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16. Abstract

The objective of the research was to develop a procedure for evaluating the operations along one-way and two-way freeway frontage roads. The developed procedure is based on the arterial analysis chapter of the *Highway Capacity Manual* and includes consideration of the delays that are incurred at ramp junctions. To develop the level-of-service analysis procedure, data were collected at several locations within Texas. Travel time, volume, and access density data were collected at 20 one-way frontage road sites and nine two-way frontage road sites. Delay data were collected at six ramp junctions. The results showed that signalized intersections have the greatest impact on the operations along a frontage road. Access density (i.e., the number of driveways and unsignalized intersections per km) affects the operations along a frontage road segment when greater than 20 acs/km on one-way frontage roads and greater than 16 acs/km on two-way frontage roads. The developed procedure produced reasonable results when compared to the average speed measured at six one-way and six two-way frontage road sites. Predicted results were generally within 3 km/h of measured speeds.

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DEVELOPMENT OF LEVEL-OF-SERVICE ANALYSIS PROCEDURE FOR FRONTAGE ROADS

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IMPLEMENTATION STATEMENT

This report presents a level-of-service analysis procedure for freeway frontage roads. The results from this report will aid engineers in evaluating one-way and two-way frontage road sections. The procedure developed can be used to estimate the level of service on these types of facilities, which can aid in prioritizing frontage road improvement projects and/or predicting future operations.

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 or the Federal Highway Administration. 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).

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SUMMARY

This study developed a procedure for evaluating freeway frontage road operations. The procedure is based on the arterial analysis chapter of the *Highway Capacity Manual* (*HCM*) and includes consideration of the delay incurred at ramp junctions. Several advantages exist for using a modified version of the existing *HCM* arterial procedure. The *HCM* is the state-of-the practice in level-of-service evaluations. Individuals who perform those evaluations are familiar with the *HCM* and the accompanying software that greatly simplifies the calculation efforts. In addition, as updates are made to relevant *HCM* chapters, such as signalized intersections, users can quickly integrate those updates into the frontage road procedure.

To develop the level-of-service analysis procedure, data were collected at several locations within Texas. The selected field sites included a range of characteristics (e.g., volumes, intersection spacings, ramp locations, access densities, etc.) so that the researchers could analyze the effects of these characteristics on travel time. Sites were a minimum of 1.6 km in length, had no construction activity, had a minimum number of horizontal and vertical curves, and were distributed across the state. Travel time, volume, and access density data were collected at 20 one-way frontage road sites and nine two-way frontage road sites. A distance measuring instrument capable of recording distance traveled, travel time, and speed was used to collect the travel time data. To test previously developed models that determine the delay at ramp junctions, researchers collected delay data at six ramp junctions (two exit ramps on one-way frontage roads, and one entrance and three exit ramps on two-way frontage roads).

The findings from the field studies clearly showed that signalized intersections have the greatest impact on the operations along a frontage road. For the two-way frontage road sites studied, the ramp junctions also had significant influence on operations. When examining the effects of roadway characteristics on travel time between signalized intersections, the distance between the intersections had the greatest impact. Access density (i.e., the number of driveways and unsignalized intersections per km) noticeably affected the operations along a frontage road segment when greater than 20 acs/km on one-way frontage roads and greater than 16 acs/km on two-way

frontage roads. For the two-way frontage road sites studied, volume noticeably affected operations when it exceeded 400 vphpl.

The developed procedure was used to estimate the average speed for six one-way and six two-way frontage roads. These speeds were compared to the average speeds measured in the field. For the one-way frontage roads, the estimated average travel speeds were within 2.5 km/h of the actual travel speeds measured in the field. The two-way frontage roads procedure produced results within 3 km/h of the field data in most cases. In conclusion, the procedure presented in Chapter 7 of this report is appropriate for the evaluation of frontage road operations.

CHAPTER 1
INTRODUCTION

Frontage roads have operational features that are similar to both freeways and arterials. For example, frontage roads have ramps (common to freeways) and signalized intersections (common to arterials). Because of this mix of elements, using the existing procedures within the *Highway Capacity Manual* (*HCM*) (1) to estimate capacity and/or level of service would be incomplete. For example, using the arterial analysis procedure would not sufficiently consider the delay at ramp junctions. A procedure created specifically for frontage roads is needed so that the Texas Department of Transportation (TxDOT) can adequately design frontage roads for expected volumes and predict traffic operation over a range of conditions. The procedure could also be used as a guide in the selection of alternatives in solving operational problems.

The existing methodology for arterials contained in the *HCM* represents current research findings and state-of-the-practice in evaluating the operations of a facility that has signalized intersections. Operations along a frontage road are also dominated by signalized intersections, so it is sensible to use the arterial analysis procedure as a basis for the frontage road analysis procedure. Modifications needed to the arterial analysis procedure include consideration of the delay incurred at ramp junctions. In addition, the time required to traverse a set distance on a frontage road may be different than that on an arterial because of the presence of freeway ramps and because frontage roads only have driveways on one side of the facility. After the procedure is developed and refined, step-by-step instructions on how to conduct a level-of-service evaluation of freeway frontage roads are needed.

OBJECTIVE

The overall objective of this study was to develop a procedure to estimate the level of service on a freeway frontage road. In support of that objective, the research also was to examine the effects that certain roadway characteristics (such as access density and volume) have on the operations of a frontage road.

ORGANIZATION

This report is divided into eight chapters. Chapter 1 contains some background information concerning frontage roads and defines the problem statement and research objective. Chapter 2 contains a summary of procedures provided in the *HCM* for estimating capacity and level of service on arterial streets. Included in the chapter is a summary of previous research on estimating the delay to frontage road vehicles at ramp junctions. Chapter 2 also presents how access density influences travel speed on multi-lane highways.

Chapter 3 provides a description of the study design. The site selection and data collection procedures, as well as the data reduction strategies, are described for both one-way and two-way frontage roads included in the study. In addition, this chapter presents a summary of the statistical analysis procedures.

Chapter 4 contains the study results for one-way frontage roads, and Chapter 5 summarizes the results for the study of two-way frontage roads. Chapter 6 describes the development of the level-of-service analysis procedure and compares the field data and the findings from the proposed procedure.

Chapter 7 presents the proposed level-of-service analysis procedure for one-way and two-way frontage roads. Finally, Chapter 8 presents the conclusions drawn from this research.

The appendices contain supporting materials. Data collected on the one-way frontage roads and the speed and travel time plots generated from that data are contained in Appendix A and Appendix B, respectively. Similar information for the two-way frontage roads are contained in Appendix C and Appendix D, respectively.

Appendix E contains the worksheets for the frontage road level-of-service analysis procedure. Appendix F presents how to perform the procedure as summarized in a flow chart. Both metric and English unit flow charts are provided. Appendix G discusses how to use the *Highway Capacity Software* (Release 2.1) to determine frontage road level of service.

CHAPTER 2
PREVIOUS STUDIES

The *HCM* (1) contains a procedure for estimating the levels of service on arterial streets. Although this procedure may not be applied directly to frontage roads, they can certainly be used as a framework. This chapter presents a summary of the procedures contained in the *HCM* for estimating the level of service on arterial streets and for calculating delay incurred at signalized intersections. In addition, a summary of a study that developed procedures to estimate the delay incurred by frontage road vehicles at freeway ramps is included. Finally, this chapter discusses the effects of driveways and unsignalized intersections on traffic operations.

1994 *HIGHWAY CAPACITY MANUAL*

Chapter 11 ("Urban and Suburban Arterials") of the *1994 HCM* (1) contains a procedure for estimating the level of service on arterials. This chapter, however, does not contain procedures for determining the capacity of arterials because arterial capacity is largely dominated by the capacity of the signalized intersections. Therefore, the *HCM* recommends following the procedures in Chapter 9 ("Signalized Intersections") for capacity analysis. Following is a summary of these two chapters. In addition, a summary of Chapter 10 ("Unsignalized Intersections") is provided for estimating capacity and level of service at two-way and four-way stop-controlled intersections.

Chapter 11: Urban and Suburban Arterials

The *HCM* defines arterials as facilities with a primary function of serving through traffic. Arterials may have signalized intersections spaced from 60 m (in downtown areas) to as much as 3.2 km (in other areas). Urban and suburban arterials may include any of the following: multilane divided arterials; multilane undivided arterials; two-lane, two-way arterials; and one-way arterials.

The *HCM* states that traffic operations on arterial streets are primarily influenced by the following three factors:

- the arterial environment
- the interaction between vehicles
- the effect of traffic signals

The arterial environment includes the geometric characteristics such as the number of lanes, median type, speed limit, and spacing between signalized intersections. Also included in the arterial environment are the effects of adjacent land use such as access density, parking availability, and pedestrian activity. All these factors affect drivers' speed along an arterial, which in turn affects the expected level of service for motorists.

Traffic density, the number of trucks and buses, and the percentage of turning movements influence interaction between vehicles. Interaction influences drivers' ability to drive the desired speed. Drivers in a traffic stream caught behind slower moving vehicles will change lanes to maintain their desired speed. However, as the traffic density increases, drivers' ability to maneuver decreases, resulting in a decrease in travel speed. This, in effect, influences the overall level of service for the arterial.

Application Procedure

Of all of the factors influencing arterial operations, signalized intersections have the greatest effect. Signalized intersections largely control the capacity of the arterial. Total intersection delay includes the time that vehicles are stopped and deceleration and acceleration time. The following factors control the delay incurred at signalized intersections: the green time to cycle length ratio on the arterial approach, the quality of traffic signal progression, and the traffic volume.

The *HCM* procedure for estimating arterial level of service is based upon the average travel speed. This measure of effectiveness (MOE) includes the running time along the arterial segments and delay at intersections. The *HCM* defines level of service as average travel speed of all through-vehicles on the arterial. The following paragraphs discuss the methodology to predict average travel speed and estimate arterial level of service.

The *HCM* defines six levels of service that range from primarily free flow (LOS A) to speeds one third to one quarter free flow speed (LOS F). Figure 2-1 summarizes seven steps to predict arterial level of service .

Step 1 involves establishing the location and length of arterial to be considered. Data to be collected in this step include information concerning the physical parameters of the arterial, traffic signal timing information, and traffic volumes.

The purpose of Step 2 is to determine the arterial class and free-flow speed. Three classes are defined based on the arterial's function and design. The functional categories include both principal and minor arterials, and the design categories include suburban design, urban design, and intermediate design.

In Step 3, the arterial section under investigation is divided into segments. A section is composed of a group of segments with similar characteristics (e.g., length, speed limit, and general land use). A segment is a one-directional distance from one signalized intersection to the next.

Steps 4 and 5 involve computing the arterial running time and intersection approach delay. The running time is estimated based on the arterial class, free-flow speed, and average segment length. The intersection parameters required to estimate approach delay include the cycle length, green time to cycle time ratio, volume to capacity ratio, capacity of the through-lanes, and the quality of signal progression.

In Step 6, the average travel speed is computed by segment and over the entire arterial under investigation. An equation is provided to compute the arterial travel speed based upon the following factors: average section length; total running time for all segments; and total intersection approach

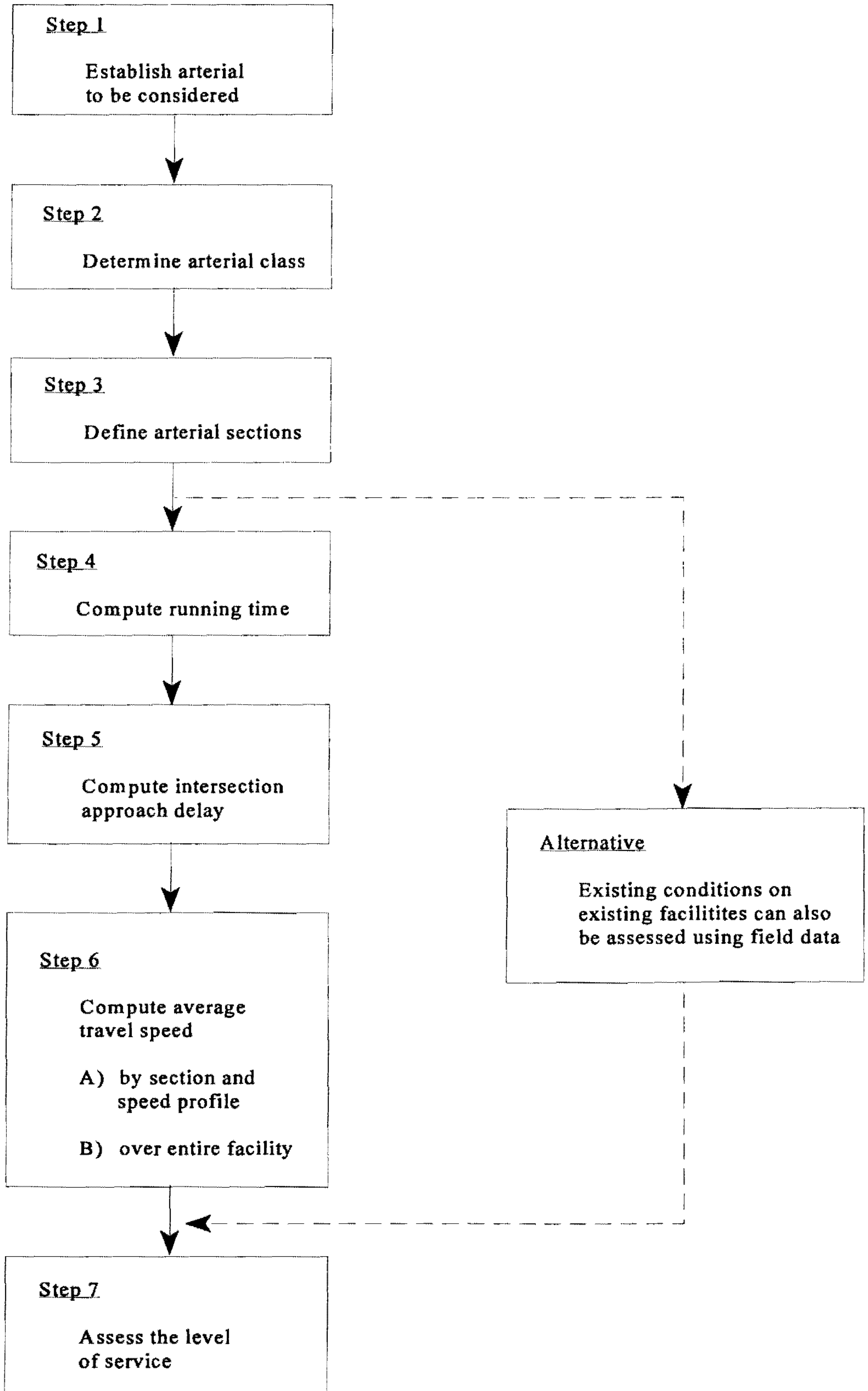

Figure 2-1. ***HCM*** **Arterial Level-of -Service Methodology. (1)**

delay for all signalized intersections in the study area. Finally, Step 7 involves estimating the arterial level of service based on arterial class and average travel speed.

Planning Applications

The 1994 version of the *HCM* (1) includes a procedure for approximating arterial level of service for future conditions. This procedure should only be used for planning applications and should not be used for design or operational analyses. The planning method might be applied in the following scenarios:

- only estimates of level of service are desired
- field data are lacking
- longer planning horizons are used
- individuals with limited transportation planning experience are involved

One major difference between the planning method and the arterial analysis procedure is the treatment of left-turning vehicles at signalized intersections. For the planning method, it is assumed that a left-turn bay will be provided for left-turning vehicles, and a separate signal phase will accommodate left-turning vehicles. This assumption significantly reduces the amount and complexity of data required to estimate approach delay at signalized intersections.

To perform a planning analysis, the inputs (or assumed defaults) required include characteristics of the traffic, roadway, and signals. Table 2-1 summarizes the required data. The following paragraphs define some *HCM* terms.

Planning Analysis Peak Hour Factor (*K*). The planning analysis peak hour factor represents the percentage of average annual daily traffic (AADT) occurring in the peak hour. For planning purposes, many possible peak hours may be appropriate. *K*30 (the 30 highest hourly volumes of the year) is widely accepted as the design hour in nonurban areas. *K*100 approximates

Table 2-1. Required Data for Performing a Planning Analysis.

	Inputs Required
Traffic Characteristics	Annual Average Daily Traffic (AADT) Planning Analysis Peak Hour Factor (*K*) Directional Distribution Factor (D) Peak Hour Factor (PHF) Adjusted Saturation Flow Rate Percentage of Turns from Exclusive Lanes
Roadway Characteristics	Number of Through Lanes (N) Free-Flow Speed Arterial Classification Medians Left-Turn Bays or Exclusive Left-Turn Lanes
Signal Characteristics	Arrival Type Signal Type Cycle Length (C) Effective Green Ratio (g/C)

the typical weekday peak hour during the peak season in developed areas and is frequently used in long-range urban transportation models. *K*200 to 400 is a better representation of a typical peak hour of the year. In many urban areas, general ranges for *K*30, *K*100, and *K*200 to 400 are 8.5 to 11.0 percent, 8.0 to 10.0 percent, and 7.0 to 9.0 percent, respectively. The analyst needs to determine the appropriate peak hour.

Adjusted Saturation Flow Rate. Many factors affect the saturation flow rate per lane (see *HCM*, Chapter 9). For a planning analysis, these adjustments may reasonably be combined and multiplied by the ideal saturation flow rate to determine an adjusted saturation flow rate. Based on the ideal saturation flow rate of 1,900 passenger cars per hour of green time per lane (pcphgpl), a reasonable range for urban arterials during the peak hour is 1,750 to 1,850 pcphgpl.

Percentage of Turns from Exclusive Lanes. Turns from exclusive lanes represent the percentage of vehicles performing left- or right-turning movements at signalized intersections from lanes solely dedicated to turning movements. The planning methodology assumes that left turns are accommodated by separate lanes and phases so that they have minimal effect on through vehicles. Adding the percentage of right turns to the percentage of left turns is reasonable (assuming a left-turn bay or lane) to determine the percentage of turns from exclusive lanes where a separate right-turn lane exists.

Number of Through Lanes. Since significant delays seldom occur in midblock portions of arterials, an important parameter is the number of through and shared right-turn lanes at signalized intersections. However, when significant midblock delays occur or reasonable lane continuity between intersections is not maintained, caution should be used in strictly applying the concept of the number of such lanes.

Free-Flow Speed. For planning purposes, an arterial's free-flow speed should be based on actual studies of the road or on studies of similar roads and should be consistent with arterial classifications. The actual or probable posted speed limit may be used as a surrogate for free-flow speed if comparable roadway free-flow speed studies do not exist.

Medians. Medians are painted, raised, or grassed areas that separate opposing midblock traffic lanes and are wide enough to serve as a refuge for turning vehicles. For planning purposes, the adjusted saturation flow rate may be reduced five percent for roadways that do not have medians.

Left-Turn Bays or Exclusive Left-Turn Lanes. Left-turn bays or lanes are storage areas at signalized intersections to accommodate left-turn movements. The length of these bays or lanes must be sufficient to accommodate left turns so that the through movement is not impeded. For planning purposes, the saturation flow rate should be reduced 20 percent for roadways that do not have left-turn bays at major intersections. (This value is 15 percent in addition to the five percent reduction for a roadway that does not have a median).

Effective Green Ratio (g/C). The parameter g/C is the ratio of the time at signalized intersections allocated for through traffic movement (red clearance minus the startup lost time minus effective green time) to the cycle length (C). An arterial's through g/C for each intersection is desirable; however, for broad planning purposes, a weighted g/C may be appropriate. The weighted g/C of an arterial is the average of the critical intersection through g/C and the average intersection through g/C. For example, if an arterial section has three signalized intersections with effective green time (g/C) of 0.4, 0.7, and 0.7, the critical intersection has a g/C of 0.4 (the lowest g/C); the average intersection has a g/C of 0.6 [(0.4 + 0.7 + 0.7)/3], and the weighted g/C is 0.5 (the average of the critical g/C and the average g/C) [(0.6 + 0.4)/2]. Thus, the weighted g/C takes into account the adverse impact of the critical intersection and the overall quality of flow for the arterial length. Average weighted effective green ratios for arterials vary by road purposes and by areas.

The procedure for conducting a planning analysis consists of the following seven steps:

- Step 1. Convert daily volumes to the planning analysis hour by an appropriate planning analysis peak hour factor (*K*).
- Step 2. Multiply the planning analysis peak hour by the directional distribution factor (D) to obtain hourly directional volumes.
- Step 3. Adjust the hourly directional volumes based on PHF and turns from exclusive lanes to yield estimated through volumes for 15-minute service flow rates.
- Step 4. Calculate the running time on the basis of arterial classification, intersection spacing, and free-flow speed.
- Step 5. Calculate the intersection total delay on the basis of adjusted saturation flow rates, number of lanes (N), arrival type, signal type, cycle length (C), and effective green ratio (g/C) for each intersection.
- Step 6. Calculate the average travel speed using running time and intersection total delay.
- Step 7. Obtain arterial level of service on the basis of the average travel speed.

The quality of results from the planning analysis will depend upon the number of default values used in the analysis procedure. Using site-specific data will result in more accurate results, while using default values will produce only a rough estimate of the level of service. One suggestion for producing a more accurate level of service estimate for planning applications is to use detailed turning movement and signal timing information with projected traffic volumes.

Chapter 9: Signalized Intersections

Chapter 11 of the *HCM* does not offer a procedure for determining the capacity of arterials because arterial capacity is largely dominated by the capacity of the signalized intersections. Therefore, the *HCM* recommends following the procedures outlined in Chapter 9 for a capacity analysis.

Signalized intersections can be evaluated by investigating the capacity and level of service of each approach and/or the level of service of the intersection as a whole. In the *HCM* procedures, capacity is evaluated in terms of the ratio of demand flow rate to capacity (v/c, where capacity is reached when v/c=1.00), while level of service is evaluated on the basis of average stopped delay per vehicle. Both capacity and delay must be considered to evaluate the overall operation of a signalized intersection; however, the two concepts are not as strongly correlated as they are for other facility types. For example, at a given approach to a signalized intersection, an unacceptable level of service (LOS F, delays greater than 60 seconds) may be witnessed, while v/c ratios are acceptable (< 1.00). This situation occurs when a combination of the following conditions exists: the cycle length is long, the lane group in question has a long red time, and/or the signal progression for the subject movements is poor. Furthermore, acceptable delay values (LOS A-E) at an approach to a signalized intersection may be associated with saturated flow (v/c=1.00) if the cycle length is short and/or the signal progression is favorable for the subject movement.

The operational analysis of signalized intersections is complex and requires detailed information. Therefore, the analysis procedure has been divided into the following five modules:

1. *Input Module*: Requires information concerning geometric conditions (number of lanes, lane widths, turn bays, parking conditions, etc.), traffic conditions (traffic volumes by

movement, peak hour factor, percent heavy vehicles, arrival type, etc.), and signalization conditions (cycle length, green times, type of operation, phase plan, etc.).

2. *Volume Adjustment Module*: Peak hour traffic volumes are converted to flow rates for a peak 15-minute analysis period. Lane groups for analysis are defined and lane group flows are adjusted to account for unbalanced lane utilization.

3. *Saturation Flow Rate Module*: Saturation flow rates (flow rates that could be accommodated if the green phase was always available to the approach) are calculated for each lane group.

4. *Capacity Analysis Module*: Results from previous modules are used to compute the capacity and volume/capacity ratios for each lane group and the critical volume/capacity ratio for the overall intersection.

5. *Level-of-Service Module*: The average stopped delay per vehicle is computed for each lane group and is used to predict the intersection level of service.

Procedures for estimating capacity and level of service at signalized intersections could be applied to intersections on frontage roads. However, an important concern regarding this approach is whether the intersection is the controlling factor of capacity on frontage roads. The *HCM* suggests that this is the case for arterials; however, other influences on frontage roads, such as ramps and weaving, may influence intersections or may control capacity and/or level of service.

Chapter 10: Unsignalized Intersections

Chapter 10 of the *HCM* presents specific procedures for estimating the capacity and level of service for unsignalized intersections. Separate procedures are provided for the analysis of two-way and all-way stop-controlled intersections. These procedures will be updated in the next revision of the *HCM*. Updates to *HCM*, Chapter 10 is expected to be available in late 1997.

Two-Way Stop-Controlled Intersections

The capacity of a two-way stop-controlled intersection is based on the following three factors: the distribution of gaps in traffic streams on the major street, the size of gap required by the driver on the minor street controlled approach, and follow-up time (i.e., time between departure of vehicles in queue on minor street) required by each driver in queue. The *HCM* procedures for estimating capacity are based upon the assumption that the gaps on the major street are randomly distributed. Therefore, these procedures will be less reliable for situations in which major street traffic streams travel in platoons (i.e., signalized intersection spacing less than 1.6 km).

Following are the procedures outlined in the *HCM* for estimating the level of service at two-way stop-controlled intersections:

1. Define existing geometric and traffic conditions for the intersection under study.
2. Determine the conflicting traffic through which each minor street movement, and the major street left turn, must cross.
3. Determine the size of the gap in the conflicting traffic stream needed by vehicles in each movement crossing a conflicting traffic stream.
4. Determine the capacity of the gaps in the major traffic stream to accommodate each of the subject movements that will use these gaps.
5. Adjust the calculated capacities to account for impedance and the use of shared lanes.
6 Estimate the average total delay for each of the subject movements and determine the level of service for each movement and for the intersection.

Impedance from other minor street flows may be caused by shared use of a lane by more than one movement (i.e., left, right, and/or through) or conflicting turning movements. For conflicting turning movements, it is assumed that gaps will be used by vehicles in the following priority order: (1) right turns from minor street; (2) left turns from major street; (3) through movements from minor street; and (4) left turns from minor street. The *HCM* provides techniques for estimating the effects of these impedances on intersection capacity.

The *HCM* defines *potential intersection capacity* as the "capacity under ideal conditions for a specific subject movement." This capacity assumes that the intersections under investigation will not be blocked by conflicting traffic flows, each minor street movement is provided with a separate lane, and there are no impedances from other movements. This capacity can be estimated based on the conflicting traffic volume on the major street and the critical gap acceptance.

The capacity of the minor movement is estimated based on gap acceptance theory. Potential capacity is calculated using the following formula:

$$c_{p,x} = \frac{3600}{t_f} e^{\frac{-[\Sigma V_{c,y}]t_0}{3600}} \qquad [2\text{-}1]$$

where:

$c_{p,x}$ = potential capacity of minor movement x, pcph

$V_{c,y}$ = volume of traffic in conflicting stream y, vph

t_0 = $t_g - (t_f/2)$

t_g = critical gap, sec

t_f = follow-up time, sec

In estimating potential capacity, it is assumed that each movement is provided with an exclusive lane; however, this is often not the case. For example, left-turning, through, and right-turning vehicles may be required to use the same lane. Therefore, the *HCM* provides a method of estimating *shared lane capacity*. This capacity is based on the volume and movement capacity for each movement sharing the lane.

The level-of-service criteria for two-way stop-controlled intersections is based upon average total delay. The total delay is calculated using the following formula:

$$D = \frac{3600}{c_{m,x}} + 900T \left[\frac{V_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{V_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{V_x}{c_{m,x}}\right)}{450T}}\,\right] \qquad [2\text{-}2]$$

where:

- D = average total delay, sec/veh
- V_x = volume for movement x, vph
- $c_{m,x}$ = capacity of movement x, vph
- T = analysis period, h (for a 15-min period, use T = 0.25)

Table 2-2 shows criteria for estimating the level of service at two-way stop-controlled intersections.

Table 2-2. Level-of-Service Criteria for Non-Signalized Intersections.

Level of Service	Average Total Delay (sec/veh)
A	≤ 5
B	> 5 and ≤ 10
C	> 10 and ≤ 20
D	> 20 and ≤ 30
E	> 30 and ≤ 45
F	> 45

All-Way Stop-Controlled Intersections

The *HCM* contains a methodology for estimating capacity and level of service for all-way stop-controlled intersections. In the procedures, each intersection approach is analyzed independently. The approach under study is called the *subject* approach. The approach on the opposite side is called the *opposing* approach, and the cross road approaches are identified as *conflicting* approaches.

The level-of-service criteria is based on approach total delay. The recommended steps for determining the capacity and level of service are outlined as follows:

1. Determine input data.
2. Estimate approach capacity.
3. Estimate approach total delay.
4. Determine level of service.
5. Check range of model validity.

The following equations are used for calculating approach capacity and approach total delay:

$$c = 1000V_{ps} + 700V_{po} + 200L_s - 100L_o - 300LT_{po} + 200RT_{po} - 300LT_{pc} + 300RT_{pc} \qquad [2\text{-}3]$$

$$V_s = V_{sl} + V_{st} + V_{sr} \qquad [2\text{-}4]$$

$$V_o = V_{ol} + V_{ot} + V_{or} \qquad [2\text{-}5]$$

$$V_c = V_{c1l} + V_{c1t} + V_{c1r} + V_{c2l} + V_{c2t} + V_{c2r} \qquad [2\text{-}6]$$

$$V_{po} = \frac{V_o}{V_s + V_o + V_c} \qquad [2\text{-}7]$$

$$V_{ps} = \frac{V_s}{V_s + V_o + V_c} \qquad [2\text{-}8]$$

$$LT_{po} = \frac{V_{ol}}{V_o} \qquad [2\text{-}9]$$

$$LT_{po} = \frac{V_{c1l} + V_{c2l}}{V_c} \qquad [2\text{-}10]$$

$$RT_{po} = \frac{V_{or}}{V_o} \qquad [2\text{-}11]$$

$$RT_{pc} = \frac{V_{c1r} + V_{c2r}}{V_c} \qquad [2\text{-}12]$$

$$D = e^{3.8(V_S / C)} \qquad [2\text{-}13]$$

where:

- c = capacity of subject approach, vph
- D = average total delay on subject approach, sec/veh
- L_s = number of lanes on subject approach
- L_o = number of lanes on opposing approach
- V_S = volume on subject approach, vph
- V_O = volume on opposing approach, vph
- V_C = volume on conflicting approach, vph
- LT_{po} = proportion of volume on opposing approach turning left
- LT_{pc} = proportion of traffic on conflicting approaches turning left
- RT_{po} = proportion of traffic on opposing approach turning right
- RT_{pc} = proportion of traffic on conflicting approaches turning right
- V_{ps} = proportion of intersection volume on subject approach
- V_{po} = proportion of intersection volume on opposing approach

Figure 2-2 illustrates the definitions of the variables for volumes used in the above equations. Table 2-2 shows the level-of-service criteria for all-way stop-control, which is the same criteria used for two-way stop-control.

The capacity and delay equations given above were developed based on specific ranges of variables, and the equations are only valid for these ranges. Table 2-3 lists the ranges for which the equations are valid. The analyst should check the ranges in this table each time a calculation is made.

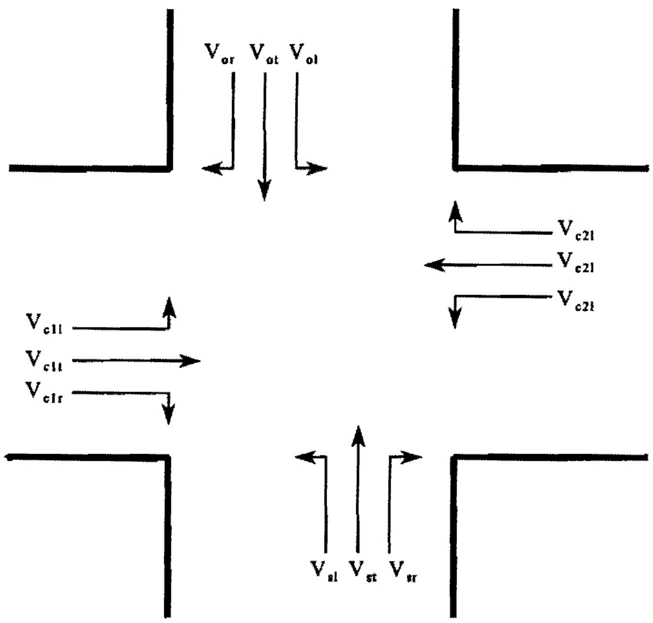

Figure 2-2. Variables for Capacity Estimation at All-Way Stop-Controlled Intersection.

Table 2-3. Range of Input Variable for Which Delay and Capacity Equations Are Valid.

Variable	Minimum Value	Maximum Value
Volume Distribution, proportion		
Subject approach	0.20	0.50
Opposing approach	0.00	0.50
Conflicting approach	0.20	0.80
Number of Approach Lanes		
Subject approach	1	3
Opposing approach	0	3
Conflicting approach	1	5
Proportion of Left Turns on		
Opposing approach	0.00	0.36
Conflicting approach	0.00	0.71
Proportion of Right Turns on		
Opposing approach	0.00	0.62
Conflicting approach	0.00	0.52

DELAY AT RAMPS

A 1986 study by Gattis, et al. (2) was conducted to investigate the delay incurred by frontage road traffic at ramps. The objective of the study was to develop procedures to estimate the delay to frontage road vehicles at exit and entrance ramps on two-way frontage roads. Field studies were conducted at four frontage road sites located in medium-sized towns in Texas. The field sites included three two-way frontage road sites and one one-way frontage road site (for comparison purposes). The following four cases were investigated in the study (see Figures 2-3 and 2-4):

- Case 1: one-way frontage road intersection with exit ramp converging movement (used for comparison with two-way frontage road delay)
- Case 2: two-way frontage road intersection with exit ramp, converging movement
- Case 3: two-way frontage road intersection with exit ramp, opposing movement
- Case 4: two-way frontage road intersection with entrance ramp, opposing movement

The data collection efforts involved tracking both ramp vehicles and frontage road vehicles as they traveled through the area of the ramp-frontage road intersection. This approach allowed the researchers to estimate the delay to frontage road vehicles by comparing the travel time of vehicles that yielded to ramp traffic with the travel times of those vehicles that did not yield.

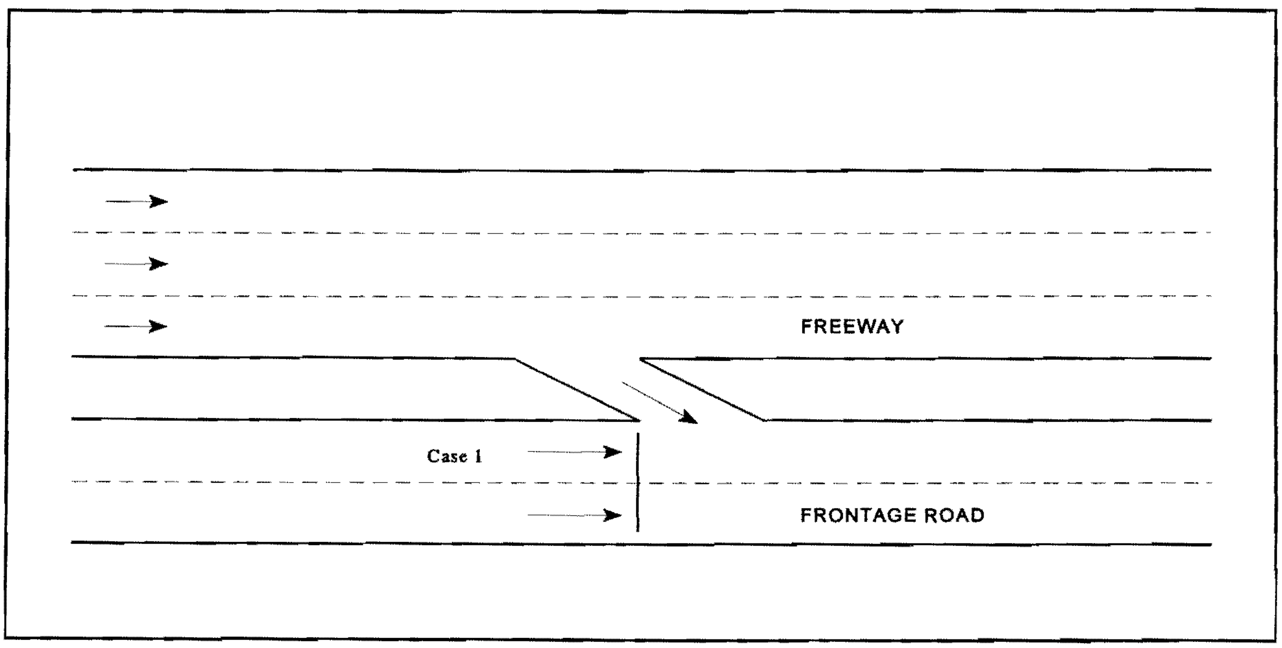

Figure 2-3. Computing Delay at Exit Ramps on One-Way Frontage Roads.

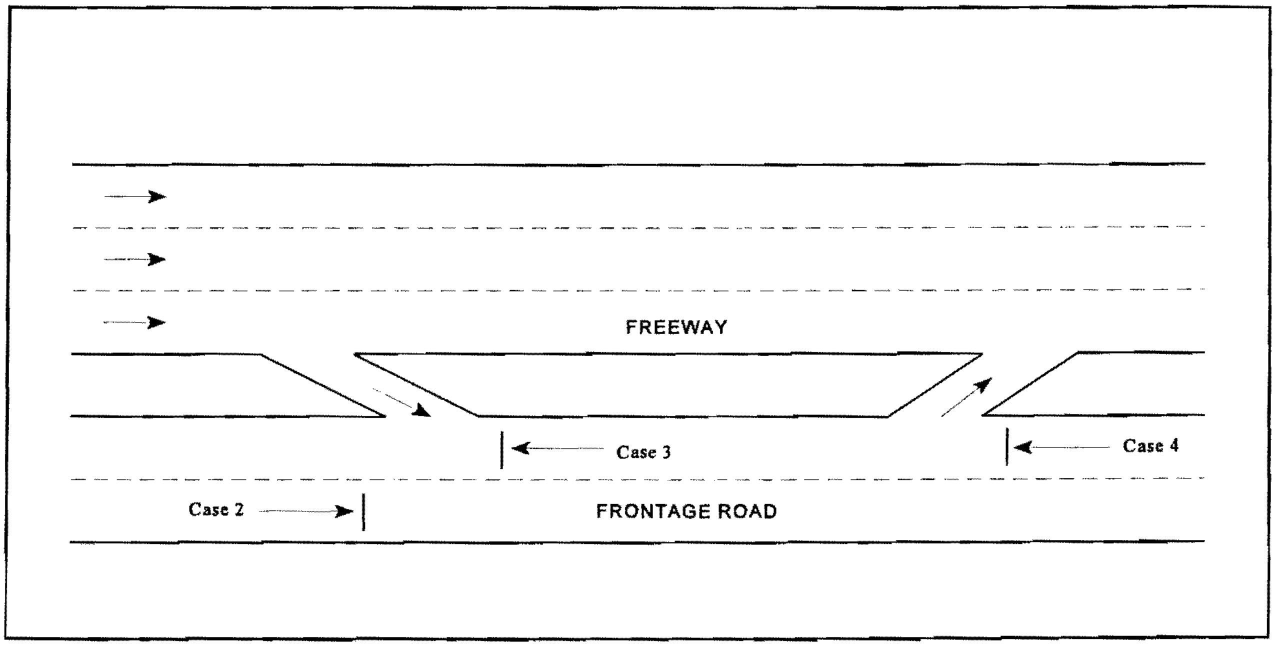

Figure 2-4. Computing Delay at Ramps on Two-Way Frontage Roads.

Theoretical models were developed to predict delay by assuming the ramp traffic arrivals could be described using the Poisson process. In addition, the interaction of frontage road traffic with ramp traffic was viewed as a queuing system. When the arrival time headway of ramp vehicles is less than the required service time for the frontage road vehicles, a queue is formed on the frontage road. Therefore, the time that a driver on the frontage road waits for an adequate headway in the ramp traffic is the time waiting to be "served." For lighter ramp volumes, this service time might be zero (i.e., the frontage road vehicles do not have to yield at the ramp-frontage road intersection). However, as ramp volumes and frontage road volumes increase, the service time increases, thereby increasing the delay at the ramp.

To estimate the queuing delay at the ramp, the researchers assumed that the frontage road capacity at the ramp was the same as the service rate. Therefore, the potential capacity for frontage road vehicles at the intersection was equal to the part of the total time period with adequate headways for frontage road vehicles to proceed divided by the headway at which frontage road vehicles would follow each other through the intersection. By knowing the frontage road flow rate (a) and the service rate (u), the average queuing system delay (W) in seconds per vehicle was calculated using the following equation:

$$W = 1/(u - a) \qquad [2\text{-}14]$$

The above equation can be used to predict average queuing delay; however, the researchers recognized that non-queuing sources of delay also existed. For example, the delay to frontage road vehicles at ramps also includes the time lost while resuming to normal speed after yielding. Therefore, to develop models for predicting total frontage road delay, the field-measured total delay was regressed against the queuing delay.

Using the assumptions discussed above in combination with results from the field studies, the following equations were developed to predict frontage road delay for each of the four case studies:

Case 1: Exit Ramp, One-Way Converging

$$C_R = N(1858 - 1.5259\,Q_R) \qquad [2\text{-}15]$$

$$W = 1 / (u - a) \qquad [2\text{-}16]$$

$$D_R = -0.0719 + 1.0922\,W \qquad (R^2 = 0.32) \qquad [2\text{-}17]$$

Case 2: Exit Ramp, Two-Way Converging

$$C_R = 1724 - 1.6120\,Q_R \qquad [2\text{-}18]$$

$$W = 1 / (u - a) \qquad [2\text{-}19]$$

$$D_R = -0.0719 + 1.0922\,W \qquad (R^2 = 0.32) \qquad [2\text{-}20]$$

Case 3: Exit Ramp, Two-Way Opposing

$$C_R = 1444 - 1.6564\,Q_R \qquad [2\text{-}21]$$

$$W = 1 / (u - a) \qquad [2\text{-}22]$$

$$D_R = -1.6451 + 1.7785\,W \qquad (R^2 = 0.83) \qquad [2\text{-}23]$$

Case 4: Entrance Ramp, Two-Way Opposing

$$C_R = 1535 - 1.3852\,Q_R \qquad [2\text{-}24]$$

$$W = 1 / (u - a) \qquad [2\text{-}25]$$

$$D_R = 0.0538 + 1.3027\ W \qquad (R^2 = 0.73) \qquad [2\text{-}26]$$

where:

- C_R = frontage road capacity per direction at a ramp, 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 which approach the entrance ramp from the converging direction, whether they enter the ramp or not)
- u = service rate, veh/sec (C / 3600)
- a = frontage road flow rate, veh/sec (volume / 3600)
- N = number of frontage road lanes

In using the equations to predict frontage road delay at ramps, the first step is to calculate the available frontage road capacity per lane, C_R. Second, the queuing delay, W, is calculated. Third, given W, the total delay per vehicle, D_R, is calculated.

Based upon the coefficient of determination (R^2), the equations for Case 3 were the most reliable. Nevertheless, the researchers stated that while the relationships for the other cases were not as strong, their similarity with the Case 3 models shows that they are fairly reliable.

ACCESS DENSITY

"The spacing of access for driveways and streets is an important element in the planning, design, and operation of roadways. Access points are the main source of accidents and congestion" (3). The presence of too many access points increases the number of conflict points, which reduces safety and increases travel times, delay, and vehicle emissions. Garber and White (4) state that as the density of intersections or businesses increases, the accident rates increase at nearly the same rate.

Chapter 7 of the *HCM* (1) states that the number of access points along the right side of the roadway is an important influence on free-flow speed for multilane highways. It also states that drivers adjust their speed based on the mere existence of access points. The procedures in Chapter 7 maintain that for every 16 access points per kilometer that affect a given direction of travel on a multilane highway, travel speed will be reduced by 4 kilometers per hour (see Table 2-4). Multilane highways are similar to urban arterials in some respects. However, multilane highways lack the regularity of traffic signals and tend to have greater control on the number of access points per mile. Also, multilane highway design standards are generally higher than arterial design standards, and the speed limits on multilane highways are often 8 to 24 km/h greater than speed limits on urban arterials.

Table 2-4. Access Point Density Adjustment.

ACCESS POINTS PER KILOMETER	REDUCTION IN FREE-FLOW SPEED (km/h)
0	0.0
16	4.0
32	8.1
48	12.1
64 or more	16.1

Because frontage roads are more similar to urban arterials than to multilane highways, this report examines the effects of access density on frontage road travel time and level of service. Access density on frontage roads will be measured as the total of the number of unsignalized intersections and the number of driveways per length of frontage road as the following equation shows:

$$\text{Access Density, acs/km} = \frac{\text{Num of Unsignalized Intersections} + \text{Num of Driveways}}{\text{Length, km}} \qquad [2\text{-}27]$$

CHAPTER 3
STUDY DESIGN

Several measures of effectiveness are available to evaluate the operations on frontage roads, including the traditional measures of speed and delay. Speed is used in the current arterial street procedure in the *HCM* (1) and is also easy to explain and understand. Because frontage road operations are similar to arterial operations, speed was selected as the proposed measure of effectiveness for this study.

In developing a method for predicting speed for different frontage road configurations, travel time studies were conducted. Travel time studies provide data on the time it takes to traverse a section of roadway under various conditions. Travel speed can be determined by dividing the length of the study section by the travel time. The level of service can then be estimated based upon the travel speed.

Several travel time studies were conducted at existing one-way and two-way frontage road sites in Texas. Following are discussions on the site selection, data collection, data reduction, and data analysis efforts.

SITE SELECTION

An exhaustive search was made to find one-way and two-way frontage road sites appropriate for this study. The selected field sites included a range of characteristics (e.g., volumes, intersection spacings, ramp locations, access densities, etc.) so that the researchers could analyze the effects of these characteristics on travel time. Site selection was based on the following criteria:

- continuous frontage road section at least 1.6 km in length
- a range of intersection types (cross roads, exit ramps, and entrance ramps)
- high volumes during the peak periods

- a range of access densities
- no construction activity on the site
- minimum number of horizontal and vertical curves

These factors were considered in an extensive statewide search for appropriate sites. Consultation of county maps yielded a preliminary list of possible study sites. TxDOT personnel in the study site districts were requested to provide average daily volumes and other information related to the defined site selection criteria. Based upon those responses, several cities were chosen for data collection on the basis that they met most or all of the requirements. Each city had several potential sites, and the sites were investigated individually when the research team traveled there for data collection. Figure 3-1 shows the cities included in the field studies.

Each frontage road site was typically 1.6 km to 6.4 km in length and contained several intersections with exit ramps, entrance ramps, and cross roads. Field data were collected at 20 one-way frontage road sites and nine two-way frontage road sites. Table 3-1 contains descriptions for the one-way field sites, and Tables 3-2 and 3-3 contain descriptions for the two-way field sites. The data in Tables 3-2 to 3-3 are separated into two categories: *with* (frontage road vehicles traveling in the same direction of freeway vehicles) and *opposing* (frontage road vehicles traveling in opposite direction of freeway vehicles). Figure 3-2 illustrates the use of these terms.

DATA COLLECTION

To identify factors affecting operations on frontage roads, travel time studies were conducted at the selected field sites. The travel time studies involved driving the field sites in a test car while monitoring travel time and speed. While conducting these studies, the data collection personnel also collected information concerning site characteristics and obtained traffic volumes at various locations. At selected frontage road sites, video data were collected at freeway entrance and exit ramps to estimate the delay incurred by frontage road vehicles at ramps. Following are discussions of activities performed during the data collection efforts.

Figure 3-1. Locations of One-Way and Two-Way Frontage Road Field Study Sites.

Table 3-1. Sites Selected for One-Way Frontage Road Travel Time Evaluation.

Site Num	City/ Fwy	Location	Dir.	Date/Day of Study	Time of Study	Num of Runs	Length of Site (km)	Num of Lanes	Access Density (acs/km)	Signals Per Kilometer	Speed Limit (km/h)
1	Dallas/ IH 35E	Beltline - Valley View	South	5/15/95 Mon.	3:45 pm - 6:45 pm	8	3.5	2-3	19	2.3	56
2	Dallas/ IH 35E	Valley View - Beltline	North	5/15/95 Tues.	3:30 pm - 6:30 pm	8	3.5	2-3	22	2.3	56
3	Dallas/ IH 635	Hillcrest - Noel	West	5/16/95 Tues.	3:00 pm - 6:15 pm	12	3.1	2-3	14	2.6	64
4	Dallas/ IH 635	Inwood - Hillcrest	East	5/16/95 Tues.	3:00 pm - 6:15 pm	12	3.2	2-3	16	2.4	64
5	Dallas/ SH 183	Story - Carl	East	5/16/95 Tues.	6:30 am - 10:00 am	8	4.3	2	19	1.8	56
6	Dallas/ SH 183	Carl - Story	West	5/16/95 Tues.	6:30 am - 10:00 am	8	4.3	2	16	1.8	56
7	Dallas/ SH 360	Avenue J - Randol Mill	South	5/17/95 Wed.	3:00 pm - 6:30 pm	13	2.1	2	11	3.7	64
8	Dallas/ SH 360	Randol Mill - Avenue J	North	5/17/95 Wed.	3:00 pm - 6:30 pm	13	2.1	2	13	3.7	64
9	Houston/ US 290	Hollister - Gessner	West	4/26/95 Wed.	6:30 am - 10:00 am	10	4.0	2-3	5	1.9	64, 81
10	Houston/ US 290	Gessner - Hollister	East	4/26/95 Wed.	6:30 am - 10:00 am	10	4.0	2-3	11	1.9	64, 81

Table 3-1 (continued). Sites Selected for One-Way Frontage Road Travel Time Evaluation.

Site Num	City/ Fwy	Location	Dir.	Date/Day of Study	Time of Study	Num of Runs	Length of Site (km)	Num of Lanes	Access Density (acs/km)	Signal Per Kilometer	Speed Limit (km/h)
11	Houston/ US 290	Mangum - 43rd	West	4/26/95 Wed.	3:00 pm - 6:30 pm	12	3.4	2-3	14	2.3	56, 64
12	Houston/ US 290	43rd - Mangum	East	4/26/95 Wed.	3:00 pm - 6:30 pm	12	3.5	2-3	10	2.3	64
13	San Angelo/ LP 306	Knicker - Sherwood	West	6/19/95 Mon.	6:45 am - 8:45 am	8	3.7	2	8	2.1	72
14	San Angelo/ LP 306	Sherwood - Knicker	East	6/19/95 Mon.	6:45 am - 8:45 am	8	3.7	2	8	2.1	72
15	Lubbock/ LP 289	University - Slide	West	6/21/95 Wed.	7:00 am - 9:00 am	13	5.0	2	10	1.6	72, 89
16	Lubbock/ LP 289	Slide - University	East	6/21/95 Wed.	7:00 am - 9:00 am	5	5.0	2	12	1.6	72, 89
17	Amarillo/ IH 40	Wolfin - Olsen	West	6/22/95 Thurs.	6:45 am - 9:45 am	12	2.6	2	27	3.1	56, 72
18	Amarillo/ IH 40	Olsen - Wolfin	East	6/22/95 Thrus.	6:45 am - 9:45 am	12	2.6	2	18	3.1	56, 72
19	Amarillo/ IH 27	Moss - Western	South	6/23/95 Fri.	3:30 pm - 7:00 pm	12	3.9	2	21	2.1	72
20	Amarillo/ IH 27	Western - Parker	North	6/23/95 Fri.	3:30 pm - 7:00 pm	12	4.0	2	19	1.9	56, 72

Table 3-2. Sites Selected for Two-Way Frontage Road Travel Time Evaluation.

Site Num	City	Fwy	Location	Side of Fwy	Direction of Travel		Date/Day of Study	Time of Study
					With[a]	*Opposing*[b]		
21	Gainesville	IH 35	853 m South of FM 372 - 1067 m North of FM 1202	West	South	North	07/20/95 Thurs.	11:00 am - 1:30 pm
22	Sulphur Springs	IH 30	Loop 301 - South Broadway	North	West	East	07/21/95 Fri.	3:00 pm - 6:00 pm
23	Sulphur Springs	IH 30	2012 m East of Broadway - 1250 m West of Broadway	South	East	West	07/22/95 Sat.	7:00 am - 9:00 am
24	New Braunfels	IH 35	1524 m South of SH 46 - 640 m North of SH 46	East	North	South	08/09/95 Wed.	7:00 am - 10:00 am
25	New Braunfels	IH 35	Schmidt Avenue - FM 725	East	North	South	08/09/95 Wed.	3:30 pm - 5:30 pm
26	Hillsboro	IH 35	732 mSouth of Corsicana - 396 m North of Brandon	West	South	North	08/11/95 Fri.	12:00 pm - 2:00 pm
27	Hillsboro	IH 35	1097 m South of Corsicana - FM 286	East	North	South	08/22/95 Tues.	4:00 pm - 6:30 pm
28	Huntsville	IH 35	518 m South of FM 1374 - SH 75	West	South	North	08/23/95 Wed.	2:30 pm - 5:00 pm
29	Huntsville	IH 35	427 m South of FM 1374 - SH 75	East	North	South	08/11/95 Fri.	11:30 am - 1:30 am

[a] Traveling in same direction of freeway traffic (see Figure 3-2).
[b] Traveling in opposite direction of freeway traffic (see Figure 3-2).

Table 3-3. Sites Selected for Two-Way Frontage Road Travel Time Evaluation.

Site Num	City	Fwy	Number of Runs		Length of Site (km)	Access Density (acs/km)	Signals Per Kilometer	Speed Limit (km/h)
			With[a]	*Opposing*[b]				
21	Gainesville	IH 35	8	8	3.4	7	0	64, 81
22	Sulphur Springs	IH 30	10	10	4.2	8	0.6	81
23	Sulphur Springs	IH 30	8	8	3.2	11	0.8	81, 89
24	New Braunfels	IH 35	11	8	2.3	9	1.1	64, 72
25	New Braunfels	IH 35	9	9	3.4	14	1.6	64
26	Hillsboro	IH 35	8	7	2.1	15	1.3	56, 72
27	Hillsboro	IH 35	8	8	2.1	9	1.3	56, 72
28	Huntsville	IH 35	8	8	6.3	7	1.3	64, 81
29	Huntsville	IH 35	8	8	6.4	9	1.3	64, 81

[a] Traveling in same direction of freeway traffic. (see Figure 3-2).
[b] Traveling in opposite direction of freeway traffic. (see Figure 3-2).

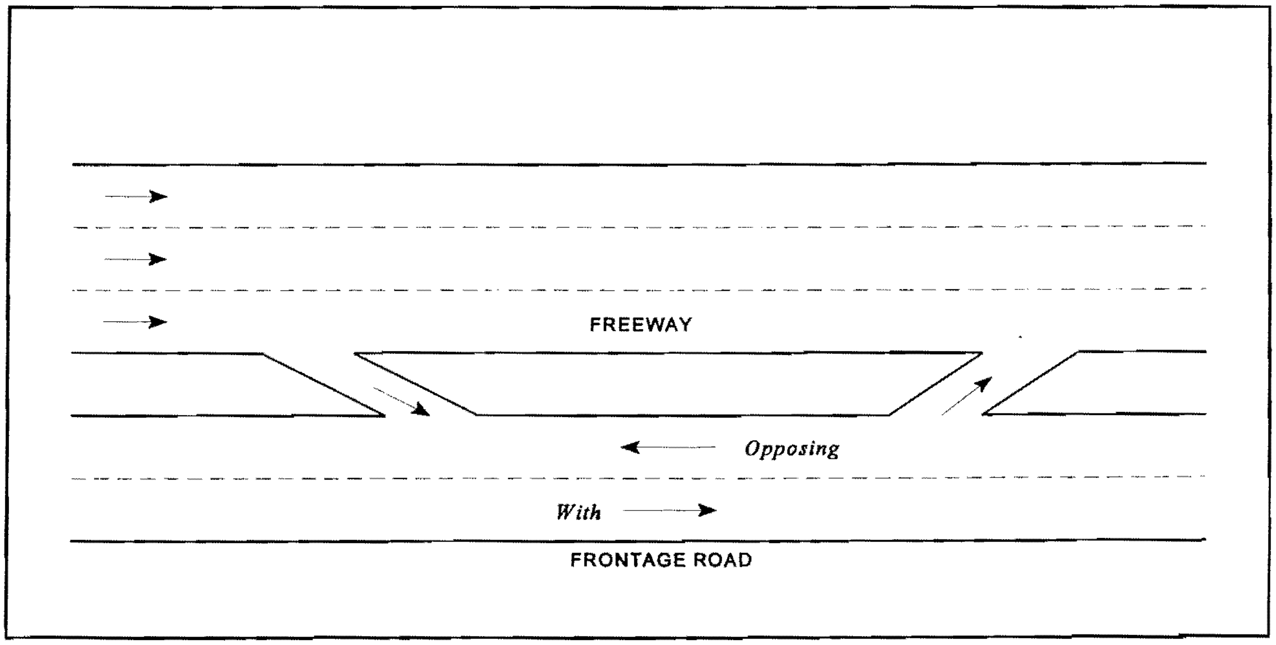

Figure 3-2. Use of the Terms *With* and *Opposing* on Two-Way Frontage Roads.

Travel Time Studies

Travel Time Data

The device used to collect travel time information was an in-vehicle Digital Measuring Instrument (DMI). The DMI was capable of recording the distance traveled, travel time, and speed. The instrument was calibrated before each data collection trip by driving a local test site and measuring the distance between two markers of known distance.

Before beginning the travel time runs at each site, starting and ending points on the frontage road were selected. These points were typically at crossroad intersections. Using a laptop computer and software developed at the Texas Transportation Institute, distance, travel time, and speed data from the DMI were recorded in one-half second increments.

Two persons were used to conduct the travel time runs: one to drive the test vehicle and one to operate the data collection equipment. The driver of the vehicle approached the beginning of the site at a normal speed while the passenger prepared the DMI and the laptop computer. As the vehicle crossed over the starting point, the data recording began. While driving through the site, the driver of the test vehicle attempted to drive at the same speed as the surrounding traffic. On one-way frontage roads, the driver also attempted to pass as many vehicles as passed the test vehicle.

For most one-way frontage road sites, DMI data were collected on both sides of the freeway, essentially collecting two sites at one time. After completing a travel time run on one side of the freeway, the driver of the test vehicle crossed the freeway and collected data on the other side.

Travel time data were collected in both directions along the two-way frontage road sites. After completing a travel time run in one direction, the driver of the test vehicle made a U-turn and collected data for the opposing direction.

Travel time runs typically began during the off-peak period and continued through the peak period. This period, usually between 2 and 3½ hours, typically yielded between 8 and 12 travel time runs per site.

Site Information

While the travel time runs were being conducted, other personnel collected site information. Using an additional DMI, the location of the freeway ramps and crossroads were recorded with respect to the starting point of the travel time runs. In addition, the location of each driveway was recorded along with the type of development associated with the driveway. Other information obtained included the speed limits, location and lengths of turning bays at crossroads, type of control used at each crossroad (e.g., no control, stop sign, or traffic signal), and signal timing information at each signalized intersection.

After the site information was collected, detailed sketches of each site were made. By studying the travel time data for different frontage road sections, the site information could be used to aid the researchers in determining which factors significantly affected travel time.

Volume Data

Mobile traffic counters with pneumatic tubes were used to record traffic volumes at selected locations within each site. These electronic counters recorded unsupervised traffic counts over a given period. At each field site, traffic volumes were recorded in five-minute increments for the periods that the travel time runs were made.

For the one-way frontage road sites, traffic counters were placed between signalized intersections, typically at each exit ramp (see Figure 3-3). Two tubes were used at each exit ramp; one tube counted the exit ramp traffic while the second tube counted the frontage road traffic. Occasionally, the research team placed the tubes at a midblock location or at an entrance ramp instead of (or in addition to) the exit ramp.

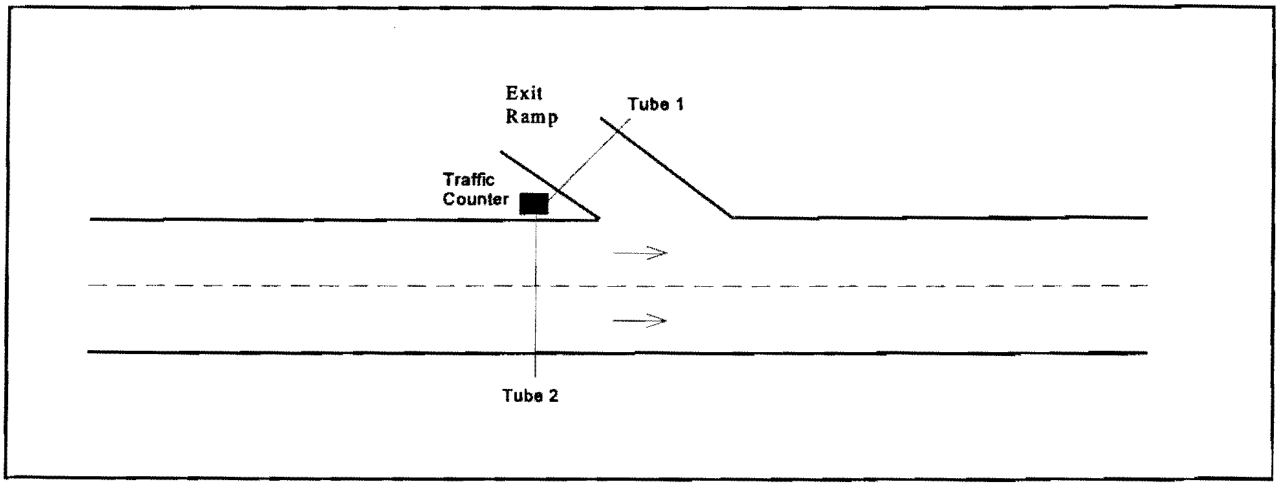

Figure 3-3. Typical Road Tube Configuration at One-Way Frontage Road Site.

For the two-way frontage road sites, traffic counters were placed at all freeway exit and entrance ramps. Counter locations were selected so that volumes could be obtained for the ramp and for both directions along the frontage road. Figure 3-4 illustrates a typical road tube configuration used at a two-way frontage road site.

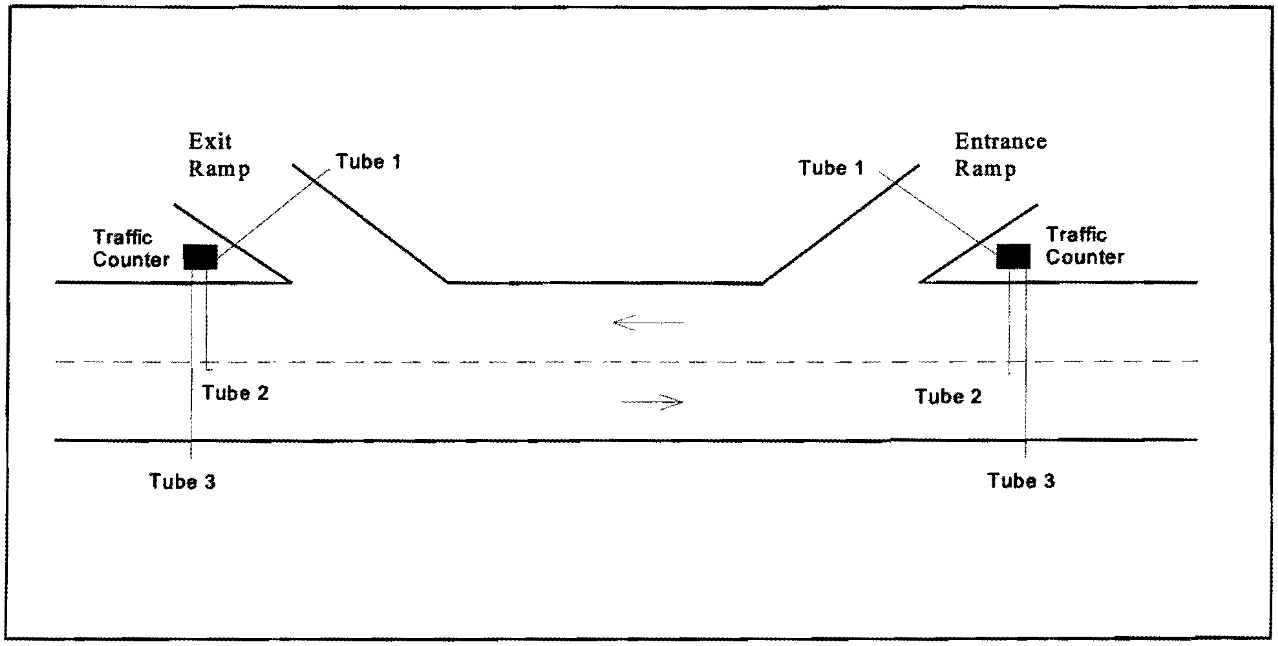

Figure 3-4. Typical Road Tube Configuration at Two-Way Frontage Road Site.

At each field site, the counters were programmed to turn on automatically and to begin counting approximately one hour before the intended start of the travel time runs. After the travel time runs were complete, the volume data were extracted from the counters and stored on a laptop computer.

Delay at Ramps

Gattis, et al. (2) developed procedures for estimating the delay to frontage road vehicles at freeway ramps on one-way and two-way frontage roads (see Chapter 2 for a summary of this study). To evaluate these procedures, field data were collected at two one-way frontage road sites and four two-way frontage road sites. Three of the two-way frontage road study sites included a freeway exit ramp and one site included a freeway entrance ramp. Each study site consisted of a frontage road section that extended a given distance upstream and downstream of a ramp. Table 3-4 provides descriptions of the one-way and two-way frontage road sites.

Table 3-4. Two-Way Frontage Road Sites Selected for Delay-at-Ramps Evaluation.

Site	City	Frontage Road	Location	Interchange Quadrant	Ramp Type	Day/Date of Study	Time of Study
A	Houston	One-Way	IH 610 @ San Felipe	Northwest	Exit	7/1/96 Mon.	7:00 - 9:00 am
B	Houston	One-Way	IH 610 @ Evergreen	Northwest	Exit	7/1/96 Mon.	4:00 - 6:00 pm
C	Huntsville	Two-Way	IH 45 @ SH 30	Northeast	Entrance	8/22/95 Tues.	3:30 - 6:00 pm
D	Huntsville	Two-Way	IH 45 @ SH 30	Northwest	Exit	8/23/95 Wed.	2:30 - 4:30 pm
E	New Braunfels	Two-Way	IH 35 @ Walnut	Southeast	Exit	8/9/95 Wed.	3:00 - 6:00 pm
F	New Braunfels	Two-Way	IH 35 @ Lp 337	Southeast	Exit	8/9/95 Wed.	7:00 - 10:00 am

The objectives of the field studies were to measure the travel times of frontage road vehicles passing through the frontage road-ramp intersection area and calculate the frontage road delay for various traffic volumes. Two cameras were used to record traffic operations at each site; one recorded operations upstream of the ramp and the other recorded operations downstream of the ramp. These views were used to calculate the travel time of frontage road vehicles passing through the frontage road/ramp intersection area. To measure travel times, the team marked points upstream and downstream of the ramp by waving an orange flag during the video taping. To calculate the total delay to frontage road vehicles due to the ramp, the upstream point was placed in areas before vehicles began to decelerate, and the downstream point was placed in areas after vehicles had accelerated to the desired speed. The team also measured the locations of driveways and crossroads with respect to the ramp.

After viewing the video tapes of the two-way frontage road sites, technicians discovered that Site F (see Table 3-3) could not be used because of a high volume driveway near the exit ramp. The location of this driveway affected the travel times (and delays) of vehicles traveling through the frontage road-ramp intersection area. Therefore, only the data at Sites C, D, and E were used for the two-way frontage road field study.

DATA REDUCTION

For this study, a frontage road *section* was defined as that area of a frontage road under investigation. Sections were composed of *segments*, and segments were divided into *links*. A *segment* was defined as an area of a frontage road that contained similar frontage road and traffic characteristics (such as number of lanes, number of driveways, type of development, and traffic volume). Segments are typically bound by either signalized intersections or stop-controlled intersections. The term *link* describes a length of a frontage road between two nodes (i.e., logical break points such as ramps or intersections). Figure 3-5 illustrates the use of the terms *section, segment, link,* and *node.* To study the traffic operations at the existing two-way frontage road sites, the researchers divided each frontage road segment into links and analyzed the operations on each link.

Travel Time Studies

Data reduction for the travel time studies involved retrieving information from the traffic counters and from the DMI files. For the two-way frontage road sites, volume counts were recorded at each freeway ramp for both the ramp traffic and the frontage road traffic (in each direction). This resulted in the researchers obtaining volume counts for all ramps and frontage road links. From the travel time runs, the times that the test vehicle was on a specific link could be determined and matched with the associated five-minute volume counts. For the one-way frontage road sites, traffic volumes were only obtained at selected locations and therefore were not available for each link.

Two databases were created to compile the information for the one-way and two-way frontage road sites. The statistical analysis software, *SAS,* was used to develop the databases. The databases were created using data from the site sketches and from the reduced volume and travel time files. The information in the databases was coded by link for each field site.

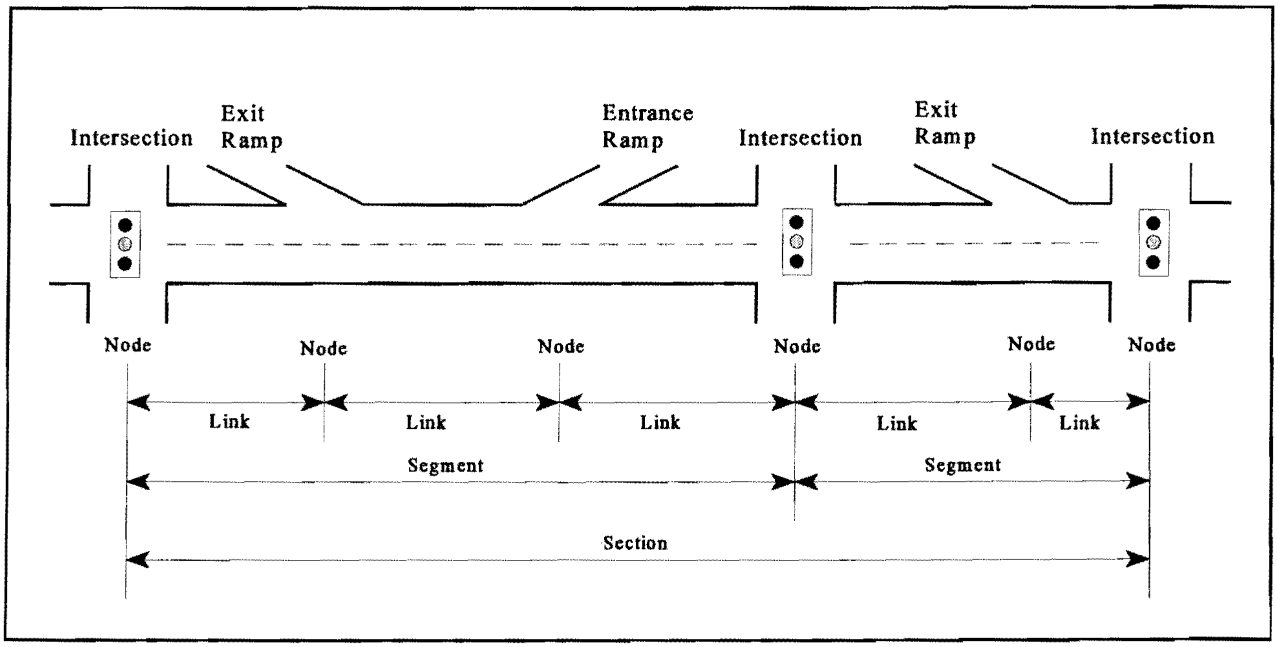

Figure 3-5. Terminology Used to Describe Frontage Roads.

In each database, site characteristics were translated from graphical format into a numerical format. Each link was defined by a beginning and ending node. Beginning and ending node types were assigned a code number based on the following classifications:

1. Signalized Intersection
2. Four-Way Stop-Controlled Intersection
3. Two-Way Stop-Controlled Intersection
4. Freeway Entrance Ramp without Auxiliary Lane
5. Freeway Entrance Ramp with Auxiliary Lane
6. Freeway Exit Ramp without Auxiliary Lane
7. Freeway Exit Ramp with Auxiliary Lane

Tables 3-5 and 3-6 provide summaries of the type and number of links included in the one-way and two-way field studies, respectively.

Table 3-5. Summary of Link Data for One-Way Frontage Road Sites.

Beg Node[a]	End Node[a]	Num	Min. Length (m)	Max. Length (m)	Beg Node[a]	End Node[a]	Num	Min. Length (m)	Max. Length (m)
1	1	4	442	707	5	1	3	296	378
1	4	26	113	546	5	6	1	1245	1245
1	5	13	101	561	5	7	10	569	924
1	6	8	265	655	6	1	8	207	524
1	7	8	163	771	6	4	10	197	479
4	1	12	241	811	6	6	2	469	503
4	2	1	284	284	6	7	1	564	564
4	4	1	701	701	7	1	33	110	655
4	6	7	325	962	7	5	1	165	165
4	7	16	360	1263	7	6	1	849	849

[a] Node Classifications:

1. Signalized Intersection
2. Four-Way Stop-Controlled Intersection
3. Two-Way Stop-Controlled Intersection
4. Freeway Entrance Ramp without Auxiliary Lane
5. Freeway Entrance Ramp with Auxiliary Lane
6. Freeway Exit Ramp without Auxiliary Lane
7. Freeway Exit Ramp with Auxiliary Lane

Table 3-6. Summary of Link Data for Two-Way Frontage Road Sites.

Beginning Node[a]	Ending Node[a]	*With*[b]			*Opposing*[c]		
		Number	Min. Length (m)	Max. Length (m)	Number	Min. Length (m)	Max. Length (m)
1	4	11	177	500	0	--	--
1	6	1	396	396	13	61	424
2	6	2	625	448	1	262	262
3	6	1	8439	849	0	--	--
4	1	0	--	--	11	177	500
4	6	13	436	2399	5	274	1058
6	1	13	61	424	1	396	396
6	2	1	262	262	2	625	448
6	3	0	--	--	1	849	849
6	4	5	274	1058	13	436	2399
6	6	1	1088	1088	1	1088	1088

[a] Node Classifications:

1. Signalized Intersection
2. Four-Way Stop-Controlled Intersection
3. Two-Way Stop-Controlled Intersection
4. Freeway Entrance Ramp
6. Freeway Exit Ramp

[b] Traveling in same direction of freeway traffic (see Figure 3-2).
[c] Traveling in opposite direction of freeway traffic (see Figure 3-2).

Table 3-7 provides a sample of the information contained in each database. Again, the information is presented by link for each field site. First, the database included a description of the link characteristics (e.g., beginning and ending node types, link length, access density, and speed limit). The access density was obtained by adding the number of driveways and unsignalized intersections and dividing by the link length. The databases also provided information concerning the travel time runs performed on each link. This information included the frontage road and ramp

Table 3-7. Sample of Database Containing Site Information.

Site	Link	Beg Node	End Node	Link Lengt h (m)	Access Density (acs/km)	Speed Limit (km/h)	Run	Fntg Road Vol (vph)	Entr Ramp Vol (vph)	Exit Ramp Vol (vph)	Speed (km/h)			Travel Time (sec)
											Min	Max	Avg	
1	1	1	4	64	6.3	72	1	450	110	--	8	52	32	7.1
1	1	1	4	64	6.3	72	2	485	90	--	8	56	35	6.5
1	1	1	4	64	6.3	72	3	460	85	--	13	61	39	6.0
1	1	1	4	64	6.3	72	4	430	100	--	11	48	31	7.5
1	1	1	4	64	6.3	72	5	490	120	--	35	63	50	4.6
1	1	1	4	64	6.3	72	6	515	110	--	11	56	37	6.2
1	1	1	4	64	6.3	72	7	555	130	--	44	68	56	4.1
1	2	4	5	123	8.7	72	1	340	110	230	50	64	56	7.9
1	2	4	5	123	5.4	72	2	395	90	280	56	71	64	6.9
1	2	4	5	123	5.4	72	3	375	85	250	60	69	64	6.9
1	2	4	5	123	5.4	72	4	330	100	315	48	76	63	7.1
1	2	4	5	123	5.4	72	5	370	120	340	56	77	68	6.6
1	2	4	5	123	5.4	72	6	405	110	410	55	71	64	6.9
1	2	4	5	123	5.4	72	7	425	130	400	68	72	69	6.4

volumes during the travel time run; the minimum, maximum, and average speed on the link; and the total travel time.

The creation of the databases gave the researchers a single source of site characteristics for each site. This simplified data manipulation and ensured that key characteristics for each site were recorded consistently.

Delay at Ramps

The objective of the delay at ramps analysis was to estimate the delay incurred by frontage road vehicles at freeway ramp junctions. The delays calculated in the field were used to evaluate procedures developed by Gattis et al. (2) to estimate delay at ramps.

On one-way frontage roads, vehicles experience delay only at exit ramps. However, on two-way frontage roads, the delay at ramps is based upon the type of ramp (exit or entrance) and the direction that the frontage road vehicle is traveling (*with* or *opposing* the freeway traffic). At exit ramps on two-way frontage roads, both the *with* and *opposing* frontage road vehicles experience delays. At an entrance ramp, however, only the *opposing* frontage road vehicles are delayed. Gattis et al. developed equations to estimate delay for the following cases:

Case 1: one-way frontage road, exit ramp *with* movement
Case 2: two-way frontage road, exit ramp *with* movement
Case 3: two-way frontage road, exit ramp *opposing* movement
Case 4: two-way frontage road, entrance ramp *opposing* movement

Two one-way frontage road sites, Sites A and B (see Table 3-3), were used to evaluate the equations for Case 1. Two of the two-way frontage road field sites (Sites D and E) were used to evaluate the equations for Cases 2 and 3. For Case 4, only one field site (Site C) was used. (Figures 2-2 and 2-3 illustrate the approaches of concern for the four cases.)

Data reduction efforts began by locating the points (upstream and downstream of the ramp) on the video tape that the technicians marked with an orange flag. These locations were marked on a clear sheet of plastic that covered the video monitor and were used to measure the travel time of frontage road vehicles traveling through the frontage road/ramp intersection area. To measure travel times, the technicians recorded the time a vehicle entered the system, tracked the vehicle through the system, and recorded the time that the vehicle left the system. The travel times were determined for each frontage road vehicle passing through the system. While measuring the travel times of frontage road vehicles, the technician recorded whether the frontage road vehicle was delayed at the ramp. A frontage road vehicle was categorized as delayed if the driver yielded to a ramp vehicle or was delayed due to another yielding vehicle.

For each field site, travel times were reduced for a one-hour period and were summarized in five-minute increments. Five-minute volume counts were also obtained for the freeway ramp, frontage road *with*, and frontage road *opposing* vehicles.

Technicians entered the travel time and volume data into a spreadsheet program for data manipulation. After an initial investigation of the five-minute aggregated data, it was determined that the results were highly variable due to random effects. The researchers believed that 15-minute averages would help overcome this randomness. Therefore, the data were aggregated into "sliding" 15-minute periods in five-minute increments (for example, 4:00 to 4:15, 4:05 to 4:20, 4:10 to 4:25, etc.).

Based on the delay units used by Gattis et al., average 15-minute delays in the field were computed as average delay per vehicle, in seconds. Delay was computed by subtracting the average travel time of vehicles that did not yield at the ramp from the average travel time of vehicles that yielded.

DATA ANALYSIS

Travel Time Studies

The objective of the travel time analysis was to learn how specific variables (e.g., traffic volume, access density, node type, etc.) affected the operations on both one-way and two-way frontage roads. To investigate the relationship between travel time/speed on the frontage road and other variables, several plots were generated. In addition, regression analyses were performed to learn the extent to which certain variables affected travel time.

Plots

To generate plots, each database and the raw data files from the travel time study were converted into a format so that they could be imported into a spreadsheet program. Using this spreadsheet program, the following plots were generated for both the one-way and two-way frontage road sites:

- Speed versus Cumulative Distance
- Travel Time versus Cumulative Distance
- Speed versus Volume
- Speed versus Access Density
- Travel Time versus Link Length

Statistical Analyses

Statistical analyses were performed on the one-way and two-way frontage road data to determine which factors had significant effects on travel time. The statistical analysis package, *SAS*, was used to perform stepwise regression on the established databases.

Linear regression models can be used to express a dependent variable as a function of a single independent variable. Linear regression models are expressed as $y = b + m(x)$, where y = dependent variable, x = independent variable, b = y-intercept, and m = slope. Multiple regression models are used to express a dependent variable as a function of two or more independent variables and are expressed as $y = b + m_1(x_1) + m_2(x_2) + \ldots$. Stepwise regression is a procedure that can be used to select the best multiple regression model (5). In other words, stepwise regression helps to identify those independent variables (x_1, x_2, ...) having the greatest effect on the dependent variable (y).

Stepwise regression works by starting with one independent variable and adding variables one at a time until certain criteria are met (5). The criterion used in this analysis was the coefficient of determination, R^2. The coefficient of determination is the portion of variability in the dependent variable explained by the independent variables. For each step in the stepwise regression procedure, the R^2 value is computed. The procedure is continued until no significant increase in R^2 is noted, and the resulting model is assumed to be the best-fitting regression equation.

The independent variables included in the regression analysis were: link length, frontage road volume, access density, and free-flow speed. Stepwise regression helped determine which factors affected travel time the most and was used to develop an equation to predict travel time based on these factors.

Delay at Ramps

The data collected at the one-way and two-way frontage road field sites were used to evaluate the procedures developed by Gattis et al. (2) for estimating frontage road delay at ramps. After the 15-minute delays were calculated from the field, the 15-minute frontage road and ramp volumes were used with the equations developed by Gattis et al. to predict delay. The predicted delay was then compared with the delay determined from the field data.

DEVELOP LEVEL-OF-SERVICE ANALYSIS PROCEDURE

The field data collected during this study were used to determine the factors affecting the level of service on one-way and two-way frontage roads. The level-of-service analysis procedure developed was based upon the procedure outlined in the *HCM* (1) for evaluating traffic operations on arterial streets. The procedure involves calculating the average travel speed along a frontage road and using the travel speed to estimate the level of service.

The average travel speed will be calculated based upon the estimated total travel time. The total travel time consists of the running time, intersection delay, and ramp delay. Data collected during the travel time studies in the field were used to develop a procedure for estimating the running time for one-way and two-way frontage roads. The intersection delay can be estimated using the *HCM* procedures. The expected delay at ramps will be calculated using the procedures developed by Gattis et al. (2).

CHAPTER 4

RESULTS FOR ONE-WAY FRONTAGE ROADS

The results from the field studies were used to identify variables affecting traffic operations on one-way frontage roads. Link data reduced from the field sites included the following: link length, node types, number of lanes, speed limit, number of driveways, number of crossroads, traffic signal timing information, and five-minute volume counts (for those links on which volumes were counted). The results from the data analyses were used to help develop a procedure to estimate the level of service for one-way frontage roads.

SPEED AND TRAVEL TIME VERSUS CUMULATIVE DISTANCE

The test vehicle used in the field was equipped with a distance measuring instrument (DMI) that can measure speed, travel time, and distance traveled in one-half second increments. A laptop computer was used to store the information from the DMI, and a different file was created for each travel time run. The analysis of the travel time data began with plotting the measured speed and travel time against the distance traveled. The plots provided a representation of travel speed and travel time along the frontage road segments. Appendix B includes speed and travel time plots for each of the 20 sites.

Figures 4-1 and 4-2 show samples of the speed versus cumulative distance plots. Figure 4-1 is the speed versus distance plot for Site 5, while Figure 4-2 is for Site 13 (see Table 3-1 for descriptions of the field sites). Each run made along the frontage road segment is represented by a different symbol and is identified by the time that the travel time run began. Along the bottom of the graph (under the x-axis) is a straight-line representation of the frontage road. Intersections are shown as straight vertical lines, and ramps are shown as angled lines. The forward sloping lines represent exit ramps, and the backward sloping lines represent entrance ramps.

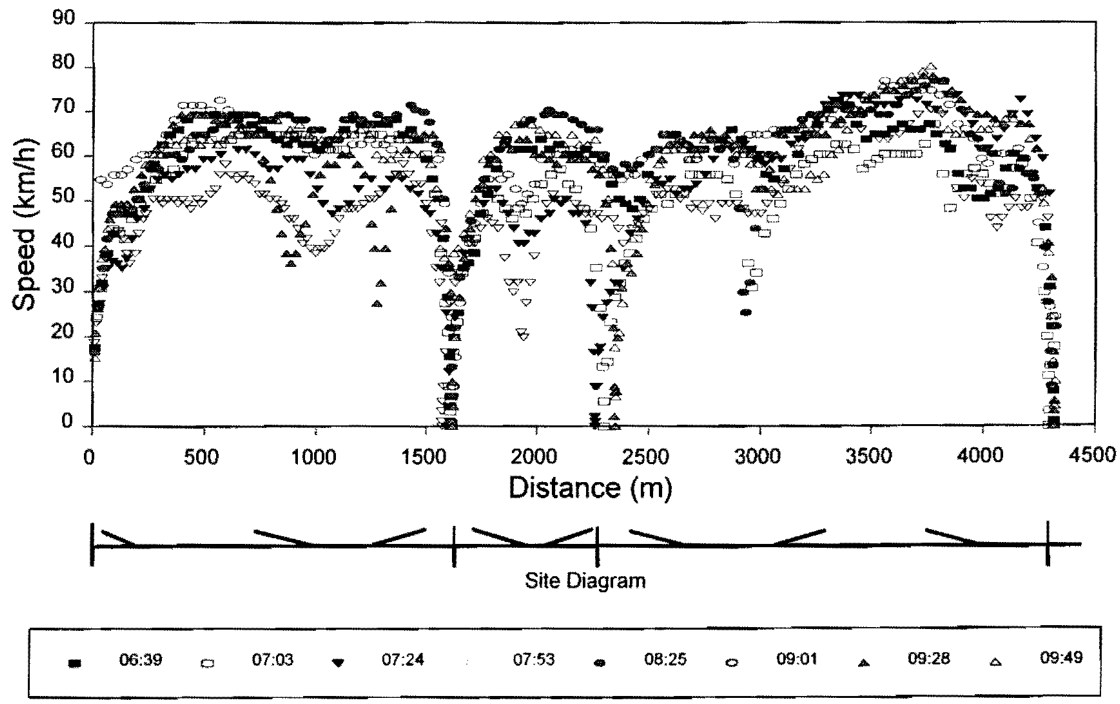

Figure 4-1. Speed Versus Cumulative Distance for Site 5.

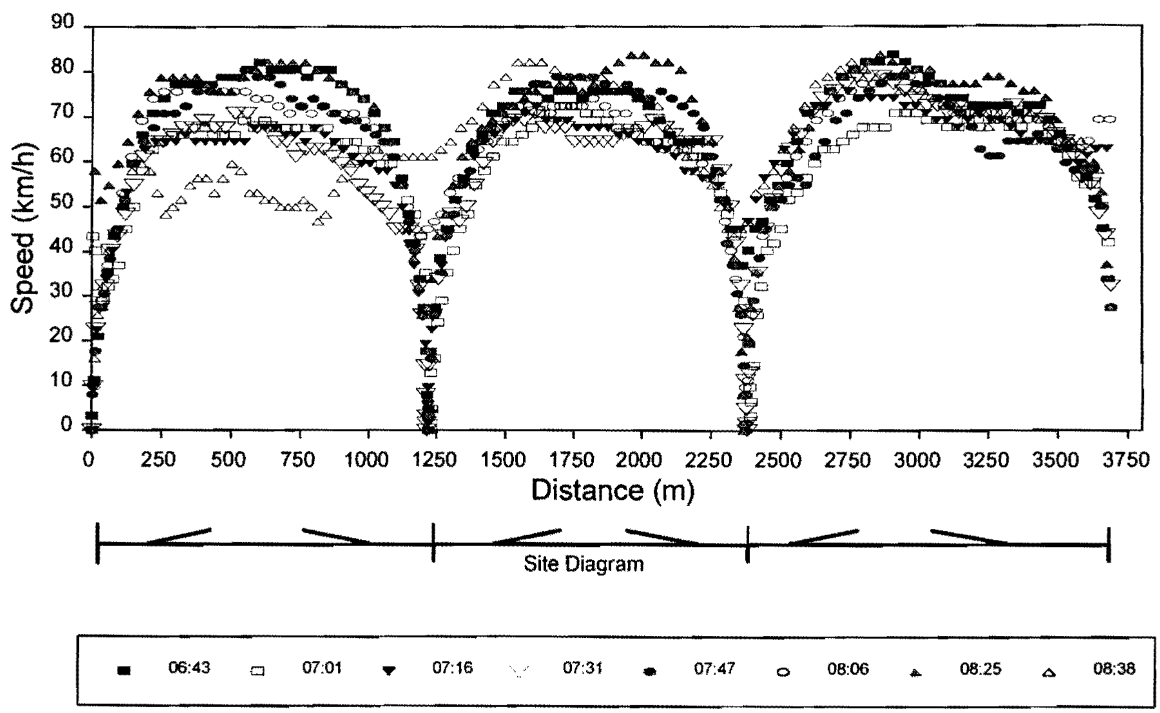

Figure 4-2. Speed Versus Cumulative Distance for Site 13.

These plots clearly show the influence of the signalized intersections on travel speed. In most runs, the test vehicle achieved speeds between 70 and 80 km/h. In almost every run, however, the test vehicle was required to slow and stop at the signalized intersections. **These plots clearly reveal that an evaluation of the operations along a frontage road must consider the effects of the traffic signal.**

Figure 4-1 shows the effects of exit ramps on travel speed. At the exit ramp located at approximately 1000 m, speed reductions of approximately 15 to 20 km/h are observed for some travel time runs. Speed reductions are also observed at most of the other exit ramps located at this site. Figure 4-1 also shows delays that may be caused by other factors such as driveways or cross streets. For example, at approximately 2900 m, reductions in speed occurred for those travel time runs beginning at 07:03 and 08:25. Investigation of the field sketch for Site 5 revealed that driveways leading to two restaurants were found in this area.

For Site 13 (see Figure 4-2), the signalized intersections again heavily influenced traffic operations. The exit ramps, however, had minimal effects on traffic operations when compared to the effects of exit ramps at Site 5 (see Figure 4-1). An investigation of the site sketches revealed that all exit ramps at Site 13 were followed by an auxiliary lane, while most of the exit ramps at Site 5 merged directly with the frontage road. These findings reveal the benefits of providing auxiliary lanes at exit ramps.

The variability in speed for the different runs at Site 13 may be caused by the traffic volume level present during different runs. Additional analyses will be used to test this theory developed based upon the speed/distance plots.

Another method of presenting the data is to use cumulative travel time rather than speed. The speed measurements showed major fluctuations between consecutive nodes, in most cases between 0 and 80 km/h. Travel time data provide a single value to represent the time spent traversing the segment. Figures 4-3 and 4-4 show the cumulative travel time versus cumulative distance for Sites 5 and 13, respectively. Again, the plots clearly show the influences of the intersections. The travel

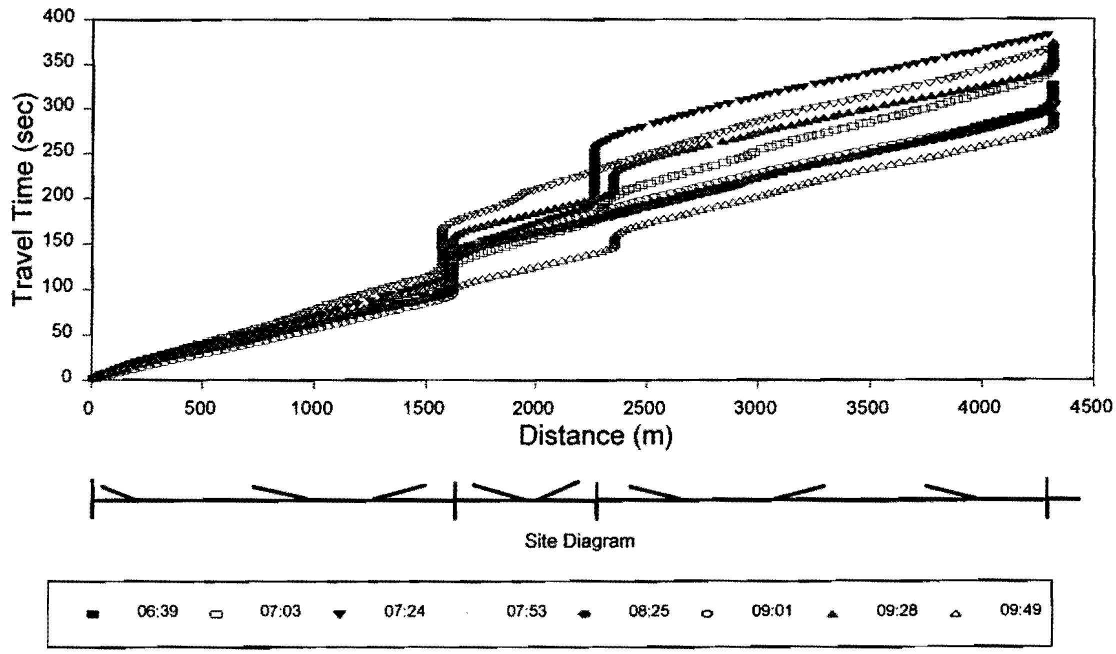

Figure 4-3. Travel Time Versus Cumulative Distance for Site 5.

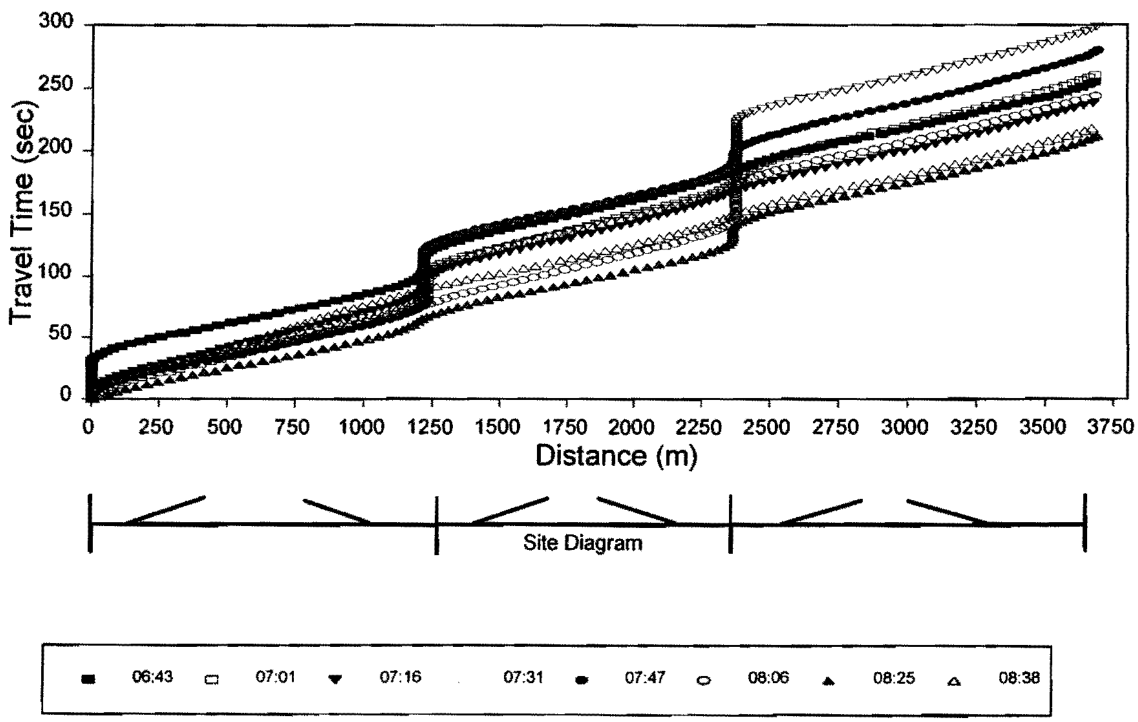

Figure 4-4. Travel Time Versus Cumulative Distance for Site 13.

time plot can also more clearly reveal the runs with the slowest overall speeds. For example, the run beginning at 07:31 at Site 13 had the largest total travel time (see Figure 4-4); however, this information is not clearly shown in the plot of speed versus distance (see Figure 4-2).

The speed/distance and travel time/distance plots provide an appreciation of the operations at the 20 field sites. Maximum speeds along the segments were generally high, and the signalized intersections had the greatest influence on operations. Therefore, these results agree with the arterial chapter (Chapter 11) of the *HCM* (1), which states that the capacity along arterials is controlled by the signalized intersections.

DEVELOPING LEVEL-OF-SERVICE ANALYSIS PROCEDURE

The *HCM* level-of-service analysis procedure for arterial streets uses average running speed to predict level of service. Average running speed is computed by dividing the total length of the arterial under investigation by the total travel time. The total arterial travel time is composed of the running time and the delay at signalized intersections. A similar procedure may be used for evaluating frontage road operations.

The delay at signalized intersections can be estimated using the procedures in the *HCM*, Chapter 9. These procedures contain some limitations (for example, the procedures poorly predict the operations for closely spaced intersections); however, they are one of the best techniques for signalized intersection evaluation and are the accepted state-of-the-practice. It was beyond the scope of this research to develop new procedures for evaluating operations at signalized intersections; therefore, the *HCM*, Chapter 9, procedures were selected to be used for estimating delay at signalized intersections on frontage roads.

Besides delay at signalized intersections, the delay at exit ramps must also be considered for one-way frontage roads. The delay at exit ramps is dependent upon several factors including: frontage road volume, exit ramp volume, presence of an auxiliary lane at the exit ramp, and the typical yielding behavior of drivers within a city. For those field sites investigated, the frontage road delay at ramps was negligible for those sites that included auxiliary lanes at the exit ramps. Therefore, procedures were needed for estimating the delay to frontage road vehicles at exit ramps without an auxiliary lane and at exit ramps when frontage road drivers consistently yield to exit ramp vehicles.

The *HCM,* Chapter 11, provides procedures for estimating the running time on arterial streets. The running time is estimated based on arterial classification, running speed, and section length. A similar procedure will be developed to predict running time on frontage roads. Following is a discussion of the efforts to develop a procedure for estimating running time on one-way frontage roads and to estimate delay at exit ramps without auxiliary lanes.

PREDICTING RUNNING TIME

Researchers used the field data to develop a method for predicting running time for use in estimating frontage road level of service. The first step was to decide how certain variables affected running time. The variables investigated included link type, link length, frontage road volume, and approach density. The study consisted of generating several plots and performing regression analyses.

Link Type

As discussed in Chapter 3 of this report, each frontage road segment studied was divided into links. A link describes a length of frontage road between two nodes, or logical break points (e.g., ramps or intersections). The nodes for this analysis are summarized as follows:

- Signalized Intersection
- Stop-Controlled Intersection
- Freeway Entrance Ramp
- Freeway Exit Ramp

To learn how link type affected traffic operations, delays were computed and compared for each link type. Delay was computed by subtracting the optimum link travel time from the actual link travel time. Optimum travel time was computed for each link by dividing the link length by the free-flow speed, which was selected using the speed versus cumulative distance plot for each site (see Appendix A). The actual travel time for each link was obtained from the field data.

Figure 4-5 shows the delay estimates for each link type. The link types are defined by beginning and ending node types using the following abbreviations: N = entrance ramp, X = exit ramp, I = signalized intersection, and S = stop-controlled intersection. As expected, the link types producing some of the highest delays were those ending with a signalized intersection (link types I-I, X-I, and N-I).

Link type N-X is an entrance ramp followed by an exit ramp; however, the highest delays are on the same order as that for links ending in a signalized intersection (link types I-I, X-I, and N-I). An inspection of the field data revealed that the high delays for link type N-X (entrance ramp to exit ramp) occurred on a link followed by a relatively short (110 m) link type X-I (exit ramp to intersection). Therefore, queues from the link type X-I were spilling back on the link type N-X, causing similar delays. This finding reveals that the operations on each link type do not operate independently of one another and that operations on one link can affect operations on an upstream link.

Figure 4-5 also shows very high delays for link types I-N and I-X (beginning with a signalized intersection). An investigation of the field data revealed that much of the delay shown on the links beginning with a signalized intersection actually occurred on the links immediately upstream (i.e., links ending in a signalized intersection). The following paragraphs explain.

While in the field, technicians took measurements to find the beginning and ending points of each link with respect to the starting point for the travel time run. Knowing the beginning and ending points for each link, the time that the test vehicle entered and exited a link could be determined by matching the distance and travel time measurments recorded by the DMI. The link travel times could then be computed by subtracting the time that the vehicle entered the link from the time that the vehicle left the link.

A problem occurs when the distance measured from the DMI during a certain travel time run does not match the measured beginning and ending points for a specific link. These inaccuracies in the measurements of distance traveled occurred due to the different maneuvers (such as lane changes) made by the driver of the test vehicle during each travel time run. Because the distance measurements made during each travel time run did not match exactly, obtaining an exact link travel time for each run was difficult. These inaccuracies have a major effect only on those links that include a signalized intersection (i.e., encounter high delays).

Eliminating those data points influenced by the signals provided a way of evaluating the sections between the signals (i.e., predict running time). A link was said to be influenced by a nearby signal if the minimum speed along the link was 8 km/h or less. Figure 4-6 shows the data for all links with a minimum speed greater than 8 km/h. Minimizing the effects of signalized intersections greatly reduced the variability in the data. In addition, the delays for each link type are all less than 20 seconds.

Link Length

Figure 4-7 illustrates the relationship between link travel time and link length for all link types included in the study. Significant variation in travel time is present for the different lengths. Most of the variation in travel time was due to the effects of signalized intersections. Figure 4-8 shows the data for all links with a minimum speed greater than 8 km/h. In this figure, a strong relationship between running time and link length is evident, and the effects of signalized intersections are reduced. The data represented in this figure were used in the following evaluations.

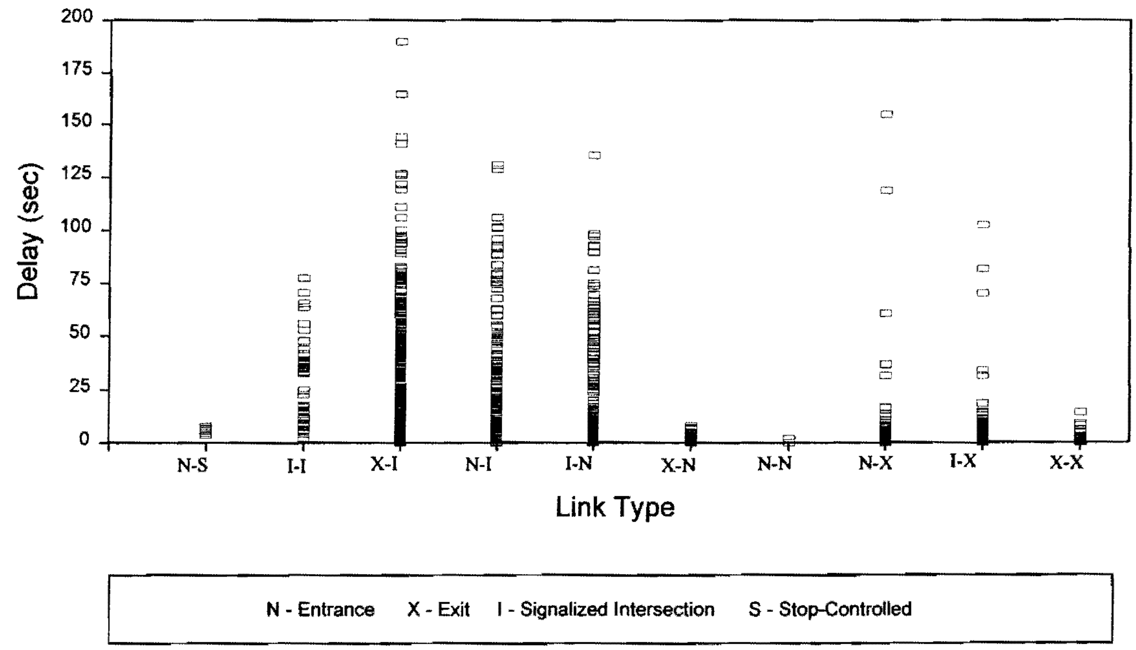

Figure 4-5. Link Delay Versus Link Type.

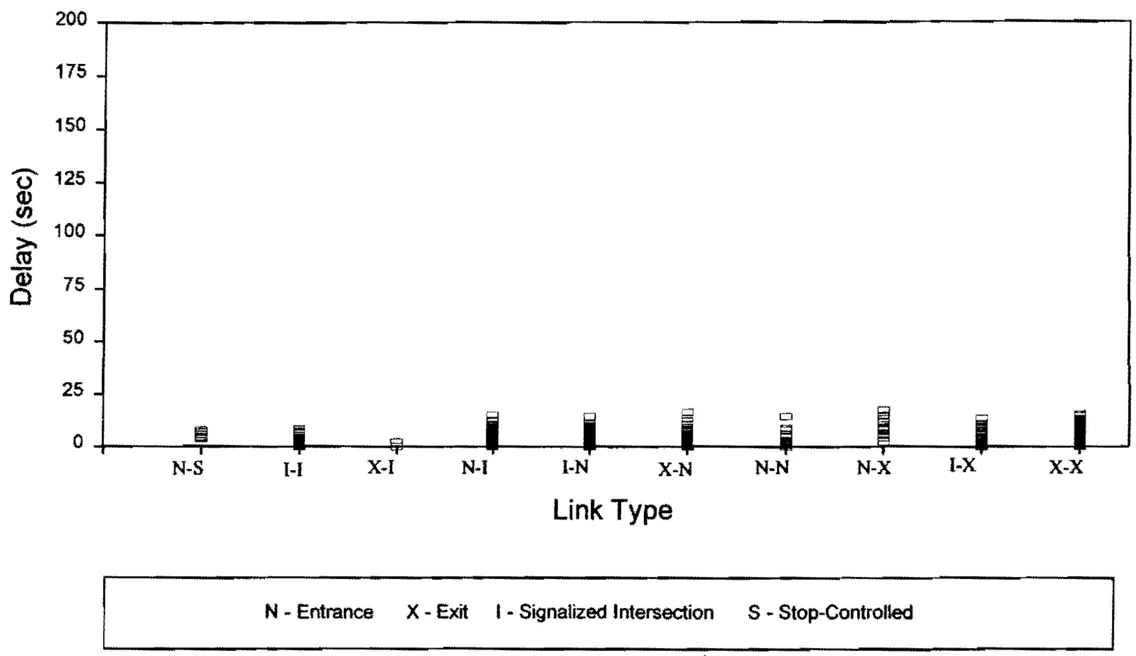

Figure 4-6. Link Delay Versus Link Type for Speeds Greater Than 8 km/h.

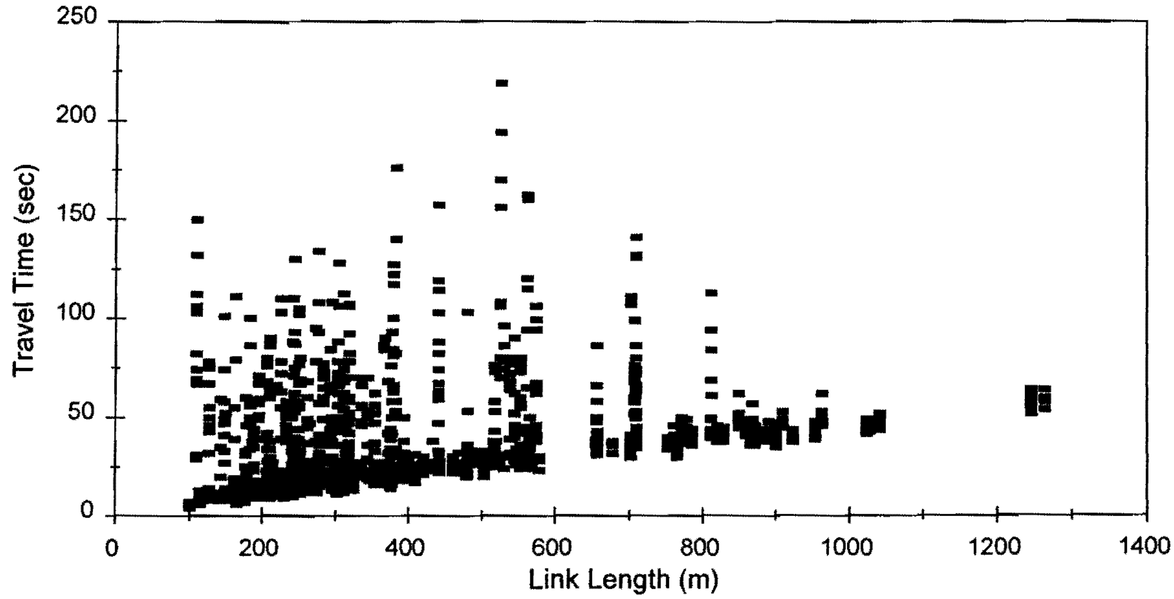

Figure 4-7. Travel Time Versus Link Length.

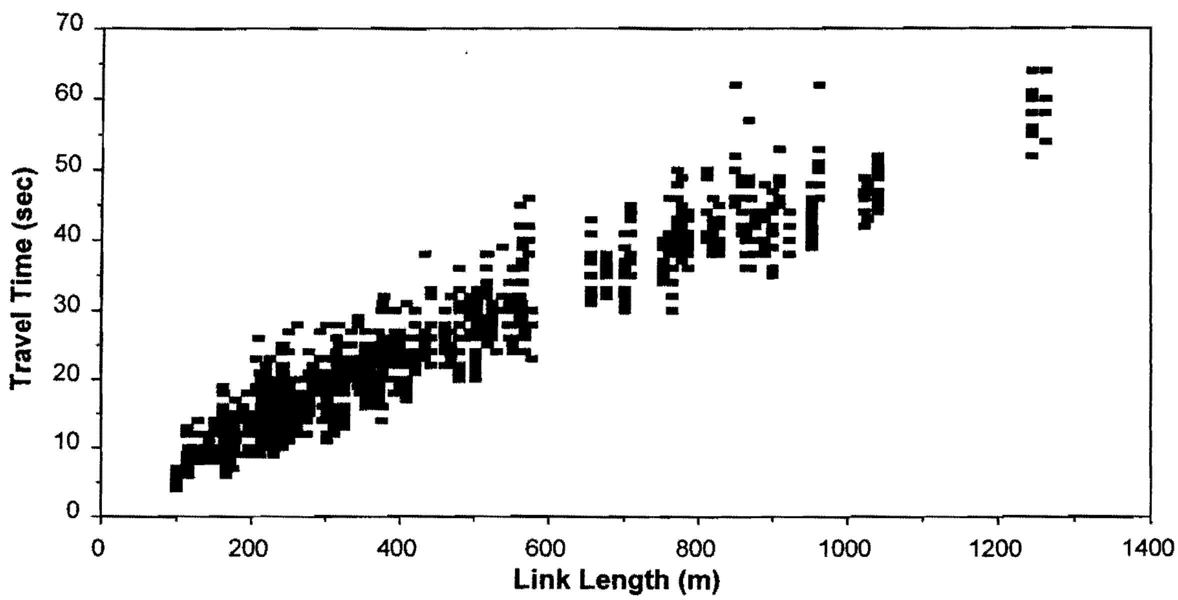

Figure 4-8. Travel Time Versus Link Length for Speeds Greater Than 8 km/h.

Because travel time is heavily influenced by link length, evaluating the effects of other variables (such as volume and access density) on travel time would be difficult without accounting for link length. For example, consider studying the effect of volume on travel time for two different links with lengths of 1.0 km and 2.0 km, respectively. The second link is 1.0 km longer than the first link; therefore, because of the heavy influence of link length on travel time, deciding how volume affected travel time would be difficult.

For this reason, speed was used in the place of travel time to determine the effects of volume and access density on traffic operations. Travel time and length are included in the calculation of speed, and therefore account for the strong relationship between travel time and length.

Frontage Road Volume

Figure 4-9 illustrates the relationship between average speed and frontage road volume for those links on which volumes were obtained. Observing this figure, no major correlation between speed and volume is apparent for the 20 field sites. The researchers initially thought that the high variability in speed was due to the varying site characteristics at each field site. Therefore, new plots were generated to show the speed/volume relationships for each of the 20 field sites. To improve readability, the data points from the 20 sites were plotted on two figures. The relationships between speed and volume for Sites 1 through 10 are shown in Figure 4-10, and Figure 4-11 shows the relationships for Sites 11 through 20.

Observing Figures 4-10 and 4-11, correlations between average speed and volume appear to exist for some, but not all, sites. For Site 3 (see Figure 4-10), the volumes ranged from approximately 0 to 600 vehicles per hour per lane (vphpl). For lower volumes (below 300 vphpl), a maximum speed of 80 km/h is observed. At higher volumes (above 500 vphpl), the maximum speed is around 60 km/h. However, for Site 6, the volumes ranged from approximately 50 to 750 vphpl, and the maximum speed remained relatively constant around 70 km/h. Observing the data for Site 15 in Figure 4-11, the volume ranged from 50 to 1150 vphpl; however, again, no apparent correlation between speed and volume was noted.

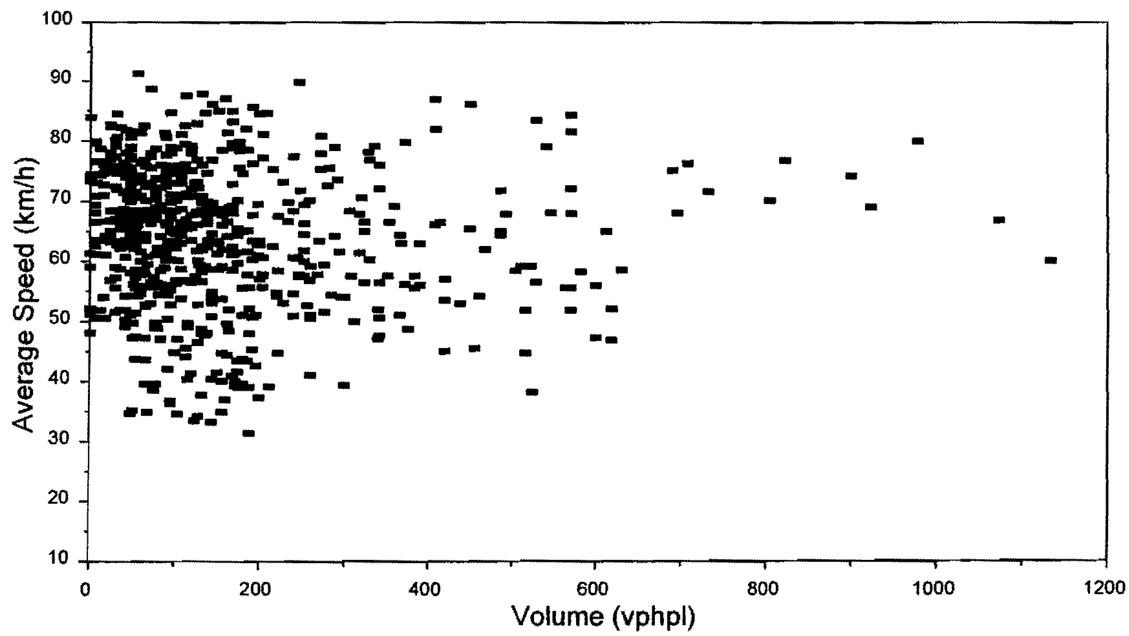

Figure 4-9. Average Speed Versus Volume.

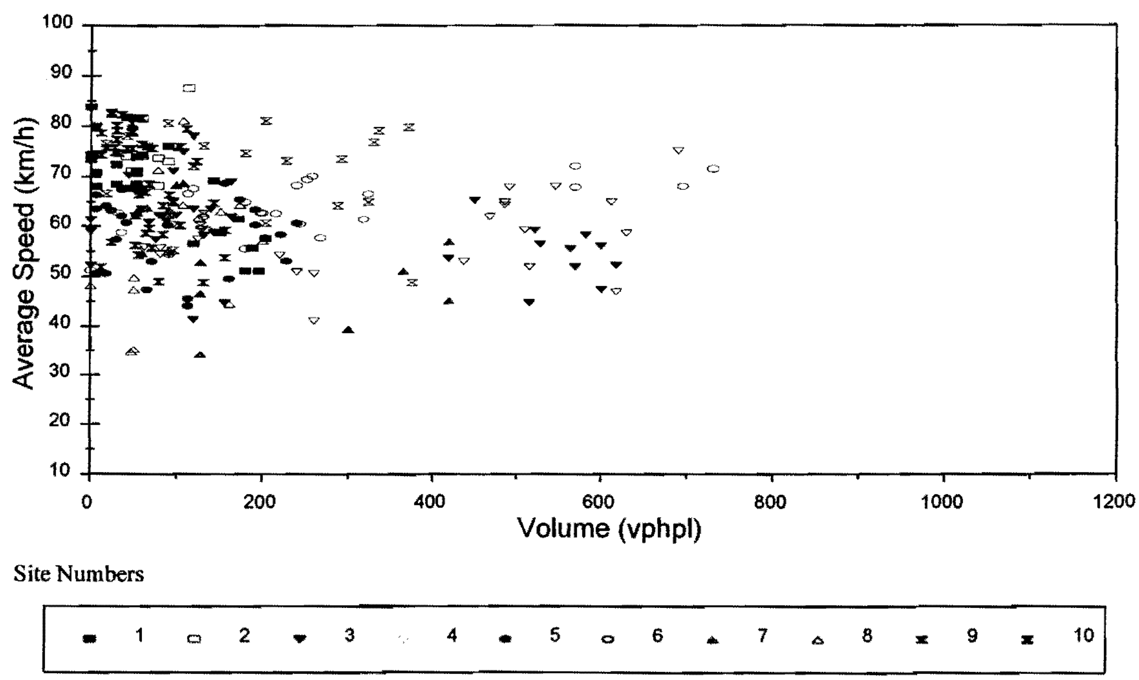

Figure 4-10. Average Speed Versus Volume for Sites 1 to 10.

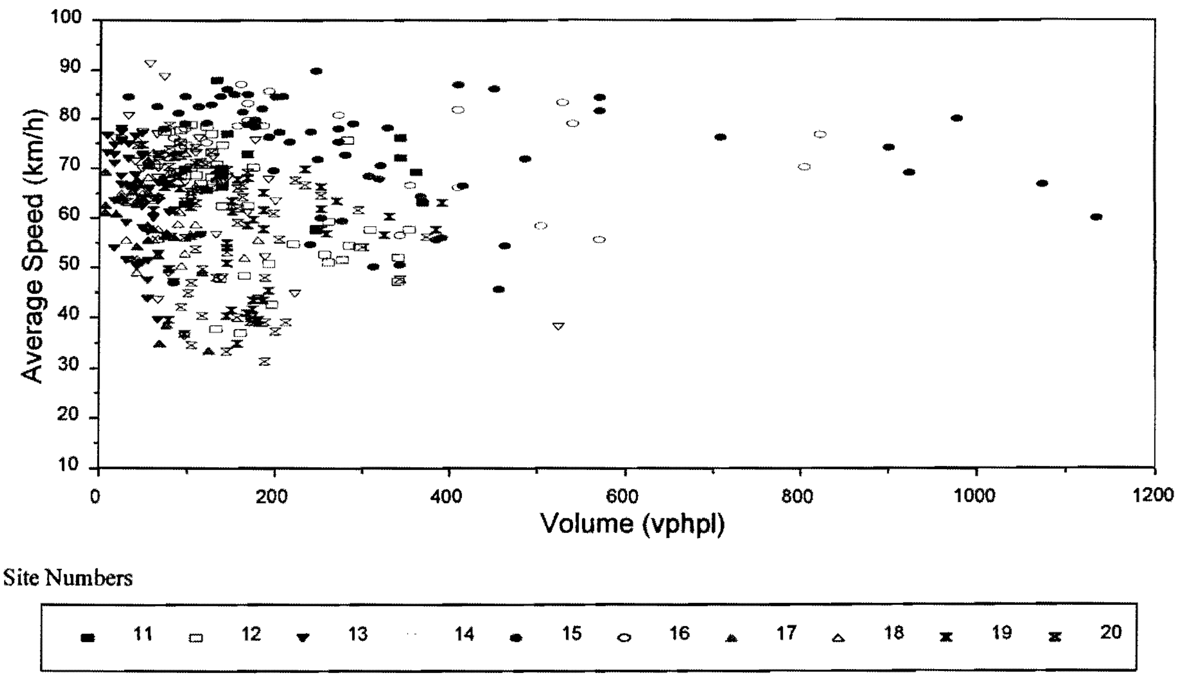

Figure 4-11. Average Speed Versus Volume for Sites 11 to 20.

An investigation of the field data for Site 3 (where speed decreased for increasing volume) revealed that the decrease in speed could have been attributed to the link type. For this site, the lower speeds occurred on links that included a freeway exit ramp followed by an entrance ramp. This type of link involves one-sided weaving operations because of the exit ramp and entrance ramp maneuvers. Due to the weaving operations, the speeds on the link are more sensitive to increasing volumes. Therefore, for the volume ranges observed at the field sites, a relationship between speed and volume was not defined.

Another cause of the high variability in operating speed shown in Figure 4-9 could be due to the varying speed limits present on the different links. Speed limits at the 20 field sites varied between 56 km/h to 89 km/h. Figure 4-12 shows the speed versus volume data by the link's speed limit. For those links with a 89 km/h speed limit, there appears to be a small decrease in speed for increases in volume. The data for the links with a 72 km/h speed limit, however, do not show the same type of relationship. Links with a 72 km/h speed limit have some of the highest and lowest

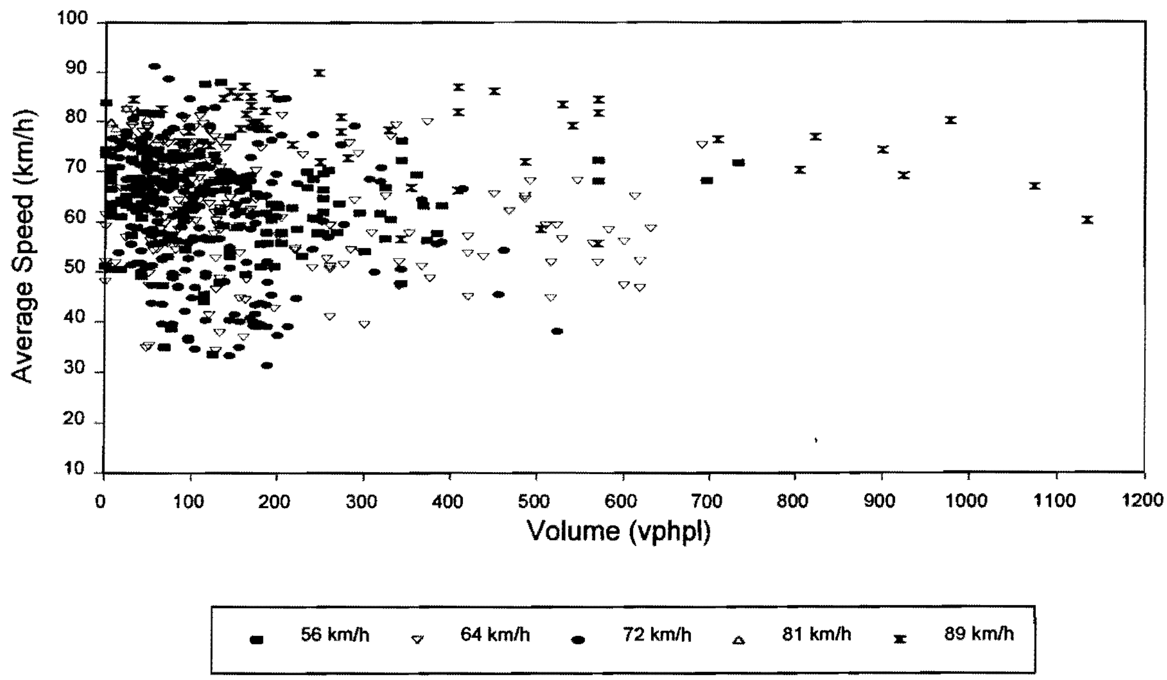

Figure 4-12. Average Speed Versus Volume By Speed Limit.

speeds for specific volume levels. Even when the data is subdivided by speed limit, there still appears to be no correlation between speed and volume for the data collected at the 20 field sites.

Access Density

The access density was calculated for each link by summing the number of driveways and unsignalized intersections on the link and dividing by the link length. The access density is defined as the number of access points per kilometer (acs/km). It can also be converted to average spacing by taking the inverse of the density value. Figure 4-13 illustrates the relationship between average speed and access density for each link included in the 20 study sites. Observing this figure, a high variability exists between speed and access density; however, the variability decreases with increasing access density. Figures 4-14 and 4-15 were generated to study the speed/access density relationships for the different speed limits. The relationships between speed and access density for speed limits of 56 and 64 km/h are shown in Figure 4-14, and Figure 4-15 shows the relationships for speed limits of 72 to 89 km/h.

As the data in Figures 4-13 to 4-15 illustrate, a correlation between speed and access density exists (i.e., speed is slower for high access density). A critical access density value exists at approximately 20 acs/km. For example, below 20 acs/km, maximum speeds around 90 km/h are observed. For access densities above 20 acs/km, most of the speeds observed do not exceed 72 km/h. For access densities above approximately 20 acs/km, increases in travel time of about 10 to 15 percent may exist. At 20 acs/km, access points (i.e., driveways and unsignalized intersections) are spaced at an average of 50 m apart.

The plots of access density versus average speed by speed limit also shows a relationship between "higher speed" facilities and "lower speed" facilities. Links with high speed limits (e.g., 81 and 89 km/h) had the lower access densities (see Figures 4-14 and 4-15). All links with access densities greater than 20 acs/km were on facilities with speed limits of 72 km/h and below. One interpretation of the data could be that lower speeds are measured on high access density roads only because they have lower speed limits. The data for the 72 km/h roads, however, do not agree with this interpretation. The 72 km/h speed limit exists on roads with access densities that range between 0 and 37 acs/km. The plot of average speed versus access density (see Figure 4-16) clearly shows a downward trend of speed (especially for the maximum speeds observed).

These observations support the hypothesis that access density influences driver behavior. Based upon the findings from this study, the number (or spacing) of driveways and unsignalized intersections affect drivers' speeds when a threshold value is reached. Below that critical number of driveways/unsignalized intersections, drivers' speeds do not appear to be influenced.

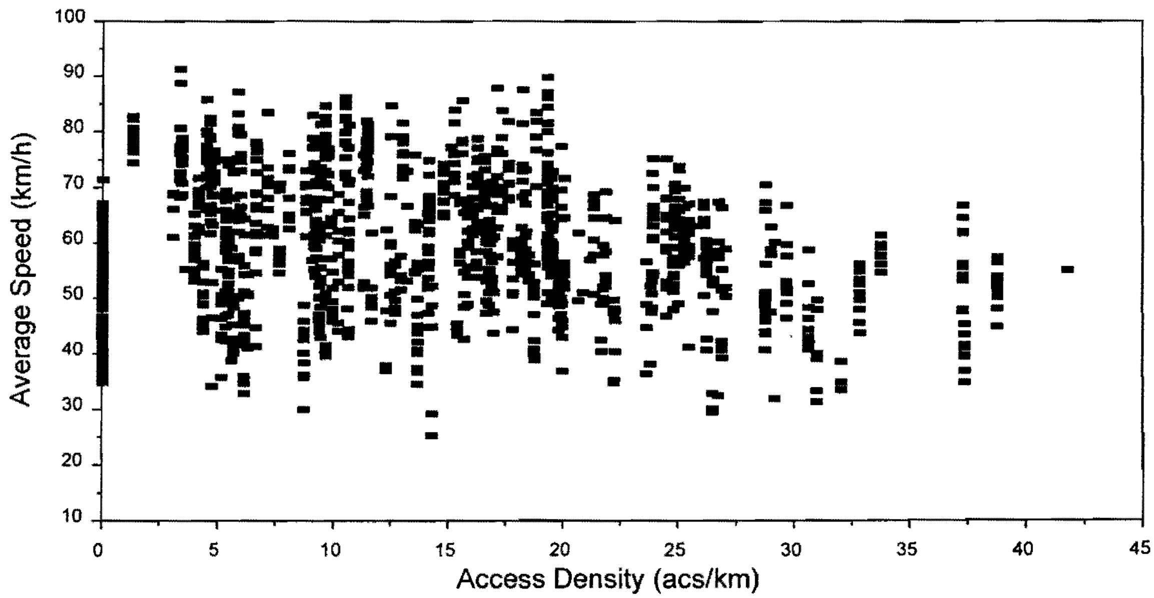

Figure 4-13. Average Speed Versus Access Density.

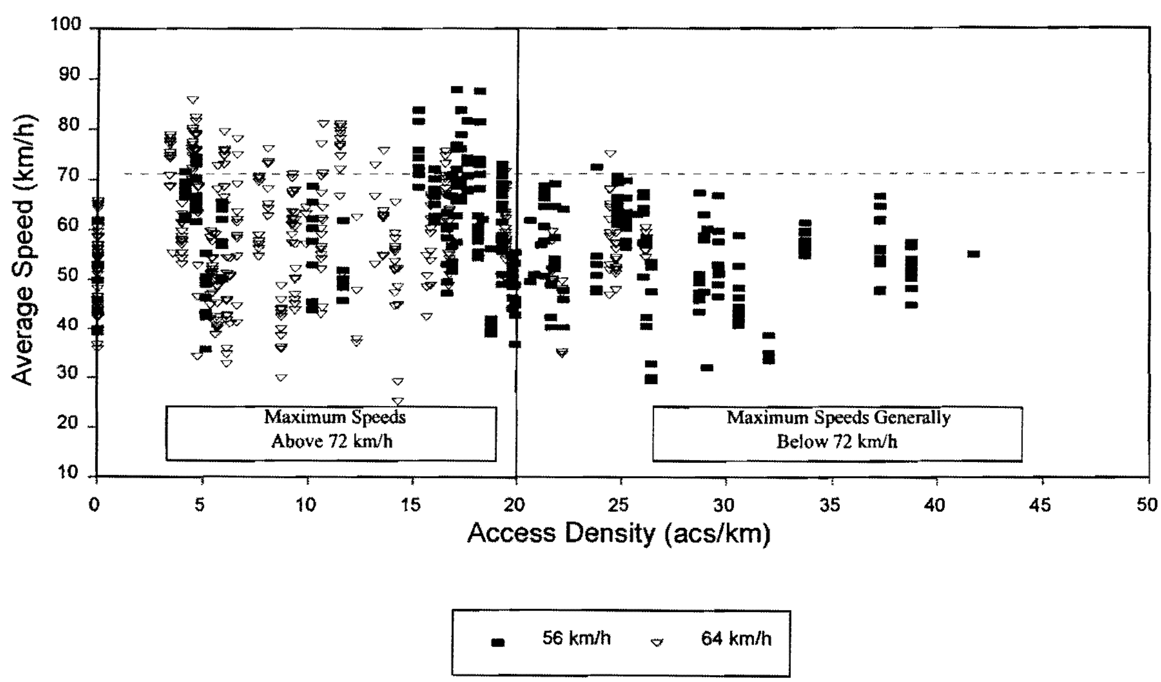

Figure 4-14. Average Speed Versus Access Density for Speed Limits of 56 and 64 km/h.

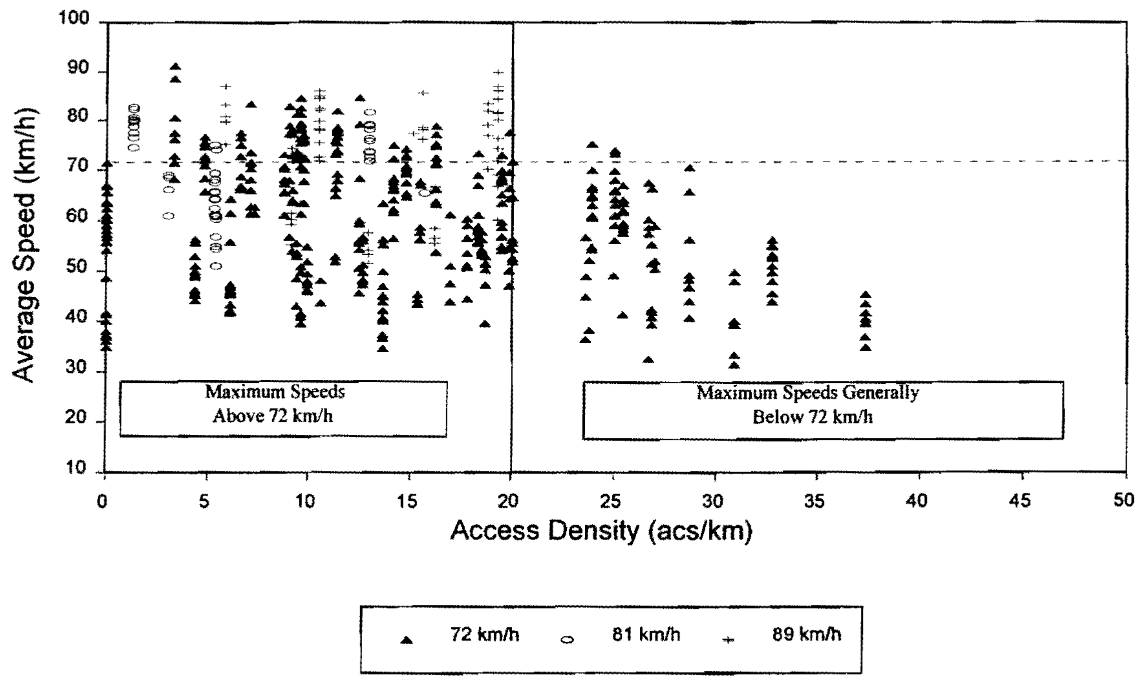

Figure 4-15. Average Speed Versus Access Density for Speed Limits of 72, 81, and 89 km/h.

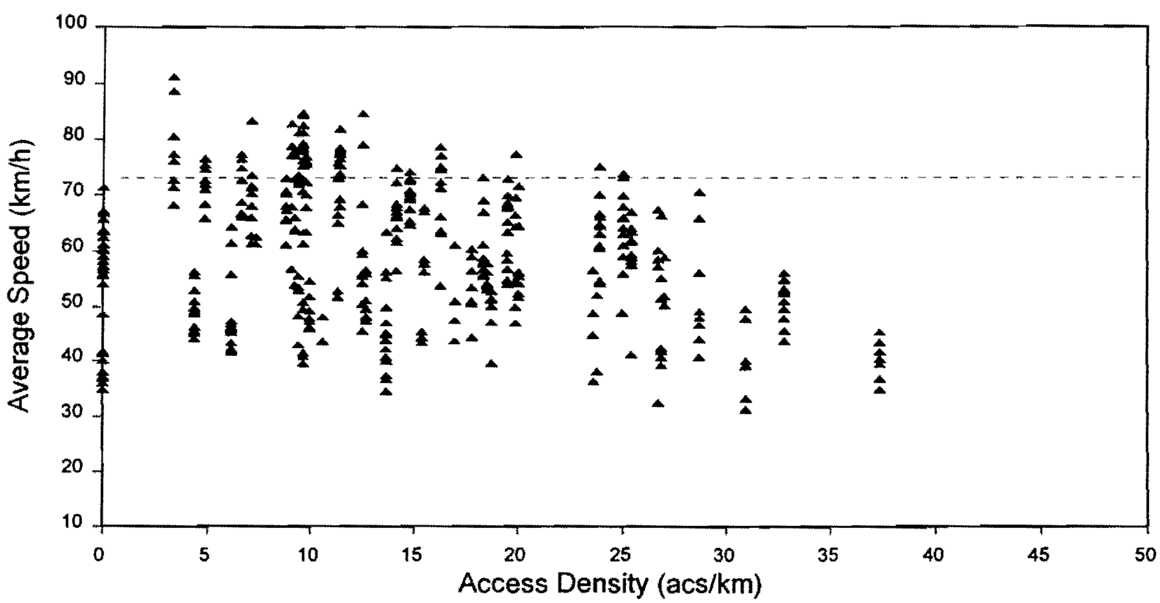

Figure 4-16. Average Speed Versus Access Density for a Speed Limit of 72 km/h.

Regression Analysis

To develop an equation for predicting running time for one-way frontage road links, a database was created consisting of all the appropriate link data to be included in the analysis. The database included the following information for each link: maximum and minimum speed, total link travel time, link length, frontage road volume (for those links on which volumes were obtained), access density, and free-flow speed. To reduce the effects of signalized intersections, the link data that included minimum speeds below 8 km/h were deleted.

The database was then used with the statistical analysis package, *SAS.* Using stepwise regression, variables were investigated to determine the factors affecting running time. These variables included link length, frontage road volume, access density, and free-flow speed.

The stepwise regression procedure involved starting with one independent variable and adding variables one at a time until there was no longer a significant increase in the coefficient of determination, R^2. The resulting model was assumed to be the best-fitting regression equation. Table 4-1 shows the equations and corresponding R^2 values.

Table 4-1. Results From Stepwise Regression for One-Way Frontage Roads.

Step	Model[a]	R^2
1	RT = 0.0504(L)	0.976
2	RT = 0.0467(L) + 0.1955(AD)	0.982
3	RT = 0.0422(L) + 0.1209(AD) + 0.0361(FFS)	0.982
4	RT = 0.0443(L) + 0.1208(AD) + 0.0358(FFS) + 0.0005(V)	0.982

[a] RT = link running time,sec
L = link length, m
AD = access density, acs/km
V = frontage road volume, vph
FFS = free-flow speed, km/h

The initial variable entered in Step 1 of the stepwise regression process was link length, producing an R^2 value of 0.976. Link length was followed by access density, free-flow speed, and frontage road volume, respectively. The R^2 values calculated in Steps 2, 3, and 4 increased very little from that calculated in Step 1. Results from the stepwise regression signify that running time is heavily dependent upon link length and is relatively independent of the other variables. Therefore, the researchers recommend using the equation in Step 1 of Table 4-1 to predict running time for one-way frontage road links.

PREDICTING EXIT RAMP DELAY

Delay is a variable that is difficult to estimate because of the sensitivity to local and environmental conditions. The delay at exit ramps is especially difficult to estimate because it is based on factors such as driver aggressiveness, gap acceptance, and local driving customs. While collecting data at field sites in various areas around the state of Texas, drivers were observed behaving differently from city to city. For example, at field sites in one city, frontage road drivers would always yield the right-of-way to exit ramp vehicles. In another city, however, the frontage road drivers drove more aggressively and sometimes the exit ramp drivers adjusted their speed for an adequate gap in the frontage road stream. Therefore, factors affecting delay at ramps vary not only from driver to driver, but also from city to city.

Proposed Model

In the study conducted by Gattis et al. (2), procedures were developed to estimate delay at ramps on two-way frontage roads for the following scenarios: exit ramp, *with*; exit ramp, *opposing*; and entrance ramp, *opposing*. In their research, the researchers also develop a procedure to estimate delay at one-way frontage roads (for purposes of comparing delay on one-way frontage roads to delay on two-way frontage roads). Models were developed for estimating ramp delay by using theoretical principles and data collected in the field. The theoretical principles involved treating the frontage road-ramp intersection area as a queuing system and assuming that the ramp traffic arrival could be described using the Poisson process. Based on these principles, equations were developed for predicting the frontage road capacity and the queuing delay. Delay measured in the field was then regressed against the queuing delay to develop equations for predicting total frontage road delay.

The one-way frontage road field site selected by Gattis et al. included a defined range of traffic volumes. Because of the limited field data, the researchers concluded that the resulting model for estimating ramp delay on one-way frontage roads could predict incorrect values for higher queuing delays. Therefore, to estimate ramp delay on one-way frontage roads, the researchers recommended using the equation developed for predicting total delay for exit ramp, *with* vehicles on two-way frontage roads. Chapter 2 shows the proposed models for estimating delay for one-way frontage roads as Equations 2-15 to 2-17.

Evaluation of Model

To evaluate the model developed by Gattis et al., additional field data were collected at two field sites (see Table 4-2). At each site, operations were recorded using video cameras. Both field sites included a three-lane frontage road section with an exit ramp followed by a downstream signalized intersection. As shown in Table 4-2, Site A included high exit ramp volumes and relatively low frontage road volumes. In contrast, Site B included high frontage road volumes and relatively low exit ramp volumes. The models developed by Gattis et al. were evaluated by comparing delays measured in the field to those predicted by the models.

Table 4-2. Description of Field Sites for Evaluation of Delay at Exit Ramps.

Site	City	Location	Number of Frontage Road Lanes	Ramp-To-Intersection Spacing (m)	Average Volume (vph)	
					Frtg. Road	Exit Ramp
A	Houston	IH 610 @ San Felipe	3	229	315	1535
B	Houston	IH 610 @ Evergreen	3	244	1255	220

One hour of data was reduced from each site. The data were aggregated into "sliding" 15-minute periods in five-minute increments (for example, 4:00 to 4:15, 4:05 to 4:20, etc.). Tables 4-3 and 4-4 compare the results from Sites A and B to the procedures developed by Gattis et al.

The evaluation of the Site A data revealed a limitation in the models developed by Gattis et al. Because the exit ramp volumes observed at this site were so high (from 1300 to 1800 vph), the equation to estimate frontage road capacity (C_R) produced negative values. The equation to predict frontage road capacity is as follows: C_R = Number of Lanes (1858 - 1.5259 * Ramp Volume). Therefore, exit ramp volumes exceeding approximately 1200 vph will produce negative results when using this equation. The frontage road delays observed in the field were on the order of only one to two seconds (see Table 4-3). The low delays were primarily due to the low frontage road volumes.

Table 4-3. Comparison of Site A Field Data to Ramp Delay Model.

Time Increment	Exit Ramp Volume (vph)	Frontage Road Volume (vph)	Field Delay (sec/veh)	Gattis et al. Procedures		
				C (vph)	W (sec/veh)	Delay (sec/veh)
7:00-7:15	1380	240	0.9	-743	N/A	N/A
7:05-7:20	1316	276	1.1	-450	N/A	N/A
7:10-7:25	1476	260	1.1	-1182	N/A	N/A
7:15-7:30	1516	332	1.2	-1365	N/A	N/A
7:20-7:35	1532	316	1.0	-1439	N/A	N/A
7:25-7:40	1488	320	1.3	-1237	N/A	N/A
7:30-7:45	1492	316	1.6	-1255	N/A	N/A
7:35-7:50	1640	372	1.6	-1933	N/A	N/A
7:40-7:55	1708	380	1.6	-2244	N/A	N/A
7:45-8:00	1800	360	1.5	-2665	N/A	N/A
Average	1535	315	**1.3**			**N/A**

Table 4-4. Comparison of Site B Field Data to Ramp Delay Model.

Time Increment	Exit Ramp Volume (vph)	Frontage Road Volume (vph)	Field Delay (sec/veh)	Gattis et al. Procedures		
				C (vph)	W (sec/veh)	Delay (sec/veh)
4:30-4:45	192	1220	1.7	4695	1.0	1.0
4:35-4:50	192	1230	1.7	4695	1.1	1.1
4:40-4:55	184	1148	1.2	4731	1.0	1.0
4:45-5:00	160	1140	1.3	4841	1.0	1.0
4:50-5:05	176	1156	1.0	4768	1.0	1.0
4:55-5:10	216	1280	1.4	4585	1.1	1.1
5:00-5:15	232	1400	1.1	4511	1.2	1.2
5:05-5:20	264	1392	2.1	4365	1.2	1.3
5:10-5:25	276	1336	1.7	4310	1.2	1.2
5:15-5:30	280	1240	2.4	4292	1.2	1.2
Average	220	1255	**1.6**			**1.1**

The data from Site B resulted in more reasonable results. The exit ramp volumes at Site B were considerably less than those at Site A; however, the frontage road volumes were much higher. At Site B, the average delay predicted by the model differed from the average delay calculated in the field by 0.5 seconds (see Table 4-4). The average delays observed at Site B were still on the order of only one to two seconds per vehicle.

The results of this evaluation revealed that the delay model developed by Gattis et al. works well when exit ramp volumes are low to moderate (i.e., below approximately 1200 vph). When exit ramp volumes exceed 1200 vph, the calculated frontage road capacity at the exit ramp approaches zero and becomes negative.

Both Sites A and B included three lanes on the frontage road. Observing the operations at these sites, researchers found that only the vehicles in the inside and middle lanes yielded to exit ramp vehicles. Frontage road vehicles in the outside lane did not yield at the ramp.

Techniques are not currently 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"). Research on revising Chapter 10 is anticipated to be completed and approved by the Transportation Research Board Highway Capacity and Quality of Service Committee in late 1997. The proposed techniques include delay estimates at two-way and four-way stop-controlled intersections. Traffic behavior at an exit ramp junction is similar to the traffic behavior at a two-way stop-controlled intersection (i.e., the drivers of the frontage road vehicles stop and search for an acceptable gap in the vehicle flow on the ramp). Therefore, the two-way stop-controlled technique should be appropriate. Additional investigation should be conducted to determine if this technique accurately predicts delay at these types of exit ramp junctions.

CHAPTER 5
RESULTS FOR TWO-WAY FRONTAGE ROADS

Two-way frontage roads usually exist in suburban or rural areas where traffic volumes are relatively low, development is limited, and interchange spacing is long. In these situations, two-way frontage roads typically provide less travel time for drivers to reach their destination than one-way frontage roads would because a circuitous route is avoided.

Operational and safety problems arise for two-way frontage roads when traffic volumes reach moderate to high levels. Because state law requires frontage road vehicles to yield to freeway ramp traffic, frontage road delays increase as traffic volumes increase. As with one-way frontage roads, two-way frontage road vehicles traveling in the same direction as freeway traffic must yield at exit ramps. However, two-way frontage roads typically only have one lane in each direction and, therefore, drivers are faced with higher delays at exit ramps. Two-way frontage road vehicles traveling in the opposite direction of freeway traffic must yield at both exit ramps and entrance ramps, causing even further delay. For these reasons, operations on two-way frontage roads may be more sensitive than one-way frontage roads to factors such as traffic volume, access density, and ramp location.

The goal of this study was to analyze operations on existing two-way frontage roads and to determine how certain factors affect frontage road operations. Similar to the analysis of one-way frontage roads, the factors investigated in this study included: link type, link length, frontage road volume, access density, and free-flow speed. The study of two-way frontage road operations involved generating several plots using data from travel time runs and performing a regression analysis.

SPEED AND TRAVEL TIME VERSUS CUMULATIVE DISTANCE

Plotting speed and travel time versus cumulative distance gave the researchers an overall view of how certain frontage road characteristics (such as ramps and signalized intersections) affected operations. For each field site, separate plots were made for travel time runs traveling in the same direction of freeway traffic (designated as *with*) and travel time runs traveling in the opposite direction of freeway traffic (designated as *oppose*). Appendix D provides plots of speed versus cumulative distance and travel time versus cumulative distance for the nine two-way frontage roads.

Figures 5-1 and 5-2 show examples of the speed versus cumulative distance plots. Figure 5-1 represents Site 23 *with* travel time runs, and Figure 5-2 represents Site 23 *oppose* (see Table 3-2 for a description of Site 23). Comparing the speed/distance plots for two-way frontage roads to those for one-way frontage roads (see Figures 4-1 and 4-2) reveals that the two-way frontage road vehicles typically experienced a higher variation in speed. As discussed above, two-way frontage road operations are more sensitive to certain characteristics because the two-way frontage road sites had only one lane in each direction. For example, comparing Figure 5-1 with Figures 4-1 and 4-2, the test vehicle experienced higher delays at exit ramps on two-way frontage roads. In addition, observing the data points in Figure 5-1 for the travel time run that began at 07:08, a delay was incurred by the test vehicle at approximately 2650 m. Looking at the site sketch for Site 23, several driveways and a cross street are in this area. Therefore, the test vehicle was most likely delayed due to a vehicle turning into or out of one of these access points. A similar situation occurred at approximately 1450 m for the travel time run that began at 08:51.

Comparing Figure 5-2 to Figure 5-1, the test vehicle experienced much higher delays when traveling in the opposite direction of freeway traffic. One reason for this was due to the requirement that *opposing* frontage road vehicles yield at both exit and entrance ramps. (See Figure 3-2 for an explanation of the terms *with* and *opposing*.) At entrance ramps, *opposing* frontage road vehicles were observed yielding to all *with* frontage road vehicles (not just those using the entrance ramp).

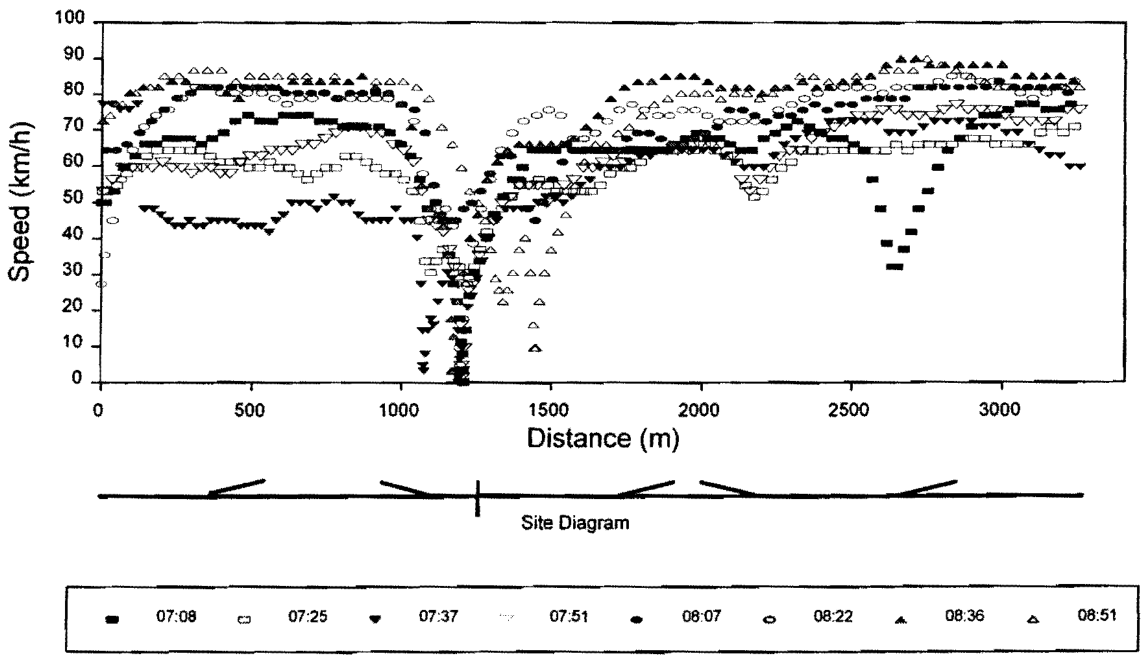

Figure 5-1. Speed Versus Cumulative Distance for Site 23 (*With*).

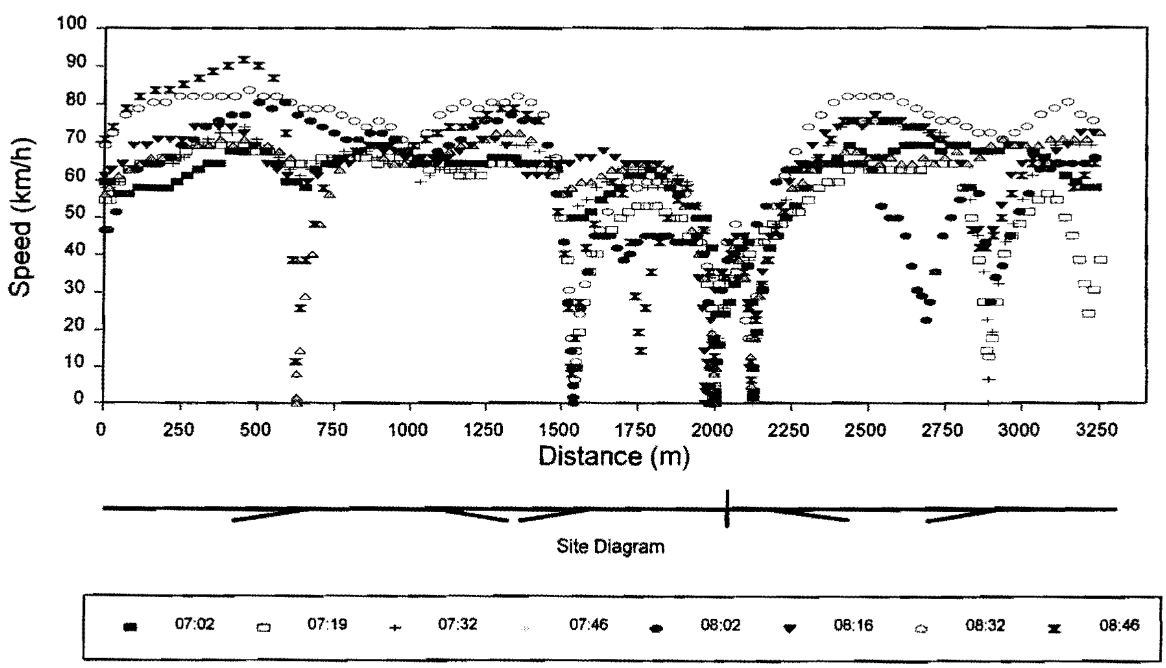

Figure 5-2. Speed Versus Cumulative Distance for Site 23 (*Opposing*).

For the drivers traveling in the *opposing* direction, at freeway entrance ramps it is difficult to determine which *with* vehicles are entering the freeway until the *with* vehicles reach the entrance ramp. Therefore, the delay to *opposing* frontage road vehicles at entrance ramps is heavily dependent upon the total *with* frontage road volume. Examples of the test vehicle being delayed at access points are shown at approximately 1760 m (for 08:46), 2700 m (for 08:02), and 3200 m (for 07:19).

Figures 5-3 and 5-4 show travel time versus cumulative distance for Site 23 *with* and *opposing*, respectively. These figures again reveal the significant effects that the signalized intersections have on frontage road operations. Delays can also be observed at some ramps; however, the signalized intersections still have the greatest influence on operations.

Figure 5-4 also illustrates the effects of signal progression. For most of the *opposing* travel time runs, the test vehicle had to stop for the signalized intersection at approximately 2000 m. The delay at this intersection was about 60 seconds for most of the travel time runs. However, the test vehicle incurred minimal delay at the intersection for the run that began at 08:32, resulting in a significant reduction in overall travel time.

The steps for evaluating operations on arterial streets, presented in Chapter 11 of the *HCM*, will again be followed to develop a procedure for estimating the capacity and level of service on two-way frontage roads. To calculate the average running speed for two-way frontage roads, the total frontage road travel time must first be estimated. The total travel time will consist of the running time, delay at ramps, and delay at signalized intersections. The procedures contained in Chapter 9 of the *HCM* can be used for estimating the capacity and delay at signalized intersections on two-way frontage roads. Procedures developed by Gattis et al. (2) may be used for estimating delay at ramps but must first be evaluated. A new procedure may need to be developed for predicting running time on two-way frontage roads. The following discussions summarize the efforts to develop a procedure for estimating running time and to evaluate the procedures developed by Gattis et al.

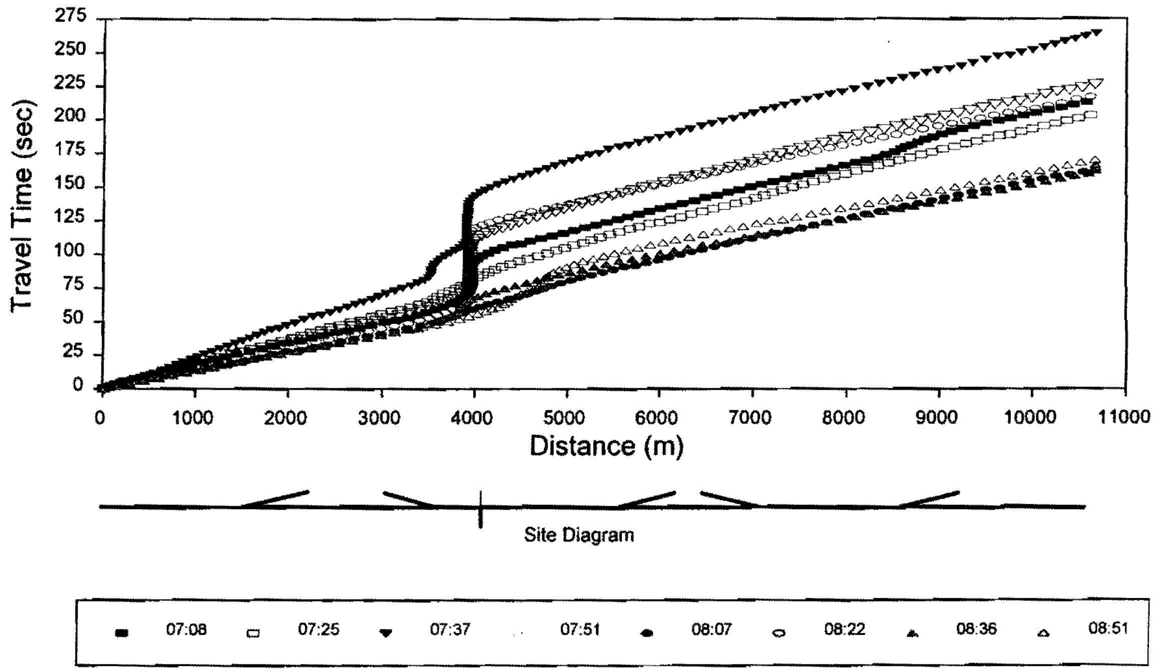

Figure 5-3. Travel Time Versus Cumulative Distance for Site 23 (*With*).

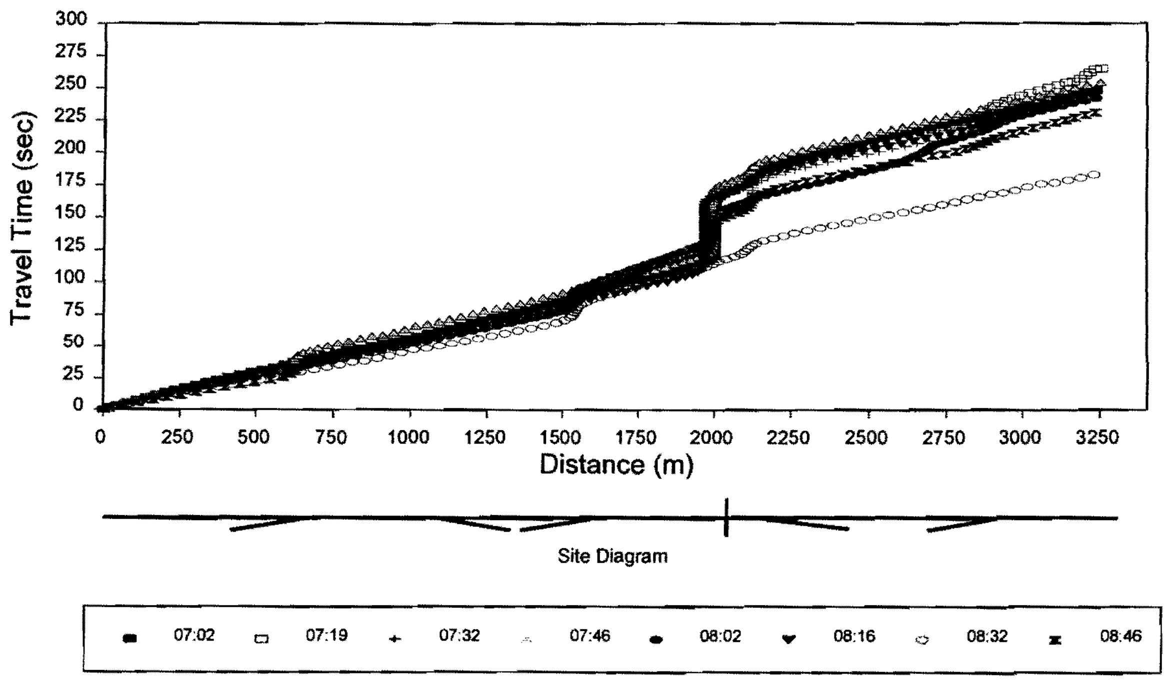

Figure 5-4. Travel Time Versus Cumulative Distance for Site 23 (*Opposing*).

PREDICTING RUNNING TIME

To develop a procedure for estimating running time on two-way frontage roads, the first step was to determine what effect certain factors, such as link length, volume and access density, had on frontage road operations. These efforts consisted of generating more plots and performing a regression analysis. The first factor investigated was link type.

Link Type

To evaluate how link type affected traffic operations on two-way frontage roads, link delays were calculated for each field site. Comparisons were then made by plotting link delay against link type. Each link type was defined by the beginning and ending nodes. The nodes for the two-way frontage road analysis are summarized as follows:

- Signalized Intersection
- Four-Way Stop-Controlled Intersection
- Two-Way Stop-Controlled Intersection
- Freeway Entrance Ramp
- Freeway Exit Ramp
- Arbitrary Beginning or Ending Point

Link delay was evaluated separately for those travel time runs made while traveling in the same direction as the freeway traffic and those traveling in the *opposing* direction. Figures 5-5 and 5-6 show how delay varied by link type for two-way frontage road *with* and *opposing*, respectively. As expected, these figures reveal that the highest delays and the greatest variability are incurred at the signalized intersections.

Based on the findings from Chapter 4 (for one-way frontage roads), the effects of signalized intersections were minimized by deleting those link travel times that had speeds below 8 km/h from

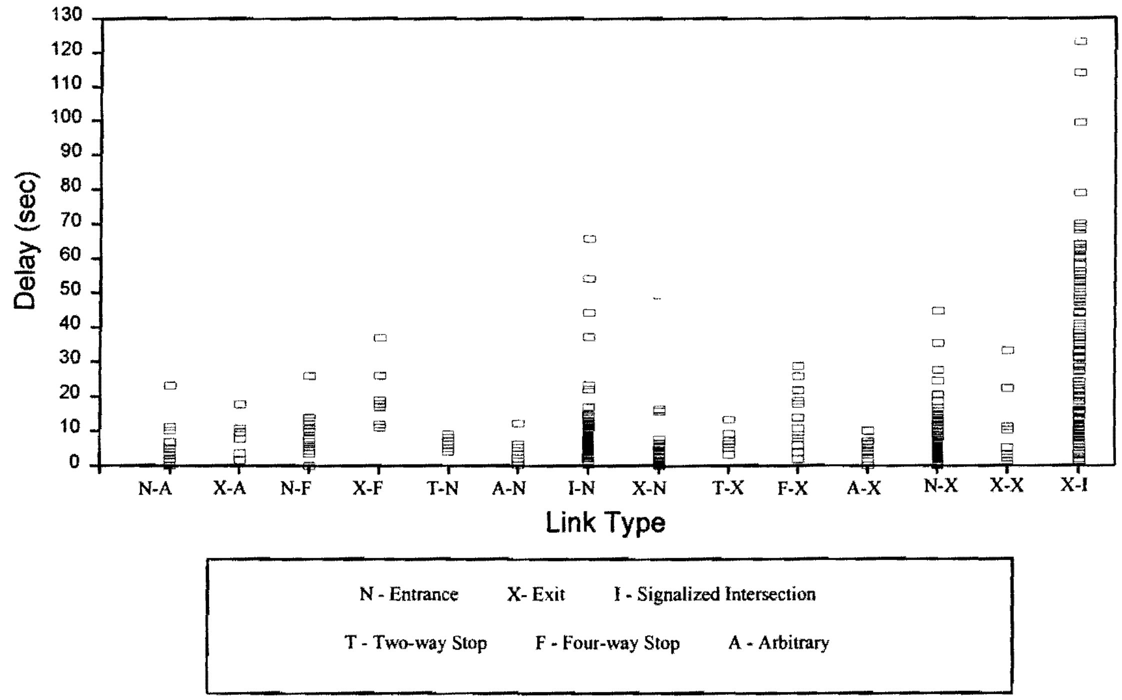

Figure 5-5. Link Delay Versus Link Type (*With*).

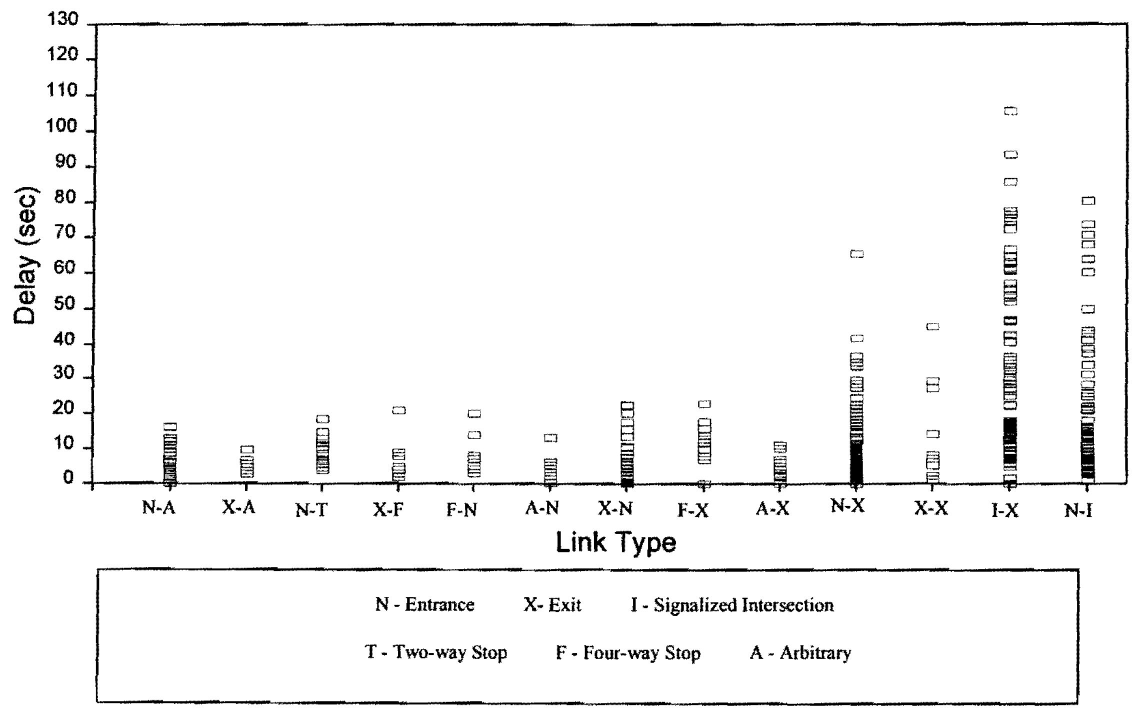

Figure 5-6. Link Delay Versus Link Type (*Opposing*).

the database. Deleting this data minimized the effects that signalized intersections and ramps had on two-way frontage road operations. The resulting database was used for all further evaluations.

Figures 5-7 and 5-8 show the data for all links with a minimum speed greater than 8 km/h for frontage road *with* and *opposing*, respectively. Minimizing the effects of signalized intersections reduced the variability in the data for frontage road *with* (see Figure 5-7). Maximum delays, however, still approached 40 seconds. Maximum delays occurred on those links that ended in an exit ramp or links that included a signalized intersection for either the beginning or ending node. Therefore, two-way frontage road operations in the *with* direction are greatly influenced by signalized intersections and exit ramps.

Figure 5-8 reveals that frontage road vehicles traveling in the *opposing* direction experienced higher delays. The maximum delays approach 50 seconds. The highest delays again occurred on those links that included a signalized intersection or links ending in an exit ramp; however, significant delays also occurred on those links ending in an entrance ramp. Therefore, two-way frontage road operations are not only significantly affected by signalized intersections, but operations are also affected by entrance and exit ramps.

Link Length

Figure 5-9 illustrates the relationship between running time and link length for both *with* and *opposing* travel time runs. The link running times for two-way frontage roads were estimated by deleting those link travel times that had speeds below 8 km/h from the database. As expected, Figure 5-9 reveals that a strong relationship exists between running time and link length for two-way frontage roads, and the relationship appears independent of the direction of travel. Observing Figure 5-9, the variability in the data increases with increasing link length. Comparing these results with the results for one-way frontage roads (see Figure 4-9), more variability exists for the two-way frontage road data; however, the maximum link lengths were approximately twice as long for the two-way frontage road sites as compared with the one-way frontage sites.

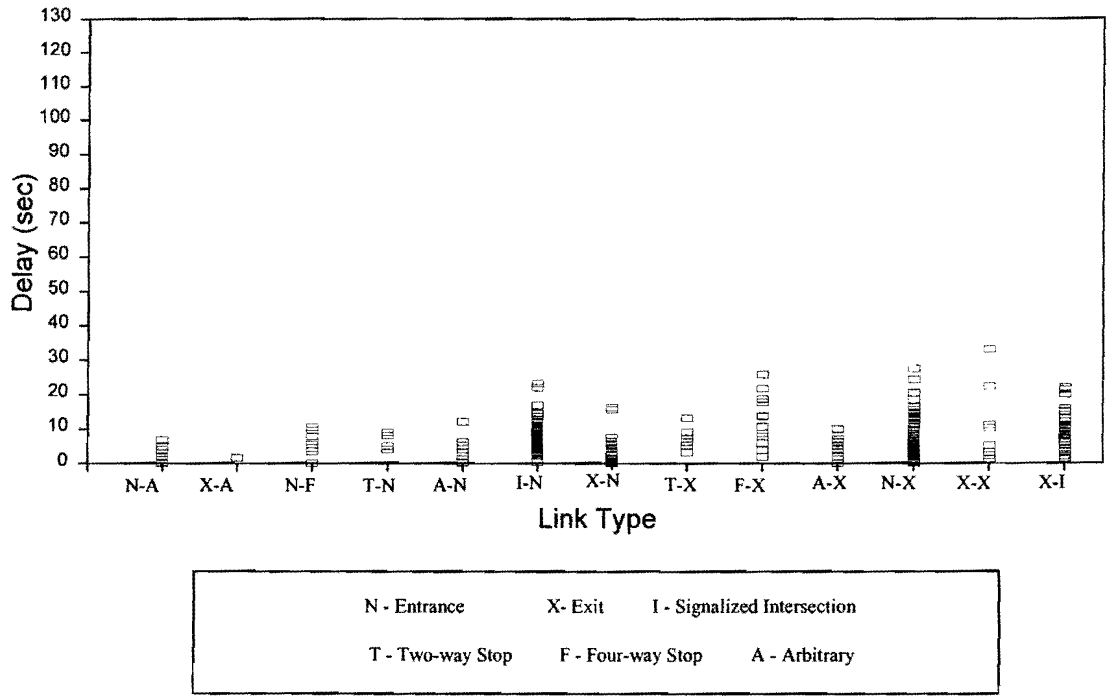

Figure 5-7. Link Delay Versus Link Type for Speeds Greater than 8 km/h (*With*).

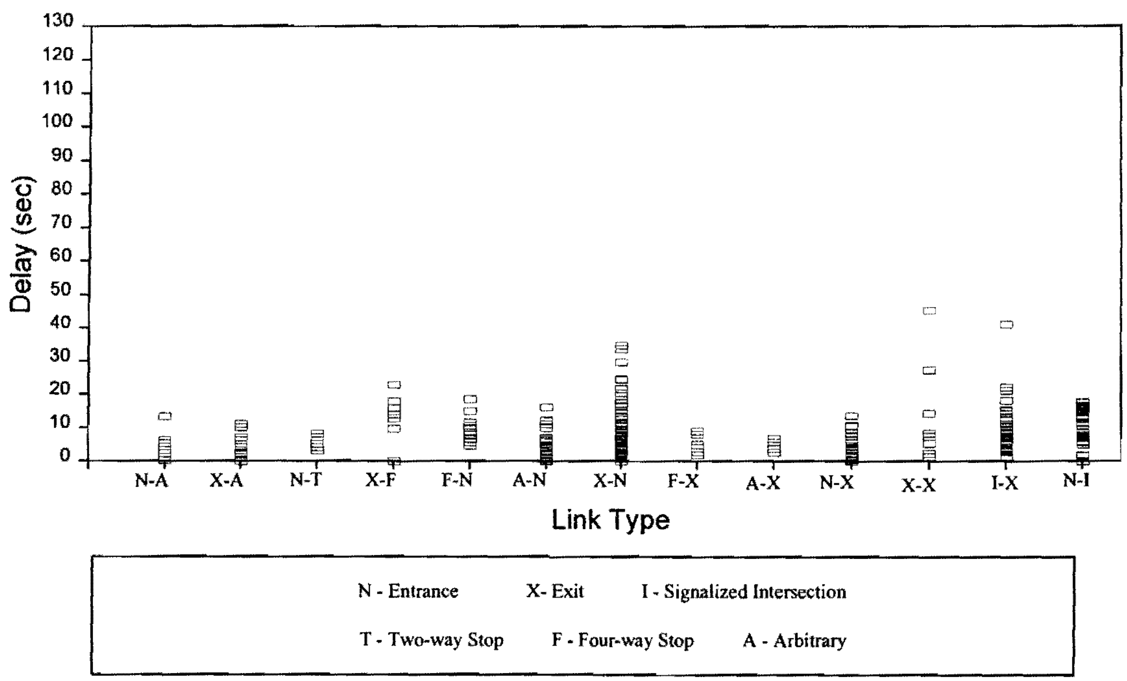

Figure 5-8. Link Delay Versus Link Type for Speeds Greater than 8 km/h (*Opposing*).

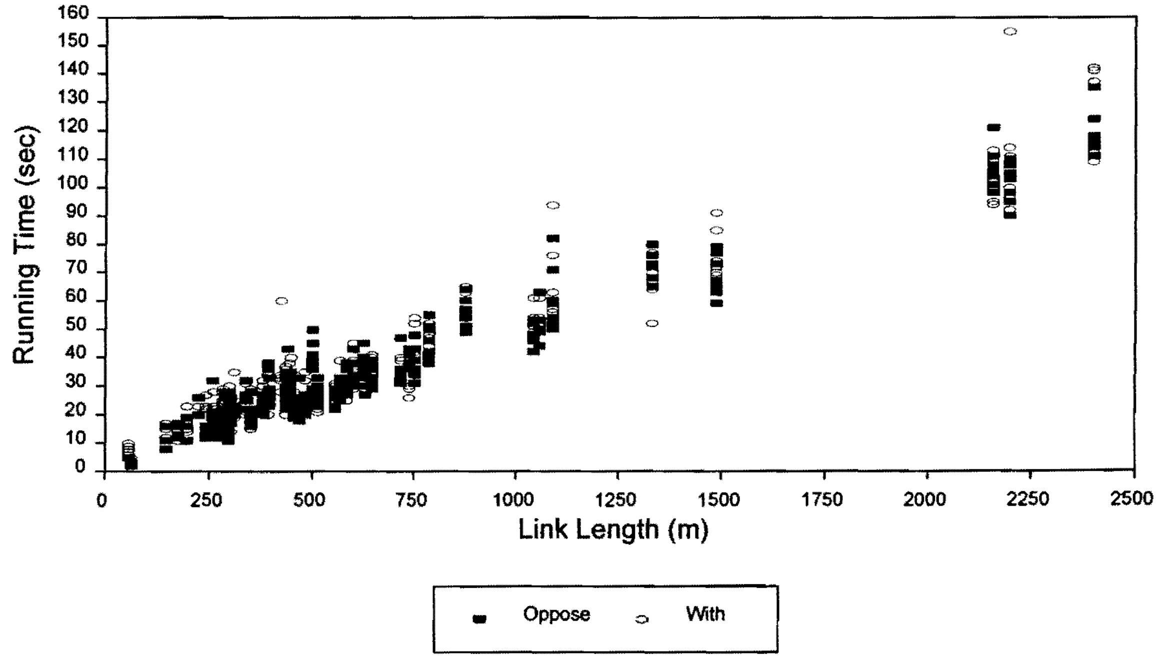

Figure 5-9. Running Time Versus Link Length.

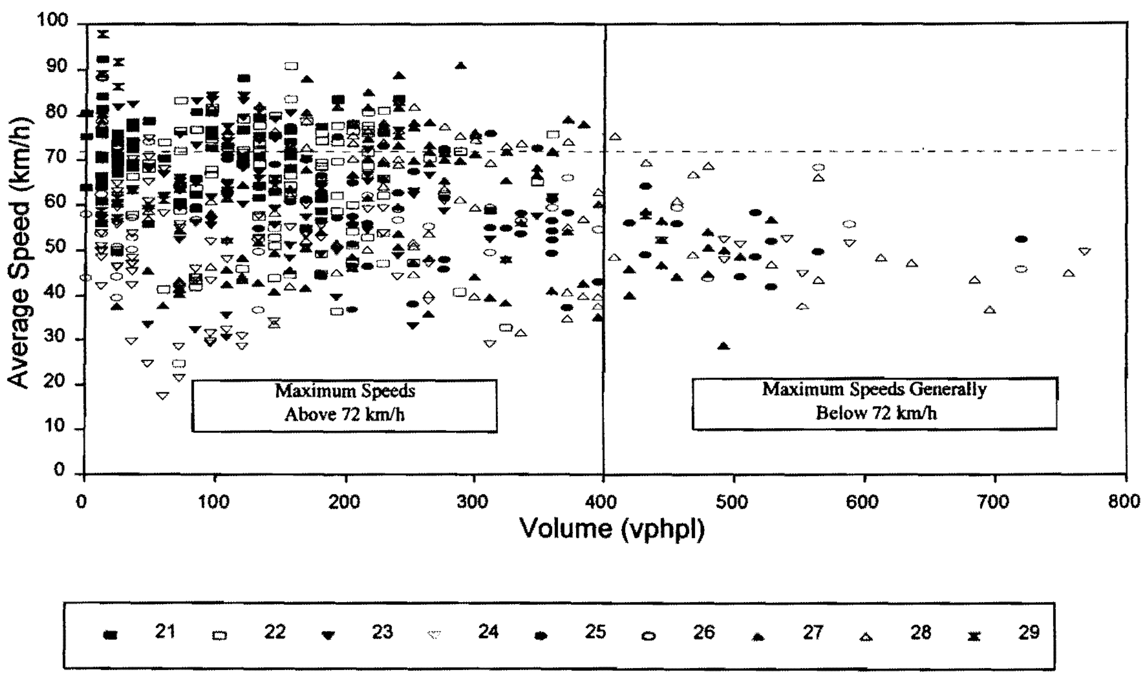

Figure 5-10. Average Speed Versus Volume.

Frontage Road Volume

Because travel time is heavily dependent upon link length, the effects of traffic volume and access density on frontage road operations were investigated using average link speed. For the two-way frontage road sites, frontage road and ramp volumes were counted at each exit and entrance ramp location; therefore, volume counts were obtained for each link at the field sites.

Figure 5-10 shows the relationship between average speed and volume per lane for each link on the nine study sites. As shown in this figure, a high variability exists between speed and volume, and the variability decreases with increasing volume. In addition, the maximum speeds begin to drop above approximately 400 vphpl. Below 400 vphpl, maximum speeds of 90 km/h are observed while above 400 vphpl, most speeds are below 72 km/h. For example, the maximum speeds for Site 27 exceed 89 km/h for volumes below 400 vphpl but do not exceed 64 km/h for volumes above 400 vphpl. Therefore, for the two-way frontage road sites studied, a critical volume of approximately 400 vphpl existed above which traffic operations began to break down. Above 400 vphpl, travel times may increase by as much as 10 to 15 percent.

Access Density

Figure 5-11 illustrates the relationship between average speed and link access density for each of the study sites. Observing the data, some correlation between speed and access density is apparent. For example, maximum speeds of 80 to 90 km/h are reached at most sites for lower access densities. For higher access densities, the maximum speeds do not exceed 72 km/h. The critical access density occurs at approximately 16 accesses per kilometer (acs/km). Therefore, two-way frontage road operations are noticeably influenced when densities are above approximately 16 acs/km. Increases in travel time of about 10 to 15 percent may exist for access densities above this critical value.

Figures 5-12 and 5-13 illustrate the average speed versus access density relationships for links with speed limits of 56 to 72 km/h and 81 to 89 km/h, respectively. The links with speed limits

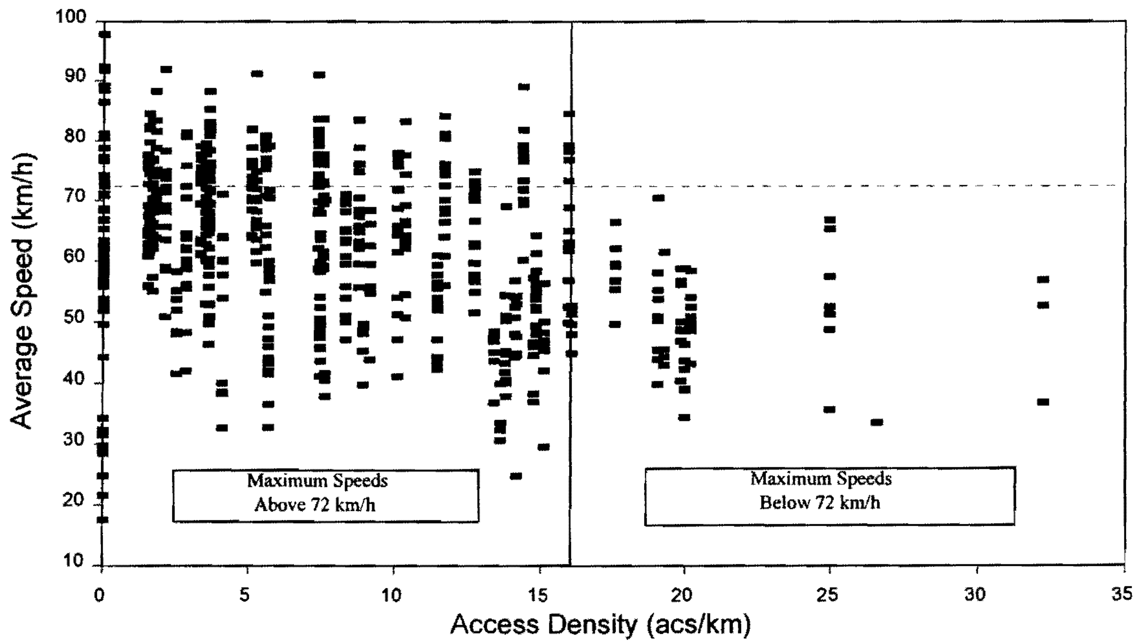

Figure 5-11. Average Speed Versus Access Density.

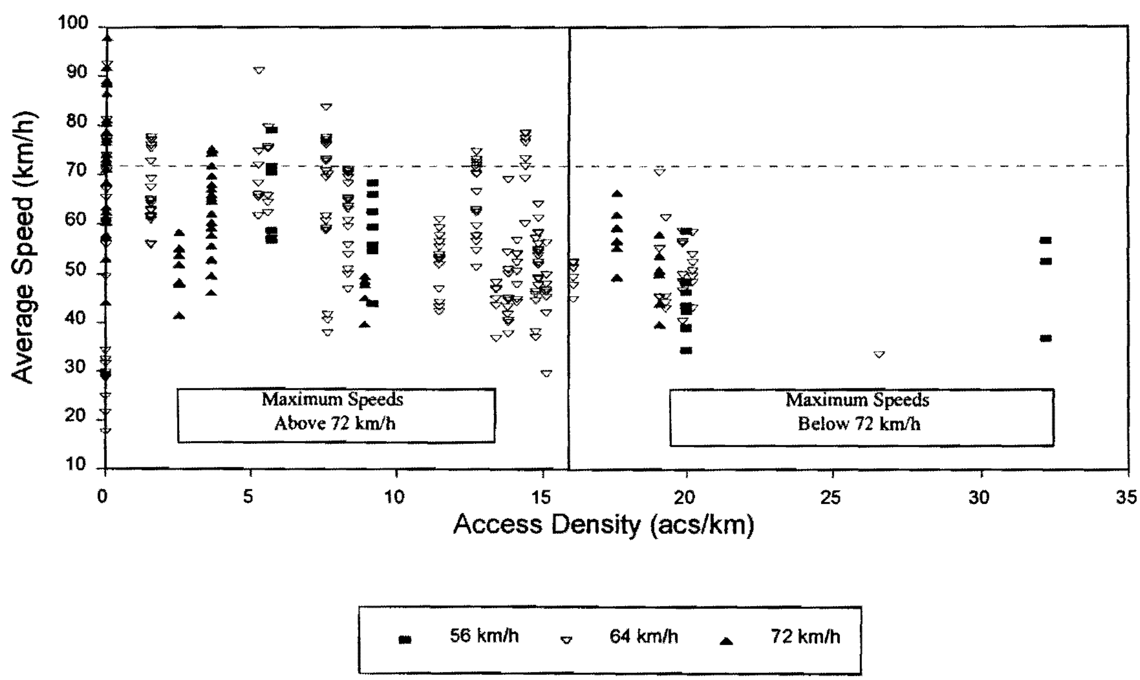

Figure 5-12. Average Speed Versus Access Density for Speed Limits of 56, 64, and 72 km/h.

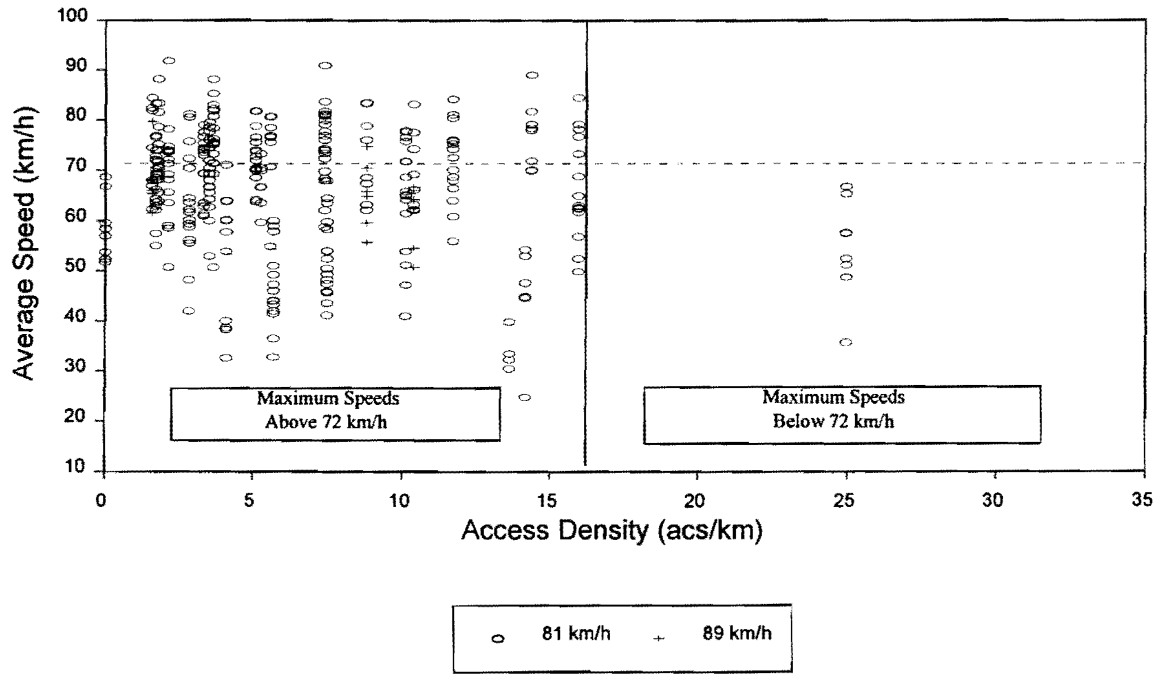

Figure 5-13. Average Speed Versus Access Density For Speed Limits of 81 and 89 km/h.

between 56 and 72 km/h show a strong downward trend in average speed as access density increases. Similar to the findings for one-way facilities, two-way facilities with speed limits of 81 or 89 km/h generally have access densities that are lower than the critical value identified in this research. Figure 5-13 shows that only one link had a speed limit of 81 km/h and an access density greater than 16 acs/km. Similar to one-way frontage roads, these observations support the hypothesis that access density influences driver behavior. The number (or spacing) of driveways and unsignalized intersections noticeably affect drivers' speeds when a threshold value is reached. Below that critical number of driveways/unsignalized intersections, drivers' speeds on two-way facilities are only mildly (or not at all) affected.

Regression Analysis

To develop an equation for predicting running time for two-way frontage road links, another database was created consisting of all the appropriate link data. Like the database for one-way frontage roads, the two-way frontage road database included the following information for each link: maximum and minimum speed, total link travel time, link length, frontage road volume, access density, and free-flow speed. To reduce the effects of signalized intersections, the link data that included minimum speeds below 8 km/h were deleted.

Using *SAS,* stepwise regression was performed on the database to learn how different factors affected running time. The factors investigated included the following: link length, frontage road volume, access density, and free-flow speed. The results from the regression analysis are shown in Table 5-1.

Table 5-1. Results From Stepwise Regression for Two-Way Frontage Roads.

Step	Model[a]	R^2
1	RT = 0.0519(L)	0.976
2	RT = 0.0488(L) + 0.4428(AD)	0.982
3	RT = 0.0479(L) + 0.3226(AD) + 0.0095(V)	0.982
4	RT = 0.0469(L) + 0.2428(AD) + 0.0081(V) + 0.0251(FFS)	0.982

[a]RT = link running time, sec
L = link length, m
AD = access density, acs/km
FFS = free-flow speed, km/h
V = frontage road volume, vph

The variable entered in Step 1 of the stepwise regression process was link length, producing an R^2 value of 0.976. Link length was followed by access density, frontage road volume, and free-flow speed, respectively. The R^2 values calculated in Steps 2, 3, and 4 increased very little from that calculated in Step 1. Like the results from the one-way frontage road analysis, the results from the stepwise regression signify that running time is heavily dependent upon link length and is relatively independent of the other variables. Therefore, the researchers recommend using the equation in Step 1 of Table 5-1 to predict running time for two-way frontage road links.

PREDICTING DELAYS AT RAMPS

Gattis et al. (2) developed methods for predicting delay at ramps on two-way frontage roads. Equations were developed to estimate frontage road delay for the following cases: exit ramp *with*; exit ramp *opposing*; and entrance ramp *opposing*. Table 5-2 shows the recommended equations for predicting ramp delay on two-way frontage roads.

Table 5-2. Equations for Predicting Ramp Delays for Two-Way Frontage Roads.[a]

Scenario	C_R	W	D_R
Exit Ramp *With*	$1724 - 1.6120(Q_R)$	1 / (u-a)	-0.0719 + 1.0922(W)
Exit Ramp *Opposing*	$1444 - 1.6564(Q_R)$	1 / (u-a)	-1.6451 + 1.7785(W)
Entrance Ramp *Opposing*	$1535 - 1.3852(Q_R)$	1 / (u-a)	0.0538 + 1.3027(W)

[a]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 entrance ramp *opposing*, includes all vehicles that approach the entrance ramp from the *with* direction, whether they enter the ramp or not)
u = service rate, veh/sec (C / 3600)
a = frontage road flow rate, veh/sec (volume / 3600)

As shown in Table 5-2, three values are calculated to estimate frontage road delay: frontage road capacity (C_R), average queuing system delay (W), and average total delay (D_R). The model for estimating frontage road capacity is based on headway acceptance and is the same as service rate in queuing theory. The queuing delay is estimated by assuming that the ramp-frontage road intersection area operates as a queuing system. Because non-queuing sources of delay (such as deceleration/acceleration lost time) also exist, total delay measured in the field was regressed against queuing delay to develop models for predicting the average total delay.

The resulting equations for predicting frontage road delay at ramps are expressed as a function of ramp volume, frontage road volume, and gap acceptance parameters. For entrance ramp *opposing* delay, Gattis et al. recommended assuming that the ramp volume 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. In the field, most frontage road vehicles traveling in the *opposing* direction were observed yielding to all vehicles approaching from the *with* direction at the entrance ramps.

The equations in Table 5-2 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. In their report, Gattis et al. state that the actual delays at field sites may vary from the predicted delay, depending upon the average accepted gap of frontage road drivers. The researchers also state that the recommended models should not be used when the queuing system delay is less than 2.5 seconds per vehicle.

Evaluation of Models

To evaluate the equations developed by Gattis et al., field data were collected, and the delays measured in the field were compared with predictions from the equations. Video data were collected at four field sites (see Table 3-3); however, because Site F included a high volume driveway near the exit ramp, only data from Sites C through E were used for the field study.

Two of the field sites (Sites D and E) were used to evaluate the equations for exit ramp *opposing* and exit ramp *with*. For entrance ramp *opposing*, only one field site (Site C) was used. One hour of data representing the peak period was reduced from each site. The data were aggregated into "sliding" 15-minute periods in five-minute increments (for example, 4:00 to 4:15, 4:05 to 4:20, 4:10 to 4:20, etc.). Tables 5-3 and 5-4 compare the results from the field to the results from the procedures developed by Gattis et al. for exit ramp *opposing* vehicles. Tables 5-5 and 5-6 show comparisons for the exit ramp *with* vehicles, and Table 5-7 compares the results for entrance ramp *opposing* vehicles.

For exit ramp *opposing* delay at Site D (see Table 5-3), the 15-minute frontage road delays measured in the field were consistently lower than the delays predicted by the model. Averaging the delays over the entire hour of data collection revealed that the average delay computed in the field was 0.9 seconds below the average delay predicted by the model (3.2 sec/veh compared with 4.1 sec/veh). For exit ramp *opposing* delay at Site E (see Table 5-4), however, the average delay from the field was 0.5 seconds higher than that predicted by the model.

Therefore, for the two field sites used to evaluate the exit ramp *opposing* delay model, the model overestimated the delay for one site and underestimated delay for the other site. The model did not consistently predict delays too low, nor did it consistently predict delays too high.

The average exit ramp *with* delay calculated at Site D was very close (0.1 seconds) to the average delay predicted by the model (see Table 5-5). Looking at the delay calculated for each 15-minute period, the model predicted higher delays for some periods and lower delays for others. The variation in the field-calculated delay was most likely due to the variation of arrival times for frontage road and exit ramp vehicles. Similar results were obtained for the exit ramp *with* delay at Site E (see Table 5-6). For this site, the model-predicted delay was 0.3 seconds higher than that calculated in the field.

Table 5-7 shows the results from the evaluation of the entrance ramp *opposing* delay model. The 15-minute delays calculated in the field were consistently lower than those predicted by the model. The average delay from the field was 1.3 seconds lower than that predicted by the model (4.5 sec/veh compared with 5.8 sec/veh). One reason for the deviation of the field data from the model predictions may be the assumption that the *opposing* frontage road vehicles yield to all *with* frontage road vehicles at the entrance ramp. For example, some situations may exist in which the *opposing* frontage road vehicle only yields to the *with* vehicle if the *with* vehicle is displaying a turn signal. Since only one field site was used to evaluate the model, determining whether the model predicts consistently high delays is difficult.

In summary, the models developed by Gattis et al. for calculating exit ramp *with* and exit ramp *opposing* delays predicted values relatively close to those observed in the field. For both scenarios, the models slightly over-predicted average delays for one field site and slightly under-predicted average delays for the other site. In other words, the models did not consistently under-predict nor over-predict average delay. Therefore, the researchers recommended using these models to predict delay for two-way frontage road vehicles at ramps.

For the entrance ramp *opposing* case, the model over-predicted delay for the one field site studied. However, because the difference in delay between the model and the field data was only about one second, and because the entrance ramp *opposing* delay model generally agrees with the other models, the researchers concluded that this model should also be used for estimating frontage road delays at ramps in the evaluation of level of service on two-way frontage roads.

Table 5-3. Exit Ramp *Opposing* Delay (Site D).

Time Increment	Exit Ramp Volume (vph)	Frontage Road Volume (vph)	Field Delay (sec/veh)	Gattis et al. procedures		
				C_R (vph)	W (sec/veh)	D_R (sec/veh)
3:00 - 3:15	108	184	3.8	1265	3.3	4.2
3:05 - 3:20	104	152	3.1	1271	3.2	4.1
3:10 - 3:25	84	164	2.7	1305	3.2	3.9
3:15 - 3:30	104	108	2.2	1271	3.1	3.9
3:20 - 3:35	120	108	3.6	1245	3.2	3.9
3:25 - 3:40	124	132	4.3	1238	3.3	4.1
3:30 - 3:45	104	148	4.6	1271	3.2	4.0
3:35 - 3:50	92	212	4.1	1291	3.3	4.3
3:40 - 3:55	88	212	2.0	1298	3.3	4.2
3:45 - 4:00	88	220	1.8	1298	3.3	4.3
Average	102	164	**3.2**			**4.1**

Table 5-4. Exit Ramp *Opposing* Delay (Site E).

Time Increment	Exit Ramp Volume (vph)	Frontage Road Volume (vph)	Field Delay (sec/veh)	Gattis et al. procedures		
				C_R (vph)	W (sec/veh)	D_R (sec/veh)
3:00 - 3:15	304	296	12.5	940	5.6	8.3
3:05 - 3:20	284	308	11.8	974	5.4	8.0
3:10 - 3:25	264	316	11.9	1006	5.2	7.6
3:15 - 3:30	212	312	6.5	1092	4.6	6.6
3:20 - 3:35	236	268	7.1	1053	4.6	6.5
3:25 - 3:40	248	212	6.3	1033	4.4	6.2
3:30 - 3:45	248	156	5.3	1033	4.1	5.7
3:35 - 3:50	260	136	4.1	1013	4.1	5.6
3:40 - 3:55	256	164	3.4	1019	4.2	5.8
3:45 - 4:00	280	160	1.9	980	4.4	6.2
Average	260	233	**7.1**			**6.6**

Table 5-5. Exit Ramp *With* Delay (Site D).

Time Increment	Exit Ramp Volume (vph)	Frontage Road Volume (vph)	Field Delay (sec/veh)	Gattis et al. procedures		
				C_R (vph)	W (sec/veh)	D_R (sec/veh)
3:00 - 3:15	108	152	3.8	1385	2.9	3.1
3:05 - 3:20	104	188	3.1	1390	3.0	3.2
3:10 - 3:25	84	176	2.9	1418	2.9	3.1
3:15 - 3:30	104	168	1.2	1390	2.9	3.1
3:20 - 3:35	120	188	2.4	1368	3.0	3.3
3:25 - 3:40	124	216	2.4	1368	3.1	3.3
3:30 - 3:45	104	228	4.2	1402	3.1	3.3
3:35 - 3:50	92	225	3.7	1410	3.0	3.2
3:40 - 3:55	88	198	3.4	1406	3.0	3.2
3:45 - 4:00	88	208	3.7	1413	3.0	3.2
Average	102	195	**3.1**			**3.2**

Table 5-6. Exit Ramp *With* Delay (Site E).

Time Increment	Exit Ramp Volume (vph)	Frontage Road Volume (vph)	Field Delay (sec/veh)	Gattis et al. procedures		
				C_R (vph)	W (sec/veh)	D_R (sec/veh)
3:00 - 3:15	304	304	6.0	1233	3.4	3.7
3:05 - 3:20	284	284	6.3	1141	3.6	3.9
3:10 - 3:25	264	264	5.6	1169	3.6	3.8
3:15 - 3:30	212	212	2.4	1241	3.3	3.5
3:20 - 3:35	236	236	2.2	1208	3.4	3.7
3:25 - 3:40	248	248	1.0	1191	3.5	3.7
3:30 - 3:45	248	248	1.3	1191	3.6	3.9
3:35 - 3:50	260	260	3.9	1174	3.6	3.8
3:40 - 3:55	256	256	2.0	1180	3.4	3.7
3:45 - 4:00	280	280	6.3	1147	3.5	3.7
Average	260	260	**4.0**			**3.7**

Table 5-7. Entrance Ramp *Opposing* Delay (Site C).

Time Increment	Frontage Road Volume *With* (vph)	Frontage Road Volume *Opposing* (vph)	Field Delay (sec/veh)	Gattis et al. procedures		
				C_R (vph)	W (sec/veh)	D_R (sec/veh)
4:30 - 4:45	292	204	3.8	1130	3.9	5.1
4:35 - 4:50	300	200	4.1	1119	3.9	5.2
4:40 - 4:55	316	216	3.4	1097	4.1	5.4
4:45 - 5:00	328	228	3.1	1080	4.2	5.6
4:50 - 5:05	332	208	2.6	1075	4.1	5.5
4:55 - 5:10	352	232	2.9	1047	4.4	5.8
5:00 - 5:15	408	208	5.6	969	4.7	6.2
5:05 - 5:20	400	228	5.4	980	4.8	6.3
5:10 - 5:25	404	232	4.8	975	4.8	6.4
5:15 - 5:30	392	236	6.9	992	4.8	6.3
Average	352	219	**4.5**			**5.8**

CHAPTER 6

DEVELOPMENT OF ANALYSIS PROCEDURE

Chapter 11 of the *HCM* (1) provides procedures for evaluating the operations on urban and suburban arterials. These procedures, however, may not be appropriate for direct use on frontage roads because frontage roads contain characteristics (such as freeway exit and entrance ramps) that are not present on arterial streets. Although the *HCM* procedures may not be applied directly to frontage roads, they can be used as a framework.

Several advantages exist for modifying the existing *HCM* arterial procedure rather than developing a new procedure. For example, the *HCM* is the state-of-the-practice in the level-of-service/capacity evaluations. Individuals who perform those evaluations are familiar with the *HCM*. The *HCM* and/or work being done to update the *HCM* contains the most current information on evaluation techniques not only for arterial streets but for signalized intersections and stop-controlled intersections as well. Professionals evaluating operations on frontage roads will need access to those updates to produce the most accurate results.

The *Highway Capacity Software* (*HCS*) is a computerized version of the worksheets contained in the *HCM*. As such, it provides a simpler and faster method for calculating the capacity or level of service of a facility. Desirably, these benefits should also be available in evaluating frontage roads. Because the *HCS* allows the user to add delay and modify total travel time, the software can be used to evaluate frontage roads. The user will need to calculate the delays at ramp junctions and stop-controlled intersections and enter those delays into the *HCS* worksheets.

The intent of this task was to modify the *HCM* arterial analysis procedures to evaluate operations on one-way and two-way frontage roads. Following is a discussion of the modifications made to the *HCM* procedures and an evaluation of the procedures.

MODIFICATION OF *HCM* PROCEDURES

The *HCM* arterial level-of-service criteria is based upon average travel speed. The average travel speed includes the time that vehicles are in motion and the time that they are delayed at intersections. This value is computed by dividing the length of the arterial under investigation by the total travel time. The total travel time consists of the running time and the delay at signalized intersections and/or other locations. The steps outlined in the *HCM* for evaluating arterial operations are summarized as follows:

- Step 1: Establish Roadway to be Considered
- Step 2: Determine Roadway Class
- Step 3: Define Roadway Sections
- Step 4: Compute Running Time
- Step 5: Compute Intersection Approach Delay
- Step 6: Compute Average Travel Speed
- Step 7: Assess the Level of Service

The procedures listed above may also be used for evaluating frontage road operations. Following is a discussion of these procedures as they apply to evaluating frontage road operations.

Step 1: Establish Roadway to be Considered

The first step in evaluating frontage road operations is to select the roadway to be investigated. This process involves determining the location and length of frontage road to be included in the analysis and gathering the needed data (e.g., roadway characteristics, traffic data, and signal data).

Step 2: Determine Roadway Class

In the arterial analysis procedures, the arterial class is associated with estimating the running time and predicting the level of service. The *HCM* defines three arterial classes based on the arterial's function and design. The functional categories include principal arterials and minor arterials. The design categories consist of suburban, intermediate, and urban. Based on the findings from the travel time runs, Class 1 best represents both one-way and two-way frontage roads. Class 1 defines a principal arterial with a typical suburban design and free flow speeds ranging from 56 to 72 km/h. The free flow speeds were generally 64 km/h for two-way facilities and 72 km/h for one-way facilities.

Step 3: Define Roadway Section

For analysis purposes, the frontage road section under investigation should be divided into similar segments. A *segment* defines an area of frontage road with similar roadway and operational characteristics, such as speed limits, traffic volumes, and access densities. A segment is typically bound by signalized intersections but may include any combination of frontage road links.

Step 4: Compute Running Time

The total travel time along a section of frontage road is composed of the running time, intersection approach delay, and ramp delay. The *running time* is the time that it takes a vehicle to traverse a given segment of frontage road without being delayed by signalized intersections, stop-controlled intersections, or freeway ramp junctions. The *HCM* provides a procedure for estimating running time based on segment length, free flow speed, and arterial class. Table 6-1 contains running times recommended in the *HCM* for Class 1 arterials.

As stated previously, frontage roads possess different characteristics than arterials. For example, frontage road operations are influenced by freeway ramps and have driveways on only one side of the roadway. For these reasons, new equations were developed to predict running time on frontage roads. Researchers used travel time data collected in the field to analyze the factors affecting travel time. Results from regression analyses revealed that only length significantly affected frontage road travel time. Therefore, regression equations were developed to predict running time on one-way and two-way frontage roads based on length. Figure 6-1 shows a comparison of running times from the regression equations and the *HCM*. Free flow speeds observed in the field ranged from 64 to 81 km/h (see Appendices B and D); therefore, the *HCM* recommended running times for 64 km/h and 72 km/h are included in the figure.

Table 6-1. Arterial Running Time Recommended in *HCM* for Class 1 Arterials. (1)

Free Flow Speed (km/h)	72	64	56
Average Segment Length (km)	Running Time Per Kilometer (sec/km)		
0.32	68	72	78
0.48	62	63	68
0.64	58	60	65
0.81	55	58	64
1.61	50	56	64

Figure 6-1 shows that the running times predicted by the regression equations for one-way and two-way frontage roads are very similar. The travel time for two-way frontage roads is only slightly greater than the predicted travel time for one-way frontage roads, and the difference becomes greater as length increases. The estimated travel time from the *HCM* for a free flow speed of 72 km/h is also very similar to the predicted travel times for both one-way and two-way frontage roads, especially at longer lengths (i.e., above 0.8 kilometers). At shorter lengths, the *HCM* (for a free flow speed of 72 km/h) predicts travel times as much as five seconds higher than the regression equations. For a free flow speed of 64 km/h, the *HCM* consistently predicts greater travel times than the regression equations.

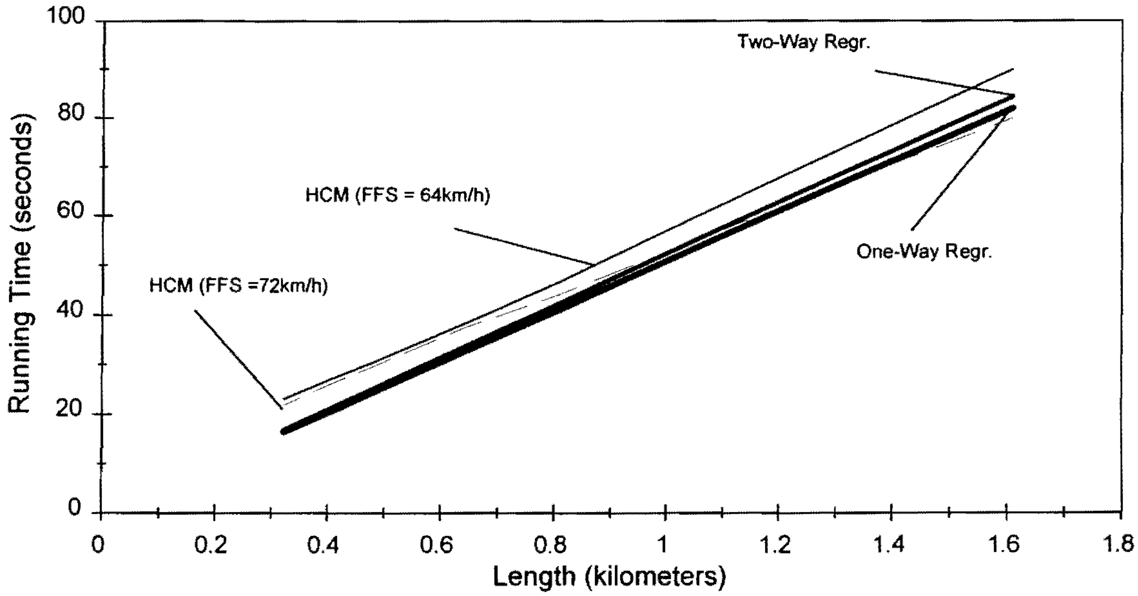

Figure 6-1. Comparison of Running Times from HCM and Regression Equations.

Results from the field studies showed that access density affected running time. For both one-way and two-way frontage road sites, critical access densities existed at which the maximum observed speeds dropped noticeably. Above these critical values, travel times were predicted to increase by as much as 10 percent. The critical access density for one-way frontage roads was approximately 20 acs/km. For two-way frontage roads, the critical value occurred at approximately 16 acs/km.

For one-way frontage roads, the field results showed no correlation between frontage road volume and speed. For two-way frontage roads, however, a critical volume existed above which a noticeable drop in maximum speeds was observed. Above a critical volume of approximately 400 vph, travel times were again predicted to increase by as much as 10 percent.

Step 5: Compute Intersection Approach Delay

Additional delays for frontage road vehicles are incurred at signalized intersections, stop-controlled intersections, and ramp junctions. Chapter 9 of the *HCM* includes procedures for estimating approach delay at signalized intersections. These procedures involve estimating stop delay based on intersection capacity, cycle length, green/cycle time ratio, and volume/capacity ratio. The total delay (including deceleration and acceleration time) is estimated by multiplying the stopped delay by an adjustment factor.

Chapter 10 of the 1994 *HCM* includes procedures for predicting approach total delay at two-way and all-way stop-controlled intersections. These procedures will be updated in the next version of the *HCM.* The revised procedures in Chapter 10 are expected to be available in late 1997.

The delay incurred by frontage road vehicles at freeway ramps will be estimated using the procedures developed by Gattis et al. (2). For two-way frontage roads, both *opposing* and *with* movements are delayed at exit ramps. In addition, the *opposing* movement on two-way frontage roads is also delayed at entrance ramps. Results from the field studies revealed that vehicles on one-way frontage roads only experienced significant delays at exit ramps when an auxiliary lane was not present. Frontage road delay at ramps for all other cases was negligible.

Step 6: Compute Average Travel Speed

The average travel speed is computed by dividing the length of the frontage road section under investigation by the total travel time. The total frontage road travel time is computed by summing the running time, intersection approach delays (both signalized and stop-controlled), and ramp delay.

Step 7: Assess the Level of Service

The *HCM* level-of-service criteria for evaluating arterial operations are based upon roadway class and average travel speed. The criteria recommended for Class 1 roadways in the HCM will be used for evaluating operations on one-way and two-way frontage roads.

EVALUATION OF ANALYSIS PROCEDURE

After the *HCM* arterial analysis procedures were modified for estimating frontage road operations, the next step was evaluation. This was accomplished by using the *Highway Capacity Software (HCS)*, Release 2.1. This program includes the arterial analysis procedures in Chapter 11 of the 1994 *HCM.* The arterial analysis module requires the user to input a description of the roadway characteristics (i.e., class, free flow speed, segment length, and other general information) and signal timing information (i.e., cycle length, intersection capacity, green/cycle time ratio, volume/capacity ratio, and arrival type) and uses the Chapter 11 procedures to estimate the level of service. Both one-way and two-way frontage road analysis procedures were evaluated.

One-Way Frontage Roads

From the 20 one-way frontage road sites studied (see Table 3-1), six were chosen for this analysis (sites 7, 8, 13, 14, 17, and 19). The primary criterion for selecting sites for use in the analysis was the available intersection approach volume. Because frontage road and ramp volumes were only recorded at selected locations along the frontage road, intersection approach volumes could not be computed for all sites. The selected sites included a range of intersection spacings, frontage road volumes, access densities, and freeway exit and entrance ramp locations.

Table 6-2 lists the site information used in the evaluation procedure. The segment lengths and access densities for the selected field sites were computed from the schematics of the field sites. Free flow speeds were estimated from the plot of speed versus cumulative distance (see Appendix B) and with consideration of the posted speed limit. The cycle length and green/cycle time ratio (g/C) were measured in the field. This information was not always precise because some of the signalized intersections were semi-actuated. In these circumstances, average values were used. Arrival type was estimated using the plots of travel time versus cumulative distance (see Appendix B). The arrival type was typically selected to be either 2 or 3. These arrival types represent from unfavorable progression to random arrival. Intersection capacity was estimated by multiplying the saturation flow rate by the g/C ratio. The saturation flow rate was assumed to be 1800 vphgpl (saturation flow rates between 1750 and 1850 vphgpl are suggested in the *HCM*).

Table 6-2. Site Information Used in Evaluation of One-Way Frontage Roads.

Site	Num of Runs	Total Length (km)	Sgmnt [a]	Sgmnt Length (km)	Access Density (acs/km)	Free Flow Speed (km/h)	Range of g/C Ratios [b]	Range of v/c Ratios	Cycle Length Range [b] (sec)
7	12	2.1	1	0.6	6.7	72	0.57	0.09 - 0.13	90 - 105
			2	0.5	0.0	64	0.22	0.73 - 1.10	125 - 136
			3	1.0	13.2	72	0.26 - 0.32	0.09 - 0.30	100 - 110
8	12	2.1	1	1.0	19.0	72	0.26 - 0.31	0.20 - 0.41	120 - 144
			2	0.5	8.6	64	0.25 - 0.27	0.50 - 1.15	60 - 120
			3	0.6	10.0	72	0.37 - 0.48	0.10 - 0.51	88 - 92
13	8	3.7	1	1.2	9.8	72	0.23	0.10 - 0.20	70
			2	1.2	11.2	72	0.47	0.03 - 0.11	83
			3	1.3	3.0	72	0.36	0.04 - 0.17	70
14	8	3.7	1	1.3	3.8	72	0.31	0.11 - 0.25	52
			2	1.2	10.5	72	0.31	0.17 - 0.40	52
			3	1.2	9.2	72	0.30	0.56 - 1.20	100
17	12	2.60	1	0.9	31.6	64	0.18	0.35 - 0.96	80
			2	0.9	22.2	72	0.36	0.05 - 0.12	55
			3	0.8	23.9	72	0.18	0.50 - 1.10	100
19	12	3.9	1	1.2	22.1	72	0.25 - 0.28	0.15 - 0.53	95 - 120
			2	0.9	29.5	64	0.25 - 0.34	0.20 - 0.41	80 - 100
			3	1.8	14.7	72	0.29 - 0.50	0.25 - 0.49	50 - 75

[a] Frontage road section was divided into like-segments for analysis purposes.
[b] Varied from off-peak to peak periods.

At each field site, 8 to 12 travel time runs were made. The frontage road sites were analyzed by dividing each site into segments (from signalized intersection to signalized intersection). Running time on each segment was estimated using two different procedures: (1) the *HCM* arterial analysis procedures as determined by the *HCS* and (2) the regression equation developed from the field data. Signalized intersection delay was computed by *HCS* using the *HCM* procedures. Delays at exit ramps were computed for those ramps without auxiliary lanes and were entered in the *HCS* as "Other Delay." The total travel times calculated were averaged for all runs at each site, and average travel speeds were computed by dividing the length of the site by the average travel times. The calculated average travel speeds were then compared to those measured in the field. Table 6-3 shows the results of the evaluation.

The average travel speeds predicted by the *HCS* using the *HCM* procedures to predict running time were typically lower than the travel speeds calculated from the regression equation (see Table 6-3 and Figure 6-2). Using the regression equation to estimate running time, researchers found the average travel speeds were typically higher than those in the field. For both procedures, however, the estimated average travel speeds were within plus or minus 2.5 km/h of the actual travel speeds measured in the field.

From the results of the evaluation, the researchers concluded that the modified *HCM* arterial analysis procedures can be used to estimate the average travel speed on one-way frontage roads. Because the regression equation to calculate running time was derived from field data collected at existing frontage road sites, the researchers recommend using the regression equation in conjunction with the *HCM* procedures for calculating the total travel time. The *HCM* procedures for calculating running time, however, also produced reasonable results.

Table 6-3. Evaluation of One-Way Frontage Road Analysis Procedure.

Site	Segment	Running Time (sec)		Avg. Intrsct. Delay (sec)	Exit Ramp Delay[c] (sec)	Average Travel Speed (km/h)		
		HCM	Regrsn. Equation[a,b]			*HCS* *HCM* Running Time	*HCS* Regrsn. Running Time	Field
7	1	35.3	30.2	12.9	N/A	32	34	34
	2	30.6	24.5	74.8				
	3	52.6	49.9	28.9				
8	1	56.3	50.7	39.1	N/A	32	35	34
	2	30.6	23.7	44.6				
	3	35.4	30.2	20.2				
13	1	63.7	62.2	21.7	N/A	55	55	53
	2	60.1	58.9	14.4				
	3	67.3	67.1	16.2				
14	1	67.3	67.1	13.2	N/A	48	48	48
	2	60.1	58.1	14.1				
	3	62.3	60.5	56.6				
17	1	56.1	49.5	36.3	N/A	34	37	35
	2	51.0	47.7	12.7				
	3	50.2	46.8	70.2				
19	1	67.8	65.7	37.4	2.7	45	48	47
	2	67.8	60.3	25.8	1.3			
	3	80.0	81.9	23.3	1.1			

[a] Running time was increased by 10 percent for those segments with an access density greater than 20 acs/km.
[b] Regression equation: running time = 0.0504 (segment length, m).
[c] Frontage road delay at exit ramps without auxiliary lanes.

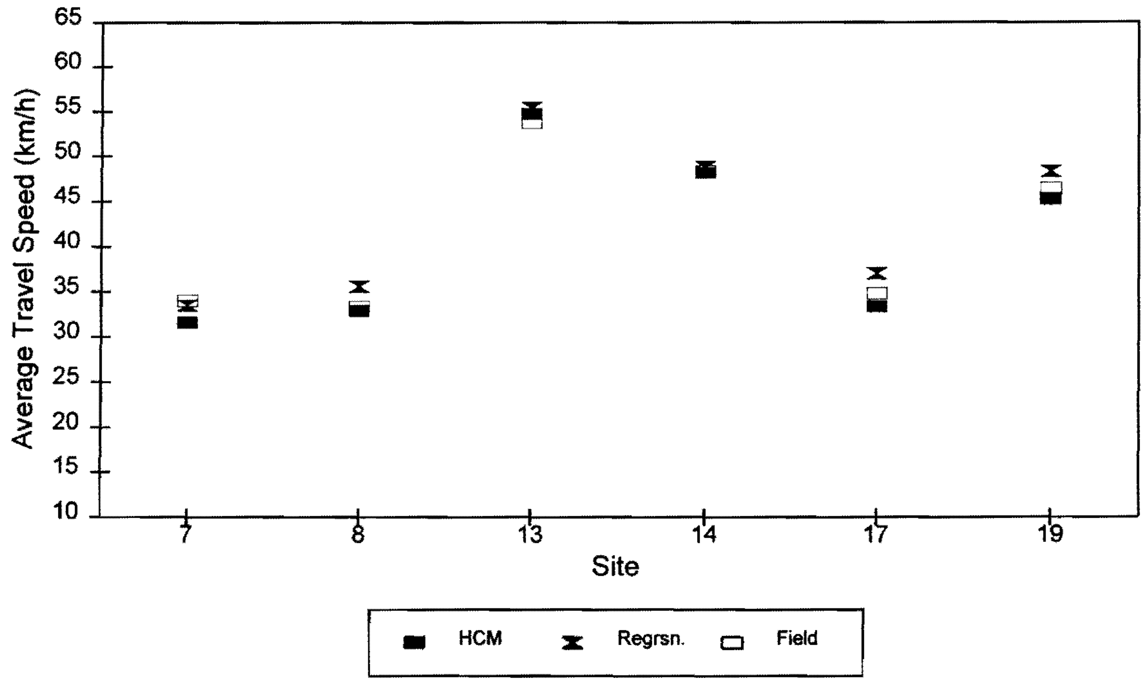

Figure 6-2. Evaluation of Level-of-Service Procedure for One-Way Frontage Roads.

Two-Way Frontage Roads

The *HCS* was again used to evaluate the procedure for estimating the level of service on two-way frontage roads. From the nine two-way frontage road sites (see Table 3-2), three were selected for this analysis (Sites 25, 27, and 28). Both the *with* data (traveling in the same direction as freeway traffic) and *opposing* data (traveling in the opposite direction of freeway traffic) were used. For each site, the modified *HCM* procedures were used to estimate average travel speed (in each direction), and these values were compared to the field data.

Table 6-4 lists the site information used in the evaluation procedure. The segment lengths, access densities, free flow speeds, and signal timing information were obtained using the same procedure used for the evaluation of one-way frontage roads (see preceding section). The arrival type was assumed to be 2 (representing unfavorable progression). The saturation flow rate was assumed to be 1800 vphgpl. Table 6-5 shows the results of the evaluation.

The total travel time at each site included the running time, delay at signalized intersections, and delay at ramp junctions. In the *with* direction, delays were experienced only at exit ramps; however, for the *opposing* direction, delays were experienced at exit and entrance ramps. The running time was estimated using two different procedures: (1) the *HCM* arterial analysis procedures and (2) the regression equation developed from the field data.

The average speeds calculated using the *HCM* arterial analysis procedures were generally within 3 km/h (see Table 6-5 and Figure 6-3) of the speeds measured in the field. The regression equations predicted lower travel times, which resulted in higher average travel speeds as compared to the field data. The differences were between 1 and 8 km/h. Based on these results, the developed frontage road analysis procedure produces reasonable values.

Table 6-4. Site Information Used in Evaluation of Two-Way Frontage Roads.

Site	Direction of Travel	Num of Runs	Total Length (km)	Sgmnt [a]	Sgmnt Length (km)	Access Density (acs/km)	Free Flow Speed (km/h)	Range of g/C Ratios [b]	Range of v/c Ratios	Cycle Length [b] (sec)
25	*With*	9	3.09	1	1.77	9.04	56	0.20	0.23 - 0.87	170
				2	1.32	16.66	56			
	Opposing	9	3.09	1	1.32	16.66	56	0.34	0.42 - 0.88	182
				2	1.77	9.04	56			
27	*With*	8	6.39	1	0.56	16.16	64	0.31	0.17 - 0.46	65
				2	2.91	10.29	64	0.34	0.25 - 0.56	90
				3	2.89	5.53	64	0.37	0.04 - 0.22	70
	Opposing	8	6.39	1	2.89	5.53	64	0.34	0.17 - 0.64	90
				2	2.91	10.29	64	0.31	0.24 - 0.54	65
				3	0.56	16.16	64			
28	*With*	8	6.26	1	2.77	6.15	64	0.29	0.25 - 0.69	90
				2	2.98	5.36	64	0.37	0.36 - 0.61	65
				3	0.50	11.91	64			
	Opposing	8	6.26	1	0.50	11.91	64	0.37	0.07 - 0.29	65
				2	2.98	5.36	64	0.29	0.46 - 0.83	90
				3	2.77	6.15	64	0.21	0.50 - 1.03	70

[a] Frontage road section was divided into like-segments for analysis purposes.
[b] Varied from off-peak to peak periods.

Table 6-5. Evaluation of Two-Way Frontage Road Analysis Procedure.

Site	Direction of Travel	Segment	Running Time (sec)		Average Intersection Delay (sec)	Entrance Ramp Delay[c] (sec)	Exit Ramp Delay (sec)	Average Travel Speed (km/h)		
			HCM	Regrsn. Equation[a,b]				*HCS*		Field
								HCM Running Time	Regrsn. Running Time	
25	*With*	1	112	92	62	--	7	42	47	39
		2	85	75	0					
	Opposing	1	85	75	57	14	6	41	45	37
		2	112	92	0					
27	*With*	1	35	32	18	--	12	53	57	54
		2	163	151	25					
		3	161	150	16					
	Opposing	1	161	150	25	10	14	54	58	52
		2	163	151	14					
		3	35	32	0					
28	*With*	1	155	144	28	--	7	60	60	54
		2	167	155	18					
		3	31	26	0					
	Opposing	1	31	26	15	15	9	48	51	50
		2	167	155	33					
		3	155	144	40					

[a] Running time was increased by 10 percent for those segments with an access density greater than 16 acs/km or frontage road volumes exceeding 400 vphpl.
[b] Regression equation: running time = 0.0519(segment length, meters).
[c] Entrance ramp delay for *opposing* direction only.

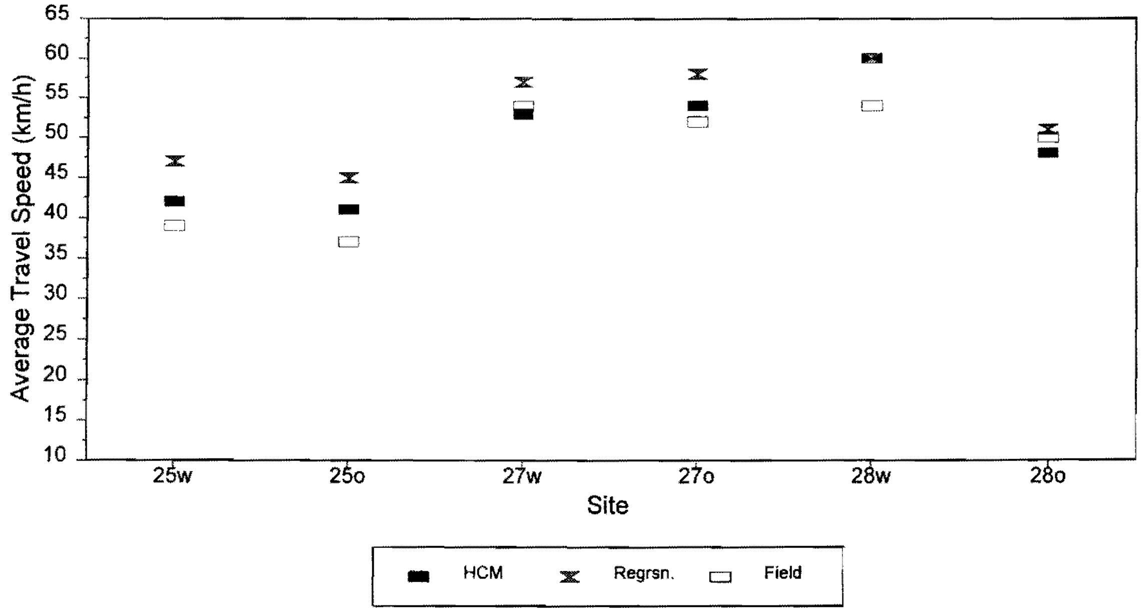

Figure 6-3. Evaluation of Level-of-Service Procedure for Two-Way Frontage Roads.

CHAPTER 7
LEVEL-OF-SERVICE ANALYSIS PROCEDURE

OPERATIONS APPLICATION

The procedure for determining frontage road level of service has been divided into seven steps (see Figure 7-1). The procedure listed in Figure 7-1 applies to both one-way and two-way frontage roads. The evaluation for two-way frontage roads differs from the one-way frontage road evaluation 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 area 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

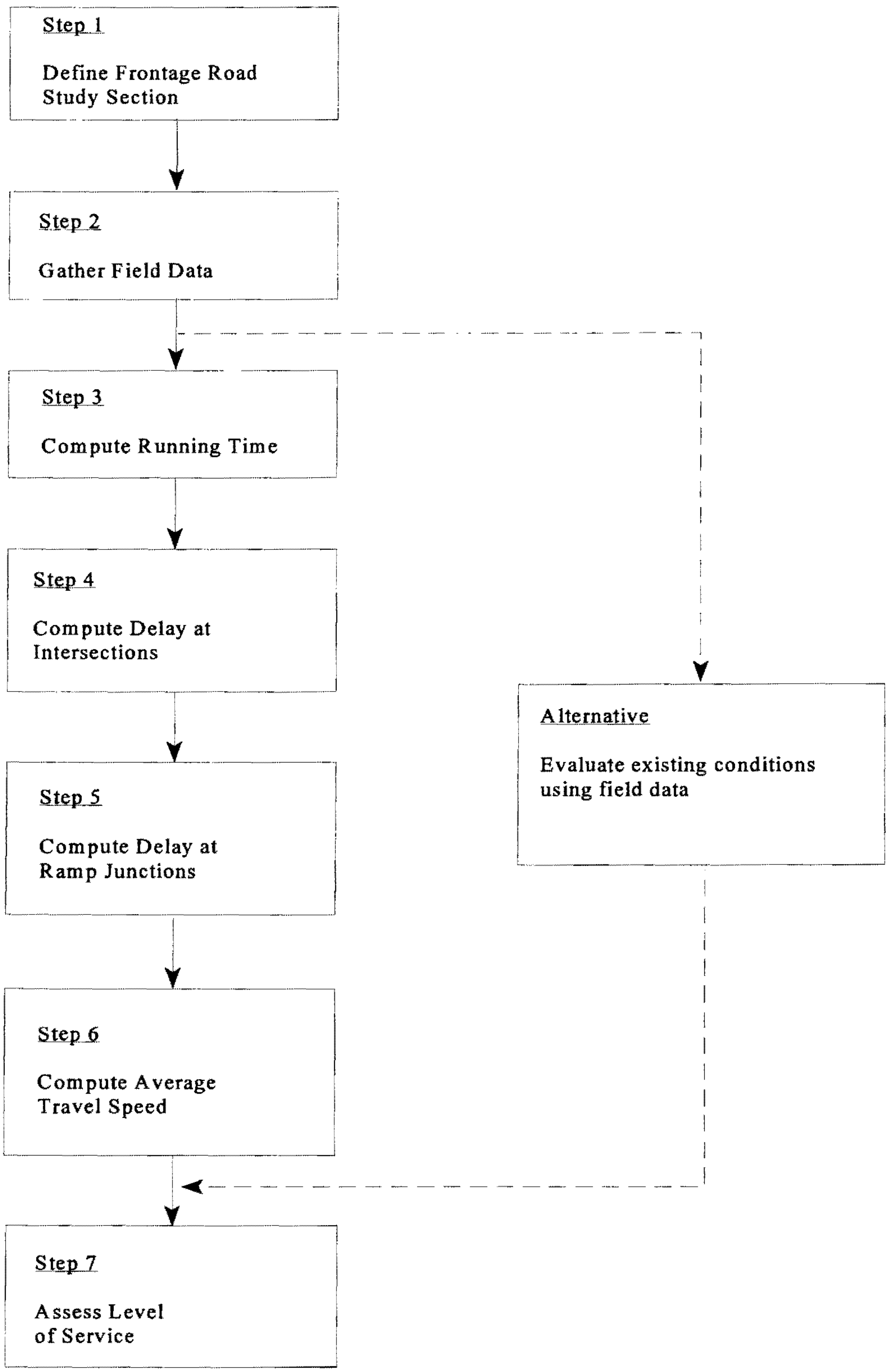

Figure 7-1. Level-of-Service Analysis Procedure.

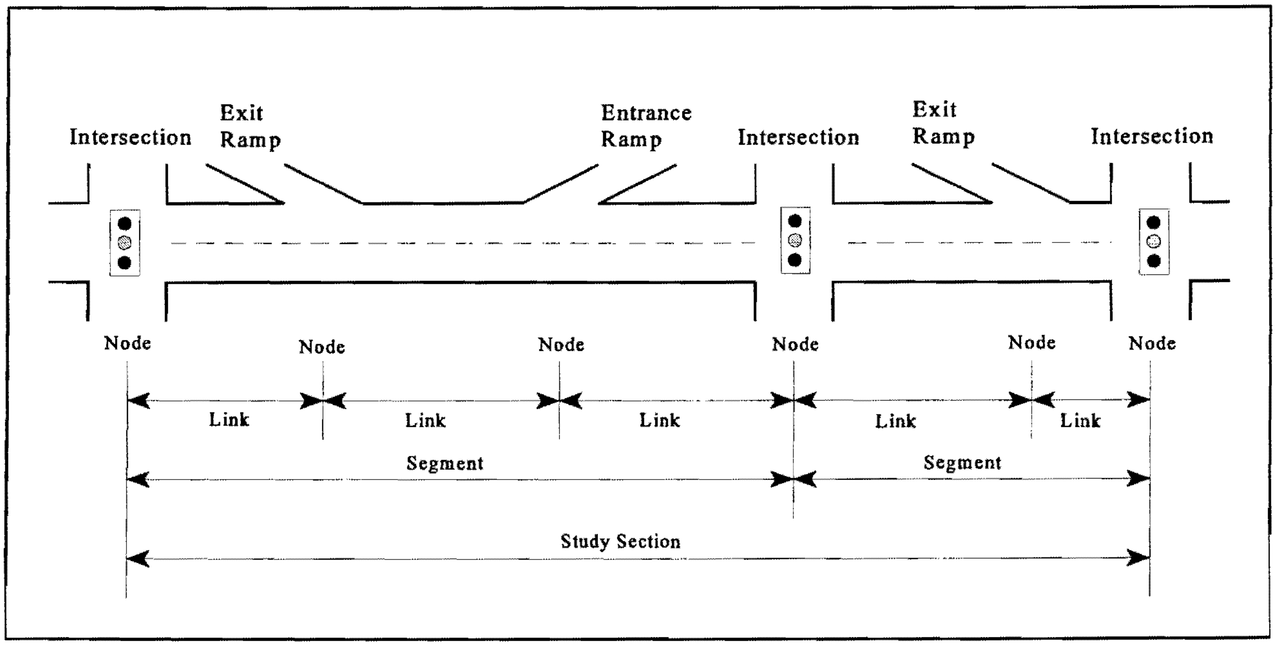

Figure 7-2. Terminology Used to Describe Frontage Roads.

its beginning and ending *nodes* (e.g., exit ramp, entrance ramp, signalized intersection, etc.). Figure 7-2 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 7-1 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 it takes a vehicle to traverse a given section of roadway without being delayed by intersections or ramps. Procedures for estimating running time

Table 7-1. Data Required for Evaluating Frontage Road Operations.

Type of Data	Data Required	Frontage Road One-Way	Frontage Road 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)	✔	✔

were 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 7-2 shows these regression equations.

Table 7-2. 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 increased 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 increased by as much as 10 percent.

Table 7-3 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 7-3 are increased by 10 percent when access

Table 7-3. 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 7-2. The running time values are increased by 10 percent when there are greater than 20 acs/km for a one-way frontage road, greater than 16 acs/km for a two-way frontage road, or greater than 400 vphpl on a two-way frontage road.

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:

$$D_I = 1.3 * d \qquad [7\text{-}1]$$

where:

D_I =intersection total delay, sec/veh

d = intersection stopped delay, sec/veh

Intersection stopped delay is calculated using the following equations:

$$d = d_1 DF + d_2 \qquad [7\text{-}2]$$

$$d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]} \qquad [7\text{-}3]$$

$$d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}] \qquad [7\text{-}4]$$

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 7-5)

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 7-4)

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 7-4 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 7-5 shows values of DF recommended in the *HCM.*

Table 7-4. 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 7-5. 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 7-1 through 7-4 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 7-2. Table 7-6 shows level-of-service criteria for signalized intersections suggested in the *HCM.*

Table 7-6. 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. (2), 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 7-7.

As shown in Table 7-7, 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 7-7 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 (see Chapter 4) 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 7-8. Using the capacity equations for ramp volumes above those in this table will produce invalid results.

Table 7-7. 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 7-8. 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:

$$S = \frac{3,600(L)}{RT + D_I + D_R} \qquad [7\text{-}5]$$

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 7-9. 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 7-9. 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 7-9 to assess the level of service.

PLANNING APPLICATIONS

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. Chapter 2 of this report includes a description of the planning application procedure presented in the *HCM*. 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 7-3 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.

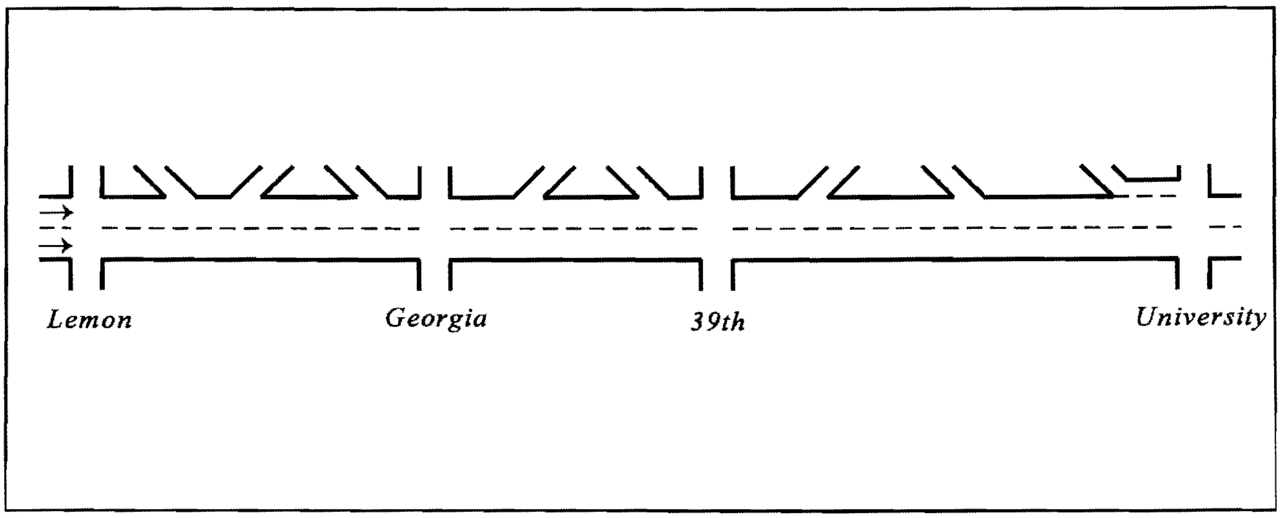

Figure 7-3. Schematic of One-Way Frontage Road Study Section.

Step 2: Gather Field Data

The required field data include roadway characteristics, traffic data, and signal data (see Table 7-1). Assumptions include random arrival and a saturation flow rate of 1800 vphpl. Tables 7-10 and 7-11 summarize collected field data.

Table 7-10. 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 7-11. 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 7-4). Running times are obtained from Table 7-3.

Step 4: Compute Intersection Delay

Intersection delay is computed on the Signalized Intersection Delay Worksheet (see Figure 7-5). 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 7-4. 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 7-1 through 7-4. Intersection level of service is based on stopped delay (d) and may be estimated using the criteria in Table 7-6. 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 7-6). 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

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	1.2	21.2	67					
2	1.1	18.2	55					
3	1.6	16.2	81					

[a] Use field data or values from Table 7-3
[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 7-9.

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 7-4. 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 7-4

[b] Equation 7-3 $d_1=\frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]}$

[c] Table 7-5

[d] Equation 7-4 $d_2=173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}]$

[e] Equation 7-2 $d=d_1DF+d_2$

[f] Equation 7-1 $D_I=1.3*d$

[g] Table 7-6

Figure 7-5. 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 $\leq$ 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] $D_R = -0.0719 + 1.0922\,(W)$

Figure 7-6. 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 7-5). This information is entered on the Frontage Road Level-of-Service Worksheet (see Figure 7-7).

Step 7: Assess Level of Service

The frontage road speeds for each segment are now compared to the criteria in Table 7-9 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 7-7, 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

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	1.2	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 7-3
[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 7-9.

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 7-7. 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 7-8 illustrates the frontage road length to be analyzed.

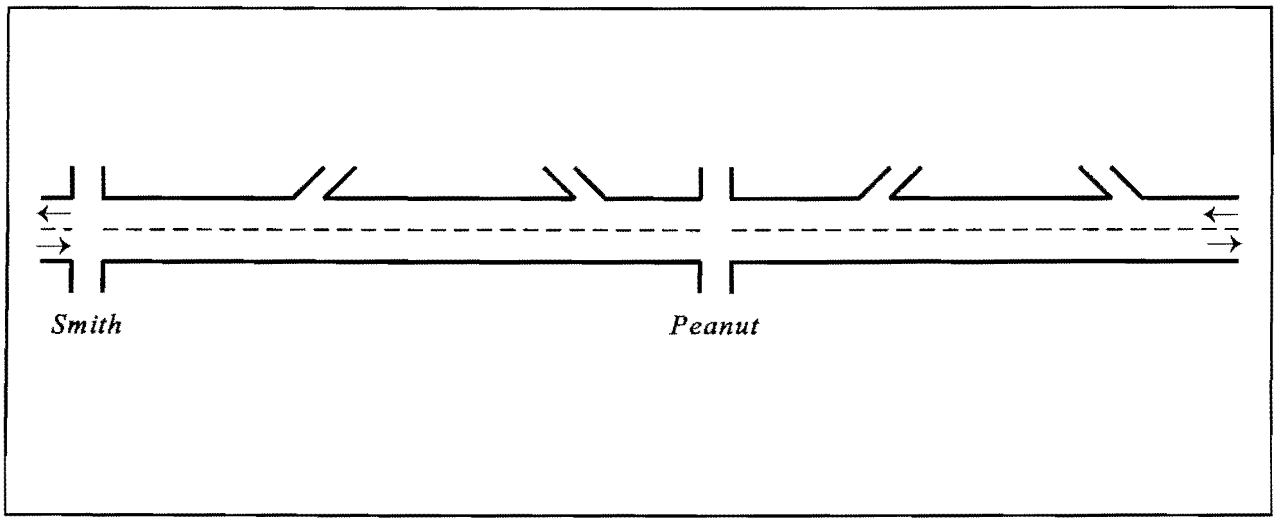

Figure 7-8. Schematic of Two-Way Frontage Road Study Section.

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 7-1). The saturation flow rate is assumed to be 1800 vphgpl. Tables 7-12 and 7-13 summarize the required field data.

Table 7-12. 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 7-13. 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 7-9). Running times are computed from Table 7-3.

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 7-3
[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 7-9.

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 7-9. Compute Running Time.

Step 4: Compute Intersection Delay

Intersection delay is computed on the Signalized Intersection Delay Worksheet (see Figure 7-10). 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 7-4. 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 7-1 through 7-4. Intersection level of service is based on stopped delay (d) and may be estimated using the criteria in Table 7-6. 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 7-11). 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 7-5). This information is entered on the Frontage Road Level-of-Service Worksheet (see Figure 7-12).

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 7-4

[b] Equation 7-3 $d_1=\frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]}$

[c] Table 7-5

[d] Equation 7-4 $d_2=173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}]$

[e] Equation 7-2 $d=d_1DF+d_2$

[f] Equation 7-1 $D_I=1.3*d$

[g] Table 7-6

Figure 7-10. 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 7-11. 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

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	
2	1.3	15.9	68	0.0	3.0	71.0	65.9	

[a] Use field data or values from Table 7-3
[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 7-9.

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 7-12. 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 7-9 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 7-13, 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 7-3
[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 7-9.

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 7-13. 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.

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

Step 4. Determine running time.

The running time rate is obtained from Table 7-3 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 7-1 through 7-4. 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

$$d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]} \qquad [7\text{-}3]$$

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

d_1 = 21.9 sec

From Table 7-4, m = 16 for arrival type 3. From Table 7-5, DF = 0.85 for non-coordinated, semiactuated signals.

$$d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}] \qquad [7\text{-}4]$$

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

d_2 = 2.6 sec

Determine intersection stopped delay (d).

$$d = d_1 \times \mathrm{DF} + d_2 \qquad [7\text{-}2]$$

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

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

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

$$D_I = 1.3 \times d \qquad [7\text{-}1]$$

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

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

Step 6. Determine average travel speed using Equation 7-5.

$$S = \frac{3{,}600(L)}{RT + D_I + D_R} \qquad [7\text{-}5]$$

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

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

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 7-9, the frontage road level of service is "C."

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

The objective of this project was to develop a procedure to evaluate freeway frontage roads. This report documents the research performed in developing this procedure and presents the step-by-step instructions on how to conduct a level-of-service evaluation of freeway frontage roads. Based on the research performed, the following conclusions and recommendations are made:

CONCLUSIONS

- The performance of a freeway frontage road can be evaluated using the procedure presented in Chapter 7. The procedure is based on the arterial analysis chapter of the *Highway Capacity Manual* (Chapter 11) (1). Consideration of the effects of the ramp junctions on performance is included.

- 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 running 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.* Users of the frontage road level-of-service procedure (i.e., Chapter 7 of this report) can either use the running times calculated with the *HCM* table or refine those values by using 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 greater than 16 acs/km on two-way frontage roads.

- The models developed by Gattis et al. (2) for predicting delay at ramp junctions are appropriate when used within their acknowledged limitation range.

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

RECOMMENDATIONS

- The procedure presented in Chapter 7 of this report is appropriate for the evaluation of frontage road operations.

- When the operations along a frontage road are dominated by a specific location, such as a two-sided weaving section between an exit ramp and a downstream intersection or a one-sided weaving section between an exit ramp and entrance ramp connected by an auxiliary lane, the analyst should conduct additional analyses using the appropriate methods [see TxDOT 1393-4F report (6)].

- Additional research is needed in the following areas to determine:
  — delay at high volume ramps (see Table 7-5 for ramp volumes), and
  — delay at exit ramps on one-way frontage roads when traffic on all frontage road lanes stops at the ramp junction.

REFERENCES

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

2. Gattis, J.L., C.J. Messer, and V.G. Stover. *Delay to Frontage Road Vehicles at Intersections With Ramps.* Report FHWA/TX-86/402-2. Texas Transportation Institute, Texas Department of Transportation. June 1988.

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4. Garber, N. J. and T.E. White, *Guidelines for Commercial Driveway Spacing On Urban and Suburban Arterial Roads,* August 1996.

5. Ott, R.L. *An Introduction to Statistical Methods and Data Analysis — Fourth Edition.* Duxbury Press, Wadsworth Publishing Company, Belmont, California. 1993.

6. Fitzpatrick, K., R.L. Nowlin, and A.H. Parham. *Procedures to Determine Frontage Road Level of Service and Ramp Spacing.* Report FHWA/TX-97/1393-4F. Texas Transportation Institute, Texas Department of Transportation. 1996.

APPENDIX A

ONE-WAY FRONTAGE ROAD DATA

Appendix A contains the roadway characteristics for each of the 20 one-way frontage road sites. The tables for each site also list the available volumes collected on the frontage road for each travel time run. In addition, the available g/C ratio (and the calculated capacity) for each intersection is shown. These data were used during the evaluation of one-way frontage roads. The legend for the tables follows:

- g/C = green time to cycle time ratio
- Capacity = saturation flow rate (vphgpl) x number of lanes x g/C, vph
- LEN = length in meters for each link, m
- #L = average number of lanes per link (a value of 2.5 means a change in the number of lanes within the section)
- SL = speed limit for link, km/h
- FF Spd = free flow speed for link, km/h
- AD = calculated access density, acs/km
- Volume = flow rates for specific links and travel time runs, determined by converting 5-min volumes into hourly flow rates, vph
- R# = travel time run number

Site 1, Dallas, IH 35E, Southbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Beltline	Entrance					516	2.5	56	72	11.63													
2	Entrance	Crosby	0.16	0.12	648	878	311	3	56	72	19.32	582	534	612	456	564	432	516	354					
3	Crosby	Entrance					242	3	56	72	20.66													
4	Entrance	Exit					568	2	56	72	17.61	0	108	6	6	0	84	102	78					
5	Exit	Valwood	0.19	0.13	720	1035	183	3	56	72	21.80													
6	Valwood	Entrance					262	3	56	80	30.59													
7	Entrance	Exit					1245	2	56	80	15.26	0	60	120	108	108	174	102	60					
8	Exit	Valley View	0.25	0.22	780	900	277	2	56	80	21.66													

Site 2, Dallas, IH 35E, Northbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Valley View	Entrance					194	2	56	72	5.14													
2	Entrance	Exit					1263	2	56	72	18.21	90	78	150	120	120	228	156	174					
3	Exit	Valwood	0.23	0.25	1260	1350	311	3	56	72	41.77													
4	Valwood	Entrance					252	3	56	64	23.83													
5	Entrance	Exit					578	2	56	64	17.31													
6	Exit	Crosby	0.22	0.22	1168	1168	206	3	56	64	29.12													
7	Crosby	Exit					275	3	56	64	21.85													
8	Exit	Beltline	0.18	0.19	990	1013	525	3	56	64	15.25	450	456	414	462	432	174	258	186					

Site 3, Dallas, IH 635, Westbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Hillcrest	Exit					564	2	64	64	19.49													
2	Exit	Entrance					354	2	64	64	14.13	0	834	1200	1158	1200	1230	1122	1050	900	1032	1134	1038	
3	Entrance	Preston	0.24	0.22	872	778	708	2	64	64	11.29													
4	Preston	Entrance					115	3	64	64	8.70													
5	Entrance	Exit					325	3	64	64	9.23	0	258	222	282	198	252	300	150	234	180	180	126	
6	Exit	Montfort	0.25	0.28	1350	1519	279	3	64	64	3.59													
7	Montfort	Exit					368	2.5	64	64	21.73													
8	Exit	Noel	0.64	0.80	3480	4320	302	3	64	64	6.62	0	360	468	492	426	360	492	354	276	324	420	396	

Site 4, Dallas, IH 635, Eastbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Inwood	Entrance					501	3	64	64	3.99													
2	Entrance	Montford	0.37	0.40	1980	2160	318	3	64	64	6.29	0	648	516	600	720	660	678	780	804	816	858	780	
3	Montford	Entrance					242	3	64	64	24.76													
4	Entrance	Exit					382	2.5	64	64	26.18	234	198	150	174	150	198	264	276	324	402	312	336	
5	Exit	Preston	0.27	0.23	1440	1227	109	3	64	64	27.42													
6	Preston	Exit					656	2	64	64	16.78													
7	Exit	Entrance					409	2	64	64	24.47	876	1032	1086	978	966	966	936	1230	1260	1380	1014	1224	
8	Entrance	Hillcrest	0.36	0.28	1280	1000	561	2	64	64	5.34													

Site 5, Dallas, SH 183, Eastbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Story	Exit					213	1.5	56	64	18.75													
2	Exit	Exit					849	2	56	64	5.89													
3	Exit	Entrance					230	2	56	64	26.07	6	12	12	60	72	48	36	30					
4	Entrance	Mac Arthur	0.39	0.32	1405	1161	336	2	56	64	20.86													
5	Mac Arthur	Exit					301	2	56	64	19.94													
6	Exit	Entrance					196	2	56	64	10.19	72	144	228	228	312	402	384	342					
7	Entrance	O'Conner	0.43	0.33	1530	1181	242	2	56	64	28.96													
8	O'Conner	Exit					371	2	56	64	29.63													
9	Exit	Entrance					420	2	56	64	16.65	84	126	180	324	450	378	474	438					
10	Entrance	Exit					859	2	56	64	4.66													
11	Exit	Carl	0.19	0.33	675	1181	315	2	56	64	22.25													

Site 6, Dallas, SH 183, Westbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Carl	Entrance					172	2	56	64	0.00													
2	Entrance	Exit					962	2	56	64	4.16	264	240	396	648	1134	1458	504	480					
3	Exit	Exit					504	2	56	64	33.74													
4	Exit	O'Conner	0.32	0.39	1152	1409	344	2	56	64	26.20													
5	O'Conner	Exit					279	2	56	64	28.72													
6	Exit	Entrance					165	3	56	64	18.16	90	108	270	408	732	804	600	534					
7	Entrance	MacArthur	0.22	0.28	800	990	296	2	56	64	30.44													
8	MacArthur	Exit					303	2	56	64	19.80													
9	Exit	Exit					469	2	56	64	21.33													
10	Exit	Entrance					312	2	56	64	16.04	228	180	354	630	1140	1392	516	426					
11	Entrance	Story	0.29	0.25	1042	900	545	2	56	64	18.36													

Site 7, Dallas, SH 360, Southbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Avenue J	Exit					377	2.5	64	64	10.62													
2	Exit	Lamar	0.57	0.57	3086	3060	211	3	64	64	4.74	0	270	324	330	378	282	336	300	294	348	384	384	294
3	Lamar	Entrance					100	3	64	64	0.00													
4	Entrance	Six Flags Dr.	0.23	0.22	821	778	379	2	64	64	0.00	0	726	840	888	600	690	720	744	750	840	930	834	768
5	Six Flags Dr.	Entrance					176	2	64	64	5.69													
6	Entrance	Randol Mill	0.32	0.26	1168	953	811	2	64	64	13.56	0	120	174	198	132	180	108	192	108	180	192	282	144

Site 8, Dallas, SH 360, Northbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Randol Mill	Exit					771	2	64	64	19.46													
2	Exit	Six Flags Dr.	0.31	0.26	2220	1850	225	4	64	64	22.20	0	198	186	156	198	198	192	168	210	180	348	360	156
3	Six Flags Dr.	Exit					163	2	64	64	6.13													
4	Exit	Avenue H	0.51	0.56	2744	3000	304	3	64	64	9.87	0	450	504	312	318	684	600	438	546	504	594	594	576
5	Avenue H	Entrance					213	3	64	64	9.37													
6	Entrance	Avenue J	0.37	0.48	1330	1718	375	2	64	64	10.66	0	192	168	156	210	270	324	246	174	342	318	354	372

Site 9, Houston, 290, Eastbound

Link	Node A	Node B	g/C		Capacity		LEN	# L	SL	FF Spd	AD	Volume (vph)												
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Hollister	Tidwell	0.54	0.47	2903	2546	442	3	64	80	15.85	210	150	192	312	270	468	432	396	234	468			
2	Tidwell	Entrance					502	3	64	80	5.98	78	84	96	306	336	372	216	0	168	198			
3	Entrance	Exit					865	2	64	80	4.63	18	30	60	90	48	114	84	126	96	90			
4	Exit	Fairbanks	0.21	0.17	1157	900	377	3	64	80	0.00													
5	Fairbanks	Entrance					366	3	64	80	0.00	36	72	126	168	252	288	252	156	192	198			
6	Entrance	Exit					765	2	81	80	1.31	6	12	6	48	18	54	90	120	54	66	282		
7	Exit	Gessner	0.33	0.46	1774	2469	655	3	81	80	3.05													

Site 10, Houston, 290, Eastbound

Link	Node A	Node B	g/C		Capacity		LEN	# L	SL	FF Spd	AD	Volume (vph)												
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Gessner	Entrance					560	3	81	80	5.36													
2	Entrance	Exit					923	2	81	80	13.00													
3	Exit	Fairbanks	0.27	0.21	1440	1157	319	3	81	80	15.70													
4	Fairbanks	Entrance					367	3	81	80	5.45													
5	Entrance	Exit					871	2	64	80	11.48	30	30	240	672	738	660	408	360	180	84			
6	Exit	Tidwell	0.41	0.47	2224	2546	372	3	64	80	8.07	156	132	396	876	930	972	864	870	684	390			
7	Tidwell	Hollister	0.38	0.33	2025	1774	538	3	64	80	16.74	171	288	780	1131	1083	1317	924	951	612	387			

Site 11, Houston, 290, Westbound

Link	Node A	Node B	g/C		Capacity		LEN	# L	SL	FF Spd	AD	Volume (vph)												
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Mangum	Entrance					296	3	56	72	16.89													
2	Entrance	Exit					760	2	56	72	17.10	276	270	192	288	186	240	258	276	486	714	684		
3	Exit	34th	38	20	2052	1080	233	3	56	72	25.77	984	1278	852	1104	912	1062	1086	1206	1470	1710	1692		
4	34th	Antoine	46	22	2484	1188	707	3	56	72	21.22	486	654	648	720	708	678	612	738	1062	1110	1098		
5	Antoine	Entrance					260	3	64	72	0.00													
6	Entrance	Exit					890	2	64	72	4.49													
7	Exit	43rd	27	18	1458	972	210	3	64	72	14.31													

Site 12, Houston, 290, Eastbound

Link	Node A	Node B	g/C		Capacity		LEN	# L	SL	FF Spd	AD	Volume (vph)												
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	43rd	Entrance					392	3	64	72	7.65													
2	Entrance	Exit					877	2	64	72	3.42	186	180	210	222	174	198	174	144	120	240	252	180	
3	Exit	Antoine	29	15	1566	810	244	3	64	72	12.30	408	498	552	576	480	498	480	480	396	678	636	546	
4	Antoine	34th	20	20	1080	1080	573	3	64	64	15.70	522	573	513	660	663	645	588	489	456	678	570	714	
5	34th	Entrance					366	3	64	64	5.46	1014	822	774	1014	882	1056	768	1110	888	846	924	774	
6	Entrance	Exit					785	2	64	64	16.55	348	270	222	270	246	306	246	558	258	198	270	168	
7	Exit	Mangum	18	20	972	1080	303	3	64	64	13.19													

Site 13, San Angelo, Loop 306, Westbound

Link	Node A	Node B	g/C		Capacity		LEN	# L	SL	FF Spd	AD	Volume (vph)												
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Knickerbocker	Entrance					179	2	64	72	5.60													
2	Entrance	Exit					868	1.5	72	72	9.22	66	84	96	24	66	12	30	24					
3	Exit	College Hills		0.23		831	177	2	72	72	16.97	90	108	114	126	168	96	108	150					
4	College Hills	Entrance					186	2	72	72	26.85													
5	Entrance	Exit					752	1.5	72	72	6.65	24	36	48	54	48	60	36	60					
6	Exit	Southwest		0.47		1688	214	2	72	72	18.72	54	84	132	162	180	132	174	108					
7	Southwest	Entrance					143	2	72	72	0.00													
8	Entrance	Exit					828	1.5	72	72	4.83	30	54	18	72	36	42	6	18					
9	Exit	Sherwood		0.36		1286	346	2	72	72	0.00	54	108	126	96	222	132	114	90					

Site 14, San Angelo, Loop 306, Eastbound

Link	Node A	Node B	g/C		Capacity		LEN	# L	SL	FF Spd	AD	Volume (vph)												
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Sherwood	Entrance					226	2	72	72	0.00													
2	Entrance	Exit					899	1.5	72	72	3.34	48	72	24	60	96	72	84	108					
3	Exit	Southwest		0.31		1125	189	2	72	72	10.60	168	180	120	192	276	168	132	204					
4	Southwest	Entrance					201	2	72	72	9.94													
5	Entrance	Exit					819	1.5	72	72	9.76	96	72	192	300	288	72	168	264					
6	Exit	College Hills		0.31		1125	127	2	72	72	23.60	192	156	288	420	444	108	264	312					
7	College Hills	Entrance					150	2	72	72	0.00													
8	Entrance	Exit					909	2	72	72	8.80	108	216	228	336	192	108	288	132					
9	Exit	Knickerbocke		0.30		1620	126	3	72	72	23.77	600	744	996	1572	960	564	1080	708					

Site 15, Lubbock, Loop 289, Westbound

Link	Node A	Node B	g/C		Capacity		LEN	# L	SL	FF Spd	AD	Volume (vph)												
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	University	Entrance					315	2	72	80	12.68													
2	Entrance	Exit					1041	1.5	72	80	9.61	288	252	474	408	300	312	132	102	258	426	264	168	144
3	Exit	Indiana	0.28	0.27	994	975	251	2	72	80	19.88	624	684	1650	894	660	612	192	162	390	828	732	480	480
4	Indiana	Entrance					319	2	72	80	9.42													
5	Entrance	Exit					953	1.5	89	80	10.50	324	372	486	420	408	252	204	96	48	276	258	216	228
6	Exit	Quaker	0.31	0.29	1128	1050	552	2	72	80	9.06	810	972	1536	1254	420	408	252	336	636	594	642	558	228
7	Quaker	Exit					480	2	72	80	12.50	912	924	678	780	768	612	192	240	414	546	498	384	558
8	Exit	Entrance					311	2	89	80	19.30	1842	2142	2268	1950	1800	1410	486	324	810	1134	1134	894	972
9	Entrance	Slide	0.22	0.29	793	1050	701	2	72	80	7.13													

Site 16, Lubbock, Loop 289, Eastbound

Link	Node A	Node B	g/C		Capacity		LEN	# L	SL	FF Spd	AD	Volume (vph)												
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Slide	Exit					494	2	89	50	16.20	816	1140	1008	684	708								
2	Exit	Entrance					479	2	89	50	18.81	816	1608	1644	1080	1056								
3	Entrance	Quaker		0.28		1013	529	2	89	50	15.11													
4	Quaker	Entrance					547	2	89	50	9.15													
5	Entrance	Exit					1023	1.5	89	50	5.87	180	408	252	252	240								
6	Exit	Indiana		0.26		947	244	2	89	50	20.51	396	1104	888	804	648								
7	Indiana	Entrance					309	2	89	50	12.93													
8	Entrance	Exit					1027	2	89	50	15.58	192	312	372	168	384								
9	Exit	University		0.33		1800	273	3	72	50	7.33	576	1260	1020	756	900								

Site 17, Amarillo, I-40, Westbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Wolflin	Entrance					113	2	56	72	26.46													
2	Entrance	Exit					524	2	56	72	24.83	12	36	12	12	84	48	48	72	108	72	108	84	
3	Exit	Western		0.18		945	250	3	56	72	32.01	84	228	228	204	402	366	612	372	606	462	528	594	
4	Western	Entrance					335	2	72	72	32.81													
5	Entrance	Avondale		0.36		1309	517	2	72	72	15.48	60	60	12	84	108	156	162	108	180	120	84	96	
6	Avondale	Exit					399	2	72	72	20.04	162	72	24	108	84	198	198	54	126	108	126	222	
7	Exit	Bell		0.18		972	440	3	72	72	24.98	342	324	348	426	528	960	816	636	834	660	678	840	

Site 18, Amarillo, I-40, Eastbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Bell	Entrance					486	2	72	72	18.51													
2	Entrance	Olsen		0.30		1080	354	2	72	72	11.31	0	0	84	96	96	144	84	216	192	120	36	144	
3	Olsen	Exit					709	2	72	72	18.34	120	108	60	168	354	180	204	216	132	60	132	120	
4	Exit	Western		0.25		1350	148	3	72	72	27.00	258	276	222	336	576	486	384	510	492	402	378	348	
5	Western	Entrance					163	2	72	72	6.14													
6	Entrance	Exit					518	2	56	72	19.30	48	96	84	156	156	102	84	216	108	258	96	192	
7	Exit	Paramount		0.19		1013	283	3	56	72	21.17	132	240	186	300	300	294	270	384	312	480	228	366	

Site 19, Amarillo, I-27, Southbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Moss	Exit					265	2	56	72	26.40													
2	Exit	Entrance					238	2	56	72	25.24	504	774	492	462	504	768	648	660	540	372	504	372	
3	Entrance	Exit					460	2	72	72	23.90													
4	Exit	Georgia	0.25	0.28	893	996	207	2	72	72	9.65	282	204	294	264	330	474	300	294	240	300	150	156	
5	Georgia	Entrance					244	2	72	72	28.71													
6	Entrance	Exit					433	2	72	72	25.41													
7	Exit	45th	0.34	0.25	1224	911	241	2	72	72	37.38	372	186	372	348	378	300	312	318	342	342	282	354	
8	45th	Entrance					195	2	72	72	15.38													
9	Entrance	Exit					777	2	72	72	14.15	96	96	204	144	120	174	156	144	168	144	174	168	
10	Exit	Exit					564	2	72	72	19.51	264	372	330	348	288	516	300	264	336	288	336	312	
11	Exit	Western	0.26	0.29	1421	1588	229	3	72	72	4.37													

Site 20, Amarillo, I-27, Northbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	# L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13
1	Western	Entrance					393	2	72	72	17.80													
2	Entrance	Entrance					701	2	72	72	11.41													
3	Entrance	Exit					677	2	72	72	14.78	204	234	156	444	336	282	156	204	180	216	216	132	
4	Exit	45th	0.17	0.22	918	1192	219	3	72	72	13.67	312	342	306	600	468	534	372	300	276	288	288	234	
5	45th	Entrance					262	2	72	72	26.70													
6	Entrance	Exit					360	2	72	72	25.02	312	264	324	276	396	252	306	228	192	210	210	156	
7	Exit	Georgia	0.28	0.32	1530	1731	162	3	72	72	30.95	522	564	564	510	636	564	510	432	492	492	492	348	
8	Georgia	Entrance					293	2	72	72	13.67													
9	Entrance	Exit					369	2	72	72	16.27	84	216	132	156	246	210	228	174	60	120	120	48	
10	Exit	Entrance					268	2	56	72	37.28	324	600	288	504	684	588	744	468	264	408	408	312	
11	Entrance	Parker					283	2.5	56	72	38.81													

APPENDIX B

ONE-WAY FRONTAGE ROAD

TRAVEL TIME AND SPEED PLOTS

Appendix B contains the travel time and speed plots for each of the 20 one-way frontage road sites.

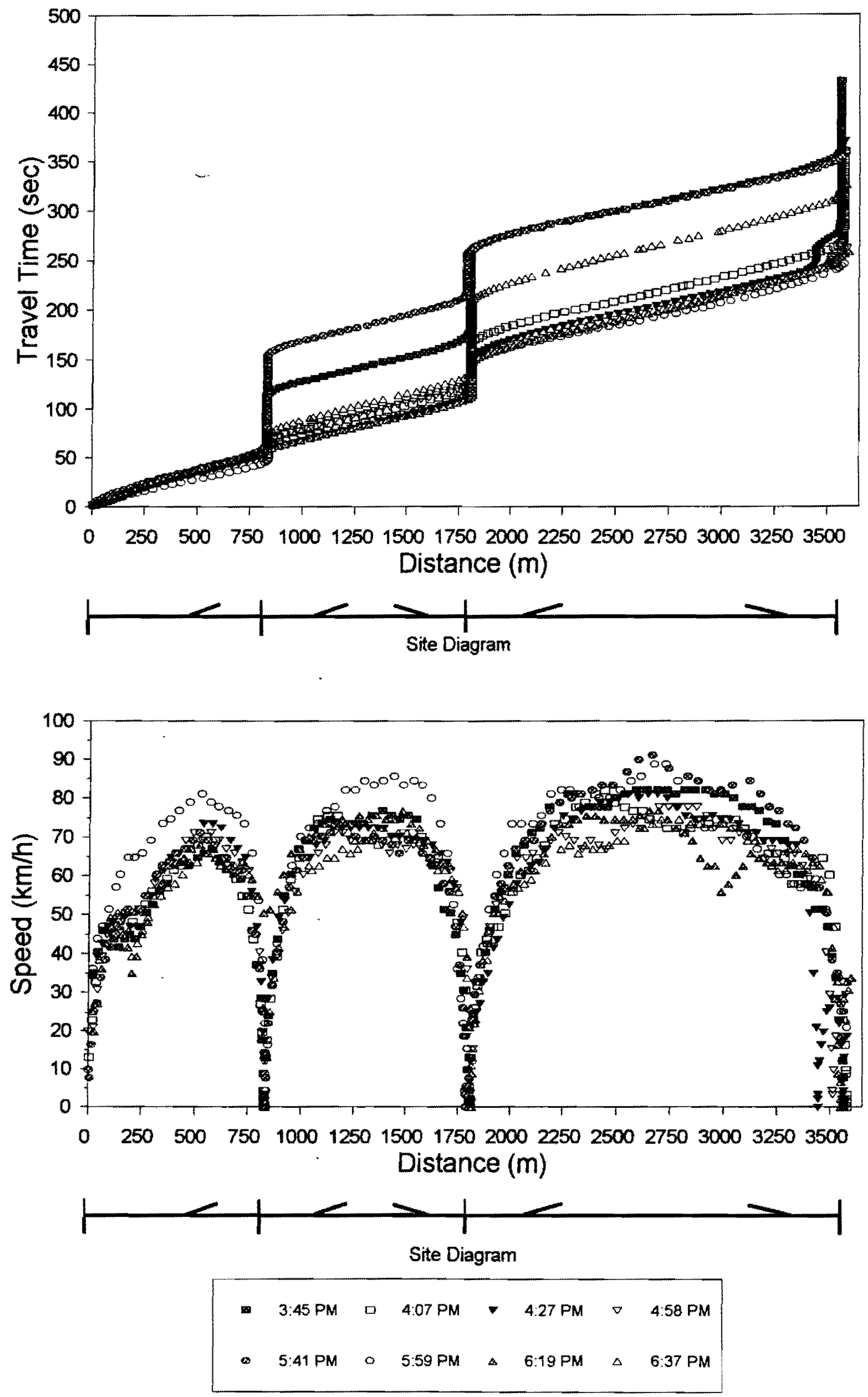


Figure B-1. Travel Time or Speed Versus Cumulative Distance for Site 1.

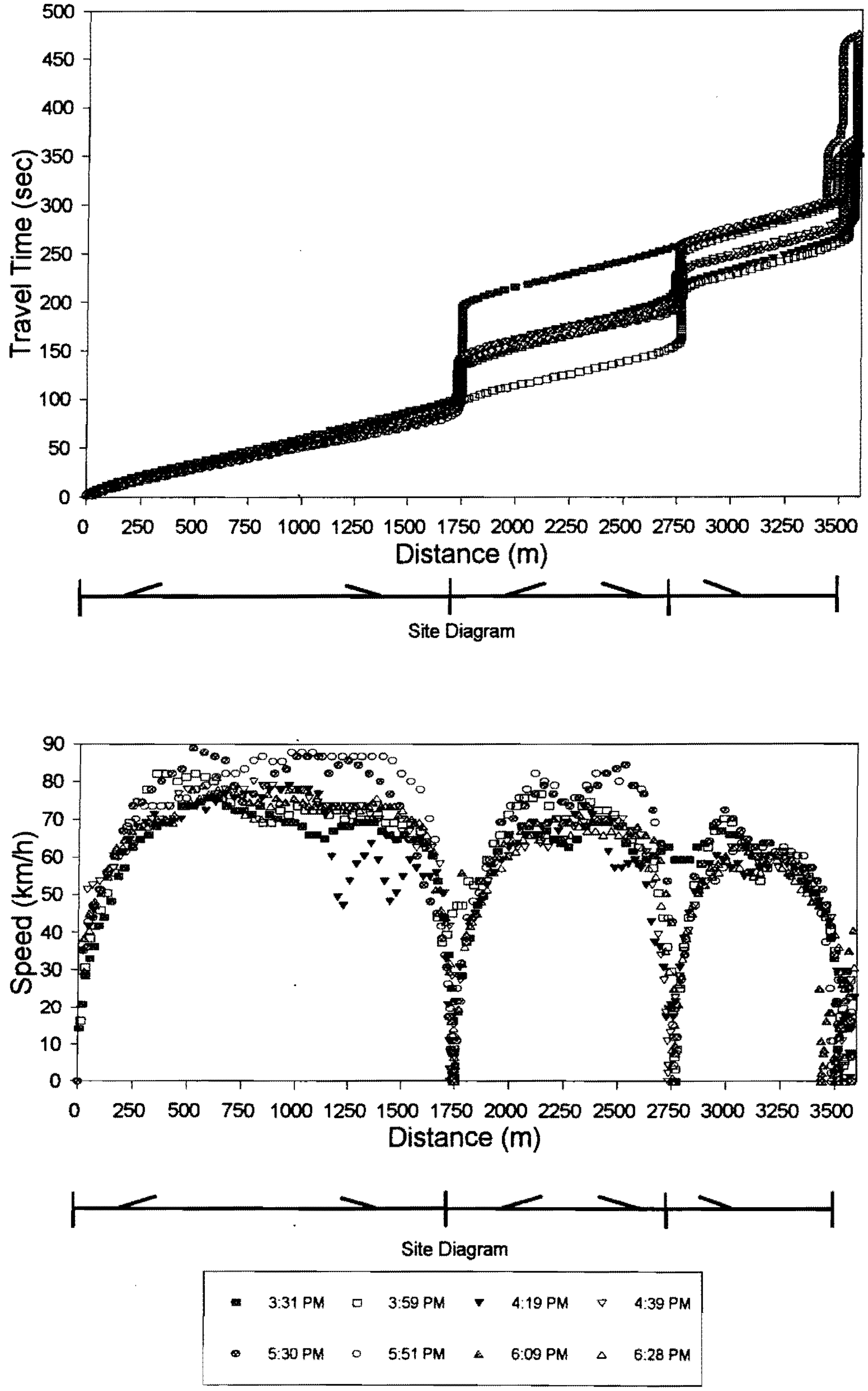


Figure B-2. Travel Time or Speed Versus Cumulative Distance for Site 2.

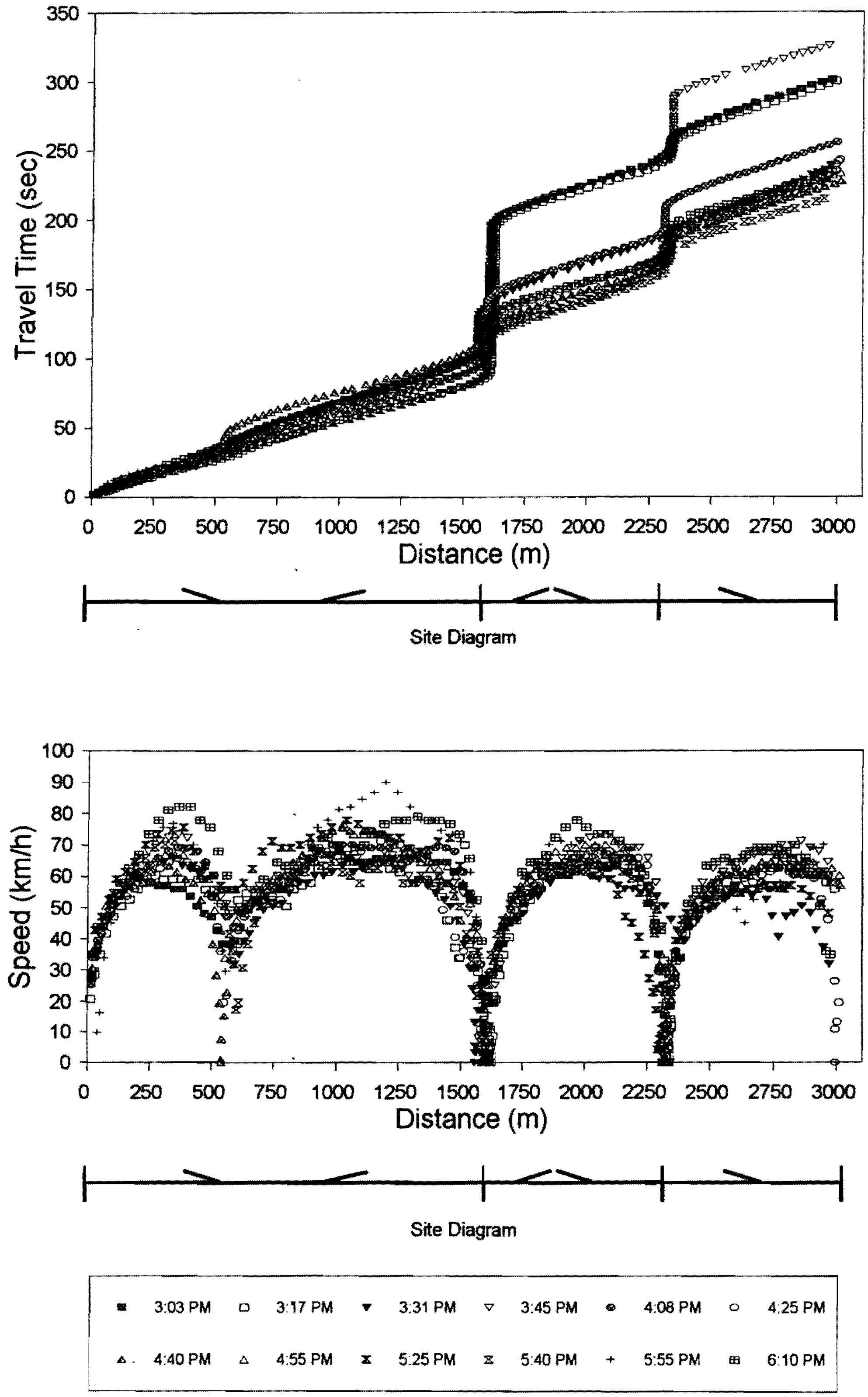
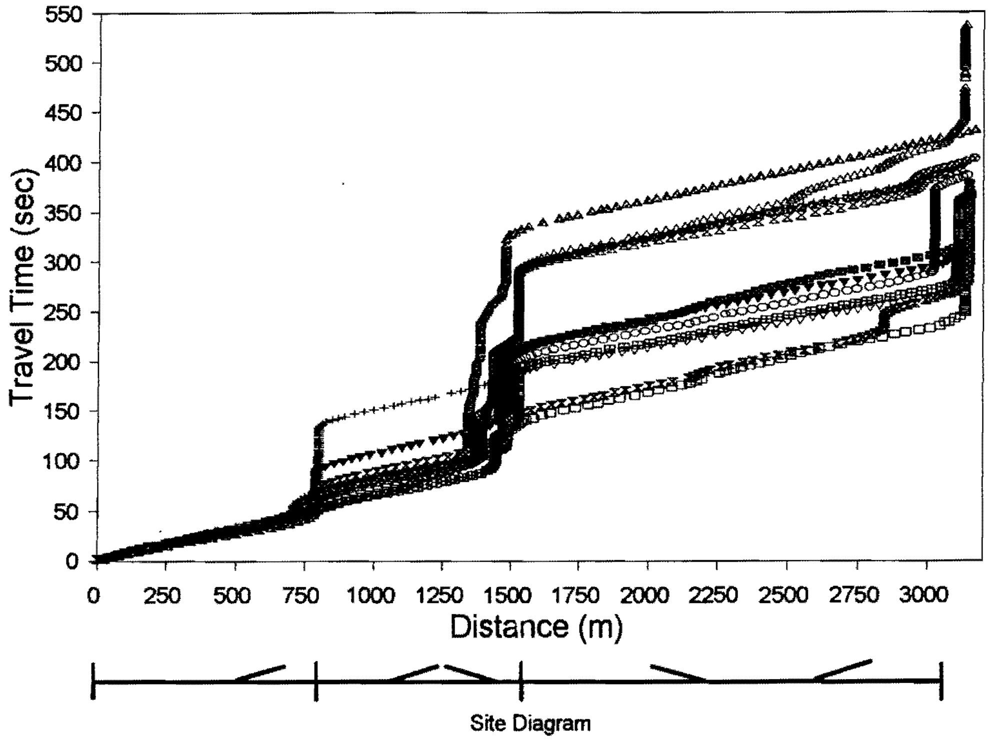


Figure B-3. Travel Time or Speed Versus Cumulative Distance for Site 3.



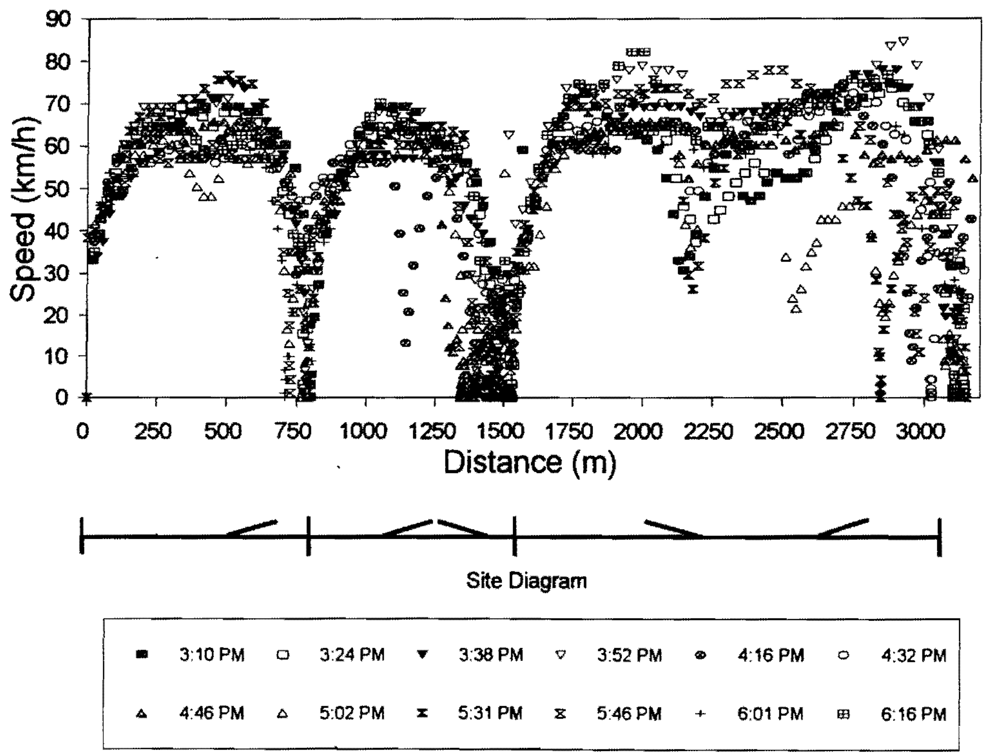


Figure B-4. Travel Time or Speed Versus Cumulative Distance for Site 4.

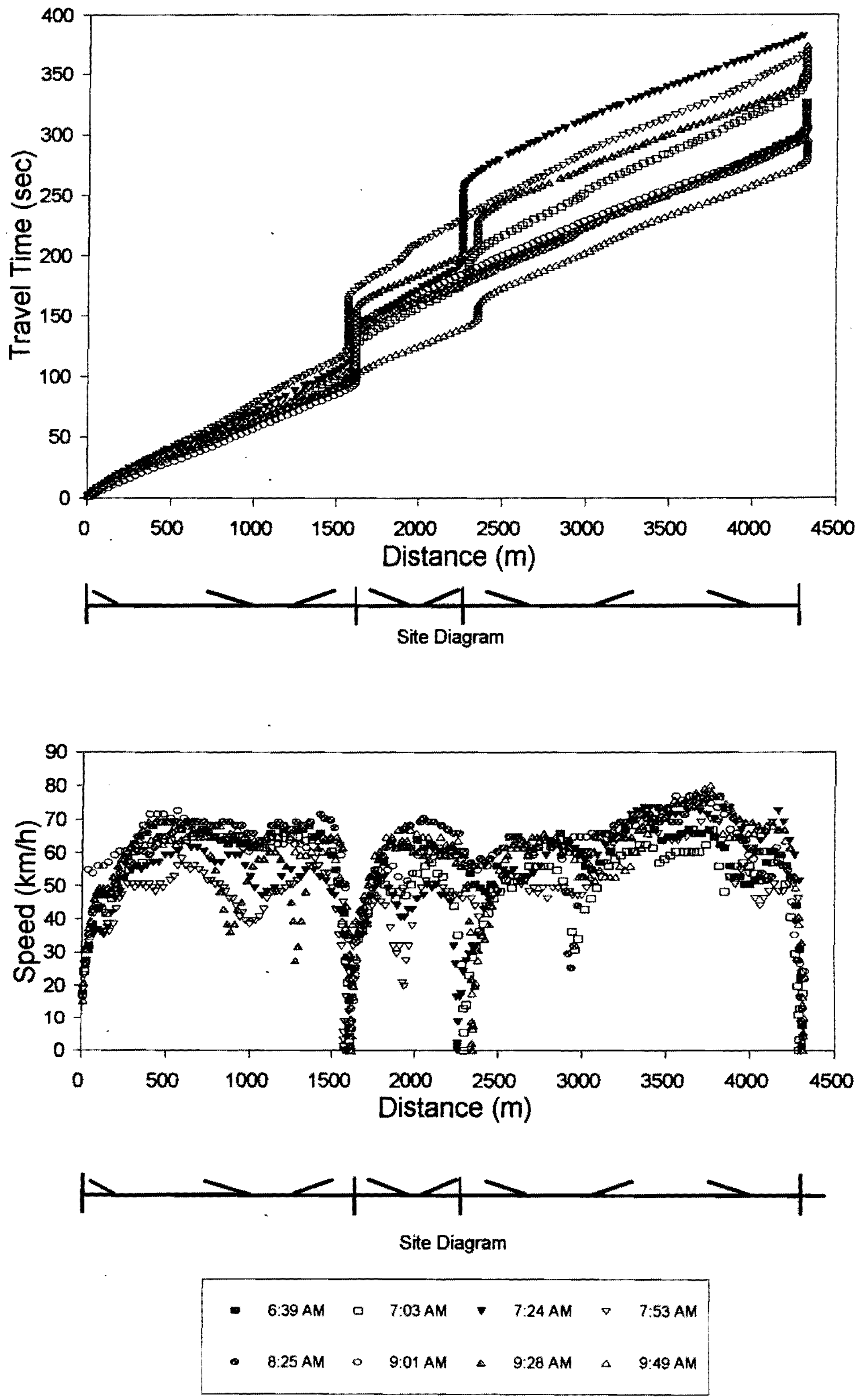


Figure B-5. Travel Time or Speed Versus Cumulative Distance for Site 5.

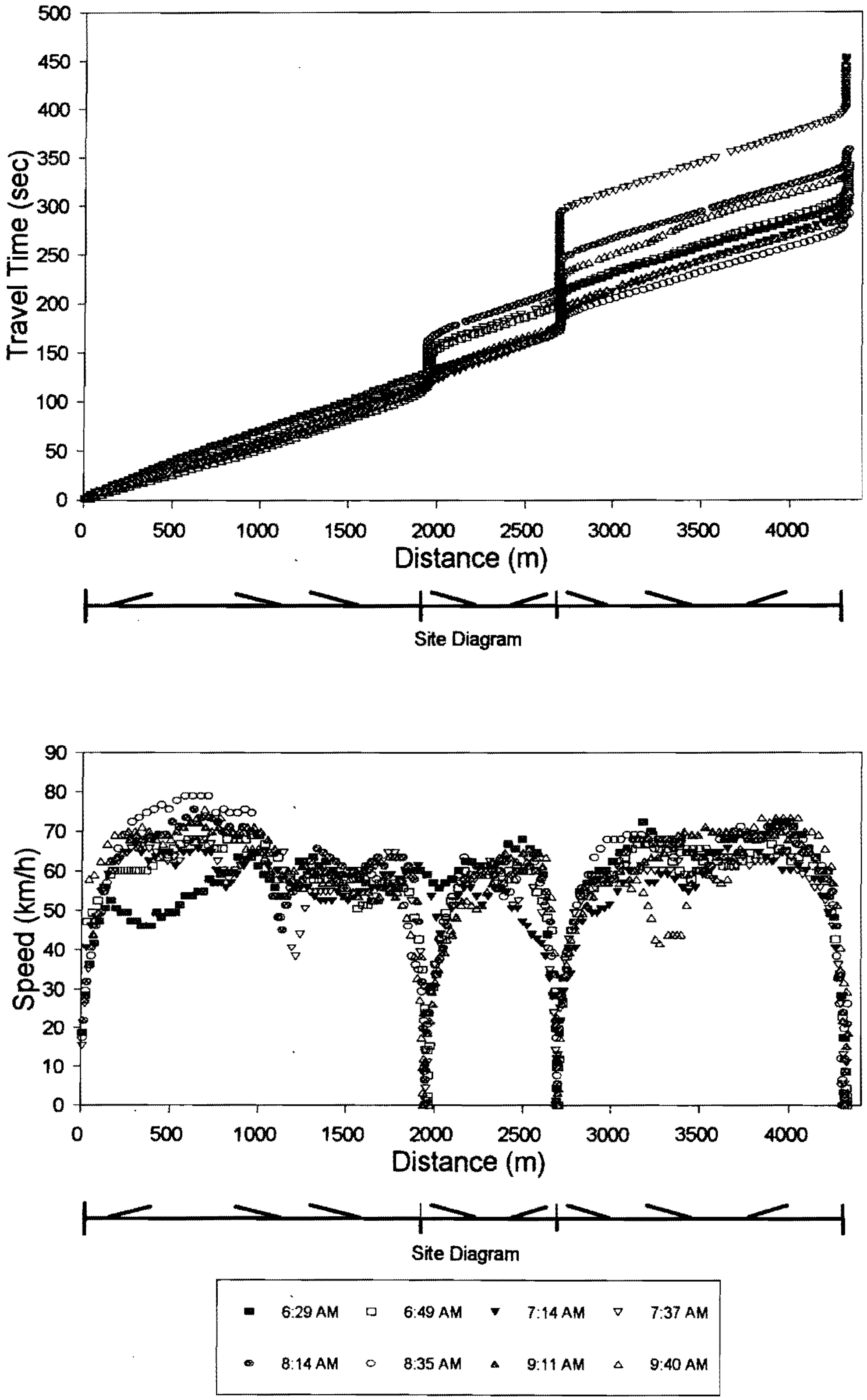
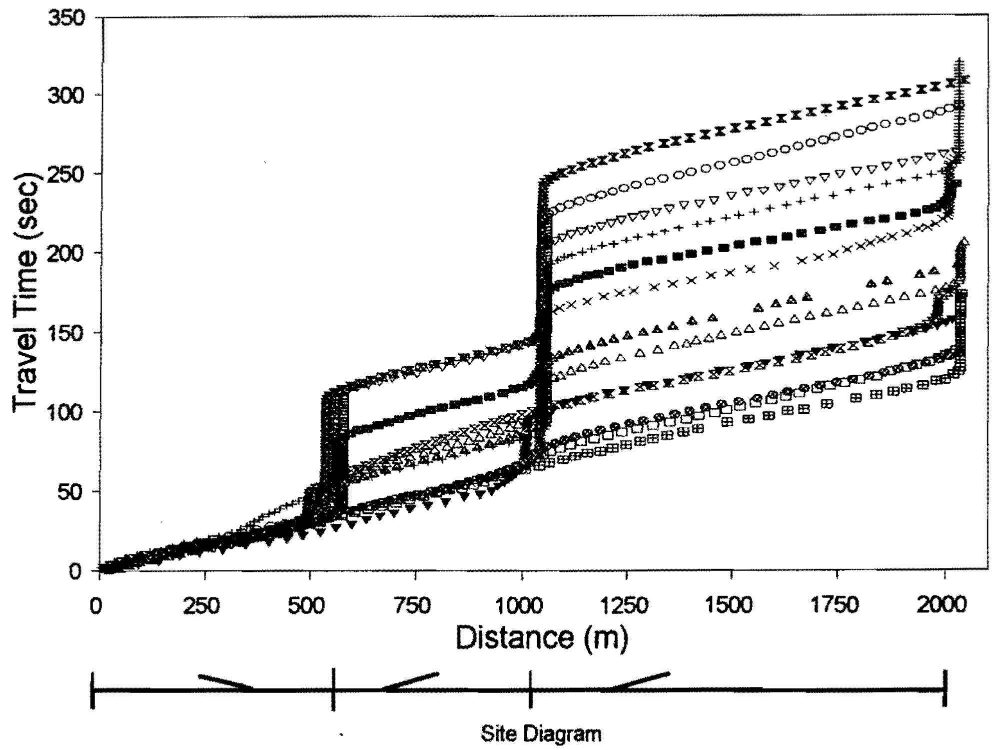


Figure B-6. Travel Time or Speed Versus Cumulative Distance for Site 6.



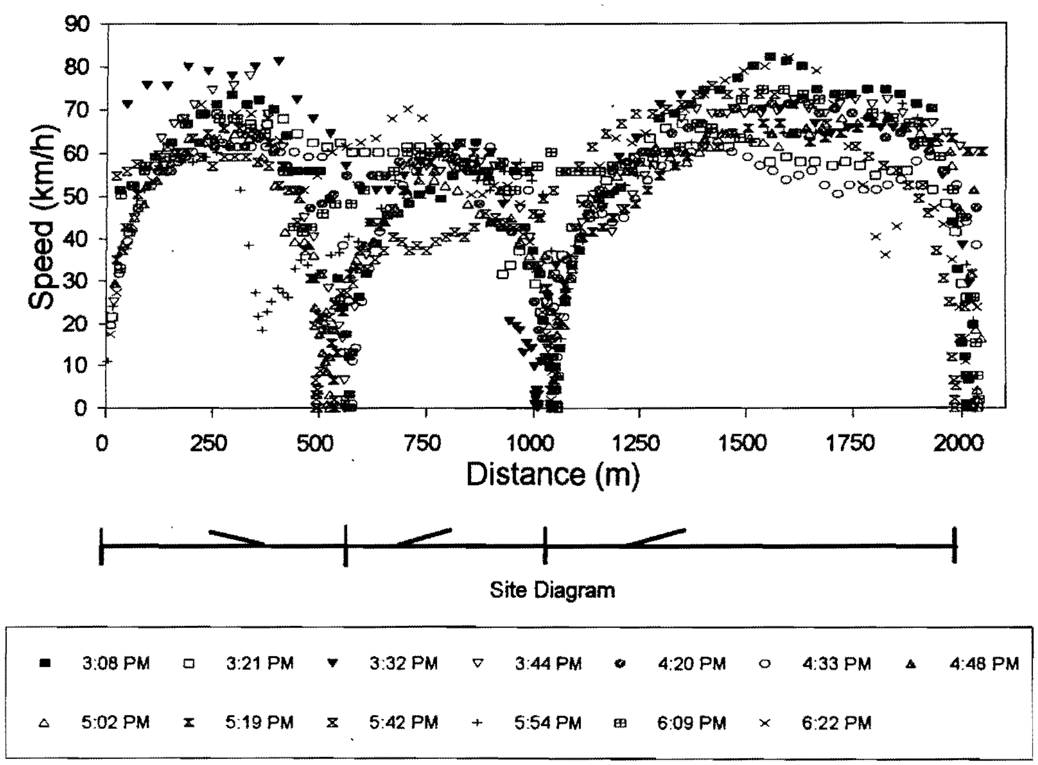
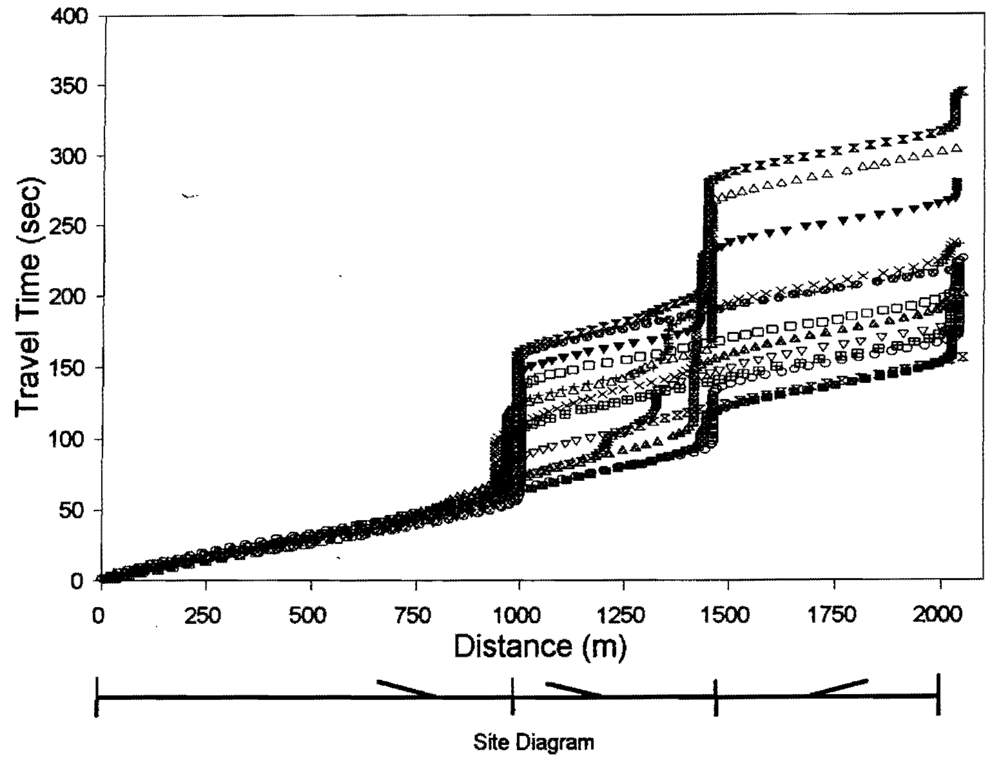


Figure B-7. Travel Time or Speed Versus Cumulative Distance for Site 7.



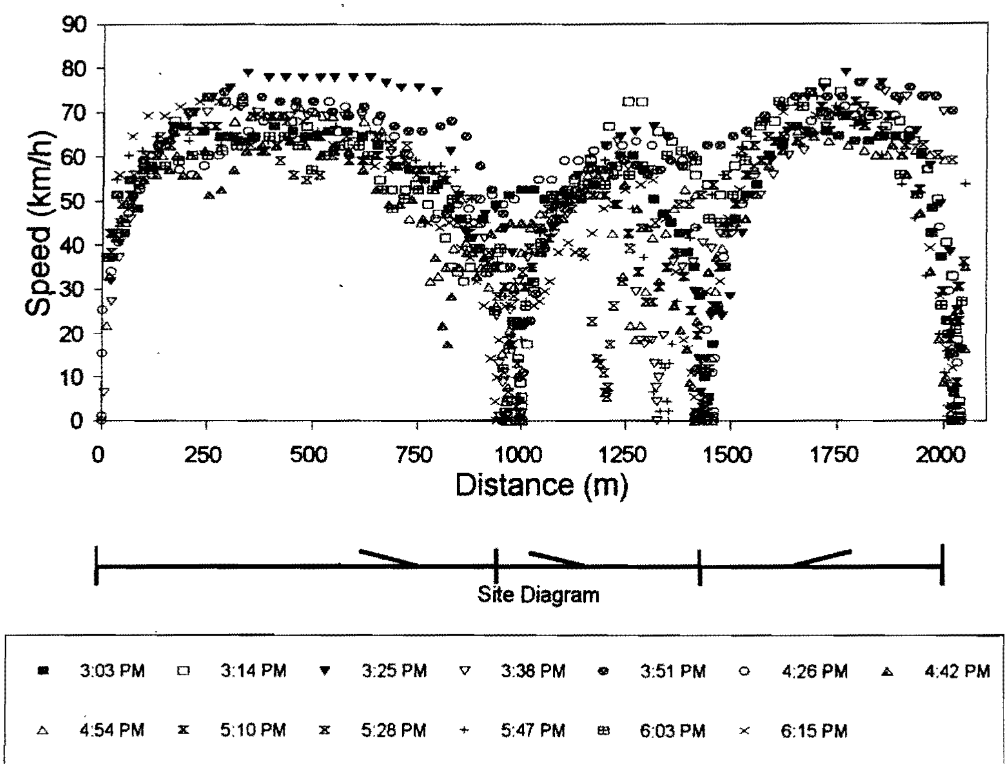
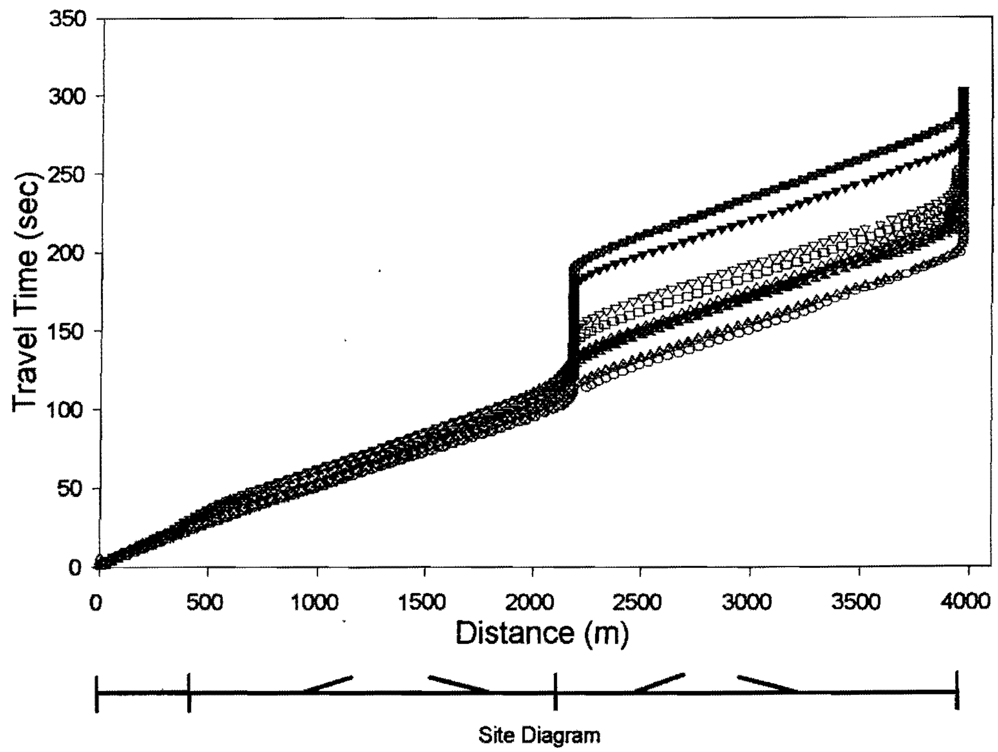


Figure B-8. Travel Time or Speed Versus Cumulative Distance for Site 8.



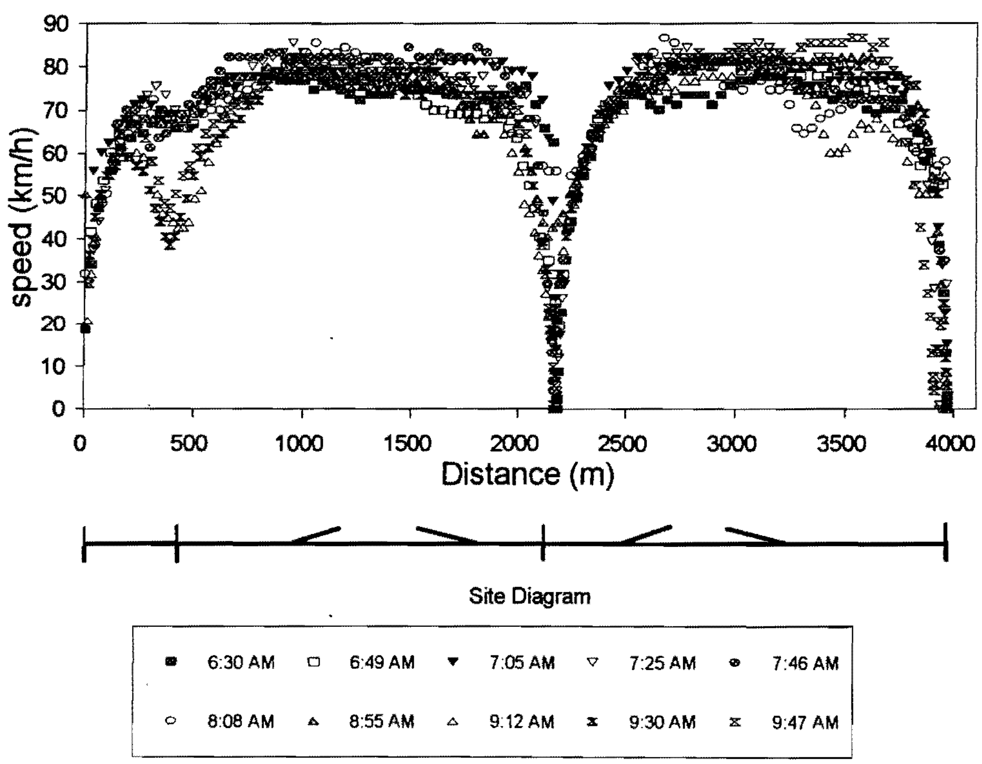


Figure B-9. Travel Time or Speed Versus Cumulative Distance for Site 9.

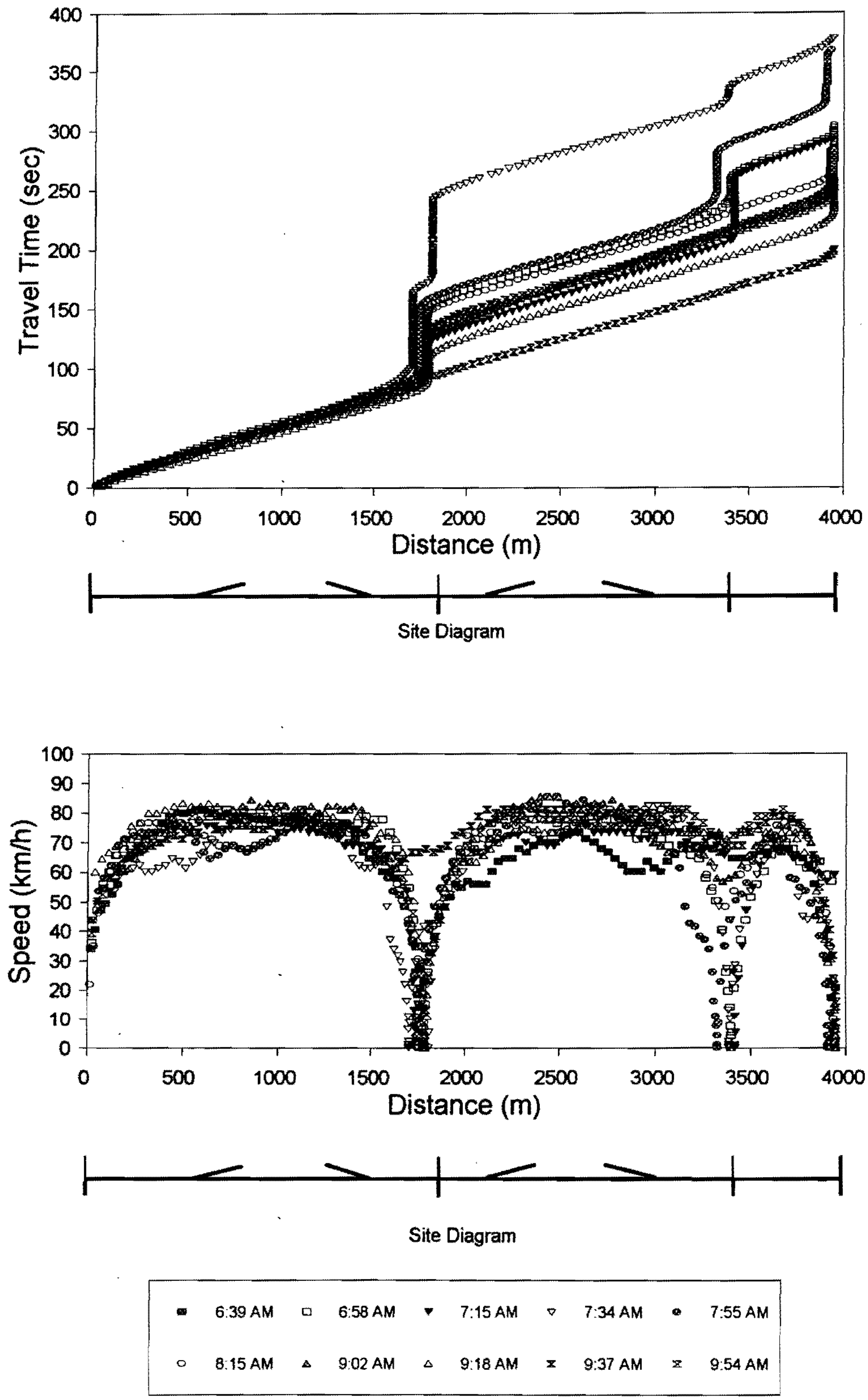


Figure B-10. Travel Time or Speed Versus Cumulative Distance for Site 10.

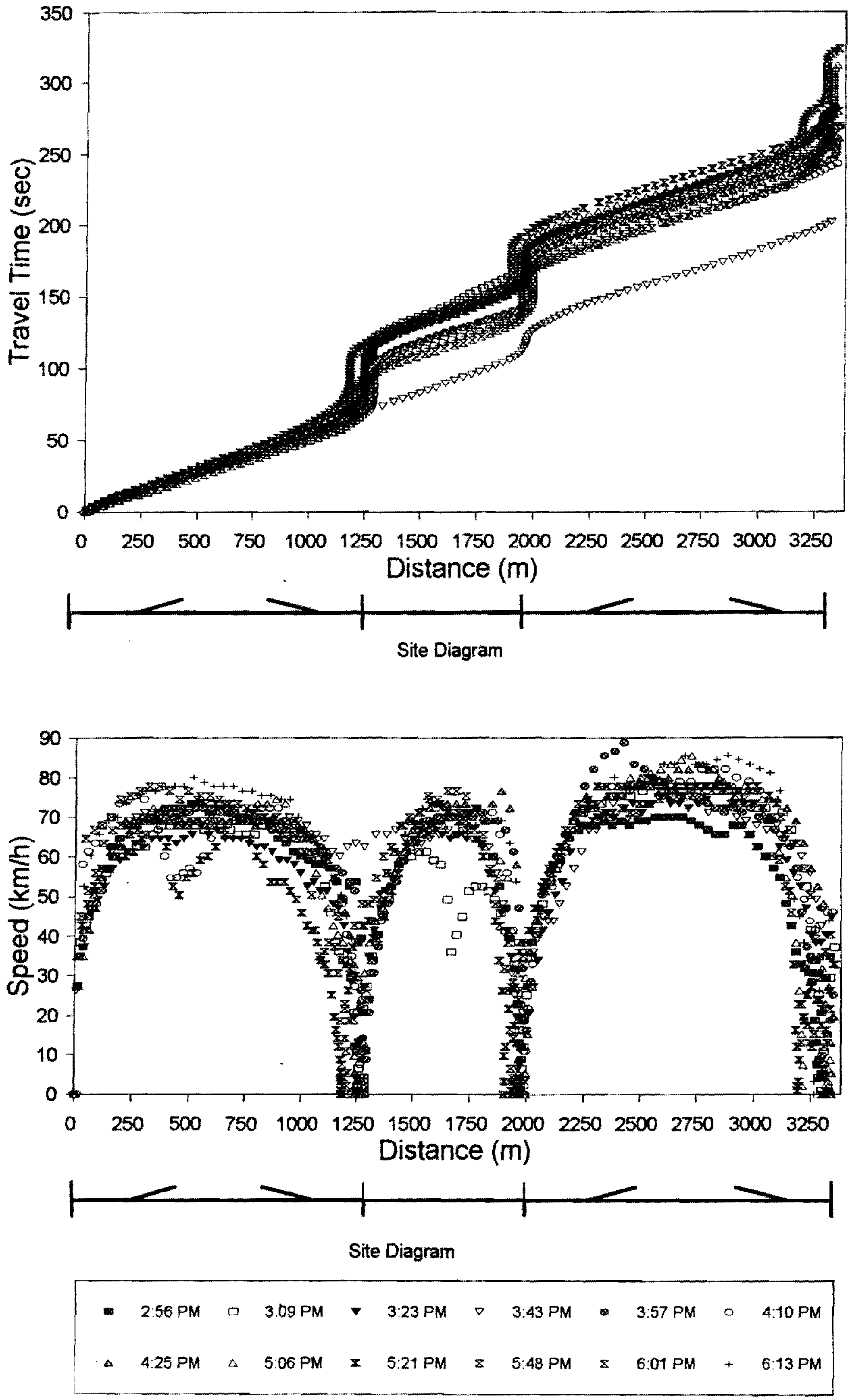


Figure B-11. Travel Time or Speed Versus Cumulative Distance for Site 11.

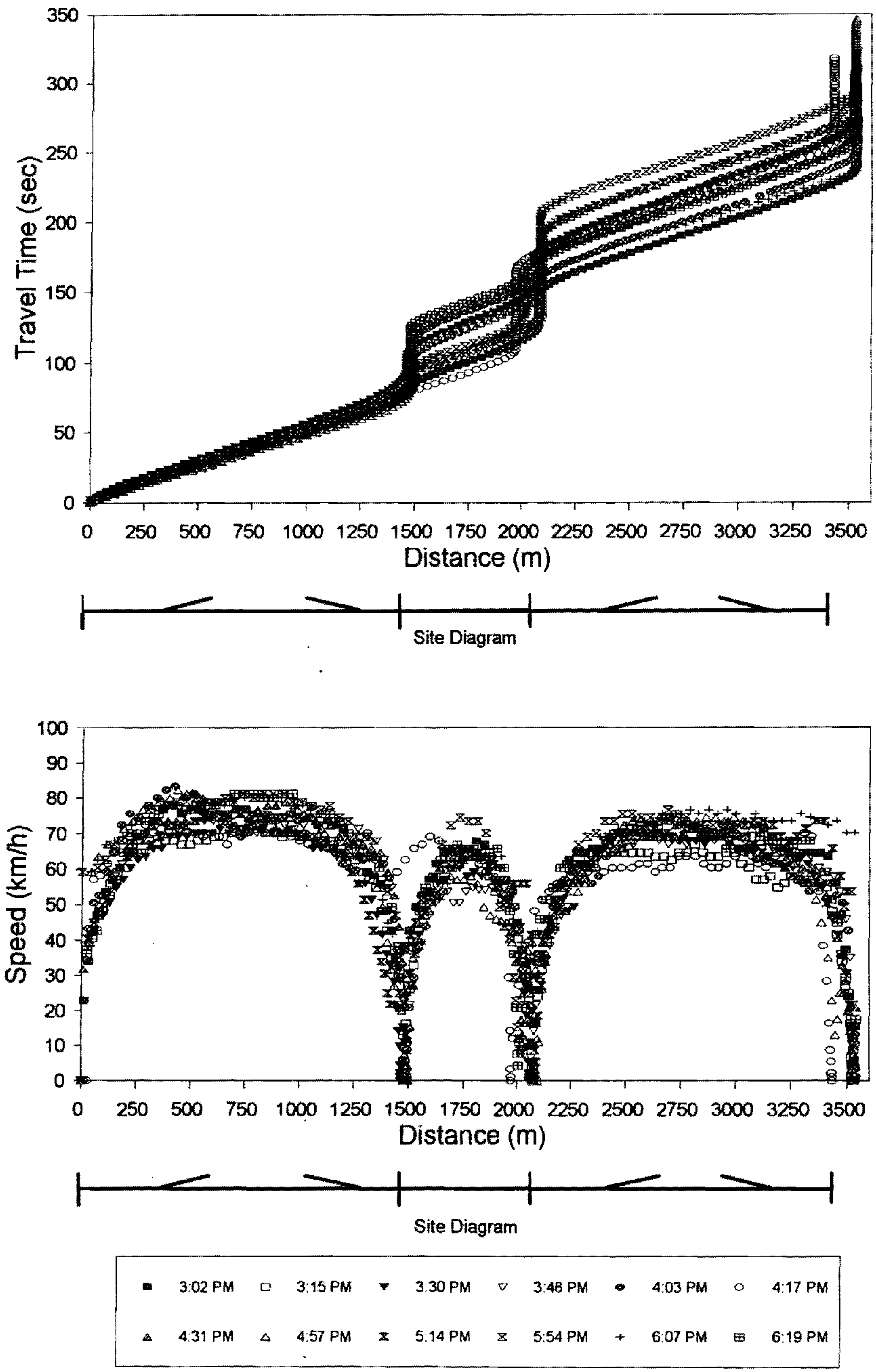
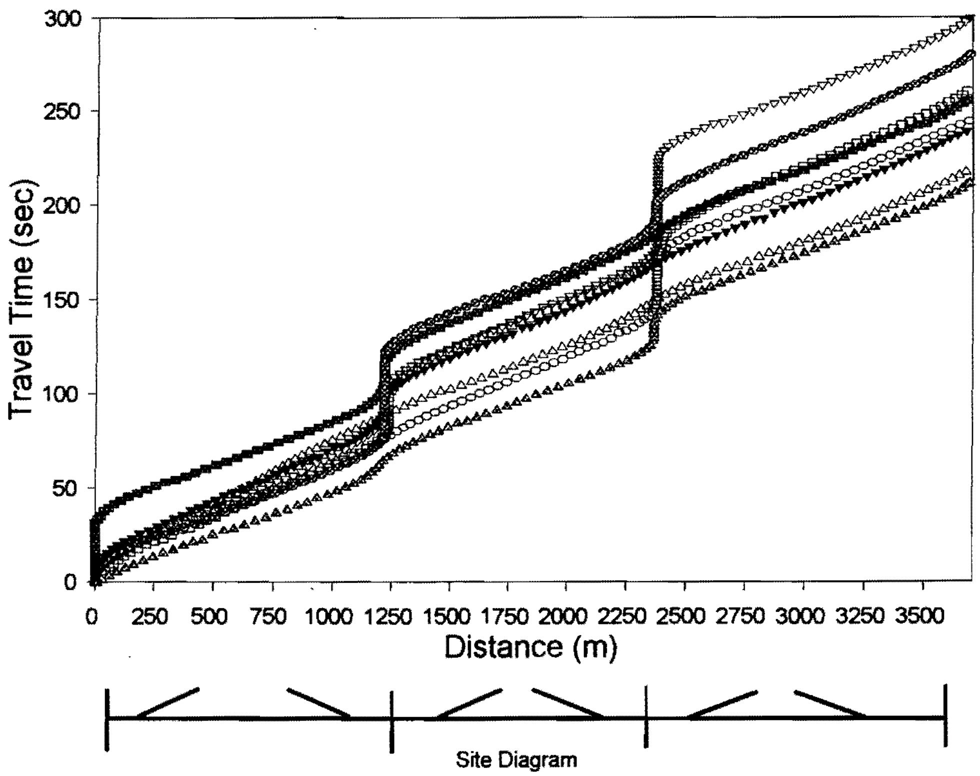


Figure B-12. Travel Time or Speed Versus Cumulative Distance for Site 12.



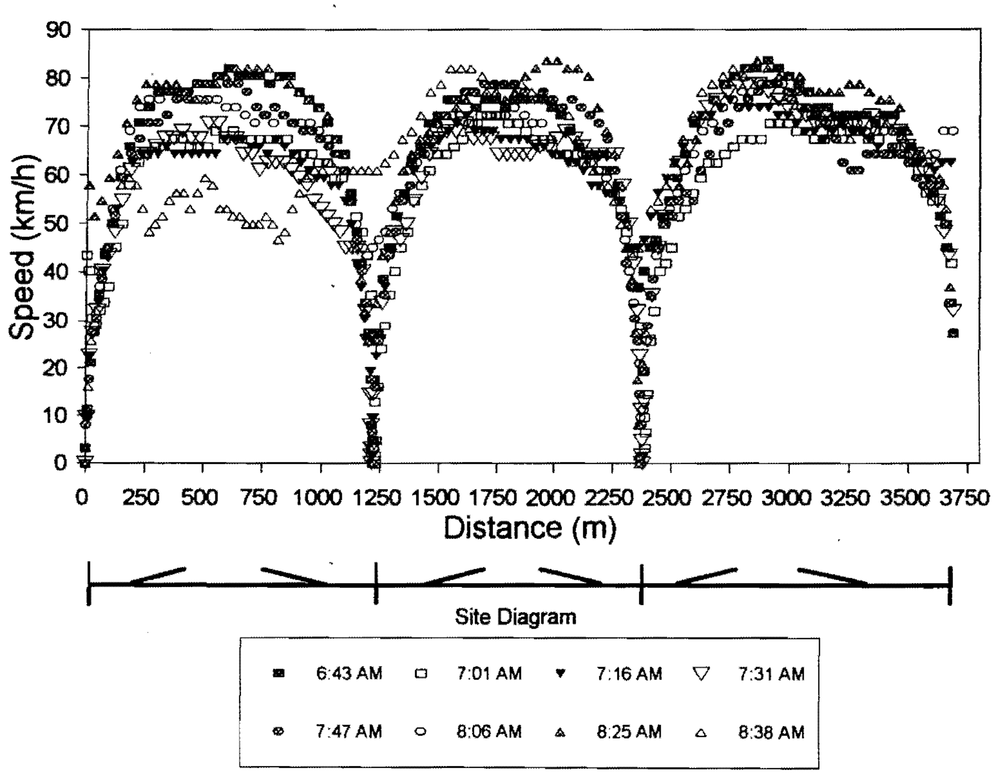
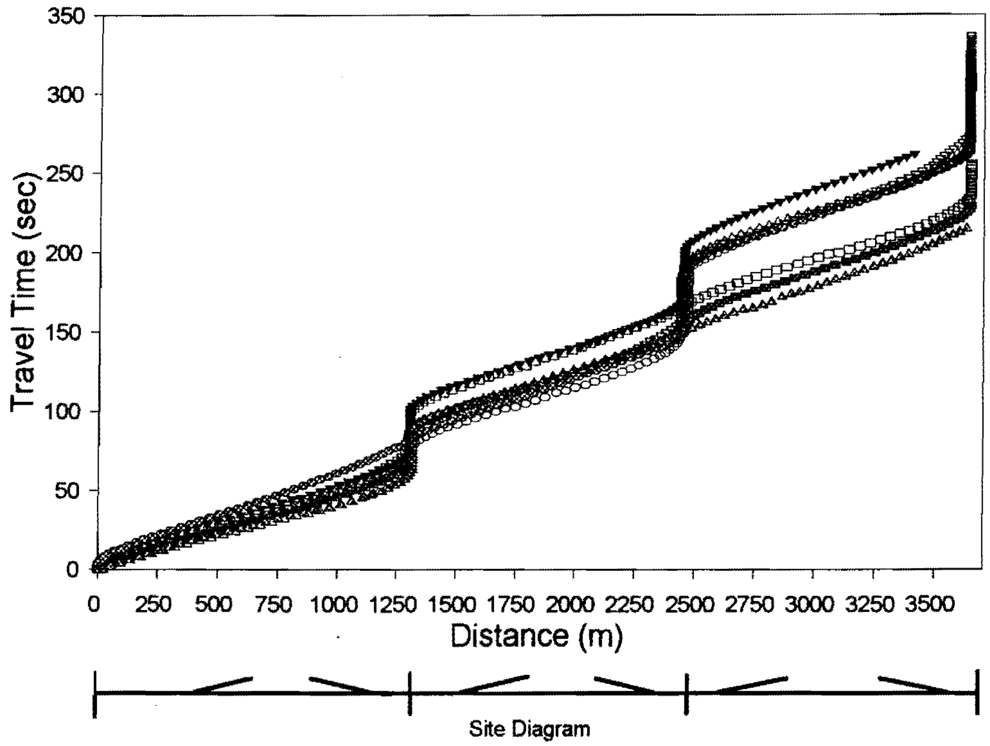


Figure B-13. Travel Time or Speed Versus Cumulative Distance for Site 13.



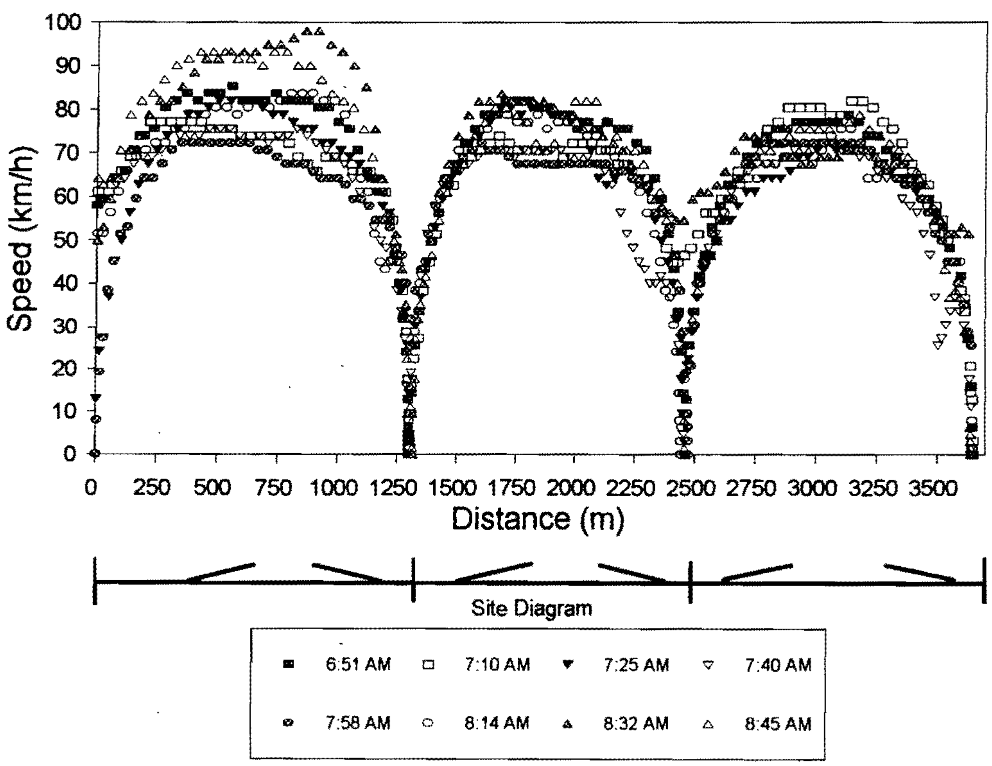


Figure B-14. Travel Time or Speed Versus Cumulative Distance for Site 14.

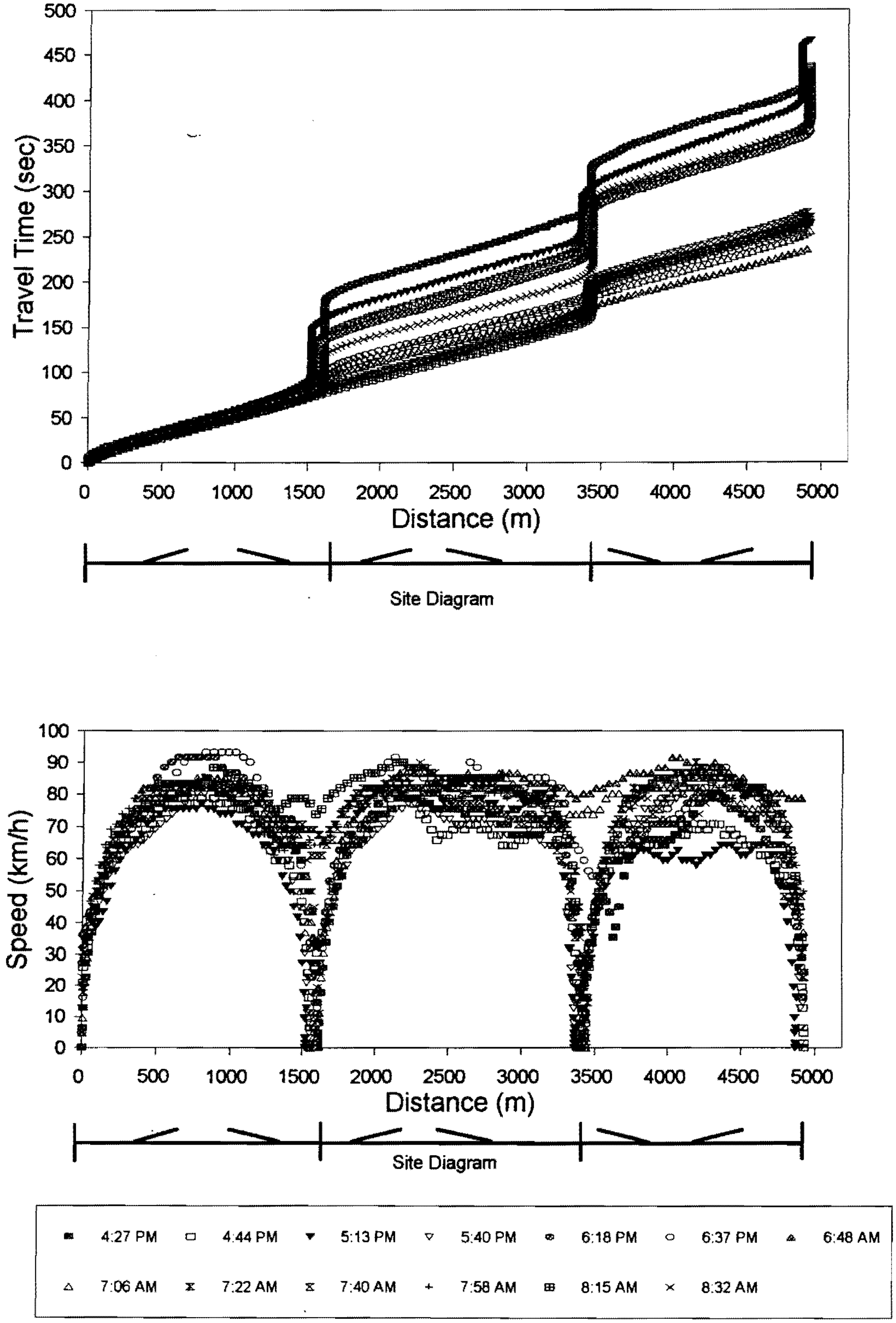


Figure B-15. Travel Time or Speed Versus Cumulative Distance for Site 15.

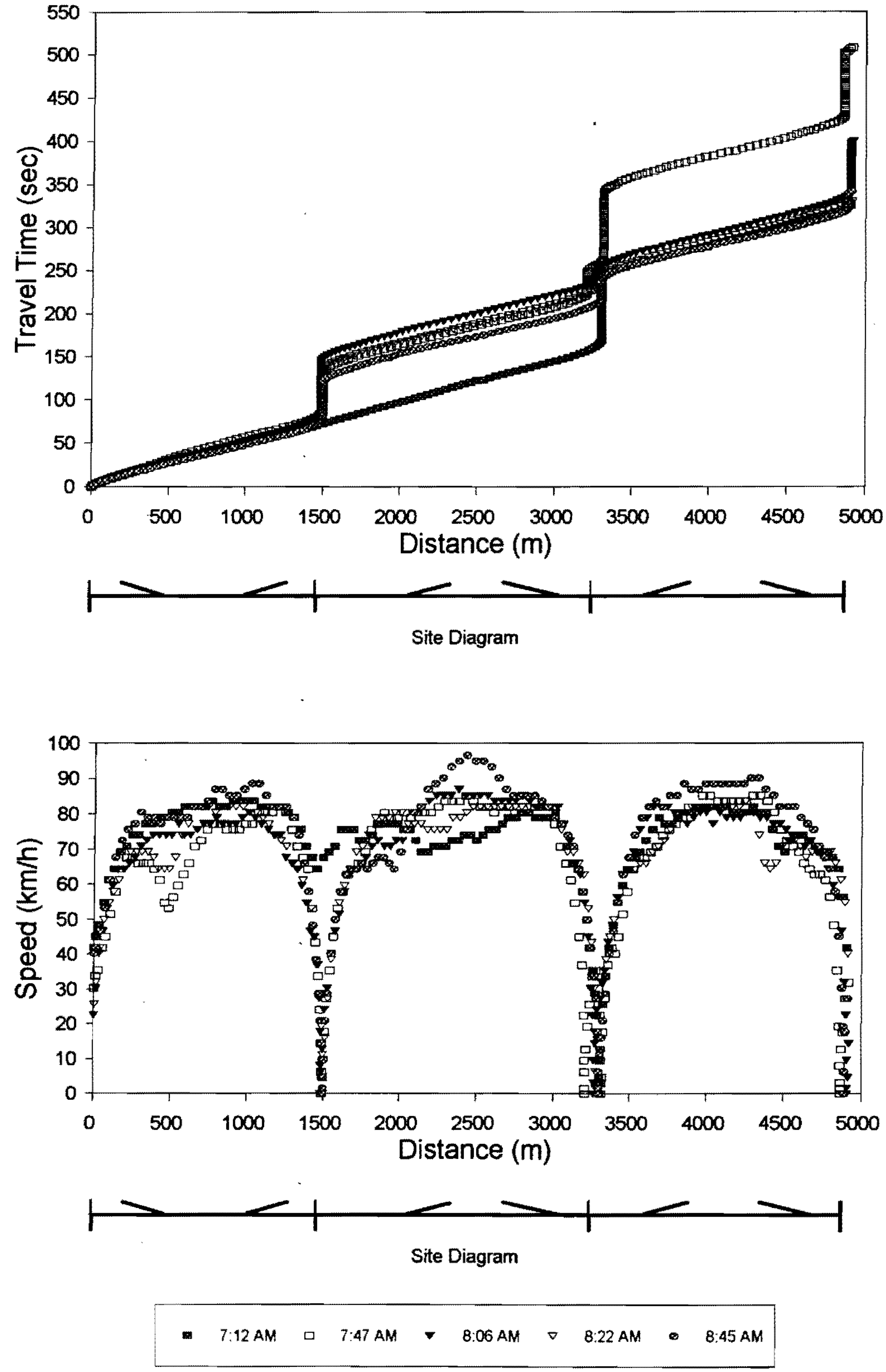
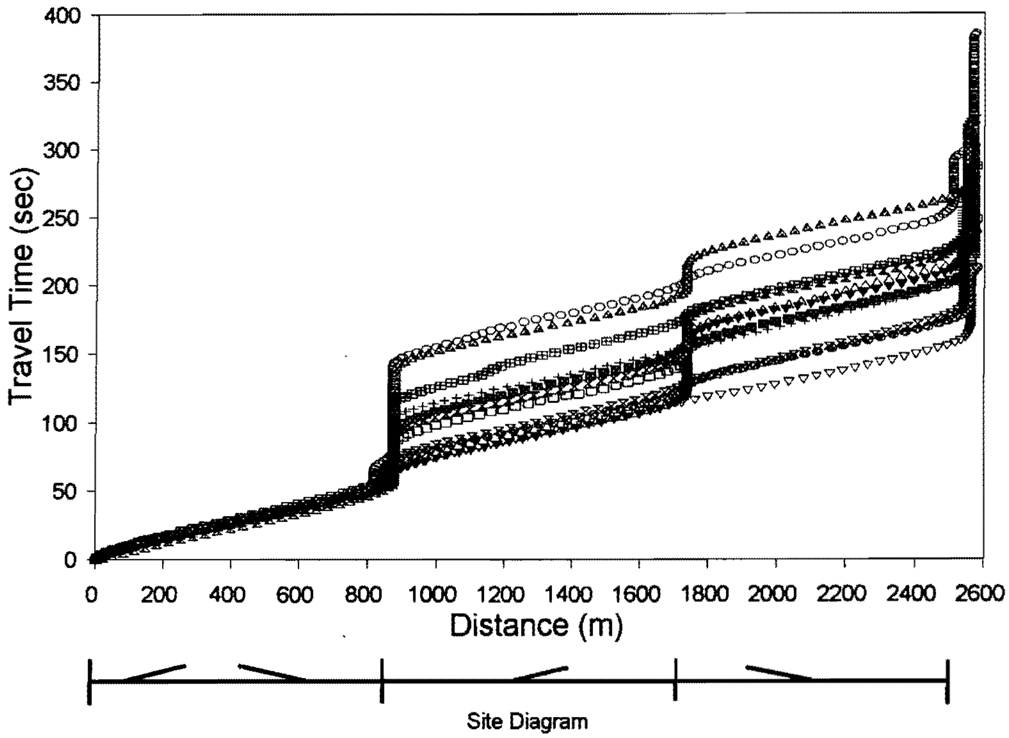


Figure B-16. Travel Time or Speed Versus Cumulative Distance for Site 16.



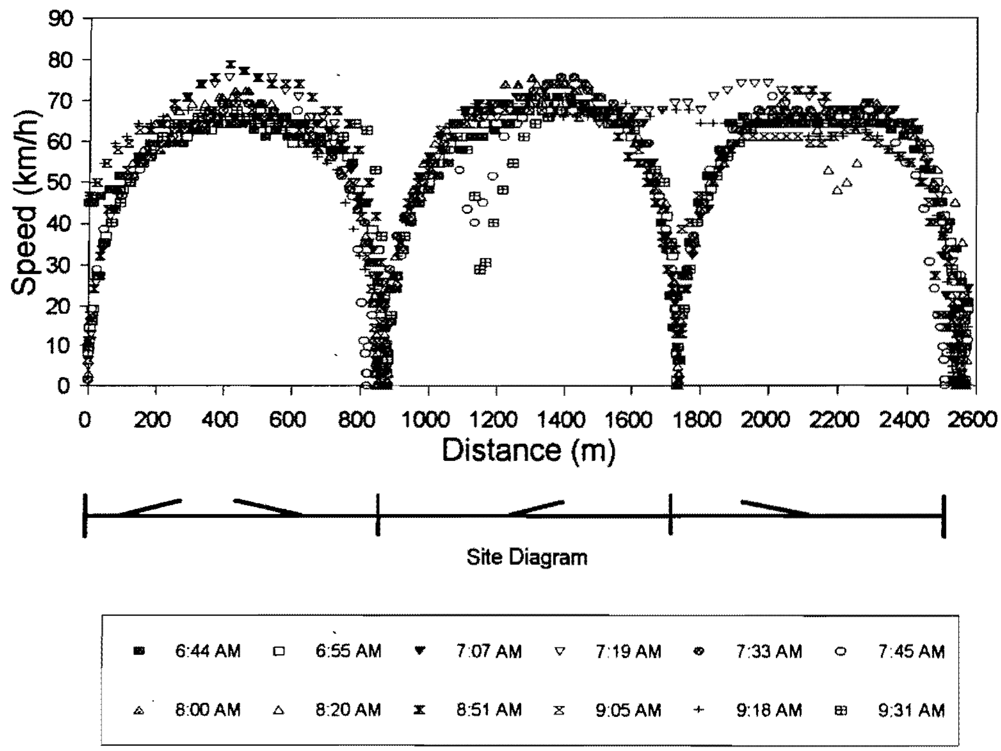


Figure B-17. Travel Time or Speed Versus Cumulative Distance for Site 17.

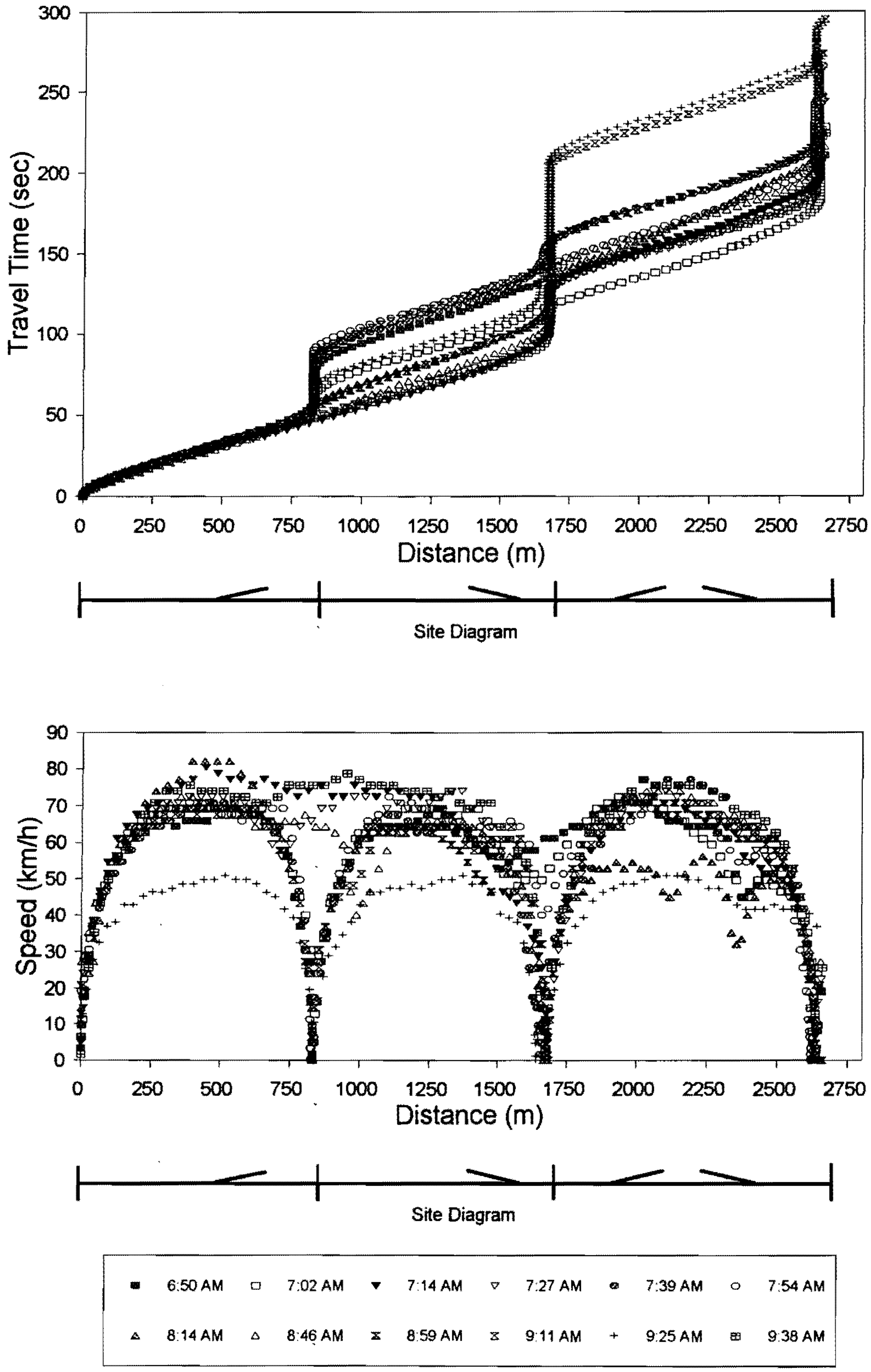
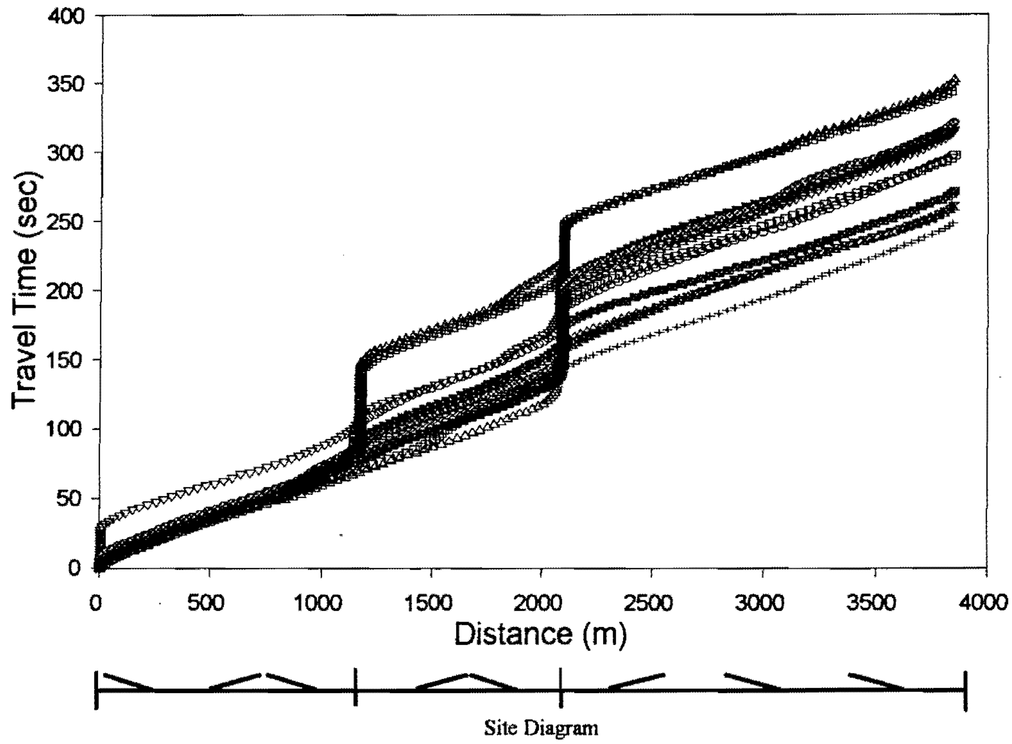


Figure B-18. Travel Time or Speed Versus Cumulative Distance for Site 18.



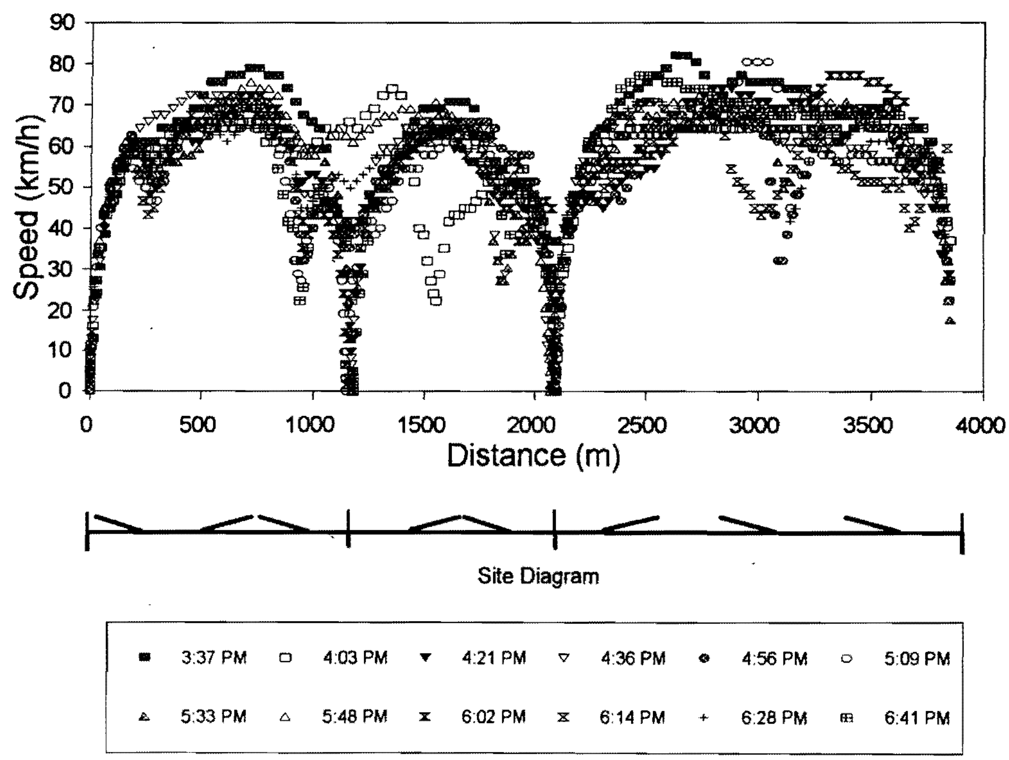
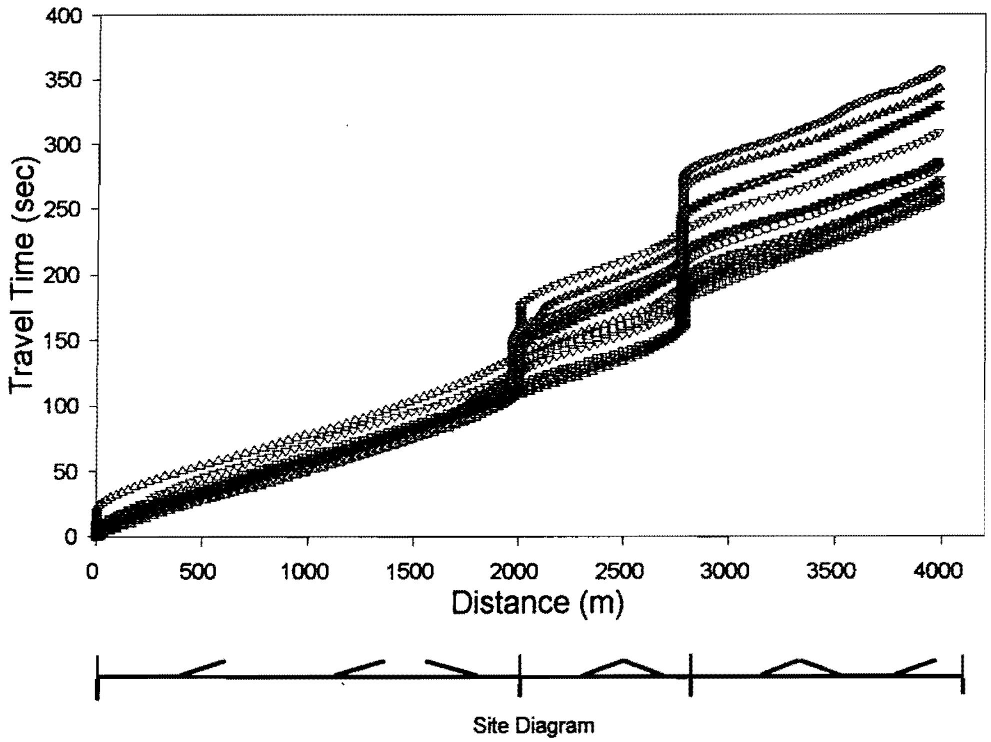


Figure B-19. Travel Time or Speed Versus Cumulative Distance for Site 19.



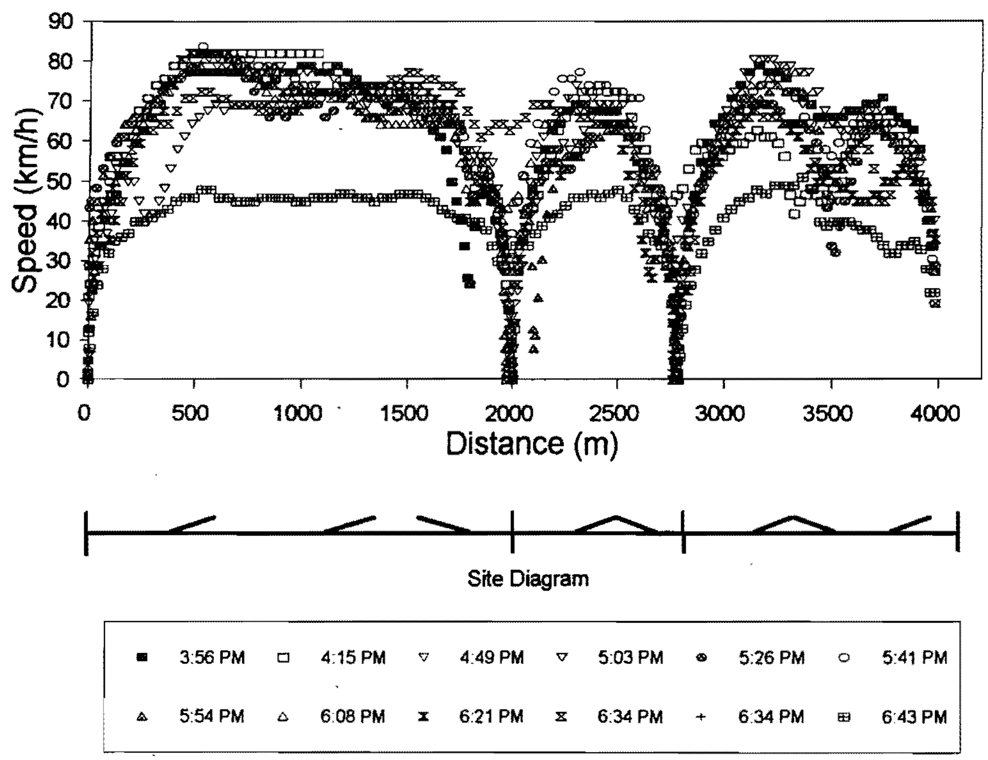


Figure B-20. Travel Time or Speed Versus Cumulative Distance for Site 20.

APPENDIX C

TWO-WAY FRONTAGE ROAD DATA

Appendix C contains the roadway characteristics for each of the nine two-way frontage road sites. The tables for each site also list the available volumes collected on the frontage road for each travel time run. In addition, the available g/C ratio (and the calculated capacity) for each intersection is shown. These data were used during the evaluation of two-way frontage roads. The legend for the tables follows:

- g/C = green time to cycle time ratio
- Capacity = saturation flow rate (vphgpl) x number of lanes x g/C, vph
- LEN = length for each link, m
- #L = average number of lanes per link (a value of 2.5 means a change in the number of lanes within the section)
- SL = speed limit for link, km/h
- FF Spd = free flow speed for link, km/h
- AD = calculated access density, acs/km
- Volume = flow rates for specific links and travel time runs, determined by converting 5-min volumes into hourly flow rates, vph
- R# = travel time run number

Site 21o, Gainesville, IH 35, Northbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)							
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R3	R5	R7	R9	R11	R13	R15
6	Overlay	Entrance					63	2	64	80	0.00	12	12	24	12	24	12	24	48
5	Entrance	Exit					1057	2	64	80	7.57	72	84	96	120	132	180	84	108
4	Exit	Entrance					717	2	64	80	5.58	72	84	96	120	144	192	84	132
3	Entrance	F.M. 1202					468	2	81	80	2.14								
2	F.M. 1202	Exit					555	2	81	80	1.80	12	12	48	24	24	12	36	0
1	Exit	Beg. of Guardrail					513	2	81	80	11.70	12	12	0	24	24	12	36	0

Site 21w, Gainesville, IH 35, Southbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)							
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R2	R4	R6	R8	R10	R12	R14	R16
1	Beg. of Guardrail	Exit					513	2	81	72	11.70	12	12	24	12	24	36	12	12
2	Exit	FM 1202 (CRR)					555	2	81	72	1.80	12	12	24	12	24	36	12	12
3	FM 1202 (CRR)	Entrance					468	2	81	72	2.14								
4	Entrance	Exit					717	2	81	72	5.58	96	84	120	36	132	96	156	168
5	Exit	Entrance					1057	2	64	72	7.57	96	36	120	36	132	96	156	168
6	Entrance	Overlay					63	2	64	72	0.00	12	24	12	12	12	24	36	12

Site 22o, Sulphur Springs, IH 30, Eastbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)									
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R4	R6	R8	R10	R12	R14	R16	R18	R19
8	S. Broadway	Exit					423	2	81	80	14.18	204	144	204	204	180	72	156	180	168	168
7	Exit	Exit					1088	2	81	80	10.11	204	144	204	204	180	288	180	180	216	216
6	Exit	Entrance					1487	2	81	80	7.40	192	216	228	96	228	132	180	180	192	216
5	Entrance	Exit					567	2	81	80	3.53	156	216	156	96	216	84	156	144	228	132
4	Exit	Entrance					585	2	81	80	1.71	120	36	144	72	132	84	180	120	84	72
3	Entrance	Loop 301					176	2	81	80	5.69	120	36	60	72	132	60	180	144	84	72
2	Loop 301	Exit					319	2	81	80	0.00	12	36	48	24	36	36	36	48	36	60
1	Exit	Stop Bar					849	2	81	80	7.07	12	36	48	24	36	36	36	48	36	60

Site 22w, Sulphur Springs, IH 30, Westbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)									
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R4	R6	R8	R10	R12	R14	R16	R18	R19
1	Stop Bar	Exit					849	2	81	80	7.07	12	24	36	36	0	24	24	24	24	24
2	Exit	Loop 301					319	2	81	80	0.00	12	24	36	36	0	24	24	24	24	24
3	Loop 301	Entrance					176	2	81	80	5.69	36	48	60	120	36	36	132	144	192	120
4	Entrance	Exit					585	2	81	80	1.71	48	72	60	48	108	36	132	144	192	120
5	Exit	Entrance					567	2	81	80	3.53	48	156	108	108	108	108	132	180	120	108
6	Entrance	Exit					1487	2	81	80	7.40	96	156	120	192	96	156	132	192	144	156
7	Exit	Exit					1088	2	81	80	10.11	132	204	360	228	228	288	348	228	348	216
8	Exit	S. Broadway	0.22	0.22	400	400	423	2	81	80	14.18	132	204	360	228	228	288	348	228	348	216

Site 23o, Sulphur Springs, IH 30, Eastbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	#L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R3	R5	R7	R9	R11	R13	R15
7	Underpass	Entrance					628	2	89	80	1.59		72	132	108	156	120	120	60
6	Entrance	Exit					455	2	89	80	8.79		84	180	144	168	144	144	72
5	Exit	Entrance					482	2	89	80	10.38		168	180	264	240	144	228	216
4	Entrance	Broadway	0.29		520		440	2	89	80	24.99	144	168	180	264	240	144	180	216
3	Broadway	Exit					147	2	81	80	13.64	108	48	84	96	192	108	84	180
2	Exit	Entrance					751	2	81	80	15.97	108	72	96	108	192	108	84	180
1	Entrance	FM Route Sign					351	2	81	80	2.85	72	72	84	108	96	108	72	144

Site 23w, Sulphur Springs, IH 30, Westbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	#L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R3	R5	R7	R9	R11	R13	R15
1	F.M. Route Sign	Entrance					351	2	81	80	2.85	60	96	144	204	144	240	144	120
2	Entrance	Exit					751	2	81	80	15.97	120	96	168	252	144	240	204	120
3	Exit	Broadway	0.29		520		147	2	81	80	13.64	120	96	156	192	144	192	204	204
4	Broadway	Entrance					440	2	81	80	24.99	48	24	72	108	108	48	84	60
5	Entrance	Exit					482	2	81	80	10.38	72	60	108	108	24	108	84	108
6	Exit	Entrance					455	2	81	80	8.79	72	60	48	132	24	48	108	96
7	Entrance	Underpass					628	2	81	80	1.59		60	48	132	24	36	168	96

Site 24o, New Braunfels, IH 35, Northbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	#L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R3	R4	R6	R8	R10	R12	R14	R16	R17	R18
7	Concrete Median	Exit					58	2	64	64	0.00	72	48	72	96	132	120	96	96	108	60	144
6	Exit	Entrance					436	2	64	64	11.47	84	120	36	108	132	216	168	228	108	84	168
5	Entrance	SH 46	0.28	0.20	511	367	311	2	64	64	16.08	84	120	36	84	132	216	168	228	96	84	168
4	SH 46	Exit					198	2	64	72	15.14	12	36	12	36	36	24	24	24	36	24	48
3	Exit	Entrance					600	2	64	72	8.33	12	36	12	48	36	24	24	24	36	24	48
2	Entrance	Exit					275	2	72	72	3.64	12	24	12	48	48	24	24	24	24	24	24
1	Exit	90					396	2	72	72	2.53	12	24	12	48	48	24	12	12	24	24	24

Site 24w, New Braunfels, IH 35, Southbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	#L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R2	R5	R7	R9	R11	R13	R15	R19
1	90	Exit					396	2	72	64	2.53	72	156	60	108	72	96	12	36
2	Exit	Entrance					275	2	72	64	3.64	72	156	60	108	72	96	12	36
3	Entrance	Exit					600	2	64	72	8.33	12	36	36	72	36	48	12	24
4	Exit	SH 46	0.28	0.20	511	367	198	2	64	72	15.14	12	36	36	72	36	48	12	24
5	SH 46	Entrance					311	2	64	72	16.08	132	264	120	120	180	216	168	180
6	Entrance	Exit					436	2	64	72	11.47	132	264	132	228	180	276	240	180
7	Exit	Concrete Median					58	2	64	72	0.00	120	264	132	180	168	276	240	180

Site 25o, New Braunfels, IH 35, Northbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	#L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R2	R4	R6	R8	R10	R12	R14	R16	R18
5	Exit	Entrance					875	2	64	72	14.86	360	276	252	288	324	372	504	528	492
4	Entrance	Walnut	0.34		603		445	2	64	72	20.22	360	276	252	288	324	372	504	528	492
3	Walnut	Exit					338	2	64	72	14.78	204	204	252	216	120	192	276	252	180
2	Exit	Entrance					646	2	64	72	1.55	204	204	180	216	348	192	276	252	180
1	Entrance	Schmidt Ave.					786	2	64	72	12.72		84	108	228	276	108	132	144	204

Site 25w, New Braunfels, IH 35, Southbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	#L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R3	R5	R7	R9	R11	R13	R15	R17
1	Schmidt Ave.	Entrance					786	2	64	72	12.72	180	108	144	96	132	120	180	156	108
2	Entrance	Exit					646	2	64	72	1.55	180	168	240	156	144	132	312	156	228
3	Exit	Walnut	0.34		603		338	2	64	72	14.78	192	168	240	156	144	132	312	144	84
4	Walnut	Entrance					445	2	64	72	20.22	252	228	276	336	0	312	360	360	336
5	Entrance	Exit					875	2	64	72	14.86	336	360	432	312	432	360	420	360	336

Site 26o, Hilsboro, IH 35, Northbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	#L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R1	R3	R6	R8	R10	R12	R14
5	Arbitrary	Entrance					296	2	72	72	0.00	36	12	12	24	48	36	12
4	Entrance	Corsicana		0.36		643	436	2	56	72	9.18	36	12	12	24	48	36	12
3	Corsicana	Exit					341	2	56	72	32.22	96	252	132	132	216	156	84
2	Exit	Brandon Rd.					625	2	72	72	17.60	96	252	132	132	216	156	108
1	Brandon Rd.	Exit					262	2	72	72	19.07	12	36	36	24	24	0	0

Site 26w, Hilsboro, IH 35, Southbound

Link	Node A	Node B	g/C Peak	g/C Off	Capacity Peak	Capacity Off	LEN (m)	#L	SL (km/h)	FF Spd (km/h)	AD (acs/km)	Volume (vph) R2	R4	R5	R7	R9	R11	R13	R15
1	Exit	Brandon Rd.					262	2	72	72	19.07	0	12	24	24	36	12	12	12
2	Brandon Rd.	Exit					625	2	72	72	17.60	216	312	240	264	240	336	216	312
3	Exit	Corsicana		0.36		643	341	2	56	72	32.22	216	312	240	264	192	336	216	312
4	Corsicana	Entrance					436	2	56	72	9.18	48	24	12	24	24	12	12	12
5	Entrance	Arbitrary					296	2	72	72	0.00	48	24	12	0	24	12	12	12

Site 27o, Huntsville, IH 45, Northbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)							
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R2	R4	R6	R8	R10	R12	R14	R16
9	SH 75	Exit					393	2	64	89	7.63	36	24	72	120	72	24	60	24
8	Exit	Entrance					2198	2	81	89	3.64	204	240	228	300	132	132	216	192
7	Entrance	SH 30	0.34		620		302	2	64	89	16.57	240	264	396	264	360	252	108	204
6	SH 30	Exit					259	2	64	89	19.30	120	72	144	168	120	72	108	48
5	Exit	Avenue "S"					1332	2	64	89	12.01	324	228	216	276	324	348	288	216
4	Avenue "S"	Entrance					1042	2	64	89	3.84	240	264	396	264	360	252	108	228
3	Entrance	FM 1374	0.31		554		283	2	64	89	17.64	204	192	228	300	132	192	216	192
2	FM 1374	Exit					290	2	64	89	24.12	120	72	144	168	120	72	108	48
1	Exit	Arbitrary					268	2	81	89	7.47	120	72	144	168	120	72	108	48

Site 27w, Huntsville, IH 45, Southbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)							
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R3	R5	R7	R9	R11	R13	R15
1	Arbitrary	Exit					268	2	81	89	7.47	156	108	120	252	96	180	144	168
2	Exit	FM 1374	0.31		554		290	2	64	89	24.12	372	264	324	444	192	444	240	312
3	FM 1374	Entrance					283	2	64	89	17.64	384	252	264	276	252	240	252	240
4	Entrance	Avenue "S"					1042	2	81	89	3.84	384	372	264	276	252	240	252	240
5	Avenue "S"	Exit					1332	2	81	89	12.01	348	156	240	216	204	156	240	288
6	Exit	SH 30	0.34		620		259	2	81	89	19.30	696	348	492	576	420	300	384	420
7	SH 30	Entrance					302	2	64	89	16.57	264	168	276	240	300	240	288	168
8	Entrance	Exit					2198	2	81	89	3.64	264	168	276	240	300	240	300	168
9	Exit	SH 75	0.37		669		393	2	81	89	7.63	528	384	552	456	516	336	396	408

Site 28o, Huntsville, IH 45, Northbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)							
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R3	R5	R7	R9	R11	R13	R15
8	Arbitrary	Exit					244	2	81	80	4.10								
7	Exit	FM 1374		0.37		665	262	2	64	80	19.07	156	96	84	48	48	192	96	156
6	FM 1374	Exit					290	2	64	80	13.81	396	264	240	192	252	216	252	396
5	Exit	Entrance					2399	2	81	80	3.34	396	288	372	372	300	240	432	456
4	Entrance	SH 30		0.29		520	299	2	64	80	13.39	312	288	372	372	372	240	432	420
3	SH 30	Exit					226	2	64	80	26.60	204	156	168	96	204	252	144	240
2	Exit	Entrance					2158	2	81	80	5.10	204	156	168	96	204	252	144	240
1	Entrance	SH 75		0.21		386	381	2	81	80	0.00	396	264	240	192	252	216	252	396

Site 28w, Huntsville, IH 45, Southbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)							
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R2	R4	R6	R8	R10	R12	R14	R16
1	SH 75	Entrance					381	2	81	80	0.00	72	72	156	144	132	300	168	192
2	Entrance	Exit					2158	2	81	80	5.10	144	204	228	168	228	360	228	216
3	Exit	SH 30		0.29		520	226	2	64	80	26.60	300	288	336	300	324	528	360	372
4	SH 30	Entrance					299	2	64	80	13.39	264	276	276	288	132	420	372	324
5	Entrance	Exit					2399	2	81	80	3.34	312	276	276	288	336	324	408	336
6	Exit	FM 1374		0.37		665	290	2	64	80	13.81	384	384	348	372	468	492	552	480
7	FM 1374	Exit					262	2	64	80	19.07								
8	Exit	Arbitrary					244	2	81	80	4.10								

Site 29o, Hilsboro, IH 35, Northbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)							
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R2	R4	R6	R8	R10	R12	R14	R16
4	FM 286	Entrance					448	2	72	89	8.93								
3	Entrance	SH 22		0.31		566	500	2	56	89	20.01								
2	SH 22	Exit					351	2	56	64	5.71	12	24	12	12	12	12	24	12
1	Exit	Arbitrary					738	2	72	64	0.00	12	24	12	12	12	12	24	12

Site 29w, Hilsboro, IH 35, Sothbound

Link	Node A	Node B	g/C		Capacity		LEN	#L	SL	FF Spd	AD	Volume (vph)							
			Peak	Off	Peak	Off	(m)		(km/h)	(km/h)	(acs/km)	R1	R3	R5	R7	R9	R11	R13	R15
1	Arbitrary	Exit					738	2	72	72	0.00	24	36	24	60	24	36	24	24
2	Exit	SH 22		0.31		566	351	2	56	72	5.71	24	36	24	60	24	36	24	24
3	SH 22	Entrance					500	2	56	64	20.01								
4	Entrance	FM 286					448	2	72	64	8.93								

APPENDIX D
TWO-WAY FRONTAGE ROAD
TRAVEL TIME AND SPEED PLOTS

Appendix D contains the travel time and speed plots for each of the nine two-way frontage road sites by direction (e.g., with or oppose).

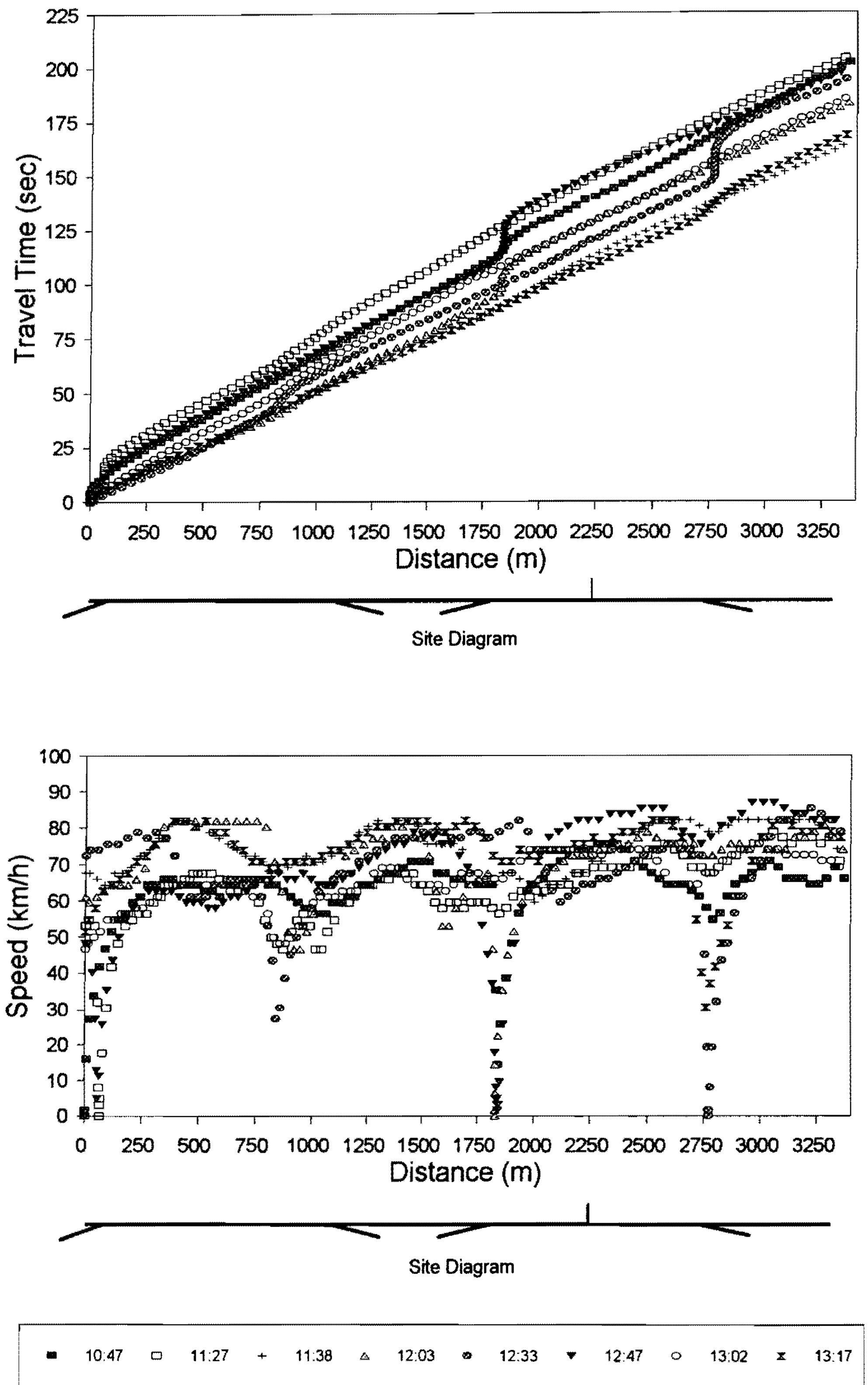


Figure D-1. Travel Time or Speed Versus Cumulative Distance for Site 21o.

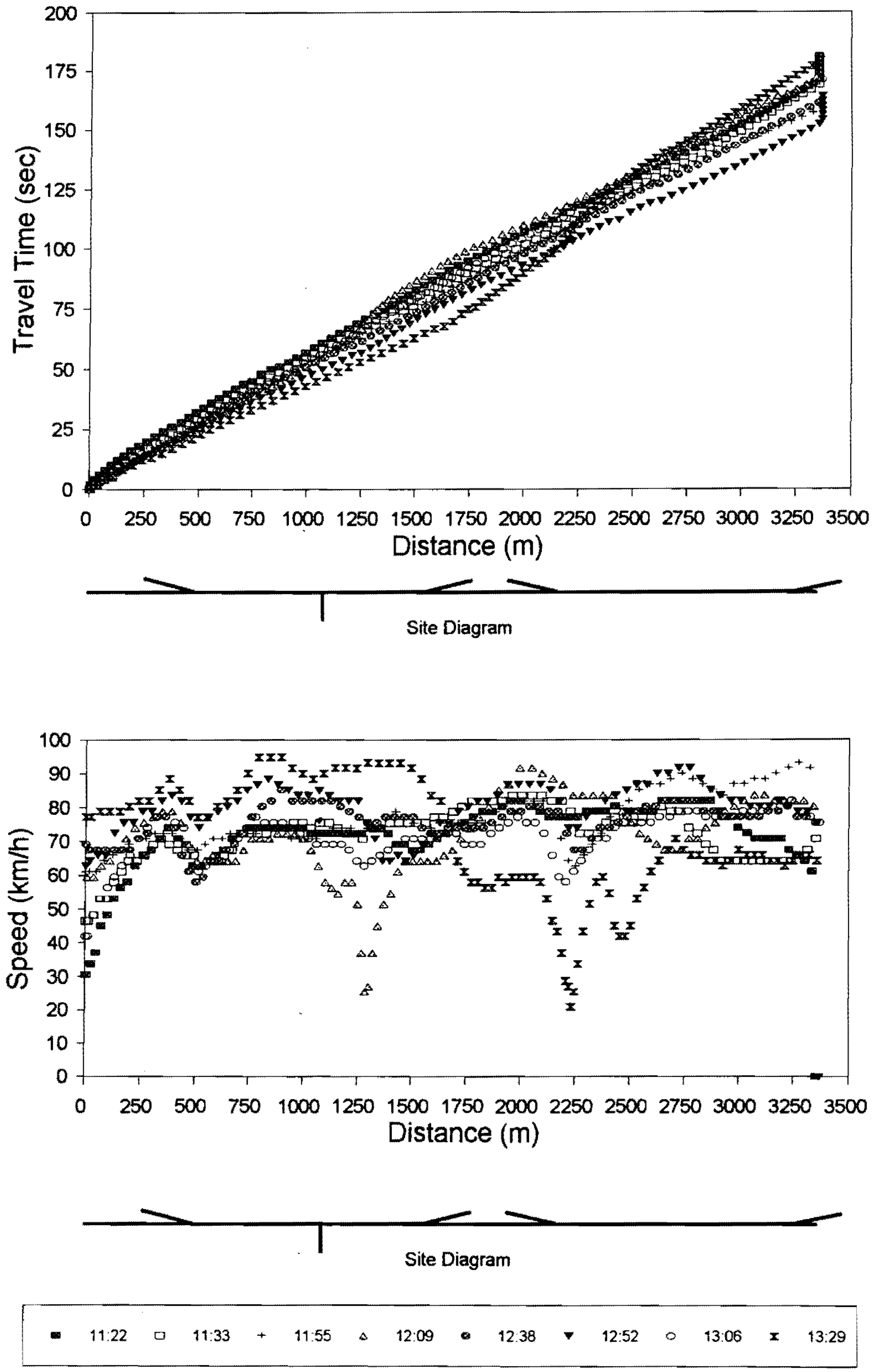
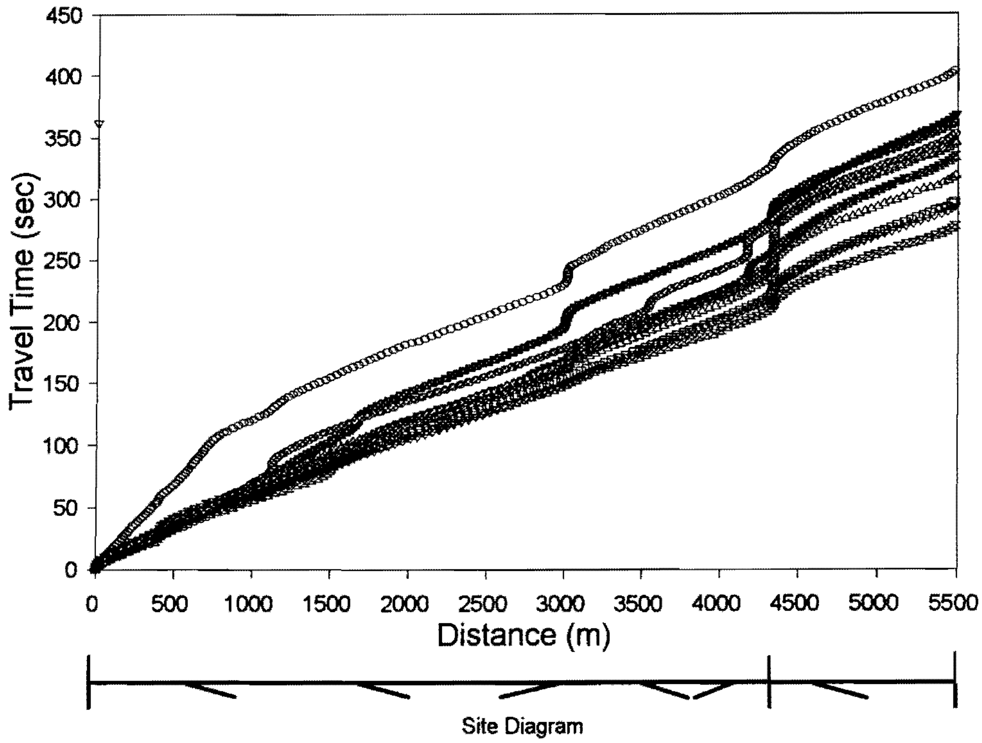


Figure D-2. Travel Time or Speed Versus Cumulative Distance for Site 21w.



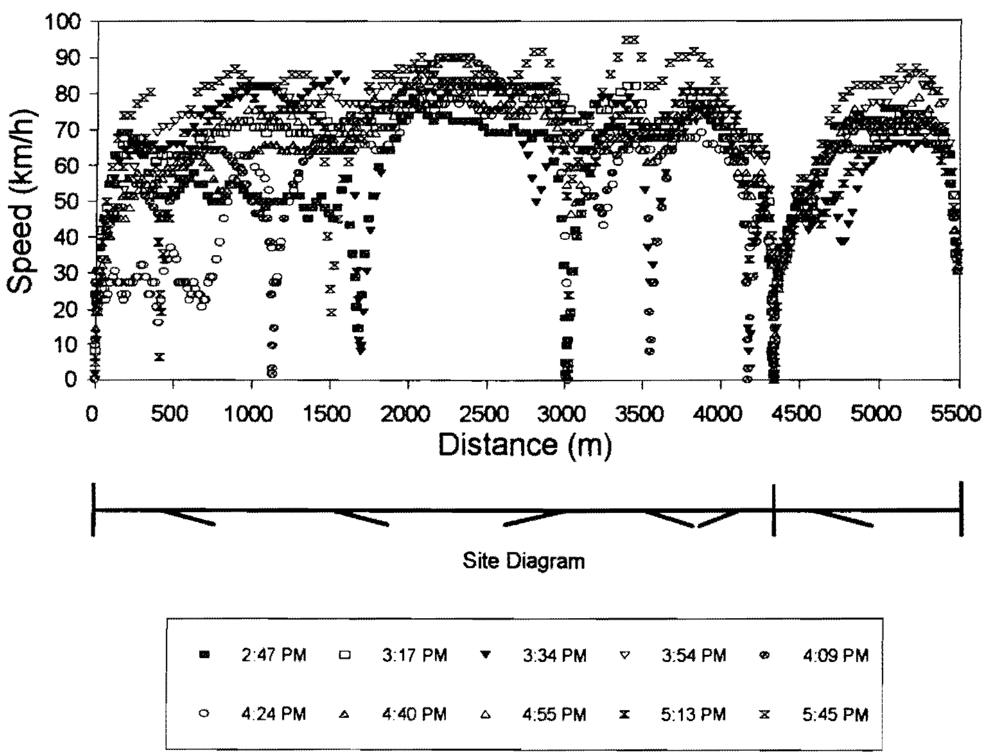
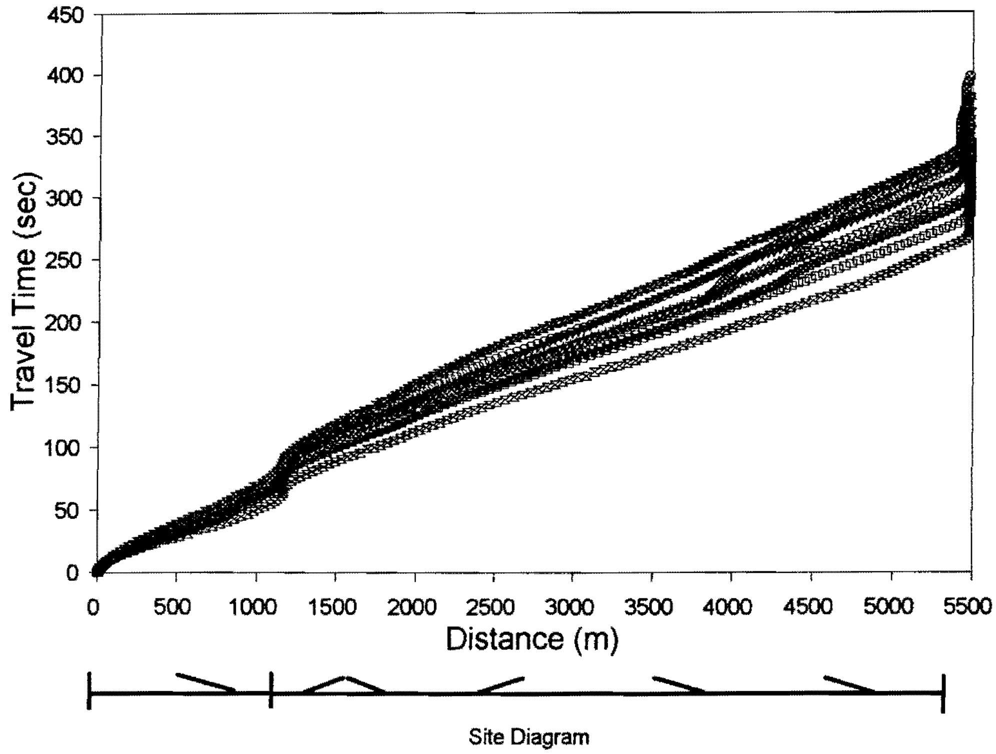


Figure D-3. Travel Time or Speed Versus Cumulative Site Distance for Site 22o.



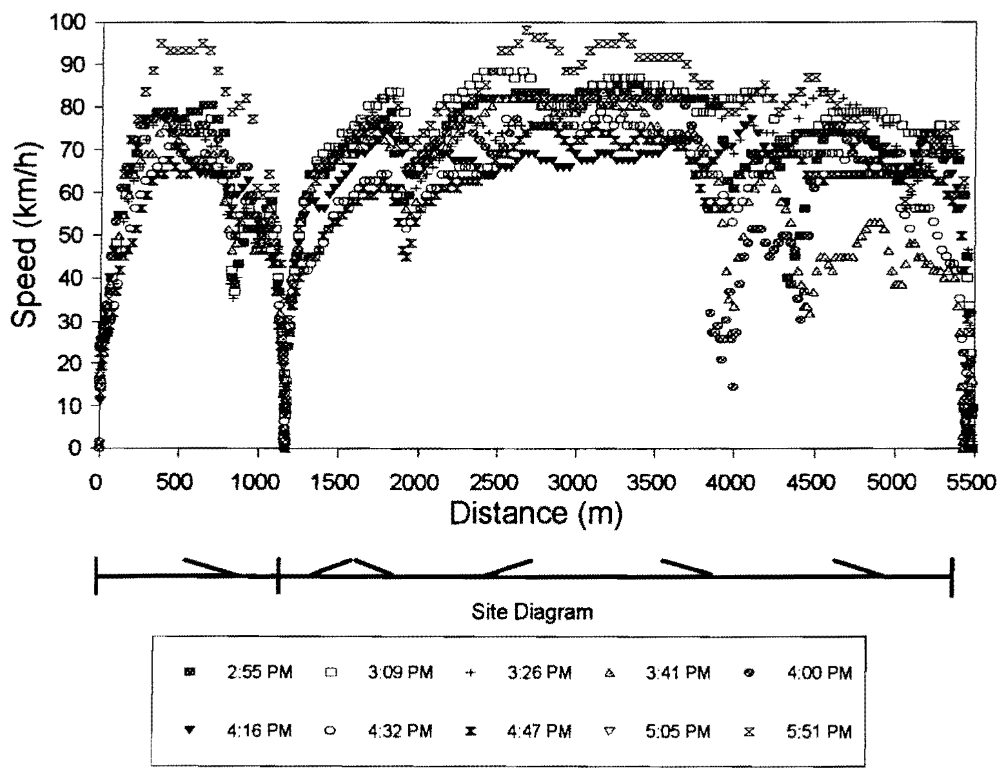
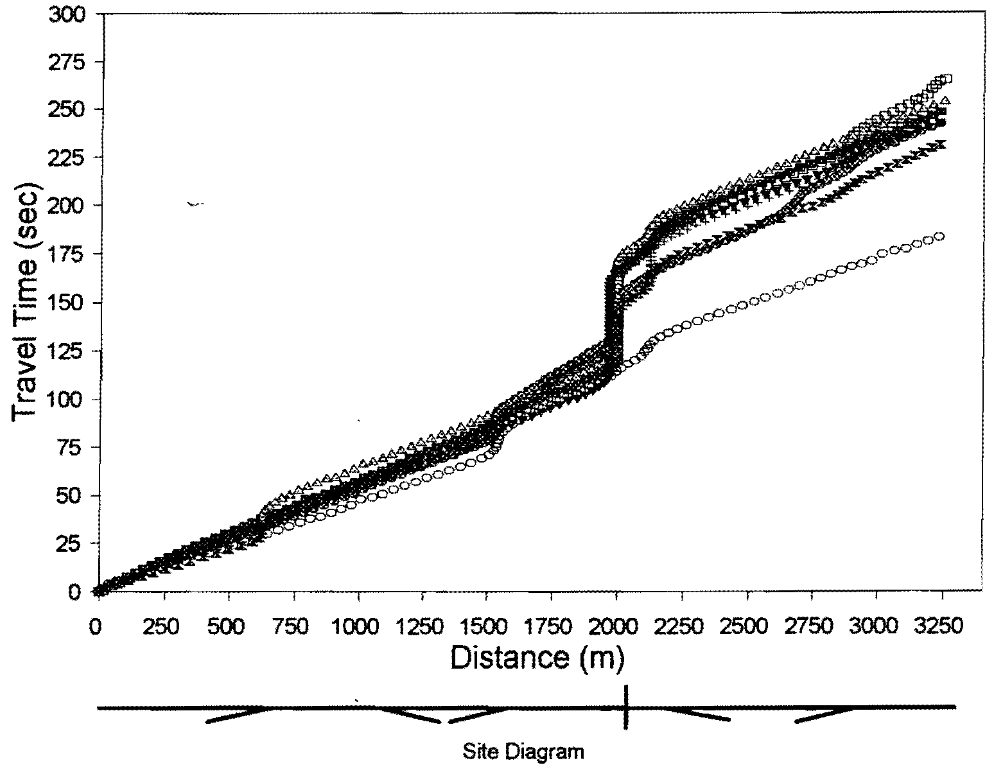


Figure D-4. Travel Time or Speed Versus Cumulative Distance for Site 22w.



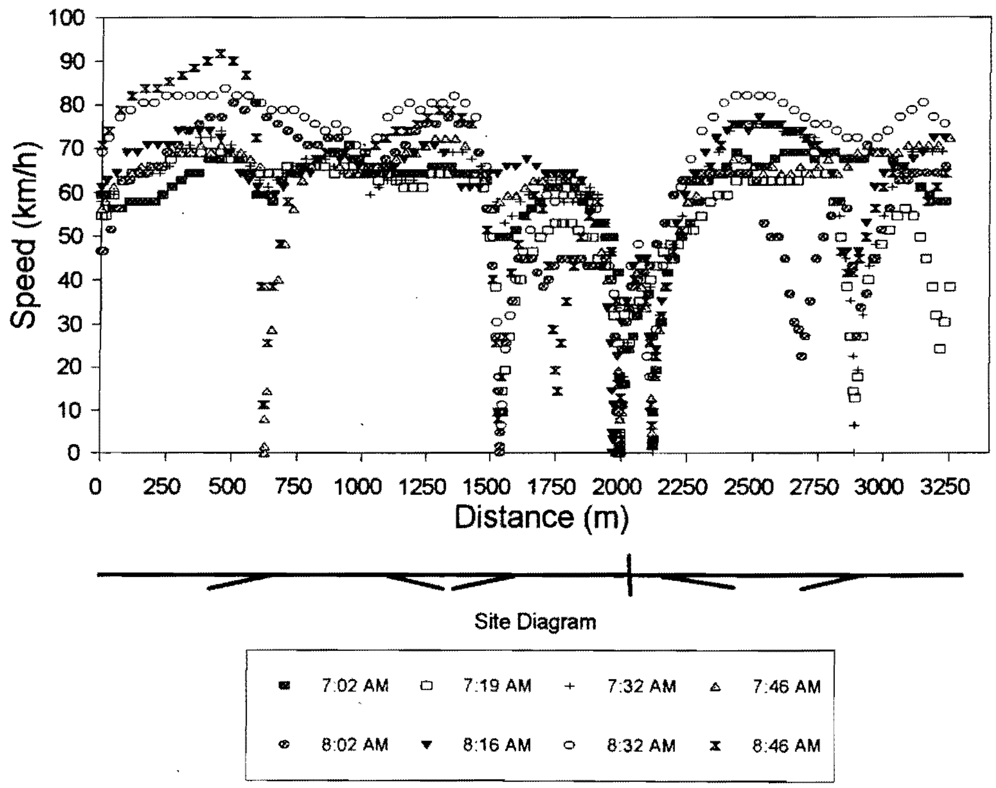


Figure D-5. Travel Time or Speed Versus Cumulative Distance for Site 23o.

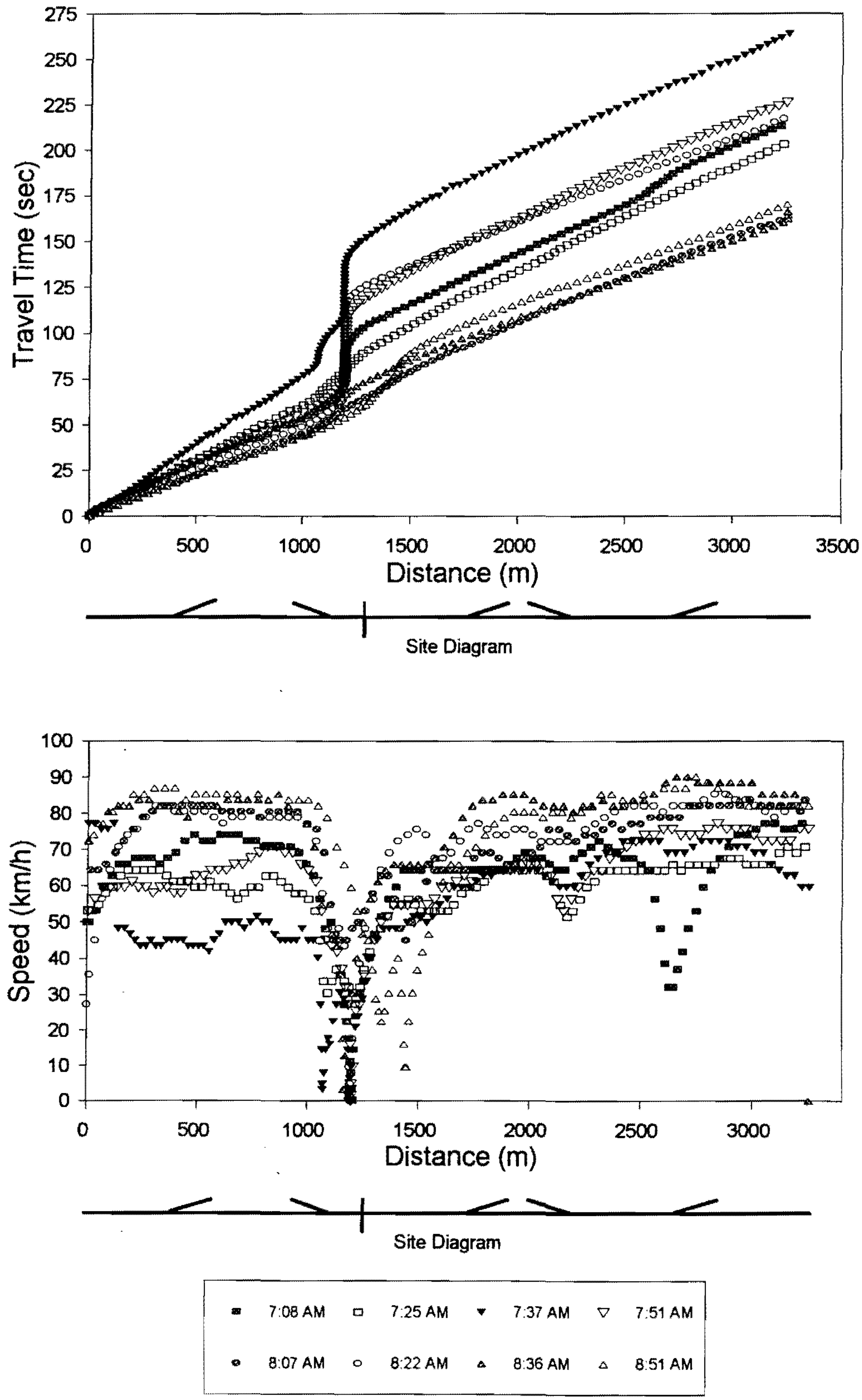
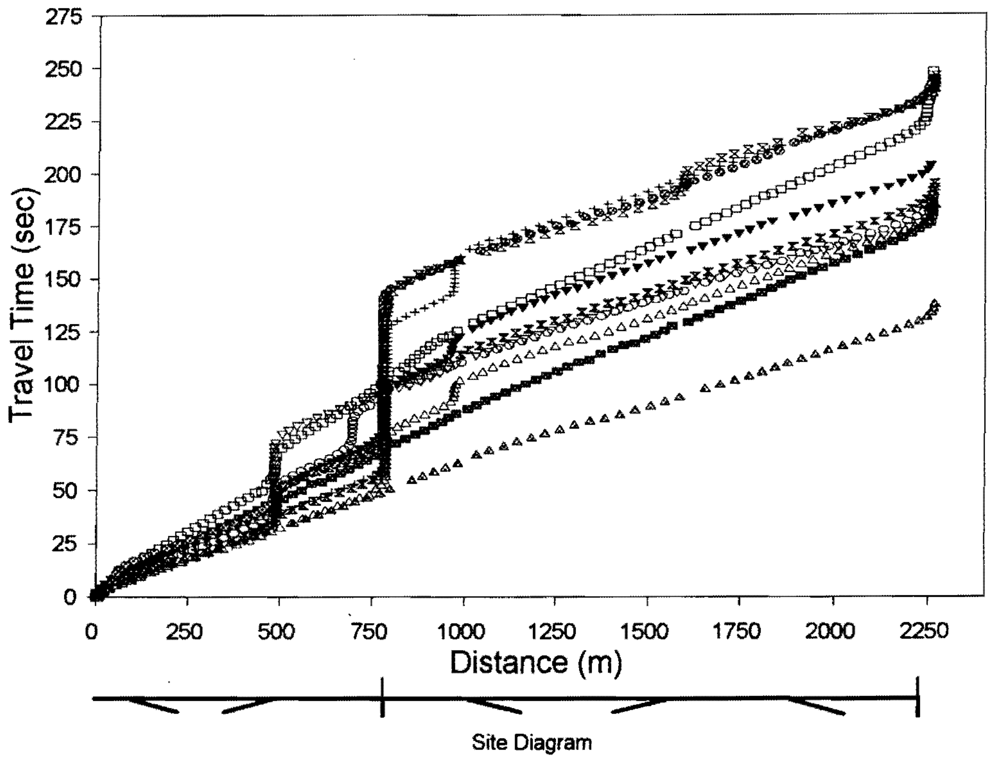


Figure D-6. Travel Time or Speed Versus Cumulative Distance for Site 23w.



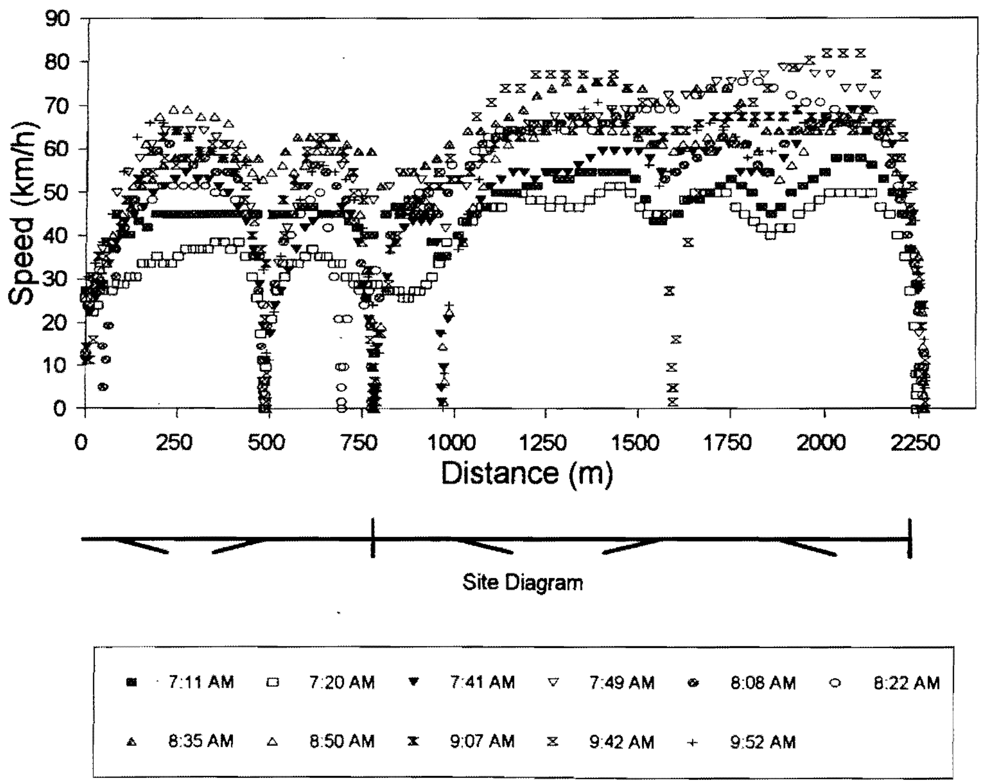


Figure D-7. Travel Time or Speed Versus Cumulative Distance for Site 24o.

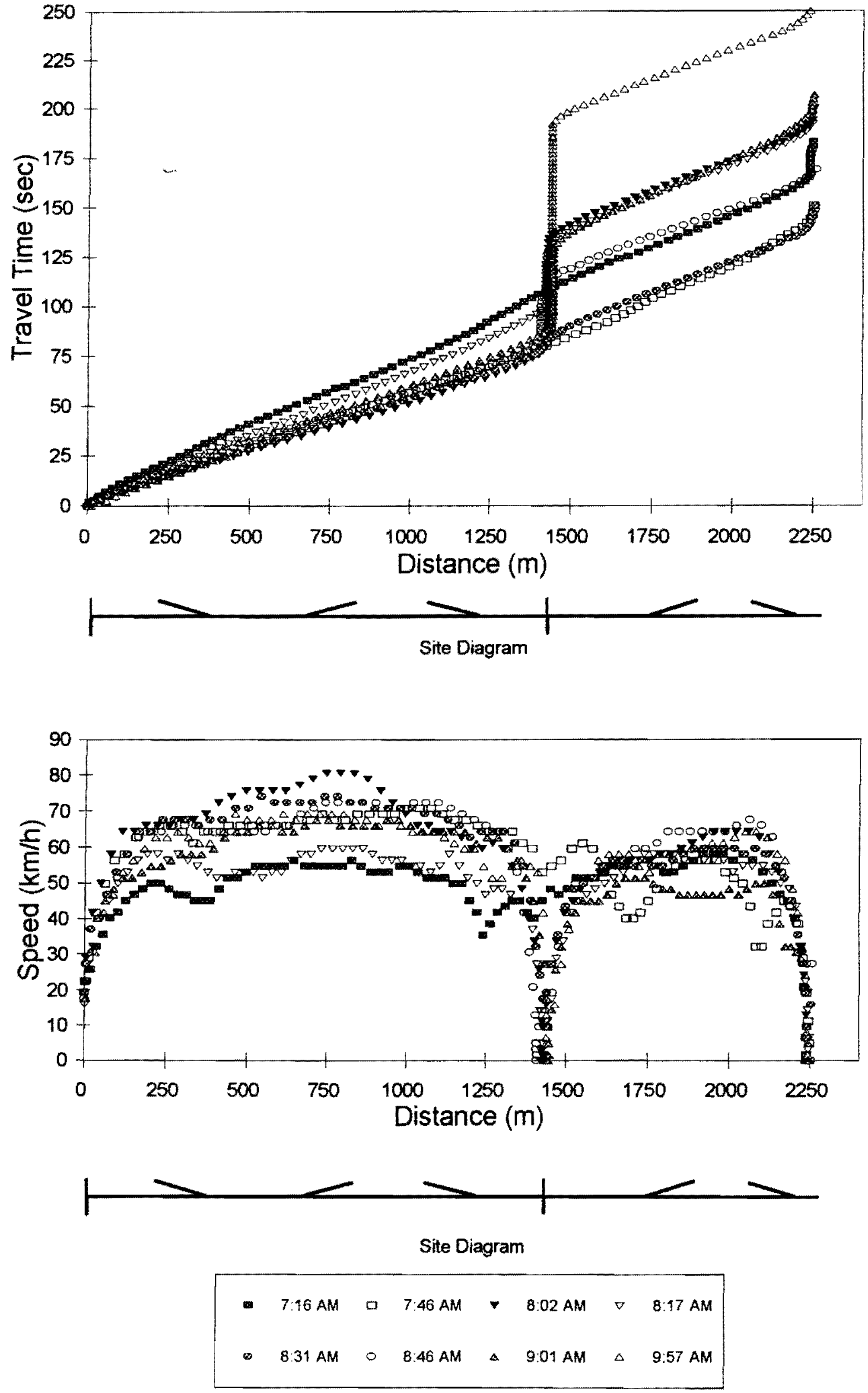
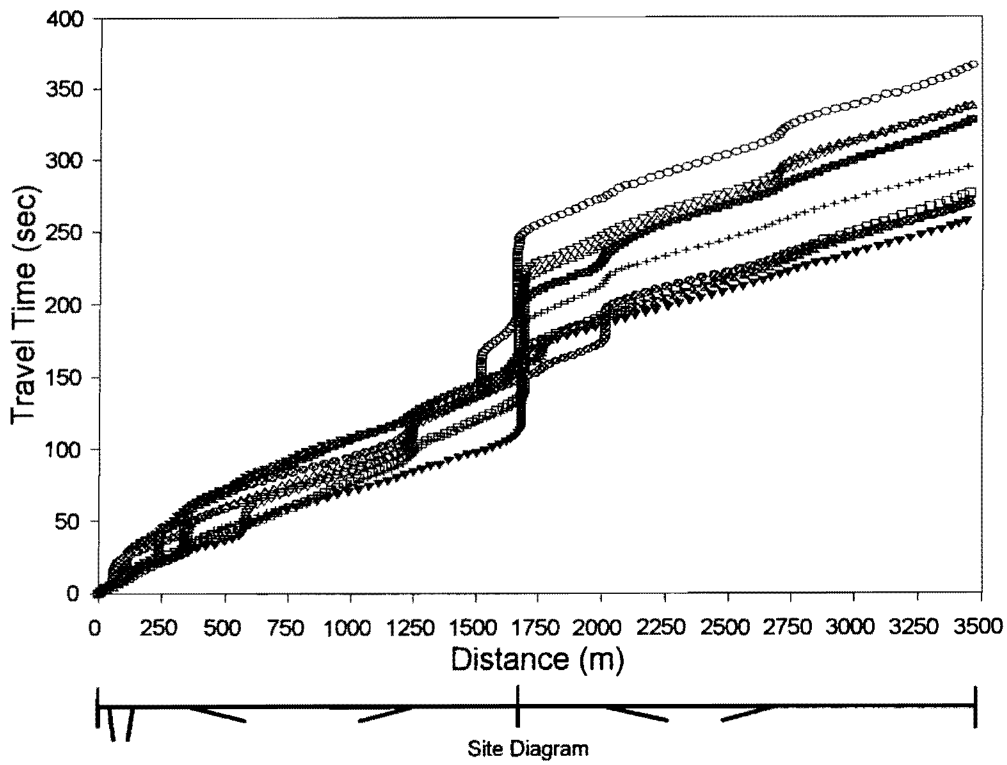


Figure D-8. Travel Time or Speed Versus Cumulative Distance for Site 24w.



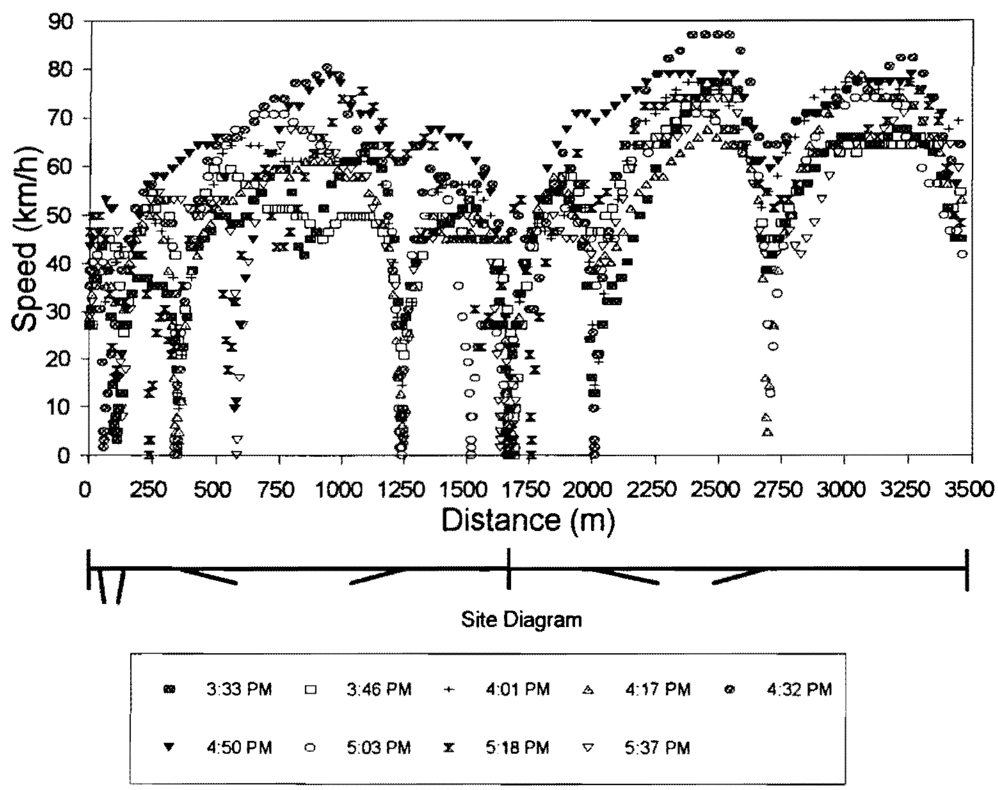


Figure D-9. Travel Time or Speed Versus Cumulative Distance for Site 25o.

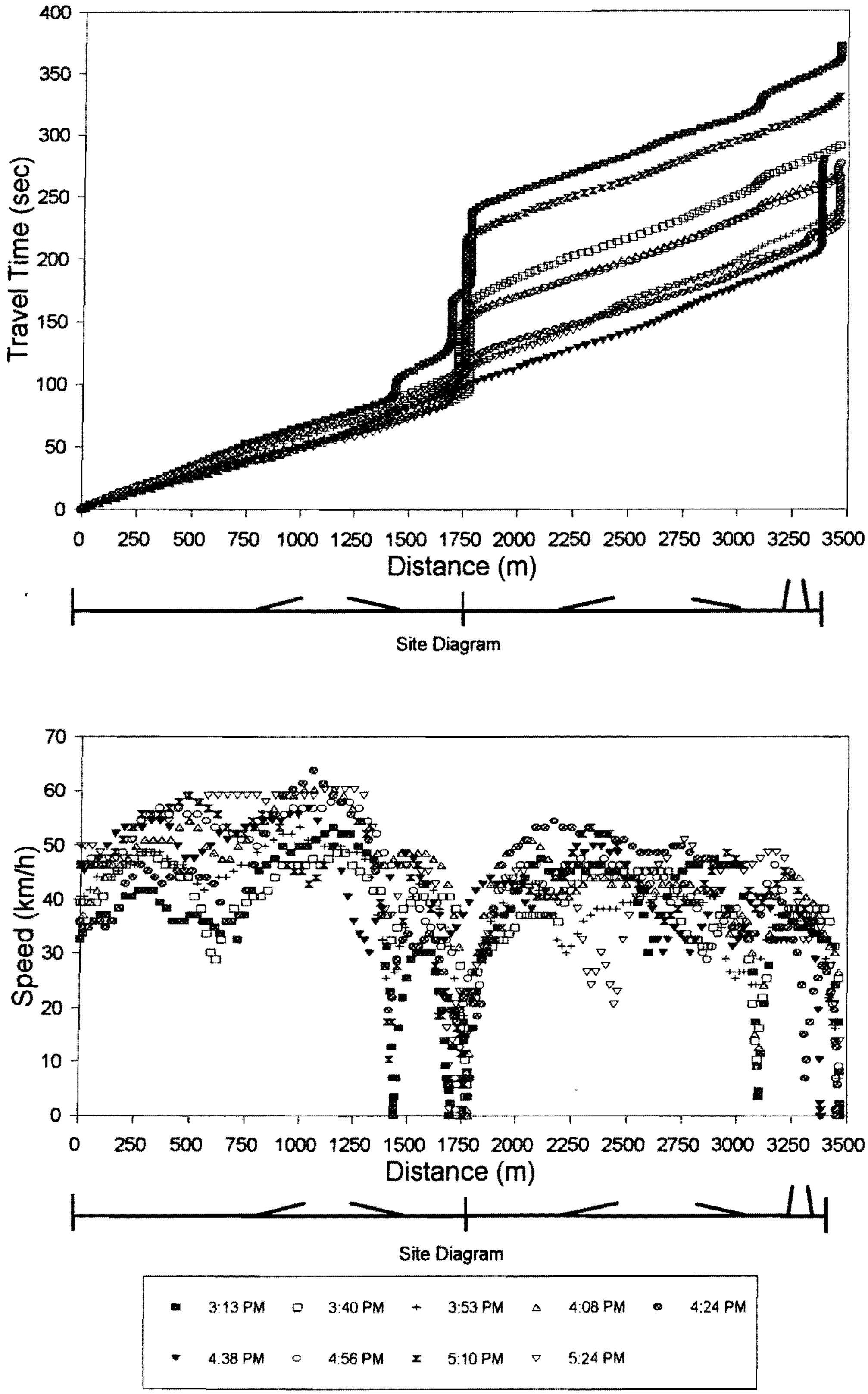


Figure D-10. Travel Time or Speed Versus Cumulative Distance for Site 25w.

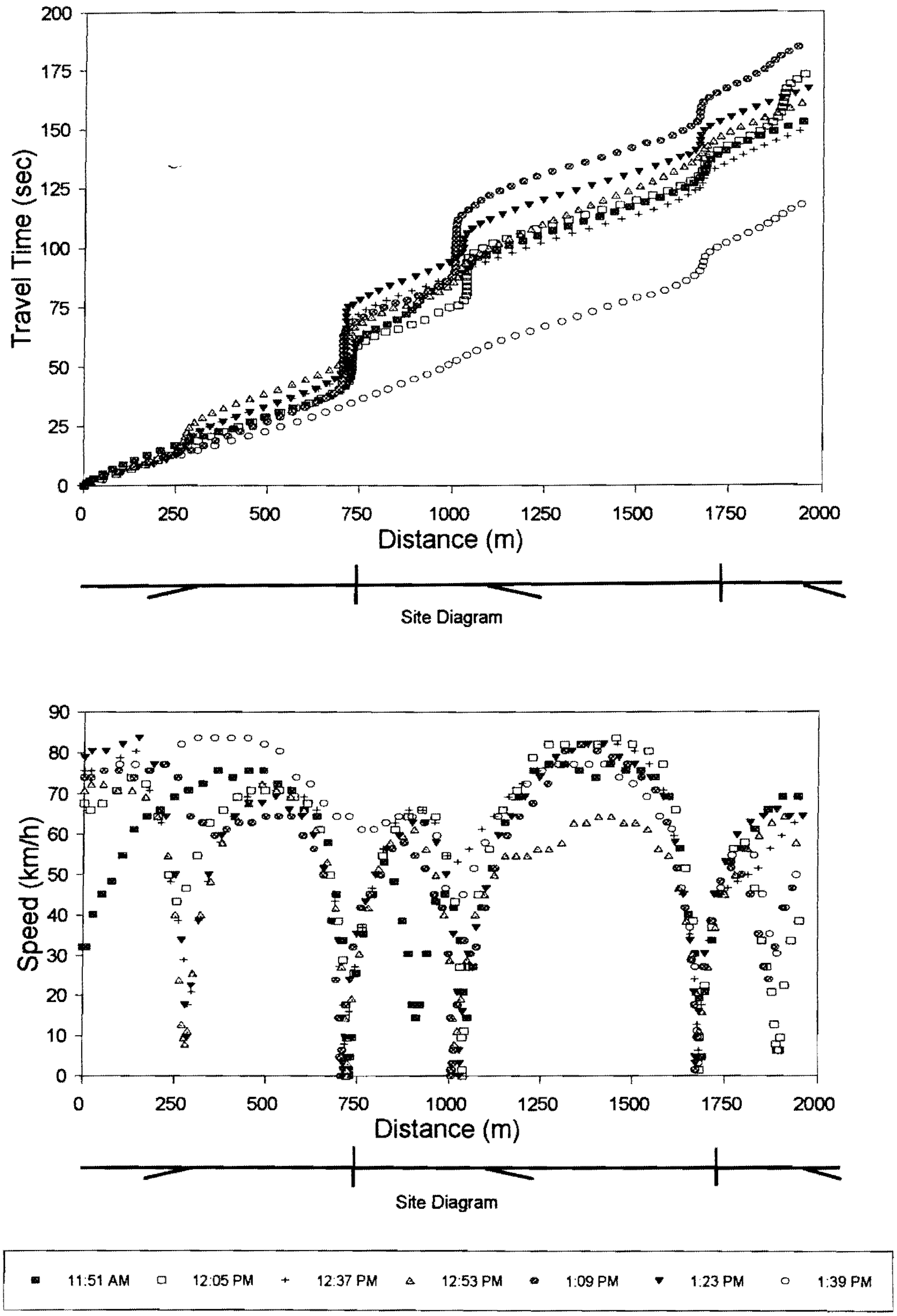


Figure D-11. Travel Time or Speed Versus Cumulative Distance for Site 26o.

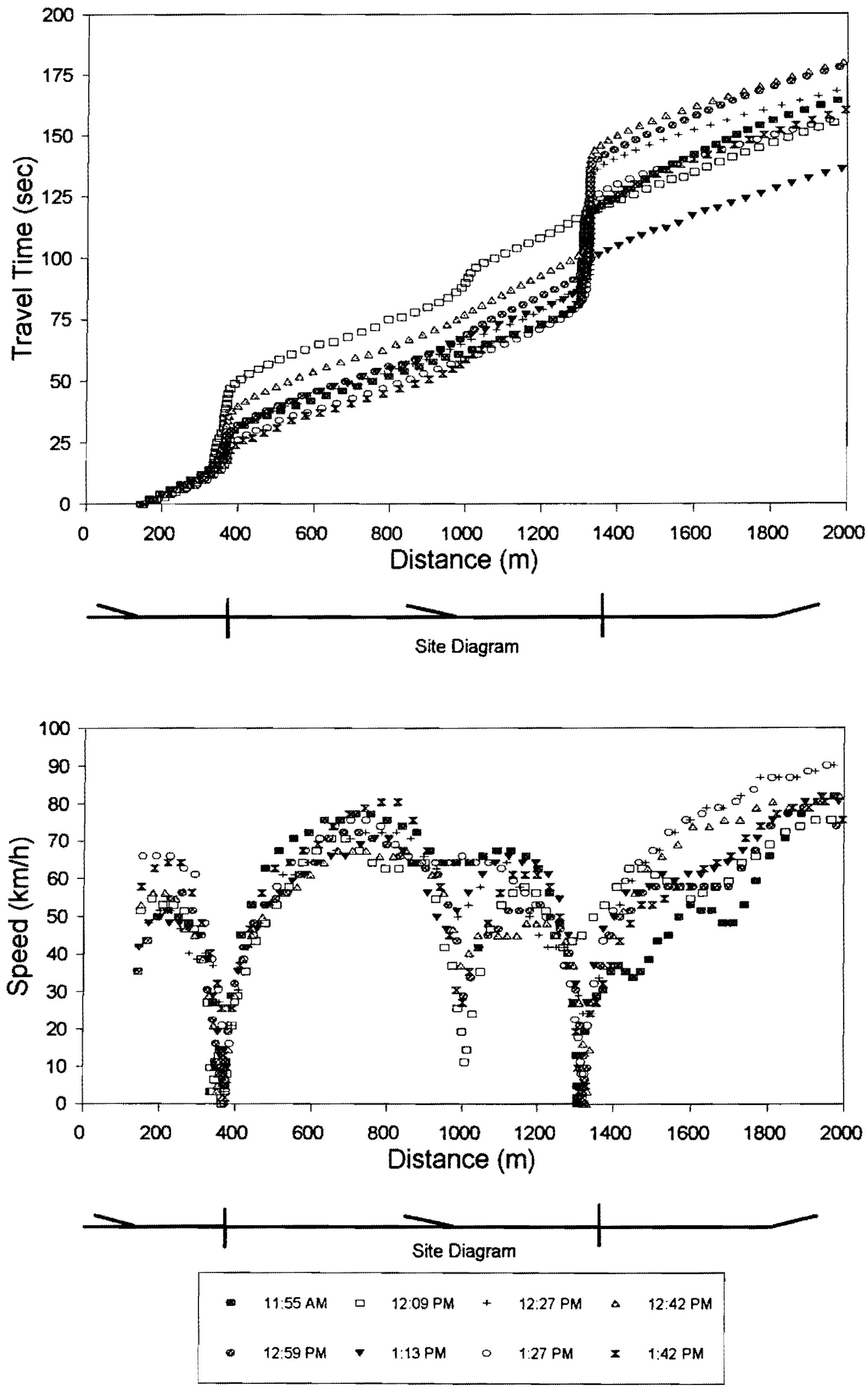


Figure D-12. Travel Time or Speed Versus Cumulative Distance for Site 26w.

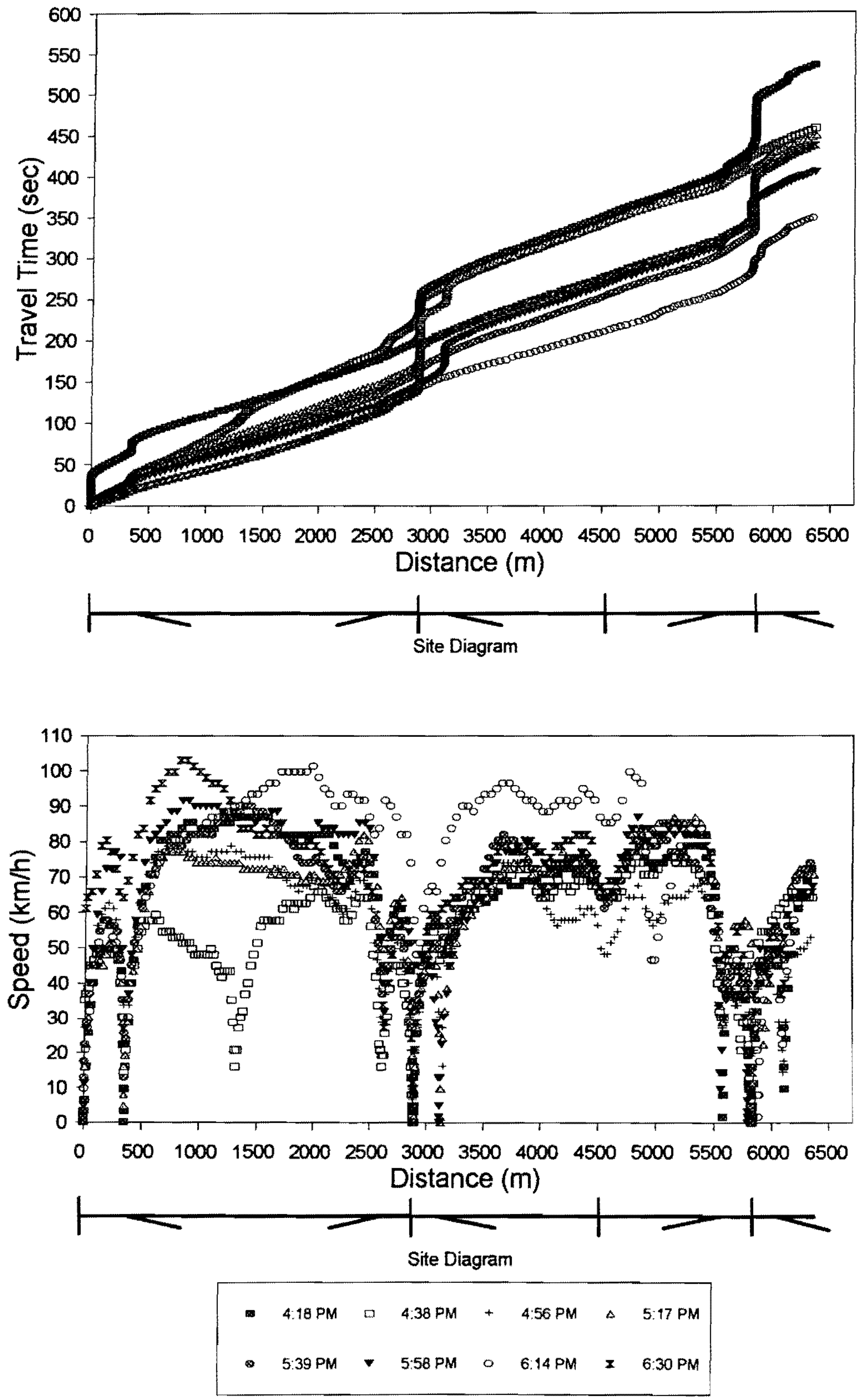
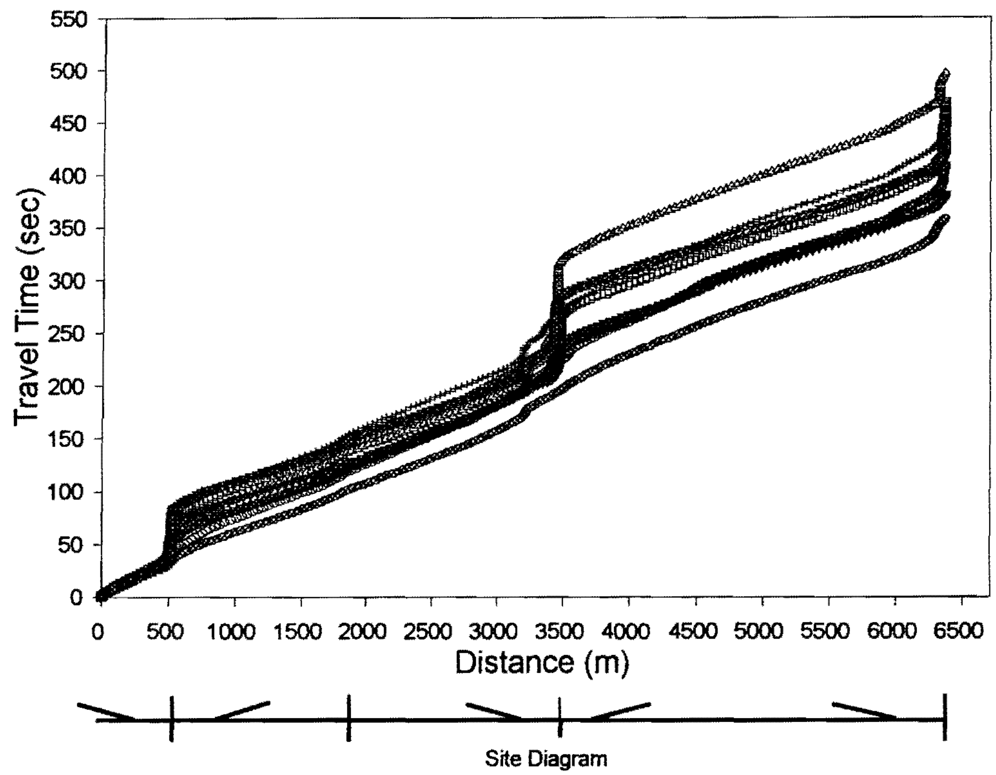


Figure D-13. Travel Time or Speed Versus Cumulative Distance for Site 27o.



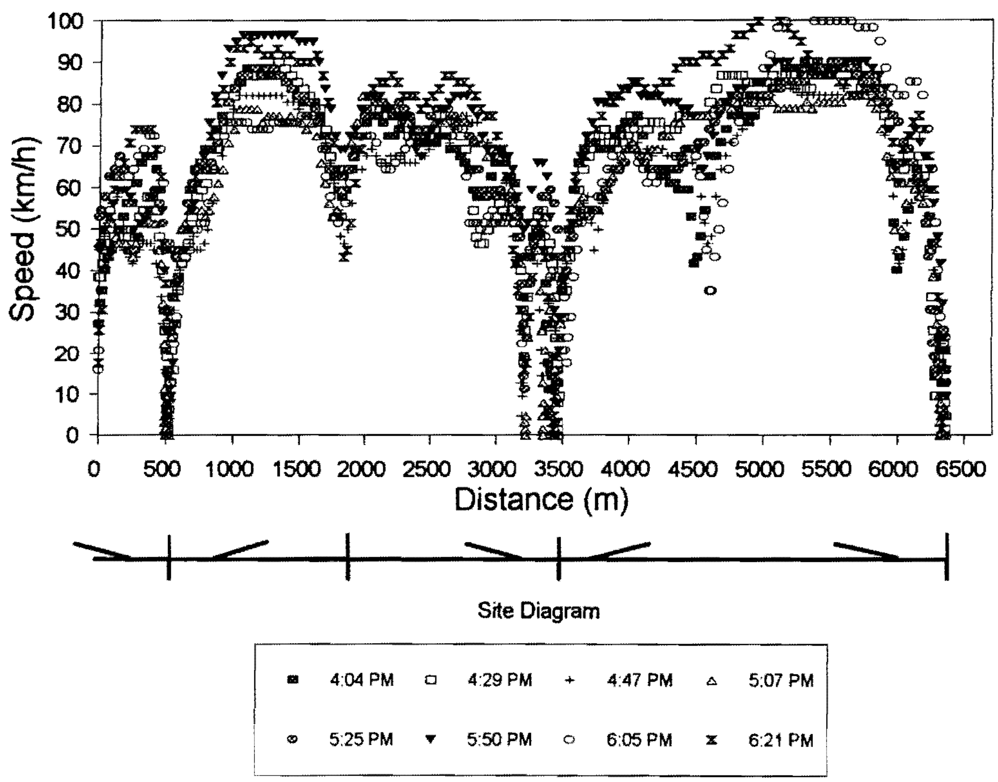
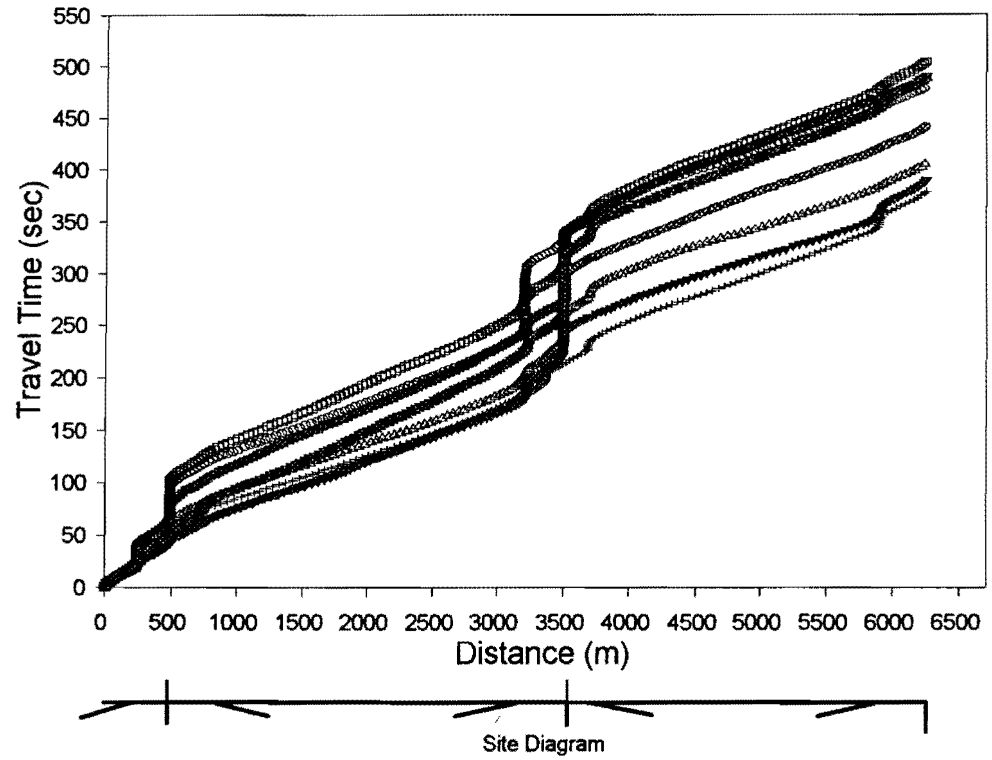


Figure D-14. Travel Time or Speed Versus Cumulative Distance for Site 27w.



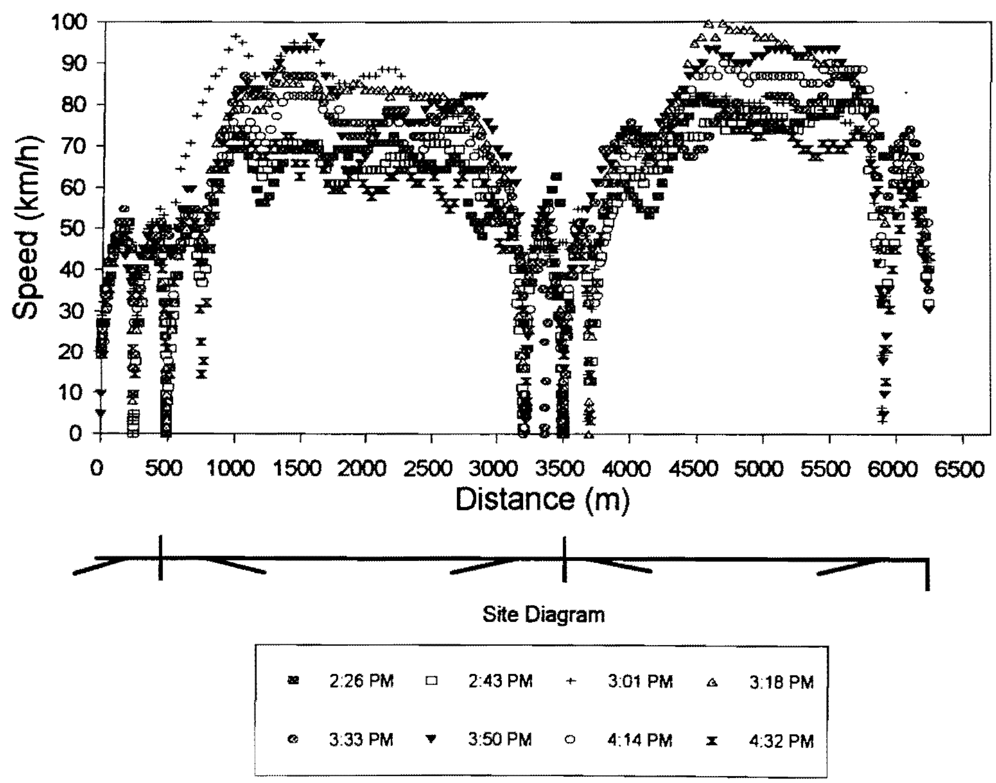
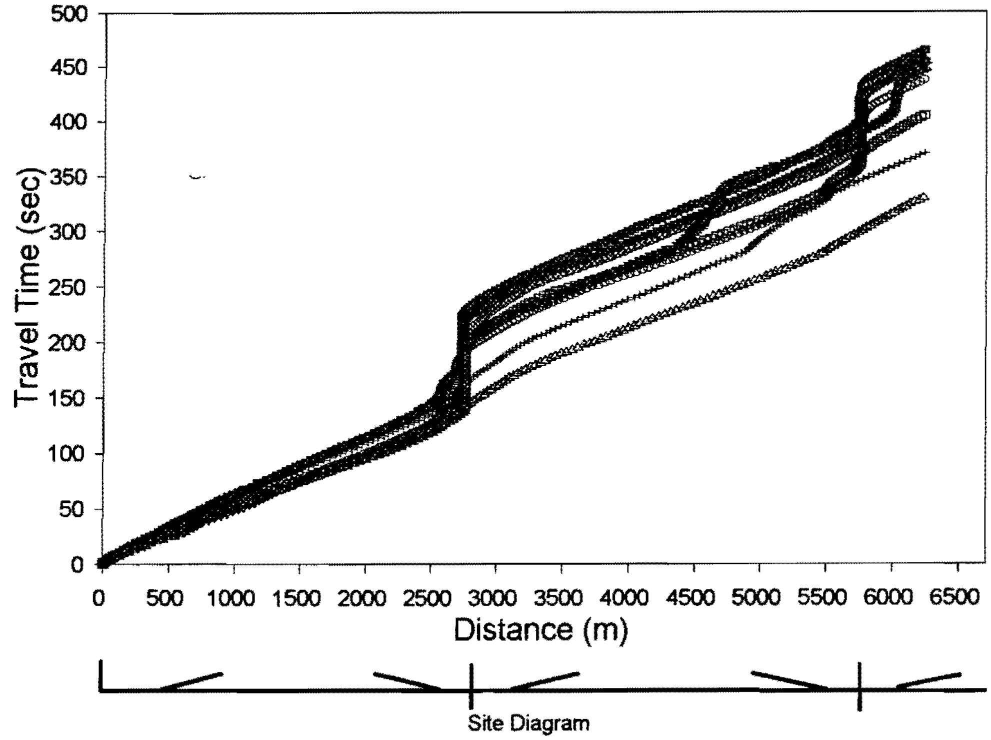


Figure D-15. Travel Time or Speed Versus Cumulative Distance for Site 28o.



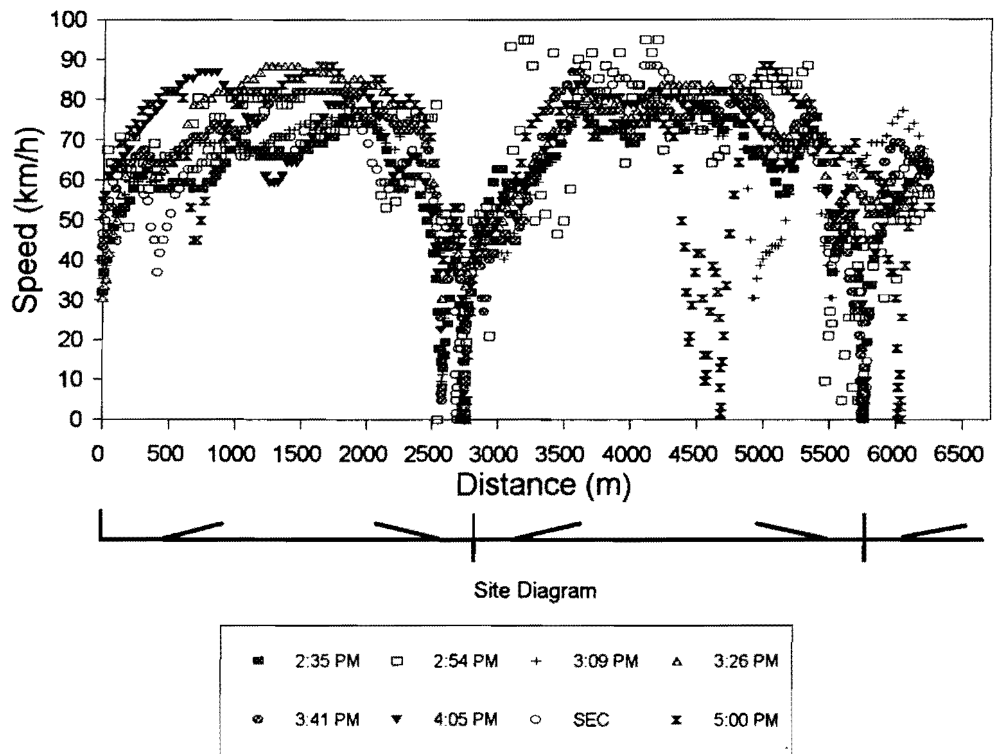
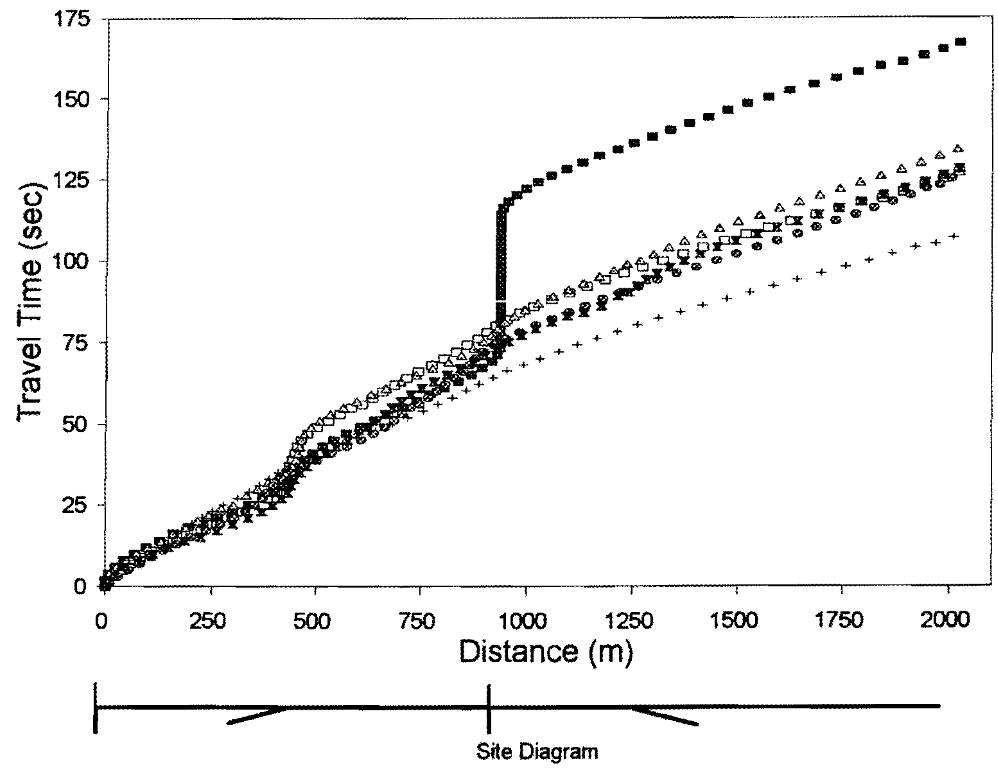


Figure D-16. Travel Time or Speed Versus Cumulative Distance for Site 28w.



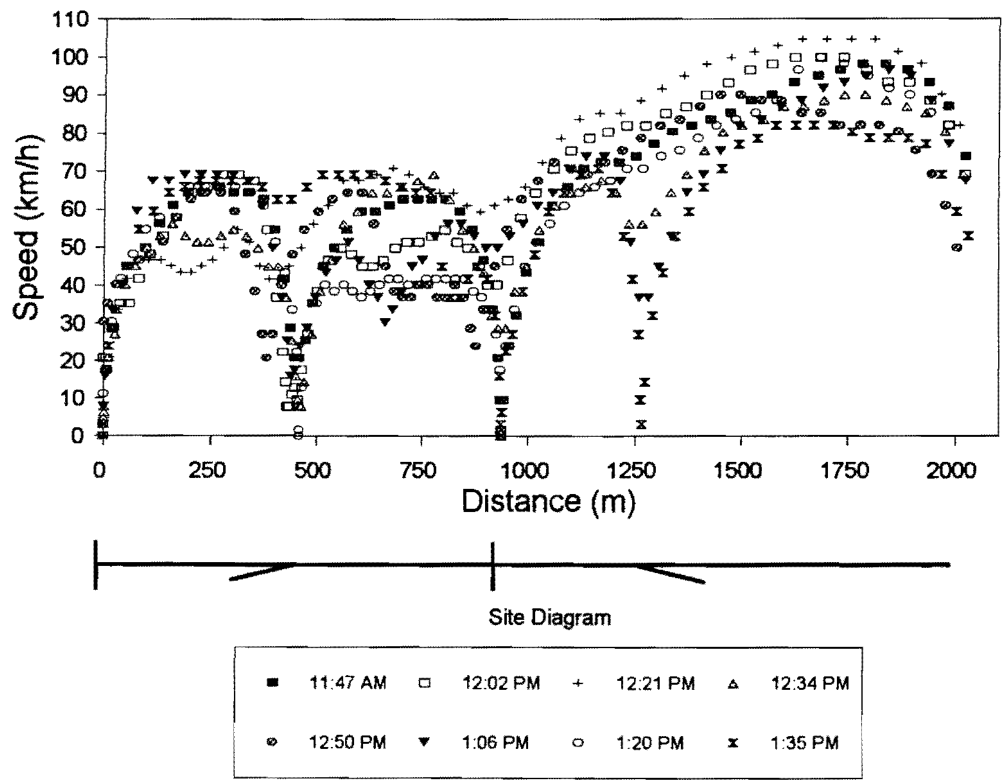


Figure D-17. Travel Time or Speed Versus Cumulative Distance for Site 29o.

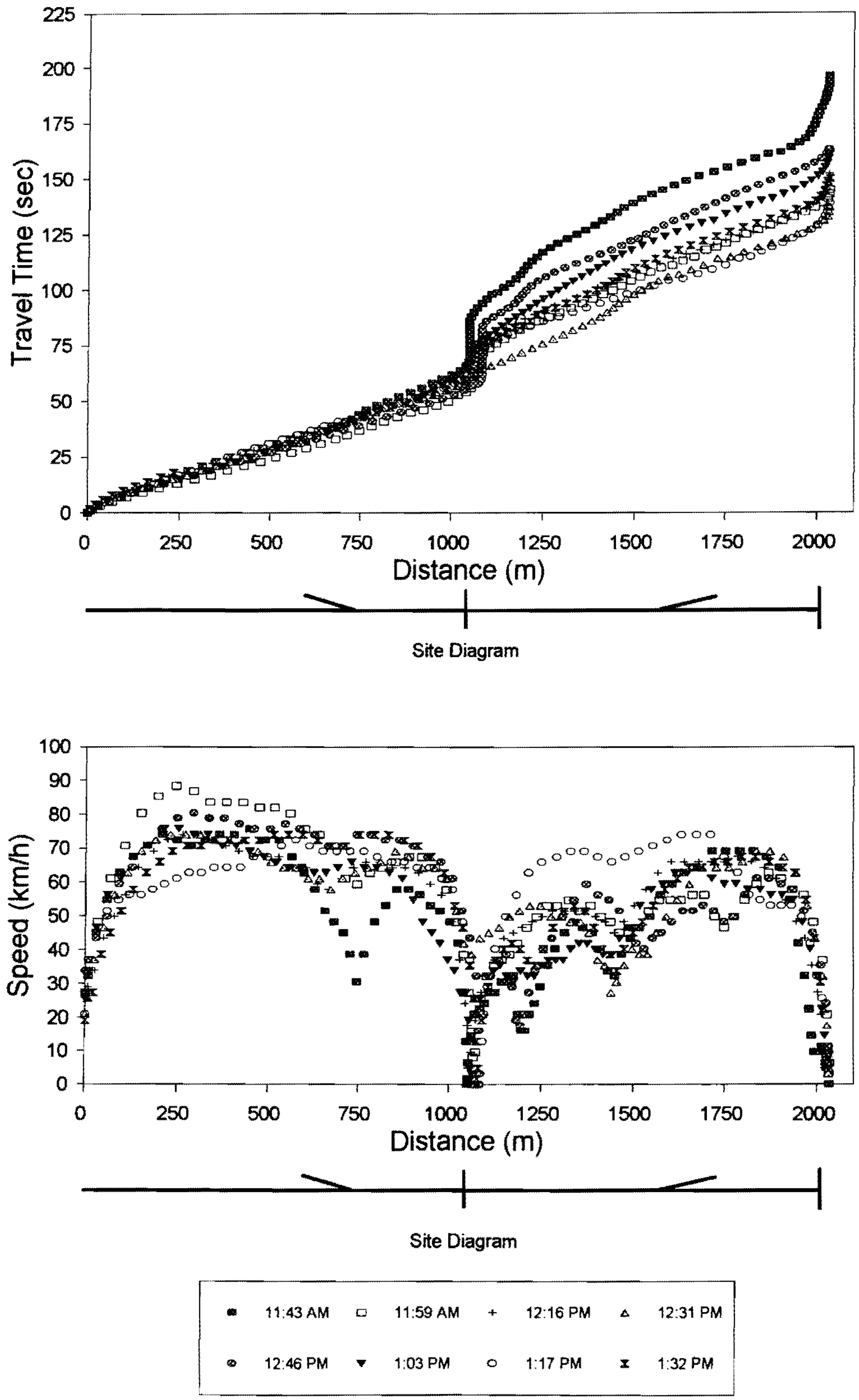


Figure D-18. Travel Time or Speed Versus Cumulative Distance for Site 29w.

APPENDIX E

FRONTAGE ROAD LEVEL-OF-SERVICE WORKSHEETS

Appendix E contains the worksheets to be used in the level-of-service procedure.

FRONTAGE ROAD LEVEL-OF-SERVICE WORKSHEET

Location: ______________________________ Direction: ______________________ - bound

Description: ____________________________ Type: _________________________

Date: _________________________________ Prepared By: ____________________

Segment	Segment Length (km) L	Access Density (acc/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]

[a] Use field data or values from Table 7-3
[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 7-9.

Sum of Travel Times, sec (ΣT) = ____________

Total Frontage Road Length, km (ΣL) = ____________

Average Frontage Road Speed, km/h = $3600\ (\Sigma L) / (\Sigma T)$ = ____________

Frontage Road LOS = ____________

SIGNALIZED INTERSECTION DELAY WORKSHEET

Location: ______________________ Direction: ______________________ - bound

Description: ______________________ Type: ______________________

Date: ______________________ Prepared By: ______________________

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]

[a] Table 7-4

[b] Equation 7-3 $d_1 = \frac{0.38C[1-(g/C)]^2}{1-(g/C)[Min(X,1.0)]}$

[c] Table 7-5

[d] Equation 7-4 $d_2 = 173X^2[(X-1)+\sqrt{(X-1)^2+mX/c}]$

[e] Equation 7-2 $d = d_1 DF + d_2$

[f] Equation 7-1 $D_I = 1.3 * d$

[g] Table 7-6

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 ≤ 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 F

FRONTAGE ROAD LEVEL-OF-SERVICE ANALYSIS

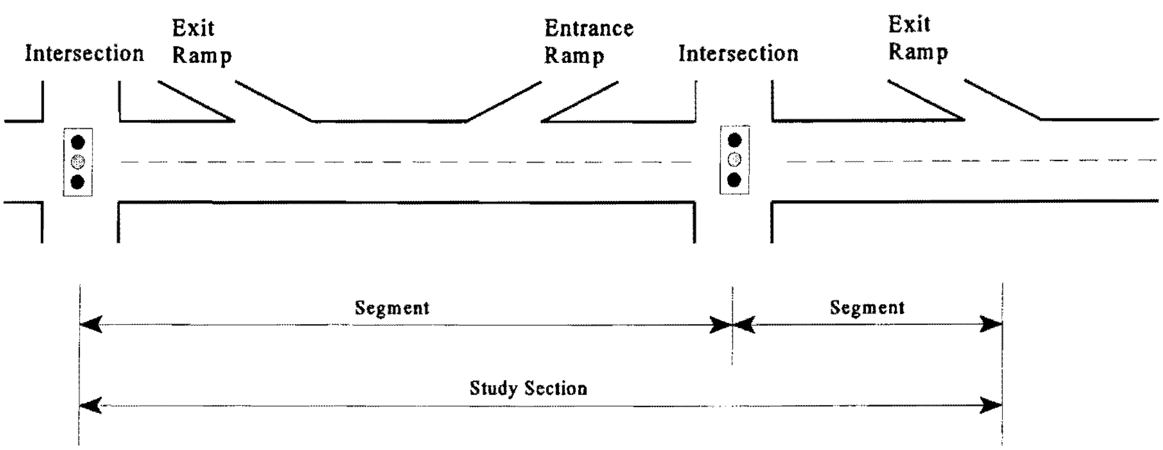
FLOW CHARTS

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

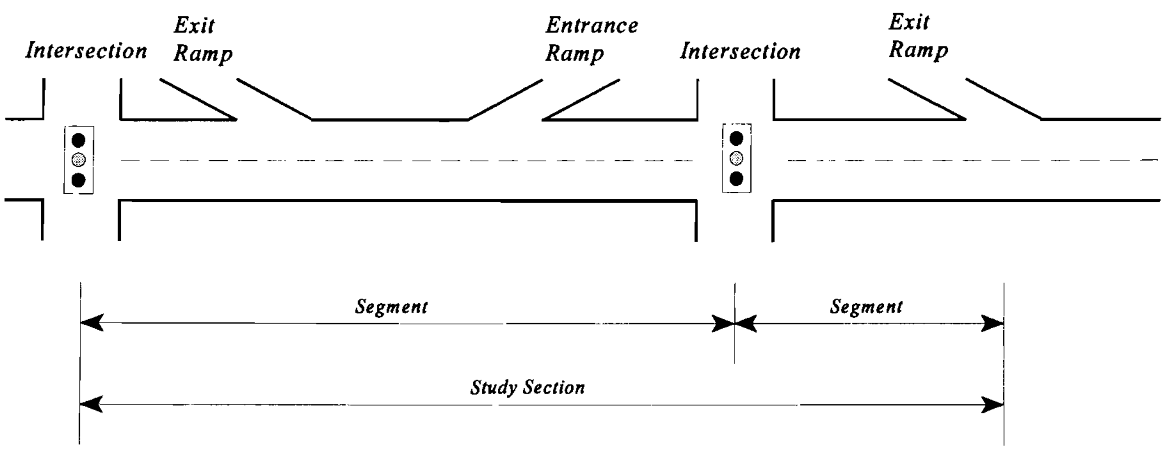
STEP 7 - ASSESS LEVEL OF SERVICE

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:

$$D_I = 1.3 * d$$

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

$$d = d_1 DF + d_2$$

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

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 G

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 G-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 G-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 7-4). 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 7-3. (*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 G-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 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.

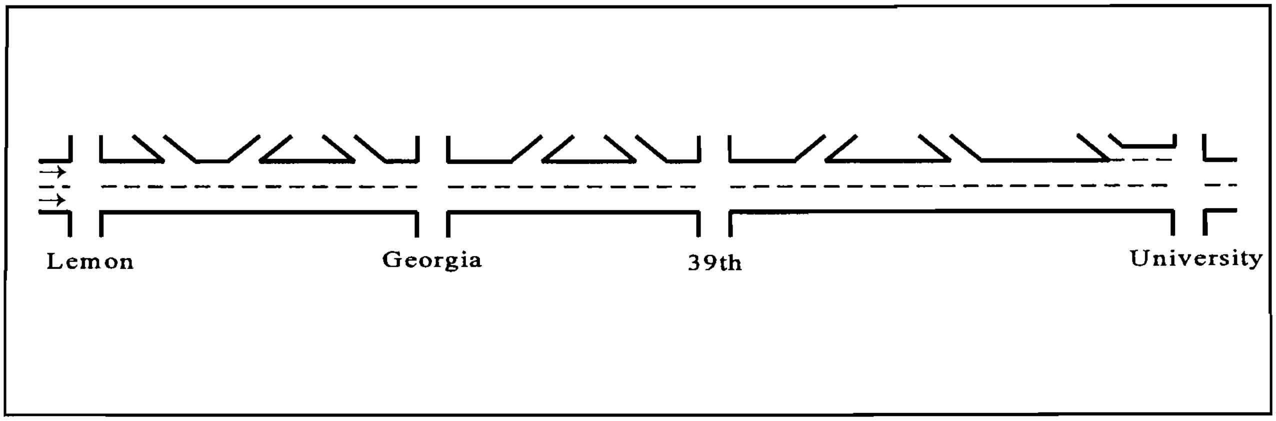


Figure C-1. Schematic of One-Way Frontage Road Study Section.

Step 2: Gather Field Data

The required roadway data (summarized in Table 7-1) are shown in Table G-2, while the traffic data are listed in Table G-3. Table G-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 G-2). Arterial Classification is entered as 1 because frontage road characteristics are similar to those of Arterial Classification 1.

Table G-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 G-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 G-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 G-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 G-3). However, these values can be adjusted for frontage roads by using the running time values in Table 7-3. The running times determined for frontage roads were similar to the assumed running times for arterials (see Figure 6-1). Therefore, adjustments are not required; use engineering judgement. The running times listed in Table G-5 are obtained from Table 7-3.

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 G-3. Compute Initial Running Time.

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

Segment	Boundaries	Length (mi / km)	Running Time from Table 7-3 (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 7-3 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 G-4). (The hints shown in Table G-1 provide information on calculating capacity and v/c.) Arrival Type is matched with the *HCM* arrival type definitions which are provided in Table 7-4. 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 G-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 G-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 G-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 G-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 G-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 G-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 7-3 plus the intersection delay and ramp delay values (see Figure G-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 G-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 G-8. Adjust Sum of Time.

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 screen (see Figure G-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 7-9) 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 G-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 G-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.

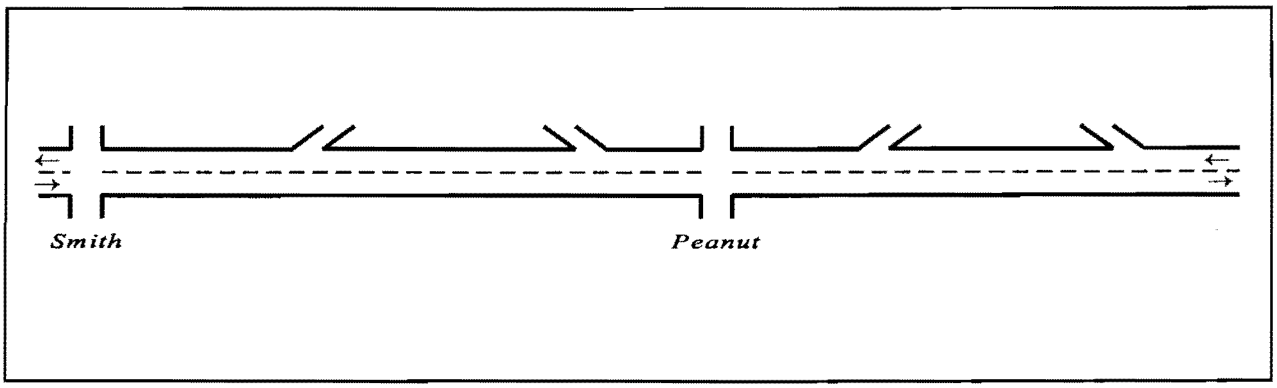


Figure G-9. Schematic of Two-Way Frontage Road Study Section.

Step 2: Gather Field Data

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

Table G-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 G-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 G-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 G-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 G-11). However, these values may be adjusted for frontage roads by using the running time values in Table 7-3. The running times determined for frontage roads were similar to the assumed running times for arterials (see Figure 6-1). Therefore, adjustments are not required; use engineering judgement. The running times listed in Table G-9 are obtained from Table 7-3.

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

Figure G-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 G-11. Compute Initial Running Time.

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

Segment	Intersection	Length (mi / km)	Running Time from Table 7-3 (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 7-3 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 G-12). (The hints shown in Table G-1 provide information on calculating capacity and v/c). Arrival Type is matched with the *HCM* arrival type definitions which are provided in Table 7-4. 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 G-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 G-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 G-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 G-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 G-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 G-15). The Sum of Time values can now be adjusted so that they equal the running time values from Table 7-3 plus the intersection delay and ramp delay values (see Figure G-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 G-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 7-9) 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 G-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 G-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 G-16. Adjust Sum of Time.