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TRANSPORTATION RESEARCH CIRCULAR

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Transportation Research Board, National Academy of Sciences, 2101 Constitution Avenue, Washington, D.C. 20418

INTERIM MATERIALS ON HIGHWAY CAPACITY

modes
1 highway transportation
2 public transit
5 other

subject areas
12 planning
21 facilities design
54 operations and traffic control
55 traffic flow, capacity, and measurements

ERRATA

Transportation Research Circular 212
Interim Materials on Highway Capacity

Prior to the distribution of Transportation Research Circular 212, the Committee on Highway Capacity and Quality of Service had an opportunity to discuss and review it during the Fifty-Ninth Annual Meeting of the Transportation Research Board. At that time, the need for certain corrections in the text became apparent. This errata sheet and the separate page showing Figures 1 and 2 for "A New Technique for Design and Analysis of Weaving Sections on Freeways" have been prepared as a result. Readers are urged to report any similar problems or difficulties they encounter on the User Evaluation forms at the back of the Circular.

Page 11. Table 6. In the fourth column, replace the value of 1375 for level of Service D by the value of 1225 shown for Level of Service E, and vice versa. This correction applies also in Table 6 on pages 17, 19, 21, and 23.

Page 29. Step 11. Second line. Change 1606 to 1604.

Page 31. Step 11(R). Second line. Change 1423 to 1422.

Page 47. Second line below Table 2. Change the term M_4 to M_H

Page 49. Table 3. In the second column, change the dash to E. This correction applies also in Table 3 on pages 51, 55, 59(both places), 63, 67(both places), and 69.

Page 100. Table 26. Last line. Change 10 to 8 and 800 to 640.

Page 101. Table 26 (Continued). Last line. Change 1600 to 1400 and 2400 to 1920.

Page 108. References 4 & 5. Change Levinson, H. W. to Levinson, H. S.
Reference 16. Change Research Engineering Society to Doctor of Engineering Science.
Reference 17. Change 1971 to 1977.

Page 129. Figure 12. The portion of the Figure to the right of LOS E (i.e., for Average Flow values greater than 25) should be disregarded. This correction applies also to Figure 12 on page 135.

Page 142. Step 2b, Condition 1. Change 187.8 to 187.

Page 143. Step 1d. Last line. Change 7.6 to 6.7.

Page 169. Second Column. Line 13. Change Level of Service ..."as 3" to "as C".
Lines 34-40. Change Level of Service terms 1, 2, 3, 4, and 5 to A, B, C, D, and E, respectively.
Line 53. Change Level of Service 3 to C.
Last Paragraph. Change Level of Service terms 1, 2, and 3 to A, B, and C, respectively.

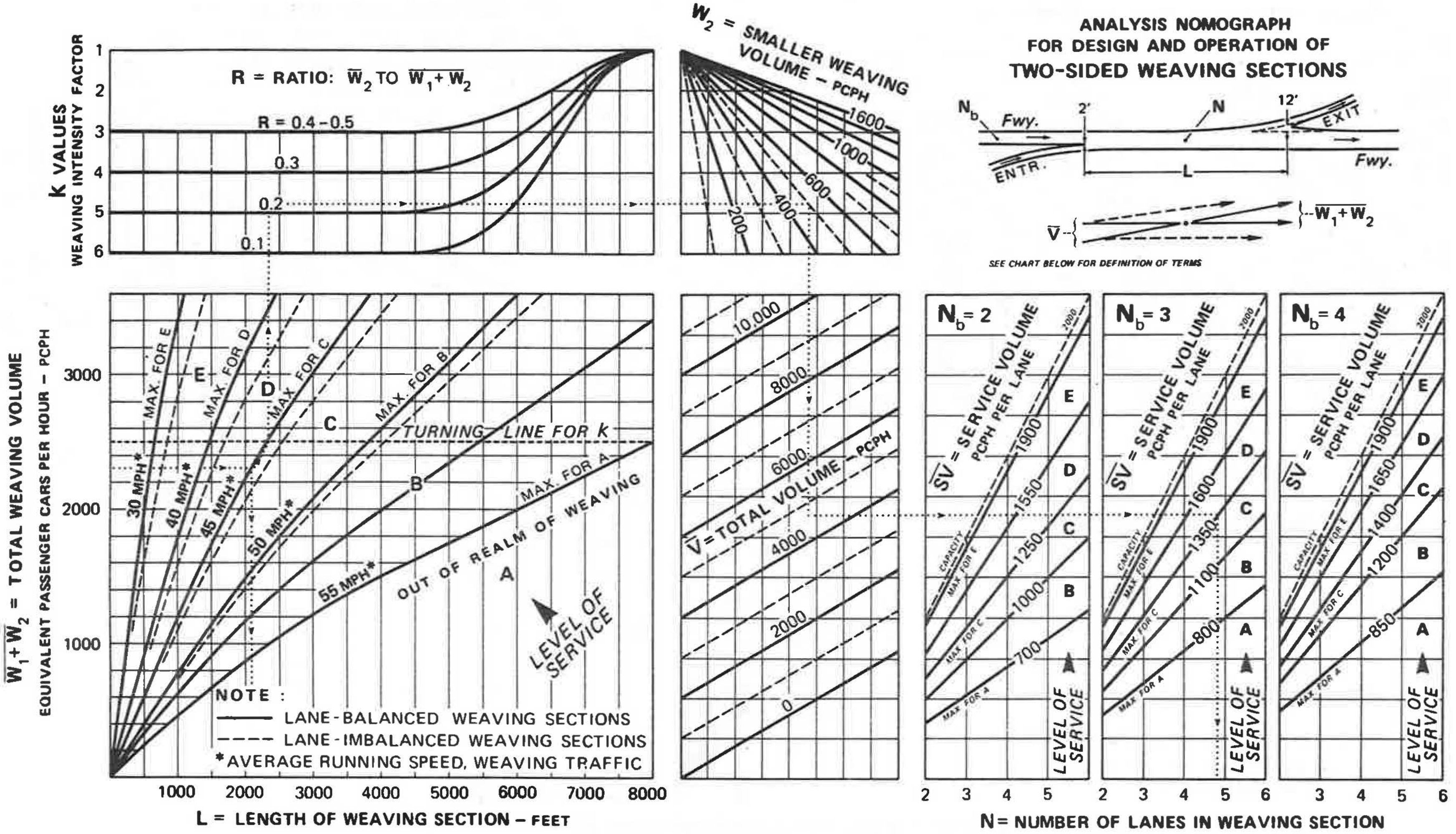
Page 170. Table 2.2. In the three columns headed LOS, change 1, 2, 3, and 4 to A, B, C, and D, respectively.

Page 227. First Column. Last line of Problem 1. Change 5 to E.

Page 264. Table 5.2. In first Column (Levels of Service) change 1, 2, 3, 4, and 5 to A, B, C, D, and E, respectively.

Page 269. Insert Figures 1 and 2 (separate sheet) here.

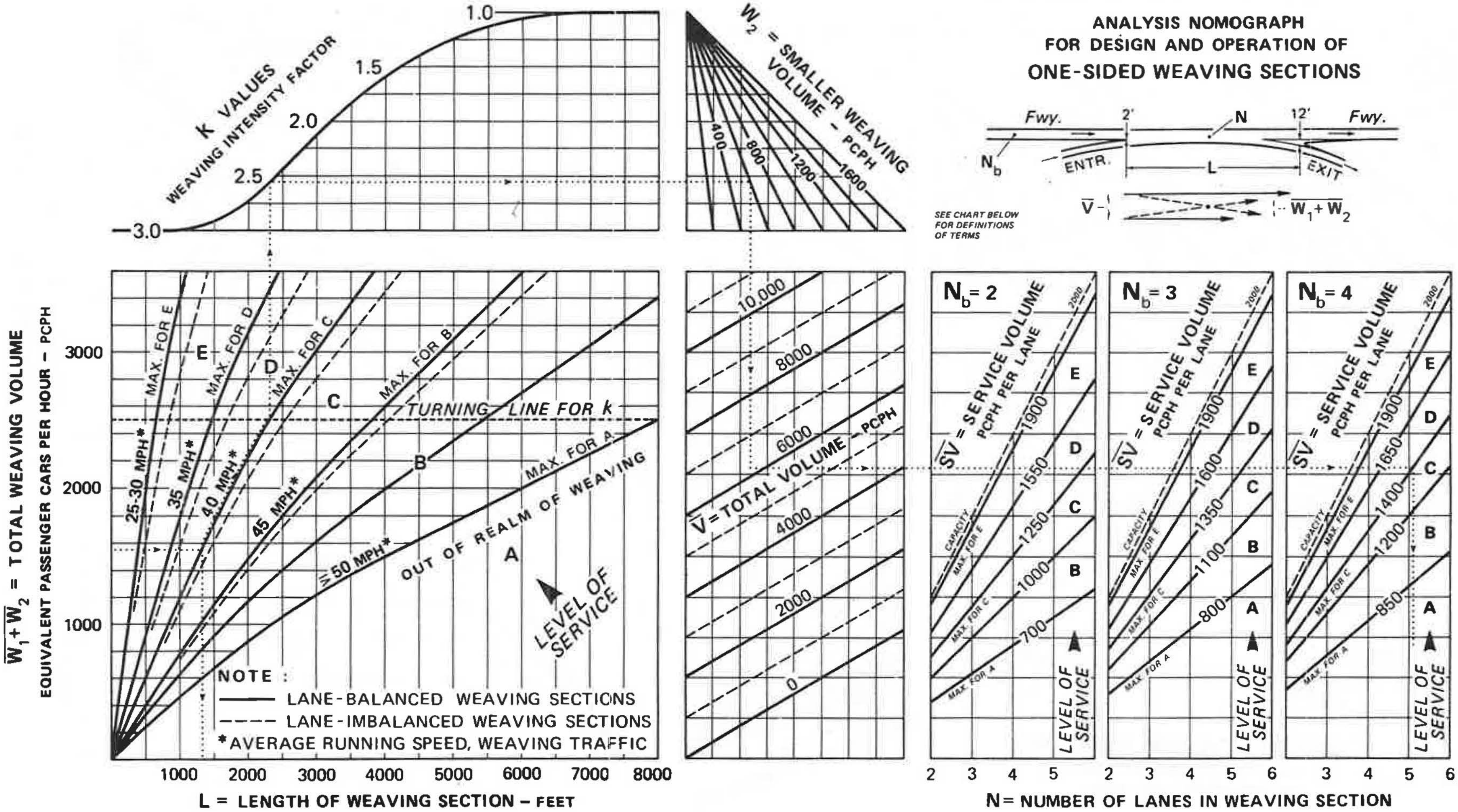
Transportation Research Circular 212, Interim Materials on Highway Capacity. Figures 1 and 2 for A New Technique for Design and Analysis of Weaving Sections on Freeways (See page 267).



NOMOGRAPH FOR DESIGN AND ANALYSIS OF WEAVING SECTIONS—TWO-SIDED CONFIGURATIONS

FIGURE 2

Transportation Research Circular 212, Interim Materials on Highway Capacity. Figures 1 and 2 for A New Technique for Design and Analysis of Weaving Sections on Freeways (See page 267).



NOMOGRAPH FOR DESIGN AND ANALYSIS OF WEAVING SECTIONS—ONE-SIDED CONFIGURATIONS

FIGURE 1

TRANSPORTATION
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Table of Contents

INTRODUCTION TO CIRCULAR

The 1965 Highway Capacity Manual has become an essential tool in planning, designing, operating, and managing roadway facilities. The production and distribution of thirty thousand copies of the document indicate its extensive use and acceptance. Yet, since 1965, emphasis has shifted from new facility construction to the upgrading of existing facilities and the implementation of operational techniques. Factors such as the presence of pedestrians, bicycles, and bus transit along roadways--and the needs for environmental and user cost analyses--have become important in engineering and planning. Though considerable research on these and other topics has been undertaken since publication of the 1965 Manual, no single document has incorporated the results in a readily usable format. Thus, this Circular addresses the need - identified by a survey of Highway Capacity Manual users - for a compilation of current procedures to supplement the Manual. This is not to say that the Circular replaced the Manual as an accepted procedure where the Manual is referred to in such terms by law or regulation. Decisions on its acceptability in that sense will have to be made by others.

The choice of a Transportation Research Circular as the publication medium has been quite deliberate. By definition, Circulars contain information of immediate interest but not necessarily of long-lasting value. They usually reflect the decisions of a TRB Committee to disseminate the results of pertinent research findings. They are quickly readied for publication and they are broadly distributed. This circular is expected to be of immediate interest to highway designers, traffic engineers, and research analysts through its provision of up-to-date information and improved procedures in selected areas of highway capacity analysis. Some of these procedures are widely accepted already; others are in need of field validation. Some may not withstand the rigors of extensive application; they may be either dropped or modified before being advanced as recommended practice in a permanent publication. Nevertheless, it is the considered judgment of the TRB Committee on Highway Capacity and Quality of Service that the methods presented here can be put to use until such time as a revised Manual becomes available.

Because the Circular has been prepared rapidly for publication, the reader may notice that different sections have different formats in their presentations. Inconsistencies not only in style but in symbols and terminology may appear (a small subcommittee of the TRB Committee is directing its attention to this and related issues). Readers who wish to call attention to these as well as other technical problems are encouraged to use the evaluation sheets attached as the last pages for that purpose.

The content of the Circular is varied and comes from a number of sources. The first sections come from Project 3-28, "Development of an Improved Highway Capacity Manual", conducted by JHK & Associates for the National Cooperative Highway Research Program. Two of the subjects treated ("Critical Movement Analysis" and "Unsignalized Intersections") are drawn from current practices.

The sections titled "Transit" and "Pedestrians" are based on material initially prepared by Herbert S. Levinson and Jeffrey M. Zupan, respectively, both members of the TRB Committee on Highway Capacity and Quality of Service.

The freeway sections of the Circular were prepared by Roger P. Roess, Elliot Linzer, William R. McShane and Louis J. Pignataro, of the Polytechnic Institute of New York, for the Implementation Division in the Federal Highway Administration's Office of Research and Development. Under the Heading of "Freeway Capacity Procedures", the work treats basic freeway segments, weaving areas, ramps, and the freeway as a total facility.

The last section of the Circular, also dealing with weaving area analysis, is based on material prepared by Jack E. Leisch.

Throughout, appropriate references to other sources and background material are cited. As the probable forerunner for parts of a revised Highway Capacity Manual, though, the Circular is intended for use on a stand-alone basis. If readers encounter difficulties in this regard, such comments transmitted on the evaluation sheets will also be welcomed. Reader experiences, both positive and negative, are required in order to develop a more responsive revision of the Highway Capacity Manual.

Looking to the future, it seems probable that some additional interim materials or even revisions to the present contents may appear. Again, their issuance through the medium of Transportation Research Circulars seems likely. Given the schedule for completion of research that is now ongoing or likely to begin in early 1980, the Committee on Highway Capacity and Quality of Service is projecting that 1983 will be the earliest date that the user community can expect to receive a wholly revised Capacity Manual. This circular, and any subsequent one, is thus anticipated to serve for some considerable time as a supplement to the 1965 Manual.

NCHRP PROJECT 3-28: DEVELOPMENT OF AN IMPROVED HIGHWAY CAPACITY MANUAL

Preface

This research was conducted by JHK & Associates as National Cooperative Highway Research Program (NCHRP) Project 3-28, Phase I. The Traffic Institute at Northwestern University served as subcontractor on the project. Mr. William R. Reilly of JHK & Associates was Principal Investigator, and the principal professional for the Traffic Institute was Mr. Ronald C. Pfefer.

Other key team members were James H. Kell, Ruel H. Robbins, Richard A. Presby, and Iris J. Fullerton of JHK & Associates. Technical editor of these materials was Mr. David A. Kell of JHK & Associates --who also served as production supervisor during final layout and paste-up. For the Traffic Institute, Jack Hutter, Alex Sorton, and Robert Seyfried provided valuable input to the work. Other personnel in both agencies also contributed to the research effort.

Appreciation is extended to the Transportation Research Board's Committee on Highway Capacity and Quality of Service for their cooperation in surveying users, for conducting workshops at the 1978 Annual Meeting of the Transportation Research Board and for reviewing these interim materials prior to publication.

Special acknowledgement is due to two individuals. Mr. Herbert Levinson of Wilbur Smith and Associates served as principal author of the Transit section, and Mr. Jeffrey Zupan of the Regional Plan Association contributed the basic work leading up to the Pedestrian section of these interim materials.

The NCHRP Project 3-28 Panel played an important part in guiding the research, and took an active role as "users" in providing insights and suggestions on the contents and format of the interim sections included in this volume.

Contents

ORGANIZATION OF THESE MATERIALS

This report comprises the first set of interim materials which will be distributed prior to the publication of a new "Highway Capacity Manual" in the mid 1980's. These interim materials are intended for application by HCM users in the 1980-1982 period. A user response form is included at the end of this document to permit users of these materials to communicate their comments to the Transportation Research Board directly. This user response will be vital in identifying desirable revisions to the interim materials prior to their inclusion in the new manual. Users are encouraged to send to TRB their observations, including actual data and analyses.

The interim materials provided in this section are Critical Movement Analysis, Unsignalized Intersections, Transit, and Pedestrians. Development of these sections has been carried out as part of NCHRP Project 3-28, "Development of an Improved Highway Capacity Manual." Project 3-28 was started in 1977, and the final report on Phase I of the Project was submitted to NCHRP in August, 1979. The final report describes the user surveys, the assessment of research and literature, the process used for developing the interim materials included here, and the proposed research program needed to produce documentation for a new Highway Capacity Manual.

Each of the interim materials in this report is introduced with a "DISCUSSION" which explains the background and the conceptual framework for the technique. The technique itself is explained and references are cited. The "USER APPLICATIONS" section then leads the user through a step-by-step description of the calculation, and several numerical examples are provided. Completed calculation forms are provided and shown for each example. Also, a blank form is provided in each section, except for the "Transit" material, which does not utilize a calculation form.

Critical Movement Analysis

Critical Movement Analysis is based on work conducted in the 1960's and 1970's by various researchers and practitioners. Of particular importance are the works of McInerney and Petersen, and of Messer and Fambro. The project team did, however, make major changes in previously reported methods to devise the final technique as presented herein. Mr. William R. Reilly, Principal Investigator of NCHRP Project 3-28, had primary responsibility for deriving the final procedure.

Critical Movement Analysis allows the HCM user to analyze the urban signalized intersection as an entire unit. The overall intersection level of service and the effects on level of service of design and operational changes can be determined. Also, guidelines on ranges of vehicle delay expected under different levels of service are included. The technique is divided into PLANNING applications for relatively simple and quick computations; and OPERATIONS AND DESIGN applications for a more detailed solution. Both applications are similar in concept and both allow the user to analyze intersections operating with pretimed signals, vehicle actuated signals, and multiphase signals with phase overlap.

For determination of capacity or level of service of a single intersection approach, the 1965 HCM remains the principal tool until the new HCM is produced.

Unsignalized Intersections

The procedure for capacity analysis of unsignalized intersections is an adaptation, in content and format, of a German technique reported in the Organisation for Economic Co-Operation and Development (OECD) report, "Capacity of At-Grade Junctions." Mr. James H. Kell, of the NCHRP Project 3-28 team, was most directly responsible for revising and adapting this technique to the point where it can be of use to the HCM user.

Only those unsignalized intersections that are controlled by two-way STOP signs or by YIELD signs can be analyzed by this technique. The procedure is not applicable to uncontrolled intersections or four way STOP sign controlled intersections.

Initially, the capacity or maximum flow of vehicles in passenger car equivalents is calculated for each minor approach movement. These values are then compared to the existing demand for each movement and the probable delay and level of service is estimated.

The assumption is made that major street traffic is not affected by the minor street movements. Left turns from the major street to the minor street are influenced only by the opposing major street through flow. Minor street flows, however, are impeded by all other conflicting movements. The procedure includes adjustments for mutual interference to the minor street traffic streams, such as the additional adverse effect of main street vehicles waiting to make left turns.

In order to treat these potential impedences, it is necessary to structure the computational procedures and deal with individual traffic movements in the following order:

1. Right turns into the major road;
2. Left turns from the major road;
3. Through traffic crossing the major road; and
4. Left turns into the major road.

In addition, the method takes into account the lane configuration on the minor street and includes appropriate adjustments for movements that use the same lane (shared lane).

The application of this technique and subsequent user comments may lead to a linking of this method to standard warrants for traffic signal installation. However, at this time no attempt has been made to relate the two procedures.

Transit

Bus transit on urban streets and expressways and, to a lesser extent, rail transit, is described in the Transit section of this document. This material was developed by Mr. Herbert S. Levinson, of Wilbur Smith and Associates. The NCHRP 3-28 Project Team participated with Mr. Levinson in the final review of the material.

The HCM user will be able to apply these materials to the analysis of capacity and level of service of bus lanes, busways, and rail transit lines. Analysis techniques for determining the number of bus berths needed, given bus flows and passenger service times, are described. Also, considerable data on characteristics of existing transit systems are included, to illustrate the operating experience of transit properties.

Although calculation forms are not included in this section, several example problems do indicate the application of the concepts and numerical values involved with transit capacity.

Pedestrians

Development of the pedestrian section was initiated with Mr. Jeffrey L. Zupan's presentation of his discussion paper, "Pedestrian Facilities," at the 1978 TRB Annual Meeting. Mr. Zupan, of the Regional Plan Association, worked with the NCHRP 3-28 project team during 1978 to expand and finalize the materials. Mr. Ruel Robbins of JHK & Associates and Mr. Alex Sorton of the Traffic Institute were instrumental in developing this section for the project team. These materials provide the HCM user with an analytical tool to analyze the flow characteristics of walkways (e.g., sidewalks) and intersection crosswalks. The section does not address other pedestrian facilities (such as stairways, escalators, and elevators), although standard reference documents describing such facilities are cited.

The analysis procedure is based on the amount of space available per person and walking speed, with space being the principal determinant of level of service. The "effective width" of a walkway is determined by using width adjustments based on the effects of various fixed objects. The technique can be used to either analyze the flow characteristics and levels of service of an existing facility, or to determine a walkway design for a given design level of service. The new concept of "platoon flow" is introduced and can be applied by the HCM user for conditions where peaking is substantial over short periods.

For crosswalks, a method is presented for the analysis of both the intersection reservoir area and for the crosswalk itself. The adequacy of either a planned or an existing crosswalk and reservoir are also determined by applying the technique.

DISCUSSION

Introduction

Critical Movement Analysis is a procedure which allows for capacity and level of service determination for signalized intersections. The analysis incorporates the effects of geometry and traffic signal operation and results in a level of service determination for the intersection as a whole operating unit.

The ability of a line of vehicles to discharge past a point is the key principle involved. Rarely can a discharge rate of 2000 passenger cars per hour of green be surpassed. Because of time lost due to queue start up and signal change intervals the maximum discharge of a single lane at signalized intersections typically varies from 1500 to 1800 passenger cars per hour of green. The 1965 Highway Capacity Manual (HCM) (1) states that a single lane at a traffic signal can accommodate 2000 and 1500 passenger cars per hour of green respectively, for a perfectly coordinated signal where all vehicles pass through without stopping, and for a signal where all vehicles must stop.

Definitions

Approach - The portion of an intersection leg which is used by traffic approaching the intersection.

Capacity - The maximum number of vehicles that has a reasonable expectation of passing over a given roadway or section of roadway in one direction during a given time period under prevailing roadway and traffic conditions.

Change Interval - Yellow time plus all red time occurring between two phases.

Critical Volume - A volume (or combination of volumes) for a given street which produces the greatest utilization of capacity (e.g., needs the greatest green time) for that street. Given in terms of passenger cars or mixed vehicles per hour per lane.

Cycle Time - The period in seconds required for one complete sequence of signal indications.

Delay - Stopped time delay per approach vehicle, in seconds per vehicle.

Green Time - The length of a green phase plus its change interval, in seconds.

Hourly Volume - The number of (mixed) vehicles that pass over a given section of a lane or roadway during a time period of one hour.

Level of Service - A measure of the mobility characteristics of an intersection, as determined by vehicle delay and a secondary factor, volume/capacity ratio.

Local Bus - A bus having a scheduled stop at the intersection under analysis.

Passenger Car Equivalency - For a given vehicle, the number of through moving passenger cars it is equivalent to, based on its headway and delay creating effects.

Passenger Car Volumes - The volumes expressed in terms of passenger cars, following the application of passenger car equivalency factors to vehicular volumes.

Period Volume - A design volume, based on the flow rate within the peak 15 minutes of an hour, and converted to an equivalent hourly volume.

Peak Hour Factor - A measure of peaking characteristics within the peak hour, equal to:

$$\text{PHF} = \frac{\text{Peak Hour Volume}}{4(\text{Highest 15 minute Volume})}$$

Phase - A part of the cycle allocated to any traffic movement or combination of traffic movements receiving right of way simultaneously during one or more intervals.

Probable Phase - A phase within the probable sequence of phases which represents the sequence of a multi-phase signal controller most likely to occur under given traffic conditions.

Through Bus - A bus not having a designated stop at the intersection under analysis.

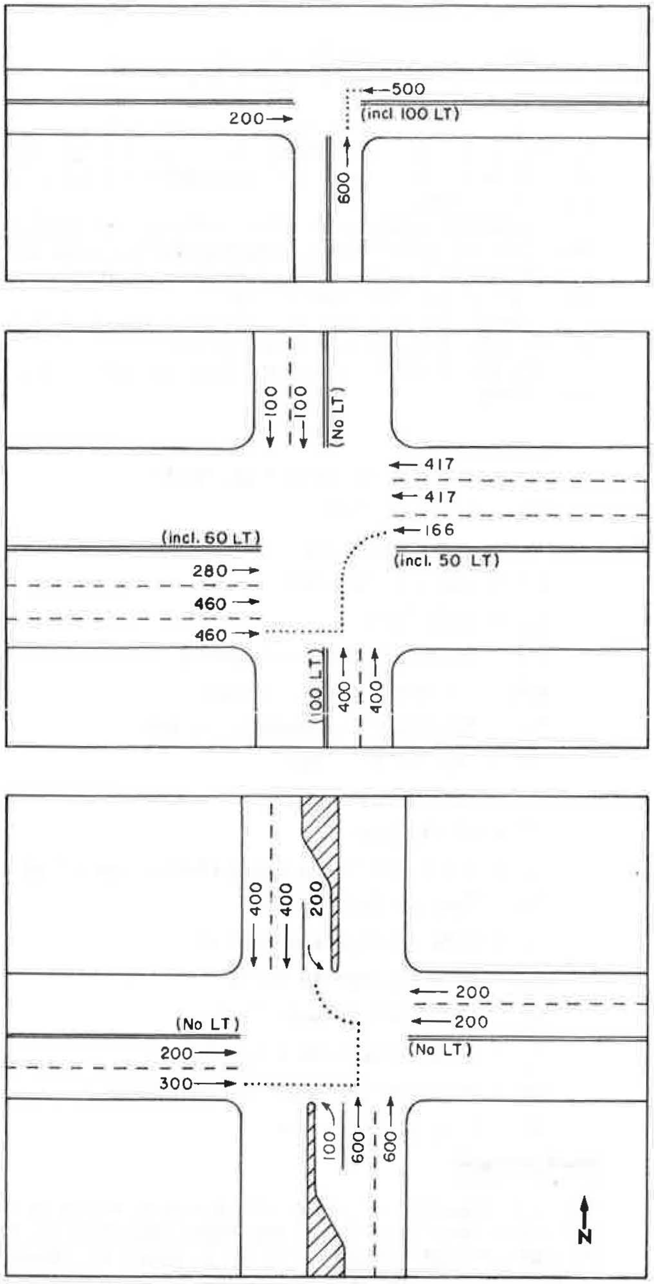
Truck - A vehicle having six or more tires on the pavement.

G/C = Green time/Cycle time ratio
HV = Hourly Volume
LB = Local Bus (Number per hour)
LOS = Level of Service
LT = Left Turn
PCE = Passenger Car Equivalency
pch = Passenger cars per hour
PCV = Passenger Car Volume, in pch
PHF = Peak Hour Factor
PV = Period Volume
RT = Right Turn
T = Truck and Through Bus (Percentage of HV)
TH = Through Traffic
U = Lane Utilization Factor
v/c = Volume/Capacity ratio
V_L = Left Turn Volume, in vph
V_0 = Volume Opposing a V_L, in vph
vph = Vehicles per hour (mixed traffic)
W = Lane Width factor

Background

The development of Critical Movement (then called "critical lane") Analysis was first reported in 1961 by Capelle and Pinnell (2) in a study of diamond interchanges. In 1971, McInerney and Petersen (3) explained the technique as applied to traffic planning work. In 1975, Trout and Loutzenheiser (4) reported on field tests and questionnaire results related to application of the method. Messer and Fambro (5) proposed a detailed procedure for critical movement analysis to assess design alternatives. In 1978, it was determined by NCHRP Project 3-28 (6) that many planners and engineers were usng the method, both for detailed traffic signal and geometric design, and for planning studies. The technique seems to be gaining greater acceptance, not only in North America but also overseas. For example, the Swedish Capacity Manual (7) contains a form of critical movement analysis in its chapter on intersections.

Figure 1. Critical Movements, PLANNING Applications



- Single Lane Approaches
- Two Phase Signal
- Critical Movements:
  500 and 600 vph
- Sum of Critical Volumes:
  500 + 600 = 1100 vph

- Two and Three Lane Approaches
- Two Phase Signal
- Critical Movements, by lane:
  400 and 460 + 50 vph
- Sum of Critical Volumes:
  400 + 510 = 910 vph

- Two and Three Lane Approaches
- Five Phase Actuated Signal
- Note:
  For the east-west street, the critical volume is 300 vph. For the north-south street the greatest demand for green time will occur with the conflicting movement totaling 800 vph (600 + 200 LT). The conflicting movement totaling 500 vph (400 + 100 LT) would require less green time and will be satisfied if the 800 vph critical volume is satisfied.
- Sum of Critical Volumes:
  300 + 800 = 1100 vph.

Note:
The above examples relate to PLANNING applications of Critical Movement Analysis. OPERATIONS AND DESIGN applications of the method use a somewhat different procedure for combining critical volumes, and express volumes in terms of passenger cars per hour (pch) instead of in terms of vehicles per hour (vph)

Analytical Base

There is at each signalized intersection a combination of conflicting movements which must be accommodated. Figure 1 shows several examples of critical movement combinations. Regardless of the complexity of the intersection and its traffic signal operations, the critical volumes (when placed on a per lane basis) cannot physically be accommodated beyond the 2000 passenger cars per hour of green (pchg) limit, and in practice cannot be accommodated beyond about 1500 to 1800 pchg. The latter values take into account the time headway between successive vehicles, the starting delay for a queue of vehicles, and the lost time due to signal change intervals.

Time headway (average headway, once the initial queue start-up time has been experienced), starting delay, and the amount of lost time due to yellow and red intervals must be considered in order to assess the capacity of a single lane. Numerous researchers have proposed formulae for calculating capacity of a single lane based on these factors. Table 1 gives several of the more prominent formulae for

Table 1. Capacity Calculation Techniques

Reference	Formula	Calculated Capacity[a]
1. Berry-Gandhi (8) Method	Cap (in vph) = $\frac{3600(G + \lambda Y - D + H)}{CH}$ where: Cap = Capacity of the signalized approach D = Starting time delay, in seconds, elapsing from beginning of green indication to the instant the rear wheels of the first vehicle cross the reference line (usually, the stop line) H = Average headway time, in seconds, for all vehicles in a compact platoon that cross the reference line. λ = Proportion of the length of yellow indication, for a loaded cycle, which is utilized up to the time the last vehicle in a compact platoon crosses the reference line C = Length of signal cycle, in seconds G = Length of green indication, in seconds Y = Length of yellow indication, in seconds vph = Vehicles per hour pch = Passenger cars per hour	$= \frac{3600[40 + (0.5)(4) - 3 + 2.1]}{(80)(2.1)}$ = 881 vehicles per hour
2. Capelle-Pinnell (2) Critical Lane Method	Cap (in vph) = $\left(\frac{G - D}{H} + 2\right)\left(\frac{3600}{C}\right)$ where: D = Starting delay--the time for the first two vehicles to enter H = Average time headway for the third, fourth, fifth, etc. vehicles to enter G = Length of green indication, in seconds C = Cycle length, in seconds	$= \left(\frac{40 - 5.0}{2.1} + 2\right)\left(\frac{3600}{80}\right)$ = 840 vehicles per hour
3. Messer-Fambro (5)	Cap (in pch) = SG/C where: C = Cycle length, in seconds S = Saturation flow, in passenger cars per hour of green, measured empirically as in the Australian Method (9, 10) and assumed as 1800 passenger cars per hour of green in this example (a typical value for a through lane) G = Effective green time, in seconds = green + yellow - 4.0 seconds	= [1800(40 + 4.0 - 4.0)]/[80] = 900 passenger cars per hour
4. Bellis-Reilly (11, 12, 13) Method	Cap (in pch) = $\left(\frac{3600}{C}\right)\left(\frac{G + 3}{H}\right)$ where: G = Length of green indication, in seconds C = Cycle length, in seconds H = Average time headway, in seconds	$= \left(\frac{3600}{80}\right)\left(\frac{40 + 3}{2.1}\right)$ = 921 passenger cars per hour
5. British (14) Method	Cap (in pch) = $\frac{160WG}{C}$ where: W = Width of lane, in feet G = Effective green time, in seconds = green + yellow - 4.0 seconds C = Cycle length, in seconds	$= \frac{(160)(12)(42)}{80}$ = 1000 passenger cars per hour
6. 1965 Highway Capacity Manual (1)	USE: Figure 6.8, p. 135. Use a 24 ft. width to place the analysis in a more representative section of the charts. Assume no turns and no trucks or through buses, and no local buses. Also, assume PHF = 0.85 and Metro Area population = 500,000. THEN: Cap (in pch) = (2100 vphg)(G/C)(PHF/Pop)(Location)(Left Turns)(Right Turns)(Trucks and Buses) = (2100)(40/80)(1.06)(1.25)(1.10)(1.10)(1.05) = 1610 passenger cars per hour per approach	= 805 passenger cars per hour

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[a] Problems based on suburban arterial street with 12 ft. lanes, headway average = 2.1 seconds, starting delay for first vehicle only = 3.0 seconds, G/C = 40/80 seconds, yellow time = 4 seconds, with 2 seconds used for traffic movement. **All results are on a per-lane basis.** (1 foot = .305 meter)

estimating capacity, and includes a numerical example.

The computations in Table 1 indicate that very little variation exists in the value used for capacity of a standard 12 foot wide (3.7 m) lane at an urban signalized intersection with ideal traffic conditions (no trucks, buses, or turning motions). Three of the models shown give capacities of approximately 900 pch for a green time/cycle time (G/C) ratio of 0.5. The British method, which has been known to give considerably higher computed values for capacity than North America methods, shows a computed capacity 12 percent higher. The 1965 HCM yields a capacity value of 805 pch (G/C = 0.50), or about 10% below the other methods.

Because of the close agreement between Berry-Gandhi (8), Capelle-Pinnell (2), Messer-Fambro (5), and Bellis-Reilly (11, 12, 13), an average value of 1800 passenger cars per hour of green (pchg) for a 12 foot (3.7 m) through traffic lane--with no trucks, buses, turns, or pedestrian interference--can be used as a base value for capacity in the critical movement analysis technique. It should be noted that the British capacity procedures use--for a 13 foot (4.0 m) wide lane--a capacity of 1950 pchg.

The factors which are considered of prime importance in modifying the capacity value of 1800 pchg for a single 12 foot (3.7 m) lane are as follows:

1. Lane Width
2. Buses and Trucks
3. Bus Stop Operations
4. Left Turns
5. Right Turns and Pedestrian Activity
6. Parking Activity
7. Peaking Characteristics (Peak Hour Factor)

Other factors--such as vertical grade and type of driver using the intersection--may be of importance in modifying the capacity value, but little research has been accomplished in these areas. Also, field measurement of saturation flow allows the HCM user to establish a capacity value for any intersection approach or lane without explicitly defining each modifying factor.

1. Lane Width. The critical movement procedure proposed by Messer and Fambro (5) includes a reduction in calculated capacity of 10 percent for lane widths between 9.0 and 9.9 feet (2.7 m and 3.0 m). For lanes 10.0 feet (3.0 m) or wider, no adjustment in capacity is made. Note that these adjustments increase the passenger car volume (PCV) rather than reduce capacity.

Using the Australian procedures (9, 10), capacity adjustments are made for lanes not falling in the 10.0 to 12.0 foot (3.0 m to 3.7 m) range. Adjustments for the value of capacity are:

Lane Width (feet):	8.0	9.0	13.0	14.0	15.0
Lane Width (meters):	2.4	2.7	4.0	4.3	4.6
Adjustment Value:	-12%	-7%	+3%	+4½%	+6%

Application of the 1965 HCM, with the assumed conditions used in Table 1, gives adjustment values of - 29% for the equivalent of a 9 foot (2.7 m) lane and + 19% for the equivalent of a 14 foot (4.3 m) lane. Table 2 combines these concepts into a readily applied set of values. These adjustments rely principally on the Messer-Fambro work, but include upward adjustments in capacity for wide traffic lanes as included in most other methods.

One important concept to note is that under peak traffic conditions, lane widths in the 10 to 13 foot (3.0 to 4.0 m) range have little effect on saturation flow or capacity. However, it is likely that if comfort and safety were to be considered in intersection level of service (LOS), lane width differences would have a greater impact on LOS than they will in the proposed new HCM; with its emphasis on mobility rather than quality of flow.

2. Buses and Trucks. Trucks, and buses not having a designated stop at the intersection under analysis (called "through" buses), reduce capacity because the time headway of these vehicles tends to be longer than the 2.0 second average implied by a capacity set at 1800 pchg.

There are two means available for including the effects of trucks and buses. First, each truck or bus can be converted to an equivalent number of passenger cars, and the volume used in the analysis

Table 2. Lane Width Adjustments

	Adjustment Factors to Capacity for Lane Width (ft.)								
Reference	8	9	10	11	12	13	14	15	16
Berry-Gandhi (8)	(Suggest use of Australian factors)								
Messer-Fambro (5)	NA[a]	1.10	1.00	1.00	1.00	1.00	1.00	1.00	NA
Australian (9), (10)	1.12	1.07	1.00	1.00	1.00	0.97	0.96	0.94	--[b]
Recommended[c] Adjustment Factors	8.0-9.9 feet W = 1.10		10.0-12.9 feet W = 1.00			13.0-15.9 feet W = 0.90			

[a] NA denotes data not available.

[b] For 16-foot wide approaches, two 8-foot lanes would be assumed.

[c] Recommended for use in Critical Movement Analysis (OPERATIONS AND DESIGN Application, Step 8)

Source: As cited above and W.R. Reilly (NCHRP Project 3-28) (1 foot = .305 meter) stated in terms of passenger cars per hour rather than (mixed) vehicles per hour. Second, the capacity of the lane can be reduced and the analysis carried out using vehicles per hour. For PLANNING applications of Critical Movement Analysis, average geometric and traffic conditions are assumed and the work is carried out in terms of mixed vehicles per hour (vph). For OPERATIONS AND DESIGN applications, the analysis is performed in terms of passenger cars per hour (pch).

The passenger car equivalency (PCE) for trucks and through buses in the 1965 HCM can be inferred from the adjustment factors used. The approximate PCE value is 2.0. In essence, this means that the time headway for these vehicles is twice that for passenger cars, or 4.0 seconds if the assumption of a 2.0 second average headway for passenger cars is used.

The recommended average PCE value for converting trucks and through buses is 2.0 (recall that six or more tires on the pavement is the working definition of "truck").

3. Bus Stop Operations. As with trucks and through buses, the effect of bus stops in or adjacent to a traffic lane is to increase the average time headway. In the development of the 1965 HCM, PCE values for local buses ranged from 1.0 to 7.0 (16). Future research is expected to result in a clear definition of the impacts on delay and capacity of bus stop operation. For an average value to apply in the critical movement analysis procedure, a PCE value of 5.0 for each local bus appears to be reasonable. This implies an average headway of 10 seconds per bus, and would be applied to all buses having a designated stop at the intersection.

For example, if 30 buses per hour stop at a nearside bus stop, with 33 percent of them stopping on red, and 67 percent on green, a total time headway for all buses is assumed to be (30 × 5.0 × 2 seconds), or 300 seconds. The 300 seconds of headway might principally be used by only 20 buses having to stop on the green for an average of 13 seconds each. The remaining 10 buses, stopping on the red interval, would create only 40 seconds of time headway, or about 4.0 seconds per bus. This latter figure relates to the recommended equivalency of 2.0 PCE for through buses and trucks.

The actual effects of a stopping bus will vary considerably depending upon bus stop location, bus dwell time, parking activity, lane configuration, and traffic volumes. However, until further research is accomplished, the figure of 5.0 PCE per local bus appears to be useful average value.

4. Left Turns. Left turning vehicles are treated in considerable detail in most capacity computation techniques. The reason for this is simple--left turns (unless removed from through traffic lanes by provision of exclusive turn lanes) have a large impact on capacity and on vehicular delay, which will be the principal determinant of level of service in the new HCM.

The most direct means of taking into account the delaying effects of left turn vehicles is to convert them to pch using PCE values. It is anticipated that future research will lead to a range of PCE values for various combinations of geometry, traffic volumes, opposing traffic volumes, and signal phasing for left turns.

Different methods use varying PCE values for left turns. The British method sets 1.75 PCE as the average value for lanes with left turning and through movements. The 1965 HCM uses adjustment factors which show an approximate PCE value of between 4.0 and 2.0 for narrow and wider approaches, respectively. For a single lane, the typical effect can be on the order of 3.0 PCE per left turn operating from a left-through lane. The actual effect varies depending on geometric and traffic factors and especially on the volume of opposing traffic.

The Messer-Fambro method describes a detailed procedure for considering left turns in critical movement calculations. Three distinct factors are described for left turn adjustments. Included are a PCE adjustment to all traffic for approaches without left turn bays, a PCE adjustment to left turn traffic for approaches with left turn bays, and a PCE adjustment to non-left turn traffic for approaches with left turn bays of inadequate length (thus creating blockages in the through lane). Although this latter factor has not been included in the critical movement procedure, the user may wish to refer to Messer and Fambro's research (5) for details on the effects of left turn storage bay lengths.

Table 3 gives the PCE values for left turns for use when applying the critical movement procedure. These values are to be considered as "average" values for a broad range of traffic and geometric conditions. Future research may lead to a more precise formulation of left turn PCE values by incorporating other variables, in addition to "opposing traffic."

Table 3. PCE Values: Left Turn Effects

Left Turns Allowed from Left-Through Lanes[a]					
1. No Turn Phase	Opposing Volume, in vph: 1 left turn equals:	0-299 1.0 PCE	300-599 2.0 PCE	600-999 4.0 PCE	1000 + 6.0 PCE
2. With Turn Phase	1 left turn equals 1.2 PCE				
Left Turns Allowed from Left Turn Bays Only[b]					
3. No Turn Phase	Opposing Volume, in vph: 1 left turn equals:	0-299 1.0 PCE	300-599 2.0 PCE	600-999 4.0 PCE	1000 + 6.0 PCE
4. With Turn Phase	1 left turn equals 1.05 PCE				

[a] PCE Values are used in Step 5, PLANNING applications, to develop a distribution of volumes among several traffic lanes. PCE Values are also used in Step 7, OPERATIONS AND DESIGN applications, to convert left turn volumes to passenger car volumes prior to adding them to through and right turn volumes, in pch.

[b] PCE Values are used in Step 7, OPERATIONS AND DESIGN applications, to convert left turn volumes (operating from a turn bay) to passenger car volumes, in pch.

Source: W. R. Reilly (NCHRP Project 3-28), based on a synthesis of various data, including Ref. (5).

5. Right Turns and Pedestrian Activity. For simplicity, the adverse effect of right turns on intersection capacity can be considered as zero if little or no pedestrian interference occurs in the parallel conflicting crosswalk. If considerable pedestrian activity exists, then a right-turning vehicle has a similar effect as a local bus, creating a greater average time headway and producing greater vehicular delay.

A study of the Australian documents (9, 10) indicates that lanes with right turn activity might show a reduction in vehicle capacity of from fifteen to thirty-five percent. The 1965 HCM (1) indicates a PCE value of approximately 1.5 for right turns on a two-lane approach. However, for one-lane approaches this value may rise to 4.0. The British (14) use a PCE value of 1.25 for right turning vehicles (actually left turns in Britain) when the right turns comprise greater than 10 percent of the total traffic. In Australia PCE values of 1.25 and 2.50 are used for right turns of automobiles and heavy vehicles, respectively.

In the Messer-Fambro (5) technique, a right turn adjustment is made, based on the radius of the corner and the percentage of traffic making the turn. Also, an adjustment is made for the vehicles which may turn right on red. Such adjustments are not of prime importance and have not been included in the critical movement procedure presented herein.

The PCE values for right turns recommended for use in Critical Movement Analysis are given in Table 4. The values listed are considered as "average" for a broad range of traffic and geometric conditions and are based on a synthesis of information from many sources. Future research may lead to a more definitive set of PCE values for right turns relative to pedestrian activity.

Table 4. PCE Values: Right Turn Effects

Type of Activity	PCE Value for Right Turning Vehicle
1. Little pedestrian activity (0 to 99 peds. per hour) in parallel conflicting crosswalk	1.00
2. Moderate pedestrian activity (100 to 599 peds. per hour) in parallel conflicting crosswalk	1.25
3. Heavy pedestrian activity (600 to 1,199 peds. per hour) in parallel conflicting crosswalk	1.50
4. Extremely heavy pedestrian activity (1,200 or more peds. per hour) in parallel conflicting crosswalk	2.00 or greater[a]

[a] as determined from local conditions.

Source: W. R. Reilly (NCHRP Project 3-28), based on a synthesis of various data.

6. Parking Activity. Little or no definitive research work on parking and its capacity effects has been completed. However, the British do use a formula to compute these effects, as follows:

Loss in Approach Width, in feet,

$$= 5.5 - \frac{0.9(Z - 25)}{K}$$

where:

Z = Clear distance, in feet, from stop line to parked car

K = Green time, in seconds

(1 foot = .305 meter)

The British formula, assuming a green time of 30 seconds, infers that there is no effect on the approach capacity if parking is approximately 200 feet (61 m) or more away from the stop line.

Most North American techniques do not explicitly consider a reduction in capacity due to parking, if the parking ends 250 feet (76 m) before the intersection. For a curbside lane where parking is allowed, 8 feet (2.4 m) should be allowed for the parking lane and its friction effects, with the remaining width being assigned to the moving lane in the capacity computations. For parking which extends into the 250 foot (76 m) area, the HCM user must use judgment on the value or lack thereof of the additional width gained at the point where parking is prohibited. Because of the lack of definitive research on parking effects, this factor has not been included in the critical movement procedure.

7. Peaking Characteristics. To convert peak 15 minute flow rates to 1 hour volumes, some type of factor must be applied. Messer and Fambro indicate that the peak 15 minute flow along urban arterials consistently exceeds the average 15 minute flow during the peak hours by twenty to thirty percent. In the 1965 HCM (1) an "average" condition at urban intersections is assumed to be that the peak 15 minute flow will exceed the average 15 minute flow by about 15 percent. This results in a peak hour factor (PHF) of 0.85.

Because the HCM user may wish to use either a 15 minute peak flow rate or the peak 1 hour volume for design or analysis, a relationship between the two is needed.

Generally, PHF will vary with such factors as volume/capacity ratio, size of city, and type of adjacent activity. The data leading to the publication of the 1965 HCM indicated (16) that the average value for PHF at all sites was 0.85. Thus, the "average" PHF (if no additional information is available) which can be assumed for analysis is 0.85. The HCM user can easily develop a set of specific Peak Hour Factors by taking a limited amount of field data on different classes of streets.

The importance of PHF is that the base figure of 1800 pchg per lane is based on the assumption that the PHF is 1.0 (i.e., flow in the peak hour is uniform by 15 minute period). If we assume one hundred percent green time in an ideal traffic lane, the maximum flow rate in a 15 minute period would be 450 (i.e., 1800 ÷ 4) passenger cars. If a PHF of 0.85 is used, the corresponding flow rate expressed in terms of hourly volume would be:

Hourly Volume (HV), in pch,
= (PHF)(4)(Highest 15 min. Flow)
= (0.85)(4)(450) = 1530 pch

This represents a fifteen percent reduction in volume on an hourly basis when compared with conditions where PHF is equal to 1.0.

Lane Utilization

Critical Movement Analysis is based on "per lane" volumes. Thus, for movements (e.g., left turn, through, and right turn) which take place from more than one lane, it is necessary to estimate the volume in each of the lanes affected. In this manner, the highest lane volume can be identified and used in the analysis.

Reilly and Bellis (11, 12, 13) indicate that a traffic movement carried in two lanes could break down into a 55% / 45% split, by lane. A traffic movement carried in three lanes might divide into a 40% / 35% / 25% split.

In the critical movement analysis proposed by Messer and Fambro (5) a lane utilization factor is applied. For two lanes, a 55% / 45% split in volume is assumed. For three lanes, 40% of the total movement is assumed to occur in the most heavily used lane. Many HCM users have used analyses based on the assumption that volume is distributed approximately equally by lane, especially under peak conditions.

Lane utilization factors (U) were developed by the NCHRP 3-28 Project Team, based on the research cited above, and modified according to operational experience. The value for U when 2 lanes are utilized represents a 52.5% / 47.5% split. The value for U when 3 lanes are utilized assumes that approximately 37% of the volume is carried in the most heavily used lane. This represents a compromise between the HCM and Messer-Fambro procedures.

Table 5 contains the adjustment factors to be applied for lane utilization. For use in OPERATIONS AND DESIGN applications, average adjustments for lane utilization of 1.05 and 1.10 are recommended for two lane and three lane situations. These adjustments increase the passenger car volume for vehicles in the two or three lanes due to volume imbalances by lane.

Table 5. Lane Utilization Adjustments

Lanes Utilized	1	2	3
Utilization Factor (U)	1.00	1.05	1.10

Source: W. R. Reilly (NCHRP Project 3-28), based on a synthesis of various data.

An example of the effects of lane distribution can be seen by assuming two approach lanes, each capable of carrying 900 pch with a G/C ratio of 0.50. When a volume of 900 pch is reached in the most heavily traveled lane, a volume of only 814 pch will be using the second lane, assuming a 1.05 lane utilization factor. Thus a total capacity of 1714 pch (five percent less than the ideal 1800 pch) can be achieved by two lanes.

Levels of Service

As part of the critical movement technique, a set of guidelines on volume/capacity (v/c) ratio, average delay values, and sum of critical volumes is presented for use, review, and comment by HCM users. Table 6 gives the recommended thresholds for the sum of critical volumes for Levels of Service A through E for both the PLANNING and the OPERATIONS AND DESIGN applications.

Table 6. Level of Service Ranges

PLANNING Applications (in vph)

Level of Service	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	900	855	825
B	1050	1000	965
C	1200	1140	1100
D	1350	1275	1375
E	1500	1425	1225
F	---------not applicable---------		

OPERATIONS AND DESIGN Applications (in pch)

Level of Service	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	1000	950	900
B	1200	1140	1080
C	1400	1340	1270
D	1600	1530	1460
E	1800	1720	1650
F	---------not applicable---------		

Source: W. R. Reilly (NCHRP 3-28) and Ref. (5)

In comparing the v/c ranges used in Table 6 with those implied from the 1965 HCM (1), the following can be noted (using the example conditions given in Table 1): Levels of Service (LOS) A, B, C, D, and E are represented by v/c ratios of approximately 0.71, 0.75, 0.81, 0.92, and 1.00, respectively. Thus, the recommended values in Table 6 closely follow the 1965 HCM for defining LOS C, D, and E, but produce more ample ranges of v/c values for levels A and B. The threshold volume levels of Table 6 are expressed in vehicles per hour (vph) for the PLANNING application and in passenger cars per hour (pch) for the OPERATIONS AND DESIGN application. The levels of service defined in Table 6 relate to the critical approaches and/or lanes at the intersection. "Non-critical" lanes will tend to operate at better levels.

Delay

Because delay will be the principal determinant of signalized intersection level of service in the new HCM, Table 7 is included. The delay values given are not yet an integral part of the Critical Movement Analysis procedure but are presented as an initial step in developing a range of delay values which can be related to intersection level of service. The values of Table 7 do not take into account the offset relationship between adjacent signals. Synthesis of data from a number of sources has been used to produce Table 7. HCM users may find it useful to compare the table with locally obtained delay data.

Table 7. Delay and Level of Service

Level of Service	Typical v/c Ratio	Delay Range[a] (secs. per veh.)
A	0.00-0.60	0.0-16.0
B	0.61-0.70	16.1-22.0
C	0.71-0.80	22.1-28.0
D	0.81-0.90	28.1-35.0
E	0.91-1.00	35.1-40.0
F	varies	40.1 or greater

[a] Measured as "stopped delay" as described in Ref. (17). Delay values relate to the mean stopped delay incurred by all vehicles entering the intersection. Note that traffic signal coordination effects are not considered and could drastically alter the delay range for a given v/c ratio.

Source: W. R. Reilly (NCHRP Project 3-28), based on a synthesis of various data.

Summary

Table 8 contains a summary list of values used in the conceptual and applied aspects of the critical movement technique.

Critical Movement Analysis: Strategy

Critical Movement Analysis can be used in two general categories of problems: PLANNING applications and OPERATIONS AND DESIGN applications. In each case the fundamentals are the same. However, the level of detail is greater for OPERATIONS AND DESIGN applications.

Critical Movement Analysis is a tool to be used for study of the intersection as an operating whole. For specific analysis of a single approach, the procedure outlined by the 1965 HCM (1) remains a valuable tool.

The key assumption in the technique is that there is a combination of lane volumes which must be accommodated in 1 hour through the middle of a signalized intersection. The sum of these volumes, termed "critical volume" by Capelle and Pinnell (2), cannot exceed the saturation flow characteristics of the intersection. In essence, 1800 pch would be the maximum value under ideal conditions for the critical volume, with 1500 vph being an average value for typical conditions.

PLANNING Applications

In these applications, an important reference work is that of McInerney and Petersen (3). The only tabular material used is that found in Table 6 which gives a single value for the maximum sum of critical lane volumes, in vehicles per hour, assuming "average" traffic, signal, and geometric conditions, and Table 3, which is used to apportion traffic among several lanes.

The focus of this tool is to allow for a rapid approximation of level of service. None of the detailed individual adjustment factors need be applied to obtain a solution. The solution is for typical average conditions and should not necessarily be used for detailed design or operational decisions.

OPERATIONS AND DESIGN Applications

A principal source used for developing this more detailed application of Critical Movement Analysis is Messer and Fambro's 1977 paper (5). Many of the concepts and values from this work have been revised or extended to reflect work found in other source documents.

Table 6 gives the level of service standards which apply to this detailed application. Previous sections contain descriptions of various adjustment procedures and factors used. Table 8 provides a summary of these factors.

An explanation and examples of the step-by-step procedure is given under the heading of "USER APPLICATIONS" later in this section.

Table 8. Summary Factors for Critical Movement Analysis

Element	Values
1. Capacity, per lane ideal conditions	1800 pch
2. Capacity, per lane average-to-good urban conditions	1500 vph
3. Green time	Assumed as actual green time plus change interval time
4. PCE values for vehicle type	1.0 = passenger car or motorcycle 2.0 = truck or through bus 5.0 = local bus
5. Peak Hour Factor	0.85 = typical, or use actual field measurements
6. PCE values for left and right turns	Left turns (see Table 3) Right turns (see Table 4)
7. Lane Utilization (U)	Two lanes, volume divides 52.5% / 47.5% Three lanes, volume in heaviest lane is 36.6% of total
8. Lane Width (W)	8.0-9.9 feet, W = 1.1 10.0-12.9 feet, W = 1.0 13.0-15.9 feet, W = 0.9

Source: W. R. Reilly (NCHRP Project 3-28)

USER APPLICATIONS

Methodology

The intent of this section is to set forth the detailed procedures, with example problems, to be used in Critical Movement Analysis. The examples are divided into two groups: PLANNING applications with quick and simple solutions, and OPERATIONS AND DESIGN applications with more complex detailed solutions. A Calculation Form has been developed for each of the two groups of applications. These forms are shown in the following pages. Detailed definitions, the analytical framework, and references used in Critical Movement Analysis, are described in the preceeding section entitled "DISCUSSION."

PLANNING applications are carried out in terms of mixed vehicles per hour (vph). OPERATIONS AND DESIGN applications are carried out in terms of passenger cars per hour (pch).

Definitions

The abbreviations and symbols used in critical movement analysis are defined below. A more detailed set of definitions of concepts and terms is found in the preceeding "DISCUSSION".

G/C = Green time/Cycle time ratio
HV = Hourly Volume
LB = Local Bus (Number per hour)
LOS = Level of Service
LT = Left Turn
PCE = Passenger Car Equivalency
pch = Passenger cars per hour
PCV = Passenger Car Volume, in pch
PHF = Peak Hour Factor
PV = Period Volume
RT = Right Turn
T = Truck and Through Bus (Percentage of HV)
TH = Through Traffic
U = Lane Utilization Factor
v/c = Volume/Capacity ratio
V_L = Left Turn Volume, in vph
V_0 = Volume Opposing a V_L, in vph
vph = Vehicles per hour (mixed traffic)
W = Lane Width factor

PLANNING Applications: Procedure

The PLANNING application of Critical Movement Analysis is based on average or better conditions of geometry and traffic. The solutions can resolve the following questions:

1. What is the operating level of service for a signalized intersection as a whole?

2. If a design level of service is set, what changes in lane geometry or demand volume will be necessary to achieve that level?

3. What changes in lane configuration or signal phasing will have the greatest impact on operating level of service?

Step-By-Step Approach

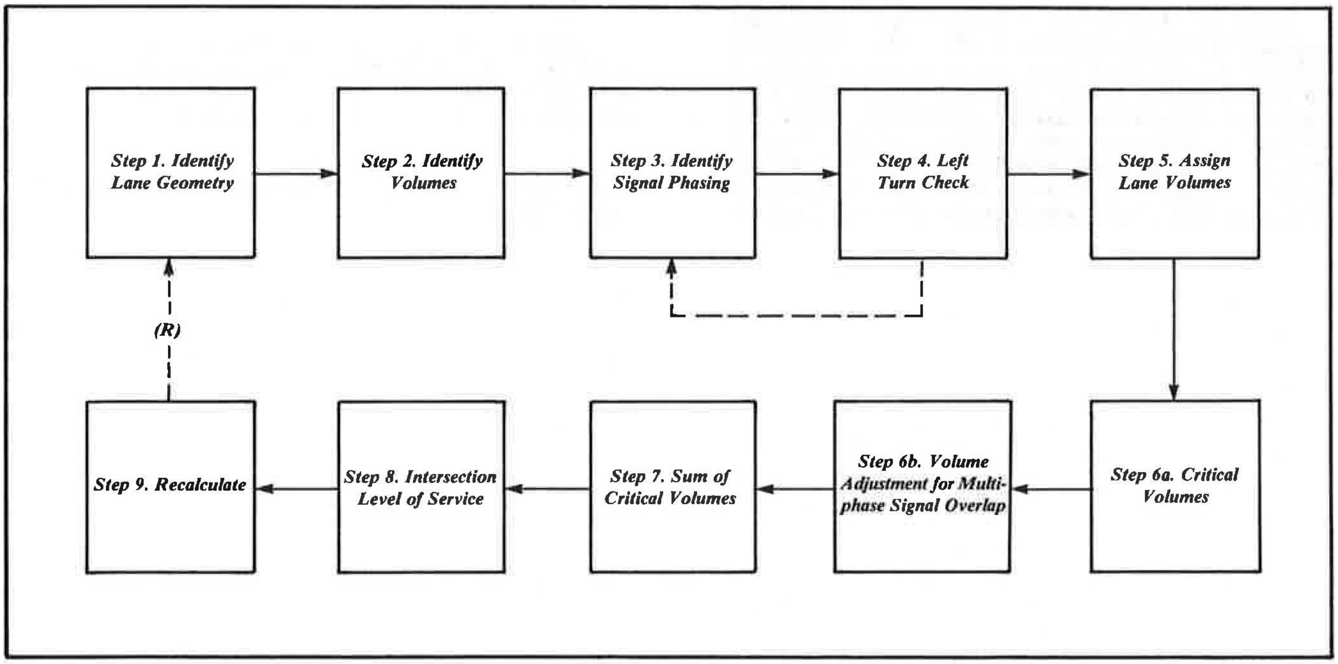
The steps followed in solving a problem by this technique are described below. Figure 2 contains an illustration of the steps followed, which are:

Step 1. Identify Lane Geometry - the assumed or known lane configuration for each approach is identified, by type of lane.

Step 2. Identify Volumes - the assumed or known traffic volumes for the design hour or analysis hour are identified in vehicles per hour. Left turn volumes, through, and right turn volumes are identified for each intersection approach.

Step 3. Identify Phasing - the signal phasing to be used for analysis is identified.

Figure 2. Procedure for Critical Movement Analysis, PLANNING Applications



Step 4. Left Turn Check - for an assumed phasing with no left turn phases, a check is made on the probability of clearing the identified left turn volume. On the change interval, 2.0 times the number of cycles per hour gives the maximum number of lefts that can clear on the change interval. Use 90 left turns per hour if no information on number of cycles per hour is available. Additionally, the number of vehicles per hour that can clear through opposing traffic during the green interval is estimated by:

$$V_L = (G/C)(1200) - V_O$$

where:

V_L = Left Turn Volume, in vph, that can clear through opposing traffic on the green interval

G/C = Green time/Cycle time ratio for opposing flow (V_O). If no other design information is available, estimate by lane volume ratio.

V_O = Volume of Opposing through plus right turn traffic, in vph.

Note that the green time in the G/C ratio is considered as the green interval plus the change interval. If the sum of the two left turn volumes described above is less than the analysis volume, a separate left turn phase can be considered, by returning to Step 3. If the sum is greater than the left turn analysis volume, no special left turn phasing needs to be considered and the analysis moves to Step 5.

The purpose of the left turn check is to determine whether all left turn movements not controlled by an exclusive turn phase can be accommodated. If not, the assumption on signal phasing can be changed to provide for left turn phasing. In many cases (e.g., analysis of existing conditions), no change in phasing is assumed and the analysis continues, with the analyst knowing that the non-satisfied left turns will create operating difficulties and be subject to excessive delay.

Step 5. Assign Lane Volumes - the volumes are assigned to the appropriate lanes. If no left turn lanes exist, the left turn volume is converted to a pch volume (Table 3) and the remaining through plus right turn volume is assumed to be in pch units. The The sum of these two pch volumes is then divided equally among all approach lanes, However, in all cases. the entire left turn volume must be assigned to the lane(s) from which the turns are made, and the remaining pch volume for through and right turn traffic is distributed equally among the remaining lanes. Following this distribution, the pch volume is converted back to vehicles per hour for the lane carrying the left turn.

If a left turn lane exists, the left turn volume in vehicles per hour is assigned to that lane and the through plus right turn volume is divided equally among the through and through-right lanes. For the special case of a double left turn lane, fifty-five percent of the total left turn volume is assigned to one left turn lane and forty-five percent to the other.

Step 6. Critical Volumes - for each signal phase, the highest total of conflicting traffic (on a per lane basis) is identified. For a two phase signal, the "highest total of the through (or through plus right turn if no exclusive right turn lane exists) plus the opposing left turn volume" is selected. For a three-to-eight phase ("multiphase") signal, each phase listed in the typical (i.e., most probable) phase sequence has one critical volume. The most probable phase sequence represents the sequence of a multiphase signal most likely to occur under the volume conditions assigned in Step 5. Where an exclusive right turn lane exists, such a lane is often not included in the critical analysis if right turns on red are permitted. However, such a lane can be included if the analyst believes that it might carry the most critical volume for that approach. Some reduction (30 percent is typical) in the assigned right turn volume (Step 5) may be made to allow for right turns made on red. If right turns on red are not permitted, an exclusive right turn lane is included in the analysis. Note that Calculation Form 1 contains Step 6a, which is used for two phase signals, and Step 6b, which is used for multiphase signals. In Steps 6a and 6b, a street operating without separate turn phases must have the opposing left turns added to the through volume to obtain the critical volume for that street.

Step 7. Sum of Critical Volumes - the critical volumes, for each phase, are summed.

Step 8. Intersection Level of Service - the sum of critical volumes is compared with Table 6, and an intersection level of service is identified.

Step 9. Recalculate - depending on the solution found in Step 8, a change in geometry, demand volume, or signal phasing can be made, and a recalculation --Steps1(R) through 9(R)--is performed.

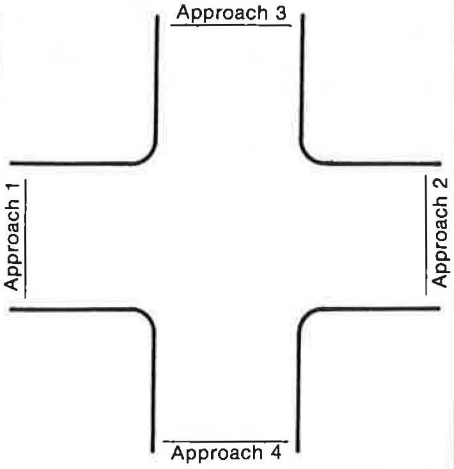
Calculation Form 1 is used for PLANNING applications.

Critical Movement Analysis: PLANNING
Calculation Form 1

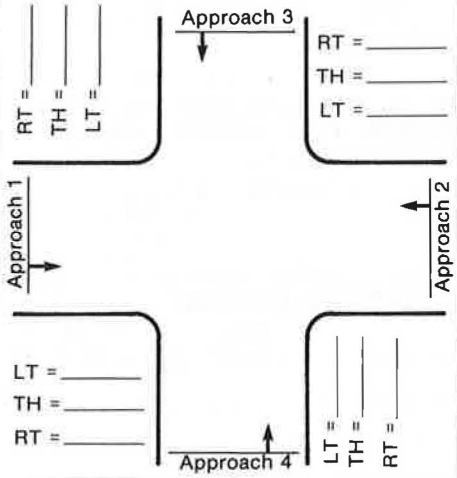
Intersection ______________________________ **Design Hour** ____________

Problem Statement ____

Step 1. Identify Lane Geometry



Step 2. Identify Volumes, in vph



Step 3. Identify Phasing

☐ ________

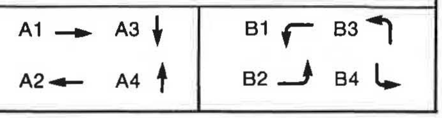
☐ ________

☐ ________

☐ ________

☐ ________

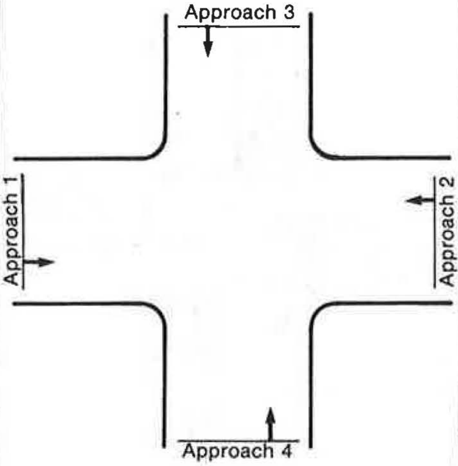
☐ ________



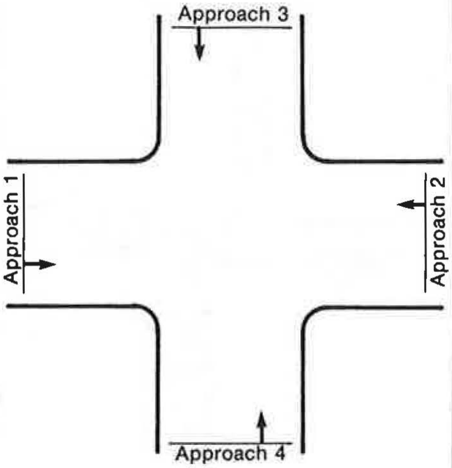
Step 4. Left Turn Check

	Approach			
	1	2	3	4
a. Number of change intervals per hour				
b. Left turn capacity on change interval, in vph				
c. G/C Ratio				
d. Opposing volume in vph				
e. Left turn capacity on green, in vph				
f. Left turn capacity in vph (b + e)				
g. Left turn volume in vph				
h. Is volume > capacity (g > f)?				

Step 5. Assign Lane Volumes, in vph



Step 6a. Critical Volumes, in vph (two phase signal)



Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph

Step 7. Sum of Critical Volumes

________ + ________ + ________ + ________

= __________ vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

☐

Step 9. Recalculate

Geometric Change ____________________

Signal Change ____________________

Volume Change ____________________

Comments

____

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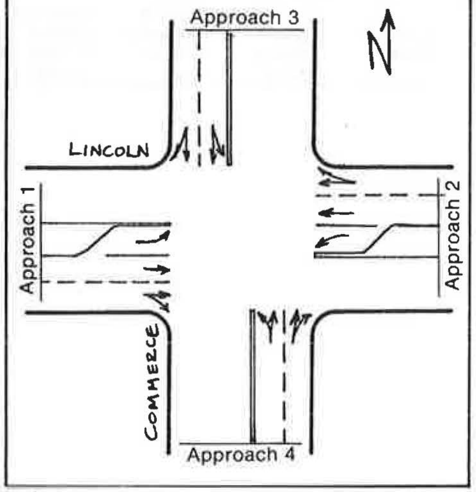
Critical Movement Analysis: PLANNING
Calculation Form 1

Example 1

Intersection LINCOLN AND COMMERCE **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND EXISTING LOS. CAN LT BE HANDLED WITH 2 ϕ ?

Step 1. Identify Lane Geometry



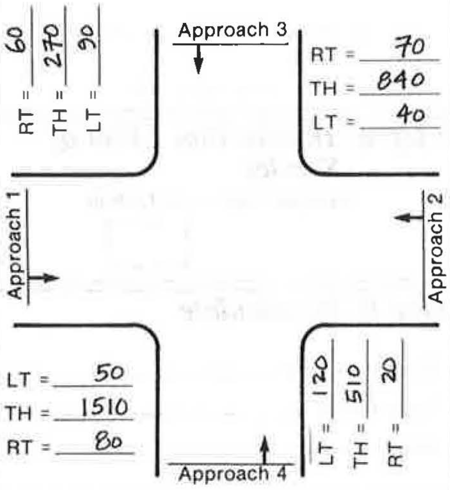
Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour	40	40	40	40
b. Left turn capacity on change interval, in vph	80	80	80	80
c. G/C Ratio	.55	.55	.45	.45
d. Opposing volume in vph	910	1590	530	330
e. Left turn capacity on green, in vph	0	0	10	210
f. Left turn capacity in vph (b + e)	80	80	90	290
g. Left turn volume in vph	50	40	90	120
h. Is volume > capacity (g > f)?	No	No	No*	No

Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph
	2ϕ		

Step 2. Identify Volumes, in vph



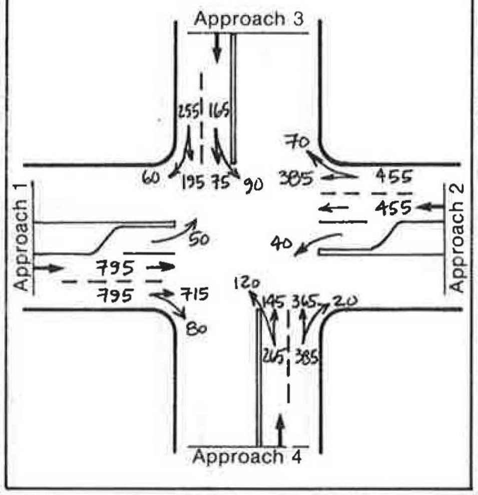
Approach 3: RT = 60, TH = 270, LT = 90

Approach 2: RT = 70, TH = 840, LT = 40

Approach 1: LT = 50, TH = 1510, RT = 80

Approach 4: LT = 120, TH = 510, RT = 20

Step 5. Assign Lane Volumes, in vph



Step 7. Sum of Critical Volumes

795 + 40 + 385 + 90

= 1310 vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

D

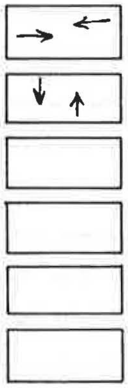
Step 9. Recalculate

Geometric Change ADD LT LANES TO APPROACHES 3 AND 4

Signal Change

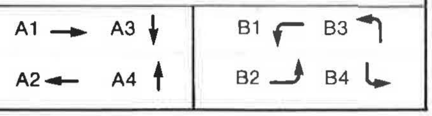
Volume Change

Step 3. Identify Phasing 2ϕ

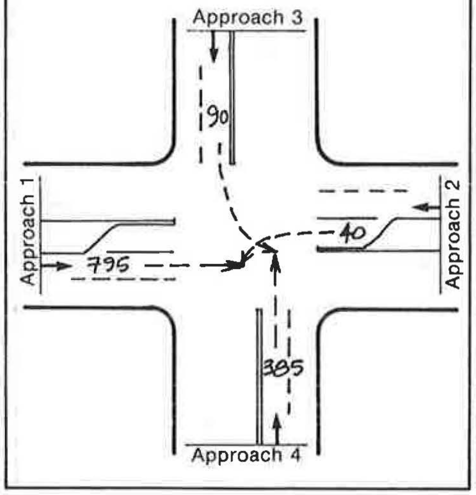


A1B2 or A2B1

A3B4 or A4B3



Step 6a. Critical Volumes, in vph (two phase signal)



Comments

*NOTE THAT LEFT TURN DEMAND FOR APPROACH 3 EQUALS CAPACITY

PLANNING Applications: Example 1

Problem

Lane configuration and peak hour volumes are shown on Calculation Form 1 for an existing urban intersection. The following three questions must be answered:

1. What is the intersection level of service?

2. Can left turns be handled without installing an exclusive phase?

3. If left turn lanes are added on Approaches 3 and 4 what changes, if any, may be expected in the level of service?

Analysis

Step 1. Identify Lane Geometry. Existing lane configuration is shown on Calculation Form 1.

Step 2. Identify Volumes. Existing peak hour volumes (vph) are shown on Calculation Form 1. Approaches are numbered 1, 2, 3, and 4, from the west, east, north, and south, respectively.

Step 3. Identify Phasing. A two phase signal operation exists.

Step 4. Left Turn Check. A 90 second peak hour cycle length is used. Forty cycles per hour times 2.0 left turns per cycle result in 80 left turns per hour made on the change interval. Additionally, left turns made through opposing traffic on the green interval, assuming a 0.55 G/C ratio for Approaches 1 and 2 and a 0.45 G/C ratio for Approaches 3 and 4 are calculated by the formula:

$$V_L = (G/C)(1200) - V_0.$$

For all directions, the capacity for left turns is equal to or greater than left turn demand. Therefore, the two phase signal operation is adequate. Note that for left turns from Approach 3, demand and capacity are equal at 90 vph.

Step 5. Assign Lane Volumes. For Approaches 1 and 2, left turn volumes are assigned to the left turn lanes and through plus right turn volumes are divided equally between the remaining lanes.

For Approaches 3 and 4, factors from Table 3 are used to convert 90 and 120 left turns (with 530 vph and 330 vph opposing, respectively) to 180 and 240 pch, respectively. Thus, a total pch volume of 510 (from Approach 3) and 770 (from Approach 4) is computed. On a per lane basis, 255 pch and 385 pch, from Approaches 3 and 4, respectively, are computed.

For Approach 3, the left lane is assigned 255 pch, of which 180 pch is due to left turn vehicles. The right lane is also assigned 255 pch, comprised of through and right turn traffic. Therefore, the left lane carries 165 vph (90 left turns plus the difference between 180 and 255) and the right lane carries 255 vph.

For Approach 4, the left lane is assigned 385 pch, of which 240 pch are due to left turn vehicles. The right lane is also assigned 385 pch, comprised of through and right turn traffic. Thus, the left lane carries 265 vph (120 left turns plus the difference between 240 and 385) and the right lane carries 385 vph.

Table 6. Level of Service Ranges

Level of Service	PLANNING Applications (in vph)		
	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	900	855	825
B	1050	1000	965
C	1200	1140	1100
D	1350	1275	1375
E	1500	1425	1225
F	---------not applicable---------		

OPERATIONS AND DESIGN Applications (in pch)

(deleted)

The per lane volumes are entered in Step 5 of Calculation Form 1.

Step 6. Critical Volumes. Critical volumes for phase A1A2, on Approaches 1 and 2, is 795 + 40 LT or 455 + 50 LT. Use 835. Critical Volumes for phase A3A4 on Approaches 3 and 4 is 255 + 120 LT or 385 + 90 LT. Use 475. These volumes are graphically shown in Step 6A on the form.

Step 7. Sum of Critical Volumes. The sum of critical volumes is 835 + 475 or 1310 vph.

Step 8. Intersection Level of Service. Using Table 6, this value falls within the range of 1201 to 1350 vph or Level of Service D for two phase signals. The left turns can be handled using the geometry shown and a two phase signal.

Step 9. Recalculate. To determine the effect on level of service of adding left turn lanes on Approaches 3 and 4, return to Step 1 and recompute.

(Continued)

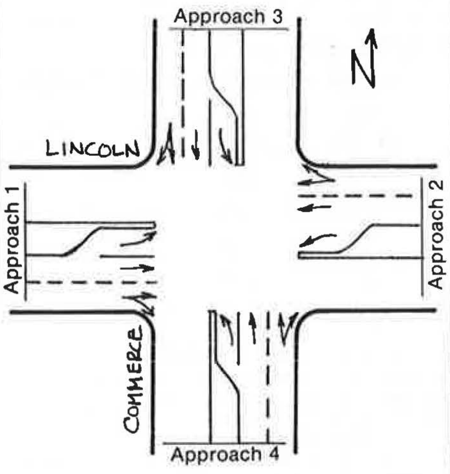
Example 1
(Recalculation)

Critical Movement Analysis: PLANNING
Calculation Form 1

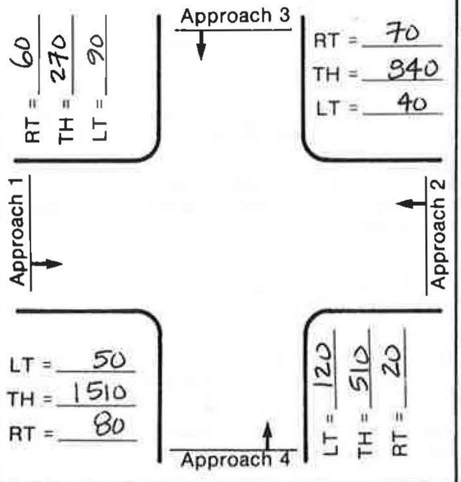
Intersection LINCOLN AND COMMERCE **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND CHANGE IN LOS BY ADDING LEFT-TURN LANES

Step 1. Identify Lane Geometry



Step 2. Identify Volumes, in vph



RT = 60
TH = 270
LT = 90

RT = 70
TH = 840
LT = 40

LT = 50
TH = 1510
RT = 80

LT = 120
TH = 510
RT = 20

Step 3. Identify Phasing 2φ

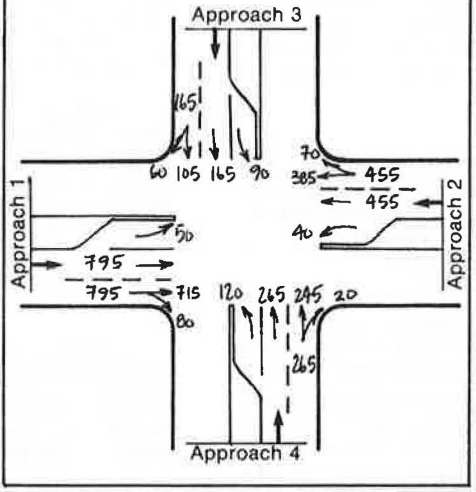
→ ←	A1B2 or A2B1
↓ ↑	A3B4 or A4B3

A1 → A3 ↓ B1 ↰ B3 ↰
A2 ← A4 ↑ B2 ↱ B4 ↳

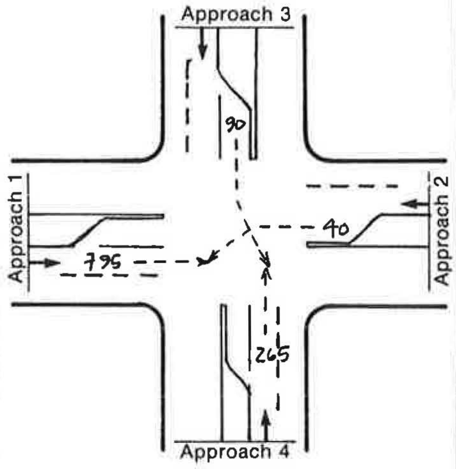
Step 4. Left Turn Check

	Approach 1	Approach 2	Approach 3	Approach 4
a. Number of change intervals per hour	40	40	40	40
b. Left turn capacity on change interval, in vph	80	80	80	80
c. G/C Ratio	.55	.55	.45	.45
d. Opposing volume in vph	910	1590	530	330
e. Left turn capacity on green, in vph	0	0	10	210
f. Left turn capacity in vph (b + e)	80	80	90	290
g. Left turn volume in vph	50	40	90	120
h. Is volume > capacity (g > f)?	No	No	No*	No

Step 5. Assign Lane Volumes, in vph



Step 6a. Critical Volumes, in vph (two phase signal)



Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph

2φ

Step 7. Sum of Critical Volumes

795 + 40 + 265 + 90
= 1190 vph

Step 8. Intersection Level of Service
(compare Step 7 with Table 6)

C

Step 9. Recalculate
NOT NECESSARY
Geometric Change
Signal Change
Volume Change

Comments
*NOTE THAT LEFT TURN DEMAND FOR APPROACH 3 EQUALS CAPACITY.

(Example 1)

Note: "(R)" denotes a recalculation.

Step 1(R). Identify Lane Geometry. Left turn lanes are added on Approaches 3 and 4.

Step 2(R). Identify Volumes. Volumes, in vph are shown on the form.

Step 3(R). Identify Phasing. The existing two phase signal will be analyzed.

Step 4(R). Left Turn Check. Step 4(R) is identical to the preceeding Step 5.

Step 5(R). Assign Lane Volumes. Left turns are assigned to left turn lanes and through plus right turn volumes are distributed equally to the remaining lanes.

Step 6(R). Critical Volumes. Critical volumes for phase A1A2 on Approaches 1 and 2 are 795 + 40 LT or 455 + 50 LT. Use 835. Critical volumes for phase A3A4 on Approaches 3 and 4 are 165 + 120 LT or 265 + 90 LT. Use 355.

Step 7(R). Sum of Critical Volumes. The sum of the critical volumes is (835 + 355) or 1190 vph.

Table 6. Level of Service Ranges

PLANNING Applications (in vph)

Level of Service	Maximum Sum of Critical Volumes Two Phase	Three Phase	Four or more Phases
A	900	855	825
B	1050	1000	965
[✓ C]	1200	1140	1100
D	1350	1275	1375
E	1500	1425	1225
F	---------not applicable---------		

OPERATIONS AND DESIGN Applications (in pch)

(deleted)

Step 8(R). Intersection Level of Service. Using Table 6, the value of 1190 vph falls within the range of 1051 to 1200, or Level of Service C for two phase operation.

Step 9(R). Recalculate. No recalculation is necessary as it is demonstrated that left turn lanes alter the intersection Level of Service D to C.

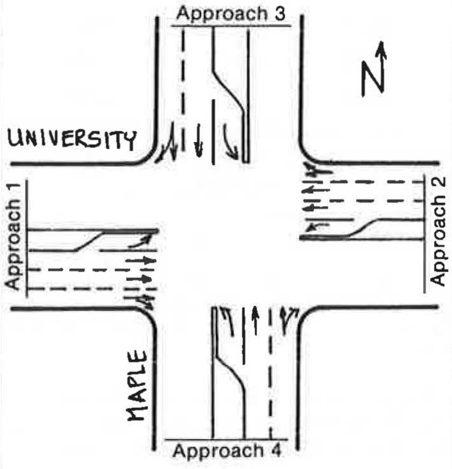
Critical Movement Analysis: PLANNING
Calculation Form 1

Example 2

Intersection UNIVERSITY AND MAPLE **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND EXISTING LOS

Step 1. Identify Lane Geometry



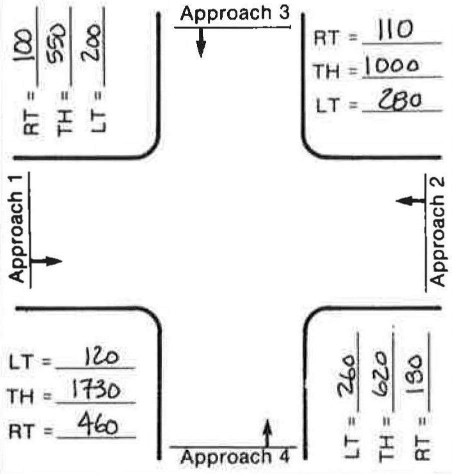
Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour		8ϕ		
b. Left turn capacity on change interval, in vph				
c. G/C Ratio				
d. Opposing volume in vph				
e. Left turn capacity on green, in vph				
f. Left turn capacity in vph (b + e)				
g. Left turn volume in vph				
h. Is volume > capacity (g > f)?				

Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph
B2B1	120(B2)	280-120=160(B1)	120 ↘ LT from 2
A2B1	160 (B1)	370-160=210(A2)	160 ↗
A1A2	730(A1) OR 210(A2)		730
B4B3	200(B4)	260-200=60(B3)	200 ↘ LT from 4
A4B3	60(B3)	400-60=390(A4)	60 ↗
A3B4	325(A3) OR 340 (A4)		340

Step 2. Identify Volumes, in vph



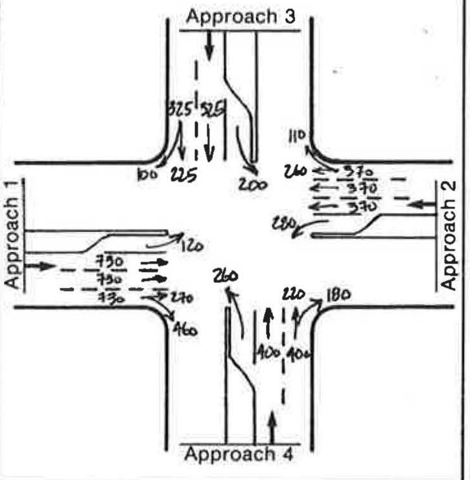
RT = 100
TH = 550
LT = 200

RT = 110
TH = 1000
LT = 280

LT = 120
TH = 1730
RT = 460

LT = 260
TH = 620
RT = 130

Step 5. Assign Lane Volumes, in vph



Step 7. Sum of Critical Volumes

280 + 730 + 260 + 340

= 1610 vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

FAILURE* —

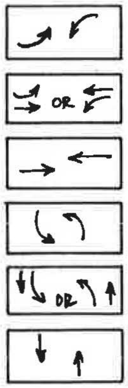
Step 9. Recalculate

Geometric Change 1 THRU LANE-APPROACHES 3 & 4 / 1 RT LANE-APPROACHES 1 & 2

Signal Change

Volume Change

Step 3. Identify Phasing 8ϕ



B2 B1

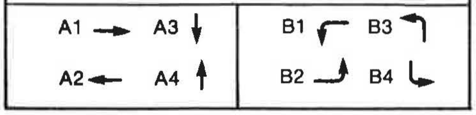
A1 B2 OR A2 B1

A1 A2

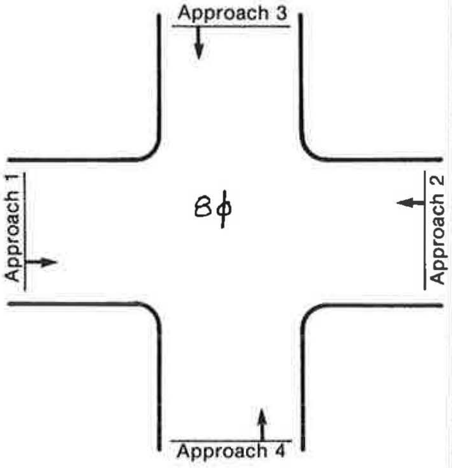
B4 B3

A3 B4 OR A4 B3

A3 A4



Step 6a. Critical Volumes, in vph (two phase signal)



Comments

* INTERSECTION WILL NOT OPERATE WITHOUT VERY LONG QUEUES AND EXCESSIVE DELAYS.

PLANNING Applications: Example 2

Problem

Lane configuration and design hour volumes (with left turn lanes on all approaches) are shown on the calculation form for a major new suburban intersection. The following information is needed.

1. The whole intersection level of service if an eight phase signal operation is used.

2. Change in level of service if an additional through lane is added to Approaches 3 and 4, and a right turn lane to Approaches 1 and 2.

Analysis

Step 1. Identify Lane Geometry. The assumed lane configuration is shown on the form.

Step 2. Identify Traffic Volumes. Design hour volumes are shown on the form.

Step 3. Identify Phasing. An eight phase signal is planned, with left turn arrows for each direction. The left turns are allowed only on the arrow (in a protected mode).

Step 4. Left Turn Check. Each left turn movement has a protected phase. Therefore, the left turn check is not needed.

Step 5. Assign Lane Volumes. Left turns are assigned to left turn lanes and through plus right turn volumes are distributed equally to the remaining lanes.

Step 6. Critical Volumes. Using Step 3, the phase sequence which most likely will appear under the volumes of Step 5 is: B2B1, A2B1 , A1A2, B4B3, A4B3, and A3A4. For example, since left turn volume from Approach 2 (B1) is greater than left turn volume from Approach 1 (B2), B1 will continue receiving a green arrow after B2 has been terminated. Thus, A2B1 is selected as the most probable phase, rather than A1B2.

Table 6. Level of Service Ranges

PLANNING Applications (in vph)

Level of Service	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	900	855	825
B	1050	1000	965
C	1200	1140	1100
D	1350	1275	1375
Ⓔ	1500	1425	1225
F	---------not applicable---------		

OPERATIONS AND DESIGN Applications (in pch)

(deleted)

Using the most probable phase sequence, the through plus right turn volume which moves during the concurrent display of a left arrow is subtracted from the total through plus right turn volume and the remaining volume is carried over to the next phase. This calculation is listed in Step 6b on the form.

Step 7. Sum of Critical Volumes. The sum of critical lane volumes for all phases is 120 + 160 + 730 + 200 + 60 + 340, or 1610 vph.

Step 8. Intersection Level of Service. Using Table 6, the critical sum of 1610 vph falls beyond Level of Service E (1375 vph) for eight phase control. Therefore, the intersection will not operate without unacceptable delays.

Step 9. Recalculate. Return to Step 1 and recalculate to determine the effects of adding a through lane on Approaches 3 and 4, and a right turn lane on Approaches 1 and 2.

(Continued)

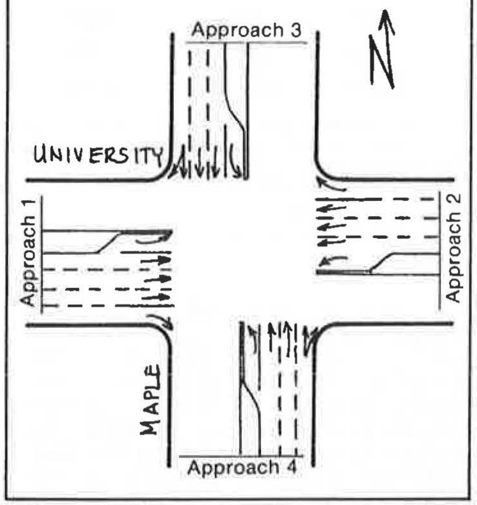
Critical Movement Analysis: PLANNING
Calculation Form 1

Example 2
(Recalculation)

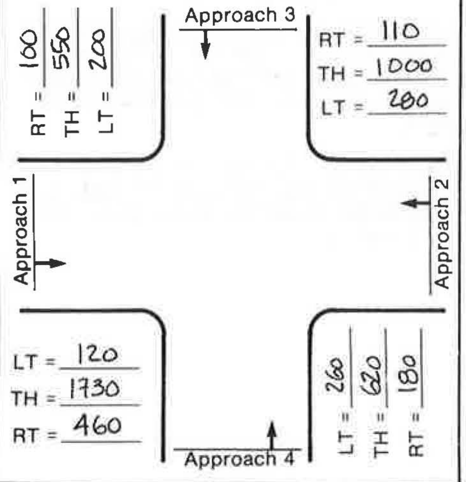
Intersection UNIVERSITY AND MAPLE **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND CHANGE IN LOS BY ADDING ADDITIONAL THRU AND RT LANES

Step 1. Identify Lane Geometry



Step 2. Identify Volumes, in vph



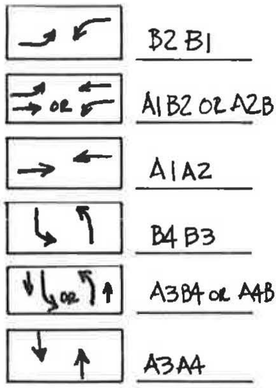
Approach 3: RT = 180, TH = 550, LT = 200

Approach 2: RT = 110, TH = 1000, LT = 280

Approach 1: LT = 120, TH = 1730, RT = 460

Approach 4: LT = 260, TH = 620, RT = 180

Step 3. Identify Phasing 8φ



B2 B1

A1 B2 OR A2 B1

A1 A2

B4 B3

A3B4 OR A4B3

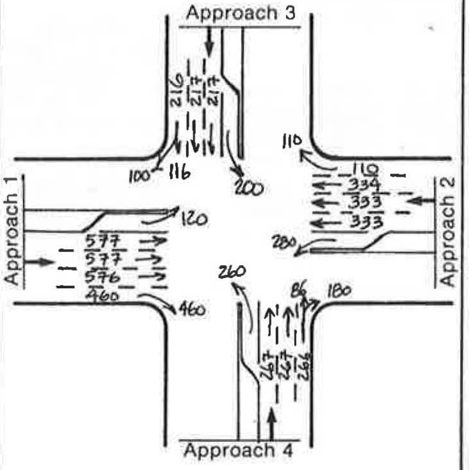
A3A4

A1 → A3 ↓ B1 ↰ B3 ↰
A2 ← A4 ↑ B2 ↱ B4 ↳

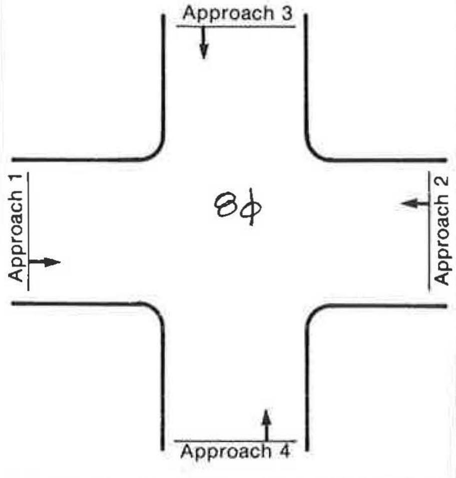
Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour		8φ		
b. Left turn capacity on change interval, in vph				
c. G/C Ratio				
d. Opposing volume in vph				
e. Left turn capacity on green, in vph				
f. Left turn capacity in vph (b + e)				
g. Left turn volume in vph				
h. Is volume > capacity (g > f)?				

Step 5. Assign Lane Volumes, in vph



Step 6a. Critical Volumes, in vph (two phase signal)



Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph
B2B1	120(B2)	280-120=160(B1)	120
A2B1	160 (B1)	334-160=174(A2)	160
A1A2	577(A1) OR 174(A2)		577
B4B3	200 (B4)	260-200=60(B3)	200
A4B3	60(B3)	267-60=207(A4)	60
A3A4	217(A3) OR 207(A4)		217

LT from 2

LT from 4

Step 7. Sum of Critical Volumes

280 + 577 + 60 + 217

= 1334 vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

E

Step 9. Recalculate

NOT NECESSARY

Geometric Change ______

Signal Change ______

Volume Change ______

Comments

(Example 2)

Note: "(R)" denotes a recalculation.

Step 1(R). Identify Lane Geometry. The new lane geometry to be analyzed is shown on the form.

Step 2(R). Identify Volumes. Design hour volumes are shown on the form.

Step 3(R). Identify Phasing. An eight phase signal is assumed, with left turn arrows for each direction. Left turns are allowed only on the arrow (in a protected mode).

Step 4(R). Left Turn Check. Each left turn movement has a protected phase. Therefore, the left turn check is not needed.

Step 5(R). Assign Lane Volumes. Left turns are assigned to left turn lanes and right turns are assigned to exclusive right turn lanes, on Approaches 1 and 2. Remaining volumes are distributed equally to the remaining lanes.

Step 6(R). Critical Volumes. Using Step 3, the phase sequence which most likely will appear under volumes of Step 5 is; B2B1, B1A2, A1A2, B4B3, A3B4, and A3A4. For example, since the left turn volume from Approach 2 (B1) is greater than left turn volume from Approach 1 (B2), B1 will continue receiving a green arrow after B2 has been terminated. Thus, A2B1 is selected as the most probable phase, rather than A1B2.

Using the most probable phase sequence, the through plus right turn volume (except where right turns have an exclusive lane) which moves during a left arrow is subtracted from the total through plus right turn volume and the remaining volume is carried over to the next phase. Note that exclusive right turn lanes are not included in the critical volume analysis when right turns on red are permitted unless the analyst considers this lane to be critical. In this example, right turns on red are permitted.

Table 6. Level of Service Ranges

	PLANNING Applications (in vph)		
	Maximum Sum of Critical Volumes		
Level of Service	Two Phase	Three Phase	Four or more Phases
A	900	855	825
B	1050	1000	965
C	1200	1140	1100
D	1350	1275	1375
E	1500	1425	1225
F	---------not applicable---------		
	OPERATIONS AND DESIGN Applications (in pch)		
	(deleted)		

Step 7(R). Sum of Critical Volumes. The sum of critical volumes for all phases is 120 + 160 + 577 + 200 + 60 + 217, or 1334 vph.

Step 8(R). Intersection Level of Service. Using Table 6 1334 vph falls within the range of 1226 to 1375, for Level of Service E for eight phase control.

Step 9(R). Recalculate. Recalculations could be made to determine the improvement in level of service from other geometric or signal changes, such as addition of double left turn lanes.

OPERATIONS AND DESIGN Applications: Procedure

The OPERATIONS AND DESIGN application of Critical Movement Analysis allows for specific adjustments to be made for traffic and roadway conditions. In essence, there are five adjustments, related to the following factors: vehicle mix, peaking characteristics, turns, lane utilization (i.e., volume distribution) and lane width.

This procedure follows a similar pattern as the PLANNING application, but works in passenger car units (pch) rather than in mixed vehicle units (vph). The level of service is taken from Table 6, using the listing for this application.

The OPERATIONS AND DESIGN procedure can be used for determining the following:

1. What is the operational level of service for a signalized intersection, given information on demand volume, lane configuration, signal operation, and traffic and geometric conditions?

2. What will be the effects of geometric or traffic signal operation changes on intersection level of service?

3. What changes are necessary at an intersection to achieve a desired level of service, given a known demand volume?

Step-By-Step Approach

The procedure uses a step-by-step approach which is briefly explained below, and shown in Figure 3.

Step 1. Identify Lane Geometry - the assumed or known lane configuration for each approach is identified. Lane widths are noted for all lanes.

Step 2. Identify Hourly Volumes - the design volume (or existing volume) is identified by movement (in vph) for each approach. The percentage of trucks and through buses and the number of local buses is also indicated for each approach.

Step 3. Identify Phasing. Movements are identified according to information shown in Step 3 of the form.

Step 4. Left Turn Check - for an assumed phasing with no left turn phases, a check is made on the probability of clearing the identified left turn volume. On the change interval, 2.0 times the number of cycles per hour gives the maximum number of lefts that can clear on the change interval. Use 90 left turns per hour if no information on number of cycles per hour is available. Additionally, the number of vehicles per hour that can clear through opposing traffic during the green interval is estimated by:

$$V_L = (G/C)(1200) - V_O$$

where:

G/C = Green time/Cycle time ratio for opposing flow (V_O). If no other design information is available, estimate by lane volume ratio.

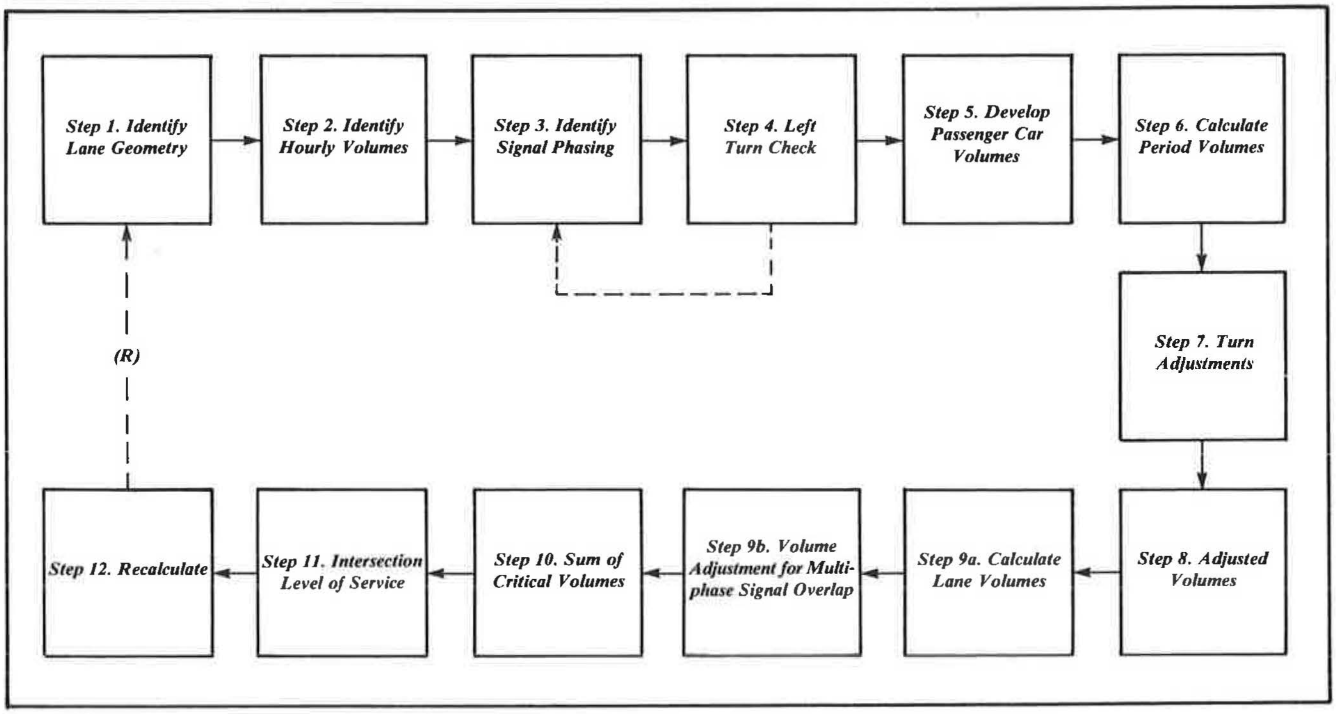
V_O = Volume of opposing through plus right turn traffic, in vph. (Right turn volumes can be excluded from V_O if the exit is wide enough to minimize interference with V_L.)

Note that the green time in the G/C ratio is considered to be the green interval plus change interval. Use 0.50 as a value for G/C if no other design information is available.

If the sum of the two left turn volumes described above is less than the analysis volume, a separate left turn phase can be considered, by returning to Step 3. If the sum is greater than the left turn analysis volume, no special left turn phasing needs to be considered and the analysis moves to Step 5.

The purpose of the left turn check is to determine whether all left turn movements not controlled by an exclusive turn phase can be accommodated. If not, the assumption on signal phasing can be changed to provide for left turn

Figure 3. Procedure for Critical Movement Analysis, OPERATIONS AND DESIGN Applications



moves to Step 5.

The purpose of the left turn check is to determine whether all left turn movements not controlled by an exclusive turn phase can be accommodated. If not, the assumption on signal phasing can be changed to provide for left turn phasing. In many cases (e.g., analysis of existing conditions), no change in phasing is assumed and the analysis continues, with the analyst knowing that the non-satisfied left turns will create operating difficulties and be subject to excessive delay.

Step 5. Develop Passenger Car Volumes - hourly volumes (HV) in mixed traffic terms (vph), are converted to an equivalent number of passenger cars per hour (pch). Each through bus and truck is 2.0 PCE, each passenger car or motorcycle is 1.0 PCE and each local bus (those with a designated stop at the intersection) is 5.0 PCE. The volumes, in pch, are computed for each traffic movement (e.g., left turn, through, and right turn) served by one or more lanes.

$$PCV = HV + T(HV) + 4(LB)$$

where:

PCV = Passenger Car Volume, in pch

HV = Hourly Volume, in vph

T = Trucks + Through Buses, as a decimal percentage of HV

LB = Local Buses (buses which have a scheduled stop at the intersection) per hour

Step 6. Calculate Period Volumes - from the passenger car volumes, design period volumes are calculated. The following formula is used:

$$PV = PCV/PHF$$

where:

PV = Period Volume, in pch

PCV = Passenger Car Volume, in pch

PHF = Peak Hour Factor

A PHF of 0.85 to 0.90 is average for many urban streets. The period volume implies that the analysis is based on a peak flow rate within the peak hour. The period volumes are assigned to each movement served by one or more lanes.

Step 7. Turn Adjustments - the period volumes are adjusted to account for turning movements.

For left turns, there are two general cases. First, for left turns made from left through lanes, a PCE value is obtained from Table 3, items 1 or 2. Item 1 is applied for cases with no turn phase where the opposing traffic (from Step 2) is the principal determinant of the PCE value. Item 2 is applied in cases where a turn phase exists and the turning movement is the determinant.

Second, for situations with an exclusive left turn lane, PCE values are taken from Table 3, items 3 and 4. For left turns having no phase (item 3), the opposing volume must be determined, from Step 2. For turns made on an exclusive phase (item 4), a single PCE value (1.05) is used to account for the effective increase in volume due to the turn.

The PCE values obtained are multiplied by the appropriate left or right turn volume to obtain a total PCV volume (in pch).

Step 8. Adjusted Volumes - the PCV volumes from Step 7 are multiplied by "lane utilization" factors (U, from Table 5) and by "lane width" factors (W, from Table 2).

For movements having more than one lane, the average of the lane widths is used to enter Table 2. This averaging is a simplifying assumption for this method. Note that only the width available to moving vehicles is used. For example, assume that an intersection approach has two lanes for through plus right turn traffic. One lane is 11 feet (3.4 m) wide and the other is 18 feet (5.5 m) wide, with 5 feet (1.5 m) striped for a bike lane. The average lane width would be (11 + 18 - 5) / 2 = 12 feet (3.7 m). The 12 feet (3.7 m) would be used to enter Table 2.

The lane utilization and lane width adjustments result in a final passenger car per hour (pch) volume for each movement carried by one or more lanes at the intersection. This volume, in pch, has been derived by applying five adjustments to the base volume in vph.

$$\text{Adjusted PCV} = U \times W \times PCV$$

Step 9. Calculate Lane Volumes - the adjusted volume for each movement, from Step 8, is divided by the number of lanes available for the movement in Step 9a. For example, if a left turn lane is provided, one (1) lane is available for that left turn movement. If two additional lanes, for through and right turns, are provided on the same intersection approach, then two lanes would be used for the through plus right turn adjusted volume. For the special case of a double (two abreast) left turn lane, two (2) lanes are used and the lane utilization adjustment accounts for volume imbalance by lane. The computation for lane volume, in pch, is as follows:

$$\text{Lane Volume} = \text{Adjusted PCV} \div \text{Number of Lanes}$$

For analysis of two phase signals, only Step 9a need be completed. For signals having multiphase (three to eight phases) operation a calculation of probable phasing and adjusted critical volumes in Step 9b is necessary. The most probable phase sequence represents the sequence of a multiphase signal most likely to occur under the volume conditions assigned in Step 9a.

The "volume carryover" computation is performed by subtracting the through plus right turn volume which moves during a left arrow from the total through plus right turn volume for that movement. The remainder is carried over to the next "probable" phase. The "adjusted critical volume", in pch, is then selec- for each probable phase.

Step 10. Sum of Critical Volumes - using Table 9, the critical combination of lane volumes for each of two streets is determined. The movement descriptions in Table 9 relate to Figure 4. Footnote c in Table 9 makes an important distinction between this OPERATIONS AND DESIGN application and that of PLANNING. Care must be exercised in carefully following the critical volume summation procedure given in Table 9.

Step 11. Intersection Level of Service - using the sum from Step 10, a comparison with level of service values given in Table 6 is made, based on the type of signal phasing used for the analysis.

Step 12. Recalculate - changes can be made in the assumed lane geometry, signal phasing, or volumes and a recalculation made--Steps 1(R) through 12(R).

Summary

The step-by-step approach described above is illustrated on Calculation Form 2. Two example problems utilizing this form are presented on the following pages.

Figure 4. Identification of Intersection Movements

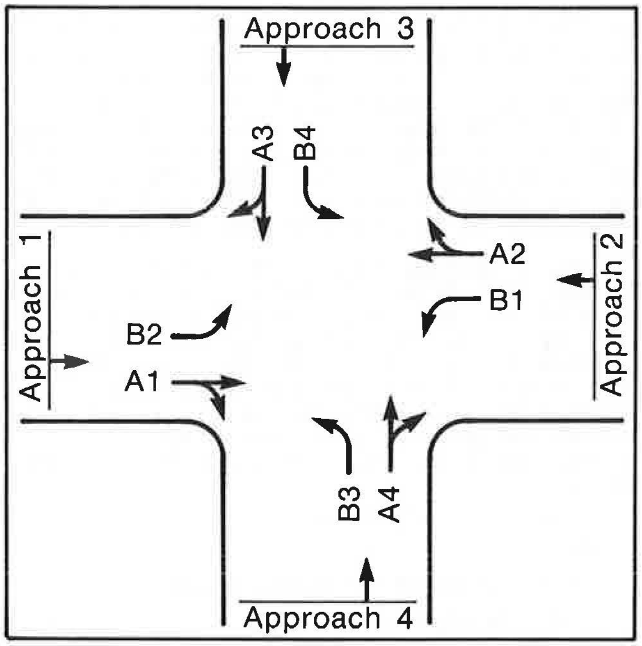


Table 9. Combining Critical Movements, OPERATIONS AND DESIGN Applications

Signal Phasing and Intersection Geometry	Approaches[a]	Critical Movements[b]
One phase, no left turn bay	1 and 2 3 and 4	A1B2 or A2B1 A3B4 or A4B3
One phase, with left turn bay	1 and 2 3 and 4	A1 or A2 or B1 or B2 A3 or A4 or B3 or B4
Two phases, no overlap, with left turn bay		
1. Leading or lagging left turns, from both directions	1 and 2 3 and 4	A1 or A2 + B1 or B2[c] A3 or A4 + B3 or B4[c]
2. Leading or lagging left turns, from one direction	1 and 2 3 and 4	B1 + A1 or A2[d] B3 + A3 or A4[d]
Two phases, with overlap, with left turn bay		
1. Leading or lagging left turns, from both directions	1 and 2 3 and 4	A1 + B1 or A2 + B2 A3 + B3 or A4 + B4
2. Leading or lagging left turns, from one direction	1 and 2 3 and 4	B1 + A1 or A2[d] B3 + A3 or A4[d]

[a] See Figure 4 for an identification of intersection movements and approaches.

[b] By approach, on a per lane basis. Select the maximum of the alternatives shown.

[c] Note that the critical volume on a given street is the single highest volume. Combining through traffic and opposing left turns is not done in OPERATIONS AND DESIGN applications. This is a major difference between these applications and PLANNING applications. Messer and Fambro have established, through actual use of the method (particularly, the identification of critical volumes) that the results have conceptual validity and are useful for design work.

[d] Assume arrow is for movements B1 and B3. Other combinations are possible, depending on intersection configuration.

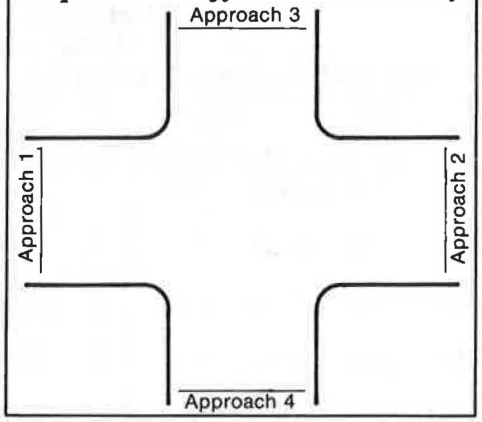
Source: W. R. Reilly (NCHRP 3-28), based on Messer-Fambro (5).

Critical Movement Analysis: OPERATIONS AND DESIGN
Calculation Form 2

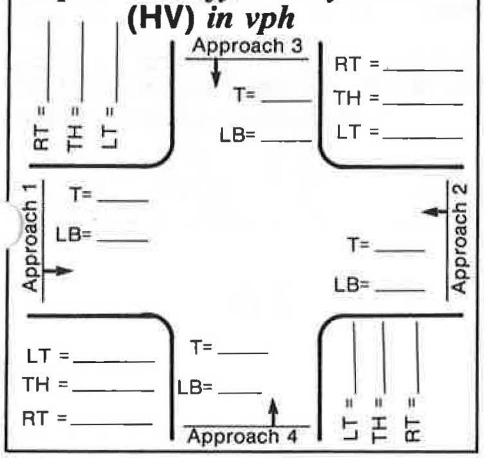
Intersection ______________________________ **Design Hour** ____________

Problem Statement ____

Step 1. Identify Lane Geometry



Step 2. Identify Hourly Volumes (HV) *in vph*



Step 3. Identify Phasing

☐ ____ ☐ ____

☐ ____ ☐ ____

☐ ____ ☐ ____

A1 → A3 ↓

A2 ← A4 ↑

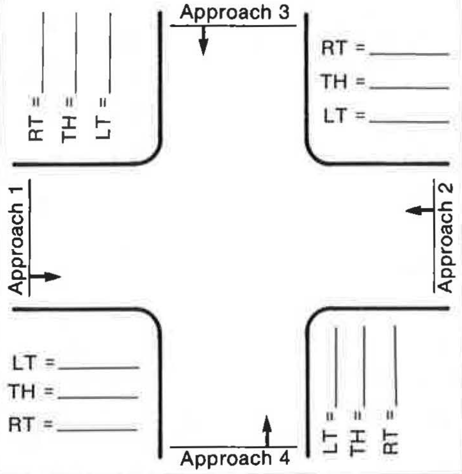
B1 ↰ B3 ↰

B2 ↱ B4 ↳

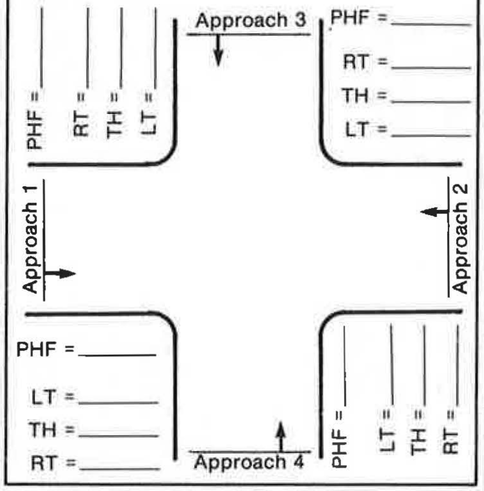
Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour				
b. Left turn capacity on change interval, in vph				
c. G/C Ratio				
d. Opposing volume in vph				
e. Left turn capacity on green, in vph				
f. Left turn capacity in vph (b + e)				
g. Left turn volume in vph				
h. Is volume > capacity (g > f)?				

Step 5. Develop Passenger Car Volumes (PCV) in pch



Step 6. Calculate Period Volumes (PV) *in pch*



Step 7. Turn Adjustments

Approach	
Movement	
Turn	
Turn volume (PV from Step 6)	
Opposing vol. in vph from Step 2	
Ped. vol/hour	
PCE LT from Table 3	
LT vol. in pch	
PCE RT from Table 4	
RT vol. in pch	
TH vol. in pch from Step 6	
Total PCV in pch	

Step 8. Adjusted Volumes / *Step 9a. Calculate Lane Volumes*

Movement	Total PCV (Step 7)	U	W	Adjusted PCV (U×W×PCV)	No. of Lanes	PCV per Lane

Step 9b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in pch	Volume Carryover to next phase	Adjusted Critical Volume in pch

Step 10. Sum of Critical Volumes

________ + ________ + ________ + ________

= ________ pch

Step 11. Intersection Level of Service

(compare Step 10 with Table 6)

☐

Step 12. Recalculate

Geometric Change ____________________

Signal Change ____________________

Volume Change ____________________

Comments ____________________

____

____

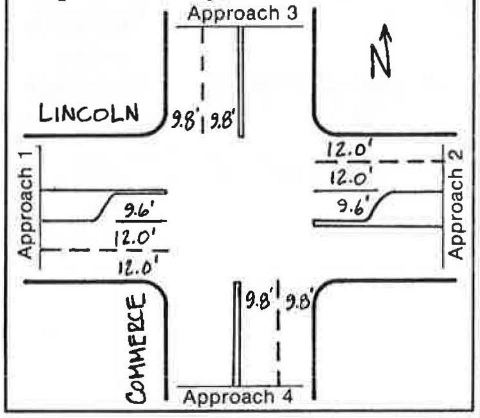
Critical Movement Analysis: OPERATIONS AND DESIGN
Calculation Form 2

Example 1

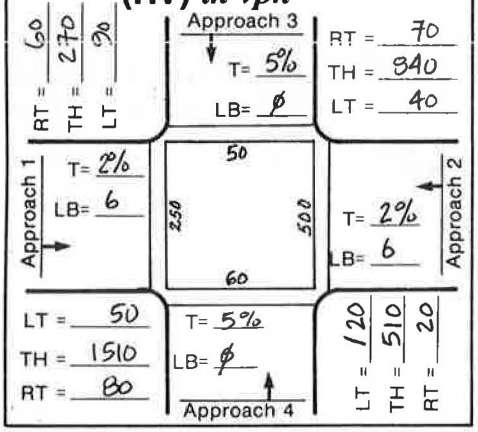
Intersection LINCOLN AND COMMERCE **Design Hour** 4:30-5:30 p.m.

Problem Statement FIND EXISTING LOS CAN LT BE HANDLED WITH 2ϕ?

Step 1. Identify Lane Geometry



Step 2. Identify Hourly Volumes (HV) in vph



Approach 3: RT = 60, TH = 270, LT = 90; T= 5%; LB= 0

Approach 2: RT = 70, TH = 840, LT = 40; T= 2%; LB= 6

Approach 1: T= 2%; LB= 6; LT = 50, TH = 1510, RT = 80

Approach 4: T= 5%; LB= 0; LT = 120, TH = 510, RT = 20

Step 3. Identify Phasing 2ϕ

⇄	A1B2 or A2B1
↓↑	A3 B4 or A4 B3
☐	
☐	
☐	
☐	

A1 → A3 ↓

A2 ← A4 ↑

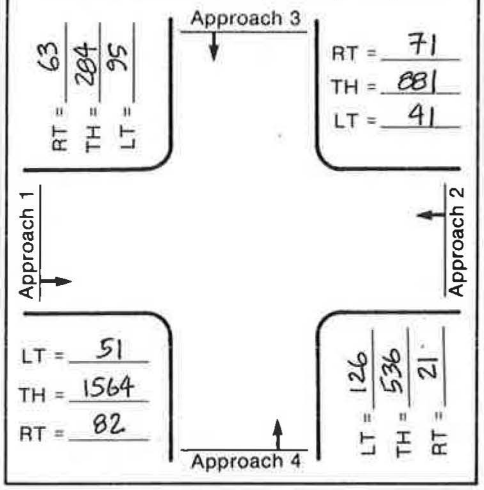
B1 ↰ B3 ↰

B2 ↱ B4 ↳

Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour	45	45	45	45
b. Left turn capacity on change interval, in vph	90	90	90	90
c. G/C Ratio	.50	.50	.50	.50
d. Opposing volume in vph	910	1590	530	330
e. Left turn capacity on green, in vph	0	0	70	270
f. Left turn capacity in vph (b + e)	90	90	160	360
g. Left turn volume in vph	50	40	90	120
h. Is volume > capacity (g > f)?	No	No	No	No

Step 5. Develop Passenger Car Volumes (PCV) in pch



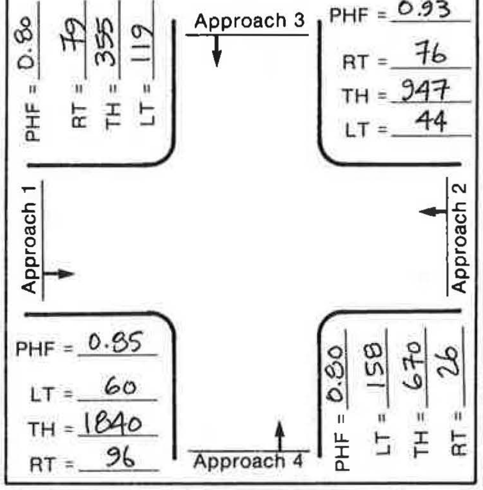
Approach 3: RT = 63, TH = 284, LT = 95

Approach 2: RT = 71, TH = 881, LT = 41

Approach 1: LT = 51, TH = 1564, RT = 82

Approach 4: LT = 126, TH = 536, RT = 21

Step 6. Calculate Period Volumes (PV) in pch



Approach 3: PHF = 0.80, RT = 79, TH = 355, LT = 119

Approach 2: PHF = 0.93, RT = 76, TH = 947, LT = 44

Approach 1: PHF = 0.85, LT = 60, TH = 1840, RT = 96

Approach 4: PHF = 0.80, LT = 158, TH = 670, RT = 26

Step 7. Turn Adjustments

Approach	1		2		3		4	
Movement	B2	A1	B1	A2	B4	A3	B3	A4
Turn	LT	RT	LT	RT	LT	RT	LT	RT
Turn volume (PV from Step 6)	60	96	44	76	119	79	158	26
Opposing vol. in vph from Step 2	910	-	1590	-	530	-	330	-
Ped. vol/hour	-	60	-	50	-	250	-	500
PCE LT from Table 3	4.0	-	6.0	-	2.0	-	2.0	-
LT vol. in pch	240	-	264	-	238	-	316	-
PCE RT from Table 4	-	1.00	-	1.00	-	1.25	-	1.25
RT vol. in pch	-	96	-	76	-	99	-	33
TH vol. in pch from Step 6	-	1840	-	947	-	355	-	670
Total PCV in pch	240	1936	264	1023	238	454	316	703

Step 8. Adjusted Volumes / *Step 9a. Calculate Lane Volumes*

Movement	Total PCV (Step 7)	U	W	Adjusted PCV (U×W×PCV)	No. of Lanes	PCV per Lane
B2	240	1.00	1.10	264	1	264
B1	264	1.00	1.10	290	1	290
A1	1936	1.05	1.00	2033	2	1016
A2	1023	1.05	1.00	1074	2	537
A3B4	692	1.05	1.10	799	2	400
A4B3	1019	1.05	1.10	1177	2	588

Step 9b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in pch	Volume Carryover to next phase	Adjusted Critical Volume in pch

2ϕ

Step 10. Sum of Critical Volumes

(B2 or B1 or A1 or A2) + (A3B4 or A4B3)

1016 (A1) + 588 (A4B3) + ______ + ______

= 1604 pch

Step 11. Intersection Level of Service

(compare Step 10 with Table 6)

E*

Step 12. Recalculate

Geometric Change ADD 11' L.T LANE TO APPROACHES 3 AND 4.

Signal Change ______

Volume Change ______

Comments

*BORDERLINE BETWEEN LOS D AND E

OPERATIONS AND DESIGN: Example 1

Problem

Lane configuration and peak hour volumes are given as shown on Calculation Form 2 for an existing urban intersection. Note that the geometry and volumes are identical to those of Example 1 under the heading of PLANNING Applications. Additionally, information is known about vehicle mix, peak hour factor, turns, and lane width.

The following questions must be answered:

1. What is the intersection level of service?

2. Can left turns be accommodated without installing an exclusive phase?

3. If left turn lanes (11 feet wide) are added to Approaches 3 and 4, what change--if any--in level of service will occur?

Analysis

Step 1. Identify Lane Geometry. Existing lane configuration and lane widths are shown on the form. Approaches are numbered 1, 2, 3, and 4 from the west, east, north, and south, respectively.

Step 2. Identify Hourly Volumes. Existing peak hour volumes (in vph) are shown on the form. Also, information on trucks plus through buses (T) and local buses (LB) is given. A pedestrian volume in pedestrians per hour is given for each of four crosswalks.

Step 3. Identify Phasing. A two phase signal exists. Movements are identified according to information shown in Step 3 of the form.

Step 4. Left Turn Check. The number of left turns, in vph, that can be made without an exclusive signal phase is computed. A cycle length of 80 seconds exists, with a 50/50 split.

Step 5. Develop Passenger Car Volumes. Using mixed vehicle volumes from Step 2, the passenger car volume, in pch, is computed for each movement. The percentage of trucks plus through buses is taken as constant for all movements on a given intersection approach.

$$PCV = HV + T(HV) + 4(LB)$$

An example of this calculation for approach 1 thru traffic is:

$$PCV = 1510 + (.02)\ (1510) + 4(6) = 1564$$

Step 6. Calculate Period Volumes. The period volume for each movement is computed, using the following formula:

$$PV = PCV/PHF$$

The peak hour factor for each approach is given on the form.

Step 7. Turn Adjustments. Tables 3 and 4 are used to determine the PCE values associated with left and right turns, respectively. The opposing volume, in vph, and the pedestrian volume, in pedestrians per hour, are taken from Step 2.

For example, there is a period volume of 79 right turns from approach 3 (step 6), and the adjustment factor for a crosswalk having 250 pedestrians per hour (from Table 4) is 1.25. This results in a "RT vol. in PCH" of 79 x 1.25 = 99.

Step 8. Adjusted Volumes. The total PCV volume (in pch) developed for each movement in Step 7 is adjusted for lane utilization and for lane width. The adjustment factors are based on information from Step 1 and Tables 2 and 5.

Step 9. Calculate Lane Volumes. The PCV volume, in pch, for each movement from Step 8 is divided by the number of lanes available for that movement. Step 9a on the form is used.

Step 10. Sum of Critical Lane Volumes. Using Table 9 and Figure 4 as guides, the sum of critical volumes on each of the two intersecting streets is obtained:

Step 11. Intersection Level of Service. The sum of 1606 pch falls in the range of 1601 to 1800 which Table 6 indicates to be Level of Service E for a two phase signal.

$$\text{Sum} = (B2 \text{ or } B1 \text{ or } A1 \text{ or } A2) + (A3B4 \text{ or } A4B3)$$
$$= A1 + A4B3 = 1016 + 588 = 1604$$

Step 12. Recalculate. To determine the effects of adding 11 foot (3.4 m) wide left turn lanes to Approaches 3 and 4, Steps 1 through 11 are repeated.

Table 6. Level of Service Ranges

PLANNING Applications (in vph) (deleted)			
OPERATIONS AND DESIGN Applications (in pch)			
Level of Service	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	1000	950	900
B	1200	1140	1080
C	1400	1340	1270
D	1600	1530	1460
E	1800	1720	1650
F	---------not applicable---------		

Comment

Note that in the PLANNING application, Example 1 resulted in a Level of Service D for these conditions. This illustrates the point that application of specific adjustment factors may result in a different answer when compared with the simplified PLANNING application, which is based on typical average-to-good urban conditions.

(Continued)

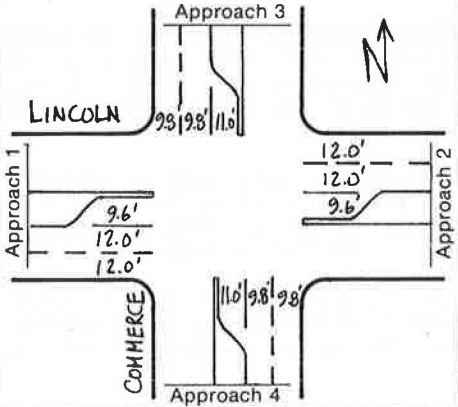
Critical Movement Analysis: OPERATIONS AND DESIGN
Calculation Form 2

Example 1
(Recalculation)

Intersection LINCOLN AND COMMERCE **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND CHANGE IN LOS BY ADDING LT LANES

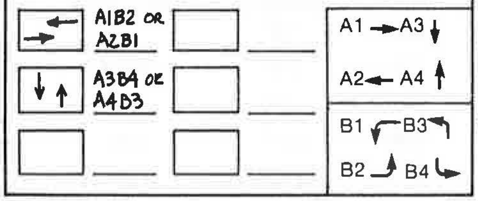
Step 1. Identify Lane Geometry



Step 2. Identify Hourly Volumes (HV) in vph

Approach 3: RT = 60, TH = 270, LT = 90; T = 5%, LB = 0
RT = 70, TH = 840, LT = 40
Approach 1: T = 2%, LB = 6
Approach 2: T = 2%, LB = 6
Pedestrians: 50, 250, 500, 60
LT = 50, TH = 1510, RT = 80
Approach 4: T = 5%, LB = 0
LT = 120, TH = 510, RT = 20

Step 3. Identify Phasing



Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour	45	45	45	45
b. Left turn capacity on change interval, in vph	90	90	90	90
c. G/C Ratio	.50	.50	.50	.50
d. Opposing volume in vph	910	1590	530	330
e. Left turn capacity on green, in vph	0	0	70	270
f. Left turn capacity in vph (b + e)	90	90	160	360
g. Left turn volume in vph	50	40	90	120
h. Is volume > capacity (g > f)?	NO	NO	NO	NO

Step 5. Develop Passenger Car Volumes (PCV) in pch

Approach 3: RT = 63, TH = 284, LT = 95
RT = 71, TH = 881, LT = 41
Approach 1
Approach 2
LT = 51, TH = 1564, RT = 82
Approach 4: LT = 126, TH = 536, RT = 21

Step 6. Calculate Period Volumes (PV) in pch

Approach 3: PHF = 0.80, RT = 79, TH = 355, LT = 119
PHF = 0.93, RT = 76, TH = 947, LT = 44
Approach 1
Approach 2
PHF = 0.85, LT = 60, TH = 1940, RT = 96
Approach 4: PHF = 0.80, LT = 158, TH = 670, RT = 26

Step 7. Turn Adjustments

Approach	1		2		3		4	
Movement	B2	A1	B1	A2	B4	A3	B3	A4
Turn	LT	RT	LT	RT	LT	RT	LT	RT
Turn volume (PV from Step 6)	60	96	44	76	119	79	158	26
Opposing vol. in vph from Step 2	910	-	1590	-	530	-	330	-
Ped. vol/hour	-	60	-	50	-	250	-	500
PCE LT from Table 3	4.0	-	6.0	-	2.0	-	2.0	-
LT vol. in pch	240	-	264	-	236	-	316	-
PCE RT from Table 4	-	1.00	-	1.00	-	1.25	-	1.25
RT vol. in pch	-	96	-	76	-	99	-	33
TH vol. in pch from Step 6	-	1940	-	947	-	355	-	670
Total PCV in pch	240	1936	264	1023	236	454	316	703

Step 8. Adjusted Volumes / Step 9a. Calculate Lane Volumes

Movement	Total PCV (Step 7)	U	W	Adjusted PCV (U×W×PCV)	No. of Lanes	PCV per Lane
B2	240	1.00	1.10	264	1	264
B1	264	1.00	1.10	290	1	290
A1	1936	1.05	1.00	2033	2	1016
A2	1023	1.05	1.00	1074	2	537
B4	236	1.00	1.00	236	1	236
B3	316	1.00	1.00	316	1	316
A3	454	1.05	1.10	524	2	262
A4	703	1.05	1.10	812	2	406

Step 9b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in pch	Volume Carryover to next phase	Adjusted Critical Volume in pch
2φ			

Step 10. Sum of Critical Volumes

(B2 OR B1 OR A1 OR A2) + (B4 OR B3 OR A3 OR A4)
1016 (A1) + 406 (A4) + ____ + ____
= 1422 pch

Step 11. Intersection Level of Service

(compare Step 10 with Table 6)

D

Step 12. Recalculate

NOT NECESSARY
Geometric Change ____
Signal Change ____
Volume Change ____

Comments

(Example 1)

Note: "(R)" denotes a recalculation.

Step 1(R). Identify Lane Geometry. Assumed lane configuration is shown on the form.

Step 2(R). Identify Hourly Volumes. Identical to original Step 2.

Step 3(R). Identify Phasing. Identical to original Step 3.

Step 4(R). Left Turn Check. Identical to original Step 4.

Step 5(R). Develop Passenger Car Volumes. Identical to original Step 5.

Step 6(R). Calculate Period Volumes. Identical to original Step 6.

Step 7(R). Turn Adjustments. On approaches 3 and 4, movements B4 and B3 are carried in exclusive left turn lanes. Otherwise, the adjustments are identical to original Step 7.

Step 8(R). Adjusted Volumes. With the addition of the 11.0 foot (3.4 m) turn lanes to Approaches 3 and 4, movements B4 and B3 will have specific values of U and W. Therefore calculations of adjusted volumes, in pch, will be separated by movement (i.e., A3, A4, B3, B4).

Step 9(R). Calculate Lane Volumes. The adjusted volume from Step 8 is divided by the number of lanes available for each movement. Step 9a on the form is used.

Step 10(R). Sum of Critical Volumes. Using Table 9 and Figure 4 as guides, the sum of critical volumes on each of the two intersecting streets is obtained.

Step 11(R). Intersection Level of Service. Using Table 6, the sum of 1423 pch falls in the range of 1401 to 1600 which implies a Level of Service **D** for a two phase signal.

Table 6. Level of Service Ranges

PLANNING Applications (in vph)

(deleted)

OPERATIONS AND DESIGN Applications (in pch)

Level of Service	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	1000	950	900
B	1200	1140	1080
C	1400	1340	1270
[✓ D]	1600	1530	1460
E	1800	1720	1650
F	---------not applicable---------		

Step 12(R). Recalculate. No recalculation is necessary. The solution is as follows:

1. Intersection level of service, with existing geometry, is **E** (See Step 11).

2. Left turns can be accommodated, with no special signal phasing, on all four approaches--see Step 4.

3. The level of service changes to **D** when left turn lanes are added on Approaches 3 and 4--see Step 11(R).

Comment

Note that in the PLANNING application, Example 1 (Recalculation) resulted in a Level of Service **C** for these conditions. This illustrates the point that application of specific adjustment factors may result in a different answer when compared with the simplified PLANNING application, which is based on typical average-to-good urban conditions.

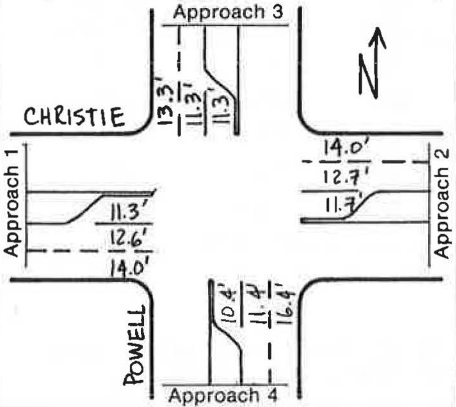
Critical Movement Analysis: OPERATIONS AND DESIGN
Calculation Form 2

Example 2

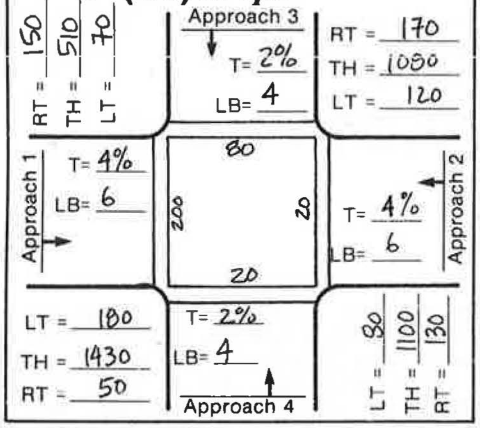
Intersection CHRISTIE AND POWELL **Design Hour** 4:30-5:30 p.m.

Problem Statement WILL 2 ϕ ACCOMODATE LEFT TURNS?

Step 1. Identify Lane Geometry



Step 2. Identify Hourly Volumes (HV) in vph



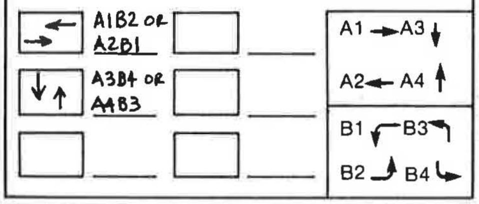
Approach 3: RT = 150, TH = 510, LT = 70, T= 2%, LB= 4

Approach 2: RT = 170, TH = 1080, LT = 120, T= 4%, LB= 6

Approach 1: T= 4%, LB= 6, LT = 180, TH = 1430, RT = 50

Approach 4: T= 2%, LB= 4, LT = 80, TH = 1100, RT = 130

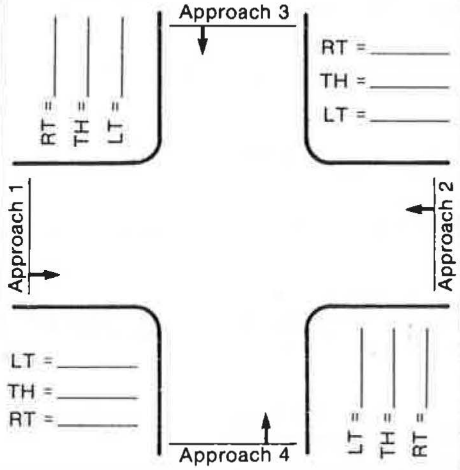
Step 3. Identify Phasing



Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour	50	50	50	50
b. Left turn capacity on change interval, in vph	100	100	100	100
c. G/C Ratio	.55	.55	.45	.45
d. Opposing volume in vph	1250	1480	1230	660
e. Left turn capacity on green, in vph	0	0	0	0
f. Left turn capacity in vph (b + e)	100	100	100	100
g. Left turn volume in vph	180	120	70	80
h. Is volume > capacity (g > f)?	YES	YES	NO*	NO*

Step 5. Develop Passenger Car Volumes (PCV) in pch



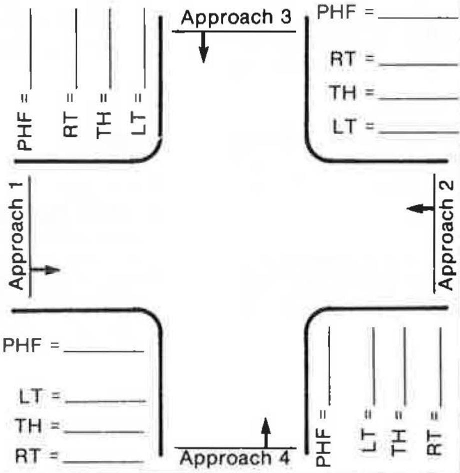
RT = ______ TH = ______ LT = ______

RT = ______ TH = ______ LT = ______

LT = ______ TH = ______ RT = ______

LT = ______ TH = ______ RT = ______

Step 6. Calculate Period Volumes (PV) in pch



PHF = ______ RT = ______ TH = ______ LT = ______

PHF = ______ RT = ______ TH = ______ LT = ______

PHF = ______ LT = ______ TH = ______ RT = ______

PHF = ______ LT = ______ TH = ______ RT = ______

Step 7. Turn Adjustments

Approach
Movement
Turn
Turn volume (PV from Step 6)
Opposing vol. in vph from Step 2
Ped. vol/hour
PCE LT from Table 3
LT vol. in pch
PCE RT from Table 4
RT vol. in pch
TH vol. in pch from Step 6
Total PCV in pch

Step 8. Adjusted Volumes / *Step 9a. Calculate Lane Volumes*

Movement	Total PCV (Step 7)	U	W	Adjusted PCV (U×W×PCV)	No. of Lanes	PCV per Lane

Step 9b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in pch	Volume Carryover to next phase	Adjusted Critical Volume in pch

Step 10. Sum of Critical Volumes

______ + ______ + ______ + ______

= ______ pch

Step 11. Intersection Level of Service

(compare Step 10 with Table 6)

☐

Step 12. Recalculate

Geometric Change ______

Signal Change ______

Volume Change ______

Comments * 2ϕ INADEQUATE, TRY 5ϕ

OPERATIONS AND DESIGN: Example 2

Problem

An existing urban intersection is being analyzed for possible changes in signalization and/or geometry. Lane configuration and peak hour volumes are known. Additionally, information is known about vehicle mix, peak hour factors, turns and lane widths.

The following questions are to be answered:

1. Is a two phase signal adequate to accommodate left turns? If not, what phasing will accommodate them?

2. What is the existing level of service for the selected phasing?

Step 1. Identify Lane Geometry. Existing lane configuration and lane widths are shown on the form.

Step 2. Identify Hourly Volumes. Existing peak hour volumes (in vph) are shown. Also, information on trucks plus through buses (T) and local buses (LB) is given. A pedestrian volume in pedestrians per hour is given for each of the four crosswalks.

Step 3. Identify Phasing. A two phase signal exists. Movements are identified according to information given in Figure 4.

Step 4. Left Turn Check. The number of left turns, in vph, that can be made without an exclusive signal phase is computed. A cycle length of 72 seconds exists with a 55/45 (Approaches 1 and 2/Approaches 3 and 4) split.

For Approaches 1 and 2 the demand exceeds the capacity, whereas the demand is less than the capacity for Approaches 3 and 4. A two phase signal is therefore inadequate and a recalculation must be done with a five phase signal.

Critical Movement Analysis: OPERATIONS AND DESIGN

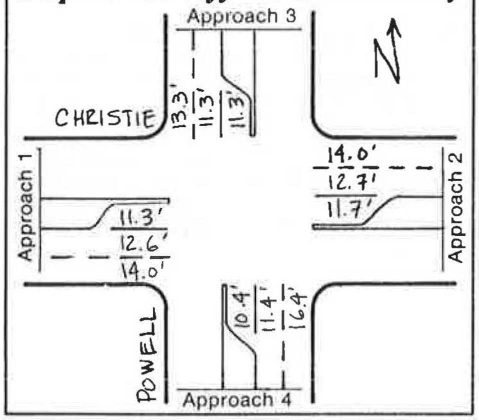
Example 2
(Recalculation)

Calculation Form 2

Intersection CHRISTIE AND POWELL **Design Hour** 4:30 - 5:30 p.m.

Problem Statement FIND LOS

Step 1. Identify Lane Geometry



Step 2. Identify Hourly Volumes (HV) in vph

Approach 3: RT = 150, TH = 510, LT = 70; T = 2%, LB = 4

Approach 2: RT = 170, TH = 1090, LT = 120; T = 4%, LB = 6

Approach 1: T = 4%, LB = 6; LT = 180, TH = 1430, RT = 50

Approach 4: T = 2%, LB = 4; LT = 80, TH = 1100, RT = 130

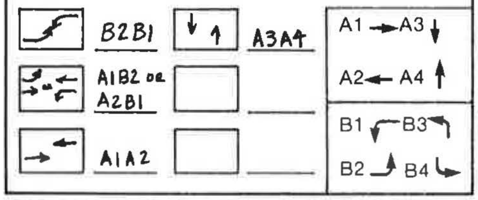
80

200

20

20

Step 3. Identify Phasing



*Step 4. Left Turn Check**

	Approach 1	2	3	4
a. Number of change intervals per hour			50	50
b. Left turn capacity on change interval, in vph			100	100
c. G/C Ratio			.40	.40
d. Opposing volume in vph			1230	660
e. Left turn capacity on green, in vph			0	0
f. Left turn capacity in vph (b + e)			100	100
g. Left turn volume in vph			70	80
h. Is volume > capacity (g > f)?			No	No

Step 5. Develop Passenger Car Volumes (PCV) in pch

Approach 3: RT = 162, TH = 520, LT = 71

Approach 2: RT = 201, TH = 1123, LT = 125

Approach 1: LT = 187, TH = 1497, RT = 76

Approach 4: LT = 82, TH = 1122, RT = 149

Step 6. Calculate Period Volumes (PV) in pch

Approach 3: PHF = 0.91, RT = 186, TH = 571, LT = 78

Approach 2: PHF = 0.92, RT = 218, TH = 1221, LT = 136

Approach 1: PHF = 0.92, LT = 203, TH = 1616, RT = 83

Approach 4: PHF = 0.87, LT = 94, TH = 1290, RT = 171

Step 7. Turn Adjustments

Approach	1		2		3		4	
Movement	B2	A1	B1	A2	B4	A3	B3	A4
Turn	LT	RT	LT	RT	LT	RT	LT	RT
Turn volume (PV from Step 6)	203	83	136	218	78	186	94	171
Opposing vol. in vph from Step 2	1250	-	1480	-	1230	-	660	-
Ped. vol/hour	-	20	-	80	-	200	-	20
PCE LT from Table 3	1.05	-	1.05	-	6.0	-	4.0	-
LT vol. in pch	213	-	143	-	468	-	376	-
PCE RT from Table 4	-	1.00	-	1.00	-	1.25	-	1.00
RT vol. in pch	-	83	-	218	-	233	-	171
TH vol. in pch from Step 6	-	1616	-	1221	-	571	-	1290
Total PCV in pch	213	1699	143	1439	468	804	376	1461

Step 8. Adjusted Volumes / *Step 9a. Calculate Lane Volumes*

Movement	Total PCV (Step 7)	U	W	Adjusted PCV (U×W×PCV)	No. of Lanes	PCV per Lane
B2	213	1.00	1.00	213	1	213
B1	143	1.00	1.00	143	1	143
A1	1699	1.05	0.90	1606	2	803
A2	1439	1.05	0.90	1360	2	680
B4	468	1.00	1.00	468	1	468
B3	376	1.00	1.00	376	1	376
A3	804	1.05	1.00	844	2	422
A4	1461	1.05	0.90	1381	2	690

Step 9b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in pch	Volume Carryover to next phase	Adjusted Critical Volume in pch
B2B1	143(B1)	213-143=70(B2)	143
A1B2	70(B2)	803-70=733(A1)	70
A1A2	733(A1) OR 680(A2)		733
A3A4	468(B4) OR 376(B3) OR 422 (A3) OR 691 (A4)		690

LT from 1

Step 10. Sum of Critical Volumes

213 + 733 + 690 + ____ = 1636 pch

Step 11. Intersection Level of Service

(compare Step 10 with Table 6)

E

Step 12. Recalculate

NOT NECESSARY

Geometric Change ____

Signal Change ____

Volume Change ____

Comments *NEED ONLY CHECK UNPROTECTED PHASES.

(Example 2)

Note: "(R)" denotes recalculation.

Step 1(R). Identify Lane Geometry. Identical to original Step 1.

Step 2(R). Identify Volumes. Identical to original Step 2.

Step 3(R). Identify Phasing. A five phase signal is to be analyzed with protected phases for Approaches 1 and 2.

Step 4(R). Left Turn Check. The estimated G/C ratio is amended to 0.40 (Approaches 3 and 4), 0.40 (Approaches 1 and 2), and 0.20 for left turns (Approaches 1 and 2). Approaches 3 and 4 do not have exclusive phases, therefore a left turn check of these approaches must be made. This estimated split of the cycle time is adequate for determining item a in left turn check.

Step 5. Develop Passenger Car Volumes. Using mixed vehicle volumes from Step 2, the passenger car volume, in pch, is computed for each movement. The percentage of trucks plus through buses is taken as constant for all movements on a given intersection approach.

Step 6. Calculate Period Volumes. The period volume for each movement is computed in pch. The peak hour factor for each approach is given on the form.

Step 7. Turn Adjustments. Tables 3 and 4 are used to determine the PCE values associated with left and right turns, respectively. The opposing volume in vph and the pedestrian volume, in pedestrians per hour are taken from Step 2.

Step 8. Adjusted Volumes. The total PCV volume in pch developed for each movement in Step 7 is adjusted for lane utilization and lane width. The adjustment factors are based on information from Step 1 and Tables 2 and 5.

The average of both through plus right lanes on all approaches was used to arrive at a value for W. For example, in A1 the average of 12.6 and 14.0 is 13.3, therefore W = 0.90 was used.

Step 9. Calculate Lane Volumes. Lane volumes, in pch, are determined by dividing the adjusted volumes from Step 8 by the number of lanes available.

Table 6. Level of Service Ranges

PLANNING Applications (in vph)

(deleted)

OPERATIONS AND DESIGN Applications (in pch)

Level of Service	Maximum Sum of Critical Volumes		
	Two Phase	Three Phase	Four or more Phases
A	1000	950	900
B	1200	1140	1080
C	1400	1340	1270
D	1600	1530	1460
[✓ E]	1800	1720	1650
F	---------not applicable---------		

Using Step 3(R), the phase sequence which most likely will appear, with lane volumes of Step 9a, is as follows: B2B1, A1B2, A1A2, and A3A4. For example, since left turn volume from Approach 2 (B1) in Step 8 is less than left turn volume from Approach 1 (B2), A1B2 is more probable than A2B1.

Using the most probable phase sequence, the through plus right turn volume which moves during a left arrow is subtracted from the total through plus right volume, and the remaining volume is carried over to the next phase.

Step 10. Sum of Critical Volumes. Using Table 9 and Figure 4 as guides, the "critical volume," in pch, for each phase is obtained. From Step 9, the sum of the critical volumes is:

Sum = B2 + (A1 or A2) + (B4 or B3 or A3 or A4)

= B2 + A1 + A4 = 213 + 733 + 690 = 1636 pch

Step 11. Intersection Level of Service. The sum 1636 pch falls in the range of 1461 to 1650, which Table 6 indicates as Level of Service E for a five phase signal.

Step 12. Recalculate. No further recalculation is necessary. The solution is as follows:

1. A two phase signal is not adequate to handle left turns from Approaches 1 and 2. A five phase signal with exclusive left turn phases for these approaches will accommodate the left turns.

2. The intersection level of service with a five phase signal is E.

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(15) Berry, D. S., "Other Methods for Computing Capacity of Signalized Intersections," Paper for 56th Annual Meeting of the TRB, Washington, D.C., Jan. 1977, vp.

(16) Normann, O. K., "Variations in Flow at Intersections as Related to Size of City, Type of Facility and Capacity Utilization," Highway Research Board Bulletin-352, Washington, D.C., HRB, 1962, pp. 55-99.

(17) Reilly, W. R., Gardner, C. C., and Kell J. H., A Technique for Measurement of Delay at Intersections, San Francisco, CA., and Tucson, AZ., JHK & Associates, Sept 1976, 3 Vols.

Unsignalized Intersection Capacity Calculation Form

Intersection ______________________________

Location Plan:

D

A

B

C

Counts:
Date ____________
Day ____________
Time ____________
Control ____________
Prevailing Speed ____________

Hourly Demand Traffic Volumes from _____ to _____, _____ m

Approach	A ←→			B →←			C ↘↑↙			D ↗↓↖		
Movement	A_L ↗	A_T →	A_R ↘	B_L ↖	B_T ←	B_R ↙	C_L ↖	C_T ↑	C_R ↗	D_L ↘	D_T ↓	D_R ↙
Volume												
pch (see Table 1)		▨	▨		▨	▨						

Step 1 **Right Turn from C/D**	C_R ↗	D_R ↙
Conflicting Flows = M_H =	$\frac{1}{2} A_R + A_T =$	$\frac{1}{2} B_R + B_T =$
(from Fig. 1)	_____ + _____ = _____ vph	_____ + _____ = _____ vph
Critical Gap from Table 2 T_g =	_____ sec	_____ sec
Capacity from Fig. 2 =	$M_{No} = M_1 =$ _____ pch	$M'_{No} = M'_1 =$ _____ pch
Demand =	$C_R =$ _____ pch	$D_R =$ _____ pch
Capacity Used =	$100 (C_R/M_1) =$ _____ %	$100 (D_R/M'_1) =$ _____ %
Impedance Factor from Fig. 3 =	$P_1 =$ _____	$P'_1 =$ _____
_____ Shared Lane – See Step 3		
_____ No Shared Lane – Available Reserve	$M_1 - C_R =$ _____ pch	$M'_1 - D_R =$ _____ pch
Delay & Level of Service (Table 3)	____________ ☐	____________ ☐
Step 2 **Left Turn from B/A**	B_L ↖	A_L ↗
Conflicting Flows = M_H =	$A_R + A_T =$	$B_R + B_T =$
(from Fig. 1)	_____ + _____ = _____ vph	_____ + _____ = _____ vph
Critical Gap from Table 2 T_g =	_____ sec	_____ sec
Capacity from Fig. 2 =	$M_{No} = M_2 =$ _____ pch	$M'_{No} = M'_2 =$ _____ pch
Demand =	$B_L =$ _____ pch	$A_L =$ _____ pch
Capacity Used =	$100 (B_L/M_2) =$ _____ %	$100 (A_L/M'_2) =$ _____ %
Impedance Factor from Fig. 3 =	$P_2 =$ _____	$P'_2 =$ _____
Available Reserve =	$M_2 - B_L =$ _____ pch	$M'_2 - A_L =$ _____ pch
Delay & Level of Service (Table 3)	____________ ☐	____________ ☐
Step 3 **Thru Movement from C/D**	C_T ↑	D_T ↓
Conflicting Flows = M_H =	$\frac{1}{2}A_R + A_T + A_L + B_L + B_T + B_R$	$\frac{1}{2}B_R + B_T + B_L + A_L + A_T + A_R$
(from Fig. 1)	___ + ___ + ___ + ___ + ___ + ___	___ + ___ + ___ + ___ + ___ + ___
(M_T & M'_T are used in Step 4)	$M_H = M_T =$ _____ vph	$M_H = M'_T =$ _____ vph
Critical Gap from Table 2 T_g =	_____ sec	_____ sec
Capacity from Fig. 2 =	$M_{No} =$ _____ pch	$M'_{No} =$ _____ pch
Adjust for Impedance	$M_{No} \times P_2 \times P'_2 = M_3 =$ _____ pch	$M'_{No} \times P'_2 \times P_2 = M'_3 =$ _____ pch
Demand =	$C_T =$ _____ pch	$D_T =$ _____ pch
Capacity Used =	$100 (C_T/M_3) =$ _____ %	$100 (D_T/M'_3) =$ _____ %
Impedance Factor from Fig. 3	$P_3 =$ _____	$P'_3 =$ _____

Unsignalized Intersection Capacity Calculation Form (continued)

Step 3 (Continued)	C_T ↑	D_T ↓
____ No Shared Lane Available Reserve = Delay & Level of Service (Table 3)	 $M_3 - C_T =$ ____ pch ____ ☐	 $M_3' - D_T =$ ____ pch ____ ☐
____ Shared Lane with Left Turn See Step 4		
Shared Lane Demand = ____ Shared Lane with Right Turn Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_R + C_T = C_{RT} =$ ____ pch $M_{13} = \frac{(C_R + C_T)}{(C_R/M_1) + (C_T/M_3)}$ $M_{13} =$ ____ pch $M_{13} - C_{RT} =$ ____ pch ____ ☐	$D_R + D_T = D_{RT} =$ ____ pch $M_{13}' = \frac{(D_R + D_T)}{(D_R/M_1') + (D_T/M_3')}$ $M_{13}' =$ ____ pch $M_{13}' - D_{RT} =$ ____ pch ____ ☐
Step 4 **Left Turn from C/D**	C_L ↰	D_L ↲
Conflicting Flows = M_H = (M_T & M_T' were calculated in Step 3) Critical Gap from Table 2 T_g = Capacity from Fig. 2 = Adjust for Impedance	$M_T + D_T + D_R =$ ____ + ____ + ____ = ____ vph ____ sec $M_{No} =$ ____ pch $M_{No} \times P_2 \times P_2' \times P_1' \times P_3' = M_4$ $M_4 =$ ____ pch	$M_T' + C_T + C_R =$ ____ + ____ + ____ = ____ vph ____ sec $M_{No}' =$ ____ pch $M_{No}' \times P_2' \times P_2 \times P_1 \times P_3 = M_4'$ $M_4' =$ ____ pch
____ No Shared Lane Demand = Available Reserve = Delay & Level of Service (Table 3)	$C_L =$ ____ pch $M_4 - C_L =$ ____ pch ____ ☐	$D_L =$ ____ pch $M_4' - D_L =$ ____ pch ____ ☐
Shared Lane Demand = ____ Shared Lane with Thru Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_T + C_L = C_{TL} =$ ____ pch $M_{34} = \frac{(C_T + C_L)}{(C_T/M_3) + (C_L/M_4)}$ $M_{34} =$ ____ pch $M_{34} - C_{TL} =$ ____ pch ____ ☐	$D_T + D_L = D_{TL} =$ ____ pch $M_{34}' = \frac{D_T + D_L}{(D_T/M_3') + (D_L/M_4')}$ $M_{34}' =$ ____ pch $M_{34}' - D_{TL} =$ ____ pch ____ ☐
Shared Lane Demand = ____ Shared Lane with Thru & Right Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_R + C_T + C_L = C_{RTL} =$ ____ pch $M_{134} = \frac{C_R + C_T + C_L}{(C_R/M_1) + (C_T/M_3) + (C_L/M_4)}$ $M_{134} =$ ____ pch $M_{134} - C_{RTL} =$ ____ pch ____ ☐	$D_R + D_T + D_L = D_{RTL} =$ ____ pch $M_{134}' = \frac{D_R + D_T + D_L}{(D_R/M_1') + (D_T/M_3') + (D_L/M_4')}$ $M_{134}' =$ ____ pch $M_{134}' - D_{RTL} =$ ____ pch ____ ☐

Overall Evaluation ____________________

____

____

Unsignalized "T" Intersection Capacity Calculation Form

Intersection ______________________________

Location Plan:

Counts:
Date ____________
Day ____________
Time ____________
Control ____________
Prevailing Speed ____________

A **B**

C

Hourly Demand Traffic Volumes from ______ to ______ , ______ m

Approach	A ↘→		B ←↙		C ↖↗	
Movement	A_T →	A_R ↘	B_L ↙	B_T ←	C_L ↖	C_R ↗
Volume						
pch (see Table 1)						

Step 1	**Right Turn from C**	C_R ↗
	Conflicting Flows = M_H =	½ A_R + A_T =
	(from Fig. 1)	______ + ______ = ______ vph
	Critical Gap from Table 2 T_g =	______ sec
	Capacity from Fig. 2 =	$M_{No} = M_1$ = ______ pch
______	Shared Lane – See Step 3	
______	No Shared Lane Demand =	C_R = ______ pch
	Available Reserve =	$M_1 - C_R$ = ______ pch
	Delay & Level of Service (Table 3)	______________ ☐
Step 2	**Left Turn from B**	B_L ↙
	Conflicting Flows = M_H =	A_R + A_T =
	(from Fig. 1)	______ + ______ = ______ vph
	Critical Gap from Table 2 T_g =	______ sec
	Capacity from Fig. 2 =	$M_{No} = M_2$ = ______ pch
	Demand =	B_L = ______ pch
	Capacity Used =	100 (B_L/M_2) = ______ %
	Impedance Factor from Fig. 3 =	P_2 = ______
	Available Reserve =	$M_2 - B_L$ = ______ pch
	Delay & Level of Service (Table 3)	______________ ☐
Step 3	**Left Turn from C**	C_L ↖
	Conflicting Flows = M_H =	½A_R + A_T + B_L + B_T =
	(from Fig. 1)	____ + ____ + ____ + ____ = ______ vph
	Critical Gap from Table 2 T_g =	______ sec
	Capacity from Fig. 2 =	M_{No} = ______ pch
	Adjust for Impedance	$M_{No} \times P_2 = M_3$ = ______ pch
______	No Shared Lane Demand =	C_L = ______ pch
	Available Reserve =	$M_3 - C_L$ = ______ pch
	Delay & Level of Service (Table 3)	______________ ☐
	Shared Lane Demand =	$C_R + C_L = C_{RL}$ = ______ pch
______	Shared Lane with Right Turn Capacity of Shared Lane =	$M_{13} = \dfrac{(C_R + C_L)}{(C_R/M_1) + (C_L/M_3)}$ M_{13} = ______ pch
	Available Reserve =	$M_{13} - C_{RL}$ = ______ pch
	Delay & Level of Service (Table 3)	______________ ☐

Overall Evaluation ______________________________

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USER APPLICATIONS

Introduction

To facilitate the calculation of capacities at unsignalized intersections, computational forms have been developed. These forms were designed to enable practitioners to apply the methodology previously discussed.

In order to introduce and explain the "Unsignalized Intersection Capacity Calculation Forms," this description of user applications will cover the following subject areas: (1) the concepts underlying the organization of the forms, (2) the definitions of terms used in the forms, and (3) three examples of how the forms are applied in measuring unsignalized intersection capacity.

Concepts

The basic concept of the analysis is a sequential evaluation of the individual flows in the intersection. It is assumed that the thru movement on the major street and the right turns from the major street are unimpeded and have the right of way over all side street traffic and left turns from the major street. If this is not the case and there is congestion on the major street such that side street vehicles cannot enter, the intersection is operating at Level of Service F. This is usually caused by a downstream condition that limits the flow through the intersection under consideration. The intersection cannot be analyzed for capacity until the downstream condition is ameliorated.

All other flows in the intersection cross, merge with, or are affected by other flows. The concept used in this procedure is to process the individual movements in a sequential manner. For each movement in turn, all conflicting flows are summed, and a critical gap is determined. A graphic solution provides the potential capacity of that movement. Adjustments are made for mutual interference to the individual streams, e.g., the additional adverse effect of main street vehicles waiting to make left turns on minor street vehicles waiting to cross the major street.

Consideration is first given to the right turns from the minor street. This maneuver is affected by the thru vehicles (in the curb lane only) coming from the left on the major street. It is also affected by right turns from the major street. Although these right turns do not directly conflict with traffic entering from the minor street, they do have an inhibiting effect on the minor street traffic unless a separate turning lane for this movement is present. To allow for this effect, a flow equal to one half of the right turns is added to the other conflicting flows. This value may be reduced or eliminated where large radius turning areas for the right turns and/or STOP or YIELD control of these turns is present.

Left turns from the major street are considered next. The ability to make this maneuver is affected by the opposing thru flow and the right turns from the opposing approach. Large radius curb returns or channelized right turn areas reduce or eliminate the effect of right turning vehicles.

Vehicles crossing the major street are then analyzed. The conflicting flows include all major street traffic except the near side right turns which are treated as described earlier. Impedances caused by major street left turning vehicles being delayed in making their turns are applied to reduce the potential capacity of the crossing flow.

The final flow to be analyzed is left turns from the minor street. In addition to all of the conflicts that crossing vehicles encounter, left turns also conflict with the thru and right turns from the opposing minor street approach. Impedances caused by these minor street conflicting flows as well as those caused by the major street left turns must be applied.

The above discussion has assumed that each minor street flow (right, thru, and left) has had one exclusive lane. Generally this is not the case. In fact, frequently all three maneuvers share the same lane. When a single lane must be shared by two or three different movements, there is bound to be interference.

The disadvantage of shared lanes is not important at intersections with wide approach areas or large radius corners, because vehicles can stop side by side at the intersection (e.g., a right turning vehicle can bypass a delayed crossing vehicle).

Where the vehicles are confined to a single lane, the shared lane capacity equation (see DISCUSSION, above) is used. The shared lane capacity is applicable only to a traffic stream that has the same proportion of movements as used in the equation. For example, if the existing flows are 100 thru and 60 left and the shared lane capacity is 240 this implies a stream flow of 150 thru and 90 right.

Once the capacities of the individual flows are determined they are compared to the existing or projected flows. A zero or negative difference indicates intersection failure and extreme congestion. A positive difference indicates an available reserve capacity, the magnitude of which determines Level of Service and expected delay.

Only those unsignalized intersections that are controlled by two-way STOP signs or by YIELD signs can be analyzed by these techniques. The procedure is not applicable to uncontrolled intersections or four-way STOP sign controlled intersections.

Definitions

The following terms are utilized in the calculation of unsignalized intersection capacity.

Available Reserve - The difference between capacity and present or future demand. It is used to define Level of Service and expected delay.

Capacity Used - The percentage of the capacity of a movement that is used by the existing or projected demand.

Conflicting Flows (M_H) - The sum of all existing or projected flows that conflict with the movement under consideration. These flows are in vehicles per hour. The specific elements of M_H are shown in Figure 1.

Critical Gap (T_g) - The minimum gap (in seconds) needed by drivers to negotiate the conflicting flows. Critical gaps suggested by the original developers for passenger cars are contained in Table 2. These critical gaps are dependent upon the intended maneuver, the type of control (STOP or YIELD), the major street prevailing speed, and the number of lanes on the major street.

Demand - The existing or projected traffic flow in the stream under consideration. Demand is given in passenger car equivalents per hour. Conversion factors are contained in Table 1.

Impedance (P) - The interference caused by a conflicting stream becoming congested thereby

DISCUSSION

The methodology described has been adapted from the OECD publication "Capacity of At-Grade Intersections"* which in turn is a translation of a German document.** Although this material has been available for a number of years, it has not been extensively applied in the United States. This chapter clarifies the methodology and presents a simplified computational procedure.

Only those unsignalized intersections that are controlled by two-way STOP signs or by YIELD signs can be analyzed by these techniques. The procedure is not applicable to uncontrolled intersections or four-way STOP sign controlled intersections.

The capacity or maximum flow of vehicles (M_{No}) in passenger car equivalents is calculated for each minor approach movement. These values are then compared to the existing demand for each movement and the probable delay and level of service is estimated.

The assumption is made that major street traffic is not affected by the minor street movements. Left turns from the major street to the minor street are influenced only by the opposing major street thru flow. Minor street flows, however, are impeded by all other conflicting movements. The methodology includes adjustments for mutual interference to the minor street traffic streams, e.g., the additional adverse effect of main street vehicles waiting to make left turns.

In order to treat these potential impedances, it is necessary to structure the computational procedures and deal with individual traffic movements in the following order:

1. Right turns into the major road;
2. Left turns from the major road;
3. Through traffic crossing the major road; and
4. Left turns into the major road.

In addition, the method takes into account the lane configuration on the minor street and has appropriate adjustments for movements that use the same lane (shared lane).

Calculations are based upon maximum hourly flows using the intersection at a given time. A given intersection may have to be tested for different time periods during the day to account for varying directional flows. Traffic flows at existing intersections are obtained from traffic counts to evaluate present conditions. For new intersections or to analyze future conditions, forecasts of the flows must be made.

The methodology requires a specific sequence of steps which is presented below.

The preliminary steps include the gathering of basic data. Required are: the general layout of the intersection; the number of lanes on the major street; the lane configuration on the minor street approaches; the type of contol (STOP or YIELD); and the prevailing speed on the major street. Left, right, and through volumes are required for each approach (this is, of course, simplified in the case of a "T" intersection). Minor street movements and left turn movements from the major street should be converted to passenger car equivalents per hour (pch) to account for approach grade and traffic mix. Table 1 provides estimated conversion factors.

Table 1. Converting Existing Traffic Flow to Passenger Car Equivalents per hour

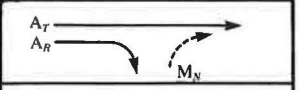
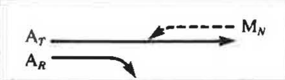
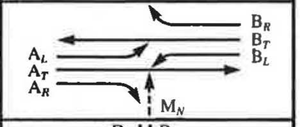
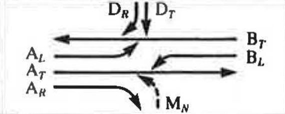
Grade / Type of Vehicle	-4%	-2%	0%	+2%	+4%
Motorcycles	0.3	0.4	0.5	0.6	0.7
Passenger Cars	0.8	0.9	1.0	1.2	1.4
Trucks/R.V.'s	1.0	1.2	1.5	2.0	3.0
Truck-Trailers	1.2	1.5	2.0	3.0	6.0
Motor Vehicle*	0.9	1.0	1.1	1.4	1.7

*Approximate value used for estimate calculations.

The computational methodology is described below in the sequence that is followed.

The conflicting individual traffic streams ( ) that must be considered in each step when evaluating the minor street are shown in Figure 1.

Figure 1. Definition of Conflicting Traffic Streams

Step 1	Right turns into major street	$M_H = ½A_R + A_T$	
Step 2	Left turns from major street	$M_H = A_R + A_T$	
Step 3	Crossing major street	$M_H = ½A_R + A_T + A_L + B_L + B_T + B_R$	
Step 4	Left turns into major street	$M_H = ½A_R + A_T + A_L + B_L + B_T + B_R + D_T + D_R$	

Note: In Step 1, if there is more than one lane on the major street, A_T is the flow in the curb lane only.
In Steps, 1, 3, and 4, if a turning lane is present for major street right turns, A_R can be omitted.
In Steps 2 and 3, large radius turning areas for right turns off the major street and/or STOP or YIELD control of these turns reduce or eliminate the effect of A_R and B_R.
For complementary movements, reverse the major street movements (A and B) and minor street movements (C and D).

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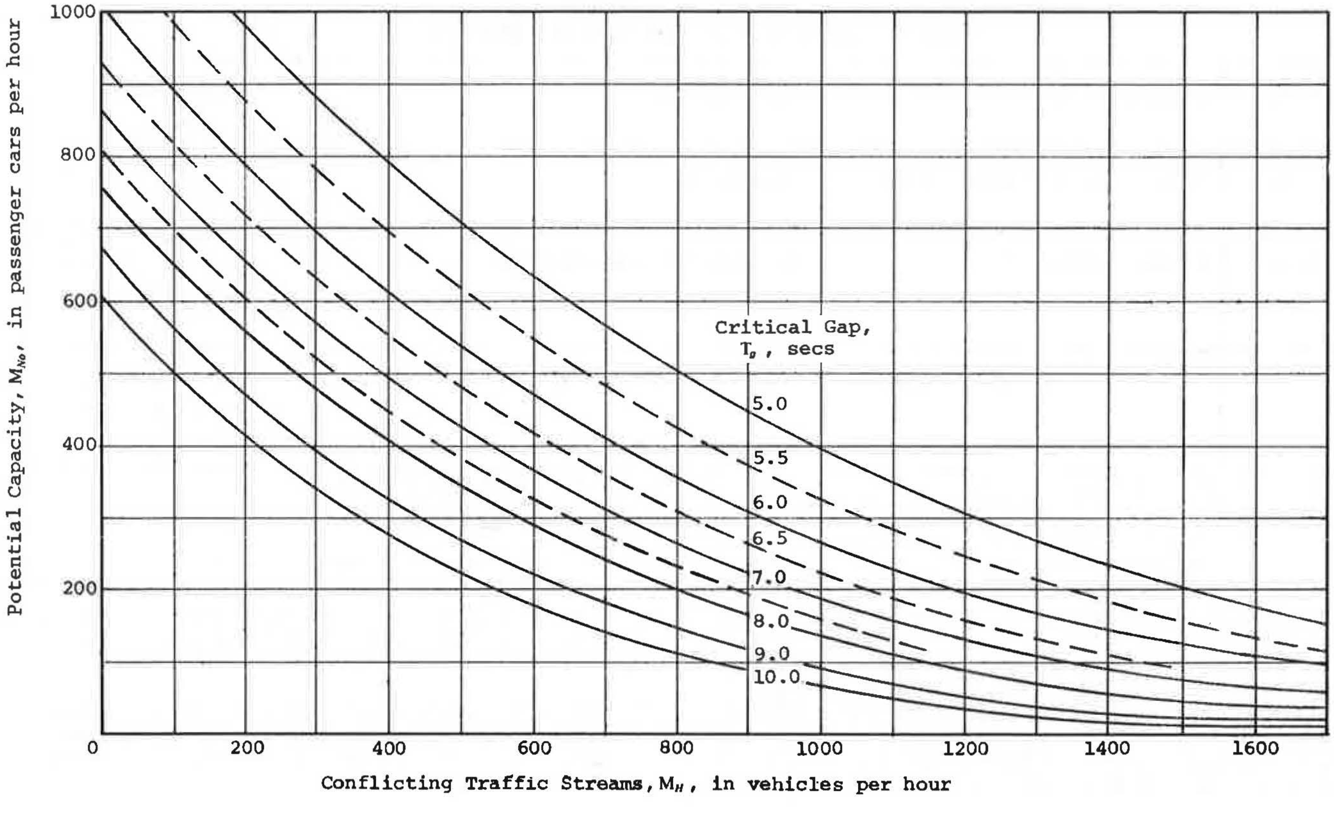
* Organisation for Economic Co-Operation and Development, "Capacity of At-Grade Junctions", Paris, 1974.

** Forschungsgesellschaft fur das Strassenwesen, "Merkblatt for Lichtsignalanlagen an Landstrassen, Ausgabe 1972", Forschungshesellschaft fur das Strassenwesen, Koln, 1972.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



In steps 1, 3, and 4, there is not a direct conflict between the movement under consideration (M_N) and the right turn from the major street (A_R). However, this right turning traffic does have an inhibiting effect on the minor street traffic. To allow for this effect, half of this flow is included in M_H. This influence diminishes where large radius curves or deceleration areas are available and disappears entirely when a separate turning lane is provided.

Conflicting traffic streams (M_H) are used as existing or forecast volumes in vehicles per hour and are not converted into passenger car equivalents. Bicycles on the major road are included if it is apparent that they cause restrictions to minor street movement.

Vehicles emerging from the minor road (and left-turning vehicles from the major road) can only do so if the available gaps in the conflicting streams are long enough for them to execute their desired maneuver. The critical gap can be used to describe the minimum gap required by drivers affected by the intersection.

Table 2 shows critical gaps which apply to passenger cars. These critical gaps are dependent upon the intended maneuver, the type of control (STOP or YIELD), the major road prevailing speed, and the number of lanes on the major street.

Once the relevant conflicting flow (M_H) and the critical gap (T_g) are determined for a given movement, the maximum capacity (M_{No}) is read from Figure 2. This maximum capacity is the largest flow that can be achieved from the minor movement into the intersection, assuming the following conditions prevail:

a) the traffic on the major road does not block the major road;

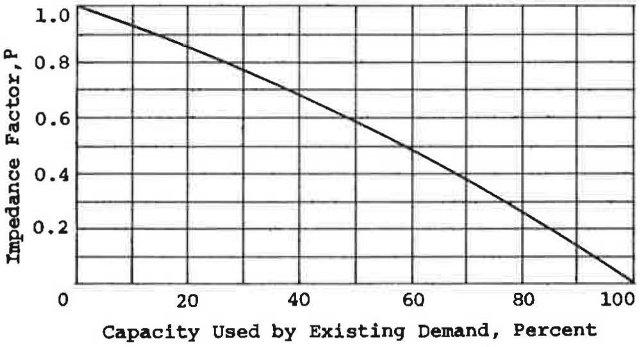
b) congestion at nearby intersections does not back up into the intersection under consideration; and

c) a separate lane is provided for the exclusive use of the minor street movement under consideration.

If these conditions exist, the actual capacity is the value obtained from Figure 2 for right turns from the minor road and left turns off the major road. Additional adjustments are necessary for crossing traffic and left turns from the minor road due to congestion interference. Further adjustments may also be required for shared lane conditions.

There is a possibility that traffic turning off the major road may become congested and interfere with minor road traffic. In the case of left turns off the minor road, congestion in the opposing through traffic and right turns may also have congestion interference. To compensate for this factor, the maximum capacity (M_{No}) is reduced. For this purpose, an impedence factor (P) is obtained for each relevant traffic stream from Figure 3. This value is applied to any minor road movement that is likely to be impeded by such congestion. The "P" value defines the probability that this minor road movement remains unaffected. The entry value in Figure 3 is the percent of capacity used or the ratio between the existing or forecast demand (in pch) of a potentially congesting flow and the capacity (M_{No}) of that stream, expressed as a percentage (i.e., $100[\ B_L/M_2\ ]$).

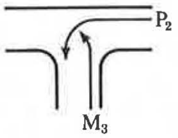
Figure 3. Capacity Reduction caused by Congestion



The following equations show how the basic capacities for each vehicular movement are to be reduced.

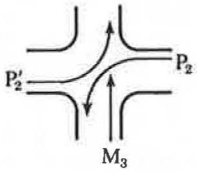
1. Left turns into the major street at a "T" intersection:

$$M_3 = M_{No} \times P_2$$



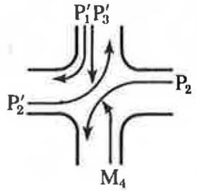
2. Thru traffic crossing the major street at a 4-way intersection:

$$M_3 = M_{No} \times P_2 \times P_2'$$



3. Left turns into the major street at a 4-way intersection:

$$M_4 = M_{No} \times P_2 \times P_2' \times P_1' \times P_3'$$



The procedures up to this point have assumed that each minor flow has an exclusive lane for that particular movement. This is frequently not the case, as often two or more movements must share a lane. There is bound to be interference between the movements sharing the lane. This interference may be reduced or eliminated at intersections with large radius corners, because two or more vehicles can stop side by side. In such cases, the adverse effect of the shared lane can be neglected.

If the shared lane on the minor approach continues as a single lane up to the near edge of the major street, as is the case where small curb returns are used, the capacity of the shared lane can be determined by the following equation:

$$\frac{1}{M_{134}} = \frac{x}{M_1} + \frac{y}{M_3} + \frac{z}{M_4}$$

where:

M_{134} = Capacity of all streams using the shared lane

x,y,z = Proportion of right, thru, and left movements, respectively

M_1, M_3, M_4 = Capacity of the right, thru, and left individual streams, respectively

Note: Only those movements included in the shared lane are included in the computation (e.g., if the shared lane is right turning and thru vehicles, only the first two terms are used and the last term is omitted).

A simpler computational form that can readily be set up on a programmable calculator is as follows:

$$M_{134} = \frac{C_R + C_T + C_L}{(C_R/M_1) + (C_T/M_3) + (C_L/M_4)}$$

where:

C_R, C_T, C_L = Demand of the right, thru, and left movements, respectively

A sample program for a Hewlett-Packard model 33E programmable calculator is presented in an appendix to this section.

The calculated capacity is then compared with the existing or projected traffic demand. This requires the conversion of the existing (or projected) minor approach traffic streams (e.g., C_T) into passenger car equivalents (pch). If there are appreciable grades on the minor street approaches, the gradient effect must also be considered. Conversion factors are contained in Table 1. Bicycles, if considered, are treated like motorcycles.

If the existing or projected traffic demand is greater than the calculated capacity, a failure or breakdown condition occurs. This is not Level of Service F. Level of Service F occurs when the major street traffic backs up from a downstream condition and blocks the minor street such that the minor street vehicles cannot enter the intersection.

A minor street approach operating at or near capacity has very long traffic delays and lengthly queues. These conditions can be tolerated only if they are relatively rare occurrences.

The difference between the capacity figure and the existing or projected flows is defined as the reserve capacity. Traffic delay and the resulting Level of Service are directly related to the magnitude of the reserve capacity. Suggested ranges of reserve capacities for the various levels of service are shown in Table 3.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays
0 to 99	E	Very Long traffic delays
Less than 0	E	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

The reserve capacity concept applies only to an individual stream (or shared lane stream) and assumes that all these streams remain constant. Even in a shared lane, the reserve capacity assumes the same proportion as the current contributing streams. The summation of individual reserve capacities for the various movements is incorrect and should not be done.

Once the capacity of all of the individual movements have been calculated and their Levels of Service and expected delay determined, an overall evaluation of each minor street approach or the total minor street is made. Normally, the worst condition or Level of Service defines the overall evaluation, but this may be tempered by engineering judgement.

Computational forms for unsignalized "T" and 4-way intersections are shown on the following pages.

blocking the intended maneuver. It is based upon the percentage of capacity used by the conflicting stream and is obtained from Figure 3.

M_H - See Conflicting Flows

M_{No} - See Potential Capacity

P - See Impedance

pch - passenger car equivalents per hour (See Table 1 for conversion factors).

Potential Capacity (M_{No}) - The capacity obtained from Figure 2 before adjusting for impedances and/or shared lanes. It is in terms of passenger car equivalents per hour.

Shared Lane Capacity - The result of the combining equation which relates flows and exclusive lane capacities to determine the capacity of the shared lane.

Shared Lane Demand - The sum of the individual flows using the shared lane in pch.

T_g - See Critical Gap

vph - vehicles per hour

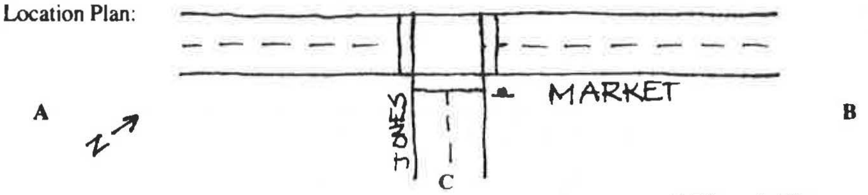
Following are three examples which display the capacity calculation procedure. The first of these examples begins on the following two facing pages.

Example 1

Unsignalized "T" Intersection Capacity Calculation Form

Intersection JONES DRIVE AT MARKET STREET

Location Plan:



Counts:
Date JUNE 20
Day TUESDAY
Time 3-7 PM
Control STOP
Prevailing Speed 30 MPH

Hourly Demand Traffic Volumes from 4:30 to 5:30, P m

Approach	A		B		C	
Movement	A_T →	A_R ↘	B_L ↙	B_T ←	C_L ↖	C_R ↗
Volume	250	40	150	300	40	120
pch (see Table 1)			165		44	132

Step 1	**Right Turn from C**	C_R ↗
	Conflicting Flows = M_H = (from Fig. 1)	$\frac{1}{2} A_R + A_T =$ 20 + 250 = 270 vph
	Critical Gap from Table 2 T_g =	6.0 sec
	Capacity from Fig. 2 =	$M_{No} = M_1 =$ 720 pch
✓	Shared Lane – See Step 3	
____	No Shared Lane Demand =	C_R = ____ pch
	Available Reserve =	$M_1 - C_R$ = ____ pch
	Delay & Level of Service (Table 3)	____ ☐
Step 2	**Left Turn from B**	B_L ↙
	Conflicting Flows = M_H = (from Fig. 1)	$A_R + A_T =$ ____ + ____ = ____ vph
	Critical Gap from Table 2 T_g =	____ sec
	Capacity from Fig. 2 =	$M_{No} = M_2$ = ____ pch
	Demand =	B_L = ____ pch
	Capacity Used =	$100 (B_L/M_2)$ = ____ %
	Impedance Factor from Fig. 3 =	P_2 = ____
	Available Reserve =	$M_2 - B_L$ = ____ pch
	Delay & Level of Service (Table 3)	____ ☐
Step 3	**Left Turn from C**	C_L ↖
	Conflicting Flows = M_H = (from Fig. 1)	$\frac{1}{2}A_R + A_T + B_L + B_T =$ ____ + ____ + ____ + ____ = ____ vph
	Critical Gap from Table 2 T_g =	____ sec
	Capacity from Fig. 2 =	M_{No} = ____ pch
	Adjust for Impedance	$M_{No} \times P_2 = M_3$ = ____ pch
____	No Shared Lane Demand =	C_L = ____ pch
	Available Reserve =	$M_3 - C_L$ = ____ pch
	Delay & Level of Service (Table 3)	____ ☐
	Shared Lane Demand =	$C_R + C_L = C_{RL}$ = ____ pch
____	Shared Lane with Right Turn Capacity of Shared Lane =	$M_{13} = \frac{(C_R + C_L)}{(C_R/M_1) + (C_L/M_3)}$ M_{13} = ____ pch
	Available Reserve =	$M_{13} - C_{RL}$ = ____ pch
	Delay & Level of Service (Table 3)	____ ☐

Overall Evaluation ____________________

Example 1

The first example is a T intersection in an urban area. Market Street is a two lane collector and Jones Drive is a two lane local street serving a housing development. Jones Drive is controlled by a STOP sign. There is no widening in the vicinity of the intersection and the corner radii are 20 feet (6 m). The terrain is level and the prevailing speeds are approximately 30 mph (50 kph). There is a relatively heavy left turn from Market Street and conversely a heavy right turn from Jones.

Problem

Residents of the housing development on Jones Drive have complained that there is substantial delay in the late afternoon turning right into Market Street. According to the residents, this delay is caused by both left and right turns from Jones Drive having to use the single lane available. They have requested that a right-turn-only lane be constructed. This complaint is to be evaluated.

Analysis

The "Unsignalized 'T' Intersection Capacity Calculation Form" is used for the analysis. A sketch plan of the intersection showing pertinent features is drawn (see opposite page), and volume data are obtained. In this case, a turning movement count was made on Tuesday, June 20 from 3 to 7 p.m. The peak hourly volumes occurred during the period from 4:30 to 5:30. These peak demand volumes are inserted into the table. Each approach is designated by a letter. **A** and **B** are the major street approaches and **C** is the minor street approach. For a four-way intersection **D** is used for the other minor street approach. The subscript after the letters (i.e., **L**, **T**, and **R**) identifies the movement -- left, thru, and right respectively.

For the calculation process, some of the volumes in vehicles per hour (vph) should be converted to passenger car equivalents per hour (pch). This is accomplished through the use of Table 1. No classification count was made in this example, therefore the last row is used. Because it is level terrain, the factor of 1.1 is used. Calculating passenger car equivalents is not necessary for the major street except for the left turn.

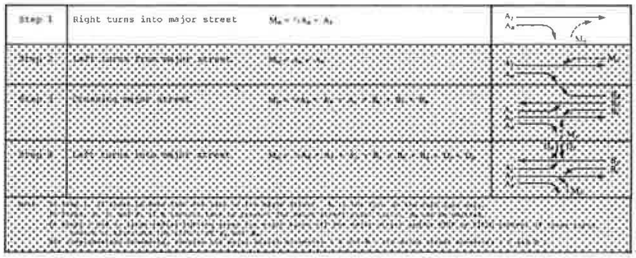
Table 1. Converting Existing Traffic Flow to Passenger Car Equivalents per hour

Grade / Type of Vehicle	-4%	-2%	0%	+2%	+4%
Motorcycles	0.3	0.4	0.5	0.6	0.7
Passenger Cars	0.8	0.9	1.0	1.2	1.4
Trucks	1.0	1.2	1.5	2.0	3.0
Truck-Trailers	1.2	1.5	2.0	3.0	6.0
Motor Vehicle*	0.9	1.0	1.1	1.4	1.7

*Approximate value used for estimate calculations.

Step 1 covers right turns from the minor street. The conflicting flows which affect these right turns are determined in accord with Figure 1 and as shown on the form. Because there is no separate lane or traffic control for the right turning major street vehicles, their effect cannot be reduced or eliminated. Therefore, half the right turn movement (20) and the total thru movement (250) is summed to provide the total conflicting flow (M_H=270). This value is in vehicles per hour.

Figure 1. Definition of Conflicting Traffic Streams



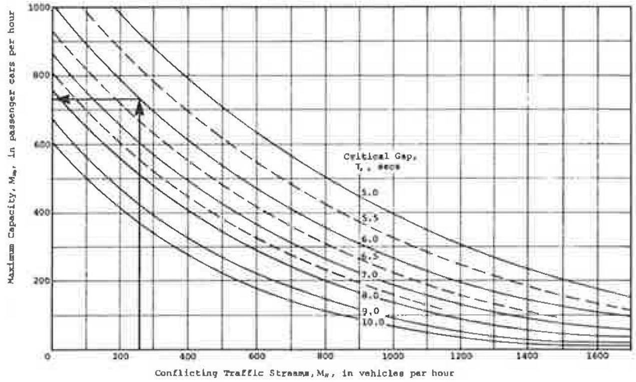
The critical gap is then selected based upon maneuver, type of control, prevailing speed, and number of lanes on the major street. Critical gaps are tabulated in Table 2 for passenger cars. If there are significant numbers of trucks (>5%), the critical gap may be increased to accommodate the heavier vehicles. Grades on the minor approach may also affect the critical gap. In this example, right turn onto the major street, passing a STOP sign, prevailing speed of 30 mph (50 kph), and a two-lane major street -- the critical gap is 6.0 seconds.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

The values for critical gap (T_o) and conflicting flow (M_4) are used to enter Figure 2 to determine potential capacity (M_{No}). This value in Step 1 is also called M_1 and is used in later computations.

Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



This is the end of Step 1 in this example because the right turning vehicles share the single lane with left turning vehicles. The shared lane computation will be completed in Step 3. If this had been an exclusive lane for right turns, Step 1 would have been completed by bringing down from the data table the existing right turn demand and subtracting it from the potential capacity to obtain the available reserve. The available reserve is used to determine delay and Level of Service.

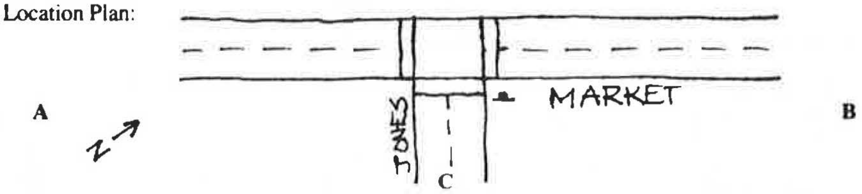
(Continued)

Example 1

Unsignalized "T" Intersection Capacity Calculation Form

Intersection JONES DRIVE AT MARKET STREET

Location Plan:



Counts:
Date JUNE 20
Day TUESDAY
Time 3-7 PM
Control STOP
Prevailing Speed 30 MPH

Hourly Demand Traffic Volumes from 4:30 to 5:30, P m

Approach	A		B		C	
Movement	A_T →	A_R ↘	B_L ↰	B_T ←	C_L ↖	C_R ↗
Volume	250	40	150	300	40	120
pch (see Table 1)			165		44	132

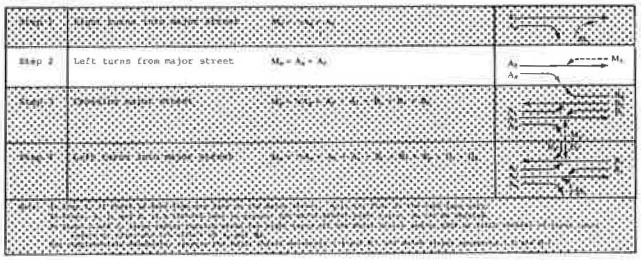
Step 1	**Right Turn from C**	C_R
	Conflicting Flows = M_H =	½ A_R + A_T =
	(from Fig. 1)	20 + 250 = 270 vph
	Critical Gap from Table 2 T_g =	6.0 sec
	Capacity from Fig. 2 =	M_{No} = M_1 = 720 pch
✓	Shared Lane – See Step 3	
___	No Shared Lane Demand =	C_R = ___ pch
	Available Reserve =	$M_1 - C_R$ = ___ pch
	Delay & Level of Service (Table 3)	___ ☐
Step 2	**Left Turn from B**	B_L
	Conflicting Flows = M_H =	A_R + A_T =
	(from Fig. 1)	40 + 250 = 290 vph
	Critical Gap from Table 2 T_g =	5.0 sec
	Capacity from Fig. 2 =	M_{No} = M_2 = 900 pch
	Demand =	B_L = 165 pch
	Capacity Used =	100 (B_L/M_2) = 18 %
	Impedance Factor from Fig. 3 =	P_2 = 0.87
	Available Reserve =	$M_2 - B_L$ = 735 pch
	Delay & Level of Service (Table 3)	NO DELAY A
Step 3	**Left Turn from C**	C_L
	Conflicting Flows = M_H =	½A_R + A_T + B_L + B_T =
	(from Fig. 1)	20 +250 +150 +300 = 720 vph
	Critical Gap from Table 2 T_g =	___ sec
	Capacity from Fig. 2 =	M_{No} = ___ pch
	Adjust for Impedance	$M_{No} \times P_2 = M_3$ = ___ pch
___	No Shared Lane Demand =	C_L = ___ pch
	Available Reserve =	$M_3 - C_L$ = ___ pch
	Delay & Level of Service (Table 3)	___ ☐
	Shared Lane Demand =	$C_R + C_L = C_{RL}$ = ___ pch
___	Shared Lane with Right Turn Capacity of Shared Lane =	$M_{13} = \frac{(C_R + C_L)}{(C_R/M_1) + (C_L/M_3)}$ M_{13} = ___ pch
	Available Reserve =	$M_{13} - C_{RL}$ = ___ pch
	Delay & Level of Service (Table 3)	___ ☐

Overall Evaluation ___

(Example 1)

Step 2 deals with the left turns off of the major street onto the minor street. The conflicting flows are again determined as shown in Figure 1. In this case, the total right turns (40) from the major street are included in the conflicting flow and are added to the thru movement (250) resulting in a total conflicting flow of 290 vph.

Figure 1. Definition of Conflicting Traffic Streams



[illegible]	[illegible]	[illegible]	
Step 2	Left turns from major street	[illegible]	
[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	

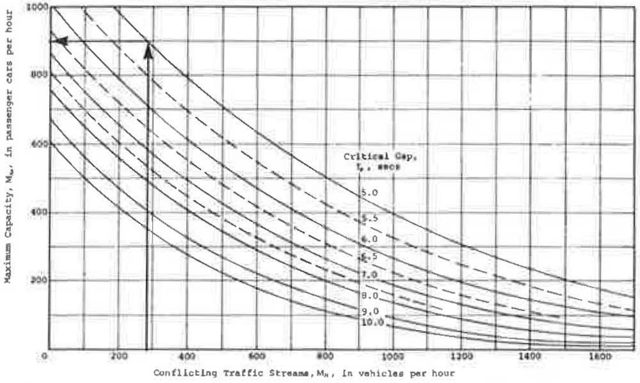
The critical gap is selected from Table 2. For left turns from a 30 mph (50 kph) two-lane major road, the critical gap is 5.0 seconds.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

Using the conflicting flow value of 290 vph and the critical gap of 5.0 seconds to enter Figure 2, results in a potential capacity of 900 pch.

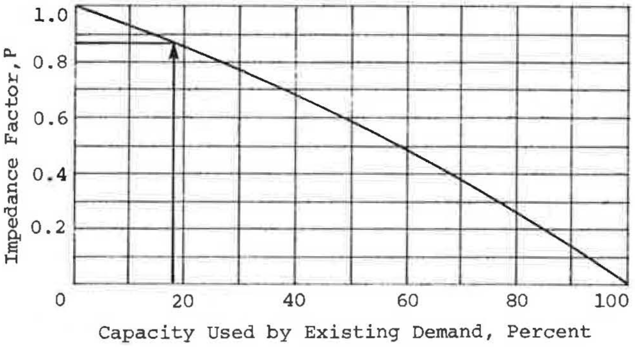
Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



Continuing Step 2 requires the determination of the impedance factor that this movement will have on other minor street flows. The left turn demand in passenger car equivalents per hour (165 pch) is brought down from the data table. The percentage of the potential capacity that is actually used is computed by dividing the demand (165 pch) by the capacity (900 pch) and multiplying by 100 to convert to a percentage. The nearest percent (i.e., 18%) is adequate accuracy.

The percentage of capacity used (18) is the entry point to the graph on the capacity reduction graph (Figure 3). The resulting impedance factor is 0.87. This factor is used in Step 3 to reduce the potential capacity of conflicting minor street movements caused by possible congestion in the left turning stream. The more potential capacity this stream uses the greater the impedance to the conflicting minor street flow.

Figure 3. Capacity Reduction caused by Congestion



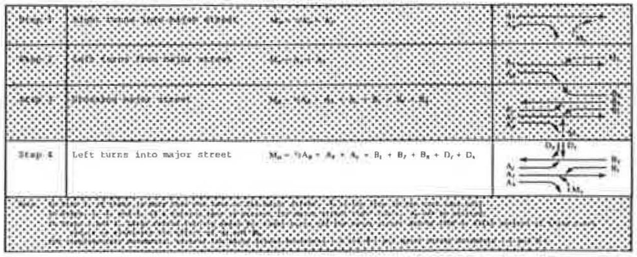
The available reserve is then computed by subtracting the demand (165 pch) from the capacity (900 pch) resulting in 735 pch. This value is then compared to Table 3 to find the Level of Service (LOS) and probable delay. In this example, the left turn movement has a LOS of A and anticipates virtually no delay.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays
0 to 99	E	Very Long traffic delays
Less than 0	—	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

Step 3 examines the critical left turn movement from the side street. Again the conflicting flow is determined as described in Figure 1. Note: Step 3 for a "T" intersection corresponds to Step 4 for a four-way intersection and some of the movements do not exist. In this example, all major street movements are used including half the right turns (20), both thru movements (250 and 300), and the left turns (15) or a total of 720 vph.

Figure 1 Definition of Conflicting Traffic Streams



[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	
Step 4	Left turns into major street	[illegible]	

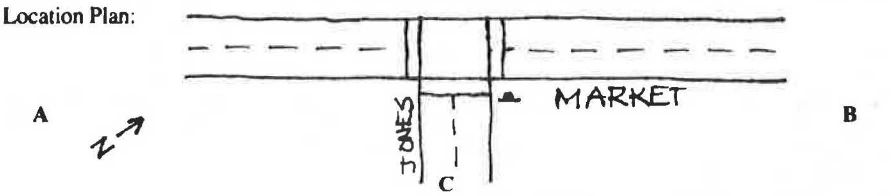
(Continued)

Example 1

Unsignalized "T" Intersection Capacity Calculation Form

Intersection JONES DRIVE AT MARKET STREET

Location Plan:



Counts:
Date JUNE 20
Day TUESDAY
Time 3-7 PM
Control STOP
Prevailing Speed 30 MPH

Hourly Demand Traffic Volumes from 4:30 to 5:30 , Pm

Approach	A		B		C	
Movement	A_T →	A_R ↘	B_L ↱	B_T ←	C_L ↰	C_R ↱
Volume	250	40	150	300	40	120
pch (see Table 1)			165		44	132

Step 1	**Right Turn from C**	C_R
	Conflicting Flows = M_H =	½ A_R + A_T =
	(from Fig. 1)	20 + 250 = 270 vph
	Critical Gap from Table 2 T_g =	6.0 sec
	Capacity from Fig. 2 =	$M_{No} = M_1$ = 720 pch
✓	Shared Lane — See Step 3	
___	No Shared Lane Demand =	C_R = ___ pch
	Available Reserve =	$M_1 - C_R$ = ___ pch
	Delay & Level of Service (Table 3)	☐
Step 2	**Left Turn from B**	B_L
	Conflicting Flows = M_H =	A_R + A_T =
	(from Fig. 1)	40 + 250 = 290 vph
	Critical Gap from Table 2 T_g =	5.0 sec
	Capacity from Fig. 2 =	$M_{No} = M_2$ = 900 pch
	Demand =	B_L = 165 pch
	Capacity Used =	100 (B_L/M_2) = 18 %
	Impedance Factor from Fig. 3 =	P_2 = 0.87
	Available Reserve =	$M_2 - B_L$ = 735 pch
	Delay & Level of Service (Table 3)	NO DELAY $\boxed{A}$
Step 3	**Left Turn from C**	C_L
	Conflicting Flows = M_H =	½A_R + A_T + B_L + B_T =
	(from Fig. 1)	20 +250 +150 +300 = 720 vph
	Critical Gap from Table 2 T_g =	7.5 sec
	Capacity from Fig. 2 =	M_{No} = 265 pch
	Adjust for Impedance	$M_{No} \times P_2 = M_3$ = 231 pch
___	No Shared Lane Demand =	C_L = ___ pch
	Available Reserve =	$M_3 - C_L$ = ___ pch
	Delay & Level of Service (Table 3)	☐
	Shared Lane Demand =	$C_R + C_L = C_{RL}$ = 176 pch
✓	Shared Lane with Right Turn Capacity of Shared Lane =	$M_{13} = \frac{(C_R + C_L)}{(C_R/M_1) + (C_L/M_3)}$ M_{13} = 471 pch
	Available Reserve =	$M_{13} - C_{RL}$ = 295 pch
	Delay & Level of Service (Table 3)	AVERAGE DELAY $\boxed{C}$

Overall Evaluation CURRENTLY HIGH LOS C. ADDING RT LANE WOULD MAKE RT LANE OPERATE AT LOS A (CAPACITY [720] - DEMAND [132]) = AVAIL. RESERVE OF 588 = LOS A
LT WOULD BE LOS D (CAPACITY [231] - DEMAND [44]) = AVAIL. RESERVE OF 187 = LOS D

(Example 1)

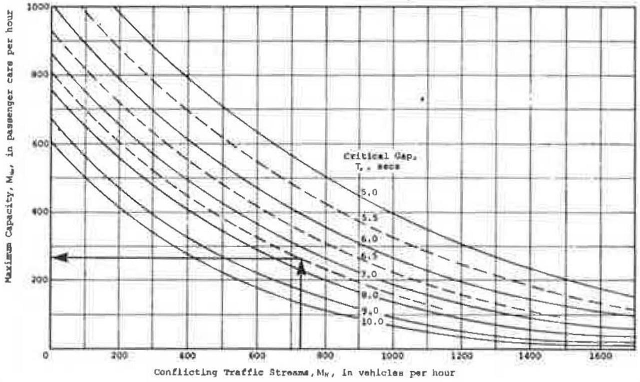
Table 2 gives a critical gap of 7.5 seconds for a left turn passing a STOP sign.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

The potential capacity is read from Figure 2 using a conflicting flow of 720 vph and a critical gap of 7.5 seconds. The value is 265 pch. This potential capacity must be reduced because of impendance created by the left turns from the major street. The impedance factor was computed in Step 2 (0.87). Reducing the potential capacity (265) by this factor yields 231 pch as the potential capacity of the left lane. If there were an exclusive lane for the left turns, the calculated value would be the capacity and by, subtracting the demand, the available reserve would be found. Comparing the available reserve with Table 3 would provide the LOS and the expected delay.

Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



However, in this example, right and left turns share a common lane. The capacity of the shared lane is computed by the formula shown on the calculation form, which results in a shared lane capacity of 471 pch. This capacity is for a shared lane with a 3 to 1 ratio of right turns to left turns and these specific major street flows. Subtracting the combined demand of 176 pch leaves an available reserve of 295 pch. Comparing this value with Table 3 gives a LOS of C and average traffic delays.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays
0 to 99	E	Very Long traffic delays
Less than 0	—	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

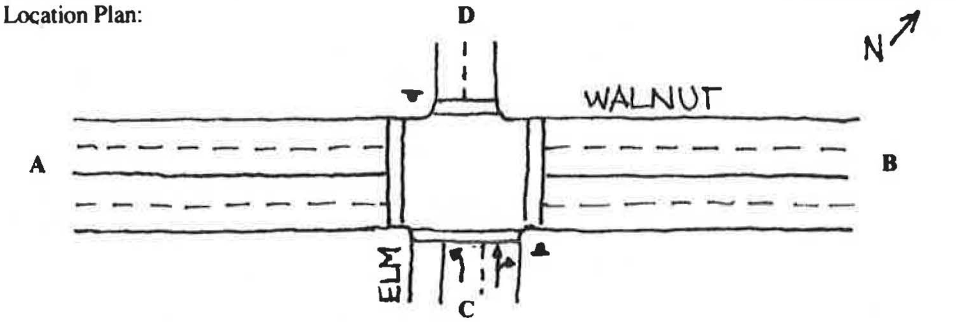
The overall evaluation section indicates the effect of adding a right-turn lane. Essentially, the uncompleted parts of the form were completed to obtain the additional LOS values. Adding a right-turn lane would virtually eliminate delay to three-fourths of the Jones Drive approach traffic. Average delay for left turns is greater than the average for the combined stream in the shared lane resulting in a lower level of service for the left turns.

Example 2

Unsignalized Intersection Capacity Calculation Form

Intersection ELM STREET AND WALNUT STREET

Location Plan:



Counts:
Date OCTOBER 17
Day TUESDAY
Time 7-10 AM
Control STOP
Prevailing Speed 30 MPH

Hourly Demand Traffic Volumes from 7:15 to 8:15 Am

Approach	A			B			C			D		
Movement	A_L	A_T	A_R	B_L	B_T	B_R	C_L	C_T	C_R	D_L	D_T	D_R
Volume	30	250	50	60	300	100	40	120	50	10	100	25
pch (see Table 1)	33	▨	▨	66	▨	▨	44	132	55	11	110	28

Step 1 **Right Turn from C/D**	C_R	D_R
Conflicting Flows = M_H = (from Fig. 1)	½ A_R + A_T = 25 + 250/2 = 150 vph	½ B_R + B_T = 50 + 300/2 = 200 vph
Critical Gap from Table 2 T_g =	6.0 sec	6.0 sec
Capacity from Fig. 2 =	$M_{No} = M_1$ = 840 pch	$M'_{No} = M'_1$ = 790 pch
Demand =	C_R = 55 pch	D_R = 28 pch
Capacity Used =	$100 (C_R/M_1)$ = 7 %	$100 (D_R/M'_1)$ = 4 %
Impedance Factor from Fig. 3 =	P_1 = 0.95	P'_1 = 0.97
C&D Shared Lane – See Step 3		
____ No Shared Lane – Available Reserve	$M_1 - C_R$ = ____ pch	$M'_1 - D_R$ = ____ pch
Delay & Level of Service (Table 3)	____ ☐	____ ☐

Step 2 **Left Turn from B/A**	B_L	A_L
Conflicting Flows = M_H = (from Fig. 1)	A_R + A_T = ____ + ____ = ____ vph	B_R + B_T = ____ + ____ = ____ vph
Critical Gap from Table 2 T_g =	____ sec	____ sec
Capacity from Fig. 2 =	$M_{No} = M_2$ = ____ pch	$M'_{No} = M'_2$ = ____ pch
Demand =	B_L = ____ pch	A_L = ____ pch
Capacity Used =	$100 (B_L/M_2)$ = ____ %	$100 (A_L/M'_2)$ = ____ %
Impedance Factor from Fig. 3 =	P_2 = ____	P'_2 = ____
Available Reserve =	$M_2 - B_L$ = ____ pch	$M'_2 - A_L$ = ____ pch
Delay & Level of Service (Table 3)	____ ☐	____ ☐

Step 3 **Thru Movement from C/D**	C_T	D_T
Conflicting Flows = M_H = (from Fig. 1) (M_T & M'_T are used in Step 4)	½A_R + A_T + A_L + B_L + B_T + B_R ___ + ___ + ___ + ___ + ___ + ___ $M_H = M_T$ = ____ vph	½B_R + B_T + B_L + A_L + A_T + A_R ___ + ___ + ___ + ___ + ___ + ___ $M_H = M'_T$ = ____ vph
Critical Gap from Table 2 T_g =	____ sec	____ sec
Capacity from Fig. 2 =	M_{No} = ____ pch	M'_{No} = ____ pch
Adjust for Impedance	$M_{No} \times P_2 \times P'_2 = M_3$ = ____ pch	$M'_{No} \times P'_2 \times P_2 = M'_3$ = ____ pch
Demand =	C_T = ____ pch	D_T = ____ pch
Capacity Used =	$100 (C_T/M_3)$ = ____ %	$100 (D_T/M'_3)$ = ____ %
Impedance Factor from Fig. 3	P_3 = ____	P'_3 = ____

Example 2

This example is an urban four-way intersection. Walnut Street is an arterial and Elm Street is a two-lane collector in a grid system with STOP sign control. The northbound Elm approach has been widened to provide an exclusive lane for left turns. Curb radii are 25 ft. (7-1/2 m). The terrain is level, and the prevailing speed is 30 mph (50 kph).

Problem

Complaints have been received of extensive backups and long delays on Elm Street at this intersection particularly in the morning peak.

Analysis

The Unsignalized Intersection Capacity Calculation Form is used. A sketch plan of the intersection is drawn and volume data are obtained. In this case, a turning movement count was made from 7 to 10 a.m. on Tuesday, October 17. The peak volumes occurred during the period 7:15 to 8:15 and are inserted in the volume data table.

Many of the volumes must be converted from vehicles per hour (vph) to passenger car equivalents per hour (pch) by the factors in Table 1. No classification count was made, therefore the last row is used, and the factor is 1.1. This conversion is not necessary for major street thru and right turns.

Table 1. Converting Existing Traffic Flow to Passenger Car Equivalents per hour

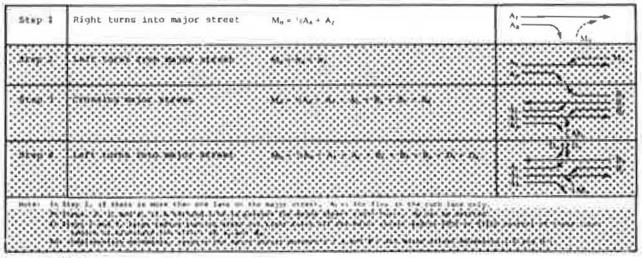
Grade / Type of Vehicle	-4%	-2%	0%	+2%	+4%
Motorcycles	0.3	0.4	0.5	0.6	0.7
Passenger Cars	0.8	0.9	1.0	1.2	1.4
Trucks	1.0	1.2	1.5	2.0	3.0
Truck-Trailers	1.2	1.5	2.0	3.0	6.0
Motor Vehicle*	0.9	1.0	1.1	1.4	1.7

*Approximate value used for estimate calculations.

The calculation form covers two pages and each step has two columns. The left column deals with the **C** approach (or in this case the northbound approach) while the right column **D** or southbound approach. Both columns are computed for each step. Computed values in the right column are designated by a prime (e.g., M'_{No}).

Step 1 covers right turns from the minor street. The conflicting flows that affect these right turns are determined in accord with Figure 1. Note that Figure 1 only shows right turns from the **C** approach, the similar turns from the other approach are treated in the same manner. The conflicting flows for **C** are half the right turns (25) and the thru movement (250) divided by 2 (see the first note on Figure 1) for a total of 150. Likewise for **D**, half the right turns (50) and half the thru (300/2) equals 200.

Figure 1. Definition of Conflicting Traffic Streams



Step 1 | Right turns into major street | $M_R = \frac{1}{2}A_R + A_r$

Step 2 | Left turns from major street

Step 3 | Crossing major street

Step 4 | Left turns into major street

Note: In Step 1, if there is more than one lane on the major street, A_r is the flow in the curb lane only.

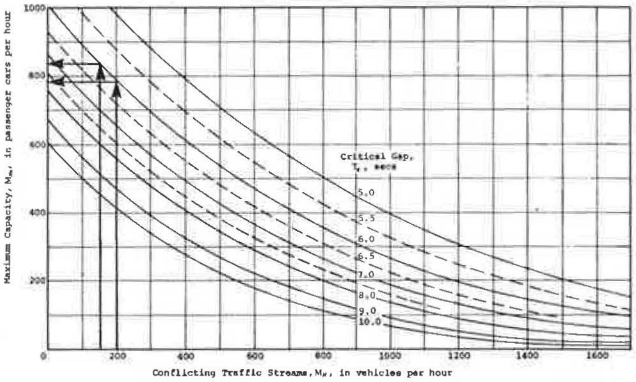
The critical gap is the same for the two minor street approaches and is selected from Table 2. For right turns into a 30 mph (50 kph), four lane major road, the value is 6.0 seconds.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

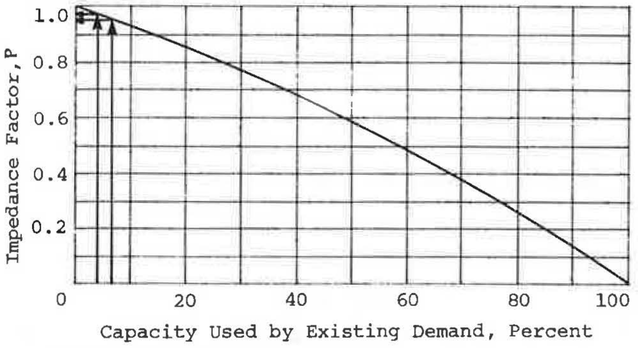
The conflicting flows (150 & 200) and the critical gap (6.0) are used to enter Figure 2. 840 pch and 790 pch are the potential capacities for the right turning movements for the south and north approaches respectively.

Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



Impedance factors must be calculated for both approaches for later use in Step 4. The right turn demands in pch are brought down from the data table. The percent of capacity used is computed for each approach - 7% for the south approach and 4% for the other. These values are used to enter Figure 3 to obtain the impedance factors of 0.95 and 0.97.

Figure 3. Capacity Reduction caused by Congestion



Both the south and the north approaches have right turns sharing lanes with other movements (i.e. in the south approach, the right turn shares with the thru movement; while in the north approach, all three movements share the same lane). This sharing of lanes is indicated at the far left of the form and the remainder of Step 1 is omitted.

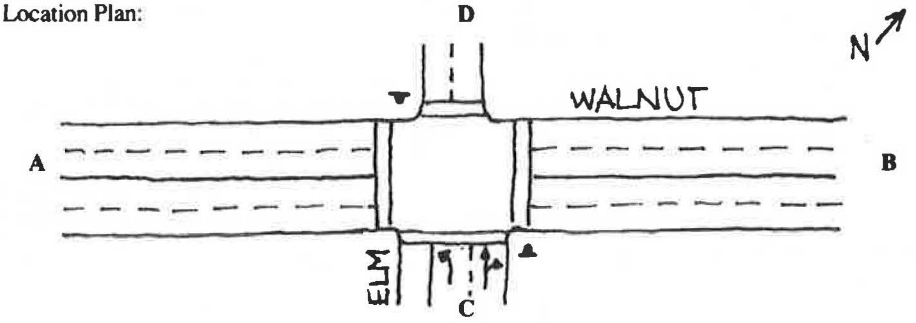
(Continued)

Example 2

Unsignalized Intersection Capacity Calculation Form

Intersection ELM STREET AND WALNUT STREET

Location Plan:



Counts:
Date OCTOBER 17
Day TUESDAY
Time 7-10 AM
Control STOP
Prevailing Speed 30 MPH

Hourly Demand Traffic Volumes from 7:15 to 8:15 A m

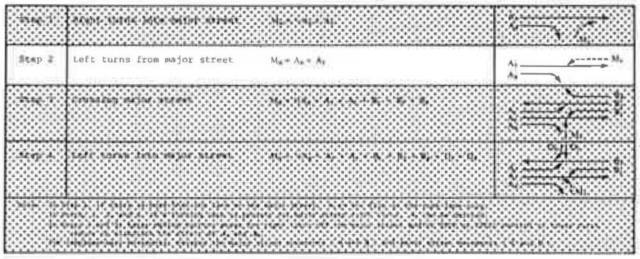
Approach	A			B			C			D		
Movement	A_L	A_T	A_R	B_L	B_T	B_R	C_L	C_T	C_R	D_L	D_T	D_R
Volume	30	250	50	60	300	100	40	120	50	10	100	25
pch (see Table 1)	33	▨	▨	66	▨	▨	44	132	55	11	110	28

Step 1 **Right Turn from C/D**	C_R	D_R
Conflicting Flows = M_H =	½ A_R + A_T =	½ B_R + B_T =
(from Fig. 1)	25 + 250/2 = 150 vph	50 + 300/2 = 200 vph
Critical Gap from Table 2 T_g =	6.0 sec	6.0 sec
Capacity from Fig. 2 =	$M_{No} = M_1$ = 840 pch	$M'_{No} = M'_1$ = 790 pch
Demand =	C_R = 55 pch	D_R = 28 pch
Capacity Used =	$100 (C_R/M_1)$ = 7 %	$100 (D_R/M'_1)$ = 4 %
Impedance Factor from Fig. 3 =	P_1 = 0.95	P'_1 = 0.97
C&D Shared Lane – See Step 3		
____ No Shared Lane – Available Reserve	$M_1 - C_R$ = ____ pch	$M'_1 - D_R$ = ____ pch
Delay & Level of Service (Table 3)	____ ☐	____ ☐
Step 2 **Left Turn from B/A**	B_L	A_L
Conflicting Flows = M_H =	A_R + A_T =	B_R + B_T =
(from Fig. 1)	50 + 200 = 250 vph	100 + 300 = 400 vph
Critical Gap from Table 2 T_g =	5.5 sec	5.5 sec
Capacity from Fig. 2 =	$M_{No} = M_2$ = 780 pch	$M'_{No} = M'_2$ = 700 pch
Demand =	B_L = 66 pch	A_L = 33 pch
Capacity Used =	$100 (B_L/M_2)$ = 8 %	$100 (A_L/M'_2)$ = 5 %
Impedance Factor from Fig. 3 =	P_2 = 0.94	P'_2 = 0.97
Available Reserve =	$M_2 - B_L$ = 714 pch	$M'_2 - A_L$ = 667 pch
Delay & Level of Service (Table 3)	NO DELAY A	NO DELAY A
Step 3 **Thru Movement from C/D**	C_T	D_T
Conflicting Flows = M_H =	½A_R + A_T + A_L + B_L + B_T + B_R	½B_R + B_T + B_L + A_L + A_T + A_R
(from Fig. 1)	___ + ___ + ___ + ___ + ___ + ___	___ + ___ + ___ + ___ + ___ + ___
(M_T & M'_T are used in Step 4)	$M_H = M_T$ = ____ vph	$M_H = M'_T$ = ____ vph
Critical Gap from Table 2 T_g =	____ sec	____ sec
Capacity from Fig. 2 =	M_{No} = ____ pch	M'_{No} = ____ pch
Adjust for Impedance	$M_{No} \times P_2 \times P'_2 = M_3$ = ____ pch	$M'_{No} \times P'_2 \times P_2 = M'_3$ = ____ pch
Demand =	C_T = ____ pch	D_T = ____ pch
Capacity Used =	$100 (C_T/M_3)$ = ____ %	$100 (D_T/M'_3)$ = ____ %
Impedance Factor from Fig. 3	P_3 = ____	P'_3 = ____

(Example 2)

In Step 2, the left turns from the major street are treated. The conflicting volumes are determined in accord with Figure 1. Note that the total thru and right turn volumes are used resulting in flows of 300 vph and 400 vph.

Figure 1. Definition of Conflicting Traffic Streams



[illegible]	[illegible]	[illegible]	
Step 2	Left turns from major street	[illegible]	
[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	
[illegible]			

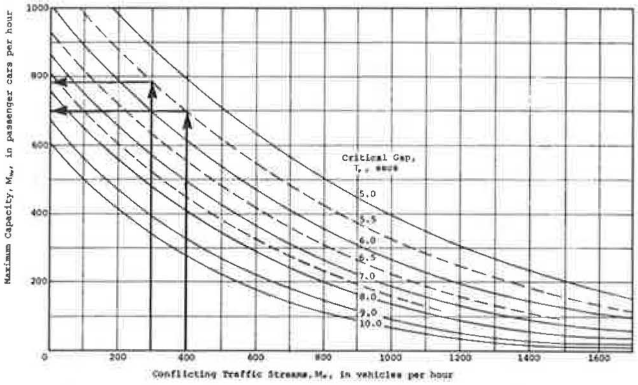
The critical gap of 5.5 seconds is obtained from Table 2.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

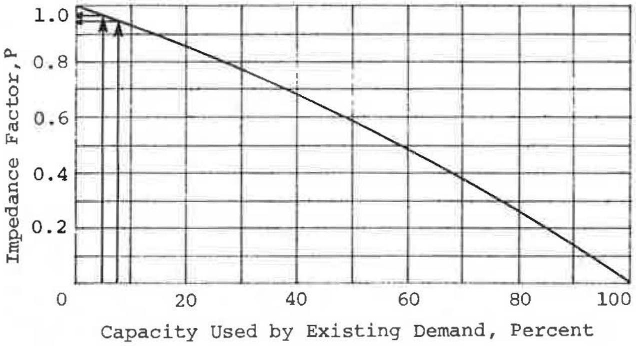
Using the above values, Figure 2 is entered to provide capacities of 780 pch and 700 pch respectively.

Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



Impedance factors are necessary for later steps. The percent of capacity being used is calculated and impedance factors obtained from Figure 3. Factors derived from figure 3 are 0.97 and 0.94.

Figure 3. Capacity Reduction caused by Congestion



The available reserve for each left turn is then calculated by subtracting the demand from the capacity. These available reserves of 714 pch and 667 pch, respectively, when compared to Table 3, indicate an excellent operating condition with Level of Service **A** and no expected delay.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays
0 to 99	E	Very Long traffic delays
Less than 0	—	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

Example 2 continues with Step 3 on the following pages.

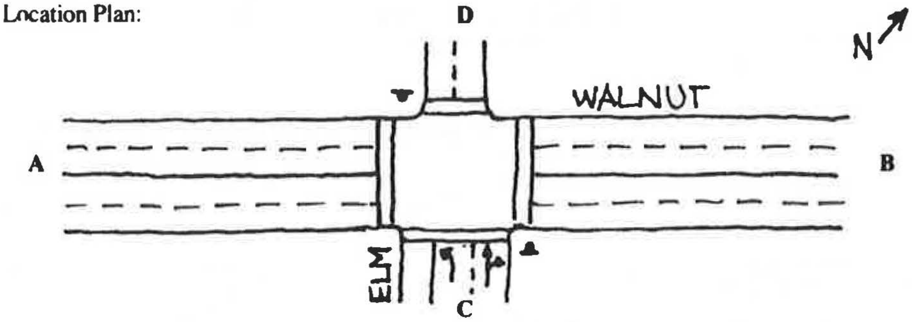
(Continued)

Example 2

Unsignalized Intersection Capacity Calculation Form

Intersection ELM STREET AND WALNUT STREET

Location Plan:



Counts:
Date OCTOBER 17
Day TUESDAY
Time 7-10AM
Control STOP
Prevailing Speed 30 MPH

Hourly Demand Traffic Volumes from 7:15 to 8:15 A m

Approach	A			B			C			D		
Movement	A_L	A_T	A_R	B_L	B_T	B_R	C_L	C_T	C_R	D_L	D_T	D_R
Volume	30	250	50	60	300	100	40	120	50	10	100	25
pch (see Table 1)	33	▨	▨	66	▨	▨	44	132	55	11	110	28

Step 1 **Right Turn from C/D**	C_R	D_R
Conflicting Flows = M_H = (from Fig. 1)	½ A_R + A_T = 25 + 250/2 = 150 vph	½ B_R + B_T = 50 + 300/2 = 200 vph
Critical Gap from Table 2 T_g =	6.0 sec	6.0 sec
Capacity from Fig. 2 =	$M_{No} = M_1$ = 840 pch	$M'_{No} = M'_1$ = 790 pch
Demand =	C_R = 55 pch	D_R = 28 pch
Capacity Used =	100 (C_R/M_1) = 7 %	100 (D_R/M'_1) = 4 %
Impedance Factor from Fig. 3 =	P_1 = 0.95	P'_1 = 0.97
C&D Shared Lane – See Step 3		
____ No Shared Lane – Available Reserve	$M_1 - C_R$ = ____ pch	$M'_1 - D_R$ = ____ pch
Delay & Level of Service (Table 3)	☐	☐

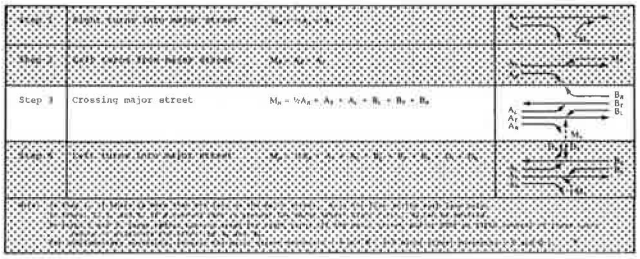
Step 2 **Left Turn from B/A**	B_L	A_L
Conflicting Flows = M_H = (from Fig. 1)	A_R + A_T = 50 + 200 = 250 vph	B_R + B_T = 100 + 300 = 400 vph
Critical Gap from Table 2 T_g =	5.5 sec	5.5 sec
Capacity from Fig. 2 =	$M_{No} = M_2$ = 780 pch	$M'_{No} = M'_2$ = 700 pch
Demand =	B_L = 66 pch	A_L = 33 pch
Capacity Used =	100 (B_L/M_2) = 8 %	100 (A_L/M'_2) = 5 %
Impedance Factor from Fig. 3 =	P_2 = 0.94	P'_2 = 0.97
Available Reserve =	$M_2 - B_L$ = 714 pch	$M'_2 - A_L$ = 667 pch
Delay & Level of Service (Table 3)	NO DELAY A	NO DELAY A

Step 3 **Thru Movement from C/D**	C_T	D_T
Conflicting Flows = M_H = (from Fig. 1) (M_T & M'_T are used in Step 4)	½A_R + A_T + A_L + B_L + B_T + B_R 25 + 250 + 30 + 60 + 300 + 100 $M_H = M_T$ = 765 vph	½B_R + B_T + B_L + A_L + A_T + A_R 50 + 300 + 60 + 30 + 250 + 50 $M_H = M'_T$ = 740 vph
Critical Gap from Table 2 T_g =	7.5 sec	7.5 sec
Capacity from Fig. 2 =	M_{No} = 250 pch	M'_{No} = 260 pch
Adjust for Impedance	$M_{No} \times P_2 \times P'_2 = M_3$ = 228 pch	$M'_{No} \times P'_2 \times P_2 = M'_3$ = 237 pch
Demand =	C_T = 132 pch	D_T = 110 pch
Capacity Used =	100 (C_T/M_3) = 58 %	100 (D_T/M'_3) = 46 %
Impedance Factor from Fig. 3	P_3 = 0.50	P'_3 = 0.62

(Example 2)

Conflicting flows are determined for both entering minor street approaches as indicated in Figure 1. Half of the near side right turns plus all of the rest of the entering major street traffic yields 765 vph and 740 vph respectively. Note these values are also labeled M_T and M_T' and are used later in Step 4.

Figure 1. Definition of Conflicting Traffic Streams



[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	
Step 3	Crossing major street	$M_H = ½A_R$ + [illegible]	
[illegible]	[illegible]	[illegible]	
[illegible]			

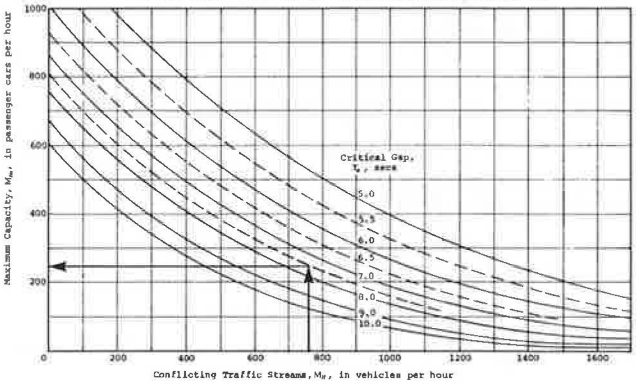
The critical gap is 7.5 seconds as indicated in Table 2.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

The above values are used to enter Figure 2 to obtain potential capacity. Note that only the first entry is illustrated on Figure 2 because the lines are very similar. The two potential capacities are 250 pch and 260 pch respectively. These values must be adjusted for the impedance caused by the left turns off of the major street. The impedance factors (P_2 and P_2') were determined in Step 2 and are 0.94 and 0.97 respectively. Each thru movement must be multiplied by both factors to yield the exclusive lane capacity.

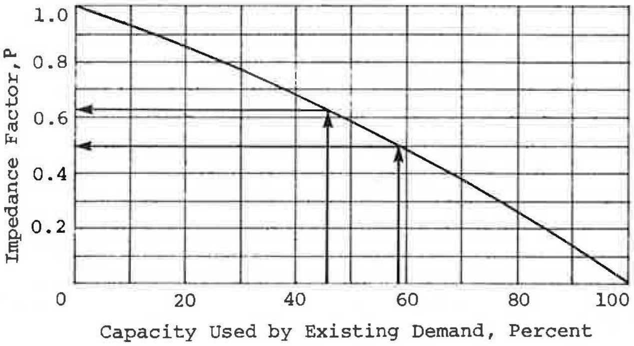
Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



Additional impedance factors must be determined for use in Step 4. Demand values of 132 pch and 110 pch are brought down from the data table and the percent of capacity is determined (58% and 46%).

These percentages are used to enter Figure 3 to obtain impedance factors of 0.50 and 0.62.

Figure 3. Capacity Reduction caused by Congestion



The remaining part of Step 3 is on the back of the form. The first section is for exclusive lanes (i.e., no shared lane). Because this example does not have exclusive lanes for the thru movement, this section is omitted. The second section is for a shared lane with left turns. The southbound approach (D) has all thru movement sharing the lanes, therefore a "D" is inserted in the space at the far left. The analysis of this shared lane will take place in Step 4.

(Continued)

Example 2

Unsignalized Intersection Capacity Calculation Form (continued)

Step 3 (Continued)		C_T ↑	D_T ↓
______	No Shared Lane Available Reserve = Delay & Level of Service (Table 3)	$M_3 - C_T =$ ______ pch ______ ☐	$M'_3 - D_T =$ ______ pch ______ ☐
D	Shared Lane with Left Turn See Step 4		
C	Shared Lane Demand = Shared Lane with Right Turn Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_R + C_T = C_{RT} =$ 187 pch $M_{13} = \frac{(C_R + C_T)}{(C_R/M_1) + (C_T/M_3)}$ $M_{13} =$ 290 pch $M_{13} - C_{RT} =$ 103 pch LONG DELAYS $\boxed{\text{D}}$	$D_R + D_T = D_{RT} =$ ______ pch $M'_{13} = \frac{(D_R + D_T)}{(D_R/M'_1) + (D_T/M'_3)}$ $M'_{13} =$ ______ pch $M'_{13} - D_{RT} =$ ______ pch ______ ☐
Step 4	**Left Turn from C/D**	C_L	D_L
	Conflicting Flows = M_H = (M_T & M'_T were calculated in Step 3) Critical Gap from Table 2 T_g = Capacity from Fig. 2 = Adjust for Impedance	$M_T + D_T + D_R =$ 765 + 100 + 25 = 890 vph 8.0 sec $M_{No} =$ 170 pch $M_{No} \times P_2 \times P'_2 \times P'_1 \times P'_3 = M_4$ $M_4 =$ 93 pch	$M'_T + C_T + C_R =$ 740 + 120 + 50 = 910 vph 8.0 sec $M'_{No} =$ 160 pch $M'_{No} \times P'_2 \times P_2 \times P_1 \times P_3 = M'_4$ $M'_4 =$ 69 pch
C	No Shared Lane Demand = Available Reserve = Delay & Level of Service (Table 3)	$C_L =$ 44 pch $M_4 - C_L =$ 49 pch VERY LONG DELAYS $\boxed{\text{E}}$	$D_L =$ ______ pch $M'_4 - D_L =$ ______ pch ______ ☐
______	Shared Lane Demand = Shared Lane with Thru Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_T + C_L = C_{TL} =$ ______ pch $M_{34} = \frac{(C_T + C_L)}{(C_T/M_3) + (C_L/M_4)}$ $M_{34} =$ ______ pch $M_{34} - C_{TL} =$ ______ pch ______ ☐	$D_T + D_L = D_{TL} =$ ______ pch $M'_{34} = \frac{D_T + D_L}{(D_T/M'_3) + (D_L/M'_4)}$ $M'_{34} =$ ______ pch $M'_{34} - D_{TL} =$ ______ pch ______ ☐
D	Shared Lane Demand = Shared Lane with Thru & Right Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_R + C_T + C_L = C_{RTL} =$ ______ pch $M_{134} = \frac{C_R + C_T + C_L}{(C_R/M_1) + (C_T/M_3) + (C_L/M_4)}$ $M_{134} =$ ______ pch $M_{134} - C_{RTL} =$ ______ pch ______ ☐	$D_R + D_T + D_L = D_{RTL} =$ 149 pch $M'_{134} = \frac{D_R + D_T + D_L}{(D_R/M'_1) + (D_T/M'_3) + (D_L/M'_4)}$ $M'_{134} =$ 226 pch $M'_{134} - D_{RTL} =$ 77 pch VERY LONG DELAYS $\boxed{\text{E}}$

Overall Evaluation INTERSECTION OPERATES AT NEAR
CAPACITY IN THE PEAK HOURS.
CHECK SIGNAL WARRANTS.

(Example 2)

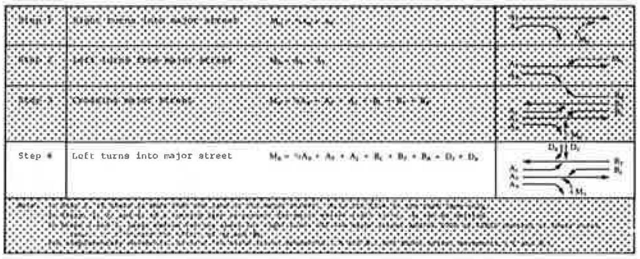
In the northbound approach (**C**), thru and right turns share a single lane. A "C" is inserted in the space at the far left and this section is completed for this approach. The capacity of the shared lane is computed as shown on the form. A programable calculator program for this equation is presented in an appendix following Example 3. The result of the computation in this example is 290 pch. The shared lane demand (sum of thru and right turns) is obtained from the data table and equals 187 pch. The difference (103 pch) is the available reserve, which in Table 3 indicates a Level of Service **D** and long traffic delays.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays
0 to 99	E	Very Long traffic delays
Less than 0	—	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

Step 4 analyzes the left turns from the minor street. These turns have many conflicting flows as shown in Figure 1. The major street flows have been summed in Step 3 and labeled M_T and M_T' respectively. These values are entered in Step 4 and the opposing approach thru and right turns are added. For the northbound approach, the conflicting flow totals 890 vph while there are 910 vph in conflicting flows for the southbound approach.

Figure 1. Definition of Conflicting Traffic Streams



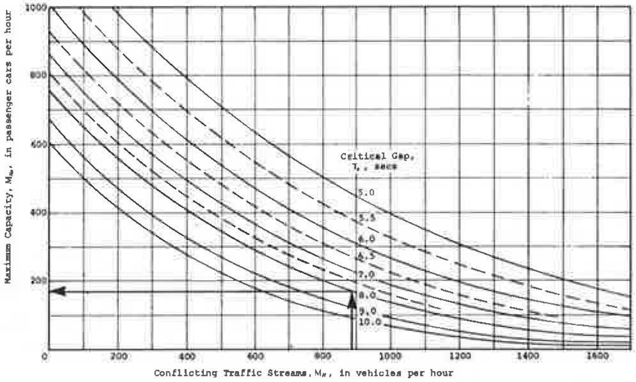
The critical gap (8.0 seconds) is selected from Table 2.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

Step 4 continues by determining the potential capacity from Figure 2. Again, only one line is illustrated because of the closeness of the values. The values from the figure (170 pch and 160 pch) must be adjusted for impedances. Note that of the adjustments for the northbound approach, P_2 , P_2' , P_1' , and P_3' , the last three (those with primes) were computed during the analysis of the southbound approach. The reverse is true for the other approach. After applying the impedance factors, the resulting capacities are 93 pch and 69 pch.

Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



The next section of Step 4 is for exclusive left turn lanes. The northbound approach does have an exclusive left turn lane. Therefore, a "C" is entered in the space at the left, and this section is completed for that approach. The left turn demand (44 pch) is obtained from the data table. Subtracting 44 pch from the capacity of 93 pch yields an available reserve of 49 pch. Table 3 indicates a Level of Service **E** with very long traffic delays.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays
0 to 99	E	Very Long traffic delays
Less than 0	—	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

The next section of the form is for shared lanes with thru and left turns. Although the southbound approach does have the thru and left turns sharing a lane, they also share it with right turns. Therefore, the last section is used to analyze the shared lane with all three movements. A "D" is inserted in the last space on the left. The complex formula is used to compute the shared lane capacity. In this example, the result is 226 pch. Subtracting the combined demand (from the data table) of 149 pch leaves an available reserve of 77 pch. Table 3 again indicates Level of Service **E** and very long traffic delays.

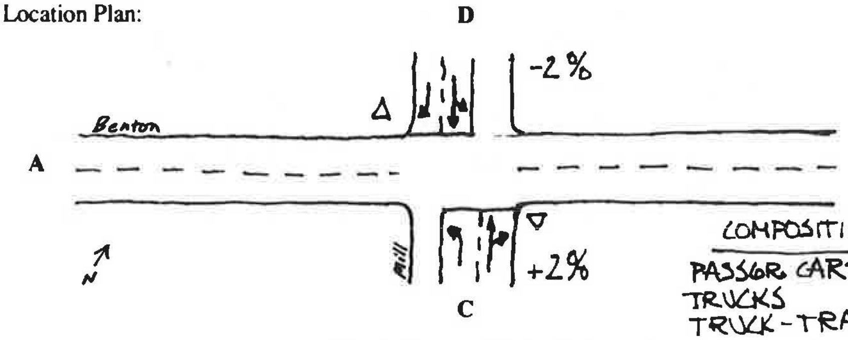
For the overall evaluation of this intersection, it is apparent that the minor street operates at near capacity in the peak hour. Very little additional traffic would cause congestion. Check current and predicted traffic against signal warrants.

Example 3

Unsignalized Intersection Capacity Calculation Form

Intersection BENTON HIGHWAY AND MILL ROAD

Location Plan:



COMPOSITION %

PASSGR. CARS	85
TRUCKS	12
TRUCK-TRAILERS	3

Counts:
Date NOVEMBER 8
Day WEDNESDAY
Time 4-5 PM
Control YIELD
Prevailing Speed 55 MPH

Hourly Demand Traffic Volumes from 4 to 5, p m

Approach	A			B			C			D		
Movement	A_L	A_T	A_R	B_L	B_T	B_R	C_L	C_T	C_R	D_L	D_T	D_R
Volume	60	120	20	40	100	40	20	40	10	10	20	120
pch (see Table 1)	65	▨	▨	44	▨	▨	27	54	14	10	19	114

Step 1 **Right Turn from C/D**	C_R	D_R
Conflicting Flows = M_H =	½ A_R + A_T =	½ B_R + B_T =
(from Fig. 1)	10 + 120 = 130 vph	20 + 100 = 120 vph
Critical Gap from Table 2 T_g =	6.0 sec	6.0 sec
Capacity from Fig. 2 =	$M_{No} = M_1$ = 860 pch	$M'_{No} = M'_1$ = 870 pch
Demand =	C_R = ______ pch	D_R = ______ pch
Capacity Used =	$100 (C_R/M_1)$ = ______ %	$100 (D_R/M'_1)$ = ______ %
Impedance Factor from Fig. 3 =	P_1 = ______	P'_1 = ______
______ Shared Lane – See Step 3		
______ No Shared Lane – Available Reserve	$M_1 - C_R$ = ______ pch	$M'_1 - D_R$ = ______ pch
Delay & Level of Service (Table 3)	______ ☐	______ ☐
Step 2 **Left Turn from B/A**	B_L	A_L
Conflicting Flows = M_H =	A_R + A_T =	B_R + B_T =
(from Fig. 1)	______ + ______ = ______ vph	______ + ______ = ______ vph
Critical Gap from Table 2 T_g =	______ sec	______ sec
Capacity from Fig. 2 =	$M_{No} = M_2$ = ______ pch	$M'_{No} = M'_2$ = ______ pch
Demand =	B_L = ______ pch	A_L = ______ pch
Capacity Used =	$100 (B_L/M_2)$ = ______ %	$100 (A_L/M'_2)$ = ______ %
Impedance Factor from Fig. 3 =	P_2 = ______	P'_2 = ______
Available Reserve =	$M_2 - B_L$ = ______ pch	$M'_2 - A_L$ = ______ pch
Delay & Level of Service (Table 3)	______ ☐	______ ☐
Step 3 **Thru Movement from C/D**	C_T	D_T
Conflicting Flows = M_H =	½A_R + A_T + A_L + B_L + B_T + B_R	½B_R + B_T + B_L + A_L + A_T + A_R
(from Fig. 1)	___ + ___ + ___ + ___ + ___ + ___	___ + ___ + ___ + ___ + ___ + ___
(M_T & M'_T are used in Step 4)	$M_H = M_T$ = ______ vph	$M_H = M'_T$ = ______ vph
Critical Gap from Table 2 T_g =	______ sec	______ sec
Capacity from Fig. 2 =	M_{No} = ______ pch	M'_{No} = ______ pch
Adjust for Impedance	$M_{No} \times P_2 \times P'_2 = M_3$ = ______ pch	$M'_{No} \times P'_2 \times P_2 = M'_3$ = ______ pch
Demand =	C_T = ______ pch	D_T = ______ pch
Capacity Used =	$100 (C_T/M_3)$ = ______ %	$100 (D_T/M'_3)$ = ______ %
Impedance Factor from Fig. 3	P_3 = ______	P'_3 = ______

Example 3

This example is a rural four-way intersection with prevailing speeds in the 55 mph (90 kpm) range. Benton Highway leads to a small community to the west. Mill Road leads to numerous farms and to a small housing development to the north. There are grades of +2% for the northbound approach and -2% for the southbound approach. Benton Highway is level. Sight distance is good and the minor street is controlled by YIELD signs. Mill Road has been recently widened in the vicinity of the intersection to provide a right turn lane for the southbound approach and a left turn lane for the northbound.

Problem

Evaluate the adequacy of the intersection during the evening peak period from 4 to 5 pm.

Analysis

The "Unsignalized Intersection Capacity Calculation Form" is used. A sketch plan of the intersection is drawn, noting the grades that exist. Volume data are obtained. In this case, a turning movement count was made from 4 to 5 pm on Wednesday, November 8. A classification count for the entire intersection was also made. These counts indicated a composition of 85% passenger cars, 12% single unit trucks, and 3% trucktrailers.

The volume data need to be converted to pch in accord with Table 1. The factor for the Benton Highway is:

$$(0.85 \times 1.0) + (0.12 \times 1.5) + (0.03 \times 2.0) = 1.09$$

The computations for the north and southbound approaches of Mill Road are:

$$(0.85 \times 1.2) + (0.12 \times 2.0) + (0.03 \times 3.0) = 1.35$$

and

$$(0.85 \times 0.9) + (0.12 \times 1.2) + (0.03 \times 1.5) = 0.95$$

Table 1. Converting Existing Traffic Flow to Passenger Car Equivalents per hour

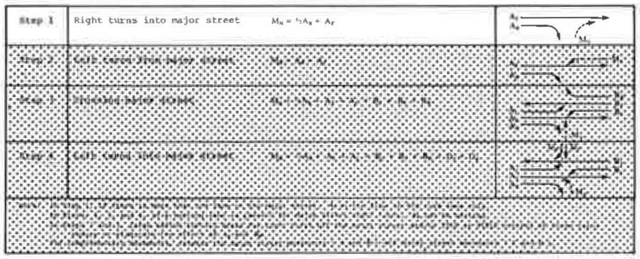
Grade / Type of Vehicle	-4%	-2%	0%	+2%	+4%
Motorcycles	0.3	0.4	0.5	0.6	0.7
Passenger Cars	0.8	0.9	1.0	1.2	1.4
Trucks	1.0	1.2	1.5	2.0	3.0
Truck-Trailers	1.2	1.5	2.0	3.0	6.0
Motor Vehicle*	0.9	1.0	1.1	1.4	1.7

*Approximate value used for estimate calculations.

These factors, when appropriately applied, convert vph to pch and account for the gradient as well.

Step 1 covers right turns from the minor street. Conflicting flows are determined as indicated in Figure 1 and entered on the form (i.e., 130 vph and 120 vph).

Figure 1. Definition of Conflicting Traffic Streams



Step 1 | Right turns into major street | $M_N = \frac{1}{2}A_R + A_T$

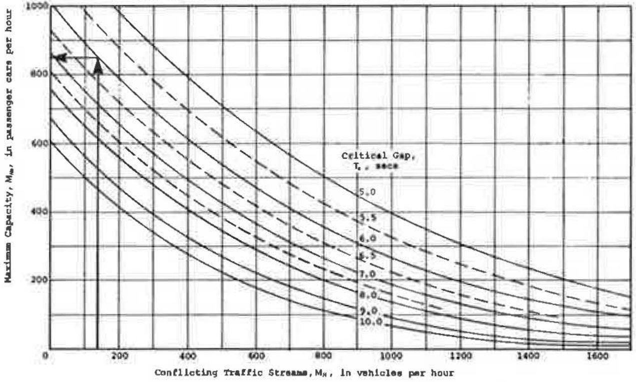
The critical gap is the same for the two minor street approaches and is selected from Table 2. For right turns into a 55 mph (90 kph), four-lane major road, the value is 6.0 seconds.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

The conflicting flows (130 & 120) and the critical gap (6.0) are used to enter Figure 2. Only one set of lines is illustrated. 860 pch and 870 pch are the potential capacities for the right turning movements for the south and north approaches respectively.

Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



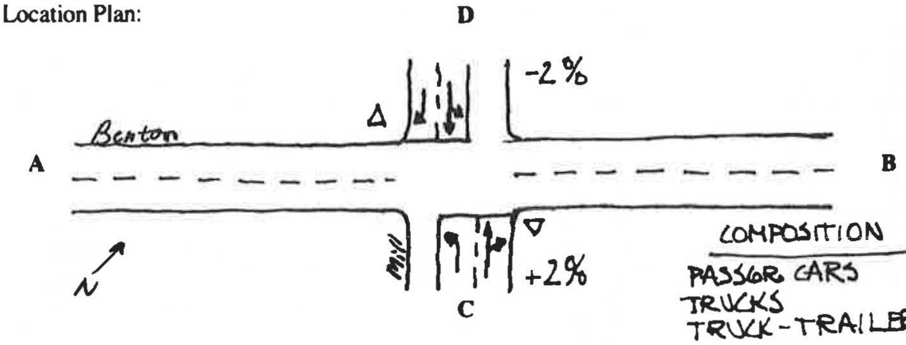
(Continued)

Example 3

Unsignalized Intersection Capacity Calculation Form

Intersection BENTON HIGHWAY AND MILL ROAD

Location Plan:



COMPOSITION	%
PASSGR CARS	85
TRUCKS	12
TRUCK-TRAILERS	3

Counts:
Date NOVEMBER 8
Day WEDNESDAY
Time 4-5 PM
Control YIELD
Prevailing Speed 55 MPH

Hourly Demand Traffic Volumes from 4 to 5, p m

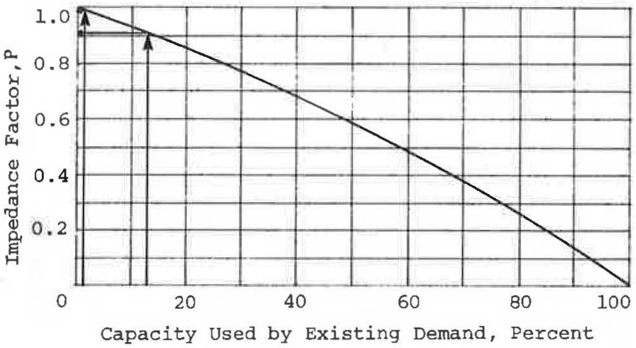
Approach	A			B			C			D		
Movement	A_L	A_T	A_R	B_L	B_T	B_R	C_L	C_T	C_R	D_L	D_T	D_R
Volume	60	120	20	40	100	40	20	40	10	10	20	120
pch (see Table 1)	65	▨	▨	44	▨	▨	27	54	14	10	19	114

Step 1 **Right Turn from C/D**	C_R	D_R
Conflicting Flows = M_H =	½ A_R + A_T =	½ B_R + B_T =
(from Fig. 1)	10 + 120 = 130 vph	20 + 100 = 120 vph
Critical Gap from Table 2 T_g =	6.0 sec	6.0 sec
Capacity from Fig. 2 =	$M_{No} = M_1$ = 860 pch	$M'_{No} = M'_1$ = 870 pch
Demand =	C_R = 14 pch	D_R = 114 pch
Capacity Used =	$100 (C_R/M_1)$ = 2 %	$100 (D_R/M'_1)$ = 13 %
Impedance Factor from Fig. 3 =	P_1 = 098	P'_1 = 091
C — Shared Lane – See Step 3		
D — No Shared Lane – Available Reserve	$M_1 - C_R$ = ____ pch	$M'_1 - D_R$ = 756 pch
Delay & Level of Service (Table 3)	☐	NO DELAY A
Step 2 **Left Turn from B/A**	B_L	A_L
Conflicting Flows = M_H =	A_R + A_T =	B_R + B_T =
(from Fig. 1)	20 + 120 = 140 vph	40 + 100 = 140 vph
Critical Gap from Table 2 T_g =	5.5 sec	5.5 sec
Capacity from Fig. 2 =	$M_{No} = M_2$ = 930 pch	$M'_{No} = M'_2$ = 930 pch
Demand =	B_L = 44 pch	A_L = 65 pch
Capacity Used =	$100 (B_L/M_2)$ = 5 %	$100 (A_L/M'_2)$ = 7 %
Impedance Factor from Fig. 3 =	P_2 = 097	P'_2 = 095
Available Reserve =	$M_2 - B_L$ = 886 pch	$M'_2 - A_L$ = 865 pch
Delay & Level of Service (Table 3)	NO DELAY A	NO DELAY A
Step 3 **Thru Movement from C/D**	C_T	D_T
Conflicting Flows = M_H =	½A_R + A_T + A_L + B_L + B_T + B_R	½B_R + B_T + B_L + A_L + A_T + A_R
(from Fig. 1)	___ + ___ + ___ + ___ + ___ + ___	___ + ___ + ___ + ___ + ___ + ___
(M_T & M'_T are used in Step 4)	$M_H = M_T$ = ____ vph	$M_H = M'_T$ = ____ vph
Critical Gap from Table 2 T_g =	____ sec	____ sec
Capacity from Fig. 2 =	M_{No} = ____ pch	M'_{No} = ____ pch
Adjust for Impedance	$M_{No} \times P_2 \times P'_2 = M_3$ = ____ pch	$M'_{No} \times P'_2 \times P_2 = M'_3$ = ____ pch
Demand =	C_T = ____ pch	D_T = ____ pch
Capacity Used =	$100 (C_T/M_3)$ = ____ %	$100 (D_T/M'_3)$ = ____ %
Impedance Factor from Fig. 3	P_3 = ____	P'_3 = ____

(Example 3)

Impedance factors must be calculated for both approaches for later use in Step 4. The right turn demands in pch are brought down from the data table. The percent of capacity used is computed for each approach - 2% for the south approach and 13% for the other. These values are used to enter Figure 3 to obtain the impedance factors of 0.98 and 0.91.

Figure 3. Capacity Reduction caused by Congestion



The northbound approach right turns share the right lane with the thru movement. This is indicated in the space to the left.

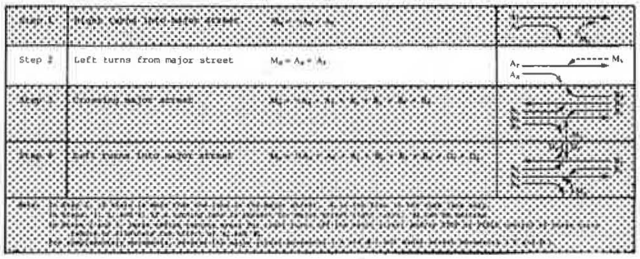
The southbound approach has an exclusive right turn lane. The demand is subtracted from the capacity to yield the available reserve (870-114=756 pch). Comparing this reserve to Table 3 indicates a Level of Service A and no delay.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays
0 to 99	E	Very Long traffic delays
Less than 0	—	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

In Step 2, the left turns from the major street are treated. The conflicting volumes are determined in accord with Figure 1 resulting in conflicting flows of 140 vph for each left turn.

Figure 1. Definition of Conflicting Traffic Streams



[illegible]	[illegible]	[illegible]	
Step 2	Left turns from major street	[illegible]	
[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	
[illegible]			

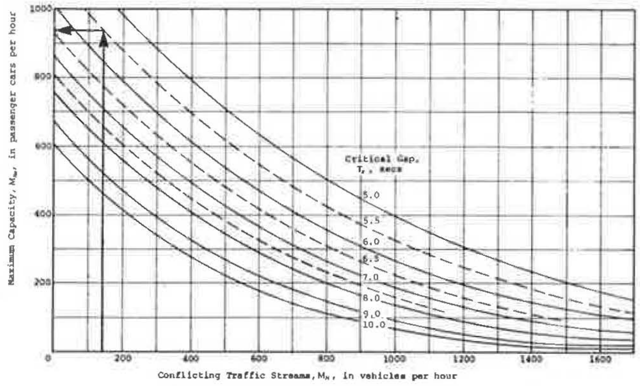
The critical gap of 5.5 seconds is obtained from Table 2.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

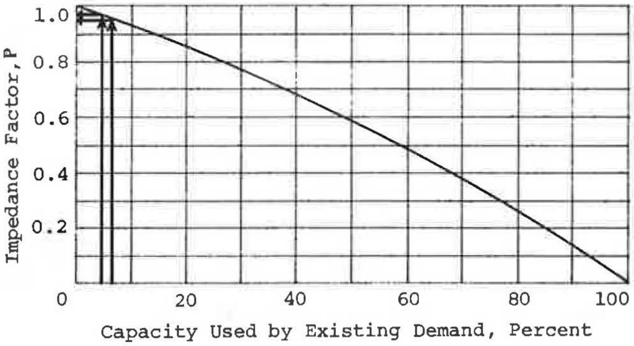
Using the above values, Figure 2 is entered to provide capacities of 930 pch for each left turn.

Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



Impedance factors are necessary for later steps. The percent of capacity being used is calculated and impedance factors obtained from Figure 3.

Figure 3. Capacity Reduction caused by Congestion



The available reserve for each left turn is then calculated by subtracting the demand from the capacity. These available reserves of 886 pch and 865 pch respectively, when compared to Table 3, indicate an excellent operating condition with Level of Service **A** and no expected delay.

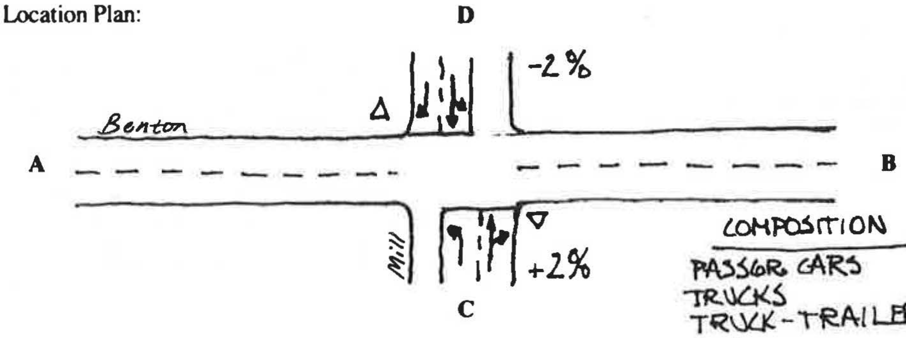
(Continued)

Example 3

Unsignalized Intersection Capacity Calculation Form

Intersection BENTON HIGHWAY AND MILL ROAD

Location Plan:



COMPOSITION	%
PASSGR. CARS	85
TRUCKS	12
TRUCK-TRAILERS	3

Counts:
Date NOVEMBER 8
Day WEDNESDAY
Time 4-5 PM
Control YIELD
Prevailing Speed 55 MPH

Hourly Demand Traffic Volumes from 4 to 5, p m

Approach	A			B			C			D		
Movement	A_L	A_T	A_R	B_L	B_T	B_R	C_L	C_T	C_R	D_L	D_T	D_R
Volume	60	120	20	40	100	40	20	40	10	10	20	120
pch (see Table 1)	65			44			27	54	14	10	19	114

Step 1 **Right Turn from C/D**	C_R	D_R
Conflicting Flows = M_H = (from Fig. 1)	$\frac{1}{2} A_R + A_T =$ 10 + 120 = 130 vph	$\frac{1}{2} B_R + B_T =$ 20 + 100 = 120 vph
Critical Gap from Table 2 T_g =	6.0 sec	6.0 sec
Capacity from Fig. 2 =	$M_{No} = M_1 =$ 860 pch	$M'_{No} = M'_1 =$ 870 pch
Demand =	$C_R =$ 14 pch	$D_R =$ 114 pch
Capacity Used =	$100 (C_R/M_1) =$ 2 %	$100 (D_R/M'_1) =$ 13 %
Impedance Factor from Fig. 3 =	$P_1 =$ 0.98	$P'_1 =$ 0.91
C Shared Lane – See Step 3		
D No Shared Lane – Available Reserve	$M_1 - C_R =$ ______ pch	$M'_1 - D_R =$ 756 pch
Delay & Level of Service (Table 3)	☐	NO DELAY A

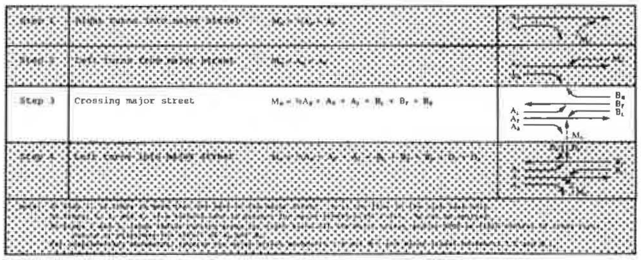
Step 2 **Left Turn from B/A**	B_L	A_L
Conflicting Flows = M_H = (from Fig. 1)	$A_R + A_T =$ 20 + 120 = 140 vph	$B_R + B_T =$ 40 + 100 = 140 vph
Critical Gap from Table 2 T_g =	5.5 sec	5.5 sec
Capacity from Fig. 2 =	$M_{No} = M_2 =$ 930 pch	$M'_{No} = M'_2 =$ 930 pch
Demand =	$B_L =$ 44 pch	$A_L =$ 65 pch
Capacity Used =	$100 (B_L/M_2) =$ 5 %	$100 (A_L/M'_2) =$ 7 %
Impedance Factor from Fig. 3 =	$P_2 =$ 0.97	$P'_2 =$ 0.95
Available Reserve =	$M_2 - B_L =$ 886 pch	$M'_2 - A_L =$ 865 pch
Delay & Level of Service (Table 3)	NO DELAY A	NO DELAY A

Step 3 **Thru Movement from C/D**	C_T	D_T
Conflicting Flows = M_H = (from Fig. 1) (M_T & M'_T are used in Step 4)	$\frac{1}{2}A_R + A_T + A_L + B_L + B_T + B_R$ 10 + 120 + 60 + 40 + 100 + 40 $M_H = M_T =$ 370 vph	$\frac{1}{2}B_R + B_T + B_L + A_L + A_T + A_R$ 20 + 100 + 40 + 60 + 120 + 20 $M_H = M'_T =$ 360 vph
Critical Gap from Table 2 T_g =	7.0 sec	7.0 sec
Capacity from Fig. 2 =	$M_{No} =$ 510 pch	$M'_{No} =$ 515 pch
Adjust for Impedance	$M_{No} \times P_2 \times P'_2 = M_3 =$ 470 pch	$M'_{No} \times P'_2 \times P_2 = M'_3 =$ 475 pch
Demand =	$C_T =$ 54 pch	$D_T =$ 19 pch
Capacity Used =	$100 (C_T/M_3) =$ 11 %	$100 (D_T/M'_3) =$ 4 %
Impedance Factor from Fig. 3	$P_3 =$ 0.92	$P'_3 =$ 0.97

(Example 3)

Conflicting flows are determined for both entering minor street approaches as indicated in Figure 1. Half of the near side right turns plus all of the rest of the entering major street traffic yields 370 vph and 360 vph respectively. Note these values are also labeled M_T and M'_T and are used later in Step 4.

Figure 1. Definition of Conflicting Traffic Streams



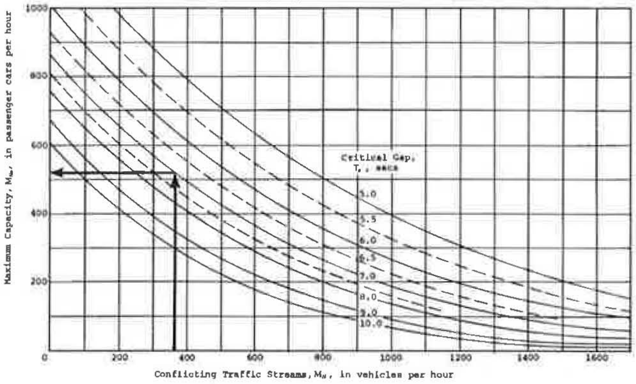
The critical gap of 5.5 seconds is obtained from Table 2.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

The above values are used to enter Figure 2 to obtain potential capacity. Note that only the first entry is illustrated on Figure 2 because the lines are very similar. The two potential capacities are 510 and 515 pch respectively. These values must be adjusted for the impedance caused by the left turns off of the major street. The impedance factors (P_2 and P'_2) were determined in Step 2 and are 0.97 and 0.95 respectively. Each thru movement must be multiplied by both factors to yield the exclusive lane capacity.

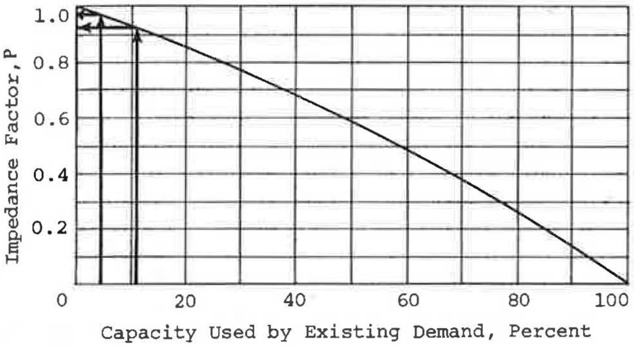
Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



Additional impedance factors must be determined for use in Step 4. Demand values of 54 pch and 19 pch are brought down from the data table and the percent of capacity is determined (11% and 4%).

These percentages are used to enter Figure 3 to obtain impedance factors of 0.92 and 0.97.

Figure 3. Capacity Reduction caused by Congestion



Example 3 Unsignalized Intersection Capacity Calculation Form (continued)

Step 3 (Continued)	C_T ↑	D_T ↓
____ No Shared Lane Available Reserve = Delay & Level of Service (Table 3)	$M_3 - C_T =$ ____ pch ____ ☐	$M'_3 - D_T =$ ____ pch ____ ☐
D Shared Lane with Left Turn See Step 4		
Shared Lane Demand = C Shared Lane with Right Turn Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_R + C_T = C_{RT} =$ 68 pch $M_{13} = \frac{(C_R + C_T)}{(C_R/M_1) + (C_T/M_3)}$ $M_{13} =$ 518 pch $M_{13} - C_{RT} =$ 450 pch LITTLE DELAY A	$D_R + D_T = D_{RT} =$ ____ pch $M'_{13} = \frac{(D_R + D_T)}{(D_R/M'_1) + (D_T/M'_3)}$ $M'_{13} =$ ____ pch $M'_{13} - D_{RT} =$ ____ pch ____ ☐
Step 4 **Left Turn from C/D**	C_L ↰	D_L ↘
Conflicting Flows = M_H = (M_T & M'_T were calculated in Step 3) Critical Gap from Table 2 T_g = Capacity from Fig. 2 = Adjust for Impedance	$M_T + D_T + D_R =$ 370 + 19 + 114 = 503 vph 8.0 sec $M_{No} =$ 340 pch $M_{No} \times P_2 \times P'_2 \times P'_1 \times P'_3 = M_4$ $M_4 =$ 277 pch	$M'_T + C_T + C_R =$ 360 + 54 + 14 = 428 vph 8.0 sec $M'_{No} =$ 385 pch $M'_{No} \times P'_2 \times P_2 \times P_1 \times P_3 = M'_4$ $M'_4 =$ 320 pch
C No Shared Lane Demand = Available Reserve = Delay & Level of Service (Table 3)	$C_L =$ 27 pch $M_4 - C_L =$ 250 pch AVERAGE DELAY C	$D_L =$ ____ pch $M'_4 - D_L =$ ____ pch ____ ☐
Shared Lane Demand = ____ Shared Lane with Thru Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_T + C_L = C_{TL} =$ ____ pch $M_{34} = \frac{(C_T + C_L)}{(C_T/M_3) + (C_L/M_4)}$ $M_{34} =$ ____ pch $M_{34} - C_{TL} =$ ____ pch ____ ☐	$D_T + D_L = D_{TL} =$ ____ pch $M'_{34} = \frac{D_T + D_L}{(D_T/M'_3) + (D_L/M'_4)}$ $M'_{34} =$ ____ pch $M'_{84} - D_{TL} =$ ____ pch ____ ☐
Shared Lane Demand = ____ Shared Lane with Thru & Right Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_R + C_T + C_L = C_{RTL} =$ ____ pch $M_{134} = \frac{C_R + C_T + C_L}{(C_R/M_1) + (C_T/M_3) + (C_L/M_4)}$ $M_{134} =$ ____ pch $M_{134} - C_{RTL} =$ ____ pch ____ ☐	$D_R + D_T + D_L = D_{RTL} =$ ____ pch $M'_{134} = \frac{D_R + D_T + D_L}{(D_R/M'_1) + (D_T/M'_3) + (D_L/M'_4)}$ $M'_{134} =$ ____ pch $M'_{134} - D_{RTL} =$ ____ pch ____ ☐

Overall Evaluation ____________________

____

____

(Example 3)

The remaining part of Step 3 is on the back of the form. The first section is for exclusive lanes (i.e., no shared lane). Because this example does not have exclusive lanes for the thru movement, this section is omitted. The second section is for a shared lane with left turns. The southbound approach (D) has all thru movements in the left lane, therefore a "D" is inserted in the space at the far left. The analysis of this shared lane will take place in Step 4.

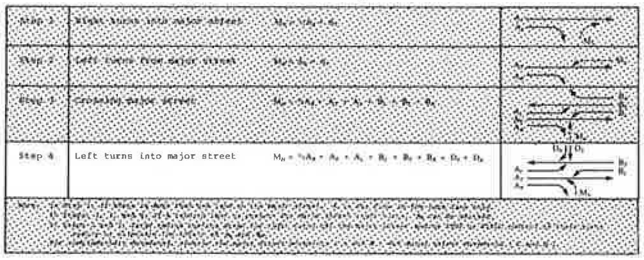
In the northbound approach (C), thru and right turns share a single lane. A "C" is inserted in the space at the far left and this section is completed for this approach. The capacity of the shared lane is computed as shown on the form. A programmable calculator program for this equation is presented in an appendix following this example. The result of the computation in this example is 518 pch. The shared lane demand (sum of thru and right turns) is obtained from the data table and equals 68 pch. The difference (450 pch) is the available reserve, which in Table 3 indicates a Level of Service **A** and little delay.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays·
0 to 99	E	Very Long traffic delays
Less than 0	—	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

Step 4 analyzes the left turns from the minor street. These turns have many conflicting flows as shown in Figure 1. The major street flows have been summed in Step 3 and labeled M_T and M_T', respectively. These values are entered in Step 4 and the opposing approach thru and right turns are added. For the northbound approach, the conflicting flow totals 503 vph while there are 428 vph in conflicting flows for the southbound approach.

Figure 1. Definition of Conflicting Traffic Streams



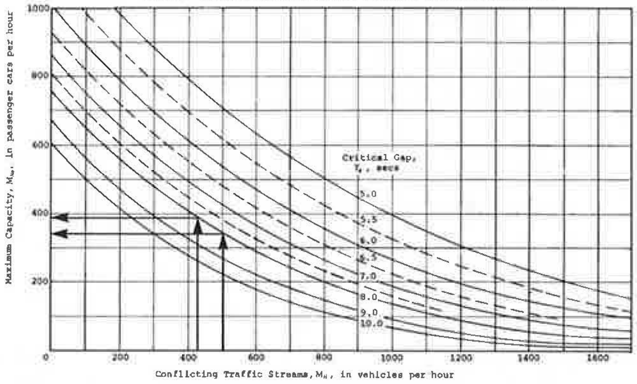
The critical gap (8.0 seconds) is selected from Table 2.

Table 2. Critical Gap for Passenger Cars, in seconds

Vehicle Maneuver and Type of Control	Prevailing Speed			
	30 mph (50 kph)		55 mph (90 kph)	
	Major Road		Major Road	
	2 lanes	4 lanes	2 lanes	4 lanes
Right Turn from Minor Road				
YIELD Control	5.0	5.0	6.0	6.0
STOP Control	6.0	6.0	7.0	7.0
Left Turn from Major Road				
No Control	5.0	5.5	5.5	6.0
Crossing Major Road				
YIELD Control	6.0	6.5	7.0	8.0
STOP Control	7.0	7.5	8.0	9.0
Left Turn from Minor Road				
YIELD Control	6.5	7.0	8.0	9.0
STOP Control	7.5	8.0	9.0	10.0

Step 4 continues by determining the potential capacity from Figure 2. The values from the figure (340 pch and 385 pch) must be adjusted for impedances. Note that of the adjustments for the northbound approach P_2, P_2', P_1', and P_3', the last three (those with primes) were computed during the analysis of the southbound approach. The reverse is true for the other approach. After applying the impedance factors, the resulting capacities are 277 pch and 320 pch.

Figure 2. Maximum Capacity based on Conflicting Volume and Critical Gap



The next section of Step 4 is for exclusive left turn lanes. The northbound approach does have an exclusive left turn lane. Therefore, a "C" is entered in the space at the left, and this section is completed for that approach. The left turn demand (27 pch) is obtained from the data table. Subtracting 27 pch from the capacity of 277 pch yields an available reserve of 250 pch. Table 3 indicates a Level of Service **C** with average traffic delays.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays
0 to 99	E	Very Long traffic delays
Less than 0	—	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

(Continued)

Example 3 Unsignalized Intersection Capacity Calculation Form (continued)

Step 3 (Continued)	C_T ↑	D_T ↓
____ No Shared Lane Available Reserve = Delay & Level of Service (Table 3)	$M_3 - C_T =$ ____ pch ____ ☐	$M'_3 - D_T =$ ____ pch ____ ☐
D Shared Lane with Left Turn See Step 4		
Shared Lane Demand = C Shared Lane with Right Turn Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_R + C_T = C_{RT} =$ 68 pch $M_{13} = \dfrac{(C_R + C_T)}{(C_R/M_1) + (C_T/M_3)}$ $M_{13} =$ 518 pch $M_{13} - C_{RT} =$ 450 pch LITTLE DELAY $\boxed{\text{A}}$	$D_R + D_T = D_{RT} =$ ____ pch $M'_{13} = \dfrac{(D_R + D_T)}{(D_R/M'_1) + (D_T/M'_3)}$ $M'_{13} =$ ____ pch $M'_{13} - D_{RT} =$ ____ pch ____ ☐
Step 4 **Left Turn from C/D**	C_L ↰	D_L ↳
Conflicting Flows = M_H = (M_T & M'_T were calculated in Step 3) Critical Gap from Table 2 T_g = Capacity from Fig. 2 = Adjust for Impedance	$M_T + D_T + D_R =$ 370 + 19 + 114 = 503 vph 8.0 sec $M_{No} =$ 340 pch $M_{No} \times P_2 \times P'_2 \times P'_1 \times P'_3 = M_4$ $M_4 =$ 277 pch	$M'_T + C_T + C_R =$ 360 + 54 + 14 = 428 vph 8.0 sec $M'_{No} =$ 385 pch $M'_{No} \times P'_2 \times P_2 \times P_1 \times P_3 = M'_4$ $M'_4 =$ 320 pch
C No Shared Lane Demand = Available Reserve = Delay & Level of Service (Table 3)	$C_L =$ 27 pch $M_4 - C_L =$ 250 pch AVERAGE DELAY $\boxed{\text{C}}$	$D_L =$ ____ pch $M'_4 - D_L =$ ____ pch ____ ☐
Shared Lane Demand = D Shared Lane with Thru Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_T + C_L = C_{TL} =$ ____ pch $M_{34} = \dfrac{(C_T + C_L)}{(C_T/M_3) + (C_L/M_4)}$ $M_{34} =$ ____ pch $M_{34} - C_{TL} =$ ____ pch ____ ☐	$D_T + D_L = D_{TL} =$ 29 pch $M'_{34} = \dfrac{D_T + D_L}{(D_T/M'_3) + (D_L/M'_4)}$ $M'_{34} =$ 407 pch $M'_{34} - D_{TL} =$ 378 pch SHORT DELAYS $\boxed{\text{B}}$
Shared Lane Demand = ____ Shared Lane with Thru & Right Capacity of Shared Lane = Available Reserve = Delay & Level of Service (Table 3)	$C_R + C_T + C_L = C_{RTL} =$ ____ pch $M_{134} = \dfrac{C_R + C_T + C_L}{(C_R/M_1) + (C_T/M_3) + (C_L/M_4)}$ $M_{134} =$ ____ pch $M_{134} - C_{RTL} =$ ____ pch ____ ☐	$D_R + D_T + D_L = D_{RTL} =$ ____ pch $M'_{134} = \dfrac{D_R + D_T + D_L}{(D_R/M'_1) + (D_T/M'_3) + (D_L/M'_4)}$ $M'_{134} =$ ____ pch $M'_{134} - D_{RTL} =$ ____ pch ____ ☐

Overall Evaluation LEFT TURNS FROM MILL ROAD HAVE SOME DELAY (LOS C AND B). ALL OTHER MOVEMENTS ARE AT LOS A

(Example 3)

The next section of the form is for shared lanes with thru and left turns as is the case in this example. A "D" is entered at the left and the shared lane capacity formula is employed to obtain a capacity of 407 pch. The combined demand of thru and left turns is 29 pch. The available reserve is 378 which Table 3 indicates as Level of Service B with short traffic delays.

The overall evaluation indicates some delay to left turns from Mill Road. All other movements have LOS A. The intersection operates satisfactorily at the present time.

Table 3. Level of Service and Expected Delay for Reserve Capacity Ranges

Reserve Capacity	Level of Service	Expected Traffic Delay
400 or more	A	Little or no delay
300 to 399	B	Short traffic delays
200 to 299	C	Average traffic delays
100 to 199	D	Long traffic delays
0 to 99	E	Very Long traffic delays
Less than 0	—	Failure - extreme congestion
(Any value)	F	Intersection blocked by external causes

APPENDIX

Program for Computing Shared Lane Capacity

The following program was developed for a Hewlett-Packard Model 33E Programmable Calculator to solve the Shared Lane Capacity equation shown below and calculate the Shared Lane Demand and Available Reserve.

Capacity of Shared Lane = $M_{134} = \dfrac{C_R + C_T + C_L}{(C_R/M_1) + (C_T/M_3) + (C_L/M_4)}$

Where:

C_R, C_T, C_L = Demand for right, thru, and left movements respectively

M_1, M_3, M_4 = Exclusive lane capacity for right, thru, and left movements respectively.

Shared Lane Demand = $C_{RTL} = C_R + C_T + C_L$

Available Reserve = $M_{134} - C_{RTL}$

Program

Key Entry	Display
f CLEAR PRGM	00-
STO 1	01- 23 1
STO 0	02- 23 0
R/S	03- 74
STO 2	04- 23 2
STO + 0	05- 23 51 0
R/S	06- 74
STO 3	07- 23 3
STO + 0	08- 23 51 0
RCL 0	09- 24 0
STO 4	10- 23 4
R/S	11- 74
g X=0	12- 15 71
1	13- 1
STO ÷ 1	14- 23 71 1
R/S	15- 74
g X=0	16- 15 71
1	17- 1
STO ÷ 2	18- 23 71 2
R/S	19- 74
g X=0	20- 15 71
1	21- 1
STO ÷ 3	22- 23 71 3
RCL 3	23- 24 3
RCL 2	24- 24 2
RCL 1	25- 24 1
+	26- 51
+	27- 51
STO ÷ 0	28- 23 71 0
RCL 0	29- 24 0
RCL 4	30- 24 4
-	31- 41
RCL 0	32- 24 0
GTO 00	33- 13 00

Registers							
R_0	M_{134}	R_1	C_R/M_1	R_2	C_T/M_3	R_3	C_L/M_4
R_4	C_{RTL}	R_5		R_6		R_7	

User Instructions

Step	Instruction	Input	Keys	Output
1	Key in program			
2	Set display		f FIX 0	
3	Initialize		f PRGM	
4	Input Right Turn Demand	C_R	R/S	
5	Input Thru Demand	C_T	R/S	
6	Input Left Turn Demand Display Shared Lane Demand	C_L	R/S	C_{RTL}
7	Input Right Turn Capacity	M_1	R/S	
8	Input Thru Capacity	M_3	R/S	
9	Input Left Capacity Display Shared Lane Capacity	M_4	R/S	M_{134}
10	Exchange X and Y Display Available Reserve		X↔Y	$M_{134} - C_{RTL}$
11	For New Problem, go to Step 4			

NOTE: All input entries must be entered. Where only two movements share a lane (e.g. Right and Thru), enter 0 for missing Demand and Capacity.

NOTE: For T intersections enter 0 for C_T and enter M_3 for M_3 and M_4.

Example 1: Single lane shared by all three movements

$C_R = 47$
$C_T = 98$
$C_L = 23$

$M_1 = 865$
$M_3 = 299$
$M_4 = 189$

Keystrokes	Display	Comment
47 R/S	47.	Enter C_R
98 R/S	98.	Enter C_T
23 R/S	168.	Enter C_L - Display shows Shared Lane Demand
865 R/S	865.	Enter M_1
299 R/S	299.	Enter M_3
189 R/S	333.	Enter M_4 - Display shows Shared Lane Capacity
X↔Y	165.	Display shows Available Reserve

Example 2: Lane shared by Thru and Left movements only

$C_R = 0$
$C_L = 98$
$C_T = 23$

$M_1 = 0$
$M_3 = 299$
$M_4 = 189$

Keystrokes	Display	Comment
0 R/S	0.	Enter 0 for no Right Turn Demand
98 R/S	98.	Enter C_T
23 R/S	121.	Enter C_L - Display shows Shared Lane Demand
0 R/S	1.	Enter 0 for no Right Turn Capacity
299 R/S	299.	Enter M_3
189 R/S	269.	Enter M_4 - Display shows Shared Lane Capacity
X↔Y	148.	Display shows Available Reserve

Example 3: T intersections with lane shared by Right and Left turns

$C_R = 132$
$C_T = 0$
$C_L = 44$

$M_1 = 720$
$M_3 = 231$

Keystrokes	Display	Comment
132 R/S	132.	Enter C_R
0 R/S	0.	Enter 0 for no Thru movement
44 R/S	176.	Enter C_L - Display shows Shared Lane Demand
720 R/S	720.	Enter M_1
231 R/S	231.	Enter M_3
231 R/S	471.	Enter M_3 again - Display shows Shared Lane Capacity
X↔Y	295.	Display shows Available Reserve

DISCUSSION

Introduction and Concepts

This chapter describes the various factors which influence bus and rail transit capacity. It summarizes results of previous studies, presents basic analytical relationships, and sets forth transit capacity guidelines for busways, freeways, arterial streets and terminals. It also contains summary materials pertaining to rail transit. The material in the chapter may be used to estimate:

- total passenger flow based on roadway condition, and mix of cars and buses
- effects of bus flows on intersection capacity
- bus berth requirements along downtown busways, at terminals, and at bus stops on city streets
- passenger movements on rail transit lines for varying car sizes, train length, service frequency, and loading conditions.

Definitions

Following are definitions of the important terms which relate to transit capacity.

Person Capacity - The maximum number of persons that can be carried past a given location during a given time period under specified operating conditions without unreasonable delay, hazard, or restriction. Usually measured in terms of persons per hour.

Person Level of Service - The quality of service offered the passenger within a transit vehicle, and determined by the available space per passenger.

Uninterrupted Flow - Transit vehicles moving along a roadway or track without stopping.

Interrupted Flow - Transit vehicles moving along a roadway or track and having to make service stops at regular intervals.

Service Time - The time, in seconds, for a passenger to board or to alight a transit vehicle.

Maximum Load Point - The point along a transit route at which the greatest number of passengers is being carried.

Dwell Time - The time, in seconds, that a transit vehicle is stopped for the purpose of serving passengers.

Seat Capacity - The number of passenger seats on a transit vehicle.

Percent Standing - The number of standing passengers expressed as a percentage of the number of seats.

Crush Capacity - The number of passengers carried by a transit vehicle with conditions at Level of Service F; the maximum number that can physically be accommodated.

Person Movement

In assessing the role and capacities of public transport, it is important to view each roadway or transit facility in terms of people carried, that is, person movement. This calls for knowledge of the occupancy of each transit vehicle, as well as the number of vehicles. For example, an urban freeway lane carrying 1800 passenger cars per lane per hour with an average occupancy of 1.5 persons would have a person movement of 2700 people per hour. Likewise, an arterial street carrying 600 automobiles per hour and 50 buses per hour, with occupancy of 1.5 and 40 respectively, would have a total person movement of 2900 persons per hour, with approximately 70 percent being carried by public transport.

Person Capacity

The person capacity for any given transport mode can be defined as "the maximum number of people that can be carried past a given location during a given time period under specified operating conditions without unreasonable delay, hazard, or restriction". More specifically, it depends on the number of vehicles per hour that can pass a given point, and their occupancy. Person capacity is a function of the type of vehicle and its size, and passenger carrying ability of each vehicle, and of frequency or headway of operation (1). The number in parentheses refers to the list of references at the end of the chapter. Thus, with a fixed number and type of vehicles passing a point, an increase in average vehicle occupancy increases person capacity.

The mix of automobiles and transit vehicles in the traffic stream results from the choice of travel mode by the traveler and, in the case of the bus, a determination by the transit operator of the number of transit vehicles to be scheduled over the facility to handle adequately the persons desiring to travel by the transit mode. The patronage that can be carried by a given bus or rail line, therefore, reflect the operating policy of the transit property with respect to minimum service frequency and passenger loading conditons (i.e., number of standees).

Observed Person and Bus Flows

The importance of public transport in increasing the person capacity of transport corridors is well documented. Although buses and rail transit cars require more room per vehicle on the street or highway because of their size and operating characteristics than private automobiles, transit vehicles carry many more passengers per unit than automobiles and, therefore, can reduce the total number of vehicles in the traffic stream.

Illustrative examples of various types of transit vehicles may be found in Table 1. The typical 40 foot (12 m) urban transit bus can normally seat 53 passengers and can carry up to 32 additional standees. Similarly, a 60 foot (18 m) articulated bus can carry 69 passengers and 41 standees. The total passenges carried will vary, depending upon bus and rail car design, and the tradeoff between seated capacity and total capacity.

Table 2 gives levels of service for conventional 40 foot (12 m) buses, based on 53 passengers per bus and 320 square feet (30 m^2), gross per vehicle. These levels of service are from the perspective of passengers on the vehicle, rather than the number of vehicles in a given channel. Similar measures for rail transit vehicles are given in subsequent sections.

Public transport vehicles carry a substantial number of peak-hour person trips across the downtown cordon, and along many urban freeways, arterials and downtown streets.

Table 1. Characteristics of Transit Vehicles

Type of Vehicle or Train	Length		Width		Typical Capacity[a]			Remarks
	ft	m	ft	m	Seats	Standees	Total	
Minibus-short haul	19.5	5.95	7.7	2.35	18	12	30	
Transit bus	30.0	9.15	8.0	2.45	36	19	55	
	35.0	10.65	8.0	2.45	45	25	80	
	40.0	12.20	8.5	2.60	53	32	85	
Articulated transit bus	54.1	16.50	8.2	2.50	48	124	172	European model
	60.0	18.30	8.5	2.60	69	41	110	US specifications
Streetcar train	140.0	42.70	9.0	2.75	177	198	375	3-car P.C.C. train[b]
Light rail car train	220.0	67.00	8.9	2.70	204	366	570	3-car train, 6-axle car, US
	170.0	51.80	7.7	2.35	128	372	500	2-car train, 8-axle car, Europe
Rapid transit train	605.0	184.40	10.0	3.05	500	1,700	2,200	10-car train, New York IND
	600.0	182.90	10.3	3.15	616	2,000	2,616	8-car train, Toronto
	700.0	213.35	10.5	3.20	720	1,280	2,000	10-car train, BART, San Francisco

[a] In any transit vehicle the total passenger capacity can be increased (and passenger comfort decreased) by removing seats and making more standing room available, and vice versa.

[b] Presidents' Conference Cars.

Source: Ref. (29)

Table 2. Levels of Service for Bus Transit

Peak-Hour Level of Service	Passengers/Seat (Approximate)	Approximate ft.^{2}/Passenger
A	0.00 - 0.50	11.9 or more
B	0.51 - 0.75	11.8 - 8.0
C	0.76 - 1.00	7.9 - 6.1
D	1.01 - 1.25	6.0 - 4.8
E (Scheduled Load)	1.26 - 1.50	4.7 - 4.0
F (Crush Load)	1.51 - 1.60	< 4.0

(1 foot = .305 meter)

Table 3 indicates the peak period use of public transport, bus and rail combined, by persons entering the central business districts of selected cities in Canada and the United States (2). Transit carries more than two-thirds of all peak-hour travelers to or from the New York, Chicago, Philadelphia, and Toronto downtown areas, and more than a third of all peak hour travelers entering or leaving most other CBD's. The variations in transit use reflect variations in population, central business district employment, extent of bus and rail transit services, and geographic characteristics.

Table 4 presents bus use statistics for urban freeways (3). Buses carry over 85 percent of all peak hour person trips through the Lincoln Tunnel, account for about half of all peak hour travelers on the San Francisco-Oakland Bay Bridge, the Shirley Highway (Virginia), Ben Franklin Bridge (Philadelphia), and the Long Island and Gowanus Expressways (New York City), and for more than a quarter of all passengers on radial freeways in many other larger cities.

Buses carry an even higher proportion of peak hour travelers on city arterial streets as shown in Table 5. More than 85 percent of peak hour person trips on Hillside Avenue, New York City; State Street, Chicago; Market Streets in Philadelphia and San Francisco; and 14th Street and Pennsylvania Avenue in Washington, D.C., are on buses. Buses accommodate more than half of all peak-hour person-trips on downtown streets in many other cities (4).

Table 3. Peak Hour Use of Public Transit by Persons Entering or Leaving the Central Business District

Urban Area	Year	Percent by Public Transport
New York, NY	1974	90[a]
Chicago, IL	1974	82[a]
Toronto, ONT	1970	68[a]
Boston, MA	1974	49[a]
Ottawa, ONT	1974	44
Cleveland, OH	1970	44[a]
Vancouver, B.C.	1970	40
Los Angeles, CA	1974	37
Detroit, MI	1974	35
Denver, CO	1977	30
Washington, DC	1968	29[b]
Dallas, TX	1971	28
Milwaukee, WI	1974	25
Minneapolis, MN	1965	20
Houston, TX	1971	14

[a] With Rail Transit

[b] Includes Pentagon Area but prior to rail transit

Source: Cordon Counts for each city, compiled in Ref. (2)

Table 4. Peak Hour Bus Volumes on Urban Freeway Facilities, Ranked by Percentage of Total Passengers Carried by Bus, in Dominant Direction of Flow Under Current Conditions

Facility	Metropolitan Area	Vehicles Per Hour		Passengers Carried[a]			Percent Carried By Bus
		Bus	Auto	Bus	Auto	Total	
Lincoln Tunnel	New York	735	3,200	32,560	5,065	37,625	85.5
I-495	New York	490	3,000	21,600	4,750	26,350	82.0
San Francisco-Oakland Bay Bridge	San Francisco-Oakland	327	8,115	13,000	10,400	23,400	55.5
Shirley Highway (I-95)	Washington, D.C.	110	3,200	5,550	4,500	10,050	53.0
Ben Franklin Bridge	Philadelphia	137	4,490	5,065	5,620	10,685	47.5
Long Island Expressway	New York	89	2,710	3,560	4,100	7,660	46.5
Memorial Bridge	Washington, D.C.	100	3,690	4,020	6,650	10,670	37.6
Lions Gate Bridge	Vancouver, B.C.	45	3,300	2,000	4,600	6,600	30.2
Schuylkill Expressway	Philadelphia	78	5,300	2,800	6,650	9,450	29.5
Southeast Expressway	Boston	65	4,200	2,450	6,000	8,450	29.0
I-71	Cleveland	35	3,200	1,850	4,500	6,350	29.0
Golden Gate Bridge	San Francisco	80	6,650	3,750	9,250	13,000	28.8
South Capitol St. Bridge	Washington, D.C.	32	3,335	1,920	5,000	6,920	27.7
George Washington Bridge	New York	108	9,440	4,245	13,215	17,460	24.3
14th St. Bridge	Washington, D.C.	79	6,565	3,295	10,425	13,720	24.0
North Lake Shore Drive	Chicago	80	9,500	4,000	14,200	18,200	22.0
John C. Lodge Freeway	Detroit	40	4,950	1,800	6,920	8,720	20.6
North Central Expressway	Dallas	32	4,000	1,200	5,600	6,800	17.5
Bayshore Freeway	San Francisco	35	6,800	2,270	10,880	13,150	17.3
South Lake Shore Drive	Chicago	24	5,700	1,400	8,000	9,400	14.9
I-5	Seattle	47	9,800	2,300	13,700	16,000	14.4
Hollywood Expressway	Los Angeles	36	7,650	1,755	10,500	12,255	14.4
North Expressway	Atlanta	24	4,550	1,070	6,380	7,450	14.4
East Memorial Shoreway	Cleveland	24	5,800	1,250	8,100	9,350	13.3
Memorial Drive	Houston	11	2,250	500	3,380	3,880	12.9
Stevenson Expressway	Chicago	16	4,600	840	6,900	7,740	10.9
Harbor Freeway	Los Angeles	23	7,200	1,050	10,000	11,050	9.5
I-45N	Houston	19	6,450	875	9,550	10,425	8.4
I-35W	Minneapolis-St. Paul	13	4,950	585	6,900	7,485	7.8
US 59	Houston	13	6,900	600	10,300	10,900	5.5
I-45S	Houston	11	6,000	505	9,000	9,505	5.3
I-10W	Houston	8	5,870	370	8,800	9,170	4.0
Jones Falls Expressway	Baltimore	3	2,780	125	3,900	4,025	3.1
Chrysler Freeway	Detroit	4	5,550	180	7,750	7,930	2.3

[a] Involves assumption in some cases as to car or bus occupancy.

Table 5. Peak Hour Bus Volumes on Urban Arterials, Ranked by Percentage of Total Passengers Carried by Bus, in Dominant Direction of Flow Under Current Conditions

Arterial Location	City	Vehicles Per Hour			Passengers Carried			Percent Carried By Bus
		Bus	Auto	Total	Bus	Auto	Total	
Nicollet Mall	Minneapolis	64	0	64	2,900	0	2,900	100.0
Market Street (East of Broad)	Philadelphia	143[b]	465	608	8,300	695	8,995	92.5
State Street @ Madison	Chicago	151[b]	465	616	6,100	660	6,760	90.0
Hillside Avenue	New York	170[b]	630	800	8,500	950	9,450	90.0
Pennsylvania Ave. @ Seventh	Washington, D.C.	120	600	720	6,000	900	6,900	87.0
Market Street @ Van Ness	San Francisco	155[b]	1,200	1,355	9,900	1,550	11,450	86.5
Main Street @ Fourth Street	Los Angeles	115	720	835	5,850	1,100	6,950	84.0
Main Street @ Harwood Street	Dallas	100	635	735	4,400	900	5,300	83.0
Hill Street @ Seventh Street	Los Angeles	109	800	909	5,250	1,200	6,450	81.5
Broad Street @ Hunter Street	Atlanta	48	290	338	1,920	435	2,355	81.5
Seventh Street @ Main Street	Los Angeles	91	705	796	4,500	1,050	5,550	81.0
Forbes Avenue @ Wood Street	Pittsburgh	47	400	447	2,300	560	2,860	79.5
Fifth Avenue @ Smithfield	Pittsburgh	47	420	467	2,300	590	2,890	79.5
Liberty Street @ Sixth Avenue	Pittsburgh	66	650	716	3,250	910	4,160	78.2
"K" Street NW @ 13th Street	Washington, D.C.	130	1,300	1,430	6,500	1,950	8,450	77.0
Eye Street @ 13th Street	Washington, D.C.	104	1,100	1,204	5,200	1,600	6,800	76.5
Smithfield Street @ Fifth Avenue	Pittsburgh	50	550	600	2,450	770	3,220	76.0
Thirteenth Street @ "F" Street	Washington, D.C.	101	1,050	1,151	5,000	1,600	6,600	75.8
Broadway @ Sixth Street	Los Angeles	78	850	928	4,000	1,390	5,390	74.5
Adams Street Bridge	Chicago	107	785	892	3,425	1,220	4,645	73.7
Granville Street @ Georgia	Vancouver	70	900	970	3,150	1,200	4,350	72.5
Wisconsin Avenue	Milwaukee	78	935	1,013	3,100	1,200	4,300	72.0
Chestnut @ 12th Street	Philadelphia	67	890	957	3,350	1,350	4,700	71.5
State Street @ Roosevelt	Chicago	72	670	742	2,305	935	3,240	71.4
Washington Street @ Wacker	Chicago	108	1,100	1,208	3,800	1,540	5,340	71.4
Wood Street @ Forsyth Ave.	Pittsburgh	55	800	855	2,700	1,120	3,820	70.8
Seventh Street @ Pennsylvania Ave.	Washington, D.C.	80	1,150	1,230	4,000	1,720	5,720	70.0
Main Street @ Pratt	Hartford	75	625	700	1,875	815	2,690	70.0
Jackson Blvd. Bridge	Chicago	88	845	933	2,815	1,325	4,140	68.0

(Continued)

Table 5. (Continued)

		Vehicles Per Hour			Passengers Carried			Percent Carried By Bus
Arterial Location	City	Bus	Auto	Total	Bus	Auto	Total	
Sixth Avenue @ Smithfield	Pittsburgh	33	560	593	1,620	780	2,400	67.6
Eglington Avenue @ Bathurst	Toronto	80	1,200	1,280	3,300	1,700	5,000	66.0
Elm Street @ Harwood	Dallas	80	1,345	1,425	3,500	1,880	5,380	65.2
Sacramento Street	San Francisco	25	410	435	1,000	535	1,535	65.0
Constitution Ave. @ 15th	Washington, D.C.	120	2,200	2,320	6,000	3,300	9,300	64.5
Spring Street @ Seventh Street	Los Angeles	111	1,500	1,611	4,450	2,500	6,950	64.0
Sixteenth Street @ Florida Ave.	Washington, D.C.	80	1,500	1,580	4,000	2,250	6,250	64.0
Fourteenth Street @ Constitution Ave.	Washington, D.C.	80	1,550	1,630	4,000	2,350	6,350	63.0
Connecticut Avenue @ Cathedral Ave.	Washington, D.C.	90	1,800	1,890	4,500	2,700	7,200	62.5
Walnut @ 15th Street	Philadelphia	48	960	1,008	2,400	1,450	3,850	62.5
Commerce Street @ St. Paul	Dallas	72	1,415	1,487	3,300	2,120	5,420	61.0
Sheridan @ Hollywood	Chicago	32	500	532	1,100	700	1,800	61.0
Michigan Avenue @ Roosevelt Rd.	Chicago	77	770	847	1,815	1,210	3,025	60.0
Asylum @ Main Street	Hartford	35	450	485	875	585	1,460	60.0
Michigan Avenue Bridge (Upper Level)	Chicago	116	1,590	1,706	3,580	2,390	5,970	60.0
Sutter Street	San Francisco	63	1,300	1,363	2,500	1,700	4,200	59.5
Madison Avenue @ 42nd Street	New York	96	2,400	2,496	4,800	3,600	8,400	57.1
Second Avenue @ 42nd Street	New York	110	2,800	2,910	5,500	4,200	9,700	56.8
First Avenue @ 44th Street	New York	110	2,800	2,910	5,500	4,200	9.700	56.8
Sixth Avenue @ Figueroa	Los Angeles	29	965	994	1,875	1,430	3,305	56.7
Georgia Avenue @ Granville	Vancouver	45	1,200	1,245	2,000	1,600	3,600	55.5
Clay Street	San Francisco	26	650	676	1,050	850	1,900	55.3
Ninth Street @ Market Street	Philadelphia	22	600	622	1,100	900	2,000	55.0
Second Avenue North	Birmingham, AL	44	1,400	1,444	2,300	1,950	4,250	54.0

(Continued)

Table 5. (Continued)

Arterial Location	City	Vehicles Per Hour			Passengers Carried			Percent Carried By Bus
		Bus	Auto	Total	Bus	Auto	Total	
Grand Avenue @ Temple Street	Los Angeles	24	855	879	1,400	1,215	2,615	53.5
Geary Street	San Francisco	43	1,250	1,293	1,720	1,630	3,350	51.4
Howard Street @ Fayette Street	Baltimore	30	470	500	790	755	1,545	51.0
Marietta @ Spring Street	Atlanta	35	1,050	1,085	1,400	1,580	2,980	47.0
Peachtree @ Ellis	Atlanta	55	1,700	1,755	2,200	2,550	4,750	46.5
Tryon Street	Charlotte, N.C.	40	1,150	1,190	1,200	1,700	2,900	41.4
Eighth Street @ Los Angeles St.	Los Angeles	30	1,155	1,185	1,290	1,835	3,130	41.3
O'Farrell Street	San Francisco	27	1,200	1,227	1,080	1,550	2,630	41.2
Trade Street	Charlotte, N.C.	30	1,030	1,000	1,000	1,500	2,500	40.0
Pratt Street @ Paca Street	Baltimore	64	2,390	2,454	2,215	3,825	6,040	36.7
Charles Street @ Madison St.	Baltimore	33	1,915	1,948	1,480	3,060	4,540	32.6
Lombard Street @ Greene St.	Baltimore	42	1,750	1,792	1,335	2,800	4,135	32.0
Eleventh Street Bridge	Washington, D.C.	54	4,120	4,174	2,870	7,735	10,605	27.1
Cathedral Street @ Eager	Baltimore	36	1,545	1,581	880	2,470	3,350	26.3
St. Paul Street @ Preston	Baltimore	45	2,815	2,860	1,375	4,505	5,880	23.4
Calvert Street @ Lexington	Baltimore	39	2,645	2,684	1,185	4,230	5,415	21.9

[a] Data involves assumptions in some cases as to car or bus occupancy.

[b] Buses operate in more than one lane.

The observations reported in these tables do not necessarily represent maximum possible bus volumes or total traffic volumes. They do, however, clearly indicate that while buses account for a relatively small proportion of the vehicles in a traffic stream, they represent a sizable part of the total person flow.

Operational Experience

The number of buses which can operate past a point in a given period of time varies widely according to specific roadway conditions and operating practices. A summary of both theory and actual practice in bus operations follows (5).

Several studies have analyzed the effects of buses on the capacity of mixed traffic roadways. They have viewed buses in terms of passenger car equivalents, assuming uninterrupted flow and no time losses for passenger discharge or pickup. Ranges in bus capacities or volumes based on these theoretical studies are shown in Table 6. Values for buses stopping in traffic are also shown for comparative purposes (6, 7). When buses do not stop, capacities of 900 or more buses per lane per hour can be achieved on exclusive bus roadways with uninterrupted flow. Theoretical simulation studies based on buses that have 30 second dwell times and operate in platoons of six between stations 0.3 miles (0.5 km) apart, result in capacities ranging from 350 to 400 buses per hour on an exclusive grade-separated busway (8).

Uninterrupted flow studies of concentrated bus movements have determined bus-car equivalency relationships. For instance, a study was made in 1962 by the Port of New York Authority in a single lane of the two lane one way north tube of the Lincoln Tunnel (9). The study site was on a level section at approximately the midpoint of the 1-1/2 mile (2.4 km) long tunnel. Automatic detecting and recording equipment was used to determine the time of passage of the front and rear of each vehicle over two points a few feet apart. A computer program then summarized a variety of vehicle characteristics, including velocity, length, and headway time.

In this single lane carrying 60 percent cars, 32 percent buses and 8 percent trucks, data were collected on 3,200 vehicles. Included were 1,200 samples where cars followed cars, and almost 400 cases of buses followed by buses. The relationship of time headway and speed were compared for these two types of flow. Results showed for cars following cars a minimum headway of 2.39 sec. at 21.6 mph (34.6 kph); for buses following buses, a minimum of 3.49 sec. at 24.2 mph (38.7 kph). The headway difference between the two types of flow was found to range from 1.3 sec. at speeds of 14.0 mph (22.4 kph) to 1.0 sec. at 41.0 mph (65.6 kph), with a 1.1 sec. difference of minimum values.

Comparison of the minimum headway times resulted in a passenger car equivalency of 1.46 for buses. Over a speed range of 14.0 to 41.0 mph (22.4 to 65.6 kph), the equivalent was found to decrease from 1.53 to 1.36, probably because the greater length of buses is a more significant influence at low speeds. In summary, it was found that a car-bus equivalent varies with speeds but that an equivalent of 1.5 cars per bus is representative of tunnel flow.

A nationwide study of the Bureau of Public Roads (now the Federal Highway Adminstration) of mixed traffic flows on expressways carrying relatively large numbers of buses (7), involved detailed recording of speed and spacing of thousands of vehicles. This study indicated an equivalency factor of 1.6 as generally applicable on both expressways and full freeways. This factor appears appropriate to each of the traffic lanes at their normal speeds.

Locations observed included:

1. Route 3 approaches to Lincoln Tunnel, New Jersey (New York City area)
2. Center Tube, Lincoln Tunnel, New Jersey (New York City area)
3. Shoreway West, Cleveland, Ohio
4. Lakeshore Drive, Chicago, Illinois
5. Mark Twain Expressway, St. Louis, Missouri
6. Bayshore Freeway, San Francisco, California
7. San Francisco-Oakland, California, Bay Bridge (temporary exclusive bus lane, lower deck)

An earlier limited study of mixed traffic conducted by the Bureau of Public Roads on the Shirley Highway near Washington, D.C., showed a 1.7 factor, and a test track study by a bus manufacturer showed a value of about 1.4.

The similarity of these several findings indicates that when buses are in motion, either exclusively bus traffic or in mixed traffic, under uninterrupted flow conditions over a broad range of levels of service, their equivalency factors will be approximately 1.6 passenger cars (7).

The capacity or service volume of an exclusive bus lane with uninterrupted flow can be computed by applying the 1.6 car equivalency factor to the computed capacity or corresponding service volume in passenger cars per hour. For example, a roadway lane having a capacity of 1,500 passenger cars per hour would have an equivalency of 940 buses per hour and one within Level of Service C at a service volume of 1,100 cars per hour, an equivalency of 690 buses per hour. This uninterrupted flow volume requires, of course, in the case of a single-lane facility, that bus stops be located off the lane and that adequate acceleration and deceleration lanes be provided.

Bus Stops. The effects of bus stops to pick up

Table 6. Typical Theoretical Bus Volumes

Facility or Source	Buses Per Hour	Headway (Seconds)	Average Bus Stop Spacing (Feet)	Average Bus Speed (mph)	Equivalent Passengers Per Hour[a]
G.M. Proving Grounds: Uninterrupted Flow (Initial Studies)	1,450[b]	2.5	No Stops	33	72,500
Highway Capacity Manual, 1965					
Freeway - Level of Service D	940	3.8	No Stops	33	47,000
Level of Service C	690	5.2	No Stops	40-60	34,500
G.M. Proving Grounds: 6-Bus Platoons, 30-sec On-Line Stops	400	9.0	Variable	15	20,000
Highway Capacity Manual, 1965					
Arterial Streets - 25-sec Loading and 25-sec Clearance	72	50	Not Cited	Not Cited	3,600
Toronto Transit Commission (Planning Criteria)	60	60	500-600	10	3,000

[a] Equivalent passenger volume assumes 50 passengers per bus.

[b] Subsequent studies have reported bus volumes of 900 to 1,000 vehicles per lane per hour, these are consistent with reported flows.

Source: Compiled from various Bus-Use Studies. Summarized in Ref. (11), p. 38

(1 foot = .305 meter
1 mph = 1.6 kph)

Table 7. Observed Peak Hour Bus Volumes

Facility or Source	Buses Per Hour	Headway (Seconds)	Average Bus Stop Spacing (Feet)	Average Bus Speed (mph)	Passengers Per Hour
Lincoln Tunnel - Uninterrupted Flow	735	4.9	No Stops	30	32,560
I-495 (New Jersey) Exclusive Bus Lane - Uninterrupted Flow	485	7.3	No Stops	30-40	21,600
San Francisco - Oakland Bay Bridge	350	10.3	No Stops	30-40	13,000
South Michigan Avenue, Chicago (5-minute rate, some multiple lane use)	228	15	Not Cited	Not Cited	11,400[a]
Hillside Avenue, New York City (Multiple lane use with lightly patronized stops)	170	21	530	Not Cited	8,500[a]
Shirley Highway Busway and 14th Street Bus Lanes, Washington, D.C. area	160	23	900 (in CBD)	35 (Freeway) 6-12 (CBD)	8,000[a]
State Street, Chicago; Market Street, Philadelphia; and Market Street, San Francisco (Multiple lanes)	150	24	300-600	6-10	6,100-9,900
K Street, Washington, D.C.	130	28	500	5-8	6,500[a]
Downtown Streets (Single Lane) with stops (various cities)	90-120	30-40	500	5-10	4,500-6,000[a]

[a] Estimated, assuming 50 passengers per bus

Source: Compiled for various Bus-Use Studies. Summarized in Ref. (4)

(1 foot = .305 meter; 1 mph = 1.6 kph)

and discharge passengers are given in Table 7, which summarizes observed bus volumes on arterials and city streets. The highest volumes, 735 buses per lane per hour in the Lincoln Tunnel and on the Port Authority Bus Terminal access ramp, are achieved on a completely exclusive right-of-way where vehicles make no stops. Where bus stop or layovers are involved, reported volumes are much less.

Stopping a bus to pick up or discharge passengers limits the capacity of a bus lane. Time must be allowed for acceleration, deceleration, and stop clearance as well as the time when the doors are open. Observed transit lane volumes where intermediate stops are made rarely exceed 120 buses per hour, although volumes of 180 buses per hour or more are feasible where stops are short or where buses use two or more lanes and stopped vehicles can be overtaken--with careful management and control of bus operations. These volumes compare with maximum streetcar volumes on city streets some 50 years ago approaching 150 cars per track per hour--under conditions of extensive queueing and platoon loading at heavy stops (10).

Terminals. Usage characteristics of major bus terminals in the United States are summarized in Table 8. During a typical peak hour, New York's 184 berth midtown terminal serves 33,000 entrants; San Francisco's 37 berth Transbay Terminal, 13,000 (before BART); and Chicago's 22 berth 95th and Dan Ryan Terminal, 5,000.

Effects of Buses on Vehicular Capacity

Buses have a reductive effect on vehicular capacity which varies according to their method of operation. In general, the time available for other vehicles will be reduced by the time preempted by buses. This time loss depends upon the number of buses in the traffic flow, and their service time requirements.

Consequently, for uninterrupted flow, buses are the equivalent of 1.6 passenger car units, in the lane in which they operate. At bus stop locations, buses will have a greater reductive effect because of the time involved in discharging and receiving passengers. The equivalency factors for these conditions depend upon the duration of the bus stop and its reductive effect on arterial street green time.

The reductive effects on other vehicles in the lane where local transit buses operate can be summarized as follows:

> Where the buses stop in a lane which is not used by moving traffic (for example in a curb parking lane), the time loss to other vehicles is approximately three to four seconds per bus. For this case, buses would either accelerate or decelerate across the intersection, thereby reducing the impeditive effects to other traffic.
>
> Where buses stop in a normal traffic lane, the time loss involves the dwell time for buses plus a time loss for stopping and starting, and the associated queueing effects on other traffic. The time loss can be estimated from the following equation for the lane in which the buses operate.

Table 8. Principal Central Area Bus Terminals

		Port Authority Bus Terminal, New York, NY	George Washington Bridge Bus Terminal, New York, NY	Greyhound Bus Terminal, Clark and Randolph Sts. Chicago, IL	Transbay Bus Terminal, San Francisco, CA	Dixie Terminal Cincinnati, OH
Development Costs[b]		$58,000,000.00	$15,300,000.00	$8,000,000.00	$11,000,000.00	Not Available
Type of Bus Service		Commuter and Intercity	Commuter and Intercity	Mainly Intercity	Intercity and Commuter	Commuter
Date Completed		1950	1963	1952	1960	Railcars -1921 Buses - 1936
Number of Bus Levels		3	2	1	1	1
Number of Bus Loading Docks		184	43	30	37	6[c]
Contiguous Transportation Facilities		Subway, Local Bus, Auto Parking	Subway, Local Bus	Subway, Local Bus, Curb Parking	Streetcar and Bus, Auto Parking	Local Bus, Auto Parking
Access Connections		Direct Ramp Connections with Lincoln Tunnel	Direct Ramp Connections with George Washington Bridge	Tunnel and Ramp Connections with Garvey St. and Wacker Dr.	Direct Ramp Connections with San Francisco-Oakland Bay Bridge	Direct Ramp Access to Suspension Bridge over Ohio River
Number of Passengers[a]	Daily	105,500	20,000	---	44,000	5,000
	Peak Hour	32,600	4,200	10,000	13,000	1,800
Number of Buses	Daily	3,350	850	---	2,200	195
	Peak Hour	730	108	---	350	48
Average Bus Occupancy	Daily	27.4	23.5	---	20.0	25.4
	Peak Hour	44.1	39.0	---	37.2	37.5
Avg. Number of Buses Per Dock	Daily	18.2	19.6	---	59.5	32.5
	Peak Hour	4.0	2.5	---	9.5	8.0
Avg. Bus Layover Time in Hours	Daily	1.32	1.22	---	0.40	0.16
	Peak Hour	0.25	0.4	---	0.16	0.08
Ancillary Land Uses		Retail Convenience Goods, Restaurants	Retail Convenience Goods, Restaurants	Retail Convenience Goods and Offices over	Retail Convenience Goods	Retail, Offices, Restaurants
Remarks		1,080 cars; saves buses 30 mins. over previous operations	Located over cross Bronx Expressway	Designed to allow office building over station	Prior to 1960 key system taxis used terminal	Former inter-urban rail terminal, shared by rail & bus 1936-1950. Bus only since 1950

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[a] One direction only bus volumes.

[b] Data on maintenance costs and revenues unavailable.

[c] Also four unloading and six loading docks.

$$\text{Time Loss (in seconds per hour)} = (G/C) \times s \times (D + L) \qquad (1)$$

where:

G/C = Green time/Cycle time ratio

s = Buses per hour that stop

D = Average Dwell Time, in seconds

L = Additional Loss due to stopping, starting, and queuing, in seconds (L = 6 seconds, assuming average conditions)

Equivalent passenger car units can be derived from this formula, for various rates of vehicle flow, dwell times, G/C ratio and bus volumes.

Bus Berth Capacity

General Considerations

The service volumes of bus routes, terminals and busways--in persons carried--is generally limited by the ability of stops, or loading areas, to pickup and discharge passengers. Just as the signalized intersection usually determines arterial street capacity, bus route capacity is determined by the passenger service times at major passenger loading and unloading points. For this reason, theoretical bus capacities for uninterrupted flow have little practical application for other than express runs.

Each bus requires a certain amount of service time at stops, which varies with the number of boarding and alighting passengers, door configuration of buses, and methods of fare collection. The minimum safe spacing between buses in motion, and the number of loading positions available at any stop also influence the total number of buses that a given stream can carry. Bus volume may be increased where vehicles can overtake or leave each other in entering or leaving loading positions.

The number of buses per lane per hour and the number of people they carry depend on a variety of roadway and operating factors. These factors include:

1. Type and characteristics of the roadway - mixed traffic versus special busway; expressways versus arterial lanes; extent of signalization, flow restrictions and interferences.
2. Mode of operation - singly or in platoons; on-line versus off-line stations; consistent arrival versus random arrival at designated loading areas.
3. Design of vehicle - seating capacity and door configuration; single versus articulated vehicles; standing versus seated loads; single versus multiple doors; number and height of steps.
4. Clearance between buses - queueing versus nonqueueing operations; low speed versus high-speed operations.
5. Frequency and duration of stops - (including dwell times); dispersed versus concentrated loadings; common or separated passenger boarding and alighting; prepayment versus on-vehicle fare collection; single-coin versus odd-penny fares.
6. Interface between buses and pedestrians at bus station.
7. Bus layover practices at terminals - intercity versus suburban operations; driver relief and schedule recovery requirements.

Analytical Relationships

The following relationships show how the various factors influence the capacity of a downtown busway or bus terminal area. They establish ranges in typical time requirements for each of the operations at a bus berth, and they identify relationships between bus passenger line-haul capacity, boarding and alighting volumes, and major parameters of equipment and facilities. These formulas should be applied to the peak 15 minutes in each rush hour, since this period usually contains the maximum boarding and alighting volumes.

Capacity of a Berth. Capacity of a berth can be estimated as follows:

Minimum headway at bus berth (h')

$$h' = aA + bB + C \qquad (2a)$$

(for two-way flow through doors)

$$h' = aA + C \qquad (2b)$$

(for exiting only; one-way flow)

$$h' = bB + C \qquad (2c)$$

(for boarding only; one-way flow)

Where passengers enter via the front door, and exit via the rear door, the greater of equations 2b or 2c determines minimum headways and dwell times.

Maximum passengers per berth per hour (G)

$$f' = 3600/h' = 3600/(aA + bB + C) \qquad (3a)$$

(for two-way flow)

$$f' = 3600/(aA + C) \qquad (3b)$$

(for exiting only)

$$f' = 3600/(bB + C) \qquad (3c)$$

(for boarding only)

The following relationships apply to a single station, assuming that loading conditions govern. Similar formulas can be derived based upon passenger interchange or unloading:

Maximum passengers per berth per hour

$$G = f'B = (3600)/(bB + C) \qquad (4)$$

Effective berths required (N) to serve J passengers per hour

$$N = J/G = \frac{J(bB + C)}{(3600)(B)} \qquad (5)$$

The basic variables used in the various analyses are defined in Table 9. Table 10 presents basic analytical relationships for a single station and contains a set of illustrative calculations, assuming that loading conditions govern.

Table 9. Basic Bus System Algebraic Variables

Symbol	Description
A	Alighting Passengers per Bus in peak 10-15 minutes
a	Alighting Service Time per Passenger, in seconds
B	Boarding Passengers per Bus in peak 10-15 minutes
b	Boarding Service Time per Passenger, in seconds
C	Clearance time between Successive Buses, in seconds (time between closing of doors on first bus and opening of doors on second bus)
D	Bus Dwell Time at Bus Stop, in seconds (time when doors are open and bus is stopped)
f	Bus Frequency, in buses per hour (all routes using the facility), at Maximum Load Point (if all buses stop at all stations, $f = N(f')$
f'	Maximum Peak Bus Frequency at a berth, in buses per berth per hour
G	Boarding Passenger Capacity per berth per hour
H	Alighting Passenger Capacity per berth per hour
h	Bus Headway on the facility, in seconds, at Maximum Load Point ($h = 3600/f$)
h'	Minimum Bus Headway at a berth, in seconds ($h' = 3600/f'$)
J	Passengers Boarding at Heaviest Stop, per hour
K	Passengers Alighting at Heaviest Stop, per hour
L	Peak Hour Load Factor, in passengers per bus seat per hour, at Maximum Load Point
N	Number of Effective Berths at a Bus Station or Stop ($N = N' \times u$)
N'	Number of Berth Spaces Provided in a Multi-berth Station
P	Line Haul Capacity of a bus facility, in persons per hour, past the Maximum Load Point (hourly flow rate based on maximum 10-15 minutes)
PHF	Peak Hour Factor
S	Seating Capacity of Bus (varies with design)
u	Berth Utilization Factor (an efficiency factor applied to the total number of berths to estimate realistic capacity of multi-berth stations. $u = N/N'$)
X	Proportion of Maximum Load Point Passengers which Board at Heaviest Stop ($X = J/P$)
Y	Proportion of Maximum Load Point Passengers which Alight at Heaviest Stop ($Y = K/P$)

Source: Adapted from Ref. (11), p. 41.

Table 10. Bus System Capacity Equations, Illustrative Example (Boarding Conditions Govern)

Variables	Equation (Hourly Rates)	Example
		Let: $C = 15$ seconds $b = 3$ seconds/passenger $B = 10$ passengers/bus $J = 2400$ boarding pass./hour
Minimum Headway at Stop	$h' = Bb + C$	$h' = 10(3) + 15 = 45$ seconds
Maximum Buses per Berth per hour	$f' = 3600/h' = \frac{3600}{Bb + C}$	$f' = 3600/45 = 80$ buses/berth/hour
Maximum Passengers per Berth per hour	$G = f'B = \frac{3600}{Bb + C}$	$G = 80(10) = 800$ pass./berth/hour
Effective Berths Required to serve J passengers per hour	$N = J/G = \frac{J(Bb + C)}{3600B}$	$N = 2400/800 = 3$ berths
Bus Frequency Required to serve J passengers per hour	$f = f'N = J/B$	$f = 3(80) = 240$ buses
		Let: $P = 6000$ passengers $b = 3$ seconds/passenger $S = 50$ passengers/bus $C = 15$ seconds $X = 0.50$ (50%)
Bus Frequency at Maximum Load Point	$f = P/S$	$f = 6000/50 = 120$ buses/hour
Passengers per Bus at Heaviest Stop	$B = X(S)$	$B = 50(0.50) = 25$ passengers/bus
Minimum Headway at Heaviest Stop	$h' = Bb + C = bX(S) + C$	$h' = 25(3) + 15 = 90$ seconds
Buses per Hour at Heaviest Stop	$f' = 3600/h' = \frac{3600}{bX(S) + C}$	$f' = 3600/90 = 40$ buses/berth/hour
Number of Effective Berths at Heaviest Stop	$N = \frac{f}{f'} = \frac{P}{S}(\frac{bXS + C}{3600}) = \frac{P(bXS + C)}{3600(S)}$	$N = 120/40 = 3$ berths

Source: Adapted from Ref. (11), p. 41.

System Capacity. The capacity of any busway or terminal and approach system will be governed by the number of passengers (a) boarding and/or alighting at the heaviest stop or (b) traveling past the maximum point (between stops), whichever is less. These condtions apply when the approach volume of buses and passengers is specified and it is desired to estimate the required number of berth positions. The sequence of analyses is as follows:

1. The maximum load point demand establishes bus frequency requirements in the corridor.
2. Bus service frequency and boarding volumes determine minimum headway per berth. (For planned systems, where no boarding counts are available, the percentage of passengers boarding at the heaviest stop is a key parameter of total passenger capacity.)
3. The maximum bus frequency per berth depends on this minimum headway.
4. Berth needs are derived from the required bus frequency at the maximum load point and the maximum bus frequency which can load at the heaviest berth.

The following equations show how maximum load point and heaviest stations parameters interrelate. These relationships assume that loading conditions govern; a similar set of equations could be derived where passenger alighting (or passenger interchange) determine capacities.

$$P = \frac{(3600)(N)(S)}{bB + C} \qquad (6)$$

Or, since boarding passengers per bus depends on bus frequency, f

$$P = \frac{3600(N)}{Xb + (C/S)} \qquad (7)$$

This relationship can also be expressed in terms of the passenger capacity per berth as follows:

$$P = \frac{NG}{X} = fS \qquad (8)$$

These equations indicate that the number of bus berths required at the heaviest stop or bus terminal varies directly with the total passengers to be served at that point, the boarding and alighting service times required per passenger, and the clearance times between buses. Table 10 contains an illustrative set of calculations.

The following example also shows how these formulas can be applied. It is desired to find the total passengers that can be carried past the maximum load point in an hour, based on the peak 15 minute flow rate. Operating policy calls for a 20 second clearance between buses (C = 20), and 50 passenger buses, and a load factor of 1.00 (S = 50). There is prepayment of fares, and the ability to load buses at 2.0 seconds per passenger (b = 2.0). System design anticipates that 50 percent of the total passengers will board at the maximum load point (X = 0.5). Four effective berths are provided. Substituting in formula (7):

$$P = \frac{(3600)(4)}{(0.5)(2)+(20/50)} \cong 10{,}286 \cong 10{,}000 \text{ persons/hr.}$$

The actual hourly volume would be less since the capacity represents four times the peak 15 minute flow rate. To calculate the hourly volume, a peak hour factor (PHF) is used. PHF is defined as the hourly volume divided by four times the highest 15 minute volume occurring within the hour. The hourly volume can be calculated by:

Hourly Volume (HV)

HV = (Peak 15 min. volume)(4)(PHF)

In this example if the peak hour factor is 0.75, the hourly volume would be 7,000 persons.

Bus system capacities can be increased (alternatively, berthing requirements can be reduced) by: (1) increasing the number of downtown (or "terminal") stations on a busway or busline thereby reducing the number of boarding and alighting passengers at the heaviest stop; (2) reducing the loading and unloading times for passengers through multiple doors on buses, pre-payment, and/or selective separation of loading-unloading; and (3) using larger buses to reduce the clearance interval time losses between successive vehicles. In summary, the person capacity of a bus lane appears to depend heavily on the number of doors per bus and the methods of fare collection.

Table 11. Suggested Use Factors for Multiple Berth Operations, Linear Stations

Berth Number	On-Line Stations[a]			Off-Line Stations[a]		
	Efficiency	Capacity Factor (Cumulative)	Factor[b]	Efficiency	Capacity Factor (Cumulative)	Factor[b]
1	100%	1.00	1.000	100%	1.00	1.000
2	75%	1.75	0.875	85%	1.85	0.925
3	50%	2.25	0.750	75%	2.60	0.867
4	25%	2.50	0.625	65%	3.25	0.812
5	Negligible	2.50	0.500	50%	3.75	0.750

[a] Assumes that buses do not overtake each other.

[b] Cumulative capacity.

Sources: Ref. (12) and Ref. (23).

Note: In Ref. (23), efficiency values were (1) 100%, (2) 73%, (3) 41%, (4) 27%, and (5) 18%. The resulting capacity factors (cumulative) were 1.00, 1.73, 2.14, 2.41, and 2.54.

Berth Use Efficiency

Actual bus route schedules may not permit an even distribution of scheduled buses among berths or an even distribution of passengers among loading positions. The actual efficiency of a system of loading positions will also vary with the type of design, and should be considered in developing capacitiesfor any given loading area. The berth efficiency factors shown in Table 11 are based on the experience at the Port of New York Authority. Further research is necessary to develop typical use factors, because experience with high-volume exclusive bus facilities is limited.

Passenger Service Times and Bus Headways

The minimum headway of buses at a stop consists of (1) actual station dwell time when the bus doors are open for boarding and alighting, plus (2) clearance times between buses. The time lost in opening and closing doors may be added to the dwell times, or incorporated in the clearance intervals.

Field observations of bus clearance times are limited. A British study (13) reported "dead time" (standing at a stop with the doors closed) of two to five seconds. Scheel and Foote (6) indicate that bus start-up times should also range from two to five seconds. The time for a bus to travel its own length after starting ranges from 5 to 10 seconds, depending on acceleration and traffic conditions. Accordingly, a reasonable estimate of clearance time per bus is 10 to 20 seconds, including door opening and closing times.

Station dwell times may be governed by boarding demand (e.g., in the PM peak when substantially empty buses arrive at a heavily used stop), alighting demand (e.g., in the AM peak at the same location), or total interchanging passenger demand (e.g., at a major transfer point on the system). In all cases, dwell time is proportional to boarding and/or alighting volumes times passenger service time.

Kraft (27) in his research on passenger service times found that the physical characteristics of vehicles such as the dimensions and number of stairs, aisle width, seating configuration and gate configuration affect service times. His investigations indicated the following:

1. There is no difference between front door and rear door alighting times.
2. Using both doors to alight requires more than one-half the time than it does to alight from one door. Time reductions of 27 to 80 percent were observed.
3. For alighting passengers, double stream doors have been observed to require 27 to 46 percent less time than single stream doors.
4. Rear door boarding times for double stream doors were observed to be 0.4 second per passenger faster than for double stream front doors, a reduction of 30 percent.
5. The use of boarding through both doors required less time than for one door, but the time requirements for two doors was more than half that required for one door.

It was difficult to determine the effects of aisle width and seating configuration because of their interaction with other elements. However, it was concluded that decreased aisle width increases passenger service time and that reducing the double seats on each side of the vehicle to a single seat on one side of the vehicle may result in reduced passenger service time.

Analysis of boarding and alighting time indicated the following:

1. Boarding service time requirements exceed those for alighting.
2. Alighting times are greater when boarding passengers are present.
3. Fewer delays to alighting and boarding passengers occurred when boarding queues were organized and orderly.
4. The presence of standees increases passenger service time. Observations of bus operations on Bloomfield Avenue in Newark, New Jersey, indicated an increase of 20 percent in boarding and alighting times when standees were present. It was observed that standees did not always interfere with the boarding and alighting of passengers.

Observed ranges in passenger service times for various bus operating and fare collection procedures are summarized in Tables 12, 12a, and 13 for both American and European experience. The appendix to this section also contains passenger service times from a recent U.S. study (27).

Boarding service times are usually greater than alighting times. However, some stop time equations in Table 12 relate to total passenger interchange. Differences among cities reflect door widths and configurations, fare collection practices, and one versus two-man operation. Some formulas also reflect the time losses resulting from opening and

Table 12. Bus Boarding and Alighting Times in Selected Urban Areas

Location	Bus Type	Boarding and Alighting Method	Fare Scheme	Fare Collection	Boarding and Alighting Relationship[a]
Louisville, Ky.	One-man	Alighting only	Flat fare	Driver	$T = 1.8 + 1.1F$
	One-man	Boarding only	Flat fare	Driver	$T = -0.1 + 2.6N$
	One-man	Simultaneous	Flat fare	Driver	$T = 1.8 + 1.0F + 2.3N - 0.02FN$
London	Two-man	Consecutive	Graduated	Conductor	$T = 1.3 + 1.5\ (N+F)$
	One-man	Consecutive	Graduated	Driver	$T = 8 + 6.9N + 1.4F$
	One-man	Simultaneous	Flat fare		
			Single coin	Mechanical	$T = 7 + 2.0N$
			Two coin	Mechanical	$T = 5.7 + 3.3N$[b]
Toronto	One-man	Simultaneous	Zonal	Fare Box	$T = 1.7N$, $T = 1.25F$, $T = 1.4(N+F)$
Copenhagen	One-man	Simultaneous	Flat fare	Split entry[c]	$T = 2.2N$
Dublin	Two-man	Consecutive	Graduated	Conductor	$T = 1.4(N+F)$
	One-man	Consecutive	Graduated	Driver	$T = 6.5N + 3.0F$
France:					
Bordeaux	One-man	Simultaneous	Flat fare	Driver	$T = 15 + 3N$
Toulouse	One-man	Simultaneous	Flat fare	Driver	$T = 11 + 4.6N$
Paris	One-man	Simultaneous	Graduated	Driver	$T = 4 + 5N$
	Two-man	Simultaneous	Graduated	Conductor	$T = 2.3N$

[a] T = stop time, in sec; N = number of passengers boarding; F = number of passengers alighting.

[b] In peak time; $T = 5.7 + 5.0N$ in off-peak time.

[c] Driver and machine.

Sources: Refs. (14) and (15).

Table 12A. Means and Variances of Observed Passenger Service Time Distributions

Location	Direction of Flow	Bus Type	Doors on Bus	Time in Seconds		Coefficient of Variation (%)
					Variance	
Montreal, Canada	Boarding	Can. Car	2	2.097	0.727	40.67
Montreal, Canada	Boarding	GMC	2	2.034	0.834	44.89
New Brunswick, NJ	Alighting	GMC	1	1.972	1.045	51.83
New Brunswick, NJ	Boarding	GMC	1	3.471	3.499	53.90
San Diego, CA	Alighting	GMC	2	1.472	0.403	43.34
San Diego, CA	Boarding	GMC	2	2.180	0.868	42.75

Source: Ref. (27)

Table 13. Reported Passenger Service Times On and Off Buses

	Conditions	Time (Seconds)
Unloading	Very little hand baggage and parcels; few transfers	1.5 - 2.5
	Moderate amount hand baggage or many transfers	2.5 - 4.0
	Considerable baggage from racks, (intercity runs)	4.0 - 6.0
Loading[a]	Single coin or token fare box	2.0 - 3.0
	Odd penny cash fares, multiple zone fares	3.0 - 4.0
	Pre-purchased tickets and registration on bus	4.0 - 6.0
	Multiple zone fares; cash; including registration on bus	6.0 - 8.0
	Prepayment before entering bus or pay when leaving bus	1.5 - 2.5

Sources: Adapted from Ref. (18), pp. 338-348, and Ref. (30), p. 7.

[a] Add one second where fare receipts are involved.

closing doors. However, this loss can be incorporated into clearance time between buses.

Examples of means, variances, and coefficients of variation of passenger service times are shown in Table 12A. Coefficients of variation generally range from 40 to 50 percent of the mean passenger service time.

American experience with single-door buses shows passenger boarding times ranging from 2 seconds (single-coin) to over 8 seconds for multiple-zone fares collected by driver (Table 13). Alighting times range from about 1-1/2 to 2-1/2 seconds for typical urban conditions to 6 seconds or more where baggage is involved.

Suggested ranges in bus service times in relation to door width, methods of operation, and fare collection practices are presented in Table 14. These bus service times, based on current

Table 14. Typical Bus Passenger Boarding and Alighting Service Times for Selected Bus Types and Door Configurations

Bus Type	Available Doors or Channels		Typical Boarding Service Times[a]		Typical Alighting Service Times
	Number	Location	Prepayment[b]	Single Coin Fare[c]	
Conventional	1	Front (F)	2.0 sec.	2.6-3.0 sec.	1.7 sec.
	1	Rear (R)	2.0 sec.	n.a.[c]	1.7 sec.
	2	Front (F)	1.2 sec.	1.8 sec.	1.0-1.2 sec.
	2	Rear (R)	1.2 sec.	n.a.[c]	1.0-1.2 sec.
	2	F & R[d]	1.2 sec.	n.a.[c]	0.9 sec.
	4	F & R[f]	0.7 sec.	n.a.[c]	0.6 sec.
Articulated	3	Front, Rear & Center	0.9 sec.[f]	n.a.[c]	0.8 sec.
	2	Rear	1.2 sec.[g]	n.a.[g]	---
	2	Front & Center[d]	---	---	0.6 sec.[g]
	6	Front, Rear & Center[e]	0.5 sec.	n.a.[c]	0.4 sec.
Special Single Unit	6	3 Double Doors[h]	0.5 sec.	n.a.[c]	0.4 sec.

[a] Typical interval in seconds between successive boarding and alighting passengers. Does not allow for clearance times between successive buses or dead time at stop.

[b] Also applies to pay-on-leave or free transfer situations.

[c] Not applicable with rear-door boarding.

[d] One each.

[e] Two double doors each position.

[f] Less use of separated doors for simultaneous loading and unloading.

[g] Double door rear loading with single exits, typical European design. Provides one-way flow within vehicle, reducing internal congestion. Desirable for line-haul, especially if 2-person operation is feasible. May not be best configuration for busway operation.

[h] Examples: Neoplan TR-40 Mobile Lounge designed by Trepal Systems, Inc., for airport apron use.

Sources: Ref. (18), Ref. (13), and Ref. (31).

experience, provide a basis for further estimating bus and person capacity. They assume that pre-payment before entering buses would reduce passenger service time, a reasonable assumption for downtown busways and bus terminals.

Passenger service times decrease as the number of door channels available to passengers increases. The time values in Table 14 reflect inefficiencies in utilizing additional doorway capacity. For example, one passenger may occupy a double door; moreover, passengers do not distribute themselves uniformly among doorway openings. The values do not, however, reflect doorway and aisle turbulence at points of heavy simultaneous boarding and alighting (See References 16 and 17 for more details).

Figure 1 shows how berth capacity can be increased by changing downtown fare collection practices on a standard versus an urban transit bus. The example shown in Figure 1 is based on the following assumptions: clearance interval, 15 sec./bus; service time with single-coin fare, 3 sec. (top curve); service time with double doors and prepaid fare, 1.2 sec. (bottom curve); and 15-minute peak passenger flow rates, stated in hourly terms. The figure also indicates how increasing the number of passengers boarding per bus tends to decrease the frequency of buses that can load at a berth. If the boarding passenger volumes are distributed over several stops so that peak boarding averages 10 passengers/bus at the heaviest stop, from 80 to 140 buses could be scheduled, depending on fare structure, door availability, and the number of alighting passengers. At outlying stops where boarding or alighting averages less than 5 passengers/bus, 120 buses/berth/hour can be scheduled when single-coin fare and single-door entry are used. Conversely, where the entire bus fills up at a given stop, only 20 to 48 buses/hour could be served.

CBD Busway Guidelines

CBD busway capacity can be computed from the preceding formulas, utilizing appropriate assumptions regarding type of bus used, maximum allowable bus loading, distribution of ridership among CBD stops and ratio of the peak 15 minute demands of the entire hour, and type of status (on or off-line) which would affect berth efficiency.

Bus Use. The number of people per bus will depend upon (1) size of vehicles (about 50 seats/regular bus to 60 seats/articulated bus), and (2) operating policies with regard to standees. To provide an acceptable level of comfort for express bus commuters with a minimum non-stop run of three to five miles, the passenger load factor in the peak 15 minute period should not exceed 1.00--i.e., there should be a seat available for each passenger (Higher load factors are acceptable on shorter bus routes). When total hourly flows are considered, a lower load factor should be assumed. Depending on land use and employer hours, conservative load factors also will minimize on-vehicle turbulence at bus stops.

Passenger Distribution at CBD Stops. A reasonable design assumption is that 50 percent of the maximum load point volume is served at the heaviest CBD busway stop--assuming a minimum of three stops in the downtown. The Washington-State Subway Station in Chicago accounts for about half of all boarding passengers at the three downtown stops on the State Street line.

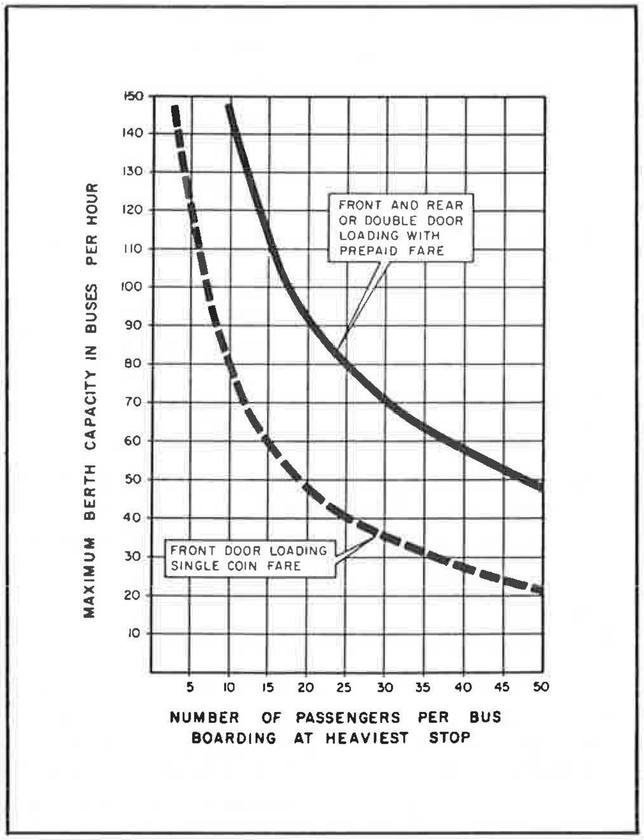
Peak Hour Factor. Typical relationships between the peak 15 minutes and peak 60 minutes for transit lines range from 0.60 to 0.85. Quinby cites a range of 0.70 to 0.95, with 0.83 as a guide, in the Transportation and Traffic Engineering Handbook (28). For example, the Los Angeles SCRTD reports peak hour factors for Route 83 of 0.66 for commuters and 0.74 for locals. For busways, an average peak hour factor of 0.67 is reasonable.

Illustrative busway capacity guidelines for central urban areas are shown in Figure 2 and given in Table 14 for a variety of bus types and service conditions. Figure 2 shows how door configuration and number of berths increases maximum load point capacity. The lower horizontal scale applies to typical through station operations while the upper scale applies to a single-station situation.

Table 15 gives assumptions used in deriving capacities. The computations assume that:

1. passengers per bus at maximum load point is 50 for conventional buses and 60 for articulated buses,
2. fity percent of the maximum load-point passengers board at the heaviest CBD stop,
3. there are three loading berths for both on-line and off-line boarding (for alternate station sizes, see Figure 2) and loading and unloading areas are separated,
4. an adjustment factor of 0.75 is used to allow for on-vehicle turbulence and schedule irregularity,
5. a peak hour load factor of 0.67 is used to convert from peak 15 minute flow rates to overall average hourly volumes, and
6. fares are prepaid (no fares collected on bus in CBD).

Figure 1. Illustrative Example of Bus Berth Capacity in Relation to Passenger Boarding Volumes



Source: Ref. (5), p.37

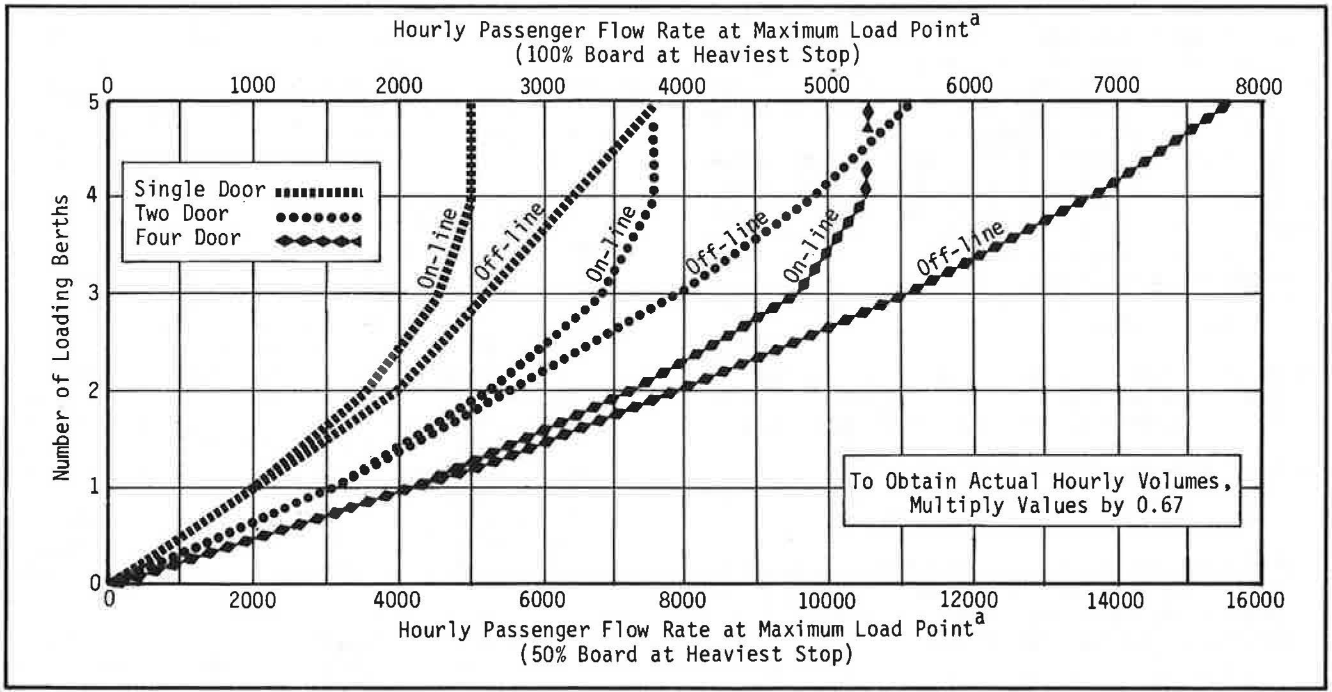
Table 15. Illustrative Bus Capacity Guidelines for CBD Busways

Loading Condition A: Single door conventional bus, simultaneous loading and unloading
Loading Condition B: Two door conventional bus, both doors loading or double-stream doors simultaneously loading and unloading
Loading Condition C: Four door condventional bus, all double-stream doors loading
Loading Condition D: Six door articulated bus, all doors loading

	Loading Condition							
	A		B		C		D	
Station→	On-line	Off-line	On-line	Off-line	On-line	Off-line	On-line	Off-line
Passengers Boarding at Heaviest Stop								
Number of Passengers	25	25	25	25	25	25	30	30
Boarding Time per Passenger, in seconds	2.0	2.0	1.2	1.2	0.7	0.7	0.5	0.5
Total Boarding Time, in seconds[a]	65	65	45	45	32½	32½	30	30
Berth Use, in buses per hour								
Maximum buses per hour per berth	55	55	80	80	111	111	120	120
Use Factor, for three berths	2.25	2.60	2.25	2.60	2.25	2.60	2.25	2.60
Total for All Berths	124	143	180	208	250	289	270	312
Adjusted Total for All Berths[b]	93	107	135	156	188	217	200	234
Passengers per hour at Heaviest Stop								
Peak[c]	4650	5350	6750	7800	9400	10850	12000	14040
Average[d]	3115	3570	4520	5200	6300	7320	8040	9360

[a] Includes fifteen second bus clearance interval
[b] Adjusted by a factor of 0.75 to account for turbulence, schedule irregularities, and the like.
[c] From Figure 2
[d] Adjusted by a factor of 0.67 from peak volume
Source: Adapted from Ref. (5), p. 39 (Table 8).

Figure 2. Typical CBD Busway Line Haul Service Volumes (Flow Rates)



Source: Adapted from Ref. (5), p. 39

[a] Based on Peak 15 minute Volumes

Resulting average hourly bus volumes at maximum load points are as shown in Table 16.

Table 16. Busway Volumes at Maximum Load Points

Type of Operation	On-Line Stations	Off-Line Stations
Conventional Bus[a]	3,100	3,550
Two Doors Available[b]	4,500	5,200
Four Doors Available[c]	6,300	7,250
Articulated 60-Passenger Bus[d]	8,050	9,350

[a] Single door for loading

[b] Double-door entrance or front and rear single doors with separate or negligible alighting

[c] Wide double-doors front and rear with separate or negligible alighting

[d] Six door channels and separate or negligible alighting

Note: Peak 15-minute flow rates would be 50 percent higher, assuming a typical load factor of 0.67.

Source: Herbert S. Levinson

Bus Terminals

Berth space requirements at major bus terminals can be computed by the preceding formulas. Computations should reflect both scheduled and actual peak period bus arrivals and departures, since intercity bus services regularly run "extras" during peak travel periods. They should also recognize the specific type of carrier operations, fare collection practices, bus door configurations, passenger arrival patterns, amount of baggage, driver layover-recovery times, terminal design, and berth configuration.

Bus service times may be increased to allow buses to meet scheduled departure times. In these cases, it may be necessary to add 5 minutes or more to computed clearance and dwell times. Current experience shows about two buses per berth per hour for intercity operations and about 10 to 12 buses per berth for commuter operation.

Illustrative berth loading capacities are shown in Table 17. Note that bus unloading capacities will approximate loading conditions. These capacities assume that each berth would be fully effective and that passenger loading times would determine bus dwell times.

Typical berth operations and resulting levels of service are as follows:

- Prepaid fare collection with use of double entrance doors can accomodate 2,400 to 2,900 passengers per hour per berth (with queueing);

- Free, prepaid, or pay-on-exit operations, with single doors, can accomodate 1,600 to 1,900 passengers per hour per berth with queueing;

- With multi-zone fares--where tickets are sold or validated by the bus driver--capacities capacities are reduced to as low as 250-500 passengers per hour per berth.

In practice, the capacities would have to be reduced to reflect actual operating conditions. These capacities should also be reduced to allow for schedule recovery and driver relief time. A maximum of two services per loading berth should be operated. Therefore, service patterns will also influence berth requirements.

Bus Operations on Urban Arterials

Bus operations along arterial streets and urban expressways are influenced by: (1) the number of other vehicles with which they must share roadway space, (2) marginal interference, (3) intersection delays, and (4) time lost in passenger boarding or alighting.

Bus Stop Spacing. Bus stop spacing is largely a matter of operating policy. Successive stops should be closest in the central business district and farthest in outlying suburban areas. Current practice suggest the following ranges:

- Central Business District - 400 to 600 feet (122 to 183m)
- "Urban" Areas - 600 to 750 feet (183 to 229m)
- "Suburban" Areas - 1200 to 1320 feet (366 to 402m)

Bus Stop Location. The location of curb stops; near side of intersection, far side, or midblock, may have a significant effect on the transit operation itself as well as on overall street capacity. Bus stop locations should be standardized within each community to the extent that service requirements and traffic conditions permit. However, locations usually involve tradeoffs between locational consistency and conflict minimization (11). Thus, where conficts would otherwise seriously impeded bus and/or vehicle flow, stops should be either relocated to adjacent intersections or eliminated.

Bus Stop Locational Guidelines. It is difficult to establish a fixed locational policy. An efficient transit operation which is in harmony with overall traffic flow requires detailed analysis of each route and stop. Choice of stop will depend upon availability of curb loading space; location of existing stops, convenience of passenger transfer, and proximity to passenger destinations. Other significant factors include bus routing patterns; directions of intersecting streets; the types of traffic flow controls (signal, stop, yield), traffic volume and turning movements; and widths of sidewalks or roads.

> Far side stops are preferable, where sight distance or signal capacity problems exist, where buses have use of curb lanes during peak travel periods, and where right or left turns by general traffic are heavy.
>
> Near side stops are preferable where transit flows are heavy, but traffic and parking conditions are not critical. From the transit operator's point of view, they make it easier to rejoin the traffic stream, particularly where curb parking is permitted in peak periods. They also allow the first bus to stop at the intersection.
>
> Midblock stops are generally applicable in downtown areas where multiple routes require long loading areas, and where stops might extend an entire block.

Bus Stop Capacity Guidelines. The length of bus stops should reflect (1) the number of buses that each stop will accomodate simultaneously in each peak 15 to 30 minute period, (2) maneuvering

Table 17. Illustrative Local and Commuter Bus Terminal Berth Loading Capacities, in Relation to Fare Collection Procedures

	Type of Fare Collection Procedure				
	Free, Prepaid, or Pay on Exit		Pay on Entry, Farebox with Single-Doorway Entrance Channel		
	Double Door	Single Door	Single Coin or Token	Odd-Penny	Multi-Zone
Passenger Headway	0.8-1.2 secs.	1.0-2.0 secs.	2.0-3.0 secs.	3.0-4.0 secs.	4.0-6.0 secs.[a] 6.0-8.0 secs.[b]
Doors Used	2[c]	2[c]	1	1	1 1
Dwell Time to Load					
50 Passengers[d] (33 through heaviest door)	34-40 secs.	30-65 secs.	100-105 secs.	150-200 secs.	200-300 secs.[a] 300-400 secs.[b]
Minimum Bus Headway					
Queued Buses[e]	50 secs.	75 secs.	160 secs.	210 secs.	310 secs.[a] 410 secs.[b]
Single Buses[f,g]	100 secs.	125 secs.	210 secs.	260 secs.	360 secs.[a] 460 secs.[b]
Equivalent Berth Capacity					
Queued Buses	72 buses/hr.	48 buses/hr.	23 buses/hr.	17 buses/hr.	17 buses/hr.[a] 8 buses/hr.[b]
Single Buses	36 buses/hr.	29 buses/hr.	17 buses/hr.	14 buses/hr.	10 buses/hr.[a] 7 buses/hr.[b]
Equivalent Passenger Load					
Queued Buses	3,600	2,400	1,150	850	600[a] 400[b]
Single Buses	1,800	1,450	850	700	500[a] 350[b]
Effective Berth Capacity					
40% in Peak 20 Minimum					
Queued Buses	2,900	1,900	900	680	480[a] 320[b]
Single Buses	1,400	1,200	580	560	400[a] 280[b]
50% in Peak 20 Minimum					
Queued Buses	2,400	1,600	770	570	400[a] 270[b]
Single Buses	1,200	1,000	570	470	330[a] 230[b]

[a] Prepurchased tickets, registered on bus by driver.

[b] Cash fare, driver makes change, and farebox prints receipts for passenger to show on exit.

[c] Assumes 67-33 split between front and rear doors.

[d] Assumes 50-seat buses loaded to seating capacity for express runs. Standees can be accommodated on relatively short express runs, but seating capacity is considered more realistic in view of the need to compete with private auto comfort.

[e] Assumes that the next bus is always waiting behind the loading bus and can pull in and be ready to load within 10 seconds (i.e., linear platform).

[f] Assumes that the next bus has to be summoned from a holding or storage area, involving a 60-second delay and/or recovery time (linear or shallow sawtooth platform).

[g] With lower times, capacities would approach those for queued operations.

Source: Adapted from Ref. (11), p. 46.

Table 18. Bus Stop and Bus Bay Capacity Requirements

Peak Hour Bus Flow	Headway Per Min.	Capacity Required (Bays) When Service Time at Stop is 10 Sec.	20 Sec.	30 Sec.	40 Sec.	50 Sec.
15	4	1	1	1	1	1
30	2	1	1	1	1	2
45		1	1	2	2	2
60	1	1	1	2	2	3
75		1	2	2	3	3
90		1	2	2	3	4
105		1	2	3	3	4
120	1/2	1	2	3	3	5
150		2	3	3	4	5
180	1/3	2	3	4	5	6

Source: Ref. (11), p. 130.

requirements of buses to enter and leave the stop, and (3) the type of stop.

The number of buses that can be handled at curbside bus stops without unacceptably long queues (and associated waiting lines) being caused varies principally with the service time per bus and, to a lesser degree, with the number of loading positions. Additional loading spaces (or additional length of bus zones) increase the capacity, but at a decreasing rate as the number of spaces increases.

The number of bus loading positions at any given stop, in turn, depends upon (1) the rate and nature of bus arrival, (2) passenger service times, and (3) allowable amount of queueing. Bus stop and bus capacity requirements based upon a Poisson (random) arrival rate and a 95 percent confidence interval are summarized in Table 18. This table gives the number of bus berths that should be provided, allowing only a 5 percent chance that the bus bays will overload. Thus it is a reasonable approximation of Level of Service **C.** Emergent criteria for bus stop capacity are as follows:

- Passenger sevice times of 20 seconds or less - one bus berth per 60 peak hour buses (this is the typical radial arterial street condition).
- Passenger service times of 30 to 40 seconds - one bus berth per 30 peak hour buses.
- Passenger service times over 40 seconds - one bus berth per 20 peak hour buses.

These results are generally consistent with the guidelines set forth in the 1965 Highway Capacity Manual (18) which found that "a bus stop can serve buses arriving at half the average service rate, or trip frequency, with well under 10 percent probability of forming queues beyond the stop." According to the manual,

> An acceptable rule of thumb might be to assume that the headways at a curbside bus stop (in minimum seconds of interval between vehicles) could be about twice the average service time per vehicle. Along any arterial, the stop with the longest service time will be the bottleneck. The capacity of the artery itself could be increased by providing different bus stops for different routes, provided vehicles could overtake each other.
>
> To illustrate: Assume that along "Main Street" the average service time at the busiest bus stop is 25 seconds, including clearance. Provided the length of the bus stop is adequate, this stop will handle buses at a minimum headway of about 50 seconds. Headways can be approximately halved (frequency of service doubled) by providing alternate sets of bus stops far enough removed from each other so as not to cause interference in entering and leaving the loading zones. Each set of stops can then handle buses at 50 second headways, and the

Table 19. Minimum Desirable Lengths for Bus Curb Loading Zones

		Loading Zone Length[a] (ft.)					
		One-Bus Stop			Two-Bus Stop		
Approximate Bus Seating Capacity	Approximate Bus Length (ft.)	Near Side[b]	Far Side[c]	Mid-Block	Near Side[b]	Far Side[c]	Mid-Block
30 and less	25	90	65	125	120	90	150
35	30	95	70	130	130	100	160
40-45	35	100	75	135	140	110	170
51-53	40	105	80	140	150	120	180
51-53[d]	40	90-105	80-100	130-145	135-150	125-145	175-210

[a] Measured from extension of building line, or from an established stop line, whichever is appropriate. Based on side of bus positioned 1 ft. from curve; if bus is as close as 6 in. from curb, 20 ft. should be added to near-side stops, 15 ft. to far-side stops, and 35 ft. to midblock stops.

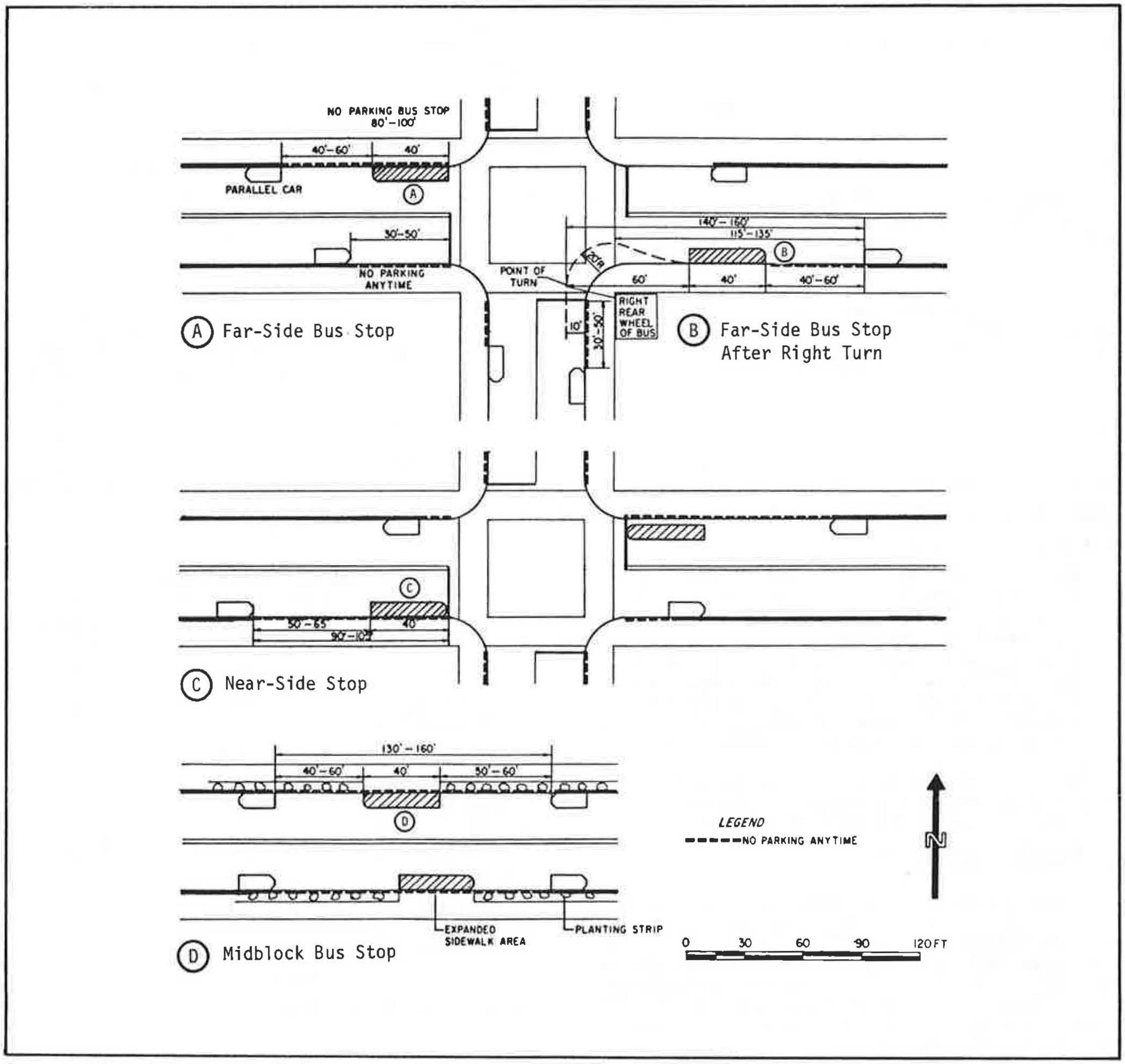
[b] Increase 15 ft. where buses are required to make a right turn. If there is a heavy right turn movement of other vehicles, near side stop zone lengths should be increased 30 ft.

[c] Based on roadways 40 ft. wide, which enable buses to leave the loading zone without passing over centerline of street. Increase 15 ft. if roadway is 16 ft. wide, and 30 ft. if roadway is 32 ft. wide.

[d] From Ref. (11).

Sources: American Transit Association, Ref. (18), and Ref. (11). (1 foot = .305 meter)

Figure 3. Illustrative Bus Stop Design Standards



Source: Adapted from Ref. (11), p.131

street as a whole can handle buses at 25 second headways, if exactly 50 percent of the buses are assigned to each set of stops, and if schedule reliability can be maintained. Ample smoothly-operating stops help assure reliability. However, it should be realized that in the case of the usual all bus-lane operation, buses would be restricted to this lane, hence, overtaking would be impossible and multiple stops would not be feasible (18).

Bus Stops on Freeways and Expressways. Bus loading zones on an exclusive roadway within a freeway right-of-way have capacities similar to those of curbside loading zones. Here again, the length of the stop and the ability of buses to overtake others are important. Given similar loading facilities, any difference does not lie in the operation of the stop itself, but in the capacity of the roadway lane leading into and away from the stop.

Bus Stop Design Guidelines. Figure 3 and Table 19 gives minimum desirable bus stop lengths for curb side loading zones. These guidelines are based on a 40 foot bus; stop lengths should be adjusted for a longer or shorter length and 45 feet should be added for each additional bus.

Arterial Street Capacity Guidelines. Arterial street bus capacity can be estimated based on the general approaches identified in Tables 9 and 10, assuming (1) prepayment or on-vehicle fare collection or pay as you leave and (2) a greater number of bus loading points than for busways. Key factors include: (1) average service time at busiest stops, (2) desired space between buses, (3) number of stopping positions, and (4) allowable queueing at stops.

Queue Behavior Parameters - Typical bus queue behavior along downtown arterial streets is

Table 20. Comparative Bus Flow and Queue Statistics

(a) Actual Flow

Location	Volume in Buses/ Hour	5-Min Flow Rate in Buses/ Hour	Headway in Secs. Aver.	Headway in Secs. Range	Max. Bus Queue At Stop
S. Michigan Ave., Chicago	174	228	21	16- 33	4
N. Michigan Ave., Chicago	112	148	32	3-150	5
Washington St., Chicago	106	132	34	3-121	4

(b) Queue Behavior at Stop Locations[a]

Queue Length (No. of Buses)	No. of Occurrences	Queue Delay in Secs. Range	Mean	Median
1	50-70	--	--	--
2	8	6-49	14	8
3	6	9-61	29	24
4	3	47-59	51	48

[a]Data represent bus volumes of 100 to 120 vehicles per hour

Source: Ref. (11), p. 42.

given in Table 20. These statistics suggest that when bus volumes exceed 100 per hour, queues of 2 to 4 buses are likely to develop approximately 20 percent of the time (19).

Service Volume Capacity Ranges - The preceding analysis, coupled with additional observations of existing service volumes suggest the representative service volumes in Table 21 for central areas and their radial approaches.

Where stops are relatively lightly patronized, such as along outlying arterials, it is reasonable to increase these volumes by about 20 percent.

Bus Priority Treatments

A growing number of cities have established or are considering exclusive bus lanes and other bus priority measures to improve person flow over city streets and highways. Bus priority measures are an essential part of transportation system management (TSM) programs which attempt to maximize transport system efficiency consistent with social, economic, and environmental objectives.

Because buses may stop within priority lanes to pick up and discharge passengers, the ability of these lanes to carry people will be affected by loading and unloading time requirements set forth earlier. Guidelines presented in the previous section can be used to estimate capacities. The following section summarizes the pertinent features, planning guidelines and potential benefits associated with various bus and high occupancy vehicle priority measures.

Table 22 gives an overview of key factors for priority treatments existing in 1979. Table 23 defines the various types of measures that can be implemented.

Operational Overview

Freeway Related Treatments. Examples include the San Bernardino, Shirley, and Pittsburgh Busways; peak hour bus pre-emption of one of two roadways on the Ottawa River Parkway; contra-flow bus lanes on the Long Island Expressway (New York City), I-495 (New Jersey), and U.S. 101 (Marin County); normal flow bus lanes on U.S. 101 (Marin County), Moanalua Freeway, Honolulu and I-95 (Miami); a special reversible bus ramp for Seattle's Blue Streak express bus service, and the bus and car pool bypass lanes at the San Francisco-Oakland Bay Bridge toll plaza; and bus and/or car pool priorities at some 50 metered freeway ramps in the Los Angeles area; and at nine locations along I-35W, Minneapolis.

Table 21. Bus Lane Service Volumes

Level of Service	Buses/Lane/Hour	Mid-Value
A - Free Flow	Under 25	15
B - Stable Flow, Unconstrained	25-45	35
C - Stable Flow, Interference	45-74	60
D - Stable Flow, Some Platooning	75-104	90
E - Unstable Flow, Queueing	105-134	120
F - Forced Flow, Poor Operation	135 and over	150[a]

[a]Results in more than one-lane operation.

Source: Herbert S. Levinson

Arterial Street Treatments. Operating installations include bus-only streets in Chicago, Madison, Minneapolis, Philadelphia, Portland, Washington, D.C. and Vancouver, B.C.; contra-flow bus lanes in Chicago, Harrisburg, Honolulu, Indianapolis, Los Angeles, San Antonio, San Juan and Seattle; median bus lanes in Chicago, Denver, and Miami; and curb bus lanes in most cities. A half-mile (0.8 km) "split-level" tunnel serves trolley and diesel buses in Harvard Square, Cambridge, Massachusetts; and buses use a half-mile (0.8 km) tunnel in Providence, Rhode Island.

Significant terminals. This type of priority treatment is typified by New York City's Midtown and George Washington Bridge terminals, San Francisco's Transbay Terminal, Chicago's 69th and 95th Street

Table 22. Summary: Bus Priority Treatments

	General Applicability To:			Design-Year Conditions		
Type of Treatment	Local Bus Service	Limited-Express Bus Service	Planning Period in Years	Range in One-Way Peak-Hour Bus Volumes	Range in One-Way Peak-Hour Bus Passenger Volumes	Related Lane-Use and Transportation Factors
FREEWAY RELATED						
Busways on special right-of-way	X	X	10-20	40-60	1,600-2,400	Urban population, 750,000; CBD employment, 50,000; 20 million sq. ft. floor space
Busways within freeway right-of-way		X	10-20	40-60	1,600-2,400	Freeways in corridor congested in peak hour
Busways on railroad right-of-way	X	X	5-10	40-60	1,600-2,400	Not well located in relation to service area. Stations required.
Freeway bus lanes, normal flow		X	5	60-90	2,400-3,600	Applicable upstream from lane-drop. Bus passenger time saving should exceed other road user delays.
Freeway bus lanes, contra-flow		X	5	40-60	1,600-2,400	Freeways 6 or more lanes; where imbalance in traffic volumes permits level of service D in off-peak travel directions.
Bus lane bypass at toll plaza		X	5	20-30	800-1,200	Adequate reservoir on approach to toll station.
Exclusive bus access ramp to nonreserved freeway or arterial lane	X	X	5	10-15	400-600	
Bus bypass lane at metered freeway ramp		X	5	10-15	400-600	Alternate surface street route available for metered traffic. Express buses leave freeways to make immediate stops.
Bus stops along freeways		X	5	5-10	50-100[a]	Generally provide at surface street level in conjunction with metered ramp.
ARTERIAL RELATED						
Bus streets	X	X	5-10	20-30	800-1,200	Commercially oriented frontage.
CBD curb bus lanes, main street	X		5	20-30	800-1,200	Commerically oriented frontage.
Curb bus lanes	X		5	30-40	1,200-1,600	At least 2 lanes available for other traffic in same direction.
Median bus lanes	X	X	5	60-90	2,400-3,600	At least 2 lanes available for other traffic in same direction; ability to separately vehicular turn conflicts from buses.
Contra-flow bus lanes, short segments	X		5	20-30	800-1,200	
Contra-flow bus lanes, extended	X	X	5	40-60	1,000-2,400	At least 2 lanes available for other traffic in opposite direction. Signal spacing greater than 500-ft. intervals.
Bus turnouts	X		5	10-15	400-600	Points of major passenger loadings on streets with more than 500 peak-hour autos using curb lane.
Bus preemption of traffic signals	X		1-5	10-15	400-600	Wherever not constrained by pedestrian clearance or signal network constraints.
Special bus signals and signal phases, bus-actuated	X		1-5	5-10	200-400	At access points to bus lanes busways, or terminals; or where special bus turning movements must be accommodated.
Special bus turn provisions	X		1-5	5-10	200-900	Wherever vehicular turn prohibitions are located along bus routes.

[a] Boarding or alighting passengers in peak hour.

Source: Ref. (11), p. 28.

Table 23. Functional Classification of Bus Priority Treatments on Urban Highways

	Type of Bus Service			
Element	Rapid Transit	Express	Local	Demand Actuated
FREEWAYS OR FREEWAY RELATED				
1a. Special busways				
Exclusive subway (underpass in street or private right-of-way)	X	X	X	
Exclusive elevated way	X	X	X	
Other exclusive bus right-of-way (at grade or depressed)	X	X	X	
Exclusive use of freeway median (reversible lanes, other)	X	X		
Exclusive busway in nonhighway rights-of-way (railroad; public utilities)	X	X		
1b. Reserved freeway lane (peak hours; all day)				
Inside lane reserved, buses flow with traffic	X	X		
Inside lane reserved, buses flow against traffic	X	X		
1c. Bus ramps				
Exclusive bus ramps to freeway to nonreserved lanes	X	X		
Bus bypass lanes at toll booths	X	X		
Special bus access to exclusive bus lanes	X	X		
Ramp metering with preferential bus treatment	X	X		
1d. Bus bays or turnouts on freeways		X		
URBAN ARTERIALS				
2a. Exclusive bus streets (peak hours; all day)				
CBD street or alley	X	X	X	X
Other arterial		X	X	X
2b. Exclusive bus lanes (peak hours; all day)				
Flow with traffic				
CBD, curb lanes	X	X	X	
CBD, center lanes		X	X	
Arterials, curb lanes		X	X	X
Arterials, center lanes		X	X	X
Flow against traffic (contra-flow lanes)				
CBD	X	X	X	
Arterials		X	X	X
2c. Exclusive bus bypass of congested locations				
Underpass		X	X	X
Overpass (exclusive bus lanes)		X	X	X
Other exclusive short rights-of-way		X	X	X
Special bus turning lanes		X	X	X
2d. Bus stops				
Bus bays or turnouts	X	X	X	X
Curb loading and unloading platforms	X	X	X	X
Median loading and unloading platforms	X	X	X	X
2e. Special traffic signalization				
Bus preemption of intersection (driver actuated)		X	X	
Bus presence detector in street		X	X	
Special bus signal phase (turn provision)		X	X	
2f. Special traffic control				
Turn lanes for buses only	X	X	X	
Permissive bus turns (other movements prohibited)		X	X	
BUS TERMINALS				
3a. Central area terminals	X	X		
3b. Outlying terminals, park-ride facilities				
Freeway related	X	X		
Arterial related		X	X	X
Rapid transit related	X	X	X	X
OTHER TRAFFIC OPERATIONAL MEASURES				
4a. Permissive bus turns (where movements by other vehicles are prohibited)		X	X	
4b. Parking prohibitions or restrictions to facilitate bus flow		X	X	
4c. STOP sign or YIELD sign protection for streets used by buses		X	X	

[a] Passenger facilities amenity and bus service operating procedures not identified.

Source: Ref. (4), p. 12.

bus bridges over the Dan Ryan Expressway, Toronto's bus-rail terminals, and Cleveland and Washington's extensive bus-rail-car interchange facilities. Park-and-ride lots are tied to express bus services in many communities including Washington, Miami, Los Angeles, Boston, New York, Hartford, Portland and Providence.

Car Pool Priorities. Programs of this type were first installed on the San Francisco-Oakland Bay Bridge in 1971, and on Boston's I-93 in 1973. Since then, reserved car pool lanes have been installed on freeways in Honolulu, Miami, Oakland, Portland, San Francisco, and Washington, and on arterial streets in Honolulu and Miami. Many metered ramps in the Los Angeles area provide car pool bypass lanes.

Appraisal. Most bus priority measures constitute reserved bus lanes on city streets - usually in the direction of traffic flow. However, the number of bus only streets—such as State Street in Chicago, Nicollet Mall in Minneapolis, and Chestnut Street in Philadelphia—is increasing.

Busways and reserved lanes on freeways are mainly found or are being proposed in larger American Cities—usually with a large downtown employment and heavy peak hour bus ridership. Within the last few years, there has been a tendency for medium-sized cities such as Miami and Portland to install normal flow bus and car pool lanes. The trend is to implement operationally oriented measures because of the time, costs, and complexity associated with major new construction.

Distribution of buses in central areas remains an important challenge, and communities are giving this item increased attention. Freeway-related treatments generally provide good access to the CBD perimeter, but do not substantially improve service within the downtown core. Terminals are not always located near major employment concentrations and may require secondary distribution, and curb bus lanes may not be effective. Consequently, there have been several attempts to install contra-flow bus lanes in downtown areas.

Many bus priority measures have produced important passenger benefits—especially those relating to freeways. They achieve peak hour travel time savings of 5 to 30 minutes—savings which compare favorably with those resulting from rail transit extensions.

Successful priority treatments are usually characterized by: (1) an intensively developed downtown area with limited street capacity and high all day parking costs, (2) a long-term reliance on public transport, (3) highway capacity limitations on approaches to downtown, (4) major water barriers which limit road access to the CBD and which channel bus flows, (5) fast non-stop bus runs for considerable distances, (6) bus priorities on approaches to or across water barriers, (7) special bus distribution within the CBD—often off-street terminals, and (8) active traffic management, maintenance, and operations programs.

Planning Criteria

Planning and implementing bus priority measures require: (1) a reasonable concentration of bus services, (2) a high degree of bus and car congestion, and (3) community willingness to support public transport. There is little value in providing bus priority measures where service is poor, costly, or non-existent; where there are neither buses nor congestion; and where the community has no desire to maintain and improve bus service.

Planning calls for a realistic assessment of demands, costs, benefits, and impacts. The objective is to apply measures which (1) alleviate existing bus service deficiencies, (2) achieve attractive and reliable bus service, (3) serve demonstrated existing demands, (4) provide reserve capacity for future growths in bus trips, (5) attract auto drivers, and (6) relate to long-range transit improvement and downtown development programs.

Key factors include: (1) the intensity and growth prospects of the city center; (2) the historic and potential future reliance on public transport; (3) street width, configuration, continuity, and congestion; (4) the suitability of existing streets (and expressways) for express bus service; (5) bus operating speeds and service reliability in the city center; (6) availability of alternate routes for displaced auto traffic; (7) locations of major employment centers in relation to bus routes; (8) goods and service vehicle loading requirements; (9) express and local bus routing patterns; (10) bus passenger loading requirements along curbs; and (11) community attitudes and resources.

Bus priority measures must fit real world street systems. They must be reasonable, not only in how they improve bus service, but how they impact other traffic as well. Community acceptance and support is essential--especially over the long run. Effective enforcement and maintenance are also necessary elements in priority treatments for buses and car pools.

Buses must be able to enter and leave priority lanes easily, and alternative routings must be available for potentially displaced automobile traffic. New problems should not be created, nor should existing problems merely be transferred from one location to another.

Before any treatment is placed into effect, an a priori assessment should be made of its benefits and impacts. This is important to provide a rational basis for implementing the treatment and to assure its operational viability. A commitment also should be obtained from appropriate government agencies regarding enforcement and maintenance.

Traffic management and bus priority studies of urban freeways are, in reality, freeway operations studies. Demands, queues, and densities, as well as speeds and volumes, should be clearly identified. Various computer models may prove useful in analyzing and simulating priority lane and ramp control strategies.

Guidelines for Specific Priority Treatments

Busways. Busways, unlike most other bus priority measures, may require substantial capital investments. Off-street busways in the city center generally will require peak hour one-way bus volumes of 60 buses per hour, existing bus speeds less than 6 mph (10 kph), and an intensively congested area which extends for more than 1 mile (1.6 km).

Busways should be of economical design and should be built, wherever feasible, for lower per-mile capital costs than rail transit lines. Shoulders generally will not be necessary in view of the relatively low bus volumes (one to three buses per minute each way) and the low incidence of bus breakdown.

Capacities of 6,000 to 9,000 persons per hour can be achieved with 40 foot (12 m) buses and with conventional linear station designs.

Contra-Flow Freeway Lanes. Contra-flow bus lanes are an adaptation of the reversible lane concept applied to urban freeways. Costs are minimal, enforcement is easy, and passenger safety is good, but application depends on suitable road

Table 24. Summary: On-Street Bus Service Options (Bus Lanes and Bus Streets)

Item	Curb Bus Lanes-Normal Flow	Curb Bus Lanes-Contra-Flow (Extended)	Median Bus Lanes	Bus Streets
Applicability	Generally 30-40 buses/hour; 1,200-1,600 people/hour/one-way. Preferably two lanes available for other traffic in same direction. Ability to restrict turns.	40-60 buses/hour; 1,600-2,400 persons/hour/one-way. At least two lanes available for other traffic. Signal spacing greater than 500-foot intervals.	60-90 buses/hour; 2,400-3,600 persons/hour/one-way. At least two lanes available for other traffic. Ability to separate turn conflicts from buses.	60-80 buses/hour; 2,400-3,600 persons/hour/one-way. Commercially oriented frontage. Ability to service buildings (from alternate locations or in off-peak periods.) Minimum garage across requirements along street.
Principal Design Features	10' lanes delineated by paint and signs.	Lanes separated by paint or physical barrier. Opposing left turns prohibited or specially treated.	Lanes separated by paint and/or block-long pedestrian islands at least 5' wide with access at adjacent intersections.	Minimum 22-33' width. Four-lane operation on wide street.
Capacity	60-90 buses/lane/hour desirable (4,500 people) 90-120 maximum (6,000 people).	60-90 buses/lane/hour desirable (4,500 people) 90-120 maximum (6,000 people).	60-90 buses/lane/hour desirable (4,500 people) 90-120 maximum (6,000 people).	60-90 buses/lane/hour desirable For 2 lanes one-way 60 buses/lane/hour desirable.
Construction Costs Per Mile (1974 levels)	\$3,000-\$6,000	\$4,000-\$100,000	\$15,000-\$100,000	\$500,000-\$2,000,000 Costs may be lower where existing streets can be used without physical changes.
Speed				
Minutes/Mile	5 to 10	5 to 8	5 to 8	5 to 7
Minutes/Mile Saved	1.5 to 5.0	1.5 to 6.5	1.5 to 6.5	1.5 to 8
Remarks:	May impact curb access and deliveries. Difficult to enforce.	May impact curb access and deliveries. Self-enforcing.	Difficult for buses to leave lanes in central area. Passengers must cross traffic to board buses. Excellent potential for express service on approaches to city center.	Requires alternate traffic routes. May enhance CBD by removing cars and increasing sidewalk width. Excellent visibility. Self-enforcing. Optimal distribution in medium-sized cities.

Source: Ref. (11). (1 mph = 1.6 kph, 1 min/mile = 0.6 min/km)

geometry, and maintenance costs are high. They should be applied only on six and eight lane freeways, where peak hour traffic is highly imbalanced, and all auto traffic enters and leaves from the right. The bus lane can be separated from opposing traffic by a one lane buffer on eight lane freeways and by traffic posts on six lane freeways. Buses must run non-stop throughout the extent of the contra-flow lanes.

