SURROGATE MEASURES FOR EVALUATING NEW SIGNAGE AND INTERSECTION DESIGNS

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ZHONGYUAN ZHU

Dr. Praveen Edara, Dissertation Supervisor

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The undersigned, appointed by the dean of the Graduate School, have examined the dissertation entitled

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presented by Zhongyuan Zhu,

a candidate for the degree of doctor of philosophy,

and hereby certify that, in their opinion, it is worthy of acceptance.

Professor Praveen Edara

Professor Carlos Sun

Professor Timothy Matisziw

Henry Brown

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ABSTRACT

Transportation agencies faced with the challenge of enhancing safety on roadways are looking for alternative solutions to designing roads and signage. When deciding whether the alternative design is superior to the traditional one, decision makers need methods and quantitative data to evaluate these alternatives. This dissertation provides two accessible methods to compare different alternative designs and illustrates them using case studies.

The first method involves using speed-based statistical measures that are extracted from video-based traffic surveillance. This method was more accurate in collecting vehicle speeds than the speeds extracted from video-based data collection systems. It was then utilized to evaluate the effectiveness of an alternative merge sign in work zones. This alternative sign consists of an arrow pointing the merge direction and text describing the lane closure, while MUTCD sign is graphical. The case study measured driver behavior characteristics including speeds and open lane occupancies. The results indicate that open lane occupancy was higher for the test sign in comparison to the MUTCD sign upstream of the merge sign. The occupancy values at different distances between the merge sign and the taper were similar for both the test and MUTCD signs, but the test sign encouraged up to 11% more cars to be in the open lane immediately upstream of the merge sign. Passenger cars stayed in the closed lane longer, or closer to the taper, than trucks. The merging behavior of truck drivers did not vary significantly with the type of merge sign deployed in the work zone. The analysis of speed characteristics did not reveal substantial differences between the two sign configurations. The mean speeds with

the MUTCD configuration were 1.3 mph and 2 mph lower than the test configuration at the merge sign and taper locations, respectively.

The second method utilizes microscopic traffic simulation to evaluate alternative designs. This method is ideal for projects where video monitoring of the entire study of interest is not feasible. Evaluating alternative designs with crash data usually requires a long time span to build the facility and record crash data over at least one year after the facility has been open to traffic. In addition to that, new facility needs to be built or altered if other design features are to be tested. With microscopic simulation, the time cost for the study is greatly shortened and different design aspects can be tested in a risk-free environment. Two case studies are presented to illustrate this simulation method. The first case study involves a work zone while the second case study focuses on evaluating a J-turn intersection design. The spacing of U-turn and the inclusion of acceleration and deceleration lanes were evaluated, in the J-turn study. A simulation analysis was conducted to study the impact of different design variables on the safety of J-turns. A base simulation model was created and calibrated using field data collected in a previous Missouri Department of Transportation (MoDOT) project on J-turns. The calibrated model was then used to study various combinations of major road and minor road volumes and design variables. The simulation analysis helped develop guidance on recommended spacing for various major road and minor road volume scenarios. For all the studied scenarios, the presence of acceleration lane resulted in significantly fewer conflicts. Thus, acceleration lanes are recommended for all J-turn designs, including lower volume sites. Second, while U-turn spacing between 1000 feet and 2000 feet was

found to be sufficient for low volume combinations, a spacing of at least 1500 feet and 2000 feet are recommended for medium and high volumes, respectively.

1. INTRODUCTION

Transportation agencies are interested in researching new designs and signage that can improve safety, mobility, and driver comprehension. How can agencies compare the performance of any new design or signage to existing alternatives? As the new alternatives have not been implemented, performance measures typically used to quantify safety and operations such as crash frequency, delay and travel time reliability are not yet available. Thus, surrogate measures of performance and alternative methods of evaluation are necessary to answer the above question. This dissertation introduces two methods for evaluating alternative signage and intersection design.

The first method utilizes field data and statistical analysis of speed-based measures such as average speeds, speed differential, and open lane occupancy. Although it is widely used in transportation engineering studies, there is a lack of research regarding the process and importance of raw data collection and extraction. Speed data can be collected using radar guns deployed at the study site. A procedure was established for collecting field data and extracting the performance measures from the video data.

The first case study involved comparing the graphical-only lane closed sign (W4-2) from the Manual on Uniform Traffic Control Devices (MUTCD) Section 6F.24 against a MERGE/arrow sign on one side and a RIGHT LANE CLOSED sign on the other side (Figure 1.1.1).

Figure 1.1.1 MUTCD sign (left) and Alternative Sign (right)

Although the graphical-only MUTCD signage for work zones has been in use for several years, it is not known if the signage recommended by the MUTCD offers the highest safety for all jurisdictions. This dissertation measured driver behavior characteristics including speeds and open lane occupancies. The measurements were taken at a work zone on Interstate 70 in Missouri. The study found that the open lane occupancy upstream of the merge sign was higher for the test sign in comparison to the MUTCD sign. The occupancy values at different distances between the merge sign and the taper were similar for both signs. The test sign had 11% more traffic in the open lane upstream of the merge sign. In terms of safety, it is desirable for vehicles to occupy the open lane as far upstream from the taper as possible to avoid conflicts due to the lane drop. The analysis of speed characteristics did not reveal substantial differences between the two sign configurations. The 85th percentile speeds with the MUTCD sign were only one mph and two mph lower than the test sign at the merge sign and taper locations, respectively. In considering all the aforementioned performance measures, the alternative sign configuration was not superior, but performed equal to the MUTCD sign configuration.

The second method is using calibrated microscopic simulation model to generate safety and operational performance measures i.e. delays and conflicts. One crucial aspect of using a microscopic simulation model for evaluation is calibration. An uncalibrated model may lead to unreasonable claims and decision making. Thus, it is important to calibrate the model as close to the real-world conditions as possible. The common ways of calibrating a simulation model are presented in chapter two. Since most commonly used calibration processes are time consuming and not easy to perform, this dissertation

presents a calibration process that utilizes easily obtainable speed data and travel time data. The process is then demonstrated with a long-term work zone field study and its potential is further demonstrated in a J-turn study. The J-turn case study also showed that the use of simulated traffic conflicts as a surrogate safety measure is a viable option for safety evaluation prior to implementing such designs.

The J-turn design has been in use in North Carolina (as Superstreet), Maryland (as Restricted Crossing U-turn), Michigan, among other states. In the last decade, Missouri DOT has replaced several high crash intersections on rural high speed expressways with J-turns. Figure 1.1.2 illustrates a J-turn maneuver at one of Missouri's J-turn site (Edara, 2013). One difference between the Missouri J-turns from other states (e.g. North Carolina) is that they have all been installed on rural high speed expressway intersections replacing a two-way stop control. Much of the existing research on J-turns evaluates the performance of signalized J-turns, typically installed in urban areas. Studies evaluating the performance of unsignalized J-turns on rural high speed expressways is limited. A recent study (Edara et al., 2015) quantified the overall safety benefits of J-turns in Missouri. Hummer et al. (2010) documented the safety of unsignalized Superstreet intersections in North Carolina.

Despite their long use, there is no specific national guidance on the design of J-turns. Specifically, there are no recommendations on the spacing between the main intersection and the U-turn. Similarly, there is no guidance on when acceleration lanes are recommended, i.e., at what traffic volume.

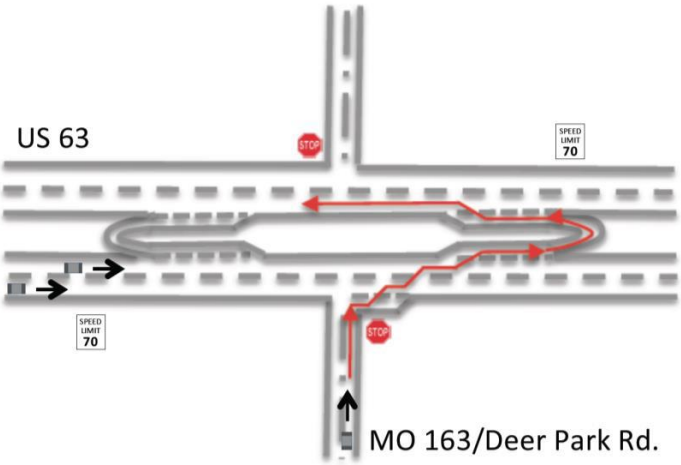


Figure 1.1.2 Minor-Road Left-Turn Maneuver at J-turn Intersection (Edara, 2013)

This part of the dissertation addresses this gap in practice by developing guidance on spacing and acceleration lanes. Calibrated simulation models were used to study various volume scenarios and J-turn design variables.

2. LITERATURE REVIEW

2.1. Video-based Methods in Before-and-after Studies

Video recording is an essential tool to traffic engineers and researchers. It is commonly used in before and after studies to provide data for both visual qualitative information and quantitative data. It usually follows a field data collection and playback analysis process. For example, Jeng (2005) led a typical before and after study on evaluating the human factors on vehicle entrapment on railroad tracks using high definition videos. The study collected total of 8 hours for each of the three sites and trained five reviewers to extract the volume and movement of the vehicles, as well as quantifying potential collision incidents. The surrogate measures such as “abnormal turning curve” and “improper brakes-hesitation upon crossing tracks” were also recorded.

Gordon et. al. (2012) submitted a report on developing a relatively low-cost and portable video capture system that can be installed on a site-by-site basis. This is an automated image processing system which uses virtual vehicle loops for traffic counts and is able to identify certain signatures of irregular vehicle movements. This system requires stable image and clear background referencing image for accurate data generation.

Following this framework, Paul (2013) conducted a case study using a video data set which includes 45.3 hours across four entrances and four exits to a highway. After collecting all the trajectory data, a time to collision (TTC) analysis was conducted. They found the TTC numbers remained the same before and after the treatment.

Similarly, Sayed et. al. (2012) developed an automated video-based safety evaluation system which claimed to be from 84.7% to 94.4% accurate in tracking

vehicles. This system required careful calibration of the footage to convert in-video coordinates to real world coordinates. Sayed et al. (2013) then conducted a series of case studies using the system and used TTC as a surrogate measure for crash frequency. This system in theory is able to handle high flow scenario with adequate accuracy and efficiency but it required camera calibration on each new location which may require additional time to set up.

Davis et al. (2014) utilized a portable 30-foot camera recording system to collect video data and followed the same "video-coordinates to real-world-coordinates" calibration process as previous mentioned studies, in which control points were marked in the video and their real-world and video coordinates were recorded. The vehicles' video coordinates were later translated into real-world coordinates to construct complete vehicle trajectories. Despite the low resolution of the camera, the calibrated results were close to radar-gun-measured speeds.

Gong et al. (2014) conducted a simple study on the speed bump on Chinese campuses using video-based before and after method where 230 vehicles were recorded and individually tracked for analysis. This method required every vehicle's trajectory to be tracked manually thus it is not recommended for large scale projects.

Recently, Kirk (2016) evaluated the effectiveness of active warning bike sign in Portland, Oregon. This typical before and after study compared crash frequency as well as conflicts derived from 48 hours of video data. The reduction in both conflicts and crashes was found to be statistically significant at 95 percent confidence level.

Unlike aforementioned studies which utilized video-based automated vehicle detection programs to calculate speeds, this dissertation paired camcorders with radar

guns to record vehicle speed directly. This method is both cost-effective and accurate since video-based automated vehicle detection systems are expensive to buy and maintain. These systems also require extensive calibration to convert video coordinates into real world coordinates. Even slight error in the calibration process can result in inaccurate speed readings. Moreover, the video footage needs to be stable for the virtual detection loop algorithm to accurately detect and compute vehicle speeds. Radar gun method is more accurate although it requires good shooting angles to capture all the lanes. It should be noted that the video detection systems perform well for collecting occupancy and volume data.

2.2. Microscopic Simulation Calibration

Calibration of microscopic traffic simulation models ensures the accuracy and reliability of the generated performance measures.

Depending on the availability of empirical data and the structure of the simulation model, certain parameters are more critical than others in the calibration process. In general, speeds, capacity and driving behavior parameters such as headway, acceleration rate are among the most commonly used calibration parameters. In 2011, Mamun et al. (2011) reported on a large scale microscopic simulation model calibration. They presented a sequential calibration process in the following order: capacity, speed distribution, lane distribution, headway distribution, local, and network. This sequential approach, however, does not guarantee that the parameters already selected to optimize a performance measure (e.g. capacity) will also optimize a different performance measure in the future steps (e.g. lane or headway distribution).

Hoffman and Hitscherich (2014) from PTV group encouraged using historical speed and volume data as an input for calibration. Zhang et al. (2016) proposed a similar method for short-term work zones, which involved collection of speeds within the work zone and the construction of a cumulative frequency distribution plot of passenger cars and trucks. The authors then recommended using five control points in the cumulative frequency plot instead of ten to speed up the calibration process without sacrificing much accuracy.

Li et al. (2015) proposed an approach based on survey and in-car tests to use acceleration and deceleration distributions for calibration. GPS was used to record participants driving session to obtain acceleration, deceleration and speed data. The data

was then categorized according to driver aggressiveness. The acceleration and deceleration distributions were then input into VISSIM to calibrate the model.

When calibrating car following model parameters, capacity is a commonly used performance measure for work zones. For example, Kan et al. (2014) calibrated a 2-to-1 freeway work zone by using speed distribution as an initial input to generate a capacity value and then varying the two car following parameters CC0 (stand still distance) and CC1 (headway time) to reproduce a certain speed and queue length to match previous capacity value. Chatterjee et al. (2009) also calibrated work zone capacities by adjusting several CC parameters in VISSIM. They recommended parameter values for different lane closures and truck percentages. Durrani et al. (2016) statistically analyzed field data and suggested a parameter value table for VISSIM's Wiedemann 99 Model parameters for cars, heavy vehicles and motorcycles. For alternative intersection designs, ,Li et al. (2013) developed a procedure which used critical gap and following headway in VISSIM's Wiedemann 74 car following Model to calibrate roundabout capacity. This dissertation will introduce a different approach using travel time to calibrate the model at a certain capacity.

2.3. Traffic Conflict and SSAM

A traffic conflict is defined as an event where two or more road users are approaching each other with a risk of collision if they remain on the course (Hyden, 1975). It is the most used surrogate safety measure especially when crash frequency data is not available or hard to obtain. Traffic conflicts can be extracted from the output of traffic simulation models.

The Federal Highway Administration (FHWA) sponsored the development of an analytical tool called "Surrogate Safety Assessment Model (SSAM)" (Gettman and Head, 2003) that automates the process of extracting conflicts from the vehicle trajectories obtained from simulation models. Gettman and Head (2003) selected 83 urban signalized intersections and simulated them in VISSIM. The SSAM analysis result showed significant correlation to the historical crash data with a R-squared value of 0.41. This validated simulated conflicts as a surrogate for crash frequency. SSAM provides five conflict measures: time to collision (TTC), post encroachment time (PET), deceleration rate (DR), the maximum speed (MaxS) and the speed differential (DeltaS).

Huang (2012) conducted a study on comparing VISSIM and SSAM simulated conflicts with visually identified field conflicts. The field data was recorded in 15-min intervals of a total of 80 hours of peak traffic across ten sites. The video was then analyzed manually to identify all the conflicts within the intersections. The study concluded that simulated conflicts were statistically consistent with observed conflicts with a R-square value of 0.783.

Although traffic conflicts are widely used as a surrogate measure in traffic safety studies, most of the studies focus on signalized intersections. This dissertation

implemented SSAM in a rural unsignalized situation where driving behavior was different, due to higher speeds and low volumes.

3. METHODOLOGY

3.1. Data Collection and Statistical Comparisons for Before-and-after Studies

The first method when evaluating transportation facilities is statistically comparing selected field data with controlled variables. This chapter discusses some commonly used video data collection procedures and statistical tools and then presents a case study to illustrate their application. Additionally, some case-specific comparison measure can be developed to further differentiate the original design and the alternative design. Is this case, new concept called "open lane occupancy" is developed to address the different sign's effectiveness in encouraging lane changes before drivers reaching the taper.

3.1.1. Basic Speed-Related Measures

Mean speed, standard deviation and 85th percentile speed is considered most commonly used speed-related measures. When conducting before-and-after comparisons, these measures are collected and compared statistically. The standard t-test can be used for comparing means and the F-test can be used for comparing variances. The 85th percentile speed is usually calculated to assess if vehicles are compliant with the posted speed limits. The 85th percentile speeds across different merge sign configurations can be statistically compared using a test similar to the T-test described in Hou et al. (2012).

3.1.2. Video Data Collection and Processing

Checklist of items was developed for the video monitoring of study sites and the data reduction. The checklist is as follows:

1. Plan ahead and survey for best locations to deploy cameras and other field equipment.

2. Charge the batteries and synchronize the clock on all cameras.

3. Bring back-up battery, memory card and camera.

4. Before data collection period begins, start the recording and measure a few control points that can be used for calibration and data reduction. A few markers must be visible in the camera view and the distances to these markers must be measured using a measuring wheel.

5. Pair each radar gun with a camera recording the display. Check frequently if the radar gun is functioning as expected.

6. To extract the data, all video recordings should be marked and synchronized through camera clock.

7. Track vehicles visually and individually to record their exact speeds and vehicle type (car, truck). This process can be automated by developing programs to read the display on the radar gun.

8. Aggregate the data to generate mean speed, variance, and volume. The merge locations are also aggregated based on predefined distances to the merge sign.

3.1.3. Case Study

Roadway construction and maintenance activities involve lane closures that necessitate vehicles to merge into open lanes. The Manual of Traffic Control Devices (MUTCD 2009) provides information on temporary traffic control (TTC) plans to be implemented at both short-term and long-term work zones. The TTC plans include information regarding the appropriate signs to use and the locations to place them inside a work zone. The MUTCD TTC plan adapted to meet the standards from the Missouri Department of Transportation (MoDOT) Engineering Policy Guide (EPG 2014) is shown

in Figure 3.1.1. In this case study, the performance of an alternative TTC plan with slightly different signage than that recommended by the MUTCD is evaluated. In this alternative TTC plan, the graphical lane closed sign is to be replaced with a MERGE/arrow sign on one side and RIGHT LANE CLOSED sign on the other side as shown in Figure 3.1.2. In order to test the new TTC plan, a MUTCD request for experimentation was submitted by MoDOT and approved by FHWA in early 2013.

In a recent study, Ishak et.al (2012) found that the advance warning area right before the taper has the highest crash rates of the entire work zone. Thus, effective signage that encourages safer driving behavior in this area is desirable. A review of the existing literature did not reveal any studies investigating the effectiveness of different static merge signs in work zones Feldblum (2005) investigated a new static merge sign at intersections. A rating system was developed based on visually inspecting the speed changes of merging vehicles. In this system, a vehicle received a higher rating if it experienced a lower speed change during merging. Actual vehicle speeds were not measured. The study found that the new static sign had better overall rating than the MUTCD sign.

One goal of this dissertation is to compare the safety performance of the new static merge sign configuration with the MUTCD merge sign in a work zone. Field studies were conducted at a work zone site on I-70 in Missouri. Video monitoring was used to record merging locations and radar guns were used to collect vehicle speeds. The field data was analyzed and several measures of the effectiveness of the sign in encouraging early merges were computed. These measures include distribution of traffic in the open

and closed lanes at various distances from the taper, 85th percentile speeds, mean speeds, and speed variance.

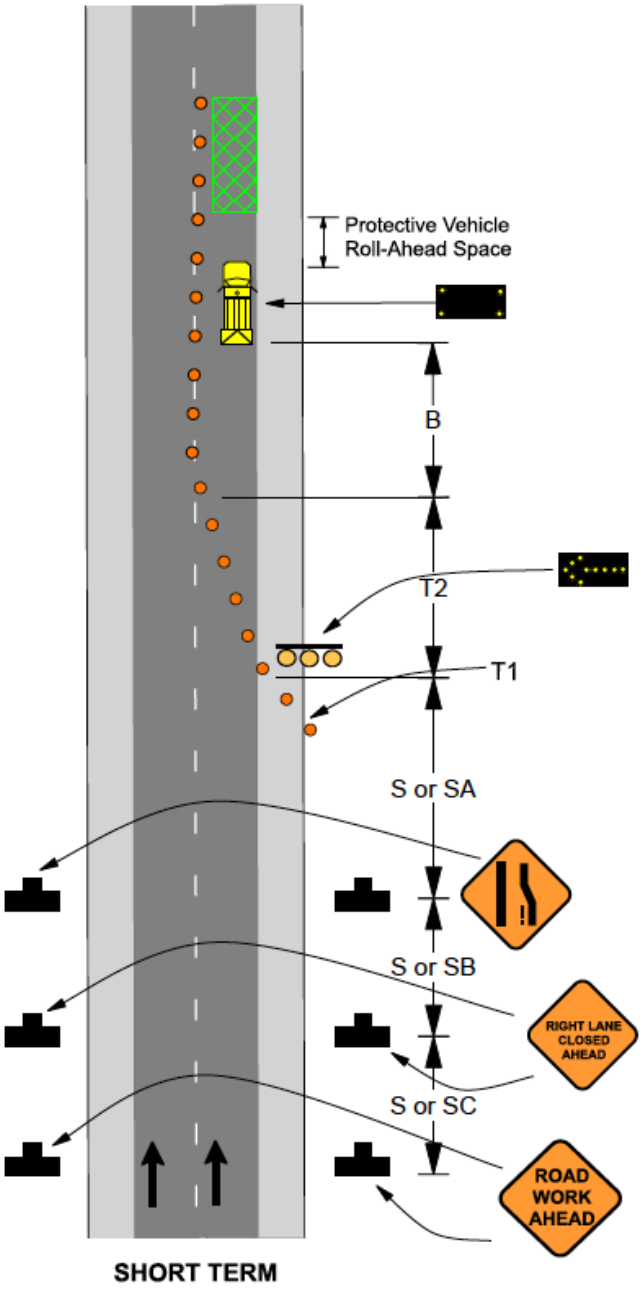


Figure 3.1.1 Missouri MUTCD-based temporary traffic control plan for a stationary lane closure on a divided highway

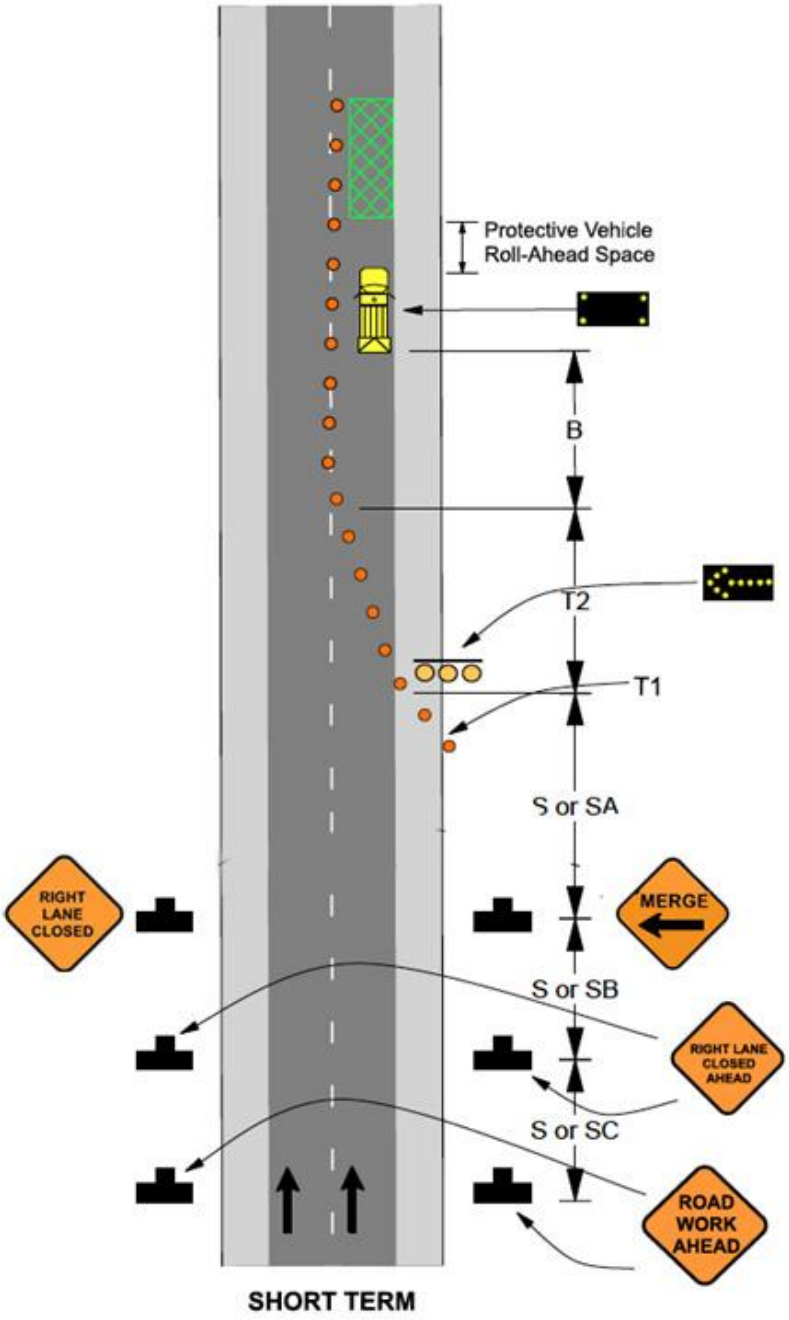


Figure 3.1.2 Test merge sign temporary traffic control plan for a stationary lane closure on a divided highway

3.1.3.1. Site Description

Two short-term work zones involving a left lane closure on a two-lane segment of westbound I-70 near Boonville, MO, are considered in this study. The work activity at the sites involved patching of the deck of the bridge spanning the Lamine River, MO. The work zones were at the same location and at approximately the same time of day on different days. The data collection period was from 11:30 am to 2:00 pm and was chosen based on the peak hourly traffic volumes for that location. In accordance with the TTC plan, merge signs were placed 1000 feet upstream of the taper. The new static text merge sign, called the test sign hereafter, was deployed on April 22nd, 2013 and the MUTCD graphical sign was tested on April 25th, 2013. The posted speed limit of the work zone remained at 70 mph.

Figure 3.1.3 illustrates the setup of the data collection effort. One radar gun was placed at the merge sign and another radar gun was placed at the taper in order to capture the longitudinal speed changes for individual vehicles. Three Sony high definition digital camcorders covered the whole study area as shown in Figure 3.1.4.

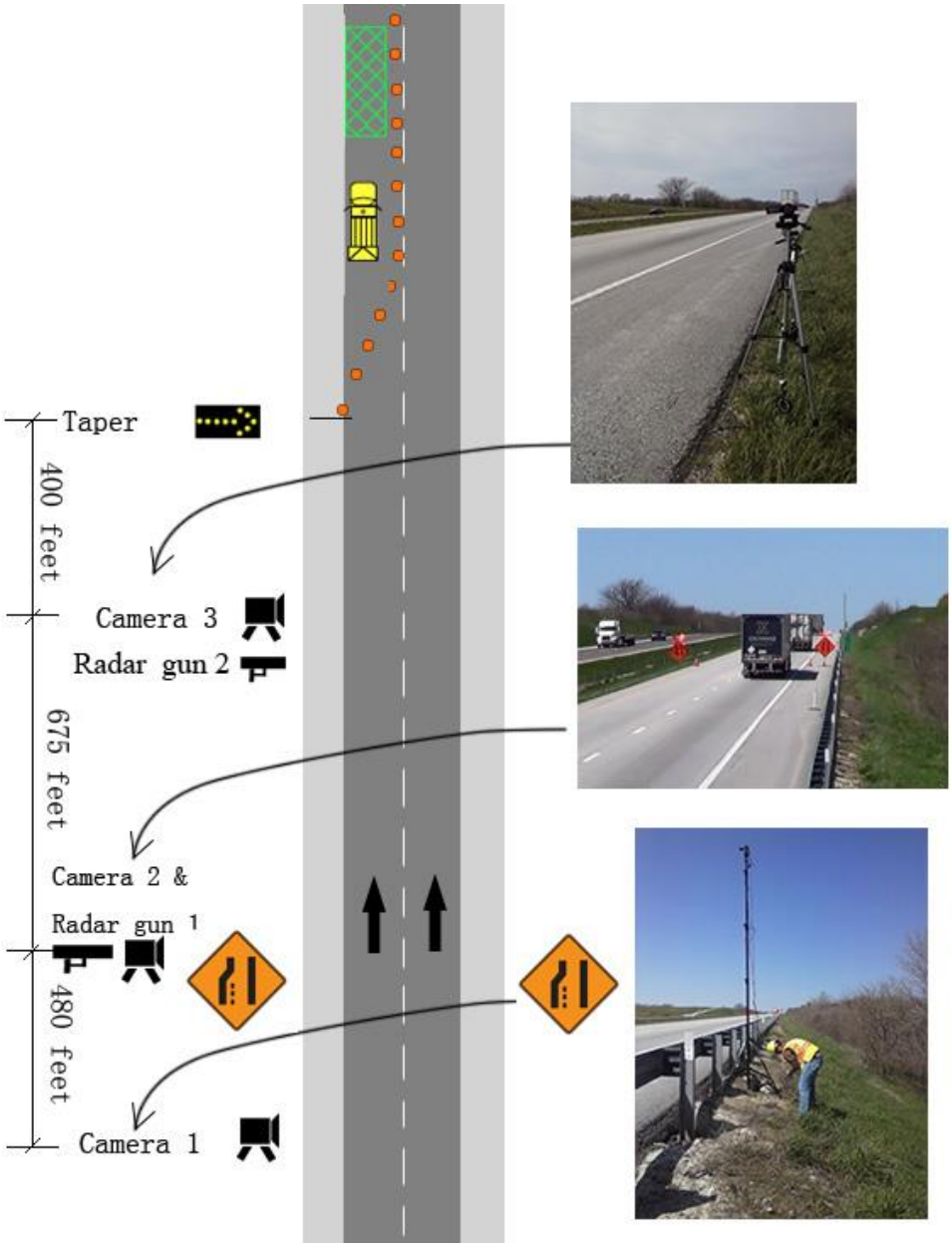


Figure 3.1.3 MUTCD plan for a stationary lane closure on a divided highway

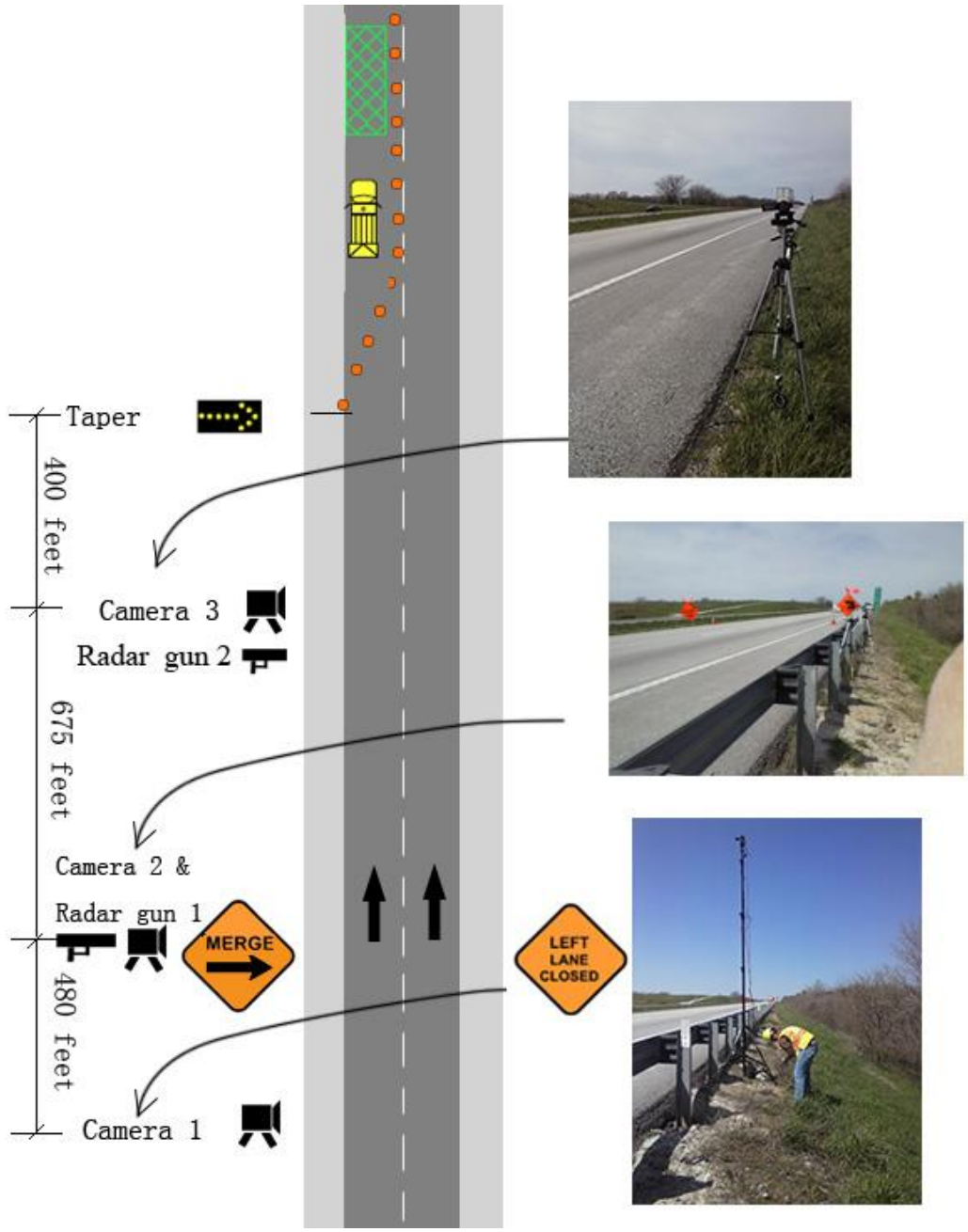


Figure 3.1.4 Test sign for a stationary lane closure on a divided highway

Camera 1 (upstream of the Merge sign): The first camera was located 480 feet upstream of the merge sign and was positioned 20 feet above ground. This camera captured the merge location data to determine where vehicles merged into the open lane.

Camera 2 (at the merge sign): A radar gun with a camera recording the speeds captured by the display was placed at the merge sign location. The radar gun was positioned so that it would start picking up vehicles from both lanes near the merge sign. The camera coverage was also used to obtain merge location data for locations 600 feet downstream of the merge sign.

Camera 3 (beginning of Taper): A radar gun capturing speeds at the beginning of taper was deployed along with an accompanying camera recording the display. This camera coverage was used to obtain merge location data 400 feet upstream of the taper. All three cameras were shooting in the direction of the taper. Camera clocks were synchronized so that all vehicle maneuvers could be monitored.

3.1.3.2. Open lane occupancy

The open lane occupancy defined as the proportion of total traffic in the open lane at a given location was computed at locations upstream and downstream of the merge sign. The location of a vehicle merge was recorded if it occurred within any of the three camera views described earlier. Every vehicle was tracked individually and visually through the area between camera 1 and the end of taper, and was divided into six zones for analysis. Figure 3.1.5 shows the six zones that were created. Whenever a vehicle merged from the left lane to the right lane, the zone in which the merging maneuver occurred was recorded.

Five delineators were used to identify the six zones in the camera coverage. Delineators were placed at 200-feet intervals for a distance of 400-feet upstream, and 600-feet downstream of the merge sign. As shown in Figure 3.1.5, zone 1 is between the first two delineators upstream of the merge sign, and zone 2 is between the second delineator and the merge sign. Zone 3 is the area between the merge sign and the third delineator. Zone 4 covers the distance between the third and fifth delineators, 400-feet upstream of the camera 3. Zone 5 is the distance between the fifth delineator and beginning of the taper. Zone 6 covers the area beyond zone 5 until the end of taper. The zones are divided in a way that merge maneuvers can be distinguished. The further close to the taper area, the shorter the distances appear during processing, due to perspective view. The lane occupancy differences were tested using a standard z test. (Milton and Arnold, 2007).

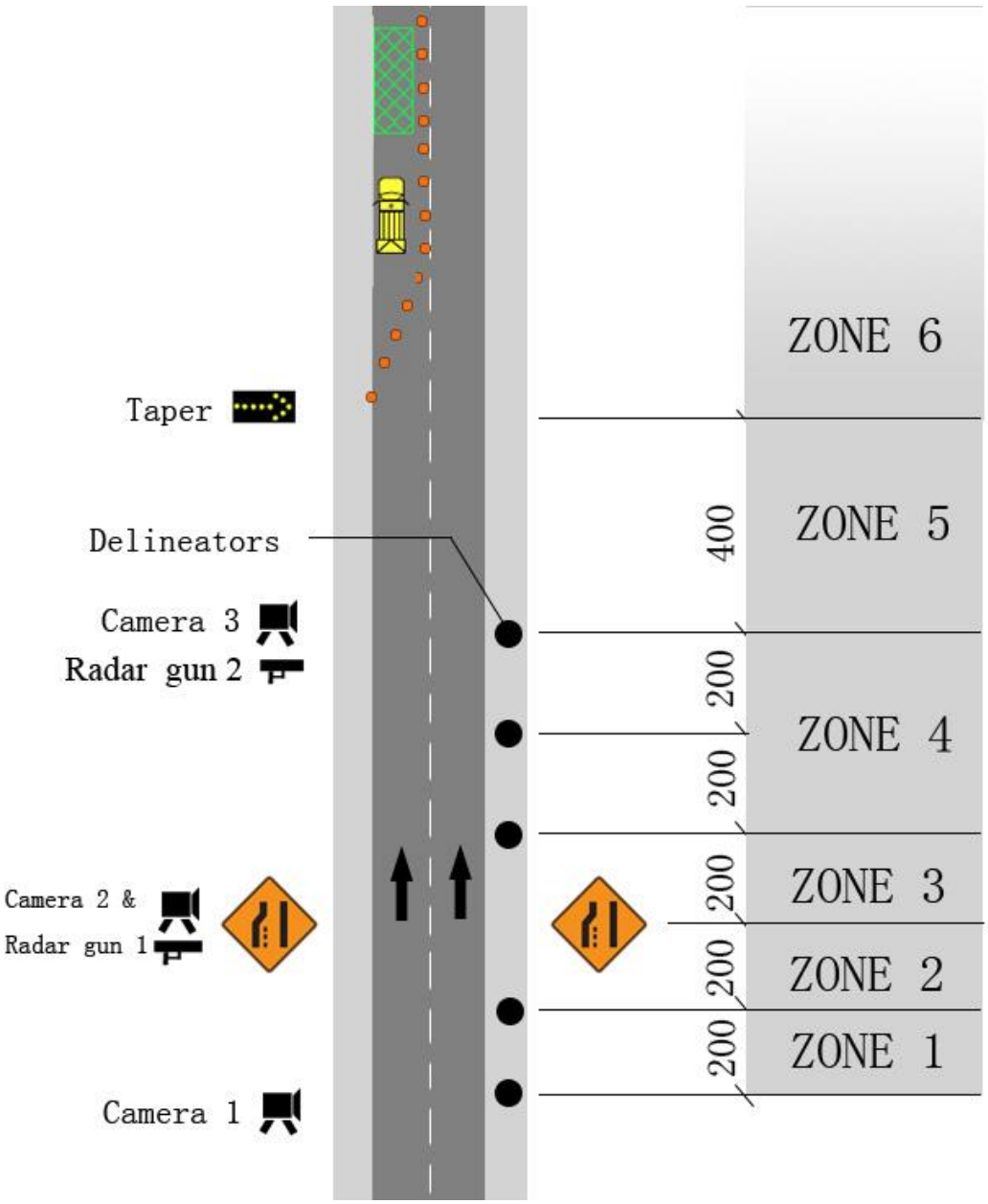


Figure 3.1.5 Analysis Zones

3.1.3.3. Case study results

The original camera views were all processed and all the vehicles were individually tracked and recorded throughout the road segment. Vehicle speeds at each radar gun were recorded and verified before aggregating. The data regarding traffic volumes and vehicle composition from each test setup were compiled for 1.5 hours and are shown in Table 3.1.1. The total number of vehicles and percentage of trucks were higher on the second day with the MUTCD configuration than on the first day with the test sign configuration. In this study, trucks were defined as all vehicles other than FHWA Classes 1 and 2, which are motorcycles and passenger cars with one or two-axle trailers including light pickups and minivans. Thus, trucks include single unit trucks and semi-and full tractor-trailers (Pickett 2012).

Table 3.1.1 Traffic Volume and Composition For The Two Sign Setups

	Test Sign (1.5 hours)	MUTCD Sign (1.5 hours)
Total Number of Vehicles	978	1041
Number of Passenger Cars	707	666
Number of Trucks	271	375
Truck percentage	27.7%	36.0%

The open lane occupancy defined as the proportion of total traffic in the open lane at a given location was computed at locations upstream and downstream of the merge sign. The low traffic volumes at the work zone site did not pose any operational issues in terms of delays or queuing. Thus, the merging locations of vehicles did not have any significant effect on the operational performance. In terms of safety, it is desirable to have vehicles occupy the open lane as far upstream of the taper as possible to avoid the likelihood of

running into the taper or work area which could result in severe crashes. The open lane occupancies at seven different locations are listed in Table 3.1.2. At the start of Zone 1, the test sign witnessed 81% occupancy in the open lane compared to 75% occupancy for the MUTCD sign. The 6% increase in open lane occupancy is desirable in terms of safety because it shows that vehicles are merging farther upstream of the work area. The open lane occupancy for the test sign continues to be higher than that of the MUTCD sign until the merge sign location. Past the merge sign, however, the open lane occupancies for both sign configurations are equal to each other. This trend is also evident in Figure 3.1.6 showing the open lane occupancies at five locations.

Table 3.1.2 Open Lane Occupancy at Different Locations (all vehicles)

Location	Distance from Merge Sign	Test Sign	MUTCD Sign	Difference	p-value
Start of Zone 1	400 feet upstream	81%	75%	6%	0.0004
End of Zone 1	200 feet upstream	82%	77%	5%	0.0022
End of Zone 2	At the merge sign	84%	82%	1%	0.1999
End of Zone 3	200 feet downstream	87%	87%	0%	0.4739
End of Zone 4	600 feet downstream	93%	93%	0%	0.4809
End of Zone 5	1000 feet downstream (Start of taper)	96%	96%	0%	0.4389
End of Zone 6	End of taper	100%	100%		

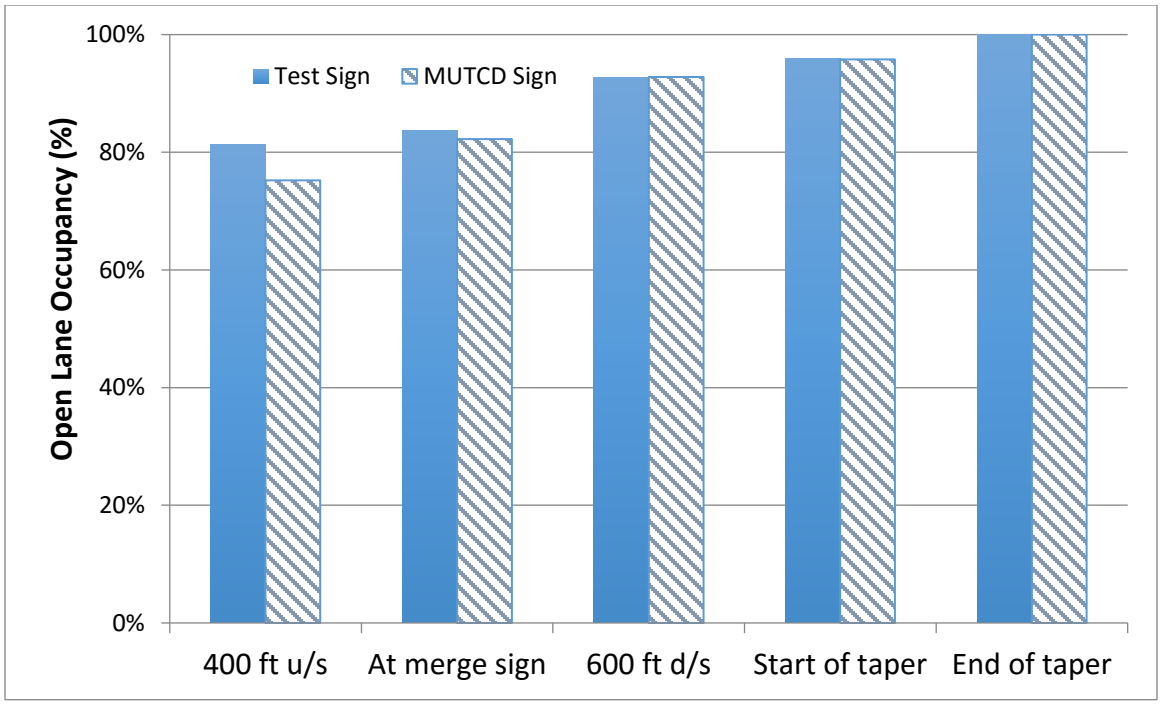


Figure 3.1.6 Open Lane Occupancies (all vehicles)

The results shown in Table 3.1.2 and Figure 3.1.6 are for all vehicles observed during the data collection period. The vehicle population was separated into passenger cars and trucks to ascertain any differences in the merging behavior across the two vehicle types. The effect of each sign setup on passenger cars are shown in Table 3.1.3 and Figure 3.1.7. The open lane occupancies at all locations until the beginning of taper were higher for the test sign than the MUTCD sign. The highest occupancy differences of 11% and 10% were observed at the two upstream locations.

Table 3.1.3 Open Lane Occupancy at Different Locations (Passenger Cars)

Location	Distance from Merge Sign	Test Sign	MUTCD Sign	Difference	p-value
Start of Zone 1	400 feet upstream	77%	66%	11%	0.0000
End of Zone 1	200 feet upstream	78%	68%	10%	0.0000
End of Zone 2	At the merge sign	80%	76%	4%	0.0391
End of Zone 3	200 feet downstream	84%	82%	2%	0.1257
End of Zone 4	600 feet downstream	92%	90%	2%	0.1172
End of Zone 5	1000 feet downstream (Start of taper)	95%	94%	1%	0.1347
End of Zone 6	End of taper	100%	100%		

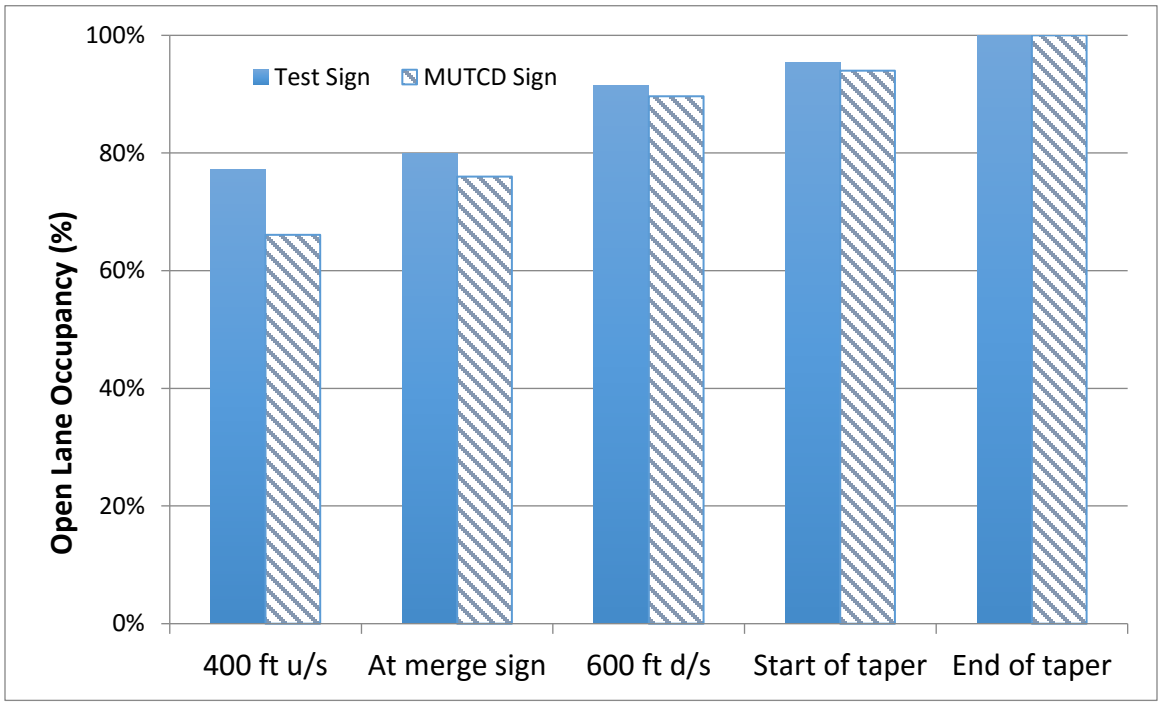


Figure 3.1.7 Open Lane Occupancies (Passenger Cars)

The open lane occupancies for trucks are detailed in Table 3.1.4 and Figure 3.1.8. The truck occupancies at all locations were higher than those observed for passenger cars for both sign setups. Several likely rationale are offered for the observed safer merging behavior of trucks compared to passenger cars. Typically, most commercial trucks trips

are work-related and the drivers thus are more likely to adopt safer driving practices such as compliance with the speed limit and early merging. Although the sight distance was not a problem at the study site, the higher line of sight for truck drivers compared to passenger car drivers helps them to detect the signage sooner thus encouraging earlier merges. Due to the work-related nature of truck trips, drivers also receive traveler information through additional means such as radio communications and third-party navigation sources that may lead to early merging. The differences in occupancies across the two signs were not as discernable for trucks as they were for passenger cars. Upstream of the merge sign, the performance of test sign was slightly better than or the same as the MUTCD sign. This trend reversed downstream of the merge sign where the performance of MUTCD sign was slightly better than or same as that of the test sign.

Table 3.1.4 Open Lane Occupancy at Different Locations (Trucks)

Location	Distance from Merge Sign	Test Sign	MUTCD Sign	Difference	p-value
Start of Zone 1	400 feet upstream	92%	91%	1%	0.3600
End of Zone 1	200 feet upstream	92%	92%	0%	0.4535
End of Zone 2	At the merge sign	93%	93%	0%	0.4951
End of Zone 3	200 feet downstream	95%	96%	-1%	0.1888
End of Zone 4	600 feet downstream	96%	98%	-2%	0.0270
End of Zone 5	1000 feet downstream (Start of taper)	97%	99%	-2%	0.0708
End of Zone 6	End of taper	100%	100%		

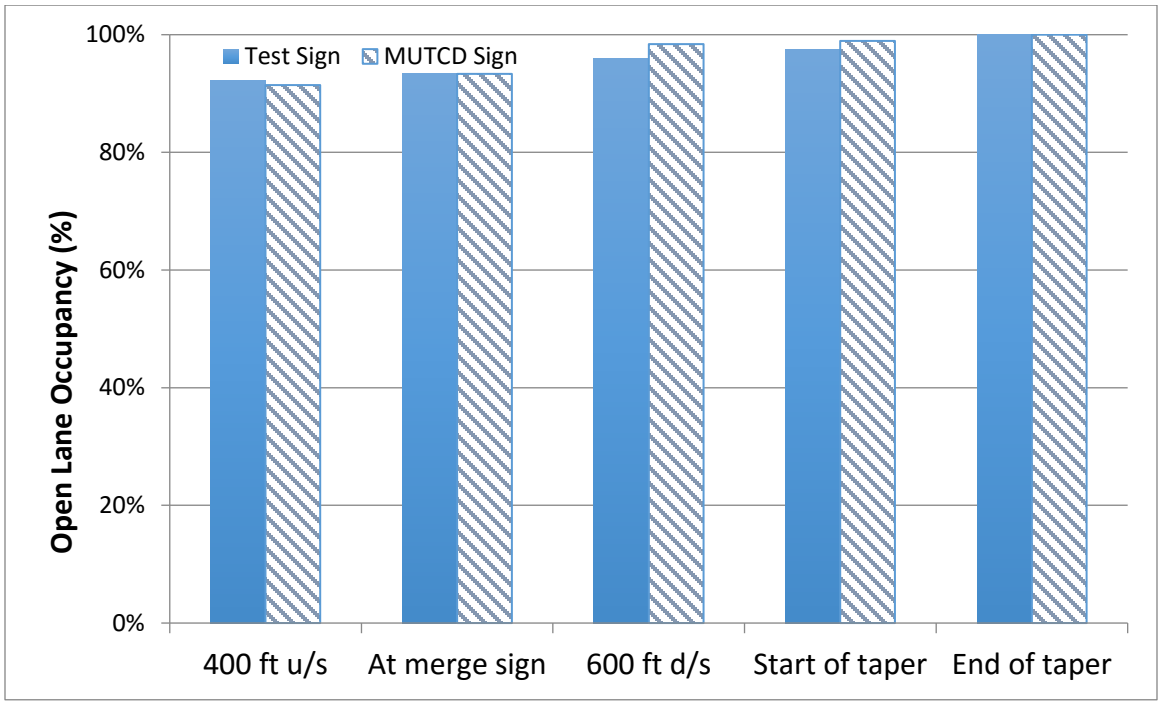


Figure 3.1.8 Open Lane Occupancies (Trucks)

In summary, the open lane occupancy values were higher for the test sign compared to the MUTCD sign upstream of the merge sign. The occupancy values between the merge sign and the taper were similar for both the test and MUTCD signs. Passenger car drivers stayed in the closed lane longer, or closer to the taper, than truck drivers. The test sign encouraged up to 11% more traffic to be in the open lane upstream of the merge sign

than the MUTCD sign. A high percentage of truck drivers, upward of 90%, already switched to the open lane upstream of the merge sign. Thus, the merging behavior of truck drivers did not vary significantly with the type of merge sign deployed in the work zone. Since it is desirable to have vehicles occupy the open lane as far upstream of the taper as possible to avoid the likelihood of severe crashes in the work area, the test sign proved to be a good alternative to the MUTCD sign.

Table 3.1.5 displays the descriptive statistics of speeds for all vehicles and by vehicle type for passenger cars and trucks. The statistics include mean speed, standard deviation of speeds, 85th percentile speeds at two locations – at the merge sign and at the taper. Statistical significance as indicated by p-values is reported for comparing means using the t-test and variances using the F-test. The speed differential between the two locations was also computed for each vehicle, i.e., the increase or decrease in speeds from merge sign to taper. The speed differentials for all vehicles were averaged and reported in the last column of Table 3.1.5. The positive sign of the mean speed differential indicates a decrease in the speeds from the merge sign to the taper.

The speeds at the merge sign and at the taper are slightly lower for the MUTCD sign than the test sign. The magnitude of the differences in mean speeds between the two sign setups was quantified using the Cohen's effect size measure (Cohen, 1988). The small values of this measure reported in Table 3.1.5 indicate that the differences in mean speeds were small. The mean speed differential values were slightly greater for the MUTCD sign than the test sign.

Table 3.1.5 Descriptive Statistics of Speeds

	Location						
All vehicles	At merge sign			At taper			Mean Speed Differential (mph)
	Speed statistics (mph)			Speed statistics (mph)			
Sign Type	Mean	Standard deviation	85th percentile	Mean	Standard deviation	85th percentile	
Test sign	66.57	5.46	72	65.05	5.74	72	2.52
MUTCD sign	65.27	5.47	71	63.14	6.01	70	2.75
p-value	<0.001	0.465	0.004	0.00	0.069	<0.001	0.078
Cohen's	0.238			0.324			
Passenger Vehicles							
Test sign	68.07	5.27	73	66.32	5.75	73	2.82
MUTCD sign	66.80	5.61	72.95	64.39	6.44	71	3.17
p-value	<0.001	0.047	0.456	<0.001	0.001	<0.001	0.045
Cohen's	0.233			0.316			
Trucks							
Test sign	62.67	3.74	67	61.72	4.16	66	1.72
MUTCD sign	62.55	3.95	67	60.91	4.35	65	2.01
p-value	0.345	0.183	0.5	0.009	0.236	0.027	0.100
Cohen's	0.030			0.190			

In summary, the speed analysis did not show any substantial differences between the test sign and the MUTCD sign. Thus, the test sign could be considered as a good alternative to the MUTCD sign given its better open lane occupancies and similar traffic speeds.

In the future, additional case studies with signs deployed at a work zone for a longer time period may be considered. However, the duration of the sign use depends on what the DOT is willing to allow.

3.2. Evaluation with microscopic simulation

3.2.1. VISSIM

VISSIM is a microscopic, stochastic, discrete time-step based simulation where individual vehicles represent the most basic elements of the simulation. It is based on the Wiedemann "psycho-physical" car-following model and lane changing model. The characteristics and behavior of individual vehicles (and drivers) affect performance measures such as speed, throughput, and queue length. One goal of the user of the simulation is to try to duplicate the field performance measures using simulation. The car-following model that represents freeway conditions, Wiedemann 99 car following model (W-99), has 10 user defined driving behavior parameters: CC0, CC1, CC2…, CC8, CC9 (PTV America, 2008). In the model, the driver can be in one of four driving modes: Free driving, Approaching, Following, and Braking. In W-99 a driver either accelerates or decelerates to change from one driving mode to other as soon as some threshold value expressed in terms of relative speed and distance is reached. Thus the whole car following process is based on repetitive acceleration or deceleration of individual vehicles with drivers having different perceptions of speed difference, desired speed, and the safety distance between two successive vehicles. Here is a brief description of the 10 driving behavior parameters used in W-99 car following model.

CC0 is the standstill distance, which defines the desired distance between two consecutive vehicles at stopped condition. The default value is 4.94 feet.

CC1 is the desired time headway for the following vehicle. Based on these values the safety distance can be computed as dxsafe = CC0+CC1* v, where v is the speed of the vehicle (PTV, 2007). The default value is 0.9. Higher CC1 values

characterize less aggressive drivers. CC2 defines the threshold that restricts longitudinal oscillation beyond safety distance in a following process. The default value is approximately 13 feet.

CC3 characterizes the entry to the "following" mode of driving. It initiates the driver to decelerate upon recognizing a slower leading vehicle. It defines the time at which the driver starts to decelerate before reaching the safety distance.

CC4 and *CC5* control the speed oscillations after the vehicle enters the "following" mode of driving. Smaller values represent a more sensitive reaction of the driver to the acceleration or deceleration of the leading vehicle. CC4 is used for negative speed difference and CC5 is used for positive speed difference. The default value of CC4/CC5 is -0.35/0.35.

CC6 represents dependency of speed oscillation on distance in the "following" state. Increased value of CC6 results in an increase of speed oscillation as the distance to the preceding vehicle increases. CC7, CC8, and CC9 parameters control the acceleration process.

The lane changing model in VISSIM is based on the driver response to the perception of the surrounding traffic. It uses gap acceptance criteria where driver changes lanes provided the available gap is greater than the critical gap. The decision to change lanes depends on the following hierarchical set of conditions: the desire to change lanes, favorable driving conditions in the neighboring lanes, and the possibility to change lanes (gap availability). Based on these conditions the lane changing phenomenon is broadly classified into two types: 1) discretionary lane change which includes drivers who want to change from slow moving lanes to fast moving lanes and, 2) necessary lane change in

case of any lane closure due to work zones, incidents, etc. A detailed description of the lane changing algorithm is presented in Wiedemann and Reiter (1992).

Necessary lane changes depend on the aggressiveness of drivers in accepting/rejecting gaps in the adjacent lanes that is represented by parameters such as acceptable and threshold deceleration values of lane changing and trailing vehicles, and the safety distance reduction factor (called *SRF*). The safety reduction factor refers to the reduction in safety distance (*dxsafe*) to the trailing and leading vehicle on the desired lane and the safety distance to the leading vehicle in the current lane. The default value of SRF is 0.6 which means the safety distance during lane changing is reduced by 40%. A lower SRF value (say 0.4) would mean that the safety distance for lane changing is reduced by 60% meaning the drivers have become more aggressive in accepting shorter gaps.

3.2.2. Calibrating Capacity using travel time data

3.2.2.1. Case study: Work Zone on I-44 at Antire road.

The first work zone site was located, southwest of St. Louis, on I-44 between Antire Rd. and Lewis Rd. The work activity involved road resurfacing from June 1, 2012 to October 19, 2012. No alternative routes were available due to its rural setting. The average annual daily traffic as of 2011 was 68,181 including 8,020 heavy vehicles (11.8% of the AADT). The original speed limit of 65 mph was reduced to 55 mph in the work zone. This long-term work zone had lane closures in both travel directions. One lane out of three lanes was closed in each direction. Because of the high demand in the westbound direction, one additional lane was opened from 3:00 pm to 7:00 pm on weekdays.

A map showing the work zone and the locations of traffic sensors is provided as Figure 3.2.1. Traffic data was extracted from the four ASTI sensors maintained by contractors, Q01 to Q04, deployed in the eastbound direction as shown in Figure 3.2.1. Travel time and delay values were computed using data obtained from the two Bluetooth sensors. Four portable dynamic message sign (DMS) trailers were used in the work zone to display real-time delay and queue information.

Sensor Q04 was located at the beginning of the eastbound work zone while sensor Q05 was at the beginning of the westbound work zone. The distances between adjacent sensors in each direction are shown in Figure 3.2.1.

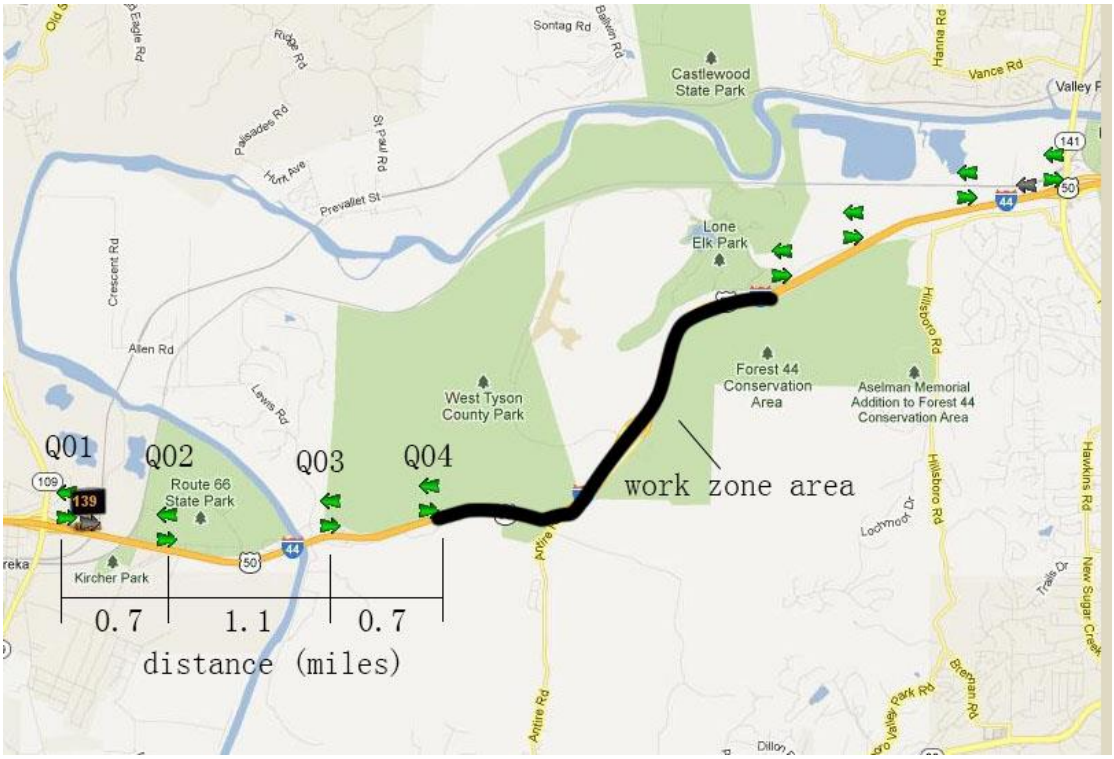


Figure 3.2.1 Location of The Work Zone And Traffic Monitoring Equipment (Google maps 2013)

Traffic sensor data was available from May 15th, 2012, to October 17th, 2012. There was some missing traffic data for two weeks in June, thus that period was excluded from the analysis. After carefully examining the entire dataset, four days, July 10th, July 12th, July 17th and July 19th were chosen for the calibration process. These days had the highest flow rates and longest queue length (more than 0.7 miles) observed throughout the entire work zone period. Only eastbound work zone data was used for calibration since one travel lane was always closed, which resulted in a 3-to-2 work zone. The hourly traffic volumes with and without work zone for the eastbound direction are shown in Figure 3.2.2.

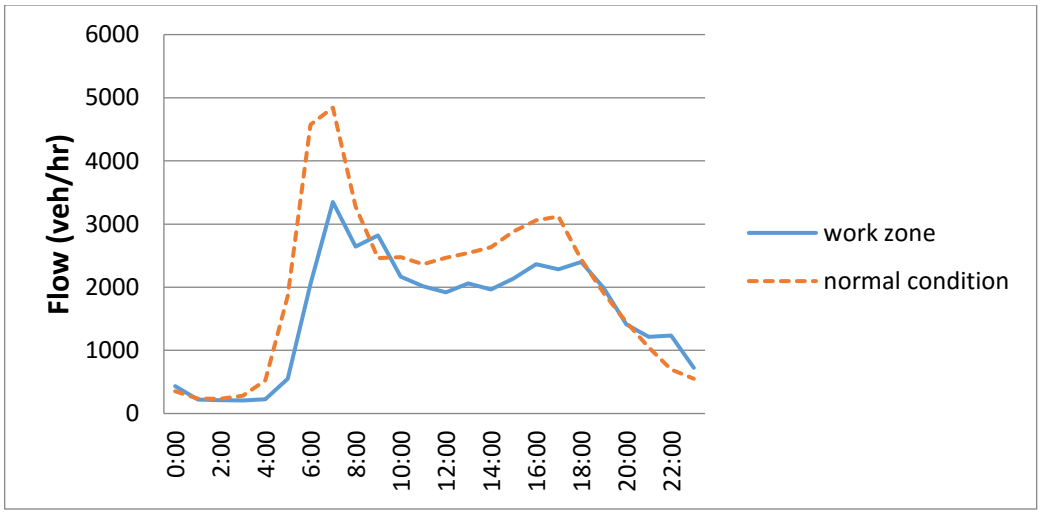


Figure 3.2.2 Hourly Volumes With and Without Work Zone at The Eastbound I-44 Site

Travel times were calculated based on the time stamp recorded by Bluetooth sensors. The two sensors were spaced at 7.3 miles. The travel times however did not show any significant delays due to the work zone. The placement of Bluetooth sensors was such that vehicles usually made up for any lost time in the work zone by speeding downstream of the work zone. Thus, the Bluetooth data did not allow capturing the delay caused by the work zone. An alternative method was used to capture the work zone delay. The speed data obtained from traffic sensors inside the work zone was used to compute the travel times. The section selected for travel time calculation was from sensor Q01 to Q04, the start of the eastbound work zone taper. Free flow travel times were also computed using the free flow speeds determined from the speed-occupancy plots. The free flow speed was computed by averaging the speed values at low occupancies, <2%. The computed free flow speed values at the four locations were: 68.6 mph at Q01, 66.1 mph at Q02, 64.6 mph at Q03, and 60.4 mph at Q04. Queue length was determined using

the same four sensors, Q01 to Q04. The queue length increases upstream of the taper (located at Q04) to 0.7 mi at Q03 to 1.8 mi at Q02 to 2.5 mi at Q01.

The calibration of VISSIM simulation model involved identifying the calibration parameters and determining the optimal value for that parameter that generates the most accurate delay and/or queue length estimates. Several driving behavior parameters control capacity and are provided as inputs. This section describes the calibration approach used for the I-44 work zone case study.

The I-44 segment including the work zone was coded in VISSIM. A screenshot of the network is shown in Figure 3.2.3. The section highlighted in brown shows the work zone.

Figure 3.2.3 I-44 Segment of Work Zone in VISSIM

The desired speed distributions were developed using speed limits and operating speeds. Two uniform distributions were used – the non-work zone speed distribution

ranging from 60 mph to 70 mph and the work zone speed distribution ranging from 50 mph to 60 mph.

The hourly traffic demand at the I-44 work zone is shown in Table 3.2.1. The morning peak period was the only time during which the demand approached capacity. Thus, only the morning peak period between 5:00 am and 11:00 am, highlighted in green in the table, was simulated. A warm-up period of 900 seconds at the beginning of simulation was used. The traffic was composed of 93% passenger cars and 7% trucks according to available on-site counts.

Table 3.2.1 Hourly Demand for The I-44 Work Zone

Time	Demand (veh/hr)
0:00 to 1:00	434
1:00 to 2:00	220
2:00 to 3:00	212
3:00 to 4:00	207
4:00 to 5:00	227
5:00 to 6:00	552
6:00 to 7:00	2048
7:00 to 8:00	3349
8:00 to 9:00	2642
9:00 to 10:00	2819
10:00 to 11:00	2169
11:00 to 12:00	2016
12:00 to 13:00	1920
13:00 to 14:00	2060
14:00 to 15:00	1966
15:00 to 16:00	2135
16:00 to 17:00	2364
17:00 to 18:00	2285
18:00 to 19:00	2400
19:00 to 20:00	1990
20:00 to 21:00	1414
21:00 to 22:00	1216
22:00 to 23:20	1234
23:00 to 24:00	721

Three performance measures, capacity, travel time, and queue length were collected from the simulations. The travel time and queue length values were used to calibrate the driving behavior parameters. Travel time was used in lieu of delay since accurate estimation of travel time also results in accurate estimation of delay. Based on the work of Edara (2009) three driving behavior parameters were chosen for calibration. These parameters are: *CC1*, *CC2*, and *SRF*. A description of these parameters was provided in an earlier chapter. The range of values for each parameter used for calibration was also based on the previous study (Edara 2009) and is shown in Table 3.2.2.

Table 3.2.2 Ranges of Driving Behavior Parameter Values

Parameters	Minimum	Maximum
CC1	0.9 sec	1.8 sec
CC2	10 feet	55 feet
SRF	0.15	0.6

After several trials the combination of *CC1* =1.3 sec, *CC2* = 35 feet, and *SRF* = 0.3, resulting in a capacity of 3,034 veh/hr was found to produce the lowest total travel time error of 2.02 minutes. The corresponding total queue length error was 2.1 miles.

4. Simulated Conflict-Based Safety Evaluation for J-turns

J-turn is an alternative highway intersection design to improve safety by reducing crossing conflicts. Since it requires a long time span to implement and collect crash data, simulated evaluation highly recommended for these situations for much shorter time. Simulation also offers more flexibility and can alter some of the design elements such as the length of the acceleration lane and length of U-turn segment for further testing scenarios. The following chapter will present a case study using simulated conflicts to find the optimum value for the distance between two U-turns under different volume scenarios in rural areas. The term U-turn spacing is used in the following chapter to describe the distance between the center of the intersection and one of the U-turns. Two types of design are tested: with or without the acceleration lanes. The procedures to be discussed in the following chapter can be extended to other alternative designs.

Coding a simulation model can be a challenging task. Creating an accurate and realistic base model requires patience and attention to detail. In this case study, one of the main obstacle in coding was: vehicles randomly changed lanes when it was not necessary. After extensive testing it was found that one key parameter in VISSIM should be varied across different spacing scenarios. As shown in Figure 4.0.1, “route: lane change distance” from the connector tab should be longer than the distance from the U-turn to the center of the intersection. This allows the vehicles to start looking for gaps to merge into the deceleration lane just before they reach the minor road which result in smooth and realistic merging maneuver.

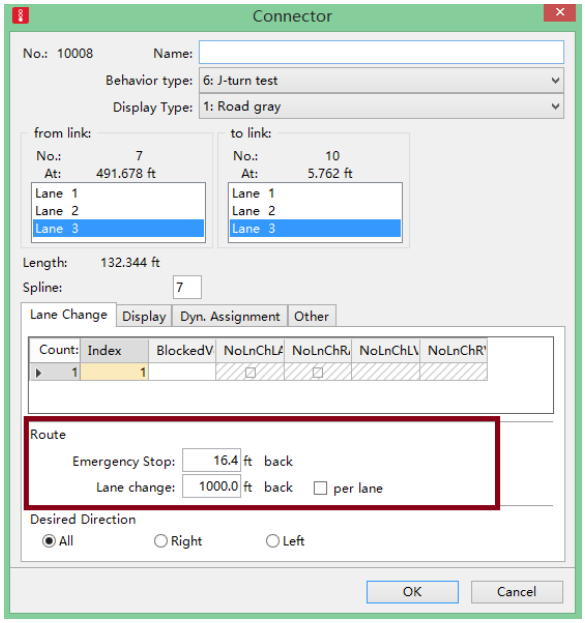


Figure 4.0.1 Connector tab from VISSIM

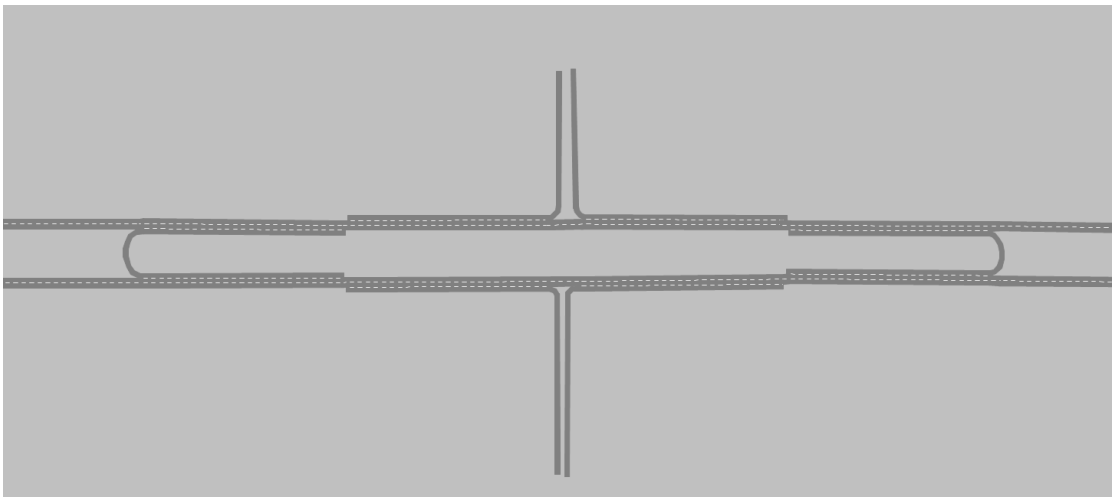
Another challenge in coding the J-turn is that the queued vehicles on the minor road extends beyond the link in high volume scenarios which result in the input volume not fulfilled. This causes the conflict numbers appearing to be low because these remaining vehicles can generate more conflicts when they are allowed onto the network. This problem is solved by simply extending the minor road so that the queue never extends beyond the link nor the vehicle input bar.

4.1. Simulation model development

4.1.1. Network Layout

Micro-simulation program VISSIM was used to analyze the effect of two different J-turn design considerations: presence or absence of acceleration lanes and the distance between the minor road and the U-turn. The simulation model used in this research is derived from the field data collected in a previous MoDOT research project from 2013 (Edara et al., 2013). The previous J-turn field site was located near Deer Park Road on Highway 63, south of Columbia, Missouri. This section of Highway 63 is a rural four-lane highway with a speed limit of 70 mph. This segment consisted mainly of tangents with no sharp horizontal curves, or steep vertical grades. The satellite image and the corresponding VISSIM simulation model layout is shown in Figure 4.1.1.

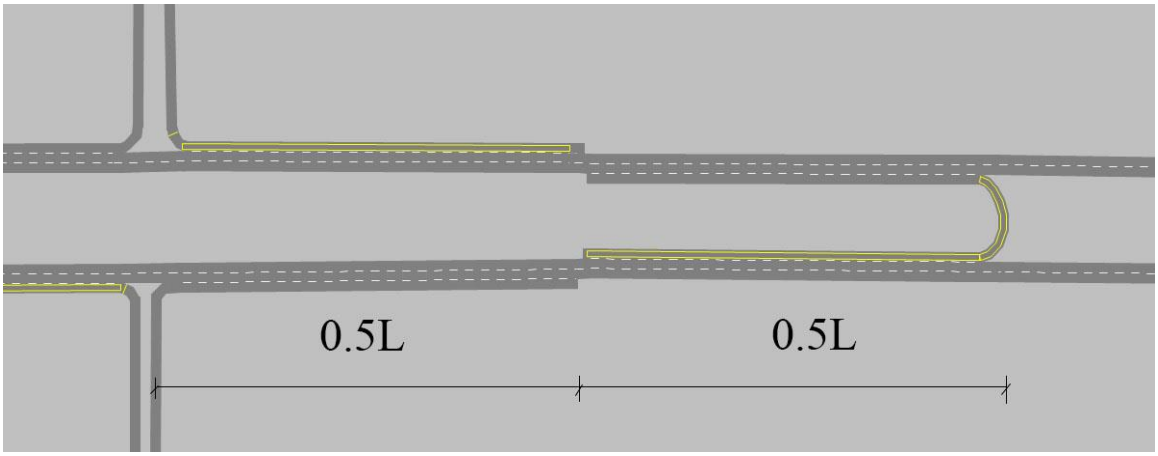
(a) Highway 63 at Deer Park Road (Google maps, 2015)



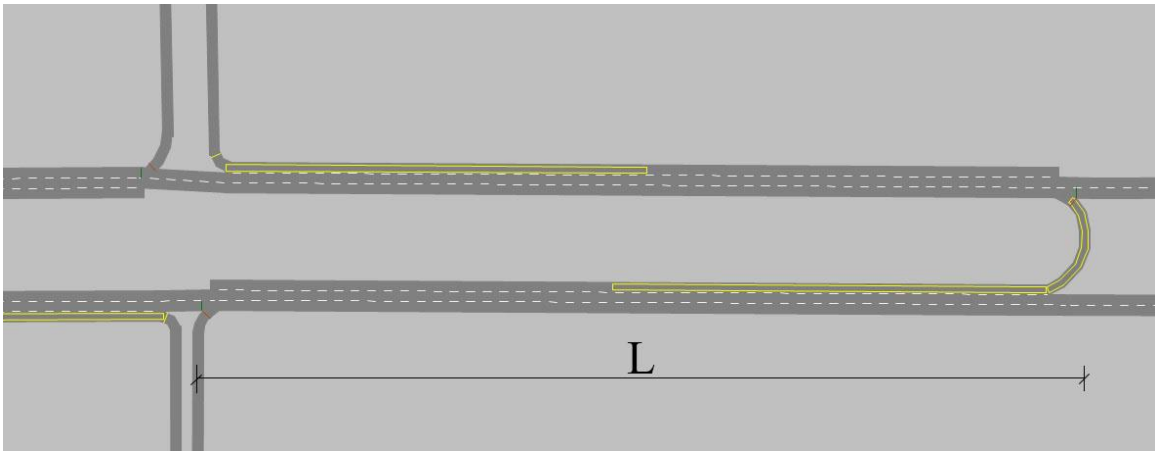
(b) VISSIM Simulation Model Layout

Figure 4.1.1 Satellite Image And Simulation Layout Of The J-Turn

For the distance between the minor road and the U-turn, three distances were analyzed–1000 feet, 2000 feet, and 3000 feet. In terms of the presence or absence of acceleration lanes, two different layouts were analyzed as shown in Figure 4.1.2. The distance from the intersection to the U-turn is noted as L and referred to as “spacing” in the following chapters. The first layout (top) includes an acceleration lane extending from the minor road to half the distance to the U-turn and deceleration lane for the U-turn starting at the end of the acceleration lane and extending to the U-turn. In the other direction, an acceleration lane is provided for vehicles merging from the U-turn lane into major road and a deceleration lane to exit to the minor road. The second layout (bottom) does not contain an acceleration lane for minor road traffic or for U-turn traffic. The deceleration lane extends the entire length between the U-turn and the minor road. These two layouts were recommended by the project’s technical advisory panel comprised of MoDOT safety engineers.



(a) Layout 1: acceleration lane present



(b) Layout 2: accelration lane not present

Figure 4.1.2 J-turn layouts with and without acceleration lane

Several parameters in VISSIM were optimized in order to accurately simulate vehicles at a J-turn. These parameters included: reduced speed areas (length and magnitude), desired speed decisions, and lane change distance upstream of a connector. For example, Figure 4.1.3 shows the lane change distance parameter window in VISSIM. This parameter specifies the upstream distance from a connector where vehicles start to look for lane changing gaps to stay on their desired path. This parameter value was based on trial and error through manual observation of the simulations. The value was different for the two layouts.

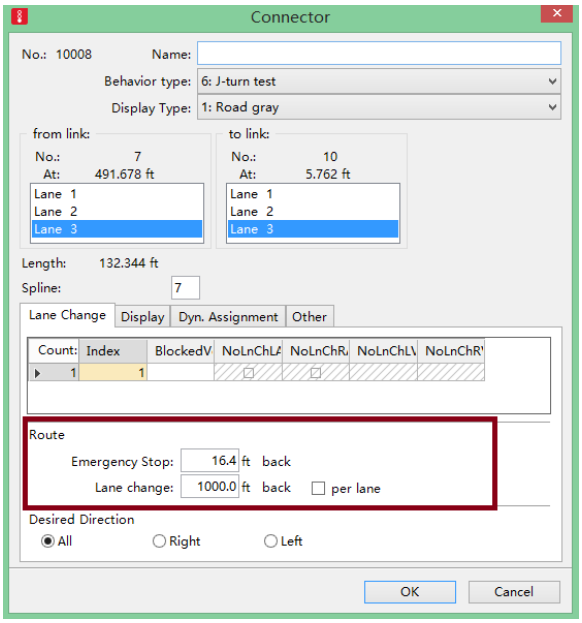


Figure 4.1.3 Connector tab from VISSIM

4.1.2. Speed Distribution

The calibration procedure in this study used disaggregated data of individual vehicle speeds measured in the field in a previous project (Edara et al., 2013). Thus, the calibration procedure was more robust than the state of practice that relies on aggregated sensor speeds on a roadway. A map of the field data collection equipment placement used in Edara et al. (2013) is shown in Figure 4.1.4. Several cameras and radar guns were used to extract traffic volumes and vehicle speeds (see Figure 4.1.5). The AM peak period data was collected in the Southbound direction and PM peak period was collected in the Northbound direction.

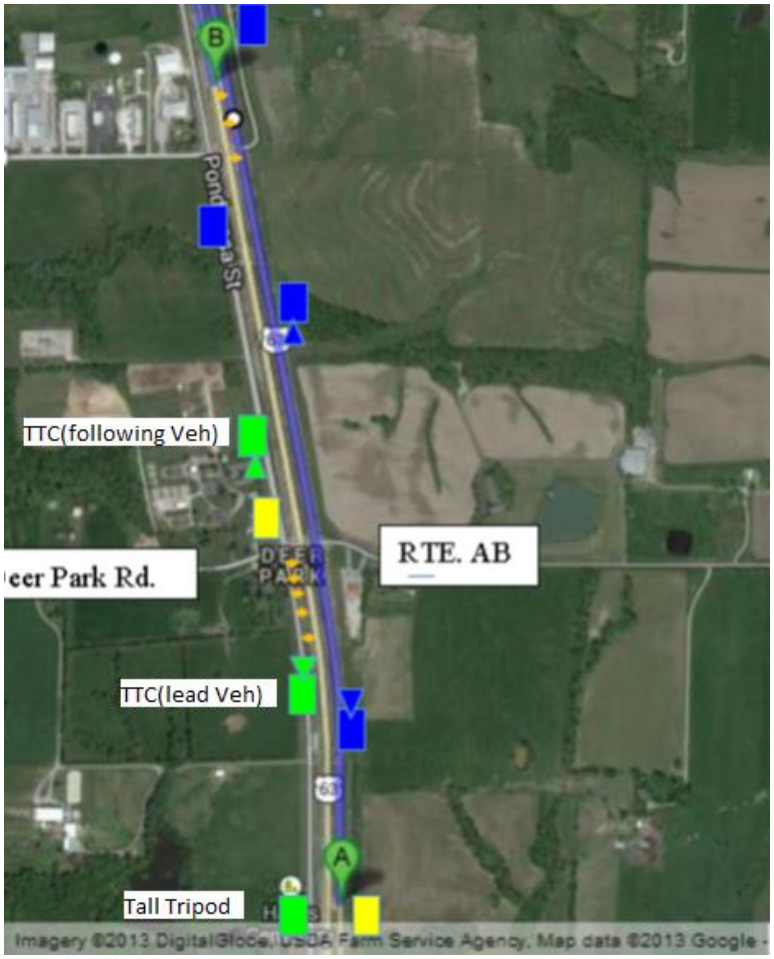


Figure 4.1.4 Data collection equipment in Edara et al. (2013)

Figure 4.1.5 Radar speed gun view

The speed distribution of merging vehicles from the minor road (Route E) and the major road vehicles are shown in Figures 4.1.6 and 4.1.7. The 85th percentile speeds of passenger cars and trucks on the major road were 75 mph and 70 mph and for merging vehicles was 64 mph.

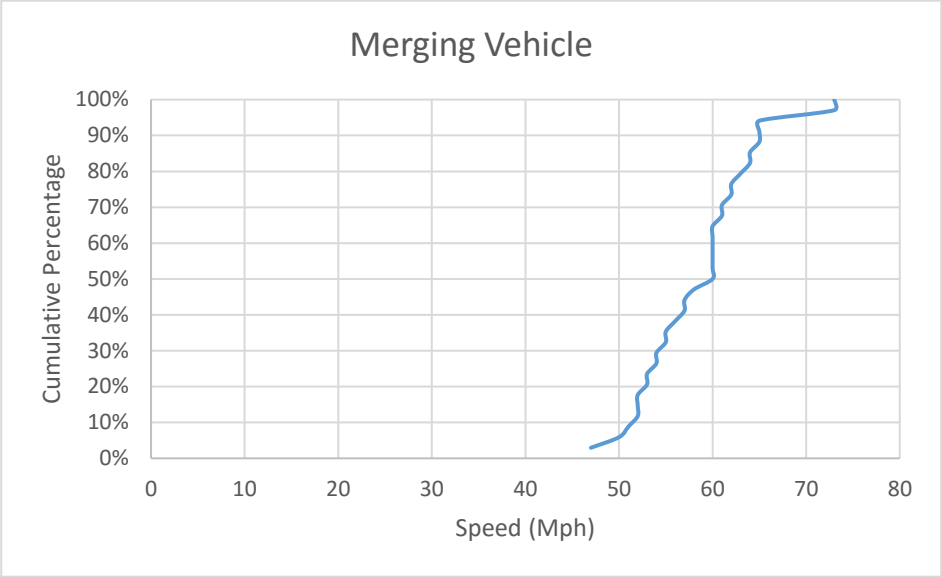


Figure 4.1.6 Merging Vehicle Speed Distribution

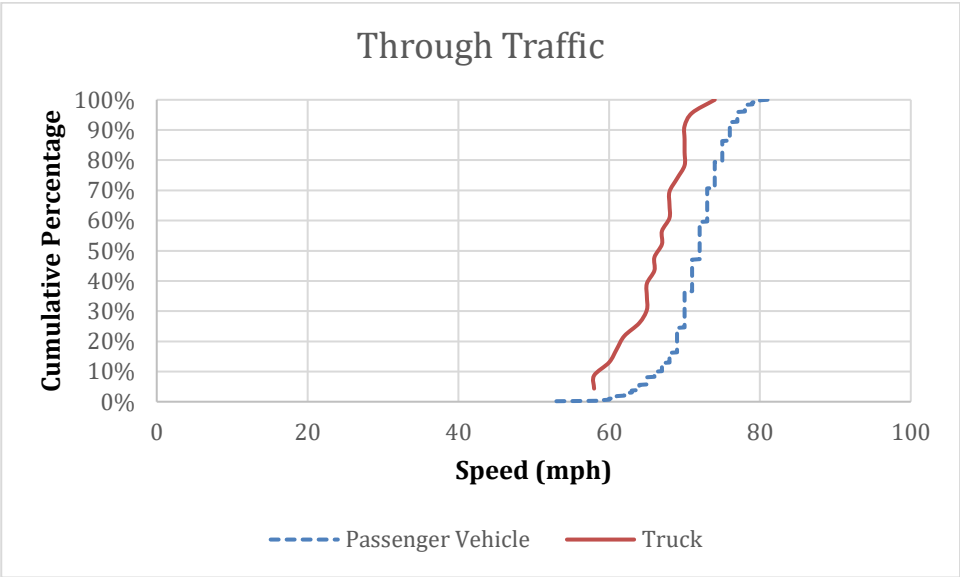
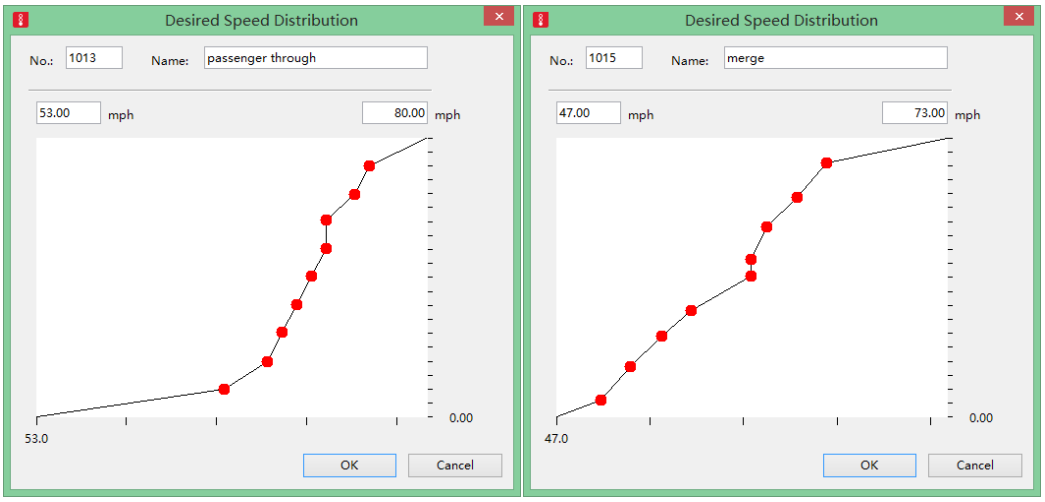


Figure 4.1.7 Through Traffic Speed Distribution

These speed distributions were then defined in VISSIM using the desired speed distribution parameter window shown in Figure 4.1.8.



(a)Passenger Through (b)Merging/Diverging

Figure 4.1.8 Desired Speed Distribution in VISSIM

4.1.3. Base Vehicle Composition and Routing Decisions

Vehicle composition is obtained through one-hour volume count from 7:30 a.m. to 8:30 a.m. and is listed in Table 4.1.1. The truck percentage after route E is about 2.6%.

Table 4.1.1 Southbound volume on Hwy63

Time period		Route E right turn (on ramp)		Southbound		Southbound right turn (off ramp)	
		Passenger Vehicle	Truck	Passenger Vehicle	Truck	Passenger Vehicle	Truck
7:30:02	8:30:06	83	3	1401	42	36	7

Figure 4.1.9 AM high tripod-U-turn cutoff

Two views （Figure 4.1.9）around the U-turn are played at the same time to follow each vehicle that uses the U-turn. As a result, a total of 25 vehicles were recorded using the northbound U-turn on Hwy 63 over one-hour period, from 4:50 p.m. to 5:50 p.m. Table 4.1.2 summed up all the movements. It seems that it was not the peak hour and corresponding traffic volume needs to be collected to determine the traffic assignment ratio.

Table 4.1.2 J-turn Vehicle Movements

Movement	From	To	Total
1	hwy 63 N	Deer Park	18
2	Route E	Deer Park	3
3	Route E	hwy 63 S	4
4	hwy 63 N	hwy 63 S	0

Total flow from the south was 1403 vehicles/hour which was relatively low compared to morning peak. Truck percentage of the major road was 2.71%. The total flow from Route E was 57 vehicles/hour which indicated a major-minor road flow ratio of 24.61:1. Vehicle routing ratios were calculated based on the hourly volume count from the evening peak. According to Table 4.1.3, only 1.3% of the volume from the major road used the J-turn to turn left; and 5.3% of the minor road volume used the J-turn to cross the highway 63.

Table 4.1.3 Routing Ratios

%	Hwy 63	Route E
Through	93.4%	5.3%
Right Turn	5.3%	87.7%
Left Turn	1.3%	7.0%
Total	100%	100%

These ratios were assumed the same for both directions for the base condition.

Table 4.1.4 shows the base volume scenario used. The third column (titled Diagram) graphically illustrates each movement. The major road volumes shown in Table 4.1.4 were obtained from the field data discussed earlier. The field-observed minor road volumes were low and did not generate enough conflicts to be useful for safety analysis. Thus higher values were used.

Table 4.1.4 Base Condition Major and Minor Road Flow Rates

No.	Movement	Diagram	Veh./Hour	Total
1	Major road through	→	1443	1504
2	Major road left turn	↰	18	
3	Major road right turn	↳	43	
4	Minor road through	↑	22	308
5	Minor road left turn	↰	16	
6	Minor road right turn	↱	270	

4.2. Safety Analysis

4.2.1. Volume Scenarios

In addition to different U-turn spacing scenarios, different volume scenarios were generated for analyzing the J-turn performance. Simulations were run five times for each scenario and the averaged value of conflicts was recorded.

Table 4.2.1 shows all the 12 major and minor road flow combinations. The "Minor Road Crossing" flow column includes both minor road left-turns and minor road through movements and they each represent half of the total minor road crossing volume. The volume scenarios ranged from low volume to high volume. These twelve volume scenarios were then studied for the three U-turn spacings of 1000 feet, 2000 feet, and 3000 feet, and for the presence/absence of acceleration lane, thus resulting in a total of 72 combinations.

Table 4.2.1 Volume Scenarios

	Major Road Total (veh./hour)	Minor Road Crossing (veh./hour)	Minor Road Right Turn (veh./hour)	Total Minor/Major ratio
1	1000	150	150	30%
2	1000	250	250	50%
3	1000	350	350	70%
4	1300	195	195	30%
5	1300	325	325	50%
6	1300	455	455	70%
7	1504	226	226	30%
8	1504	376	376	50%
9	1504	526	526	70%
10	1800	270	270	30%
11	1800	450	450	50%
12	1800	630	630	70%

4.3. SSAM Setup

SSAM, FHWA's Surrogate Safety Assessment Model, has an option where unrealistic conflicts (e.g. TTC (time to collision) =0) can be filtered from the output. In this dissertation, most of the default parameters were used. Only lane change conflicts were of interest in this dissertation as J-turns eliminated crossing conflicts from the minor road movements. Figure 4.3.1 shows the filters used in this study for all volume and design scenarios. The SSAM user manual provides guidance on selecting the threshold values for the filter (Gettman and Head, 2003).

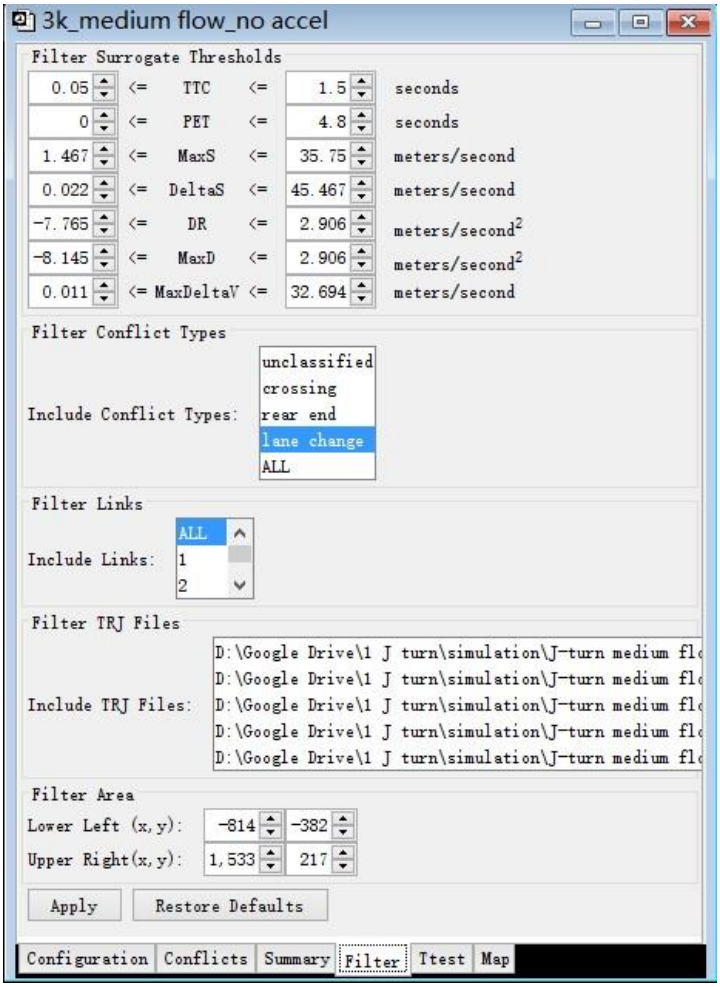


Figure 4.3.1 Applied SSAM filter of the Conflicts Analysis

4.4. Results Analysis

4.4.1. Overview

The conflicts collected for each scenario are averaged from five simulation runs. The number of non-zero TTCs for all scenarios are listed in Table 4.4.1. This dissertation will discuss these results further in the following chapters

Table 4.4.1 Conflicts for all Scenarios

Major Total	Minor/Major	Conflicts (average from 5 runs)			
		1k	2k	1k_no accel	2k_no accel
1000	30%	1.2	0.6	8.4	7
1000	50%	2.2	1	15.6	13.4
1000	70%	5.8	1	26.8	21.8
1300	30%	2.8	1	16.6	13.6
1300	50%	7.8	1	40.2	32.8
1300	70%	17	3	70	61.6
1504	30%	7	2	27.8	24.8
1504	50%	14.4	4.4	75	60.4
1504	70%	35.8	4.8	109.8	93.6
1800	30%	16.2	3.8	55.4	44.6
1800	50%	35.6	5.8	109.6	87.4
1800	70%	68.4	17.8	131.8	106.4

4.4.2. Designs with Acceleration Lanes

Figures 4.4.1 (a to d) show the average conflicts registered by SSAM from all 12 volume combinations. Conflicts are grouped by major road volume. In each chart, the x axis stands for the minor road crossing volume and the y axis stands for the conflicts. In addition to the crossing volume, an equal number of right turning vehicles were also simulated. For example, the total minor road volume for the scenario with 150 veh./hr. crossing volume is 300 veh./hr. Minor road left turning vehicles consisted of half of the total minor road volume. Each scenario was run five times using different random seeds in VISSIM and the results were averaged across the five runs. Striped bars in figures 4.4.1 (a to d) indicate 1000 feet (or 1k) spacing, squared bars indicate 2000 feet (or 2k) spacing, and dotted bars indicate 3000 feet (or 3k) spacing.

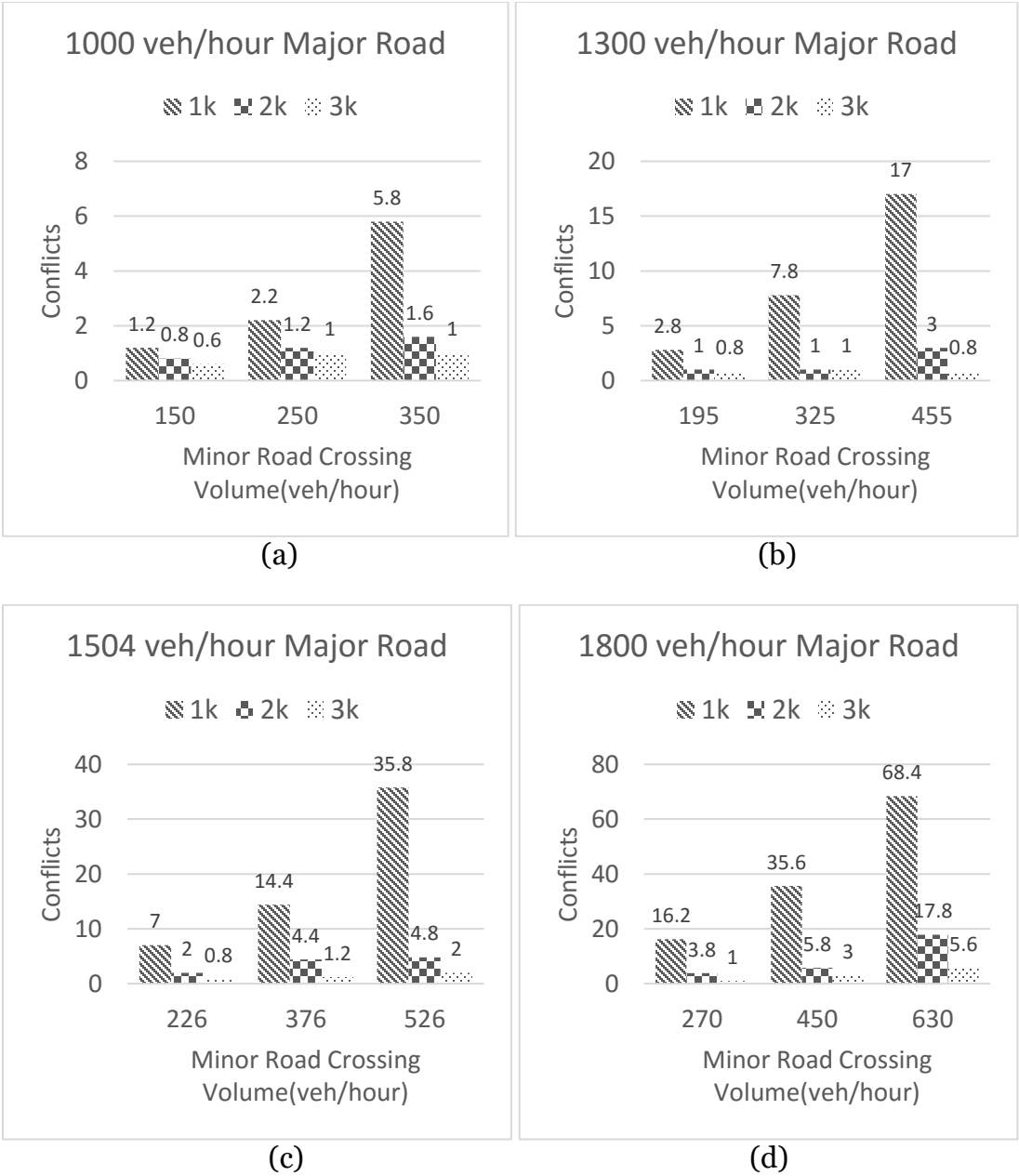
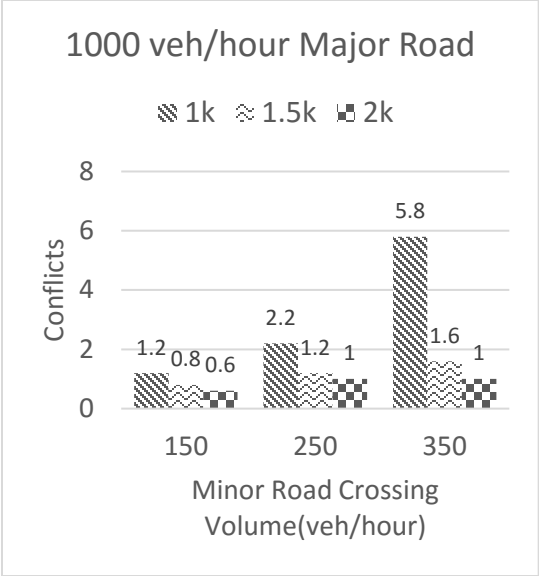


Figure 4.4.1 Conflict Counts for Designs with-acceleration Lane

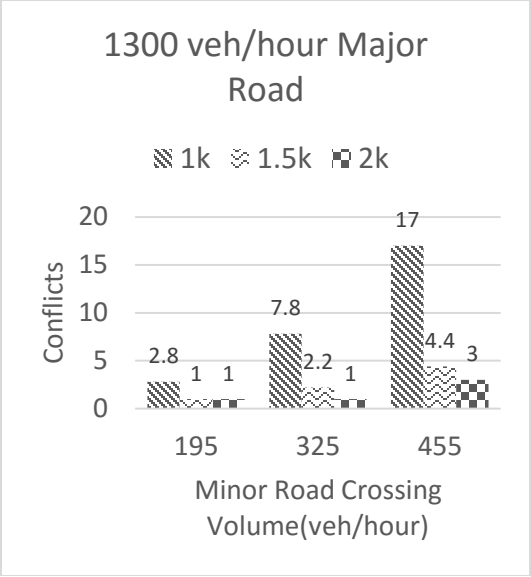
The results were consistent for all volume scenarios. The number of conflicts decreased with an increase in the spacing between the minor road and the U-turn. For example, the lowest volume combination, 1000 veh./hour total on the major road and 150 veh./hour on the U-turn, witnessed 1.2 conflicts for 1000 feet spacing, 0.6 for 2000 feet

and 0.2 for 3000 feet. This effect is more significant when the traffic volume is higher. For the highest volume scenario of 1800 veh./hour on the major road and 630 veh./hour on the U-turn, the number of conflicts dropped from 68.4 to 17.8 for 2000 feet, a difference of 50.6, and to merely 5.6 for 3000 feet.

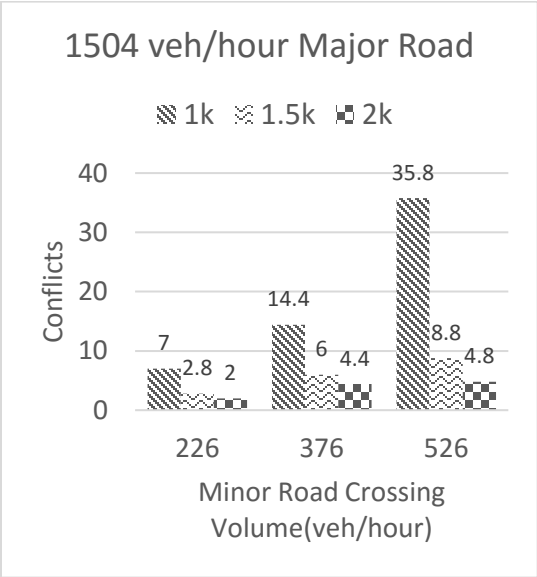
Although it is clear from the results that longer spacing values decreased the number of conflicts, the reduction of conflicts is not linear. For example, the second heaviest volume combination results in a 31 reduction in conflicts from 1000 feet to 2000 feet and merely a 2.8 reduction in conflicts from 2000 feet to 3000 feet. Thus, a spacing of 2000 feet may be sufficient for providing a good trade-off between safety and cost-effective J-turn design. To further explore the effect of U-turn spacing on conflicts, a spacing of 1500 feet was examined. The results for 1500 feet spacing are compared with 1000 feet and 2000 feet values in Figure 4.4.2.



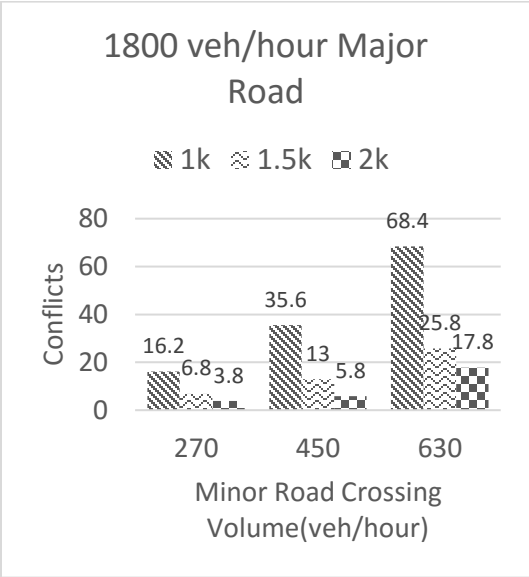
(a)



(b)



(c)



(d)

Figure 4.4.2 Conflicts reduction among 1k, 1.5k and 2k

Table 4.4.2 Reduction in Conflicts

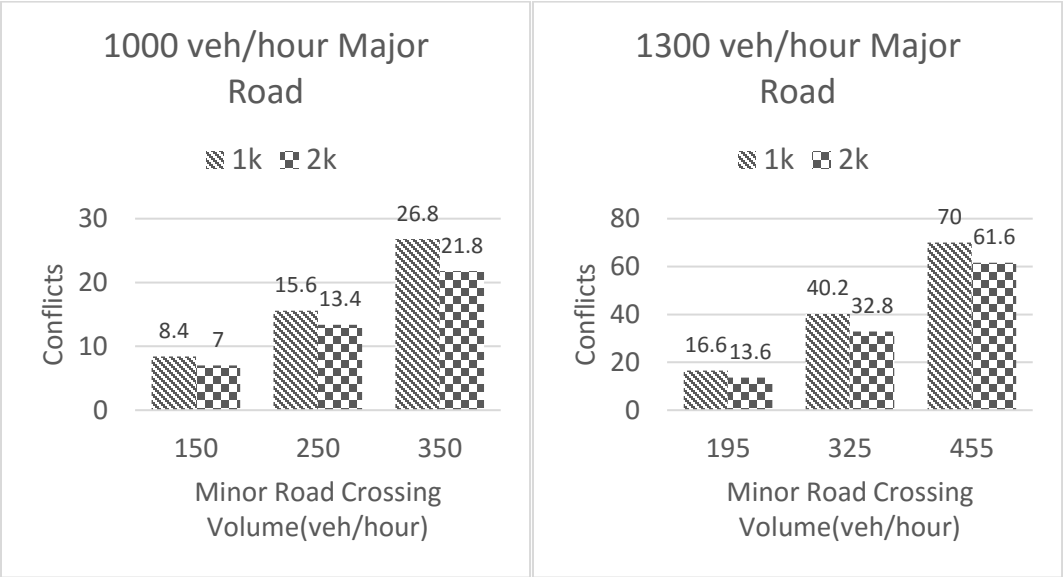
Major Total	Minor Crossing	Minor/Major ratio	Reduction	
			1k-1.5k	1.5k-2k
1000	150	30%	0.4	0.2
1000	250	50%	1	0.2
1000	350	70%	4.2	0.6
1300	195	30%	1.8	0
1300	325	50%	5.6	1.2
1300	455	70%	12.6	1.4
1504	226	30%	4.2	0.8
1504	376	50%	8.4	1.6
1504	526	70%	27	4
1800	270	30%	9.4	3
1800	450	50%	22.6	7.2
1800	630	70%	42.6	8

For designs with acceleration lane, the trend remained the same. Number of conflicts for 1500 feet spacing were between the conflicts observed for 1000 feet and 2000 feet spacing. As shown in Table 4.4.2, the reduction in conflicts as spacing increased from 1000 feet to 1500 feet was more profound than the reduction observed when spacing was increased from 1500 feet to 2000 feet.

4.4.3. Designs without Acceleration Lane

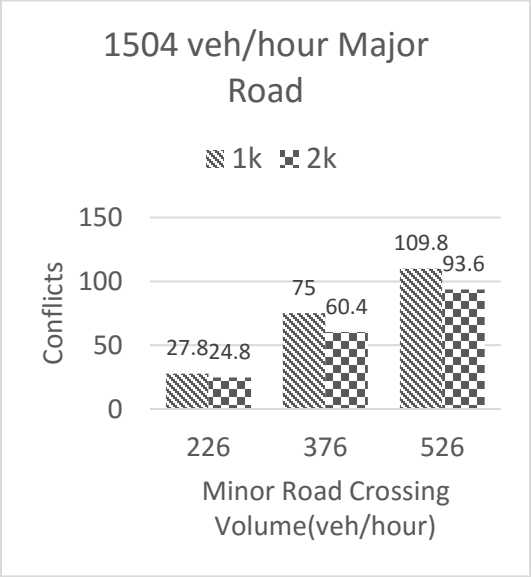
In general, the lack of acceleration lane increased the queuing on the minor road for vehicles waiting for a gap to merge into the major road. The numbers of conflicts for designs without the acceleration lane are shown in Figures 4.4.3 (a to d). Due to the lack of acceleration lanes, only two spacing combinations of 1000 feet and 2000 feet were evaluated. Overall, the number of conflicts decreased when the spacing increased from 1000 feet to 2000 feet. For example, in Figure 4.4.3 (b), conflicts dropped from 16.6 to 13.6 for 195 U-turn vehicles; 40.2 to 32.8 for 325 U-turn vehicles, and 70 to 61.6 for 455

U-turn vehicles.

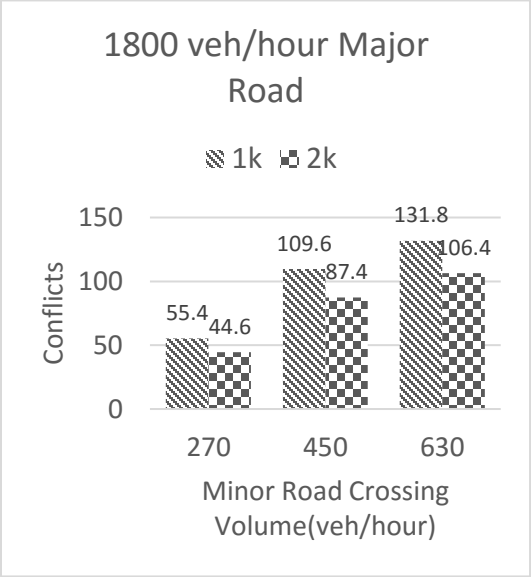


(a)

(b)



(c)



(d)

Figure 4.4.3 Conflict Counts for Design without Acceleration Lanes

The trend is consistent across all three spacing scenarios. The number of conflicts decreases when the spacing between J-turn increases. According to Figure 4.4.4, with 1000 to 1504 major road volume, 1500 feet spacing is enough when minor/major ratio is below 50%. Spacing needed to be increased to 2000 feet when. Minor/major ratio is over

50%. When the major road volume reaches 1800 veh/hour, increasing spacing from 1k feet to 1500 feet yield less efficiency comparing to that from 1500 feet to 2000 feet. Noted that with 1800 major road volume and 630 minor crossing volume, the network starts to become heavily congested, thus the higher reduction from 1000 feet to 1500 feet. It is still recommended to use at least 2k spacing with such high volume scenario.

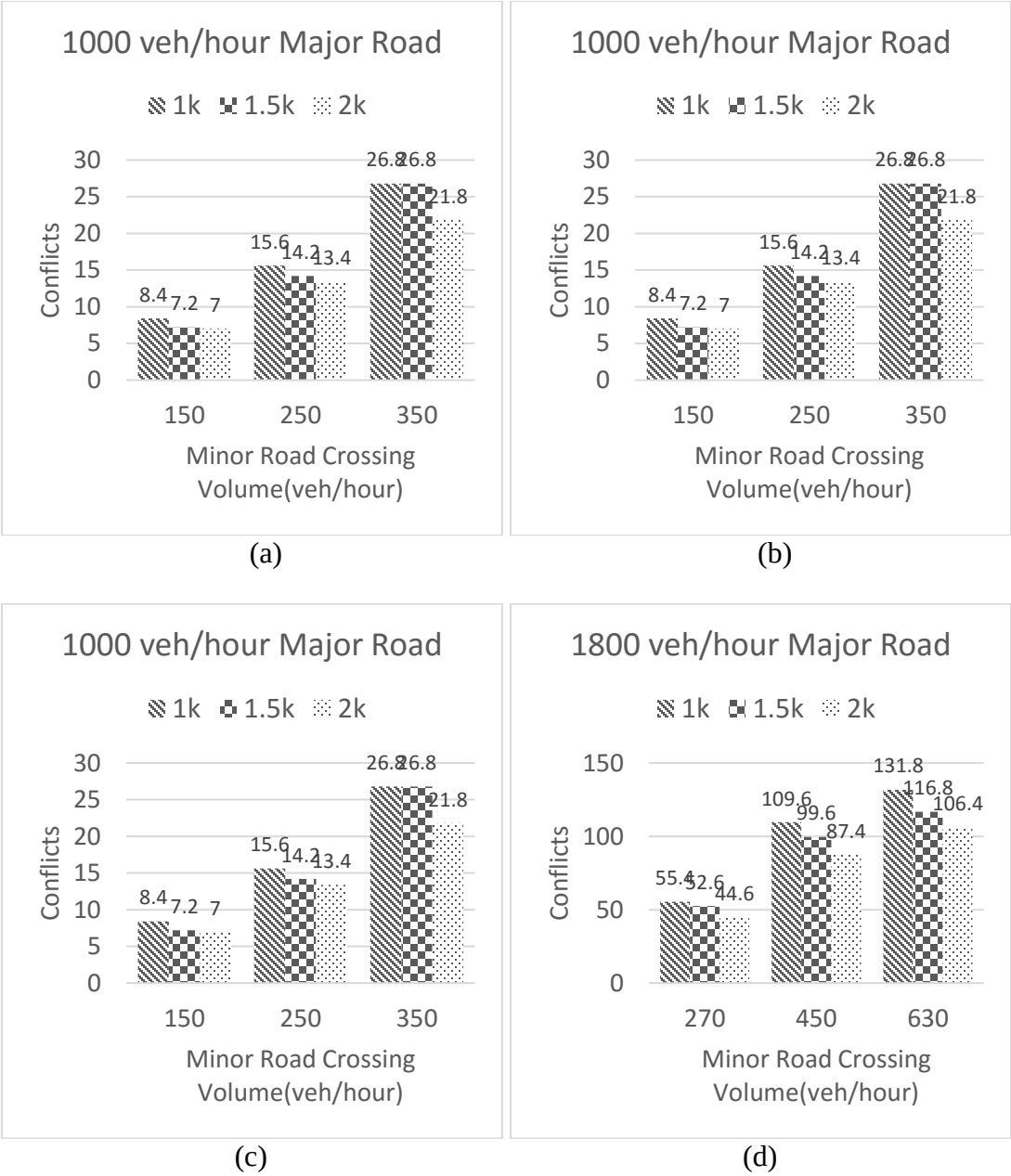


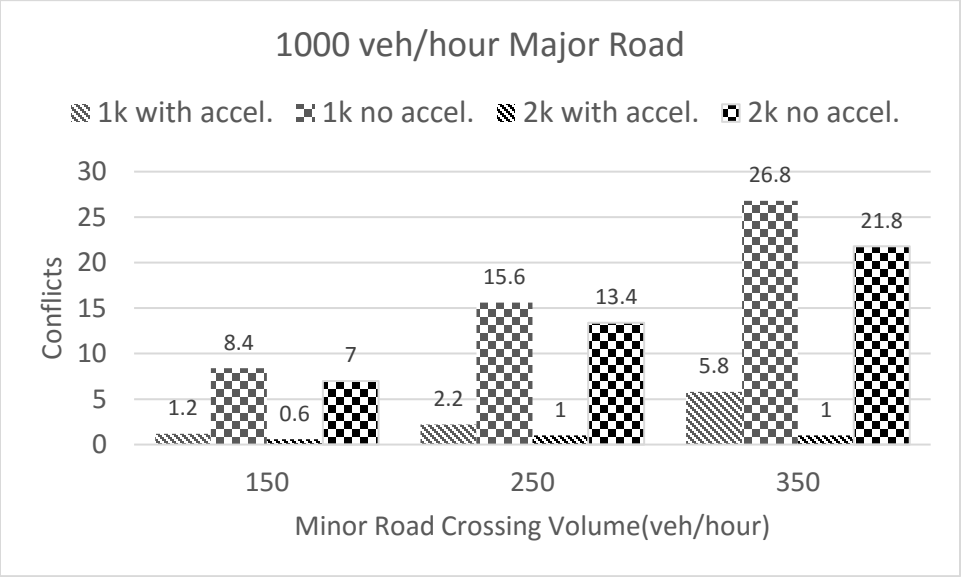
Figure 4.4.4 Conflicts reduction among 1k, 1.5k and 2k

Table 4.4.3 Reduction in Conflicts

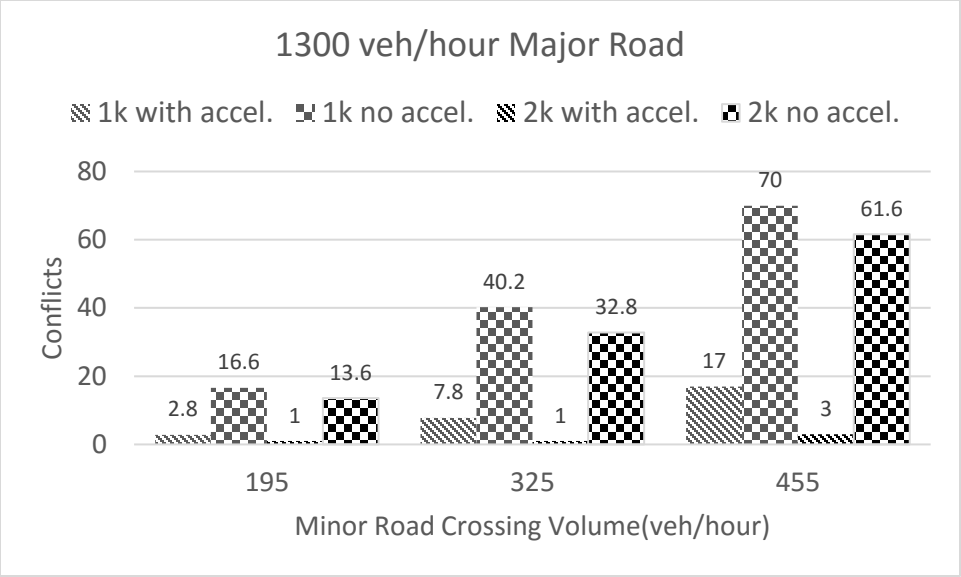
Major Total	Minor Crossing	Minor/Major ratio	Reduction	
			1k-1.5k	1.5k-2k
1000	150	30%	1.2	0.2
1000	250	50%	1.4	0.8
1000	350	70%	0	5
1300	195	30%	2	1
1300	325	50%	4.4	3
1300	455	70%	1.2	7.2
1504	226	30%	1.6	1.4
1504	376	50%	9.8	4.8
1504	526	70%	9.4	6.8
1800	270	30%	2.8	8
1800	450	50%	10	12.2
1800	630	70%	15	10.4

4.4.4. Comparison of with and without Acceleration Lane Designs

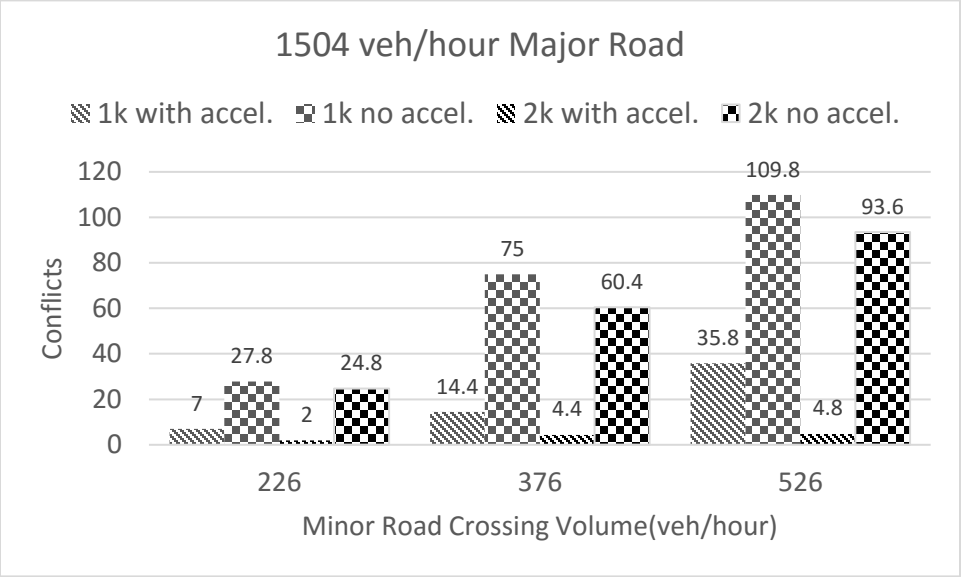
Figure 4.4.5 (a to d) compares the designs with and without acceleration lanes across all volume scenarios. In the figures, striped bars represent the designs with acceleration lanes while squared bars represent the designs without acceleration lanes. For each volume combination and the same U-turn spacing, the no-acceleration-lane design has more conflicts than the design with acceleration lane. Thus, acceleration lanes resulted in better safety for all spacing and volume combinations.



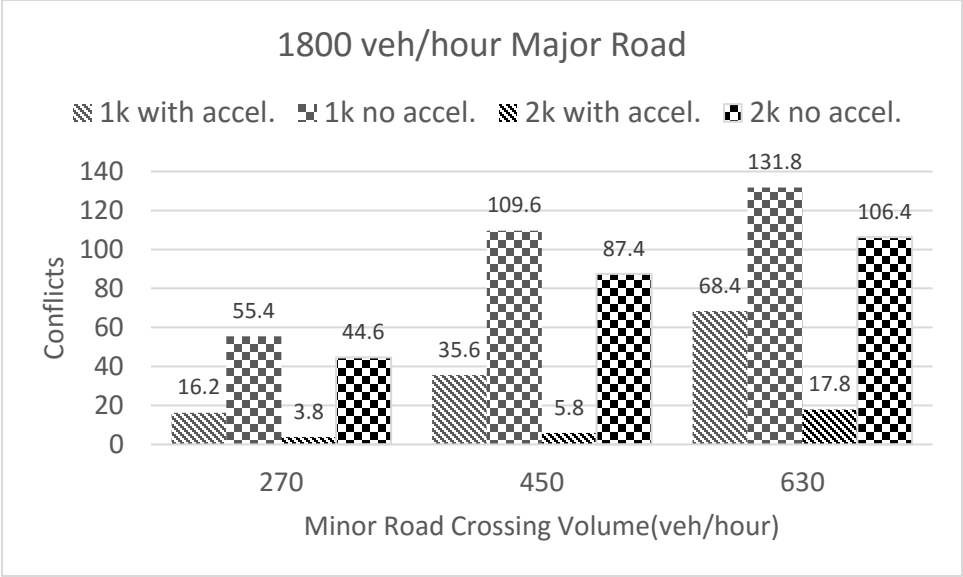
(a)


1300 veh/hour Major Road
1k with accel.
1k no accel.
2k with accel.
2k no accel.
Conflicts
80
60
40
20
0
2.8
16.6
1
13.6
7.8
40.2
1
32.8
17
70
3
61.6
195
325
455
Minor Road Crossing Volume(veh/hour)

(b)


1504 veh/hour Major Road
1k with accel.
1k no accel.
2k with accel.
2k no accel.
Conflicts
120
100
80
60
40
20
0
7
27.8
2
24.8
14.4
75
4.4
60.4
35.8
109.8
4.8
93.6
226
376
526
Minor Road Crossing Volume(veh/hour)

(c)



(d)

Figure 4.4.5 With Accel. Lane Design VS. No Accel. Lane Design

Table 4.4.4 calculates the difference of number of conflicts between the two J-turn designs. It indicates that with the same level of major road volume, the differential increases as the minor road volume increases. In 1000 feet spacing scenarios, the differential reaches peak as the volume combination reaches 1504/526 and 1800/450. This is because after the point of 1800/450, the roadway is heavily congested which result in slow speed merging and fewer conflicts. On the other hand, in the 2k feet spacing scenarios, the differential keeps rising and reaches 96.6 with 1800/630 combination but it does not seem to be reaching the peak.

Table 4.4.4 Difference in Number of Conflicts of the Two Designs

Major Total	Minor Crossing	Minor/Major ratio	Differential		
			1k	1.5k	2k
1000	150	30%	7.2	6.4	6.4
1000	250	50%	13.4	13	12.4
1000	350	70%	21	25.2	20.8
1300	195	30%	13.8	13.6	12.6
1300	325	50%	32.4	33.6	31.8
1300	455	70%	53	64.4	58.6
1504	226	30%	20.8	23.4	22.8
1504	376	50%	60.6	59.2	56
1504	526	70%	74	91.6	88.8
1800	270	30%	39.2	45.8	40.8
1800	450	50%	74	86.6	81.6
1800	630	70%	63.4	91	88.6

One goal of this project was to determine optimal spacing between the U-turn and the minor road for different volume and design combinations. Table 4.4.5 was compiled based on the results of simulation analysis. As previously concluded, acceleration lanes are safer for all volume combinations studied in this project. If acceleration lanes cannot be provided, the last column in Table 4.4.5 provides guidance on minimum spacing recommended for the different volume combinations. When acceleration lanes can be provided, the recommended spacing is lower for low volume combinations as shown in the fifth column in Table 4.4.5

Table 4.4.5 Recommended Minimum Spacing for Each Scenario

Major Total (veh/hr)	Minor Crossing (veh/hr)	Minor Crossing (veh/hr)	Total Minor/Major	With Acceleration Lane (in feet)	No Acceleration Lane (in feet)
1000	150	150	30%	1000-2000	1000
	250	250	50%	1000-2000	1000
	350	350	70%	1500	1000-2000
1300	195	195	30%	1000-2000	1000
	325	325	50%	1500	1000-2000
	455	455	70%	1500-3000	1000-2000
1504	226	226	30%	1500	1000
	376	376	50%	>1500	1000-2000
	526	526	70%	>1500	1000-2000
1800	270	270	30%	1500	1000
	450	450	50%	2000	2000
	630	630	70%	3000	2000

4.4.5. Discussion

Simulation models allow the flexibility to change design elements for testing. This dissertation showcased possible ways of testing J-turn designs. The scenarios can be expanded and other design elements can be explored. For example, the speed limits can be altered to reflect its possible effects on the number of conflicts. Commercial truck percentage can be a crucial design concern for some areas and altering the percentage in the simulation model can provide some insight on how the number of conflicts change. Moreover, there could be scenarios where acceleration lane length is limited due to lack of right of way, the simulation model helps recommend the minimum acceleration lane length for J-turns in that scenario. These evaluations can be efficiently done in a simulation model rather than constructing the design and collecting crash data in the future.

5. CONCLUSIONS

This dissertation presented surrogate speed-based measures to evaluate the effectiveness of new signage and alternative intersection designs. The two methods presented in the dissertation offer accuracy, reliability, flexibility and efficiency when comparing to existing video-based automated vehicle detection system and crash report analysis. These methods were demonstrated using two case studies – an alternative to MUTCD merge signage and J-turn intersection design. Alternative static merge signage in work zones has not been investigated in prior research. The study measured driver behavior characteristics including speeds and open lane occupancies. Field measurements were taken from the same work zone on different days using two configurations: one with the new test configuration and the other with the standard MUTCD configuration. Statistical tests were conducted to ensure that the comparisons between the experimental sign and the MUTCD sign were statistically valid. Based on an analysis of the measurements, the following conclusions were drawn:

1) Open lane occupancy was higher for the test sign in comparison to the MUTCD sign upstream of the merge sign. The occupancy values at different distances between the merge sign and the taper were similar for both the test and MUTCD signs, but the test sign encouraged up to 11% more cars to be in the open lane immediately upstream of the merge sign. In terms of safety, it is desirable for vehicles to occupy the open lane as far upstream of the taper as possible to avoid merging conflicts near the taper. Thus, the test sign proved to be a good alternative to the MUTCD sign.

2) Traffic monitoring results showed that passenger cars stayed in the closed lane longer, or closer to the taper, than did trucks. This was not unexpected given that

commercial trucks typically operate in the right lane unless for passing, truck drivers have better sight distances to spot signage earlier, and since commercial trips are work-related, drivers are more likely to adopt safer driving practices.

3) The merging behavior of truck drivers did not vary significantly with the type of merge sign deployed in the work zone. This is partly because more than 90% of truck traffic were already in the open lane upstream of the merge sign, both for the test sign and the MUTCD sign.

4) The analysis of speed characteristics did not reveal substantial differences between the two sign configurations. The mean speeds with the MUTCD configuration were 1.3 mph and 2 mph lower than the test configuration at the merge sign and taper locations, respectively. However, the effect size of 0.238 and 0.324 were very small, thus the speed results are not very useful despite being statistically significant.

There are several directions for future research in evaluating alternative signage in work zones. One is to survey motorists to obtain their perceptions towards the test sign configuration. Another is to evaluate the test sign configuration at additional work zone sites or using driving simulator experiments to further validate its performance. A third is to study the effect of sign performance as a function of the position of closed lane (right versus left lane). A fourth direction is to evaluate the test sign in a long-term construction work zone and to compare the performance with the short-term work zone evaluated in the current study. After getting accustomed to the new design, driver reaction to signs and other temporary traffic control in long-term work zones could be different from that of short-term work zones. Finally, the performance of the test sign in nighttime work zones can be investigated.

The second case study involved developing design guidance for J-turn. Specifically, the spacing of U-turn and the inclusion of acceleration and deceleration lanes was evaluated. A simulation analysis was conducted to further study the impact of different design variables on the safety of J-turns. A base simulation model was created and calibrated using field data collected in a previous MoDOT project on J-turns. The calibrated model was then used to study various combinations of major road and minor road volumes and design variables. The simulation analysis helped develop guidance on recommended spacing for various major road and minor road volume scenarios. For all the studied scenarios, the presence of acceleration lane resulted in significantly fewer conflicts. Thus, acceleration lanes are recommended for all J-turn designs, including lower volume sites. Second, while spacing between 1000 feet and 2000 feet was found to be sufficient for low volume combinations, a spacing of at least 1500 feet and 2000 feet are recommended for medium and high volumes, respectively. One topic for future research is to obtain driver feedback and response to the U-turn spacing and presence of acceleration and deceleration lanes. A driving simulator study is appropriate to obtain driver feedback on new intersection designs. It is also interesting to test lower speed limits and various commercial truck percentages. These variables can potentially affect the total number of conflicts.

Future research could examine the suitability of connected vehicle data for calibrating traffic simulation models. Microscopic traffic variables such as headways can be extracted from data obtained from connected vehicle pilot projects in the U.S. Similar headway distribution can also be obtained through driving simulators which records the

following headways directly. The obtained headway distribution could provide guidance for calibrating driving behavior parameters.

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VITA

Zhongyuan Zhu got his B.S. degree from Tongji University, China, in 2010. He received his M.S. degree in civil engineering from the University of Missouri - Columbia and continued to pursue a Ph.D. degree. Zhu's research topic includes work zone safety evaluation, ramp metering, alternative intersection design, microscopic simulation modeling. He is interested in applying new technologies in transportation researches and develop new methods for practitioners to apply in projects.