TRANSPORTATION
RESEARCH
CIRCULAR

Number 284, October 1984
ISSN 0097-8515

Transportation Research Board, National Research Council, 2101 Constitution Avenue, Washington, D.C. 20418

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PROPOSED CHAPTERS FOR THE 1985 HIGHWAY CAPACITY MANUAL— ADDENDUM 1 TO TRB CIRCULAR 281

modes

1 highway transportation
5 other

subject areas

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

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Foreword

This Circular presents final text for portions of the Third Edition of the *HIGHWAY CAPACITY MANUAL*. Chapter 8, "Two-Lane Highways," Chapter 13, "Pedestrians," and Chapter 14, "Bicycles," are appearing now in TRB Circular 284 for two reasons: To make new procedures available to practitioners as early as possible; and to provide a final review opportunity so that problems of clarity and the like will be minimized in the eventual complete Manual.

The Third Edition of the *HIGHWAY CAPACITY MANUAL*, expected in 1985, will be another milestone in a long history. Its first edition appeared in 1950. Research leading to a second edition began a few years later. Under the leadership of the Transportation Research Board (then Highway Research Board) Committee on Highway Capacity, these efforts led to the publication of Special Report 87, *Highway Capacity Manual—1965*. More research followed, and a formal project was initiated in 1977 under the National Cooperative Highway Research Program (NCHRP) to produce the third edition. One step along the way was the 1980 publication of TRB Circular 212, *Interim Materials on Highway Capacity*. Another has been TRB Circular 281, *Proposed Chapters for the 1985 Highway Capacity Manual.*

This long publication history attests to the importance of the Highway Capacity Manual for highway engineering practitioners. Now in its tenth printing, the 1965 Manual has been the Board's most widely distributed publication, with more than 30,000 copies made available. It has been translated into several languages, and it frequently serves as the primary reference for planning, design, and operational analyses of highway capacity all over the world—this despite the fact that the data upon which it is based come from North American experience. TRB Circular 212 has itself been through several printings, with more than 9,000 copies distributed in the past three years.

Yet much has changed since earlier editions, in both travel characteristics and information needs for capacity analyses. Research by many individuals, by private organizations, and by public agencies has led to new understandings and insights, leading in turn to procedural revisions and new techniques. Because some past public concerns have faded and new issues have taken their place, current requirements reflect new emphasis. All of these forces press for the new publication.

The chapters presented in this Circular are not necessarily interrelated, and they do not represent a complete section in the Third Edition. They are simply those where the work is now complete. The conveyance to TRB of any discovery of errors or recommendations for improved clarity is invited and will be received with gratitude. Although one or more Circulars may follow this one before the entire Manual is assembled, it is anticipated that production of the complete manual will begin no later than November 1984. Comments, suggestions, or criticisms should be sent before then.

The three chapters in this Circular are Chapter 8, Chapter 13, and Chapter 14. Chapter 8 replaces procedures given in the 1965 HIGHWAY CAPACITY MANUAL. Chapter 13 replaces material in Circular 212, *Interim Materials on Highway Capacity*. Chapter 14 is new text.

Acknowledgments

The fifteen chapters in the Third Edition of the *HIGHWAY CAPACITY MANUAL* will come from several sources. In some cases they represent the results of funded research specifically commissioned for the development of new Manual material. In other cases, and at the other extreme, they represent the voluntary contributions of members of the TRB Committee on Highway Capacity and Quality of Service. Still others represent mixed sources of inputs that become nearly impossible to identify and to acknowledge. Nevertheless, all chapters have two features in common. Each has been prepared by the research team assembled under Dr. Roger P. Roess of the Polytechnic Institute of New York and Dr. Carroll J. Messer of Texas Transportation Institute. And each has been thoroughly reviewed by members of the NCHRP project panel monitoring the work, by members of the TRB Committee on Highway Capacity and Quality of Service and its subcommittees, and by many individuals not affiliated with either group who have volunteered their time and interest.

The principal agencies and groups whose contributions merit recognition are listed as follows:

- The National Cooperative Highway Research Program, under the management and guidance of the NCHRP staff and project panel, has been responsible for much of the work leading to the development of these and the remaining chapters. Other research has been funded by the Federal Highway Administration, under the Office of Research, Development and Technology.
- Research agencies have included JHK & Associates, Texas A&M Research Foundation, Polytechnic Institute of New York, PRC Voorhees, and Jack E. Leisch & Associates. Others are the Traffic Institute at Northwestern University, KLD Associates, Inc., and the Minnesota Department of Transportation.
- The final responsibility for what appears in this Circular belongs to the Transportation Research Board Committee on Highway Capacity and Quality of Service and supporting staff including the Editorial and Production Offices.

CHAPTER 8

TWO-LANE HIGHWAYS

CONTENTS

I. INTRODUCTION

A two-lane highway may be defined as a two-lane roadway having one lane for use by traffic in each direction. Passing of slower vehicles requires the use of the opposing lane where sight distance and gaps in the opposing traffic stream permit. As volumes and/or geometric restrictions increase, the ability to pass decreases, resulting in the formation of platoons in the traffic stream. Motorists in these platoons are subject to delay because of the inability to pass.

Two-lane highways compose the predominant mileage of most national highway systems. They are used for a variety of functions, are located in all geographic areas, and serve a wide range of traffic requirements. Consideration of operating quality must account for these disparate traffic functions.

Efficient *mobility* is the principal function of major two-lane highways used as primary arteries connecting major traffic generators or as primary links in state and national highway networks. Such routes tend to serve long-distance commercial and recreational travelers, and may have sections of many miles through rural environments without traffic control interruptions. Consistent high-speed operations and infrequent passing delays are desirable for these facilities.

Many paved, two-lane rural roads basically serve an *accessibility* function. They provide all-weather accessibility to an area, often for relatively low traffic volumes. The provision of cost-effective access is the dominant policy consideration. High speed, while beneficial, is not the principal concern. Delay, as indicated by the formation of platoons, and the utilization of capacity are more relevant measures of service quality.

Two-lane roads also serve scenic and recreational areas where the vista and environment are to be experienced and enjoyed without traffic interruption or delay. A safe roadway is desired, but high-speed operation is neither expected nor desired.

Short sections of high-volume two-lane roads sometime serve as short connections between two major multilane roadways or urban centers. For such short links, traffic conditions tend to be better than might be expected for longer two-lane segments, and the expectations of motorists regarding service quality are generally higher than for longer sections.

For these reasons, three parameters are used to describe service quality for two-lane highways:

1. Average travel speed.
2. Percent time delay.
3. Capacity utilization.

Average travel speed reflects the mobility function of two-lane highways, and is the length of the highway segment under consideration divided by the average travel time of all vehicles traversing the segment in both directions over some designated time interval.

Percent time delay reflects both mobility and access functions, and is defined as the average percent of time that all vehicles are delayed while traveling in platoons due to the inability to pass. "Percent time delay" is difficult to measure directly in the field. The percent of vehicles traveling at headways less than 5 sec can be used as a surrogate measure in field studies.

The *utilization of capacity* reflects the access function, and is defined as the ratio of the demand flow rate to the capacity of the facility.

Level-of-service criteria utilize all three of the parameters noted above, with percent time delay being the primary measure of service quality. Speed and capacity utilization are secondary measures.

This chapter provides specific definitions and methodologies for the estimation of level of service for all types of two-lane highways. Subsequent sections provide a descriptive list of treatments for alleviating both spot and section design and/or operational problems that may arise because of high volume and/or geometric restrictions. A set of example calculations is provided to illustrate the use and application of procedures. A complete set of worksheets for all levels of analysis is also provided. Illustration 8-1 shows typical views of two-lane, two-way rural highways.

LEVELS OF ANALYSIS

This chapter is based on a comprehensive study of two-lane highway operation *(1, 2)*. Microscopic simulation combined with additional field data *(3)* and theoretical considerations were used to develop the methodology. Analysis is provided at two levels:

1. *Operational analysis*—This application is intended to determine the level of service for an existing two-lane highway with existing traffic and roadway conditions, or for projected future conditions; operational analysis applications are presented for general terrain segments and for specific grades.
2. *System planning*—This application enables planners to quickly determine the AADT volumes which can be accommodated on two-lane highways for various levels of service and terrain conditions.

Design computations cannot be readily performed for two-lane highways because the number of lanes is fixed. Modifications to grade and alignment, however, could improve the operational efficiency of a two-lane facility. For other design options, procedures for the appropriate types of facilities would be consulted. Procedures of Chapter 3, "Basic Freeway Segments," and Chapter 7, "Multilane Highways," would often be useful in investigating design alternatives.

The selection of an appropriate level of analysis is based on the objectives of the analysis, the available data base, and the accuracy requirements.

OPERATIONAL CHARACTERISTICS

Traffic operations on two-lane, two-way highways are unique. Lane-changing and passing are possible only in the face of oncoming traffic in the opposing lane. Passing demand increases rapidly as traffic volumes increase, while passing capacity in the opposing lane declines as volumes increase. Thus, unlike

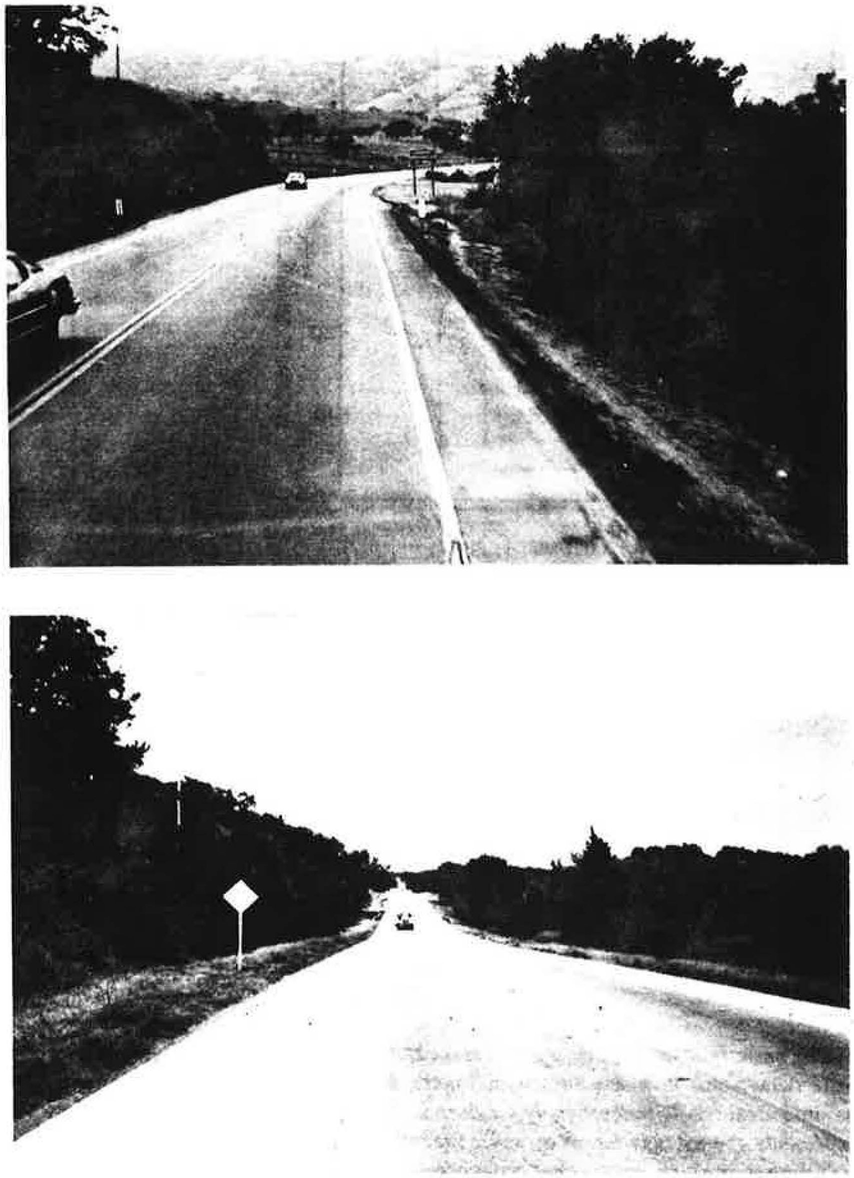


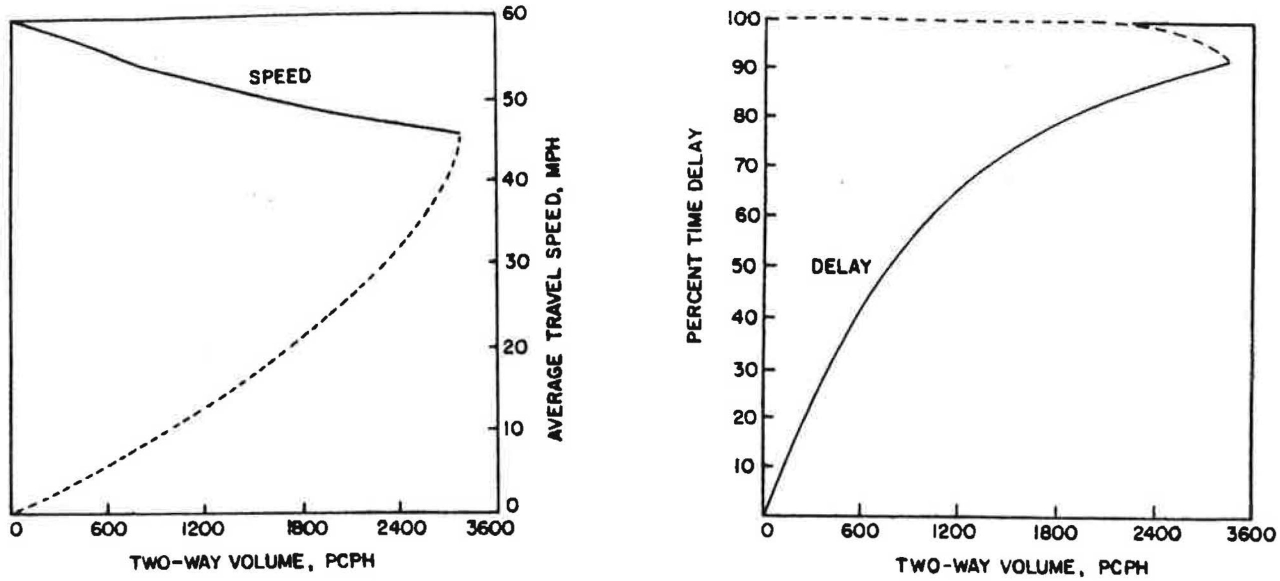
Illustration 8-1. Typical views of two-lane, two-way highways in rural environments.

other types of uninterrupted flow facilities, on two-lane highways, normal traffic flow in one direction influences flow in the other direction. Motorists are forced to adjust their individual travel speed as volume increases and the ability to pass declines. Two traffic stream characteristics, average travel speed and percent time delay, are used as operational measures describing the quality of service provided to motorists on a two-lane highway.

A relatively high running speed has become an accepted criterion for primary highway design. Mean speeds of traffic flow are frequently observed above 55 mph on primary rural highways. Research has shown that speed is fairly insensitive to volume on two-lane highways without significant grades or turning traffic *(4)*. Consequently, average speeds of less than 50 mph are judged undesirable for primary two-lane highways in level terrain because of the high percentage of time motorists would be delayed.

"Percent time delay" is the average percent of the total travel time that all motorists are delayed in platoons while traveling a given section of highway. Motorists are defined to be delayed when traveling behind a platoon leader at speeds less than their desired speed and at headways less than 5 sec. For field measurement purposes, percent time delay in a section is approximately the same as the percentage of all vehicles traveling in platoons at headways less than 5 sec *(2, 5)*.

Percent time delay reflects the changing service quality perceived by motorists under a wide range of geometric and traffic conditions. At low traffic volumes, motorists are almost never delayed because demand for passing is low, average headways are high, and the ability to pass is high. The percent time delay for such conditions is near 0 percent. As volumes approach capacity, passing demand greatly exceeds passing capacity, major platoons of traffic exist, and motorists are delayed almost



a) Relationship between average speed and flow on two-lane highways.

b) Relationship between percent time delay and flow on two-lane highways

Figure 8-1. Speed-flow and percent time delay-flow relationships for two-lane rural highways (ideal conditions).

100 percent of the time. Even though speeds may be relatively high near capacity (40 mph or more), driver frustration would be excessive if these conditions routinely existed for long periods of time.

The basic relationships between average travel speed, percent time delay, and flow are shown in Figure 8-1. These curves assume ideal traffic and roadway conditions. The average speed represents the average travel or space mean speed of all traffic traveling in both directions over the section of highway in question. Percent time delay is the average for all vehicles in the traffic stream.

IDEAL CONDITIONS

Ideal conditions for two-lane highways are defined as no restrictive geometric, traffic, or environmental conditions. Specifically, they include:

1. Design speed greater than or equal to 60 mph.
2. Lane widths greater than or equal to 12 ft.
3. Clear shoulders wider than or equal to 6 ft.
4. No "no passing zones" on the highway.
5. All passenger cars in the traffic stream.
6. A 50/50 directional split of traffic.
7. No impediments to through traffic due to traffic control or turning vehicles.
8. Level terrain.

The capacity of two-lane rural highways under these ideal conditions is 2,800 pcph, total, in both directions. This capacity reflects the impact of opposing vehicles on passing opportunities, and therefore on the ability to efficiently fill gaps in the traffic stream. This phenomenom restricts capacity to a lower value than the 2,000 pcphpl which may be accommodated on multilane uninterrupted flow facilities.

Directional distribution is defined to be 50/50 for ideal conditions. Most directional distribution factors observed on rural two-lane highways range from 55/45 to 70/30. On recreational routes, the directional distribution may be as high as 80/20 or more during holiday or other peak periods. Some variation in speed and percent time delay occurs by direction with changing directional distribution factors and volume levels. Minor changes in average traffic stream characteristics will also occur with directional distribution.

The frequency of no passing zones along a two-lane highway is used to characterize roadway design and to define expected traffic conditions. A no passing zone is defined as any marked no passing zone or, as a surrogate, any section of road wherein the passing sight distance is 1,500 ft or less. The average percentage of no passing zones in both directions along a section is used in the procedures.

The typical percentage of no passing zones found on rural two-lane highways ranges from 20 percent to 50 percent. Values approaching 100 percent can be found on sections of winding mountainous roads. No passing zones have a greater effect in mountainous terrain than on level or rolling highway segments. Heavy platoon formation along a highway section also may cause greater than expected operational problems on an adjacent downstream section having restricted passing opportunities.

II. METHODOLOGY

LEVELS OF SERVICE

As noted previously, level-of-service criteria for two-lane highways address both mobility and accessibility concerns. The primary measure of service quality is percent time delay, with speed and capacity utilization used as secondary measures. Level-of-service criteria are defined for peak 15-min flow periods, and are intended for application to segments of significant length.

Level-of-service criteria for general terrain segments are given in Table 8-1. For each level of service, the percent time delay is shown. Average travel speed is also shown, with values varying slightly by type of terrain. The body of the table includes maximum values of *v/c* ratio for the various terrain categories and levels of service A through F. The *v/c* ratios shown in Table 8-1 are somewhat different from those used in other chapters. For two-lane highways, the values given represent the ratio of flow rate to "ideal capacity," where ideal capacity is 2,800 pcph for a level terrain segment with ideal geometrics and 0 percent no passing zones. Two-lane highways are quite complex, and capacities vary depending on terrain and the degree of passing restrictions. To simplify computational procedures, *v/c* ratios are given in terms of the constant "ideal capacity" of 2,800 pcph, total in both directions of flow.

The level-of-service criteria of Table 8-1 are for extended segments of two-lane rural highways where efficient mobility is the primary objective of the facility. Where speeds have been restricted by an agency, such as through a town or village, the percentage of time delay and capacity utilization are the only meaningful indicators of level of service.

Table 8-2 gives level-of-service criteria for specific grade segments. These criteria relate the average travel speed of *upgrade* vehicles to level of service. Operations on sustained two-lane grades are substantially different from extended segments of general terrain. The speed of upgrade vehicles is seriously impacted, as the formation of platoons behind slow-moving vehicles intensifies and passing maneuvers generally become more difficult. Further, unlike general terrain segments, where the approximate average travel speed at which capacity occurs can be identified, the capacity speed for a specific grade depends on the steepness and length of the grade and volume. Because of this, estimation of capacity is complex. Thus, Table 8-2 defines separate level-of-service criteria for specific grade segments. In addition, this chapter includes special computational procedures for sustained grades on two-lane highways.

Downgrade operations are not specifically addressed by these procedures. Downgrade operations on gentle grades (less than 3 percent) are generally comparable to those on a level roadway. On more severe grades, downgrade operations are about midway between those experienced on a level roadway and those experienced on an upgrade of equivalent traffic and roadway characteristics. The principal concern on steep downgrades is the potential for "runaway" trucks.

The highest quality of traffic service occurs when motorists are able to drive at their desired speed. Without strict enforcement, this highest quality, representative of *level-of-service A*, would result in average speeds approaching 60 mph on two-lane highways. The passing frequency required to maintain these speeds has not reached a demanding level. Passing demand is

TABLE 8-1. LEVEL-OF-SERVICE CRITERIA FOR GENERAL TWO-LANE HIGHWAY SEGMENTS

		v/c RATIO[a]																			
		LEVEL TERRAIN							ROLLING TERRAIN							MOUNTAINOUS TERRAIN					
	PERCENT	AVG[b]	PERCENT NO PASSING ZONES						AVG[b]	PERCENT NO PASSING ZONES						AVG[b]	PERCENT NO PASSING ZONES					
LOS	TIME DELAY	SPEED	0	20	40	60	80	100	SPEED	0	20	40	60	80	100	SPEED	0	20	40	60	80	100
A	≤ 30	≥ 58	0.15	0.12	0.09	0.07	0.05	0.04	≥ 57	0.15	0.10	0.07	0.05	0.04	0.03	≥ 56	0.14	0.09	0.07	0.04	0.02	0.01
B	≤ 45	≥ 55	0.27	0.24	0.21	0.19	0.17	0.16	≥ 54	0.26	0.23	0.19	0.17	0.15	0.13	≥ 54	0.25	0.20	0.16	0.13	0.12	0.10
C	≤ 60	≥ 52	0.43	0.39	0.36	0.34	0.33	0.32	≥ 51	0.42	0.39	0.35	0.32	0.30	0.28	≥ 49	0.39	0.33	0.28	0.23	0.20	0.16
D	≤ 75	≥ 50	0.64	0.62	0.60	0.59	0.58	0.57	≥ 49	0.62	0.57	0.52	0.48	0.46	0.43	≥ 45	0.58	0.50	0.45	0.40	0.37	0.33
E	> 75	≥ 45	1.00	1.00	1.00	1.00	1.00	1.00	≥ 40	0.97	0.94	0.92	0.91	0.90	0.90	≥ 35	0.91	0.87	0.84	0.82	0.80	0.78
F	100	< 45	—	—	—	—	—	—	< 40	—	—	—	—	—	—	< 35	—	—	—	—	—	—

[a] Ratio of flow rate to an ideal capacity of 2,800 pcph in both directions.
[b] Average travel speed of all vehicles (in mph) for highways with design speed ≥ 60 mph; for highways with lower design speeds, reduce speed by 4 mph for each 10-mph reduction in design speed below 60 mph; assumes that speed is not restricted to lower values by regulation.

TABLE 8-2. LEVEL-OF-SERVICE CRITERIA FOR SPECIFIC GRADES

LEVEL OF SERVICE	AVERAGE UPGRADE SPEED (MPH)
A	≥ 55
B	≥ 50
C	≥ 45
D	≥ 40
E	≥ 25–40[a]
F	< 25–40[a]

[a] The exact speed at which capacity occurs varies with the percentage and length of grade, traffic compositions, and volume; computational procedures are provided to find this value.

well below passing capacity, and almost no platoons of three or more vehicles are observed. Drivers would be delayed no more than 30 percent of the time by slow-moving vehicles. A maximum flow rate of 420 pcph, total in both directions, may be achieved under ideal conditions.

Level-of-service B characterizes the region of traffic flow wherein speeds of 55 mph or slightly higher are expected on level terrain. Passing demand needed to maintain desired speeds becomes significant and approximately equals the passing capacity at the lower boundary of level-of-service B. Drivers are delayed up to 45 percent of the time on the average. Service flow rates of 750 pcph, total in both directions, can be achieved under ideal conditions. Above this flow rate, the number of platoons forming in the traffic stream begins to increase dramatically.

Further increases in flow characterize *level-of-service C*, resulting in noticeable increases in platoon formation, platoon size, and frequency of passing impediment. Average speed still exceeds 52 mph on level terrain, even though unrestricted passing demand exceeds passing capacity. At higher volume levels, chaining of platoons and significant reductions in passing capacity begin to occur. While traffic flow is stable, it is becoming susceptible to congestion due to turning traffic and slow-moving vehicles. Percent time delays are up to 60 percent. A service flow rate of up to 1,200 pcph, total in both directions, can be accommodated under ideal conditions.

Unstable traffic flow is approached as traffic flows enter *level-of-service D*. The two opposing traffic streams essentially begin to operate separately at higher volume levels, as passing becomes extremely difficult. Passing demand is very high, while passing capacity approaches zero. Mean platoon sizes of 5 to 10 vehicles are common, although speeds of 50 mph can still be maintained under ideal conditions. The fraction of no passing zones along the roadway section usually has little influence on passing. Turning vehicles and/or roadside distractions cause major shockwaves in the traffic stream. The percentage of time motorists are delayed approaches 75 percent. Maximum service flow rates of 1,800 pcph, total in both directions, can be maintained under ideal conditions. This is the highest flow rate that can be maintained for any length of time over an extended section of level terrain without a high probability of breakdown.

Level-of-service E is defined as traffic flow conditions on two-lane highways having a percent time delay of greater than 75 percent. Under ideal conditions, speeds will drop below 50 mph. Average travel speeds on highways with less than ideal conditions will be slower, as low as 25 mph on sustained upgrades. Passing is virtually impossible under level-of-service E conditions, and platooning becomes intense when slower vehicles or other interruptions are encountered.

The highest volume attainable under level-of-service E defines the capacity of the highway. Under ideal conditions, capacity is 2,800 pcph, total in both directions. For other conditions, capacity is lower. Note that the v/c ratios of Table 8-1 are not all 1.00 at capacity. This is because the ratios are relative to "ideal capacity" as discussed. Operating conditions at capacity are unstable and difficult to predict. Traffic operations are seldom observed near capacity on rural highways, primarily because of a lack of demand.

Capacity of two-lane highways is affected by the directional split of traffic. As directional split moves away from the 50/50 "ideal" condition, total two-way capacity is reduced, as follows:

Directional Split	*Total Capacity (pcph)*	*Ratio of Capacity to Ideal Capacity*
50/50	2,800	1.00
60/40	2,650	0.94
70/30	2,500	0.89
80/20	2,300	0.83
90/10	2,100	0.75
100/0	2,000	0.71

For short lengths of two-lane road, such as tunnels or bridges, opposing traffic interactions may have only a minor effect on capacity. The capacity in each direction may approximate that of a fully loaded single lane, given appropriate adjustments for the lane width and shoulder width *(5)*.

As with other highway types, *level-of-service F* represents heavily congested flow with traffic demand exceeding capacity. Volumes are lower than capacity, and speeds are below capacity speed. Level-of-service E is seldom attained over extended sections on level terrain as more than a transient condition; most often, perturbations in traffic flow as level E is approached cause a rapid transition to level-of-service F.

OPERATIONAL ANALYSIS

This section presents the methodology for operational analysis of general terrain segments and specific grades on two-lane highways. Separate procedures for general highway segments and grades are used, because the dynamics of traffic interaction on sustained two-lane grades differ from those on general terrain segments. Grades of less than 3 percent or shorter than 1/2 mile may be included in general terrain analysis. Grades both longer and steeper than these values should generally be treated as specific grades. Level, rolling, and mountainous terrain are as defined in Chapters 1 and 3.

The length of grade is taken to be the tangent length of grade plus a portion of the vertical curves at the beginning and end of the grade. About one-fourth of the length of vertical curves at the beginning and end of a grade are included in the grade length. Where two grades (in the same direction) are joined by a vertical curve, one-half the length of the curve is included in each grade segment.

The objective of operational analysis is generally the determination of level of service for an existing or projected facility operating under existing or projected traffic demand. Operational analysis may also be used to determine the capacity of a two-lane highway segment, or the service flow rate which can be accommodated at any given level of service.

Use of the Peak Hour Factor

As for other facility types, two-lane highway analysis is based on flow rates for a peak 15-min period within the hour of interest, which is usually the peak hour. The criteria of Table 8-1 refer to equivalent hourly flow rates based on the peak 15 min of flow.

These criteria are used to compute service flow rates, *SF*, which are compared to existing or projected flow rates to determine level of service. Thus, full-hour demand volumes must be converted to flow rates for the peak 15 min, as follows:

$$v = V/\text{PHF}$$

where:

v = flow rate for the peak 15 min, total for both directions of flow, in vph;
V = full-hour volume total for both directions of flow, in vph; and
PHF = peak hour factor.

When criteria are compared to flow rates, the predicted operating characteristics are expected to prevail for the 15-min period for which the flow rate applies. For many rural conditions, the analyst may wish to examine average conditions over a peak hour. Full-hour volumes, unadjusted for the PHF, are compared to criteria directly for these cases. It should be noted, however, that prediction of an average level-of-service C during a full hour may include portions of the hour operating at level D or E, while other portions operate at A or B.

The decision to use flow rates or full-hour volumes in an analysis is related to whether or not peaking characteristics will cause substantial fluctuation in operating conditions within the peak hour, and whether the impact of such fluctuations will impact design and/or operational policy decisions. In general, where the peak hour factor is less than 0.85, operating conditions will vary substantially within the hour.

Where the peak hour factor can be determined from local field data, this should be done. Where field data are not available, the factors tabulated in Table 8-3 may be used. These are based solely on the assumption of random flow and may be somewhat higher than those obtained from field studies. When level of service is to be determined for a given traffic volume, a value appropriate to the volume level on the subject segment is selected from the upper portion of the table. When a service flow rate is to be computed, a value is selected from the lower portion of the table, because volume is unknown.

Analysis of General Terrain Segments

The general terrain methodology estimates average traffic operational measures along a section of highway based on average terrain, geometric, and traffic conditions. Terrain is classified as level, rolling, or mountainous, as described previously. The general terrain procedure is usually applied to highway sections of at least 2 miles in length.

Highway geometric features include a general description of longitudinal section characteristics and specific roadway cross-section information. Longitudinal section characteristics are described by the average percent of the highway having no passing zones. The average for both directions is used. The percentage of roadway along which sight distance is less than 1,500 ft may be used as a surrogate for no passing zone data. Roadway cross-section data include lane width and usable shoulder width. Geometric data on design speed and specific grades are not used directly, but are reflected in the other geometric factors discussed.

TABLE 8-3. PEAK HOUR FACTORS FOR TWO-LANE HIGHWAYS BASED ON RANDOM FLOW

A. LEVEL-OF-SERVICE DETERMINATIONS

TOTAL 2-WAY HOURLY VOLUME (VPH)	PEAK HOUR FACTOR (PHF)	TOTAL 2-WAY HOURLY VOLUME (VPH)	PEAK HOUR FACTOR (PHF)
100	0.83	1,000	0.93
200	0.87	1,100	0.94
300	0.90	1,200	0.94
400	0.91	1,300	0.94
500	0.91	1,400	0.94
600	0.92	1,500	0.95
700	0.92	1,600	0.95
800	0.93	1,700	0.95
900	0.93	1,800	0.95
		≥ 1,900	0.96

B. SERVICE FLOW-RATE DETERMINATIONS

Level of Service	A	B	C	D	E
Peak Hour Factor	0.91	0.92	0.94	0.95	1.00

Traffic data needed to apply the general terrain methodology include the two-way hourly volume, a peak hour factor, and the directional distribution of traffic flow. Peak hour factors may be computed from field data, or appropriate default values may be selected from Table 8-3. Traffic data also include the proportion of trucks, recreational vehicles (RV's), and buses in the traffic stream. When estimates of the traffic mix are not available, the following default values for these fractions may be used for primary routes:

- P_T = 0.14 (trucks)
- P_R = 0.04 (RV's)
- P_B = 0.00 (buses)

Recreational routes would typically have a higher proportion of recreational vehicles than shown for primary rural routes.

1. *General relationship*—The general relationship describing traffic operations on general terrain segments is as follows:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV} \qquad (8\text{-}1)$$

where:

SF_i = total service flow rate in both directions for prevailing roadway and traffic conditions, for level of service *i*, in vph;

$(v/c)_i$ = ratio of flow rate to ideal capacity for level of service *i*, obtained from Table 8-1;

f_d = adjustment factor for directional distribution of traffic, obtained from Table 8-4;

f_w = adjustment factor for narrow lanes and restricted shoulder width, obtained from Table 8-5;

f_{HV} = adjustment factor for the presence of heavy vehicles in the traffic stream, computed as:

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)] \qquad (8\text{-}2)$$

where:

P_T = proportion of trucks in the traffic stream, expressed as a decimal;

P_R = proportion of RV's in the traffic stream, expressed as a decimal;

P_B = proportion of buses in the traffic stream, expressed as a decimal;

E_T = passenger-car equivalent for trucks, obtained from Table 8-6;

E_R = passenger-car equivalent for RV's, obtained from Table 8-6; and

E_B = passenger-car equivalent for buses, obtained from Table 8-6.

Equation 8-1 takes an ideal capacity of 2,800 pcph, and adjusts it to reflect a v/c ratio appropriate for the desired level of service, directional distributions other than 50/50, lane width restrictions and narrow shoulders, and heavy vehicles in the traffic stream.

2. *Adjustment for v/c ratio*—The v/c ratios given in Table 8-1 reflect a complex relationship among speed, flow, delay, and geometric parameters for two-lane highways. Specifically, v/c values vary with level-of-service criteria, terrain type, and the magnitude of passing restrictions. Note that v/c ratios at capacity are not equal to 1.00 for rolling or mountainous terrain. This is because the ratios are based on an ideal capacity of 2,800 pcph which cannot be achieved on severe terrains. Further, as the formation of platoons is more frequent where terrain is rolling or mountainous, passing restrictions have a greater effect on capacity and service flow rate than on level terrain.

3. *Adjustment for directional distribution*—All of the v/c values in Table 8-1 are for a 50/50 directional distribution of traffic on a two-lane highway. For other directional distributions, the factors shown in Table 8-4 must be applied to Table 8-1 values.

4. *Adjustment for narrow lanes and restricted shoulder width*—Narrow lanes force motorists to drive closer to vehicles in the opposing lane than they would normally desire. Restricted or narrow shoulders have much the same effect, as drivers "shy" away from roadside objects or point restrictions perceived to be close enough to the roadway to pose a hazard. Motorists compensate for driving closer to opposing vehicles by slowing down and/or by leaving larger headways between vehicles in the same lane. Both reactions result in lower flow rates being sustained at any given speed.

Factors reflecting this behavior are shown in Table 8-5, and are applied to v/c values taken from Table 8-1. Factors at capacity are higher than those for other levels of service, as the impact of narrow lanes and restricted shoulder widths is less deleterious when vehicles are already traveling at reduced speeds which prevail under capacity operation.

5. *Adjustment for heavy vehicles in the traffic stream*—The v/c ratios of Table 8-1 are based on a traffic stream consisting of only passenger cars. All vehicles having only four wheels contacting the pavement may be considered to be passenger cars. This includes light vans and pick-up trucks.

"Heavy vehicles" are categorized as trucks, recreational vehicles, or buses, and the traffic stream is characterized by the proportion of such vehicles in the traffic mix. The adjustment factor for heavy vehicles, f_{HV}, is computed using Eq. 8-2 and the passenger-car equivalents given in Table 8-6.

A wide range in the proportions of trucks and RV's in the traffic stream are found on rural highways. Equation 8-2 will yield an adjustment factor for any given mix. In addition, there is some variation in the weight distribution between heavy (> 35,000 lb) and medium-duty (≤ 35,000 lb) trucks. The equivalents of Table 8-6 assume a 50/50 distribution between heavy and medium-duty trucks. Two-lane highways serving unusually high proportions of heavy trucks, such as in coal, gravel, or timber operations, particularly those in mountainous terrain, would have higher values of E_T than those shown in the table.

The deleterious impact of heavy vehicles on two-lane highways increases markedly as terrain becomes more severe. As heavy vehicles slow on steeper grades, platoon formation becomes more frequent and severe. This effect is compounded by passing sight distance restrictions often accompanying severe terrain and leads to serious deterioration of traffic flow.

