CLEARANCE TIME REQUIREMENTS AT RAILROAD-PREEMPTED TRAFFIC SIGNALS

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Clearance Time Requirements at Railroad-Preempted Traffic Signals

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ERRATA 1

1) Add the following sentence at the end of step (1) on page 55, before the last sentence in step (1) on page 58, after the first sentence in step (1) on page 59, and after the first sentence in step (1) on page 60: "For intersections 200 ft to 500 ft from a crossing, FDOT Procedure 750-030-002-d requires either preemption or documentation of an engineering analysis to be placed on file showing that preemption would be of no value."

2) Change the distance between the stop-line and pavement edge from 12 ft to 15 ft in the middle of the "Specifics" paragraph for Example Problem 1 on page 56, and in step (5) on page 57.

3) Change the value of 17.2 to 17.3 in the last line of step (4) on page 58.

4) Change the value of 6 ft to 6.9 ft in both places where "6" is shown in step (5) on page 58.

5) Change the value of 329 ft to 328 ft in both steps (5) and (6) on page 58.

6) Change the value of 394 to 393 in both steps (6) and (7) on page 58.

ABSTRACT

Traffic signals near railroad grade crossings that have active warning systems are normally interconnected and receive a signal from the railroad track circuitry when trains are approaching. This train approach signal is utilized by the traffic signal to interrupt and preempt its normal phasing and enter into higher priority special phasings to clear the track of any vehicles that might be in the pathway of a train, and then proceed with signal phases which do not allow traffic movements to approach the track until after the train has passed. The amount of preemption time needed to clear a vehicle from the pathway of a train is necessary information for preemption signal settings but there are no definitive guidelines on how to determine this amount of time. It is usually left to the judgment of the signal engineer, and frequently unfounded assumptions are required. This study investigated the time required to clear the n^{th} vehicle in a queue off a track at railroad-preempted traffic signals. The two key time components are startup delay and repositioning time. It was found that queues where all preceding vehicles are short passenger cars cause the longest startup delays, and heavy trucks compel the longest repositioning times. The model developed is convenient because once it is decided that preemption is needed, it does not need traffic volumes or the distribution of vehicle types or estimates of average vehicle spacings including trucks, just the minimum track clearance distance, clear storage distance, and the types of vehicles which are permitted to use the roadway.

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Clearance Time Requirements at Railroad-Preempted Traffic Signals

INTRODUCTION

In 23 CFR §655.603, the national *Manual on Uniform Traffic Control Devices* (MUTCD), as approved by the Federal Highway Administrator, is designated as the national standard for all traffic control devices in the U.S. The current version was released in December 2001.

F.S. 316.0745 requires the Florida Department of Transportation (FDOT) to adopt a system of uniform traffic control devices for use on traffic facilities that are open to the public in Florida. By F.A.C. Rule 14-15.010, FDOT has adopted the national MUTCD as the standards for traffic control devices in Florida.

The MUTCD [2001], in §8A.01(11), defines traffic signal preemption as "the transfer of normal operation of traffic signals to a special control mode". The special control mode consists of distinct traffic signal phases to (1) allow any vehicles that might be trapped in the pathway of an approaching train to move forward to safe positions, and (2) suspend any traffic movements through an intersection toward the railroad track until all trains have cleared the crossing. Railroad preemption only pertains to traffic signals at roadway intersections which are located near at-grade railroad crossings having active warning devices.

In §8D.07, the MUTCD [2001] specifies the distances between traffic signals and railroad tracks for preemption to be applied:

> "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." [MUTCD, 2001, p. 8D-10]

No limit is specified in the MUTCD as to the maximum distance for which preemption should be considered.

FDOT Procedure 750-030-002-d establishes, as standards for Florida, that traffic signals within 200 ft of railroad-highway grade crossings be provided with preemption circuitry, and that preemption should also be provided at signalized intersections between 200 ft and 500 ft of railroad-highway grade crossings having active warning devices unless documentation from a traffic analysis is on file for a specific location which shows that preemption would be of no value. This extends preemption requirements beyond the MUTCD [2001], but is not inconsistent with the MUTCD.

MUTCD §8D.06 [2001] addresses train detection and indicates that flashing-light signals shall operate for at least 20 sec before the arrival of any train. If the same train detection circuitry is used to trigger preemption at nearby traffic signals, an important issue is whether 20 sec is adequate for track clearance. While 20 sec might be adequate for railroad tracks that are within 200 ft of signalized roadway intersections, it is usually grossly inadequate for separations approaching 500 ft. MUTCD §8D.06 [2001] specifies that additional warning time may be provided when determined by an engineering study, so the 20 sec can be increased, but a method is needed to assess how much track clearance time is required. The MUTCD does not offer any methodology for this purpose, only a declaration that an engineering study is needed.

Increasing the time of operation of flashing-light signals at railroad crossings may not be an acceptable method of providing longer times that are needed for preemption of intersection traffic signals at longer distances from railroad tracks. Motorist delays may become excessive tempting motorists to consider risky behaviors. Instead, different types and arrangements of train detection devices may be required. Consequently, information about the amount of track clearance time that is required for adequate safety of motorists is needed for selecting the type of train detection circuitry and designing the arrangement of the circuits. The track clearance time methodology is also needed to review and assess the signal timings already in use at existing intersections near railroad crossings. Since no accepted methodology currently exists, transportation engineers do not have reliable tools to assure consistent results. With human safety being at stake, this is extremely important.

This research investigates the time required to clear a vehicle in a queue from the pathway of a train at railroad-preempted traffic signals. It investigates the factors which affect track clearance time, and proposes a model to fulfill this information need.

BACKGROUND

There are many locations where roadways and railroad tracks are roughly parallel and in close proximity to each other. Sometimes this is a consequence of the physical terrain, where a corridor was predefined by the course of least resistance around rivers, lakes, swamps, hills and valleys. Sometimes it is a consequence of the location of the cheapest or most easily obtainable land, or the location of a corridor established by one mode in which service was needed by the other mode.

Prior to about 1830 and the advent of the steam locomotive, land travel was served mostly by roads and by only a very few miles of rails supporting horse-drawn railway vehicles in urban areas. Most of the parallel alignments of roadway and track were established during the century between about 1830 and 1930 when land travel was by horse or railroad, and no particular problems were presented by this configuration, beyond trains occasionally spooking horses. After motor vehicles replaced horses on roadways, new problems emerged from close alignments.

Need for Preemption

An important problem occurs where perpendicular roadways first cross a railroad track and then cross a nearby roadway that lies close to the track. Traffic on the perpendicular road can cross the track and then be stopped by a traffic signal or other impediments or conditions at the intersection of the two roadways. Motorists are always eager to cross a track to avoid the possibility of incurring additional delay by having to wait for a long, slow freight train to clear the crossing, if one should arrive. In their eagerness, sometimes motorists misjudge whether sufficient space exists on the other side of a track for safe storage of their vehicles. Too often they apparently do not even think about it and continue to follow the vehicle ahead until it stops. After starting to traverse a track, a motorist usually cannot stop and back up due to trailing vehicles blocking a retreat path.

Where the intersection of two roadways is signalized, and a nearby railroad crossing is signalized, as is common at many urban locations, the two signal mechanisms can be interconnected. When the railroad circuitry detects the approach of an oncoming train, the interconnection circuitry can pass an indication to the traffic signal circuitry that a train will be arriving. The traffic signal can then interrupt its current cycle, terminate the current phase and stop whatever flows are moving, and change the signal to disperse any queue of vehicles that extends backward towards the track. The intention is to allow any vehicle that might be encroaching into the pathway of the train an opportunity to move forward before the approaching train arrives. This mechanism is known as preemption of traffic signals near railroad crossings. The normal signal sequences and timings are preempted by special sequences and timings. Whatever phase the traffic signal is in when an approaching train warning is received is preempted as quickly as possible for the higher priority task of dispersing any queue of vehicles that could trap a vehicle on the track. A second stage of preemption then prohibits any movements of vehicles from the intersection towards the tracks and persists until the preemption signal from the railroad control circuit ceases, which normally terminates when a train has fully passed the roadway.

Preemption Impacts

Most of the time, hopefully, the first stage of preemption will be wasted because no roadway vehicle will be stopped foolishly in the pathway of a train. Motorists served by all phases of the roadway traffic signal, except the phase for the traffic stream that crosses the track, consequently usually suffer increased delay every time a train arrives. Motorists who crossed the track may enjoy reduced delay from the traffic signal due to preemption by being allowed to proceed immediately. This can be self-defeating if it induces motorists to try to avoid delay by deliberately and recklessly crossing a track even though they are aware that sufficient storage space is unavailable because they expect the traffic signal to change to green and allow them to proceed before a train arrives.

Although additional delay will be imposed on some motorists when preemption occurs, it only occurs when a train is approaching, which is usually only a few of the traffic signal cycles during a day. If a train approaches as the traffic stream crossing a track is in its green phase, the first stage of preemption may have no effect on other traffic movements because the track will be cleared as it would be if no train was approaching. The amount of additional delay imposed by the first stage of preemption increases with increasing distance between the track and traffic signal, and other factors.

The second stage of preemption does not really create much additional delay for any traffic movements. Traffic flows through a signalized intersection are ordinarily disrupted anyway by a train blocking the road. The second stage of preemption just prohibits traffic movements through the roadway intersection along pathways crossing the track while a train blocks the track. Since the train blocks the track anyway, the second stage of preemption just detains motorists a little farther away from the track and keeps the roadway intersection clear of queue backups that could interfere with the movements of traffic through the intersection that are unaffected by the train. Consequently, the second stage of preemption does not involve much additional delay beyond that imposed by the train blocking the track, and motorists traveling through the intersection not crossing the track may enjoy reduced delay by special phases allowing them to proceed instead of having to wait for their normal turn.

Certainly it can be argued that stopping a vehicle in the pathway of a train, whether a train is present or not, is reckless and irresponsible and should never occur. This is true. However, it does occur, and death or severe injury from a collision with a train is too severe of punishment for a momentary lapse of sensibility, if it can be averted at an affordable cost. It has to be kept in mind that the victims can be innocent passengers, such as automobile passengers or school bus passengers, who may not be directly involved in the lunacy of getting into such a situation.

Besides motorists who find themselves stopped inadvertently in the pathway of a train, there are risk-takers who do this knowingly. Sometimes they recklessly drive around gates or disregard warning lights that they observe to be flashing. Usually they do this because they perceive the train warning system to be malfunctioning or maladjusted. Often, this perception is faulty. However, it usually arises from conditioning associated with repeated experiences where a warning system was observed delivering an endless or excessively long false warning of the approach of a train, possibly due to train switching operations or a 'fail-safe' equipment failure.

Sometimes there is no train visible at a crossing in either direction as far as a motorist can see, due to visibility obstructions. From similar experiences at other crossings, sometimes this induces motorists into trying to cross a track without carefully considering how close a train could be before becoming visible or how fast a train might be coming. Sometimes a train can be seen in the distance, but motorists perceive it to be stopped or moving slowly, and underestimate how long until it will arrive. Liebowitz [1985] analyzed how false illusions regarding train size and distance away misleads motorists about train speed and travel time. Even motorists who are not aggressive gamblers sometimes can be enticed into risky behavior by conditioning from past experiences, misleading perceptions and superficial analysis during the few moments available for making gathering information and making a decision.

Relative Delays

Motorists in the U.S. often show much less respect for train warning signals than for intersection traffic signals. While a few motorists deliberately enter intersections during the first couple of seconds after a traffic signal changes to red, normally most motorists stop and remain stopped until they get a green signal, even if no other traffic is present. Of course, delays at traffic signals are usually both moderate in duration and predictable. Motorists expect to be detained by a red traffic signal a maximum of about ½ to 1½ minutes, depending primarily upon the number of signal phases and intensity of traffic congestion during the period of travel. Intersection capacities could be increased by lengthening the cycle lengths beyond the typical limit of about 2½ minutes to minimize wasted time during phase changes, but this is rarely done so motorists will not perceive that delays are excessive and contemplate disobeying traffic signals.

At a railroad crossing, a 400-foot long passenger train traveling at 80 mph will block a crossing for only 3½ seconds, but a 7,000-foot long freight train traveling at 25 mph across the same crossing will block it for 191 seconds. With a minimum of 20 seconds of advance warning actuation time, a motorist could be delayed less than ½ minute for the passenger train. With a fixed actuation point set for a minimum of 20 seconds of advance warning time for 80 mph passenger trains, the advance warning time would be 64 seconds for a 25 mph freight train. The full delay of 64 seconds during advance warning plus 191 seconds while the freight train blocks the crossing adds to 4¼ minutes of delay for motorists. Crossing delays of 6 to 7 minutes are referred to by Ruback [2001]. While a passenger train might block a crossing for less than ½ minute, and a freight train for 4¼ to 7 minutes, a traveler usually has no information about which delay it will be. However, most trains are freight trains, so motorists observe this tendency and expect a delay that exceeds by as much as 14 times what they normally incur and consider to be tolerable at traffic signals. Railroads could easily counteract this situation by shortening the lengths of freight trains, but this might lessen the profitability of their operations.

Obviously, attention needs to be directed toward the cause of the problem. Better maintenance or design of warning systems to reduce incidents of false or excessive warnings at railroad signals could help. Better driver training or better enforcement of laws prohibiting stopping or standing on railroad tracks might help. More or different traffic signs or pavement markings at railroad tracks might help. A separate report from this project presents an evaluation of an X-box pavement marking that is intended to reinforce the visual cues to motorists as to whether sufficient space to store another automobile exists at the end of a queue on the far side of a railroad track [Stephens and Long, 2002]. The X-box has been used in Europe with some success, but is not allowed in the U.S. by the *Manual on Uniform Traffic Control Devices* [MUTCD, 2001]. Perhaps barring drivers from using cell phones, personal organizers and/or fiddling with radios or tape or CD players in vehicles might help reduce motorist distractions so they will be more attentive to driving responsibilities [Peters and Peters, 2001]. However, nothing will eliminate all distractions or assure that all tired, weary or preoccupied drivers are as alert and vigilant as they always need to be. Motorists cannot be successfully prevented from talking with passengers or from thinking about matters besides driving, so the alternatives above are not complete solutions to the problem. It is not possible to preclude all driving distractions. Motor vehicles are operated by humans, and humans are prone to occasional error and/or bad judgment.

At the other extreme, no feasible system for mitigating this type of situation can usually be completely safe or completely fail-safe. While it is always desired to design a traffic control system to be safe for the worst case, almost always there is a case that is worse, although extremely improbable and often unforseen. To guarantee complete safety from roadway vehicles being hit by trains, it is simply necessary to either close all roadways or close all railroads or separate the crossing planes at all crossings. Since our society has not decreed that the costs of such safety measures are acceptable, and has not elected to do this, we are accepting some risk of loss. Of course, there are risks to almost everything, although many may be quite small. Persons demanding to be 100% safe, must choose not to travel.

We are always trying to diminish the risks within the bounds of affordable costs. While everyone wants risks to be very, very small, nobody wants to set a specific limit on what risk is acceptable when human life is at stake, as is the situation at grade crossings. This is complicated further by difficulties in quantifying various aspects of safety risks at crossings, such as the probability of each motorist accelerating a vehicle at a specific rate that will clear a queue of trailing vehicles.

Preemption Applicability

There are two distinct classes of warning devices at grade crossings. Some crossings have only signs and possibly pavement markings. At these crossings, a motorist is left to use his own senses to detect any trains approaching within hazardous proximity and make an appropriate decision about whether it is safe to cross the track. These are known as passive warnings. Other crossings have flashing lights, sometimes with gates and/or bells. The flashing lights tell a motorist when a train, which frequently cannot be seen due to visibility obstructions, is approaching within threatening proximity. These are known as active warnings.

To minimize risks, some high-risk crossings have been grade-separated where it was affordable, some at-grade crossings have been closed where the added delay and inconvenience costs to motorists were affordable, and preemption circuitry has been installed at some crossings where feasible. Some roadway intersections are equipped with traffic signals; others are not. Some railroad crossings are equipped with active warning devices that warn of approaching trains; others are not. Preemption circuitry usually can be installed only where a roadway intersection has a traffic signal and a nearby railroad crossing has active warning devices.

Since the preemption of traffic signals which are located in close proximity to railroad crossings having flashing lights does not involve inordinate monetary costs, it is often implemented. However, there are other costs, such as the delay costs associated with the lost time of other motorists at the intersection, which must be considered in addition to risks and monetary costs, when setting traffic signal preemption timings. Delay costs are imposed on other traffic at preempted signals every time a train approaches regardless of whether any roadway vehicles are actually trapped in the pathway of a train. Preemption settings involve a trade-off between the risk of deaths and injuries to motorists stopped stupidly and trapped in the pathway of an oncoming train and the delays imposed on all other motorists who are always in a hurry and intolerant of incurring much delay for the benefit of stupid motorists who (fortunately) are rarely present. Moreover, preemption can increase the risks to pedestrians and roadway traffic who are not expecting their green phases to be abruptly truncated with no apparent cause at the time, and it can increase risks to any impatient motorist-gamblers who deliberately put themselves in harm's way to take advantage of preemption allowing them to escape a little delay by relying upon preemption to save them.

Early MUTCD Preemption Concepts

Traffic signal preemption at roadway intersections in the vicinity of railroad crossings is not new. Section 3B-25 of the 1961 MUTCD specifies that it is essential for the controller of a street traffic signal to be preempted by the train-approach signal from a railroad when the traffic signal is within 200 feet of a railroad crossing to avoid conflicting aspects of the traffic signal and train-approach signal.

The 1961 MUTCD noted that two sequential stages are involved in traffic signal preemption. The first stage is to clear vehicles from the tracks. At the completion of the first stage, the second stage is to prohibit movements of vehicles over the tracks. The second stage is to persist until the preemption signal from the railroad control circuit ceases. When no trains are present, the normal sequence of the roadway traffic signal is to be designed so that vehicles are not required to stop on the tracks even though in some cases this may increase the waiting time. "There is, however, one fundamental requirement which must be observed in all cases. The preemption sequence initiated when the train first enters the approach circuit shall at once bring into effect a signal display which will permit all vehicles to clear the tracks before the train reaches the intersection" [MUTCD, 1961, p. 174].

Due to the varying and unique configurations of roadway intersections and nearby railroad crossings, the MUTCD has never specified how much clearance time must be provided. Determining this value on a case-by-case basis has been left to the expertise of qualified transportation engineers. Once the necessary clearance time has been determined, cooperation by railroad officials is needed for establishing the required track circuitry to provide the proper preemptive signal timing.

Railroad Interface

There are several railroad factors which must also be considered in preemption timing. Railroad track circuits for traffic signal preemption must operate in rapport with the activation of railroad flashing lights at the crossing, and with railroad block-signals which indicate whether the track-block ahead is clear for trains to proceed. Furthermore, the distances to the actuation points must be compatible with the track circuits for signals at adjacent roadway crossings along the track.

Train Movement Block Signals

Railroad block signals involve a different concept than is needed for roadways. Freight trains that extend over one mile in length are typical for most railroads. When traveling at 60 mph with loaded freight cars, because of its large mass, a freight train may take a quarter-mile or more for an emergency stop [Hay, 1982] to dissipate all of its kinetic energy. Dissipation of energy through friction is limited by the very low coefficient of friction between the steel wheels and steel rails, especially when wet from rain, as well as issues associated with derailment. With normal horizontal and vertical curves in a track, rarely can a train engineer continuously see one-quarter mile ahead or further to know if the track is clear of other trains or if the train needs to make an emergency stop. To solve this problem, railroads divide the track into sections called blocks, and install track circuitry and signals along the track which indicate whether the track block ahead is clear or occupied by another train. Track circuitry for train-approach signals for traffic signal preemption must operate in concert with block signal circuitry.

Track Circuits

Track circuits capitalize on the fact that steel rails conduct electricity. Early tracks were formed of segmented rails where it was merely necessary to connect an electrical conductor from the end of one rail to the beginning of the next to conduct electricity through a series of rail segments. Modern tracks are formed of continuously welded rails so it is merely necessary to cut the rail and install an insulated joint where conduction is intended to terminate. Another feature exploited in track circuitry is that steel train wheels and steel axles conduct electricity from one rail of a track to the other.

Both rails of a continuously welded track could be cut near a roadway, and also cut at 1,320 feet from the roadway, to isolate the two rail sections from the rest of the track. A power source could be connected between one rail and a flashing light at the roadway, and the other terminal of the flashing light could be connected to the other rail. This would form part of an electrical circuit, but the light would not flash because the circuit would be open at the cuts 1,320 feet away. However, when the steel wheels and axles of an approaching train first pass the rail cuts 1,320 feet away, a closed electrical circuit would be formed through the axle between the wheels from rail to rail and the light would begin flashing and continue to flash until the last steel wheels in the caboose passed the rail cuts near the roadway. This is the basic concept of track circuitry.

If the train was traveling at 45 mph (66 ft/sec) when it reached the actuation point at the cuts in the rails 1,320 feet from the roadway, the light would have begun flashing 20 seconds (1,320 feet / 66 fps = 20 sec) before the train reached the roadway. If the train was traveling at 60 mph (88 fps), then the light would have started flashing 15 seconds before the train reached the roadway. If 20 seconds of warning time was desired for trains traveling at 60 mph, then the actuation point cuts in the rails could be made at 1,760 feet from the roadway rather than at 1,320 feet.

Warning Device Failures

Two types of failures must be considered with active warning devices: failure to execute warnings when trains are present, and delivery of false warnings when trains are not present. It is critical to execute warnings when trains are present because the trains might not be visible or otherwise detectable by motorists, causing lives to be risked. False warnings are a nuisance and breed disrespect for traffic control devices, but are not as critical because lives are not at risk. Therefore, if active warning devices are to suffer failures, it is much safer to deliver a false warning than to fail to execute a needed warning.

Normally-Closed Circuit Design

If the power source in the example above was disrupted by a storm which caused a power outage from the power supply lines, the light would not flash when a train was approaching, which could be catastrophic. Consequently, track circuits have a backup battery and operate with a circuit that is normally closed having current normally flowing through the rails. A power source in series with a resistor is connected between the rails near the break in the rails at the actuation point and a relay in series with a resistor is connected between the rails near the roadway. While no train is present, current flows through the track circuit holding the relay open. When a train passes the actuation point, the steel wheels and axle serve as a shunt which causes a shorter circuit with less resistance from the power supply to the train wheels than to the relay, resulting in a disruption of current flow through the relay ahead of the train wheels. Without current to hold it open, the relay closes and activates a circuit with a power supply and backup battery to energize the flashing lights. If a storm disrupts power flowing through the track circuit when no train is present, the relay will open and activate the flashing lights, so the failure will send a false warning when trains are not present. However, a power outage will not result in the lights not flashing when a train is present. Sometimes a design where the flashing lights are activated in the event of some failure is referred to as being 'fail-safe', but this can be a misleading misnomer as discussed later.

