Technical Report Documentation Page

1. Report No. FHWA/TX-97/1393-4F	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle PROCEDURES TO DETERMINE FRONTAGE ROAD LEVEL OF SERVICE AND RAMP SPACING		5. Report Date August 1996 6. Performing Organization Code
7. Author(s) Kay Fitzpatrick, R. Lewis Nowlin, and Angelia H. Parham		8. Performing Organization Report No. Research Report 1393-4F
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135		10. Work Unit No. (TRAIS) 11. Contract or Grant No. Study No. 0-1393
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Transfer Office P.O. Box 5080 Austin, Texas 78763-5080		13. Type of Report and Period Covered Final: September 1993 - August 1996 14. Sponsoring Agency Code

15. Supplementary Notes

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.
Research Study Title: Determination of Capacity and Level of Service on Freeway Frontage Roads

16. Abstract

The main objectives of this study were to develop procedures for estimating the level of service on freeway frontage roads and to determine desirable spacings for ramp junctions. The tasks involved developing 1) procedures for analyzing frontage road weaving sections, 2) recommended spacing requirements for ramp junctions, and 3) a technique to evaluate overall operations on a continuous frontage road section. The two weaving segments analyzed included a one-sided weaving area formed by an exit ramp followed by an entrance ramp and connected by an auxiliary lane and a two-sided weaving area formed by an exit ramp followed by a downstream signalized intersection. Spacing guidelines were developed for the following frontage road sections: exit ramp to entrance ramp; exit ramp to downstream signalized intersection; and signalized intersection to metered entrance ramp. The technique to analyze overall frontage road operations can be used to estimate the level of service for a frontage road section several kilometers in length.

17. Key Words Level of Service, Frontage Roads, One-Sided Weaving, Two-Sided Weaving, Ramp Spacing	18. Distribution Statement No restrictions. This document is available to the public through NTIS: National Technical Information Service 5285 Port Royal Road Springfield, Virginia 22161

19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 148	22. Price

Form DOT F 1700.7 (8-72)

PROCEDURES TO DETERMINE FRONTAGE ROAD LEVEL OF SERVICE AND RAMP SPACING

by

Kay Fitzpatrick, P.E.
Associate Research Engineer

R. Lewis Nowlin
Assistant Research Scientist

and

Angelia H. Parham, P.E.
Assistant Research Engineer

Research Report 1393-4F
Research Study Number 0-1393
Research Study Title: Determination of Capacity and Level of Service on Freeway Frontage Roads

Sponsored by the
Texas Department of Transportation
In Cooperation with
U.S. Department of Transportation
Federal Highway Administration

August 1996

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

IMPLEMENTATION STATEMENT

This report presents procedures for estimating the level of service on freeway frontage roads. The results from this report will aid engineers in evaluating one-way and two-way continuous frontage road sections. In addition, procedures are provided for evaluating one-sided and two-sided weaving segments on one-way frontage roads. Engineers can use the procedures to estimate the level of service on these types of facilities, which, in turn, can aid in prioritizing frontage road improvement projects and/or predicting future operations. Recommended spacing requirements for ramp junctions are also contained in this report.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Texas Department of Transportation (TxDOT) or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. This report was prepared by Kay Fitzpatrick (PA-037730-E), R. Lewis Nowlin, and Angelia H. Parham (TN-100,307).

ACKNOWLEDGMENT

The Texas Department of Transportation study 1393 technical advisory panel chair, Wayne Dennis, along with the technical panel (Lilly Banda, Cheryl Flood, and Fred Marquez) are recognized for their time in providing direction and comments for this research. This study was performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

The authors would also like to recognize the following persons for helping with the data collection, reduction efforts, and report preparation: Jon Collins, Shirley Kalinec, Stacy King, Molly Marshall, Chuck McIlroy, Kelly Quy, Jason Vaughn, and Dan Walker.

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF TABLES

LIST OF TABLES

SUMMARY

Using frontage roads as a component of freeway design has important advantages, including operational flexibility to handle emergency traffic situations, accessibility to connecting streets and commercial development along the freeway corridor, and additional capacity when the freeway reaches maximum flow. The state of Texas has realized the importance and advantages of the freeway frontage road system as witnessed by the extensive incorporation of frontage roads into the Texas urban freeway system.

Techniques to estimate capacity and level of service on freeways and urban arterials are detailed in the current *Highway Capacity Manual* (*HCM*); however, these procedures cannot be applied directly to frontage roads, as they often combine features from both freeways and arterials. Even when weaving is expected to dominate frontage road operations, the speed assumptions in the *HCM* freeway weaving analysis make it unusable for frontage road analysis. Techniques must be developed to enable engineers to adequately design frontage roads for expected volumes, to predict operating conditions under a range of flows, and to guide in the selection of alternatives for solving operational problems.

The overall objectives of this study were to develop procedures for estimating the level of service on freeway frontage roads and to determine desirable spacings for ramp junctions. The study involved developing 1) procedures for analyzing frontage road weaving sections, 2) recommended spacing requirements for ramp junctions, and 3) a technique to analyze overall operations on a continuous frontage road section. The two weaving segments analyzed included a one-sided weaving area formed by an exit ramp followed by an entrance ramp connected by an auxiliary lane and a two-sided weaving area formed by an exit ramp followed by a downstream signalized intersection. Spacing guidelines were developed for the following frontage road sections: exit ramp to entrance ramp; exit ramp to downstream signalized intersection; and signalized intersection to metered entrance ramp. The technique to analyze overall frontage road operations can be used to estimate the level of service on a frontage road section several kilometers in length.

CHAPTER 1
INTRODUCTION

Frontage roads are an integral part of the Texas freeway system. They provide access to land development adjacent to the freeway and connect the freeway with local streets. In addition, frontage roads can serve as alternate routes to the freeway during congestion, maintenance activities, or emergencies. The state of Texas has realized the importance and advantages of the freeway frontage road system as witnessed by the extensive incorporation of frontage roads into the Texas urban freeway system.

Frontage roads contain characteristics of both freeways and arterial streets. Frontage roads are one-way or two-way, contain entrance and exit ramps servicing the freeway, and provide access to local driveways and low priority streets. In addition, the frontage road system is interconnected with the major streets intersecting the freeway, usually as signalized or stop-controlled intersections.

PROBLEM STATEMENT

Procedures are currently available in the *1994 Highway Capacity Manual* (*HCM*) (1) to estimate capacity and level of service on freeways and urban arterials; however, these procedures may not be appropriate for frontage roads as features from both freeways and arterials are often present. Because of this limitation, procedures must be developed to enable engineers to adequately design frontage roads for expected volumes, to predict operating conditions under a range of conditions, and to guide in the selection of alternatives for solving operational problems.

OBJECTIVES

An objective of this study was to develop procedures for estimating the level of service on freeway frontage roads. Separate procedures were developed to evaluate traffic operations for the following three scenarios: a continuous frontage road section up to several kilometers in length, a one-sided weaving area formed by an exit ramp followed by an entrance ramp connected by an auxiliary lane, and a two-sided weaving area formed by an exit ramp followed by a downstream signalized intersection. In addition, spacing guidelines were developed for the following frontage road sections: exit ramp to entrance ramp; exit ramp to downstream signalized intersection; and signalized intersection to metered entrance ramp.

ORGANIZATION

Texas Department of Transportation (TxDOT) Project 1393 developed several procedures to evaluate frontage roads and portions of frontage roads. The research conducted during the development of these procedures is documented elsewhere (2, 3, 4). This report contains the step-by-step procedures that an analyst would use to evaluate the performance along a frontage road. Figure 1-1 illustrates the different portions of a one-way frontage road that can be evaluated using techniques presented in this report. The material in Chapter 5 can also be used to evaluate the operations on a two-way frontage road section.

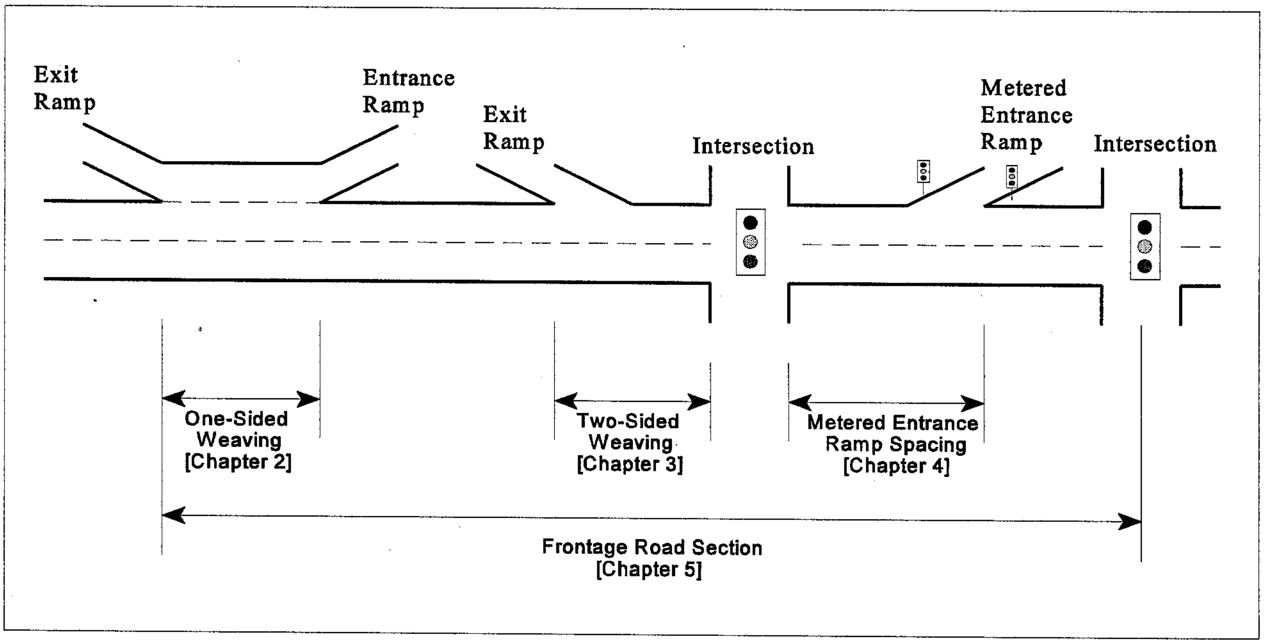


Figure 1-1. Frontage Road Analysis.

This report is divided into six chapters. Chapter 1 contains some background information concerning frontage roads and defines the problem statement, research objectives, and organization of this report. Chapter 2 provides the procedure for evaluating the operations on a one-sided weaving segment. It also presents the recommended spacing between an exit ramp and an entrance ramp when joined by an auxiliary lane. Chapter 3 contains the procedure for evaluating two-sided weaving operations when an exit ramp is followed by a signalized intersection. It also includes recommended spacing between an exit ramp and the intersection. The desired location for a ramp meter can be determined using the procedure presented in Chapter 4. The procedure provides estimates for the queue storage length and the acceleration and merging distance. Chapter 5 contains the procedure for determining level of service on freeway frontage road sections. For purposes of this procedure, a section is typically defined as being at least 0.8 km in length, with a signal spacing between 0.5 to 3.0 km. The findings and recommendations drawn from this research project are presented in Chapter 6.

Appendix A contains blank worksheets that can be used in the procedures. Summary flowcharts on how to determine the level of service on freeway frontage road sections are presented in Appendix B. Techniques on how to use the *Highway Capacity Software* to evaluate frontage roads are provided in Appendix C.

CHAPTER 2

ONE-SIDED WEAVING ANALYSIS

When all weaving movement takes place on one side of a roadway, it is referred to as one-sided weaving. One-sided weaving occurs on frontage roads when an exit ramp is followed by an entrance ramp connected by a continuous auxiliary lane (see Figure 2-1). There are many factors that influence traffic operations on one-sided weaving sections, including traffic volume, ramp spacing, and number of lanes.

The efforts documented in this chapter focus on one-sided weaving operations on one-way frontage roads. The objectives of this study were to develop a technique for evaluating one-sided weaving operations and to develop recommendations on minimum and desirable ramp spacing. To meet these objectives, both field data and computer simulation (NETSIM) were used. The intent was to use the results from the field study to calibrate a NETSIM model and use the NETSIM model to predict various measures of effectiveness (MOEs) under different conditions.

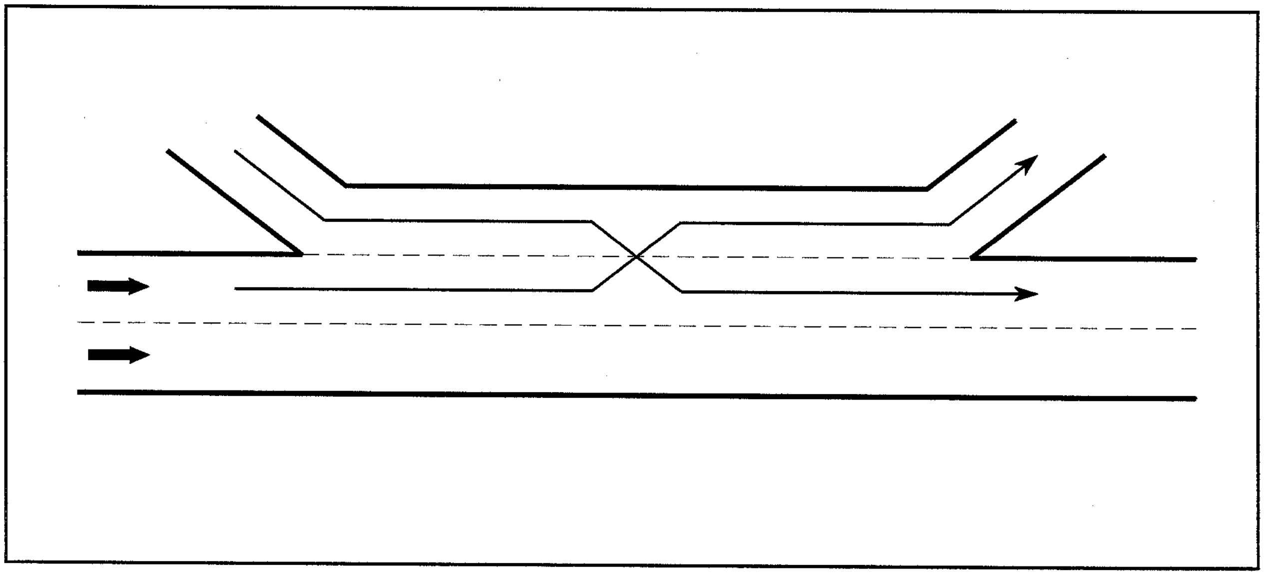


Figure 2-1. One-Sided Weaving Maneuvers on Frontage Roads.

DEVELOPMENT OF LEVEL-OF-SERVICE CRITERIA

By studying the relationships of the MOEs predicted by NETSIM, a procedure could be developed for determining the level of service (LOS) within a weaving area. The researchers investigated several MOEs, including speed, delay, travel time, and number of lane changes. After an analysis of one-sided weaving areas, it was concluded that the average speed on the weaving link (i.e., weaving speed) would be the proposed MOE. Speed is easy to measure in the field, and it is easy to explain and understand.

Findings

In an attempt to use weaving speed to determine the LOS on a weaving section, the relationships between weaving speed and several other variables were studied. These variables included weaving volume, total volume, and number of lane changes. From the analysis, it was concluded that weaving speed is most closely related to lane changes.

Figure 2-2 illustrates the relationship between weaving speed and number of lane changes per hour (lc/hr) for one-sided weaving areas with weaving lengths of 100 to 500 meters. Observing this figure, there appear to be certain critical points (or break points) in which the weaving speed begins to drop noticeably. For instance, there is a critical lane change value (approximately 2000 lc/hr) in which the weaving speed begins to drop more rapidly. Also, as the number of lane changes increases, there is another point (approximately 4000 lc/hr) in which speeds drop significantly and become more variable. The latter critical point was also evident in the relationship between the speed prior to the weaving link and lane changes (see Figure 2-3). As shown in Figure 2-3, the speeds prior to the weaving link are relatively stable up to approximately 4000 lc/hr. Above 4000 lc/hr, the speeds drop and become more variable. For both Figures 2-2 and 2-3, the 100 meter weaving sections began to break down sooner than weaving sections with lengths of 200 meters and above.

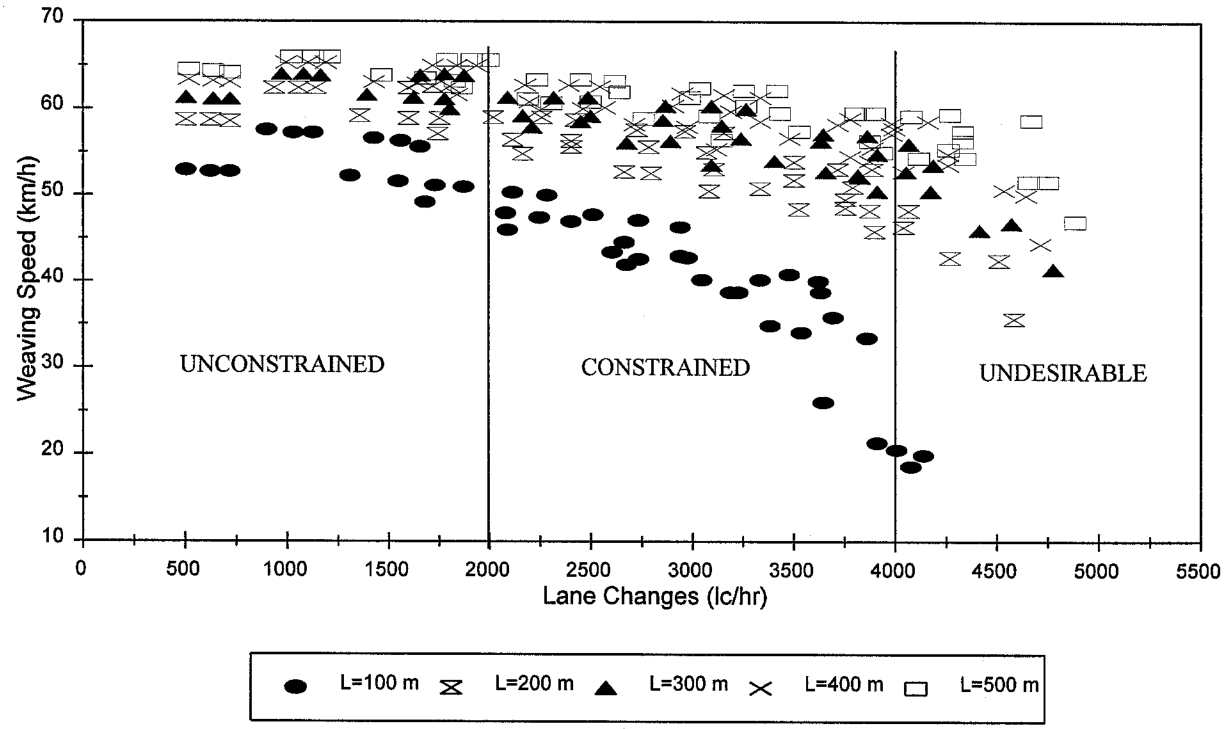


Figure 2-2. Breaking Points for Weaving Speed and Lane Change Relationship.

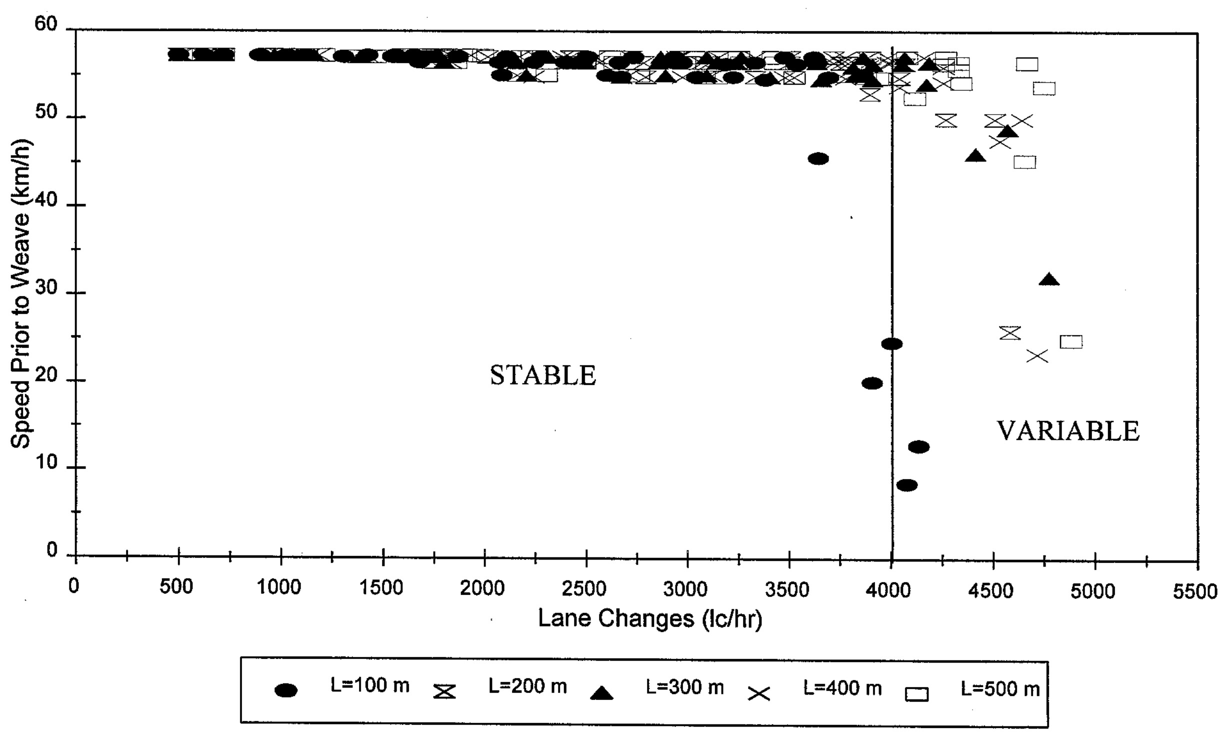


Figure 2-3. Breaking Point for Prior Speed and Lane Change Relationship.

Level-of-Service Criteria

Using the critical lane change values, each weaving section was divided into three levels of service: unconstrained, constrained, and undesirable. These three levels of service correspond to the following levels of service defined by the *HCM*: unconstrained = LOS A-B, constrained = LOS C-D, and undesirable = LOS E-F. Unconstrained operations represent free flow to stable operations in which drivers can maneuver with relatively little impedance from other traffic. Constrained operations represent stable operations in which drivers' ability to maneuver becomes more restricted due to other traffic. Undesirable operations represent unstable operations in which flows are approaching capacity and drivers' ability to maneuver is highly restricted.

Because the number of lane changes is difficult to measure in the field, a method was developed for converting lane changes to weaving volume. Weaving volume is defined as the sum of the exit ramp volume and the entrance ramp volume. Results from the field data showed that a linear relationship existed between weaving volume and the number of lane changes: average number of lane changes = 1.33 × weaving volume. Using this relationship, the level-of-service criteria were defined in terms of weaving volume. The LOS criteria are shown in Table 2-1.

Table 2-1. Level-of-Service Criteria.

Level of Service	Average Lane Changes (lcph)	Weaving Volume* (vph)
Unconstrained	< 2000	< 1500
Constrained	2000 - 4000	1500 - 3000
Undesirable	> 4000	> 3000

* weaving volume = average lane changes / 1.33

Due to the range of data included in this study, the criteria in Table 2-1 apply to one-sided weaving areas on one-way frontage roads with the following characteristics:

- frontage road section containing a freeway exit ramp followed by an entrance ramp connected by an auxiliary lane,
- either two or three frontage road through lanes, and
- spacing between exit ramp and entrance ramp of 100 to 500 meters.

TECHNIQUE FOR DETERMINING LEVEL OF SERVICE

To estimate the level of service for an existing one-sided weaving segment, the following procedures should be followed:

(1) Collect peak hour exit ramp and entrance ramp volumes for the one-sided weaving section.

(2) Calculate weaving volume (vph): weaving volume = exit ramp volume + entrance ramp volume.

(3) Compare the calculated weaving volume to the values listed in Table 2-1 to estimate the LOS.

A worksheet for determining the level of service on one-sided weaving sections is provided in Appendix A of this report.

The level-of-service criteria in Table 2-1 are not meant to represent exact divisions in LOS. The values are intended to provide a general idea of the LOS which might be expected for a particular weaving segment; therefore, engineering judgement should be used when applying these criteria.

SAMPLE CALCULATION

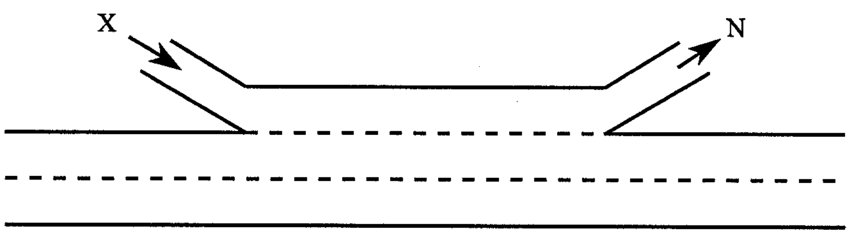
As an example, consider a one-sided weaving section on a one-way frontage road with the following peak period volumes: exit ramp volume, 750 vph; entrance ramp volume, 1000 vph. Adding the exit ramp volume and the entrance ramp volume results in a weaving volume of 1750 vph. Comparing the weaving volume to the level-of-service criteria in Table 2-1, traffic operations in this area are predicted to be operating in the constrained region (see Figure 2-4 for an example of the worksheet).

ONE-SIDED WEAVING ANALYSIS WORKSHEET

Location: IH-20　　Direction: West - bound

Description: Between 45th and Crosby

Date: 07/10/96　　Prepared By: Sally



Exit Ramp Volume (X): 750 **vph**　　**Entrance Ramp Volume (N):** 1000 **vph**

Weaving Volume (X + N): 1750 **vph**

Weaving Volume	Level of Service
< 1500 vph	Unconstrained
1500 - 3000 vph	Constrained
> 3000 vph	Undesirable

Level of Service: Constrained

Figure 2-4. Sample Calculation for One-Sided Weaving Analysis.

WEAVING LENGTH

The spacing between an exit ramp and a downstream entrance ramp can have a great effect on the operations of a weaving section. The effect of weaving length on traffic operations becomes more evident as traffic volumes increase. To illustrate this point, the results from NETSIM were used to examine the speeds of weaving vehicles on weaving sections with different lengths at high traffic volumes. In particular, the weaving speeds were examined at the boundary between unconstrained and constrained operations (2000 lc/hr), and at the boundary between constrained and undesirable operations (4000 lc/hr).

Figure 2-5 shows the relationships between weaving speed and weaving length. This figure illustrates that weaving speed decreases at a relatively low rate as weaving length decreases for lengths above 300 meters. The rate at which the speeds decrease becomes greater for weaving lengths between 200 and 300 meters, and the rate of decrease is greatest for weaving lengths below 200 meters. These findings correspond to other findings in this study that showed that the weaving sections with a length of 100 meters began to break down sooner than those weaving sections with lengths of 200 meters and above (see Figures 2-2 and 2-3). From these results, it was concluded that it is desirable to have a weaving length greater than 300 meters. If this length is not achievable, then the absolute minimum length should be approximately 200 meters.

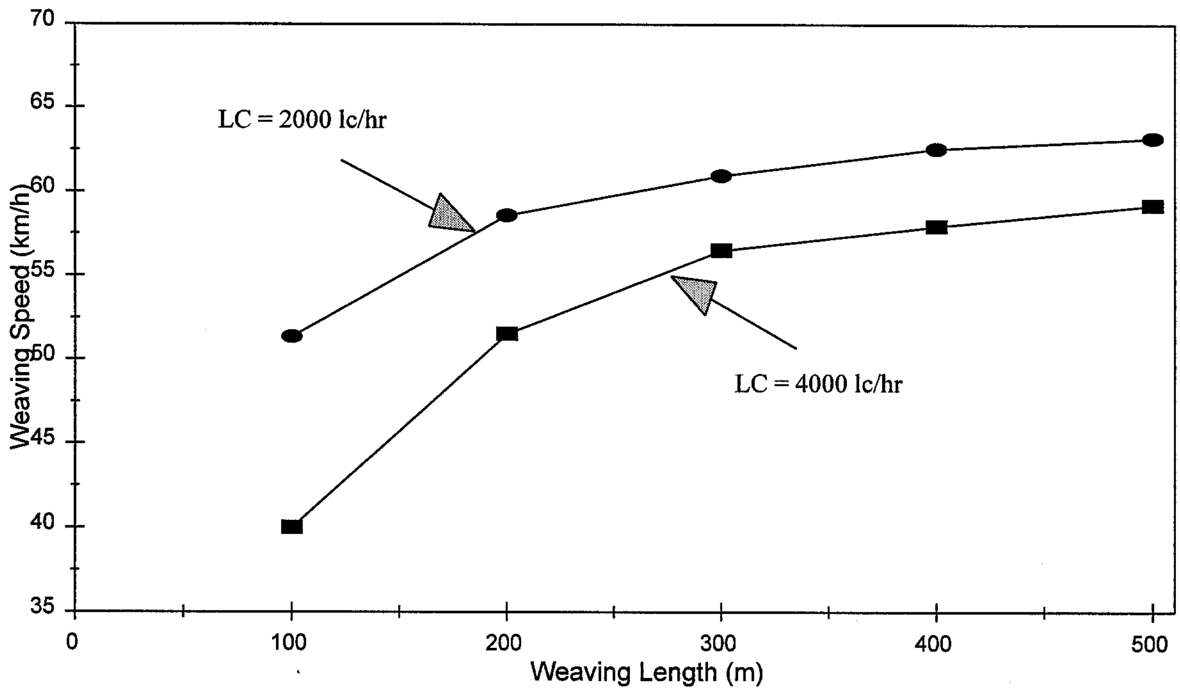


Figure 2-5. Weaving Speed and Weaving Length Relationship.

CHAPTER 3

TWO-SIDED WEAVING ANALYSIS

A frontage road section typically influenced by weaving maneuvers is the area between a freeway exit ramp and a downstream intersection. This type of area is said to have two-sided weaving operations because exit ramp vehicles desiring to make a right turn at the downstream intersection must maneuver from one side of the frontage road to the opposite side of the frontage road (see Figure 3-1). The level of operations in this type of area may be influenced by several factors, including traffic volumes, turning percentages, and ramp-to-intersection spacing.

The objectives of the study documented in this chapter were to develop a technique for evaluating two-sided weaving operations on one-way frontage roads between an exit ramp and a downstream intersection, and to develop recommended ramp-to-intersection spacings. To meet these objectives, field data and computer simulation (NETSIM) were used.

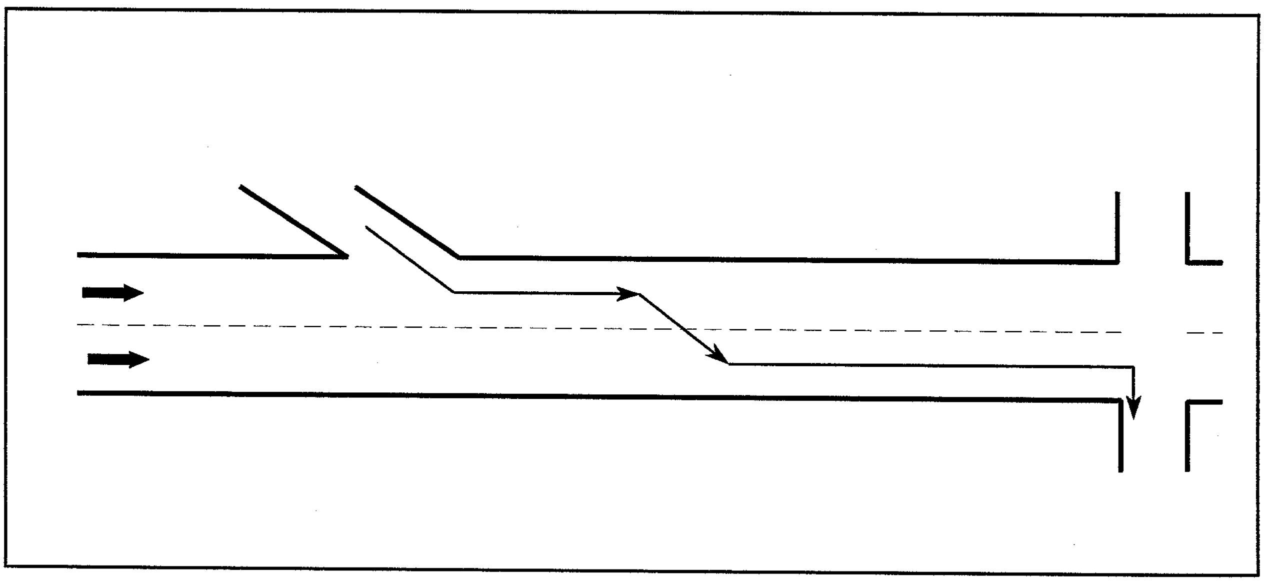


Figure 3-1. Two-Sided Weaving Maneuver Between Exit Ramp and Intersection.

DEVELOPMENT OF LEVEL-OF-SERVICE CRITERIA

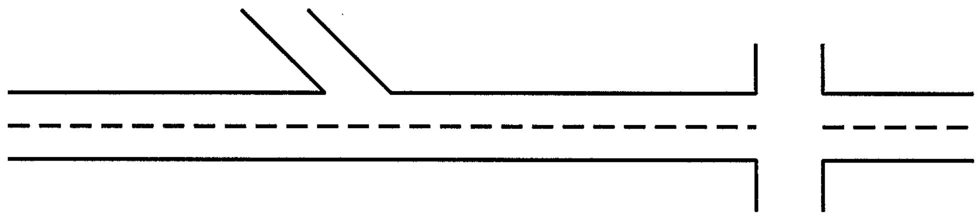
Results from the field study were used to calibrate a NETSIM model. Researchers then used the calibrated model to study two-sided weaving operations under various conditions. The variables modified during simulation included: frontage road volume (500 to 2000 vph), exit ramp volume (250 to 1250 vph), exit ramp to intersection spacing (100 to 400 meters), and percentage of exit ramp vehicles making a two-sided weaving maneuver (25 to 75 percent). In addition, three frontage road configurations were investigated: two-lane frontage road (2LFR), three lane frontage road (3LFR), and two-lane frontage road with an auxiliary lane connecting the exit ramp to the downstream intersection (2LFR+Aux). Figure 3-2 illustrates the three configurations studied.

To develop a procedure for determining the level of service on a two-sided weaving segment, several MOEs were investigated, including speed, travel time, and density. After an analysis of two-sided weaving segments using NETSIM, it was concluded that the density on the weaving link would be the proposed MOE. Density is a good measure of weaving operations because it measures the proximity of vehicles and is a reflection of drivers' freedom to maneuver.

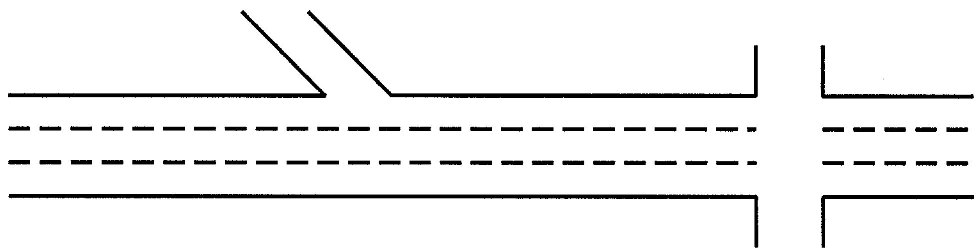
Findings

In an attempt to define level-of-service criteria, the researchers used the results from NETSIM to investigate the relationships between density and other factors. Results from the investigation revealed that a correlation exists between speed and density. Figure 3-3 illustrates the relationships between speed and density for the three frontage road configurations.

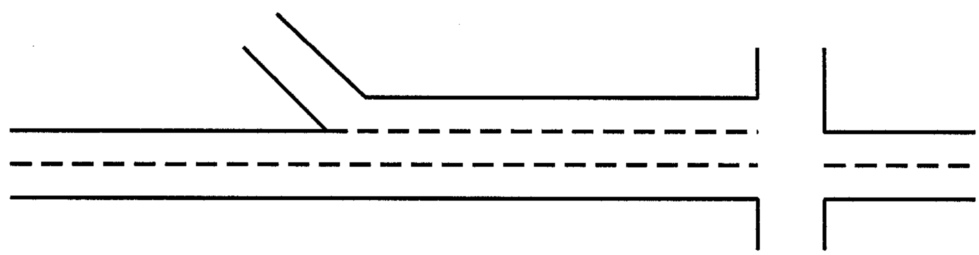
As shown in Figure 3-3, speed decreases significantly as density increases for lower density values (below approximately 40 veh/km/ln). In this range, the operations on the weaving link diminish noticeably with relatively small increases in density, and traffic operations vary from free-flow to restricted. From approximately 40 veh/km/ln to 100 veh/km/ln, the rate of decrease in speed becomes less. In this density range, traffic operations are beginning to break down and become predominately unstable. Above approximately 100 veh/km/ln, the rate of decrease begins to level off and become relatively constant, signifying that traffic operations are at their lowest level.



(a) Two-Lane Frontage Road (2LFR)



(b) Three-Lane Frontage Road (3LFR)



(c) Two-Lane Frontage Road with Auxiliary Lane (2LFR+Aux)

Figure 3-2. Three Frontage Road Configurations.

Using the relationship between speed and density, two critical values of density exist at approximately 40 and 100 veh/km/ln. These values divide the level of operations into three areas. To support the findings from computer simulation, observations at existing field sites were made.

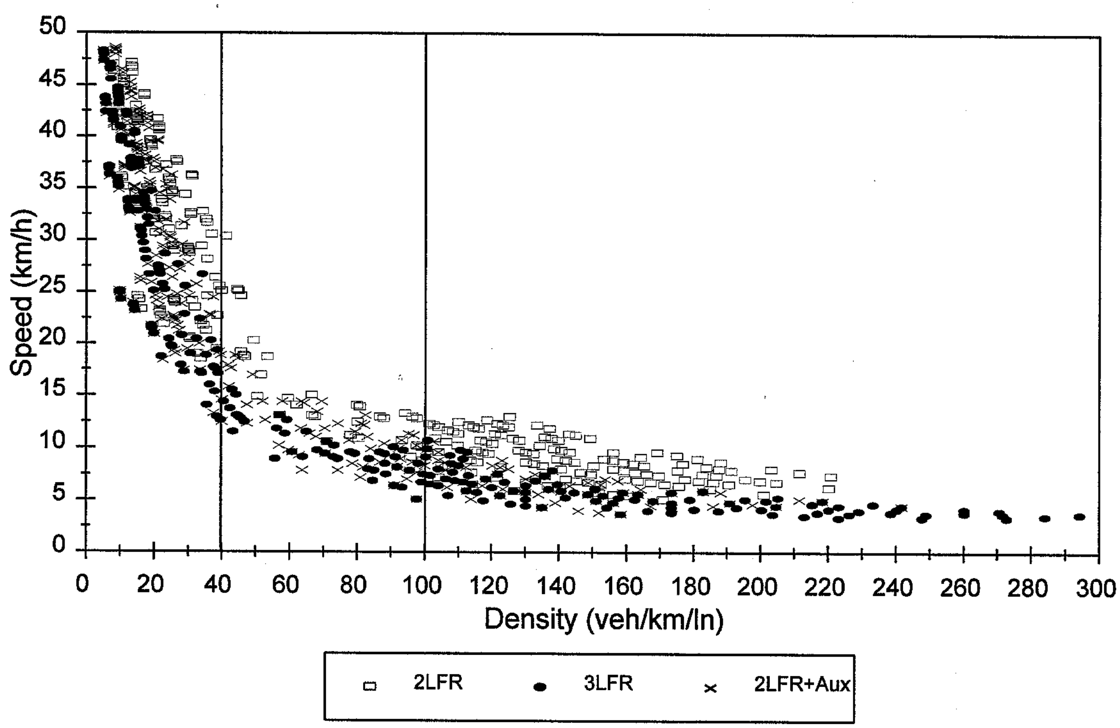


Figure 3-3. Relationship Between Speed and Density.

The objective of studying field data was to view actual two-sided weaving operations and use engineering judgement to estimate the critical densities at which there was a change in the level of service. This was accomplished by viewing the video tapes collected during the field study and estimating the level of service for varying densities.

Results from the field study corresponded to the findings derived from the relationship between speed and density for the NETSIM data. From the field data, it was determined that the critical densities dividing the levels of operations occurred at approximately 40 and 100 veh/km/ln.

Level-of-Service Criteria

Using the results from computer simulation and from the field data, traffic operations on two-sided weaving sections were divided into three levels: unconstrained, constrained, and undesirable. These three levels of operation correspond to the following levels of service defined by the 1994 *HCM* (1): unconstrained = LOS A-B, constrained = LOS C-D, and undesirable = LOS E-F.

Unconstrained operations represent predominantly free-flow operations in which drivers can maneuver with relatively little impedance from other traffic, and delay is minimal. Constrained operations represent situations in which drivers' ability to maneuver becomes more restricted due to other traffic, and delay is moderate. Undesirable operations represent situations in which flows are approaching capacity, drivers' ability to maneuver are highly restricted, and delay is high.

The level-of-service criteria are shown in Table 3-1. The ranges shown in this table are not meant to represent exact divisions in level of service; they are to be used as guides in determining the level of service on a two-sided weaving segment.

Table 3-1. Level-of-Service Criteria.

Level of Service	Density (veh/km/ln)
Unconstrained	< 40
Constrained	40 - 100
Undesirable	> 100

Predicting Density

Traffic density is defined as the number of vehicles occupying a given space at a given time. Density can be determined directly from field data; however, the process is very difficult and time consuming. In an effort to develop an easier method for estimating density, data bases were created from the NETSIM output. Stepwise regression was used to develop regression equations to predict density based on the following factors: frontage road volume, exit ramp volume, exit ramp-to-intersection spacing, and percentage of exit ramp vehicles making a two-sided weaving maneuver. With the exception of percentage of two-sided weaving maneuvers, these factors are relatively easy to collect in the field using traffic counters and a measuring wheel. To simplify the procedure of estimating percentage of two-sided weaving, the percentage of two-sided weaving vehicles was separated into the following: less than or equal to 50 percent and greater than 50 percent. The researchers felt that this separation would not affect the results since results from computer simulation showed that traffic operations were only significantly affected when the percentage of two-sided weaving maneuvers was high (i.e., above approximately 50 percent).

Density equations were derived for each of the three frontage road configurations included in the study (i.e., 2LFR, 3LFR, and 2LFR+Aux). Following are the equations that were developed:

<u>Two-Lane Frontage Road (2LFR)</u>

$$D_L = 0.034(FR) + 0.098(R) - 0.132(L) + 9.51(T) \qquad [R^2 = 0.90]$$

<u>Three-Lane Frontage Road (3LFR)</u>

$$D_L = 0.055(FR) + 0.080(R) - 0.200(L) + 27.4(T) \qquad [R^2 = 0.84]$$

<u>Two-Lane Frontage Road with Auxiliary Lane (2LFR + Aux)</u>

$$D_L = 0.021(FR) + 0.077(R) - 0.150(L) + 23.4(T) \qquad [R^2 = 0.83]$$

where:

D_L = density on weaving link, veh/km/ln
FR = frontage road volume, vph
R = exit ramp volume, vph
L = ramp-to-intersection spacing, m
T = factor based on percentage of exit ramp vehicles turning right at downstream intersection
(T = 0, Percent $\leq$ 50; T = 1, Percent > 50)

Level-of-Service Evaluation

To estimate the level of service for a particular frontage road configuration, Tables 3-2, 3-3, and 3-4 were generated. These tables contain densities based on the developed regression equations for each frontage road configuration. Calculated densities are given for various frontage road volumes, exit ramp volumes, ramp-to-intersection spacings, and percentages of exit ramp vehicles turning right at the downstream intersection (≤ 50 percent or > 50 percent). The estimated levels of service are shown using various shades: white (unconstrained), light grey (constrained), and dark grey (undesirable). The levels of service are based on the criteria shown in Table 3-1.

The criteria developed in this study did not include the effects of turn bays. Turn bays can relieve congestion, resulting in less density and improved level of service. When evaluating frontage road configurations with turn bays, engineering judgement should be used when applying the criteria developed in this study, especially when predicted densities are close to the density boundaries defining level of service (i.e., 40 or 100 veh/km/ln). For example, if a two-lane frontage road with a turn bay is predicted to have a density of approximately 105 veh/km/ln, traffic operations may be within the constrained region. If, however, the density is predicted to be 150 veh/km/ln, the traffic operations are most likely in the undesirable region.

In addition, two-sided weaving operations were analyzed in this study assuming that the cross street traffic at the intersection was moderate and the traffic signal was optimally timed to minimize overall intersection delay. Frontage road operations can be significantly impacted by poor signal timing, especially when volumes are high. Therefore, for situations in which the traffic signal is causing high delays for the frontage road approach, engineering judgement should again be used when applying the criteria developed in this study.

Table 3-2. Levels of Service for Two-Lane Frontage Roads.[a]

Spacing[b] (m)	Ramp Volume (vph)	250 vph[c]		500 vph		750 vph		1000 vph	
		≤50[d]	> 50	≤50	> 50	≤50	> 50	≤50	> 50
100	250	20	29	28	38	37	46	45	55
	500	44	54	53	62	61	71	70	79
	750	69	78	77	87	86	95	94	104
	1000	94	103	102	111	110	120	119	128
	1250	118	128	126	136	135	144	143	153
200	250	7	16	15	25	24	33	32	41
	500	31	41	40	49	48	58	56	66
	750	56	65	64	74	73	82	81	91
	1000	80	90	89	98	97	107	106	115
	1250	105	114	113	123	122	131	130	140
300	250	N/A[e]	3	2	11	10	20	19	28
	500	18	28	26	36	35	44	43	53
	750	43	52	51	61	59	69	68	77
	1000	67	77	76	85	84	94	92	102
	1250	92	101	100	110	109	118	117	127
400	250	N/A	N/A	N/A	N/A	N/A	7	6	15
	500	5	14	13	23	22	31	30	40
	750	29	39	38	47	46	56	55	64
	1000	54	64	62	72	71	80	79	89
	1250	79	88	87	97	95	105	104	113

Spacing (m)	Ramp Volume (vph)	1250 vph		1500 vph		1750 vph		2000 vph	
		≤50	> 50	≤50	> 50	≤50	> 50	≤50	> 50
100	250	54	63	62	71	70	80	79	88
	500	78	88	87	96	95	104	103	113
	750	103	112	111	121	119	129	128	137
	1000	127	137	136	145	144	154	152	162
	1250	152	161	160	170	169	178	177	187
200	250	40	50	49	58	57	67	66	75
	500	65	74	73	83	82	91	90	100
	750	89	99	98	107	106	116	115	124
	1000	114	124	122	132	131	140	139	149
	1250	139	148	147	157	155	165	164	173
300	250	27	37	36	45	44	54	52	62
	500	52	61	60	70	69	78	77	87
	750	76	86	85	94	93	103	102	111
	1000	101	110	109	119	118	127	126	136
	1250	125	135	134	143	142	152	151	160
400	250	14	24	22	32	31	40	39	49
	500	39	48	47	57	55	65	64	73
	750	63	73	72	81	80	90	88	98
	1000	88	97	96	106	105	114	113	123
	1250	112	122	121	130	129	139	138	147

[a] Density (veh/km/ln) = 0.034(FR Vol, vph)+0.098(Ramp Vol, vph) -0.132(Spacing, m)+9.51(Ramp RT%, 0 for ≤ 50%; 1 for > 50%)
[b] Spacing between exit ramp and downstream intersection
[c] Frontage road volume
[d] Percentage of ramp vehicles turning right at downstream intersection
[e] N/A - Regression equation resulted in negative density value

Unconstrained (< 40)
Constrained (40-100)
Undesirable (> 100)

Table 3-3. Levels of Service for Three-Lane Frontage Roads.[a]

Spacing[b] (m)	Ramp Volume (vph)	250 vph[c]		500 vph		750 vph		1000 vph	
		≤50[d]	> 50	≤50	> 50	≤50	> 50	≤50	> 50
100	250	14	41	27	55	41	68	55	82
	500	34	61	47	75	61	88	75	102
	750	53	81	67	95	81	108	94	122
	1000	73	101	87	114	101	128	114	142
	1250	93	121	107	134	121	148	134	162
200	250	N/A[e]	21	7	35	21	48	35	62
	500	14	41	27	55	41	68	55	82
	750	33	61	47	75	61	88	74	102
	1000	53	81	67	94	81	108	94	122
	1250	73	101	87	114	101	128	114	142
300	250	N/A	1	N/A	15	1	28	15	42
	500	N/A	21	7	35	21	48	35	62
	750	14	41	27	55	41	68	55	82
	1000	33	61	47	75	61	88	74	102
	1250	53	81	67	94	81	108	94	122
400	250	N/A	N/A	N/A	N/A	N/A	8	N/A	22
	500	N/A	1	N/A	15	1	28	15	42
	750	N/A	21	7	35	21	48	35	62
	1000	13	41	27	55	41	68	54	82
	1250	33	61	47	74	61	88	74	102
Spacing (m)	**Ramp Volume (vph)**	**1250 vph**		**1500 vph**		**1750 vph**		**2000 vph**	
		≤50	> 50	≤50	> 50	≤50	> 50	≤50	> 50
100	250	68	96	82	109	96	123	109	137
	500	88	116	102	129	116	143	129	157
	750	108	136	122	149	135	163	149	177
	1000	128	155	142	169	155	183	169	196
	1250	148	175	162	189	175	203	189	216
200	250	48	76	62	89	76	103	89	117
	500	68	96	82	109	96	123	109	137
	750	88	116	102	129	115	143	129	157
	1000	108	135	122	149	135	163	149	176
	1250	128	155	142	169	155	183	169	196
300	250	28	56	42	69	56	83	69	97
	500	48	76	62	89	76	103	89	117
	750	68	96	82	109	96	123	109	137
	1000	88	116	102	129	115	143	129	157
	1250	108	135	122	149	135	163	149	176
400	250	8	36	22	49	36	63	49	77
	500	28	56	42	69	56	83	69	97
	750	48	76	62	89	76	103	89	117
	1000	68	96	82	109	95	123	109	137
	1250	88	115	102	129	115	143	129	156

[a] Density (veh/km/ln) = 0.055(FR Vol, vph)+0.080(Ramp Vol, vph) -0.200(Spacing, m)+27.4(Ramp RT%, 0 for ≤ 50%; 1 for > 50%)
[b] Spacing between exit ramp and downstream intersection
[c] Frontage road volume
[d] Percentage of ramp vehicles turning right at downstream intersection
[e] N/A - Regression equation resulted in negative density value

Unconstrained (< 40)
Constrained (40-100)
Undesirable (> 100)

Table 3-4. Levels of Service for Two-Lane Frontage Roads with Auxiliary Lane.[a]

Spacing[b] (m)	Ramp Volume (vph)	250 vph[c]		500 vph		750 vph		1000 vph	
		≤50[d]	> 50	≤50	> 50	≤50	> 50	≤50	> 50
100	250	9	33	15	38	20	43	25	48
	500	29	52	34	57	39	62	44	68
	750	48	72	53	77	58	82	63	87
	1000	67	91	73	96	78	101	83	106
	1250	87	110	92	115	97	120	102	126
200	250	N/A[e]	18	N/A	23	5	28	10	33
	500	14	37	19	42	24	47	29	53
	750	33	57	38	62	43	67	48	72
	1000	52	76	58	81	63	86	68	91
	1250	72	95	77	100	82	105	87	111
300	250	N/A	3	N/A	8	N/A	13	N/A	18
	500	N/A	22	4	27	9	32	14	38
	750	18	42	23	47	28	52	33	57
	1000	37	61	43	66	48	71	53	76
	1250	57	80	62	85	67	90	72	96
400	250	N/A	N/A	N/A	N/A	N/A	N/A	N/A	3
	500	N/A	7	N/A	12	N/A	17	N/A	23
	750	3	27	8	32	13	37	18	42
	1000	22	46	28	51	33	56	38	61
	1250	42	65	47	70	52	75	57	81

Spacing (m)	Ramp Volume (vph)	1250 vph		1500 vph		1750 vph		2000 vph	
		≤50	> 50	≤50	> 50	≤50	> 50	≤50	> 50
100	250	30	53	35	58	40	64	45	69
	500	49	73	54	78	59	83	65	88
	750	69	92	74	97	79	102	84	107
	1000	88	111	93	116	98	122	103	127
	1250	107	131	112	136	117	141	123	146
200	250	15	38	20	43	25	49	30	54
	500	34	58	39	63	44	68	50	73
	750	54	77	59	82	64	87	69	92
	1000	73	96	78	101	83	107	88	112
	1250	92	116	97	121	102	126	108	131
300	250	N/A	23	5	28	10	34	15	39
	500	19	43	24	48	29	53	35	58
	750	39	62	44	67	49	72	54	77
	1000	58	81	63	86	68	92	73	97
	1250	77	101	82	106	87	111	93	116
400	250	N/A	8	N/A	13	N/A	19	N/A	24
	500	4	28	9	33	14	38	20	43
	750	24	47	29	52	34	57	39	62
	1000	43	66	48	71	53	77	58	82
	1250	62	86	67	91	72	96	78	101

[a] Density (veh/km/ln) = 0.021(FR Vol, vph)+0.077(Ramp Vol, vph) -0.150(Spacing, m)+23.4(Ramp RT%, 0 for ≤ 50%; 1 for > 50%)
[b] Spacing between exit ramp and downstream intersection
[c] Frontage road volume
[d] Percentage of ramp vehicles turning right at downstream intersection
[e] N/A - Regression equation resulted in negative density value

Unconstrained (< 40)
Constrained (40-100)
Undesirable (> 100)

TECHNIQUE FOR DETERMINING LEVEL OF SERVICE

To estimate the level of service between an exit ramp and an intersection on a one-way frontage road, the following procedures should be followed:

(1) From the field, collect exit ramp and frontage road volumes and determine the exit ramp-to-intersection spacing. In addition, estimate the percentage of exit ramp vehicles making a right turn at the downstream intersection as either less than or equal to 50 percent or greater than 50 percent.

(2) Based on the frontage road configuration, use Table 3-2 (2LFR), Table 3-3 (3LFR), or Table 3-4 (2LFR+Aux) to estimate the level of service.

(3) For volumes and ramp-to-intersection spacings that fall between the increments shown in the tables, one should either interpolate between the columns and rows to predict density or calculate the density using the appropriate regression equation (given at the bottom of each table).

A worksheet for determining the level of service on one-sided weaving sections is provided in Appendix A of this report.

The criteria developed in this study are not meant to represent exact divisions in level of service. The values are intended to provide a general idea of the level of service which might be expected for a particular two-sided weaving segment; therefore, engineering judgement should be used when applying these criteria. Special considerations should be given to frontage road configurations with turn bays and situations in which a signalized intersection is causing high delays for the frontage road approach.

SAMPLE CALCULATION

As an example, consider a two-lane frontage road with a ramp-to-intersection spacing of approximately 200 meters, a frontage road volume of 1000 vph, a ramp volume of 500 vph, and an exit ramp right turn percentage less than 50 percent. Using Table 3-2, the estimated density would be approximately 56 veh/km/ln. This results in a level of service in the constrained region (40 - 100 veh/km/ln). The completed worksheet is shown in Figure 3-4.

TWO-SIDED WEAVING ANALYSIS WORKSHEET

Location: IH-19 at University   Direction: South - bound

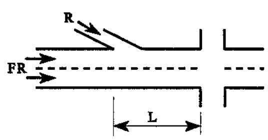
Description: 2-Lane Frontage Road

Date: 6/30/96   Prepared By: Sally

Exit Ramp Volume (R): 500 vph   Ramp Spacing (L): 200 m

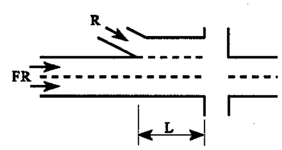
Frontage Road Volume (FR): 1000 vph   Percent 2-Sided Weaving (T): 0

[T=0 for ≤ 50%, T= 1 for > 50%]



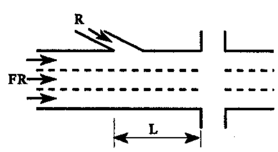
2LFR

$D_L = 0.034(FR) + 0.098(R) - 0.132(L) + 9.51(T)$



2LFR+Aux

$D_L = 0.021(FR) + 0.077(R) - 0.150(L) + 23.4(T)$



3LFR

$D_L = 0.055(FR) + 0.080(R) - 0.200(L) + 27.4(T)$

Density (D_L): 56 veh/km/ln

Density, veh/km/ln	Level of Service
< 40	Unconstrained
40 - 100	Constrained
> 100	Undesirable

Level of Service: Constrained

Figure 3-4. Sample Calculation for Two-Sided Weaving Analysis.

EXIT RAMP-TO-INTERSECTION SPACING

The spacing between an exit ramp and a downstream intersection can have a significant effect on the operations of a weaving section. In an effort to develop recommendations for minimum and desirable spacings, the regression equations developed to predict density were used. Since spacing was a variable in the equations, the equations could be used to back-calculate for spacing given frontage road volume, ramp volume, and percentage of two-sided weaving maneuvers. To estimate minimum spacing, the density value between constrained and undesirable operations (100 veh/hr/ln) was used in the equations. To estimate desirable spacings, the density value between unconstrained and constrained operations (40 veh/km/ln) was used.

Using the density equations to predict minimum and desirable ramp-to-intersection spacings, small spacings (near zero) were computed for low traffic volumes. Therefore, an absolute minimum spacing had to be selected. The 1994 AASHTO *Green Book* (5) states that ramps should connect to the frontage road a minimum of 105 meters upstream of the crossroad. It also states that desirable lengths should be several meters longer to provide adequate weaving length, space for vehicle storage, and turn lanes at the cross road. From the field studies, it was determined that the majority of drivers used between 60 and 120 meters to weave from the exit ramp to the right-most lane when frontage road traffic and/or queues from the downstream intersection did not significantly influence exit ramp driver behavior. In a study by Turner and Messer (6), a rule-of-thumb ramp-to-intersection spacing of 150 meters was recommended. This spacing corresponds to recommendations from the *Green Book* and findings from the field. Therefore, based upon findings from this study and findings from previous research, an absolute minimum exit ramp-to-intersection spacing of 150 meters is recommended. Using this minimum spacing value and the results from the regression equations, Tables 3-5 through 3-7 were generated to estimate minimum and desirable spacings for the three frontage road configurations.

Table 3-5. Minimum and Desirable Ramp-to-Intersection Spacings for Two-Lane Frontage Roads (m).

Exit Ramp Volume (vph)	Exit Ramp Right Turn Percent	Frontage Road Volume (vph)							
		500		1000		1500		2000	
		Min	Desr	Min	Desir	Min	Desir	Min	Desir
250	≤ 50%	150	150	150	150	150	150	150	235
	> 50%	150	150	150	150	150	180	150	305
500	≤ 50%	150	150	150	170	150	295	150	420
	> 50%	150	150	150	240	150	370	150	490
750	≤ 50%	150	235	150	360	150	485	150	610
	> 50%	150	305	150	430	150	555	150	680
1000	≤ 50%	150	420	150	545	150	670	150	795
	> 50%	150	490	150	620	150	740	185	865
1250	≤ 50%	150	610	150	735	175	860	300	985
	> 50%	150	680	150	805	250	930	375	1055

Table 3-6. Minimum and Desirable Ramp-to-Intersection Spacings for Three-Lane Frontage Roads (m).

Exit Ramp Volume (vph)	Exit Ramp Right Turn Percent	Frontage Road Volume (vph)							
		500		1000		1500		2000	
		Min	Desr	Min	Desir	Min	Desir	Min	Desir
250	≤ 50%	150	150	150	175	150	310	150	450
	> 50%	150	175	150	310	150	450	290	585
500	≤ 50%	150	150	150	275	150	410	250	550
	> 50%	150	275	150	410	250	550	390	685
750	≤ 50%	150	235	150	375	210	510	350	650
	> 50%	150	375	210	510	350	650	490	785
1000	≤ 50%	150	335	175	475	310	610	450	750
	> 50%	175	475	310	610	450	750	590	885
1250	≤ 50%	150	445	275	575	410	710	550	850
	> 50%	275	575	410	710	550	850	690	985

Table 3-7. Minimum and Desirable Ramp-to-Intersection Spacings for Two-Lane Frontage Roads with Auxiliary Lane (m).

Exit Ramp Volume (vph)	Exit Ramp Right Turn Percent	Frontage Road Volume (vph) 500 Min	500 Desr	1000 Min	1000 Desir	1500 Min	1500 Desir	2000 Min	2000 Desir
250	≤ 50%	150	150	150	150	150	150	150	150
	> 50%	150	150	150	155	150	230	150	295
500	≤ 50%	150	150	150	150	150	200	150	270
	> 50%	150	215	150	285	150	355	150	425
750	≤ 50%	150	185	150	255	150	325	150	400
	> 50%	150	345	150	415	150	480	150	555
1000	≤ 50%	150	315	150	385	150	455	150	525
	> 50%	150	470	150	540	210	615	280	680
1250	≤ 50%	150	445	150	515	185	585	255	655
	> 50%	200	600	270	670	340	740	410	810

CHAPTER 4

SPACING NEEDS FOR METERED ENTRANCE RAMPS

Ramp metering is a form of entrance ramp control that restricts traffic flow in order to limit the rate at which traffic can enter a freeway. Its primary function is to maintain the freeway's capacity to efficiently serve high-priority urban traffic demands. Figure 4-1 illustrates a typical ramp metering system. Traffic signals are placed on freeway entrance ramps to regulate the ramp traffic. The ramp meter signals and stop bar are placed at a predetermined point on the ramp. Ramp meters minimize congestion on the freeway by maintaining a balance between demand and capacity.

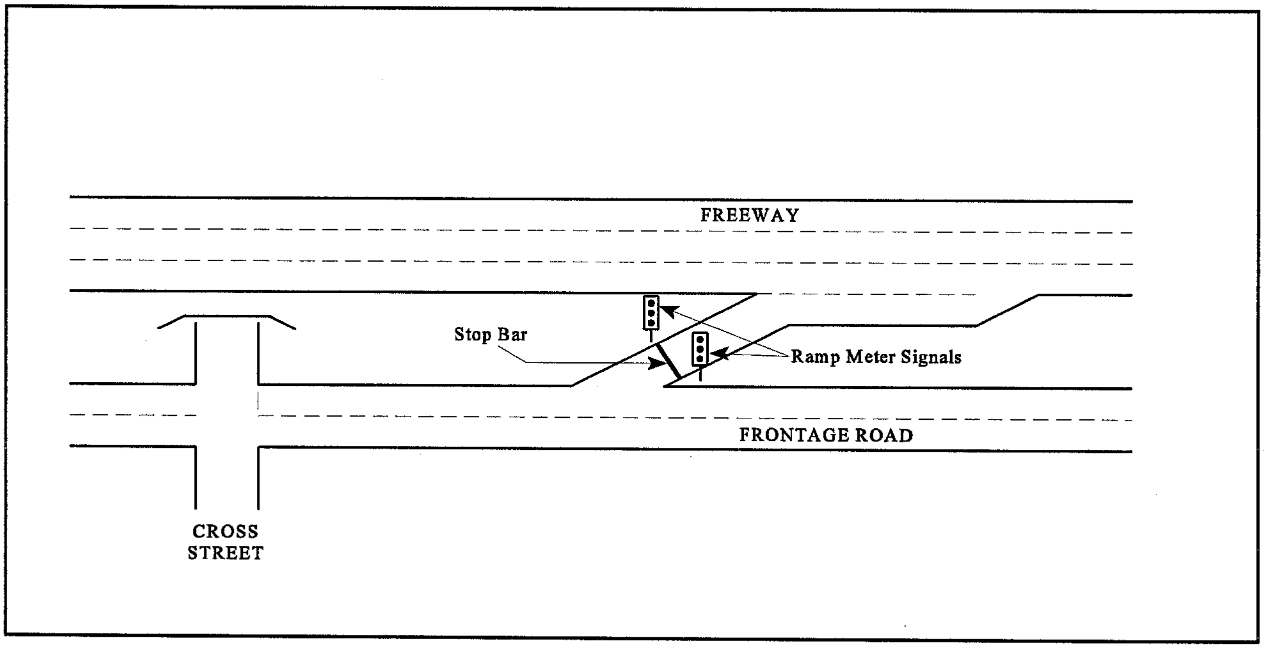


Figure 4-1. Typical Ramp Metering System.

Although ramp metering can control freeway congestion, it may also produce queues that shift congestion to surrounding surface streets. Adequate storage must be provided to assure that the queues of waiting vehicles will not seriously affect non-freeway traffic. Therefore, the spacing between a metered freeway entrance ramp and a signalized cross street intersection is critical for efficient freeway and frontage road operation. If sufficient storage space is not provided on the ramp or on the frontage road, queues formed at metered ramps may back across the cross street, causing congestion and a negative effect on traffic signal operations.

This chapter presents a methodology, developed by Sharma and Messer (7), for determining spacing needs for metered entrance ramps. An example problem using the methodology was developed and is included at the end of this chapter. The example demonstrates how to determine the distances required for ramp metering and the location of the ramp meter signal, how to check the adequacy of a given location, and how to decide upon specific geometric elements.

DETERMINING METERED ENTRANCE RAMP SPACING NEEDS

The queuing section and acceleration and merging (or metering) section are the two components needed to determine spacing requirements for ramp metering (see Figure 4-2). The queuing section is the storage distance needed for vehicles waiting to enter the freeway at the ramp signal. This distance is dependent upon the ramp demand volume and the operating capacity of the ramp metering signal. The metering section is defined as the distance between the ramp signal and the point of merge that allows a vehicle to accelerate to a reasonable merge speed and select a gap.

Sharma and Messer (7) developed a methodology for determining the distance requirements for ramp metering for a wide range of traffic volumes and freeway geometric conditions. A queue storage model was developed to determine distance requirements for queue storage, and the constant acceleration models of linear motion were used to determine the distance required for the freeway merging operation. Following is a discussion of the procedures developed by Sharma and Messer for determining spacing for ramp metering.

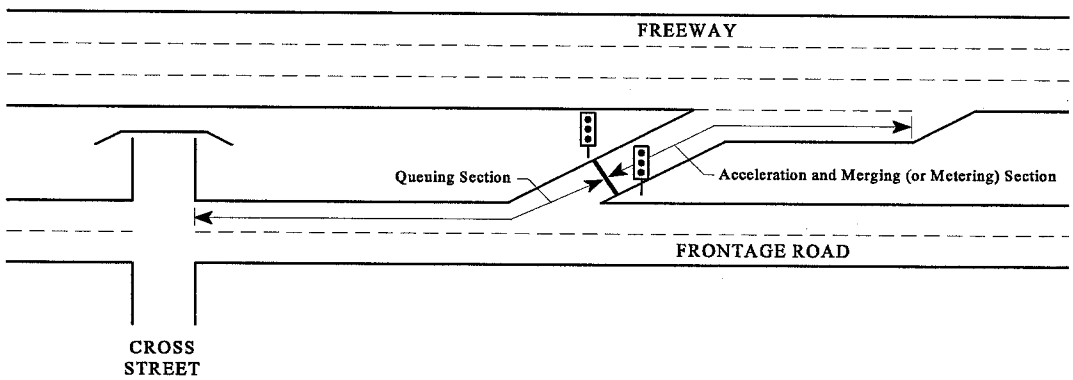


Figure 4-2. Queuing Section and Metering Section.

Queue Storage

The queue storage model relates storage distance to the ramp vehicle arrival rate, the time period under consideration, and the acceptable delay. This model was developed using the following assumptions:

- 95% Poisson arrivals.
- A storage requirement of 7.6 meters per vehicle. This was assumed because it accounts for a normal proportion of trucks in the entrance ramp traffic mix.
- A minimum ramp metering rate of 200 vph. This metering rate cycles a vehicle every 18 seconds, which is believed to be close to the maximum time a driver will wait once the ramp meter signal is reached.
- An analysis time period of four minutes. This four-minute period accounts for approximately two cycle lengths from the upstream traffic signal. (Analysis time periods of two minutes and four minutes were used in the original study because they represent approximate durations of one and two signal cycles of possible demand overload from the upstream intersection, assuming a cycle length of 120 seconds. The example

problem uses four minutes to simulate the more severe situation of two cycle lengths where additional queuing is required.)

- An acceptable delay of one to five minutes for a vehicle in queue. Acceptable ramp delay is the maximum delay for a vehicle in queue which would be accepted by the driver before major ramp signal violations begin to occur. Sharma and Messer state that a ramp delay of more than five minutes is considered unreasonable and can lead to frequent violations of the ramp meter signal.

The queue length model is represented by the following equation:

$$L_Q = \frac{0.122\,(\alpha VT)}{(1 + T/D)} \qquad [4\text{-}1]$$

where:

L_Q = Length of queue, meters
V = Vehicle arrival rate, vph
T = Analysis time period under consideration, min
D = Acceptable ramp delay, min
α = 2, a constant corresponding to 95% Poison arrivals
0.122 = a constant to account for unit conversions and the assumptions previously described

Table 4-1 lists the distance requirements for the queuing section, or the upstream part of a metered entrance ramp. Part of this distance may be accommodated on the frontage road if the left most lane is used exclusively for ramp operation. The values in Table 4-1 are based upon the queue length model. The table provides the queue storage requirements for four-minute analysis time periods for delay values of one to five minutes. Figure 4-3 illustrates the information provided in Table 4-1.

Table 4-1. Distance Requirements for Queue Storage for a Four-Minute Analysis Period (m).

Entrance Ramp Arrival Rate (vph)	Acceptable Delay (min)				
	1	2	3	4	5
200	39	65	84	98	108
300	59	98	125	146	163
400	78	130	167	195	217
500	98	163	209	244	271
600	117	195	251	293	325
700	137	228	293	342	380
800	156	260	335	390	434

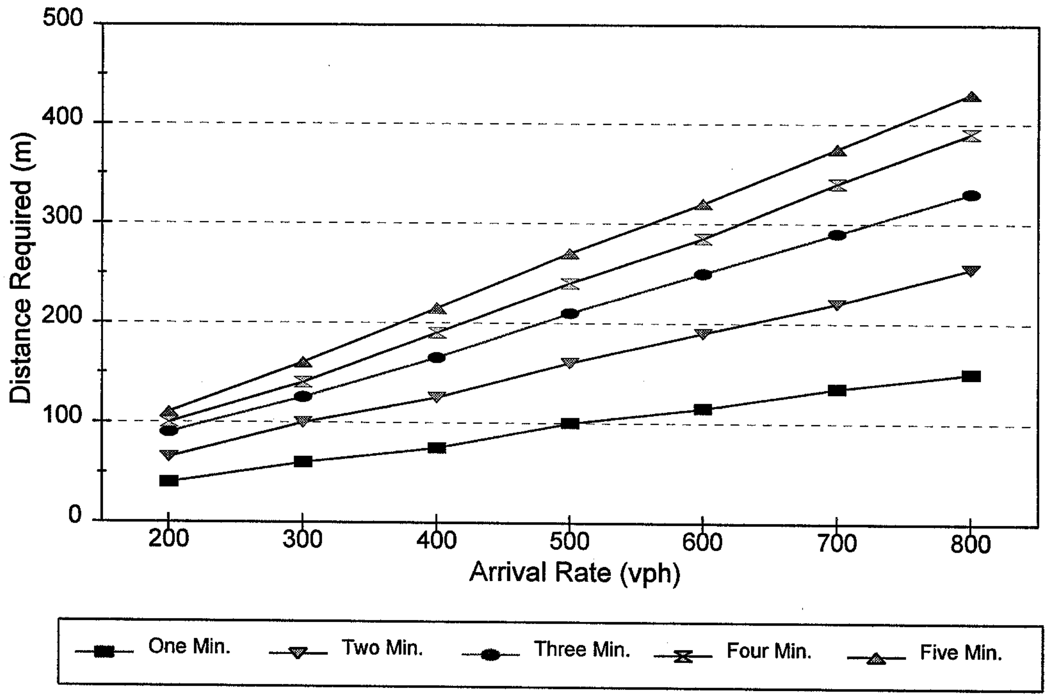


Figure 4-3. Distance Requirements for Queue Storage for a Four-Minute Analysis Period.

Merging Operation

The freeway merging operation includes the distance required to accelerate to freeway speed from the ramp meter stop bar and to find a gap in the freeway traffic stream. An acceleration rate of 3 mpsps assumes uniform acceleration, which is a rapid but usable acceleration for low speeds. A headway of 1.5 seconds over an adjacent freeway vehicle was considered acceptable for the merging operation. Constant acceleration models for linear motion were used to calculate the merging distances required.

Figure 4-4 shows the distance required to achieve freeway speed, the distance required to achieve a 1.5 second headway, and the total merge distance required. Freeway speed is defined as the speed of main lane freeway traffic and is represented on the x-axis. The freeway speeds included in this figure range from 48 to 113 km/h. This range includes the speeds most frequently observed on urban freeways during ramp metering. The distance required to merge is defined as the distance

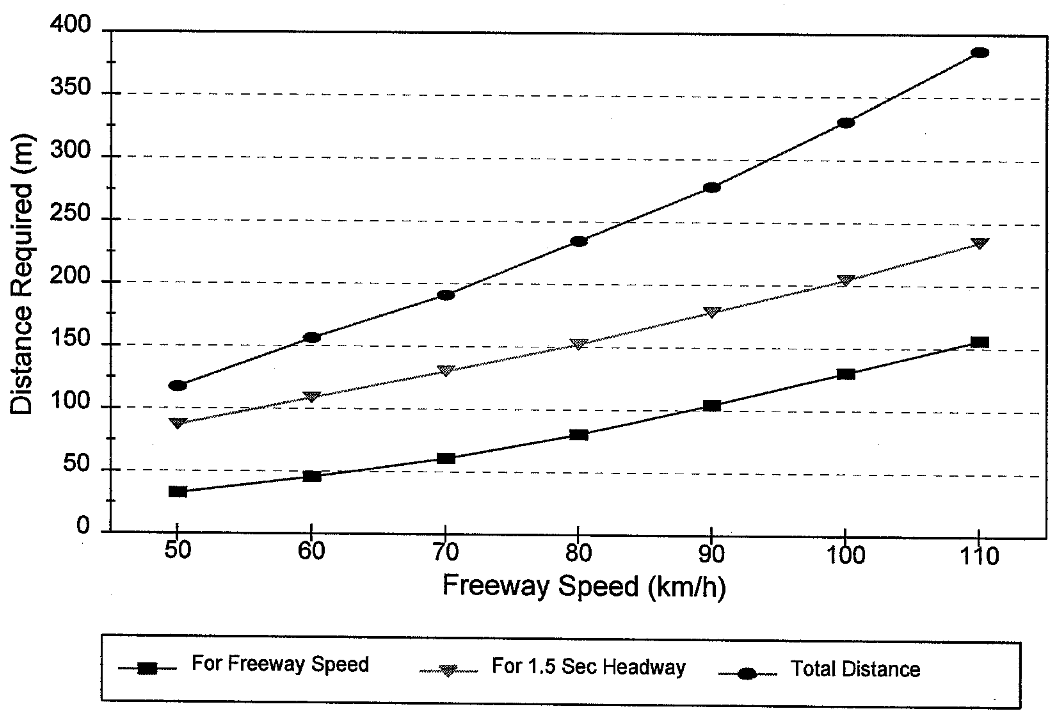


Figure 4-4. Distance Requirements for Freeway Merging Operation.

from the ramp meter stop bar to the final merge point on the freeway and is presented along the y-axis. Therefore, the distance required for a freeway merging operation, from the ramp meter stop bar to the point of merging on the freeway, can be obtained from this figure. Additional distance may be needed for ramps with positive grades due to the additional acceleration time required.

Geometric Considerations

The ramp meter signal location is critical for satisfactory operation of a metered entrance ramp. The ramp meter signal should be located to provide adequate distance downstream of the ramp to achieve a safe freeway merge and to provide adequate distance upstream of the ramp for queue storage. Additionally, more violations of the ramp meter occur when the meter is so close to the freeway that the driver can see the freeway operations.

The location of the ramp meter signal is determined by the geometry of the merge area length requirements and the frontage road separation from the freeway, and it should satisfy both safety and operational needs. Most urban freeway entrance ramps' merges are at an angle of three, four, or five degrees. Also, roadside design safety practice recommends a 9 meter clear zone adjacent to the outside freeway travel lane. The ramp meter signal is presumed to be placed 1.2 meters away from the edge of the entrance ramp travel lane, and this lane is assumed to be 4.9 meters wide. These dimensions are illustrated in Figure 4-5.

The location of the ramp meter signal in terms of the distance from the final merge point must be determined in order for a vehicle to merge safely. This location also effectively defines the signal offset, which is the distance from the edge of the freeway travel lane to the ramp meter signal post nearest to the freeway.

Figure 4-6 relates the ramp signal offset to the ramp distance available downstream of the ramp meter signal to achieve freeway merging. This figure was developed using trigonometric principles to determine the distance available on the entrance ramp, downstream from the ramp meter signal, for a given ramp meter signal offset. Figure 4-7 relates ramp signal offset to

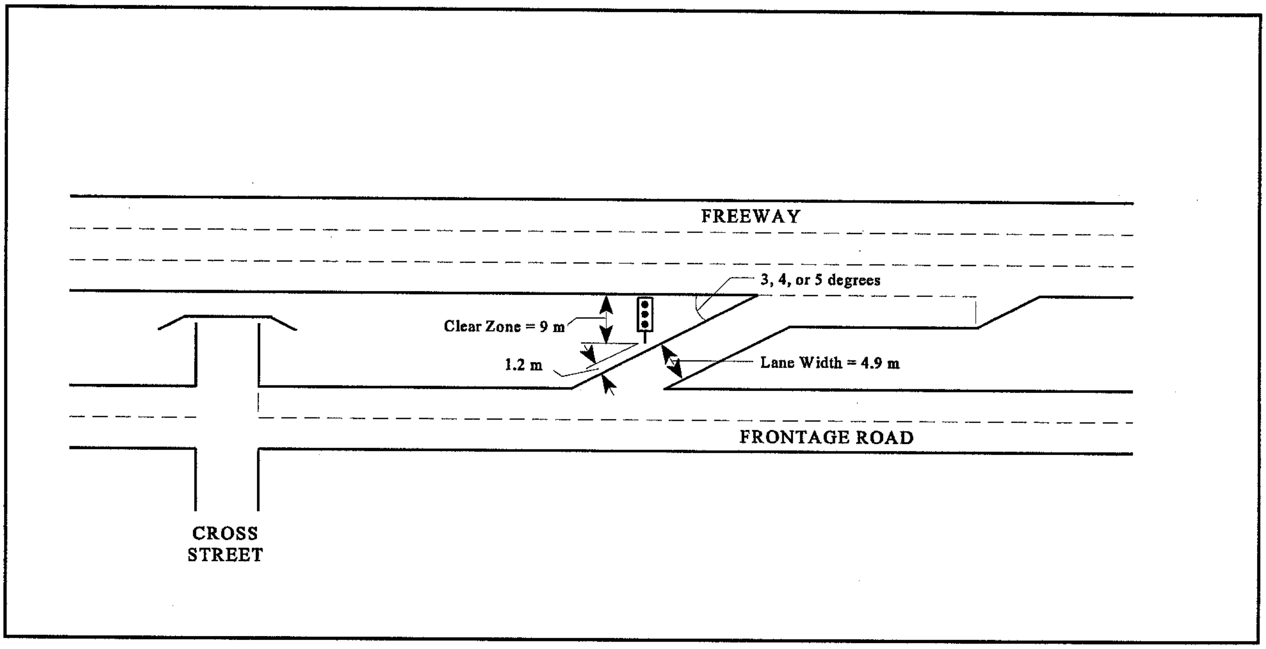


Figure 4-5. Entrance Ramp Dimensions.

maximum speed attainable by the ramp upon discharge at green. This figure was developed using laws of constant linear acceleration to determine the speed a ramp vehicle will be able to reach after leaving the ramp meter signal for a given meter signal offset and a given distance available on the ramp.

The signal offset can be determined from Figure 4-6 when the ramp distance available for merging is known. Also, Figure 4-7 can be used to determine the speed that can be achieved for the specific signal offset. By adding the length of the acceleration lane to the available ramp distance and checking this total available distance with Figure 4-4, it can be determined whether the distance requirements for safe freeway merging are satisfied.

Sharma and Messer also recommend verifying that the ramp meter signal is actually on the entrance ramp and not on the frontage road. If the ramp meter signal is on the frontage road, other problems are involved due to the dual signal heads required and the potential to cause confusion for through frontage road traffic.

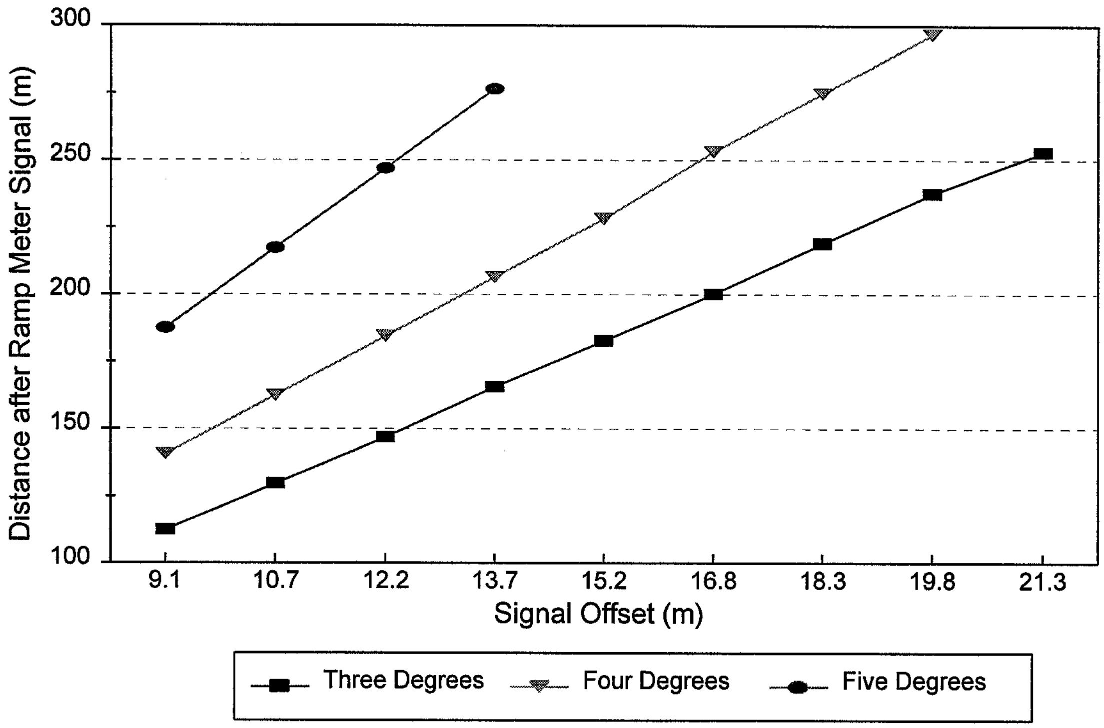


Figure 4-6. Ramp Distance Available for Ramp Signal Offsets.

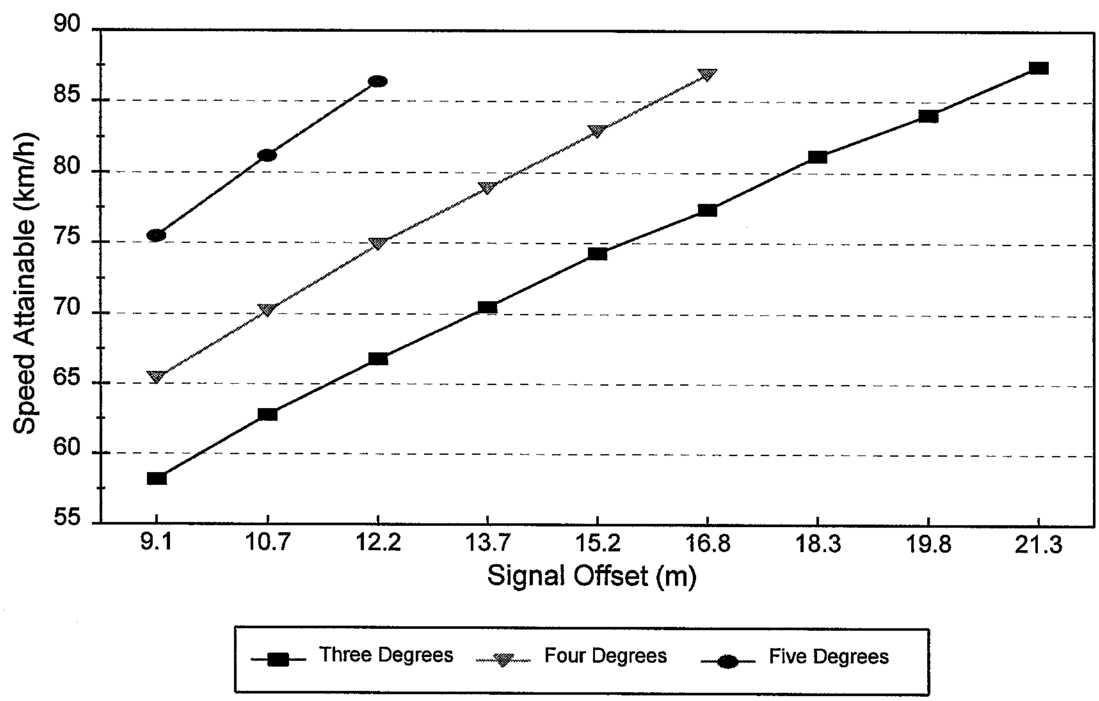


Figure 4-7. Speeds Attainable for Ramp Signal Offsets.

RECOMMENDED PROCEDURE

The work completed by Sharma and Messer provides a methodology for determining queuing and merging sections for entrance ramp metering systems. A step-by-step procedure was developed from the Sharma/Messer method and is presented in the form of an example problem. This procedure is intended to provide engineers and designers with guidelines for the planning, design, and installation of ramp metering systems. Worksheets for completing the procedures are included in Appendix A.

The procedures and methodology presented should be used for new urban entrance ramp designs in order to accommodate metering systems. This method should also be used to evaluate existing entrance ramps where metering systems are currently installed, or are proposed to be installed, to determine the potential need to redesign those ramps with deficient spacings.

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 1

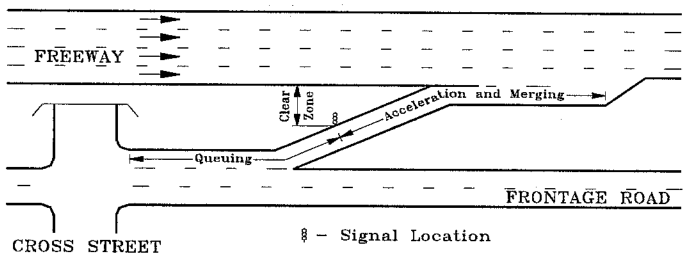
COMMENTS

Site: Example Date: 8/25/95 Time: 4:00 PM

Name: Sally Smith Checked by: K F

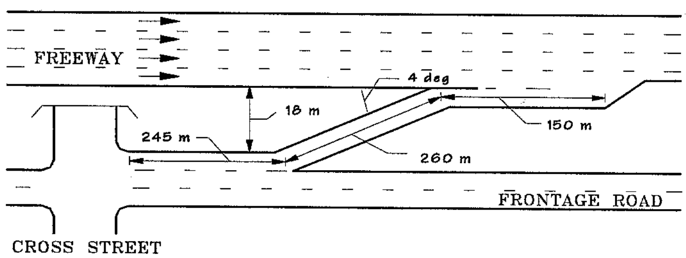
I. DESIRED SOLUTION

The following figure illustrates the design requirements: distance for acceleration and merging, ramp meter signal location and clear zone, and queue storage.



II. GEOMETRIC DATA

Frontage road leaving cross street: 2 ,3 , or 4 lanes: 3
Angle of merge = 3 ,4, or 5 degrees: 4
Separation between outside freeway travel lane and left frontage lane (edge-to-edge) = 18 m
Length of entrance ramp = 260 m
Length of acceleration lane = 150 m
Storage space available between the cross street and the entrance ramp = 245 m



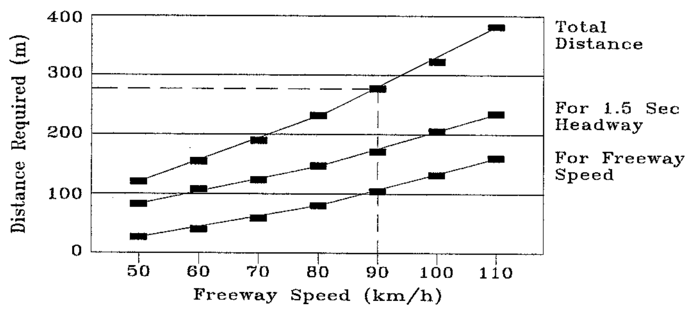
Existing Conditions

III. OPERATIONAL CONDITIONS

Entrance ramp peak hour arrival rate = 650 vph
Freeway speed = 90 km/h
Minimum ramp metering rate = 200 vph

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 2

IV. DETERMINE REQUIRED DISTANCE FOR ACCELERATION AND MERGING



The total distance required for acceleration and merging can be determined from the Total Distance curve on the above figure. The total distance is the ramp distance required for acceleration plus the merging distance required to achieve a 1.5 second headway after reaching freeway speed and before merging with the freeway travel lane.

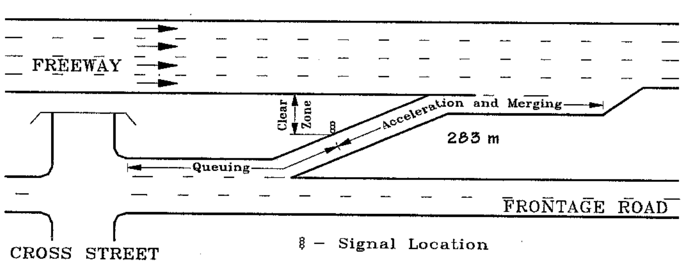
From the above figure, the total acceleration and merging distance required for a freeway speed of _90_ km/h = _283_ m.

The existing length of the freeway acceleration lane = _150_ m.

Therefore, the distance required on the entrance ramp for acceleration and merging is the *total acceleration and merging distance required* minus the *existing length of the freeway acceleration lane*:

_283_ m - _150_ m = _133_ m required on the ramp.

This distance is used to locate the signal; however, the clear-zone distance needs to be checked (see Part V).



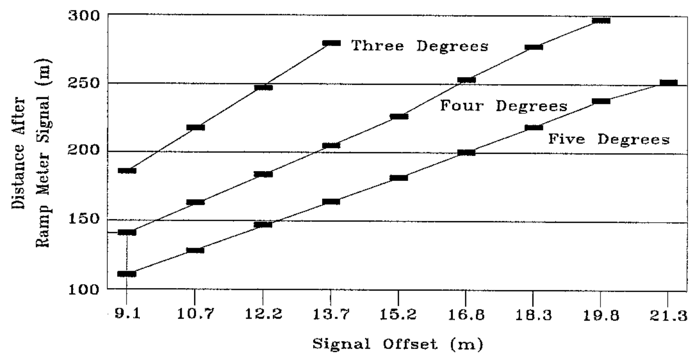
Total Distance Provided for Acceleration and Merging

COMMENTS

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 3

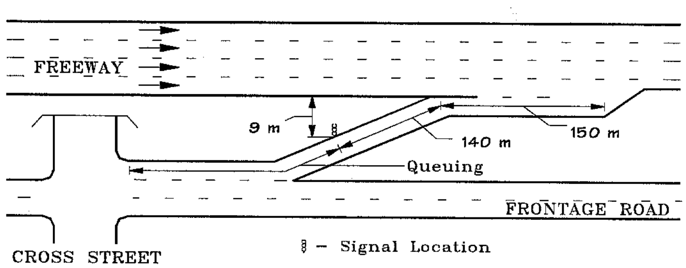
V. CHECK CLEAR ZONE

Part IV determined that a minimum of 133 m of the ramp is needed for acceleration and merging purposes. The following figure gives the minimum acceleration distance after the ramp meter signal for various signal offsets (or clear-zone distances) and for 3, 4, and 5 degree angles of merge between the ramp and freeway. The minimum desirable clear zone is 9 m, which is the distance from the outside edge of the freeway travel lane to the ramp meter signal.



For a distance of 133 m after the ramp meter signal, a clear zone of < 9 m is provided for a 4 degree merge angle. A 9 m clear zone for a 4 degree merge results in 140 m after the ramp meter signal.

Engineering judgement must be used to determine if the clear zone and the distance for accelerations and merging are adequate. If not, the ramp meter may need to be shifted to another location. **It should be verified that the ramp meter signal is on the entrance ramp and not on the frontage road.**



Clear Zone and Signal Location

COMMENTS

133 m for acceleration on the ramp provides < 9 m for a clear zone.
∴ Use 140 m on the ramp for acceleration and merging. This provides a 9 m clear zone.

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 4

VI. DETERMINE QUEUE STORAGE REQUIREMENT

The portion of the ramp not used for acceleration and merging is available for queue storage:

260 m (ramp length)

\- 140 m (portion used for acceleration and merging)

= 120 m (portion available for queue storage)

Determine the queue storage length required for an arrival rate of 650 vph and a 5 minute delay from the table below.

Entrance Ramp Arrival Rate (vph)	Acceptable Delay (min)				
	1	2	3	4	5
200	39	65	84	98	108
300	59	98	125	146	163
400	78	130	167	195	217
500	98	163	209	244	271
600	117	195	251	293	325
700	137	228	293	342	380
800	156	260	335	390	434

Required queue storage length = 352 m

Determine the available queue storage length:

245 m (on the frontage road)
\+ 120 m (on the ramp)
= 365 m (available queue storage length)

If the required queue storage length is less than the queue storage length available, the design is good.

If the provided distance is less than the required distance, some compromise between the queue storage distance and the roadside safety clear zone requirement may be made depending upon the judgement of the engineer.

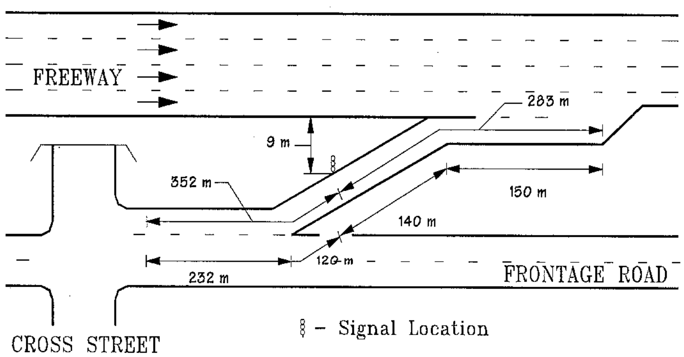
COMMENTS

Since 140 m of the entrance ramp is being used for acceleration and merging, this leaves 120 m available for queuing.

352 m is an interpolation between 325 and 380 for arrival rates of 600 and 700 vph.

The required length is less than the available length. Therefore, the design is good.

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 5

VII. SOLUTION	COMMENTS
The following sketch illustrates the solution to the design problem. 	

VIII. NOTES

If the entrance ramp is on a positive slope, additional distance may be required for acceleration.

CHAPTER 5

LEVEL-OF-SERVICE ANALYSIS PROCEDURE

OPERATIONS APPLICATION

The procedure for determining frontage road level of service has been divided into seven steps (see Figure 5-1). The procedure listed in Figure 5-1 applies to both one-way and two-way frontage roads. The analysis of two-way frontage roads differs from one-way frontage roads in the following areas: data requirements, computation of running time, and computation of delay at ramp junctions. In addition, the analysis procedure should be followed twice for two-way frontage roads (once for each direction).

The level-of-service criteria are based on average travel speed. Average travel speed is computed by dividing the length of the frontage road by the total travel time. The total travel time may be estimated either by using the procedure outlined in this chapter or by measuring it directly in the field. The following sections give descriptions of the steps for predicting the level of service for frontage road operations.

Step 1: Define Frontage Road Study Section

The first step in analyzing frontage road operations is to determine the location of the frontage road to be analyzed. The analyst must then choose the length of frontage road to include in the analysis. The frontage road section being analyzed may be bound by intersections controlled by signals or stop signs, or it may begin or end at any point, such as a freeway ramp.

After the frontage road boundaries have been defined, the frontage road study *section* should be divided into segments. Each *segment* should contain similar frontage road and traffic operational characteristics (i.e., traffic volume, speed limit, roadside development, etc.). Segments are typically bound by signalized intersections but may include any combination of links. A *link* is defined by

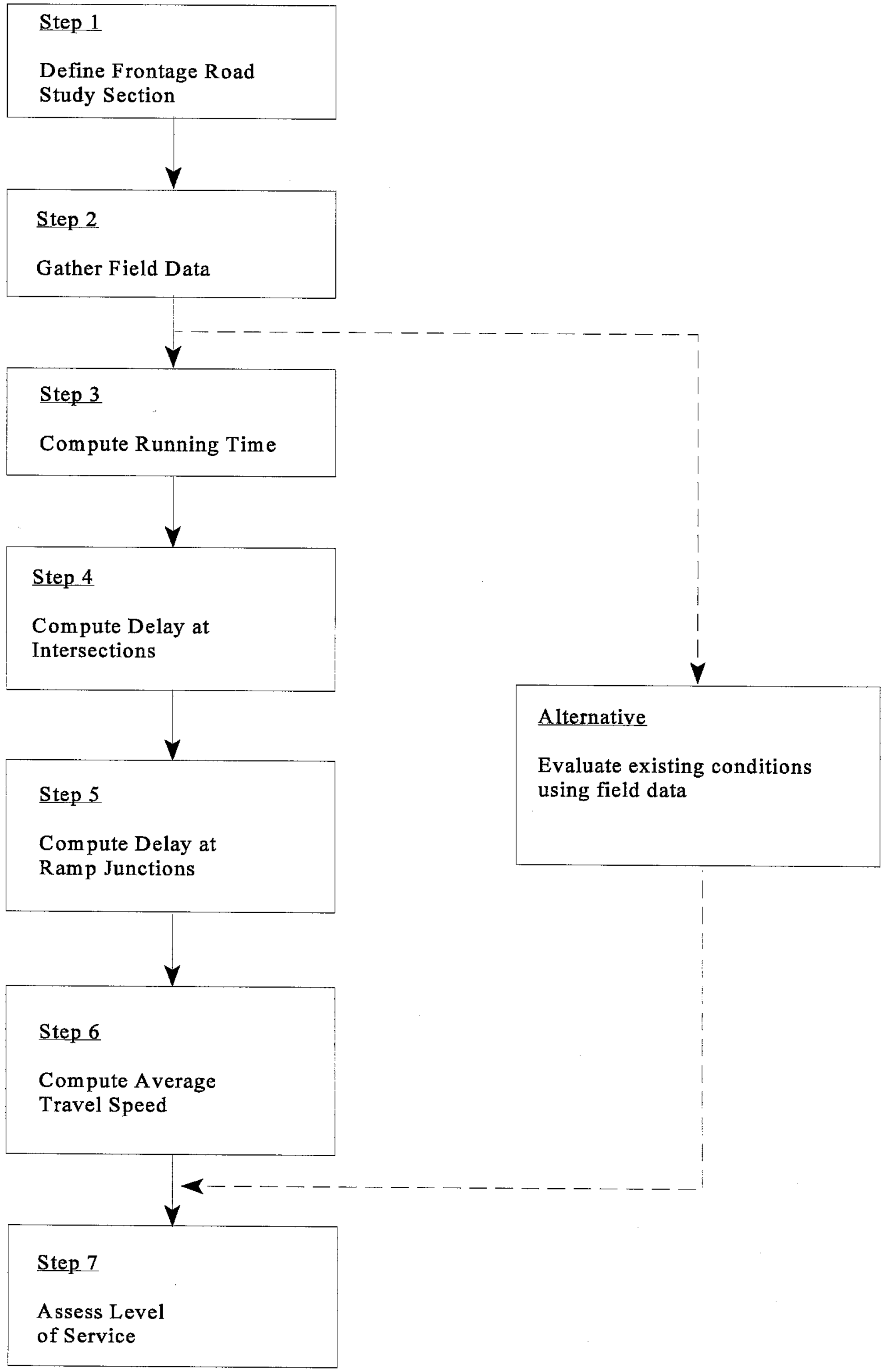


Figure 5-1. Level-of-Service Analysis Procedure.

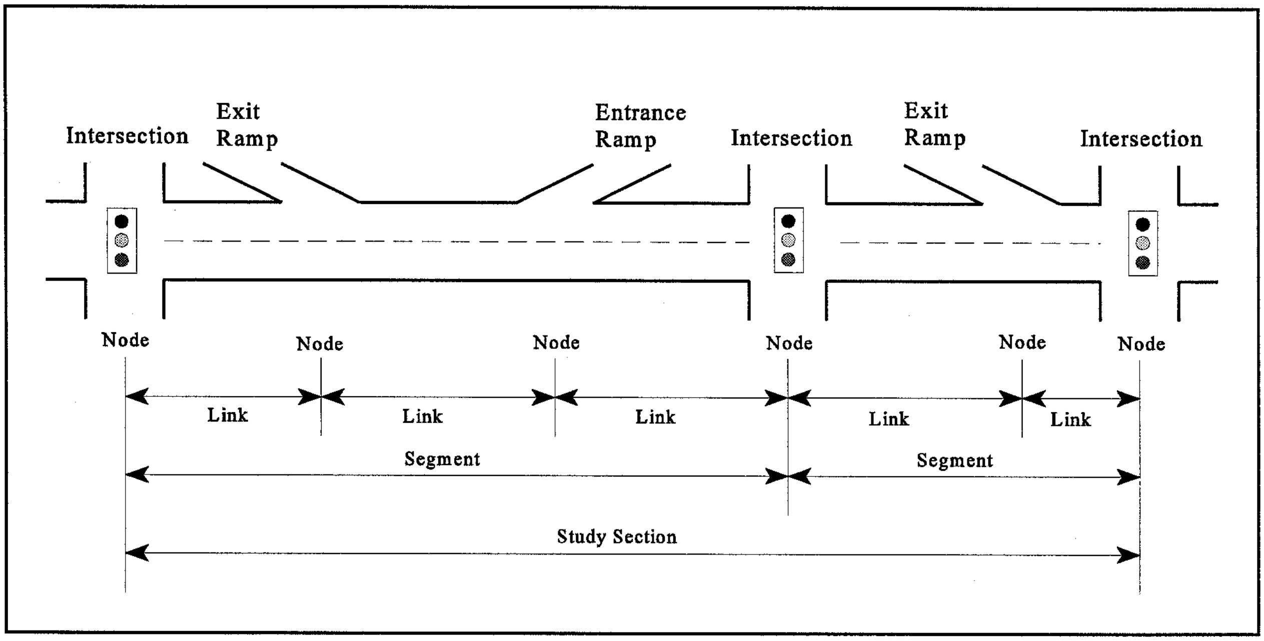


Figure 5-2. Terminology Used to Describe Frontage Roads.

its beginning and ending *nodes* (e.g., exit ramp, entrance ramp, signalized intersection, etc.). Figure 5-2 illustrates the use of the terms *node, link, segment, and study section.*

Step 2: Gather Field Data

This step involves gathering the data (e.g., roadway characteristics, traffic data, and signal data) required to perform the analysis. As mentioned earlier, total travel time may either be measured directly in the field or may be computed using the procedure in this chapter. Table 5-1 summarizes the required data for computing the total travel time for one-way and two-way frontage roads.

Step 3: Compute Running Time

The total frontage road travel time includes the running time, delay at intersections, and delay at freeway ramp junctions. The running time is the time is takes a vehicle to traverse a given section of roadway without being delayed by intersections or ramps. A procedure for estimating running

Table 5-1. Data Required for Analyzing Frontage Road Operations.

Type of Data	Data Required	Frontage Road	
		One-Way	Two-Way
Roadway Characteristics	Segment length, km	✔	✔
	Type of traffic control at intersections (e.g., no-control, stop-controlled, or traffic signal)	✔	✔
	Number of all exit and entrance ramps		✔
	Number of exit ramps without auxiliary lanes	✔	
	Segment access density, acs/km (number of driveways and unsignalized intersections per kilometer)	✔	✔
Traffic Data	Frontage road approach volume at stop-controlled and signalized intersections, vph	✔	✔
	Ramp and frontage road volumes at all exit and entrance ramps, vph		✔
	Exit ramp and frontage road volumes at exit ramps without auxiliary lanes, vph	✔	
Signal Data	Signal progression data	✔	✔
	Intersection capacity (c), vph	✔	✔
	Cycle length (C), sec	✔	✔
	Green/cycle time ratio (g/C)	✔	✔
	Volume/capacity ratio (v/c)	✔	✔

time was developed by collecting travel time data at existing frontage road sites. Regression analyses showed that length significantly affected travel time. Other factors, such as volume and free flow speed, had minor effects on travel time when compared to length.

Results from the regression analyses were used to develop equations to predict running time for both one-way and two-way frontage roads. Table 5-2 shows these regression equations.

Table 5-2. Equations for Predicting Running Time on Frontage Roads.

Frontage Road	Regression Equation[a]
One-Way	RT = 0.0504 (L)
Two-Way	RT = 0.0519 (L)

[a] RT = running time (sec)
L = segment length (m)

For two-way frontage roads, plots of average speed versus frontage road volume revealed some correlation between speed and volume. For frontage road volumes above approximately 400 vphpl, maximum speeds begin to drop noticeably (and travel times increase). Below 400 vphpl, maximum speeds of 89 to 97 km/h were observed while above 400 vphpl, most speeds were below 72 km/h. Travel times were predicted to increase by as much as 10 percent for frontage road volumes above 400 vphpl.

The analyses also showed that access density had an effect on travel time. For both one-way and two-way frontage roads, a critical value of access density existed at which speeds began to drop and travel times increase significantly. The critical values for one-way and two-way frontage roads occurred at approximately 20 and 16 acs/km, respectively. Above these critical values, travel times may again increase by as much as 10 percent.

Table 5-3 contains estimated running times for one-way and two-way frontage roads. The segments lengths included in the field data ranged from approximately 0.2 to 2.0 km for one-way and 0.2 to 3.2 km for two-way; therefore, these ranges are included in the table. If the frontage road segment lengths being evaluated fall outside of this range, the analyst should consider redefining the segments. The travel times shown in Table 5-3 are increased by 10 percent when access

Table 5-3. Running Time for One-Way and Two-Way Frontage Road Segments.

	One-Way Frontage Roads		Two-Way Frontage Roads			
Access Density (acs/km)	≤ 20	> 20	≤ 16		> 16	
Frontage Road Volume (vphpl)	All	All	≤ 400	> 400	≤ 400	> 400
Segment Length[a] (km)	Running Time, RT[b] (sec)					
0.2	10	11	10	11	11	13
0.4	20	22	21	23	23	25
0.6	30	33	31	34	34	38
0.8	40	44	42	46	46	50
1.0	50	55	52	57	57	63
1.2	60	67	62	69	69	75
1.4	71	78	73	80	80	88
1.6	81	89	83	91	91	100
1.8	91	100	93	103	103	113
2.0	101	111	104	114	114	126
2.2	N/A	N/A	114	126	126	138
2.4	N/A	N/A	125	137	137	151
2.6	N/A	N/A	135	148	148	163
2.8	N/A	N/A	145	160	160	176
3.0	N/A	N/A	156	171	171	188
3.2	N/A	N/A	166	183	183	201

[a] If segment length falls outside of 0.2 to 2.0 km for one-way and 0.2 to 3.2 km for two-way, consider redefining segments.

[b] Equations used to determine values are listed in Table 5-2.

density exceeds 20 acs/km for one-way frontage roads and exceeds 16 acs/km for two-way frontage roads. The travel times are again increased by 10 percent for two-way frontage roads when frontage road volumes exceed 400 vphpl.

Step 4: Compute Intersection Delay

For most frontage roads, intersections at major crossroads will be controlled either by a traffic signal or by stop signs. To estimate the approach delay at signalized intersections, the procedures outlined in Chapter 9 of the *HCM* are recommended. Chapter 10 of the *HCM* includes procedures for estimating approach total delay for two-way and all-way stop-controlled intersections. Updated procedures in Chapter 10 is expected to be available in late 1997. Following is a summary of the procedures in Chapter 9 of the *HCM* for calculating approach delay at signalized intersections.

Estimating Delay at Signalized Intersections

The total delay incurred at a signalized intersection includes the time that a vehicle is stopped (defined as stopped delay), as well as the time to decelerate from and accelerate to the driver's desired speed. The 1994 *HCM* defines intersection total delay as a function of stopped delay using the following equation:

$$D_I = 1.3 * d \qquad [5\text{-}1]$$

where:

D_I =intersection total delay, sec/veh

d = intersection stopped delay, sec/veh

Intersection stopped delay is calculated using the following equations:

$$d = d_1 DF + d_2 \qquad [5\text{-}2]$$

$$d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]} \qquad [5\text{-}3]$$

$$d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}] \qquad [5\text{-}4]$$

where:

d = stopped delay, sec/veh

d_1 = uniform delay, sec/veh

d_2 = incremental delay, sec/veh

DF = delay adjustment factor for either quality of progression or type of control (see Table 5-5)

X = volume/capacity ratio of lane group

C = cycle length, sec

c = capacity of lane group, vph

g = effective green time for lane group, sec

m = incremental delay calibration term representing effect of arrival type and degree of platooning (see Table 5-4)

The total delay incurred at signalized intersections will be based upon the arrival type. The arrival type is a function of the quality of progression. Table 5-4 lists the six arrival types defined in the *HCM.* The incremental delay calibration term (m) is a function of the arrival type and is also shown in this table.

The delay adjustment factor (DF) accounts for the effects of signal progression and controller type on uniform delay. To estimate the value of this factor, either the controller-type adjustment factor (CF) or the progression adjustment factor (PF) is used. Table 5-5 shows values of DF recommended in the *HCM.*

Table 5-4. Arrival Type and Incremental Delay Calibration Term (m) Values.

Arrival Type	Progression Quality	Incremental Delay Calibration Term, m
1	Very poor	8
2	Unfavorable	12
3	Random arrivals	16
4	Favorable	12
5	Highly favorable	8
6	Exceptional	4

Table 5-5. Uniform Delay Adjustment Factor (DF).

Controller-Type Adjustment Factor, CF		
Control Type	Non-Coordinated Intersections	Coordinated Intersections
Pretimed	1.00	PF as computed below
Semiactuated		
Traffic-actuated lane groups	0.85	1.00
Non-actuated lane groups	0.85	PF as computed below
Fully actuated	0.85	N/A

Progression Adjustment Factor, PF						
Green/Cycle Time Ratio, g/C	Arrival Type					
	1	2	3	4	5	6
0.20	1.167	1.007	1.000	1.000	0.833	0.750
0.30	1.286	1.063	1.000	0.986	0.714	0.571
0.40	1.445	1.136	1.000	0.895	0.555	0.333
0.50	1.667	1.240	1.000	0.767	0.333	0.000
0.60	2.001	1.395	1.000	0.576	0.000	0.000
0.70	2.556	1.653	1.000	0.256	0.000	0.000

Equations 5-1 through 5-4 should be used to compute total delay at all signalized intersections within the study section. Chapter 9 of the *HCM* contains complete descriptions of the variables used in the equations and further discussion on computing intersection delay.

Intersection Level of Service

The *HCM* defines intersection level of service in terms of average stopped delay per vehicle. Stopped delay may be computed using Equation 5-2. Table 5-6 shows level-of-service criteria for signalized intersections suggested in the *HCM.*

Table 5-6. Signalized Intersection Level-of-Service Criteria.

Intersection Level of Service	Stopped Delay per Vehicle (sec)
A	≤ 5.0
B	5.1 to 15.0
C	15.1 to 25.0
D	25.1 to 40.0
E	40.1 to 60.0
F	> 60.0

Step 5: Compute Ramp Delay

Delay incurred by frontage road vehicles at freeway ramps is more of a concern for two-way frontage roads than for one-way frontage roads. For two-way frontage roads, vehicles traveling in the same direction as freeway traffic will be required to yield only at exit ramps; however, vehicles traveling in the opposite direction will be required to yield at both exit ramps and entrance ramps. For one-way frontage roads, frontage road delay at ramps is typically only experienced at exit ramps that do not have auxiliary lanes or in those cities where all drivers on the frontage road consistently yield to exit ramp vehicles.

In a study conducted by Gattis et al. (8), procedures for predicting delay at ramps were developed. The recommended equations for predicting delay at ramps on one-way and two-way frontage roads are listed in Table 5-7.

As shown in Table 5-7, three values are calculated to estimate frontage road delay: frontage road capacity at ramp (C_R), average queuing system delay (W), and average total delay (D_R). These models were developed by assuming that the ramp-frontage road intersection area operates as a queuing system. Because of this assumption, the equations can only be used when the frontage road flow rate (a) does not exceed the service rate (u) (i.e., $u - a \geq 0$).

The resulting equations for predicting frontage road delay at ramps are expressed as a function of ramp volume and frontage road volume. Therefore, these are the only parameters that need to be obtained for estimating delay at ramps. For entrance ramp *opposing* delay on two-way frontage roads, the ramp volume should include all frontage road vehicles approaching the entrance ramp from the *with* direction, whether the vehicles actually enter the ramp or continue along the frontage road.

The equations in Table 5-7 were developed by assuming that ramp traffic arrivals could be described using the Poisson process and by estimating the gap acceptance tendencies of frontage road traffic. Actual delays at field sites may vary from the predicted delay depending upon the average accepted gap of frontage road drivers.

An evaluation of the equations for predicting frontage road delay at exit ramps on one-way frontage roads revealed a limitation of the equations for predicting frontage road capacity (C_R). Capacity is calculated from these equations by multiplying a factor by the ramp volume and subtracting this product from the maximum frontage road flow rate (i.e., maximum flow rate - factor x ramp volume). When the ramp volume multiplied by the factor exceeds the maximum flow rate, a negative capacity value results. Maximum ramp volumes for which the capacity equations produce positive values are shown in Table 5-8. Using the capacity equations for ramp volumes above those in this table will produce invalid results.

Table 5-7. Equations for Predicting Frontage Road Delay at Ramps.

Case	Frontage Road	Scenario	Frontage Road Capacity, C_R (veh/hr)	Queuing Delay, W (sec/veh)	Total Delay, D_R (sec/veh)
1	One-Way	Exit Ramp without Auxiliary Lane	$N[1858-1.5259(Q_R)]$	1/(u-a)	-0.0719 + 1.0922(W)
2	Two-Way	Exit Ramp *With*	$1724 - 1.6120(Q_R)$	1/(u-a)	-0.0719 + 1.0922(W)
3	Two-Way	Exit Ramp *Opposing*	$1444 - 1.6564(Q_R)$	1/(u-a)	-1.6451 + 1.7785(W)
4	Two-Way	Entrance Ramp *Opposing*	$1535 - 1.3852(Q_R)$	1/(u-a)	0.0538 + 1.3027(W)

Note:

- N = number of frontage road through lanes
- C_R = frontage road capacity per direction, vph
- W = average queuing system delay, sec/veh
- D_R = average total delay, sec/veh
- Q_R = hourly ramp volume, vph (for Case 4, includes all vehicles that approach the entrance ramp from the *with* direction, whether they enter the ramp or not)
- u = service rate (C / 3600), veh/sec
- a = frontage road flow rate (volume / 3600), veh/sec

Table 5-8. Maximum Ramp Volumes to Be Used with Capacity Equations.

Case	Frontage Road	Scenario	Maximum Ramp Volume (vph)
1	One-Way	Exit Ramp	1200
2	Two-Way	Exit Ramp *With*	1050
3	Two-Way	Exit Ramp *Opposing*	850
4	Two-Way	Entrance Ramp *Opposing*	1100

Currently, techniques are not available to predict delays at high-volume ramps or at ramp junctions on one-way frontage roads where all lanes of traffic consistently yield to exiting ramp vehicles. A potential solution to determine delay at these types of ramp junctions is the revision to

HCM, Chapter 10 ("Unsignalized Intersections"), which will be included in the next revision of the *HCM.* Until available, engineering judgement should be used if a frontage road segment includes these types of ramp junctions.

Step 6: Compute Average Travel Speed

The average travel speed can be computed by dividing the total length of the frontage road under consideration by the total travel time. The total travel time is composed of the total running time, total delay at intersections, and total delay at ramps. The average travel speed may be computed using the following formula:

$$S = \frac{3,600(L)}{RT + D_I + D_R} \qquad [5\text{-}5]$$

where:

- S = average travel speed, km/h
- L = length of frontage road, km
- RT = total running time, sec
- D_I = total approach delay for all signalized and stop-controlled intersections, sec
- D_R = total frontage road delay incurred at ramps, sec

Step 7: Assess Level of Service

Once the average travel speed has been computed, the level of service can be estimated using the criteria in Table 5-9. These criteria apply to both one-way and two-way frontage road operations. The criteria are not meant to represent exact divisions in level of service. The values are intended to provide a general idea of the level of service that might be expected for a particular frontage road section; therefore, engineering judgement should be used when applying these criteria.

Table 5-9. Frontage Road Level-of-Service Criteria.

Frontage Road Level of Service	Average Travel Speed (km/h)
A	≥ 56.0
B	45.0 to 55.9
C	35.0 to 44.9
D	27.0 to 34.9
E	21.0 to 26.9
F	< 21.0

Alternative Evaluation

An alternative to calculating average travel speed using the above procedure is to make travel time measurements directly in the field. Collecting field data is a more direct approach to evaluating existing frontage road operations and will produce more accurate results. An example would be to measure the total time to travel through a selected study site at various times during a peak period. After obtaining an average frontage road travel time, the travel speed would be computed by dividing the length of the study site by the average travel time. The average travel speed would then be compared to the criteria in Table 5-9 to assess the level of service.

PLANNING APPLICATION

The *HCM* planning level procedure for an arterial street level-of-service analysis can essentially be used for a similar analysis of frontage roads. The major simplifying assumption in the arterial street planning application is that left turns are accommodated by providing left-turn bays at major intersections and controlling the left-turn movement with a separate phase that is properly timed. As a result of this assumption, planning application results should not be used for intersection design or traffic operations analyses. Another assumption needed for a frontage road planning level of service is that ramp junctions do not significantly contribute to the delay along the frontage road (i.e., that all exit ramps on one-way frontage roads have auxiliary lanes). For two-way frontage roads, estimates of delay at ramp junctions need to be added. Example Calculation 3 provides an example of a planning application for a one-way frontage road.

EXAMPLE CALCULATION 1—COMPUTATION OF FRONTAGE ROAD LEVEL OF SERVICE, ONE-WAY FRONTAGE ROAD

Step 1: Define Frontage Road Study Section

The frontage road to be considered is a 3.9 km length of a two-lane, one-way frontage road in an area of moderate development. Figure 5-3 illustrates the frontage road section to be analyzed. Each of the crossroad intersections shown are controlled by traffic signals.

The selected frontage road study section is divided into the following three segments (with each segment being bound by signalized intersections): Lemon to Georgia, Georgia to 39th, and 39th to University.

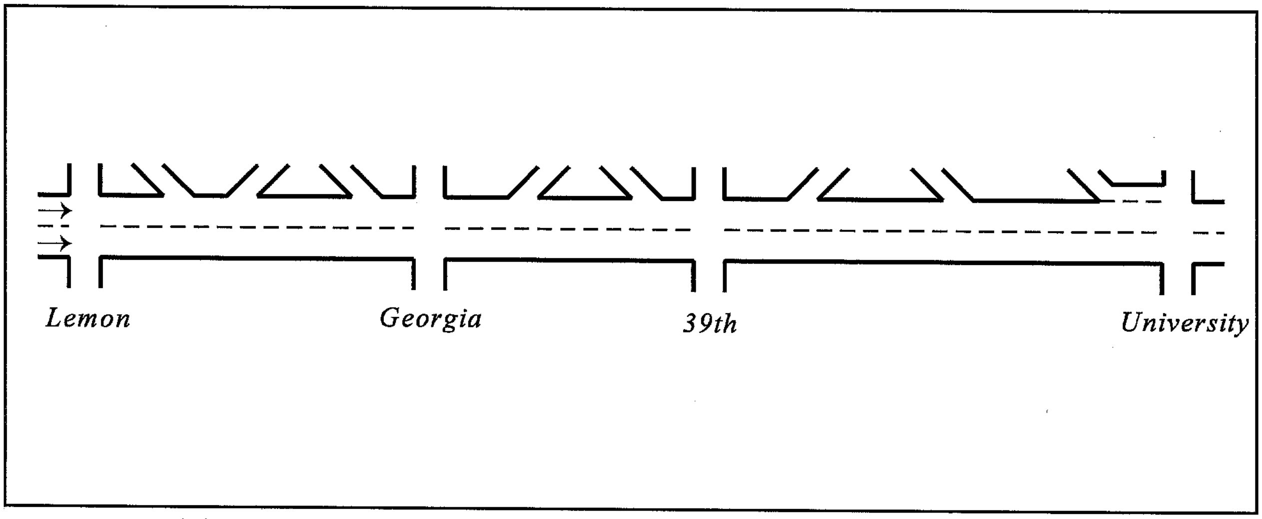


Figure 5-3. Schematic of One-Way Frontage Road Study Section.

Step 2: Gather Field Data

The required field data include roadway characteristics, traffic data, and signal data (see Table 5-1). Assumptions include random arrival and a saturation flow rate of 1800 vphpl. Tables 5-10 and 5-11 summarize collected field data.

Table 5-10. Roadway Characteristics and Traffic Data for One-Way Frontage Road Study Section.

Seg-ment	Segment Boundaries	Length (km)	Access Density (acs/km)	Number of Exit Ramps w/o Aux. Lanes	Exit Ramp Volume (vph)	Frontage Road Volume (vph)	
						At Exit Ramps	At Inter-sections
1	Lemon to Georgia	1.2	21.2	2	Exit 1: 358 Exit 2: 180	Exit 1: 193 Exit 2: 97	282
2	Georgia to 39th	1.1	18.2	1	214	115	372
3	39th to University	1.6	16.2	1	98	53	261

Table 5-11. Signal Data for One-Way Frontage Road Study Section.

Intersection	Cycle Length, C (sec)	Green/Cycle Time Ratio, g / C	Intersection Capacity, c[a] (vph)
Georgia	120	0.25	900
45th	100	0.34	1224
Western	75	0.26	936

[a] c = (Saturation flow rate)(# of lanes)(g/C)

Step 3: Compute Running Time

The segment lengths and access densities are entered on the Frontage Road Level-of-Service Worksheet (see Figure 5-4). Running times are obtained from Table 5-3.

Step 4: Compute Intersection Delay

Intersection delay is computed on the Signalized Intersection Delay Worksheet (see Figure 5-5). The first step is to enter cycle length (C), green/cycle time ratio (g/C), v/c ratio (X), capacity (c), and arrival type onto the worksheet. Arrival type is based on quality of progression and is estimated using the values in Table 5-4. Arrival Type 3 is selected because the vehicles are assumed to be random arrivals.

The next step is to compute the total delay (D_I) for each signalized intersection. Intersection total delay is computed using equations 5-1 through 5-4. Intersection level of service is based on stopped delay (d) and may be estimated using the criteria in Table 5-6. Intersection total delay is then entered on the Frontage Road Level-of-Service Worksheet.

Step 5: Compute Ramp Delay

Ramp delay is computed using the Ramp Junction Delay Worksheet (One-Way Frontage Roads). For one-way frontage roads, ramp delays are calculated for exit ramps without auxiliary lanes only. Segment 1 has two exit ramps without auxiliary lanes, and Segments 2 and 3 each have one exit ramp without an auxiliary lane. Delay for each ramp is calculated on a separate line of the worksheet (See Figure 5-6). Total ramp delay for each segment is entered in the "Ramp Delay" column on the Frontage Road Level-of-Service Worksheet.

FRONTAGE ROAD LEVEL-OF-SERVICE WORKSHEET		
Location: IH-99	Direction: North - bound	
Description: Between Lemon and University	Type: One-Way	
Date: 8-19-96	Prepared By: Sally	

Segment	Segment Length (km) L	Access Density (acs/km)	Running Time[a] (sec) RT	Intersection Total Delay[b] (sec) D_I	Ramp Delay[c] (sec) D_R	Total Travel Time[d] (sec) T	Average Travel Speed[e] (km/h) S	Frontage Road LOS by Segment[f]
1	1.2	21.2	67					
2	1.1	18.2	55					
3	1.6	16.2	81					

[a] Use field data or values from Table 5-3
[b] From Signalized Intersection Delay Worksheet
[c] From Ramp Junction Delay Worksheet
[d] $T = RT + D_I + D_R$
[e] $S = 3600(L)/T$
[f] See LOS criteria in Table 5-9.

Sum of Travel Times, sec (ΣT) = ______

Total Frontage Road Length, km (ΣL) = ______

Average Frontage Road Speed, km/h = 3600 (ΣL) / (ΣT) = ______

Frontage Road LOS = ______

Figure 5-4. Compute Running Time.

SIGNALIZED INTERSECTION DELAY WORKSHEET

Location: IH-99 Direction: North - bound

Description: Between Lemon and University Type: One-Way

Date: 8-19-96 Prepared By: Sally

Seg-ment	Cycle Length (sec) C	Green/ Cycle Time Ratio g/C	v/c Ratio X	Lane Group Capacity (vph) c	Arrival Type [a]	Uniform Delay[b] (sec) d_1	DF[c]	Incre-mental Delay[d] (sec) d_2	Inter-section Stopped Delay[e] (sec) d	Inter-section Total Delay[f] (sec) D_I	Inter-section LOS[g]
1	120	0.25	0.316	900	3	27.9	1.0	0.1	28.0	36.4	D
2	100	0.34	0.304	1224	3	18.5	1.0	0.0	18.5	24.1	C
3	75	0.26	0.279	936	3	16.8	1.0	0.0	16.8	21.9	C

[a] Table 5-4

[b] Equation 5-3 $d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]}$

[c] Table 5-5

[d] Equation 5-4 $d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}]$

[e] Equation 5-2 $d = d_1 DF + d_2$

[f] Equation 5-1 $D_I = 1.3 * d$

[g] Table 5-6

Figure 5-5. Compute Intersection Delay.

RAMP JUNCTION DELAY WORKSHEET
(ONE-WAY FRONTAGE ROADS)

Location: IH-99 Direction: North - bound

Description: Between Lemon and University Type: One-Way

Date: 8-19-96 Prepared By: Sally

Segment	Exit Ramp Hourly Volume[a] (veh/hr) Q_R	Frontage Road Hourly Volume (veh/hr) a	Potential Capacity of Frontage Road Lanes[b] (veh/hr) C_R	Queuing System Delay per Vehicle[c] (sec) W	Predicted Total Delay per Vehicle[d] (sec) D_R
1	358	193	2623	1.5	1.6
1	180	97	3167	1.2	1.2
2	214	115	3063	1.2	1.3
3	98	53	3418	1.1	1.1

[a] Q_R must be ≤ 1200; otherwise, use engineering judgement. If an auxiliary lane is present, delay is negligible.
[b] C_R = # Lanes $(1858 - 1.5259\,(Q_R))$
[c] $W = 3600 / (C_R - a)$
[d] $D_R = -0.0719 + 1.0922\,(W)$

Figure 5-6. Calculate Ramp Delay.

Step 6: Compute Average Travel Speed

To calculate the average travel speed, the total travel time for each segment must be computed. The total travel time is the sum of the running time, intersection total delay, and ramp delay. Frontage road travel speed is calculated by dividing the total length of the frontage road study section by the total travel time (see Equation 5-5). This information is entered on the Frontage Road Level-of-Service Worksheet (see Figure 5-7).

Step 7: Assess Level of Service

The frontage road speeds for each segment are now compared to the criteria in Table 5-9 to determine the level of service by segment. The overall frontage road level of service is estimated by computing the average travel speed for the frontage road. As shown in Figure 5-7, the average travel speed for the frontage road is 48.3 km/h resulting in a LOS B.

FRONTAGE ROAD LEVEL-OF-SERVICE WORKSHEET

Location: IH-99 Direction: North - bound

Description: Between Lemon and University Type: One-Way

Date: 8-19-96 Prepared By: Sally

Segment	Segment Length (km) L	Access Density (acs/km)	Running Time[a] (sec) RT	Intersection Total Delay[b] (sec) D_I	Ramp Delay[c] (sec) D_R	Total Travel Time[d] (sec) T	Average Travel Speed[e] (km/h) S	Frontage Road LOS by Segment[f]
1	1.2	21.2	67	36.4	2.8	106.2	40.7	C
2	1.1	18.2	55	24.1	1.3	80.4	49.3	B
3	1.6	16.2	81	21.9	1.1	104.0	55.4	B

[a] Use field data or values from Table 5-3
[b] From Signalized Intersection Delay Worksheet
[c] From Ramp Junction Delay Worksheet
[d] $T = RT + D_I + D_R$
[e] $S = 3600(L)/T$
[f] See LOS criteria in Table 5-9.

Sum of Travel Times, sec (ΣT) = 290.6

Total Frontage Road Length, km (ΣL) = 3.9

Average Frontage Road Speed, km/h = 3600 (ΣL) / (ΣT) = 48.3

Frontage Road LOS = B

Figure 5-7. Assess Level of Service.

EXAMPLE CALCULATION 2—COMPUTATION OF FRONTAGE ROAD LEVEL OF SERVICE, TWO-WAY FRONTAGE ROAD

Step 1: Define Frontage Road Study Section

The frontage road to be considered is a 3.1 km length of two-lane, two-way frontage that is located in an area of low to moderate development. **This example illustrates the procedure to determine the level of service for the frontage road lane that flows *with* the direction of the freeway traffic. However, the lane *opposing* freeway traffic should also be analyzed because the level of service may be different.** Figure 5-8 illustrates the frontage road length to be analyzed.

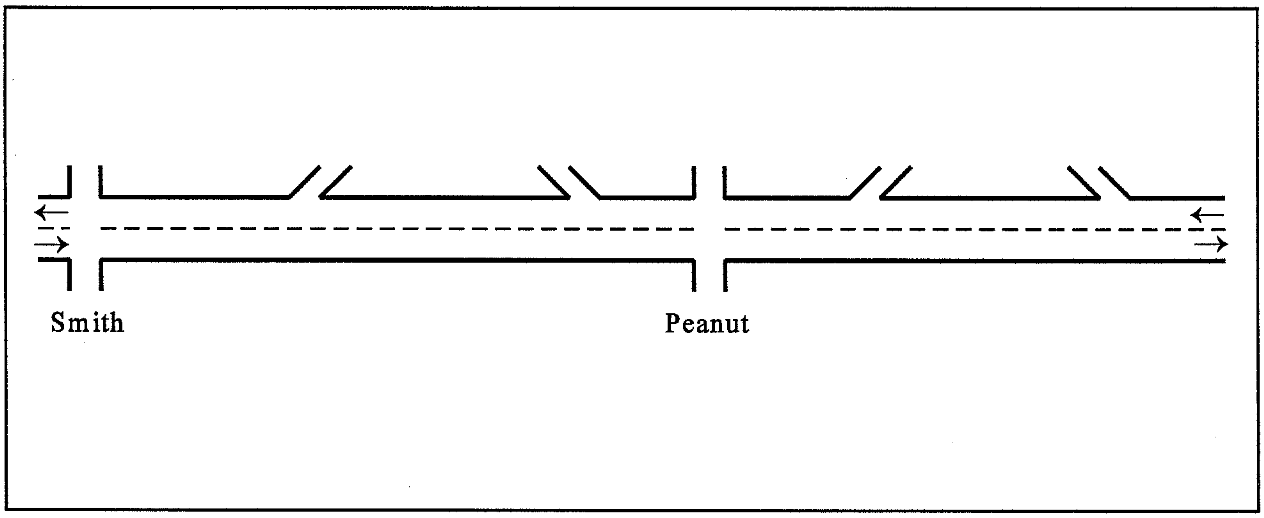


Figure 5-8. Schematic of Two-Way Frontage Road Study Section.

The selected frontage road study section is divided into the following two segments: Smith to Peanut, and Peanut to Exit Ramp.

Step 2: Gather Field Data

The required field data include roadway characteristics, traffic data, and signal data (see Table 5-1). The saturation flow rate is assumed to be 1800 vphgpl. Tables 5-12 and 5-13 summarize the required field data.

Table 5-12. Roadway Characteristics and Traffic Data for Two-Way Frontage Road Study Section.

Segment	Segment Boundaries	Length (km)	Access Density (acs/km)	Exit Ramp Volume (vph)	Frontage Road Volume (vph)	
					At Exit Ramps	At Intersections
1	Smith to Peanut	1.8	7.3	264	84	348
2	Peanut to Exit Ramp	1.3	15.9	204	96	--

Table 5-13. Signal Data for Two-Way Frontage Road Study Section.

Intersection	Cycle Length, C (sec)	g / C	Intersection Capacity, c[a] (vph)
Peanut	170	0.20	360

[a] c = (saturation flow rate)(# of lanes)(g/C)

Step 3: Compute Running Time

The segment lengths and access densities are entered on the Frontage Road Level-of-Service Worksheet (see Figure 5-9). Running times are computed from Table 5-3.

FRONTAGE ROAD LEVEL-OF-SERVICE WORKSHEET

Location: IH-50 Direction: North (With) - bound

Description: Smith to Exit Ramp Past Peanut Type: Two-Way

Date: 8-19-96 Prepared By: Sally

Seg-ment	Segment Length (km) L	Access Density (acs/km)	Running Time[a] (sec) RT	Inter-section Total Delay[b] (sec) D_I	Ramp Delay[c] (sec) D_R	Total Travel Time[d] (sec) T	Average Travel Speed[e] (km/h) S	Frontage Road LOS by Segment[f]
1	1.8	7.3	93					
2	1.3	15.9	68					

[a] Use field data or values from Table 5-3
[b] From Signalized Intersection Delay Worksheet
[c] From Ramp Junction Delay Worksheet
[d] $T = RT + D_I + D_R$
[e] $S = 3600(L)/T$
[f] See LOS criteria in Table 5-9.

Sum of Travel Times, sec (ΣT) = __________

Total Frontage Road Length, km (ΣL) = __________

Average Frontage Road Speed, km/h = 3600 (ΣL) / (ΣT) = __________

Frontage Road LOS = __________

Figure 5-9. Compute Running Time.

Step 4: Compute Intersection Delay

Intersection delay is computed on the Signalized Intersection Delay Worksheet (see Figure 5-10). The first step is to enter cycle length (C), green/cycle time ratio (g/C), v/c ratio (X), capacity (c), and arrival type onto the worksheet. Arrival type is based on quality of progression and is estimated using the values in Table 5-4. Arrival Type 3 is assumed.

The next step is to compute the total delay (D_I) for each signalized intersection. The total delay is computed using Equations 5-1 through 5-4. Intersection level of service is based on stopped delay (d) and may be estimated using the criteria in Table 5-6. The intersection total delay (D_I) is then entered on the Frontage Road Level-of-Service Worksheet.

Step 5: Compute Ramp Delay

Ramp delay is computed using the Ramp Junction Delay Worksheet (Two-Way Frontage Roads). For two-way frontage road lanes flowing *with* the frontage road traffic, ramp delays are calculated for exit ramps only (i.e., exit ramp *with*). Segments 1 and 2 each have one exit ramp. Delay for each ramp is calculated on a separate line of the worksheet (see Figure 5-11). Delay at each ramp is entered in the "Ramp Delay" column on the Frontage Road Level-of-Service Worksheet.

Step 6: Compute Average Travel Speed

To calculate the average travel speed, the total travel time for each segment must be computed. The total travel time is the sum of the running time, intersection total delay, and ramp delay. Frontage road travel speed is calculated by dividing the total length of the frontage road study section by the total travel time (see Equation 5-5). This information is entered on the Frontage Road Level-of-Service Worksheet (see Figure 5-12).

SIGNALIZED INTERSECTION DELAY WORKSHEET

Location: IH-50 Direction: North (With) - bound

Description: Smith to Exit Ramp Past Peanut Type: Two-Way

Date: 8-19-96 Prepared By: Sally

Seg-ment	Cycle Length (sec) C	Green/ Cycle Time Ratio g/C	v/c Ratio X	Lane Group Capacity (vph) c	Arrival Type [a]	Uniform Delay[b] (sec) d_1	DF[c]	Incre-mental Delay[d] (sec) d_2	Inter-section Stopped Delay[e] (sec) d	Inter-section Total Delay[f] (sec) D_I	Inter-section LOS[g]
1	170	0.20	0.233	360	3	43.7	1.0	0.0	43.7	56.9	E

[a] Table 5-4

[b] Equation 5-3 $d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]}$

[c] Table 5-5

[d] Equation 5-4 $d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}]$

[e] Equation 5-2 $d = d_1 DF + d_2$

[f] Equation 5-1 $D_I = 1.3 * d$

[g] Table 5-6

Figure 5-10. Compute Intersection Delay.

RAMP JUNCTION DELAY WORKSHEET
(TWO-WAY FRONTAGE ROADS)

Location: IH-50 Direction: North (With) - bound

Description: Smith to Exit Ramp Past Peanut Type: Two-Way

Date: 8-19-96 Prepared By: Sally

Segment	Scenario[a]	Ramp Hourly Volume (vph) Q_R	Frontage Road Hourly Volume (vph) a	Potential Capacity of Frontage Road (vph) C_R	Queuing System Delay per Vehicle (sec) W	Predicted Total Delay per Vehicle (sec) D_R
1	Exit Ramp With	264	84	1298	2.96	3.2
2	Exit Ramp With	204	96	1395	2.77	3.0

[a] Scenarios and Equations:

Exit Ramp *With*:
$C_R = 1724 - 1.6120\ (Q_R)$
$W = 3600 / (C_R - a)$
$D_R = -0.0719 + 1.0922\ (W)$

Exit Ramp *Opposing*:
$C_R = 1444 - 1.6564\ (Q_R)$
$W = 3600 / (C_R - a)$
$D_R = -1.6451 + 1.7785\ (W)$

Entrance Ramp *Opposing*:
$C_R = 1535 - 1.3852\ (Q_R)$ (Note: Q_R is assumed to be total frontage road *with* volume)
$W = 3600 / (C_R - a)$
$D_R = 0.0538 + 1.3027\ (W)$

Figure 5-11. Calculate Ramp Delay.

FRONTAGE ROAD LEVEL-OF-SERVICE WORKSHEET

Location: IH-50 Direction: North (With) - bound

Description: Smith to Exit Ramp Past Peanut Type: Two-Way

Date: 8-19-96 Prepared By: Sally

Segment	Segment Length (km) L	Access Density (acs/km)	Running Time[a] (sec) RT	Intersection Total Delay[b] (sec) D_I	Ramp Delay[c] (sec) D_R	Total Travel Time[d] (sec) T	Average Travel Speed[e] (km/h) S	Frontage Road LOS by Segment[f]
1	1.8	7.3	93	56.9	3.2	153.2	42.3	
2	1.3	15.9	68	0.0	3.0	71.0	65.9	

[a] Use field data or values from Table 5-3
[b] From Signalized Intersection Delay Worksheet
[c] From Ramp Junction Delay Worksheet
[d] $T = RT + D_I + D_R$
[e] $S = 3600(L)/T$
[f] See LOS criteria in Table 5-9.

Sum of Travel Time, sec (ΣT) = ______________

Total Frontage Road Length, km (ΣL) = ______________

Average Frontage Road Speed, km/h = 3600 (ΣL) / (ΣT) = ______________

Frontage Road LOS = ______________

Figure 5-12. Compute Average Travel Speed.

Step 7: Assess Level of Service

The frontage road speeds for each segment are now compared to the criteria in Table 5-9 to determine the level of service by segment. The overall frontage road level of service is estimated by computing the average travel speed for the frontage road. As shown in Figure 5-13, the average travel speed for the frontage road is 49.8 km/h resulting in a LOS B.

FRONTAGE ROAD LEVEL-OF-SERVICE WORKSHEET

Location: IH-50 Direction: North (With) - bound

Description: Smith to Exit Ramp Past Peanut Type: Two-Way

Date: 8-19-96 Prepared By: Sally

Seg-ment	Segment Length (km) L	Access Density (acs/km)	Running Time[a] (sec) RT	Inter-section Total Delay[b] (sec) D_I	Ramp Delay[c] (sec) D_R	Total Travel Time[d] (sec) T	Average Travel Speed[e] (km/h) S	Frontage Road LOS by Segment[f]
1	1.8	7.3	93	56.9	3.2	153.2	42.3	C
2	1.3	15.9	68	0.0	3.0	71.0	65.9	A

[a] Use field data or values from Table 5-3
[b] From Signalized Intersection Delay Worksheet
[c] From Ramp Junction Delay Worksheet
[d] $T = RT + D_I + D_R$
[e] $S = 3600(L)/T$
[f] See LOS criteria in Table 5-9.

Sum of Travel Times, sec (ΣT) = 224.1

Total Frontage Road Length, km (ΣL) = 3.1

Average Frontage Road Speed, km/h = 3600 (ΣL) / (ΣT) = 49.8

Frontage Road LOS = B

Figure 5-13. Assess Level of Service.

EXAMPLE CALCULATION 3—PLANNING APPLICATION

Description

The following information has been determined for a one-way frontage road section.

- Traffic Characteristics

  Annual average daily traffic, for both directions (AADT) = 30,000

  Planning analysis peak hour factor (K100) = 0.09

  Directional distribution factor, for northbound direction (D) = 0.55

  Peak hour factor (PHF) = 0.925

  Adjusted saturation flow = 1,850 pcphgpl

  Percentage of turns from exclusive lanes = 15

- Roadway Characteristics

  Through lanes = 2 lanes per direction

  Section length = 3.2 km

  Left-turn bays = yes

  Access density is less than 20 acs/km

- Signal Characteristics

  Signalized intersections = 4 (thus, average segment length = 0.8 km)

  Arrival type = 3 (random arrival)

  Signal types = non-coordinated, semiactuated

  Cycle length (C) = 120 sec

  Weighted effective green ratio (g/C) = 0.45

Solution

Use the following steps to determine the level of service for the northbound direction.

Step 1. Determine the two-way hourly volume for the planning analysis hour.

$$\begin{aligned}\text{Two-Way Hourly Volume} &= \text{AADT} \times \text{K}\\ &= 30{,}000 \times 0.09\\ &= 2{,}700 \text{ vph}\end{aligned}$$

Step 2. Determine the hourly directional volume based on the predominant directional flow.

$$\begin{aligned}\text{Directional Volume} &= \text{Two-Way Hourly Volume} \times \text{D}\\ &= 2{,}700 \times 0.55\\ &= 1{,}485 \text{ vph}\end{aligned}$$

Step 3. Determine the basic through-volume 15-minute flow rate.

$$\begin{aligned}\text{Flow Rate} &= (\text{Directional Volume / PHF}) \times (1 - \text{percentage of turns})\\ &= (1{,}485 / 0.925) \times (1 - 0.15)\\ &= 1{,}365 \text{ vph}\end{aligned}$$

Step 4. Determine running time.

The running time rate is obtained from Table 5-3 using one-way frontage road columns, less than 20 acs/km, and a segment length of 0.8 km. A running time of 40 sec per 0.8 km is obtained. For the 3.25 km segment, the running time is 162.5 seconds.

Step 5. Calculate total intersection delay.

The total delay (D) for all intersections is obtained using Equations 5-1 through 5-4. Following are the calculations performed to determine D.

Lane group capacity (c) = Saturation flow rate × number of lines × g/C

= 1,850 × 2 × 0.45

= 1,665

v/c ratio (X) = flow rate / lane group capacity

= 1,365 / 1,665

= 0.82

$$d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]} \quad [5\text{-}3]$$

$$d_1 = \frac{0.38 \times 120 \times [1-(0.45)]^2}{1-(0.45)[0.82]}$$

$d_1 = 21.9$ sec

From Table 5-4, $m = 16$ for arrival type 3. From Table 5-5, $DF = 0.85$ for non-coordinated, semiactuated signals.

$$d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}] \quad [5\text{-}4]$$

$$d_2 = 173(0.82)^2[(0.82-1)+\sqrt{(0.82-1)^2+(16)(0.82)/1554}]$$

$d_2 = 2.6$ sec

Determine intersection stopped delay (d).

$$d = d_1 \times \mathrm{DF} + d_2 \qquad [5\text{-}2]$$

$$d = 21.9 \times 0.85 + 2.6$$

$$d = 21.2 \text{ sec}$$

Determine intersection total delay (D_I) for all intersections (number of signalized intersections on this section is 4).

$$D_I = 1.3 \times d \qquad [5\text{-}1]$$

$$D_I = (1.3 \times 21.2) \times 4$$

$$D_I = 110 \text{ sec}$$

Step 6. Determine average travel speed using Equation 5-5.

$$S = \frac{3{,}600(L)}{RT + D_I + D_R} \qquad [5\text{-}5]$$

$$S = \frac{3{,}600(3.2)}{162.5 + 110 + 0.0}$$

$$S = 42.3 \text{ km/h}$$

Step 7. Determine the level of service for the section.

Based on an average travel speed of 42.3 km/h and the criteria in Table 5-9, the frontage road level of service is “C.”

CHAPTER 6

FINDINGS AND RECOMMENDATIONS

The objectives of this project were to develop procedures to analyze freeway frontage road operations and to determine desirable spacings for ramp junctions. Several notable findings were identified during the research. They are presented below. Additional research needs were also identified and are presented below.

FINDINGS

One-Sided Weaving

- By calculating the weaving volume (exit ramp volume + entrance ramp volume) for a one-sided weaving segment, the level of service can be estimated based on the following criteria: unconstrained (weaving volume < 1500 vph), constrained (weaving volume from 1500 - 3000 vph), and undesirable (weaving volume > 3000 vph).

- For one-sided weaving segments, it is desirable to have a weaving length greater than 300 meters. If this is not achievable, the minimum weaving length should be 200 meters.

Two-Sided Weaving

- By calculating the density for a two-sided weaving segment, the level of service can be estimated based on the following criteria: unconstrained (density < 40 veh/km/ln), constrained (density from 40 - 100 veh/km/ln), and undesirable (density > 100 veh/km/ln).

- Results from the field study revealed that the majority of drivers observed used from approximately 60 to 120 meters to weave from the exit ramp to the right-most lane on the frontage road.

- In addition, the field study showed that queues from the downstream intersection began to have significant effects on drivers making a two-sided weaving maneuver when the queue length was within approximately 90 meters of the exit ramp.

- Based upon findings from this study and findings from previous research, an absolute minimum exit ramp-to-intersection spacing of 150 meters is recommended.

Spacing Needs for Metered Entrance Ramp

- The procedures developed by Sharma and Messer (7) can be used in conjunction with the worksheets developed in this study to determine optimum spacings between intersections and metered entrance ramps.

Continuous Frontage Road Sections

- Signalized intersections have the greatest impact on the operations along a frontage road.

- For two-way frontage roads, ramp junctions also have a significant impact on operations.

- Link length has the greatest impact on travel time between signalized intersections or ramp junctions.

- The running times between signalized intersections measured at 29 frontage road sites closely matched the running times presented in the *HCM* Table 11-4. Users of the frontage road level-of-service procedure can use either the running times calculated with the *HCM* table or the refined values from the regression equations developed as part of this research.

- Access density (i.e., the number of driveways and unsignalized intersections per km) noticeably affects the operations along a frontage road segment when greater than 20 acs/km on one-way frontage roads and 16 acs/km on two-way frontage roads.

- The models developed by Gattis et al. (8) for predicting delay at ramp junctions are appropriate when used within their acknowledged limitation range.

- For the two-way frontage road sites, volume affects operations when it exceeds approximately 400 vphpl.

RECOMMENDATIONS FOR ADDITIONAL RESEARCH

Additional research is needed in the following areas:

One-Sided Weaving

- The NETSIM model used in the study predicted a relatively high percentage of frontage road-to-entrance ramp vehicles weaving from the right-most lane when compared with the field observations. In NETSIM, the frontage road vehicles wanting to access the entrance ramp did not begin the required weaving maneuvers until they reached the weaving link. According to field observations, many of the frontage road vehicles desiring to access the entrance ramp began making the required weaving maneuvers before reaching the weaving link. Therefore, improvements are recommended for NETSIM so that weaving vehicles may begin the required maneuvers before reaching the weaving link.

- Further research is recommended on one-way frontage operations between exit ramps and entrance ramps. The research should focus on lane configurations differing from that addressed in this report. Configurations identified for future study include the following: exit ramp followed by an entrance ramp with no auxiliary lane, and exit ramp followed by an entrance ramp with a lane addition beginning at the exit ramp and terminating at the downstream intersection.

Two-Sided Weaving Between an Exit Ramp and an Intersection

- The level-of-service criteria developed in this study did not take into account the effects of turn bays. Turn bays can relieve congestion resulting in less density and improved level of service. Further research should be conducted to determine the effects of turn bays on two-sided weaving operations.

- Two-sided weaving operations were analyzed in this study assuming that the cross street traffic at the intersection was moderate and the traffic signal was optimally timed to minimize overall intersection delay. Frontage road operations can be significantly impacted by poor signal timing, especially when volumes are high. Therefore, further research should be conducted in which a range of signal timings are included in the analysis of two-sided weaving operations.

Continuous Frontage Road Sections

- The equations currently used in the frontage road level-of-service evaluation to determine delay at ramps produced values similar to those observed in the field, except at high volume locations. Additional research is needed to determine the delay incurred at these high volume ramps and to develop a technique to estimate that value.

- In some locations, traffic on all frontage road lanes stops at the ramp junction. This research only examined the more common situation of the inner one or two lanes yielding to the ramp traffic. The effects on frontage road operations of having all traffic yield need to be investigated.

REFERENCES

1. *Highway Capacity Manual.* Special Report No. 209. Transportation Research Board, National Research Council, Washington, D.C. 1994.

2. Fitzpatrick, K. and R.L. Nowlin. *One-Sided Weaving Analysis on One-Way Frontage Roads.* Research Report 1393-1, Texas Transportation Institute, Texas Department of Transportation. 1995.

3. Nowlin, R.L. and K. Fitzpatrick. *Two-Sided Weaving Analysis on One-Way Frontage Roads.* Research Report FHWA/TX-97/1393-2. Texas Transportation Institute, Texas Department of Transportation. Draft Report. 1996.

4. Fitzpatrick, K., R.L. Nowlin, and A.H. Parham. *Development of Level of Service Procedure for Frontage Roads.* Research Report FHWA/TX-97/1393-3. Texas Transportation Institute, Texas Department of Transportation. Draft Report. 1996.

5. *A Policy on Geometric Design of Highways and Streets.* American Association of State Highway and Transportation Officials. 1994.

6. Turner, J. M., and C.J. Messer. "Distance Requirements for Frontage-Road Ramps to Cross Streets: Urban Freeway Design." *Transportation Research Record* 682. pp 58-64. 1978.

7. Sharma, S. and C.J. Messer. *Distance Requirements for Ramp Metering.* Research Report 1392-5. Texas Transportation Institute, Texas Department of Transportation. 1994.

8. Gattis, J.L., C.J. Messer, and V.G. Stover. *Delay to Frontage Road Vehicles at Intersections With Ramps.* Report No. FHWA/TX-86/402-2. Texas Transportation Institute, Texas Department of Transportation. June 1988.

APPENDIX A

WORKSHEETS

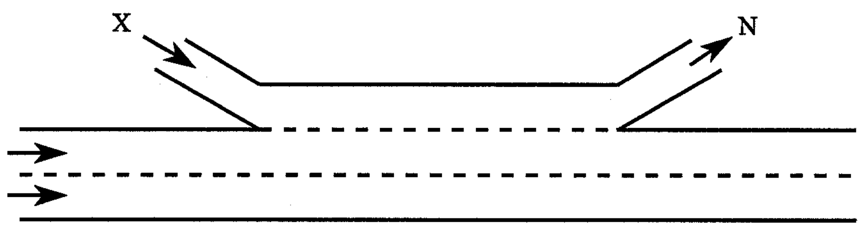
Appendix A contains the worksheets to be used for evaluating weaving segments, calculating ramp junction spacing, and determining the level of service for a continuous frontage road section.

ONE-SIDED WEAVING ANALYSIS WORKSHEET

Location: ______________________________ Direction: ___________________ - bound

Description: ____

Date: _________________________________ Prepared By: _________________



Exit Ramp Volume (X): _______ vph **Entrance Ramp Volume (N): _______ vph**

Weaving Volume (X + N): __________ vph

Weaving Volume	Level of Service
< 1500 vph	Unconstrained
1500 - 3000 vph	Constrained
> 3000 vph	Undesirable

Level of Service: ____________________

TWO-SIDED WEAVING ANALYSIS WORKSHEET

Location: ______________________ Direction: ______________ - bound

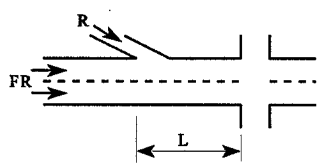
Description: ____

Date: ______________________ Prepared By: ______________

Exit Ramp Volume (R): __________ **vph** **Ramp Spacing (L):** __________ **m**

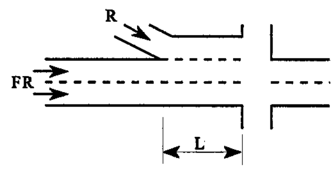
Frontage Road Volume (FR): ________ **vph** **Percent 2-Sided Weaving (T):** ______

[T=0 for ≤ 50%, T= 1 for > 50%]



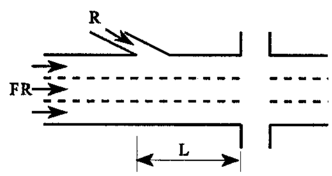
2LFR

$D_L = 0.034(FR) + 0.098(R) - 0.132(L) + 9.51(T)$



2LFR+Aux

$D_L = 0.021(FR) + 0.077(R) - 0.150(L) + 23.4(T)$



3LFR

$D_L = 0.055(FR) + 0.080(R) - 0.200(L) + 27.4(T)$

Density (D_L): __________ veh/km/ln

Density, veh/km/ln	Level of Service
< 40	Unconstrained
40 - 100	Constrained
> 100	Undesirable

Level of Service: ______________

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 1

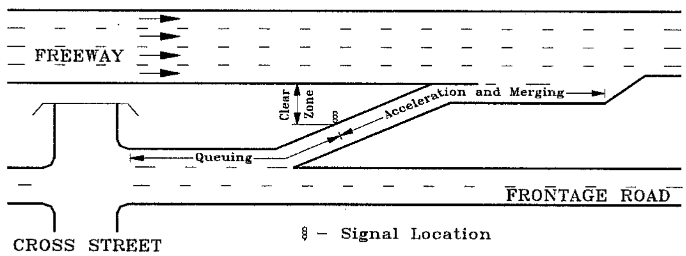
COMMENTS

Site: ________ Date: ________ Time: __________

Name: ________________ Checked by: ____________

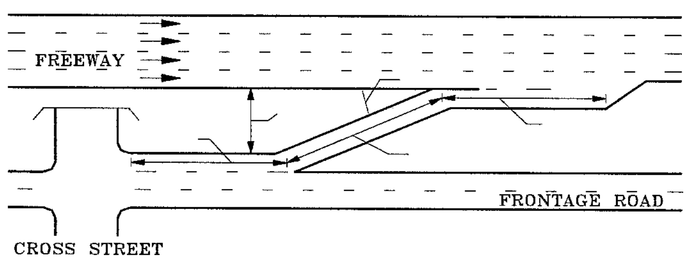
I. DESIRED SOLUTION

The following figure illustrates the design requirements: distance for acceleration and merging, ramp meter signal location and clear zone, and queue storage.



II. GEOMETRIC DATA

Frontage road leaving cross street: 2 ,3 , or 4 lanes: ____
Angle of merge = 3 ,4, or 5 degrees: ____
Separation between outside freeway travel lane and left frontage lane
(edge-to-edge) = _____ m
Length of entrance ramp = _____ m
Length of acceleration lane = ______ m
Storage space available between the cross street and the
entrance ramp = _____ m



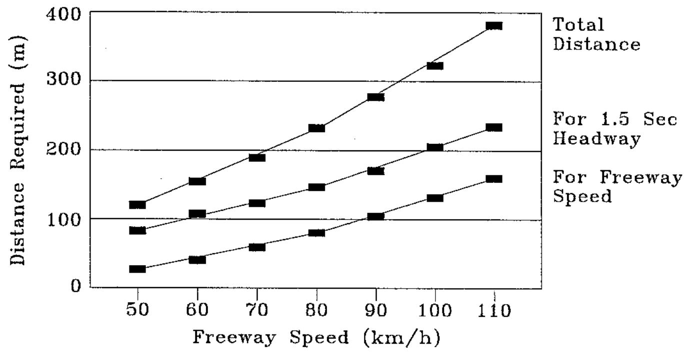
Existing Conditions

III. OPERATIONAL CONDITIONS

Entrance ramp peak hour arrival rate = ______ vph
Freeway speed = ______ km/h
Minimum ramp metering rate = ______ vph

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 2

IV. DETERMINE REQUIRED DISTANCE FOR ACCELERATION AND MERGING



The total distance required for acceleration and merging can be determined from the Total Distance curve on the above figure. The total distance is the ramp distance required for acceleration plus the merging distance required to achieve a 1.5 second headway after reaching freeway speed and before merging with the freeway travel lane.

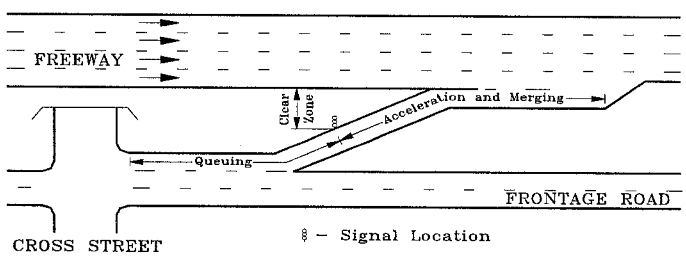
From the above figure, the total acceleration and merging distance required for a freeway speed of ______ km/h = ______ m.

The existing length of the freeway acceleration lane = ______ m.

Therefore, the distance required on the entrance ramp for acceleration and merging is the *total acceleration and merging distance required* minus the *existing length of the freeway acceleration lane*:

______ m - ______ m = ______ m required on the ramp.

This distance is used to locate the signal; however, the clear-zone distance needs to be checked (see Part V).

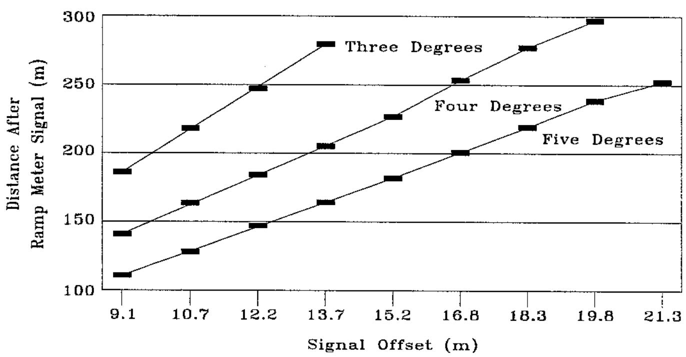


COMMENTS

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 3

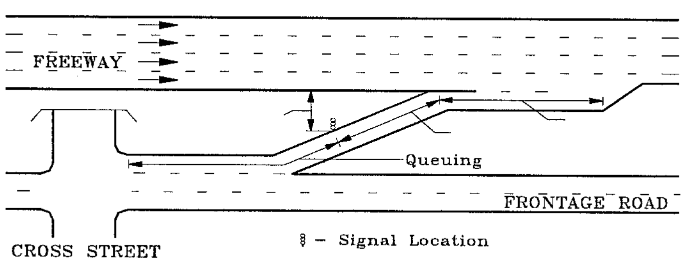
V. CHECK CLEAR ZONE

Part IV determined that a minimum of _____ m of the ramp is needed for acceleration and merging purposes. The following figure gives the minimum acceleration distance after the ramp meter signal for various signal offsets (or clear-zone distances) and for 3, 4, and 5 degree angles of merge between the ramp and freeway. The minimum desirable clear zone is 9 m, which is the distance from the outside edge of the freeway travel lane to the ramp meter signal.



For a distance of ____ m after the ramp meter signal, a clear zone of ____ m is provided for a ___ degree merge angle. A 9 m clear zone for a ___ degree merge results in ____ m after the ramp meter signal.

Engineering judgement must be used to determine if the clear zone and the distance for accelerations and merging are adequate. If not, the ramp meter may need to be shifted to another location. **It should be verified that the ramp meter signal is on the entrance ramp and not on the frontage road.**



Clear Zone and Signal Location

COMMENTS

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 4

VI. DETERMINE QUEUE STORAGE REQUIREMENT

The portion of the ramp not used for acceleration and merging is available for queue storage:

__________ m (ramp length)

\- __________ m (portion used for acceleration and merging)

= __________ m (portion available for queue storage)

Determine the queue storage length required for an arrival rate of _____ vph and a ______ minute delay from the table below.

Entrance Ramp Arrival Rate (vph)	Acceptable Delay (min)				
	1	2	3	4	5
200	39	65	84	98	108
300	59	98	125	146	163
400	78	130	167	195	217
500	98	163	209	244	271
600	117	195	251	293	325
700	137	228	293	342	380
800	156	260	335	390	434

Required queue storage length = _______ m

Determine the available queue storage length:

_______ m (on the frontage road)

\+ _______ m (on the ramp)

= _______ m (available queue storage length)

If the required queue storage length is less than the queue storage length available, the design is good.

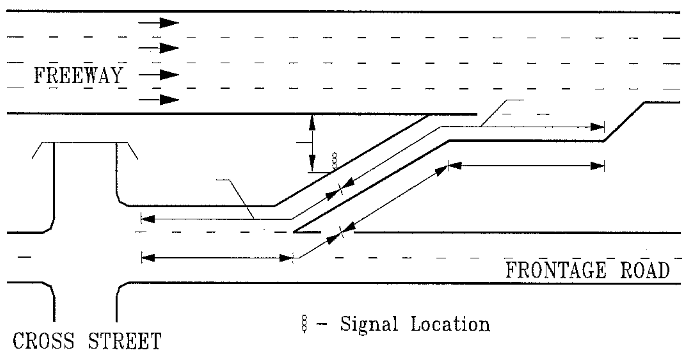
If the provided distance is less than the required distance, some compromise between the queue storage distance and the roadside safety clear zone requirement may be made depending upon the judgement of the engineer.

COMMENTS

WORKSHEET: SPACING NEEDS BETWEEN METERED ENTRANCE RAMPS AND UPSTREAM INTERSECTIONS Page 5

VII. SOLUTION

The following sketch illustrates the solution to the design problem.



COMMENTS

VIII. NOTES

If the entrance ramp is on a positive slope, additional distance may be required for acceleration.

FRONTAGE ROAD LEVEL-OF-SERVICE WORKSHEET

Location: ______________________ Direction: ______________ - bound

Description: ______________________ Type: ______________________

Date: ______________________ Prepared By: ______________________

Seg-ment	Segment Length (km) L	Approach Density (acs/km)	Running Time[a] (sec) RT	Inter-section Total Delay[b] (sec) D_I	Ramp Delay[c] (sec) D_R	Total Travel Time[d] (sec) T	Average Travel Speed[e] (km/h) S	Frontage Road LOS by Segment[f]

[a] Use field data or values from Table 5-3
[b] From Signalized Intersection Delay Worksheet
[c] From Ramp Junction Delay Worksheet
[d] $T = RT + D_I + D_R$
[e] $S = 3600 (L) / T$
[f] See LOS criteria in Table 5-9.

Sum of Travel Times, sec (ΣT) = ______________

Total Frontage Road Length, km (ΣL) = ______________

Average Frontage Road Speed, km/h = 3600 (ΣL) / (ΣT) = ______________

Frontage Road LOS = ______________

SIGNALIZED INTERSECTION DELAY WORKSHEET

Location: ____________________ Direction: ____________________ - bound

Description: ____________________ Type: ____________________

Date: ____________________ Prepared By: ____________________

Segment	Cycle Length (sec) C	Green/ Cycle Time Ratio g/C	v/c Ratio X	Lane Group Capacity (vph) c	Arrival Type [a]	Uniform Delay[b] (sec) d_1	DF[c]	Incremental Delay[d] (sec) d_2	Intersection Stopped Delay[e] (sec) d	Intersection Total Delay[f] (sec) D_I	Intersection LOS[g]

[a] Table 5-4

[b] Equation 5-3 $d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]}$

[c] Table 5-5

[d] Equation 5-4 $d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}]$

[e] Equation 5-2 $d = d_1 DF + d_2$

[f] Equation 5-1 $D_I = 1.3 * d$

[g] Table 5-6

RAMP JUNCTION DELAY WORKSHEET (ONE-WAY FRONTAGE ROADS)

Location: ____________________ Direction: ____________________ - bound

Description: ____________________ Type: ____________________

Date: ____________________ Prepared By: ____________________

Segment	Exit Ramp Hourly Volume[a] (veh/hr) Q_R	Frontage Road Hourly Volume (veh/hr) a	Potential Capacity of Frontage Road Lanes[b] (veh/hr) C_R	Queuing System Delay per Vehicle[c] (sec) W	Predicted Total Delay per Vehicle[d] (sec) D_R

[a] Q_R must be $\leq$ 1200; otherwise, use engineering judgement. If an auxiliary lane is present, delay is negligible.

[b] C_R = # Lanes $(1858 - 1.5259\,(Q_R))$

[c] $W = 3600 / (C_R - a)$

[d] $D_R = -0.0719 + 1.0922\,(W)$

RAMP JUNCTION DELAY WORKSHEET (TWO-WAY FRONTAGE ROADS)

Location: ______________________ Direction: ______________________ - bound

Description: ______________________ Type: ______________________

Date: ______________________ Prepared By: ______________________

Segment	Scenario[a]	Ramp Hourly Volume (vph) Q_R	Frontage Road Hourly Volume (vph) a	Potential Capacity of Frontage Road (vph) C_R	Queuing System Delay per Vehicle (sec) W	Predicted Total Delay per Vehicle (sec) D_R

[a] Scenarios and Equations:

Exit Ramp *With*:

$C_R = 1724 - 1.6120\ (Q_R)$
$W = 3600 / (C - a)$
$D_R = -0.0719 + 1.0922\ (W)$

Exit Ramp *Opposing*:

$C_R = 1444 - 1.6564\ (Q_R)$
$W = 3600 / (C - a)$
$D_R = -1.6451 + 1.7785\ (W)$

Entrance Ramp *Opposing*:

$C_R = 1535 - 1.3852\ (Q_R)$ (Note: Q_R is assumed to be total frontage road *with* volume)
$W = 3600 / (C - a)$
$D_R = 0.0538 + 1.3027\ (W)$

APPENDIX B

FRONTAGE ROAD LEVEL-OF-SERVICE ANALYSIS

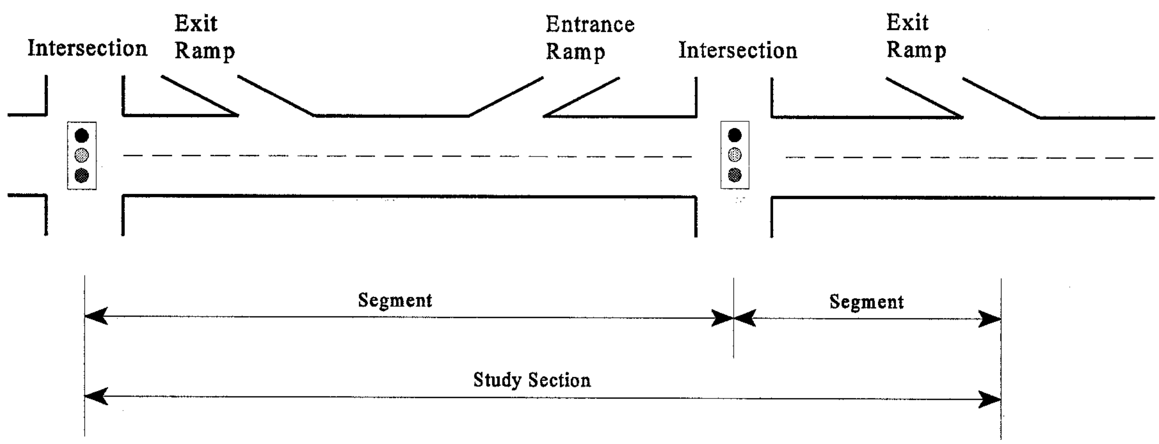
FLOW CHARTS

The following flow charts can be used as a quick reference for performing a level-of-service analysis of a frontage road. The first chart has metric units while the second has English units.

METRIC UNITS

STEP 1 - DEFINE SEGMENT

Use the following diagram to define segments:



STEP 2 - GATHER FIELD DATA

Roadway Characteristics	* Segment length, km * Type of traffic control at intersections * Number of all exit and entrance ramps (two-way only) * Number of exit ramps without auxiliary lanes (one-way only) * Segment access density, acs/km (number of driveways and unsignalized intersections per kilometer)
Traffic Data	* Frontage road approach volume at stop-controlled and signalized intersections, vph * Ramp and frontage road volumes at all exit and entrance ramps, vph (two-way only) * Exit ramp and frontage road volumes at exit ramps without auxiliary lanes, vph (one-way only)
Signal Data	* Signal progression data * Intersection capacity (c), vph * Cycle length (C), sec * Green/cycle time ratio (g/C) * Volume/capacity ratio (v/c)

STEP 3 - COMPUTE RUNNING TIMES						
	One-Way Frontage Roads		Two-Way Frontage Roads			
Access Density (acs/km)	≤ 20	> 20	≤ 16		>16	
Frontage Road Volume (vph)	All	All	≤ 400	> 400	≤ 400	> 400
Segment Length (km)	Running Time, RT (seconds)					
0.2	10	11	10	11	11	13
0.4	20	22	21	23	23	25
0.6	30	33	31	34	34	38
0.8	40	44	42	46	46	50
1.0	50	55	52	57	57	63
1.2	60	67	62	69	69	75
1.4	71	78	73	80	80	88
1.6	81	89	83	91	91	100
1.8	91	100	93	103	103	113
2.0	101	111	104	114	114	126
2.2	N/A	N/A	114	126	126	138
2.4	N/A	N/A	125	137	137	151
2.6	N/A	N/A	135	148	148	163
2.8	N/A	N/A	145	160	160	176
3.0	N/A	N/A	156	171	171	188
3.2	N/A	N/A	166	183	183	201

NOTES:
If segment length falls outside of 0.2 to 2.0 km for one-way and 0.2 to 3.2 km for two-way, consider redefining segments.
If access density is unknown, assume ≤ 20 acs/km for one way and ≤ 16 acs/km for two-way.
Access Density, acs/km = [(# of driveways + # of unsignalized intersections) / total length, km]

STEP 4 - COMPUTE DELAY AT INTERSECTIONS

Compute total intersection delay (D_I) for each signalized intersection using the following formulas:

$$D_I = 1.3 * d \qquad d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]}$$

$$d = d_1 DF + d_2 \qquad d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}]$$

where:

- d = stopped delay, sec/veh
- d_1 = uniform delay, sec/veh
- d_2 = incremental delay, sec/veh
- DF = delay adjustment factor for either quality of progression or type of control
- X = volume/capacity ratio of lane group
- C = cycle length, sec
- c = capacity of lane group, vph
- g = effective green time for lane group, sec
- m = incremental delay calibration term representing effect of arrival type and degree of platooning

STEP 5 - COMPUTE DELAY AT RAMP JUNCTIONS					
Case	Frontage Road	Scenario	Frontage Road Capacity, C_R (vph)	Queuing Delay, W (sec/veh)	Average Total Delay, D_R (sec/veh)
1	One-Way	Exit Ramp without Auxiliary Lane	N [1858 - 1.5259 (Q_R)]	1/(u-a)	- 0.0719 + 1.0922(W)
2	Two-Way	Exit Ramp *With*	1724 - 1.6120(Q_R)	1/(u-a)	-0.0719 + 1.0922(W)
3	Two-Way	Exit Ramp *Opposing*	1444 - 1.6564(Q_R)	1/(u-a)	-1.6451 + 1.7785(W)
4	Two-Way	Entrance Ramp *Opposing*	1535 - 1.3852(Q_R)	1/(u-a)	0.0538 + 1.3027(W)

NOTES:
These equations are not valid when volume exceeds capacity.
N = number of frontage road through lanes
W = average queuing system delay, sec/veh
Q_R= hourly ramp volume (For Case 4, includes all vehicles which approach the entrance ramp from the *with* direction, whether or not they enter the ramp)
u = service rate in vehicles per second (C_R / 3600)
a = frontage road flow rate in vehicles per second (volume / 3600)

STEP 6 - COMPUTE AVERAGE TRAVEL SPEED

The average travel speed is computed using the following formula:

$$S = \frac{3{,}600(L)}{RT + D_I + D_R}$$

where:

S = average travel speed, km/h
L = length of frontage road, km
RT = total running time, sec
D_I = total approach delay for all signalized and stop-controlled intersections, sec
D_R = total frontage road delay incurred at ramps, sec

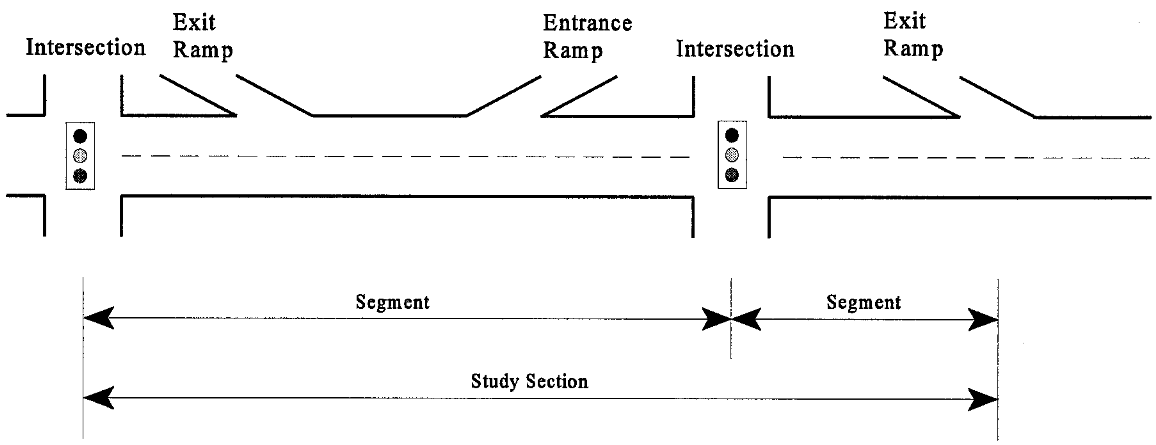
STEP 7 - ASSESS LEVEL OF SERVICE

Level of Service	Average Travel Speed (km/h)
A	≥ 56.0
B	≥ 45.0 to 55.9
C	≥ 35.0 to 44.9
D	≥ 27.0 to 34.9
E	≥ 21.0 to 26.9
F	< 21.0

ENGLISH UNITS

STEP 1 - DEFINE SEGMENT

Use the following diagram to define segments:



STEP 2 - GATHER FIELD DATA

Roadway Characteristics	* Segment length, mi * Type of traffic control at intersections * Number of exit and entrance ramps (two-way only) * Number of exit ramps without auxiliary lanes (one-way only) * Segment access density, acs/mi (number of driveway and unsignalized intersections / mile)
Traffic Data	* Frontage road approach volume at stop-controlled and signalized intersections, vph * Ramp and frontage road volumes at all exit and entrance ramps, vph (two-way only) * Exit ramp and frontage road volumes at exit ramps without auxiliary lanes, vph (one-way only)
Signal Data	* Signal progression data * Intersection capacity (c), vph * Cycle length (C), sec * Green/cycle time ratio (g/C) * Volume/capacity ratio (v/c)

STEP 3 - COMPUTE RUNNING TIMES						
	One-Way Frontage Roads		Two-Way Frontage Roads			
Access Density (acs / mi)	≤ 33	> 33	≤ 27		> 27	
Frontage Road Volume (vph)	All	All	≤ 400	> 400	≤ 400	> 400
Segment Length (mile)	Running Time, RT (seconds)					
0.1	8	9	8	9	9	10
0.2	16	18	17	19	19	21
0.3	25	27	25	28	28	31
0.4	33	36	34	37	37	34
0.5	41	45	42	46	46	51
0.6	49	54	51	56	56	62
0.7	57	63	59	65	65	72
0.8	67	72	68	74	74	81
0.9	74	81	76	84	84	92
1.0	82	90	84	93	93	102
1.1	90	99	92	102	102	112
1.2	98	108	101	111	111	122
1.3	N/A	N/A	109	120	120	131
1.4	N/A	N/A	117	129	129	142
1.5	N/A	N/A	125	138	138	152
1.6	N/A	N/A	134	147	147	162
1.7	N/A	N/A	142	156	156	172
1.8	N/A	N/A	150	165	165	182
1.9	N/A	N/A	159	175	175	192
2.0	N/A	N/A	167	184	184	202

NOTES:
If segment length falls outside of 0.1 to 1.2 mi for one-way and 0.1 to 2.0 mi for two-way, consider redefining segments.
If access density is unknown, assume ≤ 33 acs/mi for one way and ≤ 27 acs/mi for two-way.
Access Density, acs/mi = [(# of driveways + # of unsignalized intersections) / total length, mi]

STEP 4 - COMPUTE DELAY AT INTERSECTIONS

Compute total intersection delay (D_I) for each signalized intersection using the following formulas:

$$D_I = 1.3 * d \qquad d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]}$$

$$d = d_1 DF + d_2 \qquad d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}]$$

where:
d = stopped delay, sec/veh
d_1 = uniform delay, sec/veh
d_2 = incremental delay, sec/veh
DF = delay adjustment factor for either quality of progression or type of control
X = volume/capacity ratio of lane group
C = cycle length, sec
c = capacity of lane group, vph
g = effective green time for lane group, sec
m = incremental delay calibration term representing effect of arrival type and degree of platooning

STEP 5 - COMPUTE DELAY AT RAMP JUNCTIONS					
Case	Frontage Road	Scenario	Frontage Road Capacity, C_R (vph)	Queuing Delay, W (sec/veh)	Average Total Delay, D_R (sec/veh)
1	One-Way	Exit Ramp without Auxiliary Lane	N[1858 - 1.5259 (Q_R)]	1/(u-a)	- 0.0719 + 1.0922(W)
2	Two-Way	Exit Ramp *With*	1724 - 1.6120(Q_R)	1/(u-a)	-0.0719 + 1.0922(W)
3	Two-Way	Exit Ramp *Opposing*	1444 - 1.6564(Q_R)	1/(u-a)	-1.6451 + 1.7785(W)
4	Two-Way	Entrance Ramp *Opposing*	1535 - 1.3852(Q_R)	1/(u-a)	0.0538 + 1.3027(W)

NOTES:
These equations are not valid when volume exceeds capacity.
N = number of frontage road through lanes
W = average queuing system delay, sec/veh
Q_R= hourly ramp volume (For Case 4, includes all vehicles which approach the entrance ramp from the *with* direction, whether or not they enter the ramp)
u = service rate in vehicles per second (C_R / 3600)
a = frontage road flow rate in vehicles per second (volume / 3600)

STEP 6 - COMPUTE AVERAGE TRAVEL SPEED

The average travel speed is computed using the following formula:

$$S = \frac{3{,}600(L)}{RT + D_I + D_R}$$

where:

- S = average travel speed, mph
- L = length of frontage road, mi
- RT = total running time, sec
- D_I = total approach delay for all signalized and stop-controlled intersections, sec
- D_R = total frontage road delay incurred at ramps, sec

STEP 7 - ASSESS LEVEL OF SERVICE

Level of Service	Average Travel Speed (mph)
A	≥ 35.0
B	≥ 28.0 to 34.9
C	≥ 22.0 to 27.9
D	≥ 17.0 to 21.9
E	≥ 13.0 to 16.9
F	< 13.0

APPENDIX C

USING THE *HIGHWAY CAPACITY SOFTWARE* TO DETERMINE FRONTAGE ROAD LEVEL OF SERVICE

OVERVIEW OF THE *HIGHWAY CAPACITY SOFTWARE*

The *Highway Capacity Software (HCS)* is a computer version of the *Highway Capacity Manual*. It was originally developed by the Federal Highway Administration to implement the procedures contained in the *Highway Capacity Manual (HCM)*. It performs the multiple calculations that users of worksheets must complete. *HCS* Release 2.1 is the version associated with the 1994 *HCM*. The software is distributed exclusively by Mc*Trans* (Transportation Research Center, University of Florida, 512 Weil Hall, Gainesville, FL 32611-2083, phone 904-392-0378). Software support and maintenance for the *HCS* is provided by Mc*Trans*, supported by user license fees. A manual on using the *HCS* is also available from Mc*Trans*.

The Urban and Suburban Arterial module of the *HCS* contains three worksheets screens:

- Description of Arterial
- Intersection Delay Estimates
- Arterial Level of Service

The Description of Arterial screen asks for information on the name of the arterial, its class, and the number of segments. The Intersection Delay Estimate screen requests the information related to signalized intersections. The determination of the level of service for the facility is computed and shown in the Arterial Level-of-Service screen.

By using a few assumptions and modifying some of the calculated values in the screens, the *HCS* can be used to determine the level of service on a frontage road. For example, an arterial class of 1 is to be assumed for freeway frontage roads. In addition, the "Other Delay" column shown on the Arterial Level-of-Service screen is modified to account for the delay at ramp junctions. Table C-1 lists hints on how to use the *HCS* for frontage road level-of-service evaluations.

Following are examples of using the *HCS* to evaluate a one-way and a two-way frontage road. Currently, *HCS* runs in English units; therefore, the reproduction of the software's printouts are in English units. The metric values are noted in the accompanying discussion.

Table C-1. Hints for Frontage Road Analysis Using *HCS*.

HCS Screen	*HCM* 1994 (*HCS* Release 2.1)
Description of Arterial	* Divide one-way frontage road sections into segments ≥ 0.1 mi (0.2 km) and ≤ 1.2 mi (2.0 km). Divide two-way frontage road sections into segments ≥ 0.1 mi (0.2 km) and ≤ 2.0 mi (3.2 km). (A segment is typically from signal to signal but may be a traffic signal to an entrance ramp, an entrance ramp to an exit ramp, an exit ramp to a cross street, etc.) * Arterial classification is 1. * For the sites used in the evaluation, free flow speeds on the one-way frontage roads were between 40 and 50 mph (64 and 80 km/h). Two-way frontage roads typically had free flow speeds between 35 and 40 mph (56 and 64 km/h).
Intersection Delay Estimates	* For each segment, enter the cycle length, g/C, v/c, capacity, and arrival type (see Table 5-4). NOTE: for frontage road segments that do not have signals, this information may be entered as zero. * g/C = (green + yellow) / cycle length * capacity = (# of lanes)(saturation flow rate)(g/C) NOTE: This software uses a saturation flow rate of 1900 vphgpl as a default value. Saturation flow rate should reflect local conditions.
Arterial Level of Service	* Actual free flow speed can be entered. For speeds > 45 mph (72 km/h), *HCS* will produce a message saying the free flow speed is out of bounds of Table 11-4. * Under "Sum of Time," adjust running time, as desired, with values from Table 5-3. (*HCS* Release 2.1 does not allow adjustments in the "Running Time" column.) * Under "Other Delay," add delay at ramp junctions as determined from the Ramp Junction Delay Worksheet.

SAMPLE CALCULATION: ONE-WAY FRONTAGE ROAD

Step 1: Define Frontage Road Study Section

The frontage road to be considered is a 2.4 mile (3.9 kilometer) length of a two-lane, one-way frontage road in an area of moderate development. Figure C-1 illustrates the frontage road section to be analyzed. Each of the crossroad intersections shown are controlled by traffic signals. The one-way frontage road are divided into the following three segments (with each segment being bound by signalized intersections): Lemon to Georgia, Georgia to 39th, and 39th to University.

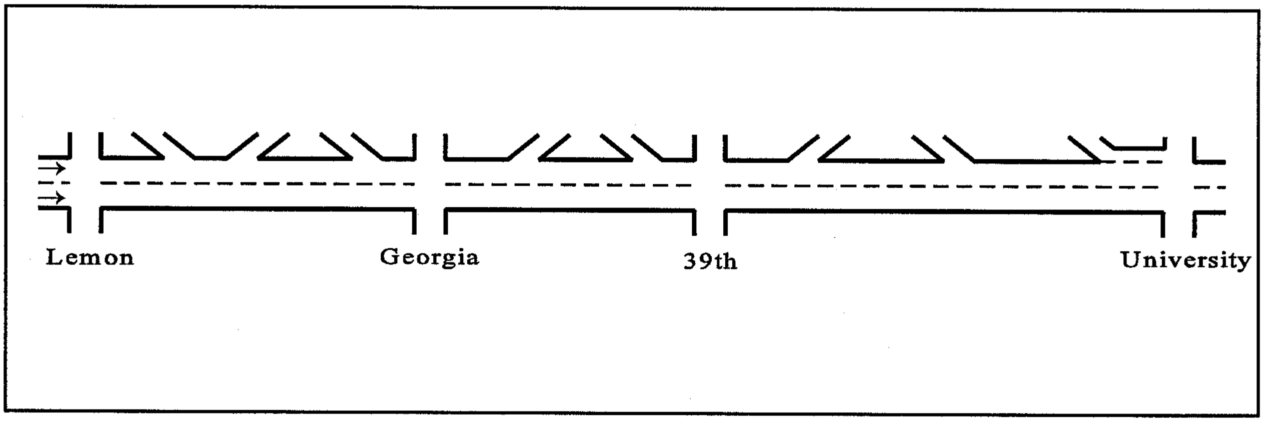


Figure C-1. Schematic of One-Way Frontage Road Study Section.

Step 2: Gather Field Data

The required roadway data (summarized in Table 5-1) are shown in Table C-2, while the traffic data are listed in Table C-3. Table C-4 lists signalized intersection data. Random arrival and a saturation flow rate of 1800 vphgpl are assumed.

Segment descriptions and free-flow speeds are entered on the Description of Arterial screen in the Urban Arterials Module (see Figure C-2). Arterial Classification is entered as 1 because frontage road characteristics are similar to those of Arterial Classification 1.

Table C-2. Roadway Data for One-Way Frontage Road Example.

Segment	Segment Boundaries	Length (mi / km)	Free Flow Speed (mi / km)	Access Density (acs/mi / acs/km)
1	Lemon to Georgia	0.73 / 1.18	45 / 72	34.2 / 21.3
2	Georgia to 39th	0.67 / 1.08	40 / 64	29.3 / 18.2
3	39th to University	1.00 / 1.61	45 / 72	26.0 / 16.2

Table C-3. Traffic Data for One-Way Frontage Road Example.

Number of Exit Ramps w/o Aux. Lanes	Exit Ramp Volume (vph)	Frontage Road Volume (vph)	
		At Exit Ramps	At Intrsct.
2	Exit 1: 358 Exit 2: 180	Exit 1: 193 Exit 2: 97	282
1	214	115	372
1	98	53	264

Table C-4. Signal Data for One-Way Frontage Road Example.

Intersection	Intersection Capacity, c[a] (vph)	Cycle Length, C (sec)	g / C	v / c
Georgia	900	120	0.25	0.316
39th	1224	100	0.34	0.304
University	936	75	0.26	0.279

[a] c = (Saturation flow rate) (# of lanes) (g/C)

```
HCS: Arterial   Release 2.1
*****************************************************************************************************

File Name . . . . . . . . . . . . .1WAYEX
Arterial . . . . . . . . . . . . . . . IH-99 Frontage Road
From / To . . . . . . . . . . . . . Lemon to University
Direction . . . . . . . . . . . . . . N
Analyst . . . . . . . . . . . . . . .  Sally
Time of Analysis . . . . . . . .
Date of Analysis . . . . . . . . 8 / 19/ 96
Other Information . . . . . . .
```

A. Description of Arterial

Seg.	Intersection File Name	Street Name	Length (mi)	Art. Class		Free Flow Speed (mph)	Sect.
		Lemon					
1		Georgia	0.73	1	*	45	1
2		39th	0.67	1		40	2
3		University	1.00	1	*	45	3

* Free flow speed is out of bounds of Table 11-4. Free-flow speed will be used as arterial speed to compute running times.

Figure C-2. Enter Frontage Road Description.

Step 3: Compute Running Time

Running times are computed by *HCS* on the Arterial Level-of-Service screen (see Figure C-3). However, these values can be adjusted for frontage roads by using the running time values in Table 5-3. The running times determined for frontage roads were similar to the assumed running times for arterials. Therefore, adjustments are not required; use engineering judgement. The running times listed in Table C-5 are obtained from Table 5-3.

C. Arterial Level of Service

Seg.	Sect.	Running Time	Int. Total Delay	Other Delay	Section Sum of Time	Sum of Length (mi)	Arterial Speed (mph)	Arterial LOS
1	1	61.6						
2	2	61.6						
3	3	80.0						

Figure C-3. Compute Initial Running Time.

Table C-5. Running Times for One-Way Frontage Road Example.

Segment	Boundaries	Length (mi / km)	Running Time from Table 5-3 (sec)
1	Lemon to Georgia	0.73 / 1.18	67
2	Georgia to 39th	0.67 / 1.08	55
3	39th to University	1.00 / 1.61	81

Running times cannot be adjusted in the "Running Time" column; therefore, they must be adjusted in the "Sum of Time" column on the *HCS* Arterial Level-of-Service screen. The difference between the *HCS* computed values and the values in Table 5-3 must be added to or subtracted from the "Sum of Time" values, which will be done in Step 5 after intersection delay and ramp delay are computed.

Step 4: Compute Intersection Delay

Cycle length, g/C, v/c, capacity, and arrival type are entered on the Intersection Delay Estimates screen (see Figure C-4). (The hints shown in Table C-1 provide information on calculating capacity and v/c.) Arrival Type is matched with the *HCM* arrival type definitions which are provided in Table 5-4. Arrival Type 3 is selected for the example. On the Intersection Delay Estimates worksheet, *HCS* computes the uniform delays, incremental delays, intersection stopped delay, intersection total delay, and intersection level of service (see Figure C-5).

B. Intersection Delay Estimates

Seg.	C	g/C	v/c	c	Arrival Type	D1	DF	D2	Inter. Stopped Delay	Inter. Total Delay	Inter. LOS
1	120	0.25	0.316	900	3						
2	100	0.34	0.304	1224	3						
3	75	0.26	0.279	936	3						

Figure C-4. Enter Intersection Data.

B. Intersection Delay Estimates

Seg.	C	g/C	v/c	c	Arrival Type	D1	DF	D2	Inter. Stopped Delay	Inter. Total Delay	Inter. LOS
1	120	0.25	0.316	900	3	27.9	1.000	0.1	28.0	36.4	D
2	100	0.34	0.304	1224	3	18.5	1.000	0.0	18.5	24.1	C
3	75	0.26	0.279	936	3	16.8	1.000	0.0	16.8	21.9	C

Figure C-5. Compute Intersection Data.

Step 5: Compute Ramp Delay

Ramp delay is computed using the Ramp Junction Delay Worksheet (One-Way Frontage Roads). For one-way frontage roads, ramp delays are calculated for exit ramps without auxiliary lanes only. Segment 1 has two exit ramps without auxiliary lanes, and Segments 2 and 3 each have one exit ramp without an auxiliary lane. Delay for each ramp is calculated on a separate line of the worksheet (see Figure C-6).

RAMP JUNCTION DELAY WORKSHEET (ONE-WAY FRONTAGE ROADS)

Location: IH-99 Direction: North -bound

Description: Between Lemon and University Type: One-Way

Date: 8-19-96 Prepared By: Sally

Segment	Exit Ramp Hourly Volume[a] (veh/hr) Q_R	Frontage Road Hourly Volume (veh/hr) a	Potential Capacity of Frontage Road Lanes[b] (veh/hr) C_R	Queuing System Delay per Vehicle[c] (sec) W	Predicted Total Delay per Vehicle[d] (sec) D_R
1	358	193	2623	1.5	1.6
1	180	97	3167	1.2	1.2
2	214	115	3063	1.2	1.3
3	98	53	3418	1.1	1.1

[a] Q_R must be ≤ 1200; otherwise, use engineering judgement. If an auxiliary lane is present, delay is negligible.
[b] C_R = # Lanes (1858 - 1.5259 (Q_R))
[c] $W = 3600 / (C_R - a)$
[d] $D_R = -0.0719 + 1.0922 (W)$

Figure C-6. Ramp Delay for One-Way Frontage Road Example.

Ramp delay is entered in the "Other Delay" column on the Arterial Level-of-Service screen (see Figure C-7).

As described in Step 3, the Sum of Time values may now be adjusted so that they equal the running time values from Table 5-3 plus the intersection delay and ramp delay values (see Figure C-8). The asterisks indicate that the values have been modified.

C. Arterial Level of Service

Seg.	Sect.	Running Time	Int. Total Delay	Other Delay	Section Sum of Time	Section Sum of Length (mi)	Arterial Speed (mph)	Arterial LOS
1	1	61.6	36.4	2.8		0.73		
2	2	61.6	24.1	1.3		0.67		
3	3	80.0	21.9	1.1		1.00		

Figure C-7. Enter Ramp Delay.

C. Arterial Level of Service

Seg.	Sect.	Running Time	Int. Total Delay	Other Delay	Section Sum of Time	Section Sum of Length (mi)	Arterial Speed (mph)	Arterial LOS
1	1	61.6	36.4	2.8	* 106.2	0.73	24.7	C
2	2	61.6	24.1	1.3	* 80.4	0.67	30.0	B
3	3	80.0	21.9	1.1	* 104.0	1.00	34.6	B

Grand sum of time:	290.6
Grand sum of length:	2.40 mi
Arterial Speed:	29.7 mph
Arterial LOS:	B

Figure C-8. Adjust Sum of Time.

Step 6: Compute Average Travel Speed

HCS calculates frontage road speed using the following equation:

$$\text{Average Frontage Road Speed} = \frac{3{,}600(\sum \text{of lengths})}{\sum \text{of time}}$$

The resulting values are shown under "Arterial Speed" on the Arterial Level-of-Service screen (see Figure C-8).

Step 7: Assess Level of Service

The frontage road speeds are now compared to the speeds in the Frontage Road Level-of-Service Table (Table 5-9) to determine the level of service. Levels of service for each segment and for the entire length of frontage road analyzed are also printed on the Arterial Level-of-Service screen (as long as the Arterial Classification was entered as 1). As shown in Figure C-8, the average travel speed for the total length of frontage road being analyzed is 29.7 mph (47.8 km/h) and the level of service is "B."

SAMPLE CALCULATION: TWO-WAY FRONTAGE ROADS

Step 1: Define Frontage Road Study Section

The frontage road to be considered is a 1.9 mile (3.1 kilometer) length of two-lane, two-way frontage road that is located in an area of low to moderate development. **This example illustrates the procedure to determine the level of service for the frontage road lane that flows *with* the direction of the freeway traffic. However, the lane *opposing* freeway traffic should also be analyzed because the level of service may be different.** Figure C-9 illustrates the frontage road section to be analyzed. The selected frontage road study section is divided into the following two segments: Smith to Peanut, and Peanut to Exit Ramp.

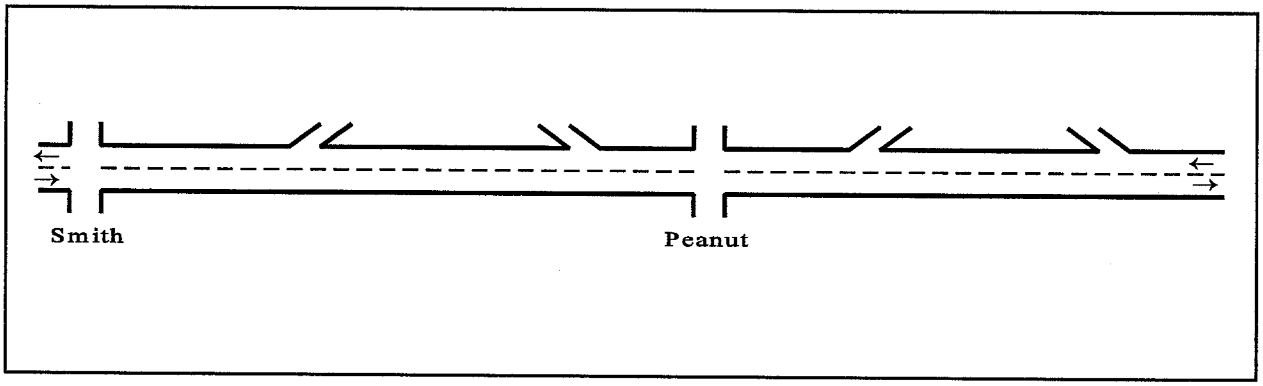


Figure C-9. Schematic of Two-Way Frontage Road Study Section.

Step 2: Gather Field Data

Tables C-6 and C-7 summarize the required field data (see Table 5-1). Table C-8 lists signalized intersection data. Random arrivals and a saturation flow rate of 1800 vphgpl are assumed.

Table C-6. Roadway Data for Two-Way Frontage Road Example.

Segment	Segment Boundaries	Length (mi / km)	Free Flow Speed (mi / km)	Access Density (acs/mi / acs/km)
1	Smith to Peanut	1.10 / 1.77	35 / 56	11.8 / 7.3
2	Peanut to Exit Ramp	0.82 / 1.32	35 / 56	25.6 / 15.9

Table C-7. Traffic Data for Two-Way Frontage Road Example.

Exit Ramp Volume (vph)	Frontage Road Volume (vph)	
	At Exit Ramps	At Intrsct.
264	84	348
204	96	--

Table C-8. Signal Data for Two-Way Frontage Road Study Section.

Intersection	Intersection Capacity, c[a] (vph)	Cycle Length, C (sec)	g / C	v / c
Peanut	360	170	0.20	0.233

[a] c = (Saturation flow rate) (g/C)

Segment descriptions and free-flow speeds are entered on the Description of Arterial screen in the Urban Arterials Module (see Figure C-10). Arterial Classification is entered as 1 because frontage road characteristics are similar to those of Arterial Classisfication 1.

Step 3: Compute Running Time

Running times are computed by *HCS* on the Arterial Level-of-Service screen (see Figure C-11). However, these values may be adjusted for frontage roads by using the running time values in Table 5-3. The running times determined for frontage roads were similar to the assumed running times for arterials. Therefore, adjustments are not required; use engineering judgement. The running times listed in Table C-9 are obtained from Table 5-3.

```
HCS: Arterial   Release 2.1
**************************************************************************

File Name . . . . . . . . . . . . . 2WAYEX
Arterial . . . . . . . . . . . . . . . IH-50 FR Northbound (WITH)
From / To . . . . . . . . . . . . . Smith to Exit Ramp past Peanut
Direction . . . . . . . . . . . . . . N
Analyst . . . . . . . . . . . . . . . Sally
Time of Analysis . . . . . . . .
Date of Analysis . . . . . . . . 8 / 19/ 96
Other Information . . . . . . .
```

A. Description of Arterial

Seg.	Intersection File Name	Street Name	Length (mi)	Art. Class	Free Flow Speed (mph)	Sect.
		Smith				
1		Peanut	1.10	1	35	1
2		Exit Ramp	0.82	1	35	2

Figure C-10. Enter Frontage Road Description.

C. Arterial Level of Service

Seg.	Sect.	Running Time	Int. Total Delay	Other Delay	Section Sum of Time	Section Sum of Length (mi)	Arterial Speed (mph)	Arterial LOS
1	1	113.1						
2	2	84.5						

Figure C-11. Compute Initial Running Time.

Table C-9. Running Times for Two-Way Frontage Road Example.

Segment	Intersection	Length (mi / km)	Running Time from Table 5-3 (sec)
1	Smith to Peanut	1.10 / 1.77	93
2	Peanut to Exit Ramp	1.06 / 1.71	68

Running times cannot be adjusted in the "Running Time" column; therefore, they must be adjusted in the "Sum of Time" column on the *HCS* Arterial Level-of-Service screen. The difference between the *HCS* computed values and the values in Table 5-3 must be added to or subtracted from the "Sum of Time" values, which will be done in Step 5 after intersection delay and ramp delay are computed.

Step 4: Compute Intersection Delay

Cycle length, g/C, v/c, capacity, and arrival type are entered on the Intersection Delay Estimates screen (see Figure C-12). (The hints shown in Table C-1 provide information on calculating capacity and v/c). Arrival Type is matched with the *HCM* arrival type definitions which are provided in Table 5-4. Arrival Type 3 is selected for the example. On the Intersection Delay Estimates screen, *HCS* computes the uniform delays, incremental delays, intersection stopped delay, intersection total delay, and intersection level of service (see Figure C-13).

B. Intersection Delay Estimates

Seg.	C	g/C	v/c	c	Arrival Type	D1	DF	D2	Inter. Stopped Delay	Inter. Total Delay	Inter. LOS
1	170	0.20	0.233	360	3						
2	0	0.00	0.000	0	0						

Figure C-12. Enter Intersection Data.

B. Intersection Delay Estimates

Seg.	C	g/C	v/c	c	Arrival Type	D1	DF	D2	Inter. Stopped Delay	Inter. Total Delay	Inter. LOS
1	170	0.20	0.233	360	3	43.4	1.000	0.1	43.4	56.5	E
2	0	0.00	0.000	0	0	0.0	0.000	0.0	0.0	0.0	

Figure C-13. Compute Intersection Delay.

Step 5: Compute Ramp Delay

Ramp delay is computed using the Ramp Junction Delay Worksheet (Two-Way Frontage Roads). For two-way frontage road lanes flowing *with* the frontage road traffic, ramp delays are calculated for exit ramps only. Segment 1 and segment 2 each have one exit ramp. Delay for each ramp is calculated on a separate line of the worksheet (see Figure C-14) .

RAMP JUNCTION DELAY WORKSHEET
(TWO-WAY FRONTAGE ROADS)

Location: IH-50 Direction: North (With) - bound

Description: Smith to Exit Ramp Past Peanut Type: Two-Way

Date: 8-19-96 Prepared By: Sally

Segment	Scenario[a]	Ramp Hourly Volume (vph) Q_R	Frontage Road Hourly Volume (vph) a	Potential Capacity of Frontage Road (vph) C_R	Queuing System Delay per Vehicle (sec) W	Predicted Total Delay per Vehicle (sec) D_R
1	Exit Ramp With	264	84	1298	2.96	3.2
2	Exit Ramp With	204	96	1395	2.77	3.0

[a] Scenarios and Equations:

Exit Ramp *With*:

$C_R = 1724 - 1.6120\ (Q_R)$
$W = 3600 / (C_R - a)$
$D_R = -0.0719 + 1.0922\ (W)$

Exit Ramp *Opposing*:

$C_R = 1444 - 1.6564\ (Q_R)$
$W = 3600 / (C_R - a)$
$D_R = -1.6451 + 1.7785\ (W)$

Entrance Ramp *Opposing*:

$C_R = 1535 - 1.3852\ (Q_R)$ (Note: Q_R is assumed to be total frontage road *with* volume)
$W = 3600 / (C_R - a)$
$D_R = 0.0538 + 1.3027\ (W)$

Figure C-14. Calculate Ramp Delay for Two-Way Frontage Road Example.

Ramp delay is entered in the "Other Delay" column on the Arterial Level-of-Service worksheet (see Figure C-15). The Sum of Time values can now be adjusted so that they equal the running time values from Table 5-3 plus the intersection delay and ramp delay values (see Figure C-16). The asterisks indicate that the values have been modified.

Step 6: Compute Average Travel Speed

HCS calculates frontage road speed using the following equation:

$$Average\ Frontage\ Road\ Speed = \frac{3{,}600(\sum of\ lengths)}{\sum of\ time}$$

The resulting values are shown under "Arterial Speed" on the Arterial Level-of-Service worksheet (see Figure C-16).

Step 7: Assess Level of Service

The frontage road speeds are now compared to the speeds in the Frontage Road Level-of-Service Table (Table 5-9) to determine the level of service. Levels of service for each segment and for the entire length of frontage road analyzed are also printed on the Arterial Level-of-Service screen (as long as the Arterial Classification was entered as 1). As shown in Figure C-16, the average travel speed for the total length of frontage road being analyzed is 30.9 mph (49.7 km/h) and the level of service is "B."

C. Arterial Level of Service

Seg.	Sect.	Running Time	Int. Total Delay	Other Delay	Section Sum of Time	Section Sum of Length (mi)	Arterial Speed (mph)	Arterial LOS
1	1	113.1	56.5	3.2		1.10		
2	2	84.5	0.0	3.0		0.82		

Figure C-15. Enter Ramp Delay.

C. Arterial Level of Service

Seg.	Sect.	Running Time	Int. Total Delay	Other Delay	Section Sum of Time	Section Sum of Length (mi)	Arterial Speed (mph)	Arterial LOS
1	1	113.1	56.5	3.2	* 152.7	1.10	25.9	C
2	2	84.5	0.0	3.0	* 71.0	0.82	41.6	A

Grand sum of time: 223.7
Grand sum of length: 1.92 mi
Arterial Speed: 30.9 mph
Arterial LOS: B

Figure C-16. Adjust Sum of Time.