Analysis of Specific Grades

The analysis of extended specific grades on two-lane highways is more complex than for general terrain segments. The analysis procedures assume that the approach to the grade is level. On such grades, the operation of upgrade vehicles is substantially

TABLE 8-4. ADJUSTMENT FACTORS FOR DIRECTIONAL DISTRIBUTION ON GENERAL TERRAIN SECTIONS

Directional Distribution	100/0	90/10	80/20	70/30	60/40	50/50
Adjustment Factor, f_d	0.71	0.75	0.83	0.89	0.94	1.00

TABLE 8-5. ADJUSTMENT FACTORS FOR THE COMBINED EFFECT OF NARROW LANES AND RESTRICTED SHOULDER WIDTH, f_w

USABLE[a] SHOULDER WIDTH (FT)	12-FT LANES		11-FT LANES		10-FT LANES		9-FT LANES	
	LOS A–D	LOS[b] E	LOS A–D	LOS[b] E	LOS A–D	LOS[b] E	LOS A–D	LOS[b] E
0	0.70	0.88	0.65	0.82	0.58	0.75	0.49	0.66
2	0.81	0.93	0.75	0.88	0.68	0.81	0.57	0.70
4	0.92	0.97	0.85	0.92	0.77	0.85	0.65	0.74
≥ 6	1.00	1.00	0.93	0.94	0.84	0.87	0.70	0.76

[a] Where shoulder width is different on each side of the roadway, use the average shoulder width.
[b] Factor applies for all speeds less than 45 mph.

impacted, while downgrade vehicles experience far less impact. As a result, level-of-service criteria presented in Table 8-2 are based on the average *upgrade* travel speed. This speed is the average speed of all vehicles traveling up the grade.

Where composite grades are present, the average grade is used in analysis. The average grade is the total rise, in feet, of the composite grade divided by the horizontal length of the grade, in feet, multiplied by 100 to adjust from a decimal to a percentage.

The average upgrade speed at which capacity occurs varies between 25 and 40 mph, depending upon the percent grade, the percentage of no passing zones, and other factors. Because operating conditions at capacity vary for each grade, the finding of capacity is not as straightforward as service flow rate computations for levels-of-service A through D, where speed is established using the criteria of Table 8-2.

Research has found that grades on two-lane highways have a more significant impact on operations than similar grades on multilane highways. Platoons forming behind slow-moving vehicles can be broken up or dissipated only by passing maneuvers using the opposing lane. On two-lane highways, the same geometric features causing platoons to form also tend to restrict passing opportunities as well. It has also been found that most passenger cars, even in the absence of heavy vehicles, are affected by extended grades, and will operate less efficiently than on level terrain. Additional operational problems due to vehicle stalls, accidents, or other incidents are not accounted for in the procedure. The effects of rain, snow, ice, and other negative environmetal factors are also not considered.

1. *Relationship between speed and service flow rate on specific grades*—Average upgrade speeds on two-lane highways may be estimated for specific grades of a given percent and length of grade, assuming a level approach to the grade. Two-way service flow rates, *SF*, may be calculated for a specific level of service, or correspondingly, for any designated average upgrade speed. The need to provide a climbing lane based on AASHTO's safety warrant is not part of the procedure, but sample calculation 5 illustrates the evaluation of a potential climbing lane.

TABLE 8-6. AVERAGE PASSENGER-CAR EQUIVALENTS FOR TRUCKS, RV'S, AND BUSES ON TWO-LANE HIGHWAYS OVER GENERAL TERRAIN SEGMENTS

VEHICLE TYPE	LEVEL OF SERVICE	TYPE OF TERRAIN		
		LEVEL	ROLLING	MOUNTAINOUS
Trucks, E_T	A	2.0	4.0	7.0
	B and C	2.2	5.0	10.0
	D and E	2.0	5.0	12.0
RV's E_R	A	2.2	3.2	5.0
	B and C	2.5	3.9	5.2
	D and E	1.6	3.3	5.2
Buses, E_B	A	1.8	3.0	5.7
	B and C	2.0	3.4	6.0
	D and E	1.6	2.9	6.5

SOURCE: Ref. 6

The service flow rate for any given average upgrade speed is given by the following relationship:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV} \quad (8\text{-}3)$$

where:

SF_i = service flow rate for level-of-service *i*, or speed *i*, total vph for both directions, for prevailing roadway and traffic conditions.

$(v/c)_i$ = v/c ratio for level-of-service *i* or speed *i*, obtained from Table 8-7;

f_d = adjustment factor for directional distribution, obtained from Table 8-8;

f_w = adjustment factor for narrow lanes and restricted shoulder width, obtained from Table 8-5;

f_g = adjustment factor for the operational effects of grades on passenger cars, computed as described below; and

f_{HV} = adjustment factor for the presence of heavy vehicles in the upgrade traffic stream, computed as described subsequently.

This relationship for specific grades is generally not applied to grades of less than 3 percent or shorter than 1/2 mile.

2. *Adjustment for v/c ratio*—Table 8-7 shows values of v/c ratio related to percent grade, average upgrade speed, and percent no passing zones. The values shown are the ratio of flow rate to an ideal capacity of 2,800 pcph, and assume that passenger cars are unaffected by extended grades. Another adjustment is applied to account for the impacts of grades on passenger-car operation. This is an important point, because a v/c ratio of 1.00 in Table 8-7 *DOES NOT* necessarily signify capacity. The solution for capacity of an extended grade is discussed later.

Values of v/c approaching or equal to 0.00 mean that the associated average upgrade speed is difficult or impossible to achieve for the percent grade and percent no passing zones indicated.

3. *Adjustment for directional distribution*—On extended grades, the directional distribution can be a critical factor affecting operations. Distributions from 100/0 to 0/100 can be encountered, with the first percentage reflecting directional flow up the grade. Table 8-8 contains adjustment factors for a full range of directional distributions.

TABLE 8-7. VALUES OF v/c RATIO[a] VS. SPEED, PERCENT GRADE, AND PERCENT NO PASSING ZONES FOR SPECIFIC GRADES

PERCENT GRADE	AVERAGE UPGRADE SPEED (MPH)	PERCENT NO PASSING ZONES					
		0	20	40	60	80	100
3	55	0.27	0.23	0.19	0.17	0.14	0.12
	52.5	0.42	0.38	0.33	0.31	0.29	0.27
	50	0.64	0.59	0.55	0.52	0.49	0.47
	45	1.00	0.95	0.91	0.88	0.86	0.84
	42.5	1.00	0.98	0.97	0.96	0.95	0.94
	40	1.00	1.00	1.00	1.00	1.00	1.00
4	55	0.25	0.21	0.18	0.16	0.13	0.11
	52.5	0.40	0.36	0.31	0.29	0.27	0.25
	50	0.61	0.56	0.52	0.49	0.47	0.45
	45	0.97	0.92	0.88	0.85	0.83	0.81
	42.5	0.99	0.96	0.95	0.94	0.93	0.92
	40	1.00	1.00	1.00	1.00	1.00	1.00
5	55	0.21	0.17	0.14	0.12	0.10	0.08
	52.5	0.36	0.31	0.27	0.24	0.22	0.20
	50	0.57	0.49	0.45	0.41	0.39	0.37
	45	0.93	0.84	0.79	0.75	0.72	0.70
	42.5	0.97	0.90	0.87	0.85	0.83	0.82
	40	0.98	0.96	0.95	0.94	0.93	0.92
	35	1.00	1.00	1.00	1.00	1.00	1.00
6	55	0.12	0.10	0.08	0.06	0.05	0.04
	52.5	0.27	0.22	0.18	0.16	0.14	0.13
	50	0.48	0.40	0.35	0.31	0.28	0.26
	45	0.49	0.76	0.68	0.63	0.59	0.55
	42.5	0.93	0.84	0.78	0.74	0.70	0.67
	40	0.97	0.91	0.87	0.83	0.81	0.78
	35	1.00	0.96	0.95	0.93	0.91	0.90
	30	1.00	0.99	0.99	0.98	0.98	0.98
7	55	0.00	0.00	0.00	0.00	0.00	0.00
	52.5	0.13	0.10	0.08	0.07	0.05	0.04
	50	0.34	0.27	0.22	0.18	0.15	0.12
	45	0.77	0.65	0.55	0.46	0.40	0.35
	42.5	0.86	0.75	0.67	0.60	0.54	0.48
	40	0.93	0.82	0.75	0.69	0.64	0.59
	35	1.00	0.91	0.87	0.82	0.79	0.76
	30	1.00	0.95	0.92	0.90	0.88	0.86

[a] Ratio of flow rate to ideal capacity of 2,800 pcph, assuming passenger-car operation is unaffected by grade.
NOTE: Interpolate for intermediate values of "Percent No Passing Zone"; round "Percent Grade" to next higher integer value.

TABLE 8-8. ADJUSTMENT FACTOR FOR DIRECTIONAL DISTRIBUTION ON SPECIFIC GRADES, f_d

DIRECTIONAL[a] DISTRIBUTION	ADJUSTMENT FACTOR	DIRECTIONAL[a] DISTRIBUTION	ADJUSTMENT FACTOR
100/0	0.58	40/60	1.20
90/10	0.64	30/70	1.50
80/20	0.70	20/80	1.93
70/30	0.78	10/90	2.51
60/40	0.87	0/100	3.32
50/50	1.00		

[a] Upgrade/Downgrade

4. *Adjustment for narrow lanes and/or restricted shoulder width*—The impact of narrow lanes and/or restricted shoulder widths on grades is the same as for general terrain segments. The appropriate factor is selected from Table 8-5, presented previously.

5. *Adjustment for passenger cars on grades*—The v/c ratios of Table 8-7 assume that passenger cars will maintain their speed on grades if unimpeded. Recent studies *(1,2)* have indicated that passenger-car operation is affected by grades, even where heavy vehicles are not present in the traffic stream. The factor f_g adjusts the v/c ratios of Table 8-7 to account for this effect. The factor is computed as:

$$f_g = 1/[1 + (P_p I_p)] \tag{8-4}$$

where:

f_g = adjustment factor for the operation of passenger cars on grades;

P_p = proportion of passenger cars in the upgrade traffic stream, expressed as a decimal;

I_p = impedance factor for passenger cars, computed as:

$$I_p = 0.02\,(E - E_o) \tag{8-5}$$

E = base passenger-car equivalent for a given percent grade, length of grade, and speed, selected from Table 8-9; and

E_o = base passenger-car equivalent for 0 percent grade and a given speed, selected from Table 8-9.

The passenger-car equivalents of Table 8-9 are used for both the passenger-car and heavy vehicle adjustment factors. The passenger-car factor adjusts from the base v/c ratios, which assume no operational impact of grades on cars, to prevailing conditions of grade. The heavy vehicle adjustment factor is based on passenger-car equivalents related to passenger cars operating on the grade specified.

6. *Adjustment for heavy vehicles in the traffic stream*—The adjustment factor for heavy vehicles is computed as follows:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)] \tag{8-6}$$

where:

f_{HV} = adjustment factor for the presence of heavy vehicles in the upgrade traffic stream;

P_{HV} = total proportion of heavy vehicles (trucks + RV's + buses) in the upgrade traffic stream;

E_{HV} = passenger-car equivalent for specific mix of heavy vehicles present in the upgrade traffic stream, computed as:

$$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1) \tag{8-7}$$

$P_{T/HV}$ = proportion of trucks among heavy vehicles, i.e., the proportion of trucks in the traffic stream divided by the total proportion of heavy vehicles in the traffic stream; and

E = base passenger-car equivalent for a given percent grade, length of grade, and speed, selected from Table 8-9.

The passenger-car equivalents presented in Table 8-9 represent an average mix of trucks, recreational vehicles, and buses in the traffic stream. This average mix is for 14 percent trucks, 4 percent RV's, and no buses. The values of E_{HV} computed by this procedure yield equivalent volumes which travel at the same average overall speed as the actual mixed traffic stream under stable flow conditions. Any tendency of vehicles to stall or perform sluggishly at high volume levels and power requirements is not accounted for in these procedures.

The existence of heavy vehicles on two-lane highway grades is a particularly difficult problem, because an increase in formation of platoons is caused at the same time as passing restrictions usually also increase. Thus, the decision of whether to provide a climbing lane for heavy vehicles is often a critical one for extended grades on two-lane highways. A common criterion sometimes used in the design of grades is to include a climbing lane where the operating speed of trucks falls 10 mph or more *(10)*. Figures 8-2 and 8-3 show speed reduction curves for a 200-lb/hp truck and a 300-lb/hp truck. The former is considered indicative of a representative truck for the average mix of trucks occurring on two-lane highways. The latter is representative of a "heavy" truck, such as heavily loaded farm vehicles, coal carriers, gravel carriers, or log carriers. The choice of which type of truck should be used is based on safety considerations. Speed reduction is related to the steepness and length of the grade in Figures 8-2 and 8-3. For a more detailed depiction of the operating characteristics of trucks on extended upgrades, the truck performance curves included in Appendix I of Chapter 3 may be consulted.

In addition to the 10-mph speed reduction criterion, a climbing lane might be considered wherever a level-of-service analysis indicates a serious deterioration in operating quality on an extended grade when compared to the adjacent approach segment of the same highway.

Heavy vehicles in the traffic stream on extended grades also cause delay to other vehicles. Delay can be evaluated as the difference in travel time between what vehicles could achieve if unimpeded by heavy vehicles and the travel time actually experienced in the mixed traffic stream. Sample calculations illustrate the computation of this delay.

7. *Capacity of specific grade segments*—Sections 1 through 6 above describe the computation of service flow rates on specific two-lane highway grades. For levels-of-service A through D, this is a simple process. The speed relating to the desired LOS

TABLE 8-9. PASSENGER-CAR EQUIVALENTS FOR SPECIFIC GRADES ON TWO-LANE RURAL HIGHWAYS, E AND E_o

GRADE (%)	LENGTH OF GRADE (MI)	AVERAGE UPGRADE SPEED (MPH)					
		55.0	52.5	50.0	45.0	40.0	30.0
0	All	2.1	1.8	1.6	1.4	1.3	1.3
3	¼	2.9	2.3	2.0	1.7	1.6	1.5
	½	3.7	2.9	2.4	2.0	1.8	1.7
	¾	4.8	3.6	2.9	2.3	2.0	1.9
	1	6.5	4.6	3.5	2.6	2.3	2.1
	1½	11.2	6.6	5.1	3.4	2.9	2.5
	2	19.8	9.3	6.7	4.6	3.7	2.9
	3	71.0	21.0	10.8	7.3	5.6	3.8
	4	ª	48.0	20.5	11.3	7.7	4.9
4	¼	3.2	2.5	2.2	1.8	1.7	1.6
	½	4.4	3.4	2.8	2.2	2.0	1.9
	¾	6.3	4.4	3.5	2.7	2.3	2.1
	1	9.6	6.3	4.5	3.2	2.7	2.4
	1½	19.5	10.3	7.4	4.7	3.8	3.1
	2	43.0	16.1	10.8	6.9	5.3	3.8
	3	ª	48.0	20.0	12.5	9.0	5.5
	4	ª	ª	51.0	22.8	13.8	7.4
5	¼	3.6	2.8	2.3	2.0	1.8	1.7
	½	5.4	3.9	3.2	2.5	2.2	2.0
	¾	8.3	5.7	4.3	3.1	2.7	2.4
	1	14.1	8.4	5.9	4.0	3.3	2.8
	1½	34.0	16.0	10.8	6.3	4.9	3.8
	2	91.0	28.3	17.4	10.2	7.5	4.8
	3	ª	ª	37.0	22.0	14.6	7.8
	4	ª	ª	ª	55.0	25.0	11.5
6	¼	4.0	3.1	2.5	2.1	1.9	1.8
	½	6.5	4.8	3.7	2.8	2.4	2.2
	¾	11.0	7.2	5.2	3.7	3.1	2.7
	1	20.4	11.7	7.8	4.9	4.0	3.3
	1½	60.0	25.2	16.0	8.5	6.4	4.7
	2	ª	50.0	28.2	15.3	10.7	6.3
	3	ª	ª	70.0	38.0	23.9	11.3
	4	ª	ª	ª	90.0	45.0	18.1
7	¼	4.5	3.4	2.7	2.2	2.0	1.9
	½	7.9	5.7	4.2	3.2	2.7	2.4
	¾	14.5	9.1	6.3	4.3	3.6	3.0
	1	31.4	16.0	10.0	6.1	4.8	3.8
	1½	ª	39.5	23.5	11.5	8.4	5.8
	2	ª	88.0	46.0	22.8	15.4	8.2
	3	ª	ª	ª	66.0	38.5	16.1
	4	ª	ª	ª	ª	ª	28.0

ª Speed not attainable on grade specified.
NOTE: Round "Percent Grade" to next higher integer value.

is selected from Table 8-2, and appropriate adjustment factors are selected for use in Eq. 8-3.

The service flow rate at capacity, i.e., SF_E, is not as easily determined, because the speed at which it occurs varies depending on the percent and length of the grade in question. For the normal range of grades, i.e., 3 to 7 percent up to 4 miles long, capacity may occur at speeds ranging from 25 to 40 mph. The speed at which capacity occurs is related to the flow rate at capacity by the following equation:

$$S_c = 25 + 3.75(v_c/1000)^2 \qquad (8\text{-}8)$$

where:

S_c = speed at which capacity occurs, in mph; and

v_c = flow rate at capacity, in mixed vph.

For convenience, the equation predicts *upgrade* speeds based on total two-way flow rates. The equation is valid for speed up to 40 mph.

If the service flow rates computed for various speeds using Eq. 8-3 and the capacity speed vs. capacity flow rate relationship of Eq. 8-8 are plotted, the two curves will intersect. The inter-

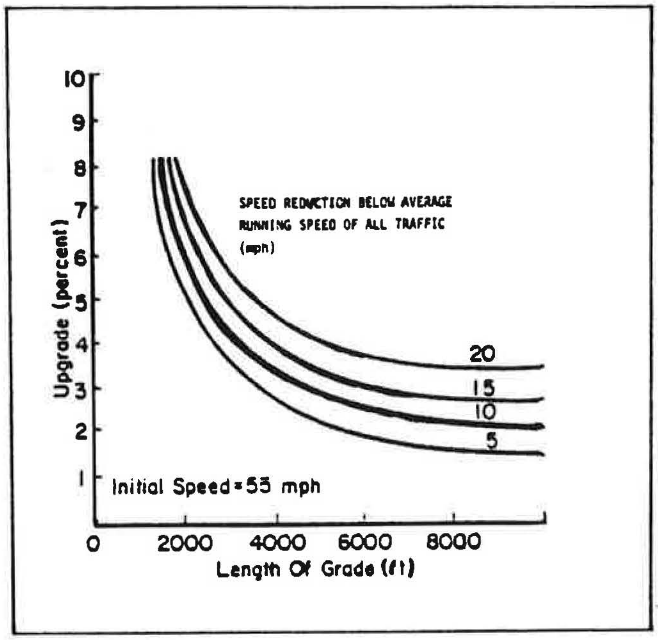


Figure 8-2. Speed reduction curve for a 200-lb/hp truck.

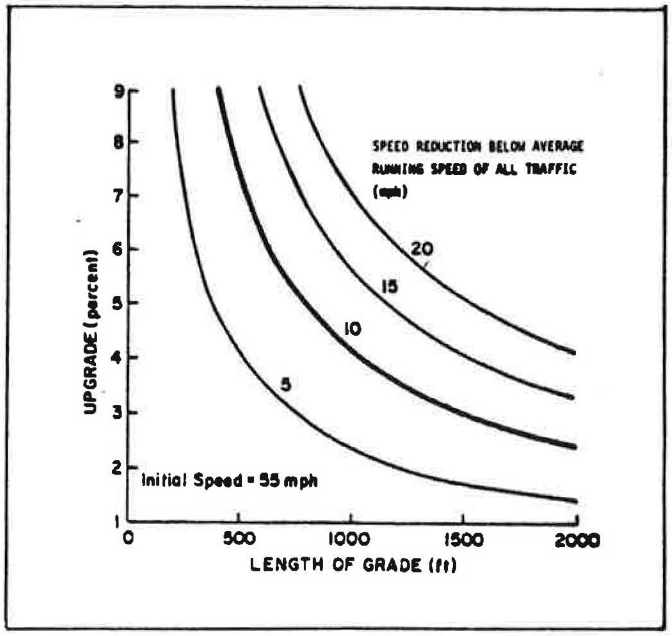


Figure 8-3. Speed reduction curve for a 300-lb/hp truck.

section defines both the speed at capacity and the flow rate at capacity for the grade in question. This procedure for determining capacity is illustrated in the sample calculations.

HIGHWAY SYSTEM PLANNING

The planning procedure enables highway operating agencies to perform very general planning and policy studies of a rural two-lane highway system. Traffic, geometric, and terrain data would be only generally classified, with traffic demand expressed in terms of an average annual daily traffic (AADT), perhaps of some future forecast year.

Table 8-10 presents estimated maximum AADT's for two-lane highways as related to:

1. Level of service.
2. Type of terrain.
3. Design hour factor, K.

The levels of service refer to operating conditions within the peak 15-min period of the day. In constructing Table 8-10, the default values of the peak hour factor (PHF) shown in Table 8-3 were assumed. For each level of service, the related percent time delay criteria were applied across all three types of terrain. The planning criteria also assume a typical traffic mix of 14 percent trucks, 4 percent RV's, and no buses. A 60/40 directional split is used, along with percent no passing zone values of 20 percent, 40 percent, and 60 percent for level, rolling, and mountainous terrain, respectively. Ideal geometrics of 12-ft lanes, 6-ft shoulders, and 60-mph design speed were used.

The AADT's presented in Table 8-10 illustrate a wide range of conditions, and were computed from service flow rates as follows:

$$AADT_i = SF_i \times \text{PHF}/K \tag{8-9}$$

where:

$AADT_i$ = the maximum AADT for level-of-service i, based on the assumed conditions described above; vpd;
SF_i = maximum service flow rate for level-of-service i, computed from Eq. 8-3, based on the assumed conditions described above, in vph;
PHF = peak hour factor, selected from Table 8-3 for the indicated level of service; and
K = design hour factor, i.e., the proportion of AADT expected to occur in the design hour.

The K-factor is normally expressed in design problems as $DHV = AADT \times K$, where the DHV is the total two-way design hour volume, and K is estimated from the ratio of the 30th HV to the $AADT$ from a similar site. The 30th HV is the 30th highest hourly volume during the year and is often used as a design volume for rural highways. Since the DHV should be less than SF_i for the selected level of service, the actual $AADT$ for a road should be less than the maximum value shown in Table 8-10. Traffic conditions occurring during the *highest* hourly volume of the year (1st HV) would usually be no worse than one level of service less than that existing for the 30th HV for most rural highways.

TABLE 8-10. MAXIMUM AADT'S VS. LEVEL OF SERVICE AND TYPE OF TERRAIN FOR TWO-LANE RURAL HIGHWAYS

K-FACTOR	LEVEL OF SERVICE				
	A	B	C	D	E
			LEVEL TERRAIN		
0.10	2,400	4,800	7,900	13,500	22,900
0.11	2,200	4,400	7,200	12,200	20,800
0.12	2,000	4,000	6,600	11,200	19,000
0.13	1,900	3,700	6,100	10,400	17,600
0.14	1,700	3,400	5,700	9,600	16,300
0.15	1,600	3,200	5,300	9,000	15,200
			ROLLING TERRAIN		
0.10	1,100	2,800	5,200	8,000	14,800
0.11	1,000	2,500	4,700	7,200	13,500
0.12	900	2,300	4,400	6,600	12,300
0.13	900	2,100	4,000	6,100	11,400
0.14	800	2,000	3,700	5,700	10,600
0.15	700	1,800	3,500	5,300	9,900
			MOUNTAINOUS TERRAIN		
0.10	500	1,300	2,400	3,700	8,100
0.11	400	1,200	2,200	3,400	7.300
0.12	400	1,100	2,000	3,100	6,700
0.13	400	1,000	1,800	2,900	6,200
0.14	300	900	1,700	2,700	5,800
0.15	300	900	1,600	2,500	5,400

NOTE: All values rounded to the nearest 100 vpd. Assumed conditions include 60/40 directional split, 14 percent trucks, 4 percent RV's, no buses, and PHF values from Table 8-3. For level terrain, 20 percent no passing zones were assumed; for rolling terrain, 40 percent no passing zones; for mountainous terrain, 60 percent no passing zones.

III. PROCEDURES FOR APPLICATION

The methodology described in the previous section is generally applied in either the *operational analysis* or *planning* modes.

Design computations, as used in this manual, focus on the determination of the number of lanes required for a given facility. Such computations have little significance for two-lane highways, where the number of lanes is fixed. Such design features as horizontal and vertical alignment, however, have a significant impact on operations. Operational analyses can be performed for alternative designs to document this impact. Where computations indicate that a two-lane highway is not adequate for existing or projected demands, various multilane options may be considered and analyzed using other chapters of this manual.

A separate section of this chapter deals with operational and design measures for two-lane highways, short of reconstructing the entire highway as a multilane facility. This material should be consulted where a two-lane facility presently has or is expected to experience operational difficulties.

OPERATIONAL ANALYSIS OF GENERAL TERRAIN SEGMENTS

The objective in operational analysis is to determine the level of service for a given segment or segments of roadway for a known existing set of conditions, or for a future set of conditions which are hypothesized and/or forecast. The general approach will be to compute service flow rates for each level of service and compare these values with the existing flow rate on the facility. This is done using Eq. 8-1:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$$

where all terms are as previously defined. A service flow rate for each LOS is computed because the heavy vehicle factor varies with LOS, and a direct solution of the equation for v/c ratio would be iterative. Users preferring to solve for v/c may do so, but must iterate until the assumed LOS used in computing the heavy vehicle factor is the same as that indicated by the v/c ratio found.

In general, the following computational steps are used. Computations may be conveniently performed on the worksheet illustrated in Figure 8-4.

1. Summarize all input data on traffic and roadway conditions, including:
   - Existing or forecast peak hour volume, in vph.
   - Peak hour factor, PHF, from local data or default value selected from Table 8-3.
   - Traffic composition (% trucks, % RV's, % buses).

WORKSHEET FOR GENERAL TERRAIN SECTIONS

Site Identification:__________ Date:______ Time:______

Name:__________ Checked by:__________

I. GEOMETRIC DATA

NORTH

shoulder ____ft

____ft

shoulder ____ft

Design Speed:_____mph
% No Passing _____%
Terrain (L,R,M):_____
Segment Length_____mi

II. TRAFFIC DATA

Total Volume, Both Dir._____vph

Flow Rate = Volume ÷ PHF
_____ = _____ ÷ ____

Directional Distribution:_______
Traffic Composition:___%T,___%RV,___%B
PHF:______

III. LEVEL OF SERVICE ANALYSIS

$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$

$f_{HV} = 1 / [1 + P_T(E_T-1) + P_R(E_R-1) + P_B(E_B-1)]$

LOS	SF =	2,800x	(v/c)x	f_d x	f_w x	f_{HV}	P_T	E_T	P_R	E_R	P_B	E_B
			TAB 8-1	TAB 8-4	TAB 8-5			TAB 8-6		TAB 8-6		TAB 8-6
A		2,800										
B		2,800										
C		2,800										
D		2,800										
E		2,800										

IV. COMMENTS Flow Rate______vph LOS=______

Figure 8-4. Worksheet for operational analysis of general terrain segments.

- Directional distribution of traffic.
- Terrain type.
- Lane and usable shoulder widths, in ft.
- Design speed, in mph.

2. Select appropriate values of the following factors for *each* LOS from the tables indicated:
   - The v/c ratio from Table 8-1.
   - The directional distribution factor, f_d, from Table 8-4.
   - The lane width and shoulder width factor, f_w, from Table 8-5.
   - Passenger-car equivalents, E_T, E_R, and E_B, for trucks, RV's, and buses, from Table 8-6.
3. Compute the heavy vehicle factor, f_{HV}, for each LOS from:

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)]$$

4. Compute the service flow rate, *SF*, for each LOS from:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$$

5. Convert the existing or forecast volume to an equivalent flow rate, as follows: $v = V/\text{PHF}$.
6. Compare the actual flow rate of step 5 with the service flow rate of step 4 to determine the level of service.

Where the level of service is found to be inadequate, the alleviation measures presented in the next section should be considered, as well as the expansion of the facility to four or more lanes. Expansion to a multilane facility should be examined using the methodology presented in Chapter 7.

OPERATIONAL ANALYSIS OF SPECIFIC GRADES

The operational analysis of specific grades is similar to the procedure for general terrain segments. The level of service for the upgrade direction is sought, and is found by comparing an actual two-way flow rate to the service flow rates for the various levels of service. As noted in the "Methodology" section, however, the determination of capacity for specific grades requires the plotting of a service flow rate-speed curve, and a curve representing the relationship of speed at capacity to flow rate at capacity. The worksheet shown in Figure 8-5 is used to simplify the following computational steps.

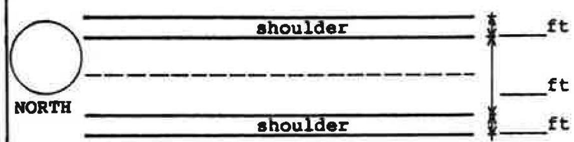
1. Summarize all required input data on traffic and roadway conditions, including:
   - Existing or forecast peak hour volume, in vph.
   - Peak hour factor, PHF, from local data or default value from Table 8-3.
   - Traffic composition (% trucks, % RV's, % buses, % passenger cars).
   - Directional distribution of traffic.

WORKSHEET FOR SPECIFIC GRADES — Page 1

Site Identification:____________________ Date:________ Time:________

Name:____________________ Checked by:________________

I. GEOMETRIC DATA



Design Speed:______mph
Grade : ____%, ____mi.
% No Passing Zones____

II. TRAFFIC DATA

Total Volume, Both Dir.______vph

Flow Rate = Volume ÷ PHF

______ = ______ ÷ ___

Directional Distribution:__________
Traffic Composition:___%T,___%RV,___%B

III. SOLVING FOR ADJUSTMENT FACTORS f_g AND f_{HV}

$f_g = 1 / [1 + P_p I_p]$

$I_p = 0.02 (E - E_o)$

$f_{HV} = 1 / [1 + P_{HV} (E_{HV} - 1)]$

$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1)$

Speed	P_p	I_p	E TAB 8-9	E_o TAB 8-9	f_g	P_{HV}	E_{HV}	$P_{T/HV}$ (P_T/P_{HV})	E TAB 8-9	f_{HV}
55										
52.5										
50										
45										
40										
30										

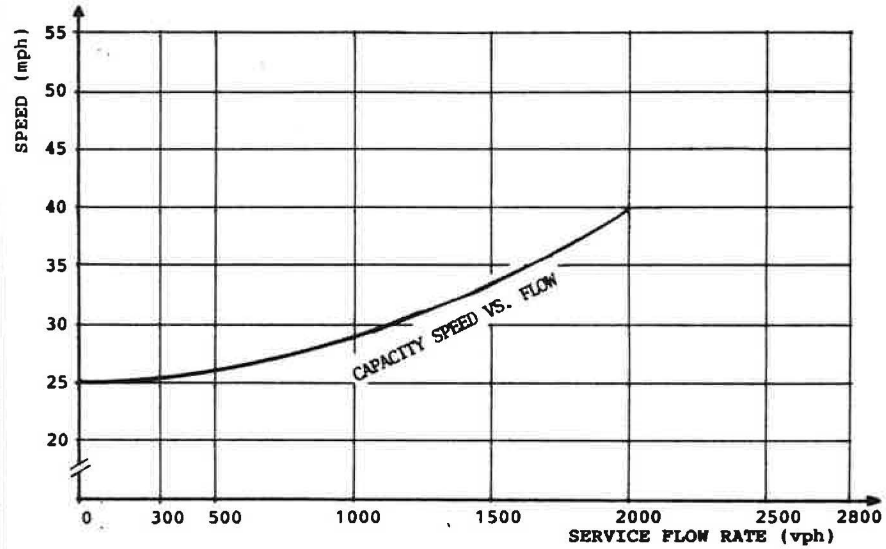
IV. SOLVING FOR SERVICE FLOW RATE

Speed (mph)	SF	2,800 x	v/c x TAB 8-7	f_d x TAB 8-8	f_w x TAB 8-5	f_g x	f_{HV}
55 - LOS A							
52.5							
50 - LOS B							
45 - LOS C							
40 - LOS D							
30							

Figure 8-5. Worksheet for operational analysis of specific grades on two-lane highways (page 1).

WORKSHEET FOR SPECIFIC GRADES — Page 2

V. PLOT SF vs Speed



Intersection of Capacity Speed vs Flow curve with Service Flow Rate vs Speed curve defines Capacity (SF_E) and Speed at Capacity (S_c)

VI. LEVEL OF SERVICE ANALYSIS

LOS	SF (from Worksheet)	Actual Flow Rate	Comments:
A			
B			
C		Level of Service	
D			
E			

Figure 8-5. Worksheet for operational analysis of specific grades on two-lane highways (page 2).

- Percent grade.
- Percent no passing zones.
- Length of grade, in miles.
- Lane and usable shoulder width, in ft.
- Design speed, in mph.

2. Select values of the following factors from the indicated tables for the following average speeds: 55 mph (LOS A), 52.5 mph, 50 mph (LOS B), 45 mph (LOS C), 40 mph (LOS D), and 30 mph. This range of speeds will allow the plotting of a service flow rate vs. speed curve to find capacity and the speed at capacity.

   - The v/c ratio from Table 8-7.
   - The directional distribution factor, f_d, from Table 8-8.
   - The lane and shoulder width factor, f_w, from Table 8-5.
   - The passenger-car equivalent, E, for the percent and length of grade, from Table 8-9.
   - The passenger-car equivalent, E_o, for a 0 percent grade, from Table 8-9.

3. Compute the grade factor, f_g, as follows:

$$f_g = 1/[1 + P_p I_p]$$
$$I_p = 0.02(E - E_o)$$

   where all values are as previously defined.
4. Compute the heavy vehicle factor, f_{HV}, for each of the speeds noted in step 2 as follows:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)]$$
$$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1)$$
$$P_{T/HV} = P_T/[P_T + P_R + P_B]$$

   where all values are as previously defined.
5. Compute the service flow rate, SF, for each of the speeds noted in step 2 as follows:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV}$$

6. Plot the service flow rates vs. speeds resulting from the computations of steps 2–5 on the grid included in the worksheet of Figure 8-5. Note that the curve for speed at capacity vs. flow rate at capacity is already drawn on this grid.
7. Find the speed at capacity and the service flow rate at capacity from the intersection of the two curves on the plot of step 6.
8. Summarize the service flow rates for each level of service on the worksheet as indicated.
9. Convert the actual or forecast volume to a flow rate, as follows: $v = V/\text{PHF}$.
10. Compare the actual flow rate of step 9 with the service flow rates of step 8 to determine the level of service.

As with general terrain segments, a two-lane highway grade displaying unacceptable operating conditions would be considered for improvement. If heavy vehicles on the upgrade are the principal difficulty, the addition of a truck climbing lane should be considered. If operational problems are more broad-based, any of the alleviation techniques discussed in the next section could be considered, as well as expansion of the facility to four or more lanes. Again, the multilane option would be examined using procedures in Chapter 7.

PLANNING

The highway system planning technique described in the "Methodology" section is easily applied. Table 8-10 may be entered with a known or forecast AADT to determine expected level of service during the peak 15 min of flow, or with a known LOS to find the maximum allowable AADT. No computations are needed to use this table, although users are cautioned that any conditions varying widely from those noted in the footnotes to Table 8-10 will indicate the need to conduct an operational analysis for the facility in question.

Users may also find Table 8-10 useful in making preliminary estimates of LOS in general section analysis.

IV. DESIGN AND OPERATIONAL TREATMENTS

Addressing those operational problems that may exist on rural two-lane highways requires an understanding of the nature of two-lane highway systems. Only about 30 percent of all travel in the United States occurs on rural two-lane roads, even though this network comprises 80 percent of all paved rural highways. For the most part, two-lane highways carry light traffic and experience few operational problems. Highway agencies are typically more concerned with pavement maintenance and roadside safety issues on such highways.

Some two-lane highways, however, periodically experience severe operational and safety problems due to a variety of traffic, geometric, and environmental causes. Special treatments for such highways may be needed before capacity levels are approached. In some areas, the two-lane rural arterial system carries a disproportionately large share of rural traffic, including significant components involved in interstate commerce. Many of these highways are located near major urban areas and are experiencing rapid growth in traffic. Heavy turning movements to roadside developments can block through traffic and increase delay.

As much as 60 percent of all two-lane highway mileage is located in terrain classified as rolling or mountainous. This, coupled with occasionally high opposing volumes, is not favorable to either passing or turning maneuvers. When these and other rural highways experience increased recreational travel, major operational problems may arise. Large numbers of recreational and other heavy vehicles in the traffic stream increase the demand for passing, while at the same time, making such maneuvers more difficult. Two-lane highways serving as major routes to recreational areas may operate at or near capacity on weekends in peak seasons.

When any of the foregoing situations exist, the frequent result is a reduced level of service, increased platooning, increased delay, an increase in questionable passing maneuvers, and generally frustrated drivers. Nevertheless, many such situations do not justify the reconstruction of the two-lane highway to a full multilane facility. In these cases, one or more of the special design and/or operational treatments discussed in this section may be useful.

A wide range of design and operational solutions are needed to address the variety of problems encountered on two-lane highways. The operational and/or safety problems on a particular section may be so severe as to call for an expansion of the facility to four or more lanes. However, limited reconstruction funds, difficult terrain, and other problems may not always permit full reconstruction of a two-lane facility as a multilane highway. Less costly and less environmentally disruptive solutions may be required. Highways experiencing less severe operational and/or safety problems, together with those experiencing site-specific reductions in level of service, may be candidates for treatment with one or more of the following alleviation techniques:

1. Realignment to improve passing sight distance.
2. Use of paved shoulders.
3. Three-lane roadways with two lanes designated for travel in one direction (passing prohibited or permitted in opposing direction).
4. Three-lane road sections with continuous two-way median left-turn lanes.
5. Three-lane roadway with reversible center lane.
6. Special intersection treatments.
7. Truck or heavy vehicle climbing lanes.
8. Turnouts.
9. Short four-lane segments.