Normal Flow Freeway Lanes. Normal flow freeway lanes for buses and car pools pose potential safety and enforcement problems although they appear more adaptable to car pools than contra-flow lanes. They may be provided by adding a lane to the existing freeway or by designating an existing lane for priority vehicle use. Caution should be exercised if an existing lane is pre-empted in the peak direction of travel—this practice generally should be avoided.

Ramp Metering. Metering freeway ramps and providing bypass lanes for buses (and car pools) has widespread applicability. This is because metering is inexpensive and because it improves general traffic flow as well. The objectives are to give high occupancy vehicles preference around the queues of waiting vehicles, and to simultaneously improve mainline freeway flow.

Storage capacity upstream from the metering point should be adequate to minimize backups onto intersecting streets. Whether the inside or outside ramp lane is metered will depend upon the geometry of the ramp terminal. Where car pool bypass lanes are provided, enforcement must be adequate.

Bus Streets and Bus Lanes. On-street bus priority treatments can be quickly implemented, have minimum environmental impacts, and have low intitial costs. Table 24 gives pertinent features and capacity guidelines for bus streets and lanes. Capacities of 3,000 to 4,500 persons per hour per lane can be achieved with good service levels.

Effectiveness is usually limited in terms of time savings and service reliability. Principal disadvantages include: (1) limited net gains in person capacity: (2) need for suitable diversionary roadways, (3) service requirements of adjacent land use, (4) potential left and right turning restrictions to general traffic, and (5) reliance on high levels of enforcement.

Rail Transit Capacity

This section contains a brief overview of peak-hour rail transit ridership in the United States and Canada, and its passenger capacity implications. More detailed information on rail transit ridership and capacity is set forth in a variety of references (20, 21, 22, 23).

Operational Experience

The operating experience for typical rail rapid transit facilities is shown in Table 25. These figures show typical AM peak hour peak direction passenger volumes per track for various U.S. and Canadian rail transit properties. The wide range of passengers carried reflects many factors, including the number, length, and size of the trains operated. Factors of importance are the peak carrying value assigned for scheduling purposes, the demands in the specific corridors, and the configuration or constraints of principal terminal stations. Track and signal capacities, station platform lengths and arrangements, and the capacities of station stairways, ramps, and escalators, further influence the limits of practical and safe operations. In each case, capacity is determined by station capacity or line capacity, whichever is smaller.

Trains Per Hour. Current 1979 rail operating characteristics are described below. There are generally less than 30 trains per track during the peak-hour in the United States and Canada, although during portions of this period slightly shorter headways are sometimes operated. In general, the 90-second headway which is possible with modern signaling systems is not realized on an hourly basis. The single exception in the PATH system, which operates 38 trains per hour on a single track under the Hudson River from the multi-track World Trade Center Terminal in New York City.

Cars Per Train. Train lengths of 4 to 10 cars are commonly operated. Maximum train lengths range up to 8 cars in Chicago, Toronto, and Washington, D.C., and 10 cars in New York City and San Francisco. The IRT Flushing Line in New York City is the only line which operates 11 cars per train.

Passengers Per Hour. Peak hour passengers carried per track past the maximum load point range upward from 5,400 in Cleveland to 36,000 in Toronto and over 50,000 in New York City. The highest volumes carried are found on the Queens-Manhattan trains passing through the 53rd Street Tunnel (53,000 persons per hour in one direction).

Capacity Factors

The capacity of rail transit line depends upon: (1) car size and train length, (2) allowable crush loads or standees as determined by scheduling policy, and (3) minimum spacing (headway) per train as determined by (a) signal control system and system alignment (horizontal and vertical), and (b) dwell times at major stations. Car lengths range from about 50 feet (15 m) in Chicago and New York (IRT, PATH) to 75 feet (23 m) in Washington, San Francisco, and New York (new cars).

Passenger capacity in the peak direction in the peak-hour can be estimated from the following formulas:

Passengers/hour

$$= \frac{\text{trains}}{\text{hour}} \times \frac{\text{cars}}{\text{train}} \times \frac{\text{seats}}{\text{car}} \times \frac{\text{passengers}}{\text{seat}} \qquad (10)$$

OR

$$= \frac{\text{cars}}{\text{hour}} \times \frac{\text{seats}}{\text{car}} \times \frac{\text{passengers}}{\text{seat}} \qquad (11)$$

Table 25. Reported Rail Rapid Transit Peak Hour Volumes (in Peak Direction)

Name of Line	Trains/Hour	Cars/Hour	Approximate Car Length[a]	Peak-Hour Passengers (Maximum Load Point)
New York City (1976)				
IRT 4,5 Lexington Express	23	230	50'	35,700
IND A,D 8th Avenue Express	23	224	60', 75'	32,700
IND E,F 53rd Street Tunnel	28	266	60', 75'	53,300
PATH Downtown (World Trade Center)[b]	38	266	50'	20,900
Toronto (1974)				
Yonge Street Subway	28	224	75'	36,000
Chicago (1974)				
Lake-Ryan	20	160	50'	14,000
North-South	20	160	50'	14,000
Cleveland (1974)				
West Side	14	52	50', 70'	5,400
San Francisco (1977)				
BART (Mission)	10	80	75'	6,500

[a] Typical values (rounded)

[b] Multiple berth terminal

Source: Various transit operating agencies, compiled by Herbert S. Levinson

(1 foot = .305 meter)

An alternative formulation, based on allowable levels of pedestrian space, is as follows:

Passengers per hour

$$=(\frac{\text{trains}}{\text{hour}} \times \frac{\text{cars}}{\text{train}} \times \frac{\text{ft}^2}{\text{car}}) \div (\frac{\text{ft.}^2}{\text{passenger}}) \qquad (12)$$

This formulation derives a person-capacity which is independent of the seating configuration and which directly relates to the area of each car. Cars which maximize total passengers generally minimize seats.

The precise values for these equations will vary among property, depending upon the type of equipment used and operating policy. Table 26 contains car

Table 26. Rapid Transit Car and Train Capacities

		Length (ft.)	Width (ft.)	Area (ft.^{2})	Seated Passengers	Total Passengers		Maximum Cars/Train	Seated Passengers/ Train
						Design	Crush		
New York City Transit Authority	IRT	51.33	8.79	451.2	44	140	180	10-11	440-484
	IND	60.50	10.0	605	50	180	220	10	500
	R-44 R-46	75.00	10.0	750.0	72-76	225	272-280	8	576-608
Port Authority of NY & NJ (PATH)		51.25	4.23	473.0	42	140	200	7	294
Chicago Transit Authority		48.25	9.33	450.1	c.50	125	135	8	400
Philadelphia (SEPTA)	Broad St.	67.50	10.00	675.0	67	N/A	281 (est)	6	450
	Market St.	55.33	9.08	502.4	55	115	200	8 (est)	440
Massachusetts Bay Transportation Authority	Blue Line	48.75	8.58	418.3	48	125	191	4	192
	Orange Line	55.31	9.28	513.3	54	175	240	4	216
	Red Line	69.81	10.35	722.5	63	208	275	4	252
New Jersey (PATCO)		67.83	10.12	686.4	80	100	200	8	640
Toronto Transit Commission	1953-1958	74.76	10.33	772.3	84	230	310	6	504
	1962-1975	57.00	10.33	588.8	62	174	233	8	496
Bay Area Rapid Transit		75.00	10.5	787.5	72	144	216	10	720
Montreal Urban Community Transit Commission		56.42	8.25	465.5	39	157	208	29	351
Greater Cleveland Regional Transit Authority	Airporter	70.25	10.41	731.3	80	120	140	4	320
	Other	48.75	10.33	403.6	54	100	197	6	324
Washington Metropolitan Area Transit Authority		75.00	10.15	761.2	80	175	240	10	800

and train capacities for U.S. and Canadian rapid transit systems. This tabulation indicates:

- 1.0 seated passenger per foot of car length;
- 10 square feet (0.1 m^2) of space per seated passenger;
- 4.0 square feet (0.4 m^2) passenger = scheduled or "design" load;
- 2.6 square feet (0.2 m^2) passenger = "crush loads".

The "crush load" factor should not be used in determining capacity, since it is not practical to assume that passenger loads will be equally

Table 26. (Continued)

		Total Passengers/ Train		Seated Passengers/ Foot of Length	Total Passengers/ Foot of Length		ft.^{2}/ Seated Passengers	ft.^{2}/Total Passengers	
		Design	Crush		Design	Crush		Design	Crush
New York City Transit Authority	**IRT**	1400	1800	0.86	2.72	3.51	10.2	3.22	2.50
	IND	1800	2200	0.83	2.97	3.64	12.1	3.36	2.75
	R-44 **R-46**	1800	2240	0.96-1.01	3.00	3.73	9.9-10.1	3.33	2.67
Port Authority of NY & NJ (PATH)		980	1400	0.82	2.73	3.90	11.3	3.37	2.36
Chicago Transit Authority		1000	1480	1.03	2.59	3.83	9.0	3.60	2.43
Philadelphia (SEPTA)	**Broad St.**	N/A	1686	0.99	N/A	4.16	10.1	N/A	2.40
	Market St.	920	1600	0.99	2.07	3.61	9.1	4.37	2.51
Massachusetts Bay Transportation Authority	**Blue Line**	500	764	0.98	2.56	3.91	8.7	3.34	2.19
	Orange Line	700	960	0.98	3.16	4.34	9.5	2.93	2.14
	Red Line	832	1100	0.90	2.98	3.94	11.4	3.47	2.62
New Jersey (PATCO)		800	1600	1.01	1.47	2.95	8.6	6.68	3.43
Toronto Transit Commission	**1953-1958**	1380	1860	1.12	3.08	4.14	9.2	3.36	2.49
	1962-1975	1392	1864	1.09	3.05	4.09	9.5	3.38	2.52
Bay Area Rapid Transit		1440	2160	0.96	1.92	2.88	10.9	5.47	3.64
Montreal Urban Community Transit Commission		1413	1872	0.69	2.78	3.69	11.9	2.96	2.23
Greater Cleveland Regional Transit Authority	**Airporter**	480	560	1.14	1.71	1.99	9.1	6.09	5.22
	Other	600	1182	1.11	2.05	4.04	9.3	5.04	2.55
Washington Metropolitan Area Transit Authority		1600	2400	1.07	2.33	3.20	9.52	4.35	3.17

Source: Herbert S. Levinson

(1 foot = .305 meter)

distributed among cars. Moreover, such loads are unacceptable to passengers, except for very brief periods. For these reasons, the scheduled loads are normally 65 to 75 percent of the crush load.

Pushkarev and Zupan suggest the use of 5.4 square feet (0.5 m^2) per passenger, as a realistic passenger capacity (22).

Typical ranges in rail rapid transit capacities are summarized in Table 27 for U.S. and Canadian operating experiences, based on 30 trains per track per hour. Ranges reflect varying car lengths (50 feet and 75 feet) (15 m and 23 m) and train sizes (6, 8, or 10 cars) and passenger load factors. These capacities can be adjusted upward or downward based on specific operating policies.

In estimating rail transit capacities and levels of service or overcrowding, it is essential to analyze the peak 15 or 20 minute periods. For example, a "scheduled level" of 200 percent standees (3.3 square feet per passenger or 0.3 m^2/pass.) would relate to the peak 15 minute period. Similarly, if an hourly capacity of 27,000 people is provided with 6-car 50 feet (15 m) trains with 200 percent standees, this implies that the peak 15 minutes would carry 6,750 people. If half of the peak hour passengers moved in this period, then the effective hourly capacity would be 13,500. In this case, the peak hour factor would be 0.5, therefore, the hourly capacity would be 0.5 × 27,000 or 13,500. These peaking characteristics further explain the differences between observed passengers and theoretical capacities.

Service levels are also shown in Table 27 for various load factors, i.e., percent standees. General guidelines are suggested as listed in Table 28.

Light Rail Systems

As with all other rail transit, the passenger carrying capacity of light rail transit (LRT) depends on vehicle size, train length, and headway. However, the realizable LRT capacities also depend on design and policy considerations which reflect specific local constraints of at grade operations and type of right-of-way. LRT trains are usually limited to a maximum of three or four cars. There are several reasons that longer trains are not used. The major reason is that longer trains could not operate on city streets without simultaneously occupying more than one intersection when traversing short blocks. Other reasons for limiting the size of trains include clearing at-grade intersections rapidly and the desirability or need to limit platform length at the stations. The Canadian light rail vehicle will be designed to operate in eight car trains, but this feature is not currently planned to be utilized in operation.

Headways for light rail systems can also vary. For operation under the control of block signaling system, as is common in rail rapid transit, 120 second headways are typical. At these headways, a high speed LRT system operating on mainly reserved right-of-way with three-unit Boeing vehicle trains would have a line capacity slightly in excess of 6,000 seated and 19,000 total passengers per hour. Under single vehicle manual operation at lower speeds, closer headways are feasible. At 60 second headways, single Boeing LRT units have a capacity of 4,000 seated and 13,000 total passengers per hour (24).

Several European systems have reported capacities ranging up to 18,000 persons per track per hour. Capacities have been reported as high as 15,000 on existing U.S. light rail systems, although current 1979 volumes are considerably lower.

Operating Experience. The Newark Streetcar Subway operates as one route with 30 single-unit cars per hour, with a minimum peak headway of 90 seconds. Its capacity is limited by equipment shortage to approximately 3,000 persons per hour.

The Philadelphia Market Street streetcar subway accommodates five routes. Single-unit cars are operated which load in platoons at downtown stations. Special variable message signs direct passengers to the correct boarding point for each car. This facility has carried 140 cars per hour, with a minimum headway of 23 seconds. Peak hour flow in 1977 was 9,000 passengers per hour, although as many as 12,000 passengers per hour have been observed in previous years.

The Boston-Tremont-Boylston streetcar subway

Table 27. Typical Rail Transit Capacities:
(Thirty Trains per Track per hour, 2 minute headway)

			Passengers Per Hour					
Cars/Train	Car Length	Approximate Seats/Train	Approximate Seat Load (1.00)[a]	50% Standees (1.50)[a]	100% Standees (2.00)[a]	150% Standees (2.50)[a]	200% Standees (3.00)[a]	250% Standees (3.50)[a]
6	50 ft.	300	9,000	13,500	18,000	22,500	27,000	40,500
180 cars/ hour	75 ft.	450	13,500	20,250	27,000	33,750	40,500	60,750
8	50 ft.	400	12,000	18,000	24,000	30,000	36,000	54,000
240 cars/ hour	75 ft.	600	18,000	27,000	36,000	45,000	54,000	81,000
10	50 ft.	500	15,000	22,500	30,000	37,500	45,000	67,500
300 cars/ hour	75 ft.	750	22,500	33,750	45,000	56,250	67,500	101,250
ft.^{2}/Passenger:			10.0	6.7	5.0	4.0	3.3	2.8
Passenger Level of Service (U.S. & Canada Conditions)			C	D	E-1	E-2	E-1	F-2

[a] Passengers per seat

Source: Herbert S. Levinson

(1 foot = .305 meter)

Table 28. Levels of Service for Rail Transit

Peak-Hour Level of Service	Passengers/Seat[a]	ft.^{2}/Passenger[a]
A	0.00-0.65	15.4 or more
B	0.66-1.00	15.2-10.0
C	1.01-1.50	9.9- 7.5
D	1.51-2.00	6.6- 5.0
E	2.01-2.50	4.9- 4.0
E (Scheduled Load)	2.51-3.00	3.9- 3.3
F (Crush Load)	3.01-3.80	3.2- 2.6

[a] Approximate

Note: Fifty percent standees reflects a load factor of 1.5 passengers per seat.

Sources: H.S. Levinson and W.R. Reilly (1 square foot = 0.09m^2)

operates single-unit two-car and three-car trains on four routes. A volume of 60 trains per peak hour was traditionally scheduled, totaling about 150 cars. At one station (Park Street), there is multi-platform loading. Cars load simultaneously in platoons at all downtown stations. The Massachusetts Bay Transportation Authority (MBTA) estimated the capacity of the subway at 15,000 persons per hour in 1971, when inbound peak hour flow approximated 12,000 persons (25). Owing to on-street interference and extensive service variety, several trains on the same route tend to platoon, which causes an uneven loading of vehicles (26). With many cars loaded to crush capacity, dwell times at major stops are excessive and peak 15 minute conditions are analogous to Level of Service F (forced flow).

These light rail (streetcar) subway experiences imply a practical limit of 120 to 150 buses per hour in CBD busways with on-line stations, even with platooning of buses. The corresponding line haul passenger capacity would range from 6,000 to 7,500 persons per hour, depending on vehicle size and load factor. Higher volumes are conceivable with off-line loading in turnouts or in a parallel lane.

USER APPLICATIONS

Methodology

Engineers and planners encounter various types of problems related to transit operations within the overall transportation system. The purpose of this section is to present procedural guidelines for user-oriented analysis of transit operations and system effects.

This section contains two parts. First, a general description of analysis types along with definitions of terms is presented. Second, example problems and appropriate step-by-step procedures for their solution are given.

The discussion and examples relate to the preceeding DISCUSSION which contains a more thorough presentation of the underlying theories.

Overview of Analysis

There are five general types of transit analysis described in this section, as listed below. For each of the general problem statements, several specialized application contexts can be developed which would relate to particular planning, design, or operations objectives.

Type 1: Effect of Buses on Highway Capacity

For a freeway traffic lane carrying mixed traffic (automobiles, buses), determine the capacity reduction resulting from buses, and the passenger car equivalent (PCE) volume.

For a lane carrying mixed traffic along an urban arterial, determine the losses occurring for auto traffic and the resulting PCE values corresponding to the operations of buses making stops, using berths located either in a through lane (on-line) or in a separated area (off-line).

For an arterial carrying mixed traffic, determine the overall capacity of the street and evaluate the level of service using the volume/capacity ratio.

Type 2: Person Flow

Using car and bus occupancies, determine the total person flow for an arterial street or freeway.

Type 3: Passenger Service Times

For various fare collection methods and bus door configurations, determine the service times for boarding and alighting passengers.

Type 4: Bus Berth Capacity

Determine the capacity of a bus berth, given the passenger loading or unloading characteristics; or, determine the number of berths needed, given the passenger loading and bus operating characteristics.

Type 5: Bus System Capacity

For an arterial street, determine the capacity of an exclusive bus lane, both in buses per hour and passengers per hour. Also, determine the operational variables and level of service for the bus system.

Type 3, 4 and 5 problem categories have received the greatest emphasis in the USER APPLICATIONS section because they require analytical procedures that differed significantly from those provided by the Highway Capacity Manual. The underlying principles and experimental information that was used in derivation of these procedures is presented in the preceeding DISCUSSION.

Definitions

Following are definitions of relevant terms used to help determine transit capacity.

Person Capacity - the maximum number of persons that can be carried past a given location during a given time period under specified operating conditions without unreasonable delay, hazard, or restriction. Usually measured in terms of persons per hour.

Person Level of Service - the quality of service offered the passenger within a transit vehicle, and determined by the available space per passenger.

Uninterrupted Flow - transit vehicles moving along a roadway or track without stopping.

Interrupted Flow - transit vehicles moving along a roadway or track and having to make service stops at regular intervals.

Service Time - the time, in seconds, for a passenger to board or to alight a transit vehicle.

Maximum Load Point - the point along a transit route at which the greatest number of passengers is being carried.

Dwell Time - the time, in seconds, that a transit vehicle is stopped for the purpose of serving passengers. Dwell Time is determined by multiplying Passenger Service Time by the number of passengers boarding or alighting.

Seat Capacity - the number of passenger seats on a transit vehicle.

Percent Standing - the number of standing passengers expressed as a percentage of the number of seats.

Crush Capacity - the number of passengers carried by a transit vehicle with conditions at Level of Service F.

Clearance - the minimum time in seconds between transit vehicles entering or leaving a stop, as determined by operating policy.

Example Problems

Introduction

The following examples and their solutions will be of interest to transit planners and highway engineers in evaluating system capacity, establishing bus schedules, and designing terminals.

Capacity of a transit stop or lane depends upon the size and loading standards of vehicles, the minimum clearance time between buses at stops, and passenger service times. Passenger service times, in turn, depend upon method of fare collection and door size and configuration. It is important to recognize that these factors are largely determined by transit system operating policy, and may vary from system to system.

The examples underscore the need to make reasonable assumptions regarding service times since they have important effects in transit system capacity.

Example 1: Effect of Buses on Freeway Capacity

Problem. Ninety (90) buses operate in the peak direction of a four lane freeway during the peak hour. The freeway also carries 3400 passenger cars in this direction. Average occupancies are 40 persons/bus and 1.5 persons per car.

It is desired to determine:

(a) the equivalent peak hour peak direction passenger car volume,

(b) the level of service—assuming 12 foot lanes and no lateral obstructions, and

(c) the total person-volume.

Analysis. Each bus is the equivalent of 2.0 passenger cars. Therefore, 90 buses are the equivalent of 180 cars (90 x 2.0 = 180). The equivalent passenger car volume is 3400 + 180 = 3580 passenger car equivalents (PCE).

Service volume for Level of Service **D**, with a peak-hour factor of 0.91, is 3300 (as indicated in the 1965 HCM). This would be exceeded by the volume on the road, suggesting that it operates in the lower end of the range for Level of Service **E**.

The total person-volume is calculated as follows:

buses: 90 @ 40 = 3600 (43%)
cars: 3400 @ 1.4 = 4760 (57%)
Total 8360 (100%)

Example 2: Effect of Buses on Arterial Street Capacity

Problem. Sixty (60) buses an hour operate along an arterial street, with a dwell time of 15 seconds per stop. It is desired to determine the total reduction in availble green time in the lane in which buses operate, under the following two cases: (1) buses stop in an adjacent parking lane, and (2) buses stop in the through traffic lane. It is further assumed that:

Green time/cycle ratio (G/C) = 0.5

Maximum capacity of through traffic lane
= 1500 cars/hour of green
= 750 cars/hour (headway = 2.4 seconds)

What is the time loss per hour for each case? What is the percentage of the total lane capacity required for bus operation? What is the passenger car equivalency (PCE) value?

Analysis for Case 1: Buses Stop in Parking Lane. Time loss in traveled lane resulting from buses maneuvering into and exiting the parking lane:

Time loss/bus = 3 seconds/bus
Time loss/hour = (bus volume)(3) = (60)(3)
= 180 secs.

Percent reduction in lane capacity:

$$\frac{\text{Time Loss/hour}}{\text{Green time/hour}} \times 100 = \frac{180}{1800} \times 100$$

= 10% reduction in available green time

This reduction in available green time results in approximately the same percentage reduction in capacity:

(% reduction)(capacity) = (0.10)(750)
= 75 passenger cars per hour reduction

Passenger Car Equivalants (PCE):

$$\text{PCE} = \frac{\text{reduction in capacity (in pch)}}{\text{number of buses}}$$

Sixty (60) buses result in a reduction of 75 passenger cars; therefore, each bus is the equivalent of 75/60 = 1.25 cars.

Analysis for Case 2: Buses Stop in Travel Lane. Time loss for buses stopping in travel lane is obtained as follows:

$$T_L = (G/C)(B)(D + L)$$

where:

T_L = Time loss, in seconds per hour
G/C = Ratio of green time to cycle time
B = Buses per hour that stop
D = Average dwell time in seconds for buses loading or unloading passengers
L = Additional loss in seconds due to stopping, starting, and queuing--assumed to be six (6) seconds under average conditions

then:

$$T_L = (0.5)(60)(15 + 6) = 630 \text{ seconds/hour}$$

Percent reduction in lane capacity:

$$\frac{\text{Time loss/hour}}{\text{Green time/hour}} \times 100 = \frac{630}{1800} \times 100$$

= 35% reduction in lane capacity

This reduction results in approximately the same percentage reduction in capacity, or:

(% reduction)(lane capacity)
= (0.35)(750) = 262 pch

Sixty (60) buses result in a reduction of 262 passenger car units; therefore, each bus is the equivalent of 262/60 = 4.37 cars.

Note that these adjustments apply only to the lane in which buses operate. Adjacent lanes in the same direction would operate with a capacity of 750 cars per hour, while the lane with buses would have a capacity of 750-262 = 488 cars and 60 buses.

Example 3: Passenger Service Times

Problem. An urban bus route has the following passenger demands at stops 1 through 6.

Stop:	1	2	3	4	5	6
Passengers alighting (A):	-	5	8	12	15	20
Passengers boarding (B):	30	10	12	5	-	-

An exact fare method of collection is used, with a 50 cent fare.

It is desired to determine the dwell times at each stop under the following two cases: (1) passengers enter via the front door and leave via the rear door, and (2) all passenger movements take place through the front door.

Analysis. Bus dwell times are computed as follows:

Case 1: One-way flow through doors

$D = bB$ or aA, whichever is greater

Case 2: Two-way flow through doors

$D = aA + bB$

where:

b = Dwell time in seconds per boarding passenger

a = Dwell time in seconds per alighting passenger

A = Number of alighting passengers

B = Number of boarding passengers

The coefficients "a" and "b" each have different values for one-way and two-way flow through a given door and depend upon door width and fare collection methods. Their values can be determined from reference tables 12, 14 and 15. From those tables, values of 3.0 seconds for boarding passengers and 1.7 seconds for alighting passengers are selected as representative of the given door and fare conditions.

The 50 cent fare is best described by the upper value for a "single coin" fare, and the lower value for an "odd penny" fare.

These factors are applied to the number of boarding and alighting passengers as follows:

Passengers	Stop: 1	2	3	4	5	6
Alighting (A)	-	5	8	12	15	20
Dwell time (aA)	-	8.5	13.6	20.4	25.5	34.0
a = 1.7 secs.						
Boarding (B)	30	10	12	5	3	-
Dwell time (bB)	90.0	30.0	36.0	15.0	9.0	-
b = 3.0 secs.						
Totals						
Dwell time, Case 1 (greater of aA or bB)	90.0	30.0	36.0	20.4	25.5	34.0
Dwell time, Case 2 (aA + bB)	90.0	38.5	49.6	35.4	34.5	34.0

Total dwell time for all stops, Case 1: 235.9 secs.
Total dwell time for all stops, Case 2: 282.0 secs.

Considering all six (6) stops, total dwell times are about 236 seconds (4 minutes), where entry and exit are separated, and 282 seconds (4.7 minutes), where both movements take place through a single door. In both cases, total dwell times would be reduced if passengers could prepay fares, or enter via both doors at stop 1.

Example 4: Berth Capacity for Unloading

Problem. A transfer facility is being built in an outlying area to facilitate transfer between feeder buses and a rail rapid transit system. It is assumed that buses will enter the facility on one minute headways, and each discharge 50 passengers. Clearance time required for one vehicle to maneuver out of the berth and for another to enter it is assumed as 20 seconds.

It is desired to know the number of unloading berths that should be provided assuming the following bus configurations:

1. Single-width door, one door used.
2. Single-width door, two doors used.

Analysis. The number of berths required for a given passenger volume can be computed from the following formula:

$$N = \frac{J(aA + C)}{(3600)A}$$

where:

N = The number of effective berths

A = The number of alighting passengers per bus

C = Clearance time per bus

J = Total passengers per hour to be served

a = Dwell time per alighting passenger

also:

$$N = \frac{aA + C}{H}$$

where:

$$H = \text{Headway between buses}$$
$$= \frac{3600}{J/A}$$

Substituting the values of $H = 60$ seconds, $C = 20$, and $A = 50$ passengers per bus, yields the following relationships:

$$N = \frac{a(50) + 20}{60}$$

The appropriate alighting service time factors, are obtained from Table 12 as follows:

1. Single width door, 1 door used:
$a = 1.7$ seconds

2. Single width door, 2 doors used:
$a = 0.9$ seconds

(Case 1) $N_1 = \frac{1.7(50) + 20}{60} = 1.75 \rightarrow$ Use 2 berths

(Case 2) $N_2 = \frac{0.9(50) + 20}{60} = 1.08 \rightarrow$ Use 2 berths

In practice, allowance should be made for: (1) some buses carrying full or standing loads during part of the peak hour, (2) buses operating at closer headways during parts of the hour, and (3) imbalanced use of doors.

One approach is to assume that buses operate with standees for design purposes. Berth requirements, assuming 75 persons per bus would be 2.46 or 3 berths, assuming use of only a single door, and 2 berths, assuming availability of both doors for passenger discharge.

Example 5: Berth Capacity for Loading

Problem. A rail-bus interchange (intermodal terminal) is planned for two urban bus lines.

Passengers pay a "single-coin" fare, and enter via the front door. Each bus has a seating capacity of 50 people, and is equipped with single-width doors. It is desired to determine the berths needed, assuming a clearance time of 15 seconds between buses. Bus frequency on Line 1 is 20 buses/hour, and 30 buses/hour on Line 2.

Analysis. The problem can be analyzed in detail in a manner similar to that for unloading.

The number of berths required for a given passenger volume can be computed from the following formula:

$$N = \frac{J(bB + C)}{(3600)B}$$

where:

N = Number of effective berths

J = Total number of passengers to be served per hour

B = Number of boarding passengers

b = Dwell time per boarding passenger

C = Clearance time per bus (in seconds)

also:

$$N = \frac{bB + C}{H}$$

where:

H = Headway between buses leaving station $\cong 3600/(J/B)$

H_1 = 180 seconds for Line 1

H_2 = 120 seconds for Line 2

Substituting the values of B = 50 passengers/bus, C = 15, and b = 3:

$$N = \frac{3(50) + 15}{H} = 165/H$$

$$N_1 = 165/H_1 = 165/180 = 0.92 \rightarrow \text{Use 1 berth (Case 1)}$$

$$N_2 = 165/H_2 = 165/120 = 1.38 \rightarrow \text{Use 2 berths (Case 2)}$$

During the peak 15 or 20 minutes, buses will probably load to their "design" or "crush" capacity. In this short period: (1) dwell times will increase and/or (2) clearance times between buses will decrease. The berths needed to accommodate loads of 75-80 passengers per bus are determined as follows:

$$N_1 = \frac{bB + C}{H_1} = \frac{3(80) + 15}{180} = 1.42 \rightarrow \text{Use 2 berths (Case 1)}$$

$$N_2 = \frac{bB + C}{H_2} = \frac{3(80) + 15}{120} = 2.20 \rightarrow \text{Use 3 berths (Case 2)}$$

Example 6: Berth Capacity for Loading at Major Stops

Problem. It is desired to determine the capacity of a bus stop for a bus line where 10 people board each bus, passenger loading time is 3 seconds per passenger, and clearance time is 15 seconds per bus. It is assumed that boarding conditions govern.

Analysis. The problem may be analyzed in detail by use of the following formula:

$$Z = \frac{(3600)B}{bB + C}$$

where:

Z = Passenger service capacity in passengers/berth/hour

B = Boarding passengers per bus

b = boarding service time per passenger

if:

B = 10, and b = 3, then:

$$Z = \frac{(3600)(10)}{3(10) + 15} = 800 \text{ passengers per hour}$$

Since 10 people board per bus, the stop has a capacity of 80 buses per hour. This relationship is also shown in Figure 1.

Example 7: CBD Bus Street or Busway

Problem. A central business district "bus-only" street (i.e., at-grade busway) serves 2000 people past the maximum load point in the peak 20 minutes. The heaviest STOP has a 20-minute boarding volume of 1000 people. It is desired to determine (1) the bus frequency, and (2) the number of berths required to accommodate the boarding passenger volume. It is assumed that:

1. "design" bus volumes are 75 persons/bus,

2. clearance time between buses at each stop is 15 seconds, and

3. a pay-as-you-leave fare system is used in the downtown area.

Analysis. Table 13 gives a range of 1.5 to 2.5 seconds per passenger for "passengers" through a single door of pay as you leave. Table 15 suggests a design value of 2.0 seconds per passenger.

The number of buses per hour can be determined from the following formula:

$$f = P/S'$$

where:

f = Bus frequency

P = Demand at maximum load point, in passengers per peak 20 minutes

S' = Passenger capacity of bus (seated + standing)

therefore:

$$f = 2000/75 = 26{,}7 \cong 27 \text{ buses per peak 20 minutes}$$

Number of berths can be obtained from the following relationships:

$$P = \frac{(3600)N}{Xb + C/S'} \qquad N = \frac{P(Xb + C/S')}{3600}$$

where:

P = Persons per hour

N = Number of effective berths

S'= Bus capacity (seated + standing)

C = Clearance between buses

X = Proportion of total boarding passengers which occurs at heaviest stop

b = Boarding time per passenger in seconds

in this example:

$P = (2000 \times 3) = 6000$

$X = (1000/2000) = 0.5$

$C = 15$ seconds

$S' = 75$ persons

$b = 2$ seconds

$C/S' = 0.2$

therefore:

$$N = \frac{6000(0.5 \times 2 + 0.2)}{3600} = \frac{6000(1.2)}{3600} = 2.0 \text{ berths}$$

Therefore, 2 effective berths should be provided. Allowing for berth "inefficiencies" 3 loading positions should be provided (Table 11). This corresponds to a cumulative capacity of 2.25 berths for "on-line" stations and 2.60 berths for "off-line" linear stations.

The Level of Service on the approaches to the central area would be D as shown in Table 21 (75-104 buses/hour lane result in Level of Service D).

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APPENDIX

Additional Data on Passenger Service Times

Table A1. Passenger Service Time Equations, Alighting Only

Location	Date	Day	Type of Fare	Service	Time	Vehicle	Door	n	R^2	S_E	$\frac{S_E}{\bar{T}}$	Equation	Acceptable Range
San Diego, California	12-4-74 12-6-74	Wednesday Friday	Mixed Exact Fare	Local	P.M.	GMC Bus	Front Single	11	.95	1.57	.27	$T = 0.1968 + 1.4545\ ALF$	$1 \le ALF \le 14$
New Brunswick, New Jersey	4-29-74 6-18-74	Monday Friday	Cash, Cash & Change	Suburban	P.M.	GMC Bus	Front Single	21	.91	5.73	.15	$T = 0.9563 + 1.9453\ ALF$	$1 \le ALF \le 30$
Louisville, Kentucky(6)	5-16-68 to 5-29-68	Monday to Friday	Mixed, Cash & Change	Local	All	GMC Bus	Both Single	121	.76	3.06	.34	$T = 1.8437 + 1.1122\ A$	$1 \le A \le 20$
	6-23-69 to 6-27-69	Monday to Friday	Mixed Exact Fare	Local	All	GMC Bus	Both Single	147	.77	2.73	-	$T = 2.2345 + 1.0792\ A$	$1 \le A \le 26$
Newark, New Jersey(34)	10-15-69 10-16-69 10-22,24-69 10-27-69	Monday to Friday	Mixed Exact Fare	Local	A.M. P.M.	GMC Bus	Both Single	64	.76	1.55	.22	$T = 3.3548 + 1.0816\ A$	$1 \le A \le 11$
Louisville, Kentucky(6)	5-16-68 to 5-29-68	Monday to Friday	Mixed, Cash & Change	Local	A.M.	GMC Bus	Both Single	27	.81	2.24	-	$T = 1.8203 + 0.9187\ A$	$1 \le A \le 20$
					Mid	"	"	71	.79	3.09	-	$T = 1.6067 + 1.2141\ A$	$1 \le A \le 19$
					P.M.	"	"	23	.66	3.15	-	$T = 2.0938 + 1.1725\ A$	$1 \le A \le 17$
Washington, D.C.(10)	6-26-67 to 9-9-67	-	Mixed, Cash & Change	Local	Peak with Standees	Bus	-	44	.98	-	-	$T = 3.945 + 0.943\ A$	$1 \le A \le 10$
					Peak-No Standees	"	-	213	.98	-	-	$T = 3.589 + 0.936\ A$	$1 \le A \le 25$
					Off-Peak-No Standees	"	-	205	.96	-	-	$T = 3.963 + 1.155\ A$	$1 \le A \le 19$
Newark, New Jersey(35)	2-25-74 2-27-74 3-13-74 3- 4-74	Monday Wednesday	Cash, Cash & Change	Local	A.M.	Trolley	Front Double	70	.76	2.76	.21	$T = 2.8236 + 0.8929\ ALF$	$3 \le ALF \le 29$
							Rear Double	87	.91	1.94	.12	$T = 2.2079 + 0.8384\ ALR$	$2 \le ALR \le 36$
							Either Double	157	.85	2.41	.16	$T = 3.0155 + 0.8202\ A$	$2 \le A \le 36$
							Both Double	70	.77	2.70	.16	$T = 4.3224 + 0.4573\ A$	$6 \le A \le 54$

Source: Ref. (27)

Table A2. Passenger Service Time Equations, Boarding Only (Cash and Change)

Location	Date	Day	Type of Fare	Service	Time	Vehicle	Door	n	R^2	S_E	$\frac{S_E}{\bar{T}}$	Equation	Acceptance Range
Montreal, Canada	7-17-74 7-18-74	Wednesday Thursday	Mixed	Local	P.M.	GMC Bus	Front Single	18	.98	2.54	.09	$T = -4.4992 + 2.3724\ BDF$	6≤ BDF ≤26
Montreal, Canada	7-17-74 7-18-74	Wednesday Thursday	Mixed	Local	P.M.	Can. Car Bus	Front Single	12	.97	2.64	.09	$T = -3.3720 + 2.3359\ BDF$	7≤ BDF ≤25
Louisville, Kentucky (6)	5-16-68 to 5-29-68	Monday to Friday	Mixed	Local	All	GMC Bus	Front Single	41	.94	2.91	.25	$T = -0.0855 + 2.5855\ BDF$	1≤ BDF ≤16
Newark, New Jersey (6)	10-15, 16-69 10-22 to 24-69 10-27-69	Monday to Friday	Mixed	Local	A.M. & P.M.	GMC Bus	Front Single	157	.87	13.87	.32	$T = 3.1599 + 3.3272\ BDF$	1≤ BDF≤ 48
Louisville, Kentucky (6)	5-16-68 to 5-29-68	Monday to Friday	Mixed	Local	Mid P.M.	GMC Bus	Front Single	26 12	.94 .94	2.72 2.72	- -	$T = 0.2396 + 2.5288\ BDF$ $T = -0.6494 + 2.7169\ BDF$	1≤ BDF ≤14 2≤ BDF ≤16
Irvington, New Jersey	2-27-74 3-28-74	Wednesday Thursday	Mixed	Suburban	A.M. & P.M.	GMC Bus	Front Single	13	.97	7.43	.20	$T = -18.5125 + 7.4099\ BDF$	3≤ BDF ≤19
Washington, D.C. (10)	6-26-67 to 9-9-67	-	Mixed	-	Peak with Standees	Bus	-	42	.99	-	-	$T = 0.845 + 3.926\ B$	1≤ B ≤8
					Peak—No Standees	"	-	191	.97	-	-	$T = 3.376 + 2.676\ B$	1≤ B ≤25
					Off-Peak No Standees	"	-	194	.98	-	-	$T = 1.948 + 3.220\ B$	1≤ B ≤25
Newark, New Jersey (35)	3-13-74	Thursday	Cash	Local	Mid	Trolley	Front Double	23 23	.90 .90	7.52 5.03	.22 .14	$T = 9.208 + 3.862\ BDF$ $T = 6.913 + 1.232\ PEC + 7.466\ PRC$	1≤ BDF ≤25 0≤ PRC ≤10 0≤ PEC ≤15
	3-25-74 3-04-74	Monday Monday	None	Local	A.M. Peak	Trolley	Both Double	12	.89	2.32	.23	$T = 1.5458 + 0.6983\ B$	4≤ B ≤38
							Front Double	12	.92	0.95	.22	$T = 0.4541 + 1.3242\ BDF$	1≤ BDF≤ 9
							Rear Double	51	.93	1.30	.17	$T = 1.6429 + 0.9213\ BDR$	2≤ BDR≤ 29

Source: Ref. (27)

Table A3. Passenger Service Time Equations, Boarding Only (Exact Change)

Location	Date	Day	Type of Fare	Service	Time	Vehicle	Door	n	R^2	S_E	$\frac{S_E}{\bar{T}}$	Equation	Acceptable Range
San Diego, California	12-4-74 12-6-74	Wednesday Friday	Mixed	Local	P.M.	GMC Bus	Front Single	23	.92	5.12	.21	$T = 0.6997 + 2.1308\ BDF$	1 ≤ BDF ≤33
Detroit, Michigan (Curbside)	9-16-74 9-17-74	Monday Tuesday	Mixed	Local	P.M.	GMC Bus	Front Single	20	.94	4.19	.17	$T = -0.8533 + 2.2300\ BDF$	1 ≤ BDF ≤ 28
Detroit, Michigan (Terminal)	9-16-74 9-17-74	Monday Tuesday	Mixed	Local	P.M.	GMC Bus	Front Single	19	.89	10.10	.18	$T = 3.6986 + 2.1889\ BDF$	1≤ BDF ≤ 49
Detroit, Michigan	9-16-74 9-17-74	Monday Tuesday	Mixed	Express	P.M.	GMC Bus	Front Single	26	.93	4.59	.20	$T = -3.3313 + 2.6054\ BDF$	2≤ BDF ≤ 25
Louisville, Kentucky (6)	6-23-69 to 6-27-69	Monday to Friday	Mixed	Local	All	GMC Bus	Front Single	31	.94	2.70	-	$T = 0.5863 + 1.9957\ BDF$	1≤ BDF ≤ 25
Newark, New Jersey (34)	10-15-69 10-16-69 10-22-69 to 10-24-69 10-27-69	Monday to Friday	Mixed	Local	All	GMC Bus	Front Single	110	.95	5.70	.24	$T = 2.3179 + 2.6736\ BDF$	1≤ BDF ≤ 38
Irvington, New Jersey	2-27-74 3-28-74	Wednesday Thursday	Mixed	Local	A.M. & P.M.	GMC Bus	Front Single	12	.94	3.38	.18	$T = -1.8147 + 2.7570\ BDF$	2≤ BDF ≤ 14
Washington, D.C. (10)	6-27-70 to 7-5-70	-	Mixed	-	Peak	Bus	-	153	.97	-	-	$T = 4.740 + 2.604\ B$	1 ≤ B ≤ 25
					Off Peak	Bus	-	248	.98	-	-	$T = 4.342 + 2.853\ B$	1 ≤ B ≤ 25

Source: Ref. (27)

Table A4. Passenger Service Time Equations, Simultaneous Boarding and Alighting

Location	Date	Day	Type of Fare	Service	Time	Vehicle	Door	n	R^2	S_E	$\frac{S_E}{\bar{T}}$	Equation	Acceptance Range
Louisville, Kentucky(6)	5-16-68 to 5-29-68	Monday to Friday	Mixed, Cash & Change	Local	All	GMC Bus	Both Single	297	.88	5.78	.26	$T = 1.7701 + 0.9727A + 2.2756BDF - 0.0234(A \cdot BDF)$	$1 \leq A \leq 24$ $1 \leq BDF \leq 36$
	6-23-69 to 6-27-69	Monday to Friday	Mixed Exact Fare	Local	All	GMC Bus	Both Single	359	.83	6.55	-	$T = 1.6043 + 0.9588A + 2.1543BDF - 0.0202(A \cdot BDF)$	$1 \leq A \leq 30$ $1 \leq BDF \leq 33$
	5-16-68 to 5-29-68	Monday to Friday	Mixed, Cash & Change	Local	A.M.	GMC Bus	Both Single	43	.79	3.85	-	$T = 3.5985 + 1.0089A - 0.0913(A \cdot B) + 0.4653(B)^2 - 0.0215(B)^3$	$1 \leq A \leq 22$ $1 \leq B \leq 17$
					Mid			191	.88	6.38	-	$T = 1.1762 + 1.3822A - 2.3041B - 0.0828(A \cdot B) + 0.0013(B)^3$	$1 \leq A \leq 21$ $1 \leq B \leq 30$
					P.M.			63	.96	4.16	-	$T = 0.4757 + 1.0987A + 2.2614B - 0.0423(A \cdot B)$	$1 \leq A \leq 24$ $1 \leq B \leq 36$
Irvington, New Jersey	2-27-74 3-28-74	Wednesday Thursday	Mixed, Cash & Change	Local	A.M. & P.M.	GMC Bus	Both Single	11	.93	11.10	.23	$T = 1.5601 + 1.2288ALF + 4.504BDF$	$1 \leq ALF \leq 20$ $1 \leq BDF \leq 23$
	2-27-74 3-28-74	Wednesday Thursday	Mixed Exact Fare	Local	A.M. & P.M.	GMC Bus	Both Single	11	.96	8.37	.21	$T = 6.0043 + 3.2503(BDF + ALF)$	$3 \leq (BDF + ALF) \leq 33$
Washington, D.C.(10)	6-26-67 to 9-9-67	-	Mixed, Cash & Change	-	Peak with Standees	Bus	-	224	.99	-	-	$T = 3.602 + 0.873A + 3.340B - 0.0298(A \cdot B)$	-
					Peak—No Standees			849	.96	-	-	$T = 3.439 + 0.911A + 2.901B - 0.0324(A \cdot B)$	-
					Off Peak with Standees			1160	.96	-	-	$T = 3.456 + 1.094A + 3.084B - 0.0554(A \cdot B)$	-
					No Standees			1114	.96	-	-	$T = 3.512 + 1.088A + 3.078B - 0.0523(A \cdot B)$	-
Newark, New Jersey(35)	3-13-74	Wednesday	Cash	Local	A.M.	Trolley	Front	15	.80	2.93	.17	$T = 3.2364 + 1.2564(ALF + BDF)$	$4 \leq (ALF + BDF) \leq 20$
	2-27-74	Wednesday	Cash & Change	Local	A.M.	Trolley	Both	18	.80	2.66	.13	$T = 3.1066 + 0.5789(B + A)$	$11 \leq (A + B) \leq 47$
San Diego, California	12-4-74 12-6-74	Wednesday Friday	Mixed Exact Fare	Local	P.M.	GMC Bus	Front	10	.78	7.54	.26	$T = -3.9599 + 2.4184(ALF + BDF)$	$5 \leq (A + B) \leq 21$

Source: Ref. (27)

Table A5. Passenger Service Time Influence Zone Parameters, Alighting Passengers Only

Width of Doors	Number of Doors on Vehicle	Number of Doors Used	Parameters C_1	Parameters C_2	Observed Range of Passengers
Single	1	1	1.0 to 13.5	1.9 to 2.8	1 to 69
	2	1	0.2 to 1.0	1.5 to 1.9	1 to 20
	2	2	1.9 to 2.0	0.9 to 1.2	1 to 37
Double	2	1	1.8 to 1.9	0.8 to 0.9	2 to 36
	2	2	3.7 to 4.0	0.4 to 0.5	6 to 54

$$T = C_1 + C_2 A$$

where

T = Passenger Service Time in Seconds

A = Number of Passengers Alighting

Source: Ref. (27)

Table A6. Passenger Service Time Influence Zone Parameters, Front Door Boarding with Payment of Fare

Method of Fare Collection	Width of Door	Type of Fare	Parameters C_1	C_2	Observed Range of Passengers
Cash and Change	Single	Multiple Zone	-20.0 to -6.6	6.1 to 8.1	2 to 63
	Single	Flat	- 3.0 to 1.0	2.3 to 3.6	1 to 52
	Double	Flat	3.4 to 9.2	0.9 to 3.9	1 to 25
Exact Fare	Single	Flat	1.0 to 4.3	1.8 to 2.9	1 to 49

$T = C_1 + C_2 B$

where

T = Passenger Service Time in Seconds

B = Number of Passengers Boarding

Source: Ref. (27)

Table A7. Passenger Service Time Influence Zone Parameters. Boarding Passengers with No Fare Payment

Width of Doors	Number of Doors on Vehicle	Number of Doors Used	Parameters C_1	C_2	Observed Range of Passengers
Single	1	1	-2.0 to 3.2	2.9 to 3.9	11 to 63
	2	1	-3.0 to -2.0*	2.4 to 2.9*	-
	2	2	2.6 to 3.1*	1.1 to 1.4*	-
Double	2	1	1.6 to 2.6	0.9 to 1.1	1 to 29
	2	2	0.5 to 0.6	0.7 to 0.8	4 to 38

$T = C_1 + C_2 B$

where

T = Passenger Service Time in Seconds

B = Number of Passengers Boarding

*Estimated parameters developed by author based on available information.

Source: Ref. (27)

Table A8. Passenger Service Time Influence Zone Parameters, Passengers Alighting and Boarding Through Front Door with Fare Payment

Method of Fare Collection	Parameters C_1	C_2	C_3
Cash and Change	1.0 to 2.0	1.2 to 2.0	2.3 to 4.5
Exact Fare	0.5 to 1.5	1.0 to 1.9	1.8 to 2.9

$$T = C_1 + C_2\,ALF + C_3\,BDF$$

where

T = Passenger Service Time in Seconds

ALF = Number of Passengers Alighting Through Front Door

BDF = Number of Passengers Boarding Through Front Door

Source: Ref. (27)

Table A9. Passenger Service Time Influence Zone Parameters, Passengers Alighting Through Both Doors and Boarding Through Front Door with Fare Payment

Method of Fare Collection	Parameters C_1	C_2	C_3	C_4
Cash and Change	-0.1 to 3.6	0.9 to 2.2	2.3 to 4.9	0.02 to 0.5
Exact Fare	0.5 to 2.4	1.0 to 1.4	2.1 to 2.9	0.02 to 0.1

$$T = C_1 + C_2\,A + C_3\,BDF - C_4\,(A \cdot BDF)$$

where

T = Passenger Service Time in Seconds

A = Number of Passengers Alighting

BDF = Number of Passengers Boarding Through Front Door

Source: Ref. (27)

Pedestrians

DISCUSSION

Concepts

The purpose of this section is to describe the basic principles of pedestrian traffic flow theory and operational experience and to present a general framework of procedures for analysis of capacity and level of service on pedestrian facilities. This section also includes a thorough presentation of specific user-oriented analytical techniques and several accompanying examples illustrating typical applications.

The scope of the pedestrian analysis is limited specifically to pedestrian areas and facilities that are generally located within the street right-of-way boundaries. The analytical framework focuses on the pedestrian flow operations on walkways such as sidewalks and crosswalks and in stopping and queueing spaces such as reservoir areas at intersections. This scope is consistent with the emphasis on "highway" analysis that is prevalent in the current Highway Capacity Manual (HCM) (1) and that will be retained, with some modifications, in the prospective new manual.

Many users of the HCM have indicated that the incorporation of pedestrian operations as a major component of urban street capacity analysis is an issue of increasing importance in planning, designing, and operating transportation systems. The emergence of safety considerations, multi-modal evaluation methods, and person-movement concepts has generated a definite need for a more comprehensive and precise documentation of pedestrian operations within the transportation system. For example, HCM users need procedures to analyze quality of service considerations that affect pedestrian behavior, such as overcrowding of pedestrians on sidewalks and at intersections that can result in safety and street capacity problems caused by pedestrian-vehicle interactions.

This analytical framework encompasses the most significant areas of influence and interaction between pedestrian and vehicular traffic components, and it will enable pedestrian analysis that is complementary to highway analysis.

This report does not address directly other major types of pedestrian facilties—stairways, escalators, elevators, moving walks, pedestrian malls—which normally are located outside the street right-of-way. Although these facilities do have certain operational characteristics in common with sidewalks, crosswalks, and reservoir areas, it is more appropriate to use other specialized analytical procedures for this group of off-line facilities. Analytical procedures for analysis of off-line pedestrian facilities are described in various reference documents, such as (2), (3), and (4).

The principles of pedestrian flow theory and operation are similar in nature to the principles of vehicular traffic flow. The fundamental relationship between speed, volume, and density for a pedestrian stream is analogous to the vehicular flow relationship. That is, as volume and density of the pedestrian stream increase from free-flow to capacity levels, the speed decreases. As density increases beyond the capacity level, both volume and speed decline rapidly.

Typically, walkways involve relatively uninterrupted flow conditions which are affected to an extent by a variety of friction factors such as stopped pedestrians and obstacles. On the other hand, crosswalks and reservoir areas experience interrupted flow characteristics which are determined by intersection control features and vehicle operations. It is therefore necessary that the analytical framework incorporate adjustment factors that are sensitive to geometric and operational elements.

The Level of Service concept, as related to vehicular traffic analysis, can also be applied usefully to pedestrian operations. This analysis enables a differentiation of various pedestrian flow conditions by the assignment of levels of service corresponding to particular density and speed criteria. The analytical framework must incorporate procedures for an integrated evaluation of both sidewalk and intersection facilities, involving uninterrupted and interrupted pedestrian flow conditions, for any given segment or longer route within the pedestrian system.

The interaction between pedestrian and vehicular traffic at the intersection is another important element of the comprehensive analytical framework. This section develops general procedures for analysis of intersection geometry and control measures and vehicle operations that affect the level of service and capacity of the pedestrian traffic system. A major research effort is being pursued by NCHRP to refine these analytical procedures and to quantify pedestrian-vehicle interaction effects at intersections. When these refined procedures are available, the analytical framework described in this report will then be revised before inclusion in the future manual.

The following sections contain a listing of major terminology definitions and a discussion of the basic principles of pedestrian flow and the level of service concept. Subsequent sections indicate the rationale for development of the general analytical framework and the detailed procedures for capacity and level of service analysis of sidewalks, crosswalks, and reservoir areas.

Definitions

Important variables and parameters incorporated in the analytical framework are defined as follows:

Speed - The rate of movement of pedestrian traffic in a specified direction, expressed as feet per minute (ft/min) or meters per minute (m/min). Speed values are used to describe stopped traffic, as well as moving traffic, on all types of walkways.

Flow - The number of pedestrians that pass a specified point on a walkway in a specified direction during a specified time period, expressed as pedestrians per minute (ped/min). "Point" refers to a line-of-sight that is usually oriented perpendicularly across the entire width of a walkway. Flow is also commonly termed "Flow Rate".

Unit Width Flow - The flow of pedestrians per unit of effective walkway width, expressed as pedestrians per minute per foot (ped/min/ft or ped/min/m).

Platoon Flow - The flow of a single platoon of pedestrians, or a series of platoons, passing a specified point.

Volume - The number of pedestrians for a time period of 15 minutes or longer.

Density - The number of pedestrians that are located within a specified walkway segment at a given instant, expressed as pedestrians per square foot (ped/ft^2 or ped/m^2).

Pedestrian Space Module - The inverse of density, expressed as space (or area) per pedestrian (ft²/ped or m²/ped).

Principles of Pedestrian Flow on Walkways

The primary characteristics of pedestrian flow on walkways are similar in nature to those of the vehicular flow. As with vehicular flow, there are numerous indicators of the degree of mobility for pedestrian facilities. One principal mobility measure is related to the free choice of speed. Other mobility indicators include the ability to pass slower pedestrians, to walk perpendicular to or in the reverse direction from the major flow of pedestrian traffic, and the general ability to maneuver without abrupt changes in speed, direction, or gait.

Other inherent characteristics of pedestrian operations are distinctly different than vehicular operations. In addition to the mobility indicators described above, there are other important aspects which contribute to the overall quality of the pedestrian walking experience. These supplemental factors include comfort, convenience, safety, security, and economy.

Comfort - climate control, walkway surface condition, and grade.

Convenience - directness of path, conflicts with standing pedestrians and obstacles, availability of ramps, and pedestrian controls.

Safety - hazards associated with vehicular traffic, obstacles, and surface condition.

Security - amount of lighting and surveillance, walkway activity level, restrictions to open view.

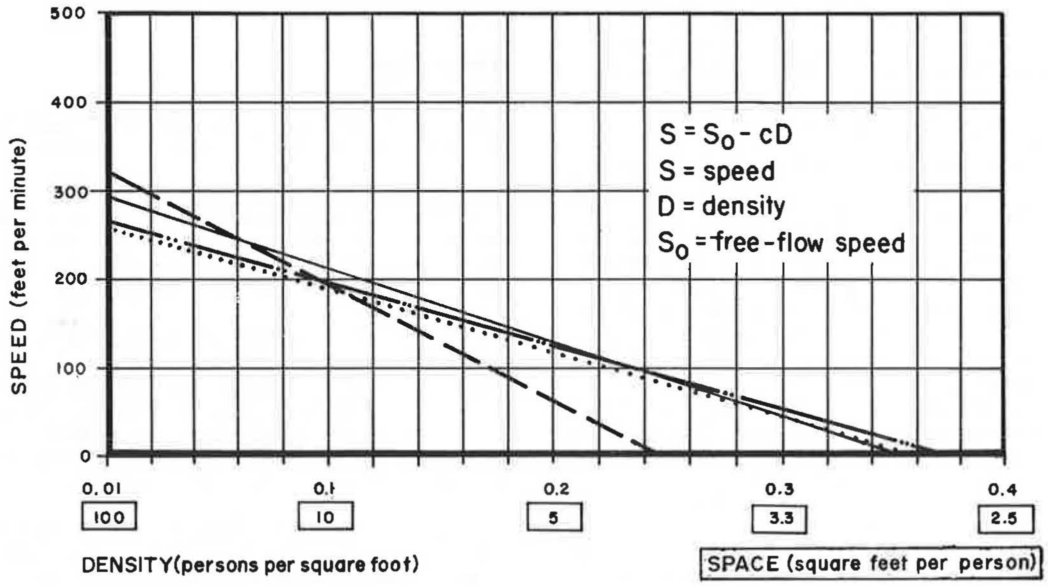
Economy - user cost (primarily associated with travel delay).

These supplemental factors are indicators of relatively intangible environmental attributes. Nevertheless, they probably have a more important effect on the pedestrian's overall quality of service than they have for vehicular traffic. Pedestrian travelers can experience high sensitivity to these types of factors that are not within their immediate control, whereas vehicular travelers presumably can control such factors to a greater degree and thereby limit their associated effects. The analytical procedures presented in the subsequent sections emphasize the mobility characteristics; however, the supplemental characteristics listed above should also be considered in certain applications when planning, designing and evaluating alternative actions with regard to the environmental attributes of pedestrian facilities.

Speed - Density (Space) Relationship

As with vehicular flow analysis, there would be

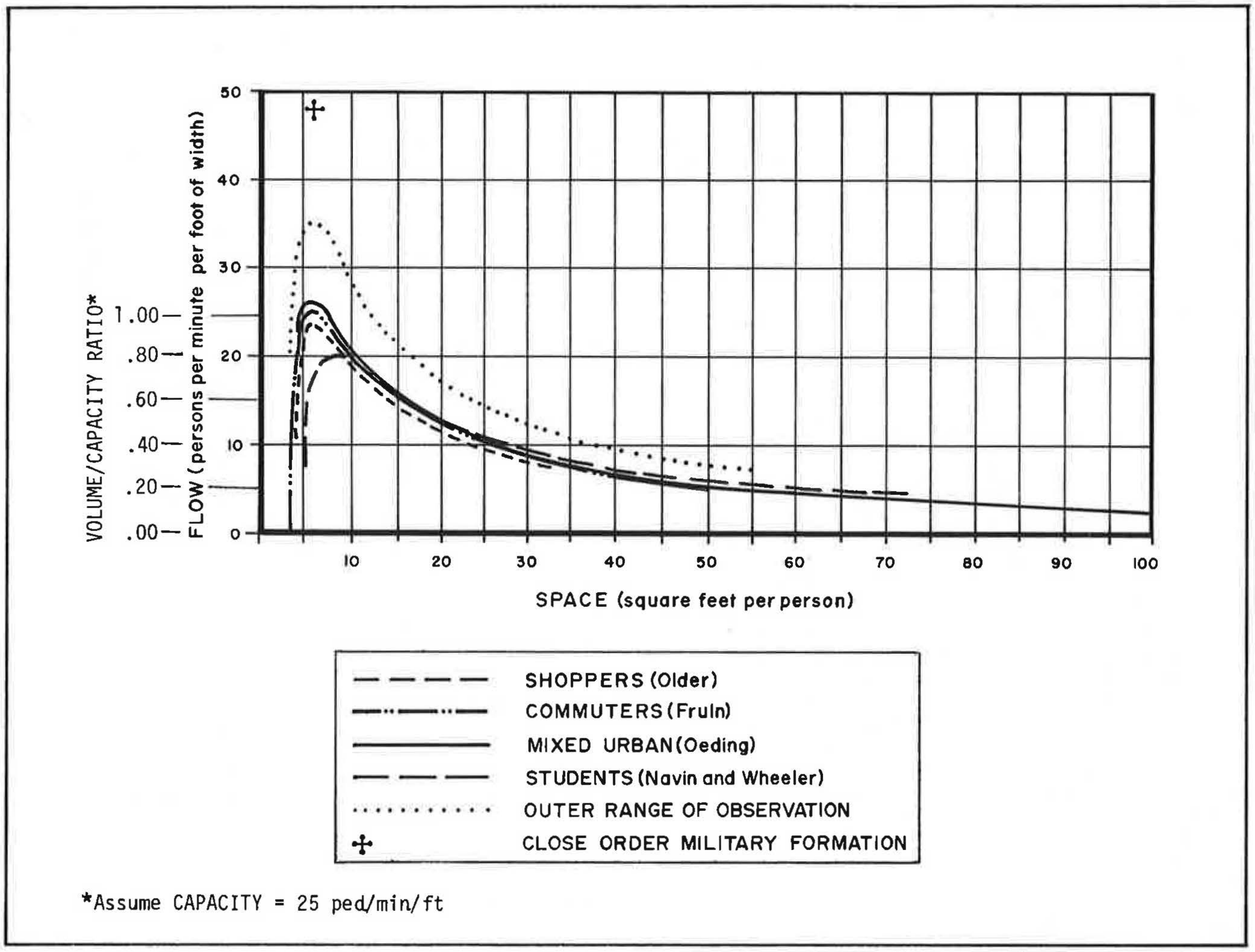
Figure 1. Speed - Density/Space Relationship



S_o ft./min.	c	$\frac{S_o}{c}$ ped./ft²	$\frac{c}{S_o}$ ft²/ped.	PEDESTRIAN TYPES (RESEARCHERS)
268	714	0.36	2.77	·········· SHOPPERS (Older)
267	722	0.37	2.70	—··—·· COMMUTERS (Fruin)
295	835	0.35	2.83	——— MIXED URBAN (Oeding)
320	1280	0.25	4.00	— — — STUDENTS (Navin and Wheeler)

Source: Adapted from Ref. (2), p. 80-84 (1 foot = .305 meter)

Figure 2. Flow - Space Relationship



*Assume CAPACITY = 25 ped/min/ft

Source: Ref. (2) (1 foot = .305 meter)

considerable difficulty in simultaneously incorporating all the various indicators of mobility of pedestrian flow in an effective analysis. To develop a reasonable approach, the analytical framework must necessarily be simplified by incorporating consolidated pedestrian traffic measures that can be directly quantified and integrated with the vehicular traffic analysis. Consequently, speed, density, and volume measures of pedestrian flow have been stipulated as the principal indicators of mobility to be used in the analytical process.

The fundamental relationship between speed, density, and volume for pedestrian flow is analogous to the vehicular flow relationship. As volume increases above a minimal level, the speed of the pedestrian stream tends to decline. This decreasing speed characteristic is directly attributable to the change in density of the pedestrian stream as volume increases. Clearly, the higher the density of flow, the lower the degree of mobility afforded the individual pedestrian and, consequently, the lower the resultant average speed of the stream.

Figure 1 shows the relationship between speed and density for a variety of pedestrian classes as determined by four investigators. The density term, when used to describe pedestrian flow and specified in persons per square foot (or square meter) will have small values usually less than 1.00. It is easier to visualize the reciprocal of density, or available space per pedestrian, which is shown as an alternative horizontal scale in Figure 1. This reciprocal is termed the "pedestrian space module" and it is measured in square feet (or square meters) per pedestrian.

Flow - Density (Space) Relationship

The basic relationship between density (space) and pedestrian flow can be developed using the fundamental relationship of vehicular traffic flow as an analogous expression. This expression, complete with appropriate pedestrian units, is:

$$\text{FLOW} = \text{SPEED} \times \text{DENSITY} \qquad (1)$$

where, FLOW is expressed as ped/min/ft (ped/min/m); SPEED is expressed as ft/min (m/min); DENSITY is expressed as ped/ft^2 (ped/m^2).

"Flow" variable used in this expression is termed "unit-width flow", which is defined as flow rate per unit width of the walkway.

An alternative expression can be developed using SPACE (pedestrian space module) as the reciprocal of DENSITY:

$$\text{FLOW} = \text{SPEED}/\text{SPACE} \qquad (2a)$$

This basic relationship between flow and space, as recorded by four investigators, is shown in Figure 2.

The conditions of maximum flow are of greatest interest since the maximum possible flow is the CAPACITY of the walkway facility. It is apparent from Figure 2 that all the different observations of maximum flow (per unit of width) fall within a very narrow range of density--that is, with space allocations per pedestrian varying between about 5 and 9 square feet (0.5 and 0.8 m²). Even the study by Oeding containing the outer range of observations indicates that maximum flow occurs in this density range, although the actual flow in this study is considerably higher than the others. As space is reduced to less than 5 square feet per pedestrian (0.5 m²/ped), the flow rate declines precipitously, and all movement eventually comes to a standstill at the minimum space allocation of 2 to 4 square feet (0.2 to 0.4 m²).

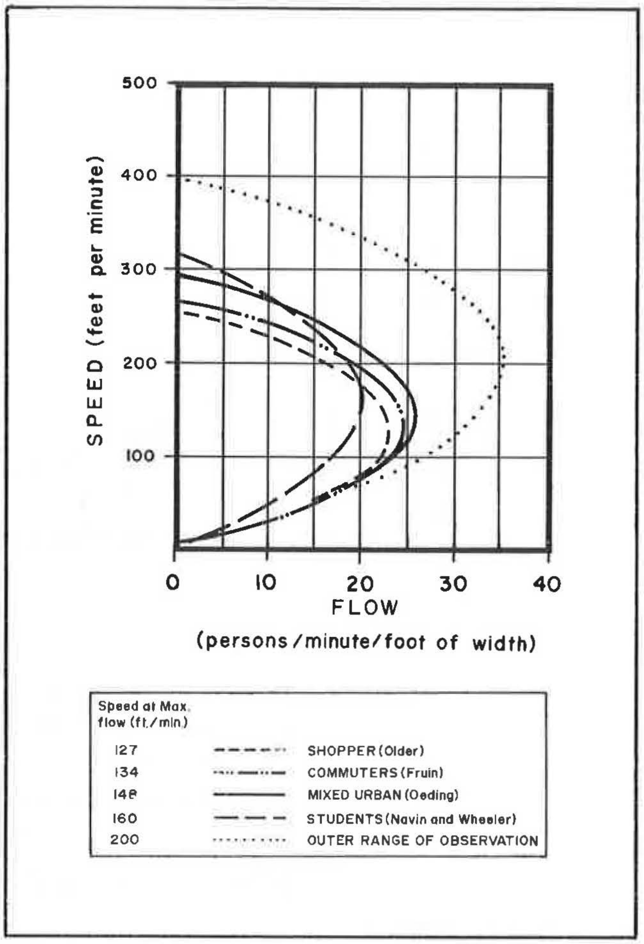
Thus, this flow-space relationship indicates that pedestrian traffic possesses characteristics which can be evaluated quantitatively using quality of flow and level of service concepts similar to vehicular traffic analysis. For example, if a walkway must be designed to accommodate a given pedestrian demand, the space allocation per pedestrian (determined by walkway width and segment length) should be no less than 5 to 9 square feet (0.5 to 0.8 m²) throughout the segment. This space allocation would just accommodate the pedestrian demand at capacity conditions, which provides a relatively low level of speed and comfort. Space allocations below this range would lead to an even lower quality of service and flow less than capacity, since the given number of incoming pedestrians would be greater than the walkway's capacity and unstable flow conditions would materialize. Conversely, space allocations above the capacity levels would accommodate the given pedestrian demand and provide a higher quality of service.

Speed - Flow Relationship

To further understand these quality of flow implications, it is necessary to define the relationship of pedestrian speed, flow, and space variables. This relationship is depicted in Figure 3, which has curves that are similar in shape to the speed-volume curves for vehicular flow.

When there are few pedestrians on the walkway and flow is low, there is a free choice of walking speed and, hence, free-flow speeds are high. As flow increases, each participant in the traffic stream is more and more affected by others and the average speed declines. As the curves suggest, at approximately half the average free-flow speed, flow is at a maximum, or capacity level. As speed drops below this point, the flow declines.

Figure 3. Speed - Flow Relationship



Source: Ref. (2) (1 foot = .305 meter)

Speed - Density (Space) Relationship

The manner in which speed varies with space is depicted graphically in Figure 4 together with the outer range of observations from Oeding. Figure 4 confirms that speed declines rapidly toward zero at space allocations less than 4 square feet per pedestrian (0.4 m²/ped) and is about half of the maximum observed speeds when space is in the 5 9 square feet per pedestrian (0.5 to 0.8 m²/ped) range.

The extreme observations depicted in Figure 4 are of special interest, for they reveal certain effects related to the distribution of walking speeds in pedestrian traffic. At the low end of the space allocation range, it is evident that the slowest walkers traveling about 150 feet per minute (46 m/min) are not able to achieve their chosen speed with space allocations below 15 square feet (1.4 m²). That is, even the slowest pedestrians cannot walk at their chosen speed when space per pedestrian drops below that level. At the high end of the space allocation range, the fast walkers wishing to travel up to 350 feet per minute (108 m/min) are not able to achieve that speed until space allocations increase to about 40 square feet (3.8 m²). Consequently, these threshold values for speeds and space allocations suggest points of demarcation that can be useful for developing level of service standards.

Effective Walkway Width

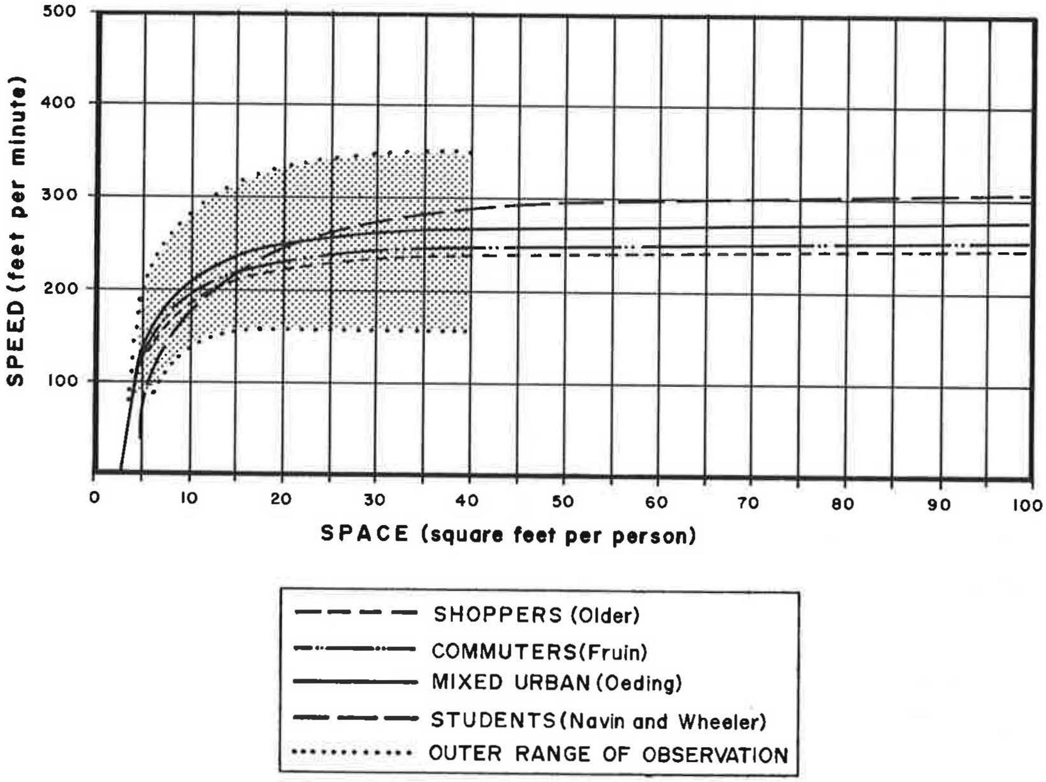
To complete this analysis of the basic relationships between pedestrian flows and corresponding space allocations the issue of "effective walkway width" must be carefully reviewed and incorporated into the analytical framework.

By analogy with highway design, some investigators in the past have used the concept of a pedestrian "lane". The "lane" is meaningful in pedestrian analysis only if one wishes to calculate how many people can walk abreast, or pass each other simultaneously, along a walkway of a given width.

To avoid interference while passing each other, two pedestrians should each have at least 2.5 feet (0.77 m) of walkway width, as observed by Oeding and Pushkarev (2). Pedestrians who know each other and are walking together will walk as close as 2 feet, 2 inches (0.65 m), center-to-center, and at this distance there is considerable likelihood of contact. Lateral spacing of less than 2 feet (0.60 m) between strangers mostly occurs, as Fruin (3) has shown, only when the space allocation is less than about 5 square feet per person (0.5 m^2/ped). Thus, multiples of about 2.5 feet (0.77 m) can be used to calculate clear walkway width necessary for a given number of people to walk abreast in a voluntary group or for one group to pass another group.

The term "clear walkway width" is related to the utilization of a walkway for pedestrian movements. Moving pedestrians will shy away from the curb and they will not press closely against building walls. Therefore, there is dead space along both edges of a walkway which must be excluded from its nominal width when calculating pedestrian design flows. Pushkarev (2) assumes this "buffer" width to be a total of 2.5 feet (0.77 m), combining both curb and building effects. Also, a strip pre-empted by standing pedestrians near a building and physical obstructions such as light poles, mail boxes, and parking meters should be excluded, though the exact effects of these impediments on pedestrian flow have not been thoroughly investigated.

Figure 4. Speed - Space Relationship



Source: Ref. (2) (1 foot = .305 meter)

Table 1. Fixed Obstacle Width Adjustment Factors[a]

Obstacle	Approximate walkway width preempted[b]	
	ft	m
Street Furniture		
Light Poles	2.5-3.5	0.8-1.0
Traffic signal poles and boxes	3.0-4.0	0.9-1.2
Fire alarm boxes	2.5-3.5	0.8-1.0
Fire hydrants	2.5-3.0	0.9-0.9
Traffic signs	2.0-2.5	0.6-0.8
Parking meters	2.0	0.6
Mail boxes (1.7 x 1.7 dimensions)	3.2-3.7	1.0-1.1
Telephone booths (2.7 x 2.7 dimensions)	4.0	1.2
Waste baskets (1.8 diameter)	3.0	0.9
Benches	5.0	1.5
Public Underground Access		
Subway stairways	5.5-7.0	1.6-2.1
Subway ventilation gratings	6.0 +	1.8
Transformer vault ventilation gratings	5.0 +	1.8
Skylights for subway stations	(not available)	
Landscaping		
Trees (5.0-6.0 pavement cut)	3.0-4.0	0.9-1.2
Planting boxes (3.7 diameter)	5.0	1.5
Commercial Uses		
Newsstands	4.0-13.0	1.2-4.0
Vending stands (fruit, vegetable, etc.)	variable	
Advertising displays	variable	
Store displays	variable	
Sidewalk cafes (two rows of tables)	variable, try 7.0	2.1
Building Protrusions		
Columns	2.5 x 2.5 to 3.0 x 3.0	0.8-0.9
Stoops	2.0-6.0	0.6-1.8
Cellar doors	5.0-7.0	1.5-2.1
Standpipe connections	1.0	0.3
Awning poles	2.5	0.8
Trucking docks	(trucks protruding)	
Garage entrances	(cars entering and exiting)	
Driveways	(cars entering and exiting)	

[a] To account for the avoidance distance normally occurring between pedestrians and obstacles, an additional 1.0-1.5 feet must be added to the preemption width for individual obstacles.

[b] Curb to edge of object, or building face to edge of object.

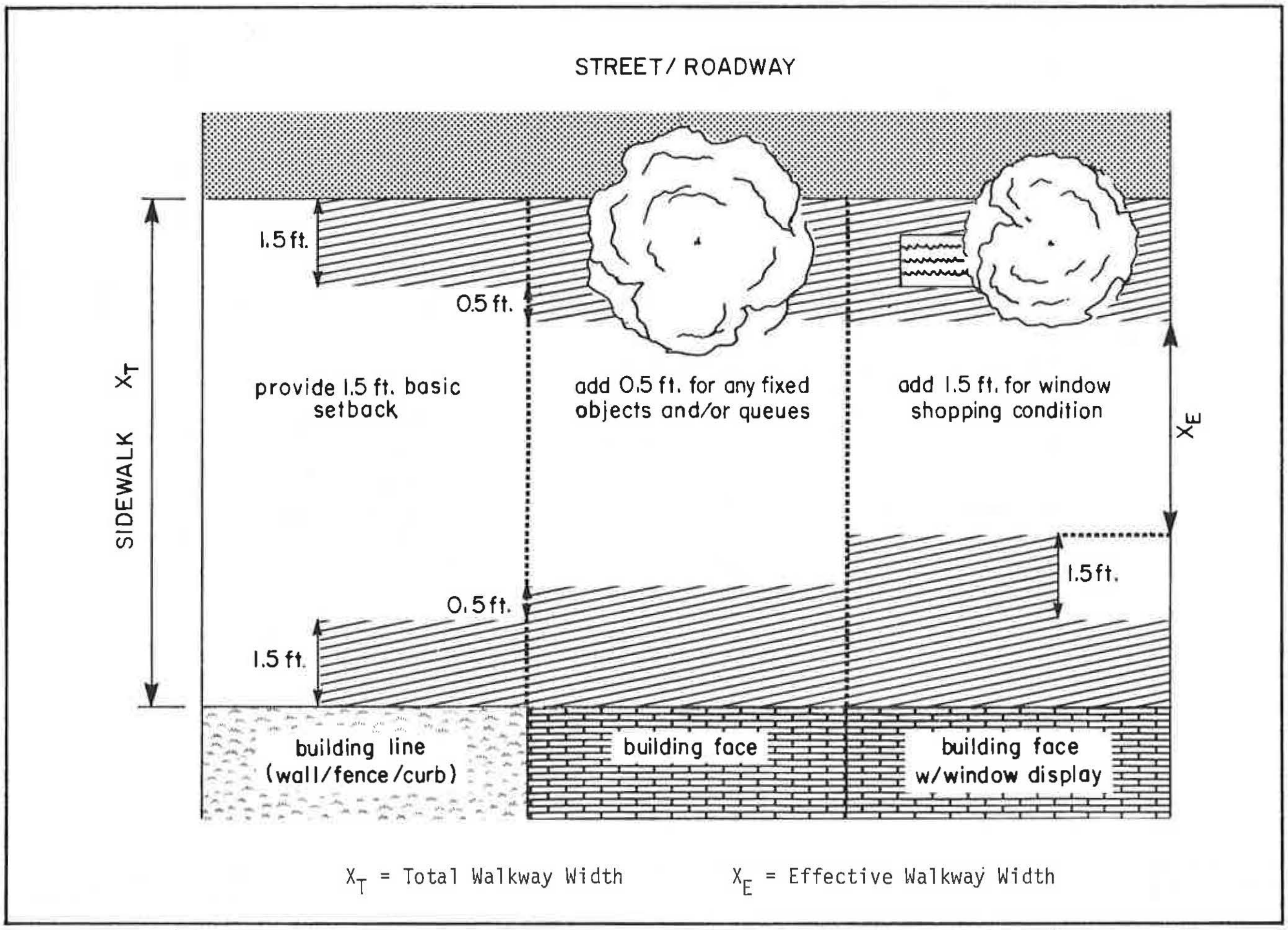
Source: Ref. (2)

(1 foot = .305 meter)

A list of typical obstructions and the estimated width of walkways that they pre-empt is provided in Table 1. Also, Figure 5 shows generally the width of walkways preempted by curbs, buildings, or fixed objects. Figure 5 may be used as a guideline when specific walkway configurations are not available.

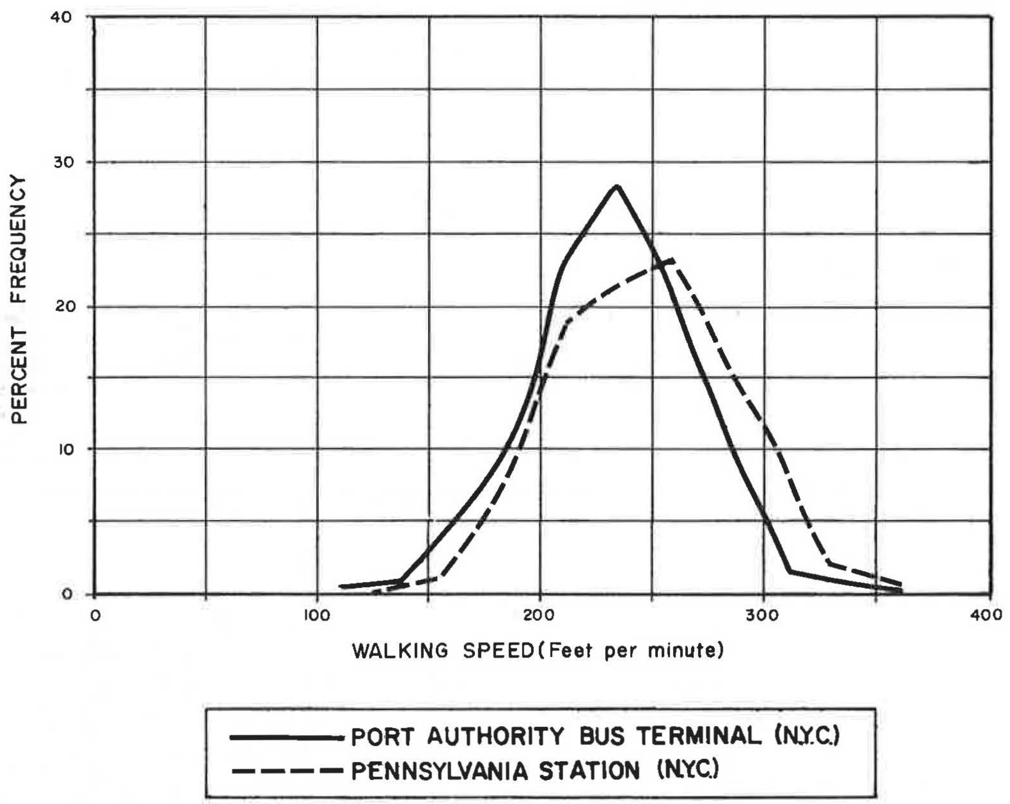
Thus, the "effective walkway width" represents that portion of the entire walkway that is reasonably available for use by the pedestrian stream moving through the area. Various width reduction adjustments, as indicated in Table 1 and Figure 5, are applied to the nominal walkway width to determine the effective width.

Figure 5. Buffer Space Width



Source: Ref. (4) (1 foot = .305 meter)

Figure 6. Free Flow Walking Speed Distribution



Source: Ref. (3) (1 foot = .305 meter)

Pedestrian Types and Trip Purposes

The analysis of pedestrian flow is generally based on mean walking speeds of groups of pedestrians. Within any group, or among groups, there can be considerable differences in flow characteristics due to trip purposes and age of pedestrian. Figure 6 indicates the typical distribution of free-flow pedestrian speeds.

Pedestrians going to and from work, using the same facilities day after day, may exhibit slightly higher walking speeds than shoppers. This has been shown in Figure 1. Older or very young persons will tend to walk at a slower gait than other groups. Shoppers not only may tend to walk more slowly than commuters, but they also can decrease the effective width of a walkway by stopping or slowing to window shop. Thus, in applying the techniques and numerical data in this section, the analyst or designer should be cognizant of pedestrian groups which have behavior that deviates widely from mean values.

Levels of Service in Walkways

Average Flow

The criteria for differentiation among the various levels of service for pedestrian flow are necessarily imprecise and the specification of demarcation points is somewhat subjective. However, it is possible to suggest appropriate ranges of space per pedestrian and of flow rates that can be used to develop quality of flow criteria.

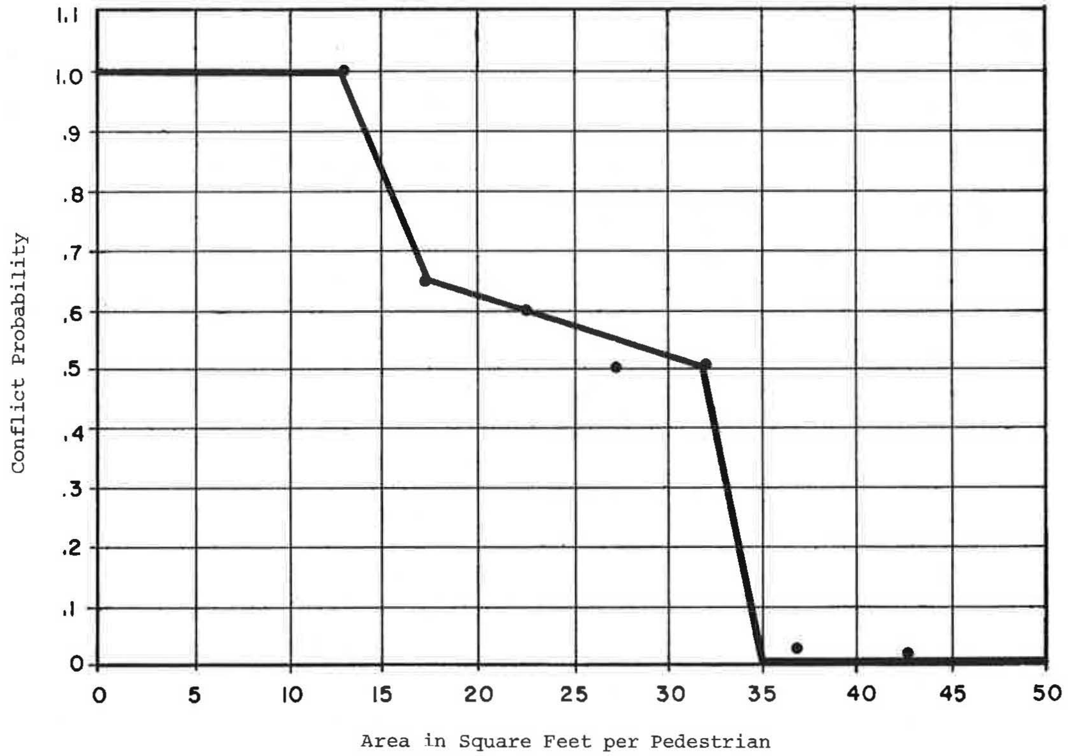
One important level of service criterion is speed. At speeds of 150 feet per minute (46 m/min) or less, a pedestrian must resort to a shuffling gait, which results in cramped unnatural movements. As Figure 4 has indicated at an average speed of 150 feet per minute (46 m/min) the corresponding space per person is in the range if 6 to 8 feet^2 (0.6 to 0.8 m^2). At space allocations of 15 square feet per person (1.4 m^2/ped) or less, even the slowest walkers who would ordinarily choose to walk at 150 feet per minute (46 m/min) are forced to slow down (shown by the cross-hatching in Figure 4). The fastest walkers cannot reach their chosen speed of 350 feet per minute (108 m/min) until space allocations are quite high. From Figure 2 it is evident that these three space allocations of about 7, 15, and 40 square feet (0.7, 1.4, and 3.8 m²) correspond approximately to the maximum flow rate at capacity, two-thirds of capacity, and one-third of capacity, respectively.

There are other significant indicators of service levels besides speed. For example, the ability of the pedestrian to freely choose his path across the path of other pedestrians is shown by Fruin (3) in Figure 7 to be impaired at space allocations below the 35 to 45 sq ft range (3.3 to 4.3 m^2). Above that level, Fruin states that the probability of "stopping or breaking of the normal walking pace" is reduced to zero. Below 15 sq ft per person (1.4 m^2/ped) virtually every crossing movement encounters a conflict. Similarly, the ability to pass slower-moving pedestrians is unimpaired above 35 square feet per person (3.3 m²/ped) but becomes progressively more difficult as space allocations drop to about 18 square feet per person (1.7 m²/ped). Below this space allocation, passing becomes virtually impossible without physical contact.

Another level of service indicator is the ability to maintain flow in the minor direction in opposition to the major flow direction. Here the quantitative evidence is somewhat less precise. For pedestrian streams of roughly equal volume distribution is each direction, there is little reduction in the capacity of the walkway compared with one-way flow because each direction occupies its proportionate share of the walkway width. However, if the bi-directional flow split is 90/10, for example, evidence suggests that for a space allocation of about 10 square feet per person (0.9 m²/ped), a capacity reduction of about 15 percent occurs. This reduction is a consequence of the inability of the minority movement to utilize its proportional share of the walkway width.

Impediments to pedestrian movement are not confined to space allocations below 40 square feet per person (3.8 m²/ped), however. Limited evidence suggests that for a higher space allocation of 60 square feet (5.7 m²/ped), it is observed that pedestrians walk in a "checkerboard" pattern rather than directly behind or alongside one another. This suggests that unconscious shifts in position occur for other pedestrians in response to the alteration of a given pedestrian. Observations also suggest that space allocations of up to 100 square feet per person (9.5 m²/ ped) are required to establish smooth movements when opposing pedestrians pass. Finally, informal observations suggest that at space allocations approaching 130 square feet per person (12.4 m²/ped) corresponding to the outer range of Hall's "public interpersonal distances" (5), the individual pedestrian is no longer influenced by other pedestrians. Examination of still higher space allocations suggests that the involuntary bunching of pedestrians in "platoons" does not completely disappear until about 500 square feet per person (47.5 m²/ ped) or higher.

Figure 7. Cross Flow Traffic: Probability of Conflict



Source: Ref. (3)

(1 foot = .305 meter)

Table 2. Comparative Level of Service Definitions for Walkways
(Not for Specific Application)

OEDING		FRUIN		PUSHKAREV	
Level of Service Label	Space (ft^2/ped)	Level of Service Label	Space (ft^2/ped)	Level of Service Label	Space (ft^2/ped)
				Open	over 530
	over 36	A	over 35	Unimpeded	130-530
				Impeded	40-130
Tolerable	18-36	B	25-35	Constrained	24-40
		C	15-25	Crowded	16-24
	11-18	D	10-15	Congested	11-16
	7-11	E	5-10	Jammed	2-11
Unacceptable	under 7	F	under 5		

Source: Ref. (2) (1 foot = .305 meter)

Level of Service Definitions

Three researchers have previously defined levels of service on the basis of the average space allocation, as summarized in Table 2. There is rather close agreement in selection of demarcation points among all three in the 10 to 40 square foot per person range (0.9 to 3.8 m^2/ped). Oeding and Fruin closely coincide throughout the entire range of space allocations, except that Fruin divides Oeding's "tolerable" level of service into two parts termed B and C. At the lowest levels of the pedestrian space spectrum, Pushkarev defines only one category, termed "jammed". At higher levels of the spectrum, above 40 square feet per person (3.8 m^2/ped), Pushkarev defines three distinct categories.

With such close agreement in the middle of the spectrum, the key issue to be resolved is how to deal with the higher and lower regions to arrive at one integral set of definitions of levels of service. Although the evidence presented by Pushkarev indicates two differentiations in quality of flow for space allocations above 130 square feet per person (12.4 m^2/ped), one level of service should be sufficient for analysis in most applications.

For the lower region of the spectrum, two separate levels of service for space allocations less than 10 square feet per person (0.9 m^2/ped) are in order, using the 6 square feet per person allocation (0.6 m^2/ped) as the dividing point. As indicated in Figure 2, one level of service with space allocation between 6 and 11 square feet per person (0.6 to 0.9 m^2/ped) would represent the capacity condition. The level of service would, therefore, represent lower space allocation, congested conditions, and flow rates below the capacity level.

Table 3 shows the adopted level of service standards. The alphabetic terminology is used to be consistent with that of vehicle flow. The primary criterion of the levels of service is space allocation; also, speed and flow rate (volume/capacity ratio) indicators can be used as supplemental criteria in special applications if deemed necessary by the analyst. The precise division points of the space allocation spectrum that was used by Pushkarev (2) are adopted in most instances in Table 3 to define the level of service standards because they correspond to reasonably progressive increments of flow rates and speeds.

Graphic illustrations and descriptions of walkway levels of service are shown in Figure 8.

Table 3. Pedestrian Levels of Service on Walkways: Based on Average Flow
(Recommended for Application)

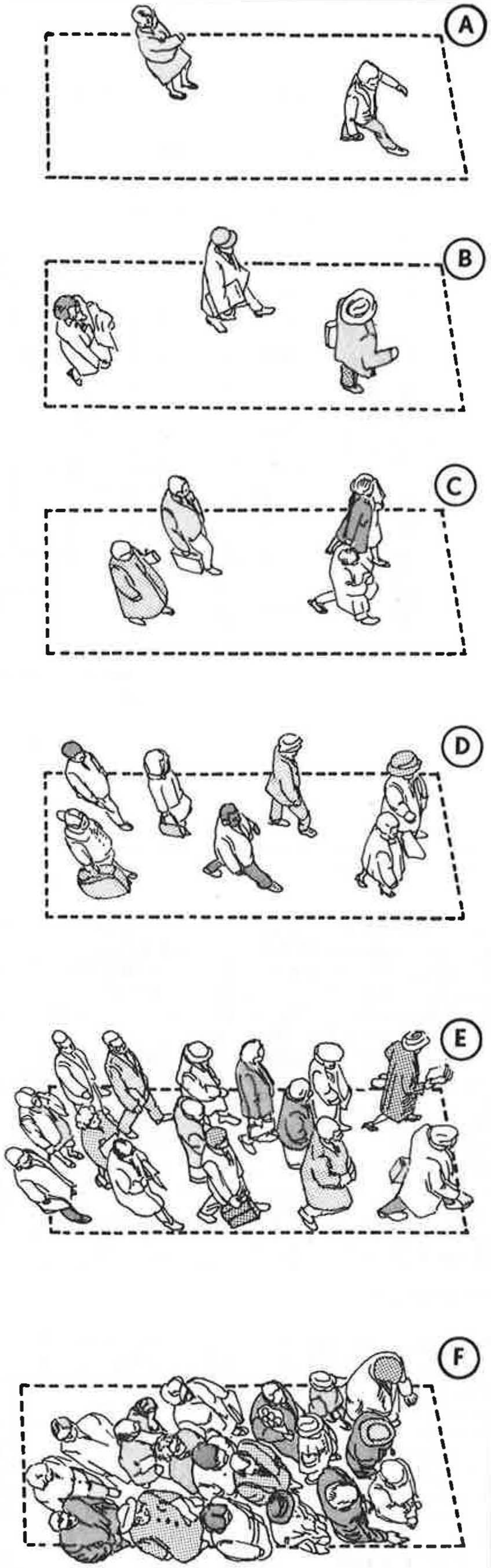
Level of Service	Space (ft^2/ped)	Average Flow Rate[a] (ped/min/ft)	Mean Speed[b] (ft/min)	Volume/Capacity[c] Ratio
A	over 40	under 6	over 250	< 0.24
B	24-40	10-6	240-250	0.24 - 0.40
C	16-24	14-10	224-240	0.40 - 0.56
D	11-16	18-14	198-224	0.56 - 0.72
E	6-11	25-18	150-198	0.72 - 1.00
F	under 6	0-25	0-150	0.00 - 1.00

[a] Flow Rate relative to effective walkway width

[b] Speeds are calculated based on Space and Flow Rate variables, using Equation (2)

[c] Assumed Capacity = 25 ped/min/ft (1 foot = .305 meter)

Figure 8. Levels of Service on Walkways



LEVEL OF SERVICE A

Average Pedestrian Space Allocation:
At least 40 square feet per pedestrian
Average Flow Rate:
6 pedestrians per minute per foot of effective walkway width

At walkway level of service A, sufficient area is provided for pedestrians to freely select their own walking speed, to bypass slower pedestrians, and to avoid crossing conflicts with others.

LEVEL OF SERVICE B

Average Pedestrian Space Allocation:
24 - 40 ft.^{2}/ped.
Average Flow Rate:
6 - 10 ped./min./ft. effective walkway width

At walkway level of service B, sufficient space is available to select normal walking speed, and to bypass other pedestrians in primarily one-directional flows. Where reverse-direction or pedestrian crossing movements exist, minor conflicts will occur, slightly lowering mean pedestrian speeds and potential volume.

LEVEL OF SERVICE C

Average Pedestrian Space Allocation:
16 - 24 ft.^{2}/ped.
Average Flow Rate:
10 - 14 ped./min./ft. effective walkway width

At walkway level of service C, freedom to select individual walking speed and freely pass other pedestrians is restricted. Where pedestrians cross movements reverse flows exist, there is a high probability of conflict requiring frequent adjustment of speed and direction to avoid contact. Designs consistent with this level of service would represent reasonably fluid flow; however, considerable friction and interaction between pedestrians is likely to occur, particularly in multi-directional flow situations.

LEVEL OF SERVICE D

Average Pedestrian Space Allocation:
11 - 16 ft.^{2}/ped.
Average Flow Rate:
14 - 18 ped./min./ft. effective walkway width

At walkway level of service D, the majority of persons would have their normal walking speeds restricted and reduced, due to difficulties in bypassing slower-moving pedestrians and avoiding conflicts. Pedestrians involved in reverse-flow and crossing movements would be severely restricted, with the occurrence of multiple conflicts with others. Designs at this level of service would be representative of the most crowded public areas, where it is necessary to continually alter walking stride and direction to maintain reasonable forward progress. At this level-of-service there is some probability of intermittently reaching critical density, causing momentary stoppages of flow.

LEVEL OF SERVICE E

Average Pedestrian Space Allocation:
6 - 11 ft.^{2}/ped.
Average Flow Rate:
18 - 25 ped./min./ft. effective walkway width

At walkway level of service E, virtually all pedestrians would have their normal walking speeds restricted, requiring frequent adjustments of gait. At the lower end of the range, forward progress would only be made by shuffling. Insufficient area would be available to bypass slower-moving pedestrians. Extreme difficulties would be experienced by pedestrians attempting reverse-flow and cross-flow movements. The design volume approaches the maximum attainable capacity of the walkway, with resulting frequent stoppages and interruptions of flow.

LEVEL OF SERVICE F

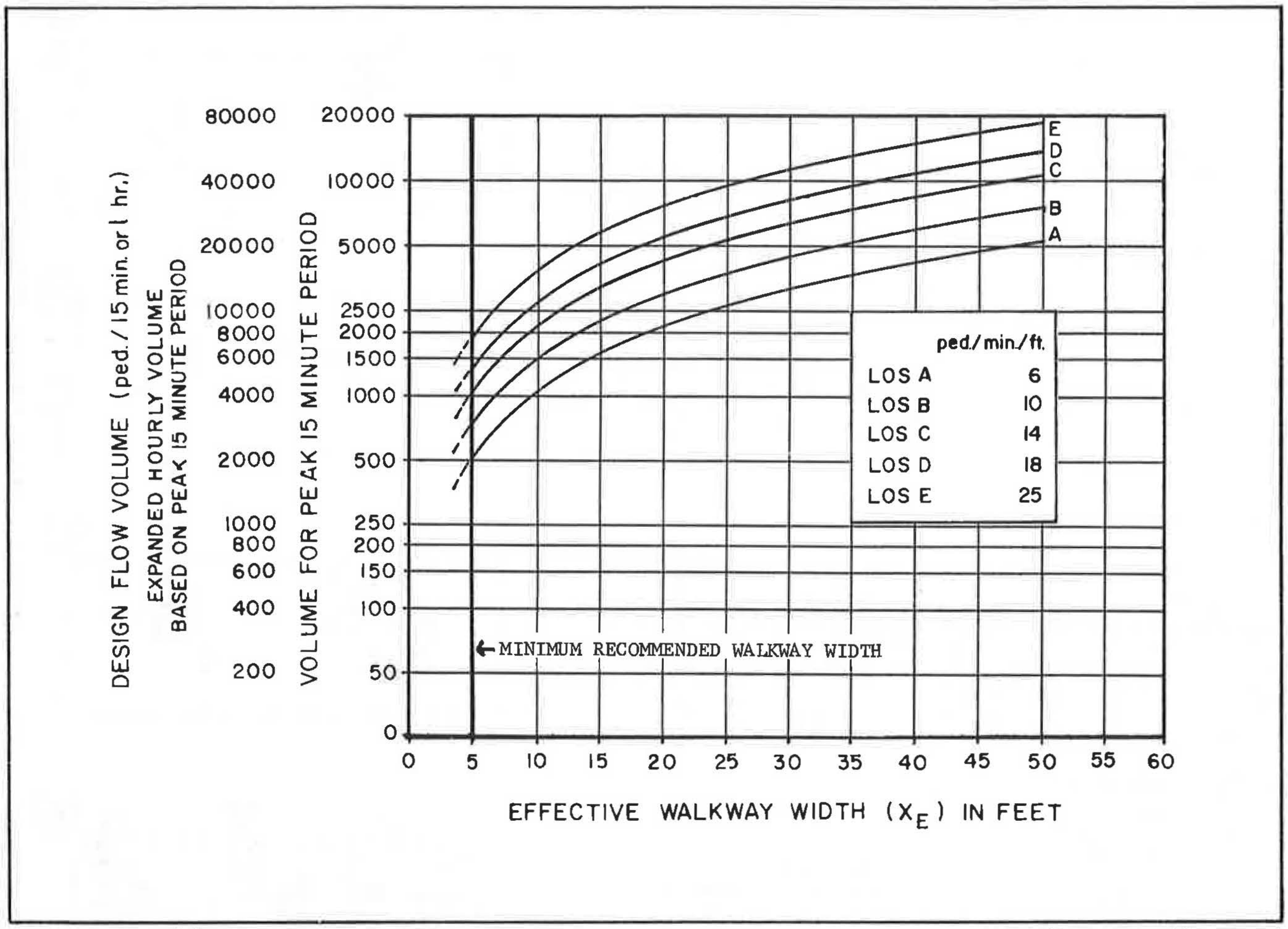
Average Pedestrian Space Allocation:
Less than 6 ft.^{2}/ped.
Average Flow Rate:
Variable, less than 25 ped./min./ft. effective walkway width

At walkway level of service F, all pedestrian walking speeds are extremely restricted, and forward progress can only be made by shuffling. There would be frequent, unavoidable contact with other pedestrians, and reverse or crossing movements would be virtually impossible. Traffic flow would be sporadic, with forward progress based on the movement of those in front. This level of service is representative of a loss of control, and a complete breakdown in traffic flow. Pedestrian areas below 5 square feet are more representative of queuing, rather than a traffic-flow situation, and this level of service is not recommended for walkway design.

Source: Adapted from Ref. (3)

(1 foot = .305 meter)

Figure 9. Effective Walkway Width - Design Considerations



Source: Ref. (4) (1 foot = .305 meter)

The level of service relationship can also be depicted graphically as in the monograph in Figure 9. Thus, if the analyst is given any two variables, such as effective walkway width and volume, the third variable, level of service, can be determined directly from this nomograph.

It must be emphasized that the level of service standards presented in Table 3 are based generally on the assumptions that pedestrians are uniformly distributed throughout the walkway segment and that flow is even, or homogenous in time. The flow rate in Table 3 is expressed in terms of a 1 minute interval and the uniform flow rates should not be extrapolated to longer periods of time without consideration of the special characteristics of platoons presented in the following section.

Platoon Flow

The definitions of the possible average flow rates at different levels of service (Table 3) will be of limited usefulness unless time intervals are specified over which these rates can reasonably be applied. That average flow rates may be misleading is clearly illustrated by the minute-by-minute variations shown in Figure 10. The data are for two locations in Lower Manhattan and are generally characteristic of many concentrated CBD locations in other cities. The maximum 15-minute flow rates average 1.4 and 1.9 pedestrians per minute per foot of effective walkway width (4.6 and 6.2 ped/min/m) during the two measurement periods. However, the diagrams indicate that flow during one minute can, on occasion, be more than 100 percent higher than the flow during the next minute, particularly when the overall volume is relatively low. Even during the peak 15 minute periods, incremental variations of 50 to 100 percent frequently occur from one minute to the next.

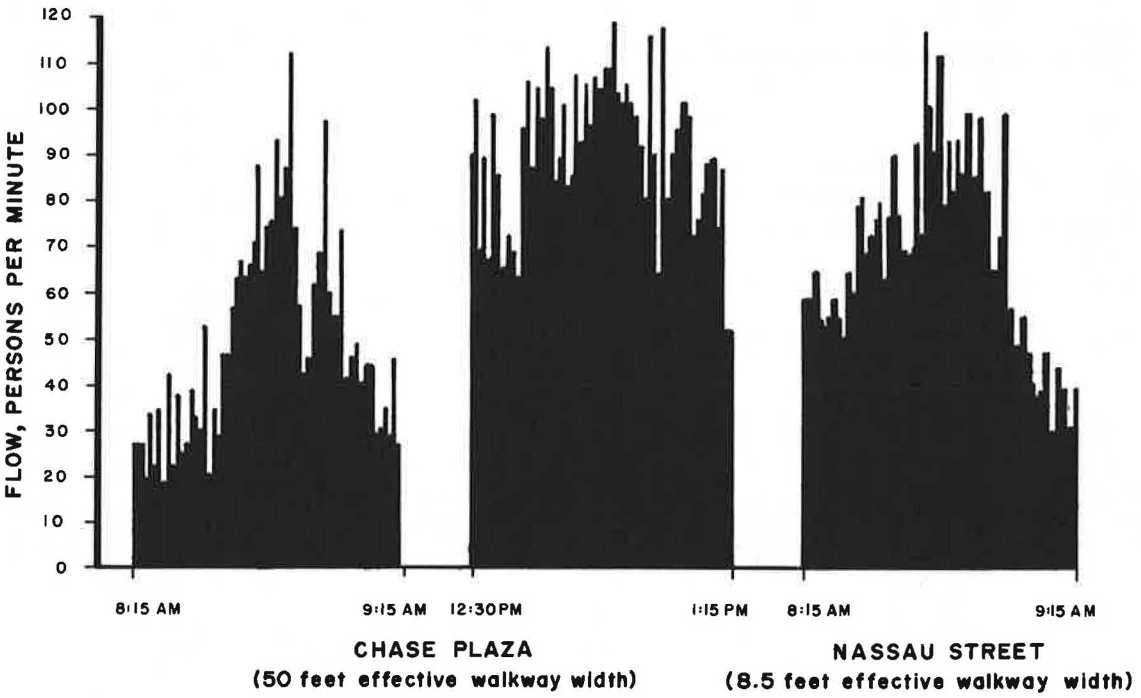
By comparing the scatter in the diagrams to the 15 minute average, it is evident that the highest 1 minute flow within each of the 15 minute periods exceeds the average by at least 20 percent in nearly all cases and by a maximum of 75 percent on occasion. The third highest 1 minute flow exceeds the average by 10 to 30 percent. Even the seventh highest minute can be as much as 20 percent higher than the 15 minute average. In general, 50 to 75 percent of all pedestrians are moving in streams having 1 minute flow rates significantly greater than the 15 minute average.

It is clear that any facility designed for the average flow in a 15 minute period will afford lower quality of flow for a sizable portion of the pedestrian traffic using it. However, it would be extravagant to design for peak 1 minute flows that may be 150 percent of the average, but which may only occur with a 1 or 2 percent probability. To resolve this dilemma, a relevant time period must be determined through closer investigation of the short-term fluctuation of pedestrian flow.

Short-term fluctuation is present in any unregulated pedestrian traffic flow because participants in the traffic stream arrive at a given spot at random. Thus, purely due to the laws of chance, in one minute a section of sidewalk may receive many pedestrians, whereas in the mext minute it may receive very few. In an urban situation, this random uneveness is exaggerated by three additional factors. The most important factor is the interruption of flow and queue formation caused by signalized intersections. In addition, transit facilities and, to a lesser extent, elevators create interruptions by the release of groups of people in very short intervals of time, with pauses during which no flow may occur at all. Until they have a chance to dissipate, pedestrians in these types of groups proceed more-or-less together as a platoon. Lastly, if passing is impeded because of insufficient space, faster pedestrians will slow down behind slower ones, and a bunching of pedestrians occurs. All of these involuntary groups of pedestrians are termed "platoons".

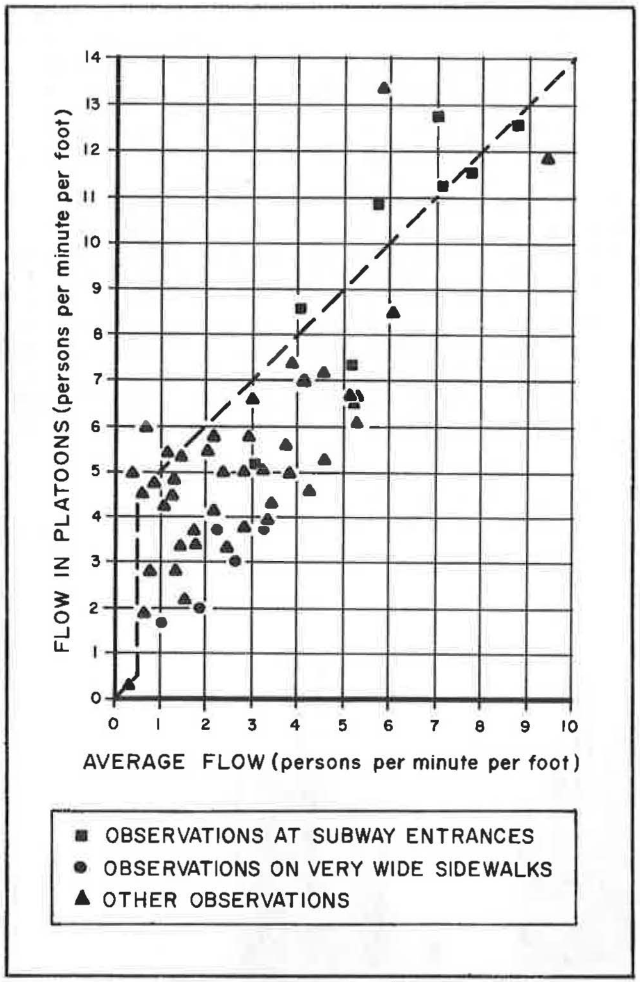
Platoons can be defined quantitatively using either of two observation procedures — termed "positive" or "negative". In the positive definition, platoons are timed and counted when it appears to the observer that a wave of above-average density and volume is swelling up in the traffic stream and passing the observation point on the walkway. In the negative definition, by contrast, relatively low volume gaps in flow are timed and the stragglers passing the observation point during these conditions are counted; then the non-platoon time and flow are subtracted from total time and slow to determine platoon characteristics. Judging from the experince of several researchers, it is recommended that the positive procedural definition be used in subsequent analysis. Platoons must be timed in seconds to avoid the arbitrary missing of periods of platoon flow with periods of non-platoon flow that can both occur during longer periods of time.

Figure 10. Minute-by-Minute Variations in Pedestrian Flow



Source: Ref. (2) (1 foot = .305 meter)

Figure 11. Relationship of Platoon Flow to Average Flow



Source: Ref. (2) (1 foot = .305 meter)

The scatter diagram in Figure 11 indicates the platoon flow rate (positive definition) in comparison with the average flow rate for 58 observation periods (mostly 5 to 6 minutes duration). The dashed line approximates the upper limit of platoon flow observations. The mathematical expression of this line relating platoon flow to average flow is:

PLATOON FLOW = AVERAGE FLOW + 4 (3)

where FLOW is expressed as ped/min/ft. The metric equivalent of this equation is as follows:

PLATOON FLOW = AVERAGE FLOW + 13.1 (3a)

where FLOW is expressed as ped/min/m.

The form that Equation 3 takes—a constant increment added to the average flow—indicates that platooning has a relatively greater impact at low volumes than at high volumes. A measure of this impact is the ratio of platoon flow to average flow, termed the "platoon factor." Figure 12 indicates the relation of the platoon factor to average flow. The platoon factor is relatively large in the low flow region, but it diminishes as the flow increases. For example, for an average flow of 2 persons per minute per foot (6.5 ped/min/m), the platoon flow is substantially larger, about 6 persons per minute per foot (19.5 ped/min/m)--corresponding to a platoon factor of 3.0. For a higher average flow condition of 10 persons per minute per foot (32.5 ped/min/m), the platoon factor is significantly lower, equaling 1.40. Since most pedestrian analysis will focus on the higher flow levels, platoon factors in the range of about 1.5 to 1.1 are of greater interest.

This pattern is not illogical, since gaps between platoons tend to fill up as flow increases. It does, however, point clearly to one possible design objective: there appears to be a need for minimum walkway standards that apply even at relatively low average flow rates, because there is always the probability of a sudden relatively large platoon. However, at low flow rates where wide walkways also exist, the data shown in Figure 11 suggest that the platoon flow rate is not likely to be greater than 2-4 pedestrians per minute per foot (6.6 to 13.1 ped/min/m) above the average flow rate. The upper flow limit is generally not experienced for low flow on wide walkways because platoons can more easily dissipate.

It is clear that an "average" flow rate, even if it refers to a period as short as one minute, may not be entirely relevant to defining the condition of the majority of pedestrians in a traffic stream who are in platoons. To the pedestrian within a platoon, it is small consolation that a few seconds prior to his arrival, the section of walkway on which he is now experiencing congested conditions was virtually empty. For example, if the objective is to provide a relatively high measure of mobility, then the time period truly relevant for design does not appear to be 15 minutes, 1 minute, or any other

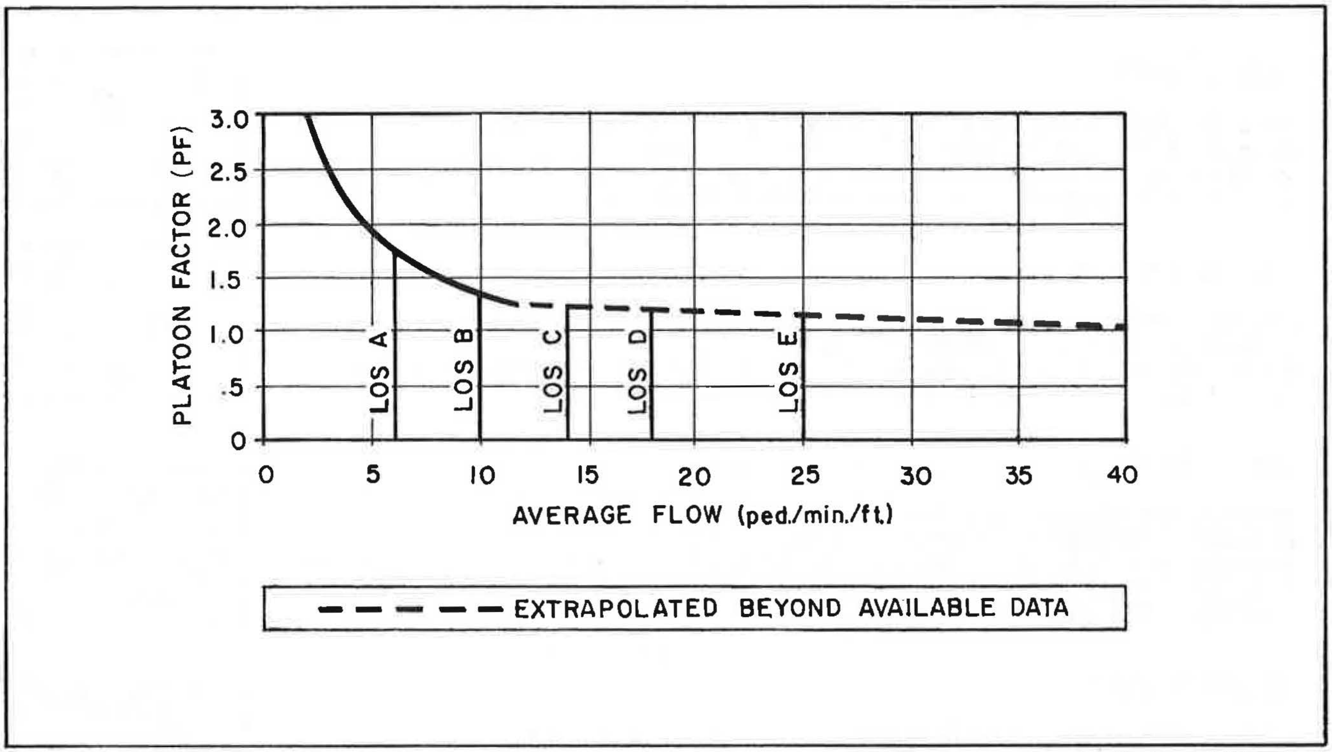
Table 4. Pedestrian Levels of Service on Walkways: Related to Platoon Flow

Average Flow Conditions			Platoon Flow Conditions		
Level of Service	Space, in ft.^{2}/ped.	Average Flow Rate, in ped./ft./min.	Level of Service	Space, in ft.^{2}/ped.	Platoon Flow Rate, in ped./ft./min.
A	130 +	< 2	A	40 + [a]	< 6
A	40 - 130	2 - 6	B	24 - 40	6 - 10
B	24 - 40	6 - 10	C	16 - 24	10 - 14
C	16 - 24	10 - 14	D	11 - 16	14 - 18
D	11 - 16	14 - 18	E	6 - 11	18 - 25
E	6 - 11	18 - 25	F	< 6	0 - 25
F	< 6	0 - 25	F	< 6	0 - 25

[a] On wide walkways, involuntary platoons occur infrequently at this low pedestrian flow level.

(1 foot = .305 meter)

Figure 12. Relationship of Platoon Factor to Average Flow



(1 foot = .305 meter)

arbitrary time span, but rather the intermittent periods during which flow in platoons occurs. Since this "time in platoons" is composed of short spans of variable length, the most convenient way to deal with it is to take a time interval that is appropriate from the viewpoint of longer cyclical variation, say 15 to 30 minutes, and then design not for the average, but for the platoon flow rate likely to occur during sub-intervals within that period.

The platoon factor corresponding to the average flow, indicated in Figure 12, can therefore be used (in appropriate circumstances) to calculate a new design volume for determination of geometric specifications (such as width) of a relatively high-design pedestrian facility. Alternatively, direct platoon measurements can be made at comparable facilities in a particular city to incorporate locational variations and other factors that cannot be predicted reliably from the New York City data used in Figure 12.

Design Criteria

To summarize this discussion of pedestrian flow effects, the correspondence between flow rates and space allocations for average flow and platoon flow is presented in Table 4. Platoon flow rates are calculated on the basis of Equation (3), and platoon space allocations are approximated from these flow rates using Figure 2.

Although the level of service standards are usually defined on the basis of average flow, as in Table 3, it is also possible to indicate representative levels of service for platoon flow based on the corresponding platoon space allocation, as shown in Table 4. Thus, the general rule-of-thumb derived from the available platoon data and the associated flow relationships is as follows:

- The level of service occurring in platoons is generally about one level of service lower than the level indicated based on the average flow criterion.

The selection of an appropriate design objectives—such as either to accomodate average flow or platoon flow at a target level of service—is determined by the analyst with considerations of various policy-oriented factors. For example, if the design objective is to provide a Level of Service C in platoon flows, space allocations must be in the range of 16 to 24 square feet per pedestrian in platoons (1.5 to 2.3 m²/ped). In this case, platoon flows of 10 to 14 pedestrians per minute per foot (33 to 46 ped/min/m) would be accomodated at the designated level of service. The corresponding average flow characteristics are space allocations of 24 to 40 square feet per pedestrian (2.3 to 3.7 m²/ped) and flow rates of 6 to 10 pedestrians per minute per foot (20 to 33 ped/min/m). Therefore, based on average flow conditions in Table 3, the designer would calculate an effective walkway width that provides Level of Service B for average flow which corresponds to Level of Service C for platoon flow. Also, the size of the urban area under investigation may determine if platoon considerations are significant in the pedestrian analysis. In large urban areas, pedestrian platooning needs to be incorporated in the analysis. However, in smaller cities (or in outlying areas of larger urban settings) the duration of platooning may be so insignificant that it need not be addressed in the analysis.

Additional examples of various design implications regarding average and platoon flow conditions are presented in detail under the heading of "USER APPLICATIONS" later in this section.

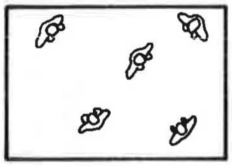
Levels of Service in Reservoir Areas

The location of the reservoir area is defined, typically, by the convergence of two perpendicular

Figure 13. Standing Levels of Service

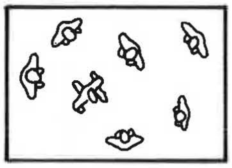
LEVEL OF SERVICE A

Average Pedestrian Area Occupancy: 13 sq. ft./person or more
Average Inter-person Spacing: 4 ft., or more
Description: standing and free circulation through the queuing area is possible without disturbing others within the queue.



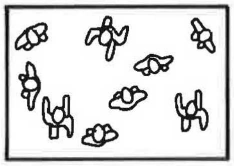
LEVEL OF SERVICE B

Average Pedestrian Area Occupancy: 10-13 sq. ft./person
Average Inter-person Spacing: 3.5-4.0 ft.
Description: standing and partially restricted circulation to avoid disturbing others within the queue is possible.



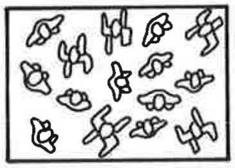
LEVEL OF SERVICE C

Average Pedestrian Area Occupancy: 7-10 sq. ft./person
Average Inter-person Spacing: 3.0-3.5 ft.
Description: standing and restricted circulation through the queuing area by disturbing others within the queue is possible, this density is within the range of personal comfort.



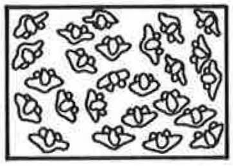
LEVEL OF SERVICE D

Average Pedestrian Area Occupancy: 3-7 sq. ft./person
Average Inter-person Spacing: 2-3 ft.
Description: standing without touching is possible; circulation is severely restricted within the queue and forward movement is only possible as a group; long term waiting at this density is discomforting.



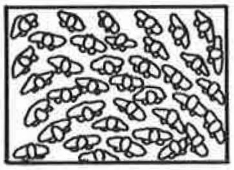
LEVEL OF SERVICE E

Average Pedestrian Area Occupancy: 2-3 sq. ft./person
Average Inter-person Spacing: 2 ft. or less
Description: standing in physical contact with others is unavoidable; circulation within the queue is not possible; queuing at this density can only be sustained for a short period without serious discomfort.



LEVEL OF SERVICE F

Average Pedestrian Area Occupancy: 2 sq. ft./person or less
Average Inter-person Spacing: close contract with persons
Description: virtually all persons within the queue are standing in direct physical contact with those surrounding them; this density is extremely discomforting; no movement is possible within the queue; the potential for panic exists in large crowds at this density.



Source: Ref. (3) (1 foot = .305 meter)

sidewalks at a street corner adjacent to the general traffic intersection area. This type of pedestrian facility generally experiences the most concentrated activities within the pedestrian network, characterized by varied interactions between several coincident pedestrian streams which may either be in moving or queueing states at any given instant. In comparison with the preceding analytical framework for sidewalk facilities, there are inherent complexities of pedestrian operations at intersections that have limited the development of analytical procedures.

The general analytical framework for calculating capacity and level of service for reservoir areas requires specification of the following input factors:

- Pedestrian multi-directional flow levels and interactions
- Facility design features
- Signal operation characteristics
- Level of service standards and criteria

There is only limited empirical data which describes pedestrian operations at intersection areas. Consequently, the underlying relationships among these input factors have not yet been studied rigorously, and the important concepts regarding mobility measures, levels of service, and capacity have not been developed fully.

Generally, two or more pedestrian streams simultaneously are utilizing a reservoir space at an intersection, and two distinct types of area requirements occur:

> Circulation Area is necessary to accommodate multi-directional pedestrian flows not interrupted by the signal cycle phase.
> Holding Area is necessary to accommondate the build-up (or queueing) of those pedestrians waiting for the traffic signal to change in favor of their desired crossing.

Both the holding and circulation functions of the reservoir area can be evaluated based on level of service standards "for queueing conditions." Figure 13 describes the standards and attendant operational characteristics for queueing facilities. The reservoir area is not utilized strictly for queueing, since substantial multi-directional stream flows also occur through the area. However, the queueing facility level of service standards do provide a useful analogy and a basis for analysis. The analyst can modify the approach and evaluation parameters used in queueing analysis, as necessary, to conform with the mixed functions of the reservoir area.

Specific analytical procedures and examples of crosswalk evaluation are presented in detail under the "USER APPLICATIONS" heading later in this section.

Levels of Service in Crosswalks

The pedestrian flow characteristics in crosswalks are similar in nature to the operations on sidewalks. Thus, the basic uninterrupted flow relationships between speed, density, and volume for crosswalk flow are consistent with sidewalk flow. As the density of pedestrians in the crosswalk increases beyond the low-flow levels, the speed declines and volume increases to the capacity condition. For densities greater than the capacity level, speed continues to decrease, and volume also decreases.

There are certain characteristic features that differentiate crosswalk operations from sidewalk flow. One principal dissimilarity is the degree of interrupted flow. Because of signal phasing conditions, the incidence of platoon flows and their interactions in the crosswalk is somewhat greater, generally, than what occurs in the typical mid-block area of a sidewalk. Also, the desired (free-flow) speeds of pedestrians in the crosswalk tend to be higher and more uniformly distributed than for sidewalks, although for more congested conditions queue discharge and platoon flow operations will significantly restrain the prevailing speeds.

The level of service concept that was developed primarily for uninterrupted flow of pedestrians on sidewalks can be utilized to evaluate crosswalk operations, as well. Since substantial dissimilarities are apparent, however, the level of service standards should only be considered as relatively coarse descriptions of the quality of service in crosswalk analysis.

Specific analytical procedures and examples of reservoir area evaluations are presented in detail under the following heading.

USER APPLICATIONS

Methodology

The purpose of this section is to present a number of specific techniques and a detailed procedural framework for user-oriented analysis of capacity and level of service on pedestrian facilities. Typical example problems are also included, and the step-by-step solution process is elaborated in each case to illustrate various types of applications.

These specific applications procedures were developed from the basic principles of pedestrian flow theory and operational experience which were analyzed in the preceding "DISCUSSION."

The scope of the pedestrian analysis is limited specifically to pedestrian areas and facilities that are generally located within the street right-of-way boundaries. Three general types of facilities are addressed:

1. Sidewalk - typically located adjacent to the street; generally involved in a "mid-block" walkway analysis of interrupted flow.

2. Reservoir Area - typically located on the sidewalk at both ends of the crosswalk; generally involved in an "intersection" analysis of queueing conditions.

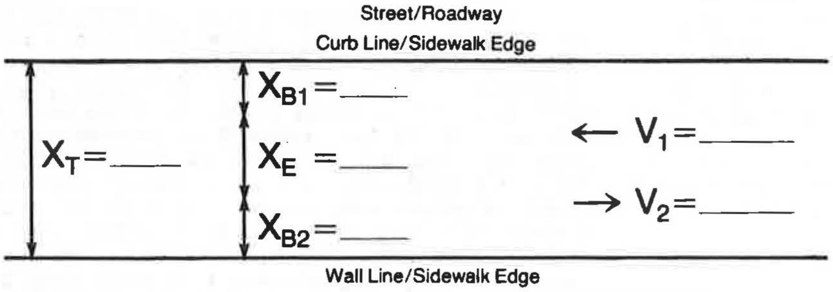
3. Crosswalk - typically located across a street; generally involved in an "intersection" analysis of interrupted flow.

The analytical procedures presented in this section have been developed to enable the calculation of capacity and levels of service for these three kinds of pedestrian facilities. The procedures are inherently flexible in nature and can be used at either a general or a detailed level to achieve a wide variety of specific applications objectives.

Pedestrians–Midblock Walkway Analysis
Calculation Form 1

Location ______________________________

Location Plan:



Counts:

Date ____________

Day ____________

Time ____________

☐ One Way ☐ Two Way

Peak 15 minute Pedestrian Count
from _:__ to _:__ __.m.

- **Pedestrian Volume**

  Pedestrians/15 minutes (V_D)

  $V_D = V_1 + V_2$

  =_____ + _____

  =_____ peds./15 min.

- **Walkway Width Analysis**

  Total Sidewalk Width (X_T)

  Buffer Space/Dead Space (X_B)

  (See Table 1 and Figure 5)

  Effective Width (X_E)

  If X_E is to be determined, use Figure 9

  X_T=_____ ft.

  $X_B = X_{B1} + X_{B2}$

  =_____ ft.

  $X_E = X_T - X_B$

  =_____ ft.

- **Walkway Level of Service**

  Level of Service (LOS) from Figure 9

  Pedestrian Unit Flow Rate (F) for LOS

  LOS = ☐

  $F = V_D \div 15X_E$

  =_____ ped./ft./min.

- **Platoon Analysis**

  (analysis to be used if platooning is anticipated or observed during a significant portion of the time)

  Platoon Factor (PF) from Figure 12

  Platoon Volume (V_P)

  Level of Service (LOS) from Figure 9

  PF =_____

  $V_P = V_D \times PF$

  =_____ peds./15 min.

  LOS = ☐

(1 foot = .305 meter)

Midblock Walkway Analysis

Calculations are based upon maximum 15 minute pedestrian volumes utilizing the midblock walkway. A midblock walkway may have to be tested for different time periods during the day to account for varying directional flows. Pedestrian volumes are obtained from manual counts to analyze present conditions. For new locations or to analyze future conditions, forecasts of the flows must be made.

The methodology requires a specific sequence which is presented below (use Calculation Form 1):

1. The preliminary steps include the gathering of basic data--such as peak fifteen minute pedestrian volumes (V_D), total sidewalk width (X_T) and identification of obstacles in the walkway.

2. The effective width of walkway must be determined. The effective walkway width (X_E) is not the total walkway width (X_T), but rather that portion of the sidewalk section actually available for pedestrian travel, i.e., free of any physical obstruction or impedences.

3. Thus, from Figure 9, knowing any two variable, the third can be determined. The variables are Pedestrian Volume (V_D), Level of Service (LOS), and Effective Width (X_E). The following may be determined:

   a. knowing effective width (X_E) and pedestrian volume (V_D), the level of service (LOS) can be determined;

   b. knowing the effective width (X_E) of walkway and its stated level of service (LOS), the 15 minute pedestrian service volume (V_D) can be found for the stated (LOS) ; and

   c. lastly, knowing the 15 minute pedestrian volume (V_D) and level of service (LOS), the effective walkway width (X_E) can be determined.

4. Buffer width or dead space (X_B) requirements must be subtracted from the total sidewalk width (X_T) to determine the effective walkway width (X_E), or added to the effective width (X_E) to determine total sidewalk width (X_T). Buffer or dead space widths are indicated in Figure 5 and Table 1. Figure 5 shows the general buffer width requirements when specific obstacle locations are not known. Table 1 provides specific information about width requirements for known obstacles.

The above methodology only considered average pedestrian flow conditions and does not take into account the surging of pedestrian flows (platooning). If platooning is anticipated or observed, special analysis should be used. Figure 12 provides a platoon factor (PF) for a specified level of service.

Example 1

A given sidewalk segment on Third Street of the pedestrian network plan has a counted 15 minute peak flow demand volume of 1250 pedestrians. A 14 foot sidewalk section has a curb on one side and stores with window shopping displays on the other side. No other sidewalk impediments exist within the sidewalk section.

Problem

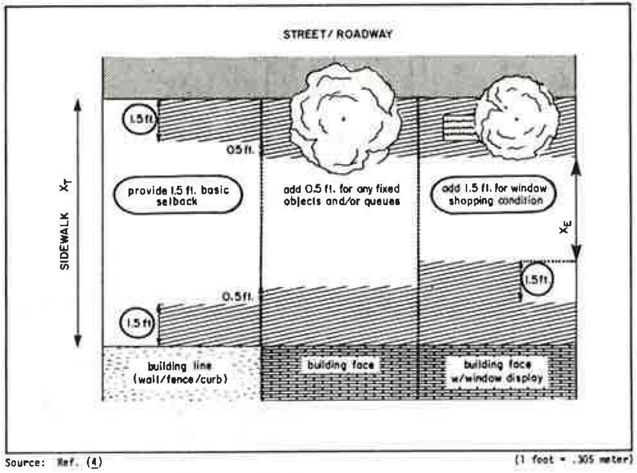
Determine the Level of Service (LOS) at which the sidewalk operates.

Analysis

For midblock walkway analysis, Calculation Form 1 is used. A location plan of the walkway showing pertinent features is completed and volume data are obtained. In this case the count was made on Thursday, May 31, from 4 to 6 p.m.. The peak 15 minute volume occurred during the interval from 4:45 to 5:00. This peak 15 minute volume (V_D) serted into Calculation Form 1.

The total width (X_T) of 14 feet is entered into the location plan. The buffer space is determined from Figure 5. A curb requires a buffer width of 1.5 feet (X_{B1}) and window displays require a buffer width of 3.0 feet (X_{B2}). After combination, total buffer width (X_B) is 4.5 feet. Thus the effective width (X_E) is: 14 - 4.5 = 9.5 feet.

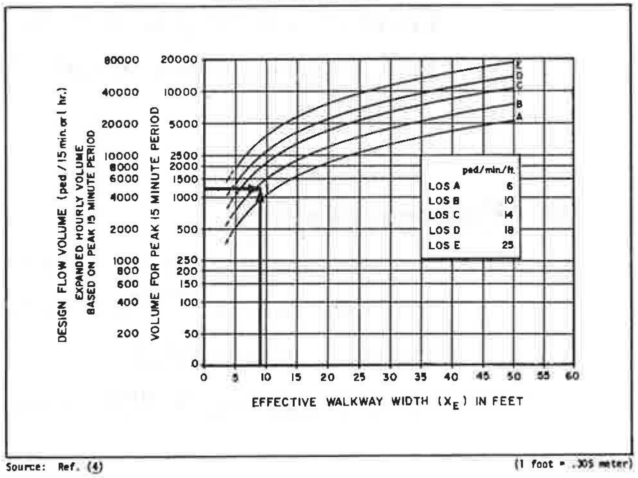
Figure 5. Buffer Space Width Requirements



Source: Ref. (4) (1 foot = .305 meter)

The values for pedestrian volumes (V_D) and effective width (X_E) are used to enter Figure 9 to determine the level of service (LOS). Figure 9 shows that LOS = B, and that the corresponding pedestrian unit flow rate (F) = 8.8 ped/min/ft.

Figure 9. Effective Walking Width-Design Considerations



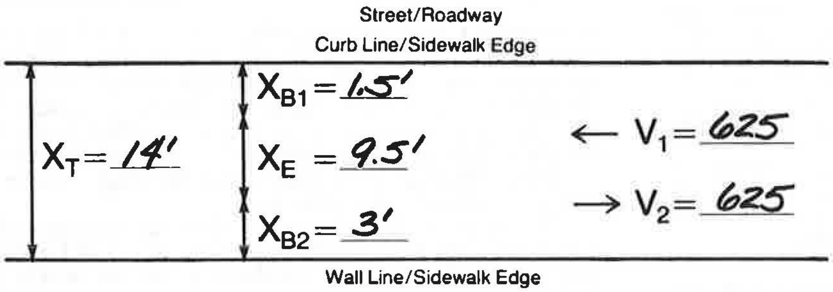
Source: Ref. (4) (1 foot = .305 meter)

Pedestrians–Midblock Walkway Analysis
Calculation Form 1

Example 1

Location THIRD STREET (EAST SIDE) AT #4573

Location Plan:



$X_T =$ 14'

$X_{B1} =$ 1.5'

$X_E =$ 9.5'

$X_{B2} =$ 3'

← $V_1 =$ 625

→ $V_2 =$ 625

Counts:

Date MAY 31, 1979

Day THURSDAY

Time 4-6 P.M.

☐ One Way ☐ Two Way

Peak 15 minute Pedestrian Count
from 4:45 to 5:00 P.m.

- **Pedestrian Volume**

  Pedestrians/15 minutes (V_D)

  $V_D = V_1 + V_2$

  = 625 + 625

  = 1250 peds./15 min.

- **Walkway Width Analysis**

  Total Sidewalk Width (X_T)

  $X_T =$ 14 ft.

  Buffer Space/Dead Space (X_B)
  (See Table 1 and Figure 5)

  $X_B = X_{B1} + X_{B2}$

  = 4.5 ft.

  Effective Width (X_E)
  If X_E is to be determined, use Figure 9

  $X_E = X_T - X_B$

  = 9.5 ft.

- **Walkway Level of Service**

  Level of Service (LOS) from Figure 9

  LOS = $\boxed{B}$

  Pedestrian Unit Flow Rate (F) for LOS

  $F = V_D \div 15X_E$

  = 8.8 ped./ft./min.

- **Platoon Analysis**

  (Optional analysis to be used if platooning is anticipated or observed during a significant portion of the time)

  Platoon Factor (PF) from Figure 12

  PF = 1.45

  Platoon Volume (V_P)

  $V_P = V_D \times PF$

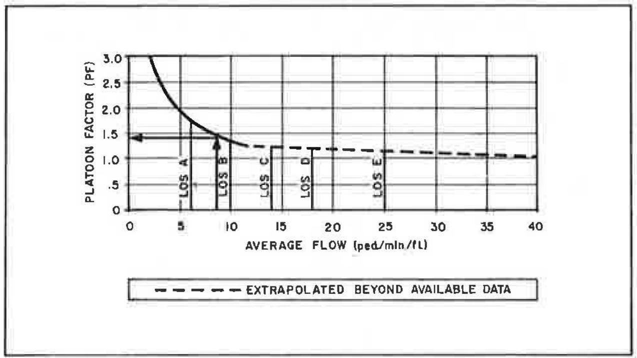
  = 1813 peds./15 min.

  Level of Service (LOS) from Figure 9

  LOS = $\boxed{C}$

(1 foot = .305 meter)

Figure 12. Platoon Flow-Average Flow Relationship



(1 foot = .305 meter)

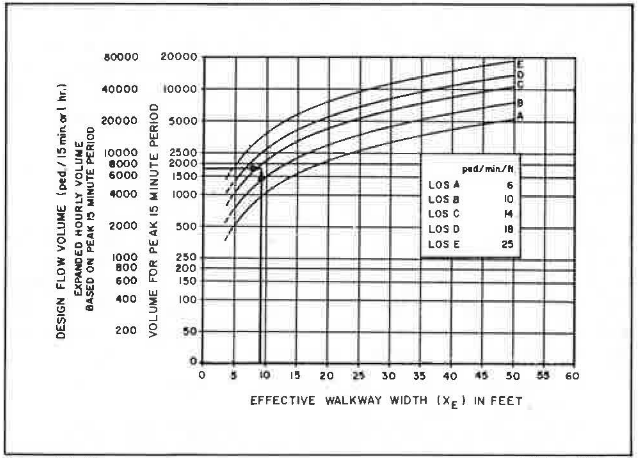
To determine platoon volume (V_P) enter Figure 12 with pedestrian unit flow of 8.8 ped/min/ft and determine the platoon factor (PF) of 1.45. The platoon volume and flow rate can then be determined: (V_P) = 1250 X 1.45 = 1813 ped/15 min. Entering Figure 9 with the effective width (X_E) and platoon volume (V_P) the level of service is determined: **LOS = C.**

The complete series of calculations is indicated on completed Calculation Form 1.

Walkway Reservoir Area Requirements

Generally, the most concentrated area of pedestrian activity within the downtown walkway network occurs at signalized street intersections. At these intersection areas, pedestrian flows along two sidewalk corridors intersect each other and one of these flows is interrupted by the traffic signal phasing which regulates street crossing. Since these areas have higher concentrations of pedestrians and cross traffic, they are the least desirable place for sidewalk impediments that could further constrict traffic flow.

Figure 9. Effective Walking Width-Design Considerations



Source: Ref. (4)

(1 foot = .305 meter)

Area requirements for the walkway system at signalized street crossing areas are of two types: (1) Circulation Area and (2) Holding Area. Circulation Area is necessary to accommodate traffic flow not interrupted by the signal cycle phase, while Holding Area is necessary to accommodate the maximum build-up of those pedestrians waiting for the traffic signal to change in favor of their desired crossing. See Figures 14 and 15 for an illustration of the above concepts.

Figure 14. Area Required for Condition 1 Movement Vectors

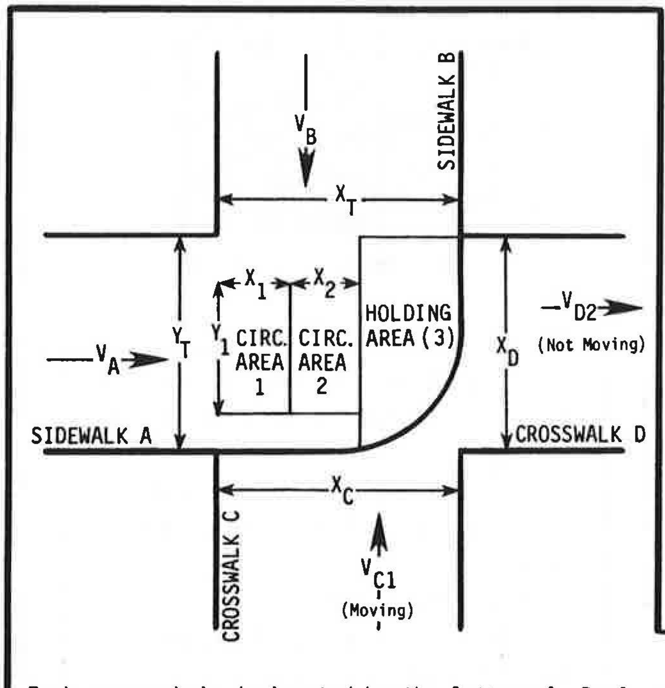
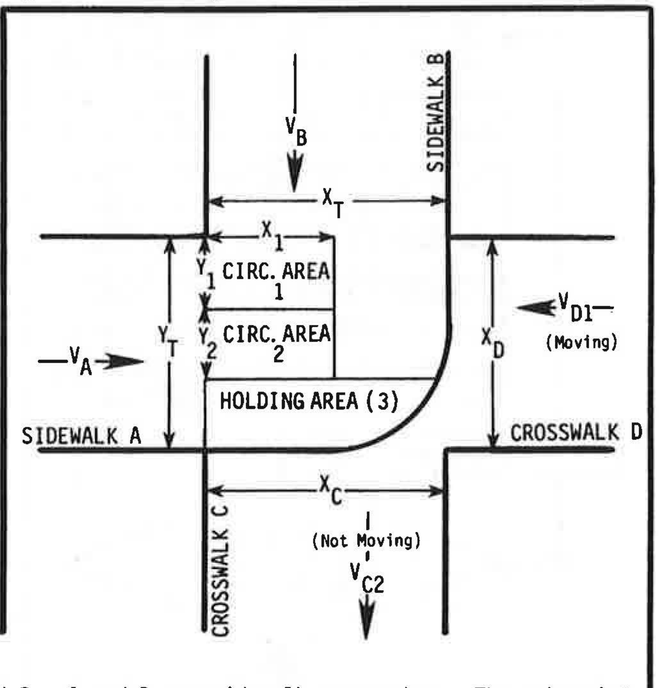


Figure 15. Area Required for Condition 2 Movement Vectors



Each approach is designated by the letters A, B, C, and D. A and B are sidewalk approaches. The subscript of the volume vectors (V) identifies the movement on each approach. The designation 1 in a subscript indicates pedestrians walking toward the intersection, and the designation 2 indicates pedestrians leaving the intersection. Total Signal Cycle Length (TS), Curb Radius (r), Cross Time (CT), and Queue Time (QT) for each signal phase must be known. All volumes are for 15 minute peaks only.

The methodology employed to calculate circulation area requirements at intersections entails the summation of area requirements for each set of incoming pedestrian volume vectors that intersect each other perpendicularly during a particular light phase. The circulation area must be large enough to avoid a high probability of conflict with crossing streams of pedestrians (Table 5).

Holding area requirements at intersections are determined by applying a queueing requirement (based on a given level of service) to the projected peak build-up of pedestrians waiting at the intersection for the signal phase to change in favor of their desired street crossing. The minimum recommended space module (Q) for queueing in holding areas is 5 ft.^{2}/person (Queueing Level of Service D, as indicated in Figure 13).

Total area required at the sidewalk intersection area is the sum of the required circulation area and holding area at the intersection. However, the area requirements at a sidewalk intersection will vary depending upon the particular phase of the signal cycle. Therefore, calculations are necessary for each phase of the cycle to determine which phase requires the maximum area, i.e., the most space-consuming phase of the total signal cycle.

An overview of the procedure for determining walkway reservoir requirements at signalized intersections is shown in Figure 16. Once the total area required at the sidewalk intersection is determined, it must be compared against the available area at the intersection (Figure 17). Use Calculation Form 2 for this type of reservoir area analysis.

Figure 16. Procedure for Determining Intersection Reservoir Area Requirements

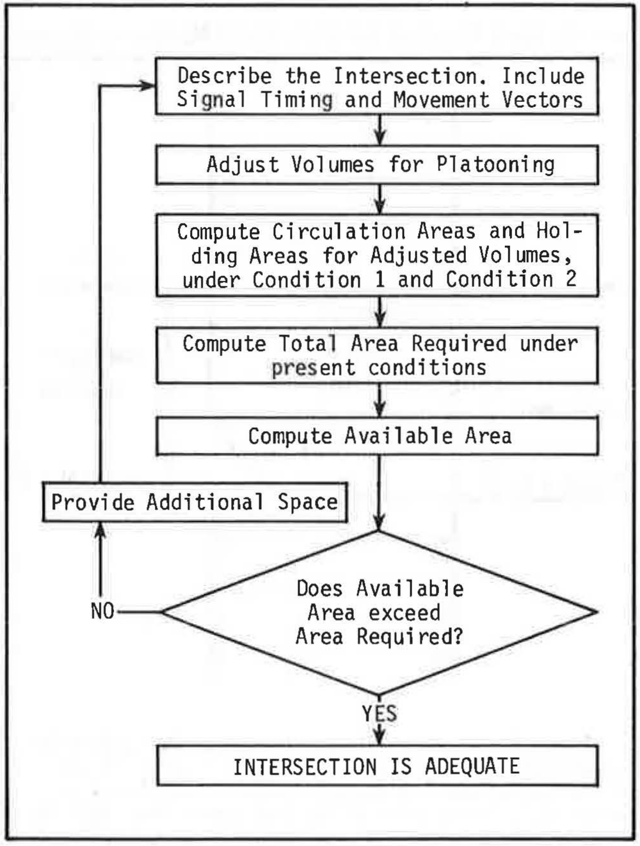


Table 5. Pedestrian Space Requirements for Cross Flow Traffic and Probability of Conflict

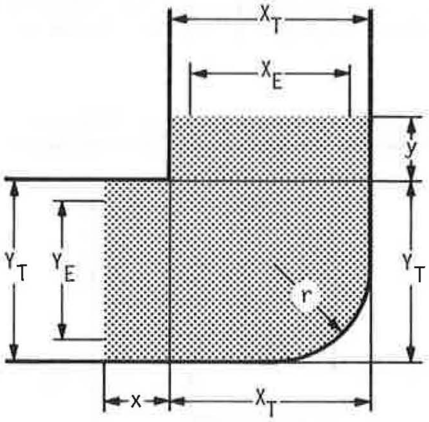
Level of Service	Space (ft^2/ped)	Conflict Probability
A	over 35	0.1 or less
B	24-35	0.5-0.1
C	18-24	0.75-0.5
D	13-18	0.90-0.75
E	6-13	0.98-0.90
F	under 6	0.98 or more

Source: Ref. (3) (1 foot = .305 meter)

Crosswalk Analysis

The determination of crosswalk capacity and level of service is somewhat similar to the midblock walkway analysis. Knowing the 2-way peak 15 minute pedestrian volume using the crosswalk and adjusting this volume for platooning, one utilizes Figure 9 to determine the necessary crosswalk width for a stipulated level of service. Calculation Form 3 is used for this type of crosswalk analysis.

Figure 17. Intersection Reservoir Area Design Alternatives



X_T = Total Walkway X width, in feet
X_E = Effective Walkway X width, in feet
$x = X_T \div 3$
Y_T = Total Walkway Y width, in feet
Y_E = Effective Walkway Y width, in feet
$y = Y_T \div 3$
r = Curb radius, in feet
= Available Area, A_A, in square feet
$A_A = 1.67(X_T)(Y_T) - 0.215r^2$

Source: Adapted from Ref. (4) (1 foot = .305 meter)

Pedestrians–Intersection Reservoir Area Analysis
Calculation Form 2

Intersection ______________________________

Corner:
NW ☐ NE ☐
SW ☐ SE ☐

Total Signal Cycle (TS) = _____ sec.
Curb Radius (r) = _____ ft.

Counts:
Date __________
Day __________
Time __________

Signal Phasing	Area Required for Condition 1	Area Required for Condition 2

Signal Phasing Condition 1

Condition 2

Figure 14

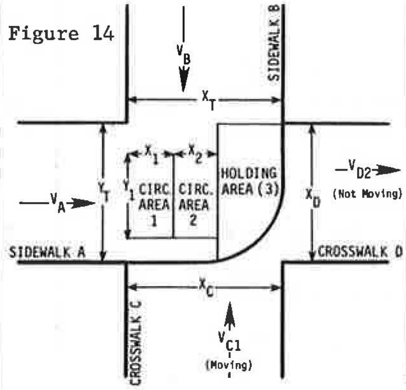
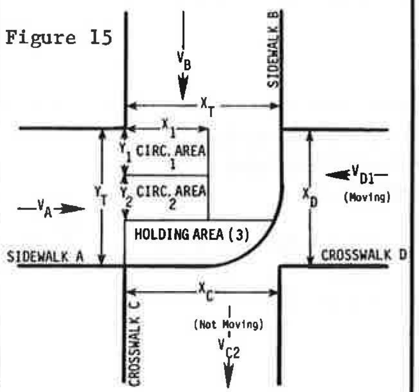


Figure 15



Each approach is designated by the letters A, B, C, and D. A and B are sidewalk approaches. The subscript of the volume vectors (V) identifies the movement on each approach. The designation 1 in a subscript indicates pedestrians walking toward the intersection, and the designation 2 indicates pedestrians leaving the intersection. Total Signal Cycle Length (TS), Curb Radius (r), Cross Time (CT), and Queue Time (QT) for each signal phase must be known. All volumes are for 15 minute peaks only.

Movements	V_A	V_B	V_{C1}	V_{D2}	V_A	V_B	V_{C2}	V_{D1}
Pedestrian Volumes								
Cross Time	▨	▨	$CT_1=$	▨	▨	▨	▨	$CT_2=$
Queue Time	▨	▨	▨	$QT_1=$	▨	▨	$QT_2=$	▨

Step	Condition 1	Condition 2
1. Compute circulation areas for each condition. 1a. Adjust volumes V_A and V_B for platooning (see Figure 12)	$V_{A(p)} = V_A \times PF$ $= \_\_\_\_ \times \_\_\_\_$ $= \_\_\_\_\_\_$ peds./15 min.	$V_{B(p)} = V_B \times PF$ $= \_\_\_\_ \times \_\_\_\_$ $= \_\_\_\_\_\_$ peds./15 min.
1b. Adjust volumes V_{C1} and V_{D1} for platooning (see Figure 12)	$V_{C1(p)} = V_{C1} \quad [TS \div (CT_1 - 3)]$ $= \_\_\_ \; [\_\_\_ \div (\_\_\_ - 3)]$ $= \_\_\_\_\_\_$ peds./15 min	$V_{D1(p)} = V_{D1} \quad [TS \div (CT_2 - 3)]$ $= \_\_\_ \; [\_\_\_ \div (\_\_\_ - 3)]$ $= \_\_\_\_\_\_$ peds./15 min.
1c. Using volumes derived from Steps 1a and 1b, determine effective widths of circulation areas using Figure 9.	$Y_1 = \_\_\_\_$ ft. use $V_{A(p)}$ $X_1 = \_\_\_\_$ ft. use $V_{B(p)}$ $X_2 = \_\_\_\_$ ft. use $V_{C1(p)}$	$X_1 = \_\_\_\_$ ft. use $V_{B(p)}$ $Y_1 = \_\_\_\_$ ft. use $V_{A(p)}$ $Y_2 = \_\_\_\_$ ft. use $V_{D1(p)}$

(1 foot = .305 meter)

(Continued)

Pedestrians – Intersection Reservoir Area Analysis

Calculation Form 2 (continued)

<table>
<tr><th>Step</th><th>Condition 1</th><th>Condition 2</th></tr>
<tr><td>1d. Determine number of pedestrians in circulation area (P_C).</td><td>$P_{C1} = \frac{V_{A(p)}(X_1 + X_2) + (V_{B(p)} + V_{C1(p)})\ Y_1}{2700}$
$= \frac{\_\_(\_\_+\_\_) + (\_\_\_\_+\_\_\_\_)\_\_}{2700}$
= ________ pedestrians</td><td>$P_{C2} = \frac{V_{B(p)}(Y_1 + Y_2) + (V_{A(p)} + V_{D1(p)})\ X_1}{2700}$
$= \frac{\_\_(\_\_+\_\_) + (\_\_\_\_+\_\_\_\_)\_\_}{2700}$
= ________ pedestrians</td></tr>
<tr><td>1e. Determine circulation area (A_{circ}) using Table 5.</td><td>$A_{circ1} = P_{C1} \times A_p$
= ____ × ____
= ______ ft.^{2}</td><td>$A_{circ2} = P_{C2} \times A_p$
= ____ × ____
= ______ ft.^{2}</td></tr>
<tr><td>2. Compute Holding Areas for Condition 1 and Condition 2.
2a. Adjust volumes V_{C2} and V_{D2} for platooning (Figure 12)</td><td>$V_{D2(p)} = (V_{D2} \div 15) \times PF$
= (____ ÷ 15) × ____
= ______ peds./min.</td><td>$V_{C2(p)} = (V_{C2} \div 15) \times PF$
= (____ ÷ 15) × ____
= ______ peds./min.</td></tr>
<tr><td>2b. Compute Holding Areas (HA_1 and HA_2). Queuing Space Requirement (Q) from Figure 13.</td><td>$HA_1 = \frac{V_{D2(p)} \times Q \times QT_1}{60}$
= (___ × ___ × ___) ÷ 60
= ______ ft.^{2}</td><td>$HA_2 = \frac{V_{C2(p)} \times Q \times QT_2}{60}$
= (___ × ___ × ___) ÷ 60
= ______ ft.^{2}</td></tr>
<tr><td>3. Compute total area required (A_T). Use Table 1 to derive a value for A_{dead}.</td><td>$A_{T1} = HA_1 + A_{circ1} + A_{dead}$
= ___ + ____ + ____
= ______ ft.^{2}</td><td>$A_{T2} = HA_2 + A_{circ2} + A_{dead}$
= ___ + ____ + ____
= ______ ft.^{2}</td></tr>
<tr><td>3a. Select maximum area (A_{Tmax}) of Conditions 1 and 2 from Step 3.</td><td colspan="2">$A_{Tmax.}$ = ________ ft.^{2}</td></tr>
<tr><td>4. Compute available area (A_A) (see Figure 17).</td><td colspan="2">$A_A = 1.67(X_T)(Y_T) - 0.215r^2$
= 1.67(_____)(_____) – (0.215)(____^{2})
= ________ ft.^{2}</td></tr>
<tr><td>5. Compare Available area (A_A) to maximum area (A_{Tmax}). If maximum area exceeds available area by at least 10%, the intersection is deficient.</td><td colspan="2">$A_A \times 1.10 > A_{Tmax.}$
_____ × 1.10 = _____ > _____
☐ True: Intersection is adequate. ☐ False: Intersection is deficient.</td></tr>
</table>

(1 foot = .305 meter)

Pedestrians–Intersection Crosswalk Analysis

Calculation Form 3

Intersection ______________________________

Corner:
NW ☐ NE ☐
SW ☐ SE ☐

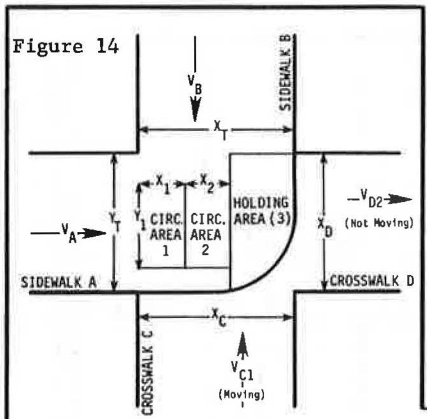
Total Signal Cycle (TS) = ______ sec.
Curb Radius (r) = ______ ft.

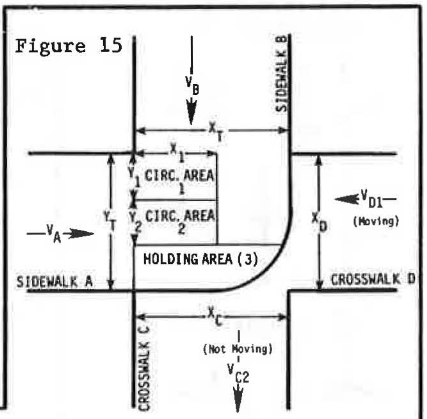
Counts:
Date ______________
Day ______________
Time ______________

Signal Phasing	Area Required for Condition 1	Area Required for Condition 2

Condition 1

Condition 2





Each approach is designated by the letters A, B, C, and D. A and B are sidewalk approaches. The subscript of the volume vectors (V) identifies the movement on each approach. The designation 1 in a subscript indicates pedestrians walking toward the intersection, and the designation 2 indicates pedestrians leaving the intersection. Total Signal Cycle Length (TS), Curb Radius (r), Cross Time (CT), and Queue Time (QT) for each signal phase must be known. All volumes are for 15 minute peaks only.

Movements	V_A	V_B	V_{C1}	V_{D2}	V_A	V_B	V_{C2}	V_{D1}
Pedestrian Volumes								
Cross Time			CT_1=					CT_2=
Queue Time				QT_1=			QT_2=	

Step	Crosswalk C, Condition 1	Crosswalk D, Condition 2
1. Determine crosswalk widths required (X_C and X_D) 1a. Determine total volumes in crosswalk (V_C and V_D)	$V_C = V_{C1} + V_{C2}$ = ______ + ______ = ____________ peds./15 min.	$V_D = V_{D1} + V_{D2}$ = ______ + ______ = ____________ peds./15 min.
1b. Adjust crosswalk volumes for surging.	$V_{C(p)} = V_C[TS \div (CT_1 - 3)]$ = _____[_____ ÷ (_____ − 3)] = ____________ peds./15 min.	$V_{D(p)} = V_D[TS \div (CT_2 - 3)]$ = _____[_____ ÷ (_____ − 3)] = ____________ peds./15 min.
1c. Determine crosswalk widths required (X_C and X_D) using Figure 9.	X_C = ________ ft.	X_D = ________ ft.

(1 foot = .305 meter)

Pedestrians–Intersection Reservoir Area Analysis
Calculation Form 2

Example 2

Intersection STATE STREET AT ELM STREET

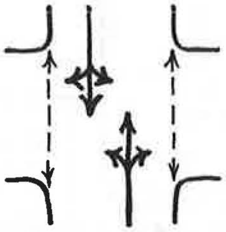
Corner:
NW ☑ NE ☐
SW ☐ SE ☐

Total Signal Cycle (TS) = 80 sec.
Curb Radius (r) = 20 ft.

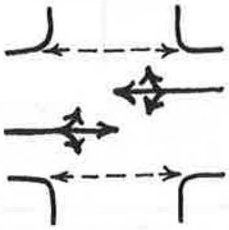
Counts:
Date AUG. 30, 1979
Day THURSDAY
Time 4:30–6 P.M.

Signal Phasing

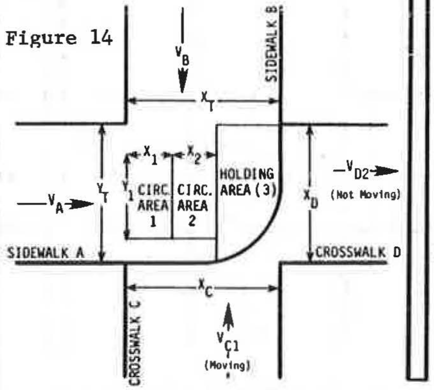
Condition 1

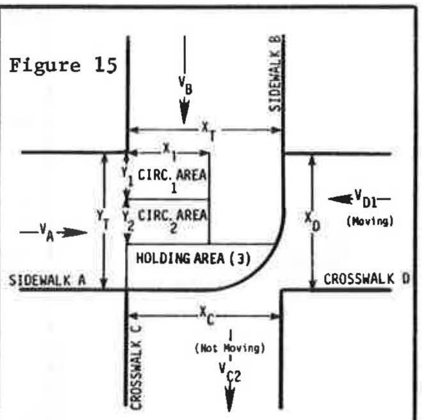


Condition 2



Area Required for Condition 1	Area Required for Condition 2





Each approach is designated by the letters A, B, C, and D. A and B are sidewalk approaches. The subscript of the volume vectors (V) identifies the movement on each approach. The designation 1 in a subscript indicates pedestrians walking toward the intersection, and the designation 2 indicates pedestrians leaving the intersection. Total Signal Cycle Length (TS), Curb Radius (r), Cross Time (CT), and Queue Time (QT) for each signal phase must be known. All volumes are for 15 minute peaks only.

Movements	V_A	V_B	V_{C1}	V_{D2}	V_A	V_B	V_{C2}	V_{D1}
Pedestrian Volumes	900	800	600	400	900	800	700	400
Cross Time	▨	▨	$CT_1 =$ 48	▨				$CT_2 =$ 32
Queue Time	▨	▨	▨	$QT_1 =$ 48			$QT_2 =$ 32	▨

Step	**Condition 1**	**Condition 2**
1. Compute circulation areas for each condition.		
1a. Adjust volumes V_A and V_B for platooning (see Figure 12)	$V_{A(p)} = V_A \times PF$ $= 900 \times 1.0$ $= 900$ peds./15 min.	$V_{B(p)} = V_B \times PF$ $= 800 \times 1.0$ $= 800$ peds./15 min.
1b. Adjust volumes V_{C1} and V_{D1} for platooning (see Figure 12)	$V_{C1(p)} = V_{C1}\ [TS \div (CT_1 - 3)]$ $= 600\ [80 \div (48 - 3)]$ $= 1068$ peds./15 min	$V_{D1(p)} = V_{D1}\ [TS \div (CT_2 - 3)]$ $= 400\ [80 \div (32 - 3)]$ $= 1100$ peds./15 min
1c. Using volumes derived from Steps 1a and 1b, determine effective widths of circulation areas using Figure 9.	$Y_1 = 5$ ft. use $V_{A(p)}$ $X_1 = 5$ ft. use $V_{B(p)}$ $X_2 = 5$ ft. use $V_{C1(p)}$	$X_1 = 5$ ft. use $V_{B(p)}$ $Y_1 = 5$ ft. use $V_{A(p)}$ $Y_2 = 5$ ft. use $V_{D1(p)}$

(1 foot = .305 meter)

(Continued)

Example 2

Problem

At State Street and Elm Street, the intersecting sidewalks each have a total width of 16 feet. The corner radius is 20 feet. The total signal cycle length is 80 seconds and is two-phase with a 48 second/32 second split. The north-south street has 48 seconds of green + yellow and the other street has the remaining 32 seconds. The 15 minute pedestrian volumes are as follows:

V_A = 900 peds./15 min.
V_B = 800 peds./15 min.
V_{C1} = 600 peds./15 min.
V_{C2} = 700 peds./15 min.
V_{D1} = 400 peds./15 min.
V_{D2} = 400 peds./15 min.

Solve for Level of Service C for the circulation area and Level of Service D for the holding area. Assume no platooning exists.

1. Determine if the intersection sidewalk reservoir has a sufficient area to accommodate the above given pedestrian flows.
2. Determine the needed width for the crosswalks.

Analysis

The Intersection Reservoir Area and Crosswalk Calculation Forms are used for the analysis. A sketch of the intersection is shown in Form 2, showing the two conditions corresponding to the typical two-phase signal cycle. The 15 minute pedestrian volumes are represented by vectors with value identification nomenclature. Also shown on Form 2 are the circulation and holding areas that must be computed for the two signal phases. Volume data is obtained. In this case, a pedestrian count was done on Thursday, August 30, 1978 from 4:00 to 6:30 p.m. For future conditions, volume projections are used. The peak 15-minute pedestrian volumes are inserted into Form 2 for both conditions 1 and 2. Specify the signal cycle time (TS), phasing, walktime (WT), and queue time (QT) of the signal operation.

Step 1. The Intersection Circulation Areas for Conditions 1 and 2 are Computed.

Step 1a. Pedestrian volumes passing through circulation areas have to be adjusted for platooning. The pedestrian volumes (V_A, V_B) are adjusted and a subscript (P) is indicated when platooning conditions have been accounted for. Figure 12 provides the platooning factors (PF) for different Level of Service (LOS). If no platooning exists, the platoon factor is 1.0.

For V_A and V_B, which are the inbound sidewalk pedestrian volumes, adjust for platooning:

$$V_{A(p)} = V_A \times PF$$
$$V_{B(p)} = V_B \times PF$$

However, in this example, it is assumed that no platooning occurs and PF is equal to 1.00.

$$V_{A(p)} = 900 \times 1 = 900 \text{ peds./15 min.}$$
$$V_{B(p)} = 800 \times 1 = 800 \text{ peds./15 min.}$$

Step 1b. The pedestrian volumes V_{C1} and V_{D1} which are the inbound crosswalk volumes, represent a special case. The actual peak demand for the incoming sidewalk vector will be relatively greater than the peaks for the other three vectors, because the measurement period for the crossing is considerably less than the total time for the measurement period. The hourly peak for V_{C1} and V_{D1} is proportional to the ratio of the signal cycle time (TS) (in seconds) to the total cross time (in seconds) for the crossing minus three seconds associated with pedestrian start-up delay prior to beginning to cross:

$$V_{C1(p)} = V_{C1} \times [TS \div (CT_1 - 3)]$$
$$V_{D1(p)} = V_{D1} \times [TS \div (CT_2 - 3)]$$

where:

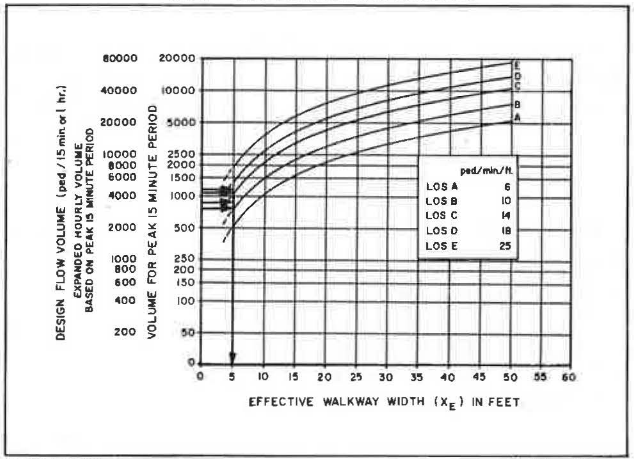
TS = Total signal time, in seconds
CT- 3 = Total cross time less 3 seconds start-up delay

$$V_{C1} = 600 \times [80 \div (48 - 3)]$$
$$= 1068 \text{ peds./15 min.}$$

$$V_{D1} = 400 \times [80 \div (32 - 3)]$$
$$= 1100 \text{ peds./15 min.}$$

Step 1c. Determine the effective widths for the circulation area for conditions 1 and 2. Knowing $V_{A(p)}$, $V_{B(p)}$, $V_{C1(p)}$ and $V_{D1(p)}$ and entering Figure 9 for a specified Level of Service C, the effective widths (X_1, X_2 , Y_1 , and Y_2) can be determined. Knowing the effective widths, the circulation area

Figure 9. Effective Walking Width-Design Considerations



Source: Ref. (4) (1 foot = .305 meter)

Pedestrians – Intersection Reservoir Area Analysis
Calculation Form 2 (continued)

Step	Condition 1	Condition 2
1d. Determine number of pedestrians in circulation area (P_C).	$P_{C1} = \frac{V_{A(p)}(X_1 + X_2) + (V_{B(p)} + V_{C1(p)})\ Y_1}{2700}$ $= \frac{900\ (5 + 5) + (800 + 1068)5}{2700}$ = 6.8 pedestrians	$P_{C2} = \frac{V_{B(p)}(Y_1 + Y_2) + (V_{A(p)} + V_{D1(p)})\ X_1}{2700}$ $= \frac{800\ (5 + 5) + (900 + 1100)5}{2700}$ = 6.7 pedestrians
1e. Determine circulation area (A_{circ}) using Table 5.	$A_{circ1} = P_{C1} \times A_p$ $= 6.8 \times 24$ = 163 ft.²	$A_{circ2} = P_{C2} \times A_p$ $= 6.7 \times 24$ = 161 ft.²
2. Compute Holding Areas for Condition 1 and Condition 2.		
2a. Adjust volumes V_{C2} and V_{D2} for platooning (Figure 12)	$V_{D2(p)} = (V_{D2} \div 15) \times PF$ $= (700 \div 15) \times 1.0$ = 46.7 peds./min.	$V_{C2(p)} = (V_{C2} \div 15) \times PF$ $= (400 \div 15) \times 1.0$ = 26.7 peds./min.
2b. Compute Holding Areas (HA_1 and HA_2). Queuing Space Requirement (Q) from Figure 13.	$HA_1 = \frac{V_{D2(p)} \times Q \times QT_1}{60}$ $= (46.7 \times 5 \times 48) \div 60$ = 187.8 ft.²	$HA_2 = \frac{V_{C2(p)} \times Q \times QT_2}{60}$ $= (26.7 \times 5 \times 32) \div 60$ = 71 ft.²
3. Compute total area required (A_T). Use Table 1 to derive a value for A_{dead}.	$A_{T1} = HA_1 + A_{circ1} + A_{dead}$ = ____ + ____ + ____ = ______ ft.²	$A_{T2} = HA_2 + A_{circ2} + A_{dead}$ = ____ + ____ + ____ = ______ ft.²
3a. Select maximum area (A_{Tmax}) of Conditions 1 and 2 from Step 3.	$A_{Tmax.}$ = ______ ft.²	
4. Compute available area (A_A) (see Figure 17).	$A_A = 1.67(X_T)(Y_T) - 0.215r^2$ = 1.67(______)(______) − (0.215)(______²) = ______ ft.²	
5. Compare Available area (A_A) to maximum area (A_{Tmax}). If maximum area exceeds available area by at least 10%, the intersection is deficient.	$A_A \times 1.10 > A_{Tmax.}$ _____ × 1.10 = _____ > _____ ☐ True: Intersection is adequate. ☐ False: Intersection is deficient.	

(1 foot = .305 meter)

can be determined. (The minimum effective width is 5 feet. This allows two pedestrians to pass one another without colliding.)

For $V_{A(p)}$ of 900 Y_1 = 5 feet

For $V_{B(p)}$ of 800 X_1 = 5 feet

For $V_{C1(p)}$ of 1068 X_2 = 5 feet

For $V_{D1(p)}$ of 1100 Y_2 = 5 feet

Step 1d. The space in the circulation areas 1 and 2 must be large enough that the crossing streams of pedestrians have sufficient area to avoid an intense probability of conflict.

The number of pedestrians (P_C) who are simultaneously located within the calculated circulation area (A_{circ1} and A_{circ2}) can be determined for conditions 1 and 2:

$$P_{C1} = \frac{V_{A(p)}(X_1 + X_2) + (V_{B(p)} + V_{C1(p)})Y_1}{2700}$$

$$P_{C2} = \frac{V_{B(p)}(Y_1 + Y_2) + (V_{A(p)} + V_{D1(p)})X_1}{2700}$$

Thus when these two equations are combined, one can determine the total pedestrians in the circulation area. It is assumed pedestrians will walk at a lower speed of 3 feet/second because of the crossing conflicts, while the average walking speed is usually 4 feet/second. Note that at 3 feet/second a pedestrian will travel 2700 feet in 15 minutes.

$$P_{C1} = \frac{900(5 + 5) + (800 + 1068)5}{2700}$$

$$= 6.8 \text{ pedestrians}$$

$$P_{C2} = \frac{800(5 + 5) + (900 + 1100)5}{2700}$$

$$= 7.6 \text{ pedestrians}$$

Step 1e. Table 5 is checked to determine if the calculated circulation area has enough space to afford an acceptable level of probability of conflict. Table 5 indicates the amount of area per pedestrian that is required for a given probability of conflict at a given level of service.

Once a level of probability of conflict is chosen, the square feet per pedestrian is known. If we know the number of pedestrians in the circulation area from Step 1d and choose an acceptable level of conflict from Table 5, we can then multiply the space requirement per pedestrian (A_p ft²) from Table 5 times the number of pedestrians in the circulation area. Thus, circulation area A_{circ} is determined. This calculation is carried out for conditions 1 and 2. This determines the area needed to maintain a given level of conflict for a given level of service.

$$A_{circ1} = P_{C1} \times A_p$$

$$A_{circ2} = P_{C2} \times A_p$$

From Table 5, for Level of Service **C** we need 24 ft²/peds to achieve probability of conflict of 0.5. Thus, A_p = 24 ft².

Table 5. Pedestrian Space Requirements for Cross Flow Traffic and Probability of Conflict

Level of Service	Space (ft^2/ped)	Conflict Probability
A	over 35	0.1 or less
B	24-35	0.5-0.1
[✓ C]	18-[✓ 24]	0.75-[✓ 0.5]
D	13-18	0.90-0.75
E	6-13	0.98-0.90
F	under 6	0.98 or more

Source: Ref. (3) (1 foot = .305 meter)

$$A_{circ(p)} = P_C \times A_p$$

$$A_{circ1} = 6.8 \times 24 = 163 \text{ ft.}^2$$

$$A_{circ2} = 6.7 \times 24 = 161 \text{ ft.}^2$$

Step 2a. The vectors V_{C2} and V_{D2}, which are the outbound crosswalk volumes, are adjusted in the same manner as V_A and V_B, except that V_{C2} and V_{D2} do not remain as 15 minute volumes. Instead, V_{C2} and V_{D2} are converted from a peak 15 minute volume to a peak one minute volume. One minute volumes are needed to compute the holding areas.

$$V_{C2(p)} = (V_{C2} \div 15) \times PF$$

$$V_{D2(p)} = (V_{D2} \div 15) \times PF$$

$$V_{C2(p)} = (700 \div 15) \times 1.0 = 46.7 \text{ peds./min.}$$

$$V_{D2(p)} = (400 \div 15) \times 1.0 = 26.7 \text{ peds./min.}$$

Step 2b. Compute holding area for conditions 1 and 2 (HA_1 and HA_2). V_{C2} and V_{D2} are the pedestrian volumes that are queueing up waiting for the signal to change so they can cross the street for each phase. Since $V_{C2(p)}$ and $V_{D2(p)}$ are per minute flows they have to be changed to per-second flows by dividing by 60 (sec/min) flow rates of pedestrian per second. Thus, if the Queue Time (QT) or non-walk time is known, then the average number of pedestrians waiting per cycle can be determined. The queuing space requirement (Q) for a specified Level of Service is derived from Figure 13 and then the holding area (HA) can be determined.

$$HA_1 = \frac{V_{D2(p)} \times Q \times QT_1}{60}$$

$$HA_2 = \frac{V_{C2(p)} \times Q \times QT_2}{60}$$

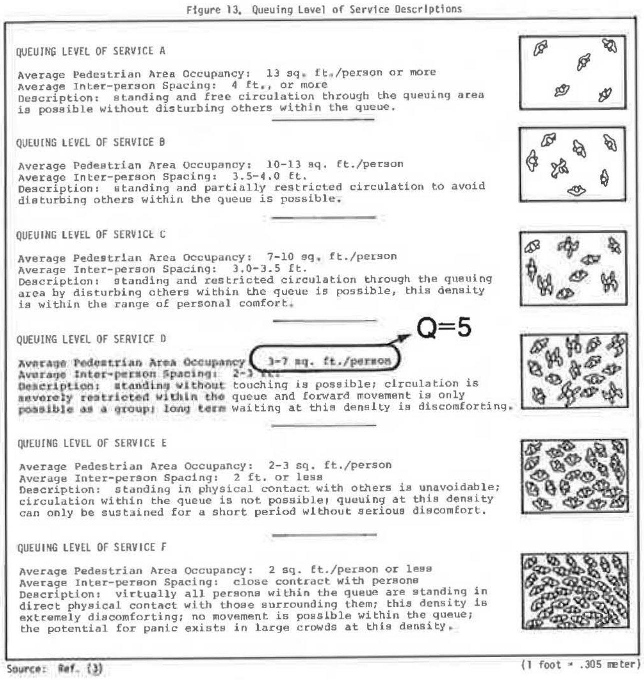
Where QT equals the non-green time (or red time) faced by a pedestrian moving in the direction of the V_{C2} or V_{D2} vector.

Pedestrians – Intersection Reservoir Area Analysis
Calculation Form 2 (continued)

Step	Condition 1	Condition 2
1d. Determine number of pedestrians in circulation area (P_C).	$P_{C1} = \frac{V_{A(p)}(X_1 + X_2) + (V_{B(p)} + V_{C1(p)})\ Y_1}{2700}$ $= \frac{900(5+5) + (800+1068)5}{2700}$ $= 6.8$ pedestrians	$P_{C2} = \frac{V_{B(p)}(Y_1 + Y_2) + (V_{A(p)} + V_{D1(p)})\ X_1}{2700}$ $= \frac{800(5+5)+(900+1100)5}{2700}$ $= 6.7$ pedestrians
1e. Determine circulation area (A_{circ}) using Table 5.	$A_{circ1} = P_{C1} \times A_p$ $= 6.8 \times 24$ $= 163$ ft.^{2}	$A_{circ2} = P_{C2} \times A_p$ $= 6.7 \times 24$ $= 161$ ft.^{2}
2. Compute Holding Areas for Condition 1 and Condition 2.		
2a. Adjust volumes V_{C2} and V_{D2} for platooning (Figure 12)	$V_{D2(p)} = (V_{D2} \div 15) \times PF$ $= (700 \div 15) \times 1.0$ $= 46.7$ peds./min.	$V_{C2(p)} = (V_{C2} \div 15) \times PF$ $= (400 \div 15) \times 1.0$ $= 26.7$ peds./min.
2b. Compute Holding Areas (HA_1 and HA_2). Queuing Space Requirement (Q) from Figure 13.	$HA_1 = \frac{V_{D2(p)} \times Q \times QT_1}{60}$ $= (46.7 \times 5 \times 48) \div 60$ $= 187$ ft.^{2}	$HA_2 = \frac{V_{C2(p)} \times Q \times QT_2}{60}$ $= (26.7 \times 5 \times 32) \div 60$ $= 71$ ft.^{2}
3. Compute total area required (A_T). Use Table 1 to derive a value for A_{dead}.	$A_{T1} = HA_1 + A_{circ1} + A_{dead}$ $= 187 + 163 + 0$ $= 350$ ft.^{2}	$A_{T2} = HA_2 + A_{circ2} + A_{dead}$ $= 71 + 161 + 0$ $= 232$ ft.^{2}
3a. Select maximum area (A_{Tmax}) of Conditions 1 and 2 from Step 3.	$A_{Tmax.} = 350$ ft.^{2}	
4. Compute available area (A_A) (see Figure 17).	$A_A = 1.67(X_T)(Y_T) - 0.215r^2$ $= 1.67(16)(16) - (0.215)(20^2)$ $= 342$ ft.^{2}	
5. Compare Available area (A_A) to maximum area (A_{Tmax}). If maximum area exceeds available area by at least 10%, the intersection is deficient.	$A_A \times 1.10 > A_{Tmax.}$ $342 \times 1.10 = 376 > 350$ ☑ True: Intersection is adequate. ☐ False: Intersection is deficient.	

(1 foot = .305 meter)

Figure 13. Queuing Level of Service Descriptions



QUEUING LEVEL OF SERVICE A

Average Pedestrian Area Occupancy: 13 sq. ft./person or more
Average Inter-person Spacing: 4 ft., or more
Description: standing and free circulation through the queuing area is possible without disturbing others within the queue.

QUEUING LEVEL OF SERVICE B

Average Pedestrian Area Occupancy: 10-13 sq. ft./person
Average Inter-person Spacing: 3.5-4.0 ft.
Description: standing and partially restricted circulation to avoid disturbing others within the queue is possible.

QUEUING LEVEL OF SERVICE C

Average Pedestrian Area Occupancy: 7-10 sq. ft./person
Average Inter-person Spacing: 3.0-3.5 ft.
Description: standing and restricted circulation through the queuing area by disturbing others within the queue is possible, this density is within the range of personal comfort.

QUEUING LEVEL OF SERVICE D

Average Pedestrian Area Occupancy 3-7 sq. ft./person
Average Inter-person Spacing: 2-3 ft.
Description: standing without touching is possible; circulation is severely restricted within the queue and forward movement is only possible as a group; long term waiting at this density is discomforting.

QUEUING LEVEL OF SERVICE E

Average Pedestrian Area Occupancy: 2-3 sq. ft./person
Average Inter-person Spacing: 2 ft. or less
Description: standing in physical contact with others is unavoidable; circulation within the queue is not possible; queuing at this density can only be sustained for a short period without serious discomfort.

QUEUING LEVEL OF SERVICE F

Average Pedestrian Area Occupancy: 2 sq. ft./person or less
Average Inter-person Spacing: close contract with persons
Description: virtually all persons within the queue are standing in direct physical contact with those surrounding them; this density is extremely discomforting; no movement is possible within the queue; the potential for panic exists in large crowds at this density.

Source: Ref. (3)

(1 foot = .305 meter)

$$HA_1 = \frac{46.7 \times 5 \times 48}{60} = 186.8 \text{ ft.}^2$$

$$HA_2 = \frac{26.7 \times 5 \times 32}{60} = 71.2 \text{ ft.}^2$$

Step 3. Compute Total Area (A_T) Required for Conditions 1 and 2. HA is determined in Step 2b, and A_{circ} is determined in Step 1e. To determine A_{dead} use Table 1 to the compute area taken up by street furniture in the sidewalk (for this problem assume $A_{dead} = 0$).

$$A_T = HA + A_{circ} + A_{dead}$$

$$A_{T1} = 187 + 163 + 0 = 350 \text{ ft.}^2$$

$$A_{T2} = 71 + 161 + 0 = 232 \text{ ft.}^2$$

Step 3a. Select the maximum area (A_{Tmax}) by taking the larger value of conditions 1 and 2 in Step 3.

$$A_{Tmax} = 350 \text{ ft.}^2$$

Step 4. Compute the Available Area at the Intersection. As indicated in Figure 17, use the equation: $A_A = 1.67(X)(Y) - 0.215r^2$ (X and Y are the total width of sidewalks A and B; r is the radius used for the intersection walkway).

$$A_A = 1.67(X_T)(Y_T) - 0.215r^2$$
$$= 1.67(16)(16) - 0.215(20^2) = 342 \text{ ft.}^2$$

Step 5. Compare the Available Area (A_A) Against the Maximum Area (A_{Tmax}). The maximum Area (A_{Tmax}) should not exceed the Available Area by more than 10%. Otherwise, if it does exceed the Available Area by more than 10%, the intersection is deficient and additional space must be provided to accommodate the flow.

$$342 \times 1.10 = 376 \text{ ft.}^2 > 350 \text{ ft.}^2$$

Further steps utilizing Form 3 are described in the following pages.

Figure 14. Area Required for Condition 1 Movement Vectors

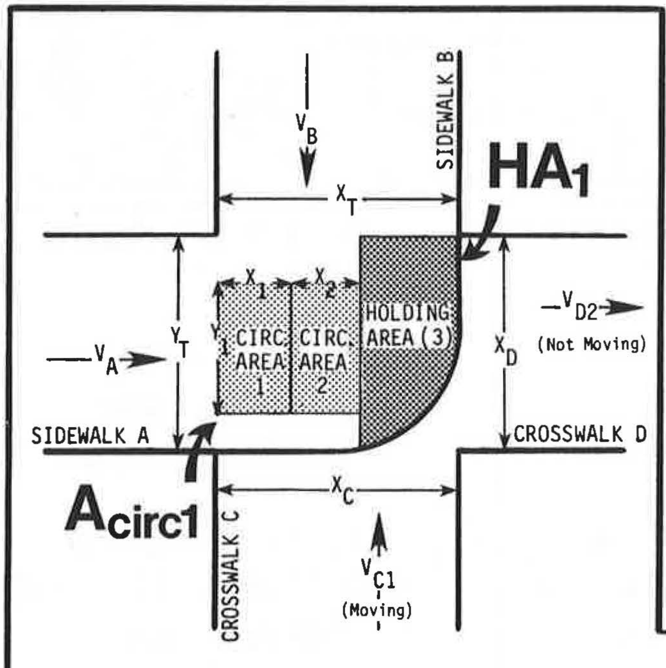
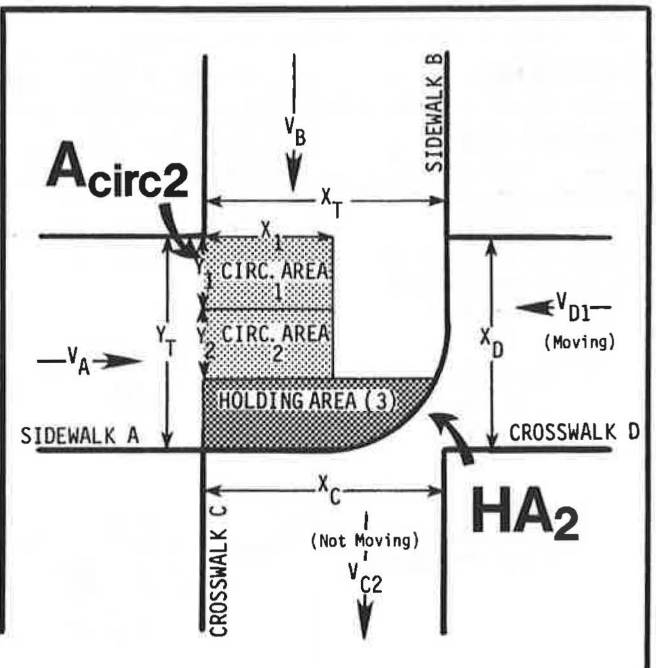


Figure 15. Area Required for Condition 2 Movement Vectors



Each approach is designated by the letters A, B, C, and D. A and B are sidewalk approaches. The subscript of the volume vectors (V) identifies the movement on each approach. The designation 1 in a subscript indicates pedestrians walking toward the intersection, and the designation 2 indicates pedestrians leaving the intersection. Total Signal Cycle Length (TS), Curb Radius (r), Cross Time (CT), and Queue Time (QT) for each signal phase must be known. All volumes are for 15 minute peaks only.

Pedestrians–Intersection Crosswalk Analysis
Calculation Form 3

Intersection STATE STREET AT ELM STREET

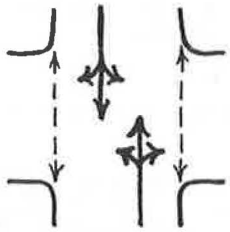
Corner:
NW ☑ NE ☐
SW ☐ SE ☐

Total Signal Cycle (TS) = 80 sec.
Curb Radius (r) = 20 ft.

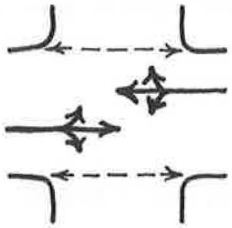
Counts:
Date AUG 30, 1979
Day THURSDAY
Time 4:30–6:30 P.M.

Signal Phasing

Condition 1



Condition 2



Area Required for Condition 1	Area Required for Condition 2

Figure 14

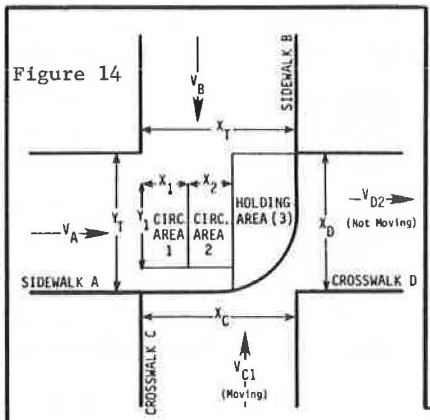
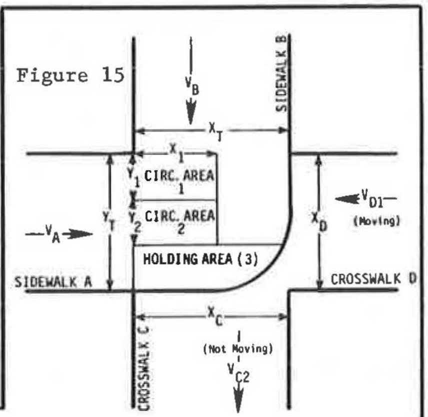


Figure 15



Each approach is designated by the letters A, B, C, and D. A and B are sidewalk approaches. The subscript of the volume vectors (V) identifies the movement on each approach. The designation 1 in a subscript indicates pedestrians walking toward the intersection, and the designation 2 indicates pedestrians leaving the intersection. Total Signal Cycle Length (TS), Curb Radius (r), Cross Time (CT), and Queue Time (QT) for each signal phase must be known. All volumes are for 15 minute peaks only.

Movements	V_A	V_B	V_{C1}	V_{D2}	V_A	V_B	V_{C2}	V_{D1}
Pedestrian Volumes	—	—	600	400	—	—	700	400
Cross Time	▨	▨	CT_1 = 48	▨	▨	▨	▨	CT_2 = 32
Queue Time	▨	▨	▨	QT_1 = 48	▨	▨	QT_2 = 32	▨

Step	Crosswalk C, Condition 1	Crosswalk D, Condition 2
1. **Determine crosswalk widths required** (X_C and X_D)		
1a. Determine total volumes in crosswalk (V_C and V_D)	$V_C = V_{C1} + V_{C2}$ = 600 + 700 = 1300 peds./15 min.	$V_D = V_{D1} + V_{D2}$ = 400 + 400 = 800 peds./15 min.
1b. Adjust crosswalk volumes for surging.	$V_{C(p)} = V_C[TS \div (CT_1 - 3)]$ = 1300 [80 ÷ (48 − 3)] = 2310 peds./15 min.	$V_{D(p)} = V_D[TS \div (CT_2 - 3)]$ = 800 [80 ÷ (32 − 3)] = 2206 peds./15 min.
1c. Determine crosswalk widths required (X_C and X_D) using Figure 9.	X_C = 11 ft. (LOS=C)	X_D = 10 ft. (LOS=C)

(1 foot = .305 meter)

Additional Steps to Calculate Crosswalk Widths

Step 1a. To evaluate the cross walk width requirements associated with the pedestrian movements in crosswalks C and D, the following procedures are employed. Use Form 3 for this analysis. Compute the two-directional volumes for each crosswalk. For crosswalk C, this total volume (V_C) is the sum of V_{C1} and V_{C2} which are the peak volumes for condition 1. Also, for crosswalk D, V_D is the sum of V_{D1} and V_{D2} which are the peak volume for condition 2. (The subscripts C and D indicate the crosswalk; subscript C refers to the vertical (north-south) crossing and subscript D refers to the horizontal (east-west) crossing.)

$V_C = 600 + 700 = 1300$ peds./15 min.

$V_D = 400 + 400 = 800$ peds./15 min.

Step 1b. Compute the adjusted volume for each crosswalk, to accommodate surging. The results of the calculation are shown on the completed Calculation Form 3.

Step 1c. Determine the required crosswalk widths for Level of Service **C** using Figure 9.

The complete series of steps followed is shown on the filled-in Calculation Form 3.

REFERENCES

(1) Highway Research Board, Highway Capacity Manual, HRB Special Report 87, Washington, D.C., 1965, 411 pp.

(2) Pushkarev, B., and Zupan, J. M., Urban Space for Pedestrians, Cambridge, MA, MIT Press, 1975, 212 pp.

(3) Fruin, J. J., Pedestrian Planning and Design, New York Metropolitan Association of Urban Designers and Environmental Planners, 1971, 206 pp.

(4) RTKL Associates, Inc., Feasibility Analysis and Design Concepts and Criteria for Communitywide Separated Pedestrian Networks, Phase III, Draft Pedestrian Planning Procedures Manual, Vols. I - III, Maryland, 1977.

(5) Hall, E. T., The Hidden Dimension, New York, Doubleday and Company, Inc., 1966, 216 pp.

WEAVING ANALYSIS: BACKGROUND OF TWO INTERIM PROCEDURES

The release of two interim procedures (Polytechnic Institute of New York and J. Leisch) dealing with weaving analysis poses special problems over the interim period before the new Highway Capacity Manual is finalized. A most desirable goal for the final Manual is agreement on a single procedure. Therefore, user evaluation and feedback are of special importance so that an intelligent choice or synthesis can be made regarding the procedure to be adopted. Undoubtedly, there will also be some limited research carried out during the interim evaluation period which might lead to improvements.

Users of the two procedures should have some familiarity with their backgrounds. The Polytechnic Institute of New York (PINY) procedure was developed as part of a FHWA contract effort to develop new freeway capacity procedures -- the goal was to develop procedures based on research accomplished since the 1965 HCM. Part of the "accomplished research" was NCHRP 3-15, Weaving Area Operations Study, also performed by PINY. The NCHRP 3-15 weaving procedure, documented in NCHRP Report No. 159, was found to be difficult to apply, so much so that a special effort was made to simplify the structure to make it more easily applied and understood, while still retaining its demonstrated accuracy and sensitivity to lane configuration, seen by PINY as a major factor influencing operations. PINY researchers used the same data base as the NCHRP 3-15 study which consisted of 38 sites from the 1963 BPR Urban Area Weaving Capacity Study and 14 sites collected specifically for the NCHRP study. As the modified PINY weaving procedure is now a portion of the freeway capacity procedures, the procedure was recalibrated to reflect modified service volume concepts developed in the overall freeway procedures. In summary, the PINY weaving procedure was part of a monitored research effort which has been presented in part at the Transportation Research Board Annual Meetings and reviewed by the Highway Capacity and Quality of Service Committee, the NCHRP 3-28 panel, and the contractor for the NCHRP 3-28 effort.

The Leisch weaving technique has a much different history. It was first introduced to the user community through an article published in the March 1979 issue of ITE Journal. Essentially, it was an in-house development by Jack E. Leisch & Associates. As such, it was unfunded externally and not subject to outside monitoring. The individuals involved in its development felt they had a significant contribution to make in design practice for weaving sections based on analysis of weaving data and experience in the highway design profession. Inherent in its development was a practicality of application oriented toward the designer user. However, because the procedure is not supported by a research report, the user is forced to accept at face value the accuracy of the procedure as well as the strength of its foundation. The data used in the development was to some extent identical with that used in the PINY effort. The 1963 BPR data were used, but mainly for Levels of Service D and E. The data gathered by PINY in the 14 sites specifically for NCHRP 3-15 were also used as taken from summaries given in NCHRP Report 159. Finally, other data not as yet specified were used. The developed procedure builds on the 1965 HCM and also on Mr. Leisch's involvement as a consultant to PINY on the NCHRP 3-15 effort. The Leisch weaving procedure is intended to be used with the freeway materials currently in the 1965 HCM or with Mr. Leisch's reformatting and expansion of the 1965 HCM entitled Capacity Analysis Techniques for Design and Operation of Freeway Facilities, 1974, FHWA Report RD-74-24. Chapter V of the latter report deals with WEAVING SECTIONS and would be replaced by the new procedure.

The user of these interim materials hopefully will apply and evaluate both weaving procedures. User ease of application, gaps and inconsistencies within each procedure, and accuracy are all of prime importance. It is recognized that the question of accuracy might require some data collection, but it is highly essential that this type of feedback be obtained.

FREEWAY CAPACITY PROCEDURES

ROGER P. ROESS, ELLIOT LINZER, WILLIAM R. McSHANE,
and LOUIS J. PIGNATARO, Transportation Training and
Research Center, Polytechnic Institute of New York

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PREFACE

The "Freeway Capacity Procedures" section of this circular is the result of a Federal Highway Administration-sponsored effort to synthesize the best available information and results on freeway capacity and related subjects into a cohesive set of procedures for design and analysis. The work was performed by the Transportation Training and Research Center of the Polytechnic Institute of New York. The procedures were authored by Drs. Roger P. Roess, William R. McShane, Louis J. Pignataro and Mr. Elliot Linzer, all of the Polytechnic.

The procedures developed are based upon a synthesis of available information, a number of pilot field studies, and a comprehensive re-evaluation of the data base collected for the National Cooperative Highway Research Program-sponsored "Weaving Area Operations Study," also conducted at the Polytechnic. The latter re-evaluation, which was spurred by additional research results not available at the time of the weaving study, resulted in the formulation of the weaving procedures contained in the "Freeway Capacity Procedures" section.

Where gaps in available information exist, they exist also in the freeway procedures, and are so noted. Like the 1965 HCM, the procedures are no better than the data and information available for their formulation. They improve on the 1965 HCM in that additional research results have become available since its publication which have been adopted herein. This is not to suggest that more research in the area is not needed, for there are many topics and subjects, some of which are as basic as speed-flow relationships, which are in need of further study -- and larger data bases for that study.

It is hoped that these "Freeway Capacity Procedures" will help formulate the freeway content of the forthcoming HCM revision, due in the mid-1980's. It is hoped, though, that additional research and information will be available to further improve these techniques for that revision. Capacity analysis techniques must be dynamic and continuously reviewed, as driving habits and vehicles change the basic characteristics of highway flow. These procedures are merely one step of what should be a process of periodic review and revision.

In its use, these procedures should be a useful tool to designers, operational analysts, and planners in evaluating freeway conditions. These procedures do not make decisions, but provide results which, along with economics, environmental concerns, energy impacts, etc. will be factors for professionals and decision-makers to consider in making and executing those decisions.

CHAPTER I - INTRODUCTION AND BASICS

In 1950, the first edition of the Highway Capacity Manual (1) was published as a practical guide to the design and evaluation of streets and highways in terms of their traffic-carrying ability. One of the principal purposes of this manual was to assure consistency of procedures in the national program of highway design and construction. In general, the data bases for the 1950 HCM were sparse, and the collective judgment of the outstanding professionals of the time was exercised through the Highway Capacity Committee of the Highway Research Board to produce a set of workable procedures.

In 1953, the committee was reconvened to begin work on an improved manual, which was eventually published in 1965. The 1965 HCM, like its predecessor, was to be a practical guide in capacity analysis for design and operational evaluation. Reflecting the changing needs of the profession, the new manual (2) devoted a significant amount of attention to freeways and freeway components, such as weaving and ramps. It also reflected much-improved data bases, which had been collected by a variety of governmental agencies over the span of years between 1950 and the early 1960's. Substantial professional judgment, however, was still needed to close gaps in the available information. The 1965 HCM remains the current standard in the field, but the Transportation Research Board Committee on Highway Capacity and Quality of Service is now working on a further updating which will result in a new manual being published in the early to mid-1980's.

The committee's work is being assisted by several major contracts for research in highway capacity, funded by the National Cooperative Highway Research Program and the Federal Highway Administration. This document is the result of one of these, specifically, "Freeway Capacity Analysis Procedures," sponsored by the FHWA.

Since the publication of the 1965 HCM, a number of significant research efforts have taken place relating to freeway capacity analysis, including:

- a comprehensive study of weaving area operations, sponsored by NCHRP and conducted by the Polytechnic Institute of New York (3).
- simulation studies treating the effect of trucks and traffic regulations on traffic streams for both two-lane and multilane roadways, conducted by the Midwest Research Institute (4,5).
- the development of simplified procedures for freeway capacity analysis by J. Leisch for the FHWA (6).
- a study of truck weight/horsepower ratios conducted for NCHRP at Penn State University (7).
- studies on the effect of recreational vehicles on two-lane, two-way traffic flow, conducted by A. Werner (8).

These chapters contains procedures for freeway design and operational analysis, and represents an updating of procedures found in the 1965 HCM based upon the above and other research which has taken place in the field. It is intended that these procedures eventually be used, perhaps in modified form, as the basis for freeway chapters of the upcoming revision to the HCM. Their presentation here will enable them to be rigorously tested and evaluated by users in the field.

The Freeway Facility

The freeway is a type of facility which is rather unique, in that it is the only form of highway which offers totally "uninterrupted" flow, that is, the traffic stream is not interrupted at any time by factors external to the traffic stream. There are no STOP signs or signals, no at-grade crossings, no pedestrian access, no direct access to abutting lands, and all vehicle entries and exits are made at ramps. As such, the operation of the facility is highly sensitive to changes in traffic demand, even short-term fluctuations, as there are no metering devices to spread short-term peaks. Further, virtually everything which happens in terms of the overall operation of freeway traffic streams is the result of interaction between vehicles of the traffic stream, and of the interaction between vehicles and geometric characteristics of the highway. It is the objective of the procedures detailed herein to relate operating characteristics to the geometric and traffic conditions which exist during a defined time interval on a specified segment of freeway.

A. Components of a Freeway

In general, a freeway is composed of three different types of components:

- Basic Freeway Segments: Segments of the freeway which are not affected by merging or diverging maneuvers at nearby ramps, or by weaving movements;
- Weaving Areas: Segments of the freeway where two or more vehicle flows must cross each other's path along a length of the freeway. These are usually formed when merge areas are followed by diverge areas. They are also formed when a freeway on-ramp is followed by an off-ramp and the two are connected by a continuous auxiliary lane;
- Ramp Junctions: Points at which on- and off-ramps join the freeway. The junction formed at this point is an area of turbulence due to concentrations of merging or diverging movements.

Figure 1.1 illustrates several examples of these types of freeway components. This document presents procedures for the design and operational analysis of each type of component in Chapters II, III, and IV respectively.

In each case, procedures are designed to consider components of uniform type (basic, weaving, ramp), geometry (grades, number and width of lanes, lateral clearance, curvature), and traffic conditions (volume and percentages of trucks, buses, and recreational vehicles). Analysis of any extended length of freeway begins by breaking it up into such uniform components. Detailed instructions on how this is accomplished are given in Chapters II, III, and IV for the various types of segments.

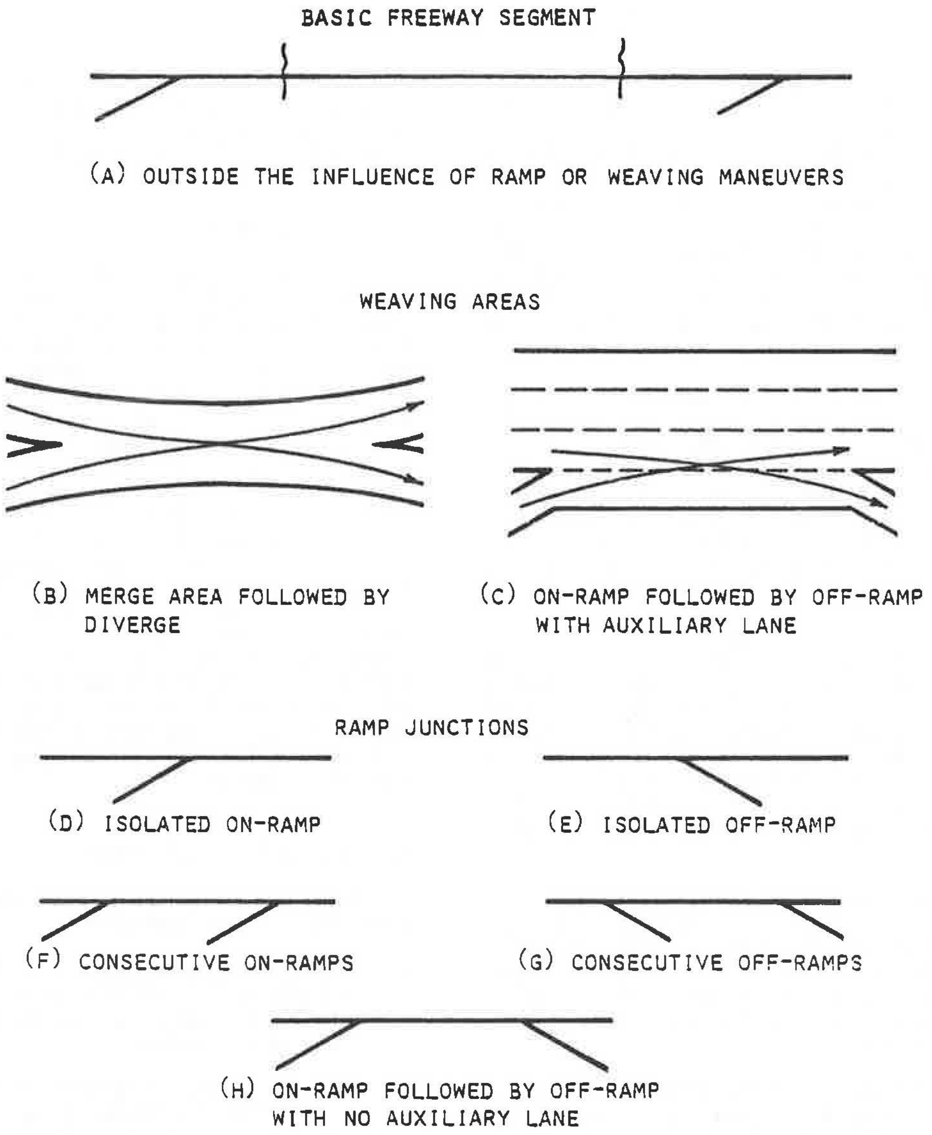


FIGURE 1.1

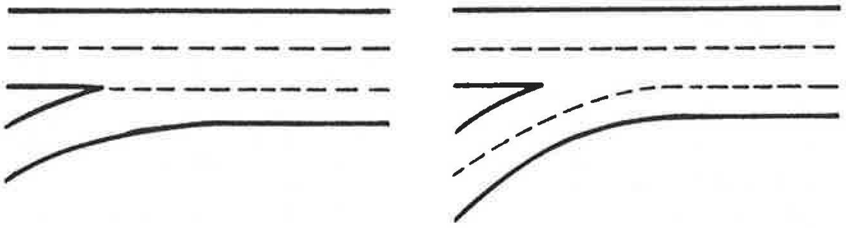
FREEWAY COMPONENTS ILLUSTRATED

B. Overall Considerations

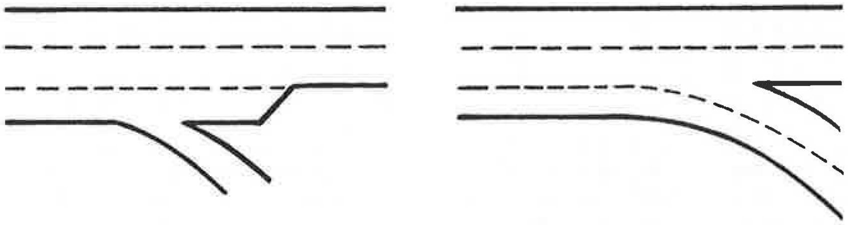
While the basic freeway unit for consideration using the procedures detailed herein is a segment of uniform characteristics as described above, it must be remembered that the freeway operates as a cohesive unit, and that the operation of one component often influences that of others. Should one freeway component break down, the resulting congestion will spread upstream into adjacent segments. Thus, segments may become congested when procedures herein indicate that their operation should be acceptable. The procedures detailed herein do not account for problems in one segment which are caused by problems in another--yet the analyst or designer must be keenly aware of such potentials.

Another critical factor is the transition between adjacent segments of freeway. There are generally several different ways in which a transition may be handled between adjacent freeway components. These alternatives generally involve the concepts of lane balance and configuration. Figure 1.2 illustrates two prime examples, that of a lane addition and that of a lane drop. Figure 1.2(a) illustrates two alternative ways of adding a lane to a freeway at a ramp. Figure 1.2(b) gives the corresponding alternatives for dropping, or subtracting, a lane at a ramp. In both cases, the design of the upstream and downstream freeway sections are not affected by the transition alternate selected. The quality of the merge or diverge by which the transition is made will, however, be greatly different in each case. Chapter IV procedures can adequately analyze the effect of each alternative on the merge or diverge itself, but cannot take into account the differing impacts on upstream and downstream components.

The difference between the alternatives of Figure 1.2 is in the configuration of lanes. In all cases, the transition is between a 3-lane freeway segment and a 2-lane freeway segment. In the first alterna-



(A) TWO ALTERNATES FOR ADDING A FREEWAY LANE AT A RAMP



(B) TWO ALTERNATES FOR DROPPING AT A RAMP

FIGURE 1.2

ILLUSTRATION OF LANE BALANCE
AND CONFIGURATION

tive, single-lane ramps are used, and entering or exiting traffic must do so in the lane closest to the shoulder. In the second of the alternatives, two-lane ramps are used, allowing merging or diverging to take place in two different lanes. In the diverge case, this provides for lane balance; that is, one more lane leaving the diverge point than is entering it. Obviously, the choice of which alternative to use will depend on the ramp analysis itself, and the designer or analyst's impression of the likely upstream and downstream impacts, which are not clearly defined by procedures.

The concept of lane balance is an important one. Lane balance provides the driver approaching a diverge point with a "choice" lane; that is, there is one lane from which the driver may proceed along either leg of the diverge without a lane change. Such a design has the potential to reduce erratic movements resulting from last-second decisions and provides greater flexibility in lane utilization. Since lane balance does require 2-lane ramps, however, it cannot be provided in all cases.

A number of other factors may also influence the operation of the freeway as a whole. Weather and incidents often have a critical effect on operations. Procedures herein are based upon good weather conditions and the absence of traffic incidents. Any deviation from these conditions will adversely impact operations, depending upon the severity of the weather and/or incidents. In areas where ice and snow are the rule rather than the exception, some allowance must be made for this. Similarly, where the number of incidents is high, and their occurence frequent in peak periods, some allowances should be considered. No specific allowance can be specified, however, due to the wide variability of these effects.

Freeway surveillance and control is another factor to be considered. In a number of urban areas where freeways are subject to regular peak hour congestion, freeway surveillance and control systems have been installed. These systems continuously monitor freeway flow, using detectors at several locations, and adjust the ramp metering rates according to those flows using one strategy or another. Some aim to quickly identify incidents and react to them to provide for speedy clearance. Several use variable message signs to advise drivers on safe speeds and approaching hazards. A large part of many such systems is ramp control, in which vehicles are permitted to enter the freeway one at a time at specified intervals. This prevents the clogging effect of several ramp vehicles forcing their way into a crowded stream at the same time, with the resultant breakdown in freeway flow, and may actually reduce the total number of vehicles entering the freeway at any given point. Ramp-metering rates may be preselected by time of the day, may be responsive to freeway detector information, or may be based upon the detection of gaps in the approaching freeway stream.

The principal point, however, is that freeways having such systems may be operated better than similar freeways without. Freeway surveillance and control systems may have a beneficial effect on a freeway, one that cannot be taken fully into account using the procedures presented herein. Again, the user must be aware of this potential. (See Chapter IV, "Ramp Metering")

Another factor of interest in some urban areas is the existence of exclusive high-occupancy vehicle lanes on the freeway. These lanes come in a wide variety of forms, and are becoming increasingly important with the current emphasis on transportation systems management solutions. They are intended to improve the people - carrying capacity of the freeway without major capital expense and to provide additional incentives to use mass transit. The most common forms include the exclusive bus lane or high-occupancy vehicle lane which is used by buses, car-pools and, in N.Y.C., taxis.

It is not the intent of this document to analyze in detail the transit-related impacts of high occupancy vehicle lanes. A"transit" chapter prepared by Levinson and others is included elsewhere in this circular as an "interim capacity material.". These lanes do, however, have an impact on the operation of the remaining non-exclusive lanes of the freeway which must be considered.

Where high-occupancy vehicle lanes are done on a contra-flow basis, (that is, the exclusive lane is taken from the opposing direction of flow, usually an off-peak or reverse-peak flow), the following impacts often occur: 1) The primary traffic flow in remaining peak-direction lanes is reduced, as high occupancy vehicles have been removed to a contra-flow lane, 2) the number of lanes available to reverse peak traffic is reduced by one, or two, if a full-lane buffer between opposing flows is provided, 3) flexible posts or other dividers used to separate opposing flows may hinder reverse-peak vehicles in remaining lanes, and 4) the process of entering and/or leaving the exclusive lane may be disruptive to other vehicles in the traffic stream.

Where high occupancy vehicle lanes are installed in the primary flow lanes of the freeway, these impacts are somewhat different: 1) the primary flow vehicles not eligible to use the high occupancy lane are restricted one lane less than is normally available, 2) as both high-occupancy and other vehicles travel in the same direction, there is no need for flexible posts or other dividers which might hinder flow, 3) the process of entering and/or leaving the special lane is generally not highly disruptive, although increased lane-changing activity near the lane's entry and exit points may result.

Chapter V attempts to provide approximate procedures for analyzing some of these effects. It should be remembered, however, that the intent of such lanes is to optimize the person-carrying ability of the freeway as opposed to its vehicle-carrying ability This involves many planning considerations beyond the scope of this document.

Lastly, the designer or operational analyst must keep in mind that the freeway interacts with other facilities making up the traffic system. The operation of other freeways, and of surrounding streets and arterials may all have an impact on the freeway, and vice-versa. Ramp control may benefit the quality of freeway operations, but it may also divert vehicles to the local arterial network, where they could create more congestion. Arterials and intersections may create queues which extend down freeway ramps and affect freeway operations. These and other potential interactions should always be considered in conjunction with specific analyses conducted using procedures detailed herein.

Chapter V of these freeway capacity procedures gives a detailed treatment of the consideration of the overall freeway facility, including the issues and factors discussed above, as well as others. Where quantitative guidelines can be given, they are. In other cases, logical procedures are suggested, or qualitative instructions given.

Structure of the Document

The remainder of Chapter I of these procedures discusses basic characteristics of freeway flow, the level of service concept, and basic definitions of terminology. Chapters II, III, and IV contain detailed procedures for the analysis of Basic Freeway Segments, Weaving Areas, and Ramps and Ramp Terminals respectively. Each of these chapters is divided into three parts:

- Basic Characteristics: A discussion of the basic characteristics of flow in the type of segment under consideration, and the factors which affect them, as well as a presentation of the equations and/or methods which are used in textbook fashion.
- Computational Procedures: A step-by-step set of instructions for performing a design or analysis.
- Sample Problems: A variety of problems are presented, solved, and discussed, representing a broad range of possibilities for each type of segment being considered.

Organization of the chapters in this manner enables the user to avoid reading through the discussion of "characteristics" after the user has become familiar with the techniques presented. It also allows the user to follow a condensed step-by-step procedure for the consideration of each type of segment in a straightforward manner.

As previously discussed, Chapter V treats the analysis of the freeway as a whole in a detailed way, and gives instructions on the combining of individual segment analyses and their interpretation, as well as discussing some of the system elements noted previously.

Definitions and Terminology

In general, specific terms are defined for the user within the chapter sections in which they occur. This section is intended to present the major terms which will be utilized throughout the document, and those of which the user should have a strong conceptual understanding before proceding into specific procedures.

A. Traffic Flow Measures

Of particular interest are those measures which are used to define or characterize the condition of a traffic stream. These are defined and discussed below:

(1) Speed: a rate of motion expressed as distance per unit time, generally as miles per hour or kilometers per hour. A general measure, speed may be observed in a variety of ways.

In characterizing the speed of a traffic stream, it is clear that some representative value should be used, as each vehicle within the stream has a different speed. For the purposes of these procedures, the average running speed is the measure which will be used. Average running speed is selected as it is easily determined from traffic stream measurements, and it is the most statistically relevant measure which is easily obtained. The average running speed is computed by taking the length of the highway segment under consideration and dividing it by the average travel time of vehicles to traverse the segment. Thus, if travel times t_1, t_2, t_3,.....t_n are measured for n vehicles traversing a segment of length, the average running speed would be:

$$S = \frac{\ell}{\sum_{i=1}^{n} t_i/n}$$

where: S = average running speed, in mph

ℓ = length of the highway segment, in miles

t_i = travel time of the ith vehicle to traverse the segment

n = number of travel times recorded

For example, if the following travel times were observed for vehicles traversing a one mile highway segment:

1.0 minutes	(.0167 hrs.)
1.2 minutes	(.0200 hrs.)
1.7 minutes	(.0283 hrs.)
1.1 minutes	(.0183 hrs.)

the average travel time would be computed as (.0167 + .0200 + .0283 + .0183)/4 or .0208 hours. The average running speed is, therefore:

S = 1.0 miles/0.0208 hours = 48.08 mph (76.9 km/h)

For capacity analysis, speeds are best measured by observing travel times over a known length of highway. The length taken may be as short as several hundred feet for ease of observation. Radar meters give point speeds which can be averaged to give a "time mean speed" which is usually 1-3 mph higher than average running speed.

(2) Volume: the number of vehicles passing a point on a highway or highway lane during one hour, expressed as vehicles per hour.

(3) Rate of Flow: the number of vehicles passing a point on a highway or highway lane during some period of time less than one hour, expressed as an equivalent rate in vehicles per hour.

The distinction between volume and rate of flow must be clearly understood. A volume represents an actual number of vehicles passing a point in one hour. A rate of flow represents the number of vehicles which would pass a point in one hour if they continued to arrive at a rate similar to that observed in a shorter time span. A rate of flow is computed by dividing the number of vehicles observed passing the point in question by the time (in hours) during which they were observed. Thus, 100 vehicles observed in a 15-minute period represents a rate of flow of:

100/0.25 hours = 400 vehicles per hour

The difference between volume and rate of flow is illustrated by the following example. The following traffic counts were made during a one hour study period:

5:00 - 5:15	1000 veh.
5:15 - 5:30	1200 veh.
5:30 - 5:45	1100 veh.
5:45 - 6:00	1000 veh.

The volume for this hour is the sum of these counts, or 4,300 vph. The rate of flow, however, varies in each 15-minute period. During the peak period, the flow rate is 1200 veh./0.25 hrs. = 4,800 vph. Note that 4,800 vehicles do not actually pass the point in question in one hour, but that they do pass the point at that rate for 15 minutes.

Consideration of peak rates of flow is of critical importance, as a freeway breakdown of several minutes may take several hours to clear up. The procedures and guidelines presented in this document are based upon uniform flow rates. This is done so that peak flow rates may be used directly, and to insure that the description of operations during these periods is meaningful.

The unit of time to be used for freeway flow rates has historically been taken as 5 minutes. In recent times, a number of practitioners have utilized 15 minutes (9) for practical reasons. Further, the results of the NCHRP-sponsored Weaving Area Operations Study (3) included the conclusion that 5 minute periods were statistically unstable. Many operational analysts, however, prefer the use of the shorter 5-minute periods, as they allow for the examination of shorter-term fluctuations in conditions. For the purposes of these procedures, either may be used, as long as the rate of flow is uniform for the period considered.

Peak rates of flow are related to volumes through the use of the peak hour factor, which is defined below:

$$PHF = \frac{\text{VOLUME (for one hour)}}{\text{PEAK RATE OF FLOW (within the hour)}}$$

Then, for 5-minute flow periods:

$$PHF = \frac{V}{12 \times N_5}$$ where 12 is the number of 5 minute periods in an hour

and, for 15-minute flow periods:

$$PHF = \frac{V}{4 \times N_{15}}$$ where 4 is the number of 15-minute periods in an hour

where: PHF = peak hour factor
V = volume (for full hour)
N_5 = maximum 5-minute count during hour of interest
N_{15} = maximum 15-minute count during hour of interest

Where the peak hour factor is known, full hour volumes may be converted to peak flow rates within the hour:

Peak Flow Rate = V/PHF

The latter conversion from full-hour volumes, which are usually given, to peak flow rates, is also quite important. As the procedures herein are based upon peak flow rates, such a conversion is often necessary before computations may begin on any particular problem.

(4) Density: the number of vehicles occupying a given length of highway or highway lane, averaged over time, usually expressed as vehicles per mile.

Density is virtually never measured directly, as this requires costly elevated or aerial photography. Density, rate of flow, and average running speed are, however, related by the equation:

$$F = S \times D$$

where: F = rate of flow, in vph
S = Average running speed, in mph or km/h.
D = density, in vpm or veh/km

Knowing rate of flow and average running speed, density can be computed from this relationship.

The parameters discussed in this section (average running speed, volume, rate of flow, peak hour factor, density) are those which are most often used in describing a traffic stream. These will be referred to throughout these procedures.

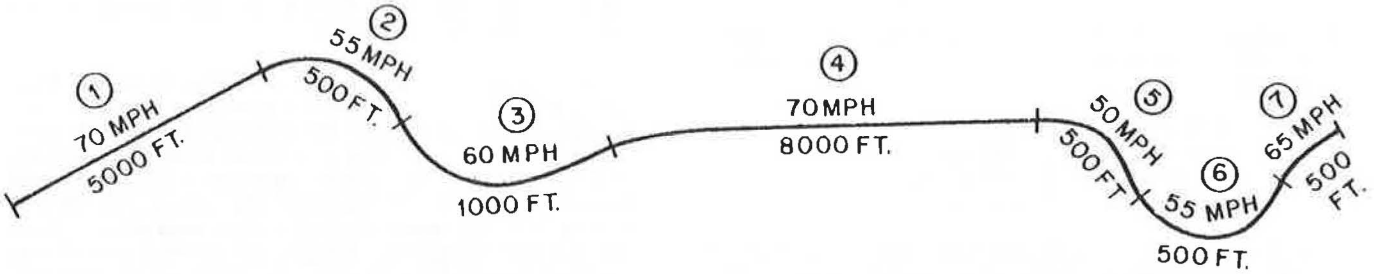
B. Capacity Terms

Freeway Capacity may be defined as the maximum rate of flow which may be accommodated by a uniform freeway segment under prevailing roadway and traffic conditions in the specified direction of interest.

Roadway Conditions refer to the geometric characteristics of the freeway segment under study, i.e., grades, number and width of lanes, lateral clearance, design speeds, configuration of lanes, curvature, etc.

Traffic Conditions refer to traffic composition, generally expressed in terms of the percentages of trucks, buses, and recreational vehicles in the traffic stream.

Capacity analysis of freeway segments of significant length is based upon Average Highway Speed (AHS). Average highway speed is the weighted average design speed of the segment, where the design speed of each component is weighted by the length of the component (straight sections are assigned a design speed of 70 mph or 112 km/h.). The computation of AHS for an extended freeway segment is illustrated in Figure 1.3.



SEGMENT	LENGTH (FT.)	DESIGN SPEED (MPH)	DESIGN SPEED X LENGTH
1	5000	70	350,000
2	500	55	27,500
3	1000	60	60,000
4	8000	70	560,000
5	500	50	25,000
6	500	55	27,500
7	500	65	32,500
	16,000		1,082,500

AHS = 1,082,500/16,000 = 67.66 MPH

NOTE: 1 mph = 1.6 km/h.
1 ft. = 0.3048 m.

FIGURE 1.3

ILLUSTRATION OF THE COMPUTATION OF AVERAGE HIGHWAY SPEED

Other terms and concepts are introduced and defined as they are used in subsequent chapters and sections of this manual.

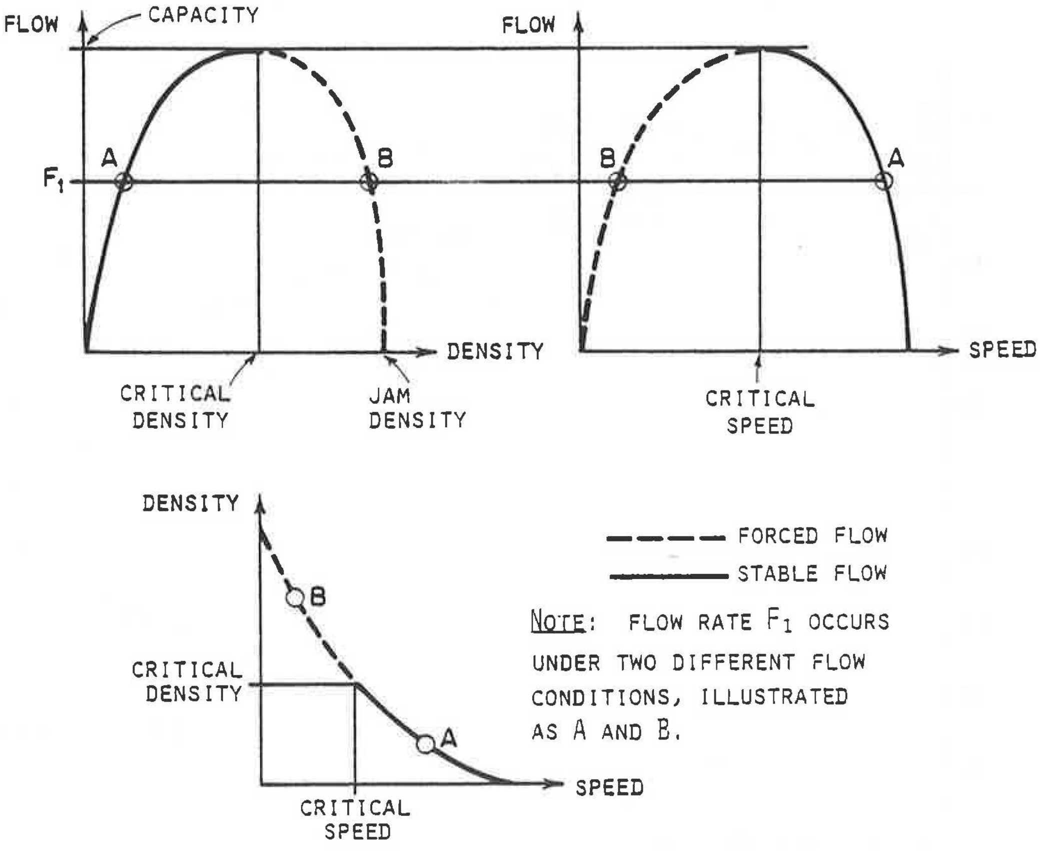
Characteristics of Freeway Flow

A. General

The relationship between the three primary parameters of freeway flow has been discussed previously. Rate of flow, average running speed, and density are macroscopic measures which are used to describe the condition of a traffic stream. Although the relationship F = S x D seems to suggest that a given rate of flow may occur at numerous combinations of speed and density, this is not true. Only a limited number of F, S, and D combinations will occur, as there are additional relationships between F and S, F and D, and S and D which control those combinations that may occur. Figure 1.4 illustrates the general form of these relationships, which are the basis for capacity analysis. The exact shape of these curves depends upon the prevailing roadway and traffic conditions which exist on any given highway segment.

Note that a rate of flow of zero occurs under two conditions: (1) when there are no cars on the road density is zero and speed is theoretically anything an individual driver would select, and (2) when the density becomes so high that all movement stops - speed is zero. This latter density is called "jam density." As figure 1.4 illustrates, any rate of flow, other than capacity, may occur under two different conditions, one of high density and low speed, and one of high speed and low density. The entire high-density, low-speed side of the curves is the region of forced, or breakdown flow. The entire stable region of flow occurs on the low-density, high-speed side of the curves.

The peak of the speed-flow and density-flow curves represents the maximum rate of flow, or capacity. The density at which this occurs is referred to as "critical density," and the speed at which it occurs is "critical speed." As capacity is approached, flow becomes more unstable, as available gaps in the traffic stream are fewer. At capacity, there are no usable gaps in the traffic stream, and any entering vehicles and/or vehicles changing lanes within the traffic stream will create a disturbance which cannot be effectively damped, or dissipated. Thus, operation of a facility at capacity is difficult to maintain for long periods of time, and forced flow becomes almost unavoidable. For this reason, freeways are designed to operate in the more stable regions of flow, at volumes less than capacity.



NOTE: FLOW RATE F_1 OCCURS UNDER TWO DIFFERENT FLOW CONDITIONS, ILLUSTRATED AS A AND B.

FIGURE 1.4

RELATIONSHIPS AMONG SPEED, FLOW, AND DENSITY

B. Freeway Flow Under Ideal Conditions

As has been noted, the calibration of any given flow relationship depends upon the prevailing roadway and traffic conditions existing in the highway segment of interest. As a basis for freeway capacity analysis, a set of conditions called "ideal conditions" has been established. These conditions include:

- 12 foot (3.7 m.) lane widths
- at least 2 lanes for the exclusive use of vehicles in each direction
- a minimum of 6 feet (1.8 m.) between the edge of the travel lanes and the nearest obstacle or object at the roadside
- no trucks, buses, or recreational vehicles in the traffic stream (i.e., only passenger cars in the stream).

Figure 1.5 illustrates the relationship between speed and flow for freeways under ideal conditions for various values of AHS. These relationships, which were estimated from the literature (9, 10) and original field studies (11) associated with this effort, are representative of urban commuter traffic streams. The user of these procedures may wish to modify the relationships for other types of traffic, such as weekend, rural, etc. The data base for Figure 1.5 was not sufficient to discern regional differences, although these may also be a factor in some cases. Individual sections of these procedures give general guidelines on the modification of procedures to account for other types of flow.

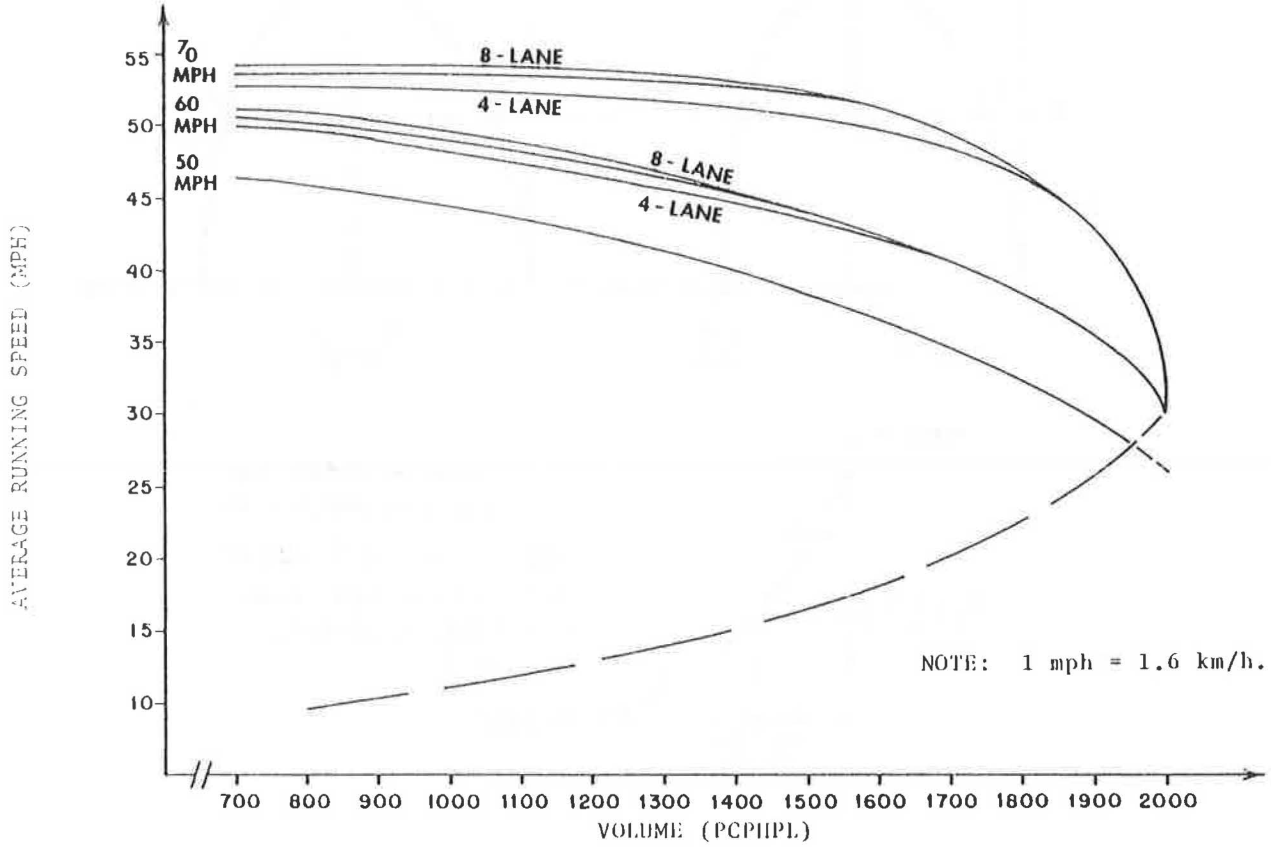
In extreme cases, users may wish to collect on-site field data and plot their own set of representative speed-flow curves. For most peak conditions, which are the principal periods for design and analysis, the curves presented in Figure 1.5 (and subsequent guidelines based upon it), are reasonably accurate, and may be used directly. There are two characteristics of the curves in Figure 1.5 that are worthy of note:

- there is a substantial range of flow over which speed is insensitive to flow, a range which extends to fairly high flow rates.
- as flow approaches capacity, speed drops off at an extremely sharp rate.

These two characteristics are of critical importance, and together they indicate that speed may not be as good an indicator of service quality as was previously indicated in the 1965 HCM, particularly for high-type 70 mph designs.

C. Factors Affecting Freeway Flow Under Ideal Conditions

Flow relationships existing under ideal conditions are altered by any prevailing conditions which are not ideal.



NOTE: 1 mph = 1.6 km/h.

FIGURE 1.5

SPEED - FLOW CHARACTERISTICS UNDER IDEAL CONDITIONS

1. Trucks, Buses, and Recreational Vehicles The presence of non-passenger cars in the traffic stream can have a marked effect on the flow characteristics of any given highway segment, due to two reasons:

- these vehicles are larger than passenger cars, and therefore occupy more roadway space.
- the operating capabilities of these vehicles (acceleration, deceleration, maintenance of speed, etc.) are generally inferior to those of passenger cars, and their presence introduces incongruities into the traffic stream, and creates gaps in the stream which cannot always be filled.

The latter effect, particularly on sustained upgrades is extremely deleterious, as heavily loaded trucks maintain extremely low speeds, creating large gaps in the traffic stream. Figure 1.6 illustrates the acceleration and deceleration characteristics of trucks on grades. The curves are representative of a "typical" truck with a weight/horsepower ratio in the range of 250-350 lbs/hp which is used as the basis for these procedures. Individual procedures contain instructions on how to estimate the impact of trucks of different typical weight/horsepower ratios on traffic streams. Figure 1.6 is partially taken from Reference 7. This is worthy of note, as there are a number of studies which have resulted in similar curves either by simulation (the most prevalent technique) or by field measurement. The results of these vary considerably, particularly with respect to final crawl speeds. The curves selected for use herein represent a rough "middle ground" among the studies available.

Illustrations 1.1 and 1.2 depict the deleterious effects of trucks and other non-passenger cars in the traffic stream.

2. Restrictive Geometrics Table 2.1, discussed in Chapter 2, depicts the effect of reduced average highway speed on speed-flow characteristics. Other restrictive geometric elements, such as narrow lanes, or roadside objects too close to the pavement edge, will also have deleterious effect on flow.

Illustrations 1.3 and 1.4 depict the restrictive effects of narrow lanes and/or restrictive lateral clearances. Narrow lanes cause vehicles to travel closer to one another laterally. Drivers tend to compensate by leaving larger headways, thus reducing flow at any given speed. Restrictive lateral clearances have much the same effect, as drivers move further out in the lane than they normally would, to put more distance between themselves and

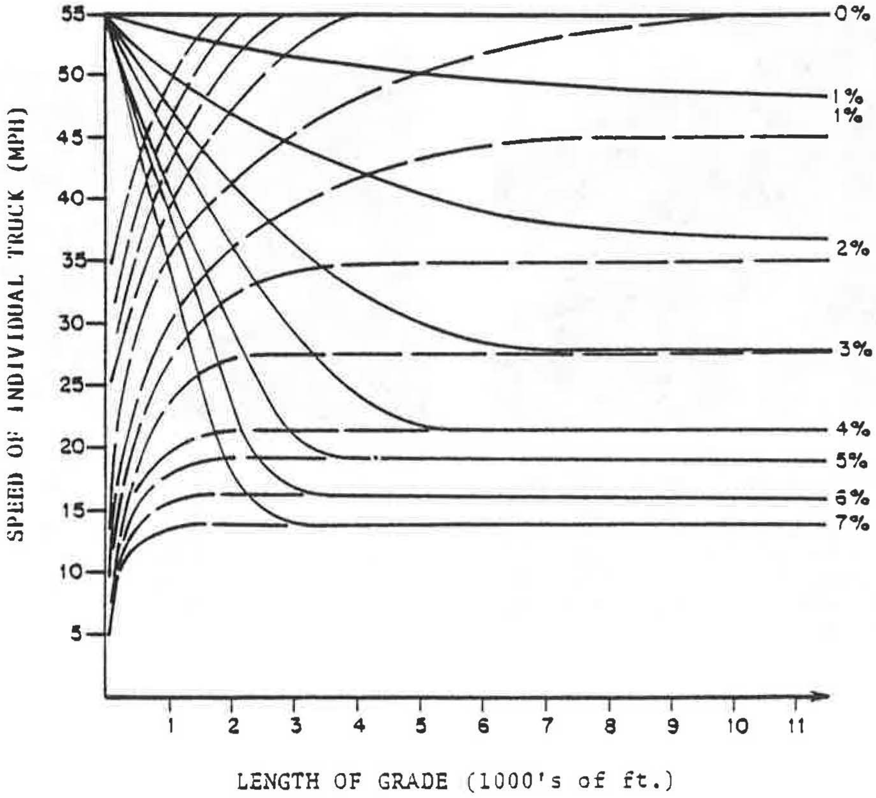


FIGURE 1.6

ACCELERATION - DECELERATION CURVES FOR A TYPICAL TRUCK (250 - 350 LBS/HP)

NOTE: 1 mph = 1.6 km/h.
1 ft. = 0.3048 m

Illustration 1.1 Note the formation of large gaps in front of slow-moving trucks climbing the upgrade.

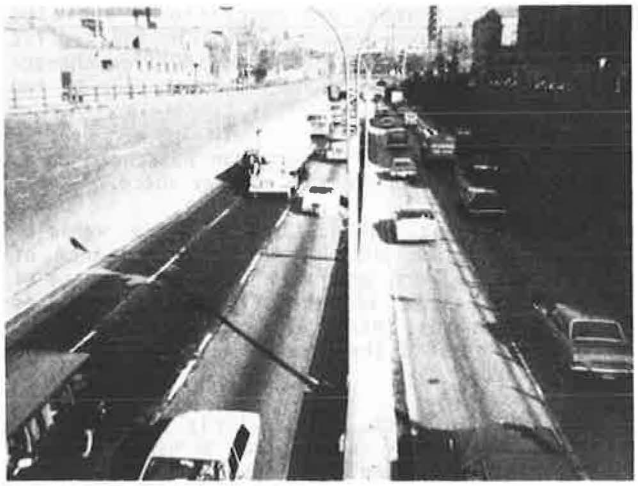


Illustration 1.2 Even on relatively level terrain, the appearance of large gaps in front of commercial vehicles is unavoidable.

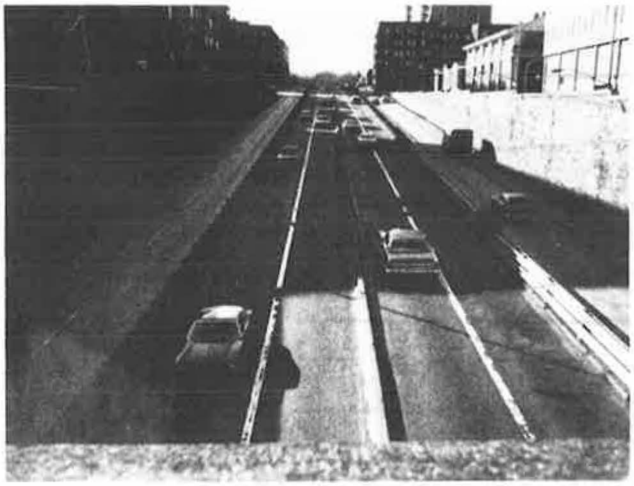


Illustration 1.3 Note how vehicles shy away from both roadside and median barriers, driving as close to the lane marking as possible. The existence of narrow lanes compounds the problem, making it difficult for two vehicles to travel alongside one another.

Illustration 1.4 In this case, vehicles shy away from the roadside barrier. Note that this causes a shift in the placement of vehicles in each lane. In all three lanes, vehicle placement is skewed towards the median.

roadside (or median) objects. Thus, vehicles again are closer to each other laterally, and drivers compensate by leaving longer headways between vehicles.

The effects of narrow lanes and/or restrictive lateral clearance impact flows throughout the range of stable speeds.

The Level of Service Concept

The 1965 HCM defines Level of Service as, "A qualitative measure of the effect of a number of factors, which include speed and travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs." It goes on to indicate that "in practice, selected specific levels are defined in terms of particular limiting values for certain of these factors."

This document follows this general principle, and like the 1965 HCM, defines six Levels of Service, A through F, for freeways. The user should take care, however, to avoid confusing the criteria defined herein, which differ considerably from those of the 1965 HCM. Service A through F, representing the best through the worst operating conditions respectively, are illustrated pictorially in Illustrations 1.5 to 1.10.

Level of Service A represents virtually completely free-flow conditions, in which the speed of individual vehicles is controlled only by driver desires and prevailing conditions, not by the presence or interference of other vehicles. Ability to maneuver within the traffic stream is unrestricted.

Levels of Service B, C, and D represent increasing levels of flow rate with correspondingly more interference between vehicles of the traffic stream. Average running speed of the stream remains relatively constant through a portion of this range, but the ability of individual drivers to freely select their speed becomes increasingly restricted as the Level of Service worsens.

Level of Service E is representative of operation at or near capacity conditions. Few gaps are available, and the ability to maneuver within the traffic stream is severely limited, and speeds are low, in the range of 30 mph. Operations at this level are unstable, and a minor disruption may cause rapid deterioration of flow into Level of Service F.

Level of Service F represents forced, or breakdown, flow. At this level, stop-and-go patterns and waves have already been set up in the traffic stream, and operations at a given point may vary widely from minute to minute, as will operations in short adjacent highway segments, as congestion waves propagate through the traffic stream. Operations at this level are highly unstable and unpredictable.

Levels of Service are defined in greater detail for each type of freeway segment in Chapters II, III, and IV.

The philosophy carried throughout these procedures is that Level of Service is a quality measure which should be defined in terms of parameters which describe the experience and perception of service quality, as seen through the eyes of the individual motorist. The parameters used should also be macroscopic and easily measured if they are to be useful to the practitioner.

Speed is one of the principal parameters which can be directly experienced by the driver, and is one which has traditionally been associated with the defining of Levels of Service. Unfortunately, the speed-flow relationships of Figure 1.5 indicate that for a large range of volumes, speed is insensitive to flow levels. Thus, while speed is clearly a principal ingredient in the perception of service quality, it cannot be the only parameter involved in defining Levels of Service.

Drivers also experience directly the proximity of other vehicles, difficulty of executing lane changes and other internal maneuvers, difficulty in entering or leaving the facility, etc. Many of these factors cannot be easily measured in a direct way, but all are generally related to density, a measure describing the number of vehicles physically present in a unit of freeway length.

For the purposes of these procedures, Levels of Service are defined using the parameters of average running speed and density. In keeping with the philosophy of the 1965 HCM, safety is introduced in a secondary manner by not allowing better Levels of Service on highways with restricted AHS.

Using speed and density, Levels of Service are defined for basic freeway segments. Levels of Service for weaving areas and ramps are based upon speed primarily, but are directly related to the definitions for basic freeway segments.

Note that these concepts are based primarily upon Level of Service as perceived by the driver. Existing information is not sufficient to allow consideration of a broader concept of service, which would treat passengers, freight, and non-user service quality as well.

The basis of capacity analysis, however, is the relationship between various flow levels and Level of Service under prevailing conditions. Computational procedures are utilized to determine these relationships. Level of Service guidelines for ideal conditions correlate flow levels with each Level of Service. The flow level for any given Level of Service is called a service volume. No service volumes are defined for Level of Service F, which is an unstable condition.

Note that a Level of Service represents a range of operating conditions. Service volume is defined as the maximum rate of flow which may be accommodated under prevailing traffic and roadway conditions while still maintaining a quality of service appropriate to the indicated Level of Service. Levels of Service are defined by speed and density. The values of service volume which are tabulated and computed herein are in accord with the speed-flow relationships depicted in Figure 1.5. The guidelines given herein reflect observed speed-flow behavior, and are not arbitrarily defined.

The Level of Service concept is central to capacity analysis. Given that it is not advisable to have freeways operate or designed to operate at capacity, it is necessary that practitioners be able to identify maximum rates of flow which can be supported under a selection of operating conditions. The Level of Service concept permits this.

Illustration 1.5 Level of Service A.

Illustration 1.6 Level of Service B.

Illustration 1.7 Level of Service C.

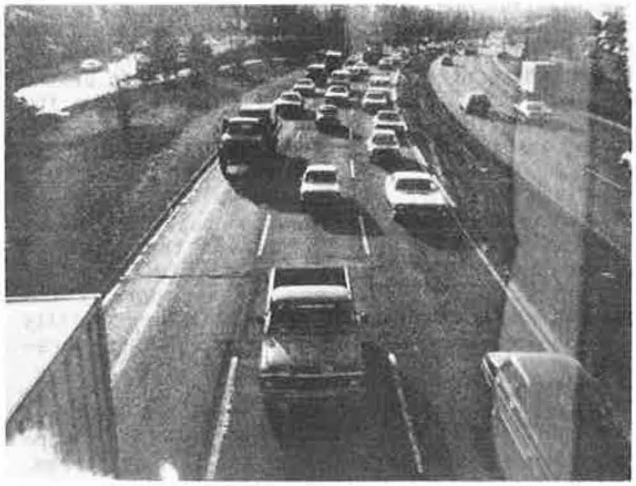


Illustration 1.8 Level of Service D.

Illustration 1.9 Level of Service E.

Illustration 1.10 Level of Service F.

Using the Freeway Capacity Procedures

In general, procedures herein may be used in two ways:

- design: given a set of forecasted demand volumes, traffic characteristics and a known set of design standards for AHS, lane width, and lateral clearance, procedures may be used to determine geometric characteristics - the number and configuration of lanes. For weaving areas and ramps, this may require some trial-and-error computations.
- analysis: given a set of known volumes, traffic characteristics, and geometrics, the situation may be analyzed to determine Level of Service, or if more detail is needed, speeds and densities.

Analysis is the most widely applicable usage, as some design will require trial-and-error analyses. Analysis is also extremely useful in evaluating the effectiveness of planned spot or segment improvements on existing facilities.

The user is cautioned that these procedures are intended to be used as a guide, and do not replace the responsibility for decision-making or selection among viable alternatives. Procedures included herein provide the user with reasonably accurate estimates of likely operating conditions, given a specified set of prevailing conditions. The engineer must still decide which trial design or improvement to select, based upon estimates of performance, cost-effectiveness, environmental impact, and other factors. These procedures do not make decisions, but hopefully provide meaningful information to engineers and planners who must.

Chapter I - References

1) Highway Capacity Manual, Bureau of Public Roads, 1950.

2) "Highway Capacity Manual," Transportation Research Board Special Report 87, Transportation Research Board, 1965.

3) Pignataro, et al, "Weaving Areas: Design and Evaluation," National Cooperative Highway Research Program Report 159, Transportation Research Board, 1975.

4) St. John, et al, "Freeway Design and Control Strategies as Affected by Trucks and Traffic Regulations," Report No. FHWA-RD-75-42, Midwest Research Institute, April 1975.

5) St. John and Kobett, "Grade Effects on Traffic Flow Stability and Capacity," NCHRP 3-19, Midwest Research Institute, August 1974.

6) Leisch, Capacity Analysis Techniques for Design and Operation of Freeway Facilities, Federal Highway Administration, 1974.

7) Review of Vehicle Weight/Horsepower Ratio as Related to Passing Lane Design, NCHRP Project 20-7, Penn State University, 1978.

8) Werner and Morrall, "Determining Passenger Car Equivalencies of Trucks, Buses and Recreational Vehicles for Rural Two-Lane Highways," to be published in Transportation Research Board in 1978-79.

9) Southern State Parkway Improvement Study, Jones Beach State Parkway Authority; Howard, Needles, Tammen, and Bergendoff, April 1977.

10) Abramson and Amster, "Testing and Evaluating Deterministic Models of Traffic Flow," Report No. 1041-1, USDOT, FHWA, November 1968.

11) Field surveys conducted on The Southern and Northern State Parkways, Nassau County, New York, April 1978.

CHAPTER II - BASIC FREEWAY SEGMENTS

I. BASIC CHARACTERISTICS

Basic freeway segments may be defined as those segments whose performance is unimpeded by the presence of nearby ramp junctions and/or weaving areas.

The 1965 Highway Capacity Manual (1) defines the general influence range of ramp terminals and weaving areas as follows:

- on-ramp terminals - 500 ft. (152 m.) upstream; 2,500 ft. (762 m.) downstream
- off-ramp terminals - 2,500 ft.(762 m.) upstream; 500 ft. (152 m.) downstream
- weaving areas - 500 ft. (152 m.) upstream; 500 ft.(152 m.) downstream (in addition to weaving area itself)

A study by Worrall, et al (2) led to the conclusion that the upstream effect of off-ramps may extend as far as 6,250 feet (1905 m.) upstream of the ramp under congested flow, and be as little as 1,750 feet (533 m.)for free-flow conditions.

These influence distances are, of course, general guidelines. The influence range experienced in any specific case is dependent upon numerous factors, including relative traffic volumes, geometrics, and other local conditions. Particularly on urban freeways, where forced flow conditions may frequently occur, a breakdown at a ramp or weaving area may influence miles of adjacent freeway. Further, these guidelines were developed for right-hand ramps. The effect of left-hand ramps may be expected to be greater than the values shown.

The definition of "basic freeway segment" is, therefore, subject to engineering judgment. Rural freeways will consist primarily of basic segments, while urban freeways may have relatively few truly "basic" sections. During periods of light flow, however, even weaving areas and segments in the vicinity of ramp junctions may behave similarly to basic segments.

Chapter V describes detailed procedures for design and analysis of overall freeway sections, accounting for the difficulty in identifying basic freeway segments.

Levels of Service

As was discussed in Chapter I, Levels of Service for basic freeway segments are based upon two parameters directly related to the road users perception of service quality:

- speed (Average Running Speed)
- proximity to other vehicles (Density)

The use of density as a second descriptor is necessitated by the characteristics of typical speed-volume curves (Chapter I, Figure 1.5) which show a broad range of volumes over which speed remains primarily constant. Further, it introduces the factor of proximity to other vehicles into the definition of service levels, a factor of great importance to the road user.

A. Criteria

The criteria for Level of Service on basic freeway segments are shown in Table 2.1.

The criteria are taken from the typical speed-volume curves in Chapter I, that is to say, they are based upon an observed relationship. Thus, for the speeds and densities shown in Table 2.1, the flow rates shown are to be expected on a typical freeway (for the base conditions of the table). To meet the criteria for a Level of Service, both the speed and density values must be met -- since the criteria are based upon observed behavior, if one is met, the other will also generally be met.

It should, however, be noted that observed speed-volume-density relationships do vary, and that observations on any particular freeway may differ somewhat from the values in Table 2.1. Table 2.1 represents typical conditions.

B. Base Conditions

Table 2.1 is developed for primarily "ideal" conditions, which consist of:

- no trucks, buses, or recreational vehicles in the traffic stream (i.e., passenger cars in the traffic stream)
- 12-foot (3.66 m.) lane widths
- no lateral obstructions closer than 6 feet (1.83 m.) to the pavement edge.

Table 2.1 accounts for the effect of AVERAGE HIGHWAY SPEED(AHS), which is the weighted average design speed for the section under consideration. The flow rates shown are for periods of uniform flow. Usually 5-minute or 15-minute periods are considered. A period usually of primary interest is the maximum 5- or 15-minute flow rate within the peak hour of the day.

C. Description of Levels of Service

Levels of Service were defined to generally fulfill the pictorial illustrations and general descriptions of Levels of Service in Chapter I. Table 2.1 values are in accordance with observed speed-volume relationships shown in Chapter I, Figure 1.5.

The service levels were further defined to represent reasonable ranges in average running speed, density, and service volume. The table has two characteristics of note:

- At Levels of Service A-C, speed is relatively insensitive to flow rate.
- At Levels of Service D and E, speed is highly sensitive to flow changes. Thus, the range of service volumes over these levels is relatively small.

TABLE 2.1

LEVEL OF SERVICE FOR BASIC FREEWAY SEGMENTS

LEVEL OF SERVICE	PERFORMANCE CRITERIA FOR LEVELS OF SERVICE		MAXIMUM SERVICE VOLUMES (ONE DIRECTION) FOR LEVELS OF SERVICE DURING UNIFORM PERIODS OF FLOW (PCPH)			
	SPEED MPH (Km/h)	DENSITY PC/mi/LN (PC/km/LN)	4-Lane (2 ea.dir)	6-Lane (3 ea.dir)	8-Lane (4 ea.dir)	EA. ADD LANE
			AHS = 70 MPH (112 km/h.)			
A	≥50(80)	≤15(9.4)	1600	2400	3280	820
B	≥50(80)	≤25(15.6)	2500	3900	5400	1350
C	≥48(77)	≤35(21.9)	3400	5100	6800	1700
D	≥40(64)	≤47(29.4)	3850	5775	7700	1925
E	≥30(48)	≤67(41.9)	4000	6000	8000	2000
F	<30(48)	>67(41.9)	-	highly variable		-
			AHS = 60 MPH (96 km/h.)			
A	*	*	*	*	*	*
B	≥45(72)	≤25(15.6)	2300	3525	4800	1200
C	≥43(69)	≤35(21.9)	3050	4575	6100	1525
D	≥38(61)	≤47(29.4)	3600	5400	7200	1800
E	≥30(48)	≤67(41.9)	4000	6000	8000	2000
F	<30(48)	>67(41.9)	-	highly variable		-
			AHS = 50 MPH (80 km/h.)			
A	*	*	*	*	*	*
B	*	*	*	*	*	*
C	≥40(64)	≤35(21.9)	2800	4200	5600	1400
D	≥35(56)	≤47(29.4)	3300	4950	6600	1650
E	≥30(48)	≤67(41.9)	4000	6000	8000	2000
F	<30(48)	>67(41.9)	-	highly variable		-

* Level of Service not achievable due to reduced safety on highways with restricted AHS

Level of Service A is in the category of free flow operation. Average running speeds of 50 mph (80 km/h) and above prevail on freeways with 70 mph (112 km/h) AHS. Vehicles are almost unimpeded in their ability to maneuver within the traffic stream, and to enter and leave it at ramps. The average spacing between vehicles is approximately 330 feet(101 m.), or 16 car lengths, affording the driver a high level of physical and psychological comfort. The effects of incidents or point breakdowns in traffic are easily absorbed at this level, and while they will deteriorate the Level of Service in the vicinity of the incident, standing queues will generally not form, and traffic quickly returns to Level of Service A on passing the incident.

Level of Service B may also be considered to be free flow. Average running speeds of 50 mph (80 km/h) or greater still prevail on freeways with 70 mph (112 km/h) AHS, though vehicles are more closely spaced, at about 191 feet (58 m.) or 10 car lengths. The ability to make lane changes, or to enter or leave the traffic stream is somewhat restricted, but not at all difficult. The level of physical and psychological comfort provided is still high. The effects of minor incidents and point breakdowns are still easily absorbed, though the local deterioration in service may be more severe than at Level of Service A.

Level of Service C provides for stable operation, but deterioration of service as volume increases occurs quickly in this range. Vehicles still maintain a good average running speed, 48 mph (77 km/h) for freeways with 70 mph (112 km/h) AHS, but freedom to maneuver within the traffic stream is clearly restricted. Average spacing between vehicles is about 6 to 7 car lengths, or 130 feet (40 m.). The proximity to vehicles in adjacent lanes also becomes noticeably restrictive. Incidents and point breakdowns are not easily absorbed, unless they are minor in nature. Queues may be expected to form behind any significant blockage.

Level of Service D borders on unstable flow. Speeds in the range of 40 mph (64 km/h) can be maintained on highways with AHS = 70 mph (112 km/h) if no incidents occur. The ability to maneuver within the traffic stream is severely restricted, as the average spacing between cars is reduced to about 5 car lengths or 92 feet (28 m.) and gaps in adjacent lanes are infrequent. Minor incidents or breakdowns may cause extensive queuing.

Level of Service E describes capacity operation, and is quite unstable. Speeds of about 30 mph (48 km/h) prevail, but there are virtually no usable gaps in the traffic stream. Because of this, any vehicle entering the traffic stream or attempting to change lanes will cause a disturbance which the traffic stream cannot easily absorb. Incidents and breakdowns will result in immediate and extensive queue buildup, as the traffic stream does not have sufficient flexibility to dampen even minor disturbances to flow. Vehicles are spaced at an average of 60 feet (18.3 m.) or three car lengths.

Level of Service F represents forced, or breakdown, flow. Once demand is such that density rises above 67 PC/mi/LN(41.9 PC/km/LN), it is virtually impossible to maintain uniform moving flow. Conditions will vary considerably from minute to minute, as traffic is brought to a halt, and then moves surprisingly well for a short distance before again being stopped. This condition is highly unstable, and it is impossible to define representative parameters. Speeds vary widely, but will generally range below 30 mph (48 km/h) as an average.

D. Use of Criteria

1. The Peak Flow Rates Table 2.1 is based upon uniform flow rates which are analogous to full hours in which the peak hour factor is 1.00.

When using the criteria of Table 2.1 for design purposes, the table must be entered with the peak flow rate during the design hour, which can be computed as:

$$\text{Peak Flow Rate (pcph)} = \frac{\text{Directional Design Hour Volume (pcph)}}{\text{PHF}}$$

Specification of criteria in this way enables them to be used to describe variations in service quality that regularly occur in periods of time less than one hour. For example, if a 6-lane freeway with 70 mph (112 km/h) AHS had a flow rate of 5,500 pcph for 15 minutes, and a flow rate of 5,000 pcph for the rest of the hour, this could be described as Levels of Service D and E respectively (Table 2.1), rather than trying to describe it as a single level for the entire hour.

2. Analysis. Table 2.1 may be entered with any uniform flow rate for any period desired. A uniform flow rate is the rate of flow for a given time interval, where the rate of flow does not vary significantly for sub-periods of the given interval. Thus, if from 2:00-2:15 PM a uniform flow rate of 2000 pcph is observed on a freeway, the table would be entered with that value to obtain the Level of Service for that 15-minute period. The table should NOT be entered with a non-uniform flow rate. Consider the following observed flows on an 8-lane freeway:

time	count	flow rate	level of service
5:00-5:10	1000	6000	C
5:10-5:20	1100	6600	C
5:20-5:30	1300	7800	E
5:00-5:30	3400	6800	C

In the previous example, if the flow rate during the full 1/2 hour is considered in aggregate, the Level of Service would be described as 3 (Table 2.1). It can be seen, however, that during the 1/2-hour, two distinct Levels of Service have existed. Where feasible, Levels of Service should be described for UNIFORM FLOW PERIODS, that is, periods of time during which the flow rate does not significantly vary.

If an analysis of the worst existing Level of Service is desired, the table should be entered with the peak flow rate, which is computed as:

$$\text{Peak Flow Rate (pcph)} = \frac{\text{Actual Peak Hour Volume (pcph)}}{\text{PHF}}$$

3. Design AASHTO (3,4) currently specifies the use of Level of Service B in rural design and Levels of Service C and D in urban design. These are based upon the definitions of Level of Service found in the 1965 HCM. Design usage of the Level of Service criteria herein must be approached differently.

Particularly for freeways with a 70 mph (112 km/h) average highway speed, the difference between the maximum volume which can be accommodated at Level of Service 3 and capacity is slight. Given the margin of error in standard traffic forecasting techniques, design at the maximum boundaries of Levels of Service 3, 4 and 5 (capacity) as described herein is not advisable for highways with AHS = 70 mph (112 km/h). Design should be restricted to Levels of Service 1 and 2 in Table 2.1, with level 3 also being acceptable for highways with AHS = 50 mph (80 km/h) or 60 mph (96 km/h).

Because of the parabolic shape of observed speed-volume curves (see Figure 1.5), and because the better Levels of Service encompass large volume ranges, the designer may wish to perform a design for a condition WITHIN one of the Levels of Service, rather than at a boundary condition. To aid the designer and present him with a wider range of design options, Table 2.2 has been developed. It shows, for uniform increments in the v/c ratio, the average running speed and Level of Service which would result. Note that all of the useful design values, including a v/c of 0.80 fall within Level of Service 3 or better, except for AHS = 50 mph (80 km/h) and AHS = 60 mph (96 km/h), 8-lane freeways. It is recommended that NO design be attempted with a v/c greater than 0.80, and this only in extraordinary cases.

The table is useful in that a design might be attempted on a highway with AHS = 70 mph (112 km/h) with a v/c of 0.80, which is WITHIN Level of Service 3, whereas the BOUNDARY condition for Level of Service 3 is too close to capacity to be used for design. Further, the large volume range in Level of Service 2 may make it desirable to design at some intermediate level, without going all the way to Level of Service 1.

TABLE 2.2

V/C* VALUES FOR USE IN DESIGN

AHS	V/C RATIO	AVERAGE RUNNING SPEED, DENSITY, AND LEVEL OF SERVICE RESULTING								
		4-Lane			6-Lane			8-Lane		
		Speed mph	Density (PC/mi/LN)	LOS	Speed mph	Density (PC/mi/LN)	LOS	Speed mph	Density (PC/mi/LN)	LOS
70	0.200	52	7.6	1	54	7.4	1	54	7.4	1
	0.400	- average maximum value for Level of Service 1 -**								
	0.600	52	23.1	2	53	22.6	2	54	22.6	2
	0.650	- average maximum value for Level of Service 2 -**								
	0.800	49	32.7	3	51	31.4	3	51	31.4	3
60	0.200	50	8.0	2	51	7.8	2	51	7.8	2
	0.400	49	16.3	2	50	16.0	2	51	15.6	2
	0.576	- average maximum value for Level of Service 2 -**								
	0.600	47	25.5	3	48	25.0	2	48	25.0	2
	0.763	- average maximum value for Level of Service 3 -**								
	0.800	- results in Level of Service 4; DO NOT USE IN DESIGN								
50	0.200	47	8.5	3	47	8.5	3	47	8.5	3
	0.400	46	17.4	3	46	17.4	3	46	17.4	3
	0.600	43	27.9	3	43	27.9	3	43	27.9	3
	0.700	- average maximum value for Level of Service 3 -**								
	0.800	- results in Level of Service 4; DO NOT USE IN DESIGN								

* Volume-to-capacity ratio

** Average v/c for 4-, 6-, and 8-lane freeways of boundary condition.

NOTE: 1 mph = 1.6 km/h - 1 PC/mi/LN = 0.63 PC/km/LN

To convert a v/c value drawn from Table 2.2 to a maximum service volume (MSV) analogous to the MSV values of Table 2.1 for boundary conditions, the following formula may be used:

$$MSV = 2000\ (v/c)\ (N)$$

where N is the number of lanes in one direction. Note that a maximum service volume computed as above is representative of a PHF of 1.00, as are values drawn from Table 2.1.

Table 2.2 also shows average v/c ratios for boundary conditions. Values are "average," as exact boundary v/c ratios differ for 4-6- and 8-lane freeways. Thus, in design, Table 2.2 may be used directly, whether design is to be attempted at a boundary condition or for some intermediate condition within the Level of Service.

4. Driver Population Tables 2.1 and 2.2 are based upon observed speed-flow relationships representing primarily urban and suburban commuter traffic. It has been generally observed that other driver populations behave quite differently, although there does not yet exist a sufficient body of data to calibrate additional curves.

Of particular interest is weekend traffic, which consists of substantial numbers of infrequent drivers in addition to those who also commute by car. Weekend trip purposes are far more varied than during commuting hours, and include recreation, shopping, cultural, and social trips as well as others. In general, weekend motorists will use a freeway less efficiently than commuters, that is to say, for any given speed, the volume accommodated will be less than that shown in Table 2.1. There is even some question as to whether weekend flow rates approach 2000 pcphpl on ideal freeways.

The extent of the reduction in service volumes due to weekend traffic varies according to local conditions, and again, there is little available data to quantify this effect. In using Table 2.1, it is recommended that the maximum service volume be reduced by 10 to 15 percent where weekend traffic is being considered. There is some evidence, particularly from California, that reductions for weekend traffic may be even larger than this.

Drivers in predominantly rural areas also differ significantly from commuters, but even less is known about rural traffic characteristics than is known about weekend traffic. The question is not as significant in this case, as rural highways do not often experience worse than Level of Service C. Any inaccuracy in Table 2.1 values in this range would not have a significant operational effect if used as is.

Factors Affecting Service Volumes and Level of Service

Tables 2.1 and 2.2 allow the determination of a maximum service volume for a given Level of Service which is achieved under ideal conditions consisting of a) 12-foot (3.66 m.) minimum lane widths, b) no lateral obstructions closer than 6 feet (1.83 m.) to the pavement edge and c) no trucks, buses, or recreational vehicles in the traffic stream. Any existing or design conditions which vary from these base conditions require a downward adjustment in the maximum service volume.

Adjustments are made through the application of multiplicative adjustment factors which convert maximum service volume (MSV) to an actual service volume (SV).

A. Adjustment for Lane Width and Lateral Clearance (W)

Lane widths which are narrower than 12 feet (3.66 m.) have a restrictive effect on traffic. Vehicles are forced to travel closer to each other (laterally) than normally. Drivers compensate for this by driving more cautiously and allowing greater longitudinal spacing between vehicles.

Lateral obstructions produce much the same effect. Objects close to the roadway edge at the roadside or in the median cause drivers in lanes adjacent to the obstruction to position themselves further away from the roadway edge than normal. This causes the lateral distance between vehicles to be reduced, just as in the case of narrow lanes.

Considerable judgment must be exercised in identifying actual lateral obstructions. They may be continuous, such as a retaining wall, concrete median barrier, or certain types of guardrail, or they may be periodic, such as light poles or bridge abutments. In some cases, drivers may become accustomed to lateral obstructions, in which case, their effect becomes negligible. Certain types of guardrail, even when closer to the pavement edge than 6 feet (1.83 m.), do not cause drivers to "shy away" from the pavement edge. The safety-type median barrier is a good example of this.

The 1965 HCM indicated that "low" barriers (smaller than 6" or 15.2 cm. in height) often did not influence driver behavior. This, again, is subject to some judgment. The low median barrier of Illustration 2.1 below is of a very dangerous type, and does cause drivers to "shy away" as much as is possible. The extent of the influence of low curbs varies according to what is on the other side of the curb, the width of the lane it borders, and other factors. Many modern designs use a 13-foot lane anywhere it is edged with a barrier curb. This effectively eliminates any impact of the curb on traffic. Current AASHTO design standards do not recommend use of barrier curbs on freeways.

Illustrations 2.1 and 2.2 show cases where lateral obstructions and lane width restrictions are evident. In the first case, median and roadside obstructions are at the pavement edge. Lane widths of 10 feet also exist in this case. In the second case, roadside obstructions exist 2-3 feet from the pavement edge with a 12-foot lane width. The type of median barrier shown here constitutes an obstruction, due to the use of barrier curbs without lane-widening.

Illustrations 2.3 and 2.4 depict freeways free of lane width and/or lateral obstruction restrictions. In both cases, 12-foot lanes are provided, as is a shoulder clear of objects for more than 6 feet. (1.83 m.). While median barriers are closer to the pavement than 6 feet (1.83 m.), neither design produces any observable "shying away" on the part of drivers, and would therefore not be considered to be lateral obstructions.

The multiplicative adjustment factors for restricted lane width and reduced lateral clearance are discussed under "Computational Procedures."

B. Adjustments for the Presence of Trucks, Buses, and Recreational Vehicles in the Traffic Stream

Maximum service volumes shown in Tables 2.1 and/or 2.2 are for a traffic stream consisting only of passenger cars. There are three general categories of vehicles which have markedly different operating characteristics than those of passenger cars:

- trucks
- buses
- recreational vehicles

The presence of any of these vehicles in the traffic stream will reduce the actual service volume due to their size, operating characteristics and interactions with other vehicles. Note that 4-wheeled, 2-axle vans may be considered to be passenger cars.

These vehicles affect operations in a number of ways, principally because they introduce vehicles with different operating capabilities into the traffic stream. It is the interaction of vehicles with widely varying operating characteristics that produces most of the reduction in service volume, not the fact that they are larger, and therefore occupy more space.

Procedures for adjusting computations to account for the presence of trucks, buses, and recreational vehicles in the traffic stream are discussed under "Computational Procedures."

II. COMPUTATIONAL PROCEDURES

General Equation

Computational procedures for basic freeway segments are relatively straightforward, and involve a single equation:

$$SV = MSV \times W \times Q$$

where

SV = service volume under prevailing traffic and roadway conditions for the Level of Service under consideration

MSV = maximum service volume under ideal conditions, taken directly from Table 2.1 or computed from Table 2.2 using the equation:

$$MSV = 2000 \times N \times V/C$$

Illustration 2.1 Lane width and lateral clearance problems are evident on this old section of parkway.

Illustration 2.2 Lateral obstructions at the roadside are 2 to 3 feet from the pavement on this freeway. The median barrier also presents an obstruction.

Illustration 2.3 Ideal geometric conditions are evident here. The safety-type median barrier does not pose an obstruction for capacity purposes.

Illustration 2.4 Another freeway with ideal geometrics is shown here. The type of median barrier used does not constitute a lateral obstruction, primarily because the median lane is widened to 13 feet.

W = adjustment factor for the combined effect of restricted lane widths and lateral clearance problems

Q = adjustment factor for the combined effect of trucks, buses, and recreational vehicles in the traffic stream

N = number of lanes in one direction

V/C = volume to capacity ratio.

Service volumes so computed are representative of a PHF of 1.00, or peak flow rates within the hour under consideration. In design the service volume (S/V) is directional design hour volume (DDHV), adjusted to represent a peak rate of flow.

Design Procedure

The steps for using the procedures are described herein. In design, geometrics, such as alignment, lane widths and lateral clearances, are set by the design standards being used. The designer uses capacity procedures to determine the number of lanes which will be required to provide the desired Level of Service. For design, Table 2.2 is used, with the designer selecting a V/C ratio commensurate with the intended design Level of Service, at either a boundary or intermediate point.

1) Select values for design: V/C ratio, average highway speed, lane width, lateral clearance, grades, etc.

2) Adjust the Directional Design Hour Volume (DDHV) to represent the peak flow rate by dividing by the PHF. This is the value of service volume used in computations:

$$SV = DDHV/PHF$$

3) Find correction factors W and Q (if needed). Note that factor Q may be slightly different for 4- and 6- or 8-lane freeways. A detailed discussion of these factors is given below.

4) Insert the values found in steps 2 and 3 in the equation:

$$SV = 2000 \times N \times V/C \times W \times Q$$

and solve for N, the number of lanes required for <u>one</u> direction of the freeway:

$$N = SV/(2000\ V/C\ W\ Q)$$

If the design indicates the possible need for a truck climbing lane, the operation of the facility as a whole should be checked using the technique specified for truck climbing lanes, which is discussed later.

Analysis Procedure

In analysis, actual traffic volumes and geometrics are generally known, and the Level of Service is to be found, using the following steps:

1) Convert the actual volume (demand) to the service volume by dividing the PHF. This is the actual SV to be used in computations.

$$SV = \text{demand volume}/PHF$$

2) Find correction factors W and Q, using techniques discussed below.

3) Compute the MSV provided by the facility under analysis:

$$MSV = SV/(W \times Q)$$

4) Compare computed MSV to MSV in Table 2.1 to determine the Level of Service at which the facility is operating.

Where a truck climbing lane is present, a Level of Service must be assumed. The number of trucks using the lane are then subtracted from mixed lanes, and the Level of Service on the mixed lanes is determined using remaining traffic. This is repeated until the assumed Level of Service is equal to that computed. A special technique for treatment of truck climbing lanes is discussed later in this chapter.

In analysis, it is also possible to determine Level of Service for periods other than peak flow. Any period of uniform flow may be analyzed by using the flow rate of interest instead of the peak flow rate in step 1.

Finding W, Correction Factor for Lane Width and Lateral Clearance

The factor for adjusting MSV to reflect the combined effect of reduced lane width and/or restricted lateral clearance is found in Table 2.3. The factors in this table are drawn from the 1965 HCM, as there has been no research on this topic to enable an updating.

The effect of reduced lane width and restricted lateral clearance is most severe on 4-lane freeways and less severe on 6- and 8-lane freeways.

The table is read directly, except in the case of a lateral obstruction on both sides of the freeway where the left-side obstruction and the right-side obstruction are at different distances from the pavement edge. In such cases, the table is entered with an obstruction on both sides at a distance which is the average of the two obstructions. For example, if there is an obstruction 3 feet from one side and another 5 feet from the other side, the table would be entered with an obstruction on both sides, 4 feet from the pavement edge. Interpolated values can be taken from the table in cases of intermediate lane widths, eg. 8.5 ft., etc.

Finding Q, Correction Factor for the Combined Effect of Trucks, Buses, and Recreational Vehicles

The procedure for adjusting maximum service volume to reflect mixed traffic streams is carried out in two steps.

- finding the PASSENGER CAR EQUIVALENT of each truck, bus, or recreational vehicle for the traffic and roadway conditions under consideration - the passenger car equivalent (E_T, E_B, or E_R) for trucks, buses, and recreational vehicles respectively represents the number of passenger cars which would utilize the SAME PERCENTAGE OF THE ROADWAY'S CAPACITY as one

TABLE 2.3

CORRECTION FACTOR FOR THE COMBINED EFFECT OF RESTRICTED LANE WIDTH AND LATERAL CLEARANCE (W)

Distance From Edge of Traveled Way To Obstruction*	Adjustment Factor (W)							
	Obstruction on One Side of One-Direction Roadway				Obstruction on Both Sides of One-Direction Roadway			
	Lane Width							
ft. (m.)	12 ft. (3.7m.)	11 ft. (3.4m.)	10 ft. (3.0m.)	9 ft. (2.7m.)	12 ft. (3.7m.)	11 ft. (3.4m.)	10 ft. (3.0m.)	9 ft. (2.7m.)
4-lane divided highway, (2 lanes each direction)								
≥6 (1.8)	1.00	0.97	0.91	0.81	1.00	0.97	0.91	0.81
5 (1.5)	0.99	0.96	0.90	0.80	0.99	0.96	0.90	0.80
4 (1.2)	0.99	0.96	0.90	0.80	0.98	0.95	0.89	0.79
3 (0.9)	0.98	0.95	0.89	0.79	0.96	0.93	0.87	0.77
2 (0.6)	0.97	0.94	0.88	0.79	0.94	0.91	0.86	0.76
1 (0.3)	0.93	0.90	0.85	0.76	0.87	0.85	0.80	0.71
0 (0.0)	0.90	0.87	0.82	0.73	0.81	0.79	0.74	0.66
6- or 8-lane divided highway, (3 or 4 lanes each direction)								
≥6 (1.8)	1.00	0.96	0.89	0.78	1.00	0.96	0.89	0.78
5 (1.5)	0.99	0.95	0.88	0.77	0.99	0.95	0.88	0.77
4 (1.2)	0.99	0.95	0.88	0.77	0.98	0.94	0.87	0.77
3 (0.9)	0.98	0.94	0.87	0.76	0.97	0.93	0.86	0.76
2 (0.6)	0.97	0.93	0.87	0.76	0.96	0.92	0.85	0.75
1 (0.3)	0.95	0.92	0.86	0.75	0.93	0.89	0.83	0.72
0 (0.0)	0.94	0.91	0.85	0.74	0.91	0.87	0.81	0.70

* Certain types of obstructions, high-type median barriers in particular, do not cause any deleterious effect on traffic flow. There is some evidence that driver reaction to barriers is based in part on their perception of likely vehicle damage should the barrier be hit. Concrete median barriers, which generally restrict damage to scuffed tires, are usually not a problem.

truck, bus, or recreational vehicle for prevailing roadway and traffic conditions.

- using the values for E_T, E_B, and/or E_R, a single multiplicative correction factor for the COMBINED EFFECT of trucks, buses, and recreational vehicles in the traffic stream is calculated.

A. Passenger Car Equivalents for General Freeway Segments

It is often possible to consider an extended length of freeway containing upgrades, downgrades, and level segments. This can be done where no one grade is long enough to have a significant impact on the operation of the general section. As a rule, general section analysis can be used where no one grade of 3% or greater is longer than approximately 1/4 mile (402 m.) or longer than 1/2 mile (804 m.) for grades of less than 3%.

To use extended section analysis, the freeway must be classified by type of terrain:

- Level Terrain: any combination of grades, length of grades, horizontal or vertical alignment which permits trucks to maintain approximately the same speed as passenger cars. This generally includes grades no greater than 1%.

- Rolling Terrain: Any combination of grades, length of grades, horizontal or vertical alignment which causes trucks to reduce their speeds substantially below those of passenger cars, but which does not cause trucks to operate at crawl speeds for any significant length of time.

- Mountainous Terrain: any combination of grades, length of grades, horizontal or vertical alignment which causes trucks to operate at crawl speed for a significant distance or at frequent intervals.

Table 2.4 shows values of E_T, E_B, and E_R for extended sections of level, rolling, or mountainous terrain. Values of E_T and E_B are drawn from the 1965 HCM; values of E_R are estimates based upon References 5 and 6.

B. Passenger Car Equivalents for Specific Grades of Significant Length

Any grade of more than 1/4 mile (402 m.) (for grades of more than 3%) or 1/2 mile (804 m.) (for grades of 3% or less) should be treated separately. On such grades, the effect of trucks, buses, and recreational vehicles is intensified as significant gaps form due to nonuniform vehicle operating characteristics. A single grade of significant length can seriously reduce the maximum service volume, and may present a severe constriction or "bottleneck," even though the same number of lanes is provided upstream and downstream of the grade.

Passenger car equivalents for trucks and recreational vehicles on specific grades are primarily based upon a simulation study of mixed vehicle flow conducted at the Midwest Research Institute(5). A study of vehicle operating characteristics conducted at Penn State(6) was also used. Passenger car equivalents for buses are the same as those in the 1965 HCM, as there is virtually no information available upon which to update them. Equivalents are calibrated in such a way that they represent the NUMBER OF PASSENGER CARS WHICH WOULD UTILIZE THE SAME PERCENTAGE OF THE CAPACITY OF A ROADWAY AS ONE TRUCK, BUS, OR RECREATIONAL VEHICLE UNDER PREVAILING ROADWAY AND TRAFFIC CONDITIONS.

Tables 2.5, 2.6, and 2.7 show the passenger car equivalents for buses, trucks, and recreational vehicles respectively on UPGRADES of significant length.

In general, if the downgrade is not so severe as to cause trucks to shift into a low gear, the downgrade may be considered as if it were an extended section of level terrain. Reference 7, based upon the MRI simulations, suggests that for grades less than 4%, or shorter than 3000 ft., such an approach is reasonable. Where downgrades do cause vehicles to shift into lower gears, the passenger car equivalent is best estimated by taking field measurements of speed, and using the equivalent for a comparable upgrade condition. Reference 7 provides a rough procedure for evaluation of downgrades.

For the most part, the passenger car equivalents for trucks in Table 2.6 are lower than those in the 1965 HCM, except for small percentages of trucks where they are higher. In general, as the percentage of trucks increases, the passenger car equivalent decreases. The effect of each truck on traffic is highest when only a few are present. As truck

TABLE 2.4

PASSENGER CAR EQUIVALENTS ON EXTENDED FREEWAY SECTIONS*

FACTOR	TYPE OF TERRAIN		
	Level	Rolling	Mountainous
E_T for trucks	2	4	8
E_B for buses	1.6	3	5
E_R for rec.veh.	2	3	4

* At Level of Service A, these values are highly variable - Table 2.4 values are used as averages for this condition.

TABLE 2.5

PASSENGER CAR EQUIVALENTS FOR BUSES ON UPGRADES OF SIGNIFICANT LENGTH

Grade %	Passenger Car Equivalent, E_B	
	Level of Service A-C	Level of Service D-E
0-3	1.6	1.6
4*	1.6	1.6
5*	4	2
6*	7	4
7*	12	10

* use generally restricted to grades over 1/4 mile long

TABLE 2.6

PASSENGER CAR EQUIVALENTS OF TRUCKS ON UPGRADES OF SIGNIFICANT LENGTH

GRADE (%)	LENGTH (mi)*	PASSENGER CAR EQUIVALENT (E_T)															
		4-Lane Freeways (2 lanes each direction)								6- or more-Lane Freeways (3 or more lanes in each direction)							
		Percent Trucks															
		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
0	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0 -1/4	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/4-1/2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/2-3/4	4	4	4	4	3	3	3	3	4	4	4	3	3	3	3	3
	3/4- 1	5	4	4	4	3	3	3	3	5	4	4	4	3	3	3	3
	1 -1-1/2	6	5	5	5	4	4	4	3	6	5	5	4	4	4	3	3
	>1-1/2	7	5	5	5	4	4	4	3	7	5	5	5	4	4	3	3
2	0 -1/4	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4-1/2	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
	1/2-3/4	8	6	6	5	5	4	4	4	8	6	6	5	5	4	4	4
	3/4- 1	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1 -1-1/2	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5
	>1-1/2	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
3	0 -1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4-1/2	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	1/2-3/4	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	3/4- 1	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	>1	14	10	10	9	8	8	7	7	12	9	9	8	7	7	7	7
4	0 -1/4	7	5	5	5	4	4	4	4	7	6	6	5	4	4	3	3
	1/4-1/2	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	1/2-3/4	13	9	9	8	7	7	7	7	11	9	9	8	7	6	6	6
	3/4- 1	15	10	10	9	8	8	8	8	12	10	10	9	8	7	7	7
	>1	17	12	12	1	9	9	9	9	13	10	10	9	8	8	8	8
5	0 -1/4	8	6	6	6	5	5	5	5	8	7	7	6	5	5	5	5
	1/4-1/2	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	1/2-3/4	20	15	15	14	11	11	11	11	14	11	11	10	9	9	9	9
	>3/4	22	17	17	16	13	13	13	13	17	14	14	13	12	11	11	11
6	0 -1/4	9	7	7	7	6	6	6	6	10	7	7	6	5	5	5	5
	1/4-1/2	17	12	12	11	9	9	9	9	13	10	10	9	8	8	8	8
	>1/2	28	22	22	21	18	18	18	18	20	17	17	16	15	14	14	14

* where the length of grade is a boundary value, always use the longer length range

percentages increase, trucks tend to concentrate in right-hand lanes, producing more uniform flow, and DECREASING the effect of each truck on the traffic stream. This should not be confused with the total effect of all trucks on traffic, which INCREASES with increasing truck percentages.

The equivalents in Table 2.6 are based upon of trucks with average weight/horsepower ratios in the range of 250 to 350 lbs./hp. The literature(5,6,7,8,9) indicates that this range is representative of normally-occurring truck populations.

In some cases, the designer or analyst may have reason to believe that the truck population would be different from what normally occurs. On freeways having a significant percentage of farm vehicles weight to horsepower ratios will be higher than 350. Where small single-unit trucks prevail, ratios lower than 250 might be representative. For this reason, Appendix 1 to this chapter includes tables for E_T for non-standard truck populations with weight to horsepower ratios of:

- ≥ 350 lbs/hp
- 150 lbs/hp

Weight/horsepower ratios in the first group are rarely higher than 600 lbs/hp.

Recreational vehicles include motor homes, motor campers, camper and boat trailers and others. These vary widely, but typically have weight to horsepower ratios of 60 lbs./hp.; assuming maximum performance of the recreational vehicle. As the use of recreational vehicles has increased dramatically over the past five to ten years, their effect on recreational routes has also increased. The provision of passenger car equivalents in Table 2.7 for them enables the designer or analyst to account for their presence in the traffic stream.

TABLE 2.7

PASSENGER CAR EQUIVALENTS OF RECREATIONAL VEHICLES ON UPGRADES OF SIGNIFICANT LENGTH

GRADE (%)	LENGTH (mi)*	PASSENGER CAR EQUIVALENT (E_R)															
		4-Lane Freeways (2 lanes each direction)								6- or more Lane Freeways (3 or more lanes each direction)							
		Percent Recreational Vehicles															
		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
0-2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0 -1/4	3	2	2	2	2	2	2	2	3	2	2	2	2	2	2	2
	1/4-1/2	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
	1/2-3/4	6	4	4	3	3	3	3	3	5	4	4	3	3	3	3	3
	3/4- 1	7	5	5	4	4	4	4	4	6	5	5	4	4	4	4	4
	>1	7	5	5	5	4	4	4	4	6	5	5	5	4	4	4	4
4	0 -1/4	5	4	4	4	4	4	3	3	5	4	4	4	3	3	3	3
	1/4-1/2	7	5	5	5	4	4	4	4	6	5	5	4	3	3	3	3
	1/2-3/4	8	6	6	5	4	4	4	4	6	5	5	4	3	3	3	3
	3/4- 1	9	7	7	6	5	5	4	4	7	6	6	5	4	4	4	4
	>1	9	7	7	6	5	5	4	4	7	6	6	5	4	4	4	4
5	0 -1/4	5	4	4	4	4	4	3	3	5	4	4	4	4	4	3	3
	1/4-1/2	8	6	6	6	5	5	4	4	7	6	6	5	4	4	4	4
	>1/2	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
6	0 -1/4	5	4	4	4	4	4	3	3	5	5	5	5	4	4	3	3
	1/4-1/2	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
	>1/2	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5

* where the length of grade is a boundary value, always use the longer length range.

C. Passenger Car Equivalents for Trucks on Composite Upgrades

The case of trucks on a composite upgrade is an interesting one. If a 3% grade of 1/2 mile (2640 feet, 804 m.) is followed immediately by a 5% grade of 1/2 mile (2640 feet, 804 m.), what value of E_T should be used?

The simplest procedure suggests that the average grade should be used, that is to say, a grade equal to the total rise in elevation divided by the horizontal length traversed. For the problem above, assuming 15% trucks on a 6-lane freeway:

$$\text{total rise} = 2640 \times 0.03 + 2640 \times 0.05$$
$$= 211.2 \text{ ft. } (64.4 \text{ m.})$$

$$\text{avg. grade} = 211.2/5280 = 0.04 \text{ or } 4\%$$

Table 2.6 would then be entered with a 4% grade of 1 mile (1.6 km.) in length, and an E_T of 9 found.

This technique is, however, approximate. A method developed by J. Leisch is more exact(10). It involves making use of the truck deceleration curves illustrated in Chapter 1, Figure 1.6.

For the purposes herein, the Ref. 10 method has been simplified by eliminating consideration of vertical curves. The error introduced because of this is generally small, and considering the overall accuracy of capacity procedures, negligible. The method involves finding a percent grade of total length equal to that of the composite grade under consideration which results in the same final speed of trucks.

For the example cited above, a single grade of 5280 ft. (1 mile, 1.6 km.) will be found which results in the same final speed of trucks as 1/2 mile (804 m.) of 3% grade followed by 1/2 mile (804 m.) of 5% grade.

Figure 2.1 illustrates the solution. Entering the acceleration-deceleration curves (Chapter 1, Figure 1.6) with 2640 ft. (1/2 mile, 804 m.) and a 3% grade, point 1 is located, and a speed of 38 mph is read on the speed scale. Thus, trucks enter the 5% grade at a speed of 38 mph. Point 2 is located by the intersection of 38 mph and a 5% grade. Trucks enter the 5% grade as if they were 1200 feet (365.8 m.) along such a grade having started from level terrain. As trucks now travel another 2640 feet (804 m.) along the 5% grade, their final speed is found at point 3, located at the intersection of 5% and 1200 + 2640 or 3840 feet (1170 m.). The final speed of trucks is, therefore, 19 mph (30.4 km/h.).

Now, a single continuous upgrade of 5280 ft. (1.6 km.) which also results in a final speed of 19 mph must be found. Point 4 is the intersection of 19 mph and 5280 feet (1.6 km.), and indicates a grade of 5%. Thus, a value of E_T is chosen for:

- 5% grade, 1 mile (1.6 km.) long

rather than a 4% grade of 1 mile (1.6 km.) as was the case using the average grade technique. The

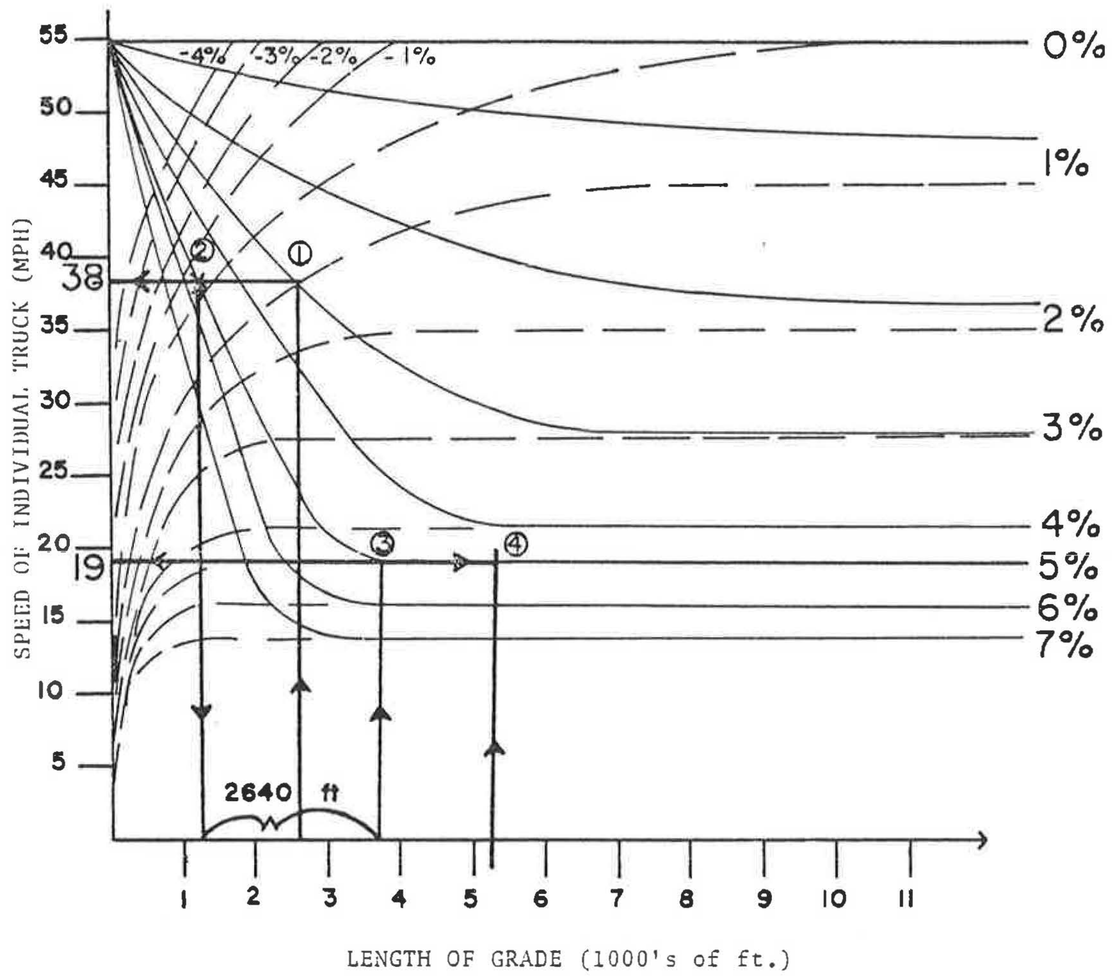


Figure 2.1

AN ILLUSTRATION OF THE LEISCH TECHNIQUE FOR FINDING E_T ON COMPOSITE UPGRADES

NOTE: 1 mph = 1.6 km/h.
1 ft. = 0.3048 m.

E_T value (15% trucks, 6-lane freeway) is 11, significantly different from the value of 9 found previously.

In general, the modified Leisch procedure may be described in the following steps:

a) Enter truck acceleration-deceleration (Chapter I, Figure 1.6) curves with the initial grade and length of initial grade. Find the speed at the end of the initial grade, which is the speed at which trucks enter the second grade.

b) Find the length along the second grade which results in the same speed as found in step a. This is used as the starting point for length along the second grade.

c) Starting with the length found in (b) above, add the length of the second grade, and find the speed at the end of the second grade.

d) If there are additional grades, repeat steps (b) and (c) for each subsequent grade until the final speed is found. If there are no additional grades, the speed found in step (c) is the final speed.

e) Enter the acceleration-deceleration curves with the final speed and the total length of the variable grade. Find the equivalent uniform grade.

Some composite grades may include downgrade portions. The procedure for these is the same, only the curves for negative grades are used. To be conservative, it is never assumed that trucks accelerate to speeds greater than 55 mph, even on downgrades.

On some composite grades, the final speed IS NOT the slowest speed experienced along the grade. In such cases, the E_T is found considering the point of slowest speed along the grade, and the length of the grade to that point.

IN GENERAL, THE LEISCH TECHNIQUE NEED NOT BE USED IF THE TOTAL LENGTH OF GRADE IS LESS THAN APPROXIMATELY 2500 FT. THE AVERAGE GRADE TECHNIQUE IS ACCEPTABLE FOR SUCH CASES. It is not necessary to use the Leisch technique for recreational vehicles due to the approximate nature of equivalents for these. Due to the form of passenger car equivalents for buses, the Leisch technique cannot be applied to bus equivalents.

D. Computing Combined Correction Factor for Trucks, Buses, and Recreational Vehicles

Once E_T, E_B, and/or E_R are found, it is possible to compute a correction factor for the combined effect of such vehicles on capacity or service volume. If more than one category of these vehicles are present in significant percentages in the traffic stream, the combined correction factor should be computed as follows:

$$Q = 100/[100 + P_T(E_T-1) + P_B(E_B-1) + P_R(E_R-1)]$$

where: Q = combined correction factor

P_T, P_B, P_R = percent trucks, buses, and recreational vehicles respectively in the traffic stream

E_T, E_B, E_R = passenger car equivalents for trucks, buses, and recreational vehicles, respectively.

Where only one type of these vehicles is present (generally trucks), or where percentages of two are negligible compared to the dominant vehicle, Table 2.8 may be used to find the factor Q.

In general, any type of vehicle making up 2% or more of the traffic stream should be separately considered. Where two types of vehicles are present in small quantities with respect to the third, all non-passenger cars would be assumed to be of the dominant type. For example, a traffic stream having 15% trucks, 1% recreational vehicles, and 1% buses would be thought of as having approximately 17% trucks.

A Special Procedure for Consideration of Truck Climbing Lanes

On many long and/or steep grades, it is necessary to consider the provision of a truck climbing lane. This is not the same as adding a lane to the freeway

TABLE 2.8

ADJUSTMENT FACTOR FOR THE PRESENCE OF TRUCKS,
BUSES, AND RECREATIONAL VEHICLES IN THE TRAFFIC STREAM*

PASSENGER CAR EQUIVALENT	ADJUSTMENT FACTOR, Q														
	PERCENTAGE OF TRUCKS, P_T (OR OF BUSES, P_B)(OR OF REC. VEH., P_R)														
	1	2	3	4	5	6	7	8	9	10	12	14	16	18	20
2	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.93	0.92	0.91	0.89	0.88	0.86	0.85	0.83
3	0.98	0.96	0.94	0.93	0.91	0.89	0.88	0.86	0.85	0.83	0.81	0.78	0.76	0.74	0.71
4	0.97	0.94	0.92	0.89	0.87	0.85	0.83	0.81	0.79	0.77	0.74	0.70	0.68	0.65	0.63
5	0.96	0.93	0.89	0.86	0.83	0.81	0.78	0.76	0.74	0.71	0.68	0.64	0.61	0.58	0.56
6	0.95	0.91	0.87	0.83	0.80	0.77	0.74	0.71	0.69	0.67	0.63	0.59	0.56	0.53	0.50
7	0.94	0.89	0.85	0.81	0.77	0.74	0.70	0.68	0.65	0.63	0.58	0.54	0.51	0.48	0.45
8	0.93	0.88	0.83	0.78	0.74	0.70	0.67	0.64	0.61	0.59	0.54	0.51	0.47	0.44	0.42
9	0.93	0.86	0.81	0.76	0.71	0.68	0.64	0.61	0.58	0.56	0.51	0.47	0.44	0.41	0.38
10	0.92	0.85	0.79	0.74	0.69	0.65	0.61	0.58	0.55	0.53	0.48	0.44	0.41	0.38	0.36
11	0.91	0.83	0.77	0.71	0.67	0.63	0.59	0.56	0.53	0.50	0.45	0.42	0.38	0.36	0.33
12	0.90	0.82	0.75	0.69	0.65	0.60	0.57	0.53	0.50	0.48	0.43	0.39	0.36	0.34	0.31
13	0.89	0.81	0.74	0.68	0.63	0.58	0.54	0.51	0.48	0.45	0.41	0.37	0.34	0.32	0.29
14	0.88	0.79	0.72	0.66	0.61	0.56	0.52	0.49	0.46	0.43	0.39	0.35	0.32	0.30	0.28
15	0.88	0.78	0.70	0.64	0.59	0.54	0.51	0.47	0.44	0.42	0.37	0.34	0.31	0.28	0.26
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
21	0.83	0.71	0.63	0.56	0.50	0.45	0.41	0.38	0.36	0.33	0.29	0.26	0.24	0.22	0.20
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
23	0.82	0.69	0.60	0.53	0.48	0.43	0.39	0.36	0.34	0.31	0.27	0.25	0.22	0.20	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.18
25	0.80	0.67	0.58	0.51	0.46	0.41	0.37	0.34	0.32	0.29	0.26	0.23	0.20	0.18	0.17

* Not to be used where more than one type of such vehicles individually comprise 2% or more of the traffic stream. For values outside the range of this table, use equation for Q directly.

in general, as it is intended for slow-moving vehicles only. It will normally contain 100% trucks or other slow-moving vehicles, with remaining trucks and slow-moving vehicles sharing mixed traffic lanes.

There are no exact or specially calibrated procedures for analyzing truck climbing lanes. The techniques recommended herein are approximate, but will afford a general idea of how such a lane would operate.

First, it is necessary to estimate how many trucks will use the truck climbing lane. E_T for a lane containing 100% trucks may be assumed to be the lowest value given in Table 2.6 for the percent and length of grade. This is reasonable, as E_T decreases with increasing truck presence. The capacity of the truck climbing lane, C_T, may then be directly computed:

$$C_T = 2000/E_T$$

If it is assumed that trucks utilize the same percentage of the capacity of the truck climbing lane as is utilized on the remainder of the freeway (a conservative view), the values shown in Table 2.9 may be used. In areas where regulations REQUIRE the use of climbing lanes, higher values should be assumed.

The number of trucks utilizing the truck climbing lane for a given Level of Service may then be computed at:

$$SV_T = C_T \times P_c = 2000\ P_c/E_T$$

where SV_T is the number of trucks or slow-moving vehicles using the climbing lane.

Remaining trucks or slow-moving vehicles are then assumed to share mixed traffic lanes with passenger cars and other vehicles. The mixed vehicle lanes may then be designed or analyzed using normal techniques.

Examples using this technique are included in the "Sample Problem" section.

III. SAMPLE PROBLEMS

1) Design: A Basic Case

How many lanes must be provided through an extended section of level terrain to accommodate the following demand:

- DDHV = 4000 vph
- PHF = 0.90
- Traffic Composition 12% trucks
  no buses or recreational vehicles
- Design Level of Service = C

Solution: The solution involves computation of THE MINIMUM NUMBER OF LANES REQUIRED TO PROVIDE FOR AN ACCEPTABLE Level of Service C design for a service volume of:

$$SV = 4000/0.90 = 4444 \text{ vph.}$$

As design cannot be attempted at the Level of Service C boundary condition (for AHS = 70 mph or 112 km/h), a v/c value of 0.80 will be used. This is taken from Table 2.2, which indicates the resulting design would be at an intermediate point within Level of Service C.

The following design geometrics will be adopted:

- 70 mph (112 kph) AHS
- 12 ft. (3.7 m.) lanes
- no lateral obstructions.

As there are no lateral obstructions, and lane widths are 12 feet, there will be no reduction in service volume due to lane width or lateral clearance restrictions, which is to say W = 1.00.

For 12% trucks on level terrain, E_T = 2 (Table 2.4) and Q = 0.89 (Table 2.8).

Then: $SV = 2000\ N\ v/c\ W\ Q$

$4444 = 2000\ N\ (0.8)\ (1.00)\ (0.89)$

$N = 4444/2000\ (0.8)\ (1.00)\ (0.89) = \underline{3.12 \text{ lanes}}$

As a v/c of 0.8 represents the maximum feasible design value, and since 0.12 lanes cannot be provided, the minimum design would be <u>4</u> lanes in each direction, or an 8-lane freeway.

The design problem most properly ends here. Because the design provides for some excess service volume, however, it would be interesting to see what Level of Service will actually exist at the design flow rate.

TABLE 2.9

PERCENT OF TRUCK CLIMBING LANE CAPACITY UTILIZED*

Level of Service	Percent Capacity Utilized, P_c (approx.)
A	0.50
B	0.72
C	0.87
D	0.93
E	0.95

* percentages based upon v/c values suggested in Table 2.1 for 70 mph (112 km/h) AHS

To analyze the situation, the known value of SV is used to compute the effective MSV.

MSV = SV/(W x Q)

where: SV = 4444 vph, as before

MSV = Unknown

W = 1.0 as before

Q = 0.89 as before

MSV = 4444/(1.0 x 0.89)

MSV = 4993 pcph

Comparing the effective MSV to Table 2.1 values, it is seen that the actual Level of Service will be B, despite the fact that design was attempted at a v/c of 0.80 (Level C). Because partial lanes may not be provided, a better initial operating level than intended has resulted.

2) Design: Truck Climbing Lane Case

A long segment of rural freeway is to be designed for Level of Service B (threshold). The DDHV is 2200 vph, including 20% trucks, at a PHF of 0.95. A 5-mile segment of level terrain is followed by a 3% sustained grade of 1 mile.

Solution: Again, it will be decided to provide ideal geometric conditions i.e., 70 mph AHS, 12-foot lanes, and no lateral obstructions.

From Table 2.2, a v/c value of 0.65 is used for the Level of Service B boundary. Then

SV = 2000 N v/c W Q

where: SV = 2200/0.95 = 2400

v/c = 0.65

W = 1.00

E_T = 2 (Table 2.4)

Q = 0.83 (Table 2.8)

2400 = 2000 N (0.65) (1.00) (0.83)

N = 2400/2000 (0.65) (1.00) (0.83)

N = 2.2 SAY 3

From the computation, it is seen that a 6-lane freeway is needed.

Consider now the 3%, 1 mile grade:

SV = 2000 N v/c W Q

where: SV = 2400 E_T = 7 (Table 2.6)

v/c = 0.65 Q = 0.45 (Table 2.8)

W = 1.00

2400 = 2000 N (0.65) (1.00) (0.45)

N = 2400/2000 (0.65) (1.00) (0.45)

N = 4.1

As 4.1 is a minimum value, it might be tempting to raise the value to 5. However, the level terrain segment entering the grade has only 3 lanes, and the addition of 2 lanes on the grade is not practical.

Thus, an 8-lane freeway is required, or more properly, 4-lanes are needed on the upgrade. Lacking further information, the downgrade would be treated as if it were level terrain, and as previously, only 3 lanes would be required on the downgrade.

As a lane needs to be added on the upgrade, and presumably dropped thereafter, the total design will have to carefully consider the issues of lane balance and configuration.

It appears that the additional upgrade lane might be specified as a truck climbing lane. Thus its operation should be checked using the procedure for truck climbing lanes. As the minimum E_T for a 3% grade of 1 mile is 7, the capacity of the truck climbing lane is:

2000/7 = 286 trucks/hour

At Level of Service B, it is expected that 72% of that capacity would be utilized (Table 2.9), so that the number of trucks expected to use the climbing lane is:

286 x 0.72 = 206 trucks/hour

The total number of trucks in the traffic stream is 2200 x 0.20 or 440 trucks/hour. Therefore, the 3 lanes of mixed traffic will carry 2200-206 = 1994 vph, 234 of which are trucks (12%).

Therefore, an analysis will be made for a 6-lane freeway carrying 1994 vph with 12% trucks and a PHF = 0.95.

The effective MSV on the three normal freeway lanes is computed as:

MSV = SV/(W x Q)

where: SV = 1994/0.95 = 2099 vph

W = 1.00

E_T = 7 (Table 2.6)

Q = 0.58 (Table 2,8)

MSV = 2099/(1.0 x 0.58) = 3619 pcph

Comparing this to the criteria in Table 2.1, the Level of Service is B.

Because the upgrade is not actually an 8-lane freeway, but a 6-lane freeway with a truck climbing lane, its Level of Service is actually B, not C as might have been thought since 4 lanes are provided where the initial computation indicated a need for 4.1 lanes.

The final design, therefore, results in a 6-lane freeway with a truck climbing lane on the sustained 3% upgrade.

3) Design: Recreational Facility

A sustained 5% upgrade of 1-1/2 miles is to be redesigned on a freeway serving a national park. After redesign, the road is expected to carry a

DDHV of 1000 vph, 20% of which are recreational vehicles and 5% of which are buses. There is no appreciable truck traffic. The PHF = 0.95, and a design for the Level of Service A boundary condition is desired.

Solution: In this problem, both buses and recreational vehicles are present in significant quantities, and must be separately considered. Again, a design including ideal geometrics is assumed (W = 1.00). Note that service volumes computed from Table 2.1 are reduced by 10% as recommended, as this recreational route will attract its peak demand on weekends. Table 2.2 values are representative of commuter traffic. Considering the upgrade:

SV = 2000 N V/C (0.90) W Q

where: SV = 1000/0.95 = 1052

V/C = 0.40 (Table 2.2 for Level of Service 1 boundary)

W = 1.00

E_B = 4 (Table 2.5)

E_R = 5 (Table 2.7)

Q = 100/[100 + 5 (4-1) + 20 (5-1)]
= 0.51

0.90 = reduction in service volume to account for weekend traffic

then: 1052 = 2000 N (0.40)(0.90)(1.00)(0.51)

N = 1052/2000(0.40)(0.90)(1.00)(0.51)

N = 2.86 SAY 3

The computation results in a determination that a 6-lane freeway, or 3 lanes on the upgrade are required.

Considering the downgrade as if it were level terrain:

SV = 2000 N V/C (0.90) W Q

where: SV = 1052

V/C = 0.40

W = 1.00

E_B = 1.6 (Table 2.4)

E_R = 2 (Table 2.4)

Q = 100/[100 + 5 (1.6-1) + 20 (2-1)]
= 0.813 = 0.81

then: 1052 = 2000 N(0.40)(0.90)(1.00)(0.81)

N = 1052/2000(0.40)(0.90)(1.00)(0.81)

N = 1.80 SAY 2

For a design flow of 1052 vph, a 4-lane freeway is required, or 2 lanes downgrade. Thus, it appears that the section should be a 4-lane freeway with a climbing lane for slow-moving vehicles on the upgrade.

An attempt could be made to analyze the climbing lane using the procedure for truck climbing lanes. The climbing lane procedure is, however, approximate, and its use with recreational vehicles, for which E_R values are also highly approximate, is not justified.

4) Analysis: A Basic Case

An older urban freeway serving a peak hour volume of 2000 vph has the following characteristics:

- 11-foot lanes
- obstructions immediately at the pavement edge at both the roadside and median
- 6% trucks
- AHS = 60 mph
- PHF = 0.95
- 4-lane freeway
- rolling terrain

Evaluate the Level of Service on the facility. How much additional demand could be accommodated before the boundary of Level of Service E is reached?

Solution: The effective MSV on the facility is computed as -

MSV = SV/(W x Q)

where: SV = 2000/0.95 = 2105 vph

W = 0.79 (Table 2.3, 11 ft. lanes, obstruction on both sides at 0 ft.)

E_T = 4 (Table 2.4).

Q = 0.85 (Table 2.8)

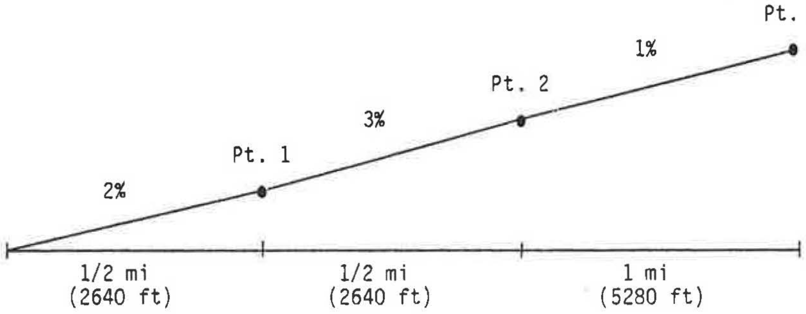
MSV = 2105/(0.79 x 0.85) = 3135 pcph

Comparing this to criteria in Table 2.1, it is seen that the Level of Service is D. Capacity is 4000 pcph, so that an additional 865 pcph could be accommodated before the limit of Level of Service 5 is reached. Converting this to vph for prevailing conditions:

Remaining Capacity = 865 x 0.79 x 0.85 = 581 vph

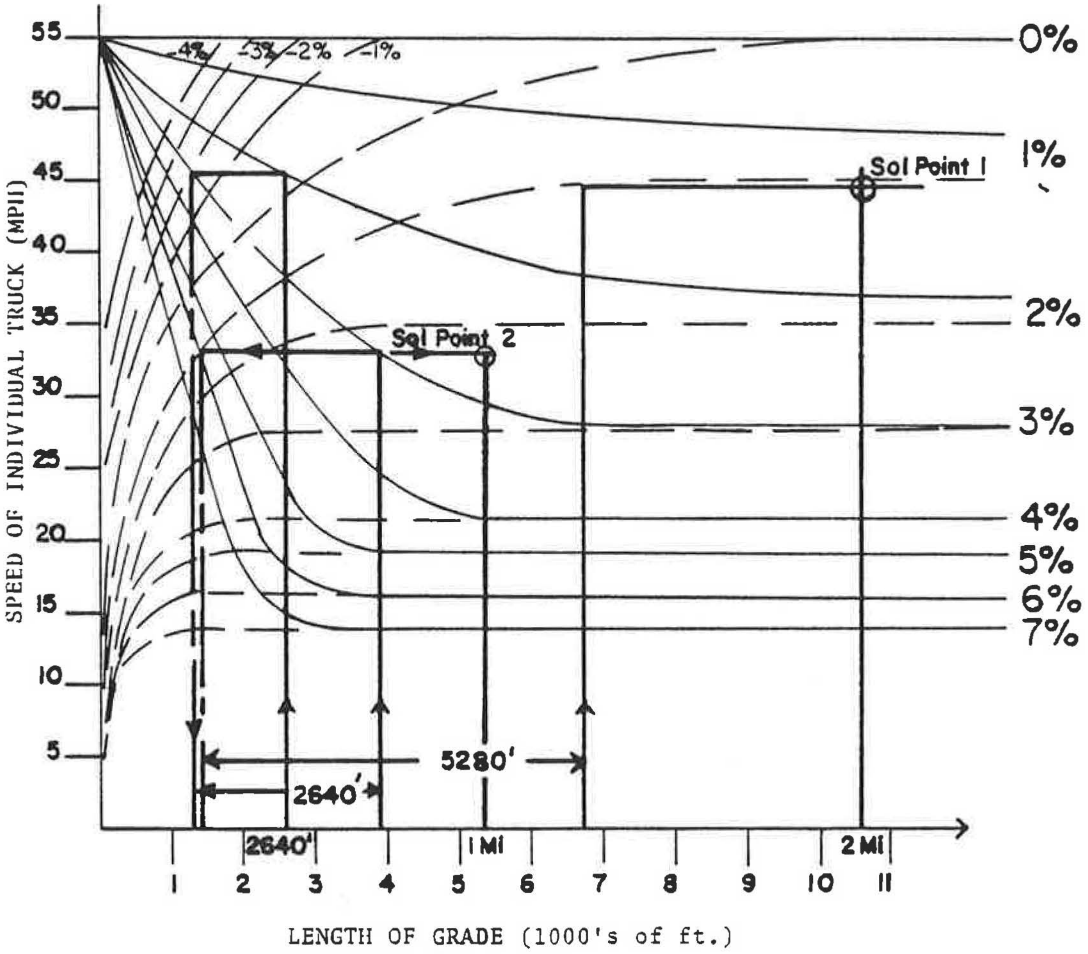
5) Analysis: Composite Grade

Consider the compound upgrade shown in the figure below:



A six lane freeway with a 70 mph design speed carries a peak hour volume of 3500 vph with 5% trucks at a PHF of 0.85. The freeway has 12-foot lanes, a 20-foot clear median, but has rock cliffs 2 ft. from the right edge of the pavement in both directions. At what Level of Service does the freeway operate during peak periods - upgrade and downgrade?

Solution: The key to the upgrade solution is finding a single grade of 2 mi. which has the same effect on truck speeds as the composite grade shown above. The solution for this is illustrated in the figure which follows.



ACCELERATION-DECELERATION CURVES FOR A STANDARD TRUCK (250-350 lbs/hp)

NOTE: 1 mph = 1.6 km/h.
1 ft. = 0.3048 m.

The acceleration-deceleration curves are entered at 2640 ft. and 2%. The speed at this point is then transferred to the 3% grade curve (which occurs at about 1300 ft. along the 3% grade). A length of 2640 ft. is advanced along the 3% grade (to about 3940 ft.), and the speed at the end of the 3% grade is found. This speed is then transferred to the 1% grade curve. Note that only the acceleration curve is intercepted, indicating that trucks will accelerate on the 1% grade. The speed at the end of the grade is found by moving along the 1% grade curve a distance of 5,280 ft.

The uniform grade of 2 mi. which would produce this speed is found by the intersection of the speed (a little less than 45 mph) and 2 mi. A grade of 1-1/3% (approx.) is indicated by solution point 1, which may be used to find a value of E_T for Point 3 on the diagram. From Table 2.6, E_T is 5.7 by interpolation.

It must be considered, however, that the maximum impact of trucks occurs at the end of the 3% grade, at Point 2, when they are traveling only about 33 mph. This, as shown by solution point 2, is equivalent to a 2-3/4%, 1 mi. grade. Therefore, the upgrade Level of Service should be found using an E_T for a 2-3/4% grade of 1 mile. The downgrade may be treated as if it were level terrain, lacking further information.

The effective MSV must be computed for both the upgrade and the downgrade:

MSV = SV/(W x Q)

where SV = 3500/0.85 = 4118 vph

W = 0.97 (Table 2.3, 2 ft. obst. on one side, 12-foot lanes)

E_T (upgrade) = 9.25 (Table 2.6, 1-mile, 2-3/4%)

Q (upgrade) = 0.705 (Table 2.8)

E_T (downgrade) = 2 (Table 2.4)

Q (downgrade) = 0.95 (Table 2,8)

MSV upgrade = 4118/(0.97 x 0.705) = 6022 pcph (Level of Service F)

MSV downgrade = 4118/(0.97 x 0.95) = 4469 pcph (Level of Service C)

From these results, it would appear that a truck climbing lane should be considered for at least the first mile of the upgrade, and possibly for the entire upgrade.

6) Design: Farm Vehicles

A rural freeway segment of 3/4 mile on a 3% grade is to be designed to a v/c ratio of 0.60. It will have a demand of 1900 vph during the design hour with 15% trucks and a PHF of 0.95. Trucks are expected to be predominantly of the farm-to-market variety, with high weight/horsepower ratios.

Solution: From Table 2.2, for a v/c ratio of 0.60, the Level of Service will be B, assuming that 70 mph geometrics, 12-foot lanes and adequate clearances are provided.

As the trucks in question are heavily loaded, the value of E_T will be drawn from Table A2.2 for trucks with a weight/horsepower ratio of greater than 350 lbs./hp. This table is included as part of the appendix to this chapter. E_T, from Table A2.2, for 15% trucks on a 3/4-mile 3% upgrade would be 12 for a 4-lane freeway and 10 for a 6- or more-lane freeway.

Then, for the upgrade:

SV = 2000 N V/C W Q

where: SV = 1900/0.95 = 2000

V/C = 0.60

W = 1.00

E_T = 10 (assuming 6- or more-lane freeway)

Q = 0.425 (Table 2.8)

2000 = 2000 N (0.60) (1.00) (0.425)

N = 2000/2000 (0.600 (1.00) (0.425)

N = 3.92 SAY 4 lanes

and for the downgrade, where:

E_T = 2 (Table 2.4, level terrain)

Q = 0.87 (Table 2.8)

2000 = 2000 N (0.60) (1.00) (0.87)

N = 2000/2000 (0.60) (1.00) (0.87)

N = 1.92 SAY 2 lanes

This suggests that 2 climbing lanes would be required for the upgrade, which is not realistic. If a design were adopted which consisted of a 4-lane freeway with one climbing lane on the upgrade, the resulting operating conditions could be analyzed.

The number of trucks expected to use the climbing lane is given by:

$SV_T = 2000\ P_c/E_T$

where: E_T = 10 (Table A2.2, minimum value for grade, 100% trucks)

P_c = 0.72 (Table 2.9, Level of Service B)

SV_T = 2000 (0.72)/10 = 144 trucks

The 2 remaining freeway lanes therefore must still accommodate a total volume of 1900 - 144 = 1756 vph, of which 1900 (0.15) - 144 = 141 are trucks (8%). The effective MSV for these lanes is therefore:

MSV = SV/W x Q

where SV = 1756/0.95 = 1848

W = 1.00

E_T = 13 (Table A2.2)

Q = 0.51

MSV = 1848/(1.00) (0.51) = 3624 pcph

This indicates a Level of Service of D for the remaining lanes (Table 2.1). Since a Level of Service of B was assumed to obtain P_c from Table 2.9, the solution should be tried again with an assumption of Level of Service C. Then:

P_c = 0.87 (Table 2.9)

E_T = 10 (as before)

and:

SV_T = 2000/(0.87)/10 = 174 trucks

Remaining freeway lanes must accommodate 1900 - 174 = 1726 vph, of which 1900 (0.15) -174 = 111 are trucks (6%).

Then: MSV = SV/W x Q

where: SV = 1726/0.95 = 1817

W = 1.00

E_T = 14 (Table A2.2)

Q = 0.56

MSV = 1817/(1.00) (0.56) = 3245 pcph

From Table 2.1, this indicates a Level of Service of C, consistent with the starting assumption.

Thus, the upgrade will only operate at Level of Service C, even with a truck climbing lane. This is less than the Level of Service B which other segments will experience with the v/c of 0.60, but must realistically be tolerated.

CHAPTER II - REFERENCES

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3) A Policy on Design of Urban Highways and Arterial Streets, American Association of State Highway and Transportation Officials, Washington, D.C., 1973.

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7) Freeway Design Elements on Grades, Implementation Package 77-18, Federal Highway Administration, USDOT, August 1977.

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9) St. John and Kobett, "Grade Effects on Traffic Flow Stability and Capacity," Final Report, NCHRP Project 3-19, Midwest Research Institute, August 1974.

10) Leisch, Capacity Analysis Techniques for Design and Operation of Freeway Facilities, Federal Highway Administration, 1974.

Appendix - Values of E_T for Non-Standard Truck Populations

TABLE A2.1
VALUES OF E_T FOR LIGHT TRUCKS
(wt/hp $\leq$ 150 lbs/hp)

GRADE (%)	LENGTH (mi)*	4-Lane Freeways								6- or More-Lane Freeways							
		Percent Trucks															
		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
0	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
2	0 -3/4	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	>3/4	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2
3	0 -1/4	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/4-1/2	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2
	1/2-3/4	4	4	4	3	3	3	3	3	3	3	3	3	3	3	2	2
	3/4- 1	4	4	4	3	3	3	3	3	3	3	3	3	3	3	3	3
	1 -1-1/2	5	5	5	4	4	4	3	3	4	3	3	3	3	3	3	3
	>1-1/2	6	5	5	4	4	4	3	3	4	3	3	3	3	3	3	3
4	0 -1/4	3	2	2	2	2	2	2	2	3	2	2	2	2	2	2	2
	1/4-1/2	5	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/2-3/4	6	4	4	4	3	3	3	3	4	3	3	3	3	3	3	3
	3/4- 1	7	5	5	5	4	4	3	3	4	4	4	3	3	3	3	3
	>1-1/2	8	6	6	5	4	4	3	3	5	4	4	3	3	3	3	3
5	0 -1/4	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
	1/4-1/2	6	4	4	4	4	4	3	3	5	4	4	3	3	3	3	3
	1/2- 1	7	5	5	5	4	4	3	3	6	4	4	3	3	3	3	3
	1 -1-1/2	9	6	6	6	4	4	3	3	7	5	5	4	4	4	3	3
	>1-1/2	12	8	8	7	5	5	4	4	8	6	5	4	4	4	3	3
6	0 -1/4	5	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4-1/2	8	6	6	5	4	4	3	3	6	5	5	4	3	3	3	3
	1/2- 1	12	8	8	7	5	4	3	3	8	6	5	4	4	4	3	3
	>1	16	10	9	8	6	5	4	4	10	7	6	5	4	4	3	3

* Where the length of grade is a boundary value, always use the longer length range

TABLE A2.2

VALUES OF E_T FOR HEAVY TRUCKS
(wt/hp $\geq$ 350 lbs/hp)

GRADE (%)	LENGTH (mi)*	4-Lane Freeways								6- or More-Lane Freeways							
		Percent Trucks															
		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
0	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0 -1/4	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
	1/4-1/2	5	4	4	4	4	4	3	3	5	4	4	4	4	4	3	3
	1/2-3/4	7	5	5	5	4	4	4	4	7	5	5	4	4	4	4	4
	3/4- 1	8	6	6	6	5	5	4	4	8	6	6	5	5	5	4	4
	1 -1-1/2	10	7	7	6	5	5	4	4	10	7	7	6	5	5	4	4
	>1-1/2	11	8	8	7	6	6	5	5	11	8	8	7	6	6	5	5
2	0 -1/4	8	6	6	6	5	5	4	4	7	5	5	5	5	5	4	4
	1/4-1/2	10	7	7	7	6	6	5	5	9	6	6	6	6	6	5	5
	1/2-3/4	12	9	9	8	8	7	6	6	11	8	8	7	7	7	6	6
	3/4- 1	14	10	10	9	9	8	7	7	13	9	9	8	8	7	6	6
	1 -1-1/2	16	11	10	9	9	8	8	8	15	10	10	9	9	8	7	7
	>1-1/2	16	12	11	10	10	9	8	8	15	11	11	10	9	8	7	7
3	0 -1/4	11	10	10	9	8	8	7	7	9	8	8	8	7	7	6	6
	1/4-1/2	13	12	12	11	9	9	8	8	11	10	10	9	8	8	7	7
	1/2-3/4	16	14	13	12	11	10	10	10	13	12	12	11	10	9	8	8
	3/4- 1	19	15	15	14	13	12	12	12	16	13	13	13	12	11	10	10
	>1	22	16	16	15	15	14	14	14	18	14	14	14	13	12	11	11
4	0 -1/4	13	11	11	10	10	9	8	8	11	9	9	9	9	8	8	8
	1/4-1/2	18	13	13	13	12	12	12	12	13	11	11	11	11	10	9	9
	1/2-3/4	22	15	15	15	14	14	14	14	16	13	13	13	13	12	11	11
	3/4- 1	24	20	19	18	17	17	17	17	19	15	15	15	15	14	13	13
	>1	26	20	20	19	19	19	19	19	21	17	17	17	16	16	14	14
5	0 -1/4	19	16	16	16	16	16	16	16	17	13	13	12	12	12	11	11
	1/4-1/2	26	21	21	21	21	21	21	21	22	17	17	16	16	16	15	15
	1/2-3/4	33	27	27	27	27	27	27	27	27	21	21	20	20	20	19	19
	>3/4	40	32	32	32	32	32	32	32	31	25	25	24	24	24	23	23

* Where the length of grade is a boundary value, always use the longer length range.

CHAPTER III - WEAVING AREAS

I. BASIC CHARACTERISTICS

Weaving is defined as the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway, without the aid of traffic signals. Weaving sections are formed when two or more one-way roadways merge forming a single roadway, then diverge, or separate to form two or more roadways again. Figure 3.1 illustrates the formation of a weaving section.

If entry and exit roadways are referred to as "legs," vehicles traveling from leg A to leg D must cross the path of vehicles traveling from leg B to leg C. Flows A-D and B-C are therefore referred to as weaving flows, while flows A-C and B-D are "non-weaving" or outer flows. Figure 3.1 illustrates a simple weaving section, which is formed by a single merge point followed by a single diverge point. Multiple weaving sections (discussed later in this chapter) are formed where two or more merge points are followed by a diverge or where a single merge point is followed by two or more diverge points.

Weaving areas, of necessity, entail intense lane-changing maneuvers, as weaving vehicles must move into a lane appropriate to their exit leg. Because of this, non-weaving vehicles may also execute a greater-than-normal number of lane changes to reach outside lanes and avoid weaving turbulence. Due to this turbulence, the operation of freeway vehicles is considerably different from that on open freeway segments, and vehicles occupy more space than on basic sections.

The requirement that many drivers execute lane changes to complete their weaving movements introduces a new geometric factor to consider - length. The length of the weaving section restricts the time and distance in which a weaving driver must make the needed lane changes. Thus, as length is decreased, the intensity of lane-changing is increased for any given volume. The length of a weaving section is illustrated in Figure 3.2, from a point at which the two entry legs are separated by two (2) feet (0.61 m.) to a point at which the exit legs are separated by twelve (12) feet (3.7 m.)

Parameters of Interest

There are a number of parameters which affect the operation of weaving areas. For convenience, they are listed and defined below.

L = length of the weaving section, in feet (note: 1 ft. = 0.3048 m.)

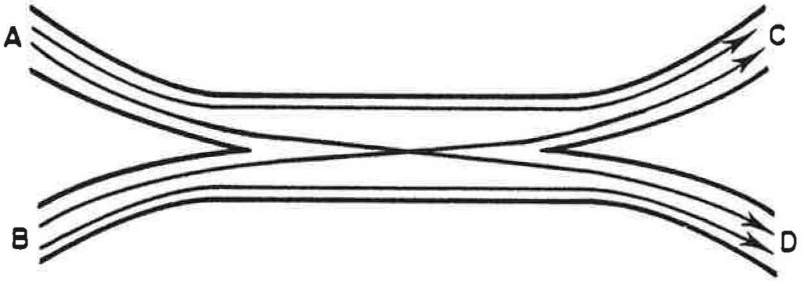


FIGURE 3.1

FORMATION OF A WEAVING SECTION

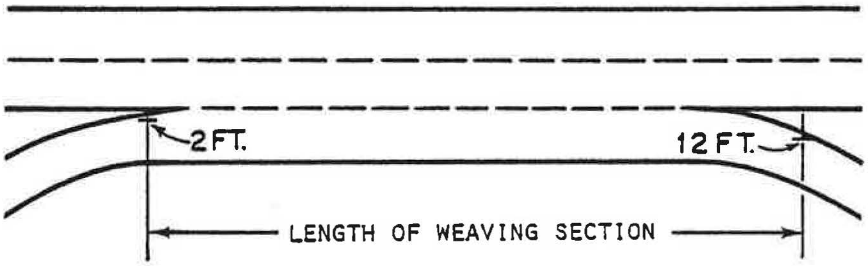


FIGURE 3.2

MEASURING THE LENGTH OF A WEAVING SECTION

NOTE: 1 FT. = 0.3048 M.

L_H = length of the weaving section, in hundreds of feet (note: 1 ft. = 0.3048 m.)

N = number of lanes in the weaving section

N_W = number of lanes theoretically utilized by weaving vehicles

N_{NW} = number of lanes theoretically utilized by non-weaving vehicles

V = total volume in the weaving section, pcph

V_W = total weaving volume, pcph

V_{NW} = total non-weaving volume, pcph

V_{W1} = larger of the two weaving flows, pcph

V_{W2} = smaller of the two weaving flows, pcph

R = weaving ratio, V_{W2}/V_W

VR = volume ratio, V_W/V

S_W = average running speed of weaving vehicles, mph (note: 1 mph = 1.6 kph)

S_{NW} = average running speed of non-weaving vehicles, mph (note: 1 mph = 1.6 kph)

S = $S_{NW} - S_W$

N_W/N = portion or fraction of total lanes used by weaving vehicles

Note that by definition: $N = N_W + N_{NW}$

Levels of Service

Weaving areas are often complex, and do not always exhibit homogeneous operating characteristics. For reasons which are discussed in the next section, weaving and non-weaving flows may experience operating conditions which are highly dissimilar. Thus, Levels of Service are separately defined for both weaving and non-weaving flows. In general, the speed of weaving vehicles is expected to be within 5 mph of that of non-weaving vehicles for acceptable operations. Average speed thresholds were selected in concert with the Levels of Service on basic freeway segments, modified by the observed relationship between speed and volume of non-weaving vehicles in a weaving section. This relationship is shown below:

$$V_{NW} = 1500\ N_{NW} - 50\ S_{NW} + 1900$$

The relationship is valid throughout the range of stable flow, which implies S_{NW} greater than or equal to 25 mph (40 kph).

Note that due to the increased turbulence in weaving areas, the relationship between non-weaving volume and non-weaving speed is linear. Speed is sensitive to volume levels throughout the range of stable non-weaving flows.

Table 3.1 defines Levels of Service in weaving sections in accord with the preceding discussion.

TABLE 3.1

LEVELS OF SERVICE IN WEAVING AREAS

NON-WEAVING VEHICLES

Level of Service	Avg. Running Speed of Non-Weaving Vehicles MPH (km/h)
A	$S_{NW} \geq 50$ (80)
B	$S_{NW} \geq 45$ (72)
C	$S_{NW} \geq 40$ (64)
D	$S_{NW} \geq 35$ (56)
E	$S_{NW} \geq 30$ (48)
F	$S_{NW} < 30$ (48)

WEAVING VEHICLES

Level of Service for Weaving Vehicles is ___ the Level of Service for Non-Weaving Vehicles	IF ΔS is MPH (km/h)
the same as	$\Delta S \leq 5$ (8)
1 level poorer than	$\Delta S \leq 10$ (16)
2 levels poorer than	$\Delta S \leq 15$ (24)
3 levels poorer than	$\Delta S \leq 20$ (32)
4 levels poorer than	$\Delta S \leq 25$ (40)

The volume and design parameters associated with these levels of service are defined by equations, in which speed is an explicit parameter. These are presented later.

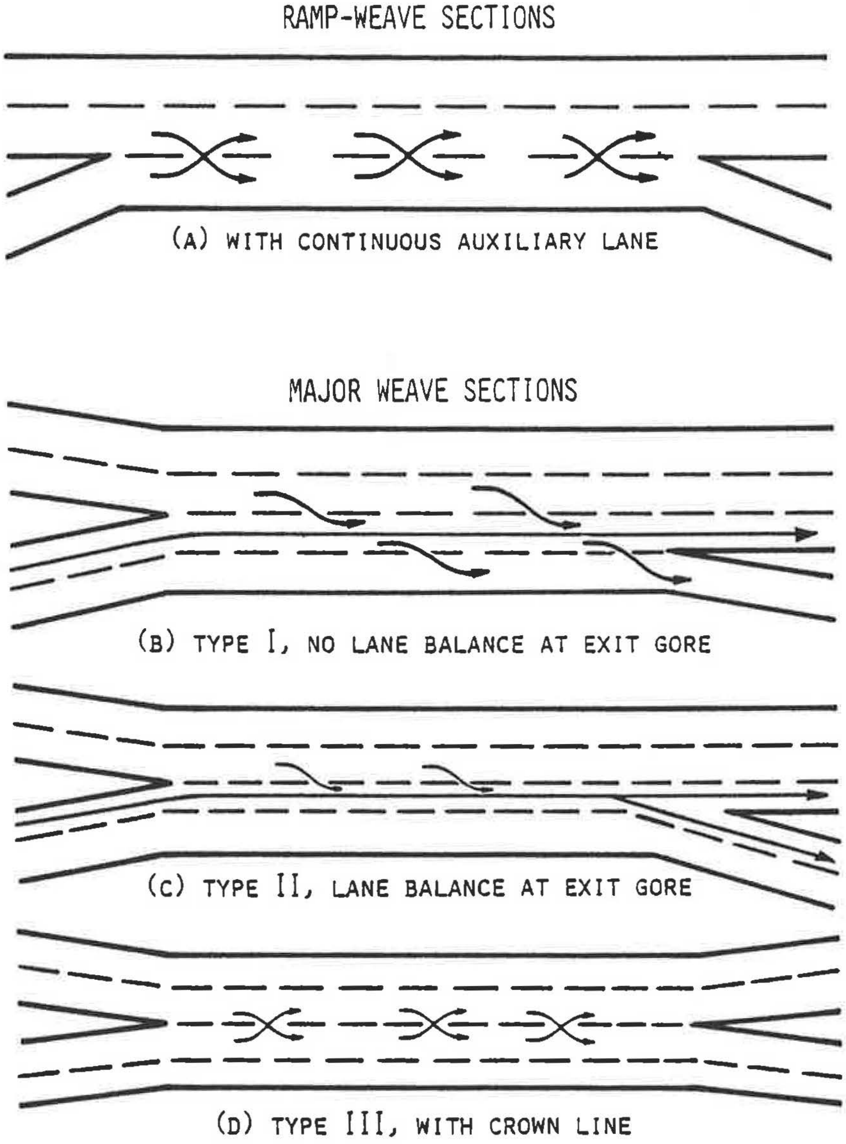
Configuration

One of the most important factors influencing the operation of weaving areas is configuration. Configuration refers to the relative placement of lanes in the weaving section vis-a-vis entry and exit legs. Configuration has a drastic influence on the number of lane changes which are required of weaving vehicles, and the ability of weaving vehicles to occupy a larger proportion of the total lanes.

For each configuration, there is a practical maximum number of lanes which weaving vehicles may occupy, N_W(max.). When weaving volumes are such that they would tend to occupy more than N_W(max) if a natural balance of lane utilization were struck, the section is constrained. In such sections, weaving vehicles are restricted to N_W(max) and experience more congestion than might be expected, while non-weaving vehicles have a disproportionate number of lanes available for their use. Thus, in constrained sections, non-weaving vehicles often display speeds considerably faster than weaving vehicles, an unhealthy operating condition.

In sections where weaving and non-weaving flows compete for space and strike a natural balance in which N_W is less than N_W(max.), the section is considered to be unconstrained.

There are four basic types of weaving configuration within two broad categories of sections. These are illustrated in Figure 3.3, and are described below.



RAMP-WEAVE SECTIONS

(A) WITH CONTINUOUS AUXILIARY LANE

MAJOR WEAVE SECTIONS

(B) TYPE I, NO LANE BALANCE AT EXIT GORE

(C) TYPE II, LANE BALANCE AT EXIT GORE

(D) TYPE III, WITH CROWN LINE

FIGURE 3.3

CONFIGURATIONS FOR WEAVING AREAS

Ramp-Weaves are one of the two broad categories of weaving sections. They are formed by a one-lane on-ramp followed by a one-lane off-ramp, where the ramps are joined by a continuous auxiliary lane. IT SHOULD BE NOTED HERE THAT ON-RAMPS FOLLOWED BY OFF-RAMPS WITHOUT AUXILIARY LANES ARE NOT CONSIDERED TO BE WEAVING AREAS AND ARE TREATED USING RAMP PROCEDURES PRESENTED IN CHAPTER IV.

Figure 3.3A illustrates a ramp-weave, of which there is only one type. Note that this type of weaving section is divided by a lane marking which begins at the entrance gore and ends at the exit gore. This "weaving crown lane" essentially divides the section into two distinct parts. Wherever a weaving crown-line is present, all weaving vehicles must execute at least one lane change, a condition which tends to limit the number of lanes which can be utilized by weaving vehicles.

Ramp-weaves are also distinctive in that all weaving vehicles are involved in a ramp movement which often requires reduced speed due to restrictive geometry. Thus, many weaving vehicles are accelerating or decelerating through the weaving area, and their speed, relative to that of non-weaving vehicles is more dependent upon length than upon other factors. Thus, a substantial difference in the average running speeds of weaving and non-weaving vehicles does not necessarily indicate that the section is constrained, that N_W(max.) has been reached.

Major Weaves are different from ramp-weaves in that at least three of the entry and exit legs involve multiple lanes, and are generally primary or major connector roadways whose geometry does not require reduced speeds compared to the open freeway. In such sections, weaving vehicles are not expected to accelerate or decelerate through the section, and the relative speeds of weaving and non-weaving vehicles result from competition for roadway space. Where no constraint exists, this competition will result in a balance in which weaving vehicles travel at an average speed within 5 mph (8 km/h) of non-weaving vehicles. Where larger speed differentials exist, it is virtually always an indication of constraint, that N_W(max.) has been reached.

There are three distinct types of major weaving sections, as shown in Figure 3.3B, C and D. Types I and II major weaving sections provide a "through" lane for one of the weaving movements, that is, one weaving movement may be made without executing a lane change. The difference between types I and II is that type I sections do not have lane balance at the exit gore, while type II sections do. Lane balance exists when the number of lanes leaving a diverge point is equal to the number entering plus 1. This is accomplished by having one lane divide to two at the gore, giving a vehicle in that lane a choice, in that it may proceed down either exit leg without changing lanes.

In type I major weaving sections, where lane balance does not exist, one weaving movement is made without a lane change. The other weaving movement, however, requires a minimum of two lane changes. In type II major weaving sections, the other weaving movement may be made with only a single lane change. As a result type II major weaving sections are more efficient than type I's. Type II major weaving sections have larger N_W(max.) values, and for a given set of volumes, operate at higher average speeds than similar type I major weaving segments.

Type III major weaving sections are similar to ramp-weaves in that they have a weaving crown line, and all weaving vehicles must make at least one lane change. They are different in that entry and exit roadways are not single-lane ramps with restrictive geometry. These sections are, however, rare. The data base used to develop the procedure described herein did not contain type III cases. It is recommended that such sections be analyzed using ramp-weave procedures.

Configuration is a central concept in weaving area design and analysis. It is not sufficient to determine N and L, as a given pair may exist in any of four different configurations, all with differing operating characteristics. It is vital to note that configuration can be changed only if the placement or design of entry and/or exit legs is altered. This generally entails the addition or subtraction of a lane or lanes on one or more legs, a decision which may have implications on adjacent freeway sections.

Equations

The equations presented herein are the result of a recalibration of the NCHRP Weaving Procedure (1). That data base was re-examined in light of additional data and information developed since its original calibration. The result is a simplified procedure with slightly improved accuracy over the original NCHRP form, and one which retains the original's advantages over the method specified in the 1965 HCM (2).

For each type of weaving configuration, there are three basic equations:

(1) An equation governing the maximum value of N_W for the configuration.

(2) An equation governing the relationship between weaving and non-weaving vehicle speeds.

(3) An equation governing the portion or fraction of total lanes utilized by weaving vehicles.

In each case, one of the equations is secondary, that is, valid only when the weaving section is not constrained, only when $N_W < N_W$(max.). For ramp-weaves this is equation (3), for major weaves, equation (2).

Table 3.2 lists the equations, which are presented in nomograph form under "Computational Procedures."

The equations show logical trends in most cases. Equations governing N_W/N indicate that as length increases, N_W/N, and therefore N_W, decreases- all else being equal. Thus, the principle of trading length for width is valid, though somewhat restricted by other factors, as will be shown.

For short lengths, the difference between S_{NW} and S_W becomes large for ramp-weave sections, as has previously been discussed. For major weaves, the speed equations yield small differences, as would be expected from a secondary equation, valid only in situations that are not constrained.

The equation for N_W(max.), type II major weaves, shows an interesting trend - that N_W(max.) increases as length decreases. It might have been expected that the reverse is more logical: as length increases, weaving vehicles have more opportunity to

TABLE 3.2

WEAVING AREA EQUATIONS

TYPE OF EQUATION	RAMP-WEAVES & TYPE III MAJOR WEAVES	MAJOR WEAVES	
		TYPE I	TYPE II
1) Maximum no. of weaving lanes, N_W(max.)	N_W(max.) = 2.0 (PRIMARY^1)	^{2}log N_W(max.) = 0.714 + 0.480 log R (r = 0.788; PRIMARY^1)	log N_W(max.) = 0.896 + 0.186 log R - 0.402 log L_H (r = 0.655; PRIMARY^1)
2) Speed Relationship	log S_W = 0.142 + 0.694 log S_{NW} 0.315 log L_H (r = 0.883; PRIMARY^1)	S_W = 15.031 + 0.819 S_{NW} - 23.527 VR (r = 0.982; SECONDARY^1)	S_W = 2.309 + 0.871 S_{NW} + 4.579 VR (r = 0.931; SECONDARY^1)
3) Portion of total lanes used by weaving vehicles, N_W/N	log N_W/N = 0.340 + 0.571 log VR - 0.438 log S_W + 0.234 log L_H (r = 0.764, SECONDARY^1)	N_W/N = 0.761 - 0.011 L_H - 0.005 ΔS + 0.047 VR (r = 0.723; PRIMARY^1)	N_W/N = 0.085 + 0.703 VR + (234.763/L) = 0.018 ΔS (r = 0.834; PRIMARY^1)

1) PRIMARY indicates that the equation is valid for all cases.

SECONDARY indicates that the equation is valid only when the section is <u>not</u> constrained, i.e., where $N_W \leq N_W$(max.)

2) Data base for this equation limited to lengths in the range 400-700 ft. - for other lengths, use 85% of the value given by Type II equation.

3) r = correlation coefficient (a measure of how well the equation represents real data: 1.00 is perfect, 0.00 worst)

NOTE: S_W, S_{NW} in mph (1 mph = 1.6 kph)
L in ft, L_H in 100 FT. (1 FT. = 0.3048 M.)
Variables as previously defined, pgs. 68, 69

use outside lanes, executing multiple lane changes to weave, and thus N_W(max.) should also increase. Non-weaving vehicles, however, give weaving vehicles wider berth in shorter sections to avoid higher levels of turbulence. The peculiar characteristics of ramp-weaves, on the other hand, result in a virtually constant value of N_W(max.).

Note that the weaving area equations do not include volume explicitly, but deal only in ratios. Space is also allocated through the use of a ratio (N_W/N). Real volume enters through the non-weaving equation, cited earlier. and N enters as a real or assumed integer value.

II. COMPUTATIONAL PROCEDURES

Computational procedures for weaving sections are utilized in the analysis mode only. Given volumes and known geometrics, the procedure is used to compute expected speeds for weaving and non-weaving flows. Design is accomplished by trial and error.

The trial and error design procedure is consistent with the fact that the design of any weaving area is constrained to a relatively few choices for N, and a relatively small length range. N is controlled by the design of entry and exit legs. Generally only one or two integer values are feasible, and even where entry and exit leg designs are reconsidered, no more than three values are real alternatives. Approximate length is controlled by the location of interchange points. Interchange and ramp design alternatives may allow a range of a few hundred feet for L, but larger changes would generally require major changes to the freeway proper and/or intersecting facilities. The design assumed for initial trial will generally involve the maximum value of N feasible without altering the design of input and output legs and a length consistent with the interchange design initially contemplated.

Simple Weaving Sections

Procedural steps for the analysis of simple weaving sections are outlined below. The analysis is made easier through the construction of a weaving diagram. The weaving diagram is a schematic drawing showing weaving and non-weaving flows in a weaving section. Figure 3.4 illustrates the construction of a weaving diagram.

Note that the weaving diagram depicts actual flows in straight-line schematic form. The relative placement of exit and entry points (A,B,C,D) must remain as actually exists to insure proper placement of flows relative to each other. Volumes shown on the weaving diagram should represent peak flow rates in pcph. The computation of parameters illustrated in Figure 3.4 is accomplished using these peak flow rates.

Computations are accomplished in the following steps:

Step 1. Adjust all volumes to represent peak flow rates in pcph, using the following equation:

$$\text{Peak Flow Rate (PCPH)} = \frac{\text{Volume (VPH)}}{\text{PHF X Q}}$$

Where: PHF = peak hour factor
Q = commercial/recreational vehicle factor obtained from Chapter II.

Step 2. Construct weaving diagram and compute weaving parameters as shown in Figure 3.4, using the adjusted flows of Step 1.

Steps 3-9 represent a trial-and-error procedure in which a value of S_{NW} is assumed and checked by computation. A solution is reached when the assumed and computed values agree. After gaining familiarity with the procedure, no more than 3 trials should be required for closure.

A key feature of the trial-and-error procedure is the determination of whether or not the section in question is constrained, i.e. N_W(max.) has been reached, or unconstrained. The latter represents a case in which a "natural balance" has been struck as a result of unhindered competition among flows for roadway space. The former indicates that a configurational constraint has restricted weaving vehicles to N_W(max.) where they would otherwise have claimed a larger portion of the roadway.

As the determination of constrained or unconstrained states depends upon the value of S_{NW} assumed, IT IS VITAL THAT TRIALS START WITH A HIGH VALUE, WITH SUBSEQUENT TRIALS MOVING TOWARD LOWER SPEEDS.

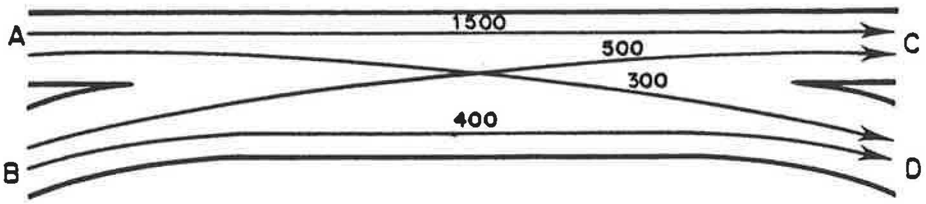
Step 3. Assume a value of S_{NW} (for unskilled users, 50 or 60 mph is a reasonable starting point.)

Step 4. Determine S_W using Figure 3.5 for the appropriate type of weaving section. A sample determinations of S_W is shown on the figure. In all cases the figure is entered on the vertical axis with the assumed value of S_{NW}. A horizontal line is constructed to the intersection with the appropriate L line for ramp-weaves, or VR for major weaves. A vertical line is constructed from this intersection to the horizontal axis, where a value of S_W is read. If the user prefers, the equations of Table 3.2 may be used directly.

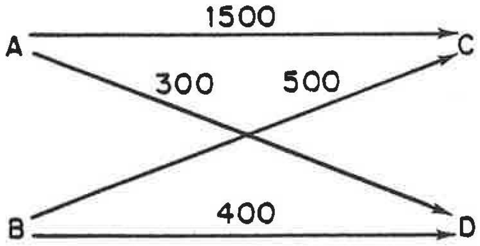
Step 5. Determine N_W(max.) from Figure 3.6 for Major Weaves. N_W(max.) for ramp-weaves is 2.0. The footnote on Figure 3.6 indicates an approximation procedure for Type I sections outside the 400-700 foot range.

The figure is entered from the vertical axis with a value of R. A horizontal line is constructed to the intersection with the appropriate L line. A vertical line is constructed from this intersection to the horizontal axis, where a value of N_W(max.) is read. An example is shown on Figure 3.6. The user may choose to use the equations of Table 3.2 directly.

The comparison between the TYPE II major weave 500 ft. curve and the TYPE I major weave curve (for a similar length range) clearly indicates the superiority of Type



SECTION AND FLOWS



WEAVING DIAGRAM

THEN:

V_{W1} = WEAVING FLOW WITH THE HIGHEST NUMERIC VALUE (500)

V_{W2} = WEAVING FLOW WITH THE SMALLEST NUMERIC VALUE (300)

V_W = TOTAL WEAVING FLOW (500+300=800)

V_{o1} = NON-WEAVING FLOW WITH THE HIGHEST NUMERIC VALUE (1500)

V_{o2} = NON-WEAVING FLOW WITH THE SMALLEST NUMERIC VALUE (400)

V = TOTAL VOLUME (500+300+1500+400=2700)

R = WEAVING RATIO = V_{W2}/V_W (300/800=0.375)

VR = VOLUME RATIO = V_W/V (800/2700=0.296)

FIGURE 3.4

CONSTRUCTION OF WEAVING DIAGRAMS AND
COMPUTATION OF PARAMETERS

II major weave sections, as they are less restrictive of N_W. In general, Type I major weave designs should be avoided where possible.

Step 6. Determine N_W/N using Figure 3.7 for the appropriate type of section. An example is shown.

In all cases, the figure is entered on the left-hand horizontal axis with a value of S_W for ramp-weaves, or L for major weaves. A vertical line is constructed to the intersection with the appropriate L curve for ramp-weaves or ΔS curve for major weaves ($\Delta S = S_{NW}-S_W$). From this intersection, a horizontal line is constructed to the intersection with the appropriate VR curve. From this second intersection, a vertical line is constructed to the right-hand horizontal axis where a value of N_W/N is read. The user may elect to use the equations of Table 3.2 directly.

Step 7. Compute $N_W = N \times (N_W/N)$ and compare N_W with $N_W(max)$ to determine whether the section is constrained or unconstrained.

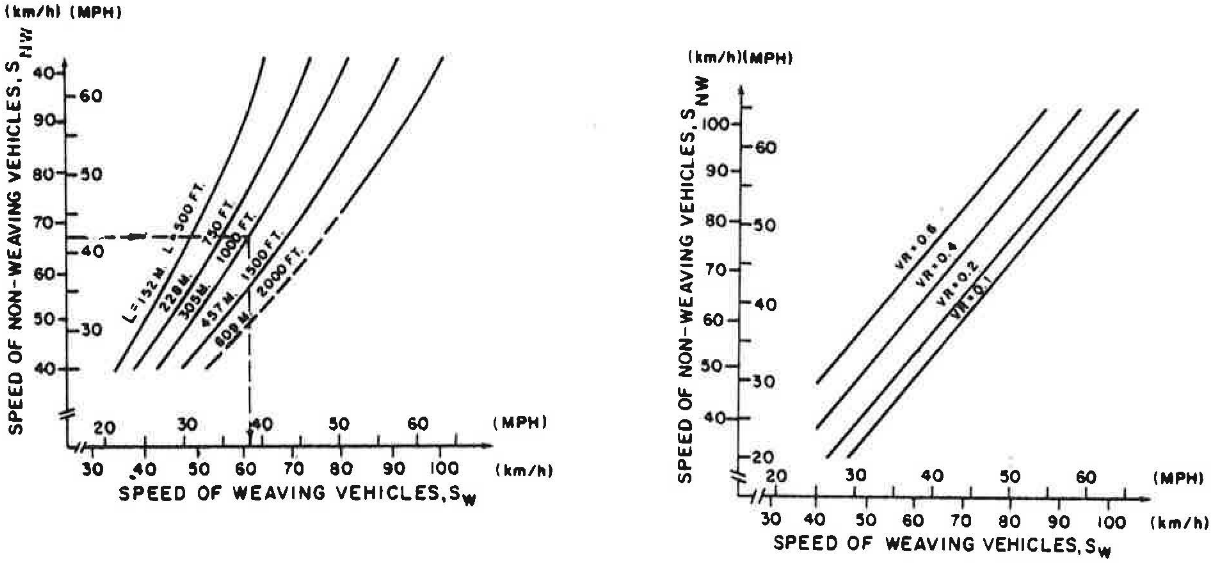
If $N_W > N_W(max.)$, section is CONSTRAINED (go to step 8)

If $N_W \leq N_W(max.)$, section is UNCONSTRAINED (go to step 9)

Step 8. a) Set $N_W = N_W(max.)$; $N_W/N = N_W(max.)/N$

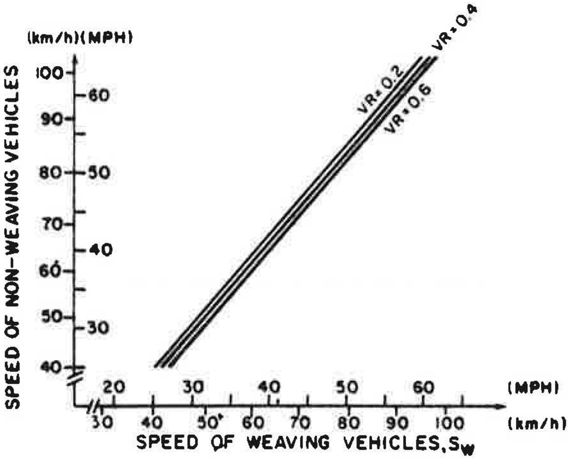
b) Compute $N_{NW} = N - N_W(max.)$

c) Determine S_{NW} from Figure 3.8. An example is shown on the figure, which is entered on the vertical axis with a value of V_{NW}



(a) RAMP-WEAVES
(All Cases)

(b) TYPE I MAJOR WEAVES
(Unconstrained Cases Only)



EXAMPLE

Ramp-weave
S_{NW} = 42 mph
L = 1000 ft.
Thus, S_W = 38 mph

(c) TYPE II MAJOR WEAVES
(Unconstrained Cases Only)

FIGURE 3.5
SPEED RELATIONSHIPS FOR WEAVING AREAS

EQUATIONS $^*\text{LOG } N_W(\text{MAX.}) = 0.714 + 0.480 \text{ LOG } R$ (TYPE I)

$\text{LOG } N_W(\text{MAX.}) = 0.896 + 0.189 \text{ LOG } R - 0.402 \text{ LOG } L_H$ (TYPE II)

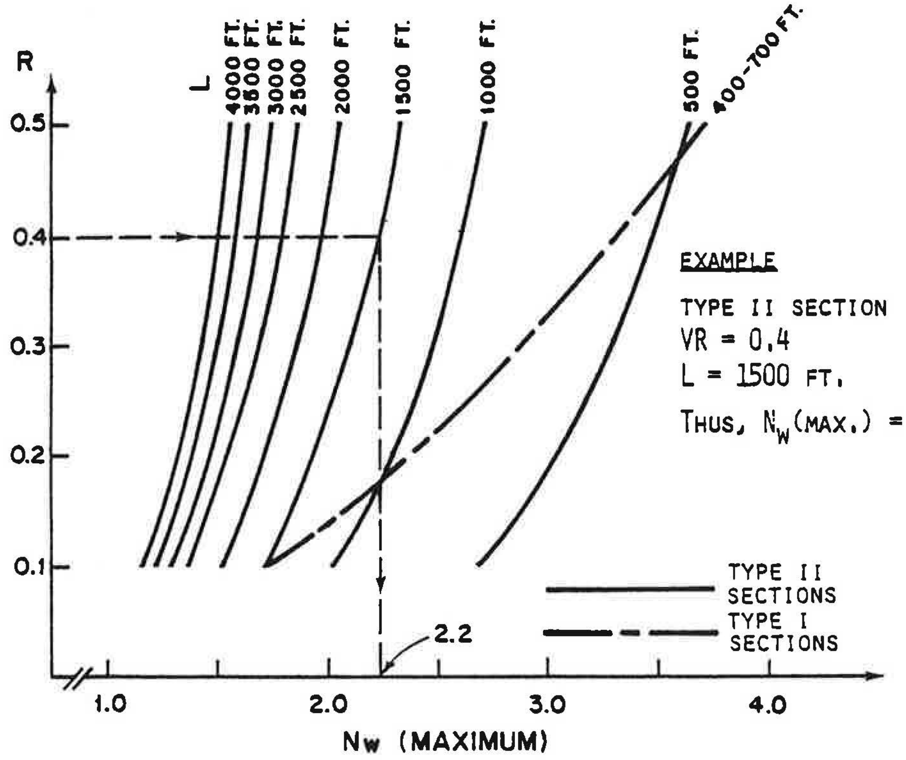


FIGURE 3.6

MAXIMUM VALUES OF N_W IN MAJOR WEAVING SECTIONS

*DATA BASE FOR TYPE I CURVE LIMITED TO LENGTHS IN THE ORDER OF 400 TO 700 FEET. FOR OTHER LENGTHS, MULTIPLY THE VALUE FROM THE TYPE II CURVE OF APPROPRIATE LENGTH BY 0.85 AS A ROUGH ESTIMATE.

NOTE: 1 FT. = 0.3048 M.

A horizontal line is constructed to the intersection with the appropriate N_{NW} curve. A vertical line is constructed from this intersection to the horizontal axis where a value of S_{NW} is read. The user may wish to use the indicated equation directly.

d) Determine S_W from Figure 3.5 for ramp-weaves and Figure 3.7 for Major Weaves. These figures represent primary relationships which are valid for both constrained and unconstrained cases. Secondary relationships are invalid for constrained cases. Figure 3.7 is entered with a value of N_W/N for this step, and is used in reverse to the use in Step 6.

e) Using S_{NW} and S_W, and comparing to the criteria in Table 3.1, determine non-weaving and weaving Levels of Service for existing or assumed conditions.

The problem is complete - no further trials are needed once segment is determined to be constrained.

Step 9. a) Compute $N_{NW} = N - N_W$

b) Determine S_{NW} using Figure 3.8.

c) If S_{NW} determined in b) is not equal to S_{NW} assumed (within ± 2 mph), assume another speed and repeat computations.

d) If S_{NW} determined in b) equals S_{NW} assumed, then take S_{NW} and S_W computed in Step 4 and determine Levels

EXAMPLE

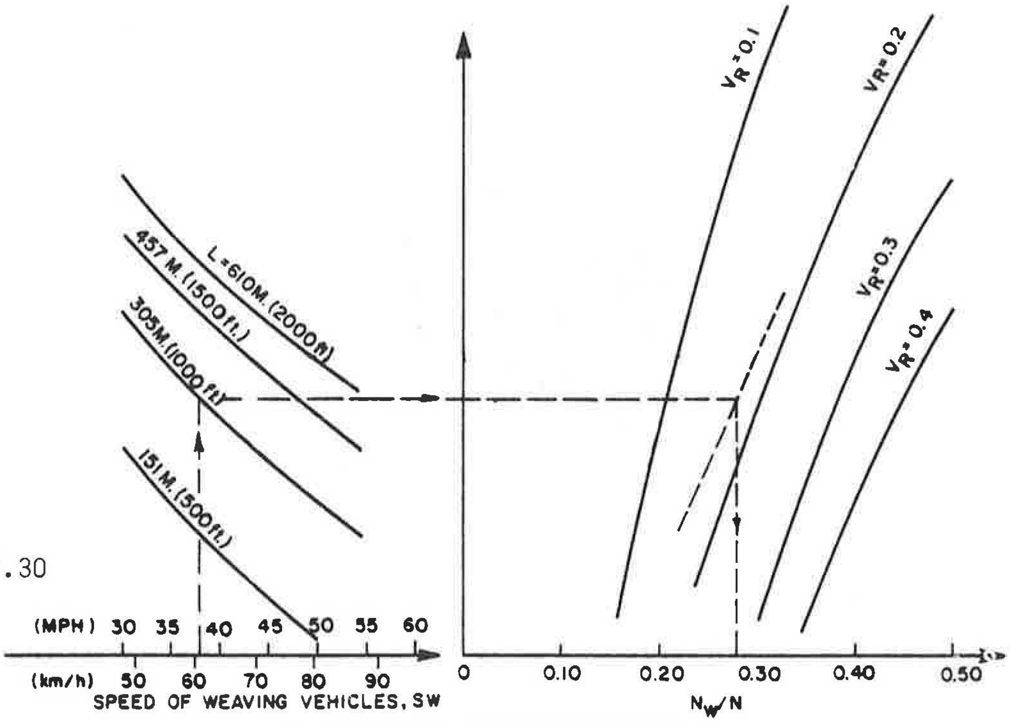
S_W = 37 mph,

L = 1000 ft.,

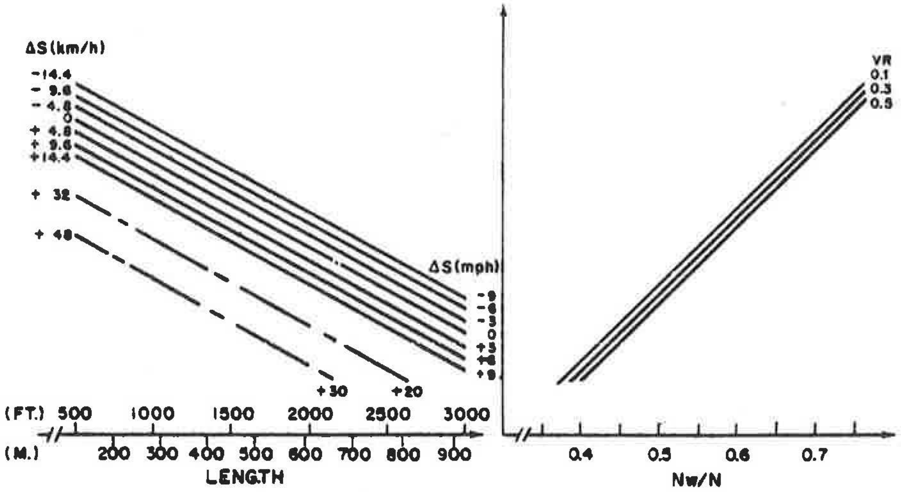
VR = 0.18

Ramp-weave

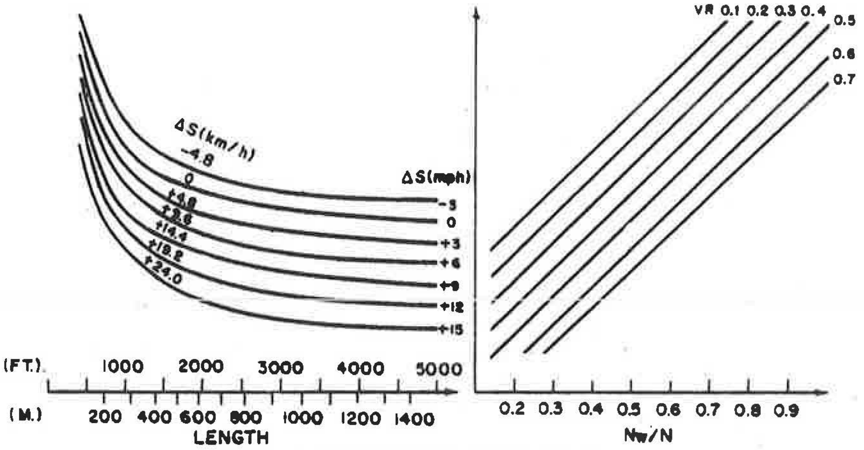
Thus, N_W/N = 0.30



(a) RAMP-WEAVES (Unconstrained Cases Only)



(b) TYPE I MAJOR WEAVES (All Cases)



(c) TYPE II MAJOR WEAVES (All Cases)

FIGURE 3.7

SHARE OF THE ROADWAY RELATIONSHIPS
FOR WEAVING VEHICLES

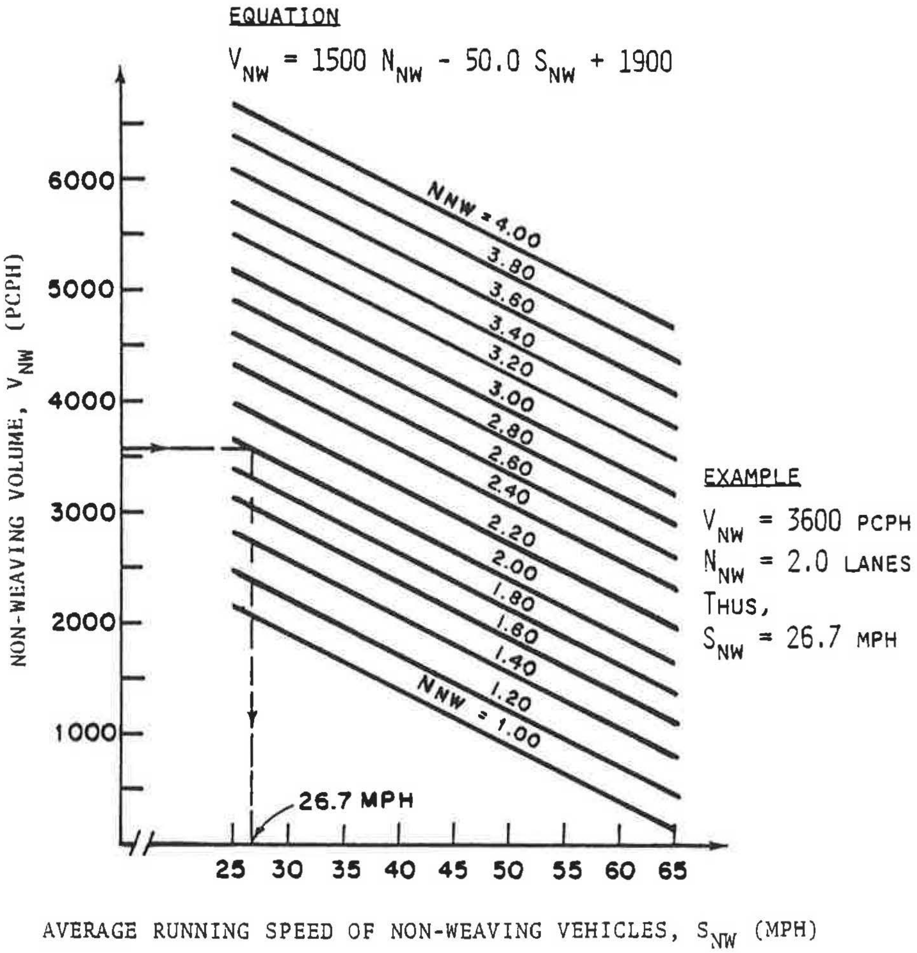


FIGURE 3.8

SPEED-FLOW RELATIONSHIP FOR NON-WEAVING
VEHICLES IN A WEAVING SECTION

NOTE: 1 mph = 1.6 km/h.

of Service for weaving and non-weaving vehicles by comparing to the criteria of Table 3.1.

It is important to note that while initial trials may appear to be unconstrained, as the user closes to a solution, later trials may show a constraint. This underscores the need to choose trial speeds properly; i.e., beginning with high speeds to low. If this is not done, a constraint may appear, ending the computation, while an unconstrained solution exists at another speed.

Table 3.3 gives an index to the figures used in computational procedures.

Multiple Weaving Sections

The 1965 HCM (2) contains procedures for the analysis of two-segment multiple weaving sections based upon the principle of proportional distribution of overlapping weaving movements by length. The Weaving Area Operations Study (3) found that the assumption of proportional weaving was not supported by field data collected in that study. Rather, each weaving movement tends to concentrate in a single segment of a multiple weave, the selection of which depends upon two principles, which are competing:

> necessity to weave -- most drivers will not weave until they have to - i.e., weaves will tend to concentrate in the last segment before their diverge point.
>
> Presegregation -- most drivers enter a weaving section already in lanes appropriate to their desired travel path; drivers may execute an "early" weave in a multiple weaving section to presegregate themselves from other components of flow.

The governing principle will virtually always be the one which results in the least interference between component flows in the segment. For all practical purposes, these principles result in the same distribution of flows for all multiple weaving sections of a similar type.

TABLE 3.3

INDEX TO FIGURES USED IN WEAVING PROBLEMS

EQUATION TYPE	RAMP-WEAVES AND TYPE III MAJOR WEAVES	MAJOR WEAVES	
		TYPE I	TYPE II
Speed Relationship	Figure 3.5^1	Figure 3.6^2	Figure 3.8^2
Portion of Roadway Used By Weaving Vehicles	Figure 3.9^2	Figure 3.10^1	Figure 3.11^1
Maximum N_W	N_W = 2.0	Figure 3.8^1	Figure 3.8^1
Non-Weaving Vehicles	Figure 3.12	Figure 3.12	Figure 3.12

1) Figure depicts PRIMARY relationship

2) Figure depicts SECONDARY relationship

Figures 3.9 and 3.10 illustrate the component flows for multiple weaves formed by a single merge followed by two diverge points and those formed by two merge points followed by a single diverge respectively. It is judged that these will virtually always hold, except in cases where one segment of the weave is extremely short with respect to the other segment (a ratio of 4:1 or more). In such cases, only those weaves which must take place in the short segment will - all others will occur in the longer segment.

In Figure 3.9, movement 5 must weave with both movements 3 and 4 within the first segment. Movement 2, which could weave with movement 3 throughout both segments, in fact weaves only in segment 2.

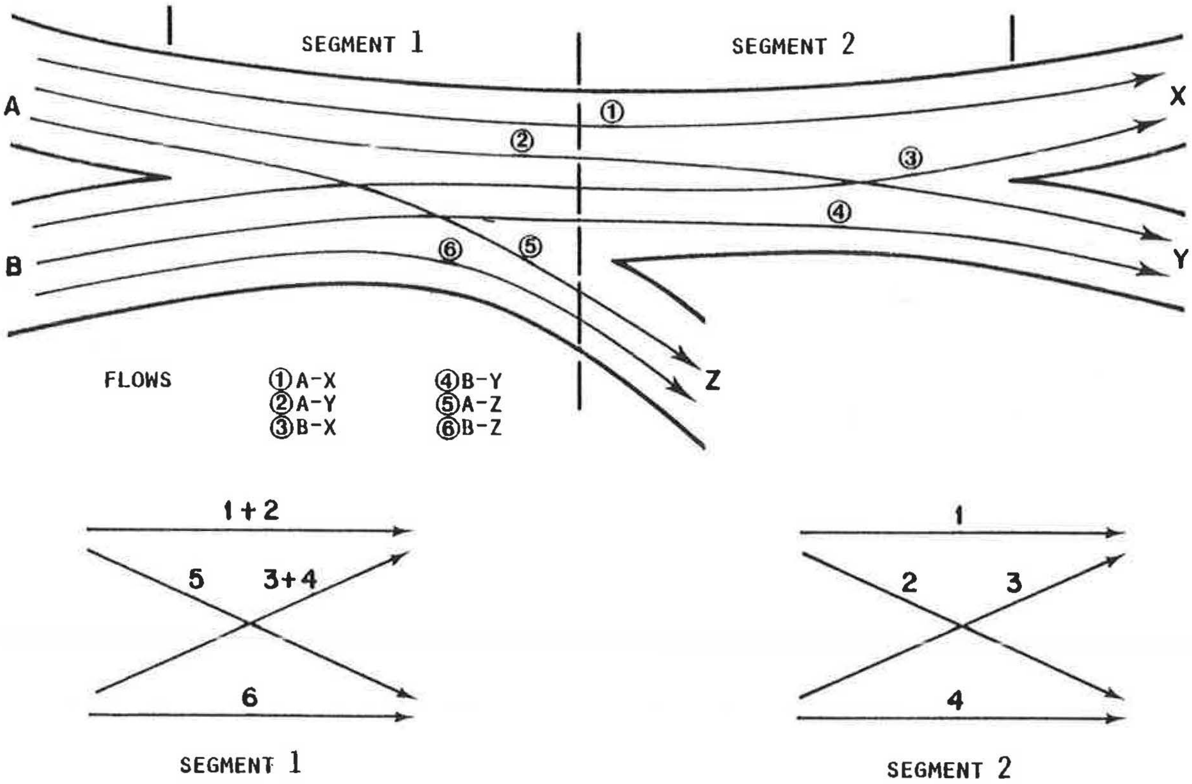


FIGURE 3.9

WEAVING FLOWS IN A MULTIPLE WEAVE FORMED BY A SINGLE MERGE FOLLOWED BY TWO DIVERGE POINTS

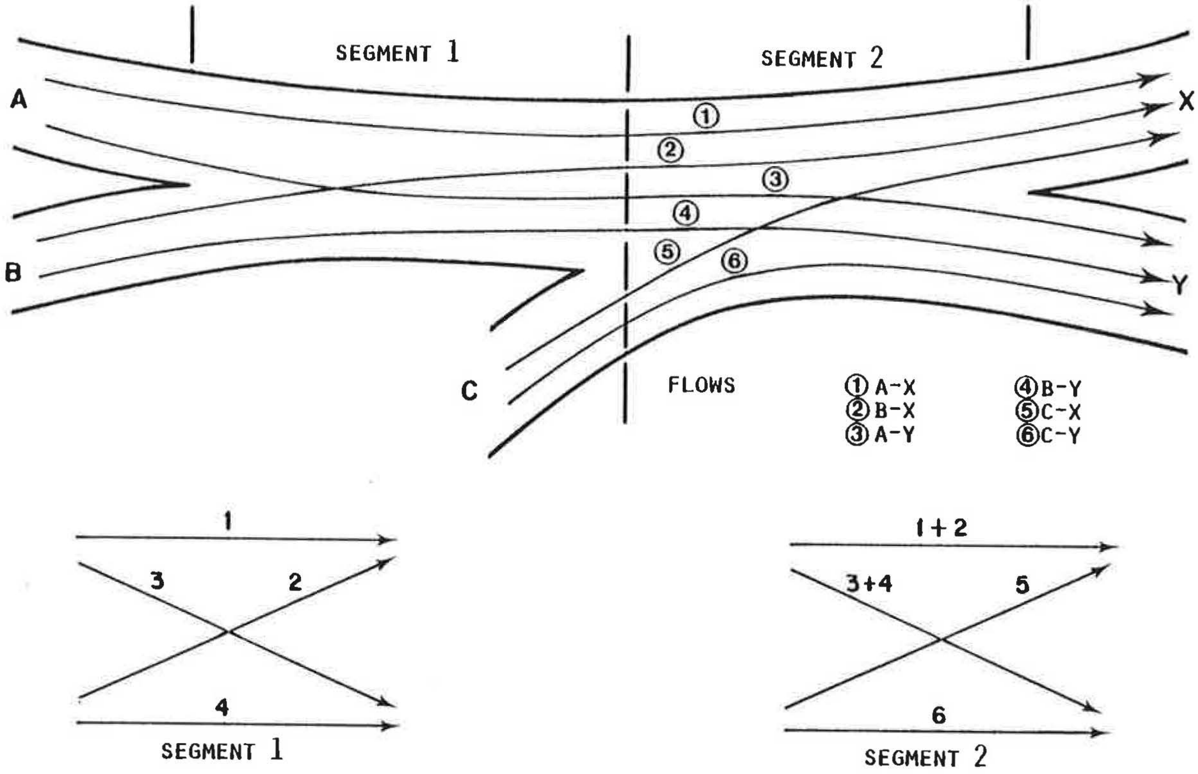


FIGURE 3.10

WEAVING FLOWS IN A MULTIPLE WEAVE FORMED BY TWO MERGE POINTS FOLLOWED BY A SINGLE DIVERGE

This is an application of the "necessity to weave" principle, resulting in a late weave intended to avoid conflict with other weaving movements in segment 1.

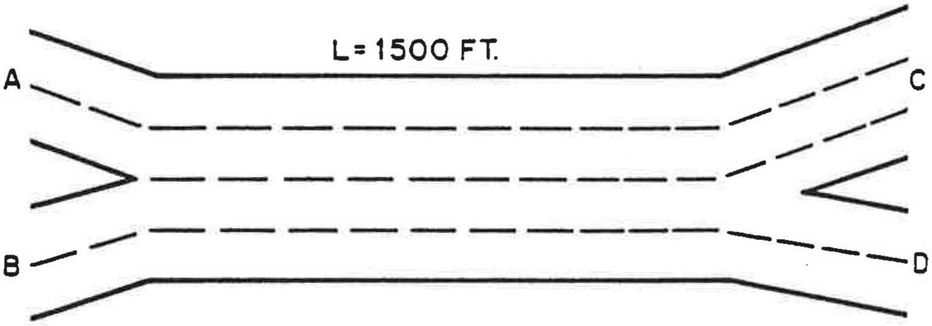
Figure 3.10 depicts the situation in which movements 3, 4 and 5 must weave in the second segment of the weaving area. Movements 2 and 3 could conceivably weave throughout both segments of the weave. In fact, this weave occurs virtually entirely in segment 1, an application of the "pre-segregation" principle. In this case, these vehicles execute an early weave to avoid conflict with other weaving movements on segment 2.