Direct current (DC) was used first in track circuits and is still widely used. Alternating current (AC) is used with electric trains, and is also used by conversion to DC for the track relay with a half-wave rectifier connected across the rails at the actuation point, and all other equipment located in a housing near the crossing.

Three Track Circuits

Since trains can approach a crossing from either direction, the track circuit needs to extend from the actuation point on one side of the roadway to the actuation point on the other side. However, the flashers do not need to continue operating after the train caboose crosses the roadway and travels to the actuation point on the other side. The warning needs to start flashing when the front of a train reaches the actuation point and only needs to continue flashing until the rear of a train has crossed the roadway. Thus, with trains approaching from either direction on a track, two approach zones are needed with each approach extending from its actuation point to slightly past the roadway. This creates an overlap of the zones at the roadway. Since the current flow for two separate circuits cannot normally use the same rail conductors, a separate island circuit is created for the track segment between insulated joints on each side of the roadway. This is called a ‘three track circuit system’ with the island circuit operating in conjunction with both approach circuits. Normally the island extends a short distance on each side beyond the roadway pavement. The standard island is 120 feet long with its center located at the center of the roadway pavement.

Audio-Frequency Overlay Circuitry

Insulated joints for roadway warning circuits can interfere with the continuity of the train-movement block-signal system. One solution to this problem is to use audio-frequency overlay circuitry which does not require insulated joints. A high-frequency alternating-current transmitter is connected to the rails at one end of the zone and a receiver tuned to the same frequency which measures circuit voltage, electric current level and signal phase is connected to the rails at the other end. The transmitter shoots a signal down the rail. The receiver listens for the signal and converts the alternating current to direct current to operate the relay controlling the warning device circuit.

Motion-Sensor Systems

An extension of this concept is known as a motion-sensor system, which continues monitoring the signal impedance to detect if it is changing. This information is used to determine in which direction a train is moving, or if it is stopped. If a train stops in the detection zone, the warning devices can be deactivated. A motion-sensor system can be valuable where a roadway crosses a track near a train station or near switching operations. If a train stops at a station or for any other reason within the detection zone, the crossing warning devices can be deactivated allowing traffic flow to resume until the train begins to move again. Section 8D.06 of the MUTCD [2001] prescribes using such devices to prevent excess signal activation for station stops and switching operations.

A fault with the motion-sensor system is that a stray wire or a piece of metal banding from lumber on flatcars or any other conductive material that might fall across the rails can create a shunt and be misinterpreted by the warning system as a train stopped on the tracks. It can cause a brief activation of the warning system when it first appears and then deactivation when it does not move. Meanwhile, a train approaching the crossing could proceed undetected until it crossed the stray shunt. If the stray shunt was located close to the crossing, traffic might be provided almost no warning time instead of the minimum warning time prescribed by law. This violates the notion of crossing devices being ‘fail-safe’ by never failing to execute a needed warning when trains are present.

Train Speed Variability

Passenger trains can be operated at 80 mph on FRA Class 4 track [49 CFR 213.9(a)]. If the desired advance warning time is 20 seconds, the actuation point would need to be set at 2,347 feet from a crossing for 80 mph trains. If a 25 mph freight train actuated the signal, it would take 64 seconds before it traveled 2,347 feet and reached the crossing. Since the actuation point is nearly one-half mile away, motorists probably would not see the train when the crossing lights began flashing. They are usually looking for an enormous locomotive in reasonably close view, and are not expecting to look for a dot in the weeds a half-mile away. If they did see it, they would likely conclude that the signal system was malfunctioning and there was plenty of time for them to breach the warning and cross the track safely. If sight distance was limited such that they could not see the train, after waiting 20 to 40 seconds they would likely conclude that the signal was malfunctioning and begin crossing the track. Research has confirmed that warning times in excess of 30 to 40 seconds induce many motorists to engage in risky crossing behavior [Richards and Heathington, 1990a]. This also breeds disrespect for traffic control devices, which violates §1A.04 of the MUTCD [2001]. 23 CFR 655.603 declares the MUTCD to be national standards. Due to adoption of the MUTCD by state statutes, excessive warning times inducing disrespect for traffic control devices violates state laws. The cause is having the actuation point set for the fastest train, but having slower trains also using the track. If the actuation point had been set for slower trains, then inadequate warning time would result for faster trains. Ideally, all trains across a crossing need to be traveling at about the same speed, but this is rare because railroads select their train speeds for their convenience, not the roadway users' convenience. The solution to this dilemma is to use constant warning-time devices at locations where trains travel at widely different speeds across a crossing.

Constant Warning-Time Systems

Constant warning-time systems are an extension of motion-sensor systems. After the initial detection of a train at the actuation point, the system delays for a brief time, perhaps 4 seconds, and detects again to assess the distance to the train and determine how far the train moved between detections. Dividing the distance the train moved by the time elapsed between detections produces an estimate of train speed. (Actually, the system may cycle repeatedly during the delay period, calculate the speed during each cycle, and then average the values as the final estimate of train speed.). Dividing the distance between the train and the crossing by the estimated train speed produces an estimate of how long until the train reaches the crossing. Subtracting the desired warning time from the time until the train reaches the crossing produces an estimate of the delay time needed before activating the flashing lights to yield the desired (constant) warning time at the crossing. Regardless of train approach speed, the warning time should result being approximately the same. Section 8D.06 of the MUTCD [2001] prescribes using such devices where train speeds vary.

Constant warning-time systems perform well where trains operate at constant speeds, even though different trains may travel at different constant speeds. However, railroads operate trains at their convenience, not so that their constant warning-time systems for motorists function properly. If trains increase speed after entering the detection circuit, they will arrive at the crossing earlier than predicted and the warning times will be too short and not constant in duration among different trains. Constant warning-time systems do not necessarily produce constant warning times.

Other Detection Technologies

Other technologies for train detection, especially off-track sensors such as microwave sensors, Doppler-radar sensors, magnetic anomaly sensors, induction loop sensors, infrared sensors, wheel-counter sensors, acoustical sensors, ultrasonic sensors, vibration/seismic sensors, global positioning system (GPS) sensors, video-imaging, etc., are under development or being evaluated, but on-track circuitry is normally what is employed for traffic signal preemption [Venglar et al., 2000].

Interconnection

The track circuits used for train detection are designed to control the activation of flashing lights at the crossing to warn traffic of approaching trains. The same circuit can be interconnected to nearby roadway traffic signal controllers for preemption.

The physical interface between the railroad signal system and the traffic signal system remains a single simple on/off circuit. Section 8D.07 of the MUTCD [2001] specifies that the electrical interconnection circuit shall be normally energized when no train is present and the approach of a train shall de-energize the circuit to actuate the traffic signal controller preempt. Consequently, a power failure in the railroad circuitry or inadvertent severing of the connection will activate preemption.

A more refined interface standard for the interconnect between railroad signals and traffic signals is being defined as a part of the National Intelligent Transportation Systems (ITS) Architecture under User Service #30 [http://www.fra.dot.gov]. This is known as the Highway-Rail Intersection (HRI) architecture, and §8A.02 of the MUTCD [2001] declares that HRI components may be specified for new equipment installations at grade crossings.

Preemption Guidance

There have been several widely-published sources of information available related to preemption. Part 3.3.10 of the Signal Manual of the Association of American Railroads [AAR, 1996] provides calculation instructions and recommendations on approach warning times at active crossings as well as information on the track circuitry and motion detection processors of the railroad industry. This has now been transferred to AREMA [2000]. This information does not include preemption times for nearby traffic signals.

The MUTCD [2001] has prescribed minimum warning times for flashing-light actuation and time delays for gate operations. A companion manual in conjunction with prior editions of the MUTCD, the Traffic Control Devices Handbook [TCDH, 1983], provides several typical preemption sequence diagrams for different railroad and roadway configurations. It also specifies, "When preempted by train movements, the traffic control signal (after provision of the proper phase change intervals) will immediately provide a short green interval to the approach crossing the track" [TCDH, 1983, p. 8-36] to clear any vehicles that may be on or so close to the track to be in danger, but it does not address the length of the "short green interval"

A compilation of state-of-the-art information on railroad-highway grade crossings is presented in FHWA's Railroad-Highway Grade Crossing Handbook [FHWA Handbook, 1986]. The typical preemption sequence diagrams from the TCDH [1983] are included, but the timing for queue clearance is not addressed.

The standard in the industry has been "A Recommended Practice" published by the Institute of Transportation Engineers in 1979 [ITE, 1979] which provides recommendations for when to preempt traffic signals near active railroad crossings and which preemption sequences to use.

Interest by public officials in track clearance at railroad-preempted traffic signals escalated after October 25, 1995 when 7 children were killed and 24 more were injured in a train-bus collision in Fox River Grove, Illinois. The school bus was stopped for a traffic signal after crossing the railroad track. About 3 feet of the rear of the bus was hit by a high-speed commuter train. The bus had crossed the tracks before the train approached and before the flashing lights and gates were actuated. The driver misjudged the amount of vehicle storage space available on the other side of the tracks in comparison to the length of the school bus. The storage space was inadequate, so the back of the bus extended slightly into the pathway of the train. The traffic signal was preempted, but changed to green for the bus only 2 to 6 seconds prior to impact. The National Transportation Safety Board determined that the preemption timing was insufficient, among other deficiencies which contributed to the crash [NTSB,1996].

After the Fox River Grove crash, ITE quickly published an updated edition of its recommended practice [ITE, 1997]. Shortly afterward, a synthesis of information was published by the Transportation Research Board [NCHRP Synthesis 271, 1999]. In addition to the compilation of technical material, the Synthesis contains a valuable annotated bibliography of pertinent literature related to traffic signals near railroad crossings.

There are several documents discussing signal preemption in Florida. Section 7.4.6 of the “Plans Preparation Manual” [FDOT, 2000], §A.4.8 of the “Rail Manual” [FDOT, 1998], and an FDOT Departmental Procedure on “Signalization Preemption Design Standards” [FDOT, 1996], all address circumstances where preemption of traffic signals is to be implemented or considered. Section III.5 of the “Traffic Engineering Manual” [FDOT, 1997] contains detailed information on preemption.

Worst-Case Design

Where safety is concerned, there is usually a desire to design for the worst case so that all cases will be encompassed. Yet, the worst case is usually difficult to define. There always seems to be a case that is worse. Moreover, a trade-off exists between the extent of a safety net provided for an errant driver in the worst conditions and the penalties imposed on all other drivers during all other times for the benefit of the errant driver, who is not even expected to be present most of the time.

AASHTO Design Policies

The AASHTO Green Book [AASHTO, 1994] deals with the design case by identifying classes of vehicles and choosing specifications for a design vehicle in each class. This avoids the pitfall of trying to identify the worst-case vehicle. Section 8A.01 of the MUTCD [2001] embraces this concept by defining some standards for traffic control design at railroad-highway grade crossings which include design vehicles. The design vehicle is defined in §8A.01 as the longest vehicle permitted by statute of the road authority to use the roadway. This definition is fine if the worst case arises from the longest vehicles, but it is little help if the worst case may involve the shortest or slowest vehicles.

The length of AASHO's original passenger-car design-vehicle was 20 feet, based on the average length of passenger vehicles at that time being about 17 feet [AASHO, 1940, p.71]. In the 1954 Blue Book, AASHO adopted a length of 19 feet as the length for the passenger-car design-vehicle based on a table of dimensions of 32 popular 1953 passenger vehicles which averaged 17.2 feet in length and ranged from 15.1 to 20.2 feet. AASHO stated that the design vehicle should be one with dimensions "larger than almost all vehicles in its class" [AASHO, 1954, p. 71]. The 19-foot length selected for the design vehicle exceeded the lengths of 90% of the 1953 vehicles listed. By reducing the 1940 design length by one foot, such that about 10% of current vehicles exceeded the design length, AASHO was focusing not on the worst case, but a case involving only few exceedences. There is no indication that AASHO intended 10% as an acceptable level for exceedences. Even though the average length of the passenger vehicle fleet has since become shorter [Long, 2002], AASHTO has continued to retain the design length of 19 feet for passenger vehicles in the recent Green Book [AASHTO, 1994], probably because there are still some passenger vehicles of this length or longer that are produced and used in the current fleet. Long [2002] reported measurements of passenger cars up to 19.7 feet in length, pickup trucks up to 22.5 feet and vans up to 25.0 feet.

AASHTO [1994] uses 30 feet as the design length for single-unit trucks. Long [2002] reported measurements of vehicles in this class up to 34.0 feet. Fancher and Gillespie indicate 35 feet as the maximum length for this class of vehicle [NCHRP Synthesis 241, 1997].

Consequently, AASHTO design vehicles do not represent the worst case, but a length which is not exceeded by many vehicles in the class. This design policy of applying reasonable design characteristics rather than attempting to identify worst-case scenarios, which can always be violated by some unforeseen and/or improbable worse case, is germane to railroad grade crossings as well as the other design applications of the Green Book.

Bicycles

Until rather recently, there was a recognition that bicycles were incompatible with motor vehicles, and bicycle facilities were often designed to help keep bicycles out of traffic streams on arterial roads. With the new 'Share the Road' concept, bicycles are now probably the worst-case vehicles involved in preemption, not for risks to the bicycle riders themselves, but for their impacts on other traffic. The typical bicycle rider at the front of a left-turning queue surely does not accelerate from a stop very quickly, nor attain a very high speed, during the clearing of the queue behind him before a train arrives.

The analysis in this study pertains to motor vehicles, but it should be borne in mind that bicycles may warrant special consideration.

CLEARANCE TIME

Definitions are important in understanding the elements of clearance time. The recently revised MUTCD [2001] provides definitions for some new terminology as well as terms which have become common in use.

Clearance Configuration

In §8A.01(8), the MUTCD [2001] defines minimum track clearance distance as "the length along a highway at one or more railroad tracks, measured either from the highway stop line, warning device, or 3.7 m (12 ft) perpendicular to the track centerline, to 1.8 m (6 ft) beyond the track(s) measured perpendicular to the far rail, along the centerline or edge line of the highway, as appropriate, to obtain the longer distance". The minimum track clearance distance is illustrated schematically in Figure 1. For a single track of standard gage, having a nominal

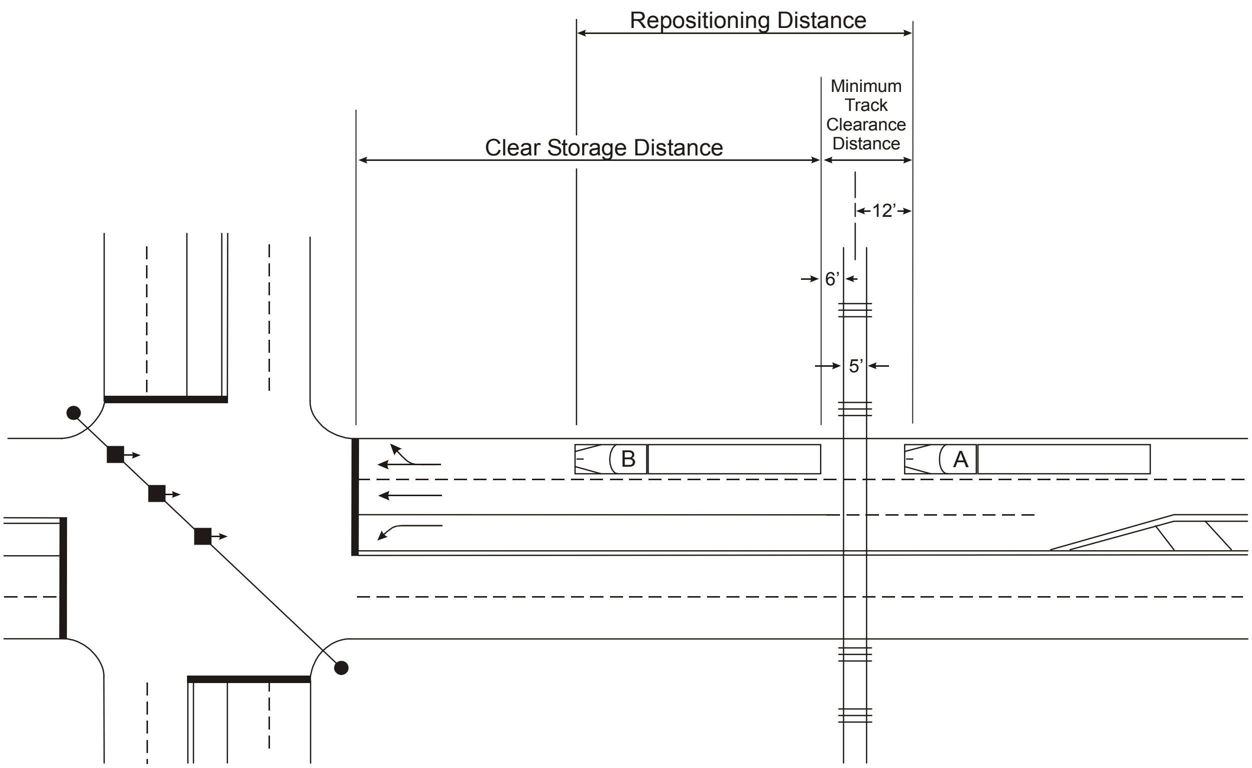

FIGURE 1 Schematic Diagram.

separation of 4'-8½" between the insides of the rails, the perpendicular distance between the centers of the rails is about 5 ft. With a minimum of 12 ft to the track centerline, 2.5 ft from the track centerline to the center of the far rail, and 6 ft of buffer for train overhang beyond the far rail, the minimum track clearance distance adds to 20.5 ft for a single track that is perpendicular to a roadway. For any other configuration, this distance is longer. This is the distance which must be completely cleared during preemption time by the repositioning of any vehicles which have any portion stopped within this area when an approaching train is detected, such as Vehicle A in Figure 1 which encroaches slightly into this area, to a new position completely beyond this area as shown by Vehicle B in Figure 1.

In §8A.01(3), the MUTCD [2001] defines clear storage distance as the space available for vehicle storage measured between 6 ft from the rail nearest the highway intersection to the intersection stop-line or normal stopping point on the highway, as illustrated schematically in Figure 1. This terminology seems awkward because most persons would probably not think of this area as being clear when any vehicles are within it waiting on the traffic signal to turn green, but such vehicles would be clear of the railroad track if totally contained within this area. Another paradox to most persons is the fact that the time needed to clear all vehicles from the minimum track clearance distance will not necessarily clear all vehicles from this area unless it is very short. When approaching the roadway intersection from the track, the clear storage distance begins exactly at the end of the minimum track clearance distance.

Critical Time

The clearance time of a vehicle starting from within a queue can generally be considered as being composed of two time intervals. The first interval is the startup delay. It begins at the instant when a signal turns green allowing queued vehicles to move, and ends at the instant when the final vehicle at risk in a queue initiates movement. The second interval is the repositioning time. It involves the time needed for the last vehicle to accelerate from rest and travel a distance sufficient to clear the crossing. Repositioning time begins at the same instant that the startup delay ends. The critical clearance time is the sum of the startup delay and repositioning time of the last vehicle at risk in a queue.

To start a vehicle in motion to clear the minimum track clearance distance, any vehicles queued in both the clear storage distance and the minimum track clearance distance must be moved. The length of queue to be moved to allow repositioning is equal to the clear storage distance plus the minimum track clearance distance. The repositioning distance is the length of the minimum track clearance distance plus the length of the design vehicle.

Startup Delay

Startup delay is associated with the fact that queued vehicles begin moving sequentially, rather than simultaneously, in somewhat of a Doppler wave effect. Each vehicle must wait for the vehicle ahead of it to move out of its way in order to get started. This sequential process is attributable to human reaction time in addition to a likely motorist desire to allow the gap between successive vehicles to widen beyond what is acceptable while stopped since it is well established that motorists allow larger gaps between vehicles as speed increases. If two successive vehicles started simultaneously to accelerate at the same rate, the distance between them at the end of acceleration would be the same as while stopped in the queue.

Startup delay relates more to the number of preceding vehicles in a queue, rather than to actual distances between stop-line and front bumper, because both the front and rear of each vehicle begins moving at the same instant regardless of the length or type of the vehicle. The front bumper of a passenger car queued behind two 74 ft long combination trucks would likely be around 180 ft from the stop-line. The same car queued behind six 16 ft long passenger cars would likely be about the same distance back. However, only two vehicles need to start before the car can move in the truck queue, but six vehicles must start sequentially before the car can move in the automobile queue.   For startup delay, the distance to the front of a queue is not as important as the number of preceding vehicles to the front of the queue.

If each motorist's response time is the same regardless of position in a queue, then the startup delay until any specific vehicle could start to move could be determined by simply multiplying the vehicle's ordinal position in a queue by the average motorist response time. Each vehicle behind the leading vehicle has preparation time from watching the traffic signal and leading vehicles begin to move to predict when the vehicle ahead of it should start to move. Since, the first vehicle lacks this preparation time, some slight additional delay time might be evoked by the driver of the lead vehicle in getting started. Allowing for excess startup time by the lead vehicle, the startup delay for the n^{th} vehicle in a queue can be described by a simple model

$$d_n = \tau + n * T \qquad (1)$$

where

d_n = startup delay for the n^{th} vehicle in a queue (sec),
τ = excess startup time of the lead vehicle in a queue (sec),
n = ordinal 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).

Previous Study Findings

Startup delays were studied in five traffic lanes at two intersections for five days by George and Heroy [1966]. Their data to conform very closely to the simple startup delay model shown above. Their data show that a value of 1.25 sec for the uniform response time and 0.5 sec for excess startup time of the lead driver fit their sample observations very closely. Although they did not disclose standard deviations, they reported that 85% of their observations were less than 1.4 seconds per vehicle. While the uniform startup time was consistently 1.25 sec per vehicle in all five lanes, the excess startup time varied slightly from 0.2 sec to 0.7 sec. No significant differences were found between queues in left-turning lanes and through lanes, nor between peak-hour queues and off-peak queues. The average distance occupied by stopped vehicles was found to be 25 ft. Although the mix of vehicles was not reported, there was no indication that queues involving trucks were rejected. The authors indicated that 6.2% of their 6,615 samples involved excess delay due to driver inattention.