Selection of the appropriate treatment requires identification of the probable causes of the operational and safety problems existing, and the determination of cost-effectiveness of the design alternatives for a given set of highway geometric, traffic, and system constraints. The following discussions address the use of alleviation measures on two-lane highways. They are intended to provide the user with general information, and should not be construed as firm guidelines or criteria.

PASSING SIGHT DISTANCE

The opportunity to pass, given a constant volume, is a function of the availability of passing sight distance. Provision of passing sight distance is an important component in basic two-lane highway design and, as illustrated by Tables 8-1 and 8-7, has a critical impact on capacity and service flow rate. Where long queues are likely to form because of severe passing restrictions, every effort should be made to continuously and completely disperse the platoon once significant passing sight distance is regained. In these passing sections, short segments with passing sight distance restrictions should be avoided where possible. Inclusion of periodic passing lanes for each direction should be considered where the distance between segments with passing sight distance available is long and queuing extensive.

PAVED SHOULDERS

A roadway that is constructed with structurally adequate paved shoulders can be used to assist in dispersal and breakup of platoons. Slower moving vehicles may temporarily use the shoulder to permit faster vehicles to pass, returning to the travel lane when passing maneuvers have been completed. In Texas and Canada, where some agencies construct wide shoulders for a total roadway width of 40 to 44 ft, a high percentage of the driving population uses the shoulder in this manner—particularly in western Canada where long distance recreational travel is heavy during the summer. Illustration 8-2 presents a typical use of paved shoulders as described previously.

Five states allow the use of shoulders for slow-moving vehicles at all times. An additional ten states permit such use under specified conditions.

THREE-LANE HIGHWAYS

Three-lane roadways are a rational intermediate solution to four-lane expansions for two-lane highways experiencing operational problems. Because of funding and terrain constraints, three-lane roadways may be considered for spot and segment improvements. There are numerous methods for using the third travel lane on such segments.

In the 1940's and 1950's, the third (center) lane was used for passing by vehicles in either direction—the first vehicle to occupy the center lane had the right-of-way. This condition was found to be hazardous, particularly in hilly terrain. This use of three-lane highways in the United States has been generally discontinued.

Other three-lane highway treatments are being safely and efficiently applied, including the use of passing lanes, turning lanes, and climbing lanes.

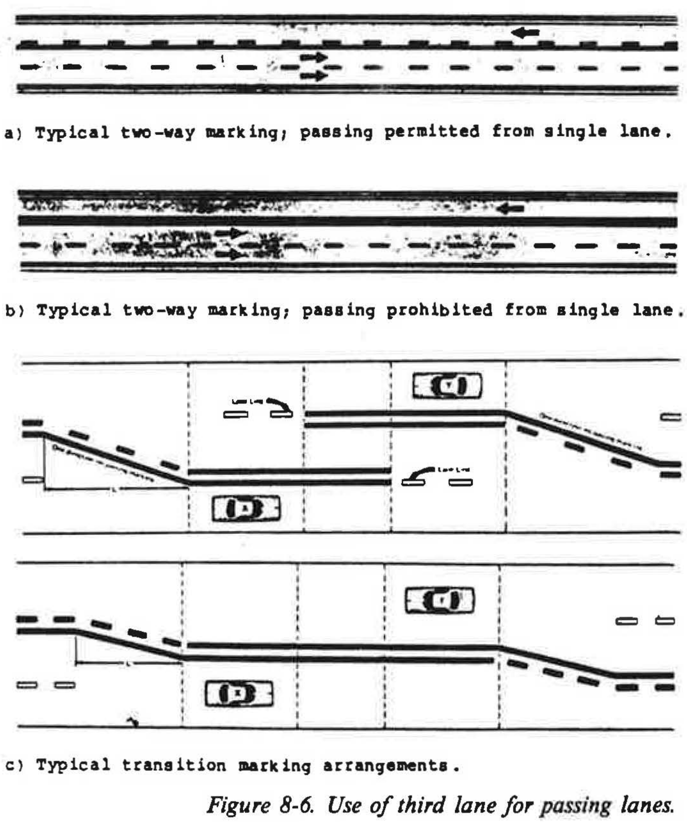
Passing Lanes

This three-lane roadway design assigns the third (center) lane to one direction of travel for a short distance (approximately 1 mile), then alternates the assignment of the passing lane to the other direction. This cyclic process may be continued along an entire highway section, or may be combined in an urban fringe area with two-way left-turn lanes and/or specific intersection turning treatments.

In a rural setting, intermittently spaced passing lane sections have been successfully used to break up platoons and reduce delay. Two lanes are provided for unimpeded passing in one direction for 1 to 2 miles followed by a transition to two lanes

Illustration 8-2. Slow-moving vehicle uses the shoulder of a two-lane rural highway, permitting faster vehicles to pass.

of similar design for the opposing flow. Advance signing advises motorists of the next upcoming passing lane to reduce driver anxiety and frustration. Two operational markings are practiced: passing in the single-lane direction may be permitted if passing sight distance is available, or passing in the single-lane direction may be prohibited. Figure 8-6 depicts these markings, and various methods of providing for the transition when the direction of the passing lane is changed. Permissive passing for the one-lane direction is not used by some agencies when the AADT exceeds about 3,000 vpd.



a) Typical two-way marking; passing permitted from single lane.

b) Typical two-way marking; passing prohibited from single lane.

c) Typical transition marking arrangements.

Figure 8-6. Use of third lane for passing lanes.

TABLE 8-11. SPACING OF PASSING LANES ON TWO-LANE HIGHWAYS

Two-Way Peak Hourly Volume (vph)	400	300	200
Distance to Next Passing Lane (miles)	5	6.5	9

An analytic study of passing lane requirements was conducted in Ontario, Canada (*7*). This study recommended that passing lanes should consistently be from 1.0 to 1.25 miles long. This length was found to be adequate to disperse most platoons, to provide for additional transition zones, and yet not be too long to change drivers' expectations about the true nature of the highway. Table 8-11 gives the recommended spacing between passing lanes in a given direction which resulted from the study.

Continuous Two-Way Median Left-Turn Lanes

On two-lane highways having sizable left-turn traffic, a single travel lane in each direction often becomes subject to long delays as vehicles await opportunities to complete left turns. By providing a continuous refuge area for left-turning traffic, the two-way left-turn lane can help to maintain through traffic capacity, with the added benefit of separating opposing flows. The ability to pass, however, is eliminated.

Two-way left-turn lanes are not usually used where speeds are less than 25 mph or more than 50 mph, and are most often used in urban fringe areas or on a major route passing through a small town or village.

Reversible Lane

This is another use of the third (center) lane of a three-lane highway which is most applicable where travel demands are of a tidal nature—that is, extreme directional splits occur. The center lane is reversed by time of day to match the peak flow. The center lane is controlled by overhead signs or traffic signals indicating the direction of travel assigned at the time. Passing is not permitted in this application in the direction of the single lane.

The reversible lane technique is most applicable to routes joining residential areas and high-employment centers, and for many recreational routes.

Intersection Treatments

Conventional analysis of two-lane highways assumes uninterrupted flow, which is normally representative of rural conditions. With increasing development occurring in some rural areas, and in suburban fringe areas, the demand for high-volume access and egress can grow. Major intersections along two-lane highways become more common and important to the overall quality of flow on main routes. Adequate protected turning lanes for both left and right turns are useful in minimizing disruption to through traffic. Bypass lanes for through traffic may be considered where a protected left-turn lane is not feasible, particularly where paved shoulders are provided and/or where T-intersections are involved.

Detailed analysis of intersections may be performed using the procedures of Chapter 9, "Signalized Intersections," and Chapter 10, "Unsignalized Intersections."

Climbing Lanes

Traditional climbing lanes also form three-lane cross sections when used in conjunction with two-lane highways. They are generally applied as a spot improvement, most often on steep, sustained grades which cause heavy vehicles, particularly heavy trucks, to travel at slow speeds. This reduces capacity, creates platoons, and increases delay. Additionally, safety problems may arise when the reduction in speed of heavy trucks exceeds 10 mph along the grade.

Estimated operating speed characteristics of trucks are illustrated in Figures I.3-1, I.3-2, and I.3-3 in Appendix I of Chapter 3. Resulting lengths of grade producing 10-mph speed reductions are plotted in Figures 8-2 and 8-3, presented earlier in this chapter. AASHTO presently warrants a climbing lane wherever the speed of a 300-lb/hp truck is reduced by 10 mph or more. Recent studies, however, indicate that a 200-lb/hp truck is more representative of existing rural truck populations.

Climbing lanes are generally considered as an alleviation treatment wherever the following criteria are met:

1. Upgrade traffic flow rate exceeds 200 vph.
2. Upgrade truck flow rate exceeds 20 vph.
3. One of the following conditions exist:
   - Level-of-service E or F exists on the Grade.
   - A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.
   - A 10-mph or greater speed reduction is expected for a typical heavy truck.

It is noted that these are not absolute criteria, but are intended as general guides for the consideration of climbing lanes on grades. They apply only to climbing lanes on two-lane highways and should not be used in conjunction with consideration of climbing lanes on multilane highways.

Turnouts

The use of turnouts for improving the level of service on two-lane, two-way highways is more prevalent in the rolling and mountainous terrain of the western United States. Turnouts are short segments of a third lane added to one side of the highway or the other which permit slow vehicles at the head of platoons to pull off the main roadway, allowing faster vehicles to pass. Turnouts are used satisfactorily on both upgrades and downgrades, as well as on level terrain, to improve traffic flow. Impeding motorists are legally required to use turnouts where provided under certain prescribed conditions, which vary by state.

A recent study of operational characteristics revealed that few drivers actually stop at turnouts (*8*). Several additional conclusions drawn from this study included:

1. Turnouts are safe when properly used.
2. A series of turnouts at regular intervals can provide considerable delay reduction.

3. Turnouts are not a substitute for a passing or climbing lane of adequate length.
4. About 10 percent of all platoon leaders use properly designated turnouts.
5. Large trucks tend to avoid turnouts.

Turnouts are a short but functional treatment of irritating causes of operational delay. A western state recommends that the length of turnouts vary with approach speed according to the criteria of Table 8-12 (*9*):

Approach speeds of potential turnout-users vary with prevailing traffic and roadway conditions, and differ between upgrades and downgrades. Turnout lengths of more than 500 ft are only used on downgrades exceeding 3 percent where high approach speeds are expected to exist. Lengths greater than 600 ft are never designed, as drivers may mistakenly attempt to use them as passing lanes.

TABLE 8-12. LENGTH OF TURNOUTS ON TWO-LANE HIGHWAYS

Approach Speed (mph)	25	30	40	50	55	60
Minimum Length of Turnout (ft)	200	200	250	375	450	535

SHORT FOUR-LANE SECTIONS

Short sections of four-lane cross section may be constructed along a primarily two-lane highway to break up platoons, to provide the desired frequency of safe passing zones, and to eliminate interference from low-speed vehicles. Such sections are particularly advantageous in rolling terrain, or where the alignment is winding or the profile includes critical grades from both directions. The decision to use a short four-lane segment, as compared to using a three-lane option, may be based on long-range planning objectives for the facility, availability of rights-of-way, existing cross section, topography, and on the desire to reduce platooning and passing problems.

The transition from a two-lane to a four-lane roadway should be designed to provide sufficient sight distance for passing. For the length of four-lane segments, AASHTO suggests that they be sufficiently long to permit several vehicles in line behind a slow-moving vehicle to pass before reaching the normal section of two-lane highway. Four-lane sections of 1.0 to 1.5 miles should be sufficiently long to dissipate most queues formed, depending on volume and terrain conditions. Further, it is noted that sections of four-lane highway, particularly divided sections, longer than 2 miles may cause drivers to lose their sense of awareness that the road is basically a two-lane facility.

V. SAMPLE CALCULATIONS

CALCULATION 1—FINDING SERVICE FLOW RATES FOR A GENERAL TERRAIN SEGMENT

1. *Description*—A segment of rural two-lane highway is expected to have the following characteristics:
   a. *Roadway characteristics*—70-mph design speed; 12-ft lanes; 10-ft paved shoulders; level terrain; 0 percent no passing zones; length = 5 miles.
   b. *Traffic characteristics*—70/30 directional split; 10 percent trucks; 5 percent recreational vehicles; 1 percent buses; 84 percent passenger cars.

What is the capacity of the section? What is the maximum flow rate which can be accommodated at level-of-service C?

2. *Solution*—The solution to this problem is found by computing the service flow rates for levels-of-service C and E (capacity), using Eq. 8.1:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$$

where

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)]$$

The following values are selected for use in these computations:

$(v/c)_C$ = 0.43 (Table 8-1, level terrain 0 percent no passing zones, LOS C);
$(v/c)_E$ = 1.00 (Table 8-1, level terrain, 0 percent no passing zones, LOS E);
f_d = 0.89 (Table 8-4, 70/30 split);
f_w = 1.00 (Table 8-5, 12-ft lanes, > 6-ft shoulders);
E_T = 2.2 for LOS C, 2.0 for LOS E (Table 8-6, level terrain);
E_R = 2.5 for LOS C, 1.6 for LOS E (Table 8-6, level terrain);
E_B = 2.0 for LOS C, 1.6 for LOS E (Table 8-6, level terrain);
P_T = 0.10 (Given);
P_R = 0.05 (Given); and
P_B = 0.01 (Given).

Then:

$$f_{HV}(\text{LOS C}) = 1/[1 + 0.10(2.2 - 1) + 0.05(2.5 - 1) + 0.01(2.0 - 1)] = 0.83$$

$$f_{HV}(\text{LOS E}) = 1/[1 + 0.10(2.0 - 1) + 0.05(1.6 - 1) + 0.01(1.6 - 1)] = 0.88$$

WORKSHEET FOR GENERAL TERRAIN SECTIONS

Site Identification: SMITH BLVD. Date: 9/18/83 Time: 5-6 PM

Name: JOHN JONES Checked by: ______

I. GEOMETRIC DATA

NORTH

shoulder — 10 ft

24 ft

shoulder — 10 ft

Design Speed: 70 mph
% No Passing: 0 %
Terrain (L,R,M): L
Segment Length: 5 mi

II. TRAFFIC DATA

Total Volume, Both Dir. ____ vph

Flow Rate = Volume ÷ PHF
____ = ____ ÷ ____

Directional Distribution: 70/30
Traffic Composition: 10 %T, 5 %RV, 1 %B
PHF: ______

III. LEVEL OF SERVICE ANALYSIS

$S_{Fi} = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$

$f_{HV} = 1 / [1 + P_T(E_T-1) + P_R(E_R-1) + P_B(E_B-1)]$

LOS	SF =	2,800x	(v/c)x TAB 8-1	f_d x TAB 8-4	f_w x TAB 8-5	f_{HV}	P_T	E_T TAB 8-6	P_R	E_R TAB 8-6	P_B	E_B TAB 8-6
A		2,800										
B		2,800										
C	889	2,800	0.43	0.89	1.00	0.83	.10	2.2	.05	2.5	.01	2.0
D		2,800										
E	2193	2,800	1.00	0.89	1.00	0.88	.10	2.0	.05	1.6	.01	1.6

IV. COMMENTS Flow Rate ______ vph LOS= ______

Figure 8-7. Worksheet summarizing solution to Calculation 1.

WORKSHEET FOR GENERAL TERRAIN SECTIONS

Site Identification: Overland Road Date: 9/18/85 Time: 4-5 PM

Name: Thomas Smith Checked by: ______

I. GEOMETRIC DATA

NORTH

shoulder — 2 ft

22 ft

shoulder — 2 ft

Design Speed: 60 mph
% No Passing: 80 %
Terrain (L,R,M): M
Segment Length: 10 mi

II. TRAFFIC DATA

Total Volume, Both Dir. 180 vph

Flow Rate = Volume ÷ PHF
207 = 180 ÷ 0.87

Directional Distribution: 60/40
Traffic Composition: 5 %T, 10 %RV, 0 %B
PHF: 0.87

III. LEVEL OF SERVICE ANALYSIS

$S_{Fi} = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$

$f_{HV} = 1 / [1 + P_T(E_T-1) + P_R(E_R-1) + P_B(E_B-1)]$

LOS	SF =	2,800x	(v/c)x TAB 8-1	f_d x TAB 8-4	f_w x TAB 8-5	f_{HV}	P_T	E_T TAB 8-6	P_R	E_R TAB 8-6	P_B	E_B TAB 8-6
A	23	2,800	0.02	0.94	0.75	0.588	.05	7	.10	5.0		
B	127	2,800	0.12	0.94	0.75	0.535	.05	10	.10	5.2		
C	211	2,800	0.20	0.94	0.75	0.535	.05	10	.10	5.2		
D	371	2,800	0.37	0.94	0.75	0.508	.05	12	.10	5.2		
E	941	2,800	0.80	0.94	0.88	0.508	.05	12	.10	5.2		

IV. COMMENTS Flow Rate 207 vph LOS= C

Figure 8-8. Worksheet summarizing solution to Calculation 2.

and:

$$SF_C = 2{,}800 \times 0.43 \times 0.89 \times 1.00 \times 0.83 = \mathit{889\ vph}$$

$$SF_E = 2{,}800 \times 1.00 \times 0.89 \times 1.00 \times 0.88 = \mathit{2{,}193\ vph}$$

Thus, the highway will have an expected capacity of 2,193 vph, total in both directions, and can accommodate a flow rate of up to 889 vph at level-of-service C. The worksheet for general terrain sections may be used to perform these computations, as shown in Figure 8-7.

CALCULATION 2—FINDING LEVEL OF SERVICE FOR A GENERAL TERRAIN SEGMENT

1. *Description*—A two-lane rural highway carries a peak hour volume of 180 vph and has the following characteristics:
   a. *Roadway characteristics*—60-mph design speed; 11-ft lanes; 2-ft shoulders; mountainous terrain; 80 percent no passing zones; length = 10 miles.
   b. *Traffic characteristics*—60/40 directional split; 5 percent trucks; 10 percent recreational vehicles; no buses; 85 percent passenger cars.

At what level of service will the highway operate during peak periods?

2. *Solution*—The solution is found by comparing the actual flow rate to service flow rates computed for each LOS. The actual flow rate is found as:

$$v = V/\text{PHF}$$

where:

V = 180 vph (Given)
PHF = 0.87 (Default value, Table 8-3, 200 vph)

and:

$$v = 180/0.87 = 207 \text{ vph}$$

Service flow rates are computed from Eq. 8-1:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$$
$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)]$$

where:

v/c = 0.02 for LOS A, 0.12 for LOS B, 0.20 for LOS C, 0.37 for LOS D, 0.80 for LOS E (Table 8-1, mountainous terrain, 80 percent no passing zones);

f_d = 0.94 (Table 8-4, 60/40 split);

f_w = 0.75 for LOS A through D, 0.88 for LOS E (Table 8-5, 11-ft lanes, 2-ft shoulders);

E_T = 7 for LOS A, 10 for LOS B, C, 12 for LOS D, E, (Table 8-6, mountainous terrain);

E_R = 5.0 for LOS A, 5.2 for LOS B-E (Table 8-6, mountainous terrain);

P_T = 0.05 (Given); and

P_R = 0.10 (Given).

Then:

$$f_{HV}(\text{LOS A}) = 1/[1 + 0.05(7 - 1) + 0.10(5.0 - 1)] = 0.588$$
$$(\text{LOS B, C}) = 1/[1 + 0.05(10 - 1) + 0.10(5.2 - 1)] = 0.535$$
$$(\text{LOS D, E}) = 1/[1 + 0.05(12 - 1) + 0.10(5.2 - 1)] = 0.508$$

and:

$$SF_A = 2{,}800 \times 0.02 \times 0.94 \times 0.75 \times 0.588 = 23 \text{ vph}$$
$$SF_B = 2{,}800 \times 0.12 \times 0.94 \times 0.75 \times 0.535 = 127 \text{ vph}$$
$$SF_C = 2{,}800 \times 0.20 \times 0.94 \times 0.75 \times 0.535 = 211 \text{ vph}$$
$$SF_D = 2{,}800 \times 0.37 \times 0.94 \times 0.75 \times 0.508 = 371 \text{ vph}$$
$$SF_E = 2{,}800 \times 0.80 \times 0.94 \times 0.88 \times 0.508 = 941 \text{ vph}$$

If the actual flow rate of 207 vph (which represents the flow rate during the peak 15 min of flow) is compared to these values, it is seen that it is higher than the service flow rate for LOS B (127 vph), but is less than the service flow rate for LOS C (211 vph). Therefore, the level of service for the highway is C for the conditions described.

This problem illustrates several points. On severe terrain, such as the situation for this problem, "good" operating conditions can be sustained only at low flow rates. The capacity of the roadway is also severely limited, reaching only 941 vph, which is approximately one-third of the ideal capacity of 2,800 vph. Note that the v/c ratio used in the computation of capacity is only 0.80. This is because all v/c ratios in the two-lane methodology are referenced to the ideal capacity of 2,800 vph, which cannot be achieved in severe terrain with passing sight distance restrictions.

This solution may be summarized or done on the general terrain section worksheet, as shown in Figure 8-8.

CALCULATION 3—FINDING SERVICE FLOW RATES FOR A SPECIFIC GRADE

1. *Description*—A rural two-lane highway in mountainous terrain has a 6 percent grade of 2 miles. Other relevant characteristics include:
   a. *Roadway characteristics*—12-ft lanes; 8-ft shoulders; 60 percent no passing zones.
   b. *Traffic characteristics*—70/30 directional split; 12 percent trucks; 7 percent recreational vehicles; 1 percent buses, 80 percent passenger cars; PHF = 0.85.

What is the maximum volume which can be accommodated on the grade at a speed of 40 mph (LOS D, Table 8-2)?

2. *Solution*—Service flow rate on specific grades is computed using Eq. 8-3, as follows:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV}$$

where:

$$f_g = 1/[1 + P_p I_p] \text{ from Eq. 8-4}$$

$$I_p = 0.02\,(E - E_o] \text{ from Eq. 8-5}$$

and:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV}(E_{HV} - 1)] \text{ from Eq. 8-6}$$

$$E_{HV} = 1 + (0.25 + P_{T/HV})\,(E - 1) \text{ from Eq. 8-7}$$

The following values are used in these computations:

$(v/c)_D$ = 0.83 (Table 8-7, 40 mph, 6 percent grade, 60 percent no passing zones);
f_d = 0.78 (Table 8-8, 70/30 split, 70 percent upgrade);
f_w = 1.00 (Table 8-5, 12-ft lanes, >6-ft shoulders);
E = 10.7 (Table 8-9, 40 mph, 6 percent for 2-mile grade);
E_o = 1.3 (Table 8-9, 40 mph, 0 percent grade);
$P_{HV} = P_T + P_R + P_B = 0.12 + 0.07 + 0.01 = 0.20$; and
$P_{T/HV} = P_T/P_{HV} = 0.12/0.20 = 0.60$.

Then, computing factors f_g and f_{HV}:

$$I_p = 0.02\,(10.7 - 1.3) = 0.188$$
$$f_g = 1/[1 + (0.80 \times 0.188)] = 0.87$$
$$E_{HV} = 1 + (0.25 + 0.60)\,(10.7 - 1) = 9.25$$
$$f_{HV} = 1/[1 + 0.20(9.25 - 1)] = 0.38$$

The service flow rate for the peak 15 min is now computed using Eq. 8-3:

$$SF_D = 2{,}800 \times 0.83 \times 0.78 \times 1.00 \times 0.87 \times 0.38 = 599 \textit{ vph}$$

Since the question asks for a maximum *volume*, rather than a flow rate, the service flow rate is converted to a full hour volume as follows:

$$V = SF \times \text{PHF} = 599 \times 0.85 = 509 \textit{ vph}$$

Thus, the maximum full-hour volume which can be accommodated at 40 mph, or LOS D, on the grade described is 509 vph. The maximum flow rate is 599 vph.

CALCULATION 4—FINDING LEVEL OF SERVICE AND CAPACITY OF A SPECIFIC GRADE

1. *Description*—A rural two-lane highway in mountainous terrain has a grade of 7 percent, 2 miles long. It currently carries a peak hour volume of 500 vph. Other relevant characteristics include:

a. *Roadway characteristics*—60-mph design speed; 11-ft lanes; 4-ft shoulders; 80 percent no passing zones.
b. *Traffic characteristics*—80/20 directional split; 4 percent trucks; 10 percent recreational vehicles; 2 percent buses; 84 percent passenger cars; PHF=0.85.

At what level of service does the grade operate? What upgrade speed can be expected during the peak 15 min of flow? What is the capacity of the grade? If the approach speed to the grade is 55 mph, what delay is incurred by vehicles climbing the grade?

2. *Solution*—The finding of capacity for a specific grade requires plotting of the service flow rate vs. speed curve which results from Eq. 8-3:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV}$$

where:

$$f_g = 1/[1 + P_p I_p]$$
$$I_p = 0.02\,(E - E_o)$$

and:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)]$$
$$E_{HV} = 1 + (0.25 + P_{T/HV})\,(E - 1)$$

Capacity is found at the point where this curve intersects the speed at capacity vs. flow rate at capacity curve on the specific grade worksheet. The upgrade speed is found by entering this curve with the actual flow rate.

To plot the curve, the procedure recommends computing service flow rate points for the following speeds: 55 mph (LOS A), 52.5 mph, 50 mph (LOS B), 45 mph (LOS C), 40 mph (LOS D), and 30 mph. These points would be plotted on the specific grade worksheet of Figure 8-5, and a smooth curve constructed. Once capacity is determined, the service flow rates for every LOS will be known, and the actual LOS can be determined by comparing the actual flow rate to the computed values.

The following values are used in these computations:

v/c = 0.00 for 55 mph, 0.05 for 52.5 mph, 0.15 for 50 mph, 0.40 for 45 mph, 0.64 for 40 mph, 0.88 for 30 mph (Table 8-7, 7 percent grade, 80 percent no passing zones);
f_d = 0.70 (Table 8-8, 80/20 split);
f_w = 0.85 for 55–45 mph, 0.92 for 40–30 mph (Table 8-5, 11-ft lanes, 4-ft shoulders);
E = 88.0 for 52.5 mph, 46.0 for 50 mph, 22.8 for 45 mph, 15.4 for 40 mph, 8.2 for 30 mph (Table 8-9, 7 percent grade, 2 miles, no value given for 55 mph);
E_o = 1.8 for 52.5 mph, 1.6 for 50 mph, 1.4 for 45 mph, 1.3 for 40 mph, 30 mph (Table 8-9, 0 percent grade);
P_p = 0.84 (Given);
$P_{HV} = P_T + P_R + P_B = 0.04 + 0.10 + 0.02 = 0.16$; and
$P_{T/HV} = P_T/P_{HV} = 0.04/0.16 = 0.25$.

Values of f_g may now be computed as follows:

$$I_p(52.5) = 0.02(88.0 - 1.8) = 1.724$$
$$(50.0) = 0.02(46.0 - 1.6) = 0.888$$

$(45.0) = 0.02(22.8 - 1.4) = 0.428$
$(40.0) = 0.02(15.4 - 1.3) = 0.282$
$(30.0) = 0.02(\ 8.2 - 1.3) = 0.138$

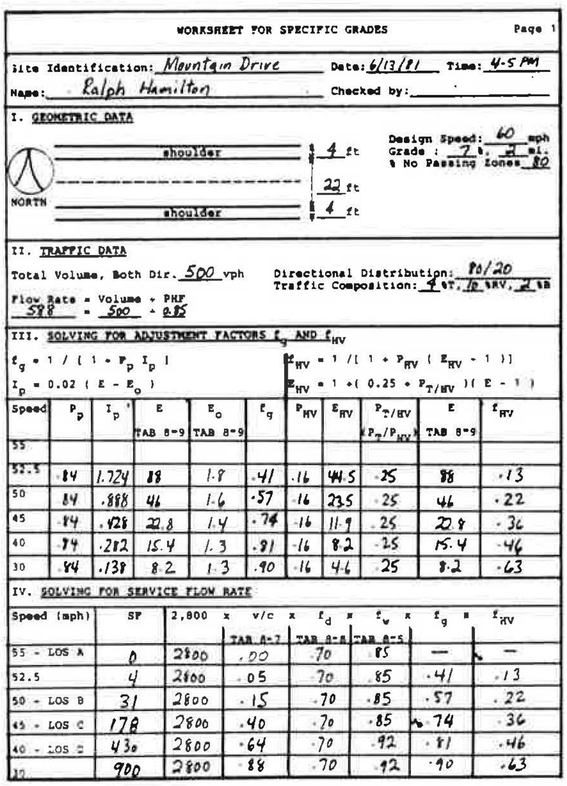
$f_g(52.5) = 1/[1 + 0.84(1.724)] = 0.41$
$(50.0) = 1/[1 + 0.84(0.888)] = 0.57$
$(45.0) = 1/[1 + 0.84(0.428)] = 0.74$
$(40.0) = 1/[1 + 0.84(0.282)] = 0.81$
$(30.0) = 1/[1 + 0.84(0.138)] = 0.90$

Values of f_{HV} are also computed:

$E_{HV}(52.5) = 1 + (0.25 + 0.25)(88.0 - 1) = 44.5$
$(50.0) = 1 + (0.25 + 0.25)(46.0 - 1) = 23.5$
$(45.0) = 1 + (0.25 + 0.25)(22.8 - 1) = 11.9$
$(40.0) = 1 + (0.25 + 0.25)(15.4 - 1) = 8.2$
$(30.0) = 1 + (0.25 + 0.25)(\ 8.2 - 1) = 4.6$
$f_{HV}(52.5) = 1/[1 + 0.16(44.5 - 1)] = 0.13$
$(50.0) = 1/[1 + 0.16(23.6 - 1)] = 0.22$
$(45.0) = 1/[1 + 0.16(11.9 - 1)] = 0.36$
$(40.0) = 1/[1 + 0.16(\ 8.2 - 1)] = 0.46$
$(30.0) = 1/[1 + 0.16(\ 4.6 - 1)] = 0.63$

Having computed all relevant factors, the total two-way service flow rates for the designated speeds may be computed:

SPEED	2,800 ×	v/c ×	f_d ×	f_w ×	f_g ×	f_{HV} =	SF
55.0	2,800	0.00	0.70	0.85	—	—	0 vph
52.5	2,800	0.05	0.70	0.85	0.41	0.13	4 vph
50.0	2,800	0.15	0.70	0.85	0.57	0.22	31 vph
45.0	2,800	0.40	0.70	0.85	0.74	0.36	178 vph
40.0	2,800	0.64	0.70	0.92	0.81	0.46	430 vph
30.0	2,800	0.88	0.70	0.92	0.90	0.63	900 vph


WORKSHEET FOR SPECIFIC GRADES Page 1

Site Identification: Mountain Drive Date: 6/13/81 Time: 4-5 PM
Name: Ralph Hamilton Checked by:

I. GEOMETRIC DATA

shoulder 4 ft
22 ft
shoulder 4 ft
NORTH

Design Speed: 60 mph
Grade: 7 %, 2 mi.
% No Passing Zones 80

II. TRAFFIC DATA

Total Volume, Both Dir. 500 vph
Directional Distribution: 80/20
Traffic Composition: 4 %T, 12 %RV, 2 %B
Flow Rate = Volume ÷ PHF
588 = 500 ÷ 0.85

III. SOLVING FOR ADJUSTMENT FACTORS f_g AND f_{HV}

$f_g = 1 / [1 + P_p I_p]$
$I_p = 0.02 (E - E_o)$
$f_{HV} = 1 / [1 + P_{HV} (E_{HV} - 1)]$
$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1)$

Speed	P_p	I_p	E TAB 8-9	E_o TAB 8-9	f_g	P_{HV}	E_{HV}	$P_{T/HV}$ (P_T/P_{HV})	E TAB 8-9	f_{HV}
55										
52.5	.84	1.724	88	1.8	.41	.16	44.5	.25	88	.13
50	.84	.888	46	1.6	.57	.16	23.5	.25	46	.22
45	.84	.428	22.8	1.4	.74	.16	11.9	.25	22.8	.36
40	.84	.282	15.4	1.3	.81	.16	8.2	.25	15.4	.46
30	.84	.138	8.2	1.3	.90	.16	4.6	.25	8.2	.63

IV. SOLVING FOR SERVICE FLOW RATE

Speed (mph)	SF	2,800 x	v/c x TAB 8-7	f_d x TAB 8-8	f_w x TAB 8-5	f_g x	f_{HV}
55 - LOS A	0	2800	.00	.70	.85	—	—
52.5	4	2800	.05	.70	.85	.41	.13
50 - LOS B	31	2800	.15	.70	.85	.57	.22
45 - LOS C	178	2800	.40	.70	.85	.74	.36
40 - LOS D	430	2800	.64	.70	.92	.81	.46
30	900	2800	.88	.70	.92	.90	.63

Note that the low or zero service flow rates for 55.0 and 52.5 mph indicate that these average upgrade speeds are virtually impossible to maintain on the upgrade described in this problem.

These computations are summarized on the specific grade worksheet shown in Figure 8-9. The curve defined by these points is also plotted on the worksheet. The intersection of the plotted curve with the speed at capacity vs. flow rate at capacity curve indicates that capacity is *950 vph*, total in both directions, which occurs at an average upgrade speed of *28.0 mph.*

To find the existing level of service, the volume of 500 vph is converted to a flow rate for the peak 15-min period:

$$v = V/\mathrm{PHF} = 500/0.85 = 588 \text{ vph}$$

The plotted curve is entered on the worksheet with 588 vph, and the upgrade speed is found to be *37 mph.* Because this speed is less than 40 mph, the minimum value for LOS D (Table 8-2), but greater than the speed at capacity (28 mph), the level of service is E. This can also be determined by comparing the actual flow rate of 588 vph with the service flow rate for LOS D (40 mph) of 430 vph and capacity (950 vph).

The last part of this problem asks to find the delay incurred by vehicles traveling up the grade. "Delay" is defined as the difference in travel time experienced by vehicles traversing the upgrade at the existing speed and the travel time which would be experienced if they were able to maintain their approach speed on the grade. Thus:

Travel time at 55.0 mph = (2 miles/55 mph) × 3600 sec/hour
= 130.9 sec/veh

Travel time at 37.0 mph = (2 miles/37 mph) × 3600 sec/hour
= 194.6 sec/veh

Delay = 194.6 − 130.9 = 63.7 sec/veh

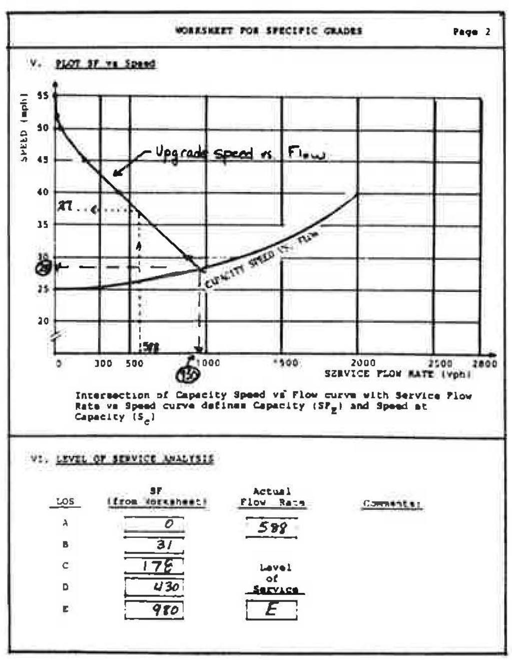


Figure 8-9. Worksheet for Calculation 4 (pages 1 and 2).

CALCULATION 5—CONSIDERATION OF A CLIMBING LANE

1. *Description*—A rural two-lane highway has a 4 percent upgrade of 1½ miles, and has the following other characteristics:
   a. *Roadway characteristics*—level terrain approach; 12-ft lanes; 8-ft shoulders; 40 percent no passing zones.
   b. *Traffic characteristics*—DHV = 400 vph; 15 percent trucks; 5 percent recreational vehicles; 1 percent buses; 79 percent passenger cars; 60/40 directional split; PHF = 0.85.

Is the addition of a climbing lane justified at this location?

2. *Solution*—It is assumed that a climbing lane on a two-lane highway is generally justified when the following conditions are met:

1. Upgrade flow rate is greater than 200 vph.
2. Upgrade truck flow rate is greater than 20 vph.
3. One of the following occurs:
   a. The grade operates at LOS E or F.
   b. The typical heavy truck reduces its speed by more than 10 mph on the grade.
   c. The LOS on the grade is two or more levels poorer than on the approach to the grade.

Each of these conditions should be checked to justify the construction of the climbing lane:

Upgrade flow rate = 400 × 0.60/0.85 = 282 vph > 200 vph OK
Upgrade trucks = 400 × 0.15 × 0.60/0.85 = 42 vph > 20 vph OK

To justify a climbing lane, only one of the conditions specified in item 3 must be demonstrated. The LOS will be E or worse if the actual flow rate exceeds the service flow rate for LOS D. This value is computed using Eq. 8-3:

$$SF_D = 2{,}800 \times (v/c)_D \times f_d \times f_w \times f_g \times f_{HV}$$

where:

$$f_g = 1/[1 + P_p I_p]$$

$$I_p = 0.02\,(E - E_o)$$

and:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)]$$

$$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1)$$

The following values are used:

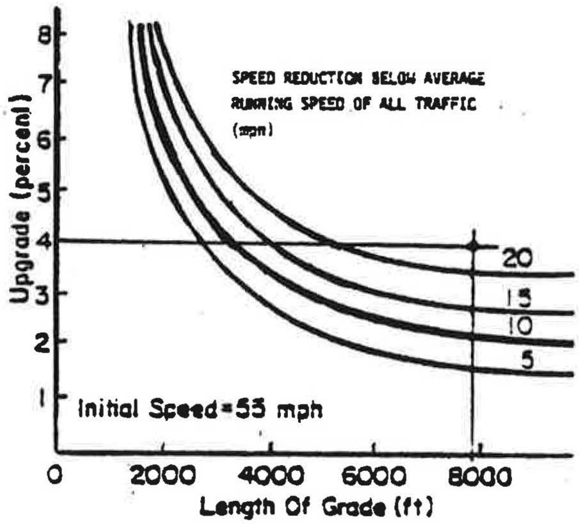
$(v/c)_D$ = 1.00 (Table 8-7, 4 percent grade, 40 mph, 40 percent no passing zones);
f_d = 0.87 (Table 8-8, 60/40 directional split);
f_w = 1.00 (Table 8-5);
E = 3.8 (Table 8-9, 4 percent, 1½-mile grade, 40 mph);
E_o = 1.3 (Table 8-9, 0 percent grade, 40 mph);
P_{HV} = 0.15 + 0.05 + 0.01 = 0.21; and
$P_{T/HV}$ = 0.15/0.21 = 0.71.

Using these values to compute the service flow rate at level-of-service D:

$$I_p = 0.02(3.8 - 1.3) = 0.05$$
$$f_g = 1/[1 + (0.79 \times 0.05)] = 0.96$$
$$E_{HV} = 1 + (0.25 + 0.71)(3.8 - 1) = 3.69$$
$$f_{HV} = 1/[1 + 0.21(3.69 - 1)] = 0.64$$
$$SF_D = 2{,}800 \times 1.00 \times 0.87 \times 1.00 \times 0.96 \times 0.64 = 1{,}497 \text{ vph}$$

The actual flow rate is the DHV divided by the PHF, or 400/0.85 = 471 vph. As this is clearly *less* than the service flow rate for LOS D, the existing LOS is *not* E, and this condition is *not* met.

The next condition to investigate is whether a 10-mph speed reduction of heavy trucks would exist on the grade described. Based on the assumption that the typical truck on this grade has a weight/horsepower ratio of 200 lb/hp, Figure 8-2 is used to estimate the speed reduction experienced as shown below:



It can be seen that the speed reduction will be well in excess of 20 mph, which is greater than 10 mph, fulfilling the last required condition for justifying a climbing lane. Note that because only one of the conditions in item 3 needs to be satisfied, it is not necessary to investigate the third condition.

It can be concluded that a climbing lane is justified on the basis of the stated criteria.

CALCULATION 6—PLANNING APPLICATION 1

1. *Description*—A rural two-lane highway in mountainous terrain is located in an area where the design hour factor, *K*, is 0.14. What is the maximum AADT which can be accommodated without the LOS falling below D during the peak 15-min flow period?

2. *Solution*—The solution is simply found by entering Table 8-10 with mountainous terrain, LOS D, and a *K*-factor of 0.14. The maximum permissible AADT is found to be *2,700 vpd.*

CALCULATION 7—PLANNING APPLICATION 2

1. *Description*—A rural two-lane highway is located in rolling terrain in an area where the design hour factor, K, is 0.12. Its current AADT is 5,000 vpd. What is the likely LOS during the peak 15 min of flow?

2. *Solution*—Again, the solution is straightforward using Table 8-10. The maximum AADT's for the various levels of service are found for rolling terrain and a K-factor of 0.12. The 5,000 AADT is seen to fall between the maximum values for LOS C (4,400 vpd) and LOS D (6,600 vpd). The LOS is therefore expected to be D during the peak 15 min of flow.

CALCULATION 8—PLANNING APPLICATION 3

1. *Description*—A two-lane highway carrying an AADT of 6,600 vpd is located in level terrain in an area where the design hour factor, K, is 0.12. The area has a traffic growth rate of 5 percent per year. The responsible highway agency's policy is to expand two-lane highways to four lanes before the level of service becomes E during peak periods. In how many years will expansion of the facility have to be completed under this policy? If it will take 7 years to construct a four-lane highway, how long will it be before the construction project should begin?

2. *Solution*—The policy requires that expansion of the highway be completed before the AADT exceeds the maximum allowable value for LOS D. From Table 8-10, the maximum AADT for LOS D, for level terrain and a K-factor of 0.12, is 11,200 vpd.

The question now becomes: How many years will it take an AADT of 6,600 vpd to grow to 11,200 vpd at a rate of 5 percent per year? Therefore:

$$11{,}200 = 6{,}600(1 + 0.05)^n$$
$$n = 10.9 \text{ years}$$

Construction should begin in 10.9 − 7 years, or in 3.9 years.

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APPENDIX I

TABLES, FIGURES, AND WORKSHEETS FOR USE IN ANALYSIS OF TWO-LANE HIGHWAYS

TABLE 8-1. LEVEL-OF-SERVICE CRITERIA FOR GENERAL TWO-LANE HIGHWAY SEGMENTS

		v/c RATIO[a]																				
		LEVEL TERRAIN							ROLLING TERRAIN							MOUNTAINOUS TERRAIN						
	PERCENT TIME DELAY	AVG[b] SPEED	PERCENT NO PASSING ZONES						AVG[b] SPEED	PERCENT NO PASSING ZONES						AVG[b] SPEED	PERCENT NO PASSING ZONES					
LOS			0	20	40	60	80	100		0	20	40	60	80	100		0	20	40	60	80	100
A	≤ 30	≥ 58	0.15	0.12	0.09	0.07	0.05	0.04	≥ 57	0.15	0.10	0.07	0.05	0.04	0.03	≥ 56	0.14	0.09	0.07	0.04	0.02	0.01
B	≤ 45	≥ 55	0.27	0.24	0.21	0.19	0.17	0.16	≥ 54	0.26	0.23	0.19	0.17	0.15	0.13	≥ 54	0.25	0.20	0.16	0.13	0.12	0.10
C	≤ 60	≥ 52	0.43	0.39	0.36	0.34	0.33	0.32	≥ 51	0.42	0.39	0.35	0.32	0.30	0.28	≥ 49	0.39	0.33	0.28	0.23	0.20	0.16
D	≤ 75	≥ 50	0.64	0.62	0.60	0.59	0.58	0.57	≥ 49	0.62	0.57	0.52	0.48	0.46	0.43	≥ 45	0.58	0.50	0.45	0.40	0.37	0.33
E	> 75	≥ 45	1.00	1.00	1.00	1.00	1.00	1.00	≥ 40	0.97	0.94	0.92	0.91	0.90	0.90	≥ 35	0.91	0.87	0.84	0.82	0.80	0.78
F	100	< 45	—	—	—	—	—	—	< 40	—	—	—	—	—	—	< 35	—	—	—	—	—	—

[a] Ratio of flow rate to an ideal capacity of 2,800 pcph in both directions.
[b] Average travel speed of all vehicles (in mph) for highways with design speed ≥ 60 mph; for highways with lower design speeds, reduce speed by 4 mph for each 10-mph reduction in design speed below 60 mph; assumes that speed is not restricted to lower values by regulation.

TABLE 8-2. LEVEL-OF-SERVICE CRITERIA FOR SPECIFIC GRADES

LEVEL OF SERVICE	AVERAGE UPGRADE SPEED (MPH)
A	≥ 55
B	≥ 50
C	≥ 45
D	≥ 40
E	≥ 25–40[a]
F	< 25–40[a]

[a] The exact speed at which capacity occurs varies with the percentage and length of grade, traffic compositions, and volume; computational procedures are provided to find this value.

TABLE 8-3. PEAK HOUR FACTORS FOR TWO-LANE HIGHWAYS BASED ON RANDOM FLOW

A. LEVEL-OF-SERVICE DETERMINATIONS

TOTAL 2-WAY HOURLY VOLUME (VPH)	PEAK HOUR FACTOR (PHF)	TOTAL 2-WAY HOURLY VOLUME (VPH)	PEAK HOUR FACTOR (PHF)
100	0.83	1,000	0.93
200	0.87	1,100	0.94
300	0.90	1,200	0.94
400	0.91	1,300	0.94
500	0.91	1,400	0.94
600	0.92	1,500	0.95
700	0.92	1,600	0.95
800	0.93	1,700	0.95
900	0.93	1,800	0.95
		≥ 1,900	0.96

B. SERVICE FLOW-RATE DETERMINATIONS

Level of Service	A	B	C	D	E
Peak Hour Factor	0.91	0.92	0.94	0.95	1.00

TABLE 8-4. ADJUSTMENT FACTORS FOR DIRECTIONAL DISTRIBUTION ON GENERAL TERRAIN SECTIONS

Directional Distribution	100/0	90/10	80/20	70/30	60/40	50/50
Adjustment Factor, f_d	0.71	0.75	0.83	0.89	0.94	1.00

TABLE 8-5. ADJUSTMENT FACTORS FOR THE COMBINED EFFECT OF NARROW LANES AND RESTRICTED SHOULDER WIDTH, f_w

USABLE[a] SHOULDER WIDTH (FT)	12-FT LANES		11-FT LANES		10-FT LANES		9-FT LANES	
	LOS A–D	LOS[b] E	LOS A–D	LOS[b] E	LOS A–D	LOS[b] E	LOS A–D	LOS[b] E
0	0.70	0.88	0.65	0.82	0.58	0.75	0.49	0.66
2	0.81	0.93	0.75	0.88	0.68	0.81	0.57	0.70
4	0.92	0.97	0.85	0.92	0.77	0.85	0.65	0.74
≥ 6	1.00	1.00	0.93	0.94	0.84	0.87	0.70	0.76

[a] Where shoulder width is different on each side of the roadway, use the average shoulder width.
[b] Factor applies for all speeds less than 45 mph.

TABLE 8-6. AVERAGE PASSENGER-CAR EQUIVALENTS FOR TRUCKS, RV'S, AND BUSES ON TWO-LANE HIGHWAYS OVER GENERAL TERRAIN SEGMENTS

VEHICLE TYPE	LEVEL OF SERVICE	TYPE OF TERRAIN LEVEL	ROLLING	MOUNTAINOUS
Trucks, E_T	A	2.0	4.0	7.0
	B and C	2.2	5.0	10.0
	D and E	2.0	5.0	12.0
RV's E_R	A	2.2	3.2	5.0
	B and C	2.5	3.9	5.2
	D and E	1.6	3.3	5.2
Buses, E_B	A	1.8	3.0	5.7
	B and C	2.0	3.4	6.0
	D and E	1.6	2.9	6.5

SOURCE: Ref. 6

TABLE 8-7. VALUES OF v/c RATIO[a] VS. SPEED, PERCENT GRADE, AND PERCENT NO PASSING ZONES FOR SPECIFIC GRADES

PERCENT GRADE	AVERAGE UPGRADE SPEED (MPH)	PERCENT NO PASSING ZONES					
		0	20	40	60	80	100
3	55	0.27	0.23	0.19	0.17	0.14	0.12
	52.5	0.42	0.38	0.33	0.31	0.29	0.27
	50	0.64	0.59	0.55	0.52	0.49	0.47
	45	1.00	0.95	0.91	0.88	0.86	0.84
	42.5	1.00	0.98	0.97	0.96	0.95	0.94
	40	1.00	1.00	1.00	1.00	1.00	1.00
4	55	0.25	0.21	0.18	0.16	0.13	0.11
	52.5	0.40	0.36	0.31	0.29	0.27	0.25
	50	0.61	0.56	0.52	0.49	0.47	0.45
	45	0.97	0.92	0.88	0.85	0.83	0.81
	42.5	0.98	0.96	0.95	0.94	0.93	0.92
	40	1.00	1.00	1.00	1.00	1.00	1.00
5	55	0.25	0.21	0.16	0.12	0.08	0.04
	52.5	0.40	0.34	0.27	0.22	0.18	0.13
	50	0.61	0.51	0.42	0.34	0.28	0.22
	45	0.97	0.84	0.71	0.60	0.52	0.46
	42.5	0.98	0.87	0.78	0.70	0.63	0.56
	40	0.99	0.89	0.81	0.74	0.69	0.63
	35	1.00	0.92	0.87	0.82	0.79	0.76
	30	1.00	0.95	0.92	0.90	0.88	0.86
6	55	0.12	0.09	0.07	0.05	0.04	0.02
	52.5	0.27	0.22	0.17	0.14	0.12	0.08
	50	0.49	0.39	0.32	0.26	0.22	0.17
	45	0.89	0.75	0.63	0.53	0.46	0.41
	42.5	0.93	0.81	0.73	0.65	0.59	0.52
	40	0.97	0.86	0.79	0.72	0.67	0.61
	35	1.00	0.92	0.87	0.82	0.79	0.76
	30	1.00	0.95	0.92	0.90	0.88	0.86
7	55	0.00	0.00	0.00	0.00	0.00	0.00
	52.5	0.13	0.10	0.08	0.07	0.05	0.04
	50	0.34	0.27	0.22	0.18	0.15	0.12
	45	0.77	0.65	0.55	0.46	0.40	0.35
	42.5	0.86	0.75	0.67	0.60	0.54	0.48
	40	0.93	0.83	0.75	0.69	0.64	0.59
	35	1.00	0.92	0.87	0.82	0.79	0.76
	30	1.00	0.95	0.92	0.90	0.88	0.86

[a] Ratio of flow rate to ideal capacity of 2,800 pcph, assuming passenger-car operation is unaffected by grade.
NOTE: Interpolate for intermediate values of "Percent No Passing Zone"; round "Percent Grade" to the next higher integer value.

TABLE 8-8. ADJUSTMENT FACTOR FOR DIRECTIONAL DISTRIBUTION ON SPECIFIC GRADES, f_d

DIRECTIONAL[a] DISTRIBUTION	ADJUSTMENT FACTOR	DIRECTIONAL[a] DISTRIBUTION	ADJUSTMENT FACTOR
100/0	0.58	40/60	1.20
90/10	0.64	30/70	1.50
80/20	0.70	20/80	1.93
70/30	0.78	10/90	2.51
60/40	0.87	0/100	3.32
50/50	1.00		

[a] Upgrade/Downgrade

TABLE 8-9. PASSENGER-CAR EQUIVALENTS FOR SPECIFIC GRADES ON TWO-LANE RURAL HIGHWAYS, E AND E_o

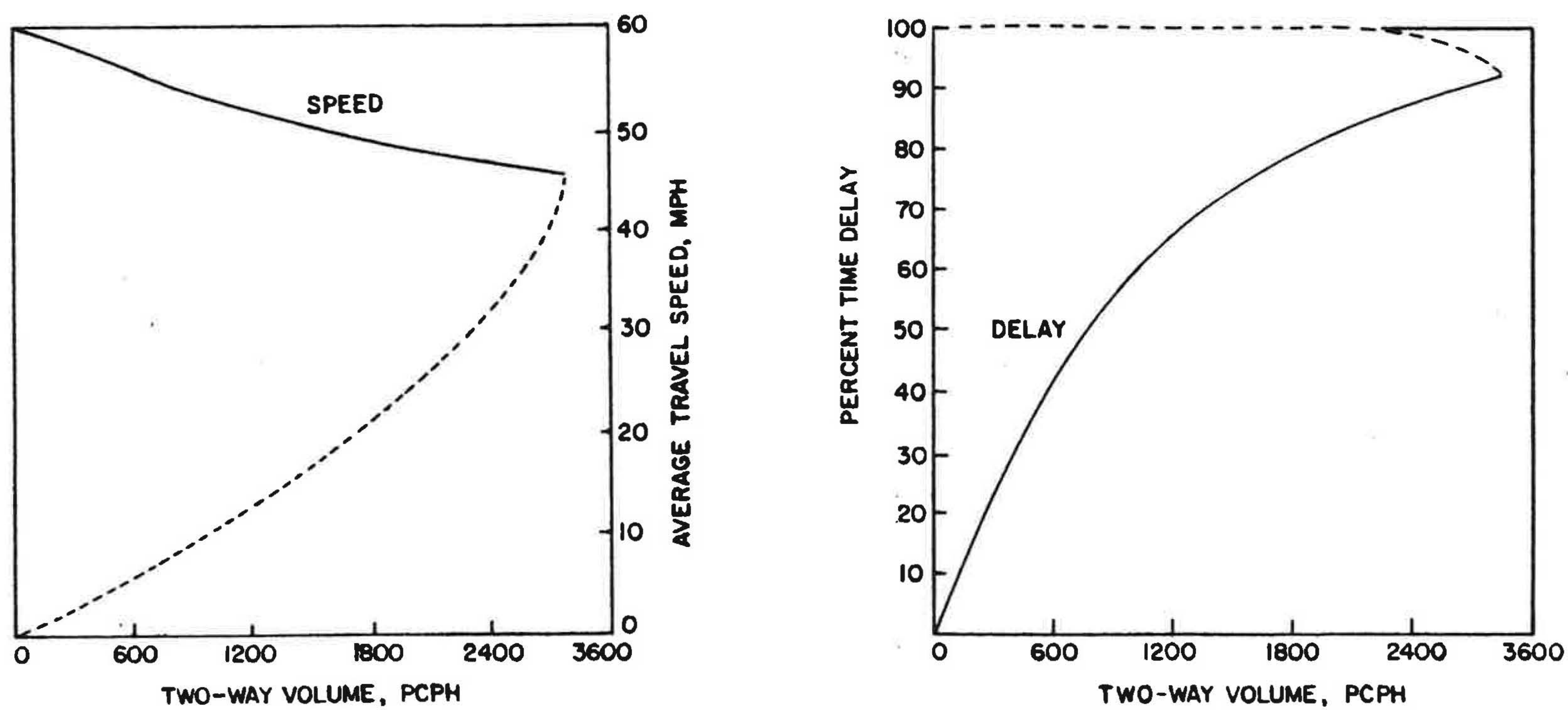
GRADE (%)	LENGTH OF GRADE (MI)	AVERAGE UPGRADE SPEED (MPH) 55.0	52.5	50.0	45.0	40.0	30.0
0	All	2.1	1.8	1.6	1.4	1.3	1.3
3	¼	2.9	2.3	2.0	1.7	1.6	1.5
	½	3.7	2.9	2.4	2.0	1.8	1.7
	¾	4.8	3.6	2.9	2.3	2.0	1.9
	1	6.5	4.6	3.5	2.6	2.3	2.1
	1½	11.2	6.6	5.1	3.4	2.9	2.5
	2	19.8	9.3	6.7	4.6	3.7	2.9
	3	71.0	21.0	10.8	7.3	5.6	3.8
	4	*	48.0	20.5	11.3	7.7	4.9
4	¼	3.2	2.5	2.2	1.8	1.7	1.6
	½	4.4	3.4	2.8	2.2	2.0	1.9
	¾	6.3	4.4	3.5	2.7	2.3	2.1
	1	9.6	6.3	4.5	3.2	2.7	2.4
	1½	19.5	10.3	7.4	4.7	3.8	3.1
	2	43.0	16.1	10.8	6.9	5.3	3.8
	3	*	48.0	20.0	12.5	9.0	5.5
	4	*	*	51.0	22.8	13.8	7.4
5	¼	3.6	2.8	2.3	2.0	1.8	1.7
	½	5.4	3.9	3.2	2.5	2.2	2.0
	¾	8.3	5.7	4.3	3.1	2.7	2.4
	1	14.1	8.4	5.9	4.0	3.3	2.8
	1½	34.0	16.0	10.8	6.3	4.9	3.8
	2	91.0	28.3	17.4	10.2	7.5	4.8
	3	*	*	37.0	22.0	14.6	7.8
	4	*	*	*	55.0	25.0	11.5
6	¼	4.0	3.1	2.5	2.1	1.9	1.8
	½	6.5	4.8	3.7	2.8	2.4	2.2
	¾	11.0	7.2	5.2	3.7	3.1	2.7
	1	20.4	11.7	7.8	4.9	4.0	3.3
	1½	60.0	25.2	16.0	8.5	6.4	4.7
	2	*	50.0	28.2	15.3	10.7	6.3
	3	*	*	70.0	38.0	23.9	11.3
	4	*	*	*	90.0	45.0	18.1
7	¼	4.5	3.4	2.7	2.2	2.0	1.9
	½	7.9	5.7	4.2	3.2	2.7	2.4
	¾	14.5	9.1	6.3	4.3	3.6	3.0
	1	31.4	16.0	10.0	6.1	4.8	3.8
	1½	*	39.5	23.5	11.5	8.4	5.8
	2	*	88.0	46.0	22.8	15.4	8.2
	3	*	*	*	66.0	38.5	16.1
	4	*	*	*	*	*	28.0

* Speed not attainable on grade specified.
NOTE: Round "Percent Grade" to next higher integer value.

TABLE 8-10. MAXIMUM AADT'S VS. LEVEL OF SERVICE AND TYPE OF TERRAIN FOR TWO-LANE RURAL HIGHWAYS

K-FACTOR	LEVEL OF SERVICE				
	A	B	C	D	E
LEVEL TERRAIN					
0.10	2,400	4,800	7,900	13,500	22,900
0.11	2,200	4,400	7,200	12,200	20,800
0.12	2,000	4,000	6,600	11,200	19,000
0.13	1,900	3,700	6,100	10,400	17,600
0.14	1,700	3,400	5,700	9,600	16,300
0.15	1,600	3,200	5,300	9,000	15,200
ROLLING TERRAIN					
0.10	1,100	2,800	5,200	8,000	14,800
0.11	1,000	2,500	4,700	7,200	13,500
0.12	900	2,300	4,400	6,600	12,300
0.13	900	2,100	4,000	6,100	11,400
0.14	800	2,000	3,700	5,700	10,600
0.15	700	1,800	3,500	5,300	9,900
MOUNTAINOUS TERRAIN					
0.10	500	1,300	2,400	3,700	8,100
0.11	400	1,200	2,200	3,400	7.300
0.12	400	1,100	2,000	3,100	6,700
0.13	400	1,000	1,800	2,900	6,200
0.14	300	900	1,700	2,700	5,800
0.15	300	900	1,600	2,500	5,400

NOTE: All values rounded to the nearest 100 vpd. Assumed conditions include 60/40 directional split, 14 percent trucks, 4 percent RV's, no buses, and PHF values from Table 8-3. For level terrain, 20 percent no passing zones; were assumed; for rolling terrain, 40 percent no passing zones; for mountainous terrain, 60 percent no passing zones.



a) Relationship between average speed and flow on two-lane highways.

b) Relationship between percent time delay and flow on two-lane highways.

Figure 8-1. Speed-flow and percent time delay-flow relationships for two-lane rural highways (ideal conditions).

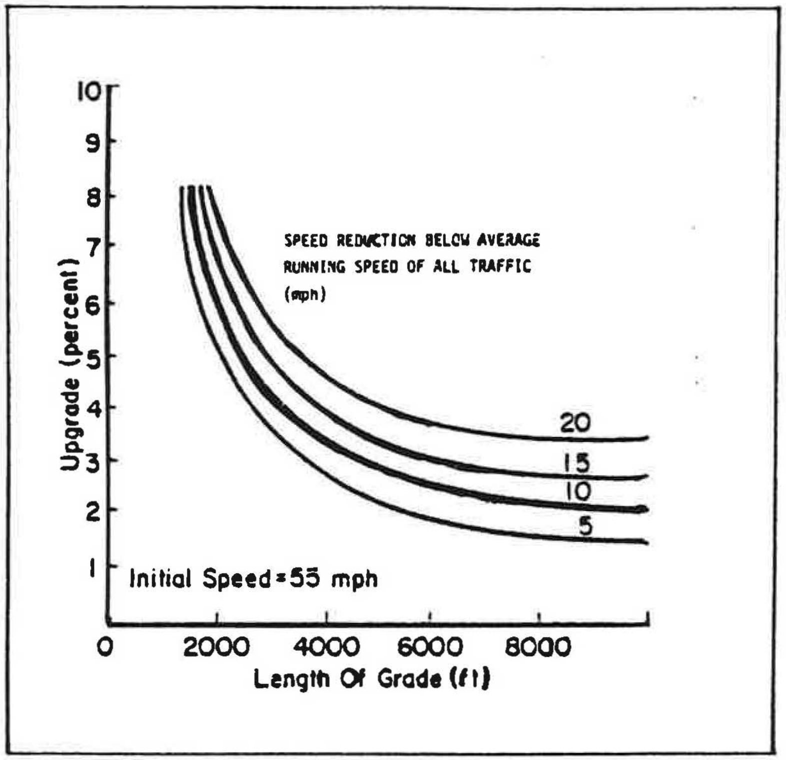


Figure 8-2. Speed reduction curve for a 200-lb/hp truck.

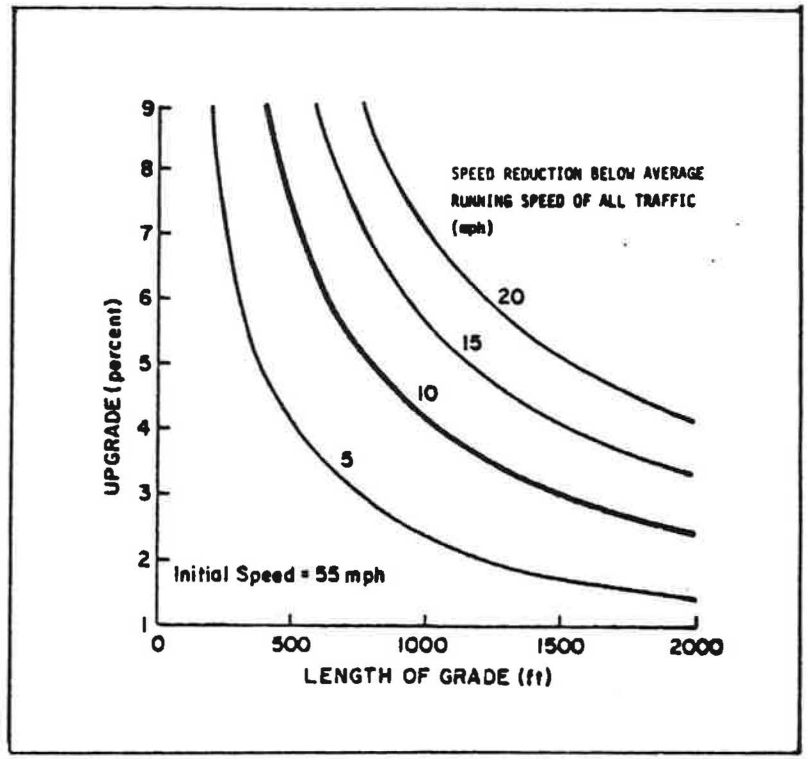


Figure 8-3. Speed reduction curve for a 300-lb/hp truck.

WORKSHEET FOR GENERAL TERRAIN SECTIONS

Site Identification:______________________ Date:________ Time:________

Name:__________________________________ Checked by:______________

I. GEOMETRIC DATA

NORTH

shoulder ____ft

____ft

shoulder ____ft

Design Speed:______mph
% No Passing ________%
Terrain (L,R,M):______
Segment Length______mi

II. TRAFFIC DATA

Total Volume, Both Dir._____vph

Flow Rate = Volume ÷ PHF
_________ = _______ ÷ ___

Directional Distribution:____________
Traffic Composition:___%T,___%RV,___%B
PHF:__________

III. LEVEL OF SERVICE ANALYSIS

$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$

$f_{HV} = 1 / [1 + P_T(E_T-1) + P_R(E_R-1) + P_B(E_B-1)]$

LOS	SF =	2,800x	(v/c)x	f_d x	f_w x	f_{HV}	P_T	E_T	P_R	E_R	P_B	E_B
			TAB 8-1	TAB 8-4	TAB 8-5			TAB 8-6		TAB 8-6		TAB 8-6
A		2,800										
B		2,800										
C		2,800										
D		2,800										
E		2,800										

IV. COMMENTS

Flow Rate________vph LOS=_______

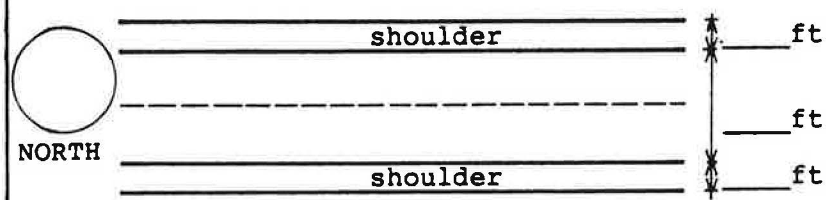
WORKSHEET FOR SPECIFIC GRADES

Page 1

Site Identification:________________________ Date:__________ Time:_________

Name:____ Checked by:________________

I. GEOMETRIC DATA



Design Speed:______mph
Grade : ____%, ____mi.
% No Passing Zones____

II. TRAFFIC DATA

Total Volume, Both Dir.______vph

Flow Rate = Volume ÷ PHF
_________ = _______ ÷ ___

Directional Distribution:____________
Traffic Composition:___%T,___%RV,___%B

III. SOLVING FOR ADJUSTMENT FACTORS f_g AND f_{HV}

$f_g = 1 / [1 + P_p I_p]$

$I_p = 0.02 (E - E_o)$

$f_{HV} = 1 /[1 + P_{HV} (E_{HV} - 1)]$

$E_{HV} = 1 +(0.25 + P_{T/HV})(E - 1)$

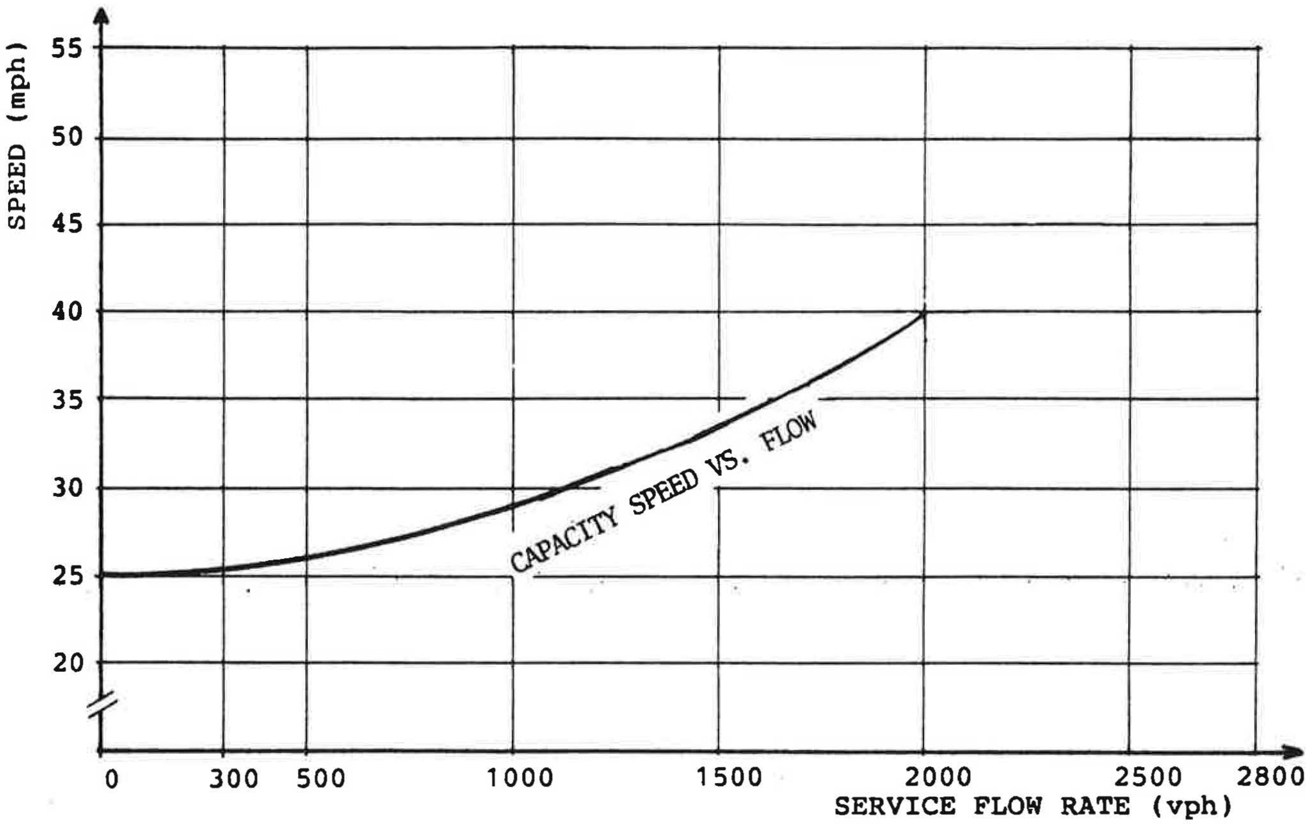
Speed	P_p	I_p	E TAB 8-9	E_o TAB 8-9	f_g	P_{HV}	E_{HV}	$P_{T/HV}$ (P_T/P_{HV})	E TAB 8-9	f_{HV}
55										
52.5										
50										
45										
40										
30										

IV. SOLVING FOR SERVICE FLOW RATE

Speed (mph)	SF	2,800 x	v/c x TAB 8-7	f_d x TAB 8-8	f_w x TAB 8-5	f_g x	f_{HV}
55 - LOS A							
52.5							
50 - LOS B							
45 - LOS C							
40 - LOS D							
30							

WORKSHEET FOR SPECIFIC GRADES Page 2

V. PLOT SF vs Speed



Intersection of Capacity Speed vs Flow curve with Service Flow Rate vs Speed curve defines Capacity (SF_E) and Speed at Capacity (S_c)

VI. LEVEL OF SERVICE ANALYSIS

LOS	SF (from Worksheet)	Actual Flow Rate	Comments:
A			
B			
C		Level of Service	
D			
E			

APPENDIX II

GLOSSARY AND SYMBOLS

GLOSSARY

design hour factor—Proportion of 24-hour volume occurring during the design hour for a given location or area; designated by the letter *K*.

no passing zone—A segment of two-lane, two-way highway along which passing is prohibited in one or both directions.

passing sight distance—The visibility distance required to allow drivers to execute passing maneuvers in the opposing traffic lane on two-lane, two-way highways.

three-lane highway—A highway having a three-lane cross section; the third lane may be used in a variety of ways, including as a passing lane, a two-way left-turn lane, or a climbing lane.

turnout—A short section of lane added to a two-lane, two-way highway for the purpose of allowing slow-moving vehicles to leave the main roadway and stop, thereby permitting faster vehicles to pass.

two-lane highway—A roadway having a two-lane cross section with one lane for each direction of flow, on which passing maneuvers must be made in the opposing lane.

two-way left-turn lane—The center lane on a three-lane or multilane highway which is used continuously for vehicles turning left in either direction of flow.

two-lane highway capacity—The maximum number of vehicles, total in both directions, which can be accommodated by a uniform uninterrupted flow segment of two-lane highway under prevailing traffic and roadway conditions. This capacity does not take into account the effect of fixed interruptions, such as traffic signals.

SYMBOLS

D the design hour directional factor; the percentage of total design hour traffic expected in the peak direction of flow.

E passenger-car equivalent for a standard mix of heavy vehicles on a specific grade of a two-lane, two-way rural highway; the standard mix includes 14 percent trucks and 4 percent recreational vehicles.

E_{HV} passenger-car equivalent for the prevailing mix of heavy vehicles on a two-lane, two-way rural highway specific grade.

E_o passenger-car equivalent for a standard mix of vehicles on a grade of 0 percent on a two-lane, two-way rural highway.

f_g multiplicative adjustment factor for the affect of specific grades on the operation of passenger cars on two-lane, two-way highways.

I_p impedance factor reflecting the affect of specific grades on the operation of passenger cars on two-lane, two-way highways.

K the design hour factor; the percentage of AADT expected to occur during the design hour.

P_{HV} the total proportion of heavy vehicles in the traffic stream; i.e., the proportion of trucks, recreational vehicles, and buses in the traffic stream.

$P_{T/HV}$ the proportion of trucks among heavy vehicles in the traffic stream; i.e., the proportion of trucks in the traffic stream divided by the total proportion of heavy vehicles in the traffic stream.

CHAPTER 13

PEDESTRIANS

CONTENTS

I. INTRODUCTION

The purpose of this chapter is to describe the basic principles of pedestrian traffic flow, and to present a general framework and procedures for the analysis of pedestrian facilities. The scope is limited to sidewalks, crosswalks, and street corners, but the analysis techniques can be applied to other pedestrian facilities. The chapter includes examples illustrating several typical applications.

Pedestrian activity can be a major component in urban street capacity analysis, and pedestrian characteristics are an important factor in the design and operation of transportation systems. Concentrated pedestrian movement occurs at public events, in and near transit terminals, high-rise buildings, department stores, theaters, stadia, parking garages, and other major traffic generators. Pedestrian safety, trip patterns, and convenience are also a necessary consideration in all multimodal traffic and transportation studies. Table 13-1 presents some high pedestrian volumes observed in several major urban centers.

The concentration of pedestrian activity at street corners and crosswalks makes them critical traffic links for both sidewalk and street networks. An overloaded corner or crosswalk not

TABLE 13-1. OBSERVED PEDESTRIAN FLOW RATES IN URBAN AREAS*

LOCATION	TIME	WALKWAY WIDTH (FT)	AVG. FLOW RATES FOR FULL HOUR		PEAK FLOW RATES FOR PERIODS LESS THAN 1 HOUR	
			PED/MIN	PED/MIN/FT	PED/MIN	PED/MIN/FT
BOSTON						
Washington St (1960)	12–1 PM	7.0	53	7.6	—	—
CHICAGO						
CTA (1976)	PM	—	—	5.2	—	—
State St/Wash (1960)	12–1 PM	25.0	112	4.5	—	—
State St/Wash (1972)	4–5 PM	25.0	93	3.7	—	—
State St/Wash (1939)	12–1 PM	25.0	206	8.2	—	—
State St/Mad (1929)	—	25.0	342	13.7	471 (15 min)	18.8
State St/Mad (1929)	—	20.0	287	14.4	368 (15 min)	18.4
Soldiers Fld (1940)	—	21.5	202	9.4	298 (1 min)	13.9
Dyche Stadium (1940)	—	10.0	114	11.4	167 (5 min)	16.7
LOS ANGELES						
Broadway (1940)	—	18.0	—	—	125 (12 min)	6.9
DES MOINES AND AMES, IOWA						
Veteran's Aud. (1975)	10 PM	8.2	—	—	—	20.0 (5 min) 22.2 (1 min)
College Creek Footbridge (1975)	12 Nn	6.0	—	—	—	22.3 (5 min) 31.8 (1 min)

CY Stephens Auditorium (1975)	4:40 PM	7.5	—	—	—	31.9 (5 min) 39.2 (1 min)
Iowa State Univ. Armory	1 PM	2.8	—	—	—	28.7 (1 min)
New York City						
Madison Av (1969)	12–1 PM	13.0	167	12.8	—	—
Fifth Av (1969)	12–1 PM	22.5	250	11.1	—	—
Lexington Av (1969)	12–1 PM	12.0	100	8.3	—	—
Eighth Av (1969)	PM	15.0	167	11.1	—	—
42nd Street (1969)	PM	20.0	105	5.3	—	—
Port Authority Bus Terminal (1965)	PM	—	—	25.0	—	—
Washington D.C.						
7th St SW (1968)	PM	10.0	42	4.2	—	—
F Street NW (1981)	PM	15.0	19	1.3	—	—
Seattle						
CBD (1976)	PM	—	—	—	—	9.6
San Francisco						
CBD (1976)	PM	—	—	—	—	10.8
Winnipeg						
CBD Street (1980)	3–4 PM	17.0	74	4.4	—	—

* Compiled by H. Levinson and R. Roess from:

1. *Chicago Loop Pedestrian Movement Study*, City of Chicago, Chicago, Ill., 1973.
2. Pushkarev, B., and Zupan, J., *Urban Space for Pedestrians*, Regional Plan Association, New York, N.Y., 1976.
3. *Traffic Circulation and Parking Plan-CBD Urban Renewal Area-Boston, Mass.*, Barton-Aschman Associates, 1968.
4. "Traffic Characteristics," *Traffic and Transportation Engineering Handbook*, Institute of Transportation Engineers, Prentice-Hall, Englewood Cliffs, N.J., 1976.
5. "Characteristics and Service Requirements of Pedestrians and Pedestrian Facilities," Informational Report, *ITE Journal*, Institute of Transportation Engineers, Washington, D.C., May 1976.
6. Carstens R., and Ring, S., "Pedestrian Capacity of Shelter Entrances," Technical Note, *Traffic Engineering*, Institute of Transportation Engineers, Washington, D.C., December 1970.

only affects pedestrian convenience, but can delay vehicle turning movements, thereby reducing the capacity of the intersection and connecting streets.

The principles of pedestrian flow analysis are similar to those used for vehicular flow. The fundamental relationship between speed, volume, and density is the same. As the volume and density of a pedestrian stream increases from free-flow to more crowded conditions, speed and ease of movement decreases. When the pedestrian density exceeds a critical level, volume and speed become erratic and rapidly decline.

Pedestrian flow on sidewalks is affected by reductions in effective walkway width caused by various items of street "furniture," such as parking meters, light standards, mail boxes, and trash cans, and by interruptions to flow caused by traffic signals. The traffic signal cycle also results in queues of waiting pedestrians at street corners, which decreases corner circulation capacity and concentrates crossing pedestrians into denser platoons.

The level-of-service (LOS) concept, first used to define relative degrees of convenience on highways, is also applicable to pedestrian facilities. With this concept, such convenience factors as the ability to select walking speeds, bypass slower pedestrians, and avoid conflicts with others are related to pedestrian density and volume. The concept can also be applied to degrees of crowding in queuing areas, such as sidewalk corners, transit platforms, and other waiting areas.

The following sections define pedestrian traffic terminology, develop the principles of pedestrian flow, present the concept of pedestrian level of service, and provide detailed analysis procedures for use.

PEDESTRIAN CAPACITY TERMINOLOGY

Pedestrian analysis uses some familiar traffic terms, as well as others not used elsewhere in the manual. The following listing defines the major terms used throughout this chapter:

1. *Pedestrian speed* is the average pedestrian walking speed, generally expressed in units of feet per second.
2. *Pedestrian flow rate* is the number of pedestrians passing a point per unit time, expressed as pedestrians per 15 minutes or pedestrians per minute; "point" refers to a perpendicular line of sight across the width of a walkway.
3. *Unit width flow* is the average flow of pedestrians per unit of effective walkway width, expressed as pedestrians per minute per foot.
4. *Platoon* refers to a number of pedestrians walking together in a group, usually involuntarily, because of signal control and other factors.
5. *Pedestrian density* is the average number of pedestrians per unit of area within a walkway or queuing area, expressed as pedestrians per square foot.
6. *Pedestrian space* is the average area provided for each pedestrian in a walkway or queuing area, expressed in terms of square feet per pedestrian; this is the inverse of density, but is a more practical unit for the analysis of pedestrian facilities.

PRINCIPLES OF PEDESTRIAN FLOW

The qualitative measures of pedestrian flow similar to those used for vehicular flow are the freedom to choose desired speeds and to bypass others. Other measures more specially related to pedestrian flow include the ability to cross a pedestrian traffic stream, to walk in the reverse direction of a major pedestrian flow, and to generally maneuver without conflicts and changes in walking speed or gait.

Additional environmental factors which contribute to the walking experience, and therefore to perceived level of service, are the comfort, convenience, safety, security, and economy of the walkway system.

1. *Comfort* factors include weather protection, climate control, arcades, transit shelters, and other pedestrian amenities.
2. *Convenience* factors include walking distances, pathway directness, grades, sidewalk ramps, directional signing, directory maps, and other features making pedestrian travel easy and uncomplicated.
3. *Safety* is provided by separation of pedestrians from vehicular traffic, horizontally in malls and other vehicle-free areas, and vertically using overpasses and underpasses. Traffic control devices can provide for time separation of pedestrian and vehicular traffic.
4. *Security* features include lighting, open lines of sight, and the degree and type of street activity.
5. *Economy* aspect relates to the user costs associated with travel delays and inconvenience, and to the rental value and retail development as influenced by pedestrian environment.

These supplemental factors can have an important effect on the pedestrian perception of the overall quality of the street environment. While auto users have reasonable control over most of these factors, the pedestrian has virtually no control over them. Although the bulk of this chapter emphasizes level-of-service analysis, which relates primarily to pedestrian flow measures, such as speed and space, these environmental factors should always be considered because they can greatly influence pedestrian activity.

Pedestrian Speed-Density Relationships

The fundamental relationship between speed, density, and volume for pedestrian flow is analogous to vehicular flow. As *volume* and *density* increase, pedestrian *speed* declines. As density increases, and pedestrian space decreases, the degree of mobility afforded the individual pedestrian declines, as does the average speed of the pedestrian stream.

Figure 13-1 shows the relationship between speed and density for a variety of pedestrian classes as determined by four researchers, including two European sources. The density term, when used to describe pedestrian streams and specified in persons per square foot will have small values, generally under 1.00. The reciprocal of density, space, is also shown on Figure 13-1.

Flow-Density Relationships

The relationship between density, speed, and flow for pedestrians is of the same form as for vehicular traffic streams, that is:

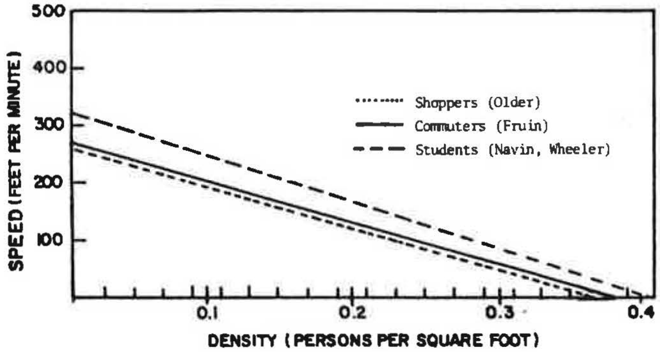


Figure 13-1. Relationships between pedestrian speed and density. (Source: Ref. *2)*

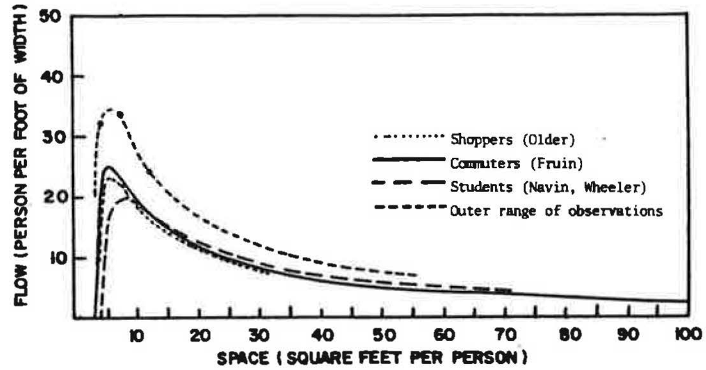


Figure 13-2. Relationships between pedestrian flow and space. (Source: Ref. *2)*

$$\text{Flow} = \text{Speed} \times \text{Density}$$

$$v = S \times D \qquad (13\text{-}1)$$

where *flow* is expressed as pedestrians per minute per foot, *speed* is expressed as feet per minute, and *density* is expressed as pedestrians per square foot.

The flow variable used in this expression is the "unit width flow" defined earlier. An alternative and more useful expression can be developed using the reciprocal of density, or *space,* as follows:

$$\text{Flow} = \text{Speed}/\text{Space}$$

$$v = S/M \qquad (13\text{-}2)$$

The basic relationship between flow and space, as recorded by several researchers, is illustrated in Figure 13-2.

The conditions at maximum flow are of interest because this represents the *capacity* of the walkway facility. From Figure 13-2, it is apparent that all observations of maximum unit flow fall within a very narrow range of density—that is, with the average space per pedestrian varying between 5 and 9 sq ft/ped. Even the outer range of these observations indicates that maximum flow occurs at this density, although the actual flow in this study is considerably higher than the others. As space is reduced to less than 5 sq ft/ped, the flow rate declines precipitously. All movement effectively stops at the minimum space allocation of 2 to 4 sq ft/ped.

These relationships show that pedestrian traffic can be evaluated qualitatively by using level-of-service concepts similar to vehicular traffic analysis. At flows near capacity, an average of 5 to 9 sq ft/ped is required for each moving pedestrian. However, at this level of flow, the limited area available restricts pedestrian speed and the pedestrian's freedom to maneuver within the pedestrian stream.

Speed-Flow Relationships

Figure 13-3 illustrates the relationship between pedestrian speed and flow. These curves, similar to vehicular flow curves, show that when there are few pedestrians on a walkway (low flow levels), space is available to choose higher walking speeds. As flow increases, speeds decline because of closer interactions

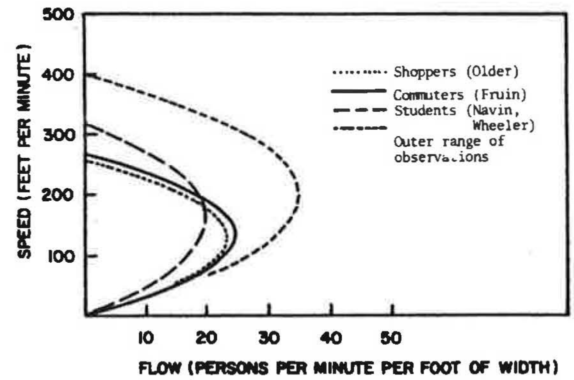


Figure 13-3. Relationships between pedestrian speed and flow. (Source: Ref. *2)*

with other pedestrians. When a critical level of crowding occurs, movement becomes more difficult, and both flow and speed decline.

Speed-Space Relationships

Figure 13-4 further confirms the relationships of walking speed and available space, and suggests some points of demarcation that can be used to develop level-of-service criteria. The outer range of observations shown on Figure 13-4 indicates that at an average space of about 15 sq ft/ped, even the slowest pedestrians cannot achieve their desired walking speed. Faster pedestrians wishing to walk at speeds up to 350 ft/min are not able to achieve such speeds until average space is 40 sq ft/ped or more. The space values of 15 and 40 sq ft/ped become critical points in defining level-of-service boundaries, as is illustrated in the "Methodology" section of this chapter.

EFFECTIVE WALKWAY WIDTH

The concept of a pedestrian "lane" has sometimes been used to analyze pedestrian flow, comparable to the analysis of a

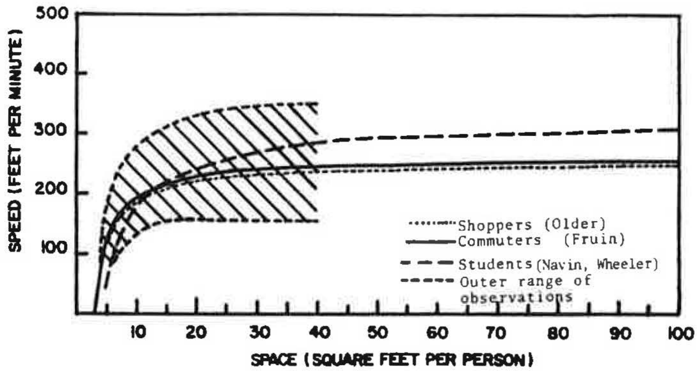


Figure 13-4. Relationships between pedestrian speed and space. (Source: Ref. *2)*

highway lane. The "lane" should not be used in pedestrian analysis, because photographic studies have shown that pedestrians do not walk in organized lanes. The "lane" concept is meaningful only in determining how many persons can walk abreast on a given walkway width, as in the case of determining the minimum sidewalk width to permit two pedestrians to conveniently pass by each other.

To avoid interference while passing each other, two pedestrians should *each* have at least 2.5 ft of walkway width, as observed by Oeding and Pushkarev (*2*). Pedestrians who know each other and are walking close together will each occupy a width of 2 ft, 2 in., a distance at which there is considerable likelihood of contact due to body sway. Lateral spacing less than this occurs only in the most crowded of situations.

The term "clear walkway width" is related to the portion of a walkway that can be effectively used for pedestrian movements. Moving pedestrians will shy away from the curb, and will not press closely against building walls. Therefore, unused space must be subtracted when determining pedestrian LOS. Further, a strip preempted by pedestrians standing near a building (as in window shopping) and/or near physical obstructions such as light poles, mail boxes, and parking meters, should also be excluded.

The degree to which point obstructions (poles, signs, hydrants) influence pedestrian movement and reduce effective walkway width is not extensively documented. While a single such obstruction would not reduce the effective width of an entire walkway, it would have such an effect in the immediate vicinity of the obstruction.

A list of typical obstructions and the estimated width of walkways which they preempt is provided in Table 13-2. Figure 13-5 shows the width of walkway preempted by curbs, buildings,

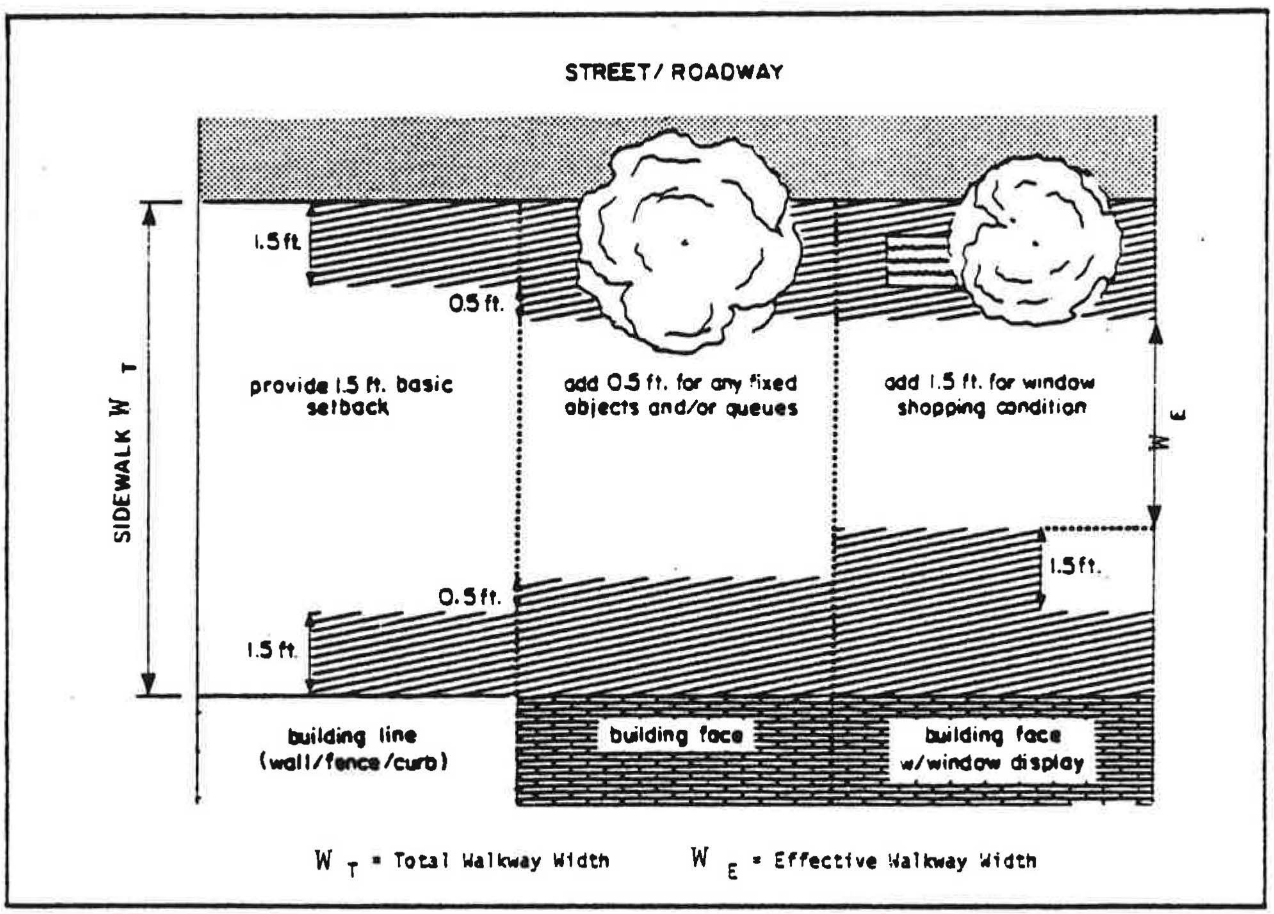


Figure 13-5. Preemption of walkway width. (Source: Ref. *4)*

TABLE 13-2. FIXED OBSTACLE WIDTH ADJUSTMENT FACTORS FOR WALKWAYS*

OBSTACLE	APPROX. WIDTH PREEMPTED (FT)ᵃ
STREET FURNITURE	
Light Poles	2.5–3.5
Traffic Signal Poles and Boxes	3.0–4.0
Fire Alarm Boxes	2.5–3.5
Fire Hydrants	2.5–3.0
Traffic Signs	2.0–2.5
Parking Meters	2.0
Mail Boxes (1.7 ft by 1.7 ft)	3.2–3.7
Telephone Booths (2.7 ft by 2.7 ft)	4.0
Waste Baskets	3.0
Benches	5.0
PUBLIC UNDERGROUND ACCESS	
Subway Stairs	5.5–7.0
Subway Ventilation Gratings (raised)	6.0+
Transformer Vault Ventilation Gratings (raised)	5.0+
LANDSCAPING	
Trees	2.0–4.0
Planting Boxes	5.0
COMMERCIAL USES	
Newsstands	4.0–13.0
Vending Stands	variable
Advertising Displays	variable
Store Displays	variable
Sidewalk Cafes (two rows of tables)	variable, try 7.0
BUILDING PROTRUSIONS	
Columns	2.5–3.0
Stoops	2.0–6.0
Cellar Doors	5.0–7.0
Standpipe Connections	1.0
Awning Poles	2.5
Truck Docks (trucks protruding)	variable
Garage Entrance/Exit	variable
Driveways	variable

* To account for the avoidance distance normally occurring between pedestrians and obstacles, an additional 1.0 to 1.5 ft must be added to the preemption width for individual obstacles.
ᵃ Curb to edge of object, or building face to edge of object.
SOURCE: Ref. 2

or fixed objects. Figure 13-5 may be used as a guideline when specific walkway configurations are not available.

PEDESTRIAN TYPE AND TRIP PURPOSES

The analysis of pedestrian flow is generally based on *mean*, or average, walking speeds of groups of pedestrians. Within any group, or among groups, there can be considerable differences in flow characteristics due to trip purposes, land use, type of group, age, and other factors. Figure 13-6 shows a typical distribution of free-flow walking speeds.

Pedestrians going to and from work, using the same facilities day after day, exhibit higher walking speeds than shoppers. This has been shown in Figure 13-1. Older or very young persons will tend to walk at a slower gait than other groups. Shoppers not only tend to walk slower than commuters, but may decrease the effective walkway width by stopping to window shop. Thus, in applying the techniques and numerical data in this chapter, the analyst should adjust for pedestrian behavior which deviates from the regular patterns represented in the basic speed volume and density curves.

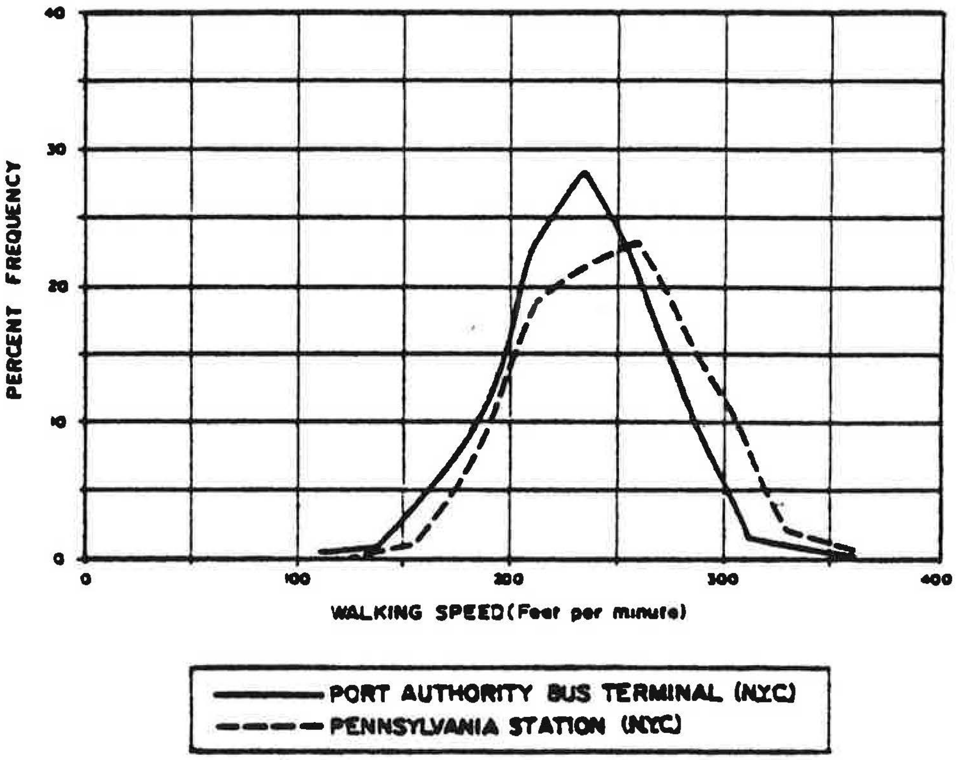


Figure 13-6. Typical free-flow walkway speed distribution. (Source: Ref. *3)*

II. METHODOLOGY

LEVELS OF SERVICE IN WALKWAYS

The criteria for various levels of service (LOS) for pedestrian flow are based on subjective measures that may be somewhat imprecise. However, it is possible to define ranges of *space per pedestrian, flow rates,* and *speeds* which can be used to develop quality of flow criteria.

Speed is an important level-of-service criterion because it can be easily observed and measured, and because it is a descriptor of the service pedestrians perceive. At speeds of 150 ft/min or less, most pedestrians resort to an unnatural "shuffling" gait. Figure 13-4 shows that this speed corresponds to a space per pedestrian in the range of 6 to 8 sq ft/ped. At 15 sq ft/ped or less, even the slowest walkers are forced to slow down (shown by the cross-hatching in Figure 13-4). The fastest walkers cannot reach their chosen speed of 350 ft/min until areas are over 40 sq ft/ped. Further, from Figure 13-2, it is evident that these three space values, 6, 15, and 40 sq ft/ped correspond approximately to the maximum flow at capacity, two-thirds of capacity, and one-third of capacity, respectively.

There are other significant indicators of service levels. For example, the ability of the pedestrian to cross a pedestrian stream is shown by Fruin (*3*) in Figure 13-7 to be impaired at areas below the 35- to 40-sq ft/ped range. Above that level, Fruin states that the probability of "stopping or breaking the normal walking gait" is reduced to zero. Below 15 sq ft/ped, virtually every crossing movement encounters a conflict. Similarly, the ability to pass slower pedestrians is unimpaired above 35 sq ft/ped, but becomes progressively more difficult as space allocations drop to 18 sq ft/ped, a point at which passing becomes virtually impossible.

Another level-of-service indicator is the ability to maintain flow in the minor direction in opposition to a major pedestrian flow. Here the quantitative evidence is somewhat less precise. For pedestrian streams of roughly equal flow in each direction, there is little reduction in the capacity of the walkway compared with one-way flow, because the directional streams tend to separate and occupy a proportional share of the walkway. However, if the bidirectional split is 90-10, and space is 10 sq ft/ped, capacity reductions of about 15 percent have been observed. This reduction is a consequence of the inability of the minority flow to utilize a proportional share of the walkway.

Photographic studies show that pedestrian movement on sidewalks is affected by the presence of other pedestrians, even at areas above 40 sq ft/ped. At 60 sq ft/ped, pedestrians have been observed walking in a "checkerboard" pattern, rather than

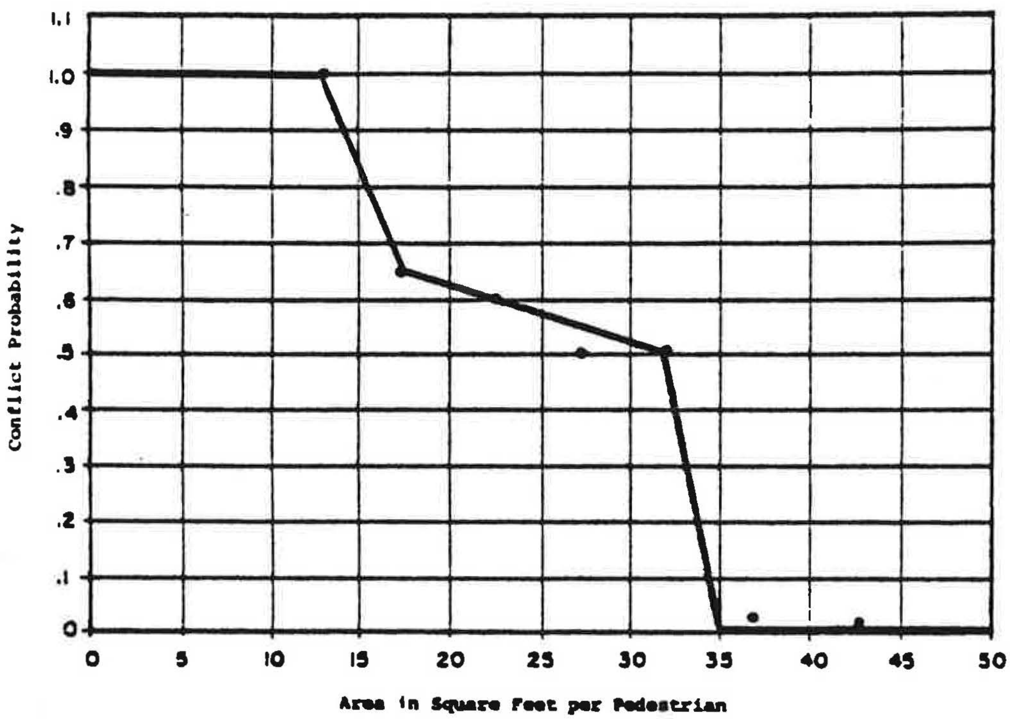


Figure 13-7. Cross-flow traffic—probability of conflict. (Source: Ref. *3)*

TABLE 13-3. PEDESTRIAN LEVEL OF SERVICE ON WALKWAYS*

LEVEL OF SERVICE	SPACE (SQ FT/PED)	EXPECTED FLOWS AND SPEEDS		
		AVE. SPEED, S (FT/MIN)	FLOW RATE, v (PED/MIN/FT)	VOL/CAP RATIO, v/c
A	$\geq$ 130	$\geq$ 260	$\leq$ 2	$\leq$ 0.08
B	$\geq$ 40	$\geq$ 250	$\leq$ 7	$\leq$ 0.28
C	$\geq$ 24	$\geq$ 240	$\leq$ 10	$\leq$ 0.40
D	$\geq$ 15	$\geq$ 225	$\leq$ 15	$\leq$ 0.60
E	$\geq$ 6	$\geq$ 150	$\leq$ 25	$\leq$ 1.00
F	< 6	< 150	----Variable----	

* Average conditions for 15 min.

directly behind or alongside each other. These same observations suggest that up to 100 sq ft/ped are required before completely free movement occurs without conflicts, and that at 130 sq ft/ped, individual pedestrians are no longer influenced by others (*5*). Bunching or "platooning" does not completely disappear until space is about 500 sq ft/ped or higher.

Walkway Level-of-Service Criteria

Table 13-3 shows the criteria for pedestrian level of service. The primary measure of effectiveness used in defining pedestrian level of service is *space,* the inverse of density. Mean speed and flow rate are shown as supplementary criteria. *Capacity* is taken to be 25 ped/min/ft, a representative value from Figures 13-2 and 13-3.

Graphic illustrations and descriptions of walkway levels of service are shown in Figure 13-8.

It should be noted that the pedestrian LOS, according to the criteria of Table 13-3, is quite good in most areas, as the high pedestrian flows required for the poorer levels generally occur only in and around major activity centers. In most areas, the design of walkways is based on the minimum widths required for voluntary pedestrian groups to pass each other and similar factors, rather than on the flow rate.

The LOS criteria apply to pedestrian flow and the space provided for that flow. Pedestrian facilities may also include extensive space intended to enhance the general environment that is not used or intended to handle basic pedestrian movements. When analyzing pedestrian flow rates per unit width of walkway, such space *should not be included.* Thus, pedestrian space intended to provide for window shopping, browsing, or

LEVEL OF SERVICE A

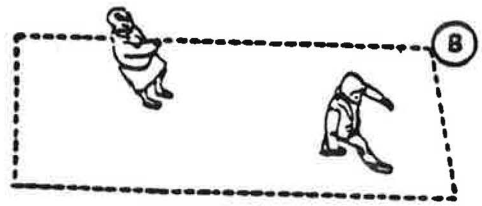
Pedestrian Space: ≥ 130 sq ft/ped Flow Rate: ≤ 2 ped/min/ft

At walkway LOS A, pedestrians basically move in desired paths without altering their movements in response to other pedestrians. Walking speeds are freely selected, and conflicts between pedestrians are unlikely.

LEVEL OF SERVICE B

Pedestrian Space: ≥ 40 sq ft/ped Flow Rate: ≤ 7 ped/min/ft

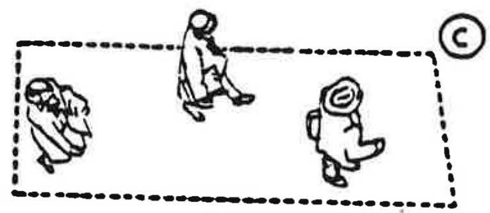
At LOS B, sufficient area is provided to allow pedestrians to freely select walking speeds, to bypass other pedestrians, and to avoid crossing conflicts with others. At this level, pedestrians begin to be aware of other pedestrians, and to respond to their presence in the selection of walking path.



LEVEL OF SERVICE C

Pedestrian Space: ≥ 24 sq ft/ped Flow Rate: ≤ 10 ped/min/ft

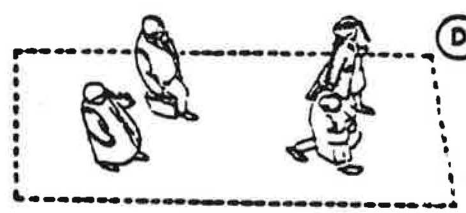
At LOS C, sufficient space is available to select normal walking speeds, and to bypass other pedestrians in primarily unidirectional streams. Where reverse-direction or crossing movements exist, minor conflicts will occur, and speeds and volume will be somewhat lower.



LEVEL OF SERVICE D

Pedestrian Space: ≥ 15 sq ft/ped Flow Rate: ≤ 15 ped/min/ft

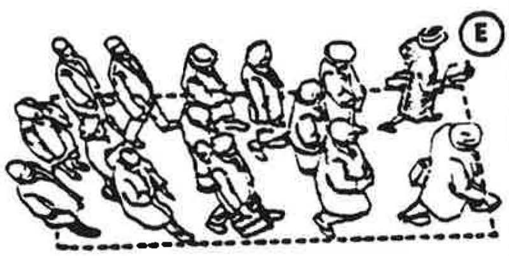
At LOS D, freedom to select individual walking speed at to bypass other pedestrians is restricted. Where crossing or reverse-flow movements exist, the probability of conflict is high, and its avoidance requires frequent changes in speed and position. The LOS provides reasonably fluid flow, however, considerable friction and interaction between pedestrians is likely to occur.



LEVEL OF SERVICE E

Pedestrian Space: ≥ 6 sq ft/ped Flow Rate: ≤ 25 ped/min/ft

At LOS E, virtually all pedestrians would have their normal walking speed restricted, requiring frequent adjustment of gait. At the lower range of this LOS, forward movement is possible only by "shuffling." Insufficient space is provided for passing of slower pedestrians. Cross- or reverse-flow movements are possible only with extreme difficulties. Design volumes approach the limit of walkway capacity, with resulting stoppages and interruptions to flow.



LEVEL OF SERVICE F

Pedestrian Space: ≤ 6 sq ft/ped Flow Rate: variable

At LOS F, all walking speeds are severely restricted, and forward progress is made only by "shuffling." There is frequent, unavoidable, contact with other pedestrians. Cross- and reverse-flow movements are virtually impossible. Flow is sporadic and unstable. Space is more characteristic of queued pedestrians than of moving pedestrian streams.

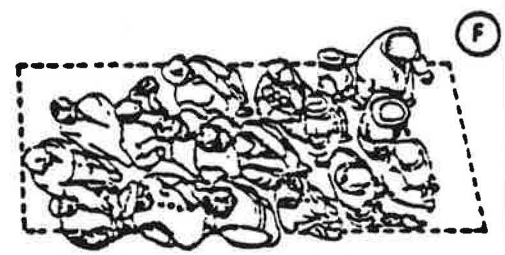


Figure 13-8. Illustration of walkway levels of service.

simply sitting or standing in informal groups should not be considered to be part of the effective walkway width.

It should also be emphasized that the level-of-service criteria of Table 13-3 are based on the assumption that pedestrians distribute themselves uniformly throughout the effective walkway width. Pedestrian flow is subject to wide variability on a minute-by-minute basis, and the analyst must consider the effects of platooning as described in the next section.

Effect of Pedestrian Platoons

The average flow rates at different levels of service are of limited usefulness unless reasonable time intervals are specified. Figure 13-9 illustrates that "average flow rates" can be misleading. The data shown are for two locations in Lower Manhattan, but the pattern is generally characteristic of many concentrated CBD locations. The maximum 15-min flow rates average 1.4 and 1.9 ped/min/ft of effective walkway width during the periods measured. However, Figure 13-9 shows that flow during a 1-min interval can be more than double the rate in another, particularly at relatively low flows. Even during the peak 15-min period, incremental variations of 50 to 100 percent frequently occur from one minute to the next.

Depending on traffic patterns, it is clear that a facility designed for average flow can afford lower quality of flow for a proportion of the pedestrian traffic using it. However, it is extravagant to design for extreme peak 1-min flows which occur only 1 percent or 2 percent of the time. A relevant time period must therefore be determined through closer evaluation of the short-term fluctuations of pedestrian flow.