III. SAMPLE PROBLEMS

In all sample problems, judgment has been used to pick an initial value of S_{NW} for trial. The inexperienced user should generally start with an assumption of 50 mph.

Problem 1: A Major Weaving Section

Consider the following weaving area:



Volumes:

AC = 1815 vph	5% Trucks
AD = 692 vph	PHF = 0.91
BC = 1037 vph	Level Terrain
BD = 1297 vph	

At what Level of Service will the facility operate during peak periods?

SOLUTION: The first step of the solution is the conversion of peak hour volumes in mixed vph to peak flow rates in pcph. This is done by dividing the above volumes by PHF and Q, the commercial vehicle factor, or

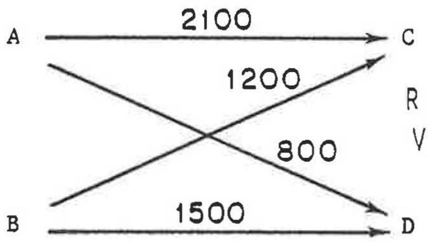
$$\text{Peak Flow Rate} = \frac{\text{Peak Hour Volume}}{\text{PHF X Q}}$$

Where PHF = 0.91

Q = 0.95 (Table 2.5, 5% trucks, E_t = 2 from Table 2.4)

Then: AC = 1815/ (0.95 X 0.91) = 2100 pcph
AD = 692/ (0.95 X 0.91) = 800 pcph
BC = 1037/ (0.95 X 0.91) = 1200 pcph
BD = 1297/ (0.95 X 0.91) = 1500 pcph

and a weaving diagram may be drawn.



R = 800/2000 = 0.400
VR = 2000/5600 = 0.357

The analysis should begin with an assumption of S_{NW}. As the total volume is 5600 pcph in 4 lanes, or 1400 pcphl, moderately low speeds might be expected. A speed of 35 mph might reasonably be tried. Note that this is a Type II major weaving section, as it provides a "through" lane for one weaving movement, and has lane balance at the exit gore.

Try: S_{NW} = 35 mph

Then:
- Find S_W using Figure 3.5 S_W = 34.5 mph Therefore, ΔS = 35.0 - 34.5 = + 0.5 mph
- Find N_W(max.) using Figure 3.6 N_W(max.) = 2.20 lanes. (This is the example shown on Figure 3.8).
  Note: This value remains constant through all iterations.
- Find N_W/N using Figure 3.11 N_W/N = 0.49
- N_W = 4 x 0.49 = 1.96 lanes As N_W (1.96) is less than N_W(max.), the section is UNCONSTRAINED
- N_{NW} = 4 - 1.96 = 2.04 lanes
  Now find S_{NW} using Figure 3.12, using 2.04 lanes and V_{NW} = 3600 S_{NW} = 28 mph

as this does not agree closely with the assumed value of 35 mph, another trial is necessary.

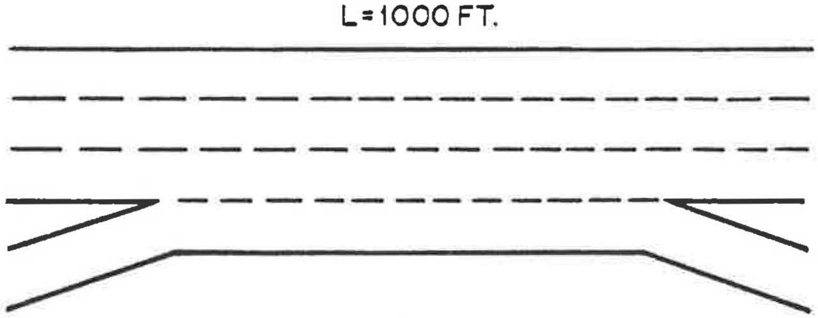
The first assumed value was obviously too high. For a second trial, it might be thought that a value between the assumed and computed speeds should be selected. One factor tempers this. The procedure is such that as the assumed S_{NW} is decreased, so also is the computed value, but at a slower rate. The two values do not naturally work towards one another. The second trial, therefore, should be lower than 28 mph.

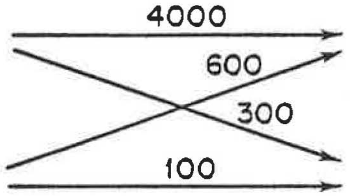
TRY: S_{NW} = 25 mph
- Find S_W from Figure 3.5
  S_W = 25.5 mph
  Therefore, ΔS = 25.0 - 25.5 = 0.5 mph
- N_W(max.) = 2.20 (from previous trial)
- Find N_W/N from Figure 3.7
  N_W/N = 0.510
- N_W = 4 x 0.510 = 2.04 lanes
  as 2.04 is less than N_W(max.) of 2.20, section is still UNCONSTRAINED
- N_{NW} = 4-2.04 = 1.96 lanes
  Now find S_{NW} Figure 3.8, using 1.96 lanes and V_{NW} = 3600
  S_{NW} = 25 mph

As the assumed and computed values agree, the solution is completed. The section will operate in an unconstrained condition with S_{NW} = 25 mph and S_W = 25.5 mph. From Table 3.1, this is Level of Service F for non-weaving vehicles - and Level of Service F for weaving vehicles - an obviously undesirable condition.

Problem 2: A Ramp-Weave Section

Consider the following weaving section. All volumes are given as peak flow rates in pcph:





R = 300/900 = 0.333
VR = 900/5000 = 0.180

SOLUTION: As the total flow of 5000 pcph in 4 lanes represents an average of 1250 pcph, a relatively good speed might be expected. Noting that this is a ramp-weave, and that trials should begin from high speeds to low, a first trial will be made at S_{NW}=60 mph.

TRY S_{NW} = 60 mph

- Find S_W from Figure 3.5

  S_W = 49.0 mph
- N_W(max.) = 2.0 for all ramp-weaves (Table 3.2)
- Find N_W/N from Figure 3.7

  N_W/N = 0.26
- N_W = 4 x 0.26 = 1.04 lanes

  As 1.04 is less than N_W(max.) of 2.0, section is UNCONSTRAINED
- N_{NW} = 4 - 1.04 = 2.96 lanes

  Now, find S_{NW} from Figure 3.8, using 2.96 lanes and V_{NW}= 4100. S_{NW} = 45 mph

Since fairly close agreement between assumed and computed values has not been realized, a second trial is made at a value somewhat less than 45 mph.

TRY: S_{NW} = 42 mph

- Find S_W from Figure 3.5

  S_W = 38 mph
- N_W(max.) = 2.0 (from first trial)
- Find N_W/N from Figure 3.7

  N_W/N = 0.29
- N_W = 4 x 0.29 = 1.16

  As N_W is less than 2.0, section is UNCONSTRAINED
- N_{NW} = 4 - 1.16 = 2.84

  Now, find S_{NW} from Figure 3.8, using 2.84 lanes and V_{NW} = 4100.

  S_{NW} = 56 mph

The agreement is exact, and the problem is complete. The facility is expected to operate in the unconstrained state with S_{NW} = 42 mph and S_W= 38 mph, corresponding to Levels of Service C for non-weaving vehicles and C for weaving vehicles.

Problem 3: Illustration of a Constrained Section

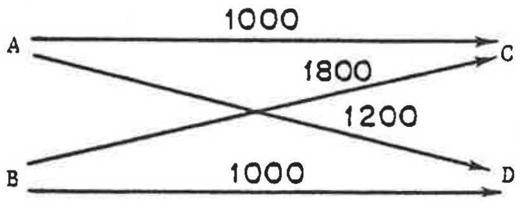
Consider the weaving segment in Problem 1, with a reduced length of 1000 feet and the following volumes:

AC = 655 vph	BC = 1178 vph
AD = 785 vph	BD = 655 vph

PHF = 0.85; 10% trucks; Rolling Terrain

SOLUTION: These volumes must again be converted to peak flow rates in pcph. This is done by dividing the PHF and Q. For 10% trucks in rolling terrain, E_t = 4 (Table 2.4) and Q = 0.77 (Table 2.8). Thus:

AC = 655/ (0.85 X 0.77) = 1000 pcph
AD = 785/ (0.85 X 0.77) = 1200 pcph
BC = 1178/ (0.85 X 0.77) = 1800 pcph
BD = 655/ (0.85 X 0.77) = 1000 pcph



R = 1200/3000 = 0.40
VR = 3000/5000 = 0.60

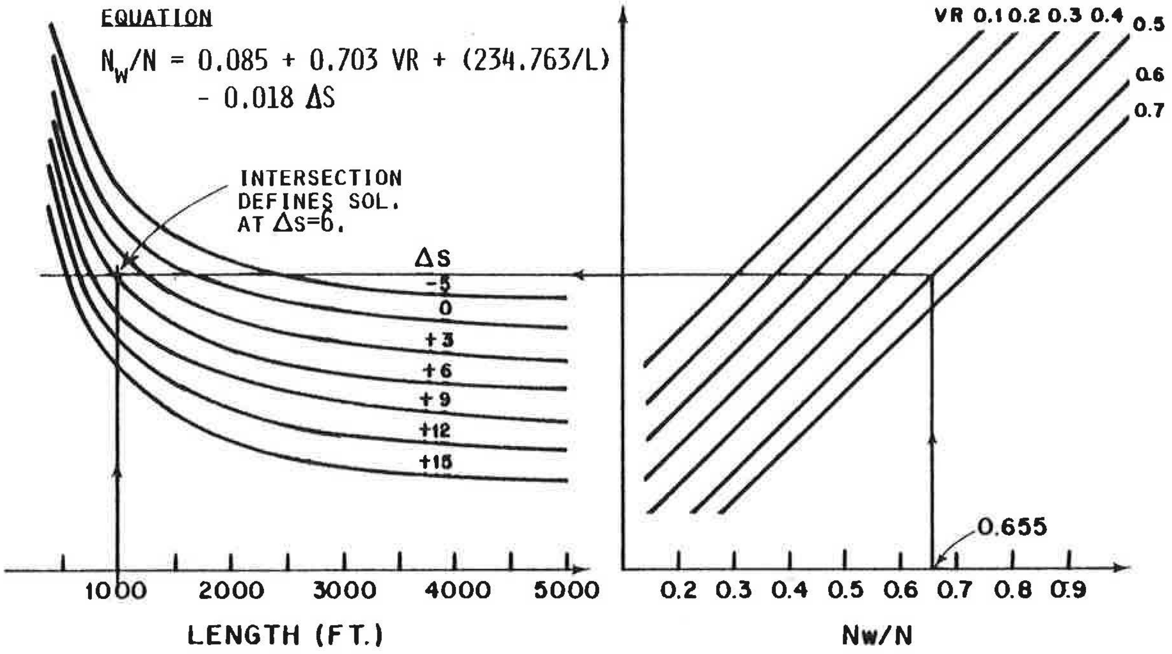


FIGURE 3.11

SOLUTION FOR ΔS IN PROBLEM 3 USING FIGURE 3. 7

NOTE: 1 MPH = 1.6 KPH

Although the total volume of 5000 vehicles in the 4 lane (1250 pcphpl) is not unusually high, 60% of these are weaving vehicles, and the potential for congestion is present. A first trial of S_{NW} = 45 mph appears reasonable.

TRY: S_{NW} = 45 mph

- Find S_W using Figure 3.5
  S_W = 44.3 mph
  Therefore, ΔS = 45.0 - 44.3 = 0.7 mph
- Find N_W(max.) from Figure 3.6
  N_W(max.) = 2.62
- Find N_W/N = 0.740 from Figure 3.7
- N_W = 0.74 X 4 = 2.96 lanes
  As 2.96 is more than the N_W(max.) of 2.62 lanes, the section is CONSTRAINED
- Therefore, set N = 4

  N_W = 2.62

  N_{NW} = 4 - 2.62 = 1.38

  N_W/N = 2.62/4 = 0.655

- Find S_{NW} from Figure 3.8, using 1.38 lanes and V_{NW} = 2000.
  S_{NW} = 39.5 mph
- Find ΔS from Figure 3.7
  ΔS=6 mph (this solution is illustrated in Figure 3.11: ΔS is defined by the intersection of the two constructed lines in Figure 3.11)

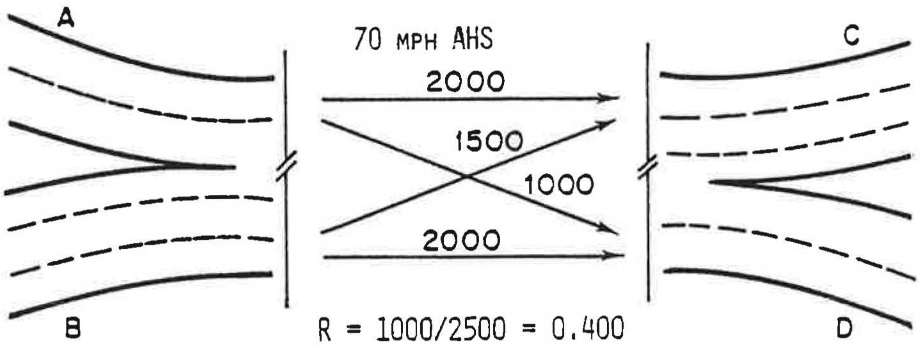
  S_W = 39.5 - 6.0 = 33.5 mph
- Computations are complete. Levels of Service are D for non-weaving vehicles and E for weaving vehicles (Table 3.1).

To be absolutely certain that a section is indeed constrained, a trial at a higher speed should be made to see if an unconstrained solution exists at a higher speed. In this case, the constraint is clear, and a trial of 50 or 55 mph would confirm the constraint.

Problem 4: A Design Application

A weaving section is being considered as a major junction between two urban freeways. The pre-determined configuration of entry and exit legs follows, as do demand volumes, expressed as peak flow rates in pcph. Design constraints limit the length to a maximum of 1500 feet. A design Level of Service commensurate with that existing on entry and exit roadways is desired.

SOLUTION: To determine the desired Level of Service, it is first necessary to find the Level of Service anticipated on entry and exit roadways. This is done below:



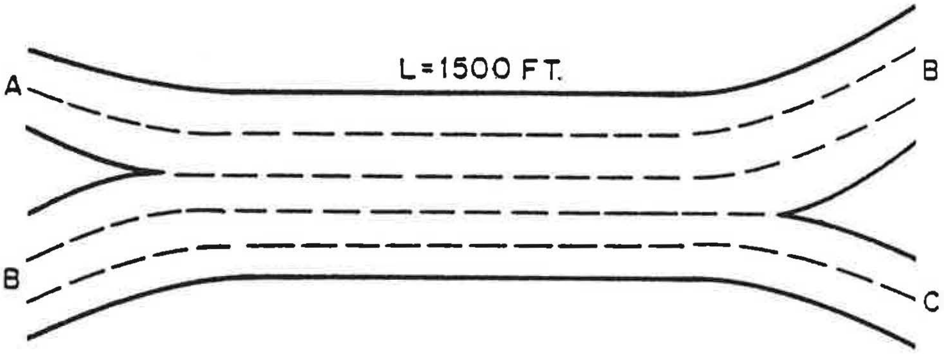
R = 1000/2500 = 0.400

VR = 2500/6500 = 0.385

Leg	Volume	Level of Service (Table 2.1)
A	3000	C (4-lane freeway)
B	3500	B (6-lane freeway)
C	3500	B (6-lane freeway)
D	3000	C (4-lane freeway)

Therefore, a design at Level of Service B, would be most desirable, if it can be achieved, but Level C would also be acceptable. The obvious trial design would consist of 5 lanes and the maximum allowable length of 1500 ft., as shown below.

- Find N_W/N from Figure 3.7
  N_W/N = 0.61
- N_W = 5 x 0.61 = 3.05 lanes.
  As 3.05 lanes is more than N_W(max.) of 1.90, the section is CONSTRAINED.
- Set N_W = 1.87, N_{NW} = 5 - 1.87 = 3.13
  Then, N_W/N = 1.87/5 = 0.374



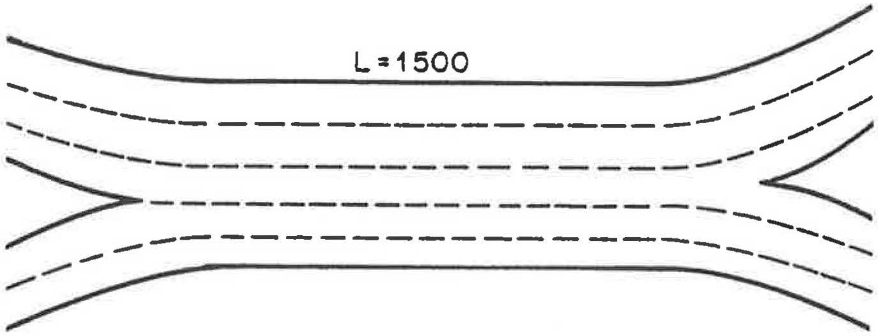
Note that this is a Type 1 major weaving area. An analysis must now be conducted to determine how well the proposed segment would perform. The 6500 total flow in 5 lanes is 1300 pcphpl with 40% weaving vehicles. An initial trial of S_{NW} = 40 mph appears to be reasonable.

TRY: S_{NW} = 40 mph

- Find S_W from Figure 3.5
  S_W = 38.5 mph
  Therefore, ΔS = 40.0 -38.5 = 1.5 mph
- Find N_W(max.) from Figure 3.6 - note the footnote which instructs user to take 85% of the corresponding Type II value for lengths outside the 400-700 ft. range. N_W(max.) = 0.85 (2.20) = 1.87 lanes
  (solution for 2.20 is illustrated on Figure 3.6)
- Find S_{NW} from Figure 3.8, using V_{NW}= 4000 and N_{NW}= 3.13 S_{NW} = 52.0 mph (Level of Service A, Table 3.1)
- Find ΔS from Figure 3.7, as illustrated in Figure 3.12. ΔS = 40 mph -- a very rough estimate, as the value is quite a bit off the calibrated range of the algorithm. Thus, S_W = 12.0 mph. This is Level of Service F from Table 3.1.

Operations in the proposed segment are obviously intolerable, and the Type I design could not be adopted. The constraint is extreme, and further trials at higher speeds are not really necessary.

A Type II design of 1500 feet might be attempted by adding a lane to leg D. This lane would then be dropped at a convenient location at least 3,000 to 4,000 feet downstream. The new proposed design is illustrated below:



Another analysis will be conducted, starting with an assumption of S_{NW} = 40 mph.

TRY: S_{NW} = 40

- Find S_W using Figure 3.5
  S_W = 38.8 mph
  Therefore ΔS = 40.0 - 38.8 = 1.2 mph
- Find N_W(max.) using Figure 3.6
  N_W(max.) = 2.2 (This is the problem illustrated in Figure 3.6)
- Find N_W/N using Figure 3.7
  N_W/N = 0.48
- N_W = 5 x 0.48 = 2.4 lanes
  As 2.4 is greater than N_W(max.) of 2.20, the section is CONSTRAINED, although it could be considered to be a borderline case.
- Set N_W = 2.20, N_{NW} = 2.80
  Therefore N_W/N = 2.2/5 = 0.44
- Find S_{NW} from Figure 3.8, using V_{NW}= 4000 and N_{NW}= 2.80.
  S_{NW} = 42.5 mph (Level of Service C, Table 3.1)
- Find S_W from Figure 3.7
  ΔS = 4 mph
  S_W= 42.5 - 4 = 38.5 mph (Level of Service C, Table 3.1)

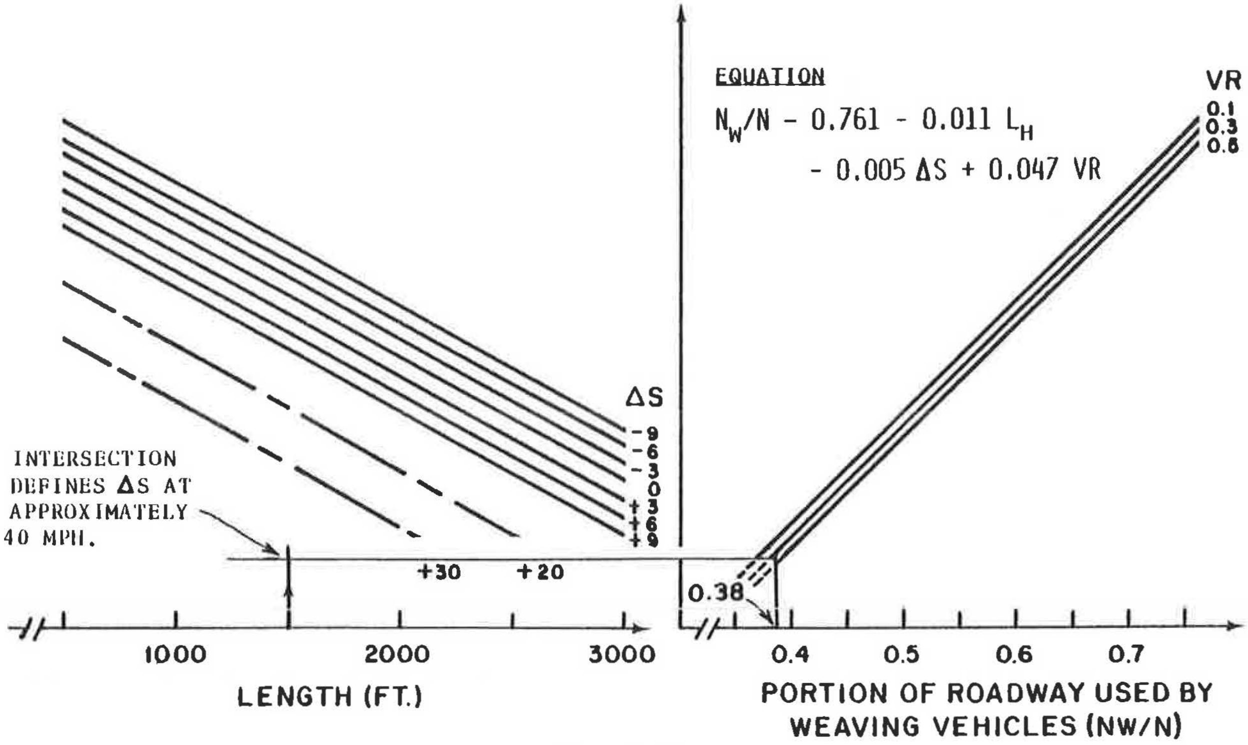


FIGURE 3.12

SOLUTION FOR ΔS IN PROBLEM 4 USING FIGURE 3. 7

NOTE: 1 mph = 1.6 km/h.
1 ft. = 0.3048 m.

To be sure that the determination of constraint is valid, a trial with a speed of S_{NW} = 50 mph should be made. This is done here as the constraint is clearly borderline, and because the solution speeds are higher than 40 mph, which was assumed. A quick check yields the following:

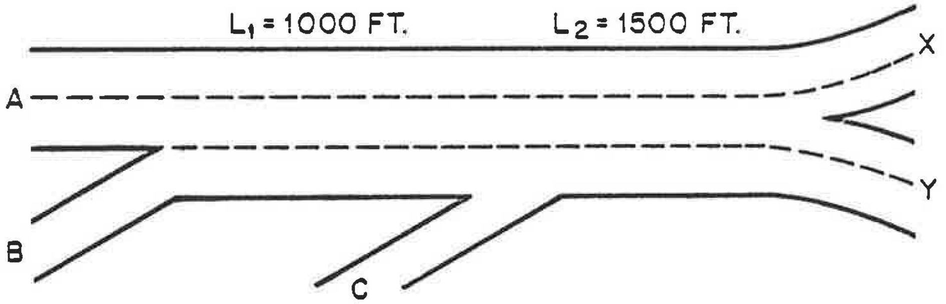
If S_{NW} = 50 mph, S_W = 47, ΔS = 3 mph, N_W/N = 0.47; N_W = 2.35 lanes which is still greater than N_W(max.) of 2.2. The constraint is therefore valid, and the above solution holds.

The Type II design could be adopted, as Level of Service of C is acceptable during peak periods.

This problem, however, clearly indicates the great advantage of Type II major weaving sections over Type I major weaves, which are far more restrictive to weaving vehicles, and produce generally worse operations than similar Type II major weaving sections.

Problem 5: A Multiple Weave

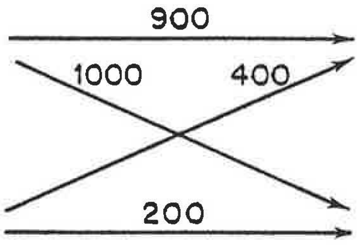
At what Level of Service would the following weaving section operate. All volumes are expressed as peak flow rates in pcph:



Flows:
A-X= 900 pcph (1) B-X=400 pcph (2) C-X=300 pcph (5)
A-Y=1000 pcph (3) B-Y=200 pcph (4) C-Y=100 pcph (6)

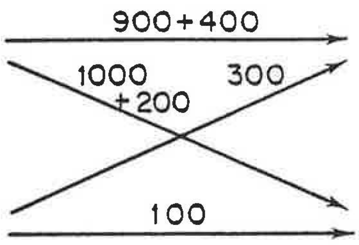
SOLUTION: Referring to Figure 3.10, weaving diagrams for the two segments may be constructed as follows:

SEGMENT 1



$R = 400/1400 = 0.286$
$VR = 1400/2500 = 0.560$

SEGMENT 2



$R = 300/1500 = 0.200$
$VR = 1500/2900 = 0.517$

Note that flow numbers conforming to those in Figure 3.10 have been indicated above for convenience. Both segments would be treated as Type II major weaves. This is based upon the fact that one weaving movement can be made without a lane change, and the other with only one lane change. Were two lane changes required for the second weaving movement, Type I procedures would be used; and if all weaving movements required a lane change, ramp-weave equations would be used.

Consider segment 1, with 2500 pcph in 3 lanes, with a length of 1000 ft. (305 m.). High speeds would be expected, and an initial trial of S_{NW} = 50 mph will be made.

TRY: S_{NW} = 50 mph

- Find S_W using Figure 3.5
  S_W = 48 mph
  Therefore ΔS = 50 - 48 = 2 mph
- Find N_W(max.) using Figure 3.6
  N_W(max.) = 2.46 lanes
- Find N_W/N from Figure 3.7
  N_W/N = 0.68
- N_W = 0.68 x 3 = 2.04 < 2.46 lanes
  UNCONSTRAINED
  N_{NW} = 3 - 2.04 = 0.96 lanes
- Find S_{NW} from Figure 3.8
  S_{NW} = 46 mph (outside range of nomograph)

A second trial with S_{NW} = 42 mph will be made.

TRY: S_{NW} = 42 mph

- Find S_W from Figure 3.5
  S_W = 41.6 mph
  Therefore, ΔS = 0.4 mph
- Find N_W/N from Figure 3.7
  N_W/N = 0.710
- N_W = 3 x 0.71 = 2.13 lanes < 2.46 lanes
  UNCONSTRAINED
  N_{NW} = 3 - 2.13 = 0.87 lanes
- Find S_{NW} using Figure 3.8
  S_{NW} = 42 mph

As agreement between assumed and completed values is exact, computations cease here. Non-weaving vehicles will average 42 mph (Level of Service C, Table 3.1) and weaving vehicles 41.6 mph (Level of Service C, Table 3.1) in segment 1.

Consider segment 2, with 2900 pcph in 3 lanes, with a length of 1500 ft. (457 m.). An initial trial is made at 50 mph.

TRY: S_{NW} = 50 mph

- Find S_W from Figure 3.5
  S_{NW} = 48 mph
  Δ S = 50 - 48 = 2 mph
- Find N_W(max.) from Figure 3.6
  N_W(max.) = 1.9 lanes
- Find N_W/N from Figure 3.7
  N_W/N = 0.57
- N_W = 0.57 x 3 = 1.71 < 1.90
  UNCONSTRAINED
  N_{NW} = 3 - 1.71 = 1.29
- Find S_{NW} from Figure 3.8
  S_{NW} = 49 mph

Segment 2 will, therefore, operate with non-weaving speeds of 50 mph (Level of Service A, Table 3.1) and weaving speeds of 48 mph (Level of Service A, Table 3.1). Segment 2 operates better than segment 1 primarily due to its longer length and lower percentage of weaving vehicles. (The equations and nomographs illustrate both these points.)

CHAPTER III - REFERENCES

1) Pignataro, et al, "Weaving Areas: Design and Analysis," NCHRP Report 159, 1975.

2) "Highway Capacity Manual," TRB Special Report 87, Transportation Research Board, 1965.

3) Pignataro, et al, Weaving Area Operations Study, Final Report, National Cooperative Highway Research Program Project 3-15, Polytechnic Institute of Brooklyn, Brooklyn, N.Y., Nov. 1973.

CHAPTER IV - RAMPS AND RAMP JUNCTIONS

I. BASIC CHARACTERISTICS

Introduction

A ramp may be described as a length of roadway providing an exclusive connection between two highway facilities. This chapter concerns itself with the capacity of ramps for which at least one of the connecting facilities is a freeway. The capacity of such ramps is controlled by one of three elements:

- the ramp-freeway terminal
- the ramp proper
- the ramp-street system interface (generally an at-grade intersection)

Procedures for analyzing the capacity of at-grade intersections are included in Chapter 6 of the 1965 Highway Capacity Manual (1)and in other parts of this circular, and are not treated herein. This chapter details procedures for ramp-freeway junctions, which are usually the critical element to consider, and for ramps proper.

The design and operational characteristics of ramps and ramp-freeway junctions may have a great impact on the overall operation of a freeway. Merging and diverging vehicles will directly influence freeway vehicles in the freeway lane adjacent to the ramp, normally the lane nearest the outside shoulder (lane 1). Insufficient capacity at these merge and diverge points may create problems on the freeway itself, as exiting vehicles are prevented from leaving the freeway expeditiously, and entering vehicles cannot join the freeway stream smoothly.

There is great variability in the physical design of ramps and ramp terminals. The procedures herein are primarily applicable to high-type designs, though, where noted, some relationships may be applied to substandard cases. Physical design standards for ramps and ramp terminals are given in AASHTO (2,3), and should be carefully considered.

This chapter presents a discussion of ramp operational characteristics as well as specific computational procedures for capacity and Level of Service analyses. It treats the following subject areas:

- the ramp-freeway junction
- the ramp proper
- ramp control

The latter subject is treated qualitatively with general quantitative guidelines given. It is an area which will become increasingly important in urban area freeway operation, where construction of new or expanded facilities is difficult.

Operational Characteristics

A freeway ramp terminal is an area of competing traffic demands for space. Upstream freeway demand competes with on-ramp demand in merge areas. On-ramp demand is usually generated locally, though arterials and collector streets may bring vehicles to the ramp from more distant origins. The freeway demand upstream of an on-ramp is the composite of demands on all upstream on- and off-ramps. In the merge area, on-ramp vehicles attempt to find gaps in the lane adjacent to the ramp (usually the shoulder lane, or lane 1). As the ramp volume increases, greater pressure is exerted upon lane 1 vehicles to move into other freeway lanes in order to avoid the turbulence in the merge area.

The situation is a dynamic one in which both the freeway volume and the ramp volume are independent variables which could be altered by making changes in the ramp location and design or in the design of the upstream freeway itself. Ramp volumes may be controlled directly through the imposition of ramp control.

Off-ramp operations are similar, though it is a diverging maneuver which takes place rather than a merge. Again, the principal interaction between exiting traffic and through vehicles occurs in lane 1, upstream of the off-ramp. Exiting vehicles must occupy the lane adjacent to the ramp, and as their numbers increase, pressure on through vehicles to move into other lanes also increases. Where 2-lane off-ramps are present, the influence of diverging movements may spread over several lanes of the freeway. Again, given a basic freeway design and ramp location, the demand volumes involved are more-or-less independent variables, or inputs to an analysis.

Procedures given herein treat ramp and freeway volumes as given inputs to a ramp capacity analysis, with Level of Service, or operational characteristics as the output. This is consistent in that the ramp is a point location whose detailed design or analysis would be considered only in conjunction with an overall facility whose design or characteristics have already been established.

A ramp will operate properly only if the three elements of concern -- ramp-freeway junction, ramp proper, ramp-street system interface (if any exists) -- are compatably designed.

1) Ramp-Freeway Terminal: the ramp-freeway junction should be designed to allow vehicles to enter or leave the freeway traffic stream at the normal speed of the traffic stream, that is, the design should be such that acceleration and deceleration takes place on the ramp. This requires that adequate acceleration or deceleration lanes be provided, and that horizontal and vertical curvature near the ramp-freeway junction allow operation at appropriate speeds. Visibility is also a critical factor: on-ramp vehicles should have an adequate view of the adjacent freeway lane to allow timely selection of a gap in the freeway stream; off-ramp vehicles should be able to assess conditions at the gore area and on the ramp proper in time to permit appropriate driving decisions. Any factor which requires that merging or diverging vehicles slow significantly on the freeway itself will cause disruption to the smooth flow of traffic on the freeway. In extreme conditions, this disruption will spread to all freeway lanes.

2) Ramp Proper: the ramp proper must be designed to carry the expected ramp volume under appro-

priate operating conditions. It should provide for adequate storage of vehicles where queuing, particularly from signalized at-grade intersections, is expected. The storage of queued vehicles should not extend to a point on the ramp where it would interfere with exiting traffic at the ramp-freeway terminal. Ramps should also be designed to allow stalled or slow-moving traffic to be passed. This is a design requirement of AASHTO on ramps longer than 1000 feet.

3) Ramp-Street System Interface: the nature of many ramp-street system interface points require that they be controlled in some way, either by signs or signals. The control aspects of these at-grade intersections are complex, particularly where diamond interchanges are concerned, and will not be discussed in these procedures. In general, the design and operation of interface points should minimize queue buildups on the ramp itself, subject to constraints imposed by street-side concerns.

A ramp will operate efficiently only if all three elements have been properly designed. It is critical to note that a breakdown of any one of these elements will adversely affect the operation of the entire ramp.

Ramp-Freeway Terminals

The ramp-freeway junction is usually the critical element in ramp design and operation. Merging and diverging movements which occur at these terminals should take place at the speed of the freeway traffic stream and without disruption to that stream.

Because merging and diverging maneuvers occur in the freeway lane adjacent to the ramp, the amount and character of traffic in this lane is a critical concern. For the common case of a right-hand ramp, this is the lane next to the outside shoulder, which will be referred to herein as lane 1. Computational procedures, therefore, concentrate on estimatng of the volume in lane 1. In general, the volume in lane 1 of a freeway, a freeway, immediately upstream of a ramp is basically dependent on:

- the type of ramp (on-or off-ramp)
- traffic volume on the ramp (V_r)
- total freeway volume, immediately upstream of the ramp (V_f)
- the distance to adjacent upstream or downstream ramps (D_u, D_d)
- the volume on the adjacent upstream or downstream ramps (V_u, V_d)

The procedure presented herein allows for the analysis of isolated ramps, or ramps in association with one adjacent upstream or downstream ramp. Where a ramp has both an adjacent upstream and downstream ramp, it will generally be considered twice, first in conjunction with the adjacent upstream ramp, then with the downstream ramp. This procedure is discussed under "Computational Procedures for Ramp-Freeway Terminals" and illustrated under "Sample Problems."

Figure 4.1 illustrates the various ramp configurations for which direct procedures are available. Note that the case of an on-ramp followed by an off-ramp, with an auxiliary lane connecting them, is not considered a ramp configuration. These are considered to be ramp-weave sections which are analyzed using procedures presented in Chapter III. The procedures discussed herein are adapted from the 1965 Highway Capacity Manual, but are simplified and modified based upon the results of the Weaving Area Operations Study, NCHRP Project 3-15(4).

Because of the wide variety of ramp configurations, there will be cases that are not specifically covered by the procedures contained herein. In such cases, approximations may be made by utilizing a procedure most closely representing the case at hand. Such approximations will be discussed at greater length later in this chapter.

A. Critical Elements for Consideration

The computation of lane 1 volumes enables the evaluation of the following critical volumes, referred to as "checkpoint volumes":

- merge volume (V_m): merge volume equals the lane 1 volume plus the ramp volume which joins it at an on-ramp terminal, which is the entire ramp volume in the case of one-lane ramps, or the ramp volume in the lane adjacent to lane 1 in the case of two-lane ramps. A two-lane on-ramp may or may not be associated with a lane addition at the ramp.
- diverge volume (V_d): diverge volume is the volume in lane 1 which divides to a ramp volume and a volume continuing on the freeway at an off-ramp terminal. In cases of a two-lane ramp, there may be two such diverge volumes, one associated with each lane of the off-ramp. A two-lane off-ramp may or may not be associated with a lane drop at the ramp.
- weaving volume (V_w): in cases of an on-ramp followed by an off-ramp (without an auxiliary lane), functional weaving takes place between the ramps as on-ramp vehicles cross the path of off-ramp vehicles. This weaving volume is usually expressed as a uniform rate of weaving vehicles per 500 feet (153 m.)of weaving length (the distance between ramps).
- freeway volume (V_f): for each ramp terminal, the total freeway volume should be checked against the criteria given in Table 2.1. For on-ramps, the check is made at a point immediately after the merge; for off-ramps, immediately before the diverge.

Figure 4.2 illustrates the relationship between these critical volumes. It is these volumes which determine the level of operation of a particular ramp-freeway junction, and it is these volumes for which Level of Service standards are given.

B. Level of Service Criteria

Level of Service criteria for merge (V_m),diverge (V_d), and weaving (V_w) volumes at ramp-freeway terminals are given in Table 4.1. These criteria are modified from the 1965 Highway Capacity Manual Table 8.1 to reflect units of passenger cars per hour, uniform periods of flow (i.e. PHF = 1.00), and the freeway criteria of Table 2.1. Criteria for total freeway volume (V_f) are the same as those shown in Table 2.1, which is consulted directly for this checkpoint.

(a) Isolated on-ramp **(b) Isolated off-ramp**

(c) Adjacent on-ramps **(d) Adjacent off-ramps**

(d) On-ramp followed by off-ramp (no auxiliary lane) **(e) Off-ramp followed by on-ramp**

(f) Lane addition **(g) Lane drop**

(h) Major diverge **(i) Major merge**

FIGURE 4.1

RAMP CONFIGURATIONS COVERED BY PROCEDURES

TABLE 4.1

CHECKPOINT VOLUMES AT RAMP-FREEWAY TERMINALS
FOR UNIFORM FLOW RATES (PHF = 1.00)

Level of Service	Merge[a] Volume (pcph)	Diverge[b] Volume (pcph)	Weave[c] Volume per 500 ft.(153 m.) of Weaving Length (pcph)
A	≤ 750	≤ 800	500
B	751 - 1200	801 - 1300	501 - 700
D	1201 - 1550	1301 - 1650	701 - 1300
D	1551 - 1800	1651 - 1900	1301 - 1550
E[d]	1800 - 2000	1901 - 2000	1551 - 2000
F	widely variable		

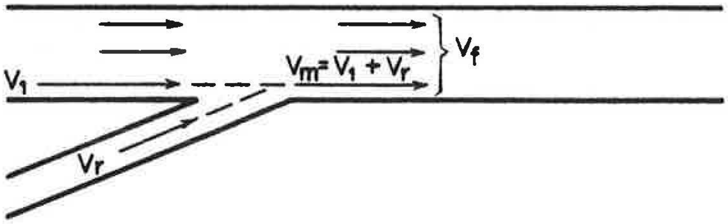
a. lane 1 volume plus ramp volume (for 1-lane ramps)

b. lane 1 volume immediately upstream of off-ramp

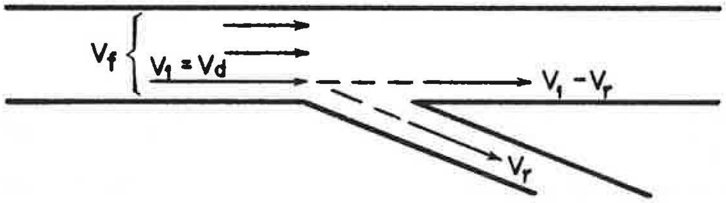
c. weaving vehicles between on-ramp, off-ramp pair per 500 ft. (153 m.) of length

d. capacity

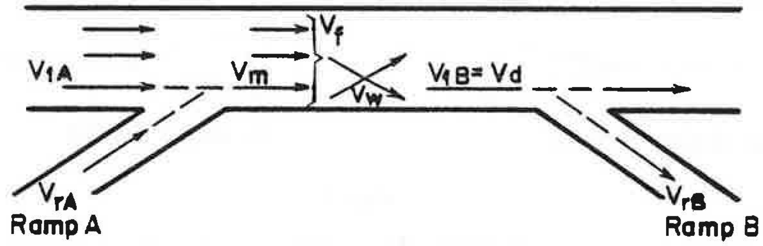
NOTE: for total freeway service volumes, see Table 2.1.



(a) Checkpoint volumes at an on-ramp



(b) Checkpoint volumes at an off-ramp



(c) Checkpoint volumes at an on-ramp followed by an off-ramp (no auxiliary lane)

FIGURE 4.2

CHECKPOINT VOLUMES FOR RAMP-FREEWAY TERMINALS

These criteria are intended to reflect volumes which can be accommodated while permitting the freeway as a whole to operate at the stated Level of Service in the vicinity of the ramp. Thus, for merge, diverge, and weaving volumes shown for Level of Service C, operation of the freeway as a whole is expected to be as described in Chapter 2 for basic freeway section Level of Service C, assuming V_f does not preclude this. Vehicles in lane 1 would be expected to experience some turbulence beyond that experienced on open sections, but average speeds and densities equivalent to those in Table 2.1 would, on the average, be maintained. The operation of merging, diverging, and weaving vehicles at each Level of Service is described in the paragraphs which follow.

Level of Service A represents unrestricted operation. Entering and exiting vehicles have little effect on other freeway flows. Merging is smoothly accomplished with little or no speed adjustments required to fill gaps; diverge movements encounter no significant turbulence.

At Level of Service B, entering vehicles may have to adjust their speed slightly to fill lane 1 gaps, but exiting vehicles still experience little difficulty. Vehicles not involved directly in merging and/or diverging maneuvers are not seriously affected.

Level of Service C is the limit of relatively stable flow, though small changes in volumes may make the situation unstable. Both lane 1 and entering vehicles must adjust speeds to make smooth merges, and under heavy ramp flows, minor queuing on on-ramps may occur. Some slowing may also occur at diverge points. The operation of all freeway vehicles is somewhat affected due to a general increase in turbulence, but overall speed and density is not seriously deteriorated.

At Level of Service D, smooth merging becomes difficult, and both entering and lane 1 vehicles must frequently change their speed to achieve acceptable merging operations. Slowing at diverge areas also becomes noticeable. On heavily used on-ramps, queues may become a disruptive factor.

Level of Service E represents capacity operation. Merging movements create significant turbulence, but continue without noticeable queuing. Diverging movements entail significant storing in the diverge area. Other freeway flow components attempt to avoid turbulence by moving towards the inside lanes.

At Level of Service F, all merging is on a stop-and-go basis, and ramp queues and freeway lane 1 backups are extensive. Much turbulence is created as vehicles attempt to change lanes to avoid merge and diverge areas. Considerable delay is encountered in the vicinity of the ramp terminal (and perhaps for quite a distance upstream on the freeway), and conditions vary widely from minute to minute, as unstable conditions create "waves" of alternately good and forced flow.

As is the case for other components, design should preferably be at Levels of Service A and B. For some urban cases, Levels of Service C may also be an acceptable design level. The instability of Levels of Service D and E, however, particularly considering the accuracy of volume forecasts, makes design at these levels questionable. An underestimate of 200 pcph in ramp merge or diverge volumes may make the difference between Level of Service D operation and Level of Service E.

II COMPUTATIONAL PROCEDURES FOR RAMP-FREEWAY JUNCTIONS

When the design of a ramp is being considered, the ramp location and general freeway design are already determined (at least for a particular computational trial). Thus, ramp and freeway demand volumes are also known, and are inputs rather than outputs of the computation. In analysis as well, volumes are known. Given known geometrics and volumes, computational procedures are established to determine the existing Levels of Service. Thus, design is accomplished by trial-and-error, assuming a configuration, applying forecasted volumes, and finding the resulting Levels of Service.

This design procedure is not difficult as the number of possible designs in any given instance is generally limited. As other major elements of the freeway would already have been considered before specific location, rather than the design of a ramp terminal more in question.

A step-by-step computational procedure for the analysis of ramp terminals is given below:

(1) Establish the geometrics and demand volumes for the case under study. This includes adjacent ramp configurations.

(2) Compute V_1, the lane 1 volume immediately upstream of the ramp in question using one of the nomographs included in the Appendix to this chapter. Where no appropriate nomograph exists, V_1 may be approximated using Table 4.3 and Figure 4.3. Table 4.2 provides an index to aid in the selection of the appropriate nomograph.

(3) Find the percentage of trucks in the lane 1 volume, using Figure 4.6.

(4) Convert all volumes in mixed vehicles (V_1, V_r, V_f) to pcph by dividing each by an appropriate adjustment factor for trucks, buses and recreational vehicles. These factors are computed using the procedures presented in Chapter 2.

(5) Compute all checkpoint volumes. In general, for single-lane ramps (Figure 4.2 illustrates):

V_m = merge volume = $V_1 + V_r$
V_d = diverge volume = V_1

V_w = weaving volume; expressed as a rate per 500 ft. (153 m.) of length

V_f = as a checkpoint is taken at a point just downstream of an on-ramp and just upstream of an off-ramp

(6) Convert all checkpoint volumes to peak flow rates by dividing each by the PHF.

(7) Compare all converted checkpoint volumes to the criteria in Table 4.1 to determine Level of Service.

Each of these steps is discussed in more detail and illustrated in the sections which follow.

STEP 1 - Establish Ramp Geometry and Volumes

In analysis, these two factors are known. In design, a geometric configuration will be assumed, and forecasted demand volumes are assigned to the freeway and ramp(s).

The establishment of a configuration includes the type, location of, and volumes on adjacent ramps, and is the basis for selection of a nomograph or approximation procedure for use in analysis. As the nomographs primarily deal with ramp pairs, an individual ramp with both an adjacent upstream and downstream ramp will often be considered twice, as part of a pair with each. For initial purposes of this step, "adjacent" should consider adjacent ramps within 6000 feet of the ramp in question. Individual nomographs include more detailed criteria for when an "adjacent" ramp may be considered as being isolated, and when it must be considered as a combination with adjacent ramps.

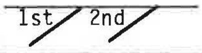
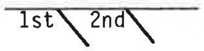
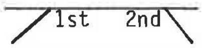
STEP 2 - Computation of Lane 1 Volume

The computation of V_1 is the critical step in any ramp analysis. These procedures provide 13 nomographs for the computation of V_1 under certain given configurations. In situations not covered specifically by these, or where situations cannot be reasonably approximated by these, approximations may be made using Table 4.3 and Figure 4.3. Table 4.2 provides an index to these procedures.

Each of the nomographs included in The Appendix to this chapter contains a complete set of instructions for use, and details the conditions under which use is acceptable. These should be carefully noted, particularly where an approximation is involved. Special instructions for such cases are included. The equation for each nomograph is also prominently displayed. Where greater precision is desired, the direct use of the equation is recom-

TABLE 4.2

INDEX TO NOMOGRAPHS AND PROCEDURES FOR ANALYSIS OF RAMP TERMINALS

Configuration	4-Lane Freeway (2 lanes ea.dir.)		6-Lane Freeway (3 lanes ea.dir.)		8-Lane Freeway (4 lanes ea.dir.)	
	1st Ramp	2nd Ramp	1st Ramp	2nd Ramp	1st Ramp	2nd Ramp
Isolated One-Lane On-Ramp 	Fig.A4.1	-	Fig.A4.6	-	Fig.A4.9	-
Isolated One-Lane Off-Ramp 	Fig.A4.2	-	Fig.A4.7	-	Approximate using Table 4.3 and Fig.4.3	-
Adjacent One-Lane on-Ramps  1st 2nd	Fig.A4.1	Fig.A4.5	Fig.A4.6	Fig.A4.8	Approximate using Table 4.3 and Fig.4.3	Approximate using Table 4.3 and Fig. 4.3
Adjacent One-Lane Off-Ramps  1st 2nd	see note 1	Fig.A4.2	see note 2	Fig.A4.7	Approximate using Table 4.3 and Fig.4.3	Approximate using Table 4.3 and Fig.4.3
On-Ramp Followed by Off-Ramp  1st 2nd	Fig.A4.1	Fig.A4.3	Fig.A4.6	Fig.A4.7	Fig.A4.10	Approximate using Table 4.3 and Fig.4.3
Off-Ramp Followed by On-Ramp  1st 2nd	Treat as Isolated			Fig.A4.6	Treat as Isolated Ramps	
Loop Ramps 	Fig.A4.4	Fig.A4.3	Fig.A4.6	Fig.A4.7	Fig.A4.10	Approximate using Table 4.3 and Fig.4.3
Two-Lane On-Ramps 	N.A.*	-	Fig.A4.11		N.A.*	-
Two-Lane Off-Ramps 	-	N.A.*	-	Fig.A4.12	-	N.A.*

mended, though in most cases, the precision provided by the nomographs is adequate.

Table 4.3 and Figure 4.3 are used only where nomographs are not available for the particular configuration being considred. Table 4.3 and Figure 4.3 were calibrated in California using data for primarily Level of Service D (as defined in the 1965 HCM). When used, they yield approximate results.

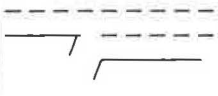
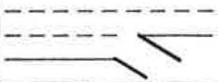
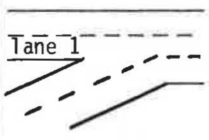
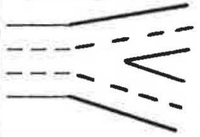
Lane 1 volumes are computed by obtaining the percentage of "through" vehicles in lane 1 from Table 4.3, the percentage of off-ramp vehicles in lane 1 from Figure 4.3I and/or the percentage of on-ramp in lane 1 from Figure 4.3II. Total lane 1 volume is the sum of these elements, where a "through" vehicle is defined as one not involved in any ramp movement within the configuration being considered.

The following brief examples illustrate the computation of V_1 using the nomographs, as well as by Table 4.3 and Figure 4.3.

a. Example a - Nomograph Procedure

Find the lane 1 volumes upstream of ramps A and B as illustrated below:

TABLE 4.2 (cont.)

Configuration	4-Lane Freeway (2 lanes ea.dir.)		6-Lane Freeway (3 lanes ea.dir.)		8-Lane Freeway (4 lanes ea. dir.)	
	1st Ramp	2nd Ramp	1st Ramp	2nd Ramp	1st Ramp	2nd Ramp
Addition of Lane at on-Ramp 	Merge criteria in Table 4.1 may be applied directly to the on-ramp volume as a checkpoint.					
Dropping a Lane at Off-Ramp 	Diverge criteria in Table 4.1 may be applied directly to the off-ramp volume as a checkpoint.					
Major Junctions 	Assume that Ramp Lane B carries an amount of traffic equal to the merge checkpoint volume in Table 4.1 for the assumed Level of Service. Ramp Lane A then carries the remaining ramp traffic. Compute Lane 1 volume using Figure A4.1 (4-lane freeway), A4.6 (6-lane freeway), or A4.9 (8-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) 	N.A.*	-	Fig.A4.13	-	N.A.*	-

* Not Available

TABLE 4.2 NOTES

1) Use Figure A4.2 to find V_1 in advance of the first ramp, but enter with a V_r which is equal to the total volume on both off-ramps. This technique is valid where the distance between ramps is less than 800 ft. (243.8 m.). Where the distance between ramps is between 800 and 4000 ft. (243.8 and 1219.2 m.), use Table 4.3 and Figure 4.3 to approximate the situation. Where the distance between ramps is greater than 4000 ft. (1219.2 m.), the ramps are treated as if they were independent (isolated).

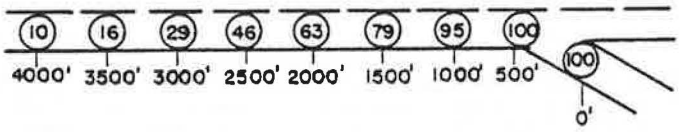
2) Use Figure A4.7 to find V_1 in advance of the first ramp, but enter with a V_r which is equal to the total volume on both off-ramps. This technique is valid where the distance between ramps is less than 800 ft. (243.8 m.). For other distances, see note 1 above.

TABLE 4.3[a]

APPROXIMATE PERCENTAGE OF THROUGH[b] TRAFFIC REMAINING IN LANE 1 IN THE VICINITY OF RAMP TERMINALS

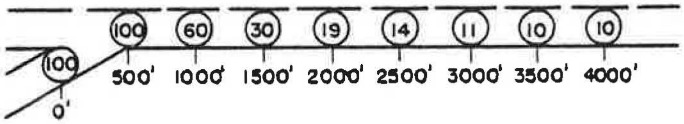
Total Through Volume, One Direction (vph)	Through Traffic Remaining in Lane 1		
	8-Lane Freeway	6-Lane Freeway	4-Lane Freeway
≥6500	10	--	--
6000-6499	10	--	--
5500-5999	10	--	--
5000-5499	9	--	--
4500-4999	9	18	--
4000-4499	8	14	--
3500-3999	8	10	--
3000-3499	8	6	40
2500-2999	8	6	35
2000-2499	8	6	30
1500-1999	8	6	25
≤1499	8	6	20

I.



Percentage of Off-Ramp Traffic Present in Lane 1 at Various Distances from Ramp

II.



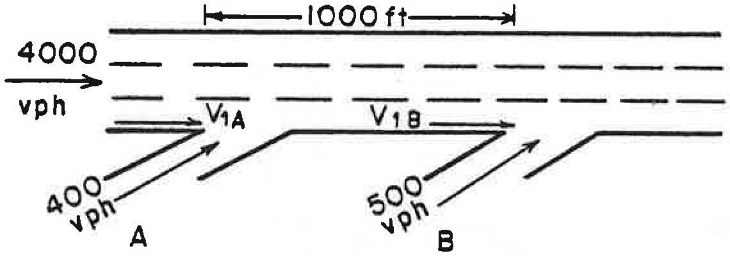
Percentage of On-Ramp Traffic Present in Lane 1 at Various Distances from Ramp

NOTE: If percentage found in this figure is less than the percentage of through volume in lane 1 (given in Table 4.3), use the percentage given for through volume (Table 4.3)

FIGURE 4.3

PERCENTAGE OF RAMP VEHICLES IN LANE 1

(adapted from Reference 1, Figure 8.23a)



From Table 4.2, Figure A4.6 is used to compute the lane 1 volume immediately upstream of ramp A(V_{1A}), while Figure A4.8 is used for ramp B(V_{1B}).

Note that the use of Figure A4.6 for ramp A is an approximation. Following instruction No. 3 under "Conditions for Use," a value of 5 will be used for 640 V_a/D_d, as no downstream off-ramp exists (an on-ramp exists here). Further, instruction 2 requires that V_u be taken as 50, as no upstream off-ramp exists. Thus, using the equation directly:

$$V_1 = -121 + 0.244V_f - 0.085\ V_u + 640\ V_d/D_d$$

where: $V_f = 4000$; $V_u = 50$; $640\ V_a/D_d = 5$

$$V_{1A} = -121 + 0.244(4000) - 0.085(50) + 5$$

$$V_{1A} = 856 \text{ vph}$$

Figure 4.4 illustrates the same solution using the nomograph, $V_{1A} = 860$ vph. The difference between computed and nomograph values is due to the scale precision of the nomograph. <-

Figure A4.8 may be applied directly for the solution of lane 1 volume immediately upstream of ramp B. Note that for ramp B, V_f is equal to 4000 vph plus the 400 vph entering at A, or 4400 vph. Using the equation:

$$V_1 = 574 + 0.228\ V_f - 0.194\ V_r - 0.714\ D_u + 0.274\ V_u$$

where: $V_f = 4400$, $V_r = 500$, $D_u = 1000$, $V_u = 400$

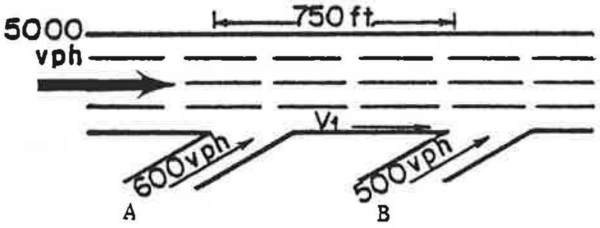
$$V_{1B} = 574 + 0.228(4400) - 0.194(500) - 0.714(1000) + 0.274(400)$$

$$V_{1B} = 876 \text{ vph}$$

Figure 4.5 illustrates the same solution using the nomograph, $V_{1B} = 870$ vph. The difference between computed and nomograph solutions is once again due to the scale precision of the nomograph.

b. Example b: Approximate Procedure

Find the lane 1 volume upstream of ramp B, as illustrated below:



In this problem, the lane 1 volume for the second ramp is the desired solution. Table 4.2 indicates that the approximate procedure of Table 4.3 and Figure 4.3 should be used in this case. (See "Adjacent On-Ramps-2nd Ramp-8-lane freeway".)

V_1 for this problem will consist of two components: the "through" vehicles remaining in lane 1 and the upstream on-ramp vehicles remaining in lane 1.

Table 4.3 is used to compute the number of through vehicles in lane 1. The number of vehicles NOT involved in any ramp maneuvers (through vehicles) is 5000. From Table 4.3, 9% of these remain in lane 1. Therefore:

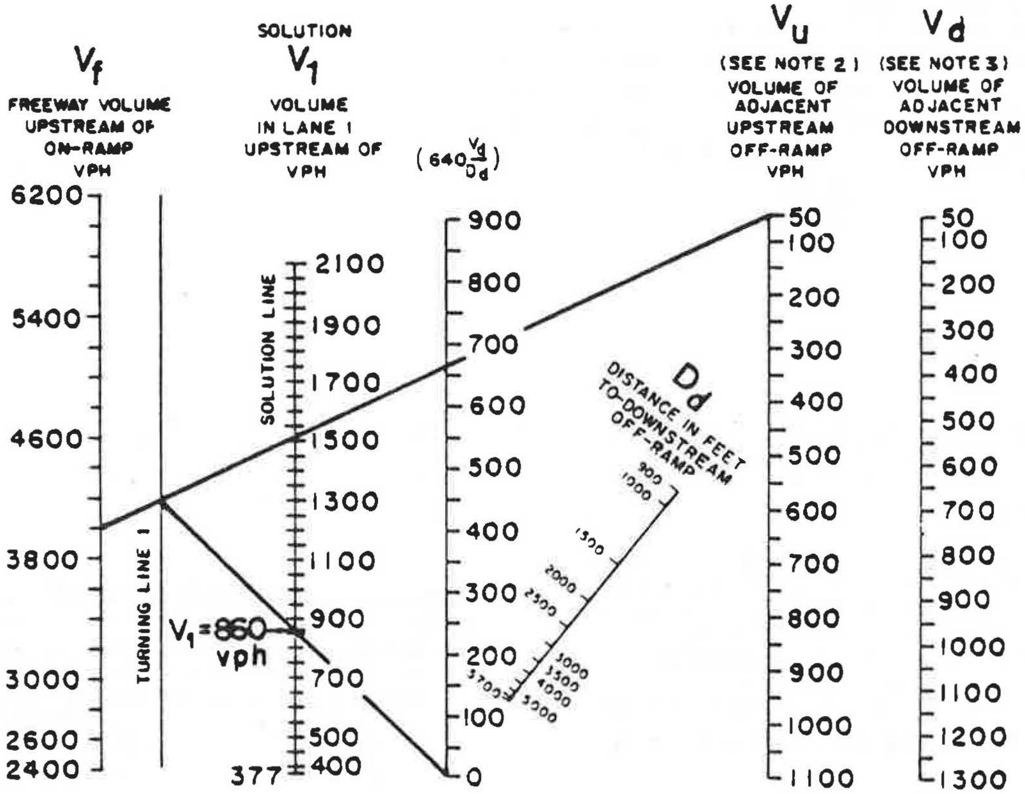
$$V_1(\text{through}) = 0.09 \times 5000 = 450 \text{ vph}$$

Figure 4.3 is used to compute the amount of adjacent ramp traffic in lane 1 at the point in question. The point in question is located 750 ft. downstream of the adjacent on-ramp.

From Figure 4.3, case II, and interpolating between 500 ft. and 1000 ft., 80% of on-ramp traffic is still in lane 1 at a point 750 ft. downstream. Therefore:

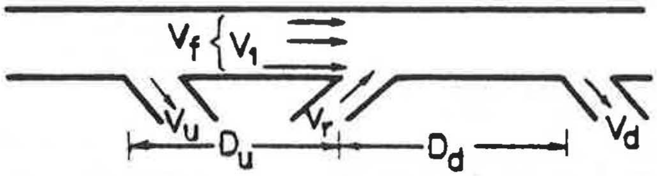
$$V_1(\text{on-ramp}) = 0.80 \times 600 = 480 \text{ vph}$$

and: $V_1 = 450 + 480 = 930$ vph



EQUATION: $V_1 = -121 + 0.244V_f - 0.085V_u + 640\,{}^{V_d}/_{D_d}$

DIAGRAM:



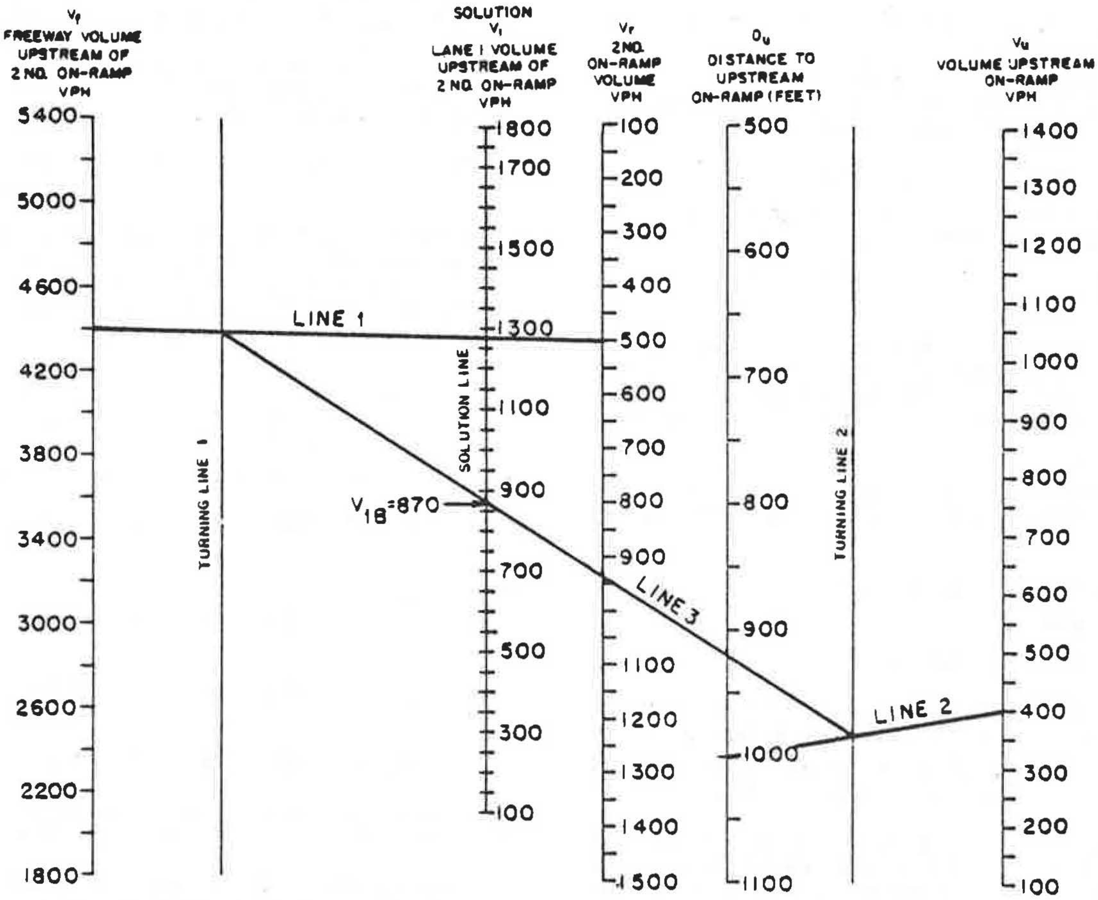
CONDITIONS FOR USE:

1) Single lane on-ramps on 6-lane freeways with or without upstream and/or downstream off-ramps, with or without acceleration lane.
2) If there is no upstream off-ramp within 2600 ft., use V_u=50.
3) If there is no downstream off-ramp within 5,700 ft., and V_f < 5000 vph, use 640 V_d/D_d=5, and skip step 2 below.
4) Normal range of use: V_f=2400 to 6200vph; V_u=50 to 1100vph; V_d=50-1300vph
V_r=100 to 1700vph; D_d=900 to 5700 ft.; D_u=900-2600 ft.

STEPS IN SOLUTION:

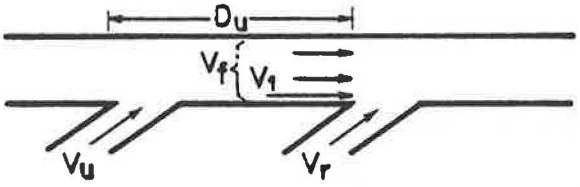
1) Draw a line from V_f value to V_u value, intersecting turning line 1.
2) Draw a line from V_d value to D_d value, intersecting 640 V_d/D_d line.
3) Draw a line from the step 1 intersection with turning line 1 to the 640 V_d/D_d value of step 2; read solution at intersection with V_1 line.

FIGURE 4.4
AN ILLUSTRATION OF A NOMOGRAPH SOLUTION FOR
V_{1A} OF EXAMPLE 1 USING FIGURE A4.6



EQUATION: $V_1 = 574 + 0.228V_f - 0.194V_r - 0.714\,D_u + 0.274\,V_u$

DIAGRAM:



CONDITIONS FOR USE:

1) Single lane on-ramps on 6-lane freeways with adjacent upstream on-ramps, with or without acceleration lanes.

2) Normal range of use: V_f = 1800 to 5400 vph; V_r = 100 to 1500 vph
V_u = 100 to 1400 vph; D_u = 500 to 1000 ft.

STEPS IN SOLUTION:

1) Draw a line from V_f value to V_r value, intersecting turning line 1.

2) Draw a line from V_u value to D_u value, intersecting turning line 2.

3) Draw a line from intersesction of step 1 to that of step 2; read solution on V_1 line.

FIGURE 4.5
AN ILLUSTRATION OF A NOMOGRAPH SOLUTION FOR V_{18} IN EXAMPLE 1 USING FIGURE A4.8

STEP 3 - Convert All Volumes to PCPH

Volumes V_1, V_r, and the total freeway volume should now be converted to pcph units. Volumes in mixed vehicles per hour may be converted to pcph by dividing by the appropriate commercial/recreational vehicle correction factor, from Table 2.8.

Before V_1 may be appropriately converted, it is necessary to find the percentage of trucks in V_1. Truck distribution is not uniform across all freeway lanes, as trucks tend to concentrate in right-hand lanes.

Figure 4.6 illustrates the average percentage of total trucks which will be in lane 1 under various conditions. This percentage may then be converted to one which represents the proportion of trucks in V_1.

Consider Ramp A of Example a in the previous section. For a 6-lane freeway, the following was given:

V_f = 4000 vph (before merge)
V_r = 400 vph
V_1 = 856 vph (computed)

If the freeway volume includes 8% trucks, and the ramp volume 10% trucks, what percentage of V_1 is trucks, and what will the truck percentage for the freeway be at a point just beyond the merge?

Entering Figure 4.6 with a volume of 4000 vph (read on scale as 40), rising vertically to the "6-lane freeways" curve and projecting horizontally, it is found that 52% of all trucks will be in lane 1. Then:

Number of trucks on freeway = 0.08 x 4000 = 320
Number of trucks in lane 1 = 0.52 x 320 = 166
% trucks, lane 1 = (166/856) x 100 = 19.4% say 19%

Number of trucks on freeway (after merge)
=320+(400x0.10)=360
Total freeway volume (after merge) = 4000 + 400
= 4400
% trucks, freeway(after merge) = (360/4400)x100
= 8.2% say 8%

Once these percentages are established, each volume may be converted to pcph by dividing by the appropriate truck factor, extracted from the appropriate tables in Chapter 2. In this case, assuming level terrain for the segment in question:

item	volume (vph)	% trucks	E_T^*	Q^{**}	volume (pcph)
freeway vol. (before merge)	4000	8	2	0.93	4000/0.93 =4,301
freeway vol. (after merge)	4400	8	2	0.93	4400/0.93 =4,731
ramp volume	400	10	2	0.91	400/0.91 = 440
lane 1 volume	856	19	2	0.84	856/0.84 =1,019

* Table 2.4 ** Table 2.8

Having converted all component volumes to pcph, it is now possible to compute checkpoint volumes directly.

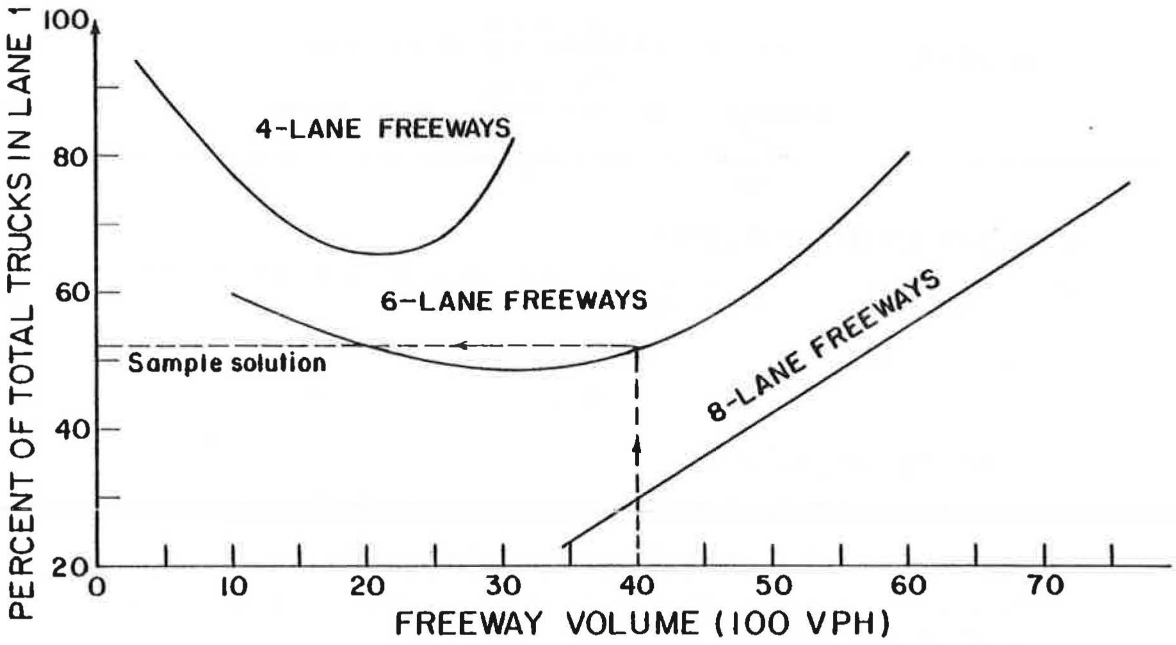


FIGURE 4.6 - TRUCK PRESENCE IN LANE 1

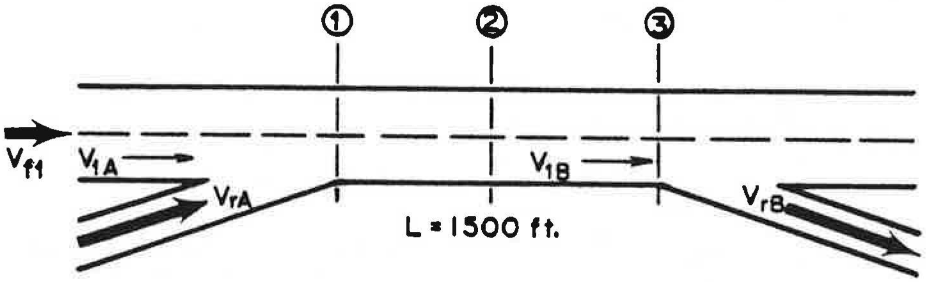
(Source: adapted from Refs. 1 and 4)

STEP 4 - Compute Checkpoint Volumes

For each ramp analysis, there are up to four checkpoint volumes for each ramp, or pair of ramps:

- merge volume (V_m) - in any merge situation, two lanes will join to form a single lane. The merge volume is the sum of the volumes in the two lanes which join. In the standard case of a one-lane, right-side on-ramp, the merge volume is the sum of the ramp volume (V_r) and the volume in lane 1 immediately prior to the merge (V_1).
- diverge volume (V_d) -the diverge volume is the total volume in a freeway lane immediately upstream of a point at which that lane divides into two separate lanes. For the case of a one-lane, right-side off-ramp, this is equal to the volume in lane 1 (V_1).
- total freeway volume (V_f) - total freeway volume must be checked at critical points. It is always checked immediately upstream of an off-ramp and immediately downstream of an on-ramp.
- weaving volume (V_w) - in cases where an on-ramp is followed by an off-ramp (with no continuous auxiliary lane connecting them), the amount of weaving volume per 500 (153 m.)feet of length is used as a criteria. Thus, for 800 weaving vehicles in a 1000 ft. (305 m.) length, V_w would equal 800 (500/1000) or 400.

Figure 4.7 illustrates the computation of checkpoint volumes. Note that for the problem illustrated, only one freeway volume check is made for both ramps, at the point of maximum freeway volume which occurs anywhere between the two ramps. Note further that the computation of weaving volume



CHECKPOINTS

(1) Merge (immediately after on-ramp) at Point ①:

$$V_m = V_{1A} + V_{rA}$$

(2) Diverge (immediately before off-ramp) at Point ②:

$$V_d = V_{1B}$$

(3) Freeway Checkpoint Volume (upstream of off-ramp between the ramps) at Point ③:

$$V_f = V_{f1} + V_{rA}$$

(4) Weaving Volume (assuming that none of the on-ramp vehicles also use the off-ramp), per 500 ft. (153 m.) of length:

$$V_w = (V_{rA} + V_{rB}) \left(\frac{500}{1500}\right)$$

FIGURE 4.7

COMPUTATION OF CHECKPOINT VOLUMES FOR A CASE OF AN ON-RAMP FOLLOWED BY AN OFF-RAMP per 500 feet (153 m.) of weaving length requires the assumption that none of the on-ramp vehicles exit by the off-ramp immediately downstream. In some cases, the actual component flows may be known, and no assumption would be necessary.

STEP 5 - Convert All Checkpoint Volumes to Peak Flow Rates

Before comparing checkpoint volumes to the criteria given in Table 4.1, they must be adjusted to reflect uniform peak flow rates rather than full-hour volumes. This is done by dividing each checkpoint volume by the PHF. Off-peak periods may be checked by using any period in which flow rates are uniform.

STEP 6 - Find the Level of Service

The Level of Service for a given analysis is found by comparing the checkpoint volumes (expressed as peak flow rates) for merging, diverging, and weaving with the criteria given in Table 4.1. Total freeway volume is checked versus the criteria given in Table 2.1.

In many cases, the various operational elements (merge, diverge, weaving, total volume) will not be in balance, i.e., the Level of Service for the merge may be different than for the diverge, etc. In such cases, the worst resultant Level of Service is assumed to govern the section in question. Further, the analysis will clearly indicate which element or elements of operation are controlling the situation. Thus, if a merge volume is creating a difficulty, efforts at improvement should be keyed to the design and operation of the merge point, rather than other elements.

SPECIAL CASE - 10 Lane Freeways (5-lanes in each direction)

Ten-lane (five lanes in one direction) freeway sections in urban areas is becoming more frequent. Often, these segments involve ramp junctions which need to be designed or analyzed. While no specific procedures for the consideration of such ramps exist, Ref. 5 contains an approximate procedure which is useful.

Table 4.4 shows approximate standards for considering 10-lane sections as equivalent 8-lane segments by computing an equivalent freeway volume which when used in conjunction with procedures for 8-lane freeways, results in the same lane 1 volumes as exist on 10-lane freeways.

Thus, in the case of an off-ramp on a 10-lane freeway carrying a volume of 6400 vph, procedures for a similar configuration on an 8-lane freeway would be used, but with a freeway volume of:

6400 x 0.85 = 5440 vph

The lane 1 volume computed in this way is an approximation of the actual lane 1 volume occurring on the 10-lane freeway.

When considering such segments, other special considerations include:

(1) trucks in lane 1 - truck presence may be computed using the 8-lane freeway curve in Figure 4.6. This is a "worst case" assumption, as little information exists concerning truck distribution on 10-lane freeways.

(2) freeway volume - the freeway volume check may be made directly in Table 2.1, and does not require an approximation. The 10-lane freeway may be checked directly using the actual freeway volume.

SPECIAL CASE - Left-Hand Ramps

While not normally recommended, left-hand ramps do exist on some freeways, and often occur on collector-distributor roadways.

Reference 5 again contains an approximate procedure for treating such ramps, which involves two modifications to normal procedures:

(1) lane i volumes The freeway lane of interest is not lane 1 in the case of left-hand ramps, but lane i, the left-most lane of the freeway adjacent to the ramp. To compute lane i volumes, which are generally higher than corresponding lane 1 volumes, the lane 1 volume is computed as normally done. Then:

lane i volume = 1.25 x lane 1 volume (on-ramps)
lane i volume = 1.10 x lane 1 volume (off-ramps)

(2) truck presence The percentage of trucks present in lane i is approximated as follows.

For 4-lane freeways, the percentage of through trucks is 25% of total through trucks on the freeway. In the case of on-ramps, no additional trucks would be in lane i, but in the case of off-ramps, ALL exiting trucks would be in lane i.

TABLE 4.4

CONVERSION FACTORS FOR CONSIDERATION OF RAMPS ON TEN-LANE FREEWAYS

Ramp Type	Freeway Volume (5-lane segment)	Conversion Factor
ENTRANCE	all volumes	0.78
EXIT	$\leq$ 4000 vph	1.00
	4001 - 5500 vph	0.90
	5501 - 7000 vph	0.85
	$\geq$ 7000 vph	0.80

For 6-or-more-lane freeways, no through trucks are assumed to be in lane i. As with 4-lane freeways, no on-ramp trucks would be in lane i prior to the ramp junction, but ALL off-ramp trucks would be in lane i.

EFFECTS OF RAMP GEOMETRY - A Qualitative Treatment

There is no question that the specific geometrics of a ramp terminal can affect its operation. Such characteristics as grade, the differential between freeway and ramp grade, angle of convergence, length of acceleration and deceleration lanes and others influence the overall operation and, perhaps, the capacity of a ramp.

Unfortunately, there is little in the way of quantitative information in this regard. A study by Wattleworth, et al (7) resulted in observations of the effects of geometric parameters on speed distributions at on-ramps, but these are difficult to relate to capacity and service volume.

In a textbook by Drew (8) he develops a methodology for on-ramp merge "capacity" based upon gap-acceptance models, calibrated with a small data base. The technique allows for the effect of varying angle of convergence and length of auxiliary lane. While the Drew technique cannot be directly integrated into the procedures presented herein - his use of the term "capacity" is not synonomous with the use herein, or in the 1965 HCM - it can be used to illustrate the potential effect of angle of convergence and acceleration lane length on capacity and service volume.

Table 4.5 illustrates the percentage of gaps accepted by merging vehicles for various combinations of angle of convergence and length of acceleration lane. The table is based upon an "ideal" case of 2° convergence angle and a 1200-foot (365.8 m.) acceleration lane. It is also based upon a lane 1 volume of 1000 vph, although similar percentages could be shown for other V_1 values.

USERS OF THIS DOCUMENT SHOULD BE CAUTIONED THAT THIS TABLE IS ONLY ILLUSTRATIVE. IT CANNOT, FOR EXAMPLE, BE USED TO MODIFY THE RESULTS OF PROCEDURES PRESENTED HEREIN, AS THOSE PROCEDURES DO NOT REPRESENT THE "IDEAL" CASE ASSUMED BY DREW. IN FACT, THE DATA BASE FOR THE PROCEDURES INCLUDES NUMEROUS "SUB-IDEAL" CASES BY THE CRITERIA OF TABLE 4.5.

TABLE 4.5 IS MEANT, THEREFORE, TO PROVIDE A GENERAL INSIGHT INTO THE POTENTIAL EFFECTS OF THESE TWO IMPORTANT DESIGN PARAMETERS, RATHER THAN TO SERVE AS A SPECIFIC COMPUTATIONAL DEVICE.

Designers should be careful to provide for adequate geometric design, as defined in AASHTO policies (2,3), and analysts should be aware that poorly designed ramps may not operate with the conditions specified herein. Nevertheless, the field studies upon which the 1965 HCM was based showed some remarkably high merge volumes at such poorly-designed ramp junctions, where almost all traffic consisted of "repeat" drivers familiar with the site. Many such cases were included in the calibration of the procedures presented herein.

III. COMPUTATIONAL PROCEDURES FOR RAMPS PROPER

There is very little information concerning operational characteristics on ramps proper. Because most problems occur at ramp terminals, most quantitative studies have been concerned with terminal operations, not the ramp itself.

Some basic design standards exist in AASHTO policies (3) but these are not related to specific operational characteristics. J. Leisch (5) has adapted this material to provide a broader set of criteria, but again, they are not related to specific operational characteristics.

Ramps differ considerably from the freeway mainline in that:

1) they are roadways of limited length and width (often one lane)

2) the design speed of the ramp is frequently lower than that of the roadways it connects

3) on single-lane ramps, where passing is not possible, the adverse effect of trucks and other slow moving vehicles is more pronounced than on a multi-lane roadway

4) acceleration and deceleration often takes place on the ramp itself

5) at ramp-street system interfaces, queuing may develop on the ramp.

Because of these distinct characteristics, it is difficult to adjust basic freeway criteria to approximate criteria for ramps. Reference 5 gives instructions for estimating the approximate capacity of ramps. Service volumes for other Levels of Service are not as easily found, nor are there clear definitions of what type of operation is associated with each Level of Service. Table 4.6 gives approximate service volumes for ramps. Capacity estimates were generated from Reference 5. For consis-

TABLE 4.5

THE EFFECT OF RAMP GEOMETRICS ON GAPS ACCEPTED BY MERGING VEHICLES*
(% of ideal case)

angle of convergence	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

* for V_1 = 1000 vph

TABLE 4.6

APPROXIMATE SERVICE VOLUMES FOR SINGLE LANE* RAMPS
(PHF = 1.00; PCPH)

Level of Service	RAMP DESIGN SPEED mph (kph)				
	≤20(32)	20-30	30-40	40-50	≥ 50
A	**	**	**	**	700
B	**	**	**	1000	1050
C	**	**	1125	1250	1300
D	**	1025	1200	1325	1500
E	1250	1450	1600	1650	1700
F	widely variable				

* for 2-lane ramps, multiply above values by 1.7 for ≤ 20 mph
1.8 for 20-30 mph
1.9 for 30-40, 40-50 mph
2.0 for ≥ 50 mph

** Level of Service not achievable due to restricted design speed.

tency, other Levels of Service were specified at v/c ratios similar to those for the same level on basic freeway segments. Extant data does not permit each level to be precisely described in terms of operating characteristics.

These values may be adjusted for non-passenger vehicle presence and lane width-lateral clearance restrictions using the appropriate factors from Chapter 2. Their use in this context, it should be remembered, is approximate.

It should be noted further that Table 4.6 refers only to the ramp proper. Even though up to 1700 pcph may be accommodated in a single-lane ramp, this does not guarantee that they can be accommodated in a single-lane ramp-freeway terminal. As a general rule-of-thumb, where volumes exceed 1500 pcph, a two-lane ramp and a two-lane ramp freeway terminal are needed.

Further, even where a one-lane ramp and terminal is sufficient from the capacity point of view, a two-lane ramp is generally provided if:

- the ramp is longer than 1000 feet, to provide opportunities to pass slower moving vehicles.
- queues are expected to form on the ramp from a controlled ramp-street junction, to provide additional storage.
- ramp is located on a steep grade or has minimal geometrics.

If a two-lane ramp is provided for any of the above reasons, it is usually tapered to a single-lane prior to the ramp-freeway junction.

Two lane loop ramps are normally not be provided due to the restrictive geometry of a loop. In such cases, flush shoulders are generally provided with sufficient width to allow stalled vehicles to be passed. In cases where two-lane loops are provided, lanes must be wider than 12 feet.

The guidelines included here are most useful in design, where alternate ramp configurations may be developed for detailed analysis using the ramp-freeway terminal procedures. In analysis, total ramp volumes may be quickly checked to insure that adequate capacity is provided. Rarely, however, will the ramp proper itself be a controlling factor in either design or analysis.

IV. RAMP-STREET SYSTEM INTERFACE

These procedures do not treat the subject of ramp-street system interfaces. The 1965 HCM contains detailed procedures for the analysis of signalized at-grade intersection. A procedure for unsignalized intersections is included as an interim capacity material elsewhere in this circular.

Where the ramp-street interface is a high-type design the procedures for ramp-freeway junctions may be used to approximate conditions.

V. RAMP METERING

Ramp metering has been used as an effective method of improving freeway operations at a number of on-ramp locations, and is now a generally accepted practice. It has the following advantages related to facility capacity:

- it allows the full capacity of downstream sections to be effectively utilized by avoiding upstream bottle-necks which prevent demand from reaching capacity levels;
- it sometimes permits a desired Level of Service to be attained and maintained on the facility.

In addition, it has specific operational advantages at particular problem sites. For instance, it has been used successfully to enhance safety in cases of poor sight distance or short ramps. It has also been used to disperse platooned arrivals from signals at local streets.

Reference (9) is an important treatment of "Urban Freeway Surveillance and Control: The State of The Art," and addresses both ramp metering and the system use of ramp controls. Within the context of the present document, attention will focus on basic concepts and their relation to unmetered ramps.

A. Types of Ramp Metering

There are several levels of ramp-metering:

- fixed time metering
- real time metering
- gap-acceptance metering
- greenband metering

Fixed time metering uses a pre-selected ramp signal timing rate, usually employing pretimed controllers. This rate may vary by time of day. The signal normally rests in the red indication. The green is initiated by an on-ramp vehicle crossing a detector. Thus, an approaching vehicle is not confused by changing signal indications on an empty ramp.

Real-time metering uses a mainline sensor to estimate the current demand, from which the permissible ramp flow is computed using a predetermined relationship. The ramp signal entry rate is then adjusted in cognizance of this level.

Gap-acceptance metering is intended to match entering vehicles to specific gaps in the lane adjacent to the ramp, to enhance the probability that they will enter the freeway stream efficiently and to lessen potential conflicts. Sensors in lane 1 are used to detect gaps which are then projected forward to the merge area.

Greenband metering is an additional level of sophistication which adds a series of closely spaced signals on the ramp which display a green progression leading the ramp vehicle to the merge in a more precise fashion. Although specific applications may find special use for these versions, most installations are of the simpler types described in previous paragraphs. Applications of this type of metering have not been successful to date.

The basic set-up of a ramp-control installation is illustrated in Figure 4.8.

B. Implications on Capacity and Service Volume

The basic purpose of ramp metering is to assure that free flow is maintained in freeway lanes, without breakdown into congested flow with its shock waves, stop-and-go opration, and resultant loss of traffic-carrying capability below inherent capacities.

The methods and procedures presented in this chapter for the computation of ramp-freeway junction service levels are based upon data collected at uncontrolled ramps. At the present time, no detailed data exists from which to establish shifts in lane usage, truck presence, or capacity due to the presence of a metered ramp. In general, it is reasonable to consider ramp metering as a way of assuring that ramp flows do not exceed certain anticipated or planned levels, and to use the methods presented herein to determine the lane 1 volume and the Level of Service for a given ramp volume, V_r.

In uncontrolled situations, V_r prevails and determines the lane 1 volume by its presence and influence on entering. Ramp metering merely controls the maximum value of V_r.

In operations, demand-responsive control sometimes alters the usual pattern of V_r determining V_1: the control relationship reacts to the upstream Lane 1 volume and sets the metering rate (V_r maximum) based upon a maximum merge volume ($V_1 + V_r$). Thus, the nature of the ramp control modifies the usual sequence of cause and effect.

Because V_r may be controlled through the setting of maximum metering rates, it is often useful to use the procedures herein to compute V_r as the dependent variable. By so doing, a maximum V_r can be set which allows the facility to operate at a selected Level of Service. This is a trial-and-error process, in that V_r must be assumed to compute a V_1 value. To compute a V_r maximum for a given Level of Service:

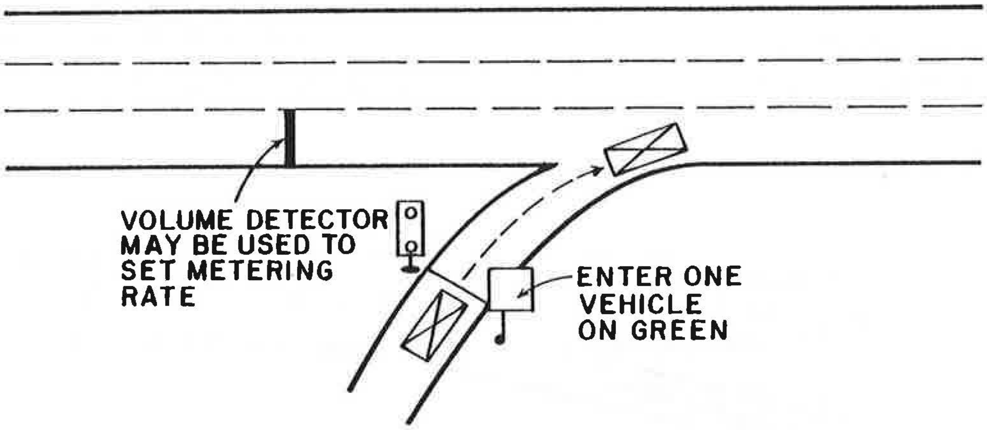


FIGURE 4.8

A TYPICAL RAMP METERING INSTALLATION

(1) Find the merge SV (V_m) for the Level of Service of interest (Table 4.1)

(2) Assume a value for V_r

(3) Compute V_1 using the procedures described in this chapter

(4) Compute $V_r = V_m - V_1$

(5) Continue computations until the V_r assumed in (1) matches that computed in (4).

Of course, all values must be converted to pcph and peak flow rates as derscribed elsewhere in this chapter.

In cases of severe platooning of arrivals at an entrance ramp, it is quite likely that ramp control will induce a better mainline distribution of freeway vehicles. This does not mean that it is necessarily different than that of a "normal" uncontrolled ramp in the procedures of this chapter. Indeed, it is reasonable to expect that the uncontrolled ramp with heavy platoon arrivals caused an abnormal lane distribution.

Metering rates must be kept reasonable if driver observance is to be reliably maintained. In this regard, a metering rate with a red interval of more than 15 seconds will invite widespread violations.

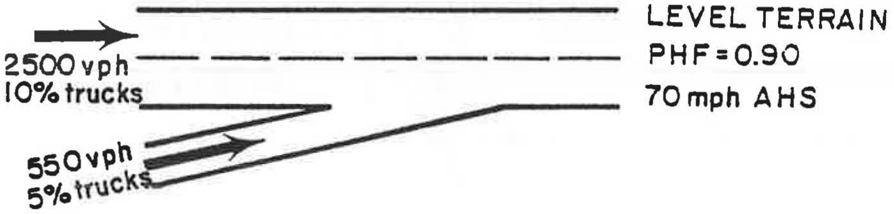
C. System Coordination of Controlled Ramps

An overall facility can be regulated by judicious use of ramp controls, and successful projects exist in Minneapolis (the only actual real-time system in operation), Los Angeles, Dallas, and other locations. The overall plan is established by use of the procedures contained in this document, but the detailed computations and methodologies fall within the general topic of freeway surveillance and control more than within the topic of capacity and Level of Service methods. Interested persons should refer to Reference 9. The topic of evaluating overall Level of Service is addressed herein in Chapter 5.

VI. SAMPLE PROBLEMS

Problem 1: Isolated On-Ramp

Consider the following on-ramp:



What Level of Service would be expected to prevail?

Solution: Using the index provided in Table 4.2, Figure A4.1 is chosen as the appropriate nomograph for this case.

Thus, V_1, immediately upstream of the ramp, may be computed as:

$$V_1 = 136 + 0.345\ Vf - 0.115\ V_r$$

$$V_1 = 136 + 0.345\ (2500) - 0.115\ (550)$$

$$V_1 = 935 \text{ vph}$$

or found from the nomograph as approximately 930 vph.

From Figure 4.6, about 67% of all trucks on the freeway will be in lane 1 immediately upstream of the ramp. Thus,

Total trucks on freeway = 2500 x 0.10 = 250
trucks in lane 1 = 250 x 0.67 = 168
% trucks in lane 1 = (168/935) x 100 = 18%

Now, V_1, V_r, and V_f may be converted to pcph. Values of E_T are taken from Table 2.4 of Chapter 2, and values of Q from Table 2.8.

item	volume (vph)	E_T	% trucks	Q	volume (pcph) = $\frac{\text{volume(vph)}}{Q}$
V_1	935	2	18	0.85	1100
V_r	550	2	5	0.95	579
V_f	2500	2	10	0.91	2747

Checkpoint volumes may now be computed:

$$V_m = V_r + V_1 = 579 + 1100 = 1679 \text{ pcph}$$

$$V_f(\text{after merge}) = V_f(\text{before merge}) + V_r$$

$$= 2747+579 = 3326 \text{ pcph}$$

These are now expanded to peak flow rates by dividing by the peak hour factor. The Level of Service for the merge is found by comparing V_m to Table 4.1 criteria; for the freeway volume, it is found by comparing to Table 2.1.

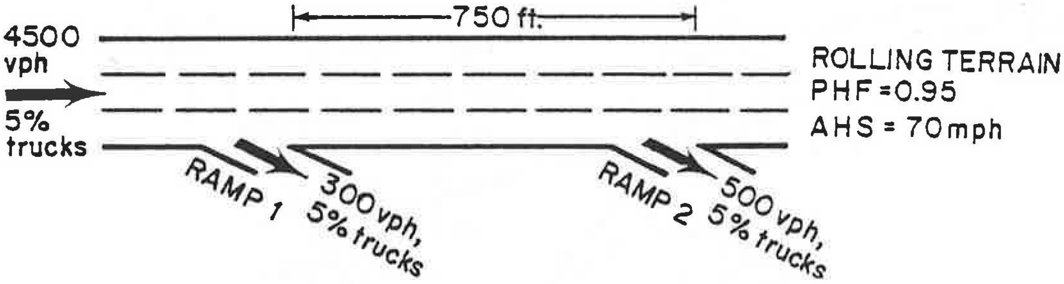
V_m = 1679/0.90 = 1866 pcph (Level E, Table 4.1)

V_f = 3326/0.90 = 3696 pcph (Level D, Table 2.1)

In this case, the merge area is the controlling feature (an undesirable condition), and Level of Service 5 prevails.

Problem 2 - Consecutive Off-Ramps

Consider the following ramp configuration:



At what Level of Service would the two off-ramps be expected to operate?

Solution: As indicated in Table 4.2, Note 2 must be consulted when analyzing the first ramp. Note 2 specifies the use of Figure A4.7 for this ramp, but instructs that V_r should be taken to be equal to the total off-ramp volume on both ramps. Figure A4.7 is also used for the second ramp.

RAMP 1: As there is no upstream on-ramp, the value "215 V_u/D_u" will be taken as 2, as directed by item 2 under "Conditions for Use" on Figure A4.7. As noted above, V_r will be taken as 300+500=800 for consideration of the first ramp. Then:

$$V_1 = 94 + 0.231\ V_f + 0.473\ V_r + 215\ V_u/D_u$$

$$V_1 = 94 + 0.231\ (4500) + 0.473\ (800) + 2$$

$$V_1 = 1514 \text{ vph}$$

RAMP 2: For ramp 2, V_f equals 4500 - 300 or 4200 vph. Further, 215 V_u/D_u will still be set at 2.

$$V_1 = 94 + 0.231\ (4200) + 0.473\ (500) + 2$$

$$V_1 = 1303 \text{ vph}$$

Figure 4.9 illustrates the nomograph solutions for both of these values. (V_1 = 1500 for Ramp 1; 1300 for Ramp 2)

The percentage of trucks in the respective lane 1 volumes should now be computed:

RAMP 1: % total trucks in lane 1 = 56% (Figure 4.6)
total trucks on freeway = 4500 x 0.05 = 225
trucks in lane 1 = 225 x 0.56 = 126
% trucks in lane 1 = (126/1514) x 100 = 8.3% say 8%

RAMP 2: % total trucks in lane 1 = 53% (Figure 4.6)
total trucks in lane 1 = 4200 x 0.05 = 210
trucks in lane 1 = 210 x 0.53 = 111
% trucks in lane 1 = (111/1301) x 100 = 8.5% say 9%

All volumes are now converted to pcph in the table below:

item	volume (vph)	E_T^*	% Trucks	Q**	volume (pcph) = $\frac{\text{volume(vph)}}{Q}$
V_f	4500	4	5	0.87	5172
$V_r(1)$	300	4	5	0.87	345
$V_r(2)$	500	4	5	0.87	575
$V_1(1)$	1514	4	8	0.81	1869
$V_1(2)$	1301	4	9	0.79	1647

* Table 2.4
** Table 2.8

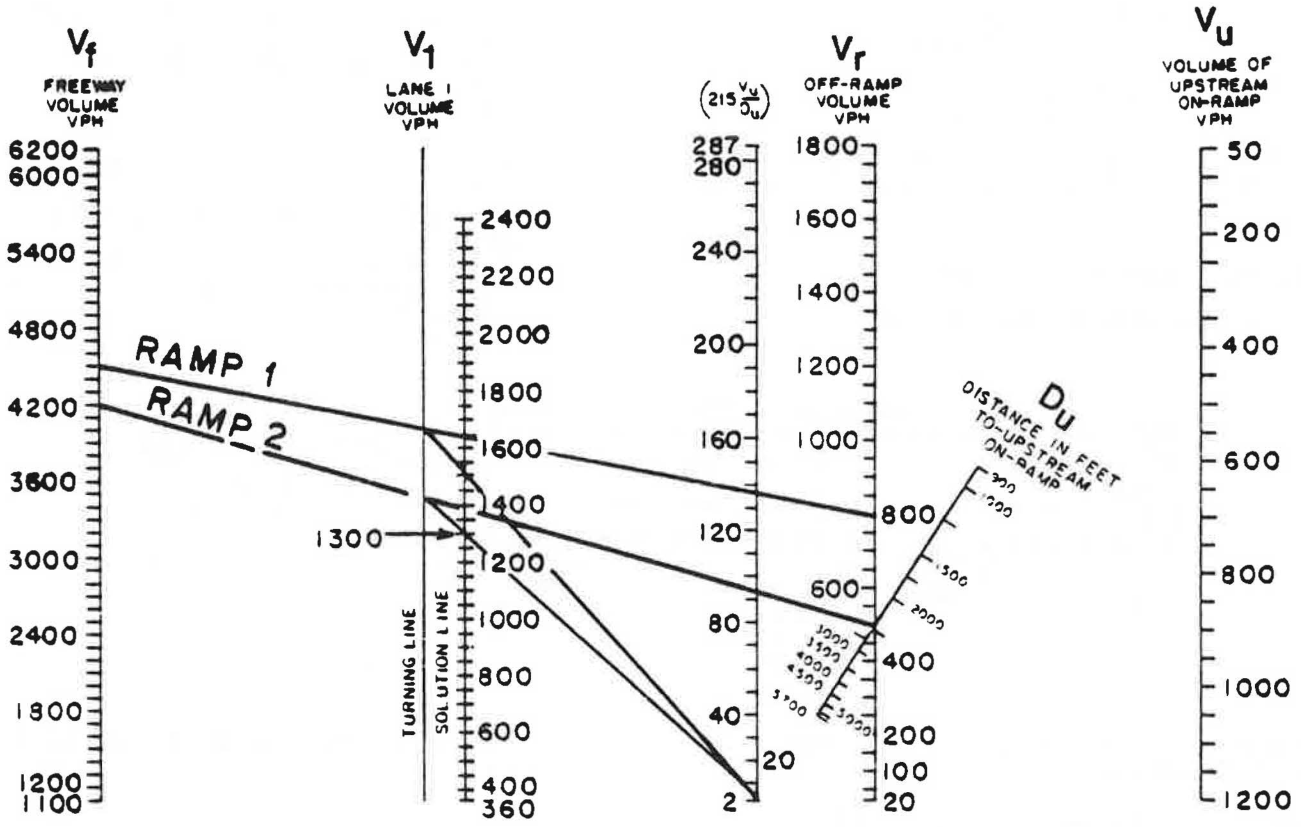
Three checkpoint volumes are of interest: freeway volume at the maximum point (before the two ramps), and the diverge volume before each of the off-ramps. Each checkpoint volume must be expanded by the peak hour factor to reflect peak flow rates and compared with the standards in Tables 2.1 and 4.1.

V_f = 5172/0.95 = 5444 pcph (Level D, Table 2.1)

V_d(RAMP 1)=V_1(RAMP 1)=1869/0.95=1967 pcph (Level E, Table 4.1)

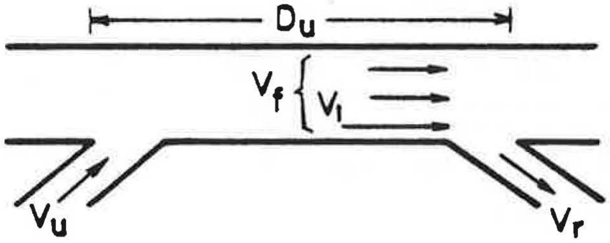
V_d(RAMP 2)=V_1(RAMP 2)=1647/0.95=1734 pcph (Level D, Table 4.1)

In this situation, the diverge at Ramp 1 is clearly the critical restrictive element, causing the overall Level of Service to be E. The high lane 1 volume at that point, however, is greatly influenced by the presence of the second, more highly utilized off-ramp. The diverge movement at Ramp 1, per se, is not really a problem - the total lane 1 volume is. In such a situation, other than considering combining the two ramps at a single exit point, there are no minor improvements which could be made to alleviate the situation.



EQUATION: $V_1 = 94 + 0.231 V_f + 0.473 V_r + 215 V_u/D_u$

DIAGRAM:



CONDITIONS FOR USE:

1) Single-lane off-ramp on a 6-lane freeway with or without upstream on-ramp, with or without deceleration lane.
2) If there is no upstream on-ramp within 5700 ft., skip step 2 below, and set 215 V_u/D_u to 2.
3) Normal range of use: V_f = 1100 to 6200 vph; V_r = 20 to 1800 vph; V_u = 50 to 1200 vph; D_u = 900 to 5700 ft.

STEPS IN SOLUTION:

1) Draw line from V_f value to V_r value, intersecting turning line.
2) Draw line from V_u value to D_u value, intersecting 215 V_u/D_u line.
3) Draw line from intersection point of step 1 to that of step 2; read solution on V_1 line.

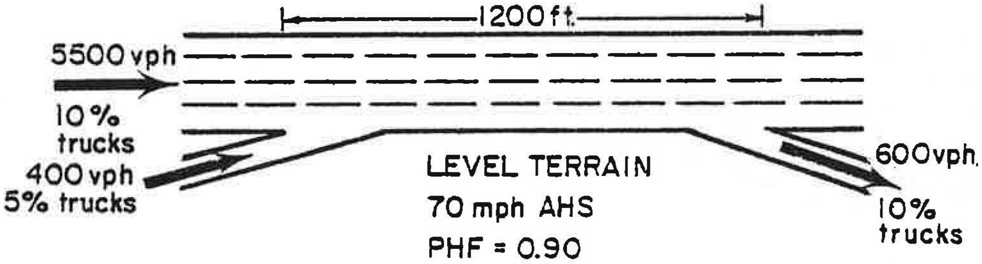
FIGURE 4.9

NOMOGRAPH SOLUTIONS FOR V_1, SAMPLE PROBLEM 2

(Nomograph is Figure A4.7)

Problem 3: On-Ramp Followed by an Off-Ramp

Consider the following configuration:



At what Level of Service would the section operate?

Solution: Table 4.2 indicates that the on-ramp be analyzed using Figure A4.10. The off-ramp situation must be approximated using Table 4.3 and Figure 4.3.

ON-RAMP: Note that the distance of 1200 ft. between ramps falls outside of the calibrated range of 1500 to 3000 ft. for Figure A4.10. Nevertheless, it appears reasonable to extend this range slightly for use here, rather than resort to Table 4.3 and Figure 4.3. In this case, Figure A4.10, though not directly applicable, is used to approximate the situation. Thus,

$$V_1 = -353 + 0.199\ V_f - 0.057\ V_r + 0.486\ V_d$$

$$V_1 = -353 + 0.199\ (5500) - 0.057\ (4000 + 0.486\ (600)$$

$$V_1 = 1010 \text{ vph}$$

From Figure 4.6, the percentage of total trucks in lane 1 is 49. Therefore:

total trucks on freeway = 5500 x 0.10 = 550
trucks in lane 1 = 550 x 0.49 = 270
% trucks in lane 1 = (270/1010)x100 = 26.7% say, 27%

OFF-RAMP: The freeway volume in advance of the off-ramp is 5500 + 400 = 5900 vph. It contains 5500 (0.10) + 400 (0.05) = 570 trucks, or (570/5900) x 100 = 9.7%.

The "through" volume for this section, that is, the volume not involved in either ramp movement, is 5500-600 or 4900 vph. The lane 1 volume immediately in advance of the off-ramp consists of:

9% of the through vehicles (Table 4.3)
100% of the off-ramp vehicles (Figure 4.3 I)
48% of the on-ramp vehicles (Figure 4.3 II, interpolating between 1000 ft. and 1500 ft.)

Thus: $V_1 = 0.09\ (4900) + 1.00\ (600) + 0.48\ (400)$

$V_1 = 1233$ vph

From Figure 4.6, this lane 1 volume contains 54% of the total trucks on the freeway. Therefore:

total trucks on freeway = 5900 x 0.097 = 572
trucks in lane 1 = 572 x 0.54 = 309
% trucks in lane 1 = (309/1233)x100 = 25.1, say 25%

Now, each volume must be converted to pcph and expanded to a peak flow rate by dividing by the PHF. This is done in the table below. Note that the freeway volume is taken between the ramps, at a point where it is the maximum value.

item	Vol. (vph)	% trucks	E_T^*	Q**	vol.(pcph)$=\frac{\text{vol(vph)}}{Q}$	$\frac{\text{vol.(pcph)}}{\text{PHF}}$
V_1(on)	1010	27	2	0.79	1278	1420
V_1(off)	1233	25	2	0.80	1541	1712
V_f	5900	10	2	0.91	6484	7204
V_r(on)	400	5	2	0.95	421	468
V_r(off)	600	10	2	0.91	659	732

* Table 2.4
** Table 2.8 or $Q = 100/[100 + P_T\ (E_T - 1)]$

Critical checkpoint volumes may now be computed and compared with the criteria in Tables 2.1 and 4.1.

$V_m = V_1(\text{on}) + V_r(\text{on}) = 1420+468$
$= 1888$ pcph (LEVEL E, Table 4.1)

$V_d = V_1(\text{off}) = 1712$ pcph (LEVEL D, Table 4.1)

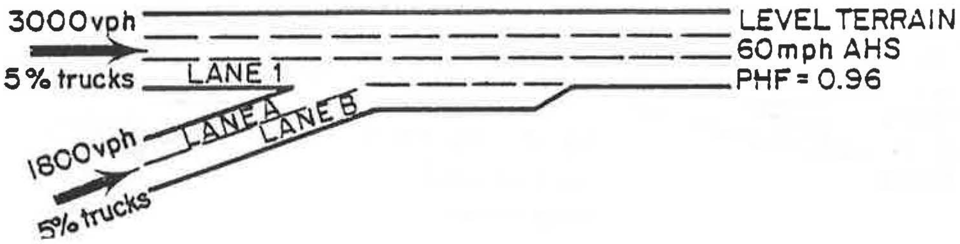
$V_w = [V_r(\text{on}) + V_r(\text{off})][500/1200]$
$= [468+732][0.417] = 500$ pcph
(LEVEL A, Table 4.1)

$V_f = 7204$ pcph (LEVEL D, Table 2.1)

In this case, Level of Service E will prevail due to the merge at the on-ramp, the diverge at the off-ramp, and general freeway conditions. Weaving is not expected to create problems in this section.

Problem 4: Two-Lane On-Ramp

Consider the following two-lane on-ramp:



What Level of Service would be expected at this location?

Solution: Table 4.2 indicates that Figure A4.11 should be used for this problem. Note, in Figure A4.11, that the solution to this problem entails two merges - the first when the lane 1 volume merges with the lane A volume, the second when the lane B volume merges with the total volume from the first merge. The second merge is the most critical for analysis.

Figure 4.10 illustrates the nomograph solution for V_1 and for V_{1+A}. <---Fi

Therefore: V_{1+A} = 1700 vph

V_1 = 352 vph

$V_A = V_{1+A} - V_1$ = 1700 - 352 = 1348 vph

V_B = 1800 - 1348 = 452 vph

V_f (after merging) = 4800 vph

Each of these must be converted to pcph. To accomplish this, it is necessary to assume that there are 5% trucks in both lanes A and B of the ramp. Procedures do not give specific guidance on this point, and lacking specific information, and equal percentage of truck traffic in each ramp lane would be assumed. From Figure 4.6, 49% of the total trucks on the freeway are in lane 1 immediately in advance of the ramp. Thus:

item	vol.(vph)	% trucks	E_T^*	Q**	vol.(pcph)= $\frac{\text{vol.(vph)}}{Q}$	$\frac{\text{vol.(pcph)}}{\text{PHF}}$
V_1	352	21	2	0.83	424	442
V_{1+A}	1700	8	2	0.93	1828	1904
V_A	1348	5	2	0.95	1419	1478
V_B	452	5	2	0.95	476	496
V_f	4800	5	2	0.95	5053	5264

* Table 2.4
** Table 2.8 or $Q = 100/[100 - P_T (E_T - 1)]$

Checkpoint volumes may now be computed and compared with standards:

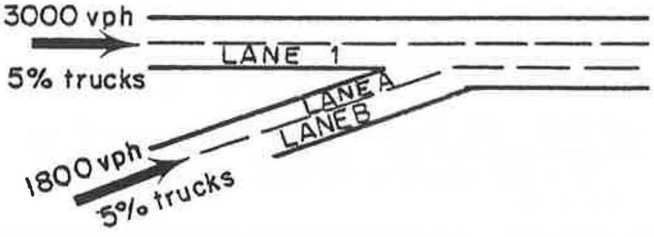
$V_{m1} = V_1+V_A$ = 442+1478 = 1920 pcph (LEVEL E, Table 4.1)

$V_{m2} = V_{1+A}+V_B$ = 1904+496 = 2400 pcph (LEVEL F, Table 4.1)

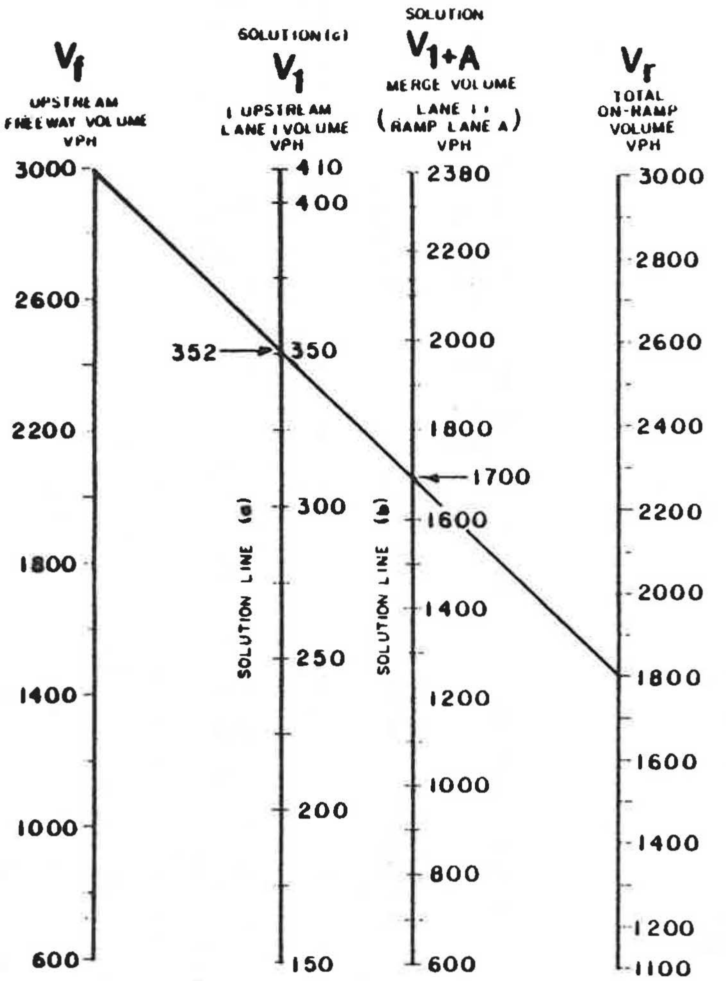
V_f = 5264 pcph (LEVEL D, Table 2.1)

Obviously, the second merge volume of 2400 pcph would not actually occur. However, it is clear that during peak periods of flow, great congestion will exist in the vicinity of this merge-congestion caused by the merge itself. Level of Service Fis highly likely.

The addition of a lane at this point which would be carried for a significant distance might be considered. If this is not possible, the deletion of a lane from the main freeway approaching the merge might be considered, creating a major junction with geometry as shown below:



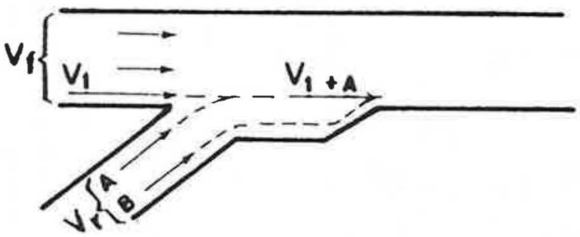
From Table 4.2, this alternative may be analyzed using Note 5, which describes a multi-step trial-and-error process.



EQUATION: (a) $V_1 = 54 + 0.070V_f + 0.049V_r$

(b) $V_{1+A} = -205 + 0.287V_f + 0.575V_r$

DIAGRAM:



CONDITIONS FOR USE:

1) Two-lane on-ramps on 6-lane freeways with acceleration lane of at least 800 feet in length.

2) Normal range of use: V_f = 600 to 3000 vph
V_r = 1100 to 3000 vph

STEPS IN SOLUTION:

1) Draw line from V_f value to V_r value. Read V_1 on V_1 line, V_{1+A} on V_{1+A} line.

2) Compute $V_A = V_{1+A}-1$; $V_B = V_r-V_A$

3) Check L. of S. for two merge points: $V_{m1} = V_1+V_A$; $V_{M2} = V_{1+A}+V_B$

FIGURE 4.10

SOLUTION FOR V_1 AND V_{1+A} IN SAMPLE PROBLEM 4

(Nomograph is Figure A4.11)

From Table 4.2, this alternative may be analyzed using Note 5, which describes a multi-step trial-and-error process.

If Level of Service D is assumed, lane B is assumed to carry 1800 pcph or 1800 x 0.95 = 1710 vph. Thus, lane A would carry only 90 vph. At Level of Service C, lane B would carry 1550 pcph or 1550 x 0.95 = 1473 vph. Lane A would carry 327 vph. At Level B, lane B carries 1200 x 0.95 = 1140 vph and lane A carries 660 vph. These values are drawn from Table 4.1 in accordance with Note 5 and converted to vph.

The lane 1 volume is found using Figure A4.1. Therefore:

$V_1 = 136 + 0.345\ V_f - 0.115\ V_r$

where V_r is taken to be only the lane A volume. As the assumption of Level of Service 2 resulted in the most reasonable distribution of the ramp volume, that level is tried first:

$V_1 = 136 + 0.345\ (3000) - 0.115\ (660)$

$V_1 = 1096$ vph

From Figure 4.6, lane 1 will contain 80% of all trucks on the freeway, or:

trucks in lane 1 = 3000 (0.05) (0.8) = 120
% trucks in lane 1 = (120/1096) x 100 = 10.9%, say 11%

The checkpoint volume of interest here is the merge volume consisting of the lane 1 volume plus the lane A volume. Converting these to pcph and dividing by the PHF:

item	vol.(vph)	% trucks	E_T^*	Q**	vol.(pcph)= $\frac{\text{vol.(vph)}}{Q}$	$\frac{\text{vol.(pcph)}}{\text{PHF}}$
V_1	1096	11	2	0.90	1218	1269
V_A	660	5	2	0.95	695	724

* Table 2.4
** Table 2.8

Then: $V_m = V_1 + V_A = 1269 + 724 = 1993$ pcph
(LEVEL E, Table 4.1)

As Level of Service B was assumed, and Level E resulted, a second trial, starting with Level C should be made: Then, $V_A = 327$, and:

$V_1 = 136 + 0.345\ (3000) - 0.115\ (327) = 1133$ vph

As previously, lane 1 will contain 120 trucks, or (120/1133) x 100 = 10.6%, say 11%. Converting V_1 and V_A to pcph, and dividing by the PHF:

item	vol.(vph)	% trucks	E_T^*	Q**	vol.(pcph)= $\frac{\text{vol.(vph)}}{Q}$	$\frac{\text{vol.(pcph)}}{\text{PHF}}$
V_1	1133	11	2	0.90	1259	1311
V_A	327	5	2	0.95	344	359

* Table 2.4
** Table 2.8

Then: $V_m = 1311 + 359 = 1770$ pcph
(LEVEL D, Table 4.1)

It is now clear that the merge Level of Service will be in the D range. A third trial could be made to confirm this, but the computations thus far leave no other alternative. Remember that the total freeway volume downstream of the merge remains the same as in the initial problem, and produces Level of Service D, so that a balanced operation will result.

It is, however, certain that the alternate geometry produces a more orderly merge, and greatly improved overall operation. The first design tended to force ramp vehicles into lane A, whereas the second makes a great deal more use of lane B. Further, by "adding" a lane, lane B vehicles do not merge. The removal of a lane on the freeway at this point is not critical, as the Level of Service provided by the three initial lanes is out of balance with the merge and downstream conditions. Two lanes is sufficient for balanced operation. If a third lane were needed at some more distant upstream point, a lane drop would have to be designed before approaching the vicinity of the merge in question.

Problem 5: Ramp Proper

A loop ramp with a design speed of 25 mph is expected to carry 800 vph, 10% of which are trucks. If the PHF = 0.90 and the ramp is on a 1400 ft., 4% upgrade, what design should be adopted and what Level of Service can be expected.

Solution: First, the demand volume must be adjusted to reflect pcph and a peak flow rate. Note that 1400 ft. 1/4 mile. From Table 2.6, $E_T = 4$ (using 4-lane freeway values). From Table 2.8, Q = 0.77. Thus, the adjusted demand flow rate is:

800/(0.77 x 0.90) = 1154 pcph

From Table 4.6, a one-lane ramp would provide for Level of Service E (design speed = 25 mph). Since the ramp is longer than 1000 ft., however, it is advisable to provide for flush shoulders wide enough to permit passing of a stalled vehicle. Two lane loop ramps are not normally provided.

Provision of a better Level of Service requires an improvement in the design speed used. A 40-50 mph design speed would result in Level of Service C operations, a more acceptable result.

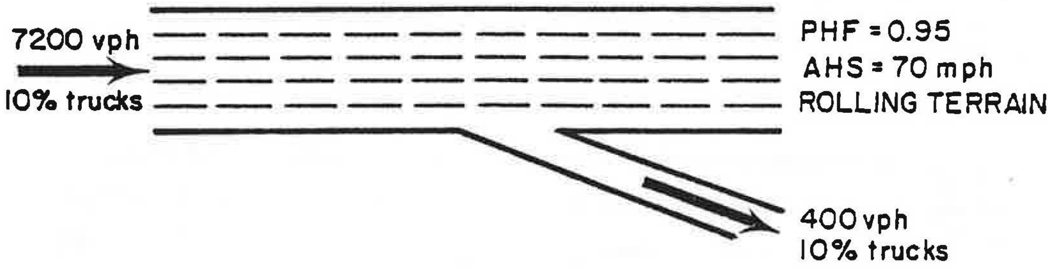
A 40-50 mph design speed will create an extremely long loop ramp, consuming a great deal of land in its wake. The designer is faced with several options here:

- accept a lower Level of Service, using a loop ramp of 25 mph design speed
- use a 40-50 mph loop ramp and accept the inefficiency of the design
- design a direct interchange not requiring a loop ramp. (This involves costly structures.)

In this case, only the first and last alternatives could reasonably be considered.

Problem 6: Isolated Off-Ramp on a 10-Lane Freeway (Five-Lane Freeway Segment)

The following off-ramp occurs on a five-lane freeway segment (10-Lane Freeway):



What Level of Service would be expected to prevail?

Solution: From Table 4.4, the segment may be treated as though it were a four-lane segment with a freeway volume of:

V_f = 7200 x 0.80 = 5760 vph

From Table 4.2, for an 8-lane freeway, the lane 1 volume must be approximated using Table 4.3 and Figure 4.3, using a freeway volume of 5760 vph. From Table 4.3, 10% of through traffic will remain in lane 1 in the vicinity of the ramp; 100% of off-ramp traffic must also be in lane 1 immediately upstream of the ramp. Thus,

Through Vehicles in Lane 1 = (5760 - 400) x 0.10 = 536 vph

Off-Ramp Vehicles in Lane 1 = $\underline{400 \text{ vph}}$

V_1 = 936 vph

From Figure 4.6, using an 8-lane freeway with V_f of 5760 vph, the percentage of total trucks in lane 1 is 52%. Thus:

total trucks on freeway = 5760 x 0.10 = 576 vph
total trucks in lane 1 = 576 x 0.52 = 300 vph
% trucks in lane 1 = (300/936) x 100 = 32%

Then:

item	vol.(vph)	E_T^*	Q**	vol.(pcph)= $\frac{\text{vol.(vph)}}{Q}$	$\frac{\text{vol.(pcph)}}{PHF}$
V_1	936	4	0.51	1835	1932
V_r	400	4	0.77	519	547
V_f	7200	4	0.77	9350	9842

* Table 2.4
** Table 2.8, or computed

Computing checkpoint volumes:

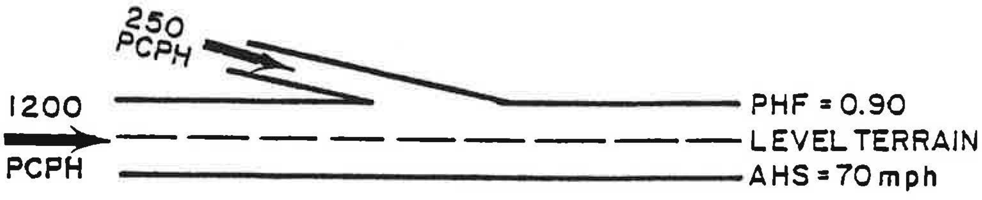
V_d = V_1 = 1932 pcph (Level E, Table 4.1)

V_f = 9842 pcph for 5-lanes (Level E, Table 2.1)

The segment will operate at Level of Service E. All elements are in balance.

Problem 7 - Left-Hand On-Ramp

Consider the following left-hand on ramp:



At what Level of Service would the section be expected to operate?

Solution: In this problem, the volume in the left-hand lane must be computed immediately upstream of the ramp. The "SPECIAL CASE-Left Hand Ramps" section of this chapter indicates that this volume (V_i) can be approximated as 1.25 times V_1 for a similar configuration of a right-hand ramp.

From Table 4.2, the equivalent V_1 is found using Figure A4.1. Use of this nomograph results in:

V_1 = 520 pcph

Thus: V_i = 520 x 1.25 = 650 pcph

Computing checkpoint values and dividing by the PHF:

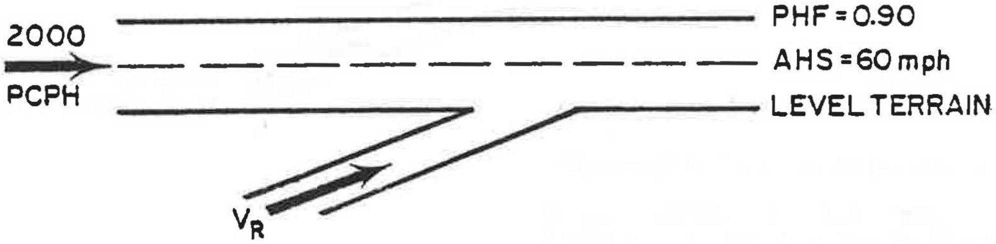
V_m = (650 + 250)/0.90 = 1000 pcph (Level B, Table 4.1)

V_f = (1200 + 250)/0.90 = 1611 pcph (Level B, Table 2.1)

The facility will operate at Level of Service B.

Problem 8: Ramp Metering

It is desired to control V_r by establishing a maximum flow rate through ramp metering at the following location:



If a fixed-time ramp meter is utilized, 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?

Solution: The question asks for a solution of a maximum value of V_r such that the Level of Service is C. The trial-and-error method described in Section V of this chapter is used.

From Table 4.1, the maximum merge service volume for Level of Service C is 1550 pcph (peak flow rate). For a PHF of 0.90, this is equivalent to an hourly volume of 1550 x 0.90 = 1395 vph. Considering the situation given in the problem, a tabular computation may be set up as follows:

Assumed V_r	V_1 (Fig. A4.1)	Computed Vr ($1395-V_1$)	Comparison
200	810	585	NG
400	775	620	NG
600	760	635	NG
650	750	645	OK SAY 650 PCPH

A metering rate of 650 PCPH or one car every 3600/650 = 5.54, say 5.5 seconds would be set.

These computations are, naturally, more complex where volumes contain mixed vehicles, but the procedure and basic approach are as illustrated herein.

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

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

and considering that

$$V_r = 1395 - V_1 \qquad (2)$$

Substituting for V_1 in equation 2,

$$V_r = 1395 - (136+0.345\ V_f - 0.115\ V_r)$$

$$V_r = 1259 - 0.345\ V_f + 0.115\ V_r$$

$$0.885\ V_r = 1259 - 0.345\ V_f$$

$$V_r = \frac{1259-0.345\ V_f}{0.885}$$

$$V_r = \frac{1259-0.345(2000)}{0.885} = 643 \text{ pcph}$$

CHAPTER IV - REFERENCES

1) "Highway Capacity Manual," TRB Special Report 87, Transportation Research Board, Washington, D.C., 1965.

2) A Policy on Design of Urban Highways and Arterial Streets, American Association of State Highway and Transportation Officials, Washington, D.C., 1973.

3) A Policy on Geometric Design of Rural Highways, American Association of State Highway and Transportation Officials, Washington, D.C., 1965.

4) Pignataro, et al, Weaving Area Operations Study, Final Report, National Cooperative Highway Research Program Project 3-15, Polytechnic Institute of Brooklyn, Brooklyn, NY, Nov. 1973.

5) J. Leisch, Capacity Analysis Techniques for Design and Operation of Freeway Facilities, Report No. FHWA-RD-74-24, U.S. Department of Transportation, Washington, D.C., Feb. 1974.

6) Pignataro, Traffic Engineering: Theory and Practice, Prentice-Hall Inc., Englewood Cliffs, NJ, 1974, Ch. 28.

7) Wattleworth, et al, "Operational Effects of Some Entrance Ramp Geometrics on Freeway Merging," Texas Transportation Institute Report 430-4, Texas A&M University, 1967.

8) Drew, Traffic Flow Theory and Control, McGraw-Hill, New York, 1968.

9) Everall, Urban Freeway Surveillance and Control: The State of the Art, Federal Highway Administration, USGPO Stock No. 5001-00058, June 1973.

Appendix

NOMOGRAPHS FOR THE SOLUTION OF LANE 1 VOLUMES

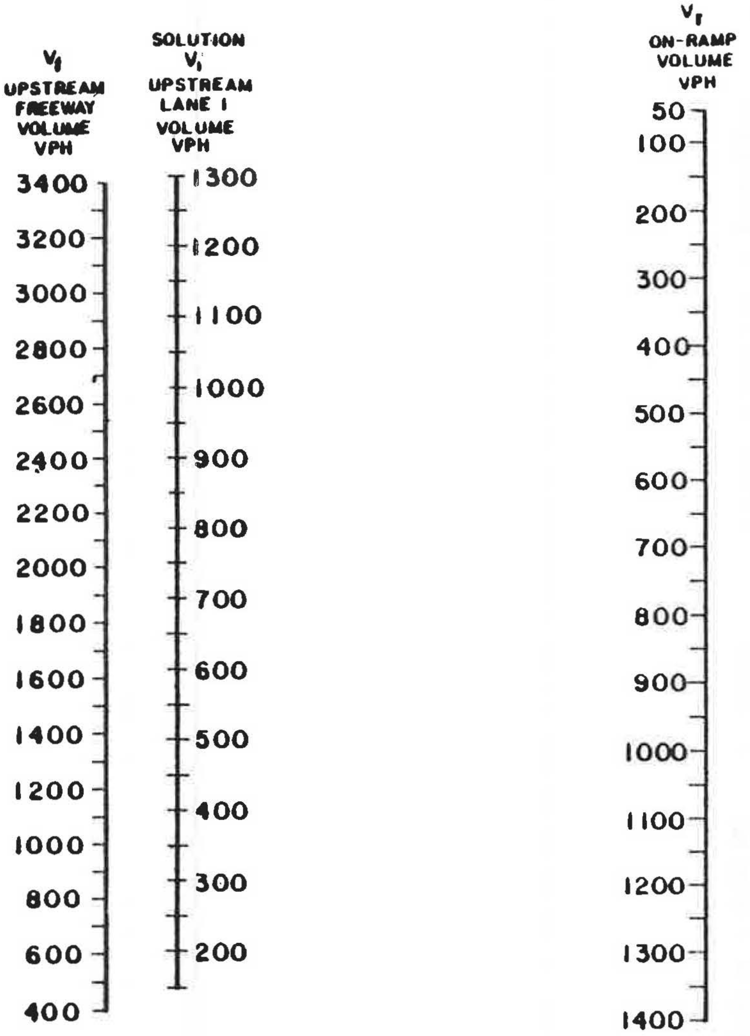
In using the nomographs in this Appendix, note the following:

CONDITIONS FOR USE specify configurations for which the nomograph and accompanying equation were apply. Where use is indicated for both ramps "with or without acceleration/deceleration lanes," the data base used in calibrating the nomograph included both, and no statistically significant differences (with or without) were observed. "Normal range of use" indicates the range of data used to calibrate the nomograph. Use outside this range should be limited to cases close to the range, and should be done with caution.

CONDITIONS FOR USE also contain instructions for using nomographs to approximate configurations not covered elsewhere.

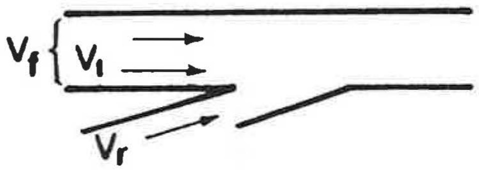
STEPS IN SOLUTION are a step-by-step set of instructions for using each nomograph.

EQUATION shows the mathematical relationship for each nomograph, which may be used directly for greater precision in V_1 computations.



EQUATION: $V_1 = 136 + 0.345V_f - 0.115V_r$

DIAGRAM:



CONDITIONS FOR USE:

1) Single-lane on-ramp (not a loop) on 4-lane freeway, with or without acceleration lane.

2) For use only when no adjacent upstream on-ramp exists within 2000 feet.

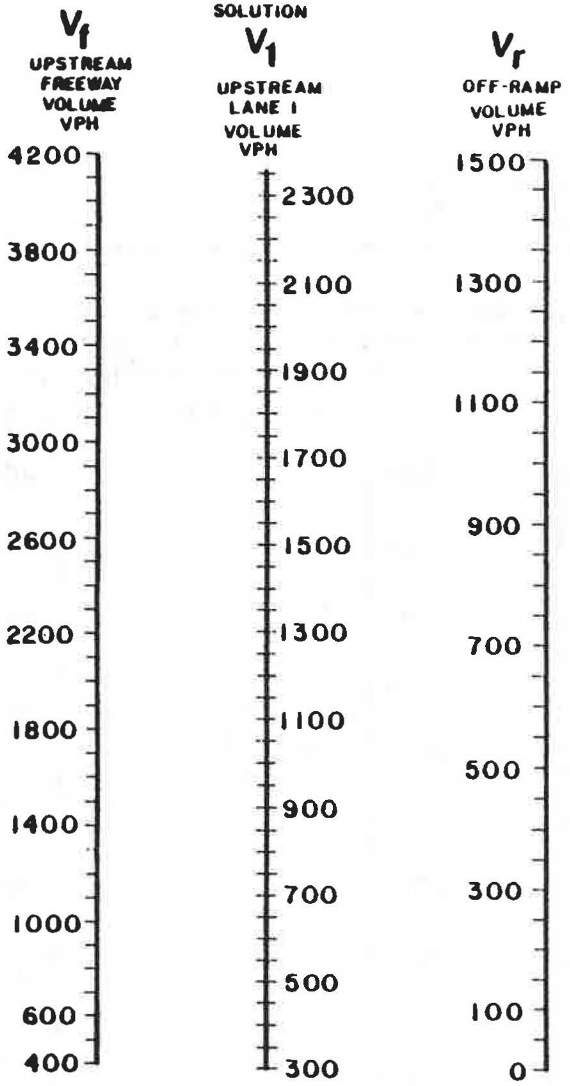
3) Normal range of use: V_f = 400 to 3400 vph
V_r = 50 to 1400 vph

STEPS IN SOLUTION:

1) Draw line from V_f value to V_r value; read solution on V_1 line.

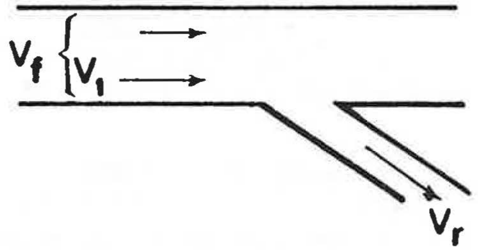
FIGURE A4.1

DETERMINATION OF LANE 1 VOLUME UPSTREAM
OF ONE-LANE ON-RAMPS ON 4-LANE FREEWAYS
(2 lanes in each direction)



EQUATION: $V_1 = 165 + 0.345 V_f + 0.520 V_r$

DIAGRAM:



CONDITIONS FOR USE:

1) Single-lane off-ramp on a 4-lane freeway, with or without a deceleration lane.

2) For use only when there is no adjacent upstream on-ramp within 3200 ft.

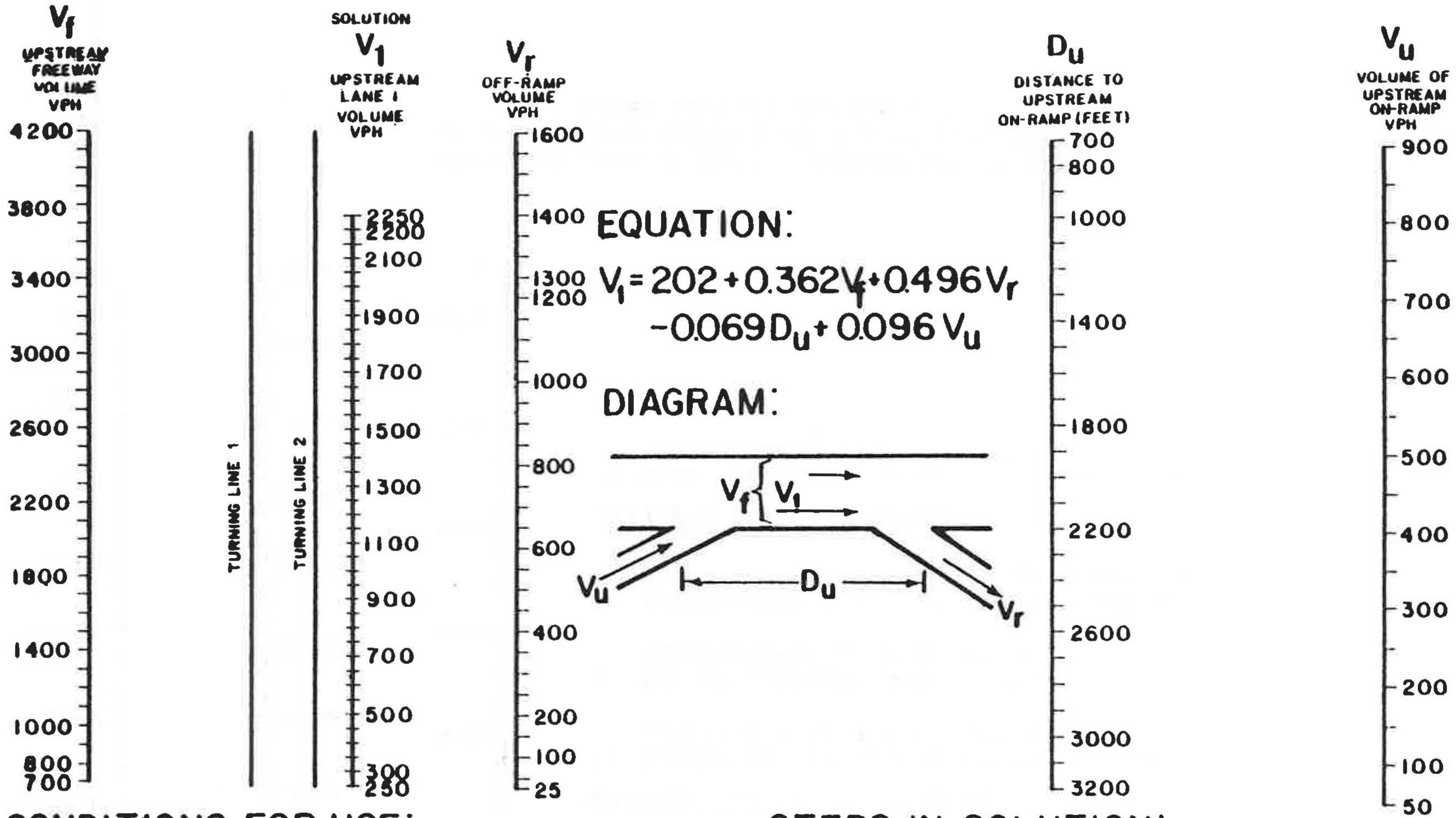
3) Normal range of use: V_f = 400 to 4200 vph
V_r = 50 to 1500 vph

STEPS IN SOLUTION:

1) Draw line from V_f value to V_r value; read solution on V_1 line.

FIGURE A4.2

DETERMINATION OF LANE 1 VOLUME UPSTREAM OF ONE-LANE OFF-RAMPS ON 4-LANE FREEWAYS (2 lanes in each direction)



CONDITIONS FOR USE:

1) Single-lane off-ramp on a 4-lane freeway, with or without a deceleration lane, with an adjacent upstream on-ramp within 3200 ft.

2) Normal range of use:
V_f = 70 to 4200 vph
V_r = 50 to 1600 vph
V_u = 50 to 900 vph
D_u = 700 to 3200 ft.

STEPS IN SOLUTION:

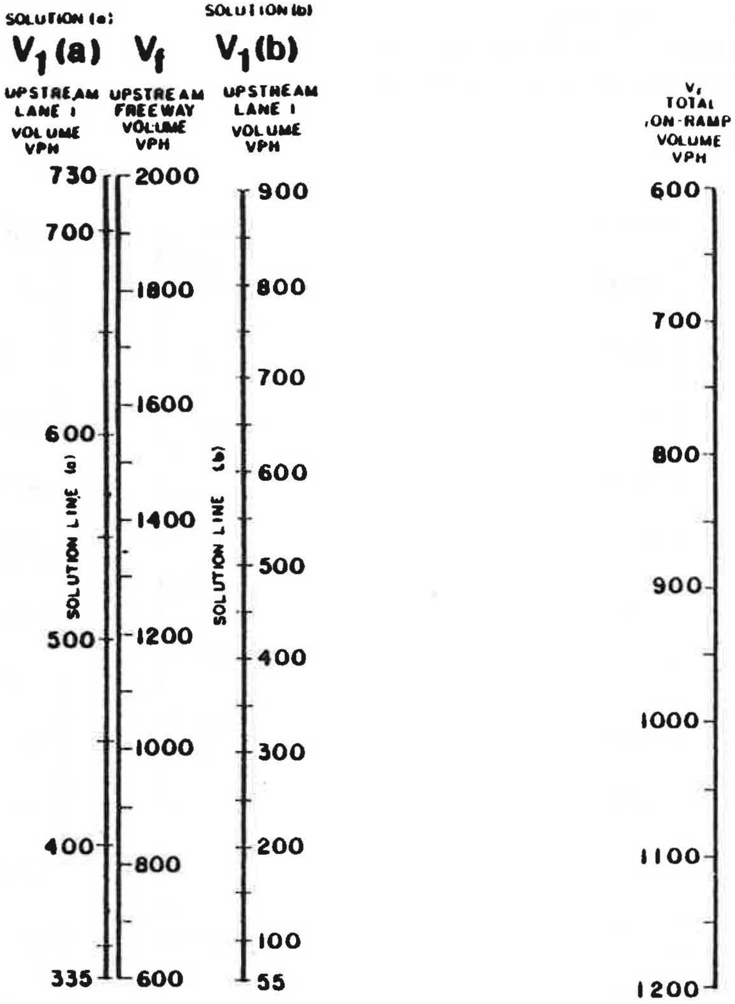
1) Draw line from V_f value to V_r value, intersecting turning line 1.

2) Draw a line from the point defined in step 1 to the D_u value, intersecting turning line 2.

3) Draw a line from the point defined in step 2 to the V_u value. Read the result from the V_1 line.

FIGURE A4.3

DETERMINATION OF LANE 1 VOLUME UPSTREAM OF ONE-LANE OFF-RAMPS ON 4-LANE FREEWAYS (2 lanes in each dir.) WITH ADJACENT UPSTREAM ON-RAMPS



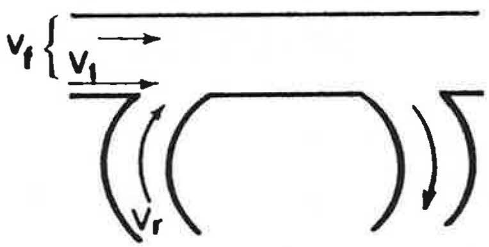
EQUATION: $aV_1 = 166 + 0.280V_f$

(for $V_r < 600$ VPH)

$bV_1 = 128 + 0.482V_f - 0.301V_r$

(for V_r between 600-1200 VPH)

DIAGRAM:



CONDITIONS FOR USE:

1) Single-lane, loop-type on-ramp on a 4-lane freeway with or without an acceleration lane.

2) Normal range of use:
V_f = 600 to 2000 vph
V_r = 600 to 1200 vph (b)
0 to 600 vph (a)

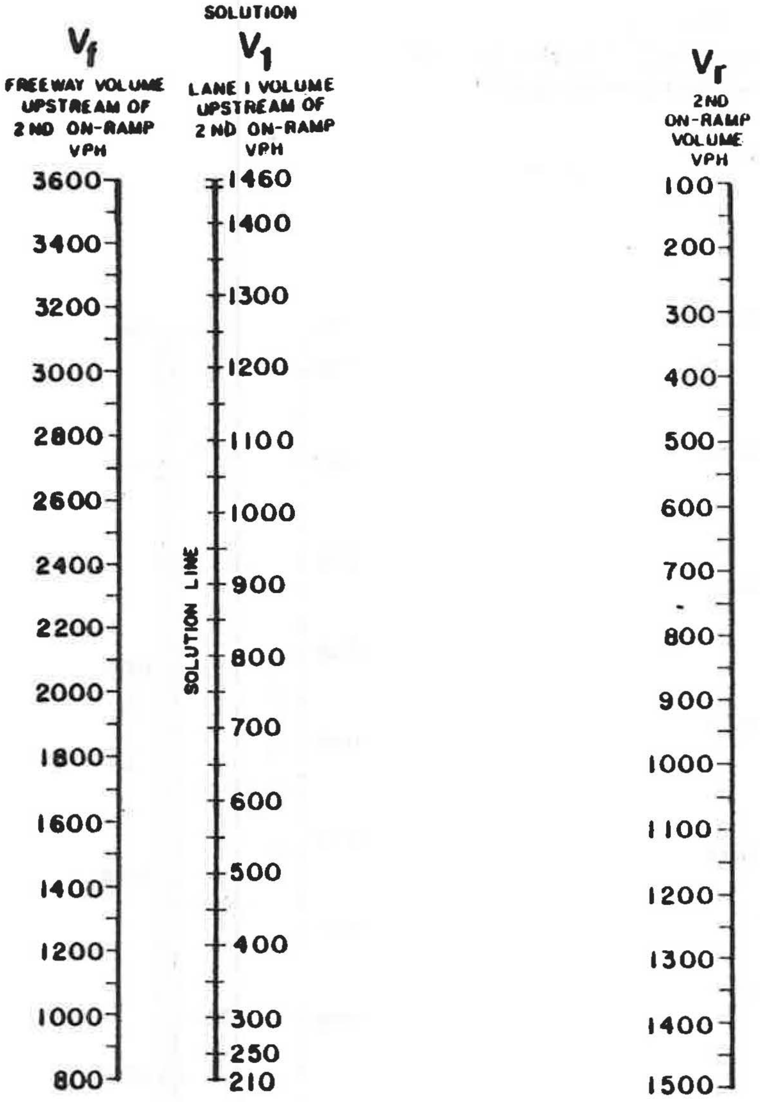
STEPS IN SOLUTION:

1) If V_r < 600 vph, read solution on solution line (a), horizontally to the left of V_f value

2) If V_r is between 600 and 1200 vph, draw a line from V_f value to V_r value; read result on solution line (b).

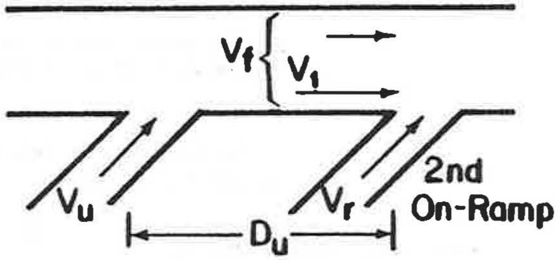
FIGURE A4.4

DETERMINATION OF LANE 1 VOLUME UPSTREAM OF ONE-LANE, LOOP-TYPE ON-RAMPS ON 4-LANE FREEWAYS (2 lanes in each direction)



EQUATION: $V_1 = 123 + 0.376V_f - 0.142V_r$

DIAGRAM:



CONDITIONS FOR USE:

1) Single-lane on ramp on 4-lane freeway with adjacent upstream on-ramp within 400 to 2000. feet (with or without acceleration lane).

2) Not accurate where $D_u \leq 400$ or $V_u \geq 1000$ vph.

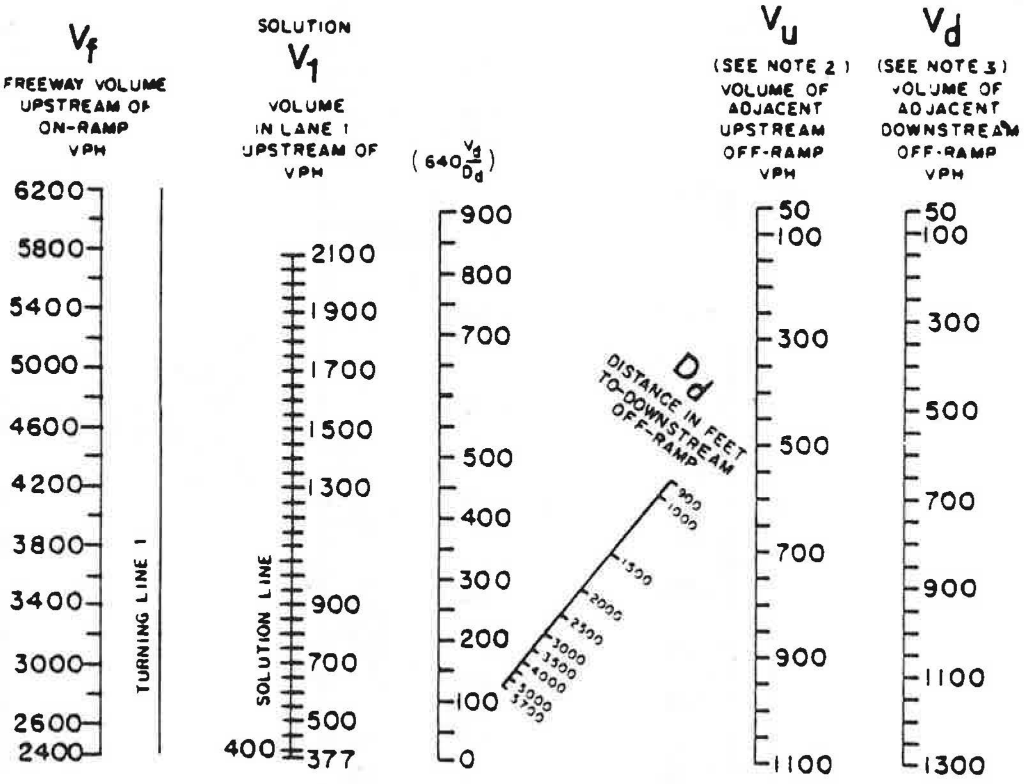
3) Normal range of use: V_f = 800 to 3600 vph
V_r = 100 to 1500 vph
V_u = 100 to 1000 vph
D_u = 400 to 2000 ft.

STEPS IN SOLUTION:

1) Draw a line from V_f value to V_r value; read solution on V_1 line.

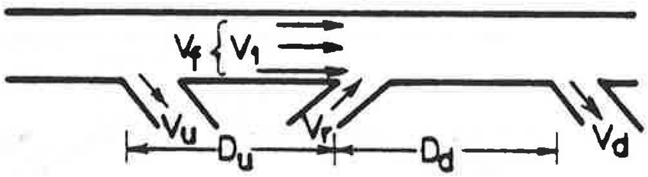
FIGURE A4.5

DETERMINATION OF LANE 1 VOLUME UPSTREAM OF ONE-LANE ON-RAMPS ON 4-LANE FREEWAYS (2 lane in each direction) WITH ADJACENT UPSTREAM ON-RAMPS



EQUATION: $V_1 = -121 + 0.244V_f - 0.085V_u + 640\,{}^{V_d}/_{D_d}$

DIAGRAM:



CONDITIONS FOR USE:

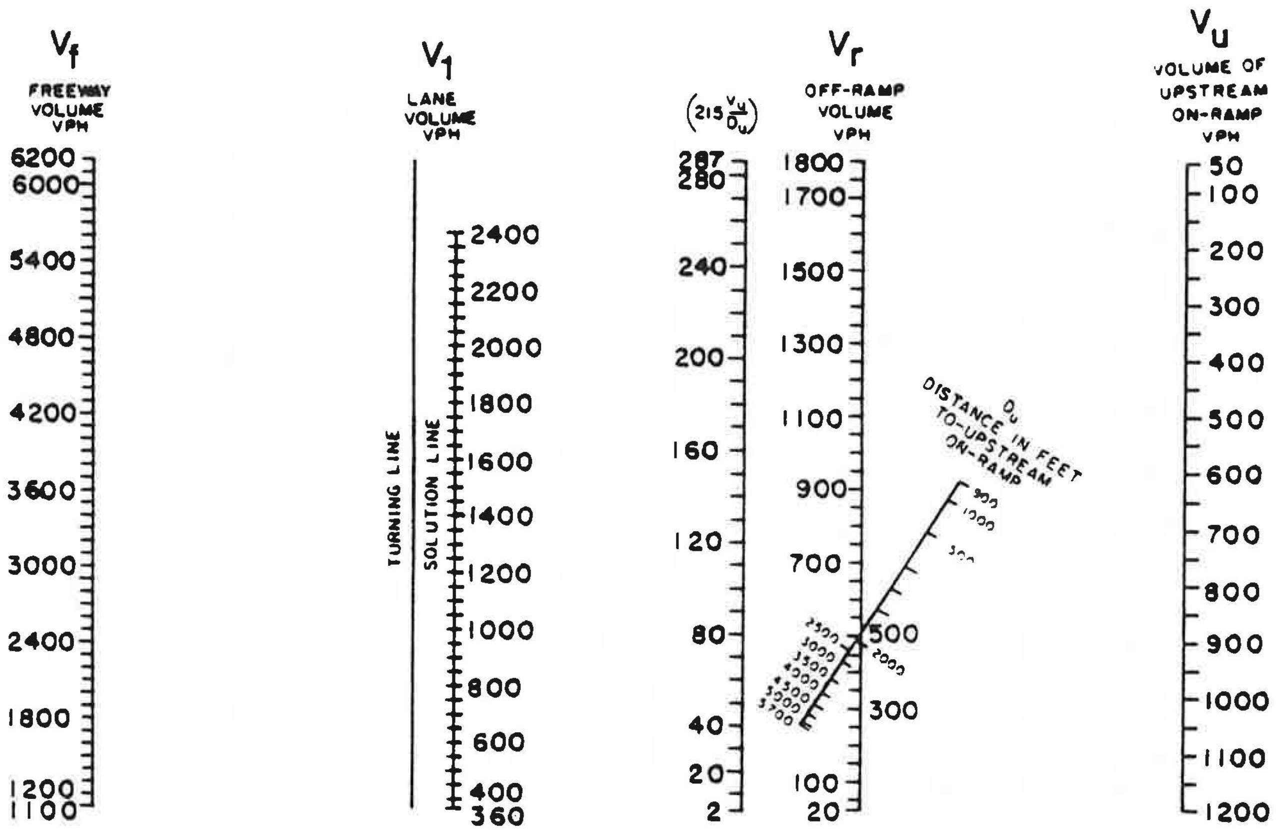
1) Single-lane on-ramps on 6-lane freeways with or without upstream and/or downstream off-ramps, with or without acceleration lane.
2) If there is no upstream off-ramp within 2600 ft., use $V_u = 50$.
3) If there is no downstream off-ramp within 5700 ft., and $V_f \leq 5000$ vph, use $640 \frac{V_d}{D_d} = 5$, and skip step 2 below.
4) Normal range of use: V_f=2400 to 6200 vph; V_u= 50 to 1100 vph; V_d= 50 to 1300 vph. V_r= 100 to 1700 vph; D_d=900 to 5700 ft.; D_u=900 to 2600 ft.

STEPS IN SOLUTION:

1) Draw a line from V_f value to V_u value, intersecting turning line 1.
2) Draw a line from V_d value to D_d value, intersecting 640 V_d/D_d line.
3) Draw a line from the step 1 intersection with turning line 1 to the 640 V_d/D_d value of step 2; read solution at intersection with V_1 line.

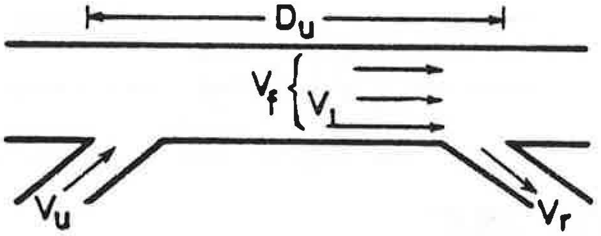
FIGURE A4.6

DETERMINATION OF LANE 1 VOLUME UPSTREAM OF
ONE-LANE ON-RAMPS ON 6-LANE FREEWAYS (3 lanes in each dir.)
WITH OR WITHOUT ADJACENT OFF-RAMPS



EQUATION: $V_1 = 94 + 0.231 V_f + 0.473 V_r + 215 V_u/D_u$

DIAGRAM:



CONDITIONS FOR USE:

1) Single-lane off-ramp on 6-lane freeway with or without upstream on-ramp with or without deceleration lane.

2) If there is no upstream on-ramp within 5700 ft., skip step 2 below, and set 215 V_u/D_u = 2.

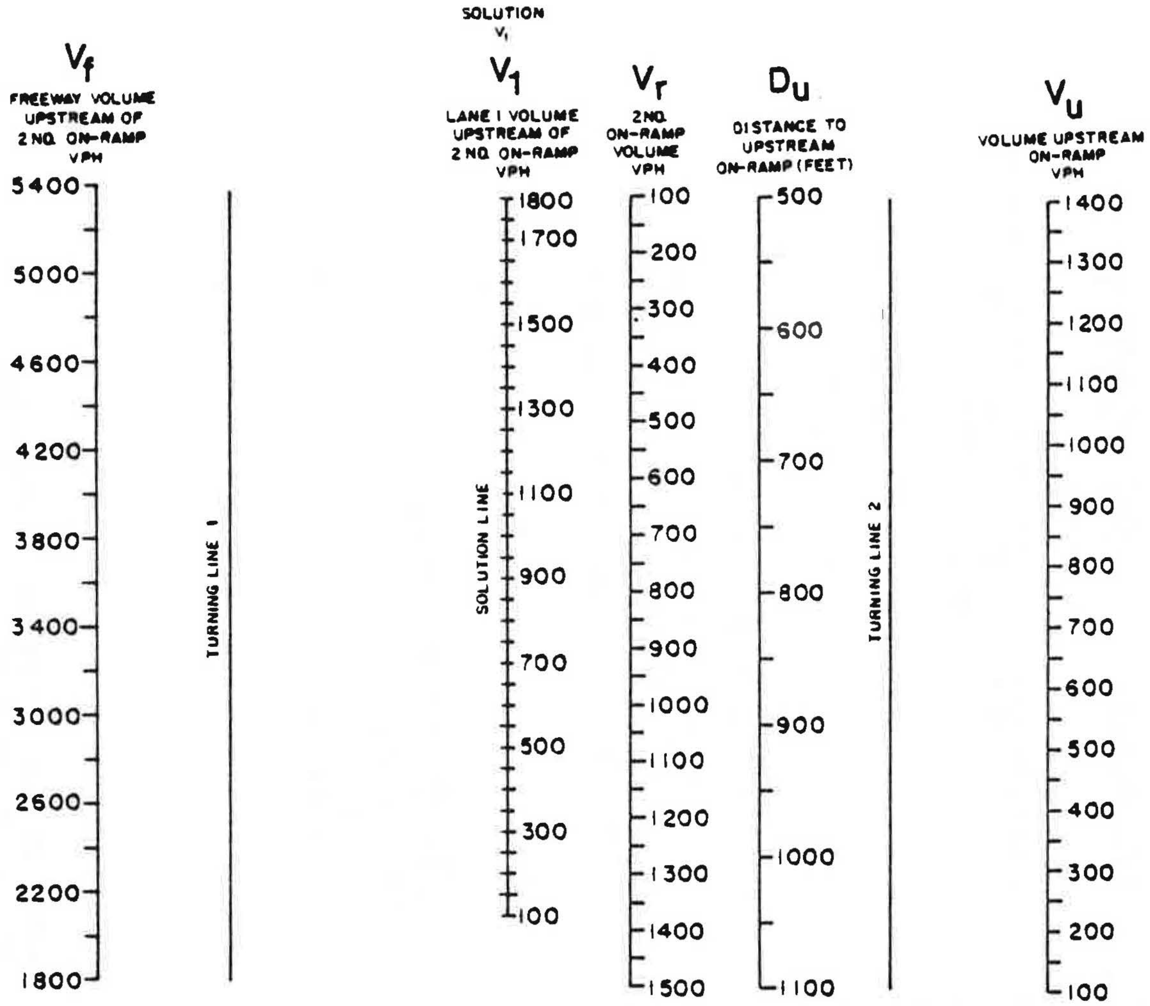
3) Normal range of use: V_f = 1100 to 6200 vph; V_r = 20 to 1800 vph;
V_u = 50 to 1200 vph; D_u = 900 to 5700 ft.

STEPS IN SOLUTION:

1) Draw a line from V_f value to V_r value, intersecting the turning line.
2) Draw a line from V_u value to D_u value, intersecting the 215 V_u/D_u line.
3) Draw a line from intersection point on the turning line of step 1 to the value on the 215 $\frac{V_r}{D_u}$ line of step 2; read solution on V_1 line.

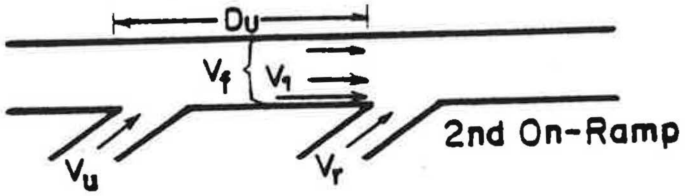
FIGURE A4.7

DETERMINATION OF LANE 1 VOLUME UPSTREAM OF
ONE-LANE OFF-RAMPS ON 6-LANE FREEWAYS
(3 lanes in each direction)



EQUATION: $V_1 = 574 + 0.228 V_f - 0.194 V_r - 0.714 D_u + 0.274 V_u$

DIAGRAM:



CONDITIONS FOR USE:

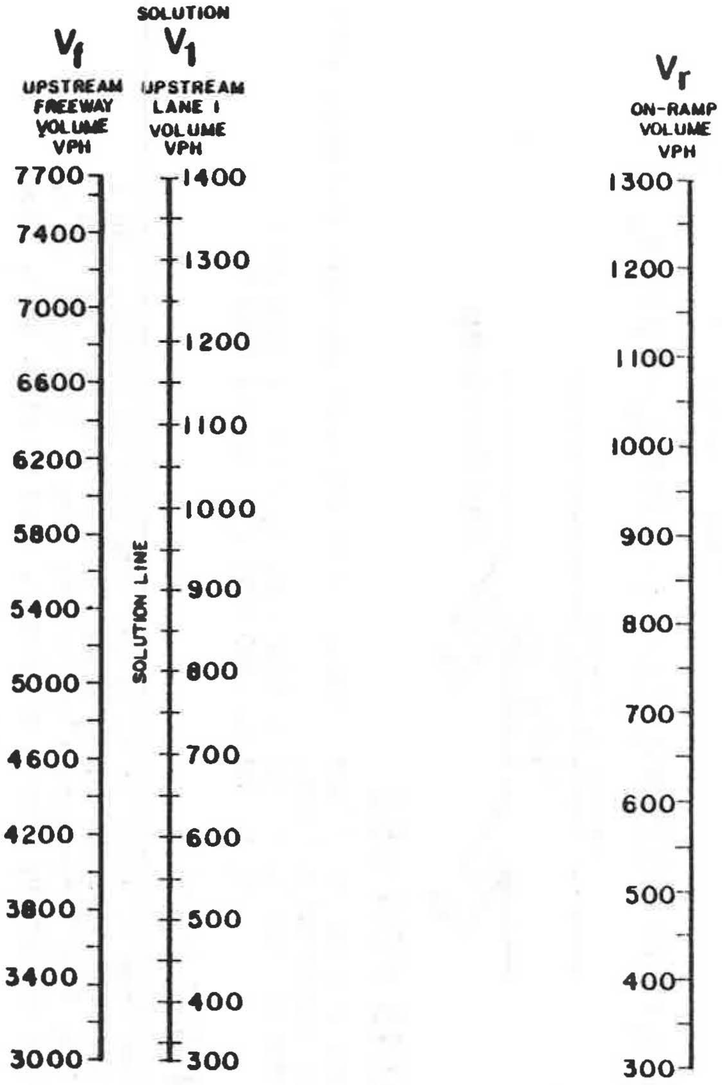
1) Single-lane on-ramp on 6-lane freeways with adjacent upstream on-ramps, with or without acceleration lanes.
2) Normal range of use: V_f = 1800 to 5400 vph; V_r = 100 to 1500 vph;
V_u = 100 to 1400 vph; D_u = 500 to 1000 ft.

STEPS IN SOLUTION:

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

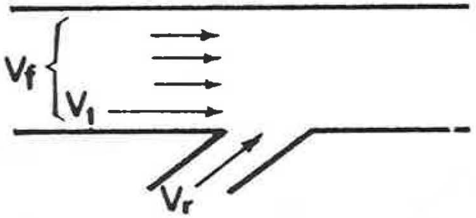
FIGURE A4.8

DETERMINATION OF LANE 1 VOLUME UPSTREAM OF ONE-LANE ON-RAMPS ON 6-LANE FREEWAYS (3 lanes in each direction) WITH UPSTREAM ON-RAMPS



EQUATION: $V_1 = -312 + 0.201 V_f + 0.127 V_r$

DIAGRAM:



CONDITIONS FOR USE:

1) Single-lane on-ramp on 8-lane freeway with or without acceleration lane.

2) Not for use if there is adjacent downstream off-ramp within 3000 ft.

3) Normal range of use: V_f = 3000 to 7700 vph
V_r = 300 to 1300 vph

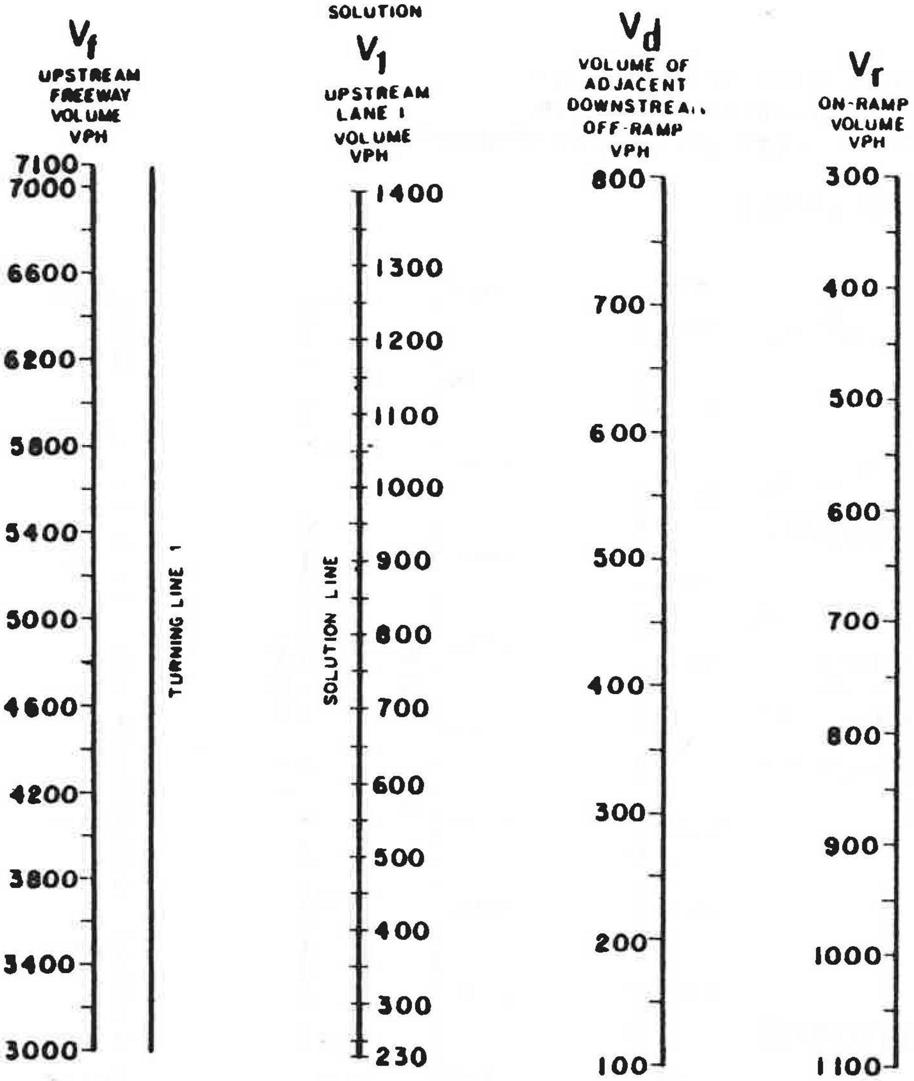
STEPS IN SOLUTION:

STEPS IN SOLUTION:

1) Draw a line from V_f value to V_r value; read solution on V_1 line.

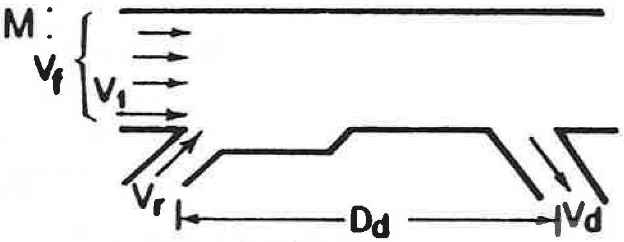
FIGURE A4.9

DETERMINATION OF LANE 1 VOLUME UPSTREAM OF ONE-LANE ON-RAMPS ON 8-LANE FREEWAYS
(4 lanes in each direction)



EQUATION: $-353 + 0.199 V_f - 0.057 V_r + 0.486 V_d$

DIAGRAM:



CONDITIONS FOR USE:

1) Single-lane on-ramp on 8-lane freeway with acceleration lane, with adjacent downstream off-ramp within 1500 to 3000 ft..

2) Normal limits of use:
V_f = 3000 to 7100 vph
V_r = 300 to 1100 vph
V_d = 100 to 800 vph
D_d = 1500 to 3000 ft.

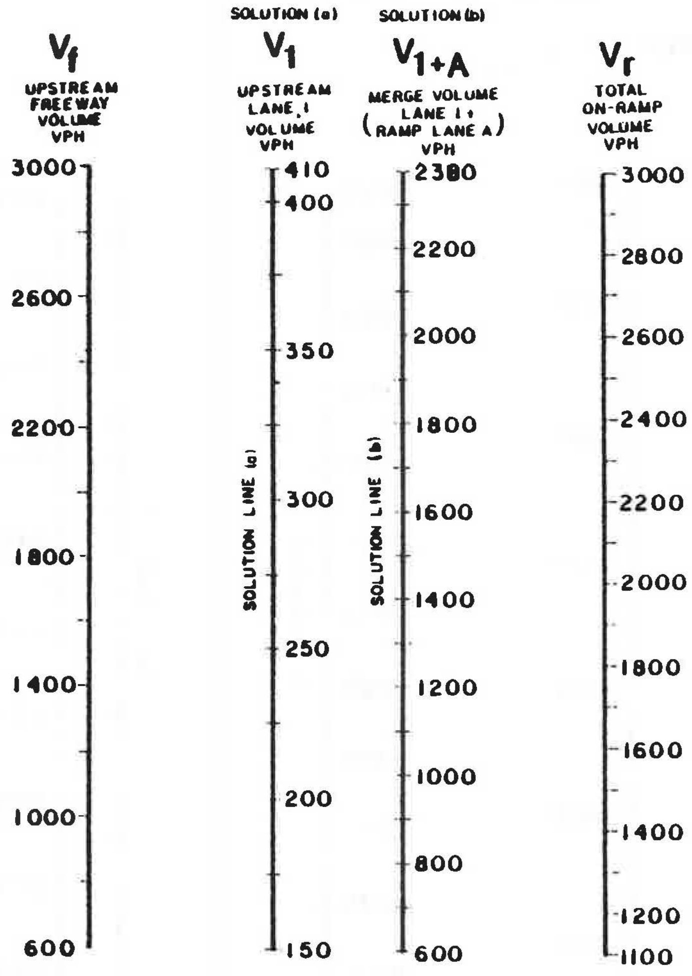
STEPS IN SOLUTION:

1) Draw a line from V_f value to V_r value; intersecting turning line.

2) Draw line from intersection of step 1 with turning line 1 to V_d value; read result on solution line.

FIGURE A4.10

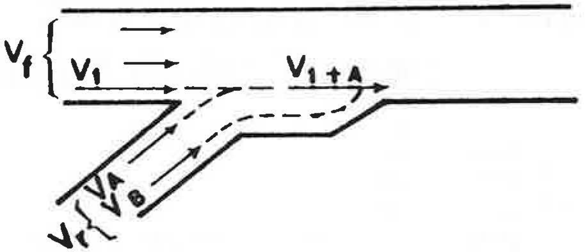
DETERMINATION OF LANE 1 VOLUME UPSTREAM OF ON-RAMPS ON 8-LANE FREEWAYS (4 lanes in each direction) WITH ADJACENT DOWNSTREAM OFF-RAMPS



EQUATION: (a) $V_1 = 54 + 0.070V_f + 0.049V_r$

(b) $V_{1+A} = -205 + 0.287V_f + 0.575V_r$

DIAGRAM:



CONDITIONS FOR USE:

1) Two-lane on-ramps on 6-lane freeways with acceleration lane of at least 800 feet in length.

2) Normal range of use: V_f = 600 to 3000 vph
V_r = 1100 to 3000 vph

STEPS IN SOLUTION:

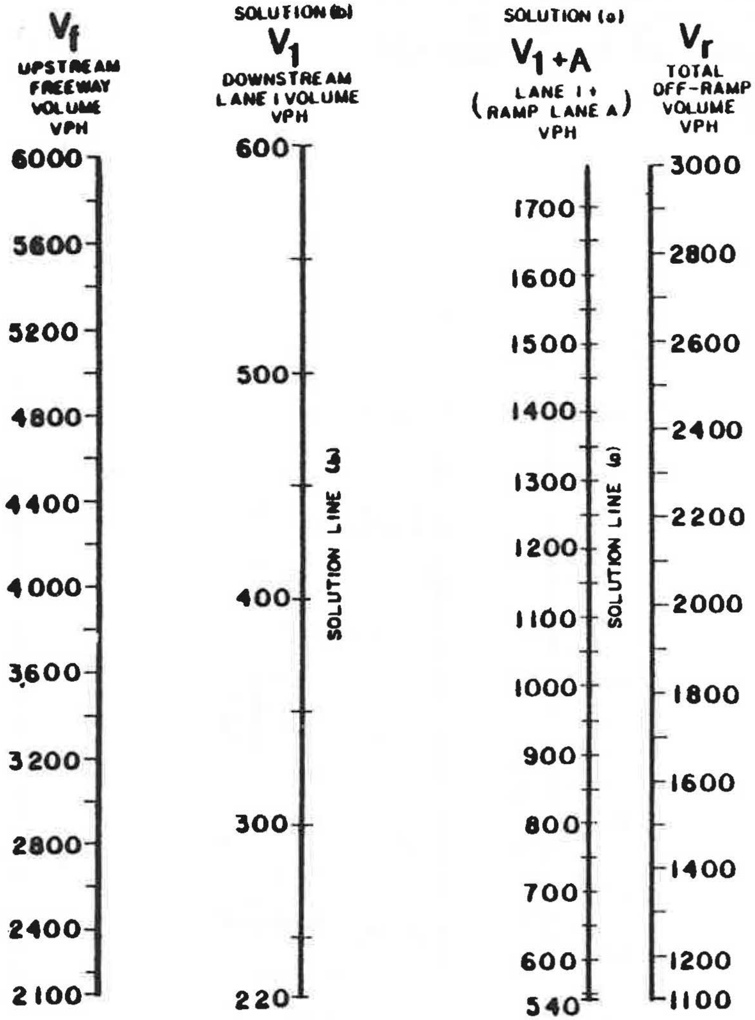
1) Draw line from V_f value to V_r value. Read V_1 on V_1 line, V_{1+A} on V_{1+A} line.

2) Compute $V_A = V_{1+A} - V_1$; $V_B = V_r - V_A$

3) Check Level of Service for two merge points: $V_{m1} = V_1 + V_A$ and $V_{m2} = V_{1+A} + V_B$

FIGURE A4.11

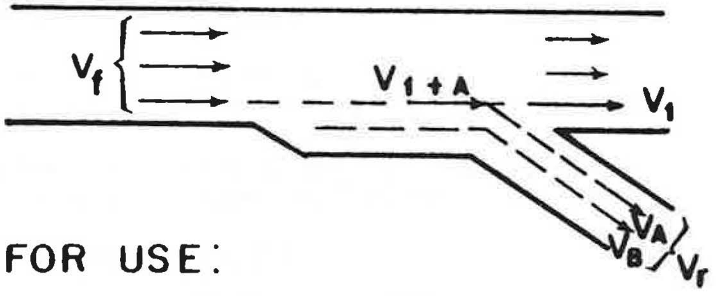
DETERMINATION OF LANE 1 VOLUME UPSTREAM OF TWO-LANE ON-RAMPS ON 6-LANE FREEWAYS (3 lanes in each direction)



EQUATION: (a) $V_{1+A} = -158 + 0.035 V_f + 0.567 V_r$

(b) $V_1 = 18 + 0.060 V_f + 0.072 V_r$

DIAGRAM:



CONDITIONS FOR USE:

1) Two-lane off-ramps on 6-lane freeways with deceleration lanes of at least 700 feet in length.

2) Normal range of use: V_f = 2100 to 6000 vph
V_r = 1100 to 6000 vph

STEPS IN SOLUTION:

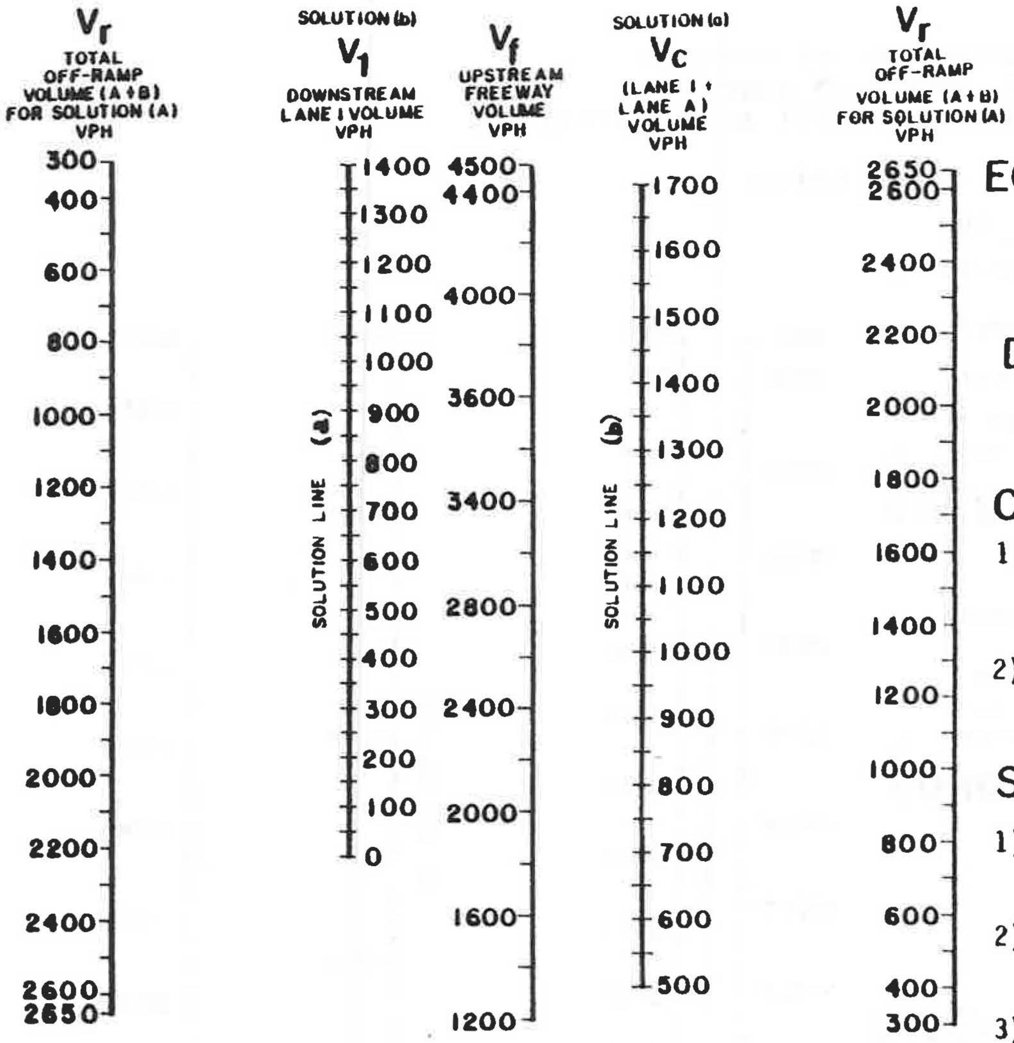
1) Draw line from V_f value to V_r value; read V_1 solution on V_1 line, V_{1+A} solution on V_{1+A} line.

2) Compute: $V_A = V_{1+A} - V_1$; $V_B = V_r - V_A$

3) Check Level of Service for two diverge volumes: $V_{d1} = V_{1+A}$ and $V_{d2} = V_B$

FIGURE A4.12

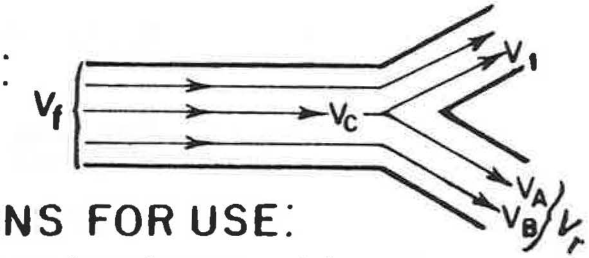
DETERMINATION OF LANE 1 VOLUME UPSTREAM OF TWO-LANE OFF-RAMPS ON 6-LANE FREEWAYS
(3 lanes in each direction)



EQUATION: (a) $V_C = 64 + 0.285V_f + 0.141V_r$

(b) $V_1 = 173 + 0.295V_f - 0.320V_r$

DIAGRAM:



CONDITIONS FOR USE:

1) Major diverge junctions on a 6-lane freeway, with three (3) lanes dividing to two 2-lane roadways

2) Normal range of use: V_f = 1200 to 4500 vph
V_r = 300 to 2650 vph

STEPS IN SOLUTION:

1) Draw line from V_f value to V_r value on the far right-hand scale; read V_c on solution (a) line.

2) Draw a line from V_f value to V_r value on the far left-hand scale; read V_1 on solution (b) line.

3) Compute $V_A = V_c - V_1$ and $V_B = V_r - V_A$

4) Check Level of Service for two diverge volumes: $V_{d1} = V_c$; $V_{d2} = V_B$

FIGURE A4.13

DETERMINATION OF CRITICAL LANE VOLUMES AT A MAJOR FORK OF A 6-LANE FREEWAY (3 lanes in each direction) WHICH DIVIDES INTO TWO 4-LANE FREEWAYS (2 lanes in each dir.)

CHAPTER V - THE FREEWAY AS A TOTAL FACILITY

Chapters II, III, and IV of these procedures have treated in detail the design and analysis of basic freeway segments, weaving areas, and ramp terminals respectively. This Chapter addresses how these elements are to be combined into a complete freeway design or analysis, and a number of special features which may be present and have a significant impact on operations.

Combined Analysis of Freeway Segments

A. Design

When approaching the design use of procedures herein, it is necessary to clearly indicate the kinds of information which would generally be available, and what results are desired. Capacity analysis is only one of several inputs into the design process. Others include geometric standards, safety standards, standards for signing, etc.

These procedures are utilized primarily in the design of cross-sectional elements (number of lanes, lane widths, shoulders, etc.) and in the selection of lane configurations for individual freeway elements. In general, the following information would be available as inputs to the design problem:

- horizontal and vertical alignment
- approximate location of ramps and interchanges
- forecasted demand volumes
- forecasted demand characteristics, such as the percentages of trucks, buses, and recreational vehicles in the traffic stream, PHF, etc.

The principal problem in coordinating the design of an overall freeway facility is the segmenting of the freeway into component parts for individual consideration via the methods of Chapters II, III, and IV. In general, the following guidelines may be utilized:

1) Each section of freeway between ramps or major junction points should be considered to be a separate "basic freeway segment."

2) Within these basic freeway segments, any grade of more than 1/4-mile (if grade $\geq$ 3%) or 1/2-mile (if grade < 3%) must be considered as a separate "basic freeway segment." Any sharp change in terrain, such as from level to rolling terrain would also necessitate the division of a single segment into separate subsegments. Long basic segments with no single grade of significance may be considered as extended segments of level, rolling, or mountainous terrain, as defined in Chapter II. Downgrade segments would normally be considered to be "level terrain" unless local data allows more specific treatment (see Chapter II).

3) Each ramp junction should be considered separately, in combination with the adjacent downstream ramp, and in conjunction with the adjacent upstream ramp. Ramps which are clearly part of a weaving section would not be analyzed using ramp procedures, but would be treated in step 4 below.

4) Potential weaving and multiple weaving areas should be investigated as such. "Potential" is used in that some segments may turn out to be either weaving areas or ramp conbinations depending upon the final configuration adopted.

In application, these guidelines lead to fairly straightforward computations in the following sequence:

1) Establish design Level of Service, demand volume and traffic characteristics, horizontal and vertical alignment, and approximate ramp locations.

2) Determine the basic number of lanes required for each of the basic freeway segments identified as previously noted, using the procedures detailed in Chapter II. The basic number of lanes for each ramp may also be determined using "ramp proper" techniques described in Chapter IV.

3) The results of step 2 will suggest probable configurations for ramp junctions and potential weaving areas. Analyze each ramp junction from three points of view: 1) as an isolated ramp, 2) in combination with the adjacent downstream ramp, and 3) in combination with the adjacent upstream ramp using the procedures of Chapter IV. Usually, one or two of these views will be invalidated by those procedures, but in other cases, there will be more than one valid analysis. In such cases, the analysis indicating the poorest operations or Level of Service is taken as a final result.

4) Weaving areas should be analyzed using the procedures of Chapter III to determine likely operating conditions. Note that in design, the case of an on-ramp followed by an off-ramp must be looked at BOTH as a potential weaving section with an auxiliary lane AND a ramp combination without.

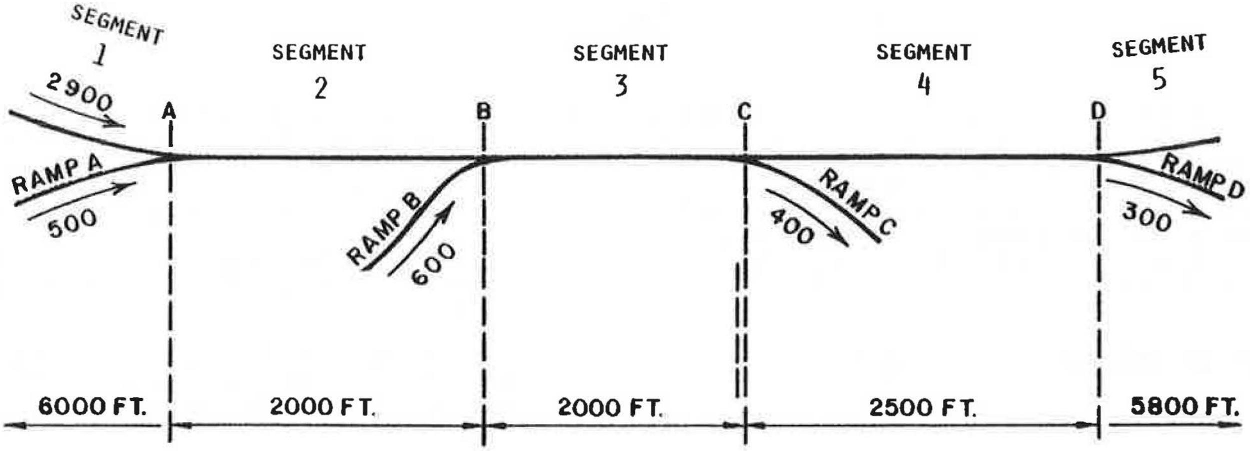
5) If the results of steps 3 and 4 are unsatisfactory, consideration may be given to:
   - altering the number and/or location of ramps (which may affect demand distribution)
   - changing the design of ramps and/or mainline segments determined in step 2 to create new configurations
   - changing the design of major interchanges to achieve different configurations, reduce weaving, etc.

Steps 2-4 are then repeated.

EXAMPLE

The simple design problem indicated in Figure 5.1 illustrates the above procedure. Note that the given demand volumes are already expressed as peak rates of flow in pcph.

STEP 1: Establish Demand, Alignment, Ramp Location
These are indicated in Figure 5.1.



FLOW RATES ON MAINLINE SEGMENTS:

segment	volume
1.	2900 PCPH
2.	2900 + 500 = 3400 PCPH
3.	3400 + 600 = 4000 PCPH
4.	4000 - 400 = 3600 PCPH
5.	3600 - 300 = 3300 PCPH

ROLLING
TERRAIN

DESIGN OBJECTIVE:
LEVEL OF SERVICE B

FIGURE 5.1

SAMPLE DESIGN PROBLEM

NOTE: 1 FT. = 0.3048 M.

STEP 2: Determine Basic Number of Lanes for Open Freeway Segments and Ramps The demand on each open freeway segment is shown in Figure 5.1. Using Table 2.1 criteria directly for Level of Service B, the number of lanes in each may be found. Note that 12-ft. (3.7 m.) lanes, adequate lateral clearance and 70 mph (112 kph) AHS are are to be provided as the result of design decisions.

Segment	Flow Rate	No. of Lanes Req'd
1	2900	3
2	3400	3
3	4000	4
4	3600	3
5	3300	3

Table 4.6 may be used to estimate the number of lanes required for each of the ramps. It will be assumed that all ramps will be designed for a minimum of 40 mph design speed. Using this criteria, all of the ramps of Figure 5.1 are single-lane ramps.

Based upon these results, the configuration illustrated in Figure 5.2 is most likely to be appropriate. Note that in this configuration, as there is an auxiliary lane between Ramps B and C, this is a weaving area. Segments 2, 3 and 4 together comprise a multiple weaving area.

STEP 3: Analyze Ramp Junctions Given that ramps B and C are definitely part of a weaving section for the trial design of Figure 5.2, the following ramp combinations remain to be analyzed using ramp procedures:

- Ramp A, Isolated
- Ramp D, Isolated

Ramp A and D could conceivably be considered both as isolated ramps with a simple weaving section in segment 3, or as part of a multiple weaving configuration with segment 3. Both cases would be analyzed.

Ramp A, Isolated From Table 4.2, Figure A4.6 is used. As the ramp is taken to be isolated, V_U is set as 50 (Note 2, Figure A4.6) and 640 V_d/D_d as 5 (Note 3, Figure A4.6).

V_1 = 600 (Figure A4.6)

V_m = 600 + 500 = 1100 Level of Service B (Table 4.1)

Ramp D, Isolated From Table 4.2, Figure A4.7 is used. Thus, for:

V_r = 300, V_f = 3600, 215 V_u/D_u = 2 ("Conditions for Use," note 2):

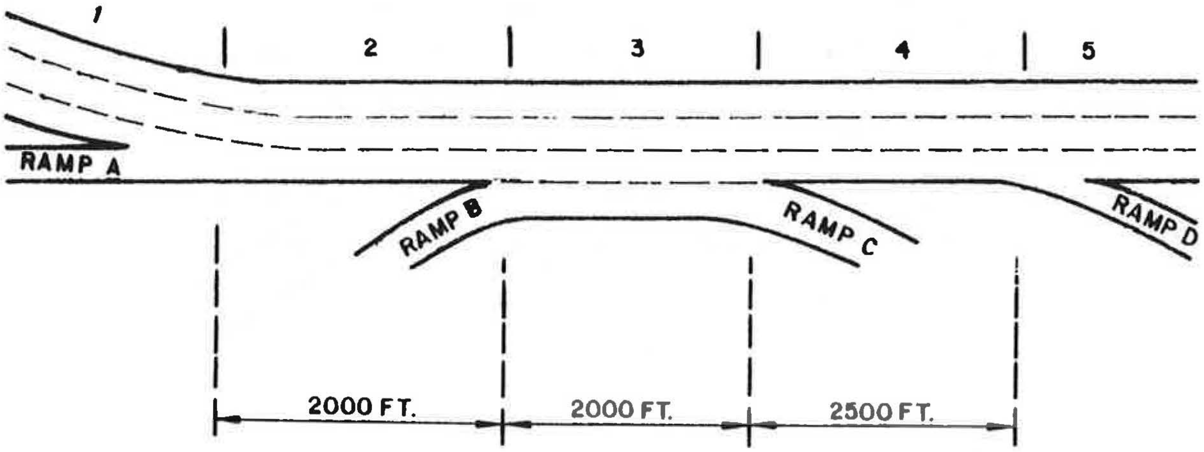


FIGURE 5.2

A LIKELY DESIGN FOR SAMPLE PROBLEM

NOTE: 1 FT. = 0.3048 M.

V_1 = 1050 (Figure A4.7)

V_d = 1050 Level of Service B (Table 4.1)

Ramps B and C should not be considered as a part of a ramp configuration, as the trial design of Figure 5.2 shows them to be in a weaving configuration, analyzed as such in step 4.

STEP 4: Analyze Potential Weaving Areas Segments 2 and 3 should be considered as a multiple weave. For the purposes of this analysis, all off-ramp vehicles at C will be assumed to originate from the freeway mainline, a worst-case assumption. Figure 5.3 depicts the resulting flows and weaving diagrams.

Segment 2 As one of the Segment 2 weaving movements is made with no lane change and another with one lane change, this is a Type II section. For Segment 2:

VR = 900/3400 = 0.26

R = 400/900 = 0.44

A trial speed of 50 mph will be assumed to begin computations.

- S_W = 47 mph (Figure 3.5)
- N_W(max.) = 1.9 lanes (Figure 3.6)
- N_W/N = 0.33 (Figure 3.7)

  N_W = 0.33 X 3 = 0.99 lanes (UNCONSTRAINED)
- N_{NW} = 3 - 0.99 = 2.01 lanes

  S_{NW} = 48 mph (Figure 3.8)

To obtain a more exact agreement between the assumed and computed values of S_{NW}, and a second trial with S_{NW} = 47 mph will be tried:

1) S_W = 45 mph (Figure 3.5)

2) N_W(max.) = 1.9 lanes (as before)

3) $N_{W/N}$ = 0.35 (Figure 3.7)

N_W = 0.35 x 3 = 1.05 lanes

4) N_{NW} = 3 - 1.05 = 1.95 lanes

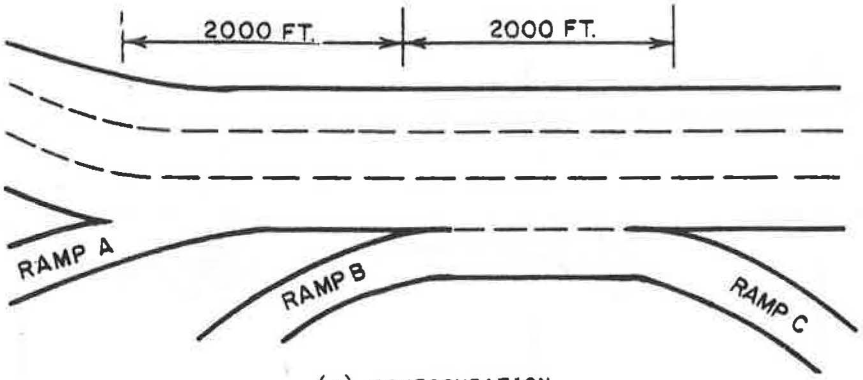
S_{NW} = 46.5 mph (Figure 3.8)

Thus, S_{NW} would be expected to be 47 mph (Level of Service B, Table 3.1) and S_W would be 45 mph (Level of Service B, Table 3.1). This is acceptable for the desired design Level of Service.B.

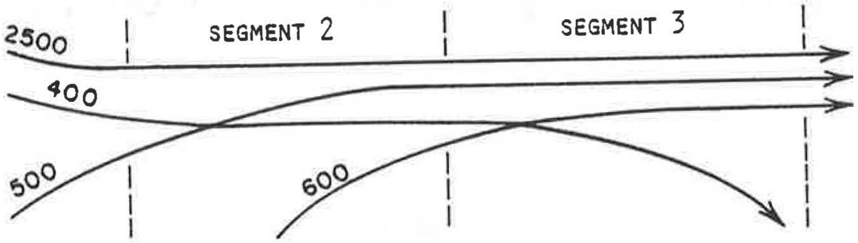
Segment 3 This should be considered as a ramp-weave, as it has an auxiliary lane, as shown in Figure 5.3, and all weaving vehicles make at least one lane change. Note that consideration of segment 3 of the multiple weave is the same as considering it as a simple weaving section.

For Segment 3:

VR = 1000/4000 = 0.25



(A) CONFIGURATION



(B) FLOWS

(C) WEAVING DIAGRAMS

FIGURE 5.3
CONSIDERATION OF MULTIPLE WEAVE

NOTE: 1 FT. = 0.3048 M.

$R = 400/1000 = 0.40$

Considering this alternative, assume $S_{NW} = 50$ mph

- $S_W = 53$ mph (Figure 3.5)
- $N_W(\text{max.}) = 2.0$
- $N_W/N = 0.35$ (Figure 3.7)

  $N_W = 0.35X4 = 1.40 < 2.0$ (UNCONSTRAINED)
- $N_{NW} = 4 - 1.40 = 2.60$

  $S_{NW} = 55$ mph (Figure 3.8)

A second trial will be made with S_{NW} assumed to be 57 mph.

- $S_W = 58$ mph (Figure 3.5)
- $N_W(\text{max.}) = 2.0$
- $N_W/N = 0.34$ (Figure 3.7)
- $N_W = 4\ (0.34) = 1.36$ lanes
- $N_{NW} = 4 - 1.36 = 2.64$ lanes

  $S_{NW} = 56$ mph (Figure 3.8) OK

Thus, S_{NW} would be expected to be 57 mph (Level of Service A, Table 3.1) and S_W would be 58 mph (Level of Service A, Table

3.1). This is acceptable for the desired design Level of Service.

Obviously, in this option, excellent operating conditions will exist (since 4 lanes are provided for a total of 4000 pcph). Level of Service A has been shown to prevail.

The ramp-weave section will provide service which is better than on adjacent segments. As slightly better operations in the ramp-weave section will exist, this will allow for some growth in weaving traffic.

Segments 3 and 4 should now be looked at as a multiple weaving area, as is illustrated in Figure 5.4. Again, it will be assumed that no on-ramp vehicles at B leave the freeway at C or D (a worst-case assumption).

Segment 3, in this case, remains the same as previously, so no additional analysis is required.

Segment 4, however, should be analyzed as a Type II weaving section, as one weaving movement is made with no lane change, and the other requires only one lane change. For Segment 4:

$VR = 900/3600 = 0.25$

$R = 300/900 = 0.33$

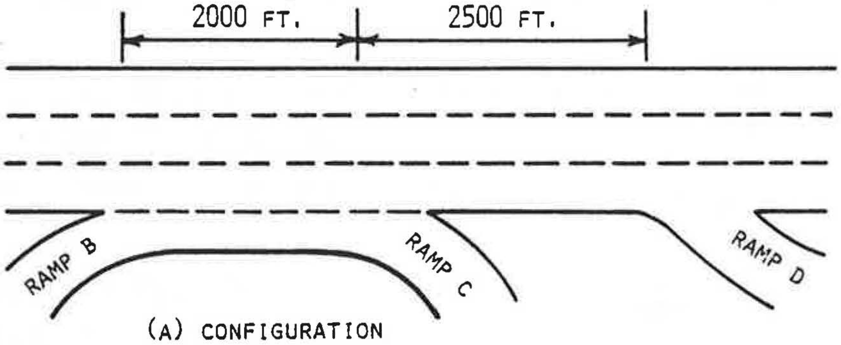
A first trial will be made at $S_{NW} = 50$ mph.

1) $S_W = 47$ mph (Figure 3.5)

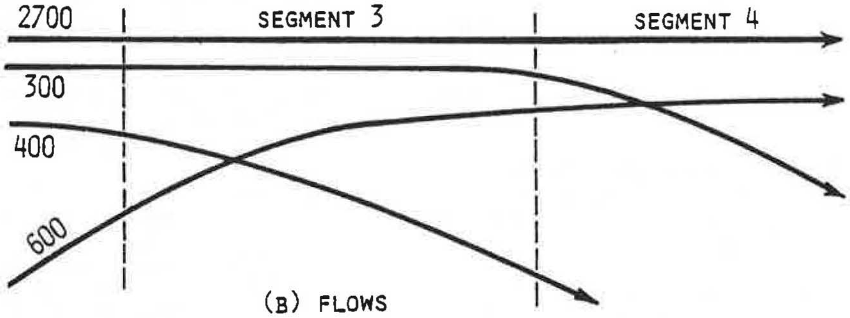
2) $N_W(\text{max.}) = 1.7$ lanes (Figure 3.6)

3) $N_W/N = 0.30$ (Figure 3.7)
$N_W = 0.30 \times 3 = 0.90$ lanes

4) $N_{NW} = 3 - 0.90 = 2.10$ lanes
$S_{NW} = 48$ mph (Figure 3.8)



(A) CONFIGURATION



(B) FLOWS

(C) WEAVING DIAGRAMS

FIGURE 5.4

CONSIDERATION OF A MULTIPLE WEAVE

To obtain closer agreement between assumed and computed values of S_{NW}, a second trial will be made at S_{NW} = 47 mph.

1) S_W = 45 mph (Figure 3.5)

2) N_W(max.) = 1.7 lanes (as before)

3) N_W/N = 0.32 (Figure 3.7)
N_W = 0.32 x 3 = 0.96 lanes

4) N_{NW} = 3 - 0.96 = 2.04 lanes
S_{NW} = 46 mph (Figure 3.8) OK

Thus, S_{NW} would be expected to be 47 mph and S_W 45 mph for the conditions proposed. This results in Level of Service B operation for both weaving and non-weaving vehicles (Table 3.1).

B. Analysis

The analysis approach for total freeway evaluation is quite similar to the design approach, but is simpler in that there are no alternates to consider. All volumes, goemetrics, and traffic conditions are known, and the freeway may be segmented with certain knowledge of ramp locations, weaving configurations, etc.

Once the freeway has been divided into uniform segments according to the guidelines previously noted, the following computational sequence may be followed:

1) Determine the Level of Service for each potential basic freeway segment using the procedures of Chapter II.

2) Determine the Level of service for each ramp junction, considering each ramp:

- as an isolated ramp
- in conjunction with the adjacent downstream ramp
- in conjunction with the adjacent upstream ramp

These checks are made using the procedures of Chapter IV. Ramps which are clearly part of a weaving configuration would not be examined using Chapter IV procedures.

3) Determine the Level of Service of each weaving and multiple weaving segment using the procedures of Chapter III.

Where a given segment falls under several of these analyses, the analysis resulting in the worst Level of Service is the controlling solution.

Once the analysis of segments is complete, the overall interpretation of results is subject to the exercise of judgment. As was discussed in Chapter II, the 1965 HCM (1) gives guidelines on the extent of influence of weaving areas and ramp junctions. Other research has yielded varying results which tend to indicate that the extent of influence of any individual element can range from as little as several hundred feet to more than a mile. As it is not possible to exactly determine the extent of such impacts, weaving and ramp junction areas which operate at Levels of Service poorer than adjacent segments should be viewed with caution.

A graphic technique presented in the 1965 HCM is useful as a tool to get a pictorial overview of overall operations. The technique assumes standard areas of influence as follows:

on-ramps - 500 ft. upstream, 2500 ft. downstream

off-ramps - 2500 ft. upstream, 500 ft. downstream

weaving areas - 500 ft. upstream of on-ramp and 500 ft. downstream of off-ramp.

Figure 5.5 illustrates the technique, with which levels of service are plotted for each segment. The illustration shown clearly indicates that the "bottle-neck" or limiting segment is the weaving area of segment 4. As long as the indicated operations hold, segment 4 will operate poorly, at Level of Service E, while other segments could operate at Levels B and C, if not prevented from doing so by spillback from segment 4. As noted previously, the exact extent of this spillback cannot be determined with certainty. What can be said is that segment 4 should not have an extended effect as long as it does not break down, i.e., slip to Level of Service F.

If more flow is added, segment 4 would be the first to break down -- and segment 4 is the most susceptable to breakdowns caused by incidents, weather or other extraneous factors. Once breakdown occurs here, the spatial and time extent of the breakdown can be estimated using techniques detailed in the next section.

C. Analysis of Breakdown Conditions

The two basic numbers supplied by the 1965 HCM for capacity under ideal conditions for uninterrupted and interrupted flow are critical to the understanding of the general response of freeways to breakdowns:

- under ideal conditions, the maximum flow rate which can be accommodated under uninterrupted flow conditions is 2000 pcphpl.
- once stopped, vehicles cannot depart from a standing queue at a rate greater than 1500 pcphpl, under ideal conditions. It should be noted that there is some controversy over this limit, and some engineers accept a value as high as 1700 pcphpl.

When breakdown occurs on a freeway segment, and stop-and-go conditions are formed, in effect, a standing queue has been formed. Even though the stop point of the queue may move through the traffic stream in a wave-like motion, the queue is real, and capacity under ideal conditions drops from 2000 pcphpl to something in the range of 1500-1700 pcphpl.

Consider the case illustrated in Figure 5.6 a three-lane freeway segment operating under ideal conditions with a demand of 5500 pcph during a peak hour, 4500 pcph during the hour after the peak, and 3000 pcph thereafter. What will occur if an incident blocks one lane for 15 minutes at the beginning of the peak period?

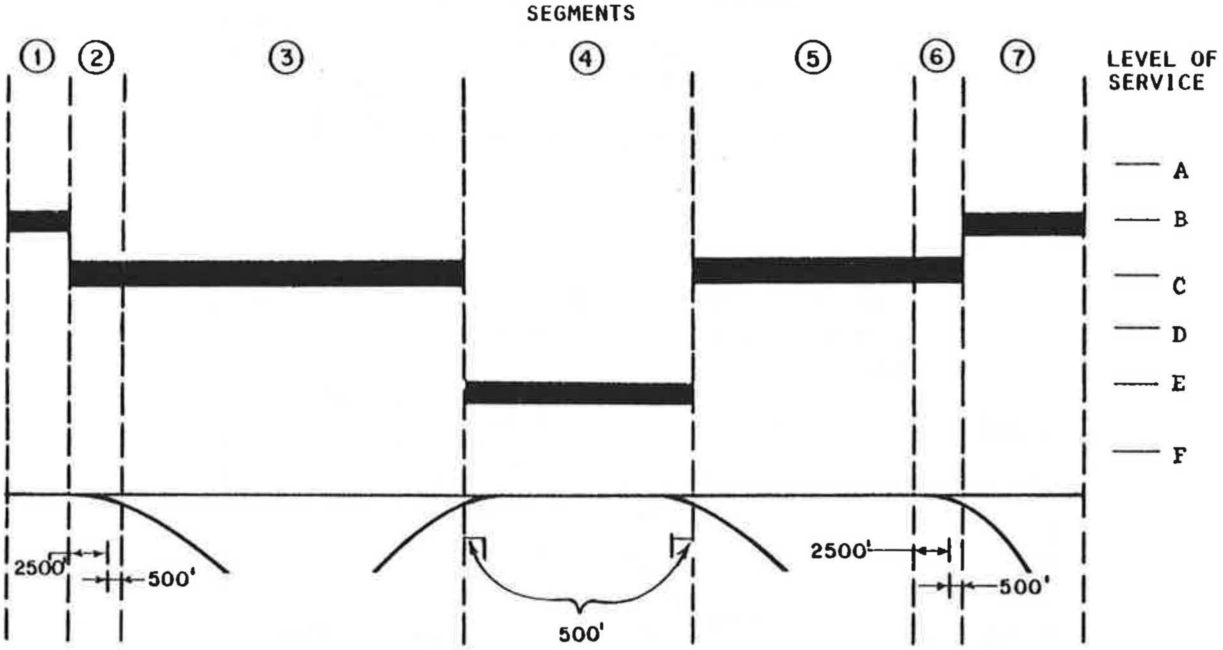


FIGURE 5.5

A GRAPHIC REPRESENTATION OF OVERALL LEVEL OF SERVICE

NOTE: 1 FT. = 0.3048 M.

The following operational effects should be anticipated:

1) When blockage occurs, capacity immediately drops from 6000 pcph to 4000 pcph or lower, which quickly creates stop-and-go queues, due to the 5500 pcph demand. This further deteriorates capacity to 3000 pcph (assuming a drop to 1500 pcphpl with 2 lanes open. Thus, during the first 15 minutes, 5500/4 = 1375 pc arrive and only 3000/4 = 750 pc are processed, and a queue of 615 pc is formed behind the blockage.

2) After the blockage is removed, capacity improves to 1500 X 3 = 4500 pcph, as standing queues still exist. Full capacity cannot be regained until all queues are dissipated. Thus, in the ensuing 45 minutes, 5500 X 3/4 or 4125 pc arrive and 4500 X 3/4 or 3375 pc are processed. The queue continues to build to 625 + 750 = 1375 pc.

3) During the second hour, 4500 pc arrive, and exactly 4500 pc are processed. The queue is stable, but does not dissipate.

4) Thereafter, the queue will dissipate, as 3000 pcph arrive, and 4500 pcph may be processed. The 1375 queued vehicles dissipate in 1375/(4500-3000)=0.92 hours, and full capacity is restored, some 2.92 hours after the occurrence of a 15-minute blockage. The queue length (assuming 3 lanes and 25 feet per vehicle) reached (1375/3) X 25 = 11,458 feet, or more than 2 miles at its peak, which lasted for one full hour.

Figure 5.6 illustrates this graphically.

This technique is approximate, and does not account for many microscopic properties of unstable freeway flows. It is, however, useful in estimating the overall effect of a breakdown in one location on overall operations.

Freeway Surveillance and Control

A complete treatment of this subject is beyond the current scope, of this report, but there are excellent references on the subject. The interested reader is referred to a state-of-the-art report by FHWA (2) and to the results of NCHRP Project 3-22A, which is concurrent with the present efforts (3).

A. Background

It is important to recognize that freeway surveillance and control is employed relatively commonly, and that it has a number of potential advantages. Some of the key potential advantages are:

- relief of congestion, by virtue of exercising control over excessive entries;
- decrease in delay, for the same reasons;
- protection of Level of Service;
- response to freeway incidents.

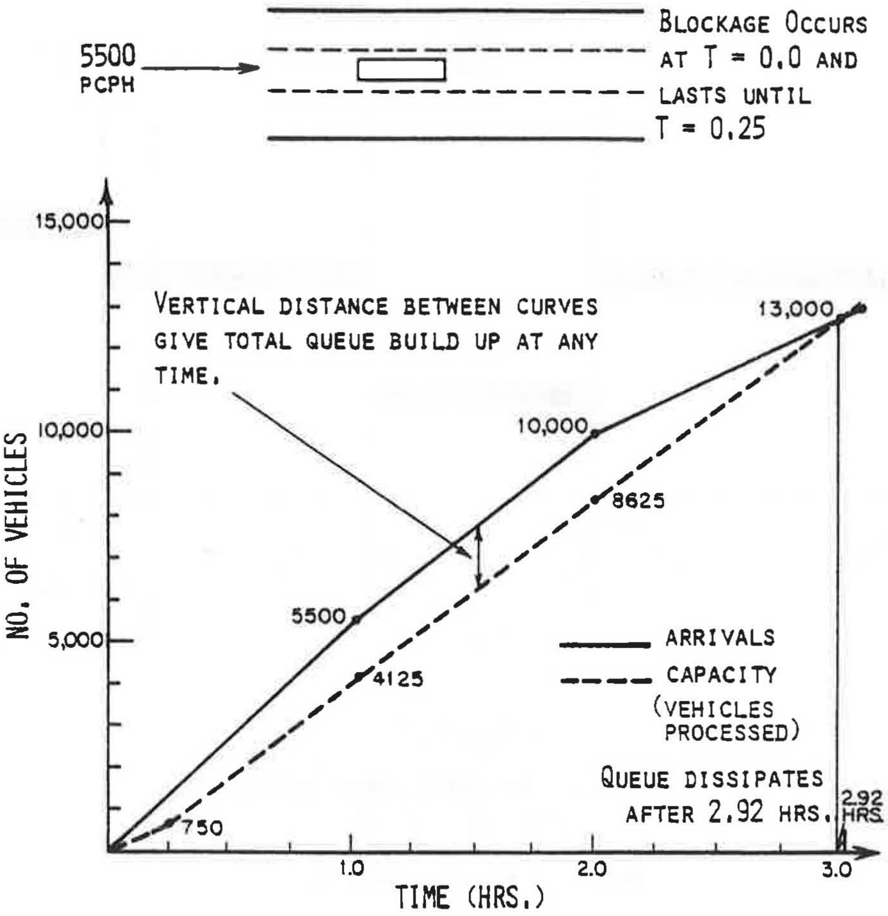


FIGURE 5.6

EFFECTS OF BREAKDOWN ILLUSTRATED

There is an interesting distinction between the first and third items: a freeway can be controlled with a single objective - to avoid breakdown (by restricting entries at appropriate locations), or it can be controlled so that some specified Level of Service is maintained. In the latter case, one may specify ramp metering rates in anticipation of future growth in demand. Thus, freeway management can be used at the planning stage, and not simply as an operational correction.

A freeway management system may be pre-planned, or it may be responsive to traffic variations. Further, it may or may not have explicit response to incidents.

B. Control Elements

The principal elements which are added to the facility because of a surveillance and control/management effort are:

- Vehicular Detectors;
- Ramp Metering;
- Video and/or Other Observation;
- Control Policies, Implemented by Central Computer or Other Hardware;
- Static and perhaps Variable Message Signing, to inform motorists of alternate routes and/or conditions.

At the time of this writing, radio communication with the motorists via CB or other means is also being actively considered.

Of these elements, the ramp metering is the most essential, for it is the most positive control action exercised. Chapter IV has addressed the lack of detailed knowledge on Lane 1 effects of metering, but its known advantages in control are in "smoothing out" disruptive arrival platoons. It is useful to consider an illustration of the ramp and mainline effects of a metered ramp, in order to make that discussion meaningful.

Consider the situation of Figure 5.7: an on-ramp has the demand depicted ranging from 250 to 575 vph (flow rate); the mainline has 3500 vph already on it, with a capacity of 4000 vph. Clearly, if the ramp demand is allowed to enter, a Level of Service F situation will occur upstream of the ramp. How may the ramp be metered to avoid this? What delay and queue will occur at the ramp because of this.

To not exceed capacity on the mainline, the ramp must be metered at 500 vph. This means 1

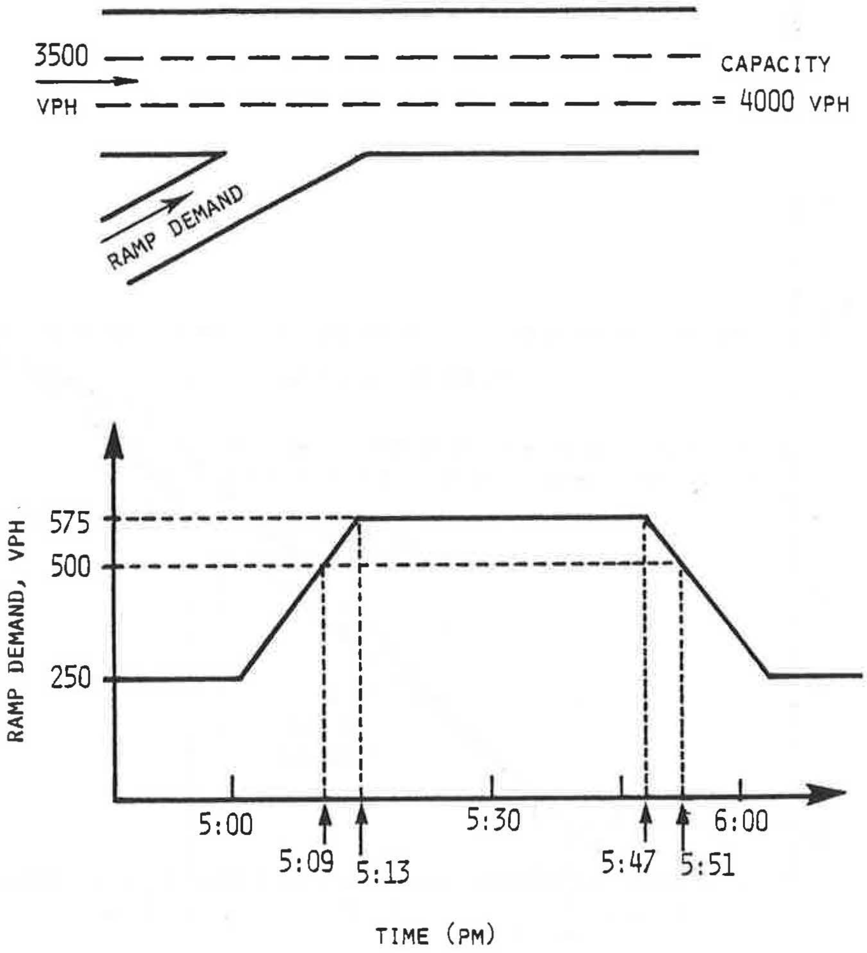


FIGURE 5.7

AN ILLUSTRATION OF A RAMP-METERING NEED

vehicle every (3600/500) = 7.2 seconds. With a green-red signal at the ramp, this would usually mean 2 seconds of green followed by 5.2 seconds of red. This may be implemented in a number of ways, including a conventional electromechanical controller, another local controller (possible a microprocessor), or a command from a remote computer.

From Figure 5.7, ramp demand reaches the 500 vph level at approximately 5:09 PM, and does not decrease below that level again until 5:51 PM. In the interim, a queue will form and continue to enlarge, as illustrated in Figure 5.8.

Figure 5.8 is a plot of ramp vehicles vs. time. At any given time, the horizontal distance between the demand and vehicles serviced curves is the delay/vehicle, and the vertical distance between the curves is the queue length. From Figure 5.8, the maximum delay/vehicle would occur at 5:51 PM, and would be approximately 5 minutes. The queue length at this time would be about 50 vehicles.

It should be noted here, however, that many drivers will be unwilling to accept 6 minute delays. (In Los Angeles, 1-2 minute delays are the maximum usually observed.) Many of the queued vehicles might be expected to seek alternate routings to avoid the delay. Thus, a critical consideration on ramp metering is the availability of alternate routes, and the impact of diverted traffic on them.

It should also be noted that some freeway management systems operate on nothing more than application of the above principle in a consecutive set of freeway segments: the section input is monitored; the segment capacity is known; the ramp input is not allowed to cause mainline flow to exceed capacity.

Some of the cities currently using some form of freeway surveillance are: Chicago, San Antonio, Milwaukee, Houston, Minneapolis, Fort Worth, Toronto, San Jose, Dallas, San Francisco, Detroit, and Los Angeles.

C. Determination of Problems and Control

Freeway management is more frequently motivated by operational problems: one or more sections are

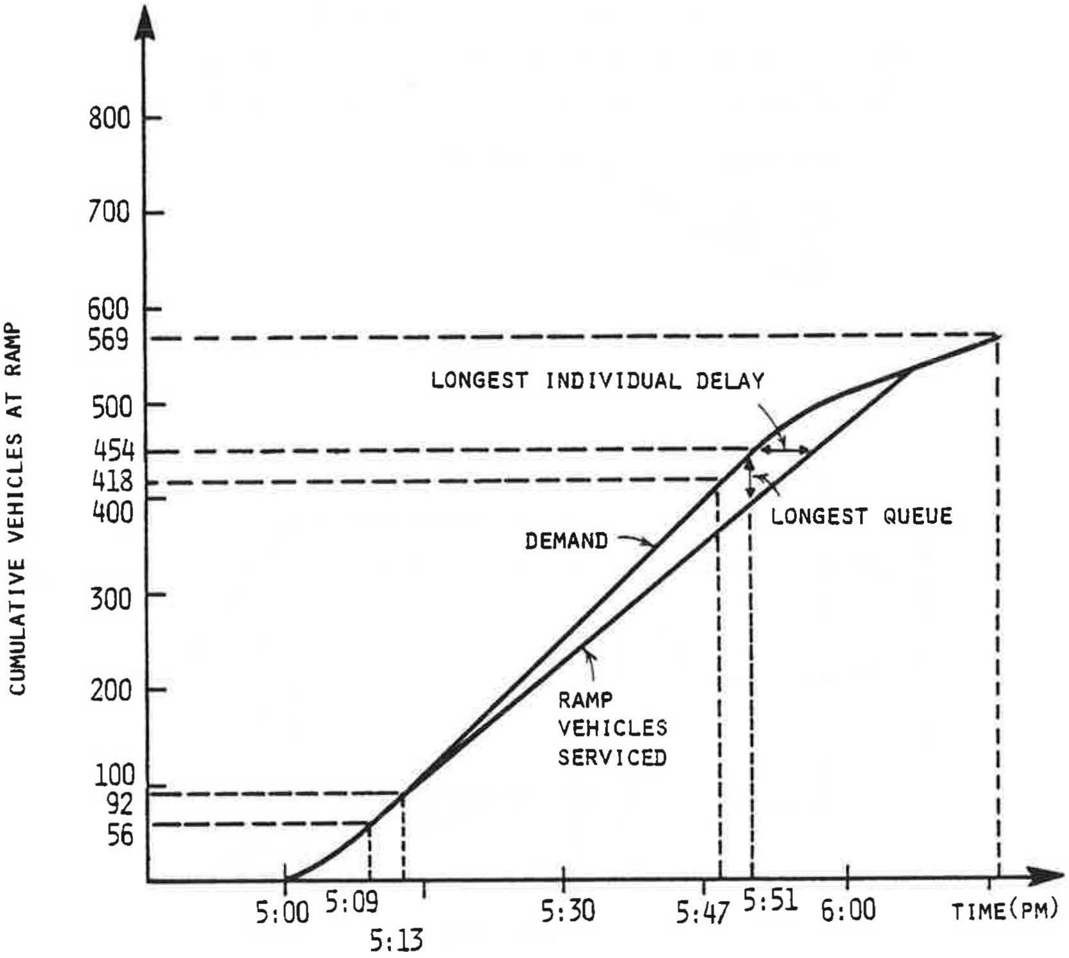


FIGURE 5.8

A PLOT OF CUMULATIVE RAMP DEMAND AND OUTPUT

"bottlenecks," with significant mainline congestion occuring. The problem is then to alleviate the congestion and to maintain a Level of Service better than F. In some cases, the project includes construction at some locations to provide additional capacity, or includes the incorporation of high occupancy vehicle lanes.

Although an entire treatment of freeway management is not appropriate in the present context, two problem areas deserve special mention: hidden bottlenecks, and origin-destination patterns.

Figure 5.9 depicts a hypothetical freeway with five sections, and with the input demands shown. Clearly, demand will exceed capacity in Section 3, and Level of Service F will result. Stop-and-go operation can occur in all upstream sections, depending upon the duration over which demand

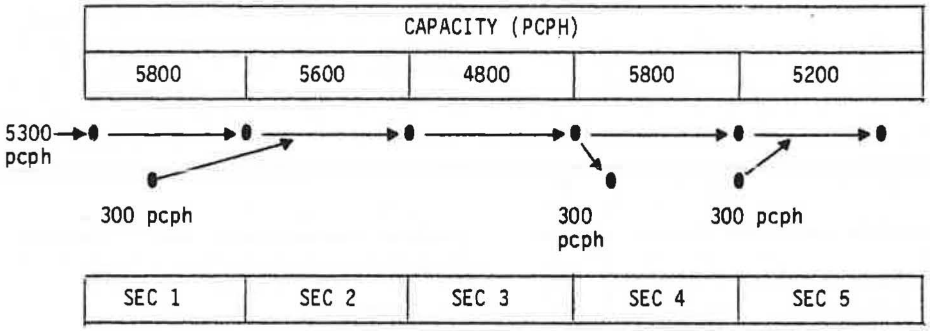


FIGURE 5.9

POTENTIAL FOR HIDDEN BOTTLENECKS

exceeds capacity (i.e., over which the congestion has a chance to spread).

In practice, the capacities are not computed and one simply observes severe congestion in Section 2, caused by Section 3.
The congestion may spread to Section 1 if the peak period is long enough or Section 2 is short.

Assume that some physical reconstruction, perhaps coupled with decreasing the 5300 PCPH input via ramp metering further upstream, alleviates the problem. Lacking the capacity figures for all sections, one may overlook the fact that if Section 3 now outputs a flow rate higher than 5200 PCPH, a bottleneck will appear at Section 5 for the first time. It was always there, but only the solution of the Section 3 problem allowed the demand to attain levels necessary to exhibit it. That is, it was "hidden" by the upstream bottleneck in Section 3.

Certainly, one must conduct a complete capacity analysis of the facility to avoid the "hidden bottleneck" problem. In doing so, one must anticipate changes in flow due to the improvements. For instance, is the off-ramp in Section 4 shown at a level of 300 PCPH because it is the true demand or because it is the observed amount which could get past the original bottleneck? In addition, one must recognize that the service volumes in some sections (e.g., weaving sections) are functions of the traffic mix, which may change.

Because the flow pattern may be distorted, it is important to have some knowledge of the origin-destination of the traffic. Further, the origin-destination pattern influences what can be done and what should be done. Consider a freeway on which virtually all the outlying ramp entries stay on the facility until it terminates in the downtown area. Consider an identical physical facility, but with traffic using it for many short trips, with much outlying traffic exiting before another "layer" of traffic enters. The control opportunities and the equity of various control options vary radically between these two extremes.

D. Incidents

Incidents occur relatively commonly on traffic facilities, although it is standard practice to design to a Level of Service for the non-incident condition. Clearly, incidents require attention because they

- disrupt the level of service being provided;
- reduce the capacity radically;
- present hazards to the motorists, particularly those directly involved.

Certainly incident response is desired in order to provide assistance to the motorists involved -- tow, medical, police as the need arises. Incident response can also be directed to minimizing the impact on other vehicles and to recovering use of the facility.

One study (4) showed that an incident removed to the shoulder on a three lane facility still reduced capacity by one-third; a single-lane blockage reduced capacity by 50%; a two-lane blockage reduced capacity by 79%. In addition to the magnitude of the impact, the duration must also be considered. Refer to Figure 5.10, which identifies four critical phases of an incident history. Analogous to the ramp metering illustration in Section B above, the effect can persist long after the incident itself is removed, due to the backups created. One facility (5) estimated that peak period incidents were responsible for more delay than recurrent peak period congestion at the location in question.

Incidents may be detected by video observation, audio reports (call-boxes, CB), or roadway sensors. There is some recent FHWA work on the last subject.

Incidents may be responded to by some combination of required assistance, ramp restrictions or closure, and alternative route advisories. The control actions may be preplanned or dynamic decisions.

Table 5.1 shows a compilation of data from several sources on the impact of incidents on traffic capacity. Incident effects are combined in the table with the impacts of various types of maintenance procedures which normally result in temporary lane closures.

Weather

Freeway capacity is affected by weather. The most extreme case is represented by heavy snowfalls which cause multiple lane closings. However, a variety of weather conditions -- rain, snow, fog, glare, and others -- affect capacity without such dramatic evidence of their existence.

Quantitative information is sparse, but some indications do exist: one study found that rain reduced capacity by 14-19% (6,7): another found a typical figure of 8% for rain (8) although much variation was observed. Indeed, the substantial variations due to the intensity of the weather condition and the specifics of the location are entirely rational. Most importantly, it must be recognized that 10-20% reductions are typical and higher percentages are quite possible. These effects must be considered in facility design, particularly when adverse conditions are common.

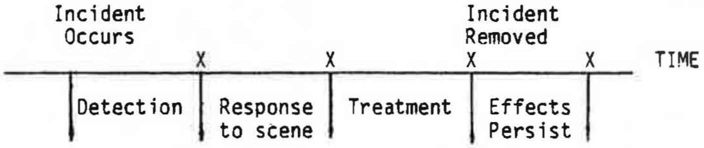


FIGURE 5.10

PHASES OF AN INCIDENT

TABLE 5.1

TYPICAL CAPACITY FLOW RATES

No. of lanes one direction (Normal Operation)	2	3 or 4	4
No. of lanes open one direction	1	2	3
Type of operation:*			
-- Median barrier or guardrail repair	1500 vph	3200 vph	4800 vph
-- Pavement repair, mudjacking, pavement grooving	1400 vph	3000 vph	4500 vph
-- Stripping, resurfacing, slide removal	1200 vph	2600 vph	4000 vph
-- Pavement markers	1100 vph	1400 vph	3600 vph
-- Middle lanes - any reason	----	2200 vph	2400 vph
Accidents**			
-- Incident occurring in moving traffic lane with one lane blocked	1300 vph	2700 vph	4300 vph
No. of lanes in one direction	2	3	4
Accidents**			
-- Incident occurring in shoulder with no lanes blocked	3000 vph	4600 vph	6300 vph

* From Forbes, C.E. et al. "Reducing Motorist Inconvenience Due to Maintenance Operation on High-Volume Freeways," HRB Special Report 116, 1971, pp. 181-188.

** From Goolsby, Merrill E. "Influence of Incidents on Freeway Quality of Service," HRR No. 349, 1971.

High-Occupancy Vehicle Lanes on Freeways (HOV Lanes)

The existence of exclusive high-occupancy vehicle lanes on freeways raises two issues: (1) what is their capacity, and what are the operating characteristics of such lanes, and (2) what effect does their presence have on the operation of the remainder of the freeway.

A. Capacity Analysis for HOV Lanes

This issue is addressed by Levinson (9) in his draft of a "Transit" chapter submitted to the Highway Capacity and Quality of Service Committee of the Transportation Research Board.

The issue is a complex one. High-occupancy vehicle lanes come in many forms, including:

- exclusive bus lanes
- exclusive bus/taxi lanes
- exclusive bus/car pool lanes, with varying occupancy restrictions
- exclusive bus/taxi/car pool lanes.

In addition, each type may be implemented as a contra-flow lane, with the exclusive lane taken from the opposing freeway lanes, or as a concurrent flow lane, in which the lane is taken from freeway lanes in the same direction of flow. These lanes are adopted to provide for smooth and speedy flow of passengers in vehicles using the lanes, and are used to circumvent freeway segments operating at or near breakdown conditions. The contrast of high-occupancy vehicles progressing smoothly while other vehicles are mired in heavy congestion is also intended to act as an inducement to motorists to abandon their car for a bus or car pool.

Thus, it is unthinkable for such a lane to operate at, or anywhere near, capacity or at a low Level of Service. To do so would defeat its function and purpose. The issue of the "capacity" of such lanes is, therefore highly speculative, as few (if any) existing lanes approach this condition at any time. If defining capacity of such lanes is a problem, the selection of criteria for defining Levels of Service is even more difficult. Average travel time might be one appropriate measure -- but should this be on a per vehicle or per person basis?

Since the goal of an HOV lane is to increase the person-capacity of a freeway without costly capital expenditures, strong consideration should be given to the development of capacity and Level of Service criteria in terms of persons rather than vehicles. The literature, while reporting many site-specific studies and accompanying data, has not yet approached this question in a comprehensive way. A comprehensive study would be required to address the many issues involved in the formulation of HOV lane Level of Service criteria in a serious fashion, one which would be well beyond the scope and resources of this current work.

The issues involved in defining capacity analysis techniques for high-occupancy vehicle lanes are, therefore, too complex to be resolved within these provcedures, and the available data base too variable. Levinson's work (9,10,11) provides a useful guide, and there are numerous studies of existing operations (12-22) which may be utilized for general insight on the subject. The portion of this circular devoted to transit capacity is the result of Levinson's work.

B. Effect of HOV Lanes on Freeway Operations

The existence of a HOV lane on a freeway influences the operation of remaining freeway lanes in three ways:

- a lane is removed from one direction of flow (occasionally two are removed, the second being used as a buffer lane)
- cones or other devices used to demark the lane (where used) pose lateral obstructions to flow in the adjacent lane, if a buffer lane is not provided
- the movement of vehicles into or out of the HOV lane may be disruptive to other traffic.

Unfortunately, there is no meaningful body of data which has quantified these effects. Estimates of the first two factors can, however, be made using techniques presented in Chapter II.

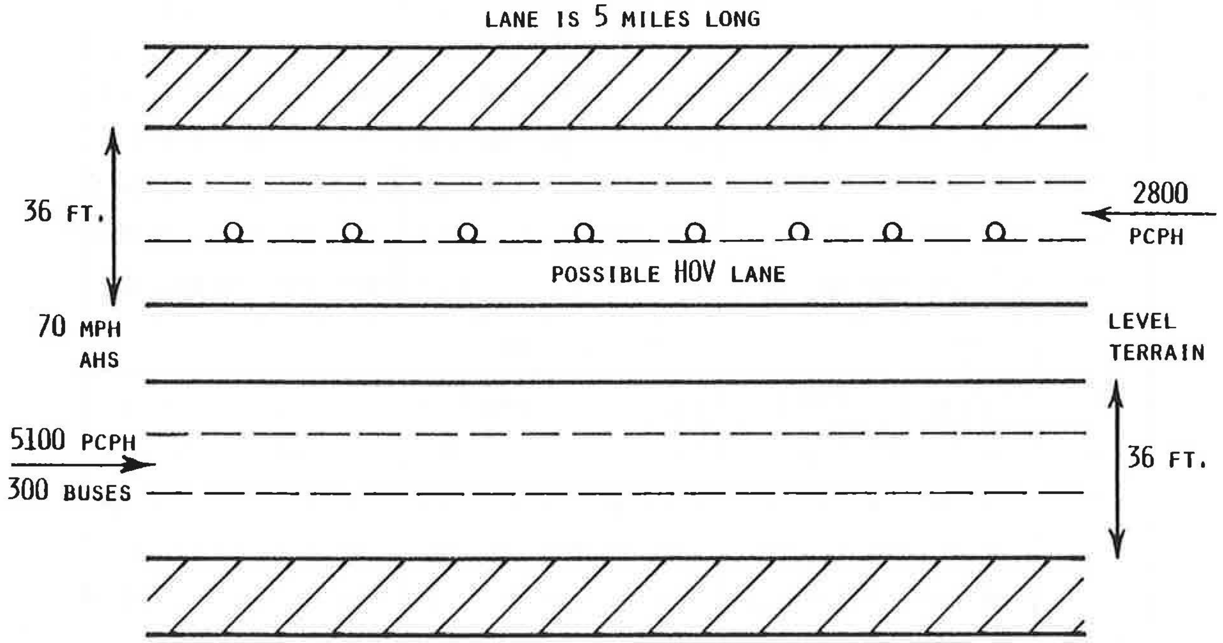
The removal of a lane is simply handled by assuming that the 8-lane freeway becomes a 6-lane freeway, and the 6-lane freeway a 4-lane, etc. The effect of cones or other dividers may be estimated by treating them as lateral obstructions at the roadside edge. Depending upon their placement, they may also have the effect of narrowing the lane as well.

In contra-flow lanes, this latter effect is marked, as vehicles shy away from the imposing opposite flow of large vehicles at relatively high speeds. In some instances, an entire adjacent lane is taken out of service to act as a buffer zone.

Figure 5.11 illustrates a problem using this estimating technique. The problem is to analyze the impact of a proposed contra-flow lane on Level of Service in the direction from which the lane is taken, and on the concurrent direction of flow.

Before HOV Lane is Initiated

Primary Flow =	5100 + (1.6 x 300) = 5580 pcph in 3 lanes of 12 ft. each, with no lateral obstructions. Level of Service D, approximate speed 43 mph (Table 2.1, interpolate for speed)
Contra-flow =	2800 pcph in 3 lanes of 12 ft., with no lateral obstructions. Level of Service B, speed 50 mph (Table 2.1)



NOTES: ALL VOLUMES ARE EXPRESSED AS PEAK FLOW RATES; ONE BUS = 1.6 PASSENGER CARS

FIGURE 5.11

EXAMPLE FOR ANALYSIS OF HOV LANE IMPACT

ONE BUS = 1.6 PASSENGER CARS

NOTE: 1 MI. = 1.6 KM.

After HOV Lane Is Initiated

Primary flow = 5100 pcph in 3 lanes of 12 ft. with no lateral obstructions. Level of Service C, approximate speed 48 mph (Table 2.1)

Contra-flow = 2800 pcph in 2 lanes with a lateral obstruction at 0' feet on one side - assume 11 foot lanes due to divider placement -- W = 0.87 (Table 2.3) -- effective flow = 2800/0.87 = 3218 pcph. Level of Service C, approximate speed 49 mph (Table 2.1, interpolate for speed)

On the basis of this approximate analysis, the creation of the new lane improves flow in the concurrent direction by removing buses from the stream. Level of Service improves from D to C, and average running speed increases from 43 mph to 48 mph, which saves each vehicle (5/43 - 5/48) 60 = 0.78 minutes.

Level of Service on the freeway in the reverse direction decreases from B to C, and speed from 50 mph to 49 mph, causing each vehicle to lose (5/49 - 5/50) 60 = 0.12 minutes.

While not totally definitive, this approximate technique is useful in evaluating the gross effects of HOV lane implementation on remaining freeway flows. These impacts would have to be evaluated in light of the benefits and costs of the HOV lane itself and related issues.

Some General Guidelines for Planners

During the initial stages of transportation system planning, it is often necessary for planners to consider alternate freeway facilities as part of their investigations. During this process, the planner will need some general information on freeway capacity and performance. Certainly all of the procedures detailed herein are valid, but they are often too detailed for use at this level, where ramp locations, specific configurations, etc. are not yet known or even contemplated.

TABLE 5.2

SERVICE VOLUMES FOR FREEWAYS WITH AHS = 70 MPH,
12-FOOT LANES AND ADEQUATE LATERAL CLEARANCE

Level	4-lane freeways (2 lanes in each dir.)					6-lane freeways (3 lanes in each dir.)					8-lane freeways (4 lanes in each dir.)				
of	Percent Trucks														
Service	0	5	10	15	20	0	5	10	15	20	0	5	10	15	20
	LEVEL TERRAIN														
1	1600	1520	1455	1390	1330	2400	2280	2185	2088	1990	3280	3115	2985	2855	2720
2	2500	2375	2275	2175	2075	3900	3705	3550	3390	3235	5400	5130	4915	4700	4480
3	3400	3230	3094	2960	2820	5100	4845	4640	4435	4235	6800	6460	6190	5915	5645
4	3850	3660	3500	3350	3195	5775	5485	5255	5025	4795	7700	7315	7000	6700	6390
5	4000	3880	3640	3480	3320	6000	5700	5460	5220	4980	8000	7600	7280	6960	6640
	ROLLING TERRAIN														
1	1600	1390	1230	1105	1010	2400	2088	1848	1655	1510	3280	2855	2525	2265	2065
2	2500	2175	1925	1725	1575	3900	3390	3005	2690	2460	5400	4700	4160	3725	3402
3	3400	2960	2620	2345	2142	5100	4435	3925	3520	3215	6800	5915	5235	4690	4285
4	3850	3350	2965	2655	2425	5775	5025	4445	3985	3640	7700	6700	5930	5315	4850
5	4000	3480	3080	2760	2520	6000	5220	3080	2760	2520	8000	6960	6160	5520	5040
	MOUNTAINOUS TERRAIN														
1	1600	1185	945	785	670	2400	1850	1415	1175	1010	3280	2425	1935	1605	1375
2	2500	1850	1475	1225	1050	3900	2885	2300	1910	1640	5400	3995	3185	2645	2270
3	3400	2515	2005	1665	1430	5100	3775	3010	2500	2142	6800	5030	4010	3330	2855
4	3850	2850	2270	1885	1615	5775	4275	3405	2830	2425	7700	5700	4545	3775	3235
5	4000	2960	2360	1960	1680	6000	4440	3540	2940	2520	8000	5920	4720	3920	3360

The planner must be able to quickly determine whether a 4-, 6-, 8-, or more-lane freeway is likely to be needed in a given system configuration, as well as whether a freeway facility is needed at all. The latter determination requires information on non-freeway facilities which are beyond the scope of this manual. Table 5.2 does, however, provide a set of easily used service volumes for freeways, assuming that ideal geometrics are provided, for a variety of traffic conditions.

While these guidelines are useful for initial sketch-planning, all freeway facilities resulting from such plans will have to be subjected to segment-by-segment detailed analysis using the methods presented in this manual.

CHAPTER V - REFERENCES

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A NEW TECHNIQUE FOR DESIGN AND ANALYSIS OF WEAVING SECTIONS ON FREEWAYS

INTRODUCTION

The technique for design and analysis of weaving sections on freeways described below was developed by Jack E. Leisch, President of Jack E. Leisch & Associates, Evanston, Illinois. This material has been adapted from an article in the March, 1979 issue of ITE Journal.

The procedure was developed independently by Mr. Leisch based upon the 1963 BPR data base (1), the procedures of the 1965 Highway Capacity Manual (HCM) (2), Mr. Leisch's involvement in NCHRP Project 3-15 (3), and numerous unpublished studies (4). The purpose of the ITE Journal article was to introduce an updated technique for use in the improvement of operational efficiency and safety in the rehabilitation of congested and outmoded freeways. The article presents the primary results of Mr. Leisch's considerable experience and analysis in a form to permit application of those areas of major concern.

The procedure for weaving sections as presented here is primarily for design purposes. Analysis cases must be done in reverse, and require engineering judgment when the two levels of service differ materially.

This technique is intended to be used with the freeway materials currently in the 1965 HCM, and not with the freeway treatment contained in the Freeway Section of this Circular. The speed-volume relation assumed herein is that of the 1965 HCM; the level of service definitions are those of the 1965 HCM (except that average speed-relations are used for weaving movements); and the truck factors used in the calibration are those of the 1965 HCM.

FORM OF PRESENTATION

The primary results are presented to the reader in the form of two composite graphs which permit the solution of the majority of weaving problems on freeways. The end product is embodied in the nomographs of Figures 1 and 2.

The weaving section configuration and identification were determined to have a pronounced effect upon operational characteristics. Designations of one-sided and two-sided weaving sections were specifically established; these were further subdivided into "lane-balanced" and "lane-imbalanced" sections and included within the basic graphs. Application of nomographs in the solution of problems is simple and direct as described further on. The terminology in the 1965 HCM has been maintained for the most part.

DEVELOPMENT OF METHOD

The development of this technique for design and analysis of weaving sections was approached on the basis of the following premise and objectives:

1. The procedure was to be simple and easy to use.

2. The format, to all feasible extent, was to follow the makeup and terminology applied in the 1965 HCM.

3. The revised procedure was to make appropriate and extensive use of both the 1963 BPR data base, supplementary data and development of the NCHRP Project 3-15 on "Weaving Area Operations Study."

4. Statistical analyses, independently performed, to reflect the data of both projects, were to play a major role in establishing the needed relationships.

5. Elements of analytical modeling and rational formulations were to be utilized to supplement and expand upon portions of findings determined statistically to provide a sufficiently broad spectrum for application.

The adherence to these objectives, and the confidence that the general framework of procedure in Chapter 7 of the 1965 HCM (despite some of its shortcomings) could provide a definitive avenue of investigation, gave impetus to the study. The 1963 BPR data base furnished the major basis of investigation, coupled with numerous elements of research derived in the NCHRP 3-15 Project. The latter provided valuable information and insights previously unavailable; this further aided the development of the revised or updated methodology reported herein. Direct experience of the author in design, construction, and operation of weaving sections has provided an additional dimension to the practical considerations of real conditions to be reflected in the results.

With this direction, the course of research and development followed a series of separate but interrelated analyses. These were made possible with a mechanism by which the data could be reduced to simulate operationally "balanced" sections. In addition, weaving section configurations must be identified as one of two basic forms: one-sided (ramp weaves, irrespective of numbers of ramp lanes); or two-sided (where entering and existing traffic from opposite sides weave across the freeway flow, producing disruptive tendencies within an otherwise state of maintained route continuity). These two basic forms are geometrically and operationally different, but when properly "balanced" and grouped into speed categories, good correlation was found by regression analysis of weaving volume-speed-length relationships. For each basic configuration a different set of curves evolved.

The analysis further verified that when weaving sections are operationally balanced, there is another ingredient within the volume-speed-length relationship which, as in the 1965 HCM, may be referred to as a measure of weaving intensity, or factor "k". For one-sided weaving sections the maximum value clustered at 3 and reduced downward toward 1 as length and speed increased for a given volume. Distribution of k values from the analysis took the form shown in the upper-left portion of Figure 1. For two-sided weaving sections, interestingly enough there was no correlation with speed, but there was a very definite relationship between values of k and R (ratio of the smaller weaving volume, W_2, to the total weaving volume, $W_1 + W_2$). The values of k also clustered around 3 but only when W_1 and W_2 were nearly equal, or when R was of the order of 0.5 or 0.4 For lesser ratios of R, the value of k increased rapidly to a maximum of 6 for relatively small W_2 movements, or

R values of 0.1 or less. The maximum of k = 6 can be rationally deduced. The k values of 3 to 6 all reduced necessarily to 1 as the length of section approached and passed into the "out of the realm of weaving" zone. Distribution of k values, with the R variable, was charted from the analysis as indicated in the upper-left portion of Figure 2.

The average running speeds of weaving traffic for the one-sided section, where ramp traffic weaves along one side of the freeway, were taken to be 5-mph below the average through speeds, without inhibiting the freeway level of service. This is a logical deduction from the data base, and in general agreement with the philosophy that at ramps the entering and exiting traffic is acceptable with slight momentary speed reduction.

Accepting this compatibility, the average running speeds of weaving traffic for the various freeway levels of service are shown within the lower-left portion of the nomograph, Figure 1. The average freeway speeds for the various levels of service have been converted from the current "operating speeds" as presently shown in the 1965 HCM.

The average running speeds of weaving traffic on the two-sided section, where the through freeway traffic is crossed by entering-exiting movements, were considered to be the same as the speeds of freeway traffic itself. This, of course, is logical since here the main freeway movement is an element of the weaving traffic. The speeds and the correspondingly designated levels of service, as indicated by the regression curves developed from the available data, collaborated this relationship. Thus, the average running speeds shown for the various levels of service in the lower-left portion of the nomograph, Figure 2, are indicative of both the weaving traffic and the freeway traffic.

The right-hand elements of each nomograph are derived from the basic equation:

$$N = (V + (k - 1)\ W_2) \div SV$$

Although this is a rational expression used in the 1965 HCM, the correlation of k factors with volume-speed-length relationship determined through the mechanism of balancing the weaving sections, provides real N values for specified SV (service volumes) per lane. The results are in fractional rather than an integer number of lanes. The final result must be rounded to a whole number of lanes. When the lane adjustment is slight, such as several tenths of a foot per lane, the overall results are practically unaffected. With more radical adjustments in lane requirements or other features, the nomographs still permit the determination of the controlling levels of service, or an adjusted level of service, or an approximation of an adjusted speed. The total nomograph allows for an integrated solution of all elements of a weaving section.

It may be noted in both nomographs that three widths of basic lanes, N_b, are indicated for the major approach to the weaving section. The three right-hand elements of both Figures 1 and 2 are provided for this purpose, allowing for the variation in service volumes, SV, with the width of traveled way. The numerical values of SV are predicated on the 1965 HCM with a preselected peak-hour factor built into the indicated service volumes.

The levels of service presented in Figures 1 and 2 are for the weaving and non-weaving traffic separately (the lower left and lower right sections, respectively). The levels of service are closely related to the speed ranges (for weaving traffic) and volume ranges (for non-weaving traffic) of the 1965 HCM. They are not equivalent to the designations used in other parts of this Circular, and should not be intermixed when reporting results without a clear statement to this effect.

Referring back to the lower-left portion of the nomograph, it is noted that the solid curves are for lane-balanced weaving sections, i.e., with a proper arrangement of lanes operationally including an "optional" lane; the dashed curves are representative of sections lacking lane balance, requiring longer weaving dimensions for given levels of service.

SAMPLE PROBLEMS

To provide some preliminary indication in the use of the updated procedure the following examples are presented.

Example 1

The problem to be investigated is a one-sided lane-balanced weaving section formed along the freeway between two interchanges. The design calls for Level of Service C. The volumes noted have all been converted to equivalent passenger cars per hour (pcph). Referring to the weaving configuration at the upper-right portion of Figure 1 in describing the example, the approach freeway volume is 5100 pcph on 4 lanes with 4500 pcph proceeding through and 600 pcph departing at the next exit. At the entrance ramp 1100 pcph are merging, 950 pcph are proceeding on the freeway, and 150 pcph are destined to the next exit. The total volume through the weaving section amounts to 6200 pcph. The problem is to determine the minimum spacing (for weaving) between ramps and the number of lanes required through the weaving section to maintain Level of Service C operation.

Enter with weaving volumes (W_1+W_2) of 600 + 950 = 1550, proceed right to the 40-mph curve (maximum for C) and turn downward to read a minimum required weaving length, L, of 1300 feet. Then, from the original intersection point proceed along the 40-mph curve to the "turning line for k," and continue upward to intersect the k values curve (no need to read k), at this point turn right and proceed to the smaller weaving volume, W_2 of 600, followed by a downward turn to V = 6200; then a horizontal projection to Level of Service C line (1400 pcph) for N_b = 4 produces, with a downward projection, a total number of lanes, N, of 5.2. A rounding to 5 lanes would be close enough to maintain a balanced section. Theoretically, this barely places the operation into Level of Service D zone and a proportional decrease of weaving speed by approximately 1 mph. (In this case the slight difference may be ignored.

Example 2

Here a two-sided weaving section is formed by an entering ramp on the right and an exiting ramp on the left, as diagrammed in the upper part of Figure 2. The existing section is badly congested and is slated for improvement as required in length and width to produce an operationally balanced facility at Level of Service C. The total volume of $\overline{V}$ = 4700 includes 1800 pcph $(\overline{W_1})$ proceeding through on the freeway, and 500 pcph $(\overline{W_2})$

crossing the freeway from entrance to exit ramp.

Following the solution arrows on the nomograph, it is noted that with $\overline{W_1+W_2}$ of 2300 and Level of Service C, the spacing between ramps has to be increased to at least 2100 feet. Proceeding further through the graph with R = 0.22, $\overline{W_2}$ = 500, and a proposed N_b of 3 lanes, the required number of lanes in the weaving section is indicated to be 4.8. A rounding of N to 5 lanes would be appropriate, maintaining reasonable balance with possibly a very slight improvement in operation.

ASSESSMENT OF METHOD

The graphic presentation for analysis permits a variety of problems to be investigated and solved rapidly. Having all the major variable selected on a composite nomograph provides for flexibility in the procedure and a means of comparing alternative solutions without calculations. An important aspect is that the application of a "balanced" section, which in turn minimizes the number of variables required, permits the nomograph and its associated procedure.

All analyses and relations have been predicated on the present definitions of levels of service and the measures related to them. One of the adjustments reflected in the methodology is the use of average running speeds, taken from the data of field experiments expressed as space mean speeds; however, the basis of the 1965 HCM operating speeds of the various levels of service were maintained in making the conversion to the more usable average running speeds. The numerical service volume have also been retained from the 1965 HCM, except for the means of "building-in" the peak-hour factor within the base service volumes.

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