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Analysis Of Queue Characteristics At Signalized Intersections Near Highway-Railroad Grade Crossing

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ANALYSIS OF QUEUE CHARACTERISTICS AT SIGNALIZED INTERSECTIONS NEAR HIGHWAY-RAILROAD GRADE CROSSING

by

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INTRODUCTION

1.1 Background

Rail-highway crossings pose a unique set of challenges related to safety that requires agencies involved in highway, and railroad transportation, to work together to alleviate safety and operational issues. Rail-highway at grade crossings are controlled by various traffic control devices including signs, pavement markings, red flashers, gates and other directional separators. Crossing gates and other active devices are used where the potential for automobile-rail crashes is elevated due to the increased frequency of trains and vehicular traffic. The crash potential increases further due to driver violations/non-compliancy of traffic control features at the crossings. Therefore, addressing causal factors of risky driver behavior is essential to creating a safe rail-highway at grade crossing.

A federally mandated report summoned in 1995 included the following statement "Shortly after the tragic collision of a commuter train with a school bus in Fox River Grove, Illinois, that resulted in seven deaths on October 25, 1995, then Secretary of Transportation Federico Peňa asked Michael Huerta, the Associate Deputy Secretary and Director of the Office of Intermodalism, to head up a task force to look into grade crossing safety. The purpose of the task force was to review the decision making process for designing, constructing, maintaining, and operating railroad-highway grade crossings."*(1)* The task force investigated rail-highway grade crossing-related issues and provided a comprehensive report to Transportation Secretary Slater in June of 1997.

This report *(1)* further states that "It has been a long-standing and desirable engineering practice to preempt highway intersection traffic signals in close proximity to railroad-highway grade crossings that have active warning devices. The purpose of the preemption is to allow sufficient time for any motor vehicle inadvertently stopped on a railroad-highway grade crossing to proceed off the track prior to the arrival of a train." *(1)* As a response to this report, all states actively pursued interconnection of traffic signals located in close proximity of all rail-highway crossings. All states also started reviewing at grade crossings that were known to have crash history and/or have anecdotal evidence of high violation.

Safety issues at such locations arise if a train arrives when traffic from the intersection has queued back onto the tracks. Similarly, traffic flow issues arise when traffic queued for a train arrival at the rail crossing spills back into the intersection. To alleviate both safety and operational issues associated with signalized intersections that are in close proximity to a roadway-rail grade crossing, it is necessary to coordinate or "interconnect" the railroad signals with the traffic signals at the nearby intersection. The coordination of railroad signals and traffic signals at a nearby intersection is often performed using a special signal "preemption" strategy, which transfers the traffic signal from the normal operational mode to a special control mode upon detection of an arriving train *(2)*.

The need for preemption arises due to the differences in the right-of-way assignment principles between roadway and railroad signals (i.e., approaching trains ALWAYS have the right-of-way, while roadway signals alternately assign right-of-way based on time or actuation). Because of these differences, the adjacent roadway traffic signals MUST be preempted by an arriving train to give the train priority over all other movements. The objective of a preempt is to take control of the traffic signal at a nearby intersection to provide for the passage of a train, no matter what the status of the normal traffic signal operation at the time the preemption occurs *(3)*. Successful traffic signal pre-emption provides two functions: 1) initially clears the tracks of any queued vehicles and 2) disallows any movements that would intersect the tracks as the traffic may potentially spill back into the intersection.

Issues associated with railroad-traffic signal preemption were brought to the national forefront in the aftermath of the previously referenced 1995 fatal train-bus crash in Illinois *(4)*. This collision occurred because the rear of the bus protruded onto the tracks while stopped for a traffic signal on the adjacent roadway. The crash resulted in seven fatalities. Shortly after the collision, state and federal transportation and railroad agencies began thorough investigations of their own policies and/or procedures to determine ways to minimize the occurrence of a similar situation within their jurisdiction. A national taskforce (assembled as a result of the collision) found two widespread issues associated with the interconnections between the railroad crossing and nearby traffic signals *(2)*.

- No specific guidelines existed as to when to provide interconnection of the highway-rail grade crossings with traffic signals at nearby intersections relative to vehicle storage space between the intersection and the railroad crossing.
- There is ineffective communication between the multiple parties that are responsible for highway-rail grade crossings.

The MUTCD provides guidance as to when signal preemption should be provided *(5)*. Section 8C.09 of the MUTCD states that preemption should be provided if the intersection is located within 200 feet of a rail-highway at grade crossing with flashing signals, although intersections farther than 200 feet may be considered for preemption *(5)*. However, research has suggested that the need for preemption should be based on a detailed queuing analysis at each individual location, rather than a pre-specified distance, such as 200 feet *(6)*. A nationwide survey found that many states desired additional guidance as to when to provide preemption, as preemption was often deemed necessary well beyond the 200 foot rule *(4)*. Items that were suggested for consideration when determining the need for preemption included traffic volumes, number of lanes, traffic signal timing, saturation flow rates, vehicular arrival characteristics, and vehicle types.

Several existing guidelines *(7, 8)* also recommend performing analysis of traffic queues at the signalized intersections to determine if preemption is necessary. If the 95th percentile queues extend across the nearby tracks, or traffic stopped at the crossing backs up and disrupt the operation at a signalized intersection upstream, traffic signal should be preempted.

1.2 Problem Statement

Analysis of traffic queues at signalized intersections located in close proximity to railroad grade crossing is crucial for determining if a traffic signal needs to be preempted for safe operation of vehicular and train traffic at such locations. Although automobile/train crashes are rare events however, such collisions are generally quite severe. It is therefore extremely important to adequately analyze traffic queues at such locations in order to implement safe design that will minimize the risk of future crashes. Such queuing analysis becomes even more critical where direct observations of traffic queues are not possible or where the assessment is needed for a future location. Inadequate estimation of queues from signalized intersections to the nearby railroad grade crossing can lead to severe safety issues. Underestimation of queue lengths may lead to an unsafe decision by not implementing signal preemption for safe clearance of vehicular traffic before the arrival of a train at the crossing. Whereas the design based on significantly overestimated queues may cause unnecessary traffic delays consequently leading to violations of the active traffic control devices at the crossing.

Several analytical and simulation based methods exist that can be used for estimating traffic queue lengths at signalized intersections near railroad grade crossing. For example, deterministic analytical methods for queue prediction are provided in the Highway Capacity

Manual (HCM) *(9)*, ITE guidelines*(7)*, Synchro *(10)* while simulation based methods are provided in software such as Sim-Traffic, VISSIM. The deterministic analytical based procedures are generally based on fundamental assumptions and have limited applicability. Traffic simulation techniques offer model specification to detailed level and have the capability to model actual traffic conditions however several simulation based methods exist and there is inadequate guidance on using a particular method for queue estimation, particularly when also considering railroad signal interconnection. In order to select a suitable method for reasonable queue estimation at signalized intersections near highway-rail grade crossings, there is a need to evaluate these methods and understand their application in order to determine the most appropriate method for queue prediction at interconnected railroad crossings.

1.3 Research Objectives

The main objective of this research is to analyze and evaluate the existing queue estimation procedures to (i) help the designers in selecting an appropriate approach, and (ii) develop a procedure for reasonable estimation of queue lengths at signalized intersections that are in close proximity to highway-railroad grade crossing. In this context, this research included the following tasks:

1. Review state-of-the-art
2. Review current state-of-the-practice
3. Analyze the current queue length estimation procedures
4. Compare different simulation based methods to each other and to other methods

1.4 Organization

Chapter 2 provides a review of existing literature pertaining to traffic signal preemption near railroad-highway grade crossings and also includes a review of existing queue estimation procedures. Chapter 3 consists of a current state-of-the-practice review conducted through states' MUTCD supplements, design manuals, and/or developed guidelines. Chapter 4 provides additional information obtained from states through an online survey. Chapter 5 presents an analysis and comparison of different queue estimation procedures and provides a recommended procedure for improved estimation of traffic queues at signalized intersections for preemption evaluation. Chapter 6 puts together recommendations for determining queue clearance distance based on various existing signalized intersection approaches and also provides recommended queue clearance time. The findings of this dissertation, conclusions and recommendations are presented in Chapter 7.

1.5 Contribution

The main contribution of this research is providing a procedure for reasonable estimation of queue spillback for preemption evaluations at signalized intersections near highway-railroad grade crossing. Four existing queue estimation methods including HCS, Synchro (deterministic analytic models), Sim-Traffic and VISSIM (micro-simulation models) have been evaluated for their adequacy in estimating the queue lengths by capturing the impact of various important traffic factors that affect the resulting extent of queues. Based on the analysis findings, a microscopic simulation based procedure is developed using Sim-Traffic for estimating the queue lengths at signalized intersections near highway-rail grade crossings to help evaluate the need for signal preemption. In addition, recommendations have been developed, if preemption is necessary, for determining queue clearance distance and minimum track clearance time

The recommended procedure is developed considering minimizing the risk of underestimated queues or unsafe design at such locations, and simplify the design and decision-making process.

CHAPTER 2

LITERATURE REVIEW

The literature review presented in this chapter consists of two parts. First part includes an introduction to signal preemption starting out with base criteria for identifying the need for signal preemption and various components of the preemption system including train detection systems, preemption sequence, and minimum warning time. It then presents a review of serval existing methods for estimating traffic queue lengths at signalized intersections in order to determine if preemption is necessary. Some portion of the literature is referenced from the recently completed research in Michigan *(11)* that compiled the current information on traffic signal coordination with railroad crossing devices from federal, state and other sources.

2.1 Federal Preemption Guidance

At the time of the Fox River Grove crash and subsequent task force investigation, the Federal Manual on Uniform Traffic Control Devices (MUTCD) (1988 version) provided only limited guidance on the use of preemption, stating the following in Section 8C-6:

"When highway intersection traffic control signals are within 200 feet of a grade crossing, control of the traffic flow should be designed to provide the vehicle operators using the crossing a measure of safety at least equal to that which existing prior to the installation of such signals."

The 1988 MUTCD also provided guidance <u>against</u> the use of preemption for railroad crossings beyond 200 feet, stating later on in Section 8C-6:

"Except under unusual circumstances, preemption should be limited to the highway intersection traffic signals within 200 feet of the grade crossing."

The 2000 “Millennium” Edition of the MUTCD was the first MUTCD revision to occur after the comprehensive report produced by the task force. The 2000 MUTCD included newly revised language providing additional direction regarding railroad preemption, most significantly, the consideration for providing coordination between railroad and traffic signals at crossings located beyond 200 feet from the intersection. The specific language regarding railroad preemption exists in Section 8D.07 and is provided as follows:

“When a highway-rail grade crossing is equipped with a flashing-light signal system and is located within 60 m (200 ft) of an intersection or mid-block location controlled by a traffic control signal, the traffic control signal should be provided with preemption in accordance with Section 4D.13.

Coordination with the flashing-light signal system should be considered for traffic control signals located farther than 60 m (200 ft) from the highway-rail grade crossing. Factors to be considered should include traffic volumes, vehicle mix, vehicle and train approach speeds, frequency of trains, and queue lengths.”

The preemption guidance that exists in the current (2009) MUTCD remains unchanged from the 2000 MUTCD language.

2.2 Identifying Locations for Preemption

Although the MUTCD provides relatively clear guidance on the use of preemption for signalized intersections within 200 feet of a roadway-highway grade crossings with signals, a nationwide survey found that many states desired additional guidance as to when to provide preemption, when they were beyond the 200 foot rule *(4)*. Research has suggested that the need for pre-emption should be based on a detailed queuing analysis at each individual location, rather than a pre-specified distance, such as 200 feet *(6)*. Researchers have recommended that the need for railroad preemption control should be determined based on the 95th percentile queue length determined either by using simulation or other analytical methods, as suggested by the MUTCD *(12, 13)*. Prior research *(6, 14-16)* has suggested that the queue lengths from a traffic signal are primarily dependent on the 1) approach lane volumes, 2) cycle length, 3) effective green time, 4) turning movement type, 5) left turn signalization, and 6) arrival patterns. A general example of a typical candidate location for train pre-emption is shown in Figure 2 *(4)*.

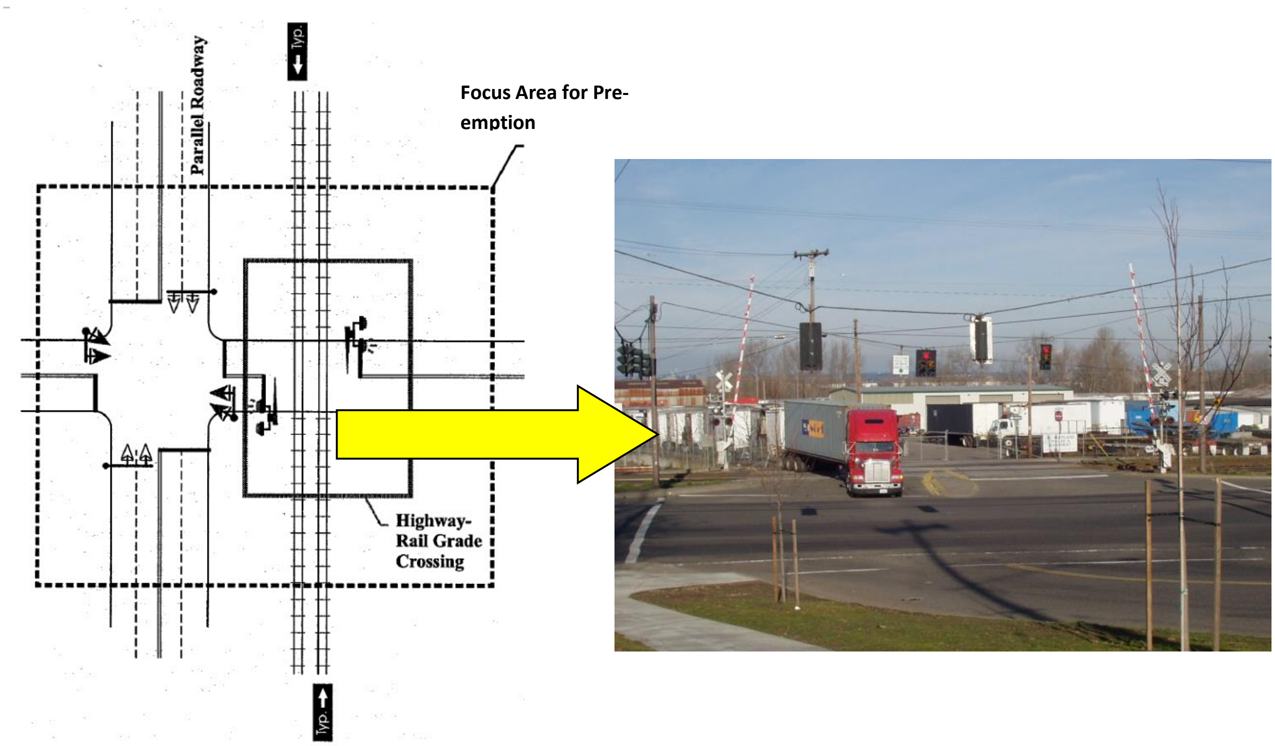


Figure 1. General Example of Candidate Train Preemption Location *(4)*

2.3 Train Detection Systems

Activation of the railroad warning devices utilizes two primary systems: (1) fixed distance and (2) constant warning time. Track circuitry-based detection systems work in the following manner. The rails conduct a small current (supplied by a battery or other power source) through a fixed section of track causing the two rails to act as a closed electrical circuit.

The circuit remains intact until shorted (or shunted) by an approaching train, causing the circuit to de-energize and triggering the warning devices for the vehicular traffic which includes red flashers as well as gates. A nearby traffic signal that is interconnected with the track circuitry will also receive notification of the approaching train through the interconnection circuit. An example of train detection, using track-circuitry, is shown in Figure 1.

The limits of the track circuit are established by the use of insulated joints in the rail, which electrically isolate the particular section of rail. To provide the minimum necessary warning time for all train arrivals, the track circuit must extend an adequate distance from the location of the approach train detection so that adequate warning time is provided even for the fastest possible arriving train. The distance from the crossing location that the detection circuitry must extend is given by the following equation:

$$d_f = r_f \times MWT$$

where: d_f =approach circuit distance for the fastest train operating on the track

r_f = fastest allowable train speed for the track in question

MWT = minimum warning time provided to crossing users

Placement of the detection circuitry, according to the previous equation, will conservatively trigger the railroad warning devices based on the fastest possible approaching train. The drawback to this fixed distance detection is that excessive warning time is provided for slow moving trains. The actual amount of warning time provided by fixed distance detection systems can be determined based on the following equation:

$$t_a = d_a / r_a$$

where: t_s = warning time provided to motorists

d_a = approach circuit distance

r_a = speed of approaching train

Therefore, if the circuitry was designed to provide 20 seconds of warning time for a maximum train approach speed of 40 mph, a train approaching at 10 mph will provide 80 seconds of warning time to motorists prior to arrival of the train at the crossing.

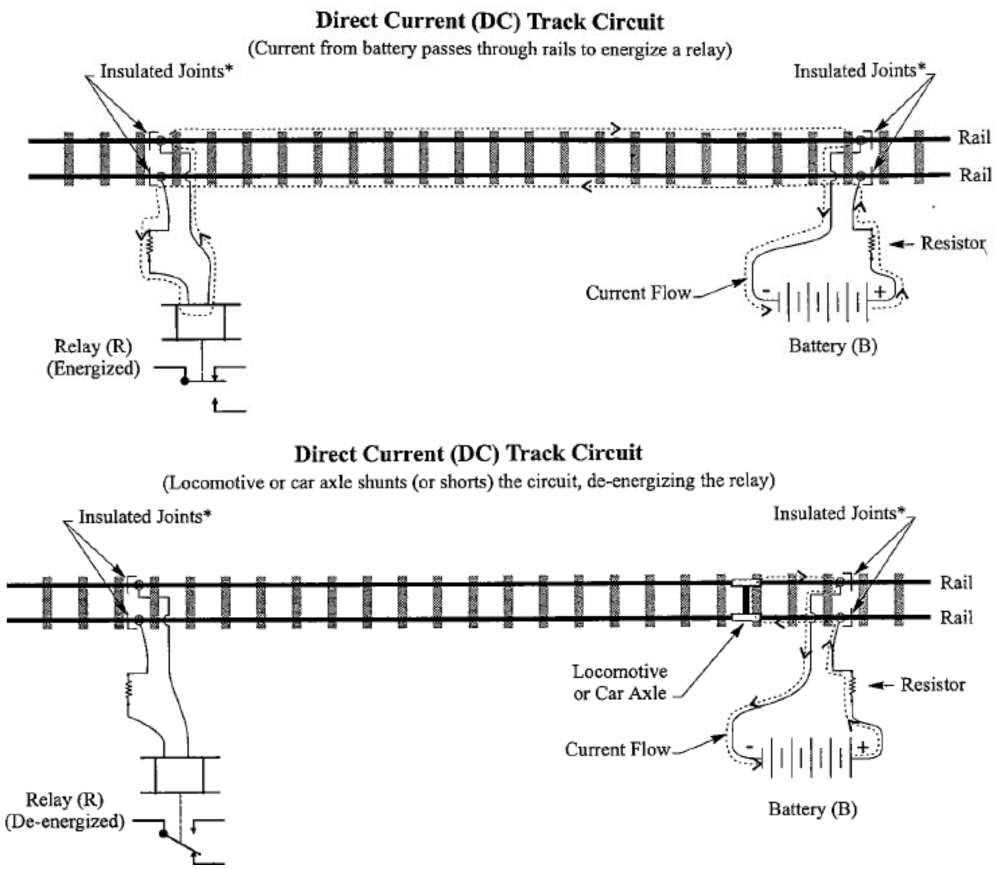


Figure 2. Detection of Train Arrival Using Track-Circuitry ***(4)***

Constant warning time systems also utilize the track circuitry; however, it helps to reduce excessive warning times for motorists caused by a slow moving train by predicting the train arrival time based on position and speed calculations. The grade-crossing warning devices are only activated when the train is approximately located at the preselected minimum warning time away from the crossing. A late-1990's nationwide survey found that constant warning time arrival detection is one of the most common types of track circuits *(4).*

To help improve the reliability of train detection systems, advanced train detection technology is also being developed and utilized. These systems, which do not utilize the standard track circuitry and are often based on tracking of GPS signals from within the train itself, have been developed to provide enhanced reliability of train arrival time prediction and improved cost-effectiveness with respect to installation and maintenance *(4, 17, 18)*.

2.4 Preemption Sequence and Minimum Warning Time

Methods for inputting pre-emption routines depend on the traffic signal controller. Most modern traffic signal controllers allow for several default and user-programmable pre-emption routines including those for railroad crossings *(4)*. For the traffic signal controller to call a train pre-emption mode, data on the train arrival must be received from the interconnection circuit that runs from the railroad equipment to the appropriate interface (i.e., plug in location) on the traffic signal controller. Issues may arise with determining the appropriate interface location on the traffic signal controller as they are frequently inconsistently labeled or not labeled at all *(4)*. All modern traffic signal controller units provide the same basic train pre-emption sequence, which is depicted for a simple two-phase traffic signal and adjacent rail crossing template as shown in Figure 3 *(19)*. The basic pre-emption sequence includes *(4)*:

- Entry into pre-emption mode,
- Termination of the current interval in operation,
- Initiation of "clear track" intervals,
- Initiation of pre-emption hold interval, and
- Return to normal operations.

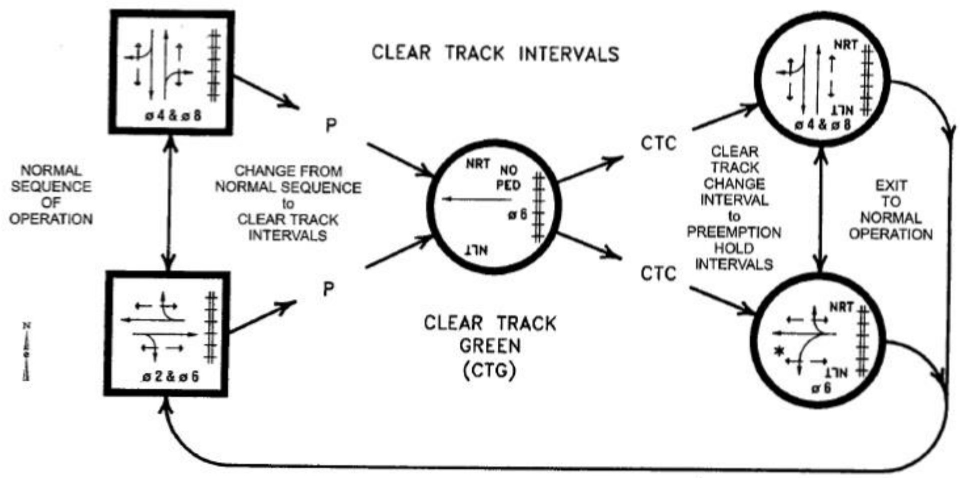


Figure 3 Example Basic Template for Signal Preemption Sequence *(19)*

Federal law requires that a minimum of 20 seconds of warning time to roadway users must be provided by the railroad crossing signals prior to a train arriving at the rail grade crossing *(20)*. The use of pre-emption at an intersection might require detecting the train much sooner than 20 seconds prior to arrival in order to adequately clear vehicles off the tracks. The train detection system must extend far enough along the tracks to detect the fastest allowable train in order to provide 20 seconds of warning time (or more, if pre-emption is needed). Calculation of the maximum amount of pre-emption time needed for the traffic signals depends on several factors, such as the time needed to right-of-way transfer time, queue clearance time, and additional separation time between the queue clearance and train arrival. The pre-emption timeline and railroad crossing timeline are detailed in a generalized schematic in Figure 4 *(2)*.

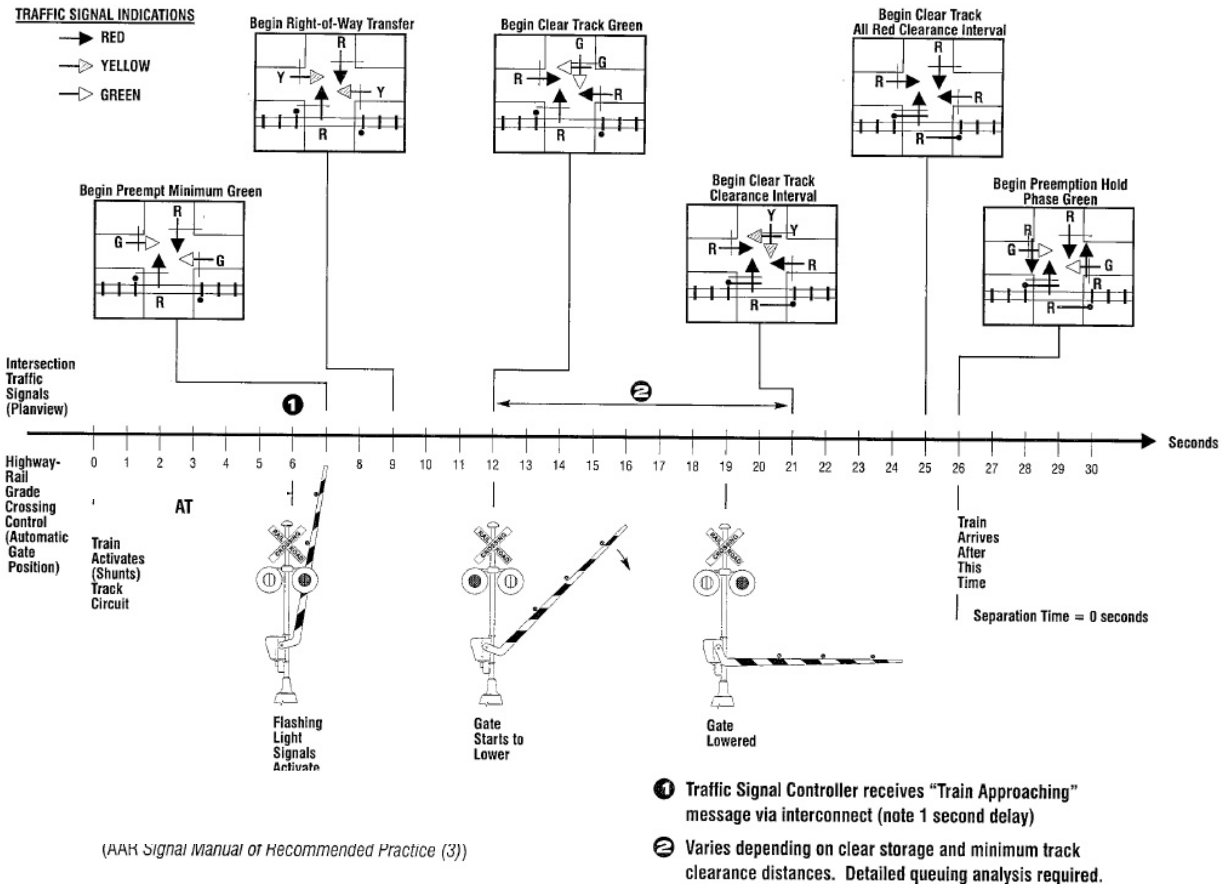


Figure 4. Example Signal Preemption Timeline *(2)*

In order for pre-emption to work effectively, a high level of motorist compliance is necessary both at the railroad grade crossing and the nearby intersection. Motorist violations at the railroad crossing, such as crossing the tracks when the railroad signals are flashing or driving around gates, etc. will increase the risk of crash occurrence no matter how effective the signal pre-emption strategy. Violations at the nearby signalized intersection, such as red-light-running or jaywalking by pedestrians will also have a negative impact on the ability to effectively clear the tracks of any queued vehicles. Motorist compliance at the rail grade crossing would also be improved if train engineers improve compliance with the maximum grade crossing obstruction time limit of 5 minutes (Michigan Compiled Law Service, Railroad Code of 1993, Sec

462.391*(21)*). Drivers may become impatient at railroad crossing locations where trains are known to obstruct the tracks for more than 5 minutes, which may lead to increased levels of non-compliance with the railroad crossing warning lights and/or gates.

The literature has suggested that the greatest challenges associated with safe and efficient pre-emption timing are connected with 1) terminating the current phase when pre-emption is called, particularly if a pedestrian clearance phase is required, and 2) determining adequate track clearance time. While it is not desirable to terminate the pedestrian clearance phase – essentially trapping pedestrians in the intersection – it is allowable per the MUTCD and is often necessary to provide adequate track clearance time. Strategies have been suggested to help alleviate the issues associated with prematurely terminating pedestrian clearance intervals due to train pre-emption, and include special train-activated signing for pedestrians *(4)* and auxiliary detection systems strictly for pedestrian clearance protection *(17, 18, 22)*. Various strategies have been suggested for determination of the track clearance green interval, some of which are based on real-time presence data for the queue storage area *(23)*. The most important considerations for timing of the track-clearance green interval are start-up delay and repositioning time *(24)*, which are largely dependent on vehicle type/length. For example, queues that include only short passenger vehicles cause the longest start-up delays, whereas the same queue distance composed of heavy trucks only will have the longest repositioning time. Strategies have also been suggested for providing the optimal pre-emption timing plan to minimize the impacts on a dense network of signals *(25)*.

A brief description on each interval of preemption sequence is provided in the following sub-sections.

2.4.1 Initiating Preemption Mode– Entry into Preemption

The entry into preemption sequencing starts usually by the signal controller unit immediately upon notification from the railroad detection system that a train is approaching the crossing. The signal for initiating the preemption mode is generated when the electric circuit is de-energized.

2.4.2 Termination of the active phase– Right-of-Way Transfer

Once the entry into preemption mode starts a transition phase is required to transfer the right-of-way from the normal operation mode to the approach where the railroad crossing is located. This transfer/termination of current phase (except for the track clearance phase) should be executed providing a minimum green time, yellow change and red clearance interval, called right-of-way transfer time. The minimum green value is usually provided based on the agency's practice and maybe omitted and the clearance intervals (yellow and all red) according to the MUTCD in section 4D.27.08 *(26)* should not be shortened and/or omitted. To prevent any delays initiating the track clearance green interval, the MUTCD *(26)* allows for two exceptions in normal signal operation;

(i). It allows shortening/truncation of any pedestrian walk and/or pedestrian clearance interval in order to prevent the delays initiating the track clearance green interval. Note, however, that in case of an intersection with high pedestrian activity (such as urban locations), the shortening/truncating the pedestrian walk and/or clearance intervals can put pedestrians on risk by placing them in stranded situation.

(ii). If the preemption call is received at the time signal phase for the approach crossing the tracks is active and in yellow change interval, the signal can turn from yellow to green indication without providing red clearance indication. This may, however, create a possible left-turn trap if the opposing traffic does not have a protected left-turn phasing or an all red clearance interval is not provided *(7).*

In any event, the right-of -way transfer time, as recommended in ITE recommended practice *(7),* should be the longest time determined through considering all phases of the signalized intersection. The longest/maximum time could be the time needed for the worst-case condition before the initiation of track clearance green phase, which should be analyzed by considering any equipment response time to preemption call and any signal phase's minimum green, yellow change and red clearance interval and/or pedestrian walk and clearance intervals for conflicting traffic *(8).*

Several parameters that can impact right-of-way transfer include vehicle clearance intervals, pedestrian intervals, minimum green times, high speed roads, or intersection configuration. Research has shown that one of the most crucial assumptions that designers have to make is to estimate maximum RTT for a new installation *(23).* A research study *(27)* has found that the standard preemption strategy truncated pedestrian clearance phase in 40 percent of railroad preemption calls. Improved strategies that provide advance warning time (at least 90s) can eliminate the possibility of cutting pedestrian clearance phase *(27).*

2.4.3 Clear Track Interval—Track Clearance Phase

As the name implies the track clearance interval is the time needed to clear the queue that has built up between the signalized intersection and the crossing a safe distance away from the tracks. The track clearance phase includes (i). Minimum track clearance green time (ii). Track pedestrian clearance interval (iii). Yellow clearance interval and (iv). All red clearance interval.

Track green time (or queue clearance time) is defined in the FHWA Railroad-Highway Grade Crossing handbook *(8)* as "*The time required for the design vehicle of maximum length* *stopped just inside the minimum track clearance distance to start up, move through, and clear the entire minimum track clearance distance."* Where, the minimum track clearance distance is measured from the gate/warning device/railroad stop line or 12 feet perpendicular to the centerline of the tracks, to 6 feet beyond the tracks measured perpendicular to the far track, either along the center or edge line of the road to get the longer distance (*MUTCD Section 8A.01.).* Figure 5 below illustrates the minimum track clearance distance.

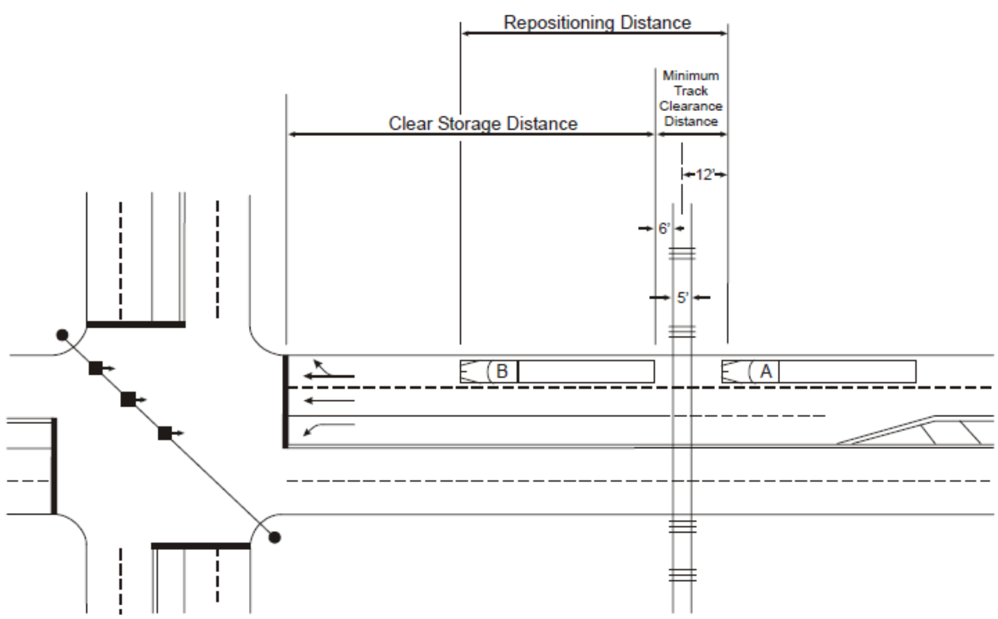


Figure 5. Minimum Track Clearance Distance *(24)*

The track clear green time or queue clearance time should be based on the time necessary for a stopped queue to start-up and acceleration to safely clear the tracks. ITE recommended practice *(7)* points out four crucial considerations while determining the track clearance green time. These include;

- Minimum track clearance distance
- clear storage distance (elaborated in Fig. 3)
- Start-up time of a vehicle within the minimum track clearance distance and the vehicles in the queue ahead;

- Time required for the design vehicle to clear the minimum track clearance distance or clear storage distance (CSD) in case where the design vehicle cannot be safely stored in the CSD.
- ITE recommend practice *(7)* suggests considering a typical separation time ("*component of maximum preemption time during which the minimum track clearance distance is clear of vehicular traffic prior to the arrival of the train"*) of 15 sec.

There are procedures available in the literature that use some mathematical equations for estimating the clear track intervals. These equations are primarily based on driver starting up, acceleration time. The two methods commonly found in the literature are Marshall and Berg procedure *(12)* and Gary Long Method/University of Florida Method *(24).*

2.4.3.1 Marshall and Berg Procedure

The Marshall and Berg procedure *(12)* estimates the time, needed to move a vehicle within a stopped queue, using the shockwave theory in which the rate of queue dissipation is equal to the rate at which the starting wave moves backwards through the queue. The time until the last vehicle starts moving can thus be determined through the following expression.

$t_1=LK_j/2.94s$

Where L = length of a standing queue (in feet) that needs to be cleared between the intersection stop bar and the far gate at the crossing.

k_j = jam density (vpm)

s = saturation flow rate (vph), can be estimated by using Highway Capacity Manual (HCM)

The other component of time which is the time the vehicle takes in accelerating/moving from the tracks to a safe area can be determined through this simple formula

$$t_2 = \frac{\sqrt{2(L + 2D + W)}}{a}$$

Where L represents length of the design vehicle,

D = clearance distance on either side of the tracks as shown in figure 5

W = width of the crossing (ft.)

a = acceleration characteristics/rate for the design vehicle (ft/sec^2)

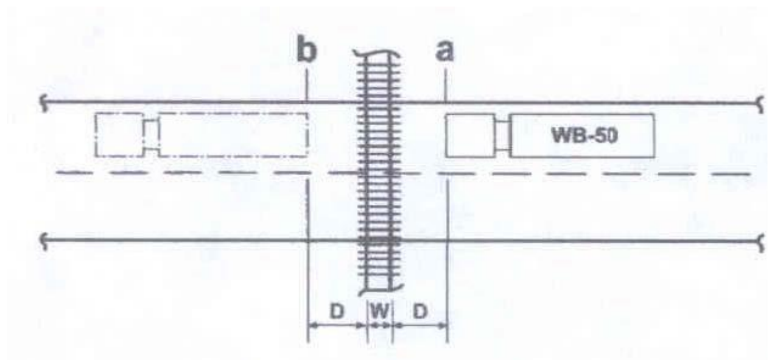


Figure 6. Clearance Distance on either side of the Crossing *(8)*

2.4.3.2 University of Florida Method

The other method known as Gary Long's procedure *(24)* is relatively new and is utilized by Texas Transportation Institute (TTI) in its "Guide for Traffic Signal Preemption near Railroad Grade Crossing"*(28).*

This procedure also involves determining the two components of the time similar to the other procedures. The first component i.e. start-up time is calculated by a simple linear model *(29).*

. $dn = \tau + n * T$

d_n= Start up delay for the nth vehicle in a queue (sec)

τ = Excess startup time of the lead vehicle in a queue (sec)

n = Position of a specific vehicle in a queue (n=1, 2, 3…) and

T = Uniform startup response time of each driver in a queue (sec)

The time should be calculated for the last vehicle in the queue that needs to be cleared, because the drive has to wait for preceding vehicles before he is able to start moving.

TTI in its "Guide for Traffic Signal Preemption near Railroad Grade Crossing" *(28)* provides some recommended values for T and τ. For example for average passenger car and WB 50 truck the values are as follows.

Passenger car: T = 1.2s, τ = 1.0 s

Truck WB 50: T=1.0s, τ = 3.0 s

To determine the time a vehicle takes in accelerating from one point to other, Long's procedure uses the following model *(30)* that predicts the travel distance of the vehicle as a function of the travel time and the grade of the roads. The equation can be solved for travel time (t) by inputting other variables in the model.

$$d = \left(\frac{\alpha \pm Gg}{\beta}\right) t - \frac{\left(\frac{\alpha \pm Gg}{\beta} - v_0\right)\left(1 - e^{-\beta t}\right)}{\beta}$$

Where d = Clearance distance in ft. (measuring from the rear of the vehicle)

t = Vehicle travel time (sec) from a stopped position to the distance'd'.

v_o= Initial speed, set to 0 for starting from rest (fps)

G = Average road surface grade over repositioning distance, set to 0 for flat grade (ft. /ft.)

g = Gravitational constant (32.174 fps^2 near sea-level)

α = Initial vehicle acceleration from rest (fps^2), and

β = rate of reduction in acceleration with increasing speed (sec^{-1})

The values of α and β can either be obtained from collecting the data in the field or else AASHTO provides values of these coefficients for different vehicle classification obtained through various field studies. For example the values provided for passenger cars and WB-50 trucks are as follows.

Passenger car; $\alpha = 6.6$, $\beta = 0.12$

Truck WB 50; $\alpha = 1.2$, $\beta = 0.02$

The procedure provides guidance on selecting the values of excess and uniform star-up time between vehicles and also provide values for acceleration rate coefficients on the basis of vehicle type (pass car, truck) and characteristics (slow, average or fast car).

It should be noted that special considerations are needed in case of any possible delays in clearing the tracks due to any prevailing site specific conditions. These conditions may include (i). Vehicular yielding for any stranded pedestrians (because of truncating/shortening the pedestrian clearance phase during right-of-way transfer). (ii). Absence of protected left turn phasing for the track clearance approach which may cause vehicles turning left to keep yielding for the opposing side vehicles without realizing that the other side has red indication.

Another critical issue is the situation of "preempt trap" which can occur when the track clearance phase ends before the gates and warning devices are activated *(29).* This usually occurs when advance preemption warning time is provided. In order to alleviate the issue of the preempt trap, some kind of "gate down" confirmation logic can be used in the signal controller for ending the track clearance phase.

2.4.4 Holding/Dwell Phase

Dwell/holding phase consists of only that phase/phases of the normal operation mode which can be allowed during the period when the railroad preemption warning devices are active. During the signal preemption sequences, the traffic signal indications should be such that it will prevent vehicles from entering or turning towards the tracks (MUTCD section 8B.08 *(5)*).

To restrict turning movements towards the tracks, changeable message indications (shown in Figure 7) for 'No Right/Left Turn towards the Tracks' or left and right turn signal heads can be used.

Figure 7. Changeable Message Sign to Restrict Turning Movements *(5)*

2.4.5 Return to Normal Operation—*Exit Phase*

The return to normal operations starts after the railroad preemption warning devices are deactivated. The common practice is to return the green phase to the approach crossing the tracks, but it is recommended to give priority to approaches that are expected to have longer queues.

2.5 Minimum Warning Time

A design engineer must carefully determine the minimum warning time to ensure the safe preemption operation at the location. The minimum warning time should be based on the time necessary to terminate the current phase, plus time necessary to clear any vehicle that may be stopped on the track, with an additional safety interval of 4 to 8 seconds *(12).*

The MUTCD Section 8D.06 *(5)* requires that the railroad crossing warning signals start flashing a minimum of 20 seconds before the arrival of a train at the crossing. The AREMA Communication and Signal Manual *(31)* refers to the MUTCD specified 20 second interval as the' minimum time' (MT) and further defines minimum warning time as the sum of the 'minimum time' and clearance time (calculated primarily on the basis of minimum track clearance distance). In addition to that it recommends considering additional time in calculating the minimum warning time, if needed, for equipment response time, buffer time (accounts for variation in train arrival time) and advance preemption time.

To put it together, the minimum warning time should be calculated on the basis of the following elements.

- Preempt Verification and Response Time
- Minimum right-of-way transfer time =Max[(Flashing Don't Walk Interval + All Red), (Min Green +yellow +All Red)]
- Minimum Track Clearance Time (*refer Fig 5*) considering for the design vehicle (longest legal truck combination) worst case scenario
- Additional safety interval (e.g. 4-8 seconds *(12))* to account for variability in the provide warning time

2.6 Pre-Signals

Pre-track signals are often used in Michigan and other states as a means by which to ensure that the tracks remain clear. Pre-signals are positioned prior to the rail grade crossing and the intersection for traffic heading towards the intersection. Pre-signals are coordinated with the intersection traffic signals in a similar manner to the paired signals at a divided roadway intersection, in that the pre-signal changes to red several seconds prior to the signal at the intersection (i.e., the downstream signal), thereby allowing all vehicles ample time to clear the tracks. Pre-signals provide the advantage of keeping the tracks clear during each and every cycle regardless of whether a train is arriving or not. An example of a pre-signal schematic is shown in Figure 8 *(2)*.

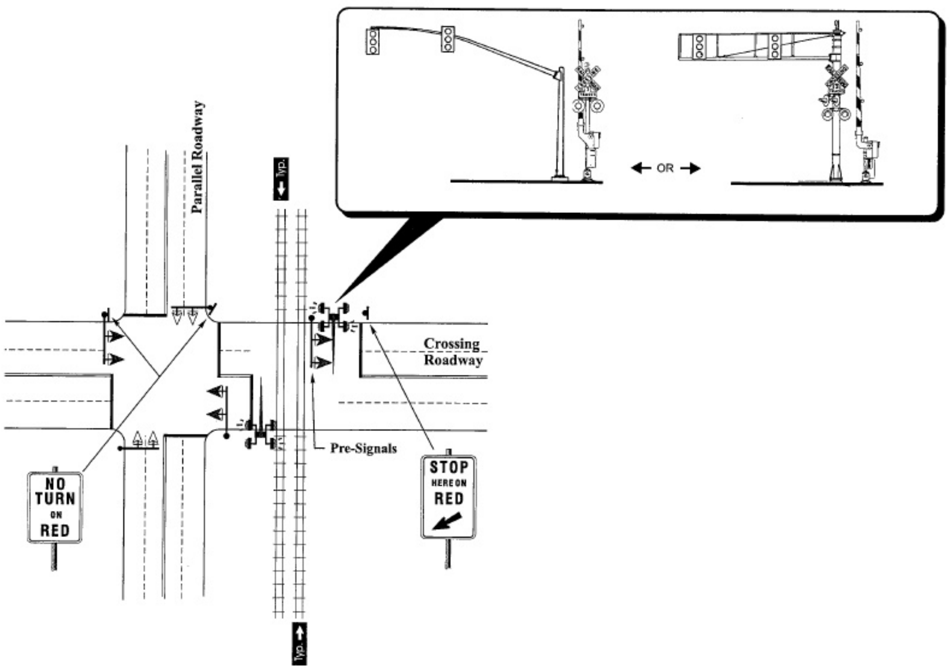


Figure 8. Example of a Pre-Track Traffic Signal Installation *(2)*

2.7 Methods for Estimating Traffic Queues

Research has suggested that a detailed queuing analysis should be performed at locations where traffic signals are located in close proximity to railroad crossings to provide guidance on determining the need for signal preemption. If the resulting 95th percentile queue (the typical performance percentile for preemption consideration *(12)*) reaches the railroad tracks, the traffic signal must be interconnected to the active warning/flashing light system to allow the implementation of safe clearance of automobile traffic before the arrival of a train. The queuing analysis requires data from the critical approach(es) including traffic volumes, number of lanes and turn lane designation, signal phasing and timing, percentage of heavy vehicles etc.

If the field observations of queue lengths during critical traffic hours are not possible, some analytical models and procedures or simulation techniques can be used to determine the queue length estimates. These include;

- ITE Guidelines *(7)*
- Highway Capacity Manual Procedure *(9)*
- Manual of Traffic Signal Design Nomograph *(25)*
- Traffic Analysis Software such as Synchro *(10)*
- Simulation Models (Sim-Traffic, VISSIM,CORSIM or others)

2.7.1 ITE Guidelines

The ITE Guidelines *(7)* recommend using the Northwestern University formula *(32)* that involves a simple deterministic computation of 95th percentile queue length. The formula is as follows.

$L = 2qr\,(1+p)\,25$; if $v/c<0.9$

Where; L= length of queue (ft.)

q=vehicle flow rate (veh/lane/sec)

r=effective red time (red +yellow) (sec)

p=proportion of heavy vehicles in traffic flow (as a decimal)

v = traffic volume on subject approach

c = capacity of subject approach

A parameter of 25 has been used in the formula to account for the effective length (i.e., actual length + headway b/w the cars) of a passenger car. The factor of 2 is used to account for random arrivals of traffic. In cases where the v/c ratio is between 0.90 and 1.0, the following equation applies:

L= 2qr (1+Δx) (1+p) (25)

If the v/c ratio exceeds 1.0 or the intersection is otherwise over saturated, the guide recommends using Highway Capacity Manual analysis or traffic simulation models (e.g., Sim-Traffic) to determine the 95th percentile queue lengths.

2.7.2 Highway Capacity Manual Procedure

The Highway Capacity Manual defines back of queue as "*The back of queue is the number of vehicles that are queued depending on the arrival patterns of vehicles and on the number of vehicles that do not clear the intersection during a given green phase (overflow)".(9)*

The HCM 2000 *(9)* gives the average back-of-queue as the basic measure to calculate the 'percentile' back of queue. In order to determine the back-of-queue at signalized intersections, the manual gives the following basic equation:

$Q=Q_1+Q_2$

Q= maximum distance in vehicles over which queue extends from stop line on average signal cycle (veh),

Q_1= first-term queued vehicles (veh), and

Q_2= second-term queued vehicles (veh)

The definition and computation of each term as described in HCM *(9)* are presented below.

The first term, Q_1 is the average back of queue, determined first by assuming a uniform arrival pattern and then adjusting for the effects of progression for a given lane group. The first term is calculated using the following equation *(9)*:

$$Q_1 = PF_2 \frac{\left({V_L C}/{3600}\right)\left(1-\frac{g}{C}\right)}{1-\left[\frac{\min(1.0, X_L) g}{C}\right]} \quad \text{…………………………………… ………………………Eq. 1}$$

Where; Q_1= first-term queued vehicles (veh),

PF_2= adjustment factor for effects of progression,

VL = lane group flow rate per lane (veh/hr.)

C = cycle length (s)

g = effective green time (s), and

X_L= ratio of flow rate to capacity (V_L/C_L ratio)

Q_1 represents the number of vehicles that arrive during the red phases and during the green phase until the queue has dissipated. The adjustment factor for the effects of progression is calculated by the following equation:

$$PF_2 = \frac{\left(1-\frac{R_P\,g}{C}\right)\left(1-\frac{V_L}{S_L}\right)}{\left(1-\frac{g}{C}\right)[1-R_P\left(\frac{V_L}{S_L}\right)]} \quad \text{……Eq. 2}$$

Where; PF_2= adjustment factor for effects of progression

V_L = lane group flow rate per lane (veh/h)

S_L = lane group saturation flow rate per lane (veh/h)

g= effective green time (s)

C = cycle length (s), and

R_p = platoon ratio [P(C/g)]

The second term, Q_2, is an incremental term associated with randomness of flow and overflow queues that may result because of temporary failures, which can occur even when demand is below capacity. This value can be an approximate cycle overflow queue when there is no initial queue at the start of the analysis period. Initial queue at the start of the analysis period is also accounted for in the second term, Q_2. The equation shown below is used to compute the second term of the average back of queue.

$$Q_2 = 0.25\, C_L\, T\, [(X_L - 1) + \sqrt{(X_L - 1)^2 + \frac{8k_B X_L}{C_L T} + \frac{16k_B Q_{bL}}{(C_L T)^2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Eq. 3}$$

The adjustment factors in the second term are calculated using the following equation:

$K_B = 0.12\, l\, (S_L \frac{g}{3600})^{0.7}$ …….. (pretimed signal operations)

$K_B = 0.101\, l\, (S_L \frac{g}{3600})^{0.6}$……… (actuated signal operations)

Where

K_B = second-term adjustment factor related to early arrivals,

S_L = lane group saturation flow rate per lane (veh/h)

g = effective green time (s), and

l= upstream filtering factor for platoon arrivals

2.7.3 Manual of Traffic Signal Design Nomograph

The nomograph presented in the Manual of Traffic Signal Design *(33)* and also referred in an ITE published research *(12)* can be utilized to estimate the 95th percentile queue for isolated intersections, assuming a random arrival pattern of vehicles (Figure 9).

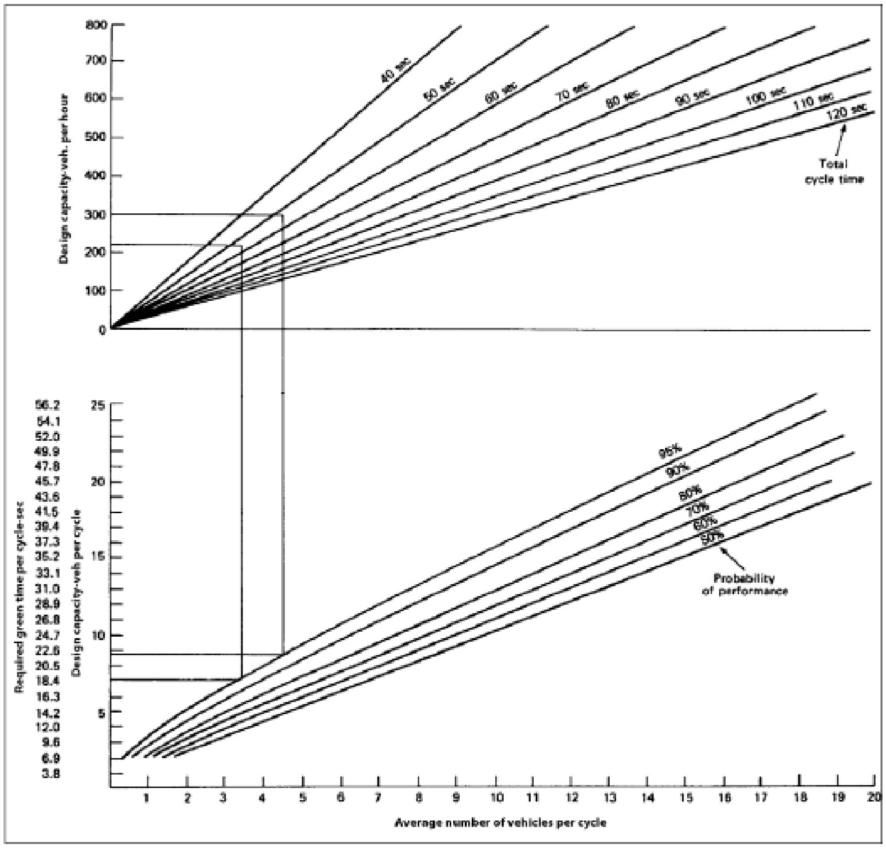


Figure 9. Design Curves for Queue Length Estimate ***(33)***

2.7.4 Synchro Queue Length Model

Synchro software computes 50th and 95th percentile queues. The procedure used in Synchro Version 7.0 to compute 95th percentile queue length as presented in the User Guide *(10)* is as follows.

***<u>Step 1:</u>* Estimate the peak hour lane volume on the critical approach.**

$$v_{crit} = .1\left(\frac{ADT}{N_{dir}N_{lanes}}\right)$$

Where:

v_{crit} = estimated peak hour lane volume on the critical approach (vehicles per hour per lane)

ADT = Average Daily Traffic [User Input]

N_{dir} = Number of traffic directions (1 or 2) [User Input]

N_{lanes} = Number of traffic lanes approaching the intersection [User Input]

***<u>Step 2:</u>* Calculate the 95th percentile lane volume on the critical approach.**

$$v_{crit95} = v_{crit}\left(1 + 1.64\left(\frac{\sqrt{v_{crit} * C/3600}}{v_{crit} * C/3600}\right)\right)$$

Where:

v_{crit95} = estimated 95th percentile peak hour lane volume on the critical approach (vehicles per hour per lane)

C = cycle length (seconds) [User Input]

***<u>Step 3:</u>* Determine if the 95th percentile approach volume represents oversaturated traffic conditions.**

Assume oversaturated traffic conditions if

$v_{crit95} \geq \left(\frac{g}{C}\right)1800$, where g = green time on the critical approach (sec)

Assume undersaturated traffic conditions if

$v_{crit95} < \left(\frac{g}{C}\right)1800$, where g = green time on the critical approach (sec)

***<u>Step 4:</u>* Calculate the length of the 95th percentile queue on the critical approach.**

For undersaturated traffic conditions:

$$Q_{95} = \frac{v_{crit95}}{3600}(C - g - 3)\left(1 + \frac{1}{\frac{1800}{v_{crit95}} - 1}\right) * 25 * \text{Length Adj.}$$

Where:

Q_{95} = estimated 95th percentile peak hour queue length on the critical approach (ft)

C = cycle length (seconds) [User Input]

25 = assumed vehicle storage length in queue (ft)

Length Adj. = Vehicle length adjustment factor to account for trucks

For oversaturated traffic conditions:

$$Q_{95} = 2\left(\frac{v_{crit95} * C}{3600} - \frac{1800 * g^2}{C * 3600}\right) * 25 * \text{Length Adj.}$$

Synchro procedure is not applicable under permitted-protected left turn operation. In addition, this procedure does not capture certain traffic conditions or variations in traffic flow such as "*spillback between intersections, spillback beyond turning bays, forced lane changes, unbalanced lane use for downstream turns, and other subtle traffic flow interactions" (10).*

2.7.6 Stochastic Queuing Models

Stochastic queuing models are those which operate on the basis of some assumed probability distributions. In a stochastic queuing system, the state of the system changes probabilistically over time. Most of the stochastic queuing models are based on classic queuing theories that describe the queuing system (i.e. vehicle arrival and departure) by some probability distributions most commonly Poisson arrival and exponential service times. Also the probability of a system evolving in future depends only on the present state and not on the past events (Markov Chains). Some of the models describing such queuing systems include M/M/s, M/G/s and M/D/s models etc. These are general notations used to describe the distributions used in the queuing model. The first letter refers to the distribution of inter-arrival times, second letter for distribution of service times, and the third letter for number of servers (service providing units) respectively. The three types of distributions that most of the queuing models use are *(34)*.

M= Exponential distribution

D=degenerate or deterministic distribution (constant times)- deterministic arrivals

G= general independent distribution (any arbitrary distribution)

Ek= Erlang distribution

Following the above notations, some examples of the queuing models are M/M/1, M/G/1 and M/D/s queuing systems.

2.7.7 Traffic Simulation Models

A simulation technique is defined by May *(35)* as "*a numerical technique for conducting experiments on a digital computer, which may include stochastic characteristics, be microscopic or macroscopic in nature, and involve mathematical models that describe the behavior of a transportation system over extended periods of real time*".

*"**Microsimulation** is the modeling of individual vehicle movements on a second or sub-second basis for the purpose of assessing the traffic performance of highway and street systems, transit and pedestrians (36)".*

Traffic simulation models involve imitating and running the operation of a system on a computer. The operation of a system is described by various randomly generated events using some probability density functions. By running this imitated system, statistical observations can be obtained of the performance of this system. Several basic formulations are needed to describe the simulation model. These include the following *(34)*.

- *A definition of the state of the system (e.g. number of arrivals/vehicles in a queuing system)*
- *Identify the possible states of the system that can occur*

- *Identify the possible events (e.g. arrivals and service completions in a system) that would change the state of the system*
- *A provision for a simulation clock that will record the passage of time*
- *Random number generating method*
- *A formula for identifying state transitions that are generated by the various kinds of events.*

Once the simulation model runs, a time increment method advances the simulation and keeps record of the events that are generated, updates the system and also record the system performance during that interval. The two methods for time incrementing include fixed time increment and next-event increment. In both cases the desired information on system performance measures is recorded in per the time increment. A brief description on two how these methods operate is as follows.

Fixed-Time Increment: Moves the simulation forward by a fixed time increment and updates the system by utilizing information on events that occurred during this interval and on the current state of the system *(34)*.

Next-Event Increment: This method updates the system on the basis of its current state and also by randomly producing the time increments for occurrence of a new event (not generated earlier) that can result from the current state *(34)*.

Traffic flow is characterized by various factors related to roadway geometry, driver behaviors, and vehicle interactions. Some of these parameters include speed, flow, density, headways, gaps, lane changing behavior, etc. Most of the macroscopic analytical models use average traffic stream characteristics and simple speed, density, flow relationships. With the development of microscopic car following models, it was made possible to incorporate individual driver behavior into the traffic model. The basic assumption behind the microscopic car following model was that a driver behavior following another driver can be modeled on the basis of some established roadway rules.

Microscopic traffic simulation techniques combine various such models, probability distributions and mathematical formulations (a few described above) to model and operate the traffic flow in the system. To model the queue formation, most of these techniques use some probability density functions such as described by the elementary queuing theories e.g. Poisson arrival and exponential inter-arrival times.

The required basic input in simulation modeling includes the intersection geometry, peak hour traffic volumes per movement, traffic signal timing and phasing, percentage of heavy vehicles, speeds etc. The other model parameters that can be calibrated to represent actual traffic conditions include time headways, gap acceptance, turning speeds, lane changing behavior, vehicle characteristics, lane blockage percentage, maximum queues and several others.

2.7.8 Prior Research on Comparing Analytical and Simulation based Methods for Measuring Delays and Queue Lengths

In 1989 Oppenlander *(37)* study on determining design lengths for turn lanes utilized a probability based queuing model that assumed Poisson Arrival and exponential service times with single server to obtain the queue length distributions over various parameters. It was later found that the assumptions were not quite applicable for real conditions. 1994 follow-up research by Oppenlander pointed out the inadequacy of the queuing model used in 1989 work and used a simulation model in his following studies *(14-16)* highlighting the practicality of simulation based techniques for determining the probability distributions of queue lengths under various traffic operations *(6)*.

A New Mexico study in 2011 *(38)* evaluated queue estimation from four different models including HCS+, Synchro, Sim-Traffic and TEAPAC and compared the output to the observed maximum queues at the study locations. The study results found that HCS+ and TEAPAC generally underestimated the observed queues under low v/c ratios whereas over-estimated at high v/c ratios. The also study found Synchro model underestimating the observed queues in most cases. Sim-Traffic simulation model was found most closely matching the observed queues (while also somewhat over-estimating it) in particular by calibrating it for 95th percentile traffic volumes and using a simulation period almost equal to the cycle length at the signalized intersection.

Traffic simulation software such as Sim-Traffic, VISSIM etc. have different capabilities and require different level of coding and calibration however, it is important to know the underlying default parameters and definitions to understand the applicability of each simulation technique when they are well-suited. Sim-Traffic, for example, in various research studies *(38, 39)* have been found to provide quite reasonable/close estimate of queue lengths as compared with the observed queues. Another study that compared two methods (HCS+ and Sim-Traffic) for estimating queue lengths over left-turn lanes concluded that micro-simulation model (SimTraffic) is a better choice in situations where the impact of traffic from adjacent intersections needs to be accounted for left-turn queue estimation *(40)*.

Traffic operation on left-turn lanes is relatively complex as compared to through traffic lanes. A research study on left-turn lane design lengths at signalized intersections *(41)* suggested that effects of lane overflow and blockage can have a significant impact on queue formation particularly under permitted and protected left turn operations, and should be adequately accounted for. The analytical models because of their split phasing assumption are unable to capture the effects of overflow and blockage *(41)*.

Simulations technique can be effectively used to analyze such traffic operations and have been used in the past for determining left-turn lane storage lengths. In a recent study *(39)* different macroscopic methods and microscopic simulation models for estimating design lengths of left-turn lanes are compared. The methods that were investigated included HCS+, Synchro (macroscopic), Sim-Traffic and VISSIM (microscopic) and the model outputs from these models were compared to the actual observed queues. Sim-Traffic among the four was found to provide the closest estimates of left-turn queue lengths as compared with the observed queues. HCS+ was found underestimating the observed left-turn queues while Syncho outputs were closer to Sim-Traffic. VISSIM simulation models in this study was found in most cases over-estimating the left-turn lane queue lengths as compared with the observed queues and those estimated through Sim-Traffic. The reason indicated was that under the left turn lane over flow conditions, the queue counter in VISSIM on left turn lane tends to consider the adjacent through vehicles.

2.8 Summary

The analytical/macroscopic models for estimating queues such as provided in HCM method *(9)*, ITE recommended model *(19)* and Synchro *(10)* are limited to their model assumptions, certain conditions (e.g., under-saturated, uniform arrival), and are unable to capture variations in traffic flow or any traffic conditions that are different from the assumptions used in these models. In addition to that various factors that can impacts delays and queues, cannot be adequately captured in these models. For example, using HCS, various factors that may significantly impact the resulting delays and queues, cannot be captured such as vehicle mix, vehicle interactions between adjacent and opposing lanes, residual queues at the end of a cycle, approach speed, car-following and lane-changing behavior, and complex traffic operations. Synchro model for estimating queues also become invalid with complex phasing such as permitted-protected left turn operation and do not capture any variations due to spillback between intersections, spillback beyond turning bays, forced lane changes, unbalanced lane use for downstream turns, and other subtle traffic flow interactions *(10)*.

The delays computed in HCM method includes average control delay that only accounts for delays due to deceleration, stopped delays, queue start-up and acceleration time. The roadway speed that may impact time spent in deceleration or acceleration are not accounted for in the HCM delay computations. Drivers traveling at high speed roadways may need more time in decelerating than those traveling at lower speeds. Residual queues at the end of a cycle are not added to the next cycle in the HCM based computer tools i.e. HCS, however HCM does provide directions on including them.

Microscopic simulation techniques offer modeling the traffic flow to detailed level and have the capability to capture the impact of all the aforementioned factors, individual driver behaviors and vehicle interactions in a traffic stream and if properly calibrated can replicate the actual traffic conditions. However, there exist several micro-simulations models such as Sim-Traffic, VISSIM, CORSIM etc. and in order to select a particular simulation method for analysis it is important to investigate their applicability, how the performance measures such as delay, queue lengths etc. are defined and computed and if the underlying model assumptions provide reliable analysis and estimates of the desired performance measures particularly in estimating queues at signalized intersections near highway railroad grade crossings for preemption evaluations.

The research in this dissertation is conducted to further investigate the queue computation from the current analytical procedures and advanced micro-simulation software in order to develop recommendations for selecting reliable method when analyzing traffic queues at signalized intersections for preemption evaluation.

CHAPTER 3

CURRENT STATE-OF-THE-PRACTICE

The policies and practices of state transportation agencies, published guidelines, state MUTCDs, and other available documents/manuals providing information on traffic signal preemption near highway-railroad grade crossings and intersections in close proximity were reviewed. The review focused on the following:

1. Signal Preemption Criteria
2. Factors for considering signal preemption
3. Queue Length Determination

3.1 Signal Preemption Criteria

A review of state DOT practices found that a variety of procedures exist to determine locations where railroad preemption should be applied. Most base their decision on the distance between the traffic signal and the railroad crossing – generally 200 feet per the MUTCD. However, some states consider other factors for signal preemption, such as queue length.

The MUTCD *(5)* considers intersection proximity as a critical factor for interconnecting warning devices at the crossing to the nearby traffic signal at the intersection. It states that the interconnection of signals at the highway-railroad grade crossing occur if it is within 200 feet of a signalized intersection or midblock location. All 50 states websites were searched for their MUTCD and other relevant documents/information out of which no information was found online for 15 states. A review of the MUTCD supplements available online found that at least 21 states follow the FHWA MUTCD 200 feet recommendation. For the others, preemption guidance is provided for intersections located at distances greater than 200 feet from the railroad grade crossing. For example, Florida DOT procedure 750-030-002-f *(42)* requires preempting the signals located within 200 feet of rail-highway grade crossing and also suggests preempting the signals at intersections if a railroad grade crossing with active railroad warning devices is located 200 feet to 500 feet upstream based on an engineering study. South Carolina provides similar guidance *(43)* recommending that the preemption be provided for intersections up to 500 feet away from the tracks. Oregon *(44)* guideline requires preempting the traffic signals if they are within 215 feet of the rail grade crossing. Details for each state are summarized in Table 1 at the end of this chapter.

3.2 Consideration of Other Factors

In instances where intersections are located more than 200 feet from the railroad crossing, MUTCD *(5)* provides several factors to be considered for signal preemption. These factors include traffic volumes, vehicle mix, queue lengths, vehicle and train approach speeds and frequency of trains. At least 18 states follow the guidance of the federal MUTCD when determining if interconnection is needed for signalized intersections located beyond 200 feet from the crossing. Some states provide additional guidance for locations beyond 200 feet, including more factors to be considered, details on which are as follows.

Alabama *(45)* recommends warranting the signal preemption (in addition to the intersection proximity within 200 feet) in the following scenarios. *1). "Analysis indicates that vehicle queues from the traffic signal have the potential to extend into or past the rail crossing; 2). Analysis indicates that vehicle queues caused by a passing train have the potential to extend into the signalized intersection and obstruct traffic flow."* It recommends that signal should be preempted if the 95$^{\%}$ maximum queue backs up to within 8ft. of the nearest rail. For estimating queue length with 95 percent probability, Alabama refers to procedure suggested by Marshall and Berg *(12)* i.e. Nomographs from the Manual of Traffic Signal Design *(46)*.

Louisiana traffic signal design manual *(47)* provides the same warrants for providing traffic signal preemption as those recommended by Alabama. For determining the queue lengths and queue clearance time Louisiana points out the potential of significant variation in results using different computational methods from observed queues and therefore suggests field observations to account for real conditions at the site. However, it refers to computational methods that may be needed for future locations or traffic situations.

