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Traffic Control Device Evaluation Program: FY 2017

Technical Report 9-1001-14-4

Cooperative Research Program

TEXAS A&M TRANSPORTATION INSTITUTE

COLLEGE STATION, TEXAS

in cooperation with the
Federal Highway Administration and the
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16. Abstract

This report presents findings on the activities conducted in the Traffic Control Device Evaluation Program during the 2017 fiscal year. The research on sponsored changeable message signs (continued from the previous year) was terminated by the Federal Highway Administration. Also continued from the previous year, the update for determining time requirements for traffic signal preemption at highway-railroad grade crossings was completed. New efforts included a focus on wrong-way driving mitigation efforts throughout Texas and a safety evaluation of wet-weather pavement markings in Texas. The final activity was an effort to develop a method to compute roadway grade from streaming data.

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TRAFFIC CONTROL DEVICE EVALUATION PROGRAM: FY 2017

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DISCLAIMER

This research was performed in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the FHWA or TxDOT. This report does not constitute a standard, specification, or regulation.

This report is not intended for construction, bidding, or permit purposes. The researcher in charge of this project was Paul Carlson.

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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CHAPTER 1. SPONSORED CHANGEABLE MESSAGE SIGNS

FHWA EXPERT AND REVIEW AND MEETING

The request to experiment originally submitted to Federal Highway Administration (FHWA) for the sponsored logo signs called for an on-road study to be completed in fiscal year (FY) 17.

FHWA Manual on Uniform Traffic Control Devices (MUTCD) staff hired three human factors experts to review the report on the legibility and glare testing completed by TTI in summer 2015. This review took place in summer 2016 and culminated in a visit to TTI by the review panel in early September 2017. Those attending this meeting were:

- Martin Calawa, FHWA MUTCD team.
- Dr. Linda Boyle, Professor of Industrial Engineering at University of Washington.
- Dr. Don Fisher, Professor of Industrial Engineering at University of Massachusetts-Amherst, with a joint appointment at the United States Department of Transportation (USDOT) Volpe Lab.
- Dr. Andrew Kun, Associate Professor, Electrical and Computer Engineering, University of New Hampshire.
- Eric Nadler, USDOT Volpe National Lab. Coordinator for expert review.
- Toni Whitfield, FHWA Texas Office.

The one-day meeting consisted of detailed review of the methodology and results of the study presented by Sue Chrysler, Paul Carlson, and statistician Eun Sug Park. The group also visited the RELLIS campus during the day and at night to view the signs and to observe the testing procedure in the instrumented vehicles. The expert group was satisfied with the methods, results, and conclusions and recommended to FHWA that the on-road study be allowed to proceed. A revised Phase II plan was submitted to FHWA on September 28, 2017, incorporating these changes.

Through correspondence with Mr. Calawa, researchers know that the expert group met via teleconference October 19, 2017, but were not privy to those discussions. On November 8, 2016, researchers received written summary of comments from the panel from Mr. Calawa on the revised Phase II (on-road study) plan. The experts were asked if they thought there were any safety concerns with the on-road study and each of them reported that they felt it was safe to proceed. Researchers were advised by Mr. Calawa that FHWA upper management would be reviewing the use of sponsored logos from a policy perspective.

In January 2017, TxDOT and TTI met with FHWA Office of Operations in FHWA headquarter office to discuss the progress and answer questions. At that time, FHWA announced they were planning to issue a Request for Comment (RFC) in the *Federal Register* to identify other possible concerns they have not yet discovered with their own efforts and the expert panel efforts. A schedule for RFC was discussed. Initially, FHWA asked for the research to be paused until they had time to issue an RFC and digest the comments. An alternative was proposed was presented where the research would be allowed to continue as planned while the RFC was being prepared and issued. The meeting ended with an action item for FHWA to decide if the simultaneous effort could be pursued rather than pausing the research. In June 2017, FHWA followed up with correspondence that terminated the previously approved research plan. FHWA's reasoning was based on applicable statutes and regulations but they did not provide specific references. The FHWA correspondence is included as Appendix A.

OTHER ACTIVITIES

A research paper based on the closed-course testing was submitted for presentation at the 2017 Transportation Research Board meeting. It received strong reviews and was accepted for presentation and publication immediately and without revision. A poster presentation was made during the meeting in January 2017, and the article was subsequently published in the *Transportation Research Record:*

> Chrysler, S. T., Carlson, P. J., Brimley, B., and Park, E. S. (2017). "Effects of full matrix color changeable message signs on legibility and roadway hazard visibility." *Transportation Research Record: Journal of the Transportation Research Board*, (2617), 9–18.

The two Daktronics dynamic messages signs (DMSs) were removed from the TTI RELLIS campus on October 19, 2016, with the assistance of the Texas Department of Transportation (TxDOT) Houston District and their contractors. They were stored at the Bryan District maintenance yard until summer 2017 when they were installed on I-45 north of Huntsville on a new ITS project there. The signs were activated in August 2017.

CHAPTER 2. WRONG-WAY DRIVING MITIGATION EFFORTS IN TEXAS

INTRODUCTION

The Texas A&M Transportation Institute (TTI) has been deeply involved in wrong-way driving (WWD) analysis and countermeasure evaluation since the beginning of the interstate highway system in the 1950s. In the 1960s and 1970s, TTI conducted some of the first WWD related research (*1*, *2*). More recently, in 2012, TTI quantified the wrong-way crash issue on freeways in Texas and evaluated the effectiveness of select WWD countermeasures as part of TxDOT Project 0-6769 (*3*). TTI researchers have also conducted studies to develop and test WWD alerts that can be posted on DMSs or received in vehicles to warn right-way drivers of a possible event (*3*, *4*). Currently, TTI researchers are developing connected vehicle applications that will detect wrong-way vehicles, notify the transportation agency and law enforcement, and alert affected travelers (TxDOT project 0-6867-01). In addition to these research projects, TTI researchers assist many TxDOT districts with on-going wrong-way crash analysis and countermeasure evaluations.

As part of the project documented herein, TTI researchers were tasked with summarizing the countermeasure implementation and other actions taken by TxDOT since 2010 in the San Antonio, Houston, Fort Worth, Dallas, and Austin Districts to mitigate WWD on freeways. TTI researchers also reviewed the available data sources in three of these regions that could be used to analyze the WWD issue and countermeasure effectiveness. Based on these data, TTI researchers identified future evaluations that can be conducted. TTI researchers also hosted a WWD forum where attendees discussed recent research and countermeasure implementations in Texas and Florida.

SUMMARY OF WWD ACTIVITIES

The following section summarizes the actions taken by TxDOT since 2010 to mitigate WWD in the San Antonio, Houston, Fort Worth, Dallas, and Austin Districts.

San Antonio District

In August 2010, the San Antonio Police Department (SAPD) began to use an emergency call signal (i.e., E-Tone) for its radio network when a wrong-way driver was reported to 911. In January 2011, SAPD implemented a code in their computer-aided dispatch (CAD) system that specifically identified all WWD events. Similarly, in March 2011, TxDOT TransGuide traffic management center operators began logging all WWD events, not just those that resulted in a crash. In May 2011, TxDOT TransGuide operators began displaying wrong-way driver warning messages on DMSs when an E-tone was issued (previously they waited to display warning message until the wrong-way driver was visually verified). Two of these procedures (i.e., code in the SAPD CAD system and TxDOT logging all WWD events) created databases that could be used to determine the WWD trends in San Antonio.

In May 2011, public transportation and law enforcement agencies in the San Antonio area created a WWD task force to share information and identify the means to address and reduce WWD activity. Participating agencies included:

- TxDOT San Antonio District.
- TxDOT Traffic Operations Division.
- City of San Antonio (CoSA) Police Department (SAPD).
- CoSA Public Works Department.
- Bexar County Sheriff's Office.
- FHWA Texas Division.
- City of Balcones Heights Police Department.
- Southwest Research Institute (SwRI).
- TTI.

The task force used various methods to document WWD activity in San Antonio, with the purpose of identifying where WWD countermeasure deployment would be most meaningful and effective. After analyzing the various WWD event data sources and the information details available from each source, analysts determined that insufficient information existed to link WWD events with specific freeway ramps where wrong way drivers entered the freeway network. Accordingly, there was no logical means that could be devised for prioritizing the treatment of one freeway ramp over another. The task force concluded that treatment of an entire freeway corridor was necessary to determine the effectiveness of WWD countermeasures.

To assist with the selection of a test corridor in San Antonio, TTI researchers imported WWD data points from SAPD 911 call logs, TxDOT's TransGuide operator logs, and the TxDOT Crash Record Information System (CRIS) into a geographic information system (GIS) database to form a single set of WWD event locations. TTI researchers then used spatial analysis functions native to the ArcView GIS platform to create a density map of WWD activity in San Antonio that used a color ramp from green (low WWD density) to red (high WWD density) to emphasize locations of the most intense WWD activity. Figure 1 shows the resulting map for 2011.

This map shows that the highest number of WWD events occurred at US 281 and Airport Boulevard. In addition, US 281 from I-35 to Stone Oak Parkway contained the most WWD events of all the corridors analyzed. Based on this information, the task force selected the 15-mile US 281 corridor from I-35 (near downtown) to just north of Loop 1604 (the far north central side of San Antonio) as the Wrong Way Driver Countermeasure Operational Test Corridor.

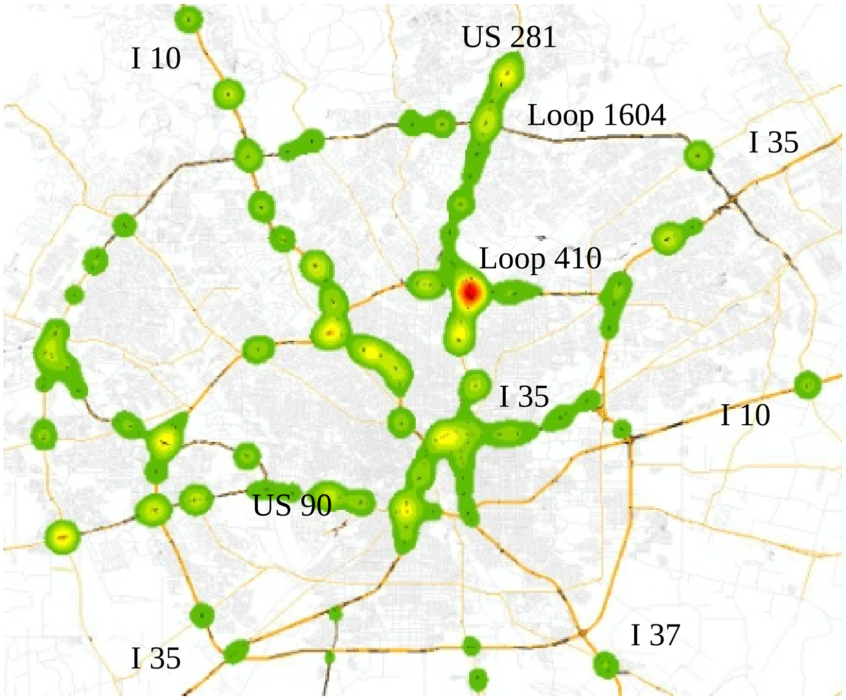


Figure 1. 2011 WWD Density Map of San Antonio.

Between March 2012 and July 2012, TxDOT San Antonio District staff and their contractors installed WRONG WAY signs with flashing red light emitting diodes (LEDs) around the border at each exit ramp in the US 281 test corridor (see Figure 2). The purpose of the flashing red LEDs was to increase the conspicuity of WRONG WAY signing at night. The signs were set to flash under low ambient light conditions (i.e., at night and during some inclement weather events), whether or not a wrong-way vehicle was detected. TxDOT felt that this operation would catch the attention of a wrong-way driver approaching on the frontage farther away, instead of waiting until they were driving up the ramp. Also, sustained false alarm issues with the detection equipment led to deactivation of the detection component.

Where the length and design of the exit ramp allowed, WRONG WAY signs with flashing red LEDs around the border supplemented the existing, static WRONG WAY signs. On shorter ramps, the WRONG WAY signs with flashing red LEDs around the border replaced the existing static WRONG WAY signing. The battery for the signs was encased in the sign pole and charged by a small solar array attached to the top of the sign support.

Figure 2. Example of WRONG WAY Sign with Flashing Red LEDs around Border.

Previous research found a 38 percent reduction in the WWD events on the US 281 test corridor after the installation of WRONG WAY signs with flashing red LEDs around the border at all exit ramps in the corridor (*3*). This finding was based on WWD event data extracted from SAPD 911 logs from January 2011 to April 2014. A more recent update of this analysis that included data through April 2016 found a 32 percent reduction in WWD events on the US 281 test corridor. While this reduction in WWD events is slightly less than initially found, the expanded data set findings show the long-term effectiveness of these signs. Based on these positive findings, the TxDOT San Antonio District has implemented WRONG WAY signs with flashing red LEDs around the border at ramps along I-35, I-10, and Loop 1604. Additionally, the TxDOT San Antonio District will be installing multiple-radar wrong-way driver detection systems on US 90 during summer and fall 2017 (see Figure 3). Included with these systems are flush-mounted bars of flashing red LEDs that affix to the sides of existing WRONG WAY signs. Other future countermeasure implementations include installing lowered height (3 ft) WRONG WAY signs on I-10.

In addition to these efforts, TTI personnel partnered with TxDOT and CoSA staff to conduct reviews of priority exit ramps, and went with TxDOT to review signing and markings at all ramps in the San Antonio area to ensure the presence and accuracy of standard wrong-way signing. TTI researchers have also worked with TxDOT San Antonio District staff to identify, test, and troubleshoot several technologies that can be used to improve WWD event information gathering and WWD event response. Table 1 documents in more detail all of the activities taken by TxDOT in the San Antonio District to address WWD from 2010 to 2017.

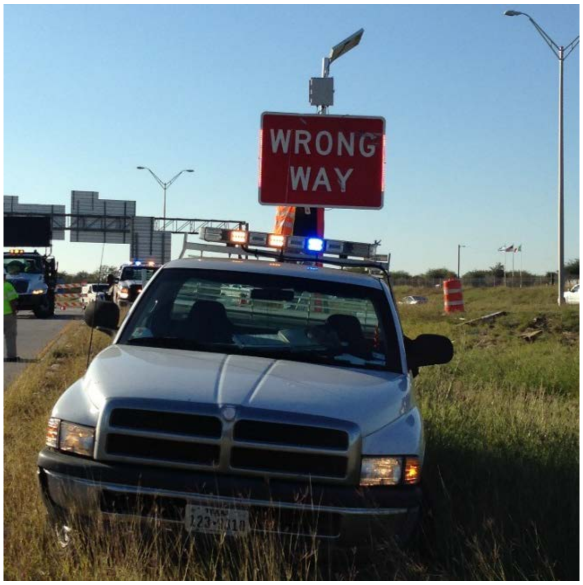


Figure 3. Example of Multiple-Radar Detection WWD Warning System with Flashing LED Panels.

Table 1. San Antonio Agency Actions for WWD Events 2010–2017.

Date of Change	WWD Response Action Change
August 2010	SAPD implemented an emergency call signal (i.e., E-tone) for its radio network for WWD events.
November 2010	SAPD authorized the use of portable spike strips in certain situations to stop wrong-way drivers.
January 2011	SAPD implemented a code in their 911/CAD system to identify WWD events (effectively began creation of 911/CAD WWD logs).
March 2011	TxDOT TransGuide traffic management center operators began documenting all WWD events (previously documented only if a crash resulted).
May 2011	TxDOT TransGuide operators began displaying WWD warning messages on DMSs to right-way drivers when E-tone issued (previously waited to display warning message until wrong-way driver was visually verified). San Antonio WWD task force formed. Started monthly meetings.
June 2011	TxDOT and staff from other task force agencies performed a few exit ramp site visits to identify issues in the field and identify whether or not signing or marking deficiencies of any kind existed.
July 2011	SAPD traffic crash investigators asked to determine entry point used by wrong-way drivers if possible. Test installation of a radar unit to identify WWD events in the field installed at I-35 and Nogalitos southbound exit. Signal from radar unit received at TxDOT TransGuide, but discovered that modifications were needed to TransGuide's Lonestar software to properly receive WWD event signal. WWD research problem statement developed by task force and sent to TxDOT's research office.
September 2011	TTI created the first GIS-based map of San Antonio WWD events using TransGuide and SAPD 911 logs. Using this map, the task force determined that the initial WWD countermeasure deployment would be along US 281 from downtown to Loop 1604.

Date of Change	WWD Response Action Change
October 2011	The task force selected flashing LED border-illuminated WRONG WAY signs in conjunction with radar units (WWD event detection devices) as the preferred WWD countermeasures for test corridor implementation. The task force also recommended mainlane systems that include blank-out WRONG WAY signs.
November 2011	TxDOT installed the first flashing LED border-illuminated WRONG WAY sign at I-35/Nogalitos test location.
December 2011	SwRI completed changes to TxDOT's Lonestar software to allow receipt of a WWD event signal from radar units.
January 2012	TxDOT approved use of internal funding to install WWD countermeasures on US 281 test corridor ramps.
February 2012	Media event hosted at TxDOT TransGuide to announce the US 281 WWD test corridor and other WWD countermeasure efforts.
March 2012	TxDOT began flashing LED border-illuminated WRONG WAY sign installation on US 281 (southern end of corridor near downtown). Funding approval was sought for ramp WWD countermeasures on I-35 on the north and west sides of downtown. TxDOT and TTI performed ramp signing and marking reviewed on US 281 and I-35 downtown.
April 2012	Over half of the US 281 ramp flashing LED border-illuminated signs installed (installation began from the south and reached Bitters Road).
May 2012	TxDOT and TTI delivered a presentation at the Intelligent Transportation Society America Annual Meeting on the San Antonio WWD task force and US 281 test corridor.
June 2012	TxDOT and TTI continue ramp signing and marking reviews on US 90.
July 2012	Flashing LED border-illuminated WRONG WAY sign installation effectively completed on US 281. TxDOT installed the first ramp radar detection units at Nakoma; however, communications to TxDOT TransGuide were not yet enabled.
September 2012	Nine radar units installed on US 281, to date. SwRI completed testing of ramp radar units (detection of wrong-way vehicles and transmission of detection signal).
October 2012	14 radar units installed on US 281, to date. TxDOT began construction project on I-35 from Judson Road to FM 3009. Flashing LED border-illuminated WRONG WAY signs to be installed as ramps are reconstructed.
December 2012	Radar installation on US 281 completed from downtown to Bitters Road. Calibration and orientation of the radars with the manufacturer were ongoing because false signals to TxDOT TransGuide proved to be problematic.
March 2013	Ramp radar manufacturer visited US 281 test corridor to help resolve false call detection issues. Flashing LED border-illuminated WRONG WAY sign installation began on I-35 from Laredo Street to US 281/I-37 (downtown area).
April 2013	SwRI demonstrated the mainlane WWD detection and response system installed at its test track to TxDOT.
July 2013	TxDOT completed installation of flashing LED border-illuminated WRONG WAY signing along I-35 from Laredo Street to US 281/I-37. Task force group merged its meetings into monthly meetings of the San Antonio Traffic Incident Management Group.

Date of Change	WWD Response Action Change
June 2013–August 2013	TxDOT and TTI perform broad review of WWD ramp signing and markings on Loop 410, I-10, I-35, I-37, and US 90 in the San Antonio urban area.
December 2013	TxDOT installed first wrong-way mainlane system on I-10 at Callaghan/Wurzbach. TxDOT completed flashing LED border-illuminated WRONG WAY sign installation on I-35 from Judson Road to FM 3009.
January 2014	TxDOT began flashing LED border-illuminated WRONG WAY sign installation on I-10 from Huebner to Loop 410.
March 2014	TxDOT completed flashing LED border-illuminated WRONG WAY sign installation on I-10 from Huebner to Loop 410. TxDOT installed second wrong-way mainlane system on I-35 at Judson Road.
May 2014	Testing of mainlane WWD systems on I-10 and I-35 revealed detection issues that cannot be quickly resolved; systems deactivated until detection issues can be resolved.
November 2014	TxDOT received award from ITS Texas for its WWD initiative in San Antonio.
June 2015	TxDOT installed flashing LED border-illuminated WRONG WAY signs on I-10 from I-37 to US 87.
October 2015	TxDOT receives National Roadway Safety Award for San Antonio WWD initiative from the USDOT (FHWA) and the Roadway Safety Foundation.
November 2015	Flashing LED border-illuminated WRONG WAY signs installed on Loop 1604 (new freeway section) from Bandera Road to Culebra Road.
January 2016	TxDOT installed flashing LED border-illuminated WRONG WAY signs on I-10 from Huebner to Loop 1604.
December 2016	TxDOT installed a multiple-radar detection and verification wrong-way warning system at a test site (Loop 410 EB exit to Southton Road).
January 2017	TxDOT completed installation of retroreflective red tape on WRONG WAY and DO NOT ENTER sign posts on ramps along most of Loop 410 around the north, west, and south sides of town (from I-35 NE counter-clockwise around Loop 410 to I-37).
Summer/Fall 2017	TxDOT to install multiple-radar detection and verification wrong-way warning system on US 90 exit ramps from I-35 to Loop 1604.
October 2017	TxDOT to install lowered height (3 ft) WRONG WAY signs on I-10 from Foster Road to Graytown Road.

Houston District

In 2014, the TxDOT Houston District requested that TTI examine the WWD issue on Houston freeways. The primary focus of this evaluation was crashes from 2009 to 2013, since the Houston Police Department (HPD) did not specifically tag WWD events in their 911 call logs at that time. Researchers obtained the crash data from the TxDOT CRIS database. Researchers identified patterns in WWD crashes by time of day, driving under the influence of drugs or alcohol, day of the week, month of the year, age of the wrong-way driver, gender of the wrong-way driver, and light condition. Researchers also plotted the location of the WWD crashes.

Researchers again determined that insufficient information existed to link WWD crashes with specific freeway ramps where wrong-way drivers entered the freeway network. Accordingly, there was no logical means that could be devised for prioritizing the treatment of one freeway ramp over another. Ultimately certain WWD countermeasures, such as wrong-way arrows on the pavement and red retroreflective material on sign supports, should be deployed for an entire freeway corridor or zone. Other supplemental countermeasures, such as WRONG WAY signs with flashing red LEDs around the border or internally illuminated WRONG WAY signs, should be deployed at specific ramp locations based on field condition assessment of ramp geometry, driver expectation, and other characteristics.

Using data from the TxDOT CRIS database, researchers conducted ArcMap-based spatial analysis to identify higher-density WWD crash locations, and assist in prioritizing and selecting locations for countermeasure deployment. Based on this analysis, the TxDOT Houston District was divided into three subareas:

- I-610 Loop and freeway segments inside the I-610 Loop (Priority Zone I).
- Sam Houston Tollway and freeway segments between I-610 and within 3 miles outside the Sam Houston Tollway (Priority Zone II).
- TxDOT Houston District freeways not covered in prior efforts (Priority Zone III).

In 2014 and 2015, a field condition assessment was conducted to catalog deficiencies in existing wrong-way related signage and markings along exit ramps, as well as identify any atypical locations that should be considered for supplemental treatments. In total, 894 exit ramps were reviewed that stretched over 1125 miles. Based on the findings from this assessment, the TxDOT Houston District worked to address the identified deficiencies and installed red retroreflective material on sign supports and wrong-way arrows on the pavement. In addition, the TxDOT Houston District considered additional countermeasures at select locations, including the installation of WRONG WAY signs with flashing red LEDs around the border at three ramps. The TxDOT Houston District also procured and tested two different types of WWD detection systems.

Currently, the TxDOT Houston District is installing multiple-radar wrong-way driver detection systems with flashing red LED flush-mounted enhancement bars (see Figure 3). To date, nine of these systems have been installed at the following locations:

- I-69 SB at Jackson.
- I-45 SB at Dallas.
- I-69 SB at McGowen.
- I-45 NB at N. Main.
- I-69 NB at Main.
- I-45 SB at Houston.
- I-45 SB at Scott.
- I-69 SB at Shepherd (not operational – waiting on piece of equipment).
- I-69 SB at Fannin – (currently under repair).

In August 2017, TxDOT will install these systems at the following locations:

- I-45 SB at Pierce.
- I-45 SB at Allen.
- I-10 EB at Jensen.

Fort Worth District

In 2015, TTI researchers assisted in the identification and conceptual development of safety projects for the approximately $620,000 in regional funding administered by the North Central Texas Council of Governments (NCTCOG) to combat WWD on the western side of the Dallas-Fort Worth region within Tarrant County. Researchers worked with TxDOT Fort Worth District staff and other stakeholders (i.e., City of Arlington and City of Fort Worth) to develop a plan to combat WWD in Tarrant County with a technology focus for countermeasure evaluation. Researchers used a three-step planning process:

1. Analyze and aggregate WWD data, including:
    a. Crashes from 2007 to 2014 (see Figure 4).
    b. 911 reports (City of Arlington).
    c. Exposure data (average daily traffic data for Tarrant County freeways).
    d. Site conditions (e.g., ramp geometry, signs, and markings).
    e. Local knowledge/engineering judgement.
2. Identify and prioritize corridors.
3. Develop countermeasure recommendations.

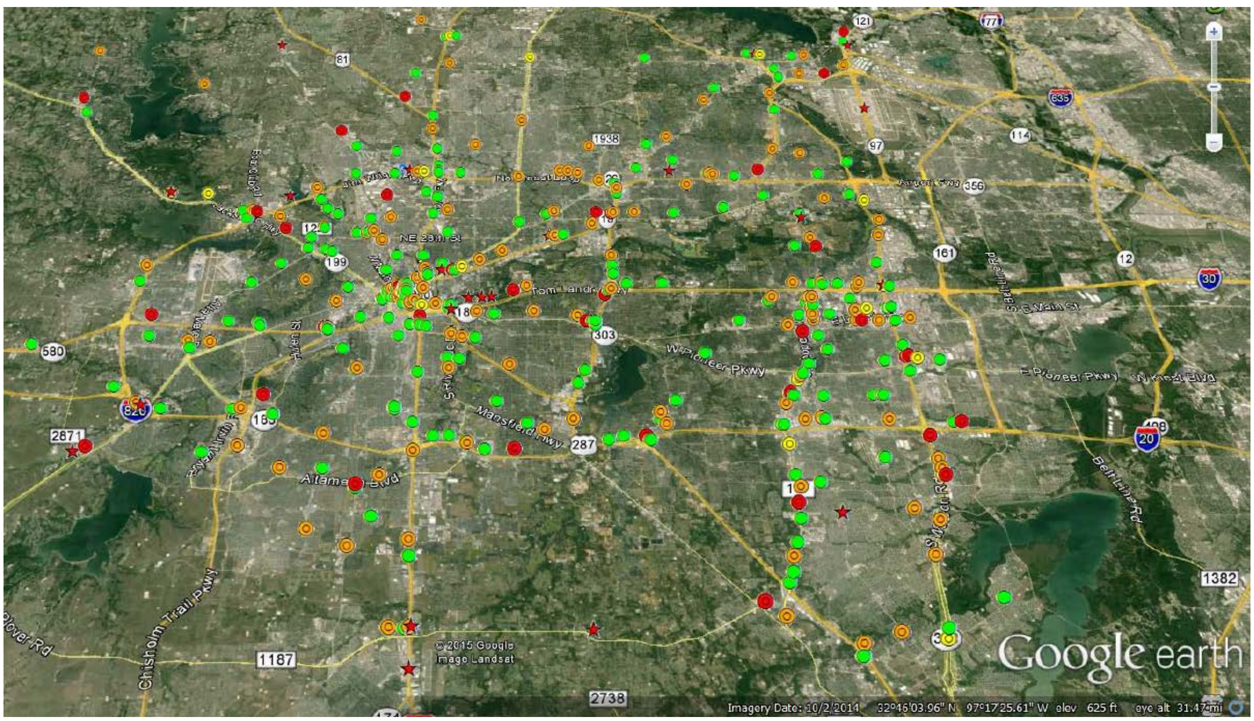


Figure 4. Google Earth Display of Tarrant County WWD Crashes (2007–2014).

The *Tarrant County Priority Corridor and Wrong-Way Driving Countermeasures Plan* identified seven priority corridors to implement traditional devices (i.e., wrong-way arrows on the pavement, red retroreflective material on sign posts, and simplified markings), innovative devices (i.e., lowered DO NOT ENTER and WRONG WAY signing), and active technology (i.e., LED signs [see Figure 5]).

Figure 5. Solar-Powered LED Signs with Reflective Tape and Camera Monitoring System.

TxDOT developed the detailed plans (Control Section Job [CSJ] 0902-00-138) and let a project to implement recommended countermeasures based on available funding in June 2016. The project implemented WWD detection equipment, WRONG WAY signs with flashing red LEDs around the border, pavement markings, signing, high definition radar vehicle sensing device vehicle-alert modules and wireless communications in the following three corridors:

- I-30 (University Drive to AT&T Way/Baird Farm Road).
- I-35W (I-30 to Belknap Road).
- SH 360 (Spur 303/Pioneer Parkway to Trinity Boulevard).

The construction was completed in January 2017 at a cost of approximately $887,000. Many sites have closed-circuit television cameras deployed that are capturing both still and video images when wrong-way vehicles are detected.

TTI staff will be working with TxDOT Fort Worth District staff to monitor and evaluate the performance of sites with countermeasures deployed over the next several years. The evaluation will include assessing wrong-way entries and successful turnarounds at sites with cameras. The pilot lowered sign implementation on the SH 360 corridor between Pioneer Parkway and Trinity Boulevard will also be a focus area of the performance evaluation study being done as part of the interagency agreement.

Dallas District

The Dallas Traffic Management Team and the Dallas Section of the Texas Institute of Transportation Engineers developed a task force to get the consensus of agency leaders on a solution that provides uniform and consistent traffic control on cross streets (particularly at freeway diamond interchanges), meets the MUTCD and American Association of State Highway and Transportation Officials guidelines, and addresses the needs of both the general public motorists and impaired drivers. The task force's recommended solutions included:

- Elimination of conflicting lane assignment signs and markings.
- Addition of straight arrow markings in extended bays.
- Addition or relocation of larger ONE WAY signs on signal mast arms as close to the left-turn as possible.
- Remove left arrow on TxDOT and North Texas Tollway Authority (NTTA) trailblazer signs in advance of the intersection.

These recommendations were developed with the objectives of being highly effective for impaired drivers and promotion of consistent traffic control for better driver compliance. Implementation would cost approximately $5,000 per interchange assuming two left-turn lanes on the cross-street approaches. One million dollars in funding has been secured for use in Dallas County from the Regional Toll Revenue program to implement the wrong-way signing solutions recommended by the task force.

NCTCOG, TxDOT Dallas District, and nine Dallas County cities used the $1 million in regional funding to initiate Phase I of the Wrong-Way Driving Pilot Mitigation Project in 2014. Phase I focused on implementing a standard set of countermeasures at 350 diamond interchanges throughout Dallas County. The standard countermeasure package included replacement of conflicting lane and arrow markings, reflective tape on sign posts, signal enhancements, and other intersection-related improvements.

Eight cities (Carrollton, Farmers Branch, Garland, Grand Prairie, Irving, Mesquite, Richardson, and Rowlett) have installed the Phase I countermeasures as part of a TxDOT contract (CSJ 0918-00-234 – January 2014). This effort has also been expanded into Collin and Denton Counties, where improvements are either in the design or construction phase in Allen, Carrollton (Denton County), Lewisville, and McKinney as part of a $1.8 million TxDOT project (CSJ 0918-00-263 – September 2016). The City of Dallas has 19 intersections under construction and an additional 42 in the design phase.

Austin District

In early 2017, a consultant completed a WWD study for the TxDOT Austin District that included short-, mid-, and long-term recommendations for addressing WWD in the Austin area. They used a three-step analysis involving roadway and traffic conditions, five years of crash data, and the latest research and summaries on the state of the practice. Similar to other studies, WWD crashes in the Austin District were found to occur at night and involve impaired drivers.

Based on this report, the TxDOT Austin District is working to lower DO NOT ENTER and WRONG WAY signs and replace/install wrong-way arrows on freeway exit ramps in the region. The first phase involved lowering the sign height of 44 signs from 7 ft to 3 ft on I-35 from Riverside Drive to US 183. This first phase also included replacing 31 wrong-way arrows and installing 5 new wrong-way arrows along the same corridor. The implementation of these WWD countermeasures is part of the TxDOT Austin District safety initiative and will be expanded to other freeways in the future as funding allows. The Central Texas Regional Mobility Authority has also lowered DO NOT ENTER and WRONG WAY signs at exit ramps on the three tollroads they oversee in the Austin District (i.e., 183A, Manor Expressway, and 183 South).

REVIEW OF DATA SOURCES

The following sections document the data sources reviewed in the San Antonio, Houston, and Austin Districts.

San Antonio District

As discussed previously, both SAPD and TxDOT TransGuide traffic management center operators log all WWD events, not just those that resulted in a crash. Through their involvement in the San Antonio WWD task force, both SAPD and TxDOT shared the WWD subcomponent of their data logs for the border-illuminated WRONG WAY signing evaluation. For three and a half years, SAPD devoted staff time to the post-processing of 911 event data to extract more detailed WWD event information that may be of use to other agencies in designing and implementing countermeasures. The TxDOT TransGuide operator logs generally included only the events that occurred in their coverage area—about half of the freeways in the San Antonio region.

TTI researchers have also extracted WWD-involved crash information from the TxDOT CRIS database. However, since CRIS only documents WWD-related crashes, rather than any event where WWD activity is observed, the number of records was small compared to the SAPD 911 WWD call logs and the TxDOT TransGuide operator WWD logs.

Since 2011, TTI staff have maintained the San Antonio region's database of WWD event activity in cooperation with TxDOT and SAPD. Researchers routinely analyze these data to generate descriptive statistics of WWD event activity, including the location, duration, and length of WWD events. For previous analyses, researchers primarily used the SAPD 911 call logs for any statistical analysis since these logs began in January 2011, continued without interruption through April 2014, and contained more data points. However, for a period of time, WWD event data from May 2014 onward were only available from the TxDOT TransGuide logs. This was due to changes at SAPD that limited their ability to post process the 911 data and provide detailed WWD event data on a regular basis.

In early 2017, discussions between SAPD and TTI resulted in TTI receiving the SAPD 911 call log WWD event data from May 2014 through February 2017. However, with multi-agency focus on countermeasure deployment rather than development, SAPD no longer post-processes the 911 call log entries for detailed information. Instead, SAPD has only shared more general WWD 911 event data with other agencies. Table 2 and Table 3 show the information contained in each WWD event entry in the SAPD 911 call log before and after May 2014, respectively. With supplemental data processing, both call log types were used to create intensity maps of WWD activity in the San Antonio area and guide WWD countermeasures installations. TTI continues to coordinate with SAPD to obtain monthly updates on WWD events reported to 911. TTI also continues to receive and process the TxDOT TransGuide WWD event data on a monthly basis.

Table 2. SAPD 911 WWD Event Data from January 2011 to May 2014.

Data Element	Comments
Date	MM/DD/YY
Time	24:00
Day	Day of week (Mon, Tue …)
Incident #	SAPD incident entry identifier from 911 call log
Highway	Roadway name for facility where WWD occurred
At/Near	Nearest cross street to observed WWD location
Travel	Direction of WWD (NB in SB, etc.)
Known Entry	Freeway ramp where WWD entered mainlanes, if known
Details	911 call taker notes on event; may include car description
Result	SAPD code or case number (if vehicle stopped)

Note: Spreadsheet color indexing was used to denote multiple calls relating to one overall WWD event.

Table 3. SAPD 911 WWD Event Data from May 2014 to Present.

Data Element	Comments
Master Incident Number	SAPD incident entry identifier from 911 call log
Case Number	SAPD case number (if vehicle stopped)
Battalion	SAPD patrol battalion where WWD event occurred
Time Phone Pickup	911 call date and time; MM/DD/YY 24:00
DOW	Day of week (Mon, Tue …)
Problem	911 call type; all entries are WWD
Response Area	SAPD section assigned to call
Location Name	Supplemental location information, if known
Address	Roadway and nearest intersecting street to observed WWD
LAT	Latitude of WWD event
LON	Longitude of WWD event

Houston District

As mentioned previously, for the initial assessment of the WWD issue in the TxDOT Houston District, researchers only used crash data since HPD did not have a mechanism for identifying WWD events in their 911 call logs. In 2015, HPD began using a specific code to tag WWD events in their 911 call database to allow for quicker capture of data for analysis. TTI researchers have recently began receiving, researching, and analyzing the CAD system logs for WWD events that have been reported to law enforcement but did not necessarily result in a crash. Table 4 shows the information contained in each WWD event entry in the HPD 911 call log. Researchers recently geolocated and plotted the data from 2015 and 2016. In addition, researchers were able to conduct an initial revised GIS assessment of locations of interest. However, additional research is needed in terms of locational accuracy for the data.

TTI researchers contacted 28 other local agencies that have a presence along the freeway system in the six-county TxDOT Houston District to obtain information about their CAD systems. Among these entities there are multiple types of CAD systems in use. Outside of the City of Houston, no other local agency has a designated code in their CAD system to identify WWD events. Instead, WWD events are typically coded as traffic complaint, reckless driving, or traffic hazard. While most CAD systems allow for the dispatcher's notes to be searchable, the amount of time need to handle such a request was highly variable. The majority of the agencies contacted require a formal Open Records Request to obtain data.

Table 4. HPD 911 WWD Event Data.

Data Element	Comments
Code	HPD incident entry wrong-way identifier (2468)
Text	Description of incident (TRAFFIC HAZARD/Wrong Way Driving)
Date	MM/DD/YYYY
Time	24:00
Month Number	Number of the month (1–12)
Day Number	Number of the day (1–31)
Hour	Hour of the day (1–24)
Month	Name of the month (Jan, Feb, …)
Day	Day of week (Mon, Tue …)
Location	Address provided by caller
District	HPD patrol district
X	Longitude of WWD event
Y	Latitude of WWD event

At many ramps were supplemental countermeasures (such as the multiple-radar wrong-way driver detection systems with active notification) are installed no wrong-way entry data were collected before their installation. TTI researchers wanted to investigate the feasibility of using a portable camera system to monitor WWD events at select ramps to determine if this equipment could be used to collect wrong-way entry data before the implementation of detection systems. TTI researchers identified a segment of I-45 in downtown Houston with a high incidence of CAD-reported WWD events. Researchers then worked with TxDOT Houston District personnel to conduct video monitoring for a location with three potential exit-ramp feeders onto the system that may be contributing to that segments WWD events (I-45 exit to Pierce/Bagby/Jefferson Street). Researchers received and reviewed approximately 80 of 120 hours of video footage. Intermittent camera outages and rain reduced the amount of footage available to identify WWD events. Overall, researchers identified one wrong-way driver and a potential issue with signage (several vehicles entered the wrong exit, backed up, and proceeded down their intended exits).

Austin District

In April 2017, TTI researchers communicated with the Austin Police Department (APD) about the availability of WWD event data in their 911 call logs. It was learned that there was no call type to capture this in the APD 911 call data. After this conversation, APD implemented a new code to denote incidents where the suspect vehicle is traveling the wrong-way on highways, expressways, their entrances and exits, flyovers, and frontage roads. Table 5 shows the information contained in each WWD event entry in the APD 911 call log. APD expects WWD event data to be available in six months.

Table 5. APD 911 WWD Event Data from April 23, 2017, to July 5, 2017.

Data Element	Comments
Date	MM/DD/YY
Time	24:00
Call Number	APD incident entry identifier from 911 call log
Address	Roadway name, roadway section or address for facility where WWD occurred
Initial Call Type Description	The call type description that was identified initially
Final Call Type Description	The final determination of the call type description
Long	Longitude of the WWD location
Lat	Latitude of the WWD location

WWD FORUM

TTI has been working with the University of Central Florida (UCF) on a National Cooperative Highway Research Program project investigating WWD crashes on divided highways. As part of this collaboration, a UCF professor planned a visit to TTI in College Station, Texas, to discuss a modeling methodology for identifying WWD hotspots. TTI researchers thought this would be an opportunity to exchange knowledge and lessons learned about the WWD issue and countermeasures between Texas and Florida agencies. Therefore, TTI invited staff from TxDOT, NTTA, the Florida's Turnpike Enterprise, the Central Florida Expressway Authority, and several other Texas agencies with an interest in WWD. This event could be attended in-person or via webinar. Overall, 19 people attended this inaugural Texas WWD Forum. Table 6 contains the agenda. The event was very well received and the majority of the participants expressed interest in attending such forums once a year. Future forums would include a broader audience, provide updates on activities across the state, and allow for the exchange of lessons learned and evaluation results.

Table 6. Inaugural Texas WWD Forum Agenda.

Time	Agenda Item	Speaker(s)
	Main Events	
9:00 a.m. to 9:15 a.m.	Introductions	Melisa Finley
9:15 a.m. to 10:45 a.m.	UCF WWD Hotspots™ Modeling Methodology	Haitham Al-Deek
10:45 a.m. to 11:00 a.m.	Break	
11:00 a.m. to 11:30 a.m.	TTI Wrong-Way Driving Efforts	Melisa Finley
11:30 a.m. to 12:00 p.m.	San Antonio Wrong-Way Driving Efforts	John Gianotti
12:00 p.m. to 1:00 p.m.	Lunch	
1:00 p.m. to 1:30 p.m.	Dallas/Fort Worth Wrong-Way Driving Efforts	Scott Cooner
1:30 p.m. to 2:00 p.m.	NTTA's Wrong-Way Driving Efforts	Eric Hemphill
2:00 p.m. to 2:30 p.m.	Other Wrong-Way Driving Efforts in Texas	Attendees
2:30 p.m. to 3:00 p.m.	Open Discussion/Closing	Attendees
	Ancillary Events	
3:00 p.m. to 3:15 p.m.	Break	
3:15 p.m. to 3:45 p.m.	Visibility Research Laboratory Tour	Adam Pike
3:45 p.m. to 4:15 p.m.	Driving Simulator Tour	Laura Higgins Myung Ko
4:15 p.m. to 5:00 p.m.	RELLIS Tour	Melisa Finley

FUTURE EFFORTS

The WWD event data compiled by TTI researchers in several regions in Texas have proven invaluable for identifying problem areas and examining the potential for both near- and long-term study of WWD countermeasures. Continued oversight and coordination of ongoing activities, as well as new investigations and evaluations, is critical for the reliability of the evaluations, the ability to compare findings across the state, and the sharing of lessons learned. Below is a list of a few recent and forthcoming WWD countermeasure implementations that TTI researchers could assist with and evaluate in future phases of this research project.

- Recently, as part of an implementation project led by TTI, the Houston and San Antonio Districts received 44 multiple-radar wrong-way driver detection systems with flashing red LED flush-mounted enhancement bars. These systems will be installed at exit ramps in these two regions during summer/fall 2017. The existing WWD event databases maintained by TTI staff can be used to assess the effectiveness of these devices. Using similar data collection and analysis techniques will be critical to ensure that the effectiveness of the countermeasure can be compared across the two regions.
- In the Dallas District, researchers could evaluate the effectiveness of the consistent application of countermeasures at diamond interchanges in Collin, Dallas, and Denton Counties.
- In the Austin District, researchers could evaluate the effectiveness of lowered DO NOT ENTER and WRONG WAY signs, as well as replacing/installing wrong-way arrows, at freeway exit ramps along a portion of I-35. The evaluation could be expanded as these countermeasures are applied to additional corridors in the region.

In addition, TTI researchers can help other districts or areas in Texas quantify their WWD issue, conduct reviews of exit ramps, and select and evaluate mitigation measures. Finally, TTI can continue to organize and host a Texas WWD Forum that facilitates the exchange of knowledge on WWD activities across the state.

CHAPTER 3. SAFETY EVALUATION OF WET-WEATHER PAVEMENT MARKINGS

Wet-night conditions pose a significant safety hazard to motorists for many reasons. One safety issue is that typical pavement markings lose their visibility and are generally unable to adequately delineate the roadway in wet-night conditions. TxDOT often supplements typical markings with retroreflective markers to aid in wet-night visibility, but these markers can fail and not provide supplement guidance to drivers. To aid in wet-weather conditions, some pavement markings are designed to provide increased levels of retroreflectivity in wet-night conditions. These markings are typically more expensive than standard markings, and their durability has not been adequately studied.

This initial study seeks to explore the safety effect on wet-night crashes of using non-standard pavement markings. Researchers developed a database of roadway segments with non-standard markings and acquired the roadway characteristics and crash information for those segments. To increase the robustness and statistical validity of the results, researchers employed two different evaluation methods to assess the safety benefits of the wet-weather pavement markings: Empirical Bayes (EB) before-after analysis and Full Bayes (FB) before-after analysis with comparison groups.

DATA COLLECTION METHODOLOGY

The following information was gathered to conduct the safety analysis of the wet-weather pavement markings:

- Pavement marking location and retroreflectivity data.
- Roadway inventory data.
- Crash data.
- Weather data.

Pavement Marking Location and Retroreflectivity Database

The location of the pavement markings was based on contractor-supplied mobile retroreflectivity reports. The retroreflectivity readings are recorded for every marking line (center lines, skip lines, edge lines) on the roadway. The initial retroreflectivity data were measured by the contractor and provided to TxDOT. Table 7 describes the information provided in the mobile retroreflectivity report.

Table 7. Information Provided in Mobile Retroreflectivity Reports.

Variable	Description
County	The county in which the markings were applied
Roadway	The roadway on which the markings were applied
Date	The date on which the retro readings are taken
Color	The color of the marking
Material	Marking material type
From and To	Start and end point of the project
Left and Right Retro	Retroreflectivity readings of the markings

TxDOT provided researchers two sets of data. The first set was an Excel® table with summarized information from numerous sets of contractor supplied mobile retroreflectivity information. TxDOT also included the project number for each of the jobs listed. The second set of data was hard copies of the mobile retroreflectivity data collected by the contractors that had not yet been manually entered into the database. The data in the hard copies did not have the project numbers. Researchers entered the rest of the data in the format TxDOT had started to generate a single database of information based on the contractor-supplied mobile retroreflectivity readings. Each individual project was given a unique ID (U_ID) so that the information gathered in future steps could be linked together.

There is a need to combine three databases for the crash study—the marking location, type, and retroreflectivity database, the roadway inventory database (RhiNo database), and the crash database (CRIS database). The databases are integrated through the means of the U_ID and the distance from origin (DFO) of start and end point of each of the projects is maintained the same in all three databases. The DFO information identifies the roadway segment that is being considered.

The project numbers can be used to obtain the plans and specifications files for each of the projects. Some of the projects have the Reference Marker (RM) and distance from RM both for the start and end point for each of the stretches over which the marking retro is evaluated. However, researchers observed that several project plans and specifications do not specify the RM data. The RM data need to be converted to the DFO data to be useful for the data analysis. The DFO data can also be obtained if the global positioning system (GPS) coordinates of the end points are available. The coordinates of the start and end point have been obtained through the following means:

1. The hard copies of the mobile retroreflectivity data have the GPS coordinates of the point where the retro readings are evaluated. The coordinates of the start and end point of the roadway segment can be obtained from these data.
2. In the cases where the hard copies of data were not provided and the RM information could not be found, researchers obtained the GPS data from Google Earth®. The name of the project stretch along with the name of the start and end point of the stretch can be used to obtain the coordinates of the point. However some of the data points may not be reliable since the names of the start and end points do not always clearly define a point in Google Maps.

Most of the project stretches had either the RM data or the coordinates of the start and end point (obtained from the field retro data or Google Maps). However a few sites did not have either of these data and were not useful for further analysis. These segments were discarded. For some segments, the DFO information could not be obtained and these sites were also discarded. The methodology to evaluate DFO using the RM or GPS coordinates is described in the next section.

Steps to Calculate DFO from RM or GPS Coordinates

The DFO information of start and end points of each treatment segment were identified using the statewide point data set of Texas Reference Marker locations maintained by the Transportation Planning and Programming Division of TxDOT (*5*). The detailed steps are shown below:

Step 1: Obtain the reference marker or GPS coordinate information of start and end points.

(a) Reference Marker—In the case where the location information of a treatment segment is available from the TxDOT Daily Work Report, the reference markers of the start and end points can be identified.

(b) GPS Coordinates—In the case where the location information is not available in the TxDOT Daily Work Report, the GPS coordinates of start and end points are identified in Google Maps.

Step 2: Match the RM or coordinates with the Texas Reference Marker Database.

(a) Reference Marker—If the RM of start or end point of a treatment segment is available, the RM can be matched in the Texas Reference Marker database.

(b) GPS Coordinates—The distance between start or end points and the reference markers listed in the Texas Reference Marker database are calculated using the GPS coordinates. The equations used to calculate distance between two GPS coordinates are shown below:

$$d = R * c \qquad (1)$$

$$c = 2 * \arccos(\sin(x_1) \times \sin(x_2) + \cos(y_1) \times \cos(y_2) \times \cos(\Delta)) \qquad (2)$$

Where,

d = distance between two points.
R = the radius of the Earth (i.e., 6,378,137 m).
x_1, x_2, y_1, y_2 = coordinates of two points.
Δ = the absolute difference.

Summarizing the Location and Retroreflectivity Database

The retroreflectivity database has several rows for every project. This is because most of the roads usually have multiple pavement markings. The retroreflectivity readings are collected for each line on the road. The material types for each of these line types maybe different even in the same stretch. The retroreflectivity readings are taken in both directions for many yellow markings. Researchers summarized the data for each of the sites into a single row. It was observed that there are a maximum of two material types on any given project stretch. In the case of center lines, the retroreflectivity values for the two lines are averaged and a single value is reported. The values for marking evaluated in both the directions were averaged and a single value was reported.

Roadway Inventory Database

The road inventory data are collected and updated regularly by TxDOT (usually once every year). The road inventory data are an essential component of this study since the annual average daily traffic (AADT), road geometrics, and traffic characteristics are expected to affect the number of crashes. The TxDOT road inventory has the data for the years 2011 to 2015. The study considers data over 7 years, between 2011–2017. Thus some assumptions have to be made for the 2016 and 2017 data. These assumptions will be described later. The road inventory database of TxDOT has numerous parameters. However in this study, only a few parameters are considered. The first column of the RhiNo data is the U_ID, which would enable researchers to identify the project stretch and thus the start and end DFO.

The RhiNo data are extracted through the aid of ArcGIS and statistical software called R. The start and end point of the RhiNo data do not exactly match with the start and end DFO from the retroreflectivity report data. This is because the start and end DFO in RhiNo are the start DFO of the first segment in the stretch and the end DFO of the last segment in that stretch. The first and last segments considered in a stretch are based on the actual DFOs of the project stretch. The segments closest to the actual DFOs (start and end of stretch) are considered for extracting the inventory data. These segments may not be the same over the years. Thus the total project length is not expected to be same in every year.

The parameters considered in this study and the methodology adopted are described in this section:

1. AADT – A single project stretch may have several segments of different lengths and different AADTs. The presence of intersections on the stretch can sometimes create a large variation of AADT between the segments. In this study, the AADT of a project stretch is evaluated by taking the weighted average of the AADTs of all the segments, with segment length being the weight. The AADTs are evaluated the same way in every year. The AADT for the years 2016 and 2017 does not exist in the database. Researchers forecasted the AADTs in 2016 and 2017 using two methodologies:

   - Researchers evaluated the percentage change in AADT in the consecutive years and averaged these values. This average percentage was used to forecast the AADT for 2016 and 2017.
   - Researchers developed a linear model using 5 years AADT data for every project stretch and use that to project the AADTs in the years 2016 and 2017.

3. Length of the section – This parameter gives the total length of the project stretch. The length of the section is determined by adding the sum of the length of segments that are

part of that project stretch. The values are evaluated every year. Researchers used the data of the year 2015 in the year 2016 and 2017. The length is given in miles in RhiNo.

4. Functional System – This parameter gives the functional class of the project roadway. The functional class of individual segments that form a project stretch may be different but this only happens for long stretches. Researchers used the mode of the functional class of all the segments and assign it as the functional class of the project stretch. This value is evaluated for every year. We use the same value as 2015 in the years 2016 and 2017.

5. Rural/Urban Classification – This parameter gives the type of area near the project stretch. Even for this parameter, researchers used the mode of the values of each of the segments that constitute the stretch and assign it to the stretch. Researchers evaluated the value for every year and use the value of year 2015 for the years 2016 and 2017.

   The codes for the rural/urban classification as defined in the RhiNo database are:

   1=Rural (Population < 5,000).
   2=Small Urban (Population 5,000–49,999).
   3=Urbanized (Population 50,000–199,999).
   4=Large Urbanized (Population 200,000+).

6. Speed Limit – This parameter gives the speed limit of the project stretch. There may be changes in the speed limit among the segments that form the project stretch. The mode of the speed limits is taken and assigned to the entire project stretch. The speed limits are evaluated every year up to 2015 and the value of 2015 is used in 2016 and 2017. The speed limit is given in mph in the RhiNo database.

7. Number of Lanes – The mode of the number of lanes of the segments in a project stretch is assigned to the entire stretch. The values in 2015 are again used for 2016 and 2017. The number of lanes does not include the turning, climbing, or auxiliary lanes but includes Super 2 and exclusive high occupancy vehicle/high occupancy toll lanes.

8. Surface Type and Median Type – The similar methodology described earlier is used where the mode of the segment values is assigned to the entire stretch. The values for 2015 are assigned to 2016 and 2017.

   The codes for the surface type in the RhiNo database are:
   - 01=Road is unpaved (unpaved).
   - 02=Low Type Bituminous Surface-treated (paved, flex).
   - 03=Intermediate Type mixed (paved, flex).
   - 04=High Type Flexible (paved, flex).
   - 05=High Type Rigid (paved, concrete).
   - 06=High Type Composite (paved, flex).
   - 99=Unknown (New in 2014).

The codes for the median type in the RhiNo database are:
0=No median.
1=Curbed.
2=Positive Barrier.
3=Unprotected.
4=One-way pair.
5=Positive Barrier Flexible.
6=Positive Barrier SemiRigid.
7=Positive Barrier Rigid.

Note: Include Median Type 1 and 3 for Medians that include Grass, Gravel, dirt, etc.

9. Median Width – The median width of the project is the mode of the median width of the segments that constitute the stretch. The value is evaluated every year. For the years 2016 and 2017, the value in 2015 is used. The median width is given in feet.

10. Lane Width – The lane width is not directly specified in the RhiNo database. The surface width is given, and the number of lanes is given. The lane width is calculated as the surface width divided by the number of lanes. The lane width is given in feet. The individual segments in a project stretch may have different lane widths. The lane width of the project stretch is calculated as the weighted average of all the segments in that stretch. The weights are the individual segment lengths. The years 2016 and 2017 use the same data as year 2015.

11. Shoulder Width – While the shoulder width is given for both the inside and outside shoulder, only one value of shoulder width was reported (the average of the inside and outside shoulder widths). Again the individual segments within a project stretch may have different shoulder widths. However the shoulder width of the project stretch is calculated as the weighted average of the shoulder widths of the segments. The weights are the segment lengths. The shoulder width is specified in feet in RhiNo. The shoulder width is evaluated every year. The values in year 2015 are used for the years 2016 and 2017.

Researchers gathered the above information and included it with the location and retroreflectivity data that were previously put into the database. Each of the U_IDs has the values for all these parameters for the seven years. Thus each U_ID has seven rows of data in the database, one row per year of analysis.

Crash Database

The crash data are usually recorded by the police and later processed, which includes adding additional variables usually related to geometric and traffic conditions. TxDOT has a database for the crash data, which is known as the CRIS database. The CRIS database has the data for several years but TxDOT started validating the crash data only in 2011. The data in 2011 and the later are considered in this study since they are reliable. The data are available up to May 31, 2017 (at the time when these data were processed, the complete data for a particular month are only available up to May).

The crash data in this study are summarized for each section. The crash data do not account for the direction of roadway in which the crash occurred. The crashes in this study are summarized by the month for every year from January 2011 to May 2017. The period is used to identify the before and after period for each of the project stretches. The months before the markings are applied are marked as −1, the month during which the marking is applied is marked as 0, and the months after which the markings are applied are marked as 1. The crash counts are obtained by using ArcGIS and R by considering the crashes between the start and end DFO of every stretch.

The crashes in this study should be the ones that are related or potentially affected by the presence or absence of markings or affected by the retroreflectivity of the markings. The total count crashes in the study are the crashes that are obtained after excluding the following types of crashes (since these cannot be attributed only to the pavement markings):

- Pedestrian crashes and animal crashes.
- Crashes resulting from driving under influence.
- Bicyclist crashes.
- Work zone crashes.
- Intersection crashes.

The total crashes on a project stretch are calculated after excluding the above types of crashes. In this study, researchers are interested in specific types of crashes (potentially affected by the markings). The crash types included are the following:

- Night/day crashes (light condition).
- Wet/dry crashes (weather conditions).
- Run off road crashes.
- Head on collisions.
- Same direction side swipe collisions.
- Night fatal and injury crashes.
- Night head on crashes.
- Night same direction side swipe collisions.
- Night property damage only crashes.
- Night time ran off road crashes.
- Night time ran off road crashes of fatal and injury crashes.

Weather Database

The weather data were collected to identify the number of rainy days in a year to get an estimation of the exposure of the roadway segments to wet-weather. If the number of rainy days does not vary significantly from year to year analyzing the wet crashes without exposure may be acceptable but if the number of rainy days/precipitation varies greatly year to year, there is a need to establish some control by including exposure in the model.

Researchers sought weather information for all months between January 2011 and May 2017, the period over which the crash data were collected. Researchers used data from https://www.wunderground.com/ (*6*). However, researchers cannot consider these data to be highly reliable since they are not certified weather data. Researchers found certified weather data at https://www.ncdc.noaa.gov/ (*7*), which is the website of National Climatic Data Centre of the National Oceanic and Atmospheric Administration (NOAA). There are shortcomings to these data as well since the data are only available from January 2011 and December 2013. Researchers decided to use the wunderground data and validate them with the NOAA data.

Methodology to Summarize Weather Data

Researchers adopted the following methodology to summarize the weather data:

1. Choosing the weather station to record the data – The wunderground information has weather records from hundreds of stations located all across the United States. The project segments are located at many cities in the Atlanta District, and it would be difficult to identify the weather station of every project segment to summarize the data. The team decided to choose the central city within each county to summarize the data of all the segments in that county. The wunderground website gave the most reliable weather station closest to the city entered, and in all the cases, it was the data of the nearest airport. Researchers decided to use this approach and summarize the weather data based on the nearest airports. In some cases, a group of counties use data from the same airport.

   The county and the respective airport are as below:

   - Bowie County – Texarkana Regional Airport.
   - Cass County – Harrison County Airport.
   - Harrison County – Harrison County Airport.
   - Marion County – Harrison County Airport.
   - Panola County – Harrison County Airport.
   - Camp County – Mount Pleasant Airport.
   - Morris County – Mount Pleasant Airport.
   - Titus County – Mount Pleasant Airport.
   - Upshur County – Gilmer Municipal Airport.

   Whenever the data are missing or seemed incorrect at one of the airports during some of the months, the data from the next nearest airport were taken.

2. Rainfall thresholds – Researchers decided to consider a period (day or night) wet when the total rainfall exceeded 0.1 in. during the day or night period. Any rainy period with total precipitation less than 0.1 in. were considered dry.
3. Rain during day or night – The study summarizes the number of rainy days and rainy nights in every month for the entire study period. The day in this study is defined as the time between the sunrise and sunset. The other time is defined as night. If the rain occurs during the day time, it is counted as a rainy day and if the rain occurs during the night it is counted as a rainy night. When the rain occurs during both day and night, it is counted both as a rainy day and rainy night.

Weather Database Limitations

The amount of rainfall is not considered directly. A threshold of 0.1 in. is used but the effects of a very high rainfall are different than that of a smaller rainfall. The rainfall occurs only during part of the day or part of the night. However the exposure for the entire day/night could be determined by a single hour of precipitation.

CRASH ANALYSIS

Researchers assessed the safety benefits of wet-night pavement markings on the following six crash types, which were considered to be most relevant target crashes: wet-night, dry-night, wet-night fatal and injury, dry-night fatal and injury, wet-night run off road, and dry-night run off road crashes. To increase the robustness and statistical validity of the results, researchers employed two different evaluation methods to assess the safety benefits of wet-weather pavement markings: EB before-after analysis and FB before-after analysis with comparison groups. Each method is described in detail in this report.

There was a total of 135 segments were included in the safety study. Initially 336 segments were considered but were reduced due to lack of complete information. It was assumed that the existing marking prior to implementation of the wet-weather markings were standard pavement markings. The yearly crash data aggregated at each segment for years 2011–2016 were analyzed. The implementation year when wet-weather pavement markings were installed at each segment varies from 2011–2016. Table 8 gives the number of segments and the corresponding mileage for each implementation year.

Table 8. Number of Segments and Miles Included for Each Year.

Implementation Year	Number of Segments	Miles
2011	19	101.5
2013	18	67.1
2014	13	110.0
2015	54	270.1
2016	31	189.0
Total	**135**	**737.7**

In addition to the information on the treatment (implementation of wet-weather pavement markings), researchers incorporated important roadway characteristic variables including number of lanes, median width, lane width, shoulder width, and AADT, as well as the number of rainy nights and the number of rainy days (per year), which play a role of exposure variables for wet crashes, into the analysis. Table 9 provides the descriptive statistics for the roadway variables and weather variables used in the analysis.

Table 9. Descriptive Statistics for Atlanta District Roadway Segments.

Segment Variable	Number of Segments (miles): 135 Segments (737.7 mi)		
	Minimum	**Maximum**	**Average**
Number of Lanes	2	5	2.60
Median Width (ft)	0	40	0.94
Lane Width (ft)	10	13.9	11.6
Shoulder Width (ft)	0	17.1	4.6
AADT	92.3	23348	3788.8
Segment length	0.36	29.13	5.46
Number of Rainy Nights	22	57	35.1
Number of Rainy Days	14	58	36.6

Empirical Bayes Before-After Analysis

The EB methods have been regarded as a statistically defensible methods that can cope with several threats to validity of observational before-after studies including the regression-to-the-mean bias, changes in traffic volumes, and the effects of other unmeasured factors that might change from the before to the after period. In the EB method, safety performance functions (SPFs) developed based on the data from the reference sites are used to estimate the expected crash frequencies at the treated sites had treatments not been applied. Negative binomial regression models, are often used to derive the SPFs. While the success of an EB evaluation largely depends on reliable estimation of SPFs, it is often hard to identify a reference group that is similar enough to the treatment group in the roadway characteristics, weather, and traffic volumes. In this evaluation, daytime crashes obtained from mostly the same sites as nighttime crashes are used as a reference group. The wet-weather pavement markings provide higher levels of retroreflectivity, which is a nighttime visibility property of the markings. The night crashes, especially in wet conditions are the target crashes, thus daytime crashes can serve as the reference group because the different striping material should have no impact on daytime crashes. The 19 segments with the implementation year 2011 and the 31 segments with the implementation year 2016 in Table 8 were excluded from the treatment group in the EB analysis because there were no before data (for 19 segments) or no after data (for 31 segments). However, the daytime crashes from those segments could still be used in developing SPFs based on the reference group data. Appendix B provides steps of the EB procedure used in the current data analysis. Note that in this evaluation, SPFs are calibrated for each year of the before and after periods rather than just for each period.

The negative binomial regression models with indicator variables for year (2011–2016) to control for general trends along with the variables in Table 9 as independent variables were employed to develop SPFs based on the reference group (daytime crashes). Table 10 presents the estimated coefficients for SPFs for wet-day, dry-day, wet-day run off road, and dry-day run off road crashes. The predicted number of nighttime crashes had wet-weather pavement markings not been installed can then be obtained by applying a multiplier α_f (computed as the number of nighttime crashes divided by the number of daytime crashes) to the SPF for daytime crashes. Note that α_f needs to be estimated based on the segments where wet-weather pavement markings were not installed. The crashes for 2011 to 2015 of 31 segments with the implementation year 2016 were used for computing α_f. The estimated multipliers (α_f) for the crash types considered in this study are also included in Table 10.

Table 10. Estimates of Coefficients for SPFs Developed Based on a Reference Group Consisting of Daytime Crashes at 135 Segments (737.7 Miles).

Variable		**Wet-Day**	**Dry-Day**	**Wet-Day Run Off Road**	**Dry-Day Run Off Road**
Year	2011	-4.5080	-5.9117	-4.0821	-1.0371
	2012	-4.7954	-5.7381	-4.3616	-0.7646
	2013	-4.5979	-6.0596	-4.4133	-0.9945
	2014	-4.6857	-5.8905	-4.3858	-0.8240
	2015	-3.8881	-5.8105	-3.9716	-0.4541
	2016	-4.4578	-5.6820	-4.2061	-0.7200
Number of Lanes		-0.1247	-0.0314	-0.0647	-0.1033
Median Width		-0.0414	-0.0123	-0.0326	-0.0027
Lane Width		-0.3039	-0.2038	-0.3291	-0.1922
Shoulder Width		0.0269	-0.0349	0.0605	-0.0147
Log(Segment Length)		1.1315	0.8826	1.0751	0.9990
Log(AADT)		0.9921	0.9079	0.7558	0.5457
Log(Rainy Nights)		-0.4941	0.5168	-0.2035	-0.2397
Log(Rainy Days)		-0.1285	-0.4160	-0.0521	-0.5386
Dispersion		0.7314	0.2792	0.9533	0.0712
Pearson chi-square/DF		0.9777	1.0614	0.9498	1.0471
α_f		0.58	0.56	0.71	0.75

Table 11 presents the results of an EB before-after evaluation for nighttime crashes. For each type of nighttime crash in Table 11, the SPFs estimated from the corresponding daytime crashes after multiplying α_f were used to predict the expected number of crashes had wet-weather pavement markings not been installed. For wet-night fatal injury crashes and dry-night fatal injury crashes, the SPFs from wet-day crashes and dry-day crashes were used to predict the expected number of wet-night fatal injury crashes and dry-night fatal injury crashes, after multiplying α_{FI_NT}=0.43 (where α_{FI_NT} is the number of night fatal injury crashes divided by the total number of night crashes) to the corresponding α_f (0.58 for wet-night crashes and 0.56 for dry-night crashes), because wet-day fatal injury crashes and dry-day fatal injury crashes were not available in the current data. As it is also possible to obtain the multipliers for wet-night fatal injury crashes and dry-night fatal injury crashes as $0.58\times\alpha_{FI\_WN}$ and $0.56\times\alpha_{FI\_DN}$ (where α_{FI_WN}=0.29 is the number of wet-night fatal injury crashes divided by the number of wet-night crashes, and α_{FI_DN}=0.48 is the number of dry-night fatal injury crashes divided by the number of dry-night crashes), a sensitivity analysis with respect to multipliers was conducted for wet-night fatal injury crashes and dry-night fatal injury crashes. In this sensitivity analysis, the SPF for wet-day crashes multiplied by α_{FI_WN}=0.29 and 0.58 and the SPF for dry-day crashes multiplied by α_{FI_DN}=0.48 and 0.56 were used to predict the expected number of wet-night fatal injury crashes and dry-night fatal injury crashes, respectively. The results from the sensitivity analysis are also presented in Table 10 and noted by the * symbol. The results of Table 10 support positive safety effects of wet-weather pavement markings for nighttime crashes. Although the effects are statistically insignificant (except for wet-night fatal injury crashes) due to the limited sample size, it is expected that the effects would become statistically significant as more data are gathered.

Table 11. Results of EB Before-After Evaluations Based on the Nighttime Crashes Obtained from 85 Segments (447.2 Miles).

Crash Type	Crashes in the After Period		$\hat{\theta}$ (S.E.)	95% CI for θ	90% CI for θ	Percent Crash Reduction
	Observed (L)	EB Estimate ($\hat{\pi}$)				
Wet-Night	39	47.4	0.813 (0.155)	(0.510, 1.116)	(0.559, 1.067)	18.7
Dry-Night	116	122.2	0.945 (0.104)	(0.741, 1.150)	(0.774, 1.117)	5.5
Wet-Night Fatal Injury	12	19.7	0.600 (0.188)	(0.232, 0.968)	(0.292, 0.908)	**40.0**
Wet-Night Fatal Injury*	12	15.1	0.778 (0.244)	(0.300, 1.256)	(0.378, 1.178)	22.2
Dry-Night Fatal Injury	45	49.7	0.901 (0.150)	(0.608, 1.194)	(0.656, 1.147)	9.9
Dry-Night Fatal Injury*	45	53.2	0.841 (0.139)	(0.568, 1.114)	(0.612, 1.069)	15.9
Wet-Night Run Off Road	26	31.7	0.810 (0.183)	(0.452, 1.168)	(0.510, 1.109)	19.0
Dry-Night Run Off Road	72	76.6	0.939 (0.116)	(0.711, 1.166)	(0.749, 1.129)	6.1

Notes: 1. EB estimate **(** $\hat{\pi}$ **)** is the predicted number of crashes in the after period had wet-weather pavement markings not been installed; 2. $\hat{\theta}$: estimated index of effectiveness; 3. Percent Crash Reduction $=100(1-\hat{\theta})$; 4. SE: Standard Error; 5. CI: Confidence Interval; 6. *Indicates the results from the sensitivity analysis using different multipliers to adjust for the values of the SPFs from the Wet-Day Run Off Road crashes and Dry-Day Run Off Road crashes for prediction; 7. Statistically significant results with 95 percent confidence level are shown in **bold**.

Full Bayes Before-After Analysis with Comparison Groups

Researchers also analyzed nighttime crashes by employing a FB before-after evaluation method. Although the EB method has been widely used as a safety evaluation tool in observational before-after studies for more than two decades, there are some known limitations of EB:

1. It requires a development and calibration of reliable SPFs based on a fairly large reference group that is assumed to be as similar as possible to the treatment group other than the treatment.
2. Uncertainty in the estimated SPFs is not reflected in the final safety effectiveness estimate of EB. See Park et al. for more in-depth discussions of these issues (*8*).

In the application of EB described in the previous section, the SPFs were estimated based on daytime crashes and then the estimated SPFs were again multiplied by the estimated ratio of nighttime crashes and daytime crashes at the segments where wet-weather pavement marking were not installed until 2016. Because both the SPF coefficients and the estimated ratios (αf) are sample quantities, there are inherent uncertainties associated with them, but the mechanism of EB does not allow those uncertainties to be incorporated into the estimated index of effectiveness or percent crash reduction.

The FB methods have been introduced as an alternative to the EB methods, which can cope with the aforementioned issues of EB (*9*). They have been successfully applied in many observational before-after studies over the last decade (*8, 10,11*). However, there has been some confusion on the concept of FB at the beginning and some of the early applications of FB were actually a hybrid of EB and FB, rather than genuine FB. Park et al. (*8*) developed a FB multivariate approach to before-after evaluation with a comparison group/comparison groups within the formal Bayesian modeling framework (rather than as a hybrid of EB and FB), and provided clear guidelines for step-by-step implementation of the before-after FB evaluations. The FB evaluation of wet-weather pavement markings in this study builds on the basic modeling framework of Park et al. and is further modified so that it can model the nature of the current crash data better (*8*).

Modeling Framework for FB Analysis of Before-After Designs with Comparison Groups

A before-after evaluation design with comparison groups is adapted as a study design for FB analysis to assess safety effectiveness of wet-weather pavement markings. The FB methods generally refer to the estimation methods and can be applied to any study designs including both cross-sectional designs and before-after designs. On the other hand, EB in safety analysis refers to the combination of a specific study design (a before-after design with a reference group) and the estimation method. Although some researchers have been referring to a hybrid of EB and FB (that is basically confined within the same framework of EB with some modifications in estimating the predicted crash count without treatment) as FB (see *12, 13*), the FB methods are much more general and should not be restricted to any single framework.

Park et al. (*8*) generalized multivariate Poisson Lognormal models, developed in Park and Lord (*14*), for jointly modeling the crash frequencies of different severities or crash types obtained from multiple sites (cross-sectional data), to analyze before-after data with a comparison group/comparison groups. In this study, researchers employed a univariate framework with

Poisson-gamma mixture models (instead of Poisson Lognormal models) because Poisson-gamma mixture models can lead to explicit marginal distributions (negative binomial distributions) for observed crash frequencies while Poisson Lognormal models do not have explicit marginal distributions.

The modeling framework of Poisson-gamma mixture models for a fully Bayesian before-after evaluation with comparison groups is presented below. Let y_{it} denote an observation at site *i* ($i = 1, \ldots, I$) during time (year) *t* ($t = 1, \ldots, T$). That is, y_{it} is the number of crashes occurred in year *t* at site *i*. Let *K* be the number of covariates and $X_{it} = \left(1, X_{1it}, \ldots, X_{Kit}\right)$ be a (*K*+*1*)-dimensional vector of covariates. Let $\boldsymbol{\beta} = \left(\beta_0, \beta_1, \ldots, \beta_K\right)'$ denote the (*K*+*1*)-dimensional column vector of the regression coefficients for the crash count. Let υ_{it} denote a vector of yearly random effects corresponding to site *i* and year *t*, explaining extra-Poisson variability. Suppose that, conditional on υ_{it} and $\boldsymbol{\beta} \in \mathrm{R}^{K+1}$, the crash count at site *i* in year *t*, y_{it}, follows a Poisson distribution with mean μ_{it}, in other words:

$$y_{it} \left| \upsilon_{it}, \boldsymbol{\beta} \right. \sim Poisson\left(\mu_{it}\right) \tag{3}$$

where

$$\mu_{it} = \upsilon_{it} \exp\left(X_{it}\boldsymbol{\beta}\right). \tag{4}$$

The y_{it} ’s are independent given the μ_{it} ’s.

$$\upsilon_{it} \sim Gamma\left(\eta, 1/\eta\right) \tag{5}$$

Under the model (3)–(5), the marginal distribution of y_i is given as a negative binomial (NB) distribution with mean λ_i and variance $\lambda_i \left[1 + \lambda_i / \eta\right]$ where $\lambda_i = \exp\left(X_i\boldsymbol{\beta}\right)$.
Let the elements of the covariate vector $X_{it} = \left(1, X_{1it}, \ldots, X_{Kit}\right)$ be:

$X_{1it} = Trt_i$,
$X_{2it} = time$,
$X_{3it} = Trt_i \times time$,
$X_{4it} = \mathbf{I}\left[t > t_{0i}\right]$
$X_{5it} = Trt_i \times \mathbf{I}\left[t > t_{0i}\right]$,
$X_{6it}, \ldots, X_{Kit}$: roadway characteristic variables such as lane width, shoulder width, number of lanes, log(AADT), etc. for the *i*th site,

where
Trt_i =1 if the *i*th site is a treatment site and is zero otherwise.

time = *t*th year in the study period ($t = 1, 2, \ldots, T$).

t_{0i} = year in which the countermeasure was installed at site *i* (for a site in the comparison group, it is defined to be the same year as that for the corresponding treatment group), and $\mathbf{I}[t>t_{0i}]$ is the intervention variable, which takes a value of 1 if *t* belongs to the after period and zero otherwise. Then, Equation 4 can be re-written as follows:

$$\begin{aligned}\mu_{it} = \upsilon_{it}\exp\Big(&\beta_0 + \beta_1 Trt_i + \beta_2 time + \beta_3 Trt_i \times time + \beta_4 \mathbf{I}\left[t > t_{0i}\right] \\ &+ \beta_5 Trt_i \times \mathbf{I}\left[t > t_{0i}\right] + \beta_6 X_{6it} + \cdots + \beta_K X_{Kit}\Big)\end{aligned} \qquad (6)$$

This model can be viewed as a change-point model, which assumes that at the time of implementation, there is a possible change in the level for time at treatment sites that might be attributable to the implementation of the countermeasure. Specifically, the coefficient for $X_{5it} = Trt_i \times \mathbf{I}\left[t > t_{0i}\right]$ represents the cumulative effect of the countermeasure at the treatment site. Note that the comparison group also has the imaginary before and after periods defined the same as those for the matching treatment group although no treatment is applied to sites in the comparison group (see *8*). Note that for each group (Comp: Comparison, Trt: Treatment) and period (B: Before, A: After), Equation (6) can be represented in terms of *mean crash count* versus *time* as follows:

$$\left(\mu_{it}\right)_{Comp,B} = \upsilon_{it}\exp\left(\beta_0 + \beta_2 time + \beta_6 X_{6it} + \cdots + \beta_K X_{Kit}\right)$$

$$\left(\mu_{it}\right)_{Comp,A} = \upsilon_{it}\exp\left(\beta_0 + \beta_4 + \beta_2 time + \beta_6 X_{6it} + \cdots + \beta_K X_{Kit}\right)$$

$$\left(\mu_{it}\right)_{Trt,B} = \upsilon_{it}\exp\left\{\beta_0 + \beta_1 + \left(\beta_2 + \beta_3\right) time + \beta_6 X_{6it} + \cdots + \beta_K X_{Kit}\right\}$$

$$\left(\mu_{it}\right)_{Trt,A} = \upsilon_{it}\exp\left\{\beta_0 + \beta_1 + \beta_4 + \beta_5 + \left(\beta_2 + \beta_3\right) time + \beta_6 X_{6it} + \cdots + \beta_K X_{Kit}\right\}$$

A fully Bayesian analysis of model given in Equations 3–6 requires the (second-level) prior distributions for the parameters, $\beta_0, \beta_1, \beta_2, \ldots, \beta_K$ and η, to be chosen. Implementation of such model calls for simulation-based methods such as a Markov chain Monte Carlo (MCMC) method (*15*,*16*). Once the posterior samples for model parameters and the true averages crash counts (μ_{it}) are obtained, the steps given below (extracted from Park et al. (*8*) can be followed to estimate the effects (θ) of wet-weather pavement markings.

Steps for Implementing Fully Bayesian Before-After Evaluations with Multiple (G) Comparison Groups

Step 1. Specify the hyperparameter values, $\left(c_0, C_0, r_0, R_0\right)$, for prior distribution of model parameters.

Step 2. Obtain the draws of model parameters and the expected annual crash frequency for each site (*i*) and year (*t*) by MCMC.

Step 3. Obtain posterior distributions of crash frequencies during the before period for the treatment group (μ_{TA}), during the after period for the treatment group (μ_{TA}), during the before period for the comparison group (μ_{CB}), and during the after period for the comparison group (μ_{CA}) by taking an average of the expected crash frequencies over the appropriate years and the sites.

Step 4. Obtain a posterior distribution of the ratios of the expected crash frequencies before and after periods for the comparison group (comparison ratio) for the g^{th} comparison group by:

$$R_{C(g)} = \frac{\mu_{CA(g)}}{\mu_{CB(g)}}, \qquad g = 1, \ldots, G .$$

Step 5. Obtain a posterior distribution of the predicted frequencies that would have occurred without treatment in the after period for the g^{th} treatment group as:

$$\pi_{(g)} = \mu_{TB(g)} R_{C(g)} .$$

Step 6. Obtain a posterior distribution of the index of effectiveness (of the countermeasure) for the crashes as:

$$\theta = \frac{\sum_{g=1}^{G} \mu_{TA(g)}}{\sum_{g=1}^{G} \pi_{(g)}} = \frac{\sum_{g=1}^{G} \mu_{TA(g)}}{\sum_{g=1}^{G} \left\{ \mu_{TB(g)} R_{C(g)} \right\}} .$$

Step 7. Obtain the point estimates for β_k and θ as the sample means of corresponding posterior distributions.

Step 8. Obtain the uncertainty estimates for β_k and θ as the sample standard deviations of corresponding posterior distributions.

Step 9. Construct the 95 percent (or 90 percent) credible intervals of β_k and θ using the 2.5th (or 5th) percentiles and the 97.5th (or 95th) percentiles of the corresponding posterior distributions. If the credible interval contains the value 1, then no significant effect has been observed. The credible interval placed below 1 (i.e., the upper limit of the interval is less than 1) implies that the countermeasure has a significant positive effect (i.e., a reduction in crashes) on safety. The credible interval placed above 1 (i.e., the lower limit of the interval is greater than 1) implies that the countermeasure has a significant negative effect (i.e., an increase in crashes) on safety.

FB Analysis of the Effects of Wet-Weather Pavement Markings

The daytime crashes used as reference groups in EB analysis can be used as comparison groups in this FB analysis. Unlike the EB analysis, which cannot incorporate the uncertainty in the SPF estimates from the reference group into the estimated safety effects, the FB analysis can incorporate uncertainty in model parameters into the final safety effectiveness estimate. The treatment group consists of nighttime crashes from segments where wet-weather pavement markings were installed during 2011 through 2016. As in the case of EB analysis, the 19 segments with the implementation year 2011 and the 31 segments with the implementation year 2016 in Table 8 were excluded from the treatment because there were no before data (for 19 segments) or no after data (for 31 segments). Thus, the FB analysis is also based on crashes from 85 segments (447.20 mi) where wet-weather pavement markings were installed in 2013, 2014, or 2015.

Wet-night, dry-night, wet-night fatal injury, dry-night fatal injury, wet-night run off road, and dry-night run off road crashes were fitted by the Poisson-gamma mixture model. This approach included a change point in Equation (6) with predictors, an indicator function specifying whether a segment is a treatment site or a comparison site, time trend (year), treatment by time, an indicator functions specifying whether it belongs to the before or the after period, treatment by implementation date, number of lanes, median width, lane width, shoulder width, log of segment length, log of AADT, log of number of rainy nights, and log of umber of rainy days. The steps for implementing fully Bayesian before-after evaluations with three comparison groups (corresponding to implementation years 2013, 2014, and 2015) presented in the previous section were followed. For the prior distributions of the model parameters, proper but diffuse priors were used to reflect the lack of precise knowledge on the parameters a priori. The inferences on the parameters of interest were made based on the samples from the posterior distribution obtained by the MCMC algorithm coded in MATLAB.

Table 12 summarizes the results from the FB analysis based on 5,000 posterior samples collected for 50,000 iterations by subsampling every 10th sample after the first 10,000 draws are discarded. Note that the regression coefficient for the intervention at the treatment sites, β_5, is negative for all six crash types suggesting that crashes decreased after the intervention (installation of wet-weather pavement markings) for the treatment group compared to those for the comparison group. The estimated index of effectiveness ($\hat{\theta}$) was obtained by accounting for the changes in unmeasured factors between the before and the after period using the comparison ratio following Steps 4–6 described above. The uncertainty estimates for the estimated index of effectiveness, the posterior standard deviation (Std Dev), and 95 percent (or 90 percent) credible interval play the same role as the standard error and the 95 percent (or 90 percent) confidence interval in traditional or EB approaches. From Table 12, there are much larger crash reductions for wet night crashes compared to dry night crashes. There is a statistically significant decrease in Wet Night Fatal Injury crashes with 90 percent probability.

Table 12. Results of FB Safety Evaluation for Wet-Weather Pavement Markings for Wet-Night Crashes and Dry-Night Crashes.

Regression Coefficients	Predictors	Crash Type					
		Wet-Night	**Dry-Night**	**Wet-Night Fatal Injury**	**Dry-Night Fatal Injury**	**Wet-Night Run Off Road**	**Dry-Night Run Off Road**
β_0	Intercept	**-5.8677**	**-5.6012**	**-6.1603**	**-5.3501**	**-4.5155**	**-3.2192**
β_1	Trt	**-0.6669**	**-0.8300**	**-1.5905**	**-1.7669**	**-0.6775**	**-0.3931**
β_2	time	0.0500	-0.0596	0.1238	-0.0220	0.0150	0.0205
β_3	Trt×time	0.0118	0.0613	-0.0699	0.0619	0.0019	0.0109
β_4	I[$t>t_0$]	0.0706	**0.4093**	-0.0521	**0.3411**	-0.0682	0.1739
β_5	Trt×I[$t>t_0$]	-0.2646	-0.3067	-0.2699	-0.3097	-0.0514	-0.0537
β_6	Number of Lanes	-0.0299	-0.0228	-0.0771	-0.0700	0.0694	-0.0517
β_7	Median Width	**-0.0250**	**-0.0134**	-0.0318	**-0.0098**	**-0.0258**	0.0709
β_8	Lane Width	**-0.3729**	**-0.2048**	**-0.2867**	**-0.2108**	**-0.3557**	**-0.1759**
β_9	Shoulder Width	0.0248	**-0.0296**	0.0167	**-0.0378**	**0.0492**	**-0.0393**
β_{10}	Log(Length)	**1.0777**	**0.9421**	**1.1345**	**0.9063**	**1.0491**	**1.0419**
β_{11}	Log(AADT)	**0.8443**	**0.8042**	**0.9140**	**0.8568**	**0.6474**	**0.5049**
β_{12}	Log(Rainy Night)	0.5117	0.3603	0.1747	0.4767	0.1621	0.3626
β_{13}	Log(Rainy Day)	-0.2695	-0.1186	-0.3346	-0.3653	-0.0854	-0.5171
Index of effectiveness		**Wet-Night**	**Dry-Night**	**Wet-Night Fatal Injury**	**Dry-Night Fatal Injury**	**Wet-Night Run Off Road**	**Dry-Night Run Off Road**
$\hat{\theta}$		0.7376	0.9536	*0.5647*	0.9772	0.9019	0.9899
Std Dev		0.1459	0.1230	0.1933	0.1581	0.2913	0.1608
95% Credible Interval		(0.4780, 1.0541)	(0.7427, 1.2182)	(0.2715, 1.0261)	(0.6889, 1.2866)	(0.5085, 1.6440)	(0.7011, 1.3314)
90% Credible Interval		(0.5138, 1.0009)	(0.7684, 1.1716)	(0.3041, 0.9264)	(0.7284, 1.2400)	(0.5423, 1.4659)	(0.7593, 1.2672)
$100(1-\hat{\theta})$: Percent reduction		26.2%	4.6%	*43.5%*	2.3%	9.8%	1.0%

Notes 1. $\hat{\theta}$ is the estimated index of effectiveness; 2. Std Dev represents the posterior standard deviation for θ; 3. $100(1-\hat{\theta})$ denotes the estimated percent crash reduction; 4. Statistically significant results with 95 percent (*90 percent*) probability are shown in **bold** (*in italic*).

SUMMARY

Table 13 summarizes the results from the EB analysis and the FB analysis. The results from both analyses appear to be consistent. Although the uncertainty estimates from FB are larger than those from the EB approach in general, it is a natural consequence of incorporating parameter uncertainty that is ignored in EB into the safety effectiveness estimates of FB.

In conclusion, both evaluation results lend support to positive safety effects of wet-weather pavement markings for nighttime crashes (especially wet-night crashes). Even though currently the effect is statistically significant only for wet-night fatal injury crashes, it is expected that the effects on other types of wet night crashes also become statistically significant as more data are gathered.

Table 13. Comparison of Safety Effectiveness Estimates for Wet-Weather Pavement Markings Obtained by Different Before-After Evaluation Approaches.

	Percent Crash Reduction (Uncertainty Estimate)					
Approach	**Wet- Night**	**Dry- Night**	**Wet-Night Fatal Injury**	**Dry-Night Fatal Injury**	**Wet-Night Run Off Road**	**Dry-Night Run Off Road**
EB	19% (16%)	6% (10%)	**40%** (19%)	10% (15%)	19% (18%)	6% (12%)
FB	26% (15%)	5% (12%)	*44%* (19%)	2% (16%)	10% (29%)	1% (16%)

Note: 1. Uncertainty estimate is standard error for EB and posterior standard deviation for FB; 2. Statistically significant results with 95 percent (*90 percent*) confidence/probability are shown in **bold** (*in italic*).

NEXT STEPS

To build upon the current data collection and analysis, researchers plan to conduct additional work with the data set. This additional work will include the following:

- Expand the data set to incorporate 2017 crashes.
- Attempt to gather 2012 site information data.
- Attempt to include sites that were removed from this analysis due to lack of complete information.
- Evaluate crashes from the sites with no before or after data using an advanced control group design using matching methods and perform generalized linear regression analysis with propensity score matching.
- If enough data are available, conduct analysis for particular marking types.
- If enough data are available, conduct analysis for particular roadway types.
- Incorporate the initial retroreflectivity values.
- Collect in service retroreflectivity values to model degradation.
- Explore benefit-cost analysis.

CHAPTER 4. UPDATE TO THE GUIDE FOR DETERMINING TIME REQUIREMENTS FOR TRAFFIC SIGNAL PREEMPTION AT HIGHWAY-RAIL GRADE CROSSINGS

BACKGROUND

The existing "Guide for Determining Time Requirements for Traffic Signal Preemption at Highway-Rail Grade Crossings" worksheet was developed as a deliverable (*17*) for project 0-4265. The specific objectives of the project were:

- To increase safety at highway-rail grade crossings with nearby traffic signal-controlled highway intersections.
- To reduce the disruption in coordinated traffic signal operations along arterials with railroad preemption.

Researchers achieved these objectives by 1) examining safety, human factors, and operational problems at traffic signals near grade crossings; 2) identifying and evaluating potential solutions to these problems for their effectiveness and applicability in Texas; and 3) combining applicable solutions into a guideline document that will help TxDOT staff recognize and address the special circumstances associated with signals near grade crossings. The guidelines researchers developed were used to evaluate and improve safety and existing operations, and assist in the design future operations at highway-rail grade crossings.

The worksheet that was developed provided specific guidelines for the following safety concerns:

- Safety Concern #1: Abbreviating normal pedestrian clearance and vehicular minimum green times.
- Safety Concern #2: Gates descending on stationary vehicles or trapping vehicles in a queue on the tracks with nowhere to go.
- Safety Concern #3: Failure to consider the longer length and slower acceleration of heavy vehicles.
- Safety Concern #4: Not providing sufficient time between the last vehicle leaving the crossing and the train arriving at the crossing.
- Safety Concern #5: Non-supervised interconnection circuits and failsafe traffic signal controller preempt inputs.
- Safety Concern #6: Preemption over large distances.

The worksheet facilitated addressing these safety concerns by calculating parameters for complex highway-rail interactions at at-grade signalized intersections. Detailed instructions were provided to enable the use of the worksheet.

EXISTING WORKSHEET

The three-page worksheet was developed in PDF format. There were two versions of the PDF worksheet. One was a worksheet that was fillable electronically, which then performed calculations to determine various parameters to determine the preemption needs at the grade crossing being analyzed. Figure 6 illustrates a sample of the worksheet that was created in project 0-4265 and that has been adopted by TxDOT and many agencies in Texas and outside the state.

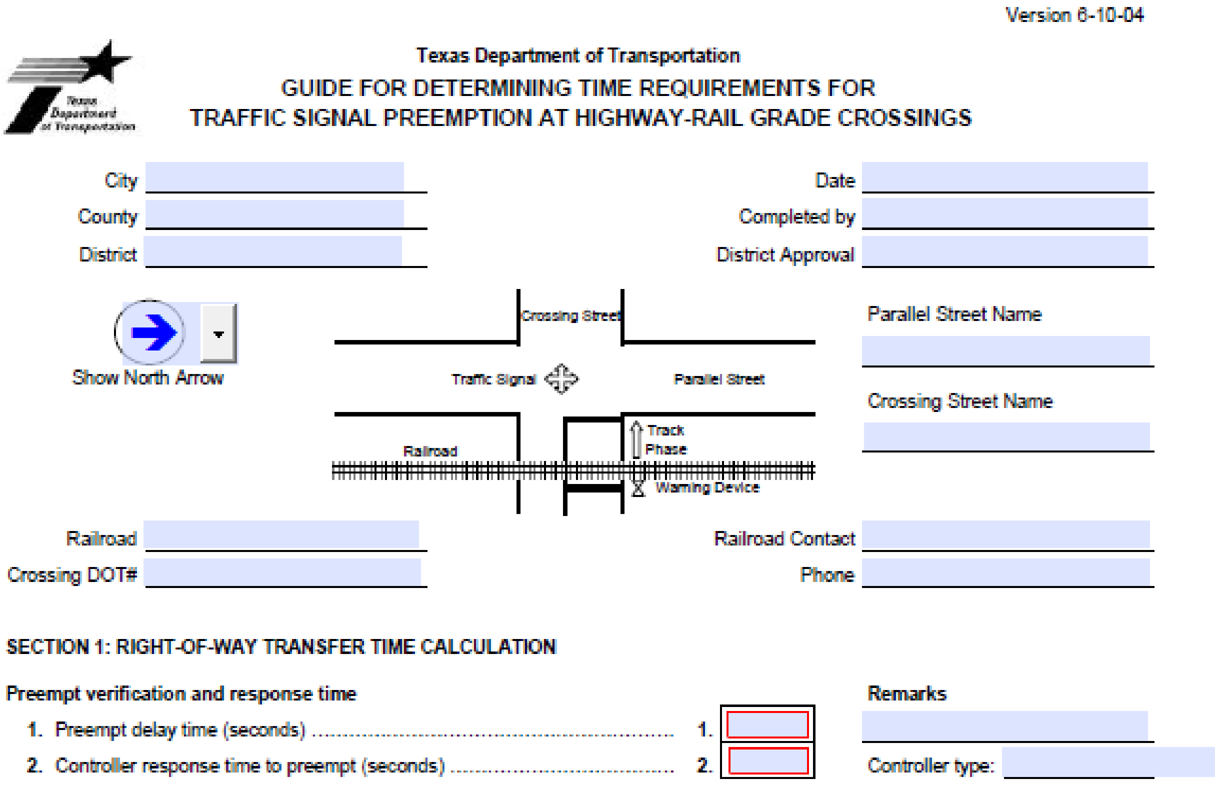


Figure 6. A Sample of the Existing Worksheet Developed in Project 0-4265.

The worksheet is supported by a 16-page instructions document that gives detailed guidance about the using the worksheet. The instruction document includes graphs and tables that the engineer conducting the analysis could use to determine various parameters affecting the preemption requirements at the grade crossing. These include the definition of design vehicles, acceleration characteristics for design vehicles, impact of grade on the acceleration rates for various design vehicles, parameters to estimate acceleration rates over larger distances, and the impact of gate descent on various design vehicles. Figure 7 illustrates a portion of the instructions for filling up the preemption worksheet.

Version 6-10-04

INSTRUCTIONS
for the
Texas Department of Transportation
GUIDE FOR DETERMINING TIME REQUIREMENTS FOR
TRAFFIC SIGNAL PREEMPTION AT HIGHWAY-RAIL GRADE CROSSINGS

USING THESE INSTRUCTIONS

The purpose of these instructions is to assist TxDOT personnel in completing the 2003 Guide For Determining Time Requirements For Traffic Signal Preemption At Highway-Rail Grade Crossings, *also known as the Preemption Worksheet. The main purpose of the Preemption Worksheet is to determine if additional time (advance preemption) is required for the traffic signal to move stationary vehicles out of the crossing before the arrival of the train.*

If you have any questions about completing the Preemption Worksheet, please contact the Mr. David Valdez in the Traffic Operations Division at telephone 512-416-2642 or email DVALDEZ@dot.state.tx.us. For any feedback on the Draft version of the Worksheet or Instructions, please contact Mr. Roelof Engelbrecht from the Texas Transportation Institute at 979-862-3559 or roelof@tamu.edu.

SITE DESCRIPTIVE INFORMATION:

Enter the location for the highway-rail grade crossing including the (nearest) **City**, the **County** in which the crossing is located, and the Texas Department of Transportation (TxDOT) **District** name. When entering the District name, do not use the dated district numbering schema; use the actual district name.

Figure 7. Instructions for Completing the Preemption Worksheet.

The worksheet and the accompanying instructions have been extensively used over the last 10 years by TxDOT not only to analyze existing grade crossings but also propose new grade crossings. These documents were also used by various cities in Texas and other municipalities outside Texas.

OVERVIEW OF THE MODIFICATIONS

With time, as more and more users were using the worksheet, some areas were identified that could improve the worksheet. These included simplifying entering some fields and identifying additional scenarios that the worksheet does not consider. While instructions for most of the entries in the worksheet required a small paragraph, some required about 4–5 paragraphs, consulting graphs and tables. Specifically, Line 24, which determines “Time for design vehicle to accelerate through the design vehicle clearance distance” has over four paragraphs along with a graph and table, making it very complicated for the user to determine the entry. Hence TxDOT had the following overarching objectives for the modifying the worksheet:

- Standardize guidance for items that have a large effect on the requested APT such as minimum green, pedestrian timing, and vehicle gate interaction.
- Provide default values in some fields so that the user need not determine those values. The user, however, can change the values for analyzing grade crossings where the default values may not be suitable.
- Identify entries in the form that are rarely used and can be eliminated.

- Eliminate confusion on how to use the form when there is existing APT.
- Minimize the decision making for someone filling out the form (other than basic measurements).
    - Pedestrian truncation decision.
    - Automatically read from the graphs and tables to minimize user errors and simplify the process.
- Improve the figures in the worksheet.

MODIFICATIONS MADE TO THE WORKSHEET

Based on the multiple meetings with TxDOT engineers and other experts in the industry, the following modifications to the worksheet were listed below. They were described in detail in the FY 2016 report (*18*).

1. Provided guidance on the storage of the worksheet.
2. Defined WB-67 as the default design vehicle with a length of 75 ft.
3. Improved the figures.
4. Addressed various scenarios of a design vehicle turning left toward the track from the street parallel to the tracks:
    a. Design vehicle being the first in the queue for a preempt at the start of green.
    b. Design vehicle already in motion in the queue for a preempt during the green.
    c. Design vehicle blocking the vehicle on the track phase.
    d. Design can or cannot be stored between the parallel road and the railroad tracks.
5. Developed guidance for pedestrian truncation.
6. Identified the duration of green indications for the track phase after the gates are down for various scenarios.

Based on feedback received from TxDOT engineers in the Traffic Operations Division and various districts, the following additional modifying were made to the worksheet and instructions in FY 2017:

1. Calculations for the values have been simplified and verified for accuracy.
2. Some bugs that were observed, were identified and eliminated.
3. A RESET button has been inserted in the form to ensure that prior values in the form are cleared out.
4. The instructions for the form were developed and fine-tuned.

Figure 8 illustrates the final preemption worksheet, and Figure 9 illustrates the instructions to the worksheet. Appendix C includes the latest versions of the worksheet and the instructions.

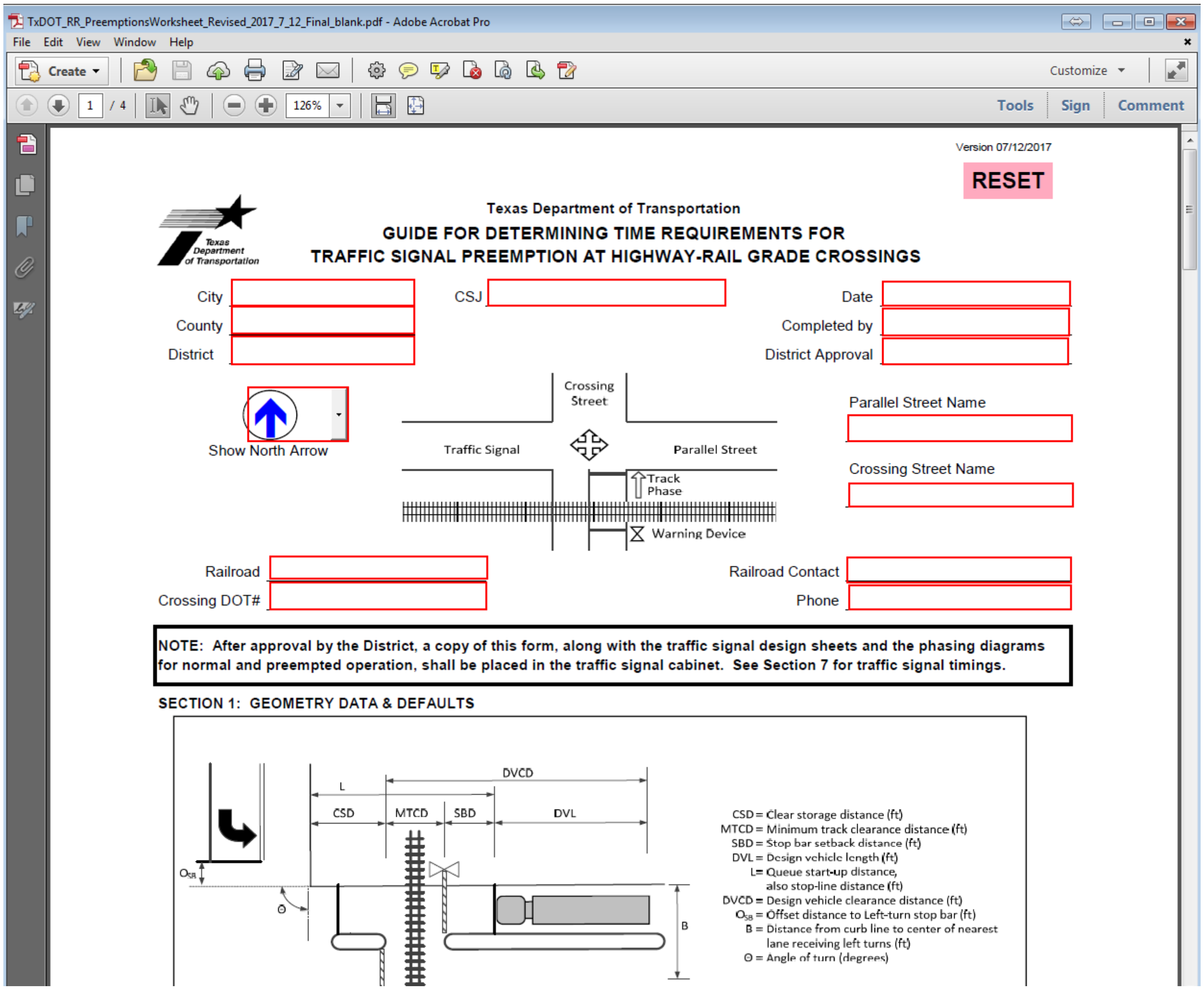
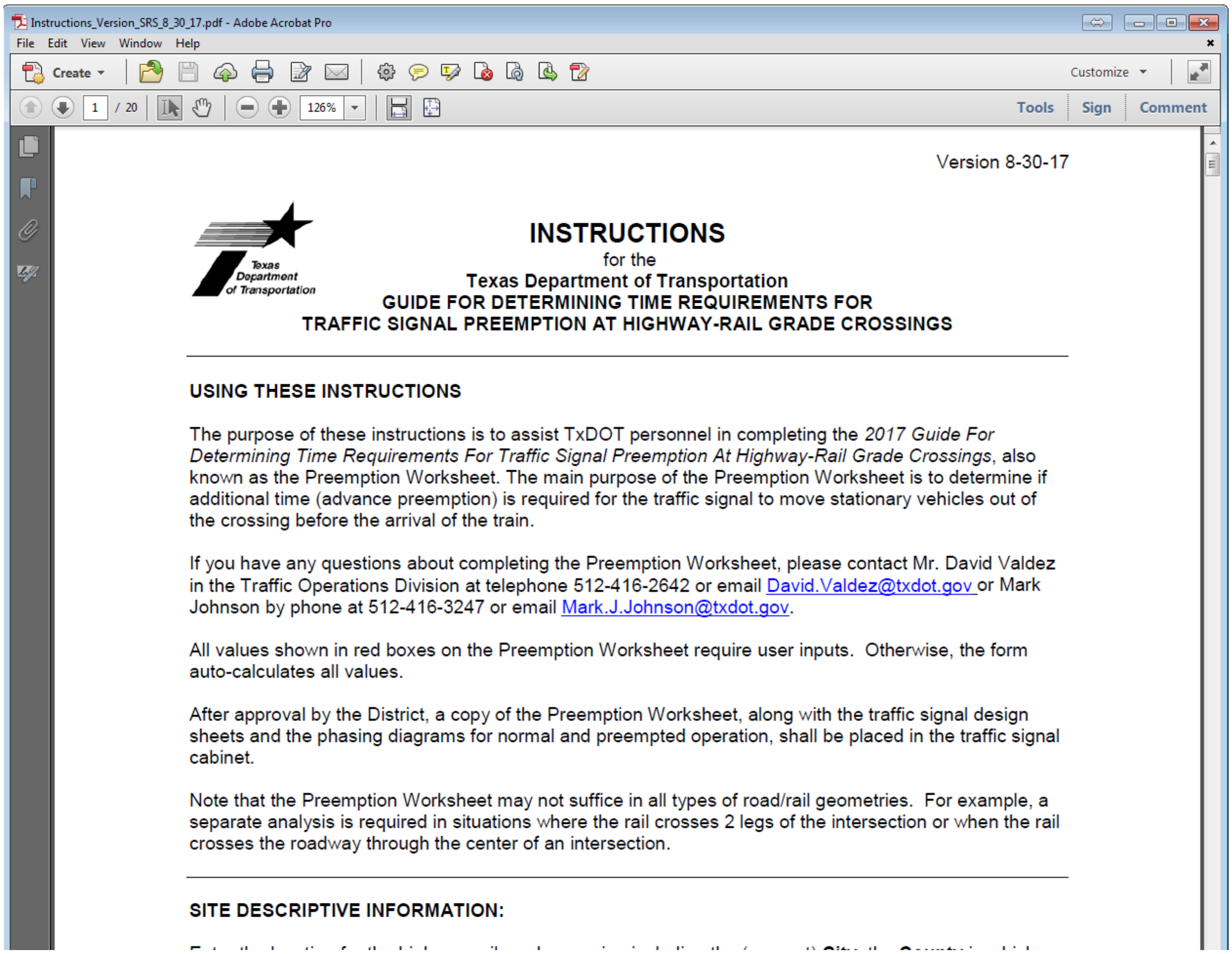


Figure 8. Revised Preemption Worksheet.



Version 8-30-17

INSTRUCTIONS
for the
Texas Department of Transportation
GUIDE FOR DETERMINING TIME REQUIREMENTS FOR
TRAFFIC SIGNAL PREEMPTION AT HIGHWAY-RAIL GRADE CROSSINGS

USING THESE INSTRUCTIONS

The purpose of these instructions is to assist TxDOT personnel in completing the *2017 Guide For Determining Time Requirements For Traffic Signal Preemption At Highway-Rail Grade Crossings*, also known as the Preemption Worksheet. The main purpose of the Preemption Worksheet is to determine if additional time (advance preemption) is required for the traffic signal to move stationary vehicles out of the crossing before the arrival of the train.

If you have any questions about completing the Preemption Worksheet, please contact Mr. David Valdez in the Traffic Operations Division at telephone 512-416-2642 or email David.Valdez@txdot.gov or Mark Johnson by phone at 512-416-3247 or email Mark.J.Johnson@txdot.gov.

All values shown in red boxes on the Preemption Worksheet require user inputs. Otherwise, the form auto-calculates all values.

After approval by the District, a copy of the Preemption Worksheet, along with the traffic signal design sheets and the phasing diagrams for normal and preempted operation, shall be placed in the traffic signal cabinet.

Note that the Preemption Worksheet may not suffice in all types of road/rail geometries. For example, a separate analysis is required in situations where the rail crosses 2 legs of the intersection or when the rail crosses the roadway through the center of an intersection.

SITE DESCRIPTIVE INFORMATION:

Figure 9. Preemption Worksheet Instructions.

SUMMARY

The Guide for Determining the Time Requirements for Traffic Signal Preemption at Highway-Rail Grade Crossings is being updated. A draft has been presented to the reviewers at TxDOT and other preemption experts and is being reviewed. Currently TTI researchers are assessing the feasibility of converting this Excel spreadsheet into PDF format.

CHAPTER 5. DEVELOPING A METHOD TO COMPUTE ROADWAY GRADE FROM STREAMING SURVEY DATA

BACKGROUND

In TxDOT implementation project 5-5439, researchers developed a system to measure horizontal curve geometry using a laptop computer, a GPS receiver, and an electronic ball-bank indicator. This system consists of two software programs: the Texas Roadway Analysis and Measurement Software (TRAMS) executable program, which records and processes data from the devices; and the Texas Curve Advisory Speed (TCAS) spreadsheet program, which uses the curve geometry computations from TRAMS to determine the appropriate curve advisory speed. These programs implement the guidance developed in TxDOT research project 0-5439 to set curve advisory speeds based on a measurement of curve geometry and an estimate of the average truck speed at the midpoint of the curve. The following curve geometry attributes are required as input for TCAS: radius, superelevation rate, and deflection angle.

In TxDOT research project 0-6714, researchers developed guidance to assist practitioners in determining whether a horizontal curve of interest needs a surface treatment to increase the pavement friction supply. The underlying principle of this guidance is margin-of-safety analysis, which involves computing the difference between friction supply (which depends on pavement skid resistance) and friction demand (which depends on vehicle speed, curve radius, superelevation rate, and grade). The guidance was assembled in the form of a spreadsheet program called Texas Curve Margin of Safety (TCMS). The following curve geometry attributes are required as input for TCMS to conduct the margin-of-safety analysis: radius, superelevation rate, deflection angle, and grade.

Because the TCAS and TCMS programs use many of the same data inputs, it is feasible to merge them into a single program. The addition of real-time measurement of roadway grade into the TRAMS program would facilitate use of the merged spreadsheet program to determine curve advisory speed and assess margin of safety. In this project activity, researchers investigated options to extract roadway grade from a data stream. Researchers considered data streams from three types of devices: an inclinometer, a GPS receiver, and a barometric altimeter. Additionally, researchers developed software code and interface features for a spreadsheet program that incorporates the capabilities of TCAS and TCMS. These efforts are expected to continue in TxDOT research project 0-6960, which will commence in September 2017.

LITERATURE REVIEW

Various applications require the collection and analysis of roadway geometry data streams. Some of these applications include maintaining roadway inventory databases, analyzing roadway operational and safety performance, modeling vehicle dynamics for the purpose of assessing occupant comfort, facilitating implementation of connected or automated vehicle technology, and assessing the adequacy of traffic control devices or pavement friction at key points on the roadway such as horizontal curves.

The literature includes discussion of three data sources that can be used to compute roadway grade. These data sources include:

- Inclinometers, which measure angles for one or more axes of a moving vehicle.
- GPS receivers, which provide latitude, longitude, and elevation coordinates at a specified time interval for the purpose of computing velocity and position.
- Barometric altimeters, which use measurements of air pressure and temperature to provide altitude values.

Issues with computing roadway grade from these sources are described in the following three sections.

Inclinometer

An inclinometer can be used to measure angles of rotation about the vehicle's pitch axis as shown in Figure 10. The pitch axis is represented as the circular symbol and runs into the page. Figure 10 illustrates the combined contributions of roadway grade (θ) and vehicle pitch (λ) to the reading that would be obtained from an inclinometer.

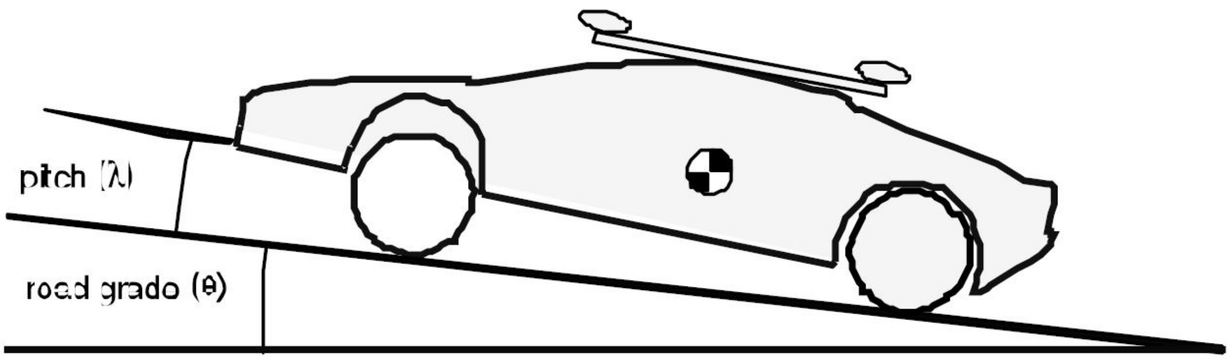


Figure 10. Vehicle Pitch Angle and Grade (*19*).

Hashemi et al. described the relationship between vehicle pitch angle, roadway grade, longitudinal acceleration and braking, and vehicle characteristics including suspension stiffness and weight distribution (*20*). They modeled the vehicle using sprung-mass kinematics and derived a system of equations to describe vehicle roll and pitch. They observed that the equations describing rotation about the pitch axis include both angles θ and λ, such that it is necessary to use height sensors on all four wheels to measure suspension displacements and de-couple the two angles. By using an instrumented vehicle that included these sensors and other components, they were able to measure roadway grade.

Hashemi et al. tested the accuracy of their grade measurements by deliberately inducing error through longitudinal acceleration and braking, and found that the grade calculation algorithm still produced accurate results. In one example test run, they showed that the difference in suspension displacement between the front and rear axes was about 3–4 in. Their instrumented vehicle had a length of about 9 ft between the two axes; for this length, a 3.5-in. increase in displacement between the axes would increase the vehicle pitch angle by about 1.9°, or overestimate the roadway grade by about 3.2 percent (i.e., a grade of 5 percent would be computed as 8.2 percent). Their instrumented vehicle system was able to compensate for this error source by measuring longitudinal acceleration and incorporating it into the calculations.

GPS Receiver

GPS receivers provide data sentences with standard formats that are specified by the National Marine Electronics Association (*21*). These standard formats provide information relevant to position, speed, heading, and altitude. It is generally suggested that GPS provides excellent position data, but the error for GPS-measured altitude is roughly 50 percent higher than for GPS-measured position (*22*).

Vahidi et al. investigated methods to use GPS-measured altitude (or elevation) data to compute roadway grade (*23*). They showed that the raw elevation data are generally accurate but often has errors of about 25 ft due to measurement fluctuations (see Figure 11). To improve the computed altitude measurements, they suggested a modeling method called recursive least squares with forgetting. Figure 12 shows their comparison of estimated and actual grade.

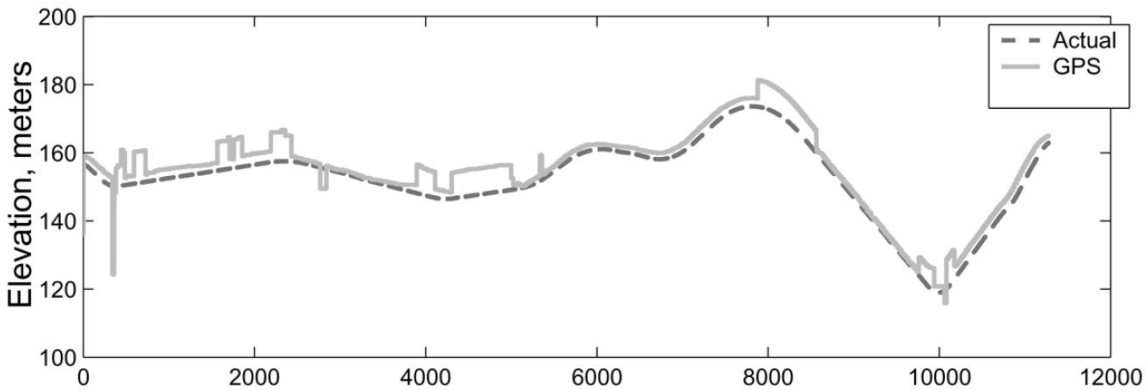


Figure 11. Accuracy of Raw GPS-Measured Elevation (*23*).

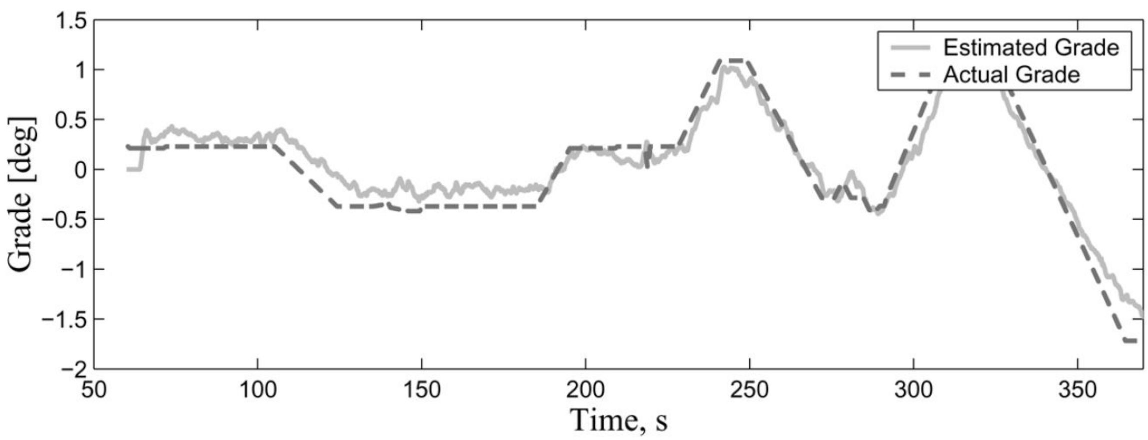


Figure 12. Accuracy of Improved GPS-Computed Grade (*23*).

Barometric Altimeter

Yazdani et al. evaluated the accuracy of roadway grade measurements obtained from a GPS receiver with a built-in barometric altimeter (*24*). They discussed several error sources that can combine to yield uncertainty as high as about 33 ft for GPS-measured horizontal position and observed that elevation measurements are typically subject to more random error than horizontal position measurements. To improve elevation estimates, they suggested using a GPS receiver with a barometric altimeter. They also acknowledged that differential GPS systems can yield more precise elevation measurements but at significant cost (e.g., $15,000 for one system).

Yazdani et al. suggested that a combination GPS/barometer device can be used to obtain accurate measurements of grade if repeated measurements are conducted on the roadway. They suggested that for roadway segments of about 500 ft in length, a sample size of 22 data collection runs would yield grade confidence intervals of about ±0.5 percent. The device that they used had a refresh rate of 1 Hz; they suggested that combining data from multiple devices to increase the number of data points would allow for fewer runs to be conducted. Yazdani et al. were seeking repeatability in raw location measurements, and while absolute errors in the range of 33 ft may occur between runs when notable time has elapsed or atmospheric conditions have changed, it is likely that measurements over a short sampling period would vary significantly less, such that computations of relative change in elevation would be repeatable.

DISCUSSION

Based on the results of the literature review, consideration of costs for the different data collection devices, and the need to ensure portability of the data collection system, researchers explored the use of elevation data from a GPS receiver with a built-in barometric altimeter. Researchers conducted a preliminary analysis of altitude change measurements obtained from a combined GPS/barometer at 1-Hz intervals, and also aggregated and averaged the measurements as follows:

- Aggregate the GPS-based elevation change estimates over a 3-second interval.
- Aggregate the barometer-based elevation change estimates over a 3-second interval.
- Average the 3-second-aggregated measurements from both data sources.

Researchers obtained ground-truth elevation data from flood zone and topography maps compiled by the Federal Emergency Management Agency (FEMA) and made available by the City of Bryan (*25*). Figure 13 shows a comparison of the averaged-and-aggregated GPS/barometer data (yellow placemarks) with the FEMA-reported elevations (pink placemarks) for a roadway segment. The roadway segment is an uphill approach to a highway overpass, with an elevation increase of about 10 ft occurring in the 200-ft distance between the turn-lane pavement markings, a grade of about 5 percent. The GPS/barometer measurements near the first 290-ft elevation placemark show elevation increases in the range of 10.0–12.6 ft.

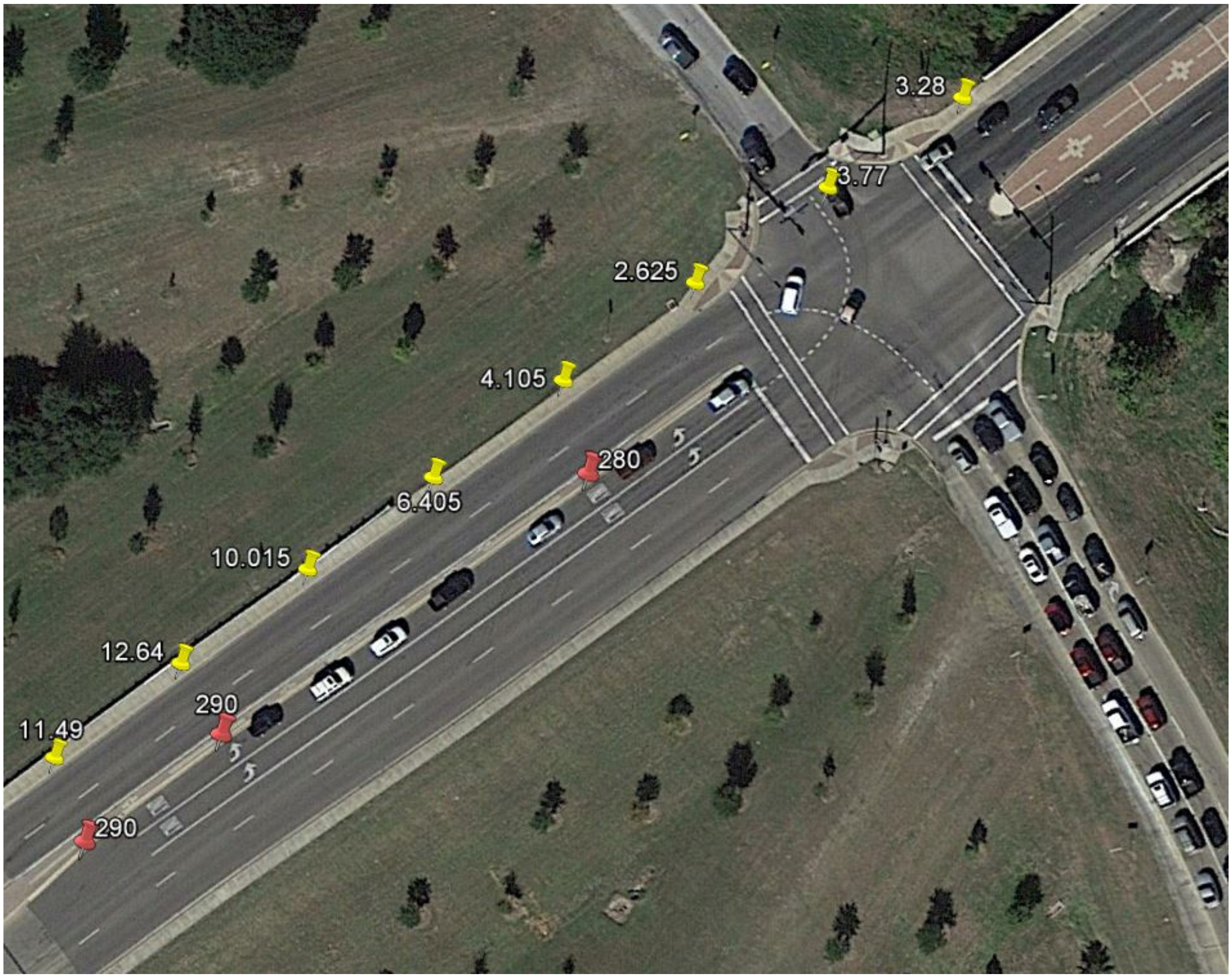


Figure 13. Comparison of Ground-Truth Elevation Data and Computed Elevation Data.

Researchers conducted an analysis of error in altitude measurements obtained from a barometric altimeter. The basis for calculating elevation change using a barometer is the hypsometric equation, which derives directly from the hydrostatic equation for the ideal gas law (*26*). This relationship is shown in Equation 7:

$$\Delta h = \frac{R_d \overline{T}_v}{g} \ln\left(\frac{p_2}{p_1}\right) \tag{7}$$

where:

Δh	=	change in altitude, m.
p_1, p_2	=	pressures at points 1 and 2 ($p_1 < p_2$), Pa.
g	=	gravitational constant (= 9.81 m/s^2).
R_d	=	dry air gas constant (= 287.04 J/kg/K).
$\overline{T}v$	=	average virtual temperature of the air layer of thickness Δh, K.

After substituting constants, Equation 7 simplifies into the working version shown in Equation 8:

$$\Delta h = 16.256 \ln\left(\frac{p_2}{p_1}\right)\left(\overline{T}_{Farenheit} + 459.67\right) \tag{8}$$

The differential of the above measurement is defined as follows:

$$err(\Delta h) = \frac{\partial \Delta h}{\partial \overline{T}_{Farenheit}} \Delta \overline{T}_{Farenheit} + \frac{\partial \Delta h}{\partial p_1} err(p_1) + \frac{\partial \Delta h}{\partial p_2} err(p_2) \tag{9}$$

$$err(\Delta h) = 16.256\left[\ln\left(\frac{p_2}{p_1}\right)\Delta \overline{T}_{Farenheit} + \left(\frac{1}{p_1} + \frac{1}{p_2}\right)\left(\overline{T}_{Farenheit} + 459.67\right) err(p)\right] \tag{10}$$

The last term is the maximum case of the general form $\left(\frac{err(p_1)}{p_1} \pm \frac{err(p_2)}{p_2}\right)$.

The following sensitivity analysis explores various scenarios of the application of the calculations outlined above, when a stream of digital data is obtained from a barometer. An estimate for the amount of error in a pressure reading was obtained from a barometer from an Android S6 device running the Physics Toolbox Suite app that was developed by Vieyra Software (*27*). The device was placed untouched on a horizontal surface for 10 seconds while the signal from the barometer was recorded. The standard deviation of this reading was measured and used as an estimate of *err*(*p*). The estimate so obtained was 0.0185 hPa. The plot in Figure 14 illustrates an approximate linear relation between *Delta p* obtained from the barometer and corresponding difference in height. This figure shows the ground truth relationship in black, the 95th-percentile frequency band around this ground truth (in dashed gray lines), and two sample noisy signals from the barometer.

From this figure, it can be seen that each *Delta p* reading is roughly expected to have the same uncertainty (about 0.32 m, or about 1 ft corresponding error in height difference) when temperature is known with certainty.

Figure 15 shows the same relationship as Figure 14 but with a temperature of 110°F, instead of 75°F. The two more relevant results of this change are: the error of the estimation of change in height increases slightly from 0.32 m to 0.34 m; and the slope of the relationship increased slightly as well. At 75°F, this slope is 13 m per 1.5 hPa, but at 110°F it changes to about 14 m per 1.5 hPa. Hence, the results of the estimation method clearly depend on the temperature at which the data are collected.

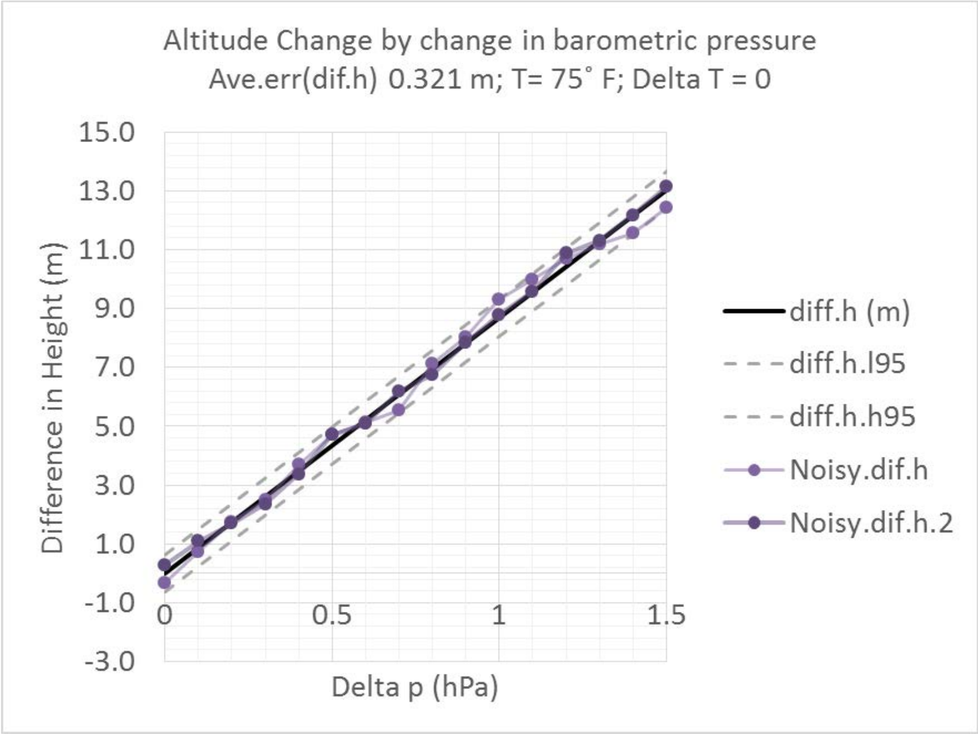


Figure 14. Change in Altitude Estimated from a Change in Barometric Pressure with Two Sample Noisy Signals.

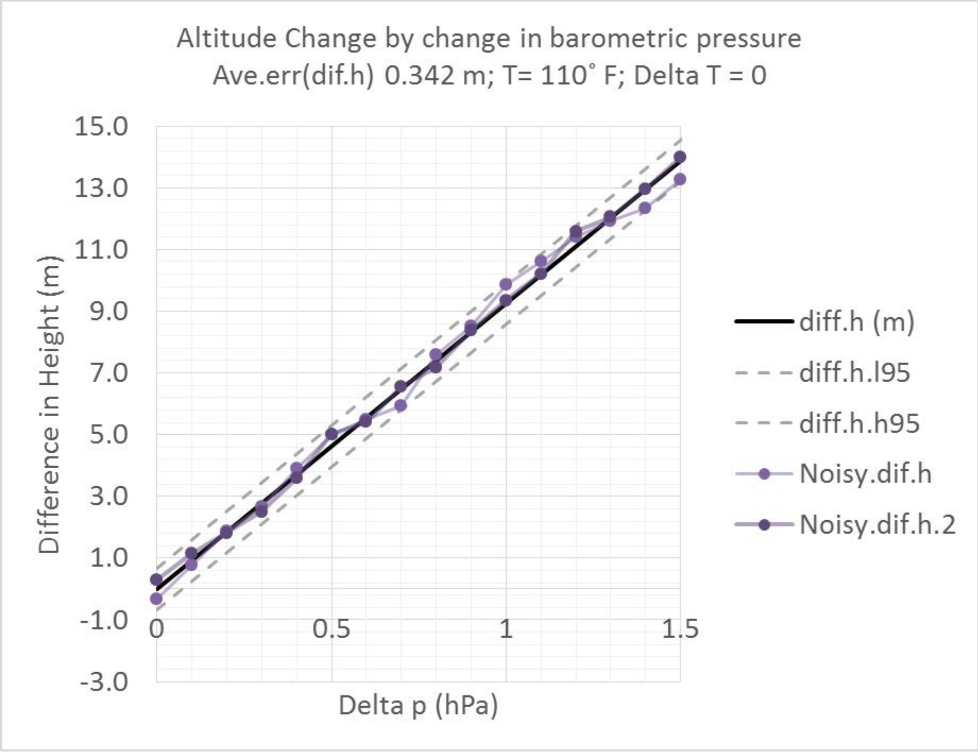


Figure 15. Example of Change in Altitude Estimated from a Change in Barometric Pressure at 110°F with Two Sample Noisy Signals.

Finally, Figure 16 shows the same relationship as Figure 14 but allowing for uncertainty associated with temperature. This uncertainty has been fixed at ±10°F. The two most crucial effects of this change are that the average error of the estimation almost doubles from 0.32 m to 0.61 m, and that more uncertainty is associated with bigger changes in pressure (i.e., the dispersion increases with increased pressure difference). These two results indicate that having a simultaneous stream from a good instantaneous thermometer would be desirable to increase accuracy of this estimation. The combined GPS/barometer device used by researchers to conduct the exploration of field-measured elevation values (see Figure 13) reports altitude, pressure, and temperature on its digital display.

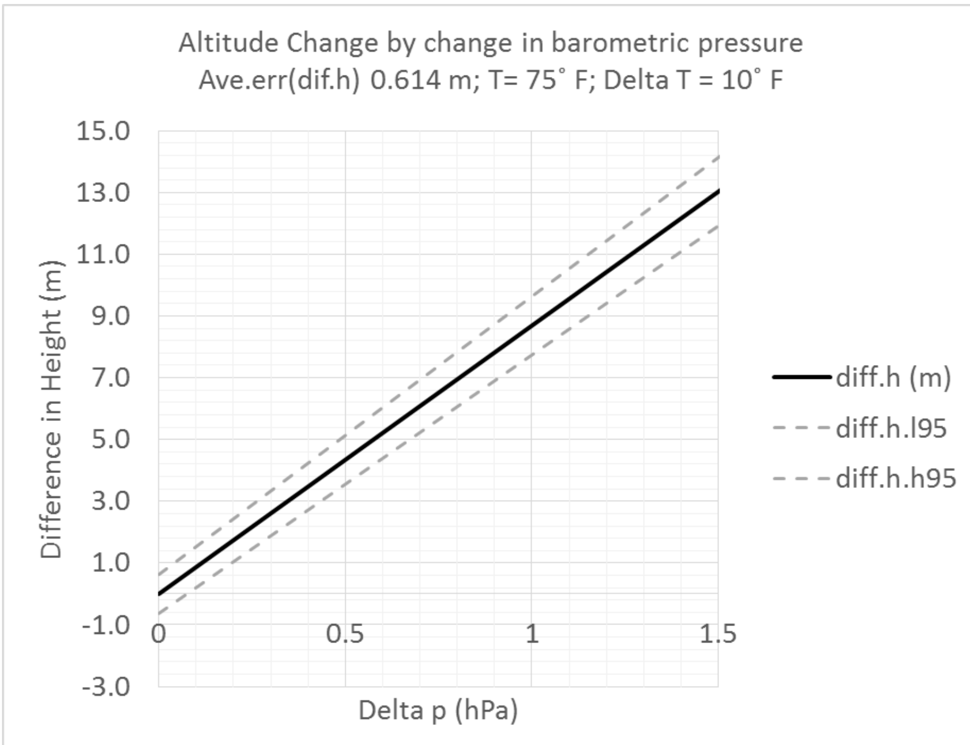


Figure 16. Change in Altitude Estimated from a Change in Barometric Pressure.

NEXT STEPS

Researchers anticipate continuing the development of the updated TRAMS and spreadsheet programs with the following efforts in TxDOT research project 0-6960:

- Conduct a statistical analysis to compare GPS/barometer-measured altitude changes with ground-truth measurements.
- Identify and purchase either a combination GPS/barometer device or a stand-alone barometer that is capable of streaming barometer data in real time.
- Update the TRAMS program and interface to accept and log the streamed GPS and barometer data at a rate of 10 Hz to facilitate more precise identification of curve beginning and ending points.
- Update the spreadsheet programs to incorporate the new altitude data and combine the capabilities of both TCAS and TCMS.

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APPENDIX A. FHWA CORRESPONDENCE

U.S. Department
of Transportation
Federal Highway Administration

1200 New Jersey Avenue, SE
Washington, D.C. 20590

In Reply Refer to:
HOTO-1

JUN -7 2017

Michael Chacon, P.E.
Director, Traffic Operation Division
Texas Department of Transportation
125 East 11th Street
Austin, TX 78701

Dear Mr. Chacon:

With this correspondence, we wish to advise you of the status of your 2016 revised request for experimentation with the display of commercial logos on electronic changeable message signs, 2(09)-83 (E) – Sponsorship Acknowledgment on CMS—TX. Your concept was proposed as an alternative to the provisions of the *Manual on Uniform Traffic Control Devices for Streets and Highways* that address acknowledgment of sponsors of highway-related services on highway signs.

The FHWA is unable to approve your request at this time based on the applicable statutes and regulations. We appreciate your interest in exploring innovative concepts and your understanding in this decision.

Sincerely yours,

Mark R. Kehrli
Director, Office of Transportation
Operations

copy: Mr. A. Alonzi, FHWA Texas Division

APPENDIX B. STEPS OF THE EB PROCEDURE EMPLOYED IN THE ANALYSIS OF ATLANTA NIGHTTIME CRASH DATA

Step 1. Develop an SPF and estimate the regression coefficients and a negative binomial dispersion parameter (*k*) using data from the reference group.

Step 2. Estimate the expected number of crashes $E\left(\kappa_{it}\right)$ for each year (*t*) in the before period at each treatment site using the SPF developed in Step 1 multiplied by α_f (the ratio of nighttime crashes and daytime crashes on segments where wet-weather pavement markings were not installed) as follows:

$$\hat{E}\left(\kappa_{it}\right) = \alpha_f \hat{\mu}_{it}$$

where $\hat{\mu}_{it}$ is the mean crash frequency estimated from the SPF developed based on the daytime crashes.

Step 3. Compute the sum of the annual SPF predictions during the before period at each treatment site by summing $E\left(\kappa_{it}\right)$ for before years:

$$P_i = \sum_{t=1}^{t_{0i}-1} \hat{E}\left(\kappa_{it}\right)$$

where t_{0i} denotes the year during which the countermeasure was installed at site *i*.

Step 4. Obtain an estimate of the expected number of crashes (M_i) before implementation of the countermeasure at each treatment site and an estimate of variance of M_i. The estimate M_i is given by combining the sum of the annual SPF predictions during the before period (P_i) with the total count of crashes during the before period as follows:

$$M_i = w_i P_i + \left(1 - w_i\right) K_i$$

where K_i is the total crash counts during the before period at site *i* and the weight w_i is given by:

$$w_i = \frac{1}{1 + kP_i}$$

where *k* is the estimated dispersion parameter of the Negative Binomial regression model developed in Step 1. An estimated variance of M_i is given by:

$$V\hat{a}r\left(M_i\right) = \left(1 - w_i\right) M_i$$

Step 5. Determine SPF predictions $\hat{E}\left(\kappa_{iy}\right)$ for each year in the after period at each treatment site, and compute C_i, the ratio of the sum of the annual SPF predictions for the after period (Q_i) and the sum of the annual SPF predictions for the before period (P_i):

$$C_i = \frac{\sum_{t=t_{0i}+1}^{T} \hat{E}\left(\kappa_{iy}\right)}{\sum_{t=1}^{t_{0i}-1} \hat{E}\left(\kappa_{it}\right)} = \frac{Q_i}{P_i}$$

Step 6. Obtain the predicted crashes ($\hat{\pi}_i$) and its estimated variance during the after period that would have occurred without implementing the countermeasure. The predicted crashes ($\hat{\pi}_i$) are given by:

$$\hat{\pi}_i = C_i M_i$$

The estimated variance of $\hat{\pi}_i$ is given by:

$$V\hat{a}r\left(\hat{\pi}_i\right) = C_i^2 V\hat{a}r\left(M_i\right) = C_i^2\left(1 - w_i\right) M_i$$

Step 7. Compute the sum of the predicted crashes over all sites in a treatment group of interest and its estimated variance by:

$$\hat{\pi} = \sum_{i=1}^{I} \hat{\pi}_i$$

$$V\hat{a}r\left(\hat{\pi}\right) = \sum_{i=1}^{I} V\hat{a}r\left(\hat{\pi}_i\right)$$

where I is the total number of sites in a treatment group of interest.

Step 8. Compute the sum of the observed crashes over all sites in a treatment group of interest by:

$$L = \sum_{i=1}^{I} L_i$$

where L_i is the total crash counts during the after period at site i.

Step 9. The index of effectiveness of the countermeasure is estimated by:

$$\hat{\theta} = \frac{L}{\hat{\pi}\left(1 + \mathrm{var}\left(\hat{\pi}\right) / \hat{\pi}^2\right)}$$

The percent change in the number of crashes at site *i* is given by $100\left(1-\hat{\theta}\right)$. If $\hat{\theta}$ is less than 1, then the countermeasure has a positive effect on safety.

Step 10. Compute the estimated variance and standard error of the index of effectiveness and the approximate 95 percent confidence interval for θ. The estimated variance and standard error of the index of effectiveness are given by:

$$V\hat{a}r\left(\hat{\theta}\right)=\hat{\theta}^2\frac{\left(1/L+V\hat{a}r\left(\hat{\pi}\right)/\hat{\pi}^2\right)}{\left(1+V\hat{a}r\left(\hat{\pi}\right)/\hat{\pi}^2\right)^2}$$

$$s.e.\left(\hat{\theta}\right)=\sqrt{V\hat{a}r\left(\hat{\theta}\right)}$$

The approximate 95 percent confidence interval for θ is given by adding and subtracting 1.96 $s.e.\left(\hat{\theta}\right)$ from $\hat{\theta}$. If the confidence interval contains the value 1, then no statistically significant effect has been observed. This does not mean that a safety effect does not exist, so all indices that were estimated are reported in this paper to show a complete picture of safety effects. A confidence interval placed below 1 (i.e., the upper limit of the interval is less than 1) implies that the countermeasure has a significant positive effect (i.e., a reduction in crashes) on safety. The confidence interval placed above 1 (i.e., the lower limit of the interval is greater than 1) implies that the countermeasure has a significant negative effect (i.e., an increase in crashes) on safety.

APPENDIX C. GUIDE FOR DETERMINING TIME REQUIREMENTS FOR TRAFFIC SIGNAL PREEMPTION AT HIGHWAY-RAIL GRADE CROSSINGS: INSTRUCTIONS AND WORKSHEET

INSTRUCTIONS

for the

Texas Department of Transportation

GUIDE FOR DETERMINING TIME REQUIREMENTS FOR TRAFFIC SIGNAL PREEMPTION AT HIGHWAY-RAIL GRADE CROSSINGS

USING THESE INSTRUCTIONS

The purpose of these instructions is to assist TxDOT personnel in completing the *2017 Guide For Determining Time Requirements For Traffic Signal Preemption At Highway-Rail Grade Crossings*, also known as the Preemption Worksheet. The main purpose of the Preemption Worksheet is to determine if additional time (advance preemption) is required for the traffic signal to move stationary vehicles out of the crossing before the arrival of the train.

If you have any questions about completing the Preemption Worksheet, please contact Mr. David Valdez in the Traffic Operations Division at telephone 512-416-2642 or email David.Valdez@txdot.gov or Mark Johnson by phone at 512-416-3247 or email Mark.J.Johnson@txdot.gov.

All values shown in red boxes on the Preemption Worksheet require user inputs. Otherwise, the form auto-calculates all values.

After approval by the District, a copy of the Preemption Worksheet, along with the traffic signal design sheets and the phasing diagrams for normal and preempted operation, shall be placed in the traffic signal cabinet.

Note that the Preemption Worksheet may not suffice in all types of road/rail geometries. For example, a separate analysis is required in situations where the rail crosses 2 legs of the intersection or when the rail crosses the roadway through the center of an intersection.

SITE DESCRIPTIVE INFORMATION:

Enter the location for the highway-rail grade crossing including the (nearest) **City**, the **County** in which the crossing is located, and the Texas Department of Transportation (TxDOT) **District** name. When entering the District name, do not use the dated district numbering schema; use the actual district name.

Next, enter the **CSJ** (Control Section Job) of the project, if applicable.

Next, enter the **Date** the analysis was performed, your (the analyst's) name next to "**Completed by**," and the status of the **District Approval** for this crossing.

To complete the reference schematic for this site, place a **North Arrow** in the provided circle to correctly orient the crossing and roadway. Record the name of the **Parallel Street** and the **Crossing Street** in the spaces provided, and remember to include any "street sign"/local name for the streets as well as any state/US/Interstate designation (i.e., "FM 1826," "SH 71," "US 290," "Interstate 35 [frontage]"). You may wish to note other details on the intersection/crossing diagram as well, including the number of lanes and/or turn bays on the intersection approach crossing the tracks and any adjacent land use.

Enter the **Railroad** name, **Railroad Contact** person's name, and **Phone** number for the responsible railroad company. Finally, record the unique 7-character **Crossing DOT#** (6 numeric plus one alphanumeric characters) for the crossing. This section also has a RESET button which clears all the data that is entered in the worksheet. This button can be used to ensure that the form is blank before starting the analysis of a new crossing.

SECTION 1: GEOMETRIC DATA & DEFAULTS

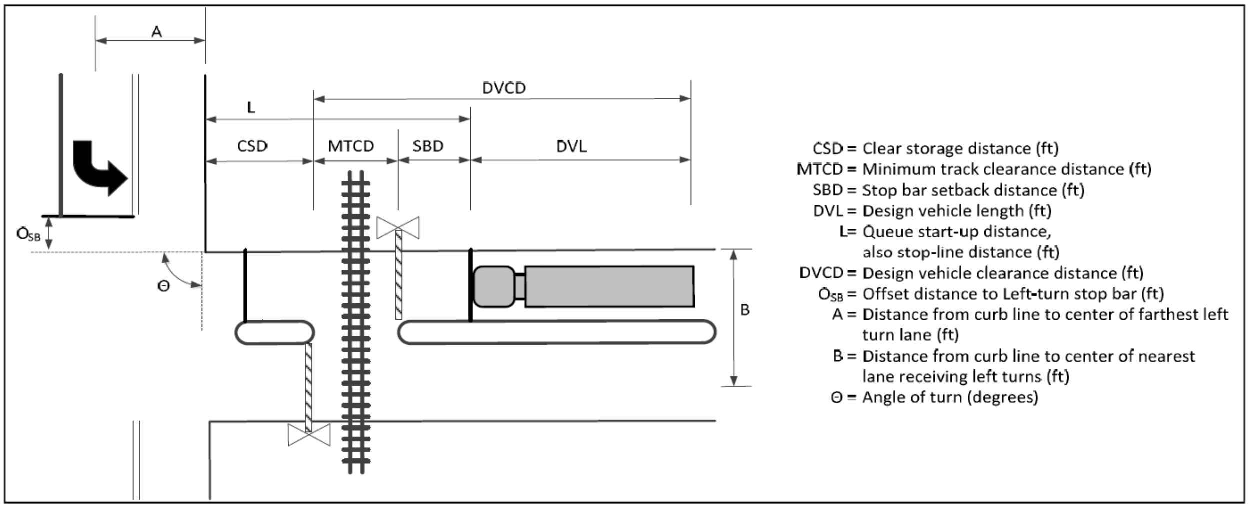

Figure 1. Geometric Data at the Grade Crossing.

GEOMETRIC DATA FOR CROSSING

Line 1. Record the **Clear storage distance** (CSD in Figure 1), in feet, as the shortest distance along the crossing street between the edge of the grade crossing nearest the signalized intersection—identified by a line parallel to the rail 6 feet (2 m) from the rail nearest to the intersection—and the edge of the street or shoulder of street that parallels the tracks. If the normal stopping point on the crossing street is significantly different from the edge or shoulder of parallel street, measure the distance to the normal stopping point. For angled (i.e., non-perpendicular) railroad crossings, always measure the distance along the inside (centerline) edge of the leftmost lane or the distance along the outside (shoulder) edge of the rightmost lane, as appropriate, to determine the shortest clear storage distance and record that value.

Line 2. Minimum track clearance distance (MTCD in Figure 1), in feet, is the length along the highway at one or more railroad tracks, measured from the portion of the railroad crossing automatic gate arm farthest from the near rail to 6 feet (2 m) beyond the tracks measured perpendicular to the far rail. For angled (i.e., non-perpendicular) railroad crossings, always measure the distance along the inside (centerline) edge of the leftmost lane or the distance along the outside (shoulder) edge of the rightmost lane, as appropriate, to determine the longest minimum track clearance distance and record that value. Where flashing light signals are used without automatic gates or where passive traffic control devices are used, the MTCD is measured from the railroad stop line.

Line 3. Stop bar setback distance (SBD in Figure 1) is the distance from the railroad warning device to the stop bar installed on the lanes approaching the crossing. If no stop bar is present, a value of zero (0) should be entered in the worksheet. The default value is 8' as per *Texas Manual on Uniform Traffic Control Devices (TMUTCD)* Section 8B.28, Para. 3.

Line 4. Width of receiving approach (B in Figure 1) is the distance from the curb or edge of the roadway for the track phase and the mid-point of the inside lane (centerline) into which vehicles turn from the roadway parallel to the tracks.

Line 5. Offset distance of left turn stop bar (O_{SB} in Figure 1) is the distance between the line extended from the curb or edge of the roadway for the track phase and the stop bar of the nearest turn lane that is used by left turning vehicles to turn onto the tracks.

Line 6. Approach grade (0 if approach is on downgrade) is the percent grade of the track phase approach. The grade is typically an average of grade measurements taken at the stop bar, 30' from the stop bar, and 60' from the stop bar on the side of the tracks opposite of the signalized intersection. See tractor-trailer location in Figure 1.

Line 7. Angle of turn at Intersection is the angle (Φ in Figure 1) that a vehicle turning left towards the tracks has to make to make the left turn maneuver. This is an approximate value and can be assumed to be 90 degrees for intersections that are nearly perpendicular.

DESIGN VEHICLE DATA

Line 8. Select Design Vehicle: The form selects the Interstate Semi-Truck as the default design vehicle. This design vehicle is recommended for all non-residential locations. For selecting a different design vehicle, check the appropriate box.

Line 9. Design vehicle length, in feet, is the length of the design vehicle, the longest vehicle permitted by road authority statute on the subject roadway. The worksheet has a default value of 75 feet based on the selection of the Interstate Semi-Truck as the default design vehicle. Selection of a different design vehicle will autofill the appropriate design vehicle length as noted in Table 1 as stated by the *AASHTO Green Book (A Policy on Geometric Design of Highways and Streets)*. The use of the Interstate Semi-Truck is recommended for non-residential locations. Note that additional truck length may be added on line 9a for special circumstances where a 75 foot vehicle is not sufficient.

Table 1. AASHTO Design vehicle lengths and heights.

Design Vehicle Type	Symbol	Length (ft)
School Bus	S-BUS 40	40
Intermediate Truck	WB-50	55
Interstate Semi-Truck	WB-67	75

Line 10. Total Design vehicle length (DVL in Figure 1), in feet, is the total length of the vehicle and is obtained by adding Line 9 and Line 9a if used.

Line 11. Centerline turning radius of design vehicle: This is the radius of the design vehicle which is auto filled based on the selection of the design vehicle.

Line 12. Passenger car vehicle length. This value is auto filled (19 feet) and is used in calculations for Line 31.

SECTION 2: RIGHT-OF-WAY TRANSFER TIME CALCULATION

Preempt Verification and Response Time

Line 13. The **preempt delay time** is the amount of time, in seconds, that the traffic signal controller is programmed to wait from the initial receipt of a preempt call until the call is "verified" and considered a viable request for transfer into preemption mode. Preempt delay time should be a whole number value entered into the controller unit for purposes of preempt call validation, and may not be available on all manufacturer's controllers.

Line 14. Unlike preempt delay time (Line 13), which is a value entered into the controller timing parameters, **controller response time to preempt** is the time that elapses while the controller unit electronically registers the preempt call. The controller manufacturer should be consulted to find the correct value (in seconds) for use here. Record the controller manufacturer and firmware version to the right.

Line 15. The sum of Line 13 and Line 14 is the **preempt verification and response time**, in seconds. It represents the number of seconds between the receipt at the controller unit of a preempt call issued by the railroad's grade crossing warning equipment and the time the controller software actually begins to respond to the preempt call (i.e., by transitioning into preemption mode).

Worst-Case Conflicting Vehicle Time

Line 16. Minimum green time during right-of-way transfer is the minimum number of seconds that any existing phase will display a green indication before the controller unit will terminate the phase through its yellow change and red clearance intervals and transition to the track clearance green interval. A default value of 5 seconds is provided to meet driver expectations and reduce the chance of a rear end collision at the intersection, but may be reduced if necessary. Note that this value is not the same as a minimum green value during normal operation; it only comes into play when a preempt call is received from the railroad. The TMUTCD allows for this minimum green time to be set as low as 0 for cases where the preemption time required from the railroad needs to be reduced.

If the current phase is green when a preempt call is received from the railroad, the amount of green time already displayed is subtracted from the minimum green time during right-of-way transfer. For example, if a phase has already been green for 3 seconds when the preempt call is processed from the railroad, the indication will stay green for 2 additional seconds (assuming a minimum green right-of-way transfer time of 5 seconds). Or, if the existing phase has been green for 5 seconds or more, the controller will terminate the green immediately (assuming a minimum green right-of-way transfer time of 5 seconds).

Line 17. If any additional green time is preserved beyond the preempt minimum green time for the worst-case vehicle phase, it should be entered here as **Other green time during right-of-way transfer**. Given the time-critical nature of the transition to the track clearance green interval during preempted operation, this value is usually zero except in unusual circumstances. One situation where other green time may be present is when a trailing green overlap is used on the worst-case vehicle phase, and the controller unit is set up to time out the trailing green overlap on entry into preemption.

Line 18. Yellow change time is the required yellow change interval time during right-of-way transfer prior to the track clearance. Section 4D.13 of the *TMUTCD* states that the normal yellow change interval shall not be shortened or omitted during the transition into preemption control. Since most controller manufacturers require one input value for yellow change time during right-of-way transfer, the highest yellow change time value for all normal phases is recommended.

Line 19. Red clearance time is the required red clearance interval time during right-of-way transfer prior to transition to track clearance. Section 4D.13 of the *TMUTCD* states that the normal red clearance interval shall not be shortened or omitted during the transition into preemption control. Since most controller manufacturers require one input value for red clearance time during right-of-way transfer, the highest red clearance time value for all normal phases is recommended.

Line 20. Worst-case conflicting vehicle time is the sum of lines 16 through 19. It will be compared with the worst-case conflicting pedestrian time to determine which of vehicle or pedestrian phase times are most critical in their impact on warning time requirements during the transition to the track clearance green interval.

Worst-case Conflicting Pedestrian Time

Line 21. Minimum walk time during right-of-way transfer (seconds) is the minimum pedestrian walk indication time. The *TMUTCD* permits the shortening (i.e. truncation) or complete omission of the pedestrian walk interval. A default value of 0 is inserted on the form and is recommended.

Line 22. Pedestrian clearance time during right-of-way transfer (seconds) is the clearance (i.e., flashing don't walk indication) time. The *TMUTCD* permits the shortening (i.e. truncation) or complete omission of the pedestrian clearance interval. A zero value allows for the most rapid transition to the track clearance green interval. See the Appendix for recommendations for pedestrian clearance time.

Line 23. Enter a **Yellow change time** if the pedestrian clearance interval does not time simultaneously with the yellow change interval of the corresponding vehicular phase; enter zero if does. Also, note than not all traffic signal controllers allow simultaneous timing of the pedestrian clearance interval and the yellow change time. Simultaneous timing of the pedestrian clearance interval and the yellow change interval (i.e. a zero value on line 23) allows for the most rapid transition to the track clearance green interval. If a non-zero value is entered, make sure to enter the yellow change time of the vehicular phase associated with your worst-case pedestrian phase. This value may not be the same value you enter on Line 19, since the worst-case pedestrian phase may not be the same as the worst-case vehicular phase.

Line 24. Enter a **Red clearance time** if the pedestrian clearance interval does not time simultaneously with the red clearance interval of the vehicular phase associated with your worst-case pedestrian phase; enter zero if does. Local policies will determine if this is allowed. Also, note than not all traffic signal controllers allow simultaneous timing of the pedestrian clearance interval and the red clearance interval. Simultaneous timing of the pedestrian clearance interval and the red clearance interval (i.e. a zero value on line 14) allows for the most rapid transition to the track clearance green interval. If a non-zero value is entered, make sure to enter the red clearance time of the vehicular phase associated with your worst-case pedestrian phase. This value may not be the same value you enter on Line 20, since the worst-case pedestrian phase may not be the same as the worst-case vehicular phase.

Line 25. Add lines 21 through 24 to calculate your **Worst-case conflicting pedestrian time**. This value will be compared to the worst-case conflicting vehicle time to determine whether vehicle or pedestrian phase times are the most critical in their impact on warning time requirements during the transition to the track clearance green interval.

Worst-case Conflicting Vehicle or Pedestrian Time

Line 26. The **Worst-case conflicting vehicle or pedestrian time** (in seconds) is computed in the worksheet by comparing lines 20 and 25 and filling in the larger of the two values.

Line 27. Calculate the **Right-of-way transfer time** by adding lines 15 and 26. The right-of-way transfer time is the maximum amount of time needed for the worst case condition, prior to display of the track clearance green interval.

SECTION 3: QUEUE CLEARANCE TIME CALCULATION

This section calculates the time required to clear the queue off the tracks before the arrival of the train. This time is impacted by the time required by a truck turning left from the street parallel to the tracks and potentially impeding the first vehicle in the queue to start moving upon the onset of the track phase. The time to clear the vehicles off the tracks is then determined by calculating the time required for the design vehicle to start moving and then moving through the design vehicle clearance distance. The section starts by asking the user if the preemption timing should account for a left turn design vehicle turning left onto the tracks.

Line 28. Are there left-turns towards the tracks?: The user has to select a box 'Yes' or 'No' to indicate whether to consider a left turn design vehicle turning left onto the tracks. If the user selects 'Yes', values for boxes 29, 31, and 32 are automatically calculated and filled in. In some cases, a value of 'No' may be selected in order to minimize the amount of preemption time requested from the railroad.

Line 29. Distance travelled by the truck during left-turn towards the tracks: This is the distance (LTL) travelled by the truck while making the turn. This calculation is used later in Line 31 and is a function of the centerline turning radius of the left turn design vehicle and the angle of the street from which the left turn design vehicle is making the left turn and is illustrated in the equation below and in Figure 2:

$$LTL = \pi R\theta/180$$

Where:
R = the centerline turning radius of the left turning design vehicle (line 11), and
Θ = angle of the street from which the left turn design vehicle is making the left turn (line 7)

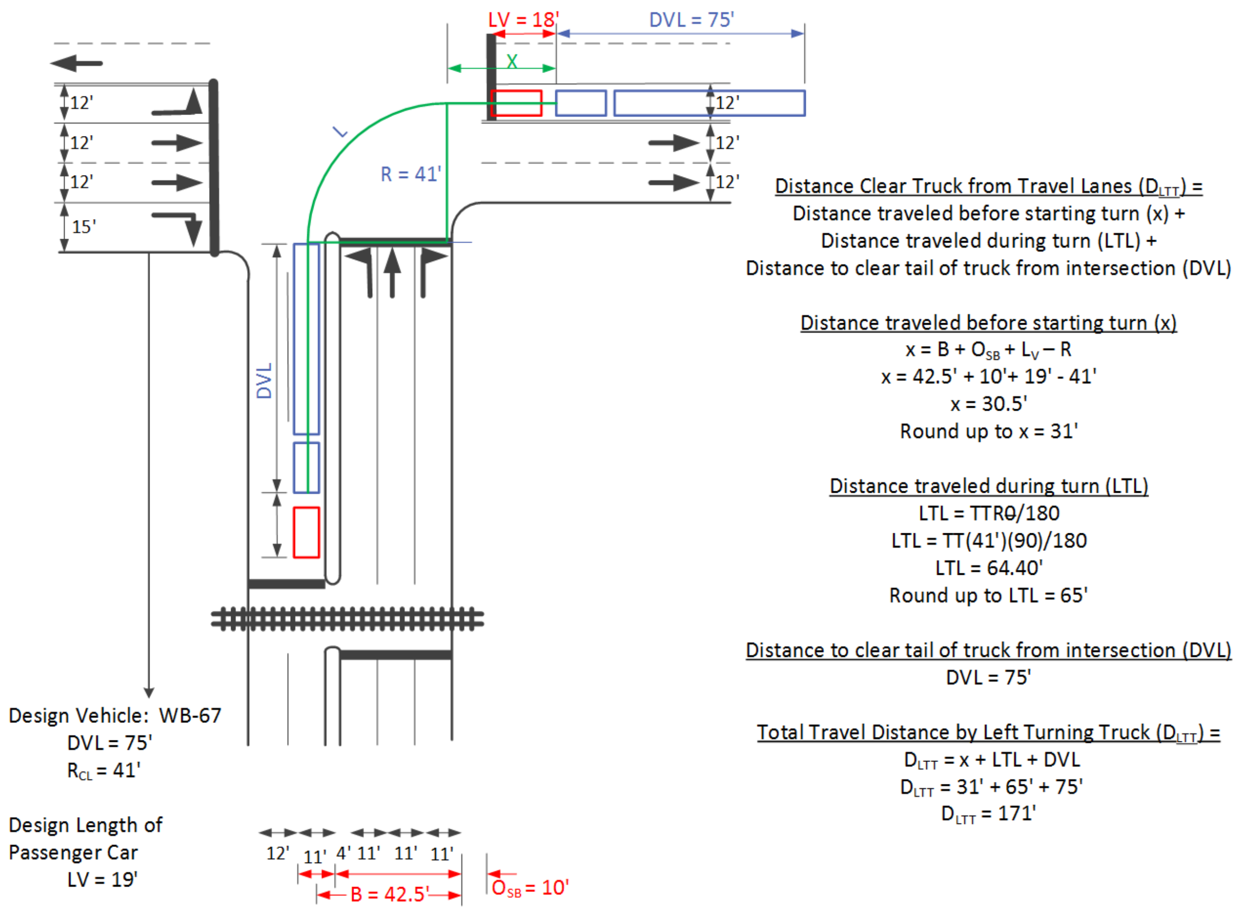

Figure 2. Sample calculation of the distance required to clear left-turning truck from travel lanes on track clearance phase.

Line 30. Travel speed of left turning truck: This is the default speed of the left turn design vehicle turning left to cross the tracks, assumed to be 10 mph.

Line 31. Distance required to clear left-turning truck from travel lanes on track clearance approach (feet): This distance is calculated from the section on geometric data and section on design vehicle. This is the distance travelled by vehicle design vehicle, assumed to be the same length as the design vehicle, making a left turn. The formula used is as follows:

$$Distance\ required = (Line\ 4 + Line\ 5 + Line\ 12 - Line\ 11) + Line\ 29 + Line\ 10$$

Where:

Line 4 = Width of receiving approach (feet)
Line 5 = Offset distance of left turn stop bar (feet)

Line 12 = Passenger car length (feet)
Line 11 = Centerline turning radius of design vehicle (feet)
Line 29 = Distance travelled by the truck during the left turn (feet)
Line 10 = Total design vehicle length (feet)

Line 32. Additional time required to clear left-turning truck from travel lanes on track clearance approach (seconds): This is the time required to travel the distance calculated in Line 31 for the expected speed stated in Line 30. The duration of yellow and red clearance timings during right-of-way transfer (Lines 18 and 19) are subtracted out, under the assumption the left turn movement began at the onset of the yellow change interval. The time required is calculated as follows:

$$Additional\ time\ required = [\frac{(line\ 31 * 3600)}{(line\ 30 * 5280)} - line\ 18 - Line\ 19]$$

Line 33. Worst-case Left Turning Truck time (seconds): This presents the impact of a left turning truck on the clearance of queue during the track phase. This value is automatically calculated and is set to '0' if 'No' is selected on line 28.

Line 34. Queue start-up distance (L in Figure 1), in feet, is the maximum length over which a queue of vehicles stopped for a red signal indication at an intersection downstream of the crossing must get in motion so that the design vehicle can move out of the railroad crossing prior to the train's arrival. It is automatically calculated by the following formula:

$$L = Line\ 1 + Line\ 2 + Line\ 3$$

Line 35. Time required for the design vehicle to start moving (seconds) is the time elapsed between either (a) the time the left turning vehicle completes its turn so it is not blocking track clearance if 'Yes' is checked on line 28 or (b) the beginning of track clearance green if 'No' is checked on line 28, and the time the design vehicle, which is located at the stop bar of the railroad crossing on the opposite side from the signalized intersection, begins to move. This elapsed time is based on a "shock wave" speed of 20 feet per second and a 2 second start-up time (the additional time for the first driver to recognize the signal is green and move his/her foot from the brake to the accelerator). The time required for the design vehicle to start moving is calculated, in seconds, as 2 plus the queue start-up distance, L (Line 34) divided by the wave speed of 20 feet per second. The time required for the design vehicle to start moving is a conservative value taking into account the worst-case vehicle mix in the queue in front of the design vehicle as well as a limited level of drive inattentiveness. This value may be overridden by local observation, but care must be taken to identify the worst-case (longest) time required for the design vehicle to start moving.

Line 36. Design vehicle clearance distance (DVCD in Figure 1) is the length, in feet, which the design vehicle must travel in order to enter and completely pass through the railroad crossing's minimum track clearance distance (MTCD). It is the sum of the minimum track clearance distance (Line 2), stop bar setback distance (Line 3), and the total design vehicle's length (Line 10).

Line 37. The **Time for design vehicle to accelerate through the design vehicle clearance distance (DVCD) on level terrain**: This is the amount of time required for the design vehicle to accelerate from a stop and travel the complete design vehicle clearance distance over a level approach. This time value, in seconds, is automatically calculated by the worksheet and filled in. The formulae used for these calculations are represented by graphs in Figure 3.

Line 38. For slower acceleration on uphill grade: If the approach over which the design vehicle has to accelerate over DVCD is an uphill grade, a factor is automatically calculated by the worksheet based on the entry in Line 6. The calculation for this factor is based on the formulae used to generate the factors in Table 2. For example, with a DVCD of 80 feet and a WB-50 intermediate truck design vehicle on a 4% uphill, the (interpolated) factor from Table 2 is 1.30. Therefore, the estimated time required for the design vehicle to accelerate through the DVCD will be 12.2 x 1.30 = 15.86 seconds, or 15.9 seconds rounded up to the next higher tenth of a second.

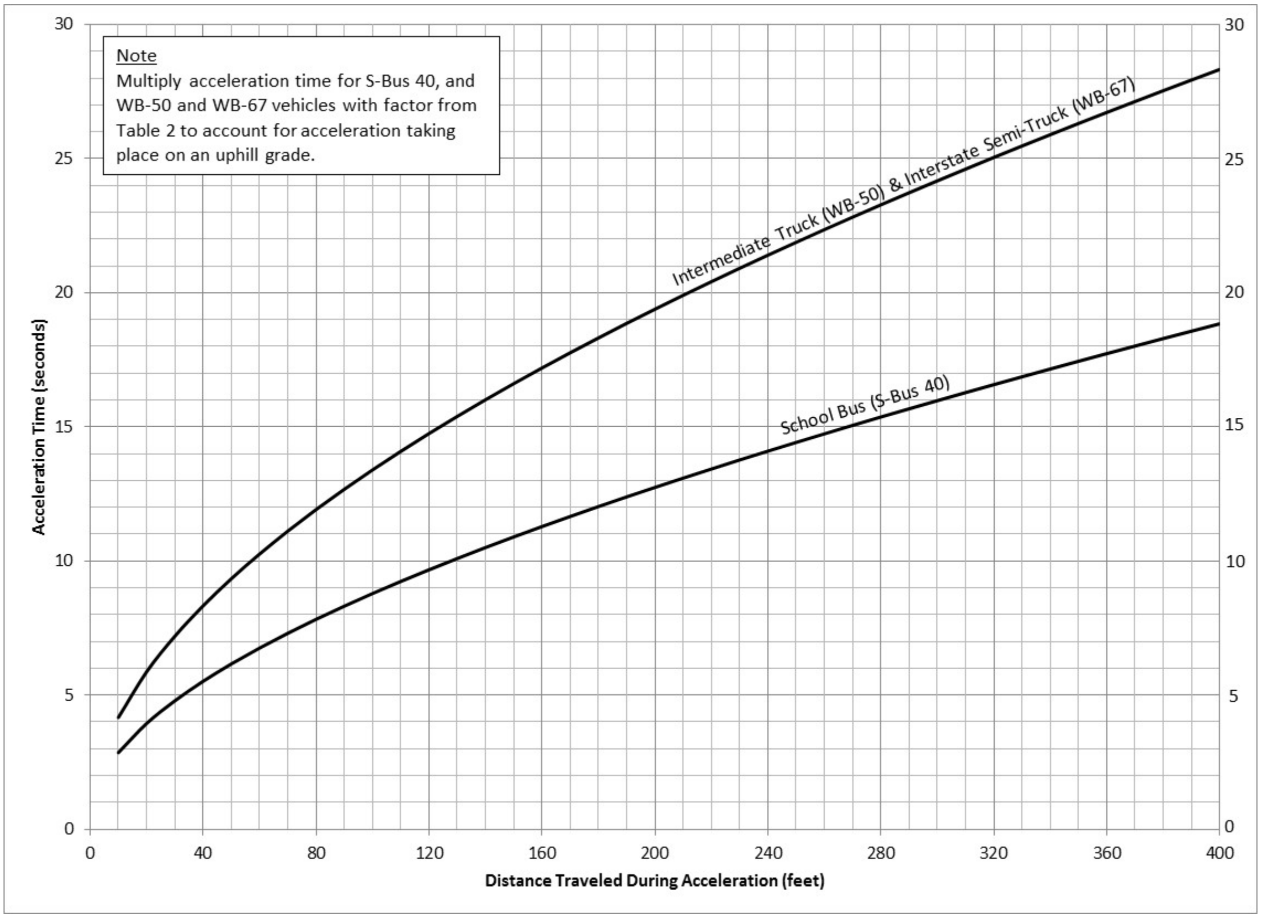

Figure 3. Acceleration time over a fixed distance on a level surface.

Table 2. Factors to account for slower acceleration on uphill grades

Acceleration Distance (ft)	Design Vehicle and Percentage Uphill Grade									
	School Bus (S-BUS 40)					Intermediate Truck (WB-50) and Interstate Semi-Truck (WB-67)				
	0-1%	**2%**	**4%**	**6%**	**8%**	**0%**	**2%**	**4%**	**6%**	**8%**
25	1.00	1.01	1.10	1.19	1.28	1.00	1.09	1.27	1.42	1.55
50	1.00	1.01	1.12	1.21	1.30	1.00	1.10	1.28	1.44	1.58
75	1.00	1.02	1.13	1.23	1.33	1.00	1.11	1.30	1.47	1.61
100	1.00	1.02	1.14	1.25	1.35	1.00	1.11	1.31	1.48	1.64
125	1.00	1.03	1.15	1.26	1.37	1.00	1.12	1.32	1.50	1.66
150	1.00	1.03	1.16	1.28	1.40	1.00	1.12	1.33	1.52	1.68
175	1.00	1.03	1.17	1.29	1.42	1.00	1.12	1.34	1.53	1.70
200	1.00	1.04	1.17	1.30	1.43	1.00	1.13	1.35	1.54	1.72
225	1.00	1.04	1.18	1.32	1.45	1.00	1.13	1.35	1.56	1.74
250	1.00	1.04	1.19	1.33	1.47	1.00	1.13	1.36	1.57	1.76
275	1.00	1.05	1.20	1.34	1.49	1.00	1.14	1.37	1.58	1.77
300	1.00	1.05	1.20	1.35	1.50	1.00	1.14	1.37	1.59	1.79
325	1.00	1.05	1.21	1.36	1.52	1.00	1.14	1.38	1.60	1.81
350	1.00	1.05	1.22	1.37	1.54	1.00	1.15	1.39	1.61	1.82
375	1.00	1.06	1.22	1.38	1.55	1.00	1.15	1.39	1.62	1.84
400	1.00	1.06	1.23	1.40	1.57	1.00	1.15	1.40	1.63	1.85

Line 39. Time for design vehicle to accelerate through DVCD (seconds) adjusted for grade: This value is calculated by multiplying the time for the design vehicle to accelerate through DVCD with the factor to account for the uphill grade (i.e. multiply Line 37 with Line 38).

Line 40. Queue clearance time is the total amount of time required (after the right-of-way transfer time) to begin moving a queue of vehicles through the queue start-up distance (L, Line 34) and then move the design vehicle from a stopped position at the far side of the crossing completely through the minimum track clearance distance (MTCD, Line 2). This value is the sum of the time required for the worst-case left turning truck to complete the turn movement (Line 33), the design vehicle to start moving (Line 35), and the time for design vehicle to accelerate through the design vehicle clearance distance (Line 39).

SECTION 4: MAXIMUM PREEMPTION TIME CALCULATION

Line 41. Right-of-way transfer time, in seconds, recorded on Line 27. The right-of-way transfer time is the maximum amount of time needed for the worst case condition, prior to display of the track clearance green interval.

Line 42. Queue clearance time, in seconds, recorded on Line 40. Queue clearance time starts simultaneously with the track clearance green interval (i.e. after right-of-way transfer), and is the time required for the design vehicle to start up and move completely out of the minimum track clearance distance.

Line 43. Desired minimum separation time is a time “buffer” between the departure of the last vehicle (the design vehicle) from the railroad crossing (as defined by the minimum track clearance distance) and the arrival of the train. Separation time is added for safety reasons and to avoid driver discomfort. If no separation time is provided, a vehicle could potentially leave the crossing at exactly the same time the train arrives, which would certainly lead to severe driver discomfort and potential unsafe behavior. The recommended value of four (4) seconds is a based on the minimum recommended value found in the

Institute of Transportation Engineer's *ITE Journal* (in an article by Marshall and Berg in February 1997). Note that this value may be reduced to as low as 0 seconds if the necessary warning time is not available.

Line 44. Maximum preemption time is the total amount of time required after the preempt is initiated by the railroad warning equipment to complete right-of-way transfer to the track clearance green interval, initiate the track clearance phase(s), move the design vehicle out of the crossing's minimum track clearance distance, and provide a separation time "buffer" before the train arrives at the crossing. It is the sum of the right-of-way transfer time (Line 27), the queue clearance time (Line 40), and the desired minimum separation time (Line 43).

SECTION 5: SUFFICIENT WARNING TIME CHECK

Line 45. Required Minimum Time, MT (seconds) is the least amount of time active warning devices shall operate prior to the arrival of a train at a highway-rail grade crossing. Section 8D.06 of the *TMUTCD* requires that flashing-light signals shall operate for at least 20 seconds before the arrival of any train, except on tracks where all trains operate at less than 32 km/h (20 mph) and where flagging is performed by an employee on the ground. The worksheet has a default value of 20 seconds filled in.

Line 46. Clearance time (seconds), typically known as CT, is the additional time that may be provided by the railroad to account for longer crossing time at wide (i.e., multi-track crossings) or skewed-angle crossings. In cases where the minimum track clearance distance (Line 2) exceeds 35 feet, the railroads' *AREMA Manual* requires clearance time of one second be provided for each additional 10 feet, or portions thereof, over 35 feet. Additional clearance time may also be provided to account for site-specific needs. Examples of extra clearance time include cases where additional time is provided for simultaneous preemption (where the preemption notification is sent to the signal controller unit simultaneously with the activation of the railroad crossing's active warning devices), instead of providing advance preemption time.

Line 47. Total Minimum Warning Time, MWT (seconds) is the sum of the minimum time (Line 45) and the clearance time (Line 46). This value is the actual minimum time that active warning devices can be expected to operate at the crossing prior to the arrival of the train under normal, through-train conditions. The term "through-train" refers to the case where trains do not stop or start moving while near or at the crossing. Note that the minimum warning time does not include buffer time (BT) or equipment response time. Buffer time is added by the railroad to ensure that the minimum warning time is always provided despite inherent variations in warning times; however, it is not consistently provided and cannot be relied upon by the traffic engineer for signal preemption and/or warning time calculations. Equipment response time is utilized up front by the railroad's constant warning technology to determine the approach speed of the train and send the preemption call to the traffic signal controller at the appropriate time.

Line 48. Required advance preemption time (APT) from railroad (seconds): This value is calculated by subtracting the total minimum warning time (MWT) in line 47 from the maximum preemption time for queue clearance in Line 44.

Line 49. APT currently provided by railroad (seconds), if provided, is the period of time that the notification of an approaching train is currently forwarded to the highway traffic signal controller unit or assembly prior to activating the railroad active warning devices. Only enter advance preemption time if you can verify from the railroad that advance preemption time is already being provided for your site. For new crossings or signals, a value of 0 seconds is entered.

SECTION 6: TRACK CLEARANCE GREEN TIME CALCULATION (IF NO GATE DOWN CIRCUIT PROVIDED)

Preempt Trap Check

Line 50. Warning Time Variability: Although the railroad guarantees a minimum duration for the APT, it is probable that in most cases the actual duration of the APT will be longer than the guaranteed duration. This variability in APT occurs due to “train handling”, which is a term that describes the acceleration and deceleration of trains on their approach to the crossing. If a train accelerates or decelerates while approaching to the crossing, the railroad warning system cannot estimate the arrival time of the train at the crossing accurately, resulting in variation in the actual duration of APT provided. This variation needs to be taken into account to ensure safe operation.

To make sure that the preempt trap does not occur we need to determine the maximum value of the APT so that a sufficiently long track clearance green interval can be provided to ensure that the gates block access to the crossing before the track clearance green ends. In the case where APT is provided, the difference between the minimum and maximum values of APT is termed excess APT. Excess APT usually occurs when the train decelerates on the approach to the crossing, or where train handling affects the accuracy of the estimated time of train arrival at the crossing so that the preempt sequence is activated earlier than expected. The amount of excess APT is increased by the following conditions:

- Increased variation in train speeds, since more trains will be speeding up and slowing down;
- Lower train speeds, since a fixed deceleration rate has a greater effect on travel time at low speeds than at higher speeds; and
- Longer warning times, because more time is available for the train to decelerate on the approach to the crossing.

The accuracy of the warning time provided by the railroad depends on many factors. These include: the duration of APT if provided, the presence of any shunting yards or stations nearby, and whether the grade crossing is near the edges of towns or cities. The longer the APT required at a grade crossings, the greater the likelihood of variability in providing the expected warning time as there is a greater impact of a change in train speed on the warning time. The presence of shunting yards and stations near the crossings, results in trains slowing down or speeding up while on the track circuit, resulting in a variability in the warning time. Finally, grade crossings that are at the borders of towns or cities frequently have restrictions on train noise and require reduced train speeds within the cities. Hence, trains start slowing on the outskirts of cities resulting in greater variability in the warning time. The analyst must assess the variability of train warning times and select either consistent warning times, low warning time variability, or high warning time variability. Selection of one of these options generates a multiplier in Line 52.

Line 51. Advance preemption time required or provided is the duration (in seconds) the preempt sequence is active in the highway traffic signal controller before the activation of the railroad active warning devices. The larger of Line 48 (required APT) and Line 49 (APT currently provided by railroad) is automatically filled in the box.

Line 52. Multiplier for maximum APT due to train handling: The multiplier for maximum APT can be estimated as 1.60 if warning time variability is high, 1.25 if warning time variability is low, or 1.00 for consistent warning times. High warning time variability can typically be expected in the vicinity of switching yards, branch lines, or anywhere low-speed switching maneuvers takes place. These values are automatically filled in based on the option selected in Line 50.

Line 53. Maximum APT is largest value (in seconds) of the advance preemption time that can typically be expected, which corresponds to the earliest possible time the preemption sequence in the traffic signal controller will be activated before the activation of the railroad grade crossing warning system (flashing lights and gates). It is the calculated by multiplying the APT required or provided by the railroad (Line 51) with the multiplier for maximum APT due to train handling (Line 52).

Line 54. Minimum duration for the track clearance green is the minimum duration (in seconds) of the track clearance green interval to ensure that the gates block access to the crossing before the track clearance green expires in the case where no advance preemption time is provided. It is necessary to block access to the crossing before the track clearance green expires to ensure that vehicles do not enter the crossing after the expiration of the track clearance green and so be subject to the preempt trap.

The 15 seconds minimum duration for the track clearance green interval is calculated from Federal regulations and requirements of the *TMUTCD*. Section 8D.06 of the *TMUTCD* requires that flashing-light signals shall operate for at least 20 seconds before the arrival of any train (with certain exceptions), while Section 8D.04 requires that the gate arm shall reach its horizontal position at least 5 seconds before the arrival of the train. For simultaneous (non-advance) preemption, the preemption sequence starts at the same time as the flashing-light signals, so to ensure that the preempt trap does not occur, a track clearance green interval of at least 15 seconds is required.

Line 55. Track clearance green time to avoid Preempt Trap: This is calculated by the form by adding the Maximum APT (Line 53) with the minimum duration of the track clearance green (Line 54). This yields the minimum time that the track clearance green interval has to be active to avoid the preempt trap.

Clearing of Clear Storage Distance
This section calculates the track clearance green and considers the time required from clearing of the design vehicle based on the geometry of the crossing.

Line 56. Time waiting on left-turn truck (seconds): This is the time needed by a truck that is turning left from the street parallel to the tracks and can potentially block the vehicle on the track phase to start moving after the onset of the track clearance phase. This is the value in Line 33.

Line 57. Time required for design vehicle to start moving, recorded on Line 35, is the number of seconds that elapses between either (a) the time the left turning vehicle completes its turn so it is not blocking track clearance if 'Yes' is checked on line 28 or (b) the beginning of track clearance green if 'No' is checked on line 28 and the time the design vehicle, which is located at the stop bar of the railroad crossing on the opposite side from the signalized intersection, begins to move.

Line 58. Design vehicle clearance distance (DVCD in Figure 1) is the length, in feet, which the design vehicle must travel in order to enter and completely pass through the railroad crossing's minimum track clearance distance (MTCD). This is the same value as recorded on Line 36.

If the CSD is smaller than the design vehicle, i.e., the spacing between the tracks and the curb is smaller than the length of the design vehicle, it is essential that the design vehicle must clear through the entire CSD before the track clearance green terminates. Hence, the CSD will be entered in Line 58.

If however the CSD is larger than the length of the design vehicle, the analyst has to decide whether to clear the design vehicle through the entire CSD or to clear the design through just the design vehicle length. If the vehicle has to clear through CSD, the CSD will be entered in Line 58. However, if the vehicle just has to clear the design vehicle length, Line 58 will be equal to DVL.

Line 59. Portion of CSD to clear during track clearance, This is the portion of the clear storage distance (CSD), in feet, that must be cleared of vehicles before the track clearance green interval ends. The worksheet automatically enters the value of CSD for this field. For intersections with a CSD greater than approximately 150 feet it is desirable—but not necessary—to clear the full CSD during the track clearance green interval. In other words, it is desirable to set Line 59 to the full value of CSD (Line 1). If the full CSD is not cleared, however, vehicles will be stopped in the CSD during the preempt dwell period, and if not serviced during the preempt dwell period, will be subject to unnecessary delays which may result in unsafe behavior. For CSD values less than 150 feet the full CSD is typically cleared to avoid the driver task of crossing the tracks followed immediately by the decision to stop or go when presented by a yellow signal as the track clearance green interval terminates.

Line 60. Design vehicle relocation distance is the distance, in feet, that the design vehicle must accelerate through during the track clearance green interval. It is the sum of the design vehicle clearance distance (Line 36) and the portion of CSD to clear during the track clearance green interval (Line 59).

Line 61. The **Time required for design vehicle to accelerate through DVRD**, **Level Terrain**: is the amount of time required for the design vehicle to accelerate from a stop and travel the complete design vehicle relocation distance (DVRD). This time value, in seconds, is calculated by the worksheet and entered in Line 61.

Line 62. Factor to account for slower acceleration on uphill grade: This is a value automatically calculated based on the grade entered in Line 6.

Line 63. Time required to accelerate design vehicle through DVRD adjusted for grade: This value is automatically calculated by multiplying Line 61 and 62.

Line 64. Time to clear portion of clear storage distance, in seconds, is the total amount of time required (after the right-of-way transfer time ends), for the left turning vehicle to get out of the way of the vehicles on the track phase (Line 56), the time required for the design vehicle to start moving (Line 57), and the time required for the design vehicle to move through the DVRD adjusted for grade (Line 63).

Line 65. The **Track clearance green interval** is the time required, in seconds, for the track clearance green interval to avoid the occurrence of the preempt trap <u>and</u> to provide enough time for the design vehicle to clear the portion of the clear storage distance specified on Line 59. The track clearance green interval time is the maximum of the track clearance green time to avoid a preempt trap (Line 55) and the track clearance green time required to clear a portion of clear storage distance (Line 64). Note that this value corresponds to a design that does not include use of a separate gate down circuit preempt.

Maximum Duration of Track Clearance Green after the gates are down (in the absence of a gate down circuit)
This section calculates the estimated duration of the track clearance green indication due to the onset of preemption after the gates are down if a separate gate down circuit preempt is not used. It is possible for a preempt to occur in any state of the traffic signal. When preempt occurs such that the right-of-way-transfer time is the maximum, it is highly likely that the track phase will continue to display the green indication after the gates are down. How long this is depends on the duration of the track clearance interval. This value will provide the analyst a tool in deciding whether or not to incorporate a gate down circuit into the preemption design. A gate down circuit, in addition to eliminating a preempt trap, provides an additional operational benefit of reducing unnecessary track clearance green time after the train has arrived at the crossing. Gate down circuits may be recommended when the value on line 68 exceeds 30 s, train counts are high, or traffic volumes on the route parallel to the tracks are high.

Line 66. Time to complete track clearance green (seconds): This is the amount of time that passes from the onset of the preemption call until the track clearance green terminates and is calculated by adding the right-of-way transfer time (Line 27) with the track clearance green interval (Line 65).

Line 67. Total time before gates are down (seconds): This is the duration for the gates to come down after the onset of preemption and is calculated by subtracting 5 seconds (the gates have to be down 5 seconds the arrival of the train) from the maximum preemption time for queue clearance (Line 44)

Line 68. Maximum Duration of Track Clearance Green after the gates are down (seconds): This value is obtained by subtracting Line 67 from Line 66 and indicates how long the track clearance indication can be green after the gates are down. The larger this value is, the more inefficient the signal operations will be. It is to be noted that this scenario would only occur when the right-of-way transfer is at its maximum value, (i.e., a green indication happens to just be starting when the preempt call is processed by the traffic signal controller). The train is assumed to travel at constant speed on the approach to the crossing.

SECTION 7: SUMMARY OF CONTROLLER PREEMPTION SETTINGS

This section summarizes in one place all the controller timings that are determined by the analysis of the Preemption Worksheet. All units are in seconds. This section should be used by the traffic signal technician when programming the controller and for field verification in the future.

Line 69. Duration Time: This value is set at 0 as a default value to ensure a preempt call is not dropped unnecessarily.

Line 70. Preemption Delay Time: This value is based on Line 14.

Right of Way Transfer Phase

Note that some traffic signal controllers may refer to this phase as ‘Selective’, ‘Entrance’, ‘Enter’ or ‘Begin’ phase.

Line 71. Minimum Green Interval: This value is based on Line 17.

Line 72. Pedestrian Walk Interval: This value is based on Line 22.

Line 73. Pedestrian Clearance Interval: This value is based on Line 23.

Line 74. Yellow Change Interval: This value is based on Line 19.

Line 75. All Red Vehicle Clearance: This value is based on Line 20.

Track Clearance Phase

Line 76. Green Interval (in the absence of gate down circuit): This value is based on Line 65.

Line 77. Green Interval (with gate down circuit): This value is based on Line 40 (queue clearance time). When using a gate down preempt circuit, only the queue clearance time is required to ensure the design vehicle sufficiently clears the tracks.

Line 78. Yellow Change Interval: This value is based on Line 19.

Line 79. All Red Vehicle Clearance: This value is based on Line 20.

Exit Phase

Note that some traffic signal controllers may refer to this phase as a ‘Return’ phase.

Line 80. Dwell/Cycle Minimum Green Time: This value is set to 0 as a recommendation to ensure that the traffic signal re-enters preemption as soon as possible in the event of a 2nd train.

Line 81. Yellow Change Interval: This value is based on Line 19.

Line 82. All Red Vehicle Clearance: This value is based on Line 20.

REFERENCES

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American Railway Engineering and Maintenance-of-Way Association (AREMA). *Manual of Recommended Practices-Signals*. 2000.

Engelbrecht, R.J., S. Sunkari, T. Urbanik, and K. Balke. *The Preempt Trap: How to Make Sure You do Not Have One*. Texas Department of Transportation Project Bulletin 1752-9, October, 2000. On the Internet at http://tti.tamu.edu/product/catalog/reports/1752-9.pdf. Link valid May 2003.

APPENDIX

GENERAL GUIDELINES FOR WHEN TO TRUNCATE PEDESTRIAN CLEARANCE INTERVALS DURING PREEMPTION EVENTS AT HIGHWAY-RAIL GRADE CROSSINGS

The Texas *Manual on Uniform Traffic Control Devices* (TMUTCD) indicates that "During the transition into preemption control … The shortening or omission of any pedestrian walk interval and/or pedestrian change interval shall be permitted." The following is intended to provide general guidance to assist local transportation officials when it *may* be appropriate to truncate the pedestrian clearance interval in the presence of preemption call at a railroad grade crossing.

- The decision on whether or not to truncate the pedestrian clearance interval rests solely with the local transportation officials, in consultation with the railroad Diagnostic Team, and should be based on an engineering study of the crossing that considers the following:

    - **Pedestrian volumes**: lower pedestrian volumes reduces the probability of a pedestrian event conflicting with a train event
    - **Frequency of preemption events**: less frequent preemption events result in fewer pedestrian clearance interval/time truncations or omissions
    - **Signal timing**: if pedestrian movements are active only during a small portion of the cycle, there is less chance that a pedestrian clearance interval will be truncated or omitted
    - **Intersection geometry**: Wider crosswalks require longer pedestrian clearance intervals that are more susceptible to truncation

- Full pedestrian clearance protection should be considered at signalized intersections located on established Safe Routes to School or crossings located near a school (elementary, middle, and/or high school) facility; or at pedestrian crossings frequented by less mobile, or mobility/sight impaired pedestrian groups.
- Full truncation or Intermediate truncation may be appropriate at intersections where the width of the pedestrian crossing is less than 40 feet, or where the pedestrian clearance interval is approximately equal to the maximum vehicle change (yellow) and clearance (all-red) intervals at the intersection. Table 3 provides guidance for when to use different truncation strategies at an intersection where the maximum roadway width to be crossed by pedestrians is less than 40 feet. The following provides a general guidance that can be used to determine the pedestrian condition present at the crossing:

    - **Very Light** – This is intended to represent conditions where pedestrians crossing at the intersection are rare. These intersections would be generally located in rural areas or lightly developed areas. These would include intersections where the Diagnostic Team and local transportation officials agree that very little pedestrian traffic exists or where no pedestrian traffic was observed during the Diagnostic Team review. As general guidance, these would be intersections where one cycle out of every 20 or more cycles was used to cross pedestrians during the heaviest pedestrian times.
    - **Light** – This is intended to represent conditions where pedestrians crossing at an intersection occur, but only occasionally. These intersections might be located generally in lightly developed areas, industrial or warehouse areas, or small urban communities. These would include intersections where the Diagnostic Team and local transportation officials agree that pedestrian traffic is "light" or where during a site visit, the Diagnostic Team observes several pedestrians in the area (but not necessarily using the intersection). As general guidance, pedestrian volumes at these intersections are such that a pedestrian is likely to cross any approach at an intersection 1 cycle every 10 to 20 cycles.
    - **Moderate** – This is intended to represent conditions where pedestrians crossing at an intersection occur with regular frequency, but not all the time. These intersections might

be located generally in more densely developed areas, including areas with residential, retail, and commercial developments. These would include intersections where the Diagnostic Team and local transportation officials agree that pedestrian traffic is moderate or where during a site visit, the Diagnostic Team observes individual or small groups of pedestrians using the intersection, but not every cycle. As general guidance, pedestrian volumes at these intersections are such that a pedestrian is likely to cross any approach at an intersection 1 cycle every 4 to 10 cycles.

- **Frequent** – This is intended to represent conditions where pedestrian are frequent users of the intersection. These might be intersections where a pedestrian crosses the intersection 1 cycle out of every 1 to 3 cycles. This is also intended to represent crossings where large groups of pedestrians may use the intersection. Under this condition, the Diagnostic Team might observe the intersection being used by pedestrians almost every cycle or every other cycle. This type of condition might be found in highly developed areas with significant residential, retail, or commercial development.
- **School Crossings/Crossing Frequented by Less Mobile (Elderly) or Mobility and/or Visually Impaired Pedestrians** – This is intended to represent conditions were the intersection is located near a school and where there are significant number of school-aged children crossing the intersection. This category of crossing also applies where less mobile populations (elderly) and mobility/visually impaired pedestrians using the pedestrian crossing. These crossings include any grade crossings found along a school's Safe Routes to School walking path(s).

Table 3. Suggested Pedestrian Clearance Strategies to be used under Different Levels of Preemption Event Frequencies and Pedestrian Conditions where the Largest Pedestrian Crossing Width <u>is less than 40 feet</u>.

	Preemption Events			
Pedestrian Conditions	**Very Light (0-5 preemption events per day)**	**Light (5-10 preemption events per day)**	**Moderate (10-20 preemption events per day)**	**Frequent (> 20 preemption events per day)**
Very Light (1 cycle out of every 20 cycles or more is used to cross pedestrians)	Full Truncation	Full Truncation	Full Truncation	Full Truncation
Light (1 cycle out of every 10 to 20 cycles is used to cross pedestrians)	Full Truncation	Full Truncation	Full Truncation	Intermediate Truncation
Moderate (1 cycle out of every 4 to 10 cycles is used to cross pedestrians)	Full Truncation	Full Truncation	Intermediate Truncation	Intermediate Truncation
Frequent (1 cycle out of every 1 to 3 cycles is used to cross pedestrians)	Intermediate Truncation	Intermediate Truncation	Intermediate Truncation	Intermediate Truncation
School Crossings/Crossing Frequented by Less Mobile (Elderly) or Mobility and/or Visually Impaired Pedestrians	Full Pedestrian Clearance			

For wide intersections (40 feet or more), Table 4 provides guidance for when to use different truncation strategies at an intersection where the minimum roadway width to be crossed by pedestrians is more than 40 feet.

Table 4. Suggested Pedestrian Clearance Strategies to be used under Different Levels of Preemption Event Frequencies and Pedestrian Conditions where the Largest Pedestrian Crossing Width is 40 feet or more.

Pedestrian Conditions	Preemption Events: Very Light (0-5 preemption events per day)	Light (5-10 preemption events per day)	Moderate (10-20 preemption events per day)	Frequent (> 20 preemption events per day)
Very Light (1 cycle out of every 20 cycles or more is used to cross pedestrians)	Full Truncation	Full Truncation	Full Truncation	Intermediate Truncation
Light (1 cycle out of every 10 to 20 cycles is used to cross pedestrians)	Full Truncation	Intermediate Truncation	Intermediate Truncation	Partial Truncation
Moderate (1 cycle out of every 4 to 10 cycles is used to cross pedestrians)	Intermediate Truncation	Partial Truncation	Partial Truncation	Full Pedestrian Clearance
Frequent (1 cycle out of every 1 to 3 cycles is used to cross pedestrians)	Partial Truncation	Partial Truncation	Full Pedestrian Clearance	Full Pedestrian Clearance
School Crossings/Crossing Frequented by Less Mobile (Elderly) or Mobility and/or Visually Impaired Pedestrians	Full Pedestrian Clearance			

- The preferred locations of crossing pedestrians at all newly installed traffic signals near railroad grade crossings is shown in Figure 4. In cases where the decision is to eliminate or truncate the pedestrian clearance intervals, agencies should consider relocating crosswalks as shown in Figure 4, where practical and where pedestrian crossing behavior permits. Relocating the crosswalks to these locations will increase the conspicuity of pedestrian that might be located in the crossing to vehicles using the track clearance phase. It allows track clearance vehicles to wait in the intersection (as opposed to on the approach) for pedestrians to finish clearing the intersection.

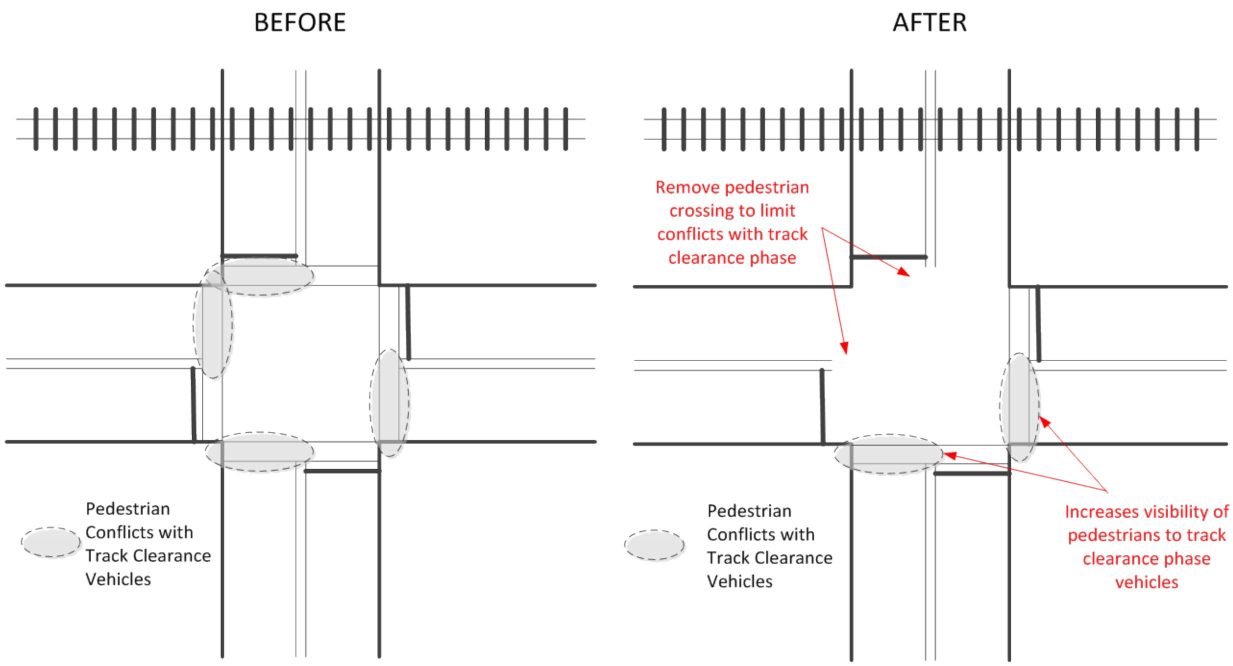


Figure 4. Recommendations for Relocating Pedestrian Crosswalks at Intersections with nearby Highway Rail Grade Crossings

The following provides a description of the different types of pedestrian clearance truncation strategies listed in Table 3 and Table 4. Truncation time (pedestrian clearance interval) is calculated by dividing the distance by 3 ft/s.

- With **Full Truncation**, the entire duration of the pedestrian clearance interval may be omitted before transitioning to the track clearance phase. Pedestrian protection is only provided during the vehicle change (yellow) and clearance (all-red) intervals of the right-of-way transition phase.
- Under the **Intermediate Truncation** strategy, the duration of the pedestrian clearance interval should be sufficient to get the pedestrian crossing from A to B and from A to D to the middle of the farthest travel lane in the pedestrian conflict zones closest to the grade crossing (see Figure 5).
- With the **Partial Truncation** strategy, sufficient time should be provided after terminating the "WALK" interval to get pedestrians crossings from B to C and crossing from C to D to the middle of the pedestrian conflict zones farthest for the grade crossing (as shown in Figure 5) to increase their conspicuity to motorists exiting the track clearance phase.
- With **Full Pedestrian Clearance**, the full amount of pedestrian clearance interval is provided after terminating the "WALK" interval before transitioning to the track clearance phase.

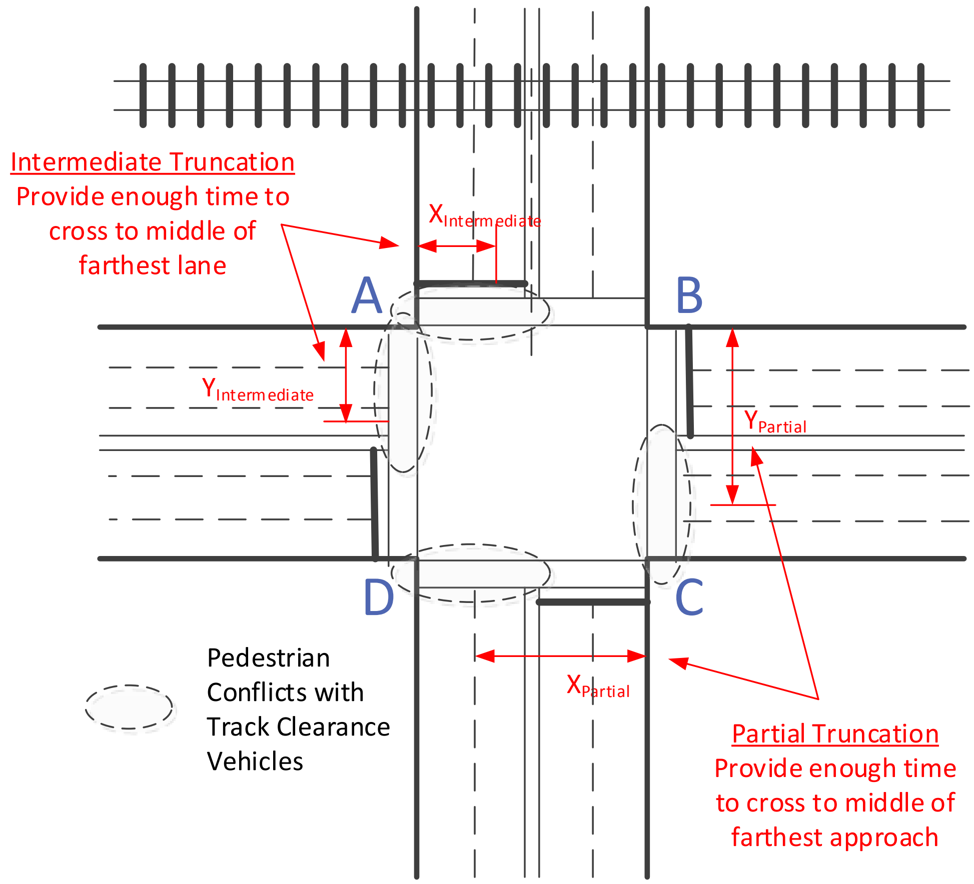


Figure 5. Crossing Distance during Truncated Pedestrian Clearance Intervals

RESET

Texas Department of Transportation

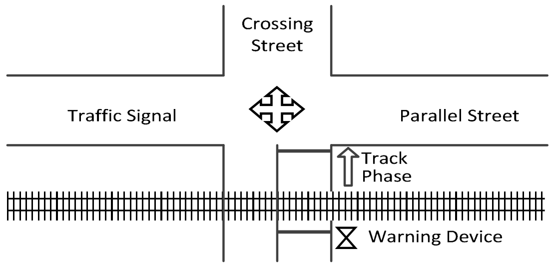
GUIDE FOR DETERMINING TIME REQUIREMENTS FOR TRAFFIC SIGNAL PREEMPTION AT HIGHWAY-RAIL GRADE CROSSINGS

City ______ CSJ ______ Date ______

County ______ Completed by ______

District ______ District Approval ______

Show North Arrow



Parallel Street Name ______

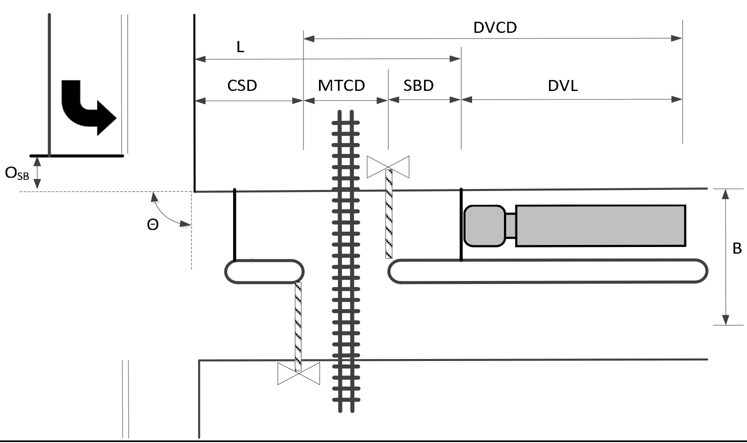
Crossing Street Name ______

Railroad ______ Railroad Contact ______

Crossing DOT# ______ Phone ______

NOTE: After approval by the District, a copy of this form, along with the traffic signal design sheets and the phasing diagrams for normal and preempted operation, shall be placed in the traffic signal cabinet. See Section 7 for traffic signal timings.

SECTION 1: GEOMETRY DATA & DEFAULTS



CSD = Clear storage distance (ft)
MTCD = Minimum track clearance distance (ft)
SBD = Stop bar setback distance (ft)
DVL = Design vehicle length (ft)
L = Queue start-up distance, also stop-line distance (ft)
DVCD = Design vehicle clearance distance (ft)
O_{SB} = Offset distance to Left-turn stop bar (ft)
B = Distance from curb line to center of nearest lane receiving left turns (ft)
Θ = Angle of turn (degrees)

GEOMETRIC DATA FOR CROSSING

				Remarks
1.	Clear storage distance (CSD, feet)	1.	0	
2.	Minimum track clearance distance (MTCD, feet)	2.	0	
3.	Stop bar setback distance (SBD, feet)	3.	8	Enter "0" if no stop bar is present
4.	Width of receiving approach (B, feet)	4.	0	
5.	Offset distance of left turn stop bar (O_{SB}, feet)	5.	0	
6.	Approach grade. % (0 if approach is on downgrade)	6.	0.0	
7.	Angle of turn at Intersection (Θ, degrees)	7.	90	

DESIGN VEHICLE DATA

8. Select Design Vehicle

☐ School Bus ☐ Intermediate Truck ☑ Interstate Semi-Truck ☐ Other

9.	Default design vehicle length (feet)	9.	75	Based on selected Design Vehicle
	a. Additional vehicle length, if needed (feet)	9a.	0	Use only if "Other" selected as Design Vehicle
10.	Total design vehicle length (DVL, feet)	10.	75	Sum of line 9 and 9a
11.	Centerline turning radius of design vehicle (R, feet)	11.	41	Based on selected Design Vehicle
12.	Passenger car vehicle length (LV, feet)	12.	19	Default value

SECTION 2: RIGHT-OF-WAY TRANSFER TIME CALCULATION

Preempt verification and response time

	Item	Line	Value	Remarks
13.	Preempt delay time (seconds)	**13.**	0	
14.	Controller response time to preempt (seconds)	**14.**	0.0	Manufacturer:
				Firmware Version:
15.	Preempt verification and response time (seconds): add lines 13 and 14	**15.**	0.0	

Worst-case conflicting vehicle time

	Item	Line	Value	Remarks
16.	Minimum green time during right-of-way transfer (seconds)	**16.**	5	Value may be adjusted to meet local conditions
17.	Other green time during right-of-way transfer (seconds)	**17.**	0	
18.	Yellow change time (seconds)	**18.**	0.0	
19.	Red clearance time (seconds)	**19.**	0.0	
20.	Worst-case conflicting vehicle time (seconds): add lines 16 through 19	**20.**	5.0	

Worst-case conflicting pedestrian time

	Item	Line	Value	Remarks
21.	Minimum walk time during right-of-way transfer (seconds)	**21.**	0	Value may be adjusted to meet local conditions
22.	Pedestrian clearance time during right-of-way transfer (seconds)	**22.**	0	Refer to instructions for pedestrian truncation guidance
23.	Vehicle yellow change time, if not included on line 22 (seconds)	**23.**	0.0	
24.	Vehicle red clearance time, if not included on line 22 (seconds)	**24.**	0.0	
25.	Worst-case conflicting pedestrian time (seconds): add lines 21 through 24	**25.**	0.0	

Worst-case conflicting vehicle or conflicting pedestrian time

	Item	Line	Value
26.	Worst-case conflicting vehicle or conflicting pedestrian time (seconds): maximum of lines 20 and 25	**26.**	5.0
27.	**Right-of-way transfer time (seconds): add lines 15 and 26**	**27.**	5.0

SECTION 3: QUEUE CLEARANCE TIME CALCULATION

28. Are there left-turns towards the tracks? ☐ Yes ☑ No

	Item	Line	Value	Remarks
29.	Distance traveled by truck during left-turn (LTL, feet):	**29.**	0	LTL = ∏RΘ/180
30.	Travel speed of left-turning truck (S_{LTT}, mph):	**30.**	10	Default value
31.	Distance required to clear left-turning truck from travel lanes on track clearance approach (feet):	**31.**	0	Equation: (line 4 + line 5 + line 12 - line 11) + line 29 + line 10
32.	Additional time required to clear left-turning truck from travel lanes on track clearance approach (seconds):	**32.**	0.0	Equation: [(line 31 * 3600) / (line 30 * 5280) - line 18 - line 19]
33.	Worst-case Left Turning Truck time (seconds): if Line 28 = 'Yes', use line 32; otherwise Use 0	**33.**	0.0	
34.	Queue start-up distance, L (feet): add lines 1 through 3	**34.**	8	
35.	Time required for design vehicle to start moving (seconds): calculate as 2+(L÷20)	**35.**	2.4	
36.	Design vehicle clearance distance, DVCD (feet): add lines 2, 3 and 10	**36.**	83	
37.	Time for design vehicle to accelerate through the DVCD (seconds), level terrain	**37.**	12.1	
38.	Factor to account for slower acceleration on uphill grade	**38.**	1.00	
39.	Time for design vehcle to accelerate through DVCD (seconds), adjusted for grade: multiply lines 37 and 38	**39.**	12.1	
40.	**Queue clearance time (seconds): add lines 33, 35 and 39**	**40.**	14.5	

SECTION 4: MAXIMUM PREEMPTION TIME CALCULATION

	Item	Line	Value	Remarks
41.	Right-of-way transfer time (seconds): line 27	**41.**	5.0	
42.	Queue clearance time (seconds): line 40	**42.**	14.5	
43.	Desired minimum separation time (seconds)	**43.**	4.0	Typical Value
44.	Maximum preemption time for Queue Clearance (seconds): add lines 41 through 43	**44.**	23.5	

SECTION 5: SUFFICIENT WARNING TIME CHECK **Remarks**

Line	Description	Line	Value
45.	Required minimum time, MT (seconds): per regulations	45.	20
46.	Clearance time, CT (seconds): (line 2 -35) / 10 (rounded up to nearest second)...	46.	0
47.	Total minimum warning time, MWT, needed (seconds): add lines 45 and 46 (excludes buffer time and equipment response time).................................	47.	20
48.	Required advance preemption time (APT) from railroad (seconds): subtract line 47 from line 44, round up to nearest full second, enter 0 if less than 0	48.	4
49.	APT currently provided by railroad (seconds): Enter "0" if new crossing or signal	49.	0

If the required advance preemption time (line 48) is greater than the amount of advance preemption time currently provided by the railroad (line 49), additional warning time must be requested from the railroad. Alternatively, the maximum preemption time (line 48) may be decreased after performing an engineering study to investigate the possibility of reducing the values on lines 13, 16, 17, 21, 22 and 43.

Remarks:

SECTION 6: TRACK CLEARANCE GREEN TIME CALCULATION (IF NO GATE DOWN CIRCUIT PROVIDED)

Preempt Trap Check **Remarks**

50. Warning Time Variability (Select One)

☐ Consistent Warning Times ☑ Low Warning Time Variability ☐ High Warning Time Variability

Line	Description	Line	Value	Remarks
51.	APT required or provided (seconds): maximum of Line 48 or Line 49.........	51.	4	See Instructions for details.
52.	Multiplier for maximum APT due to train handling	52.	1.25	
53.	Maximum APT (seconds): multiply line 51 and 52	53.	5.0	
54.	Minimum duration for the track clearance green interval (seconds)	54.	15	
55.	Track Clearance Green Time to avoid Preempt Trap (seconds): add lines 53 and 54	55.	20.0	

Clearing of Clear Storage Distance

Line	Description	Line	Value
56.	Time waiting on left-turn truck (seconds): line 33	56.	0.0
57.	Time required for design vehicle to start moving (seconds): line 35	57.	2.4
58.	Design vehicle clearance distance (DVCD, feet): line 36	58.	83

If CSD $\leq$ DVL, you must clear the design vehicle through the entire CSD during the traffic clearance phase; however, if CSD > DVL, you should consider providing enough time to clear the design vehicle from the crossing.

Is the clear storage distance (CSD) less than or equal to the design vehicle length (DVL)?

☑ YES. The design vehicle MUST clear through the entire CSD. (CSD will be entered in Line 59).

☐ NO. The design vehicle may clear through a portion of the CSD.

Do you want to clear the design vehicle through the entire CSD?

☑ YES. Clear the entire CSD. (CSD will be entered in Line 59).

☐ NO. Clear the crossing ONLY. (DVL will be entered in Line 59).

Line	Description	Line	Value
59.	Portion of CSD to clear during track clearance phase (feet)	59.	0
60.	Design vehicle relocation distance (DVRD, feet): add lines 58 and 59	60.	83
61.	Time required to accelerate design vehicle through DVRD (seconds), level terrain:	61.	12.1
62.	Factor to account for slower acceleration on uphill grade	62.	1.00
63.	Time required to accelerate design vehicle through DVRD (seconds), adjusted for grade: multiply lines 61 and 62	63.	12.1
64.	Time to clear portion of clear storage distance (seconds): add lines 56, 57 and 63	64.	14.5
65.	**Track clearance green interval (seconds): maximum of lines 55 or 64, round up to nearest full second ....**	**65.**	20

Maximum Duration of Track Clearance Green after gates are down (in absence of a gate down circuit)

Line	Description	Line	Value
66.	Total time to complete track clearance green (seconds): line 27 + line 65	66.	25.0
67.	Total time before gates are down (seconds): subtract 5 seconds from line 44 (per AREMA Manual)	67.	18.5
68.	**Maximum Duration of Track Clearance Green after gates are down (seconds): Line 66 - Line 67 ....**	**68.**	7

SECTION 7: SUMMARY OF CONTROLLER PREEMPTION SETTINGS

#	Item	Line	Value	Remarks
69.	Duration Time (seconds)	**69.**	0	Default Value
70.	Preempt Delay Time (seconds)	**70.**	0	From Line 13

#	Right of Way Transfer Phase	Line	Value	**Remarks**
71.	Minimum Green Interval (seconds)	**71.**	5	From Line 16
72.	Pedestrian Walk Interval (seconds)	**72.**	0	From Line 21
73.	Pedestrian Clearance Interval (Flashing "DON'T WALK", seconds)	**73.**	0	From Line 22
74.	Yellow Change Interval (seconds)	**74.**	0.0	From Line 18
75.	All Red Vehicle Clearance (seconds)	**75.**	0.0	From Line 19

#	Track Clearance Phase	Line	Value	**Remarks**
76.	Green Interval (seconds) (in the absence of gate down circuit)	**76.**	20	From Line 65
77.	Green Interval (seconds) with gate down circuit	**77.**	15	From Line 40
78.	Yellow Change Interval (seconds)	**78.**	0.0	From Line 18
79.	All Red Vehicle Clearance (seconds)	**79.**	0.0	From Line 19

#	Exit Phase	Line	Value	**Remarks**
80.	Dwell/Cycle Minimum Green Time (seconds)	**80.**	0	Default Value
81.	Yellow Change Interval (seconds)	**81.**	0.0	From Line 18
82.	All Red Vehicle Clearance (seconds)	**82.**	0.0	From Line 19

Remarks: