COMPARITIVE STUDY OF TYPE 2 MEDIAN CROSSOVERS AND MEDIAN U-TURNS

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ABSTRACT

Many states use rural expressway median crossovers to manage the direct left turn movements between the expressways and cross roads. These crossovers provide for separation between the two opposing traffic lanes and allow the movement for the turning and crossing traffic. As the volume increases on the major road, the traffic from the crossroad faces difficulty in finding a gap to enter the other side of the driveway. As a result these drivers will experience long travel and delay times. Sometimes the storage length provided for the expressway vehicles to make a left turn at the median crossover may get occupied completely. This may lead to a dangerous situation where the vehicles will extend back onto the expressway, obstructing the through movement traffic. The research describes the comparative study of type 2 median crossovers and Median U-Turns and estimates where the rural expressway type 2 median crossover fails in its operation. The Highway Capacity Software and VISSIM, the simulation tool, were used to obtain the performance characteristics of the median crossover based on operational parameters including travel time, delay time and Level of service. A design tool was developed that helps to make a decision on the distance required to be provided between the cross road and the Median U-Turn. This design tool is based on the volume combinations of the crossroad and the major road. Various combinations of traffic volumes have been assumed based on which, the extent to which the conventional design

option, the Median U-Turn with unsignalized condition and signalized condition will work were determined from the performance measures obtained in VISSIM. Cost estimates that include the construction costs and user costs have been made for all the three design options.

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CHAPTER 1

INTRODUCTION

Expressway median crossovers are used by major left turn, minor left turn and minor through vehicles. The median serves as a storage space for the left turning and the through movement traffic from the minor road. As described in the plans provided by Missouri Department of Transportation (MoDOT) [1], a typical Type 2 median has a median width of 60 feet. The cross roads are controlled by two-way stop signs and the median area is controlled by yield signs. A simple sketch of type 2 median crossover is shown in Figure 1-1. The detailed sketch of this crossover is shown in Appendix I.

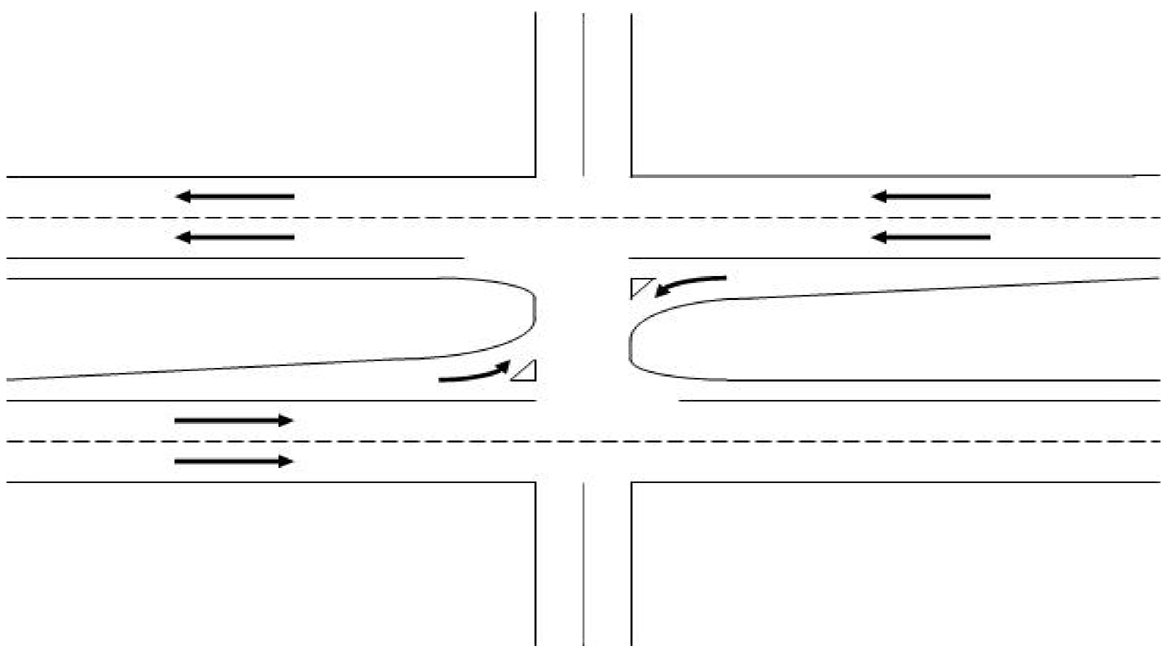


FIGURE 1-1: Typical Type 2 median crossover

There are some problems that affect the safety and operation of the Type 2 median crossover. When the volumes increase through the crossover area, these vehicles can be stored in the median and can block each other, impeding the visibility to the oncoming vehicles. As the volume increases the vehicles stored in the median can spill back and obstruct the through traffic on the major road which will produce a dangerous situation.

The objective of this thesis is to estimate the performance of Median crossovers and Median U-Turn for higher volumes, when the Type 2 median crossover may not function well. The performance measures used were travel time, delay time and queue length. The Type 2 median crossover was used as base alternative, Unsignalized Median U-Turn was considered as the first alternative and Signalized Median U-Turn was considered as the second alternative. Currently, there is no proper procedure to decide the spacing between Median U-Turn and the crossroad. An attempt has been made to prepare a design tool using Highway Capacity Software (HCS) to decide the Median U-Turn distance from the crossroad, based on various volume combinations and Level of Service (LOS).

Chapter two summarizes some of the previous work made on the characteristics of Type 2 median crossovers and Median U-Turns. Chapter three describes the application of HCS methodology to find the LOS and control delay for the present study. It provides the delay results for all the three design options. Chapter four describes the VISSIM model and also provides the advantages and additional features available in VISSIM that are not available in HCS. It also explains various elements used in the simulation software to model the Type 2 median crossover and its two alternatives. The performance measures used to compare these three design options are discussed. Chapter five describes the results and its analysis based on the performance measures and explains the limiting volumes for all the three design options to function well. Chapter six describes the cost estimates for all the three design options under two different scenarios. The construction costs and the user costs are taken in to consideration for these estimates. Chapter seven summarizes the conclusions drawn from the discussions in the earlier chapters.

Appendix I provide a detailed sketch of typical Type 2 median crossover. Appendix II provides the sample HCS output files for control delay and LOS.

CHAPTER 2

LITERATURE REVIEW

To understand the kind of work to be performed in this thesis, a literature search was conducted to understand the previous studies that were made on median crossovers. This search gave some information about the operational characteristics of Type 2 median crossovers and various alternatives that were proposed to overcome the problems. This search provided an idea of further studies that can be made to improve the performance of these median crossovers. Each pertinent search result that contributed to the areas of emphasis is annotated below.

Huaguo Zhou, Jian John Lu, Xiao Kuan Yang, Sunanda Dissanayake and Kristine M. Williams [2] studied the operational effects of U-turns as alternatives to direct left turns from driveways. In this study they considered eight sites in the Tampa and Clearwater areas of Florida (urban and sub urban areas) to compare the operational effects of the direct left turns (DLT) and right turn plus U-turn (RTUT). Field data collection including delay, travel time, traffic volume, speed, geometric data and traffic control were taken for three hundred hours and an operational effects database was developed to perform statistical analyses. Delay and travel time models of DLT and RTUT were developed as a function of major and minor traffic flow rates. A ratio model was developed to estimate how many drivers would prefer to make a RTUT rather than DLT under certain traffic flow conditions. These operational models were used to measure system performance of a full median opening versus a directional median opening.

Kim, Edara and Bared [3] have performed some simulation studies for three different cases of superstreet which is similar to Median U-Turn. In the first case one left turn lane and two through lanes on the major road was considered, the second case considered one left turn lane and three through lanes on the major road and the third case considered two left turn lanes and three through lanes on the major road. For each case microscopic traffic simulations were conducted for various traffic volumes and their performance was compared to the conventional design option. The first case was simulated for high, medium and low traffic scenarios and the remaining two cases were studied for high volumes as their application was mainly intended for sites operating under heavy traffic conditions. A 400 foot offset was assumed in the superstreet design. The traffic signal required only two phase instead of four or more phases. Phase one allowed the major road through movement and phase two allowed the major road left turn movement and the minor road through and left movements The Simulation Surrogate Safety Assessment tool was used to perform some safety evaluations. These simulation results showed that the superstreet design with one U-Turn lane performed better than the conventional design option. A smaller increase in the throughput was observed for two U-Turn lanes. The results from the surrogate safety assessment tool showed that one U-turn lane is safer compared to the conventional model. The superstreet design with two U-turn lanes was not safer than the conventional design.

Zhou, Hsu, Lu and Wright [4] developed a working model to find the optimum location of U-turn median openings on roadways. It included some traffic and geometric factors such as upstream and downstream signal timing, through traffic speed, distance between the side street and its upstream signalized intersection. A regression model was developed to predict the average weaving speed for various weaving lengths. They defined the optimum weaving length as the distance between the driveway and the U-turn median opening that minimizes the average waiting delay for U-turn movements at the U-turn median opening (i.e., the time when a RTUT vehicle arrives at a U-turn median opening is equal to the time when the last vehicle in an opposite through traffic platoon passes the U-turn median opening). Based on this an equation was developed to find the optimum weaving length.

$$L = \frac{\left[-\left(1 - 0.082\Delta t + \frac{21.5}{v}\right) + \sqrt{\left(1 - 0.082\Delta t + \frac{21.5}{v}\right)^2 + 7.05\,\frac{\Delta t}{v}}\right]}{\frac{0.164}{v}} \quad \ldots\ldots\ldots (2.1)$$

L = Optimum weaving length

V= Posted speed limit on the major roadway (km/hr)

$\Delta t =$ Function of offset of upstream and downstream signal timing, whole section length, distance between driveway and upstream signalized intersection and posted speed limit

$$\Delta t = \Delta T \times 0.278 + \frac{(l - 2L_1)}{v}$$

ΔT = Offset of upstream and downstream signal timing(s)

L_1 = Distance between driveway and the upstream signalized intersection (m)

To verify this model a case study was performed at the intersection of Fowler Avenue and 46th Street in Tampa, Florida which has a directional median design for a side street (46th street) and provides an exclusive U-turn median opening with 775 feet weaving length, which was very close to the estimated value of 725 feet. Video cameras were installed at the median opening and the waiting delay for these U-turn movements was recorded. Statistical analysis showed that about 60% of the U-turns had a zero waiting delay and about 80% of the U-turns had less than ten seconds delay. This case study demonstrates that the Median U-Turn design with an optimum weaving length obtained from the model is having either zero or small delay.

In the final report [5] on the information and options for median crossovers on four lane divided highways by Virkler, a brief summary of various alternative treatments available for median crossovers have been provided by reviewing the state DOT design manuals and drawings that are available electronically and through telephone calls to knowledgeable professionals. In the North Carolina option, the through and the left turn movements from the cross road are eliminated. Instead the cross road vehicles must make a right turn on the expressway and then take a U-turn in the median to continue to their destination. Maryland has a few of these and Michigan has a similar option, but in Michigan the through movements, controlled by a signal, are allowed from the crossroad prohibiting the left turns from the crossroad. Dewayne Sikes of the North Carolina DOT Highway Design Division commented that this option can be either signalized or unsignalized and that there is a location where a signalized intersection was changed from a conventional design with an eight phase controller to two two-phase controllers, with a significant increase in the capacity. In the Michigan option, minor road through movements are allowed. The Median U-Turn crossover eliminates left turns at the intersections. For Median U-Turn crossovers located on the major road, drivers turn left off the major road by passing through the intersection, making a U-turn at the crossover, and turning right at the crossroad. Drivers wishing to turn left onto the major road from the cross street turn right onto the major road and make a U-turn at the crossover. In Michigan, the Median U-Turns have been in use for more than 30 years and the signing has evolved to become more user friendly. Median U-Turns may be appropriate at intersections with high major street through movements, low to medium left turns from the major street, low to medium left turns from the minor street and any amount of minor street through volumes. Locations with high left turning volumes may not be suitable because the out of direction travel incurred and the potential for queue spill back at the Median U-Turn location could outweigh the benefits associated with removing left turns from the main intersection. The Michigan Department of Transportation advises the optimum location for the crossover from the main intersection to be 560 to 760 feet. Generally 660 feet is used in urban areas and 1320 feet is used in rural areas. Signing is needed to alert drivers of the presence of Median U-Turns and the restriction of left turn movement at the signalized intersection. Installing traffic signals at Median U-Turn locations requires additional storage for the U-turn moving vehicles and requires coordination with the adjacent signalized intersections. According to NCHRP 420, the collision rate along road sections having directional median openings (facilitating U-turns and left turns) versus road sections having full median openings (facilitating all movements) was 49 to 52 percent less for signalized corridors having more than one traffic signal per mile.

A study on a Michigan corridor [5] used simulation to compare Median U-Turn crossovers with two-way left-turn lanes (TWLTL). The study showed that during peak hours, the corridor with Median U-Turn crossovers had a lower travel time by 17 percent and a 25 percent higher average speed than the same corridor with a TWLTL. However vehicles made more stops on the arterial with Median U-Turn crossovers. In the non peak hours the Median U-Turn had the same efficiency as the TWLTL. Simulation studies using a range of intersection configurations (number of through lanes on the major and the minor street) and volumes from intersections in Virginia and North Carolina suggest a reduction in the overall travel time for all movements through the intersection when compared to a conventional intersection.

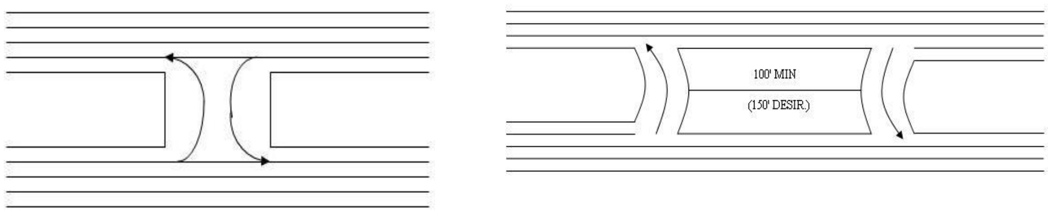
Hummer [5] described the various unconventional left turn alternatives for urban and suburban arterials that included Median U-Turns, Bowtie and superstreet crossovers. The variations of these alternatives to that of the basic model, history of each alternative, its advantages and disadvantages, driver confusion factor and when to consider these alternatives have been described.

A technical brief on synthesis of the Median U-Turn intersection treatment, safety and operational benefits presented some information on the design guidelines including the location and the design of the median crossovers on the major road [6]. There are 425 miles of boulevards with over 700 directional crossovers on the Michigan state highway system. Partial implementations or designs with similar concepts have appeared in Florida, Maryland, New Mexico and New Orleans. Hummer and Reid [7] recommended that agencies consider the Median U-Turn alternative on high design arterials where relatively high through volumes conflict with moderate or low left turn volumes, regardless of the cross street through volumes. The 2004 AASHTO Green Book [8] recommends a distance of 400 to 600 feet for the minimum spacing between the median crossover and the Median U-Turn intersection treatment (MUTIT). The Michigan Department of Transportation (MDOT) recommends a distance of 660feet (+/- 100 feet) for the median crossover from the MUTIT intersection. The Access Management Manual recommends an access spacing of 660 feet on minor arterials and 1320 feet on principal arterials between consecutive directional median openings on divided highways.

Taylor [9] performed studies to find the effects of replacing the bidirectional crossovers with directional crossovers. An eight roadway section in Michigan was considered between 1991 and 1997 and crash frequency was investigated. He observed that the average reduction in the total crash frequencies was 31 percent and average reduction in the injury crash frequencies was 32 percent. Rear-end and angle crashes have experienced the largest decrease in the crash frequency with an average 37 percent reduction in rear end crashes.

Sceuer and Kunde [10] took three years of before crash data and two years of after crash data at Grand River Avenue in Wayne County, Michigan, an eight lane boulevard with many driveways and minor cross roads. The project achieved a total crash reduction of 24 percent with a great reduction in the head on and right angle crashes.

Castronovo [11] studied the safety performance of directional and bidirectional crossovers (Figure 2-1) and found that the both of these options had the same crash rates for those sections without traffic signal. For, signalized intersections, as the density increased the directional median crossover showed a fifty percent reduction in the crash rate compared to the bidirectional median crossover.



(a) **(b)**

FIGURE 2-1: (a) Bidirectional crossover (b) Directional crossovers

In NCHRP report 524 [12], researchers studies the safety aspects of U-turns at un-signalized median openings. They analyzed the collision data at the U-turns and found that collisions occur very infrequently at the un-signalized median openings. In urban arterial corridors, un-signalized median openings had an average of 0.41 U-turn and left turn accidents per median opening per year. Un-signalized median openings in rural arterial corridors had an average of 0.20 U-turn movements and left turn accidents per median opening per year. From these results the researchers found that there is no indication that U-turns at un-signalized median openings are a general safety concern.

Dorothy et al. [13] studied the operational characteristics of TWLTL with that of the MUTIT. They used TRAF-NETSIM and performed the simulation for 3600 seconds. A cycle length of 80 seconds with 60/40 split was assumed for entering volumes on the major road and cross street. When the turning percentages were low the model assumed stop signs and when the volumes increased, the model assumed signal control. Signalized MUTIT had lower travel times than the conventional model. For lower turning percentages the directional median crossovers with stop control and directional median crossovers with signal control had approximately the same travel time.

In the next five years, Missouri Department of Transportation [14] plans to construct rural expressways with four-lane divided highways that extend up to 400 miles which will intersect a number of cross roads. For this a technical bulletin was developed to provide guidance for different levels of median opening application. A number of roadway design experts and traffic engineering teams from inside and outside MoDOT developed additional median opening options for designers to consider. These options included five types i.e. Type1 (No left turn lanes), Type 2 (Turn lanes), Type 3 (Offset lefts), Type 4 (Median U-Turn), Type 5 (Partial grade separated intersection). If these options did not meet the requirements of the traffic conditions, then an Interchange was taken as a further option. An overview of each option along with its advantages, disadvantages, model layout and additional items to be considered was described.

CHAPTER 3

APPLICATION OF HIGHWAY CAPACITY SOFTWARE

The Highway Capacity Manual methodology provides a consistent system of techniques for the evaluation of quality of service. In this study two analytical techniques were used. One is the Highway Capacity Software (which implements the Highway Capacity Manual) and the other one is VISSIM software. The Highway Capacity Manual [15] provides methods for analyzing the capacity and Level of Service (LOS) for various types of transportation facilities. LOS is a quality measure describing operational conditions of the traffic stream in terms of traffic interruptions like traffic signals, travel time, comfort and convenience. In this study HCS is used to find the LOS of the Median U-Turn by considering it as three separate sections. The first section is the location where the traffic enters the major street from the minor street which is considered as unsignalized T-legged intersection. The second section is the weaving segment where the vehicle that entered from the cross road on to the major road tries to weave on to the inner lane to take a U-turn. The third section is the location where the vehicles that entered the Median U-turn try to get onto the opposite major road which is also considered as a unsignalized T- legged intersection. A brief description of the Highway Capacity Manual (HCM) methodology for unsignalized intersections, signalized intersections and freeway weaving segments is given below.

3.1 HCM methodology for Unsignalized intersections

This section explains the methodology used to analyze the control delay and LOS for two way stop controlled intersections or unsignalized intersections. Some important factors are described in the following sections.

3.1.1 Input data requirements

The input data includes a detailed description of geometrics, control and volume. Geometric factors include the number of lanes, approach grade, median storage details. Volumes must be specified by the movement. For the analysis to reflect conditions during the peak 15 minutes, the analyst must divide the full hour volumes by the peak hour factor (PHF). And if the analyst has the peak 15 minute flow rates the PHF can be set to a value of one. The capacity model assumes the headway on the major street to be random. The presence of signals upstream from the intersection within a distance of 0.25 miles will affect the capacity of the intersection.

3.1.2 Two-stage gap acceptance

For Type 2 median crossover a two-stage gap acceptance concept is used (i.e. the intersection is considered to consist two parts). There is storage space between the two parts of the intersections to accommodate the left turning vehicles moving from the major and the minor streets and the through movement vehicles from the minor street. The conflict flow rates are defined in chapter 17 of HCM 2000. The capacity is calculated using the appropriate values of critical gap and follow-up time for the two-stage gap acceptance process in chapter 17 of HCM 2000. The capacity for the subject movement considering the two-stage gap acceptance process is computed as follows. Figure 3-1 shows the two-stage gap acceptance .An adjustment factor and an intermediate value y is computed as:

$$a = 1 - 0.32\, e^{-1.3\sqrt{m}} \qquad for\ \ m > 0 \qquad \ldots\ldots\ldots (3.1)$$

$$y = \frac{c_I - c_{m,x}}{c_{II} - V_L - c_{m,x}} \qquad \ldots\ldots\ldots (3.2)$$

Where, $m =$ Number of storage spaces in the median

$c_I =$ Movement capacity for the stage1 process

$c_{II} =$ Movement capacity for stage 2 process

$V_L =$ Major left turn flow rate, either V1 or V4 and

$c_{m,x} =$ Capacity if subject movement considering the total conflicting flow rate for both stages of a two-stage gap acceptance process

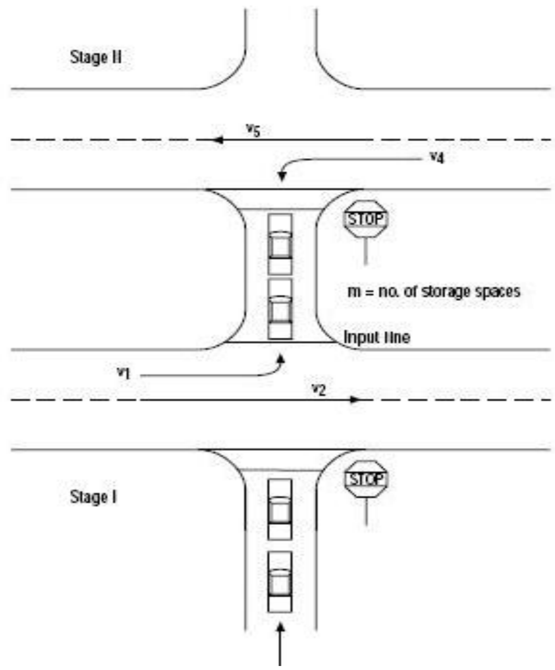


FIGURE 3-1: Two stage gap acceptance

The total capacity c_T, of the intersection for the subject movement considering the two-stage gap acceptance process is computed as (from chapter 17 in HCM 2000)

For $y \neq 1$,

$$c_T = \frac{a}{y^{m+1}}\left[y(y^m - 1)(c_{II} - V_L) + (y - 1)c_{m,x}\right] \quad \ldots\ldots\ldots (3.3)$$

For y=1,

$$c_T = \frac{a}{m+1}\left[m(c_{II} - V_L) + c_{m,x}\right] \quad \ldots\ldots\ldots (3.4)$$

3.1.3 Estimation of Queue length

Queue length is an important factor to be considered at unsignalized intersections. The probability distribution of queue lengths for any minor movement at an unsignalized intersection is a function of the capacity of the movement and the volume of the traffic being served during the analysis period. The product of the average delay per vehicle and the flow rate for the movement of interest is taken as the mean queue length. The expected total delay (vehicle-hours/hour) equals the expected number of vehicles in the time period (i.e., the total hourly delay and the average queue are numerically identical). Figure 3-2, taken from HCM (Exhibit 17-19), can be used to estimate the 95th percentile queue length for any minor movement at an un-signalized intersection during the peak 15 minute period.

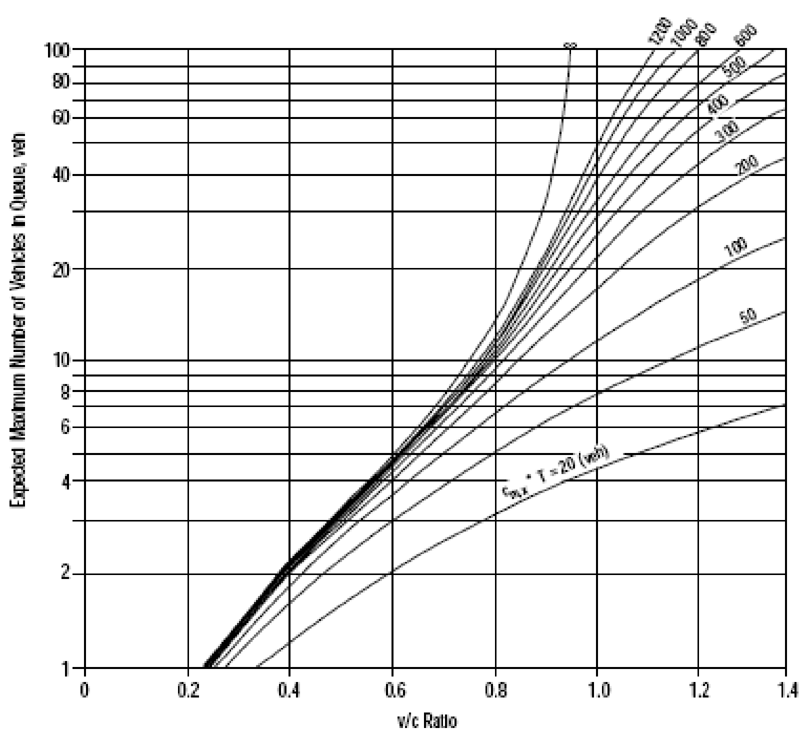


FIGURE 3-2: 95th-percentile queue length

The following equation is used to calculate the 95th percentile queue (from chapter 17 in HCM 2000):

$$Q_{95} = 900T\left[\frac{V_X}{c_{m,X}} - 1 + \sqrt{\left(\frac{V_X}{C_{m,X}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,X}}\right)\left(\frac{V_X}{c_{m,X}}\right)}{150T}}\right]\left(\frac{c_{m,X}}{3600}\right) \qquad \text{… … … (3.5)}$$

Where,

$Q_{95} =$ 95th-percentile queue (vehicles)

$V_X =$ Flow-rate for movement x (veh/hr)

$c_{m,X} =$ Capacity of movement x (veh/hr), and

$T =$ Analysis time period (h) (T = 0.25 for a 15-min period)

3.1.4 Control delay

Control delay includes the delay due to initial deceleration, queue move-up time, stopped delay and final acceleration delay. With respect to field measurements the control delay is defined as the total elapsed time from the time the vehicle stops at the end of the queue to the time the vehicle departs from the stop line. Average control delay for any particular minor movement is a function of the capacity of the approach and the degree of saturation. The following equation is used to find the control delay (from chapter 17 in HCM 2000).

$$d = \frac{3600}{c_{m,X}} + 900T \left[\frac{V_X}{c_{m,X}} - 1 + \sqrt{\left(\frac{V_X}{C_{m,X}} - 1 \right)^2 + \frac{\left(\frac{3600}{c_{m,X}} \right) \left(\frac{V_X}{c_{m,X}} \right)}{450T}} \right] + 5 \quad \ldots \ldots \ldots (3.6)$$

Where,

$d =$ Control delay (s/veh)

$V_X =$ Flow rate for movement x (veh/hr),

$c_{m,X} =$ Capacity of movement x (veh/h) and

$T =$ Analysis time period (h) (T = 0.25 for a 15-min period)

A constant value of 5 sec/vehicle is included in the equation to account for the deceleration of the vehicles from the free flow speed to the speed of the vehicle in the queue and to accelerate from the stop line to free-flow speed.

3.1.5 Determination of Level of Service

Level of Service is determined from the computed control delay and is defined for each movement. Table 3-1 (from chapter 17 of HCM 2000) is used to determine the LOS for various values of the control delay.

TABLE 3-1: Level of service criteria for TWSC intersections

Level of Service	Average control delay (s/veh)
A	0-10
B	>10-15
C	>15-25
D	>25-35
E	>35-50
F	>50

Some important factors that are involved in the determination LOS at unsignalized intersections were discussed above. Other factors include stream priority, conflicting traffic, gap acceptance, follow up time, potential capacity and impedance effects. More detailed information about these factors can be obtained from Highway Capacity Manual 2000.

3.2 HCM methodology for signalized intersections

This section explains the methodology involved to analyze the control delay and LOS at signalized intersections. Some important factors are described in the following sections.

3.2.1 Input data requirements

The input data requirements consist of geometric, traffic and signalization conditions. Geometric conditions include area type, number of lanes (N), average lane width (W ft), Grade (G%), parking, length of storage bay, existence of exclusive left turn or right turn lanes. Traffic conditions include demand volume by movement, base saturation flow rate, PHF, arrival type, proportion of vehicles arriving on green, approach speed and approach pedestrian flow rate. Signalization conditions include cycle length, green time, yellow plus all-red and clearance intervals actuated or pre-timed operation, phase plans and analysis period.

3.2.2 Determination of capacity and v/c ratio

Capacity at any signalized intersection is based on the concept of the saturation flow rate. The capacity of a given lane group is given as (from chapter 16 of HCM 2000)

$$c_i = S_i \frac{g_i}{C} \qquad \text{… … … (3.7)}$$

Where,

$c_i =$ Capacity of the lane group I (veh/hr)

$S_i =$ Saturation flow rate for lane group I (veh/hr) and

$\frac{g_i}{C} =$ Effective green ratio for the lane group i

The ratio of volume to capacity is given the symbol X and is referred to as degree of saturation. For a given lane group i, X_i is computed as (from chapter 16 of HCM 2000)

$$X_i = \left(\frac{V}{c}\right)_i = \frac{V_i}{S_{i\left(\frac{g_i}{C}\right)}} = \frac{V_{i\,C}}{S_i g_i} \qquad \text{… … … (3.8)}$$

Where,

$X_i = \left(\frac{V}{c}\right)_i$ Ratio of the lane group

$V_i =$ Actual or projected demand flow rate for lane group I (veh/hr)

$S_i =$ Saturation flow rate for lane group i (veh/hr)

$g_i =$ Effective green time for lane group i(s), and

$C =$ Cycle length

When the flow rate equals capacity, X_i will take a value of 1 and it will take a value of zero when the flow rate is equal to zero. The capacity of the entire intersection is not defined here.

3.2.3 Delay estimation

The values for the delays represent the average control delay experienced by all vehicles that arrive in the analysis period. Control delay includes the delays caused at the intersections, the slowing moving vehicles as they enter the queue and the slowdown of the vehicles due to upstream of the intersection. The average control delay for a vehicle for a given lane group is given by the following equation (from chapter 16 of HCM 2000).

$$d = d_1(PF) + d_2 + d_3 \qquad \text{… … … (3.9)}$$

Where,

d = Control delay per vehicle (s/veh)

d_1 = Uniform control delay assuming uniform arrivals (s/veh)

PF = Uniform delay progression factor, which accounts for effects of signal progression

d_2 = Incremental delay to account for effect of random arrivals and oversaturation queues, adjusted for duration of analysis period and type of signal control; this delay component assumes that there is no initial queue for lane group at start of analysis period (s/veh) and

d_3 = Initial queue delay, which accounts for delay to all vehicles in analysis period due to initial queue at start of the analysis period (s/veh)

3.2.4 Determination of Level of service

Intersection LOS is decided based on the average control delay per vehicle. When the delays have been estimated for each lane group and aggregated for each approach and the intersection as a whole Table 3-2 (from chapter 16 of HCM 2000) gives the appropriate LOS.

TABLE 3-2: Level of service criteria for signalized intersections

LOS	Control delay per vehicle (s/veh)
A	≤ 10
B	> 10-20
C	>20-35
D	>35-55
E	>55-80
F	>80

The above mentioned are some important factors involved in the computation of the LOS for signalized intersection. There are some other factors like lane grouping and determination of saturation flow rate considered for the analysis. More information on these factors can be obtained from Highway Capacity Manual 2000.

3.3 HCM methodology for freeway weaving segments

This section describes the methodology involved in determining the LOS based on the density within the weaving segment. The Highway Capacity Manual presents five distinct components to explain this methodology. Firstly it predicts the space mean speed of the weaving and the non weaving vehicles in the weaving segment. Secondly, it describes the proportional use of lanes by weaving and non-weaving vehicles, to determine if the operations are constrained or unconstrained. Thirdly, an algorithm converts the predicted speeds to an average density within the weaving segment. In the next step LOS is determined based on the density within the weaving segment. Finally capacity of the weaving segment is determined.

3.3.1 Input data requirements

Input data include geometric data, weaving and non weaving volumes, free flow speed of the weaving segment before and after the weaving segment.

3.3.2 Determination of flow rates

All the models and equations are based on peak 15-min flow rates in equivalent passenger cars per hour. Hourly volumes are converted into peak 15-min flow rates based on the following equation (from chapter 24 of HCM 2000).

$$v = \frac{V}{PHF * f_{HV} * f_p} \qquad \dots\dots\dots (3.10)$$

Where,

$v =$ Peak 15-min flow rate in an hour (pc/hr)

$V =$ Hourly volume (veh/h)

$f_{HV} =$ Heavy-vehicle adjustment factor and

$f_p =$ Driver population factor

3.3.3 Determination of weaving segment speed

The average space mean speed of all vehicles in the segment is computed using the following equation (from chapter 24 of HCM 2000).

$$S = \frac{V}{\left(\frac{V_w}{S_w}\right) + \left(\frac{V_{nw}}{S_{nw}}\right)} \qquad \ldots\ldots\ldots(3.11)$$

Where,

$S =$ Space mean speed of all vehicles in the weaving segment (mi/h)

S_w = Space mean speed of the weaving vehicles in the weaving segment (mi/h)

S_{nw} = Space mean speed of the non weaving vehicles in the weaving segment (m/hr)

$V =$ Total flow rate in the weaving segment (pc/h)

$V_w =$ Weaving flow rate in the weaving segment (pc/h) and

$V_{nw} =$ Weaving flow rate of the non weaving segment (pc/h)

3.3.4 Determination of density

The average speeds of all the vehicles can be used to determine the density for all the vehicles in the weaving segment using the following equation (from chapter 24 of HCM 2000).

$$D = \frac{\left(\frac{V}{N}\right)}{S} \qquad \ldots\ldots\ldots(3.12)$$

Where D is the average density for all vehicles in the weaving segment (pc/mi/lane)

3.3.5 Determination of the Level of service

LOS of the weaving segment is determined by comparing the computed density with Table 3-3 (from chapter 24 of HCM 2000).

TABLE 3-3: Level of service criteria for freeway weaving segments

LOS	Density (pc/mi/ln)	
	Freeway weaving segment	Multilane and collector distributor weaving segment
A	≤ 10	≤ 12
B	>10-20	>12-24
C	>20-28	>24-32
D	>28-35	>32-36
E	>35-43	>36-40
F	>43	>40

3.4 Development of Design tool and Control delays from HCS

The Highway capacity software (HCS) [16] works based on the above mentioned methodologies for unsignalized, signalized and weaving segments. In this study HCS is used to find the LOS of the Median U-Turn by considering it as three separate sections.

The first section is the location where the traffic enters the major street from the minor street which is considered as unsignalized T-legged intersection. The second section is the weaving segment where the vehicle that entered from the crossroad on to the major road tries to weave on to the inner lane to take a U-turn. The third section is the location where the vehicles that entered the Median U-Turn try to get onto the opposite major road which is also considered as an unsignalized T- legged intersection. Since the volumes were assumed in this study, the cross road volumes that were assumed ranged from 0 to 400 with an interval of 50 in between. For different volumes combinations of the major road and the minor road the maximum volume where the LOS changed from B to C, C to D, D to E and E to F were found out separately for weaving segment and the two T-legged intersections (crossroad to major road and U-turn to major road on to the opposite flow) that were defined above. One set of data assumes equal volumes approaching from eastbound and westbound on the major road and the other set of data assumes unequal volumes approaching from eastbound and westbound on the major road. The same procedure is repeated for different weaving lengths (also considered as the distance between the crossroad and the Median U-Turn) of 300 feet, 600 feet, 900 feet and 1200 feet.

3.4.1 Development of the design tool to find the distance of the Median U-turn from the crossroad

States such as Michigan, North Carolina and Maryland are currently using Median U-Turns but the distance that should be provided between the crossroad and the U-Turn is not standardized across the states. This study uses the Highway Capacity Software to find approximate limiting volumes that can maintain each LOS taking various Median U-Turn distances (i.e., distance between crossroad and the U-turn) of 300 feet, 600 feet, 900 feet and 1200 feet into consideration. Weaving occurs when two flows of traffic move to get on opposite sides of each other. The longer the distance available for the weave, the easier it is for the weave to occur. A design tool is developed from which one can decide upon the distance of the Median U-Turn from the crossroad based on the desired LOS and traffic volumes combinations of the crossroad and the major road. For unequal volumes a D factor (proportion of peak hour traffic travelling in the peak direction of flow) of 0.65 is applied as given for rural area type by Roess and MCShane [17]. HCS considers the freeway weaving for the vehicles moving from the crossroad to the Median U-Turn. In the present study since we are considering rural expressway, this design tool would be only a rough approximation. On a freeway, entering traffic from the right would use on-ramp and exiting traffic would use a left side off-ramp. On a multilane highway, entering traffic from the right must come to stop and then merge on to the highway. Exiting traffic would use the Median U-Turn. Since the HCM weaving procedure was developed for freeways, this application to multilane highways must be viewed with care. Tables 3-4 and 3-5 show various volume combinations and turning percentages for each direction taken into consideration. Tables 3-6 and 3-7 show the design tool separately for equal volumes and unequal volumes.

TABLE 3-4: Volume combinations taken into consideration

Major road volume (one way peak hour volume with equal volumes approaching from Eastbound and Westbound)	Minor road volume (vehicles/hour)			
1000	100	200	300	400
2000	100	200	300	400
2100	100	200	300	400
2200	100	200	300	400
2300	100	200	300	400
2400	100	200	300	400
2500	100	200	300	400
3000	100	200	300	400
3500	100	200	300	400
4000	100	200	300	400

TABLE 3-5: Turning volume percentages in different direction taken into consideration

Direction of travel	Percentage of volume
NBL	30
NBT	10
NBR	60
SBL	30
SBT	10
SBR	60
EBL	1
EBT	98
EBR	1
WBL	1
WBT	98
WBR	1

NBL – Northbound left turn	EBL – Eastbound left turn
NBT – Northbound through	EBT – Eastbound through
NBR – Northbound right turn	EBR – Eastbound right turn
SBL – southbound left turn	WBL – Westbound left turn
SBT – southbound through	WBT – Westbound through
SBR – Southbound right turn	WBR – Westbound right turn

Three percent of trucks have been assumed. Truck apron loop can be considered for safe turn of trucks, but in the present study since a comparative study of the three design options have been given more importance in terms of its performance measures than safety, truck apron has not been taken into geometric consideration.

Table 3-6 shows the limiting volumes of various major road and crossroad volume combinations that can satisfy a particular Median U-Turn distance and T-leg 1 and T-leg 2 for various LOS. So, if a designer have a particular volume combination obtained from the field, and want to check if he can construct a Median U-Turn with desired LOS for that volume combination, he can first verify if T-leg 1 and T-leg 2 are satisfying and them from various median U-turn distance tables in Table 3-6, he can decide one distance that satisfies a desired LOS for a given volume combination.

Table 3-6 is designed by considering equal volumes (vehicles/hour) approaching from eastbound and westbound of the major road and Table 3-7 is designed by considering unequal volumes (vehicles/hour) approaching from eastbound and westbound of the major road.

TABLE 3-6: Design tool to decide the distance between crossroad and the Median U-Turn based on the desired LOS and peak hourly one way traffic volume conditions assuming equal volumes on both the directions of the major road (WB/EB)

Equal Volumes (300ft)

Cross Road	Major Road (WB/EB)			
	B/C	**C/D**	**D/E**	**E/F**
0	1274	1666	2009	2341
50	1246	1636	1960	2326
100	1219	1609	1933	2287
150	1190	1581	1903	2256
200	1162	1550	1873	2230
250	1136	1523	1846	2199
300	1108	1496	1817	2170
350	1082	1467	1790	2143
400	1055	1440	1760	2113

Equal Volumes (900ft)

Cross Road	Major Road (WB/EB)			
	B/C	**C/D**	**D/E**	**E/F**
0	1493	1953	2334	2749
50	1462	1926	2304	2720
100	1435	1895	2273	2688
150	1405	1863	2245	2657
200	1376	1836	2215	2630
250	1349	1806	2184	2598
300	1320	1777	2154	2567
350	1293	1749	2127	2539
400	1265	1721	2098	2510

Equal Volumes (600ft)

Cross Road	Major Road (WB/EB)			
	B/C	**C/D**	**D/E**	**E/F**
0	1409	1844	2201	2591
50	1381	1813	2171	2562
100	1351	1785	2143	2532
150	1323	1753	2111	2502
200	1294	1725	2083	2471
250	1266	1697	2052	2443
300	1240	1668	2024	2413
350	1212	1642	1995	2383
400	1185	1612	1966	2355

Equal Volumes (1200ft)

Cross Road	Major Road (WB/EB)			
	B/C	**C/D**	**D/E**	**E/F**
0	1552	2037	2432	2865
50	1523	2005	2403	2837
100	1494	1974	2371	2805
150	1463	1946	2342	2773
200	1436	1916	2312	2744
250	1407	1886	2281	2714
300	1379	1857	2251	2684
350	1351	1830	2223	2652
400	1324	1800	2195	2626

Equal Volumes (Crossroad to Major road - T leg)

Cross Road	Major Road (WB/EB)			
	B/C	**C/D**	**D/E**	**E/F**
0	1731	2781	3380	3974
50	1503	2383	2835	3232
100	1295	2055	2418	2728
150	1106	1774	2085	2334
200	930	1533	1800	2010
250	767	1312	1547	1732
300	612	1116	1326	1489
350	469	932	1123	1269
400	333	762	939	1073

Equal Volumes (U-turn to Major road - T leg)

Cross Road	Major Road (WB/EB)			
	B/C	**C/D**	**D/E**	**E/F**
0	1171	1653	1843	1984
50	1089	1553	1734	1864
100	1009	1454	1627	1755
150	934	1360	1525	1653
200	861	1271	1433	1549
250	791	1187	1339	1454
300	723	1104	1254	1361
350	657	1024	1167	1274
400	593	949	1085	1189

TABLE 3-7: Design tool to decide the distance between crossroad and the Median U-Turn based on the desired weaving LOS and peak hourly one way traffic volume conditions assuming unequal volumes on both the directions of the major road (WB/EB)

Unequal Volumes (300ft)

Cross Road	Major Road (WB)			
	B/C	**C/D**	**D/E**	**E/F**
0	689	901	1076	1268
50	674	886	1061	1253
100	659	870	1045	1237
150	644	854	1029	1221
200	628	839	1013	1206
250	614	824	998	1190
300	599	809	983	1173
350	585	793	967	1159
400	571	779	952	1143

Unequal Volumes (900ft)

Cross Road	Major Road (WB)			
	B/C	**C/D**	**D/E**	**E/F**
0	807	1056	1262	1487
50	792	1042	1246	1471
100	776	1025	1230	1454
150	760	1009	1214	1438
200	744	993	1198	1422
250	729	977	1182	1405
300	714	961	1165	1389
350	699	946	1150	1373
400	684	931	1135	1357

Unequal Volumes (600ft)

Cross Road	Major Road (WB)			
	B/C	C/D	D/E	E/F
0	762	996	1190	1402
50	747	981	1174	1385
100	731	965	1158	1369
150	716	949	1143	1354
200	700	933	1126	1337
250	684	918	1111	1321
300	670	903	1095	1305
350	655	888	1079	1289
400	642	872	1063	1273

Unequal Volumes (1200ft)

Cross Road	Major Road (WB)			
	B/C	C/D	D/E	E/F
0	840	1101	1315	1550
50	824	1085	1299	1534
100	807	1068	1282	1517
150	792	1051	1266	1500
200	776	1037	1250	1484
250	761	1021	1234	1468
300	745	1004	1218	1451
350	731	989	1203	1435
400	717	973	1187	1420

Unequal Volumes (Crossroad to Major road - T leg)

Cross Road vol	Major Road (WB)			
	B/C	C/D	D/E	E/F
0	932	1497	1820	2140
50	809	1283	1527	1740
100	697	1107	1302	1469
150	596	955	1123	1257
200	501	825	969	1082
250	413	706	833	933
300	330	601	714	802
350	253	502	605	683
400	179	410	506	578

Unequal Volumes (U-turn to Major road - T leg)

Cross Road vol	Major Road (WB)			
	B/C	C/D	D/E	E/F
0	979	1335	1471	1565
50	917	1262	1393	1488
100	853	1192	1319	1411
150	794	1125	1247	1338
200	723	1058	1177	1265
250	679	993	1109	1195
300	623	928	1045	1127
350	568	869	979	1061
400	517	807	917	995

In Table 3-7, for a given WB volume the EB volume can be calculated using the D factor as follows[17].

$$EB\ volume = \frac{(WB\ Volume * 0.65)}{(1 - 0.65)} \qquad \dots\dots\dots.(3.13)$$

For various Median U-Turn distances, the limiting equal and unequal volumes of the major road with different crossroad volumes for each of the LOS B, C, D and E are shown in Figures 3-3 to 3-10. The word limiting implies that if the major road volume exceeds that particular volume, the LOS is degraded to the next LOS. For example, for a Median U-Turn distance of 600 feet as shown in Figure 3-4, the limiting volume of the major road (equal) for a crossroad volume of 300 to maintain a LOS C is 1777 and LOS B is 1320 which implies that if the volume of the major road exceeds 1777, the LOS degrades to D, if the volume lies between 1320 and 1777 the LOS is C and if the volume is less than 1320 the LOS is B.

FIGURE 3-3: Limiting one way peak hour volumes (equal volumes on eastbound and westbound) of major road for various LOS for a weaving segment of 300 feet

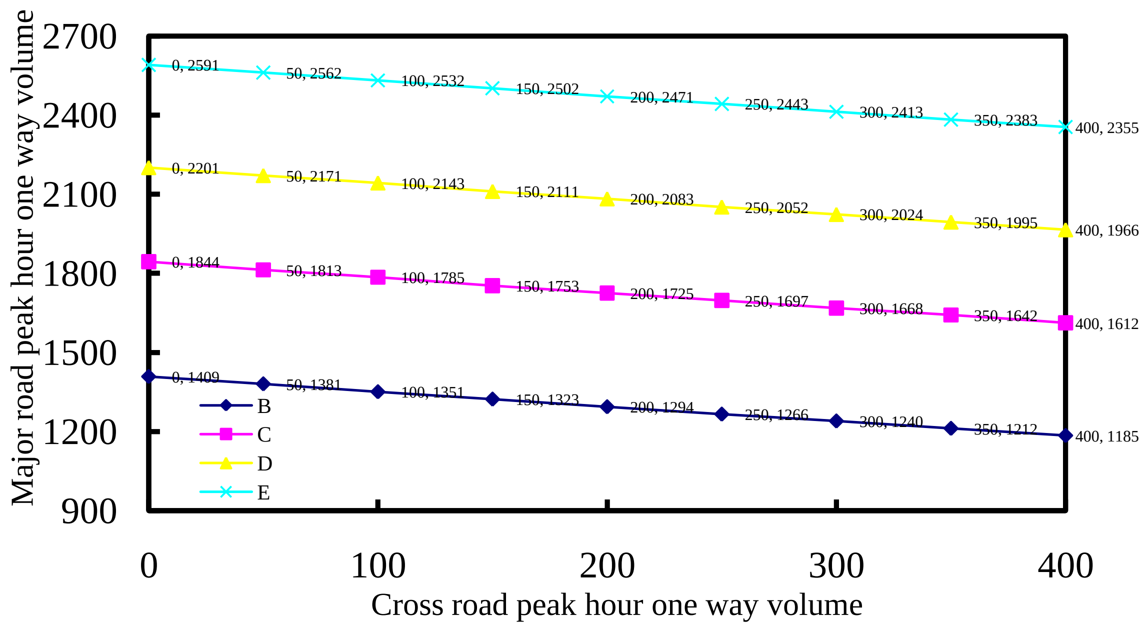


FIGURE 3-4: Limiting one way peak hour volumes (equal on eastbound and westbound) of major road for various LOS for a weaving segment of 600 feet

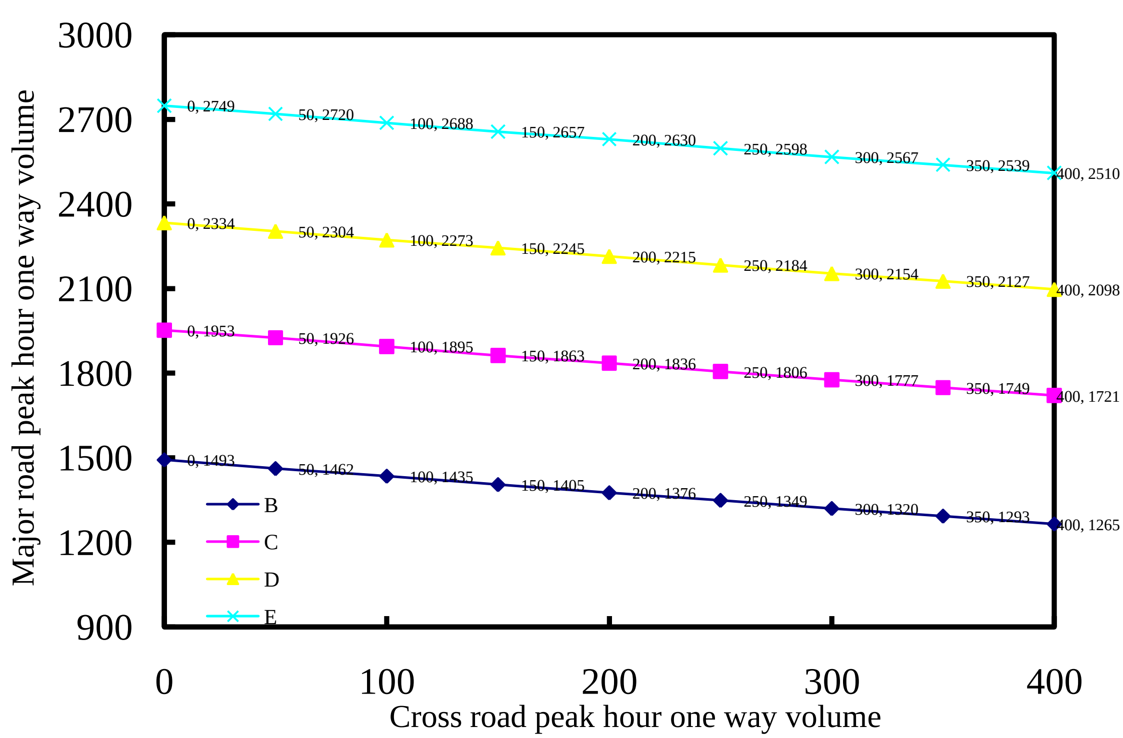


FIGURE 3-5: Limiting one way peak hour volumes (equal on eastbound and westbound) of major road for various LOS for a weaving segment of 900 feet

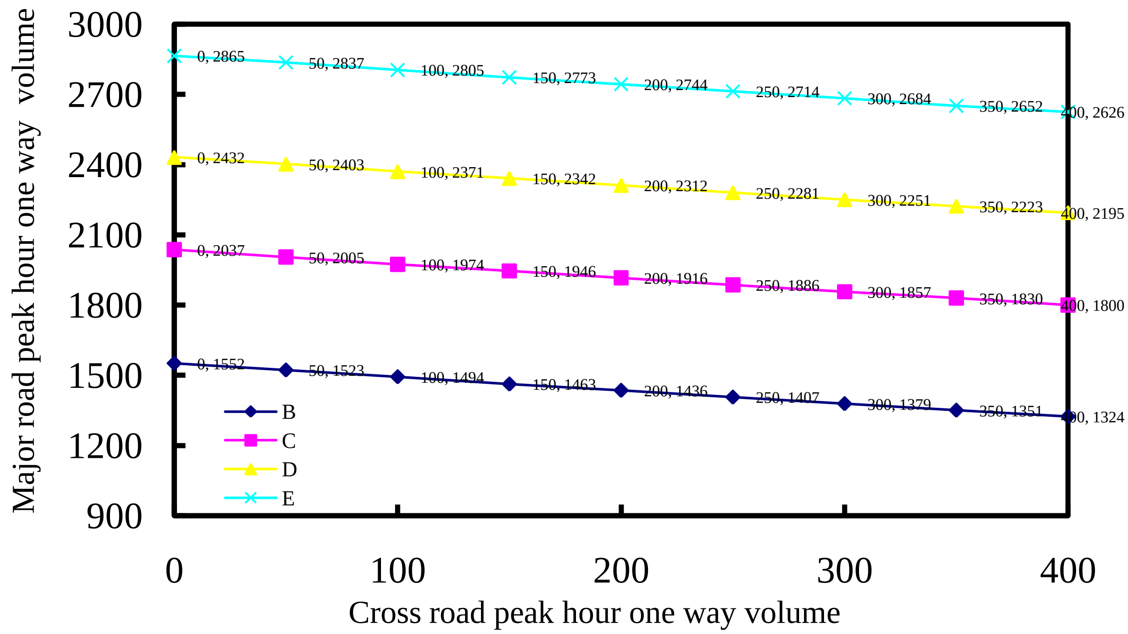


FIGURE 3-6: Limiting one way peak hour volumes (equal on eastbound and westbound) of major road for various LOS for a weaving segment of 1200 feet

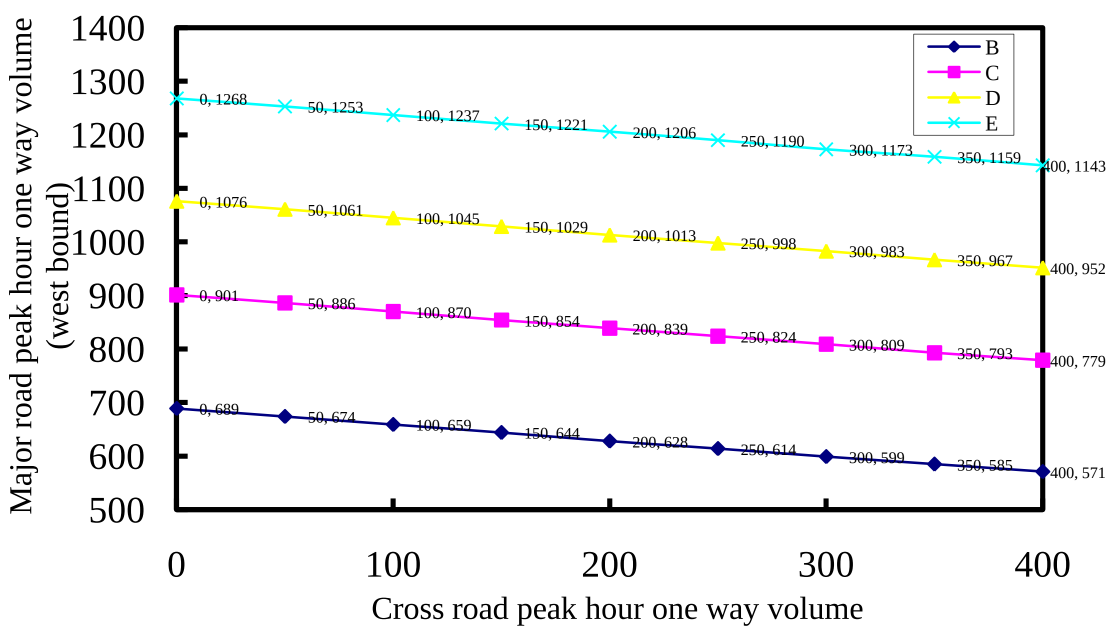


FIGURE 3-7: Limiting one way peak hour volumes (unequal on eastbound and westbound) of major road for various LOS for a weaving segment of 300 feet

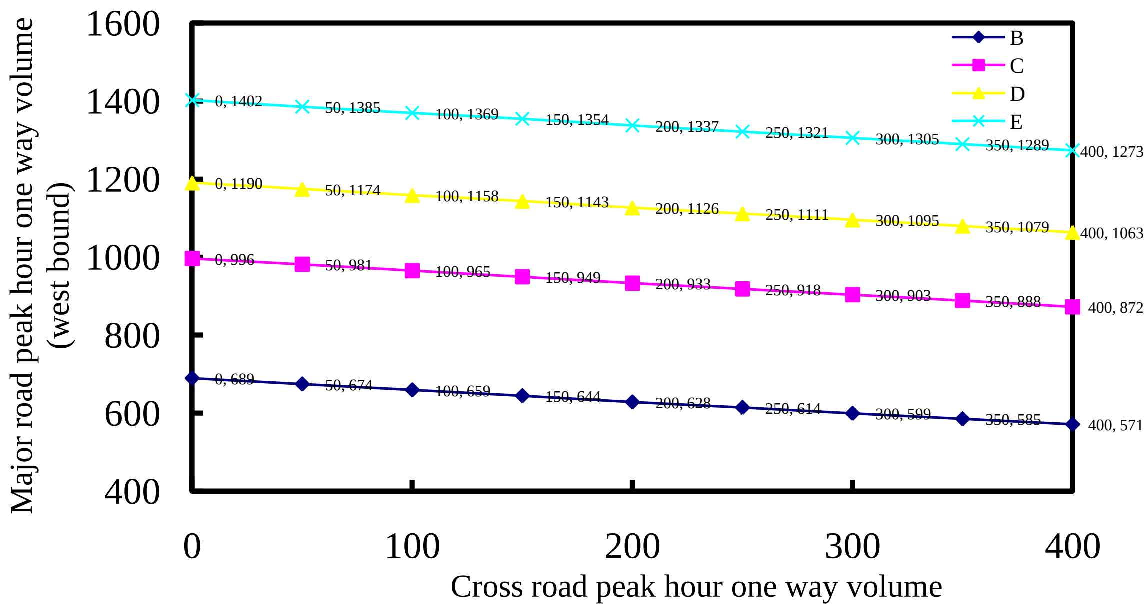


FIGURE 3-8: Limiting one way peak hour volumes (unequal on eastbound and westbound) of major road for various LOS for a weaving segment of 600 feet

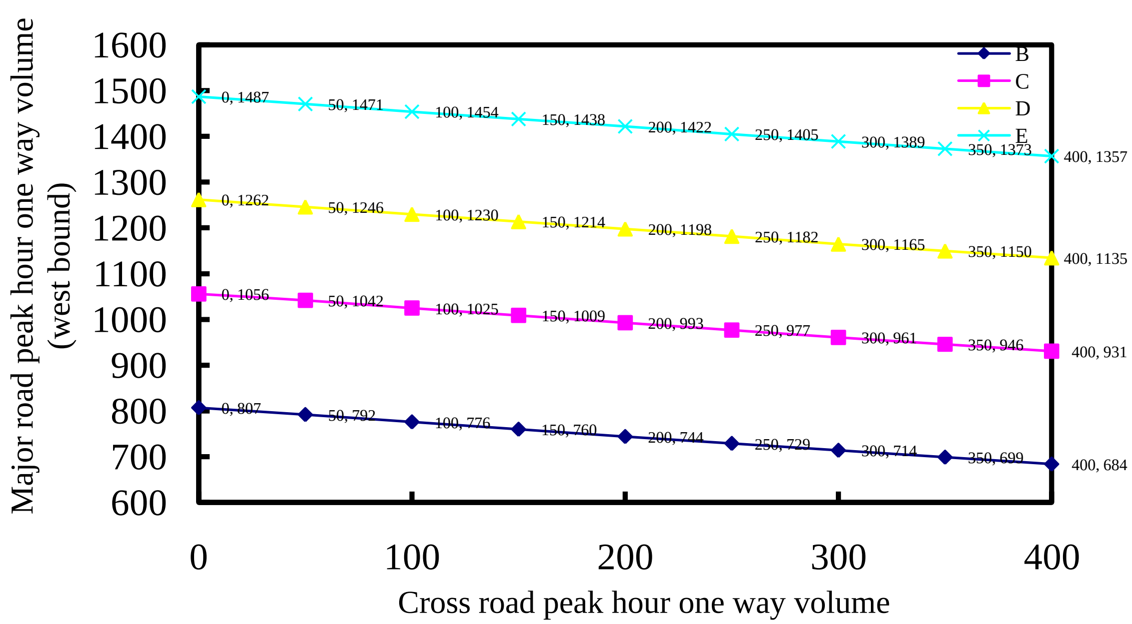


FIGURE 3-9: Limiting one way peak hour volumes (unequal on eastbound and westbound) of major road for various LOS for a weaving segment of 900 feet

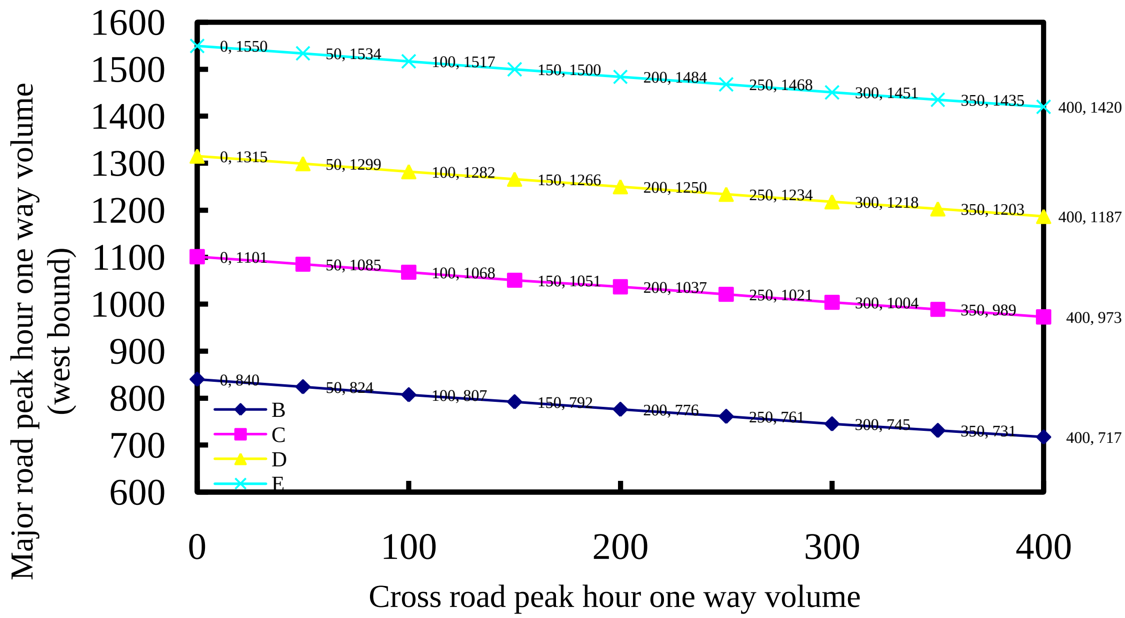
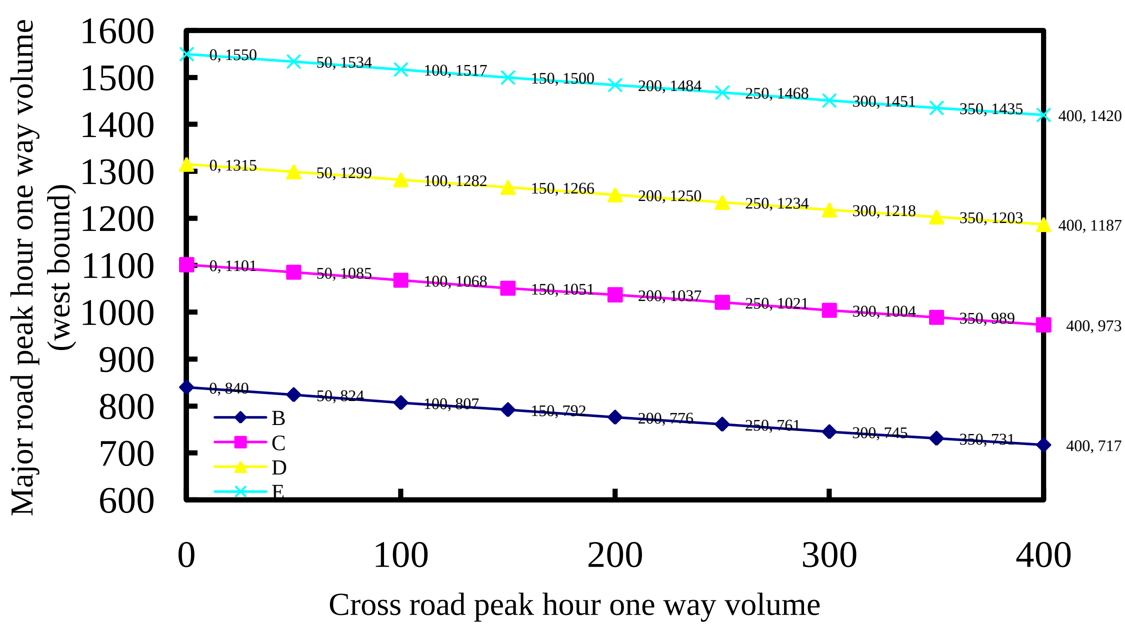


FIGURE 3-10: Limiting one way peak hour volumes (unequal on eastbound and westbound) of major road for various LOS for a weaving segment of 1200 feet



Figures 3-11 to 3-18 show the LOS limiting volumes of the major road and the crossroad for 300 feet, 600 feet, 900 feet and 1200 feet weaving segment distances, T-leg from crossroad to major road and T-leg from U-turn to major road for a desired LOS. These figures help to determine if all the three segments of the Median U-Turn can handle a considered volume combination maintaining a desired LOS. The three segments are T-leg intersection where the crossroad volume enters onto the major road, the freeway weaving segment between the crossroad and the U-turn and the T-leg intersection where the Median U-Turn volume enters onto the major road. Any one of these three segments which has the minimum volume will be the allowable volume (limiting volume) that satisfies the corresponding LOS for that particular Median U-Turn distance. For example, if we consider a major road volume of 800 (equal volume on EB and WB) and a crossroad volume of 100 with desired LOS B, the T-leg (crossroad to major road) and T-leg (U-Turn to major road) from figure 3-11 satisfies the LOS B criteria since this particular volume combination lies below those two curves. Also, this volume combination point lies below all the weaving segment distances of 300 feet, 600 feet, 900 feet and 1200 feet which indicates that all the distances satisfy the LOS B. So, we can consider a weaving segment distance of 300 feet. In figure 3-11, any point that lies below each of the six curves (300ft, 600ft, 900ft, 1200ft, T-leg CR to MR and T-leg U-turn to MR) will have a LOS B and any point that lies above that curve goes into LOS C.

FIGURE 3-11: LOS B Equal one way volume (EB/WB) on major road for peak hour with various crossroad volumes

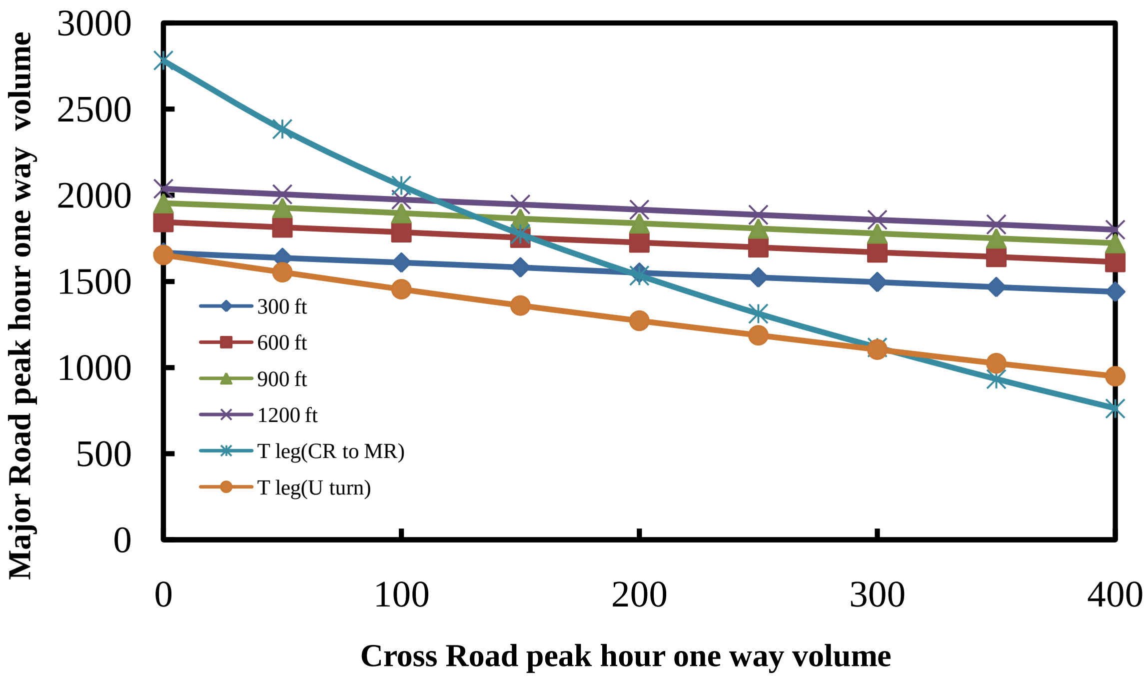


FIGURE 3-12: LOS C Equal one way volume (EB/WB) on major road for peak hour with various crossroad

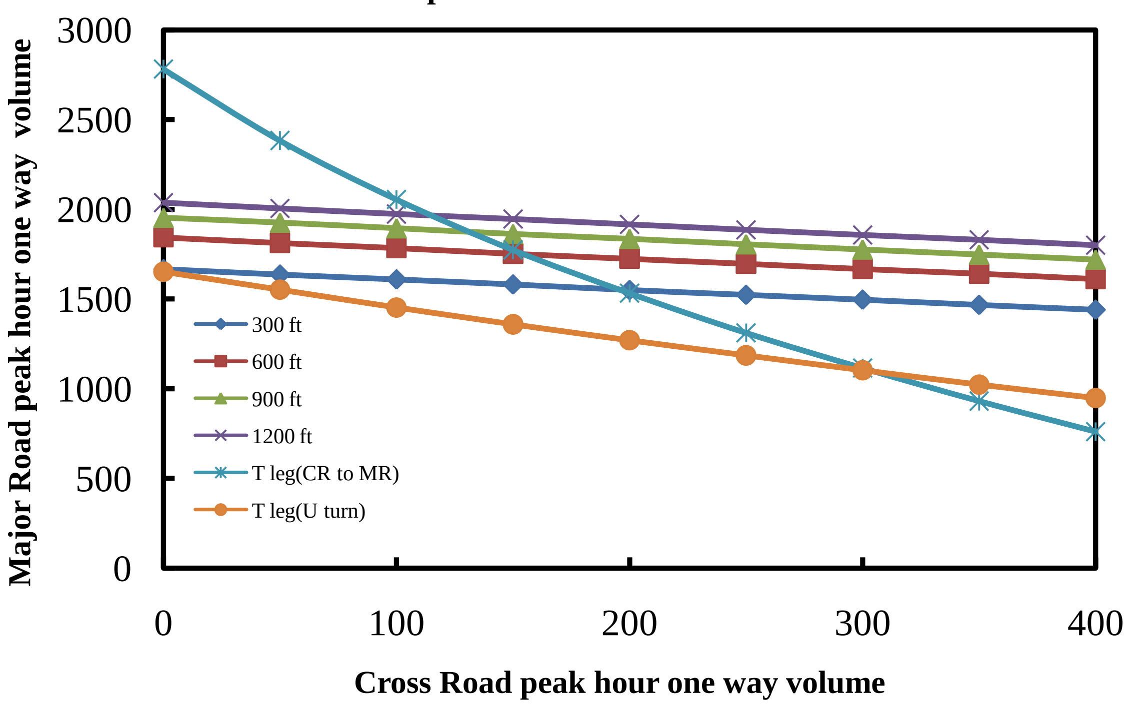


FIGURE 3-13: LOS D Equal one way volume (EB/WB) on major road for peak hour with various crossroad volumes

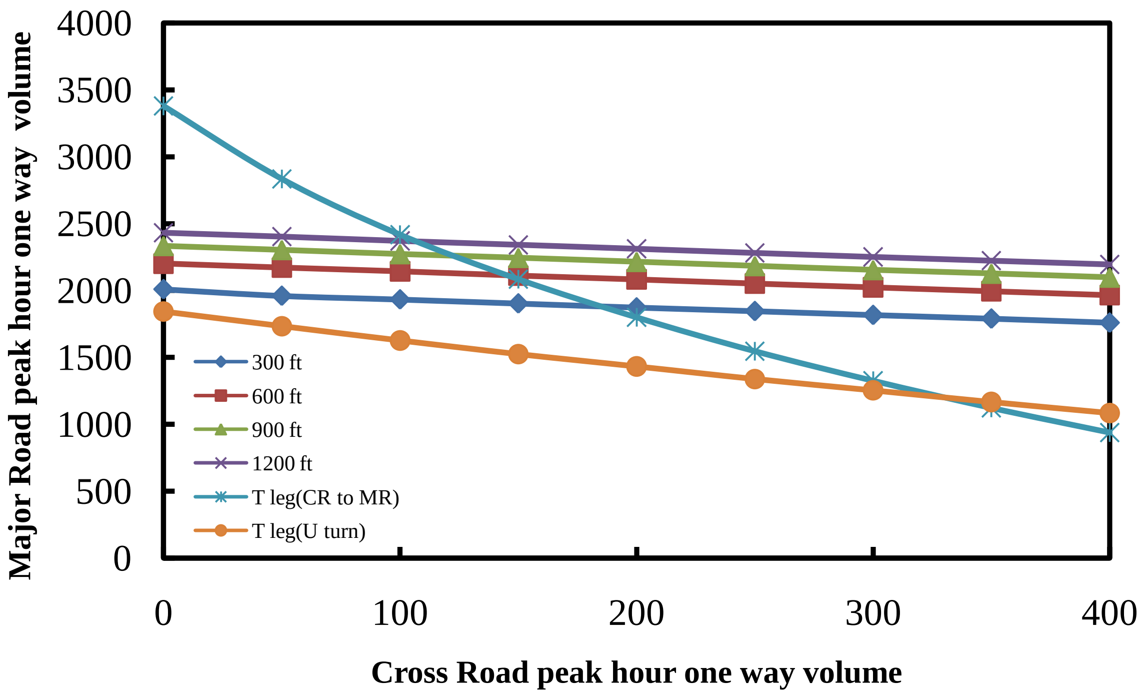


FIGURE 3-14: LOS E Equal one way volume (EB/WB) on major road for peak hour with various crossroad volumes

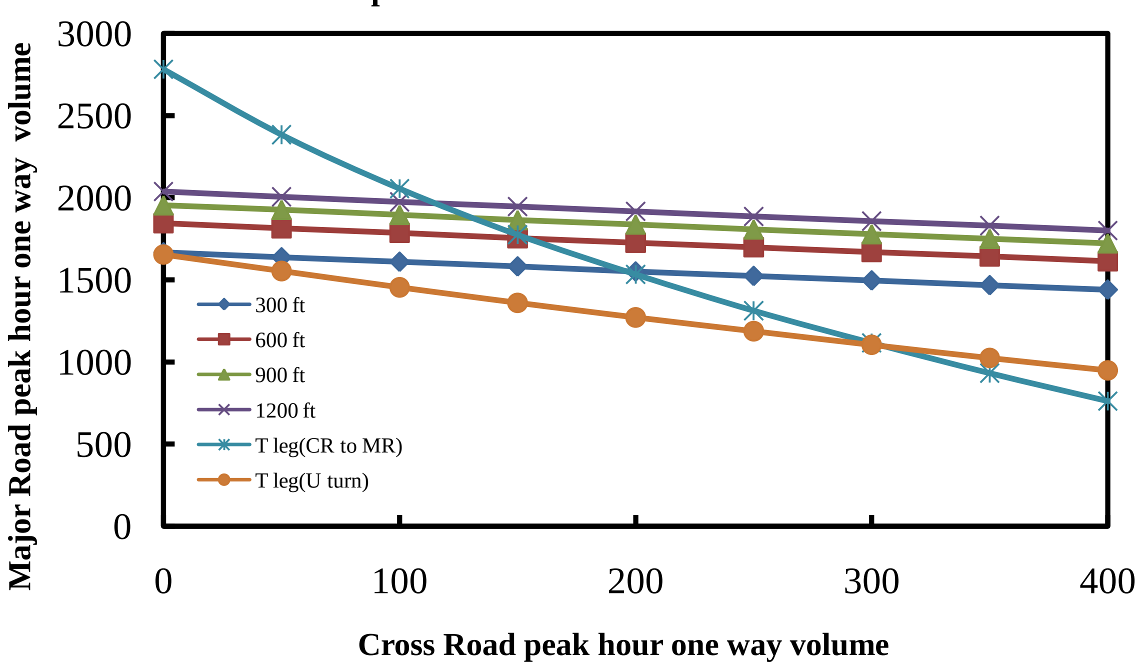


FIGURE 3-15: LOS B Unequal one way volume (WB) on major road for peak hour with various crossroad volumes

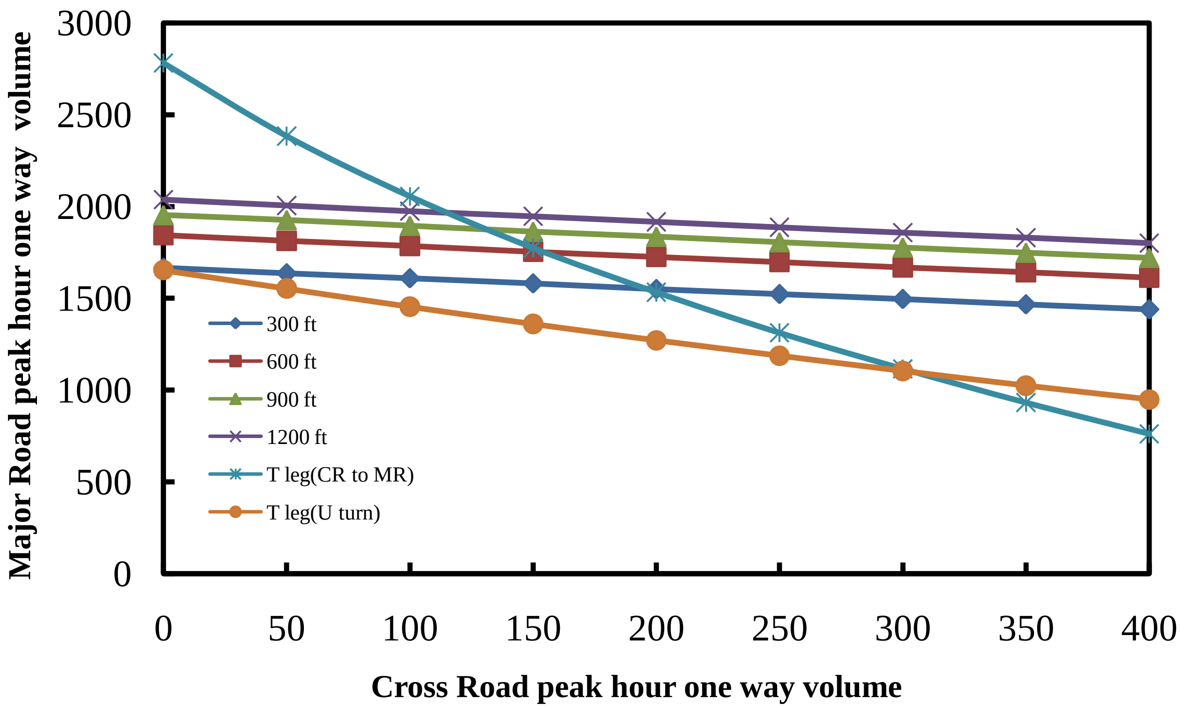


FIGURE 3-16: LOS C Unequal one way volume (WB) on major road for peak hour with various crossroad volumes

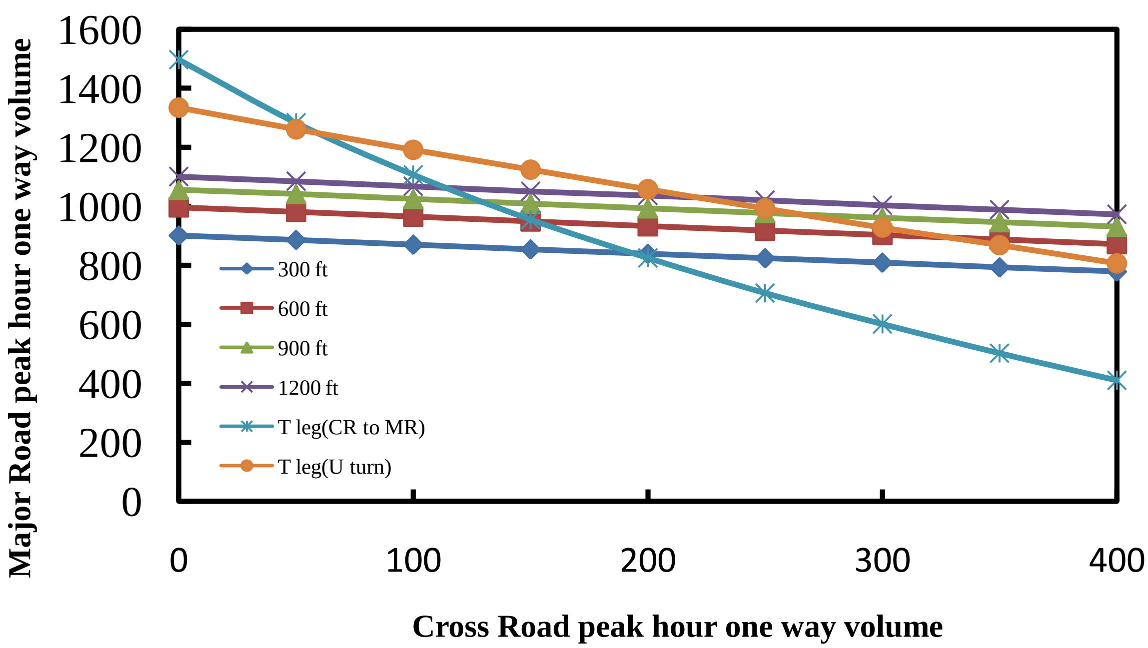


FIGURE 3-17: LOS D Unequal one way volume (WB) on major road for peak hour with various crossroad volumes

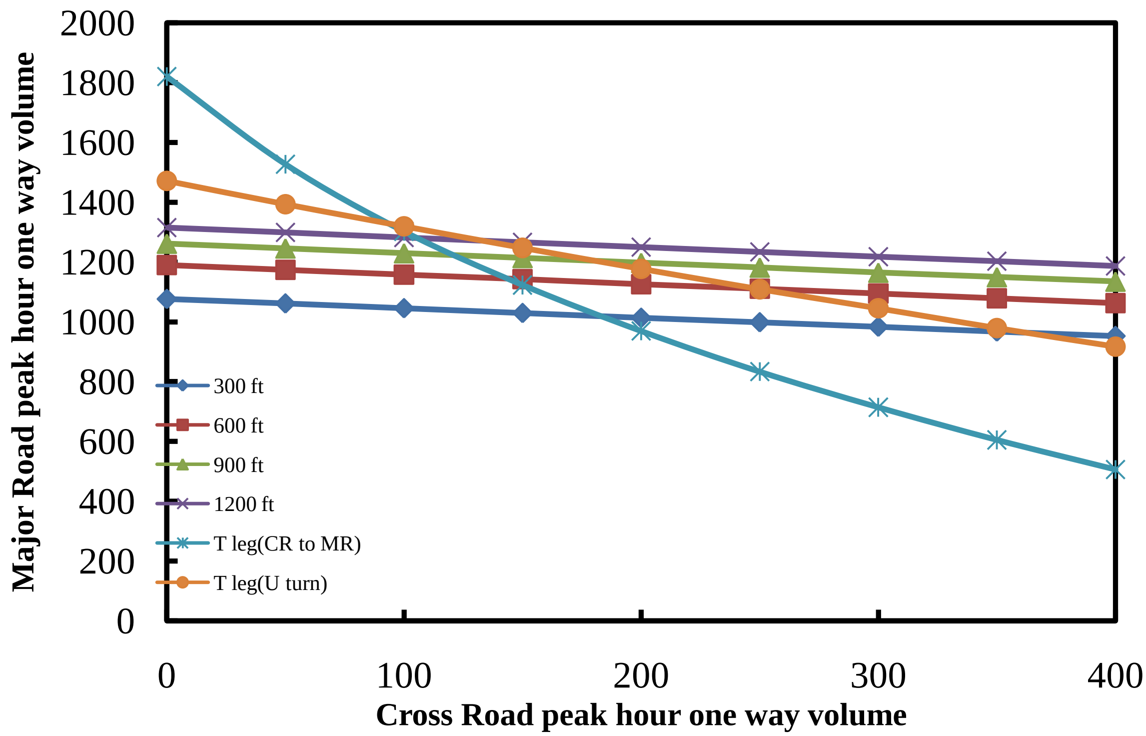
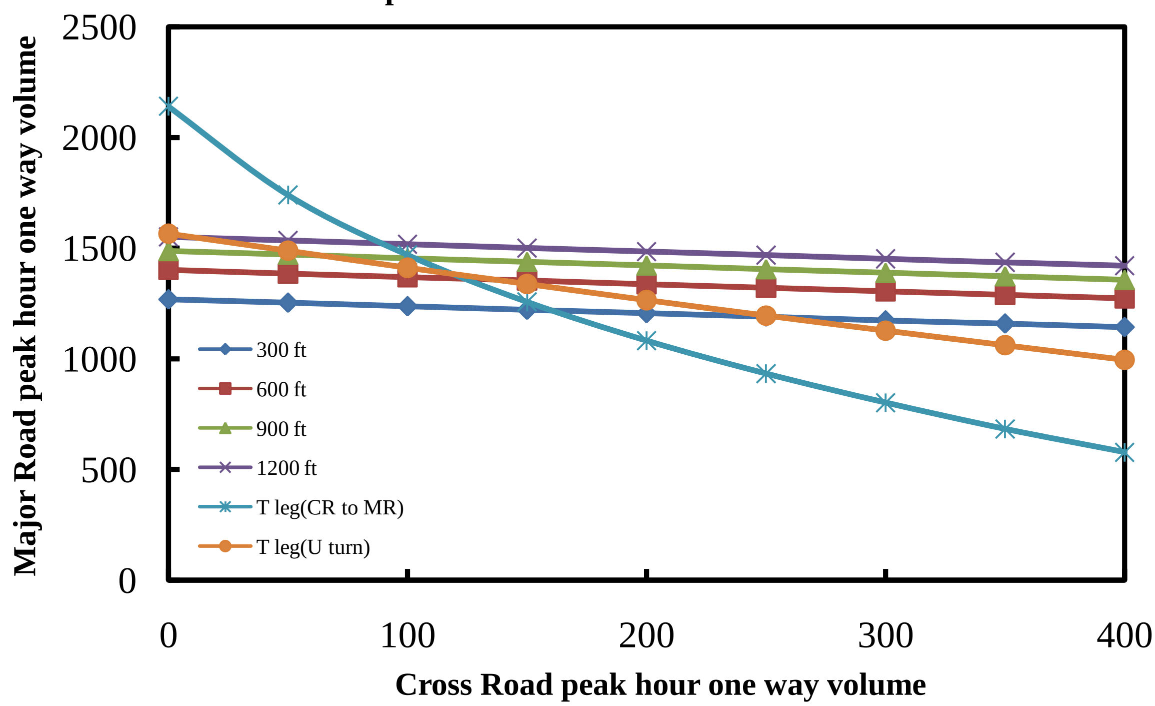


FIGURE 3-18: LOS E Unequal one way volume (WB) on major road for peak hour with various crossroad volumes



3.4.2 Control delays for Type 2 median crossovers, Unsignalized Median U-Turns and Signalized Median U-Turns

The main purpose of computing the control delays for all the three options (i.e. Type 2 median crossovers, Unsignalized Median U-turns and Signalized Median U-Turns) using HCS is to compare these results with the delays obtained from VISSIM simulation software and check how these values vary for the same volume combinations when used in two different software. Chapter five presents a more detailed discussion about delay variations.

For Type 2 median crossover the control delay is computed by considering the concept of the two-stage gap acceptance since the vehicles approaching from minor road stops at two stages (i.e. firstly the vehicle yields to the major road traffic to enter on to the median, secondly the vehicle yields to the major road traffic coming from the opposite direction to take a left turn or a through movement). In order to get the control delays for Type 2 median crossovers the TWSC option is selected in the unsignalized intersections option. The lane configuration and the volume distribution information are given. For Type 2 median crossover the median type is taken as raised curb and the median storage is given as two cars. In the output the control delay for all the directions will be obtained. The directions that are involved here are Eastbound left turn (EBL), Westbound left turn (WBL), Northbound left turn (NBL), Northbound right turn (NBR), Southbound left turn (SBL) and Southbound right turn (SBR). NBL delay counts the delay of NBL and NBT together and SBL delay counts for SBL and SBT together. The eastbound through and westbound through movements will not yield to the other movements and hence will not have any delay. The weighted volume delay is taken for all the volume combinations as shown Tables 3-8, 3-9 and 3-10. In Table 3-8, for NBL, the delays of NBL and NBT are added and for SBL, the delays of SBL and SBT are added up. The same thing is applied for volumes.

TABLE 3-8: Control delay for Type 2 median crossover from HCS

One way peak hour volumes								Control Delay (seconds)						Weighted delay (sec)
Major road volume	Cross road volume	EBL	WBL	NBL NBL+NBT	NBR	SBL SBL+SBT	SBR	EBL	WBL	NBL NBL+NBT	NBR	SBL SBL+SBT	SBR	
1000	100	10	10	40	60	40	60	10.2	10.2	25.3	12.7	25.3	12.7	17.1
1000	200	10	10	80	120	80	120	10.2	10.2	33.2	13.9	33.2	13.9	21.1
1000	300	10	10	120	180	120	180	10.2	10.2	53	15.4	53	15.4	29.8
1000	400	10	10	160	240	160	240	10.2	10.2	152	17.6	152	17.6	69.9
2000	100	20	20	40	60	40	60	18	18	252.1	24	252.1	24	99.0
2000	200	20	20	80	120	80	120	18	18	1649	32.7	1649	32.7	619.1
2000	300	20	20	120	180	120	180	18	18	7166	54.7	7166	54.7	2719.1
2000	400	20	20	160	240	160	240	18	18	*	149	*	148.6	85.8
2100	100	21	21	40	60	40	60	19.4	19.4	379.9	26	379.9	26	141.8
2100	200	21	21	80	120	80	120	19.4	19.4	2362	37.1	2362	37.1	877.0
2100	300	21	21	120	180	120	180	19.4	19.4	18522	69.6	18522	69.6	6964.4
2100	400	21	21	160	240	160	240	19.4	19.4	*	230	*	229.9	132.0
2200	100	22	22	40	60	40	60	20.9	20.9	616.5	28.3	616.5	28.3	219.8
2200	200	22	22	80	120	80	120	20.9	20.9	3479	42.7	3479	42.7	1278.8
2200	300	22	22	120	180	120	180	20.9	20.9	*	93.1	*	93.1	53.5
2200	400	22	22	160	240	160	240	20.9	20.9	*	354	354.4	*	169.1
2300	100	23	23	40	60	40	60	22.6	22.6	963.8	31	963.8	31	332.8
2300	200	23	23	80	120	80	120	22.6	22.6	5456	50.1	5456	50.1	1986.6
2300	300	23	23	120	180	120	180	22.6	22.6	*	132	*	132.3	75.3

One way peak hour volumes										Control Delay (seconds)				Weighted delay (sec)
Major road volume	Cross road volume	EBL	WBL	NBL	NBR	SBL	SBR	EBL	WBL	NBL	NBR	SBL	SBR	
				NBL+NBT		SBL+SBT				NBL+NBT		SBL+SBT		
2300	400	23	23	160	240	160	240	22.6	22.6	*	490	*	490	279.2
2400	100	24	24	40	60	40	60	24.7	24.7	1529	34	1529	34	514.5
2400	200	24	24	80	120	80	120	24.7	24.7	9049	59.3	9049	59.3	3266.2
2400	300	24	24	120	180	120	180	24.7	24.7	*	192	*	191.6	108.3
2400	400	24	24	160	240	160	240	24.7	24.7	*	646	*	646.3	367.2
2500	100	25	25	40	60	40	60	26.9	26.9	2109	37.6	2109	37.6	698.3
2500	200	25	25	80	120	80	120	26.9	26.9	48620	72.9	48620	72.9	17329.0
2500	300	25	25	120	180	120	180	26.9	26.9	*	287	*	286.9	161.0
2500	400	25	25	160	240	160	240	26.9	26.9	*	834	*	834	472.5
3000	100	30	30	40	60	40	60	44.9	44.9	*	68.1	*	68.1	41.8
3000	200	30	30	80	120	80	120	44.9	44.9	*	310	*	309.5	167.3
3000	300	30	30	120	180	120	180	44.9	44.9	*	1110	*	1110	609.5
3000	400	30	30	160	240	160	240	44.9	44.9	*	2018	*	2018	1129.5
3500	100	35	35	40	60	40	60	91.2	91.2	*	165	*	164.5	96.8
3500	200	35	35	80	120	80	120	91.2	91.2	*	1103	*	1103	576.8
3500	300	35	35	120	180	120	180	91.2	91.2	*	2430	*	2430	1315.2
3500	400	35	35	160	240	160	240	91.2	91.2	*	3786	*	3786	2096.2
4000	100	40	40	40	60	40	60	297.4	297	*	582	*	582.4	334.6
4000	200	40	40	80	120	80	120	297.4	297	*	2464	*	2464	1281.6
4000	300	40	40	120	180	120	180	297.4	297	*	4480	*	4480	2406.8
4000	400	40	40	160	240	160	240	297.4	297	*	6510	*	6510	3577.9

*Indicates that the delay results in HCM are not reliable when V/C >1

To calculate the control delay for unsignalized intersection, the delays of the vehicles that enter on to the major road from the crossroad are added to the delay of the vehicles that enter on to the major road from the Median U-Turn. Though there might be some amount of delay experienced by the vehicles while weaving from the point they enter on to the major road from the crossroad till they reach the Median U-Turn, HCS does not explicitly account for that delay. The intersection where the vehicles enter from the crossroad on to the major road is considered as T-legged (T-leg one) unsignalized intersection and the intersection where the vehicles move on to the major road from the Median U-Turn is considered as a T-legged (T-leg two) unsignalized intersection. The movements taken into consideration at T-leg one are EBT, EBR and NBR movements. EBT movement includes EBT, EBL and SBL. EBR movement includes SBT, EBR and WBL. NBR movement includes NBT, NBR and NBL. The NBR movement undergoes the delay at this intersection and its weighted volume delay is shown in the Table 3-9. The movements taken into consideration at T-leg two are NBL and WBT movements. NBL movement includes NBT, NBL and EBL turn movements. WBT movement includes WBT, WBR and WBL movements. The EBL turn movement undergoes the delay as they yield to the WBT movements. The total delay is obtained by adding the control delays obtained at T-leg one and T-leg two as shown in Table 3-9.

TABLE 3-9: Control delay (sec) for one way peak hour volume for unsignalized Median U-turn from HCS

T- leg 1 (from crossroad to the major road) for Unsignalized

Major road vol	Cross road vol	EBT (Volume) EBT+EBL+ SBL	EBR (Volume) SBT+EBR+ WBL	NBR (Volume) NBT+NBR+ NBL	control Delay NBR
1000	100	1020	30	100	12.7
1000	200	1050	40	200	15.1
1000	300	1080	50	300	19.6
1000	400	1110	60	400	30.9
2000	100	2010	50	100	23.3
2000	200	2040	60	200	43.8
2000	300	2070	70	300	231.7
2000	400	2100	80	400	848.3
2100	100	2109	52	100	25.3
2100	200	2139	62	200	53.1
2100	300	2169	72	300	335.3
2100	400	2199	82	400	1026
2200	100	2208	54	100	27.6
2200	200	2238	64	200	65.9
2200	300	2268	74	300	459.4
2200	400	2298	84	400	1207
2300	100	2307	56	100	30.3
2300	200	2337	66	200	85.1
2300	300	2367	76	300	599.8
2300	400	2397	86	400	1414

T- leg 2 (from U-turn to major road) for Unsignalized

NBL (Volume) NBT+NBL+ EBL	WBT (Volume) WBT+WBR+ WBL	control delay EBL	Total Delay (sec)
50	1000	12.4	25.1
90	1000	13.1	28.2
130	1000	13.9	33.5
170	1000	14.9	45.8
60	2000	22	45.3
100	2000	25.9	69.7
140	2000	32.1	263.8
180	2000	43.1	891.4
61	2100	23.6	48.9
101	2100	28.5	81.6
141	2100	36.5	371.8
181	2100	52.2	1078.2
62	2200	25.5	53.1
102	2200	31.5	97.4
142	2200	42.2	501.6
182	2200	65.5	1272.5
63	2300	27.8	58.1
103	2300	35.3	120.4
143	2300	49.9	649.7
183	2300	86.1	1500.1

Major road volume	Cross road volume	EBT (Volume)	EBR (Volume)	NBR (Volume)	Control Delay	NBL (Volume)	WBT (Volume)	control delay	Total Delay (sec)
		EBT+EBL+ SBL	SBT+EBR+ WBL	NBT+NBR+ NBL	NBR	NBT+NBL+ EBL	WBT+WBR+ WBL	EBL	
2400	100	2406	58	100	33.2	64	2400	30.3	63.5
2400	200	2436	68	200	117.1	104	2400	40	157.1
2400	300	2466	78	300	754.4	144	2400	60.4	814.8
2400	400	2496	88	400	1638	184	2400	119.6	1757.6
2500	100	2505	60	100	36.9	65	2500	33.1	70
2500	200	2535	70	200	164.2	105	2500	45.3	209.5
2500	300	2565	80	300	921.7	145	2500	74.3	996
2500	400	2595	90	400	1858	185	2500	169.6	2027.6
3000	100	3000	70	100	71.3	70	3000	58.7	130
3000	200	3030	80	200	728.7	110	3000	120.5	849.2
3000	300	3060	90	300	1975	150	3000	371.4	2346.4
3000	400	3090	100	400	3304	190	3000	829.6	4133.6
3500	100	3495	80	100	216.6	75	3500	153	369.6
3500	200	3525	90	200	1704	115	3500	576.6	2280.6
3500	300	3555	100	300	3535	155	3500	1270	4805
3500	400	3585	110	400	5417	195	3500	2009	7426
4000	100	3990	90	100	746.7	80	4000	615.4	1362.1
4000	200	4020	100	200	3128	120	4000	1595	4723
4000	300	4050	110	300	5716	160	4000	2646	8362
4000	400	4080	120	400	8316	200	4000	3711	12027

To obtain the delays for signalized intersections, the signalized intersections option is selected in HCS and the same procedure as stated for unsignalized intersections is followed. Here the delay values at T-leg one will remain same as that for unsignalized intersection but the delay values at T-leg two will vary as a signal is installed. The delay occurs in both the directions i.e., WBT and NBL turn movements at T-leg two. The weighted average volume delay is taken and is added to the delay obtained at T-leg one to get the total delay as shown in Table 3-10. For the signal timings the cycle length is assumed to be 70 seconds with a two phase signal. The respective signal timing information is fed in to HCS and the delays are obtained. For the signal timings the following assumption has been made. Lost time is taken as eight seconds (i.e., four seconds for each phase). The yellow time is taken as four seconds. The minimum green time is taken as eight seconds. The following distribution of green time has been assumed for left turn and through movements.

$$\frac{g_L}{g_T} = \frac{\left(\frac{LT_{Vol}}{0.85}\right)}{TH_{Vol}} \qquad \text{... ... ... (3.14)}$$

Where,

g_L = Green time for left turning movement (assumed ≥ 8.0 seconds)

g_T = Green time for through movement (assumed ≤ 54.0 seconds)

LT_{vol} = Left turn volume

TH_{vol} = Through movement volume

TABLE 3-10: Control delay (sec) for one way peak hour volume for signalized Median U-turn from HCS

T leg (from U-turn to major road) for Signalized

Major road vol	Cross road vol	EBL (volume) NBT+NBL+EBL	WBT (volume) WBT+WBR+WBL	control delay WBT	control delay EBL	Weighted Delay
1000	100	50	1000	3.1	31.6	4.5
1000	200	90	1000	3.1	37.2	5.9
1000	300	130	1000	4.2	33.5	7.6
1000	400	170	1000	5.6	30.5	9.2
2000	100	60	2000	7.2	32.7	7.9
2000	200	100	2000	7.2	39.3	8.7
2000	300	140	2000	7.2	55.8	10.4
2000	400	180	2000	7.2	131	17.4
2100	100	61	2100	8.4	32.8	9.1
2100	200	101	2100	8.4	39.6	9.8
2100	300	141	2100	8.4	56.4	11.4
2100	400	181	2100	8.4	134.9	18.4
2200	100	62	2200	10.1	32.9	10.7
2200	200	102	2200	10.1	39.8	11.4
2200	300	142	2200	10.1	57.1	12.9
2200	400	182	2200	10.1	138.9	19.9
2300	100	63	2300	12.9	33	13.4
2300	200	103	2300	12.9	40	14.1

T leg (from cross road to major road) for Signalized

EBT (volume) EBT+EBL+SBL	EBR (volume) SBT+EBR+WBL	NBR (volume) NBT+NBR+NBL	Control Delay NBR	Total Delay
1020	30	100	12.7	17.2
1050	40	200	15.1	21.0
1080	50	300	19.6	27.2
1110	60	400	30.9	40.1
2010	50	100	23.3	31.2
2040	60	200	43.8	52.5
2070	70	300	231.7	242.1
2100	80	400	848.3	865.7
2109	52	100	25.3	34.4
2139	62	200	53.1	62.9
2169	72	300	335.3	346.7
2199	82	400	1026	1044.4
2208	54	100	27.6	38.3
2238	64	200	65.9	77.3
2268	74	300	459.4	472.3
2298	84	400	1207	1226.9
2307	56	100	30.3	43.7
2337	66	200	85.1	99.2

Major road volume	Cross road volume	EBL (volume) NBT+NBL+EBL	WBT (volume) WBT+WBR+WBL	control delay WBT	EBL	Weighted Delay
2300	300	143	2300	12.9	57.8	15.5
2300	400	183	2300	12.9	143	22.5
2400	100	64	2400	19	33.2	19.4
2400	200	104	2400	19	40.5	19.9
2400	300	144	2400	19	58.5	21.2
2400	400	184	2400	19	147.3	28.1
2500	100	65	2500	37.9	33.3	37.8
2500	200	105	2500	37.9	40.7	38.0
2500	300	145	2500	37.9	59.3	39.1
2500	400	185	2500	37.9	156.4	46.1
3000	100	70	3000	361.5	34	354.0
3000	200	110	3000	361.5	41.9	350.2
3000	300	150	3000	361.5	64.3	347.3
3000	400	190	3000	361.5	181.6	350.8
3500	100	75	3500	718.4	34.6	704.1
3500	200	115	3500	718.4	43.6	696.9
3500	300	155	3500	718.4	69.4	690.9
3500	400	195	3500	718.4	216.5	691.9
4000	100	80	4000	1076	35.5	1055.6
4000	200	120	4000	1076	45.2	1046.0
4000	300	160	4000	1076	77.1	1037.6
4000	400	200	4000	1076	248.7	1036.6

EBT (volume) EBT+EBL+SBL	EBR (volume) SBT+EBR+WBL	NBR (volume) NBT+NBR+NBL	Control Delay NBR	Total Delay
2367	76	300	599.8	615.3
2397	86	400	1414	1436.5
2406	58	100	33.2	52.6
2436	68	200	117.1	137.0
2466	78	300	754.4	775.6
2496	88	400	1638	1666.1
2505	60	100	36.9	74.7
2535	70	200	164.2	202.2
2565	80	300	921.7	960.8
2595	90	400	1858	1904.1
3000	70	100	71.3	425.3
3030	80	200	728.7	1078.9
3060	90	300	1975	2322.3
3090	100	400	3304	3654.8
3495	80	100	216.6	920.7
3525	90	200	1704	2400.9
3555	100	300	3535	4225.9
3585	110	400	5417	6108.9
3990	90	100	746.7	1802.3
4020	100	200	3128	4174.0
4050	110	300	5716	6753.6
4080	120	400	8316	9352.6

More information about the methodology of HCS can be obtained from Highway capacity manual [15]**.** Sample HCS output is shown in Appendix II.

CHAPTER 4

APPLICATION OF VISSIM SIMULATION SOFTWARE

In the earlier chapter, a detailed analysis of the Highway Capacity Software and the evaluation of various parameters were described. This chapter describes the importance of VISSIM software over HCS and the deficiencies of HCS pertaining to the present analysis. There are many simulation tools available for traffic simulation and one among these tools is VISSIM simulation software, version 4.2 [18] is used in the present study. A detailed description of the deficiencies of HCS and the need to address those deficiencies with the use of VISSIM simulation software is given in this chapter. The results obtained from VISSIM are analyzed in the later part of this chapter.

4.1. Limitations of HCM/HCS analysis

HCS is used to analyze the intersection as a whole. It gives the LOS and the delay for the intersection as a whole and for each movement. For a Type 2 median crossover we consider the raised median with a storage length for two vehicles but the true picture of the median and its geometrics is not clearly established. HCS cannot analyze the Median U-Turn as a whole and cannot give the performance measures for the considered model. Instead it gives the control delays for the T-leg one intersection and T-leg two intersection separately. It does not account for the delay that the traffic experiences while travelling on the major road before reaching the Median U-Turn. To find the LOS for the vehicles that enter the major road from the crossroad and move on till the Median U-

Turn, freeway weaving option is considered, as a separate option for analysis of rural median crossovers is not available in HCS. The dynamic nature of driver behavior is not given in HCS. Considering all the above mentioned factors, HCS cannot be used to represent a true picture of Type 2 median crossover and Median U-Turn options in terms of the inputs that are given and the outputs that are obtained to analyze and apply our engineering judgment.

4.2. Selection of VISSIM

Some of the features contained in VISSIM that supported its usage over HCS in the analysis of the Type 2 median crossovers and Median U-Turns are as follows:

- VISSIM is a simulation software that can model complex intersections and design options with a precision down to millimeter.
- Type 2 median crossover and Median U-Turns can be modeled based on the field geometrics and controls.
- Stochastic distribution of speed and spacing thresholds replicate individual driver behavior characteristics.
- VISSIM presents the performance measures such as travel time, delay time and queue lengths which can be used to select the most effective alternative based can be obtained in the output.
- VISSIM also has the following additional characteristics which are not available in HCS
    1. Analysis of slow speed weaving and merging areas.

2. Capacity and operational analyses of complex station layouts for light rail and bus systems can be analyzed using VISSIM.
3. Evaluation and optimization of traffic operations in a combined network of coordinated and actuated traffic signals.
4. Easy comparison of the design alternatives including signalized and stop sign controlled intersections, roundabouts and grade separated interchanges.
5. The traffic model in VISSIM follows a discrete, stochastic, time step based microscopic model with driver vehicle units as single entities.

- Unlike HCM, which assumes random headway distribution, we can input headway distribution and minimum gap time in VISSIM.

4.3. Performance Measures

HCM uses control delay as the measure of effectiveness to find the LOS at any intersection. In the case of Type 2 median crossover and Median U-Turn considered in our study, apart from control delay, many other factors such as queue lengths and travel times can be taken as measures of effectiveness. Queue length is an important factor because excessive queue lengths can cause the vehicles to extend back on to the major road (i.e. rural expressway lane). This can cause the blockage of the traffic moving on the expressway lane. In VISSIM there are three queuing parameters (i.e., maximum queue length, average queue length and number of stops of the vehicle in the queue). In the present study the maximum queue length and average queue length are taken into consideration. The values of control delay obtained from HCS are compared with the delay values obtained from the simulation tool to check the variation in the values when obtained from two different software. The travel times, delay times and the queue lengths for all the three design options (i.e. Type 2 median crossover, unsignalized Median U-Turn and signalized Median U-Turn considered in our study are compared and further analysis was made which is discussed later). Table 4-1 describes the performance measures used.

TABLE 4-1: Description of performance measures

Measure	**Description**
Travel time (sec)	The average travel time (including the waiting or dwell times) is determined as the time the vehicle crosses the first cross section to crossing the second cross section.
Delay time (sec)	All vehicles concerned by the travel time measurements are also considered for delay time measurement. The mean time delay is calculated from all vehicles observed on a single link or several link sections.
Maximum queue length (feet)	The current queue length is measured upstream every time step. From these values the maximum is computed for every time interval.
Average queue length (feet)	The current queue length is measured upstream every time step. From these values the arithmetical average is computed for every time interval

4.4. VISSIM methodology

VISSIM is a microscopic, time step and behavior based simulation model developed to model urban traffic and public transit operations. The program can analyze traffic and transit operations under constraints such as lane configuration, traffic composition, traffic signals, transit stops etc., thus making it a useful tool for the evaluation of various alternatives based on the transportation engineering and planning measures of effectiveness. VISSIM can be applied in a variety of transportation problems [18]. There are various components in this tool that are used to construct the three design options considered in this study. Each of these components is described below along with the assumptions that were made in the study since the data has been assumed.

4.4.1. Network Background

For easy coding of the network on this tool a scaled background is considered to be important. This background can be obtained from any site for maps like google maps or teraserver. The map along with the scale on it is downloaded and loaded on to VISSIM.

4.4.2. Links and Connectors

Links and connectors are used to code the roadway network. Links give the graphical representation of the road along with length and width. Connectors are used to connect these links which are usually used at turning points, off ramps and on ramps. The width of the lane was assumed to be twelve feet in the VISSIM simulation for the present study.

There were two lanes in each direction on the expressway (Eastbound and Westbound) and one lane in each direction on the crossroad (Northbound and Southbound). For Type 2 median crossover there is exclusive right turn lane for northbound right turn and southbound right turn. In the same way there are exclusive left turn lanes for eastbound left turn and westbound left turn. The median width for Type 2 median crossovers is assumed to be 60 feet.

4.4.3 Vehicle Input

This option enables us to input the vehicular volume at the starting point of the network in the desired direction. In order to input the volume on to the network at least one traffic composition should be defined. Two traffic compositions have been assumed separately for major road and the minor road with cars and HGV's moving at a speed range of 60 to 70 mph on the major road (expressway) and 30 to 40 mph on the minor road (cross road). The time for simulation is given as 3600 seconds. The naming for these vehicle inputs were given as northbound (NB), southbound (SB), eastbound (EB) and westbound (WB). Various combinations of the volume assumptions were made as described in chapter three.

4.4.4. Routing Decisions

Routing decisions are used to assign the direction of travel for the vehicles that flow on the network. A red bar on the network indicates the point from which the vehicles are assigned a particular route and the corresponding green bar gives the destination point.

The yellow color band representation between the red bar and its associated green bar indicates the path that the vehicles take. The distribution of vehicles in each direction of flow can be given in terms of percentages or number of vehicles. In this study the percentage distribution is given as described in the previous section. The time interval for which the traffic simulation runs is given as 3600 seconds.

4.4.5. Priority rules

For non-signal protected conflicting movements the right of way is modeled using the priority rules. The priority rule consists of one stop line and one or more conflict markers. The vehicles approaching the red bar stop at that red bar and observe if they have any vehicle in the conflict zone defined by the green bar. The minimum gap time and headway distribution can be given for this parameter. There is a priority rule defined at the unsignalized Median U-Turn in the present study.

4.4.6. Stop sign control

Stop sign control in VISSIM forces the vehicle to stop for at least one time step regardless of the presence of the conflicting traffic. Stop signs can be used to model regular stop signs, right turn on red (RTOR) option and dispatch counters (e.g. customs, road toll etc.,). In case of RTOR the stop sign becomes active only when the associated signal controller phase displays red.

4.4.7. Signalized intersections

In VISSIM signalized intersections can be modeled using either fixed time control or an optional external signal state generator. Every signal controller (SC) is represented by its individual signal controller number and signal groups (signal phase) as its smaller control unit. VISSIM also discriminates signal heads and signal groups. Signal head is the actual device showing the picture of associated signal group. These signal heads are coded individually for each travel lane at the location of the signal stop line. Signal indications are updated at the end of each simulation second. In the present study fixed time signal control is used for signalized Median U-Turn option. For fixed time signal control VISSIM starts a signal cycle at second one and ends with second cycle time. Only red end and green end times need to be defined along with amber and red/amber. The values of amber and red/amber can be given as zero in order to switch them off.

4.4.8. Travel time sections

These sections are used to measure the travel time and delay time for the required sections on the network. Each travel time section consists of a start and an end cross section. The average travel time including waiting or dwell times is determined as the time the vehicle crosses the first cross section to crossing the second cross section. An output file for travel time has *.RSZ extension and it contains file title, path and name of the input file, simulation comment, date and time of evaluation, list of travel time sections that has been evaluated, travel time in seconds and number of vehicles.

4.4.9. Delay times

Based on the travel time sections VISSIM can generate the delay times independently for the vehicle classes selected in these travel time sections. As delay times are based on the travel time sections no additional definitions need to be done. The output file for delay time has *.VLZ extension and it contains file title, path and name of the input file, simulation comment, data and time of evaluation, list of delay segments that has been evaluated and table with delay data measured for each section and time interval.

4.4.10. Queue counters

The queue counters in VISSIM provides the average queue length, maximum queue length and number of vehicle stops in the queue. Queues are counted from the point the queue counter is placed on the link or connector upstream to the final vehicle that is in the queue condition. If the queue extends back on to multiple approaches the queue counter will record information for all those approaches and reports the maximum queue length.

4.4.11 Geometrics of the three design options in VISSIM

In the construction of the three design options in VISSIM, the following assumptions were made.

In the base alternative, there is an exclusive left turn lane for eastbound and westbound direction and the length of this deceleration lane is taken as 350 feet. For the Median U-

Turn the deceleration length is taken as 350 feet. This length is calculated as the braking distance and is given as [17]

$$d_b = \frac{{s_i}^2 - {s_f}^2}{30(F + 0.01G)} \qquad \ldots \ldots \ldots (4.1)$$

Where,

d_b = Braking distance in feet

s_i = Initial speed in mph

s_f = Final speed in mph

F = Forward rolling or skidding friction

G = Grade in percentage

The initial speed is taken as 60 mph and the final speed is taken as zero miles per hour with a level grade. A standard deceleration rate is adopted as 11.2 ft/s^2. The standard friction factor for braking computations is taken as 0.348. For northbound right turn and southbound right turn, an exclusive right turn lane is provided for Type 2 median crossover for deceleration and storage of vehicles. The length is of this lane is 230 feet. The distance of the Median U-Turn from the crossover is taken as 900 feet since 300 feet may be too short to accommodate the deceleration length for major road left turn movements, 600 feet may not provide adequate weaving distance and 1200 feet may be too long for the vehicles (making left turn from the crossroad) to travel.

Figure 4-1 shows the design option of Type 2 median crossover constructed in VISSIM with a median opening width of 60 feet and 12 feet lanes. Figure 4-2 shows the design option of Unsignalized Median U-Turn constructed in VISSIM with a U-turn at an assumed distance of 900 feet from the cross road. A yield sign is located at the Median U-Turn for the vehicles entering onto the major road from the U-turn. Figure 4-3 shows the design option of signalized Median U-Turn constructed in VISSIM with a two phase signal at the Median U-turn. There is a STOP sign located on the crossroad for the vehicles entering on to the major road.

FIGURE 4-1: Type 2 Median Crossover in VISSIM

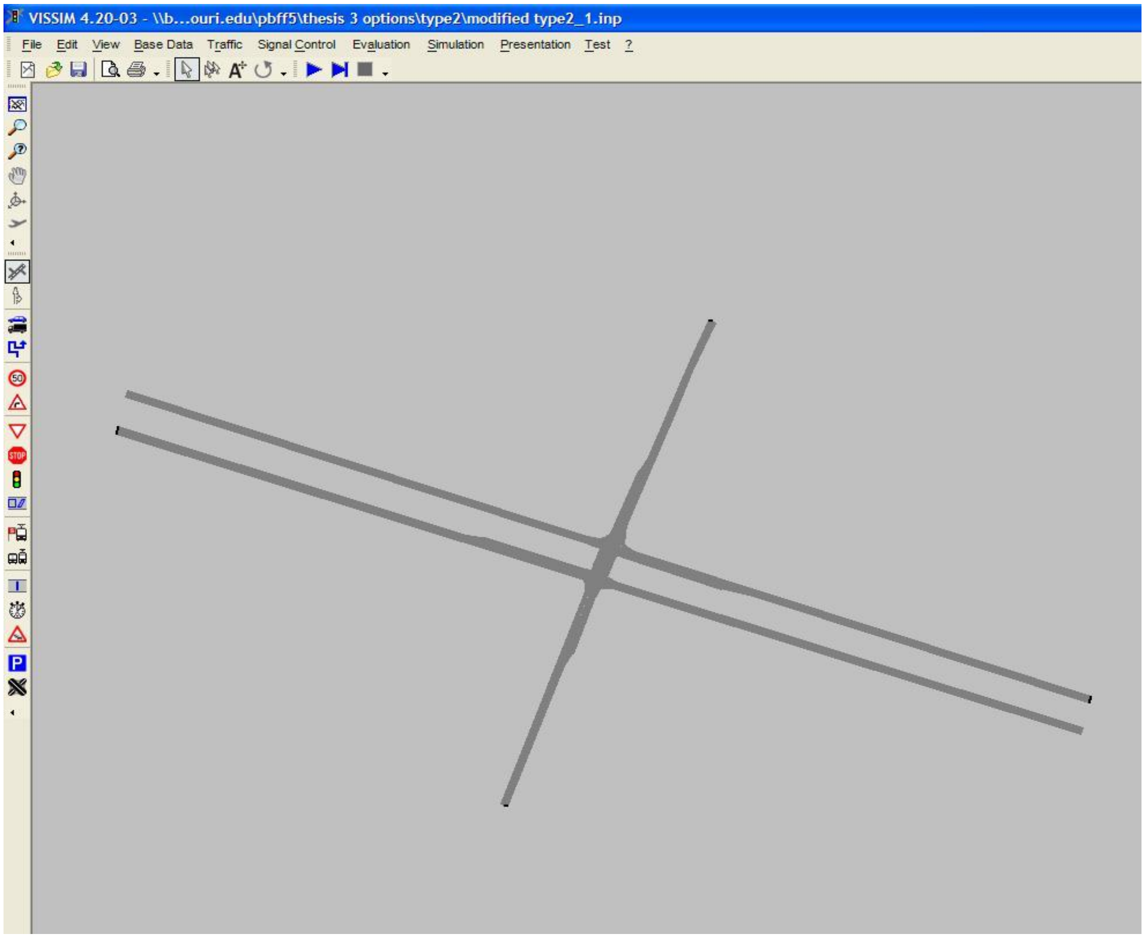


FIGURE 4-2: Unsignalized Median U-Turn in VISSIM

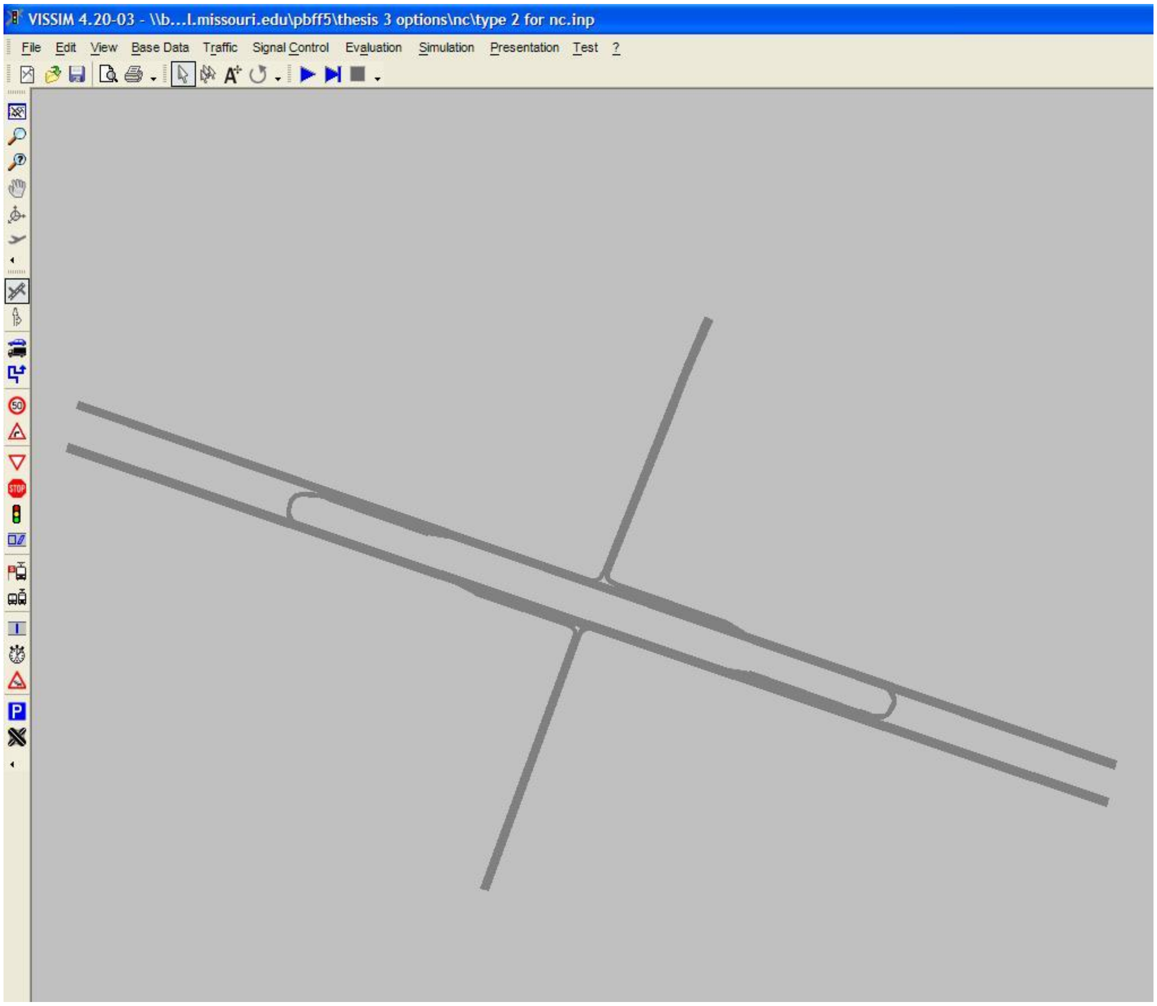
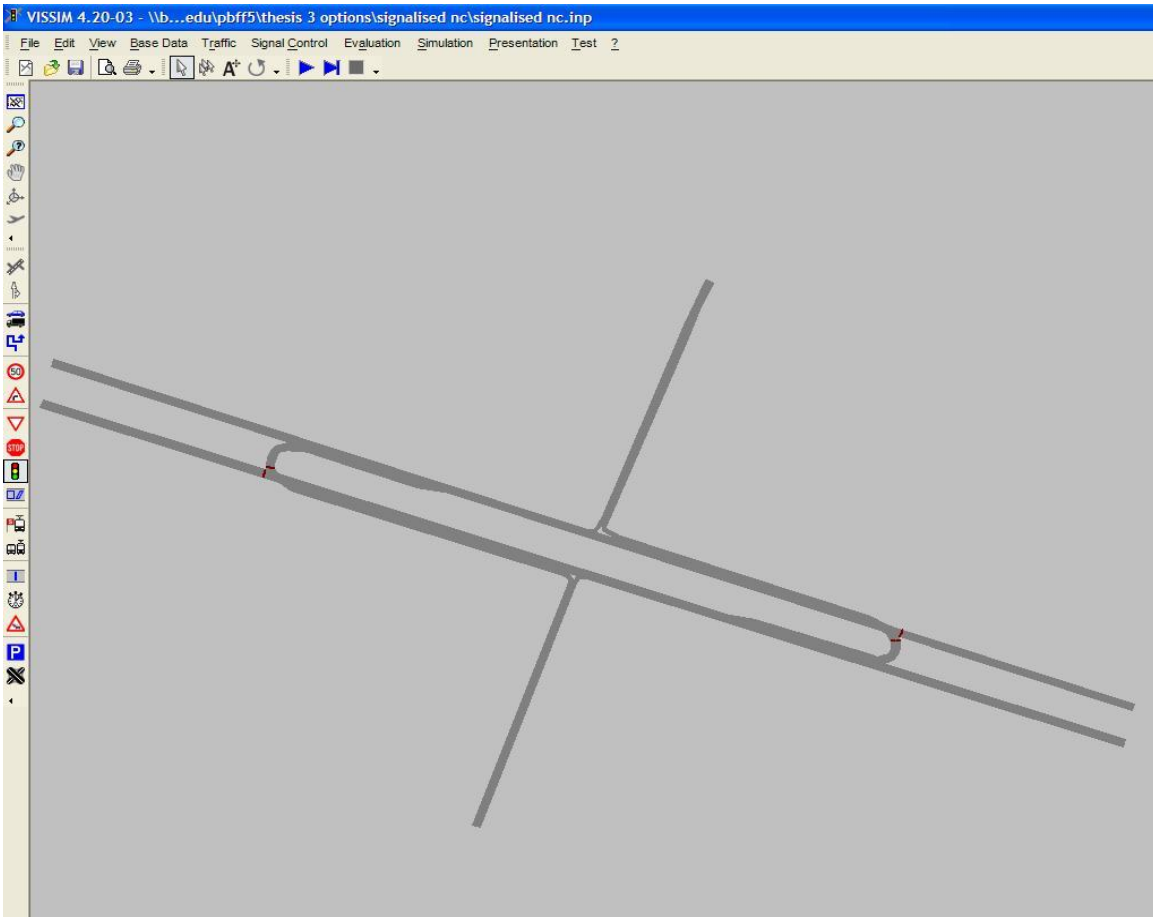


FIGURE 4-3: Signalized Median U-Turn in VISSIM



CHAPTER 5

VISSIM DATA ANALYSIS

VISSIM software tool is used to compare the three design options (i.e., Type 2 median crossover, unsignalized Median U-Turn and signalized Median U-Turn) as described in Table 5-1 in terms of travel time, delay time and queue lengths (maximum and average queue lengths). The main purpose of obtaining these performance measures is to check the maximum volume for the base alternative, first alternative and second alternative to perform well. The travel time, delay time and queue length are obtained for all the three design options and compared. The travel time and delay time are taken as weighted volume travel time and weighted volume delay time for the eight major movements that take the path of the Median U-turn.

Model	Description
Base alternative	Type 2 median crossover
First alternative	Unsignalized Median U-Turn
Second alternative	Signalized Median U-Turn

TABLE 5-1: Description of the three Design options

The major eight movements taken into consideration out of all the twelve movements are shown in Table 5-2 with asterisk.

Eight major movements (with *) considered out of the twelve movements that involve the Median U-Turn
*Northbound right turn (NBR)
*Northbound left turn (NBL)
*Northbound through (NBT)
Eastbound right turn (EBR)
*Eastbound left turn (EBL)
Eastbound through (EBT)
Westbound right turn (WBR)
*Westbound left turn (WBL)
Westbound through (WBT)
*Southbound right turn (SBR)
*Southbound left turn (SBL)
*Southbound through (SBT)

Table 5-2: Description of the eight major road movements considered

The main reason to consider only these eight movements is because they involve the Median U-Turn in their path which is considered as our alternative in the present study. The comparison of the performance measures for various volume combinations of the three design options are shown in Tables 5-3 to 5-12.

Table 5-3 shows the comparison of travel times, the shaded cell indicates the lowest travel time for each volume combination among the three design options. For lower one way peak hour volume of 1000 on the major road and with a one way peak hour crossroad volume of 100, 200, 300 and 400 Type 2 median crossover has the lower travel time. For medium one way peak hour volume of 2000 to 2500 of the major road with crossroad volume combination of 100 and 200, the unsignalized

TABLE 5-3: Comparison of weighted volume travel time in seconds of the three design options for the 8 major movements taken into consideration (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	60.2	73.3	83.1
2000	82.5	80.4	87.4
2100	95.1	81.3	89.0
2200	108.4	84.3	91.2
2300	119.6	85.1	93.2
2400	135.9	86.2	93.8
2500	148.5	87.9	94.7
3000	444.1	108.67	100.3
3500	959.2	265.82	118.7
4000	1299.90	527.74	135.26

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	61.9	76.4	85.4
2000	210.6	97.3	98.5
2100	266.0	96.2	102.9
2200	347.3	95.6	103.2
2300	420.3	102.9	106.6
2400	440.2	119.0	109.0
2500	489.9	126.7	116.7
3000	838.9	307.27	260.7
3500	1340.7	684.43	470.5
4000	1570.1	757.32	569.7

Cross road volume = 300

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	70.5	85.7	92.7
2000	363.1	196.3	149.6
2100	444.4	245.5	202.9
2200	509.1	288.6	232.9
2300	571.0	306.5	276.4
2400	650.6	343.5	278.2
2500	687.0	345.7	306.1
3000	1004.8	500.54	419.8
3500	1476.2	861.31	615.1
4000	1677.0	800.61	691.0

Cross road volume = 400

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	105.9	155.7	138.1
2000	472.8	330.2	306.0
2100	547.3	339.1	331.7
2200	570.9	355.2	335.1
2300	654.2	381.0	350.3
2400	708.3	384.0	365.3
2500	732.0	415.3	379.9
3000	1073.8	534.79	478.5
3500	1541.9	862.32	628.9
4000	1730.9	1034.89	690.8

Shading indicates the design option with lowest travel time for given major and minor road volume.

Median U-Turn has the lower travel times. For higher major road volumes i.e., 2500 to 4000 with crossroad volume combination of 100 and 200, the signalized Median U-Turn has the lower travel times. For the crossroad volumes of 300 and 400 the Type 2 median crossover has the lowest travel time for a major road volume of 1000 and beyond that volume of the major road i.e., from 2000 to 4000 and with the same cross road volumes of 300 and 400, the signalized Median U-Turns have the lowest travel times. As a conclusion, for lower volume combinations, Type 2 median crossover works well. For medium volume combinations unsignalized Median U-Turns works well and for higher volume combinations of the major and the minor road the signalized Median U-Turn works well.

Table 5-4 shows the comparison of delay times with the shaded cells indicating the lowest delay times for each volume combination among the three design options. From this table, for lower to medium volumes of the major road with a crossroad volume of 100 and 200 the unsignalized Median U-Turn experiences a lower delay time. As the crossroad volume increases to 300 and 400, the unsignalized Median U-Turn still experiences lower travel time for lower volume of 1000 on the major road. As the volume of the major road increases from 1000 to 4000 with a crossroad volume of 300 and 400, the signalized Median U-Turn experiences the lower delay time. In Highway Capacity Manual (HCM) [15], the acceptable delay (assumed to be LOS D) for unsignalized intersection is 35 seconds and for signalized intersection is 55 seconds.

TABLE 5-4: Comparison of weighted volume delay time in seconds of the three design options for the 8 major movements taken into consideration (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	21.4	14.7	24.7
2000	44.4	22.6	29.8
2100	56.8	23.6	31.4
2200	70.2	26.9	33.8
2300	81.5	27.6	36.0
2400	97.8	28.7	36.6
2500	110.6	30.6	37.5
3000	406.3	51.8	43.5
3500	921.6	209.2	62.1
4000	1262.36	471.4	79.07

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	22.6	17.29	26.3
2000	171.6	38.68	40.0
2100	227.0	37.56	44.4
2200	308.4	37.03	44.7
2300	381.5	44.41	48.3
2400	401.5	60.54	50.6
2500	451.1	68.19	58.4
3000	800.4	249.19	202.7
3500	1302.3	626.40	412.7
4000	1531.6	699.76	512.1

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	31.3	26.35	33.5
2000	323.8	137.40	90.7
2100	405.1	186.58	144.1
2200	469.7	229.76	174.0
2300	531.8	247.71	217.4
2400	611.6	284.62	219.4
2500	648.0	286.97	247.4
3000	966.0	441.99	361.3
3500	1437.4	802.80	556.9
4000	1638.0	742.43	632.8

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	66.4	96.26	78.8
2000	433.5	271.05	247.0
2100	507.8	279.89	272.7
2200	531.4	296.09	276.1
2300	614.9	322.00	291.4
2400	669.2	325.02	306.4
2500	692.8	356.29	321.0
3000	1034.7	475.98	419.7
3500	1503.0	803.49	570.3
4000	1691.7	976.35	632.2

Shading indicates the design option with lowest delay time for given major and minor road volume.

Tables 5-5 to 5-12 show the comparison of queue lengths, queue length on the eastbound indicate the queue formed on the eastbound Median U-Turn and the queue length on the westbound indicate the queue formed on the westbound Median U-Turn. Queue length on the northbound and southbound indicates the queue formed on the crossroads which is not of a major issue in the present study. The shaded cells in the queue lengths table indicate the length that that exceeds the minimum safe conditions of the traffic. From the Type 2 median crossover alternative created in VISSIM, there is an exclusive left turn lane near the median in eastbound (EB) and westbound (WB) direction with a length of 350 feet (as discussed in section 4.4.11). There may be a problem if the vehicles in this lane spill back on to the major road. The limiting length of the queue in EB and WB direction for Type 2 median crossover is 350 feet. Any value of the queue length beyond 350 feet for the base alternative is shaded in the table. For Median U-Turn, the U-turn length is taken as the storage length which is 175 feet for eastbound U-turn and 195 feet for westbound U-turn. The deceleration length is taken as 350 feet for the vehicles that take Median U-Turn. All the cells in the table for Median U-Turn (signalized or unsignalized) queue lengths with a value greater than 175 feet for EB and a value greater than 195 feet in WB direction are shaded indicating that they have exceeded the safe queue length values. For northbound and southbound directions there is no limiting value for safe queue lengths as these vehicles do not obstruct the path of any other vehicle. The values of the average and maximum queue lengths from Tables 5-5, 5-6, 5-9, 5-10 indicate that signalized Median U-Turns have the safe queue lengths for all the volume combinations in eastbound and westbound direction. The base alternative and the unsignalized Median U-Turn have the safe maximum queue lengths till the major volume is 3000 and safe average queue lengths till a major road volume of 3500 for all crossroad volumes combinations.

TABLE 5-5: Comparison of average queue lengths in feet of the three design options in eastbound (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	1	3.0
2000	0.0	3	4.0
2100	0.0	3	4.0
2200	0	4	4.0
2300	0.0	4	5.0
2400	0.0	4	5.0
2500	0.0	6	5.0
3000	0.0	14	4.0
3500	45.0	79	4.0
4000	127.0	637	3.0

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	1	7.0
2000	0.0	5	9.0
2100	0.0	8	9.0
2200	0.0	6	9.0
2300	0.0	7	10.0
2400	0.0	11	9.0
2500	0.0	9	10.0
3000	0.0	24	8.0
3500	45.0	144	5.0
4000	127.0	532	4.0

One way Cross road volume = 300

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	2	11.0
2000	0.0	9	14.0
2100	0.0	10	14.0
2200	0.0	11	14.0
2300	0.0	15	14.0
2400	0.0	11	13.0
2500	0.0	15	14.0
3000	0.0	25	10.0
3500	45.0	116	5.0
4000	127.0	722	4.0

One way cross road volume = 400

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	2	17.0
2000	0.0	9	16.0
2100	0.0	17	15.0
2200	0	14	15.0
2300	0	12	15.0
2400	0	14	16.0
2500	0	15	14.0
3000	0	26	9.0
3500	45	258	5.0
4000	127	512	5.0

Shading indicates the queue length that exceeds allowable limit of that particular design option for given major and minor road volume.

TABLE 5-6: Comparison of average queue lengths in feet of the three design options in westbound (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	0	4
2000	0.0	1	4
2100	0.0	1	3
2200	0.0	1	4
2300	0.0	2	5
2400	0.0	3	5
2500	0.0	2	4
3000	13.0	7	5
3500	187.0	29	5
4000	255.0	327	5

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	1	10
2000	0.0	3	10
2100	0.0	3	11
2200	0.0	5	10
2300	0.0	4	11
2400	0.0	6	12
2500	0.0	6	12
3000	9.0	14	9
3500	187.0	34	7
4000	255.0	500	6

One way Cross road volume = 300

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	1	12
2000	0.0	4	18
2100	0.0	4	17
2200	0.0	4	16
2300	0.0	5	16
2400	0.0	5	16
2500	0.0	5	15
3000	9.0	12	10
3500	187.0	32	7
4000	255.0	514	6

One way cross road volume = 400

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	1.0	15.0
2000	0.0	4.0	19.0
2100	0.0	6.0	16.0
2200	0.0	4.0	17.0
2300	0.0	6.0	15.0
2400	0.0	7.0	16.0
2500	0.0	9.0	16.0
3000	9.0	14.0	10.0
3500	187.0	31.0	7.0
4000	255.0	568.0	7.0

Shading indicates the queue length that exceeds allowable limit of that particular design option for given major and minor road volume.

TABLE 5-7: Comparison of average queue lengths in feet of the three design options in northbound (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	0	0.0
2000	7.0	1	1.0
2100	15.0	1	2.0
2200	28	1	2.0
2300	46.0	1	3.0
2400	62.0	2	2.0
2500	78.0	2	3.0
3000	399.0	6	7.0
3500	596.0	111	20.0
4000	604.0	366	41.0

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	2.0	2	3.0
2000	401.0	29	16.0
2100	497.0	20	25.0
2200	591.0	17	21.0
2300	662.0	33	26.0
2400	664.0	72	26.0
2500	660.0	85	43.0
3000	691.0	320	352.0
3500	696.0	601	563.0
4000	720.0	671	609.0

One way Cross road volume = 300

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	30.0	16	16.0
2000	690.0	401	134.0
2100	694.0	435	351.0
2200	723.0	513	396.0
2300	731.0	560	520.0
2400	737.0	597	495.0
2500	721.0	612	558.0
3000	744.0	682	643.0
3500	728.0	710	706.0
4000	753.0	728	712.0

One way cross road volume = 400

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	232.0	280	150.0
2000	731.0	648	661.0
2100	734.0	666	679.0
2200	742	684	655.0
2300	742	693	672.0
2400	751	677	682.0
2500	754	700	688.0
3000	770	725	707.0
3500	743	734	735.0
4000	768	747	738.0

TABLE 5-8: Comparison of average queue lengths in feet of the three design options in southbound (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	0	0
2000	1.0	0	0
2100	2.0	0	0
2200	4.0	3	0
2300	2.0	1	1
2400	3.0	0	0
2500	4.0	0	0
3000	71.0	6	1
3500	283.0	61	5
4000	848.0	314	10

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	1.0	1	1
2000	19.0	12	9
2100	23.0	15	13
2200	70.0	10	18
2300	32.0	20	13
2400	50.0	24	20
2500	138.0	36	24
3000	1089.0	434	231
3500	1227.0	622	528
4000	1010.0	655	543

One way Cross road volume = 300

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	7.0	24	20
2000	189.0	193	156
2100	401.0	403	236
2200	576.0	499	357
2300	903.0	493	444
2400	880.0	551	476
2500	860.0	522	488
3000	1355.0	636	646
3500	1335.0	712	656
4000	1050.0	721	693

One way cross road volume = 400

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	24.0	272.0	213.0
2000	705.0	658.0	610.0
2100	977.0	679.0	671.0
2200	1100.0	671.0	684.0
2300	1112.0	687.0	672.0
2400	1140.0	690.0	677.0
2500	1301.0	704.0	690.0
3000	1420.0	707.0	709.0
3500	1364.0	738.0	716.0
4000	1070.0	742.0	728.0

Tables 5-7 and 5-8 does not show any shaded region as NB and SB are not considered as critical movements whose queue effects any other movement.

TABLE 5-9: Comparison of maximum queue lengths in feet of the three design options in eastbound (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	44	63.0
2000	0.0	45	63.0
2100	0.0	68	62.0
2200	0	105	61.0
2300	0.0	61	62.0
2400	0.0	67	61.0
2500	29.0	90	62.0
3000	44.0	89	61.0
3500	178.0	235	61.0
4000	378.0	1674	63.0

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	42	66.0
2000	0.0	88	69.0
2100	0.0	102	66.0
2200	0.0	84	84.0
2300	0.0	92	104.0
2400	0.0	163	84.0
2500	29.0	104	65.0
3000	44.0	124	62.0
3500	178.0	378	62.0
4000	378.0	1674	64.0

One way Cross road volume = 300

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	82	83.0
2000	0.0	107	104.0
2100	0.0	105	86.0
2200	0.0	108	88.0
2300	0.0	186	87.0
2400	0.0	99	86.0
2500	29.0	113	68.0
3000	44.0	147	65.0
3500	178.0	347	44.0
4000	378.0	1674	61.0

One way cross road volume = 400

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	63	106.0
2000	0.0	131	129.0
2100	0.0	215	86.0
2200	0	112	107.0
2300	0	127	86.0
2400	0	133	88.0
2500	29	154	86.0
3000	44	134	64.0
3500	178	544	65.0
4000	378	1674	63.0

Shading indicates the queue length that exceeds allowable limit of that particular design option for given major and minor road volume.

TABLE 5-10: Comparison of maximum queue lengths in feet of the three design options in westbound (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	22	61
2000	0.0	44	66
2100	0.0	40	61
2200	0.0	44	65
2300	0.0	62	80
2400	0.0	86	63
2500	29.0	42	59
3000	89.0	83	62
3500	419.0	166	64
4000	1235.0	1674	61

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	39	81
2000	0.0	62	105
2100	0.0	66	86
2200	0.0	87	81
2300	0.0	92	80
2400	0.0	87	101
2500	29.0	94	84
3000	73.0	106	63
3500	419.0	177	81
4000	1235.0	1673	80

One way Cross road volume = 300

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	44	86
2000	0.0	89	102
2100	0.0	86	87
2200	0.0	82	124
2300	0.0	79	86
2400	0.0	63	83
2500	29.0	59	123
3000	73.0	134	84
3500	419.0	148	65
4000	1235.0	1674	82

One way cross road volume = 400

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	0.0	65.0	101.0
2000	0.0	85.0	101.0
2100	0.0	112.0	86.0
2200	0.0	60.0	87.0
2300	0.0	65.0	86.0
2400	0.0	67.0	103.0
2500	29.0	101.0	123.0
3000	73.0	129.0	86.0
3500	419.0	152.0	65.0
4000	1235.0	1674.0	82.0

Shading indicates the queue length that exceeds allowable limit of that particular design option for given major and minor road volume.

TABLE 5-11: Comparison of maximum queue lengths in feet of the three design options in northbound (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	96.0	63	63.0
2000	141.0	46	68.0
2100	183.0	68	68.0
2200	227	69	68.0
2300	229.0	68	90.0
2400	240.0	68	68.0
2500	250.0	68	68.0
3000	1012.0	77	113.0
3500	869.0	301	155.0
4000	890.0	813	221.0

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	101.0	90	86.0
2000	905.0	175	178.0
2100	920.0	199	200.0
2200	949.0	152	180.0
2300	909.0	217	171.0
2400	921.0	371	222.0
2500	905.0	331	218.0
3000	977.0	813	661.0
3500	869.0	831	831.0
4000	891.0	831	832.0

One way Cross road volume = 300

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	323.0	178	175.0
2000	948.0	817	410.0
2100	920.0	831	705.0
2200	922.0	823	813.0
2300	947.0	832	814.0
2400	965.0	832	813.0
2500	903.0	831	817.0
3000	1000.0	832	823.0
3500	870.0	832	832.0
4000	888.0	831	831.0

One way cross road volume = 400

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	1068.0	677	416.0
2000	903.0	831	831.0
2100	948.0	818	831.0
2200	922	831	831.0
2300	929	831	831.0
2400	992	832	831.0
2500	992	831	831.0
3000	1020	832	831.0
3500	870	831	832.0
4000	888	831	831.0

TABLE 5-12: Comparison of maximum queue lengths in feet of the three design options in southbound (one way major road peak hour volume)

One way Cross road volume = 100

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	73.0	31	31
2000	92.0	31	31
2100	112.0	31	42
2200	117.0	128	43
2300	119.0	64	48
2400	107.0	39	37
2500	118.0	52	43
3000	305.0	101	46
3500	905.0	346	148
4000	1630.0	790	127

One way cross road volume = 200

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	109.0	91	90
2000	194.0	176	177
2100	227.0	155	185
2200	436.0	118	187
2300	248.0	216	172
2400	298.0	238	140
2500	577.0	229	208
3000	1645.0	820	496
3500	1653.0	821	821
4000	1630.0	821	818

One way Cross road volume = 300

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	232.0	305	257
2000	806.0	740	489
2100	1312.0	821	678
2200	1643.0	821	815
2300	1667.0	821	812
2400	1668.0	821	821
2500	1668.0	821	814
3000	1651.0	821	821
3500	1653.0	821	821
4000	1630.0	821	814

One way cross road volume = 400

Major road volume	Type2 median crossover	Unsignalized median U-turn	Signalized median U-turn
1000	370.0	792.0	645.0
2000	1083.0	821.0	821.0
2100	1667.0	821.0	821.0
2200	1667.0	821.0	821.0
2300	1658.0	821.0	821.0
2400	1668.0	821.0	821.0
2500	1667.0	821.0	821.0
3000	1652.0	821.0	820.0
3500	1653.0	820.0	821.0
4000	1630.0	821.0	820.0

Tables 5-11 and 5-12 does not show any shaded region as NB and SB are not considered as critical movements whose queue effects any other movement.

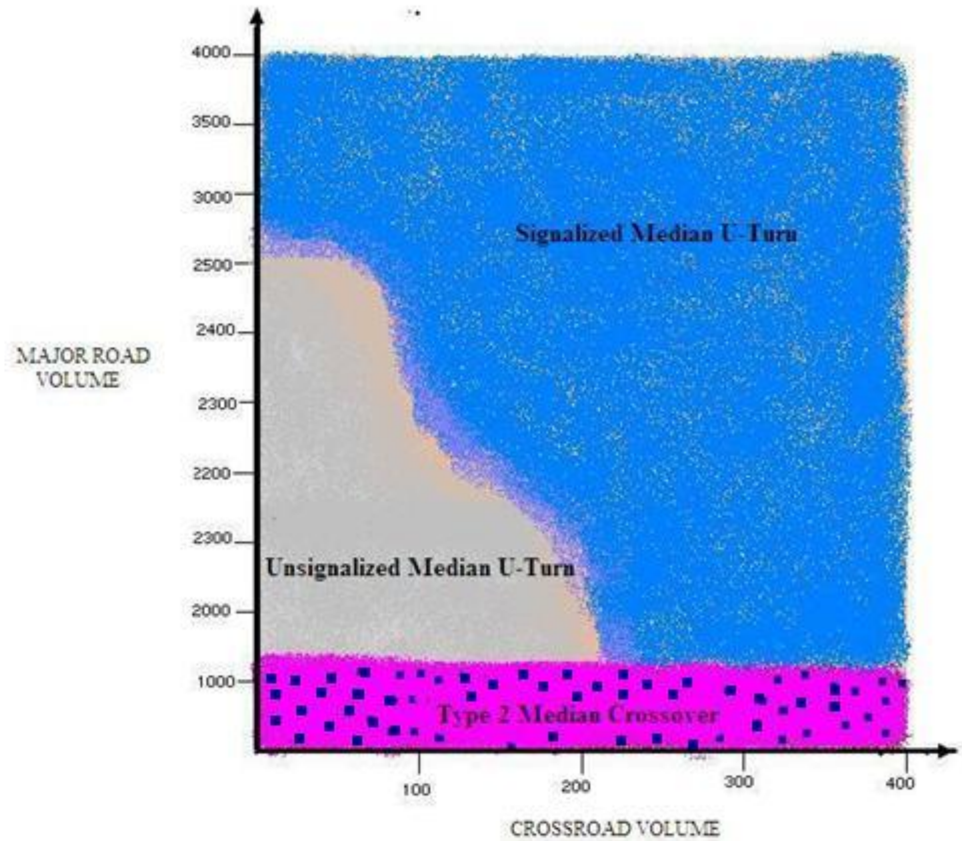


FIGURE 5-1: Sketch of design alternatives versus volume combinations

For lower major road volumes Type 2 median crossover performs well, for medium major road volumes unsignalized Median U-Turn performs well and for higher major road volumes signalized Median U-Turn performs well. In the three design options, the average and the maximum queue lengths for eastbound and westbound left turn lanes exceeded the safe limit for higher major road volumes. The above discussion is summarized in Figure 5-1. The boundaries between preferred designs are somewhat fuzzy because the analyses were based upon assumed turning proportion of volumes.

Table 5-13 shows the variation in the delay time values for the three design options when used in two different software. In HCS, the delay for the Median U-Turn is taken as the sum of the delays caused at the first T-legged intersection which is considered as crossroad to the major road intersection and the second T-legged intersection which is considered as the Median U-Turn to the major road intersection. In VISSIM the delay is calculated for the section that is considered including the dwell time and the waiting time at the intersection. HCS can calculate the delay for the intersection alone, unlike VISSIM which calculates for the whole section under consideration. This accounts for the variation in the delays for the same volume combination in two different software as shown in Table 5-13.

TABLE 5-13: Comparison of the delay times obtained from HCS and VISSIM for all the eight major movements weighted by volume (for one way major road peak hour volume)

One way peak hour cross road volume = 100

Major road volume	Type2(VISSIM)	Type2(HCS)	Unsignalized median U-turn(VISSIM)	Unsignalized median U-turn(HCS)	Signalized median U-turn(VISSIM)	Signalized median U-turn(HCS)
1000	21.4	17.1	14.7	25.1	24.7	17.2
2000	44.4	99.0	22.6	45.3	29.8	31.2
2100	56.8	141.8	23.6	48.9	31.4	34.4
2200	70.2	219.8	26.9	53.1	33.8	38.3
2300	81.5	332.8	27.6	58.1	36.0	43.7
2400	97.8	514.5	28.7	63.5	36.6	52.6
2500	110.6	698.3	30.6	70.0	37.5	74.7
3000	406.3	*	51.8	130.0	43.5	425.3
3500	921.6	*	209.2	369.6	62.1	920.7
4000	1262.36	*	471.4	1362.0	79.07	1802.3

*Indicates that the delay results in HCM are not reliable when V/C >1

One way peak hour cross road volume = 200

Major road volume	Type2(VISSIM)	Type2(HCS)	Unsignalized median U-turn(VISSIM)	Unsignalized median U-turn(HCS)	Signalized median U-turn(VISSIM)	Signalized median U-turn(HCS)
1000	22.6	21.1	17.29	28.20	26.3	21
2000	171.6	619.1	38.68	69.70	40.0	52.5
2100	227.0	877.0	37.56	81.60	44.4	62.9
2200	308.4	1278.8	37.03	97.40	44.7	77.3
2300	381.5	1986.6	44.41	120.40	48.3	99.2
2400	401.5	3266.2	60.54	157.10	50.6	137
2500	451.1	17329.0	68.19	209.50	58.4	202.2
3000	800.4	*	249.19	849.20	202.7	1078.9
3500	1302.3	*	626.40	2281.00	412.7	2400.9
4000	1531.6	*	699.76	4723.00	512.1	4174

One way peak hour cross road volume = 300

Major road volume	Type2(VISSIM)	Type2(HCS)	Unsignalized median U-turn(VISSIM)	Unsignalized median U-turn(HCS)	Signalized median U-turn(VISSIM)	Signalized median U-turn(HCS)
1000	31.3	29.8	26.35	33.50	33.5	27.2
2000	323.8	2719.1	137.40	263.80	90.7	242.1
2100	405.1	6964.4	186.58	371.80	144.1	346.7
2200	469.7	*	229.76	501.60	174.0	472.3
2300	531.8	*	247.71	649.70	217.4	615.3
2400	611.6	*	284.62	814.80	219.4	775.6
2500	648.0	*	286.97	996.00	247.4	960.8
3000	966.0	*	441.99	2346.00	361.3	2322.3
3500	1437.4	*	802.80	4805.00	556.9	4225.9
4000	1638.0	*	742.43	8362.00	632.8	6753.6

*Indicates that the delay results in HCM are not reliable when V/C >1

One way peak cross road volume = 400

Major road volume	Type2(VISSIM)	Type2(HCS)	Unsignalized median U-turn(VISSIM)	Unsignalized median U-turn(HCS)	Signalized median U-turn(VISSIM)	Signalized median U-turn(HCS)
1000	66.4	69.9	96.26	45.80	78.8	40.1
2000	433.5	*	271.05	891.40	247.0	865.7
2100	507.8	*	279.89	1078.00	272.7	1044.4
2200	531.4	*	296.09	1273.00	276.1	1226.9
2300	614.9	*	322.00	1500.00	291.4	1436.5
2400	669.2	*	325.02	1758.00	306.4	1666.1
2500	692.8	*	356.29	2028.00	321.0	1904.1
3000	1034.7	*	475.98	4134.00	419.7	3654.8
3500	1503.0	*	803.49	7426.00	570.3	6108.9
4000	1691.7	*	976.35	12027.00	632.2	9352.6

*Indicates that the delay results in HCM are not reliable when V/C >1

From this chapter, it can be concluded that from Tables 5-3 to 5-12 that, for lower volumes Type 2 median crossover performs better, for medium volumes unsignalized Median U-Turn performs better and for higher volumes signalized Median U-Turns performs better in terms of its performance measures. From the comparison table of delay obtained from HCS and VISSIM it is observed that the two different software calculate the delay in two different ways and hence the values of delay vary for the same volume combinations when used in these two software. For higher volume combination, HCS does not show any value of delay as V/C ratio is greater than one.

CHAPTER 6

COST ESTIMATES

This chapter describes the cost estimate analysis for Type 2 median cross over, unsignalized Median U-Turn and signalized Median U-Turn. It provides general guidelines for the planner to select the most desirable alternative in the project development stage of the planning process. This cost estimate analysis presents two scenarios. The first scenario includes the cost estimates for construction of a new Type 2 median crossover, a new unsignalized Median U-Turn and a new signalized Median U-Turn. The second scenario includes the additional amount of cost required to convert an existing Type 2 median crossover to unsignalized Median U-Turn, to convert an unsignalized Median U-Turn to signalized Median U-Turn and to convert a Type 2 median crossover to signalized Median U-Turn as shown in Tables 6-2 and 6-3 . For all the three design cases under second scenario an additional cost of 20% to the total cost has been assumed to account for contingency since the existing design options are being converted to a new design option.

The statewide costs for the pay items described in Table 6-1 are obtained from MoDOT (2007 Bid prices). There are two stop signs used for Type 2 median crossover, two stop signs for the unsignalized Median U-Turn and two stop signs for the signalized Median U-Turn. There are four yield signs used for Type 2 median crossover and two yield signs considered for unsignalized Median U-Turn. There are two guide signs used for unsignalized Median U-Turn and two guide signs for signalized Median U-Turn to represent the Median U-Turn ahead. The cost for each signal installation is assumed to be

$150,000 and the maintenance cost for these signals is assumed to be $20,000 per year. Breakaway assembly will be placed at each of these signs. So, a total of six breakaway assemblies are assumed for Type 2 median crossover, eight for unsignalized Median U-Turn and six for signalized Median U-Turn.

TABLE 6-1: MoDOT 2007 Bid prices

Pay Item	Description	Unit	Unit cost
201-30.00	Clearing and Grubbing	ACRE	$2,705.23
201-10.00	Class A excavation	CUYD	$3.15
304-01.43	Type 1 aggregate for base (4 in. thick)	SQYD	$5.18
310-50.02	Gravel (A) or crushed stone (B)	TONS	$22.84
403-01.16	AC PG 76-22 (SP 125B mix)	TONS	$63.31
620-59.00	4 inch white high build acrylic waterborne pavement marking paint	LIN. FT.	$0.14
620-59.01	4 inch yellow high build acrylic waterborne pavement marking paint	LIN. FT.	$0.15
903-12.40	Breakaway assembly	EACH	$77.52
903-50.04	Type SHR2L-1 sign	SQFT	$16.82
903-50.09	36" stop sign	EACH	$138.45
MODOT	Signals	EACH	150,000

All the calculations are based upon the following assumptions:

- Depth of the excavation equals 1.5 feet.
- Pavement structure consists of 10 inches AC, 4 inches granular base, 12 inches gravel sub-base.

TABLE 6-2: Scenario one - Construction costs for a new Type 2 median crossover and its alternatives

Pay item	Unit	Type 2	Unsignalized U-turn	Signalized U-turn
201-30.00	ACRE	$45,730	$47,636	$48,401
201-10.00	CUYD	$42,923	$44,093	$46,403
304-01.43	SQYD	$141,168	$145,017	$152,614
310-50.02	TONS	$280,101	$287,738	$302,813
403-01.16	TONS	$411,203	$422,472	$444,605
620-59.00	LIN. FT.	$1,646	$1,654	$1,692
620-59.01	LIN. FT.	$1,040	$1,167	$1,167
903-12.40	EACH	$465	$620	$465
903-50.04	SQFT	$639	$1,144	$908
903-50.09	EACH	$554	$277	$277
MODOT	EACH	$0	$0	$300,000
	Total cost	**$925,467**	**$951,818**	**$1,299,345**

TABLE 6-3: Scenario two - Additional construction cost incurred to convert the existing Type 2 median crossover to its alternatives

Pay item	Unit	Type 2 to unsignalized U-turn	Unsignalized U-turn to signalized U-turn	Type 2 to signalized U-turn
216-15.01	SQFT	$25,626	$3,146	$25,626
201-10.00	CUYD	$4,550	$2,695	$7,245
304-01.43	SQYD	$14,965	$8,863	$23,829
310-50.02	TONS	$29,692	$17,587	$47,279
403-01.16	TONS	$43,596	$25,810	$69,407
620-59.00	LIN. FT.	$357	$193	$550
MODOT	EACH	$0	$300,000	$300,000
	Total cost =	**$118,786**	**$358,295**	**$473,935**
	Total cost+20% of total cost =	**$142,543**	**$429,954**	**$568,722**

The user cost includes the travel time cost for all the twelve movements. For the calculation of the average daily traffic (ADT), two flow conditions (i.e., light and heavy flow conditions) have been assumed. It is assumed that heavy flow conditions exist for three hours a day and light flow condition exist for 21 hours a day. 30 percent of a day's traffic is assumed for heavy flow (peak hour) and 70 percent of a day's traffic is assumed for light flows (non peak hour). The light flow conditions are taken for a crossroad volume of 100 and for various major road volumes and the heavy flow conditions are taken for a crossroad volume of 400 for various major road volumes as shown in tables 6-4 and 6-5. According to the U.S Department of Transportation, the recommended hourly value of travel time savings per person-hour for all purposes including personal and business is $17.94 [19, 20]. The average vehicle occupancy is assumed to be 1.3 persons. Since the travel times are obtained for peak hour in VISSIM they are multiplied by the number of hours a day and number of days in a year to get average annual travel time cost. Travel time in VISSIM includes the running time of the vehicle and the delay caused due to its waiting time at the intersections and delay due to the waiting time in queue condition. So, the user cost included the travel time cost alone.

Tables 6-4 and 6-5 show the travel time cost for peak and the non peak hour traffic. Tables 6-6 show the total travel time cost per day. Graphical representation of the total travel time cost is shown in Figure 6-1. In the graph, only the major road volumes are shown for easy representation but all the twelve movements' volume and their corresponding travel times for peak hour and non peak hour have been considered to get the travel time cost per day.

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Travel time cost = $17.94 per person hour travel

Average occupancy = 1.3 persons

TABLE 6-4: Travel time cost for non peak hour traffic

Cross road vol= 100

Major road volume	Type 2	Unsignalized U-Turn	Signalized U-Turn
1000	$3,806	$3,977	$4,439
2000	$7,488	$7,644	$8,780
2100	$7,963	$8,017	$9,154
2200	$8,448	$8,446	$9,699
2300	$8,944	$8,907	$10,175
2400	$9,456	$9,265	$10,589
2500	$10,032	$9,736	$11,136
3000	$15,377	$12,506	$14,107
3500	$24,099	$18,315	$18,711
4000	$30,547	$31,443	$24,298

TABLE 6-5: Travel time cost for peak hour traffic

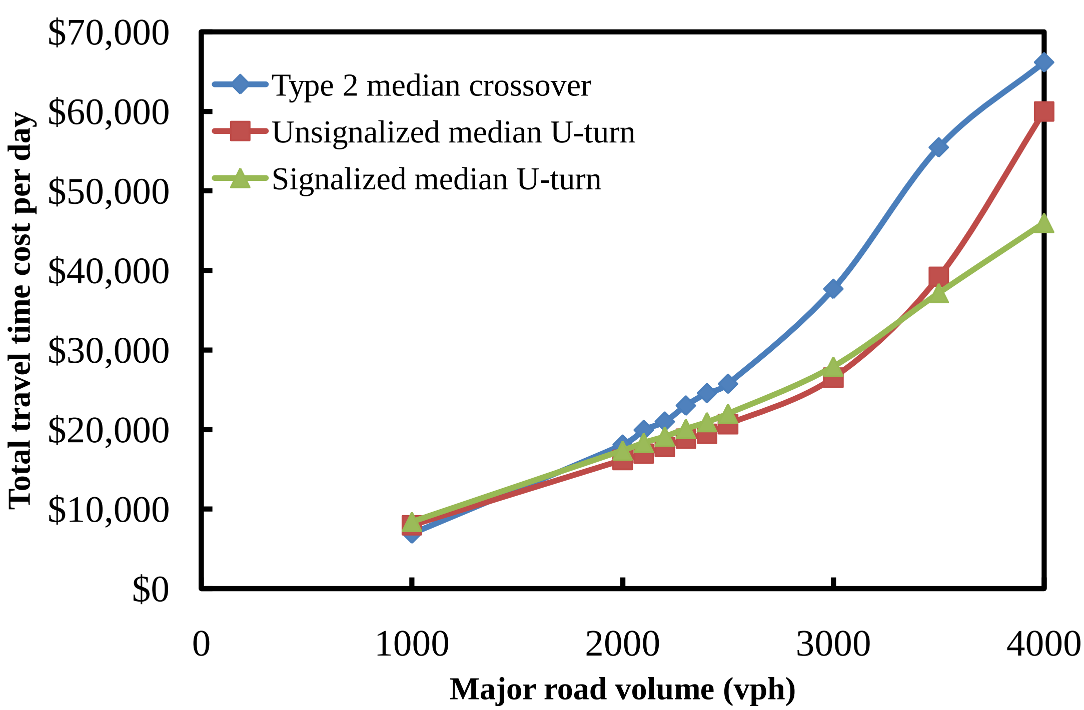
Cross road vol= 400

Major road volume	Type 2	Unsignalized U-Turn	Signalized U-Turn
1000	$3,104	$3,989	$3,954
2000	$10,589	$8,533	$8,587
2100	$11,966	$8,955	$9,181
2200	$12,533	$9,349	$9,425
2300	$14,057	$9,953	$9,908
2400	$15,113	$10,194	$10,344
2500	$15,703	$10,946	$10,855
3000	$22,299	$14,001	$13,823
3500	$31,373	$20,876	$18,455
4000	$35,621	$28,530	$21,698

TABLE 6-6: Total travel time cost per day

Major road volume	Type 2	Unsignalized U-Turn	Signalized U-Turn
1000	$6,909	$7,966	$8,394
2000	$18,078	$16,177	$17,367
2100	$19,929	$16,972	$18,335
2200	$20,981	$17,796	$19,125
2300	$23,001	$18,860	$20,082
2400	$24,569	$19,459	$20,934
2500	$25,735	$20,682	$21,990
3000	$37,676	$26,507	$27,930
3500	$55,471	$39,191	$37,166
4000	$66,169	$59,972	$45,996

FIGURE 6-1: Total travel time costs per day for various major road volumes



The annual equivalent (AE) [22] worth criterion provides a basis for measuring investment worth by determining equal payments on an annual basis. For engineering purposes, a discount rate of 3 to 4% is often assumed with no inflation for each year. In the present case the useful life is assumed to be 20 years with a discount rate of 3%.

$$\mathrm{AE}\ (3\%) = \left(\frac{\mathrm{usercosts}}{\mathrm{year}}\right) + \left(\frac{\mathrm{A}}{\mathrm{P}}, 3\%, 20\mathrm{yrs}\right)(\mathrm{P}) \qquad \ldots\ldots\ldots (6.1)$$

Where,

$$\left(\frac{\mathrm{A}}{\mathrm{P}}, 3\%, 20\mathrm{yrs}\right) = \frac{\mathrm{i}(1+\mathrm{i})^{\mathrm{n}}}{(1+\mathrm{i})^{\mathrm{n}} - 1}$$

$\mathrm{P} = \text{capital cost or construction cost}$

$\mathrm{AE} = \text{Annual equivalent}$

The annual equivalent for all the major road volumes for scenario one are obtained for Type 2 median crossover, unsignalized Median U-Turn and signalized Median U-Turn as shown in Tables 6-7, 6-8 and 6-9. The annual equivalent for all the major road volumes of scenario two for the conversion of the existing Type 2 median crossover to unsignalized Median U-Turn which is assumed as case one, the existing unsignalized Median U-Turn to signalized Median U-Turn which is assumed as case two and the existing Type 2 median crossover to signalized Median U-Turn which is assumed as case three are shown in Tables 6-10, 6-11 and 6-12.

Type 2 median crossover:

Capital cost = $926,000

Life time = 20 years

Factor for capital cost = $\left(\frac{A}{P}, 3\%, 20\text{yrs}\right)$ (P) = $62,242

TABLE 6-7: Annual equivalent cost for Type 2 median crossover

Volumes	User cost Per year $	AE
1000	$2,521,941	$2,584,183
2000	$6,598,329	$6,660,571
2100	$7,274,069	$7,336,311
2200	$7,658,042	$7,720,284
2300	$8,395,447	$8,457,689
2400	$8,967,719	$9,029,961
2500	$9,393,275	$9,455,517
3000	$13,751,737	$13,813,979
3500	$20,246,938	$20,309,180
4000	$24,151,604	$24,213,846

Unsignalized Median U-Turn:

Capital cost = $952,000

Useful life = 20 years

Factor for capital cost = $\left(\frac{A}{P}, 3\%, 20\text{yrs}\right)$ (P) = $63,990

TABLE 6-8: Annual equivalent cost for Unsignalized Median U-Turn

Volumes	User cost Per year $	AE
1000	$2,907,559	$2,971,549
2000	$5,904,695	$5,968,685
2100	$6,194,837	$6,258,827
2200	$6,495,427	$6,559,417
2300	$6,884,036	$6,948,026
2400	$7,102,445	$7,166,435
2500	$7,548,928	$7,612,918
3000	$9,675,112	$9,739,102
3500	$14,304,786	$14,368,776
4000	$21,889,959	$21,953,949

Signalized Median U-Turn:

Capital cost = $1,300,000

Useful life = 20 years

Factor for capital cost = $\left(\frac{A}{P}, 3\%, 20\text{yrs}\right)$ (P) = $87,381

User cost includes maintenance cost of $20,000

TABLE 6-9: Annual equivalent cost for signalized Median U-Turn

Volumes	User cost Per year $	AE
1000	$3,083,634	$3,171,015
2000	$6,358,993	$6,446,374
2100	$6,712,152	$6,799,533
2200	$7,000,493	$7,087,874
2300	$7,350,097	$7,437,478
2400	$7,660,734	$7,748,115
2500	$8,046,391	$8,133,772
3000	$10,214,355	$10,301,736
3500	$13,585,641	$13,673,022
4000	$16,808,504	$16,895,885

Case one:

Capital cost = $143,000

Life time = 20 years

Factor for capital cost = $\left(\frac{A}{P}, 3\%, 20\text{yrs}\right)$ (P) = $9,612

TABLE 6-10: Annual equivalent cost for Case One

Volumes	User cost Per year $	AE
1000	$2,907,559	$2,917,171
2000	$5,904,695	$5,914,307
2100	$6,194,837	$6,204,449
2200	$6,495,427	$6,505,039
2300	$6,884,036	$6,893,648
2400	$7,102,445	$7,112,057
2500	$7,548,928	$7,558,540
3000	$9,675,112	$9,684,724
3500	$14,304,786	$14,314,398
4000	$21,889,959	$21,899,571

Case two:

Capital cost = $430,000

Useful life = 20 years

Factor for capital cost = $\left(\frac{A}{P}, 3\%, 20\text{yrs}\right)$ (P) = $28,903

TABLE 6-11: Annual equivalent cost for Case Two

Volumes	User cost Per year $	AE
1000	$3,063,634	$3,092,537
2000	$6,338,993	$6,367,896
2100	$6,692,152	$6,721,055
2200	$6,980,493	$7,009,396
2300	$7,330,097	$7,359,000
2400	$7,640,734	$7,669,637
2500	$8,026,391	$8,055,294
3000	$10,194,355	$10,223,258
3500	$13,565,641	$13,594,544
4000	$16,788,504	$16,817,407

Case three:

Capital cost = $569,000

Useful life = 20 years

Factor for capital cost = $\left(\frac{A}{P}, 3\%, 20\text{yrs}\right)$ (P) = $38,246

User cost includes maintenance cost of $20,000

TABLE 6-12: Annual equivalent cost for Case Three

Volumes	User cost Per year $	AE
1000	$3,063,634	$3,101,880
2000	$6,338,993	$6,377,239
2100	$6,692,152	$6,730,398
2200	$6,980,493	$7,018,739
2300	$7,330,097	$7,368,343
2400	$7,640,734	$7,678,980
2500	$8,026,391	$8,064,637
3000	$10,194,355	$10,232,601
3500	$13,565,641	$13,603,887
4000	$16,788,504	$16,826,750

TABLE 6-13: Annual equivalent cost comparison for all the three design options for scenario one

Major road volumes (one way peak hour volume)	Type 2 median crossover	Unsignalized Median U-Turn	Signalized Median U-Turn
	AE	AE	AE
1000	$2,584,183	$2,971,549	$3,171,015
2000	$6,660,571	$5,968,685	$6,446,374
2100	$7,336,311	$6,258,827	$6,799,533
2200	$7,720,284	$6,559,417	$7,087,874
2300	$8,457,689	$6,948,026	$7,437,478
2400	$9,029,961	$7,166,435	$7,748,115
2500	$9,455,517	$7,612,918	$8,133,772
3000	$13,813,979	$9,739,102	$10,301,736
3500	$20,309,180	$14,368,776	$13,673,022
4000	$24,213,846	$21,953,949	$16,895,885

Shaded area in Tables 6-13 and 6-14 indicate the lower cost among the three design options for that particular volume combination.

TABLE 6-14: Annual equivalent cost comparison for all the three cases in scenario two

Major road volumes (one way peak hour volume)	Type 2 to unsignalized median U-turn	Unsignalized to signalized Median U-Turn	Type 2 to signalized Median U-Turn
	AE	AE	AE
1000	$2,917,171	$3,092,537	$3,101,880
2000	$5,914,307	$6,367,896	$6,377,239
2100	$6,204,449	$6,721,055	$6,730,398
2200	$6,505,039	$7,009,396	$7,018,739
2300	$6,893,648	$7,359,000	$7,368,343
2400	$7,112,057	$7,669,637	$7,678,980
2500	$7,558,540	$8,055,294	$8,064,637
3000	$9,684,724	$10,223,258	$10,232,601
3500	$14,314,398	$13,594,544	$13,603,887
4000	$21,899,571	$16,817,407	$16,826,750

Tables 6-13 and 6-14 shows the comparison of Annual Equivalent for the three options under scenario one and scenario two. From table 6-13, for major road volume of 1000, the type 2 median crossover has the lowest annual equivalent cost. For major road volumes from 2000 to 3000, the unsignalized Median U-Turn has lower annual equivalent cost. For higher major road volumes of 3500 and 4000, the signalized Median U-Turn has the lowest annual equivalent cost. From table 6-14, for major road volume of 1000 to 3000, case one has the lowest annual equivalent cost. For higher major road volumes of 3500 and 4000, case two has lower annual equivalent costs. Table 6-15 shows the annual equivalent cost comparison of case one to the user costs of type 2 median crossover, case two to user costs of unsignalized Median U-Turn and comparison of case three to user costs of Type 2 median crossover. All the costs in Table 6-15 are shown in million dollars.

TABLE 6-15: Comparison of annual equivalent costs of all the three cases and their respective user costs

Major road Volumes	Case one AE	Type2 median crossover user cost	Case two AE	Unsignalized U-Turn user cost	Case three AE	Type 2 median crossover user cost
1000	$2.917	$2.522	$3.093	$2.908	$3.102	$2.522
2000	$5.914	$6.598	$6.368	$5.905	$6.377	$6.598
2100	$6.204	$7.274	$6.721	$6.195	$6.730	$7.274
2200	$6.505	$7.658	$7.009	$6.495	$7.019	$7.658
2300	$6.894	$8.395	$7.359	$6.884	$7.368	$8.395
2400	$7.112	$8.968	$7.670	$7.102	$7.679	$8.968
2500	$7.559	$9.393	$8.055	$7.549	$8.065	$9.393
3000	$9.685	$13.752	$10.223	$9.675	$10.233	$13.752
3500	$14.314	$20.247	$13.595	$14.305	$13.604	$20.247
4000	$21.900	$24.152	$16.817	$21.890	$16.827	$24.152

All the costs in this table are in million dollars.

Figure 6-2 shows the comparison of annual equivalent costs for all the three design options under scenario one. Figure 6-3 to 6-5 shows the comparison of the annual equivalent costs of all the three cases with the user costs of existing design options respectively.

FIGURE 6-2: Annual equivalent cost comparison of all the three design options for scenario one

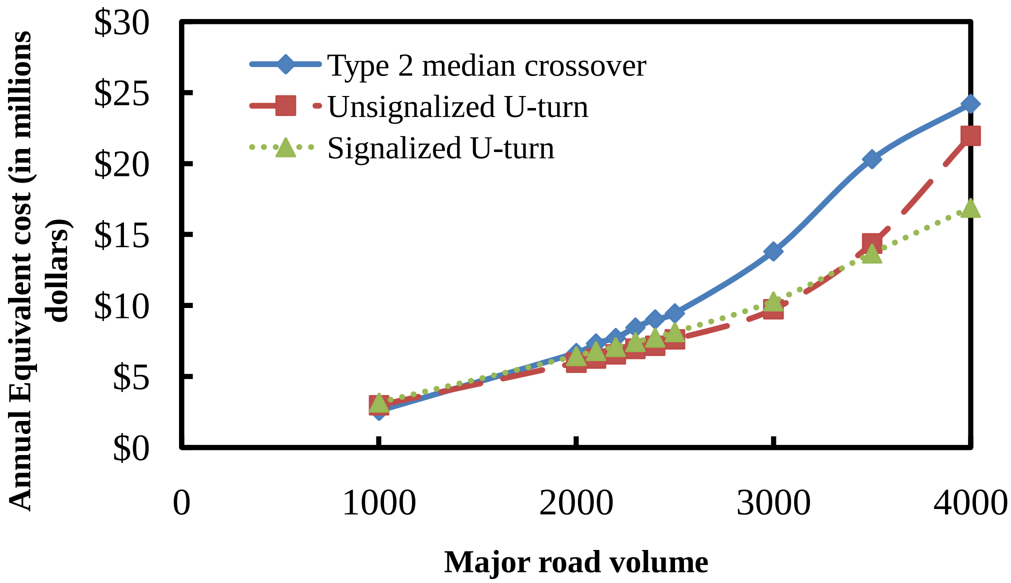


Figure 6-3: Comparison of annual equivalent costs of case one to user costs of Type 2 median crossover

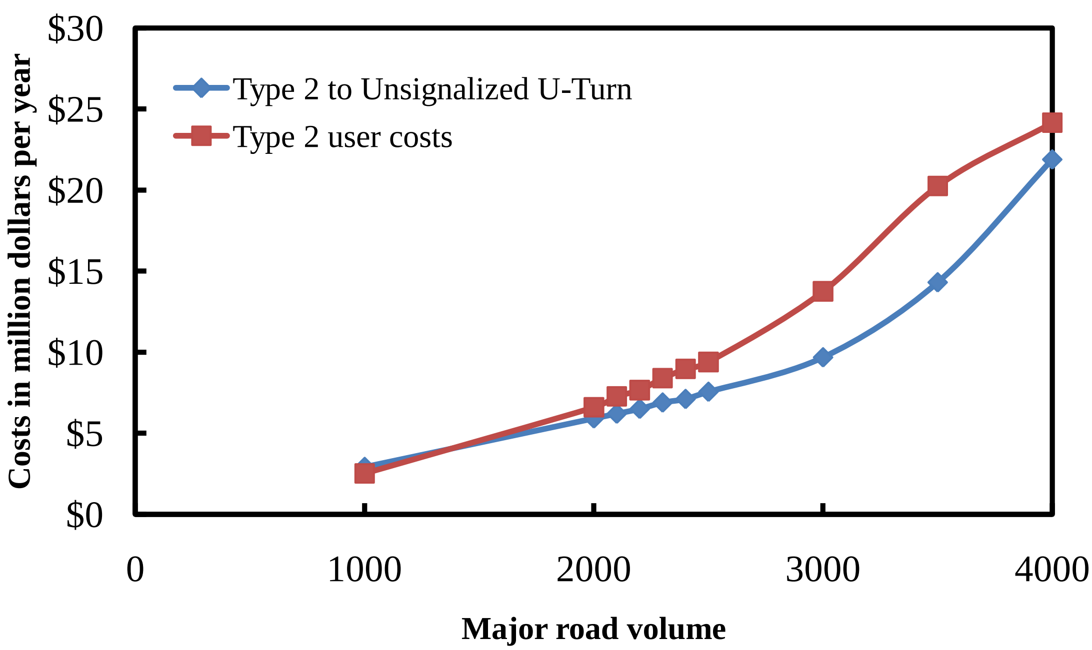


FIGURE 6-4: Comparison of annual equivalent costs of case two to user costs of Unsignalized Median U-Turn

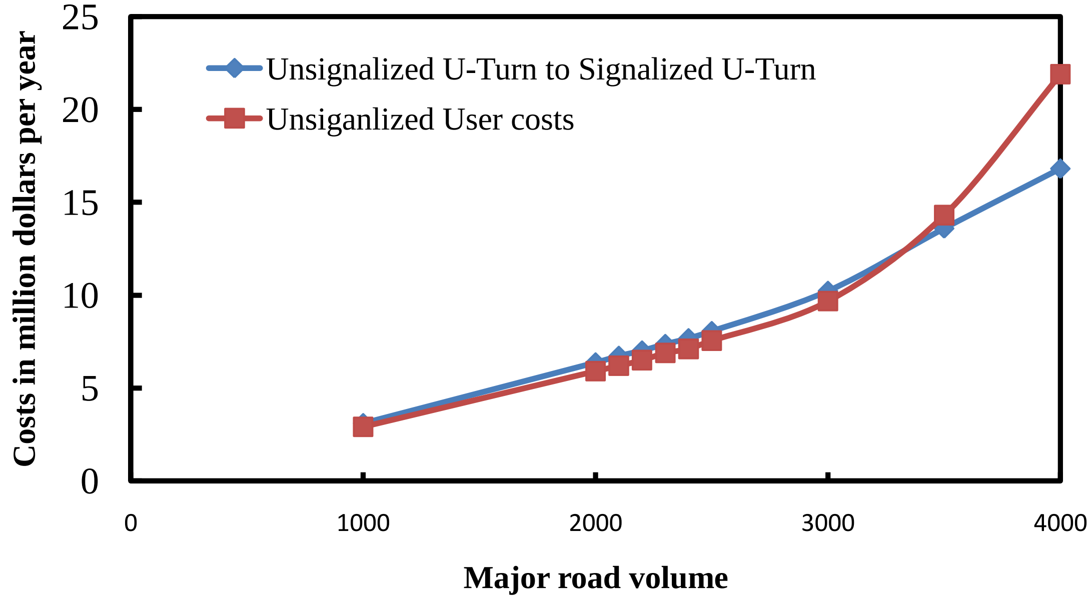
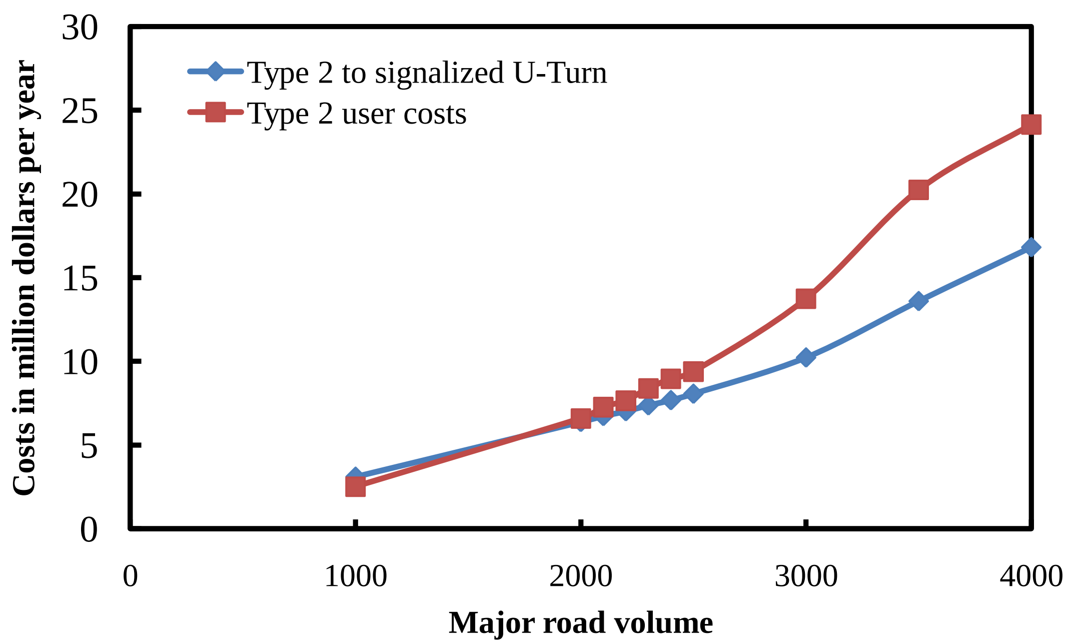


FIGURE 6-5: Comparison of annual equivalent costs of case three to user costs of Type 2 median crossover



CHAPTER 7

CONCLUSIONS

This study describes the operational characteristics of Median U-Turns and estimates where the rural expressway median crossover fails in its operation. The objective of this thesis is to estimate the performance of Median U-Turns for higher volumes when the Type 2 median crossover may not function well. As there is no proper procedure to decide the spacing between the crossroad and Median U-turn, an attempt has been made to prepare a design tool using HCS to decide this distance based on various volume combinations.

The performance of the three design options were estimated based on the three performance measures such as travel time, delay time and queue lengths. VISSIM simulation software was used to obtain these performance measures. The design tool was developed using HCS through which LOS was obtained for T-legged intersection and different lengths of freeway weaving segment (approximated as expressway weaving segment).

From this study it can be inferred that the base alternative (i.e., Type 2 median crossover) performs better for lower major road volumes, unsignalized Median U-Turn performs better for medium volumes of the major road and signalized Median U-Turn performs better for higher major road volumes using the VISSIM simulation software. A design tool was developed using Highway Capacity Software (HCS) to help determine the Median U-Turn distance from the crossroad, based on various volume combinations and Level of Service (LOS). Costs were estimated for all the three design options under two different scenarios (discussed in chapter six) as an aid for choosing the most economic one.

7.1 Traffic operation

The three design options were constructed in VISSIM and their perform measures such as travel time, delay time and queue length were observed. From these observations the following points can be interpreted:

- For lower volume of the major road (i.e., 1000) with various cross road volume combinations (i.e., 100, 200, 300 and 400), Type 2 median crossover proved to be efficient with lower travel times.
- For medium volumes of the major road (i.e., 2000 to 2500 with an interval of 100 and a crossroad volume of 100) and for medium volumes of the major road (i.e., 2000 to 2300 with an interval of 100 and a crossroad volume of 200), unsignalized Median U-turn proved to be efficient with lower travel times.
- For higher volumes of the major road (i.e., 3000 to 4000 with an interval of 500 and a crossroad volume of 100) and for higher volumes of the major road (i.e., 2400 to 4000 and a crossroad volume of 200), signalized Median U-turn proved to be efficient with lower travel times.
- For higher cross road volumes of 300 and 400 with a major road volume of 2000 to 4000, signalized Median U-turn was efficient with lower travel times.

- For major road volumes of 1000 to 2500 with a crossroad volume of 100 and a major road volume of 1000 to 2300 with a crossroad volume of 200, unsignalized Median U-turn proved to be efficient with lower delay times.
- For major road volumes of 2000 to 4000 and crossroad volume of 300 and 400, signalized Median U-Turn proved to be efficient with lower delay times.
- For higher major road volume of 3500 and 4000 with various crossroad volume combinations, Type 2 median crossover and unsignalized Median U-Turn were observed to exceed the safe maximum and average queue length limit of 350 feet (on left turn lane of eastbound and westbound) for Type 2 median crossover and 175 feet for eastbound left turn lane and 195 feet for westbound left turn lane for Median U-Turn

From the above interpretations, it can be concluded that for lower major road volumes Type 2 median crossover performs well, for medium major road volumes unsignalized Median U-Turn performs well and for higher major road volumes signalized Median U-Turn performs well. In the three design options, the average and the maximum queue lengths for eastbound and westbound left turn lanes exceeded the safe limit for higher major road volumes.

7.2 Design Tool for deciding the distance between crossroad and Median U-Turn

In developing the design tool to find the Median U-Turn distance from the crossroad using HCS, the following points were considered:

- Since HCM addresses only the weaving concept for freeways, the rural expressway weaving has been approximated to freeway weaving.
- The entire Median U-Turn has been split in to two T-legged intersections and one freeway weaving segment to calculate the total delay as HCM cannot analyze the whole Median U-Turn.

The design tool provides the required Median U-Turn distance to maintain a desired LOS for a particular major and crossroad volumes. For various Median U-Turn distances, design tool provides the limiting major road volumes to maintain a desired LOS for a given crossroad volume. If there is any particular volume scenario obtained from the field, and if a designer is willing to have a particular LOS, then the designer needs to choose that Median U-Turn distance from the design tool whose limiting value (major road volume) for the same LOS is greater than or equal to the obtained major road volume.

7.3 Cost Estimations

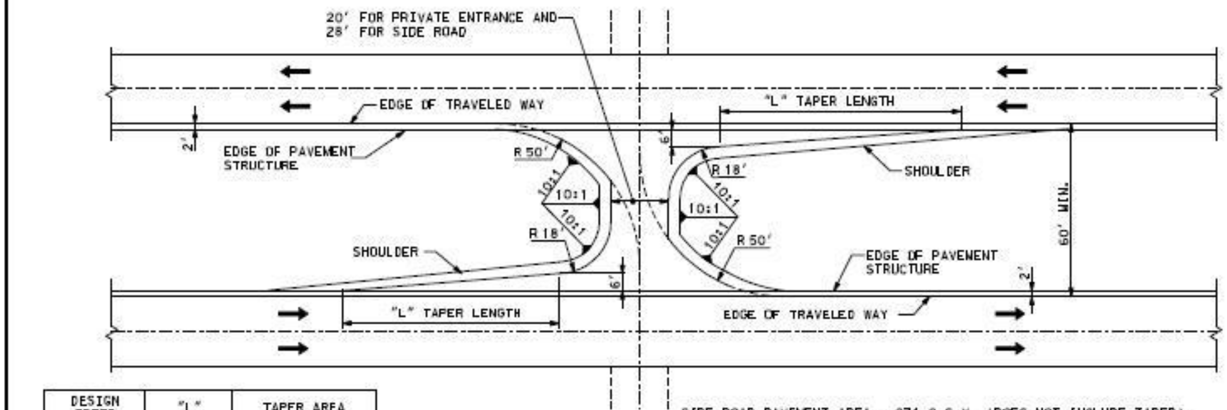
Cost estimates were made for the three design options under two different scenarios. For lower volume of the major road, Type 2 median crossover in scenario one and case one of scenario two has lesser annual equivalents ($1.8 and $2.3 million respectively) which suggests Type 2 median crossover can be used for lower major road volumes. In the case of the medium volumes of the major road for scenario one, unsignalized and signalized Median U-Turn have almost the closer values of the annual equivalents which suggests that unsignalized Median U-Turns can be used for medium volumes. For medium volumes, case two of scenario two proves to be economic. For higher volumes (2500 to 4000) of major road, signalized Median U-Turn proves to be economic in scenario one and the conversion of unsignalized Median U-Turn to signalized Median U-Turn (case two) is economic in scenario two. However, based on the field situations and volume combinations, the better alternative among the three cases of scenario two can be adopted.

7.4 Reliability of HCS and VISSIM

Both VISSIM and HCS were used to obtain the operational characteristics of the three design options taken into consideration. HCS is used to produce the LOS for the intersection as a whole and the freeway weaving in the development of the design tool. VISSIM is used to obtain the performance measures such as travel time, delay time and queue length. VISSIM explicitly estimates additional travel time for each selected travel path. HCS explicitly estimates delay for every individual movement or an isolated intersection. HCM was not calibrated to determine an expressway weave. In this study since the freeway weaving was approximated to expressway weaving, the total delay obtained from HCS for Median U-Turn (i.e., T-leg one delay+ T-leg two delay+ freeway weaving delay) may not be reliable compared to the delay obtained from VISSIM. Hence, the control delay obtained from the two software is different.

APPENDIX I

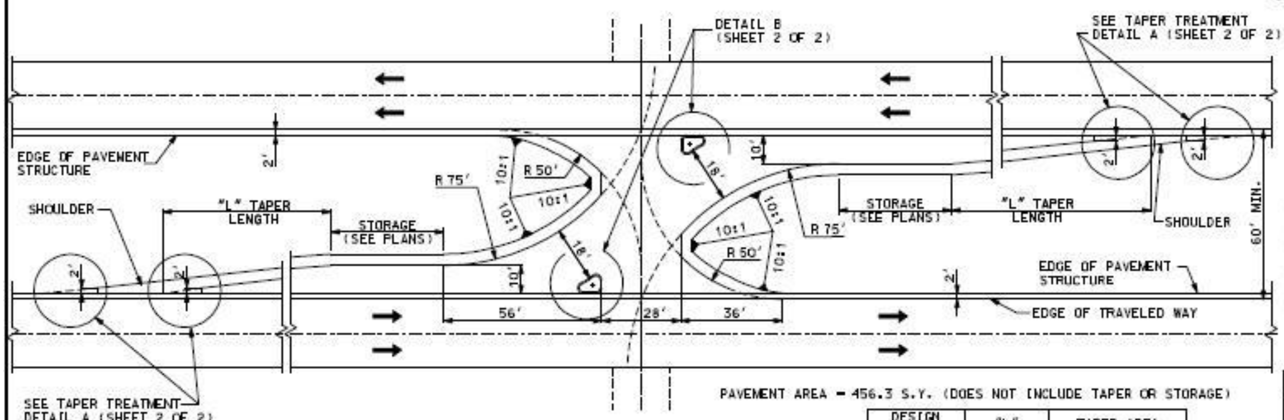
DETAILED SKETCH OF TYPE II MEDIAN CROSSOVER



DESIGN SPEED	"L"	TAPER AREA
≤ 40 mph	112.5'	75.0 S.Y.
> 40 mph	225'	150.0 S.Y.

SIDE ROAD PAVEMENT AREA = 271.0 S.Y. (DOES NOT INCLUDE TAPER).
ASSUMES 56' BETWEEN EDGE PAVEMENT STRUCTURES.
PRIVATE ENTRANCE PAVEMENT AREA = 238.1 S.Y. (DOES NOT INCLUDE TAPER).
ASSUMES 56' BETWEEN EDGE PAVEMENT STRUCTURES.

FOR PRIVATE ENTRANCES, MINOR SIDE ROADS OR FIELD ENTRANCES TYPE I



PAVEMENT AREA = 456.3 S.Y. (DOES NOT INCLUDE TAPER OR STORAGE)

DESIGN SPEED	"L"	TAPER AREA
≤ 40 mph	125'	133.3 S.Y.
> 40 mph	250'	266.7 S.Y.

ASSUMES 28' MIN. WIDTH
TAPER AREA DOES NOT INCLUDE THEORETICAL TAPER

FOR MAJOR SIDE ROADS, STATE ROUTES AND MAJOR COMMERCIAL ENTRANCES TYPE II

NOTES FOR TYPE I MEDIAN OPENINGS:

WHERE TRAVELED WAY SHOULDERS ARE TYPE B, THE MEDIAN OPENING AND TAPER, AND SHOULDERS ADJACENT THERETO, SHALL BE CONSTRUCTED OF A SEAL COAT ON THE SAME THICKNESS OF TYPE 1 OR 2 BASE USED ON THE TRAVELED WAY SHOULDER.

WHERE TRAVELED WAY SHOULDERS ARE TYPE A1 OR A2, THE MEDIAN OPENING AND TAPER, AND SHOULDERS ADJACENT THERETO, SHALL BE CONSTRUCTED OF THE SAME MATERIAL AND THICKNESS AS THE TRAVELED WAY SHOULDERS.

IN ADDITION TO THE IDENTIFIED SLOPES, SLOPES ADJACENT TO MEDIAN OPENING SHOULDERS SHALL NOT BE STEEPER THAN 5.5:1.

NOTES FOR TYPE II MEDIAN OPENINGS:

MEDIAN OPENINGS AND TAPERS SHALL BE CONSTRUCTED OF THE SAME MATERIAL AND THICKNESS AS THE TRAVELED WAY.

SHOULDERS ADJACENT TO THE MEDIAN OPENING AND TAPERS SHALL BE THE SAME AS THE TRAVELED WAY SHOULDER.

IN ADDITION TO THE IDENTIFIED SLOPES, SLOPES ADJACENT TO MEDIAN OPENING SHOULDERS SHALL NOT BE STEEPER THAN 5.5:1.

TONGUE AND GROOVE JOINT (M) WHEN MEDIAN OPENING IS OF PORTLAND CEMENT CONCRETE. (SEE DRAWING 502.05).

GENERAL NOTES:

SHOULDERS THROUGH THE MEDIAN OPENING SHALL BE THE SAME WIDTH AS THE TRAVELED WAY SHOULDER.

SHOULDER PAVEMENT QUANTITIES ARE INCLUDED IN THE PAVEMENT AREA AND TAPER AREA QUANTITIES. WHEN CALCULATING STORAGE AREA QUANTITIES, INCLUDE THE SHOULDER PAVEMENT AREAS.

FOR DETAILS ON CONCRETE MEDIAN STRIP CONSTRUCTION, SEE STANDARD PLAN 608.30.

MISSOURI HIGHWAYS AND TRANSPORTATION COMMISSION

TYPICAL MEDIAN OPENING
(DIVIDED HIGHWAYS)

DATE: ________ EFFECTIVE: 07-01-2004 203.50M 1/2

APPENDIX II

EXAMPLE HCS OUTPUT FOR TYPE 2 MEDIAN CROSS OVER AND FIRST T-LEG FOR UNSIGNALIZED MEDIAN U-TURN

By taking all the factors described in chapter 3, HCS 2000 computes the required parameters like control delay and LOS for each movement. From the input requirements discussed in chapter 3, the output of HCS 2000 for Type 2 median crossover and first T-leg of unsignalized Median U-Turn are shown in the following section. Here a sample of major road volume of 1000 and minor road volume of 100 had been taken into consideration to show the output pattern obtained from HCS.

```
                HCS+: Unsignalized Intersections Release 5.21

______________TWO-WAY STOP CONTROLSUMMARY___________________________

Analyst:              Pavani Boddapati
Agency/Co.:
Date Performed:       4/22/2008
Analysis Time Period: 1 hour
Intersection:         Type 2 median Crossover
Jurisdiction:
Units: U. S. Customary
Analysis Year:        2008
Project ID:
East/West Street:
North/South Street:
Intersection Orientation: EW                Study period (hrs):
1.00

___________Vehicle Volumes and Adjustments_________________________
Major Street:  Approach        Eastbound               Westbound
               Movement     1      2      3      |  4      5      6
                            L      T      R      |  L      T      R
____________________________________________________________________
Volume                      10     980    10        10     980    10
Peak-Hour Factor, PHF       1.00   1.00   1.00      1.00   1.00
1.00
Hourly Flow Rate, HFR       10     980    10        10     980    10
Percent Heavy Vehicles      0      --     --        0      --     --
Median Type/Storage         Raised curb               / 2
RT Channelized?
Lanes                          1    2    0             1    2    0
Configuration                   L   T   TR              L   T   TR
Upstream Signal?                   No                      No
____________________________________________________________________
Minor Street:  Approach        Northbound              Southbound
               Movement     7      8      9      |  10     11     12
                            L      T      R      |  L      T      R
____________________________________________________________________
Volume                      0      40     60        0      40     60
Peak Hour Factor, PHF       1.00   1.00   1.00      1.00   1.00
1.00
Hourly Flow Rate, HFR       0      40     60        0      40     60
Percent Heavy Vehicles      0      0      0         0      0      0
Percent Grade (%)                  0                       0
Flared Approach:  Exists?/Storage                    /
/
Lanes                          0    1    1             0    1    1
Configuration                   LT     R                LT     R
__________________________________________________________________
________Delay, Queue Length, and Level of Service___________________
Approach            EB     WB       Northbound
Southbound
Movement            1      4    |  7      8      9    |  10     11
12
```

```
Lane Config          L      L   |  LT            R    |  LT
R
____________________________________________________________________
v (vph)              10     10     40            60      40
60
C(m) (vph)           706    706    215           525     215
525
v/c                  0.01   0.01   0.19          0.11    0.19
0.11
95% queue length     0.04   0.04   0.68          0.39    0.68
0.39
Control Delay        10.2   10.2   25.6          12.7    25.6
12.7
LOS                   B      B      D             B       D
B
Approach Delay                            17.9                      17.9
Approach LOS                               C                         C
_____________________________________________________________________
```

HCS+: Unsignalized Intersections Release 5.21

```
Phone:                                        Fax:
E-Mail:

____________TWO-WAY STOP CONTROL(TWSC) ANALYSIS_____________________

Analyst:              Pavani Boddapati
Agency/Co.:
Date Performed:       4/22/2008
Analysis Time Period: 1 hour
Intersection:         Type 2 median Crossover
Jurisdiction:
Units: U. S. Customary
Analysis Year:        2008
Project ID:
East/West Street:
North/South Street:
Intersection Orientation: EW                 Study period (hrs):
1.00

_____________Vehicle Volumes and Adjustments_______________________
Major Street Movements      1      2      3      4      5      6
                            L      T      R      L      T      R
____________________________________________________________________
Volume                      10     980    10     10     980    10
Peak-Hour Factor, PHF       1.00   1.00   1.00   1.00   1.00   1.00
Peak-15 Minute Volume       2      245    2      2      245    2
Hourly Flow Rate, HFR       10     980    10     10     980    10
Percent Heavy Vehicles      0      --     --     0      --     --
Median Type/Storage          Raised curb          / 2
RT Channelized?
Lanes                          1   2    0           1   2    0
Configuration                   L  T   TR            L  T   TR
Upstream Signal?                   No                   No
```

Minor Street Movements	7 L	8 T	9 R	10 L	11 T	12 R
Volume	0	40	60	0	40	60
Peak Hour Factor, PHF	1.00	1.00	1.00	1.00	1.00	1.00
Peak-15 Minute Volume	0	10	15	0	10	15
Hourly Flow Rate, HFR	0	40	60	0	40	60
Percent Heavy Vehicles	0	0	0	0	0	0
Percent Grade (%)		0			0	
Flared Approach: Exists?/Storage				/		
/						
RT Channelized?			No			No
Lanes	0	1	1	0	1	1
Configuration	LT		R	LT		R

Pedestrian Volumes and Adjustments

Movements	13	14	15	16
Flow (ped/hr)	0	0	0	0
Lane Width (ft)	12.0	12.0	12.0	12.0
Walking Speed (ft/sec)	4.0	4.0	4.0	4.0
Percent Blockage	0	0	0	0

Upstream Signal Data

	Prog. Flow vph	Sat Flow vph	Arrival Type	Green Time sec	Cycle Length sec	Prog. Speed mph	Distance to Signal feet
S2 Left-Turn							
Through							
S5 Left-Turn							
Through							

Worksheet 3-Data for Computing Effect of Delay to Major Street Vehicles

	Movement 2	Movement 5
Shared ln volume, major th vehicles:		
Shared ln volume, major rt vehicles:		
Sat flow rate, major th vehicles:		
Sat flow rate, major rt vehicles:		
Number of major street through lanes:		

Worksheet 4-Critical Gap and Follow-up Time Calculation

Critical Gap Calculation

Movement		1 L	4 L	7 L	8 T	9 R	10 L	11 T	12 R
t(c,base)		4.1	4.1	7.5	6.5	6.9	7.5	6.5	6.9
t(c,hv)		2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
P(hv)		0	0	0	0	0	0	0	0
t(c,g)				0.20	0.20	0.10	0.20	0.20	0.10
Grade/100				0.00	0.00	0.00	0.00	0.00	0.00
t(3,lt)		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
t(c,T):	1-stage	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2-stage	0.00	0.00	1.00	1.00	0.00	1.00	1.00	0.00
t(c)	1-stage	4.1	4.1	7.5	6.5	6.9	7.5	6.5	6.9
	2-stage	4.1	4.1	6.5	5.5	6.9	6.5	5.5	6.9

Follow-Up Time Calculations

Movement	1 L	4 L	7 L	8 T	9 R	10 L	11 T	12 R
t(f,base)	2.20	2.20	3.50	4.00	3.30	3.50	4.00	3.30
t(f,HV)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
P(HV)	0	0	0	0	0	0	0	0
t(f)	2.2	2.2	3.5	4.0	3.3	3.5	4.0	3.3

Worksheet 5-Effect of Upstream Signals

Computation 1-Queue Clearance Time at Upstream Signal

	Movement 2		Movement 5	
	V(t)	V(l,prot)	V(t)	V(l,prot)
V prog				
Total Saturation Flow Rate, s (vph)				
Arrival Type				
Effective Green, g (sec)				
Cycle Length, C (sec)				
Rp (from Exhibit 16-11)				
Proportion vehicles arriving on green P				
g(q1)				
g(q2)				
g(q)				

Computation 2-Proportion of TWSC Intersection Time blocked	Movement 2		Movement 5	
	V(t)	V(l,prot)	V(t)	V(l,prot)
alpha				
beta				
Travel time, t(a) (sec)				
Smoothing Factor, F				
Proportion of conflicting flow, f				
Max platooned flow, V(c,max)				
Min platooned flow, V(c,min)				
Duration of blocked period, t(p)				
Proportion time blocked, p		0.000		0.000

Computation 3-Platoon Event Periods	Result
p(2)	0.000
p(5)	0.000
p(dom)	
p(subo)	
Constrained or unconstrained?	

Proportion unblocked for minor movements, p(x)	(1) Single-stage Process	(2) Two-Stage Process Stage I	(3) Two-Stage Process Stage II
p(1)			
p(4)			
p(7)			
p(8)			
p(9)			
p(10)			
p(11)			
p(12)			

Computation 4 and 5
Single-Stage Process

Movement	1	4	7	8	9	10	11	12
	L	L	L	T	R	L	T	R
V c,x	990	990	1535	2015	495	1535	2015	495
s								
Px								
V c,u,x								
C r,x								

C plat,x

Two-Stage Process

	7		8		10		11	
	Stage1	Stage2	Stage1	Stage2	Stage1	Stage2	Stage1	Stage2
V(c,x)	1005	530	1005	1010	1005	530	1005	1010
s		3000		3000		3000		3000
P(x)								
V(c,u,x)								
C(r,x)								
C(plat,x)								

Worksheet 6-Impedance and Capacity Equations

Step 1: RT from Minor St.	9	12
Conflicting Flows	495	495
Potential Capacity	525	525
Pedestrian Impedance Factor	1.00	1.00
Movement Capacity	525	525
Probability of Queue free St.	0.89	0.89

Step 2: LT from Major St.	4	1
Conflicting Flows	990	990
Potential Capacity	706	706
Pedestrian Impedance Factor	1.00	1.00
Movement Capacity	706	706
Probability of Queue free St.	0.99	0.99
Maj L-Shared Prob Q free St.		

Step 3: TH from Minor St.	8	11
Conflicting Flows	2015	2015
Potential Capacity	59	59
Pedestrian Impedance Factor	1.00	1.00
Cap. Adj. factor due to Impeding mvmnt	0.97	0.97

```
Movement Capacity                                57              57
Probability of Queue free St.                    0.81
0.81
_____________________________________________________________________
Step 4: LT from Minor St.                           7
10
_____________________________________________________________________
Conflicting Flows                                1535
1535
Potential Capacity                               81              81
Pedestrian Impedance Factor                      1.00
1.00
Maj. L, Min T Impedance factor                   0.79
0.79
Maj. L, Min T Adj. Imp Factor.                   0.84
0.84
Cap. Adj. factor due to Impeding mvmnt           0.74
0.74
Movement Capacity                                60              60
_____________________________________________________________________

Worksheet 7-Computation of the Effect of Two-stage Gap Acceptance
_____________________________________________________________________
Step 3: TH from Minor St.                           8
11
_____________________________________________________________________
Part 1 - First Stage
Conflicting Flows                                1005
1005
Potential Capacity                               322             322
Pedestrian Impedance Factor                      1.00
1.00
Cap. Adj. factor due to Impeding mvmnt           0.99
0.99
Movement Capacity                                317             317
Probability of Queue free St.                    0.87
0.87
_____________________________________________________________________
Part 2 - Second Stage
Conflicting Flows                                1010
1010
Potential Capacity                               320             320
Pedestrian Impedance Factor                      1.00
1.00
Cap. Adj. factor due to Impeding mvmnt           0.99
0.99
Movement Capacity                                315             315
_____________________________________________________________________
Part 3 - Single Stage
Conflicting Flows                                2015
2015
Potential Capacity                               59              59
Pedestrian Impedance Factor                      1.00
1.00
```

```
Cap. Adj. factor due to Impeding mvmnt          0.97
0.97
Movement Capacity                               57                57
__________________________________________________________________
Result for 2 stage process:
a                                               0.95
0.95
y                                               1.05
1.05
C t                                             215               215
Probability of Queue free St.                   0.81
0.81
___________________________________________________________________
Step 4: LT from Minor St.                          7
10
___________________________________________________________________
Part 1 - First Stage
Conflicting Flows                               1005
1005
Potential Capacity                              263               263
Pedestrian Impedance Factor                     1.00
1.00
Cap. Adj. factor due to Impeding mvmnt          0.99
0.99
Movement Capacity                               259               259
__________________________________________________________________
Part 2 - Second Stage
Conflicting Flows                               530               530
Potential Capacity                              506               506
Pedestrian Impedance Factor                     1.00
1.00
Cap. Adj. factor due to Impeding mvmnt          0.76
0.76
Movement Capacity                               386               386
___________________________________________________________________
Part 3 - Single Stage
Conflicting Flows                               1535
1535
Potential Capacity                              81                81
Pedestrian Impedance Factor                     1.00
1.00
Maj. L, Min T Impedance factor                  0.79
0.79
Maj. L, Min T Adj. Imp Factor.                  0.84
0.84
Cap. Adj. factor due to Impeding mvmnt          0.74
0.74
Movement Capacity                               60                60
___________________________________________________________________
Results for Two-stage process:
a                                               0.95
0.95
y                                               0.63
0.63
```

C t 209 209

Worksheet 8-Shared Lane Calculations

Movement	7	8	9	10	11	12
	L	T	R	L	T	R
Volume (vph)	0	40	60	0	40	60
Movement Capacity (vph)	209	215	525	209	215	525
Shared Lane Capacity (vph)	215			215		

Worksheet 9-Computation of Effect of Flared Minor Street Approaches

Movement	7	8	9	10	11	12
	L	T	R	L	T	R
C sep	209	215	525	209	215	525
Volume	0	40	60	0	40	60
Delay						
Q sep						
Q sep +1						
round (Qsep +1)						
n max						
C sh	215			215		
SUM C sep						
n						
C act						

Worksheet 10-Delay, Queue Length, and Level of Service

Movement	1	4	7	8	9	10	11	12
Lane Config	L	L	LT		R	LT		R
v (vph)	10	10	40		60	40		60
C(m) (vph)	706	706	215		525	215		525
v/c	0.01	0.01	0.19		0.11	0.19		0.11
95% queue length	0.04	0.04	0.68		0.39	0.68		0.39
Control Delay	10.2	10.2	25.6		12.7	25.6		12.7

```
LOS                     B      B      D            B       D
B
Approach Delay                             17.9                    17.9
Approach LOS                                C                       C
_____________________________________________________________________

Worksheet 11-Shared Major LT Impedance and Delay
____________________________________________________________________
                                                 Movement 2
Movement 5
_____________________________________________________________________
p(oj)                                               0.99
0.99
v(il), Volume for stream 2 or 5
v(i2), Volume for stream 3 or 6
s(il), Saturation flow rate for stream 2 or 5
s(i2), Saturation flow rate for stream 3 or 6
P*(oj)
d(M,LT), Delay for stream 1 or 4                    10.2
10.2
N, Number of major street through lanes
d(rank,1) Delay for stream 2 or 5
_____________________________________________________________________
```

HCS+: Unsignalized Intersections Release 5.21

```
____________TWO-WAY STOP CONTROL SUMMARY___________________________

Analyst:              Pavani Boddapati
Agency/Co.:
Date Performed:       4/22/2008
Analysis Time Period: 1 hour
Intersection:      First T-leg for unsignalized median U-turn
Jurisdiction:
Units: U. S. Customary
Analysis Year:        2008
Project ID:
East/West Street:
North/South Street:
Intersection Orientation: EW               Study period (hrs):
1.00

____________Vehicle Volumes and Adjustments_________________________
Major Street:  Approach        Eastbound              Westbound
               Movement     1      2      3     |  4      5      6
                            L      T      R     |  L      T      R
_________________________________________________________________
Volume                             1020   30
Peak-Hour Factor, PHF              1.00   1.00
Hourly Flow Rate, HFR              1020   30
Percent Heavy Vehicles             --     --                --     --
Median Type/Storage         Undivided            /
RT Channelized?                          No
Lanes                              2    1
Configuration                      T    R
Upstream Signal?                   No                      No
____________________________________________________________________
Minor Street:  Approach        Northbound             Southbound
               Movement     7      8      9     |  10     11     12
                            L      T      R     |  L      T      R
____________________________________________________________________
Volume                                    100
Peak Hour Factor, PHF                     1.00
Hourly Flow Rate, HFR                     100
Percent Heavy Vehicles                    0
Percent Grade (%)                  0                       0
Flared Approach:  Exists?/Storage                /
/
Lanes                                   1
Configuration                          R
____________________________________________________________________

________Delay, Queue Length, and Level of Service___________________
Approach            EB     WB        Northbound
Southbound
```

Movement	1	4	7	8	9	10	11	12
Lane Config					R			
v (vph)					100			
C(m) (vph)					567			
v/c					0.18			
95% queue length					0.64			
Control Delay					12.7			
LOS					B			
Approach Delay				12.7				
Approach LOS				B				

HCS+: Unsignalized Intersections Release 5.21

Phone: Fax:
E-Mail:

TWO-WAY STOP CONTROL(TWSC) ANALYSIS

Analyst: Pavani Boddapati
Agency/Co.:
Date Performed: 4/22/2008
Analysis Time Period: 1 hour
Intersection:
Jurisdiction:
Units: U. S. Customary
Analysis Year: 2008
Project ID:
East/West Street:
North/South Street:
Intersection Orientation: EW Study period (hrs): 1.00

Vehicle Volumes and Adjustments

Major Street Movements	1	2	3	4	5	6
	L	T	R	L	T	R
Volume		1020	30			
Peak-Hour Factor, PHF		1.00	1.00			
Peak-15 Minute Volume		255	8			
Hourly Flow Rate, HFR		1020	30			
Percent Heavy Vehicles		--	--		--	--
Median Type/Storage	Undivided			/		
RT Channelized?			No			
Lanes		2	1			
Configuration		T	R			
Upstream Signal?		No			No	

Minor Street Movements	7	8	9	10	11	12
	L	T	R	L	T	R
Volume			100			

```
Peak Hour Factor, PHF                   1.00
Peak-15 Minute Volume                   25
Hourly Flow Rate, HFR                   100
Percent Heavy Vehicles                  0
Percent Grade (%)                0                     0
Flared Approach:  Exists?/Storage                /
/
RT Channelized?                         No
Lanes                                 1
Configuration                        R
_____________________________________________________________________

___________Pedestrian Volumes and Adjustments______________________
Movements                    13     14     15     16
_____________________________________________________________________
Flow (ped/hr)                0      0      0      0
Lane Width (ft)              12.0   12.0   12.0   12.0
Walking Speed (ft/sec)       4.0    4.0    4.0    4.0
Percent Blockage             0      0      0      0
_____________________________________________________________________

___________________Upstream Signal Data____________________________
                 Prog.    Sat   Arrival   Green  Cycle   Prog.
Distance
                 Flow     Flow   Type     Time   Length  Speed   to
Signal
                 vph      vph             sec     sec     mph
feet
_____________________________________________________________________
S2  Left-Turn
    Through
S5  Left-Turn
    Through
____________________________________________________________________

Worksheet 3-Data for Computing Effect of Delay to Major Street
Vehicles
____________________________________________________________________
                                           Movement 2     Movement 5
_____________________________________________________________________
Shared ln volume, major th vehicles:
Shared ln volume, major rt vehicles:
Sat flow rate, major th vehicles:
Sat flow rate, major rt vehicles:
Number of major street through lanes:
_____________________________________________________________________

Worksheet 4-Critical Gap and Follow-up Time Calculation
_____________________________________________________________________
Critical Gap Calculation
Movement         1      4      7      8      9      10     11     12
                 L      L      L      T      R      L      T      R
_____________________________________________________________________
t(c,base)                                    6.2
```

```
t(c,hv)          1.00   1.00   1.00   1.00   1.00   1.00   1.00
1.00
P(hv)                                        0
t(c,g)                         0.20   0.20   0.10   0.20   0.20
0.10
Grade/100                      0.00   0.00   0.00   0.00   0.00
0.00
t(3,lt)                                      0.00
t(c,T):  1-stage 0.00   0.00   0.00   0.00   0.00   0.00   0.00
0.00
         2-stage 0.00   0.00   1.00   1.00   0.00   1.00   1.00
0.00
t(c)     1-stage                             6.2
         2-stage
____________________________________________________________________
Follow-Up Time Calculations
Movement           1      4      7      8      9     10     11     12
                   L      L      L      T      R      L      T      R
_____________________________________________________________________
t(f,base)                                    3.30
t(f,HV)          0.90   0.90   0.90   0.90   0.90   0.90   0.90
0.90
P(HV)                                        0
t(f)                                         3.3
____________________________________________________________________

Worksheet 5-Effect of Upstream Signals
_____________________________________________________________________
Computation 1-Queue Clearance Time at Upstream Signal
                                            Movement 2
Movement 5
                                         V(t)   V(l,prot)  V(t)
V(l,prot)
_____________________________________________________________________
V prog
Total Saturation Flow Rate, s (vph)
Arrival Type
Effective Green, g (sec)
Cycle Length, C (sec)
Rp (from Exhibit 16-11)
Proportion vehicles arriving on green P
g(q1)
g(q2)
g(q)
____________________________________________________________________
Computation 2-Proportion of TWSC Intersection Time  blocked
                                            Movement 2
Movement 5
                                       V(t)   V(l,prot)  V(t)
V(l,prot)
_____________________________________________________________________
alpha
beta
Travel time, t(a) (sec)
```

Smoothing Factor, F
Proportion of conflicting flow, f
Max platooned flow, V(c,max)
Min platooned flow, V(c,min)
Duration of blocked period, t(p)
Proportion time blocked, p 0.000
0.000

Computation 3-Platoon Event Periods	Result
p(2)	0.000
p(5)	0.000
p(dom)	
p(subo)	
Constrained or unconstrained?	

Proportion unblocked for minor movements, p(x)	(1) Single-stage Process	(2) Two-Stage Process Stage I	(3) Two-Stage Process Stage II
p(1)			
p(4)			
p(7)			
p(8)			
p(9)			
p(10)			
p(11)			
p(12)			

_Computation 4 and 5
Single-Stage Process

Movement	1	4	7	8	9	10	11	12
	L	L	L	T	R	L	T	R
V c,x					510			
s								
Px								
V c,u,x								
C r,x								
C plat,x								

Two-Stage Process

	7		8		10		11	
	Stage1	Stage2	Stage1	Stage2	Stage1	Stage2	Stage1	Stage2
V(c,x)								
s								
P(x)								

V(c,u,x)

C(r,x)
C(plat,x)

Worksheet 6-Impedance and Capacity Equations

Step 1: RT from Minor St. 9
12

Conflicting Flows 510
Potential Capacity 567
Pedestrian Impedance Factor 1.00
1.00
Movement Capacity 567
Probability of Queue free St. 0.82
1.00

Step 2: LT from Major St. 4
1

Conflicting Flows
Potential Capacity
Pedestrian Impedance Factor 1.00
1.00
Movement Capacity
Probability of Queue free St. 1.00
1.00
Maj L-Shared Prob Q free St.

Step 3: TH from Minor St. 8
11

Conflicting Flows
Potential Capacity
Pedestrian Impedance Factor 1.00
1.00
Cap. Adj. factor due to Impeding mvmnt 1.00
1.00
Movement Capacity
Probability of Queue free St. 1.00
1.00

Step 4: LT from Minor St. 7
10

Conflicting Flows
Potential Capacity
Pedestrian Impedance Factor 1.00
1.00
Maj. L, Min T Impedance factor
1.00

Maj. L, Min T Adj. Imp Factor.
1.00
Cap. Adj. factor due to Impeding mvmnt 1.00
0.82
Movement Capacity

Worksheet 7-Computation of the Effect of Two-stage Gap Acceptance

---

Step 3: TH from Minor St. 8
11

Part 1 - First Stage
Conflicting Flows
Potential Capacity
Pedestrian Impedance Factor
Cap. Adj. factor due to Impeding mvmnt
Movement Capacity
Probability of Queue free St.

Part 2 - Second Stage
Conflicting Flows
Potential Capacity
Pedestrian Impedance Factor
Cap. Adj. factor due to Impeding mvmnt
Movement Capacity

Part 3 - Single Stage
Conflicting Flows
Potential Capacity
Pedestrian Impedance Factor 1.00
1.00
Cap. Adj. factor due to Impeding mvmnt 1.00
1.00
Movement Capacity

Result for 2 stage process:
a
y
C t
Probability of Queue free St. 1.00
1.00

Step 4: LT from Minor St. 7
10

Part 1 - First Stage
Conflicting Flows
Potential Capacity
Pedestrian Impedance Factor
Cap. Adj. factor due to Impeding mvmnt
Movement Capacity

Part 2 - Second Stage

```
Conflicting Flows
Potential Capacity
Pedestrian Impedance Factor
Cap. Adj. factor due to Impeding mvmnt
Movement Capacity
____________________________________________________________________
Part 3 - Single Stage
Conflicting Flows
Potential Capacity
Pedestrian Impedance Factor                    1.00
1.00
Maj. L, Min T Impedance factor
1.00
Maj. L, Min T Adj. Imp Factor.
1.00
Cap. Adj. factor due to Impeding mvmnt         1.00
0.82
Movement Capacity
____________________________________________________________________
Results for Two-stage process:
a
y
C t
____________________________________________________________________

Worksheet 8-Shared Lane Calculations
___________________________________________________________________
Movement                             7      8      9     10     11
12
                                     L      T      R      L      T
R
____________________________________________________________________
Volume (vph)                                      100
Movement Capacity (vph)                           567
Shared Lane Capacity (vph)
___________________________________________________________________

Worksheet 9-Computation of Effect of Flared Minor Street Approaches
___________________________________________________________________
Movement                             7      8      9     10     11
12
                                     L      T      R      L      T
R
____________________________________________________________________
C sep                                             567
Volume                                            100
Delay
Q sep
Q sep +1
round (Qsep +1)
___________________________________________________________________
n max
C sh
SUM C sep
```

n
C act

Worksheet 10-Delay, Queue Length, and Level of Service

Movement	1	4	7	8	9	10	11	12
Lane Config					R			
v (vph)					100			
C(m) (vph)					567			
v/c					0.18			
95% queue length					0.64			
Control Delay					12.7			
LOS					B			
Approach Delay				12.7				
Approach LOS				B				

Worksheet 11-Shared Major LT Impedance and Delay

	Movement 2	Movement 5
p(oj)	1.00	1.00
v(il), Volume for stream 2 or 5		
v(i2), Volume for stream 3 or 6		
s(il), Saturation flow rate for stream 2 or 5		
s(i2), Saturation flow rate for stream 3 or 6		
P*(oj)		
d(M,LT), Delay for stream 1 or 4		
N, Number of major street through lanes		
d(rank,1) Delay for stream 2 or 5		

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