Herman et al. [1971] found that driver responses in a platoon of queued vehicles remained fairly constant as the platoon started moving and increased speed. The speed at which driver response waves propagated back through a platoon was found to be about 26 fps, and the average distance between stopped vehicles was 25.9 ft. Dividing 25.9 ft by 26 fps yields a startup response time of 1.0 sec per vehicle, which is somewhat shorter than the 1.25 sec found by George and Heroy and presumes a value of zero for excess startup time for the first driver.

Messer and Fambro [1977] measured queue lengths up to 429 ft long at two intersections having few trucks. They found average storage lengths of 23.3 to 24.0 ft per vehicle for left-turning lanes and 24.0 to 25.3 ft for through lanes. They concluded that the slight differences were insignificant so the same value could be used for both left-turning and through queues.

They also studied queue startups at three busy intersections and roughly fit a model with integer parameters to the vehicle starting times of queues with up to 18 vehicles, resulting in a uniform startup time of 1 sec per automobile (trucks and buses were considered as two automobiles) and excess time for the first driver of 2 seconds. This agrees with the 1 sec per vehicle found by Herman et al. Their plot was digitized and a model fit by linear regression to circumvent the integer parameter constraint. It explained 97.7% of the variation in the measured values and resulted in a uniform startup time of 1.1 sec per vehicle and an excess time for the first driver of 2.0 sec.

In summary, these findings indicate no significant differences in startup delays between peak and off-peak travel, no significant differences in startup delays between left-turning and through queues, and uniform startup times averaging 1.0 sec, 1.1 sec, and 1.25 sec. Excess startup times for the first vehicle averaged 0.5 sec and 2.0 sec. Motorist inattention may cause abnormal excess delay in about 6.2% of queues. No significant differences were found in queue storage space per vehicle between left-turning and though queues comprised mostly of passenger cars.

Discharge Headway Relationship

Discharge headways are another measure related to startup times. The *Highway Capacity Manual* [HCM, 2000] indicates that multiple studies have found that the discharge headways across a stop-line for drivers in a queue are constant for each driver, except for the first few drivers. The first driver in a queue has the longest response time after a signal turns green. The second driver takes less time than the first to cross the stop-line, the third driver less time than the second, etc., until after several drivers, when each remaining motorist takes about the same time to cross the stop-line as the vehicle ahead of it. The excess time of the first few drivers is usually combined and considered as startup lost time.

For ideal conditions, the HCM in Chapter 16 recommends a value for the saturation flow rate of 1,900 pc/hr/ln. This corresponds to a saturation flow headway of 1.9 sec/pc. Exhibit 8-29 of the HCM summarizes some recent studies showing saturation headways close to this value [HCM, 2000, p. 8-27]. This represents the minimum expected headway under ideal conditions. Average observed headways should be expected to be longer than 1.9 sec under most actual conditions, and vary among drivers and with different non-ideal conditions.

Startup lost time and discharge headways have often been studied at traffic signals because of being key variables affecting the capacity of a roadway in terms of the vehicles per unit of time that can be accommodated flowing through signalized intersections. However, startup lost time and discharge headways are measured at the stop-line, which confounds startup delays with acceleration rates and travel times to the stop-line. The relationships developed by Messer and Fambro [1977] of $T_f = 2 + 1 * n$ for the startup delay for the n^{th} vehicle in a queue, and $T_c = 2 + 2 * n$ for the time after the start of the green signal for the n^{th} vehicle in a queue to reach the stop-line, indicate an increase of about 1 sec per successive vehicle after startup before arriving at the stop-line. Presumably this is due to accelerating a little slower than the vehicle immediately ahead to allow the spacing between vehicles to increase with increasing speed.

Bonneson [1992] calibrated a discharge headway model by combining startup delay with acceleration to the stop-line to fit queue data collected for 12,053 through vehicles at five study sites in Florida and Texas. His model resulted in a coefficient of 1.57 sec per vehicle for the uniform response time and 1.03 sec for the excess startup time of the lead vehicle. However, this is only indirect evidence because these times were not directly measured and are confounded with other factors. Furthermore, individual terms in a regression model cannot be extracted from the full model, which included additional terms for accelerations from starting positions to the stop-line.

Data Collection and Analysis

Data were collected at four sites for this study as reported earlier by Blackadar [1998]. The four sites were chosen so that one site involved almost exclusively automobiles, for comparison with the results of the studies in the literature, one had a large proportion of heavy trucks for studying the impacts of trucks, and the other two were typical urban sites selected to represent mixes of vehicles that would normally be found in typical real-world locations. One site involved a 5% upgrade. Over 27% of the vehicles in the queues measured on the truck route were not passenger vehicles.

Data were collected for 140 queues, each containing up to 16 vehicles. A couple of outliers in the data were identified (discussed later) and the other observations were randomly assigned to two data sets. One set contained one-third of the observations from both the site with few trucks and the site with many trucks as well as all of the observations from one of the other two sites for use in model calibration and analysis. The remaining observations from the high and low truck sites and all observations from the fourth site were contained in the other set for use in testing and validation.

An analysis of variance (ANOVA) was performed using a standard statistical software package [SPSS, 1998]. No significant differences were found in startup times between queues that contained trucks and queues with only passenger vehicles. No significant differences in startup times were found between sites with level approaches and the site on a 5% upgrade. Furthermore, no significant differences in average startup times were found among the different study sites or for queues of different lengths. Consequently, separate models are not needed for different vehicle types, different vehicle lengths, different grades (up to 5%) or different queue lengths.

The uniform startup time was found to be 1.1 seconds per vehicle with a standard error of 0.1. The excess startup time of the lead driver was found to be 1.1 seconds with a standard error of 0.85. These values fall in the middle of the range of values found in the literature. The uniform startup time is the same as the smaller value reported in the most recent study in the literature, which is reassuring since the HCM [2000] reports a trend of reductions in discharge headways, possibly associated with a shorter passenger car fleet, which suggests likely collateral reductions in startup delays. While the variation in uniform startup times is remarkably small, the variation in excess lead-off time is large enough for the time not to be significantly different than zero.

The increase in excess startup time for the lead vehicle from 0.5 seconds in 1966 to 1.1 seconds today may have been affected by the change in the practice of treating the yellow interval as a clearance interval. In 1966, traffic signals were generally set to clear an intersection before releasing the next traffic movement, where the practice now is to just get the last vehicle into an intersection before releasing the next stream. Lead drivers may now have more reason to hesitate after a signal turns green to see if any stragglers are racing into the intersection.

Similar to George and Heroy [1966] reporting 6.2% of their observations involved excess delay due to driver inattention, 5% of the samples collected in this study also seemed to involve excess delay. All of the excess delay samples were at only one of the survey locations where there were plenty of distractions such as coeds, and occasionally colleagues, strolling by. With cell phones, PDAs, CD players, and other new types of distractions now available, it is a little surprising that the percentage of these incidents was lower than found in 1966.

The excess delay seemed most evident in the longest queues, involving 14 or more vehicles. The probability of one of, say, 15 drivers being distracted or inattentive is surely greater than the probability of one out of only one or two drivers, which is probably why excess delay seemed more prone to strike the longer queues. The effect is cumulative. If any one of 15 preceding drivers is inattentive, the added delay is incurred by all following motorists. The excess delay seemed to range from about 5 to 7 seconds longer than the average delay. This exceeded the range of 95% of motorists which were within about 2 seconds of the average. The entire remainder of vehicles in the queues were delayed by that amount of time, before inattentive motorists awakened to the fact that they could and should be moving, sometimes from the sound of a horn of a trailing motorist. The sample of inattentive motorists was too small to support computing a rigorous average with a maximum value at some level of confidence. Another possible explanation might be that when motorists are stopped in a queue with many vehicles ahead of them they may expect to be delayed longer than actually occurs. Not expecting to be able to move soon may tempt diversions of attention, such as quickly dialing a cell-phone call or finding a different CD to play, contributing to inattention in long queues.

The calibration and validation results are shown in Figure 2. Both the calibration and validation data sets have been superimposed in the figure. The model appears to fit the validation data as well as it fits the calibration data. A couple of the combined data points lie outside the prediction limits of the calibration data at a 95% level of confidence. This is expected since nominally only 95% of data points would be expected to be within the interval.

Only longer than average delays are critical when clearing queues near railroad tracks. Consequently, a one-sided limit was chosen to avoid influence by any delays which might be abnormally short. Since most traffic signals operate with cycle lengths which process 100 queues for each approach every 1½ hours to 3½ hours, 5 failures for a 95% level of confidence is too many. One failure every 1½ hours to 3½ hours for a 99 % level of a confidence is also too many. Consequently, a 99.9% level of confidence was chosen. Due to the possible occurrence of excess delay from inattentive drivers, it was decided that higher levels of confidence were unlikely to increase actual confidence. The 99.9% limit on expected delays is shown in Figure 2 and is given by

$$d_n\{99.9\%\} = 1.1 * (n + 1) + (32 + 0.1 * (n - 7.8)^2)^{1/2} \qquad (2)$$

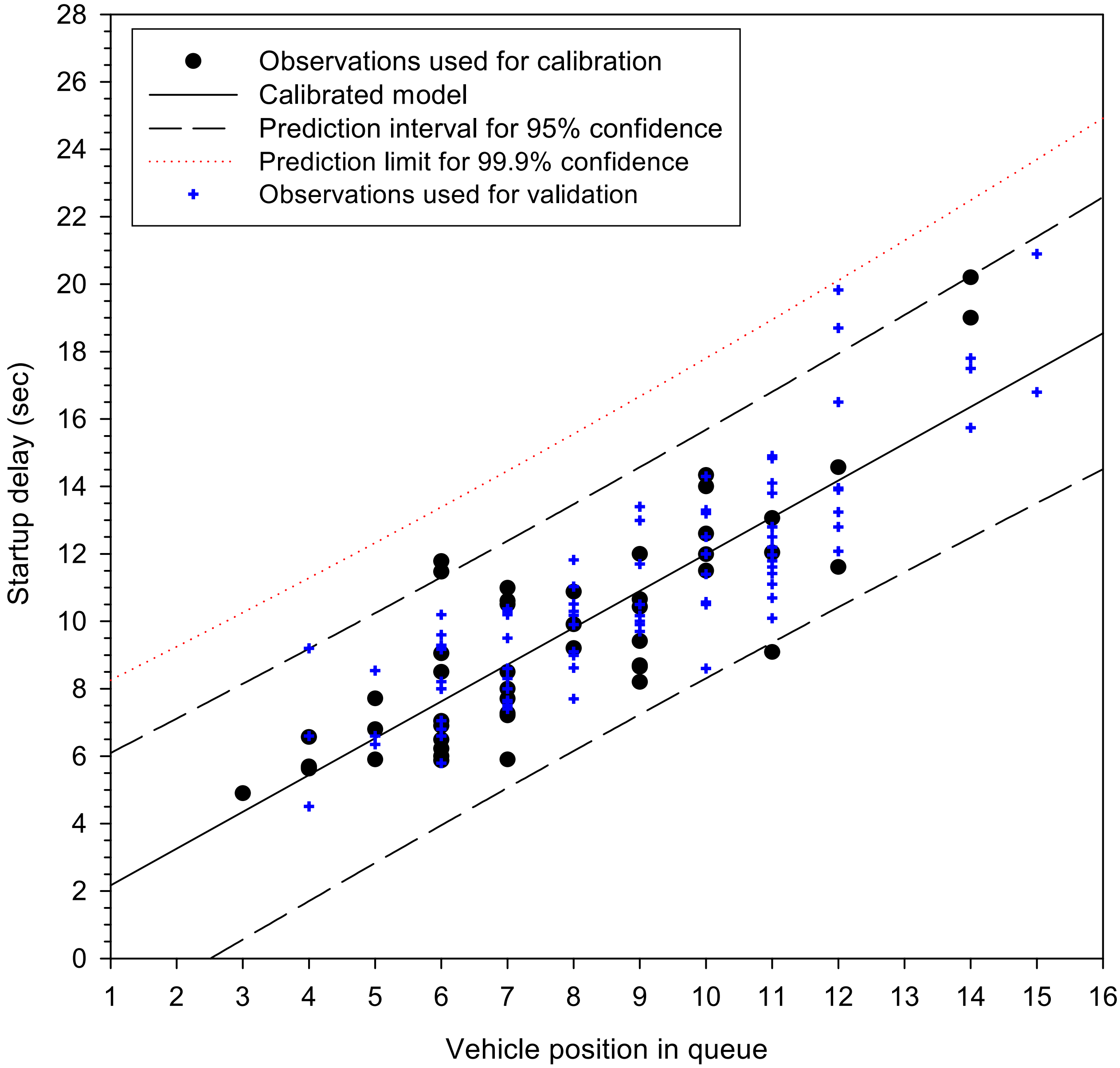

FIGURE 2 Observed startup delay by queued vehicle position.

It should be noted that none of the observations exceeded this much delay, and not many were even close. Only 1 observation in 1,000 would be expected to exceed this much delay (disregarding inattentiveness). Much fewer than 1,000 observations are contained in Figure 2.

While the linear nature of the data is obvious in Figure 2, it should be noted that none of the observations in the calibration set, and only two observations in the validation set, are below-average in length of delay at the longer queues of 14 or more vehicles. This might be due to chance, or it might be due to some increased level of inattentiveness among drivers in queues of these lengths which added some extra delay time that was not severe enough to be evident. Identifying inattentiveness is a judgment, and mild episodes are not easily recognized.

The *Highway Capacity Manual* [HCM, 2000, p. 8-27] states that research studies have found increases in saturation headways when green phases exceed 40 or 50 sec. This could also be due to driver inattentiveness. A queue of 14 vehicles at 1.9 sec headways with 2 sec of startup lost time and 5 to 7 sec of excess delay due to driver inattention requires a green phase of 34 to 36 sec, not far from 40 sec. Startup delays and saturation headways may both begin to be influenced by driver inattentiveness in queues of about 14 or more vehicles.

Queued Vehicle Spacings

To be useful, vehicle positions in queues must be converted into measurable lengths in feet or meters from the front of a queue. Since studies involving queuing models abound in the literature, it might be expected that a wealth of good information on the conversion of queue lengths should be readily available. Not so.

A possible source which immediately comes to mind is the *Highway Capacity Manual* since it offers techniques for estimating queue lengths. In these cases, however, queue lengths are represented by the numbers of vehicles in queues, not the physical lengths of queues in meters or feet. The HCM in some instances accepts an "average queue spacing" factor [HCM, 2000, p. 16-156] as a user input to convert the number of queued vehicles into feet, but no recommendation is offered as to what value should be used. "One must assume an average distance between front bumpers of successive vehicles standing in queue" [HCM, 2000, p.16-156]. Mighty helpful.

Since conversion of the number of queued vehicles into feet of storage for determining the length of vehicle storage lanes is a design issue, most engineers would look to the AASHTO references for design guidance. The matter does not seem to have been addressed in any of the nine initial policies that were published by AASHO between 1938 and 1944. The 1954 Blue Book recommends "allowing 25 feet for each arriving vehicle waiting to turn" [AASHO, 1954, p. 336] for the design of median turn bays. Table II-5 of the 1954 Blue Book indicated that 1953 passenger cars ranged from 15.1 ft to 20.2 ft in length, and the length for the P design vehicle was chosen as 19 ft. No source for the 25-ft allowance was given, but with a design length of 19 ft, an inter-vehicle spacing of 6 ft is implied.

The 1957 AASHO Red Book also specifies 25 ft for passenger car spacings [AASHO, 1957, p. 171]. The 1965 AASHO Blue Book [AASHO, 1965] contains the exact same wording as in 1954, again indicating 25 ft as the design length for passenger cars including inter-vehicle spacings. This was about the beginning of an era of increased popularity of shorter passenger cars. By the 1973 Red Book [AASHO, 1973] and the 1984, 1990 and 1994 AASHTO Green Books [AASHTO, 1984, 1990, 1994], most passenger cars were shorter than the lengths of 15.1 to 20.2 ft of 1953, yet none of these editions recommends a specific design length for use in determining the storage lengths of auxiliary lanes. These publications merely say that an auxiliary lane should be "sufficiently long" to store the number of vehicles likely to accumulate during a critical period. All editions continue to adopt 19 ft as the length of the P-vehicle.

The 25-ft spacing assumption seems to have attained widespread acceptance. It is frequently cited in the literature without any reference to a source. Examples can be found in NCHRP Report 93 [1970, p. 133], FHWA's "Design of Urban Streets" [JHK and Associates, 1980, p. 8-14], and other popular references such as *Transportation and Land Development* [Stover and Koepke, 1988, p. 144]. It is also frequently embedded as a parameter or default value in computer software packages.

It is not usually clear in the literature whether the 25-ft spacing is supposed to be the average spacing of the vehicles in a queue, or is supposed to represent the maximum spacing. If it is the average spacing, then it would be expected that the resulting storage bays should be too short roughly half of the time. The literature is also usually vague as to whether the 25-ft spacing pertains to passenger cars alone or represents some average length that includes some unknown percentages of some types of trucks. As an example, Oppenlander and Oppenlander [1999, p.85] state, "A vehicle length of 25 feet can be selected for design, unless a large proportion of trucks/buses exists on the lane(s) under consideration."

Other spacings are also specified, usually without any reference to a source. For example, Basha [1992] converts queues into storage lane lengths by multiplying by 20 ft per vehicle.

The storage space consumed by trucks is also frequently assumed without reference to any source. For example, Gattis [2000] assumes that one truck including headway is 15.2 m (50 ft), as well as the routine assumption of one automobile including headway being 7.6 m (25 ft). Venglar et al. [2000] offer recommendations for assumptions for queue spacings for several types of vehicles: 25 ft for cars, 36 ft for SU trucks, 46 ft for school buses and 61 ft for WB-15 combination trucks.

The lack of definitive data on the space occupied by vehicles in queues was recognized by ITE Technical Council Committee 5D-10 [1995]. They calculated the stacking distance needed to accommodate queues based on a front bumper to front bumper space of 22 ft per vehicle. "This 22 ft. may not be the exact space that vehicles occupy, but a value ranging from 20 ft. to 25 ft. seems appropriate for many situations."[ITE Technical Council Committee 5D-10, 1995, p. 42] The committee goes on to say that investigations are needed of the amount of space occupied per vehicle within a queue so that engineers will have the ability to project not only the number of vehicles that will be in the maximum queue for a given site, but also the queue storage length required for a site.

Studies where queues have been measured have verified that a 25-ft assumption for automobiles is not unreasonable. Herman et al. [1971] reported finding the average space per vehicle in a queue to be 25.9 ft. Messer and Fambro [1977] studied two intersections with mostly passenger cars and found an average length of about 25 ft for each queue position. As averages, somewhat longer spacings would be expected to be needed for design if more than half of the queues are to be accommodated.

When simulation software involves physical movements with graphical features depicting vehicles in motion, conversion of queued vehicles from counts to scaled images is needed, which requires values for vehicle lengths and inter-vehicle spacings. TSIS record type 58 pertains to vehicle types. It is used in TSIS only by CORSIM, for the purpose of providing greater detail in specifications [TSIS, 1998]. Because nine different vehicle types are allowed and predefined, the use of this record type is optional since default values are provided for each of the types. Two passenger cars are predefined. One is 16 ft in length and comprises 75% of the passenger car fleet. The other car is 14 ft in length and accounts for the remaining 25%. Combining the two types of cars yields an average length of 15.5 ft for the passenger car fleet. Four types of trucks are predefined. Combination trucks are 53 ft and 64 ft in length, but by default, 100% of the truck fleet is assumed to be 35-ft long single-unit trucks. Buses are predefined as 40 ft in length. CORSIM then adds 3 ft to the bumper-to-bumper lengths of each vehicle to account for inter-vehicle spacings. Inter-vehicle spacings cannot be changed except indirectly by adjusting the bumper-to-bumper lengths of vehicles. No sources are cited for the inter-vehicle spacing or for any of the predefined vehicle lengths.

Even less information was found in the literature on vehicle set-backs from intersection stop-lines, which affect the stacking lengths of queues. Bonneson's [1992] data indicate that the average speed at the stop-line of the vehicle in the lead position in queues was about 10 ft/sec at each of 5 study locations. For a leading vehicle to be traveling at 10 fps at the stop-line, it must have been stopped behind the stop-line and accelerated to 10 fps over some offset distance. Bonneson's study used tape-switches set slightly past the stop lines, so the overhang of the vehicle in front of its wheels accounts for a portion of the distance traveled while accelerating.

As reported earlier in this study, Blackadar's [1998] measurements at four intersections found the average space per queued passenger car to be 24.9 feet and an average space for queues of mixed vehicles to be 37.2 feet.

Expected Minimum Queue Lengths

Average spacings in queues are not really what is needed. Designing for the average creates failures for the half of the queues which are either above-average or below-average in length, depending upon the design situation.

For a fixed distance between a roadway intersection and a railroad crossing, the important issue is how many vehicles can be squeezed into this fixed space, because each vehicle must start and move forward in order to clear the track. The maximum number of vehicles that will squeeze into a space corresponds to the expected minimum queue lengths. Clearly, the shortness of vehicles is critical. Passenger vehicles are shorter than trucks and buses, so queues with only passenger vehicles will likely contain more vehicles to start and move, and need to be considered separately from mixed queues which also contain trucks and buses.

AASHTO [1994] prescribes maximum design lengths for vehicles, but not minimum lengths. This dearth of design information is not helpful in determining the maximum number of vehicles that are likely to squeeze into a fixed queuing space. The ITE Handbook [1992a] contains some dimensional information, as does NCHRP Synthesis 241 [1997] for several classes of trucks. These are helpful in the absence of design specifications.

A recent paper offers variations of the following equations for converting the numbers of vehicles of different types into expected lengths of queues in feet [Long, 2002]:

$$EQL = 27\ EPV + 77\ ETT + 42\ EO \tag{3}$$
$$SD = (50 + 25\ EPV + 77\ ETT + 70\ EO)^{1/2} \tag{4}$$

$$EQL_{min} = EQL - Z * SD \quad (5)$$
$$EQL_{max} = EQL + Z * SD \quad (6)$$

where EQL = expected queue length (ft)
SD = estimated standard deviation of length of queue
EPV = expected number of passenger vehicles in queue
ETT = expected number of combination trucks in queue
EO = expected number of other vehicles in queue
Z = one-tail normal deviate at a selected level of confidence

Adopting the same 99.9% level of confidence as applied for startup times results in a value for Z of 3.090. The values for EPV, ETT and EO need to be rounded to whole vehicles [Long, 2002].

These equations were applied to determine the expected minimum queue lengths for various compositions of passenger vehicles, combination trucks and other vehicles. Selected results are shown in Table 1. These results indicate the shortest distances in feet expected from the stop-line to the rear bumper of the last vehicle in 99.9% of queues.

It should be noted that the expected minimum queue length model compensates for vehicle offsets at the stop-line. Most motorists were found not stopping at the stop-line, but behind it. However, a few stopped past it. Consequently, with a 99.9% level of confidence, the models reflect that a motorist in a small car might stop completely past the stop-line, and indicate that the shortest queue length expected for a single passenger vehicle would be when its rear bumper is 0 ft from the stop-line. It was noted that the offsets of the lead-off vehicles from the stop-line varied significantly among study sites, probably due to differences in stop-line placement, so care is warranted in considering the placement of stop-lines when applying these results to other locations [Long, 2002].

Expected Maximum Startup Delays

The models for expected minimum queue lengths in Equations 3, 4 and 5, and expected maximum startup delays in Equation 2, were applied with queues composed of various numbers and types of vehicles. Since no significant differences were found in the uniform startup delays between cars and trucks, trucks can be disregarded.

The longest startup delays are caused by the most vehicles stacked in a queue. For a queue of a specified length, more cars can be encompassed than trucks, or cars and trucks mixed. While some trucks are prohibited on some roadways, cars are generally not prohibited on any roadways where trucks are allowed.

TABLE 1 Expected Minimum Queue Lengths

	Combination Trucks															
	0				1				2				3			
Passenger	Other Vehicles				Other Vehicles				Other Vehicles				Other Vehicles			
Vehicles	0	1	2	3	0	1	2	3	0	1	2	3	0	1	2	3
0	-	8	41	76	42	76	111	146	110	145	181	217	179	215	252	289
1	0[a]	32	66	101	66	100	135	171	134	170	206	242	204	240	277	314
2	23	56	90	126	90	124	160	196	159	194	231	267	229	265	302	339
3	46	80	115	150	114	149	185	221	183	219	256	293	254	290	327	364
4	70	104	139	175	138	174	210	246	208	244	281	318	279	315	352	390
5	94	129	164	200	163	199	235	272	233	269	306	343	304	341	378	415
6	118	153	189	225	188	223	260	297	258	294	331	369	329	366	403	441
7	143	178	214	251	212	248	285	322	283	320	357	394	354	391	429	466
8	167	203	239	276	237	273	310	347	308	345	382	419	379	416	454	492
9	192	228	264	301	262	299	335	373	333	370	407	445	404	442	479	517
10	216	253	289	326	287	324	361	398	358	395	433	470	430	467	505	543
11	241	278	314	352	312	349	386	424	383	421	458	496	455	493	530	568
12	266	303	340	377	337	374	411	449	409	446	484	521	481	518	556	594
13	291	328	365	402	362	399	437	474	434	471	509	547	506	544	582	620
14	316	353	390	428	388	425	462	500	459	497	535	573	531	569	607	645
15	341	378	416	453	413	450	488	526	485	522	560	598	557	595	633	671
16	366	404	441	479	438	476	513	551	510	548	586	624	582	620	658	697
17	392	429	466	504	463	501	539	577	536	573	611	649	608	646	684	722
18	417	454	492	530	489	526	564	602	561	599	637	675	633	672	710	748
19	442	480	517	555	514	552	590	628	586	624	663	701	659	697	736	774
20	468	505	543	581	540	577	615	654	612	650	688	727	685	723	761	800
21	493	531	568	606	565	603	641	679	638	676	714	752	710	749	787	826
22	518	556	594	632	591	629	667	705	663	701	740	778	736	774	813	851
23	544	582	620	658	616	654	692	731	689	727	765	804	762	800	838	877
24	569	607	645	683	642	680	718	756	714	753	791	830	787	826	864	903
25	595	633	671	709	667	705	744	782	740	778	817	855	813	851	890	929
26	620	658	696	735	693	731	769	808	766	804	843	881	839	877	916	955
27	646	684	722	761	718	757	795	834	791	830	868	907	864	903	942	980
28	671	710	748	786	744	782	821	859	817	856	894	933	890	929	968	1006
29	697	735	774	812	770	808	847	885	843	881	920	959	916	955	993	1032
30	723	761	799	838	795	834	872	911	869	907	946	985	942	980	1019	1058

[a] At 99.9% level of confidence; queue lengths in feet.

The expected maximum startup delays for queues of various lengths are shown in Figure 3. Startup delays in this figure are conveniently related to queue lengths in feet rather than in number of queued vehicles. The relationship is virtually linear and can be adequately described as follows

$$d_L = 7.2 + 0.05 * L \tag{7}$$

where

d_L = expected maximum startup delay for a vehicle starting in a queue with front bumper at position L (sec),

L = distance from the stop-line to the front bumper of a queued vehicle (ft).

While the relationship in Figure 3 has been truncated at 1,000 ft, it should be kept in mind that the queues observed in the calibration and validation data only extended to 15 vehicles and reached about one-third of this span.

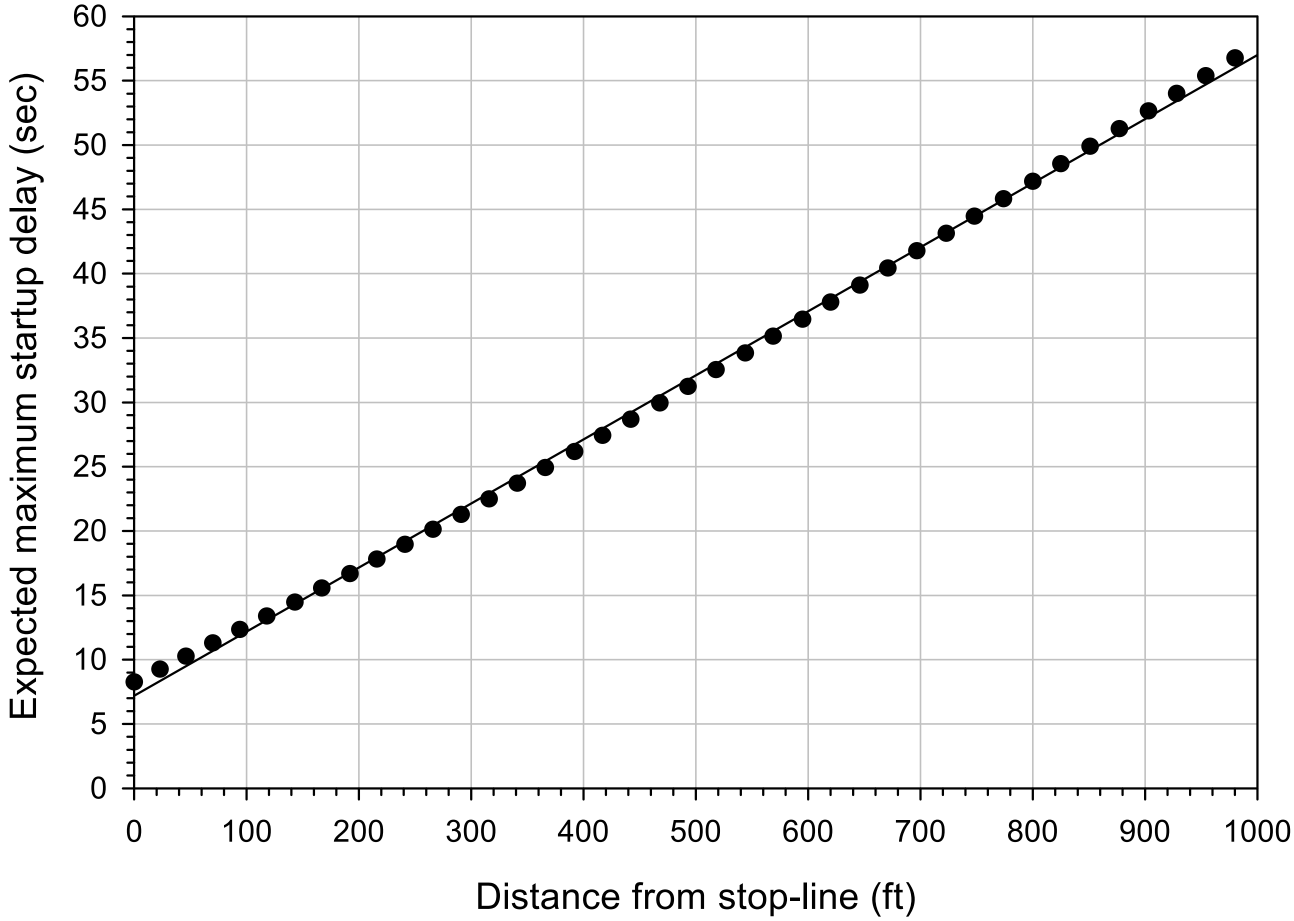

FIGURE 3 Expected maximum startup delay by distance from front of queue.

Although the relationship in Figure 3 has been fitted with a smooth line, the theoretical relationship is saw-toothed. The theoretical delay does not increase as distance is increased until the next point is reached, when delay abruptly increases to the level at the next point. Since the distances between data points are so small, the relationship is best approximated by a smooth line through the data points.

The model intercept value of 7.2 sec at 0 ft is a little conservative because the data point has a value of 7.9 sec. This is due to the slight non-linearity in the prediction limit for expected delays at 99.9% confidence. The level of confidence is highest in the middle of the observed data range, which corresponded to queues of about 8 vehicles. A queue length of 0 ft is not likely to occur between railroad crossings and roadway intersections so this slight startup aberration of 0.7 sec at this extremely short queue length is harmless.

Between queue lengths of 0 ft and 1,000 ft, the delay time of the model increases by 50 sec, from 7.2 sec to 57.2 sec. This corresponds to vehicles sequentially starting in motion in a wave propagating back through a queue at a minimum rate of 1,000 ft / 50 sec = 20 fps. This is considerably slower than the propagation rate of 26 fps reported by Herman et al. [1971]. However, Herman's rate represented an average, so approximately half of the time the actual rate would be expected to be slower and the trailing vehicles would be unable to depart in time to clear a railroad track. The theoretical approach suggested by Marshall and Berg [1997] assuming a parabolic relationship between flow rate and density, which is not an especially accurate assumption, yields a queue dissipation rate of 19.5 fps for through lanes and 17.1 fps for left-turning lanes, using the default values recommended by the authors. These queue dissipation rates seem to be a little too conservative and imply differences in startup delays between left-turning and through traffic which have not been observed to exist.

The rate determined here is expected to be sufficiently slow to accommodate nearly all startups, if not influenced by inattentive motorists, yet not excessively conservative leading to unrealistically long preemption times. It should be kept in mind that the wave propagation rate is for a queue of only passenger vehicles. The propagation would be faster with trucks in a queue, but presuming that a queue contains some trucks would mean insufficient time would be available to clear a queue that contains only passenger vehicles.

A comparison of the model results with the validation data is shown in Figure 4. Queues containing only passenger vehicles are shown. The observations appear to show a trend that is roughly parallel to the line representing expected maximum values, but offset from it. The slowest delay times seem to be about 7 seconds faster than the maximum line, which could lead to a mis-perception that the maximum expected delay might be over-estimated by 7 seconds. It must be remembered that the line was chosen based on a probability that only 1 queue in 1,000 would take more time to begin to move. There are only 33 observations in the sample shown. As the number of observations increases to 1,000, it must be recognized that some observations will likely lie closer to the line, but hopefully no more than one will lie above it.

Repositioning Time

Repositioning time is the time required to reposition every portion of a vehicle in a queue to clear the minimum track clearance distance, as illustrated in Figure 1. It starts as the vehicle departs its position in the queue and begins to move to a safe position. The distance of travel during repositioning time is measured from the front bumper of the vehicle at the time of startup to the

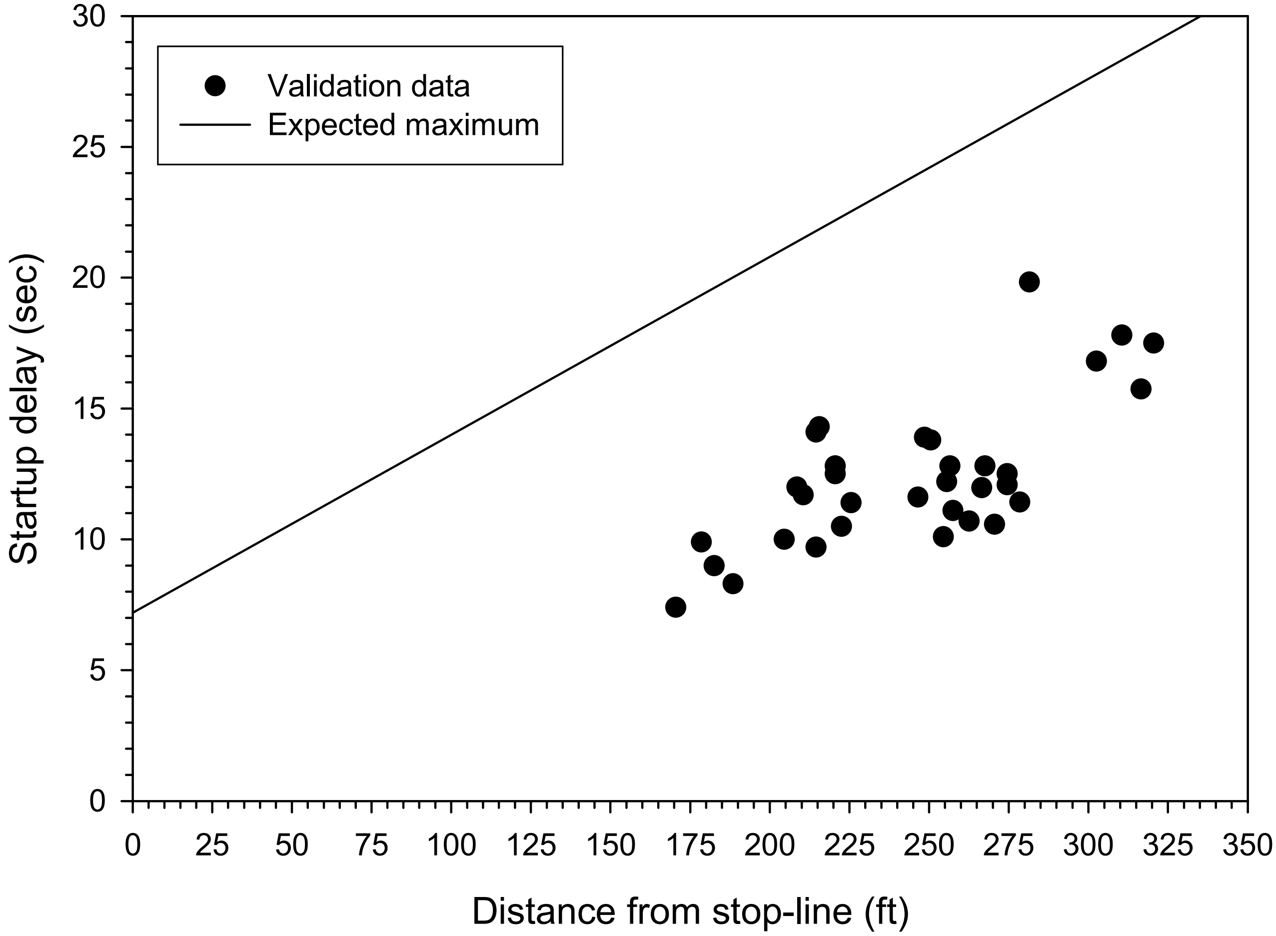

FIGURE 4 Observed and expected maximum startup delay.

front bumper of the vehicle at the point of safety, when the rear bumper is 6 ft clear of the track. The distance of travel during repositioning time includes the rear bumper traveling the length of the vehicle to the position of the front bumper and then traveling the minimum track clearance distance to the point of safety. The vehicle will be stopped at the beginning of repositioning time and in motion at the end of it.

Repositioning time pertains to the final vehicle in a queue. It depends on the type of vehicle being repositioned for two reasons. First, the vehicle length affects the repositioning distance since the length of the vehicle is part of the travel distance. Second, the acceleration of a vehicle is affected by the type of vehicle. Fully-loaded combination trucks cannot accelerate as quickly as most passenger vehicles. Additional factors can also affect truck acceleration, such as the grade of the roadway in the vicinity of the end of the queue.

Repositioning time consists of two components. The first component is the time spent accelerating from rest to a maximum speed. The second component is the time spent traveling from the end of acceleration until the rear bumper reaches the safe position. It is presumed that once a vehicle discontinues accelerating, it continues at a steady speed as long as any portion of it is within the repositioning area. If the distance to be traveled to reach the safe position is short and the design vehicle is a passenger car which can accelerate continuously, the second component of repositioning time may be zero since the vehicle may continue to accelerate while traveling through the entire repositioning distance.

There are two aspects of acceleration which must be considered. The first is the maximum acceleration of which a vehicle is capable under prevailing conditions. This acceleration might be applied in an emergency situation such as by the last vehicle in a queue when a train is approaching a crossing. The second is normal acceleration chosen by a driver. Rarely do drivers accelerate their vehicles at the maximum acceleration available to them. Heavy trucks tend to do this more often than automobiles for two reasons. The first is that heavy trucks are powered by diesel engines which are designed to be operated continuously at their maximum governed speed, unlike the gasoline engines in most automobiles. The second is that heavy trucks are rarely configured to have much spare power, so to keep up as much as possible with the pace of traffic they must deploy the full capabilities of their engines.

At the other extreme, drivers of heavy trucks do not always attempt to keep pace with other vehicles. Observations of truck accelerations have revealed many instances where truckers in typical traffic streams crossing railroad tracks have accelerated much more slowly than routinely estimated by assuming that they normally utilize their full capabilities [Long, 2000]. Consequently, it is not safe to assume that the queue of vehicles ahead of a vehicle on a railroad track will always accelerate at the maximum rate capable by the vehicles in the queue. The leading motorists may not be aware of any urgency associated with vehicles behind them.

Since repositioning time is affected by type of vehicle, three separate relationships are developed. One pertains to roadways where all vehicles other than passenger vehicles are prohibited. The second pertains to roadways where combination trucks are prohibited, but passenger vehicles, buses and/or single-unit trucks must be considered. The third pertains to roadways where all types of vehicles must be considered.

In all three cases, all vehicles in front of the last vehicle are assumed to be passenger vehicles in determining startup delays because this results in the maximum expected startup delay. For acceleration, one vehicle of the same vehicle type as the last vehicle is assumed to be at the front of the queue. The impacts of turning vehicles in a queue will be considered later.

Combination Trucks

The *Uniform Vehicle Code* [UVC, 1992] is the national standard for traffic laws underlying the traffic control devices of the MUTCD [2001]. Section 11-702 of the UVC deals with requirements for certain vehicles to stop at all railroad grade crossings. The vehicles required to stop are usually school buses, gasoline trucks and vehicles transporting other hazardous materials. After stopping, "the driver of any said vehicle shall cross only in such gear of the vehicle that there will be no necessity for manually changing gears while traversing such crossing and the driver shall not manually shift gears while crossing the track or tracks" [UVC, 1992, p. 81]. Some crossings can be exempted from this requirement, such as where the track has been abandoned or where trains travel at slow speeds and might yield for highway traffic near railroad yards. The roadways at some crossings might prohibit use of any vehicles of this nature. However, at all crossings it is always legal to stop, so it is always necessary in design to consider vehicles starting from rest. In some states the gear-shifting restriction has been extended and applies to all trucks with manual transmissions. Truck-driving schools often teach their students not to change gears over a railroad track. Authorities on truck operating characteristics state, "truck drivers are not supposed to shift gears while crossing the tracks" [NCHRP Synthesis 241, 1997, p. 29]. The general case requires presuming that truckers will not change gears over a railroad track.

FHWA's grade crossing handbook [FHWA Handbook, 1986], the Traffic Control Devices Handbook [TCDH, 1983], the AASHTO Green Books [AASHTO, 1985, 1990, 1994], and other sources all contain the same methodology for calculating minimum required sight distances at passive railroad crossings involving trucks that accelerate from rest across the tracks. This methodology involves assuming that trucks accelerate in starting gear at 1 mph/sec to a maximum speed of 6 mph, and continue at that speed until clear of the track(s) without changing gears. The sources of these assumptions about acceleration and maximum speed in starting gear are not cited. However, there has long been a perception that truckers usually maximize their acceleration since their diesel engines are built to operate steadily at full-governed speed [Gillespie, 1986]. Maximum vehicle starting accelerations for WB-15 trucks were found to range from 1 mph/sec for 400 lb/hp rigs to 2 mph/sec for 100 lb/hp rigs on level surfaces [Long, 2000], so most WB-15 trucks should be capable of 1 mph/sec starting acceleration.

Vehicle-Maximum or Driver-Preferred Acceleration

There are two issues associated with acceleration. The first is the maximum accelerating capability of a vehicle. This is easily tested and measured. It is found to be linearly-decreasing with increasing speed [Long, 2000]. Most drivers recognize that they have less acceleration capability when passing than they have when starting from rest. At the maximum speed of a vehicle, there is no acceleration capability at all or it could go faster.

The other issue associated with acceleration is the rate chosen by motorists. This is normally much less than the maximum capabilities of vehicles. It varies by type of vehicle. Similar to the maximum acceleration capabilities of vehicles, most drivers do not accelerate at a constant rate, but decrease acceleration linearly as speed increases [Long, 2000]. Normal motorist accelerations are much more difficult to measure without influencing drivers than vehicle-maximum accelerations, and there are relatively little data available. Moreover, data on acceleration variability are even harder to find for use in determining an expected acceleration that would be accepted by a high proportion of drivers. Fortunately, since each motorist in a platoon is restrained by the acceleration of the motorist ahead, accelerations tend to exhibit limited variability. Initial accelerations from rest tend to be about the same for motorists driving all vehicles except heavy trucks or other vehicles with insufficient power to keep up with the traffic stream. The decrease in acceleration as speed increases normally tapers linearly to zero at the desired travel speed.

For truck accelerations to a speed of only 6 mph, using a constant rate of acceleration is probably an adequate approximation. There is not likely to be any detectable drop-off in acceleration over a speed range of only 6 mph due to either vehicle capability limitations or driver preferences. More serious are the issues of whether 99.9% of truckers choose to accelerate at the1 mph/sec or faster capability of 400 lb/hp trucks and what maximum speed is attained by 99.9% of truckers in starting gear when crossing railroad tracks.

Long [2000] found the average starting acceleration that was chosen by truck drivers in data collected at 3 sites by Gillespie [1986] was 0.8 mph/sec. If 50% of trucks accelerate at rates below this average, the 1 mph/sec assumed for railroad crossings would not accommodate even half of all combination trucks. Most of the combination trucks, including below-average accelerating trucks, began accelerating from rest at 0.34 mph/sec or faster, and gradually reduced their acceleration rates as speeds increased [Long, 2000]. About 3% of observations accelerated even more slowly, so this only corresponds to a confidence level of about 97%.