Minnesota *(48)* in addition to signal proximity within 200 feet of the crossing also recommends preempting the signals if traffic queues are anticipated to back up over the tracks and also if vehicular traffic queued at the crossing for an approaching train reaches and interrupt the flow at a signalized intersection upstream. The method recommended for estimating the 95th percentile queue is the Northwestern University formula *(32)* recommended in ITE guidelines *(7)*. However it is further stated in the Minnesota design manual *(48)* that "*This formula cannot be used if the v/c Queue lengths for through traffic and for left turns should both be checked to determine which queue is the most critical".* In addition, this formula is not applicable for oversaturated conditions. Under such conditions, the ITE guidelines *(7)* suggests using HCM procedure *(9)* or traffic simulation model.

Ohio DOT's additional guidelines *(49)* for considering signal preemption suggest field observations of critical traffic queues. The guidelines further state that "*Queue arrival and dissipation studies or capacity analysis may be beneficial in further refining the observed queue lengths*." Ohio also recommends considering the presence of special use vehicles such as "*vehicles which haul hazardous materials, school buses or public transportation vehicles*" for assessing the need for signal preemption *(49)*.

If the crossing is more than 200 feet away from signalized intersection Tennessee *(50)* recommends providing signal preemption if queues routinely back up over the crossing. It also recommends considering preemption if the vehicular traffic queued for a train arrival at the rail crossing spills back into the intersection. Tennessee recommendations for queuing analysis include determining the 95th percentile queue lengths both in the lanes for thru and turning movements. For queue length estimation, it also suggests using the ITE recommended formula presented earlier.

Georgia *(51)* suggests giving consideration to *"frequency and duration of trains, volume of vehicular traffic, distance to the crossing and frequency of vehicular queues, the complexity of existing signal system/phasing and whether opportunities exist to serve certain movements effectively during the period when trains are using the crossing, and the spacing of traffic signals with short block lengths."*

The current Texas MUTCD follows the FHWA MUTCD guideline for preempting the signalized intersections within 200 feet of a railroad crossing. However, a report prepared by researchers at the Texas Transportation Institute *(28)* claimed the following: "A draft of the upcoming new release of the MUTCD suggests the queuing study should be performed when highway-rail intersections are located within 1000 feet of a signalized intersection."

The South Carolina DOT in its Traffic Signal Design Guidelines (Chapter 6) *(43)* addresses railroad preemption design. The factors and issues they described for consideration for interconnection of signals other than intersection proximity include:

1. *"Queuing regularly occurs within Track*

2. *Clearance Distance*
3. *Signal timing adjustments do not resolve regularity of queuing*
4. *Signal timings that are needed to serve motor vehicles result in queuing across crossing*
5. *Active railroad warning devices are existing or planned*
6. *Train speeds exceed 20 mph"*

For queuing analysis the South Carolina's signal design manual further suggests to either observe them during peak times or it suggests using Marshall and Berg's procedure *(12)* for queue estimation.

California MUTCD *(52)* in addition to a signalized intersection being within 200 feet of an active rail-highway grade crossing, lists several other scenarios when preemption of traffic signal near railroad grade crossing is needed. The scenarios include:

1. *"Where the railroad tracks run within a roadway and train speeds exceed 10 mph"*
2. *"Where the railroad tracks run along a roadway of a signalized intersection and train speeds do not exceed 10mph"*
3. *"Unusual or unique track or roadway configurations"*

The Washington State DOT *(53)* signal design manual recommends considering/evaluating those railroad crossings for signal preemption that are within 500 feet of a signalized intersection. Their guidelines provide several factors be considered for interconnection of such as 1) intersection proximity 200 feet or less (distance measured from the stop bar to the nearest rail), 2) 95% maximum queue length (determined through queuing study or traffic simulation) reaching the tracks from stop bar, or 3) queue lengths affecting the upstream traffic signal.

3.3 Summary

MUTCD 200 feet rule for providing signal preemption has been followed by most of the states in their MUTCD supplements. Several states provide additional guidance for considering signal preemption for intersections located more than 200 feet away from the grade crossing. The additional guidance primarily includes analysis of traffic queue spill back to the tracks and also if the traffic stopped for train backs up to the adjacent intersection upstream. Only few states provide further guideline on queue estimation if field observations are not possible. The procedures recommended by them include the analytical formula recommended in the ITE guidelines *(7)* or the procedure recommended in another ITE publication *(12)* that suggests using the nomograph presented in the Manual of Traffic Signal Design *(33)*. Only one state guideline document included the use of traffic simulation method to determine queue lengths for preemption evaluation.

A summary of the guidelines provided by each state as found from their MUTCD supplement or other guideline documents is presented in table 1 below.

Table 1. Summary of State DOT Practices from Design Manuals and MUTCD Supplements

State	Guideline for storage distance more than 200 feet	Guideline on Queue Estimation	Source
Alabama	1). queues from the traffic signal have the potential to extend into or past the rail crossing; 2). Analysis indicates that vehicle queues caused by a passing train have the potential to extend into the signalized intersection and obstruct traffic flow. Signal should be preempted if the 95% maximum queue backs up to within 8ft. of the nearest rail.	Nomographs from the Manual of Traffic Signal Design Manual	Traffic Signal Design Guide and Timing Manual (2007)
California	1. Where the railroad tracks run within a roadway and train speeds exceed 10 mph 2. Where the railroad tracks run along a roadway of a signalized intersection and train speeds do not exceed 10mph 3. Unusual or unique track or roadway configurations		California MUTCD (2014 Edition)
Connecticut	At signalized intersections where the rail crossing is greater than 200 feet (60 m), but high vehicular volumes are expected, a queue analysis should be conducted to ascertain if pre-emption is required.		Traffic Control Signal Design Manual 2009
Florida	Consideration for preemption should be given from 200' to 500' based on engineering study.		Signalization Pre-emption Design Standards (Topic No.:750-030-002-*f*)
Georgia	Frequency and duration of trains, volume of vehicular traffic, distance to the crossing and frequency of vehicular queues, the complexity of existing signal system/phasing and whether opportunities exist to serve certain movements effectively during the period when trains are using the crossing, and the spacing of traffic signals with short block lengths.		Traffic Signal Design Guidelines (2013)
Louisiana	Traffic from the signal is observed to back up across the railroad tracks, preemption should be used. When traffic stopped for a train at the grade crossing frequently backs up into a nearby signalized intersection, preemption may be considered.	Field Observation Method. Computations for future intersections/traffic conditions for which a similar location cannot be found for observation.	Louisiana DOTD Traffic Signal Design Manual
Maryland	Same as FHWA MUTCD		Maryland MUTCD 2011
Minnesota	If traffic queues are anticipated to back up over the tracks and also if vehicular traffic queued at the crossing for an approaching train reaches and interrupt the flow at a signalized intersection upstream.	ITE recommended formula for 95th percentile queue estimation. However, not applicable if queue length needs to be estimated both on thru and left turn lanes.	Minnesota Traffic manual
Ohio	Field observation of Critical Traffic Queues. Ohio also recommends considering the presence of special use vehicles such as “vehicles which haul hazardous materials, school buses or public transportation vehicles” for assessing the need for signal preemption	Field observations of critical traffic queues. Queue arrival and dissipation studies or capacity analysis may be beneficial in further refining the observed	Ohio Traffic manual 2002

		queue lengths.	
Oregon	Oregon guideline requires preempting the traffic signals if they are within 215 feet of the rail grade crossing.		Oregon Railroad preemption Design and Operation
South Carolina	1. “Queuing regularly occurs within Track 2. Clearance Distance 3. Signal timing adjustments do not resolve regularity of queuing 4. Signal timings that are needed to serve motor vehicles result in queuing across crossing 5. Active railroad warning devices are existing or planned 6. Train speeds exceed 20 mph”	Field observation during peak times or estimate using Marshall and Berg’s procedure i.e. (Nomograph from Traffic Signal Design Manual)	SCDOT Traffic Signal Design Guidelines (2009 Edition)
Tennessee	If queues routinely back up over the crossing or if the vehicular traffic queued for a train arrival at the rail crossing spills back into the intersection.	Determining the 95th percentile queue lengths both in the lanes for thru and turning movements. For queue length estimation, use the ITE recommended formula.	TDOT Traffic Design Manual (2012)
Washington	Recommends considering/evaluating those railroad crossings for signal preemption that are within 500 feet of a signalized intersection. 95% max queue reaching the tracks from stop bar, or queue lengths affecting the upstream traffic signal.	95% maximum queue length (determined through queuing study or traffic simulation)	WSDOT Design Manual (2011), Division 13
Wyoming	Under Development		Wyoming Traffic Studies Manual
Alaska, Arizona, Colorado, Delaware, Idaho, Illinois, Iowa, Maryland, Massachussettes, Michigan, Missouri, Nebraska, New York, New Jersey, New Mexico, North Carolina, Pennsylvania, Texas, Utah, Virginia, Wisconsin	Same as FHWA MUTCD		
Arkansas, Hawaii, Kansas, Kentucky, Maine, Mississippi, Montana, Nevada, New Hampshire, North Dakota, Oklahoma, Rhode Island, South Dakota, Vermont, West Virginia	Not Found		

3.4 Survey of States' Practices

The state-of-the-practice review from online available states manuals and guidelines did not provide detailed information related to procedures used/recommended for estimating queue lengths at signalized intersections. In order to obtain more specific information related to railroad preemption practices, an online survey was developed and distributed to states agencies. The survey questions were mainly directed to get information on the criteria used to determine the need for signal preemption and methodologies used for queue estimation at signalized intersections near highway-rail grade crossing. Several other questions were also included in the survey related to preemption design. However, this chapter only includes the survey findings related to signal preemption criteria and queue estimation procedures used in practice. A total of 17 survey responses were received. The survey questions along with the findings are presented below. The numbers and percentages presented on the charts are inter-related as multiple option selection was provided in the survey.

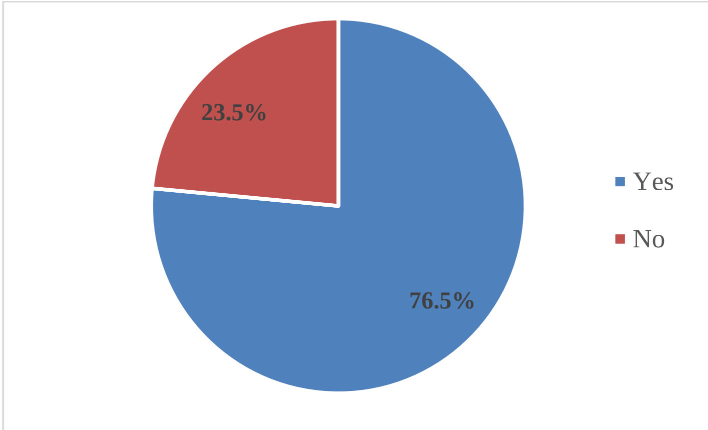
3.4.1 Survey Findings

1. *Candidate intersections for Interconnection*

The first question inquired if the agencies have maintained any guidelines or standards for selecting candidate intersections for interconnecting traffic signals with active warning devices at nearby highway-railroad grade crossings

- Most of the states were found to have maintained guidelines or standards in this regard.

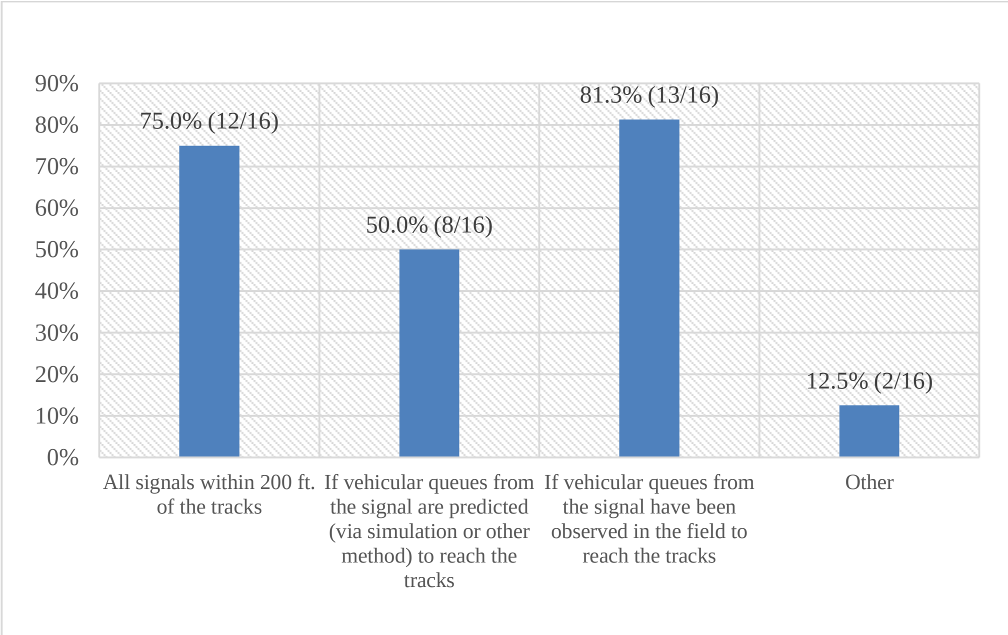
  The percentage of responses is presented below.



2. *Criteria for determining the need for interconnection*

Second question asked about the criteria that is followed by the agency to determine if traffic signal needs to be interconnected with active warning devices at nearby highway-railroad grade crossing

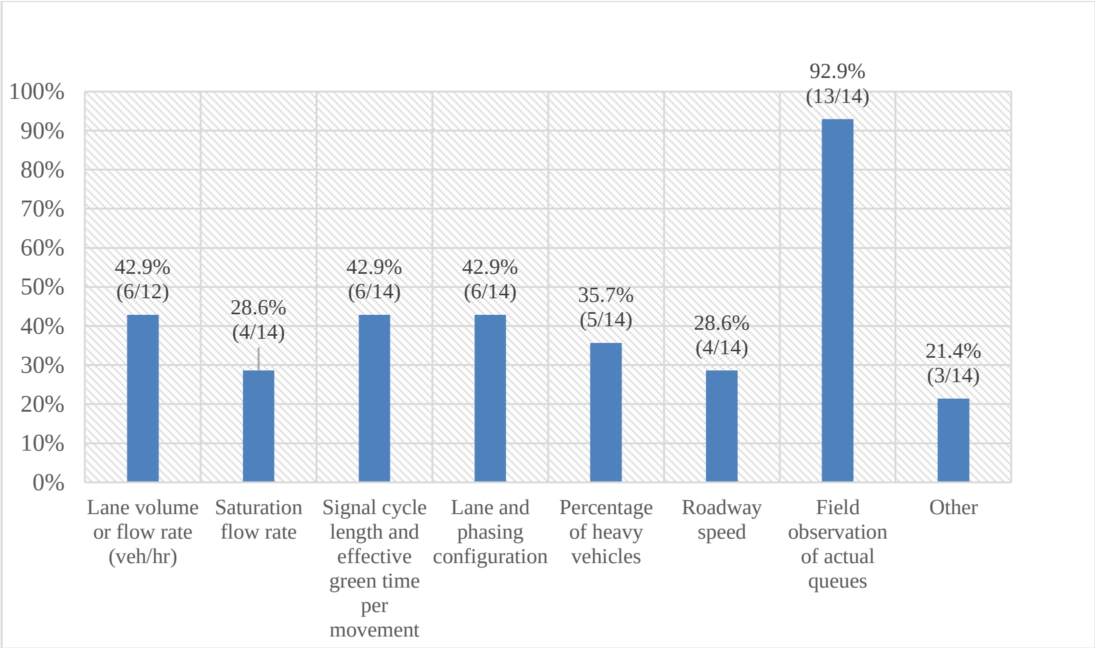
- Most of the responses (11 out of 17) reported that their state agencies follow both 200 ft. threshold as well as if traffic queues observed in the field are reaching the tracks. A few states in addition to above also included using the simulation or other method to predict if traffic queues from the signal reach the tracks.
- Only one state (South Dakota) reported the queue criteria only (not including the 200 feet minimum distance) as the basis for determining the interconnection; queues either observed or predicted through some method.
- The number and percentages of responses are presented in the chart below.



- Two states that specified additional information (Other category) included Utah and Illinois.
  - Utah Department of Transportation reported their guidelines as under development.
  - Illinois in addition to 200 feet and queue criteria provided additional information on this question and recommended treating locations beyond 200 ft. individually. A "rule of thumb" was provided for considering interconnection for such locations i.e. if queue extends across the tracks or predicted to reach up to approximately 350 ft. It was further pointed out that it may be impractical to interconnect the traffic signal and railroad grade crossing when they are at greater distances. The reasons specified included "the unpredictability of traffic dissipation both upstream and downstream of the signalized intersection, the potential for causing increased traffic congestion leading to more traffic queuing at the crossing or other nearby crossings". It was recommended (for locations at greater distances) to mitigate the traffic queuing such as signal timing adjustment, coordination with other adjacent signals or by eliminating the factors that are contributing to queue formation.

3. *Factors used for estimating queue lengths*

Question 3 asked what factors are considered by the agencies for determining queue lengths from the traffic signal. Thirteen out of seventeen respondents answered this question. The factors that the respondents specified and the corresponding number of responses for selecting those factors are presented in the figure below.



Other information/comments on this question are as follows:

– Illinois additionally reported that field observations are used at their existing intersections because they are the most reliable. It further suggested that all the above factors can be used for queue prediction for a proposed signal installation.

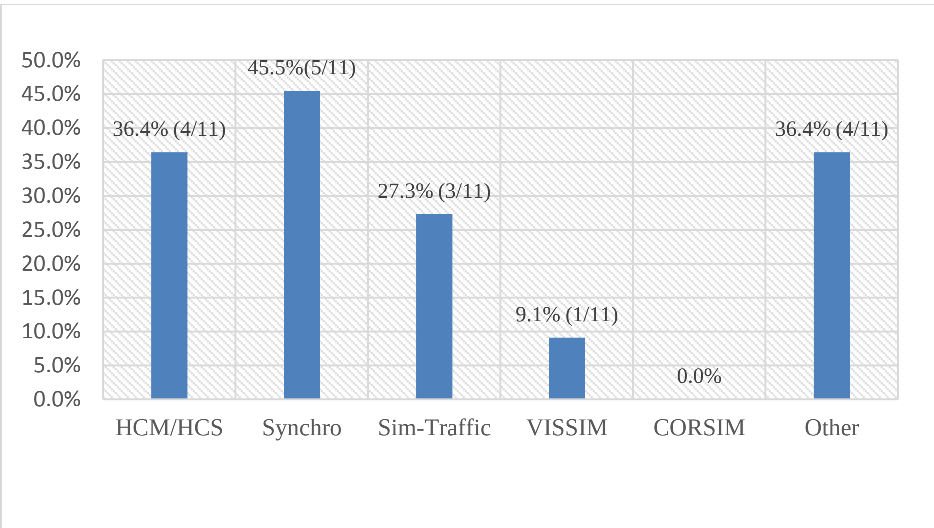
– Mississippi DOT- also specified field observation.

– Utah DOT: Guidelines under development

4. *Queue Estimation Procedure*

Next question asked what analytical procedures or computer tools the agencies use for estimating queue lengths at signalized intersections which are located in close proximity to active highway-railroad grade crossings.

Ten respondents answered this question. Highway Capacity Manual/Software procedure, Synchro and Sim-Traffic were reported most frequently. South Dakota reported use of all methods HCS, Synchro, Sim-Traffic, VISSIM. The methods number of responses along with the percentage of using each method are presented in the chart below.



The 4 responses related to "Other" category were received from agencies of Illinois, Mississippi, Ohio and Utah. The details are as follows.

- Illinois Commerce Commission reported that field observation for existing intersections are used however, highway agencies generally use HCS primarily and also simulation software tools.
- MSDOT-field observation currently

- Ohio Rail Development Commission- engaged a consultant to evaluate existing crossings that meet distance or queue criteria.
- UDOT-guidelines under development

5. *Calibrating/Validating the Queue Estimation Method*

Next question asked the respondents if they calibrate and validate the model they use for the queue estimation. Only 5 out of those respondents that answered question no. answered this question. The options to this question included the following.

- Calibration (i.e. using field data from your jurisdiction)

  Validation (i.e. comparing queue length estimation to actual observed queues)

The individual responses on this question along with the method used are presented in the following table.

Table 2. Survey Responses by State Agencies on Queue Estimation Procedures

Agency	Queue Estimation Method Used	Calibration/Validating the Model
DelDOT	HCM/HCS, Synchro, Sim-Traffic	Validation
NCDOT	HCM/HCS	Validation
MnDOT	Synchro, Sim-Trffic	Calibration
SCDOT	Synchro	Calibration & Validation Mostly determine need by field observations, however, SCDOT uses Synchro for signal timing and roadway design predictions. For signal timing both of the above methods are used. This would also be the case when used for determining need for railroad preemption.
SDDOT	HCS/HCM, Synchro, Sim-Traffic, VISSIM	Calibration
VDOT	HCM/HCS	Did not respond
Ohio DOT	Synchro	Neither Calibration nor Validation typically used

CHAPTER 4

ANALYSIS OF EXISTING QUEUE ESTIMATION METHODS

The survey responses from state DOTs showed that both analytical and simulation models are used in practice for estimating traffic queue spill back from signalized intersections to the nearby railroad-highway grade crossing. The analytical models that are mostly used include HCS and Synchro while simulation based methods, including, Sim-Traffic and VISSIM are used less frequently.

The review of state-of-the-art presented in chapter 2 reveled several limitations of the deterministic analytical models (e.g. HCS, Synchro) in accounting for the impacts of various traffic factors that can impact resulting delays and queues such as variations in traffic flow, vehicle characteristics, vehicle position within the queue, vehicle interactions between adjacent and opposing lanes, residual queues at the end of a cycle, approach speeds, driver behaviors related to car-following and lane-changing, and complex signal phasing or traffic operations. The queue estimation formulas used in capacity analysis software Synchro also become invalid under complex phasing such as permitted-protected left turns and do not capture any variations due to spillback between intersections, spillback beyond turning bays, forced lane changes, unbalanced lane use for downstream turns, and other subtle traffic flow interactions *(10)*.

Microscopic simulation models are considered more practical approach for analyzing traffic operations because of their ability to be calibrated and more accurately represent actual traffic conditions by using stochastic processes. These techniques integrate wide range of traffic factors and mathematical formulations to model traffic flow and offer calibration of many of these factors to represent actual traffic conditions. Various traffic factors that can be calibrated after traffic network coding include car-following related parameters (time headways), saturation flow rates, lane changing and driver behavior related parameters, vehicle characteristics and several others. Therefore these models have the capability to capture the impact of all the aforementioned factors for which simple analytical models are insensitive, and if properly calibrated can replicate the actual traffic conditions.

However, there exist several micro-simulations models such as Sim-Traffic, VISSIM, CORSIM etc. and in order to select a particular simulation method for analysis it is important to investigate their applicability, how the performance measures such as delay, queue lengths etc. are defined and computed and if the underlying model assumptions provide reliable analysis and estimates of the desired performance measures particularly in estimating queues at signalized intersections near highway railroad grade crossings for preemption evaluations.

4.1 Methodology

The analyses performed in this chapter include comparisons between queue lengths obtained from simulation based and other analytical based methods. The differences between each model assumptions and default parameters are identified and the impact of various traffic factors on the resulting queue lengths is analyzed. No field observed data is used to calibrate the models or validate the outcome. Vehicular delays and queues at signalized intersections are the result of many traffic factors, driving behaviors and interactions of vehicles with other vehicles in the network and system components such as traffic control devices. It is important to know that extensive field study/data are required to capture all such factors and traffic behaviors to adequately calibrate and validate the model. Data will typically include (but not limited to) the peak/critical hours data on arrival flow, saturation flow rates, signal timing and phasing, intersection and crossing geometry, vehicle-mix, vehicle lengths and spacing between queued vehicles, time headways, queue lengths, lane changing and other car-following driver behaviors etc. Also, there may be cases where the field data would not be available or hard to obtain such as for a proposed location or due to any practical difficulties. In this regard, the ability of traffic simulation to combine many analytical formulations together to describe various aspects of traffic flow and dynamically represent their interaction makes it a valuable and feasible technique for practically analyzing traffic operations.

This chapter mainly focuses on first analyzing existing micro-simulation based techniques for determining traffic queue lengths at signalized intersections, understanding their underlying car-following models and assumptions, default parameters, and determining their applicability under different traffic operations. The simulation based methods that are analyzed and compared for queue estimation in this chapter include two frequently used microscopic simulation models; VISSIM and SimTraffic. Later in this chapter, additional comparisons are made between the simulation models and several commonly used analytical computer tools such as HCS, Synchro to find the significance of differences in queue estimation. A brief description on each model characteristics, capabilities and queue computation is as follows.

4.2 Sim Traffic

SimTraffic is a widely used microscopic traffic simulation computer tool by Trafficware Corporation. It is integrated with Synchro software package and is used to simulate the traffic network based on Synchro inputs.

Queue lengths in SimTraffic are reported as average, 95th percentile and maximum observed queue during the analysis period for each lane. For queue length computations SimTraffic uses a front-of-vehicle to front-of-vehicle average passenger car length of 19.5 feet (6 meters). Queue definition used in Sim Traffic are presented in table 4.

In the Sim-Traffic Model, vehicles appear in any time interval and the number of vehicle arrivals over many time periods exhibit a Poisson distribution *(10)*.

The following parameters along with their defaults describe driver car following behavior and vehicle characteristics in the model.

Vehicle lengths

Car: 14ft. or 16ft.

Front-of-vehicle to front-of-vehicle length (including inter-vehicular spacing) for pc-19.5 feet

SU Truck: 35 ft.

Semi-Truck 1: 53 ft.

Truck DB: 64 ft.

Bus: 40 ft.

Driver Characteristics

Sim-Traffic uses 10 different driver types depending on the car-following behavior parameters including headway factor, gap acceptance factor, and factors defining lane changing and deceleration characteristics.

Gap Acceptance Factor- varies from 0.85-1.15

Headway @ 0mph: 0.35-0.65

Headway @20mph: varies from 0.8 to 1.80

Headway@50mph/80mph (s): varies from 1.00 to 2.20

Yellow deceleration (ft/s^2): varies between $7 ft/s^2$ to $12 ft/s^2$

Sim-Traffic reports Total delays which is equal to the total travel time minus the travel time for the vehicle with no other vehicles or traffic control devices. For each time slice of animation, the incremental delay is determined with the following formula *(10)*.

TD=dT*(spdmax-spd)/spdmax= Total Delay for time slice

Where

DT=time slice=0.1s

spdmax=max speed of vehicle

spd=actual speed

Total delay also includes delays due to reduced speed during turning, decelerating while approaching a turn and accelerating after taking a turn.

Delay per vehicle=TD/no. Of vehicles

4.3 VISSIM

"*VISSIM is a microscopic, time increment oriented, and behavior-based simulation tool for modeling urban and rural traffic as well as pedestrian flows*" *(54).*

For 'number of vehicular arrival' in each interval, VISSIM uses Poisson distribution ("*the distribution that expresses the probability of a given number of events occurring in a fixed interval of time and/or space if these events occur with a constant rate per time unit and independently of the time since the last event"*). The time headway distribution between each vehicle arrivals is based on a negative exponential distribution *(the probability distribution that describes the time between events in a Poisson process).* Varying number of vehicles are generated in each interval if varying random seeds are specified. During the simulation, the random number generator is queried multiple times to generate stochastic number of vehicle arrivals. This variation in volumes can go up to 10% across varying random seeds.

With regard to model specifications, VISSIM micro-simulation software offers several flexibilities in specifying model inputs and outputs providing the user to model the links and connectors with any level of complexity and modify individual vehicular and driver related characteristics.

For queue length measurements at signalized intersections, queue counters are used (at the stop bar locations) and queues are measured from the counters to the last vehicle in the queue position and can be evaluated for any small period of time *(54)*. The output includes average and maximum observed queue lengths over the time step specified. VISSIM version 7 has incorporated a percentile function to get a specific percentile value of queue lengths in addition to average and maximum observed queue lengths. For this research the 95th percentile queue has been used. Traffic queue definition and other parameters used for queue length computation in VISSIM are summarized in table 4.

The car-following model used in VISSIM is a psycho-physical model developed by Wiedemann at the Karlsruhe Institute of Technology (1974) and has been calibrated continuously ensuring the changing driving behaviors and vehicle characteristics are incorporated *(54)*. The following 4 driving states are assumed by the model.

Free Driving State: *"No influence of preceding vehicles can be observed. In this state, the driver seeks to reach and maintain his desired speed. In reality, the speed in free driving will vary due to imperfect throttle control. It will always oscillate around the desired speed."*

Approaching: *"Process of the driver adapting his speed to the lower speed of a preceding vehicle. While approaching, the driver decelerates, so that there is no difference in speed once he reaches the desired safety distance."*

Following: *"The driver follows the preceding car without consciously decelerating or accelerating. He keeps the safety distance more or less constant. However, again due to imperfect throttle control, the difference in speed oscillates around zero."*

***Braking**: "Driver applies medium to high deceleration rates if distance to the preceding falls below the desired safety distance. This can happen if the driver of the preceding vehicle abruptly changes his speed or the driver of a third vehicle changes lanes to squeeze in between two vehicles."*

For queue computations, two basic base model parameters that specify the safety distance between the vehicles and saturation flow rates include

- Additive Impact on Safety Distance
- Multiplicative Impact on Safety Distance

The definitions of these parameters are described in the table below.

Table 3. Wiedemann Car-Following Model Parameters *(54)*

Parameter	Description
Average standstill distance	Defines the average desired distance between two cars. It has a variation between -1.0 m and +1.0 m which is normally distributed at around 0.0 m with a standard deviation of 0.3 m.
Additive part of safety distance	Value used for the computation of the desired safety distance d. Allows to adjust the time requirement values.
Multiplicative part of safety distance	Value used for the computation of the desired safety distance d. Allows to adjust the time requirement values.

In addition to the above factors, speed, percentage of heavy vehicles and number of lanes also affect the saturation flow rates *(54)*. Various parameters related to car-following and other driving behaviors (such as lane changing) are integrated in VISSIM modeling technique (figure 10 below). The values for these parameters can be overridden according to the site specific data.

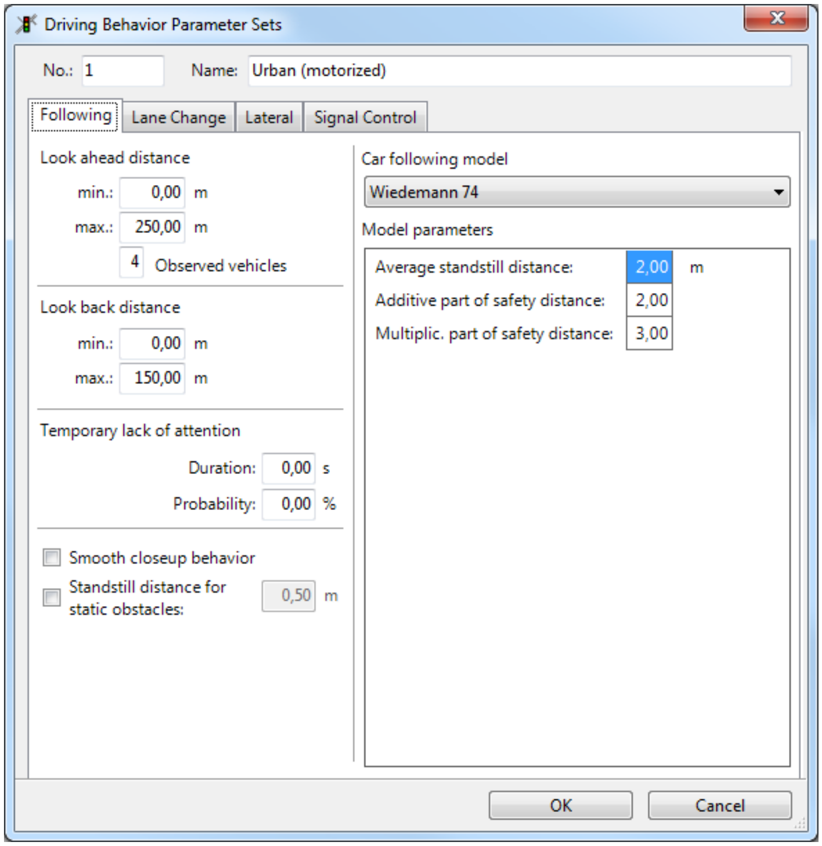


Figure 10. Driver Behavior Parameter Sets in VISSIM

4.4 Queue Definitions and Parameters used in VISSIM and Sim-Traffic Simulation Models

A brief comparison of some of the factors relating to queue formation in both VISSIM and Sim-Traffic simulation models are presented in table 4 below. Because of the differences in definitions and parameters used in queue computation, there are variations in results obtained from different simulation models. Simulation models are calibrated to represent actual conditions for specific location(s) and so the output may be valid for specific site characteristics. This is why it is important to understand underlying model parameters and definitions if model outputs are to be used for design and operational decisions.

Table 4. Queue Definition and Parameters in Sim-Traffic and VISSIM Simulation Models *(54, 55)*

Parameter	Sim-Traffic	VISSIM
Average Queue	The average queue is the average of the observed maximum queues for each time step i.e. two-minute periods.	**AVEQ=ΣQ(i)/I** AVEQ = Average Back Of Queue over analysis period Q(i) = Observed Back of Queue length at end of time step (i) I = Total number of time steps in analysis period
95 percentile Queue Computation	1.65 standard deviations above the average queue	Computed by using percentile function incorporated in version 7.0. 95^{th} percentile of maximum queue observed in each time step during the simulation period.
Vehicle Lengths	For queue length computations Sim-Traffic uses a front-of-vehicle to front-of-vehicle length of 19.5 feet (6 meters) for passenger car.	The maximum vehicle (defaults) Car length: 12.30 to 15.62 ft. HGV:33.51 ft. Average spacing between queued vehicles (default)-2.0m
Queue Definition	A vehicle is considered queued when it is traveling at less than 10 ft/s (3 m/s) and either at a stop bar or following another queued vehicle. Thus, single vehicle queues are not possible except at intersection stop bars.	VISSIM allows the user to define a queue according to the maximum vehicle speed for the beginning of the queue (default is 5 km/hr), the minimum vehicle speed at its end (default is 10 km/hr), and the maximum spacing between vehicles.

4.5 Vehicle Lengths and Inter-vehicular Spacing in Sim-Traffic and VISSIM Models

It is particularly important to recognize that the default vehicle lengths used in VISSIM are shorter than those assumed in Sim-Traffic. The default values assumed for passenger car, heavy vehicles and inter-vehicular spacing are as follows.

VISSIM
Car length: 12.30 to 15.62 ft.
HGV: 33.51 ft.
Space between queued vehicles (Average Standstill distance) - 2.00m (with some stochastic variation)

Sim-Traffic
Car: 14ft. or 16ft.
Distance between stopped vehicles- 5ft. (1.5m)
Average vehicle length (including inter-vehicular spacing) for pc-19.5 feet (6m)
SU Truck: 35 ft.

Semi-Truck 1: 53 ft.
Truck DB: 64 ft.

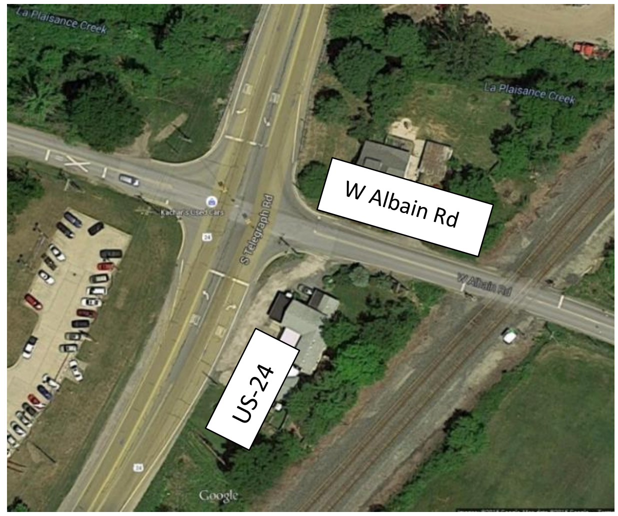
In addition, Sim-Traffic uses multiple categories of heavy vehicles with different lengths such as those for single-unit and semi-trailer trucks. Therefore, when comparing both models using the default values, the resulting lengths of queues can already expected to be shorter in VISSIM as compared to Sim-Traffic keeping everything else equal.

4.6 Using Simulation Models for Determining Queue Lengths

Traffic queue along a signalized intersection approach is mainly characterized by vehicle arrival pattern, vehicle mix, geometric elements and signal control operations. Traffic network models were developed for two existing signalized intersections in Michigan that are interconnected with the nearby railroad grade crossing warning devices. These locations were mainly picked based on different approach configurations and signal control operation. Simulation software generate random vehicular arrival based on the approach traffic volume. Additional calibration can be performed on the models however, that may only account for location specific variation. For the analysis performed in this chapter, the default model parameters were adopted for variables related to driver behavior (such as headways, lane changing behavior, gap acceptance etc.) and vehicular characteristics.

Figure 11 presents the two existing intersection configurations used for creating the traffic network models in the simulation software. The models were developed in two microscopic simulation modeling software i.e. VISSIM version 7 and Sim-Traffic version 8.

Simulations were performed for a variety of traffic scenarios and resulting queue lengths were tabulated. Tables 2-7 present 95th percentile queue lengths obtained from the micro-simulation models (VISSIM/Sim-Traffic) for various ranges of traffic volumes, vehicle mix and signal control parameters. The queue lengths are presented in feet.



(i) **(ii)**

Figure 11. Selected Intersection Configurations for Simulation Models

4.6.1 Multiple Simulation Runs

To account for the variations in simulation results, 5 multiple simulation runs were performed with different random seeding and the average queue length out of these multiple runs output was used for the analysis.

4.7 Factors included in the Analysis

Previous studies involving analysis of traffic queues at signalized intersections *(6, 12, 15, 16)* suggest that traffic queues mainly depend on approach lane volume, signal cycle length and effective green interval, movement type and phase designation, and arrival patterns. The variables that are studied in this research for analyzing traffic queue build-up at signalized intersections are as follows.

- Lane volume (vph)
- Signal cycle length

- Effective green time per movement
- Lane designation
- Signal phasing
- Percentage of heavy vehicles (HV %age)
  - Single Unit
  - Semi-trailers

The queuing analysis was performed for the following cases with respect to lane designation and signal control.

- Approach with single through lane and exclusive turn turns
- Single lane approach with shared left/through/right movements
- Exclusive Left Turn Lane
  - Permissive Left-turns

4.8 Approach with Single Through Traffic Lane and Exclusive Turn Lanes

Traffic operation on an approach with single through (thru) lane and exclusive lanes for left and right turning movements, was simulated on the modeled intersection (ii in figure 11) using both VISSIM and Sim-Traffic software for various range of analysis parameters. The following model characteristics were used for the analysis.

- Single through traffic lane with storage lanes for right and left turning traffic
- Vehicular speed on the Approach speed─35mph
- Heavy vehicles percentage─2%
- Signal cycle length=60s with 25s effective green time for the approach
- Car following and driver behavior related input- *default parameters of the simulation model*

  – Time headways

  – Gap acceptance

  – Lane changing behavior etc.

- Vehicle characteristics- *default parameters of the simulation model*

  – VISSIM: (Car length: 12.30 to 15.62 ft., HGV: 33.51 ft.)
  – Sim-Traffic: (Car length 14 to 16 ft., SU Truck: 35 ft., Semi-Truck:53 ft.)

- Analysis Period- 1 hour

- Number of multiple simulation runs─5

Table 5 below presents various queue lengths obtained from VISSIM and Sim-Traffic simulation models on an approach having a single through lane along with storage lanes for right and left turning traffic. Symmetrical traffic volumes and phase splits are assumed on the opposing approach. The values presented in table 5 are the average of 95th percentile and maximum queue lengths obtained on both the approaches. Additionally, each value represents an average of 5 multiple simulation runs output.

Table 5. Comparison of Queue Estimates on a Through Traffic Lane from Sim-Traffic and VISSIM Models *(CL=60s, EG=25s, 2%HV)*

Through Traffic Volume (vph)	Queue Lengths in feet (avg of 5 simulation runs)			
	Sim-Traffic		**VISSIM**	
	95th	**Max**	**95th**	Max
80	73.1	93.3	48.9	79.5
120	82.5	99.7	67.2	84.0
160	105.2	127.6	78.1	98.9
200	117.2	145.5	99.6	131.6
320	175.2	215.4	134.4	203.4
400	209.9	239.3	182.1	238.7
480	266.2	319.3	213.3	298.4
560	438.5	461.3	293.6	498.9
640	747.4	834.8	352	455.8

4.9 Differences between Sim-Traffic and VISSIM Queue Output

The results presented in table 5 show that the queue lengths obtained through Sim-Traffic models are mostly on the higher than those obtained through VISSIM simulation models. The difference is much higher as the volume approaches the capacity. The difference in values is due to various differences in the model assumptions and default parameters used in both the simulation models. It is important to know that the maximum queues are observed during the simulation period while 95th percentile queues are computed. The differences between 95th percentile queues are relatively more than those between the maximum observed queues. This difference is also attributed to the difference in 95th percentile computational methods used in both the models.

Sim-Traffic 95th percentile queue=1.65 standard deviations above the average queue

VISSIM 95th Percentile Queue= 95th Percentile of the max queues observed during each time step

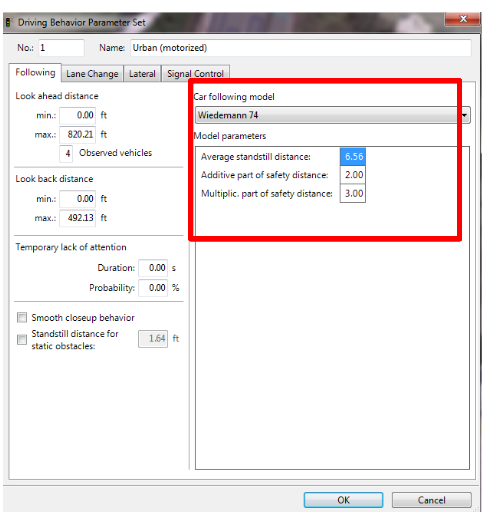
In addition, there are other differences between both the models on various parameters that impact the resulting extent of queues. These include differences in defining queue conditions, assumed vehicle lengths (passenger cars, and heavy vehicles), car-following model parameters, delay computations, parameters impacting saturation flow rates, and capacity conditions. Details on the differences in above mentioned factors are described earlier in sections 4.2-4.5 and table 4. The assumptions on average passenger car length and spacing between queued vehicles are not much different between both the models (see sec 4.5) however, for heavy vehicles, Sim-Traffic uses varying lengths including Semi-Trailers and other categories but this would only affect if much higher percentage of heavy vehicles are present in the traffic.

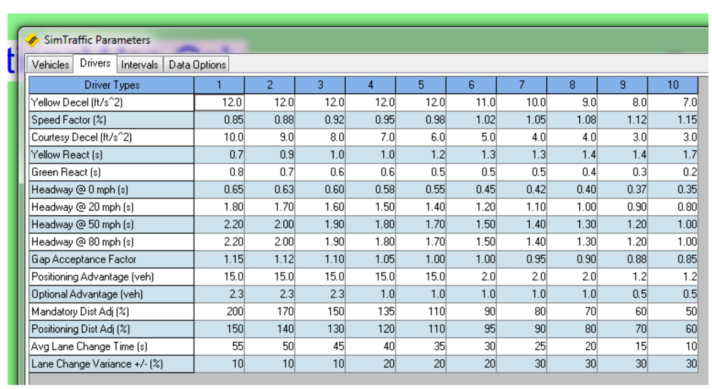
The main difference is attributed to the functions describing the arrival and departure rates at signalized intersections which depend on acceleration factors, headway factors and other car-following behaviors that are applied to the vehicles and which vary stochastically during the simulation period. The extent of queue mainly varies because of the differences in these functions and factors describing arrival, discharge, and saturation flow rates at the signalized intersection. The capacity conditions are also attributable to the arrival and discharge rate and the factors describing them. The capacity conditions arrive at lower volumes in Sim-Traffic as compared to VISSIM.

A brief description on these model assumptions and the parameters used is as follows. The car-following model used in VISSIM is "**Wiedemann 74**" (details in section 4.3) and the default values used are from the studies conducted in Germany.

"The car following model (Wiedemann 1974) has been calibrated through multiple measurements at the Institute of transport studies of the Karlsruhe Institute of Technology (since 2009 KIT – Karlsruhe Institute of Technology), Germany. Recent measurements ensure that changes in driving behavior and technical capabilities of the vehicles are accounted for (50)."

The following figure presents the windows showing some of the parameters and default values used in VISSIM and Sim-Traffic for car-following and other driver behaviors.





Driver Types	1	2	3	4	5	6	7	8	9	10
Yellow Decel (ft/s^2)	12.0	12.0	12.0	12.0	12.0	11.0	10.0	9.0	8.0	7.0
Speed Factor (%)	0.85	0.88	0.92	0.95	0.98	1.02	1.05	1.08	1.12	1.15
Courtesy Decel (ft/s^2)	10.0	9.0	8.0	7.0	6.0	5.0	4.0	4.0	3.0	3.0
Yellow React (s)	0.7	0.9	1.0	1.0	1.2	1.3	1.3	1.4	1.4	1.7
Green React (s)	0.8	0.7	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2
Headway @ 0 mph (s)	0.65	0.63	0.60	0.58	0.55	0.45	0.42	0.40	0.37	0.35
Headway @ 20 mph (s)	1.80	1.70	1.60	1.50	1.40	1.20	1.10	1.00	0.90	0.80
Headway @ 50 mph (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40	1.30	1.20	1.00
Headway @ 80 mph (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40	1.30	1.20	1.00
Gap Acceptance Factor	1.15	1.12	1.10	1.05	1.00	1.00	0.95	0.90	0.88	0.85
Positioning Advantage (veh)	15.0	15.0	15.0	15.0	15.0	2.0	2.0	2.0	1.2	1.2
Optional Advantage (veh)	2.3	2.3	2.3	1.0	1.0	1.0	1.0	1.0	0.5	0.5
Mandatory Dist Adj (%)	200	170	150	135	110	90	80	70	60	50
Positioning Dist Adj (%)	150	140	130	120	110	95	90	80	70	60
Avg Lane Change Time (s)	55	50	45	40	35	30	25	20	15	10
Lane Change Variance +/- (%)	10	10	10	20	20	20	30	30	30	30

Figure 12. Car-Following and Other Driver Behavior Parameters used in VISSIM and Sim-Traffic

Saturation Flow Rates

To model saturation flow rates, Sim-Traffic uses time headway factors. The headway factor is based on ideal saturation flow rate, lane width factor, the grade factor, the parking factor, the bus stop factor, and the area factor calculated by the following formula.

$$HWF = \frac{1.3 * 1900}{fw * fg * fp * fbs * fa * Ideal} - 0.3$$

In VISSIM, two basic model parameters that specify the safety distance between the vehicles and saturation flow rates include *(54)*.

- Additive Impact on Safety Distance *(defined in table 3)*
- Multiplicative Impact on Safety Distance *(defined in table 3)*

Delay Computation

Both Sim-Traffic and VISSIM report 'stopped' and 'total delay' per vehicle. Sim-Traffic computes total delay by taking the difference between the total travel time and the time that would be spent by a vehicle in absence of any other vehicles or control devices. The following formula is used to compute total delays in Sim-Traffic *(10)*.

Total Delay for time slice (TD) =dT*(max speed-actual speed)/max speed

Delay per vehicle=TD/no. Of vehicles

Where;

dT=time slice=0.1s

The total delay also includes delays due to reduced speed while at turning, decelerating while approaching a turn and accelerating after making a turn. The stopped delay reported in Sim-Traffic includes delay occurred when vehicle are in stopped position or at speed less than 10ft/s. VISSIM also computes the total and stopped delays for the defined travel time segments in the network. Total delay in VISSIM is defined as the difference between the actual and 'ideal' travel time for a vehicle. The actual travel time involves delays due to the impact of traffic control devices, conflict areas, and other car-following behavior. The ideal travel time considers desired speed, reduced speed areas (such as during a turn), and deceleration prior to reduced speed areas.

The differences in default values used for specifying car following parameters such as headways or safety distance, vehicle lengths, defining/calculating delays, queues, saturated flow rates and capacity conditions, all impact the resulting queue estimates from both the simulation models. It should also be noted that the traffic flow model and default values used for various traffic factors (some mentioned above) in VISSIM are based on the studies performed in Germany which (unless calibrated) may not adequately represent US driving behaviors and vehicle characteristics.

This study did not involve model calibration/validation however, referring to some of the previous studies *(38, 39)* on left-turn queues that involved field data to calibrate and validate the models while comparing Sim-Traffic, VISSIM and other queue estimation methods found Sim-Traffic in most cases closely matched the observed left-turn queues.

4.10 Validation of Results

Although various different locations and extensive field data would be needed for accurately calibrating and validating the model performance or results. However, a small comparison on a local site (in Michigan) was performed just using one of the weekday peak hour data on traffic volumes, signal timing and percentage of heavy vehicles. vehicle mix. No other data on calibration was used. Data was recorded using the video tape during a weekday morning peak period. The intersection configuration, traffic volumes and signal phasing are presented below.

Location: Nine Mile & Hilton, MI
Cycle Length=70s
Permissive-Protected Phasing
Traffic Volume (vph) (L,T,R)
EW= (35,201,17)
WB=(53,241,92)
NB=(19,100,50)
SB=(106,197,52)
Heavy Vehicles Percentage
EB-2%
WB-5%
NB-1%
SB-2%
Speed Limit
NS=30mph, EW=25mph

Cycle Length=70

Phase 1= 24s	Phase 2= 11	Phase 3= 24s	Phase 4= 11s
Effective Green= 18s	Effective Green=5s	Effective Green= 18s	Effective Green=5s
Yellow=4s	Yellow=4s	Yellow=4s	Yellow=4s
Red Clearance=2s	Red Clearance=2s	Red Clearance=2s	Red Clearance=2s

Figure 13. Model for Validation

The observed max queues (in no. of vehicles) are compared with those obtained using Sim-Traffic and VISSIM simulation models, presented in the following table. VISSIM estimated no. of vehicles for this case are lower (up to 1-2 cars) than the observed maximum queues while Sim-Traffic maximum queue estimates are little higher than the observed approximately up to 1 passenger car.

Method	**Queue Length**		
	Thru/Right Shared	**Thru**	**Left**
Max Observed (EB Approach)	5pc+1SU	6 cars	3 cars
VISSIM (max)	Max b/w T/TR shared-90.6 ft. ***(4.5 cars)***		44.2 ft. ***(2.2 cars)***
VISSIM (95th Percentile)	Max b/w T/TR shared-77 ft. *(3.8 cars)*		26.6 ft. *(1.3 cars)*
Sim-Traffic (max)	Max b/w T/TR shared-148 ft. ***(7.4 cars)***		89 ft. ***(4.4cars)***
Sim-Traffic (95th Percentile)	Max b/w T/TR shared-116 ft.*(5.8 cars)*		56 ft. *(2.8 cars)*

Where; pc=passenger cars
SU=Single Unit Truck
Conversion to no. of cars=Queue length/20
Simulation period: 60 min
Number of Runs: 5

Sim-Traffic max estimate of Thru/TR-shared is close to the observed maximum of (5pc +1SU) considering approximately 2 cars for 1 SU truck however left-turn queue is queue is higher up to 1 passenger car than the observed maximum number of vehicles. The number of vehicles estimated through VISSIM is lower than the observed queues from 1 to 2 passenger cars.

4.11 Queue Length Distributions

Table 6 presents various 95th percentile queues obtained from Sim-Traffic and VISSIM models which are plotted in Figure 14 to show the queue length distribution over varying traffic demand. The values are plotted up to the demand flow rate just below the capacity.

Table 6. 95th Percentile Queue Estimates on a Through Traffic Lane from Sim-Traffic and VISSIM Models *(CL=60s, EG=25s, 2%HV)*

Through Traffic Volume (vph)	95th Percentile Queue Length in feet *(avg of 5 simulation runs)*	
	Sim-Traffic	VISSIM
80	73.1	48.9
120	82.5	67.2
160	105.2	78.1
200	117.2	99.6
240	134.9	104.5
320	175.2	134.4
400	209.9	182.1
480	266.2	213.3
560	438.5	293.6
640	747.4	302.9
720	#2149.2	507.8

#represents over-saturated/unstable state in the simulation model (infinite queue)

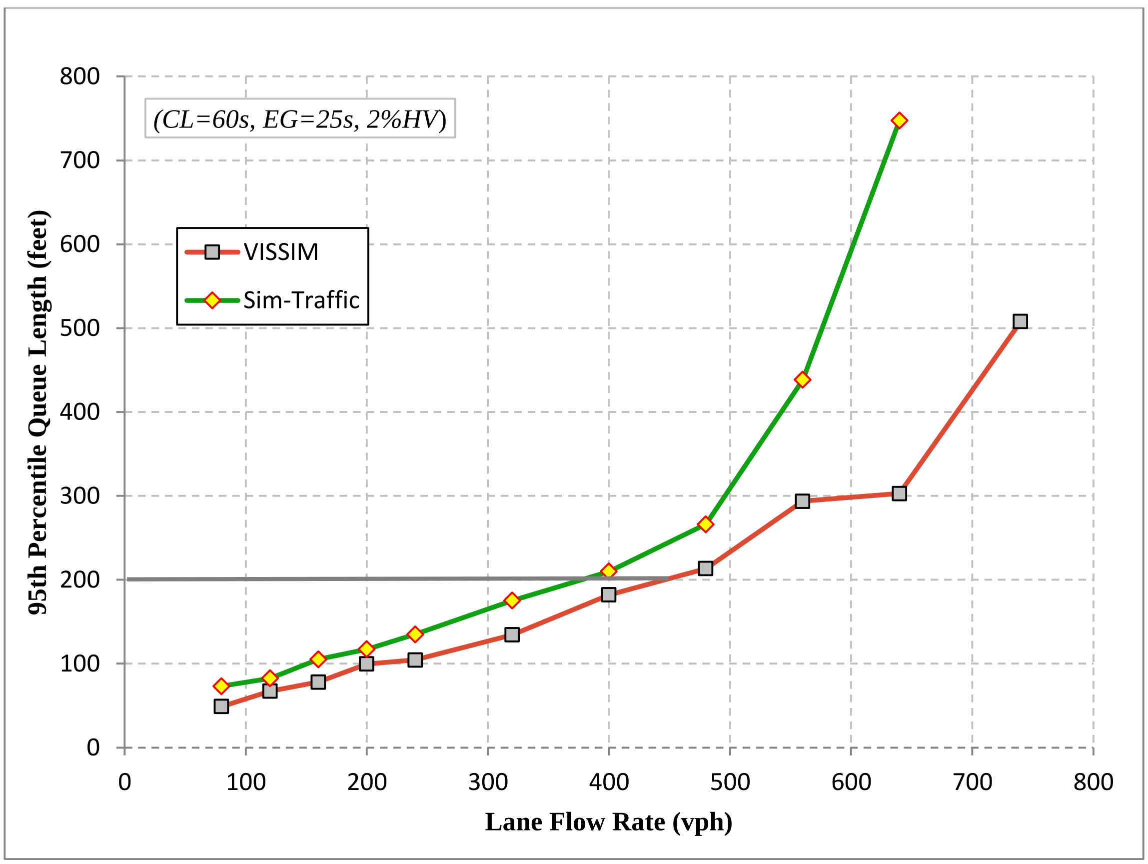


Figure 14. Queue Length Distributions from Sim-Traffic and VISSIM Models (Approach with Single Thru and Exclusive Turn Lanes)

The shape of the curves presented in figure 12 is due to the underlying probability distributions assumed for the queuing system in both Sim-Traffic and VISSIM that include Poisson arrival and negative exponential inter-arrival times/headway distribution. Going back to elementary queuing models, the resulting queue length is a function of mean arrival rate and waiting time in the queue as shown by the following expression *(34)*.

$$Expected\ Queue\ Length = Mean\ arrival\ rate \times Waiting\ time\ in\ queue$$

$$L = \lambda W$$

For a single server queuing system, the system utilization (or v/c ratio) can be expressed by taking the ratio of the mean arrival rate (λ) and mean service rate (μ),

$$\rho = \frac{\lambda}{\mu}$$

Also, as the demand flow rate approaches capacity the rapid increase occurs in the resulting queue lengths due to increase in associated delays which would further reach infinity at saturated/over-saturated ($\lambda/\mu \geq 1$) state. At flow rates exceeding capacity, queue would theoretically be equal to infinite length.

The curves in figure 13 (a and b) show the expanded queue length distributions obtained from Sim-Traffic models considering varying cycle lengths/green splits. The queue lengths are obtained for varying traffic volumes up to over-saturated state or demand flow rate exceeding capacity (using Synchro v/c ratios). As shown in figure 15 (a), with longer cycle lengths the resulting queue length increases because of lesser signal cycles or longer delays experienced by the vehicles during the analysis period. Also, as the demand flow rate approaches capacity the rapid increase occurs in the resulting queue lengths which further reaches infinity at $\lambda/\mu \geq 1$ (figure 15 (b)).

This phenomenon can again be explained by the elementary queuing models described earlier. Based on these models, the queuing system stays in a stable condition (steady-state) if arrival rate is less than the service rate or λ/μ<1. At steady state condition, the probability distribution of the state of the system remains the same over time. If the arrival rate becomes equal to or greater than service rate or at λ/μ ≥1, the queue length increases continually without being time independent and is equal to infinity as shown by the following expression from standard queuing theory *(34)*.

$$L = \frac{\rho}{1-\rho}$$

Where; L= expected number of vehicles in a queuing system

$$\rho = \frac{\lambda}{\mu}$$

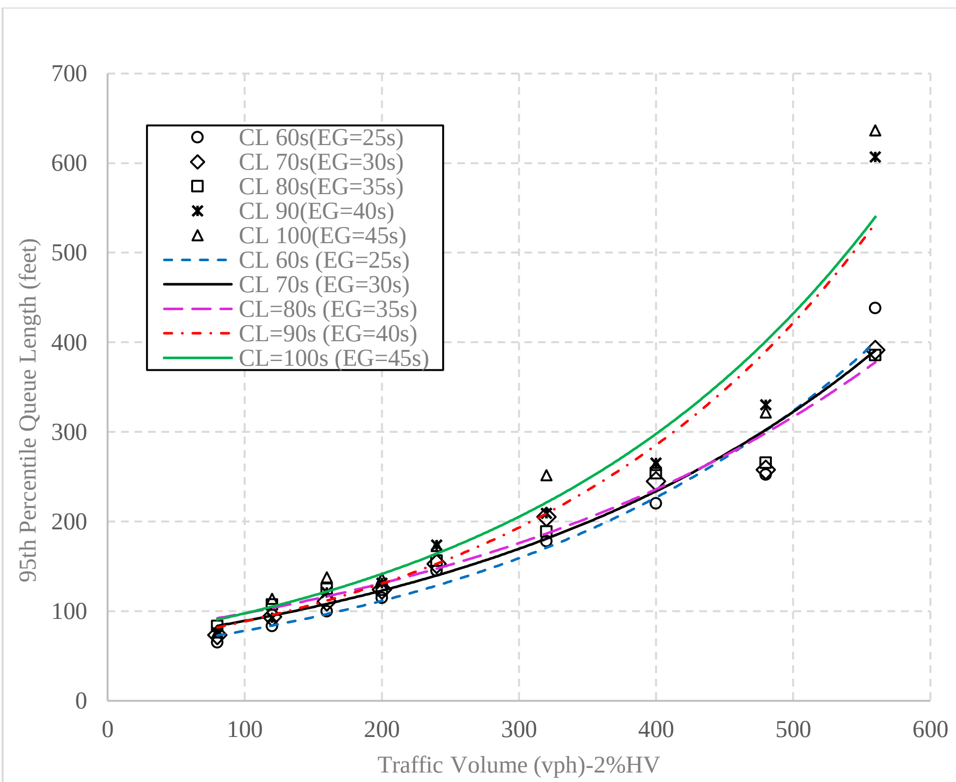


Figure 15 (a). Various Queue Length Distributions from Sim-Traffic Models (Approach with Single thru and exclusive turn lanes)

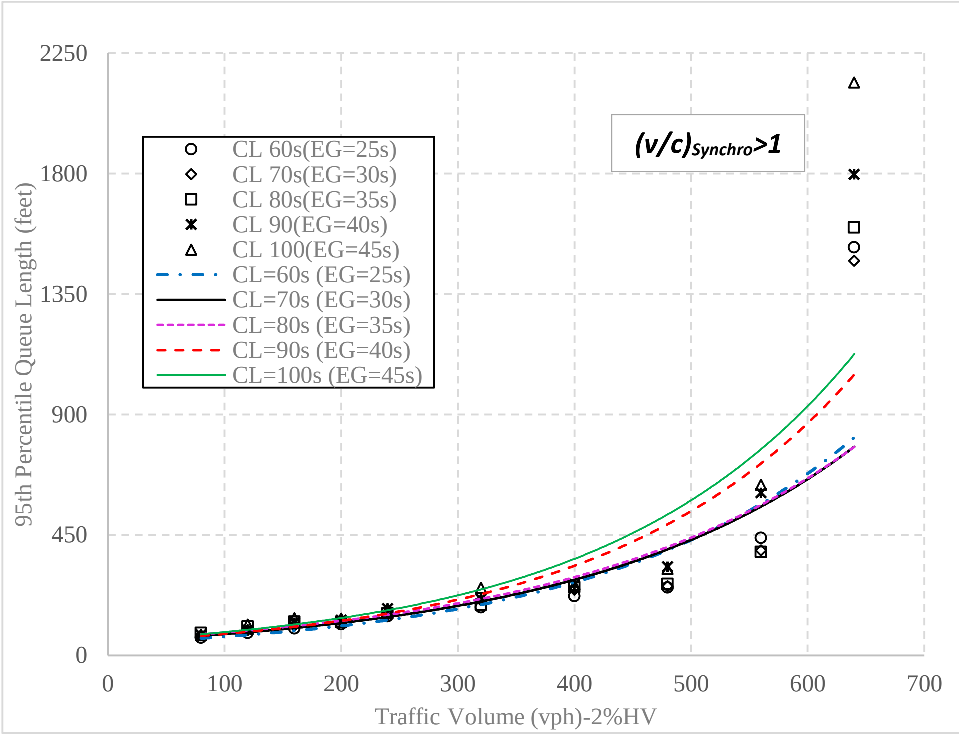


Figure 15 (b). Various Queue Length Distributions from Sim-Traffic Models Expanded up to v/c>1 (Approach with Single thru and exclusive turn lanes)

4.12 Variations due to Random Seeding and Simulation Period

To account for the variations in simulation results, 5 multiple simulation runs were performed with different random seeding and the average queue length out of these multiple runs output was used for the analysis. The variations due to varying random seeds are presented in table 7.

Additionally, simulation period also impact the resulting output. In order to show this variation, 15min, 30 min, 60 min (1 hr) and 90 minutes simulation runs were performed. The output is presented in table 8 and 9 using Sim-Traffic and VISSIM simulations respectively.

Table 7. Variations in Queue Output by Varying Random Seeding

App Vol (vph)	Random Seeding	VISSIM (95th Percentile Queue Length in ft.)			Sim-Traffic (95th Percentile Queue Length in ft.)		
		L	T	R	L	T	R
100	1	21.4	63.7	18.8	37	68	28
	2	22.2	44.7	18.0	20	77	44
	3	20.6	49.9	18.4	29	87	26
	4	22.5	37.3	20.2	25	70	23
	5	17.7	64.0	16.9	32	68	27
	Avg.	20.9	51.9	18.5	28.6	74.0	29.6
	S.D	1.9	11.8	1.2	6.5	8.2	8.3
150	1	21.9	80.1	19.9	37	84	46
	2	43.2	84.8	18.7	30	73	39
	3	22.1	67.5	18.2	36	83	35
	4	20.8	65.6	20.0	21	82	34
	5	22.2	81.8	19.3	49	100	34
	Avg.	26.1	75.9	19.2	34.6	84.4	37.6
	S.D	9.6	8.8	0.8	10.3	9.8	5.1
200	1	36.8	78.8	20.1	51	127	48
	2	21.4	84.0	20.0	72	103	40
	3	22.1	82.9	19.3	34	98	35
	4	22.2	63.3	20.0	49	108	36
	5	22.0	67.7	19.2	42	95	48
	Avg.	24.9	75.3	19.7	49.6	106.2	41.4
	S.D	6.7	9.3	0.4	14.2	12.6	6.3
250	1	35.0	129.2	20.0	53	101	36
	2	41.9	89.2	19.3	51	132	40
	3	21.8	129.6	19.9	40	118	57
	4	22.0	84.5	20.1	41	105	50
	5	22.9	65.5	19.8	63	117	52
	Avg.	28.8	99.6	19.8	49.6	114.6	47.0
	S.D	9.2	28.6	0.3	9.5	12.2	8.7
300	1	59.4	105.0	20.4	55	123	66
	2	40.4	122.1	19.8	54	120	67
	3	39.2	125.0	20.5	77	155	58
	4	40.6	112.1	19.5	71	144	72
	5	21.8	92.7	19.7	69	170	61
	Avg.	40.3	111.4	20.0	65.2	142.4	64.8
	S.D	13.3	13.1	0.4	10.2	21.2	5.4

Table 8. Variations by Varying Simulation Period using Sim-Traffic

App vol (vph)	Sim-Traffic 95th Percentile Queue Length in feet. *(avg. of 5 multiple runs)*											
	15 min simulation period			30 min simulation period			60 min simulation period			90 min simulation period		
	L	T	R	L	T	R	L	T	R	L	T	R
100	27	77	23	29	71	21	29	68	22	29	71	22
150	29	74	37	33	80	33	34	75	35	33	81	33
200	54	110	26	47	94	27	45	96	29	45	92	30
250	55	154	37	49	148	47	51	131	42	53	130	46
300	54	122	62	61	123	58	59	131	57	56	131	56
500	84	232	95	86	233	91	91	228	92	90	225	95

Table 9. Variations by Varying Simulation Period using VISSIM

App Vol (vph)	VISSIM 95th Percentile Queue Length in feet. *(avg. of 5 multiple runs)*											
	15 min simulation period			30 min simulation period			60 min simulation period			90 min simulation period		
	L	T	R	L	T	R	L	T	R	L	T	R
100	16.8	45.6	14.5	23.1	53.5	18.5	19.4	56.0	14.5	28.6	53.7	15.5
150	24.6	70.9	19.2	23.6	77.7	19.0	19.7	73.7	18.6	31.8	71.9	19.6
200	22.9	85.9	19.7	21.3	94.1	18.7	20.7	89.2	15.2	38.2	83.8	20.0
250	29.5	111.6	19.6	29.4	105.9	19.6	28.9	99.1	19.2	38.2	94.7	20.5
300	29.1	128.2	18.8	31.6	108.6	19.2	34.6	111.4	19.1	45.9	104.6	20.6
500	31.1	162.3	23.4	46.9	188.0	23.1	42.2	173.4	20.3	77.3	161.8	32.5

There is not significant variation across the simulation time periods. The queue lengths presented in the above tables are in feet. Converting it to number of vehicles would make the variations even lesser.

4.13 Single Lane Approach with Shared Left/Thru/Right (LTR) Movements

The operation of a single lane approach shared with left, thru and right movements is relatively complex for analyzing traffic queues and would also require consideration of opposing traffic volume in addition to approach volume, cycle length and effective green time.

Simulations were performed on the modeled intersection with LTR shared approach to analyze queue build-up under such traffic operation. Various queue lengths (95^{th} percentile) obtained from VISSIM and Sim-Traffic models simulations are presented in table 10 below. The approach and opposing traffic volume are considered identical in this case. In this case also Sim-Traffic estimates are higher than VISSIM.

Table 10. Comparison of Queue Estimates over a LTR Shared Lane from Sim-Traffic and VISSIM Models

Lane Volume (vph)- *10% turning traffic*	**95th Percentile Queue Length (feet)**	
	Sim-Traffic	**VISSIM**
50	40.2	34.9
100	63.8	50.1
150	84.3	78.6
200	107.2	102
250	131.5	125.6

(CL=60s, EG≈24s, 2%HV)

4.14 Left-Turn Lane Operation under Permissive Signal Control

The operation of left turn movements under permissive phasing is complex and should be carefully analyzed for determining queue spill back. As compared to exclusive signal phase, the analysis of queue build-up under permissive left-turn phasing would require consideration of opposing volume in addition to lane volume, cycle length, and effective green time. Additionally, the effect of lane overflow and blockage also could have a significant impact on the resulting queues and therefore should be carefully analyzed. Such impacts can be captured effectively using micro-simulation techniques and also be visually analyzed on the computer screen. Figure 16 includes the screenshots taken from VISSIM simulation showing left-turn lane overflow situation.

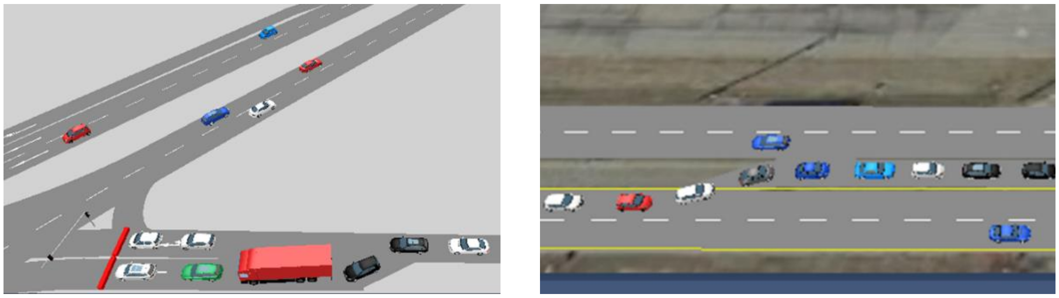


Figure 16. Left-Turn Lane Overflow

Various simulations were performed to analyze the queue build-up characteristics on left turn lanes under permissive phasing. The impact of opposing volume was included in the analysis to account for the interaction between left turns and opposing traffic. In addition, the impact of opposing traffic was further analyzed under presence of a single and multiple opposing lanes. Table 11 presents the queue lengths (95th Percentile) obtained from both VISSIM and Sim-Traffic models under the permissive left-turn signal phasing, against various left-turn lane volumes, opposing traffic volumes and signal control parameters. Each value represents the average queue lengths of 5 multiple simulation runs to account for the variation in simulation output.

Table 11. Comparison of Left-Turn Queue Estimates from Sim-Traffic and VISSIM Models

Approach Vol (vph)	Left-Turn Lane Vol (vph)	Opposing Volume (vph)	95th Percentile Queue Length (ft.)*	
			SimTraffic	VISSIM
400	100	100	51.0	59.21
		200	61.4	59.76
		300	85.2	65.3
		400	114.2	67.1
		500	194.6	76.9
		600	546.6	91.1
450	150	100	87.6	87.92
		200	119.4	86.42
		300	150.4	114.62
		400	253.4	154.31
		500	667	117.03
		600	637.6	184.08
500	200	100	107.4	100.6
		200	149.2	101.9
		300	225.8	149.6
		400	592.8	192.4
		500	632.8	204.5
		600	633	363.7
750	250	100	152.2	134.0
		200	204	148.7
		300	486.2	175.3
		400	638.2	375.1
		500	635.8	756.96
		600	637	1406.82
800	300	100	180.2	159.2
		200	383.2	205.8
		300	628.2	287.7
		400	626.6	866.7
		500	630.6	1384.3
		600	634.6	1652.1

(CL=60s, EG=24s, HV 2%) *Queue Length (average of 5 multiple runs)
Shaded area represents the left-lane overflow state

The distributions of left-turn lane queue lengths under permissive control from both the simulation models against varying opposing traffic volume are illustrated in figures 17 and 18.

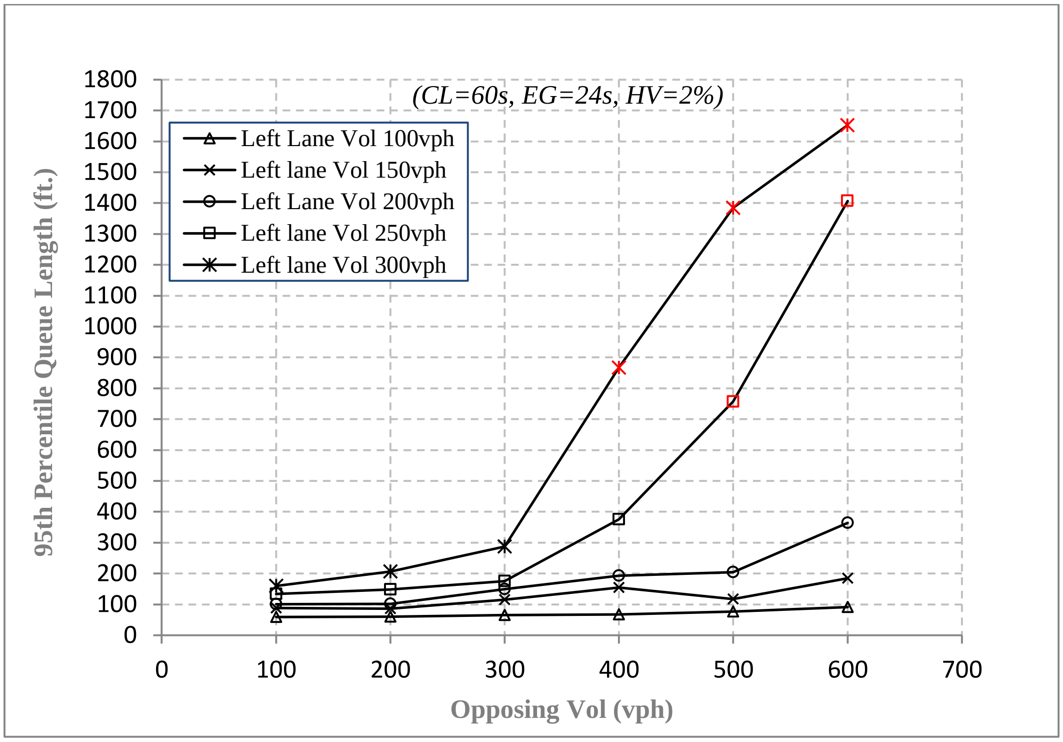


Figure 17. Left-Turn Queue Distributions under Permissive Control from VISSIM Model

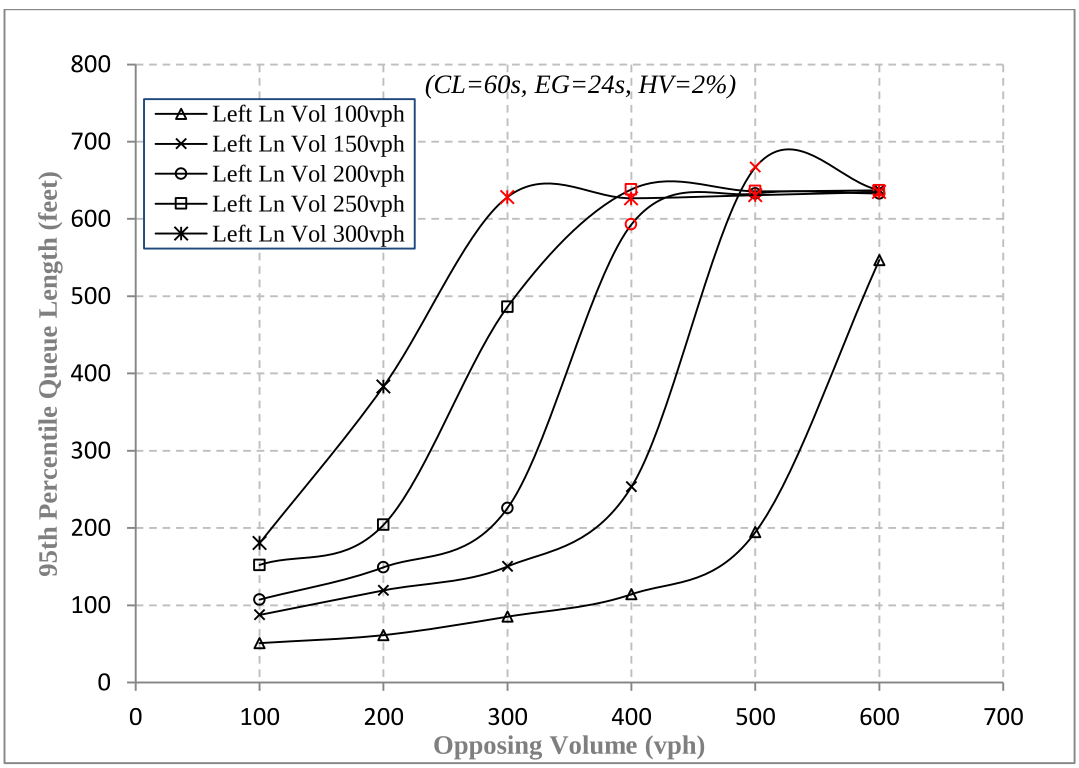


Figure 18. Left-Turn Queue Distributions under Permissive Control from Sim-Traffic Model

The marked points in figures 17 and 18 indicate the left turn lane overflow state from both VISSIM and Sim-Traffic models respectively. The queue lengths obtained from Sim-Traffic are again mostly higher than those from VISSIM except after the overflow conditions. This is because Sim-Traffic does not include queue carry over beyond the turning bay in reporting left turn queues (as shown by the marked/flatter portion of figure 18), however it is included in the adjacent lane queue estimate.

4.15 Comparison of Queue Estimation from Simulation Models with Other Analytical Models

Three different analytical methods were picked to compare the queue computation with simulation models. These included HCS, Synchro, and the Railroad Assessment Tool developed as part of the MDOT project on railroad preemption *(11)*. The following differences must be recognized before comparing these methods.

1. Length of passenger car assumed in HCS, Synchro and Railroad assessment tool is 25 ft. (including inter-vehicular spacing). Sim-Traffic uses passenger car length of 14 ft. or 16 ft. with 5 ft. spacing while passenger car lengths in VISSIM vary from 12.30 to 15.62 ft. with average spacing of 2.0m (with some stochastic variation) between the queued vehicles.

2. Simulation models (Sim-Traffic, VISSIM) assume probability based functions for describing vehicle arrival and inter-arrival time (headway distributions) and incorporate detailed car following parameters in order to include dynamic driver behaviors and vehicle interactions in the model such as parameters related to lane changing, gap acceptance, deceleration and acceleration characteristics etc. (details explained in chapter 2). Once the model is simulated the observed maximum queues as a result of the model formulations are reported which are further used to compute the percentile queues. HCS and Synchro procedures involve simple mathematical formula to compute back of queue. No initial queue is considered in HCS for under-saturated conditions (v/c<1). Similarly other deterministic analytical tools such as Synchro cannot add the impact of residual queues from one cycle to the other.

3. Queues in simulation models (Sim-Traffic) may also be of greater length than the analytical models because of several conditions that are not accounted for/reflected in them such as spillback between intersections, turn lane overflow and spill back to adjacent lane, forced lane changes, unbalanced lane use for downstream turns, and other subtle traffic flow interactions *(10)*.

4. The saturation flow rates in HCS and Synchro used for capacity, delay and queue computations are based on ideal saturation flow rates and several adjustment factors due to lane widths, heavy vehicles, approach grade, parking maneuvers, buses, area type, lane utilization, turning movements and pedestrian adjustment factors as shown in the formula below. The ideal saturation flow rate used in HCS and Synchro are as follows.

Thru Ln: 1900vphpl, Left Ln: 1000vphpl.

Adjusted saturation flow rates are calculated by the following formula.

$s = s_o\ N\ fw\ f_n\ f_{HV}\ f_g\ f_p\ f_{bb}\ fa\ f_{LU}\ f_{LT}\ f_{RT}\ fL_{pb}\ fR_{pb}$

Sim-Traffic uses time headway factors to model saturated flow rates. This factor is not used in Synchro for capacity calculations.

The headway factor is based on ideal saturation flow rate, lane width factor, the grade factor, the parking factor, the bus stop factor, and the area factor calculated by the following formula *(10)*. The default value used for HWF can be overridden according to the actual observed values.

$$HWF = \frac{1.3*1900}{fw*fg*fp*fbs*fa*Ideal} - 0.3$$

Similarly in VISSIM, two basic model parameters that specify the safety distance between the vehicles and saturation flow rates include

- Additive Impact on Safety Distance
- Multiplicative Impact on Safety Distance

5. Capacity and LOS computations in HCS and Synchro are based on HCM methodology. LOS is based on the control delay/ vehicle. There is a difference in defining delay and use it for evaluating LOS in analytical (HCS, Synchro) and microsimulation models (Sim-Traffic, VISSIM). A brief description of the differences is provided below.

6. HCS reports average control delay for signalized intersections computed through the HCM methodology that accounts for delays due to deceleration, stopped delays, queue start-up and acceleration time (HCM 2000). The roadway speed that may impact time spent in deceleration or acceleration is not considered in the HCS delay computations. Drivers traveling at high speed roadways may need more time in decelerating or accelerating than those traveling at lower speeds. In HCS queue computation procedure no initial/residual queue from previous cycle is considered at v/c<1.

7. Both Sim-Traffic and VISSIM report stopped delay and total delay per vehicle. Sim-traffic defines total delay as the total travel time minus the travel time for the vehicle with no other vehicles or traffic control devices. Total delay in VISSIM is the difference between the actual and 'ideal' travel time for a vehicle. The actual travel times involves delays due to the impact of traffic control devices, conflict areas, and other car-following behavior. The ideal travel time considers desired speed, reduced speed areas, and deceleration prior to reduced speed areas.

Table 12 below presents a comparison of queue estimates on a through traffic lane obtained from analytical methods HCS, Synchro, Railroad Assessment Tool, and simulation models including Sim-Traffic and VISSIM are presented below.

Table 12. Various Queue Estimates on a Single Through Traffic Lane from Different Analytical and Simulation Models

Through Traffic Lane Volume (vph)	**V/C Ratio** *(by HCS)*	**95th Percentile Queue Length (ft.)**				
		HCS	**Synchro**	**Railroad Assessment Tool**	**Sim-Traffic** *(avg of 5 runs)*	**VISSIM** *(avg of 5 runs)*
80	0.11	60	41	48	70	52.4
120	0.17	90	58	67	86.5	65.9
160	0.23	120	74	86	101.5	84.2
200	0.29	150	92	106	113	89.3
240	0.34	180	109	128	132.5	108.8
320	**0.46**	242.5	148	174	152.5	130.0
400	**0.57**	307.5	193	228	194	153.9
480	**0.69**	387.5	243	371*	225	195.3
560	**0.8**	495	345*	456*	295	223.1
640	**0.92**	660	421*	540*	359	255.9
700	1	865	478*	602*	558.5	296.6
750	1.05		524*		#1511.5	346.2
800						437.0
900						612.9

(CL=60s, EG=25s, HV=2%)
**red color output indicates demand reached/exceeded the signal capacity (queue estimates not reliable)*
 # represents over-saturated/unstable condition in the simulation model (infinite delay and queue)

The following table converts the above queue lengths into equivalent number of vehicles. The highlighted portion of the table is to present the critical queue estimation (more than 200 ft.) approximately between v/c 0.45-0.9.

Through Traffic Lane Volume (vph)	V/C Ratio (by HCS)	95th Percentile Queue (No of vehicles)				
		HCS	Synchro	Railroad Assessment Tool	Sim-Traffic	VISSIM
80	0.11	2.4	1.6	1.9	3.5	2.6
120	0.17	3.6	2.3	2.7	4.3	3.3
160	0.23	4.8	3	3.4	5.1	4.2
200	0.29	6	3.7	4.2	5.7	4.5
240	0.34	7.2	4.4	5.1	6.6	5.4
320	**0.46**	9.7	5.9	7	7.6	6.5
400	**0.57**	12.3	7.7	9.1	9.7	7.7
480	**0.69**	15.5	9.7	14.8	11.3	9.8
560	**0.8**	19.8	13.8	18.2	14.8	11.2
640	**0.92**	26.4	16.8	21.6	18	12.8
700	1	34.6	19.1	24.1	27.9	14.8

Note: Assumed vehicle length including inter-vehicular spacing
HCS, Synchro and Railroad Assessment Tool=25 ft. ; Sim-Traffic and VISSIM ~20 ft.

Table 13 below includes the comparison considering an approach where storage lanes are present with single thru lane. The queue lengths are obtained for over traffic demand (vph) at 60s cycle length, 25s effective green time.

Table 13. Various Queue Estimates on a Multilane Approach from Different Analytical and Simulation Models

Total Approach Volume (vph)- *10% turning vol*	Through Traffic Lane Vol (vph)	V/C ratio by HCS	95th Percentile QL (feet)				
			HCS	Synchro	Railroad Assessment Tool	Sim-Traffic	VISSIM
100	80	0.11	60	41	48	73.1	48.9
150	120	0.17	90	58	67	82.5	67.2
200	160	0.23	120	74	86	105.2	78.1
250	200	0.29	150	92	106	117.2	99.6
300	240	0.34	180	109	128	134.9	104.5
400	320	**0.46**	242.5	148	174	175.2	134.4
500	400	**0.57**	307.5	193	228	209.9	182.1
600	480	**0.69**	387.5	243	371*	266.2	213.3
700	560	**0.8**	495	345*	456*	438.5	293.6
800	640	**0.92**	660	421*	540*	747.4	302.9
900	720	>1	942.5	497*		#2149.2	507.8

(Single thru and Exclusive Turn Lanes: CL=60s, EG=25s, HV=2%)
**red color output indicates demand reached/exceeded the signal capacity (queue estimates not reliable)*
 # represents over-saturated/unstable condition in the simulation model (infinite delay and queue)

The following table converts the above queue lengths into equivalent number of vehicles.

Total Approach Volume (vph)-10% turning vol	Through Traffic Lane Vol (vph)	V/C ratio by HCS	95th Percentile QL (no. of vehicles)				
			HCS	Synchro	Railroad Assessment Tool	Sim-Traffic	VISSIM
100	80	0.11	2.4	1.6	1.9	3.7	2.4
150	120	0.17	3.6	2.3	2.7	4.1	3.4
200	160	0.23	4.8	3	3.4	5.3	3.9
250	200	0.29	6	3.7	4.2	5.9	5
300	240	0.34	7.2	4.4	5.1	6.7	5.2
400	320	**0.46**	9.7	5.9	7	8.8	6.7
500	400	**0.57**	12.3	7.7	9.1	10.5	9.1
600	480	**0.69**	15.5	9.7	14.8*	13.3	10.7
700	560	**0.8**	19.8	13.8*	18.2*	21.9	14.7
800	640	**0.92**	26.4	16.8*	21.6*	37.4	15.1
900	720	>1	37.7	19.9*		#107.46	25.4

Note: Assumed vehicle length including inter-vehicular spacing
HCS, Synchro and Railroad Assessment Tool=25 ft. ; Sim-Traffic and VISSIM ~20 ft.

The above comparisons are made between different methods to estimate queue lengths at varying flow rates (vph) increasing up to capacity conditions (v/c=1). As can be seen by the values presented in tables 8-9, the estimates provided by HCS are higher as compared to all the other methods. The difference is even higher at higher v/c ratios. The plot in figure 19 shows the queue length distributions capacity limits obtained from each method.

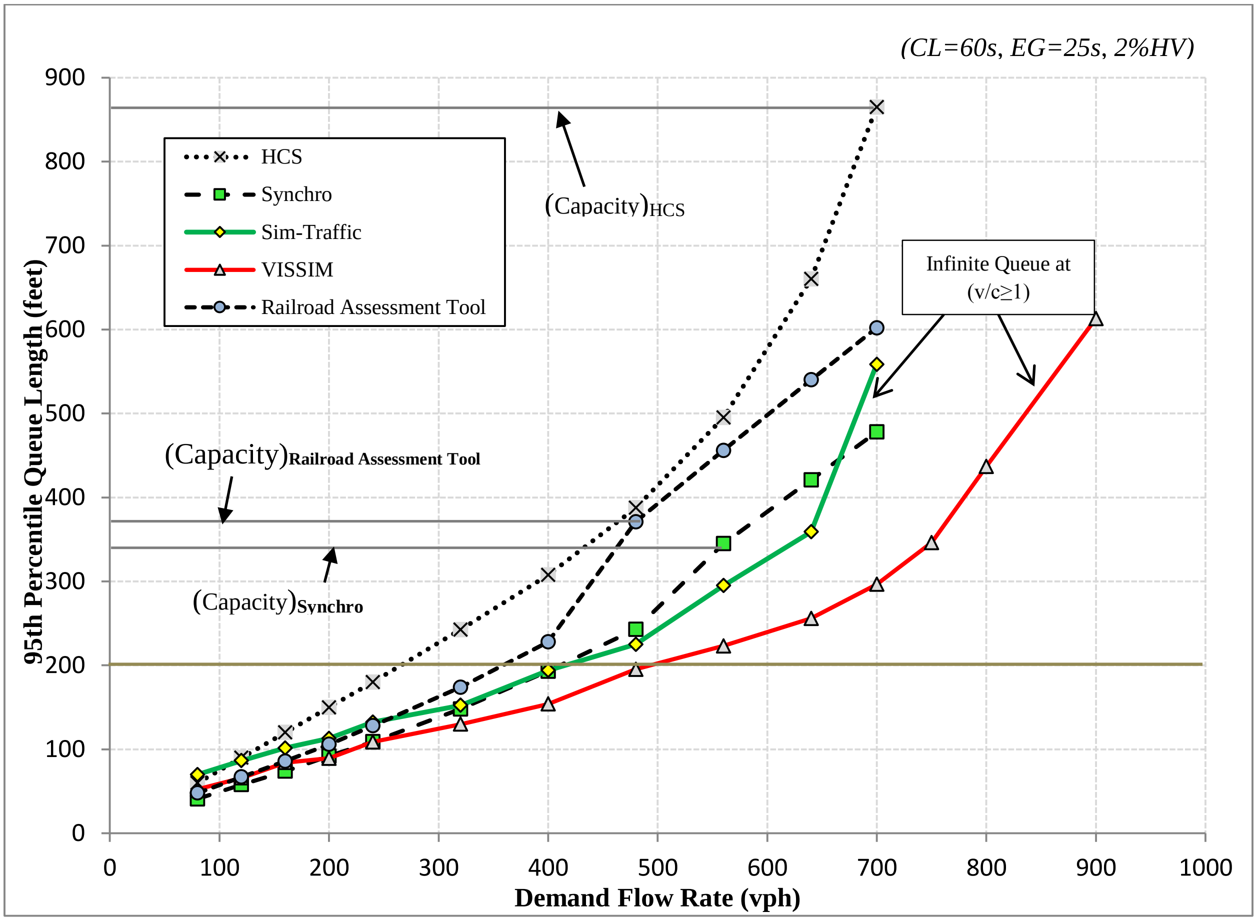


Figure 19. Queue Estimates on a Single Thru Lane from different Analytical and Simulation Models

As explained earlier there is difference in capacity, and delay calculations between all these methods which impact the resulting queue estimates. The difference in capacity limits as obtained by each method can be seen by the marked points in tables 8 and 9. VISSIM model appears to provide highest capacity and lower delay and queue estimates.

The procedure for capacity calculations in Synchro and Railroad assessment tool uses 95th percentile adjusted volumes using the following formula

$$v_{crit95} = v_{crit}\left(1 + 1.64\left(\frac{\sqrt{v_{crit} * C/3600}}{v_{crit} * C/3600}\right)\right)$$

While in simulation models capacity is reached when the flow rate becomes equal to the service rate ($\lambda \geq / \mu$) used in the models as explained in detail earlier.

In addition, while comparing the queue length results from these methods, the differences explained earlier in the beginning of section 5.9 should also be recognized; for example the default vehicle length of a passenger car in HCS, Synchro and Railroad assessment tool (25 ft.) is higher than the vehicle lengths assumed in Sim-Traffic (14 to 16 ft. with 5 ft. spacing) and VISSIM (12.30 to 15.62 ft.) because of that the queue lengths obtained from these analytical models can already be expected to be higher than those obtained from Sim-Traffic and VISSIM.

4.16 Comparison between Different Methods for Estimating Left Turn Queues under Permissive Control

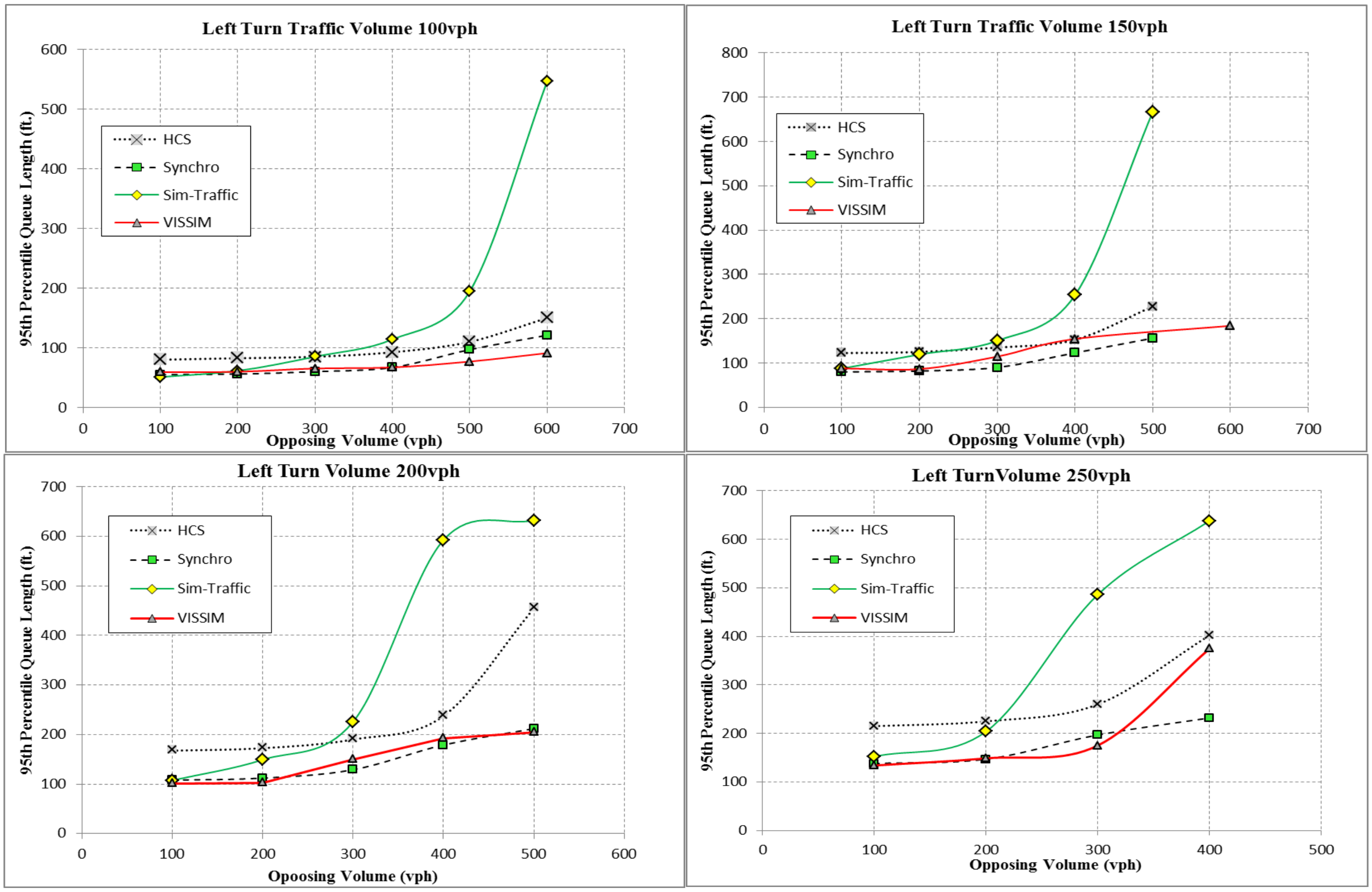
Another comparison was made on estimating left turn queue lengths on a multilane approach with exclusive left-turn lane operating under permissive control. In this case, additional factors others than those considered earlier are important to consider for analyzing queue formation such as lane changing behavior, opposing traffic volume, critical gap for left turning movements, left-turn lane overflow and blockage. These factors can impact both resulting left-turn and adjacent through traffic lane queue estimation. Table 10 below presents queue estimates for both through and left-turn lane as obtained from HCS, Synchro, Sim-Traffic, and VISIM.

Table 14. Various Left-Turn Queue Estimates from different Analytical and Simulation Models

Left-Turn Lane Vol (vph)	Opposing Volume (vph)	**$(V/C)_{HCS}$**		**HCS**		**Synchro**		**Sim-Traffic**		**VISSIM**	
		Thru	Left	Thru	Left	Thru	Left	Thru	Left	Thru	Left
Left Turn 100 Thru Vol 200	100	0.3	0.22	152.5	80	95	55	105.6	51	102.7	59.2
	200	0.3	0.25	152.5	82.5	95	56	94	61.4	102.7	59.8
	300	0.3	0.31	152.5	85	95	60	101.8	85.2	102.7	65.3
	400	0.3	0.42	152.5	92.5	95	67	103	114.2	102.7	67.1
	500	0.3	0.63	152.5	110	95	97	97.4	194.6	96.8	76.9
	600	0.3	0.9	152.5	150	95	121	681.4	546.6	96.8	91.1
Left Turn150 Thru Vol 200	100	0.3	0.34	152.5	122.5	95	80	95	87.6	98.3	87.9
	200	0.3	0.37	152.5	125	95	82	101.8	119.4	99.7	86.4
	300	0.3	0.47	152.5	135	95	90	92	150.4	98.3	114.6
	400	0.3	0.63	152.5	152.5	95	123	193.6	253.4	98.3	154.3
	500	0.3	0.95	152.5	227.5	95	156	1444.4	#667	98.3	117
	600	0.3	1.35	152.5	392.5	95	180		#637.6	98.3	184.1
Left Turn 200 Thru 200	100	0.3	0.45	152.5	167.5	95	108	99.4	107.4	100	100.6
	200	0.3	0.49	152.5	172.5	95	112	106.4	149.2	111	101.9
	300	0.3	0.63	152.5	190	95	129	93	225.8	111	149.6
	400	0.3	0.84	152.5	237.5	95	179	853	592.8	108.12	192.4
	500	0.3	1.27	152.5	455	95	212		#632.8	108.13	204.5
	600	0.3	1.79	152.5	670	95			#633	101	363.7
Left Turn 250 Thru 400	100	0.6	0.56	317.5	215	199	138	179.2	152.2	179	134
	200	0.6	0.62	317.5	225	199	147	181	204	179	148.7
	300	0.6	0.78	317.5	260	199	197	399.2	486.2	179	175.3
	400	0.6	1.06	317.5	402.5	199	232		#638.2	183.6	375.1
	500	0.6	1.59	317.5	737.5	199	265		#635.8	196	756.96
Left Turn 300 Thru 400	100	0.6	0.67	317.5	267.5	199	175	177.6	180.2	167	159.2
	200	0.6	0.74	317.5	285	199	213	295	383.2	168.2	205.8
	300	0.6	0.94	317.5	372.5	199	248		#628.2	167.5	287.7

(Permissive left-turn phasing, CL=60s, EG=25s, HV=2%)
Values in red color/shaded area indicates demand exceeded the signal capacity in the respective models
 # represents lane overflow condition. In Sim-Traffic, any spillback beyond the turn lane is counted in adjacent lane.

The graphs presented below in figure 20 are plotted using the data presented in table 14 up to the limits marked in the table.



(CL=60s, EG=24s, HV=2%)

Figure 20. Left-Turn Queue Distributions from different Analytical and Simulation Models

As explained earlier, the capacity conditions/computations are different in each method. It can also be observed form the values presented in table 14 that the HCS computed left-turn queue lengths are comparatively higher than the other methods at lower v/c ratios. As the v/c increases (0.5>v/c<1), Sim-Traffic queue estimates are higher than all the other methods. The difference is even higher near the capacity conditions (v/c=1).

It should also be noted that the adjacent lanes queue estimate from analytical models will have no variation due to opposing approach volume (see values presented in table 9) because they do not account for vehicular interaction between lanes, lane overflow, and blockage between different lanes of the approach. All these factors can have a significant impact on the resulting queues particularly at intersections operating near capacity therefore for such traffic operations, micro-simulation models provide much more practical analysis because of capturing the individual vehicles, driver behaviors and vehicle interactions. However, among the two micro-simulation models, Sim-Traffic analysis provides lesser capacity and higher queues which is preferable than lower/underestimated estimates.

4.17 Impact of Other Factors

In addition to vehicle interactions and driver behavior related impacts presented in the previous sections, the impact of various other traffic factors is included in the microscopic simulation models while recording the performance measures (delays, queues) which the deterministic models either do not include or cannot effectively account for. Some of these include impact of vehicle mix (percentage of heavy vehicles, difference in vehicle lengths, acceleration and deceleration characteristics etc.), roadway speeds (driver headways for a certain speed limit, acceleration/deceleration characteristics). Tables 15 (or figure 21) and 16 present the impact of some of these factors on resulting queue estimates obtained from the analytical based (HCS, Synchro, Railroad Assessment Tool) and micro-simulation based models (Sim-Traffic and VISSIM).

Table 15. Impact of Heavy Vehicles Percentage on the Resulting Queue Estimates from Deterministic Analytical and Simulation Models

Lane Volume (vph)	HCS			Synchro			Railroad Assesment Tool			Sim-Traffic			VISSIM		
	2%HV	5% HV	10% HV	2% HV	5% HV	10% HV	2% HV	5% HV	10% HV	2% HV	5% HV	10% HV	2% HV	5% HV	10% HV
50	37.5	37.5	37.5	27	27	27	33	35	38	38	38	47	31	31	38
100	75	75	75	47	47	48	57	61	65	62	65	74	52	52	57
150	112.5	113	115	67	68	68	81	87	92	85	95	96	78	78	84
200	150	153	152.5	89	90	91	106	113	120	98	112	117	95	99	106
250	190	190	192.5	113	114	115	133	142	151	122	128	139	140	148	149
300	230	233	235	138	140	143	162	173	183	169	168	192	151	158	163

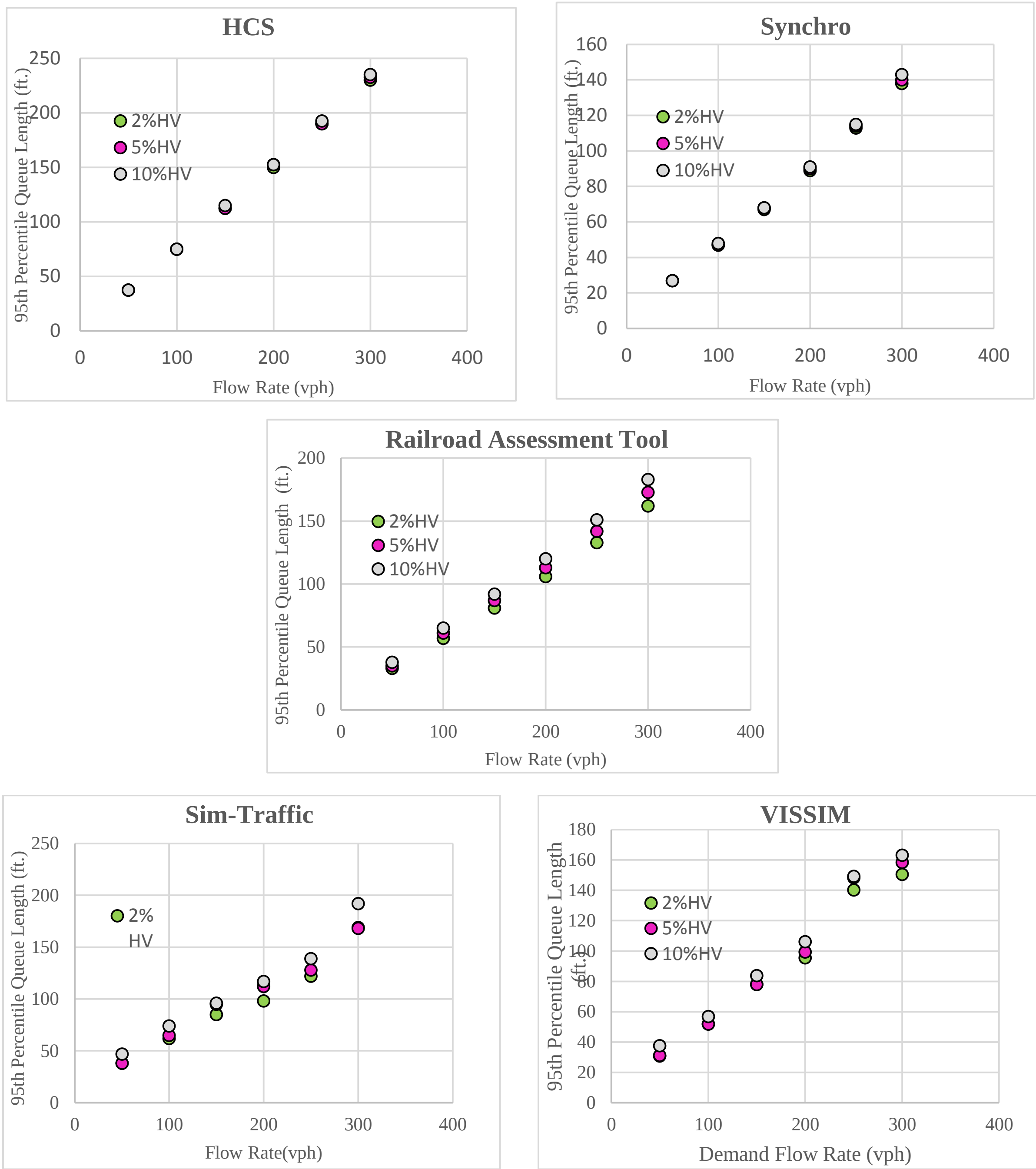


Figure 21. Impact of Heavy Vehicles Percentage on the Resulting Queue Estimates from different Deterministic Analytical and Simulation Models

As shown in figure 21, the impact of heavy vehicles is almost negligible (between 2-10% HV) from the HCS and Synchro procedures whereas variation can be observed in the simulation models output. Similarly, HCS, and Synchro models are not sensitive to roadway speed while difference in roadway speed does have an impact on the resulting queues as shown by the values presented in table 16 obtained from Sim-Traffic simulation model.

Table 16. Impact of Varying Roadway Speeds on Resulting Queue Estimates from Sim-Traffic Simulation Models

Traffic Volume (vph)		**95th Percentile Queue Length (ft.) from Sim-Traffic** *(avg of 5 sim runs)*					
		25mph		**35mph**		**55mph**	
Critical Approach	Opposing Approach	Critical Approach	Opposing Approach	Critical Approach	Opposing Approach	Critical Approach	Opposing Approach
50	100	42	59	38	55	31	46
100	150	71	85	62	77	53	67
150	200	96	106	85	99	73	83
200	250	105	133	98	131	80	113
250	300	140	157	122	145	107	130
300	350	174	181	169	186	154	182
350	400	226	245	211	236	205	228

Model parameters: Two-Lane roadway, (CL60s, EG=25s, 2%HV)

4.18 Statistical Analysis

To illustrate the significance of differences between queue estimates from each method, t-tests were applied on the queue estimates obtained in each case (values presented in table 17-20). As shown before, the difference between the values obtained from different methods becomes higher at higher v/c ratios. To show this, the t-tests were applied separately at v/c<0.5 and v/c≥0.5 as presented below.

Table 17. Results of t-test between Queue Estimates from Different Methods
(Single Thru Lane at v/c<0.5)

t-stat (p-value)

	HCS	Synchro	Railroad Assessment Tool	Sim-Traffic	VISSIM
HCS		1.72(0.06)	1.19(0.13)	1.05(0.16)	1.78(0.05)
Synchro					
Railroad Assessment Tool		0.59(0.28)		0.35(0.36)	0.59(0.28)
Sim-Traffic		1.12(0.14)			1.24(0.12)
VISSIM		0.07(0.47)			

Table 18. Results of t-test between Queue Estimates from Different Methods
(Single Thru Lane at 0.5>v/c<1)

t-stat(p-value)

	HCS	Synchro	Railroad Assessment Tool	Sim-Traffic	VISSIM
HCS		1.83(0.05)	0.86(0.2)	1.82(0.05)	3.09(0.01)
Synchro				0.11(0.45)	1.89(0.05)
Railroad Assessment Tool		1.22 (0.12)		1.22 (0.12)	3.05 (0.01)
Sim-Traffic					1.46(0.1)
VISSIM					

Table 19. Results of t-test between Queue Estimates from Different Methods
(Left-turn Lane at v/c<0.5)

t-stat(p-value)

	HCS	Synchro	Sim-Traffic	VISSIM
HCS		2.81(0.007)	0.91(0.18)	2.58(0.01)
Synchro				
Sim-Traffic		1.75(0.05)		1.504(0.078)
VISSIM		0.368(0.358)		

Table 20. Results of t-test between Queue Estimates from Different Methods
(Left-turn Lane at 0.5>v/c<1)

t-stat (p-value)

	HCS	Synchro	Sim-Traffic	VISSIM
HCS		2.89(0.004)		2.29(0.01)
Synchro				
Sim-Traffic	2.63(0.008)	4.37(0.0003)		4.05(0.0004)
VISSIM		0.31(0.38)		

4.19 Summary of Analysis Findings

While using any existing method for estimating traffic queues at signalized intersections that are in close proximity to railroad grade crossings, it is extremely important to know the specific limitations of the method, and the factors that can have a significant impact on the resulting queue estimate. An adequate analysis of traffic queues at such location is imperative in order to implement safe design that will minimize the risk of crashes. Such queuing analysis becomes even more critical where direct observations of traffic queues are not possible or where the assessment is needed for a future location.

To investigate the limitations of current queue estimation methods, their sensitivity to various traffic factors, and to help select an appropriate method for determining reliable estimates of queue lengths to be used for preemption evaluation, this research analyzed and compared different currently used analytical and simulation based methods including HCS, Syncho, Sim-Traffic and VISSIM. The comparison provided in this chapter can help understanding the appropriate application of these methods in determining queue estimates at signalized intersections near rail-highway grade crossing for preemption evaluation.

The queue length estimates at signalized intersections from all the above mentioned methods were compared for various traffic volumes, vehicle mix and signal control parameters. Impact of different lane configurations and signal phasing was also included in the analysis. The queue length parameter used in this analysis is 95th percentile queue length. Description on the differences between in each method on defining and computing queue lengths, and other assumptions on vehicle arrival pattern, car-following parameters, vehicle characteristics, delay, capacity and LOS computations etc. are provided in detail in this chapter. A few findings are presented below.

Comparing the values obtained from each method, HCS computed queue estimates are generally higher than all other methods particularly at under-saturated traffic conditions and for simple traffic operations (such as single thru lane and no turning traffic) where various traffic factors such as vehicle mix, speed, interaction between different lanes, and overflow queues do not have a significant impact on vehicular delays and queues. Similarly for such traffic operations, values obtained from the other analytical models (i.e. Synchro and Railroad Assessment Tool) were also on the higher side compared to those estimated by simulation models (using the default parameters).

For left-turn queues, deterministic analytical models HCS, Synchro, and Railroad Assessment Tool were found providing lower extent for left-turn queues than simulation models particularly for 0.5>v/c>1.0; at such traffic conditions HCS estimates on left-turn queues were lower than Sim-Traffic while Synchro estimates were lower than VISSIM. Sim-Traffic estimates on left-turn queues were significantly higher than all other methods.

The values computed from HCS are generally higher than other analytical tools i.e. Synchro and Railroad Assessment Tool however, HCS and other macroscopic analytical methods are significantly inadequate in accounting for various traffic factors that can have a significant impact on queue estimation. For example, the HCS procedure cannot account for vehicle-mix, speeds, lane overflow/blockage, impact of residual queues, and complex traffic operations. Synchro is also a macroscopic model and does not account for many of the above factors including vehicular interactions between lanes, complex left turn phasing, and spill back beyond turning bays. Microscopic-simulation models (Sim-Traffic, VISSIM) have the capability to account for the aforementioned traffic factors in computing delays and obtain information on individual vehicles position, speed, deceleration, acceleration etc. per small increment of time (per second or less than a second). In addition, these modeling techniques provide calibration of various parameters to capture the actual traffic conditions at a location.

Two simulation models Sim-Traffic and VISSIM were also analyzed and compared in this research. The differences in queue output between both the methods are attributed to the various differences in factors that impact the resulting queue lengths. These include differences in functions describing arrival and discharge rates, headway distributions, acceleration functions, default parameters used for driving behaviors, vehicle characteristics, and factors used to model saturation flow rates etc. Additionally, when comparing 95th percentile queues, the difference is also attributed to the different percentile computation methods used in both the methods. It is also important to know that the traffic flow model and parameters defaults describing the traffic flow, driver behaviors, vehicle lengths and characteristics etc. in VISSIM are based on the studies conducted in Germany which may not adequately represent (unless calibrated) US driving behaviors and vehicle characteristics.

Sim-Traffic simulation models estimates are generally higher than those obtained from VISSIM using the default values for the aforementioned factors. It was also found that using the default model parameters for car-following, driver behaviors, and vehicle characteristics etc., VISSIM model provides higher capacity and lower queue estimates as compared to Sim-Traffic, and all other analytical models HCS, Synchro particularly for through lane queus. The comparison of various queue estimates obtained from each method in this chapter showed that the difference in the queue estimates among these methods becomes more significant as the demand reaches approximately at/above 50% of the capacity ($0.5 \geq v/c < 1$).

A comparison of the maximum queues observed at a local site was performed as part of this research using the data on intersection geometry, traffic volumes, signal control, speeds and heavy vehicles. No other model calibration was done. The results showed Sim-Traffic queue estimates slightly above the observed maximum no. of vehicles, up to 1 passenger car. The 95th percentile estimate from Sim-Traffic also found to be close to the maximum observed queue (particularly left-turn queue). The no. of vehicles estimated through VISSIM were lower than the observed queues from 1 to 2 passenger cars.

4.20 Recommended Procedure for Estimating Queue Lengths for Preemption Evaluation

Based on the findings presented in previous section, a micro-simulation analysis based procedure is developed as part of this research for estimating queue lengths at various existing signalized intersection approach(es) that are in close proximity to highway-rail grade crossings for preemption evaluation.

This procedure is based on a micro-simulation technique i.e. Sim-Traffic which records information on individual vehicle position per small increment (less than a second) during the analysis period, and include impact of various important traffic factors for determining queues that only micro-simulation models are capable to capture (such as vehicle mix, vehicle interactions between lanes, driver behaviors, overflow queues, speed etc.). This procedure provides 95th percentile queue estimate by each lane group.

The required parameters for estimating queues from this procedure include

Lane flow rate (vph)

Lane designation and signal phasing

Signal Cycle length (s) and effective green time for the lane group/movement type

Percentage of heavy vehicles

The procedure assumes the following limits for vehicle lengths and driver behavior related parameters

Vehicle Characteristics

<u>Vehicle length:</u>

Car: 14ft. or 16ft.

SU Truck: 35 ft.

Semi Truck 1: 53 ft.

Truck DB: 64 ft.

Bus: 40 ft.

Inter-vehicular spacing-19.5 feet

Heavy Vehicle Length-

Driver Characteristics

Gap Acceptance Factor- varies from 0.85-1.15

Headway @ 0mph: 0.35-0.65

Headway @20mph: varies from 0.8 to 1.80

Headway@50mph/80mph (s): varies from 1.00 to 2.20

Yellow deceleration (ft/s^2): varies between 7ft/s^2 to 12 ft/s^2

The tables provided in templates 1-4 in Appendix A are developed using the above method and can be used for estimating queue lengths according to the desired lane group, traffic volume, vehicle mix, and signal control parameters.

Chapter 5 provides detailed description and recommendations on determining queue clearance distance (if preemption is needed) based on the various existing configurations and also provide procedure for determining minimum queue clearance time.

The flow chart presented in figure 22 should be followed for using the developed procedures, required input factors, and other important considerations for preemption evaluation.

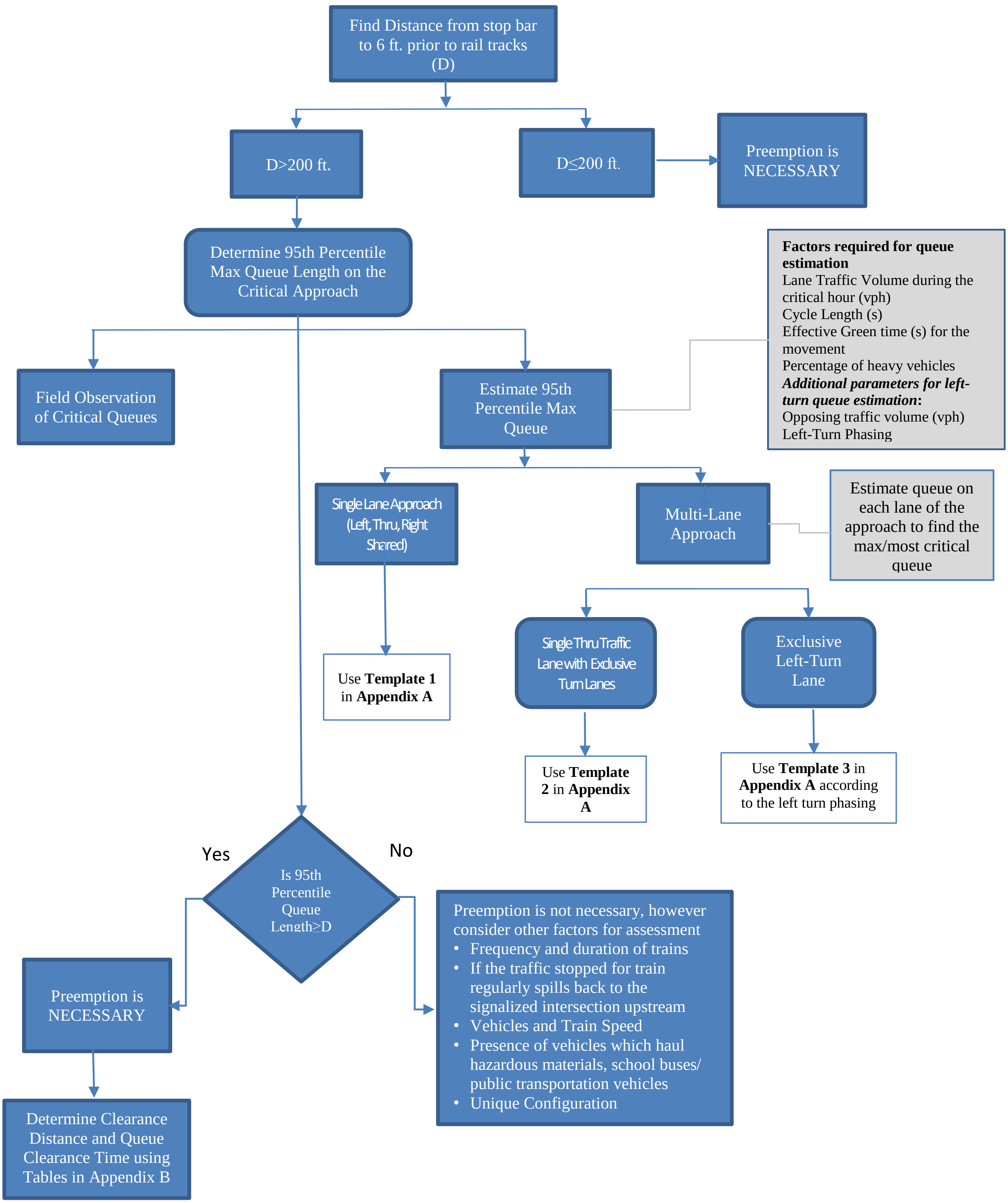


Figure 22. Preemption Evaluation Chart

CHAPTER 5

DETERMINING CLEARANCE DISTANCE AND QUEUE CLEARANCE TIME

The clearance distance is utilized in the calculation of the track clearance green time. It represents the length of the critical queue that is expected to occur at any moment of the day along the railroad crossing approach. Accordingly, the critical queue can be along the exclusive left-turn lane, the through lane or the shared movement lanes. The clearance distance is a function of many factors related to the geometry of the railroad crossing approach and the type of traffic control devices. Determining the clearance distance focuses more on the safety of the automobile traffic because train-vehicle crashes are severe. Templates were developed to cover the different configurations of highway-railroad grade crossings on which the clearance distance was identified by considering the worst case scenario. The main factors used in generating the templates are:

- Presence/absence of a gate
- Existence of pre-signals
- Lane configurations
- Type of left-turn phase (permissive only, permissive – protected, protected only) where present
- Type of railroad crossing (simple, diagonal or middle of the intersection)

5.1 Gate and Pre-Signal Present

When gates and pre-signals are present, it is expected that there will not be any queue after the pre-signal; however, driver behavior cannot be predicted, so the worst scenario is when a driver runs the red light of the pre-signal and stops on or near the tracks. There is also the possibility that the preemption call is initiated while the left turning traffic is yielding to opposing traffic. Those concerns were clarified by developing three types of the templates based on the expected geometry of the railroad crossing approach.

- Case 1: Two-Lane Road
- Case 2: Four-Lane Road (left turn is shared with through movement)
- Case 3: Roads with an Exclusive Left-Turn Lane

Case 1: Two-Lane Road

This is where the railroad crossing exists and there is one lane in each direction, which means that the through, left and right movements are shared (no storage lane are provided for left or right movements at the intersection) and have permissible turning movements. In this case if the preemption is called and the first vehicle in the queue is turning left, then it has to yield to the opposing traffic, which results in blocking the entire movement. This may not cause a safety problem if the queue does not extend into the tracks (Figure 23) but can cause a safety concern if the queue extends to the tracks (Figure 24). In order to avoid this, a doghouse signal (i.e., signal head with ball and arrow) display should be installed for the approach crossing the tracks, so that the left turning traffic will have protected movement during the track clearance phase.

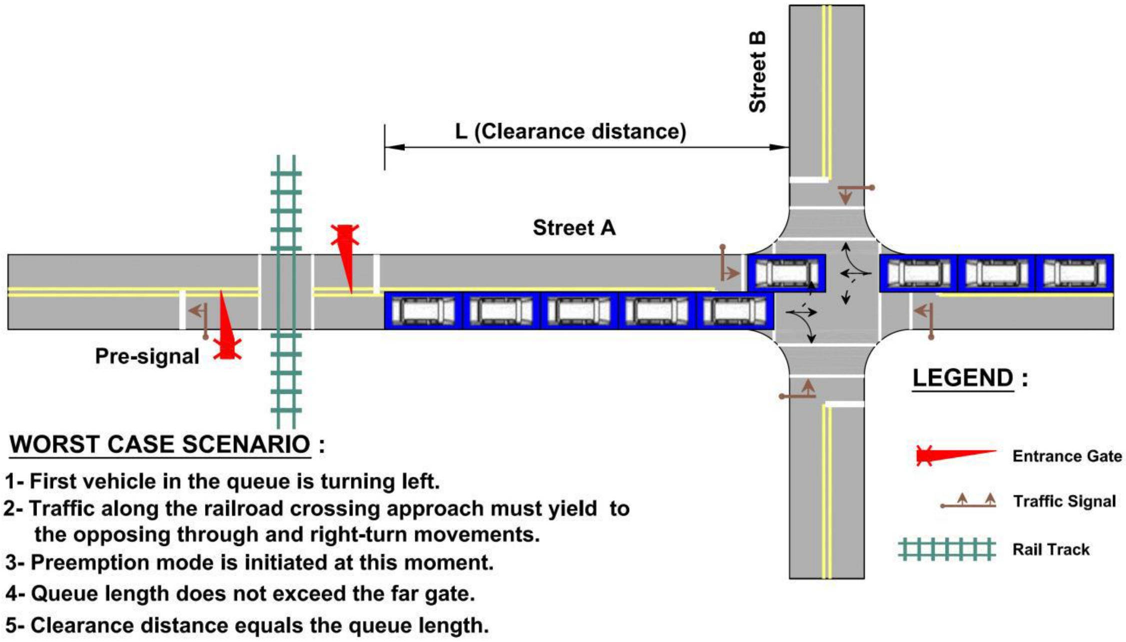


Figure 23. Clearance Distance on a Two-Lane Roadway when Queue is not extending beyond the Tracks (Gate and Pre-signal Present)

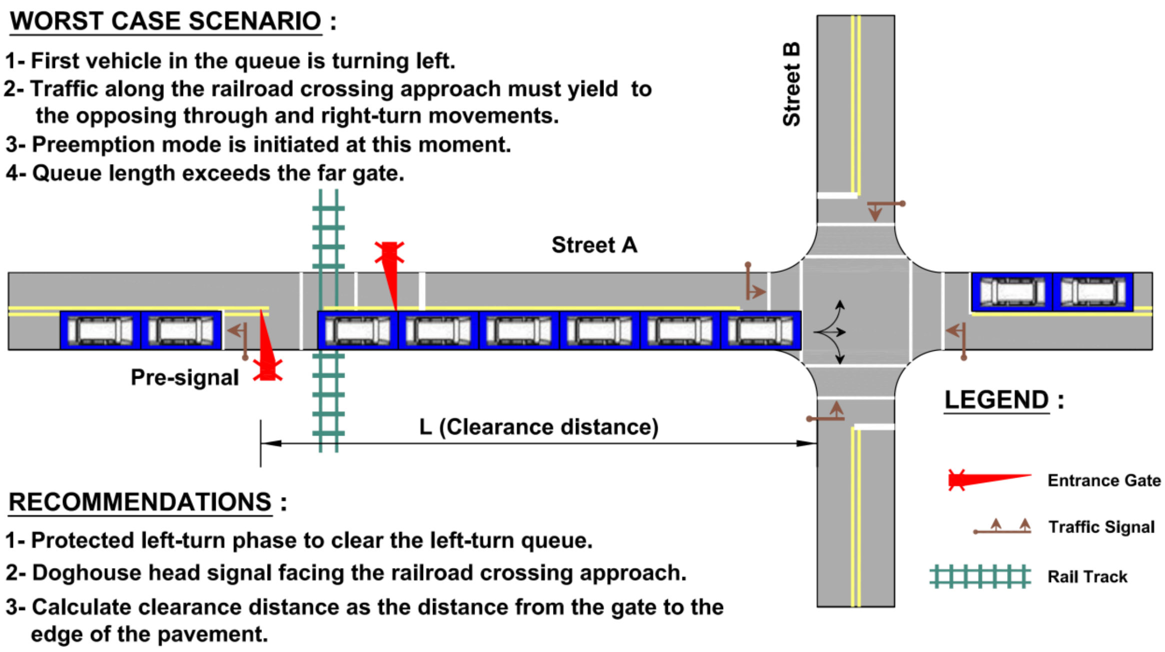


Figure 24. Clearance Distance on a Two-Lane Roadway when Queue is extending beyond the Tracks (Gate and Pre-Signal Present)

In the first case, when the queue does not reach the tracks (Figure 23), the queue length should be utilized for determining track clearance time. In the second case where the queue extends beyond the pre-signal (Figure 24), clearance distance should be the distance between the far gate and the edge of the road.

Case 2: Four-Lane Road (Left turn is shared with through movement)

Another configuration is where railroad tracks are crossing a four-lane road (two in each direction), the right and left turning movements are shared with through lanes and the traffic signal has a permissive left-turn phasing design as shown in Figure 25. The critical lane is the shared through and left turning lane because if the first vehicle in the queue is turning left, then it has to yield to opposing traffic which results in the formation of a queue. The queue length in the 'left turn shared lane' should be utilized for determining the track clearance time. If the queue length extends beyond the tracks, the clearance distance should be considered from the far gate to the edge of the road (Figure 26). A doghouse or a left-turn signal head should be installed to provide a protected phase for the left turning movement during the track clearance phase**.**

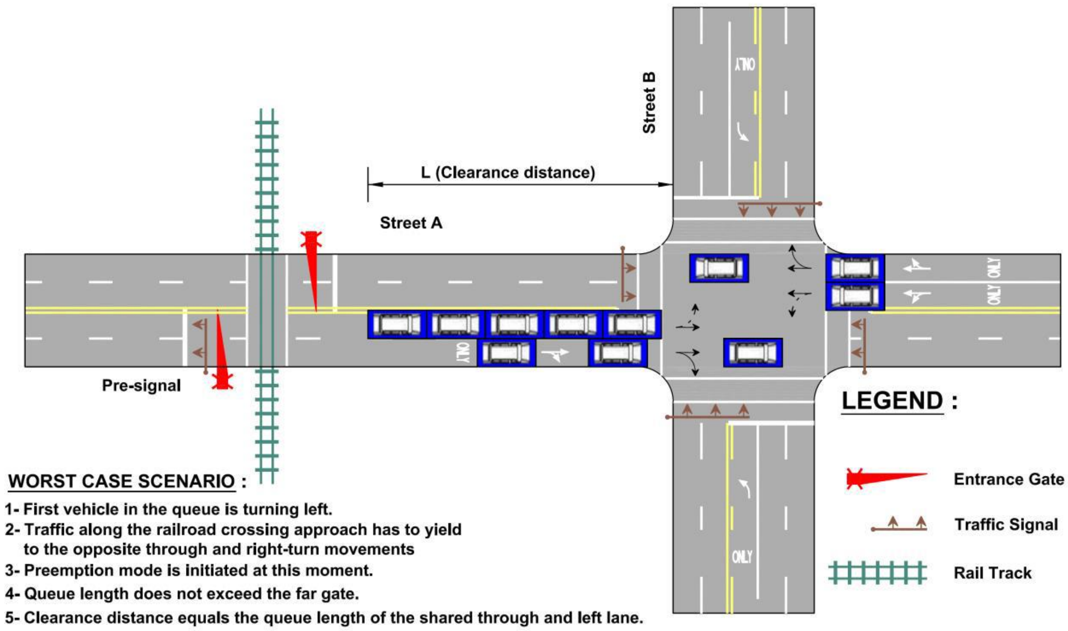


Figure 25. Clearance Distance on a Four-Lane Roadway when Queue is not extending beyond the Tracks (Gate and Pre-Signal Present)

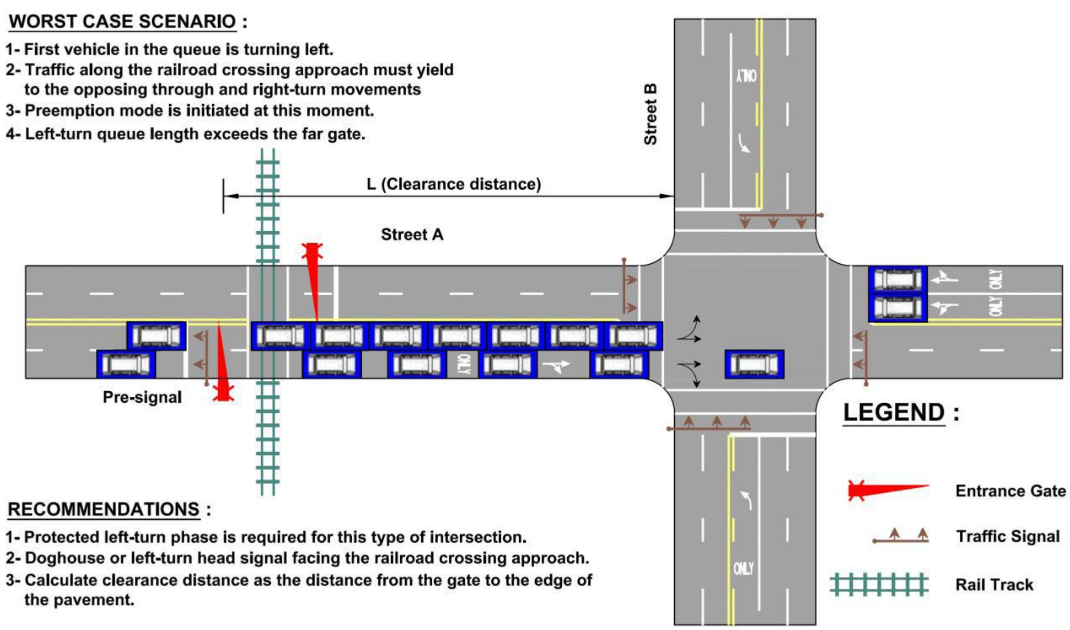


Figure 26. Clearance Distance on a Four-Lane Roadway when Queue is Extending beyond the Tracks (Gate and Pre-Signal Present)

Case 3: Roads with Exclusive Left-Turn Lane

This is similar to the four-lane road where the critical queue is expected along the left-turning lane. Two cases should be considered for this since the roadway geometry depends on the magnitude of the left turning queue and available clear storage distance (measured from the stop bar to 6 feet from the nearest track edge to the intersection):

- If the clear storage distance is more than the left turning queue length (Figure 27), then there is no need for a protected left-turn phase during the track clearance phase.
- If the clear storage distance is less than the left turning queue length (Figure 28), then a protected left-turn phase should be provided during the track clearance phase by installing a left-turn signal head facing the left lane.

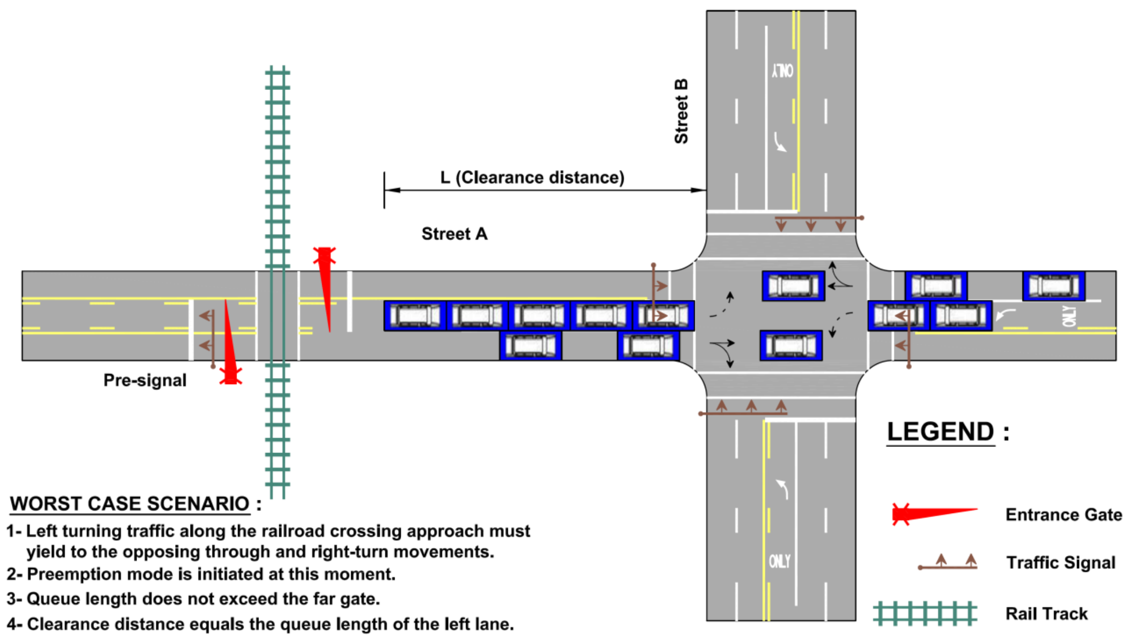


Figure 27. Clearance Distance on a Roadway with an Exclusive Left-Turn Lane When Queue is Not Extending beyond the Tracks (Gate and Pre-Signal Present)

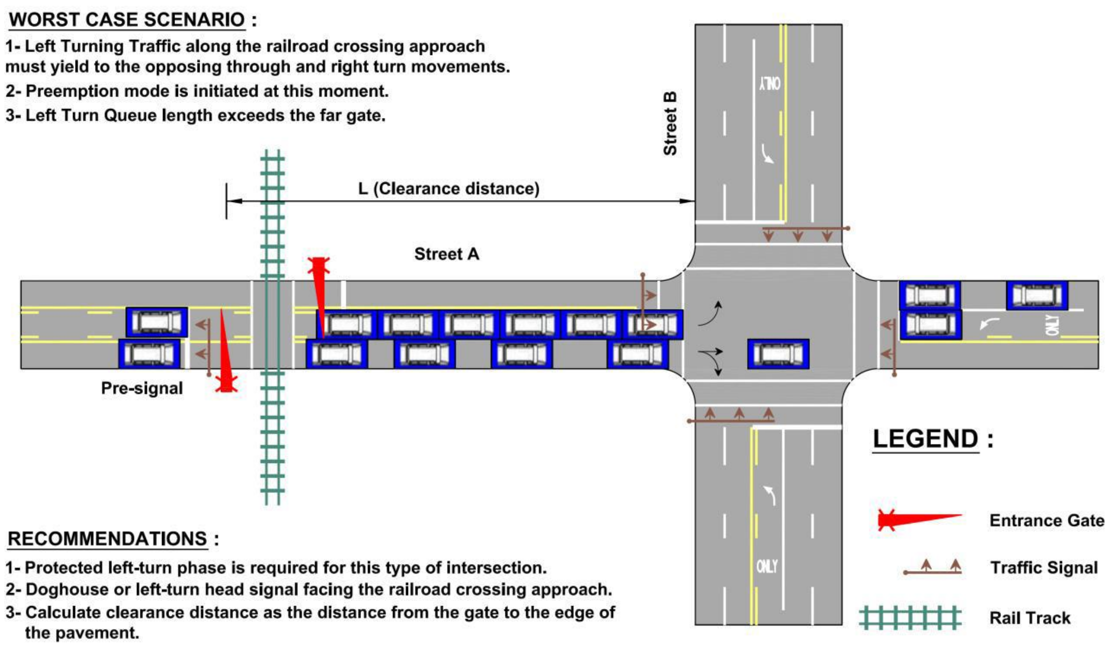


Figure 28. Clearance Distance on a Roadway with an Exclusive Left-Turn Lane When Queue is Extending beyond the Tracks (Gate and Pre-Signal Present)

5.2 Gated Railroad Crossings without Pre-Signal

If a gate is present without a pre-signal, the distance that should be considered to determine the track clearance time is minimum of:

- Clearance distance
- Maximum 95th percentile queue length (among all the lanes)

Possible worst case scenarios that can occur are as follows.