Short-term fluctuations are present in most unregulated pedestrian traffic flows because of random arrivals of pedestrians. On sidewalks, these random fluctuations are further exaggerated by the interruption of flow and queue formation caused by traffic signals. Transit facilities can create added surges in demand by releasing large groups of pedestrians in short time intervals, followed by pauses during which no flow occurs. Until they disperse, pedestrians in these types of groups move together as a platoon. Platoons can also form if passing is impeded because of insufficient space, and faster pedestrians slow down behind slower walkers.

It is important for the analyst to determine if platooning or other traffic patterns alter the underlying assumptions of average flow in LOS calculations, and to make appropriate adjustments where necessary.

In walkway sections having pronounced platooning effects, the duration and magnitude of these variations in demand should be established. This is done by timing and counting these short-term surges in demand. The magnitude and frequency of occurrence of the platoons would then be compared to the longer term 15-min average flow to provide a more accurate view of LOS conditions on the walkway segment.

The scatter diagram shown in Figure 13-10 indicates the platoon flow rate (i.e., the rate of flow within platoons of pedestrians) in comparison to the average flow rate for 58 data periods of 5- to 6-min duration. The dashed line approximates the upper limit of platoon flow observations.

The mathematical expression of this line relating maximum platoon flow rates to average flow rates is:

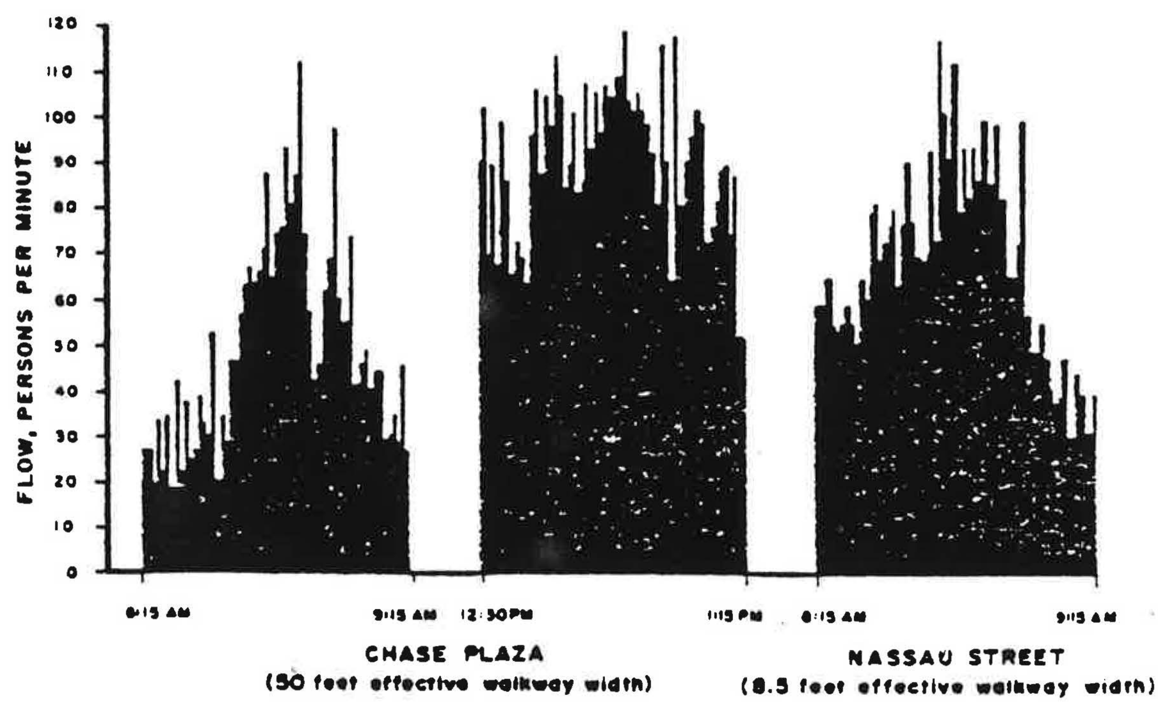


Figure 13-9. Minute-by-minute variations in pedestrian flow. (Source: Ref. *2*)

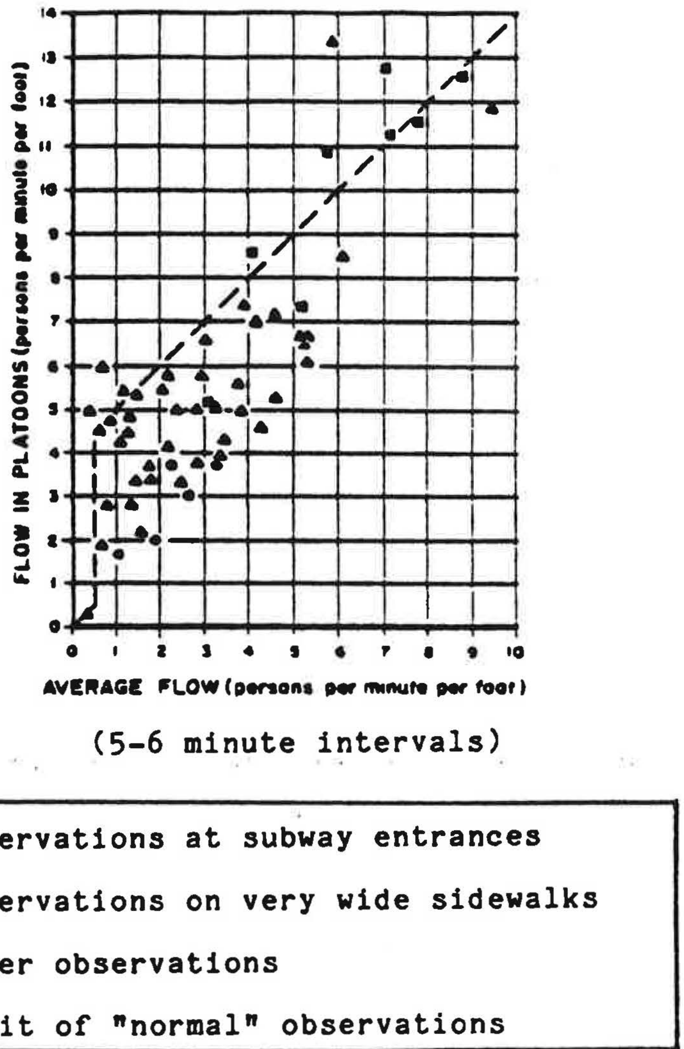


Figure 13-10. Relationship between platoon flow and average flow. (Source: Ref. *2).*

$$\text{Platoon Flow} = \text{Average Flow} + 4$$
$$v_p = v + 4 \qquad (13\text{-}3)$$

where both flows are expressed as pedestrians per minute per foot. This equation is valid for flows greater than 0.5 ped/min/ft. For lower flows, consult Figure 13-10 directly.

The form that this equation takes—a constant increment added to the average flow—shows that platooning has a relatively greater impact at low volumes than at high volumes. This pattern is logical, because gaps between platoons tend to fill up as flow increases. The equation can be used in general analyses where specific platooning data are not available.

Although the magnitude and frequency of platoons should be verified by field studies, the LOS occurring in platoons is generally *one level poorer than that determined by average flow criteria,* except for some cases of LOS E, which encompasses a broad range of pedestrian flow rates. The selection of an appropriate design objective to accommodate either average flows over a longer period, or the surges in demand occurring in platoons, depends on an evaluation of pedestrian convenience, available space, costs, and policy considerations.

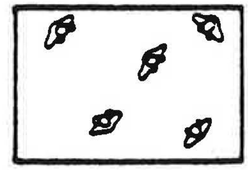
LEVELS OF SERVICE IN QUEUING AREAS

The concept of using the average space available to pedestrians as a walkway level-of-service measure can also be applied to queuing or waiting areas. In such areas, the pedestrian stands temporarily, while waiting to be served. The LOS of the waiting area is related to the average space available to each pedestrian and the degree of mobility allowed. In dense standing crowds, there is little room to move, but limited circulation is possible as average space per pedestrian is increased.

Level-of-service descriptions for standing spaces based on average pedestrian space, personal comfort, and degrees of internal mobility are shown on Figure 13-11. Standing areas in the LOS E category of 2–3 sq ft/ped are experienced only in the most crowded elevators or transit vehicles. LOS D, at 3–7 sq ft/ped, more typically exists where there is crowding, but where some internal maneuverability is still present. This commonly occurs at sidewalk corners where a large group of pedestrians is waiting to cross. Waiting areas where more space is required for circulation, such as theater lobbies and transit platforms, also require a higher LOS.

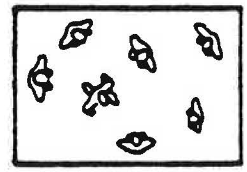
LEVEL OF SERVICE A

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.



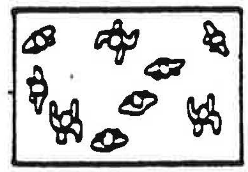
LEVEL OF SERVICE B

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.



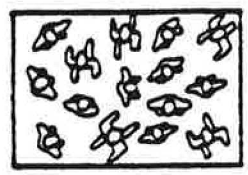
LEVEL OF SERVICE C

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.



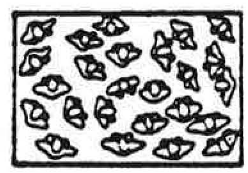
LEVEL OF SERVICE D

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.

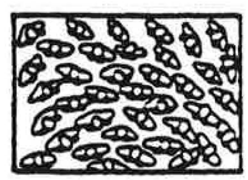


Figure 13-11. Levels of service for queuing areas. (Source: Ref. *3)*

APPLICATION OF CRITERIA

The application of these LOS criteria is relatively straightforward for walkways and waiting areas, as indicated in the previous sections. Two remaining pedestrian facilities of interest, however, present more complicated situations: street corners and crosswalks. Each of these is briefly discussed in the following sections.

Street Corners

The street corner is a more complex problem than the mid-block situation, involving intersecting sidewalk flows, pedestrians crossing the street, and others queued waiting for the signal to change. Because of the concentration of these activities, the corner is often the critical link in the pedestrian sidewalk network. An overloaded street corner can also affect vehicular operations by requiring added green crossing time or by delaying turning movements. There are two different types of pedestrian area requirements at corners:

1. *Circulation area*—Needed to accommodate (a) pedestrians crossing during the green signal phase, (b) those moving to join the red phase queue, and (c) those moving between the adjoining sidewalks, but not crossing the street.
2. *Hold area*—Needed to accommodate standing pedestrians waiting during the red signal phase.

Precise analysis of pedestrian activity at corners is difficult because of the many combinations of movements that are possible, as is illustrated in Figure 13-12. Each of the four directional movements into the corner may proceed straight ahead, or may turn left or right. This makes accurate collection of field data at busy intersections an almost impossible task. Methods for determining approximate LOS of street corners using more typically available crossing count data are given in the "Procedures for Application" section of this chapter.

The methodology is relatively straightforward and is adequate to establish problem locations which may require more detailed field study and possible remedial measures. Corrective measures could include sidewalk widenings, vehicle-turning restrictions, and/or changes in signal timing. Identifying problem areas is a primary objective of using LOS as an analytic tool.

Corners function as a "time-space" zone, with waiting pedestrians requiring less standing space, but occupying the corner for longer periods of time, and moving or circulating pedestrians requiring more space, but occupying the corner for only a few seconds. The total time-space available for these activities is simply the net area of the corner in square feet multiplied by the time of the analysis period. The analytical problem is allocation of this time-space in ways that provide a reasonable corner LOS for both waiting and moving pedestrians.

The method assumes that standing pedestrians waiting for the signal to change form a "competitive queue," in which each pedestrian occupies 5 sq ft/ped. This assumes midrange LOS D conditions within the queue, typical of many urban situations, and simplifies computational procedures. The average time moving pedestrians occupy the corner, typically in the range of 3 to 5 sec, is also assumed. This assumption of the travel time along the path of the longest dimension of a corner is actually conservative, as many pedestrians "short cut" corner edges, reducing their time-space requirements.

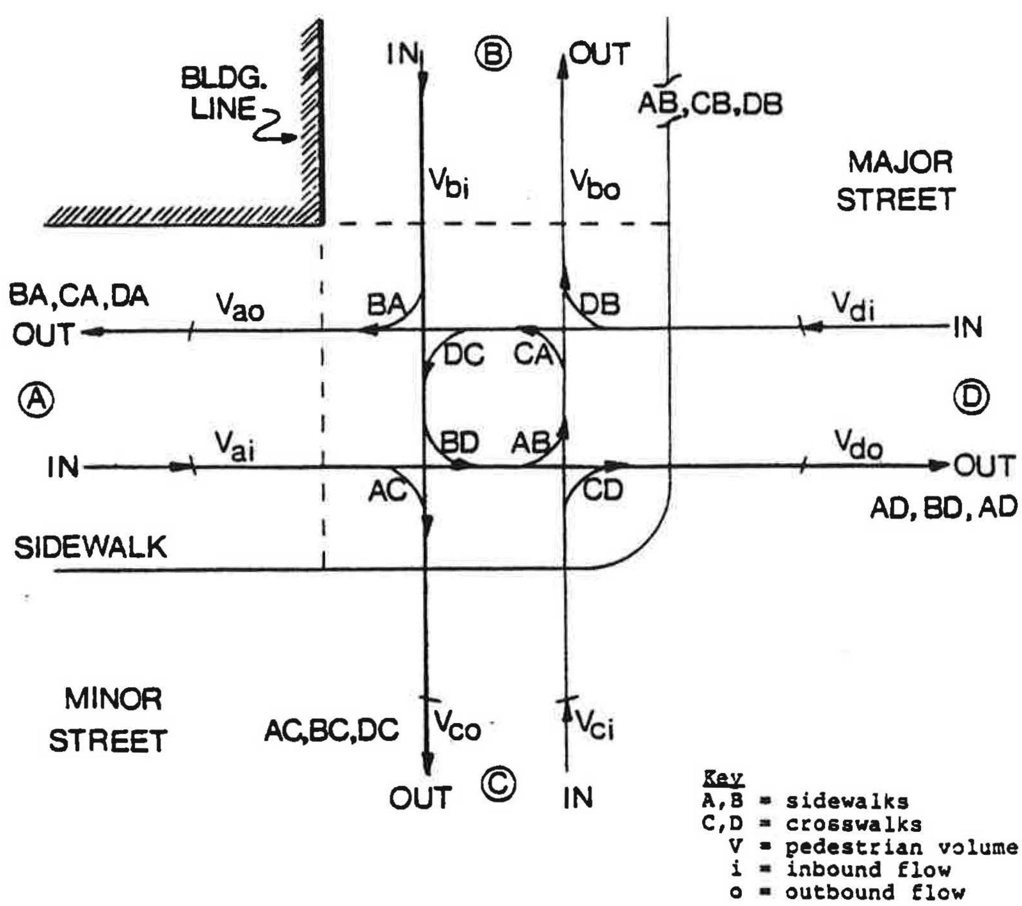


Figure 13-12. Pedestrian movements at a street corner.

Crosswalks

Pedestrian flow characteristics in crosswalks are similar to those on sidewalks, with the basic relationships of speed, density, space, and flow consistent with observed values for uninterrupted flow on walkways. However, traffic signals control movement on the crosswalk, collecting pedestrians into denser platoons, and altering the normal distribution of walking speeds.

Level-of-service concepts developed primarily for movement of pedestrians on walkways can be applied to crosswalk analysis, but signal timing and the effects of turning vehicles during the pedestrian green phase can alter the underlying assumptions of the LOS analysis. Where crosswalk analyses show low pedestrian LOS, vehicle-turning restrictions must be seriously considered.

Like corners, the crosswalk can also be analyzed as a time-space zone. The available time-space is the product of the WALK phase time less 3 sec platoon start-up time, and the area of the crosswalk in square feet. The product of pedestrian crossing flow and the average crossing time results in the demand for the space. Division of demand into the available time-space produces the space per moving pedestrian available during the green phase. This area can be compared with LOS criteria.

However, there is a brief maximum flow or surge condition during the WALK phase which must be examined. This occurs when the two lead platoons from opposite corners, formed during the waiting phase, are simultaneously in the crosswalk. Excessive pedestrian flows during this surge could cause pedestrians to drift out of the marked crosswalk area, potentially endangering them.

Neither the average nor the maximum estimate of crosswalk LOS accounts for the effects of turning vehicles during the pedestrian crossing phase. Rough estimates of pedestrian LOS degradation by turning vehicles can be made by assuming a vehicle swept path area and time in the crosswalk (time-space) decrement for each turning vehicle. An example of this is shown in the "Procedures for Application" section of this chapter. It should be noted, however, that the nature of pedestrian-vehicle interactions in the crosswalk may be greatly influenced by local right-of-way practices.

Further research is required on the interactions of pedestrian and vehicle flows in crosswalks to establish threshold values at which turning vehicles begin to seriously degrade the pedestrain crossing movement, or conversely, where heavy crosswalk flows restrict vehicular operations.

III. PROCEDURES FOR APPLICATION AND SAMPLE CALCULATIONS

In this chapter procedures for application and sample calculations are presented as a cohesive unit. Since procedures for analysis of walkways, street corners, and crosswalks are all relatively unique, illustrative calculations are shown with each procedural presentation.

ANALYSIS PROCEDURES FOR WALKWAYS

Computations for walkways are based on peak 15-min pedestrian counts. A midblock walkway should be counted for several different time periods during the day to establish variances in directional flows. For new locations or to analyze future conditions, forecasts of the flows must be made. Methods of forecasting pedestrian trip volumes and pedestrian trip generation rates for various types of land uses are contained in Ref. *8*.

Computational Steps

The methodology requires a specific sequence of computations which is presented below. Figure 13-13 is a worksheet which may be used in summarizing these computations.

1. Preliminary data needed to conduct an analysis include the following. For existing cases, field studies would be made to collect the information; for future cases, forecasts of demand and probable designs would be assumed:

- Peak 15-min pedestrian count, V_{p15}, in peds/15 min.
- Total walkway width, W_T, in ft
- Identification of obstacles in the walkway

2. The effective width of the walkway, W_E, must be determined by subtracting any unusable width from the total walkway width, W_T. Table 13-2 and Figure 13-5 can be used to estimate the unusable portion of walkway width.

3. The pedestrian unit flow rate, in ped/min/ft, is computed as:

$$v = V_{p15}/15W_E$$

4. The rate of flow within platoons may be estimated as:

$$v_p = v + 4$$

5. Levels of service for average or platoon conditions are found by comparing these flow rates to the criteria of Table 13-3.

Sample Calculation

1. *Description*—A given sidewalk segment on Third Street has a peak 15-min pedestrian flow of 1,250 ped/15 min. The 14-ft sidewalk has a curb on one side and stores with window shopping displays on the other. There are no other sidewalk obstructions. At what LOS does the sidewalk operate, on the average and within platoons?

WALKWAY ANALYSIS WORKSHEET

Location ______________________ COUNTS

City, State ______________________ Date ________

Time ________

Curb Line/Sidewalk Edge

PK 15-MIN FROM ____ to ____

W_{B1}(curb) = ft

W_{B2}(street furn.) = ft

V_1= ____

W_T= ____ W_E(effective width) = ft

V_2= ____ (ped/15 min)

W_{B3}(window shop.) = ft

W_{B4}(bldg protrusions) = ft

W_{B5}(inside clearance) = ft

Wall line/Sidewalk Edge

Pedestrian Volume

V_1 = ________ ped/15 min

V_2 = ________ ped/15 min

$V_p = V_1+V_2$ = ________ ped/15 min

Walkway Width

W_T = ________ ft

$W_B = W_{B1}+W_{B2}+W_{B3}+W_{B4}+W_{B5}$ = ________ ft

$W_E = W_T-W_B$ = ________ ft

Average Walkway LOS

$v = V_p/15W_E$ = ________ ped/min/ft

Average LOS = ________ (Table 13-3)

Platoon Walkway LOS

$v_p = v + 4$ = ________ ped/min/ft

Platoon LOS = ________ (Table 13-3)

Figure 13-13. Worksheet for walkway analysis.

2. *Solution*—The total sidewalk width of 14 ft must be reduced to account for unused "buffer" areas at the curb and building line. From Figure 13-5, the curb buffer is 1.5 ft, and the building buffer (with window shopping assumed) is 3.0 ft. Thus, the effective walkway width is 14.0 − 1.5 − 3.0 = 9.5 ft, and it is this figure that is used to determine the average and platoon flow rates.

The average unit width flow rate is computed as:

$$v = V_{P15}/15W_E$$
$$v = 1{,}250/(15 \times 9.5) = 8.8 \text{ ped/min/ft}$$

The rate of flow within platoons may then be estimated as:

$$v_p = v + 4$$
$$v = 8.8 + 4 = 12.8 \text{ ped/min/ft}$$

Table 13-3 is entered with these flow values to estimate the level of service. The LOS for average conditions is *C*, while the LOS within platoons is estimated to be *D*.

These computations can be summarized on the walkway analysis worksheet, as illustrated in Figure 13-14.

ANALYSIS PROCEDURES FOR STREET CORNERS AND CROSSWALKS

As noted previously, the analysis of street corners requires consideration of the amount of *circulation area* available for pedestrians moving through the corner, and the amount of *holding area* required for standing pedestrians waiting to cross the street. Figure 13-15 illustrates the geometrics of a typical street

WALKWAY ANALYSIS WORKSHEET

Location: Third Street
City, State: Hempstead NY

COUNTS
Date: 6/20/84
Time: PM
PK 15-MIN FROM 4:15 to 4:30

Curb Line/Sidewalk Edge

W_{B1}(curb) = 1.5 ft
W_{B2}(street furn.) = 0 ft
W_T = 14' — W_E(effective width) = 9.5 ft
W_{B3}(window shop.) = 3.0 ft
W_{B4}(bldg protrusions) = 0 ft
W_{B5}(inside clearance) = 0 ft

Wall line/Sidewalk Edge

← V_1 = 625
→ V_2 = 625
(ped/15 min)

Pedestrian Volume

V_1 = 625 ped/15 min
V_2 = 625 ped/15 min
$V_p = V_1 + V_2$ = 1250 ped/15 min

Walkway Width

W_T = 14.0 ft
$W_B = W_{B1}+W_{B2}+W_{B3}+W_{B4}+W_{B5}$ = 4.5 ft
$W_E = W_T - W_B$ = 9.5 ft

Average Walkway LOS

$v = V_p/15W_E$ = 8.8 ped/min/ft
Average LOS = C (Table 13-3)

Platoon Walkway LOS

$v_p = v + 4$ = 12.8 ped/min/ft
Platoon LOS = D (Table 13-3)

Figure 13-14. Illustration of solution to walkway problem.

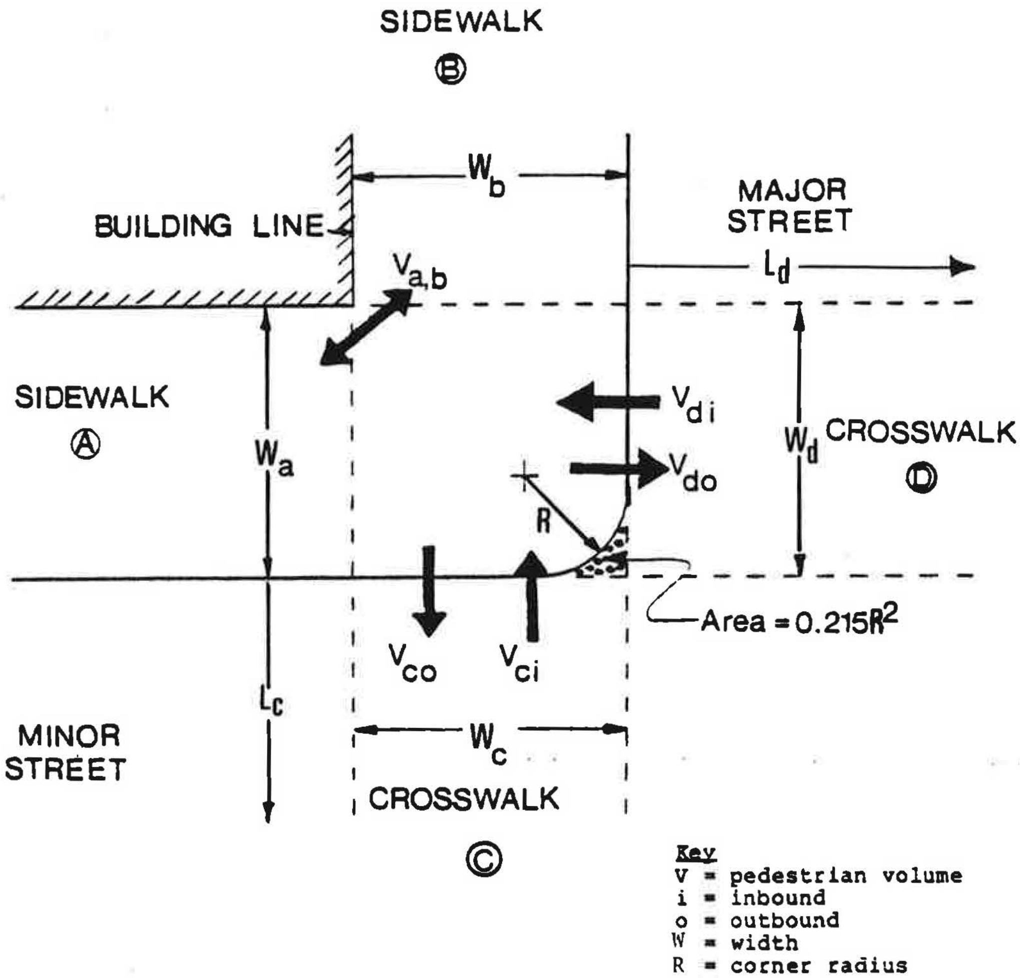


Figure 13-15. Intersection corner geometrics and pedestrian movements.

corner, and also the directional flow variables which will be used in subsequent LOS analyses.

Figures 13-16 and 13-17 show the two signal phase conditions which are analyzed both in corner and crosswalk computations. *Condition 1* is the minor street crossing phase during the major street green, and held in a queue on the major street side during the minor street red phase. *Condition 2* is the major street crossing phase, with pedestrians crossing during the minor street green, and held in a queue on the minor street side by the minor street red phase.

When making street corner computations, it is advisable to refer to Figures 13-15, 13-16, and 13-17 for graphic illustrations of the various parameters used.

The point of maximum pedestrian queuing and minimum available circulation space on the corner occurs just before the signal phase change. At this time, there is an average flow of outbound pedestrians leaving the corner, a more concentrated platoon of inbound pedestrians approaching from the opposite side of the street, and an average flow joining the pedestrian queue waiting to cross at the signal change. At this same time, there are also pedestrians moving between the intersecting sidewalks, not crossing the street.

The analysis of street corners and crosswalks is based on a comparison of available time and space to pedestrian demand. The product of time and space, i.e., time-space, is the critical parameter for consideration, because physical design limits available space and signalization controls available time.

In order to simplify the presentation and application of the time-space analysis approach, the development of relationships (equations) is presented in parallel with the solution of a sample calculation. Worksheets are illustrated in Figures 13-18 and 13-19 for crosswalk and street corner calculations respectively.

The sample calculation illustrated in the analysis of street corners and crosswalks is as follows:

1. *Description*—The sidewalks at a major and minor street intersection are each 16 ft wide, with a corner radius of 20 ft. The roadway width for the major street is 46 ft; and for the minor street, 28 ft. The signal cycle length, C, is 80 sec with a two-phase split of 48 sec of green plus amber, G_{mj}, for the major street (60 percent) and 32 sec of green plus amber, G_{mi}, for the minor street (40 percent). The 15-min peak period pedestrian crossing and sidewalk counts are shown below. Refer to Figure 13-15 for a graphic definition of flows.

Flow	*Peak 15-Min Pedestrian Count*	*Average Flow Rate (ped/min)*	*Average Flow Per Cycle (ped/cycle)*
v_{ci}	540	36	48
v_{co}	300	20	27
v_{di}	450	30	40
v_{do}	240	16	21
$v_{a,b}$	225	15	20
Totals	1,755	117	156

Note that flow rates in pedestrians per minute are rounded to the nearest integer. Pedestrians per cycle are computed by multiplying pedestrians per minute by the signal cycle length (in seconds) divided by 60 sec. For this calculation, the multiplier is 80 sec/60sec = 1.33. Pedestrians per cycle are also rounded to the nearest integer.

2. *Find—*

- The average LOS for pedestrian circulation at the street corner during a typical peak-period signal cycle.
- The average LOS for pedestrians crossing the minor and major streets.
- The decrement in average crosswalk pedestrian LOS due to five turning vehicles per cycle on the major street crossing.

Procedures for analysis of street corners and crosswalks are presented in a step-by-step fashion, along with the solution of the sample calcuation.

Street Corner Analysis (Computational Steps and Sample Calculation)

Step 1—Determine Total Available Time-Space

The total time-space available in the intersection corner for circulation and queuing, for an analysis period of t minutes, is the product of the net corner area, A, and the time t. For street corner and crosswalk analysis, t is taken to be one signal cycle and is, therefore, equal to the cycle length, C. The net corner

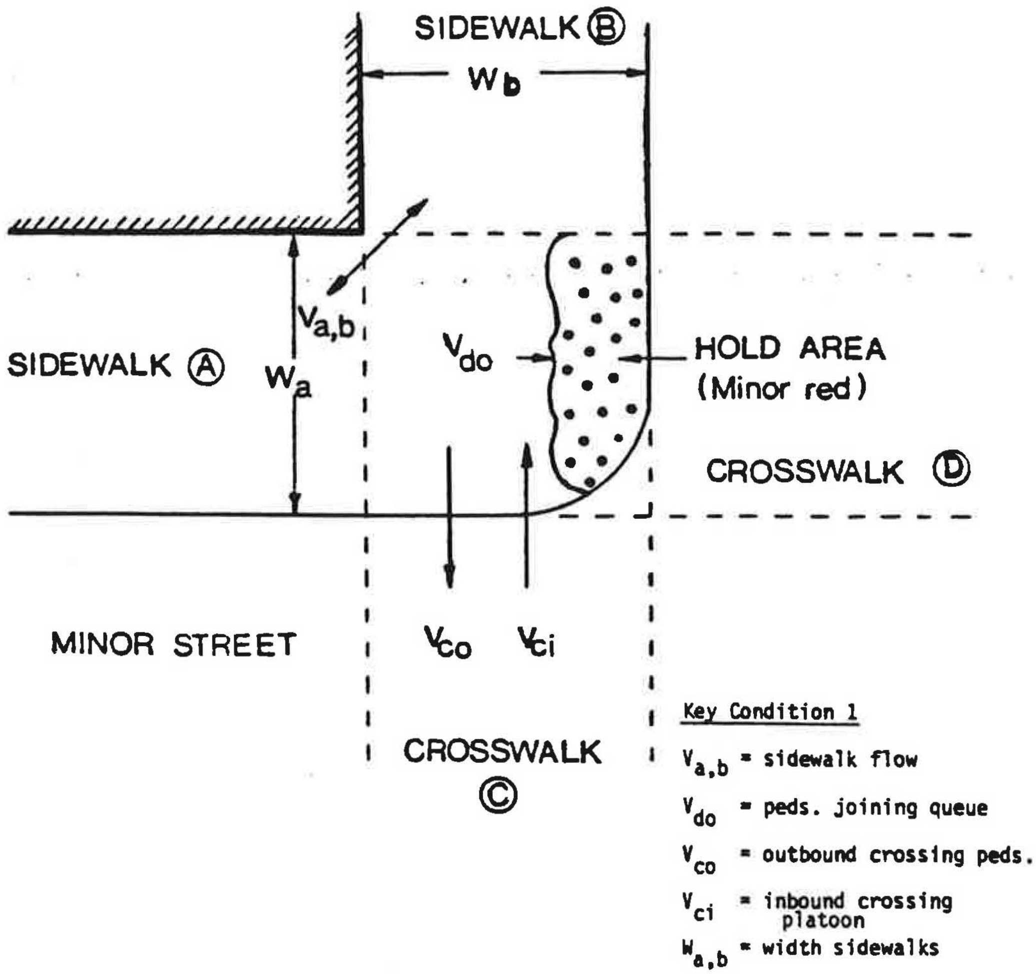


Figure 13-16. Intersection corner Condition 1—minor street crossing.

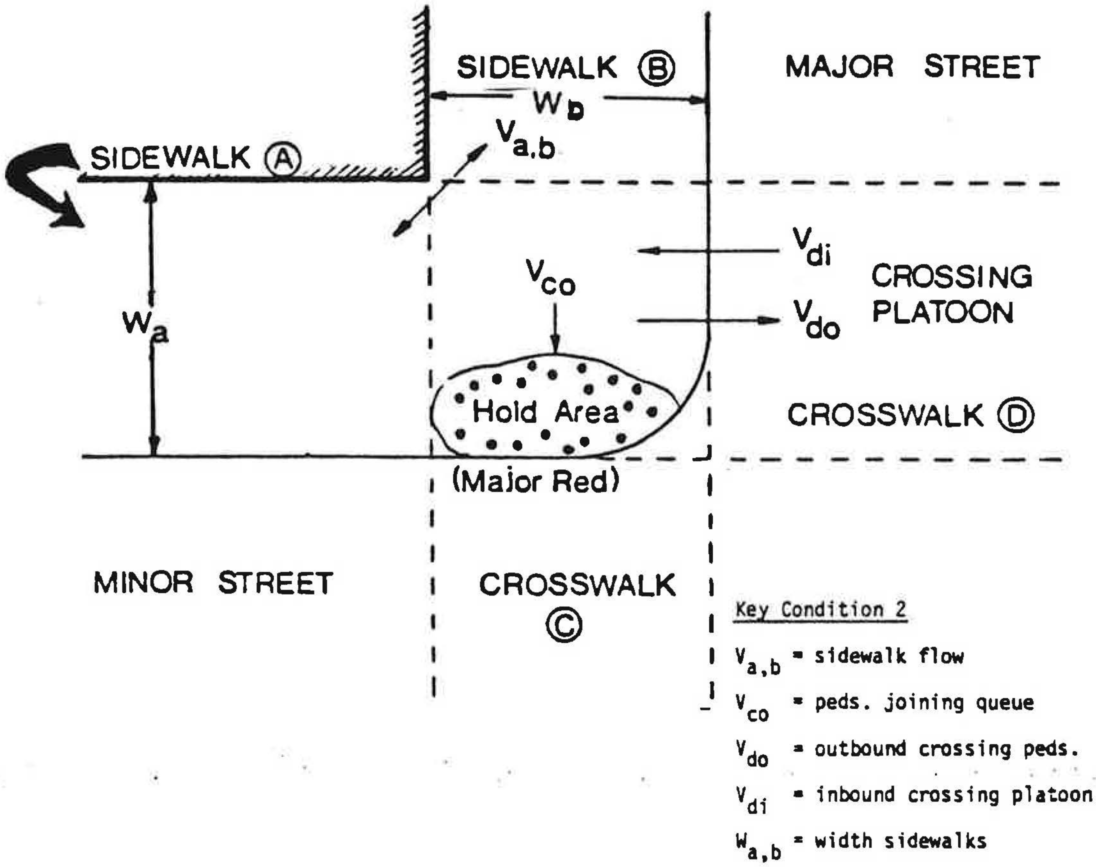


Figure 13-17. Intersection corner Condition 2—major street crossing.

area is found by multiplying the intersecting sidewalk widths, W_a and W_b, and deducting the area lost due to the corner radius and any obstructions. Then, assuming there are no obstructions in the corner area:

$$A = W_a W_b - 0.215 R^2 \qquad (13\text{-}4)$$

$$TS = A \times C/60 \qquad (13\text{-}5)$$

where:

A = area of the street corner, in sq ft;
W_a = width of the sidewalk *a*, in ft;
W_b = width of sidewalk *b*, in ft;
R = radius of corner curb, in ft.
C = cycle length, in sec; and
TS = total time-space available, in sq ft-min.

For the sample calculation described earlier, the following values may be computed:

$A = (16 \times 16) - 0.215\,(20^2) = 170$ sq ft
$TS = 170 \times 80/60 = 227$ sq ft-min

Step 2—Compute Holding Area Waiting Times

If uniform arrivals are assumed at the crossing queues, the average pedestrian holding times, Q_{tco} and Q_{tdo}, of persons waiting to use crosswalks *C* and *D*, respectively, is 1/2 the product of the outbound flows during a signal cycle (v_{co} and v_{do}, in ped/cycle), the proportion of cycle that these flows are held up, and their holding time based on the red signal phase:

For Condition 1, the minor street crossing, which occurs during the major street WALK or green phase:

$$Q_{tco} = [v_{co} \times (R_{mj}/C) \times (R_{mj}/2)]/60 \qquad (13\text{-}6)$$

For Condition 2, the major street crossing, which occurs during the minor street WALK or green phase:

$$Q_{tdo} = [v_{do} \times (R_{mi}/C) \times (R_{mi}/2)]/60 \qquad (13\text{-}7)$$

where:

Q_{tdo} = total time spent by pedestrians waiting to cross the minor street during one signal cycle, in ped-min;
Q_{tco} = total time spent by pedestrians waiting to cross the major street during one signal cycle, in ped-min;
v_{do} = the number of pedestrians per cycle crossing the minor street, in ped/cycle;
v_{co} = the number of pedestrians per cycle crossing the major street;
R_{mi} = the minor street red phase, or the DON'T WALK phase where pedestrian signals exist, in sec;

R_{mj} = the major street red phase, or the DON'T WALK phase where pedestrian signals exist, in sec; and
C = cycle length, in sec.

The term R/C is used to estimate the number of pedestrians per cycle that must wait for the green indication. The number is esimated as $v \times R/C$. Assuming that arrivals are uniformly distributed, each pedestrian that waits, does so for an average duration of R/2 sec. The division by 60 converts time from seconds to minutes.

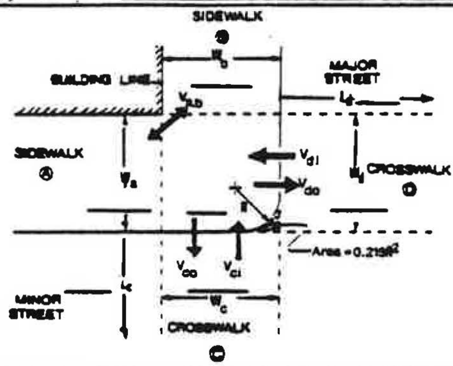
For the sample calculations, the following values are computed:

$$Q_{tco} = [27 \times 0.40 \times 32/2]/60 = 2.9 \text{ ped min}$$
$$Q_{tdo} = [21 \times 0.60 \times 48/2]/60 = 5.0 \text{ ped-min}$$

CROSSWALK ANALYSIS WORKSHEET

Location ____________

City, State ____________



SIGNAL TIMING
(secs)

C = ____
G_{mj} = ____; R_{mj} = ____
G_{mi} = ____; R_{mi} = ____

PED. VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}		
v_{co}		
v_{di}		
v_{do}		
$v_{a,b}$		
v_{tot}		

CROSSWALK AREAS

$A_c = L_cW_c$ = ________ sq ft

$A_d = L_dW_d$ = ________ sq ft

CROSSWALK TIME-SPACE

$TS_c = A_c(G_{mj}-3)/60$ = ________ sq ft-min

$TS_d = A_d(G_{mi}-3)/60$ = ________ sq ft-min

CROSSING TIMES

$t_{wc} = L_c/4.5$ = ________ sec

$t_{wd} = L_d/4.5$ = ________ sec

CROSSWALK OCCUPANCY TIME
(use ped/cycle)

$T_{wc} = (v_{ci}+v_{co})(t_{wc}/60)$ = ________ ped-min

$T_{wd} = (v_{di}+v_{do})(t_{wd}/60)$ = ________ ped-min

AVERAGE PEDESTRIAN SPACE AND LOS

$M_c = TS_c/T_{wc}$ = ______ sq ft/ped; LOS = ______ (Tab.13-3)

$M_d = TS_d/T_{wd}$ = ______ sq ft/ped; LOS = ______ (Tab.13-3)

MAXIMUM SURGE
(use ped/min)

$V_{mc} = (v_{ci}+v_{co})(R_{mj}+3+t_{wc})/60$ = ________ peds

$V_{md} = (v_{di}+v_{do})(R_{mi}+3+t_{wd})/60$ = ________ peds

SURGE PEDESTRIAN SPACE AND SURGE LOS

$M_c(MAX) = A_c/V_{mc}$ = ______ sq ft/ped; LOS = ______ (Tab.13-3)

$M_d(MAX) = A_d/V_{md}$ = ______ sq ft/ped; LOS = ______ (Tab.13-3)

Figure 13-18. Worksheet for crosswalk analysis.

Step 3—Determine Holding Area Time-Space Requirements

The holding area needs of waiting pedestrians is the product of the total waiting times determined in Step 2 (Q_{tdo} and Q_{tco}), and the average area used by a waiting pedestrian, which is taken to be 5 sq ft/ped for a competitive queue. Then:

$$TS_h = 5\,(Q_{tdo} + Q_{tco}) \qquad (13\text{-}7)$$

where TS_h equals the total time-space holding area requirements for the intersection, in sq ft-min.

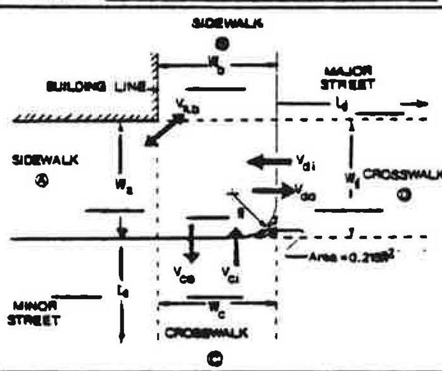
For the sample calculation, the following value is determined:

$$TS_h = 5\,(5.0 + 2.9) = 39.5, \text{ SAY } 40 \text{ sq ft-min}$$

STREET CORNER ANALYSIS WORKSHEET

Location ____________

City, State ____________



SIGNAL TIMING
(secs)

C = ______

G_{mj} = ______; R_{mj} = ______

G_{mi} = ______; R_{mi} = ______

PED. VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}		
v_{co}		
v_{di}		
v_{do}		
$v_{a,b}$		
v_{tot}		

NET CORNER AREA $A = W_aW_b - 0.215R^2$ = ______ sq ft

AVAILABLE TIME-SPACE $TS = A \times C/60$ = ______ sq ft-min

HOLD AREA WAITING TIMES
(use ped/cycle)

$Q_{tco} = [(v_{co})(R_{mj}/C)(R_{mj}/2)]/60$ = ______ ped-min

$Q_{tdo} = [(v_{do})(R_{mi}/C)(R_{mi}/2)]/60$ = ______ ped-min

HOLD AREA TIME-SPACE

$TS_h = 5\,(Q_{tco}+Q_{tdo})$ = ______ sq ft-min

CIRCULATION TIME-SPACE

$TS_c = TS - TS_h$ = ______ sq ft-min

TOTAL CIRCULATION VOLUME

$v_c = v_{ci}+v_{co}+v_{do}+v_{di}+v_{a,b}$ = ______ peds

TOTAL CIRCULATION TIME

$t_c = v_c \times 4/60$ = ______ ped-min

PEDESTRIAN SPACE AND LOS

$M = TS_c/t_c$ = ______ sq ft/ped; LOS = ______ (Tab.13-3)

Figure 13-19. Worksheet for street corner analysis.

Step 4—Determine the Net Corner Time-Space Available for Circulation

The total time-space available for circulation is the total intersection time-space minus that used for holding waiting pedestrians; or:

$$TS_c = TS - TS_h \tag{13-8}$$

where TS_c equals the total time-space available for circulating pedestrians, in sq ft-min.

For the sample calculation:

$$TS_c = 227 - 40 = 187 \text{ sq ft-min}$$

Step 5—Determine the Total Number of Circulating Pedestrians Per Cycle

The number of pedestrians which must use the available circulation time-space during each cycle is the sum of all pedestrian flows, each flow is expressed in units of ped/cycle:

$$v_c = v_{ci} + v_{co} + v_{di} + v_{do} + v_{a,b} \tag{13-9}$$

where v_c equals total number of circulating pedestrians, in ped/cycle.

For the sample calculation:

$$v_c = 48 + 27 + 40 + 21 + 20 = 156 \text{ ped}$$

Step 6—Determine the Total Circulation Time Utilized by Circulating Pedestrians

The time that pedestrians consume while walking through the corner area is taken as the product of the total circulation volume and an assumed average circulation time of 4 sec, or:

$$t_c = v_c \times 4/60 \tag{13-10}$$

where t_c equals the total circulation time, in ped-min.

For the sample calculation:

$$t_c = 156 \times 4/60 = 10.4 \text{ ped-min}$$

Step 7—Determine the Circulation Area Per Pedestrian

The circulation area per pedestrian is referred to as the "pedestrian area module," and given the symbol, *M*. It is computed as the net time-space available for circulation, TS_c, divided by the total circulation time, t_c:

$$M = TS_c/t_c \tag{13-11}$$

For the sample calculation:

$$M = 187/10.4 = 18.0 \text{ sq ft/ped}$$

Step 8—Determine the Corner Level of Service

The corner LOS is found by comparing the pedestrian area module, *M*, to the criteria found in Table 13-3. Values below LOS C indicate a potential problem that should be the subject of further field study and possible remedial actions, which could include changes in the signal timing, prohibition of vehicle-turning movements, sidewalk widening, and removal of sidewalk obstructions.

From Table 13-3, for a pedestrian area module of 18.0 sq ft/ped, the LOS for the sample calculation is found to be D. The need for further field study and possible remedial action is indicated.

Figure 13-20 illustrates the solution of the sample calculation on the street corner worksheet.

Crosswalk Analysis (Computational Steps and Sample Calculation)

Analysis procedures for crosswalks use the same basic principles of accounting for time-space. The procedure is explained in the following steps.

Step 1—Determine the Total Available Time-Space

The total time-space available in the crosswalk during one signal cycle is the product of the crosswalk area and the WALK interval for the crosswalk. Where pedestrian signals are not present, the green time −3 sec is substituted for WALK time. Note that in computing crosswalk area, the effect of the corner radius is not considered. Then:

$$A_w = W \times L \tag{13–12}$$

$$TS_w = A_w \times G_w/60 \tag{13–13}$$

where:

A_w = area of the crosswalk, in sq ft;
W = width of the crosswalk, in ft;
L = length of the crosswalk, in ft;
TS_w = Total time-space available in the crosswalk during one signal cycle, in sq ft-min; and
G_w = WALK interval, in sec.

Then, for Crosswalk C in the illustrative calculation:

$$A = 16 \times 28 = 448 \text{ sq ft}$$

$$TS_w = 448 \times (48 - 3)/60 = 336 \text{ sq ft-min}$$

and for Crosswalk D:

$$A = 16 \times 46 = 736 \text{ sq ft}$$

$$TS_w = 736 \times (32 - 3)/60 = 356 \text{ sq ft-min}$$

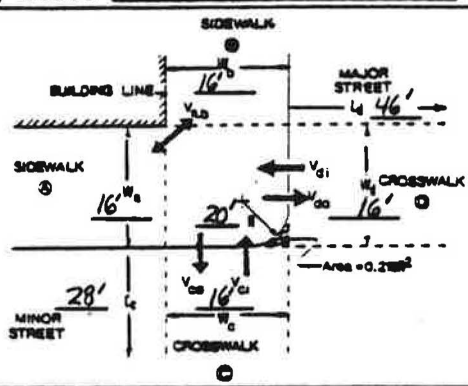
Step 2—Determine the Average Crossing Times

The average time a pedestrian occupies each crosswalk is

STREET CORNER ANALYSIS WORKSHEET

Location RALPH AVE + CROSSWAY BLVD

City, State TOWNSVILLE KY



SIGNAL TIMING (secs)

C = 80

G_{mj} = 48 ; R_{mj} = 32

G_{mi} = 32 ; R_{mi} = 48

PED. VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}	36	48
v_{co}	20	27
v_{di}	30	40
v_{do}	16	21
$v_{a,b}$	15	20
v_{tot}	117	156

NET CORNER AREA $A = W_a W_b - 0.215R^2$ = 170 sq ft

AVAILABLE TIME-SPACE $TS = A \times C/60$ = 227 sq ft-min

HOLD AREA WAITING TIMES
(use ped/cycle)

$Q_{tco} = [(v_{co})(R_{mj}/C)(R_{mj}/2)]/60$ = 2.9 ped-min

$Q_{tdo} = [(v_{do})(R_{mi}/C)(R_{mi}/2)]/60$ = 5.0 ped-min

HOLD AREA TIME-SPACE

$TS_h = 5\ (Q_{tco}+Q_{tdo})$ = 39.5 ~ 40 sq ft-min

CIRCULATION TIME-SPACE

$TS_c = TS - TS_h$ = 187 sq ft-min

TOTAL CIRCULATION VOLUME

$v_c = v_{ci}+v_{co}+v_{do}+v_{di}+v_{a,b}$ = 156 peds

TOTAL CIRCULATION TIME

$t_c = v_c \times 4/60$ = 10.4 ped-min

PEDESTRIAN SPACE AND LOS

$M = TS_c/t_c$ = 18.0 sq ft/ped; LOS = D (Tab. 13-3)

Figure 13-20. Worksheet for street corner analysis of sample calculation.

obtained by dividing the length of the crosswalk (street width) by the assumed walking speed. Average walking speed in crosswalks is taken to be 4.5 ft/sec. Then:

$$t_w = L/4.5 \qquad (13\text{-}14)$$

where:

t_w = average time spent by pedestrian in the crosswalk, in sec; and

L = length of the crosswalk, in ft.

Then, for the sample calculation Crosswalk C:

$$t_w = 28/4.5 = 6.2 \text{ sec}$$

and for Crosswalk D:

$$t_w = 46/4.5 = 10.2 \text{ sec}$$

Step 3—Determine the Total Crosswalk Occupancy Time

The total crosswalk occupancy time is the product of the average crossing time and the number of pedestrians using the crosswalk during one signal cycle. Then:

$$T_w = (v_i + v_o)\ t_w/60 \qquad (13\text{-}15)$$

where:

T_w = total crosswalk occupancy time, in ped-min;

v_i = incoming pedestrian volume for the subject crosswalk, in ped/cycle; and

v_o = outgoing pedestrian volume for the subject crosswalk, in ped/cycle.

For the sample calculation, Crosswalk C:

$$T_w = (48 + 27)\ 6.2/60 = 7.8 \text{ ped-min}$$

and for Crosswalk D:

$$T_w = (40 + 21)\ 10.2/60 = 10.4 \text{ ped-min}$$

Step 4—Determine the Average Circulation Space per Pedestrian and the Average Level of Service

The average circulation space provided for each pedestrian is determined by dividing the time-space available for crossing by the total occupancy time. This yields the average area module provided for each pedestrian, which is related to level of service by the criteria of Table 13-3.

$$M = TS_w/T_w \qquad (13\text{-}16)$$

For Crosswalk C:

$$M = 336/7.8 = 43 \text{ sq ft/ped (LOS B, Table 13-3)}$$

and for Crosswalk D:

$$M = 356/10.4 = 34 \text{ sq ft/ped (LOS C, Table 13-3)}$$

Step 5—Determine the Level of Service for the Maximum Surge Condition

Step 4 yields an analysis of conditions that are average for the WALK interval. The point at which the maximum number of pedestrians are in the crosswalk should also be examined. This occurs when the lead pedestrians in opposing crossing platoons reach the opposite corner. The area module for the surge condition is the area of the crosswalk divided by the maximum number of pedestrians in the crosswalk. Crosswalk flows in pedestrians per minute (NOT the ped/cycle units which have been used for other analysis steps) are multiplied by the DON'T WALK interval + the crossing time, t_w. The DON'T WALK interval is used to estimate the number of pedestrians queued when the WALK interval is given, and the crossing time is added to estimate the number of new arriving pedestrians during the period that the queued pedestrians cross the street. Where pedestrian signals are not present, the red interval + 3 sec is substituted for the DON'T WALK interval. Then:

$$V_m = (v_i + v_o)\ (R_w + t_w)/60 \qquad (13\text{-}17)$$

$$M = A/V_m \qquad (13\text{-}18)$$

where:

V_m = maximum number of pedestrians occupying crosswalk;

v_i = incoming crosswalk volume, in ped/min;

v_o = outgoing crosswalk volume, in ped/min; and

R_w = DON'T WALK interval, in sec.

For the sample calculation, Crosswalk C:

$$V_m = (36 + 20) \times (32 + 3 + 6.2)/60 = 38.5 \text{ ped}$$

$$M = 448/38.5 = 11.6 \text{ sq ft/ped (LOS E, Table 13-3)}$$

and for Crosswalk D:

$$V_m = (30 + 16) \times (48 + 3 + 10.2)/60 = 46.9, \text{ SAY 47 ped}$$

$$M = 736/47 = 15.7 \text{ sq ft/ped (LOS D, Table 13-3)}$$

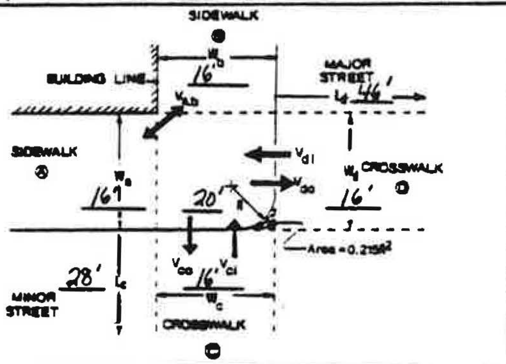
Note that the surge LOS is worse than the average LOS, particularly for Crosswalk C, where the value fell from B for average conditions to E for surge conditions. This emphasizes the need to consider both conditions.

Figure 13-21 shows the worksheet for the sample calculation discussed herein.

CROSSWALK ANALYSIS WORKSHEET

Location: RALPH AVE & CROSSWAY BLVD

City, State: TOWNSVILLE KY



SIGNAL TIMING (secs)

C = 80

G_{mj} = 48; R_{mj} = 32

G_{mi} = 32; R_{mi} = 48

PED. VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}	36	48
v_{co}	20	27
v_{di}	30	40
v_{do}	16	21
$v_{a,b}$	15	20
v_{tot}	117	156

CROSSWALK AREAS

$A_c = L_cW_c$ = 448 sq ft

$A_d = L_dW_d$ = 736 sq ft

CROSSWALK TIME-SPACE

$TS_c = A_c(G_{mj}-3)/60$ = 336 sq ft-min

$TS_d = A_d(G_{mi}-3)/60$ = 356 sq ft-min

CROSSING TIMES

$t_{wc} = L_c/4.5$ = 6.2 sec

$t_{wd} = L_d/4.5$ = 10.2 sec

CROSSWALK OCCUPANCY TIME
(use ped/cycle)

$T_{wc} = (v_{ci}+v_{co})(t_{wc}/60)$ = 7.8 ped-min

$T_{wd} = (v_{di}+v_{do})(t_{wd}/60)$ = 10.4 ped-min

AVERAGE PEDESTRIAN SPACE AND LOS

$M_c = TS_c/T_{wc}$ = 43 sq ft/ped; LOS = B (Tab.13-3)

$M_d = TS_d/T_{wd}$ = 34 sq ft/ped; LOS = C (Tab.13-3)

MAXIMUM SURGE
(use ped/min)

$V_{mc} = (v_{ci}+v_{co})(R_{mj}+3+t_{wc})/60$ = 38.5 peds

$V_{md} = (v_{di}+v_{do})(R_{mi}+3+t_{wd})/60$ = 46.9 peds

SURGE PEDESTRIAN SPACE AND SURGE LOS

$M_c(MAX) = A_c/V_{mc}$ = 11.6 sq ft/ped; LOS = E (Tab.13-3)

$M_d(MAX) = A_d/V_{md}$ = 15.7 sq ft/ped; LOS = D (Tab.13-3)

Figure 13-21. Worksheet for crosswalk analysis of sample calculation.

Estimating the Decrement to Crosswalk LOS Due to Right-Turning Vehicles

The time-space method allows for an approximate estimate to be made of the effect of turning vehicles on the average LOS for pedestrians crossing during a given green phase. This is done by assuming an average area occupancy of a vehicle in the crosswalk, based on the product of vehicle swept-path and crosswalk widths, and an estimate of the time that the vehicle preempts this space. The swept-path for most vehicles may be estimated at an average of 8 ft, and it is assumed that a vehicle occupies the crosswalk for a period of 5 sec.

For the sample calculation, each turning vehicle will preempt:

[8 ft $\times$ 16 ft (crosswalk width) $\times$ 5 sec]/60 = 10.7 sq ft-min/veh

If 5 vehicles turn during an average green phase, the total decrement to available time-space would be: 10.7 $\times$ 5 = 54 sq ft/min.

For the major street crossing (Crosswalk D), the total available time-space was computed to be 356 sq ft-min. Deducting 54 sq ft-min, only 302 sq ft-min remain for pedestrian use. The pedestrian space module is now recomputed using this figure in Eq. 13-16:

M = 302/10.4 = 29 sq ft/ped (LOS C, Table 13-3)

In this case, the decrement has not caused a reduction in the LOS, although the area per pedestrian is clearly reduced. This is an indication that the crosswalk can handle both the pedestrian demands and the turning vehicle demands without experiencing a capacity or delay problem. Where the decrement causes a significant decline in LOS, particularly where LOS F would result, further field studies and remedial action should be pursued.

IV. REFERENCES

The basic pedestrian characteristics used in this chapter were presented in Transportation Research Board Circular 212 (*1*). Pioneering references of great interest were authored by Pushkarev and Zupan (*2*) and Fruin (*3*). References *4* through *8* offer additional information for interested users of this manual.

1. "Interim Materials on Highway Capacity." *Transportation Research Board Circular 212,* Transportation Research Board, Washington, D.C. (1980).
2. Pushkarev, B., and Zupan, J., *Urban Space for Pedestrians.* MIT Press, Cambridge, Mass. (1975).
3. Fruin, J., *Pedestrian Planning and Design.* Metropolitan Association of Urban Designers and Environmental Planners, New York, N.Y. (1971).
4. *Feasibility Analysis and Design Concepts and Criteria for Communitywide Separated Pedestrian Networks.* Phase III, Draft Pedestrian Planning Procedures Manual, Vols. I–III, RTKL Associates, Baltimore, Md. (1977).
5. Hall, D., *The Hidden Dimension.* Doubleday and Co., New York, N.Y. (1966).
6. Virkler, M., and Guell, D., "Pedestrian Crossing Time Requirements and Intersections." 61st Annual Meeting of the Transportation Research Board, Washington, D.C. (Jan. 1982).
7. Fruin, J, and Benz, G., "Pedestrian Time-Space Concept for Analyzing Corners and Crosswalks." 63rd Annual Meeting of the Transportation Research Board, Washington, D.C. (Jan. 1984).
8. Kagan, L., et al., *"A Pedestrian Planning Procedures Manual."* Vols. I–II, *FHWA Report Nos. RD-78-45, RD-78-46, RD-79-47,* Federal Highway Administration, Washington, D.C. (Nov. 1978).

APPENDIX I

TABLES, FIGURES, AND WORKSHEETS FOR USE IN ANALYSIS OF WALKWAYS, CROSSWALKS, AND STREET CORNERS

TABLES *PAGE*

FIGURES

WORKSHEETS

TABLE 13-2. FIXED OBSTACLE WIDTH ADJUSTMENT FACTORS FOR WALKWAYS*

OBSTACLE	APPROX. WIDTH PREEMPTED (FT)ᵃ
STREET FURNITURE	
Light Poles	2.5–3.5
Traffic Signal Poles and Boxes	3.0–4.0
Fire Alarm Boxes	2.5–3.5
Fire Hydrants	2.5–3.0
Traffic Signs	2.0–2.5
Parking Meters	2.0
Mail Boxes (1.7 ft by 1.7 ft)	3.2–3.7
Telephone Booths (2.7 ft by 2.7 ft)	4.0
Waste Baskets	3.0
Benches	5.0
PUBLIC UNDERGROUND ACCESS	
Subway Stairs	5.5–7.0
Subway Ventilation Gratings (raised)	6.0+
Transformer Vault Ventilation Gratings (raised)	5.0+
LANDSCAPING	
Trees	2.0–4.0
Planting Boxes	5.0
COMMERCIAL USES	
Newsstands	4.0–13.0
Vending Stands	variable
Advertising Displays	variable
Store Displays	variable
Sidewalk Cafes (two rows of tables)	variable, try 7.0
BUILDING PROTRUSIONS	
Columns	2.5–3.0
Stoops	2.0–6.0
Cellar Doors	5.0–7.0
Standpipe Connections	1.0
Awning Poles	2.5
Truck Docks (trucks protruding)	variable
Garage Entrance/Exit	variable
Driveways	variable

* To account for the avoidance distance normally occurring between pedestrians and obstacles, an additional 1.0 to 1.5 ft must be added to the preemption width for individual obstacles.
ᵃ Curb to edge of object, or building face to edge of object.
SOURCE: Ref. *2*

TABLE 13-3. PEDESTRIAN LEVEL OF SERVICE ON WALKWAYS*

LEVEL OF SERVICE	SPACE (SQ FT/PED)	EXPECTED FLOWS AND SPEEDS: AVE. SPEED, S (FT/MIN)	FLOW RATE, v (PED/MIN/FT)	VOL/CAP RATIO, v/c
A	≥ 130	≥ 260	≤ 2	≤ 0.08
B	≥ 40	≥ 250	≤ 7	≤ 0.28
C	≥ 24	≥ 240	≤ 10	≤ 0.40
D	≥ 15	≥ 225	≤ 15	≤ 0.60
E	≥ 6	≥ 150	≤ 25	≤ 1.00
F	< 6	< 150	----Variable----	

* Average conditions for 15 min.

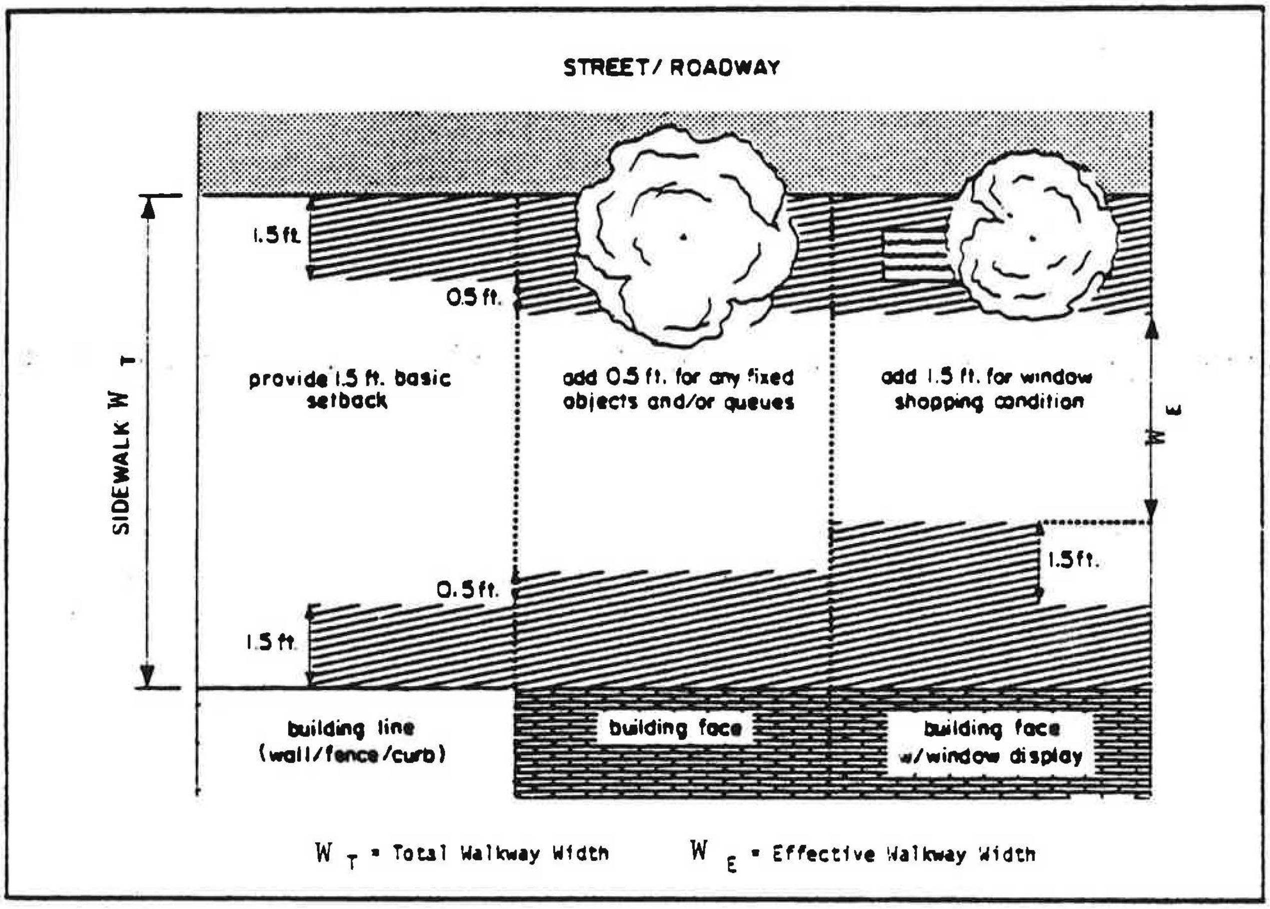
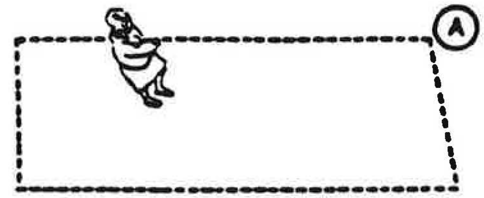


Figure 13-5. Preemption of walkway width. (Source: Ref. *4)*

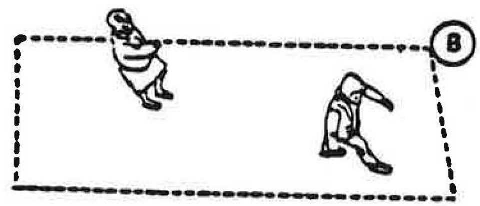
LEVEL OF SERVICE A



Pedestrian Space: $\geq$ 130 sq ft/ped **Flow Rate:** $\leq$ 2 ped/min/ft

At walkway LOS A, pedestrians basically move in desired paths without altering their movements in response to other pedestrians. Walking speeds are freely selected, and conflicts between pedestrians are unlikely.

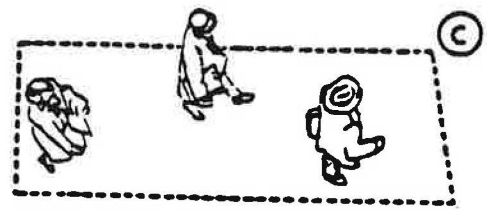
LEVEL OF SERVICE B



Pedestrian Space: $\geq$ 40 sq ft/ped **Flow Rate:** $\leq$ 7 ped/min/ft

At LOS B, sufficient area is provided to allow pedestrians to freely select walking speeds, to bypass other pedestrians, and to avoid crossing conflicts with others. At this level, pedestrians begin to be aware of other pedestrians, and to respond to their presence in the selection of walking path.

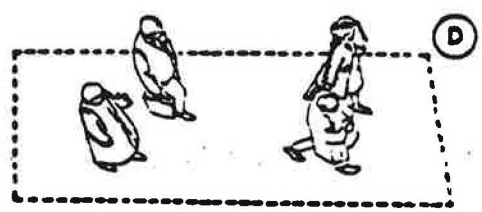
LEVEL OF SERVICE C



Pedestrian Space: $\geq$ 24 sq ft/ped **Flow Rate:** $\leq$ 10 ped/min/ft

At LOS C, sufficient space is available to select normal walking speeds, and to bypass other pedestrians in primarily unidirectional streams. Where reverse-direction or crossing movements exist, minor conflicts will occur, and speeds and volume will be somewhat lower.

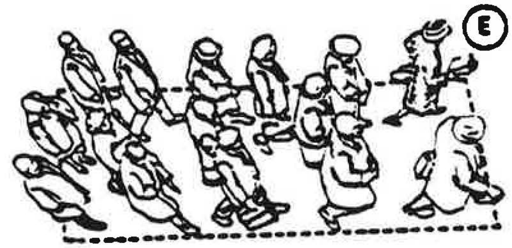
LEVEL OF SERVICE D



Pedestrian Space: $\geq$ 15 sq ft/ped **Flow Rate:** $\leq$ 15 ped/min/ft

At LOS D, freedom to select individual walking speed at to bypass other pedestrians is restricted. Where crossing or reverse-flow movements exist, the probability of conflict is high, and its avoidance requires frequent changes in speed and position. The LOS provides reasonably fluid flow, however, considerable friction and interaction between pedestrians is likely to occur.

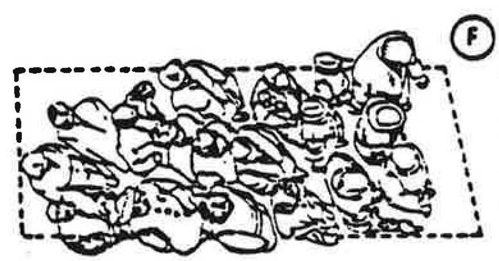
LEVEL OF SERVICE E



Pedestrian Space: $\geq$ 6 sq ft/ped **Flow Rate:** $\leq$ 25 ped/min/ft

At LOS E, virtually all pedestrians would have their normal walking speed restricted, requiring frequent adjustment of gait. At the lower range of this LOS, forward movement is possible only by "shuffling." Insufficient space is provided for passing of slower pedestrians. Cross- or reverse-flow movements are possible only with extreme difficulties. Design volumes approach the limit of walkway capacity, with resulting stoppages and interruptions to flow.

LEVEL OF SERVICE F

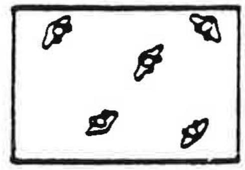


Pedestrian Space: $\leq$ 6 sq ft/ped **Flow Rate:** variable

At LOS F, all walking speeds are severely restricted, and forward progress is made only by "shuffling." There is frequent, unavoidable, contact with other pedestrians. Cross- and reverse-flow movements are virtually impossible. Flow is sporadic and unstable. Space is more characteristic of queued pedestrians than of moving pedestrian streams.

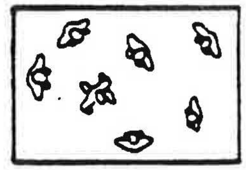
Figure 13-8. Illustration of walkway levels of service.

LEVEL OF SERVICE A



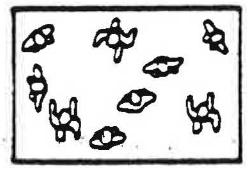
Average Pedestrian Area Occupancy: 13 sq. ft./person or more
Average Inter-person Spacing: 4 ft., or more
Description: standing and free circulation through the queuing area is possible without disturbing others within the queue.

LEVEL OF SERVICE B



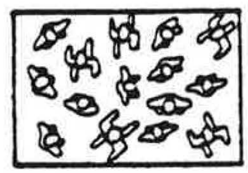
Average Pedestrian Area Occupancy: 10-13 sq. ft./person
Average Inter-person Spacing: 3.5-4.0 ft.
Description: standing and partially restricted circulation to avoid disturbing others within the queue is possible.

LEVEL OF SERVICE C



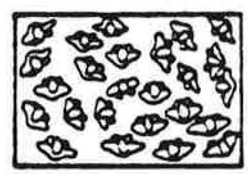
Average Pedestrian Area Occupancy: 7-10 sq. ft./person
Average Inter-person Spacing: 3.0-3.5 ft.
Description: standing and restricted circulation through the queuing area by disturbing others within the queue is possible, this density is within the range of personal comfort.

LEVEL OF SERVICE D



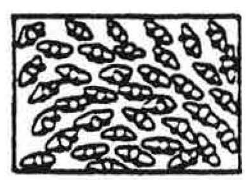
Average Pedestrian Area Occupancy: 3-7 sq. ft./person
Average Inter-person Spacing: 2-3 ft.
Description: standing without touching is possible; circulation is severely restricted within the queue and forward movement is only possible as a group; long term waiting at this density is discomforting.

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.

Figure 13-11. Levels of service for queuing areas. (Source: Ref. *3)*

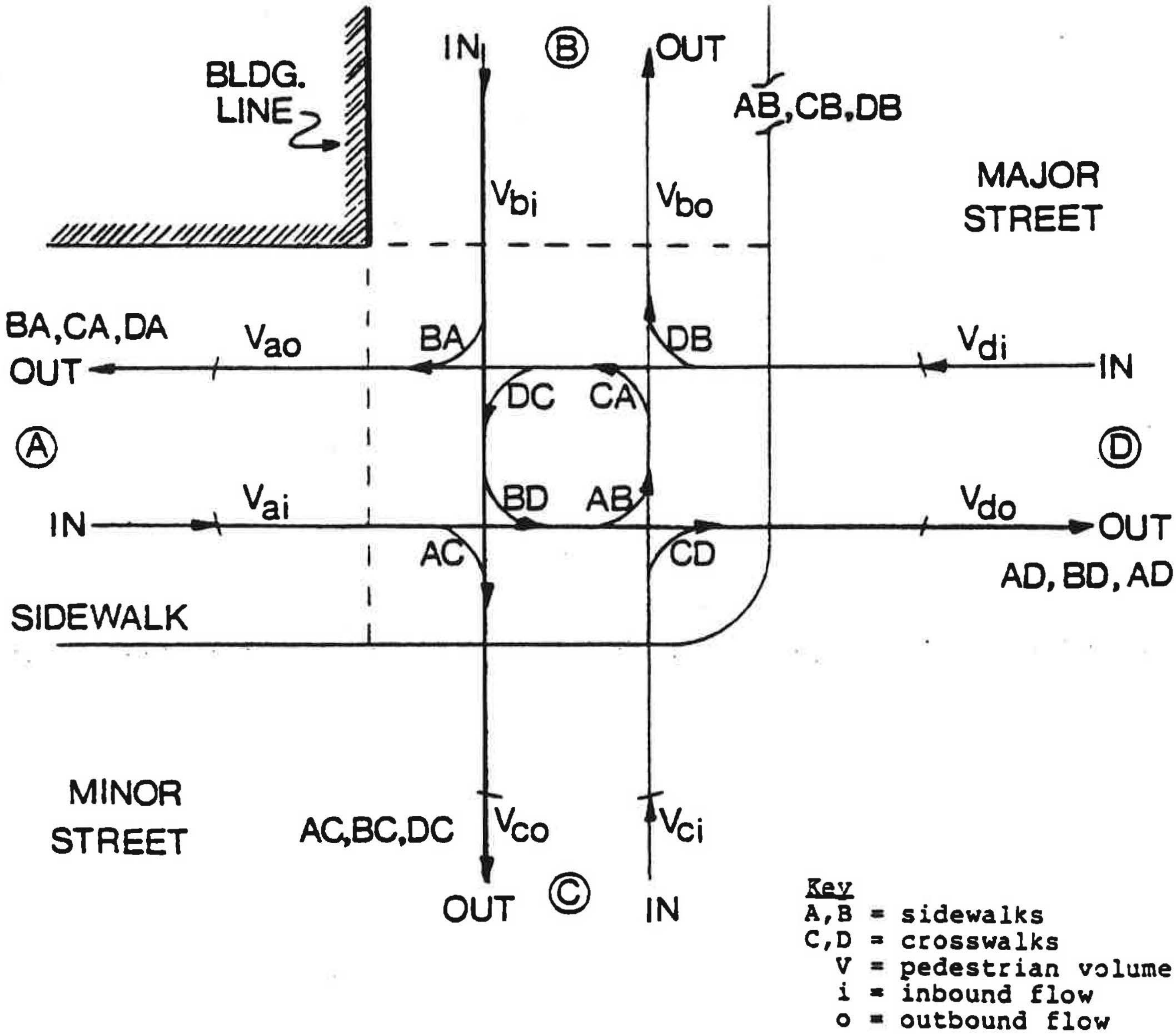
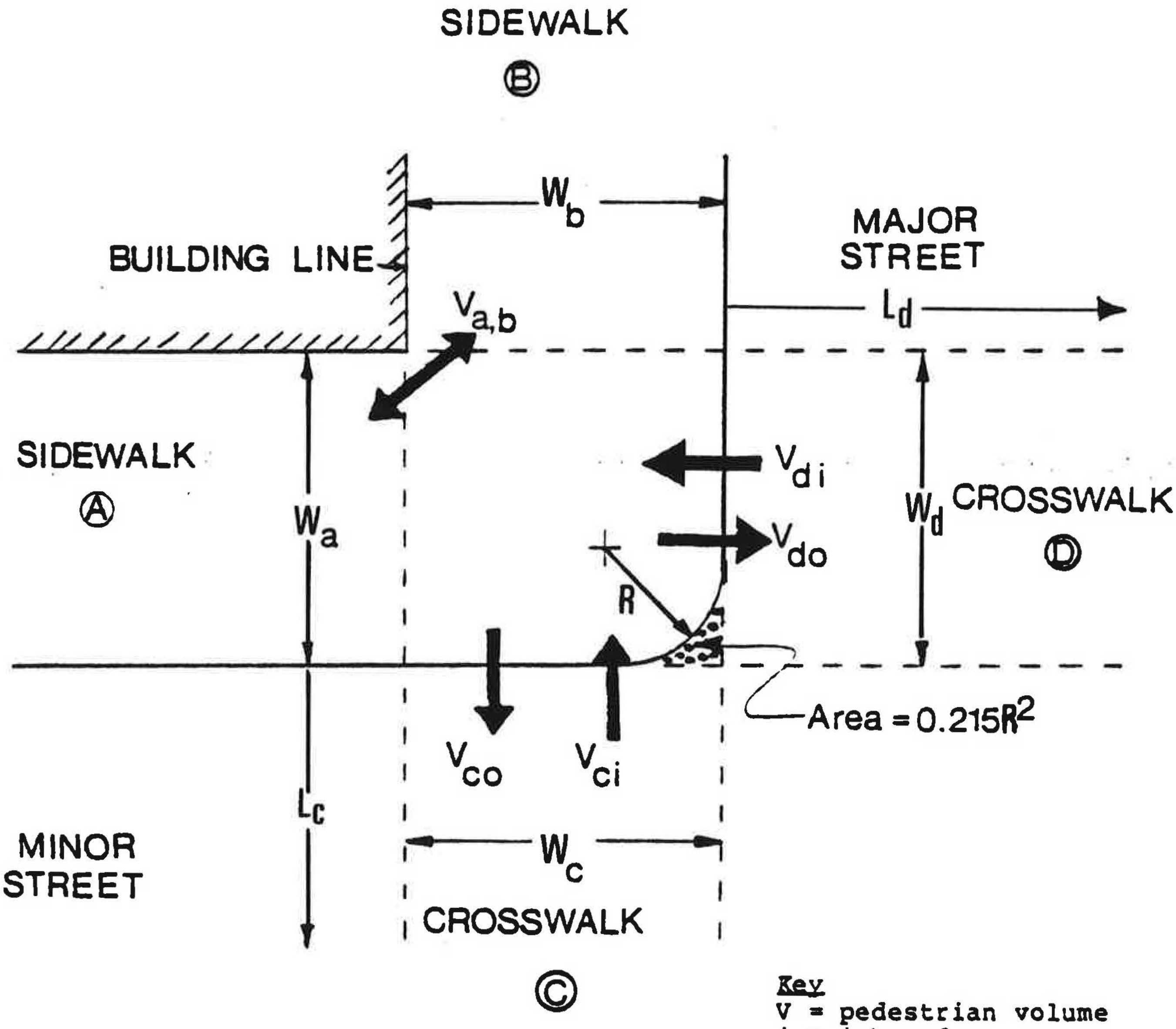


Figure 13-12. Pedestrian movements at a street corner.



Key
V = pedestrian volume
i = inbound
o = outbound
W = width
R = corner radius

Figure 13-15. Intersection corner geometrics and pedestrian movements.

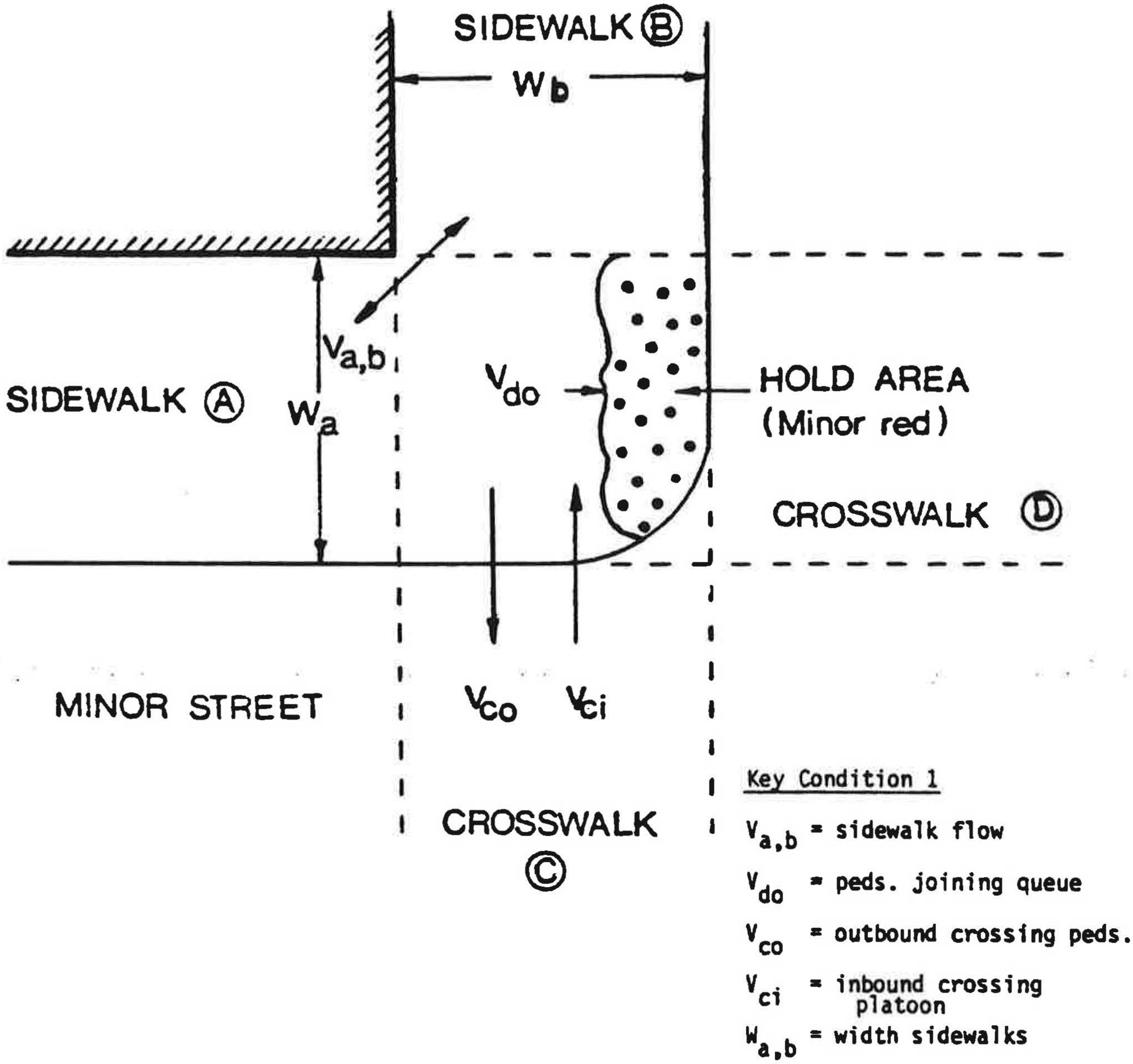


Figure 13-16. Intersection corner Condition 1—minor street crossing.

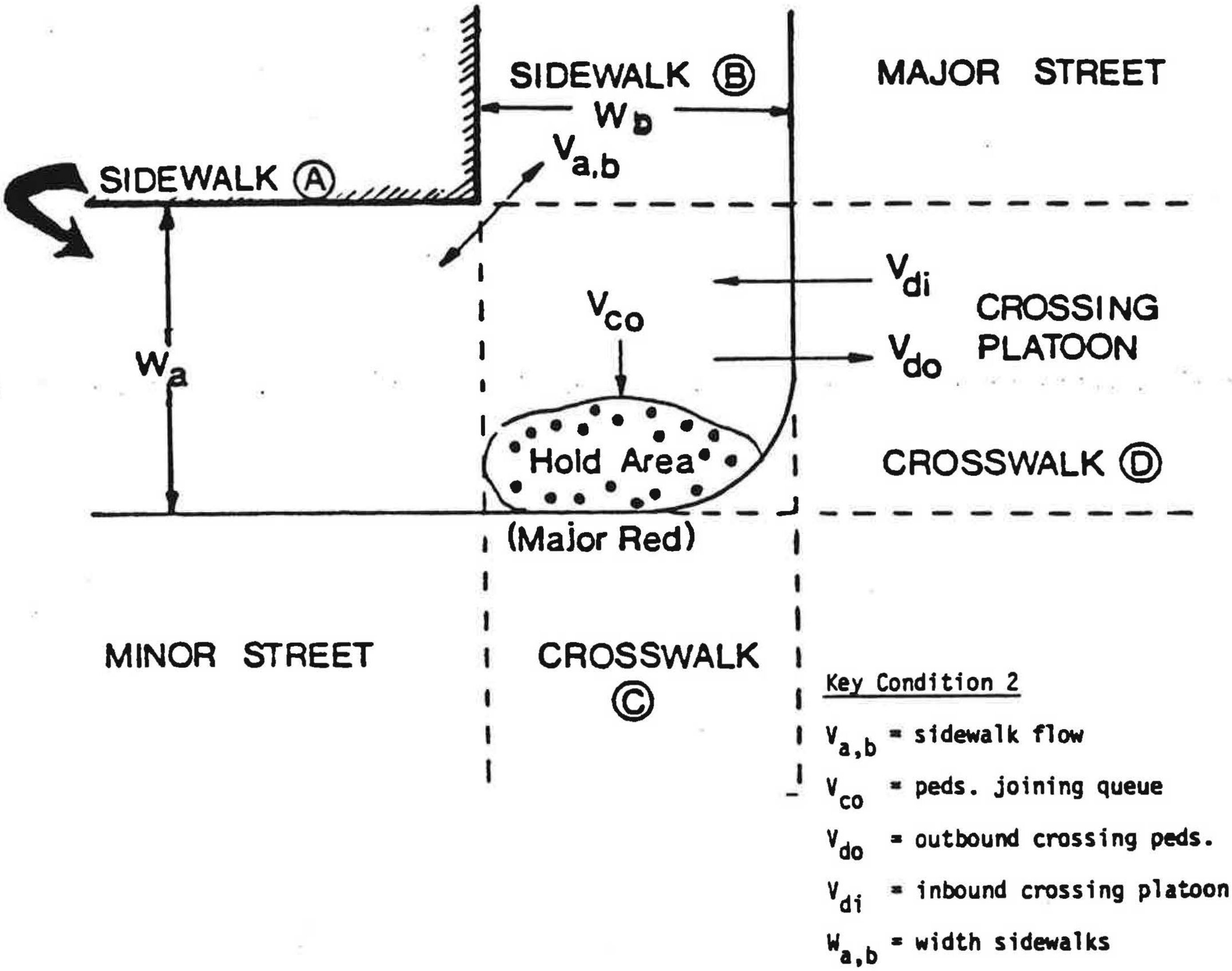


Figure 13-17. Intersection corner Condition 2—major street crossing.

WALKWAY ANALYSIS WORKSHEET

Location ______________________

City, State ______________________

COUNTS

Date __________

Time __________

PK 15-MIN FROM _____ to _____

Curb Line/Sidewalk Edge

W_{B1}(curb) = ft

W_{B2}(street furn.) = ft

W_T = ___ W_E(effective width) = ft

W_{B3}(window shop.) = ft

W_{B4}(bldg protrusions) = ft

W_{B5}(inside clearance) = ft

Wall line/Sidewalk Edge

$\leftarrow V_1$ = _____

$\rightarrow V_2$ = _____

(ped/15 min)

Pedestrian Volume

V_1 = __________ ped/15 min

V_2 = __________ ped/15 min

$V_p = V_1+V_2$ = __________ ped/15 min

Walkway Width

W_T = __________ ft

$W_B = W_{B1}+W_{B2}+W_{B3}+W_{B4}+W_{B5}$ = __________ ft

$W_E = W_T-W_B$ = __________ ft

Average Walkway LOS

$v = V_p/15W_E$ = __________ ped/min/ft

Average LOS = __________ (Table 13-3)

Platoon Walkway LOS

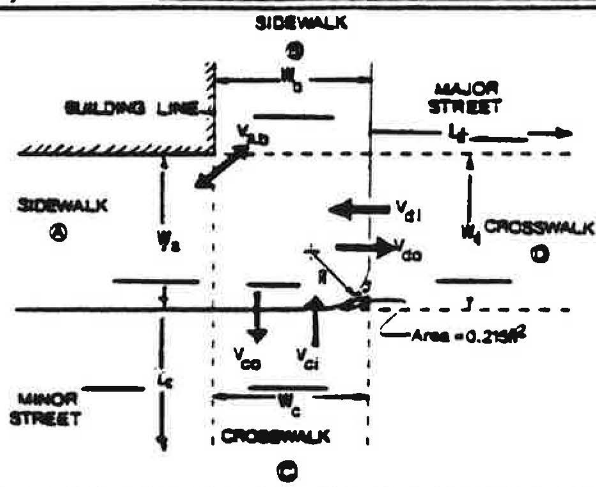
$v_p = v + 4$ = __________ ped/min/ft

Platoon LOS = __________ (Table 13-3)

CROSSWALK ANALYSIS WORKSHEET

Location ________________________

City, State ________________________



SIGNAL TIMING
(secs)

C = ______

G_{mj} = ______; R_{mj} = ______

G_{mi} = ______; R_{mi} = ______

PED. VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}		
v_{co}		
v_{di}		
v_{do}		
$v_{a,b}$		
v_{tot}		

CROSSWALK AREAS

$A_c = L_cW_c$ = ________ sq ft

$A_d = L_dW_d$ = ________ sq ft

CROSSWALK TIME-SPACE

$TS_c = A_c(G_{mj}-3)/60$ = ________ sq ft-min

$TS_d = A_d(G_{mi}-3)/60$ = ________ sq ft-min

CROSSING TIMES

$t_{wc} = L_c/4.5$ = ________ sec

$t_{wd} = L_d/4.5$ = ________ sec

CROSSWALK OCCUPANCY TIME
(use ped/cycle)

$T_{wc} = (v_{ci}+v_{co})(t_{wc}/60)$ = ________ ped-min

$T_{wd} = (v_{di}+v_{do})(t_{wd}/60)$ = ________ ped-min

AVERAGE PEDESTRIAN SPACE AND LOS

$M_c = TS_c/T_{wc}$ = ________ sq ft/ped; LOS = ________ (Tab.13-3)

$M_d = TS_d/T_{wd}$ = ________ sq ft/ped; LOS = ________ (Tab.13-3)

MAXIMUM SURGE
(use ped/min)

$V_{mc} = (v_{ci}+v_{co})(R_{mj}+3+t_{wc})/60$ = ________ peds

$V_{md} = (v_{di}+v_{do})(R_{mi}+3+t_{wd})/60$ = ________ peds

SURGE PEDESTRIAN SPACE AND SURGE LOS

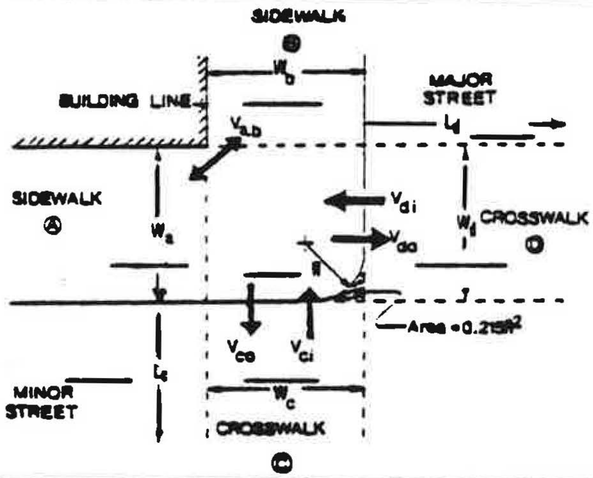
$M_c(MAX) = A_c/V_{mc}$ = ________ sq ft/ped; LOS = ________ (Tab.13-3)

$M_d(MAX) = A_d/V_{md}$ = ________ sq ft/ped; LOS = ________ (Tab.13-3)

STREET CORNER ANALYSIS WORKSHEET

Location ____________________

City, State ____________________



SIGNAL TIMING
(secs)

C = ______
G_{mj} = ______; R_{mj} = ______
G_{mi} = ______; R_{mi} = ______

PED. VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}		
v_{co}		
v_{di}		
v_{do}		
$v_{a,b}$		
v_{tot}		

NET CORNER AREA $A = W_a W_b - 0.215R^2$ = __________ sq ft

AVAILABLE TIME-SPACE $TS = A \times C/60$ = __________ sq ft-min

HOLD AREA WAITING TIMES
(use ped/cycle)

$Q_{tco} = [(v_{co})(R_{mj}/C)(R_{mj}/2)]/60$ = __________ ped-min

$Q_{tdo} = [(v_{do})(R_{mi}/C)(R_{mi}/2)]/60$ = __________ ped-min

HOLD AREA TIME-SPACE

$TS_h = 5\ (Q_{tco}+Q_{tdo})$ = __________ sq ft-min

CIRCULATION TIME-SPACE

$TS_c = TS - TS_h$ = __________ sq ft-min

TOTAL CIRCULATION VOLUME

$v_c = v_{ci}+v_{co}+v_{do}+v_{di}+v_{a,b}$ = __________ peds

TOTAL CIRCULATION TIME

$t_c = v_c \times 4/60$ = __________ ped-min

PEDESTRIAN SPACE AND LOS

$M = TS_c/t_c$ = ________ sq ft/ped; LOS = __________
(Tab.13-3)

APPENDIX II

GLOSSARY AND SYMBOLS

GLOSSARY

circulation area—The portion of a sidewalk street corner used by moving pedestrians passing through the area, in square feet.

crosswalk—The marked crossing area for pedestrians crossing a street at an intersection or designated midblock location.

effective walkway width—The width of a walkway which is usable by pedestrians; the total walkway width minus unusable "buffer" zones at the curb and building line and other unusable portions due to obstacles and obstructions in the walkway, in feet.

pedestrian—An individual traveling on foot.

pedestrian density—The number of pedestrians per unit area of pedestrian facility, in pedestrians per square foot.

pedestrian flow rate—The number of pedestrians passing a point per unit time, expressed as pedestrians per 15 minutes, or pedestrians per minute.

pedestrian area module—The space provided per pedestrian by a pedestrian facility, in square feet per pedestrian.

pedestrian speed—Average walking speed of pedestrians, in feet per second.

platoon—A number of pedestrians walking together in a group, usually involuntarily, because of signal control and other factors.

platoon flow rate—The pedestrian rate of flow within a pedestrian platoon, in pedestrians per minute.

street corner—The area encompassed within the intersection of two sidewalks.

unit width flow rate—The pedestrian rate of flow expressed in pedestrians per minute per foot of walkway or crosswalk width.

walkway—A facility provided for pedestrian movement, segregated from vehicular traffic by a curb, or provided on a separate right-of-way.

SYMBOLS

A	the area of a pedestrian facility, or portion thereof; common subscripts include c (circulation area), w (crosswalk area), h (holding area), and so on, in square feet.
D	density, in pedestrian per square foot.
v	pedestrian flow rate, in pedestrian per minute or pedestrian per cycle.
v_p	pedestrian flow rate in platoons, in pedestrian per minute or pedestrian per cycle.
M	pedestrian space, in square feet per pedestrian.
Q_{ti}	total time spent in a holding area by pedestrians in flow i, in pedestrian-minute, in one signal cycle.
Q_i	total number of pedestrians in a holding area, for flow i, in pedestrians, during one signal cycle.
r	corner radius, in feet.
S	average pedestrian speed, in feet per minute.
t_w	average time a pedestrian spends in a crosswalk, in seconds.
TS	time-space in a pedestrian area, in square feet-minute, subscripts include h (holding area time-space), c (circulation area time-space), and so on.
V_m	maximum number of pedestrians occupying a crosswalk during one signal cycle.
W_T	total width of a walkway, in feet.
W_B	unusable width of a walkway, in feet.
W_E	effective, or usable, width of a walkway, in feet.

CHAPTER 14

BICYCLES

CONTENTS

I. INTRODUCTION

A *bicycle* is defined as a vehicle having two tandem wheels, propelled soley by human power, upon which any person or persons may ride.

Bicycles make up a small percentage of the traffic stream at most locations in North America. Nevertheless, there are many locations where the impact of bicycles on the vehicular traffic stream is noticeable. Many cities have initiated extensive programs to provide facilities for bicycles in the form of designated bicycle lanes on streets and highways and bikeways with physically separated rights-of-way. The use of bicycles as a regular means of personal transportation has increased, particularly in warm climates. The bicycle is a popular mode in and around many university campuses, and is an attractive alternative in congested city areas where vehicular traffic is difficult.

While the state of knowledge concerning specific impacts of bicycles on the capacity and level of service of highway facilities is not advanced, this chapter presents some insights and procedures for approximately analyzing the effects of bicycles in the traffic stream. It also presents approximate information on the capacity of various types of bicycle facilities. Specifically, this chapter addresses the following aspects of bicycle capacity:

1. The impacts of bicycle presence on intersection capacity.
2. The impacts of bicycle presence on roadway segments between intersections.
3. The capacity of designated bicycle facilities.

The sections that follow detail these types of analyses, and illustrate their use with sample calculations.

II. METHODOLOGY AND PROCEDURES FOR APPLICATION

IMPACTS ON INTERSECTION CAPACITY

Bicycles affect the capacity and operating conditions at intersections in two principal ways:

1. Where bicycles share a lane with other vehicles, they utilize a portion of the lane's capacity. This effect is accounted for by assigning an appropriate "passenger-car equivalent" (pce) for each bicycle.
2. Where vehicles execute turning movements through a conflicting bicycle stream, they encounter opposition in addition to that normally presented by opposing vehicle streams and pedestrians. The intersection analysis techniques of Chapters 9 and 10 should be modified to account for this conflict.

Passenger-Car Equivalents for Bicycles

Table 14-1 presents the recommended values of passenger-car equivalents for bicycles. The equivalent varies with lane width and depends on whether the bicycle movement in question is "opposed" or "unopposed."

A bicycle moving straight through an intersection, encountering no significant interference from vehicles or pedestrians, is considered to be unopposed. A left-turning bicycle must cross an opposing vehicular flow on two-way streets, and would be considered to be opposed. Right-turning bicycles may or may not encounter significant pedestrian interference, and could be classified as either opposed or unopposed. Where the conflicting crosswalk flow exceeds 100 peds/hour, it is recommended that right-turning bicycles be considered opposed.

As indicated in Table 14-1, the impact of bicycles sharing vehicular lanes increases as lane width decreases. When lane widths are 14 ft or greater, bicycles tend to use a portion of the lane as a bike lane, and have little impact on vehicular flow. It should also be noted that these factors are conservative, as they assume that most bicyclists move through the intersection on the green signal.

Table 14-1 is used as follows. The number of bicycles (segregated by type of movement) is multiplied by the appropriate passenger-car equivalent values. The result is added to the vehicular volume, yielding a total equivalent vehicular volume which is used in subsequent computations. Consider a signalized intersection with a vehicular volume of 500 vph which shares a 10-ft lane with a bicycle volume of 100 bicycles/hour, one-half of which are opposed.

Then:

$$\begin{aligned}\text{Equivalent volume} &= 500 + 100(0.5)(1.2) \\ &\quad + 100(0.5)(1.0) \\ &= 500 + 60 + 50 = 610 \text{ vph}\end{aligned}$$

where 1.2 and 1.0 are the passenger-car equivalent values for opposed and unopposed bicycle movements selected from Table 14-1. Further computations would proceed using a volume of 610 vph in the procedures of Chapter 9, "Signalized Intersections."

TABLE 14-1. PASSENGER-CAR EQUIVALENT FOR BICYCLES

BICYCLE MOVEMENT	LANE WIDTH (ft) < 11	11–14	> 14
Opposed	1.2	0.5	0.0
Unopposed	1.0	0.2	0.0

Effect of Bicycles on Right-Turning Vehicles

At intersections where a curb bicycle lane is provided, right-turning vehicles encounter not only a conflicting pedestrian flow, but a conflicting bicycle flow as well. Figure 14-1 illustrates these conflicts.

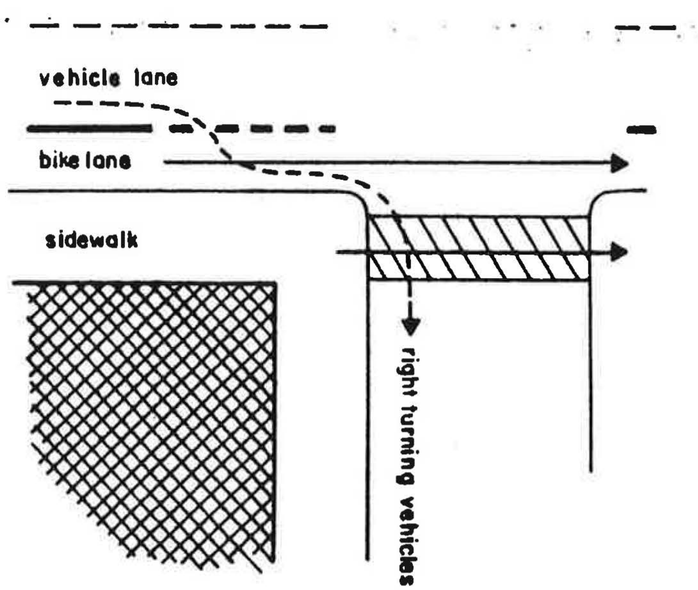


Figure 14-1. Illustration of right turn conflicts with bicycles and pedestrians.

Where such conflicts exist, right-turning vehicles experience considerably more friction than in situations where no bike lane exists. Table 9-12, of Chapter 9, "Signalized Intersections," gives adjustment factors used in correcting for the impact of pedestrian interference on right-turn saturation flow. Where a bicycle lane exists, it is recommended that this table be entered with total number of pedestrians plus bicycles which interfere with the subject right-turn movement. Thus, if a right-turn movement must cross a pedestrian flow of 100 peds/hour and a bicycle flow of 150 bicycles/hour, Table 9-12 would be entered as if the conflicting pedestrian flow were 100 + 150 = 250 peds/hour.

Where bicycles share a vehicular lane, it is not necessary to include this adjustment because the approach volume is already inflated to account for bicycle presence. Where shared-lane width is 14 ft or greater, however, it was assumed that bicycles separate into the right portion of the lane, using it essentially as a bike lane. In such cases, their impact on right-turning vehicles should be considered as indicated in this section.

Left-Turning Bicycles from Bike Lanes

Bicycles turning left out of a bike lane must mix with other vehicles as they approach the intersection and execute the left-turn maneuver. An appropriate passenger-car equivalent value is selected from Table 14-1 and added to the vehicular volume in the left-most lane. The passenger-car equivalent value for bicycles is also added to the volume in each lane the bicycles must cross in transferring from the right-hand bike lane to the left-most traffic lane.

EFFECTS OF BICYCLES ON ROADWAY SEGMENTS BETWEEN INTERSECTIONS

There is little existing data or information on the impacts of bicycles on capacity or operating conditions between intersections. Bicycles are not expected to have any impact on flow where curb-lane widths exceed 14 ft. Where bicycle volumes are less than 50/hour, impacts are also believed to be negligible, except where lanes are narrow ($\leq$ 11 ft).

One study (*1*) has indicated that vehicular intersection approach speeds are reduced by approximately 2.5 mph when bicycles are present in an adjacent bike lane.

BICYCLE FACILITIES

Bicycle facilities separated from vehicular traffic can be provided in two basic forms:

1. *Bike lane*—A portion of a roadway which has been designated by striping, signing, and pavement markings for the preferential or exclusive use of bicyclists.
2. *Bike path*—A bikeway physically separated from motorized vehicular traffic by an open space or barrier, either within the highway right-of-way or within an independent right-of-way.

There is not a great deal of information available concerning the capacity of such facilities. Planning and design criteria for bicycle facilities are available from a number of sources (*2–5*), including the *Transportation and Traffic Engineering Handbook* (*6*). A summary of available data was compiled from Ref. *2*, and is presented in Table 14-2.

Reference *3* cites the capacity of a bicycle facility as 0.22 bicycles per second per foot of bikeway. This is equivalent to 2,376 bicycles/hour for a 3-ft bikeway.

TABLE 14-2. REPORTED ONE-WAY AND TWO-WAY HIGH VOLUMES OF BICYCLE FACILITIES

TYPE OF FACILITY	NO. OF LANES[a]	RANGE OF REPORTED CAPACITIES (BICYCLES/HOUR)
One Way Bike Lane or Path	1	1,700–2,530
Two Way Bike Path	1	850–1,000
	2	500–2,000

[a] Lane widths 3–4 ft/lane
SOURCE: Adapted from Refs. *2* and *6*

It should be noted that the wide variation of reported high volumes reflects a similarly wide range in environmental conditions, skill and familiarity of cyclists, and specific geometric features of the facilities reported. Bikeway capacity is also rarely observed in practice, as demand levels are generally well below the capacity of the facility. Indeed, the planning and design documents referenced previously all emphasize the need to have bicycle facilities that provide sufficient capacity to allow good-to-excellent operating conditions if they are to be successful in encouraging bike use.

III. SAMPLE CALCULATIONS

CALCULATION 1—PASSENGER-CAR EQUIVALENTS

1. *Description*—An intersection approach with one 12-ft lane has a vehicular demand of 500 vph. It is shared by 50 bicycles/hour, 10 of which turn left and 15 of which turn right across a flow of 110 peds/hour. Convert the approach volume to an equivalent which accounts for the effects of bicycles.

2. *Solution*—Both left-turning and right-turning bicycles are considered to be "opposed." From Table 14-1, the following passenger-car equivalent values are found:

1 Through bicycle = 0.2 pce
1 Left-turning bicycle = 0.5 pce
1 Right-turning bicycle = 0.5 pce

The total equivalent demand volume on the intersection approach may then be expressed as:

$$\begin{aligned}\text{Equivalent volume} &= 500 + 25(0.2) + 10(0.5) + 15(0.5)\\ &= 500 + 5 + 5 + 7.5\\ &= 517.5, \text{ SAY } 518 \text{ vph}\end{aligned}$$

Note that this is *not* the final conclusion of the analysis of the intersection in question. If the intersection were signalized, the analysis would proceed using the procedures of Chapter 9, but with a demand volume for 518 vph on the subject approach rather than 500 vph, which is the actual vehicular demand volume. If the intersection were unsignalized, the procedures of Chapter 10 would be applied to complete the analysis.

CALCULATION 2—LEFT-TURN IMPACTS ON A MULTILANE APPROACH

1. *Description*—An intersection approach has three traffic lanes and a right-hand curb bicycle lane. The three lanes have the following approach volumes: left lane, 250 vph; center lane, 350 vph; right lane, 220 vph. There are 50 bicycles/hour executing left turns. How should the vehicular volumes be adjusted to reflect the impact of these bicycles. Traffic lanes are 11 ft wide.

2. *Solution*—From Table 14-1, each bicycle has an equivalent of 1.2 (opposed, 11-ft lanes). Thus, the 50 left-turning bicycles/hour are equivalent to $50 \times 1.2 = 60$ vph. These passenger-car equivalents should now be added to the volume in all three approach lanes. Thus, any additional analysis would proceed using the following adjusted approach volumes:

Left lane: $250 + 60 = 310$ vph
Center lane: $350 + 60 = 410$ vph
Right lane: $220 + 60 = 280$ vph

Note that the equivalents are added to each lane that is crossed by bicycles transferring from the bike lane to the left-most traffic lane.

CALCULATION 3—IMPACTS OF A BIKE LANE ON RIGHT-TURNING VEHICLES

1. *Description*—A single-lane approach at a signalized intersection is adjacent to a curb bike lane carrying 400 bicycles/hour. What right-turn adjustment factor would be selected if right-turning vehicles also interfere with a pedestrian flow of 200 pedestrians/hour? Right turns make up 20 percent of the total volume in the single lane.

2. *Solution*—Right-turn adjustment factors for right turns at signalized intersections are selected from Table 9-12 (Ch. 9). Single-lane approaches are represented by Case 7 in that table. A factor would normally be selected for 200 pedestrians/hour and 20 percent right turns, yielding an adjustment factor of 0.86, which is applied to the saturation flow rate for the approach.

Where a bicycle lane is present, however, the factor is selected as if the pedestrian volume were the total of pedestrians and bicycles. Thus, a factor is selected for $200 + 400 = 600$ pedestrians and 20 percent right turns. This factor would be 0.82. Thus, the presence of the bicycle reduces the capacity of the single-lane approach by $0.86 - 0.82 = 0.04$, or 4 percent.

IV. REFERENCES

1. Opiela, K., Khasulis, S., and Datta, T., "Determination of the Characteristics of Bicycle Traffic at Urban Intersections." *Transportation Research Record 743,* Transportation Research Board (1980).
2. *Bikeway Planning Criteria and Guidelines.* Institute of Traffic and Transportation Engineering, University of California at Los Angeles (1972).
3. *Safety and Locational Criteria for Bicycle Facilities.* Users Manual, Vol. 2, Federal Highway Administration, Washington, D.C. (1976).
4. "Pedestrian and Bicycle Accommodation and Projects." *Federal Register,* Vol 49, No. 57, Federal Highway Administration, Washington, D.C. (Mar. 22, 1984).
5. *Guide to Development of New Bicycle Facilities,* American Association of State Highway and Transportation Officials, Washington, D.C. (1981).
6. King, C., and Harkens, W., "Geometric Design." *Transportation and Traffic Engineering Handbook,* Institute of Transportation Engineers, Prentice-Hall, Englewood Cliffs, N.J. (1976).

APPENDIX I

TABLES FOR USE IN ANALYSIS OF BICYCLES

TABLE 14-1. PASSENGER-CAR EQUIVALENT FOR BICYCLES

BICYCLE MOVEMENT	LANE WIDTH (ft)		
	< 11	11–14	> 14
Opposed	1.2	0.5	0.0
Unopposed	1.0	0.2	0.0

TABLE 14-2. REPORTED ONE-WAY AND TWO-WAY HIGH VOLUMES OF BICYCLE FACILITIES

TYPE OF FACILITY	NO. OF LANES[a]	RANGE OF REPORTED CAPACITIES (BICYCLES/HOUR)
One Way Bike Lane or Path	1	1,700–2,530
Two Way Bike Path	1	850–1,000
	2	500–2,000

[a] Lane widths 3–4 ft/lane
SOURCE: Adapted from Refs. *2* and *6*

APPENDIX II

GLOSSARY

bicycle—A vehicle having two tandem wheels, propelled solely by human power, upon which any person or persons may ride.

bike—A common term used in place of "bicycle."

bike lane—A portion of a roadway which has been designated by striping, signing, and pavement markings for the preferential or exclusive use of bicyclists.

bike path—A bikeway physically separated from motorized vehicular traffic by an open space or barrier, either within the highway right-of-way or within an independent right-of-way.

bikeway—Any road, path, or way which in some manner is specifically designated as being open to bicycle travel, regardless of whether such facilities are designated for the exclusive use of bicyclists, or are to be shared with other transportation modes.

passenger-car equivalent—A value representing the number of passenger cars which would occupy the same amount of roadway space as a given number of trucks, buses, bicycles, or other vehicle type.