A prudent motorist at the tail-end of a queue would likely recognize the possibility of being in jeopardy of being hit by a train, especially if the flashing lights were activated. The motorist would surely apply the maximum acceleration that his/her vehicle was capable of providing if sufficient space ahead was cleared. Thus, the 1 mph/sec acceleration would be reasonable for combination trucks at the end of a queue. However, if a combination truck can be located at the tail of a queue, then it would also be possible for another combination truck to be at the head of a queue. The vehicle at the head of a queue might not be aware of what is happening at the tail, and might begin accelerating at a normal, more leisurely rate. No other vehicle in the queue could accelerate any faster, including the last vehicle. Consequently, the maximum acceleration of the last vehicle in a queue must be considered as being constrained by the acceleration of the slowest vehicle anywhere in the queue. This does not affect the startup delay associated with how long until the last vehicle can start to move, only the rate at which it can move once it starts. Consequently, a starting acceleration of 0.34 mph/sec should accommodate an estimated 97% of queues that contain combination trucks.

While a truck going over the tracks may not be allowed to change gears, other trucks in a queue are not prohibited from shifting gears. The trailing vehicle is therefore constrained by the normal, or below-average, driver-chosen acceleration of all other trucks ahead in the queue. It is also constrained by its own maximum vehicle-capable acceleration and speed in starting gear. This means accelerating at 0.34 mph/sec to the maximum speed of the starting gear, and continuing at that speed.

Maximum Truck Speed on Flat Pavement in Starting Gear

The maximum truck speed in starting gear depends to a large extent upon which gear is selected for starting, and its gear ratio. The UVC does not dictate that the lowest gear must be selected for starting. Unloaded trucks may start in higher gears. The lowest gear may be too low for most highway conditions even with fully-loaded trucks.

Although there are multitudes of configurations of trucks being marketed [NCHRP Synthesis 241, 1997], most trucks are expected to have a starting gear ratio around 7.5 to 8.0 [Gillespie, 1986] or equivalent. Some trucks may also have deep reduction gears, such as 12-to-1 or 15-to-1 gear ratios, for loading and unloading maneuvers in unpaved or poorly-graded areas, but such gears are too low for normal starting except on steep grades, and may only be able to reach a truck speed of 4 mph [Gillespie, 1986]. To accelerate a truck in starting gear from rest to a drive-train speed matching engine idling speed in 1 sec of clutch slipping time, Gillespie [1986] determined a gear ratio of 7.5 or higher is needed on a flat grade. For a maximum speed of 60 mph in direct drive hauling a gross weight of 80,000 lbs, the maximum speed in starting gear for a gear ratio of 7.5 would be 8 mph. For a starting gear ratio of 8.0, the maximum speed on a flat grade would be 7.5 mph. These are typical starting-gear maximum speeds for trucks with slightly below-average power/weight ratios, and having these specific gear ratios. Trucks of similar characteristics with different gear ratios would have different maximum speeds.

The calculated maximum speeds are slightly faster than the maximum speed of 6 mph assumed in the design manuals. Presumably the design manuals intend to accommodate a design vehicle which is below-average. While ITE [1992b] has accepted 8 mph as the maximum speed in starting gear, it is difficult to justify applying this speed without the traditional design sources adopting it. The proportion of trucks that are capable of traveling at 8 mph or faster in starting gear is not known, although it may be increasing. Consequently, a maximum truck speed in starting gear of 6 mph is assumed, as normal. It is expected that most trucks should be able to travel over a track at this speed or faster.

Maximum Truck Speed on Grades in Starting Gear

Trucks starting on grades at railroad crossings expend some of their power going uphill, which reduces the power available for use in attaining their maximum starting-gear speed, and reduces this speed. Gillespie's [1986] analysis indicates that grades of 4%, 8% and 12% reduce the maximum starting-gear speeds to 80%, 65% and 55%, respectively, of the flat-grade speed. Consequently, maximum starting-gear speeds for a flat-grade speed of 6 mph for 4%, 8% and 12% grades are 4.8 mph, 3.9 mph and 3.3 mph.

Repositioning Time by Vehicle Type

The type of an endangered vehicle, i.e., the last vehicle in a queue to be cleared from a railroad crossing, is defined by the design vehicle. In §8A.01(4), the MUTCD [2001] specifies that the design vehicle at a railroad grade crossing is to be "the longest vehicle permitted by statute of the road authority (State or other) on that roadway". After the design vehicle is selected, consistent with §8A.01(4), the design length of the design vehicle needs to be determined. Table 2 identifies the AASHTO design vehicles and indicates design lengths [AASHTO, 1994].

While AASHTO has numerous design vehicles, insufficient detailed data are available on motorist acceleration characteristics associated with many of them. Consequently, the AASHTO Green Books provide acceleration data for design for only P, SU and WB-15 vehicles [AASHTO, 1984, 1990, 1994]. The acceleration characteristics of sport utility vehicles, vans and pickup trucks are all considered to be similar to passenger cars or P vehicles. Buses, motor homes and cars pulling trailers are all considered to have acceleration characteristics which are typical of single-unit trucks or SU vehicles. The WB-15 vehicle is considered representative of all combination trucks. Hopefully, trucks larger than the WB-15 also have more powerful engines.

At an average acceleration from rest of 0.34 mph/sec, maximum truck speeds of 6, 4.8, 3.9 and 3.3 mph require acceleration times of 17.6, 14.1, 11.4 and 9.7 sec while accelerating over distances of 77.4, 49.7, 32.5 and 23.4 ft. Continuing at constant speed after acceleration produces the time-distance relationships shown in Figure 5 for WB-15 combination trucks on different grades. At humped railroad crossings, a truck may only be on a grade while starting from rest, and may travel more than half of the minimum track clearance distance on level or different grades.

TABLE 2 Design Characteristics of AASHTO Design Vehicles

Design Vehicle	AASHTO Symbol	Design Length	Acceleration Category
Passenger car	P	5.8 m (19 ft)	P
Single-unit truck	SU	9.1 m (30 ft)	SU
Motor home	MH	9.1 m (30 ft)	SU
Single-unit bus	BUS	12.1 m (40 ft)	SU
Car and boat trailer	P/B	12.8 m (42 ft)	SU
Car and camper trailer	P/T	14.9 m (49 ft)	SU
Intermediate semitrailer	WB-12	15.2 m (50 ft)	WB-15
Motor home and boat trailer	MH/B	16.1 m (53 ft)	WB-15
Large semitrailer	WB-15	16.7 m (55 ft)	WB-15
Articulated bus	A-BUS	18.3 m (60 ft)	WB-15
Tandem trailer	WB-18	19.9 m (65 ft)	WB-15
Semitrailer (48 ft trailer)	WB-19	21.0 m (69 ft)	WB-15
Semitrailer (53 ft trailer)	WB-20	22.5 m (74 ft)	WB-15
Triple trailer	WB-29	31.0 m (102 ft)	WB-15
Turnpike double-semitrailer	WB-35	35.9 m (118 ft)	WB-15

The effects of grades are not considered for vehicles other than combination trucks. Most of the other vehicles do not utilize their full power in starting and have sufficient reserve power to overcome all but perhaps the steepest grades, which are infrequent in Florida. Furthermore, most vehicles other than combination trucks are not required to refrain from shifting gears or have automatic transmissions, so restrictions on manual shifting do not apply.

For design purposes, AASHTO seems to have adopted operating characteristics which are about 10% below average [Long, 2000]. The problem is that it is not known what proportion of motorists operate either above or below characteristics which are 10% below average, so it is difficult to interpret what proportion of motorists are likely to be accommodated.

For left-turning and non-turning passenger vehicles, Bonneson [1990] measured accelerations and corresponding speeds from queues starting at intersections in Florida and Texas. His data for average accelerations related to speeds of non-turning passenger vehicles at at-grade intersections are shown around the line marked “Average” in Figure 6. Each data point represents the average acceleration of samples at each speed. Data for 1,449 vehicle samples are

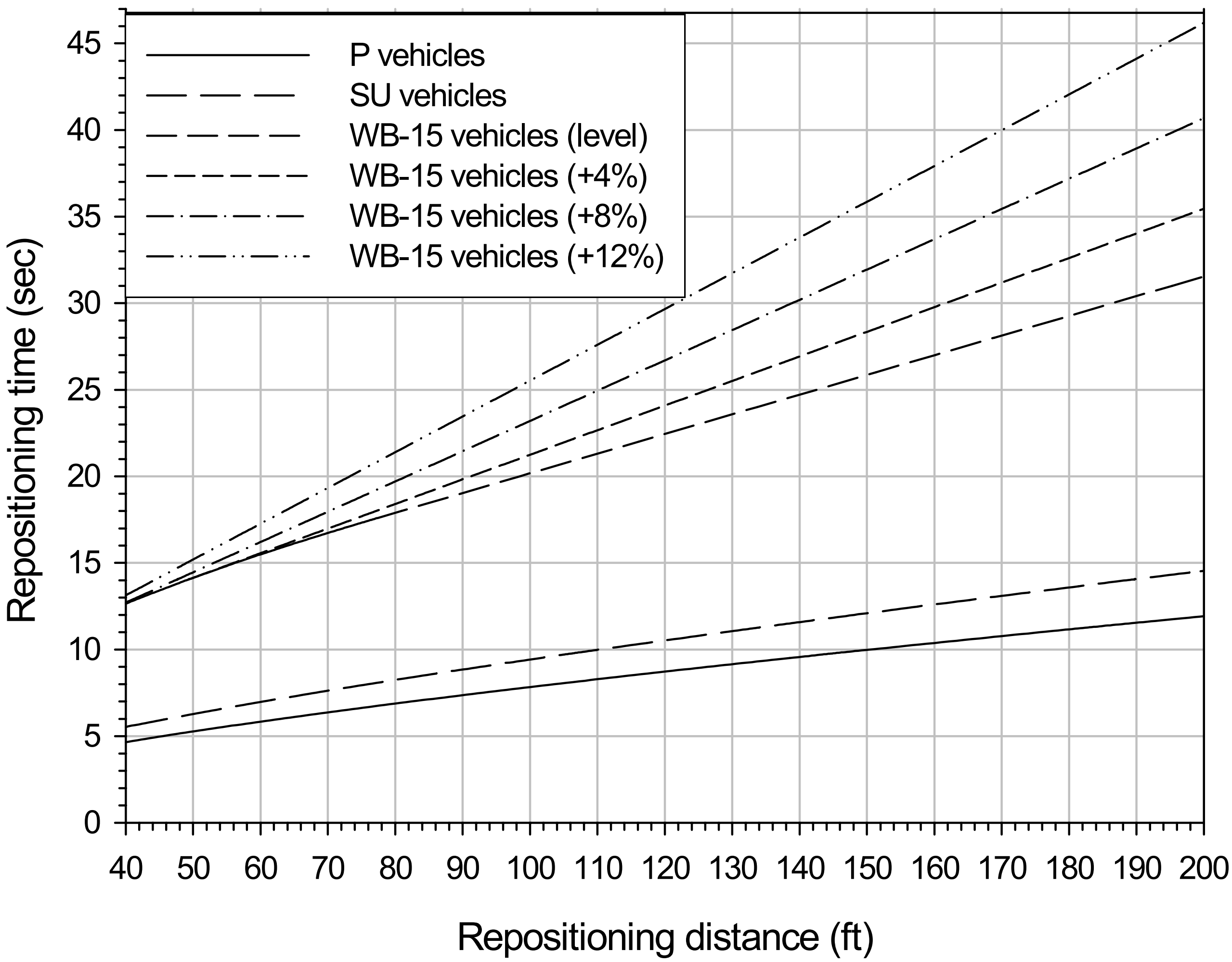

FIGURE 5 Repositioning time by vehicle type.

shown for Florida and 2,172 are shown for Texas. The number of samples averaged in each data point varies from 341 to 15. The larger samples are at slower speeds because shorter queues are more frequent. For there to be sufficient distance to accelerate from rest to speeds near 30 mph at the stop-line, the last vehicles in a queue must be at the end of rather long queues.

The data points around the line in Figure 6 marked “95% minimum” are calculated from the standard deviations reported by Bonneson [1990] related to the minimum accelerations that are expected to be exceeded by 95% of motorists. Likewise, 99% of motorists are expected to exceed the minimum acceleration line marked “99% minimum”, and 99.9% are expected to exceed the “99.9% minimum” line. It should be noted that although several points lie slightly below each line, these are calculated from standard deviations and do not represent actual observations of accelerations slower than the limit represented by the line. Although there is some variation in the standard deviations for observations at different speeds, the four lines in Figure 6, all of which were fit by linear regression, are remarkably parallel, suggesting that the variance in acceleration is virtually constant as speed changes. Furthermore, the difference between observations from Florida and observations from Texas are insignificant, so both data sets were pooled.

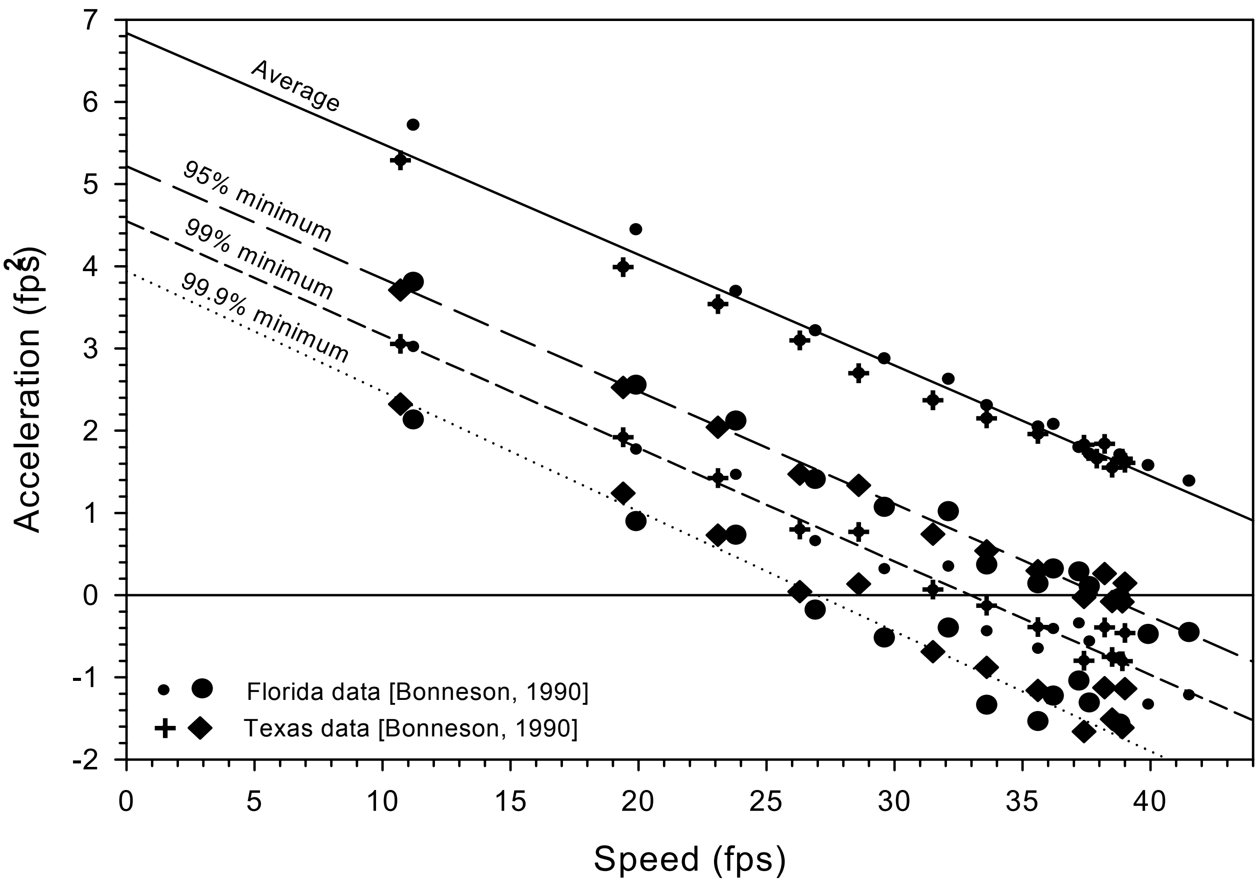

FIGURE 6 Observed non-turning passenger-vehicle accelerations.

The data in Figure 6 show linearly-decreasing accelerations with increasing speed. This relationship has also been found in other sources of acceleration observations [Long, 2000]. The intercept of each line corresponds to the initial acceleration of passenger vehicles starting from rest. The average initial acceleration was found to be 6.8 fps^2, with 95% of passenger vehicles exceeding 5.2 fps^2, 99% exceeding 4.5 fps^2, and 99.9% exceeding 3.9 fps^2. The coefficient of variation for the estimated initial acceleration is 0.143 for the pooled observations, indicating relatively tight scatter about the mean. The rate of acceleration reduction with increasing speed was -0.135 per second.

Estimating design acceleration as 10% below the average acceleration would result in a value of 6.1 fps^2. This corresponds to failing to accommodate nearly 25% of motorists, which is too high of a failure rate for use in design.

The time-distance relationships for linearly-decreasing acceleration with increasing speed are shown in Figure 5 for P vehicles, as were determined from the following equation [Long, 2000]

$$d = ((\alpha \pm Gg) / \beta)\, t - ((\alpha \pm Gg) / \beta - v_0)(1 - e^{-\beta t}) / \beta \qquad (8)$$

where

d = repositioning distance (ft),

t = repositioning time (sec),

v_0 = 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}).

For P vehicles, values of $\alpha = 4.5$ and $\beta = 0.135$ were used to represent expected minimum starting accelerations which should be exceeded by 99% of passenger vehicles.

The SU vehicles may be the most important category for design, as well as the category for which acceleration information is the scarcest. They may be the most important category because they are seldom prohibited, so restrictions allowing only passenger vehicles are unlikely. On roads where heavy trucks are prohibited, the SU vehicle usually becomes the design vehicle. Information for SU vehicles is the scarcest since many studies ignore queues with SU vehicles because they are relatively infrequent. These studies have focused on queues of purely passenger vehicles. Unlike heavy trucks, SU vehicles probably rarely accelerate at their maximum rates. Furthermore, since the SU category represents buses, motor homes, cars pulling trailers, as well as single unit trucks, there may be a more diverse mix of operating characteristics than either for passenger cars or heavy trucks. Consequently, the variances in accelerations may be larger than for passenger vehicles or heavy trucks.

Accelerations are not easy to measure with fidelity. Multiple short measurement traps along a roadway are needed because accelerations are not constant and change quickly. Sensitive, high-speed equipment is needed to measure distances and travel times accurately over short traps. The positions of measurement traps may need to be flexible since the lead vehicle of each queue does not stop close to the same location from the stop-line [Long, 2000]. Camera parallax makes it difficult to obtain sufficient accuracy by simple video-photography. Human reaction time is too slow and erratic for manual measurements. Chase-car measurements suffer from difficulties in maintaining exactly constant distances behind sample vehicles, especially since normal inter-vehicle spacings change with speed. Probably the best measurement method was applied by Bonneson [1990] using special measuring instrumentation, but he studied only queues having no trucks or SU vehicles.

From the limited information available on SU vehicle accelerations, a value of 5.0 fps^2 seems to be the best estimate for average initial acceleration [Long, 2000]. If it is assumed that the coefficient of variation might be similar to the 0.143 value found from passenger vehicle observations, then 95% of SU trucks would be expected to exceed 3.8 fps^2, 99% would exceed 3.3 fps^2, and 99.9% would exceed 2.8 fps^2. Therefore, values of $\alpha = 3.3$ and $\beta = 0.135$ were used to represent expected minimum starting accelerations which should be exceeded by 99% of SU vehicles, as plotted in Figure 5.

While an endangered vehicle at the tail of a queue would likely accelerate aggressively if warned of an approaching train, it cannot accelerate faster than the vehicle ahead of it, which cannot accelerate faster than the vehicle ahead of it, etc. It is presumed that a motorist in the front or elsewhere in a queue does not perceive any urgency that might exist at the tail, and will accelerate in a routine below-average rate.

If preemption time exceeds the 20 sec of normal train warning time, resulting in a second train detection circuit being used for traffic signal preemption, an endangered vehicle may have no clue that it is endangered at the beginning of preemption since the train warning signals could be delayed until 20 sec before train arrival. Consequently, even an endangered vehicle might accelerate after the onset of preemption at a routine, leisurely rate.

Combined Estimates

For a given type of design vehicle (and given grade for WB-15 vehicles), the startup delay and repositioning time can be combined from Figure 3 and Figure 5 to give the rudimentary clearance time before special adjustments are applied. This is presented in Figure 7 for SU vehicles and in Figure 8 for WB-15 vehicles on level pavement. These charts can be handy if special adjustments are unnecessary since the clearance time can be determined directly. Furthermore, the relative impacts of both queue length and repositioning distance can be assessed in a glance.

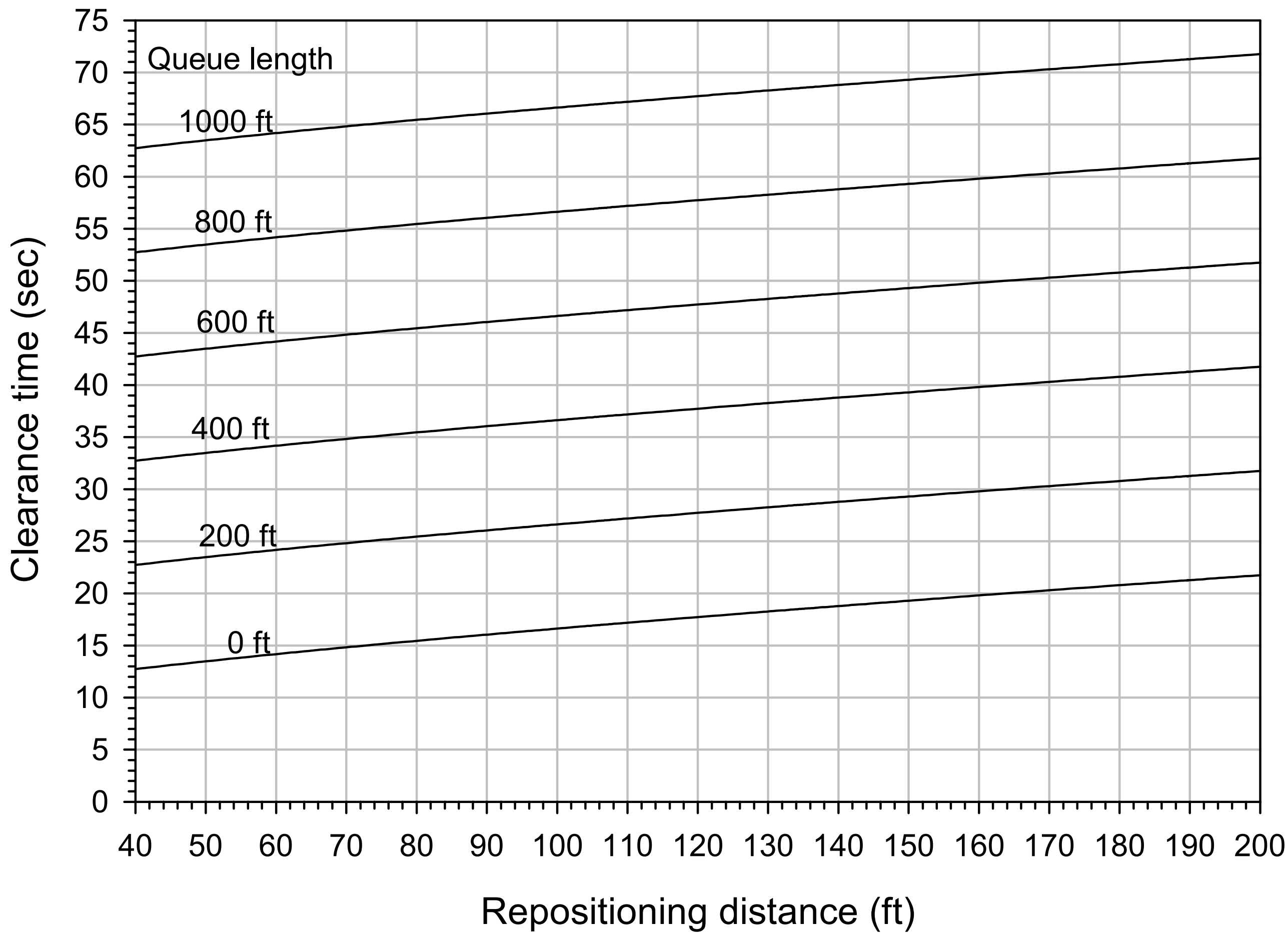

FIGURE 7 SU expected maximum clearance time.

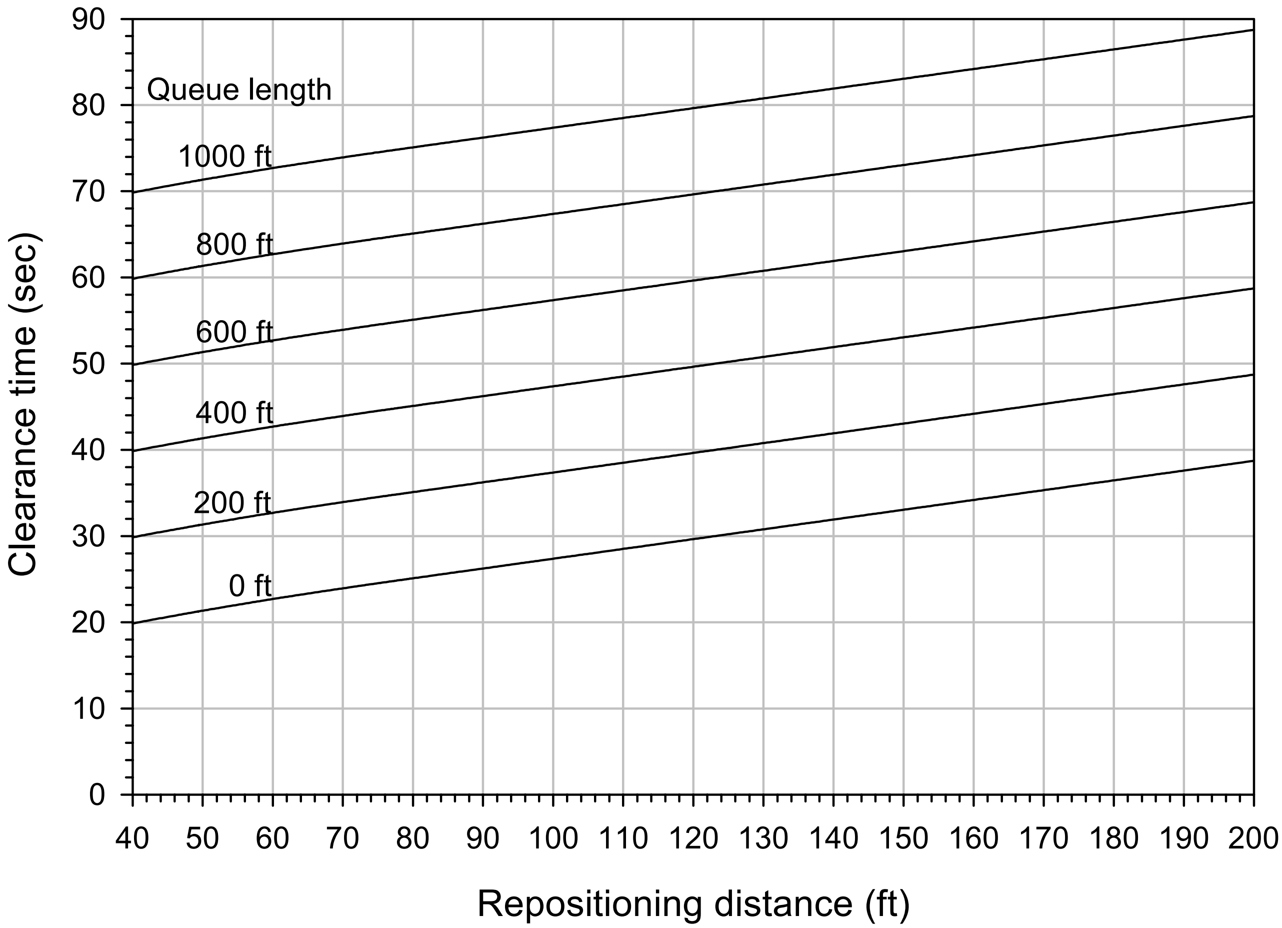

FIGURE 8 WB-15 expected maximum clearance time on level surface.

Special Considerations

There are several situations which may require special attention. These are situations which might be expected to require adjustments to the clearing times estimated from the startup-delay and repositioning-time models.

Intersection Turning Movements

As discussed earlier, the queue spacings and startup delays of traffic in turning lanes were not different from through-lanes. Data in the literature show the same initial acceleration for left-turning traffic as for through-traffic [Bonneson, 1990]. The accelerations of left-turning vehicles were found to diminish a little faster than in through-lanes, probably due to left-turning motorists reaching lower desired speeds sooner in conjunction with making a turn. Consequently, no separate acceleration rates are needed for protected left-turning flows.

Right-turning flows can be different. The radius of turn is usually much tighter for right turns, which can cause large vehicles to proceed more slowly. Combination trucks sometimes come to a halt when turning right in narrow lanes with short corner radii, and/or spill out of their lanes. Off-tracking sometimes causes truck rear wheels to climb over curbs. These situations might not affect startup delays much, but they can affect the maximum acceleration and maximum speed of a queue. To try to study this situation as part of this project, Blackadar [1998] tried to simulate intersection operations with different mixes of vehicles using CORSIM. These results were not very fruitful. It was discovered that CORSIM applies a fixed 3 ft spacing between all vehicles in all situations. Conversely, a study measuring inter-vehicular spacings found the average to be 12 ft and the variations to be large [Long, 2002]. In order to gain a more realistic representation of observed gaps using CORSIM, extra footage was added to vehicle lengths. This caused the turning vehicles to be too long to turn realistically. A revision request was recently submitted to FHWA by the TSIS contractor to revise these CORSIM features.

Right-turn bays are common throughout the U.S. for the exclusive use of right-turning traffic. Traffic flows in these lanes are generally observed to be similar to through lanes. Except for large trucks at sharp corners in tight areas, differences between traffic movements in right-turn lanes and through-lanes have not been observed to be significant. Tight right-turn geometry should simply be eliminated at preempted intersections. Widening the cross-street in the vicinity of the intersection and increasing the corner radius can eliminate this problem at much less of a cost to travelers than increasing preemption time. Consequently, no clearance time extensions are adopted for left-turning or right-turning intersection traffic relative to through traffic.

Intercepting Driveways

As longer distances between a traffic signal and railroad track are considered for preemption, it becomes more likely that there will be land that can accommodate some type of development lying between the railroad track and parallel roadway. This can lead to driveways intercepting the queue storage area between the traffic signal and railroad crossing, and traffic entering and departing on these driveways. It was suspected that startup delays could increase when queued motorists waited to allow driveway vehicles to move into the traffic stream ahead of them, and accelerations could be reduced for vehicles following a vehicle slowing to turn into a driveway.

McCoy and Heimann [1990] studied the effects of driveways near traffic signals on intersection saturation flow rates and found that vehicles turning into driveways decreased saturation flow rates at the signal due to a significant increase in discharge headways. With one vehicle leaving the roadway, headway times for the trailing vehicle increased by an average of 1.93 sec at a location 105 ft from a signal, and 1.14 sec at another location 160 ft from a signal. For queues near railroad tracks, the concern is not on saturation flow rates at the signal but what affect a turning vehicle has on the times when the vehicles behind it reached the stop-line. Discharge headways are measured between successive vehicles, so after a turning vehicle escapes from a queue, two headways became merged into one. Although the headway between the vehicle ahead and vehicle behind the turning vehicle was found to be larger than one normal saturated headway at the signal, unless it was larger than two normal headways, the trailing vehicle was not retarded. If the turning vehicle had remained in the queue with a normal saturation headway of 1.9 sec [HCM, 2000], the trailing vehicle would have crossed the stop-line at about the same time if the turning vehicle had not turned at the 105 ft location, and about three-quarters of a second later at the 160 ft location. Consequently, the trailing vehicle was not retarded by the turning vehicle, and actually arrived at the stop-line sooner than if the vehicle had not turned into the driveway at the farther location. Hence, on average, no additional clearance time is needed for roadway vehicles making right turns into driveways.

After a turning vehicle vacates the roadway, the vehicles following it apparently accelerate a little before reaching the intersection stop-line to fill in the extra gap in the queue left by the turning vehicle. Reductions in acceleration for vehicles following vehicles slowing to turn into a driveway appear to be offset by increases in acceleration after the turning vehicle has left the queue. While the average effect may be roughly offsetting, McCoy and Heimann found the standard deviations of the headway increases to be 1.28 and 1.37, respectively. These indicate a lot of variability in the impacts from turning vehicles, and suggest that a portion of the occurrences of traffic turning into driveways may retard normal queue dispersion. Moreover, the fact that a trailing vehicle can accelerate a little more to fill the extra gap towards the traffic signal is not important. The key issue is what happens to the vehicles behind the trailing vehicle. If the trailing vehicle must slow for the turning vehicle, then all vehicles behind must slow, too. Since a turning vehicle must be in motion to make a turn, the impact on each trailing vehicle would likely be on its acceleration, not its startup time. Reducing to a slower speed requires less of an inter-vehicle gap between the vehicle ahead (as noted in the old car-following rule of allowing one car-length for each 10 mph of speed), so there may be a buffering effect where each trailing vehicle slows, but not quite as much as the vehicle ahead while reducing its inter-vehicle gap, until the shock-wave dissipates after a few vehicles and the remainder of the queue returns to normal dissipation. Unless a driveway is located close to a railroad track, the impacts on trailing vehicles of vehicles turning into driveways may be insignificant. For a driveway that is close to a railroad track, there is not much opportunity for queued vehicles to turn into it since all vehicles ahead in a queue will have passed it. Consequently, no additional clearance time is added for variations in delay from roadway vehicles making right turns into driveways.

The effects of one or more vehicles turning right into a driveway on the vehicles at the end of a queue should be no worse than an entire queue turning right at an intersection. This is regularly observed in right-turn lanes, and the queues generally seem to move similarly to through-lanes. Without available data indicating otherwise, the impacts on queue dispersion of vehicles turning into driveways are considered negligible. It is presumed that a large corner radius of 20 ft or more could be required for a driveway in this sensitive area and sufficient driveway width of 35 ft or more for free movement of inbound and outbound driveway traffic would also be required. Stover and Koepke [1988] indicate an average driveway entry speed of about 12 mph for these dimensions.

With one vehicle emerging from a driveway, McCoy and Heimann found an increase in headway time by an average of 1.05 sec for the driveway closest to a traffic signal, but no significant effect from traffic turning out of the driveway located a little further away. Again, headways relate to successive vehicles at the stop-line. A vehicle emerging from a driveway into a queue creates a separate headway time at the stop-line. Yet, for queue dispersion, the concern is how the vehicle trailing the driveway vehicle is affected. By a turning vehicle entering a queue with a normal saturation headway of 1.9 sec, and an additional observed increase of 1.05 sec, the trailing vehicle was delayed by an average of 2.95 sec for the driveway located 105 ft from the stop-line and 1.9 sec for the driveway 160 ft away. McCoy and Heimann found the standard deviations of the delays to be 1.64 and 1.22 respectively, so the extra delays were quite variable.

With only two data points representing distances from the stop-line, and other confounding factors such as differences in curb radii, it is difficult to find a relationship between delay impacts and driveway distance from the stop-line. Moreover, it is the distance to the railroad track, not the stop-line, and the delay imposed on the vehicles back towards the railroad track, that are important. It appears that the delay may have been the largest at the driveway closest to the intersection because the trailing vehicle had less distance over which to accelerate to compensate for the delay. The vehicles behind the trailing vehicle may not be able to recover as much, and they are the most important in railroad clearance time. Therefore, the delay measured for the driveway closest to the intersection is considered to be the best estimate of the delay incurred by the trailing vehicle at its initial position back behind the driveway, the position where the extra delay was imposed. The extra startup delay inflicted by one vehicle, emerging by turning right from a driveway, on each vehicle in the portion of a queue behind a driveway is estimated to average 3 sec with a standard deviation of 1.6 sec. At a 99.9% level of confidence, the variations in delay for each emerging driveway passenger vehicle during preemption are estimated to be compensated by adding 8 sec (3 + 3.090 * 1.6 = 7.94 sec) of additional queue clearance time.

All of McCoy and Heimann's data pertain to queues containing only passenger cars that made no turns at the signalized intersections, and driveway vehicles which also were passenger cars. No information is available pertaining to heavy vehicles. Furthermore, all data seem to pertain to single automobiles entering or emerging from driveways. While two or more vehicles might emerge from a driveway together, after a queue begins to move it is rare that it would stop to allow more driveway vehicles out. One or multiple vehicles can always turn into a driveway, and this can provide a sufficient break in the flow for a driveway vehicle to emerge, but it might not impose much additional delay on trailing vehicles due to the added gap.

As part of this study, Cato [1998] investigated the effects of driveway traffic on queue dispersion at six sites in Daytona, Gainesville and Jacksonville. One of the first observations was that motorists stopping in a queue on a roadway near commercial driveways often leave a large gap opposite a driveway for a driveway vehicle to use whether any driveway vehicles were present or not when the vehicle stopped. This did not have much effect on startup times, but does extend a queue back farther. Another early observation was that many queues form with the leading vehicle well over the stop-line which significantly affects the discharge headways of the first few vehicles. This is the reason presented earlier for needing to consider lead vehicle offsets from the stop-line.

Cato found delays frequently imposed on the startup of the lead vehicle by opposing left-turning stragglers which kept flowing even though their signal had turned red. These were not measured explicitly, but were estimated to be mostly less than 4 sec. This would not be a problem if preemption was triggered during a different phase, but could be significant if preemption is triggered during a leading left-turn phase when left-turning traffic is not expecting their normal green time to be truncated. A solution might be to use lagging turn phases. Of course, lagging left-turns on the cross-street could pose the same problem. There were also occasional blockages by cross-street traffic spilling across the intersection, and by emergency vehicles which disrupted normal traffic flows. These conditions cannot be predicted and were excluded.

Cato investigated the impacts of queue compositions by separating queues with only passenger cars, queues with all passenger cars except one single-unit truck, and queues with all passenger cars except one combination truck. No significant differences were found in the times for the vehicle in each queue position to reach the stop-line for the three different types of queues. This comparison was also performed when interference from driveway vehicles occurred and no significant differences were found with or without the different trucks in the queues. Significant differences also were not found in conjunction with where the trucks happened to be positioned in the queues. It was concluded that adjustment factors were not needed for variations in vehicle mixes. Since the time to reach the intersection stop-line from each vehicle position in a queue involves both startup delay and acceleration, no adjustment factors for vehicle mixes for either component were found to be needed.

One of Cato's sites had a channelized right-turn lane at the intersection. No significant differences were found in the arrival times at the stop-lines for the vehicles in each queue position in the right-turn lane versus the through lane. It was concluded that the same flow properties pertaining to through traffic streams apply to right-turning traffic streams with a sufficiently large corner radius.

Since earlier work had not investigated the delays inflicted by multiple passenger vehicles turning into or out of a driveway consecutively, Cato examined movements involving two vehicles turning right into a driveway together and movements involving two vehicles and three vehicles turning right out of a driveway together, in addition to single vehicles turning into and out of driveways. The excess times to travel to the stop-line by the following vehicle ranged from 1.8 to 5.8 extra seconds per additional driveway vehicle, but the impacts of most interest were the affects on vehicles towards a railroad track, which is a different matter.

The samples of driveway vehicles contained mostly P vehicles, so it can only be conjectured that combination trucks going in and out of driveways might increase the incremental delay to track-clearance time even longer. The amount of additional time is unknown.

It is recommended that a barrier median be installed to prevent left-turns into and out of intervening driveways. This will cause some added delay and inconvenience to some of the driveway traffic, but left-turning driveway traffic is incompatible with traffic signal preemption and awkward at other times when queues are present. Obviously, there may be conflicting considerations which prevent this, such as treatments for emergency vehicles. Similarly, parking and bus stops are incompatible with preemption and should be prohibited or avoided.

Lane widths should be a minimum of 12 ft to avoid unnecessary reductions to saturated flow rates. The addition of extra lanes by intersection widening should be considered where possible to reduce the likelihood of queues extending back onto railroad tracks. This can be more cost-effective than signal preemption.

Empty Vehicle-Storage Space

It has been suggested that considering queues that fill the clear storage distance plus the minimum track clearance distance might be excessive because there might be no vehicle actually over the track. If a vehicle such as Vehicle A in Figure 1 stopped too close to the track, but not over the track, and no vehicle was over the track immediately ahead of it, then Vehicle A could creep forward while waiting for the vehicle further ahead to move forward, allowing it to start full acceleration at an earlier time, perhaps while already moving. However, it is unlikely that

Vehicle A would creep forward to a position over the track, if it initially stopped somewhere before reaching the track when there was some unoccupied space ahead, until the vehicle ahead moved far enough for there to be sufficient clear space past the track to store the whole length of Vehicle A. If Vehicle A was a combination truck waiting for the vehicle ahead not only to start, but also to move far enough forward to create clear storage for Vehicle A, this could increase the startup delay time of Vehicle A. The saving associated with not having to start a vehicle immediately ahead of Vehicle A might result in a startup time saving on some occasions, but an increase on most other occasions.

The lengthier clearance time could be determined by estimating the startup delay for the shorter queue using Figure 3. The required repositioning distance for the end of the queue to create clear storage space before starting across could be estimated as an inter-vehicle gap of 12 ft plus the length of the design vehicle from Table 2. For example, this would be 31 ft for a P vehicle, 42 ft for an SU truck and 86 ft for a WB-20 combination truck. The end of queue repositioning time to clear the required storage space before the endangered vehicle started across can be obtained from Figure 5 and the required repositioning distance of the end of the queue. This would involve an expected maximum of 3 sec for a queue containing only P vehicles, 6 sec for a queue with SU vehicles and 28 sec for a queue containing WB-20 vehicles. Then the normal repositioning time should be added as usual.

For a typical single-track crossing at a right angle having a minimum track clearance distance of 26 ft, a shorter queue by 26 ft would reduce the starting delay, using Equation 7, by 1.3 sec, so the net effect would add 1.7 sec to track clearance time for roadways designed for only P vehicles, 4.7 sec for SU design vehicles and 26.7 sec for roadways with WB-20 design vehicles. While these adjustments are tolerable for P and SU vehicles, they begin to become prohibitive for WB-20 vehicles.

Rather than be inconsistent by building this methodology into the charts for some design vehicles but not others, the adjustments have been treated as discretionary. The adjustment is small for passenger vehicles, so it is not devastating if omitted. However, it should be recognized that this scenario reflects what motorists are supposedly trained about crossing railroad tracks – do not proceed across until ample storage space is fully available on the far side. It must be expected that some motorists will abide by this. However, the issue is whether the same motorists who would put themselves in jeopardy by stopping beyond a gate or too close to a track would also defer proceeding until full storage space became available.

Joint Probabilities

Not only is it important to find the total time expected to be needed to clear most queues from being endangered, it is also important to find the likelihood of this amount of time being sufficient. There is a probability that a specific value of each component is exceeded. These probabilities can be combined to estimate the overall likelihood of a crash due to insufficient clearance time. Some of these probabilities are easier to determine than others. Several of the variables affecting clearance time have already been combined, such as queue offset from the stop-line, lengths of vehicles and space between vehicles in the queue. Several other variables must also be considered.

Insufficient data are available to resolve this issue, but it is important to begin to identify the probabilities that can be germane. The following list is only for illustration:

Probability of a train arriving when a vehicle is in jeopardy stopped over the track
Probability of belated opposing left-turns imposing extra startup delay over t sec
Probability of critical traffic flow rate being no greater than v vehicles per hour per lane
Probability of number of vehicles queued being no more than n vehicles per lane
Probability of number of passenger vehicles in the vehicle mix being no more than n vehicles
Probability of number of combination trucks in the vehicle mix being no more than n vehicles
Probability of number of other vehicles in the vehicle mix being no more than n vehicles
Probability of a queue length of n vehicles extending more than x feet from stop-line
- Probability of the lead vehicle offset from the stop-line exceeding -x feet
- Probability of the spacing between queued vehicles being more than than x feet
- Probability of the lengths of queued vehicles being longer than x feet

Probability of the excess startup time of the queue leader being over t sec
Probability of startup headways of sequential queued vehicles being over t sec
Probability of driver inattentiveness imposing extra delay over t sec
Probability of a vehicle in a queue accelerating more slowly than a_0 fps^2 imposing extra delay over t sec
Probability of vehicles exiting a queue into a driveway imposing extra delay over t sec
- Probability of one or more vehicles exiting a queue into a driveway
- Probability of exiting-delay exceeding t sec

Probability of vehicles entering a queue from a driveway imposing extra delay over t sec
- Probability of one or more vehicles entering a queue from a driveway
- Probability of entering-delay exceeding t sec

PROXIMITY CONSIDERATIONS

An important issue is where to provide traffic signal preemption. The 1948 MUTCD indicated that traffic signal preemption should be considered where traffic signals were within 500 to 1000 feet of signalized intersections [MUTCD, 1948]. This strategy was reversed in 1961. Section 3B-25 of the 1961 MUTCD states: "Except under unusual circumstances the interconnection should be limited to the traffic signals within 200 feet of the crossing" [MUTCD, 1961, p. 174]. The exact same statement is in §4B-21 of the 1971 MUTCD [p. 230]. The apparent intent was to keep the distance between a preempted traffic signal and a railroad track short so the needed preemption time will be short and the normal train warning time of 20 sec by the track circuitry at the crossing will be adequate for clearing the tracks of any motor vehicles.