Case 1: Two-Lane Road

If it is a two by two lane intersection with a two phase signal design with no pre-signal, the two phase signal does not cause any problems during preemption mode, if the 95th percentile queue length does not exceed the available clear distance from the tracks. However, if the queue extends beyond the tracks, the queue may not dissipate during the track clearance interval if the first vehicle in the queue turning left, as the turning vehicle yields to opposing traffic.

In order to ensure that the queue is cleared with one lane in each direction where the through, left and right movements are shared (no exclusive turn lane are provided for left or right movements at the intersection) may cause queue build-up. This may not cause a safety problem if the queue does not extend into the tracks (Figure 29), however can cause a serious safety issue if the queue extends to or beyond the tracks (Figure 30). In order to avoid this, a doghouse signal head (i.e., signal head with ball and arrow) should be installed for the approach crossing the tracks so that the left turning traffic may not block the queue behind.

When the queue does not reach the tracks (Figure 29), the queue length should be utilized for determining track clearance time. Where the queue extends to the tracks (Figure 30), clearance distance should be the distance from the far gate to the edge of the road.

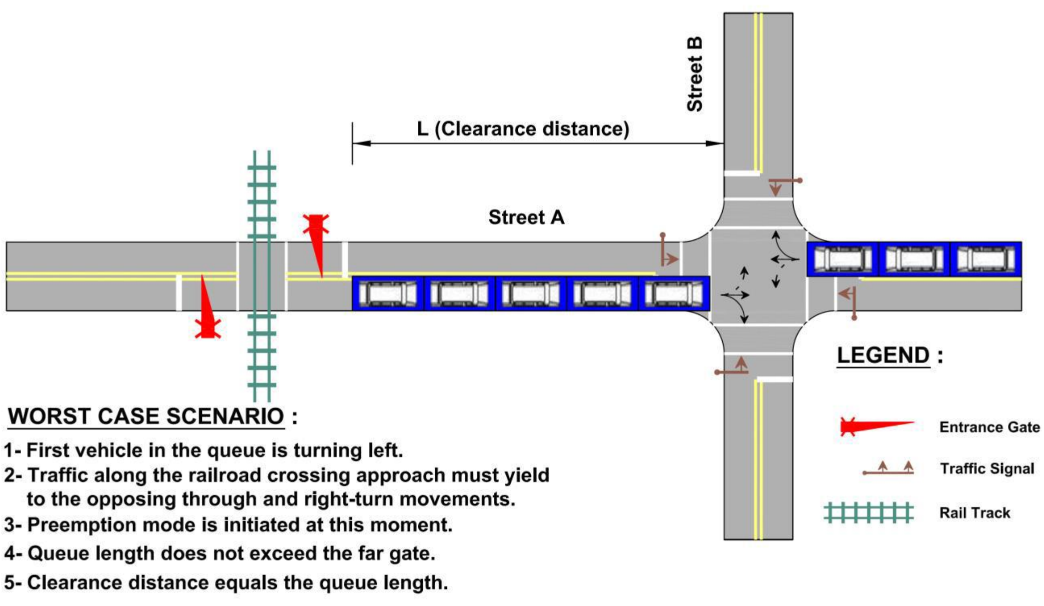


Figure 29. Clearance Distance on a Two-Lane Roadway When Queue is Not Extending beyond the Tracks (Gate Present without a Pre-Signal)

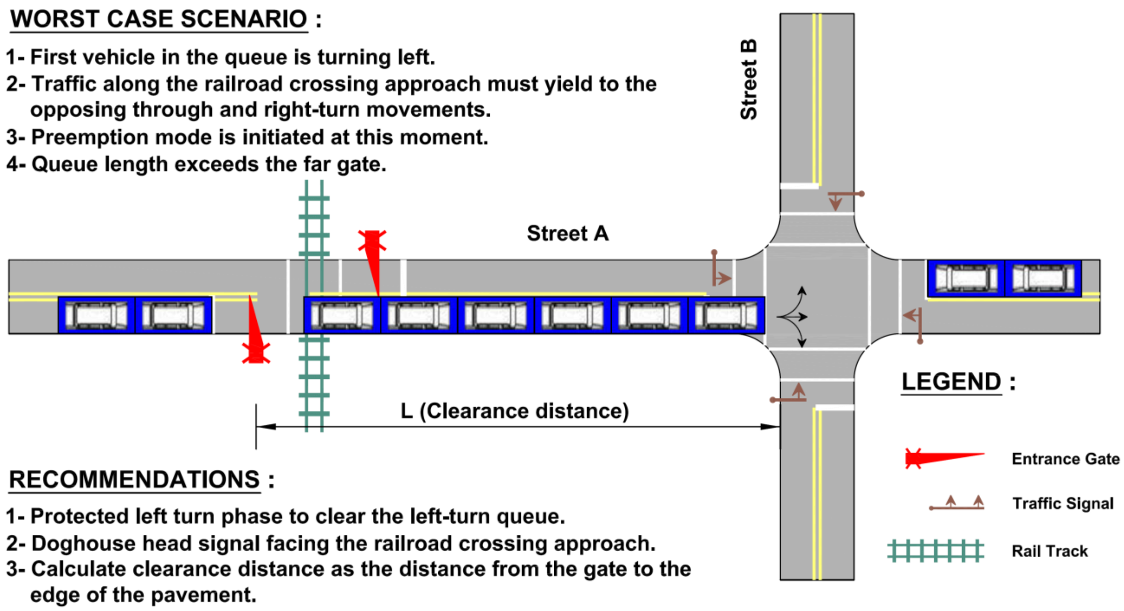


Figure 30. Clearance Distance on a Two-Lane Roadway When Queue is Extending beyond the Tracks (Gate Present without a Pre-Signal)

Case 2: Four-Lane Road

There are two scenarios for a four-lane road where the vehicles are queued within the clear storage zone, or they encroach on the train safety envelope. Shown in figure 31, it is necessary to clear the 95th percentile queue during the track clearance phase in order to avoid having vehicles stuck between the railroad crossing and the intersection during the holding phase. Figure 32, all the vehicles queued between the far gate and the intersection have to be cleared during the track clearance phase. A protected left-turn phase is required to ensure clearing the vehicles turning left. A doghouse signal head should be installed facing the railroad crossing approach.

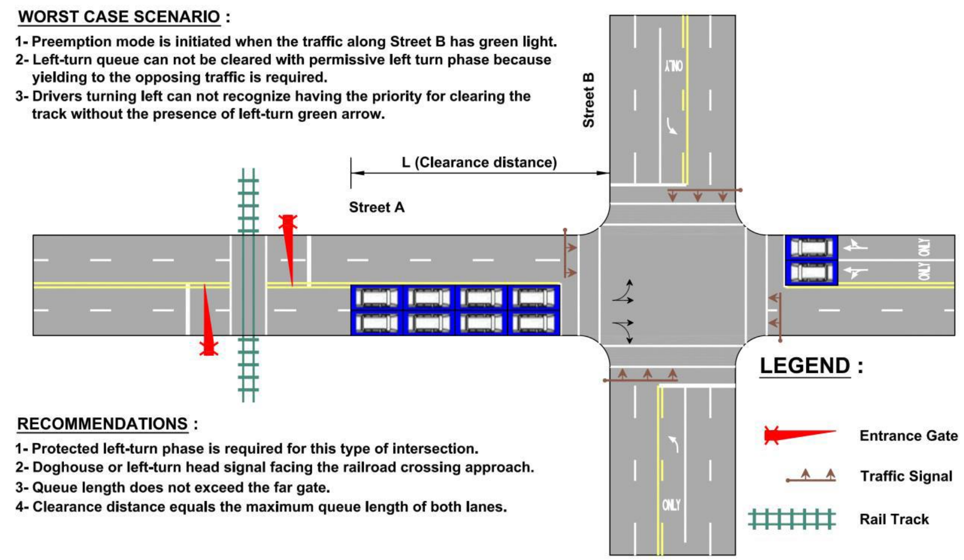


Figure 31. Clearance Distance on a Four-Lane Roadway When Queue is Not Extending beyond the Tracks (Gate Present without a Pre-Signal)

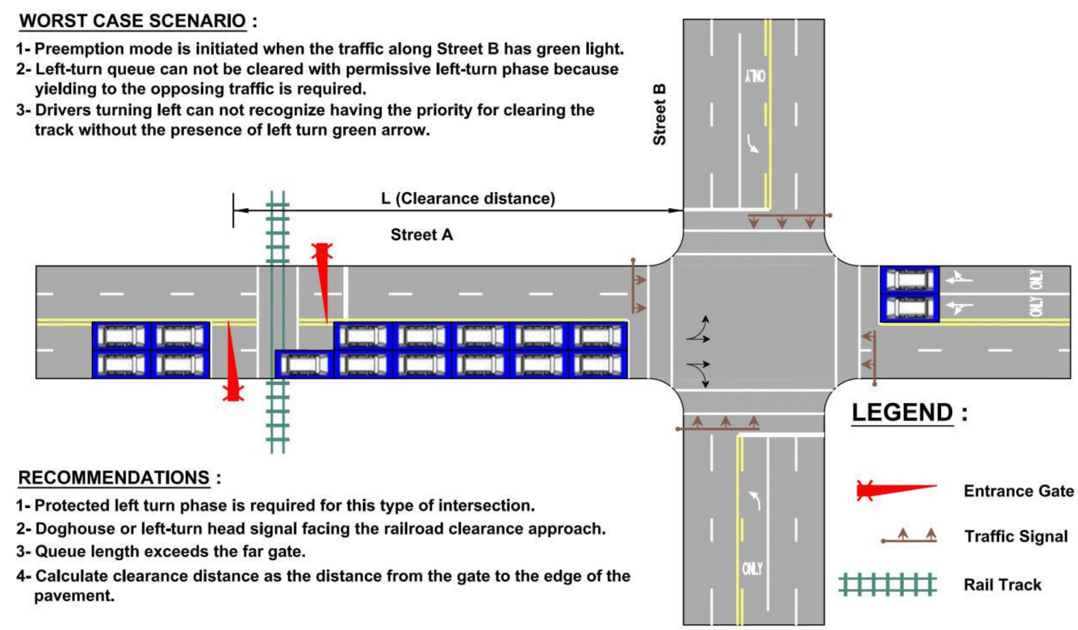


Figure 32. Clearance Distance on a Four-Lane Roadway When Queue is Extending beyond the Tracks (Gate Present without a Pre-Signal)

Case 3: Roadway with an Exclusive Left-Turn Lane

The queue length may or may not extend beyond the gate as shown in Figures 34 and 33. When the queue length is within the clear storage zone, it is necessary to clear the 95th percentile queue pertaining to the critical lane. A protected left-turn phase is necessary when the estimated left-turn queue exceeds the available storage space.

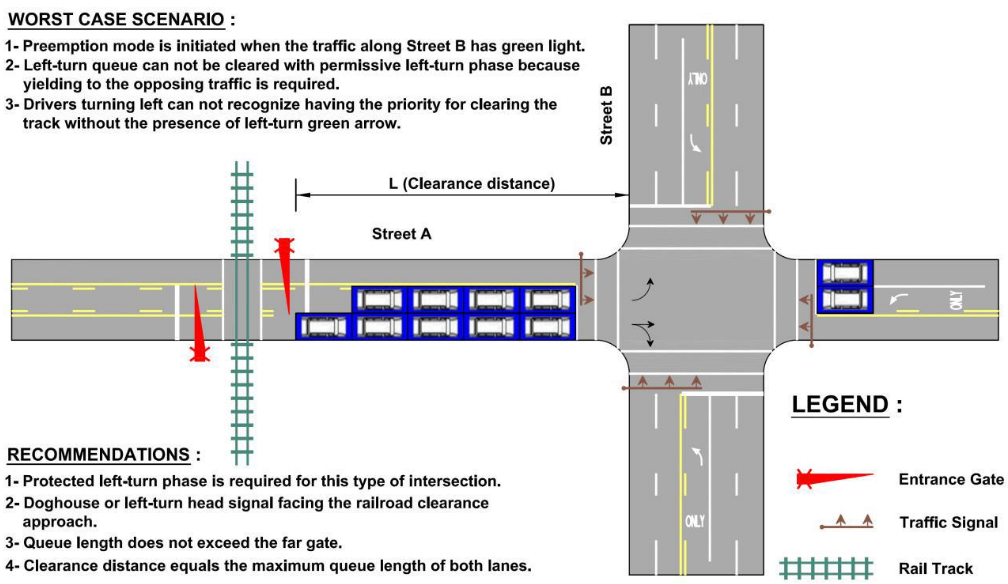


Figure 33. Clearance Distance on a Roadway with an Exclusive Left-Turn Lane When Queue is Not Extending beyond the Tracks (Gate Present without a Pre-Signal)

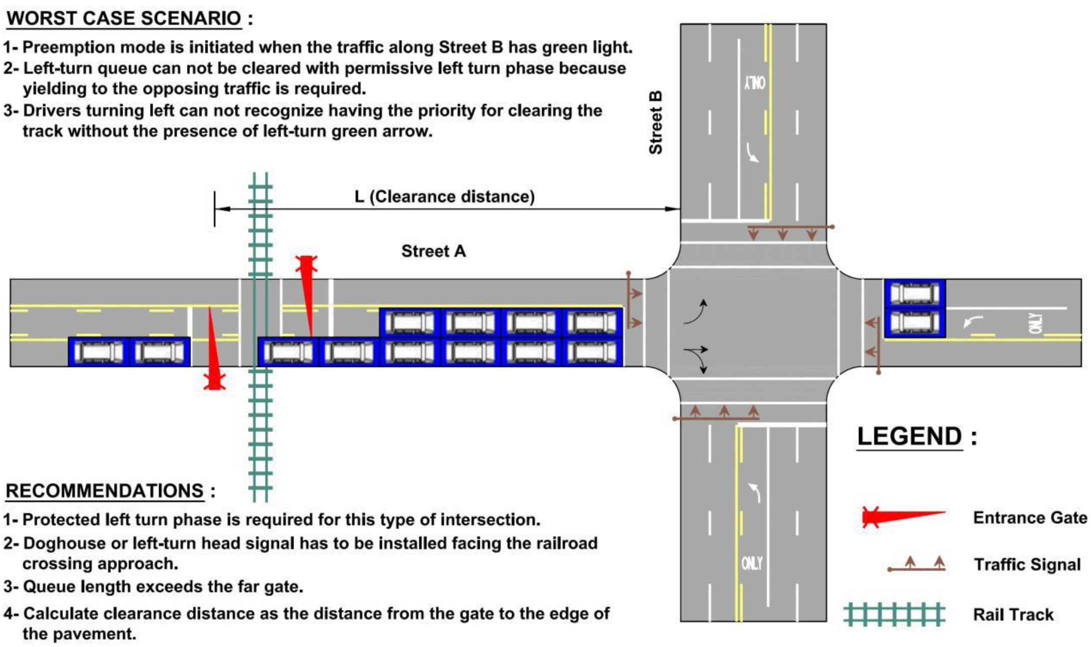


Figure 34. Clearance Distance on a Roadway with an Exclusive Left-Turn Lane When Queue is Extending beyond the Tracks (Gate Present without a Pre-Signal)

5.3 Railroad Crossing in the Vicinity of a Boulevard

For boulevards where only through lanes are present with no left-turn lane (Figure 35), the worst case scenario is considered to be the one when the traffic along the parallel approach to the tracks is moving. In this case if the preemption is called, there should be no vehicle queue on the other side. However, to assure safety, a 10-15 second track clearance time should be provided, if a vehicle runs the red light and gets stuck on the tracks, and needs to be cleared before the train arrives.

If there is no pre-signal, the clearance distance will be minimum of (a) 95th percentile queue length among all the lanes (b) the distance between the far side gate and the edge of the near side pavement.

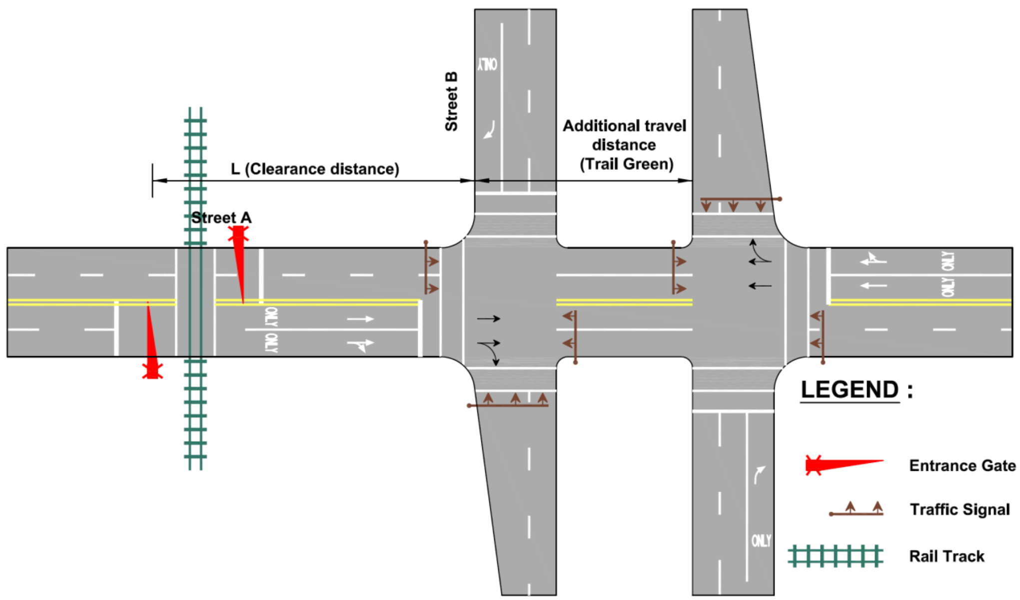


Figure 35. Clearance Distance When a Railroad Crossing is Located Near a Signalized Boulevard Intersection

5.4 Diagonal Railroad Crossings

At some railroad crossing locations, the railroad tracks may cross two approaches of an intersection which creates a critical situation regarding track clearance requirements, especially when the distance from the crossing to both lefts of the intersection are within 200 feet (Figure 36). Two track clearance intervals are required in order to ensure that no vehicles are trapped on the tracks prior to the arrival of the train. Lowering the gates closest to the tracks which control the vehicles leaving the intersection, should be delayed to clear all vehicles turning toward the railroad crossings during the track clearance intervals. As a consequence, the calculation of the minimum warning time is controlled by the time necessary for the last vehicle to clear the first railroad crossing approach, to turn to the intersection toward the second railroad crossing, and then clear it. It is expected that the minimum warning time may exceed 60 seconds which conflicts with Michigan Law (RAILROAD CODE OF 1993 (EXCERPT) Act 354 of 1993). Solving this can be achieved by:

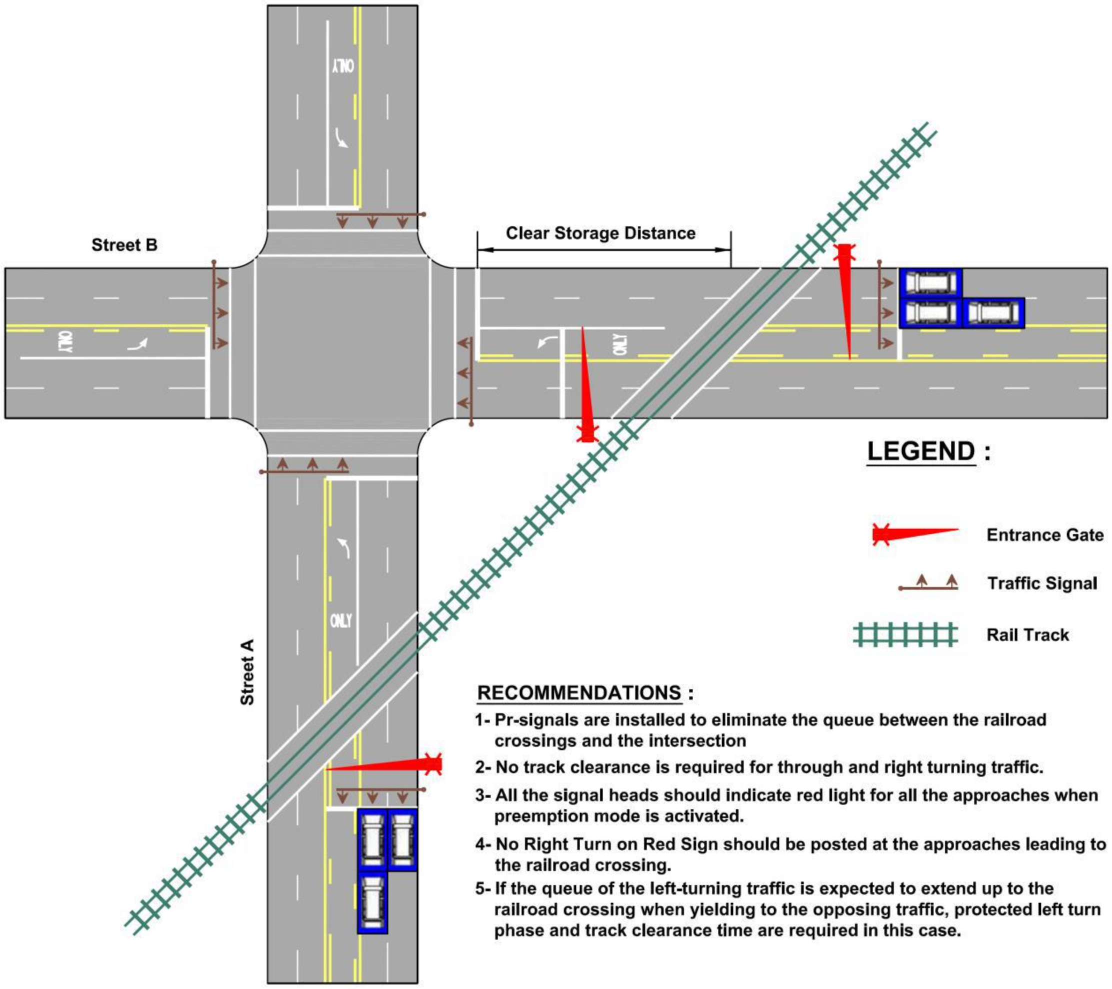


Figure 36. Skewed Railroad Crossing Two Approaches Near a Signalized Intersection

Utilizing queue prevention strategies such as pre-signals or queue cutter signals in order to ensure that there will not be any queued vehicles between the crossing and the intersection. According to ITE recommendations *(36)* and after an engineering study is completed, pre-signals can be installed at railroad crossing approaches to have a clear storage distance less than 120 feet. When the clear storage is greater than 120 feet, the pre-signal should be considered as “queue-cutter” signal. This has an adverse effect on the delay during normal operation mode because during each cycle, two trail green periods are required to clear the queue between the pre-signal and the intersection. When preemption mode is initiated, there is no need for track clearance phases after termination of the current phase due to the absence of vehicles queued between the pre-signal and the intersection. The engineer, however, may provide a minimum track clearance green of 15 seconds to clear the vehicles that may have violated the red light. If the left-turn traffic volumes generate a queue length extending beyond the railroad crossing, when yielding to the opposing through and right turning movements, a protected left-turn phase and track clearance green time will be necessary.

- Advance preemption time can be utilized to terminate the active phase and enter into track clearance for one of the railroad crossings before the railroad warning devices are turned on. After that, the warning time is used to cover the remaining track clearance of the vehicles as well as provide a separation time before the arrival of the train.
- In case the queue study for a diagonal crossing concludes that during normal operation mode only one of the railroad crossing has a queue length exceeding the clear storage distance (the second approach has enough clear storage distance to fit the queue without encroaching on the tracks), track clearance green time is required for this approach only.

5.5 Railroad Crossings in the Middle of the Intersection

At some locations where the rail tracks cross in the middle of an intersection, as shown in Figure 37, no track clearance interval is required. The traffic signal shall indicate red for all approaches.

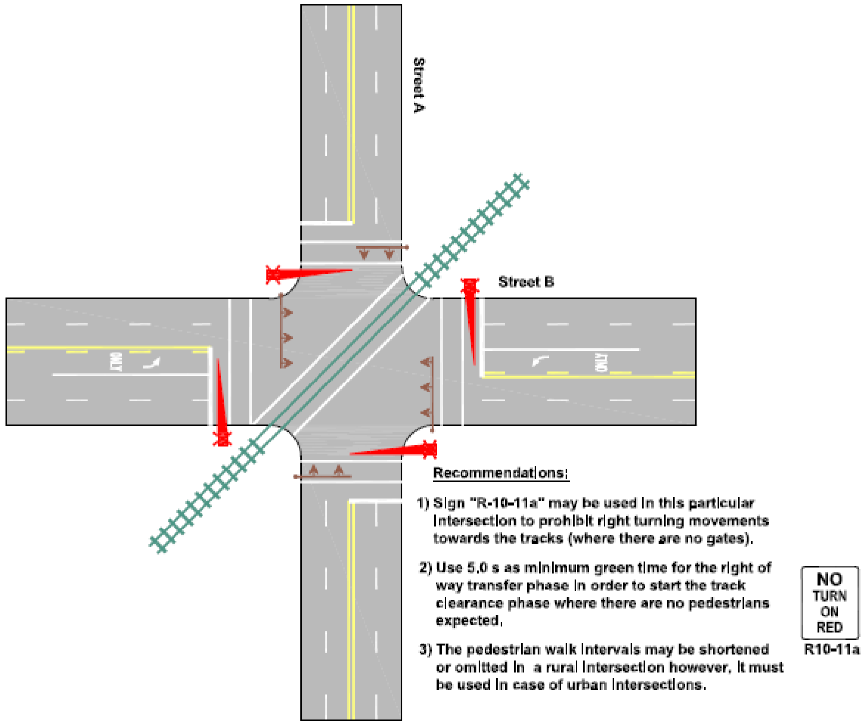


Figure 37. Railroad Crossing through the Middle of a Signalized Intersection

5.6 Queue Clearance Time

Track green time (or queue clearance time) is defined in the USDOT Railroad-Highway Grade Crossing handbook *(32)* as "*The time required for the design vehicle of maximum length stopped just inside the minimum track clearance distance to start up, move through, and clear the entire minimum track clearance distance. If pre-signals are present, this time shall be long enough to allow the vehicle to move through the intersection or to clear the tracks if there is sufficient clear storage distance. If a four-quadrant gate system is present, this time shall be long enough to permit the exit gate arm to lower after the design vehicle is clear of the minimum track clearance distance.*" The track clearance phase includes the following four intervals:

1. Minimum track clearance green time

2. Track pedestrian clearance interval

3. Yellow clearance interval

4. All red clearance interval

5.6.1 Minimum Track Clearance Green Time

The minimum track clearance green time is determined based on time necessary for a vehicle stopped on the tracks to safely enter the intersection. It is equal to the sum of (1) the start-up delay for a vehicle positioned on the tracks and (2) the subsequent travel time for the vehicle to enter the intersection, which is determined as follows:

1. Start-up delay time. This is equal to the time for the vehicle positioned on the tracks to start moving after the start of the green indication. It is influenced by both the type and number of vehicles in the queue.

2. Travel time for the vehicle to enter the intersection. This is a function of the clearance distance, approach grade, and vehicle type after the initial start-up delay.

This procedure is based on the methodology described in references *(33)* and *(38)*. Tables 1, 2 and 3 present the total track clearance green time required to clear a standing queue of length ‘L’, which represents the distance from the near edge of the intersection **across the tracks to the far side gate or flashers**. Vehicle lengths of 25 feet and 61 feet are assumed for each passenger vehicle and WB-50 (50 feet total wheelbase) tractor trailer truck, respectively, are utilized to determine the start-up delay and travel time to enter the intersection. Table 1 specifically presents the track clearance time considering only passenger cars in the queue. Tables 2 and 3 calculate the queue clearance time considering passenger cars with one and two WB-50 trucks positioned at the end of the queue. Flat grades are assumed for both. Yellow and all-red clearance intervals must also be utilized, per MDOT standard clearance interval calculation procedures.

Queue clearance time is a function of many factors such as the queue length, type of vehicle, driver behavior, the acceleration of vehicles, and weather conditions. It is divided into two parts: 1) time required for the vehicle to start moving and 2) travel time of the vehicle to cross the intersection.

Start-up Time

For track clearance interval, the control vehicle is the last vehicle in the queue. The driver has to wait for the preceding vehicles before he is able to start moving. This start up time is influenced by the type and the number of vehicles in the queue. The Florida study *(38)* suggested using the simple linear model shown below for determining the time required for the nth vehicle to start moving: $dn=\tau+n*T$

Where

dn = Start up delay for the nth vehicle in a queue (sec)

τ = Excess startup time of the lead vehicle in a queue (sec)

n = Position of a specific vehicle in a queue (n=1, 2, 3,…) and

T = Uniform startup response time of each driver in a queue (sec)

(Assumption: vehicle length = 25 feet)

T and τ values adopted by Texas Transportation Institute in "*Guide for Traffic Signal Preemption near Railroad Grade Crossing"* are used in this study project to determine the start-up time for passenger cars and WB 50 Trucks. The values are as follows.

Passenger car: T = 1.2s, τ = 1.0 s

Truck WB 50: T=1.0s, τ = 3.0 s

Vehicle Travel Time

It is the time necessary for the last vehicle in the queue to accelerate from the stopped condition and cross the intersection. The Florida study *(33)* develops a model to predict the travel distance of the vehicle as a function of the travel time and the grade of the roads:

$d=(\alpha\pm Gg\beta)t-(\alpha\pm Gg\beta - v_o)(1-e-\beta t)\beta$

d = Clearance distance in feet (includes rear bumper traveling length of the vehicle and the clearing distance)

t = Vehicle travel time (sec) from a stopped position to the distance'd'.

v_o = Initial speed, set to 0 for starting from rest (fps)

G = Average road surface grade over repositioning distance, set to 0 for flat grade (ft. /ft.)

g = Gravitational constant (32.174 fps2 near sea-level)

α = Initial vehicle acceleration from rest (fps2), and

β = rate of reduction in acceleration with increasing speed (sec-1)

The values for α and β derived from the traffic observations in the study conducted by Gary Long (2000) are as follows.

Passenger car; α = 6.6, β= 0.12

Truck WB 50; α = 1.2, β= 0.02

Table 21 provides the computed values of total queue clearance green time for a clearance distance 'L' using the above explained procedure. Extended tables considering queues containing passenger cars and trucks are provided in Appendix B.

Table 21. Queue Clearance Green Time for a Clearance Distance 'L' (passenger cars only)

NUMBER OF PASSENGER CARS IN QUEUE*	L = CLEARANCE DISTANCE (FT)	START UP DELAY TIME (S)	TRAVEL TIME TO ENTER INTERSECTION AFTER INITIAL START UP DELAY (S)	TOTAL QUEUE CLEARANCE GREEN TIME (S)	QUEUE CLEARANCE GREEN TIME ROUNDED VALUES (S)
1	25	2.2	3.0	5.2	10.0
2	50	3.4	4.3	7.7	10.0
3	75	4.6	5.3	9.9	10.0
4	100	5.7	6.3	12.0	12.0
5	125	6.9	7.1	14.0	14.0
6	150	8.1	7.9	16.0	16.0
7	175	9.3	8.5	17.8	18.0
8	200	10.5	9.3	19.8	20.0
9	225	11.7	10.0	21.7	22.0
10	250	12.8	10.6	23.4	24.0
11	275	14.0	11.4	25.4	26.0
12	300	15.2	11.8	27.0	27.0
13	325	16.4	12.4	28.8	29.0
14	350	17.6	13.0	30.6	31.0
15	375	18.8	13.6	32.4	33.0
16	400	20.0	14.1	34.1	35.0

*Each car is assumed as 25 feet, which includes nominal buffer spacing.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

Analysis of traffic queues at signalized intersections in close proximity to railroad grade crossing is crucial for determining if the signal needs to be preempted for safe operation of vehicular and train traffic at such locations. It is therefore important to adequately analyze traffic queues at such locations in order to implement safe design that will minimize the risk of future crashes. In order to help selecting an adequate method for analyzing traffic queues at such locations, this research included:

- Review of state-of-the-art
- Review of state-of-the-practice
- Analysis of queue estimation procedures
- Comparison between different procedures
- Formulation of recommendations regarding queue estimation procedures

The state-of-the-art and state-of-the-practice review findings are as follows.

1. Deterministic analytical models for determining queue lengths at signalized intersections have certain limitations that can impact the resulting estimates on traffic queues. Some of the major shortcomings identified in the state-of-the-art include 1). unable to account for any variations in traffic conditions that are different from their model assumptions 2). unable to account for various important traffic factors such as vehicle mix, speeds, overflow queues, dynamic driver behaviors and vehicle interactions, and complex signal phasing (such as permissive-protected left-turns).

2. Most states' MUTCD supplements follow the 200 feet rule for available clear storage distance as provided in the federal MUTCD for identifying candidate intersections for preemption.
3. Only a few states' design manuals/MUTCD supplements provide guidelines on considering queue build-up characteristics at signalized intersections that encroach onto the nearby railroad track in spite of being located beyond the 200 feet envelope. Very few states were found to have additional details in their design manuals regarding methods to use for queue estimation at such locations. The few states that provide guideline in this regard refer to either simple analytical formula recommended by ITE *(7)* or using the nomograph presented in the ITE journal *(33)*.
4. Most of the states' responses to an online survey showed 200 feet rule and 'observing queue lengths in the field' as base criteria for determining if signal preemption is needed. The responses that included queue estimation mostly used Synchro procedure. Second most frequently used methods included HCS and Sim-Traffic. Only one response selected VISSIM in addition to selecting other methods including HCS, Synchro, and Sim-Traffic.

<u>Conclusions on Existing Methods for Estimating Queues</u>

To further investigate the application of the existing commonly used methods for queue estimation and to help select an appropriate method to be used for preemption evaluation, this research analyzed and compared different existing analytical models (i.e. HCS, Syncho, Railroad Assessment Tool) and microscopic simulation based methods including Sim-Traffic and VISSIM. There are differences between each method in defining and/or computing capacity, delay and queues, default assumptions on vehicle lengths, and car-following parameters, which impact the resulting queue estimates. Additionally, macroscopic models i.e. HCS and Synchro do not account for various dynamic traffic factors that can impact the resulting estimation on queues.

For example, HCS and Synchro models do not include impacts on the resulting performance measures (delays, queues) due to variations in vehicle characteristics, roadway speeds, vehicular interactions between lanes, driver behaviors, lane overflow/blockage, impact of residual queues, complex signal phasing/traffic operations. Microscopic-simulation models (Sim-Traffic, VISSIM) have the capability to account for the aforementioned traffic factors in computing delays and obtain information on individual vehicles position, speed, deceleration, acceleration etc. per small increment of time (per second or less than a second). In addition, these modeling techniques provide calibration of various parameters to capture the actual traffic conditions at a location. The findings and conclusions on the comparisons between these methods on estimating queues are summarized below.

1. HCS computed queue length estimates are generally greater than all the other methods (Synchro, Railroad Assessment Tool, Sim-Traffic, VISSIM) particularly at under-saturated traffic conditions and for relatively less complex traffic operations (such as thru lanes and no turning traffic) or where traffic factors such as vehicle mix, speed, interaction between different lanes, over-flow and blockage of turn lanes, residual queues, and other driver behavior related factors do not have a significant impact on vehicular delays and queues.
2. Synchro provides lower estimates for capacity and queue lengths than HCS and Sim-Traffic. The highest capacity and lowest queue estimates were obtained from VISSIM simulation models using the default parameters on vehicle and driver related characteristics.

3. For estimating left-turn queues under permissive control the macroscopic models (HCS, Synchro) tend to provide lower estimates (particularly between v/c 0.5-1.0) than the micro-simulation models. More importantly, macroscopic models cannot adequately capture the impact of traffic interactions between adjacent and opposing lanes particularly operating at higher v/c ratios (approximately between 0.5-1.0). HCS results on queues were found lower than Sim-Traffic while Synchro provided lower estimates than VISSIM. For v/c ratios between 0.5 and 1.0, Sim-Traffic estimates on left-turn queues were significantly higher than all other methods.
4. Two simulation models Sim-Traffic and VISSIM were also analyzed and compared in this research. Both Sim-Traffic and VISSIM use Poisson arrival rates (exponential inter-arrival headways) to describe vehicle arrival. At over-saturated states or $v/c \geq 1$, the queues generated by these models tend to infinity based on the standard queueing theory assumptions of indefinite over-saturated period. Using the default model parameters for car-following, driver behaviors, and vehicle characteristics etc., VISSIM model provides higher capacity and lower delay and queue length estimates as compared to Sim-Traffic. These differences in queue output are attributed to the differences in various model parameters impacting the resulting extent of queue. These include the differences in functions describing arrival and discharge rates, headway distributions, acceleration functions, default parameters used for driving behaviors, vehicle characteristics, and factors used to model saturation flow rates etc. Additionally, when comparing 95th percentile queues, the difference is also attributed to the different percentile computation methods used in both the methods.

5. A comparison of the maximum queues observed at a local site was performed as part of this research using the data on intersection geometry, traffic volumes, signal control, speeds and heavy vehicles. No other model calibration was done. The results showed that the maximum number of vehicles in queue estimated through VISSIM was lower than the observed maximum queues (from 1 to 2 passenger cars). Sim-Traffic queue estimates were found slightly above the observed maximum number of vehicles (up to 1 passenger car). The 95th percentile estimates from Sim-Traffic were also found to be close to the maximum observed queues (particularly left-turn queue).

Recommendations

Since the MUTCD standard, which is the practice followed among most of the states, requires preempting all traffic signals within 200 feet of the at grade railroad grade crossing, evaluations would be needed for determining preemption need at those intersections which are beyond 200 ft. from the crossing. This makes such locations (or queue estimation beyond 200 feet.) more critical for adequate analysis. The analysis of various queue lengths obtained from different methods showed that the differences among these methods increase as the demand increases beyond50% of the capacity ($v/c \geq 0.5$) or when queues are much longer than 200 ft. It should also be recognized that at or beyond saturated state ($v/c \geq 1$), queue estimates are not reliable both from the deterministic analytical and simulation models. At such locations which are at greater distances and operating near capacity or under poor level of service, other strategies (such as signal timing improvement) should be considered to improve the LOS and mitigate the formation of long traffic queues before considering/implementing signal preemption. Other recommendations regarding using different methods for queue estimation are as follows.

- The deterministic analytical tools (HCS, Synchro, Railroad Assessment Tool) are significantly inadequate for analyzing many important traffic factors that can significantly impact the queue estimation from these methods such as dynamic traffic flow, vehicular interactions between lanes, driver behaviors in lane changing and turning, vehicle characteristics, speeds, overflow queues, impact of opposing traffic, lane overflow/blockage, complex traffic operations such as permissive-protected left-turn phasing etc. The impact of these factors is even greater at higher v/c ratios. It is recommended to use micro-simulation based methods for more detailed and adequate analysis of traffic conditions particularly when the aforementioned factors are involved.
- Two commonly used micro-simulation techniques include Sim-Traffic and VISSIM. Some findings on the differences that impact the resulting queue lengths in both methods, and suggestions on the suitability of using them are as follows.
  - Using the default vehicle characteristics, car-following and driver behavior parameters, Sim-Traffic simulation model typically results in greater queue length estimates than VISSIM simulation models mainly because of the differences in arrival and discharge rates, headway distributions, vehicle characteristics, acceleration functions, and default factors used to model saturation flow rates. Additionally, when comparing 95th percentile queues, the difference is also attributed to the different percentile computation methods used in both the methods.
  - Parameters defaults describing the traffic flow, driver behaviors, vehicle lengths and characteristics etc. in VISSIM are based on the studies conducted in Germany which may not adequately represent (unless calibrated) US driving behaviors and vehicle characteristics.

- Referring to some of the previous studies *(38, 39)* on left-turn queues that involved field data to calibrate and validate the models while comparing Sim-Traffic, VISSIM and other queue estimation methods found that Sim-Traffic in most cases closely matched the observed left-turn queues.

6. A comparison of the maximum queues observed at a local site showed that the maximum number of vehicles in queue estimated through VISSIM (using default parameters) was lower than the observed maximum queues (from 1 to 2 passenger cars). Sim-Traffic queue estimates (using default parameters) were found slightly above the observed maximum number of vehicles (up to 1 passenger car) which is preferable than under-estimation. The 95th percentile estimates from Sim-Traffic were also found to be close to the maximum observed queues (particularly left-turn queue).

- VISSIM software does provide additional flexibilities in model calibration as compared to Sim-Traffic however that would require extensive field data to properly calibrate and validate the model.

Due to the above described factors, it is recommended that if limitations in collecting detailed field data or time constraints exist, or the default traffic flow model parameters and vehicle characteristics are being used, Sim-Traffic is a preferable approach particularly for US locations.

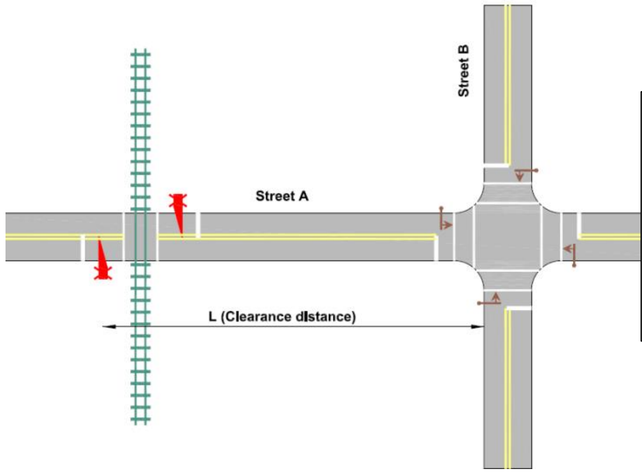
The analysis of different queue estimation methods presented in this research should facilitate in recognizing the specific limitations of different methods when estimating queues at signalized intersections near highway-rail grade crossings. Based on the analysis findings, a procedure is developed using Sim-Traffic microscopic simulation technique for estimating queue lengths on various existing signalized approaches under commonly observed ranges of traffic, vehicle-mix and signal control parameters. Additionally, if preemption is needed procedures and recommendations are provided for determining queue clearance distance and time.

The procedures and recommendations developed in this research are formulated to minimize the risk of underestimated queues or unsafe design at such locations, and simplify the design and decision-making process.

APPENDIX A

TEMPLATES FOR ESTIMATING QUEUES AT SIGNALIZED INTERSECTION APPROACHES

Template 1: Queue Estimation on a Single Lane Approach- Left-Through-Right Shared



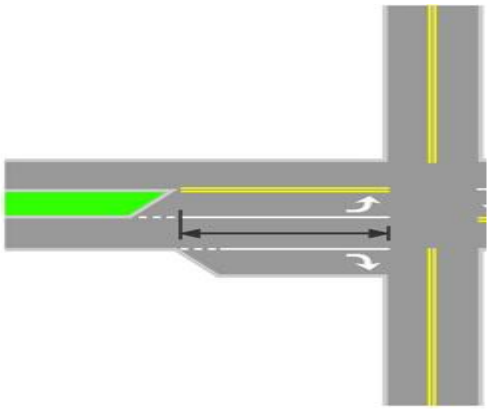
Factors required for queue estimation

- Lane Traffic Volume during the critical hour (vph)
- Cycle Length (s)
- Effective Green time (s) for through movement
- Percentage of heavy vehicles
- Opposing Traffic Volume (vph)

Cycle Length (s)	Effective Green (s)	Critical Lane Vol (vph)	Oppsing Lane Vol (vph)	95th Percentile Queue Length (ft.)					
				2% HV		5% HV		10% HV	
				Critical Ln	Opposing Ln	Critical Ln	Opposing Ln	Critical Ln	Opposing Ln
		50	100	38	55	38	66	47	68
		100	150	62	77	65	91	74	101
		150	200	85	99	95	113	96	115
		200	250	98	131	112	125	117	135
60	25	250	300	122	145	128	155	139	156
		300	350	169	186	168	191	192	212
		350	400	211	236	215	243	263	275
		400	450	541	786	579	523	840	786
		450	500	1083	1232				
		50	100	52	66	51	72	47	74
		100	150	71	96	85	102	83	112
		150	200	106	118	108	131	122	133
		200	250	116	157	133	159	126	171
80	35	250	300	150	179	160	203	181	190
		300	350	195	226	184	221	236	262
		350	400	256	260	264	278	265	318
		400	450	460	453	578	590	872	834
		450	500	1196	1204				
		50	100	45	69	47	77	57	81
		100	150	71	96	81	110	95	113
		150	200	115	128	117	147	130	155
		200	250	130	175	149	167	139	188
100	45	250	300	166	200	166	212	181	217
		300	350	205	245	209	238	238	257
		350	400	273	336	267	304	284	336
		400	450	707	790	676	783	986	806
		450	500						

Note: *Average passenger car length including spacing~19.5 ft. (to convert into equivalent no. of vehicles, divide the queue lengths by 19.5)*
Shaded area shows the left- lane overflow state (queue estimation not reliable at or beyond this point).

Template 2: Approach with Single Through Traffic Lane and Exclusive Turn lanes



Factors required for queue estimation

- Approach Traffic Volume (Thru and turning traffic) during the critical hour (vph)
- Cycle Length (s)
- Effective Green time (s) for through movement
- Percentage of heavy vehicles

CYCLE LENGTH=60s, EFFECTIVE GREEN=25s

Approach Volume (vph)	Thru Lane Volume (vph) *10%turning traffic*	**95th Percentile Queue Length (feet)**								
		2%HV			**5%HV**			**10%HV**		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
100	80	29.5	64.5	21	33.5	73.5	25.5	32.5	80	23.5
150	120	38	79	32	36	87.5	32	43.5	95.5	35
200	160	37	68.5	67.5	45.5	105.5	36.5	45.5	110	41
250	200	47.5	117	43	66	124	60.5	54.5	124.5	44.5
300	240	51	136.5	60.5	51.5	137.5	57.5	64.5	149.5	58
400	320	69.5	168	78	67	176.5	73.5	75.5	184.5	76.5
500	400	88	211.5	89	95.5	217.5	90	95.5	247	94.5
600	480	107.5	257.5	105.5	114.5	285.5	107	120	304.5	106.5

Note: *Average passenger car length including spacing~19.5 ft. (to convert into equivalent no. of vehicles, divide the queue lengths by 19.5)*

CYCLE LENGTH=80s, EFFECTIVE GREEN=35s

Approach Volume (vph)	Thru Lane Volume (vph) *10%turning traffic*	95th Percentile Queue Length (feet)								
		2%HV			5%HV			10%HV		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
100	80	26.5	73	23.5	33	79.5	30.5	31.5	86.5	32
200	160	38	108	40	41.5	113.5	45.5	44	121.5	42
250	200	53	132	46	56	131	56.5	60	141	51.5
300	240	54.5	152.5	58	62.5	161.5	62	68	164.5	67.5
400	320	71.5	187.5	83	76.5	205.5	77.5	91	217.5	90
500	400	93.5	238	100.5	92	254.5	100	96	283	98
600	480	112.5	302.5	120	109.5	315.5	117.5	117	345.5	114

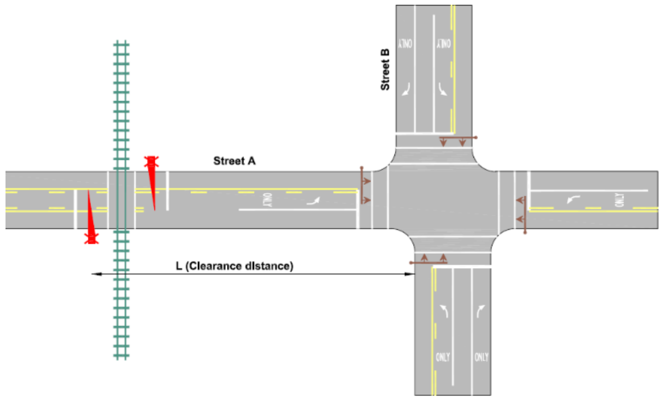
Note: *Average passenger car length including spacing~19.5 ft. (to convert into equivalent no. of vehicles, divide the queue lengths by 19.5)*

CYCLE LENGTH=100s, EFFECTIVE GREEN=45s

Approach Volume (vph)	Thru Lane Volume (vph) *10%turning traffic*	95th Percentile Queue Length (feet)								
		2%HV			5%HV			10%HV		
		Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
100	80	30	81	26	29.5	77.5	30.5	29	85.5	32
150	120	40	98	31.5	32.5	102	39	44	107.5	41
200	160	41.5	124	47	42	124.5	52.5	43	135	53.5
250	200	49	142.5	52.5	56.5	148	60	57	149	59
300	240	63.5	173	68.5	58.5	178.5	63.5	75	182.5	73
400	320	78.5	220	90.5	84	225.5	93.5	92	242.5	82
500	400	102.5	270	120	103	291	115.5	102	319	112.5
600	480	118.5	340	124	119.5	345	127	121	389	130.5

Note: *Average passenger car length including spacing~19.5 ft. (to convert into equivalent no. of vehicles, divide the queue lengths by 19.5)*

Template 3: Queue Estimation on an Approach with Exclusive Left-Turn Lane



Factors required for queue estimation
- Approach Volume (vphpl) during the peak/critical hour
- Cycle Length (s)
- Effective Green time (s) for left turns
- Percentage of heavy vehicles
- Opposing traffic volume (vph)
- Left-Turn Phasing

Permissive Left Turn Phasing

With 2% Heavy Vehicles in Traffic

Traffic Volume (2%HV)		Left Lane Queue Length in feet (95th percentile)										
Approach Vol (vph)	Left Ln Vol (vph)	*Opposing Traffic (vph)- (1 Opposing Lane)*						*Opposing Traffic (vph) (2 Opposing Lanes)*				
		100	200	300	400	500	600	350	450	500	600	700
CL=60s , Effective Green=24s												
400	100	51	62	85	114	195	547			89	105	212
450	150	88	120	151	254	667	638	88	131	237	463	719
500	200	108	149	226	593	633	633	155	286	689		
550	250	157	200	610				296	754			
CL=80s, Effective Green=34s												
400	100	75	68	93	120	182		58	78	102	115	155
450	150	113	123	143	370	763		105	138	186	213	699
500	200	140	150	243	780			157	277	575		
550	250	165	226	693				239	563	711		
CL=100s, Effective Green=44s												
400	100	73	76	98	109	190		78	83	88	131	150
450	150	123	128	162	300	674		120	144	175	296	669
500	200	146	165	203	694			155	207	435	766	
550	250	176	239	502				262	598			

Note: *Average passenger car length including spacing~19.5 ft. (to convert into equivalent no. of vehicles, divide the queue lengths by 19.5)*
Shaded area shows the left- lane overflow state (queue estimation is not reliable at or beyond this point).

With 5% Heavy Vehicles in Traffic

Traffic Volume (5%HV)		Left Lane Queue Length in feet (95th percentile)										
Approach Vol (vph)	Left Ln Vol (vph)	*Opposing Traffic (vph)-5%HV (1 Opposing Lane)*						*Opposing Traffic (vph)-5%HV (2 Opposing Lanes)*				
		100	**200**	**300**	**400**	**500**	**600**	**300**	**400**	**500**	**600**	**700**
CL=60s , Effective Green=24s												
400	100	62	76	77	122	301		74	101	80	149	283
450	150	105	110	138	468			110	123	205	642	
500	200	127	163	283				229	346	864		
550	250	164	189	717				343				
CL=80s, Effective Green=34s												
400	100	79	91	91*	128	228		69	91	107	122	136
450	150	110	124	167	300			150	136	200	608	
500	200	148	179	352	689			170	291	740		
550	250	184	208	574				499	724			
CL=100s, Effective Green=44s												
400	100	95	94	112	186	179	720	74	84	89	118	170
450	150	130	145	171	266	756		123	125	174	485	603
500	200	155	214	310	771			196	273	602		
550	250	205	236	617				293	706			

With 10% Heavy Vehicles in Traffic

Traffic Volume (10%HV)		Left Lane Queue Length in feet (95th percentile)										
Approach Vol (vph)	Left Ln Vol (vph)	*Opposing Traffic (vph)-10%HV (1 Opposing Lane)*						*Opposing Traffic (vph)-10%HV (2 Opposing Lanes)*				
		100	**200**	**300**	**400**	**500**	**600**	**300**	**400**	**500**	**600**	**700**
CL=60s , Effective Green=24s												
400	100	81	74	88	135	390		83	109	135	114	253
450	150	101	120	211	276			156	217	242	661	
500	200	154	188	282				295	590			
550	250	171	276									
CL=80s, Effective Green=34s												
400	100	81	83	105	150	358		89	101	98	149	245
450	150	109	130	215	369			115	143	198	483	
500	200	152	186	272				200	412	762		
550	250	178	264	613				298				
CL=100s, Effective Green=44s												
400	100	115	91	115	150	438		88	115	139	136	277
450	150	109	130	215	369			153	152	197	582	652
500	200	152	186	272				185	254	704		
550	250	178	264	613				326	748			

Note: *Average passenger car length including spacing~19.5 ft. (to convert into equivalent no. of vehicles, divide the queue lengths by 19.5)*
Shaded area shows the left- lane overflow state (queue estimation not reliable at or beyond this point).

Permissive-Protected Left Turn Phasing

Traffic Volume		95th Percentile Left Turn Lane Queue				
Approach Vol (vph)	Left Turn Vol (vph)	*Opposing Vol (vph) 1 Opposing Thru Lane*			*Opposing Vol (vph) 2 Opposing Lanes*	
		100	**300**	**500**	**300**	**500**
CL=80s (Thru Phase=25s, Protected Phase=15s)						
200	50	64	65	68	67	70
250	100	94	99	113	97	97
300	150	119	142	~142	126	134
350	200	148	160	196	161	170
CL=100s, (Thru Phase=35s, Protected Phase=15s)						
400	100	99	112	126	103	124
450	150	131	146		137	151
500	200	166	200		170	194
550	250	192	241		199	282
CL=120s, (Thru Phase=40s, Protected Phase=20s)						
400	100	109	131	132	111	126
450	150	135	162	194	163	171
500	200	190	200	293	197	218
550	250	211	257	384	225	287

Protected-Only Left Turn Phasing

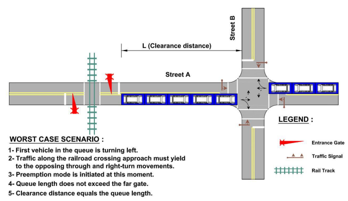
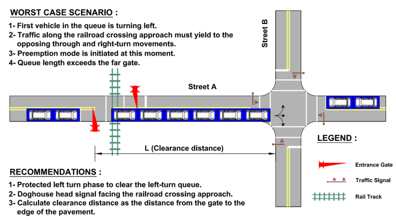
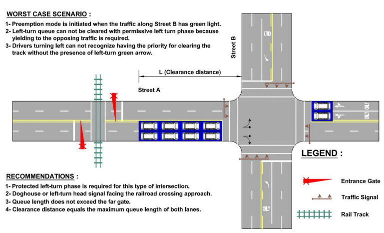
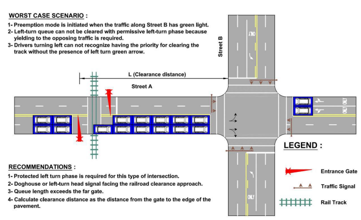
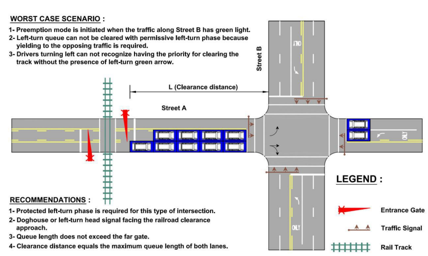
Approach Vol (vph)	Left Turn Vol (vph)	95th Percentile Left Turn Lane Queue		
		2%HV	**5%HV**	**10%HV**
CL=100s, (Thru Phase=35s, Protected Phase=15s)				
200	50	80	93	90
250	100	137	151	153
300	150	258	310	306
350	200			
CL=120s, (Thru Phase=40s, Protected Phase=20s)				
400	100	138	149	153
450	150	212	225	219
500	200	307	363	384
550	250			
CL=120s, (Thru Phase=40s, Protected Phase=30s)				
250	100	127	132	137
300	150	182	187	186
350	200	228	227	242
400	250	269	276	277
450	300			

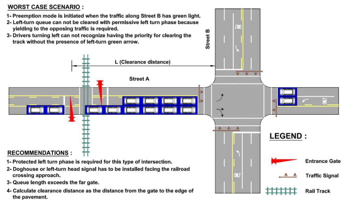
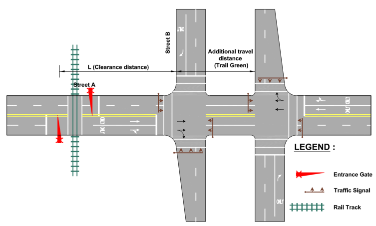
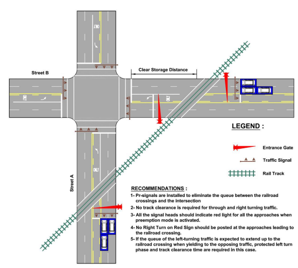
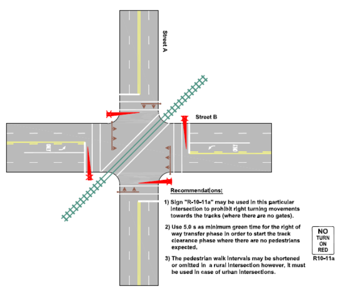
Note: *Average passenger car length including spacing~19.5 ft. (to convert into equivalent no. of vehicles, divide the queue lengths by 19.5)*
Shaded area shows the left- lane overflow state (queue estimation is not reliable at or beyond this point).

APPENDIX B

DETERMINING QUEUE CLEARANCE DISTANCE AND QUEUE CLEARANCE TIM

Table B-1. Recommendations for Determining Queue Clearance Distance

Approach Configuration	95th Percentile Queue Spill Back	Sketch	Gated Railroad Crossing without Pre-signal Recommendations
Single Lane	Queue does not extend beyond the far gate		Clearance distance should be equal to the queue length if 95th percentile queue length does not exceed the far gate.
Single Lane *(Through/right/left shared)*	Queue extends beyond the far gate		Clearance distance should be equal to the distance from the far gate to the edge of the pavement.
Two lanes, *(one shared through/left and one shared through/right)*	Queue Length does not exceed the far gate.		Clearance distance should be equal to the longer queue length (95th percentile) between both the lanes.
Two lanes, *(one shared through/left and one shared through/right)*	Queue Extends beyond the far gate		– Clearance distance should be equal to the distance from the far gate to the edge of the pavement. – Provide a dog-house or left-turn head signal facing the railroad approach.
Roadway with exclusive left-turn lane	Queue Length does not exceed the far gate.		– Clearance distance should be the max queue length of both lanes of the approach. – Provide a dog-house or left-turn head signal facing the railroad approach.

Approach Configuration	95th Percentile Queue Spill Back	Sketch	Gated Railroad Crossing without Pre-signal Recommendations
Roadway with exclusive left-turn lane	Queue Extends beyond the far gate		– Clearance distance should be equal to the distance from the far gate to the edge of the pavement. – Provide a dog-house or left-turn head signal facing the railroad approach.
Signalized Boulevard Intersection			– Clearance distance should be the minimum of (a) max 95th percentile queue length among all the lanes (b) the distance between the far side gate and the edge of the near side pavement.
Diagonal Railroad Crossings			– Pre-signals should be installed – All the signal heads should indicate red indication on the initiation of preemption mode. – No Right-turn on Red sign should be provided at the approaches leading to the railroad crossing. – If left-turn queues are expected to reach the tracks, protected left turn signal and track clearance time are needed in that case.
Railroad Crossing through the middle of a signalized intersection			– Traffic signal should show red indication for all approaches during the preemption mode. – Provide “No-Turn-on-Red” sign to prohibit right turn movements towards the tracks where there are no gates present.

RECOMMENDED VALUES FOR DETERMINING QUEUE CLEARANCE TIME

Track Clearance Green Time for a Clearance Distance 'L' (Passenger Cars Only)

NUMBER OF PASSENGER CARS IN QUEUE*	L = CLEARANCE DISTANCE (FT)	START UP DELAY TIME (S)	TRAVEL TIME TO ENTER INTERSECTION AFTER INITIAL START UP DELAY (S)	TOTAL TRACK CLEARANCE GREEN TIME (S)	TRACK CLEARANCE GREEN TIME ROUNDED VALUES (S)
1	25	2.2	3.0	5.2	10.0
2	50	3.4	4.3	7.7	10.0
3	75	4.6	5.3	9.9	10.0
4	100	5.7	6.3	12.0	12.0
5	125	6.9	7.1	14.0	14.0
6	150	8.1	7.9	16.0	16.0
7	175	9.3	8.5	17.8	18.0
8	200	10.5	9.3	19.8	20.0
9	225	11.7	10.0	21.7	22.0
10	250	12.8	10.6	23.4	24.0
11	275	14.0	11.4	25.4	26.0
12	300	15.2	11.8	27.0	27.0
13	325	16.4	12.4	28.8	29.0
14	350	17.6	13.0	30.6	31.0
15	375	18.8	13.6	32.4	33.0
16	400	20.0	14.1	34.1	35.0

*Each car is assumed as 25 feet, which includes nominal buffer spacing.

Track Clearance Green Time for a Clearance Distance 'L' (Passenger Cars + One Truck)

NUMBER AND TYPE OF VEHICLES IN QUEUE*	L = CLEARANCE DISTANCE (FT)	START UP DELAY TIME (S)	TRAVEL TIME TO ENTER INTERSECTION AFTER INITIAL START UP DELAY (S)	TOTAL TRACK CLEARANCE GREEN TIME (S)	TRACK CLEARANCE GREEN TIME ROUNDED VALUES (S)
1 truck	61	4.0	10.5	14.5	15.0
1 car + 1 truck	86	5.2	12.5	17.7	18.0
2 cars + 1 truck	111	6.4	14.3	20.7	21.0
3 cars + 1 truck	136	7.5	15.9	23.4	24.0
4 cars + 1 truck	161	8.7	17.3	26.0	26.0
5 cars + 1 truck	186	9.9	18.7	28.6	29.0
6 cars + 1 truck	211	11.1	20.0	31.1	32.0
7 cars + 1 truck	236	12.3	21.3	33.6	34.0
8 cars + 1 truck	261	13.5	22.5	36.0	36.0
9 cars + 1 truck	286	14.6	23.6	38.2	39.0
10 cars + 1 truck	311	15.8	24.7	40.5	41.0
11 cars + 1 truck	336	17.0	25.7	42.7	43.0
12 cars + 1 truck	361	18.2	26.7	44.9	45.0
13 cars + 1 truck	386	19.4	27.7	47.1	48.0
14 cars + 1 truck	411	20.6	28.7	49.3	50.0

*Each car is assumed as 25 feet, while each truck is assumed as 61 feet, representing a WB-50 tractor trailer. The vehicle lengths also include nominal buffer spacing. The truck is assumed to be positioned at the end of the queue.

Track Clearance Green Time for a Clearance Distance 'L' (Passenger Cars + Two Trucks)

NUMBER AND TYPE OF VEHICLES IN QUEUE*	L = CLEARANCE DISTANCE (FT)	START UP DELAY TIME (S)	TRAVEL TIME TO ENTER INTERSECTION AFTER INITIAL START UP DELAY (S)	TOTAL TRACK CLEARANCE GREEN TIME (S)	TRACK CLEARANCE GREEN TIME ROUNDED VALUES (S)
2 trucks	122	7.0	16.0	23.0	23.0
1 car + 2 trucks	147	8.2	17.6	25.8	26.0
2 cars + 2 trucks	172	9.4	19.0	28.4	29.0
3 cars + 2 trucks	197	10.6	20.3	30.9	31.0
4 cars + 2 trucks	222	11.8	21.6	33.4	34.0
5 cars + 2 trucks	247	13.0	22.8	35.8	36.0
6 cars + 2 trucks	272	14.2	24.0	38.2	39.0
7 cars + 2 trucks	297	15.4	25.1	40.5	41.0
8 cars + 2 trucks	322	16.6	26.2	42.8	43.0
9 cars + 2 trucks	347	17.8	27.2	45.0	45.0
10 cars + 2 trucks	372	19.0	28.2	47.2	48.0
11 cars + 2 trucks	397	20.2	29.2	49.4	50.0

*Each car is assumed as 25 feet, while each truck is assumed as 61 feet, representing a WB-50 tractor trailer. The vehicle lengths also include nominal buffer spacing. The truck is assumed to be positioned at the end of the queue.

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ABSTRACT

ANALYSIS OF QUEUE CHARACTERISTICS AT SIGNALIZED INTERSECTIONS NEAR HIGHWAY-RAILROAD GRADE CROSSING

by

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December 2015

Advisor: Dr. Timothy J. Gates

Major: Civil Engineering

Degree: Doctor of Philosophy

Analysis of traffic queues at signalized intersections which are in close proximity to highway- railroad grade crossings is of primary importance for determining if the normal signal operation needs to be preempted for railroad operations by providing a special signal mode for safe clearance of the queued vehicles from the tracks before the train arrival, and prohibiting any conflicting traffic movements towards the crossing. Such queuing analysis becomes even more critical where direct observations of traffic queues are not possible or where the assessment is needed for a future location. Inadequate estimation of queues from signalized intersections to the nearby railroad grade crossing can lead to severe safety issues. Underestimation of queue lengths may lead to an unsafe design while significantly overestimated queues may cause unnecessary traffic delays consequently leading to violations of the active traffic control devices at the crossing. In order to determine an adequate approach for reasonable estimation of queue lengths at signalized intersections near highway-railroad grade crossings, this dissertation first evaluated and compared different currently used microscopic simulation-based methods (i.e. Sim-Traffic

and VISSIM) for their adequacy in estimating the queue lengths. After that several comparisons are made between the queue estimation from the simulation-based and other deterministic analytical methods including Highway Capacity Software, Synchro, and Railroad Assessment Tool.

The comparisons drawn between each method helped identifying the differences and specific limitations of each method in including the impact of various important factors on the resulting queue estimation. The recommendations are provided on the basis of model capability to adequately count the impact of various significant traffic factors on queue estimation and considering minimizing the risk of underestimated queues.

Based on the analysis findings, a microscopic simulation based procedure is developed using Sim-Traffic for estimating the 95th percentile queue lengths on various existing signalized intersection configurations near highway-rail grade crossings to help evaluate the need for signal preemption. In addition, recommendations are developed, if preemption is necessary, for determining queue clearance distance and minimum track clearance time. The recommended procedure is developed considering minimizing the risk of underestimated queues or unsafe design at such locations, and simplify the design and decision-making process.

AUTOBIOGRAPHICAL STATEMENT

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Wayne State University, Detroit, MI (Fall 2010-Present)
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