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16. Abstract

The Texas Department of Transportation (TxDOT) and many other agencies in the U.S. prefer to time their traffic signals to maximize through progression for arterial traffic. Because of this need, the PASSER™ series of traffic signal timing optimization software has become an important asset to these agencies. Recent access management analysis of several arterial roadways in Texas required the comparison of alternatives with different signal spacing, inevitably leaving some intersections unsignalized. Since all the members of the PASSER software family cannot presently analyze unsignalized intersections, it was necessary to use the Synchro® traffic optimization tool for analysis. However, as the Synchro tool is not designed to provide arterial progression, questions were raised about the usability of the results, since TxDOT would inevitably time signals along an arterial to achieve progressed flow.

The objective of this project is to enhance PASSER V to provide the capability to analyze the impacts of unsignalized intersections, including driveways, located on signalized arterials. In projects including traffic management improvements or access management improvements, different intersections are signalized or unsignalized to achieve various operational or safety improvement objectives. With the additional feature to analyze unsignalized intersections, PASSER V users are able to make side-by-side comparisons between alternatives for these projects.

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ENHANCEMENTS TO PASSER V SIGNAL TIMING OPTIMIZATION PROGRAM

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1. INTRODUCTION

BACKGROUND

The PASSER series of traffic signal optimization software [1, 2, 3, 4] has been an asset to the Texas Department of Transportation and traffic engineers around the country since the early 1980's. Historically used to develop new signal timings for a variety of traffic signal installations, PASSER programs have also been used to analyze intersection-related geometric improvements, predict the delay, fuel consumption, or air quality impacts of proposed improvements, or even analyze corridor-level traffic management issues such as access management. PASSER programs, including the recently developed PASSER V program [4], are the only available tools for timing traffic signals to maximize arterial progression. Because of this feature, many agencies prefer these tools over delay-based programs such as TRANSYT 7F [5] and Synchro [6]. However, because PASSER programs lack the ability to model unsignalized intersections, they cannot be used for many arterial assessment and improvement projects which require consideration of signalized and unsignalized intersections. For instance, recent access management analysis of several arterial roadways in both Laredo and San Antonio, Texas required the comparison of alternatives with different signal spacing, inevitably leaving some intersections unsignalized (where signal spacing was increased). Since PASSER software cannot presently analyze unsignalized intersections, it was necessary to use the Synchro traffic optimization tool for the analysis. However, as the Synchro tool is not designed to provide arterial progression, questions were raised about the usability of the results, since TxDOT would inevitably time signals along an arterial to achieve progressed flow.

This project enhanced PASSER V to address the need for tools that provide arterial progression for arterial assessment and improvements wherein different intersections are signalized or unsignalized. The ability to model unsignalized intersections in PASSER V software will significantly increase its utility for transportation agencies, especially TxDOT, that prefer progression bandwidth-based optimization of signal timings. PASSER V integrates arterial and interchange optimization and analysis technologies previously available separately in PASSER II and PASSER III. It also provides new technology previously unavailable in PASSER programs and has a graphic user interface that makes it very easy to use. Thus, by building new features into PASSER V to extend its capabilities to analyze unsignalized intersections, PASSER V will provide a more a comprehensive analytical tool for users both within and outside Texas.

PROJECT OBJECTIVES

The overall goal of this project is to enhance the PASSER V program to serve the needs of arterial assessment and improvement projects that require the analysis of arterials with signalized intersections and unsignalized intersections/driveways. In this project, the focus of unsignalized intersections is limited to two-way-stop-controlled (TWSC) intersections. All-way-stop-controlled (AWSC) intersections are excluded because the primary interest of many agencies is to provide arterial progression, which is not defined with the presence of AWSC intersections between signalized intersections.

To accomplish the project objective, the researchers outlined the following key tasks as part of this research effort:

1. Identify all existing models for assessing the capacity and performance of an unsignalized intersection and select the most appropriate model to be included in PASSER V.
2. Conduct field studies at several sites to determine the applicability of the model selected for unsignalized intersections and use field studies to improve these models.
3. Enhance PASSER V functionality to include the analysis of TWSC intersections. This added functionality enables the analysis of:
   a. isolated TWSC intersections and driveways; and
   b. signalized arterials with TWSC intersections and driveways.
4. Update PASSER V User Guide and training materials.

ORGANIZATION OF REPORT

This report is organized as follows. Chapter 2 provides background information on gap acceptance models and provides an overview of existing models for assessing capacity of TWSC intersections. Chapter 3 documents field studies conducted in this project; including a description of the process and procedures and study results. Chapter 4 describes work conducted to enhance PASSER V. Chapter 5 provides a summary of work conducted to test the enhanced software. Finally, Chapter 6 provides summary and conclusions.

2. GAP ACCEPTANCE AND CAPACITY MODELS FOR TWSC INTERSECTIONS

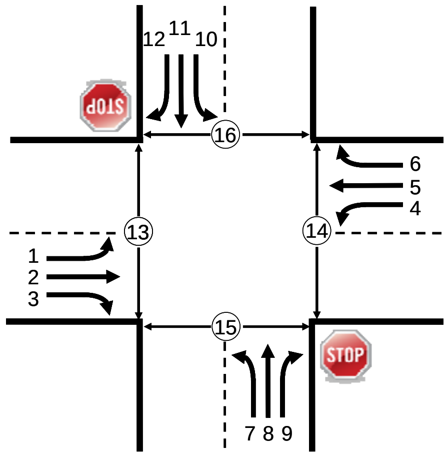
Two way stop control is frequently used at unsignalized intersections located on urban and suburban signalized arterials. In analyzing such intersections, intersection capacity is estimated first and used to derive other measures of performance. Though capacity analysis of TWSC intersections has received significant attention from the research community, available capacity analysis methodologies are far from perfect. This is partly because traffic operation at TWSC intersections depends heavily on the behavior of drivers, which often varies with traffic conditions. This dynamic makes it difficult to describe the process mathematically. The capacity analysis procedure adopted by the *Highway Capacity Manual 2000* (HCM 2000) [7], which is based on gap acceptance theory, is the most comprehensive and widely accepted methodology for the analysis of TWSC intersections at this time. This chapter describes the gap acceptance model used by HCM 2000, hereafter referred to as the HCM methodology, and discusses relevant literature and issues.

GAP ACCEPTANCE MODELS

Gap acceptance models have been widely used for modeling or explaining behavior of traffic at different types of roadway facilities. These types include; freeway merging and weaving, permitted left-turn operations at signalized intersections, and traffic movements through unsignalized intersections and traffic circles. These models assume that traffic streams can be categorized according to certain assigned or implied priorities or ranks for using the subject facility. It is further assumed that drivers in a lower-priority traffic stream will yield to traffic in all higher-priority streams and decide when gaps in those streams are large enough to provide for safe merge into or for crossing such higher-priority traffic streams.

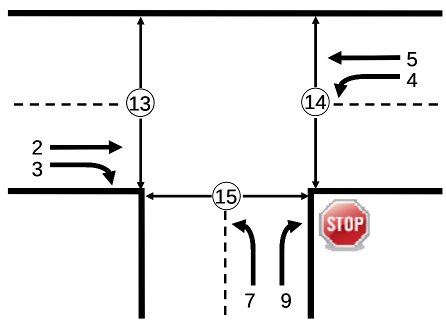
Figure 1 shows the priorities or ranks assumed for traffic movements (or streams) at a typical four-legged TWSC intersection. In this figure, the numbering of traffic movement follows HCM 2000 convention. At such a facility, minor-street traffic must yield the right-of-way to the major-street traffic. As such, the main-street through and right-turn movements (movements 2, 3, 5, and 6) are assumed to have absolute priority and are categorized as Rank 1 movements. Also regarded as Rank 1 movement are the pedestrian streams (movements 15 and 16) crossing the minor street approaches. Rank 2 movements include main-street left turns (movements 1 and 4), pedestrian streams across the main-street (movements 13 and 14), and cross-street right-turn movements (movements 9 and 12). These movements yield only to Rank 1 streams. Cross-street through movements (movements 8 and 11) must yield to all the main-street traffic, i.e., Ranks 1 and 2 movements, and are categorized as Rank 3 movements. Cross-street left turns (movements 7 and 10) have the lowest rank (Rank 4) because these movements must yield to all movements including opposing through and right-turn vehicles on the minor street.

A special case of TWSC intersection is the three-legged intersection where the cross-street approach is controlled by a stop sign. Since there is only one cross-street approach, the number of possible conflicting traffic streams is smaller than a four-legged TWSC intersection. The traffic streams and the corresponding priorities (ranks) at a T-intersection are shown in Figure 2.



Rank	Traffic Movement
1	2, 3, 5, 6, 15, 16
2	1, 4, 9, 12, 13, 14
3	8, 11
4	7, 10

Figure 1. Movement Priorities at a Four-Legged TWSC Intersection.



Rank	Traffic Movement
1	2, 3, 5, 15
2	4, 9, 13, 14
3	7

Figure 2. Movement Priorities at an Unsignalized T-Intersection.

Gap acceptance theory assumes that all traffic movements will move according to the stated priority rankings, even though observations have shown that drivers often violate these rankings. It also assumes that drivers are consistent and that the driver population is homogeneous. A driver would be considered consistent if he/she always behaves the same way when encountering the same or similar situation. For instance, a consistent driver will not reject a gap and then utilize a shorter gap for the same maneuver. The reader will agree that this is not always the case. The homogeneity assumption implies that all drivers in the population behave exactly the same way under the same or similar situations. This assumption is also weak. Nonetheless, these assumptions enable the development of a simple macroscopic model that practitioners can use to analyze otherwise very complex situations. Under these assumptions, the gap acceptance model attempts to capture the following decision process for a driver at a minor movement:

1. the higher priority streams have large enough gaps that would allow a safe maneuver, and
2. it is his/her turn to use the gap.

If both (1) and (2) are satisfied, the driver will accept the gap, enter the intersection, and complete the maneuver. To capture this process for TWSC intersections, gap acceptance models rely on two key parameters for each minor movement. These parameters, namely critical headway and follow-up time, are described next.

CRITICAL HEADWAY

Headway, measured in seconds, is the time between two successive vehicles as they pass a point on the roadway. It is measured using the same feature of both vehicles (i.e., front bumper, front axle, rear bumper, etc.). Though a minor-street driver at a TWSC intersection evaluates gap, which is headway minus vehicle length, for maneuver decision, headway is much easier to observe in the data collection process. Also, in applying gap acceptance theory, the difference between gaps and headways are generally ignored [8]. Thus, the critical headway is defined as the minimum time between successive major-stream vehicles in which a minor-movement vehicle can make a safe maneuver. In other words, no minor-movement vehicle will be serviced unless the headway is equal to or longer than the critical headway.

In reality, critical headway differs from driver to driver, and even varies for the same driver depending upon traffic conditions. For instance, drivers are willing to accept smaller than normal headways as waiting times increase during peak traffic hours. Other factors that affect critical headway include: type of minor movement (i.e., cross-street left turn), number of lanes on the main street, and visibility. However, as stated previously, the use of gap acceptance model assumes that drivers are consistent and homogeneous. Thus, a representative value of critical headway must be obtained. It should be noted that critical headway for a minor movement cannot be directly observed from field studies. It must be derived using distributions of accepted and rejected headways observed in the field. Literature contains several methods for deriving the critical headway. Some simple-to-use procedures, in addition to the maximum likelihood method, which is used by HCM methodology, are described in the following subsection.

Siegloch's Method

Siegloch's method [9] is simple and easy to use provided that there is a continuous queue on the minor-movement stream under investigation. We summarized this procedure as follows:

Step 1: Record the size of each headway, t, and the number of vehicles, n, that enter the intersection during this headway.
Step 2: Calculate the average headways that were accepted by each n observed, $E(t \mid n)$.
Step 3: Find the linear regression on the average headways against n.
Step 4: Given the intercept (t_0) and the slope (t_f) of the regression line, critical headway, t_c is estimated as $t_c = t_0 + t_f / 2$.

Though Siegloch's method is simple to use, it can only apply when a queue is continuously present at the subject movement during the data collection period. As such, this method is deemed useless for undersaturated traffic conditions.

Raff's Method

This method [10] defines critical headway as the headway for which the number of accepted headways that are shorter than it is equal to the number of rejected headways longer than it. In other words, critical headway is located at the intersection of two curves: cumulative number of accepted headways and cumulative number of rejected headways. Figure 3 demonstrates an example of employing Raff's method. In this example, the blue line (descending from left to right as the headway value increases) is the cumulative curve of rejected headways and the purple line is the cumulative curve of accepted headways.

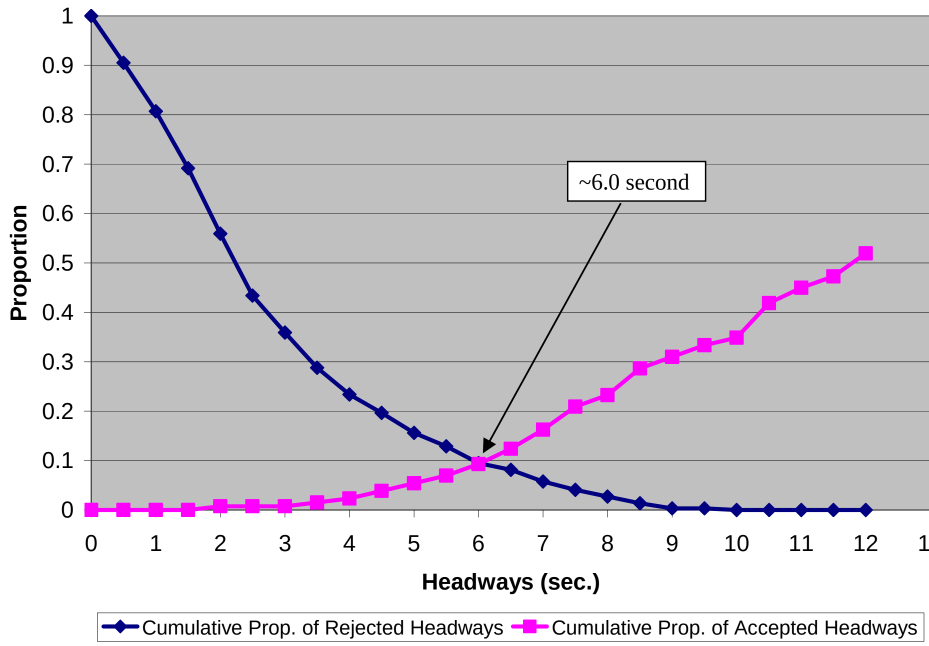


Figure 3. Raff's Method of Estimating Critical Headway.

Maximum Likelihood Method

Troutbeck [11] provided a maximum likelihood method for estimating the critical headway that can be applied to both undersaturated and saturated conditions. Let r_i and a_i be the largest rejected headway and accepted headway of driver i. Realizing that critical headway of driver i must be greater than r_i and smaller than a_i, and assuming that headway follows certain probabilistic distribution, $F(t)$, this method estimates the likelihood of driver i's critical headway is between r_i and a_i as $F(a_i)$- $F(r_i)$. In general, log-normal distribution is used as the distribution of the headways. Then, the likelihood of a sample of n drivers is:

$$L = \prod_{i=1}^{n} \left(F(a_i) - F(r_i) \right)$$

Next, mean (μ) and variance (σ^2) of the distribution are found such that the above equation is maximized. In practice, the logarithm of L is used. By setting the partial derivatives of the logarithm of L with respect to the distribution parameters to zero, mean and variance can be found by iterative numerical method. Finally, critical headway can be computed as:

$$t_c = \exp\left(\mu + 0.5\sigma^2\right)$$

Other Methods

Another simple estimation method is based on the study by Jessen [12] in which he assumed that there exists a fixed linear relationship between critical headway and follow-up time. Based on field studies, this relationship was described as follows.

$$\textit{Critical Headway} = \textit{Follow-up Time} \div 0.6$$

The definition of follow-up time and its field measurement are discussed in the next subsection. In addition to the methods mentioned above, researchers have proposed other more complex models for estimating critical headway; including logit model [13, 14], which is a weighted least square linear regression model, and probit model [15, 16, 17], which is a specification of generalized linear model in which the dependent variable can only be one or zero.

FOLLOW-UP TIME

Follow-up time applies to multiple minor movement vehicles using the same gap. Kyte et al. [18] define the follow-up time of a movement as "the time span between the departure of one vehicle from the minor stream and the departure of the next, under a condition of continuous queuing." This parameter is analogous to the lost time at signalized intersections. Like critical headway, follow-up time also varies from driver to driver and even for the same driver under different conditions, however, the consistency and homogeneity assumptions are needed for model development.

As opposed to critical headway, however, follow-up time can be directly measured in the field. The only implied requirement for obtaining this parameter from field observations is the presence of a queue. Average follow-up time must be obtained using a large enough sample size.

CAPACITY ESTIMATION

HCM Methodology

The traffic operations at TWSC intersections are complicated, mainly due to the lack of clear indication given to the drivers of minor movements and different combinations of geometric characteristics. For analysis purposes, HCM methodology assumes the ranking order described earlier. Thus, this methodology assumes that the main street through (Rank 1) movements have absolute priority and their capacity is equal to the saturation flow rate. To estimate the capacity of each minor-movement, HCM methodology suggests the following steps:

Step 1: Calculate potential capacity of each movement assuming that such movements are serviced by exclusive lanes.
Step 2: Adjust potential capacity for effects due to impedance, two-stage gap acceptance process, and upstream signals.
Step 3: Make appropriate adjustments to capacity calculations in cases where multiple movements share a lane.
Step 4: Adjust movement capacity for flared minor-street approaches.

The following sections provide further information about potential capacity, impedance, two-stage gap acceptance, and effects due to upstream traffic signals.

Potential Capacity

Potential capacity is defined assuming that:

- the TWSC intersection is not blocked by the major street traffic;
- each minor-stream movement is serviced by an exclusive lane;
- traffic on major street arrives randomly; and
- no other movement of Rank 2, 3, or 4 impede the subject movement.

In other words, potential capacity defines the potential traffic volume that can depart from the stop line for a minor stream. Calculation of potential capacity of a movement requires total conflicting flow rate, critical headway, and follow-up time for the subject movement as an input. The process accounts for the presence of heavy vehicles, approach grade, and number of legs at the intersection through appropriate adjustments to critical headway and follow-up time.

Traffic on a lower-priority movement must yield to all traffic on conflicting movements with higher priority. Thus, its potential capacity is constrained by all higher-priority conflicting volume. Generally, the impact of each higher-priority conflicting movement on a lower priority movement is different depending upon its movement type (i.e. main-street through or right turn) and geometrics. Therefore, HCM suggests that conflicting flow for a minor movement be calculated as a weighted-sum of its conflicting higher-priority movement flow rate as illustrated in Exhibit 17-4 of HCM 2000 [7].

In addition, HCM 2000 specifically addresses the treatment for a channelized right-turn movement when considering conflicting flow. For a right-turn movement to qualify as channelized, the right-turn movement must be separated by a triangular island and has to comply with a yield or stop sign. As shown in Figure 4, channelization of a right-turn movement (in this case, northbound right) increases the capacity of opposite left-turn movement (in this case, southbound left-turn). HCM methodology accommodates the effects of channelization by removing the appropriate right-turn volume from the analysis.

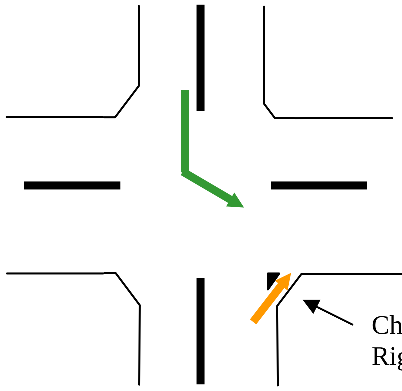


Figure 4. Channelized Right-Turn Movement.

Capacity Adjustment

As described above, potential capacity calculation is based on several assumptions, and accounts for heavy vehicles, grade, and number of approaches or legs. However, several geometric characteristics may significantly affect the capacity of minor movements. Thus, estimation of actual capacity using HCM methodology requires adjustments for additional applicable factors, including impedance, two-stage gap acceptance process, upstream signals, flared minor-street approaches, and shared-lane.

Impedance Effect. Using field data from different locations, Kyte et al. [18] verified that a higher-priority movement has additional effects on the conflicting movement with lower priority besides being part of the conflicting flow. As such, lower rank movement can only utilize the facility when the higher rank conflicting movements are queue-free at the intersection. This effect is referred to as impedance effect and is due to congestion of the higher-priority movements. To accommodate this effect, HCM methodology assesses the probability that a movement will be operating in a queue-free state and adjusts the capacities of impeded movements accordingly.

In addition to vehicle-induced impedance, pedestrians crossing the streets can also obstruct conflicting traffic streams to and from the minor streets. If there are a significant number of pedestrians, capacity calculations should account for resulting impedance.

Two-Stage Gap Acceptance. When median storage is available, minor-street left-turn and through movement may cross the TWSC intersection in two distinct stages by crossing one major stream at a time. This process is referred to as two-stage gap acceptance. The capacity of this two-stage process depends on the number of vehicles that can store in the median. Figure 5 shows a facility with a storage space of two vehicles in the median. In such a case, vehicles on the higher priority movements (i.e., eastbound left-turn) use this space first. Any available space is used by the cross street vehicles to complete the first stage of the two-stage gap acceptance. It should be noted that a two-way left-turn lane (TWLTL) may provide storage space for more vehicles.

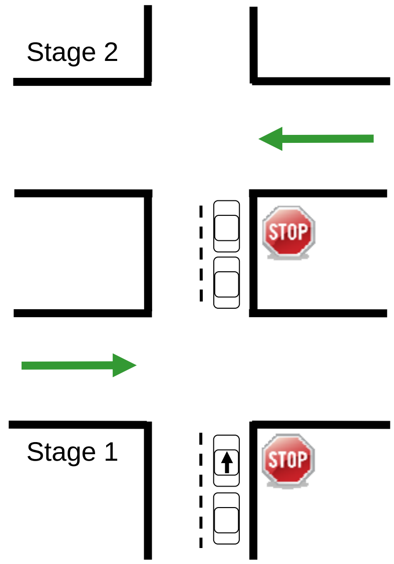


Figure 5. Two-Stage Gap Acceptance.

In this case, HCM methodology calculates the capacity for each stage separately by taking into account conflicting flow for each stage as described below:

- conflicting flow for Stage 1 is the main-street traffic from the left side,
- conflicting flow for Stage 2 of cross-street through traffic is the main-street traffic from the right-side, and
- conflicting flow for Stage 2 of cross-street left turns is the main street traffic from the right side and the opposing through and right-turn traffic.

Under this scenario, cross-street vehicles will not necessarily use a two-stage gap acceptance. There may be some drivers who will get and utilize opportunities for making the maneuver in a single stage. HCM methodology includes a procedure to account for this effect.

Upstream Signals. The presence of upstream signals will also have an impact on the operations of TWSC intersections. For example, if the majority of vehicles arriving from an upstream signal are in a compact platoon, longer headways will be available for minor movements after the platoon has crossed the intersection. Using a platoon dispersion model, the HCM methodology takes this phenomenon into consideration by assessing the probabilities of a TWSC intersection being blocked by any platoons from each direction.

If only one major approach has an upstream signal, minor movements will encounter two distinct flow profiles, namely; flow when there is no platoon (unblocked period) and flow when platoon is passing through the intersection (blocked period). However, when upstream signals exist on both sides of the TWSC intersection, vehicles on cross-street movements may face one of the following four conditions:

- no platoon,
- platoon from the left side only,

- platoon from the right side only, and
- platoons from both sides.

The joint platoon arrival patterns created by two traffic signals may be extremely complicated depending on a number of factors. For simplicity, HCM methodology incorporates the effects of each upstream signal separately and then applies additional adjustment to arrive at the total proportion of blocked and unblocked times.

Flared Minor-Street Approaches. As shown in Figure 6, when a flared approach is present, the capacity of a shared right-turn lane will increase because the extra storage space allows some of the right-turn vehicles to queue at the stop line and complete the movement without obstructing or being obstructed by other movements in the shared-lane. The increase in capacity depends on storage spaces (in terms of passenger vehicles) in the flared area and the average queue length for each movement in the shared lane. In general, longer usable storage spaces increase capacity of the shared lane. Similarly, the longer the queue length, the smaller the increase in resulting capacity.

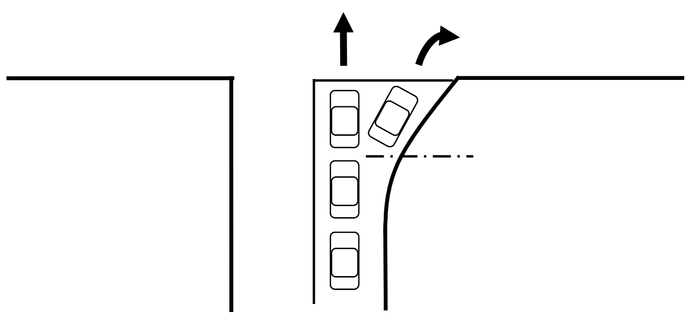


Figure 6. Flared Minor-Street Approach.

Shared-Lane Capacity. When several movements are serviced by the same lane and flared minor-street approach is not present, the lane capacity in vehicles per hour (vph) is adjusted with respect to the movements sharing the lane. HCM methodology uses the following equation to compute shared-lane capacity.

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \left(v_y / c_{m,y} \right)}$$

where: c_{SH} : capacity of the shared lane (vph)

v_y : flow rate of movement y in the subject shared lane (vph)

$c_{m,y}$: movement capacity of movement y in the subject shared lane (vph)

Limitations of HCM Methodology

While HCM methodology provides the most comprehensive approach for estimating capacities at TWSC intersections, nonetheless, it has several limitations that are worth mentioning.

First, HCM methodology treats the movements hierarchically. It begins with potential capacity and subsequently adjusts this value to entertain other factors. While this sequential and structural methodology is straightforward, it is not holistic. In other words, it does not consider joint effects of all factors simultaneously.

Second, while HCM methodology takes into account the platoon effect of the upstream signals, it fails to consider the blocking effect due to queues at the downstream signals. This deficiency may result in gross overestimation of capacity for TWSC intersections located in close proximity to traffic signals.

Third, HCM methodology only considers two-lane and four-lane streets with and without two-way left-turn lanes. It provides no data (critical headway or follow-up time) or guidance for wider arterials. Furthermore, it does not explicitly address different number of lanes (i.e., one lane on one direction and two lanes on the other) on opposing arterial approaches.

Fourth, HCM 2000 completely ignores treatment of U-turn movements, which may have significant impact on the quality of intersection operation.

Fifth, the present HCM methodology recommends the addition of pedestrian and traffic volumes to derive conflicting flows for certain movements. We strongly believe that this recommendation has a serious flaw analogous to adding apples and oranges. Additional research is needed to address this flaw.

Sixth, HCM methodology calculates conflicting flow for a minor movement as a weighted-sum of conflicting flows at higher-priority movements. However, the research that the HCM methodology is based on (Kyte et al. [18]) only verified weighting factors for the main-street through and right movements. Furthermore, we failed to find any theoretic background to support the use of the suggested weighting factors.

Additive Conflict Flows Methodology

A team of researchers in Germany developed the additive conflict flows procedure for determining the capacities at TWSC intersections based on graph theory [19, 20, 21]. The fundamental building block of this approach is the idea of a conflict group. A conflict arises if several movements must use the same area within an intersection. All such movements will constitute a conflict group. Figure 7 illustrates a conflict group with three movements.

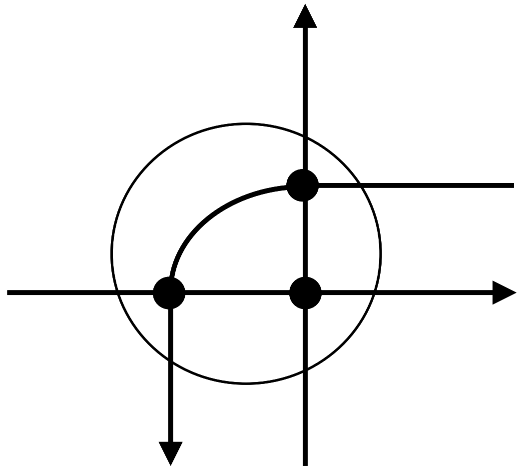


Figure 7. Conflict Group of Three Movements.

In addition to the identification of conflict groups a movement belongs to, three parameters are needed to apply this methodology to estimate the capacity of that movement. These parameters are traffic volumes of all movements in the conflict group, occupation time of the conflict area by each movement, and movement priorities. Occupation time of a specific movement is the average travel time of the subject movement vehicle through the conflict area. Assuming that the occupation time of a movement, *i*, is the same in each conflict area, the proportion of time movement *i* occupies the conflict area can be determined from movement *i*'s traffic volumes and occupation time. Thus, the proportion of time a conflict area is free from higher-priority movement can be estimated. Realizing that a movement can be serviced only when all the conflict groups to which the subject movement belongs are not used by higher-priority movements, the German researchers presented a closed-form equation for estimating the capacity of the subject movement.

The major advantage of this method is that the capacity of any movement can be expressed as a closed-form function of traffic volumes and occupation times of all conflicting movements of higher-priority. In addition, this method is flexible in that pedestrians or even bicyclists using the TWSC intersections can be modeled easily. By introducing a conflict matrix in which the element of row *i*, column *j*, $A(i,j)$, specifies the degree to which a conflicting movement *i* has priority over movement *j*, Brilon and Miltner [21] provided a mechanism to model the degree of compliance to the priority using the same graph theory-based method.

Though this method seems to be promising, empirical verifications of its validity are still needed. Also, research to date has not identified how to model the impact of platoons from upstream traffic signals. Thus, this method has not matured enough to be useful for the purpose of this project.

U-TURNS AT TWSC INTERSECTIONS

Safety concerns for drivers at TWSC intersections often result in the prohibition of through and left-turn movements from minor streets. Such restrictions, coupled with lack of alternates, often result in significant increases in U-turn traffic at adjacent downstream intersections. This change is caused by minor-street left-turn and through vehicles being forced to turn right and then turn around at the next intersection (or median opening) to reach their destinations. However, HCM 2000 is completely silent on how to accommodate U-turns at unsignalized or signalized intersections. Furthermore, a literature search found very little guidance on this subject.

Al-Masaeid [22] studied seven median openings located in Jordan and estimated U-turn capacities using both empirical and gap acceptance approaches. His empirical approach used regression models to develop estimates of U-turn capacity. However, these models cannot be incorporated into the HCM methodology. In his gap acceptance approach, critical headway and follow-up time were estimated empirically, but additional research is needed to determine if these estimates apply to conditions in the United States. In an attempt to study the gap acceptance characteristics of U-turn movements under multi-lane conditions, Yang et al. [23] collected data from 10 sites in Tampa, Florida, and estimated critical headways using Raff's method and Logit model. However, additional evidence is needed to generalize the results for use with the HCM methodology.

SUMMARY

In this chapter, briefly described the findings of literature review related to the capacity analysis of TWSC intersections. We found that the state of the art is not sufficient to address all the issues related to the operations of TWSC intersections. We also found that the HCM methodology is by far the most comprehensive and widely-accepted method. Therefore, HCM methodology for analyzing TWSC intersections was selected for incorporation into PASSER V software.

3. FIELD STUDIES

We conducted 10 field studies to gain better understanding of the operations of TWSC intersections and related issues, including the characteristics of platoons from upstream signals, and ways to calibrate the HCM model. Limited resources led to a decision to select all sites in College Station, Texas. This chapter describes these field studies.

DESCRIPTION OF SITES

TWSC intersections identified for field studies were selected to cover a wide range of geometric conditions. Table 1 provides information about the characteristics of each of these 10 sites. As identified by this table, the only desired geometric characteristic that could not be captured was the presence of a wide median with second-stage storage space of more than one vehicle.

Table 1. Geometric Characteristics of the Selected Sites.

Main Street	Site	Number of Lanes On Main Street	Median Type	Number of Legs	Upstream Signals within ¼ Mile
Texas Avenue	Lincoln Street	6	Raised	3	Yes
FM 2818	Holleman Drive	4	TWLTL	3	No
	Jones/Butler Road	4	TWLTL	3	No
	Dowling Road	4	TWLTL	3	Yes
University Drive	Eisenhower Street	6	TWLTL	4	Yes
	Shopping Center (Men's Warehouse)	6	TWLTL	3	Yes
	Title Company	6	Raised	4	Yes
Rock Prairie Road	Edelweiss Avenue	4	Raised	4	No
	Rio Grande Boulevard	4	TWLTL	3	No
	Retail Center (Walgreen)	4	TWLTL	3	Yes

Of these 10 sites, only the three TWSC intersections located on FM 2818 are adjacent to each other. Dowling Road, Jones/Butler Road, and Holleman Drive are located 0.15, 0.35, and 0.65 miles, respectively, from a signalized intersection on the east side. On the west side, there is a signalized intersection 1.1 miles from Holleman Drive. This geometric feature provided some data to study platoon dispersion. The Texas Avenue site was used in a pilot study. All other sites were used for full data collection. The next section discusses data collection and provides additional geometric information for each site during this process.

DATA COLLECTION

A video trailer was used to record traffic data in the field. This trailer was equipped with two pan-tilt-zoom cameras mounted on a 20-foot telescoping pole with an effective camera height of approximately 24 feet. The trailer was equipped with a generator, a digital video recorder (DVR), and a video monitor. A pilot study was conducted at the Texas Avenue site to determine the best trailer placement to obtain all desired information. Figure 8 illustrates the trailer and the best trailer location for that site. Figure 9 shows a sample of shots from the two cameras.

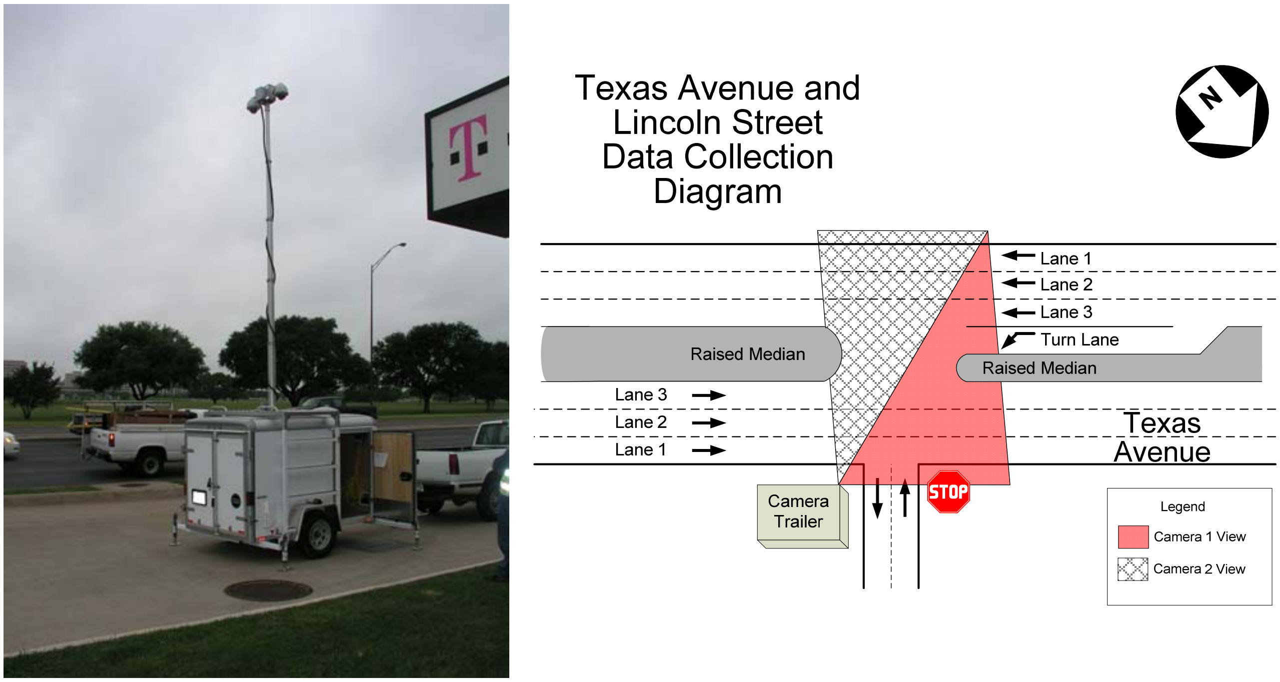


Figure 8. Video Trailer Used in the Field.

Figure 9. Camera Views from the Site on Texas Avenue and Lincoln Street.

At each of the other sites, data were recorded on a single weekday from 7:30am to 6:00pm over a period of several months. Site sketches and camera viewing zones for locations on Texas Avenue, FM 2818, University Drive, and Rock Prairie Road are shown in Figure 8, Figure 10, Figure 11, and Figure 12, respectively. During the course of these studies, video data were recorded on the DVR followed by in-lab analysis to extract desired information. Original multiplexed video was used for data extraction because multiplexing took care of the camera synchronization issue. After completing data extraction for each site, we recorded the video from each camera on a DVD for archival purposes and erased the video from the DVR.

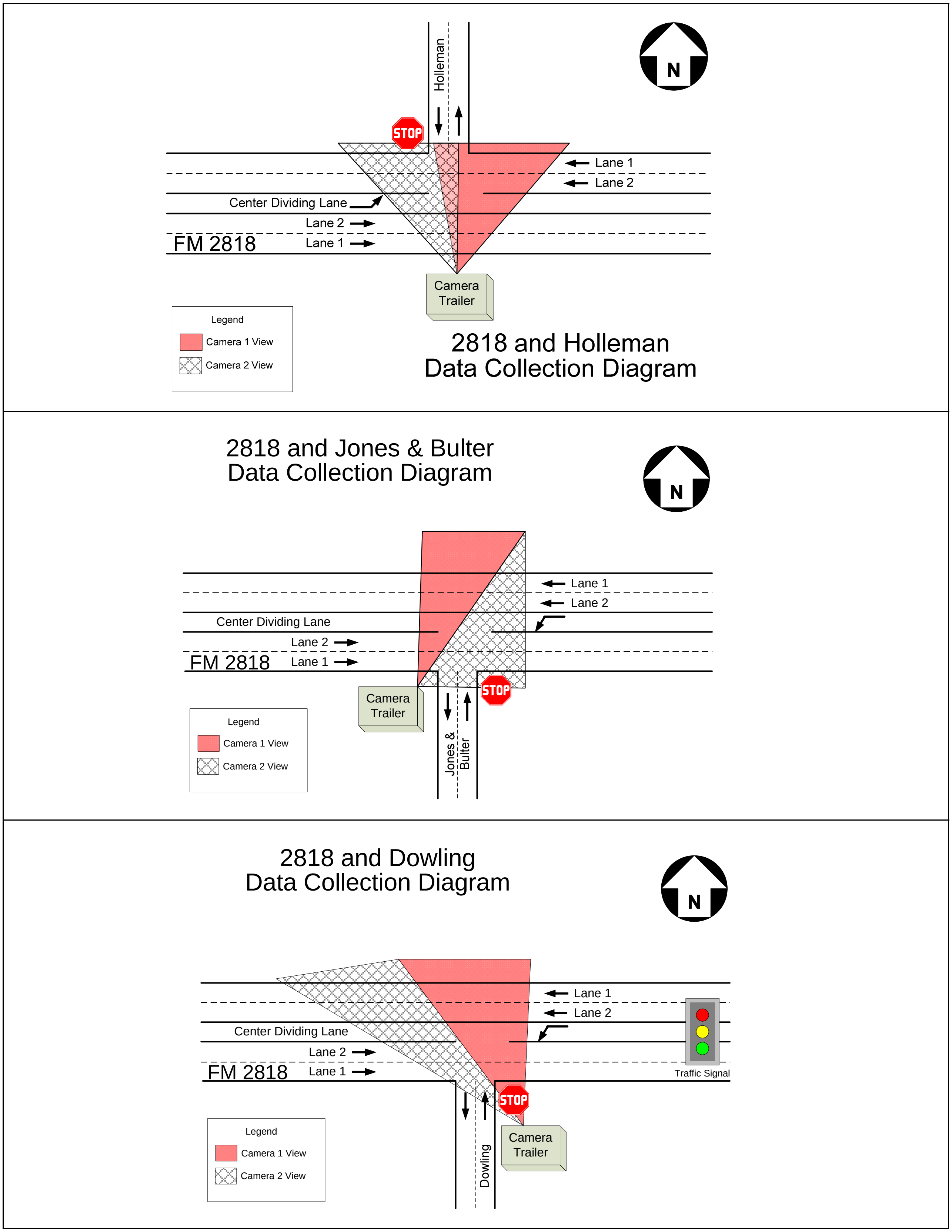


Figure 10. Camera's Viewing Horizons for Sites on FM 2818.

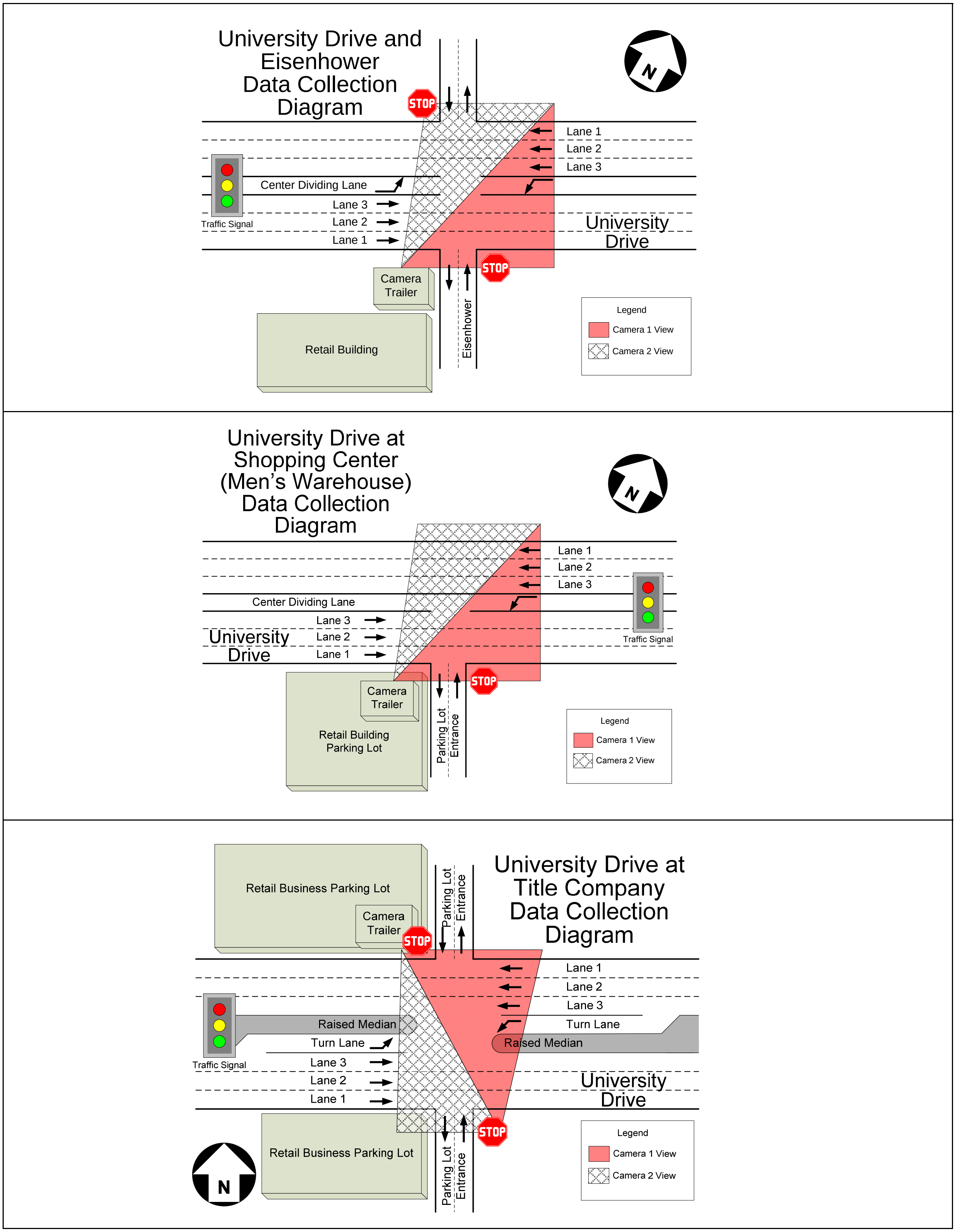


Figure 11. Camera's Viewing Horizons for Sites on University Drive.

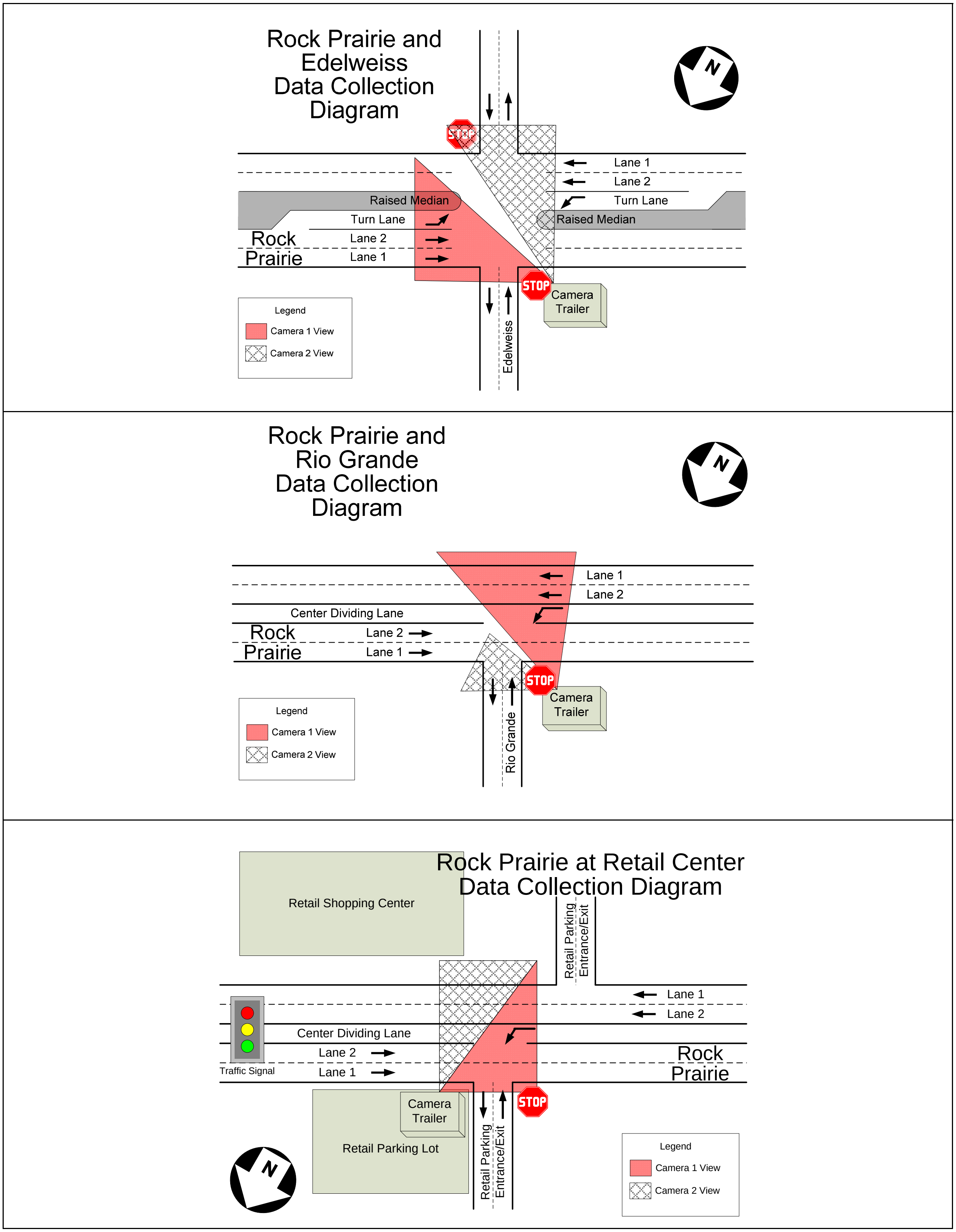


Figure 12. Camera's Viewing Horizons for Sites on Rock Prairie Road.

The desired data extraction plan was fully or partially automated using an Autoscope RackVision® video processing system. In this set up, video from each site would be run through the system with directional detectors defined in each main street lane to capture lane-by-lane actuations and additional detectors along the paths of one minor movement at a time. Time stamps from actuations at these detectors would then allow the determination of individual lane and joint headways for each major approach and information about which ones of these headways were accepted by the subject minor movement. The process would be repeated for each minor movement at each study intersection. Trial runs conducted to test the feasibility of this automation process revealed that it did not provide accurate data. Two factors, both due to insufficient camera height, contributed to data extraction errors. The first reason was occlusion, which causes vehicles to activate detectors in other lanes and in some cases even on cross-street detectors defined to distinguish turning vehicles from through vehicles. The second reason was insufficient coverage to capture the entire gap acceptance process, especially when TWLTLs were present. In the case of TWLTLs, vehicles were observed to exit the detection zone before completing the second stage of gap acceptance. Several adjustments were made to detection zone, but they either did not solve existing problems or resulted in new ones. Therefore, the desired automation process could not be utilized. Thus, we decided to process the data manually.

To facilitate the manual data gathering process, a computer program was developed. This program provides for pressing one of several keys to register the arrival time of each main-street vehicle according to its lane. This data would provide for the extraction of main street headway data. Additional keys were assigned to record various events related to minor movements. These events would be used to extract data for accepted and rejected headways. Depending on the geometric complexity of a subject intersection, one or two people were required to watch the video and record event times. This is a complex and time consuming process even with the help of a computer program. For this reason, manual data extraction was carried out only for the morning peak period (around 7:00am to 8:30am), noon peak period (around 11:30am to 1:00pm), and evening peak period (around 4:30pm to 5:30pm). Available headways, rejected headways, accepted headways, follow-up time, and vehicle counts were obtained from this data using macros in Microsoft Excel®. During the manual data collection process, we found that several segments of data were lost due to equipment malfunction. Table 2 identifies times for which useful data could be obtained.

Table 2. Data Set for Analysis.

Main Street	Site	Morning	Noon	Evening
Texas Avenue	Lincoln Street		X	
FM 2818	Holleman Drive		X	X
	Jones & Butler Road	X	X	X
	Dowling Road	X	X	X
University Drive	Eisenhower Street	X		
	Shopping Center (Men's Warehouse)	X	X	X
	Title Company	X	X	
Rock Prairie Road	Edelweiss Avenue	X	X	X
	Rio Grande Boulevard	X	X	X
	Retail Center (Walgreen)	X	X	

DATA ANALYSIS

After processing the data, we found that some of the minor movements during certain peak periods have a limited number of observations. The purpose of sampling is to obtain knowledge about a population of concern based on selected observations. As a rule of thumb, a sample with 30 observations is deemed to be a reasonable representation of the population. Therefore, during each peak period, any minor movement that has less than 30 observations should not be included for further analysis. Table 3 shows the data for further analysis based on this criterion. The numbers in this table are the actual counts (that is, number of accepted headways) for each movement.

Table 3. Sample Sizes of the Observed Movements.

Main Street	Site	Main Street			Minor Street					
		Left			Left			Right		
		M	N	E	M	N	E	M	N	E
FM 2818	Holleman Drive		136	127		67	43		133	131
	Jones & Butler Road				224	95	70			
	Dowling Road							141	139	80
University Drive	Eisenhower Street	41								
	Shopping Center (Men's Warehouse)									
	Title Company	32	36							
Rock Prairie Road	Edelweiss Avenue							47		
	Retail Center (Walgreen)					42			58	

M: Morning Peak, N: Noon Peak, E: Evening Peak

Follow-up Times

Information on follow-up time of a movement is collected from the field data when (1) there is a continuous queue at the subject movement and (2) both the lead and the following vehicles use the same headway for a maneuver. However, most minor movements at the study sites only experienced queues occasionally, and/or the number of available headways that are long enough for more than one vehicle to enter the conflicting stream were limited. Thus, we were only able to obtain an average follow-up time for the minor left-turn movement at the Jones & Butler site, which is 3.3 seconds compared to 3.5 seconds suggested by HCM 2000 for similar locations.

Critical Headways

As mentioned in the previous chapter, critical headway cannot be measured directly from the field. Assuming the drivers are homogeneous and consistent, methods described earlier can be employed to estimate the critical headway. In this section, Raff's Method and the Maximum Likelihood Method (MLM) are used to estimate the critical headways.

Recall that the critical headway is estimated from the rejected and accepted headways observed from the field. An exceptionally long headway (e.g., 15 seconds) provides little information about the critical headway as it is intuitively obvious that most drivers will accept such long headways. Gattis and Low [24] suggested that headways longer than 12 seconds should not be included in the analysis because their inclusion may skew the result. Therefore, headways longer than 12 seconds are excluded when estimating critical headway using MLM. On the other hand, Luttinen [25] mentioned that Raff's Method is not very sensitive to the inclusion of long headways, so we retain all the data when using Raff's Method for estimating the critical headway.

Left Turns on Main Street

Using Raff's method and MLM, we obtained the critical headways for the main-street left-turn movements at three sites for which we had sufficient data. Table 4 provides these results. The last column of this table shows critical headway values recommended by HCM 2000 for similar sites. It should be noted that Holleman Drive intersects FM 2818, which is a four-lane arterial, but Eisenhower and Title Company intersections are located on University Drive, which is a six-lane roadway.

Table 4. Critical Headways of Major Left-Turn Movements (in seconds).

Site		Raff's Method	MLM	HCM
Holleman Drive	Noon	5.0	6.2	4.1
	Evening	5.0	6.1	
Eisenhower Street	Morning	5.5	5.8	4.1*
Title Company	Morning	5.5	5.9	
	Noon	**7.0**	**6.8**	

* This is the value recommended by HCM 2000 for four-lane arterials. HCM 2000 states that the headway and follow-up time data suggested for four-lane arterials can be used for six-lane roadways. However, approved corrections and changes for HCM 2000 [26] recommend against this practice due to lacking data.

A quick glance at the table results in the observations listed below:

- Raff's method and MLM produce different results.
- For the Title Company intersection, estimates produced by both methods for the noon-peak period are significantly different from other estimates produced by the same methods. This discrepancy may be the result of insufficient sample size, which is barely more than 30 observations.
- With the exception of noon-peak data for the Title Company intersection:
  - Raff's method produced the most consistent results, and
  - MLM estimates for critical headway for a six-lane facility are slightly lower than those for a four-lane facility. This slight inconsistency may be due to human error in data collection or insufficient sample size.
- Another interesting observation is that both Raff's method and MLM produce longer critical headways than that suggested by HCM 2000.

Right Turns on Minor Street

Table 5 summarizes the resulting critical headways estimated for the minor right-turn movement at each site.

Table 5. Critical Headways of Minor Right-Turn Movements (in seconds).

Site		Raff's Method	MLM	HCM
Holleman Drive	Noon	6.0	5.94	6.9
	Evening	6.0	6.8	
Dowling Road	Morning	6.0	6.68	6.9
	Noon	6.5	6.69	
	Evening	5.5	6.48	
Edelweiss Avenue	Morning	**10.5**	**7.04**	6.9
Retail Center (Walgreen)	Noon	5.0	6.17	6.9

As shown in Table 5, Raff's Method produced consistent results with the exception of the Edelweiss Avenue site. Ignoring this intersection, the estimates range from 5.0 to 6.5 seconds. Similarly, MLM provides consistent estimates, except for the Edelweiss Avenue site. These estimates range from 5.94 to 6.69 seconds. In general, MLM produces larger estimates than Raff's Method. When compared to the critical headway value of 6.9 seconds suggested by HCM 2000 for four-lane roadways, both methods provided shorter critical headways.

To understand the anomaly at Edelweiss Avenue, we reviewed the original videos and traffic counts on the main street. This investigation revealed low main-street traffic volume. During the morning peak period, the total hourly flow on the conflicting main street movement was only 318 vph (159 vph per lane). This volume level resulted in a significant number of large gaps. The inclusion of these gaps in Raff's method calculation resulted in a skewed estimate of critical headway. For this site, critical headway estimation using MLM is also higher, but the skew is not as pronounced because gaps larger than 12 seconds were removed from data analysis. Based on these observations, we recommend that sites with such low volumes be excluded from such studies.

Critical headway estimated for the Walgreen driveway derived using Raff's method also raised some concerns. To understand why this estimate (five seconds) was one second lower than the average of six seconds for other estimates, we conducted a review of captured video at this site. We observed a significant platoon effect due to a signalized intersection located approximately 250 feet upstream of this driveway. Specifically, we observed that vehicles arriving from the traffic signal produced a significant number of short gaps that are rejected followed by relatively fewer large gaps. This phenomenon produced a steep curve (as shown in Figure 13) for the cumulative distribution of rejected headways, intersecting the other curve (cumulative distribution of accepted headways) at a point which accounts for only 10 percent of accepted headways. The end result is a lower than expected estimate of critical headway. Since we did not have sufficient data to verify movement capacity, it was not possible to assess which method produced a more accurate estimate of critical headway.

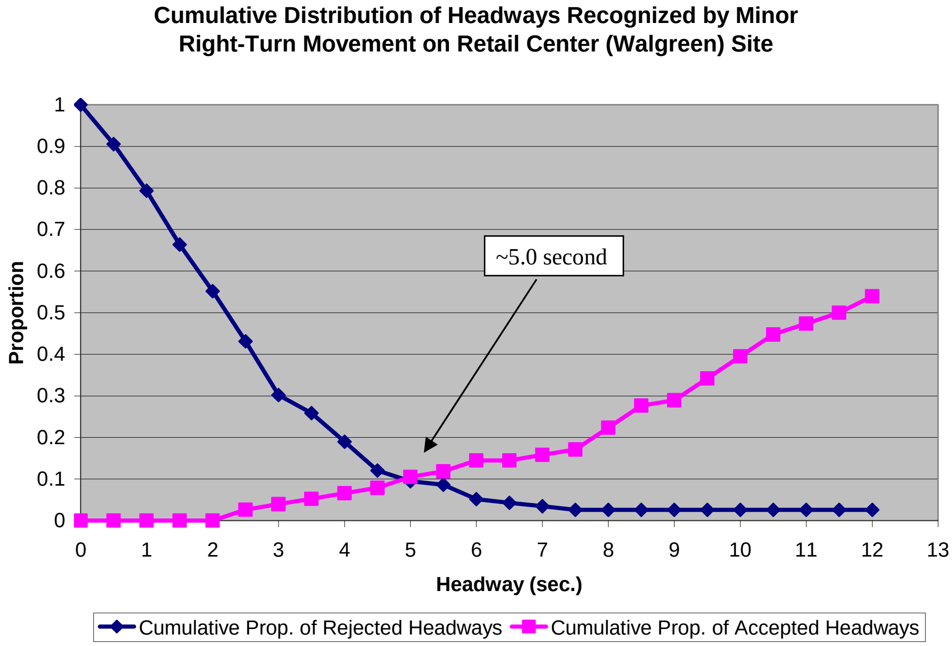


Figure 13. Critical Headway of the Right-Turn Movement on Retail Center (Walgreen) Site Using Raff's Method.

Left Turns on Minor Street

Left-turn movement on the minor street has the lowest rank, and its operation is the most complicated among all the movements at a TWSC intersection. This complexity is due to the fact that this minor-street left-turn movement must yield to, and are affected by, all other movements except opposing left-turns. In addition, other factors such as number of legs and the presence of median storage space further complicate the operation.

In collecting the data, we only recorded the times at which left-turn vehicles merged into the main street traffic stream. Information on which lane these vehicles merged to was not recorded. As a result, we were not able to determine true headways observed by left-turn vehicles. To estimate the critical headway, we used the following definitions of headway:

- Headway calculated by considering main-street traffic lanes on the near side only (traffic arriving from the left side). This is the headway that a minor-street vehicle must accept or reject during the first stage of two-stage gap acceptance process.
- Headway calculated by considering all vehicles in all near-side lanes plus vehicles only in the inside lane of the far-side approach. This method assumes that a minor-street left-turn vehicle will complete the maneuver in a single stage by crossing the near-side approach and merging into the inside lane on the far side approach.

For convenience, the term "approach headways" described headways calculated using the first definition, and when used alone the term "headways" defined those headways calculated using the second definition. Table 6 summarizes the resulting critical approach headways, and Table 7 presents the results for critical headways. The last column in each table lists HCM-suggested critical headways for single-stage and individual stages of a two-stage maneuver, shown in parenthesis.

Table 6. Critical Approach Headways of Minor Left-Turn Movements (in seconds).

Site		Raff's Method	MLM	HCM
Holleman Drive	Noon	7.25	7.29	6.8 (5.8)
	Evening	7.5	7.89	
Jones & Butler Road	Morning	7.0	7.17	6.8 (5.8)
	Noon	6.5	6.61	
	Evening	**4.5**	**6.10**	
Edelweiss Avenue	Morning	**11.5**	7.31	7.5 (6.5)
Retail Center (Walgreen)	Noon	7.5	6.99	6.8 (5.8)

Table 7. Critical Headways of Minor Left-Turn Movements (in seconds).

Site		Raff's Method	MLM	HCM
Holleman Drive	Noon	6.5	7.04	6.8 (5.8)
	Evening	6.75	7.22	
Jones & Butler Road	Morning	**4.0**	**5.69**	6.8 (5.8)
	Noon	5.0	6.06	
	Evening	**3.5**	**5.16**	
Edelweiss Avenue	Morning	**9.0**	6.33	7.5 (6.5)
Retail Center (Walgreen)	Noon	4.75	5.97	6.8 (5.8)

In regards to the first definition (approach headway), both Raff's Method and MLM provide similar and consistent estimates except for the Jones & Butler site during evening peak and Edelweiss Avenue during the morning peak. On the other hand, the estimates obtained using the second definition (Table 7) vary by site and time period.

For Edelweiss Avenue, Raff's method resulted in large estimated values for critical approach headway and critical headway (11.5 and 9 seconds) as compared to MLM. As described earlier, the reason is light traffic, which produces a significant number of large headways. As noted earlier, MLM is immune to resulting effects because its application ignores gaps larger than 12 seconds long.

A closer review of the morning peak hour data for the Jones & Butler site reveals that there was heavy traffic from the approach on the left. Although most drivers finished the maneuver in a single stage instead of stopping in the TWLTL, we observed that many drivers were more aggressive in that they used the inner lane and the TWLTL as a buffer and controlled their speed allowing them to recognize maximum gap in the merging lane. Thus, it seems that the presence of median storage encourages more aggressive behavior even when drivers do not use two-stage gap acceptance. Figure 14 illustrates this aggressive behavior and how it skews the accepted headway. In Figure 14, the left-turn vehicle L accepts the headway between vehicles 1 and 2. However, this headway is not long enough for vehicle L to finish its maneuver; instead, it is only sufficient for vehicle L to pass through lane 1. In this case, the recorded accepted headway will be significantly shorter.

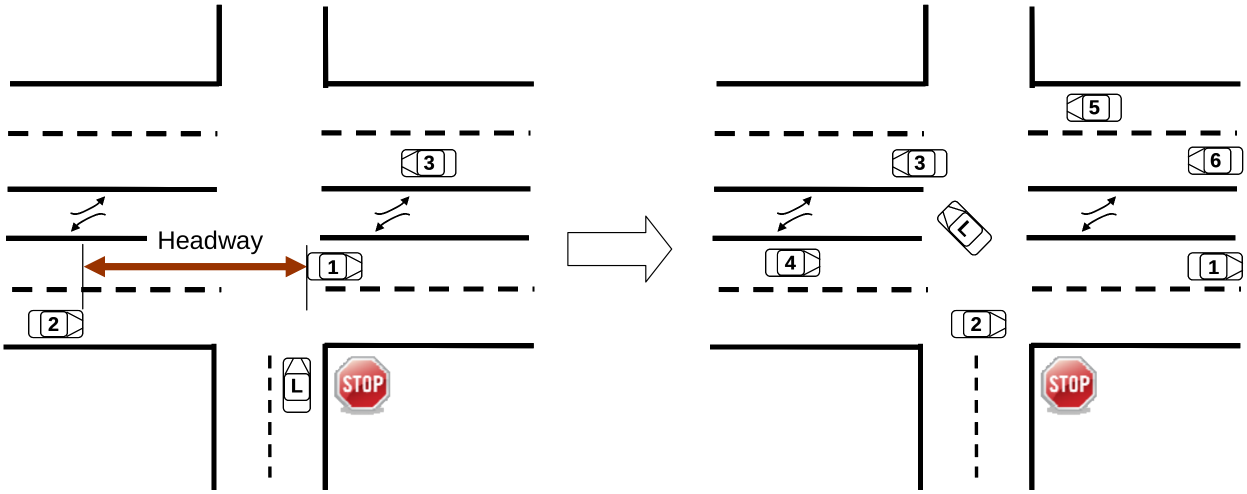


Figure 14. Left-Turn Operation.

We had concerns about the small estimated values for the evening peak period for the Jones/Butler intersection. We reviewed the video to better understand the underlying reason. This review revealed that during this time this intersection experiences heavy arterial traffic on the near side and relatively light arterial traffic on the far side during evening peak periods. We further observed that minor-street left-turn drivers were more aggressive during this period, resulting in shorter headway estimates by both methods.

To further investigate the driver behavior, we reviewed the noon-peak data for the same site. During this time the main street traffic was observed to be 70 percent of that during the morning peak period. For this period, we found that the drivers of the minor left-turn movement at this site were more patient compared to those observed during the morning and evening peak periods. This observation suggests that adjustments to critical headway to account for varying levels of conflicting flow may produce more accurate results.

Capacity

Field measurement of capacity for a movement requires the presence of a persistent queue for a sufficiently long period. The left-turn movement on Jones/Butler was the only observed movement that met this condition for a long-enough (30-minute) period during noon-peak. As shown in Figure 15, Jones/Butler and Dowling road intersections provide two access points from FM 2818 to the same residential area. Because of these geometrics, each of the two TWSC intersections predominantly serve only two movements identified in the figure. These characteristics provided an opportunity to accurately count the left-turn volume, while the queue persisted (the 30-minute period). Doubling this count produced a left-turn capacity of 300 vph for this movement.

Figure 15. Data Collection Sites on FM 2818, College Station, Texas.

To estimate the capacity using HCM methodology, we used the critical approach headways and critical headways shown in Table 6 and Table 7. In addition, since follow-up time (3.3 seconds) for this location was also available (see Follow-up Times subsection on page 21), we calculated the critical headway based on the relationship assumed by Jessen [12], which is 3.3/0.6 = 5.5 seconds. Table 8 presents the capacity estimated by HCM methodology using different critical headways.

Table 8. Estimated Capacity.

		Capacity
Critical Approach Headway	Raff's Method	401
	MLM	258
Critical Headway	Raff's Method	534
	MLM	**333**
	Jessen's Formula	**303**
	HCM	243

The reader will observe that the critical approach headway and critical headway estimated using Raff's Method resulted in significant overestimation of observed capacity, which was found to be 300 vph. The magnitudes of these overestimations are 34 percent and 78 percent, respectively. Furthermore, critical headway suggested by HCM 2000 underestimated capacity by 25 percent, whereas critical approach headway and critical headway estimated using MLM underestimated the capacity by 14 and 11 percent, respectively. Capacity estimate using Jessen's simple formula was the closest to the observed capacity, with only one percent overestimation.

Platoon Dispersion

As stated previously, the three TWSC intersections located on FM 2818 provided some insights into the platoon dispersion phenomenon. Despite the fact that data collection at these three locations was conducted on different weekdays, analysis of observed headways characteristics (i.e., compactness, magnitude, and periodicity) indicated that platoons from the east-side signalized intersection remained intact as they traveled to Holleman Drive. In the other direction, effects of platoons (observed at Holleman) were not as pronounced but were present. HCM 2000 states that the presence of upstream signals will produce non-random flows and affect the capacity of the unsignalized intersection if the signal is within 0.25 miles of the intersection. Our observations indicate that this distance is much longer. Therefore, in PASSER V enhancement (described in the next chapter) we have decided to provide for non-random arrival adjustment for signals located up to one mile away from a subject TWSC intersection.

SUMMARY

A key objective of field studies was to develop guidelines for calibrating the HCM methodology. We collected and analyzed data from 10 sites, one of which was used as a pilot study to refine data collection and reduction processes. These studies revealed that the operations of TWSC intersections are extremely complicated. There are numerous factors that would have an effect on the performance of TWSC intersections. Thus, any attempt to validate or calibrate the HCM methodology requires sufficiently large samples that include many different factors. As such, elaborate data collection, beyond the scope of this project, is needed to cover all important issues. Nonetheless, we learned several useful lessons from these field studies.

HCM 2000 defines headway as the time between two successive vehicles as they pass a point on the roadway. This definition does not precisely quantify the headway observed/recognized by a minor-movement driver for crossing and/or merging into multi-lane and multi-approach traffic streams. Furthermore, HCM 2000 or other literature does not provide any useful guidelines. Thus, the process to measure headways in the field is a subjective decision. Because of this, it is difficult to calibrate parameters for use with HCM methodology. Also, the ability to collect useful data is dictated by traffic conditions. Any site that does not have significant traffic volumes on the main street during the desired time period should not be used for data collection. Previous research had failed to point out this fact. We observed that use of facilities with light traffic may severely skew estimated critical headways. Additional research is needed to better quantify this finding, which requires better equipment for collecting data. In this study, we only have a video trailer with two cameras mounted on a mast arm. Thus, we only have limited view of the intersections. As such, important information such as in which lane a turning vehicle is merging may not be available, especially when there is a TWLTL, in which case some turning vehicles may travel on the TWLTL and merge to the main lane at a point that is far from the intersection.

4. PASSER V ENHANCEMENT

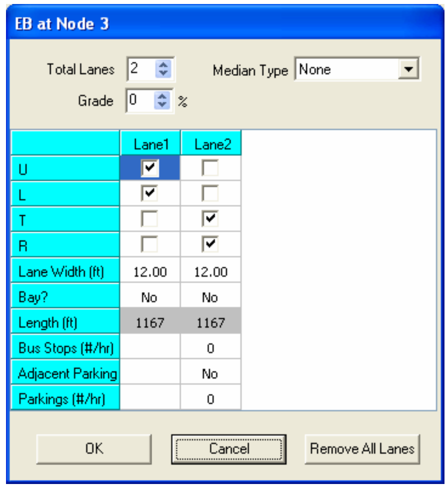
The primary application of PASSER V is the coordination of traffic signals on signalized arterials. Such facilities commonly have TWSC intersections or driveways between adjacent signalized intersections. Often, it is desirable to analyze the operational performance of such intersections and their impact on the operation of adjacent traffic signals or vice versa. Such a need arises especially when evaluating various access management alternatives, however, PASSER V lacks features to provide the analysis of TWSC intersections. The primary objective of this project was to enhance the program to fulfill this need. This chapter describes enhancements made to the PASSER V-03 (P503) program to enable the analysis of isolated TWSC intersections as well those located on signalized arterials. These enhancements include:

- modifications to program input data and output streams,
- enhancements to the user interface, including
    - modifications to existing input and output screens,
    - addition of new screens, and
    - modifications to graphical display,
- enhancements to tools, including
    - incorporation of HCM 2000 capacity analysis procedure into P503 for providing analysis of isolated TWSC intersections with and without the effect of adjacent traffic signals,
    - modification to PASSER II optimization tool in P503 for accommodating TWSC intersections while optimizing signal timings,
    - modification of GA-based optimization tool in P503 for accommodating TWSC intersections while optimizing signal timings,
    - modification to the delay analysis routine (DAR) in P503 to integrate capacity analysis of TWSC intersections during its mesoscopic simulation process. Enhanced DAR also accounts for capacity reductions at a TWSC intersection where queues from a downstream traffic signal cause full or partial blocking, and
    - modifications to the time-space diagram and volume analysis tools in P503.

The enhanced program was dubbed as PASSER V-07 (P507). The intent of this chapter is to provide information about program enhancement without going into unnecessary programming-level details.

U-TURN MOVEMENTS

P503 does not explicitly handle U-turn movements but treats them as left turns. At a majority of signalized intersections, this limitation does not cause any problems. However, the use of access management treatments that encourage U-turn movements at signalized and unsignalized intersections and median openings is on the rise, creating the need for analysis of U-turn movements separately from left-turn movements in many cases. To accommodate this need, P503's user interface and code have been enhanced. Now the user can specify when an approach has a dedicated or shared U-turn lane. Figure 16 presents a snapshot of P507's lane assignment screen for entering this information.



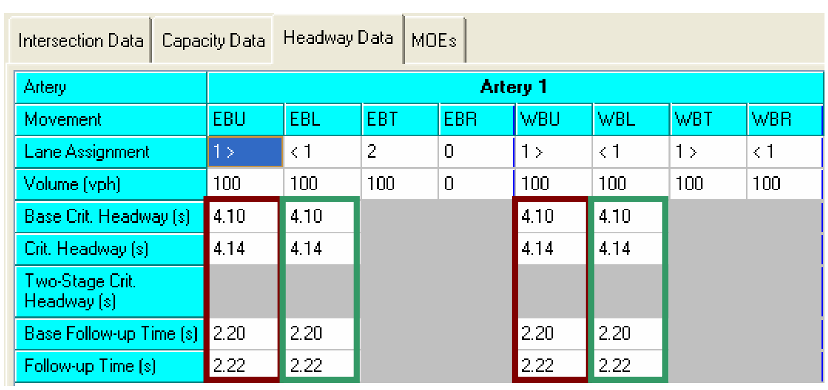
	Lane1	Lane2
U	☑	☐
L	☑	☐
T	☐	☑
R	☐	☑
Lane Width (ft)	12.00	12.00
Bay?	No	No
Length (ft)	1167	1167
Bus Stops (#/hr)		0
Adjacent Parking		No
Parkings (#/hr)		0

Figure 16. Lane Assignment Screen.

Unfortunately, the current version of HCM 2000 and other relevant literature do not provide any guidance for dealing with U-turn movements at signalized and unsignalized intersections. Many questions, such as the following, remain unanswered:

- What is the ideal saturation flow rate for a protected U-turn phase and how to adjust this rate for a shared left and U-turn lane?
- Should there be an adjustment for number of lanes in the opposite direction?
- How to calculate the capacity of protected and protected-permissive phases serving U-turns?
- What adjustments to critical headway and follow-up time values are needed when there are significant U-turns at unsignalized intersections or median openings?

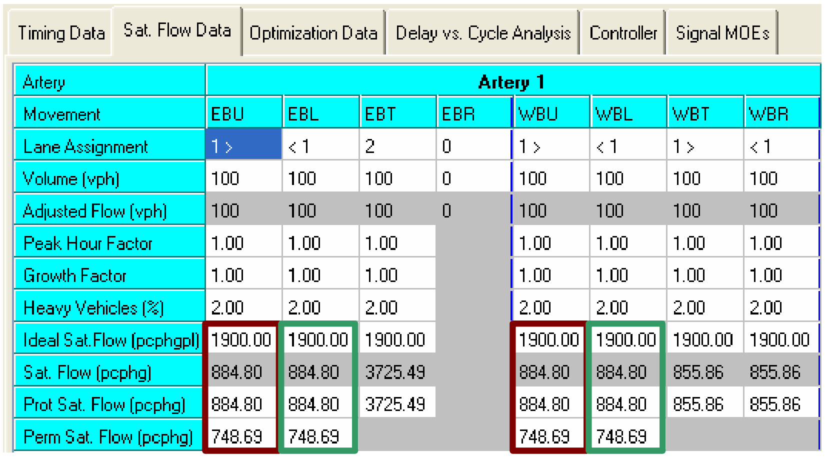
Therefore, the only way to accommodate proper analysis is to provide users the ability to enter desired information. Figure 17 illustrates the Headway Data window for a TWSC intersection. In this case, eastbound and westbound directions have single lanes for providing U- and left turns. Because of lack of guidance, the program automatically enters the same default values of headway and follow-up time for U-turns (maroon rectangles) as left turns (green rectangles). If available, more accurate values can be entered here or at the base data entry level described later.



Artery	Artery 1							
Movement	EBU	EBL	EBT	EBR	WBU	WBL	WBT	WBR
Lane Assignment	1 >	< 1	2	0	1 >	< 1	1 >	< 1
Volume (vph)	100	100	100	0	100	100	100	100
Base Crit. Headway (s)	4.10	4.10			4.10	4.10		
Crit. Headway (s)	4.14	4.14			4.14	4.14		
Two-Stage Crit. Headway (s)								
Base Follow-up Time (s)	2.20	2.20			2.20	2.20		
Follow-up Time (s)	2.22	2.22			2.22	2.22		

Figure 17. Critical Headway and Follow-up Time Input Window for a TWSC Intersection.

[Figure 18](#) illustrates the saturation flow data screen for a signalized intersection. This illustration also shows a case where both eastbound and westbound approaches have single lanes to serve left-turn and U-turn movements. As for TWSC intersections, the program uses the same saturation flow rates for the two movements identified by green and maroon rectangles but allows the user to change these values as appropriate.



Artery	Artery 1							
Movement	EBU	EBL	EBT	EBR	WBU	WBL	WBT	WBR
Lane Assignment	1 >	< 1	2	0	1 >	< 1	1 >	< 1
Volume (vph)	100	100	100	0	100	100	100	100
Adjusted Flow (vph)	100	100	100	0	100	100	100	100
Peak Hour Factor	1.00	1.00	1.00		1.00	1.00	1.00	1.00
Growth Factor	1.00	1.00	1.00		1.00	1.00	1.00	1.00
Heavy Vehicles (%)	2.00	2.00	2.00		2.00	2.00	2.00	2.00
Ideal Sat.Flow (pcphgpl)	1900.00	1900.00	1900.00		1900.00	1900.00	1900.00	1900.00
Sat. Flow (pcphg)	884.80	884.80	3725.49		884.80	884.80	855.86	855.86
Prot Sat. Flow (pcphg)	884.80	884.80	3725.49		884.80	884.80	855.86	855.86
Perm Sat. Flow (pcphg)	748.69	748.69			748.69	748.69		

Figure 18. Saturation Flow Input Window for a Signalized Intersection.

ISOLATED TWSC INTERSECTIONS

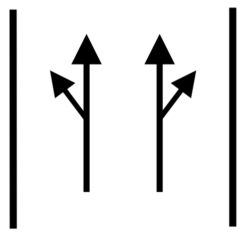
P507 provides analyses for TWSC intersections through the incorporation of HCM methodology (HCM 2000 Chapters 10 and 17). In implementing these procedures, all corrections to date [26] have been applied. In addition, modifications described in the following subsections have been made to enhance the implemented HCM methodology.

Pedestrian Effect

HCM methodology accounts for the effect of pedestrians on the capacity of TWSC intersections by making two adjustments. As mentioned in the previous chapter, the first adjustment is accomplished by adding vehicle counts to pedestrian counts to derive conflicting flow. We have found no theoretical basis for this "adding apples to oranges" approach. We believe that the behavior, and thus the effect on the TWSC operation, of pedestrians and vehicles are very different. The literature is silent on the pedestrian issue. Thus, we have decided to exclude pedestrian flow from conflicting flow calculations. The second adjustment made by HCM methodology is to accommodate impedance caused by pedestrians. Because any significant pedestrian traffic should not be ignored, we have retained this HCM suggested adjustment in the PASSER V implementation.

Shared-Lane Capacity

As opposed to HCM methodology, P503 used an iterative procedure to calculate saturation flow rates for shared movements at signalized intersections. In P507, we have implemented a similar iterative procedure for calculating the capacity of shared lanes at TWSC intersections. HCM methodology for TWSC intersections requires the user to input individual volumes of movements in shared lanes. Often, this piece of information is not readily available. P507's iterative method automatically calculates this information based on a gravity model. It should be noted that other programs (such as Synchro [6] and HCS 2000 [27]) do not provide this level of detail. The following paragraphs provide details of the P507 calculation process using the two-lane approach example shown in Figure 19. In this example left and through movements share the left lane and through and right movements share the right lane. The table in Figure 19 provides flow rates and capacities for left-turn, through, and right-turn movements, assuming exclusive lanes for each movement.



	Flow Rate	Capacity
Left	40	438
Through	100	536
Right	30	961

Figure 19. Lane Assignment and Related Information for Shared-Lane Capacity Example.

The steps of the P507 iterative procedure are:

Step 1: Create a matrix containing rows and columns identifying types of movements and the lane assignment. Equally distribute volume for each movement across all lanes serving that movement. For instance, all left-turn volume is allocated to the left lane because this is the only lane serving left turns, while through volume is equally allocated to the two lanes. Then, calculate the shared-lane capacity using HCM methodology. In this case, the shared-lane capacity of the left lane is calculated as $(40+50)/\left(40/438+50/536\right)=487.5$. Movement flow rates, initial volume distributions, and shared-lane capacities are shown below.

Movement Type	Lanes		Movement Flow Rate
	Left Lane	Right Lane	
Left	40		40
Through	50	50	100
Right		30	30
Capacity	487.5	642.6	

Step 2 (Column Operation): Allocate capacity in each column proportional to the value for each movement flow rate in that lane. For instance, the calculation for left-turn movement in the left lane is $487.5*40/(40+50)=216.7$. The following table shows results of the entire column operation.

Movement Type	Lanes		Movement Flow Rate
	Left Lane	Right Lane	
Left	216.7		40
Through	270.8	401.6	100
Right		240.9	30
Capacity	487.5	642.6	

Step 3 (Row Operation): Allocate flow rate in each row proportional to the value for movement capacity in each lane and update the shared-lane capacity accordingly. For example, the calculation for through movement in the left lane is $100 * 270.8/(270.8+401.6)=40.3$. After volume distribution is complete, update the total capacity of each lane. For instance, the capacity of the left lane is $(40+40.3)/\left({}^{40}\!/_{438}+{}^{40.3}\!/_{536}\right)=482.3$.

Movement Type	Lanes		Movement Flow Rate
	Left Lane	Right Lane	
Left	40		40
Through	40.3	59.7	100
Right		30	30
Capacity	482.3	629.0	

Steps 4 through *N*: Repeat Steps 2 and 3 until all capacity values have converged (that is, stopped changing). Had we continued, the final calculations would have been as follows:

Movement Type	Lanes		Movement Flow Rate
	Left Lane	Right Lane	
Left	258.8		40
Through	219.3	427.7	100
Right		194.1	30
Capacity	478.1	621.8	

Note that the total shared-lane capacities of the left and right lanes are 478.1 and 621.8, respectively. Also note that capacities calculated for left-turn, through, and right-turn movements are 258.8, 647.0 (219.3+427.7) and 194.1, respectively.

Platoon Dispersion Models

When upstream signals are present, HCM methodology uses a platoon dispersion model to determine the proportion of time that the TWSC intersection is blocked by platoons. P507 implements the platoon dispersion model suggested by HCM 2000 and related corrections and changes [26]. In this model, dispersion is a function of speed and distance, where dispersion continues to increase with distance. The dispersion factor in this model is independent of traffic volumes. According to Baass and Lefebvre [28], however, the amount of dispersion varies with volume. They observed that platoon dispersion initially increases with volume, starts to reduce when volume reaches 60-80 percent of link capacity, and becomes zero as volume approaches link capacity. To account for this phenomenon, Manar and Baass [29] proposed a modified platoon dispersion model. We have also added this additional platoon dispersion model to P507. P507 provides the user with an option to select the preferred platoon dispersion model (see Figure 20). Because platoon dispersion over long distances produces traffic flow patterns similar to random arrivals, P507 has been programmed to ignore the effects of traffic signals located more than one mile away from a TWSC intersection.

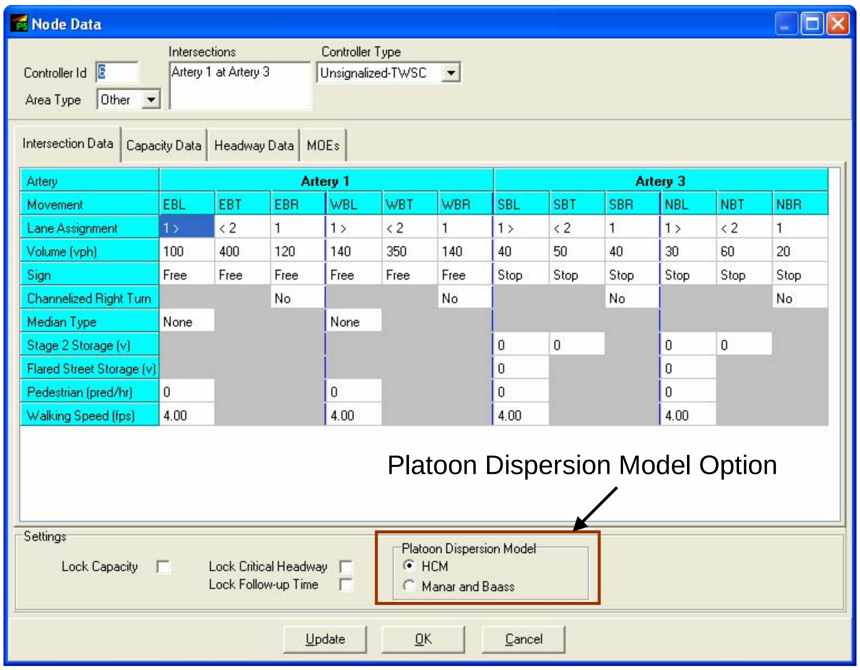


Figure 20. Screenshot of Platoon Dispersion Model Option.

When more than one TWSC intersection shares the same upstream signal as shown in Figure 21, the characteristics of a platoon originating from the upstream signal may be affected by minor movements at the first TWSC intersection (TWSC 1) before arriving at the second intersection (TWSC 2). HCM methodology does not account for such effects. In such cases, P507 treats the platoon adjustment at TWSC 2 as if TWSC 1 does not exist.

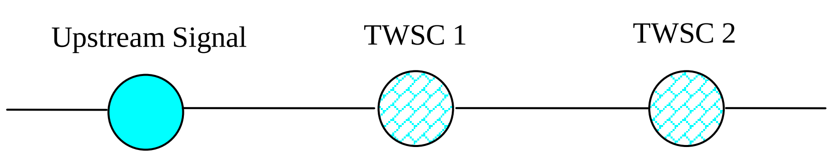


Figure 21. Two TWSC Intersections Sharing the Same Upstream Signal.

DELAY ANALYSIS ROUTINE (DAR)

DAR, a mesoscopic simulation model, was originally developed for analyzing arterials composed of solely signalized intersections. In P507, this routine has been modified to enable the analysis of signalized arterials with TWSC intersections and driveways. Platoon arrival patterns at an unsignalized intersection may change as a result of changes in signal timings at an upstream traffic signal. To accommodate such changes in platoon characteristics, DAR recalculates the capacities for movements at TWSC intersections in the system whenever it is invoked to assess performance measures or measures of effectiveness (MOEs) for a signalized arterial.

Recall that HCM methodology does not account for queue spillback from a downstream traffic signal. However, this limitation has been partially removed in a calculation performed by DAR, which uses the following steps to accommodate TWSC intersections:

1. It calculates the capacities of all TWSC intersections using the HCM methodology. In the process it applies appropriate adjustments to account for upstream signals.
2. It performs mesoscopic simulation of the entire system. In this process, it treats movements at TWSC intersections as if they are being served by permitted phases, whose lengths are equal to the system cycle length. In this process, DAR assumes that the outflow of each TWSC intersection movement is uniformly distributed with respect to its volume and capacity. During simulation, DAR also restricts outflow of any TWSC intersection movement affected by blocking due to queues at a downstream traffic signal.
3. Lastly, it adjusts capacities of any TWSC intersection movements whose capacities are reduced due to downstream blocking.

IMPACT ON OPTIMIZATION TOOLS

The optimization tools in P507 were developed to obtain optimal coordination plans for signalized arterials. As such, they are applicable to systems with at least two signalized intersections along the arterial. Such systems may contain any number of TWSC intersections.

PASSER II Tool

The PASSER II tool is applicable to signalized arterials that contain no diamond interchanges operating in three-phase or four-phase mode (Texas diamond mode). This tool assists users in developing arterial signal timings for maximizing arterial progression. During the optimization process, the PASSER II tool ignores TWSC intersections by assuming that the through phases at these intersections have continuous green indications. In other words, it is assumed that the presence of TWSC intersections will not affect the progression bands. It should be noted that all solutions from this tool are simulated using DAR to obtain performance measures. Because DAR explicitly considers TWSC intersections, the MOEs corresponding to each solution generated by this tool do account for such intersections.

PASSER III Tool

No changes were made to this tool because it applies only to isolated signalized interchanges using the Texas diamond mode.

GA-Based Tool

This tool uses a genetic algorithm to provide users the ability to time signalized arterials for maximizing arterial progression or for minimizing system delay. Depending on the optimization type selected (delay-based or bandwidth-based), it uses either the delay analysis routine or bandwidth analysis routine (BAR) for calculating the fitness values of population members during the optimization process.

If the optimization objective (fitness function) is to maximize progression, this tool treats TWSC intersections similar to the PASSER II tool. That is, TWSC intersections are assumed to have no effect on the progression bands. However, as mentioned previously, use of DAR to generate MOEs does accommodate the analysis of TWSC intersections. If the selected objective is to minimize delay, the GA tool employs DAR to obtain delay estimates during the optimization process.

Volume Analysis Tool

This tool assumes that demands of all TWSC intersection movements in the system can be served, and none of these movements will be a bottleneck. Thus, only the signalized intersections are considered in the volume analysis routine. To determine the maximum potential throughput of the system, the throughput of the TWSC intersection movements are added to the resulting throughput obtained from the volume analysis routine.

Time-Space Diagram Tool

This tool displays progression bands on a time-space diagram (T-Sp Diagram) for the currently loaded timing plan. It ignores TWSC intersections in calculating progression bands but identifies these intersections in its display by showing a horizontal green line at the location of the TWSC intersection. The green line signifies the fact that the through movements have continuous greens.

Delay/Cycle Analysis Tool

This tool displays a plot of system-wide delays versus cycle length. Because delays are calculated using DAR, this tool indirectly accounts for TWSC intersections.

SYSTEM DEFAULT VALUES

To facilitate efficient data input, PASSER V provides a capability to enter certain default values under system data. The program uses these default values to fill the appropriate field when new intersections are created. In P507, this feature has been significantly expanded from two to three categories of parameters. These three categories include general parameters, parameters for signalized intersections, and parameters related to TWSC intersections. These default parameters can be viewed and modified by clicking the “System” button on the function toolbar, as indicated in Figure 22. The following subsections provide additional information about these three categories.

Default General Parameters

Parameters under this category include peak hour factor, growth factor, heavy vehicle percentage, ideal saturation flow rate, link speed, vehicle length, drawing scale, and pedestrian walking speed (see Figure 23).

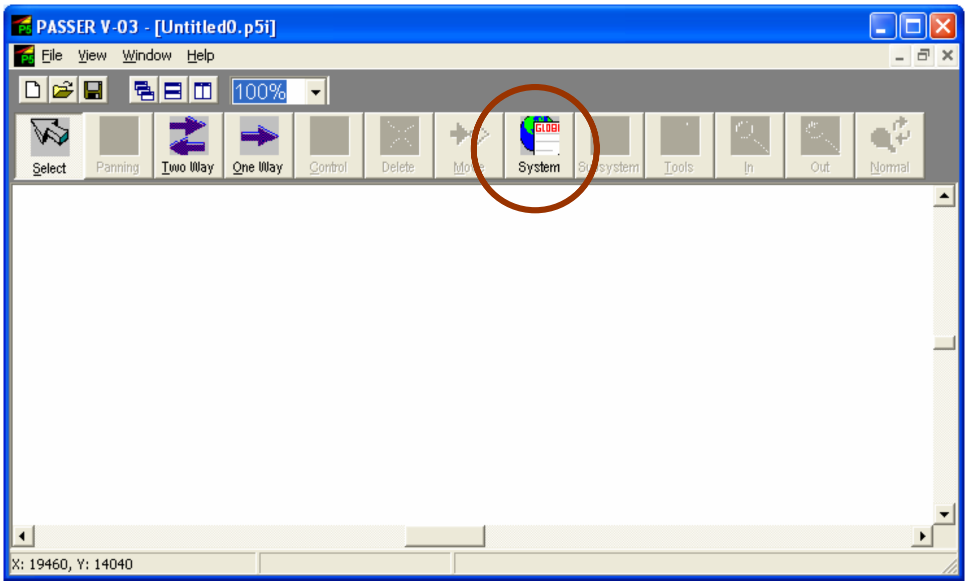


Figure 22. System Button on the Function Toolbar.

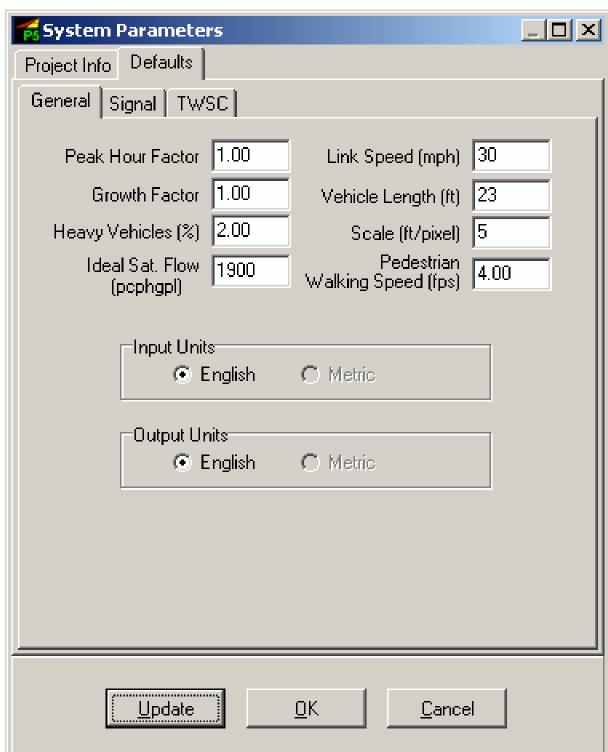


Figure 23. Default General Parameters.

Except for vehicle length and scale, most parameters under this category are used to specify data values a user wants the program to use when creating a new intersection. The value of vehicle length is used by DAR to estimate performance measures. Scale specifies the size of the drawing canvas. Increasing this value to a maximum value of 10 ft/pixel will allow the user to draw a network in a 14×11 mile area. In most cases, it may be beneficial to change at least some data values here before beginning to create a new data set. Examples of data a user may change often are the link speed and peak hour factor (PHF). For instance, if most links of a subject system have speeds of 45 mph and the user is planning on requesting the program to adjust entered volume data by a peak hour factor of 0.95, it will be worthwhile to enter these values on this screen before beginning to create the network.

Default Parameters for Signalized Intersections

Parameters under this category are related to the default timing data for signalized intersections. As shown in Figure 24, these parameters include minimum green time, yellow time, red clearance (all-red) time, lost time for left-turn and through plus right movements, and cycle length information. When the user creates a new signalized intersection, the program will use all values, except cycle length range, as default values for that intersection. Cycle length range is used as default by optimization tools. Note that any of these values can be changed later.

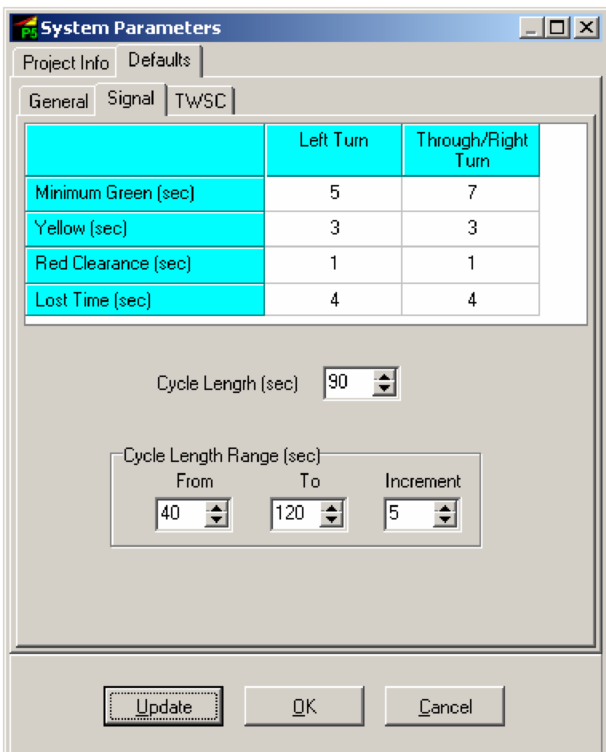


Figure 24. Default Parameters for Signalized Intersections.

Default Parameters for TWSC Intersections

Parameters under this category are the default HCM data used in the analysis of TWSC intersections. These parameters include base critical headways, base follow-up time, and adjustment factors for grade (see Figure 25). As mentioned under the U-Turn Movements section, the current HCM procedure for the analysis of TWSC intersections does not explicitly account for U-turns. PASSER V-07 provides separate fields for all applicable U-turn data to allow better calibration of U-turns if additional data were to become available. At present, the default values for U- and left turns are assumed to be the same. Furthermore, HCM methodology is based on data collected for arterials with four or fewer lanes. As such, it discourages the use of its procedure to analyze unsignalized intersections on arterials with more than four lanes. PASSER V-07 provides fields where users can enter headway and follow-up time data for six-lane arterials, if different and better data were to become available. At present, PASSER V assumes the default data for six-lane roads to be the same as that for four-lane roads. The users are encouraged to use caution when using the program for such facilities.

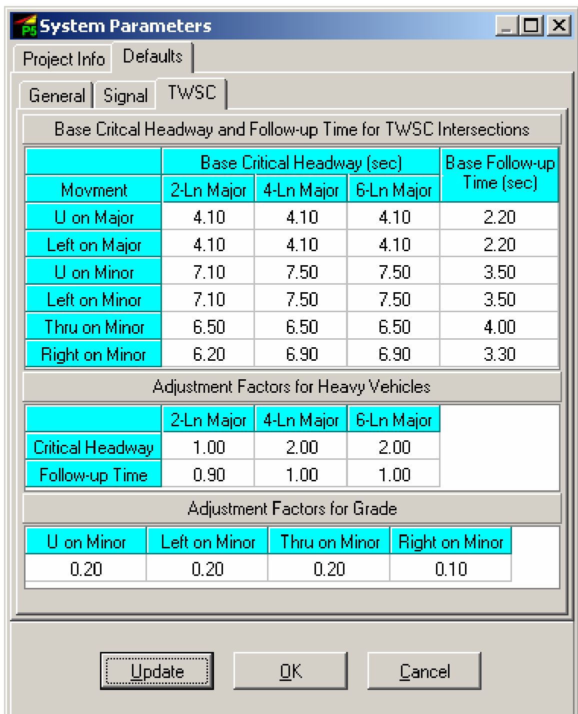


Figure 25. Default Parameters for TWSC Intersections.

PROGRAM LIMITATIONS

When a TWSC intersection is located very close to a downstream signal, queues from that signal may partially or fully block it, severely reducing its capacity. This blockage effect is not considered in the isolated intersection analysis, which replicates HCM 2000 methodology.

5. PASSER V-07 VERIFICATION

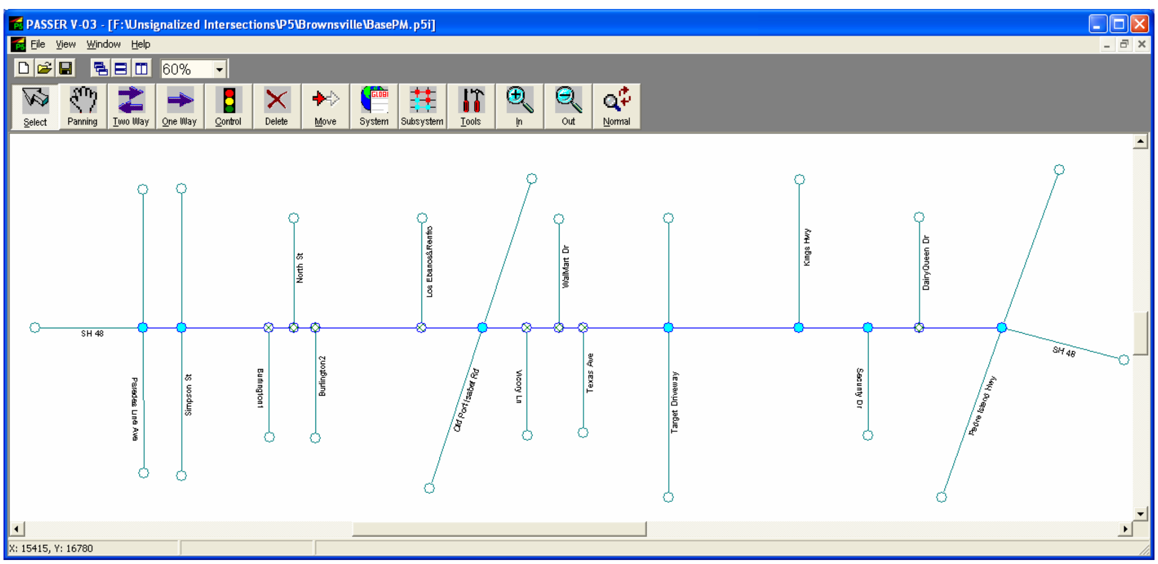
During the development and enhancement of complex software such as PASSER V, it is necessary to perform testing during and after each developmental stage to ensure that the program operates as intended. In this chapter we provide information about two stages of testing based on the two main development stages. The first stage of development was the implementation of HCM methodology for isolated TWSC intersections. The second development stage consisted of modifications to optimization and analysis tools in PASSER V to accommodate TWSC intersections.

ISOLATED TWSC INTERSECTIONS

For this stage, we used several synthetic data sets. Each data set was analyzed using HCS 2000, Synchro 6, and P507. We compared the resulting numbers to ensure that P507 produced the expected results. Based on results of these tests, we concluded that we had successfully implemented HCM methodology into PASSER V.

SIGNALIZED ARTERIALS WITH TWSC INTERSECTIONS AND DRIVEWAYS

Since the primary objective of this project was to enhance PASSER V functionality to enable the analysis and optimization of timings for signalized arterials with TWSC intersections and driveways, we conducted this testing stage using several synthetic data sets and two arterial data sets. Here, we describe the results of one of the two real data sets, an arterial system in Brownsville, Texas. Figure 26 shows P507's display of this system.



= Signalized Intersection
= TWSC Intersection

Figure 26. Test Site in Brownsville, Texas.

For this testing, we first optimized the entire system using the PASSER II tool and Synchro 6 optimization software. Then, we simulated both optimization results using SimTraffic. For this testing, we had to enter timings optimized by the PASSER II tool into Synchro. Five replications of one-hour simulations were performed using SimTraffic and results were averaged. The results for this system are summarized in Table 9.

Table 9. Simulation Results.

MOE	PASSER V-07	Synchro
Cycle Length (sec)	100	100
Throughput	4207	4236
Total Delay (hr)	92.7	84.5
Total Stops	6549	6750
Fuel Used (gal)	326.7	334.1

Coincidentally, both PASSER V-07 and Synchro 6 resulted in the same optimal cycle length, which was not necessarily the case. Since the system was undersaturated, throughput was about the same using the timing plans of PASSER V-07 and Synchro. It can be seen that Synchro 6 has a slight edge over PASSER V-07 in minimizing delay. This is not surprising because the Synchro optimization algorithm is designed to time traffic signals to minimize delay. At the same time, P507 produced timings with larger progression bands (Figure 27) and fewer stops. Again, this was an expected result.

PASSER V-07 Synchro 6

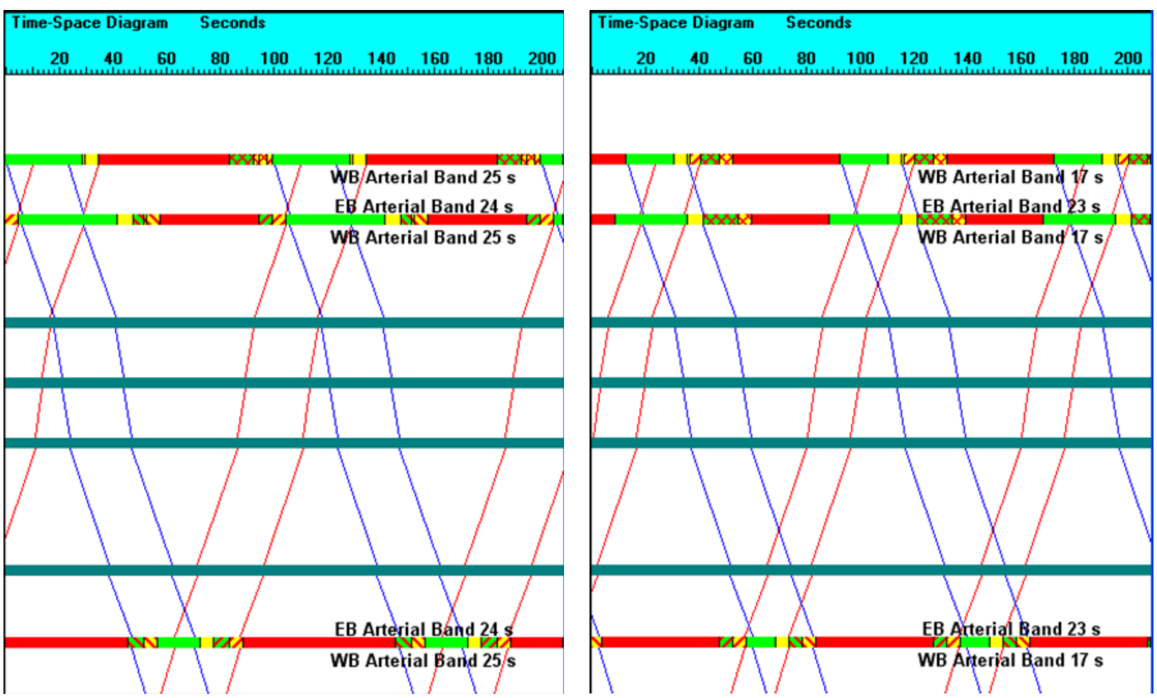


Figure 27. Time-Space Diagrams.

6. SUMMARY

The overall goal of this project was to enhance the PASSER V program to allow its use in arterial access management studies, which require explicit consideration of TWSC intersections. A detailed literature review conducted early on in the project revealed that HCM methodology was the most suitable for use in PASSER V, even though it has several weaknesses. These weaknesses include lack of a systematic approach, guidelines for application to arterials with more than two through travel lanes in each direction, and factors to account for the impact of queues at downstream traffic signals. This review also revealed that HCM 2000 and other relevant technology lacks methods for estimating capacity of U-turn traffic at signalized and unsignalized intersections.

To gain better understanding of operations at TWSC intersections, we performed field studies at 10 selected sites located in College Station, Texas. One objective of these studies was to identify ways to verify and calibrate key parameters used by HCM methodology. During the studies, data were collected using a video trailer with two cameras mounted on a telescoping pole. In-lab manual processing was conducted to extract headway and follow-up time data from these videos. As stated previously, follow-up times can be measured in the field, but critical headway must be derived from field-measured data for accepted and rejected headways. We used Raff's and maximum likelihood methods to determine critical headways for different sites. For reasons identified below, only one site provided useful data sufficient for verification purposes. These data included field estimates of follow-up time, critical headways (using Raff's method and MLM), and movement capacity. For this site, we used HCM methodology to estimate capacity using critical headway suggested by HCM 2000 and the two values of critical headways estimated from field data. In addition, we used a simple formula found in the literature to estimate critical headway using field measured follow-up time and used this estimate to obtain a fourth estimate of capacity using HCM methodology. A comparison of these four capacity estimates with the field measured capacity showed that the estimate using field measured follow-up time was the more accurate estimate. These observations suggest that there may be some merit in estimating critical headway using follow-up time. However, additional investigations should be carried out before recommending this simple method for using field data to calibrate the HCM methodology.

Field studies also revealed that the operations of TWSC intersections are extremely complicated, and a very detailed data collection plan is needed to capture data for all relevant factors. Such a plan is not feasible for most programs like PASSER V. However, should a user decide to undertake such field studies, the following guidelines could be used:

- Develop a concise definition of headway that accounts for driver perception.
- Use the necessary number of synchronized cameras located at sufficient heights to capture the entire headway acceptance and vehicle maneuver process for the studied minor movement.
- Select a site which has sufficient traffic on the main street to provide a large enough sample of headways less than 12 seconds long.

- Select a site where the subject minor movement has sufficient traffic. It is desirable to identify a site/time where the subject minor movement faces sustained queue for 15 minutes or longer if estimated capacity is to be verified using field data.

The last two recommendations also apply to cases where critical headway is to be estimated using field-measured follow-up times.

Next, we modified PASSER V to provide for the analysis of TWSC intersections. We began this process by integrating HCM 2000 isolated intersection methodology into the program. Then, we linked this methodology to various optimization and analysis tools, including PASSER II, GA-Based, DAR, and T-Sp Diagram. In integrating the TWSC intersection analysis methodology into DAR, we also provided a simple method to account for blocking due to queues at a downstream traffic signal. It should be noted that HCM 2000 and existing literature do not address this important factor in analyzing TWSC intersections.

The PASSER V enhancement process summarized above included modifications and additions to the input data stream, data structures, program output, program's graphical user interface, and input and output screens. In enhancing PASSER V, we provided room for future expandability by separating input data for U- and left turns. This feature will provide more accurate analysis should more accurate critical headway and follow-up time data become available for U-turn movement. Lastly, we expanded system-level data input of the program to allow for easily changing default values for several types of data.

The enhanced program was dubbed PASSER V-07. During and after the program enhancement process, we conducted detailed testing to ensure that the program functions as intended. This testing revealed that the program is ready for use by TxDOT and other agencies.

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