Even 200 ft may be too far to clear a track in 20 sec. Section 8B-2 of the TCDH states: "When the distance between a grade crossing and a signalized intersection is approximately 150 feet or more and traffic volumes are such that vehicle queues routinely develop onto the tracks, the typical preemption strategies may not be capable of clearing the tracks within the normal warning time provided by the train detection circuit" [TCDH, 1983, p. 8-49].

This limit of 150 ft for 20 sec of train warning time is partially supported by the findings in this study. To clear a 30-ft SU vehicle over a minimum track clearance distance of 26 ft, Figure 5 shows that 7 sec of repositioning time is needed for the total 56 ft. Figure 3 shows that the remaining 13 sec will accommodate the startup delay of a queue of about 116 ft. Subtracting the minimum track distance of 26 ft leaves a clear storage distance of 90 ft. Adding 15 ft for the stop-line setback and 6 ft from the end of the clear storage space to the rail results in 111 ft from the pavement edge to the closest rail. Therefore, 150 ft with 20 sec of train warning time may be adequate for P-vehicles, but too long for SU or heavier or longer vehicles. However, it must be recognized that queue clearance is only one component of preemption time and additional time must be provided for a yellow change interval to terminate current phases for vehicles and pedestrians, possibly some all-red clearance-extension time, equipment delay time, separation time, etc.

The question is what to do if there is more than 200 ft between the intersection and track, when queues back up onto the track. If this requires more than 20 sec of queue clearance time, a second track circuit can be installed to send an early warning to the preemption controller and the normal circuit can continue to activate the train warning devices when a train is 20 sec from the crossing. Other types of actuation devices, as discussed earlier, can also be used for the early warning. Furthermore, some of the current train detection devices can send two signals, one for preemption and another signal later for gate actuation, or pause for a delay after actuation for preemption before activating the train warning signals 20 sec before the train arrives. The key issue is that if more than 20 sec is needed for preemption, a second train warning detection mechanism is needed. While train signal activation can be extended beyond 20 sec to save money by using just one detection system for both preemption and train warnings, lengthening motorist delay beyond 20 sec encourages motorists to conclude that the system is malfunctioning and breach the warning.

The 2001 MUTCD in §8D.07 has reversed its former specification and now requires preemption at distances of less than 200 ft, and specifies that preemption should be considered at distances beyond 200 ft --

> "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." [p. 8D-10]

No limit is set as to the maximum distance for which preemption may be considered.

While §8A.01 of the MUTCD [2001] contains precise definitions for many terms, it does not provide a precise definition pertaining to the beginning and ending points of measurement between the flashing light signal system and the intersection or mid-block location specified in §8D.07. Since the specification of 200 ft is rather precise, this omission is surprising. The definition for clear storage distance is carefully defined, but not used in this specification. It seems likely that the clear storage distance might not be applicable because it references the position of the stop-line, which can be moved. The criteria for preemption should not be related to a reference point that is movable. Consequently, the reference points are considered as the closest rail to the closest edge of pavement on the crossing roadway, which is the distance that is typically measured.

The instructions above from the *Traffic Control Devices Handbook* indicate that traffic volumes must be such that vehicle queues routinely develop onto the tracks for preemption to be appropriate. This guidance seems naive. If queues occasionally extend onto a track, but it does not happen routinely, the TCDH seems to imply that preemption is not appropriate. Yet, during one of these occasions somebody can be hit and killed, which is exactly what preemption is intended to avoid. A better policy would be to consider the use of preemption if there is 1 chance in 100, or 1 chance in 1,000, that a queue could extend back onto a railroad track.

ITE [1997] recommends applying the results of Oppenlander and Oppenlander [1996] for determining expected queue lengths based on lane design-traffic volumes, signal cycle lengths, green time for the railroad approach, and the distribution of vehicle types. Critical lane volumes at signalized intersections are first obtained using the techniques in the *Highway Capacity Manual* [2000]. Oppenlander and Oppenlander assume Poisson arrivals under free-flow conditions, which ordinarily may not be applicable during peak-period traffic conditions when the longest queues are likely, nor when a nearby upstream signal releases vehicles in platoons. The same authors provide additional information for uniform arrivals [1999], exclusive left-turn lanes [1994] and permissive left-turn lanes [2002].

Oppenlander and Oppenlander's work is useful for establishing the lengths of turn bays. When queues exceed turn-bay lengths, the consequences are not severe. Traffic flows in through-lanes may be disrupted by overflowing vehicles in turn lanes backing up into through lanes at the end of a queue. Consequently, a typical level of confidence of 95% was used. At railroad crossings when queues exceed the clear storage distance, they extend across a railroad track where lives can be at stake if a train arrives. Five failures in 100 is too many to accept. A higher level of confidence should be chosen such as 99% or 99.9%.

ITE [1997] calculates the average queue length expected based on the percentages of vehicles of different types. This means that half of the time the queues should be expected to exceed this length and extend into the track area. This produces too many failures. Equations 3, 4, 5 and 6 presented here should be substituted to obtain an estimated queue length which is expected to be exceeded only 1 time in 100 or 1 time in 1,000. Furthermore, the percentages of vehicles must be rounded to yield full vehicles as discussed in Long [2002]. A queue length that accommodates 10% of a 74 ft combination truck leaves the other 90% of it to force the queue back over the crossing.

Expected Maximum Queue Lengths

Table 3 shows the expected maximum queue lengths in feet at 99.9% confidence for queues with different compositions of vehicles. Values in this table were calculated using Equations 3, 4 and 6. As can be seen in the table, queue lengths grow quickly when combination trucks and other vehicles are contained in a queue.

For purposes of analyzing whether preemption is needed, the maximum expected proportions of combination trucks and other vehicles should be considered. Using expected average percentages of trucks and other vehicles assumes that it is tolerable for queues to exceed estimates half of the time.

It must also be remembered that the total number of vehicles in a queue should represent the maximum at some given level of confidence, not just the average number of vehicles expected as often is produced by traffic estimation techniques.

It should be recognized that when using Table 3 to determine if signal preemption is needed, and then using the procedure herein to determine the minimum clearance time, an inconsistency can arise from the use of both minimum queue lengths with only passenger vehicles present and maximum queue lengths with the maximum percentages of trucks. If the expected maximum queue length is not too much longer than the distance from the signal to the track and involves some trucks, then filling the clear storage distance with automobiles to estimate the maximum startup delay may result in more vehicles than expected. Engineering judgment is needed to balance these tradeoffs, especially at long separations between signal and track where clearance times will be long.

TABLE 3 Expected Maximum Queue Lengths

	Combination Trucks															
	0				1				2				3			
Passenger	Other Vehicles				Other Vehicles				Other Vehicles				Other Vehicles			
Vehicles	0	1	2	3	0	1	2	3	0	1	2	3	0	1	2	3
0	-[a]	76	127	176	112	162	211	260	198	247	295	343	283	331	378	425
1	54	106	156	205	142	192	241	289	228	276	324	372	312	360	407	454
2	85	136	186	234	172	222	270	318	257	306	353	401	341	389	436	483
3	116	166	215	264	202	251	299	347	287	335	382	429	370	418	465	512
4	146	196	245	293	232	280	328	376	316	364	411	458	399	447	494	540
5	176	225	274	322	261	309	357	404	345	393	440	487	428	475	522	569
6	206	255	303	351	290	339	386	433	374	422	469	515	457	504	551	597
7	235	284	332	379	320	368	415	462	403	450	497	544	486	533	579	626
8	265	313	361	408	349	397	444	491	432	479	526	573	515	562	608	654
9	294	342	390	437	378	425	473	519	461	508	555	601	544	590	637	683
10	324	371	419	466	407	454	501	548	490	537	583	630	572	619	665	711
11	353	400	448	494	436	483	530	576	519	565	612	658	601	647	694	740
12	382	429	476	523	465	512	559	605	547	594	640	687	629	676	722	768
13	411	458	505	552	494	541	587	634	576	623	669	715	658	704	750	796
14	440	487	534	580	522	569	616	662	605	651	697	743	687	733	779	825
15	469	516	562	609	551	598	644	690	633	680	726	772	715	761	807	853
16	498	544	591	637	580	626	673	719	662	708	754	800	744	790	836	881
17	526	573	620	666	609	655	701	747	690	737	783	829	772	818	864	910
18	555	602	648	694	637	684	730	776	719	765	811	857	801	846	892	938
19	584	630	677	723	666	712	758	804	748	794	839	885	829	875	920	966
20	612	659	705	751	694	741	787	832	776	822	868	913	857	903	949	994
21	641	687	734	780	723	769	815	861	804	850	896	942	886	931	977	1022
22	670	716	762	808	751	797	843	889	833	879	924	970	914	960	1005	1051
23	698	744	790	836	780	826	872	917	861	907	953	998	942	988	1034	1079
24	727	773	819	865	808	854	900	946	890	935	981	1026	971	1016	1062	1107
25	755	801	847	893	837	883	928	974	918	964	1009	1055	999	1045	1090	1135
26	784	830	876	921	865	911	957	1002	946	992	1037	1083	1027	1073	1118	1163
27	812	858	904	949	894	939	985	1030	975	1020	1066	1111	1056	1101	1146	1192
28	841	886	932	978	922	968	1013	1059	1003	1048	1094	1139	1084	1129	1174	1220
29	869	915	960	1006	950	996	1041	1087	1031	1077	1122	1167	1112	1157	1203	1248
30	897	943	989	1034	979	1024	1070	1115	1059	1105	1150	1195	1140	1186	1231	1276

[a] At 99.9% level of confidence; queue lengths in feet.

TIMING NUANCES

Queue clearance times can be rather lengthy. Unfortunately, there are other delays which must also be added to queue clearance times to determine the total time needed for preemption. Some of these considerations are discussed briefly in this section for perspective. Other sources should be consulted for more complete information [ITE, 1997; AREMA, 2000; NCHRP Synthesis 271, 1999].

Some train-detection track circuitry operates by sampling the condition of a train-detection signal as a deterrent to triggering false alarms. After a train-detection signal is sensed, the circuit is sampled again after some delay to determine if the signal is still positive, perhaps multiple times, before the preemption circuit is de-energized to indicate the approach of a train. This equipment delay time might be as long as 6 sec, and must be included as part of the required preemption time. Detection equipment which responds with little delay is often preferred.

After a traffic signal controller is notified of train detection, there may be some additional equipment delay while the controller performs similar sampling of the interconnect circuit. Controller equipment which responds with little or no delay is often preferred.

If a pedestrian walk phase is in progress, MUTCD [2001] §4D.13 allows shortening or omitting any walk interval or pedestrian change interval. However, pedestrians in an intersection can block or delay the movement of a queue to clear the track. Consequently, sufficient pedestrian clearance time is needed to clear all pedestrians at a high level of confidence.

If no pedestrian phase is operating, some delay may be required to reach the minimum green time for a conflicting phase, although truncating green phases is allowed by the MUTCD.

At the end of a conflicting green phase, MUTCD §4D.13 prohibits shortening or omitting the yellow change interval or any red clearance interval that normally follows, before releasing the queue to move, so allowances for these times must be included in preemption time.

After the queue is started, the green must be held for the duration of the queue clearance time. When the rear of the design vehicle has cleared the farthest rail by 6 ft, there must be an allowance for separation time, as defined in MUTCD §8A.01(15), before the arrival of any train, which must be included in preemption time. At least 5 sec is usually allowed for separation time.

Queue Green Time

In the event that there is insufficient distance to decelerate and stop the repositioned vehicle without passing the intersection stop-line, then sufficient time must be provided to allow a repositioned vehicle to enter the intersection before transferring the green to another phase. If sufficient distance exists to decelerate and stop a repositioned vehicle before the traffic signal, this timing is preferred to avoid encouraging motorists to pull into risky positions in order to benefit from preemption.

Gate Clearance Time

Large trucks that are heavily loaded and on an upgrade to the track, as often exists, require considerable gate clearance time before the gate begins to descend. The flashing lights are required to be actuated a minimum of 3 seconds before the gates begin descent by MUTCD §8D.04 [2001]. No maximum time is specified. However, if a long time must be set to accommodate large trucks, when no truck is present, motorists are likely to conclude that their detainment time is excessive, the system must be malfunctioning, and there is plenty of time to breach the warning and cross the track. This is a dilemma. Design truck lengths are given in Table 2 and repositioning times are given in Figure 5, which can assist analysis.

Preemption Distance

The minimum preemption time usually needs to be converted into the minimum distance along a track from a crossing where train detection must commence to provide the minimum preemption time. To calculate this distance from the time, the maximum train speed is needed.

Train Speed Limits

During the late 1960s, the growth of the trucking industry was diverting revenue cargo away from railroads. Many railroads responded by cutting costs through reduced maintenance of their track and railbed. With reduced maintenance, there was concern that some railroad track could no longer sustain the movement of trains at the speeds railroads chose to operate them without increased risk of derailments. In response, the Congress assigned the Secretary of Transportation responsibility for all matters involving railroad safety in the Federal Railroad Safety Act of 1970 (Public Law 91-458). As part of a solution for railroad safety problems, the Secretary, acting through the FRA, formulated Track Safety Standards which were issued in 1971 and revised in 1982 and later. Several classes of track are defined related to its quality and capability to sustain high-speed trains. Maximum allowed train operating speeds are defined by the FRA for both freight and passenger trains in 49 CFR §213.9(a) for each class of track. As an example, freight trains are allowed to travel no faster than 60 mph, and passenger trains no faster than 80 mph, on FRA Class 4 track. 49 CFR §213 specifies multiple criteria pertaining to properties of tracks required for each class (e.g., gage, separation between rails, alinement, curvature, superelevation, crossties per mile, etc.), which the railroads are required to monitor continuously to be sure that no degradation requires demoting a section of track to a lower class (having lower speed limits).

In spite of being assigned broad powers and responsibilities for all matters associated with railroad safety [45 USC §20103], the FRA completely ignored railroad crossing safety and set train speed limits based solely on track derailment considerations. Safety needs at railroad-highway grade crossings were never considered. This has continued to the present. States and local jurisdictions tried to pass laws and regulations with speed limits to prohibit excessive train speeds based on safety needs at grade crossings. However, in 1993, the U.S. Supreme Court ruled that the FRA had preempted all state and local laws and regulations regarding train speeds when they imposed their train derailment speed limits because the FRA regulations "covered the subject matter" of train speeds at roadway crossings [Easterwood, 1993]. In a dissenting opinion, Justices expressed concern that derailment considerations did not consider all issues involved with safety and train speed at roadway crossings and concern that the ruling that state laws on train speeds were preempted purely by derailment considerations put grade crossing safety in extreme jeopardy, but the majority opinion was that the Secretary had been granted full responsibility for all matters regarding railroad safety, so the FRA was fully empowered to fix the mess. Immediately train speeds increased. The FRA has done nothing, so trains can zoom along any track without any concern about safety associated with train speed except for derailment. The FRA's counterpart, the FHWA, has funded research and published manuals associated with safety and train speed at roadway crossings, especially regarding visibility and audibility of trains at passive crossings, but the FRA has disregarded all of this information and continues to preempt state laws without imposing any regulations of its own.

Railroads normally specify their own track speed limits in their track Timetables or by issuing general orders, slow orders, special speed restrictions, etc. Timetable speeds are frequently set 1 mph slower than the FRA derailment speed limit. These are the speed limits that railroads usually report to the FRA for inclusion in its railroad crossing inventory. The railroad speed limits are set 1 mph slower than the FRA limits because, unlike roadway travel where traffic sometimes travels well under the speed limit, train engineers usually must run continuously at the speed limit to adhere to schedule. For example, Operating Rule 501 of the Canadian National/Illinois Central Railroad states: "Maximum speed must be maintained to the extent possible, consistent with safety and efficiency." [CN/IC, 1999, p. 28] Operating Rule 40 of CSX Transportation states: "Train speeds must be maintained to the extent feasible, consistent with safety." [CSX,1997] Adhering to schedule is important because trains usually operate in both directions, over much of the railroad right-of-way where there is only one mainline track, by scheduling so that trains only have to pass each other at locations where a second track exists. If one train is late in arriving, then the other must wait on it, so successful train engineers try to avoid arriving late.

When something disrupts a train's scheduling, train engineers often try to make up the delay time. With normal running being at maximum speed, this is difficult to do. Sometimes this results in exceeding the railroad's speed limits. When a crash occurs, and excessive train speed is revealed, railroad attorneys always declare that the railroad is not liable for negligence due to excessive train speed, even though the railroad's speed limit was exceeded, because the FRA derailment speed limit was not exceeded. The law and the courts agree. Train speed limits lower than the FRA derailment speed limit are unenforceable. In these cases, railroads usually agree to a settlement and demand that the records be sealed so nobody has access to the information about train speeds and other matters. Nobody really knows how frequently trains exceed a railroad's speed limits. It might be only 1 time in 1000 or 1 time in 10. It occurs frequently in crash cases, as persons who regularly testify as expert witnesses, like the author, can attest. Similarly, nobody knows how frequently trains exceed the FRA limit, but it is expected to be much less often.

For train speeds that are not likely to be exceeded too often, such as no more often than 1 time in 1000, the FRA derailment speed for a track should be used. This can be obtained by requesting from a railroad or the FRA the FRA track class that the railroad submitted to the FRA for the track at a specific crossing, as well as whether passenger trains use the track. The corresponding train speed limits can then be found in 49 CFR §213.9(a).

Constant Warning-Time Variability

Preemption can be incompatible with constant warning-time systems. Constant warning-time systems are intended to provide the same length of warning time at a crossing when either high-speed trains or slow trains are approaching. Although trains accelerate and decelerate slowly compared to roadway vehicles, typical preemption distances are so long, due to long preemption times, that trains can change speed substantially before reaching a crossing after triggering the preemption circuitry. If a train changes its speed, then the preemption time provided will either be too long or too short. Engelbrecht [2001] investigated this problem and measured variations in warning times of 30 sec, mostly attributable to train handling. This variability was similar to what was reported earlier by Richards, et al. [1990b].

Some States have found this problem to be so severe that they do not use constant warning-time equipment where railroad signals have interconnections to traffic signals. The benefit or consequence of prohibiting the use of constant warning-time equipment is that preemption would never be too short, but can certainly be too long, inciting motorist disobedience.

Sometimes railroad policy appears to avert train speed changes at crossings with automated warning devices. For example, Operating Rule 528 of the Canadian National/Illinois Central Railroad states: "When within ¼ mile of a crossing equipped with automatic warning devices, do not increase speed by more than 5 mph until the device has been operating long enough to provide warning and the crossing gates, if equipped, are fully lowered." [CN/IC, 1999, p. 34] This rule must be regularly violated for States to find constant warning-time equipment to be unusable. There is no FRA regulation prohibiting trains from changing speed and defeating constant warning-time equipment, so it must not be important to the FRA for safety. Obviously, the normal nearness of successive crossings with automated devices creates a dilemma in finding locations in which to accelerate or decelerate trains while being expected to be continuously running at maximum speed.

Preempt Trap

When trains accelerate in a constant warning-time circuit, they arrive too soon at a crossing and inadequate warning time is provided to motorists. When trains decelerate in a constant warning-time circuit with an interconnect for preemption, they can create a preempt trap. This occurs when the track clearance green phase ends before the railroad warning lights begin to flash. An approaching train will trigger preemption to clear a queue, but by reducing speed, the train does not actuate the railroad signal before the end of the queue begins moving, so trailing vehicles do not know that a train is approaching and follow the queue onto the track towards the green traffic signal. When the railroad signal is finally actuated, the queue clearance phase will have expired, and the trailing vehicles can find themselves stopped on the track in the path of the train.

Local Safety Hazards

Although the U.S. Supreme Court has declared that the FRA has preempted State and local train speed limits for roadway crossing safety by their derailment speed limits [Easterwood, 1993], there is one exception. It was under 49 USC §20106 (then 45 USC §434) that the U.S. Supreme

Court ruled that State train speed laws were preempted, but that same section of the U.S. Code also contains an escape clause pertaining to local safety hazards. Consequently, the Court recognized that, "the States may adopt more stringent safety requirements 'when necessary to eliminate or reduce an essentially local safety hazard', if those standards are 'not incompatible with' federal laws or regulations and not an undue burden on interstate commerce." [Easterwood, 1993, p. 3]

By adopting an agency rule in the Florida Administrative Code for local safety hazards, and then identifying specific crossings with constant warning-time equipment, especially those crossings interconnected to preempted traffic signals, where a train cannot be allowed to change speed within ½ mile before a crossing, nor exceed a maximum individually-prescribed speed that is reasonably safe for conditions at the crossing, the proper functionality of such devices could surely be greatly improved. This might also be implemented by local ordinance. Since it would only pertain to selected local crossings where higher than acceptable or varying train speeds create a safety problem by truncating warning times, railroads might need to be specifically notified of the track locations and maximum train speeds at each local hazard (even though the railroads already possess this information about their own equipment and timing settings). While railroads will surely argue that this will do no good and change nothing because they are operating their trains properly already, States which have found the use of constant warning-time equipment to be unsatisfactory with preemption might find reason to reverse this decision if railroads had the extra incentive of a regulation that restored liability for their negligence in the interest of public safety.

If preemption distances are too long such that they cause track circuits to overlap with adjacent crossings having automated warning devices, slower train speed limits can be prescribed to prevent the circuits from overlapping and creating a local safety hazard. Applying reduced train speed limits to solve local safety hazards does not violate Federal laws or the intent of Congress; it is precisely what Congress decreed.

In exchange for trains being granted the right-of-way over roadway vehicles at grade crossings in the 1800s, the railroads were encumbered with the duty to provide safe crossings. If train speeds are too fast for roadway crossing safety, a mechanism still exists to slow them as necessary. It is motorists who suffer delay, economic loss and the nuisance of being detained every time they must stop and wait on a train at a railroad crossing. The railroads also want motorists to be laden with the burden of paying the costs for equipment to make crossings safe, while the railroads enjoy a free ride. This is America, where there are not supposed to be privileged classes. If the public does not have sufficient funds to make railroad crossings safe, there should be no reluctance to allow a railroad to pay a small portion of its fair share for crossing safety by imposing a slower train speed limit as a remedy for local safety hazards.

See Errata

IMPLEMENTATION

The first issue that usually must be decided is whether or not preemption is to be provided. If a railroad crossing is within 200 ft of a signalized intersection, this decision is easy because MUTCD §8D.07 declares that preemption "should be provided". For railroad crossings beyond 200 ft, additional information is needed, beyond what is necessary for determining the time required to clear the n^{th} vehicle. This usually consists of determining critical lane volumes at the signalized intersection by procedures contained in the *Highway Capacity Manual* [HCM, 2000] from estimated maximum approach volumes and traffic signal timing settings. It should be kept in mind that expected maximum traffic volumes are needed, not average traffic volumes, unless failures are tolerable half of the time. Then the expected maximum numbers of vehicles queued toward the track, at a given level of confidence, need to be determined from the expected maximum lane volumes and traffic signal timings. This is sometimes achieved using the work of Oppenlander and Oppenlander for random arrivals under free-flow conditions [1996], uniform arrivals [1999], exclusive left-turn lanes [1994] or permissive left-turn lanes [2002].

Procedure

The procedure for implementing the models for determining the time required to clear the n^{th} vehicle in a queue from a track at railroad-preempted traffic signals is simple:

(1) Determine whether the signalized intersection is within 200 ft of the railroad crossing, as specified in MUTCD §8D.07, or whether expected maximum queues are likely to extend back as far as the track, using Table 3 or Equations 3, 4 and 6, such that traffic signal preemption for queue clearance is needed.

(2) Identify the appropriate design vehicle in compliance with MUTCD §8A.01(4).

(3) Obtain the design length and acceleration category of the design vehicle by consulting Table 2.

(4) Determine the minimum track clearance distance in compliance with MUTCD §8A.01(8), as illustrated in Figure 1.

(5) Determine the clear storage distance in compliance with MUTCD §8A.01(3), as illustrated in Figure 1.

(6) Add the minimum track clearance distance and the clear storage distance to get the critical queue length.

(7) Enter Figure 3 or Equation 7 with the critical queue length and get the expected progressive startup delay.

(8) Add any needed special adjustments for opposing left-turn stragglers, distracted drivers due to environment, driveway interference or other factors to get the expected maximum startup delay. The following are expected maximum delays (but mostly estimated from small samples):

See Errata

Opposing lagging left-turn stragglers: 4 sec after start of red
Left or right turns at intersection: 0 sec
Narrow lanes: 0 sec (widen lanes instead)
Sharp right-turn corner radius: 0 sec (increase radius and add cross-street turn-bay)
Combination trucks or other vehicles in queue: 0 sec (handled in Figure 5)
Grades: 0 sec (handled in Figure 5)
Distracted drivers: 7 sec per driver
Right turns into intervening driveways: 0 sec per passenger vehicle
Right turns out of intervening driveways: 8 sec per passenger vehicle
Empty vehicle-storage adjustment: see procedure in the text

(9) Add the design vehicle length and the minimum track clearance distance to get the repositioning distance.

(10) Enter Figure 5 or Equation 8 with the repositioning distance, the design vehicle type and acceleration category (and grade if the design vehicle is a combination truck) and get the expected maximum repositioning time.

(11) Add the expected maximum startup delay and expected maximum repositioning time to get the expected safe track-clearance time.

(12) Add the train-detection equipment-delay time, pedestrian minimum truncation time, yellow change interval time, train separation time and other necessary time adjustments to the expected safe track-clearance time to get the expected safe minimum-preemption time. (Outside the scope of this project.)

Examples

Example Problem 1 (See Addendum)

Specifics: A railroad track crosses a roadway near a signalized intersection having an interconnect circuit to the railroad train-detection circuitry. The distance between the rail closest to the intersection and the intersecting roadway pavement edge is 190 ft. The stop-line at the intersection is offset from the pavement edge by 12 ft. The railroad has a single track which intersects the roadway at a right angle, and has a pavement stop-line located 15 feet from the closest rail near the railroad flashing lights. The pavement is level and all vehicles except WB-29 and WB-35 trucks are permitted to use the roadway crossing the track.

Question: How much time is needed for track clearance?

Process: Following the steps given above:

(1) The MUTCD requires preemption when a track is less than 200 ft from a signalized intersection, and the distance in this example is only 190 ft., so preemption is needed.

(2) Table 2 indicates that the WB-20 truck is the longest vehicle that is not prohibited from using the roadway, so it must be selected as the design vehicle to comply with the MUTCD.

See Errata

(3) Table 2 indicates that the design length of the WB-20 truck is 74 ft, and the acceleration category for the vehicle is WB-15.

(4) The minimum track clearance distance, as illustrated in Figure 1, is 15 ft from the track stop-line to the closest rail, 5 ft between rails, plus 6 ft past the farthest rail: 15 + 5 + 6 = 26 ft.

(5) The clear storage distance, as illustrated in Figure 1, is 190 ft between the closet rail and closest pavement edge less 6 ft from the rail and less 12 ft from the stop-line: 190 - 6 - 15 = 169 ft.

(6) The critical queue length is the sum of the clear storage distance (step 5), and the minimum track clearance distance (step 4): 169 + 26 = 195 ft.

(7) Figure 3 indicates that for a critical queue length of 195 ft (step 6), the expected progressive startup delay is 17 sec.

(8) No special conditions are indicated in the problem specifics, so no adjustments for driver distractions due to the roadside environment, driveway interference or other factors can be determined to be needed to get the expected maximum startup delay.

(9) The repositioning distance is the sum of the minimum track clearance distance (step 4) and the design vehicle length (step 3): 26 + 74 = 100 ft.

(10) Figure 5 indicates that to reposition a design vehicle, having WB-15 acceleration characteristics (step 3), a distance of 100 ft on level pavement (step 9), the expected maximum repositioning time is 20 sec.

(11) The expected safe track-clearance time is the sum of the expected progressive startup delay (step 7) and the expected maximum repositioning time (step 10): 17 + 20 = 37 sec.

Answer: 37 sec of track-clearance time is needed.

Example Problem 2

Specifics: A railroad track crosses a 4-lane roadway near a signalized intersection having an interconnect circuit to the railroad train-detection circuitry. The distance between the rail closest to the intersection and the intersecting roadway pavement edge is 350 ft. Queues frequently back up across the track. The stop-line at the intersection is offset from the pavement edge by 15 ft. The railroad has two tracks with centerlines that are18 ft apart that intersect the roadway at a 60° angle. The pavement stop-line at the crossing is located 15 feet from the closest rail near the railroad flashing lights. The pavement has a 2% upgrade, 12 ft lanes, and all vehicles except WB-29 and WB-35 trucks are permitted to use the roadway crossing the track.

Question: How much time is needed for track clearance?

Process: Following the steps given above:

See Errata

(1) The MUTCD does not require preemption when a track is over 200 ft from a signalized intersection, but it does require that coordination be considered. The distance in this example is 350 ft., so coordination should be considered. Since queues frequently back up across the track, preemption is pertinent.

(2) Table 2 indicates that the WB-20 truck is the longest vehicle that is not prohibited from using the roadway, so it must be selected as the design vehicle to comply with the MUTCD.

(3) Table 2 indicates that the design length of the WB-20 truck is 74 ft, and the acceleration category for the vehicle is WB-15.

(4) The minimum track clearance distance, as illustrated in Figure 1, requires consideration of crossing skew. The distance of 15 ft from the track stop-line to the closest rail is always measured perpendicular to the track from either the left or right edge of the travel lane, whichever edge is closer. This means that the travel distance to the track parallel to the roadway will be 15 / cos 30° = 15 / 0.866 = 17.3 ft. The perpendicular distance between the closest rail and the farthest rail is 2.5 + 18 + 2.5 = 23 ft. Therefore, the travel distance between the rails along the roadway is 23 / cos 30° = 23 / 0.866 = 26.6 ft. The edge of the roadway lane which is closest to the nearest rail on one side of the track will be farthest from the track on the other side, so the extra longitudinal distance along the roadway to transition from one lane edge to the other must be added. Since the roadway is 4 lanes, there are two 12 ft lanes in each direction with a perpendicular distance from the lane edge on one side to the lane edge on the other of 24 ft. The distance along the roadway at the tracks for a pavement width of 24 ft is 24 / tan 60° = 24 / 1.732 = 13.9 ft. The travel distance along the roadway of 6 ft perpendicular to the track from the farthest rail is 6 / cos 30° = 6 / 0.866 = 6.9 ft. Adding these components, the minimum track clearance distance is 17.2 + 26.6 + 13.9 + 6.9 = 65 ft.

(5) The clear storage distance, as illustrated in Figure 1, is 350 ft between the closet rail and closest cross-street pavement edge less 6 ft from the rail and less 15 ft from the stop-line: 350 - 6 - 15 = 329 ft.

(6) The critical queue length is the sum of the clear storage distance (step 5), and the minimum track clearance distance (step 4): 329 + 65 = 394 ft.

(7) Figure 3 indicates that for a critical queue length of 394 ft (step 6), the expected progressive startup delay is 27 sec.

(8) No special conditions are indicated in the problem specifics, so no adjustments for driver distractions due to the roadside environment, driveway interference or other factors can be determined to be needed to get the expected maximum startup delay.

(9) The repositioning distance is the sum of the minimum track clearance distance (step 4) and the design vehicle length (step 3): 65 + 74 = 139 ft.

(10) Figure 5 indicates that to reposition a design vehicle, having WB-15 acceleration characteristics (step 3), a distance of 139 ft (step 9) on a 2% upgrade (interpolating

See Errata

between level pavement and a 4% upgrade), the expected maximum repositioning time is 26 sec.

(11) The expected safe track-clearance time is the sum of the expected progressive startup delay (step 7) and the expected maximum repositioning time (step 10): 27 + 26 = 53 sec.

Answer: 53 seconds of track-clearance time is needed.

Example Problem 3 (See Addendum)

Specifics: A railroad track crosses a level roadway near a signalized intersection having an interconnect circuit to the railroad train-detection circuitry. The distance between the rail closest to the intersection and the intersecting roadway pavement edge is 750 ft. The stop-line at the intersection is offset from the pavement edge by 15 ft. The railroad has a single track which intersects the roadway at a right angle, and has a pavement stop-line located 15 feet from the closest rail near the railroad flashing lights. With 99% confidence, it has been determined that a minimum of 0% to a maximum of 5% of the vehicles crossing the track are combination trucks, and 0% to 15% are other vehicles besides passenger vehicles. From an intersection critical movement analysis and a queue length analysis it was determined with 95% confidence that a maximum of 20 vehicles were expected to be queued from the intersection traffic signal.

Question: How much time is needed for track clearance?

Process: Following the steps given above:

(1) The MUTCD does not require preemption when a track is over 200 ft from a signalized intersection, but it does require that coordination be considered. The distance in this example is 750 ft., so coordination should be considered. For a queue of 20 vehicles with 0% to 5% combination trucks and 0% to 15% other vehicles, the expected composition ranges from 20 passenger vehicles and no trucks or other vehicles to 16 passenger vehicles with 4 trucks or other vehicles. The various expected mixes of vehicles are shown below with the expected maximum queue lengths from Table 3:

Pass Veh	Combos	Other	Max Len
16	1	3	719
17	1	2	701
18	1	1	684
16	0	4	683
19	1	0	666
17	0	3	666
18	0	2	648
19	0	1	630
20	0	0	612

The longest queues result from the largest percentages of the longest vehicles. Consequently, 5% combination trucks, 15% other vehicles and the rest of the queue comprised of passenger vehicles produces the longest queue of 719 ft. Because the

See Errata

longest expected queue is shorter than the 729 ft of clear storage distance to the track (750 ft - 15 ft stop-line offset - 6 ft track buffer), queues should not be expected to spill back onto the track, so preemption is unneeded. The expected maximum queue length for 16 passenger vehicles, 0 combination trucks and 4 other vehicles was not available in Table 3, so it was calculated from Equations 3, 4 and 6 as follows:

EQL = 27 * 16 + 77 * 0 + 42 * 4 = 600 ft
SD = $(50 + 25 * 16 + 77 * 0 + 70 * 4)^{1/2}$ = 27 ft
EQL_{max} = 600 + 3.090 * 27 = 683 ft

Answer: Preemption is unneeded.

Example Problem 4 (See Addendum)

Specifics: A railroad track crosses a roadway near a signalized intersection having an interconnect circuit to the railroad train-detection circuitry. The distance between the rail closest to the intersection and the intersecting roadway pavement edge is 700 ft. The stop-line at the intersection is offset from the pavement edge by 15 ft. The railroad has a single track which intersects the roadway at a right angle, and has a pavement stop-line located 15 feet from the closest rail near the railroad flashing lights. The pavement is level and combination trucks are prohibited from using the roadway that crosses the track. The single-unit bus is the design vehicle. From an intersection critical movement analysis and a queue length analysis it was determined with 95% confidence that a maximum of 20 vehicles were expected to be queued from the intersection traffic signal. With 99% confidence, it has been determined that a minimum of 0% to a maximum of 60% of the vehicles crossing the track are other vehicles, not passenger vehicles. There is a driveway into a nudist camp 550 ft from the intersecting roadway, and from field observations it was determined that 95% of the time no more than 2 motorists in a queue were distracted by this enterprise, no more than 2 passenger vehicles turned right into the driveway, and no more than 2 passenger vehicles turned right from the driveway, although the vigilance of the field crew is suspect.

Question: How much time is needed for track clearance?

Process: Following the steps given above:

(1) The MUTCD does not require preemption when a track is over 200 ft from a signalized intersection, but it does require that coordination be considered. The distance in this example is 700 ft., so coordination should be considered. For a queue of 20 vehicles with 0% to 60% other vehicles, the longest queue expected would consist of 8 passenger vehicles and 12 other vehicles. Equations 3, 4 and 6 indicate this queue would have an expected maximum length of 822 ft:

EQL = 27 * 8 + 77 * 0 + 42 * 12 = 720 ft
SD = $(50 + 25 * 8 + 77 * 0 + 70 * 12)^{1/2}$ = 33 ft
EQL_{max} = 720 + 3.090 * 33 = 822 ft

Since the expected maximum queue length (as well as other queue compositions) exceeds the clear storage distance of 679 ft (700 ft - 15 ft stop-line offset - 6 ft track buffer), queues are expected to spill back onto the track, so preemption is needed.

(2) The proper design vehicle is given as being the single-unit BUS.

(3) Table 2 indicates that the design length of the BUS is 40 ft, and the acceleration category for the vehicle is SU.

(4) The minimum track clearance distance, as illustrated in Figure 1, is 15 ft from the track stop-line to the closest rail, 5 ft between rails, plus 6 ft past the farthest rail: 15 + 5 + 6 = 26 ft.

(5) The clear storage distance, as illustrated in Figure 1, is 700 ft between the closet rail and closest pavement edge less 6 ft from the rail and less 15 ft from the stop-line: 700 - 6 - 15 = 679 ft.

(6) The critical queue length is the sum of the clear storage distance (step 5), and the minimum track clearance distance (step 4): 679 + 26 = 705 ft.

(7) Figure 3 indicates that for a critical queue length of 705 ft (step 6), the expected progressive startup delay is 43 sec.

(8) The nudist camp is a distraction to some motorists. Its driveway is far enough back from the front of the queue where drivers may think they have sufficient time before they will be able to move to divert their attention briefly towards the camp. Two distracted drivers at an expected maximum delay of 7 sec each adds 14 sec to the queue clearance time. Passenger vehicles turning into the driveway are not expected to delay the queue, but two passenger vehicles turning out of the driveway at an expected maximum delay of 8 sec each adds another 16 sec to the queue clearance time. The expected maximum startup delay is the sum of the expected progressive startup delay (step 7) plus the special delay adjustments: 43 + 14 + 16 = 73 sec.

(9) The repositioning distance is the sum of the minimum track clearance distance (step 4) and the design vehicle length (step 3): 26 + 40 = 66 ft.

(10) Figure 5 indicates that to reposition a design vehicle, having SU acceleration characteristics (step 3), a distance of 66 ft (step 9), the expected maximum repositioning time is 6 sec.

(11) The expected safe track-clearance time is the sum of the expected maximum startup delay (step 8) and the expected maximum repositioning time (step 10): 73 + 6 = 79 sec.

Answer: 79 sec of track-clearance time is needed.

CONCLUSIONS AND RECOMMENDATIONS

The preemption of traffic signals near railroad crossings has been called for by the MUTCD for over 50 years. While there are several documents discussing this matter, little specific guidance is available on the actual amount of preemption time required to clear traffic queues off of a nearby railroad track. Some values that are often assumed in estimating this time, and some rules-of-thumb that are often utilized, do not seem to be consistent with observations reported in the literature.

This study developed models for determining the time required to clear the n^{th} vehicle in a queue off a track. The models adopt a high level of confidence to minimize the risk of accidents. Yet, they avoid the "worst-case" concept to avert an invitation for litigation when an improbable or unforseen worse case results in a crash.

The models are easy to apply, and do not require the user to provide input information that must be guessed or approximated. The models require very limited input information. For determining queue clearance times, neither traffic volumes nor the distribution of vehicle types nor estimates of average vehicle spacings (including trucks) are needed. The inputs are just the minimum track clearance distance, the clear storage distance, and the types of vehicles permitted to use the roadway.

Where special conditions exist, such as driveways intercepting the roadway in the queue storage area, additional factors can affect queue clearance times and may need to be applied as adjustments. Estimation guidance for adjustment factors is provided.

Track clearance time was considered in two components. One component was the time delay incurred after a traffic signal changes to green that a vehicle in jeopardy at the track must wait while leading vehicles move forward before the vehicle in jeopardy can move forward. This was designated as startup delay time and it was found that it was a maximum when all leading vehicles were short passenger vehicles. The second component was the repositioning time while the vehicle in jeopardy accelerated forward out of harm's way. Repositioning time was found to depend on the type of vehicle being repositioned, and for heavy trucks, the grade of the roadway surface.

A weakness of the models is the lack of information on how much additional delay may be imposed on queue dispersion by right-turning combination trucks or buses at intersections having a narrow two-lane cross-street with a tight corner radius. While such locations exist, it is difficult to collect a sample of truck maneuver times because trucks try to avoid turning where these conditions exist. Rather than studying the matter further, it seems more reasonable to pursue countermeasures such as widening the intersection, providing a right-turn bay and/or increasing the corner radius to cure the defects. The cost to the public of these countermeasures is likely to be much less than the costs of preemption to compensate for such deficiencies.

In comparing the clearance times from the models herein with results estimated by other methods, it must be remembered that the times are intended to accommodate nearly all queues. Some of the factors used in other methods are average values which normally should be expected to allow failures about half of the time.

Minimum preemption times can be rather long. On most of the occasions when trains arrive and preemption is actuated, there are no motorists in jeopardy parked over a track (hopefully). Consequently, most of the time preemption just inflicts wasted time on motorists while queues of vehicles that are not endangered are cleared. Research is needed into detector circuitry to detect if no vehicles are in jeopardy so a preemption call can be aborted when it is not needed for track clearance.

Research is also needed on the delays imposed on queues by large trucks maneuvering into and out of driveways.

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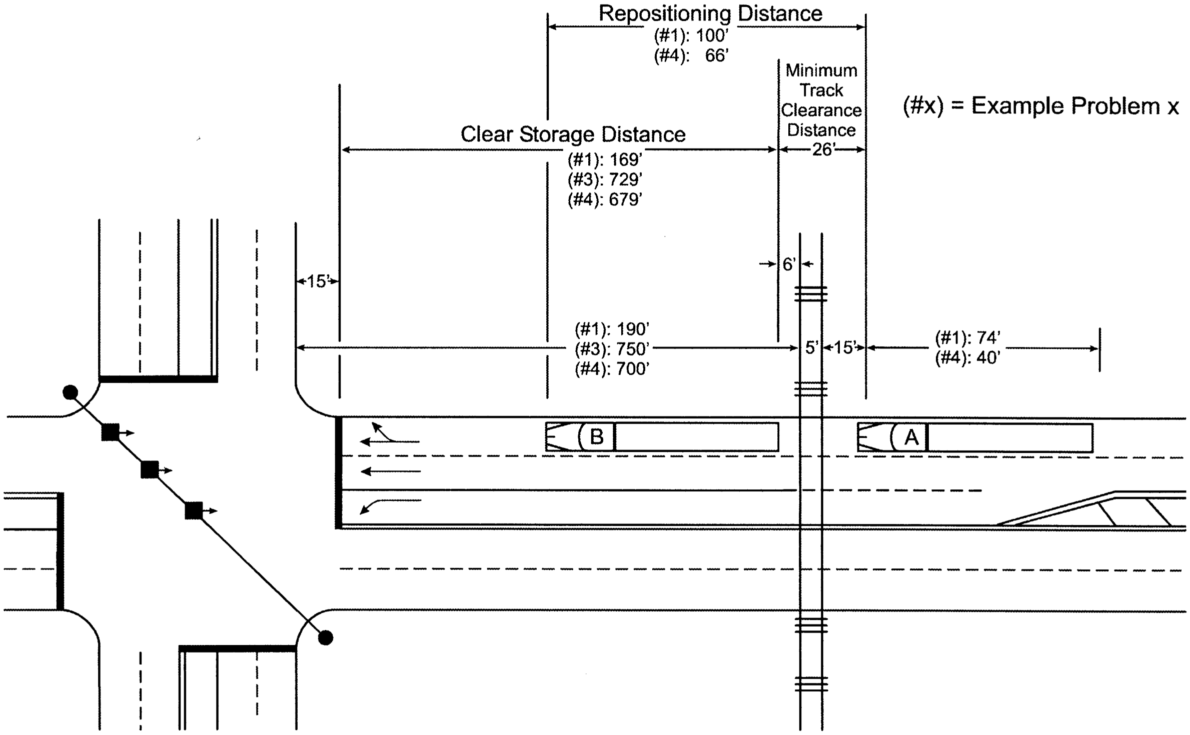
Clearance Time Requirements at Railroad-Preempted Traffic Signals

ADDENDUM 1

1) Refer to the following schematic diagram for Example Problems 1, 3 and 4:



2) Refer to the following schematic diagram for Example Problem 2: