Nebraska Transportation Center

NDOR Nebraska Department of Roads

Report SPR-P1(1) M328

Final Report
26-1121-3006-001

Investigating Operation at Geometrically Unconventional Intersections

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2014

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"This report was funded in part through grant[s] from the Federal Highway Administration [and Federal Transit Administration], U.S. Department of Transportation. The views and opinions of the authors [or agency] expressed herein do not necessarily state or reflect those of the U. S. Department of Transportation."

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A Report on Research Sponsored by

Nebraska Transportation Center

University of Nebraska-Lincoln

Nebraska Department of Roads

Lincoln, Nebraska

August 2014

Technical Report Documentation Page

1. Report No. SPR-P1(1) M328	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Investigating Operation at Geometrically Unconventional Intersections		5. Report Date 6. Performing Organization Code
7. Author(s) Anuj Sharma, Sunil Gyawali, Laurence R. Rilett		8. Performing Organization Report No. 26-1121-3006-001
9. Performing Organization Name and Address Nebraska Transportation Center 2200 Vine St. PO Box 830851 Lincoln, NE 68583-0851		10. Work Unit No. (TRAIS) 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Nebraska Department of Roads 1500 Hwy. 2 Lincoln, NE 68502		13. Type of Report and Period Covered 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

This report documents the development of decision assistance curves (DAC) for unconventional intersections, particularly median U-turns (MUT), continuous flow intersections (CFI), and jughandles. The operational measure of effectiveness such as delay, fuel consumption, and emissions were computed. An economic analysis was performed to compute the net present value (NPV) of benefits of operation and benefit to cost ratio (B/C) by estimating user's cost, non-user's cost, construction cost, and operation and maintenance cost for the life cycle period. The DAC classified the region of optimal performance of rural unconventional intersections comprising of four-lane major streets and two-lane minor streets. DAC indicated that MUT is applicable for almost all levels of volume combinations of major and minor street approach volumes under the presence of low left turning traffic. For medium to high left turning traffic, jughandle and CFI performed optimally on high major street approach volumes. Furthermore, it was also observed that for a case with medium to high left turning volumes, the use of CFI would be optimal for high major street approach volumes and high minor street approach volumes at an unbalanced condition. The use of a jughandle would be optimal for high major street approach volumes and its performance got better with increasing minor street approach volume at a balanced condition. However, the jughandle performed better at high major street approach volume and low minor street approach volume at an unbalanced condition. The study developed a spreadsheet tool called SILCC to estimate the operational measure of effectiveness, as well as to perform a life cycle cost analysis. A sample case study performed on a 24-hour rural pattern volume indicated high NPV of operational benefits and high B/C related to MUT compared to all other intersections for new construction. Though the MUT-retrofit had the highest NPV, since the construction cost of MUT-retrofit is high, a jughandle-retrofit was found to have the highest B/C.

17. Key Words		18. Distribution Statement	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 206	22. Price

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Measure of Effectiveness (MoEs)
Median U-Turn (MUT)
Continuous Flow Intersection (CFI)
Highway Capacity Manual (HCM)
Highway Capacity Software (HCS)
Right Turn Followed by U-Turn (RTUT)
Intersection Design Alternative Tool (IDAT)
Capacity Analysis and Planning of Junctions (CAP-X)
Net Present Value (NPV)
Benefit to Cost Ratio (B/C)
Life Cycle Cost Analysis (LCCA)
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Balance Factor (BF)
Left Turn Percentage (LTP)
Truck Percentage (TP)

Acknowledgments

The research team would like to thank the Technical Advisory Committee (TAC) members Matt Kneeman, Dan Waddle, Jim Knott, Ryan Huff, Virendra Singh, and Glenn Hansen for their valuable suggestions through the project period. Additionally, the research team would like to thank the persons from different State DOTs who cooperated while gathering information about the current state of practice.

Disclaimer

Abstract

This report documents the development of decision assistance curves (DAC) for unconventional intersections, particularly median U-turns (MUT), continuous flow intersections (CFI), and jughandles. The operational measure of effectiveness such as delay, fuel consumption, and emissions were computed. An economic analysis was performed to compute the net present value (NPV) of benefits of operation and benefit to cost ratio (B/C) by estimating user's cost, non-user's cost, construction cost, and operation and maintenance cost for the life cycle period. The DAC classified the region of optimal performance of rural unconventional intersections comprising of four-lane major streets and two-lane minor streets. DAC indicated that MUT is applicable for almost all levels of volume combinations of major and minor street approach volumes under the presence of low left turning traffic. For medium to high left turning traffic, jughandle and CFI performed optimally on high major street approach volumes. Furthermore, it was also observed that for a case with medium to high left turning volumes, the use of CFI would be optimal for high major street approach volumes and high minor street approach volumes at an unbalanced condition. The use of a jughandle would be optimal for high major street approach volumes and its performance got better with increasing minor street approach volume at a balanced condition. However, the jughandle performed better at high major street approach volume and low minor street approach volume at an unbalanced condition. The study developed a spreadsheet tool called SILCC to estimate the operational measure of effectiveness, as well as to perform a life cycle cost analysis. A sample case study performed on a 24-hour rural pattern volume indicated high NPV of operational benefits and high B/C related to MUT compared to all other intersections for new construction. Though the MUT-retrofit had the

highest NPV, since the construction cost of MUT-retrofit is high, a jughandle-retrofit was found to have the highest B/C.

Executive Summary

Unconventional intersections are applied in locations where conventional intersections have failed to provide the expected service in terms of congestion mitigation, emission reduction, fuel savings, and safety effectiveness. There are several types of unconventional intersection designs whose performance is governed by the condition of traffic, geometry, and controls. Deciding on the most appropriate intersection type for a particular set of conditions is a very challenging task. At present, a few tools and several studies have dealt with the development of a decision support system for the selection of optimal alternatives, however, they are either based on a preliminary planning level-of-performance analysis or a simple comparison of operational measures of effectiveness computed from simulations. An analysis of the costs and benefits related to unconventional intersection alternatives has been neglected. To solve these issues, this study was designed to develop decision assistance curves (DAC), which can classify and quantify the region and level of performance of three unconventional intersections: MUT, CFI and jughandle. The study used macro-level analysis of operation of intersections.

To get a clear idea about the use of DAC, refer to figure A, figure B, and figure C. Figure A represents a DAC labelled as MUT Threshold, which has determined the optimal region of performance of MUT and jughandle. According to this figure, a jughandle is appropriate above approximately a 1900 vph major street approach volume. Consider the volume criteria of total major street approach volume of 1900 vph and total minor street approach volume of 150 vph as

represented by point A in figures A-C. From figure A, it is clear that a jughandle is an optimal choice for this particular volume criterion. Figures B and C can be used to find how much delay savings or loss a jughandle would provide at this particular criteria. From figure B, it is clear that point A falls on the contour of magnitude 6. It indicates that the highest delay loss for the left turn movement of a jughandle (critical left turn movement) as compared to the left turn movement of standard signalized intersection at this volume criterion is 6 sec/veh. It should be noted that the positive value of contours represent delay loss and the negative value of contours represent delay savings. Similarly, from figure C, one can locate the position of point A on the contour of magnitude -3.5, which indicates that a jughandle saves the average total intersection delay 3.5 sec/veh as compared to standard signalized intersection for this particular criterion.

The overall analysis of DAC indicated that MUT is applicable for almost all levels of the volume combinations of major street approach volume and minor street approach volume under the presence of low left turning traffic. For high left turning traffic, jughandle and CFI performed optimally on high major street approach volume. Furthermore, it was also observed that, for a case with high left turning volumes, the use of CFI would be optimal for high major street approach volume and high minor street approach volume at an unbalanced flow condition. The use of jughandle would be optimal for high major street approach volume and its performance got better with increasing minor street approach volume at a balanced flow condition. However, jughandle performed better at high major street approach volume and low minor street approach volume at an unbalanced flow condition.

To find the costs and benefits related to the implementation of unconventional intersections, this study developed a spreadsheet tool called “SILCC” that is capable of performing a life cycle cost analysis (LCCA) and providing the net present value (NPV) of a

marginal benefit and marginal benefit-to-cost (B/C) ratio for any user's defined criteria. A sample case study on performed on a 24-hour rural pattern volume indicated a higher NPV of operational benefits and higher B/C related to MUT than all other intersections for new construction. Though the MUT-retrofit had the highest NPV, since the construction cost of MUT-retrofit is high, a jughandle-retrofit was found to have the highest B/C.

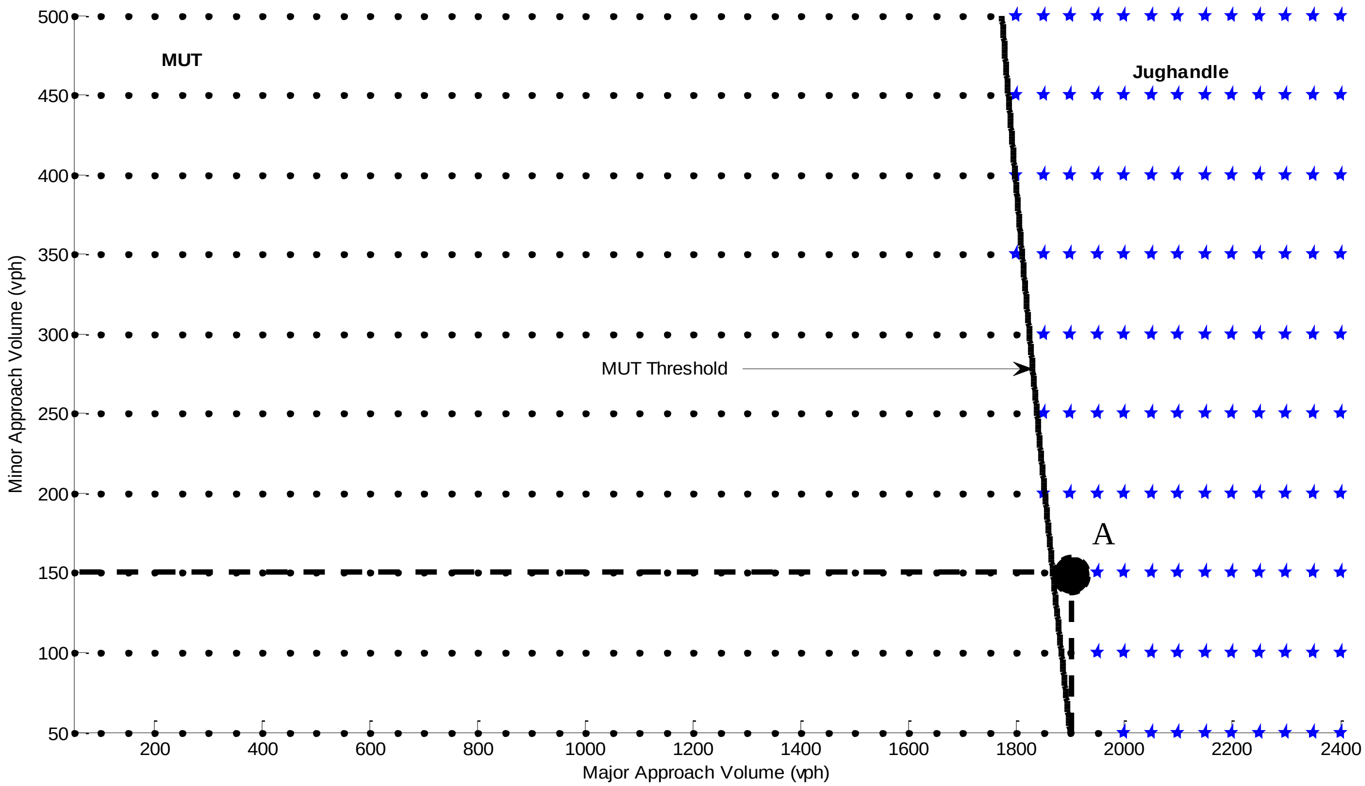


Figure A DAC for jughandle at BF=0.5, LTP=15% and TP=5%

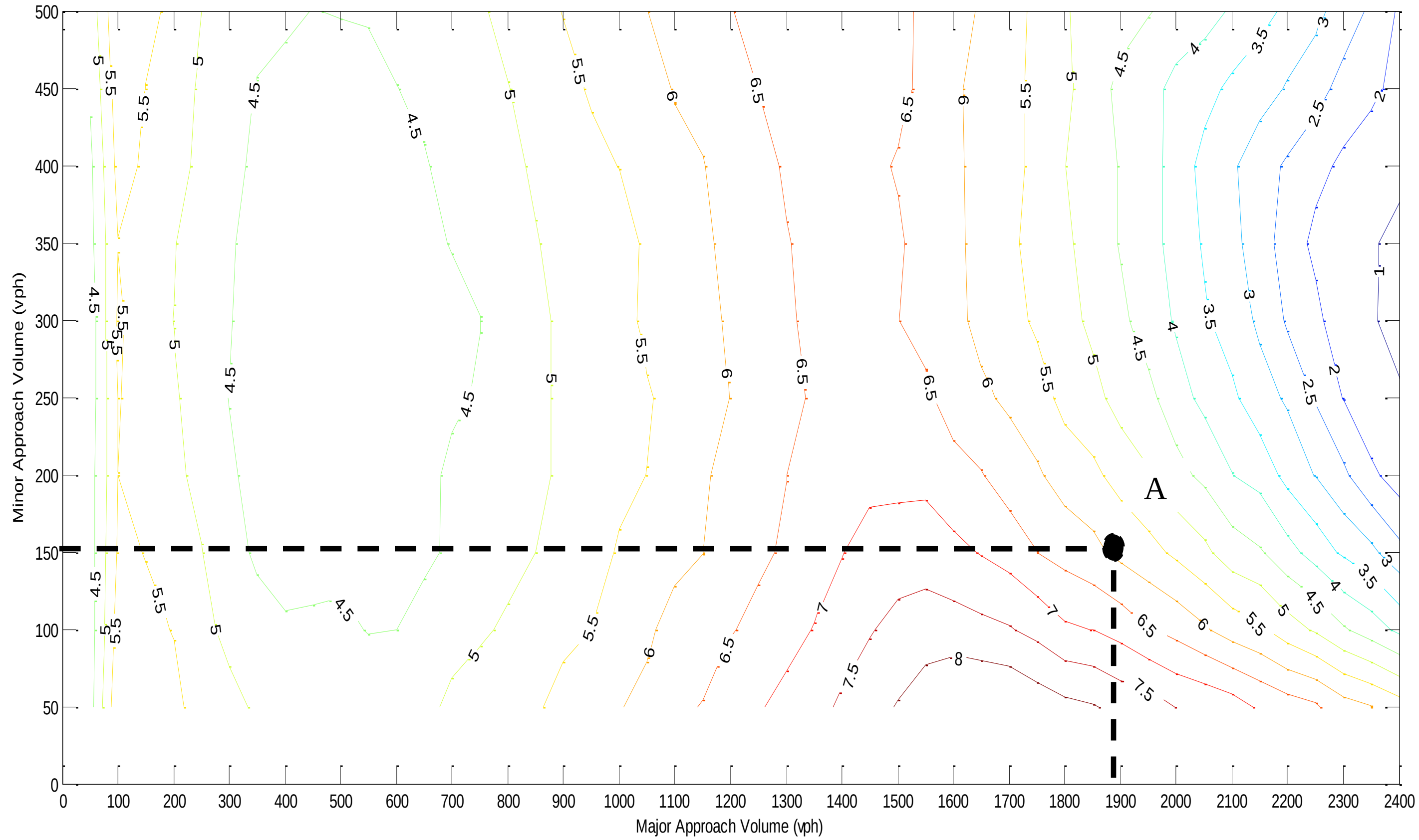


Figure B Contour plot critical of left turn delay difference for jughandle at BF=0.5, LTP=15%, TP=5%

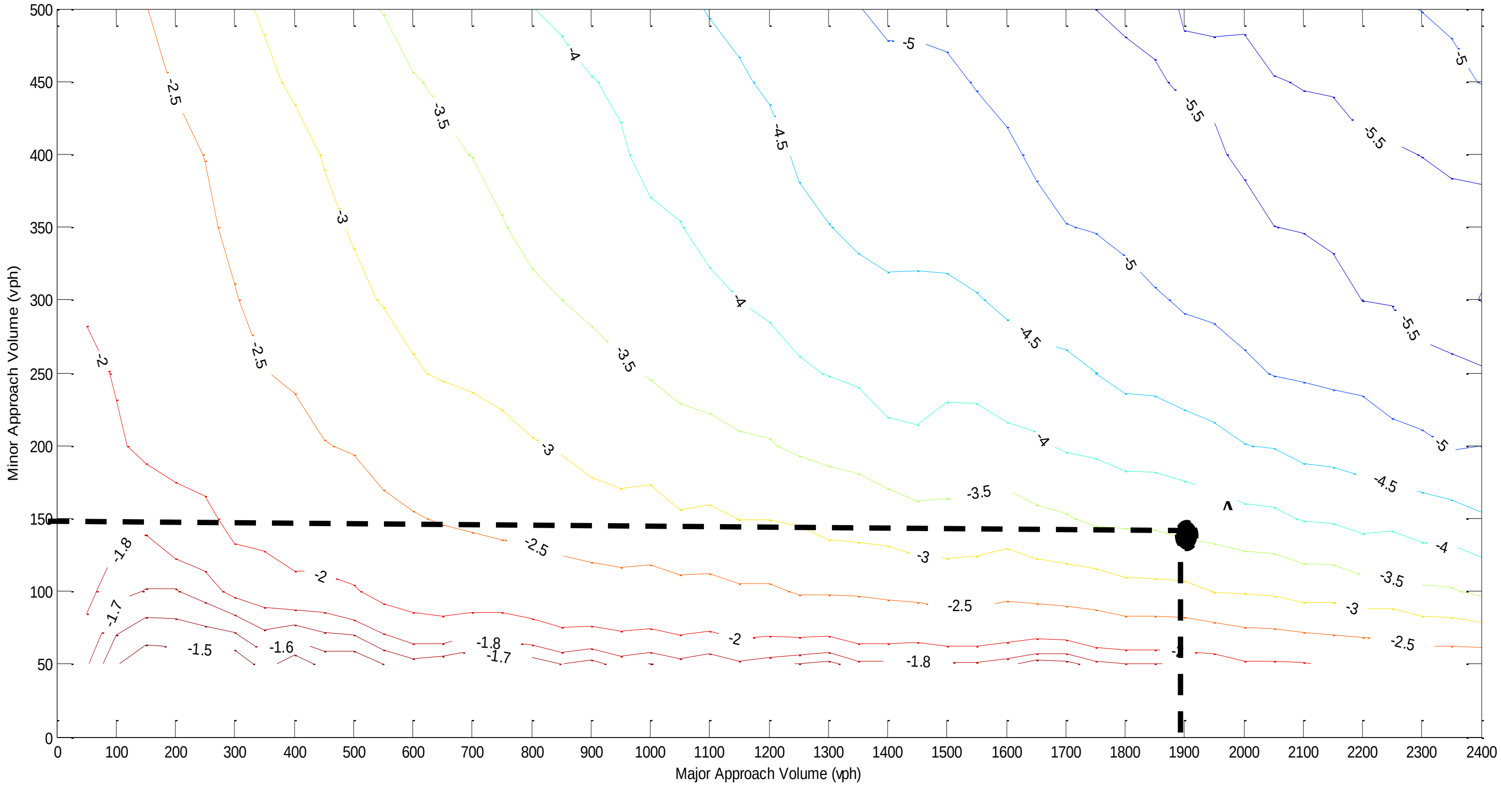


Figure C Contour plot of total intersection delay difference for jughandle at BF=0.5, LTP=15%, TP=5%

Chapter 1 Introduction and Objective

1.1 Problem Statement

The prevalent problem of traffic congestion has caused the wasting of billions of gallons of fuel, time, and money (Grant, Bowen, Day, Winick, Bauer, Chavis & Trainor, 2011). The 2012 Urban Mobility Report stated that the cost due to traffic congestion increased to $121 billion in the year 2011. This estimation was based on the travel delay of 5.5 billion hours, 2.9 billion gallons of wasted fuel, the production of 56 billion pounds of carbon dioxide (CO_2), and truck congestion costs of $27 billion. In addition, about 40-50% of all non-recurring congestion is associated with traffic incidents, and for every dollar of congestion costs, the related crash cost is $1.84 in large cities (Crashes vs. Congestion-What's the cost to Society, 2008). This scenario reflects a strong need for measures to mitigate traffic congestion to stop these losses and maintain mobility. The existing conventional transportation infrastructures alone may be insufficient to alleviate these kind of transportation problems. There is a need to investigate the development and implementation of unconventional designs to cope with this issue by overcoming the inefficiency of conventional intersections and maintaining better traffic mobility.

While planning for the implementation of unconventional intersections, the decision-making criteria to select an appropriate alternative while considering limiting factors such as traffic volumes, truck percentage, geometry, etc., play very important role. At present, few decision assistance tools are available that perform planning-level analysis and are capable of providing a hierarchical order of unconventional intersections based on their performance. The literature (Asokan, Bared, Jagannathan, Hughes, Cicu and Illota, 2010; Kirk, Jones and Statmatadias, 2011; Stamatiadis, kirk, Agrawal and Jones, 2011; Stamatadias, Kirk and Agrawal, 2012; Sangster and Rakha, 2014) indicates that these tools use critical lane volume approach

(CLV) to evaluate the performance of intersections. The details about these tools are included in chapter 2 of this report. With the exception to these tools, most of the literature about unconventional intersections (Goldbat, Mier and Fredman, 1994; Hummer and Boone, 1995; Dorothy, Maleck and Nolfe, 1997; Topp and Hummer, 2005; Pittasringkam, 2005; Tarko, Inerowicz, Lang and Villwock, 2008; Tarko, Azam and Inerowicz, 2010; Kivlins and Naudzuns, 2011; Chang, Lu and Xiangfeng, 2011; Chowdhury, 2011; Smith, 2011; El Esawey and Sayed, 2014) have compared individual performance measures to the conventional intersections in order to evaluate the performance of unconventional intersections. At present, none of the studies have developed thresholds that would allocate the region where the performance of each type of intersection is optimal. Additionally, the accountancy of the cost and benefit aspect during the decision-making process for selection of a suitable unconventional intersection is very necessary. This area is also neglected in the existing method of evaluating unconventional intersections. It could be that a certain type of unconventional intersection for a given situation may have some operational or safety benefits, but the cost of implementation is so high that its use may not be economically justified. In light of these issues, this study is designed to develop decision assistance curves that can classify and quantify the optimal level and region of the performance of unconventional intersections and help in the decision making process. This study will also develop a tool that can perform a life cycle cost analysis that incorporates all the costs and benefits of unconventional intersections.

1.2 Study Intersections

This study attempts to investigate the use of the following three types of unconventional intersections: median U-turn (MUT), continuous flow intersection (CFI), and jughandle.

A common feature of these three types of unconventional intersections is the treatment of the left turn maneuver; diverting left turning traffic from through traffic. This process is highly effective at easing the operation of an intersection system by redistributing demands over sub-intersections and the core intersection, and reducing the cycle length of signal controls. The key objective is to eliminate the protected left turn phase on the approaches to ameliorate the signal operation, which will minimize the intersection delay.

1.2.1 Median U-Turn (MUT)

The median U-turn (MUT), as shown in figure 1.1, is comprised of one signalized intersection and two median crossovers or openings. The median openings, depending on the situation, can be STOP controlled or signal controlled. The signalized intersection is only used for major and minor street through movements, and hence operates with a two phase signal. The left turners move with through movement through the signalized intersection, then divert toward the exclusive left turn lane at the median openings, and finally perform a U-turn. This kind of geometric and signal configuration is expected to improve the intersection by reducing delay, stops, and conflicts; providing better progression for the through traffic on the major street; and increasing the capacity of the intersection (Hummer, 1998(A); Techbrief: Synthesis of Median U-Turn Intersection Treatment, 2007).

1.2.2 Continuous Flow Intersection (CFI)

The continuous flow intersection (CFI) shown in figure 1.2 is comprised of five intersections, including one main signalized intersection and four other signalized crossover intersections. The left turners are diverted through a displaced left turn lane 300-400 feet ahead of the intersection and merge with the minor street at crossover intersections. Hence, the main signalized intersection is only for through traffic and operates with two signal phases. This kind of geometric and signal configuration is expected to reduce delay, travel time, fuel consumption, and pollution (Techbrief: Displaced Left-Turn Intersection, 2009).

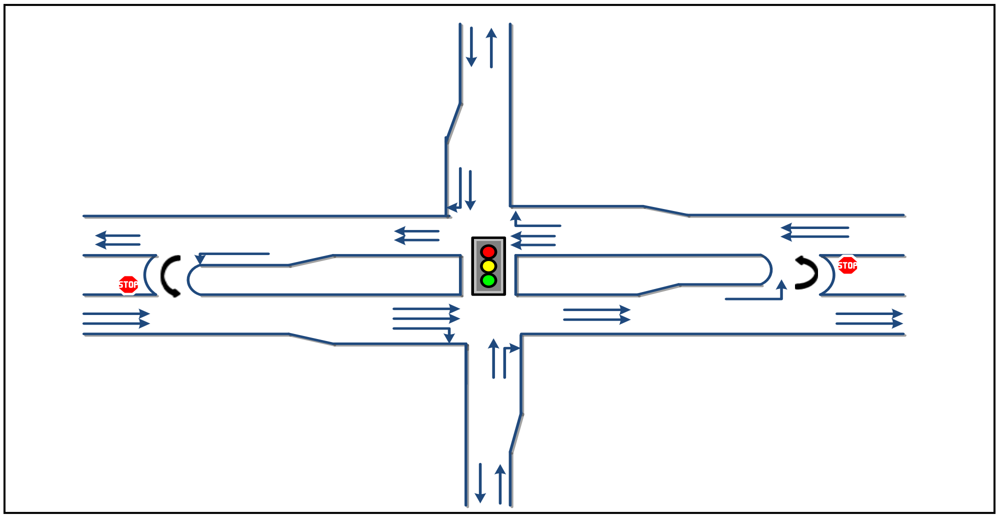


Figure 1.1 A typical median U-turn

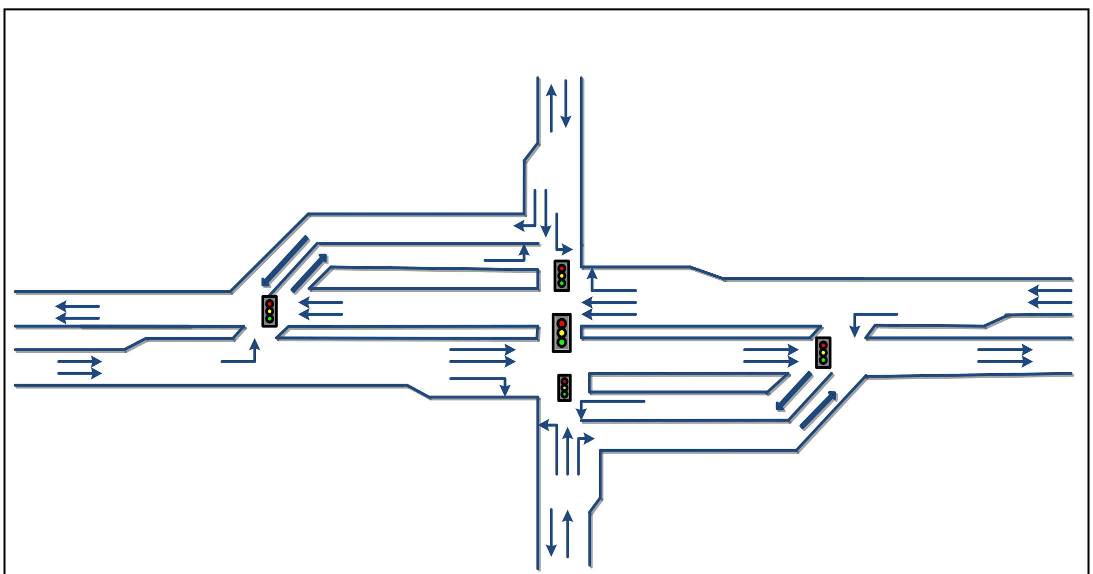


Figure 1.2 A typical continuous flow intersection

1.2.3 Jughandle

Jughandle intersections use ramps to divert left-turning vehicles from the main street. Turning left from a minor street is allowed. The signalized intersection operates for through vehicles from the major street, through vehicles from the minor street, and left-turning vehicles from the minor street. The crossovers are generally YIELD controlled. The FHWA Tech Brief on Traffic Performance of Three Typical Designs of New Jersey Jughandle Intersections (2007) has mentioned three types of jughandles: (1) forward/forward (F/F), (2) forward/reverse (F/R), and (3) reverse/reverse (R/R). A typical R/R jughandle is depicted in figure 1.3. A jughandle is expected to reduce delay and increase the capacity of an intersection (TechBrief: Traffic Performance of Three Typical Designs of New Jersey jughandle Intersections, 2007).

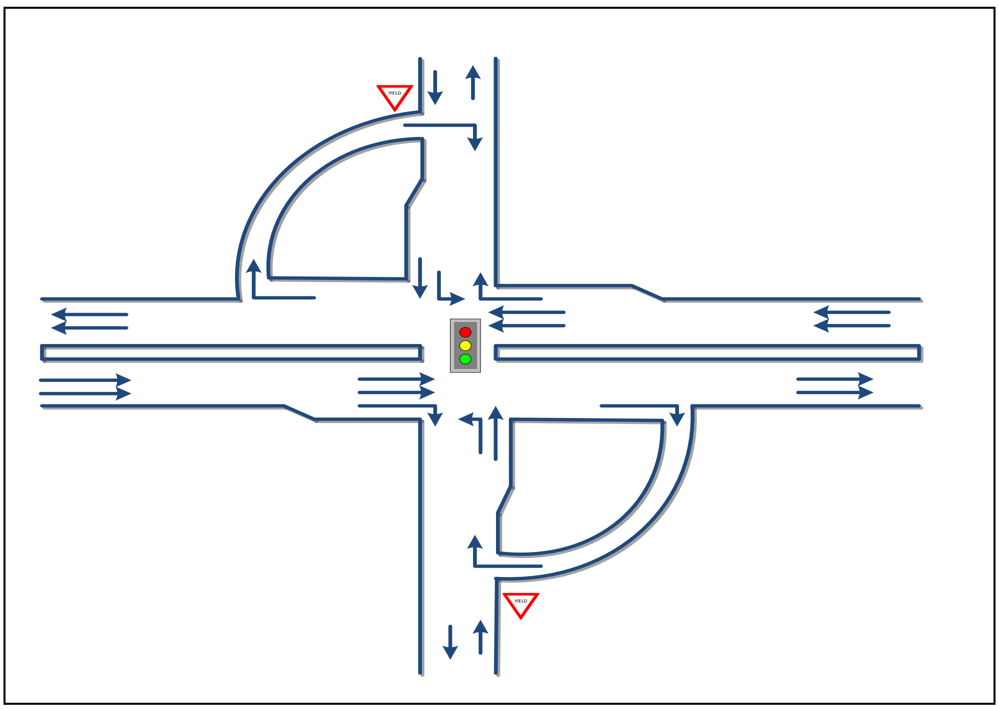


Figure 1.3 A typical R/R jughandle

1.3 Scopes and Objective

As mentioned in the preceding section, this study includes only three types of unconventional intersections: MUT, CFI and jughandle. The study will consider only fully actuated signal control operation on all of these intersections, including a simple standard signalized intersection. Furthermore, this study is limited to full MUT with no left turn at the signalized intersection and un-signalized (STOP controlled) median openings; a signalized CFI with a single controller; and jughandle with un-signalized (YELD controlled) crossovers. The study will analyze a rural intersection with a four-lane major street and two-lane minor streets. However, a tool will be developed to work on multiple sets of criteria so it can be applicable for any locations. The study carries the following specific objectives:

1. Evaluate the operation of three unconventional intersections such as MUT, CFI, and jughandle macroscopically.
2. Develop decision assistance curves that can provide decision support for the selection of suitable intersection types and quantification of the performance for the selected intersections under defined volume criteria.
3. Monetize the benefits and costs, and perform an economic analysis based on marginal benefits provided by each intersection type in comparison to a standard signalized intersection.
4. Develop a spreadsheet tool that can operate under a user's defined criteria and provide decision assistance for the selection of a suitable unconventional intersection type.

1.4 Report Organization

This report is organized into eight chapters. This chapter, "Introduction and Objectives," deals with the general background of unconventional intersections and the scope and objectives of the study. Chapter 2 contains a review of related past studies regarding the operation, safety, and cost of unconventional intersections. Chapter 3 deals with the condition of existing practices, especially the guidelines for the implementation of unconventional intersections and related costs. Chapter 4 deals with an operational analysis of these unconventional intersections. Chapter 5 is concerned with the delay sensitivity of these unconventional intersections with respect to multiple volume conditions. Chapter 6 deals with an economic analysis, including cost estimation, monetization of cost, and a life cycle cost analysis, and the development of a spreadsheet tool. Chapter 7 is the last chapter, which is comprised of a conclusion based on the study's findings and recommendations for further research.

Chapter 2 Literature Review

This section is focused on the description of past studies, their methods of evaluation, and performance related to the development and implementation of unconventional intersections based on a literature review. The literature review is comprised of three major components: (1) the description of existing tools to evaluate unconventional intersections, (2) their performance in terms of operation and safety, and (3) costs related to unconventional intersections. Limited by the scope of this study, the focus is on the MUT, CFI, and, jughandle unconventional intersections.

2.1 Existing Tools to Evaluate Unconventional Intersections

An extensive search found that there are three types of tools developed for evaluating unconventional intersections including MUT, CFI and jughandle. The following sections detail the names and description of these tools.

2.1.1 Intersection Design Alternative Tool (IDAT)

This tool was developed by a research team at the University of Kentucky. It is available for download from the CATSlab website at the Kentucky Transportation Centre upon registration. The main purpose of this tool is to provide decision assistance for engineers and planners for the selection of alternative designs based on capacity constraints (Kentucky Transportation Centre, accessed 2014). The program accepts input of peak-hour traffic and pedestrian volumes, and can work for 12 different alternative designs including CFI, MUT and jughandle. It evaluates intersection in terms of right of way, safety, and ability to accommodate access management techniques (Kentucky Transportation Centre, accessed 2014). A detailed description is also provided in a paper by Stamatiadis et al. (2012) and in a report by Stamatiadis et al. (2011). The operational evaluation procedure has utilized critical lane volume analysis

(CLA) or critical movement analysis (CMA) (Stamatiadis et al., 2011; Stamatadias et al., 2012; Kirk et al., 2011). CMA for signalized intersections represents the sum of the critical volumes assigned for each signal phase of that intersection (Kirk et al., 2011). For two-way stop control, it uses the critical approach volume, which is equivalent to critical volume for signalized intersections. The threshold for a particular intersection is fixed based on the plot of intersection control delay and critical volume. The critical thresholds determined for signalized intersections was 1,400 vph (vehicles per hour), 1,200 vph for all way stop control, 900 vph for two way stop controlled, and 1,000 vph for roundabouts (Kirk et al., 2011).

2.1.2 Alternative Intersection Selection Tool (AIST)

The AIST was developed by the Federal Highway Administration (FHWA) in 2009. The tool used the CLV approach to approximate the capacity as the sum of critical movements at an intersection per lane based on input of peak hour volumes (Asokan, Bared, Jagannathan, Hughes, Cicu and Illota, 2010). The tool can evaluate six types of intersections: (1) conventional intersections, (2) displaced left turn or continuous flow intersections (partial and full), (3) restricted crossing U-turn, (4) median U-turn (partial and full), (5) quadrant roadway intersections, and (6) roundabouts. The tool was later updated and expanded to include additional alternatives and was renamed the Capacity Analysis for Planning of Junctions (CAP-X) (Alternative Intersections by John Sangster, accessed on 2014).

2.1.3 Capacity Analysis and Planning of Junctions (CAP-X)

The CAP-X was developed by the FHWA in partnership with the Transportation Systems Institute at the University of South Florida in 2011. It is the updated form of AIST. The tool is available for download from the Transportation System Institute (TSI) website upon registration. Sangster and Rakha (2014) have done extensive analysis of the CAP-X tool and consider it a highly functional planning level tool. However, they have emphasized this tool's the need for validation. Sangster and Rakha (2014) have also documented the formulation of critical sum equations used in the tool and have demonstrated the operational limitations of intersections as predicted by this tool. The critical sum method determines the most governing movement at any time during a signal cycle, which sums the demands for critical movements in the sequence and determines a total value for demand at intersections in vehicles per hour per lane. This software is capable of assessing six different intersection designs and five different interchange designs, including CFI and MUT. The authors indicated that CAP-X does not apply adjustment factors for lane utilization or adjustment factors for saturation flow rate for turning movements, and doesn't allow variability in the end time of opposing left turn phases. The authors also stated that CAP-X software does not include jughandle intersections. Similarly, the authors indicated that CAP-X neglects the additional volumes added by the minor street left-turn movements.

2.2 Operation and Safety Performance Related to Median U-Turn (MUT)

As described in preceding chapter, the two-legged MUT is comprised of a core signalized intersection and two median openings. The left turning vehicles from minor and major streets weave toward the exclusive left turn lane to approach the median opening and finally perform a U-turn. The traffic operational phenomenon regarding this maneuver and the safety and operational performance operation of the system are described in detail in the following subsections.

2.2.1 Median Opening and Operation Related to U-Turn Maneuver

The two-legged MUT is constructed with two median openings along the major street. The MUT can be implemented with signalized or un-signalized openings (Techbrief: Synthesis of MUT Intersection Treatment, 2009). To avoid stopping on the through lane, the exclusive left-turn lanes are provided in advance of the median openings (AASHTO Green Book, 2011). The width of the median depends on the type of U-turn maneuver and type of vehicles. The values are provided in the AASHTO Green Book (2011) and a Michigan DOT report, “Information and Geometric Design Guidance Regarding Boulevards, Directional Crossovers, and Indirect (‘Michigan’) Left-Turns.” The median on a four-lane arterial should have width of 60 ft. to accommodate a tractor-semitrailer combination of trucks as the design vehicle (AASHTO Green Book, 2011; Rodgerts, Ringert, Koonce, Bansen, Nguyen, McGill, Stewart, Sugget, Neuman, Antonucci, Hardy and Courage, 2004). The minimum design of the median openings are also provided in the AASHTO Green Book (2011) and MDOT guidelines. The U-turn opening will benefit from signalization if the geometry is perfect and signal progression can be maintained. If the geometry is not perfect, STOP signs work best for U-turn openings (Dorothy, Maleck and Nolf, 1997). The signal at the median opening should accommodate a maximum queue to avoid spill overs (Rodgerts et al., 2004). The provision of median openings along the segment between intersections helps maintain the good capacity and level of service of downstream signalized intersections, rather than making vehicles to U-turn from downstream signalized intersections (Guo, Liu, Liu and Deng, 2011). There will be less delay experienced if U-turns are allowed through median openings before downstream signalized intersections as opposed to direct left turn; drivers prefer making right turns followed by a U-turn (RTUT) at median openings rather than signalized intersections (Liu, Lu, Pirinccioglu and Sokolow, 2006). It also helps to reduce travel time (Liu and Lu, 2007). Although a study (Kai, Ning and Chen-dong, 2007) mentioned that the application of an exclusive left turn lane at the median opening may not be good in some conditions, such as when there is a slow operating speed and high volume, it cannot be omitted because of its safety aspect (Chen, Qi and Liu, 2014). The existing method to evaluate median openings and the U-turn maneuver are described based on past literature.

2.2.1.1 Estimation of U-Turn Capacity and Delay

The Highway Capacity Manual (HCM, 2010) has adopted Harder's model as a procedure to estimate the capacity of minor stream movement at an un-signalized, two way stop controlled (TWSC) intersection. In Harder's model, the capacity of minor movement is expressed as a function of conflicting flow rate, critical gap, and follow-up time for minor movement. The model is expressed below (HCM, 2010).

$$C_{p,x} = \frac{V_{c,x}\, e^{-\frac{v_{c,x} t_{c,x}}{3600}}}{1 - e^{-\frac{v_{c,x} t_{f,x}}{3600}}} \tag{2.1}$$

Where,

$C_{p,x}$ = Potential capacity of movement x (veh/h),

$V_{c,x}$ = Conflicting flow rate for movement x (veh/h),

$t_{c,x}$ = Critical headway for minor movement (s), and

$t_{f,x}$ = Follow-up headway for minor movement x (s).

The HCM (2010) also provides the method to compute the critical gap and follow-up time. The critical gap depends on base critical headway, adjustment factors for heavy vehicles and heavy vehicle percentage, the adjustment factor for grade and percent grade, and the adjustment for intersection geometry. Similarly, the follow-up time depends on the base follow-up time, and the adjustment factor for heavy vehicles and heavy vehicle percentage. The HCM offers the tables to provide base critical headway and base follow-up headway values for TWSC.

Studies (Liu, John, Hu and Sokolow, 2008; Liu et al., 2006; Al Maseid, 1999) have utilized this concept when determining the capacity of the U-turn maneuver at median openings. In this context, Al Maseid (1999) used empirical and gap acceptance approaches to estimate the capacity of U-turn openings. He collected data in the country of Jordan in two sets; first, for capacity estimation models, and second, for critical gap and move up time models. He developed linear and exponential capacity models, where capacity was a function of conflicting flow. He also expanded his linear model to account for flow per lane. Al Maseid developed a delay equation where total delay was the function of conflicting flow, and a linear model for critical gap, which was the function of average total delay and the conflicting traffic stream. In the move up time model, the move up time was the function of average total delay. The equations are expressed as follows (Al Maseid, 1999):

$$C = 1{,}545 - 790\, e^{\frac{q}{3600}} \tag{2.2}$$

$$TD = 6.6\, e^{\frac{q}{1{,}200}} \tag{2.3}$$

Where,

C = Capacity of U-Turn movement (veh/hr),

q = Conflicting traffic flow (veh/hr), and

TD = Average total delay for U-Turning vehicles at median openings (s).

Al Masaeid (1999) compared the capacity from his equation with the capacity from Siegloch's equation (based on gap acceptance approach). The results were similar with 95% confidence level. However, other literature (Liu, 2006; Liu et al., 2008) has criticized these equations for use in U.S. because that study was based on data collected in Jordan. Similarly, Liu (2006) and Liu et al. (2008) estimated the potential capacity for U-turn movement at median openings on multilane highways by performing an estimation of critical gap and follow-up time for U-turning passenger cars at median openings. They collected data from six selected median openings in Tampa Bay, Florida. The critical gap for U-turns was estimated by using the maximum likelihood method and the follow-up time predicted by the linear regression equation developed in this study. The capacity estimation was done using Harder's equation. The mean critical headways were estimated as 6.9 and 6.4 s for narrow median openings (median nose width ≤ 21 ft. m) and wide median openings (median nose width ≥ 21 ft) openings (Liu, 2006; Liu and Lu, 2007; Liu et al., 2008). The study found that median width is a significant parameter affecting capacity, such that the U-turn movement on wide medians has larger potential capacity than U-turn movement with narrow median openings. The study further tested Harder's models with the capacity obtained from field data using Kyte's method (Kyte, Clemow, Mahfood, Lall and Khisty, 1991). The results showed that Harder's model provided reasonable capacity estimates for U-turn movement at median openings. Later in 2012, Liu, Qu, Yu, Wang and Cao also developed a VISSIM simulation model for U-turns at un-signalized intersections. They collected behavioral features, such as priority rule, lane selection, and turning speed of U-turns from field studies at ten sites. They compared the capacity calculated from a calibrated VISSIM model with the HCM method and field-measured capacity. They found that the VISSIM simulation yielded mean absolute percentage errors of 17.6% and 20.7% for four-lane and six-lane roadways, respectively.

Recently, Obaidat and Elayan (2013) studied gap acceptance behavior of drivers at four U-turn openings in four-lane divided highways in Jordan and developed models. The first model was developed to estimate the time gap accepted by drivers. In the model, the time gap accepted by drivers was the function of driver's age, gender, and waiting time. The second model was developed to estimate the probability of accepting the gap. In this model, the turning choice was the function of accepted gap lengths, driver's age, driver's gender, and waiting time. The models showed that the accepted gap length decreased with waiting time and the presence of a male driver. It increased with the presence of older driver groups. The turning function decreased with the presence of a young age group, but increased with the length of gap, waiting time, and presence of male drivers. In the same context, Yang (2002) studied the capacity estimation of U-turn movements at median openings using CORSIM to quantify the relationship between the capacity and conflicting flow rate for the U-turn maneuver. The study showed that there is an exponential relationship between the potential capacity and conflicting traffic volumes. The obtained relationship was consistent with the curves drawn based on the HCM 2000. The number of though lanes increased the U-turn capacity. Although the quantitative effects were not provided, the study also mentioned that distance between the U-turn bay and a downstream intersection has effect on potential capacity. Similarly, Jenjiwattanakul, Sano, and Nishiuchi (2013) evaluated the HCM 2010 gap acceptance model and proposed the adjustment method based on a volume to capacity ratio. They adjusted the potential capacities by increasing the capacity of one stream and decreasing capacity of the other stream so that the volume to capacity ratio of conflicting traffic and U-turn traffic was equal. They plotted a graph showing field capacity against the capacity estimated by the gap acceptance model, and against the capacity adjusted by balancing the volume to capacity ratio. They found that these were equivalent. The capacity estimation by the gap acceptance model was found to be systematically overestimating or underestimating the field capacity. Shihan and Mohammed (2009) used a U-SIM model to investigate the effect of five parameters: (1) gap acceptance behavior, (2) gap forcing behavior, (3) effect of opposing and advancing turning flow, (4) difference between left turn and U-turn behavior, and (5) median storage lanes. The median opening was of the bi-directional type. The result showed that with the increase in the U-turn percentage of the opposing traffic stream, the delay of U-turning vehicles in the advancing approach decreases. The study mentioned that gap forcing behavior may persist in median U-Turns. This will have an effect on safety and traffic performance. The total delay, average delay, and average queue length per vehicle were found to be positively affected by increased flow levels. The study mentioned that the U-turn maneuver is more complicated than the left turn maneuver because the drivers need larger gaps to complete the maneuver.

Al-Omari and Al-Akhras (2014) developed a delay model for un-signalized MUT at suburban four-lane divided highways. They collected data during the AM, noon, and PM peak periods on sunny days with dry pavement conditions. The developed model for average turning traffic stop delay (sec/veh) showed that turning traffic volume and conflicting traffic volume are significant parameters in predicting the delay. Both parameters have a positive effect on the log value of delay. Regarding the use of different U-turn capacity models, Aldian and Taylor (2001) studied the suitability of traffic models to calculate U-turn capacity. They estimated gap acceptance and move-up time by reducing field data. They tested the capacity with five different models: Tanner's formula, the National Association of Australian State Roads Authorities'

(NAASRA) model, the random platoon Tanner's formula, the modified random platoon Tanner's formula, and Siegloch's method. Using the Chi-square test between estimated capacity and the observed capacity, they found that the Tanner's formula can very reliably determine U-turn capacity.

2.2.1.2 Effect on Traffic Flow

Regarding the effect of U-turns on traffic, Ben-Edigbe, Rahaman, and Jailani (2013) conducted a study about kinematic waves to estimate and compare volume and density per directional flow at and before midblock facilities. They collected volume, headway, speed, and vehicle type data for eight weeks for both directional flows at two sites in Malaysia. Based on empirical analysis of the collected data, they concluded that because of the speed reduction, traffic flow rate will precede kinematic waves. Traffic safety is correlated to kinematic waves, and significant positive kinematics were found at the exit lanes. The U-turning movement at midblock induces shockwaves. Rahaman, Ben-Edigbe, and Hassan (2012) studied the extent of traffic shockwave velocity propagation induced by U-turn facilities on roadway segments, and estimated traffic shockwave velocity propagations for U-turn lanes. The results indicated that the shockwaves produced due to deceleration and diverging are less severe than the shockwaves produced by acceleration and merging. They indicated that shockwave produced by U-turn facilities can cause traffic crashes. Similarly, Combidino and Lim (2010) modelled U-turn traffic flow using a cellular automata model called Nagel Schreckenberrg (NaSch), which is based on microscopic control of car speed and driver behavior. They also checked the model prediction against empirical observations of U-turn traffic. In conclusion, the authors mentioned that U-turns promote the interaction of cars. Although some studies have shown that U-turns provide decreased congestion and increased flow compared to left turns, this study suggested that this could be possible only if there is low car flow and less lane changing maneuvers. At high traffic densities, U-turns make the situation worse instead of reducing congestion. The study recommended using lane separators as a measure to promote minimal traffic interactions. Cellular automaton was also used by Fan, Jia, Li, Tian, and Yan (2013) to study the characteristics of traffic flow at non-signalized T-shaped intersections with U-Turn movements. They found that the average control delay is a good practical means of measuring the performance of an intersection. If the inflow rate is high, U-turns can increase both the range and degree of congestion. U-turn movements in different directions have an asymmetric effect on traffic conditions. Similarly, Carter, Hummer, Foyle, and Philips (2005) studied the operational and safety effects of U-turns at standard signalized intersections. They measured vehicle headway in an exclusive left turn lane at 14 signalized intersections. The regression analysis of saturation flow data showed a 1.8% saturation flow rate loss for every 10% increase in U-turn percentage in the left turn lane. There was an additional 1.5% saturation loss for every 10% increase in U-turns if the U-turning movement was opposed by right turn overlap from the cross street. A study has also indicated that a full median opening is less safe than a directional median opening, but will produce less delay than a directional median opening (Qi, Chen, Liu, and Wang, 2014).

2.2.2 Weaving of Right Turn Followed by U-Turn (RTUT) Maneuver and Optimal Location of Median Opening

The left-turning vehicle from the cross street in MUTs requires multilane changing after it enters the main street and before it U-turns from the median opening. This process is called weaving, and the entire movement is called right turn followed by U-turn (RTUT) (Zhou, Lu, Yang, Dissanayake, and Williams, 2002). RTUT is comprised of following four steps (Zhou et al. 2002):

1. Stopping at a cross street, waiting for a gap, and turning right when a gap is available;
2. Accelerating into the through lane, weaving to the left turn lane, and deceleration to stop at U-turn;
3. Waiting for suitable gap to make U-turn; and
4. Accelerating to get to the operating speed of through vehicles.

The optimal location of the median opening from the main signalized intersection is specified as 660 ft. (AASHTO Green Book, 2011). MDOT advises that the optimal spacing of a median opening from the main intersection is 660 ft.±100 ft. (Information and Geometric Design Guidance Regarding Boulevards, Directional Crossovers, and Indirect ('Michigan') Left-Turns, accessed 2014; Hughes, Jagannathan, Sengupta, and Hummer, 2010). Hughes et al. (2010) mentions that the longer the spacing distance of median opening with respect to the main intersection, the higher the travel time, but there will be less of a probability of main road queues blocking the median opening. A long distance will allow more time for the driver to see and read signs. Hence, there should be a tradeoff between the disadvantage due to additional travel time and the advantage due to the prevention of a spillback effect with the main intersection. The operational aspects related to the weaving associated with the RTUT maneuver and the optimal location of the median opening as presented by past literature are described in detail in in following subsections.

2.2.2.1 Weaving of Right Turn Followed by U-Turn (RTUT) Maneuver

The weaving maneuver in urban and sub-urban arterials where signal spacing is less than 2.0 miles differ from freeway weaving, as the traffic flow is dominated by platoons and interruptions from traffic signals (Zhou, Hsu, Lu, and Wright, 2003). Hence, RTUT is executable on the availability of acceptable gaps between platoons, and only random arrivals or stragglers may be affected by RTUT (Zhou et al., 2003). Weaving speed is positively correlated with weaving length (Zhou et al., 2003). The models developed by Shahia and Choupanib (2009) indicate that the weaving speed of U-turning vehicles is positively affected by free flow speed, weaving length, and volume ratio, which is the ratio of the flow rate of vehicles that move freely in every lane to the flow rate of vehicles that are subjected to an access specific lane. Similarly, the weaving time of U-turning vehicles is affected positively by weaving length and total volume of weaving section, and negatively affected by volume ratio and free flow speed. The delay effect of RTUT as modelled by Zhou et al. (2002) indicates the significance of the two-directional through traffic flow rate, split, and flow rate of RTUT. Likewise, total volumes, RTUT volumes, split, and speed limit affect travel time. Travel time is negatively correlated with speed limit. It is also in agreement with a travel time model developed by Liu and Lu (2007), which indicates that the travel time at weaving sections while making a RTUT increased with offset and decreased with speed. Liu and Lu (2007) also indicate that the travel time is less on four-lane roadways than six or more lane roadways. Zhou et al. (2002) also related the selection of the RTUT maneuver by drivers against direct left turning by empirical relationship, which indicated that it is affected positively with a combination of left-turn inflow rate and major road through traffic flow rate, but negatively affected by a split. The study also indicated that travel time performance for the RTUT maneuver is better compared to that of a direct left turn under moderate and high traffic volumes. Drivers prefer to use the RTUT maneuver rather than make a direct left turn under moderate to high traffic volumes (Zhou et al. 2002). In the same context, Lu, Dissanayake, Zhou, Yang and Williams (2001) found that the RTUT maneuver increased with the major traffic flow rate and left turn flow rate from major roads. Based on empirical models developed using data collected from ten sites, delay and travel time related to the RTUT maneuver were found to be less than that related to direct left turns. There was less speed reduction on the major street due to RTUT as opposed to a direct left turn, and the running time of RTUT was found to increase linearly with weaving distance. The before and after study conducted by Lu et al. (2001) indicated 15-22% less delay related to RTUT when RTUT was forced by placing directional median openings downstream instead of allowing them for direct left turning. A similar conclusion was obtained in a study by Yang and Zhou (2004), which indicated that for a higher through volume, the delay and travel time would increase at a faster rate for a direct left turn rather than a RTUT movement. The same study also indicated that RTUT movement can perform better for a wide range of traffic conditions. In an attempt to evaluate U-turns as an alternative to a direct left turn, Liu et al. (2006) found that the RTUT maneuver at a median opening produced 24 seconds less delay than a direct left turn for 6-8 lane roadways. The average running time of a RTUT depends on the offset between the driveway and the downstream median opening, and decreases with the speed limit of the major road. The percentage of drivers selecting RTUT increases with the upstream through traffic, left turn volume from the major street, and total left turning volume at the driveway.

2.2.2.2 Optimal Location of Median Opening

The optimal weaving length minimizes the average waiting delay of the U-turning vehicle at the median opening. This is possible when a RTUT vehicle arrives at a median opening at the time the last vehicle in the platoon of opposite traffic passes the median opening (Zhou et al., 2003). A deterministic model that estimates the optimal weaving length as developed by Zhou et al. (2003) is expressed below:

$$L = \frac{-\left(1 - .082\Delta t + \frac{21.5}{v}\right) + \sqrt{\left(1 - 0.082\Delta t + \frac{21.5}{v}\right)^2 + 7.05\frac{\Delta t}{v}}}{\frac{0.164}{v}} \tag{2.4}$$

Where,

L= Optimal weaving length,

Δt = Term expressed as a function of offset of upstream and downstream signal timing, whole section length, distance between driveway and upstream signalized intersection and posted speed limit, and

v = speed limit.

The above equation indicates that the optimal weaving length is governed by (1) the offset of signal timing between upstream and downstream signals, (2) the distance between upstream and downstream signalized intersections, (3) the distance between the subject driveway and the upstream signalized intersection, and (4) the posted speed limit.

For a RTUT maneuver, Lu, Pirinccioglu, and Pernia (2005) related the offset distance between driveway exits and downstream U-turn median openings with crash and conflict rates. The developed model in the study indicated a positive correlation between the location of the median opening and the crash rate. The details of this literature will be dealt in the safety literature review of MUT. Similarly, Liu and Lu (2007) evaluated the offset of a U-turn bay and upstream driveway based on crash data analysis and travel time analysis. Based on safety and operational performance, the study identified the minimum offset for U-turn bay located at median opening as 350 ft. and 450 ft. for four-lane and 6-8 lane roadways.

2.2.3 Operational Performance of MUT

The major objective behind the implementation of unconventional intersection treatments, including MUT, is the expected improvement of the operation of existing conventional intersections (Hughes et al., 2010). MUT is thought to have more operational benefits compared to other intersections, but under specific traffic conditions. For example, Hummer (1998-B) mentioned that MUT is applicable in arterials as an intersection treatment for low to medium left turns from the arterial, low-to medium left turns from a minor street, any minor street through volume, and at the availability of minimum right of way 30 ft. wide. In this context, this subsection discusses the operational performance of MUT compared to conventional intersections or other unconventional intersections, the criteria and conditions of traffic parameters at which the MUT operates at better level, and the condition and criteria at which MUT cannot be a good alternative based on the literature review. Review notes of prominent literature that is directly related to operational performance of MUT are shown in table 2.1, table 2.2, table 2.3 and table 2.4.

2.2.3.1 Travel Efficiency of MUT

In this subsection, the performance of MUT in terms of travel time, stops, and average network speed are discussed based on past literature. In this context, Hummer and Boone (1995) studied the travel efficiency of MUT by comparing its travel time performance with conventional intersections and other unconventional intersections, such as a continuous green T-intersection (CGT) and Bowtie, in terms of travel time and percentage of stops and stop delay. The study indicated that MUT is more travel efficient at a through volume range between 400 and 700 vph. The MUT displayed superior performance in terms of overall travel time and stops in terms of through volume, but at the expense of travel time and stopped delay for left turning traffic, which were inferior to a conventional intersection. Boone and Hummer (1995) validated the use of CORSIM for an operational performance evaluation of MUT using a larger saturation flow rate as a calibrating parameter. The travel time and delay obtained from the simulation matched satisfactorily with observed site value. Later, Reid and Hummer (1999) used CORSIM to compare the performance of a MUT with signalized median openings to a superstreet and a conventional two way left turn lane (TWLTL) design. The MUT decreased the system travel time by 17.25% as compared to a conventional intersection, and the average speed of an arterial was found to be 24.74% faster than a conventional intersection. However, the average number of stops per vehicle increased by 5.56%. Compared to a superstreet, MUT performed better in terms of average speed and average number of stops per vehicle. However, the system travel time was slightly higher than that of superstreet. Again in 2001, Reid and Hummer compared the performance of seven unconventional intersections, including a MUT with conventional intersections. He considered seven intersections, comprised of these major and minor street configurations: 4x4 (2 nos.), one 4x2 (1 nos.), 6x4 (1 nos.), 6x2 (1 nos.) and 8x4 (2 nos.). Compared to conventional intersections, the most MUT could reduce the total time by 27.94%. However, MUT increased the total time in some intersections, and the highest increase of all seven intersections was 8.57%. Similarly, the highest average percent of stops that MUT could decrease compared to conventional intersections was 20.97%, however, it increased the highest average percent of stops in some intersection, the highest being by 28.4%. This result clearly shows that MUT can perform better in terms of travel efficiency, but not in terms of stops compared to conventional intersections. Similarly, Henderson and Statamatiadis (2001) studied the travel efficiency of MUT along a principal urban arterial in Kentucky. The study used CORSIM to evaluate different alternatives, including signal optimization for the existing intersection, the addition of a lane, and the implementation of MUT. A set of four intersections were studied for the implementation of MUT, and the results indicated a significant improvement on the operation of the network. Considering the whole system, speed increased by 2.4 mph and the move to total time ratio increased to 0.41. The MUTs were implemented at an additional two intersections, resulting in a total system-wide effect of a speed increase of 3.6 mph and a move to total time ratio increase by 26% (0.44).

The travel efficiency of MUT was also praised in a study by Bared and Kaisar (2002). They compared a partial MUT where direct left turns from minor streets were allowed with a conventional intersection with single and dual left turn lanes. The study found an abrupt rise in travel time savings, from 10 to 40 seconds/vehicle with 600 vph entering the flow with 10% left turns compared to a conventional intersection with a single left turn lane. The travel time saved by MUT rose to 60 seconds/vehicle for 20% left turning volume at 600 vph entering flow compared to a conventional intersection with a single left turn lane. The travel time saved by MUT was smaller at a total entering flow of 5500 vph to 6600 vph, but larger saving starts from 6600 vph for 20% left turning volume as compared to a conventional intersection with dual left turn lanes. The average travel time for U-turning traffic was 20 s/veh to 30 s/veh higher because of circuitous movement. The average proportion of stops was 20% to 40% lower for MUT for 10% left turning traffic and for 20% left turning traffic. A noticeable reduction starts at about 4,500 vph, compared to a conventional intersection with a single left turn lane. Similarly, a report developed for the Community Planning Association (COMPASS) of Southwest Idaho (2008) indicated that MUT can reduce travel time by 2% to 20% indicated that MUT can reduce travel time by 2% to 20% during non-peak hour traffic and 6% to 21% during peak hours. However, the performance of MUT for stop varies with 20% reduction to 70% increment

Table 2.1 Operational performance of MUT (Part I)

Literature	Intersection Type	Delay	Speed	Stop	Travel Time
Hummer and Boone (1995)	MUT	Stopped delay for left turning traffic increased		Less for through traffic but high for left turning traffic.	Travel efficient at through volume range 400 to 700 vph. Travel time performance of left turning traffic was inferior to conventional intersection.
Reid and Hummer (1999)	MUT		Average speed increased by 24.74%	Average number of stops increased by 5.56%	System travel time is decreased by 17.25%.
Reid and Hummer (2001)	MUT (Signalized) on 4 by 4 lanes	Average total time decreased by 0.71%		Average percentage stop increased by 28.12%	
	MUT (STOP) on 4 by 2 lanes	Average total time decreased by 27.94%		Average percentage stop increased by 20.97%	
	MUT (Signalized) on 6 by 4 lanes	Average total time decreased by 19.44%		Average percentage stop increased by 18.09%	
	MUT (STOP) on 6 by 2 lanes	Average total time decreased by 9.43%		Average percentage stop increased by 10.29%	
	MUT (Signalized) on 8 by 4 lanes	Average total time decreased by 8.99%		Average percentage stop increased by 7.46%	

Table 2.2 Operational performance of MUT (Part II)

Literature	Intersection Type	Intersection Capacity with respect to Conventional Intersection	Stop	Travel Time
Bared and Kaisar (2002)	MUT (with minor street left turning allowed)		The average proportion of stops was 20 to 40% lower for MUT for 10% and 20% left turning traffic and for 20% left turning noticeable reduction started at about 4,500 vph compared to conventional intersection with single left turn lane.	The travel time saving rised from 10 to 40 sec/vehicle for 10% left turning volume at 600 vph entering flow than ordinary intersection with single left turn lane.
				The travel time saving increased to 60 seconds/vehicle for 20% left turning volume at 600 vph entering flow than ordinary intersection with single left turn lane.
				The travel time saving was smaller at total entering flow of 5500 vph to 6600 vph and larger saving starts from 66,00 vph for 20% left turning volume than ordinary intersection with dual left turn lane.
				Average travel time for U-Turning traffic was 20 to 30 sec/veh higher because of circuitous movement.
COMPASS (2002)	MUT, CFI, jughandle	MUT can increase 50% or more capacity than a comparable double left turn	MUT operates with 20% reduction to 70% increment during non-peak hours and 2% reduction to 30% increment during peak hours	Travel time can be reduced by 2-20% during non-peak hours and by 6-21% during peak hours
Dorothy et al. (1997)	MUT			For 10% left-turning volumes, MUT with signalized median opening has lower travel time by 20 sec/vehicle, 40 sec/vehicle and 150 sec/vehicle at 30%, 50% and 70% mainline saturation, respectively than two way left turn lanes.

Table 2.3 Operational performance of MUT (Part III)

Literature	Intersection Type	Intersection Capacity with respect to Conventional Intersection	Delay	Travel Time
Jagannathan (2007)	MUT	MUT on four lanes and six lanes increases the capacity of the intersection by 20 to 50% than two way left turn lane.		
Savage (1974)	MUT	MUT can increase the corridor capacity by 20 to 50%.		
Hilderbrand (2007) Thesis	MUT, CFI, jughandle		At major street volumes 1000, 2000 and 3000 vph and minor street volumes 626 vph, MUT reduced delay by 82, 98 and 99%, respectively.	
Tarko et al. (2010)	CFI, jughandle and MUT		At total intersection volume from the range 1,000 vph to 3,000 vph, the intersection delay range related to a conventional intersection was 25 to 70 sec/veh and the intersection delay range related to MUT was 20 to 55 sec/veh.	MUT lowered travel time significantly than conventional intersection at saturated condition.
Hughes et al. (2010) (FHWA AIIR)	CFI, jughandle, MUT	MUT increased throughput of 15 to 40% than conventional intersection.		

Table 2.4 Operational performance of MUT (Part IV)

Literature	Intersection Type	Intersection Capacity with respect to Conventional Intersection	Delay
Autey et al.(2010)	MUT		MUT with un-signalized median opening performs better or at least equivalent to signalized MUT.
			For light volumes (up to 1100 vph), un-signalized MUTs performance is very good on balanced condition.
			Both MUTs with signalized and un-signalized median opening can't accommodate heavy left turning volume and high approach volume.
Kivlins and Naudzuns (2011)	MUT	MUT ensures good intersection capacity for left turning traffic flows on both arterial and cross street.	MUT ensures 73% less vehicle waiting time than arterial.
El Esawey and Sayed (2011)	MUT	MUT increased the capacity of intersection by 10% and 8% than conventional intersection in case of signalized median opening and un-signalized median opening.	
Synthesis of MUT Intersection Treatment (2009)	MUT	MUT increases the vehicle throughput ranging from 20-50%	

Table 2.5 Operational performance of MUT (Part IV)

Literature	Intersection Type	Intersection Capacity with respect to Conventional Intersection	Delay	Speed	Stop	Travel Time
Tarko et al.(2010)	CFI, MUT, jughandle		At total intersection volume from range 1, 000 vph to 3,000 vph, the intersection delay range related to conventional intersection was 25 to 70 sec/veh and the intersection delay range related to MUT was 20 to 55 sec/veh			MUT lowered travel time significantly than conventional intersection at saturated condition.
El Esawey and Sayed (2011), El Esawey and Sayed (2011)	MUT	Under balanced condition, MUT increased the capacity up to 56% better than conventional intersection.				
		The capacity of MUT intersection was about 8% to 10% higher than that of the conventional intersection.				
Reid and Hummer (1999)	CFI, jughandle, MUT Tested for two 4 X 4, one 4 X 2, two 6 X 4,one 6 X 2 and one 8 X 4 lane configurations of major and minor street		MUT reduced system time by 3.318%.	Average speed was increased by 24.49%.	Average number of stops per vehicle was increased by 5.56%. The highest average percentage of stops that MUT could decrease compared with conventional intersection was 20.97%. However, it increased the average percentage of stops in some intersection and the highest increment of all 7 intersections was by 28.4%.	The highest total time that MUT could decrease compared with conventional intersection was 27.94%. However, it increased the total time in some intersection and the highest increment of all 7 intersections was by 8.57%.

during non-peak hours and 30% increment during peak hours. Tarko et al. (2010) indicated that MUT lowered travel time significantly compared to a conventional intersection at saturated condition. Similarly, Kivlins and Naudzuns (2011) indicated that MUT could reduce the vehicle waiting time by 73%. They also indicated that the left turn lanes on an arterial intersection of a MUT intersection can produce 90% shorter congestion length at the end of green signal phase than a conventional intersection.

A study (Martin, Islam, Best, and Sharma, 2012) also evaluated MUT in combination with other alternatives. That particular study, about US 290 Corridor, led to the development of three alternatives. Alternative 1 was a combination of MUT, CFI, and traditional improvements, such as the addition of lanes. Alternative 2 was the combination of MUT and CFI. Alternative 3 was combination of MUT, CFI, and a superstreet. They used a VISSIM simulation to calculate project annual delay savings and travel time for the existing year 2010 and the future years 2015 and 2020. The results showed that Alternative 1 and Alternative 2 have significant travel time savings during the peak hour: 50% and 45% for year 2010, 45% and 30% for year 2015, and 28% and 33% for year 2020. Similarly, Pirdavani, Brijs, Bellmans and Wets (2011) estimated travel time and compared it with conventional intersections. They used a raised U-turn facility that restricted a left turn from the major street, and through and left from minor street movements. They modelled travel time based on several scenarios of the offset distance of the U-turn with a signalized intersection, traffic volume on minor and major approaches, as well as left turn percentages using AIMSUN. The results indicated that travel time for the U-turn facility for the through maneuver was always less compared to a conventional intersection. However, for left-turning vehicles on the major street, the travel time for the U-turn facility was slightly higher than a conventional intersection, but it was still less for left-turning vehicles from the minor street. Regarding the MUT with a signalized median opening, Dorothy et al. (1997) found that for 10% of left-turning volumes, MUT with a signalized median opening had lower travel time by 20 sec/vehicle, 40 sec/vehicle, and 150 sec/vehicle at 30%, 50% and 70% mainline saturation, respectively, than two way left turn lanes.

2.2.3.2 Capacity and Delay Efficiency of MUT

The reduction in the signal phases at the core intersection of MUT is explained as the reason it has an increased capacity compared to a conventional intersection (FHWA Tech brief: Synthesis of MUT Intersection Treatment, 2009). Savage (1974), and Synthesis of MUT Intersection Treatment (2007) indicated that the use of directional median openings can increase the capacity by 20%-50%. Jagnnnathan (2007) also stated that MUT on four or six-lane roads increases the intersection capacity by 20% to 50% more than a two-way left turn lane intersection. Hughes et al. (2010) mentioned that MUT can increase the throughput to 15% to 40% more than conventional intersections. Similarly, the Community Planning Association (COMPASS) of Southwest Idaho (2008) indicated that MUT can increase capacity by 50% or more compared to a conventional double left turn intersection. Similarly, Zhao, Ma, Head, and Yang (2014) measured the improvement in intersection capacity with five types of MUT with signalized median openings, including (1) a MUT with left turn prohibition from the major street, (2) a MUT with left turn prohibition from the minor street, (3) a through movement prohibition from the minor street, (4) a left turn prohibition from the major street and through movements from the minor street (5) a left turn and through movement prohibition from the minor street and (6) a left turn prohibition from the major street and through movements from the minor street. All the MUTs performed better than a conventional intersection, especially under high traffic demand. The average and highest improvements in capacity were 21% and 62.6%, respectively. The study also concluded that type 3 and type 4 MUTs are not good MUT alternatives.

A study by Henderson and Stamatiadis (2001) about the implementation of MUT along a principal urban arterial in Kentucky indicated that the average and total delay time for the whole system decreased by 28% and 24% with the implementation of MUT at four intersections. Again, with the implementation of MUT on two additional intersections, the average delay and total delay were decreased by 36% and 32%, respectively. Martin et al. (2012) found that the delay savings for the peak hour, when calculated for the existing year 2010 and using the MUT/CFI alternative, was 61%, and the delay savings for the MUT/CFI/superstreet alternative was 65%. Further, the savings for the MUT/CFI alternative and MUT/CFI/superstreet alternative for 2015 was 49% and 45%, respectively. Finally, the delay savings calculated for 2020 for these alternatives were 23% and 17%, respectively. In his thesis, Hilderband (2007) indicated that at major street volumes of 1000, 2000 and 3000 vph, and minor street volumes of 626 vph, MUT reduced delay by 82%, 98% and 99%, respectively. Tarko et al. (2010) indicated that at total intersection volume from the range 1,000 vph to 3,000 vph, the intersection delay range related to conventional intersections was 25 to 70 sec/veh, and the intersection delay range related to MUT was 20 to 55 sec/veh. A study by Autey, Sayed, and El Esawey (2010) indicated that un-signalized MUTs exhibit lower delay than signalized MUTs, but neither could accommodate an approach volume of more than 1500 vehicles/hr with 20% left turn traffic for a balanced traffic flow. Similarly, for an unbalanced traffic flow condition, they found that un-signalized MUTs outperform signalized MUTs for the two left turn volume condition of 20% and 30% at a major street volume level of 1200 vph.

El Esawey and Sayed (2011a) and El Esawey and Sayed (2011b) assessed the performance of two types of MUT against the conventional four legged intersection. Type 1 MUT prohibits left-turning traffic only from major and minor streets and is pre-timed signal controlled at the core intersection. Type 2 MUT prohibits all movement of the minor street and the left turn only movement of major street. The core intersection is not controlled with a signal. They used Synchro to optimize the signal and VISSIM for simulation, and tested the performance of the three intersections, including a conventional intersection under balanced and unbalanced traffic flow condition in terms of average intersection control delay and overall capacity of the intersection. The result indicated that Type 1 MUT performed better than Type 2 MUT. Type 2 MUT was beneficial only in very low demand approach volumes and left turning volumes. The capacity of Type 2 MUT was lower than a conventional intersection by 27% under balanced volume conditions. However, the capacity of Type 1 MUT with signalized and un-signalized median openings was 8% to 10% higher than a conventional intersection. Under unbalanced conditions, the Type 2 MUT exhibited higher delays in most cases except at a very light volume for the cross street. Type 1 MUT outperformed all other intersections in terms of exhibiting the lowest delay. For two traffic conditions with 20% and 30% left turners, Type 1 MUT with un-signalized median openings reduced delay from 3 to 6 sec/veh to 8 sec/veh. Similarly, Topp and Hummer (2005) compared the operations between traditional MUT designs at median openings along an arterial to the new MUT design that has median openings on the minor street. They used CORSIM for the simulation. The results found that the MUT with median openings at the cross street provides better operation in terms of stops, total time, and delay for most of volume combinations. They mentioned that the cross street can better accommodate left turns from a median opening because of the capacity available due to low volume. This also decreases the likelihood of stopping and queue interference with heavy traffic flow.

2.2.4 Safety Performance of MUT

A MUT intersection possesses a total of 16 conflict points, including 12 merging/diverging conflict points, 0 crossing (left turn) conflict points, and 4 other crossing points, compared to four-legged standard signalized intersection, which has 32 conflict points (Kivlins and Naudzuns , 2011). This indicates that MUT should have safety benefits compared to conventional intersections. In this section, the safety performance of MUT is described based on past studies. The literature review notes from prominent literature that deals directly with MUT-related safety are shown in tables 2.6 and 2.7.

"Information and Geometric Design Guidance Regarding Boulevards, Directional Crossovers, and Indirect ('Michigan') Left Turns" (1995) indicated that MUTs result in a reduction of 60.6% total crashes, 74.6% injury crashes, 17.1% rear end crashes, 95.5% angle crashes, and 60.6% sideswipe crashes. Similarly, Kach (1992) did comparative crash study between directional (MUT) and bi-directional signalized intersections with an objective to establish directional crossovers as a safety treatment. He studied about 15 directional signalized intersections and 30 bi-directional signalized intersections. The study identified key types of crash reduction for signalized directional intersections: angle straight by 53.1%, rear end left turn by 9.3%, sideswipe opposite by 0.3%, and head-on left turn by 35.4%. However, the treatment increased some crashes as fixed object by 7% and rear end by 91.4%. In sum, the total crash reduction was 48.91%, and total crash gained was 21.8%. The equivalent overall crash reduction was 27.11%. The result of the study showed significant reduction in total, angle and right turn, and left turn collisions. Similarly, the result also indicated significant reduction in fatal and injury collisions. Based on analysis of data collected data over 115 median openings including 105 median openings in urban arterials and 12 median openings in rural arterials, Potts, Harwood, Torbic, Richard, Gluck, Levinson, Garvey, and Ghebrial (2004) estimated average of 0.41 U-Turn plus left-turn accidents per median openings per year in urban arterial and 0.2 accidents per median opening per year in rural arterial. U-Turns represent 58% of the total median opening movement and the left turn represents 42% of the median opening movement. The study found no major safety concern on the un-signalized median opening. The study also found existence of major road angle collision, major road rear end collision, cross street collision and other or unknown collision. In terms of MUT median opening, the cross street collision is not applicable.

Mallah (2011) developed guidelines for the full opening median treatment at un-signalized intersection using predicted conflict rate as the safety measures. They collected segment geometric data, traffic volumes and traffic conflict data. They calculated conflict rates both from observations and from the model prediction. In the developed conflict rate models, the prediction of direct left turn conflict was positively governed with major road traffic volume in one direction, major road traffic volume in the opposite direction, the direct left turn volume, left turn in driveway volume and left turn in opposite driveway volume. Similarly, Qi et al. (2014) investigated about the operational and safety impacts of directional median openings on urban roadways with the help crash analysis and simulation based study. They found from the crash analysis that full median openings pose more safety and operational hazard, and hence should be avoided. To maintain the good safety condition, sufficiency in median width is very necessary. From the simulation study, it was found that when a full median opening is converted to a directional median opening, crossing conflicts will be greatly reduced, but there will be a slight

Table 2.6 Safety performance of MUT (Part I)

Literature	**Alternative Intersection**	**Safety Performance**
"Information and Geometric Design Guidance Regarding Boulevards, Directional Crossovers, and Indirect ('Michigan') Left Turns" (1995)	MUT	Reduction in total crash was 60.6%.
		Reduction in injury crash was 74.6%.
		Reduction in rear end crash was 17.1%.
		Reduction in sideswipe crash was 95.5% and 60.5% of sideswipe crashes.
Kach (1992)	MUT	Reduction in angle straight was 53.1%.
		Reduction in rear end left turn crash was 9.3%.
		Reduction in sideswipe opposite crash was 0.3%.
		Reduction in head on left turn crash was 35.4%.
		Increment in fixed object crash was 7%.
		Increment in rear end crash was 91.4%.
		The total crash reduction was 48.91%, total crash gained was 21.8% and the equivalent overall crash reduction was 27.11%.
Potts et al. (2004)	MUT Median Opening	Average of 0.41 U-turn plus left-turn accidents per median openings per year in urban arterial and 0.2 accidents per median opening per year in rural arterial were estimated. U-turns represent 58% of the total median opening movement and the left turn represents 42% of the median opening movement.
Carter et al. (2005)	MUT Median Opening	U-turn collisions were high for the sites with dual left turn lanes, protected right turn overlap, or high left turn and conflicting right turn traffic.
Pirinccioglu et al. (2006)	MUT Median Opening	The study at signalized intersection indicated that direct left turn produce two times more conflict per hour than RTUT. RTUT-related conflicts were less severe than direct left turn related conflicts. The study at median opening sites indicated 10% more direct left turn related conflict per hour than RTUT related conflicts. The conflict rate (per 1,000 vph) was 62% higher for RTUT than direct left turns.

Table 2.7 Safety performance of MUT (Part II)

Literature	**Alternative Intersection**	**Safety Performance**
Lu et al. (2005)	MUT (RTUT Movement)	The comparison showed RTUT generates 3 times less conflicts than direct left turn in terms of conflict per hour than direct left turn. In terms of conflict rate considering volume, RTUT showed 35% lower conflict than direct left turn on six or eight lane roads. Overall severity related to RTUT conflicts were 37% lower than that of direct left turn.
Xu (2001)	MUT (RTUT Movement)	RTUT reduced the total crash by 24% and injury/fatal crash by 32% than direct left turn for six lane divided arterials.
Mauga and kaseko (2010)	Median opening	Each reduction of one median opening per mile would result in crash reduction by 4.7%.
Chen et al. (2014)	Median Opening	Shorter the length of left turn lane higher is the crash modification factor.

increase in lane change conflict. In his dissertation, Liu (2006) evaluated the operational effects of U-turn movement on multilane roadways by gathering the crash histories of 179 roadway segments in central Florida. In order to complete the RTUT movement, the types of crashes during weave as explained by the study were angle/right turn crashes, sideswipe crashes, and rear end crashes. The study found that the crash rate at weaving sections decreases with the increase of the separation distance and travel time. Carter et al. (2005) analyzed the crashes of 78 intersections, 24 of which had problems with U-turns. No collisions were found in 65 sites. The remaining 13 sites had 0.33 to 3 collisions/year. U-turn collisions were high for the sites with dual left turn lanes, protected right turn overlap, or high left turn and conflicting right turn traffic.

Pirinccioglu, Lu, Liu, and Sokolow (2006) evaluated the safety of RTUT on four-lane arterials at signalized and un-signalized median openings by collecting data from 16 sites. Nine types of conflict were used; five of them were related to RTUT and four of them were related to direct left turns. The study at signalized intersections indicated that a direct left turn produced two times more conflict per hour than RTUT, but the conflict rate that takes volume into account was 5% higher for RTUT than direct left turns. RTUT-related conflicts were less severe than direct left turn related conflicts. The study at median opening sites indicated 10% more direct left turn related conflict per hour than RTUT-related conflicts. The conflict rate (per 1,000 vph), which takes traffic volumes into account, was 62% higher for RTUT than direct left turn. RTUT-related conflicts were less severe than direct left turn related conflicts. Lu et al. (2005) did a safety evaluation of RTUT and direct left turns at signalized intersections by comparing traffic conflicts. Nine types of conflict were studied, and five of them were related to RTUT movements while the rest related to direct left turns. They used two types of conflict rates: conflict per hour and conflicts per thousand vehicles. The comparison showed RTUT generates three times less conflicts than direct left turn in terms of conflict per hour. The conflict rate per volume showed that RTUT had 35% less conflicts than the direct left turn on six- or eight-lane roads. Overall severity related to RTUT conflicts were 37% lower than that of the direct left turn. Regarding conflict modeling, Agarwal (2011) estimated pedestrian safety at intersections using simulations and SSAM. He found that the conflict patterns of MUT intersections are similar to that of conventional signalized intersections.

Xu (2001) studied the safety effects of RTUT at directional median openings by conducting a before-and-after period crash analysis. The study found that RTUT can reduce the total number of crashes by 24% and injury/fatality crashes by 32% compared to direct left turns for six-lane divided arterials. Liu and Lu (2007) evaluated the offset of U-turn bays and upstream driveways based on crash data analysis and travel time analysis. The crash analysis was performed over 179 selected roadway segments, and travel time data was collected at 29 selected sites in Tampa Bay, Florida. The study mentioned that RTUT-related crashes involve angle/right turn crashes, sideswipe crashes, and rear end crashes. The model developed for crash rates on four-lane and 6-8 lane roadways (crashes/mvm) indicated that the crash rate will increase if the U–turn bay is located at a signalized intersection rather than a median opening. The crash rate was significantly affected by the offset between the driveway and U-turn bay. Based on safety and operational performance, the study also identified the minimum offset for U-turn bays located at a median opening as 350 ft. and 450 ft. for four-lane and 6-8 lane roadways. The minimum offsets were 500 ft. and 750 ft. for a U–turn at a signalized intersection on 6-8 lane roadways.

Mauga and Kaseko (2010) evaluated the safety effect of access management features in the midblock section while considering two types of median treatments, raised medians and two-way left turn lanes. The study indicated that each reduction of one median opening per mile resulted in a crash reduction of 4.7%. There has also been a study about the safety effects of length of left turn lanes at un-signalized median openings. In this context, Chen et al. (2014) developed a relationship between crash modification factors and relative length of left turn lane at un-signalized median openings with respect to the AASHTO Greenbook. The relation showed that the shorter the length of left turn lane, the higher the crash modification factor.

The public perception of the use of MUT was studied by Strickland (2012). On the basis of public comments received by the Cobb County Department of Transportation (CCDOT), the majority of the public have accepted the MUT design. However, there were some comments against the use of MUT, which were either people who were skeptical about the innovative intersection strategy or those who did not like the additional distance the left turners have to travel due to the MUT design. The author concluded his study by emphasizing the need to educate the general public about these new designs.

2.3 The Operation and Safety Performance Related to Continuous Flow Intersections (CFI)

As described in preceding chapter, the CFI operates with one main intersection and four other crossover intersections. All the intersections are signalized. The operational and safety effect brought by reduced traffic signal phases, the provision of displaced left turn lane, and the effect of the whole system are described in the following subsections.

2.3.1 Operational Performance of CFI

Like other unconventional intersections, CFI is applied as an intersection treatment to improve safety and operation. However, the extent of improvements is based on certain traffic conditions. For example, according to the alternative selection criteria developed by Hummer (1998-B), when based on left turn volume from urban and suburban arterials, CFI is applicable for all traffic volume conditions of left turns from arterials, left turns from minor streets, minor street through traffic, and the availability of two 40 ft. by 300 ft. right-of-way rectangles. In this context, this subsection deals with the operational performance of CFI compared to conventional intersections or other conventional intersections under different criteria or conditions. The review notes of prominent literature that are directly related to the operational performance of CFI are shown in table 2.8, table 2.9 and table 2.10.

2.3.1.1 Travel Efficiency of CFI

Travel efficiency represents travel time, average speed, and stops related to the operation of CFI. In this context, Tarko et al. (2008) studied the safety and operational impacts of unconventional intersections. They used VISSIM for operation performance evaluation. The study showed that at total intersection volume levels of 1730 vph, 2250 vph, and 3110 vph, CFI reduced stops by 35.56%, 7.5%, and 60.17%, respectively. Reid and Hummer (2001) compared the travel time of unconventional intersections, including CFI, by performing a simulation with traffic signal with optimal cycle length using peak hours, off peak hours, and peak hours with 15% added volume levels. The unconventional intersections were found to have lower travel time than conventional designs. Similarly, Yang, Chang, Rahwanji, and Lu (2013) studied CFI to identify queue spillback locations and develop a set of planning stage models. They also compared the operational performance of CFI to that of conventional intersections. The reductions in total travel time (hour) were 15.1% and 14.7% in the morning and evening peaks. The average number of stops per vehicle was reduced by 6.3% and 7.8% at the morning and evening peaks.

2.3.1.2 Capacity and Delay and Other Operational Efficiency of CFI

Goldbat et al. (1994) studied the effectiveness CFIs operating under multiphase actuated control. They used a TRAFNETSIM simulation model to compare both conventional and unconventional designs. The result showed that CFI performed better than conventional intersections at capacity, when the demand exceeded capacity, and with the presence of heavy left-turners such that they required left turn protection. It was found that the demand of 2000 vph on each approach exceeded the capacity of conventional designs by 20%. The capacity of CFI was exceeded on all approaches at a demand level of 3000 vph, but the capacity at this demand level was 50% higher than that of conventional intersection. A similar type of conclusion was reached by Berkowitz, Mier, Walter, Bragd (1997) in a study of CFI, which indicated that CFI can accommodate 50% more traffic than a conventional three-legged intersection design at a demand level of 3,000 vph. Simulations showed that CFI delay was 1/5 of conventional intersection delay. Mean speed of CFI was approximately twice the speed at conventional intersection. The signal efficiency increased by at least 80% and fuel consumption and emissions were reduced by 1/3 or more. “Techbrief: Displaced Left turn Intersection” (2009) indicated that a full CFI can increase 30% throughput over a conventional intersection under fully balanced opposing flow in mainlines, and 25% throughput with unbalanced main line volumes. For partial CFI, the increase in throughput was 10% for unbalanced flows and 20% for balanced flows.

Table 2.8 Operational performance of CFI (Part I)

Literature	Intersection Type	Stop	Travel Time
Reid and Hummer (2001)	CFI and other types of Unconventional Intersections		The unconventional intersections were found to have lower travel time than conventional designs.
Tarko et al. (2008)	CFI	At total intersection volume levels of 1730 vph, 2250 vph and 3110 vph, CFI reduces stop by 35.56%, 7.5% and 60.17%	
Yang et al. (2013)	CFI	The average number of stops per vehicle was reduced by 6.3% and 7.8% at morning and evening peak times.	The reductions in total travel time (hour) were 15.1% and 14.7% in the morning and evening peak times.

Table 2.9 Operational performance of CFI (Part II)

Literature	Intersection Type	Intersection Capacity with respect to Conventional Intersection	Delay and Queue	Speed	Fuel Consumption and Emissions
Glodbatt et al. (1994)	CFI	At demand level of 2000 vph, approach exceeded the capacity of conventional design by 20%. At the demand level of 3000 vph, but the capacity of CFI was 50% higher than conventional intersection. Signal efficiency increased at least by 80%.	The delay of CFI was 1/5 of conventional intersection.	Mean speed was approximately twice the speed at conventional intersection.	Fuel consumption and emissions were reduced by 1/3 or more.
FHWA Displaced Left turn Intersection (2009)	CFI	Full CFI can increase 30% throughput over conventional intersections under fully balanced opposing flow in mainlines and 25% throughput with unbalanced main line volumes. For partial CFI, the increase in throughput is 10% for unbalanced flows and 20% for balanced flows.			
Pitaksringkam (2005)	CFI		Queue and delay could be decreased by 64% and 61% respectively during PM peak hour as compared to those of unconventional intersection.		
Liu et al. (2012)	CFI		CFI reduced 42 to 86% delay with respect to conventional intersection. The through vehicle of queue length decreases by 28 to 83% and left turning queue length of CFI decreases 36 to 6%.		

Table 2.10 Operational performance of CFI (Part III)

Literature	Intersection Type	Intersection Capacity with respect to Conventional Intersection	Delay and Queue	Fuel Consumption and Emissions
Jagannathan and Bared (2004)	CFI (Type A: Crossovers on all approaches, Type B: Crossovers on major street approach, Type C: Crossover on one approach of major street		Type A: Reduction in delay was 48 to 50%, Type B: Reduction in delay was 58 to 71%, Reduction in delay in Type C: 19 to 90%	
El Esawey and Sayed (2007)	CFI and Upstream Signalized Crossover (USC)	CFI has higher capacity than conventional intersections by 90%.		
Autey et al. (2013)	MUT, CFI, Upstream Signalized Crossover (USC) and Double Crossover Intersection (DXI)	CFI showed 99% higher capacity than conventional intersection where as USC and DXI showed 50%.		
Yang et al. (2013)	CFI		The performance comparison between CFI and conventional intersections showed 34.2% reduction in average delay per vehicle (sec) in morning peak and 35.1% in afternoon peak.	CFI was found helpful in view point of fuel consumption and vehicle emission.
Park and Rakha (2010)	CFI			CFI could result in energy savings of 5% (from VISSIM) and 11% (from INTEGRATION), respectively. The decrease in HC, CO, and NOx emissions ranged from 1% to 6%.

The delay reduction capabilities of CFIs as assessed by Pitaksringkam (2005) using VISSIM simulation modeling concluded that queue and delay could be decreased by 64% and 61% respectively during PM peak hours as compared to conventional intersections. Cheong et al. (2008) assessed the operational performance of CFI, parallel flow intersections (PFI), and upstream signalized crossover (USC) intersections. The average delay comparison of through only and left-turn only traffic concluded that CFI outperformed other intersections except for some traffic conditions. For example, in balanced conditions, the average delay of through traffic for PFI was smaller than CFI under low traffic volume (1000 vph) conditions, and almost similar under a moderate traffic volume level (1500 vph). Liu, Zhang, and Yang (2012) studied CFI for urban roads, evaluating CFI with a VISSIM simulation. In terms of delay, CFI reduced 42% to 86% delay with respect to conventional intersections. The CFI through vehicle queue length decreased by 28% to 83%, and the left-turning queue length decreased from 36% to 6%. Under high or oversaturated traffic status, the traffic benefits of the CFI tended to be steady. Similarly, in their study about integrated computer system analysis, selection, and evaluation of unconventional intersection, Chang et al. (2011) developed well-calibrated VISSIM simulation models of CFI and diverging diamond interchange (DDI). They assessed operational performance under various constraints and traffic conditions, and concluded that the average intersection delay for both CFI and DDI depended on the congestion level of each sub-intersection and the ratio of maximum queue length to available bay length. Jagannathan and Bared (2004) also studied the design and operational performance of three types of CFI, including crossover on all approaches (Type A), crossovers on the major street approach (Type B) and crossover on one approach of the major street (Type C), over a wide distribution of traffic flow conditions under pre-timed signal settings. The result of the study found a 48% to 50%

delay reduction for Type A, 58% to 71% delay reduction for Type B and 19% to 90% delay reduction for Type C. El Esawey and Sayed (2007) compared the performance of four legged CFI and upstream signalized crossover (USC). They found that CFI has a higher capacity than conventional intersections by 90%. Similarly, USC has a 50% higher capacity than conventional intersections. Taberno and Sayed (2006) did a performance comparison of upstream signalized crossovers with a conventional intersection using VISSIM and found that USC can handle high traffic volumes with low overall delay.

Kaisar, Edara, Rodriguez, and Chery (2011) compared the performance of a signalized and un-signalized roundabout, a continuous flow intersection (CFI), and parallel flow intersection (PFI) as alternatives to the conventional four legged intersection. They used AIMSUN and VISSIM models to evaluate these intersections under three different level of volume; low, medium, and high entrance volumes. The CFI produced the lowest delay at medium (3,000 vph to 4,000 vph) and high (4,500 vph to 6,000 vph) volumes. The roundabout performed well at low (1,000 vph to 2,500 vph) volume. However, the overall performance was still better with CFI over all alternatives as it maintained level of service (LOS) C throughout all volume levels. Chang et al. (2011) developed deterministic queuing models for CFI and DDI. They also found that for CFI or DDI, the average intersection was dependent on the congestion level of each sub-intersection and the ratio of the maximum queue length to available bay length at each bottle-neck location. They categorized the level of impacts of queue on intersection delay at different location. For example, for a two legged CFI, the prioritized queue locations were: (1) the through queue between the main and crossover intersection, (2) the left turn queue between the main and crossover intersection, (3) the left turn queue at each crossover intersection, and (4) the through and left turn queue on the conventional legs. Autey, Sayed, and El Esawey (2013) studied four unconventional intersections: MUT, CFI, USC and DXI. The CFI outperformed all the intersections under most balanced and un-balanced volume levels. CFI showed 99% higher capacity than conventional intersections. USC and DXI had about 50% more capacity than conventional intersections. The impact of increasing the left turning volume was more prominent in conventional intersections than these unconventional intersections.

Caroll and Lahusen (2013) developed a deterministic model to study the operational effects of CFI. The model showed that a displaced left turn lane (DLTL) plays a vital role in the optimal design of CFI. Some other non-geometric elements, like speed, also have an effect on operation, but not as much as a DLTL. DLTL also controls the maximum available signal phase time of left-turning movement. Traffic volume demand is a major control over DLTL length. The authors expressed that the deterministic models can provide a more accurate and time effective result in finding the optimal design for DLTL. Yang et al. (2013) studied CFI to identify queue spillback locations and develop a set of planning stage models. They developed four equations for four types of queue at full CFI with deterministic equations and using a simulation experiment. Using experimental data, they also identified the correlation between the total delay and queue length (QL) ratio at each critical location. The mean and variance of average delay grows exponentially when the average QL ratio increased. The QL ratio is the ratio of bay length to queue length. Average delay increases linearly with the increase of total demand. Under a high congested volume, the critical lane volume could be a more reliable indicator of delay than total volume. Type 1 queue was the through queues at the major intersection. Type 2 queue was the left-turn queues at the crossover intersection. Type 3 queue was the left-turn queues at the major intersection. Type 4 queue was the through queues at the crossover intersection. They used the QL ratio to identify queue spillback locations. The performance comparison between CFI and conventional intersections showed a 34.2% reduction in average delay per vehicle (sec) in the morning peak hours and 35.1% in the afternoon peak hours.

The CFI is also helpful from the viewpoint of fuel consumption and vehicle emission. A study by Park and Rakha (2010) indicated that the use of CFI could result in energy savings of 5% (from VISSIM) and 11% (from INTEGRATION). In the case of emissions, the decrease in HC, CO, and NO_x emissions ranged from 1% to 6%. This study also indicated that CFI reduced fuel use and CO_2 emissions at both low and high demand levels of left-turn movements.

2.3.2 Safety Performance of CFI and Effect of Signing and Markings

The two-legged CFI possesses 30 conflict points compared to 32 conflict points in a conventional intersection (FHWA Techbrief: Displaced Left-turn Intersection, 2009). Hence, it can be thought that CFI would reduce crashes. This section includes a safety related literature review about CFI. The review notes from prominent literature related to the safety performance of CFI are also shown table 2.11.

A safety study of CFI in Baton Rouge indicated a 24% reduction in total crashes and a 19% reduction in fatal and injury crashes (FHWA Techbrief: Displaced Left-turn Intersection, 2009). In terms of the service of unconventional intersections to pedestrians, Jagannathan and Bared (2005) studied CFI in terms of design methodologies for pedestrian access and related pedestrian signal timings. They conducted simulations for pedestrian traffic performance of three CFI models, optimized timing for vehicular traffic. Case A was the four-legged CFI model, Case B was the two-legged CFI model, and Case C was the three-legged CFI with a displaced left turn at one side of the major street. The result from the simulation indicated that the maximum average delay experienced by any pedestrian while crossing the intersection legs with a speed of 4 ft/s were 37 s, 46 s, and 44 s for Case A , Case B and Case C , respectively. The maximum average delay per stop experienced by pedestrians while crossing the intersection legs with a speed of 4 ft/s were 15 s, 28 s, and 26 s, which are equivalent to LOS B, C, and C for Cases A, B, and C, respectively. The maximum average delay for pedestrians for diagonal crossing across two legs of CFI at a speed of 4 ft/s were 34 s, 55 s, and 26 s for Case A, Case B, and Case C, respectively. The average delay per stop for pedestrians crossing diagonally across two legs of the CFI at a speed of 4 ft/s were 11 s, 20 s, and 18 s, which are equivalent to LOS B for Case A, Case B, and Case C , respectively. All pedestrians were serviced within two cycles of signal timing for all cases. Yahl (2013), in his thesis, studied the safety effects of CFI through a before-and-after study. Data from five sites in four different states were used for study using the naïve method, the naïve with traffic factors method, and the comparison group method. The results indicated a decrease in collisions in Baton Rouge, LA, but there was an increase in collisions in the after period for the rest of the test sites. From the overall site analysis, it was found that fatal and injury collisions, rear end, and sideswipe collisions increased, while angle and other collisions decreased. The study also recommended that agencies should consider using CFI as a congestion treatment, although the safety effects are negative or neutral. The study recommended adjusting the design elements of CFI to minimize the additional crashes.

A safety study conducted by Park and Rakha (2010) regarding CFI indicated that a large number of unsafe maneuvers during the early use of CFI could be due to drivers' unfamiliarity. The study showed a 50% reduction in unsafe maneuvers by the end of first year of use. The researchers have suggested implementing a proper signing plan and information campaign as a solution to this problem. The AASHTO Green Book (2011) has suggested the use of signing, visual cues, and education to provide guidance for intersection users. Similarly, in Thomson and Hummer's study (2001) about the safe implementation of unconventional intersections, they

Table 2.11 Safety performance of CFI

Literature	Alternative Intersection	Safety Performance and Behavior
Techbrief: Displaced Left-turn Intersection (2009)	CFI	The two legged CFI possess 30 conflict points compared 32 conflict points in a conventional intersection. A safety study of CFI in Baton Rouge indicated 24% reduction in total crashes and 19% reduction in fatal and injury crashes.
Jagannathan and Bared (2005)	CFI, Case A: 4-legged CFI, Case B: 2-legged CFI, Case C: 3-legged CFI	The maximum average delay experienced by pedestrian while crossing the intersection legs with speed 4 ft/s were 37, 46 and 44 s for Case A, Case B, and Case C, respectively.
		The maximum average delay per stop experienced by pedestrian while crossing intersection legs with speed 4 ft/s were 15, 28 and 26 s which are equivalent to LOS B, C and C for Case A, Case B, and Case C, respectively.
		The maximum average delay for pedestrian for diagonal crossing across two legs of CFI at speed of 4 ft/s were 34, 55 and 26 s for Case A, Case B, and Case C, respectively
		The average delay per stop for pedestrian for diagonal crossing across two legs of CFI at speed of 4 ft/s were 11, 20 and 18 s which are equivalent to LOS B for Case A, Case B, and Case C, respectively.
Park and Rakha (2010)	CFI	There were large numbers of unsafe maneuver during the early use of CFI. The study showed 50% reduction in unsafe maneuvers by the end of first year of use.
Yahl (2013)	CFI	The study found that fatal and injury collisions, rear end and sideswipe collisions were increased. The angle and other collisions were decreased.
Thomson and Hummer (2001)	CFI	The safe implementation of unconventional intersections emphasized on signing plans and efficient public information campaigns as a measure to fulfill the adhoc and priori expectancy of the drivers to maintain safety.
AASHTO (2011)	CFI	AASHTO suggests the need of signing, visual cues, and education to provide guidance for intersection users.

emphasized signing plans and efficient public information campaigns as a measure to fulfill the ad-hoc and priori expectancy of the drivers to maintain safety. Similarly, Inman (2009) conducted a simulator study to compare alternative signing and marking options. Three strategies for navigation signing were evaluated: overhead signing and two different ground-mounted sign alternatives. The results showed that the ground-mounted signing treatment that included a "Keep Left" advance sign was as effective as an overhead navigation sign. FHWA (2008) has published a summary report regarding the evaluation of sign and marking alternatives for displaced left turn (DLT) lane intersections with main three purposes. The first purpose was to ‘‘inform recommendations for signing DLT crossovers,’’ the second was to ‘‘inform recommendations for mitigation of stop line overruns on minor street approaches to DLTS,’’ and the third was to see ‘‘the extent to which naive drivers are able to navigate a DLT for the first time.’’ The study concluded that advance signing is important and that overhead signing may not be more effective than ground-mounted signs. It does not mean that overhead signs should not be considered; it means that in some cases ground-mounted signs could be sufficient. The report mentioned that minor street approaches to DLT were not the problem. There were KEEP CLEAR markings applied, but whether the absence of overruns was due to this sign or not was not certain. The study mentioned that there was no driver confusion when drivers confronted the DLT for the first time.

2.4 The Operation and Safety Performance Related to Jughandles

As described in preceding chapter, the jughandle operates with one main intersection and two other crossover intersections. Reverse/reverse (R/R) jughandles have reverse ramps to divert left-turning vehicles. The main intersection is signalized and the crossovers are YIELD controlled. The operational and safety effect brought by this intersection modification are described in flowing subsections.

2.4.1 Operational Performance of Jughandles

Jughandles are thought to be suitable as an intersection treatment for high volume arterials with moderate to low left turning volume (Toolbox on Intersection Safety and Design, 2004; Hummer, 1998; Rodgerts et al., 2004). It is applicable for all volumes of minor street through. Hummer and Reid (1999) further added that jughandles were suitable for arterials with high through volume, moderate to low left turn volume, narrow right of way, and the distances between signals should be long so that additional right-of-way and other costs for the ramp do not exceed the savings. The extent of performance was based on certain traffic conditions that are discussed below based on the information provided by past literature. The review notes of prominent literature that are directly related to the operational performance of jughandles are shown in table 2.12.

Rodegerdts et al. (2004) indicated that jughandles have potential to reduce the overall travel time, but they do not reduce travel time and stops for left turning vehicles compared to conventional intersections. “Techbrief: Traffic Performance of Three Typical Designs of New Jersey Jughandle Intersections” (2007) indicated that the delay reduction capability of jughandles depend on the type of configuration and geometrics. The result showed that the forward/forward (F/F), reverse/reverse (R/R), and forward/reverse (F/R) jughandles will reduce the average intersection delays compared to conventional intersections by 15-35%, 20-40%, and 25-40%, respectively. Similarly, they have higher intersection capacities compared to conventional intersections for saturated conditions in the range of 20-25%, 25-30% and 25-40%. Jughandles

Table 2.12 Operational performance of jughandle

Literature	Intersection Type	Intersection Capacity with respect to Conventional Intersection	Delay and Queue	Travel Efficiency	Fuel Consumption and Emissions
Rodegerdts et al. (2004)	Jughandle			Jughandles do not reduce stops for left turning vehicles compared to conventional intersections. Jughandles have potential to reduce the overall travel time but they do not reduce travel time as compared to the conventional intersections.	
Techbrief: Traffic Performance of Three Typical Designs of New Jersey jughandle Intersections (2007)	Jughandle : Forward/Forward (F/F), Reverse/Reverse (R/R) and Forward/Reverse (F/R)	Jughandles have higher intersection capacities compared to conventional intersections for saturated conditions in the range 20-25% (F/F), 25-30% (R/R) and 25-40% (F/R). The vehicular capacity of left turn volumes on the major road decreases with ramp offsets such that offsets length reduction from 450 ft. to 230 ft. reduces the left-turn capacity on the major road approach by approximately 30%.	Jughandles reduce the average intersection delays compared to conventional intersections by 15-35% (F/F), 20-40% (R/R), and 25-40% (F/R), respectively.	Jughandles performed well in terms of travel time and number of stops per vehicle compared to conventional intersections only for near saturated conditions.	
Chowdhury (2011)	New Jersey jughandle Intersection (NJJI) with and without pre-signals	The NJJI alternative with pre-signals increases intersection capacity by 45% over conventional intersection.			NJJI reduce HC, CO and NO emissions than conventional one. Regarding fuel consumption, the mileage per gallon in an average were 13.8, 14.7 and 14.8 for conventional, NJJI without pre-signals and NJJI with pre-signals respectively.

performed well in terms of travel time and number of stops per vehicle compared to conventional intersections only for near saturated conditions. The vehicular capacity of left turn volumes in the major road decreases with ramp offsets such that offset length reduction from 450 ft. to 230 ft. reduces the left-turn capacity on the major road approach by approximately 30%. The performance of jughandles may depend on the control of crossovers as well. In this context, Chowdhury (2011) evaluated a New Jersey jughandle intersection (NJJI) with and without pre-signals. The study indicated that NJJI with pre-signals can improve operations under high volume conditions without degrading the condition at low volume. The NJJI without pre-signals performed slightly better than NJJI with pre-signals and conventional intersections under low overall volume condition. The NJJI alternative with pre-signals increased intersection capacity by 45% over conventional intersections. Both types of NJJI reduced HC, CO, and NO emissions from conventional intersections. Regarding fuel consumption, the mileage per gallon on average were 13.8, 14.7 and 14.8 for conventional intersections, NJJI without pre-signals, and NJJI with pre-signals, respectively. Regarding the use of jughandles to improve arterial corridor congestion, Furtado, Tencha and Devos (2003) did a study on suitable methods for the improvement of McKnight Boulevard in Calgary for medium term (2015) and long term (2038) under certain sets of constraints. They studied six alternatives for improvement, including a jughandle ramp at key intersections. The analysis showed that the implementation of a jughandle (4 & 6 lanes) can improve existing conditions and perform better than a conventional six-lane alternative in terms of travel time, signal delay, average speed, and LOS at AM and PM peak hour conditions for eastbound and westbound through traffic along McKnight Boulevard. The signal delay, travel time, and average speed were 139 s, 265 s, and 20 km/h, respectively, for the eastbound AM peak hour in existing conditions. Similarly, the signal delay, travel time, and average speed were 140s, 287s, and 23 km/h, respectively, for the westbound PM peak hour. With the implementation of a jughandle (four lanes) in the year 2015, signal delay, travel time, and average speed will be 33 sec, 126 sec, and 43 km/h, respectively, for the eastbound PM peak hour; and 45 sec, 154 sec, and 42 km/h for the westbound PM peak hour. The same performance measures will be 46 s, 140 s, and 39 km/h for the eastbound PM peak hour; and 38 sec, 147 sec, and 44 km/h for the westbound PM peak hour, in the year 2038 with the implementation of a jughandle (six lanes). These values were better than the conventional six-lane alternative. Similarly, the LOS at year 2038 achieved through the use of a jughandle at 4th Street West were C and B, compared to F and D with the conventional alternative for AM and PM peak hours. The LOS at Center Street were B and B with a jughandle, compared to F and E for a conventional alternative for AM and PM peak hours, respectively. Similarly, the LOS at Edmonton Trail were B and B with a jughandle, compared to F and E for a conventional alternative for AM and PM peak hours.

2.4.2 Safety Performance of Jughandles

A study (Jagannathan, Gimbel, Bared, Hughes, Persuad and Lyon, 2006) indicated that a four-legged signalized intersection with two forward jughandle ramps has a total of 26 conflict points. The four-legged signalized intersection with one forward and one reverse jughandle ramps has a total of 25 conflict points, and the four-legged signalized intersection with two reverse jughandle ramps has total of 24 conflict points. Smith (2013) indicated that a forward ramp intersecting the cross street plus a forward ramp curving left to intersect the mainline jughandle would produce 20 diverge/merge conflicts, 4 crossing (left) conflicts, and 4 crossing (angle) conflicts to total about 28 conflicts, compared to the 32 conflicts of conventional four-legged intersection and 26 conflicts of a Type A jughandle. Jagannathan et al. (2006) indicated that conventional intersections had more head-on, left turn, fatal plus injury, and property damage only crashes compared to NJJIs. Within jughandles, the reverse/reverse jughandle was found to have the lowest rate of angle and left turn crashes per million vehicle miles. The forward jughandles were found have the highest overall rate of crashes per million vehicle miles travelled, which was about 1.3 to 1.4 times the other two types, forward-reverse jughandle and reverse-reverse jughandle. There were significant differences in pedestrian injuries between conventional intersections and NJJIs. Agarwal (2011) developed conflict model concerning pedestrian safety for signalized and un-signalized intersections using a Surrogate Safety Assessment Model (SSAM) to quantify the conflicts. For a F/F jughandle, separate models were developed for ramp sections and intersections. For the ramp, the pedestrian vehicle conflict was found to be positively affected by the turn percentage and conflict volume. Similarly, the pedestrian vehicle conflict for jughandle intersections was found to be positively affected by conflict volume, number of lanes, and turn percentage. The minor approach was found to have more pedestrian vehicle conflicts. The review notes from prominent literature related to rhe safety performance of jughandles are shown table 2.13.

2.5 Cost Associated with Unconventional Intersections

In their paper, Berkowitz et al. (1997) have indicated that the cost of CFI was $638,000 based on a site in Mexico, $6,000,000 based on a site on Brooklyn, NY, and $4.4 million based in Baton Rouge, LA. The preliminary analysis by KLD associates and Francisco Mier indicated that the cost of a CFI is three times the cost of a conventional intersection. COMPASS of Southwest Idaho (2008) mentioned that the total cost of CFI was $8.55 million based on a site in Salt Lake City, including construction costs of $4 million, construction engineering cost of $300,000, a preliminary engineering cost of $1 million, and a right of way cost of $3.25 million.

Table 2.13 Safety performance of jughandles

Literature	Alternative Intersection	Safety Performance and Behavior
Jagannathan et al. (2006)	Jughandles	A four-legged signalized intersection with two forward jughandle ramps has total of 26 conflict points. The four-legged signalized intersection with one forward and one reverse jughandle ramps has total of 25 conflict points, and the four-legged signalized intersection with two reverse jughandle ramps has a total of 24 conflict points.
		The reverse-reverse jughandle was found to have lowest rate of angle and left turn crashes per million vehicle miles. The forward jughandles were found have highest overall rate of crashes per million vehicle miles travelled, which was about 1.3 to 1.4 times the other two types, F/R jughandles and R/R jughandle. There were significant differences in pedestrian injuries between conventional intersections and NJJIs.
Smith (2013)	Jughandles	The forward ramp intersecting the cross street plus forward ramp curving left to intersect the mainline jughandle would produce 20 diverge/merge conflicts, 4 crossing (left) conflicts, and 4 crossing (angle) conflicts, to a total about 28 conflicts compared to 32 conflicts of conventional four-legged intersection and 26 conflicts of a Type A jughandle.
Agarwal (2011)	Jughandles	For jughandle (F/F), separate models were developed for ramp section and intersection. For the ramp, the pedestrian vehicle conflict was found to be positively affected by turn percentage, and conflict volume. Similarly, the pedestrian vehicle conflict for jughandle intersections was found to be positively affected by conflict volume, number of lanes, and turn percentage. The minor approach was found to have more pedestrian vehicle conflicts.

Hildebrand (2007) indicated in this thesis that the estimated cost of a CFI was $5,221,064.02, which was 49% higher than the estimated cost of a conventional intersection. He indicated that the cost of a jughandle was $3,238,125, which was 7.3% lower than a conventional intersection. He also indicated that the cost of MUT was $4,681,201.11 which was 34.1% higher than a conventional intersection. "Intersection Improvement Study Phase 2 Report" from the Town of Cary indicated a total MUT project cost of $3.9 million. In his thesis, Boddapati (2008) indicated that the cost of an un-signalized MUT crossover was $951,818 based on bid prices of the Missouri Department of Transportation in the year 2007. A summary of the cost of unconventional intersections collected from different sources are presented in table 2.14.

Table 2.14 Cost of unconventional intersections

Intersection Type	Source	Cost of Construction
CFI	Berkowitz et al. (1997)	$638,000 (in Mexico)
		$6,000,000 (Brooklyn, NY)
		3 times the conventional intersection (Preliminary analysis by KLD Associates and Francisco Mier)
		The total cost of CFI was $4.4 million (Baton Rouge, LA)
	COMPASS (2002)	The total cost of CFI was $8.55 million (Salt Lake City) including construction cost of $4 million, construction engineering cost of $ 300,000, preliminary engineering cost of $1 million, and right of way cost of $3.25 million.
	Hilderband (2007)-Thesis	The estimated cost of CFI was $5,221,064.02 (49.% higher than conventional)
Jughandle	Hilderband (2007)-Thesis	The cost of jughandle was $3,238,125 (7.3% lower than conventional).
MUT	Hilderband (2007)-Thesis	The MUT costs $4,681, 201.11 (34.1% higher than conventional).
	Intersection Improvements Study Phase 2 report (Town of Cary)	Project cost of MUT was $3.9 Million.
	Boddapati (2008)	The cost of un-signalized median U-turn cross over was $951,818 based on bid prices of MODOT for year 2007.

Chapter 3 State of Practice

Email queries were sent to state agencies collect information about existing research and practices pursued by different U.S. states, especially about guidelines for the implementation, related costs, and user's feedback concerning unconventional intersection treatments. This chapter includes a description of the information collected from different state agencies. The information is also tabulated in table 3.1.

3.1 Missouri Department of Transportation (MODOT)

The responder from MODOT explained that the capacity- and turning movement-based decisions are the main criteria for the implementation of CFIs and jughandles. The responder also mentioned that they had success regarding the CFI designs in the St. Louis area, but were not sure of the reason behind the installation. MODOT has implemented ten median U-turns or J-turns to date. These were designed to tackle the angle crashes in their retrofit projects or four-lane designs. The before-and-after study indicated that there was a tremendous reduction in angle crashes, but an increase in sideswipe crashes with the use of MUT. MODOT's policy for the implementation of directional median openings with downstream U-turns (slight modification MUTs), is based on Section 233.2 At-Grade Intersection with Stop and Yield Control, and subsection 233.2.6 of the MODOT Engineering Policy Guide. The cost to install a J-turn (deviation of MUT) varied by location and geometry, which is about less than $400,000 per intersection (according to their experience with the Central District). These intersections include lengthy acceleration and deceleration lanes as well. According to the responder, the feedback for jughandles was not positive. The responder also indicated that these designs were not well received by all and it would take time before they are accepted by the majority.

Table 3.1 State of practice part I

State Agencies	Contacted Persons	Intersection Types			Cost	User's Response
		CFI	MUT	Jughandle		
MoDOT	John P. Miller	Capacity and turning movement	Capacity analysis considering weaving movements (233.2.6, MoDOT Engineering Policy Guide)	Capacity and turning movement	Variation with location and geometry (J-turn: $400,000 according to the experience in central district)	Not positive, takes time to be accepted by majority
MDOT	Imad Gedaoun	Not used	They have developed a guideline: Directional Crossovers, Michigan Preferred Left Turn Strategy	Not used	$120,000/pair (MUTs)	N/A
NCDOT	James Dunlop	In plan to implement. No specific policy, general practice to resolve traffic safety and operational problems	Superstreets No specific policy, general practice to resolve traffic safety and operational problem	Doesn't consider jughandle as treatment	No formal estimate is available. Superstreet retrofitting: 1.2 million/intersection (2 U-turn locations, median treatment at main intersections and four signals)	Negative at the beginning but, positive afterwards
NYSDOT	Rick Wilder	Based on NYSDOT intersection design and policy stated by NYSDOT Highway Design Manual Chapter 5- Basic Design. Consultation with Regional Transportation Systems Operations Engineer. The layout applicability, safety and operational performance are based on FHWA's Signalized Intersection: Informational Guide.			No standard cost for unconventional intersection. It depends on available right of way, number of lanes, local construction costs, whether or not closed design is needed and the work zone traffic control.	
IOWA DOT	John Narigon		Iowa DOT tried to seek place to install MUTs but it couldn't do that due to public's reception of the project.	The overall cost of proposed J-turn (a deviation of MUT) was $557,300.		Public expressed their concerns that J-turn would bring new problems or continue to have the same crash problem as before. It could be dangerous for buses, as they do U-turn from median opening. It wouldn't be helpful for the roads that are used by semi-trailers.

Table 3.2 State of practice part II

State agencies	Contacted persons	Intersection types			Cost	User's response
		CFI	**MUT**	**Jughandle**		
VDOT	George T. Rogerson Jr.	N/A	N/A	No specific guidelines. VDOT constructed one jughandle because of availability of right of way. Additionally, it improved signal operation of intersection.	Bid estimate: $982,910 (including work associated with closure of 3 additional crossovers)	No feedback. Assumption was made that public accepted the facility.
Kentucky Transportation Cabinet (KYTC)	Brent A. Sweger	No specific policy. University of Kentucky has developed a design tool that provides assistance in deciding the proper type of unconventional intersections.			N/A	Not known about MUT, CFI and jughandle. However, the public responses for roundabout and double crossover diamond interchange were positive.
State Highway Administration, Maryland (SHAM))	Minseo k. Kim	No specific policy. General criteria based on No Special Policy. General criteria based on delay, v/c ratio, queue length, travel time, etc.), right of way, environmental impact, construction cost, engineering judgment and experience along with human factors knowledge in addressing any potential safety concerns.			N/A	N/A

3.2 Michigan Department of Transportation (MDOT)

According to the responder from MDOT, Michigan did not use CFI or jughandle designs, and instead used the Michigan indirect left. MDOT had a detailed report about directional crossovers (MUTs), Michigan's preferred left turn strategy. According to the report, the capacity problem due to interlocking left turns within the bi-directional crossovers at major street intersections led to the concept of using directional crossovers. MDOT has implemented 700 directional crossovers on their state highway system. In terms of capacity, the study conservatively showed 20% to 50% capacity improvement. The report also included the detailed operational analysis, safety analysis, and the design of the directional left turn strategy. In terms of safety, a report from Michigan State University indicated a 31% crash reduction in the entire highway system and a 42% crash reduction in the places where bi-directional crossovers were replaced with directional crossovers. Overall, MDOT appears to have detailed documents about their implementation of MUT designs. The cost of construction, as indicated in the report for directional strategy, was $120,000/pair.

3.3 Iowa Department of Transportation (IOWA DOT)

IOWA DOT did not have any special policies regarding unconventional intersections. IOWA DOT tried to install MUTs, but could not due to public's reception of the proposal. The public expressed concerns that J-turns would bring new problems or continue to have the same crash problems as before. It could be dangerous for buses, as they make U-turns from median openings, and would not be helpful for roads that are used by semi-trailers.

3.4 Virginia Department of Transportation (VDOT)

VDOT did not have special policies. It had constructed a jughandle on Route 460. A jughandle was chosen because it could be constructed within the pre-existing VDOT right of way

(ROW), and it improved the signal timing operation of the intersection. The bid estimate of the project was $982,910, however, it included work associated with the closure of three additional cross-overs. The work that was included in the cost estimate, but was not related to the construction of the jughandle was less than 10%.

3.5 Kentucky Transportation Cabinet (KYTC)

The Kentucky Transportation Cabinet (KYTC) did not have a policy about the implementation of unconventional intersections. However, the Kentucky Transportation Center at the University of Kentucky has developed a design tool that helps determine the feasibility and lane configurations of different type of intersections. It did not have a cost estimate for intersections because they vary by location, size, and complexity.

3.6 North Carolina Department of Transportation (NCDOT)

North Carolina's general state of practice is to find anything that works to resolve traffic safety and operation problems. Their Congestion Management section reviews each project to determine the best operation for the corridor/intersection. There is no specific policy as projects are judged on an individual basis. However, they are looking to implement more superstreets on their critical corridors to maintain acceptable LOS and travel time. North Carolina does not consider jughandles for treatments anymore. North Carolina has not implemented CFIs, but is planning to implement them in near future. The responder indicated that NCDOT did not have any formal cost estimations. To retrofit existing intersections into superstreets, they estimated about $1.2 million per single location, which includes two U-turn locations, the median treatment at the main intersection, and four signals. The public's opinion was generally negative before, and positive afterward.

3.7 New York State Department of Transportation (NYSDOT)

Their practice is based on NYSDOT intersection design and policy stated by its Highway Design Manual Chapter 5: Basic Design. Section 5.9.1.3 states that unconventional intersections need special consideration and treatment and should be developed in consultation with the Regional Transportation Systems Operations Engineer. The layout, applicability, design features, safety performance, and operational performance are based on FHWA's Signalized Intersections: Informational Guide. Section 5.9.1.3 also states that jughandles are applied in the locations where operational and safety concerns preclude left turns from the median lane.

3.8 State Highway Administration, Maryland (SHAM)

The State Highway Administration, Maryland reported that there is no specific policy about the implementation of unconventional intersections except the general criteria based on delay, volume to capacity ratio, queue length, right of way, environmental impact, construction cost, engineering judgment, and experience, along with human factors in addressing any potential safety concerns.

Chapter 4 Operational Analysis

This chapter includes the procedure applied in the operational analysis of standard four-legged signalized intersections and three types of unconventional intersections: MUT, CFI, and jughandle. The operational analysis includes the volume database creation, the use of HCS for delay estimation, and a subsequent estimate of fuel consumption and emissions, which are described in detail in the following sections.

4.1 Levels and Magnitude of Fixed and Variable Parameters

Prior to the computation of the measures of effectiveness (MoEs), it was necessary to consider the reasonable magnitude of fixed parameters that represented the local condition of Nebraska or standard practices in the U.S., and fix the level of variable parameters. The fixed parameters were speed and some specific geometric related parameters. The variable parameters were volumes both of major streets and minor streets, truck percentages for both major streets and minor streets, the directional split for major streets, and the turn percentage of major streets.

4.1.2 Fixed Parameters

(A) Speed: Since this study is focused on rural roads, the operating speed at the intersections and crossovers were considered to reflect the speed condition of rural roads of Nebraska. The Nebraska Minimum Design Standards (2008) specify the design speed of 60 mph for rural major arterials in the state highway system. Similarly, it has a specified design speed of 40 mph to 50 mph for rural other arterials, 40 mph to 50 mph for rural collectors, and 30 mph to 50 mph for rural local roads in the local roads and streets system. To represent these speed conditions of rural roads in Nebraska, the operating speed of major streets was fixed at 45 mph, and the operating speed of minor streets was fixed at 35 mph. The speed of traffic at the reverse ramp of the jughandle was kept fixed at 30 mph based on the study described in “Techbrief: Traffic Performance of Three Typical Designs of New Jersey Jughandle Intersections” (2007).

(B) Offset of Median Openings and Geometry related to Median Openings for MUT: The offset of median opening was fixed at 660 ft. based on the suggestion of past literature (AASHTO Green Book, 2011; “Information and Geometric Design Guidance Regarding Boulevards, Directional Crossovers, and Indirect (‘Michigan’) Left Turns”, 1995; Potts et al., 2004). The median width was kept fixed at 60 ft. to accommodate a tractor-semitrailer combination of trucks as the design vehicle as prescribed by past literature (AASHTO Green Book, 2011; Rodgerts et al., 2004).

(C) Offset of Crossovers for Jughandles: For jughandles, the offsets of crossovers were considered 170 ft. along major streets and 150 ft. along minor streets as shown in a typical diagram of the geometry of a R/R jughandle in “Techbrief: Traffic Performance of Three Typical Designs of New Jersey jughandle Intersections” (2007).

(D) Geometric Features of CFI: The offset of east and west crossovers were considered 350 ft. from the central signalized intersection. This was based on the study by Armstrong (2014).

4.1.3 Variable Parameters

(A) Volume: The bi-directional volume beginning from 50 vph to 2400 vph at the increment of 50 mph was considered. There were a total of 48 different bi-directional volume levels for the major street. For the minor street, one directional volume, from 25 vph to 250 vph, was considered at the increment of 25 vph. Since the split for the minor street was kept constant at 0.5, both directional volumes for the minor street were equal. There were ten different unidirectional volume levels for the minor street. Since the right turn movement was to be excluded from the analysis, no volume was generated for the right turn movements for all four intersections.

(B) Truck Percentage: Three levels of truck percentage were considered: 2%, 5%, and 10%. The same truck percentages were considered for both major and minor streets.

(C) Balance Factor: The balance factor is related to the ratio of approach volume to total approach volume**.** Three levels of balance factor along the eastbound direction of the major street were considered: 0.5, 0.6, and 0.7. The balance factor of the minor street was kept fixed at 0.5.

(D) Left Turn Percentage: Three levels of left turn percentage were considered for major streets, those being 5%, 10%, and 15%. The left turn percentage of the minor street was kept fixed at 5%.

Based on directional splits (balance factor) and turn percentage, an origin-destination (O-D) volume database for all volume combinations was constructed for all four types of intersections: a standard signalized intersection, MUT, CFI, and jughandles. The combination of all these levels of variable parameters produced a total of 12, 960 combinations for each intersection.

4.2 Estimation of Delay

Delay estimation was done using HCS 2010® Streets version 6.5. The networks were coded to represent the whole system of each individual intersection. For example, MUT has two STOP controlled median openings and one central signalized intersection, CFI has four signalized crossovers and one central signalized intersection, and the jughandle has four crossovers, including two YIELD controlled crossovers and one central signalized intersection. The fully actuated signal control was assumed for all four intersection types. The details are described in following subsections.

4.2.1 Standard Signalized Intersection

The standard signalized intersection consists of two through lanes, one exclusive left turn lane, and one exclusive right turn lane in an approach. Since the right turn movement was not considered for analysis, it was not included in the lane configuration coding in HCS 2010®. The intersection operates under a fully actuated eight-phase signal control with protected left turn movements, as shown in figure 4.1. The maximum green times for all the movements were kept at 50 s, and minimum green times were left at the default 10s. The yellow change time was changed to 3 s. The base saturation flow rate was kept at 1800 pc/h/ln. The speed was set to 45 mph for the major street and 35 mph for the minor street. Respective heavy vehicle percentages for each volume combination were set. The stepwise procedure to develop the standard signalized intersection in HCS 2010® with respective input parameters is included in Appendix A.

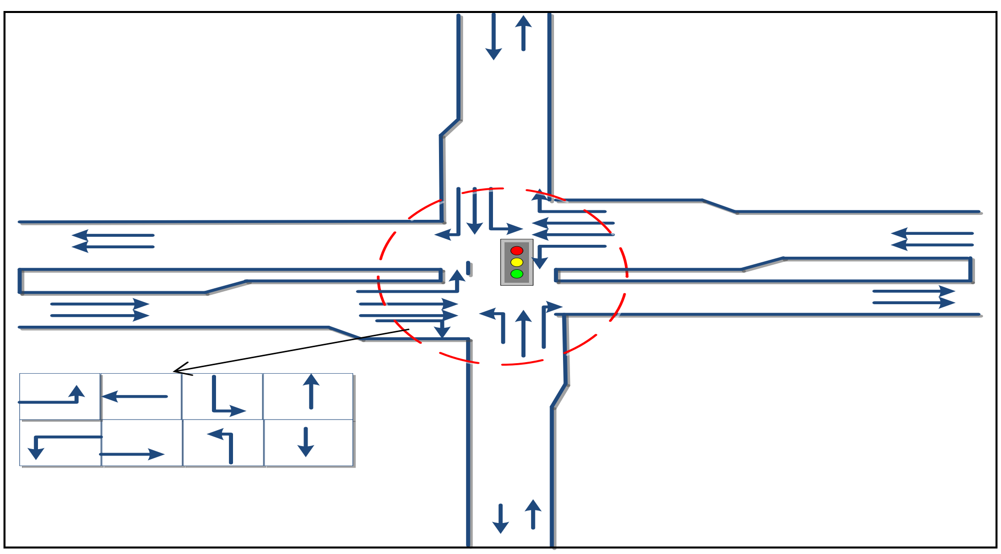


Figure 4.1 Phases and lane configuration of standard signalized intersection

4.2.2 MUT

The MUT consists of two through lanes and one exclusive right turn lane. Since the right turn movement was not considered for analysis, it was not included in the lane configuration coding in HCS 2010®. The central signalized intersection operates under fully-actuated two phase signal control, allowing through movement of the major and minor street. The median openings are STOP controlled. During network coding in HCS 2010® Streets, the un-signalized median opening was coded as equivalent to a signalized intersection, while a U–turn was coded as a permitted left turn (see fig. 4.2). This permitted left turning movement will try to sneak through the gap of opposing traffic which were provided with continuous green time throughout the cycle. To return the volumes toward the direction of U-turns, a right turn lane (northbound for west crossover and southbound for east crossover) was introduced with an equal volume to the left turn lane. Since HCS 2010® Streets does not support a single right turn lane, a same bound dummy through movement was added with zero volume. The right turn movements were allowed to turn right on red (RTOR). The central signalized intersection was coded as two phase signals. This coding technique was adopted from Armstrong (2014). However, unlike Armstrong's coding, the signal control was made fully actuated in this study. The maximum green times for all the movements were kept at 50 s and the minimum green times were left at the default 10s. The yellow change time was changed to 3 s. The base saturation flow rate was kept at 1800 pc/h/ln. The speed was set to 45 mph for the major street and 35 mph for the minor street. The respective heavy vehicle percentages for each volume combination were set. The stepwise procedure of network coding in HCS 2010 ® Streets is included in Appendix A. The input of the left turn equivalency factor was replaced by the approximate U-turn equivalency factor (Armstrong, 2014), which was calculated based on the effects of the radius of travel path using equation 4.1 from HCM 2010.

$$f_R = \frac{1}{1 + \frac{5.61}{R}} \qquad 4.1$$

Where,

f_R= U-Turn Equivalency factor = Adjustment factor to account for the effect of travel path radius

R= Radius of travel path

For median openings, the U-turn equivalency factor was calculated using a 60 ft. turn radius. The default values of critical headway and follow up headway for a permitted left turn were also replaced with values for U- Turn movement calculated using the following equations from HCM 2010 for 0% grade and no adjustment factor for intersection geometry.

$$t_{c,x} = t_{c,base} + t_{c,HV}P_{HV} \qquad 4.2$$

$$t_{f,x} = t_{f,base} + t_{f,HV}P_{HV} \qquad 4.3$$

Where,

$t_{c,x}$= critical headway,

$t_{c,base}$= base critical headway,

$t_{c,HV}$ and $t_{f,HV}$ = adjustment factor for heavy vehicles,

P_{HV} = proportion of heavy vehicles for movement,

$t_{f,x}$= follow up headway, and

$t_{f,base}$= base follow up headway

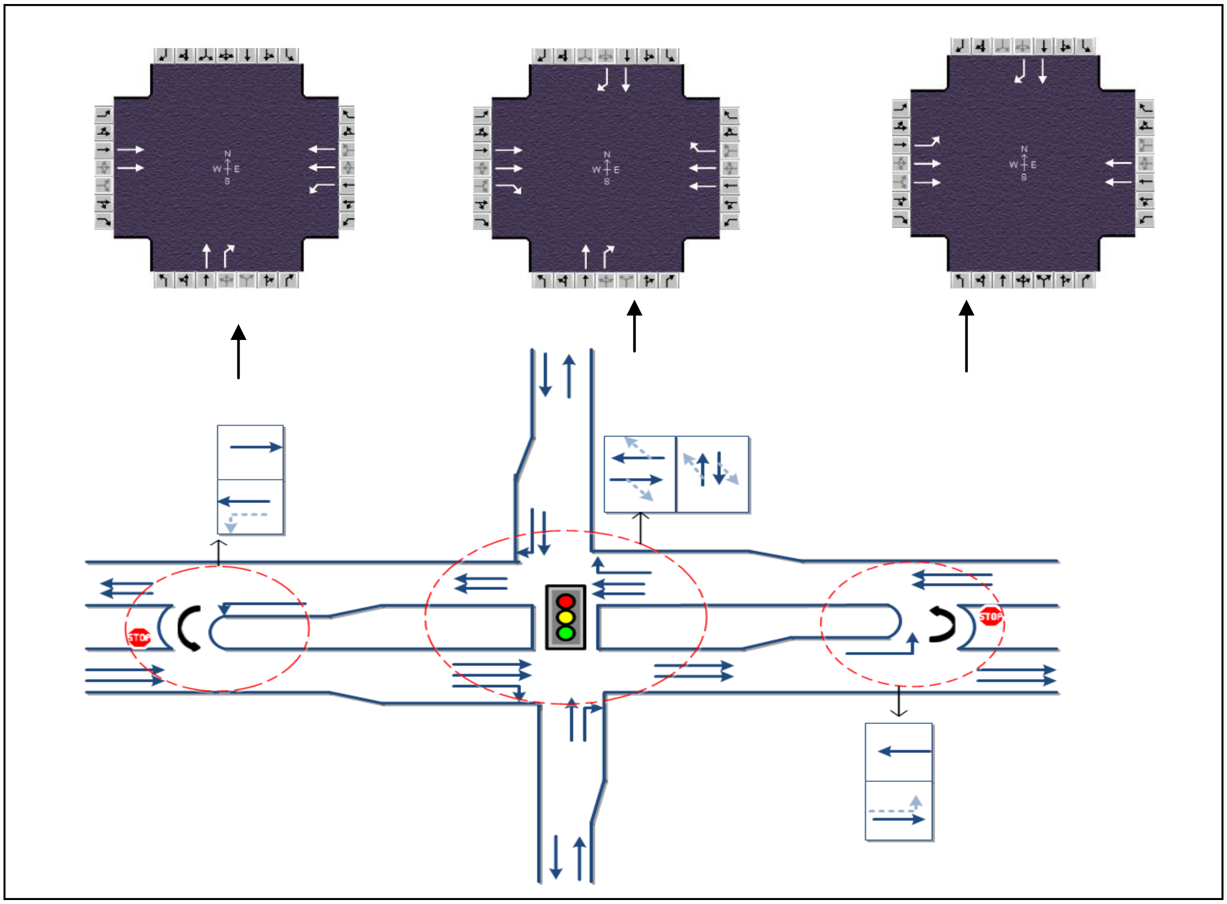


Figure 4.2 Equivalent phases and lane configuration of MUT considered in network coding in HCS 2010® Streets

The base critical headway for a U-turn from the major street can be obtained with table 4.1, based on HCM 2010. In this study, 6.9 s was chosen considering the narrow turn and four lanes. The adjustment factor for heavy vehicles for critical headway is 2 for a major street with two or three lanes in each direction (HCM 2010). The proportion of heavy vehicles for the movement corresponded to the truck percentage. Similarly, the base follow-up headway for U-turns can be obtained by following table 4.2, which is based on HCM 2010. The adjustment factor for heavy vehicles for follow-up headway was 1 for a major street with two or three lanes in each direction (HCM 2010).

4.2.3 CFI

CFI is comprised of one central signalized intersection and four signalized crossovers. While coding the movements of east and west crossovers (see fig. 4.3), the left turns at crossovers were coded as right turns. Since HCS 2010® Streets does not support a single right turn lane, a same bound dummy through movement was added with zero volume. Similarly, the left crossover movements at north and south crossovers were coded as right turn movement in the central signalized intersection. The default value of the right turn equivalency factor in HCS 2010® Streets was replaced with the left turn equivalency factor to compensate for the left and right turns trading roles. This coding technique was adopted from Armstrong (2014). However, unlike Armstrong's coding, the signal control was made fully actuated. The maximum green times for all the movements were kept at 50 s and minimum green times were left at the default 10 s. The yellow change time was changed to 3 s. The base saturation flow rate was kept at 1800 pc/h/ln. The speed was set to 45 mph for the major street and 35 mph for the minor street. Respective heavy vehicle percentages for each volume combination were set. The stepwise procedure of network coding in HCS 2010 ® Streets for CFI is included in Appendix A.

Table 4.1 Base critical headway values from HCM 2010 Exhibit 19-10

Vehicle Movement	Base Critical Headway (s)		
	Two Lanes	**Four Lanes**	**Six Lanes**
Left turn from Major	4.1	4.1	5.3
U-turn from Major	N/A	6.4 (wide)	5.6
		6.9 (narrow)	
Right turn from Minor	6.2	6.9	7.1
Through traffic on Minor	1-stage: 6.5	1-stage: 6.5	1-stage: 6.5
	2-stage, Stage I: 5.5	2-stage, Stage I: 5.5	2-stage, Stage I: 5.5
	2-stage, Stage I: 5.5	2-stage, Stage I: 5.5	2-stage, Stage I: 5.5
Left turn from Minor	1-stage: 7.1	1-stage: 7.5	1-stage: 6.4
	2-stage, Stage I: 6.1	2-stage, Stage I: 6.5	2-stage, Stage I: 7.3
	2-stage, Stage I: 6.1	2-stage, Stage I: 6.5	2-stage, Stage I: 6.7

Table 4.2 Base follow up headway values from HCM 2010 Exhibit 19-11

Vehicle Movement	Base Follow-Up Headway (s)		
	Two Lanes	**Four Lanes**	**Six Lanes**
Left turn from Major	2.2	2.2	3.1
U-turn from Major	N/A	2.5 (wide)	2.3
		3.1 (narrow)	
Right turn from minor	3.3	3.3	3.9
Through traffic on Minor	4.0	4.0	4.0
Left turn from Minor	1-stage: 7.1	1-stage: 7.5	1-stage: 6.4

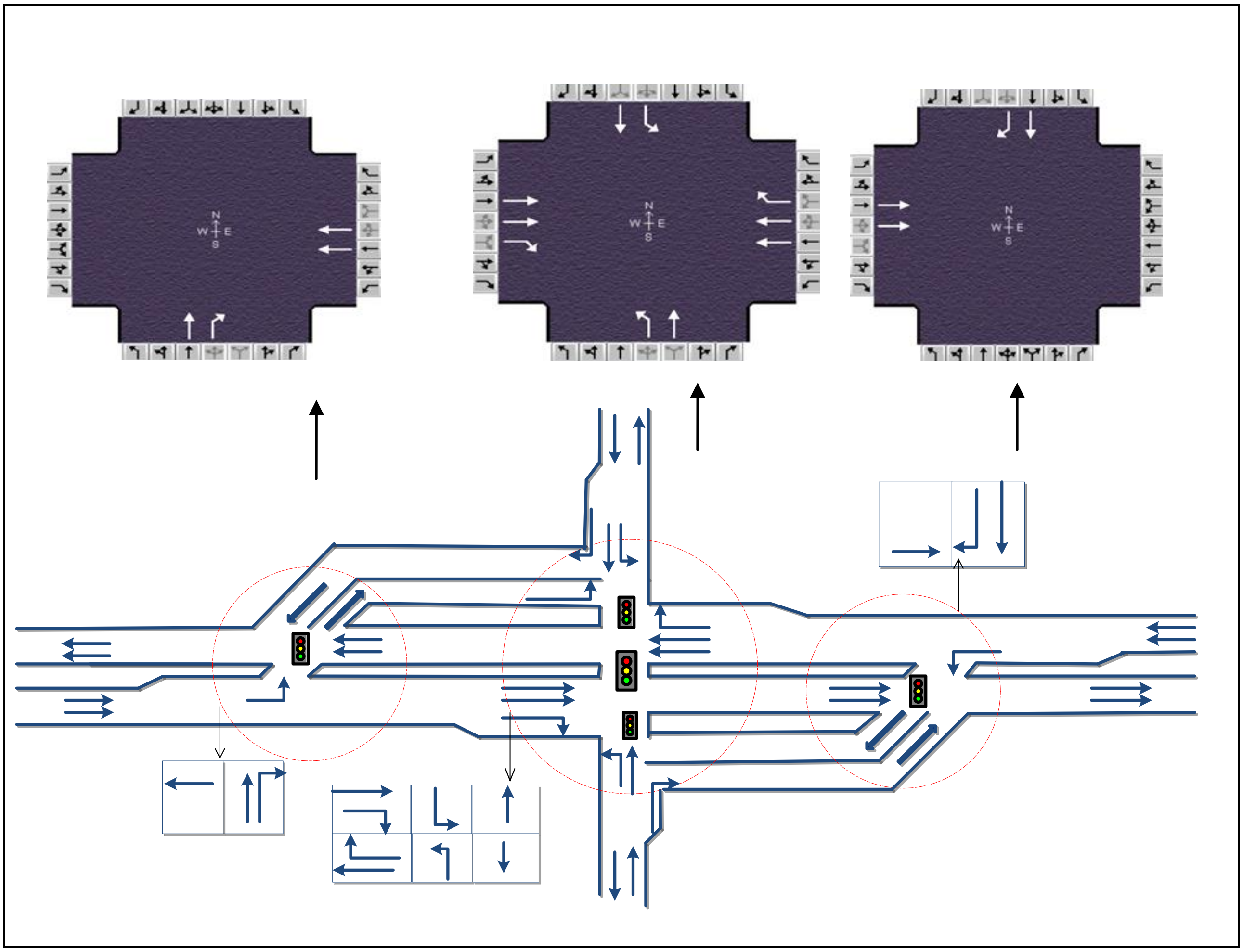


Figure 4.3 Equivalent phases and lane configuration of CFI considered in network coding in HCS 2010® Streets

4.2.4 Jughandles

The R/R jughandle has ramps forming crossovers at major and minor street segments. The ramps facilitate the flow of left turn traffic from major street approaches. The network coding of a jughandle intersection in HCS 2010® Streets is shown in figure 4.4. The right turns from ramps at minor street crossovers were coded as permitted left turns. The tradeoff was made by setting the critical headway and follow-up time of permitted left turn movement as that of right turn movement. The critical headway and follow up times were calculated using equations 4.2 and 4.3, and the respective values of base critical headway and follow up times were chosen from tables 4.1 and 4.2. The base critical headway was chosen to be 6.9 s, as it was the movement that originated from a four-lane road, and the follow-up headway was chosen to be 3.3 s. To return the volume from permitted left-turn movements toward the original direction, right turn movements with equal volume were coded at both crossovers. Since HCS 2010® Streets does not support a single right turn lane, a same bound dummy through movement was added with zero volume at both crossovers. The right turn movements at both crossovers were allowed to turn right on red (RTOR). The central signalized intersection was coded to operate with a three phase signal. For volume balancing purposes, both left turn movements from the major street were treated as RTOR movement at the central signalized intersection. The signal control was made fully actuated. The maximum green times for all the movements were kept at 50 s and minimum green times were left at the default 10 s. The yellow change time was changed to 3 s. The base saturation flow rate was kept at 1800 pc/h/ln. The speed was set to 45 mph for the major street and 35 mph for the minor street. Respective heavy vehicle percentages for each volume combination were set. The stepwise procedure of network coding in HCS 2010 ® Streets for jughandles is included in Appendix A.

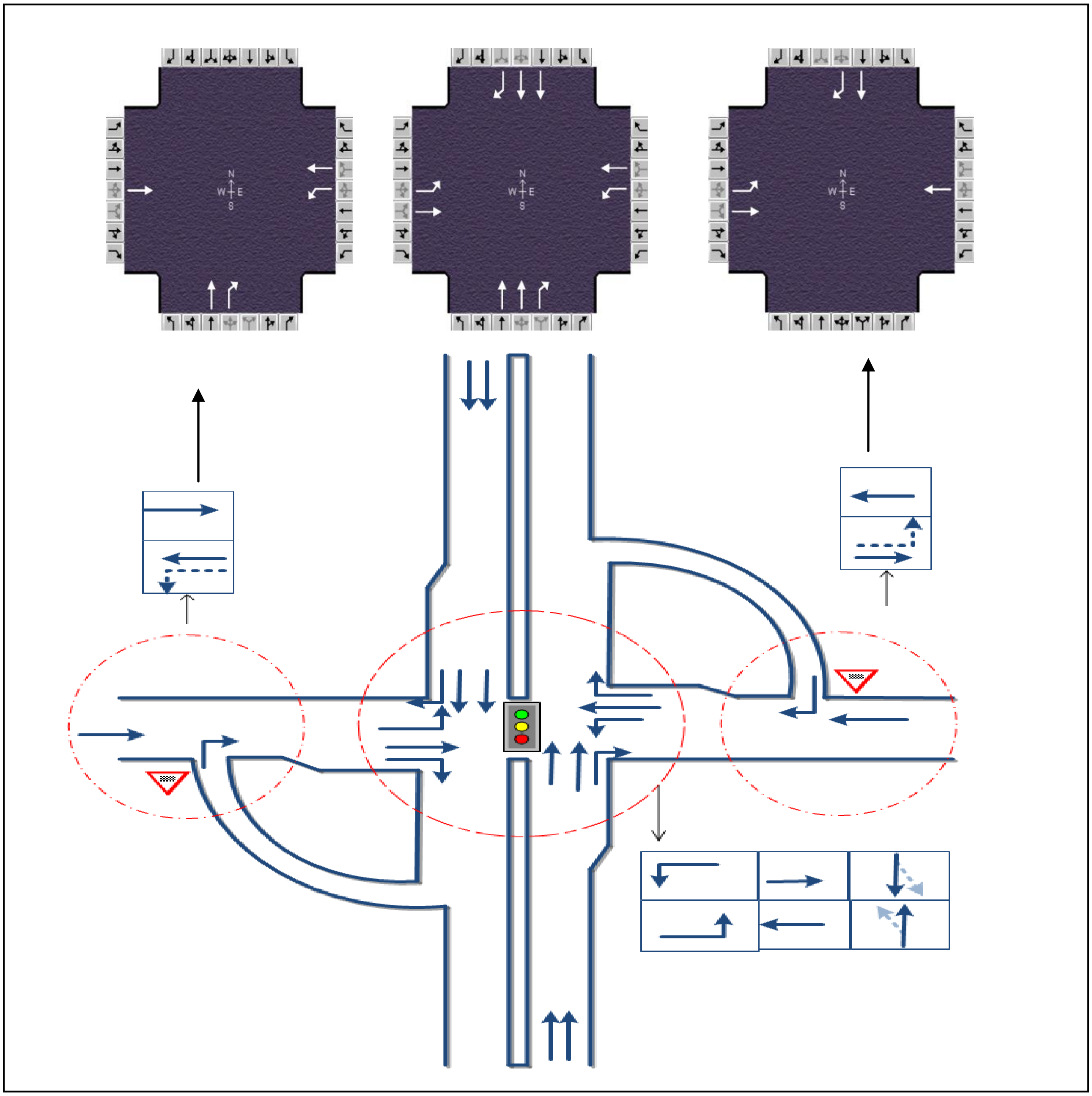


Figure 4.4 Equivalent phases and lane configurations of jughandle considered in network coding in HCS 2010® Streets

4.2.4 Delay Results for Median Openings of MUT and Crossovers at Minor Streets of Jughandles from HCS 2010® Streets

Careful observation of the HCS output confirmed that there was some error in the HCS 2010® Streets calculation of the delay at median openings and crossovers at the jughandle's minor street. As an example of this error, a screen shot of movement results from HCS 2010® Streets output for MUT is shown in figure 4.5. If we observe the capacity of westbound left movement as shown in the figure 4.5, it is higher than the saturation flow of the left turn lane, which is not true according to the original relationship between capacity and saturation flow. The problem was reported to the developer of HCS 2010® Streets and the response is yet to be received.

Movement Group Results	EB			WB			NB			SB		
Approach Movement	L	T	R	L	T	R	L	T	R	L	T	R
Assigned Movement		2		1	6			8	18			
Adjusted Flow Rate (*v*), veh/h		1051		71	1016							
Adjusted Saturation Flow Rate (*s*), veh/h/ln		1632		222	1632							
Queue Service Time (g_s), s		0.0		0.0	0.0							
Cycle Queue Clearance Time (g_c), s		0.0		0.0	0.0							
Green Ratio (*g/C*)		1.00		1.00	1.00							
Capacity (*c*), veh/h		3264		986	3264							
Volume-to-Capacity Ratio (*X*)		0.322		0.072	0.311							
Available Capacity (c_a), veh/h		17305		1941	17305							
Back of Queue (*Q*), veh/ln (50th percentile)												
Queue Storage Ratio (*RQ*) (50th percentile)												
Uniform Delay (d_1), s/veh		0.0		0.0	0.0							
Incremental Delay (d_2), s/veh		0.0		0.0	0.0							
Initial Queue Delay (d_3), s/veh		0.0		0.0	0.0							
Control Delay (*d*), s/veh												
Level of Service (LOS)												
Approach Delay, s/veh / LOS												
Intersection Delay, s/veh / LOS												

Figure 4.5 Screen shot of a part of HCS 2010® Streets result for median opening of MUT

Since similar methods have been applied for delay calculation of both median openings and crossovers at the minor street of a jughandle, it was necessary to correct the delay for both locations. To calculate the delay at median openings and crossovers at the minor street of a jughandle, a method based on queuing theory was applied, and is described in the following section.

4.2.5 Delay for Median Openings of MUT and Crossovers at the Minor Street of a Jughandle Based on Queuing Theory

The delay for the system comprised of Poisson arrivals and the Exponential service served by a single server can be computed by a M/M/1 queuing concept. The discrete random variable X (t) is said to have a Poisson distribution with a parameter of $\lambda > 0$, if the probability mass function (pmf) is given by the following expression (Hillier, Lieberman, Nag and Basu, 2010).

$$P\{X(t) = n\} = \left\{\frac{\lambda^n \times e^{-\lambda}}{n!}\right\} \qquad 4.4$$

Where,

$P\{X(t) = n\}$ = Probability mass function,

λ = rate parameter, and

n = 0, 1, 2, 3…

The expected value E (X) and variance Var (X) for the Poisson distribution are both equal to rate parameter λ. The random variable λ is said to have an exponential distribution with parameter if its probability density function (pdf) has the following expression (Banks, Carson II, Nelson and Nicol, 2009).

$$f(t;\lambda) = \lambda \times e^{-\lambda t} \qquad 4.5$$

for $t \geq 0$ and

$$f(t;\lambda) = 0 \qquad 4.6$$

for $t < 0$

The total system delay based on the M/M/1 concept is the sum of queue delay and the server's delays as expressed by the following equation (Hillier et al., 2010).

$$W_s = \frac{\lambda}{\mu \times (\mu - \lambda)} + \frac{1}{\mu} \qquad 4.7$$

Where,

W_s = Total system delay,

λ =Arrival flow rate, and

μ = Departure flow rate

The departure flow rates for the median opening and crossover are the same as the capacity of the opening or ramp junction. This is based on opposing flow, critical gap, and follow-up headway as provided by the HCM 2010 equation (equation 2.1 of this report) to compute capacity of stop-controlled movement using a gap acceptance model. The follow-up headway and critical gap were calculated based on HCM equations 19-30 and 19-31 (equations 4.2 and 4.3 of this report) for the respective values of U-turn movement at median openings of MUT and right turn movement at the crossovers of a jughandle.

4.2.6 Delay and LOS at Signalized Intersections

HCM 2010 has provided the stepwise flow chart for the delay and LOS calculation in exhibit 18-11 for a pre-timed and actuated signal. Figure 4.5 shows the procedure for an actuated signal. Step 1 to step 4 includes the calculations of adjusted flow rate and adjusted saturation flow rate. In this study, while using HCS 2010® Streets, all the saturation flow rate adjustment factors were left at default values. Since effective green time and cycle length is not known at first for an actuated signal, the estimation of a green interval duration is an iterative process. This process is described in detail in "Chapter 31: Signalized Intersections: Supplemental" of HCM 2010. According to chapter 31, the initial estimate of green interval duration for a fully actuated signal is equal to the maximum green time. The process goes through several steps until it calculates the green extension time. Finally, it computes green interval duration from average phase duration through several steps after the green extension time is calculated. The process repeats until the difference between the estimated green interval duration and the computed green interval converges by less than 0.1 s. After reaching this convergence, the actual proportion of vehicles arriving during the green phase, the phase duration, and the volume to capacity ratio are calculated. Finally, delay and LOS are computed.

The control delay for a given lane group is the sum of uniform delay (d_1), incremental delay (d_2), and initial queue delay (d_3). HCM 2010 defines the uniform delay (equation 4.8) as the average delay per vehicle due to uniform arrivals, and incremental delay (equation 4.9) is the average delay per vehicle due to random arrivals. The initial queue delay refers to unmet demand in the previous time period; it is 0 s/veh in this study.

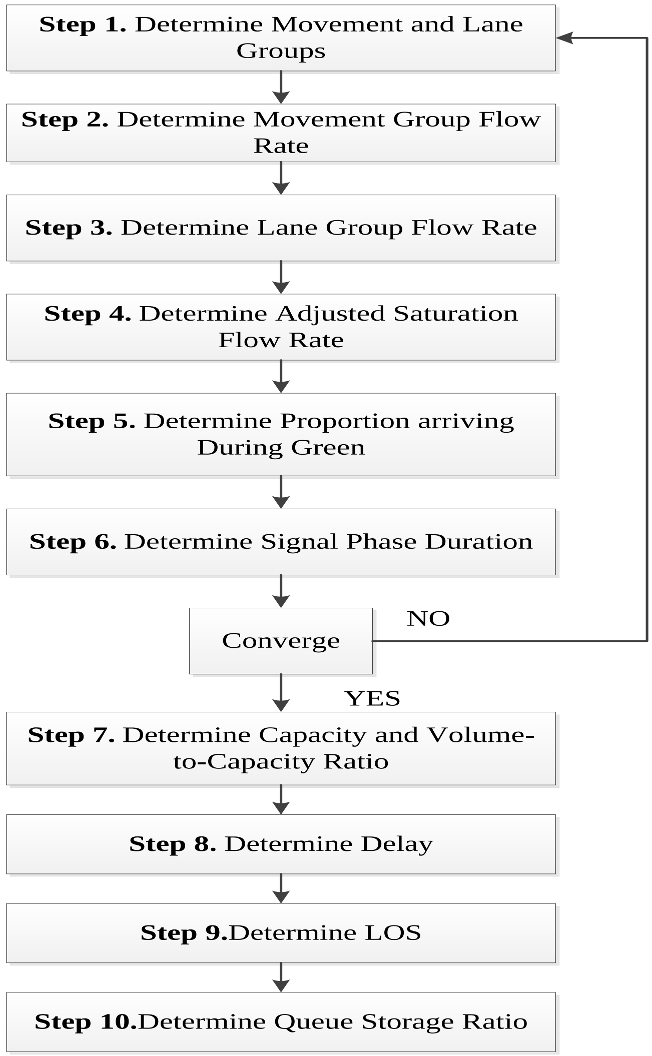


Figure 4.6 HCM method of computing delay and LOS for Actuated Signal

$$d_1 = \frac{0.5\, C\, (1 - {}^{g}/_{C})^2}{1 - \left[\min(1, X)\, {}^{g}/_{C}\right]} \tag{4.8}$$

$$d_2 = 900\, T \left[(X_A - 1) + \frac{8\, k\, I\, X_A}{C_A T} \right] \tag{4.9}$$

Where,

C = cycle Length,

g = effective green time for lane group,

X = volume to capacity ratio (v/c) or degree of saturation for lane group,

T = duration of analysis period (h),

k = incremental delay factor that is dependent on controller settings,

I = upstream filtering/metering adjustment factor,

C_A = lane group capacity (veh/h), and

X_A = lane group v/c ratio or degree of saturation.

Since the actuated signal phase has the ability to adapt its green interval duration to serve the demand on a cycle–by-cycle basis, the K factor should be taken into account for actuated signal operations, which is expressed in the following equations:

$$K = (1 - 2\,K_{min})\left({}^{v}/_{C_a} - 0.5\right) + K_{min} \;\leq\; 0.50 \qquad 4.10$$

$$K_{min} = -0.375 + 0.354\,PT - 0.0910\,PT^2 + 0.0089\,PT^3 \geq\; 0.04 \qquad 4.11$$

$$C_a = 3600\frac{g_a\,s\,N}{C} \qquad 4.12$$

$$g_a = G_{max} + Y + R_C - l_1 - l_2 \qquad 4.13$$

Where,

C_a= available capacity for lane group served by an actuated phase (veh/h),

K_{min} = minimum incremental delay factor,

PT = passage Time,

g_a = available effective green time,

s = saturation flow rate,

N = number of lanes,

C = cycle length,

G_{max}= maximum green interval,

Y = yellow interval,

R_c = red clearance interval,

l_1 = start-up lost time, and

l_2 = end lost time

The levels of service (LOS) were computed using LOS thresholds established for the automobile mode at signalized intersections listed in Exhibit 18-4 of the HCM 2010.

The operation involved in the permitted left turn movement in an exclusive lane is also provided in “Chapter 31: Signalized Intersections: Supplemental” of HCM 2010. Two effective green times are associated with permitted left turn movement. These are the effective green time for permitted left turn operation (g_p), and the effective green time associated with permitted left turn green time that is not blocked by an opposing queue (g_u). The saturation flow rate for permitted left turn operation is provided in equation 31-97 of HCM 2010 can be expressed as follows:

$$S_p = \frac{v_0 e^{-v_0 {t_{cg}}/{3600}}}{1 - e^{-v_0 {t_{fh}}/{3600}}} \qquad 4.14$$

Where,

S_p = saturation flow rate of permitted left turn movement,

v_0 = opposing demand of flow rate,

t_{cg} = critical Headway = 4.5 sec, and

t_{fh} = follow up headway = 2.5 sec

This is adjusted with saturation flow rate adjustment factors to compute the saturation flow rate for lane groups with a permitted left turn operation in an exclusive lane. This saturation flow rate is ultimately used in the estimation of the capacity of permitted left turn operations in an exclusive lane.

4.2.7 HCS Batch Run

As mentioned in previous sections, the combinations of different levels of variable parameters created 12,980 total combinations for each of the four intersections. The network files were run in batch mode with the developed codes to compute delay. Ten random combinations were picked and run again manually as a quality check. The delay obtained from the manual run of networks was found to exactly match the delay obtained from the batch run aided by code.

4.3 Estimation of Fuel Consumption

The AASHTO Red Book (2010) provides a table that gives fuel consumption in gallons per minute of delay ($gal_{c,min}$) by vehicle type, such as small car, big car, SUV, 2-axle single unit vehicle, 3-axle single unit vehicle, and combo, according to free-flow speed. This table was utilized for the computation of fuel consumption at intersections and crossovers. Six vehicle categories were combined to form two categories: cars and heavy vehicles. The $gal_{c,min}$ of the car vehicle type is the average $gal_{c,min}$ values of small cars, big cars, and SUVs. Similarly, the $gal_{c,min}$ of heavy vehicles is the average $gal_{c,min}$ values of 2-axle single unit vehicles, 3-axle single unit vehicles, and combos. Table 4.3 shows the $gal_{c,min}$ of cars and heavy vehicles computed by this method. To compute fuel consumption, the delay (sec/veh) for each intersection was separately converted to delay in vehicle minutes for cars and trucks. The vehicle minute delay of cars and trucks are multiplied with respective $gal_{c,min}$ from table 4.3 to get fuel consumption by each vehicle type. The free flow speeds were assumed to be 45 mph for the major street and 35 mph for the minor street.

Table 4.3 Fuel consumption (gallons) per minute of delay by vehicle type based on the AASHTO Red Book (2010)

Free Flow Speed (mph)	Small Cars	Heavy Vehicles (Trucks)
20	0.02	0.12
25	0.02	0.16
30	0.03	0.19
35	0.03	0.23
40	0.03	0.26
45	0.04	0.30
50	0.04	0.34
55	0.05	0.37
60	0.06	0.41
65	0.06	0.45
70	0.07	0.49
75	0.08	0.53

A table in the AASHTO Red Book (2010) that provides the fuel consumption in gallons per mile for auto and trucks with respect to the operating speed was referenced to calculate fuel consumption from travel delay related to MUT and jughandles. Fuel consumption in gallons was estimated considering a 45 mph operating speed for MUT to travel from the intersection to median openings and vice versa, 30 mph for a jughandle ramp, and 45 mph along the major street. The values from the AASHTO Red Book (2010) table are also in table 4.4.

Table 4.4 Fuel consumption related to operating speed

Speed (mph)	Gallons per Mile	
	Autos	**Trucks**
5	0.117	0.053
10	0.075	0.316
15	0.061	0.254
20	0.054	0.222
25	0.05	0.204
30	0.047	0.191
35	0.045	0.182
40	0.044	0.176
45	0.042	0.170
50	0.041	0.166
55	0.041	0.163
60	0.040	0.160
65	0.039	0.158

4.4 Estimation of Emissions

This study estimated four major types of vehicular emissions: carbon monoxide (CO), oxides of nitrogen ($NO_{x)}$, volatile oxygen compounds (VOCs), and carbon dioxide (CO_2). Cobian, Henderson, Sudeshna, Nuworsoo, and Sullivan (2009) developed the factors to convert fuel consumption in gallons to gram units of emissions, like CO, NO_x and VOCs. These factors are 69.9 gram/gallon for CO, 13.6 gram/gallon for $NO_{x,}$ and 16.2 gram/gallon for VOCs. Similarly, the U.S. Department of Energy has published a document called "Instructions for Form EIA-1605: Voluntary Reporting of Greenhouse Gases" in 2007, which relates CO_2 emissions in grams with petrol and diesel fuel consumption. The conversion factor for petrol consumption to CO_2 emissions is 17.59 g/gallon, and the conversion factor for diesel consumption to CO_2 emissions is 22.37 g/gallon. The conversions are shown in table 4.5.

Table 4.5 Relationship between fuel consumption and emissions

Emissions	Relationship with Fuel Consumption	Source of Information
CO	Fuel consumption (gallon) * 69.9 gram/gallon	Cobian et al. (2009)
NO_x	Fuel consumption (gallon) * 13.6 gram/gallon	
VOCs	Fuel consumption (gallon) * 16.2 gram/gallon	
CO_2	Fuel consumption (Petrol) (gallon)*17.59 gram/gallon + Fuel consumption (Diesel) (gallon)*22.37 gram/gallon	Instruction for form EIA-1605: Voluntary Reporting of Greenhouse Gases (2007)

Chapter 5 Sensitivity Analysis

This chapter deals with the relative delay performance of unconventional intersections under different approach volumes with respect to each other and standard signalized intersections. It includes descriptions of different types of plots used to develop decision assistance curves (DAC), which allocate the volume regions where the performance of each type of unconventional intersections can be optimal.

5.1 Decision Assistance Curves (DAC) - Classification

DAC refer to the boundary lines that allocate areas of optimal performance of unconventional intersections produced on plots with the major street and minor street approach volumes on the X and Y axes. The development of DAC includes two steps described in the following subsections.

5.1.1 Clusters

The total intersection delay of all three unconventional intersections was compared with each other for all 12,960 combinations. The intersection producing the minimal delay was chosen for each combination. Since the minimal delay from the comparison was always less than the intersection delay of a standard signalized intersection, the standard intersection delay was excluded. The group scatter plots were developed while taking the major street approach volumes, minor street approach volumes, and group based on the type of unconventional intersection that produced the minimal delay into account. The scatter plot showed the clusters of each intersection in reference to the X axis as the major street approach volume and the Y axis as the minor street approach volume. An example scatter plot showing the clusters of each unconventional intersection is shown in figure 5.1. In figure 5.1, the three intersections, MUT, jughandle, and CFI are clustered in three different locations in the plot of major street approach

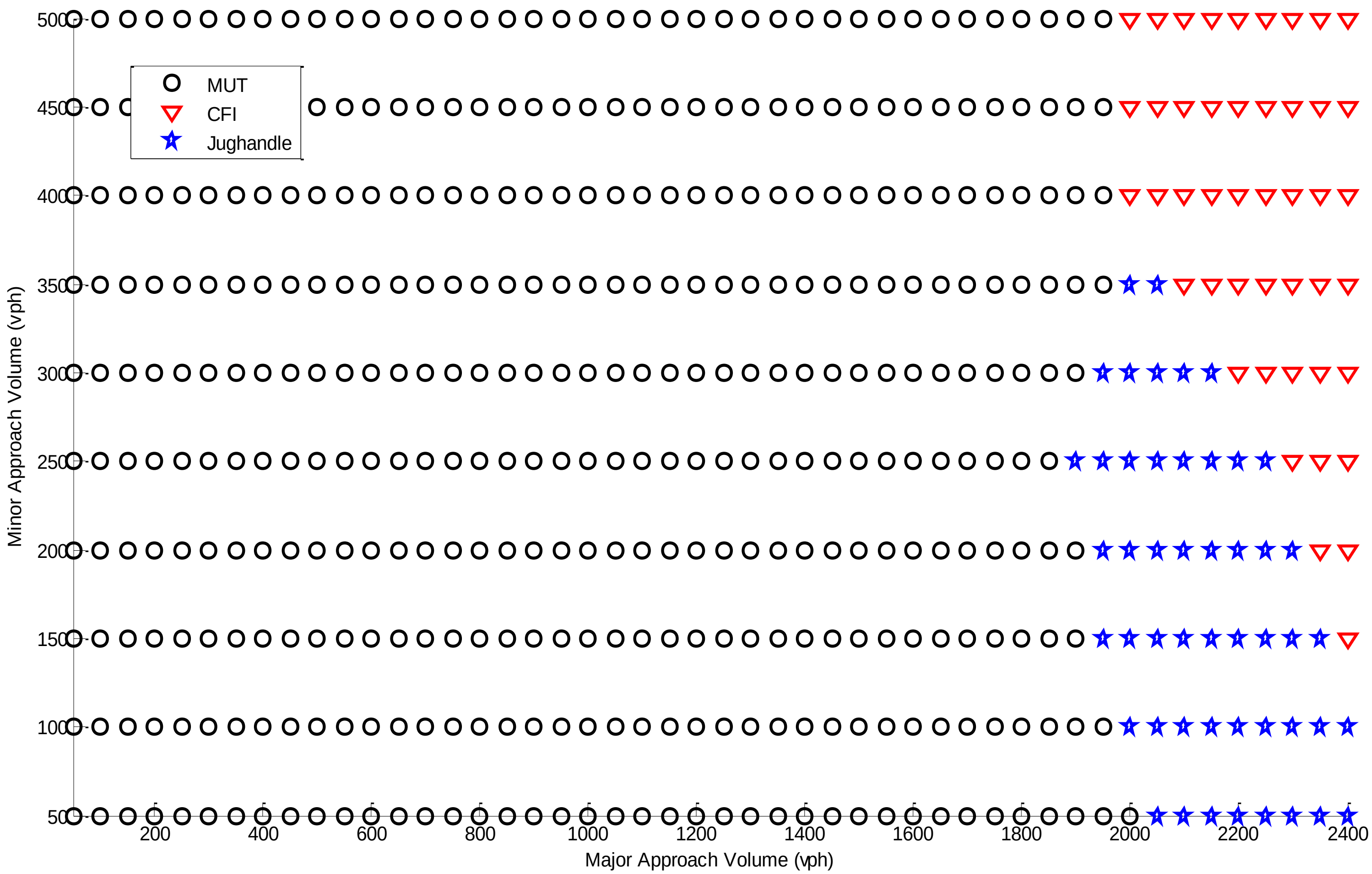


Figure 5.1 Example scatter plot showing clusters at BF=0.7, LTP=15%, TP=2%

volume (X axis) and minor street approach volume (Yaxis).

5.1.2 Application of Classifier and Development of the Equations for DAC

To separate the regions for the clusters developed as explained in section 5.1.1, the Quadratic Discriminate Analysis (QDA) was performed. The QDA was chosen over Linear Discriminant Analysis (LDA) because of three main reasons: (1) QDA allows more flexibility for covariance matrix for each class, (2) the boundary lines as observed from scatter plots were not linear, and (3) QDA classified the clusters very accurately, which can be observed from table 5.2. The DAC plots were done based on a QDA that defined the areas where the delay performance of each intersection was optimal. The equations for the DAC developed from QDA are shown in table 5.2. These equations follow the form of equation 5.2.

$$Z = K + [X_1, X_2] \times \boldsymbol{L} + [X_1, X_2] \times \boldsymbol{Q} \times [X_1, X_2]^{\boldsymbol{T}} \qquad 5.1$$

Where,

Z= quadratic Classifier,

K= constant,

L= linear coefficient matrix,

Q =quadratic coefficient matrix,

X_1 = total major street approach volume, and

X_2 = total minor street approach volume

The performance of the classifier was tested using the following performance table (table 5.1). The precision and accuracy were calculated using equations 5.2 and 5.3. The precision and accuracy of each classification are shown in table 5.2, along with the confusion matrix. The confusion matrix is simply a classification table where the first row stands for the condition for non-optimality of a particular intersection, and the second row stands for the optimality of a particular unconventional intersection. Table 5.2 also shows the re-substitution error, which is the difference between the original and predicted classification.

Table 5.1 Performance evaluation of classifier

		Conditions	
Classifier	Prediction that a particular unconventional intersection is optimal	True prediction that a particular unconventional intersection is optimal	False prediction that a particular unconventional intersection is optimal.
	Prediction that a particular unconventional intersection is not optimal	False prediction that a particular unconventional intersection is not optimal	True prediction that a particular unconventional intersection is not optimal

$$Accuracy = \frac{Number\ of\ true\ predicted\ optimality + Number\ of\ true\ predicted\ non\ optimality}{Total\ numbers} \tag{5.2}$$

$$Precision = \frac{Number\ of\ true\ predicted\ optimality + Number\ of\ true\ predicted\ non\ optimality}{Number\ of\ true\ predicted\ optimality + Number\ of\ false\ \ predicted\ \ optimality} \tag{5.3}$$

Table 5.2 Equations of DAC (Part I)

Coefficients				Combinations	Corresponding Figure in Appendix B	Corresponding Curve in Figure	Resubstitution Error	Confusion Matrix		Precision and Accuracy	
K	L	Q								Precision (%)	Accuracy (%)
-817.60	0.685	-1.44E-04	-2.26E-05	BF=0.5, LTP=10% and TP=2%	Figure B.4	MUT Threshold	0.008	34	1	100	99.17
	0.114	-2.26E-05	-9.39E-06					3	442		
-701.82	0.594	-1.25E-04	-1.26E-05	BF=0.6, LTP=10% and TP=2%	Figure B.5	MUT Threshold	0.017	34	3	99	98.33
	0.065	-1.26E-05	-7.71E-06					5	438		
-746.32	0.639	-1.37E-04	7.86E-07	BF=0.7, LTP=10% and TP=2%	Figure B.6	MUT Threshold	0.017	34	2	100	98.33
	0.000	7.86E-07	-5.50E-06					6	438		
795.14	-0.669	1.40E-04	3.42E-05			Jughandle Threshold	0.010	452	2	92	98.96
	-0.174	3.42E-05	4.44E-05					3	23		
-90.75	0.080	-1.73E-05	-3.12E-06	BF=0.5, LTP=15% and TP=2%	Figure B.7	MUT Threshold	0.008	108	2	99	99.17
	0.016	-3.12E-06	-1.70E-06					2	368		
-94.71	0.085	-1.85E-05	-1.31E-06	BF=0.6, LTP=15% and TP=2%	Figure B.8	MUT Threshold	0.017	102	3	99	99.16
	0.008	-1.31E-06	-2.77E-06					5	370		
934.23	-0.494	8.10E-05	1.23E-04			CFI Threshold	0.002	470	1	90	99.79
	-1.504	1.23E-04	9.60E-04					0	9		
-122.92	0.111	-2.44E-05	1.80E-07	BF=0.7, LTP=15% and TP=2%	Figure B.9	MUT Threshold	0.006	92	2	99	99.38
	0.000	1.80E-07	-1.58E-06					1	385		
240.24	-0.187	3.72E-05	2.24E-05			CFI Threshold	0.013	433	2	95	98.75
	-0.144	2.24E-05	4.67E-05					4	41		
-536.70	0.453	-9.56E-05	-1.59E-05	BF=0.5, LTP=10% and TP=5%	Figure B.13	MUT Threshold	0.010	43	2	100	98.96
	0.080	-1.59E-05	-6.46E-06					3	432		

Table 5.3 Equations of DAC (Part II)

Coefficients				Combinations	Corresponding Figure in Appendix B	Corresponding Curve in Figure	Resubstitution Error	Confusion Matrix		Precision and Accuracy	
K	L	Q								Precision (%)	Accuracy (%)
-498.10	0.43	-9.12E-05	-4.98E-06	BF=0.6, LTP=10% and TP=5%	Figure B.14	MUT Threshold	0.017	42	2	100	98.33
	0.03	-4.98E-06	-6.26E-06					6	430		
-585.82	0.50	-1.08E-04	-2.24E-07	BF=0.7, LTP=10% and TP=5%	Figure B.15	MUT Threshold	0.010	41	2	100	98.96
	0.00	-2.24E-07	-2.96E-06					3	434		
1007.05	-0.81	1.65E-04	4.01E-05			CFI Threshold	0.002	464	0	100	99.79
	-0.29	4.01E-05	1.11E-04					1	15		
-77.41	0.07	-1.47E-05	-2.96E-06	BF=0.5, LTP=15% and TP=5%	Figure B.16	MUT Threshold	0.010	116	2	99	98.96
	0.02	-2.96E-06	-1.85E-06					3	359		
-85.97	0.08	-1.71E-05	-3.21E-07	BF=0.6, LTP=15% and TP=5%	Figure B.17	MUT Threshold	0.023	104	5	99	97.71
	0.00	-3.21E-07	-3.05E-06					6	365		
491.65	-0.26	4.14E-05	7.68E-05			CFI Threshold	0.004	464	1	100	99.79
	-0.82	7.68E-05	4.93E-04					0	15		
-105.21	0.09	-2.09E-05	-2.94E-07	BF=0.7, LTP=15% and TP=5%	Figure B.18	MUT Threshold	0.008	99	3	99	99.17
	0.00	-2.94E-07	-1.10E-06					1	377		
194.09	-0.16	3.17E-05	1.72E-05			CFI Threshold	0.013	416	3	95	98.75
	-0.10	1.72E-05	2.08E-05					3	58		

Table 5.4 Equations of DAC (Part III)

Coefficients				Combinations	Corresponding Figure in Appendix B	Corresponding Curve in Figure	Resubstitution Error	Confusion Matrix		Precision and Accuracy	
K	L	Q								Precision (%)	Accuracy (%)
-469.95	0.40	-8.45E-05	-1.21E-05	BF=0.5, LTP=10% and TP=10%	Figure B.22	MUT Threshold	0.017	44	3	94	98.33
	0.06	-1.21E-05	-5.91E-06					5	428		
-459.48	0.40	-8.49E-05	-2.74E-06	BF=0.6, LTP=10% and TP=10%	Figure B.23	MUT Threshold	0.021	43	3	99	99.92
	0.02	-2.74E-06	-6.09E-06					7	427		
-510.67	0.44	-9.54E-05	1.66E-06	BF=0.7, LTP=10% and TP=10%	Figure B. 24	MUT Threshold	0.017	44	2	100	98.33
	-0.01	1.66E-06	-8.65E-07					6	428		
802.59	-0.66	1.38E-04	2.47E-05			CFI Threshold	0.006	456	1	95	99.38
	-0.16	2.47E-05	5.20E-05					2	21		
-74.19	0.07	-1.43E-05	-1.95E-06	BF=0.5, LTP=15% and TP=10%	Figure B.25	MUT Threshold	0.010	118	2	99	98.96
	0.01	-1.95E-06	-1.50E-06					3	357		
-82.24	0.07	-1.65E-05	1.19E-07	BF=0.6, LTP=15% and TP=10%	Figure B.26	MUT Threshold	0.006	107	5	99	98.33
	0.00	1.19E-07	-2.06E-06					3	365		
314.80	-0.20	3.57E-05	4.51E-05			CFI Threshold	0.006	453	1	96	99.38
	-0.38	4.51E-05	1.88E-04					2	24		

Table 5.5 Equations of DAC (PartIV)

Coefficients				Combinations	Corresponding Figure in Appendix B	Corresponding Curve in Figure	Resubstitution Error	Confusion Matrix		Precision and Accuracy	
K	L	Q								Precision (%)	Accuracy (%)
-104.63	0.09	-2.09E-05	1.27E-07	BF=0.7, LTP=15% and TP=10%	Figure B.27	MUT Threshold	0.015	99	3	99	99.17
	0.00	1.27E-07	-8.69E-07					1	377		
163.62	-0.14	2.85E-05	1.13E-05			CFI Threshold	0.021	399	6	92	97.92
	-0.06	1.13E-05	1.03E-05					4	71		

5.1.3 Discussion on Developed DAC

The QDA was used to develop the DAC for all 12,960 combinations under 27 combinations of three levels: of balance factor (BF), left turn percentage (LTP), and truck percentage (TP). The obtained 27 plots with thresholds are included in Appendix B in figure B.1 to figure B.27. The example study on how to use DAC plots is included in Appendix C. The following patterns were observed from the DAC:

1. For a low percentage of left turns (5%), MUT performed optimally on all conditions of BF and TP under all levels of major street approach volume and minor street approach volume from a delay-saving standpoint.
2. For a medium percentage of left turns (10%) and BF at 0.5 and 0.6, the threshold between MUT and a jughandle was found to be between 2200 vph and 2400 vph of major street approach volume. However, the threshold flared toward the left to provide a wide area for the optimal region for the jughandle as minor street approach volume increased. From this, it can be inferred that for this particular condition, a jughandle performed better at high major street volumes, and its performance increased further when the minor street approach volume increased. Still, the higher portion of the region was occupied by MUT, and there was no CFI region for this particular condition. Figure 5.2 represents this pattern. When the balance factor increased to 0.7 for the same left turn percentage (10%), CFI was introduced to the high major street approach volume and high minor street approach volume region, as shown by figure 5.3. From this, it can be inferred that high left turns on any one major street and high minor street approach volume conditions favors the application of CFI.
3. For a high percentage of left turns (15%) and BF at 0.5, which represents the balance condition under which both approaches on the major street have the same volume of traffic,

the thresholds of the jughandle shifted toward the left, between 1800 vph to 2000 vph major street approach volume. The threshold flared toward the left to provide a wide area for the optimal region for the jughandle as the minor street approach volume increased. From this, it can be inferred that at a balanced condition, the jughandle performed better at high major street volumes and performed even better when the minor street approach volume increased. Still, the higher portion of the region was occupied by MUT, and there was no CFI region at this particular condition. Figure 5.4 represents this pattern.

Similarly, when the BF increases from 0.5 to 0.6, the CFI was introduced in the region of high major street approach volume and high minor street approach volume. With a further increase in BF, from 0.6 to 0.7, the optimal region of CFI extended to low minor street approach volume. This can be visualized from figures 5.5 and 5.6. From this, it can be conferred that under high left turning volumes on both approaches, the unbalanced flow conditions also led to increased left-turning volume at a high major street approach volume, which ultimately favored CFI to serve the high left-turning volume.

4. Truck percentage did not that perceptible an effect on other conditions except at a high left turn percentage (15%) and a high balance factor (0.7), where high truck percentage favors the increment of the optimal region of CFI. This is clear from figure 5.7, figure 5.8, and figure 5.9.

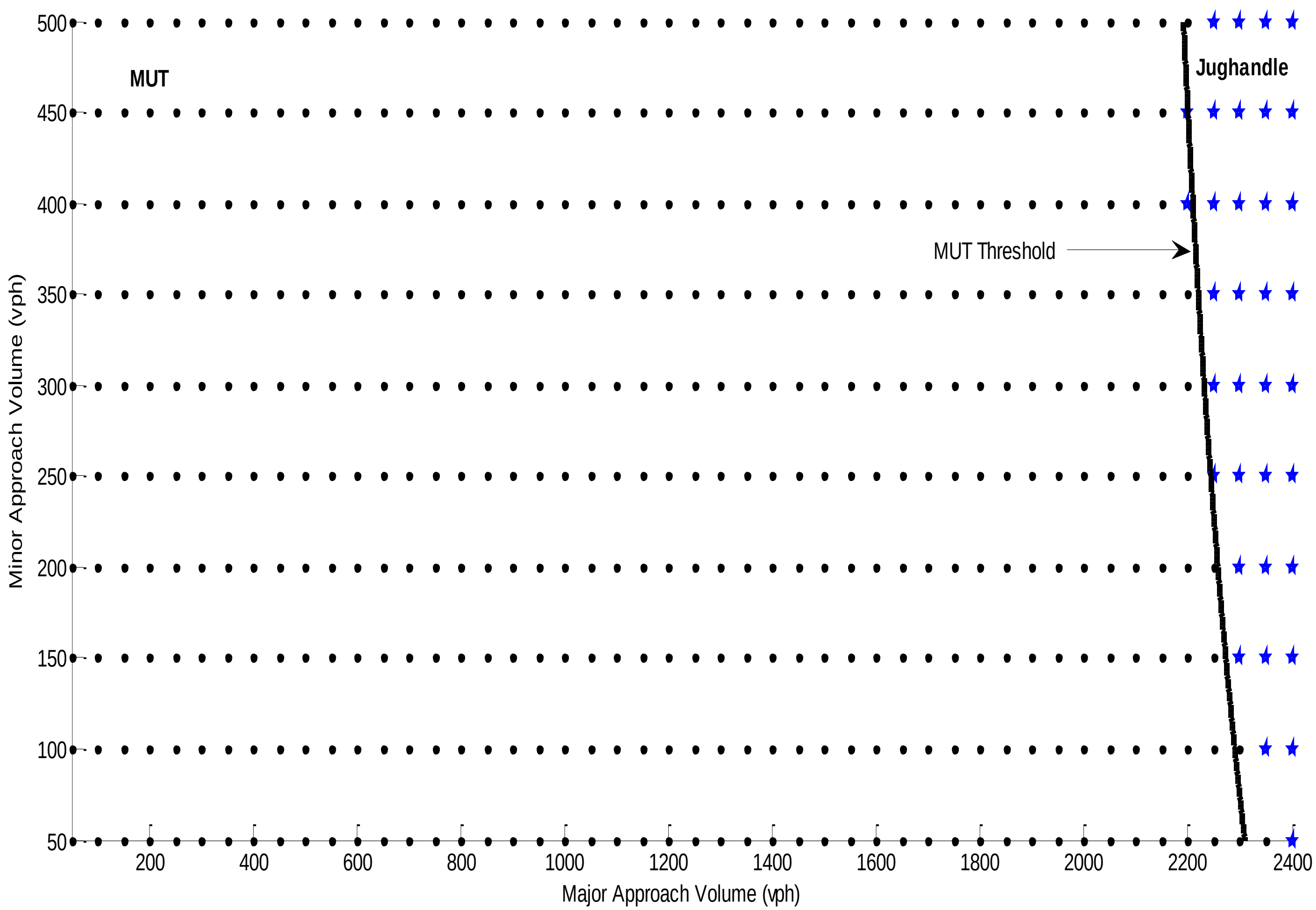


Figure 5.2 DAC for unconventional intersection at BF=0.5, LTP=10% and TP=2%

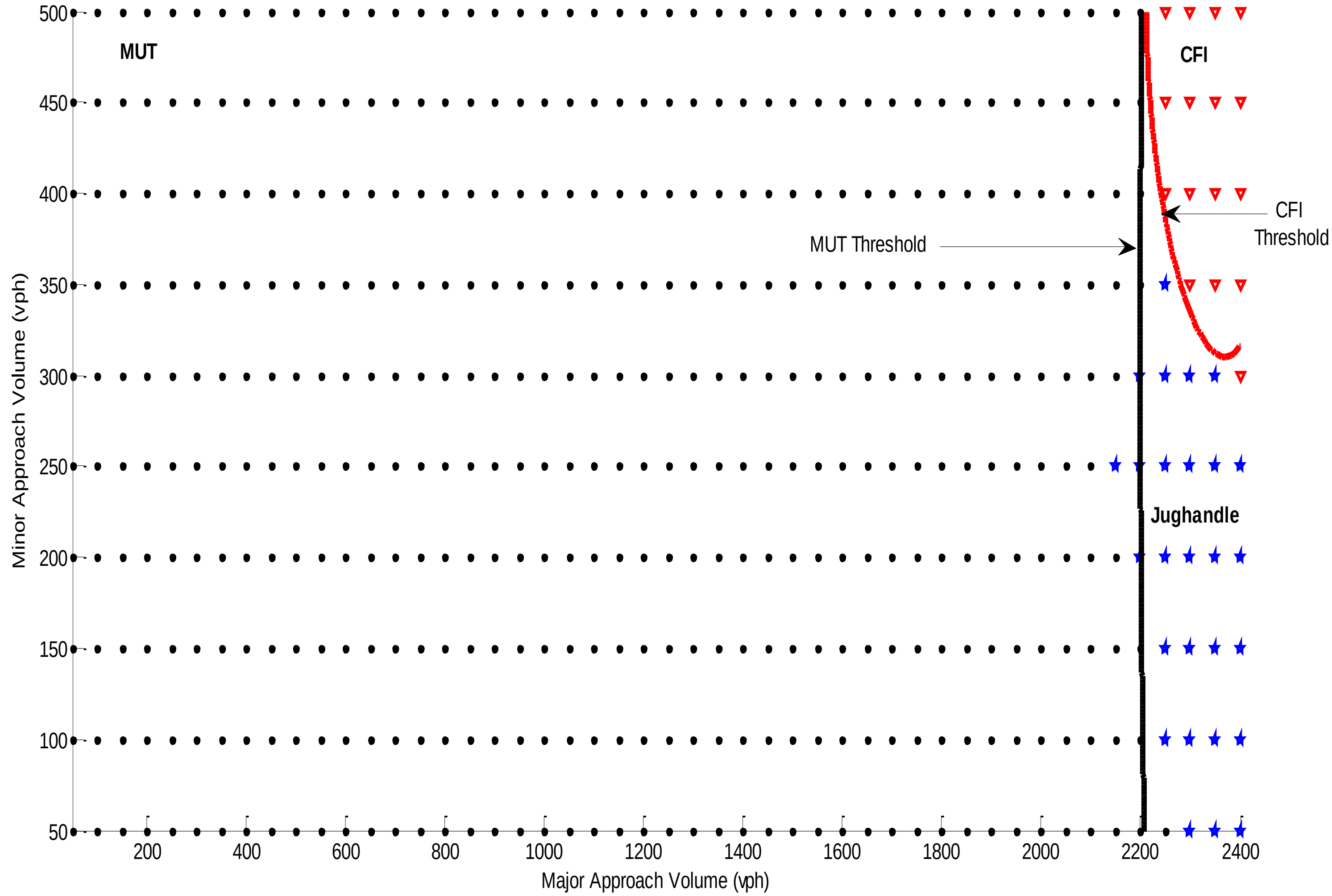


Figure 5.3 DAC for unconventional intersection at BF=0.7, LTP=10% and TP=5%

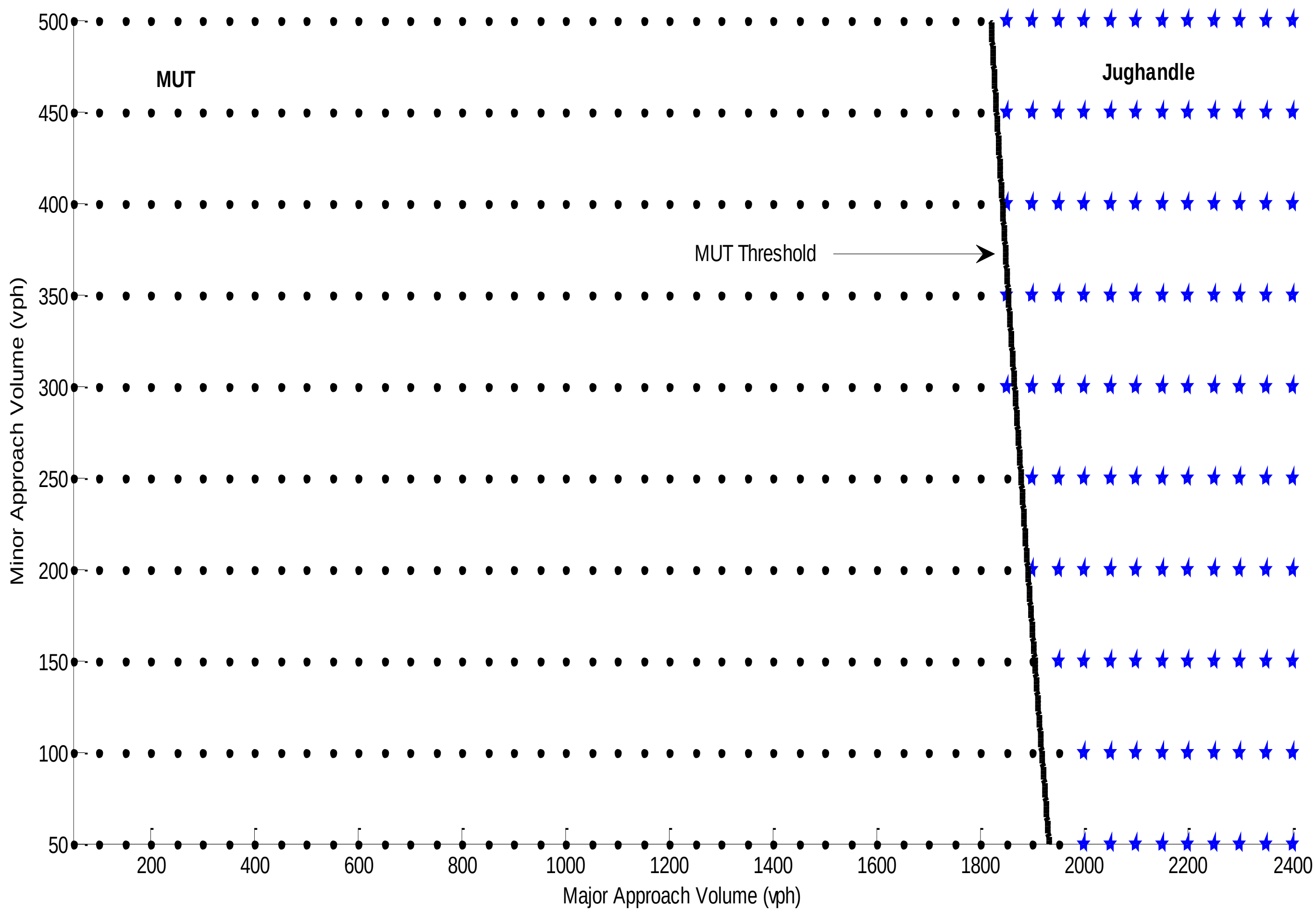


Figure 5.4 DAC for unconventional intersection at BF=0.5, LTP=15% and TP=2%

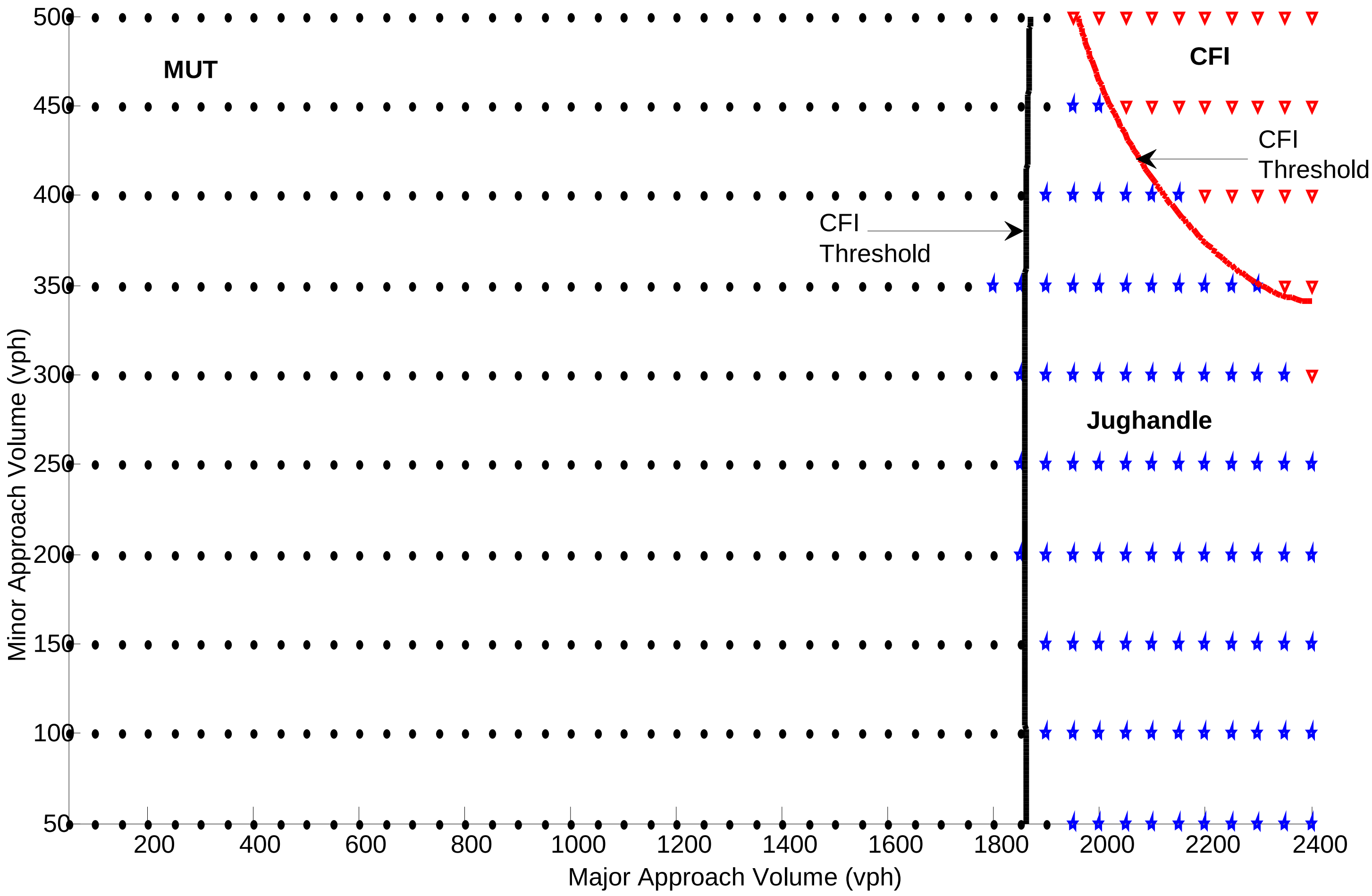


Figure 5.5 DAC for unconventional intersection at BF=0.6, LTP=15% and TP=10%

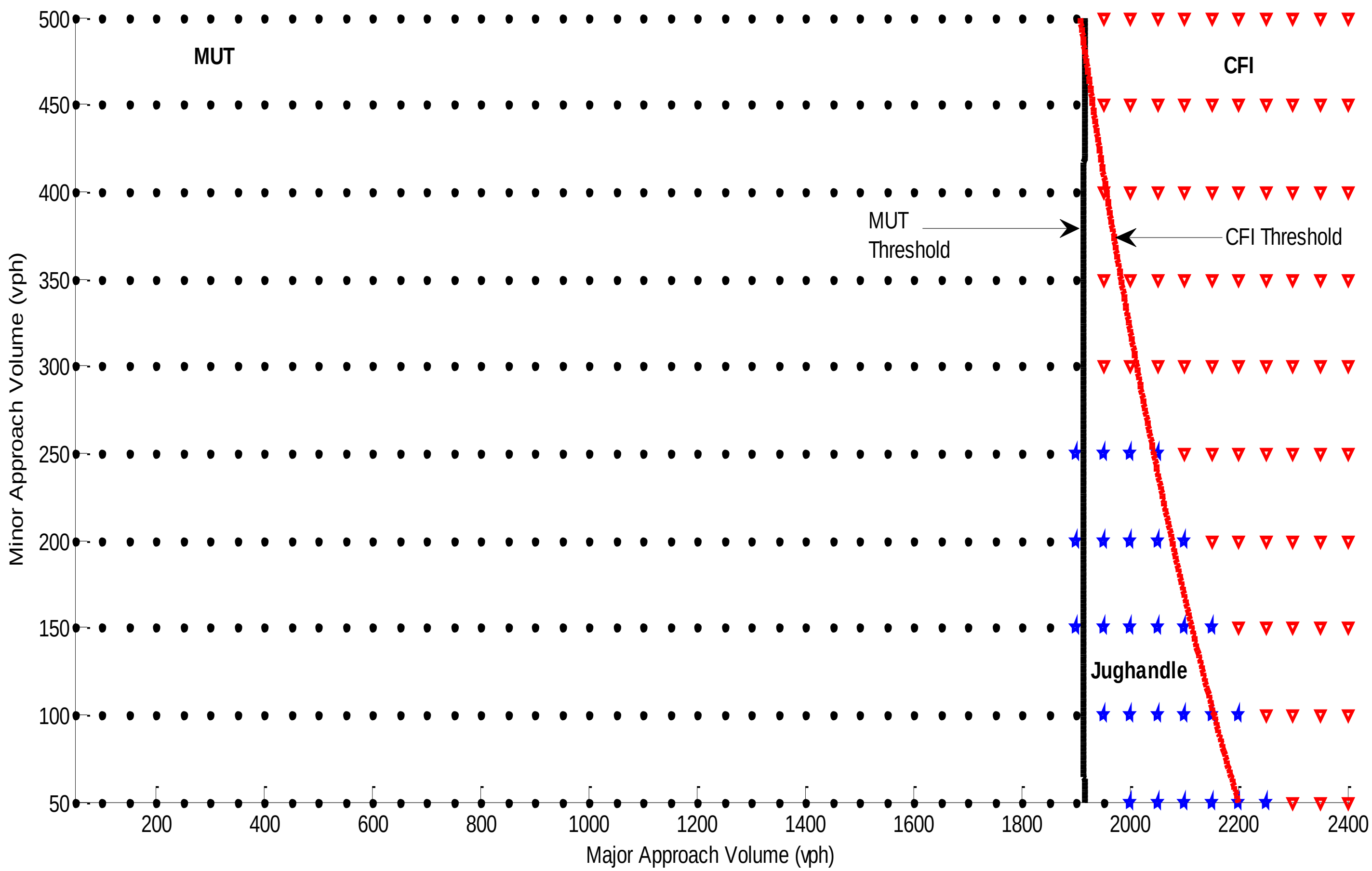


Figure 5.6 DAC for unconventional intersection at BF=0.7, LTP=15% and TP=10%

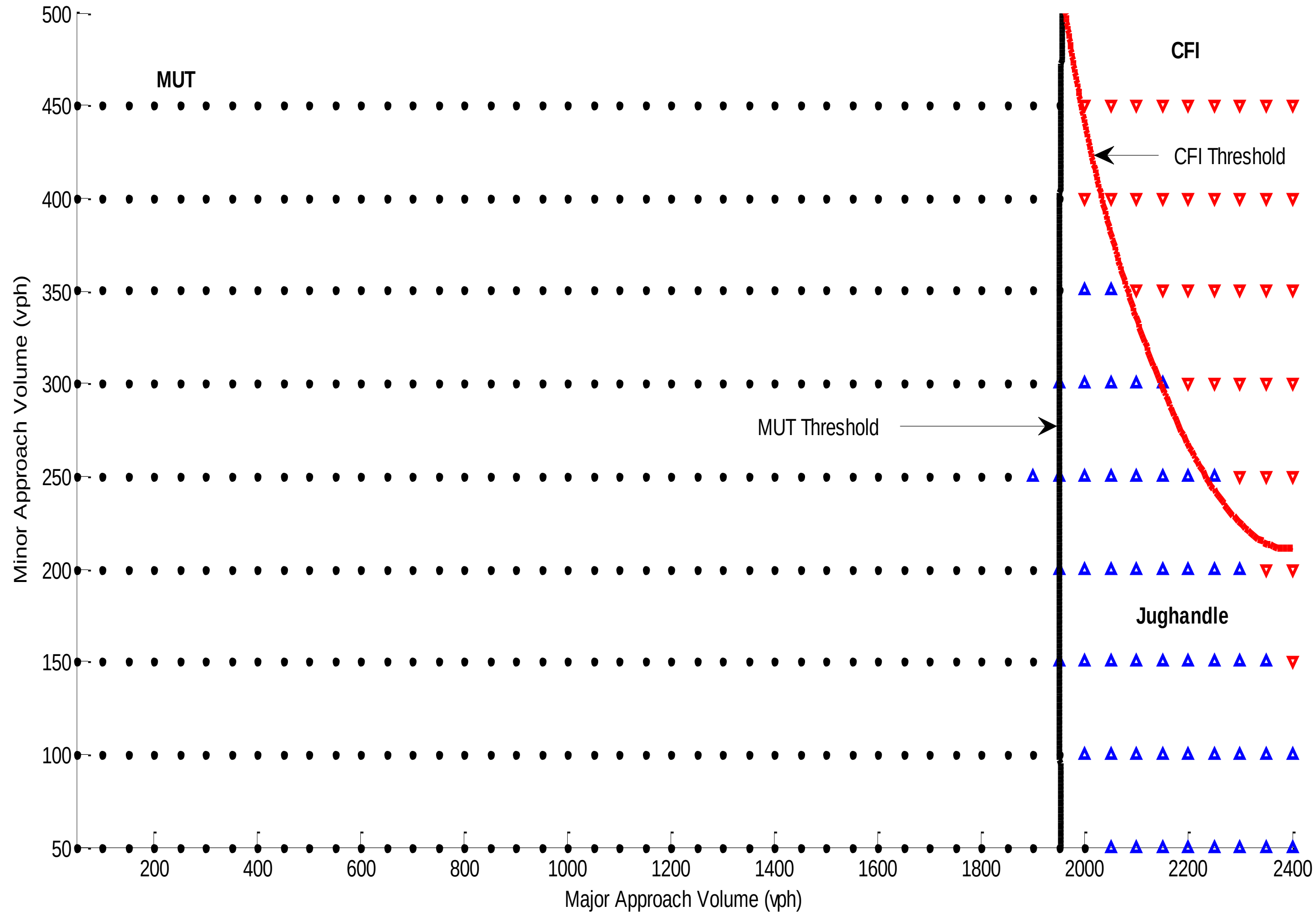


Figure 5.7 DAC for unconventional intersection at BF=0.7, LTP=15% and TP=2%

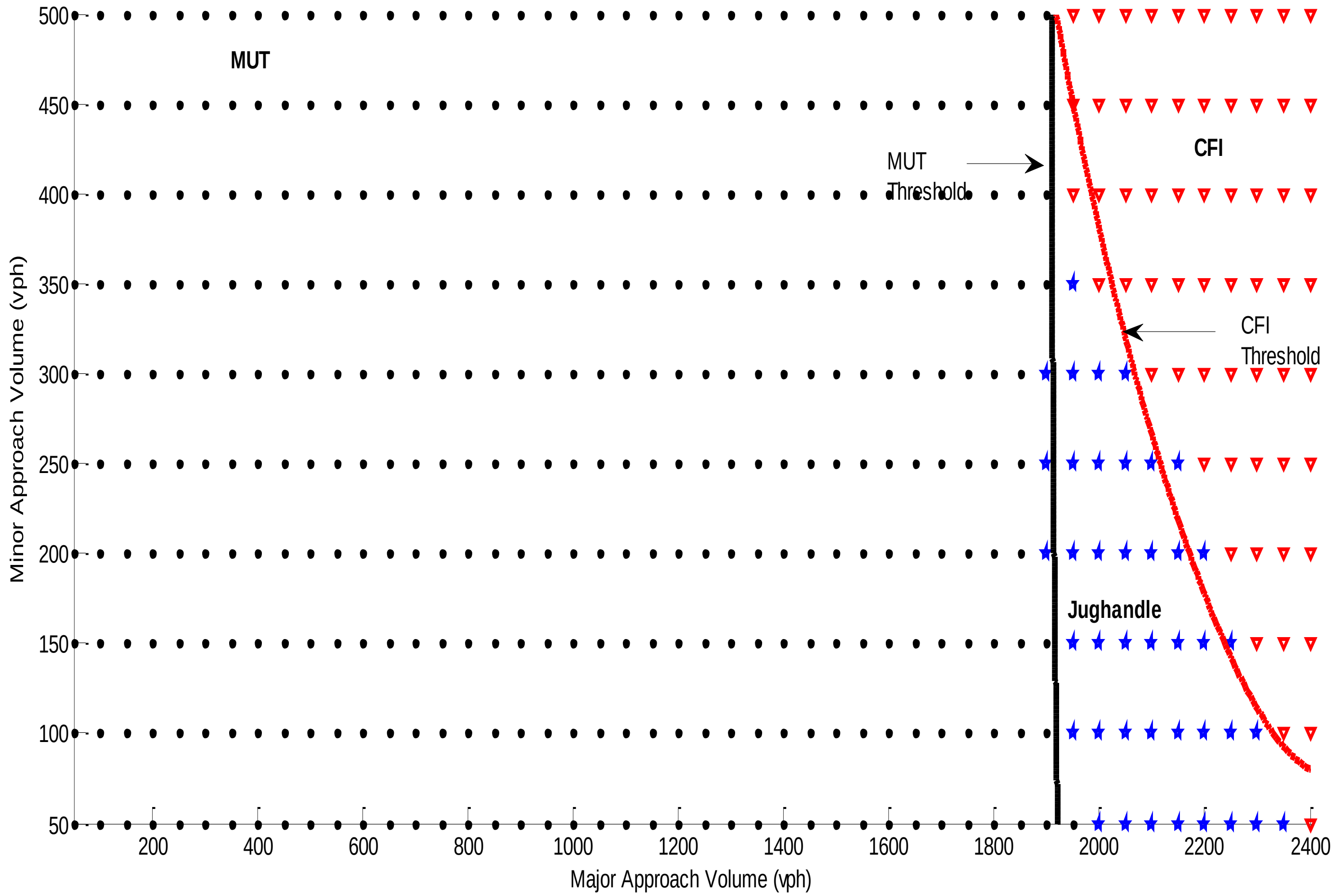


Figure 5.8 DAC for unconventional intersection at BF=0.7, LTP=15% and TP=5%

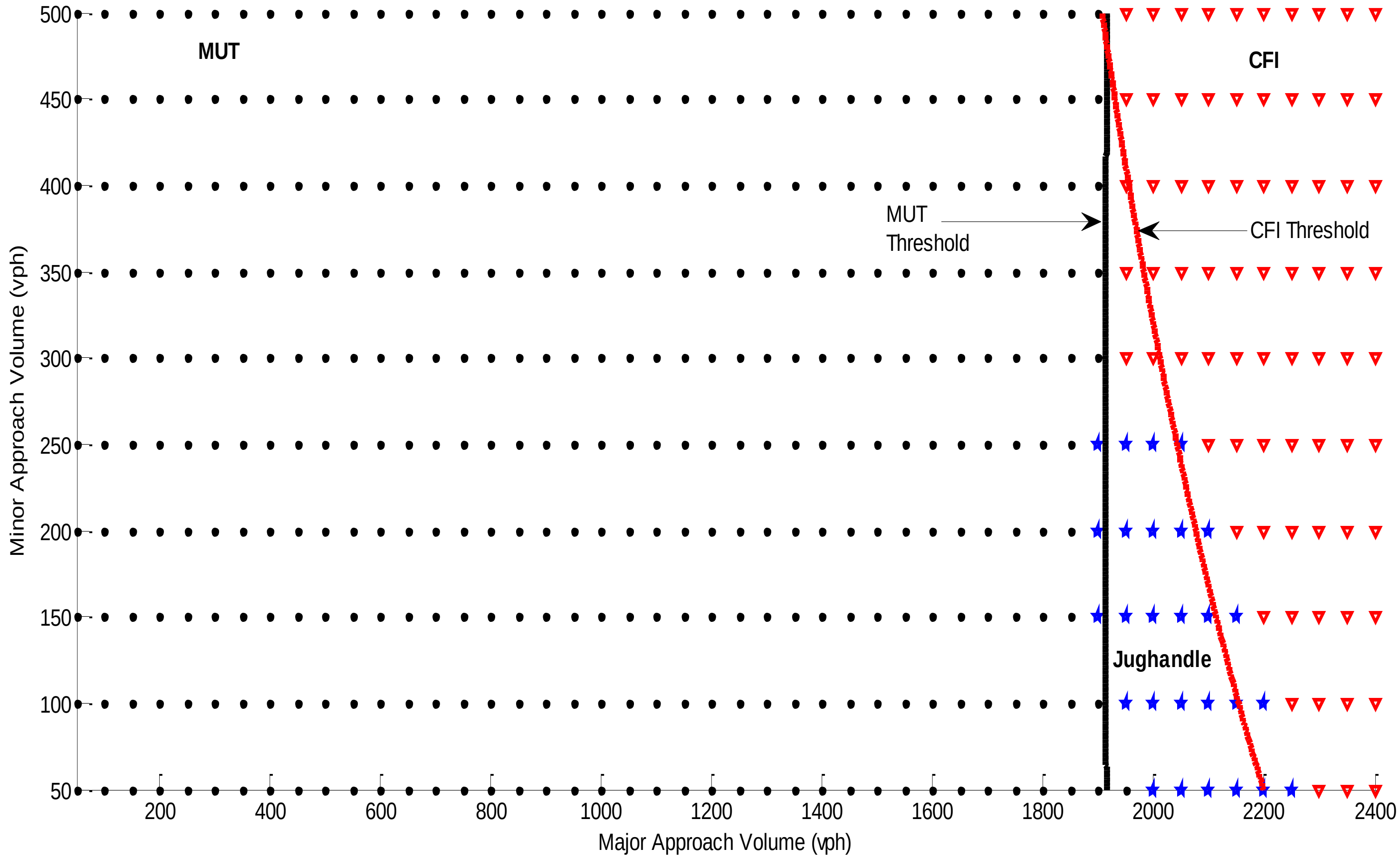


Figure 5.9 DAC for unconventional intersection at BF=0.7, LTP=15% and TP=10%

5.2 Decision Assistance Curves (DAC)-Contour

From DAC, it is possible to select the optimal choice among three unconventional intersections for any volume criteria. Furthermore, DAC was supplemented with additional information about how to obtain how much delay loss or benefit the optimal intersection would have for those volume criteria. This was done by developing two types of contour plots. The first type of contour plot was developed for the highest delay difference between the left turn movement of a standard signalized intersection and two unconventional intersections, including MUT and jughandle, in seconds per vehicle. The second type of contour plot was developed for all three unconventional intersections, indicating the total intersection delay difference with respect to a standard signalized intersection in seconds per vehicle.

5.2.1 Contour Plots for Difference in Left Turn Delay

MUT and jughandle were chosen for these plots because the left turners on a MUT and jughandle are expected to have higher delay due to additional travel delay and delay at crossovers. The plots were developed for the meshgrid of minor approach volume (50 vph to 500 vph) and major approach volume (50 vph to 2400 vph) for 27 combinations of the three levels: balance factor, truck percentage, and left turn percentage. The two example plots at 0.7 BF, 15% LTP, and 10% TP are shown in figures 5.10 and 5.11 for MUT and jughandle, respectively. The negative values of the contour represent the delay saving capability of unconventional intersections for left turns compared to a standard signalized intersection. The positive values of contour represent when unconventional intersections have higher delay than a standard signalized intersection.

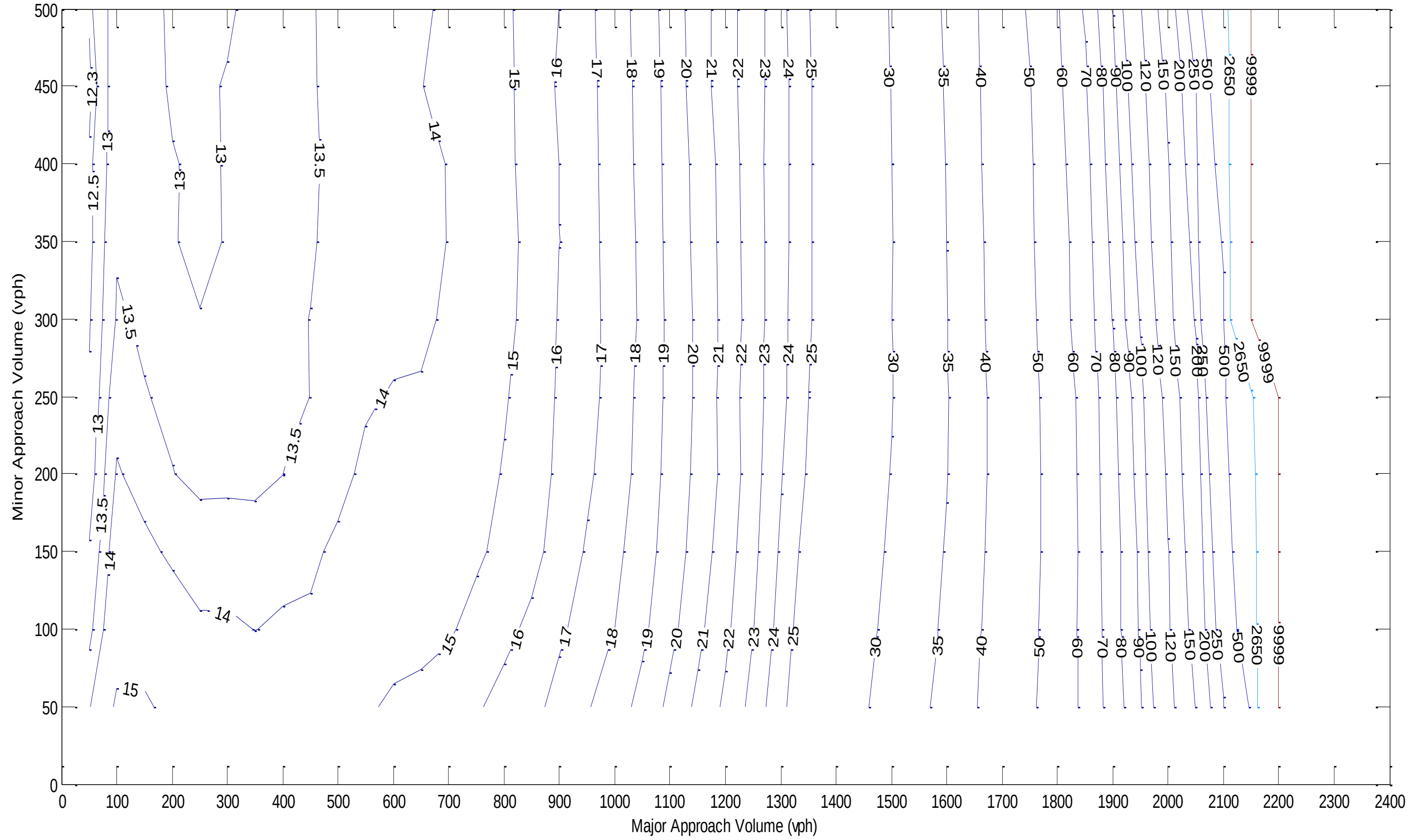


Figure 5.10 Contour plot for critical left turn delay difference between MUT and standard signalized intersection at BF=0.7, LTP=15%, TP=10%

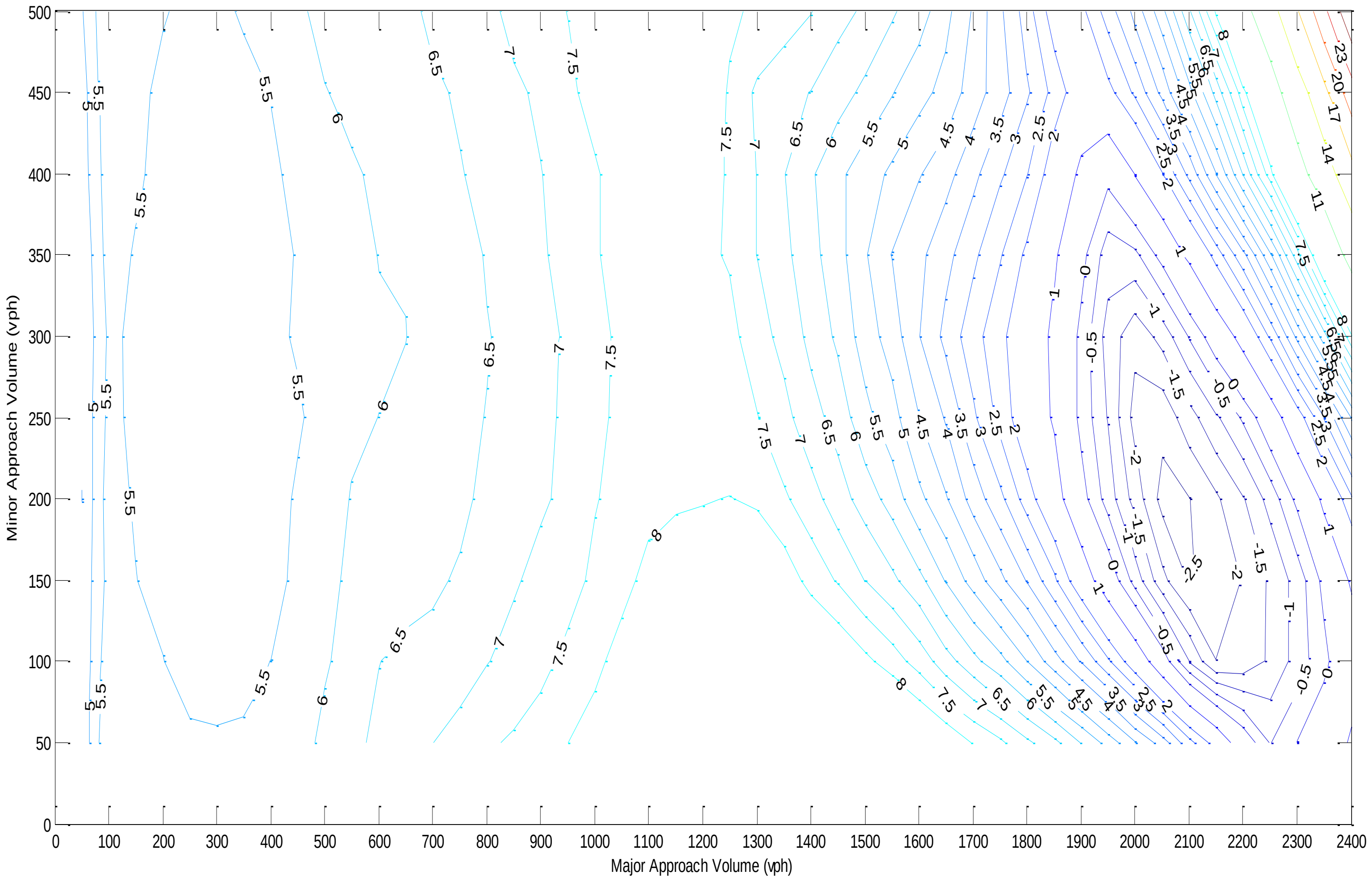


Figure 5.11 Contour plot for critical left turn delay difference between jughandle and standard signalized intersection at BF=0.7, LTP=15%, TP=10%

The contour plots indicated that the left turn delay of critical left turn movement for MUT was always higher than that of a signalized intersection. The delay difference increased at a higher rate for higher major street approach volumes. In the combinations with a high left turning volume, the left turn delay was exceptionally higher at high major approach volumes, even reaching the condition of failure. The condition of failure occurred as the left-turning volumes exceeded the capacity of the U-turn openings. In figure 5.10, the condition of failure is indicated by a contour line designated with the value of 9999. All the MUT contour plots are included in Appendix B of this report.

Unlike MUT, the contour plot indicated that the critical left turn delay difference between jughandles and standard signalized intersections was less, and even negative, for higher major street and minor street approach volumes, showing less delay or delay-saving conditions for left turn movements. All the contour plots for the jughandle are included in Appendix B of this report.

5.2.2 Contour Plots for Difference in Total Intersection Delay

Contour plots for difference in total intersection delay were developed for all three unconventional intersections. The negative values of the contour represent the delay-saving capability of unconventional intersections compared to standard signalized intersections. The positive contour values represent unconventional intersections that have a higher delay than a standard signalized intersection. The example delay difference plots are shown in figures 5.12, 5.13, and 5.14.

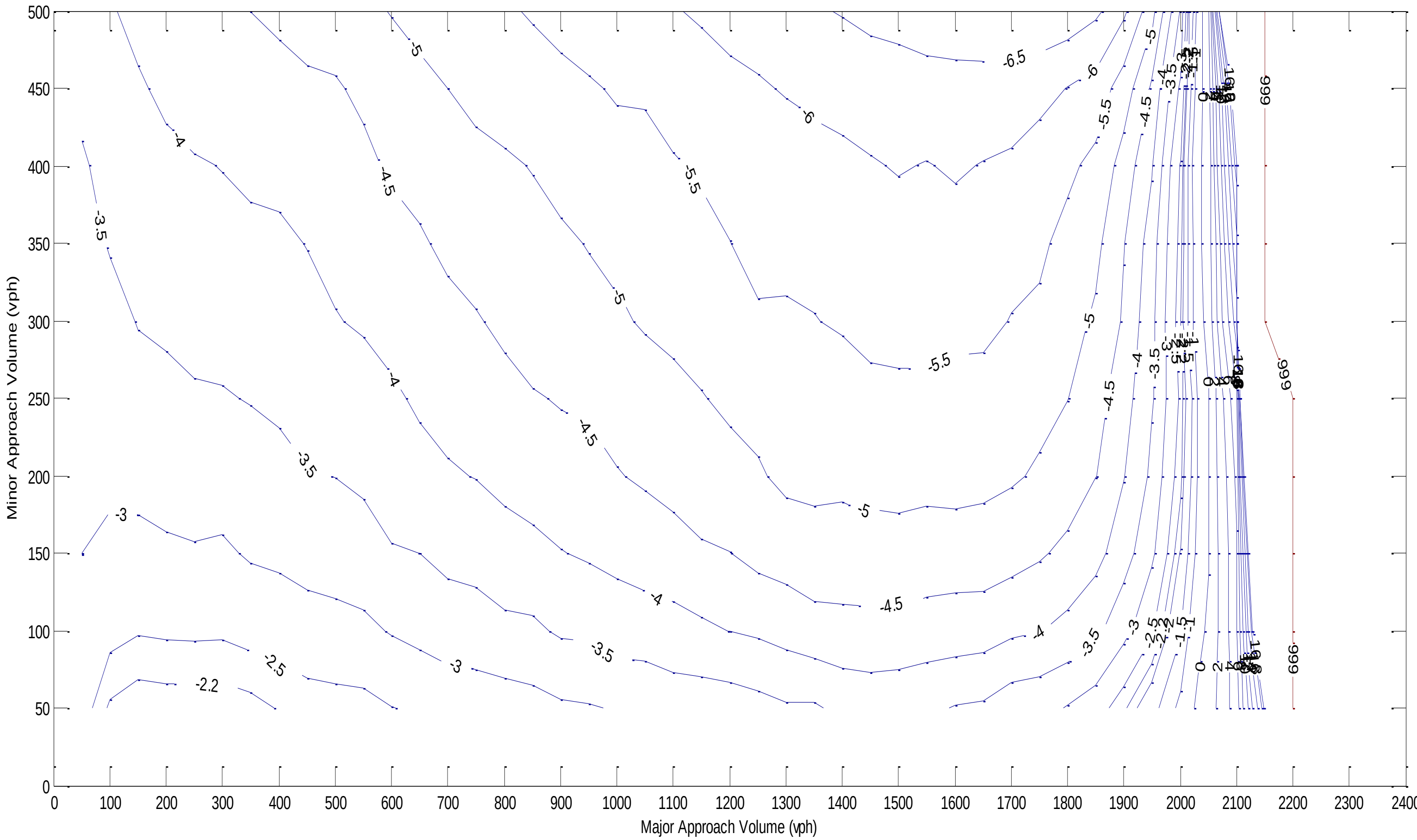


Figure 5.12 Contour plot for total intersection delay difference between MUT and standard signalized intersection at BF=0.7, LTP=15%, TP=10%

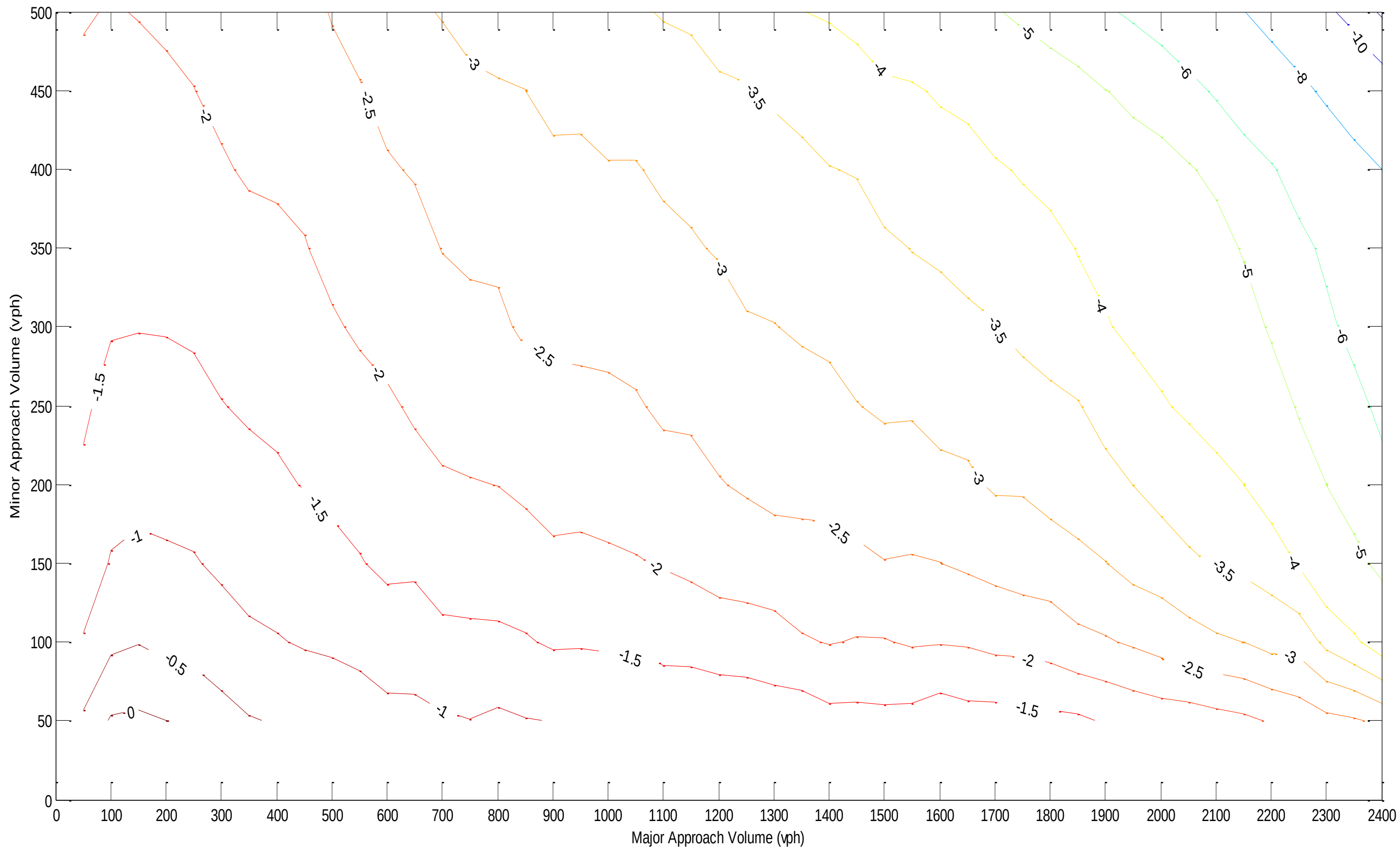


Figure 5.13 Contour plot for total intersection delay difference between CFI and standard signalized intersection at BF=0.7, LTP=15%, TP=10%

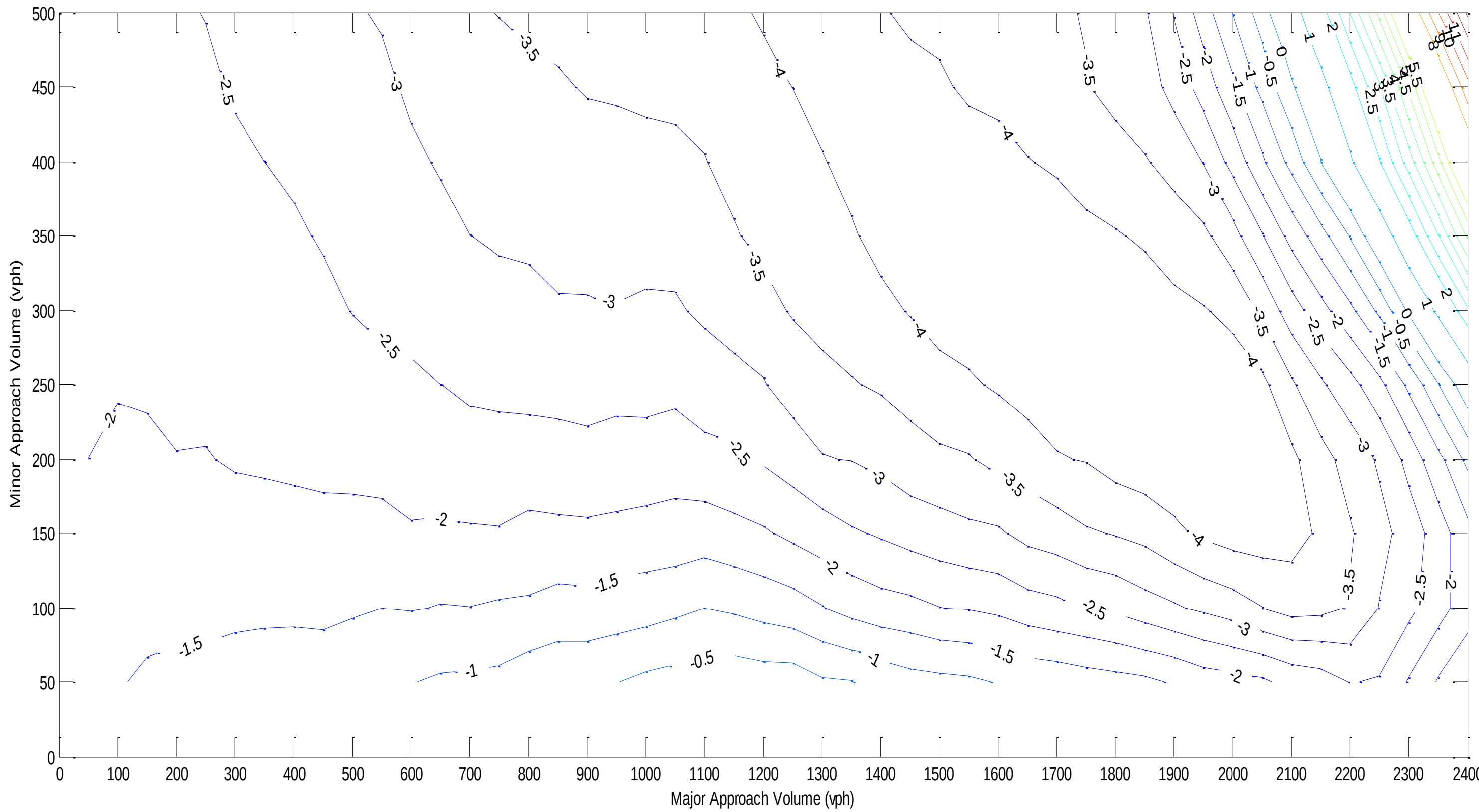


Figure 5.14 Contour plot for total intersection delay difference between jughandle and standard signalized intersection at BF=0.7, LTP=15%, TP=10%

The plots indicated that the delay saving capability of MUT decreased with an increasing major street approach volume. Under the presence of higher left turning volumes, the MUT system fails. The MUT intersection failure is depicted in the contour plot with a value of 999. The plots also indicated that the delay saving capability of CFI was good for a high major street approach volume under the presence of a high minor street approach volume. The delay-saving capability of jughandles was also good for a higher major street approach volume, but it decreased with a high minor street approach volume. For that specific condition, the delay-saving capability of jughandles increased with minor street approach volume for low to medium major street approach volumes. The plots are included in Appendix B of this report.

5.2.3 Delay Performance Information for Projected Volumes from Contour Plot

The contour plots in conjunction with the plot of projected volumes can be used to find delay savings information for projected volumes. This can be more clearly understood from by referring to an example of MUT with the contours of total delay difference, shown in figure 5.15. This is a case when major and minor street approach volumes are increasing at the rate of 2% annually. The growth rate curves for the growth rate of 1%, 2%, 3%, 4% and 5 % are also included in Appendix B. Let us consider a case with a major street approach volume of 1600 vph and a minor street approach volume of 250 vph. Point A, which lies between the contours of magnitude -5 and -6, represents the particular delay saving of about 5.4 sec/veh for the base year. Now, the delay savings for the projected volume, if both approach volumes increase at the same rate of 2% annually, can be calculated by extending line OA. The delay savings for the projected volume for the 10th year can be estimated by locating point B in line OA, as shown in figure 5.15. B represents a delay savings of approximately 3.5 sec/veh. Hence, the delay savings for the projected volumes of 1600 vph for the major street and 250 vph for the minor street for the 10th year is 3.5 sec/veh. By projecting line OA to the contour of magnitude 999, which represents the failure case, it can be observed that if both volumes of approach keep increasing by 2%, MUT will fail in 15.5 years. If the rate of increment of major and minor street approach volumes and are different, it is necessary to use different projection plots.

It is possible to find the delay of any volumes for any percentage of annual increments in minor street total approach traffic by projecting the line expressed by equation 5.4 originating from the base year point.

$$Slope\ of\ Projection\ Line = \frac{Y \times \left(1 + \frac{n}{100}\right) - Y}{X \times \left(1 + \frac{m}{100}\right) - X} = \frac{Yn}{Xm} \qquad 5.4$$

Where,

X = total major street approach volume for base year,

Y= total minor street total approach volume for base year,

m = the annual percentage increment of major street approach volume, and

n = annual percentage increment of minor street approach volume.

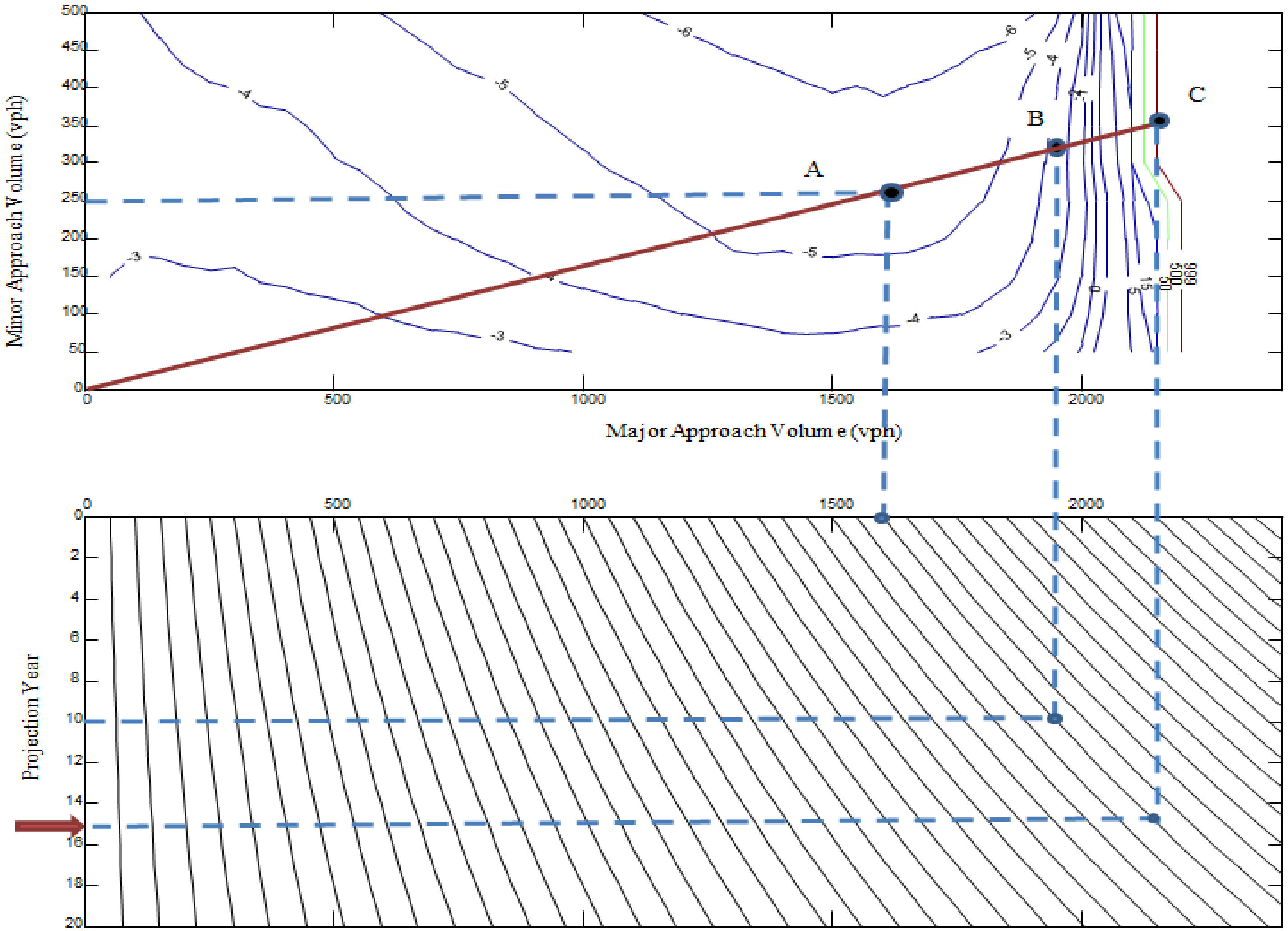


Figure 5.15 Delay savings for projected volumes (case study: MUT total intersection delay different at BF=0.7, LTP=15%, TP=10%)

Chapter 6 Economic Analysis

This chapter is comprised five major sections: (1) estimation of marginal agency cost, (2) monetization of marginal user's and non-user's benefit, (3) the life cycle cost analysis (LCCA), (4) the development of a spreadsheet tool to perform LCCA, and (5) a sample case study of LCCA using the spreadsheet tool. The outcome of this chapter will provide marginal net present value (NPV) and benefits to cost ratio (B/C) pertaining to the three unconventional intersections, which can serve as a basis for deciding the optimal alternative of unconventional intersections.

6.1 Marginal Agency Cost

The marginal agency cost includes both the marginal agency cost for new construction of unconventional intersections and retrofitting unconventional intersections over standard signalized intersections. The word "marginal" stands for the difference in quantity or cost due to the new construction of unconventional intersections or their retrofits with respect to standard signalized intersections. The marginal agency cost includes the costs of construction, preliminary engineering, and additional operation and maintenance.

The marginal construction quantities for new construction were estimated based on the additional pavement requirement, additional signals and installations with related accessories, and the additional right of way needed for new unconventional intersections as compared to standard signalized intersections. The construction quantities for retrofits were estimated based on the additional pavement requirement, the removal of existing pavements, additional signals with their installations and related accessories, etc. needed while retrofitting standard signalized intersections. NDOR's latest "English Average Unit Price (AUP) Summary July 2012-June 2013," found on their website under Item History and Info was referenced for unit price information. The items considered for the quantity estimates and the related unit prices are shown in table 6.1.

Table 6.1 Unit prices of item considered in estimation of construction quantities

Standard item no.	Item	Unit	Rate (US $)	Source of Information
1101	Concrete Pavement Removal	SY	6.81	Average Unit Price (AUP), Summary July 2010-June 2013, NDOR (accessed on 2014)
3075.56	10" Doweled Concrete Pavement, Class 47 B-3500 (Including Median Opening)	SY	30.59	
1010	Excavation (1.5')	CY	3.26	
1122.01	Remove Concrete Median Surfacing (6' Median)	SY	5.90	
A724.01	Relocate Traffic Signal	Each	470.00	
A006.98	Vehicle Detector, Type TD-A Performed	Each	263.69	
A001.7	Pull Box, Type PB-7	Each	1010.00	
A070.22	4 Inch Conduit in Trench	LF	5.91	
A072.20	4 Inch Conduit in Roadway	LF	7.83	
A077.26	16/C 14 AWG Traffic Signal Cable	LF	3.98	
A079.01	2/C #14 AWG Detector Lead-In Cable	LF	1.46	
7320.27	Traffic Sign and Post (STOP Sign, No RTOR and No Left Turn)	Each	295.85	
7500.22	Right Arrow Performed Pavement Markings	Each	420.00	
7500.25	Through and Left Arrow Performed Pavement Markings	Each	483.00	
3017.4	Concrete Class 47B-3000 Median Surfacing	SY	31.05	
A016.8	Mast Arm Signal Pole, Type MP-60	Each	8500.00	
A504.81	Install Mast Arm Signal Pole	Each	3700.00	
A 703.00	Relocate Mast Arm	Each	1470.00	
A003.10	Traffic Signal, Type TS-1	Each	493.89	
A501.00	Install Traffic Signal, Type TS-1	Each	242	
7496.05	5" Yellow Permanent Pavement Markings	LF	0.19	
7495.55	5" White Permanent Pavement Markings	LF	0.4	
N/A	Additional Land	Acre	4142.50	USDA (2012)

The unit price of land ($4142.5/acre) was calculated by referencing the "United States Land Values 2012 Summary" (2012) published by the United States Department of Agriculture. The unit price of real estate land ($2,590/acre), cropland ($4,480/acre), irrigable land ($6,000/acre), and non-irrigable land ($3,500/acre) were averaged.

The preliminary engineering cost (PE) involves expenses for activities from planning to the final design of a project (Turochy, Hoel and Doty, 2001). According to Turochy et al. (2001), most state DOTs consider the PE cost to range from 5% to 20% of construction costs depending on the project size and scope. Remaining in that range, this study considered the PE cost to be 10% of the construction cost. Contingency was assumed to be 20% of the construction cost with reference to Boddapati (2008). The unit price O&M for CFI was estimated based on the service requirement for additional signal heads, detectors, signal retiming costs, and power supply costs. Similarly for MUT and jughandle, the unit price of O&M was fixed based on the cost of landscaping the medians and areas enclosed by the reverse ramps. The agency costs were monetized by multiplying the quantities of each item by their respective unit prices. The computed marginal costs of all three unconventional intersections, considering new construction and retrofits, are shown in table 6.2. The marginal cost estimate tables for new construction and retrofits for unconventional intersections are included in Appendix D.

Table 6.2 Computed marginal cost of unconventional intersections

Intersection type	Construction Cost + Soft Cost including Contingency (US $)		Operation and Maintenance Cost (US $)	
	New construction	Retrofit	New construction	Retrofit
MUT	36762.81	680426.26	2000	2000
CFI	279226.41	439799.44	24000	24000
Jughandle	64551.15	64635.13	2000	2000

6.2 Marginal User's and Non-User's Cost and their Monetization

The user's cost pertains to the cost due to the increment in delay and fuel consumption when new unconventional intersections are constructed instead of signalized intersections, or a standard signalized intersection is retrofitted with an unconventional intersection. Similarly, the non-user's cost pertains to the cost due to increment in emissions when new unconventional intersections are constructed instead of signalized intersections or a standard signalized intersection is retrofitted with an unconventional intersection. These are calculated by subtracting the amount of each item produced by a standard signalized intersection from the amount of each item produced by the unconventional intersections. If the deducted value is negative, it is called the benefit or negative cost. Unit prices of each cost were calculated by our own rate analysis or by referencing past literature. The unit price of time (price of delay) was calculated as $12.55/hour considering the average per capita income of Nebraska in 2011, based on information provided by the U.S. Bureau of Census and the total working periods. The rate analysis is included in Appendix C. The unit prices of diesel and petrol were calculated averaging the 2012 average gas price for Nebraska, provided by AAA's Fuel Gauge Report (petrol: $3.704 /gallon, diesel: $ 3.956/gallons). The unit price of CO_2 ($0.02/kg) was referenced from the 2010 Annual Supplement to National Institute of Standards and Technology (NIST) Handbook published by the U.S. Department of Energy. The unit price of CO ($200/ton) was referenced from a technical paper by Bishop, Stedman, Peterson, Hosick and Guenther (1993). Similarly, unit prices for NO_x ($250/ton/year) and VOC_s ($180/ton/year) were referenced from Muller and Mendelson (2009) considering median damage cost. The unit prices are also listed in table 6.3. These marginal benefits were monetized by multiplying the quantities with respective unit prices.

Table 6.3 Unit prices of items

Items	Unit prices	Source of Information
Delay	$ 12.55/hour	Based on rate analysis. See Appendix D
Petrol	$ 3.704/Gallons	AAA's Fuel Gauge Report
Diesel	$ 3.956/Gallons	
CO_2	$ 0.02/Kg	U.S. Department of Energy: NIST (2010)
CO	$ 200/ton	Bishop et al. (1993)
NO_x	$ 250/ton/year	Muller and Mendelson (2009)
VOC_s	$180/ton/year	

6.3 Life Cycle Cost Analysis (LCCA)

The LCCA was performed on monetized agency, user's and non-user's marginal costs to determine the net present value (NPV) and benefit to cost ratio (B/C) of the new construction and retrofits of the unconventional intersections. The life cycle period of the retrofits was assumed to be 20 years and the discount rate was assumed to be 3% with no inflation for each year (Boddapati, 2008). The annual increase of traffic was considered 2% for major streets and 1% for minor streets. The delay for each projected volume for a 20-year period was estimated by batch running HCS 2010 Streets®. The respective fuel consumption and emissions and their annual costs were estimated. The operation and maintenance cost was assumed to be the same throughout the life cycle period. The NPV was estimated using the following equations:

$$NPV\ of\ Total = NPV\ of\ Benefits - NPV\ of\ O\&M\ Cost - Construction\ Cost + PE\ Cost \qquad 6.1$$

$$Benefit\ to\ Cost\ Ratio\ (B/C) = NPVof\ Benefits/\{NPV\ of\ O\ \&\ M\ Cost + (Construction\ Cost + PE\ Cost))\} \qquad 6.2$$

$$NPV\ of\ O\ \&\ M\ Cost = \left(\frac{1}{i}\right) \times \left\{1 - \frac{1}{(1+i)^N}\right\} \qquad 6.3$$

$$NPV\ of\ benefits = \sum_{N=o}^{20} P_N \times \frac{1}{(1+i)^N} \qquad 6.4$$

\Where,

N = life cycle period,

i = discount rate (3%), and

P_N = yearly negative or positive benefits

If any retrofits or new unconventional intersections failed due to high demand in any year throughout life cycle period, the NPV of those retrofits were calculated assuming a reduced life cycle period. The reduced life cycle period equals the time period up to which intersection operation is feasible. This case is applicable for MUT and jughandle because they were evaluated with M/M/1 queues, where the server's capacity should not be exceeded by demand. It is because the queuing system works until the utilization factor (ratio of demand and service capacity) remains less than 1.

6.4 Development of a Spreadsheet Tool to Perform Life Cycle Cost Analysis

This study developed a Microsoft Excel-based tool interfaced with HCS 2012® that can estimate a measure of effectiveness of operational performance, such as delay, fuel consumption, and emissions, and also can perform the life cycle cost analysis. The spreadsheet tool has four parts of operation: (1) the estimation of delay, fuel, and emissions in conjunction with HCS 2012®, as well as their projection for each year in the life cycle period of the intersections; (2) the marginal cost estimate of user's and non-user's benefits with respect to standard signalized intersections and monetization; (3) the estimation and monetization of construction costs, soft costs, and operation and maintenance costs; and (4) a life cycle cost analysis to estimate the NPV of benefits and the corresponding B/C of MUT, CFI, and jughandle. This spreadsheet can function as a decision assistance tool to decide the best unconventional intersection design under a user's defined criteria. The tool is named "SILCC" (Signalized Intersection Life Cycle Cost Analysis), which is a separate deliverable to NDOR as a part of this study.

6.5 A Case Study of LCCA using SILCC

For a case study, the volume pattern for rural roads was developed, following one of the 24-hour data patterns provided by Williams and Ardekani (1996). This pattern is shown in figure 6.1. The delay, fuel consumption, and emissions were estimated corresponding to the 24 hour volume data, using SILCC for the 20-year life cycle period and the specifications of 10% truck and 5% left turning traffic and the default annual increment in traffic (2% on the major street and 1% for the minor street). Additionally, the default lane configuration of a four-lane major street at 45 mph and a two-lane minor street with at 35 mph were considered. The construction estimate for retrofits and new construction for unconventional intersections that were used were the same as those discussed in section 6.1. The corresponding default rates of items used were as mentioned in previous sections. SILCC provided the results from LCCA as displayed in table 6.4. The result indicated that MUT would have the highest NPV of the various benefits for both cases, but due to its high construction cost of a retrofit, the B/C is lower than that of a jughandle. The ranking of alternatives for retrofits according to their B/C was found to follow the hierarchical order of (1) jughandle, (2) MUT, and (3) CFI. However for new construction, the ranking of alternatives according to B/C was found to follow the hierarchical order of (i) MUT, (ii) jughandle, (iii) CFI. All the intersections were found to have a B/C greater than 1, indicating that they were more beneficial than standard signalized intersections for this particular condition.

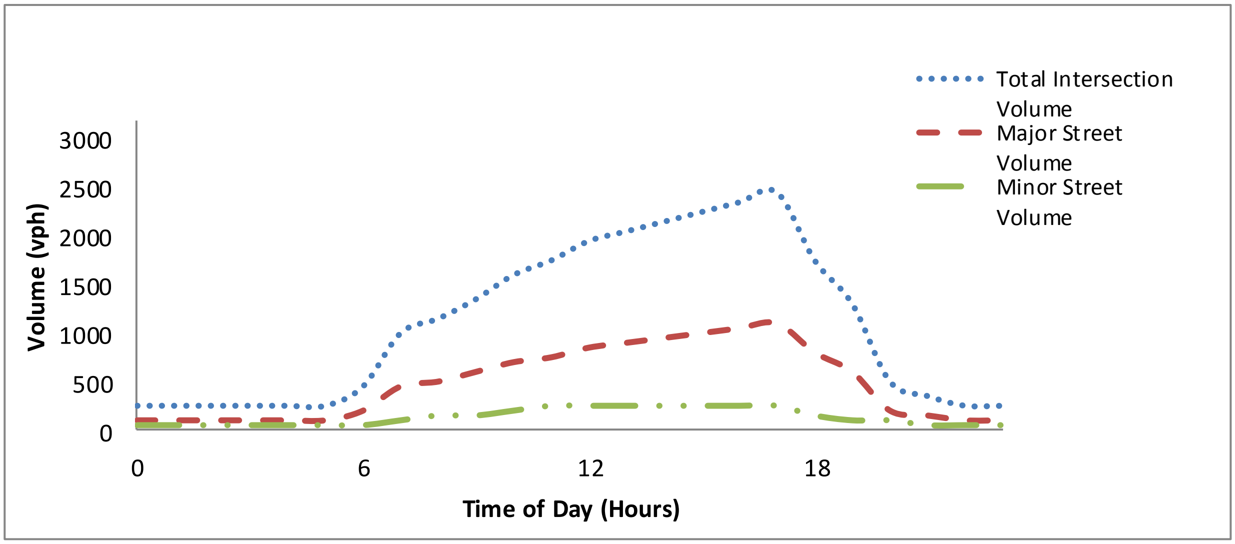


Figure 6.1 Volume pattern for a rural road

Table 6.4 LCCA result for case study

Cases	LCCA Outcomes	MUT	CFI	Jughandle
New Construction	Net Present Value (NPV) of Benefit (US $)	3720295.68	180950.96	1952272.57
	Benefit to Cost ratio (B/C)	56.93	1.28	21.7
Retrofits	Net Present Value (NPV) of Benefit (US $)	3076632.22	20377.94	1952188.59
	Benefit to Cost ratio (B/C)	5.33	1.03	21.68

Chapter 7 Conclusion and Recommendation

This study evaluated three types of unconventional intersections: MUT, CFI, and jughandle, comparing their performance with standard signalized intersections. The study at first assessed the intersections based on delay performance, and finally, performed the life cycle cost analysis over the operational benefits, operational cost, cost of construction, and cost of operation and maintenance. Based on delay performance, this study was able to define the DAC that distinguished the region of optimal performance for each type of unconventional intersection. Both graphical plots and thresholds equations were developed and are included in this report. The overall analysis of DAC indicated that MUT is applicable for almost all levels of volume combinations of major and minor street approach volume under the presence of low left-turning traffic. Under medium to high left-turning traffic, the jughandle performed optimally on high major street approach volume, and its performance got better with increasing minor street approach volume only with balanced flow conditions. Under medium to high left-turning traffic, CFI performed optimally for high major and minor street approach volumes, and the jughandle performed optimally for high major street approach volume and low minor street approach volume only at unbalanced flow conditions. CFI's optimal region expanded as the flow became more unbalanced.

Furthermore, to provide flexibility for users to decide on the best unconventional intersection design under multiple sets of conditions, a spreadsheet tool, "SILCC," was developed. SILCC has the ability to estimate the operational performance measure, complete a cost estimate, and perform LCCA. For new construction, a sample case study performed on a 24-hour rural pattern volume indicated a higher NPV for operational benefits and a higher B/C related to MUT compared to all other intersections. However, since the construction cost of a

MUT-retrofit is high, the jughandle-retrofit was found to have the highest B/C, despite the fact that MUT had the highest NPV.

The other specific conclusions reached by the study are mentioned below:

1. The DAC developed based on delay performance favored the use of MUT for all combinations of total approach volumes ranging 50 vph to 2400 vph on the major street and 50 vph to 500 vph on the minor street under a low percentage of left-turning traffic (5%).
2. Under medium left-turning traffic (10%), the jughandle performed optimally on high volume condition, or above 2300 vph for the major street total approach volume. The area of the optimal jughandle region widened flared to the left as the minor street approach volume increased. Under same condition with the balance factor increased to 0.7, which represents the condition of high volume on one direction of the major street, the CFI was introduced in the region of high major and minor street approach volumes, just above the jughandle. Still, MUT occupied the most area of the plot. These conditions can be observed from the plots provided in Appendix B. The threshold equations of are provided in table 5.2.
3. Under a high left-turning volume (15%) and balance condition (BF=0.5), the threshold between MUT and jughandle started approximately above 1800 vph and flared toward the left as the minor street approach volume increased. CFI was introduced on the top as the balance factor increased to 0.6, and extended to the bottom of the axis as the balance factor increased to 0.7. This can be explained as the increment of left-turning traffic that occurred although the left-turning percentage is the same, due to an unbalanced high flow on one approach, which made one left turn movement critical and favors the use of CFI.
4. The effect of truck percentage was not that perceptible. However, it favors CFI under a high balance factor and high left-turning percentage conditions.

5. Looking at the DAC plots in Appendix B and the DAC equations in table 5.2, table 5.3, table 5.4 and table 5.5 are recommended to have an accurate understanding of the thresholds.
6. The above criteria are based on delay performance and limited to rural intersections with a four-lane major street at 45 mph and a two-lane minor street at 35 mph. To account for the cost in decision making, the SILCC is recommended. Additionally, SILCC can provide a decision based on cost and operational benefits for any criteria that the user wants to asses; it can be used as decision assistance tool.

References

AAA's Daily Fuel Gauge Report. http://fuelgaugereport.aaa.com/?redirectto=http://fuelgaugereport.opisnet.com/index.asp. Accessed on August, 2013.

A Policy on Geometric Design of Highways and Streets. American Association of State Highway and Transportation Officials (AASHTO), 2011.

Agarwal, N.K. *Estimation of Pedestrian Safety at Intersections using Simulation and Surrogate safety Measures.* College of Engineering, University of Kentucky, 2011.

Aldian, A. and M.A.P. Taylor. Selecting Priority Junction Traffic Models to Determine U-turn Capacity at Median Opening. Proceedings of Eastern Asia Society for Transportation Studies, Vol. 3, No. 2, 2001, pp. 101-113.

Al-Masaeid, H. Capacity of U-Turn at Median Openings. *Institute of Transportation Engineers Journal*, Vol. 69, 1999, pp. 28-32.

Al-Omari, B.H and K.M.A. Al-Akharas. *Institute of Engineers Journal*, Vol. 84, No. 3, 2014, pp. 42-47.

Alternative Intersection. John Sangster. https://sites.google.com/site/trfcguy/Research/alternative-intersections. Accessed on April, 2014.

Armstrong, M. *Integrating Alternative Intersections and Interchanges into HCS 2010*. University of Florida, 2014.

Asokan, A., J. Bared, R. Jagnnathan,W.Hughes, F.Cicu and P.F. Illotta . Alternative Intersections Selection Tool-AIST. Transportation Research Board Annual Meeting 2010, Transportation Research Board of the National Academies,Washington D.C, 2010.

Autey, J, T. Sayed and M. EL Esawey. Guidelines for the Use of Some Unconventional Intersection Designs. Transportation Research Board Annual Meeting 2010, Transportation Research Board of National Academies, Washington DC, 2010.

Autey, J, T. Sayed and M. EL Esawey. Operational Performance Comparison of Four Unconventional Intersection Designs using Micros-Simulation. *Journal of Advanced Transportation*, Vol. 47, 2013, pp. 536-552.

Banks, J., J.S. Carson II, B.L. Nelson and D.M. Nicol. *Discrete-Event System Simulation*. Prentice Hall, 2009.

Bared, J. G and E.I. Kaisar. Median U-Turn Design as an Alternative Treatment for Left Turns at Signalized Intersections. *Institute of Transportation Engineers Journal*, Vol. 72-2, 2002, pp. 50-54.

Ben-Edigbe, J., R. Rahman and N.F. Jailani. Effect of U-Turning Maneuvers at Midblock Facilities on Traffic Kinematics Waves. *Journal of Applied Sciences*, Vol. 13, No.4, 2013, pp. 602-608.

Berkowitz, C.M., F. Mier, C.E. Walter, and C. Bragd. Continuous Flow Intersections: An Intelligent Transportation Solution. CD-ROM. Institute of Transportation Engineers, 1997.

Bishop, G.A, D.H. Stedman, J.E. Peterson, T.J. Hosick and P.L. Guenther. A Cost Effectiveness Study of Carbon Monoxide Emissions Reduction utilizing Remote Sensing. *Journal of Air Waste Management Association*, Vol.43, 1993, pp. 978-988.

Board of Public Roads Classifications and Standards. *Nebraska Minimum Design Standards: Counties, Municipalities, State*. Nebraska Department of Roads (NDOR), 2008.

Boddapati, P. *Comparative Study of Type 2 Median Crossover and Median U-Turns*. University of Missouri-Columbia, 2008.

Boone, J.L., and J.E. Hummer. Calibrating and Validating Traffic Simulation Models for Unconventional Arterial Intersection Designs. In *Transportation Research Record: Journal of Transportation Research Board*, No. 1500, Transportation Research Board of the National Academies, Washington, D.C., 1995, pp. 184-192.

Carrol. H. D. and D. Lahusen. Operational Effects of Continuous Flow Intersection Geometrics: A Deterministic Model. In *Transportation Research Record: Journal of Transportation Research*

Board, No. 2348, Transportation Research Board of the National Academies, Washington, D.C., 2013, pp. 1-11.

Carter, D., J.E. Hummer, R.S. Foyle and S. Philips. Operational and Safety Effects of U-turns at Signalized Intersections. In *Transportation Research Record: Journal of Transportation*, No. 1912, Transportation Research Board of the National Academies, Washington D.C., 2005, pp.11-18

Chang, G.L., Y. Lu, and Y. Xiangfeng. *An Integrated Computer System for Analysis, Selection, and Evaluation of Unconventional Intersections.* Report No. SP909B4H, 2011.

Chen, X., Y. Qi and Y. Lu. Safety Impacts of Using Short Left-turn Lanes at Unsignalized Median Openings. Transportation Research Board Annual Meeting 2014, Transportation Research Board of National Academies, Washington DC, 2014.

Cheong, S., S. Rahwanji, and G.L. Chang. Comparison of Three Unconventional Arterial Intersection Designs: Continuous Flow Intersection, Parallel Flow Intersection, and Upstream Signalized Crossover. *IEEE*, 2008.

Chowdhury, M.S. An Evaluation of New Jersey jughandle Intersection (NJJI) with and without Pre-Signals. *Transportation and Development Institute Congress*, American Society of Civil Engineers, 2011, pp. 1245-1254.

Cobian, R., T. Henderson, M. Sudeshna, C. Nuworsoo and E. Sullivan.Vehicle Emission and Level of Service Standards: Exploratory Analysis of the effects of Traffic Flow on Vehicle Greenhouse Gas Emissions. In the TRB 88th Annual Meeting Compendium of Papers DVD, Transportation Research Board of the National Academies, Washington, D.C., 2009.

Combinido, J.S.L and M.T. Lim. Modelling U-turn Traffic Flow. *Physica A*, 389, 2010, pp. 3640-3647.

Community Planning Association of Southwest Idhao (COMPASS). *Innovative Intersections: Overview and Implementation Guidelines.* High Volume Intersection Study (HVIS), Vol. 1, 2008.

Crashes vs. Congestion-What's the Cost to Society? AAA, 2008.

Dorothy, P.W., T.L. Maleck and S.E. Nolf. Operational Aspects of Michigan Design for Divided Highways. In Transportation Research Record: *Journal of Transportation Research Board*, No. 1579, Transportation Research Board of the National Academies, Washington, D.C., 1997, pp. 18-26.

El Esawey, M., and T., Sayed. Comparison of Two Unconventional Intersection Schemes. Crossover Displaced Left-Turn and Upstream Signalized Crossover Intersections. In *Transportation Research Record: Journal of Transportation Research Board*, No. 2023, Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 10-19.

El Esawey, M., and T., Sayed. Operational Performance of the Unconventional Media U-Turn Design Using Micro-Simulation. Transportation Research Board Annual Meeting 2014, Transportation Research Board of the National Academies, Washington D.C, 2011(A).

El Esawey, M., and T., Sayed. Operational Performance of the Unconventional Media U-Turn Design Intersection Design. *Canadian Journal of Civil Engineering*, Vol. 38, 2011(B), pp.1249-1261.

Energy Price Indices and Discount Factors for Life-Cycle Cost Analysis. NISTIR 85-3273-27, National Institute of Standards and Technology, U.S. Department of Commerce Technology Administration, Washington, D.C., 2012.

Fan, H. Q, B. Jia, X.G. Li, J.F Tian and X.D Yan. Characteristics of Traffic Flow at Nonsignalized T-Shaped Intersection with U-Turn Movements. *The Scientific World Journal*, Vol. 2013, 2013, pp. 1-7.

Furtado, G., G. Tencha and H. Devos. Unconventional Arterial Design: jughandle Intersection Concept for McKnight Boulevard in Calgary. Annual Conference and Exhibition, The Transportation Association of Canada, City of St. John’s Canada, 2003.

Goldbatt, R., F. Mier, and J. Friedman. Continuous Flow Intersections. *Institute of Transportation Engineers Journal*, Vol. 64, 1994, pp. 35-42.

Grant, M., B. Bowen, M. Day, R. Winick, J. Bauer, A. Chavis and S. Trainor. *Congestion Management Process: A Guidebook*. Report No. FHWA-HEP-11-011, US DOT, FHWA, 2011.

Guo, T., P. Liu, Y. Liu and W. Deng. Operational Impacts of Indirect Driveway Left-Turn Treatment at Signalized Intersections. International Conference for Chinese Transportation Professional (ICCTP), ASCE, 2011.

HCM 2000, *Highway Capacity Manual.* Transportation Research Board, Washington DC.

HCM 2010, *Highway Capacity Manual.* Transportation Research Board, Washington DC.

Henderson, S.M. and N. Stamatiadis. Use of Median U-turns to improve Traffic Flow along Urban Arterials. *Journal of the Transportation Research Forum*, Vol.40, No. 2, 2001, pp.137-145.

Hilderbrand, T.E. *Unconventional Intersection Designs for Improving Through Traffic Along with the Arterial Road.* Florida State University FAMU-FSU College of Engineering, 2007.

Hillier, F.S., G.J. Lieberman, B. Nag and P. Basu. *Introduction to Operation Research*. McGraw Hill Education (India) Pvt. Ltd., New Delhi, 2010.

Hughes, W., R. Jagannathan, D. Sengupta and J. Hummer. *Alternative Intersections/Interchanges: Informational Report (AIIR).* Report No. FHWA-HRT-090-060, U.S. Department of Transportation, 2010.

Hummer, E.J. and J.D. Reid. Unconventional Left turn Alternatives for Urban and Suburban Arterials: An Update. Transportation Research Board Circular E-C019: Urban Street Symposium, 1999.

Hummer, J. E. Unconventional Left –Turn Alternatives for Urban and Suburban Arterials-Part One. *Institute of Transportation Engineers Journal*, Vol. 11, 1998 (A), pp. 26-29.

Hummer, J. E. Unconventional Left –Turn Alternatives for Urban and Suburban Arterials-Part Two. *Institute of Transportation Engineers Journal*, Vol. 11, 1998 (B), pp. 101-106.

Hummer, J.E., and J.L. Boone. Travel Efficiency of Unconventional Suburban Arterial Intersection Designs. In *Transportation Research Record: Journal of Transportation Research Board*, No.

1500, Transportation Research Board of the National Academies, Washington, D.C., 1995, pp. 153-161.

Information and Geometric Design Guidance Regarding Boulevards, Directional Crossovers, and Indirect (‘Michigan”) Left-Turns. < http://www.bqaz.org/pdf/parkway/Info%20and%20Geometric%20Design%20Guidance.pdf)> . Accessed on April, 2014.

Inman, W. V. Evaluation of Signs and Markings for Partial Continuous Flow Intersections. In *Transportation Research Record: Journal of Transportation Research Board*, No. 2138, Transportation Research Board of the National Academies, Washington, D.C., 2009, pp. 66-74.

Instruction for form EIA-1605: Voluntary Reporting of Greenhouse Gases. Energy 473 Information Administrations (EIA), U.S. Department of Energy, Washington, D.C., 2007.

Intersection Improvement Study-Phase 2 Report. Town of Cary, North Carolina. <http://www.townofcary.org/Departments/fdts/streetsandsidewalks/streetprojects/intersectionstudy/trafficstudy.htm#_Toc222721891> Accessed on June, 2014.

Jagannathan, R. Synthesis of the Median U-Turn Intersection Treatment, Safety, and Operational Benefits. 3rd Urban Street Symposium, Washington, 2007.

Jagannathan, R., and J.G. Bared. Design and Operational Performance of Crossover Displaced. In *Transportation Research Record: Journal of Transportation Research Board*, No. 1881, Transportation Research Board of the National Academies, Washington, D.C., 2004, pp. 1-10.

Jagannathan, R., and J.G. Bared. Design and Performance Analysis of Pedestrian Crossing Facilities for Continuous Flow Intersections. In *Transportation Research Record: Journal of Transportation Research Board*, No. 1939, Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 133-144.

Jagannathan, R., M. Gimbel, J.G. Bared, W.E. Hughes, B. Persuad and C. Lyon. Safety Comparison of New Jersey jughandle Intersections and Conventional Intersections. In *Transportation Research*

Record: Journal of Transportation Research Board, No. 1953, Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 187-200.

Jenjiwattanakul, T., K. Sano and H. Nishiuchi. Capacity of U-Turn Junction at Midblock Opening on Urban Arterial Based on Balancing Volume-to-Capacity Ratio. Proceedings of the Eastern Asia Society of Transportation Studies, Vol. 9, 2013.

Kach, B. *The Comparative Accident Experience of Directional and Bi-directional Signalized Intersections.* Michigan Department of Transportation, 2004.

Kai, C., Z. Ning and Q. Chen-dong. Median Design Analysis in the Manner of U-turn Followed by Right-turn. *International Conference on Transportation Engineering*, 2007 pp.352-357.

Kaisar, I. E., P. Edara, J.D. Rodriguez-Seda and S. Chery. A Comparison of Non-Traditional Intersection Designs using Microscopic Simulation. Transportation Research Board Annual Meeting 2011, Transportation Research Board of the National Academies, Washington D.C, 2011.

Kentucky Transportation Centre, Intersection Design Analysis Tool (IDAT). http://catslab.ukytc.com/idat Accessed on April 23, 2014.

Kirk, A., C. Jones, N. Stamatiadis. Improving Intersection Design Practices. In Transportation Research Record: *Journal of Transportation Research Board*, No. 2223, Transportation Research Board of the National Academies, Washington, D.C., 2011, pp. 1-8.

Kivlins, R., and J.R. Naudzuns. Analysis of Unconventional Signalized At-Grade Intersections. *Scientific Journal of Riga Technical University*, Vol. 12, 2011, pp. 17-26.

Kyte, M., C. Clemow, N. Mahfood, B.K. Lall and C. J. Khisty. Capacity and Delay Characteristics of Two-Way Stop-Controlled Intersections. In *Transportation Research Record: Journal of Transportation Research Board*, No. 1330, Transportation Research Board of the National Academies, Washington, D.C., 1991, pp. 160-167.

Land Values 2012 Summary. United States Department of Agriculture, National Agriculture Statistics Service, 2012.

Liu, P. and J.J. Lu. Selection of the Optimal Location of a U-turn Bay for Vehicles making Right- turns Followed by U-turns. Transportation Research Board Annual Meeting 2007, Washington DC, 2007

Liu, P., J. John, F. Hu and G. Sokolow. Capacity of U-turn Movement at Median Openings on Multilane Highways. *Journal of Transportation*, Vol. 134, No. 4, 2008, pp.147-154.

Liu, P., J.J. Lu, F. Pirinccioglu and G. Sokolow. Operational Effects of U-turns as Alternatives to Direct Left–turns from Driveways or Side Streets –Some Research Findings in Florida. Transportation Research Board 85th Annual meeting, 2006.

Liu, P., X. Qu, H. Yu, W. Wang and B. Cao. Development of a VISSIM Simulation Model for U-turns at Unsignalized Intersection. *Journal of Transportation Engineering*, ASCE, Vol. 138, 2012, pp. 1333-1339.

Liu, Pan. *Evaluation of the Operational Effects of U –turn Movement*. University of South Florida, 2006.

Liu, Q., L. Zhang and W. Yang. A New Continuous Flow Intersection for Urban Road: Architecture, Design and Simulation. *Applied Mechanics and Materials*, Vol. 209-211, 2012, pp. 677-682

Lu, J., S. Dissanayake, H. Zhou, X.K. Yang and K. Williams. *Operational Evaluation of Right Turns Followed by U-turns as an Alternative to Direct Left Turns*. University of South Florida, 2001.

Lu, J.J., F. Pirinccioglu and J.C. Pernia. *Safety Evaluation of Right Turns Followed by U-turns at Signalized Intersection (6 or More Lanes) as An Alternative to Direct Left Turns Conflict Analysis.* University of South Florida, 2005.

Mallah, M. Guidelines for Median Treatment at Urban Roadways to Solve Left Turn Movement. 1st International Conference in Access Management, 2011.

Martin, A., R. Islam, M. Best and K. Sharma. Doing More with Less- Providing Innovative Mobility Solutions to TXDOT. ITE Technical Conference and Exhibit, 2012, CA, USA.

Mauga, T. and Kaseko, M. Modelling and Evaluating Safety Impacts of Access Management Features in the Las Vegas, Nevada, Valley.In *Transportation Research Record, Journal of Transportation*

Research Board, No. 2171, Transportation Research Board of National Academies, Washington DC, 2010, pp. 57-65.

Muller, N. Z., Mendelsohn, R. Efficient pollution regulation: getting the prices right. *American Economics Review*, Vol. 99, No.5, 2009, pp.1714-1739.

Nebraska Department of Road. Item History and Info. English AUP Summary July 2012-June 2013. http://www.transportation.nebraska.gov/letting/bid-item-history-info.htm. Accessed on March, 2014.

Obaidat, T.I.A and M.S, Elayan. Gap Acceptance Behavior at U-turn Median Openings-Case Study in Jordan. *Jordan Journal of Civil Engineering*, Vol. 7, No. 3, 2013, pp. 332-341.

Park, S., and H. Rakha. Continuous Flow Intersections: A Safety and Environment Perspective. IEEE, Annual Conference on Intelligent Transportation Systems, 2010, pp. 85-90.

Pirdavani, A., T. Brijs, T. Bellemans, and G. Wets. Travel Time Evaluation of a U-Turn Facility and ITS Comparison with a Conventional Signalized Intersection, 2010.

Pirinccioglu, F., J.J. Lu, P. Liu and G. Sokolow. Right Turn from Driveways followed by U-turn on Four –Lane Arterials: Is it Safer than Direct Left Turn? In *Transportation Research Record*, *Journal of Transportation*, No. 1953. Transportation Research Board of the National Academies, Washington DC, 2006, pp.172-179.

Pitaksringkarn, J.P. Measure of Effectiveness for Continuous Flow Intersection: A Maryland Intersection Case Study, 2005.

Potts, I.B, D.W. Harwood, D.J. Torbic, K.R. Richard, J.S. Gluck, H.S. Levinson, P.M. Garvey and R.S. Ghebrial. *Safety of U-Turns at Unsignalized Median Openings*. NCHRP Report 524. Transportation Research Board, 2004.

Qi Y., X. Chen, G. Liu and Y. Wang. Operational and Safety Impacts of Directional Median Opening on Urban Roadways: a Case Study in Houston, TX. Transportation Research Board Annual Meeting 2014, Transportation Research Board of the National Academies, Washington D.C, 2014.

R., Rahman, J. Ben-Edigbe and A. Hassan. Extent of Traffic Shockwave Velocity Propagations Induced by U-turn Facility on Roadway Segments. 25 th ARRB Conference-Shap ing the Future: Linking Policy, Research and Outcomes, Perth, Australia, 2012.

Reid, J.D., and J.E. Hummer. Analyzing System Travel Time in Arterial Corridors with Unconventional Designs Using Microscopic Simulation. In *Transportation Research Record: Journal of Transportation Research Board*, No. 1678, Transportation Research Board of the National Academies, Washington, D.C., 1999, pp. 208-215.

Reid, J.D., and J.E. Hummer. Travel Time Comparisons between Seven Unconventional Arterial Intersection Designs. In *Transportation Research Record: Journal of Transportation Research Board*, No. 1751, Transportation Research Board of the National Academies, Washington, D.C., 2001, pp.56-66.

Rodegerdts, L.A., J. Ringert, P. Koonce, J. Bansen, T. Nguyen, J. McGill, D. Stewart, J. Suggett, T. Neuman, N. Antonucci, K. Hardy, and K. Courage. *Signalized Intersection: Information Guide.* Publication FHWA-HRT-04-091, U.S. Department of Transportation, 2004.

Sangster, J. and H. Rakha (2014). Implications of CAP-X: Operational Limitations of Alternative Intersections. Transportation Research Board Annual Meeting 2014, Transportation Research Board of the National Academies, Washington D.C, 2010.

Savage, W.F. Directional Median Crossovers. *Traffic Engineering*, Institute of Traffic Engineers, Vol. 44, No. 11, 1974, pp.21-23.

Schrank, D., B. Eisele and T. Lomax. *2012 Urban Mobility Report*. Texas A & M Transportation Institute, Texas A & M University System, 2012.

Shahia, J. and A.A. Choupanib. Modeling the Operational Effects of Unconventional U-Turns at a Highway Intersection. *Transportmetrica*, Vol. 5-3, 2009, pp. 173-191

Shihan, J.H. and H.K. Mohammed. Traffic System Studies at Median U-Turn in Baghdad City Employing U-SIM Model. *Journal of Engineering and Development*, Vol.13, No. 1, 2009, pp. 226-237

Smith, D.P. Development of New jughandle Design for Facilitating High Volume Left Turns and U-Turns. Grove City College, 2011.

Stamatadias, N., A. Kirk and N. Agarwal. Intersection Design Tool to Aid Alternative Evaluation. In *Procedia-Social and Behavioral Sciences*, Elsevier, Vol. 53, 2012, pp. 601-610.

Stamatiadis, N., A. Kirk, N. Agrawal and C. Jones. *Improving Intersection Design Practices*. Report Number KTC-10-09/SPR-380-09-1F, Kentucky Transportation Centre, Frankfort, Kentucky, 2011.

Strickland, K. Introducing the Median U-turn Intersection in Georgia. ITE Annual Meeting, 2012.

Summary Report: Evaluation of Sign and Marking Alternatives for Displaced Left –Turn Lane Intersections. Publication FHWA-HRT-08-071. U.S. Department of Transportation, 2008.

Tabernero, V. and T. Sayed. Upstream Signalized Crossover Intersection: An Unconventional Intersection Scheme. *Journal of Transportation Engineering*, ASCE, Vol. 132, No. 11, 2006, pp. 907-911.

Tarko A., M. Inerowicz, B. Lang and N. Villwock. *Safety and Operational Impacts of Alternative (Two – Volume Report)*. Report No. FHWA/IN/JTRP-2008/23. Indiana Department of Transportation, 2008.

Tarko A., M. S. Azam and M. Inerowicz. Operational Performance of Alternative Types of Intersections –A Systematic Comparison for Indiana Conditions. 4th International Symposium on Highway Geometric Design Valencia, Spain, 2010.

Techbrief: Displaced Left-Turn Intersections. Publication FHWA-HRT-09-055. U.S. Department of Transportation, 2009.

Techbrief: Synthesis of the Median U-Turn Intersection Treatment. Publication FHWA-HRT-07-033. U.S. Department of Transportation, 2007.

Techbrief: Traffic Performance of Three Typical Designs of New Jersey jughandle Intersections. Publication FHWA-HRT-09-055. U.S. Department of Transportation, 2007.

Thompson, C.D., and J.E. Hummer. *Guidance on the Safe Implementation of Unconventional Arterial Designs.* Southeastern Transportation Centre, University of Tennessee at Knoxville, 2001.

Toolbox on Intersection Safety and Design. ITE, 2004.

Topp, A. and J.E. Hummer. Comparison of Median U-turn Design Alternatives using Microscopic Simulation. Third International Symposium on Highway Geometric Design, 2005.

Turochy, R.E., L.A. Hoel and R.S. Doty. *Highway Project Cost Estimating Methods used in the Planning Stage of Project Development.* Virginia Transportation Research Council (VTRC), 2001.

US Department of Commerce United States Census Bureau. https://www.census.gov/. Accessed on August, 2013.

User and Non-User Benefit Analysis for Highways. American Association of State Highway and Transportation Officials, 2010.

Williams, C.J. and S.A. Ardekani. *Impacts of Traffic Signal Installation at Marginally Warranted Intersections.* Report No. 1350-1F, Texas Department of Transportation, 1996.

Xu, L. Daniel B. Fambro Student Paper Award: Right Turns Followed by U-turns vs. Direct Left Turns: A Comparison of Safety Issues. *Institute of Engineers Journal*, Vol. 71, No. 11, 2001, pp.36-43.

Yahl, M.E. *Safety Effects of Continuous Flow Intersections*. North Carolina State University, 2013.

Yang, K.X. Capacity Estimation of U-turn Movements at Median Openings using Simulation. ITE Annual Meeting, 2002.

Yang, K. X., G.L. Chang, S. Rahwanji and Y. Lu. Development of Planning –Stage Models for Analyzing Continuous Flow Intersections. *Journal of Transportation Engineering*, ASCE, Vol. 139, 2013, pp. 1124-1132.

Yang, K.X and H.G. Zhou. CORSIM-Based Simulation Approach to Evaluation of Direct Left Turn versus Right Turn Plus U-Turn from Driveways. *Journal of Transportation Engineering*, ASCE, Vol. 130, 2004, pp. 68-75.

Zhao, J., W. Ma, K.L. Head and X.Yang. Optimal Intersection Operation with Median U-Turn: A Lane Based Approach. Transportation Research Board Annual Meeting 2014, Transportation Research Board of the National Academies, Washington D.C, 2014.

Zhou H., J.J. Lu, X.K. Yang, S. Dissanayakee, and K.M. Williams. Operational Effects of U-Turns as Alternatives to Direct Left Turns from Driveways. In *Transportation Research Record*: *Journal of Transportation Research Board*, No. 1796, Transportation Research Board of the National Academies, Washington, D.C., 2002, pp. 72-79.

Zhou H., P. Hsu, J.J. Lu and J.E. Wright. Optimal Location of U-Turn Median Openings on Roadways. In *Transportation Research Record*: *Journal of Transportation Research Board*, No. 1847, Transportation Research Board of the National Academies, Washington, D.C., 2003, pp. 36-41.

Appendix A Stepwise HCS Procedure To Analyze Unconventional Intersections

Studied Example Cases :

I. Bidirectional volume for major street = 1000 vph
II. Directional split (BF) = 0.5
III. Turn percentage (left turn) = 0 .05
IV. Truck percenatge = 2%
V. One directional volume for minor street = 100 vph
VI. Speed Limit on major street = 45 mph
VII. Speed Limit on minor street= 35 mph
VIII. No right turns are considered for analysis

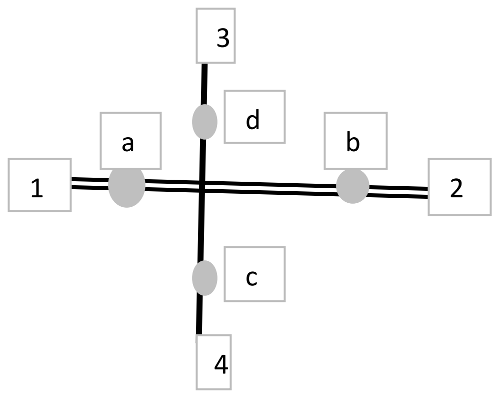


Figure A.1 Origin-Destination

Referencing Figure 1 : O-D Volumes for 1-3: 25 vph, 1-2:475 vph, 2-4: 25 vph, 2-1: 475 vph, 4-1: 5 vph, 4-3: 95 vph, 3-2: 5vph, 3-4: 95 vph

Case A: Standard signalized intersection
quick start

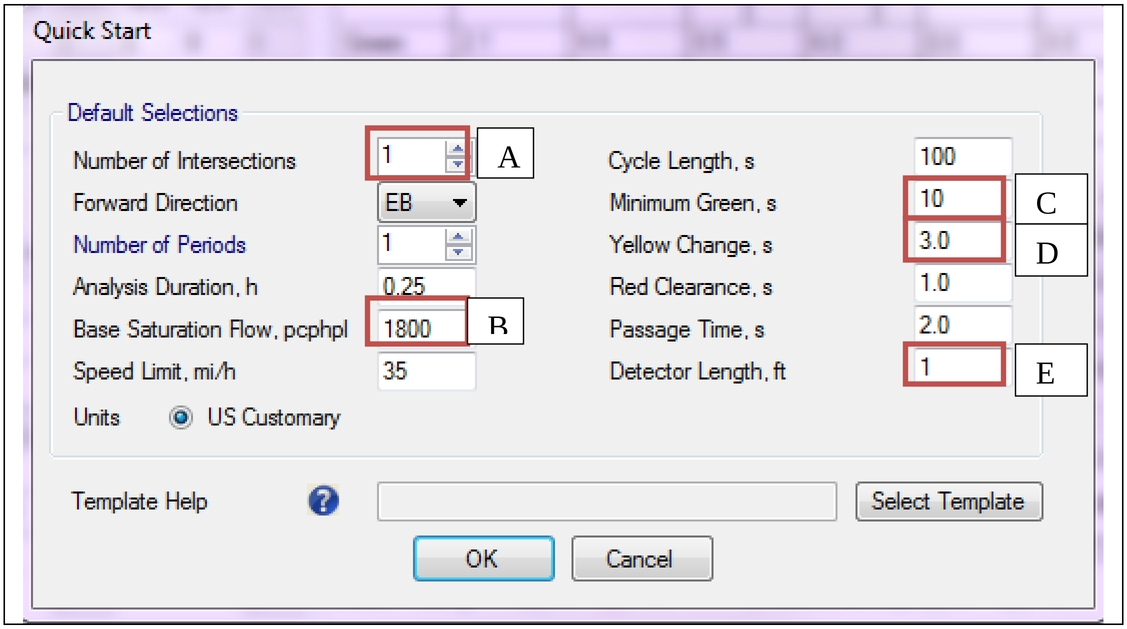
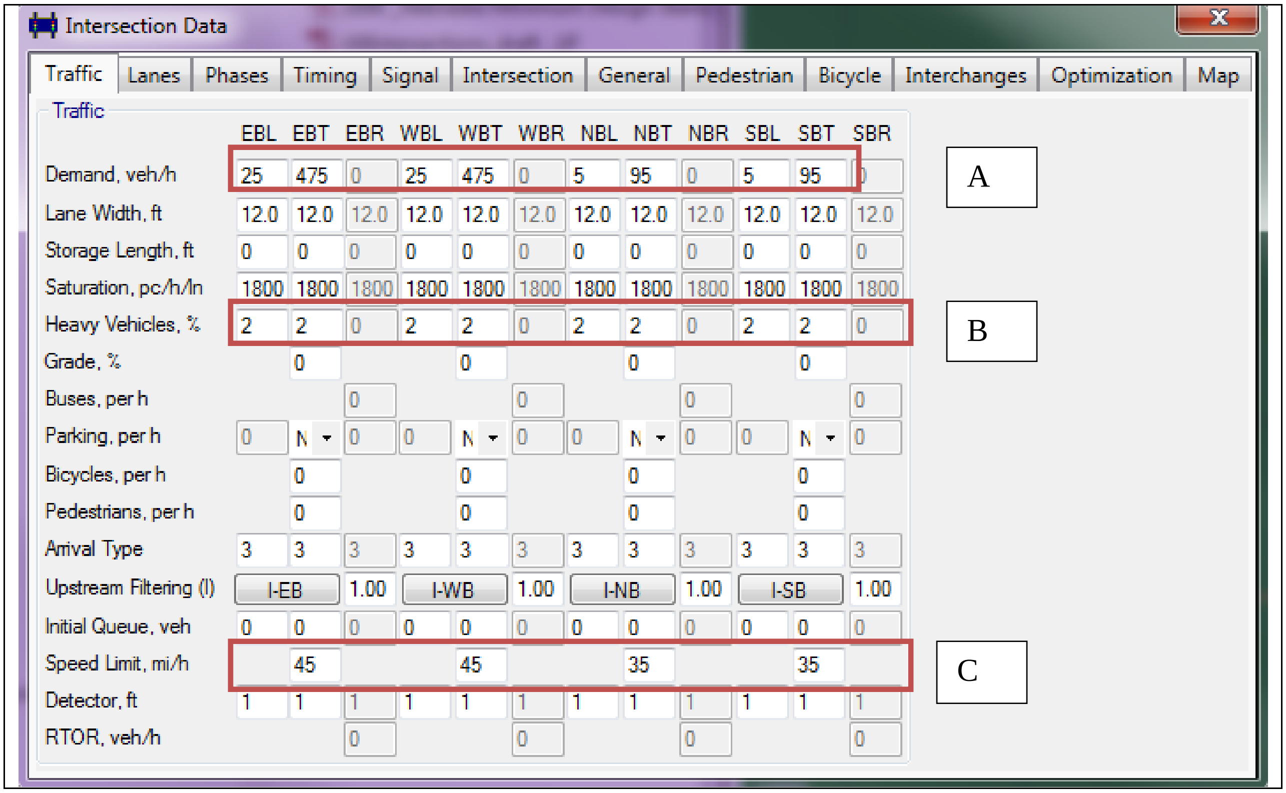


Figure A.2 Quick start inputs

A: Three intersections include two un-signalized median openings and one signalized core intersection
B: The base saturation flow rate of 1800 pcphpl was considered.
C: The minimum green was changed to 10 s.
D: The yellow change time was changed to 3 s.
E: Detector length was set to 1 ft.
All other values were left at default.

Traffic



	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Demand, veh/h	25	475	0	25	475	0	5	95	0	5	95	0
Lane Width, ft	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0
Storage Length, ft	0	0	0	0	0	0	0	0	0	0	0	0
Saturation, pc/h/ln	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
Heavy Vehicles, %	2	2	0	2	2	0	2	2	0	2	2	0
Grade, %		0			0			0			0	
Buses, per h			0			0			0			0
Parking, per h	0	N	0	0	N	0	0	N	0	0	N	0
Bicycles, per h		0			0			0			0	
Pedestrians, per h		0			0			0			0	
Arrival Type	3	3	3	3	3	3	3	3	3	3	3	3
Upstream Filtering (I)	I-EB		1.00	I-WB		1.00	I-NB		1.00	I-SB		1.00
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Speed Limit, mi/h		45			45			35			35	
Detector, ft	1	1	1	1	1	1	1	1	1	1	1	1
RTOR, veh/h			0			0			0			0

Figure A.3 Demand input

A: Demands

B: Truck Percentage

C: Speed

Lane configuration

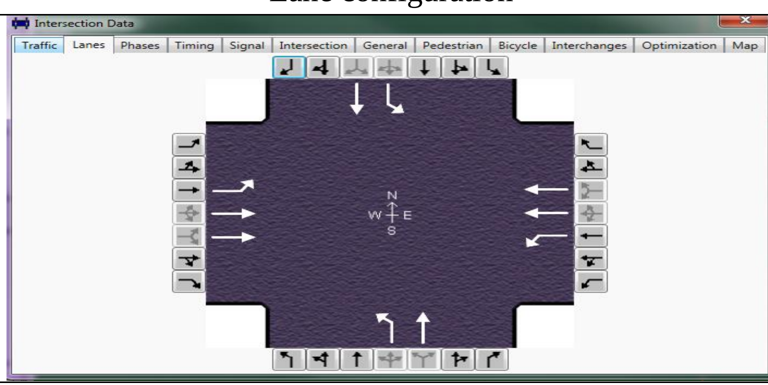


Figure A.4 Lane configuration

Note: There is a exclusive right turning lane. It was not shown in lane configuration as the right turn movement was ingored in operational analysis.

Phasing and timing

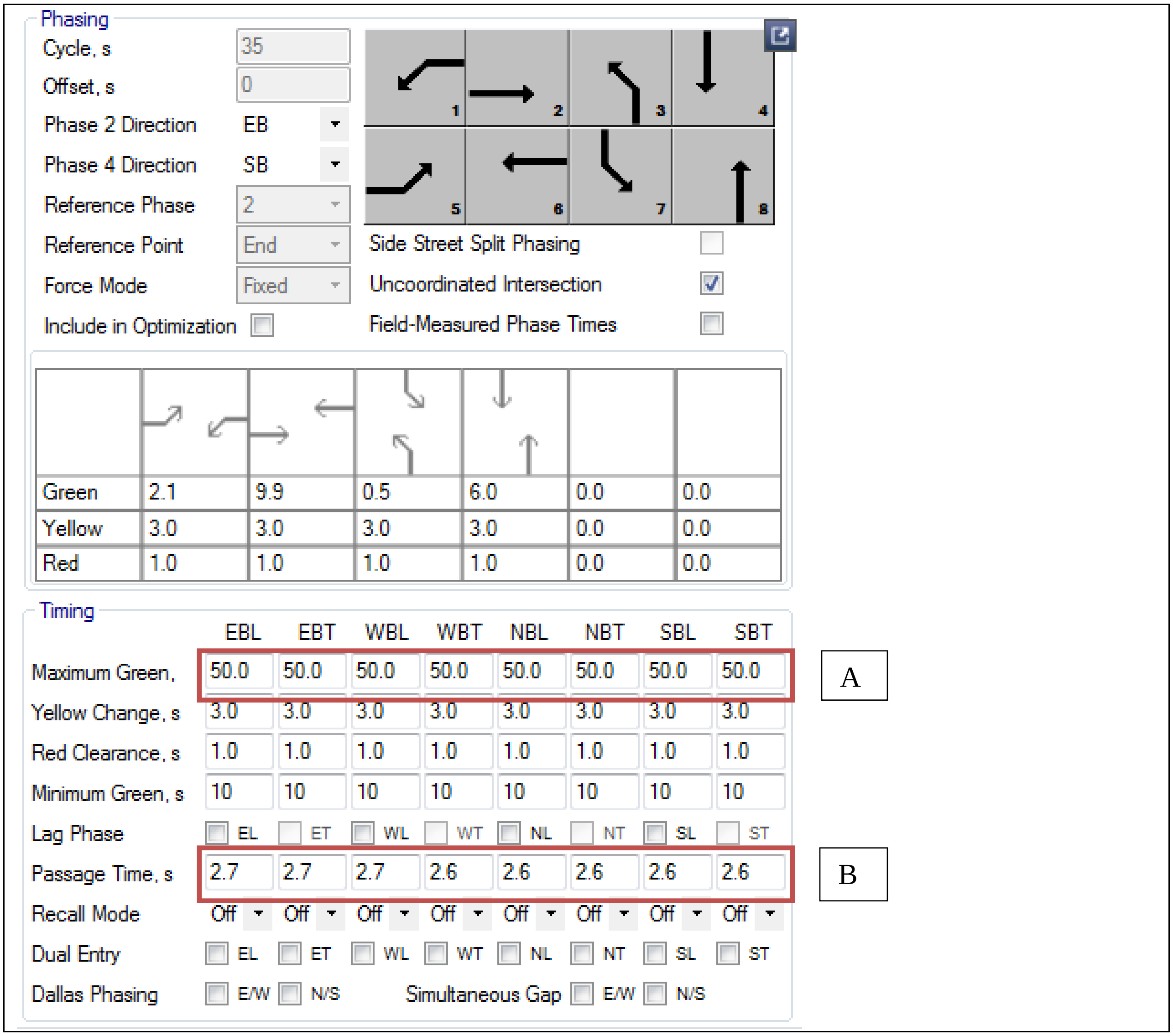
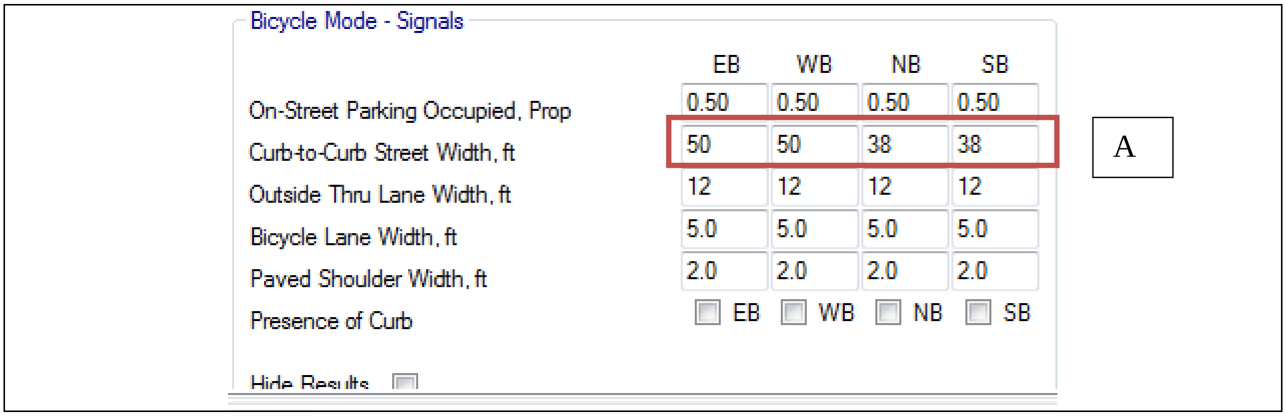


Figure A.5 Lane configuration

A: Maximum green was set to 50 for all movements
B: Passage time (PT) was calculated based on Maximum Allowable Headway (MAH) of 3 s, average lengths of small vehicles as 17 ft. and heavy vehicles as 45 ft. and estimating the combined average length (L_V) based on truck percentage, length of detection zone (L_D) of 1 foot and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship

$$PT = MAH - \frac{L_{V+}\,L_D}{1.47\,V_A}$$



Intersection width

Figure A.6 Curb to curb width

A: Curb-Curb width

EB/WB width = $4 * 12(Lane\ Width) + 1 * 2(Shoulder\ Width) = 50\ ft.$

NB/SB width = $3 * 12(Lane\ Width) + 1 * 2(Shoulder\ Width) = 38\ ft.$

Case B: Median U-Turn

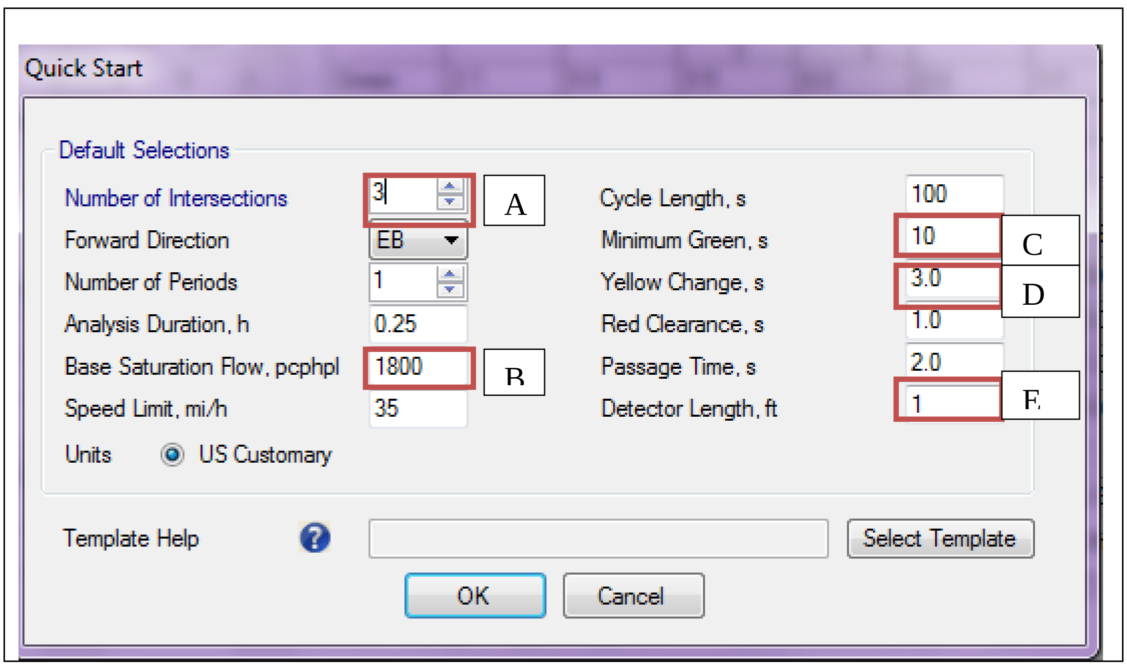


Figure A.7 Quick start inputs

A: Three intersections include two un-signalized median openings and one signalized core intersection

B: The base saturation flow rate of 1800 pcphpl was considered.

C: The minimum green was changed to 10 s.

D: The yellow change time was changed to 3 s.

E: Detector length was set to 1 ft.
All other values were left default.

Traffic (west median opening)

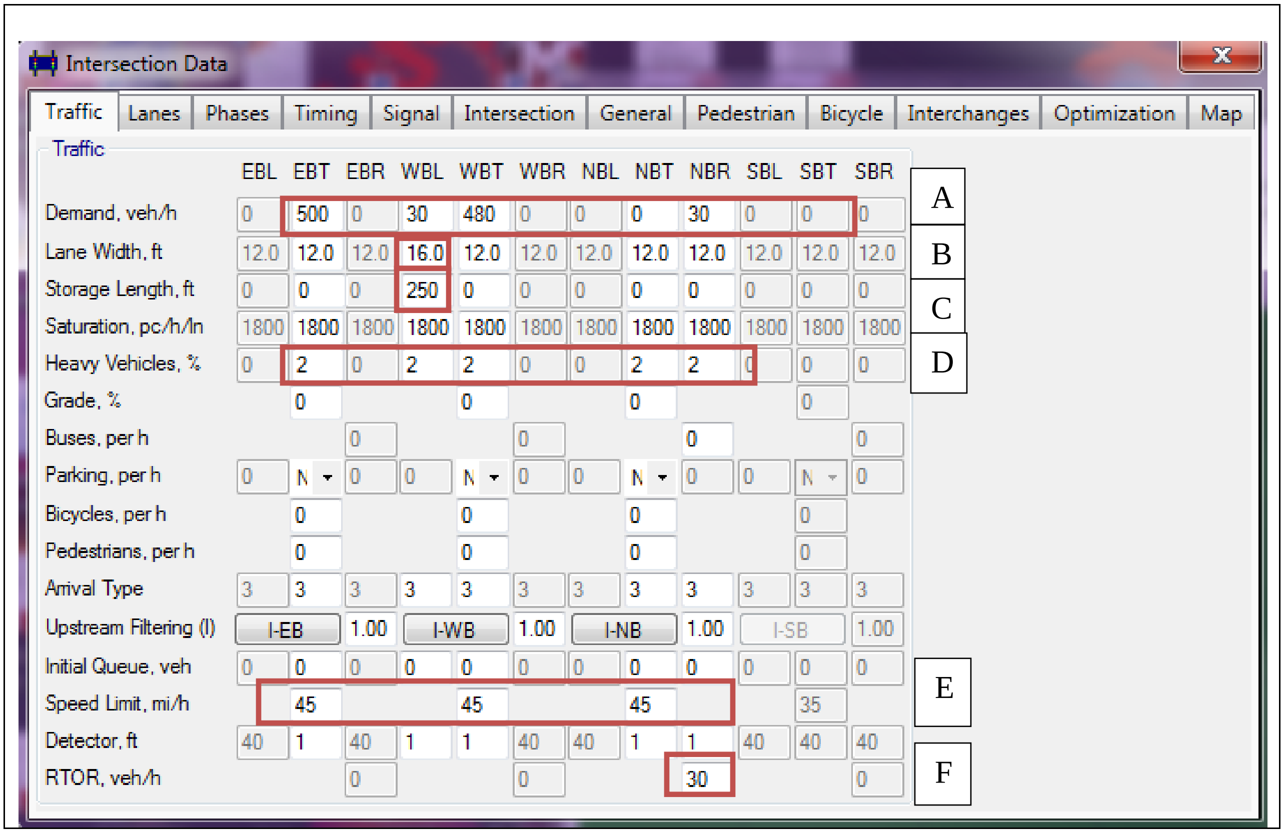


Figure A.8 Demand input for west median opening

A: Volumes for West Median
B: Lane width of U-Turn lane was set to maximum 16 ft. considering 30 ft. wide median opening
C: Storage Length
D:Truck percenatge
E: Speed (All majors)
F: RTOR Volume = NBR Volume

Lane configuration (west median opening)

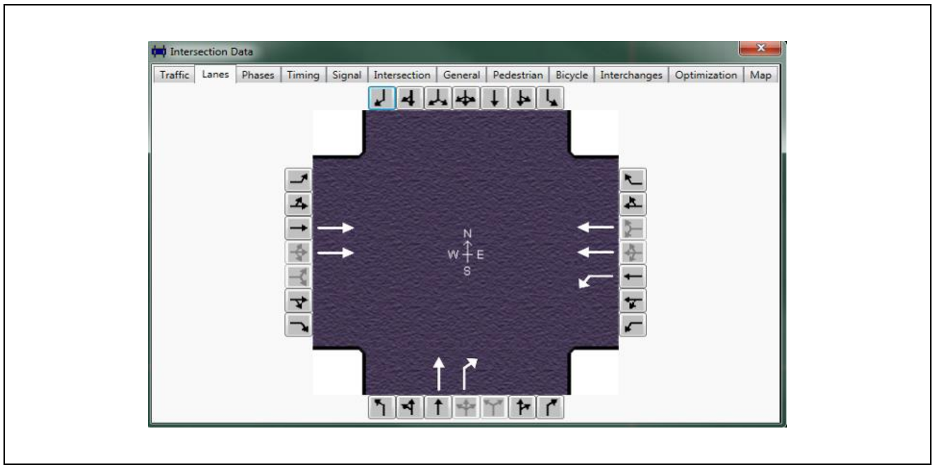
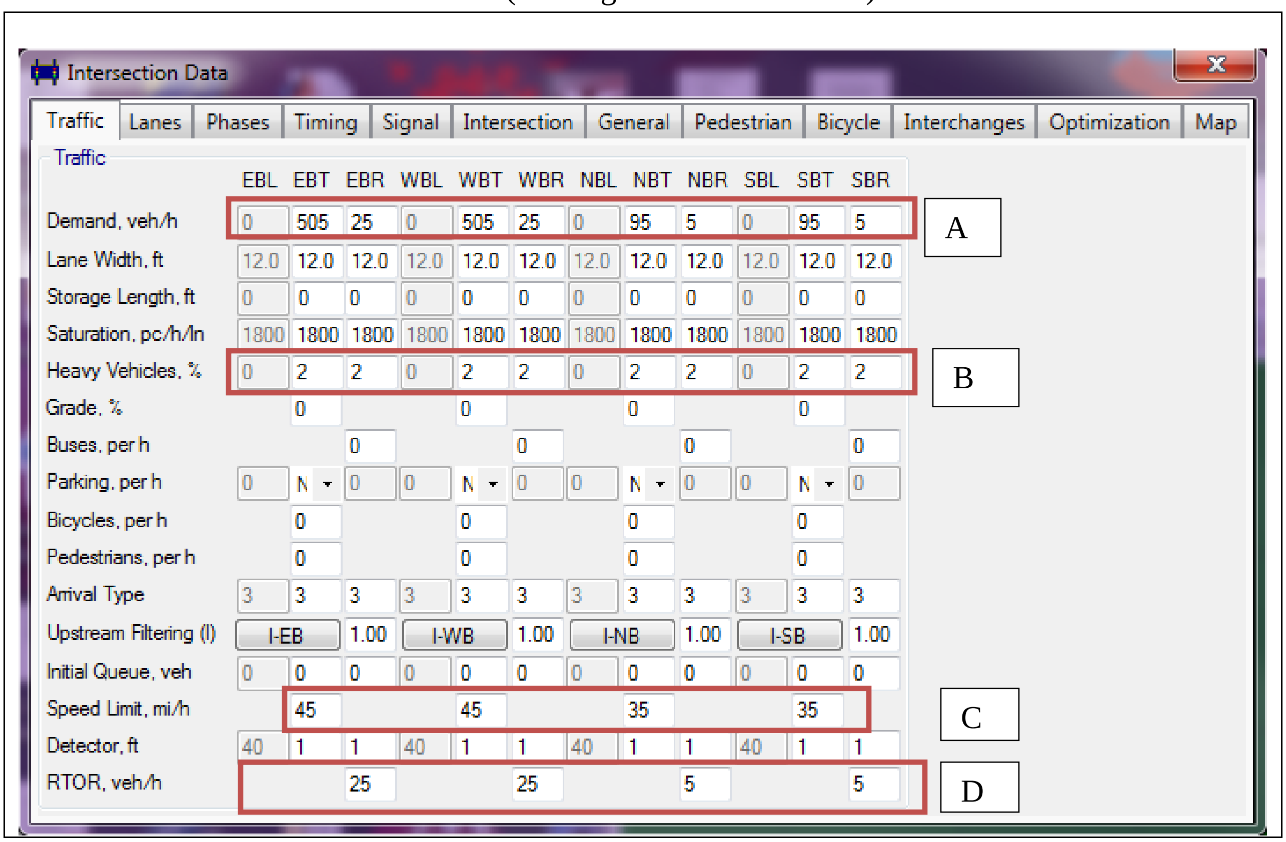


Figure A.9 Lane configuration of west median opening

Traffic (core signalized intersection)



	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Demand, veh/h	0	505	25	0	505	25	0	95	5	0	95	5
Lane Width, ft	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0
Storage Length, ft	0	0	0	0	0	0	0	0	0	0	0	0
Saturation, pc/h/ln	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
Heavy Vehicles, %	0	2	2	0	2	2	0	2	2	0	2	2
Grade, %		0			0			0			0	
Buses, per h			0			0			0			0
Parking, per h	0	N	0	0	N	0	0	N	0	0	N	0
Bicycles, per h		0			0			0			0	
Pedestrians, per h		0			0			0			0	
Arrival Type	3	3	3	3	3	3	3	3	3	3	3	3
Upstream Filtering (I)	I-EB		1.00	I-WB		1.00	I-NB		1.00	I-SB		1.00
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Speed Limit, mi/h		45			45			35			35	
Detector, ft	40	1	1	40	1	1	40	1	1	40	1	1
RTOR, veh/h			25			25			5			5

Figure A.10 Demand input for core signalized intersection

A: Demand input

B: Truck percentage

C: Speeds (Major street: 45 mph, Minor street: 35 mph)
D: All right turners were coded as RTOR

Lane configuration (Central signalized intersection)

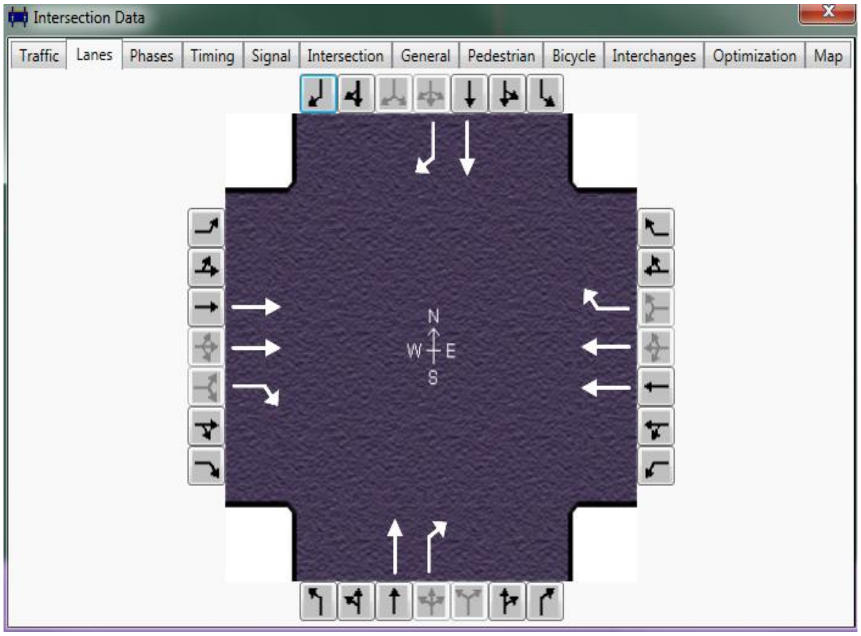


Figure A.11 Lane configuration of central signalized intersection

Traffic (east median opening)

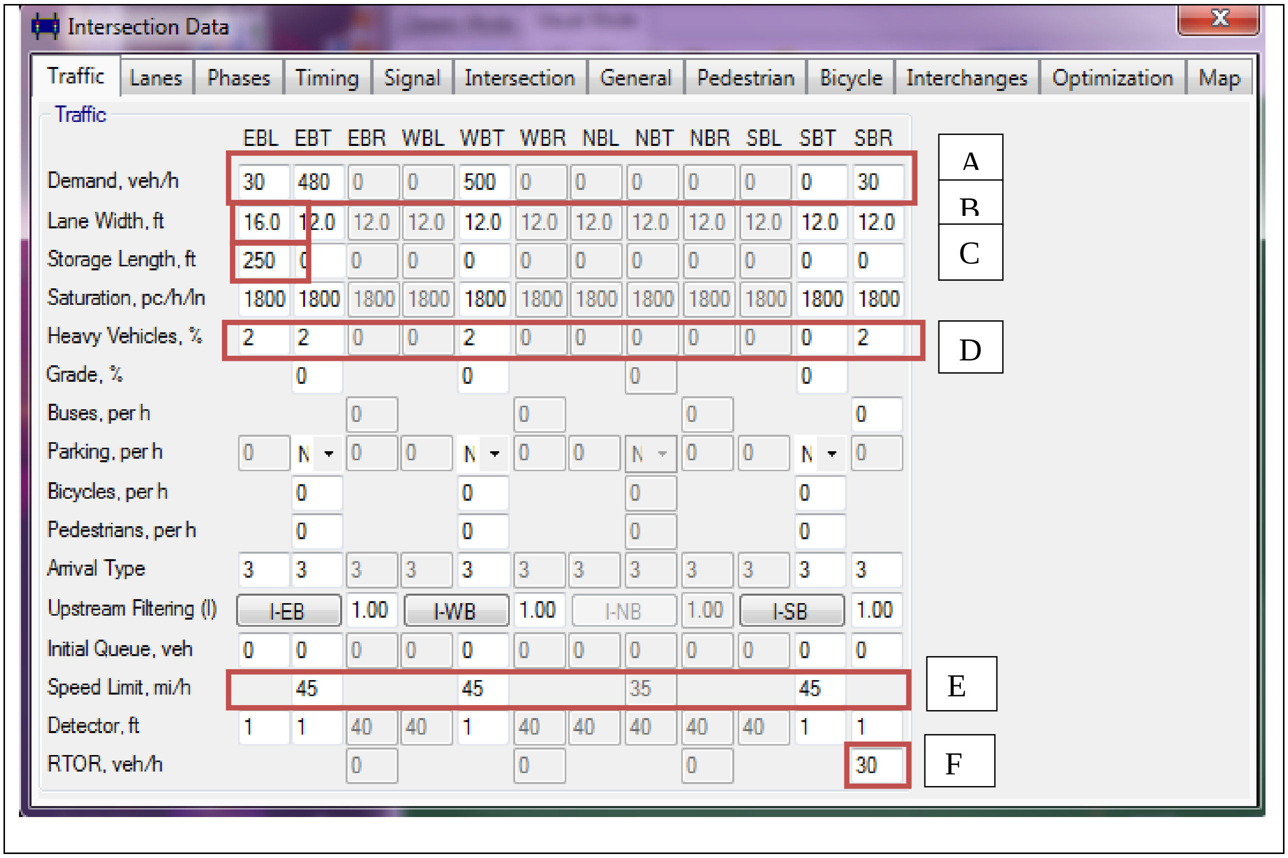


Figure A.12 Demand input for east median opening

A: Volumes for east median opening

B: Lane width of U-turn lane was set to maximum 16 ft. considering 30 ft. wide median opening

C: Storage Length = 250 ft.

D: Truck Percentage

E: Speed (All majors)

F= RTOR Volume = SBR Volume

Lane configuration (east median opening)

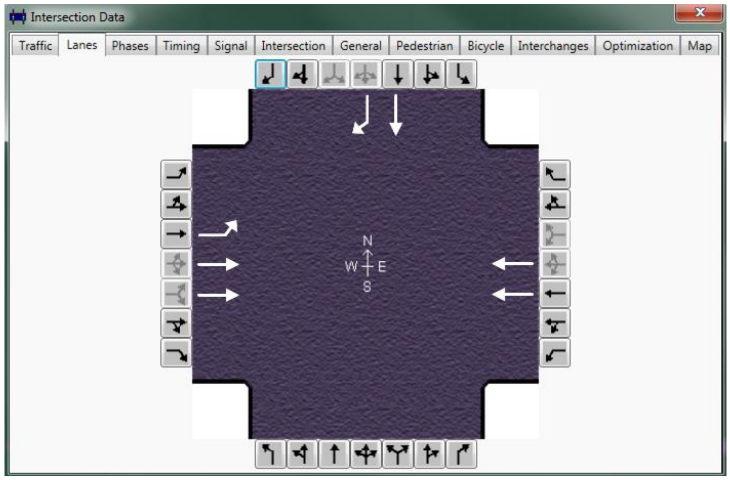


Figure A.13 Lane configuration of east median opening

Phasing and Timing (west median opening)

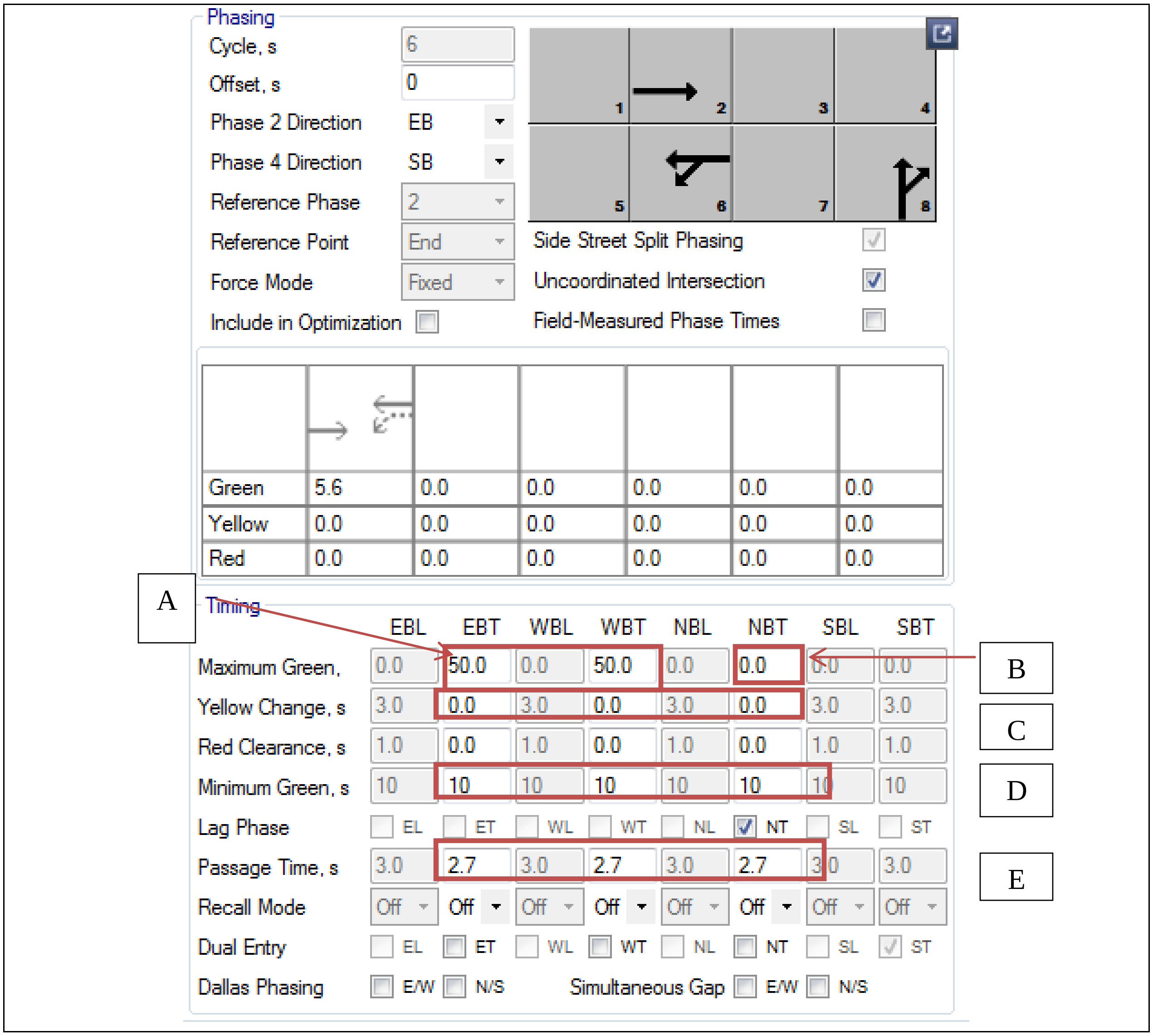


Figure A.14 Phasing and timing of west median opening

A: Maximum Green times were set to 50 s for EBT and WBT movements.

B: No green time was allocated for NBT

C: Yellow change time was set to zero

D: Red clearence was set to zero

E: Passage time (PT) was calculated based on maximum allowable headway (MAH) of 3 s, the average length of small vehicles as 17 ft. and heavy vehicles as 45 ft., and estimating the combined average length (L_V) based on truck percentage, length of detection zone (L_D) of 1 foot, and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship:

$$PT = MAH - \frac{L_{V+}L_D}{1.47\ V_A}$$

Phasing and Timing of Core Signalized Intersection

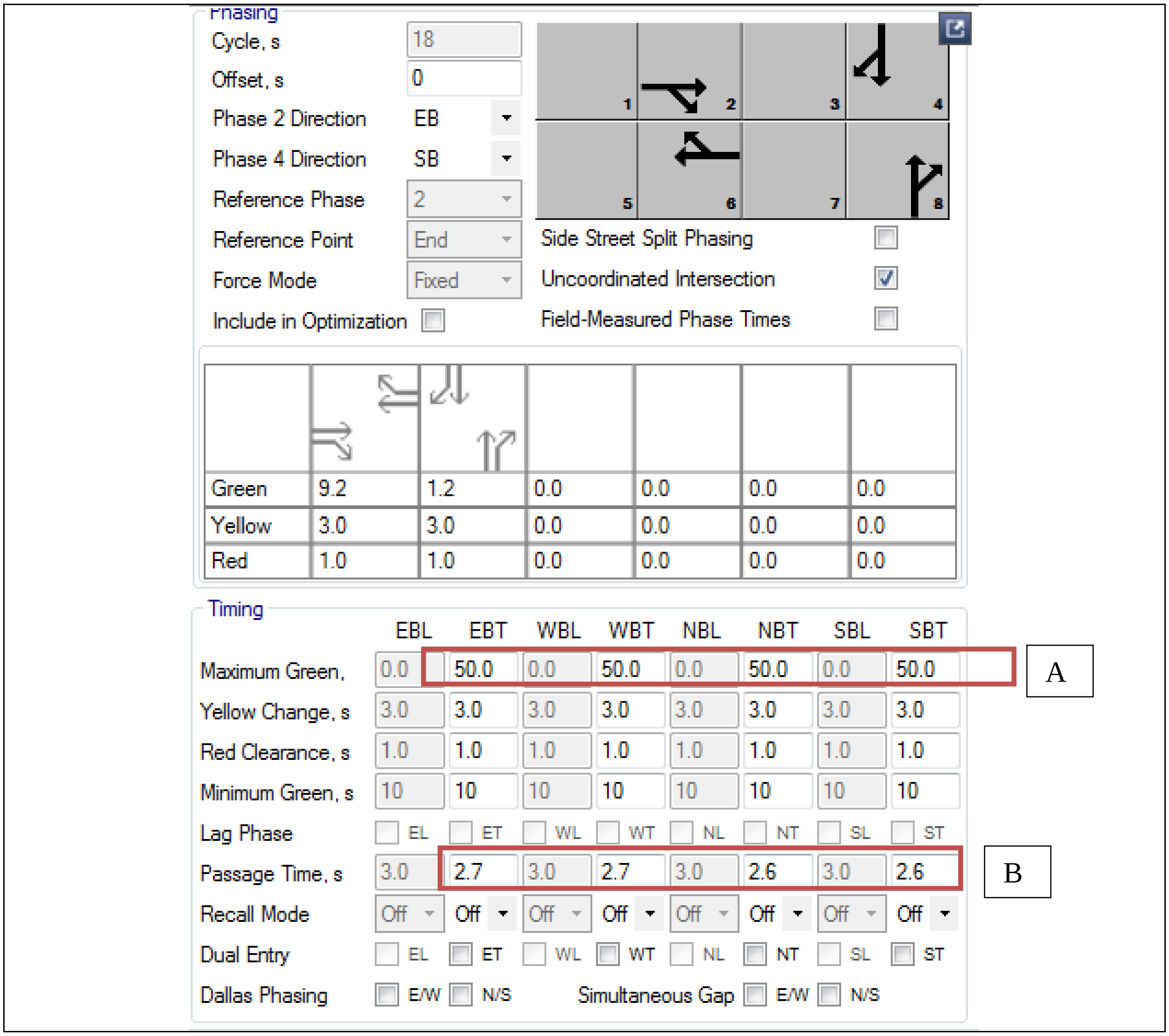


Figure A.15 Phasing and timing of core signalized intersection

A: Maximum Green times were set to 50 s for all movements.

B: Passage time (PT) was calculated based on maximum sllowable headway (MAH) of 3 s, average length of small vehicles as 17 ft. and heavy vehicles as 45 ft., and estimating the combined average length (L_V) based on truck percentage, length of detection zone (L_D) of 1 foot and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship:

$$PT = MAH - \frac{L_{V+} L_D}{1.47\ V_A}$$

Phasing and timing (east median opening)

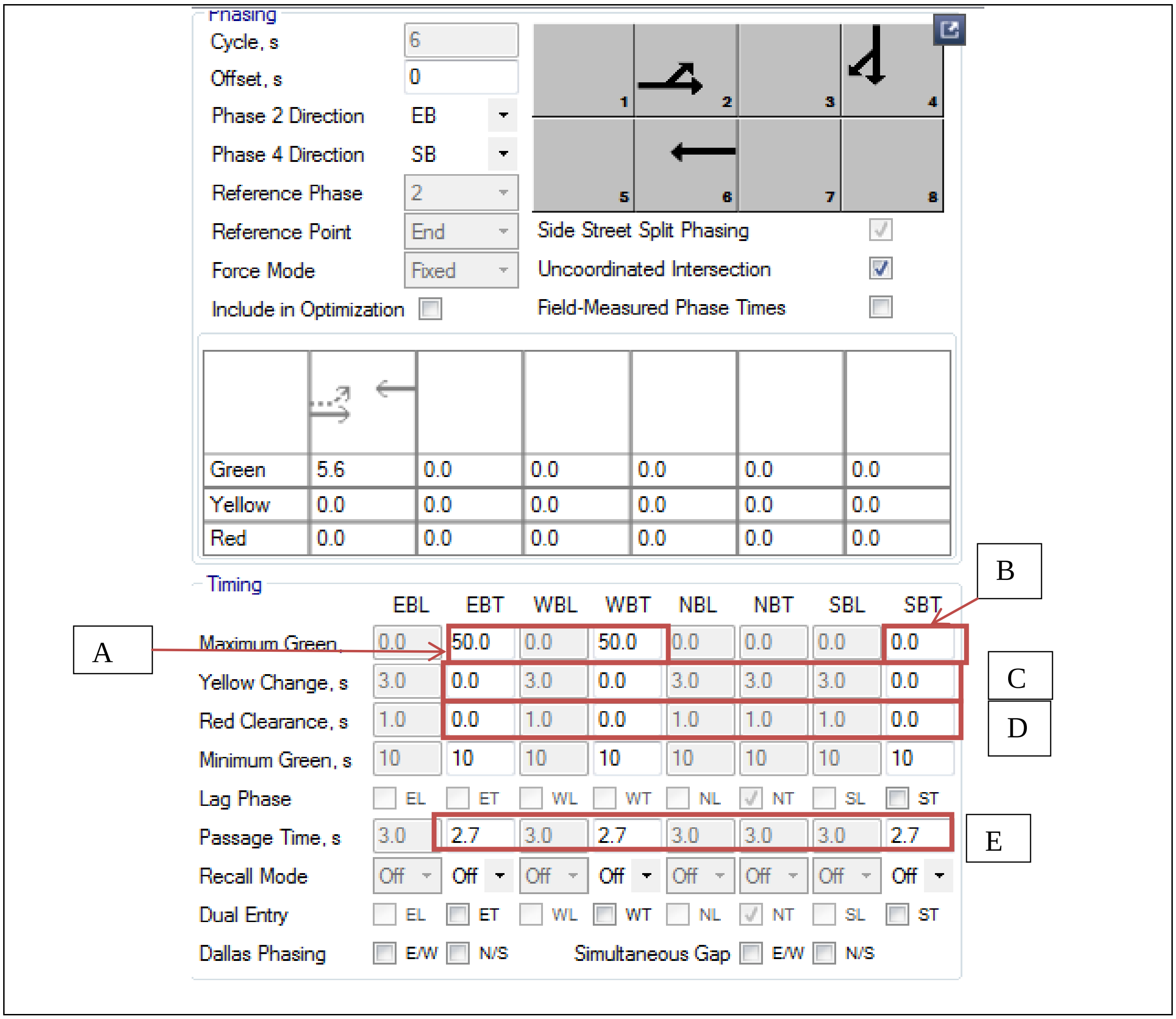


Figure A.16 Phasing and timing of east median opening

A: Maximum Green times were set to 50 s for EBT and WBT movements.

B: No green time was allocated for SBT

C: Yellow change time was set to zero

D: Red clearence was set to zero

E: Passage time (PT) was calculated based on maximum allowable headway (MAH) of 3 s, average lengths of vehicle of small vehicles as 17 ft. and heavy vehicles as 45 ft., and estimating the combined average length (L_V) based on truck percentage, length of detection zone (L_D) of 1 foot and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship:

$$PT = MAH - \frac{L_{V+} L_D}{1.47\ V_A}$$

Critical headway and Follow up headway for Median Openings

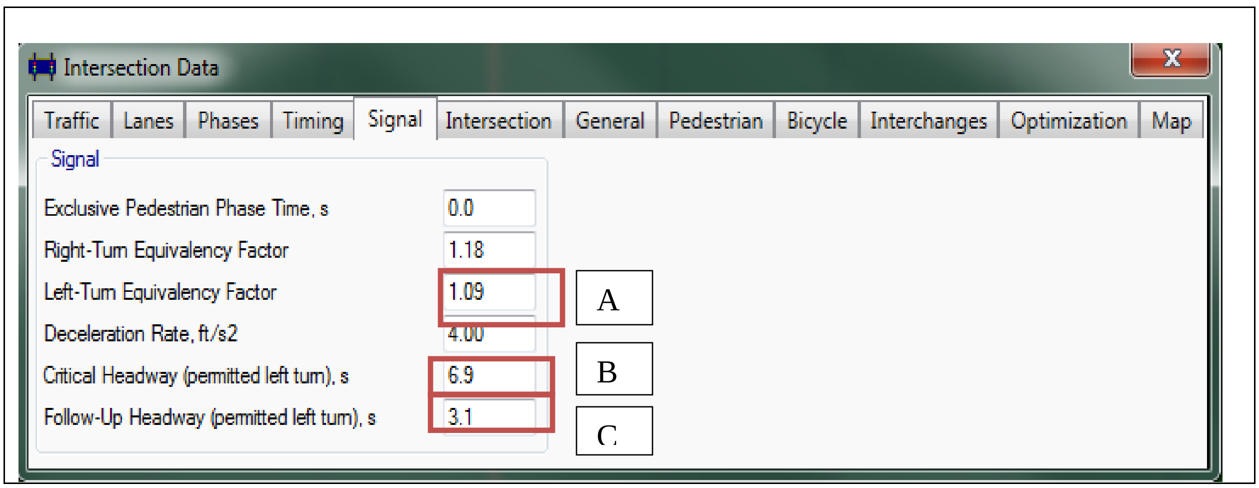


Figure A.17 Critical headway and follow up headway for U-turn movement at median openings

A: Left Turn Equivalency factor calculated for U –Turn for 60 ft. turning radius using the equation

$$f_R = \frac{1}{1 + \frac{5.61}{R}}$$

B: The Critical headway was calculated based on equation 19-30 from HCM 2010

C: The Follow up headway was calculated based on equation 19-31 from HCM 2010.

Segment and median widths

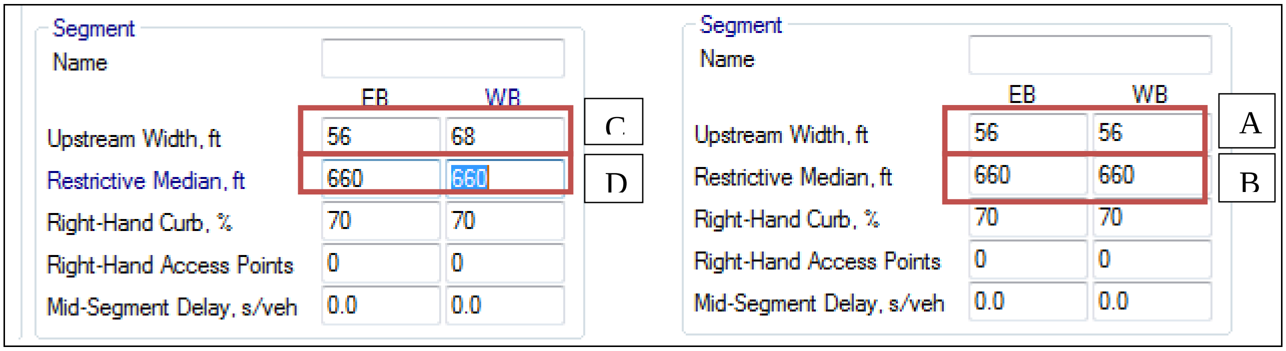


Figure A.18 Upstream segment widths for segments 1 and 2

A: Upstream segment widths (EB/WB) for segment 1,

Segment 1 EB/WB width

$$= \frac{60\ (Median\ width)}{2} + 2 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 56\ ft$$

B and D: Restrictive median lengths = Offset length = 660 ft.

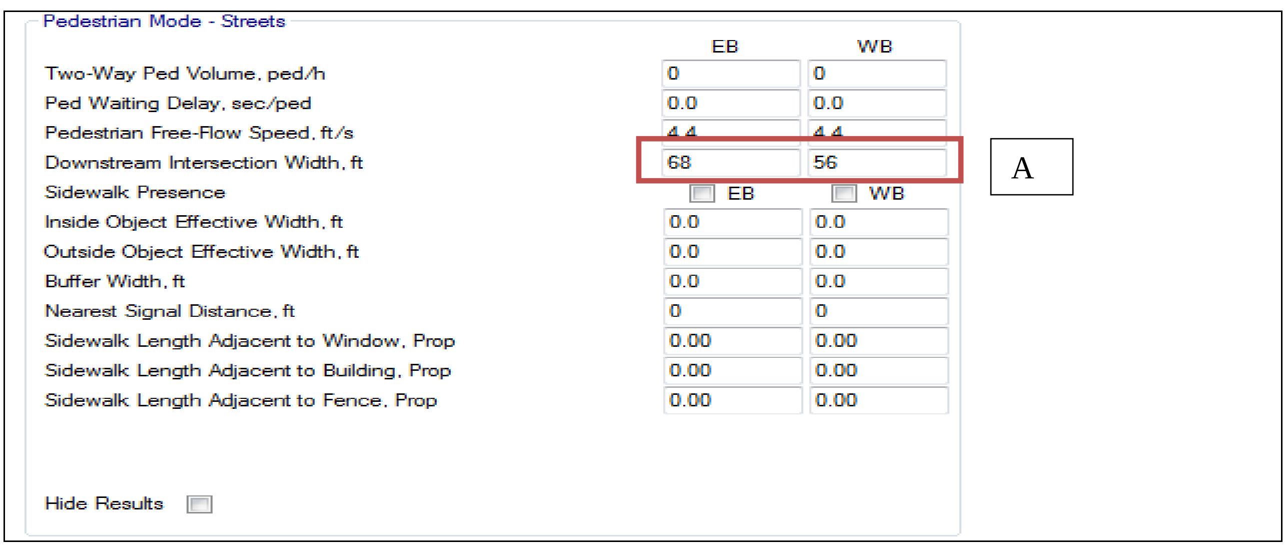
C: Upstream segment widths (EB/WB) for segment 2,

Segment 2 WB width

$$= \frac{60\ (Median\ width)}{2} + 3 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 68\ ft$$

Segment 2 width EB

$$= \frac{60\ (Median\ width)}{2} + 2 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 56\ ft$$



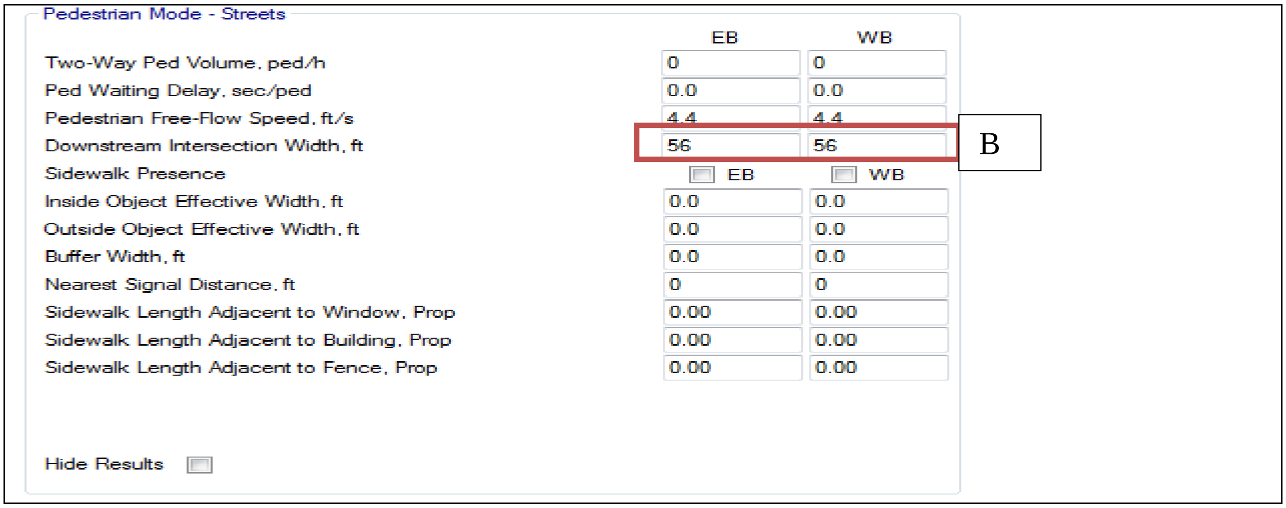


Figure A.19 Downstream segment widths for segments 1 and 2 both

A: Downstream segment widths (EB/WB) for segment 1,

Segment 1 EB width

$$= \frac{60\ (Median\ width)}{2} + 3 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 68\ ft$$

Segment 1 width WB

$$= \frac{60\ (Median\ width)}{2} + 2 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 56\ ft$$

B: Downstream segment widths for segment 2

Segment 2 EB/WB width

$$= \frac{60\ (Median\ width)}{2} + 2 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 56\ ft$$

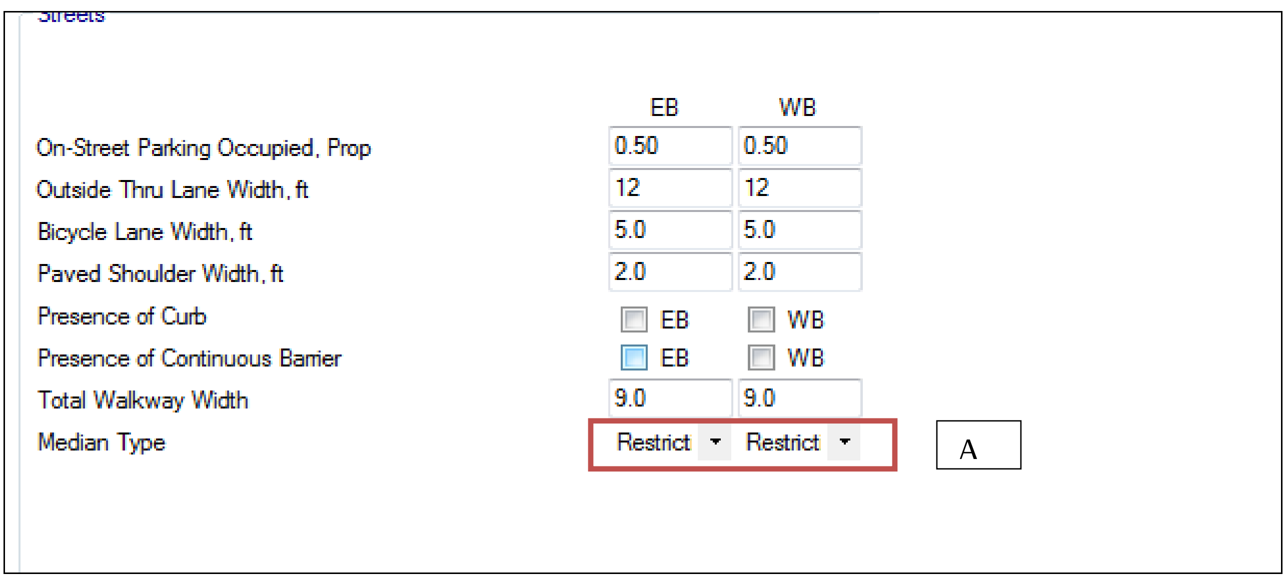


Figure A.20 Median type (applicable for segments 1 and 2)

A: Median Type = Restrictive for all cases

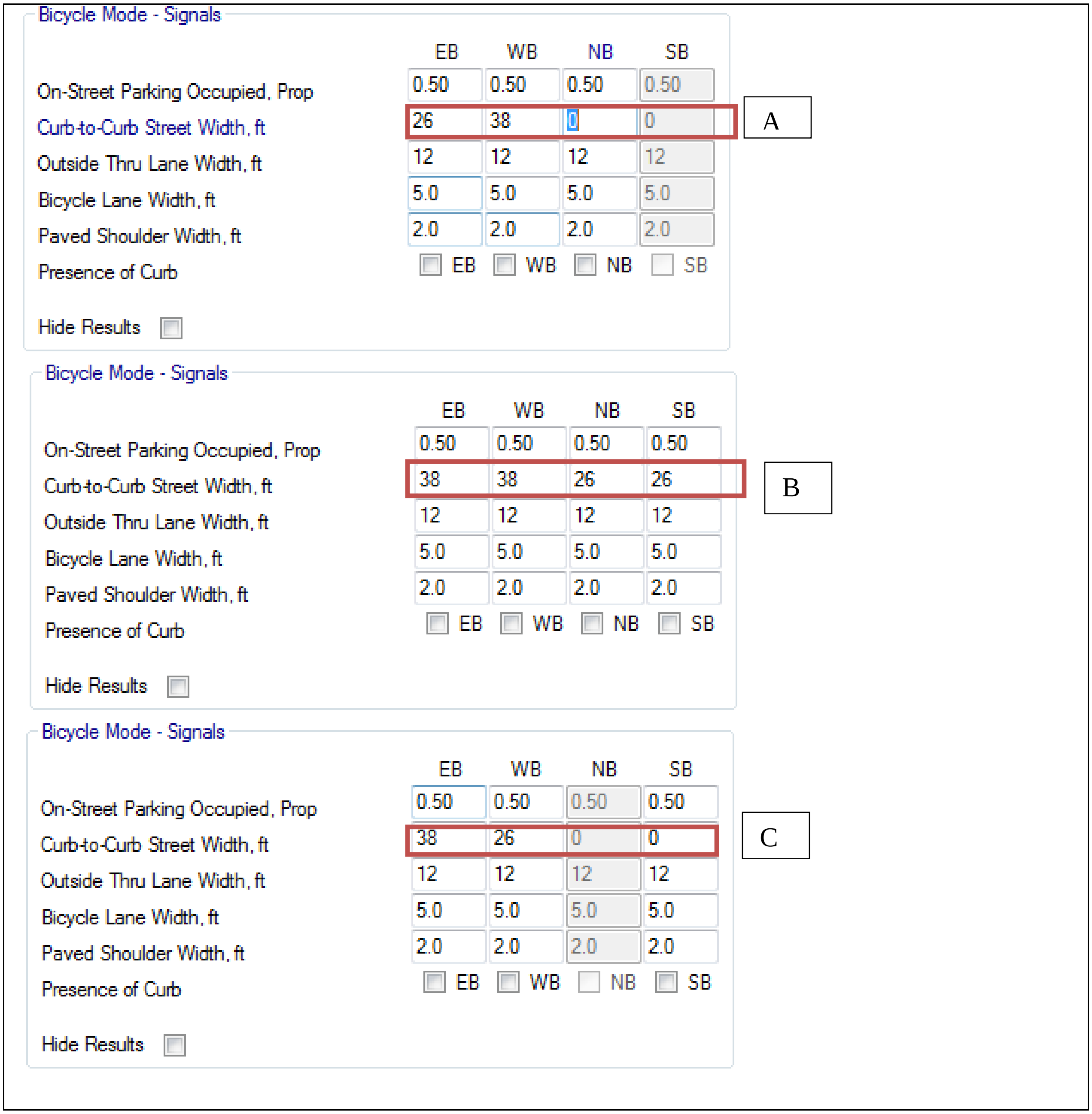


Figure A.21 Curb widths (3 intersections)

A, B, C: Curb to Curb width for west median opening, Signalized intersection and east median opening

$width\ of\ EB\ of\ west\ median\ opening\ ,WB\ of\ east\ medain\ opening\ and\ NBand\ SB\ width\ for\ mi$

$2 * 12(Lane\ Width) + 1 * 2(Shoulder\ Width) = 26$

ft.

$width\ of\ WB\ of\ west\ median\ opening\ , EB\ of\ east\ medain\ opening\ and\ EBand\ WB\ width\ for\ m$
$3 * 12(Lane\ Width) + 1 * 2(Shoulder\ Width) = 38$
ft.

Case C: CFI

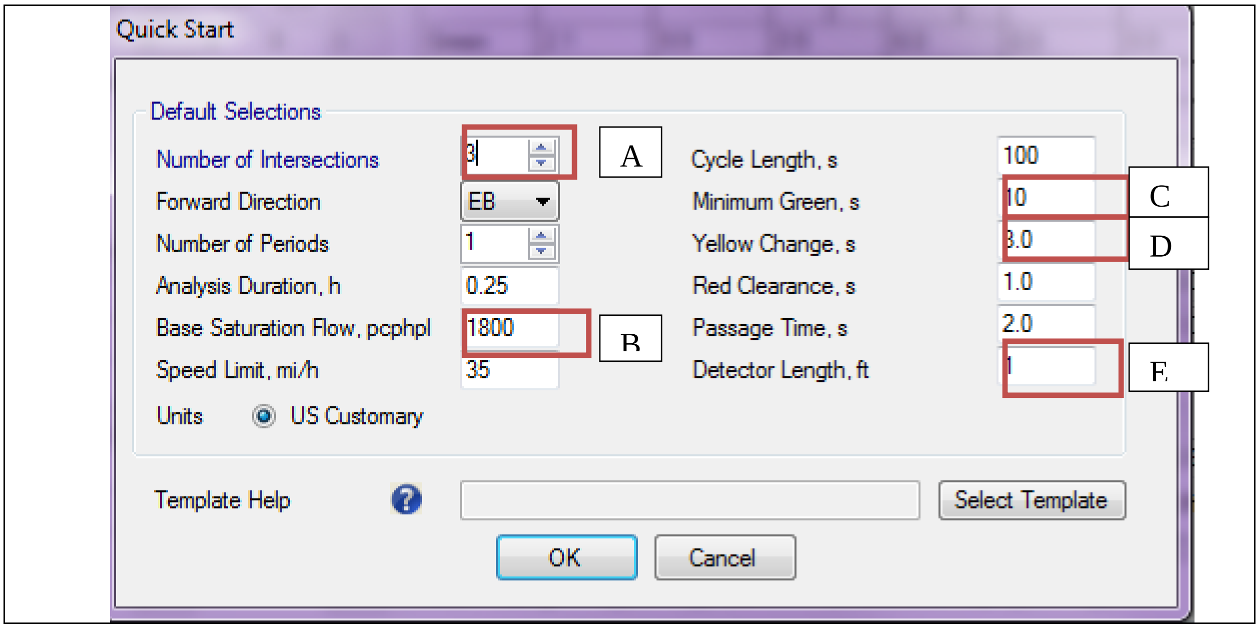


Figure A.22 Quick start inputs

A: Three intersections include two un-signalized median openings and one signalized core intersection
B: The base saturation flow rate of 1800 pcphpl was considered.
C: The minimum green was changed to 10 s.
D: The yellow change time was changed to 3 s.
E: Detector length was set to 1 ft.

Traffic Input (west crossover)

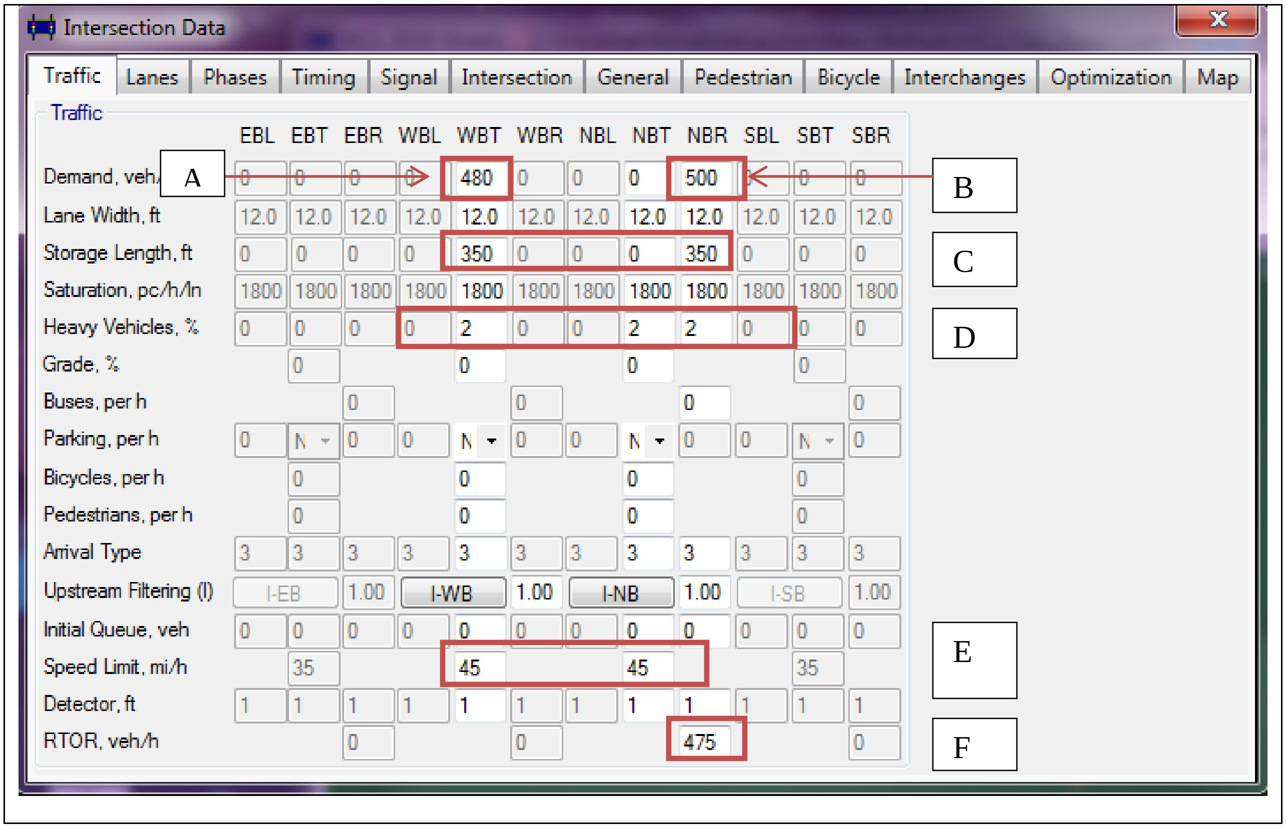


Figure A.23 Traffic input for west crossover

A: WBT Volume, B: NBR Volume = EBT + EBL (crossover)
C: Storage Length = Offset of crossover
D: Truck Percentage
E: Speed (all majors)
F: RTOR Volume = EBT

Traffic input (central core intersection)

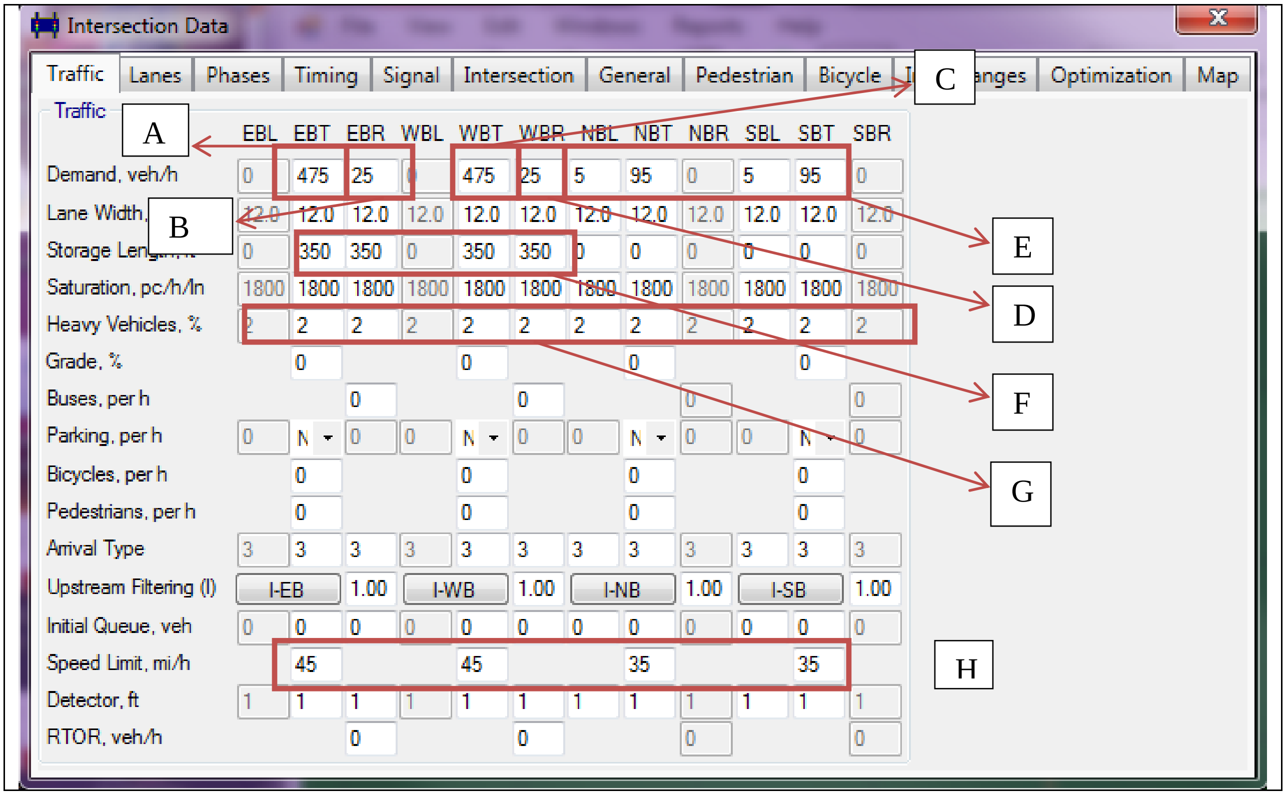


Figure A.24 Traffic input for central intersection

A: EBT Volume
B: EBR Volume = EBL (Crossover)
C: WBT Volume
D: WBL (Crossover)
E: NBL and NBT Volumes
F: Storage length = Offset of Crossover
G: Truck Percentage
H: Speeds (Major street: 45 mph, Minor street: 35 mph)

Traffic input (east crossover)

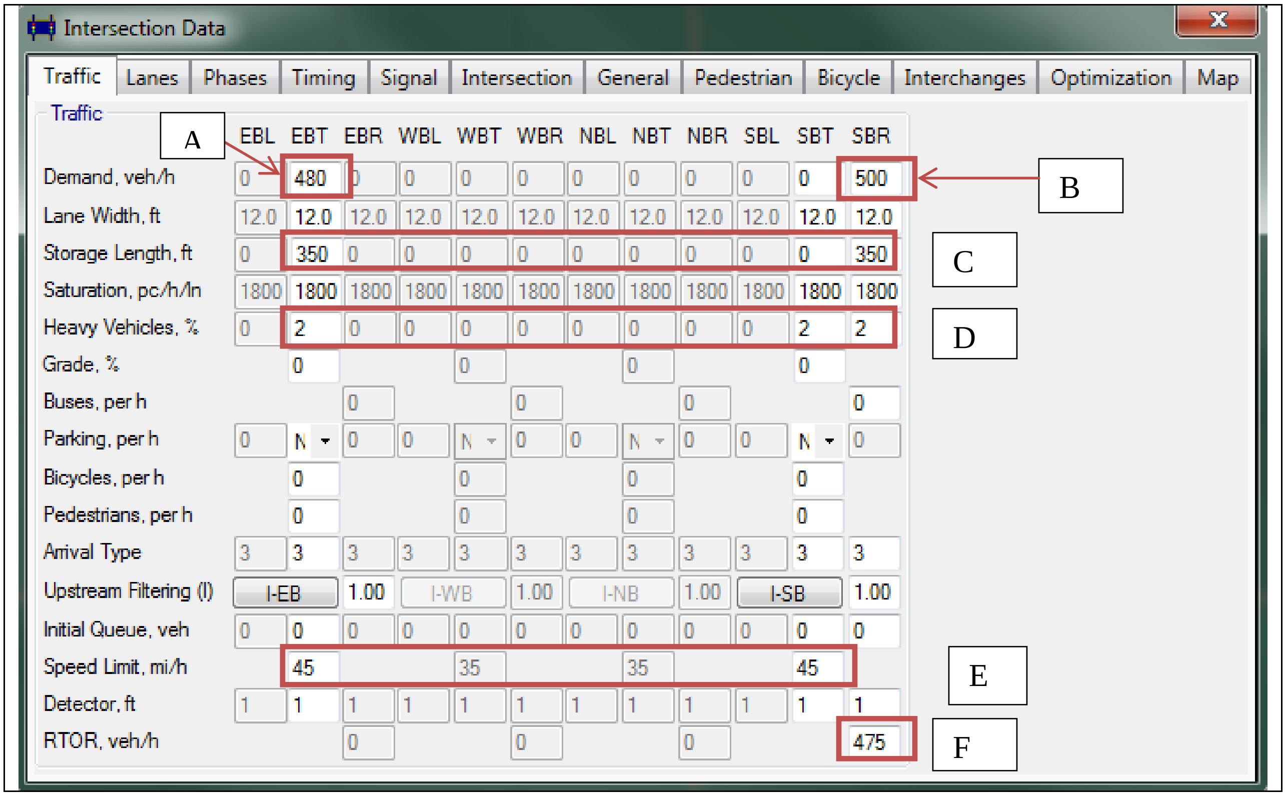


Figure A.25 Traffic input for east crossover

A: EBT Volume, B: SBR Volume = WBT + WBL (crossover)
C: Storage Length = Offset of crossover
D: Truck Percentage
E: Speed (All Majors)
F: RTOR Volume = WBT

Lane configuration (west crossover)

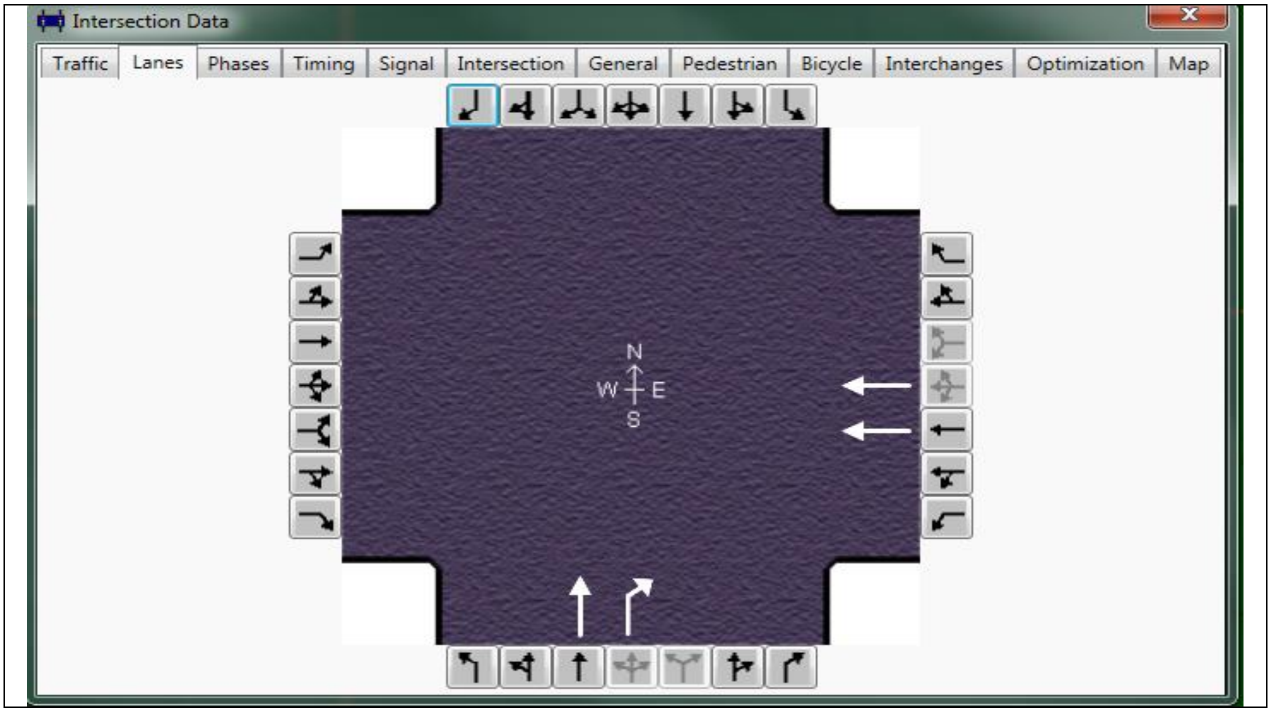


Figure A.26 Lane configuration west crossover

Lane configuration (central signalized intersection)

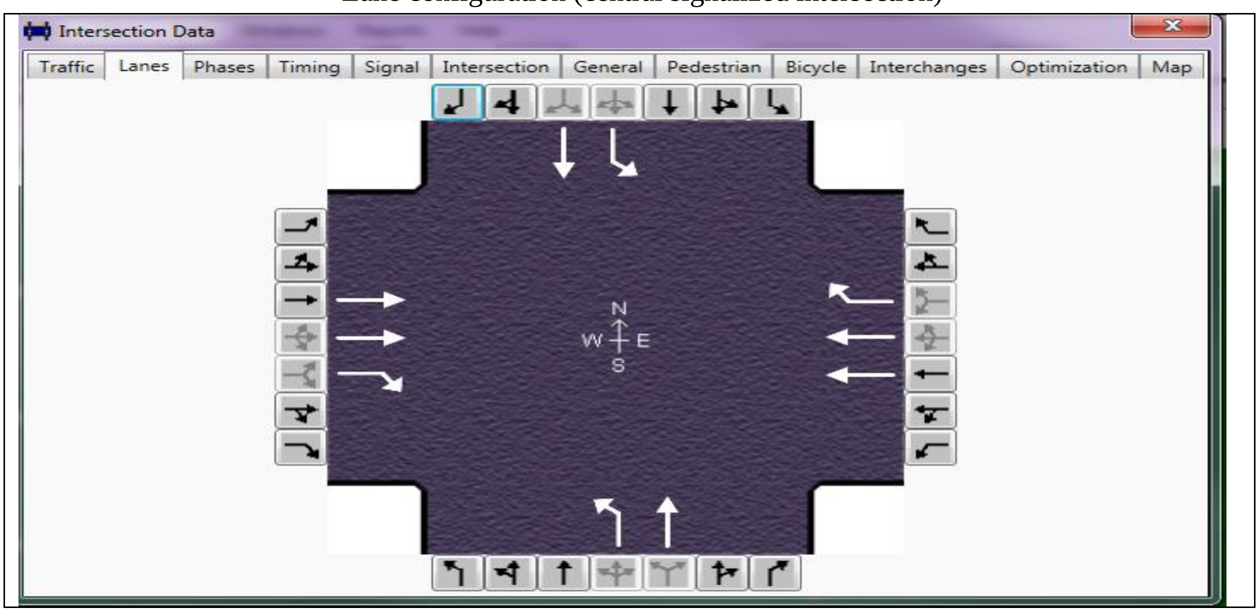


Figure A.27 Lane configuration central signalized intersection

Lane configuration (east crossover)

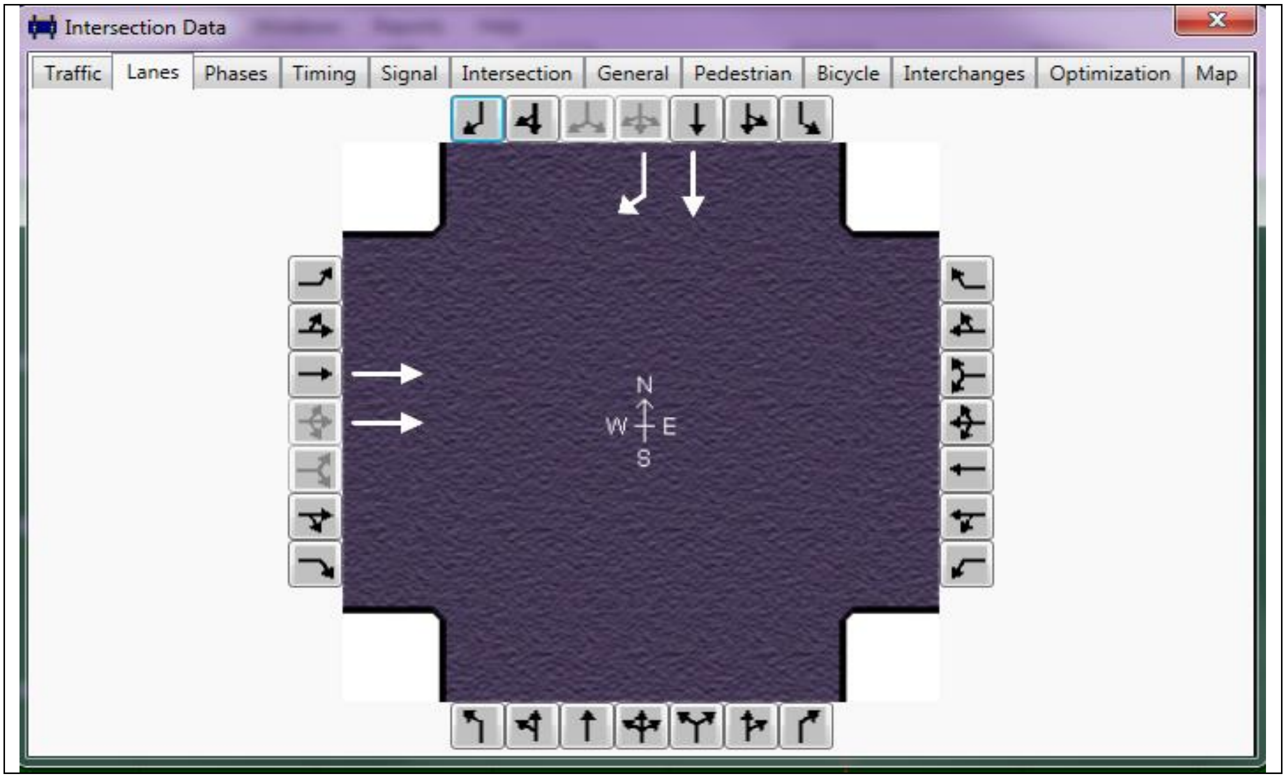


Figure A.28 Lane configuration east crossover

Equivalency factors for central intersection

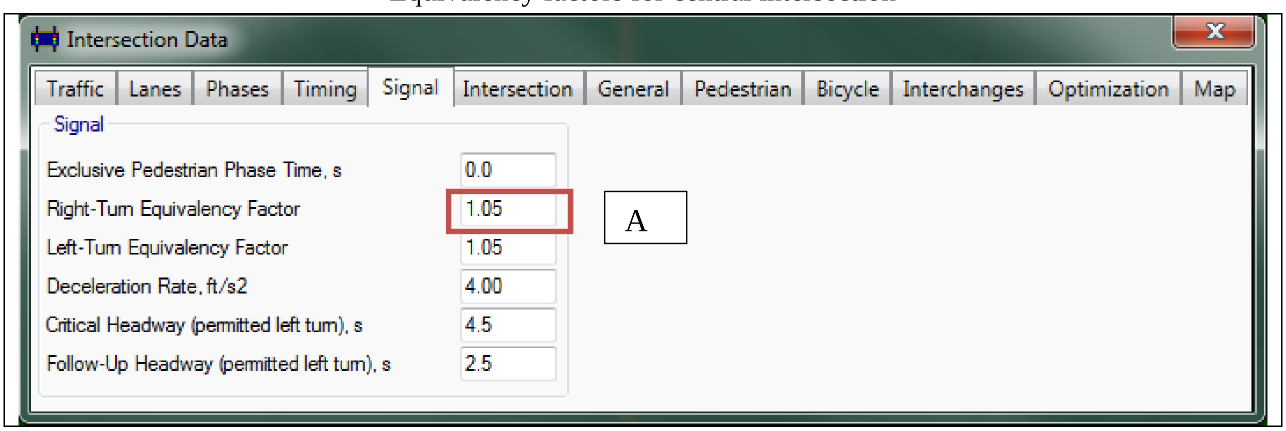


Figure A.29 Equivalency factors

A: Right turn equivalency factor was made equal to left turn equivalency factor

Phasing and timing (west crossover)

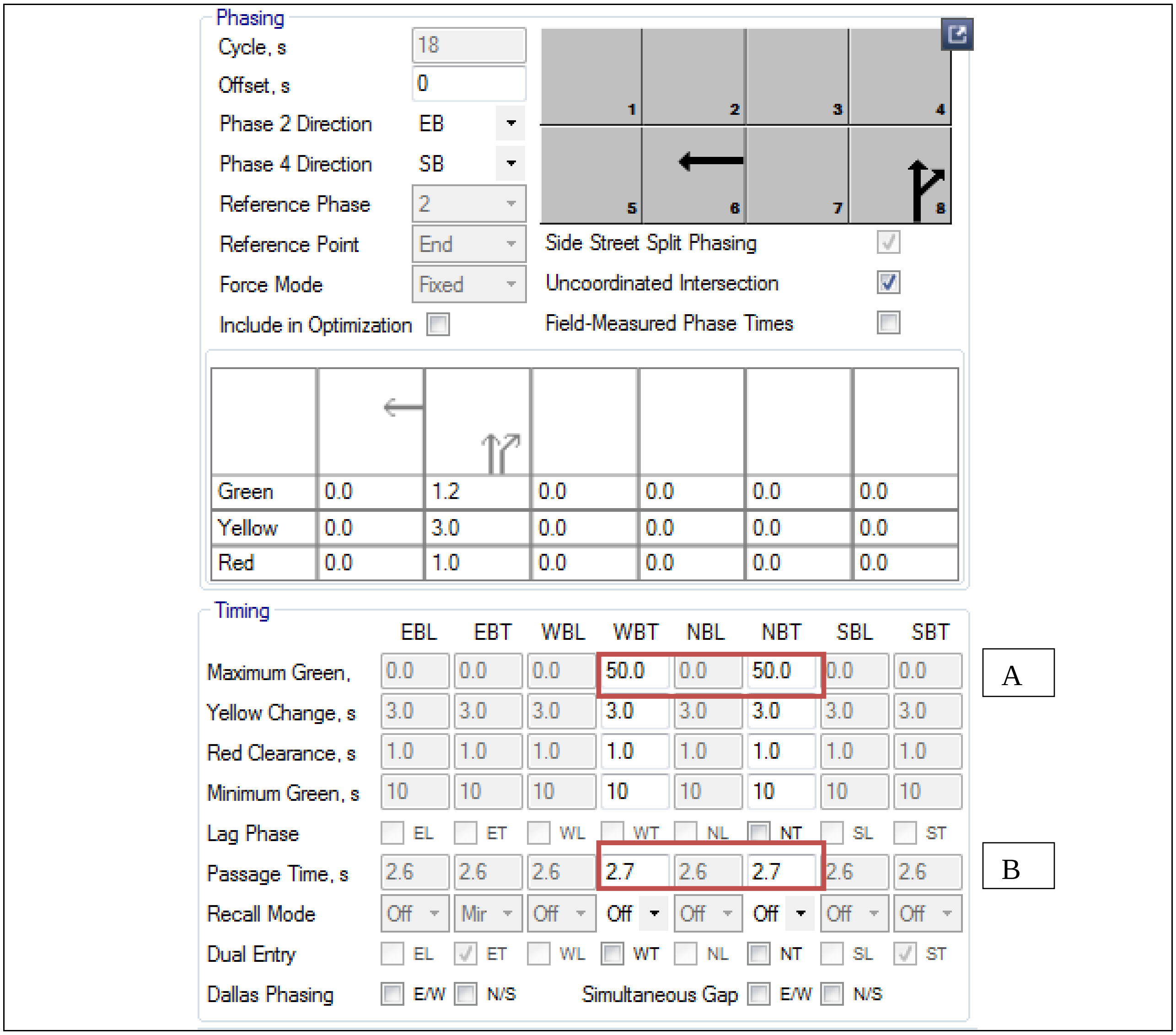


Figure A.30 Phasing and timing for west crossover

A: Maximum green times were set to 50 s for all movements.

B: Passage time (PT) was calculated based on Maximum Allowable Headway (MAH) of 3 s, average lengths of vehicle of small vehicles as 17 ft. and heavy vehicles as 45 ft. and estimating the combined average length (L_V) based on truck percentage, length of detection zone (L_D) of 1 foot and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship:

$$PT = MAH - \frac{L_{V+} L_D}{1.47\ V_A}$$

Phasing and timing (central signalized intersection)

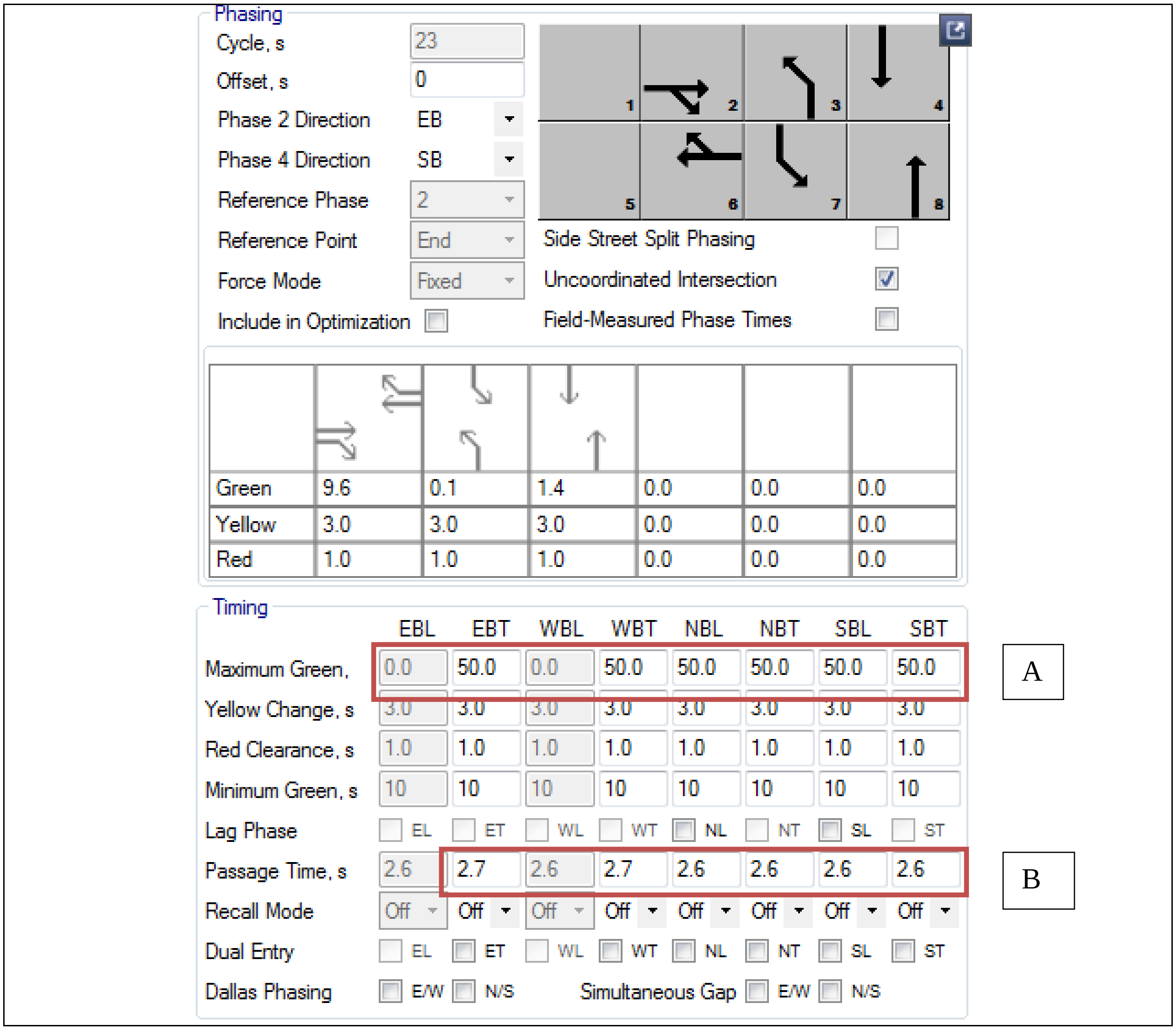


Figure A.31 Phasing and timing for central signalized intersection

A: Maximum green times were set to 50 s for all movements.

B: Passage time (PT) was calculated based on Maximum Allowable Headway (MAH) of 3 s, average lengths of vehicle of small vehicles as 17 ft. and heavy vehicles as 45 ft. and estimating the combined average length (L_V) based on truck percentage, length of detection zone (L_D) of 1 foot and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship:

$$PT = MAH - \frac{L_{V+} L_D}{1.47 \ V_A}$$

Phasing and timing (east crossover)

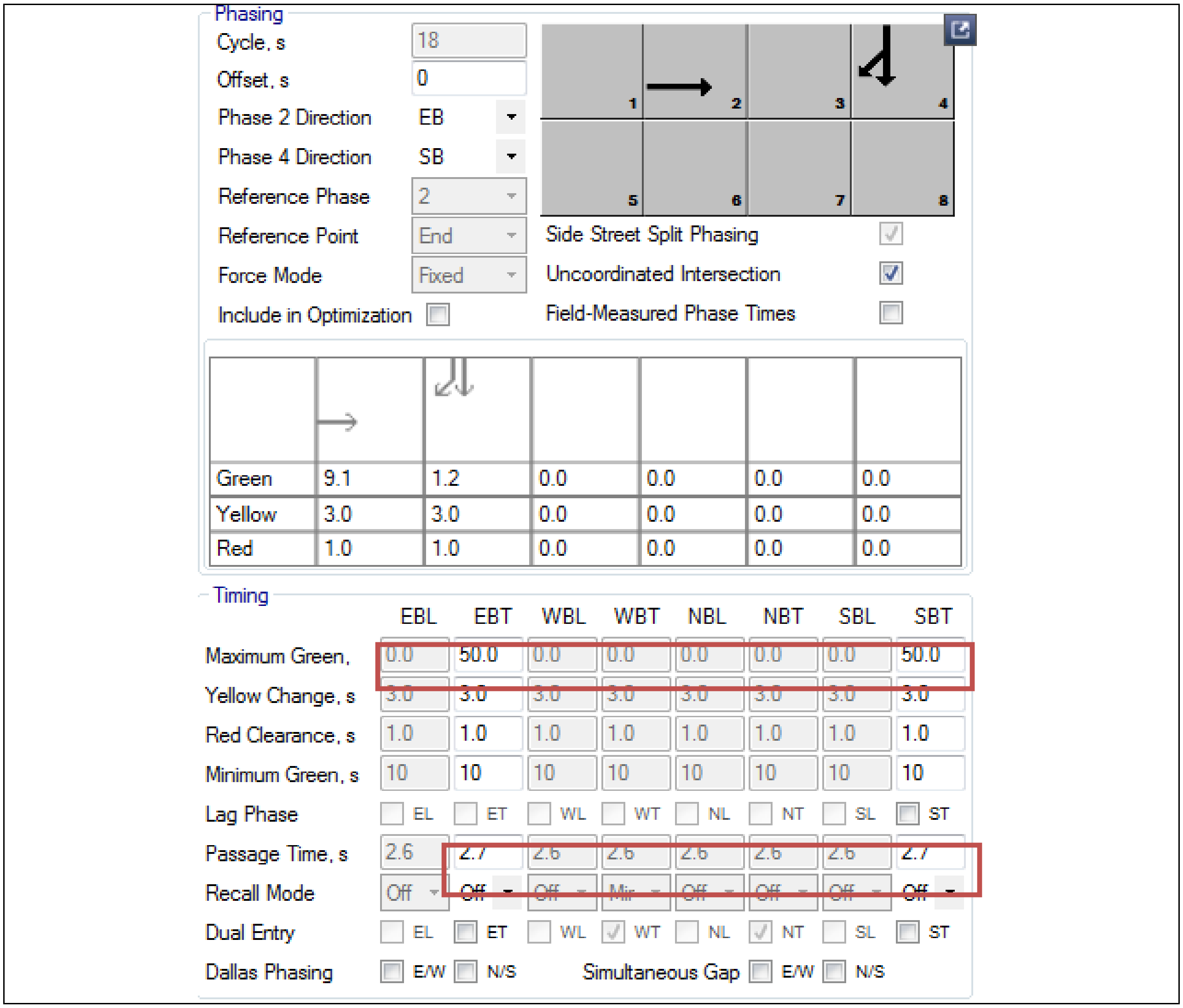


Figure A.32 Phasing and timing for east crossover

A: Maximum green times were set to 50 s for all movements.

B: Passage time (PT) was calculated based on Maximum Allowable Head (MAH) of 3 s, average lengths of small vehicles as 17 ft. and heavy vehicles as 45 ft. and estimating the combined average length (L_V) based on truck percentage, length of detection zone (L_D) of 1 foot and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship:

$$PT = MAH - \frac{L_{V+} L_D}{1.47\, V_A}$$

Segments

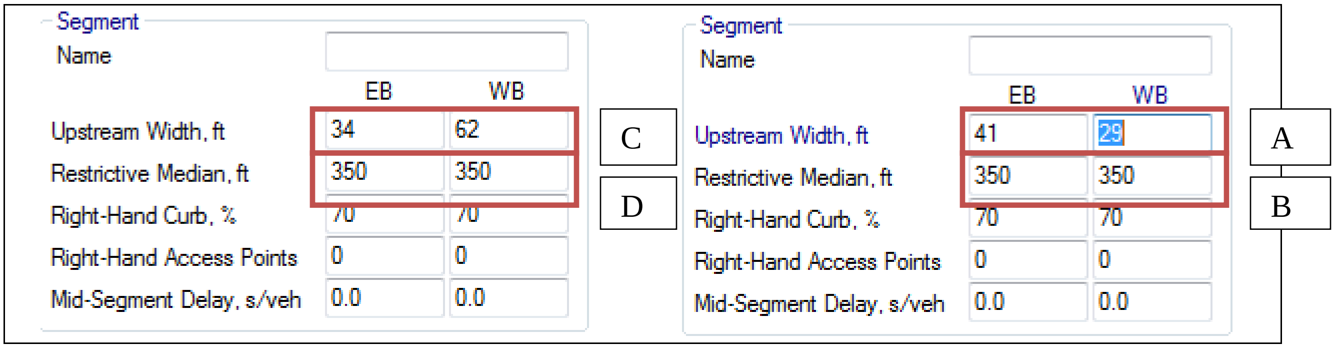


Figure A.33 Upstream segment widths

A: Upstream segment width for segment 1

Segment width for EB = $3 * 12\ (Lane\ width) + 2(Shoulder\ width) + \frac{6}{2}(Median\ width) = 41\ ft.$

Segment width for WB = $2 * 12\ (Lane\ width) + 2(Shoulder\ width) + \frac{6}{2}(Median\ width) = 29$

C: Upstream Width for Segment 2,

Segment width for WB

$= 4 * 12\ (Lane\ width) + 2 * \frac{10}{2}(Median\ widths) + 2 * 2\ (Shoulder\ width) = 62\ ft.$

Segment width for EB $= 2 * 12\ (Lane\ width) + 2 * \frac{10}{2}(Median\ widths) = 34\ ft.$

B and D: Restrictive median lengths = offset of crossover

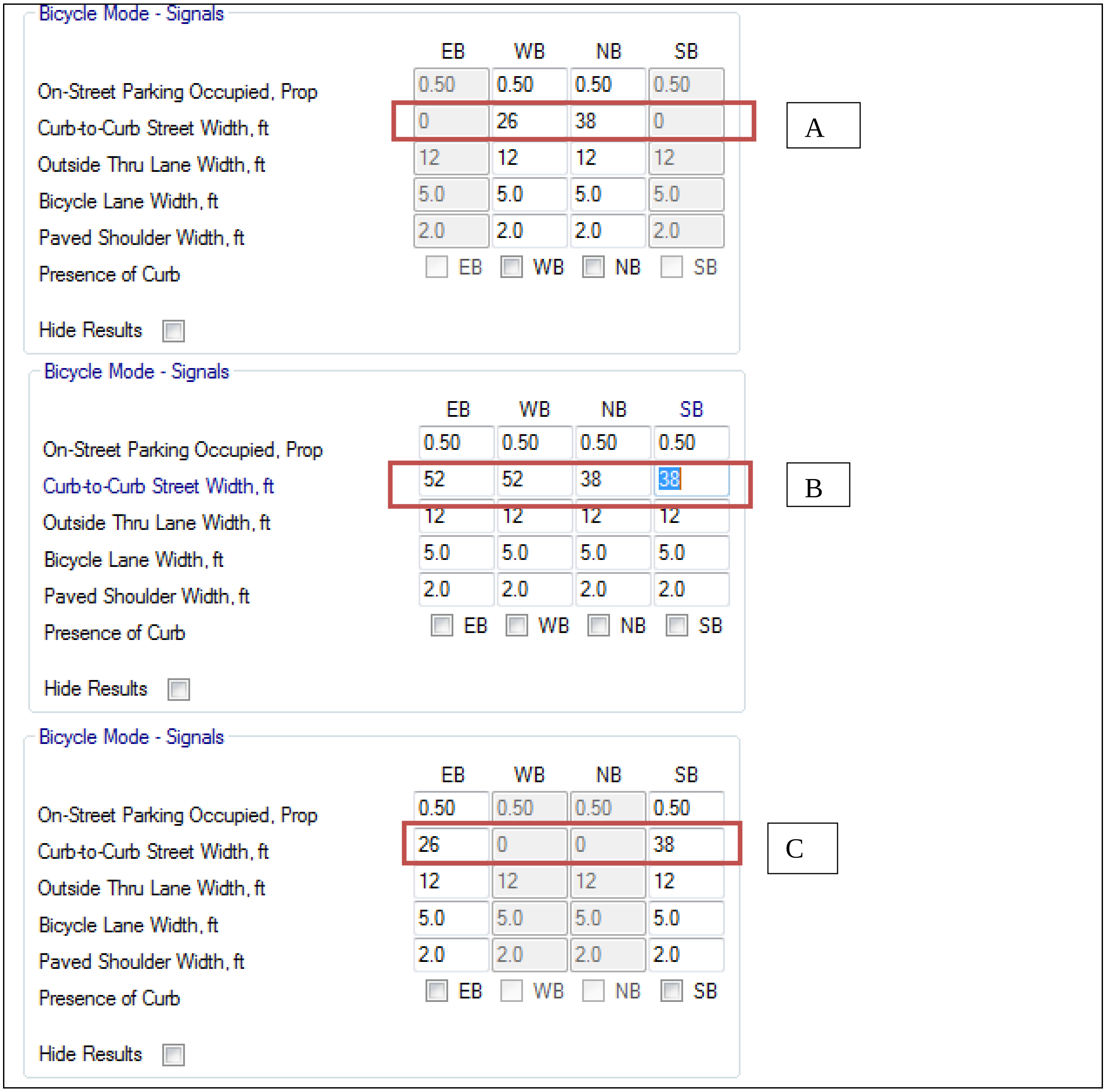


Figure A.34 Curb widths

A: Curb –Curb width for west crossover,
WB width = $2 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 26\ ft.$

NB width = Width of EB at Crossover = $3 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 38\ ft.$

B: Curb-Curb width for center signalized intersection,
EB Width = $4 * 12\ (Lane\ width) + 2 * 2\ (Shoulder\ width) = 52\ ft.$

C: Curb–Curb width for east crossover,
EB width = $2 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 26\ ft.$

SB width = Width of EB at Crossover = $3 * 12\ (Lane\ width) + 1 * 2\ (Shoulder\ width) = 38\ ft.$

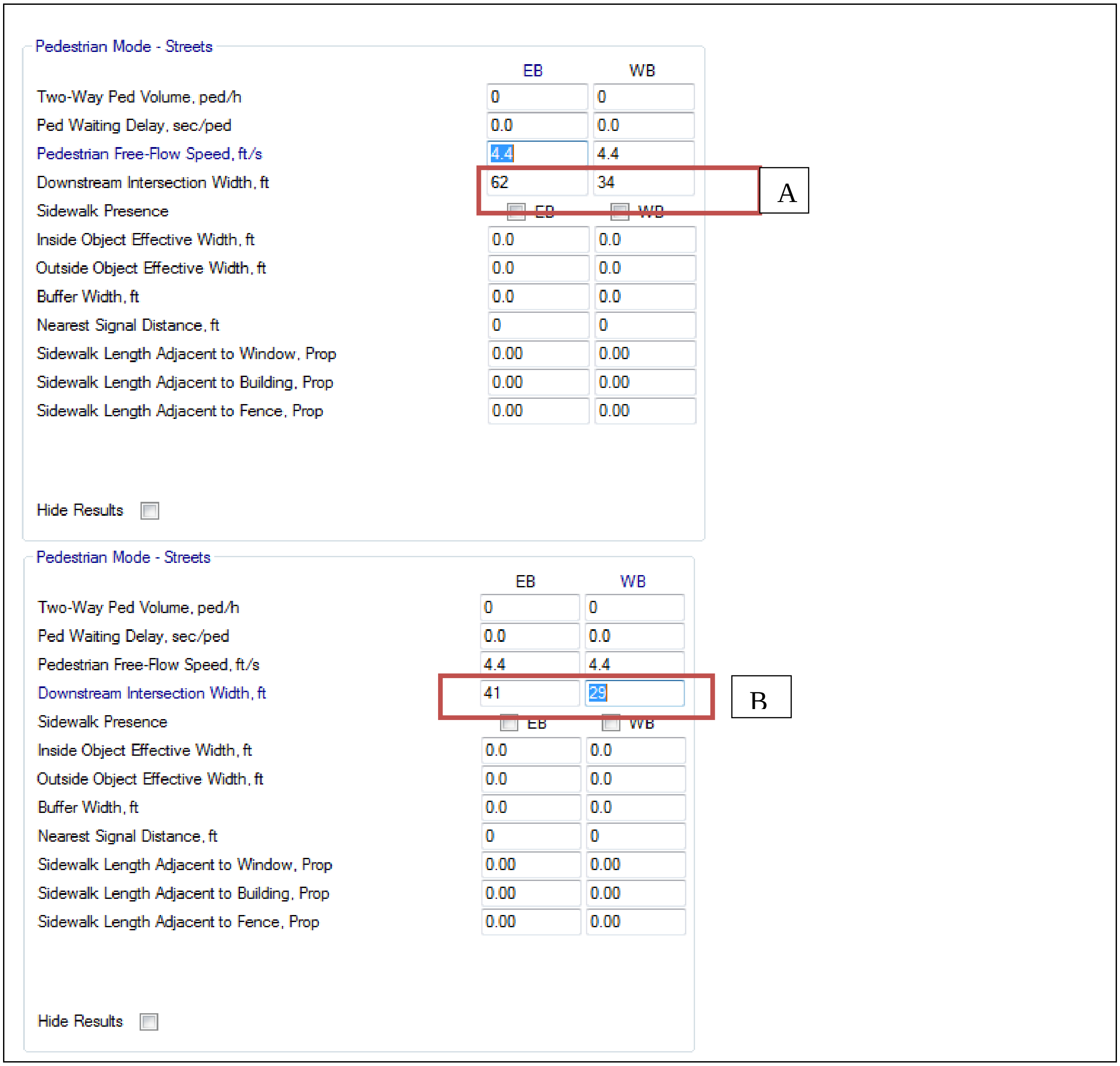


Figure A.35 Downstream segment widths for segment 1 and 2

A: Segment width for segment 1,

Segment width EB

$$= 4 * 12\ (Lane\ width) + 2 * \frac{10}{2}(Median\ width) + 2 * 2\ (Shoulder\ width) = 62 ft.$$

Segment width for segment 1WB $= 2 * 12\ (lane\ width) + 2 * \frac{10}{2}(Median\ width) = 34\ ft.$

B: Segment width for segment 2,

Segment width EB = $3 * 12\ (Lane\ width) + 2(Shoulder\ width) + \frac{6}{2}(Median\ width) = 41\ ft.$

Segment width for segment 2 WB =

$$2 * 12\ (Lane\ width) + 2(Shoulder\ width) + \frac{6}{2}(Median\ width) = 29$$

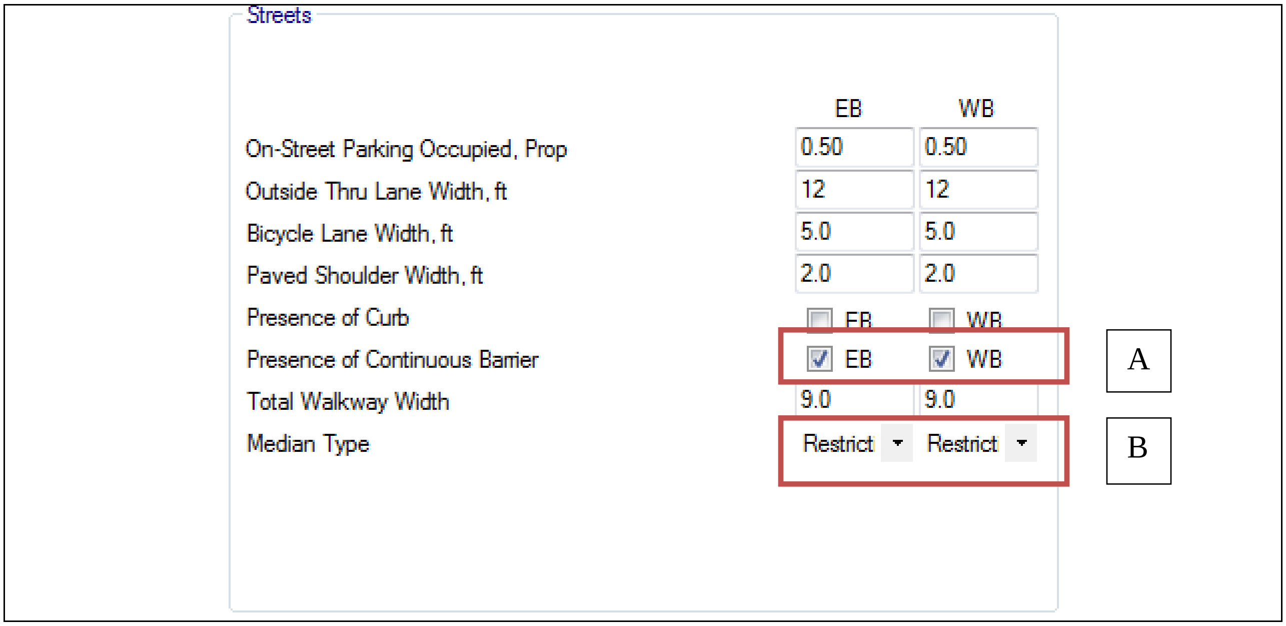


Figure A.36 Median type

A: Presence of continuous barrier was considered.
B: Presence of restrictive median was considered

Case D: Jughandle

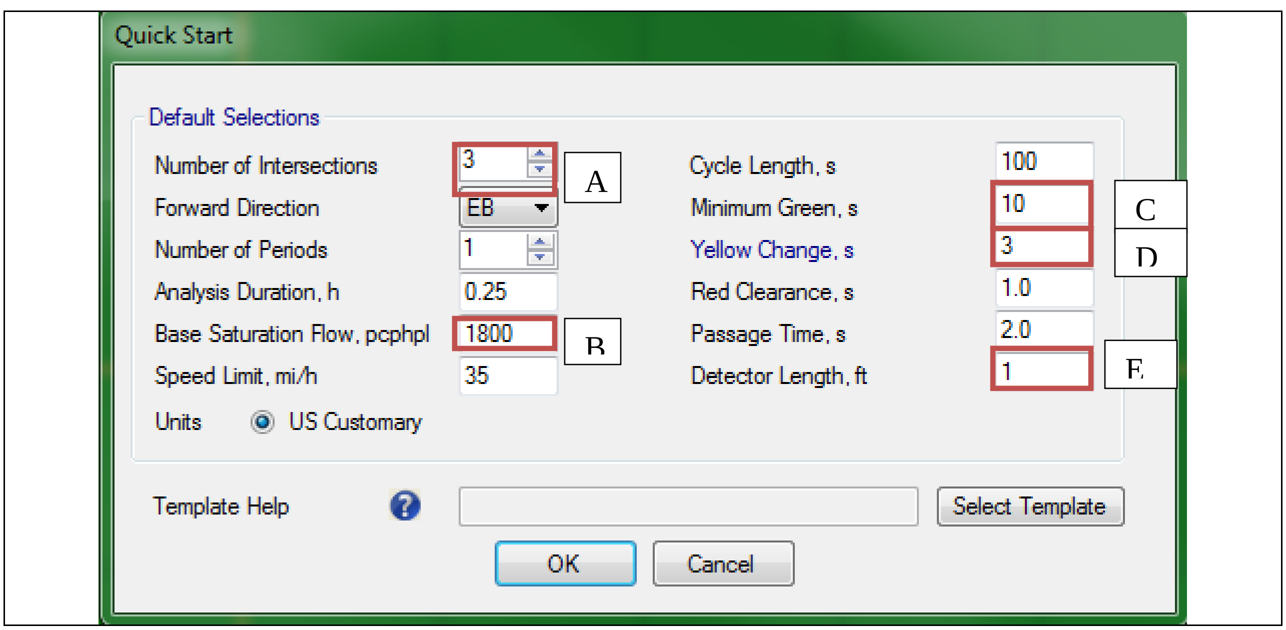
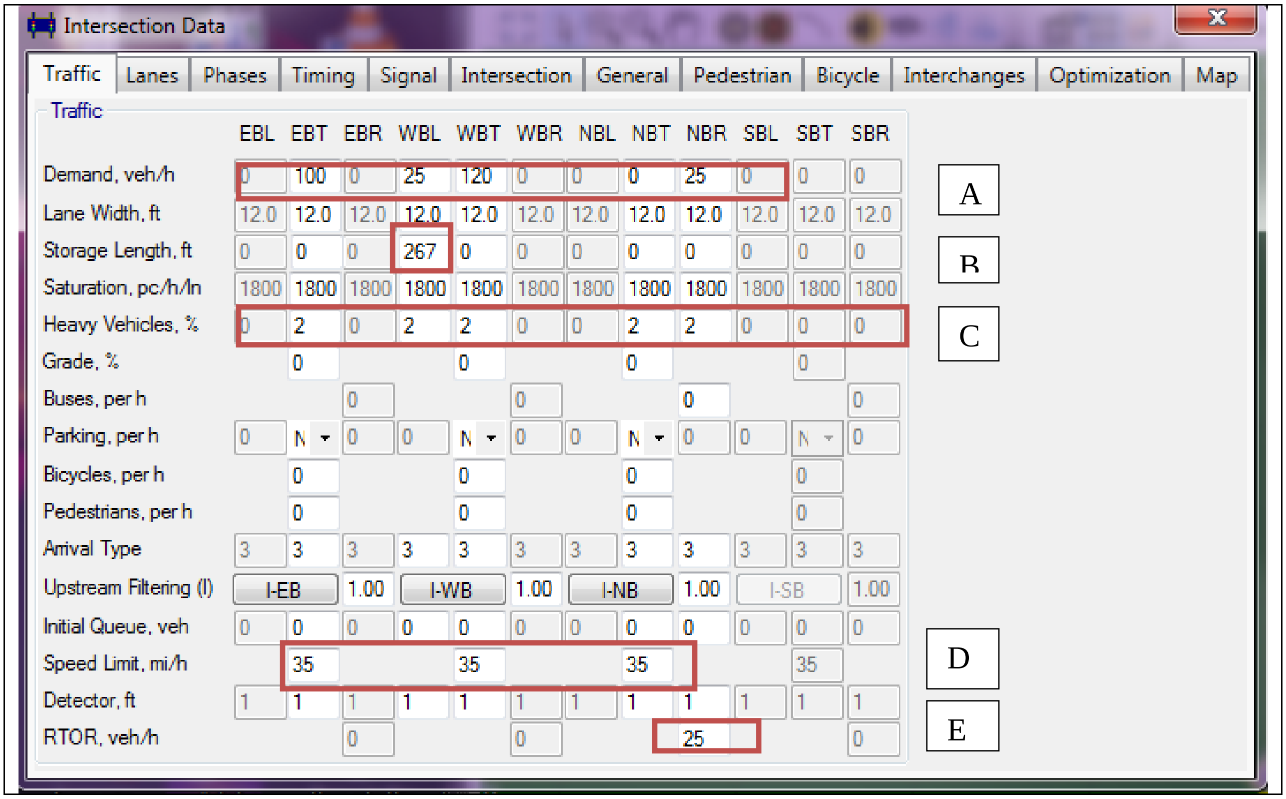


Figure A.37 Quick entry

A: Three intersections include two un-signalized median openings and one signalized core intersection
B: The base saturation flow rate of 1800 pcphpl was considered.
C: The minimum green was changed to 10 s.
D: The yellow change time was changed to 3 s.
All other values were left default.

Traffic (north crossover)

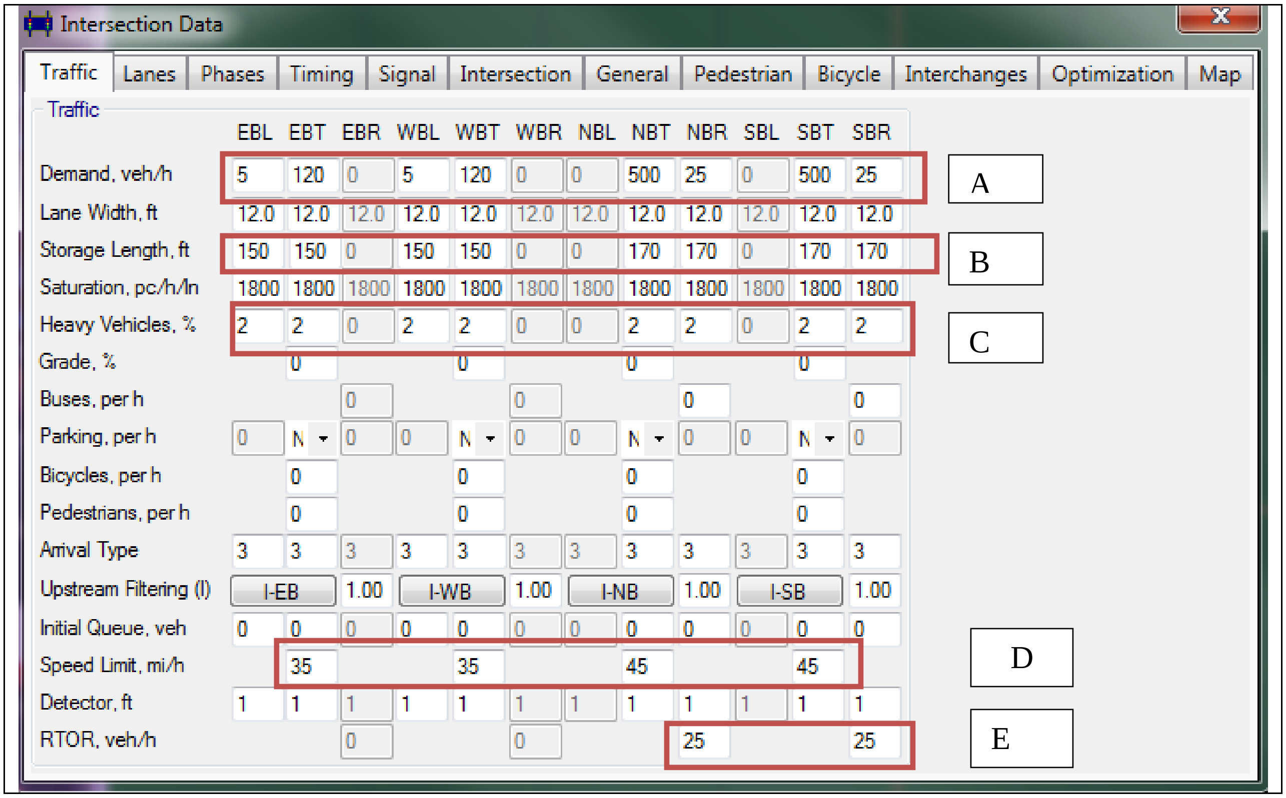


	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Demand, veh/h	0	100	0	25	120	0	0	0	25	0	0	0
Lane Width, ft	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0
Storage Length, ft	0	0	0	267	0	0	0	0	0	0	0	0
Saturation, pc/h/ln	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
Heavy Vehicles, %	0	2	0	2	2	0	0	2	2	0	0	0
Grade, %		0			0			0			0	
Buses, per h			0			0			0			0
Parking, per h	0	N	0	0	N	0	0	N	0	0	N	0
Bicycles, per h		0			0			0			0	
Pedestrians, per h		0			0			0			0	
Arrival Type	3	3	3	3	3	3	3	3	3	3	3	3
Upstream Filtering (I)	I-EB		1.00	I-WB		1.00	I-NB		1.00	I-SB		1.00
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Speed Limit, mi/h		35			35			35			35	
Detector, ft	1	1	1	1	1	1	1	1	1	1	1	1
RTOR, veh/h			0			0			25			0

Figure A.38 Traffic input of north crossover

A: Demands, EBT = Southbound volume at north crossover, WBL = Left turning volume from west leg = Volume at the ramp at the north crossover, WBT = Northbound through volume at north leg (southbound departure), NBR = Volume at the ramp at the north crossover.
B: Storage length,
Storage length for WBL = $half\ circumference\ (length\ of\ ramp) = \pi(\frac{170}{2})$, assuming diameter = 267 ft.
C: Truck percentage
D: Speed (all minors)
E: RTOR volumes =NBR

Traffic (central signalized intersection)



	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Demand, veh/h	5	120	0	5	120	0	0	500	25	0	500	25
Lane Width, ft	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0
Storage Length, ft	150	150	0	150	150	0	0	170	170	0	170	170
Saturation, pc/h/ln	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
Heavy Vehicles, %	2	2	0	2	2	0	0	2	2	0	2	2
Grade, %		0			0			0			0	
Buses, per h			0			0			0			0
Parking, per h	0	N	0	0	N	0	0	N	0	0	N	0
Bicycles, per h		0			0			0			0	
Pedestrians, per h		0			0			0			0	
Arrival Type	3	3	3	3	3	3	3	3	3	3	3	3
Upstream Filtering (I)	I-EB		1.00	I-WB		1.00	I-NB		1.00	I-SB		1.00
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Speed Limit, mi/h		35			35			45			45	
Detector, ft	1	1	1	1	1	1	1	1	1	1	1	1
RTOR, veh/h			0			0			25			25

Figure A.39 Traffic input of central signalized intersection

A: Volumes, EBL= Northbound left volume, EBT= Northbound through volume, WBL= Southbound left volume, WBT= Southbound through volume, NBT = Eastbound through volume, NBR = East bound left volume, SBT = Westbound through volume, SBR = Westbound left volume

B: Storage length,

Storage length for NBT and SBT = North and south offset of crossovers = 150 ft.

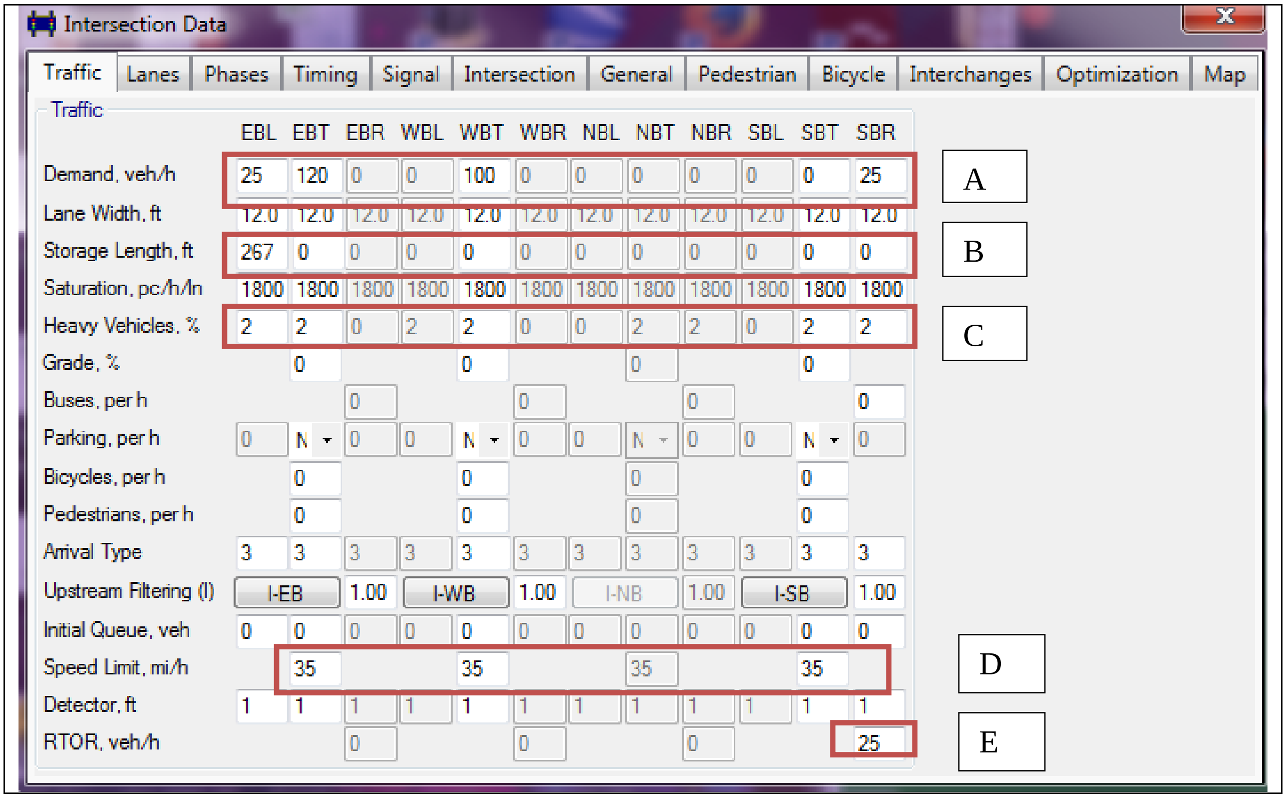
Storage length for EBT and WBT = East and west offset of crossovers = 170 ft.

C: Truck percentage

D: Speeds (Major street: 45 mph, Minor street: 35 mph)

E: NBR RTOR= NBR, SBR RTOR= SBR

Traffic (south crossover)



	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Demand, veh/h	25	120	0	0	100	0	0	0	0	0	0	25
Lane Width, ft	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0
Storage Length, ft	267	0	0	0	0	0	0	0	0	0	0	0
Saturation, pc/h/ln	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800	1800
Heavy Vehicles, %	2	2	0	2	2	0	0	2	2	0	2	2
Grade, %		0			0			0			0	
Buses, per h			0			0			0			0
Parking, per h	0	N	0	0	N	0	0	N	0	0	N	0
Bicycles, per h		0			0			0			0	
Pedestrians, per h		0			0			0			0	
Arrival Type	3	3	3	3	3	3	3	3	3	3	3	3
Upstream Filtering (I)	I-EB		1.00	I-WB		1.00	I-NB		1.00	I-SB		1.00
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Speed Limit, mi/h		35			35			35			35	
Detector, ft	1	1	1	1	1	1	1	1	1	1	1	1
RTOR, veh/h			0			0			0			25

Figure A.40 Traffic input of south crossover

A: Demands, EBL= Left turning volume from east leg = volume at the ramp at the south crossover, EBT= Southbound through volume at south leg (northbound departure), WBT: Northbound volume from south leg at south crossover, SBR= volume at the ramp at south crossover

B: Storage length,

Storage length for EBL=$half\ circumference\ (length\ of\ ramp) = \pi(\frac{170}{2})$, assuming diameter = 267 ft.

C: Truck percentage

D: Speed (all minors)

D: RTOR volumes, RTOR SBR =SBR

Lane Configurations (north crossover)

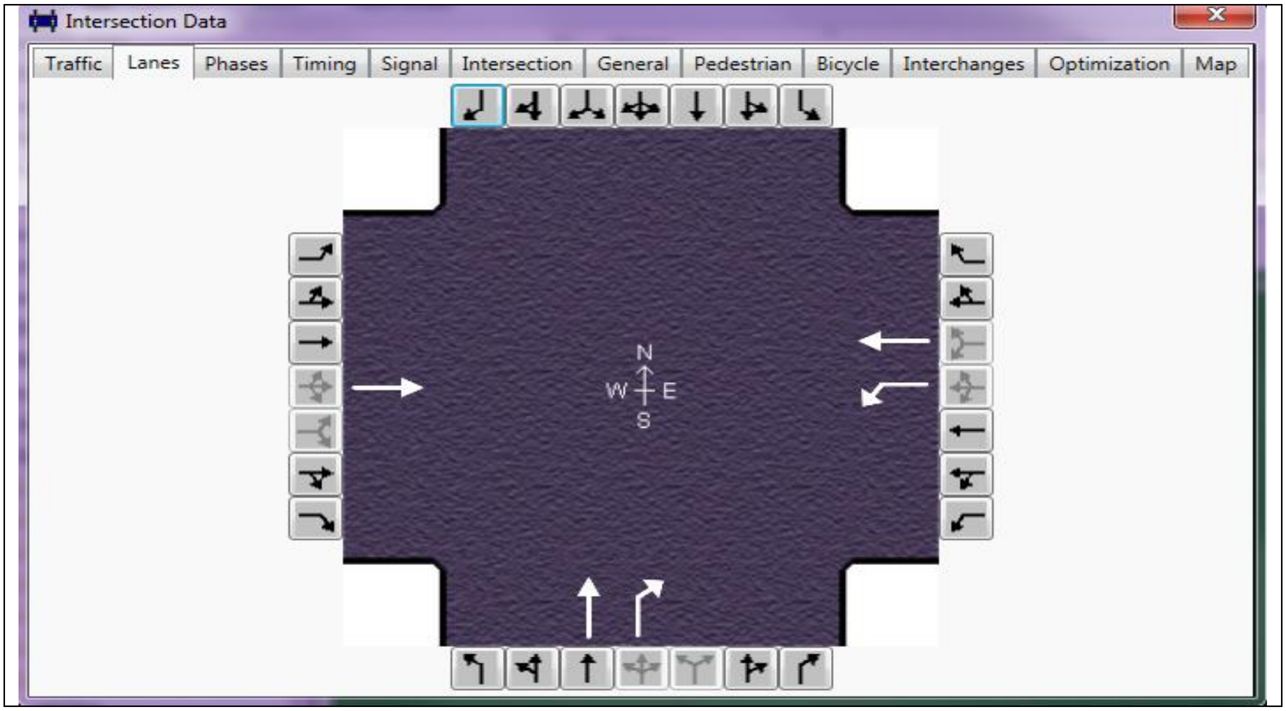


Figure A.41 Lane configuration north crossover

Lane configuration (central signalized intersection)

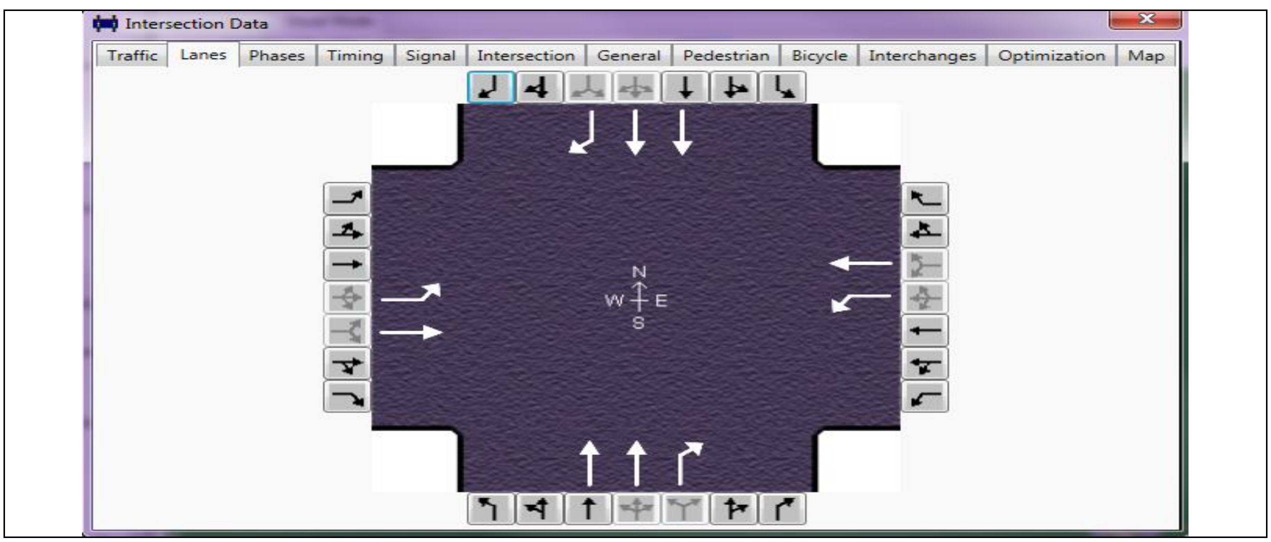


Figure A.42 Lane configuration central signalized intersection

Lane configuration (south crossover)

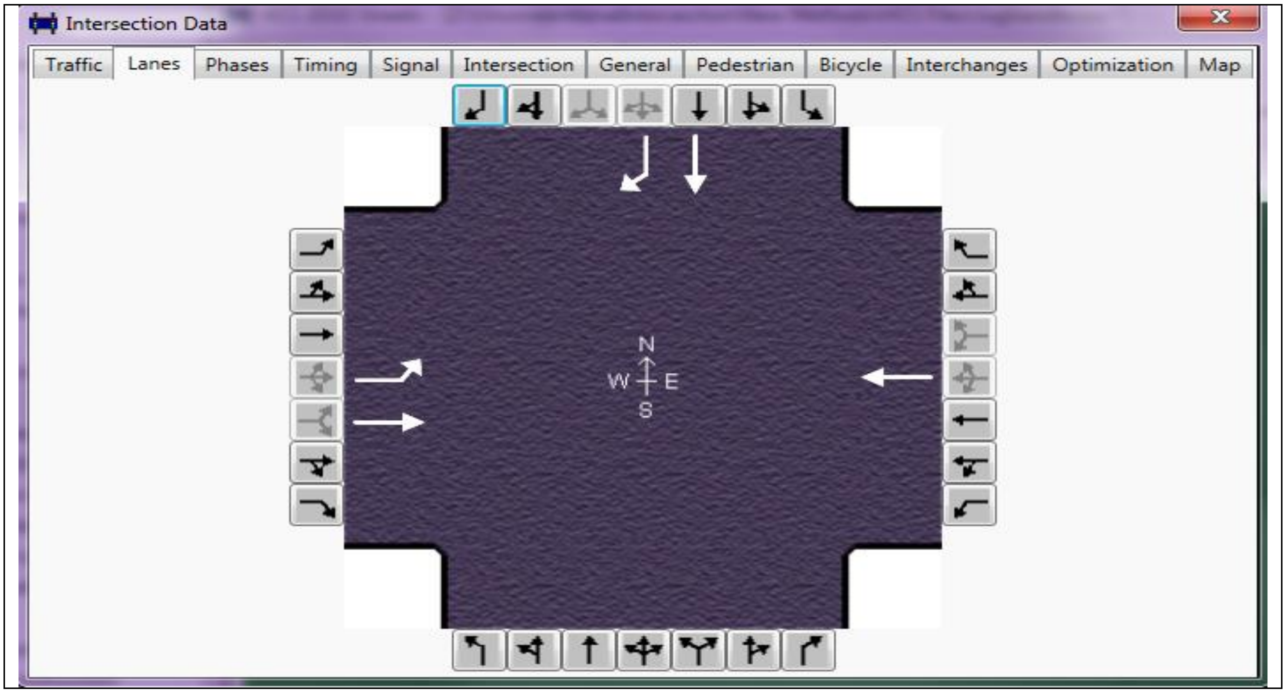


Figure A.43 Lane configuration south crossover

Phasing and timing (north crossover)

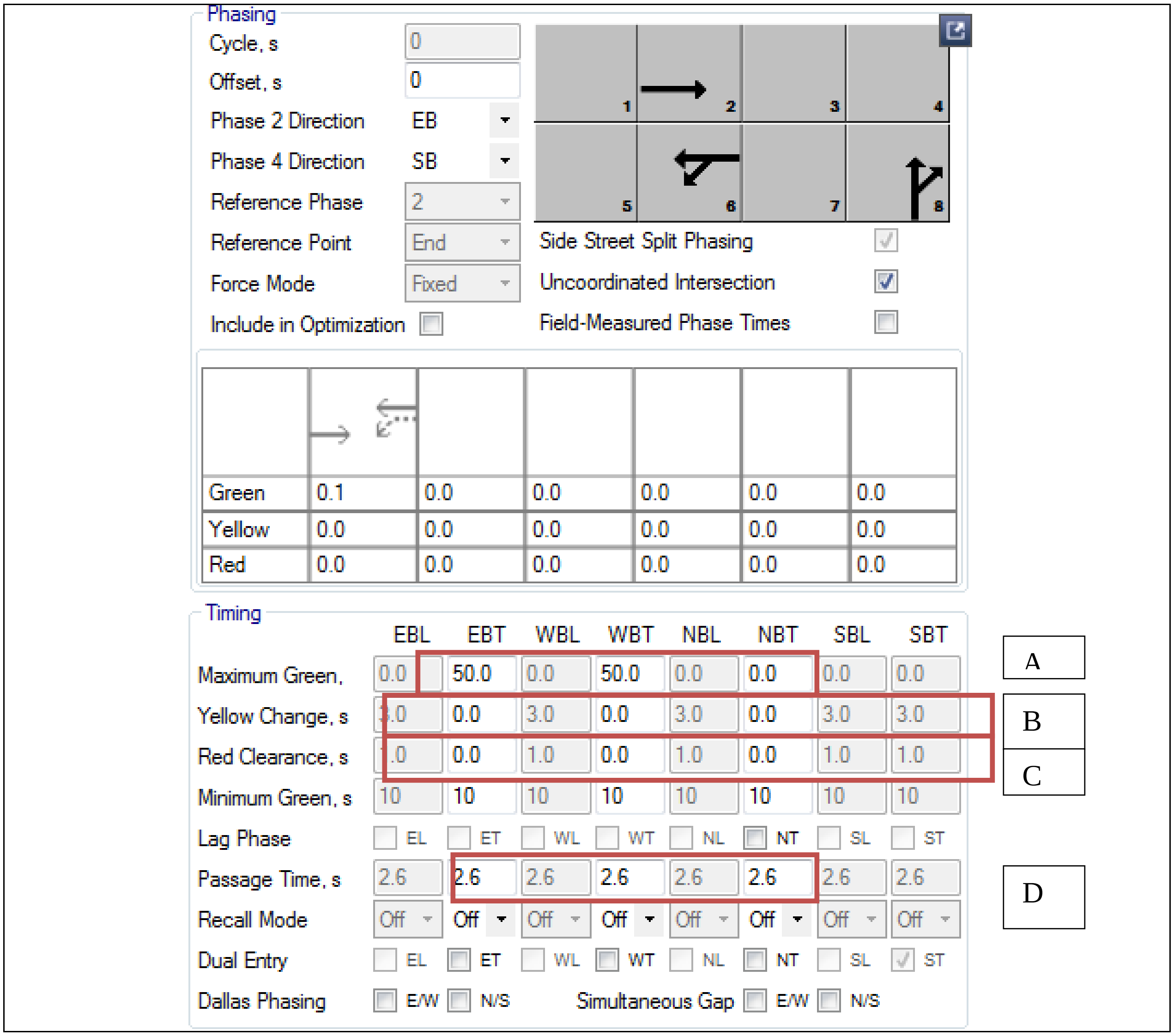


Figure A.44 Phasing of north crossover

A: Maximum green for EBT and WBT were set to 50s and for NBT, it was set to zero.

B and C: Yellow and All red were set to zero.

D: Passage time (PT) was calculated based Maximum Allowable Head (MAH) of 3 s, average lengths of small vehicles as 17 ft. and heavy vehicles as 45 ft. and estimating the combined average length (L_V) based on truck percentage, length of detection zone (L_D) of 1 foot and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship:

$$PT = MAH - \frac{L_{V+}\, L_D}{1.47\ V_A}$$

Phasing and timing (central signalized intersection)

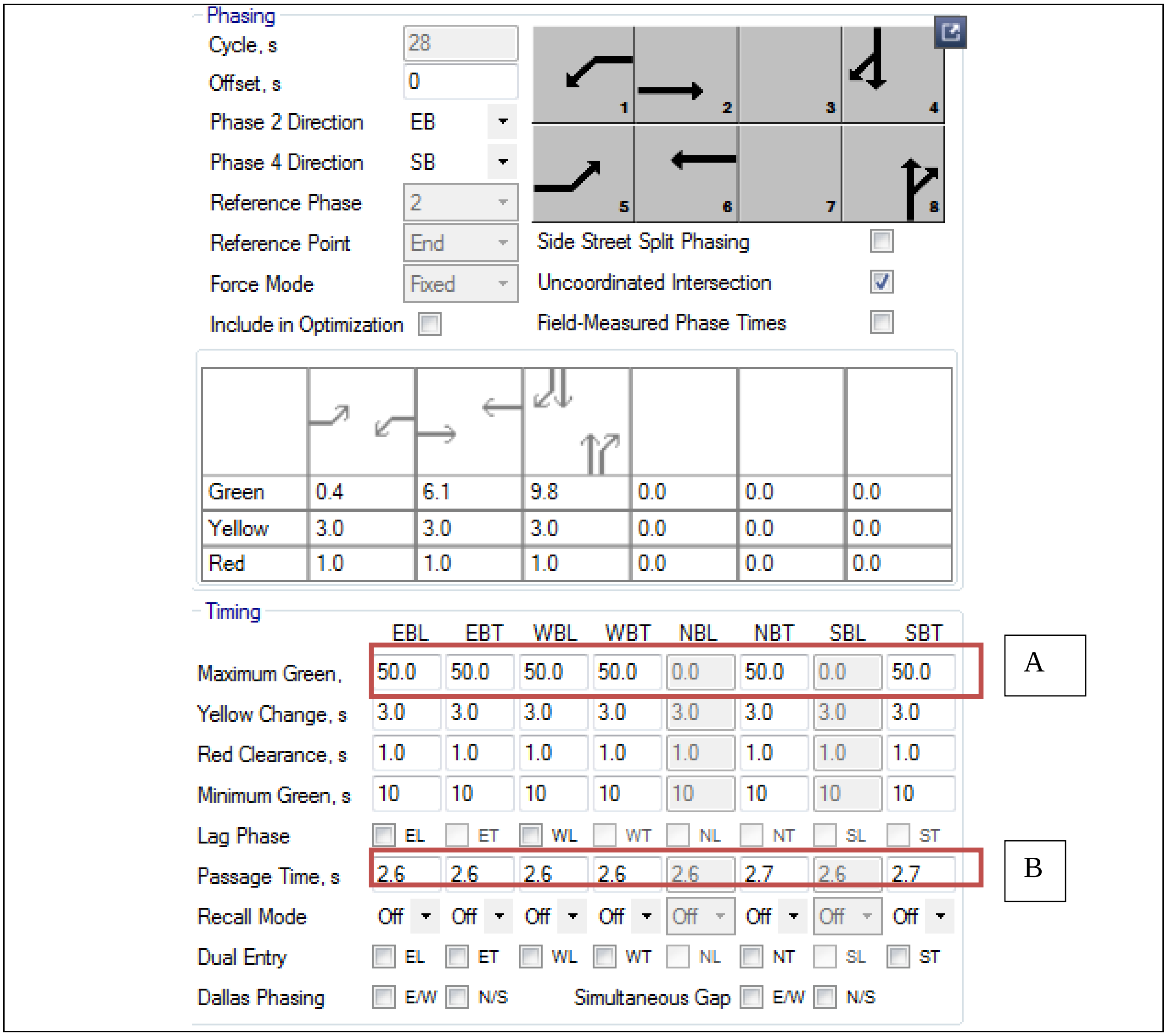


Figure A.45 Phasing for central signalized intersection

A: Maximum Green was set to 50 s for all movements.

B: Passage time (PT) was calculated based on Maximum Allowable Head (MAH) of 3 s, average lengths of vehicle of small vehicles as 17 ft. and heavy vehicles as 45 ft. and estimating the combined average length (L_V) based on truck percentage, length of detection zone (L_D) of 1 foot and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship:

$$PT = MAH - \frac{L_{V+} L_D}{1.47\, V_A}$$

Phasing and timing (south crossover)

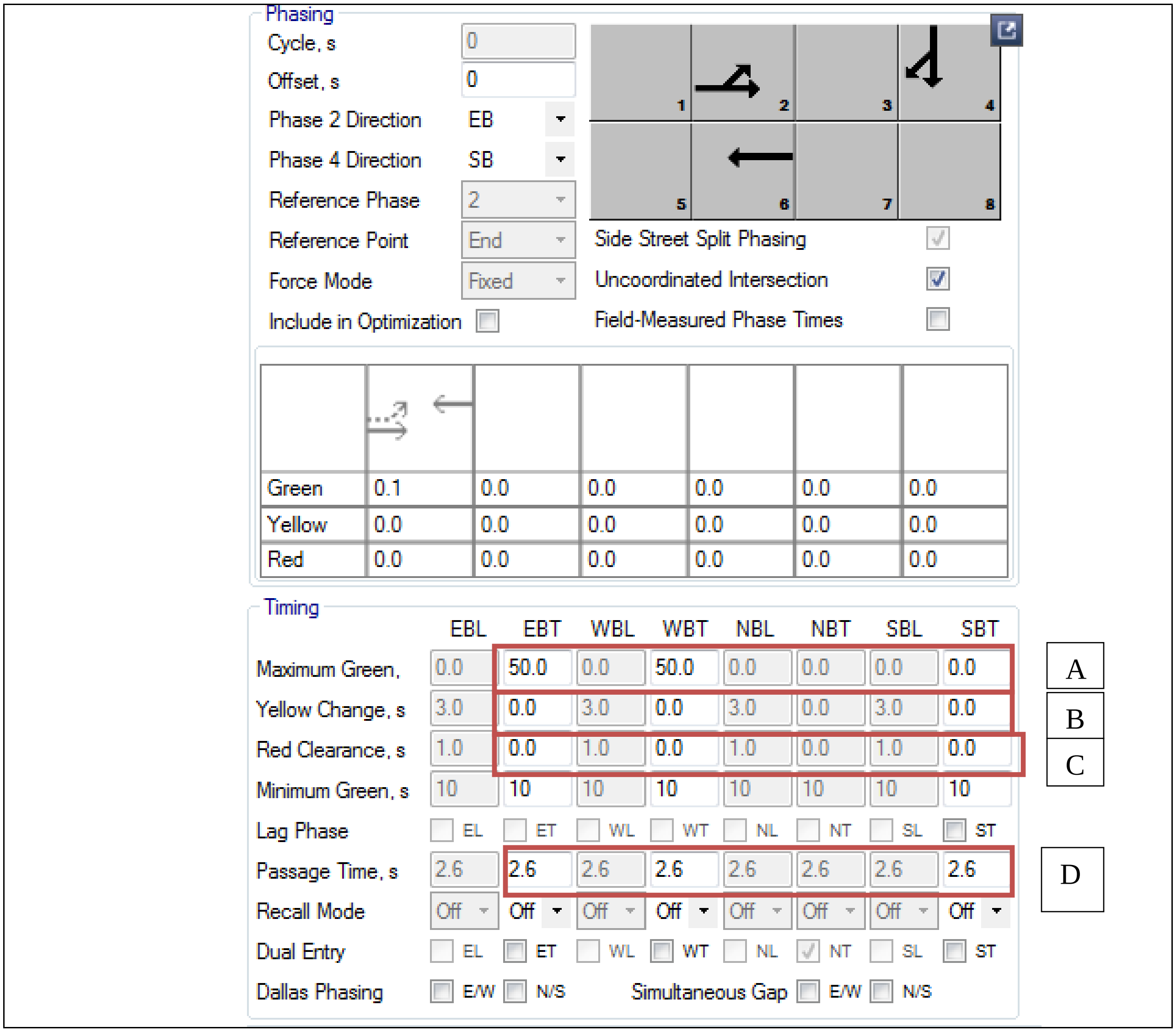


Figure A.46 Phasing for south crossover

A: Maximum Green for EBT and WBT were set to 50 s but for SBT, it was set to zero.

B and C: Yellow and All red time of all movement were set to zero.

D: Passage time (PT) was calculated based on Maximum Allowable Head (MAH) of 3 s, length of vehicle of 20 ft. (L_V), length of detection zone (L_D) of 1 foot and average approach speed of 35 mi/h and 45 mi/h (V_A) using following relationship.

Signal entry

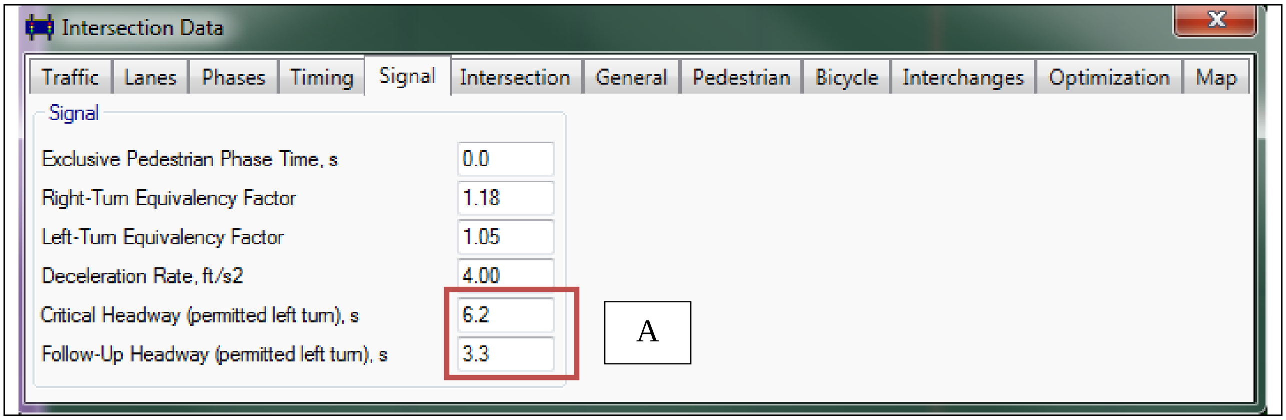


Figure A.47 Signal data for south crossover

A: The critical headway and Follow up headway for permitted left turns were entered that for right turns from the calculation.

Segment

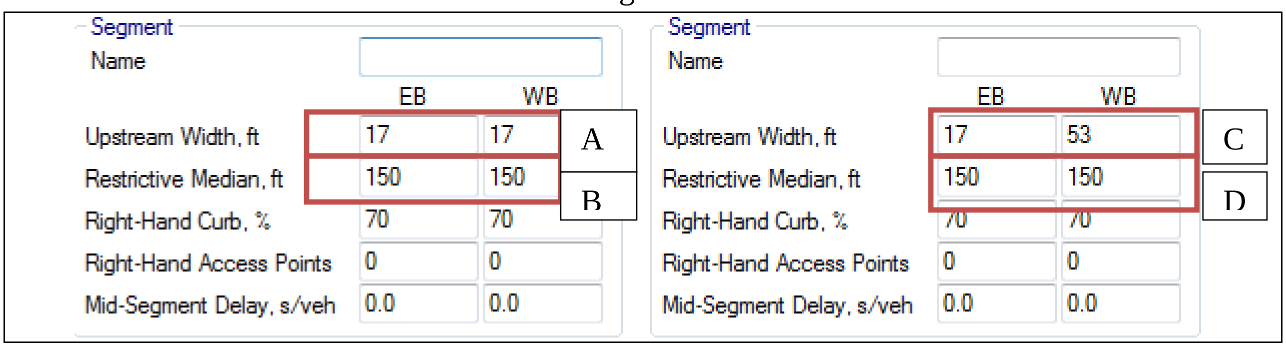


Figure A.48 Upstream segment widths for segment 1and 2

A: Segment1 widths, EB/WB width =

$1 * 12(lane\ width) + 1 * 2\ (shoulder\ width) + \frac{6}{2}\ (half\ median\ widh) = 17$

B and D: Restrictive median = offset of north and south crossovers C: Segment 2 widths, EB width =$1 * 12(lane\ width) + 1 * 2\ (shoulder\ width) + \frac{6}{2}\ (half\ median\ widh) = 17 ft.$

WB width =

$4 * 12(lane\ width) + 1 * 2\ (shoulder\ width) + \frac{6}{2}\ (half\ median\ widh) = 53\ ft.$

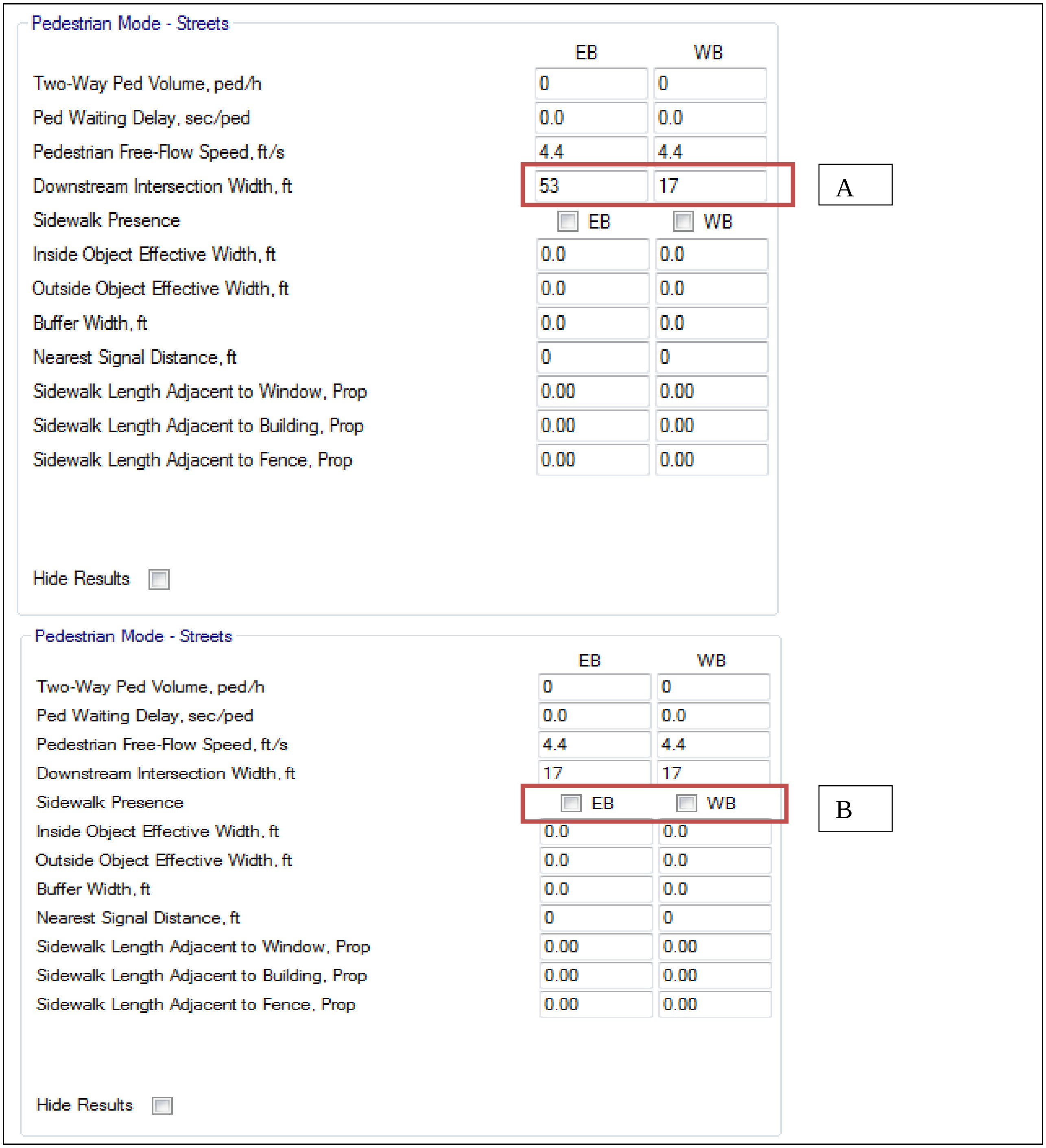


Figure A.49 Downstream intersection width segment 1 and 2

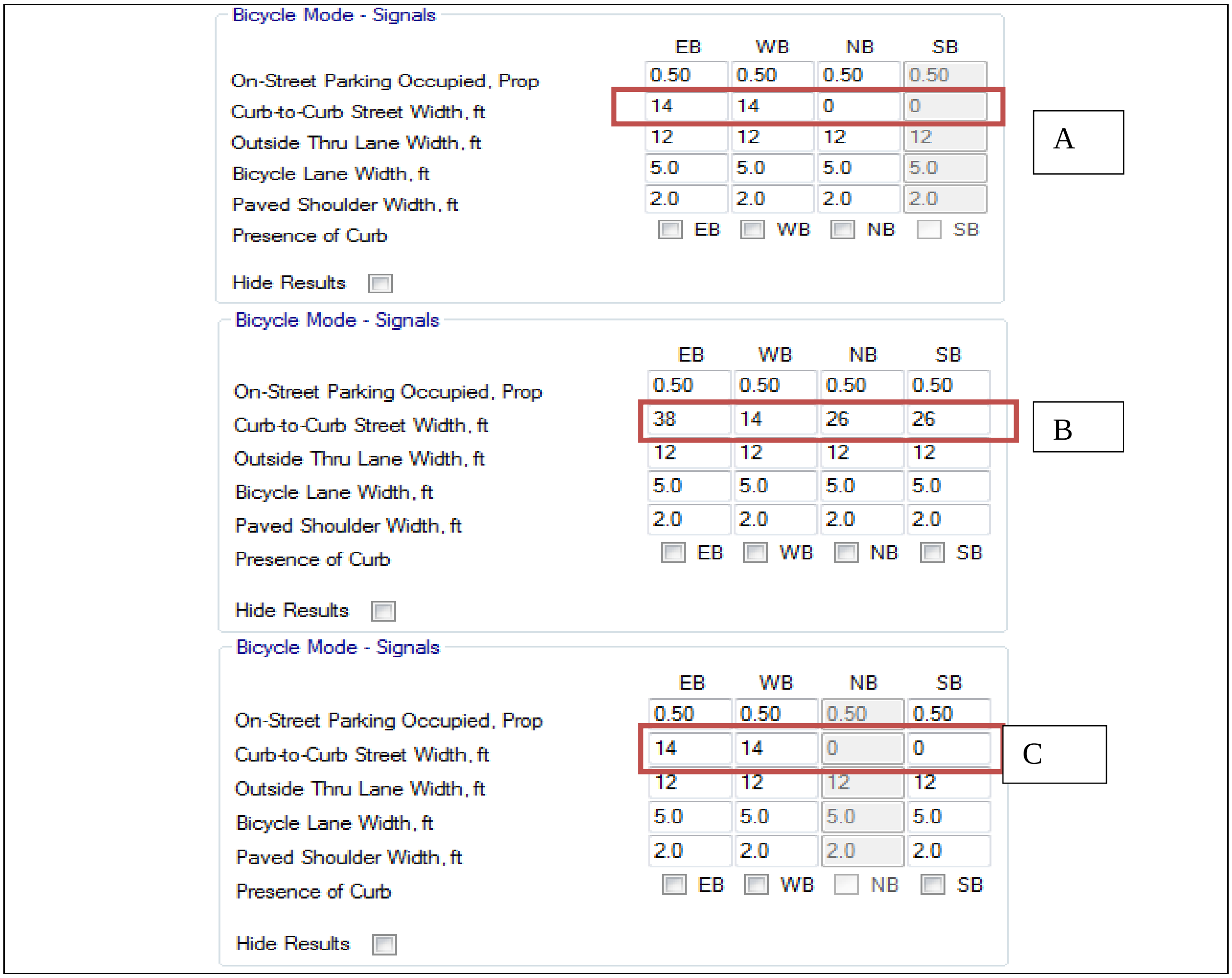
A: Downstream intersection width for segment 1, EB width =
4$* \ 12(lane\ width) + 1 * 2\ (shoulder\ width) + \frac{6}{2}\ (half\ median\ width) = 53\ ft.$

EB width =$1 * 12(lane\ width) + 1 * 2\ (shoulder\ width) + \frac{6}{2}\ (half\ median\ width) = 17 ft.$

A: Downstream intersection width for segment 2

EB/WB width

$=1 * 12(lane\ width) + 1 * 2\ (shoulder\ width) + \frac{6}{2}\ (half\ median\ widh) = 17ft.$



Bicycle Mode - Signals

	EB	WB	NB	SB
On-Street Parking Occupied, Prop	0.50	0.50	0.50	0.50
Curb-to-Curb Street Width, ft	14	14	0	0
Outside Thru Lane Width, ft	12	12	12	12
Bicycle Lane Width, ft	5.0	5.0	5.0	5.0
Paved Shoulder Width, ft	2.0	2.0	2.0	2.0
Presence of Curb	EB	WB	NB	SB

Hide Results

A

Bicycle Mode - Signals

	EB	WB	NB	SB
On-Street Parking Occupied, Prop	0.50	0.50	0.50	0.50
Curb-to-Curb Street Width, ft	38	14	26	26
Outside Thru Lane Width, ft	12	12	12	12
Bicycle Lane Width, ft	5.0	5.0	5.0	5.0
Paved Shoulder Width, ft	2.0	2.0	2.0	2.0
Presence of Curb	EB	WB	NB	SB

Hide Results

B

Bicycle Mode - Signals

	EB	WB	NB	SB
On-Street Parking Occupied, Prop	0.50	0.50	0.50	0.50
Curb-to-Curb Street Width, ft	14	14	0	0
Outside Thru Lane Width, ft	12	12	12	12
Bicycle Lane Width, ft	5.0	5.0	5.0	5.0
Paved Shoulder Width, ft	2.0	2.0	2.0	2.0
Presence of Curb	EB	WB	NB	SB

Hide Results

C

Figure A.50 Curb to curb widths for north crossover, central signalized intersection and south crossover

A: Curb to Curb width for north crossover, EB/WB width =

$1 * 12(lane\ width) + 1 * 2\ (shoulder\ width) = 14ft.$

B: Curb to Curb width for central signalized intersection

EB width = $3 * 12(lane\ width) + 1 * 2\ (shoulder\ width) = 38ft.$

WB width = $1 * 12(lane\ width) + 1 * 2\ (shoulder\ width) = 14ft.$

C: Curb to Curb width for south crossover intersection

EB/WB width = $1 * 12(lane\ width) + 1 * 2\ (shoulder\ width) = 14ft.$

Appendix B DAC Plots and Contour Plots

Appendix B is included separately as a supplement to report.

Appendix C Example Calculation of Delay and Example Use of DAC

C.1 Calculation of Delay

C.1.1 Example Study Intersection: Standard Signalized Intersection

Bi-directional volume (Major street) = 2400 vph,

Uni-directional volume (Minor street) = 250 vph, Bi-directional volume (Minor street) =500 Vph

Truck Percentage =10%, Left Turn Percentage = 15%, Balance Factor =0.7

OD Volumes (vph): Directions are referred from figure A1 of Appendix A

The calculated O-D volumes:

1-3	1-2	1-4	2-4	2-1	2-3	4-1	4-3	4-2	3-2	3-4	3-1
252	1428	0	108	612	0	13	238	0	13	238	0

The following relationship was used to calculate O-D from bidirectional volumes:

Volume (Left turn) = Bi-directional Volume *BF*LTP

Volume (Through) = Bi-directional Volume * BF * (1-LTP)

Control delay obtained from HCS:

1-3	1-2	1-4	2-4	2-1	2-3	4-1	4-3	4-2	3-2	3-4	3-1
39.1	27.2	0	38.2	17.7	0	41.4	39.3	0	41.4	39.3	0

$$Total\ \ Average\ Intersection\ delay\ \left(\frac{sec}{veh}\right) = \frac{\sum Individual\ \ movement\ delay\left(\frac{sec}{veh}\right) \times Volume\ of\ the\ movements\ (vph)}{\sum Volme\ \ of\ the\ movement\ \ (vph)}$$

= 28.75121 sec/veh

C.1.2 Example Study intersection: jughandle

Bi-directional volume (Major street) = 2400 vph,

Uni-directional volume (Minor street) = 250 vph, Bi-directional volume (Minor street)=500 Vph

Truck Percentage =10%, Left Turn Percentage = 15%, Balance Factor =0.7

OD Volumes (vph): Directions are referred from figure A.1 of Appendix A

The calculated O-D volumes:

1-3	1-2	1-4	2-4	2-1	2-3	4-1	4-3	4-2	3-2	3-4	3-1
252	1428	0	108	612	0	13	238	0	13	238	0

The following relationship was used to calculate O-D from bidirectional volumes:

Volume (Left turn) = Bi-directional volume *BF*LTP

Volume (Through) = Bi-directional volume * BF * (1-LTP)

Critical headway,

$$t_{c,x} = t_{c,base} + t_{c,HV} P_{HV}$$

= 6.3 s

Follow up headway,

$$t_{f,x} = t_{f,base} + t_{f,HV} P_{HV}$$

= 3.4 s

Capacity of north crossover,

$$C_{p,x} = \frac{V_{c,x} e^{-\frac{v_{c,x} t_{c,x}}{3600}}}{1 - e^{-\frac{v_{c,x} t_{f,x}}{3600}}} = 766.51\ vph$$

Similarly, Capacity of south crossover,

$$C_{p,x} = \frac{V_{c,x} e^{-\frac{v_{c,x} t_{c,x}}{3600}}}{1 - e^{-\frac{v_{c,x} t_{f,x}}{3600}}} = 766.51\ vph$$

Now, control delay (sec/veh) for each movement obtained from HCS:

3-2	3-4	4-1	4-3	1-2	2-1
47.6	29.2	47.6	42.6	52.3	14.2

The delay (sec/veh) for movement 1-3 (LT Delay) can be calculated as follows:

$$Delay\ (1-2) + \frac{\pi \times \frac{r}{2}}{V_{ramp} \times 1.47} + \frac{Offset\ \ of\ east\ crossover}{V_{Major\ Street} \times 1.47} + \frac{Offset\ of\ south\ croosover}{V_{Minor\ Street} \times 1.47} + \left[\frac{\lambda}{\mu \times (\mu - \lambda)} + \frac{1}{\mu}\right] \times 3$$

= 67.81sec/veh

(r = half of offset of east crossover, μ=Capacity of south crossover, λ= Volume 1-3)

Similarly, delay (sec/veh) for movement 2-4 can be calculated by same method by using same equation using r = half of offset of east crossover, μ = Capacity of north crossover and λ= volume 2-4.

Delay for 2-4 (LT Delay) =28.18 sec/veh

Control delay from HCS for central signalized intersection:

1-3	1-2	1-4	2-4	2-1	2-3	4-1	4-3	4-2	3-2	3-4	3-1
67.81	52.3	0	28.18	14.2	0	47.6	42.6	0	47.6	29.2	0

$$Total\ \ Average\ Intersection\ delay\ \left(\frac{sec}{veh}\right) = \frac{\sum Individual\ movement\ delay\left(\frac{sec}{veh}\right) \times Volume\ of\ the\ movements\ (vph)}{\sum Volme\ of\ the\ movement\ \ (vph)}$$

= 41.98 sec/veh

C.1.3 Example Study intersection: MUT

Bi-directional volume (Major street) = 2400 vph,

Uni-directional volume (Minor street) = 250 vph, Bi-directional volume (Minor street) =500 Vph

Truck Percentage =10%, Left Turn Percentage = 15%, Balance Factor =0.7

OD Volumes (vph): Directions are referred from figure A1 of Appendix A

The calculated O-D volumes:

1-3	1-2	1-4	2-4	2-1	2-3	4-1	4-3	4-2	3-2	3-4	3-1
252	1428	0	108	612	0	13	238	0	13	238	0

The following relationship was used to calculate O-D from bidirectional volumes:

Volume (Left turn) = Bi-directional volume *BF*LTP

Volume (Through) = Bi-directional volume * BF * (1-LTP)

The critical headway,

$t_{c,x} = t_{c,base} + t_{c,HV}P_{HV}$

= 7.1s

The follow-up headway,

$t_{f,x} = t_{f,base} + t_{f,HV}P_{HV}$

=3.2s

Capacity of east median opening,

$$C_{p,x} = \frac{V_{c,x}\, e^{-\frac{v_{c,x} t_{c,x}}{3600}}}{1 - e^{-\frac{v_{c,x} t_{f,x}}{3600}}}$$

=368.16 vph

Similarly, capacity of west median opening

$$C_{p,x} = \frac{V_{c,x}\, e^{-\frac{v_{c,x} t_{c,x}}{3600}}}{1 - e^{-\frac{v_{c,x} t_{f,x}}{3600}}}$$

=78.86 vph

Control delay obtained from HCS:

1-2	2-1	4-3	3-4
8.9	4.6	18.1	18.1

Delay of movement (1-3) (LT Delay) (sec/veh) can be calculated by followings:

$$Delay\ (1-2) + 2 * \frac{Offest\ of\ median\ opening}{1.47 * V_{Major\ Street}} + \left[\frac{\lambda}{\mu \times (\mu - \lambda)} + \frac{1}{\mu}\right] \times 3600$$

= 43.42 sec/veh

Here, λ = Total U-turning volume on east median opening= Volume (1-3) + Volume (4-1)

μ = Capacity of east median opening

Delay of movement (2-4) (LT Delay) (sec/veh) can be calculated by followings:

$$= Delay\ (2-1) + 2 * \frac{Offest\ of\ median\ opening}{1.47 * V_{Major\ Street}} + \left[\frac{\lambda}{\mu \times (\mu - \lambda)} + \frac{1}{\mu}\right] \times 3600$$

Here, λ = Total U-turning volume on west median opening= Volume (2-4) + Volume (3-2) = 121 vph

μ = Capacity of east median opening =78.86 vph

Since λ > μ, Median opening will fail

Delay of movement (4-1) (LT delay) sec/veh can be calculated as follows:

$$= Delay\ (4-3) + 2 * \frac{Offest\ of\ median\ opening}{1.47 * V_{Major\,Street}} + \left[\frac{\lambda}{\mu \times (\mu - \lambda)} + \frac{1}{\mu}\right] \times 3600$$

= 72.95 sec/veh

Delay of movement (3-2) can't be calculated because λ > μ, west median opening will fail.

The total average intersection delay can be found out from similar method as explained for jughandle. However, due to the failure of west median opening, the MUT system is considered not applicable for this particular condition.

C.1.3 Example Study intersection: CFI

Bi-directional volume (Major street) = 2400 vph,

Uni-directional volume (Minor street) = 250 vph, Bi-Directional Volume (Minor street)=500 Vph

Truck Percentage =10%, Left Turn Percentage = 15%, Balance Factor =0.7

OD Volumes (vph): Directions are referred from figure A1 of Appendix A

The calculated O-D volumes:

1-3	1-2	1-4	2-4	2-1	2-3	4-1	4-3	4-2	3-2	3-4	3-1
252	1428	0	108	612	0	13	238	0	13	238	0

The following relationship was used to calculate O-D from bidirectional volumes:

Volume (Left turn) = Bi-directional volume *BF*LTP

Volume (Through) = Bi-directional volume * BF * (1-LTP)

While coding in HCS network, the following volumes were considered:

CFI -A (West Crossover)				CFI Central Signalized Intersection								CFI-B (East Crossover)			
WBT	NBT	NBR	NBR-RTOR	EBT	EBR	WBT	WBR	NBL	NBT	SBL	SBT	EBT	SBT	SBR	SBR-RTOR
625	0	1680	1428	1428	252	612	108	13	238	13	238	1441	0	720	612

For CFI-A (West Crossover),

Volume (WBT) = Volume (2-1) + Volume (4-1) =625vph

NBR= Volume (1-2) + Volume (1-3) = 1680 vph

NBR- RTOR = Volume (1-2) = 1428 vph

For CFI Central Signalized Intersection

EBT = Volume (1-2) =1428 vph

EBR =Volume (1-3) = 252 vph

WBT = Volume (2-1) = 612 vph

WBR = Volume (2-4) = 108 vph

NBL = Volume (4-1) = 13 vph

NBT= Volume (4-3) = 238 vph

SBL = Volume (4-1) = 13 vph

SBT = Volume (4-3) = 238 vph

For CFI-A (East Crossover),

Volume (EBT) = Volume (1-2) + Volume (3-2) = 1441 vph

SBR= Volume (2-1) + Volume (2-4) = 720 vph

SBR- RTOR = Volume (2-1) = 612 vph

Control delay obtained from HCS:

CFI -A (West Crossover)				CFI Central Signalized Intersection								CFI-B (East Crossover)			
WBT	NBT	NBR	NBR-RTOR	EBT	EBR	WBT	WBR	NBL	NBT	SBL	SBT	EBT	SBT	SBR	SBR-RTOR
6.7	0	7.9	0	11.6	6.8	7	6	25	21.2	25	21.2	6.9	0	11.7	0

The calculated O-D Delay:

1-3	1-2	1-4	2-4	2-1	2-3	4-1	4-3	4-2	3-2	3-4	3-1
14.7	18.5	0	17.7	13.7	0	25	21.2	0	25	21.2	0

Delay (1-3) = Delay (NBR) + Delay (EBR) = 13.7 vph

Delay (1-2) = Delay (EBT, Central intersection) + Delay (EBT, East Crossover) = 18.5 sec/veh

Delay (2-4) = Delay (SBR) + Delay (WBR) = 17.7 sec/veh

Delay (2-1) = Delay (WBT, Central Intersection) + Delay (WBT, West Crossover) = 13.7 sec/veh

Delay (4-1) = Delay (NBL) =25 sec/veh

Delay (4-3) = Delay (NBT) = 21.2 sec/veh

Delay (3-2) = Delay (SBL) = 25 sec/veh

Delay (3-4) = Delay (SBT) = 21.2 sec/veh

$$Total\ \ Average\ Intersection\ delay\ \left(\frac{sec}{veh}\right) = \frac{\sum Individual\ movement\ delay\left(\frac{sec}{veh}\right) \times Volume\ of\ the\ movements\ (vph)}{\sum Volme\ of\ the\ movement\ \ (vph)}$$

= 17.63 sec/veh

<u>C.2 Example Use of DAC</u>

Consider three volume conditions of major street approach volume at Minor Street approach volume=200 vph

(1) Major street approach volume =1000 vph

(2) Major street approach volume= 2000 vph

(3) Major street approach volume= 2200 vph

In figure C.1, A represents the first condition, B represents the second condition, and C represents the third condition. From figure C.1, it is clear that A lies on the optimal performance zone of MUT, B lies on the optimal performance zone of jughandle, and C lies on the optimal performance zone of CFI.

- If A is observed in figure C.2, one can see it is located between the contours of magnitude 17 and 18.This indicates that the critical left turn delay of MUT is approximately 17.5 sec/veh higher than that of a standard signalized intersection. Again, if A is observed in figure C.3, it can be found located between the contours of magnitude -4 and -4.5. This indicates that the total average intersection delay of MUT is approximately 4.4 sec/veh lower than that of a standard signalized intersection. It can be inferred that MUT has lower total intersection delay than a standard signalized intersection, but at the expense of a higher left turn delay for critical left turn movements.
- If B is observed in figure C.4, it can be found located very near the contour of magnitude -1. This indicates that the critical left turn delay of jughandle is approximately 1 sec/veh lower than that of a standard signalized intersection. Again, if B is observed in figure C.5, it can be found located between the contours of magnitude -4. This indicates that the total average intersection delay of a jughandle is 4sec/veh lower than that of a standard signalized intersection. It can be inferred that a jughandle has lower total intersection delay as well lower critical left turn delay.

- If C is observed in figure C.6, one can see that it lies between the contours of magnitude -4.and -5. This indicates that the total intersection delay of CFI is lower than that of a standard signalized intersection by approximately 4.2 sec/veh.

This information from DAC can help the planner to decide the optimal unconventional intersection, as well as quantify the respective benefits or costs related to delay savings.

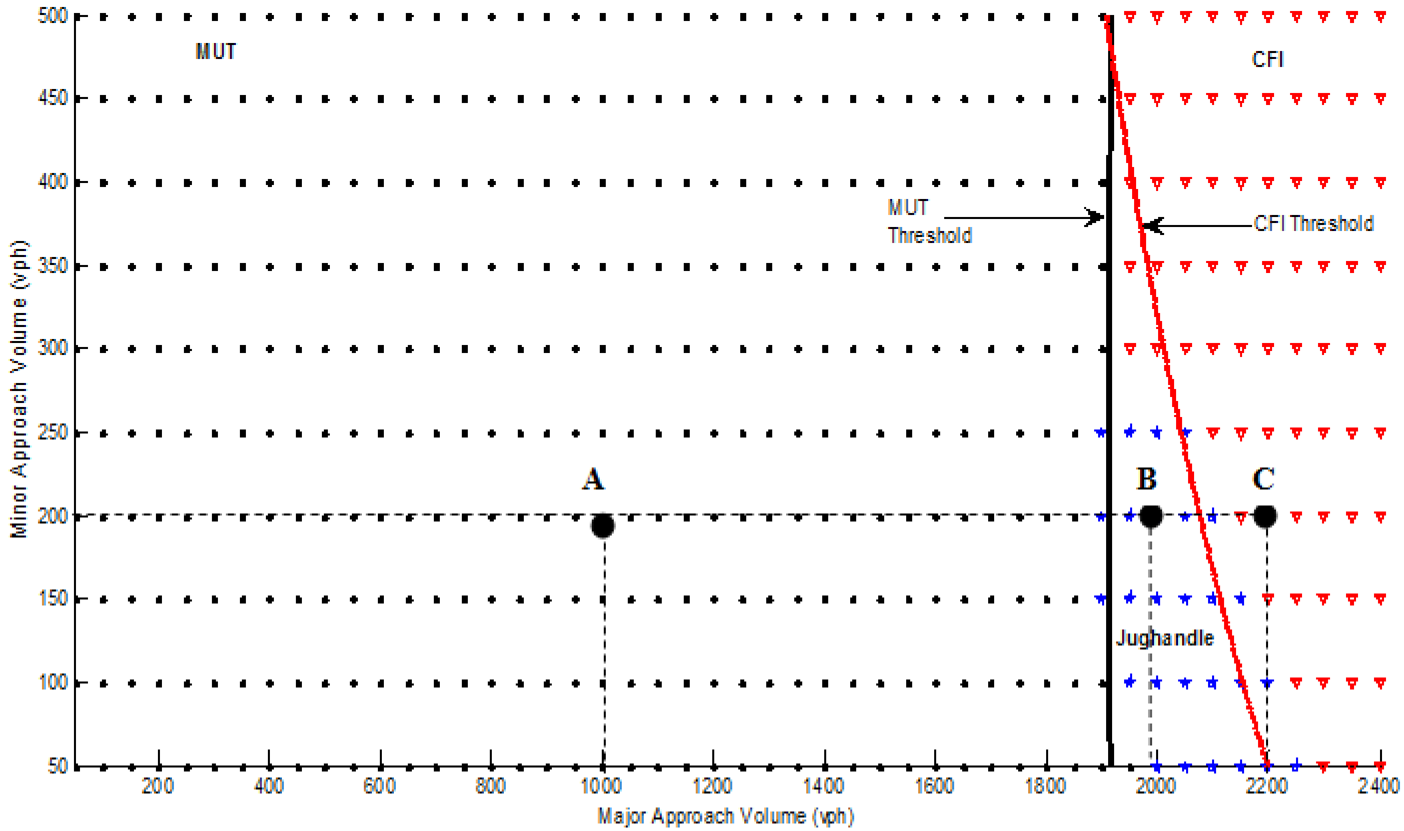


Figure C.1 DAC at BF=0.7, LTP =15%, TP =10%

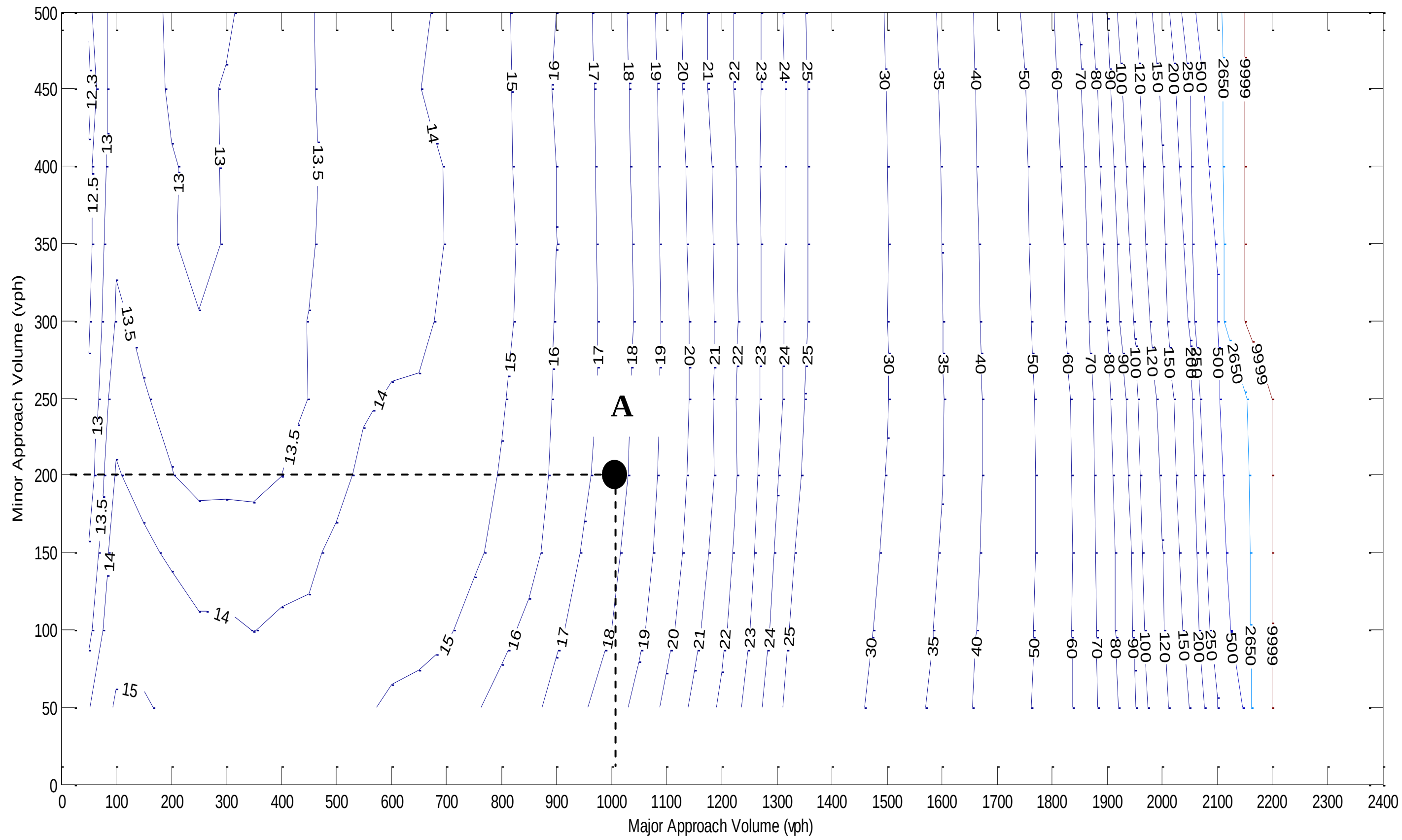


Figure C.2 Contour plot for MUT for critical left turn delay difference at BF=0.7, LTP =15%, TP =10%

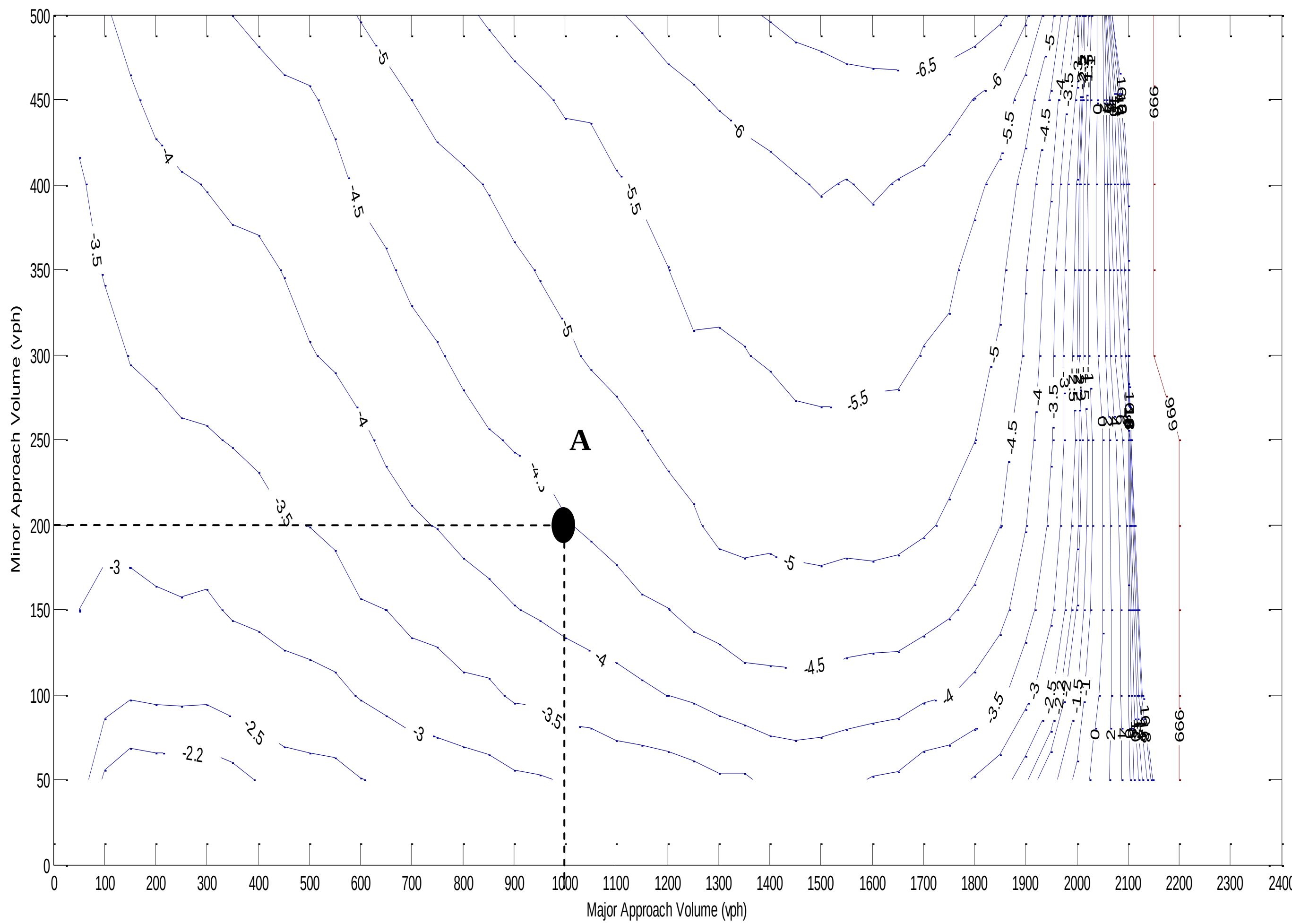


Figure C.3 Contour plot for MUT for total average intersection delay difference MUT at BF=0.7, LTP =15%, TP =10%

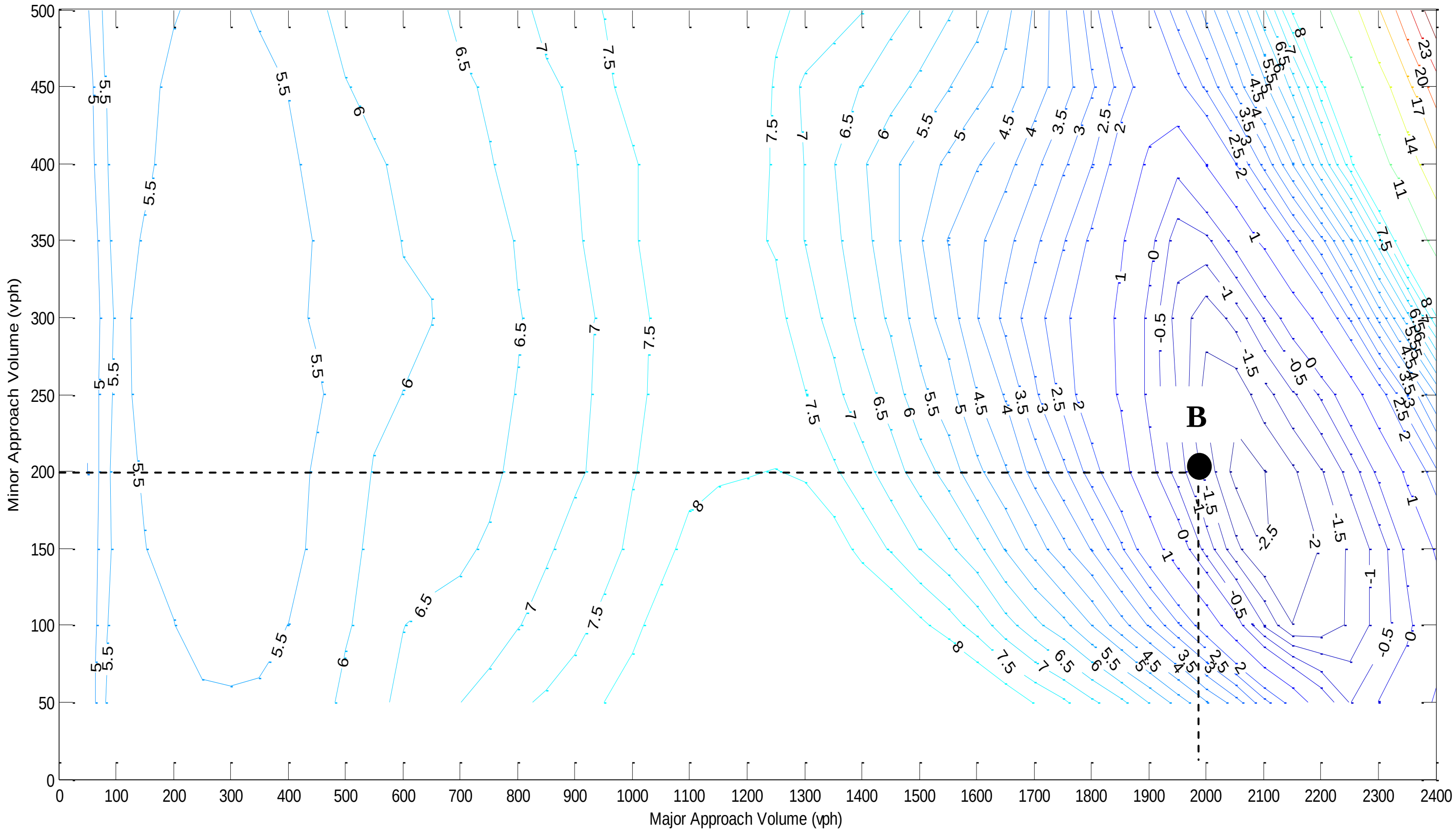


Figure C.4 Contour plot for jughandle for critical left turn delay difference at BF=0.7, LTP =15%, TP =10%

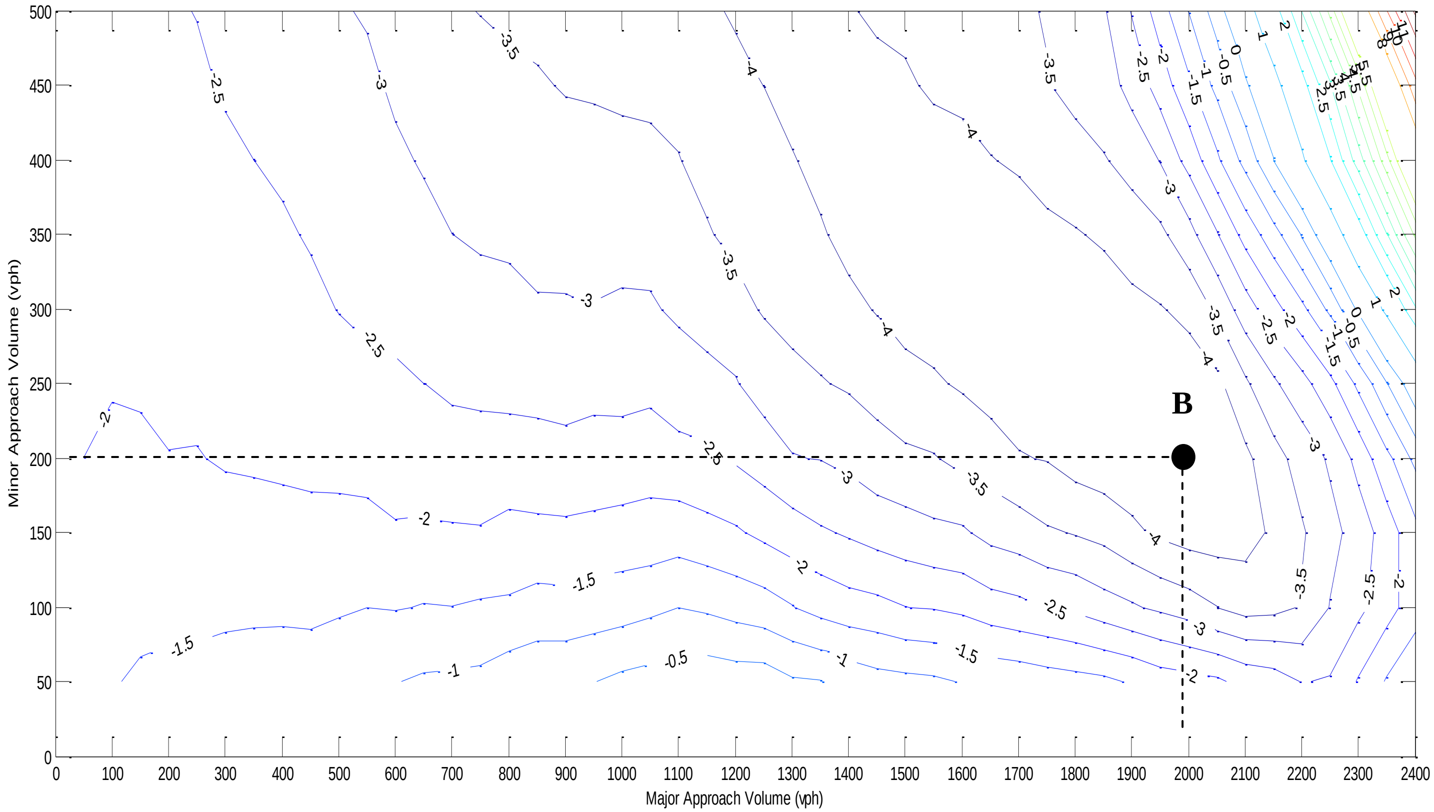


Figure C.5 Contour plot for jughandle for average total intersection delay difference at BF=0.7, LTP =15%, TP =10%

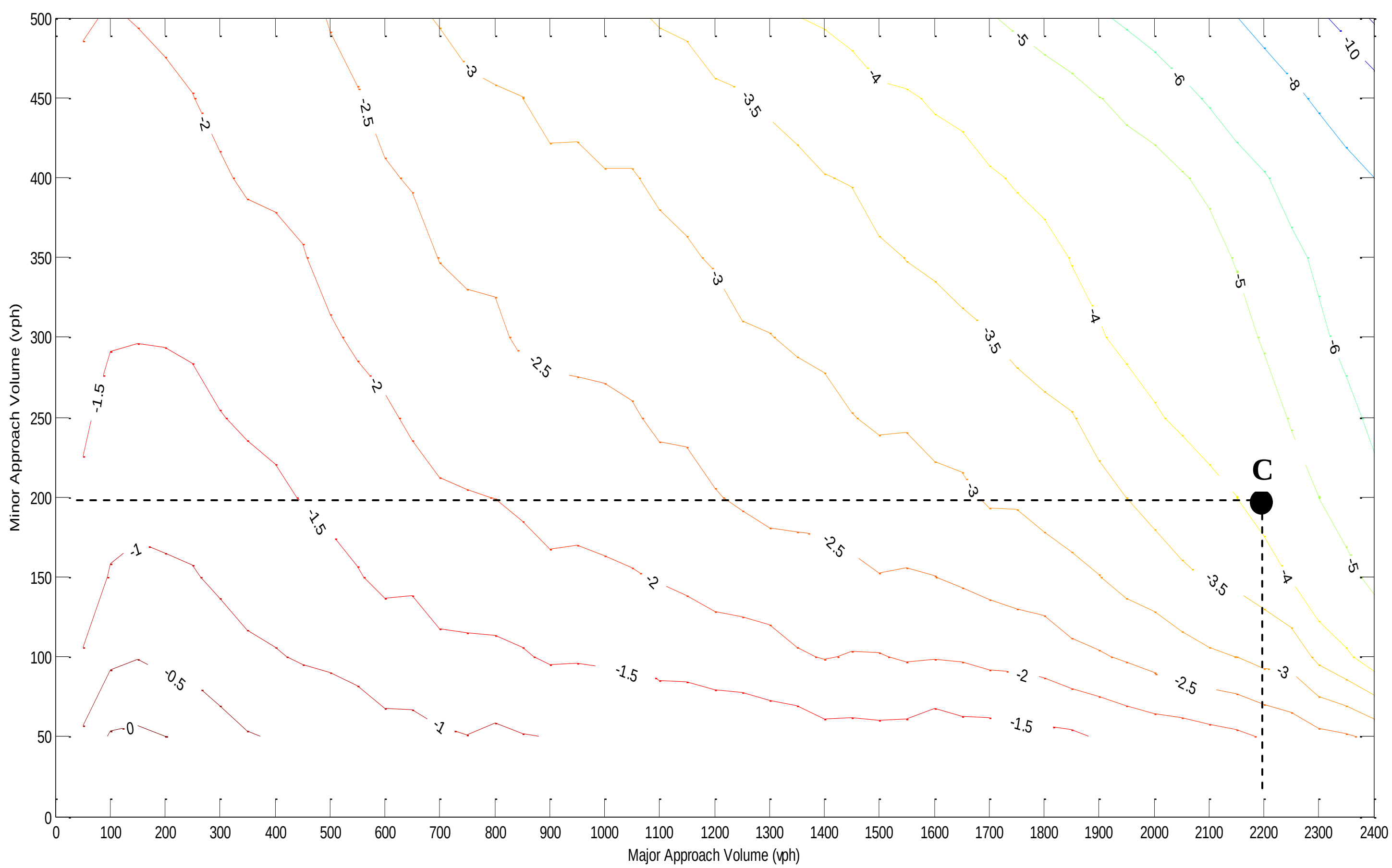


Figure C.6 Contour plot for CFI for average total intersection delay difference at BF=0.7, LTP =15%, TP =10%

Appendix D Cost

D.1. Marginal Cost of New Construction

Table D.1 Marginal cost of new construction of MUT

	Standard item no.	Item	Quantity	Unit	Rate	Amount in US Dollars
Construction Cost	3075.56	10" Doweled Concrete Pavement, Class 47 B-3500 (Including Median Opening)	399.6	SY	30.59	12223.76
	1010	Excavation (1.5')	199.8	CY	3.26	651.35
	A070.22	4 Inch Conduit in Trench	160.00	LF	5.91	945.60
	A077.26	16/C 14 AWG Traffic Signal Cable	500.00	Lf	3.98	1990.00
	7320.27	Traffic Sign and Post (STOP Sign, No RTOR and No Left Turn)	10.00	Each	295.85	2958.50
	N/A	Additional Land	2.296	Acre	4142.50	9509.87
Subtotal						28279.08
Soft Cost		Preliminary Engineering (PE)	10% of Subtotal			2827.91
		Contingency	20 %% of Subtotal			5655.82
	Total Cost in US Dollar					36762.81
Additional O & M Cost		Landscaping of median	2	Per year	1,000	2000.00

Table D.2 Marginal cost of new construction of CFI

	Standard item no.	Item	Quantity	Unit	Rate	Amount in US Dollars
Construction Cost	3075.56	10" Doweled Concrete Pavement, Class 47 B-3500	2064.6	SY	30.59	63156.11
	1010	Excavation (1.5')	1032.30	CY	3.26	3365.30
	3017.4	Concrete Class 47B-3000 Median Surfacing	0.00	SY	31.05	0.00
	A016.8	Mast Arm Signal Pole, Type MP-60	8.00	Each	8500.00	68000.00
	A504.81	Install Mast Arm Signal Pole	8.00	Each	3700.00	29600.00
	A003.10	Traffic Signal, Type TS-1	10	Each	493.89	4938.90
	A501.00	Install Traffic Signal, Type TS-1	10	Each	242.00	2420.00
	A001.7	Pull Box, Type PB-7	12	Each	1010.00	12120.00
	A070.22	4 Inch Conduit in Trench	300	LF	5.91	1773.00
	A072.10	4 Inch Conduit in Roadway	432	LF	7.83	3382.00
	A077.26	16/C 14 AWG Traffic Signal Cable	1800	Lf	3.98	7164.00
	A007.08	Vehicle Detector, Type TD-A Performed	40	Each	263.69	10547.60
	A079.1	2/C #14 AWG Detector Lead-In Cable	500	LF	1.46	730.00
	7320.27	Traffic Sign and Post (No RTOR and No Left Turn)	6	Each	295.85	1775.10
	7500.25	Left Arrow Performed Pavement Markings	6	Each	483.00	2898.00
	7496.05	5" Yellow Permanent Pavement Markings	2800	LF	0.19	532.00
	N/A	Additional Land	0.58	Acre	4142.50	2386.98
Subtotal						214789.55
Soft Cost		Preliminary Engineering (PE)	10% of Subtotal			21478.95
		Contingency	20% of Subtotal			42957.91
	Total Cost in US Dollar					279226.41
Additional O & M Cost		Additional Signal O and M (signal head, detectors, signal retiming, power supply, etc.)	4	Per year	6,000	24000.00

Table D.3 Marginal cost of new construction of jughandle

	Standard item no.	Item	Quantity	Unit	Rate	Amount in US Dollars
Construction Cost	1010	Excavation (1.5')	652.68	CY	3.26	2127.74
	3075.56	10" Doweled Concrete Pavement, Class 47 B-3500	1305.36	SY	30.59	39930.96
	7320.27	Traffic Sign and Post (Yield Sign, No RTOR and No Left Turn)	8.00	Each	295.85	2366.80
	7495.55	5" White Permanent Pavement Markings	340.00	LF	0.40	136.00
	7496.05	5" Yellow Permanent Pavement Markings	1280	LF	0.19	243.20
	N/A	Additional Land	1.17	Acre	4142.50	4850.03
Subtotal						49654.73
Soft Cost		Preliminary Engineering (PE)	10% Subtotal			4965.47
		Contingency	20% of Subtotal			9930.95
	Total Cost in US Dollar					64551.15
Additional O & M Cost		Landscaping of area enclosed by reverse curves	2	Per year	1,000	2000.00

D.2. Marginal Cost of Retrofit

Table D.4 Marginal cost of MUT retrofit

	Standard item no.	Item	Quantity	Unit	Rate	Amount in US Dollars
Construction Cost	1101	Concrete Pavement Removal	10778.10	SY	6.81	73398.86
	3075.56	10" Doweled Concrete Pavement, Class 47 B-3500 (Including Median Opening)	11910.30	SY	30.59	364336.08
	1010	Excavation (1.5')	5955.15	CY	3.26	19413.79
	1122.01	Remove Concrete Median Surfacing (6' Median)	2220.00	SY	5.90	13098.00
	A724.01	Relocate Traffic Signal	6.00	Each	470.00	2820.00
	A703.00	Relocate Mast Arm	4.00	Each	1470.00	5880.00
	A006.98	Vehicle Detector, Type TD-A Performed	24.00	Each	263.69	6328.56
	A001.7	Pull Box, Type PB-7	12.00	Each	1010.00	12120.00
	A070.22	4 Inch Conduit in Trench	160.00	LF	5.91	945.60
	A072.20	4 Inch Conduit in Roadway	432.00	LF	7.83	3382.56
	A077.26	16/C 14 AWG Traffic Signal Cable	500.00	Lf	3.98	1990.00
	A079.01	2/C #14 AWG Detector Lead-In Cable	250.00	LF	1.46	365.00
	7320.27	Traffic Sign and Post (STOP Sign, No RTOR and No Left Turn)	10.00	Each	295.85	2958.50
	7500.22	Right Arrow Performed Pavement Marking, Type 3	2.00	Each	420.00	840.00
	7500.25	Through and Left Arrow Performed Pavement Markings	6.00	Each	483.00	2898.00
	7495.55	5" White Permanent Pavement Markings	4000.00	LF	0.40	1600.00
	7496.05	5" Yellow Permanent Pavement Markings	8000.00	LF	0.19	1520.00
	N/A	Additional Land	2.296	Acre	4142.50	9509.87
Subtotal						523404.82
Soft Cost		Preliminary Engineering (PE)	10% Subtotal			52340.48
		Contingency	20% of Subtotal			104680.96
	Total Cost in US Dollar					680426.26
Additional O & M Cost		Landscaping of median	2	Per year	1,000	2000.00

Table D.5 Marginal cost of CFI Retrofit

	Standard item no.	Item	Quantity	Unit	Rate	Amount in US Dollars
Construction Cost	1122.01	Remove Concrete Median Surfacing (6' Median)	2109	SY	5.90	12443.10
	3075.56	10" Doweled Concrete Pavement, Class 47 B-3500	3196.8	SY	30.59	97790.11
	1010	Excavation (1.5')	1598.4	CY	3.26	5210.78
	3017.4	Concrete Class 47B-3000 Median Surfacing	2386.5	SY	31.05	74100.83
	A016.8	Mast Arm Signal Pole, Type MP-60	8	Each	8500.00	68000.00
	A504.81	Install Mast Arm Signal Pole	8	Each	3700.00	29600.00
	A003.10	Traffic Signal, Type TS-1	10	Each	493.89	4938.90
	A501.00	Install Traffic Signal, Type TS-1	10	Each	242.00	2420.00
	A001.7	Pull Box, Type PB-7	12	Each	1010.00	12120.00
	A070.22	4 Inch Conduit in Trench	300	LF	5.91	1773.00
	A072.20	4 Inch Conduit in Roadway	432	LF	7.83	3382.56
	A077.26	16/C 14 AWG Traffic Signal Cable	1800	Lf	3.98	7164.00
	A006.98	Vehicle Detector, Type TD-A Performed	40	Each	263.69	10547.60
	A079.01	2/C #14 AWG Detector Lead-In Cable	500	LF	1.46	730.00
	7320.27	Traffic Sign and Post (No RTOR and NO Left Turn)	4	Each	295.85	1183.40
	7500.25	Through and Left Arrow Performed Pavement Markings	4	Each	483.00	1932.00
	7495.55	5" White Permanent Pavement Markings	3800	LF	0.40	1520.00
	7496.05	5" Yellow Permanent Pavement Markings	5600	LF	0.19	1064.00
	N/A	Additional Land	0.58	Acre	4142.50	2386.98
Subtotal						338307.26
Soft Cost		Preliminary Engineering (PE)	10% Subtotal			33830.73
		Contingency	20% of Subtotal			67661.45
	Total Cost in US Dollar					439799.44
& M Cost		Additional Signal O and M (signal head, detectors, signal retiming, power supply, etc.)	4	Per year	6,000	24000.00

Table D.6 Marginal cost of jughandle retrofit

	Standard item no.	Item	Quantity	Unit	Rate	Amount in US Dollars
Construction Cost	1010	Excavation (1.5')	652.68	CY	3.26	2127.74
	3075.56	10" Doweled Concrete Pavement, Class 47 B-3500	1305.36	SY	30.59	39930.96
	7320.27	Traffic Sign and Post (Yield Sign, No RTOR and No Left Turn)	8.00	Each	295.85	2366.80
	7495.55	5" White Permanent Pavement Markings	340.00	LF	0.40	136.00
	7496.05	5" Yellow Permanent Pavement Markings	1620	LF	0.19	307.80
	N/A	Additional Land	1.17	Acre	4142.50	4850.03
Subtotal						49719.33
Soft Cost		Preliminary Engineering (PE)	10% Subtotal			4971.93
		Contingency	20% of Subtotal			9943.87
Total Cost in US Dollar						64635.13
Additional O & M Cost		Landscaping of area enclosed by reverse curves	2	Per year	1,000	2000.00

D.3. Rate Analysis for Delay

Table D.7 Rate analysis of delay

Items	Amount	Notes
Nebraska average per capita income (2011) from US Bureau of Census (US $)	26113	
Total working days in year	260	52 weeks/year
Total working hours (assuming 8 hrs/day)	2080	
Average income per hour in NE (US $)	12.55	
Cost of time per hour (US $)	12.55	

APPENDIX B DAC PLOTS, CONTOUR PLOTS AND

GROWTH PLOTS

A SUPPLEMENT

TO REPORT

“INVESTIGATING OPERATION AT GEOMETRICALLY UNCONVENTIONAL INTERSECTIONS”

B.1. DAC Plots

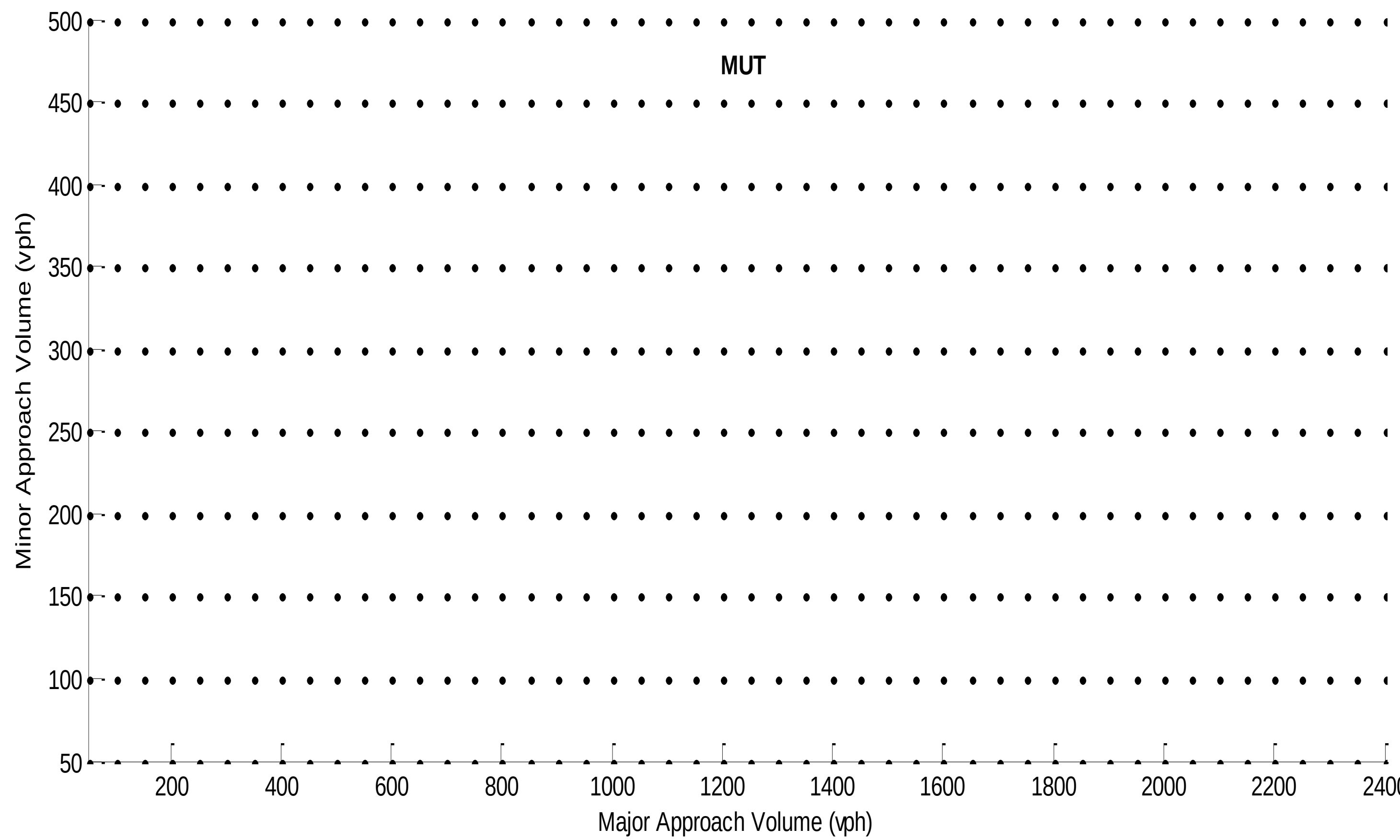


Figure B.1 DAC for unconventional intersection at BF=0.5, LTP=5% and TP=2%

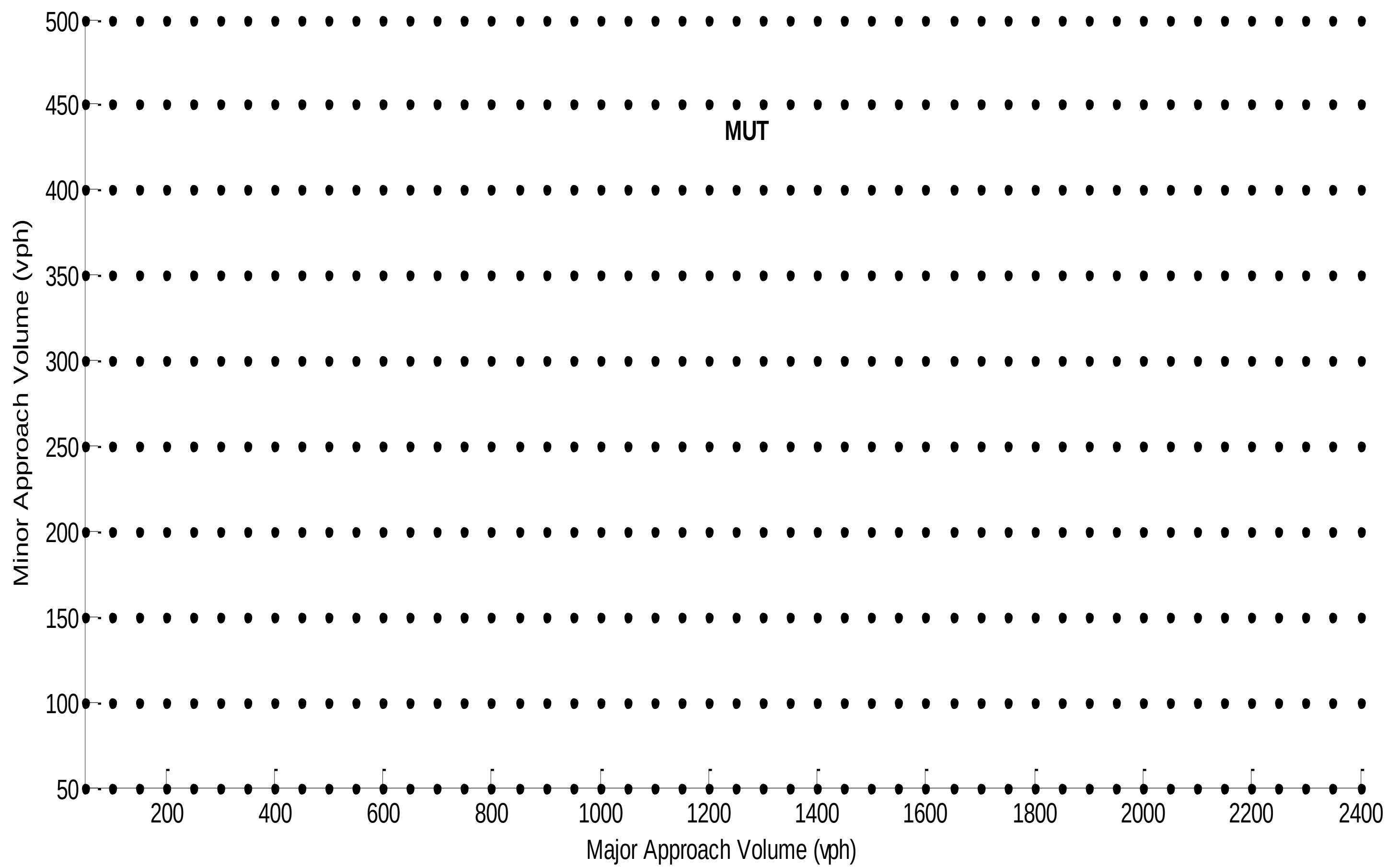


Figure B.2 DAC for unconventional intersection at BF=0.6, LTP=5% and TP=2%

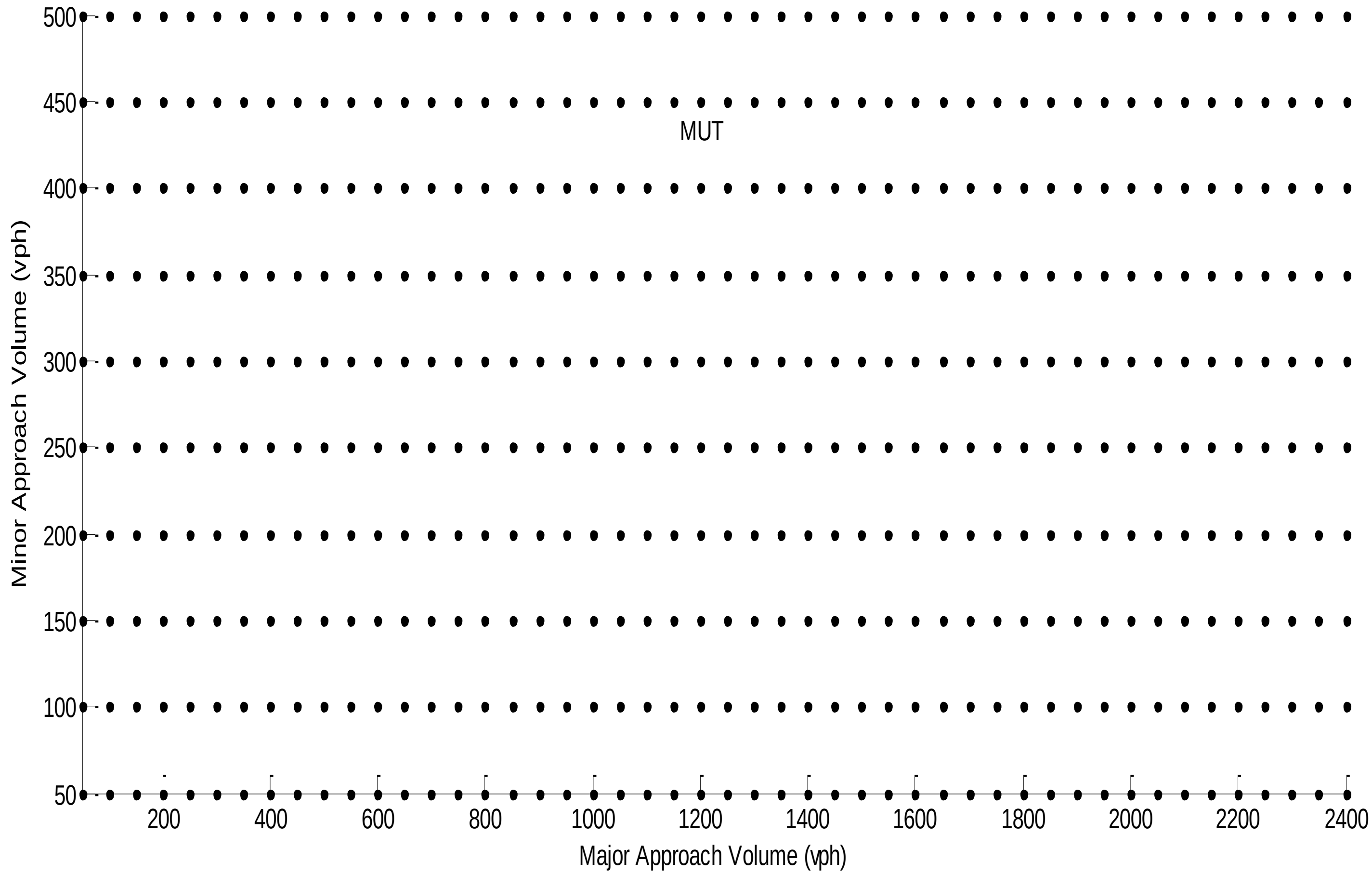


Figure B.3 DAC for unconventional intersection at BF=0.7, LTP=5% and TP=2%

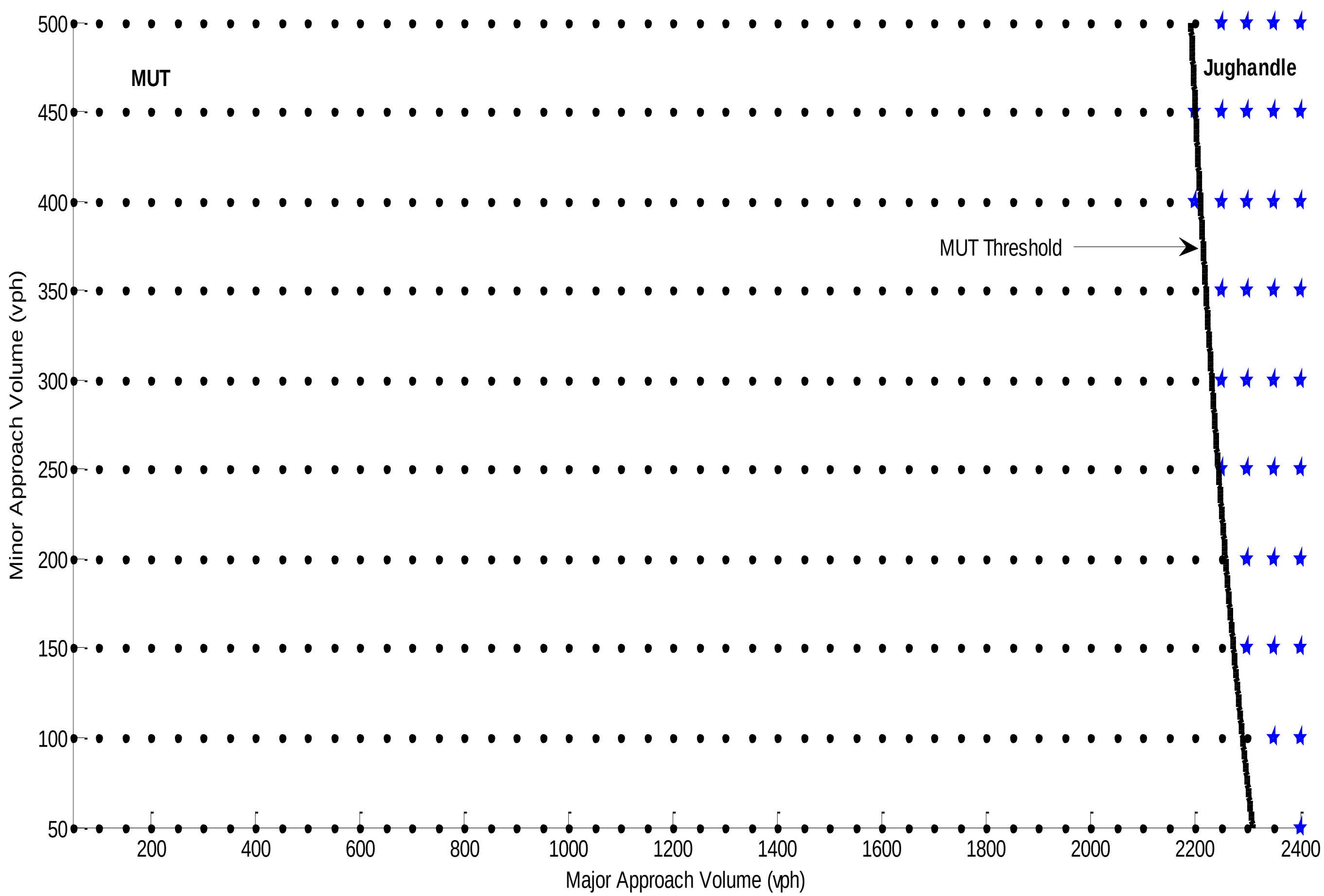


Figure B.4 DAC for unconventional intersection at BF=0.5, LTP=10% and TP=2%

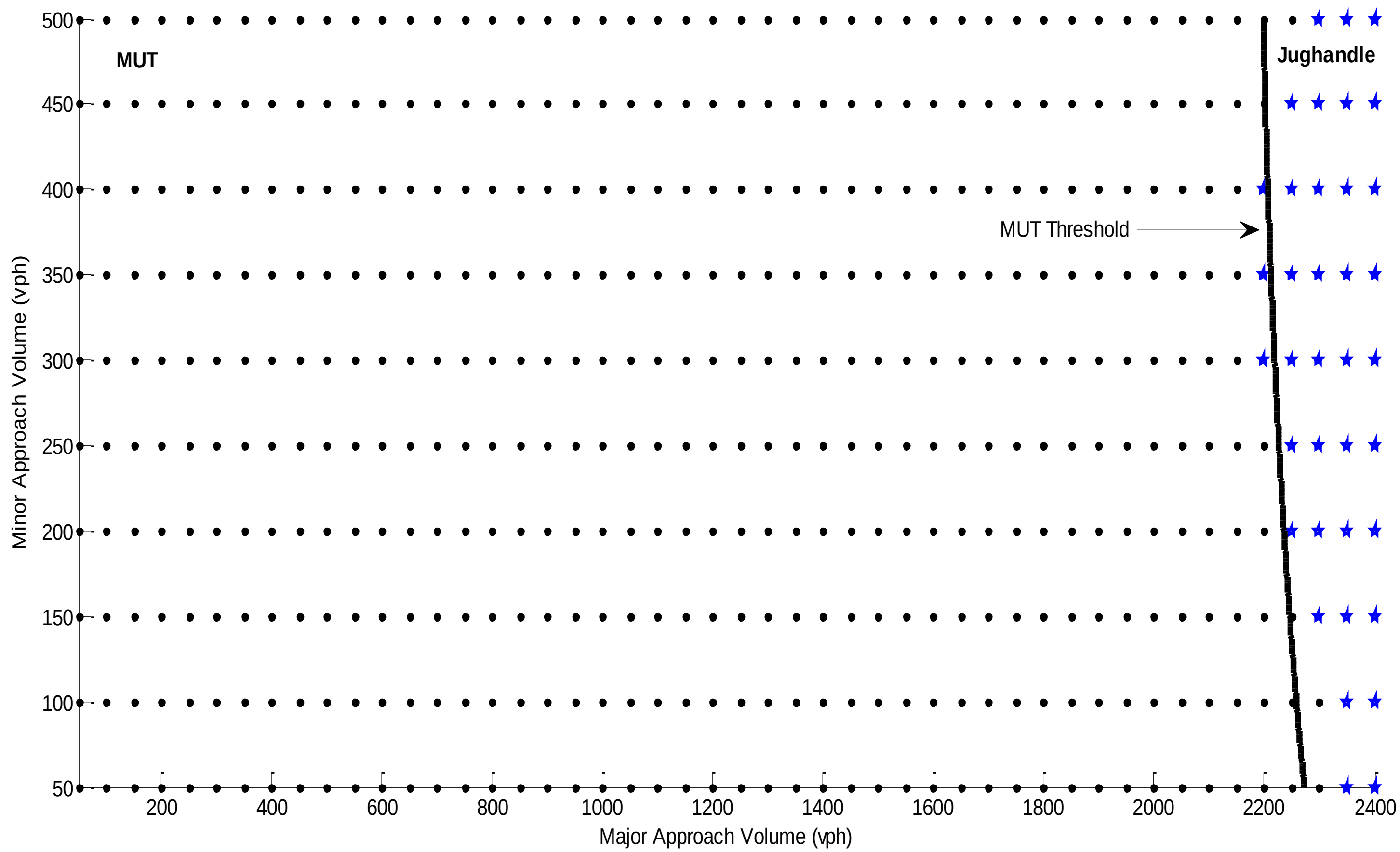


Figure B.5 DAC for unconventional intersection at BF=0.6, LTP=10% and TP=2%

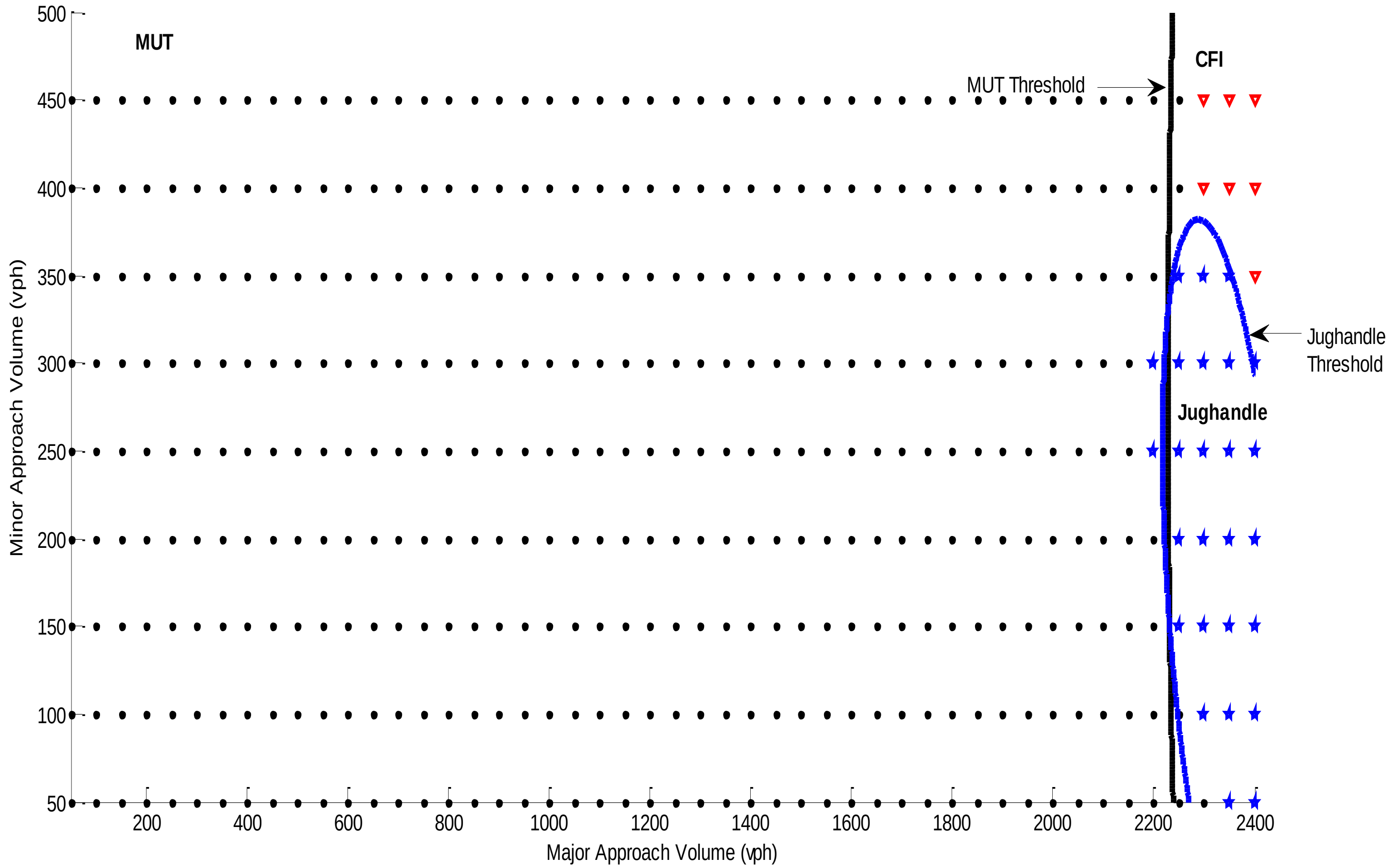


Figure B.6 DAC for unconventional intersection at BF=0.7, LTP=10% and TP=2%

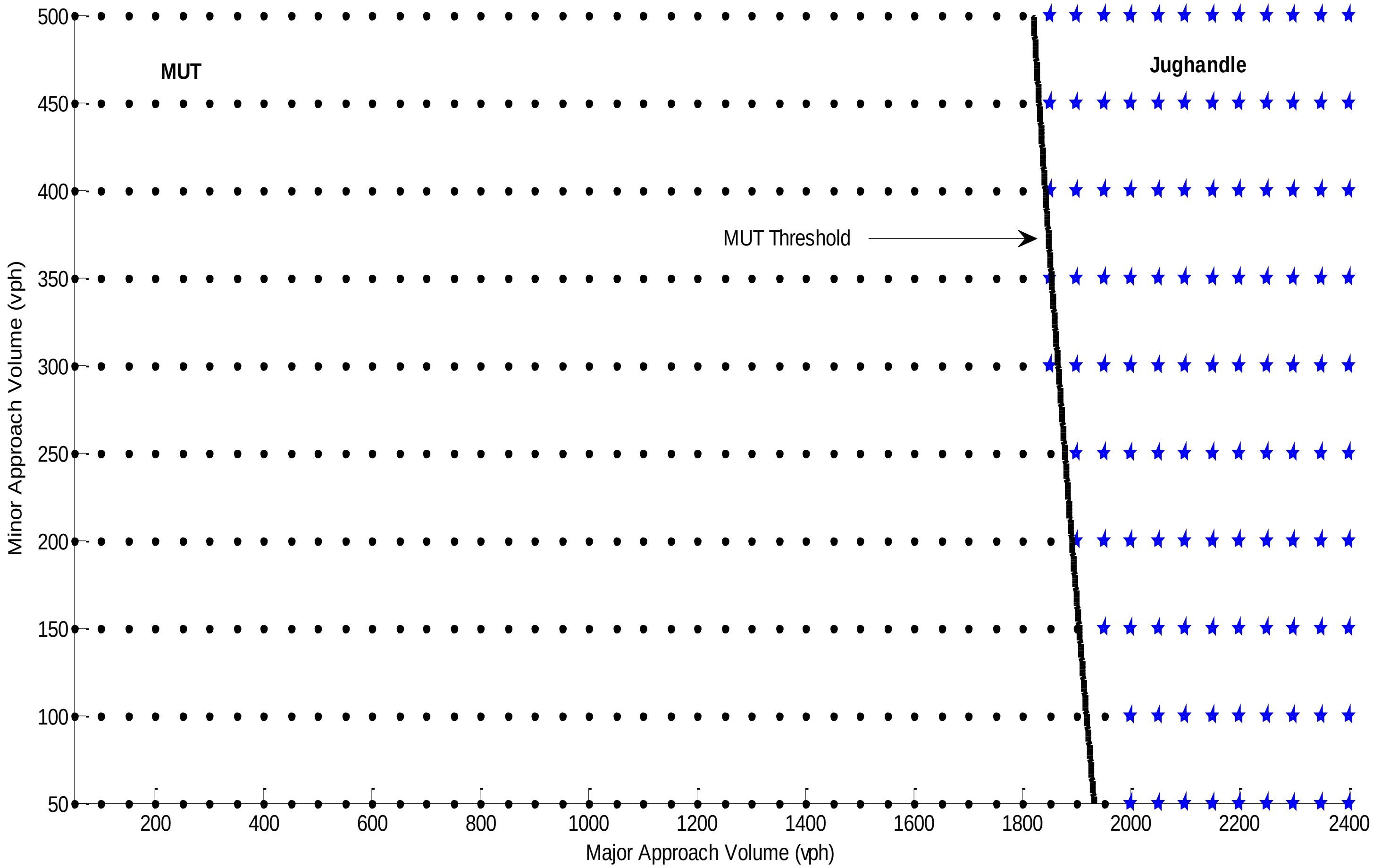


Figure B.7 DAC for unconventional intersection at BF=0.5, LTP=15% and TP=2%

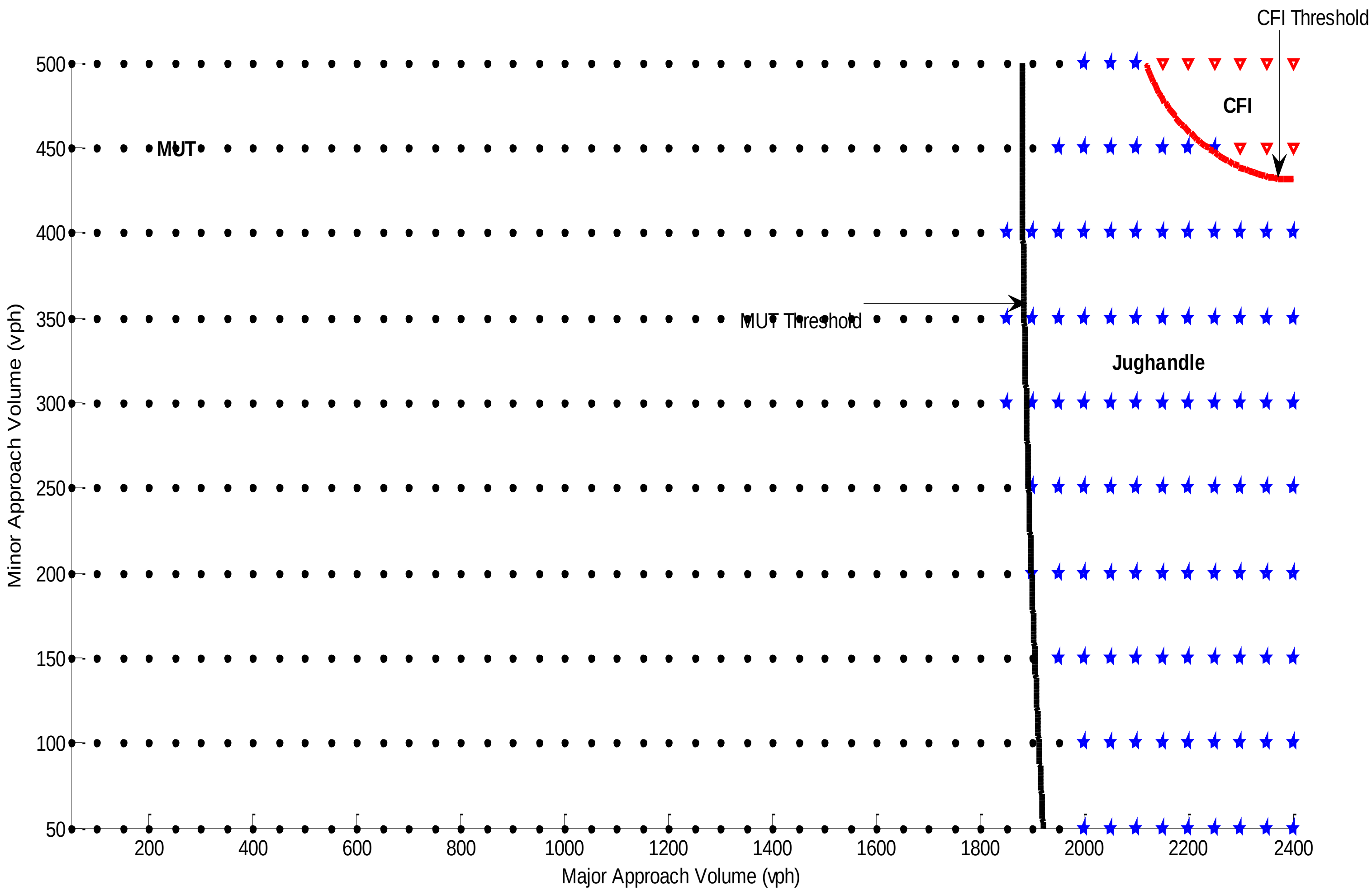


Figure B.8 DAC for unconventional intersection at BF=0.6, LTP=15% and TP=2%

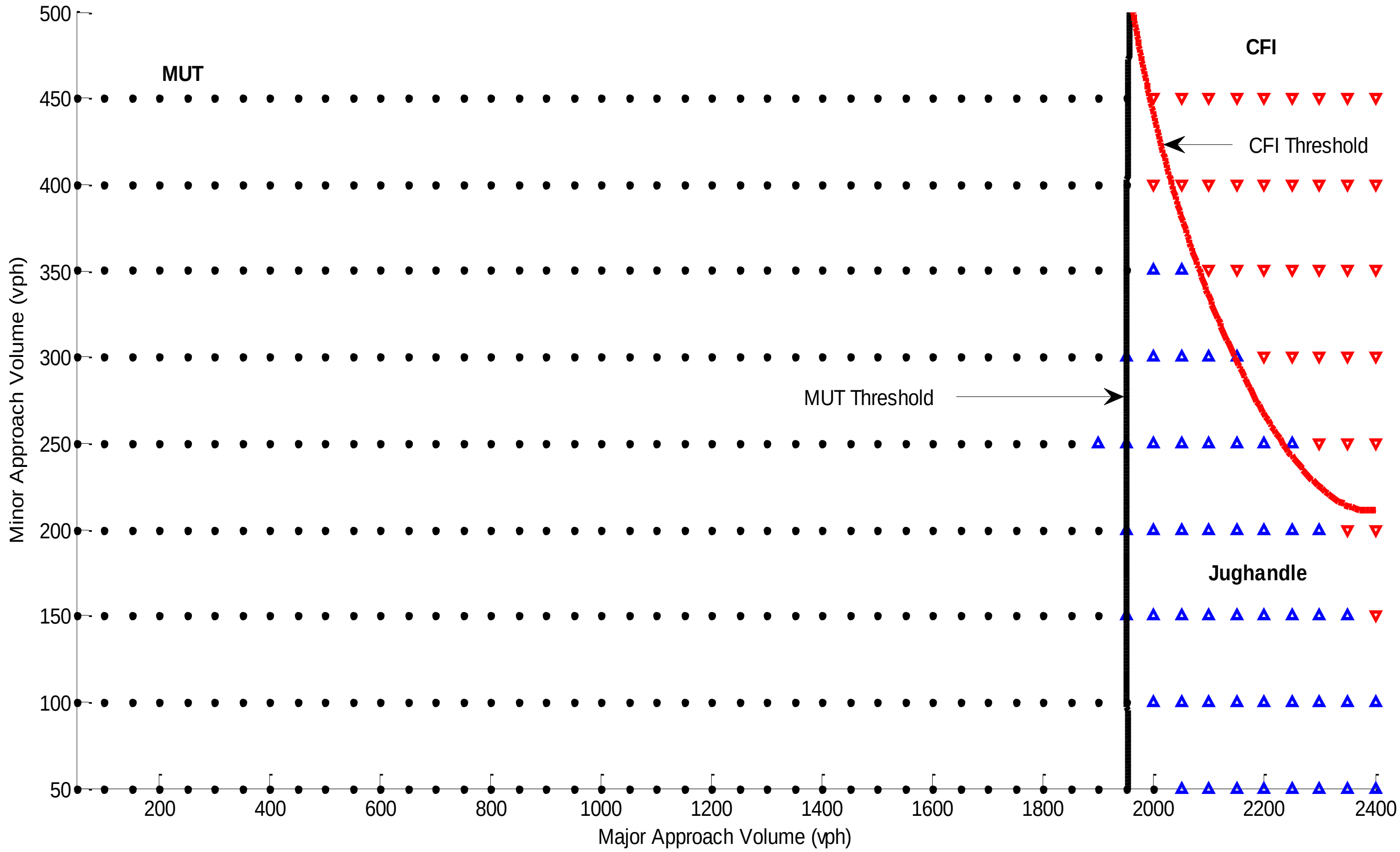


Figure B.9 DAC for unconventional intersection at BF=0.7, LTP=15% and TP=2%

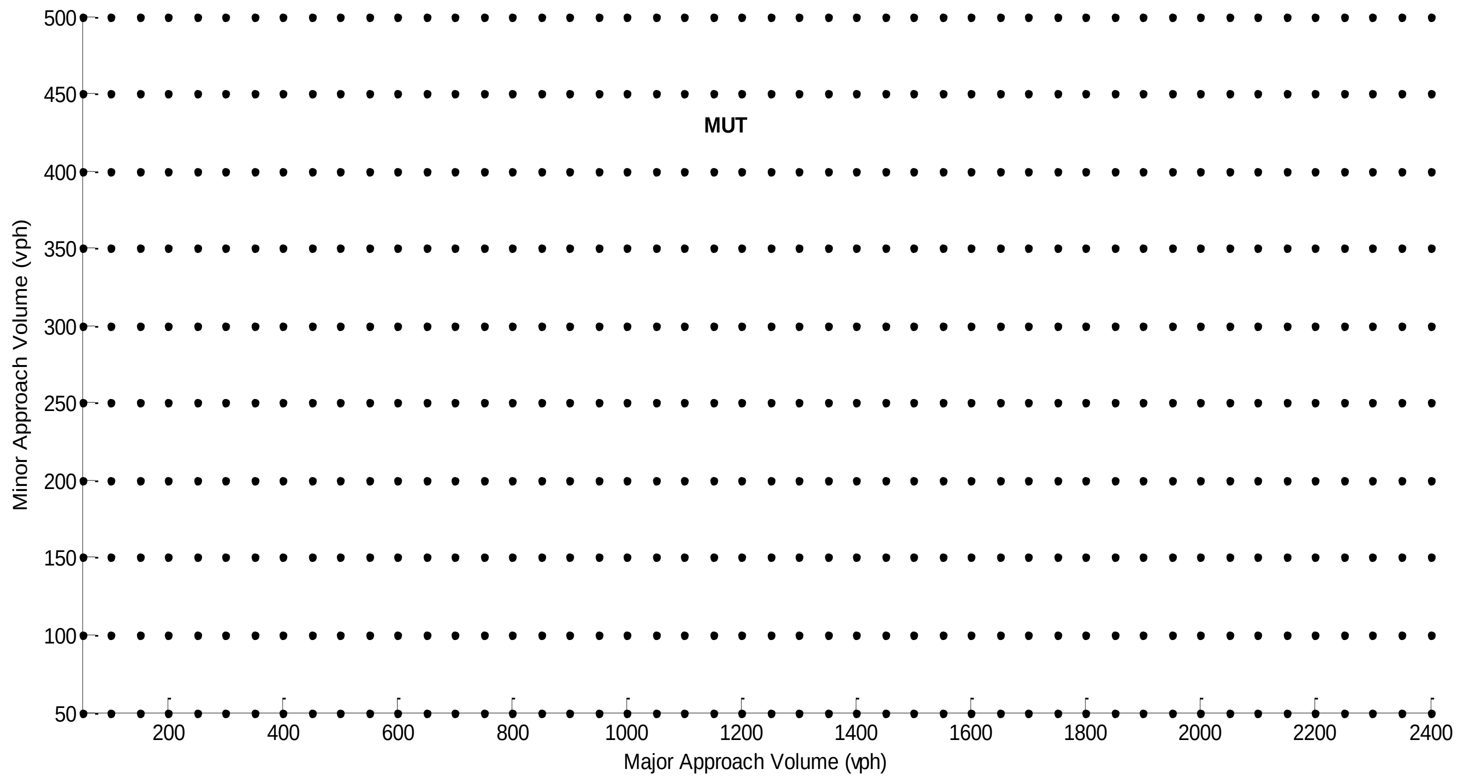


Figure B.10 DAC for unconventional intersection at BF=0.5, LTP=5% and TP=5%

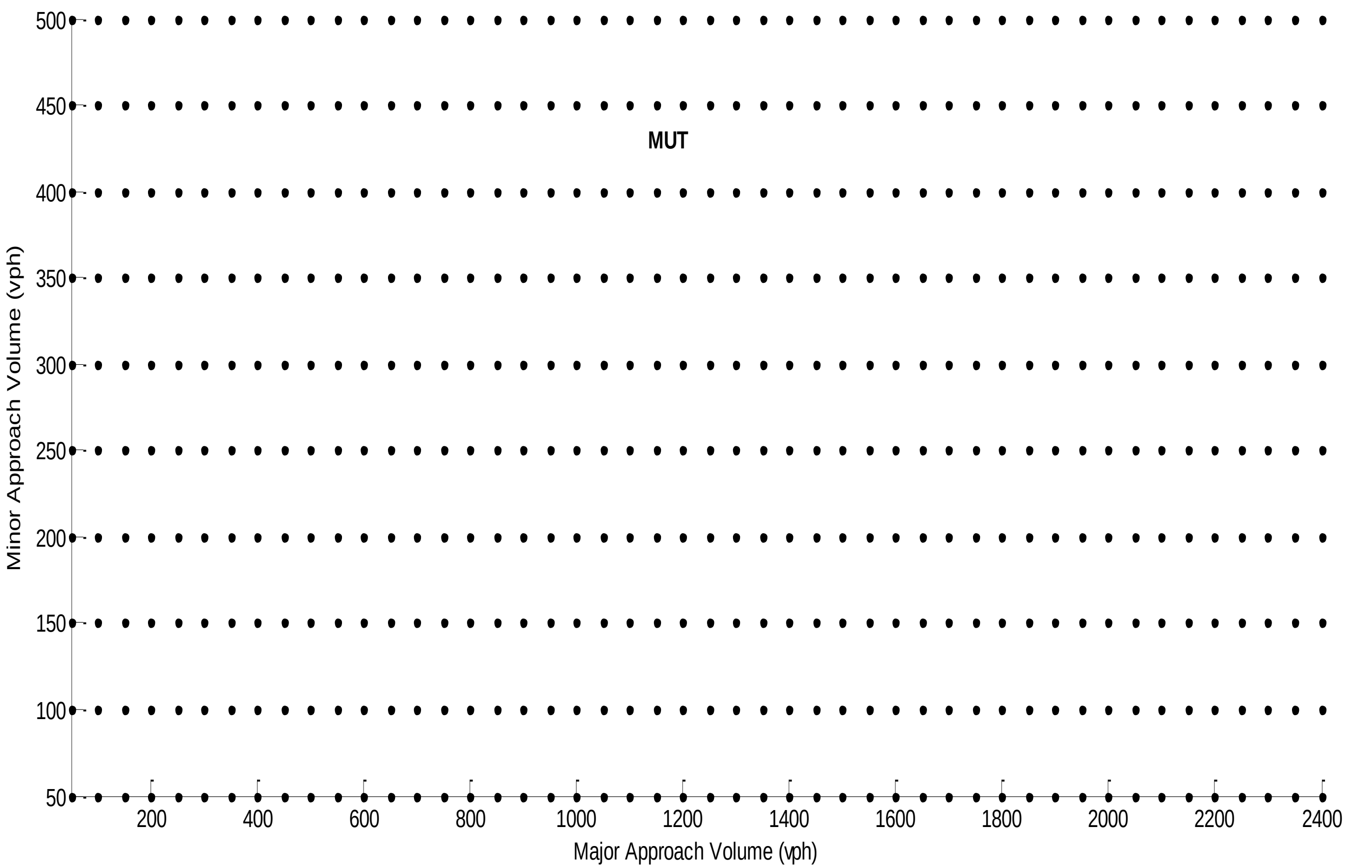


Figure B.11 DAC for unconventional intersection at BF=0.6, LTP=5% and TP=5%

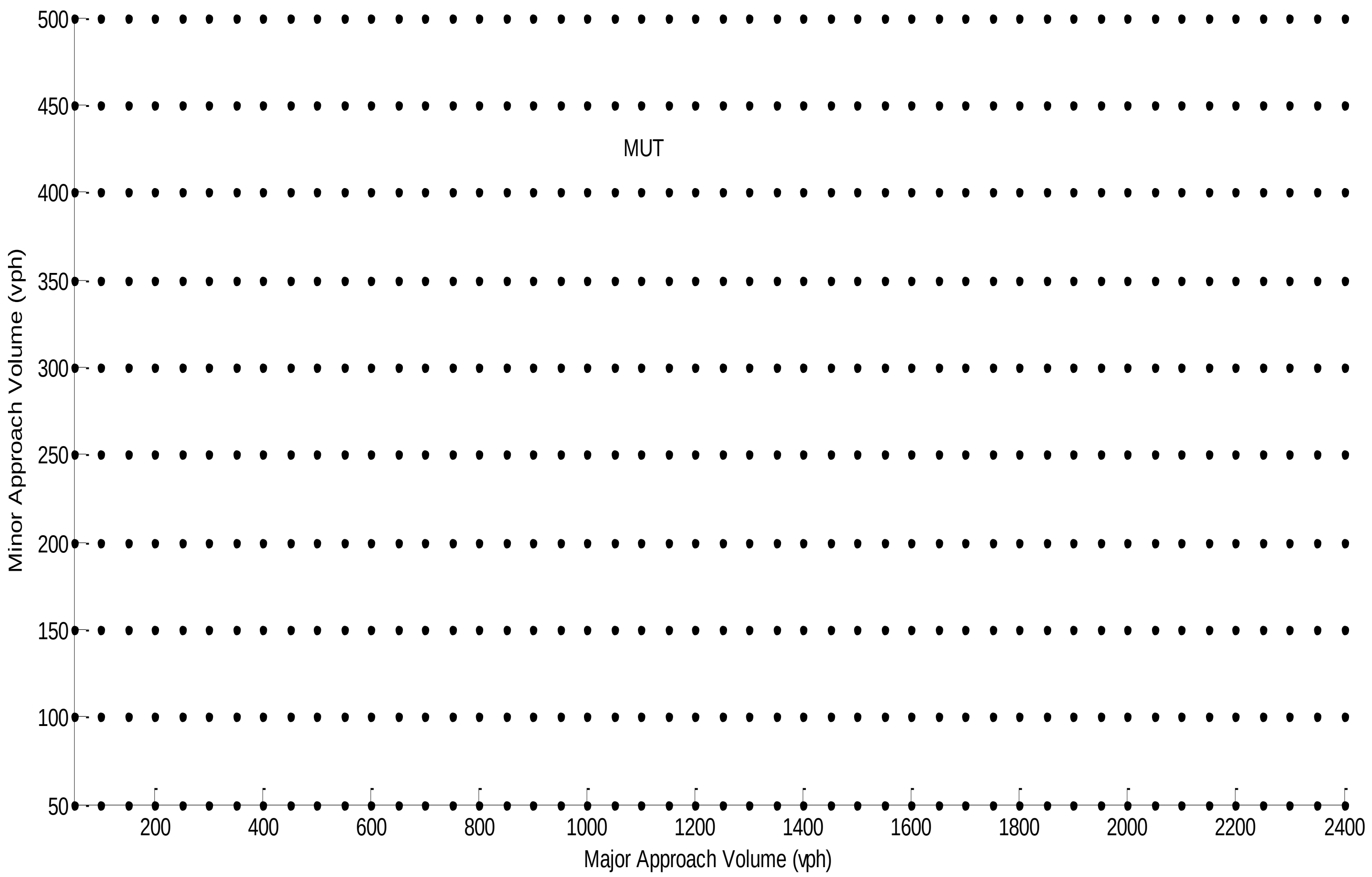


Figure B.12 DAC for unconventional intersection at BF=0.7, LTP=5% and TP=5%

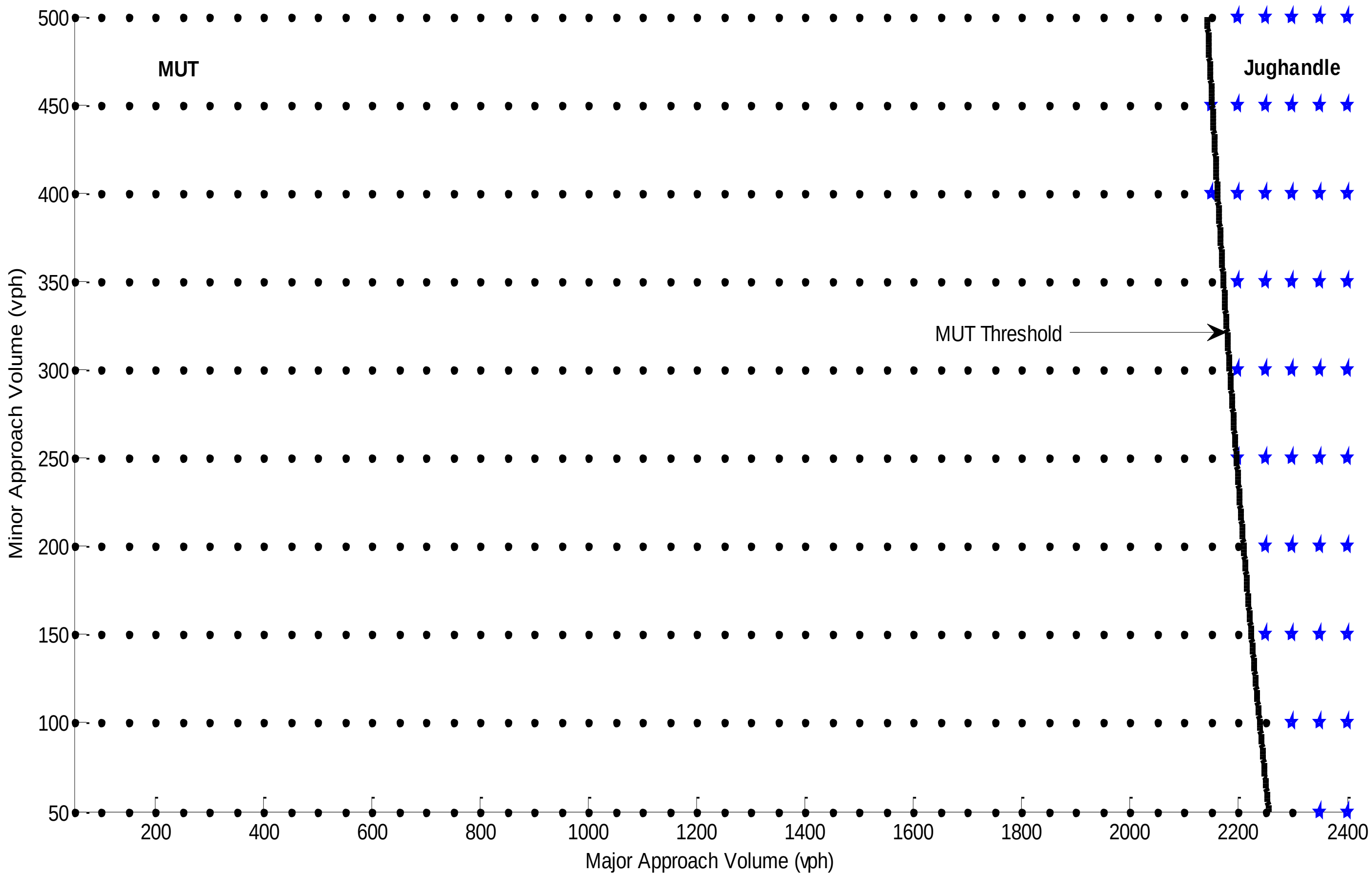


Figure B.13 DAC for unconventional intersection at BF=0.5, LTP=10% and TP=5%

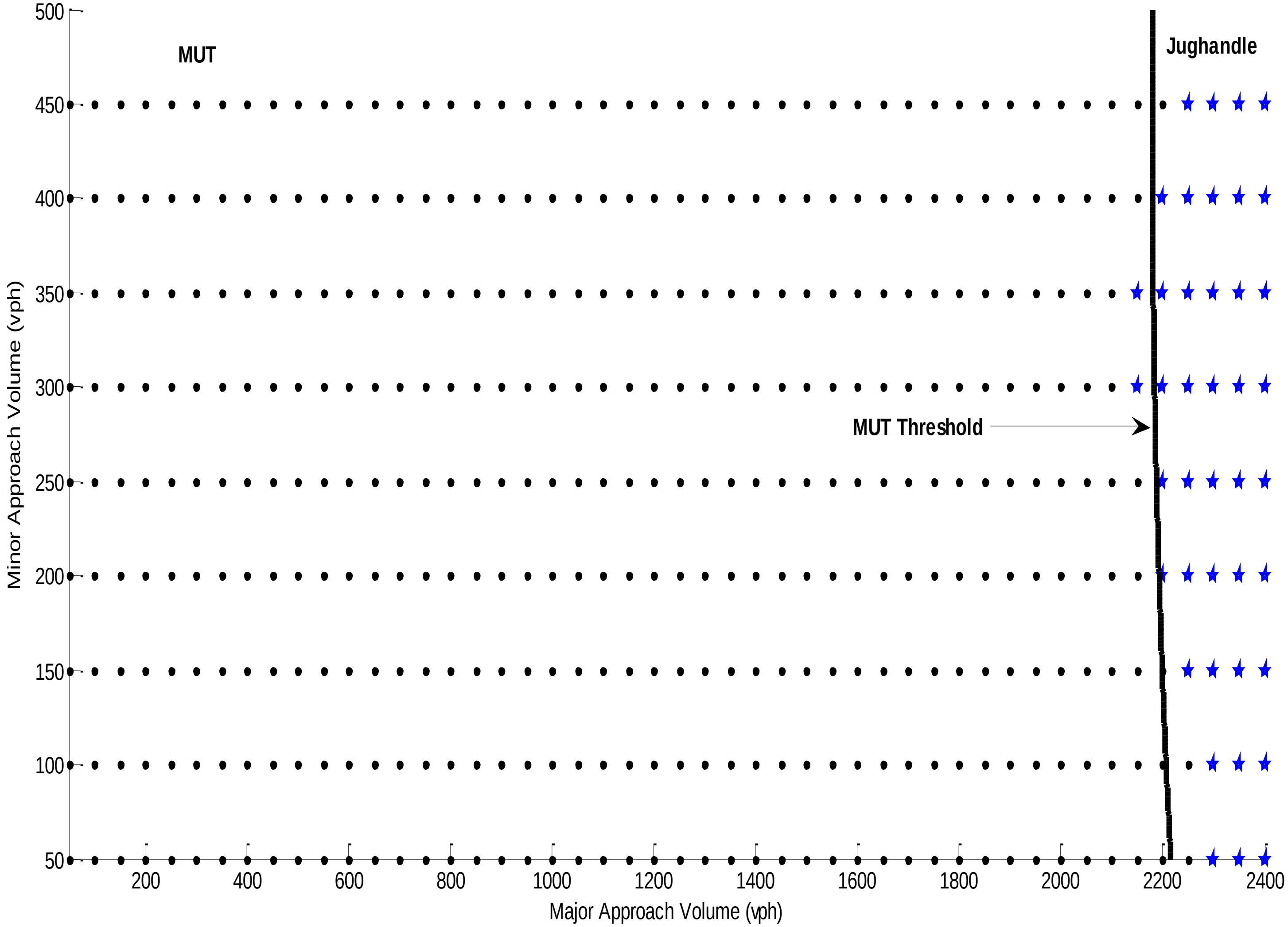


Figure B.14 DAC for unconventional intersection at BF=0.6, LTP=10% and TP=5%

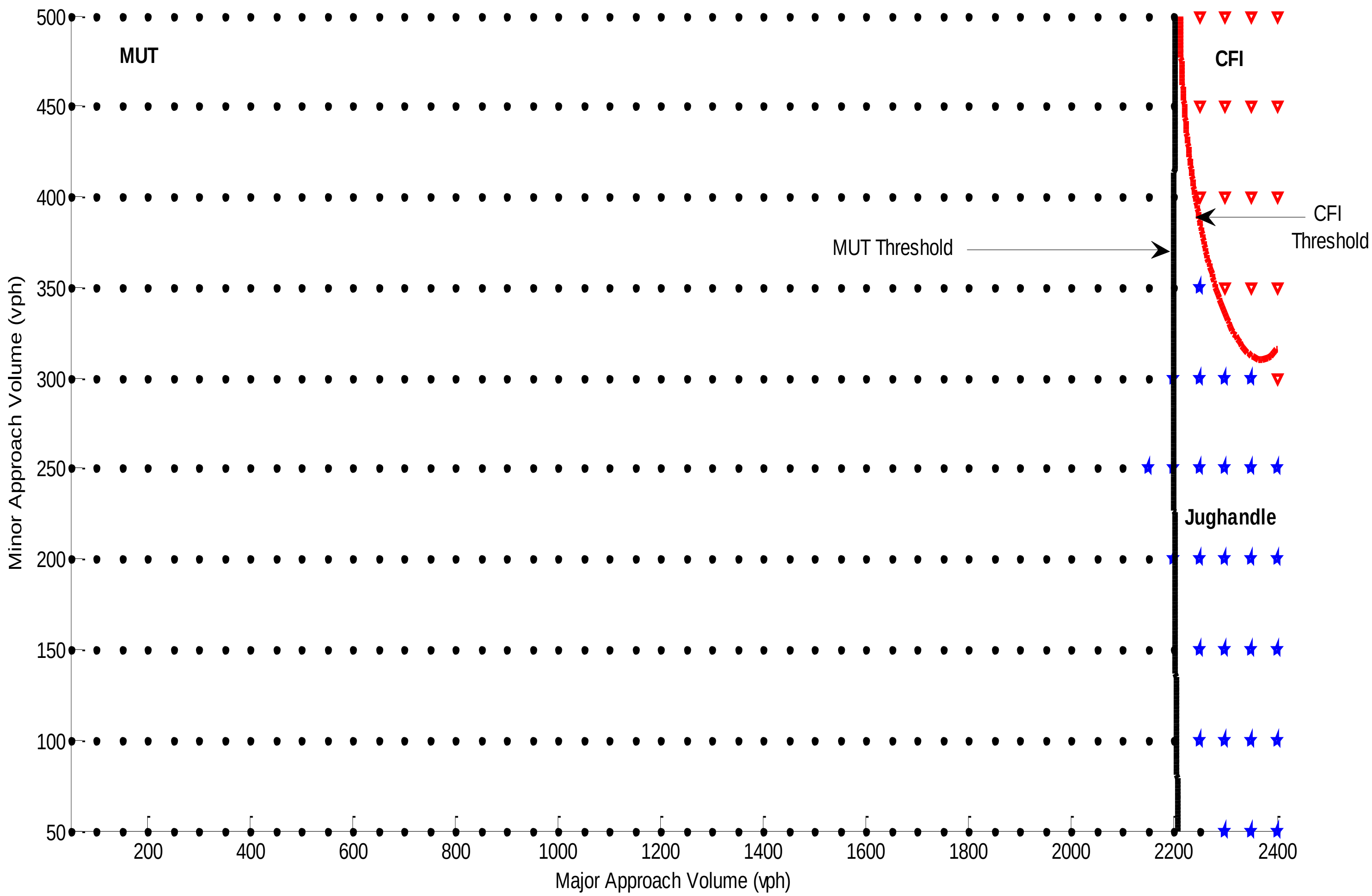


Figure B.15 DAC for unconventional intersection at BF=0.7, LTP=10% and TP=5%

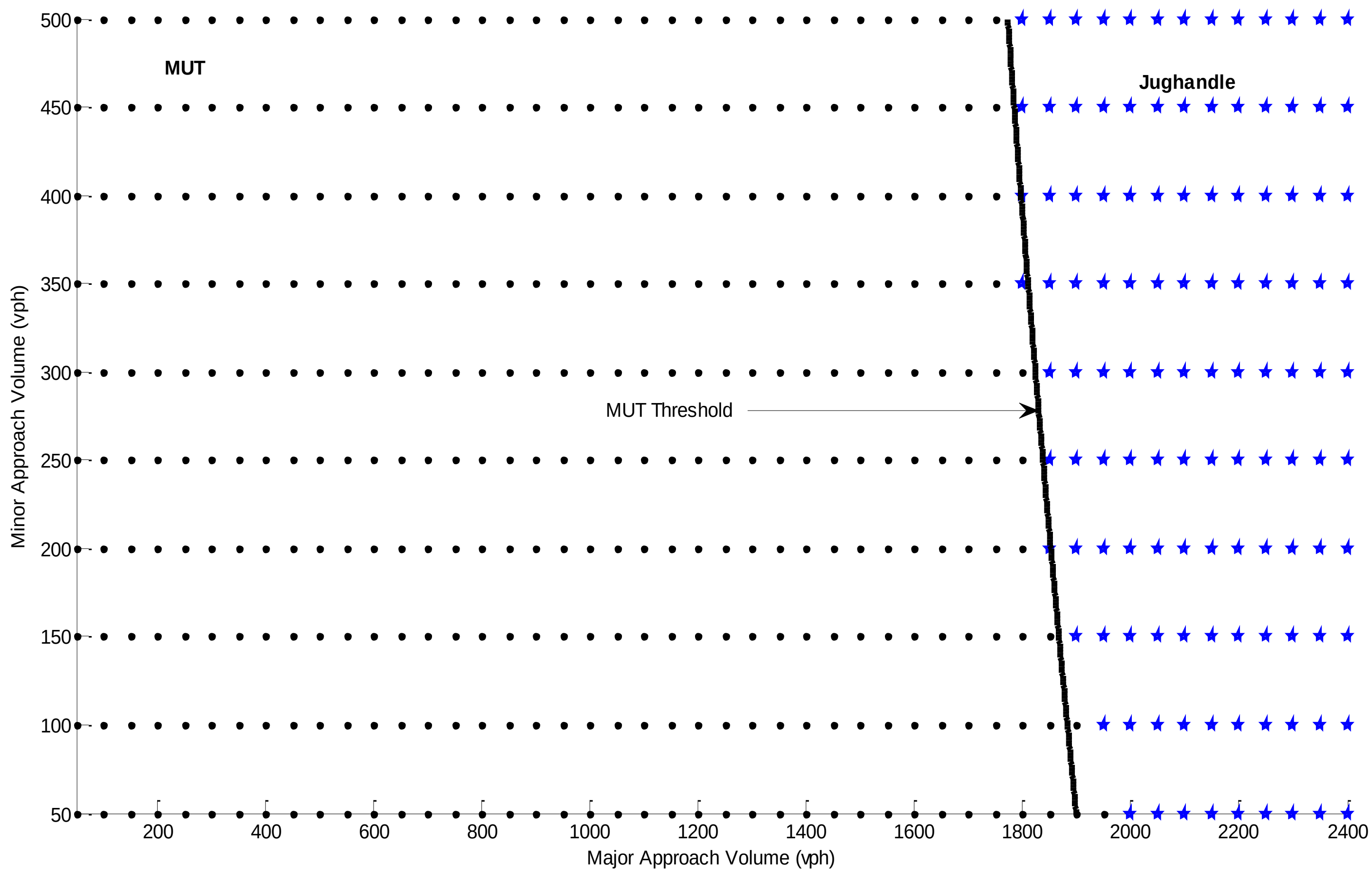


Figure B.16 DAC for unconventional intersection at BF=0.5, LTP=15% and TP=5%

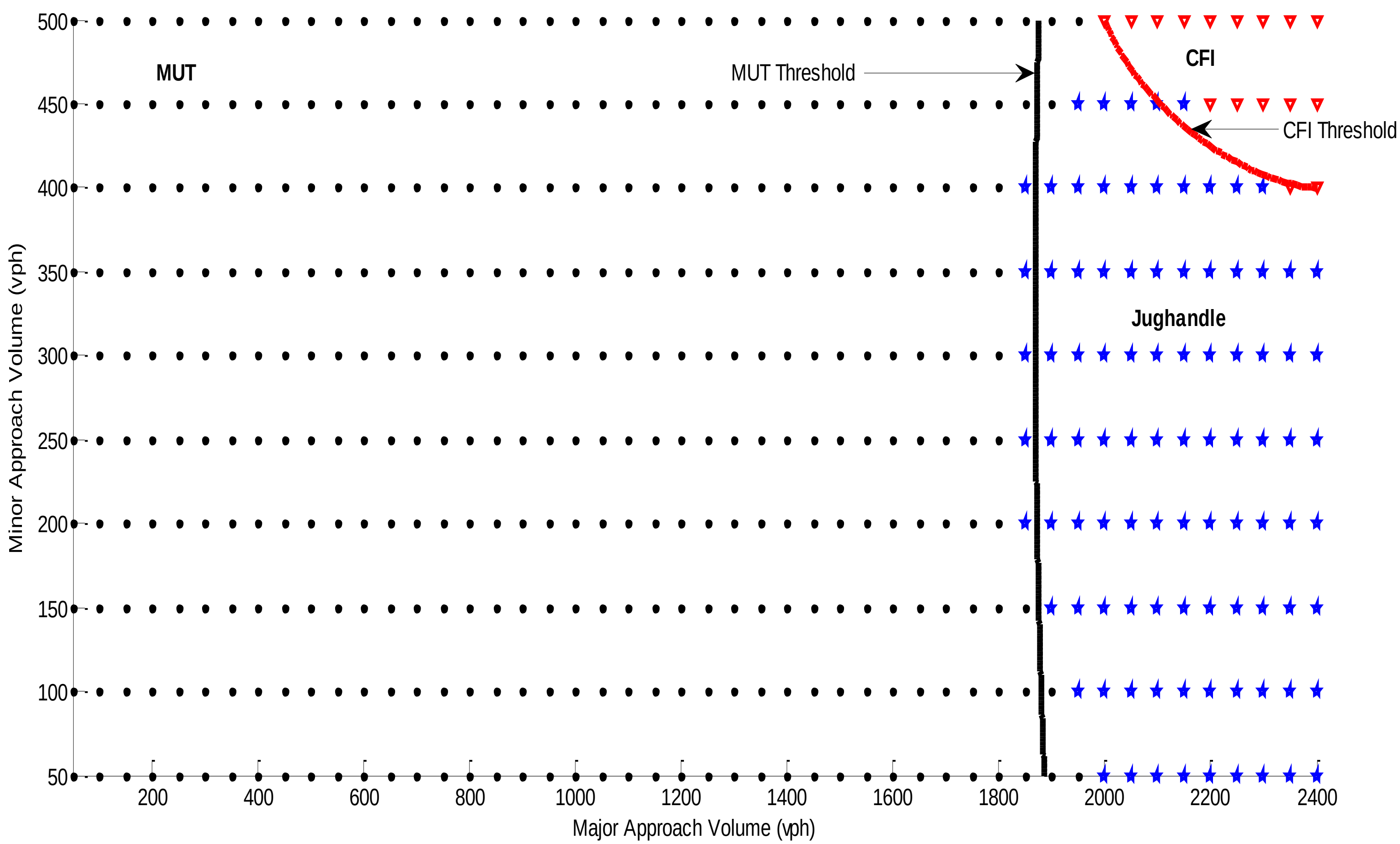


Figure B.17 DAC for unconventional intersection at BF=0.6, LTP=15% and TP=5%

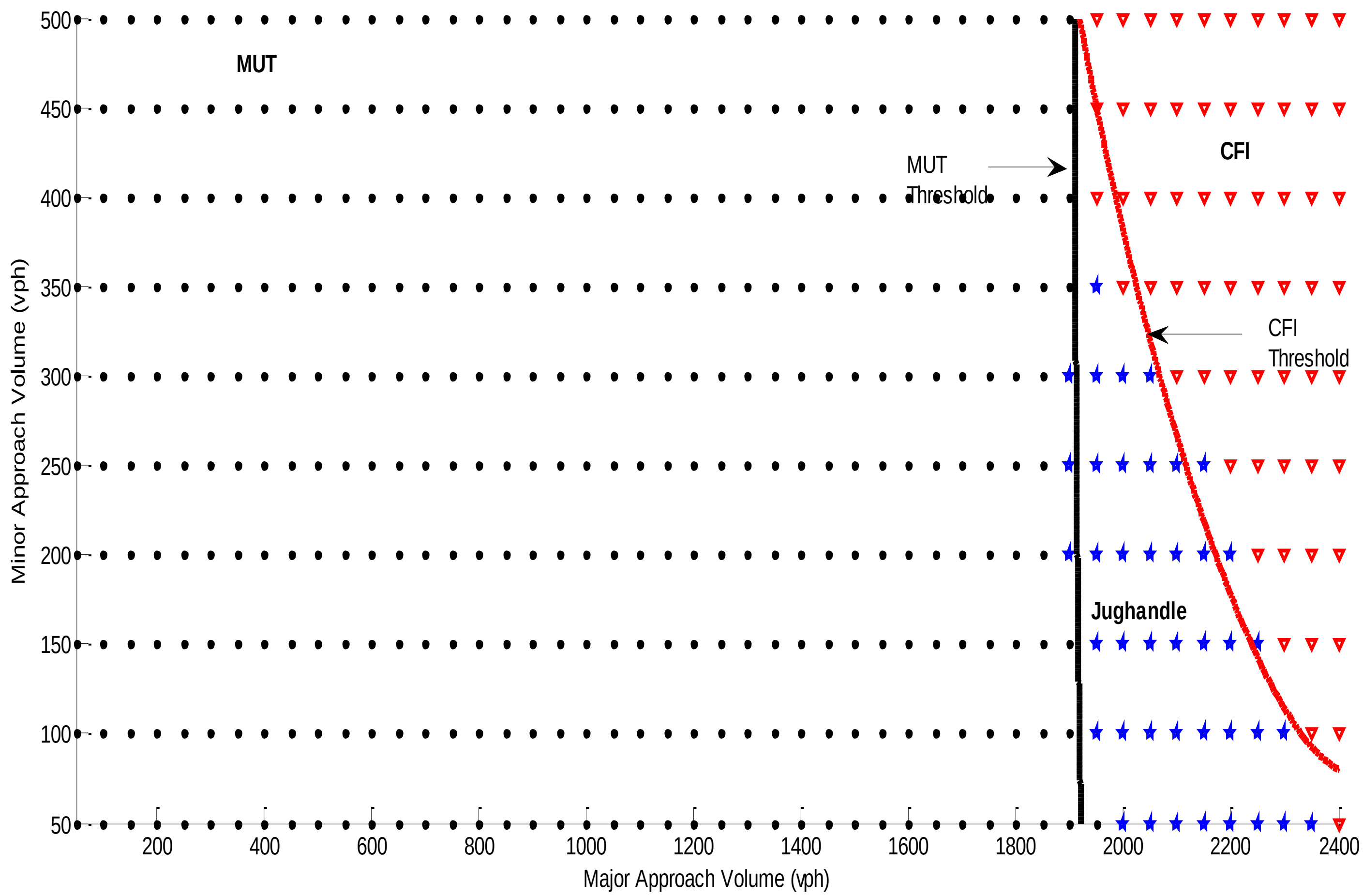


Figure B.18 DAC for unconventional intersection at BF=0.7, LTP=15% and TP=5%

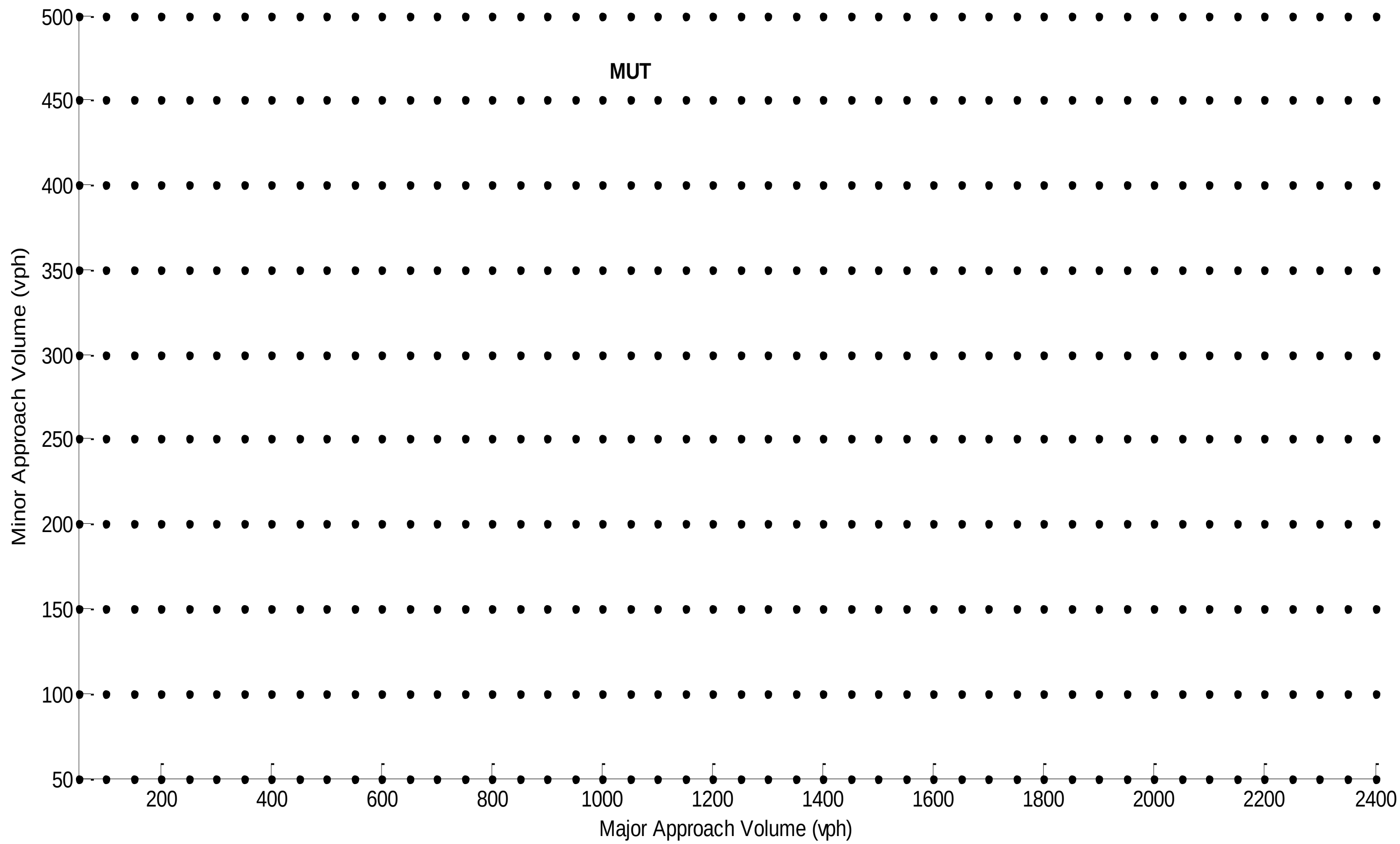


Figure B.19 DAC for unconventional intersection at BF=0.5, LTP=5% and TP=10%

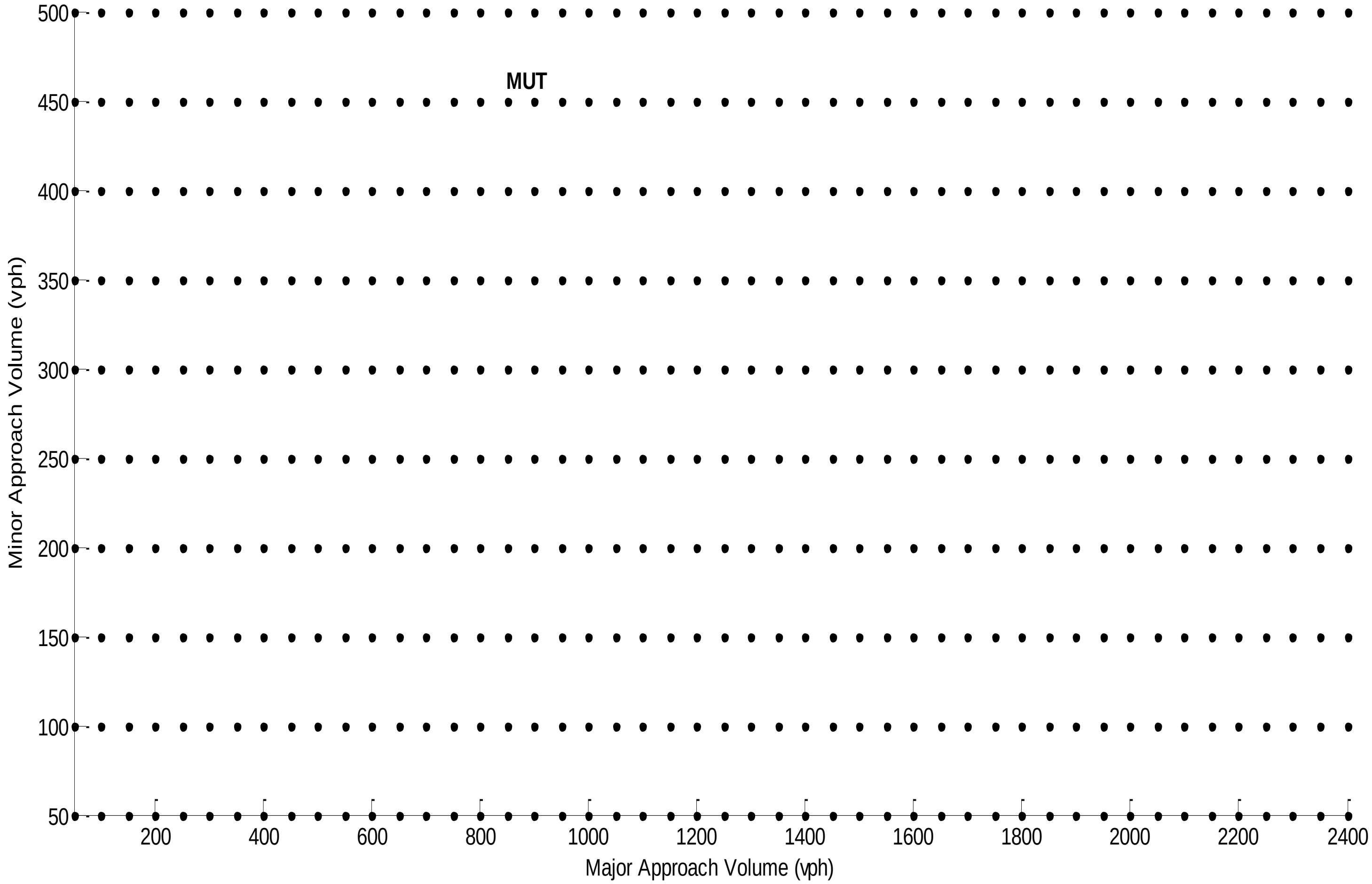


Figure B.20 DAC for unconventional intersection at BF=0.6, LTP=5% and TP=10%

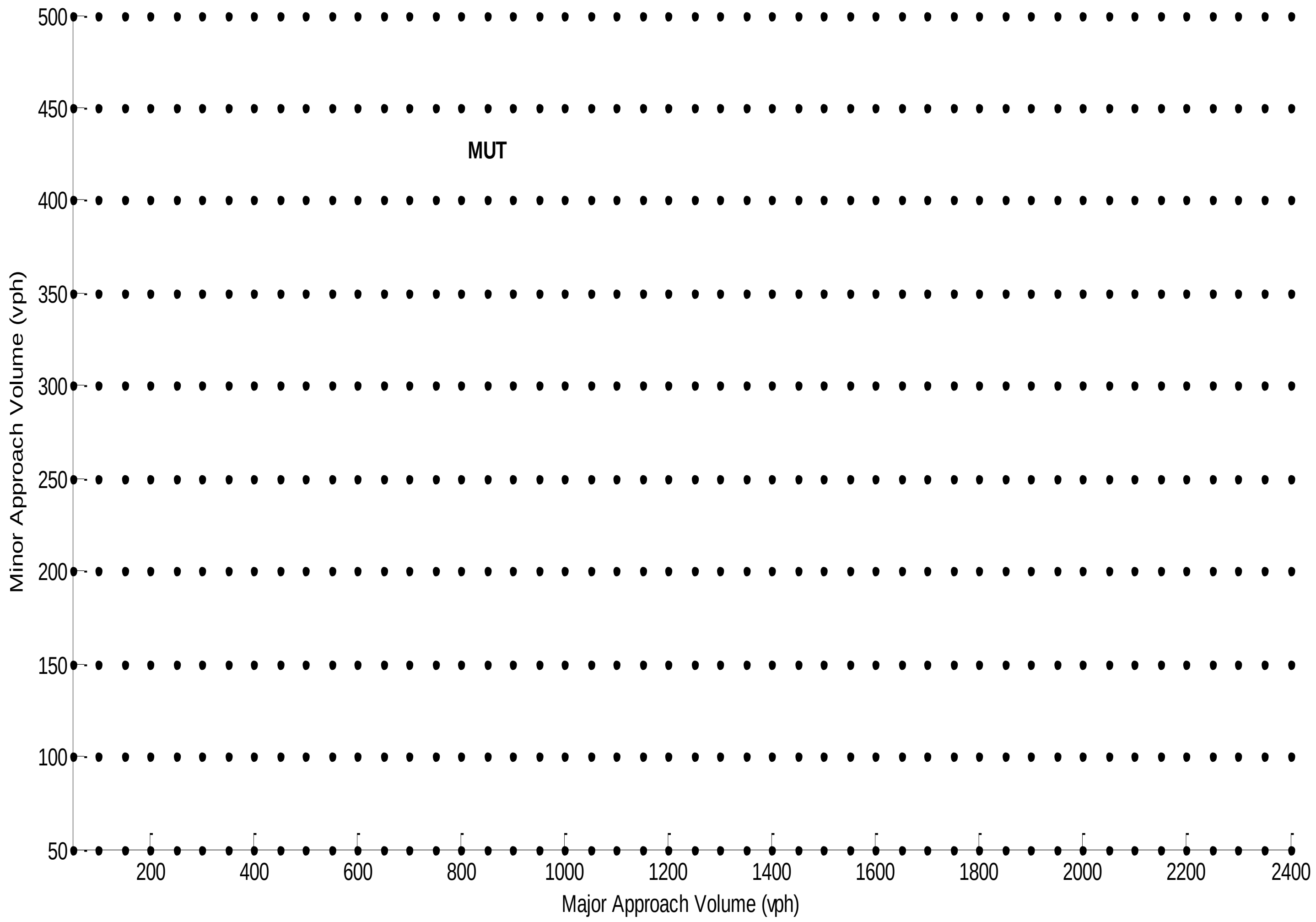


Figure B.21 DAC for unconventional intersection at BF=0.7, LTP=5% and TP=10%

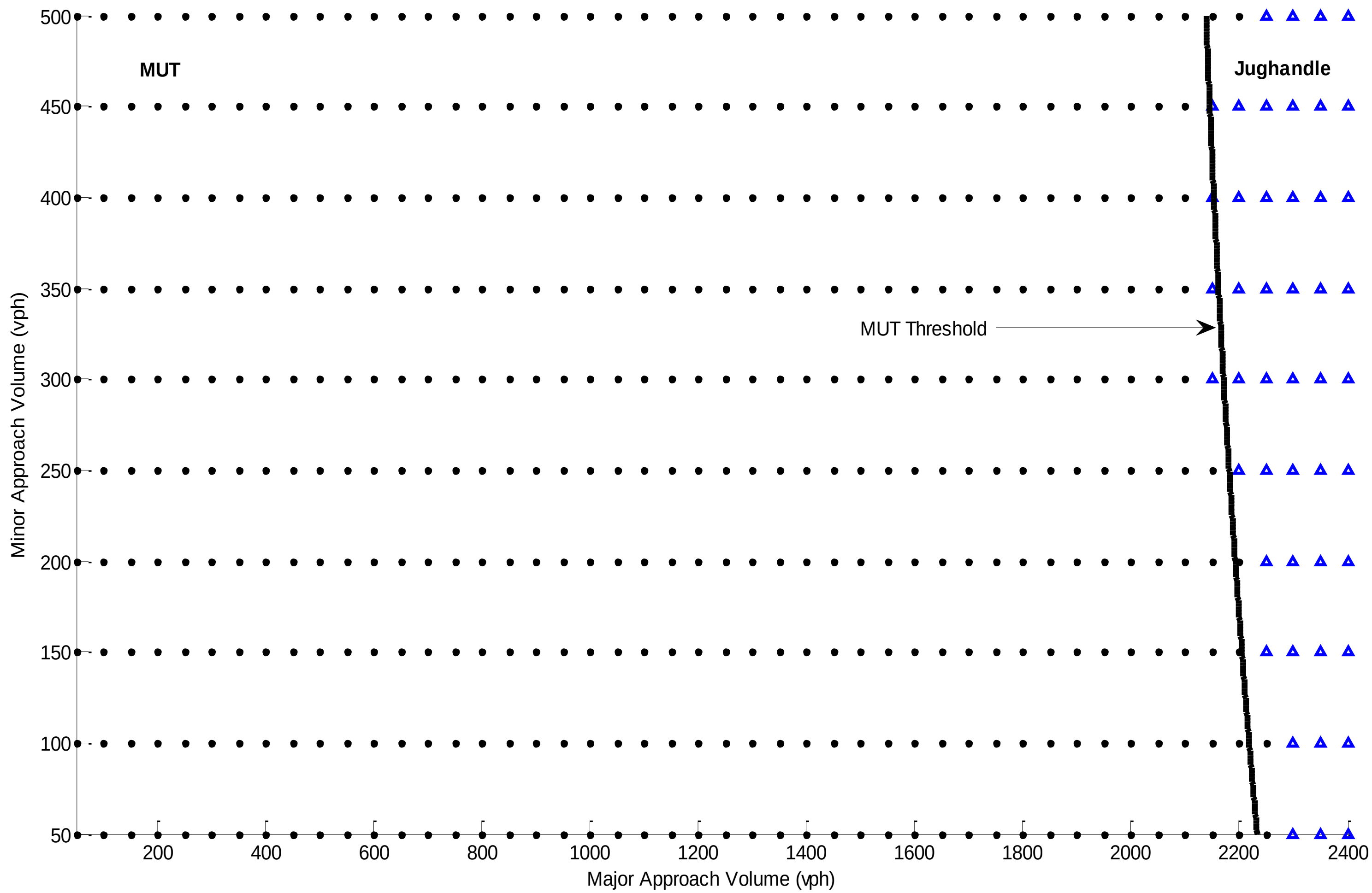


Figure B.22 DAC for unconventional intersection at BF=0.5, LTP=10% and TP=10%

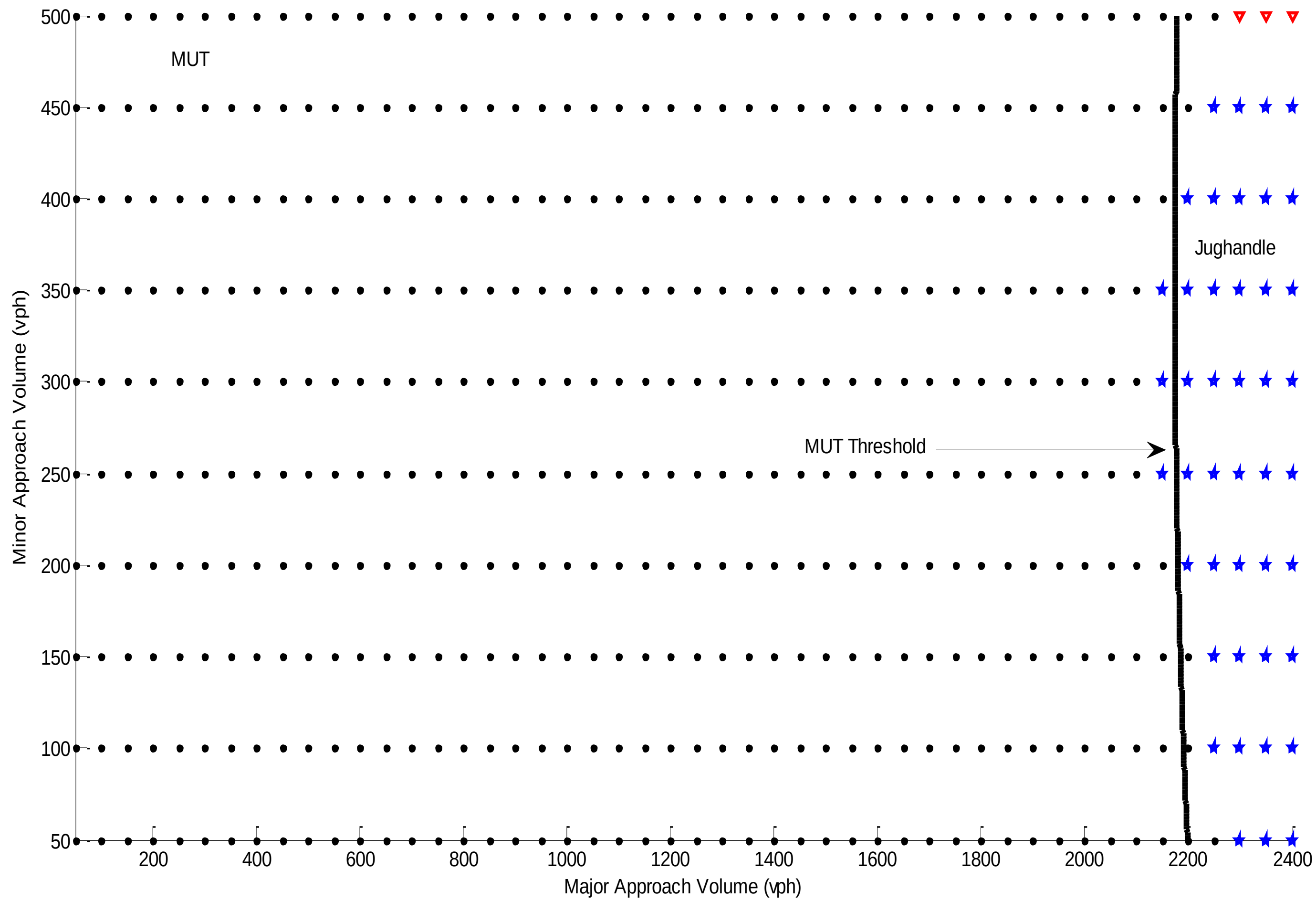


Figure B.23 DAC for unconventional intersection at BF=0.6, LTP=10% and TP=10%

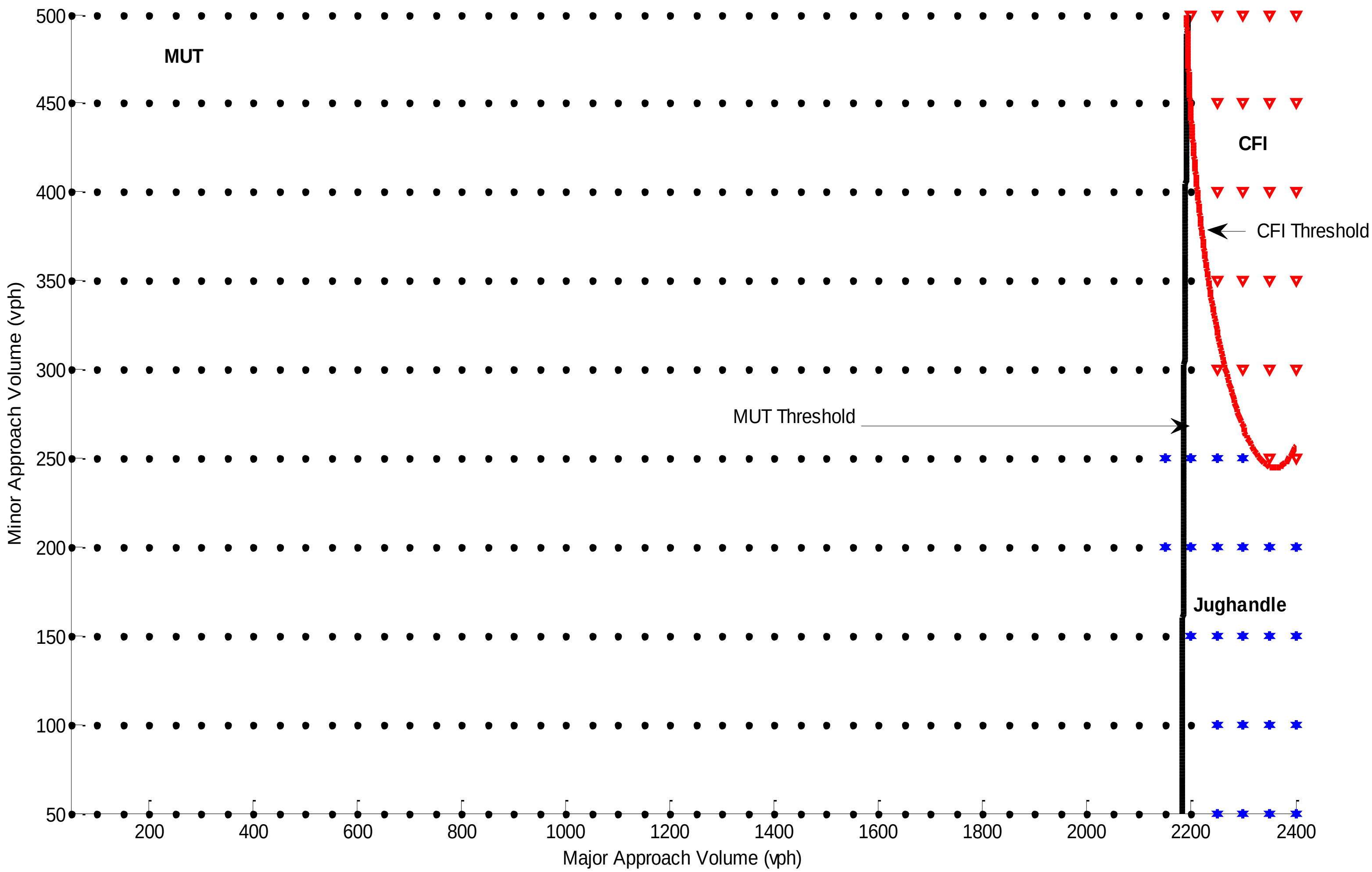


Figure B.24 DAC for unconventional intersection at BF=0.7, LTP=10% and TP=10%

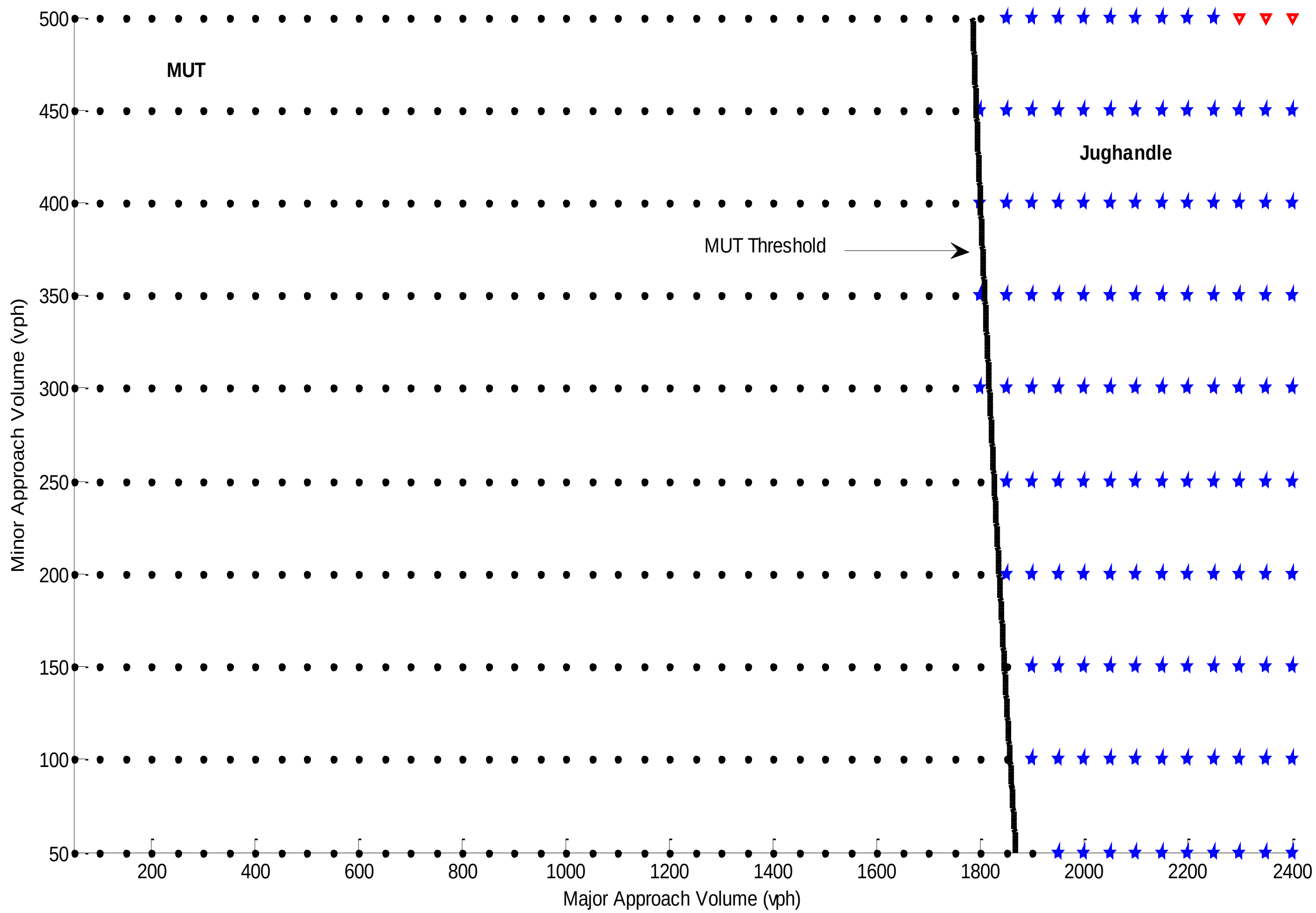


Figure B.25 DAC for unconventional intersection at BF=0.5, LTP=15% and TP=10%

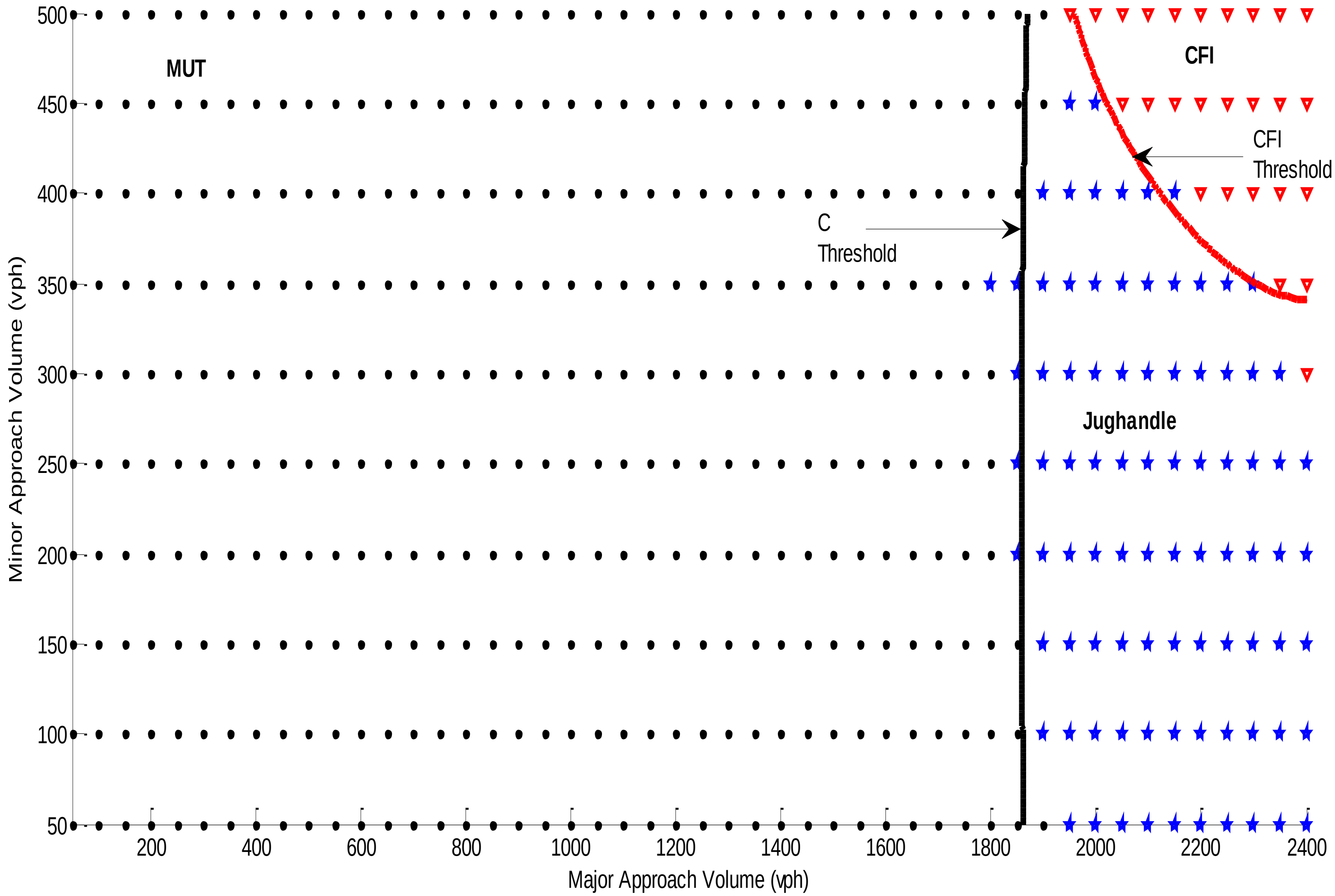


Figure B.26 DAC for unconventional intersection at BF=0.6, LTP=15% and TP=10%

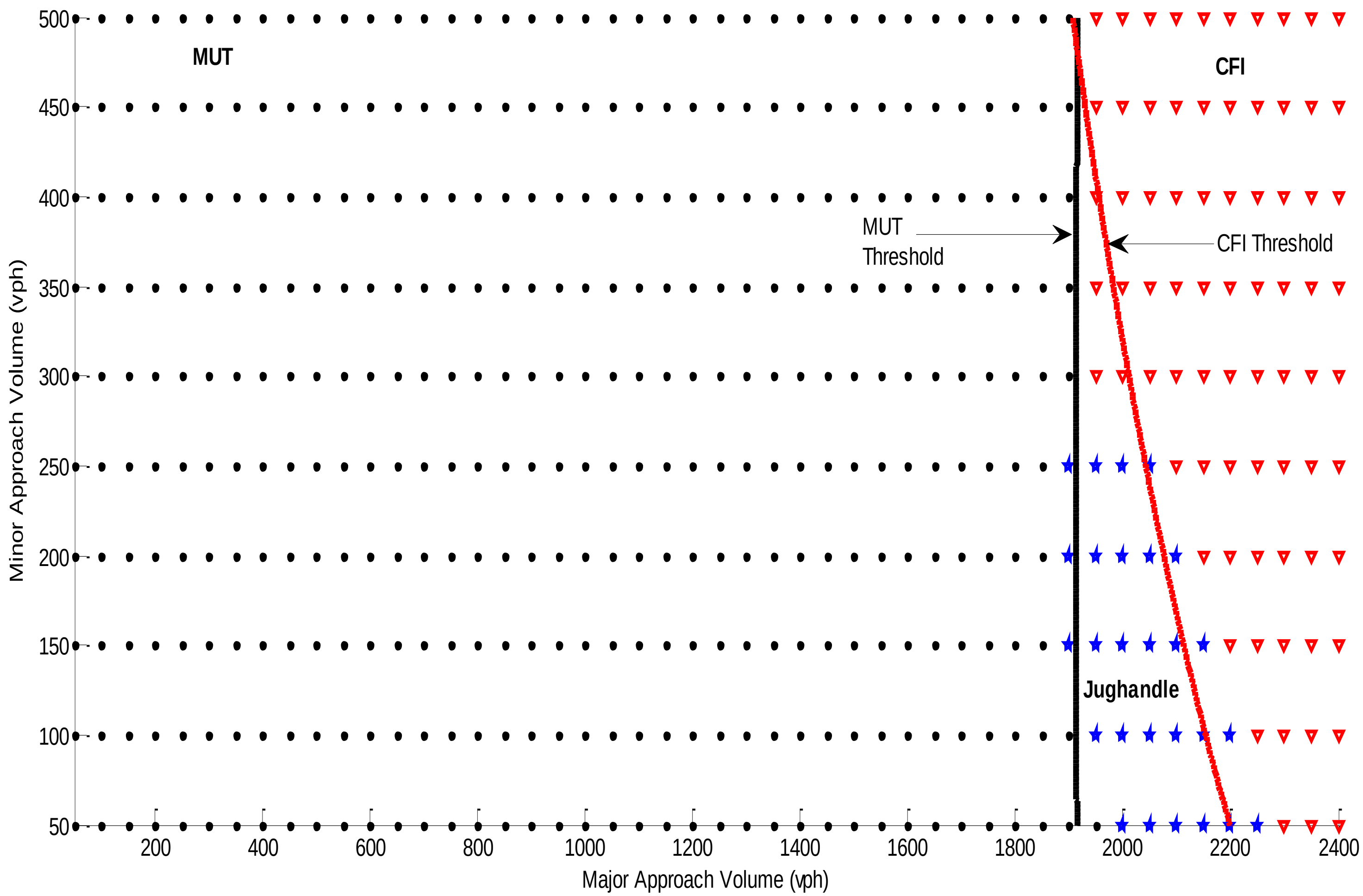


Figure B.27 DAC for unconventional intersection at BF=0.7, LTP=15% and TP=10%

B.2. Contour Plots for MUT

B.2.1 Contour Plots for Critical Left Turn Delay Difference with Standatd Signalized Intersection

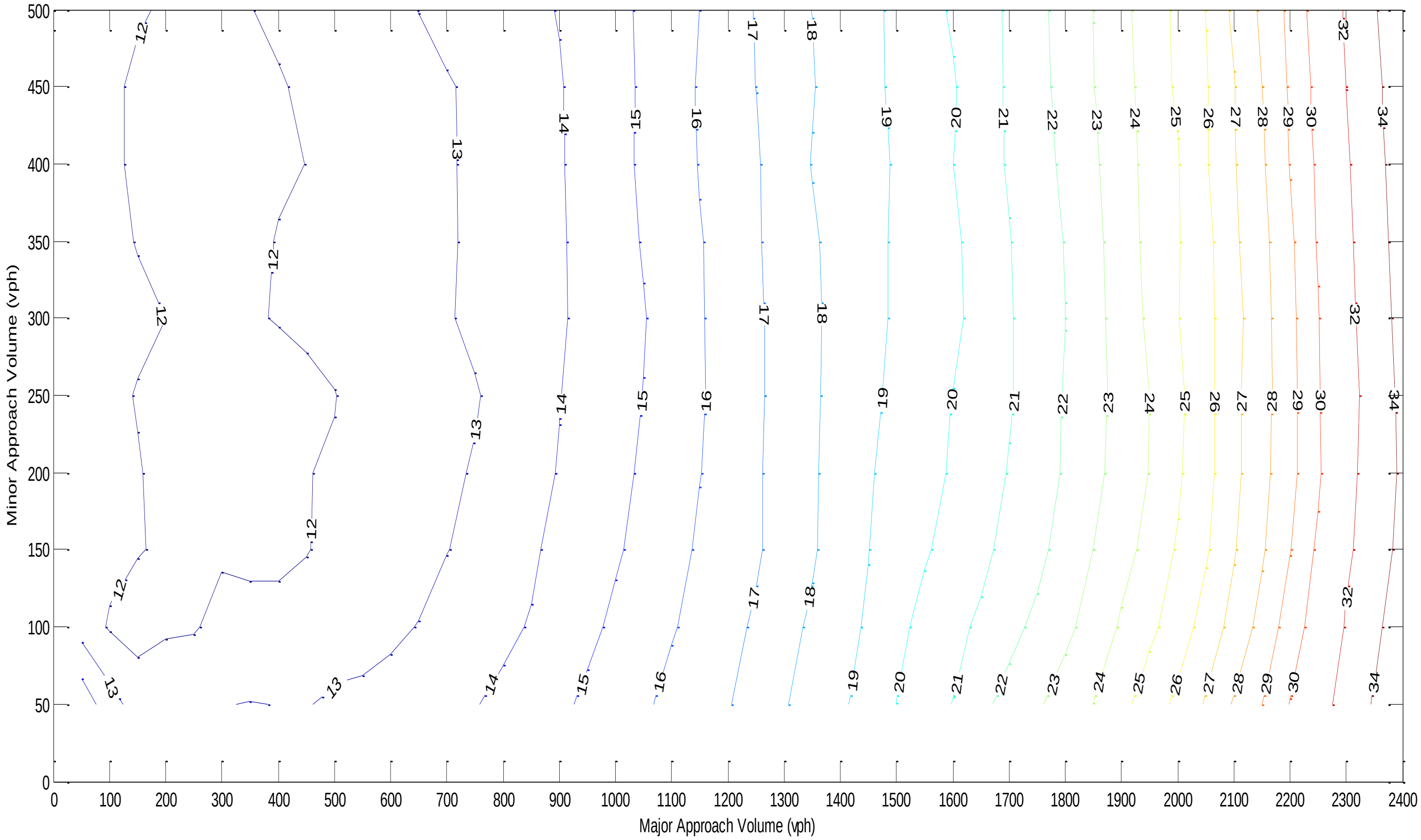


Figure B.28 Contour plot at BF=0.5, LTP=5%, TP=2%

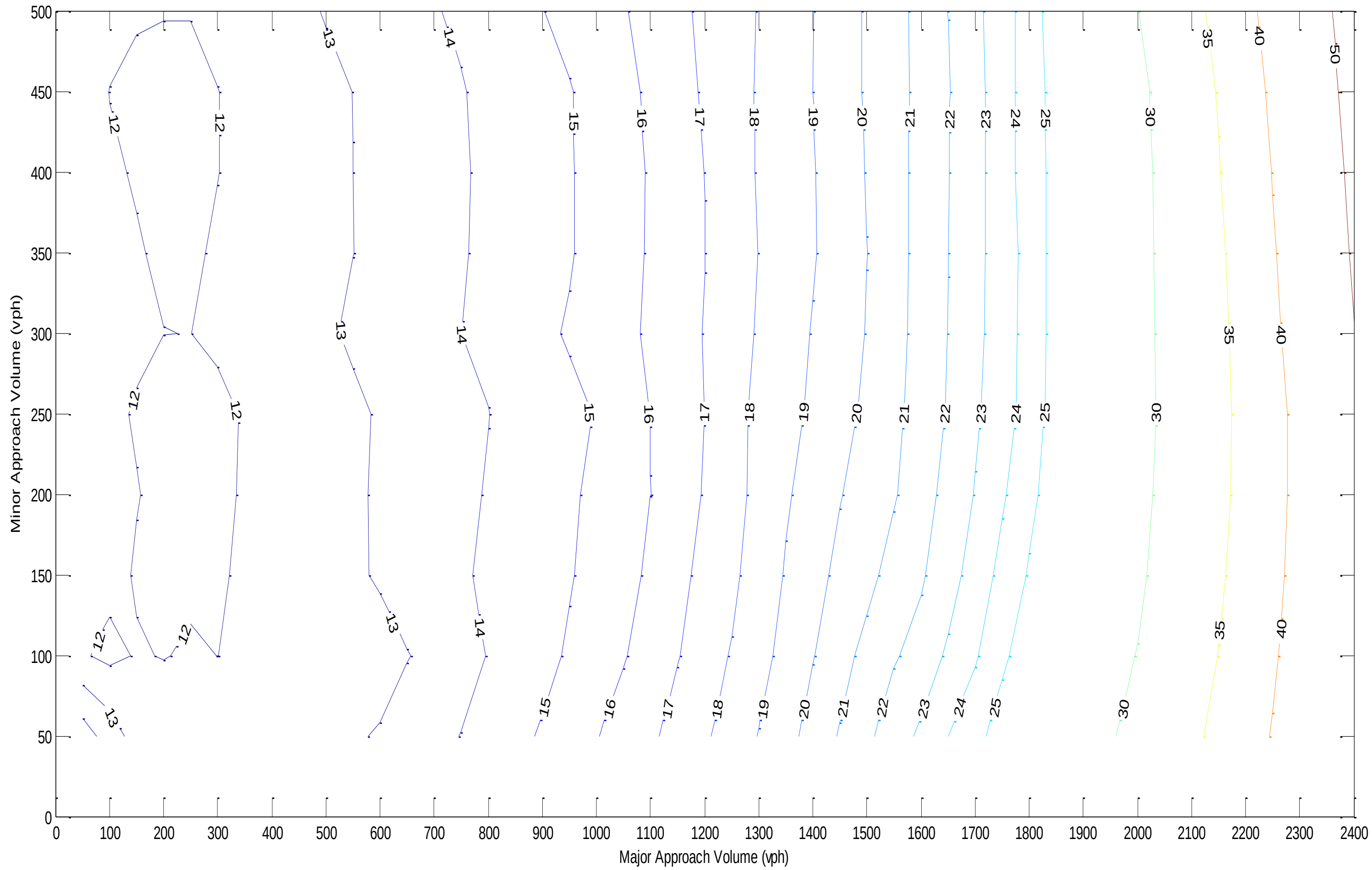


Figure B.29 Contour plot at BF=0.6, LTP=5%, TP=2%

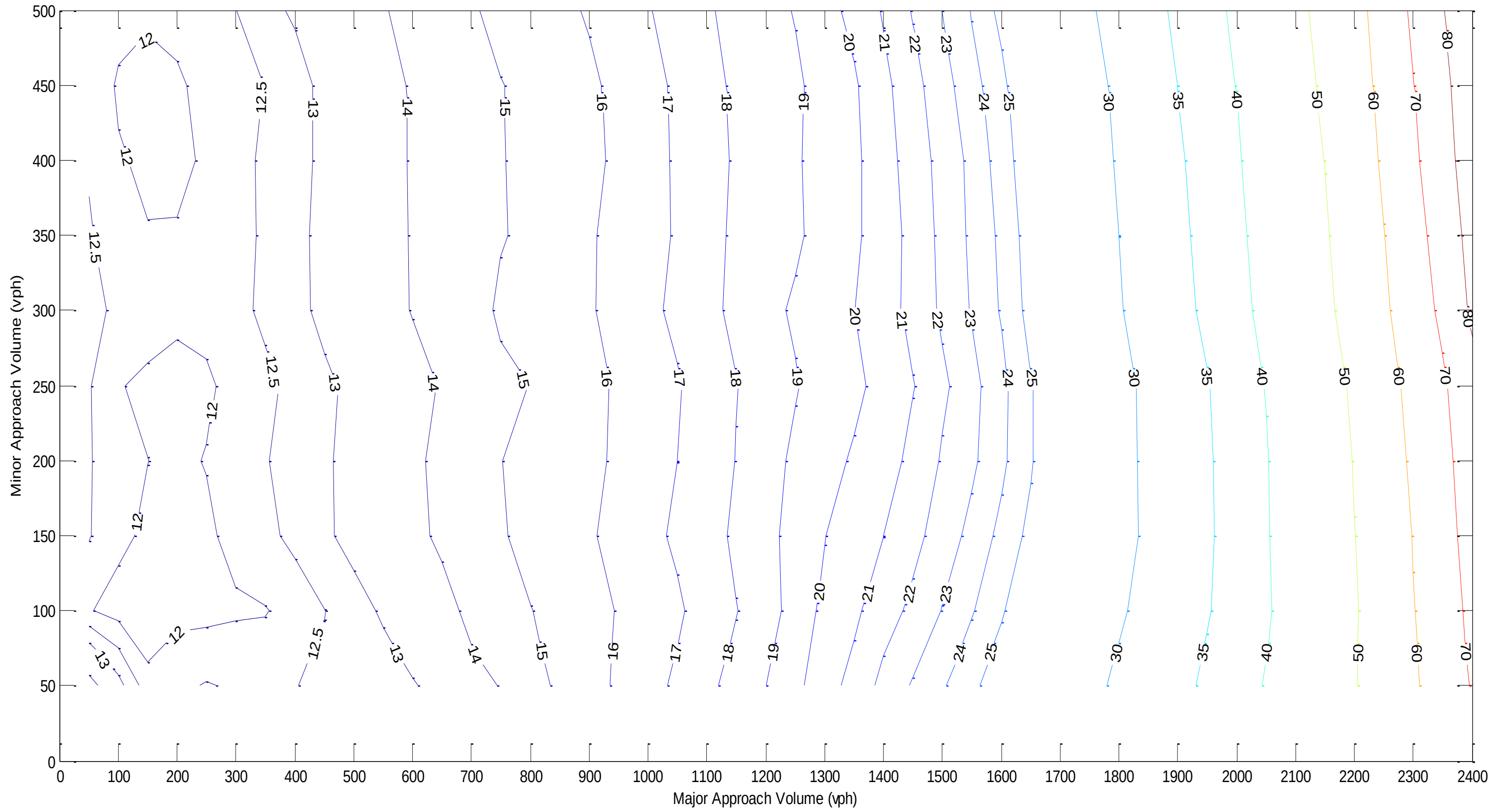


Figure B.30 Contour plot at BF=0.7, LTP=5%, TP=2%

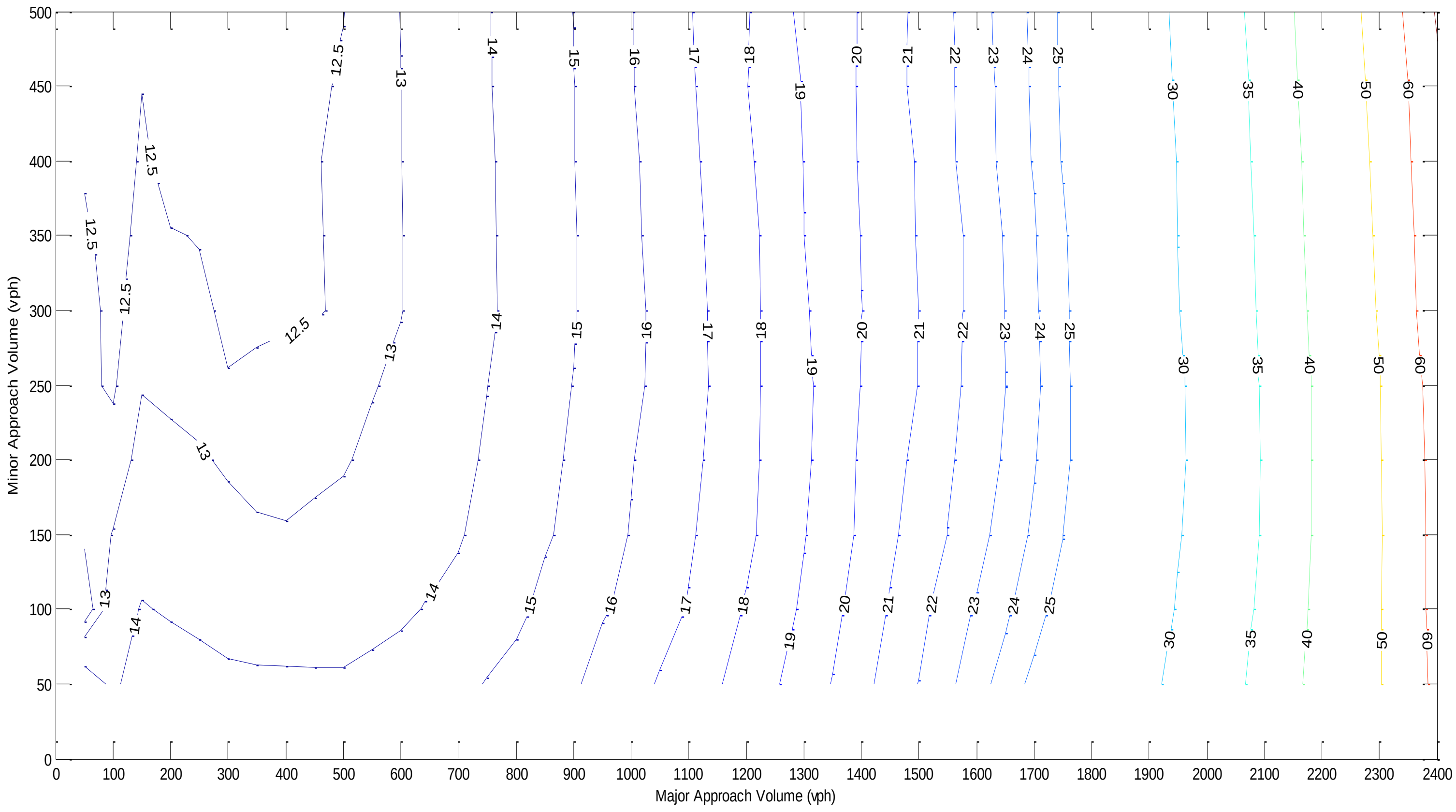


Figure B.31 Contour plot at BF=0.5, LTP=10%, TP=2%

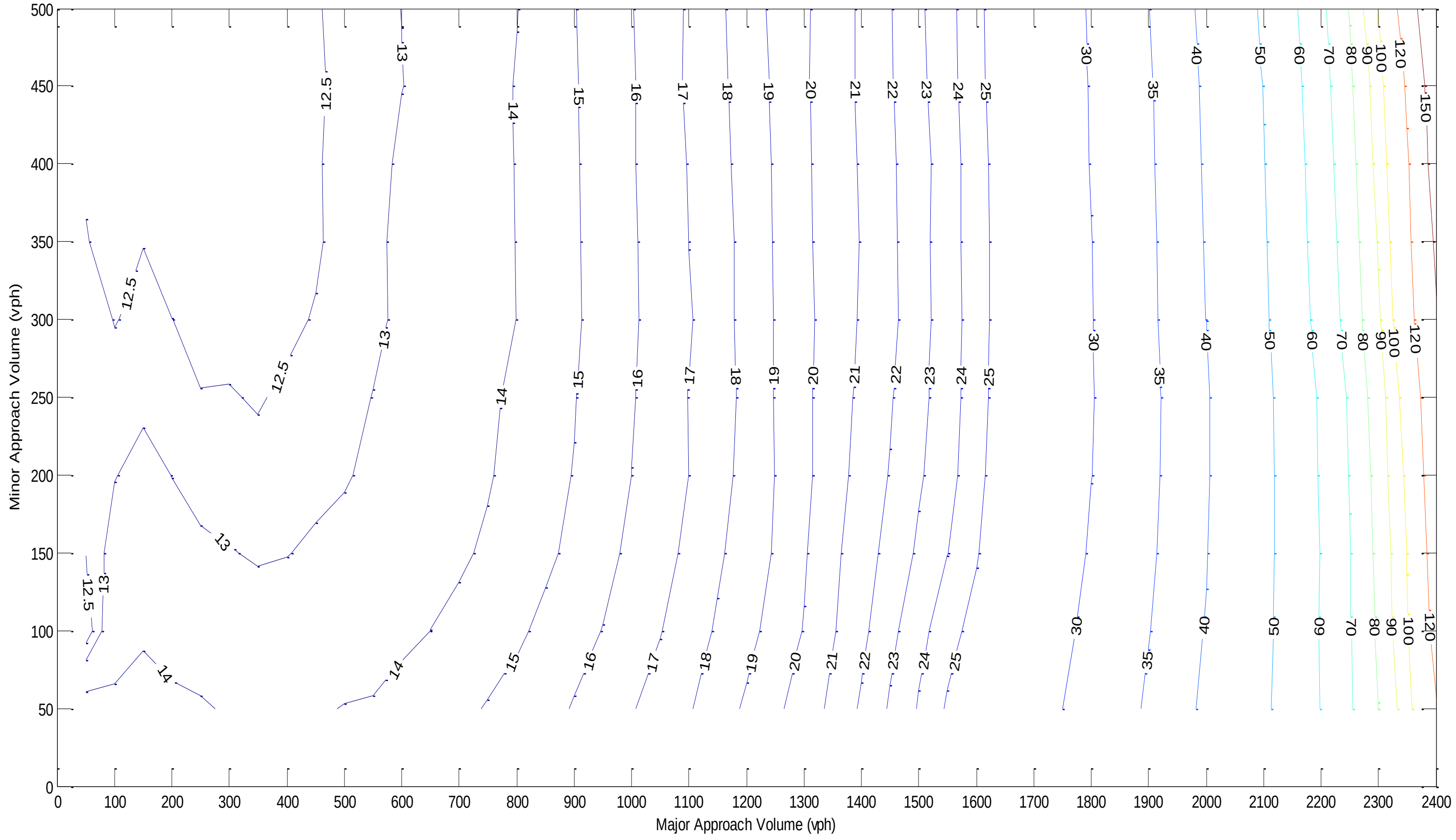


Figure B.32 Contour plot at BF=0.6, LTP=10%, TP=2%

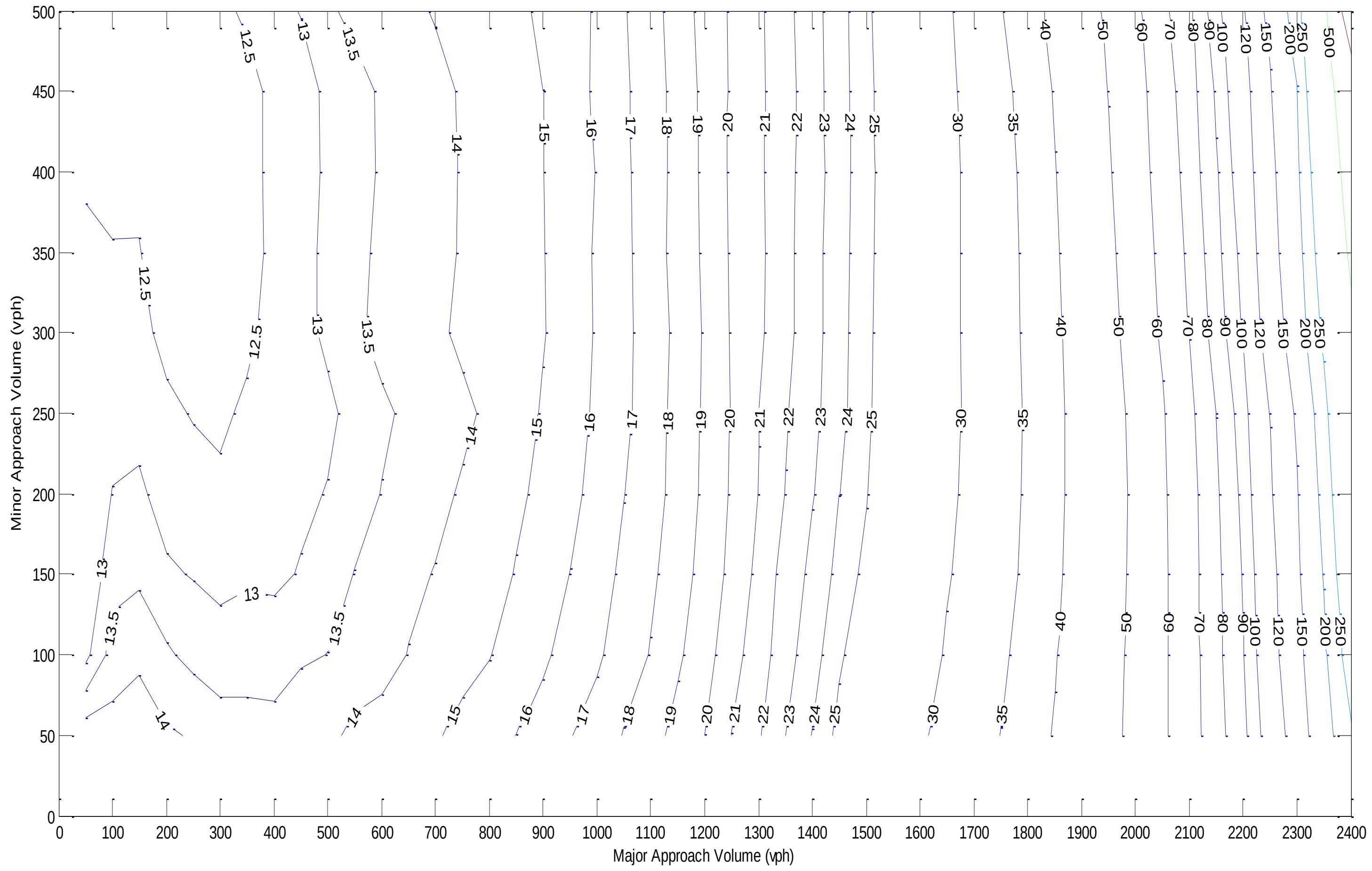


Figure B.33 Contour plot at BF=0.7, LTP=10%, TP=2%

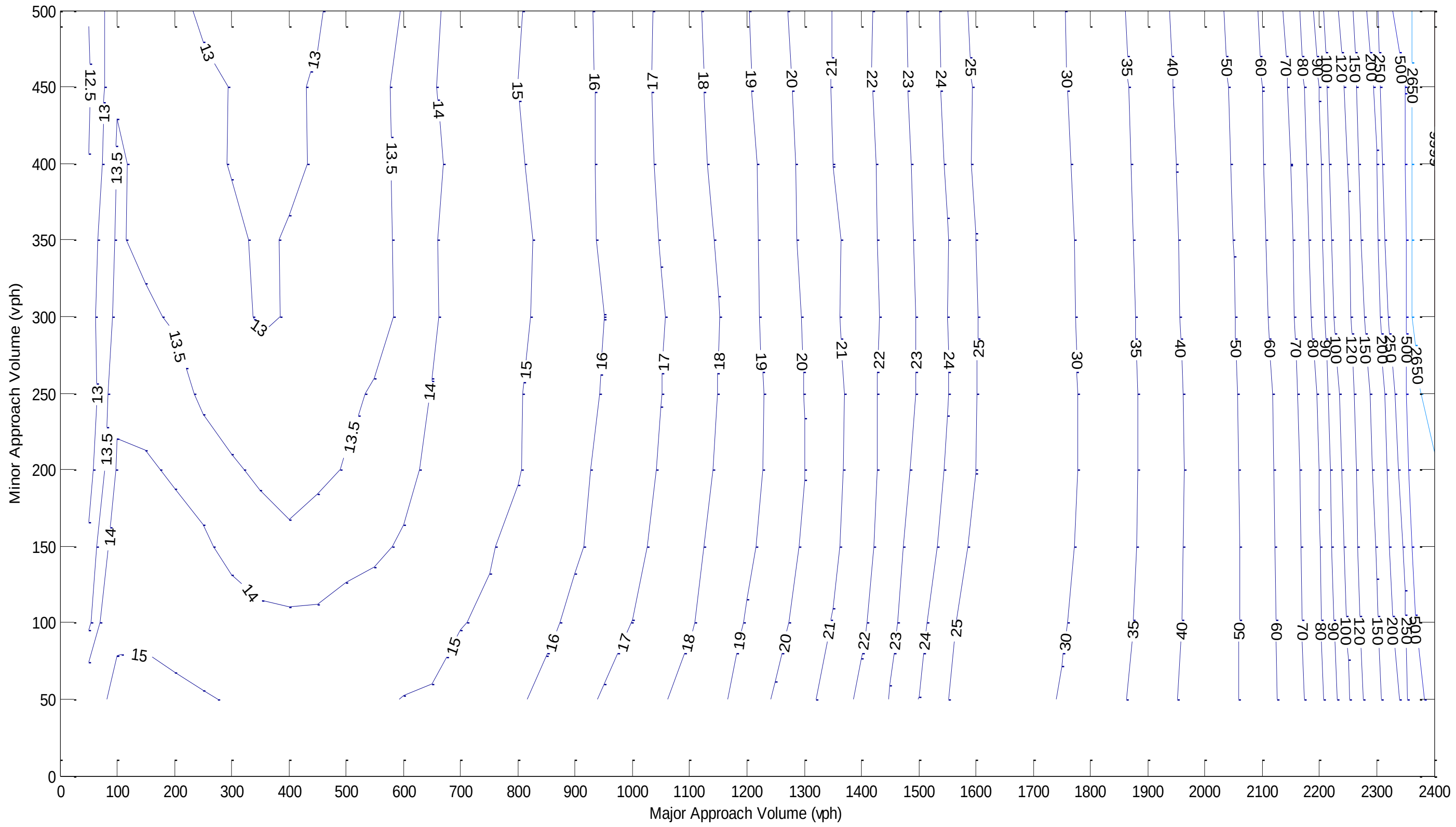


Figure B.34 Contour plot at BF=0.5, LTP=15%, TP=2%

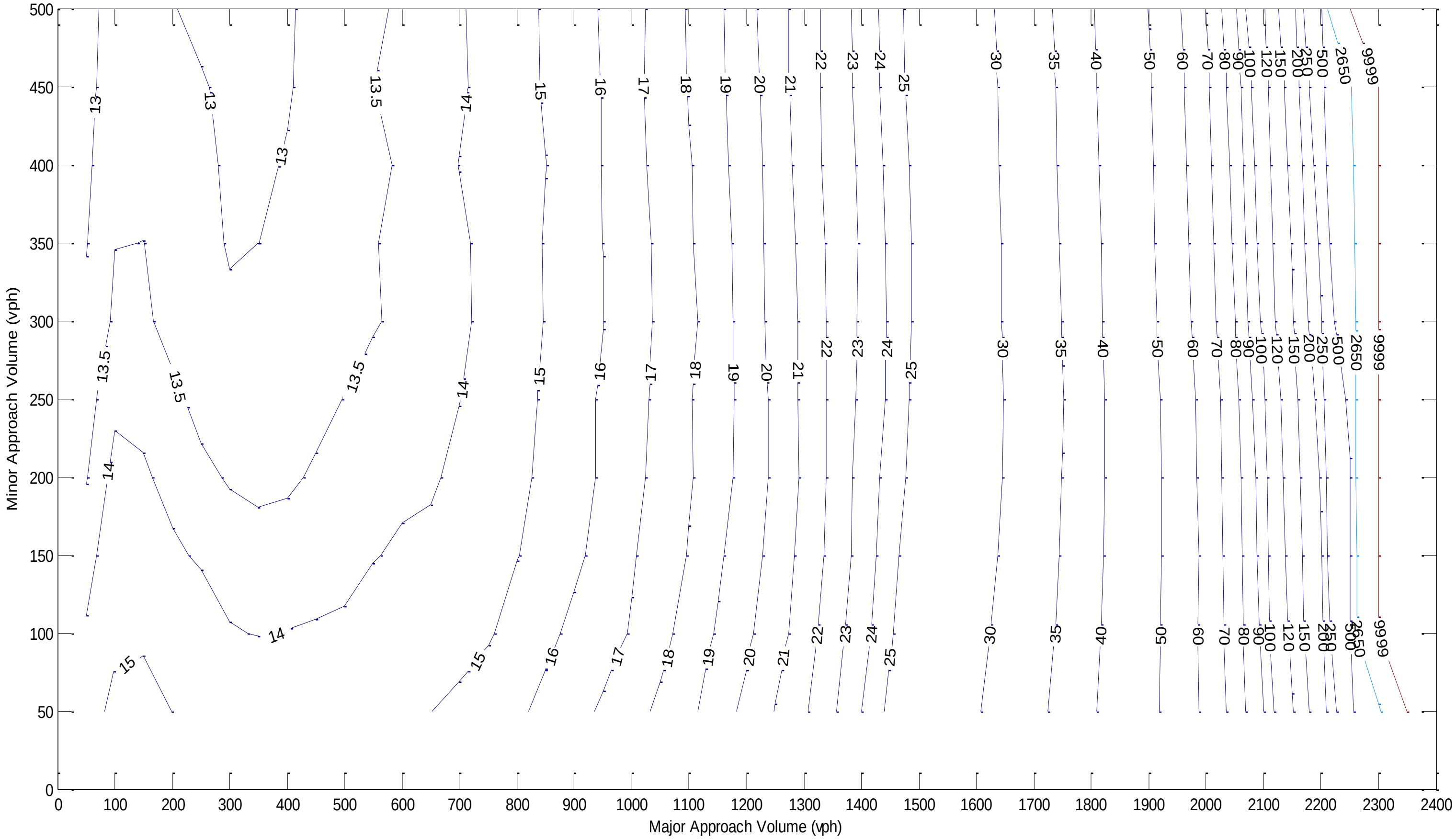


Figure B.35 Contour plot at BF=0.6, LTP=15%, TP=2%

(Note: 9999 represents failure condition of Median Opening)

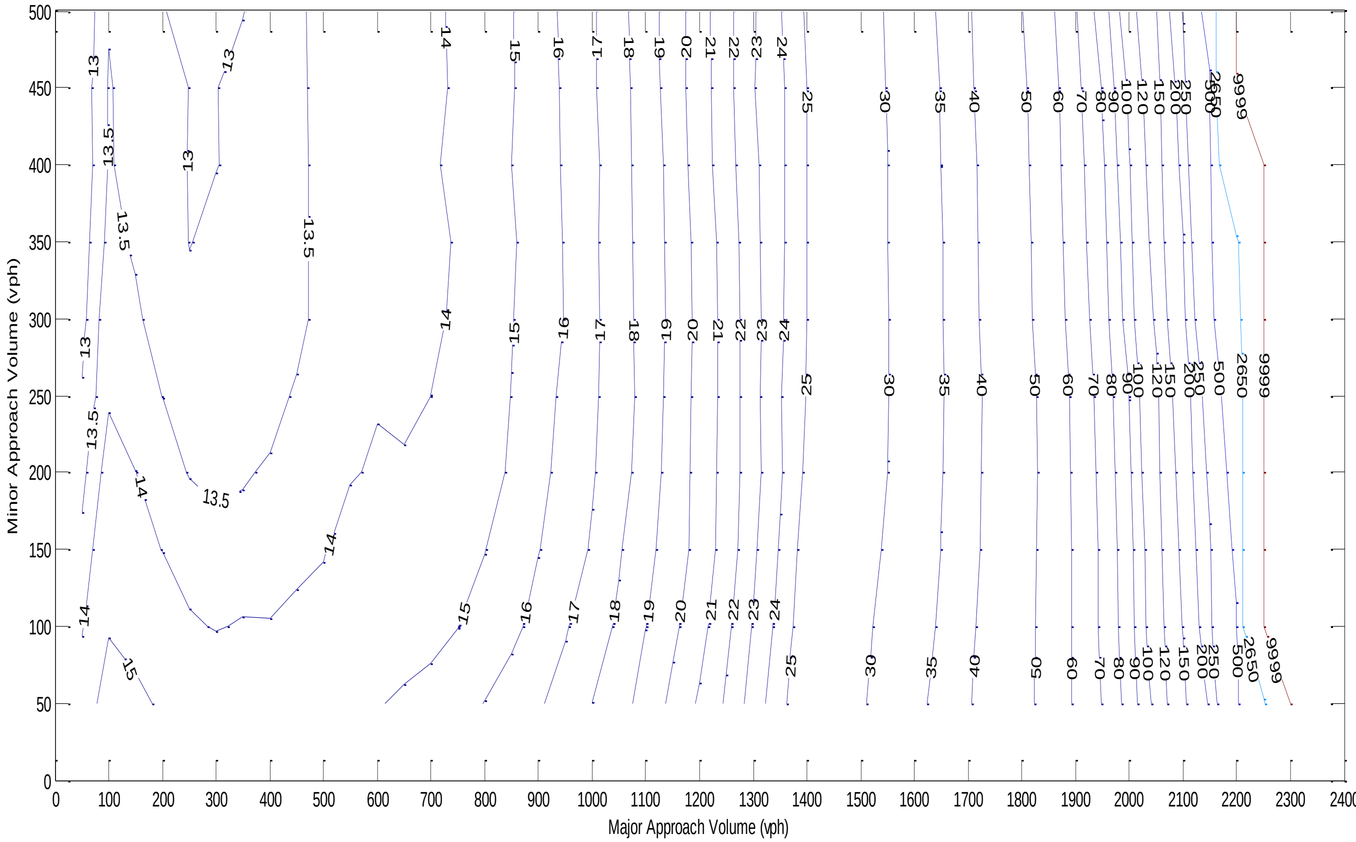


Figure B.36 Contour plot at BF=0.7, LTP=15%, TP=2%

(Note: 9999 represents failure condition of Median Opening)

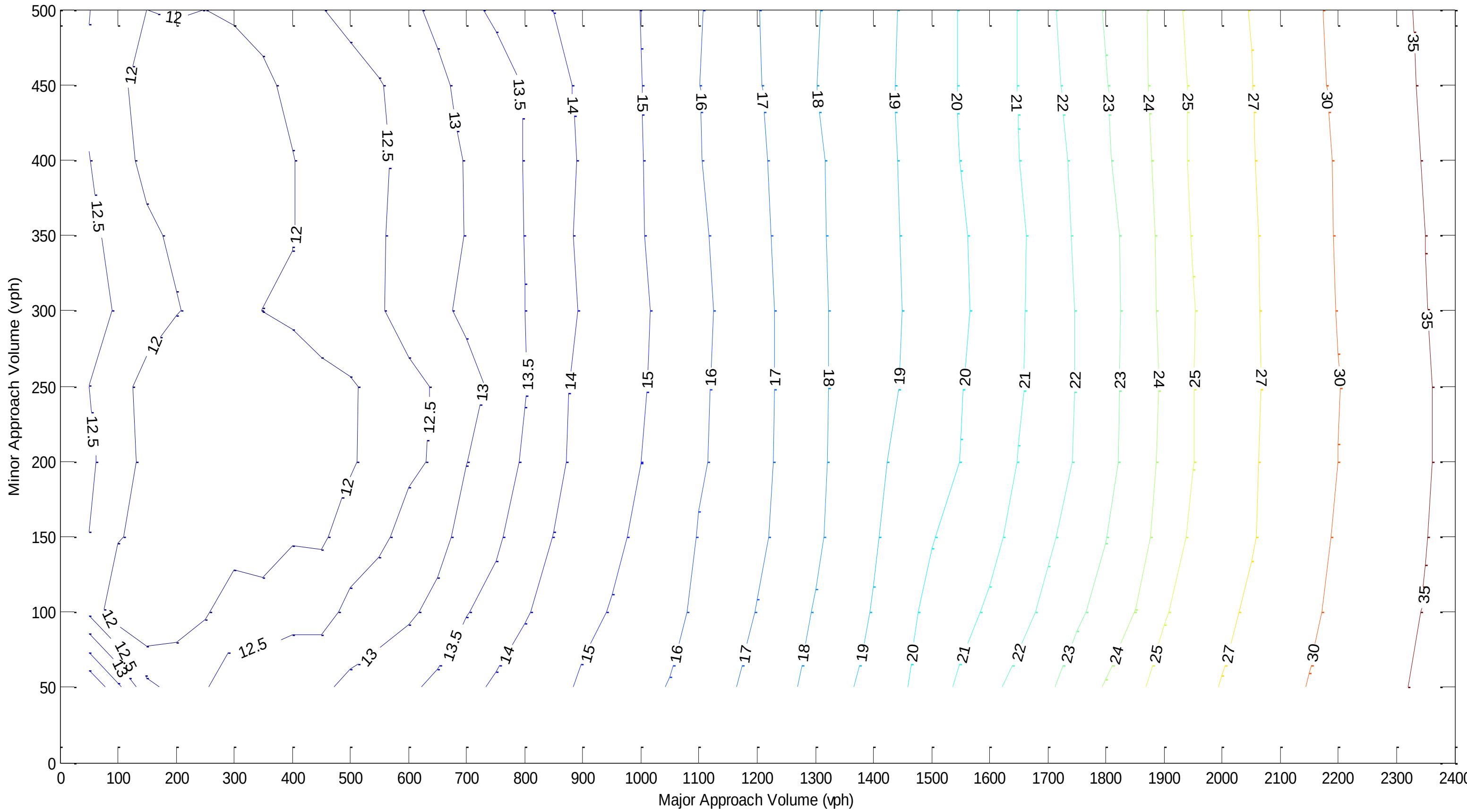


Figure B.37 Contour plot at BF=0.5, LTP=5 %, TP=5 %

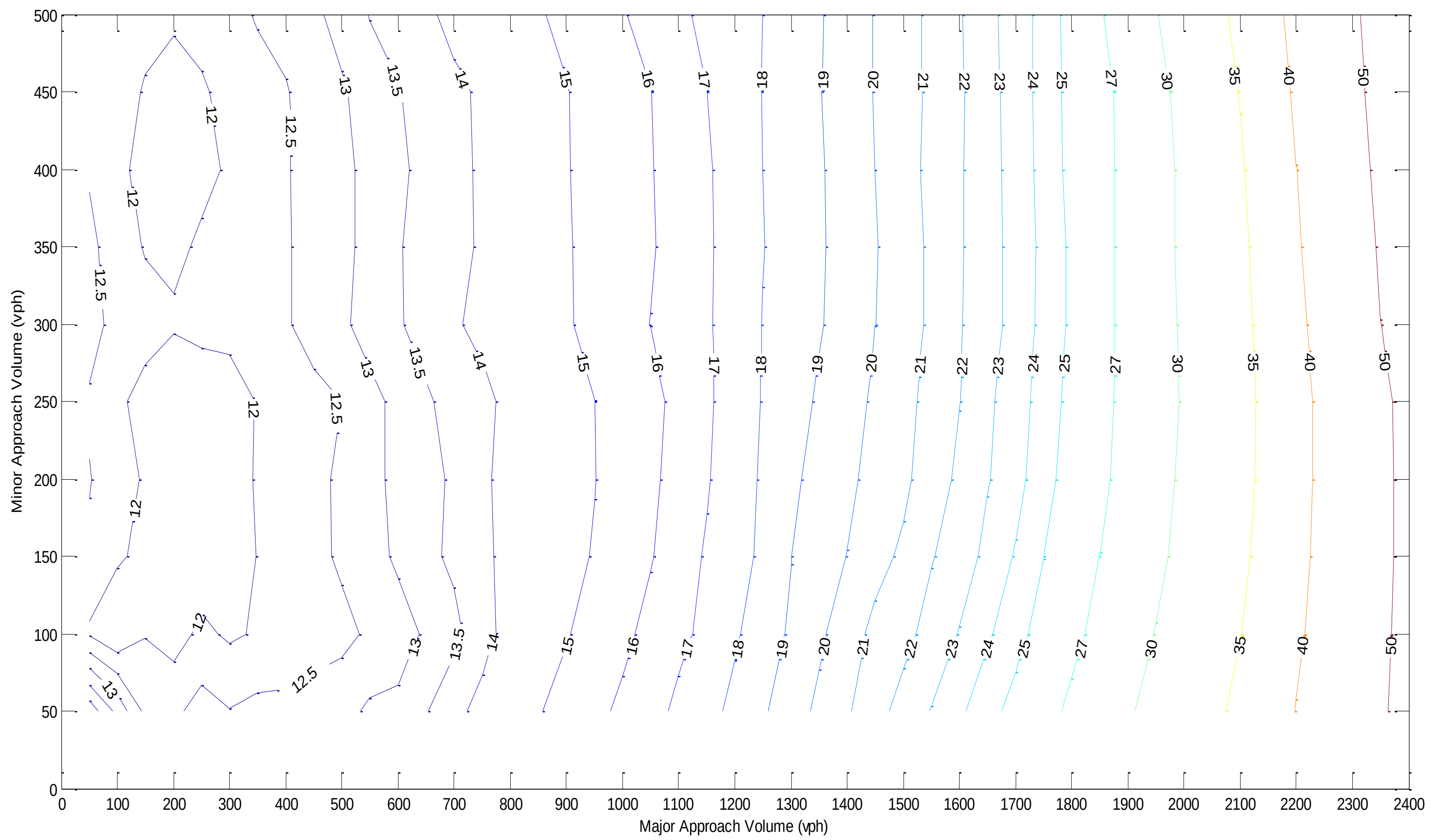


Figure B.38 Contour plot at BF=0.6, LTP=5 %, TP=5 %

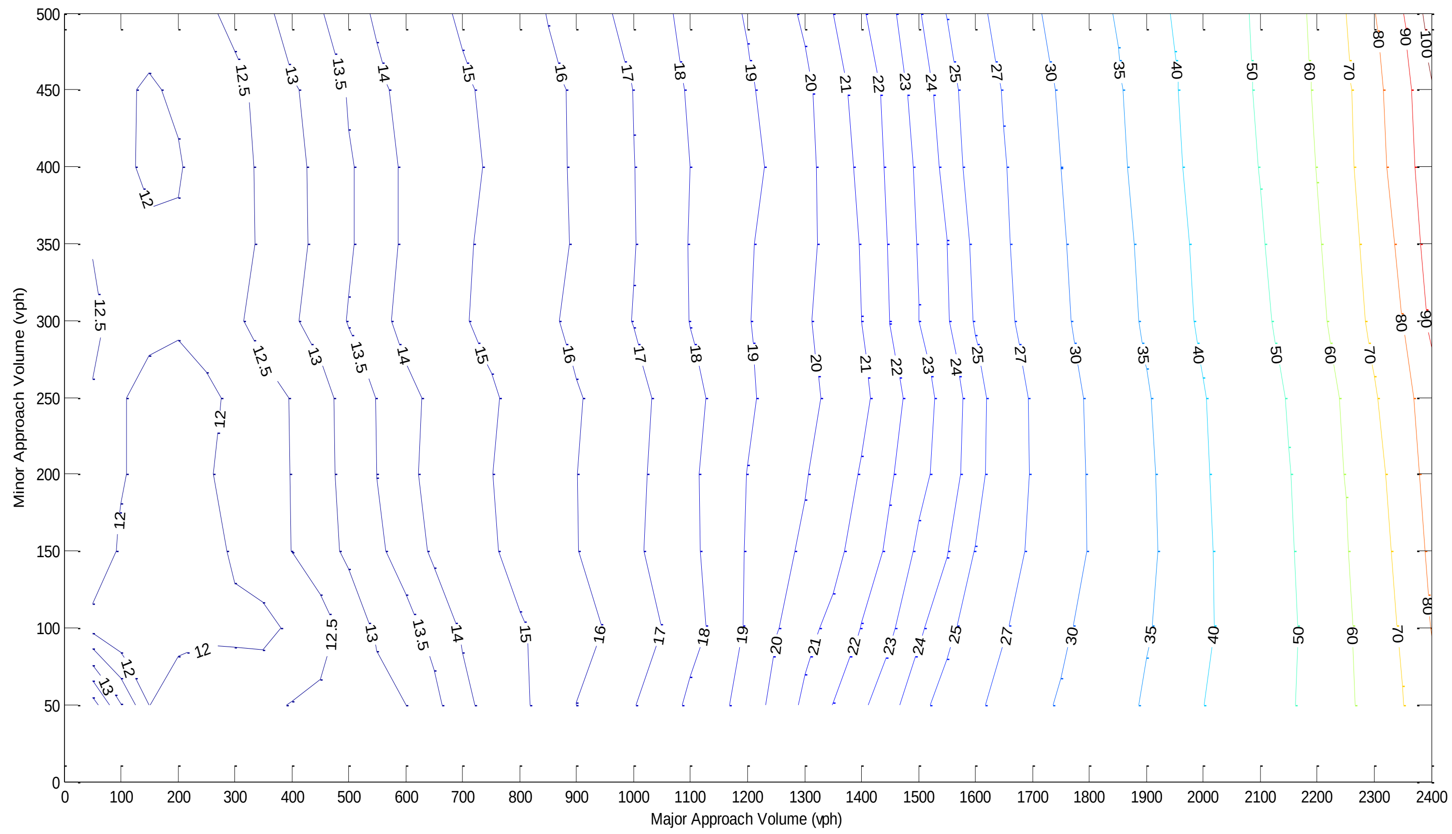


Figure B.39 Contour plot at BF=0.7, LTP=5 %, TP=5 %

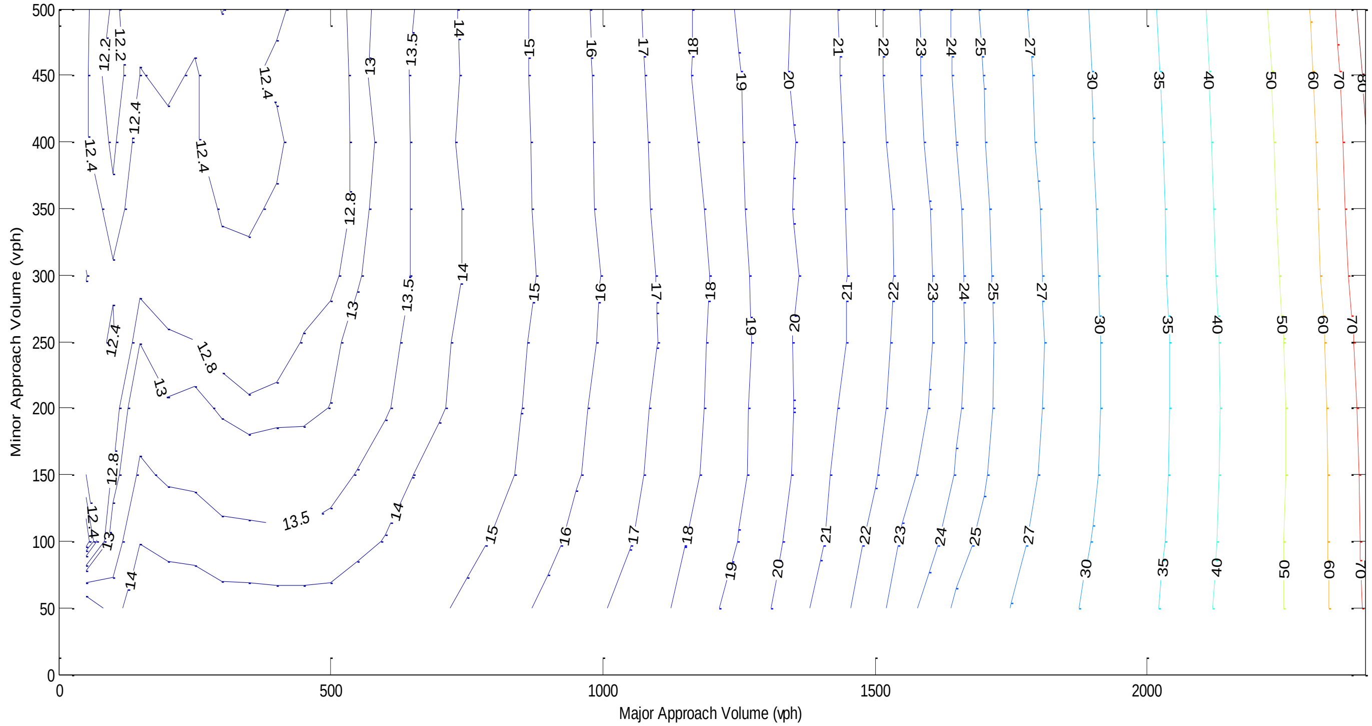


Figure B.40 Contour plot at BF=0.5, LTP=10 %, TP=5 %

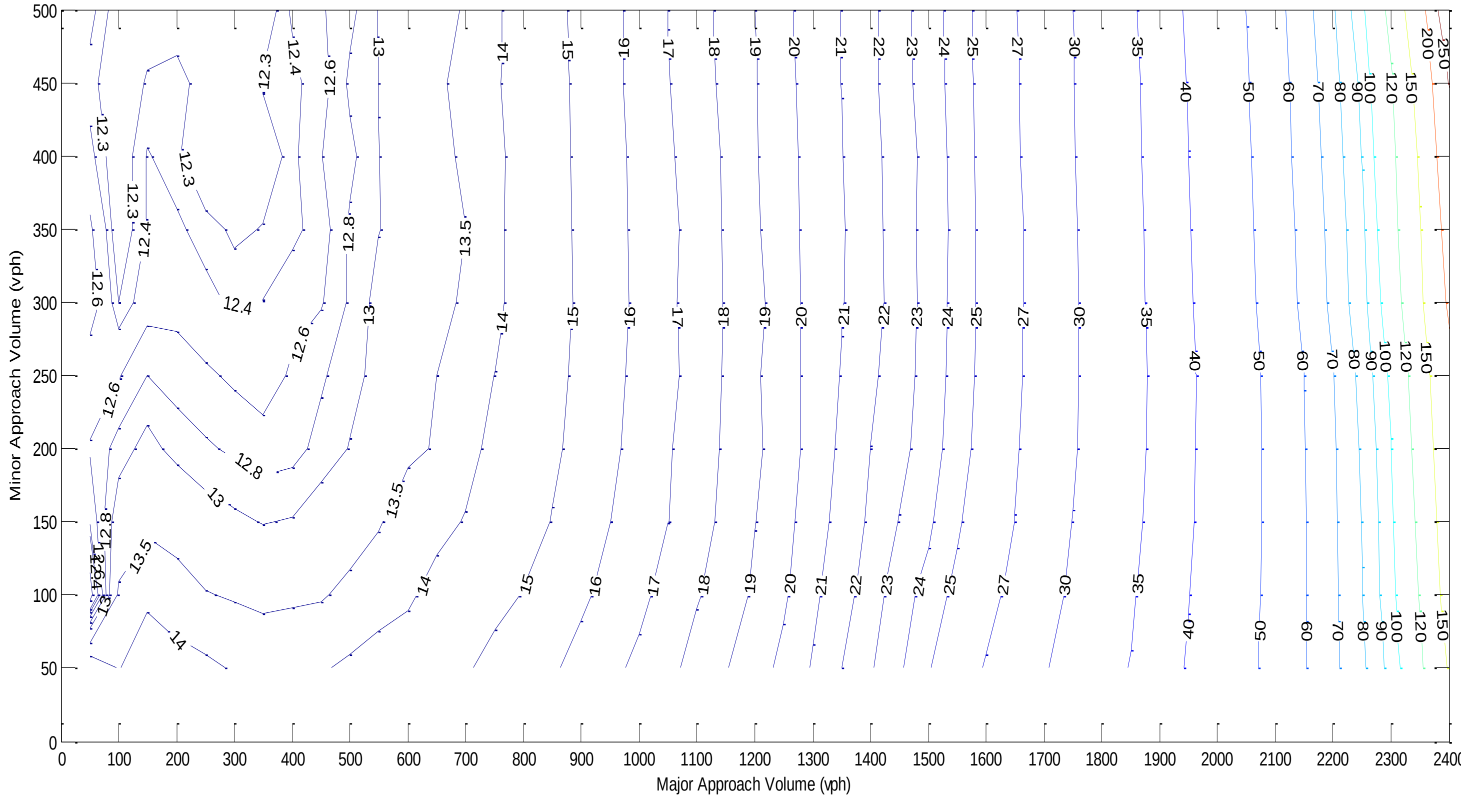


Figure B.41 Contour plot at BF=0.6, LTP=10 %, TP=5 %

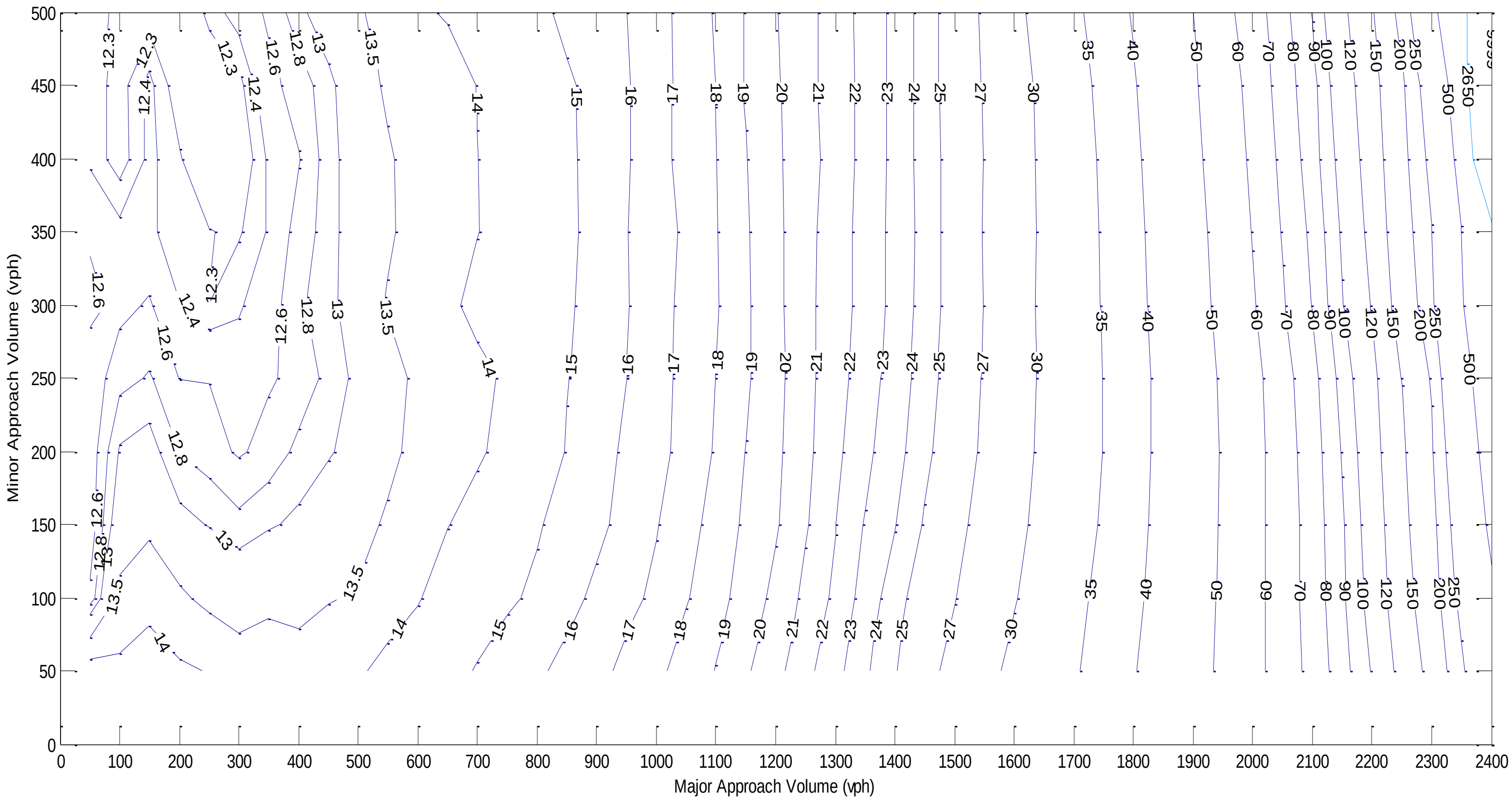


Figure B.42 Contour plot at BF=0.7, LTP=10 %, TP=5 %

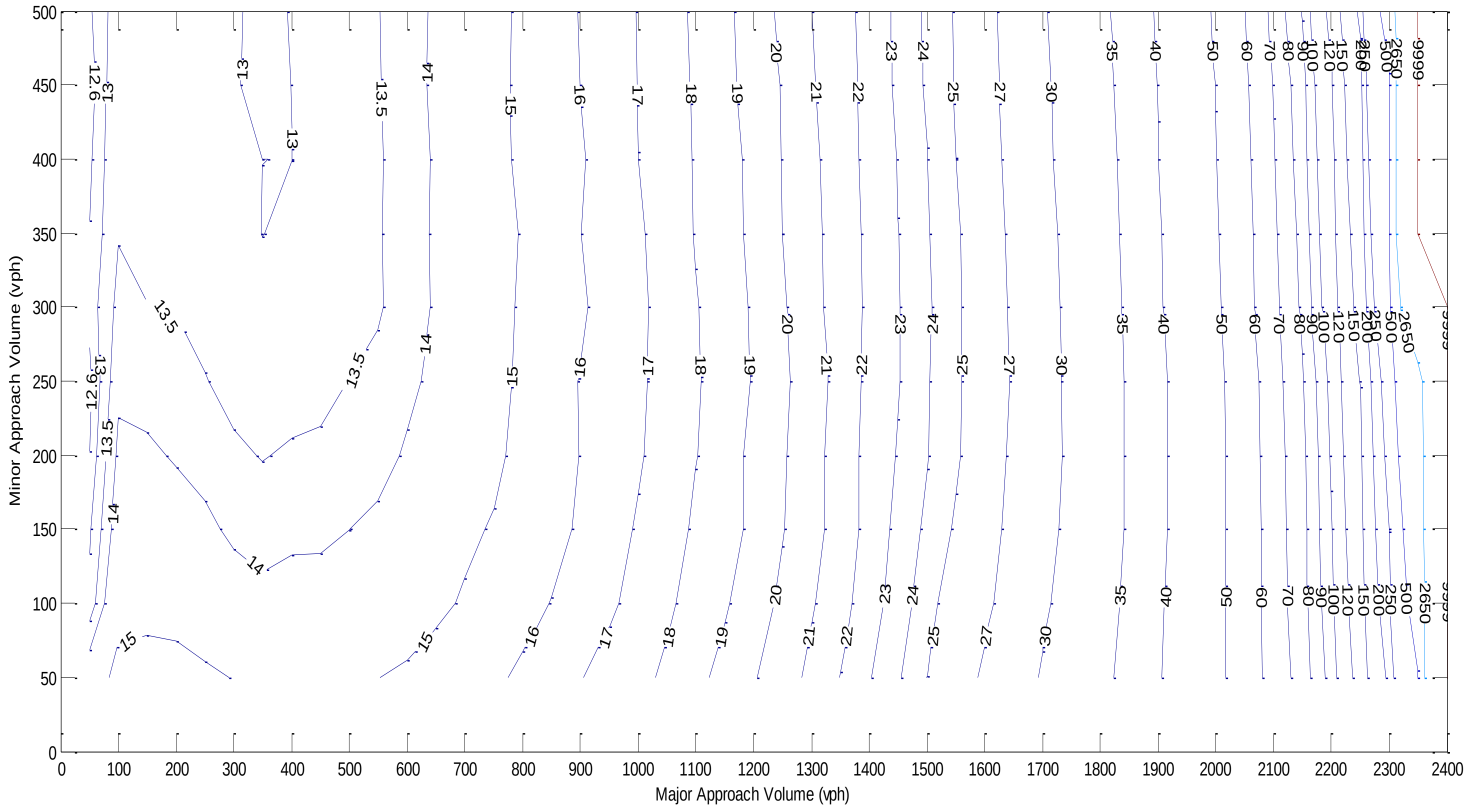


Figure B.43 Contour plot at BF=0.5, LTP=15 %, TP=5 %

Note: 9999 represents failure condition of Median Opening

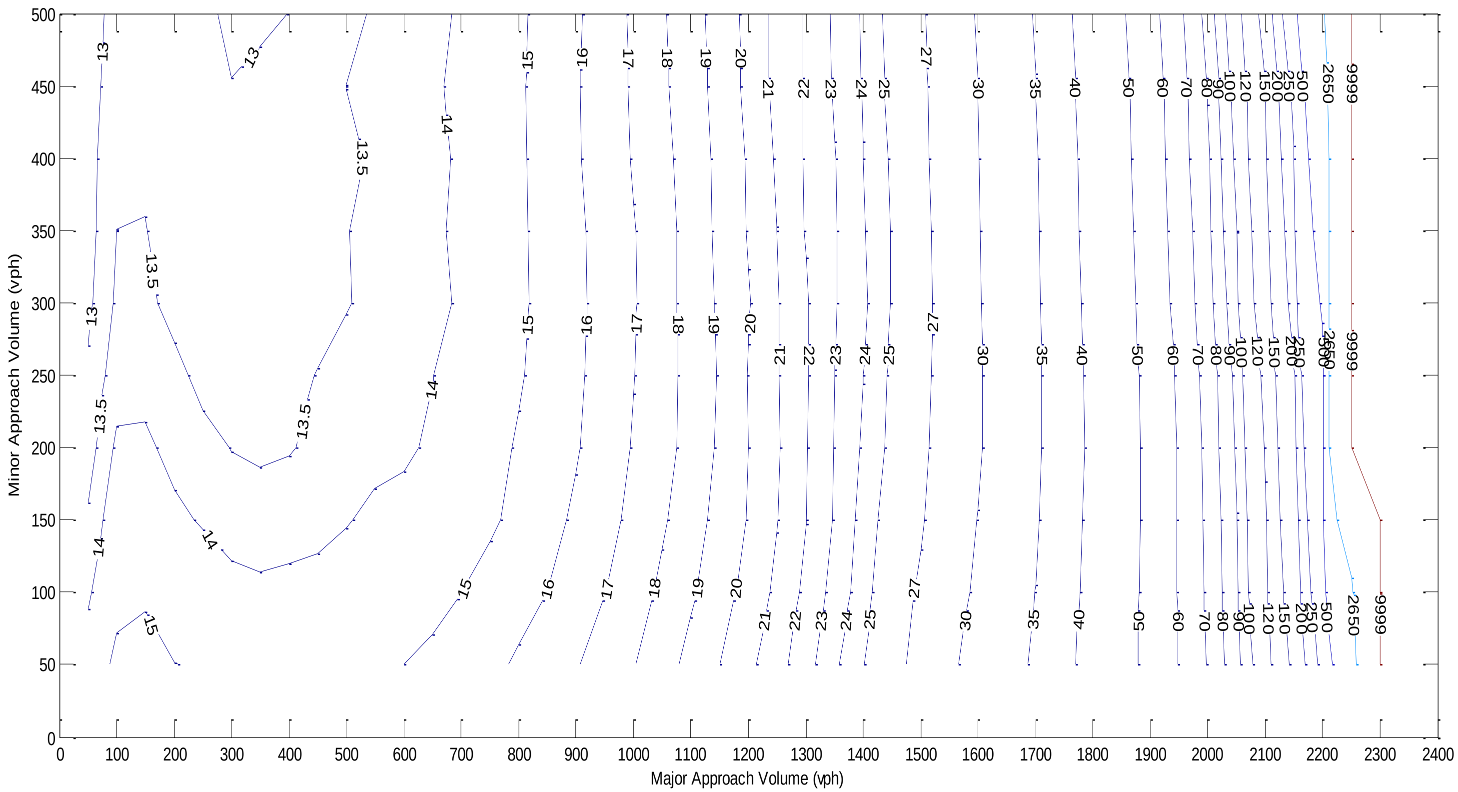


Figure B.44 Contour plot at BF=0.6, LTP=15 %, TP=5 %

(Note: 9999 represents failure condition of Median Opening)

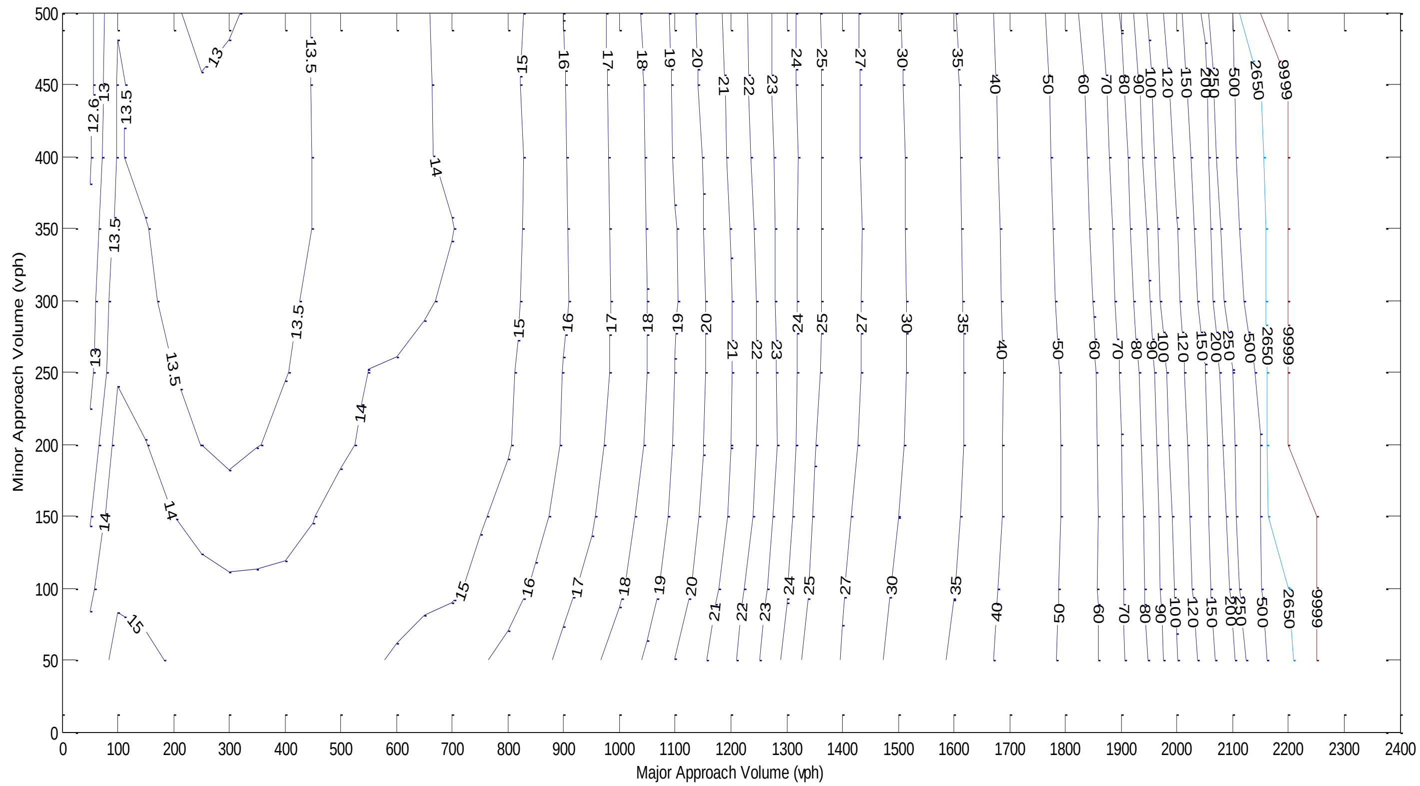


Figure B.45 Contour plot at BF=0.7, LTP=15 %, TP=5 %

(Note: 9999 represents failure condition of Median Opening)

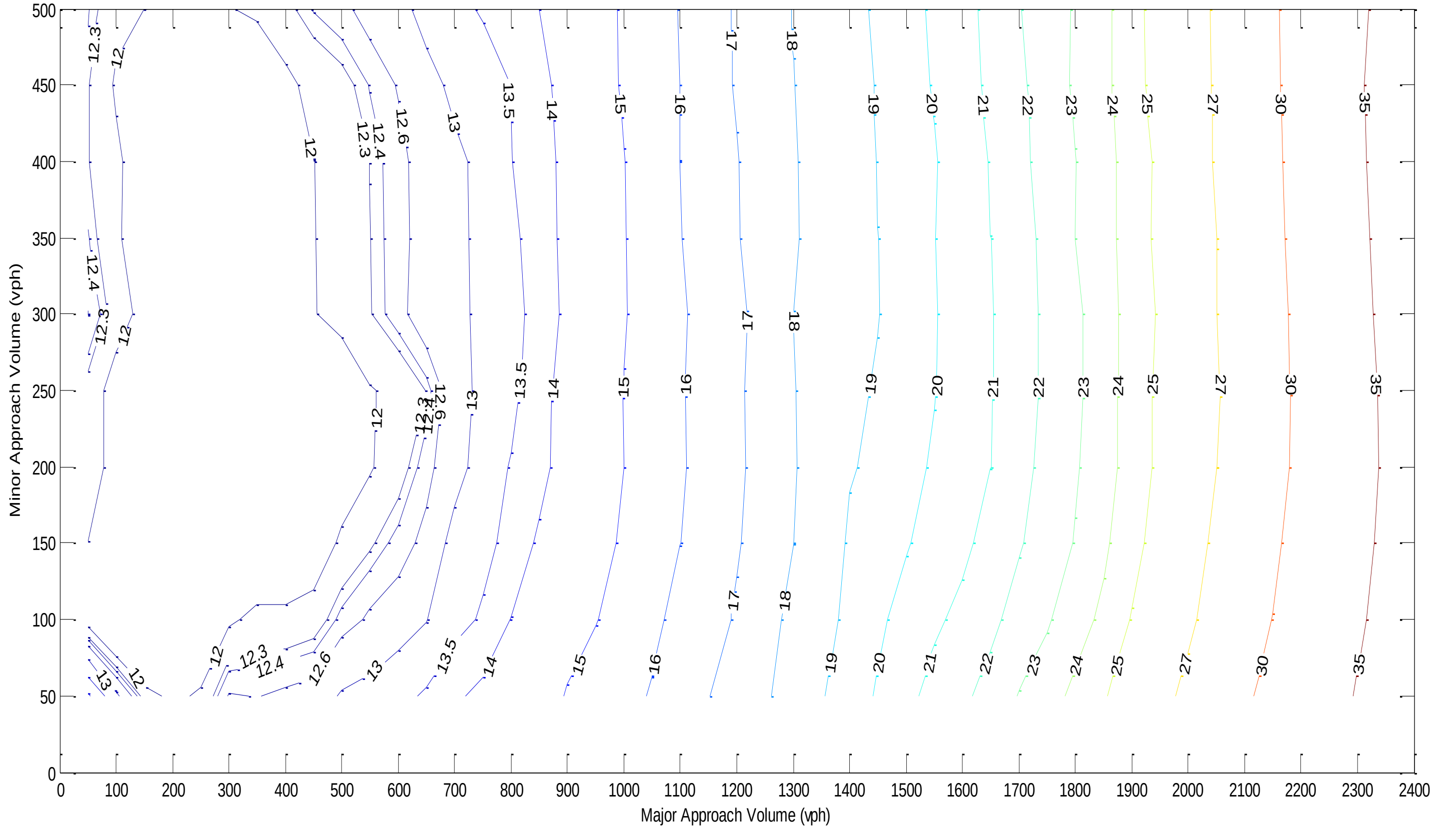


Figure B.46 Contour plot at BF=0.5, LTP=5 %, TP=10 %

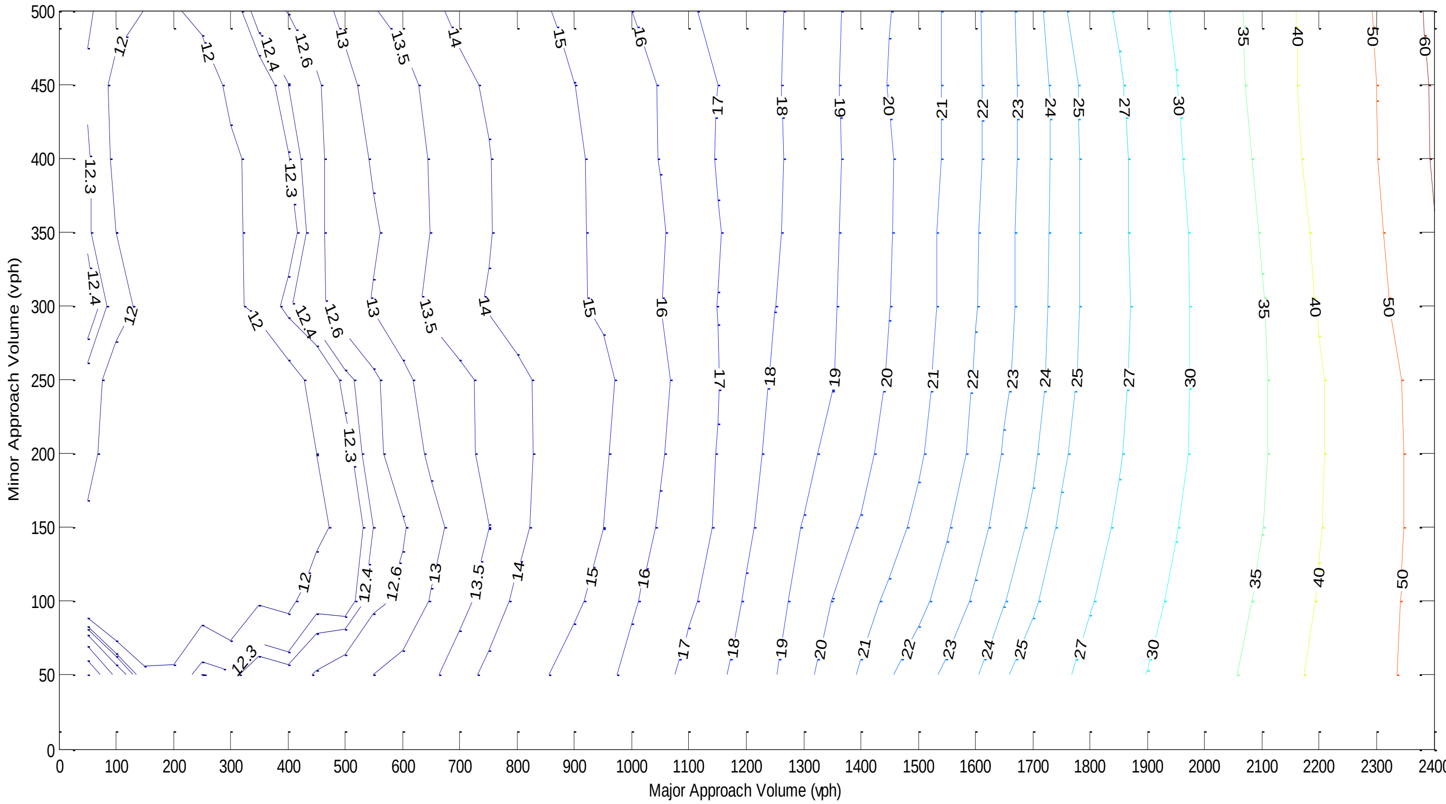


Figure B.47 Contour plot at BF=0.6, LTP=5 %, TP=10 %

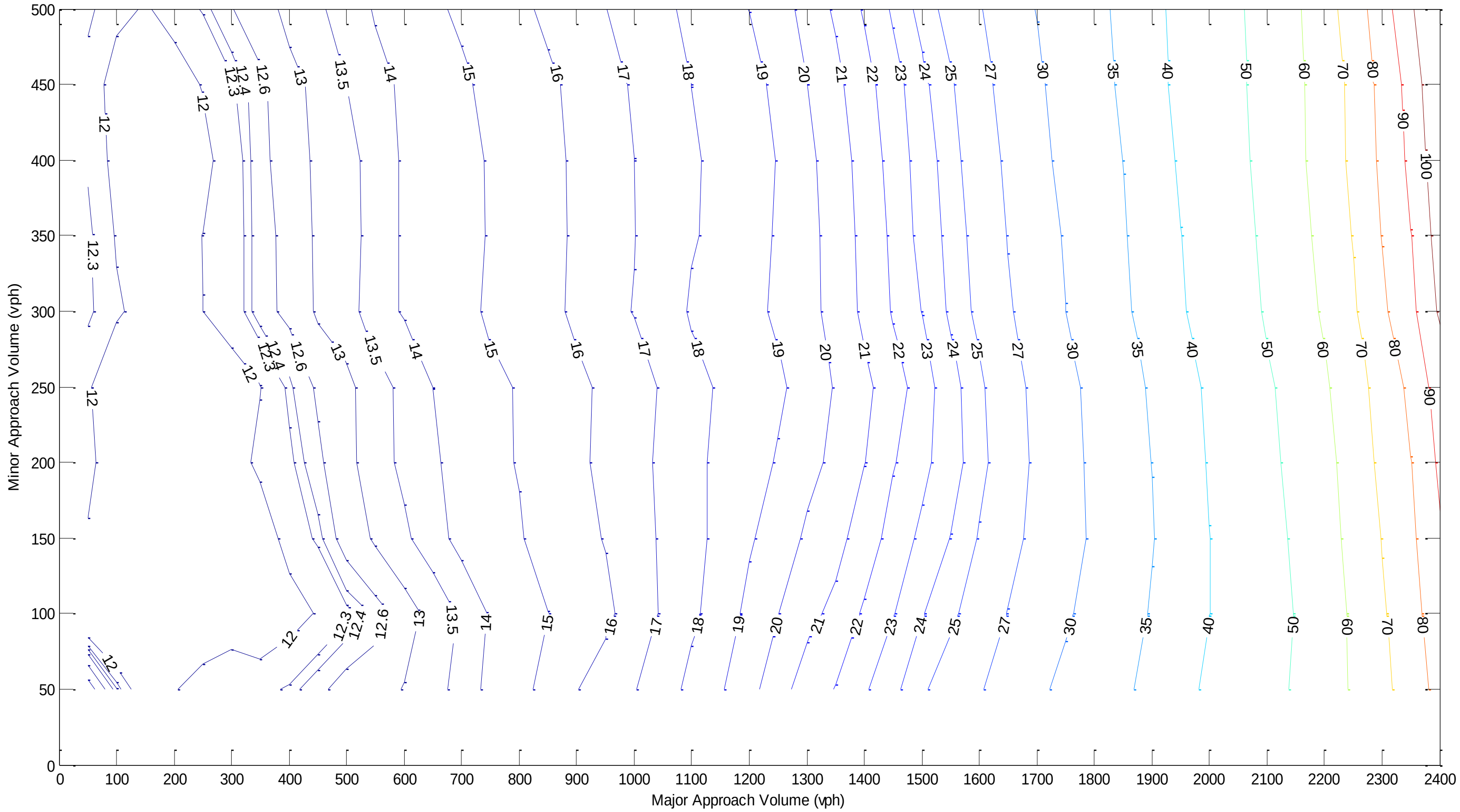


Figure B.48 Contour plot at BF=0.7, LTP=5 %, TP=10 %

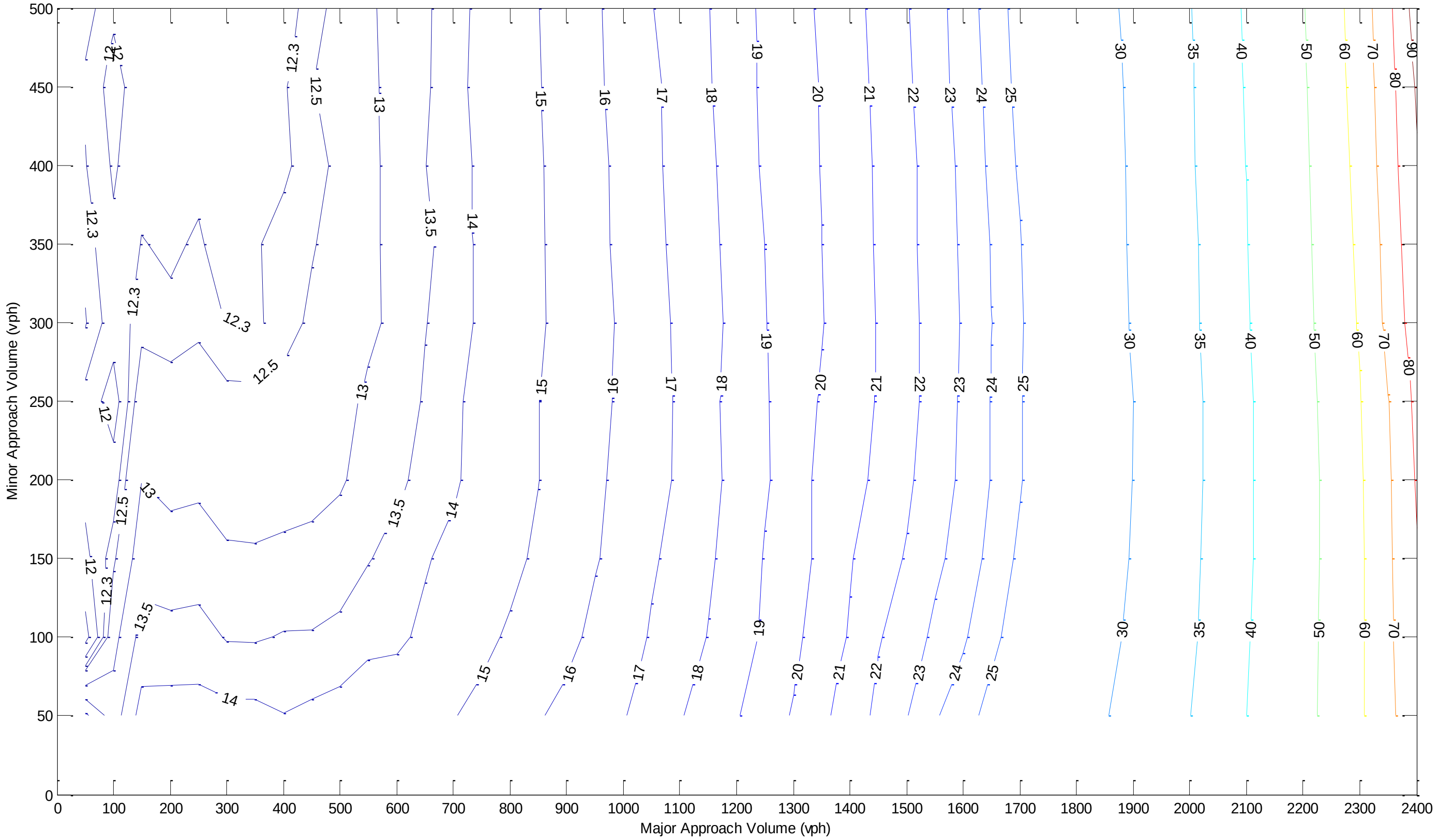


Figure B.49 Contour plot at BF=0.5, LTP=10%, TP=10 %

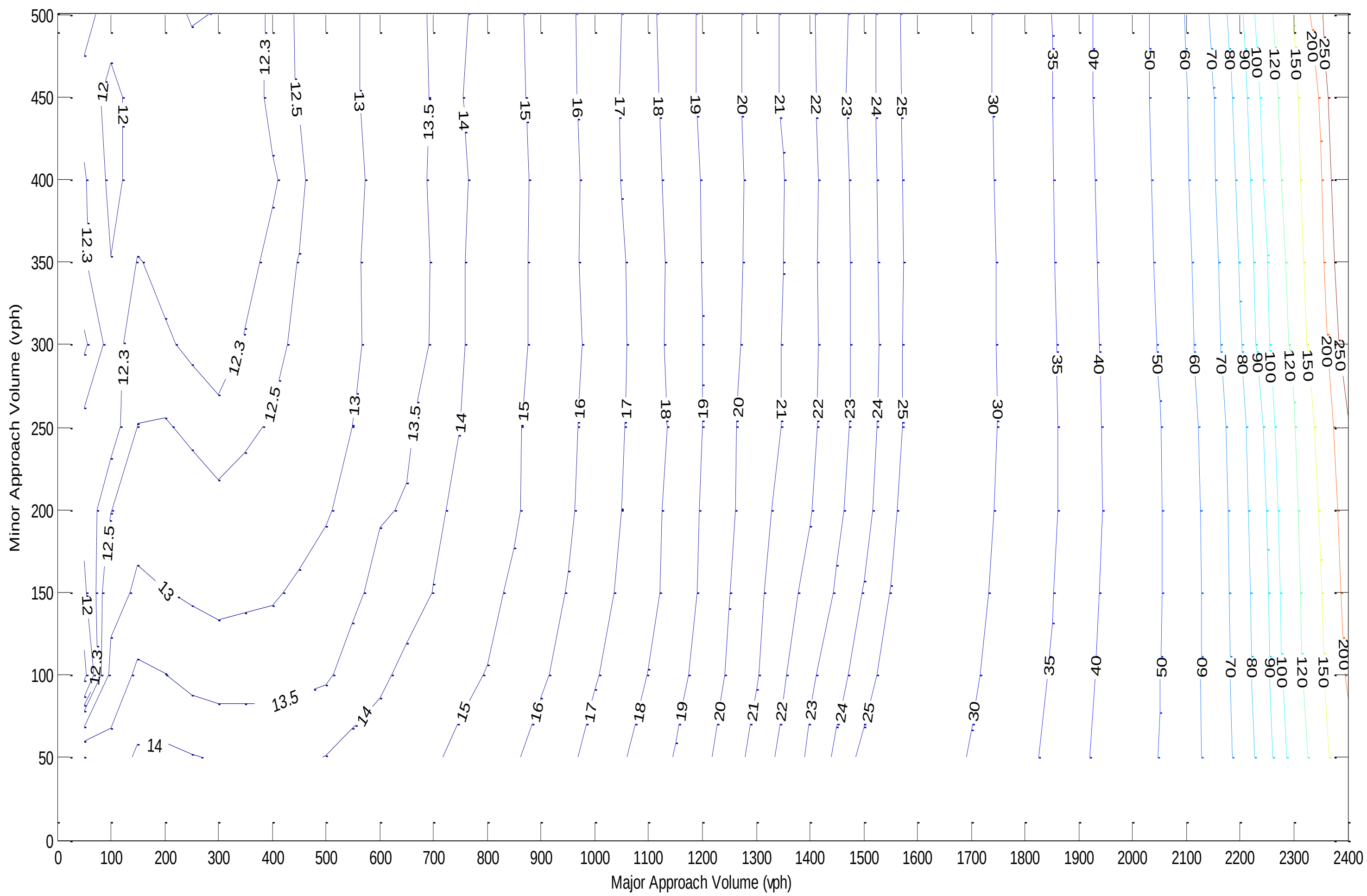


Figure B.50 Contour plot at BF=0.6, LTP=10 %, TP=10 %

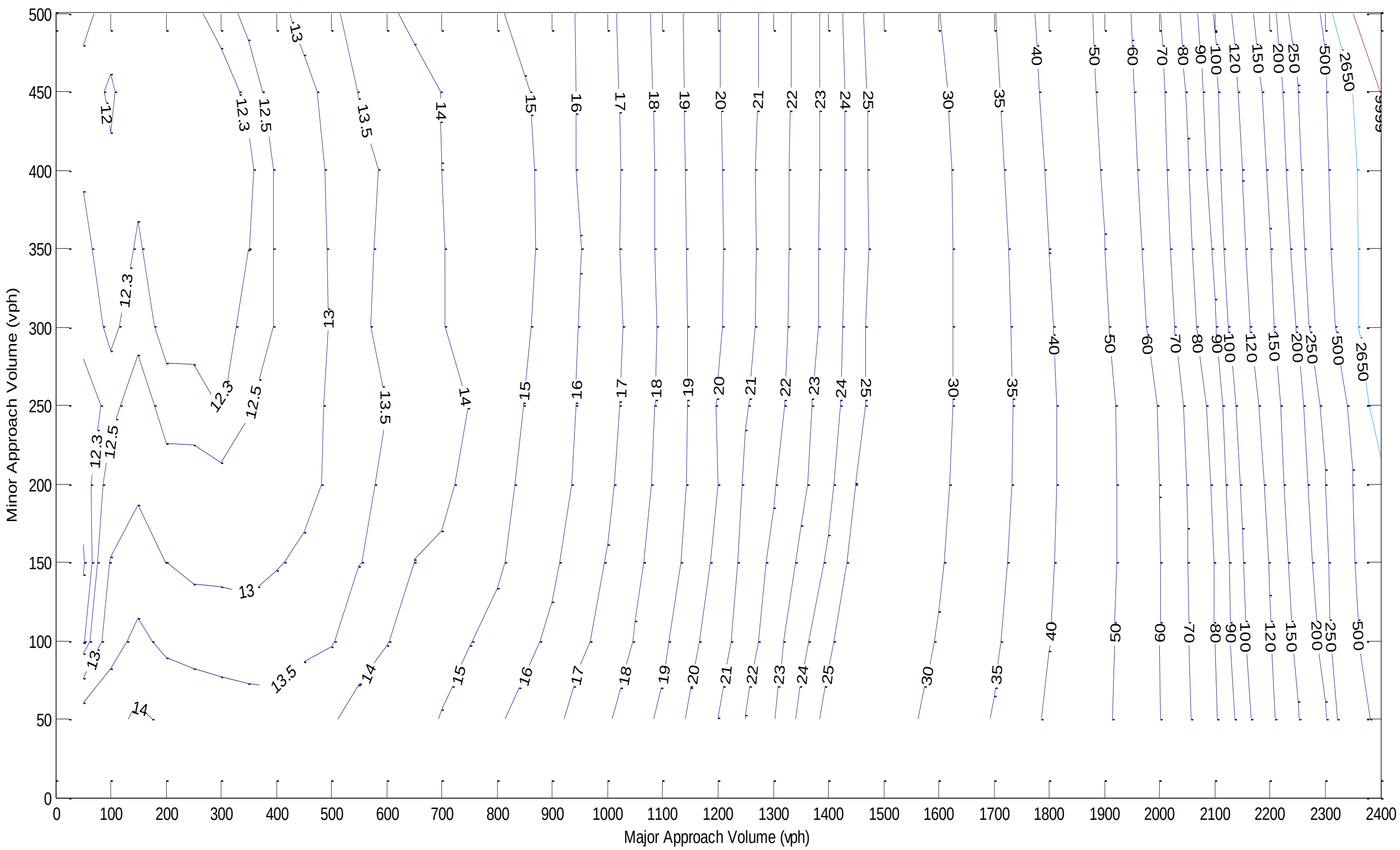


Figure B.51 Contour plot at BF=0.7, LTP=10 %, TP=10 %

(Note: 9999 represents failure condition of Median Opening)

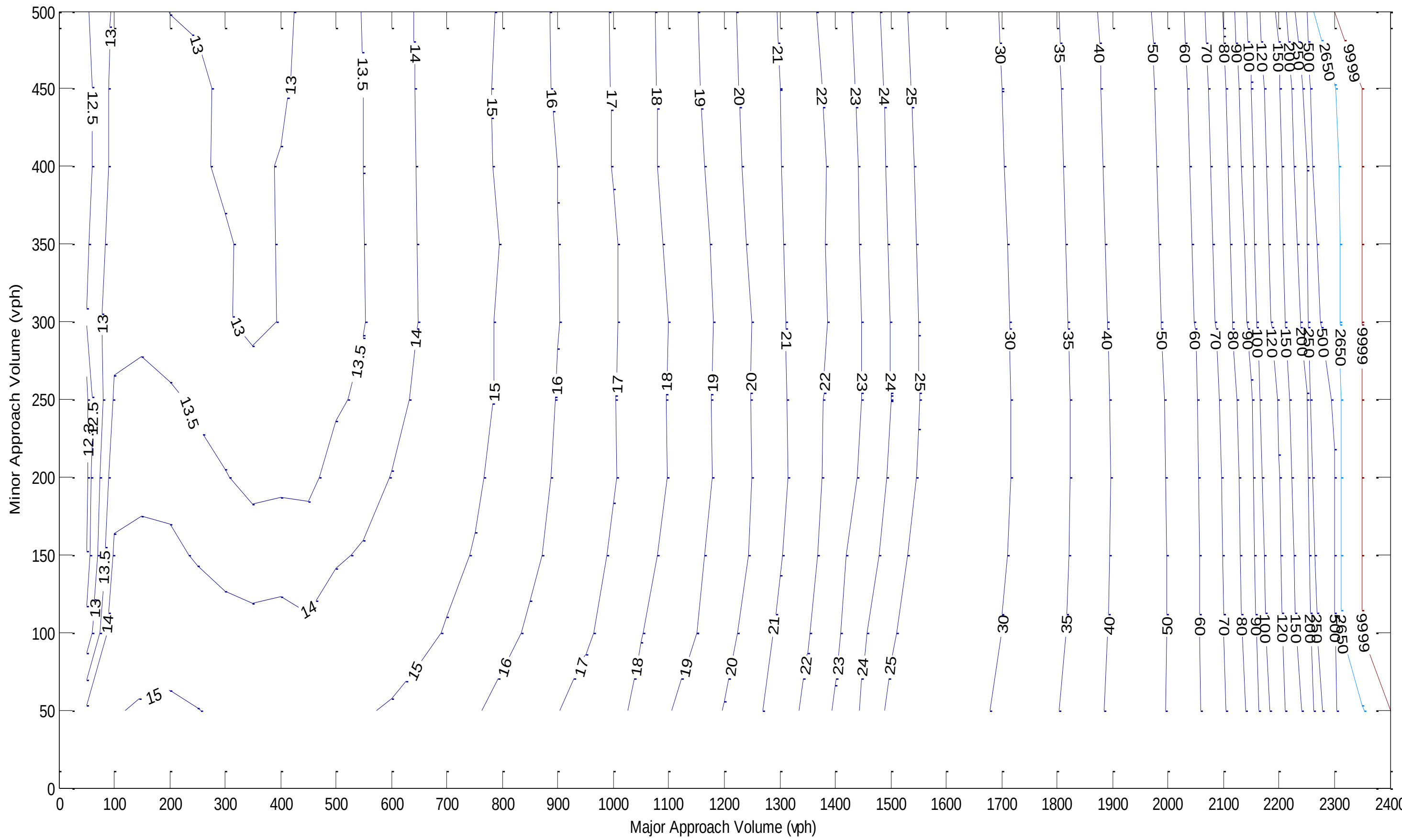


Figure B.52 Contour plot at BF=0.5, LTP=15 %, TP=10 %

(Note: 9999 represents failure condition of Median Opening)

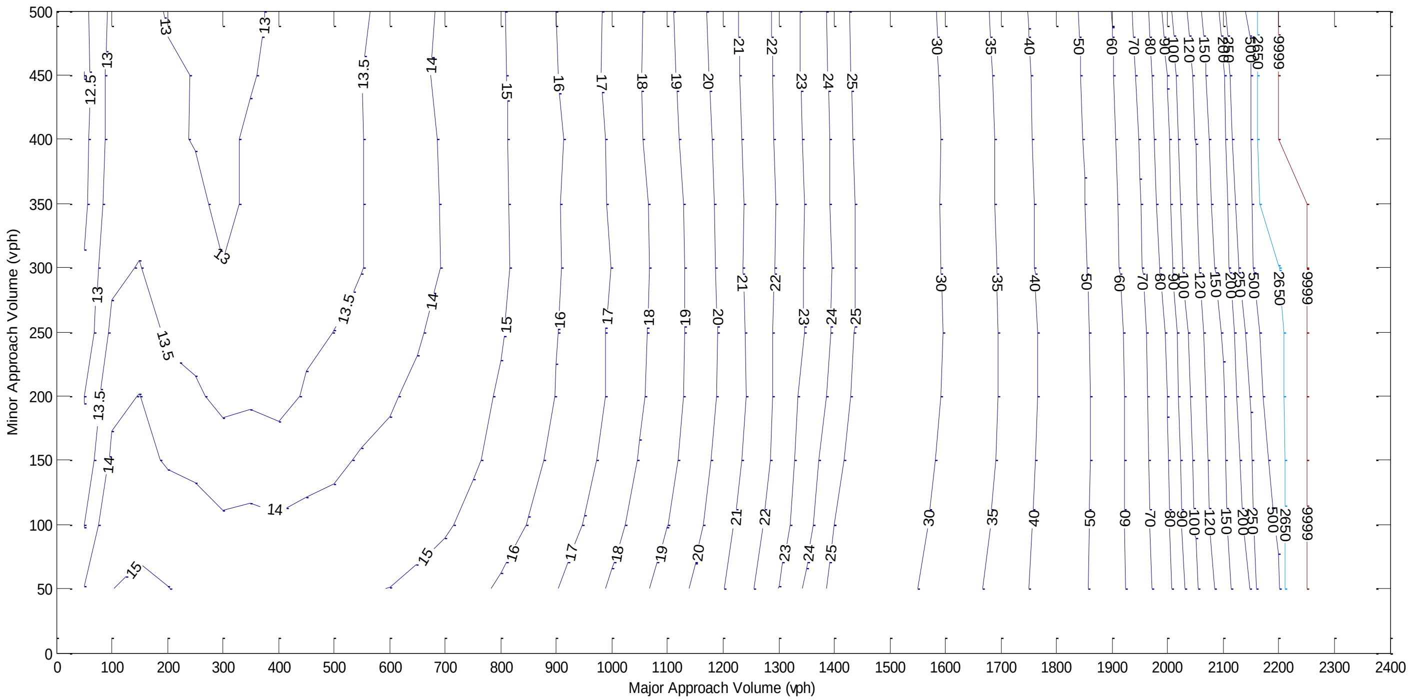


Figure B.53 Contour plot at BF=0.6, LTP=15 %, TP=10 %

(Note: 9999 represents failure condition of Median Opening)

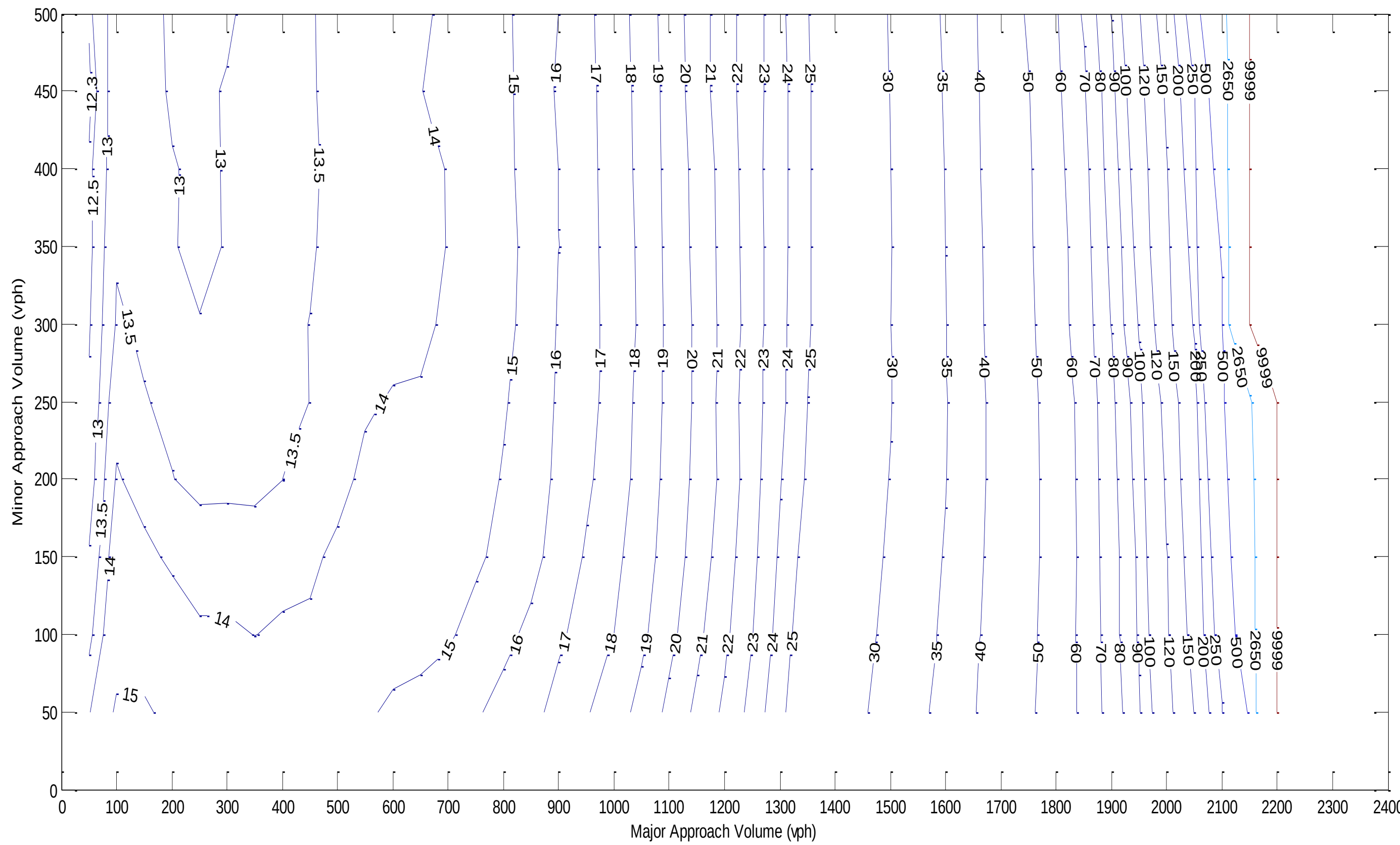


Figure B.54 Contour plot at BF=0.7, LTP=15 %, TP=10 %

(Note: 9999 represents failure condition of Median Opening)

B.2.2. Contour Plot for Total Intersection Delay Difference between MUT and Standard Signalized Intersection

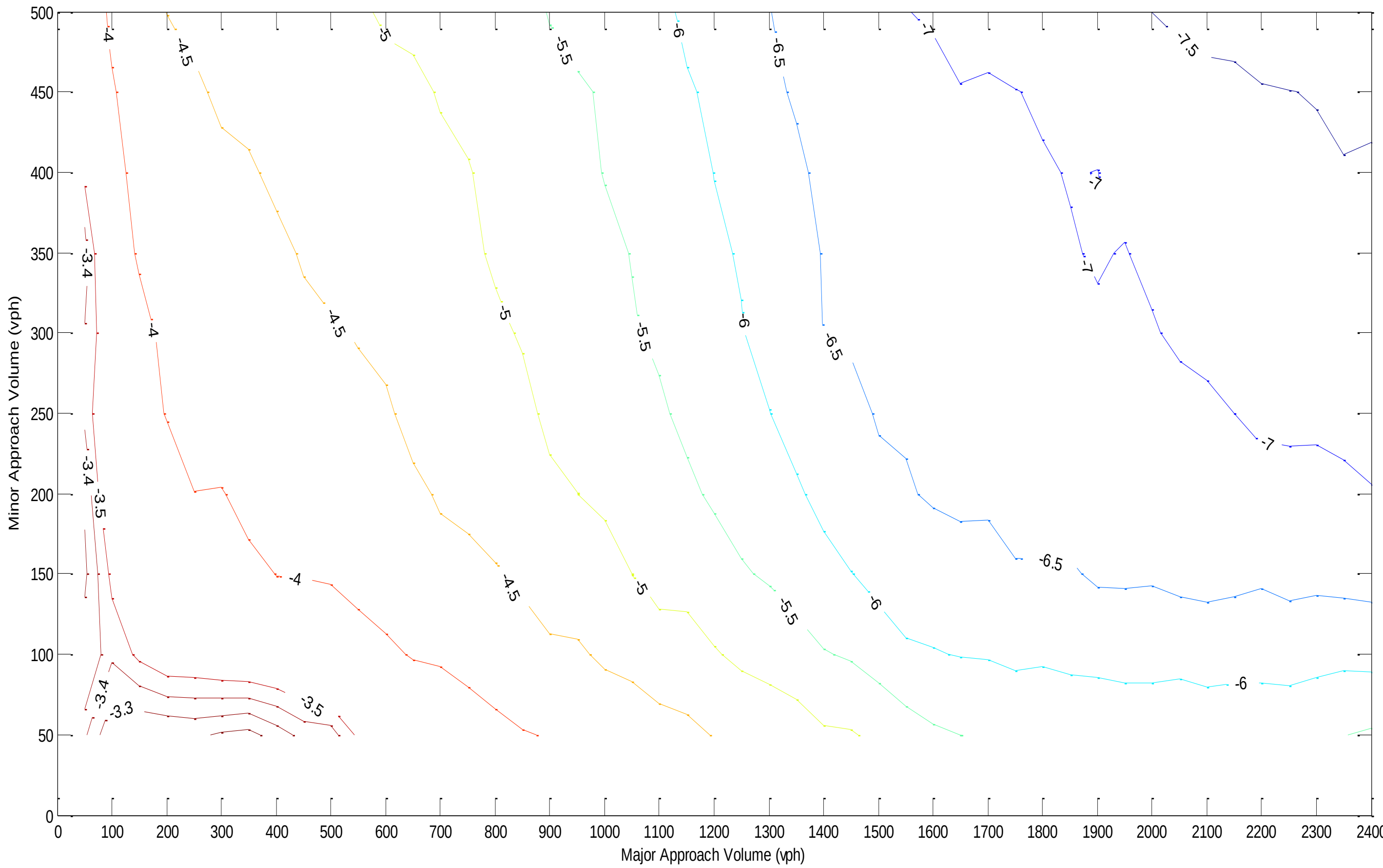


Figure B.55 Contour plot at BF=0.5, LTP=5%, TP=2%

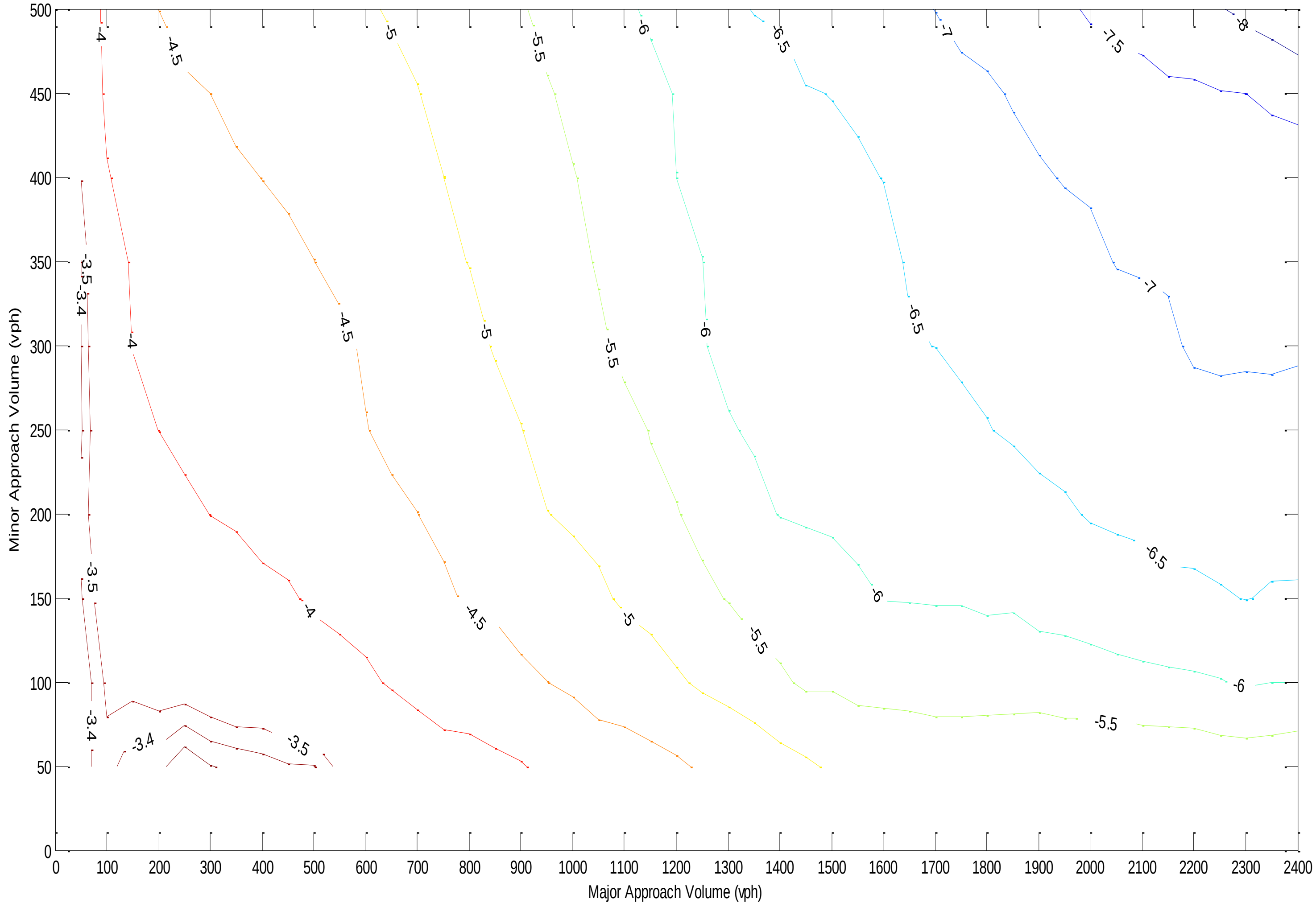


Figure B.56 Contour plot at BF=0.6, LTP=5%, TP=2%

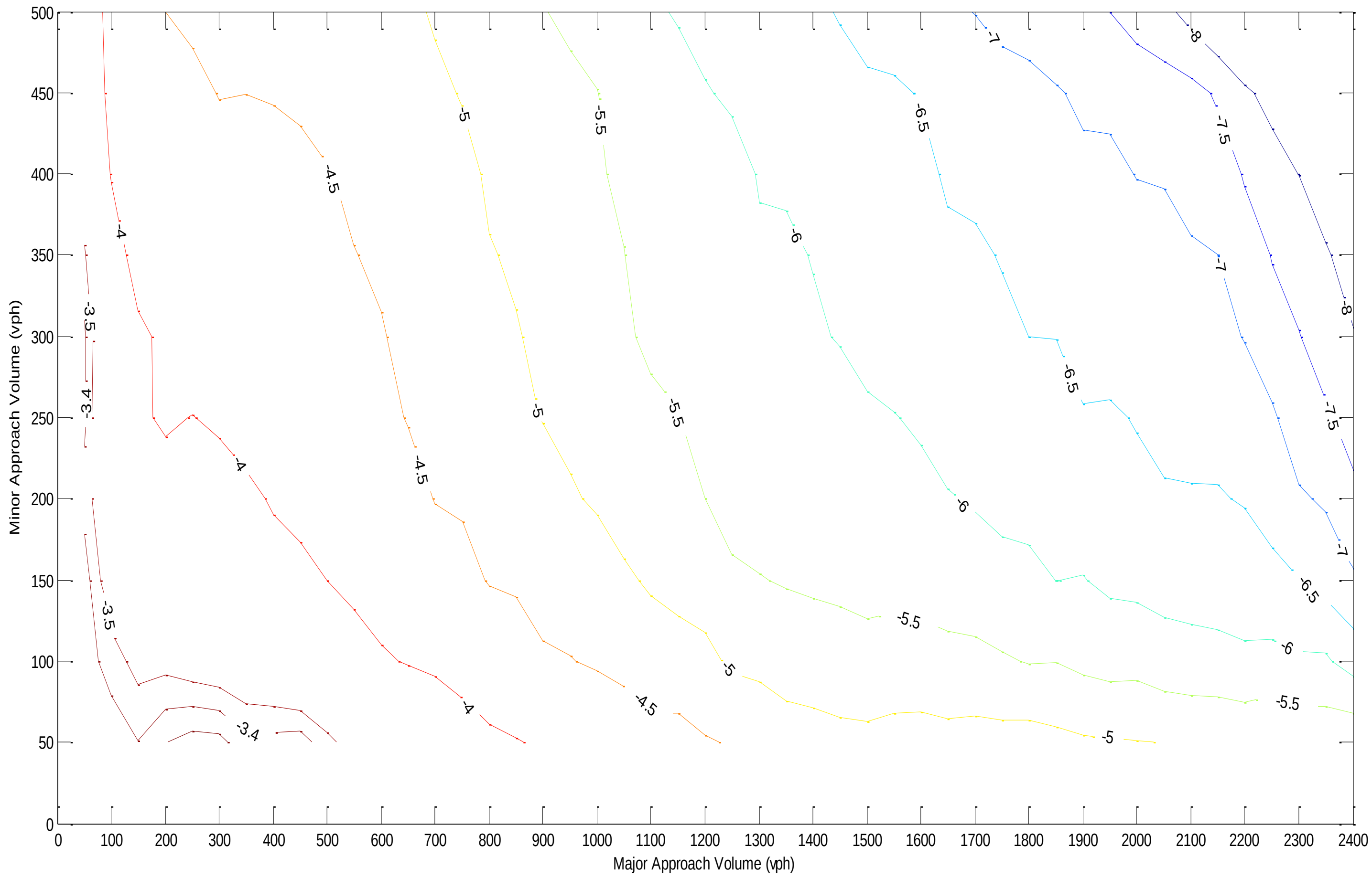


Figure B.57 Contour plot at BF=0.7, LTP=5%, TP=2%

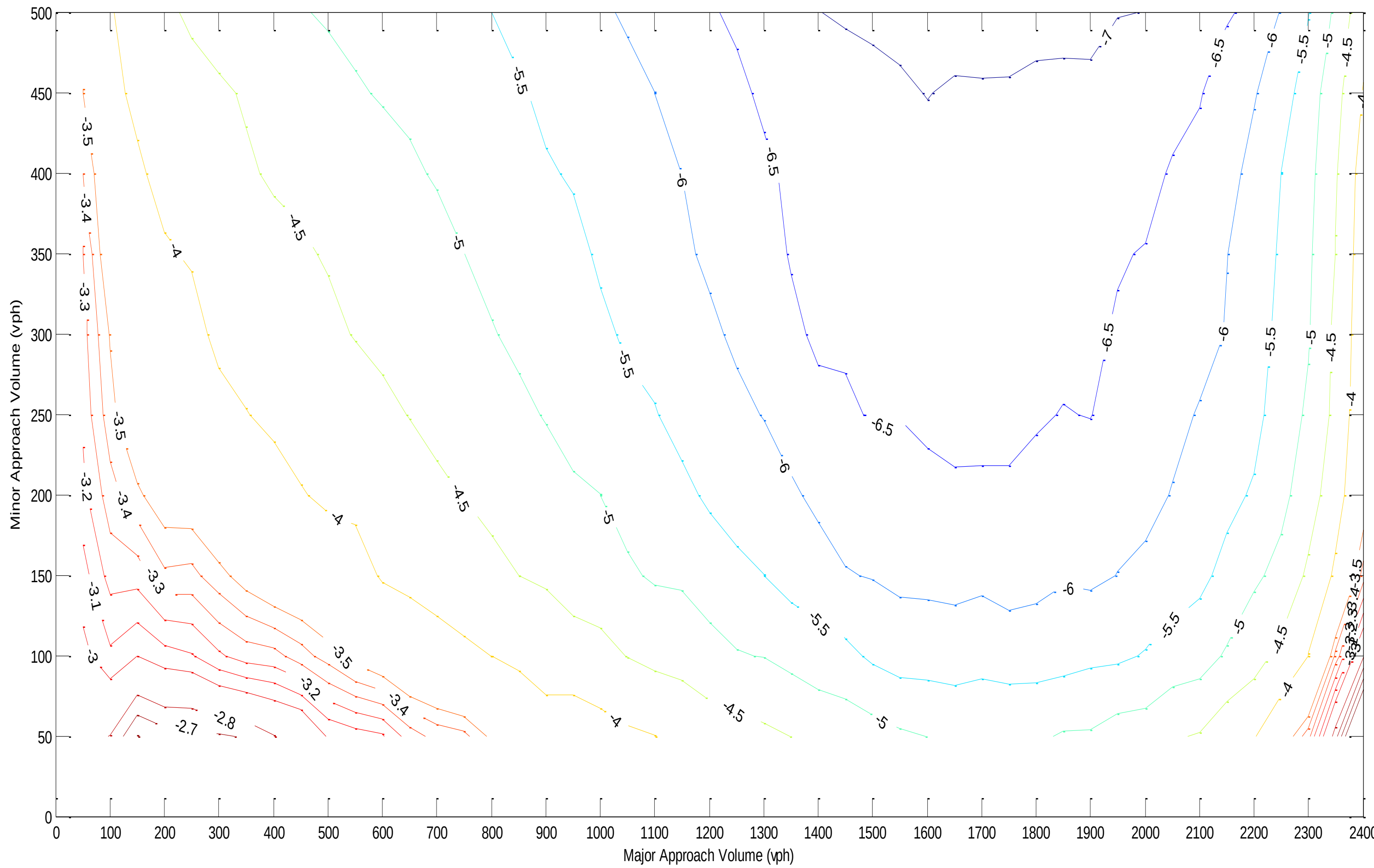


Figure B.58 Contour plot at BF=0.5, LTP=10%, TP=2%

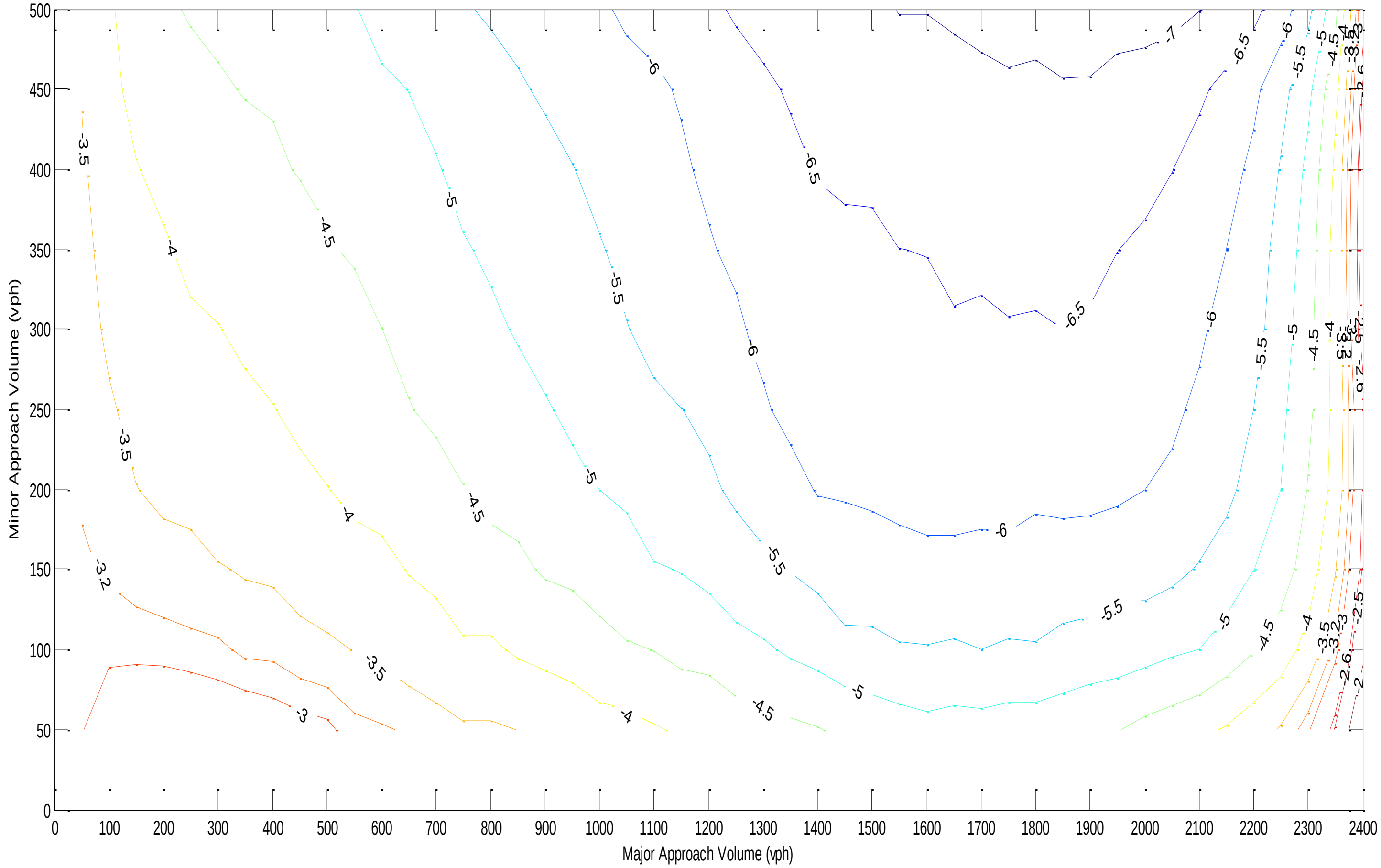


Figure B.59 Contour plot at BF=0.6, LTP=10%, TP=2%

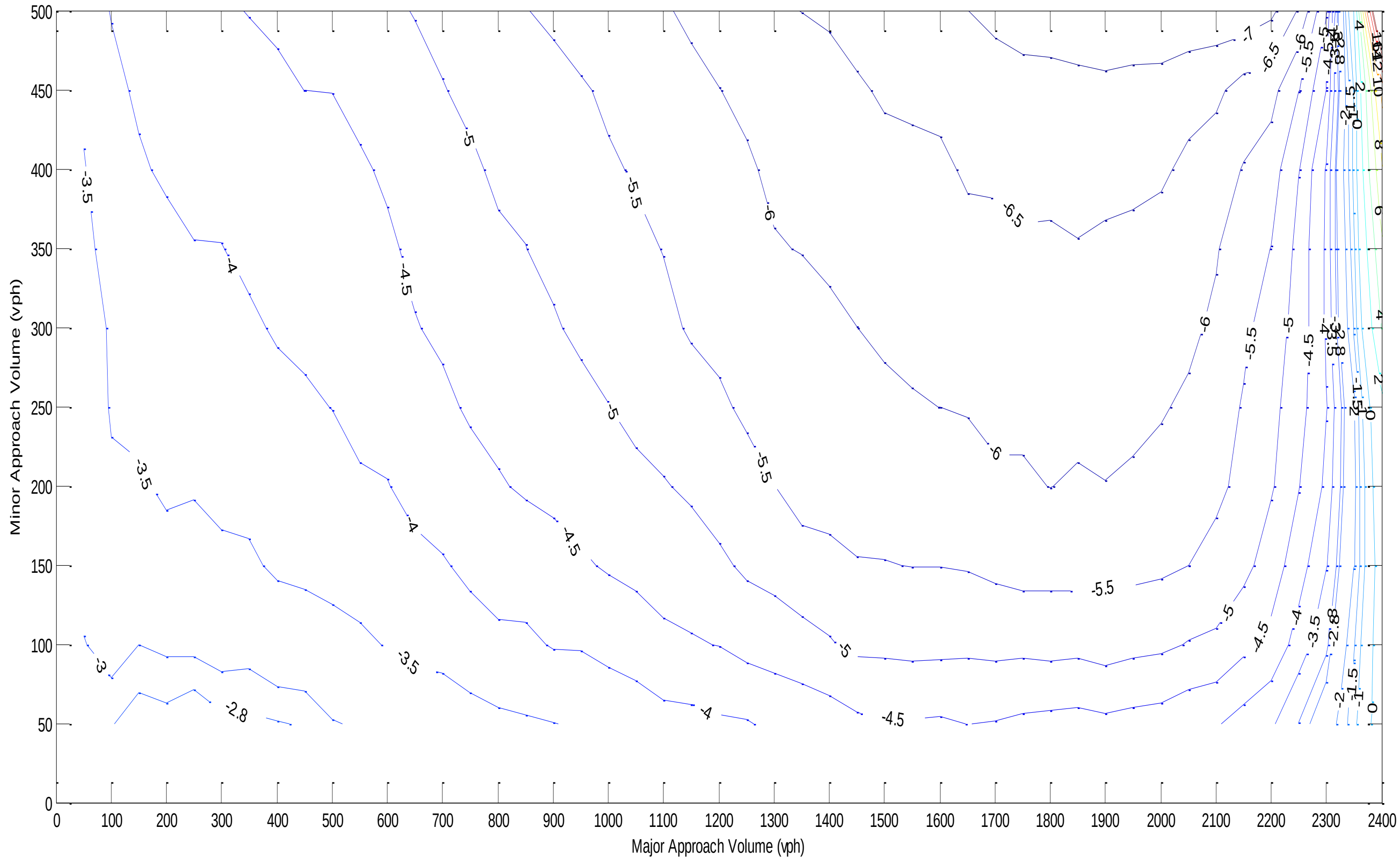


Figure B.60 Contour plot at BF=0.7, LTP=10%, TP=2%

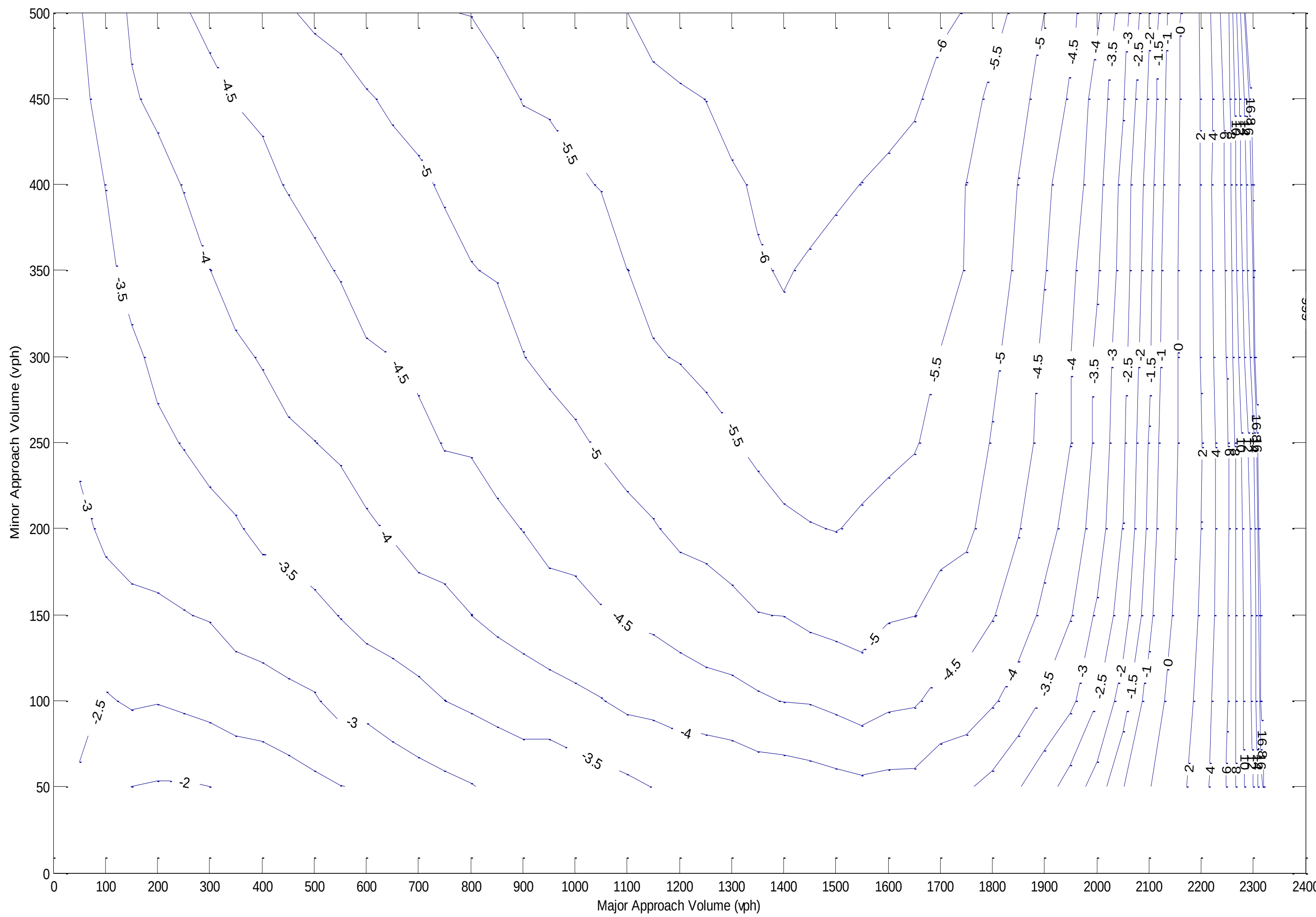


Figure B.61 Contour plot at BF=0.5, LTP=15%, TP=2%

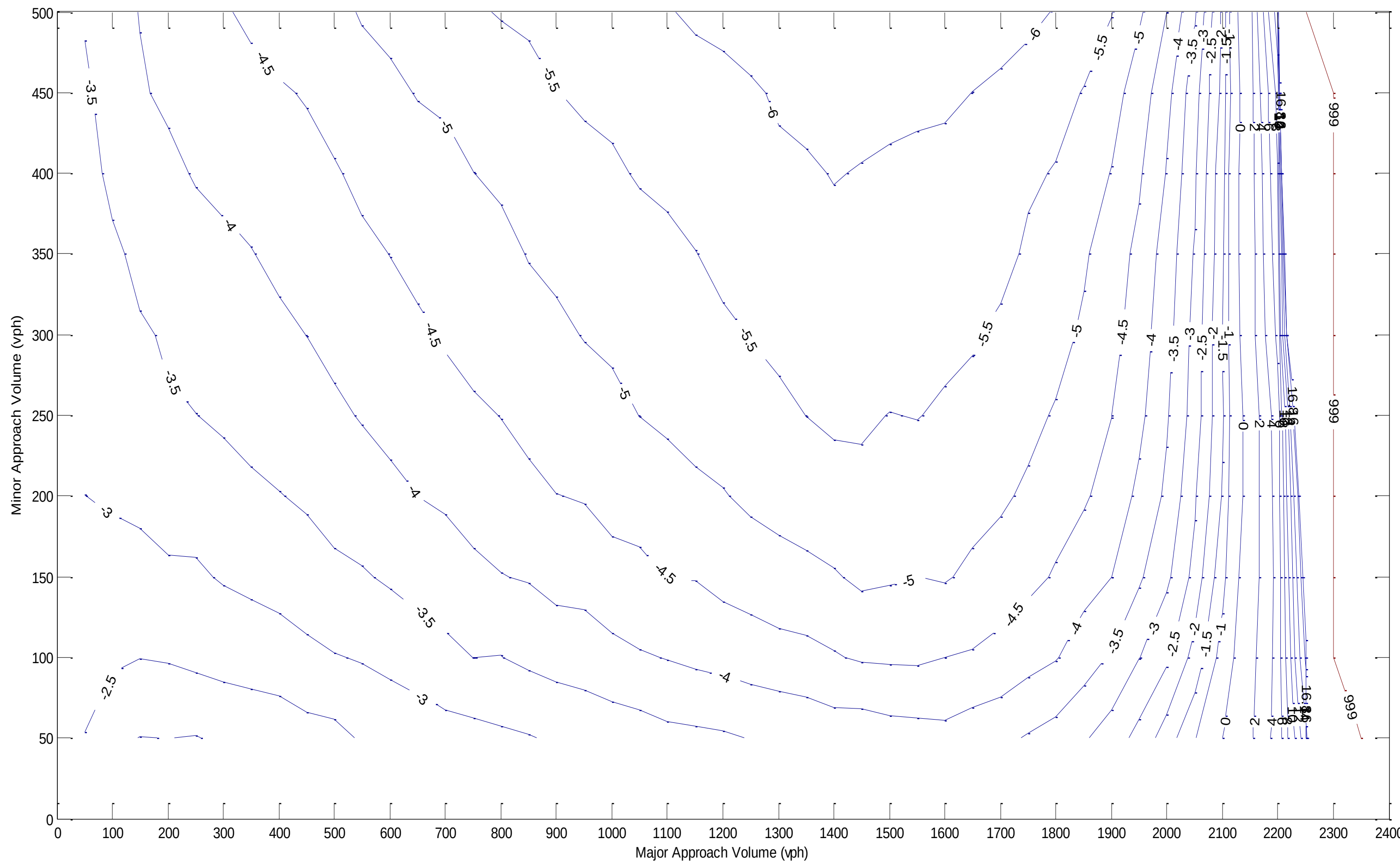


Figure B.62 Contour plot at BF=0.6, LTP=15%, TP=2%

(Note: 999 represents failure condition of Median Opening)

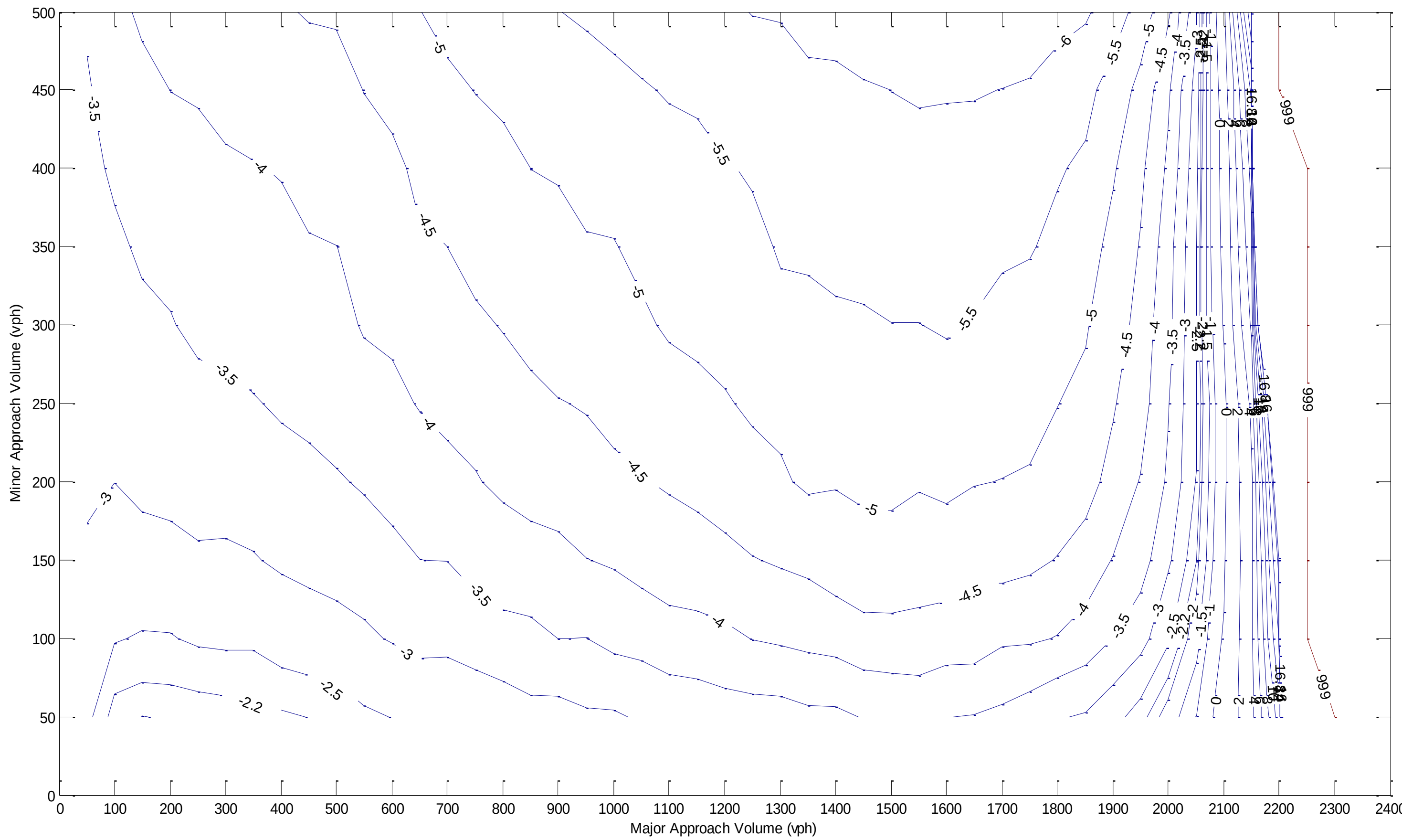


Figure B.63 Contour plot at BF=0.7, LTP=15%, TP=2%

(Note: 999 represents failure condition of Median Opening)

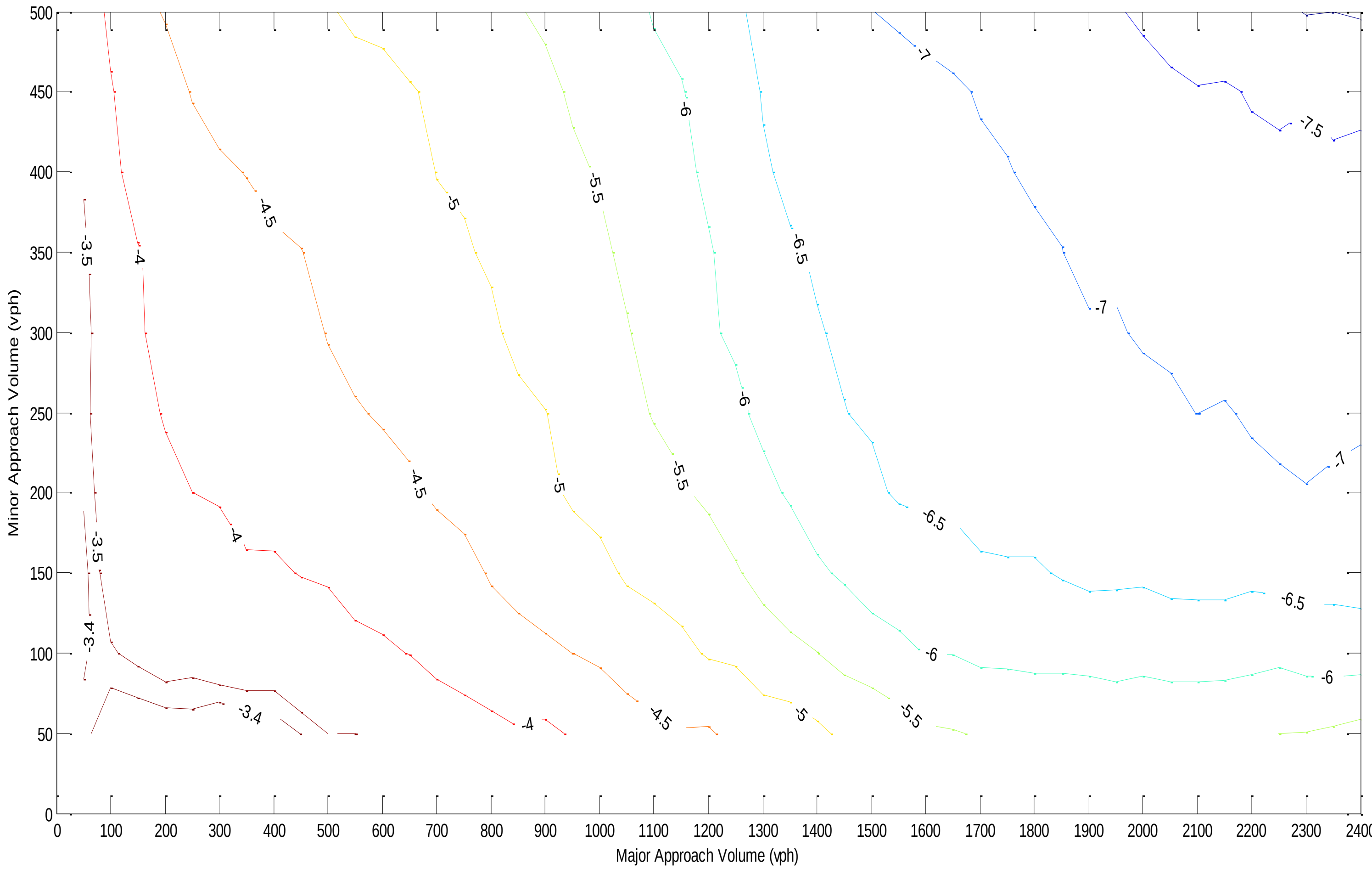


Figure B.64 Contour plot at BF=0.5, LTP=5 %, TP=5 %

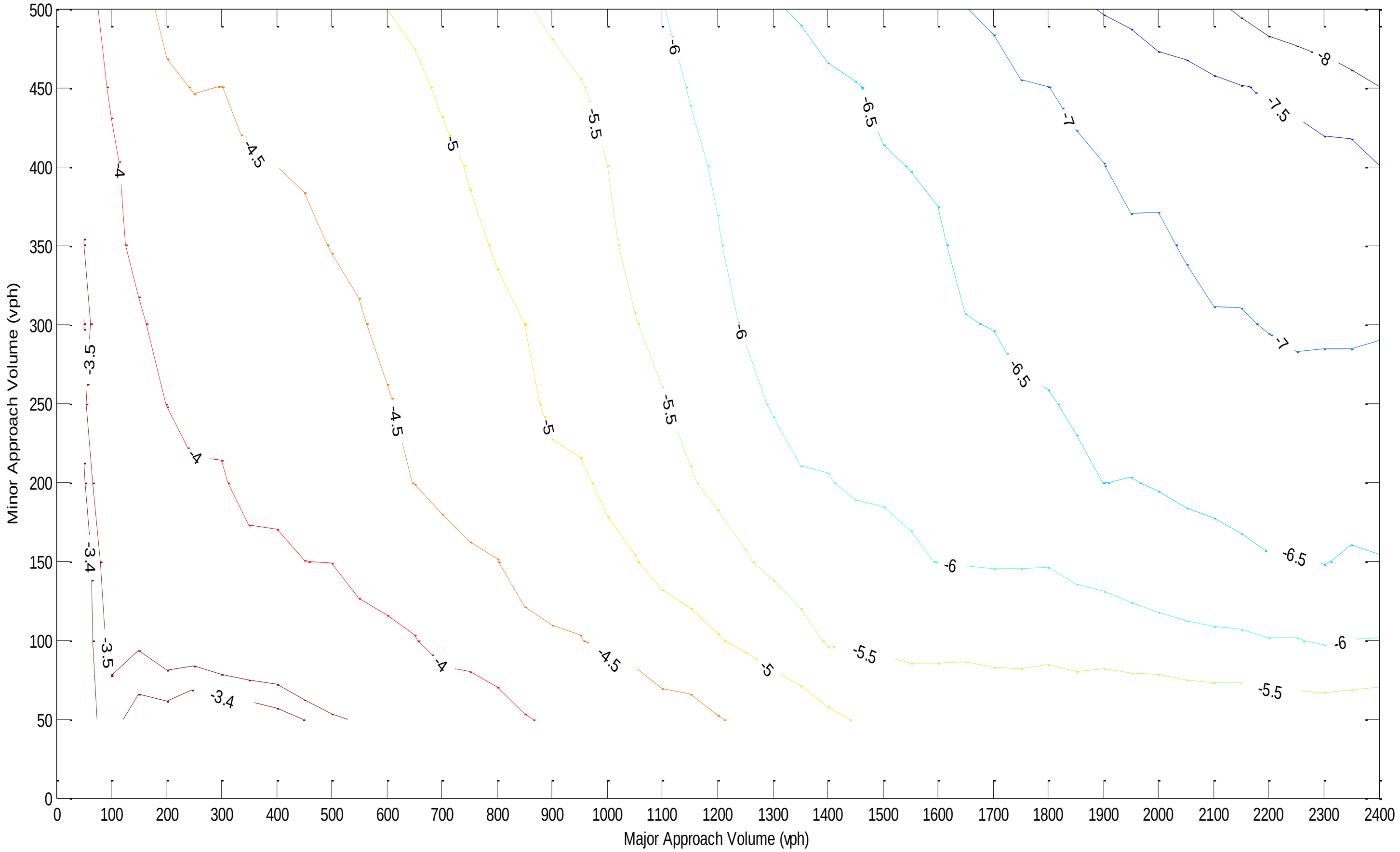


Figure B.65 Contour plot at BF=0.6, LTP=5 %, TP=5 %

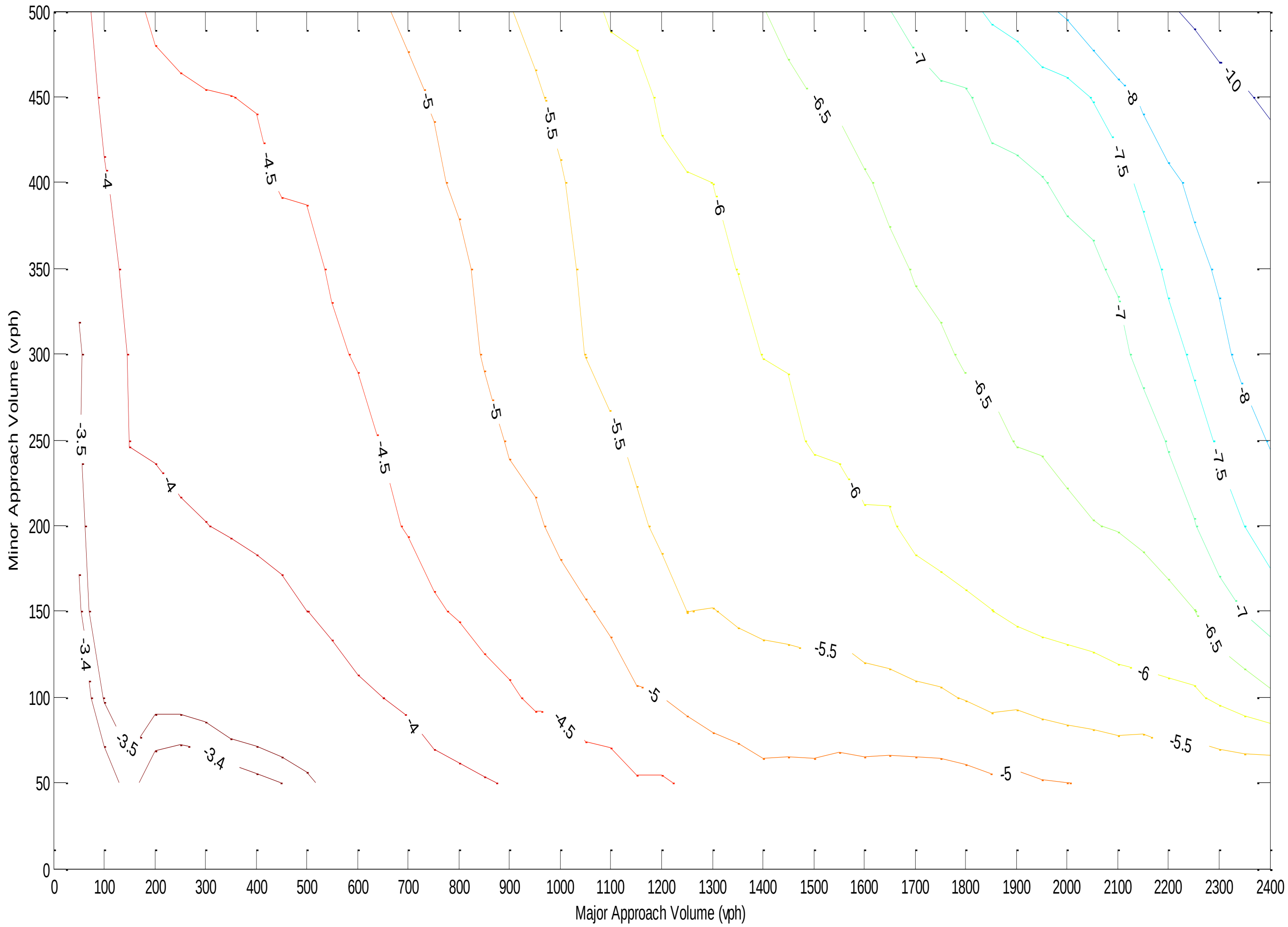


Figure B.66 Contour plot at BF=0.7, LTP=5 %, TP=5 %

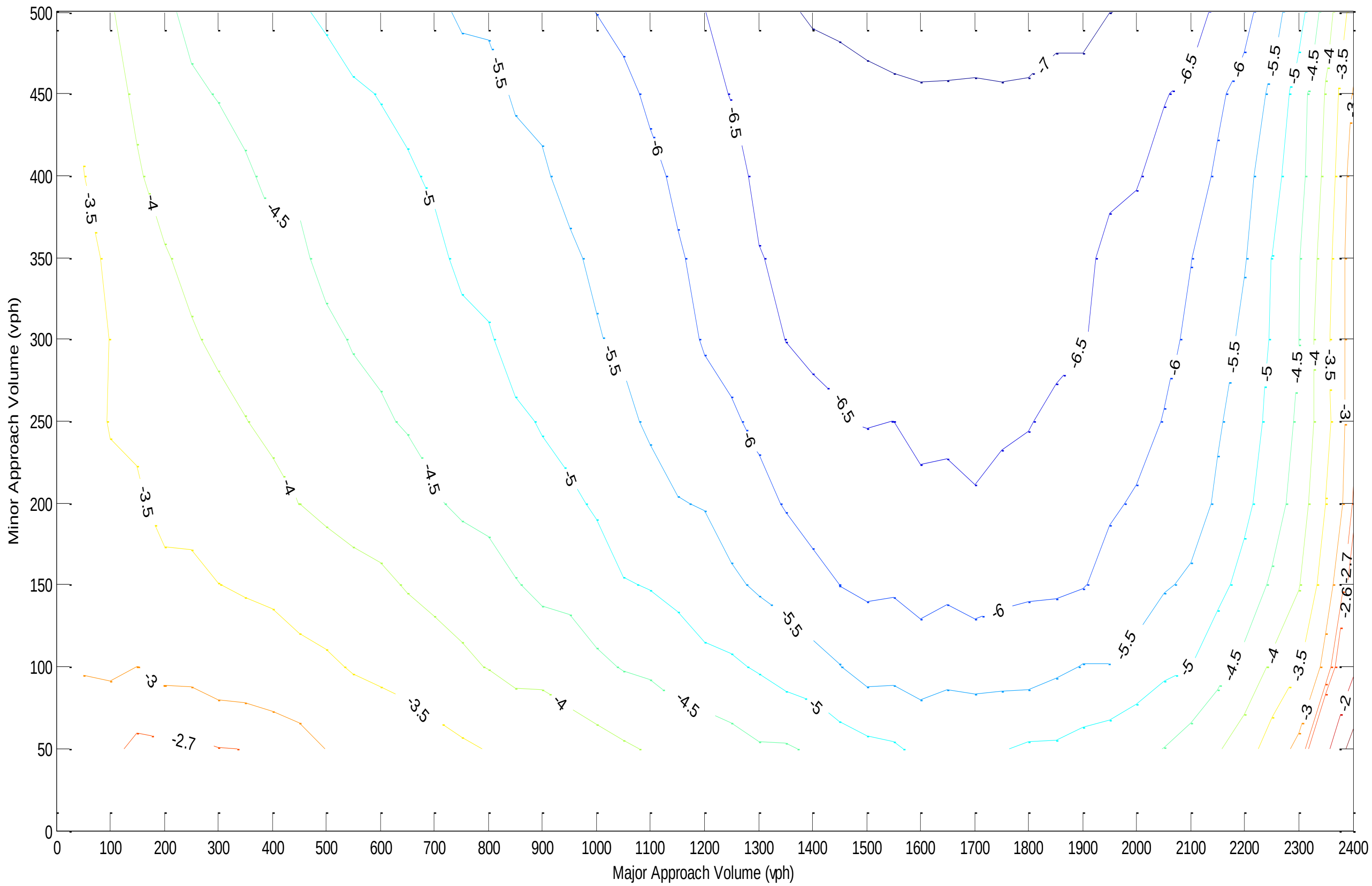


Figure B.67 Contour plot at BF=0.5, LTP=10 %, TP=5 %

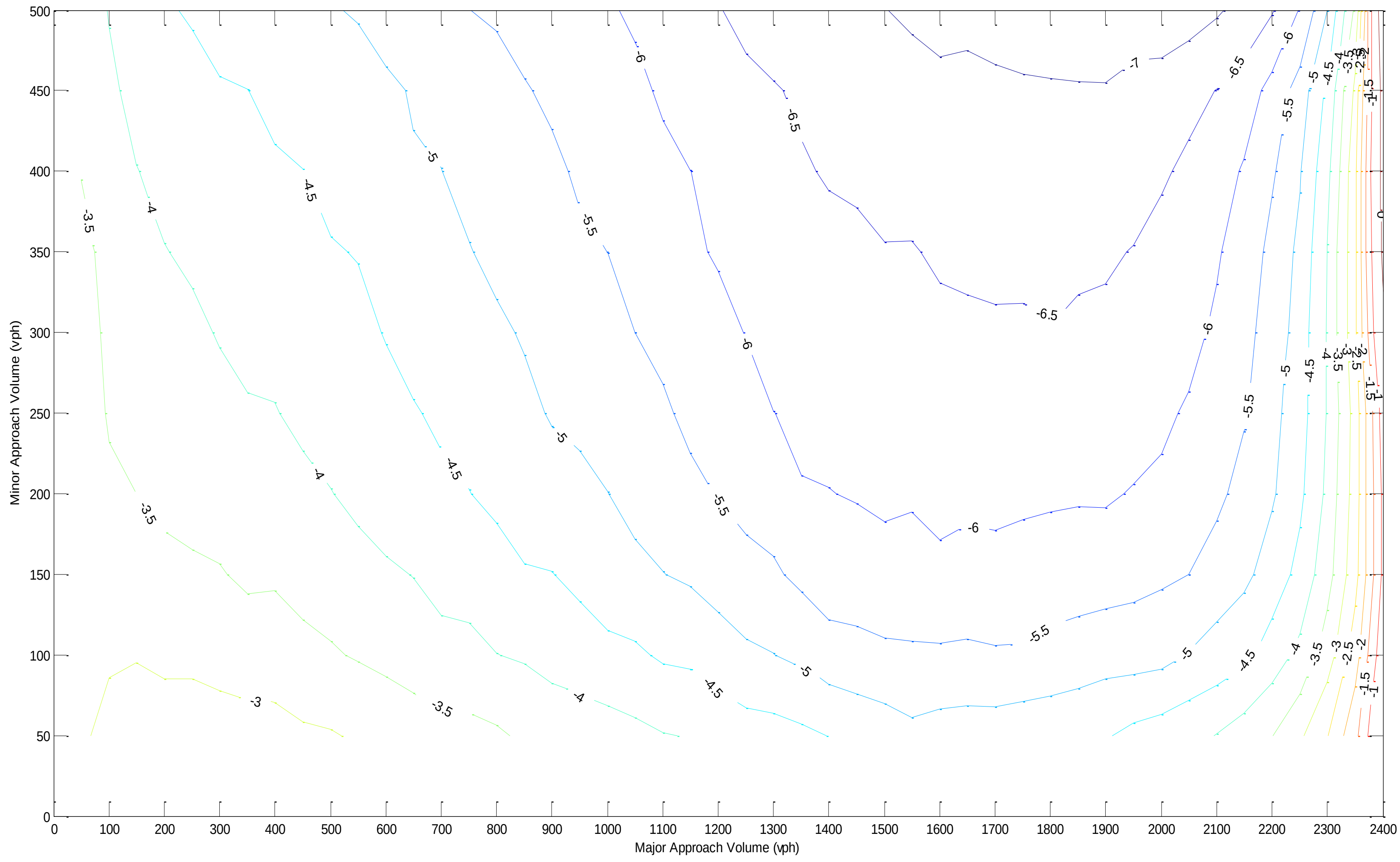


Figure B.68 Contour plot at BF=0.6, LTP=10 %, TP=5 %

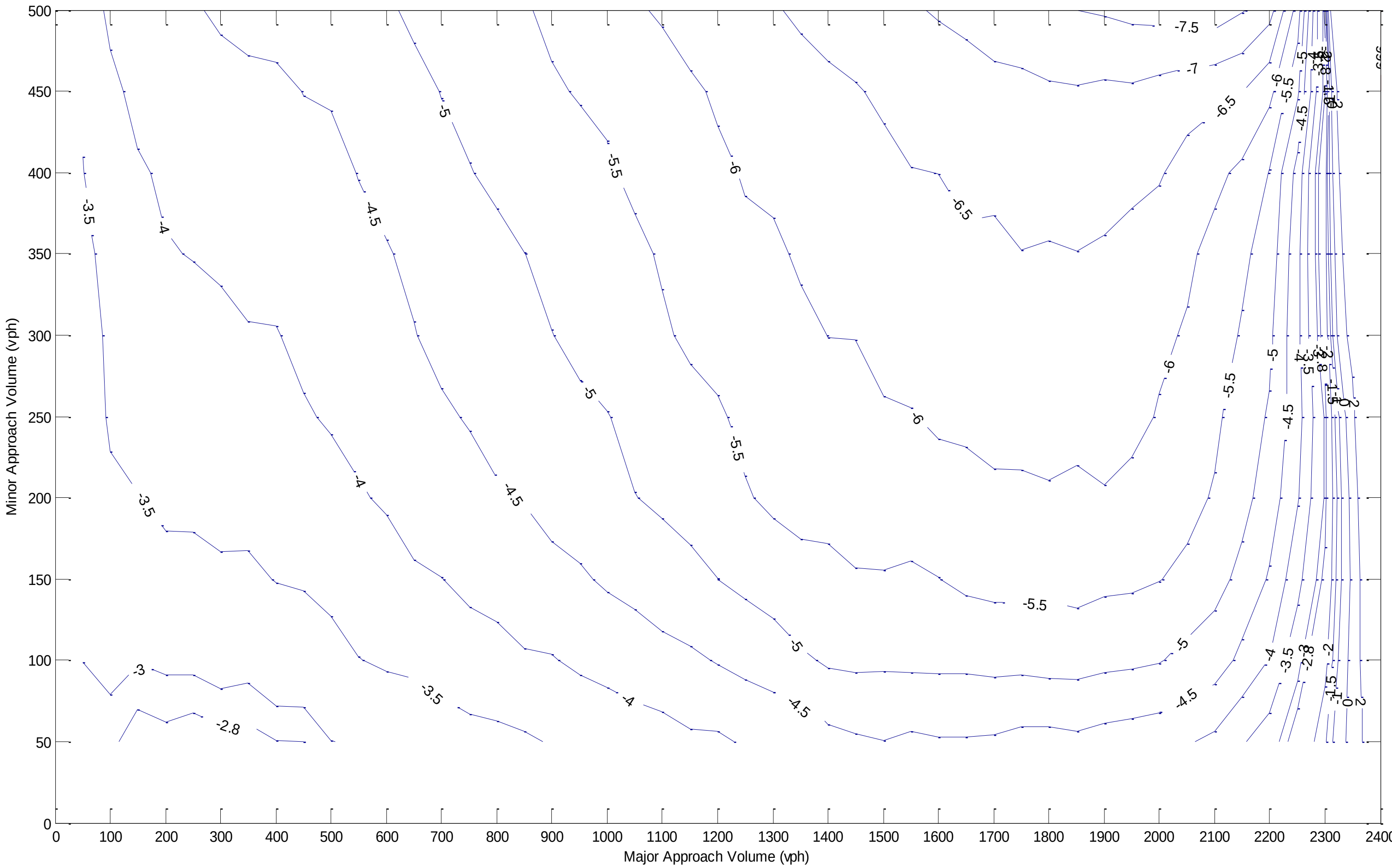


Figure B.69 Contour plot at BF=0.7, LTP=10 %, TP=5 %

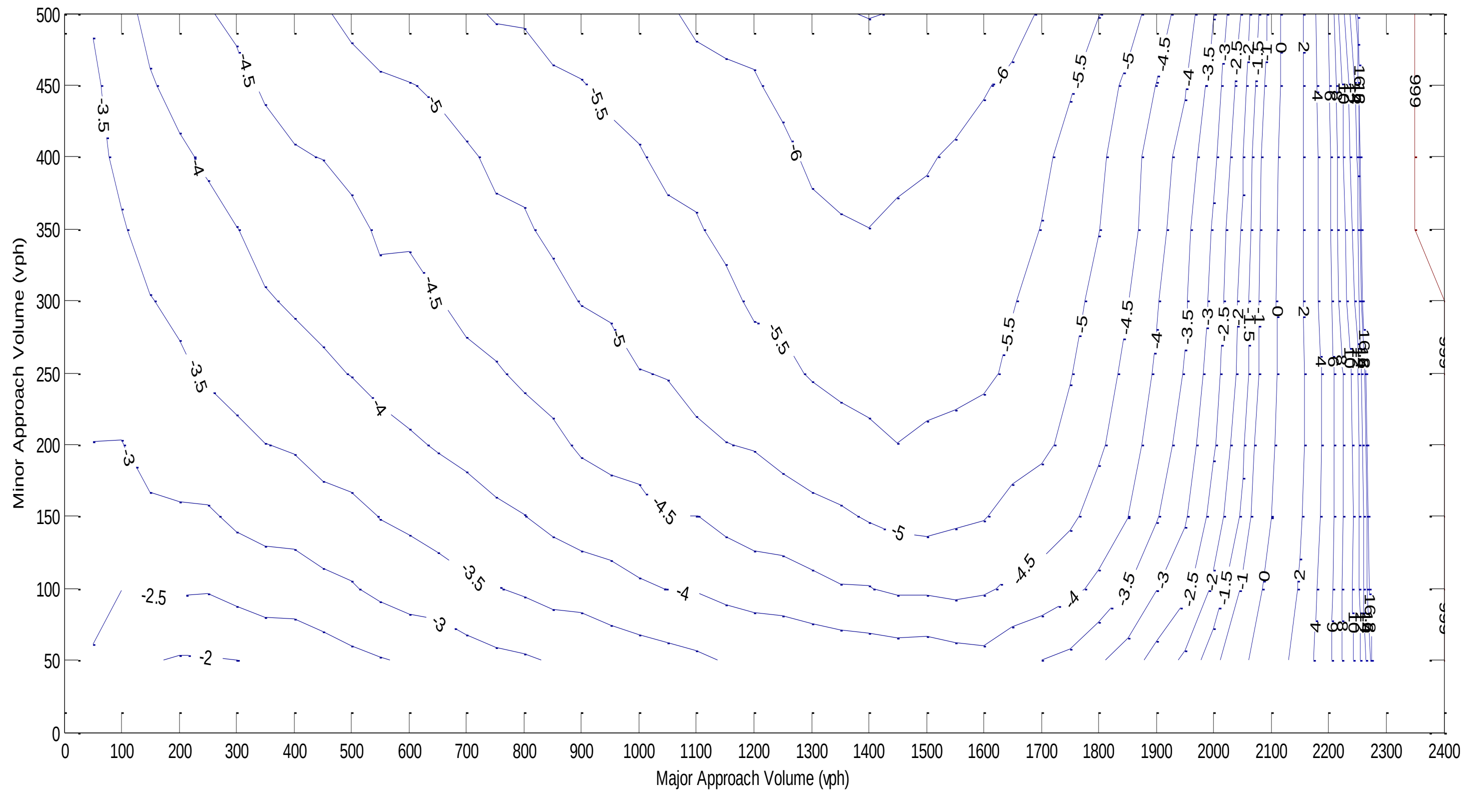


Figure B.70 Contour plot at BF=0.5, LTP=15 %, TP=5 %

Note: 999 represents failure condition of Median Opening

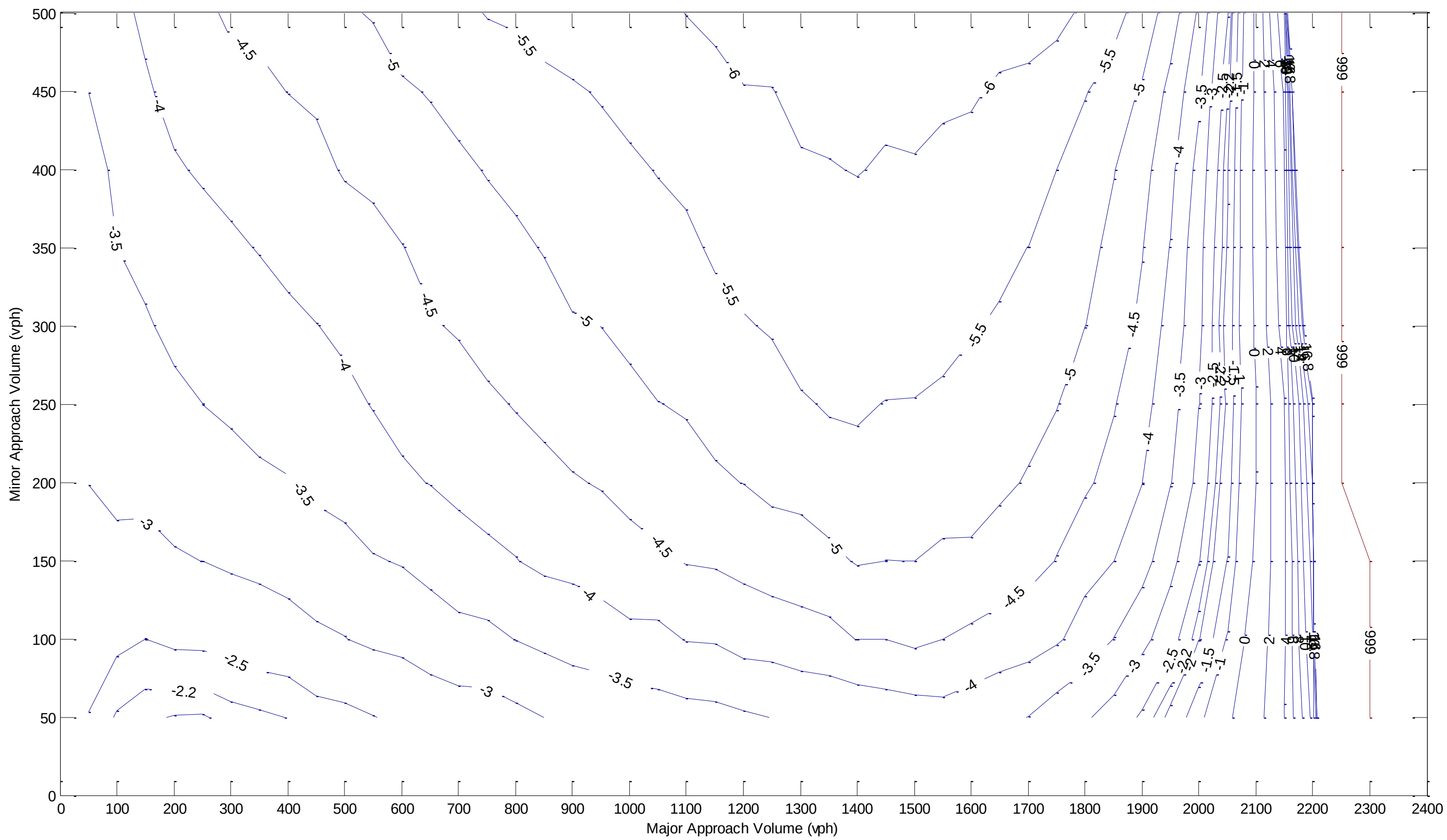


Figure B.71 Contour plot at BF=0.6, LTP=15 %, TP=5 %

(Note: 999 represents failure condition of Median Opening)

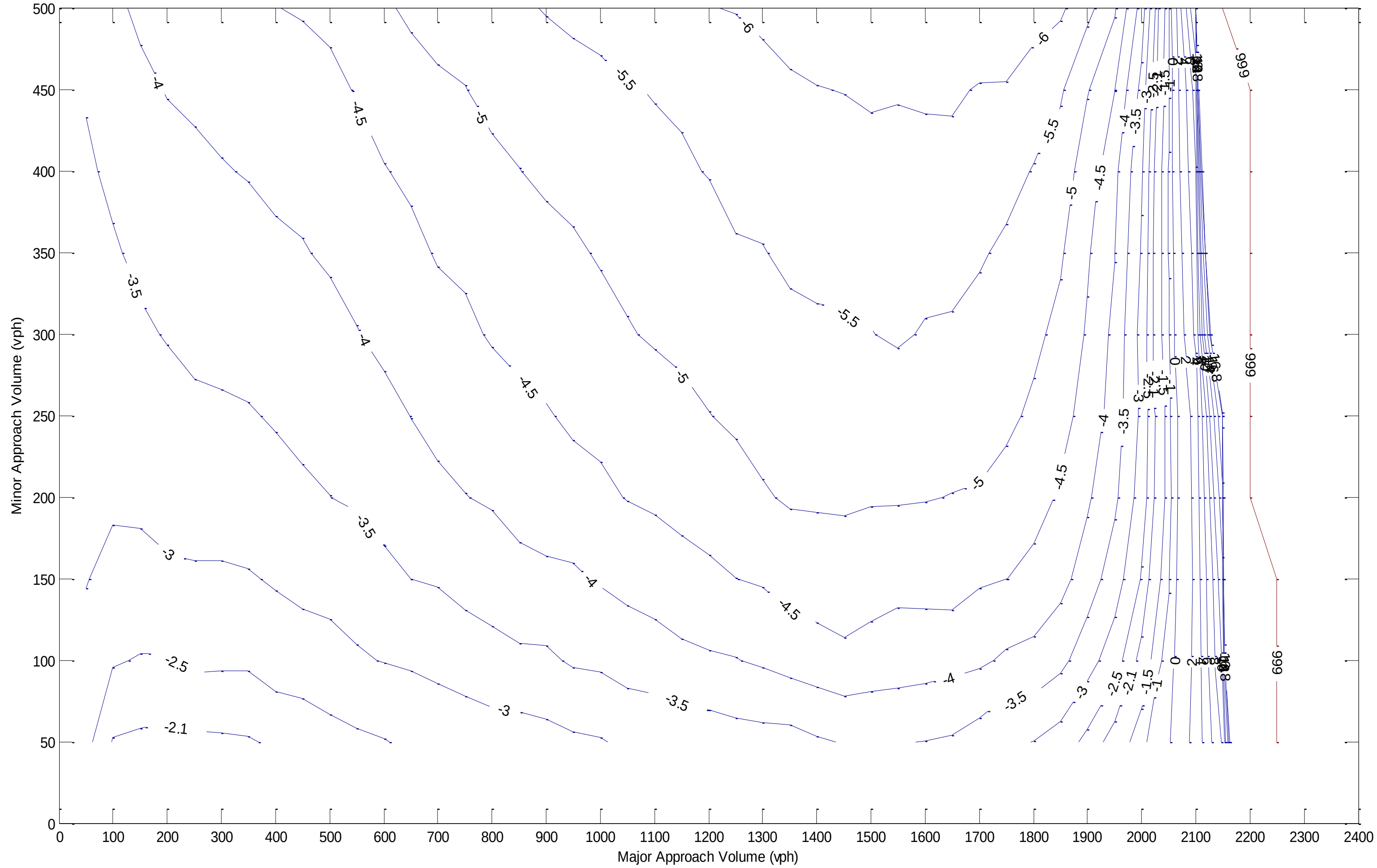


Figure B.72 Contour plot at BF=0.7, LTP=15 %, TP=5 %

(Note: 999 represent failure condition of Median Opening)

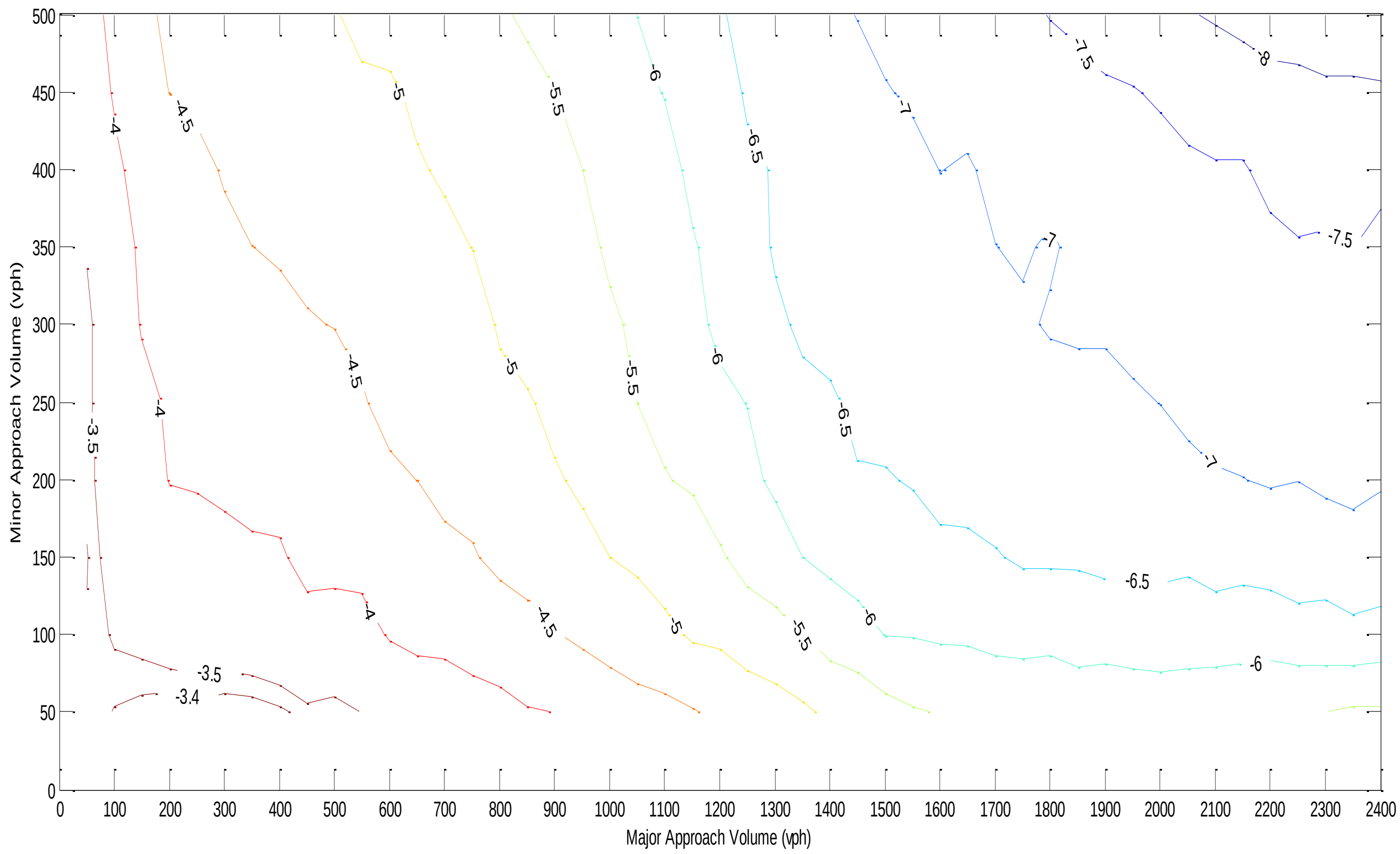


Figure B.73 Contour plot at BF=0.5, LTP=5 %, TP=10 %

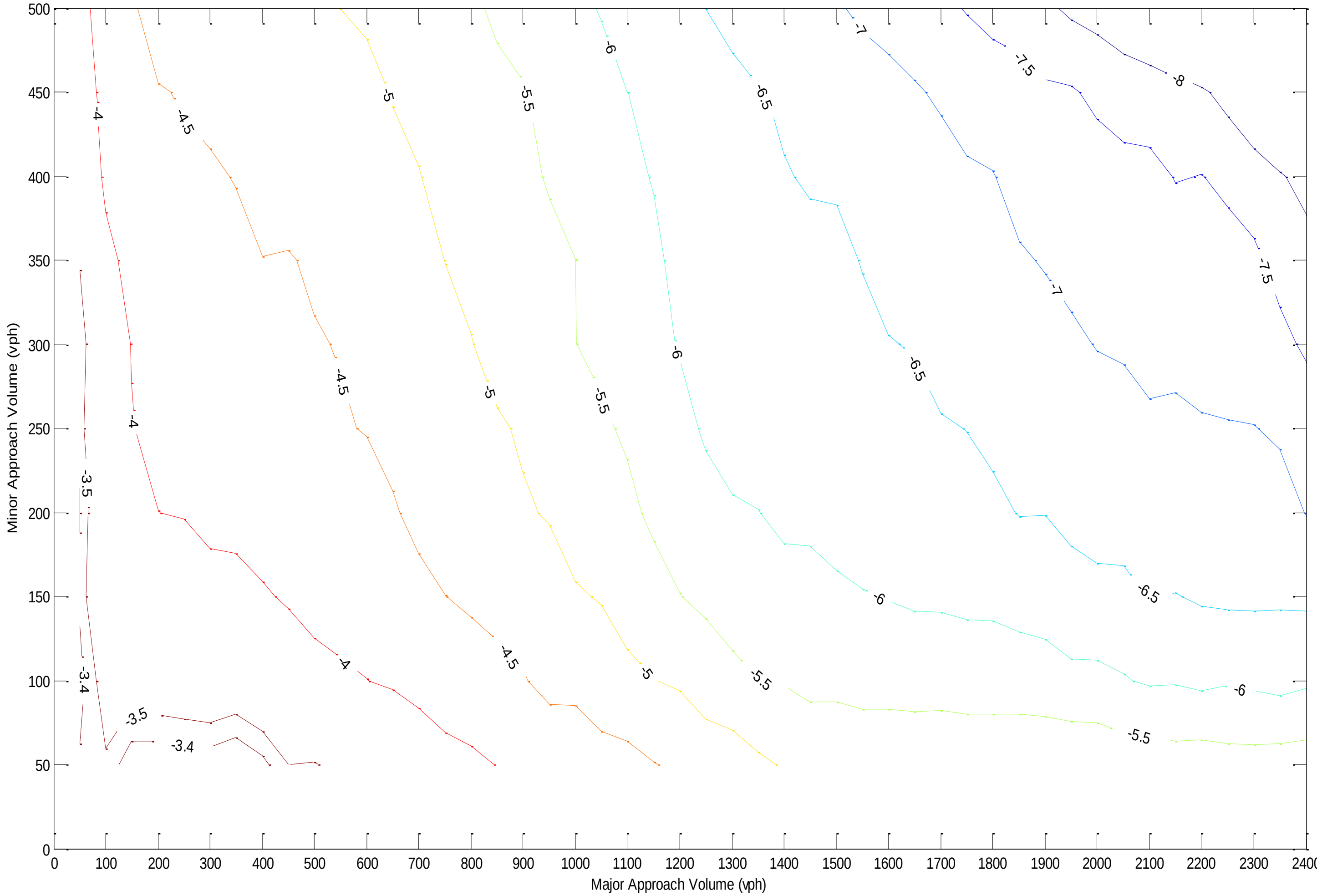


Figure B.74 Contour plot at BF=0.6, LTP=5 %, TP=10 %

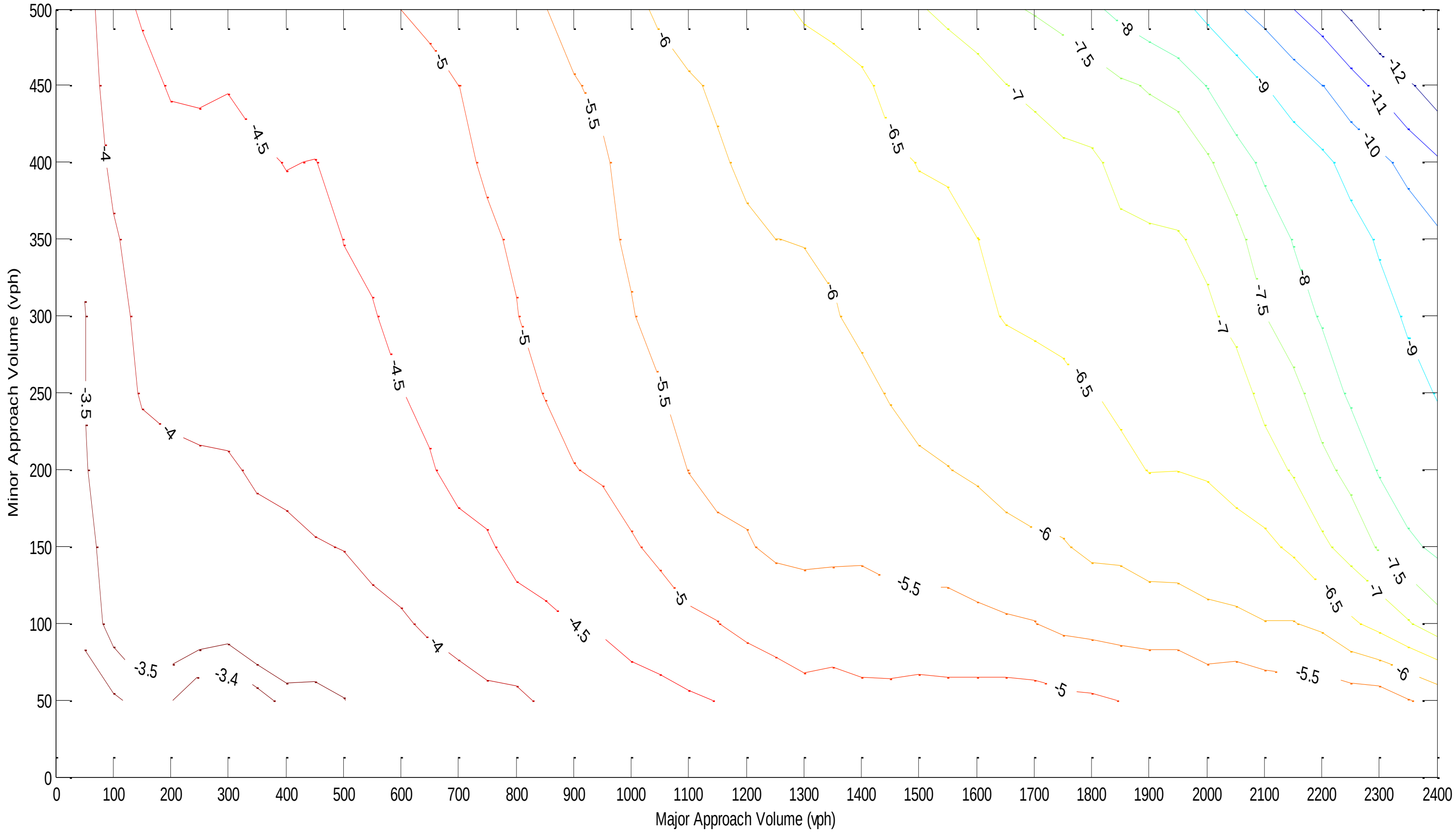


Figure B.75 Contour plot at BF=0.7, LTP=5 %, TP=10 %

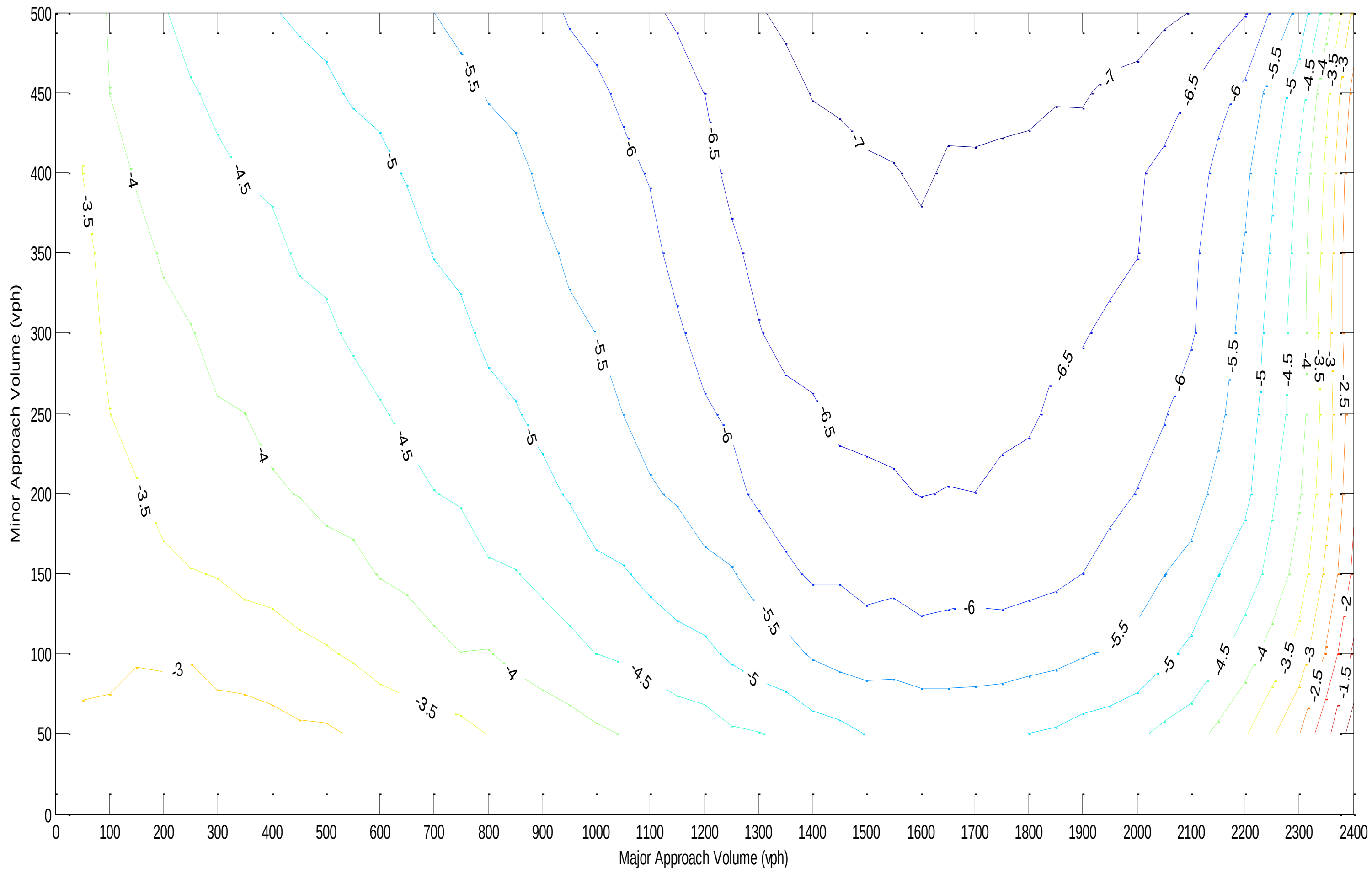


Figure B.76 Contour plot at BF=0.5, LTP=10%, TP=10 %

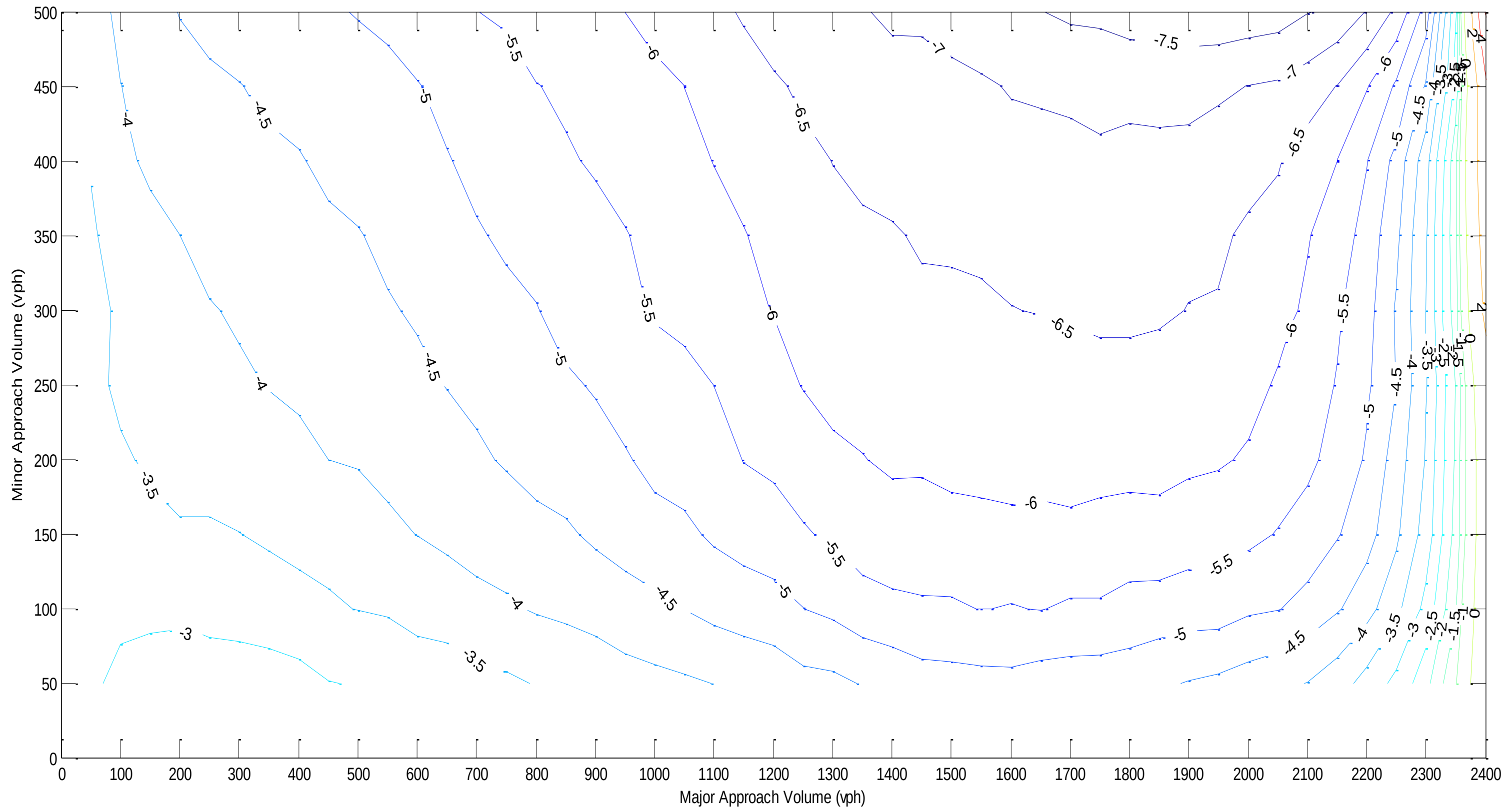


Figure B.77 Contour plot at BF=0.6, LTP=10 %, TP=10 %

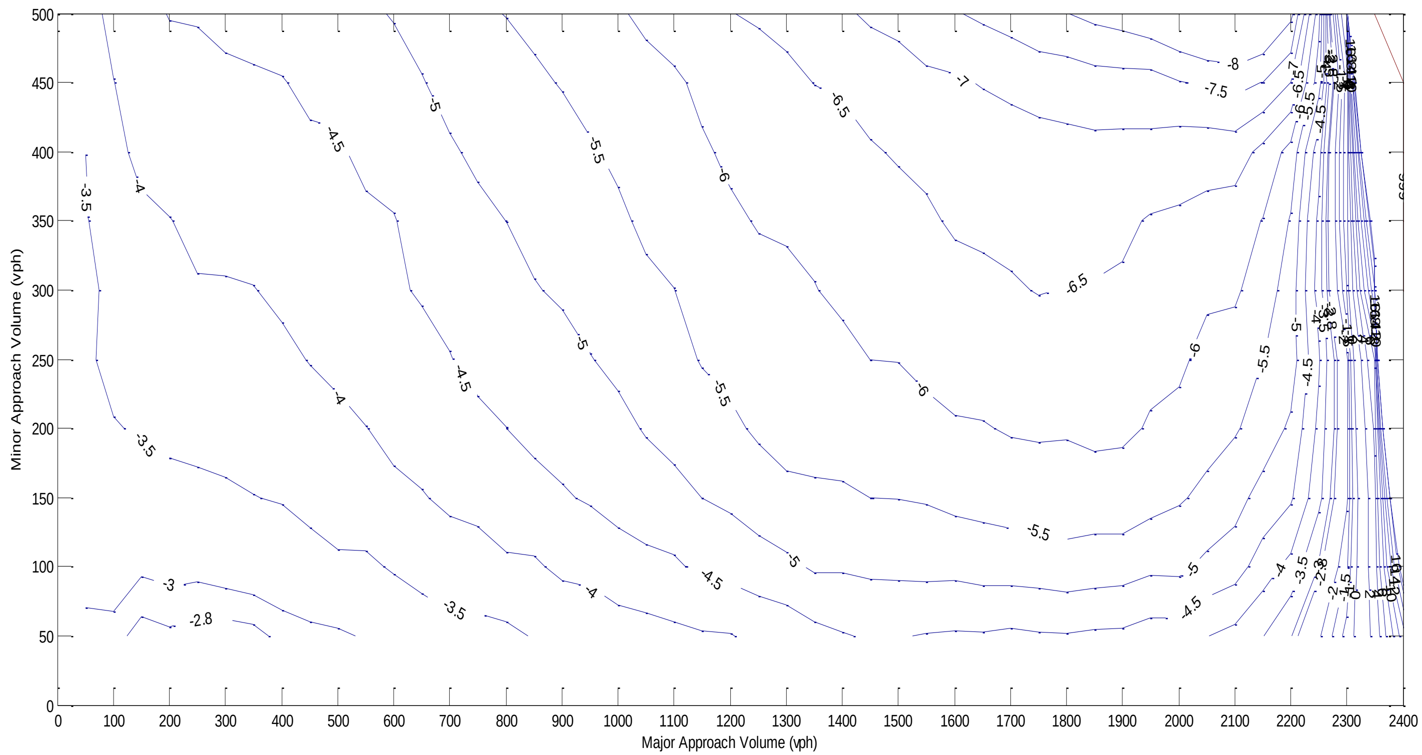


Figure B.78 Contour plot at BF=0.7, LTP=10 %, TP=10 %

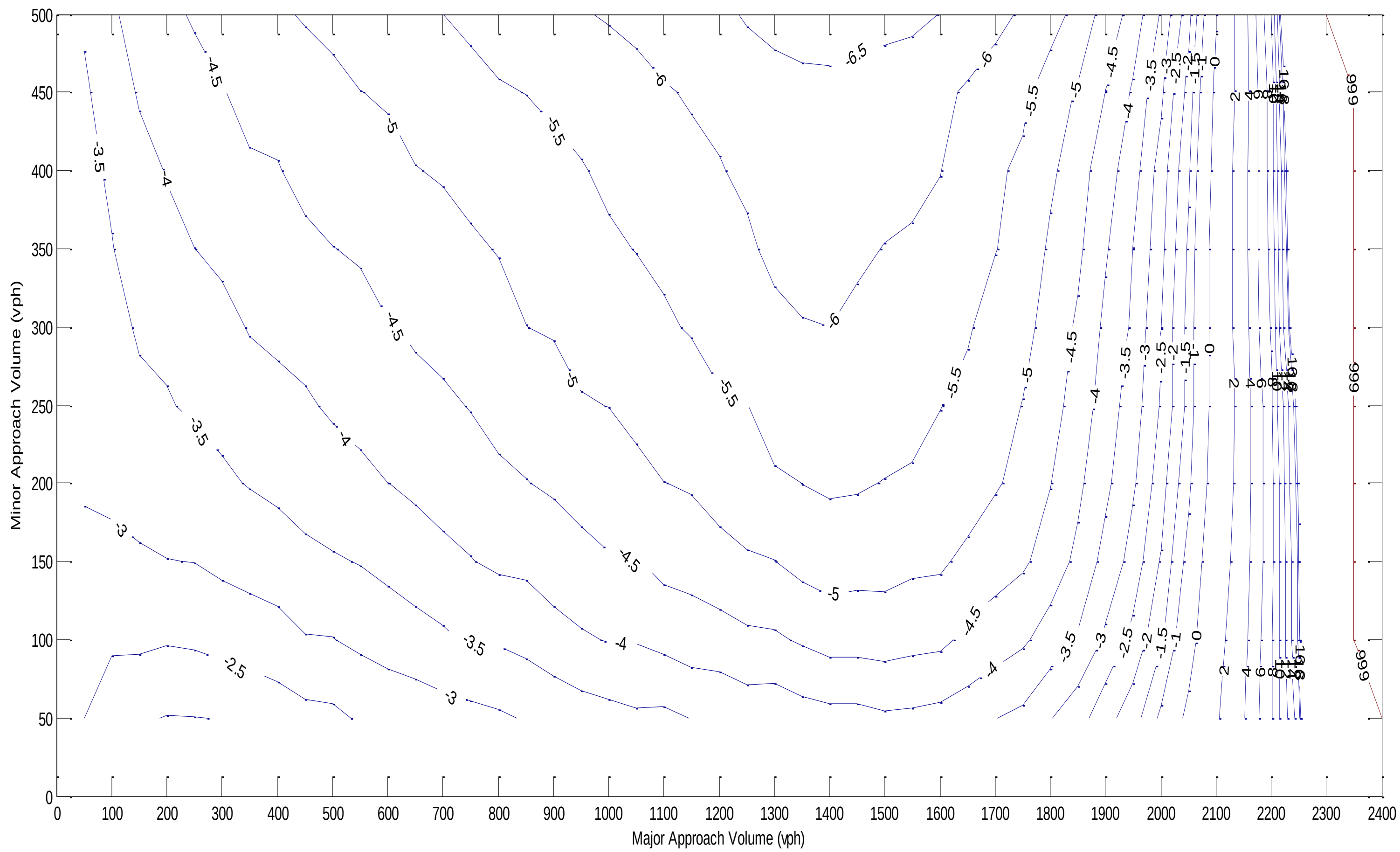


Figure B.79 Contour plot at BF=0.5, LTP=15 %, TP=10 %

(Note: 999 represents failure condition of Median Opening)

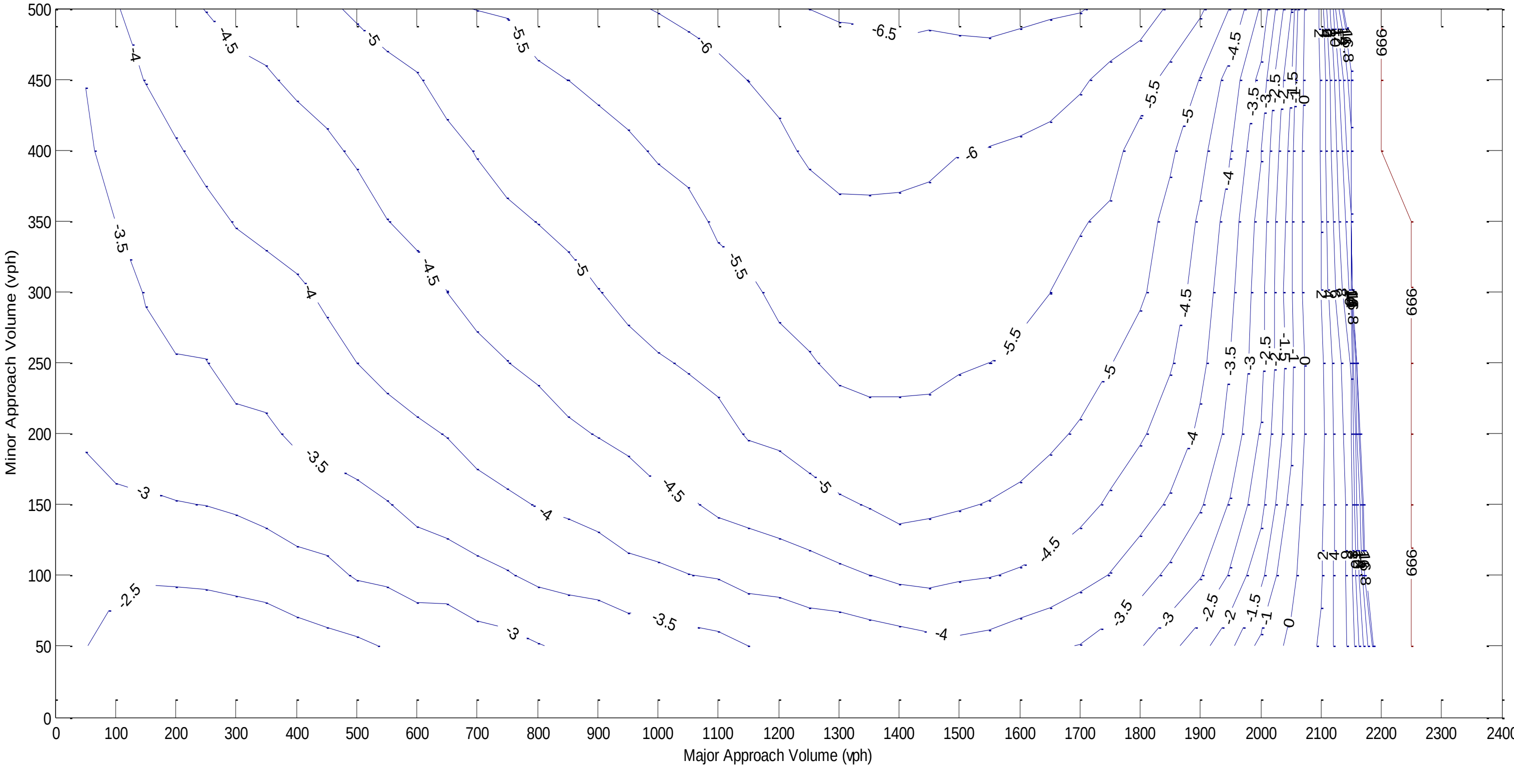


Figure B.80 Contour plot at BF=0.6, LTP=15 %, TP=10 %

(Note: 999 represents failure condition of Median Opening)

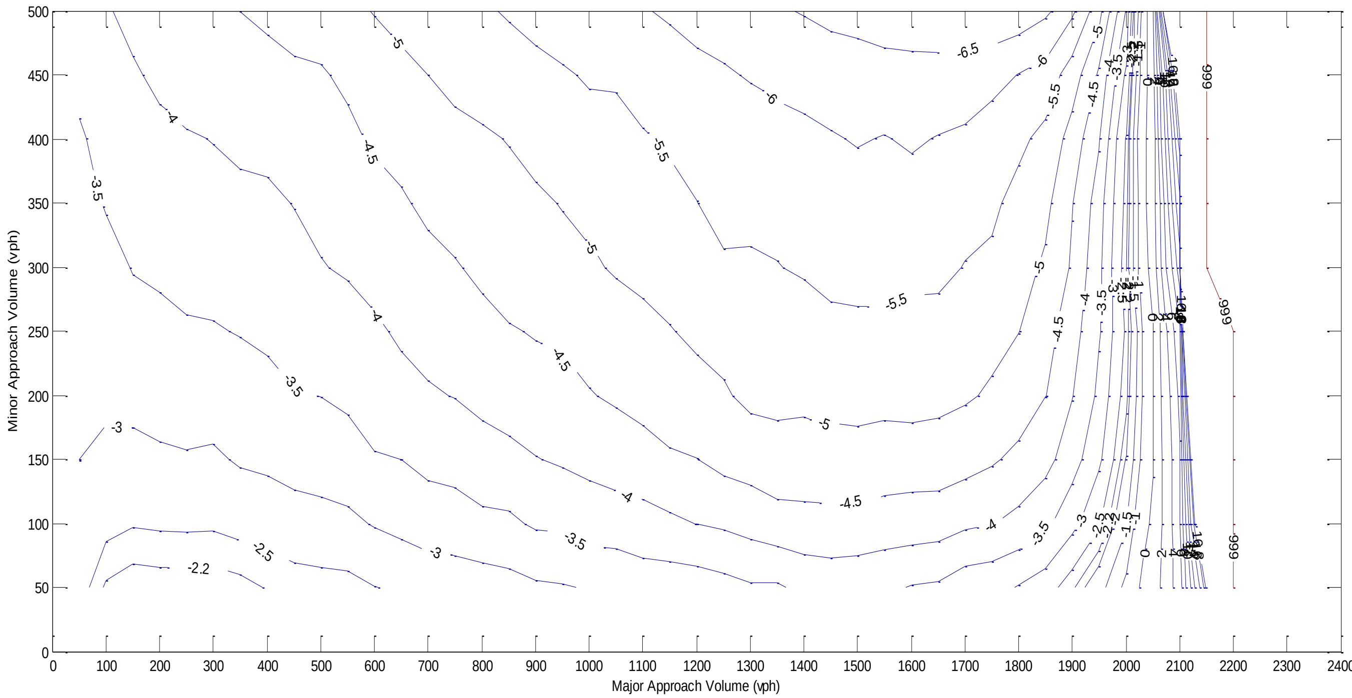


Figure B.81 Contour plot at BF=0.7, LTP=15 %, TP=10 %

(Note: 999 represents failure condition of Median Opening)

B.2. Contour Plots for Total Intersection Delay Difference of CFI with Standard Signalized Intersection

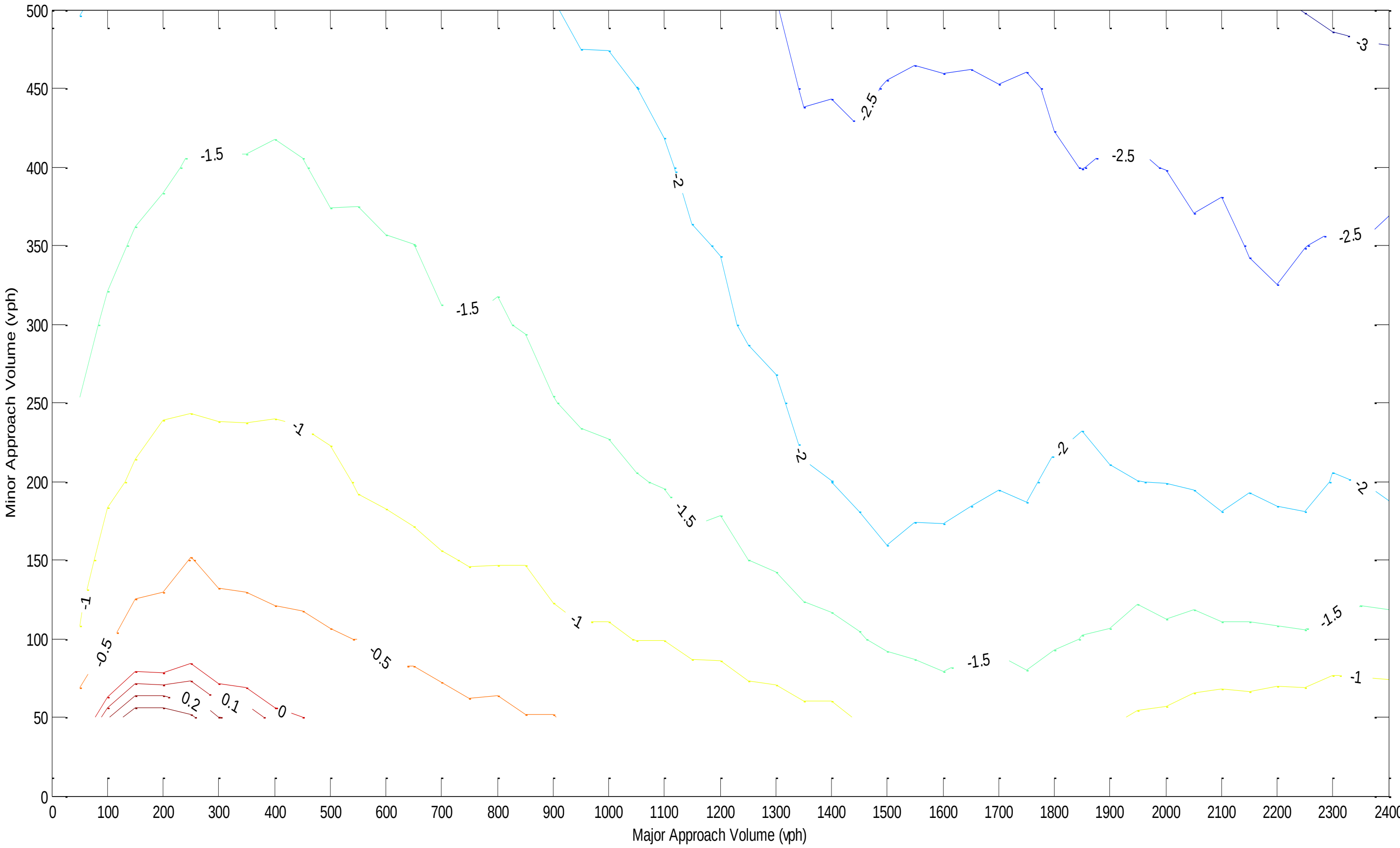


Figure B.82 Contour plot at BF=0.5, LTP=5%, TP=2%

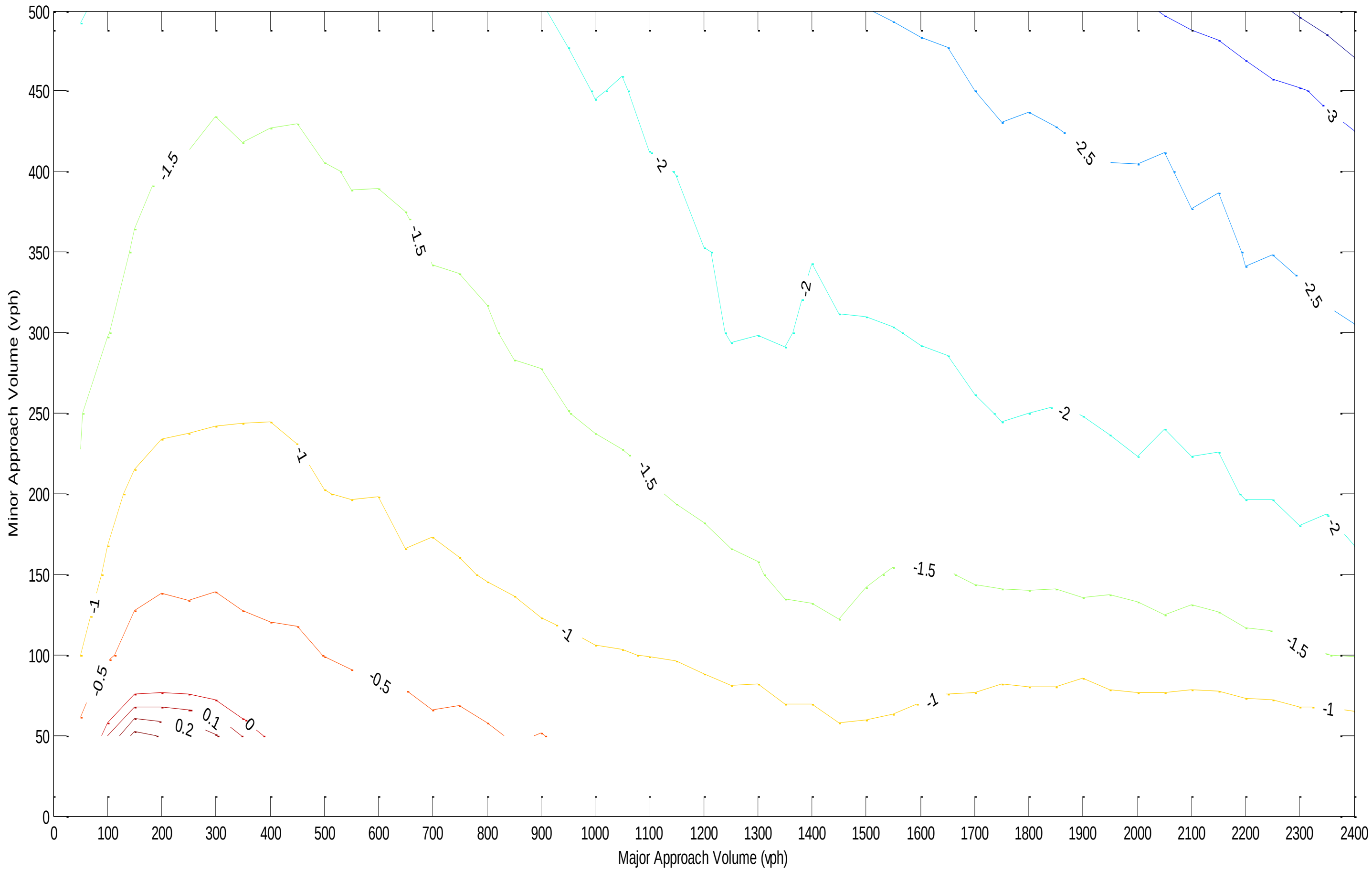


Figure B.83 Contour plot at BF=0.6, LTP=5%, TP=2%

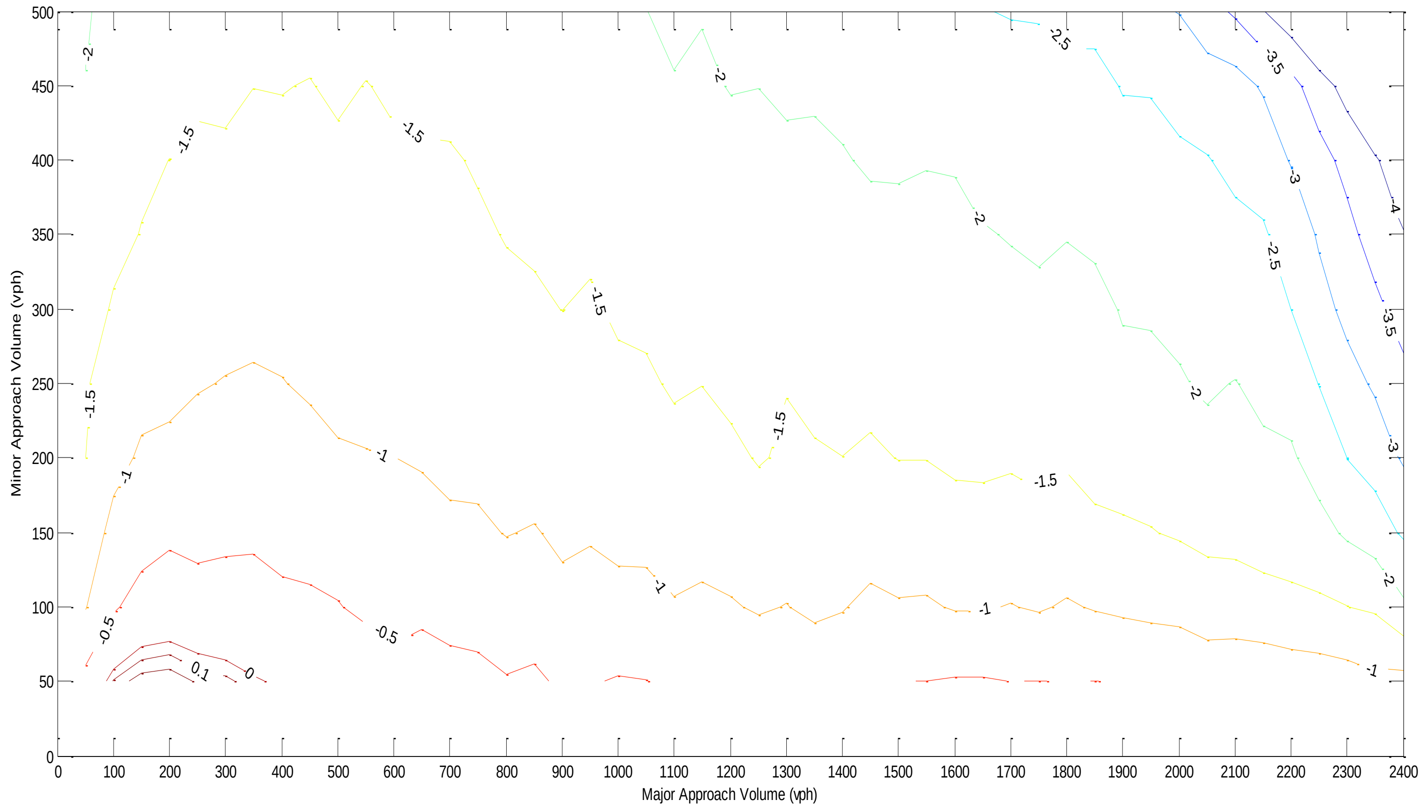


Figure B.84 Contour plot at BF=0.7, LTP=5%, TP=2%

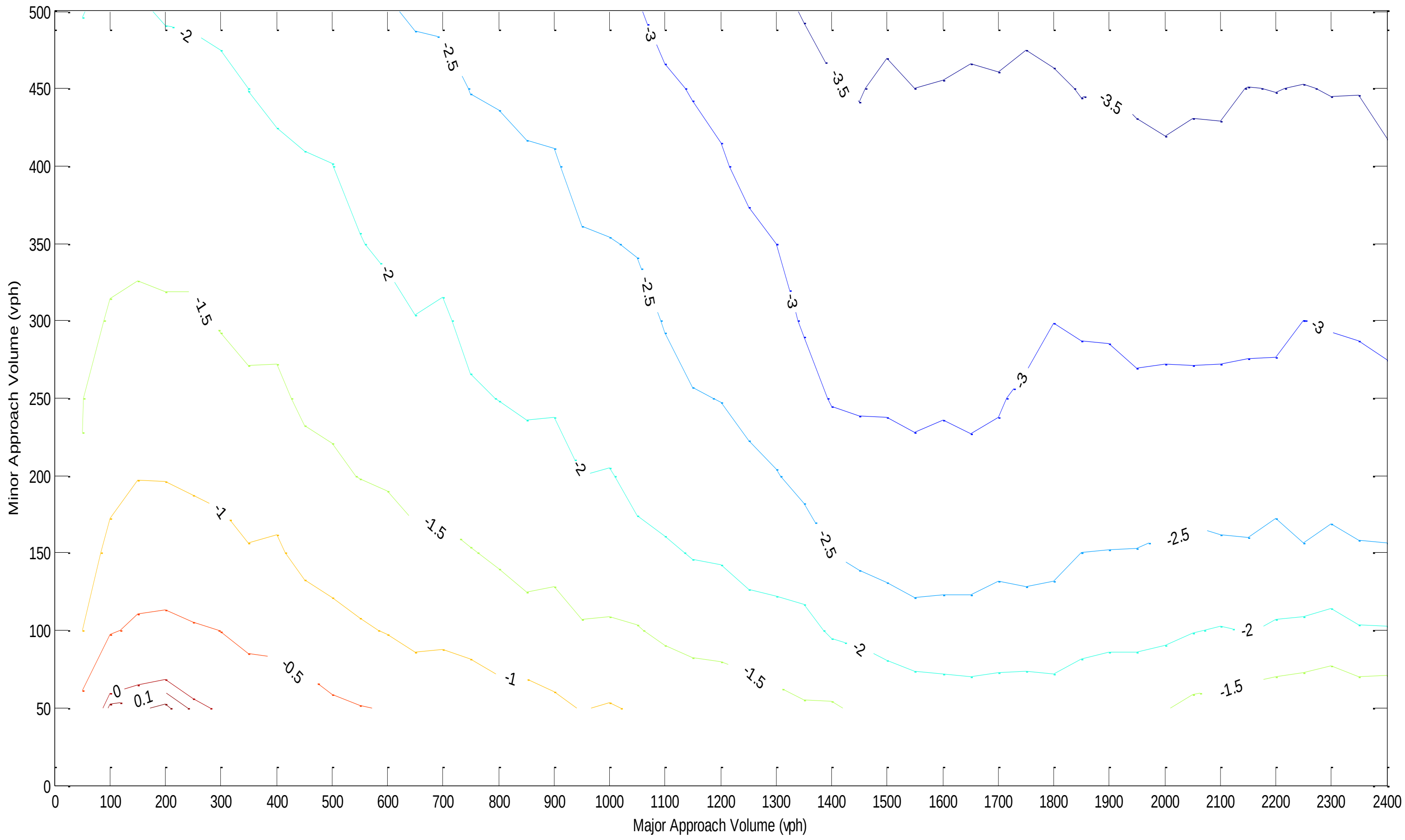


Figure B.85 Contour plot at BF=0.5, LTP=10%, TP=2%

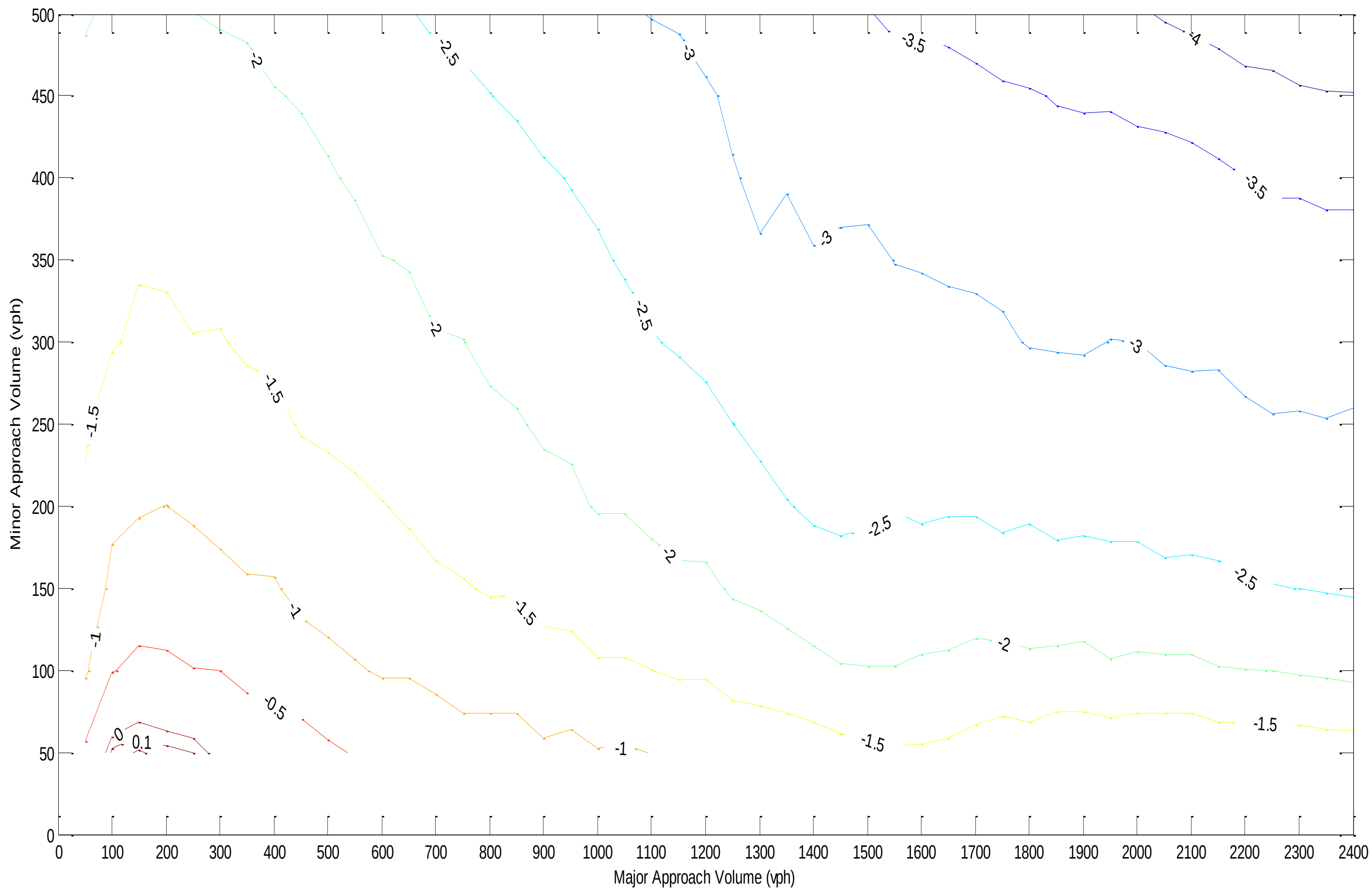


Figure B.86 Contour plot at BF=0.6, LTP=10%, TP=2%

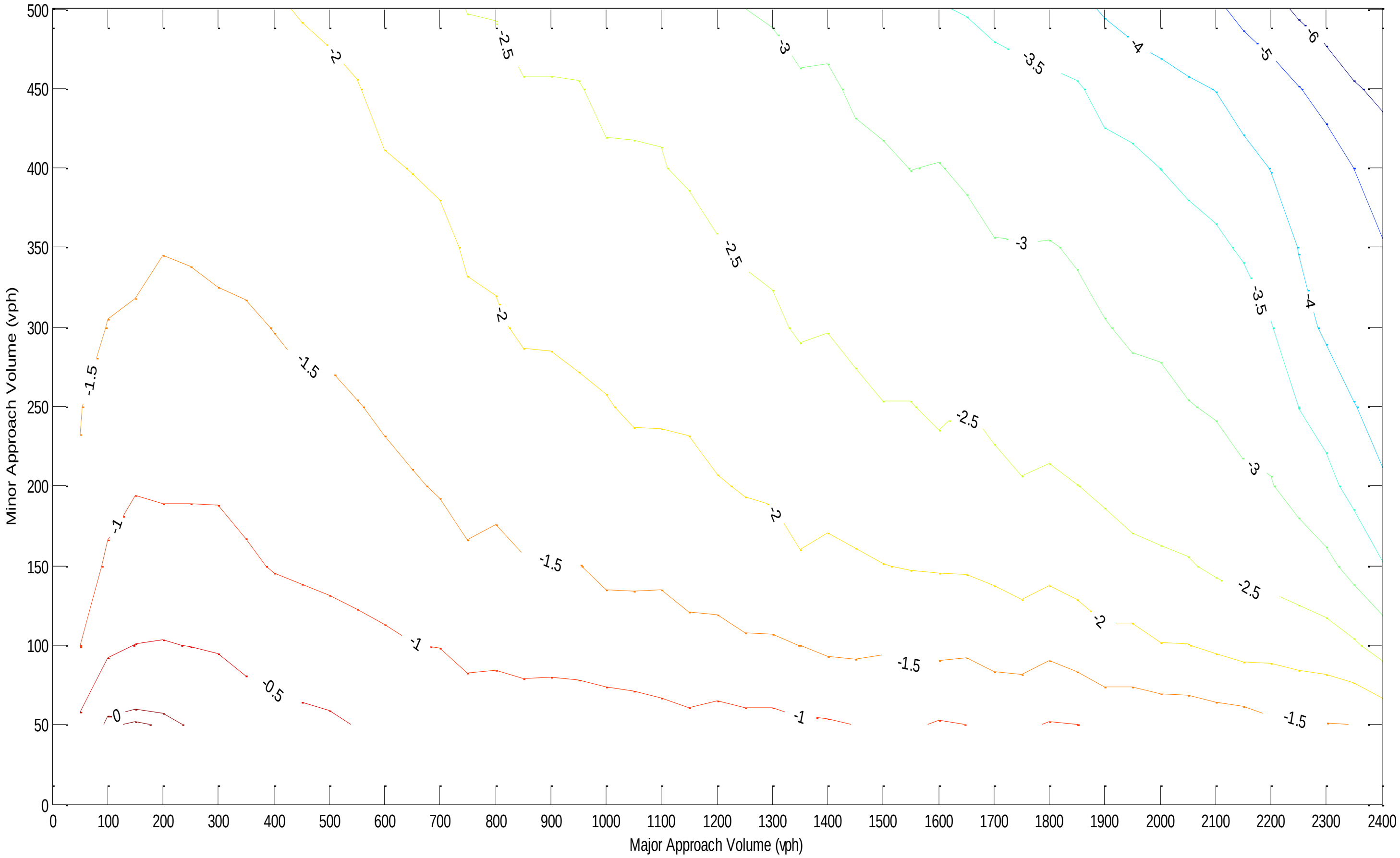


Figure B.87 Contour plot at BF=0.7, LTP=10%, TP=2%

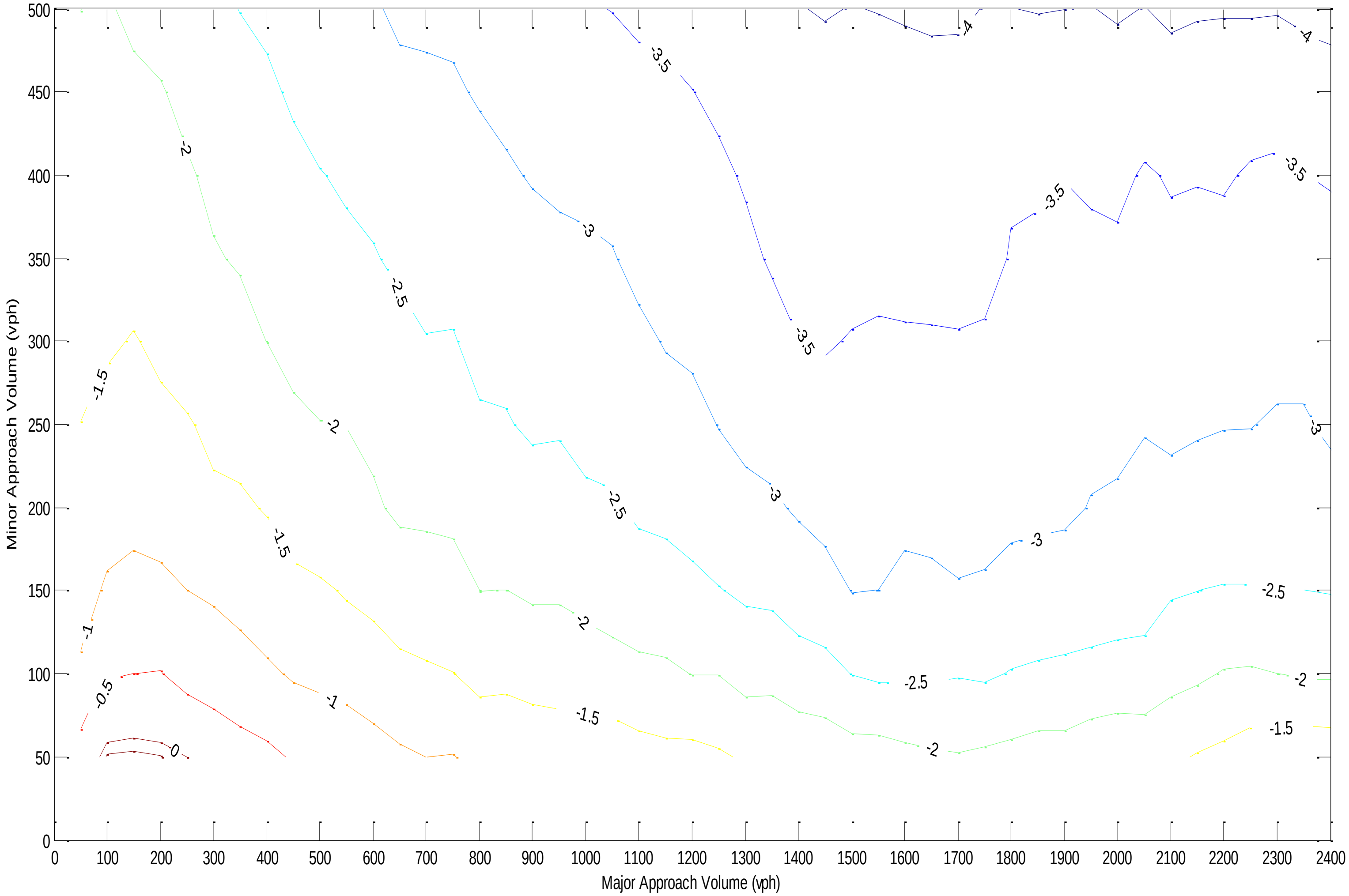


Figure B.88 Contour plot at BF=0.5, LTP=15%, TP=2%

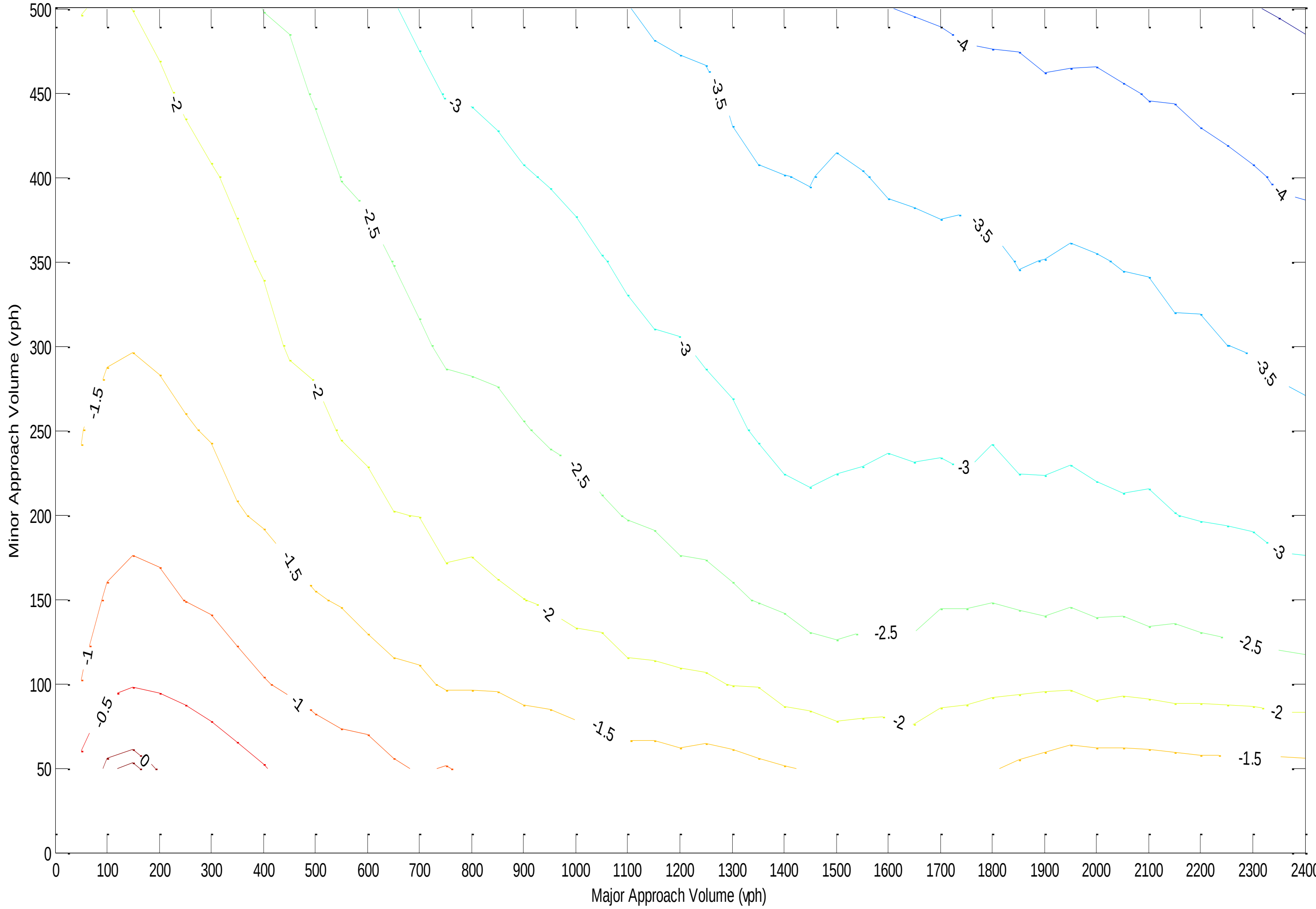


Figure B.89 Contour plot at BF=0.6, LTP=15%, TP=2%

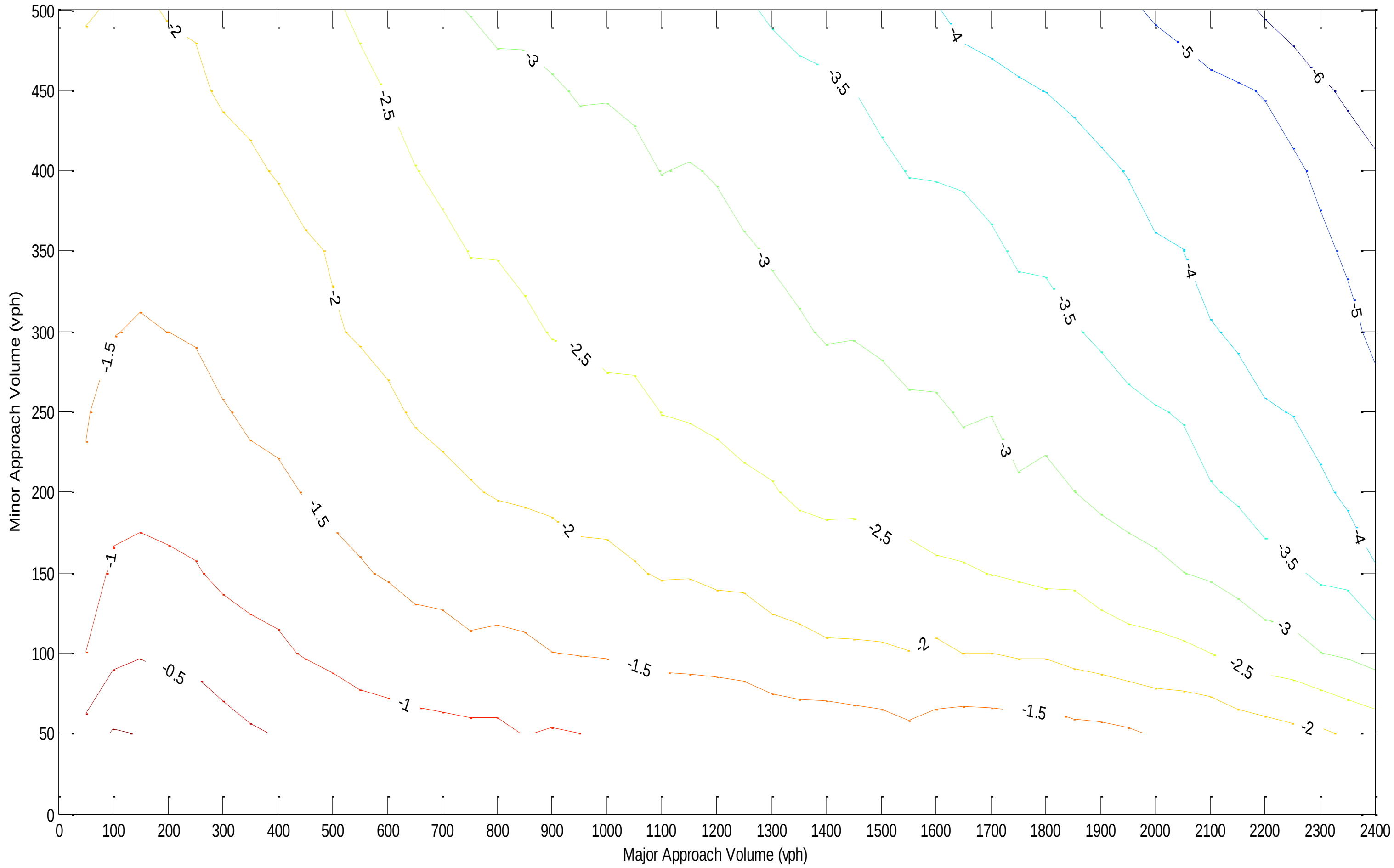


Figure B.90 Contour plot at BF=0.7, LTP=15%, TP=2%

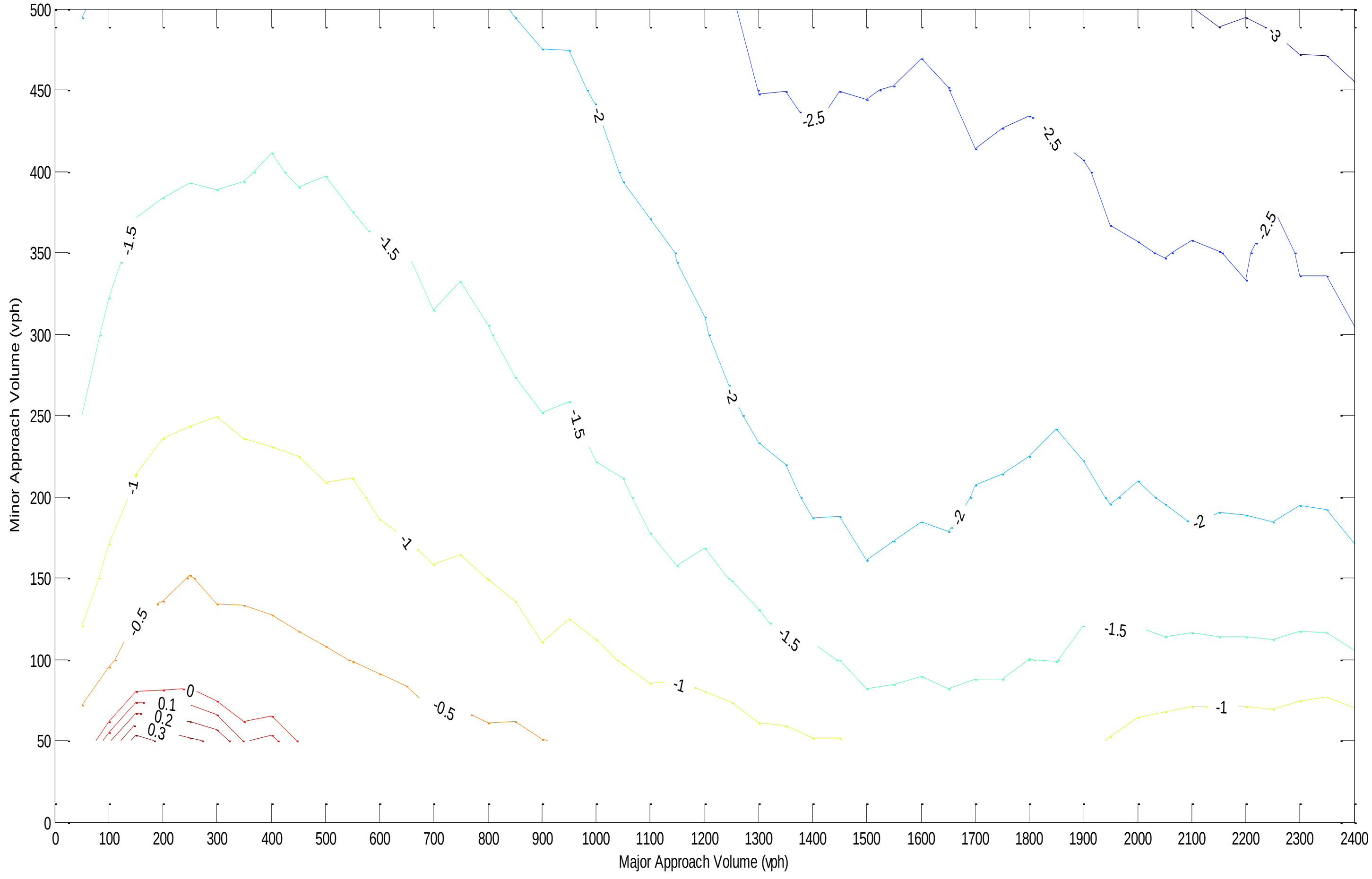


Figure B.91 Contour plot at BF=0.5, LTP=5 %, TP=5 %

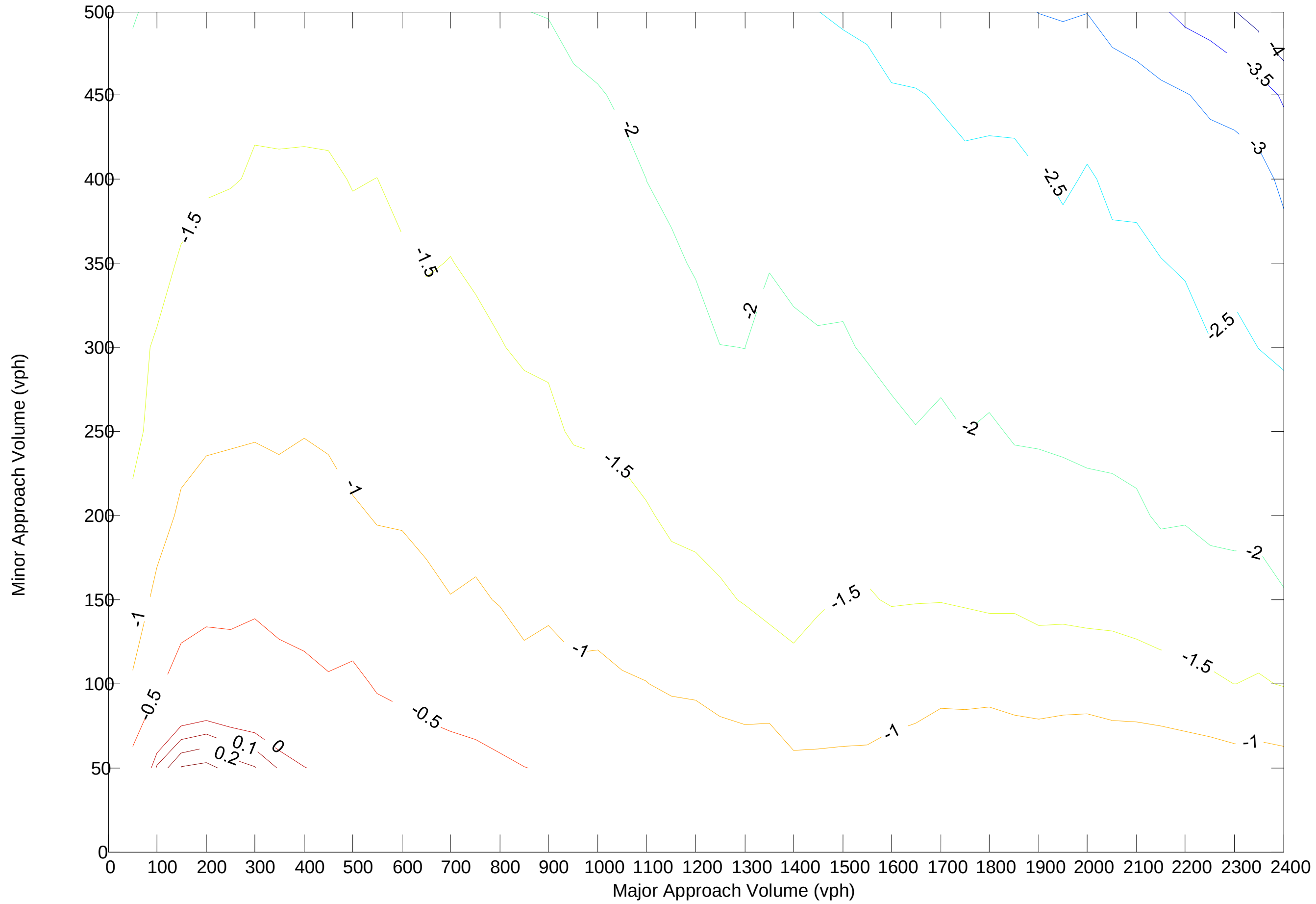


Figure B.92 Contour plot at BF=0.6, LTP=5 %, TP=5 %

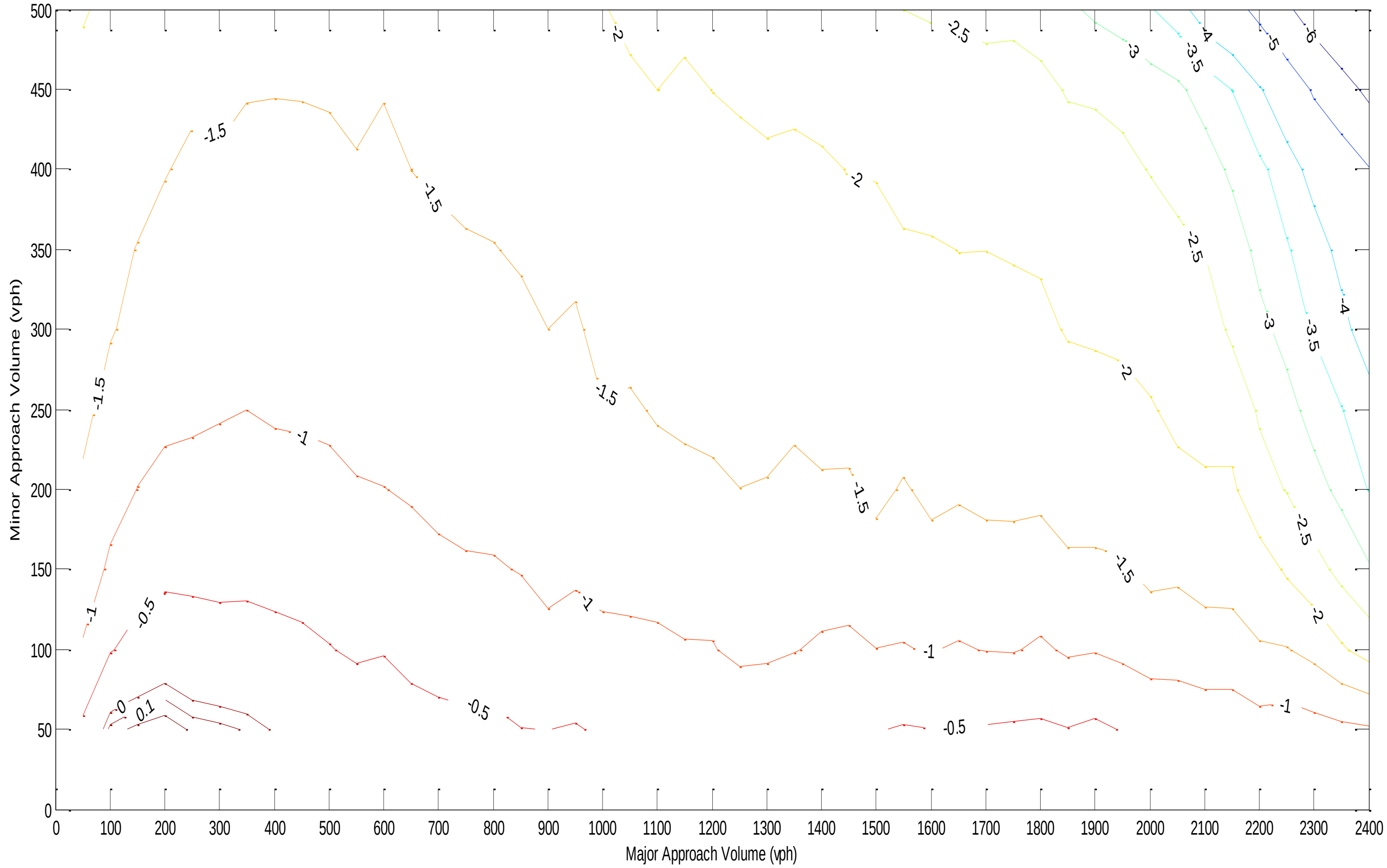


Figure B.93 Contour plot at BF=0.7, LTP=5 %, TP=5 %

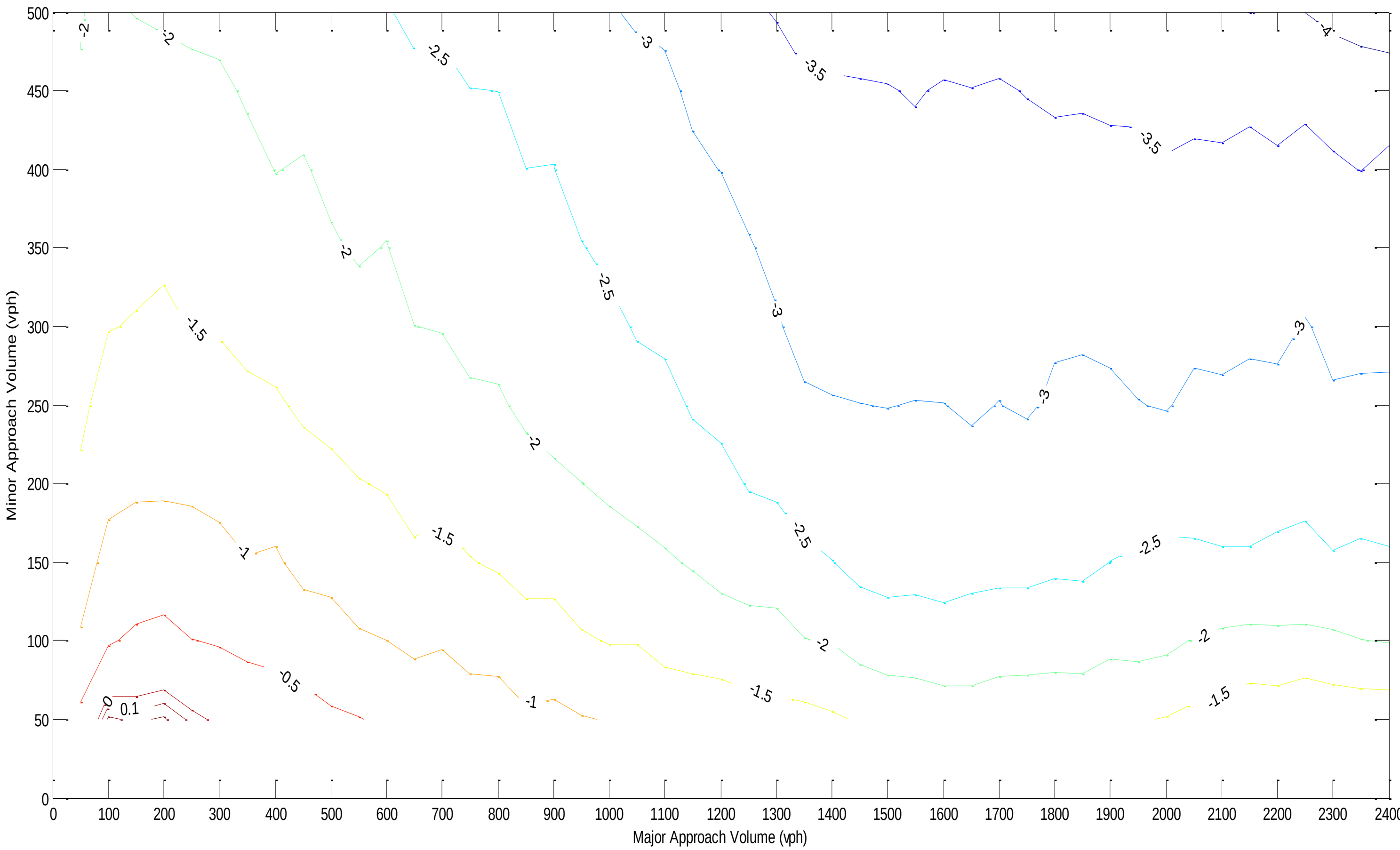


Figure B.94 Contour plot at BF=0.5, LTP=10 %, TP=5 %

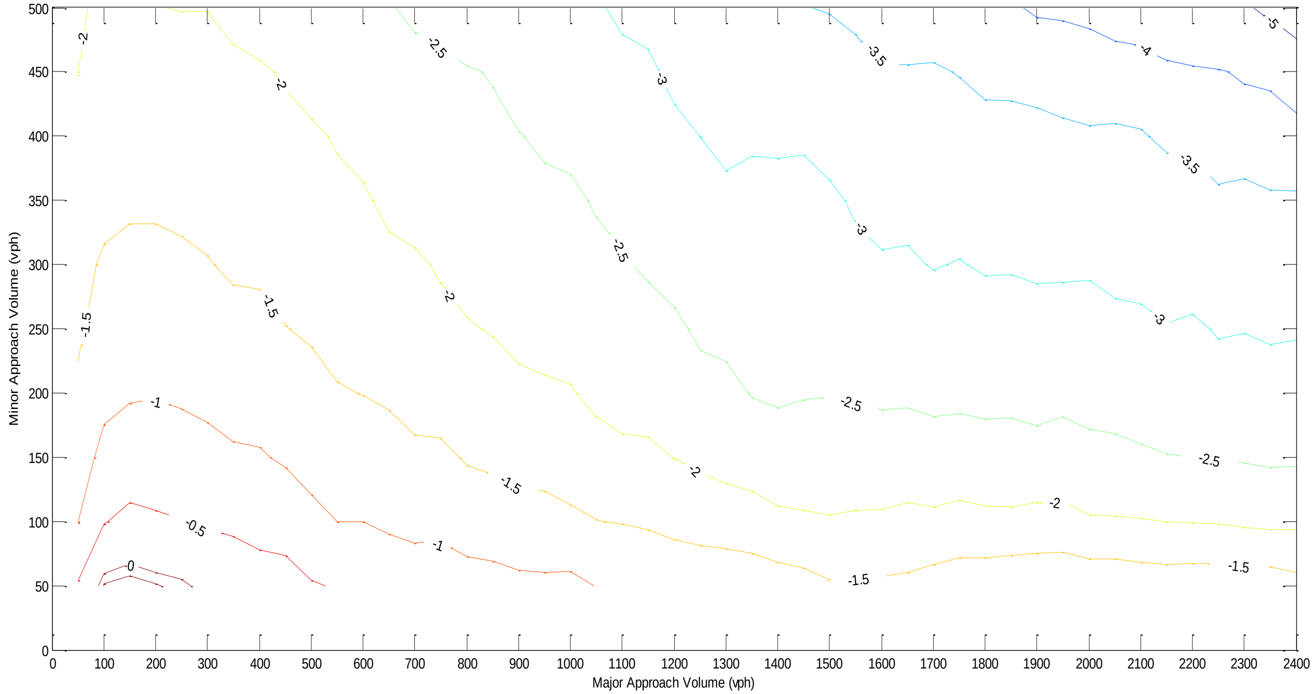


Figure B.95 Contour plot at BF=0.6, LTP=10 %, TP=5 %

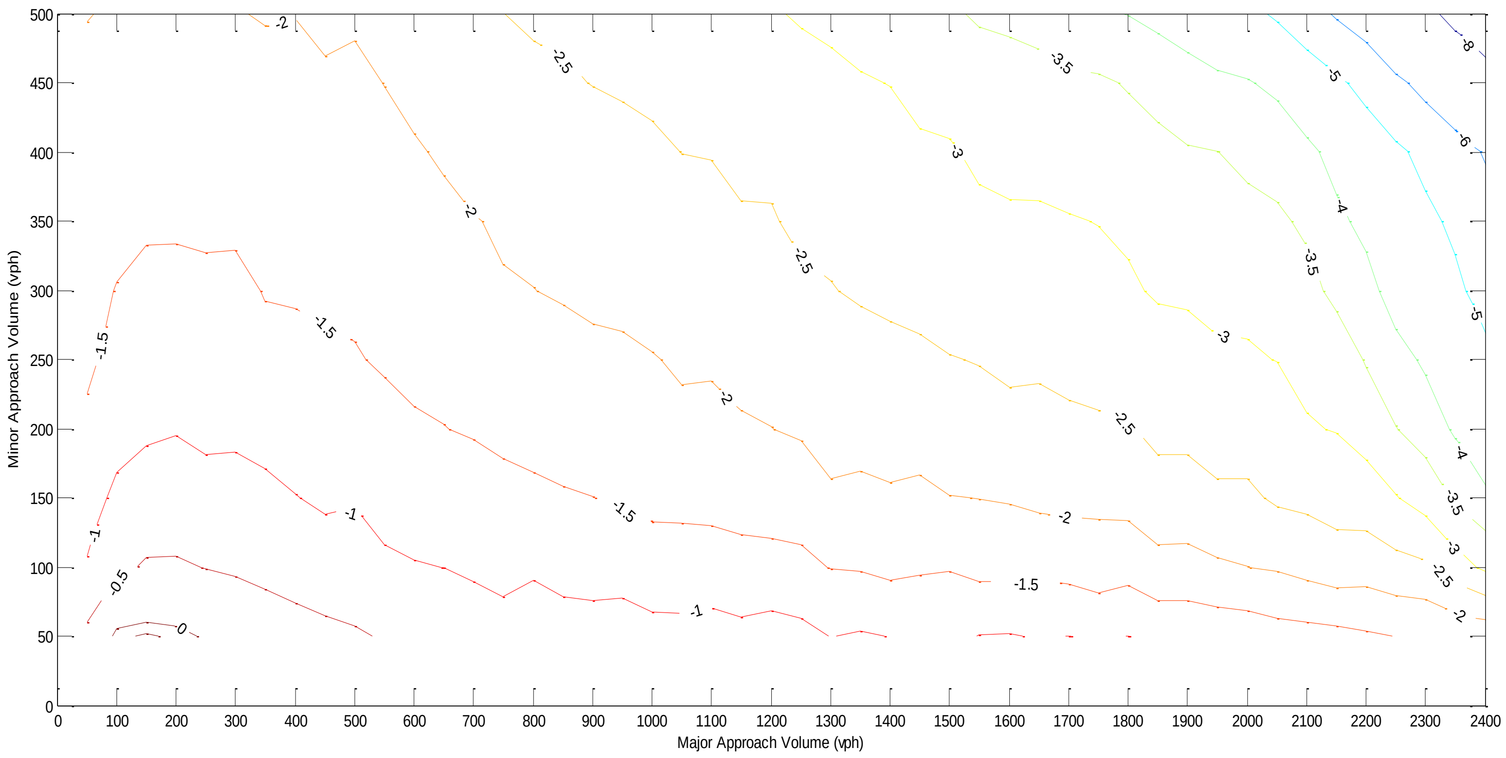


Figure B.96 Contour plot at BF=0.7, LTP=10 %, TP=5 %

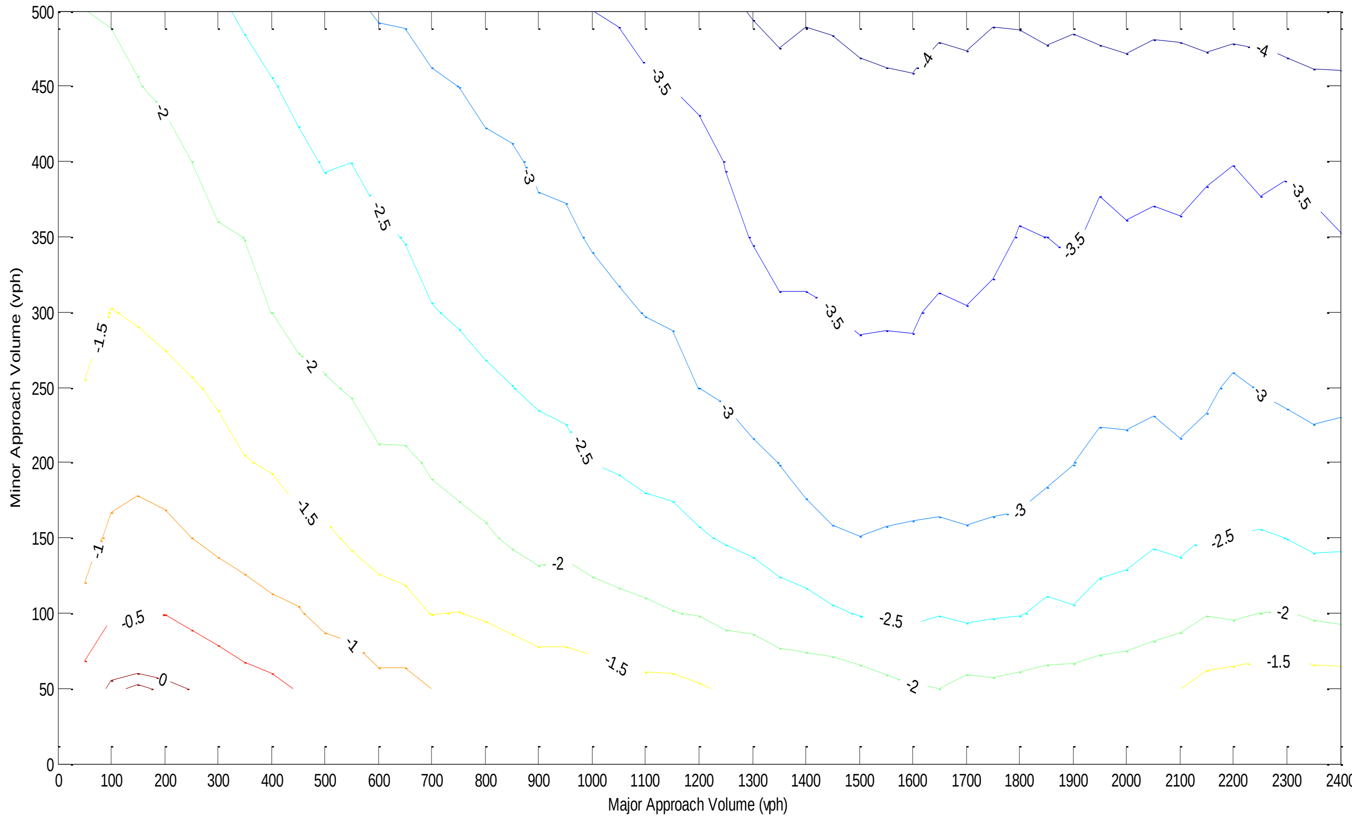


Figure B.97 Contour plot at BF=0.5, LTP=15 %, TP=5 %

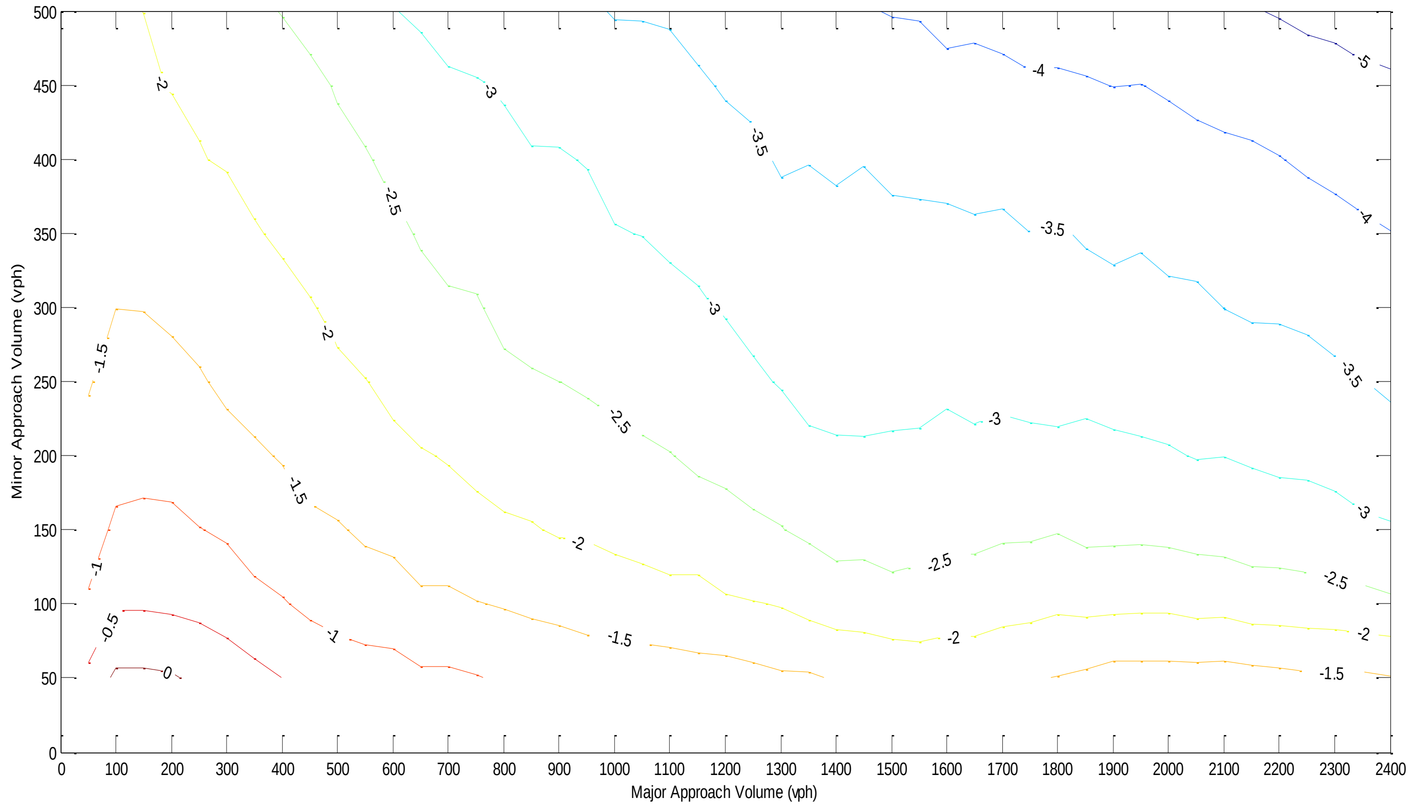


Figure B.98 Contour plot at BF=0.6, LTP=15 %, TP=5 %

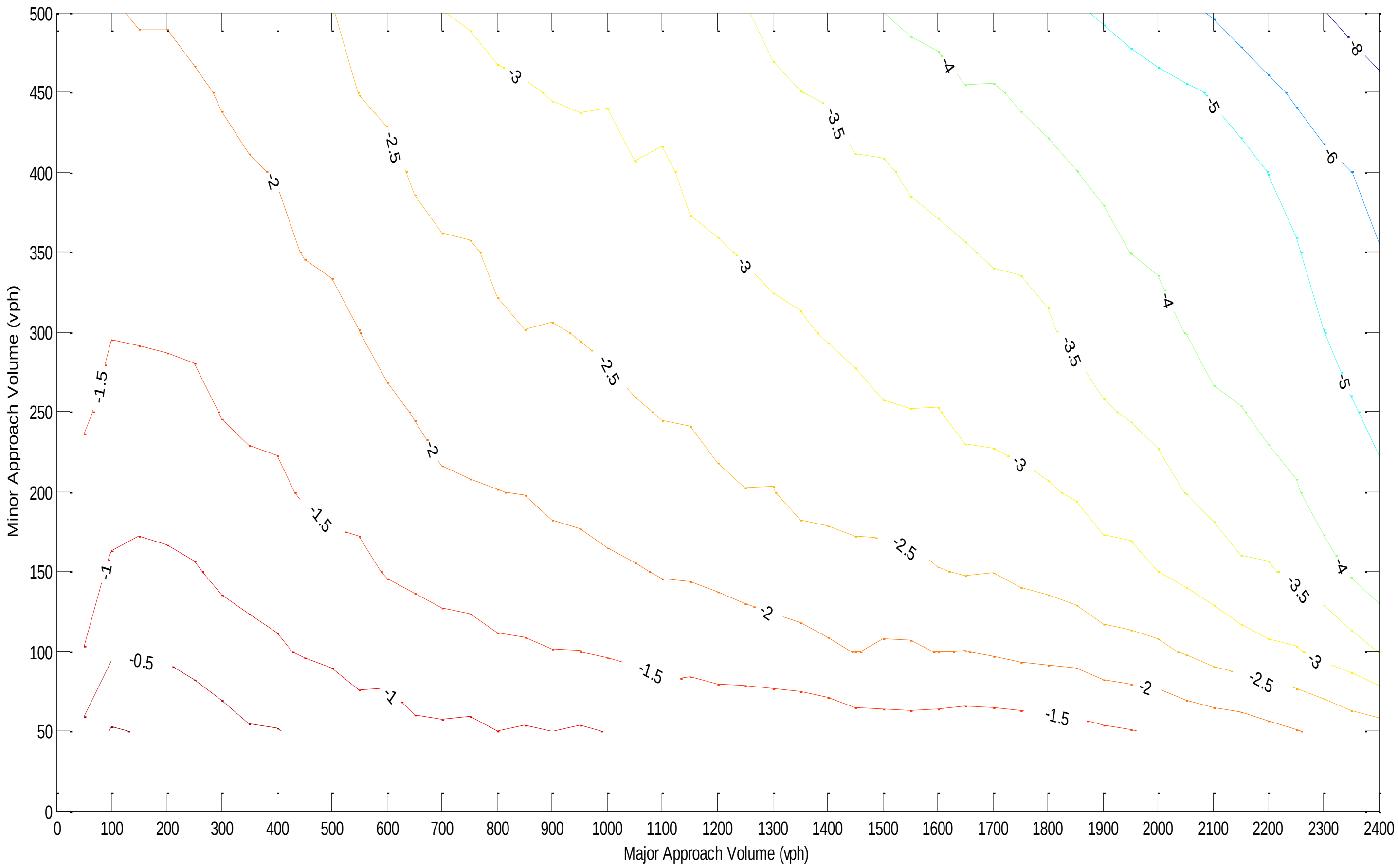


Figure B.99 Contour plot at BF=0.7, LTP=15 %, TP=5 %

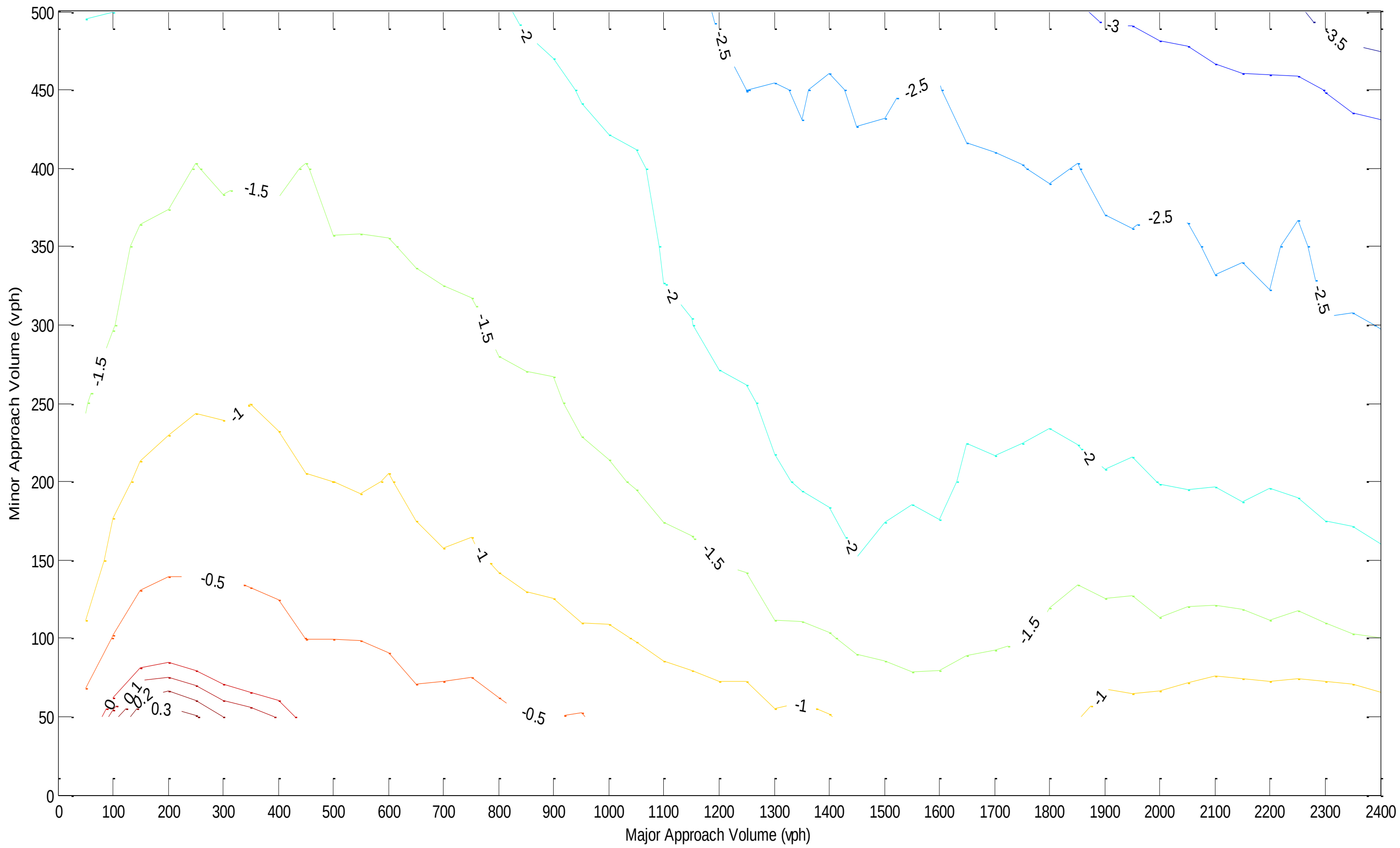


Figure B.100 Contour plot at BF=0.5, LTP=5 %, TP=10 %

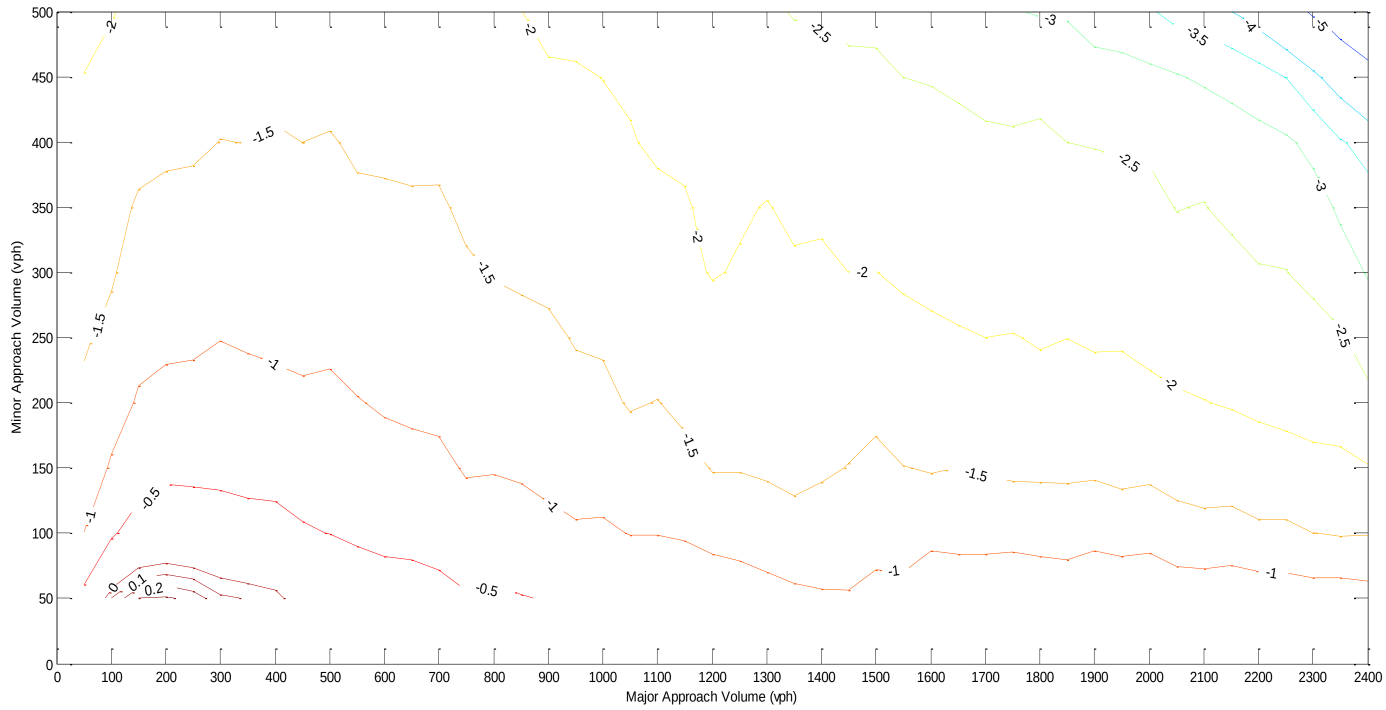


Figure B.101 Contour plot at BF=0.6, LTP=5 %, TP=10 %

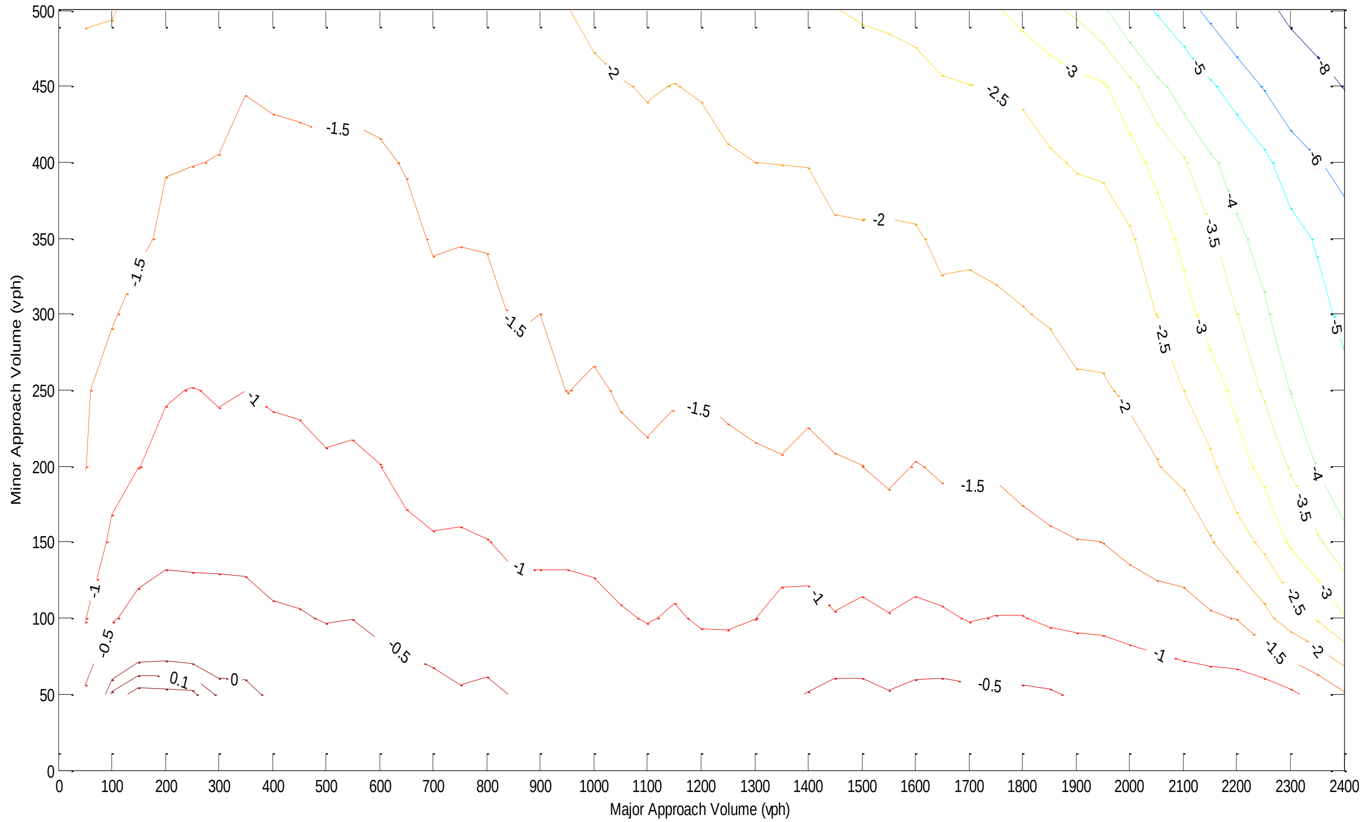


Figure B.102 Contour plot at BF=0.7, LTP=5 %, TP=10 %

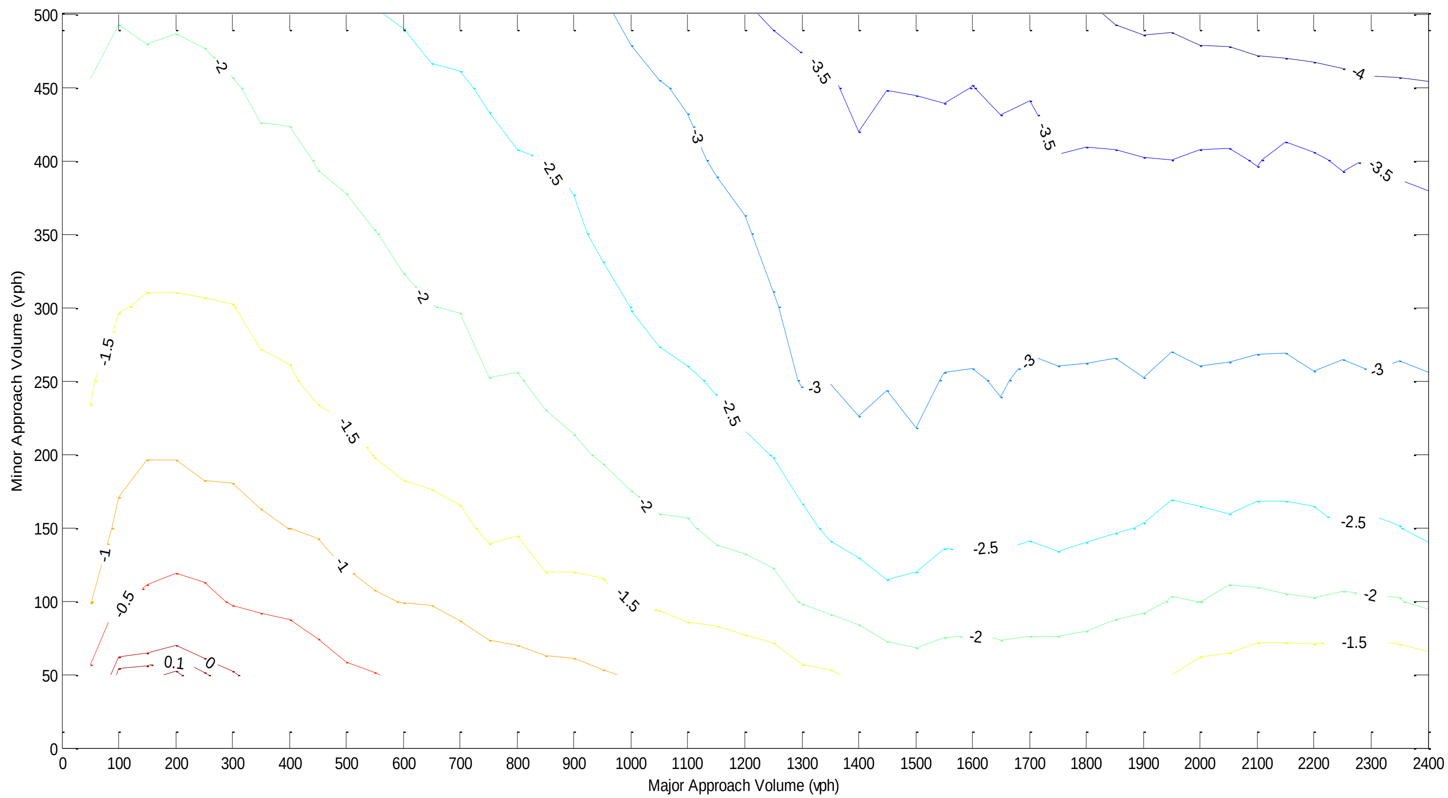


Figure B.103 Contour plot at BF=0.5, LTP=10%, TP=10 %

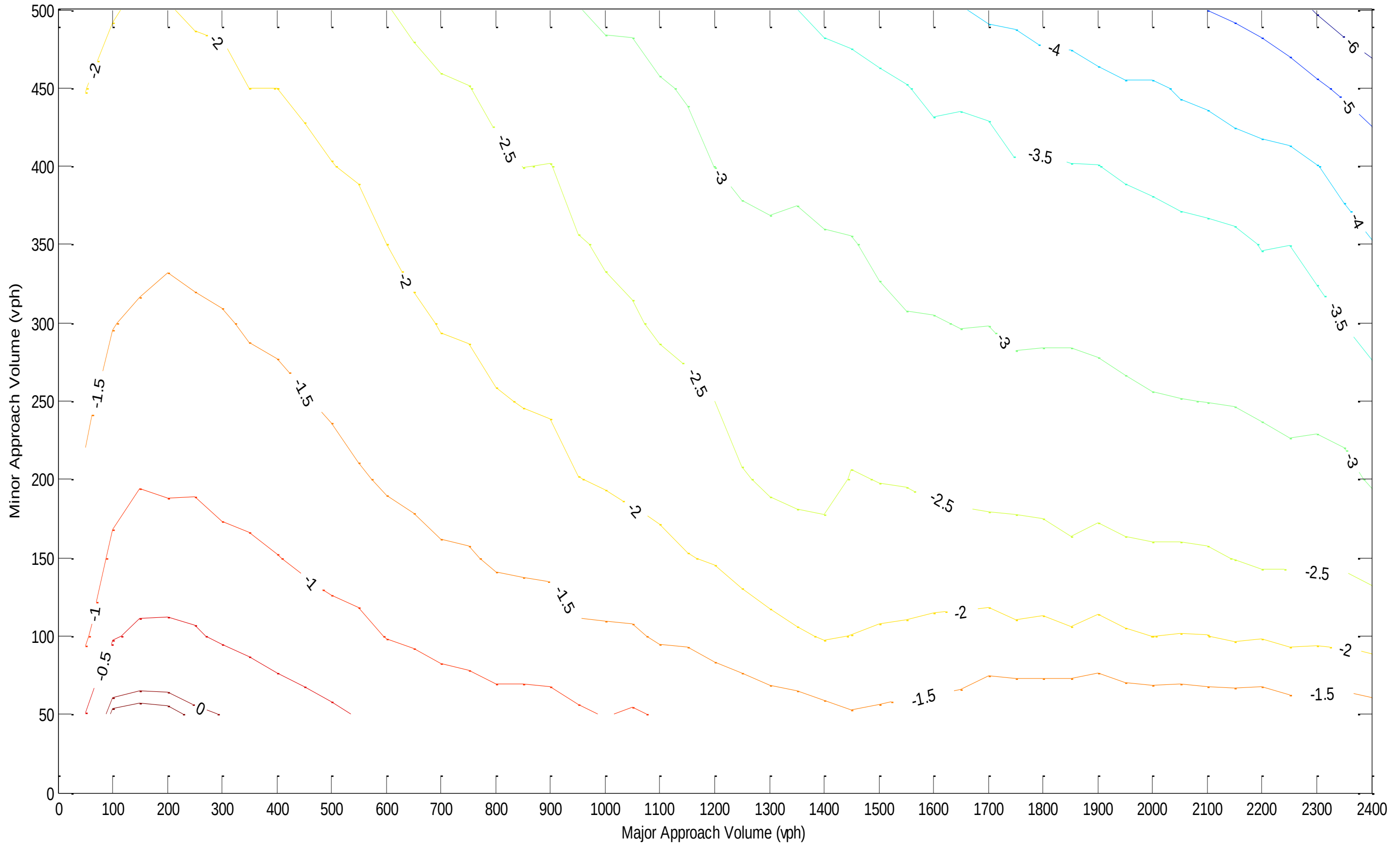


Figure B.104 Contour plot at BF=0.6, LTP=10 %, TP=10 %

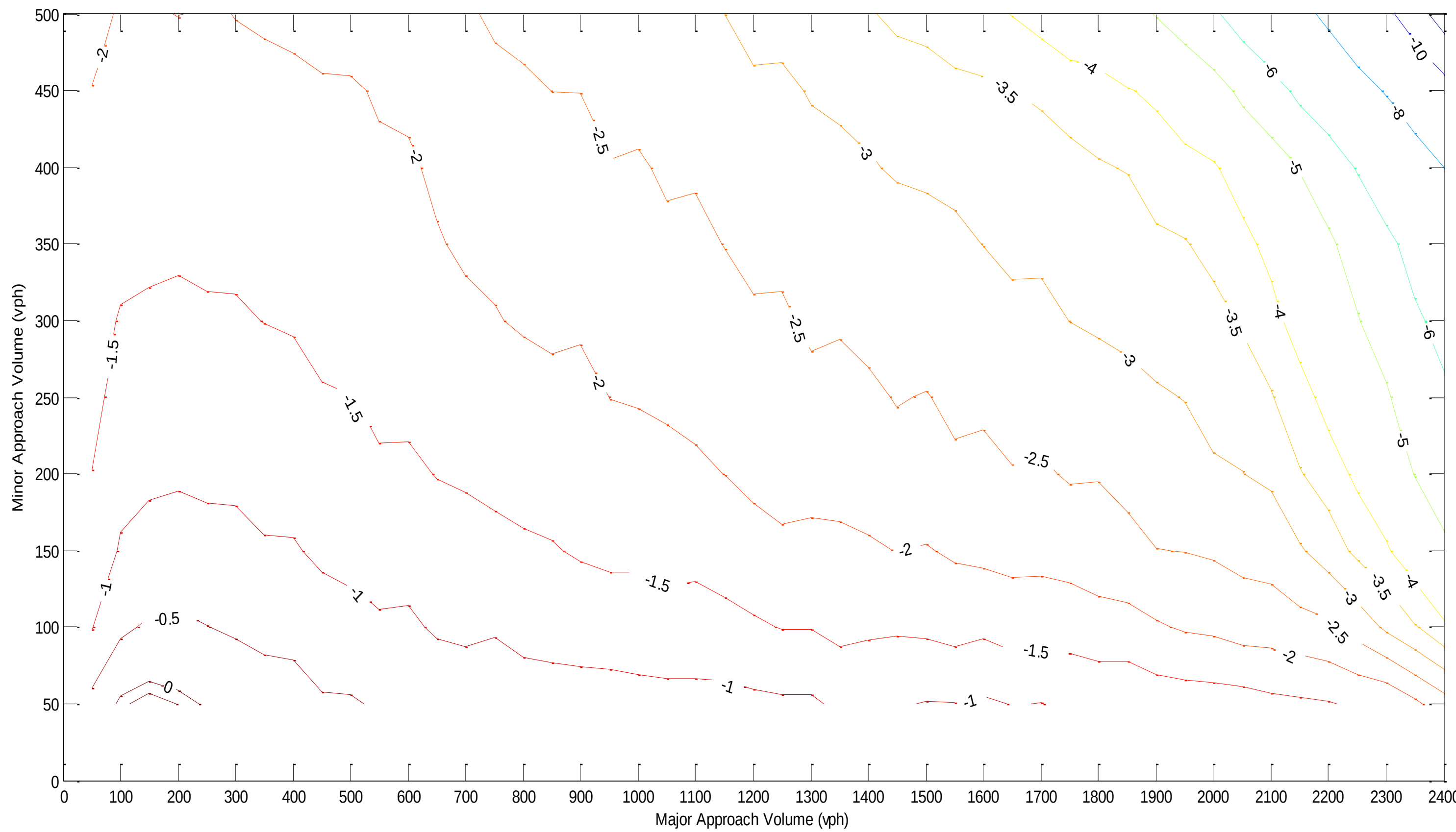


Figure B.105 Contour plot at BF=0.7, LTP=10 %, TP=10 %

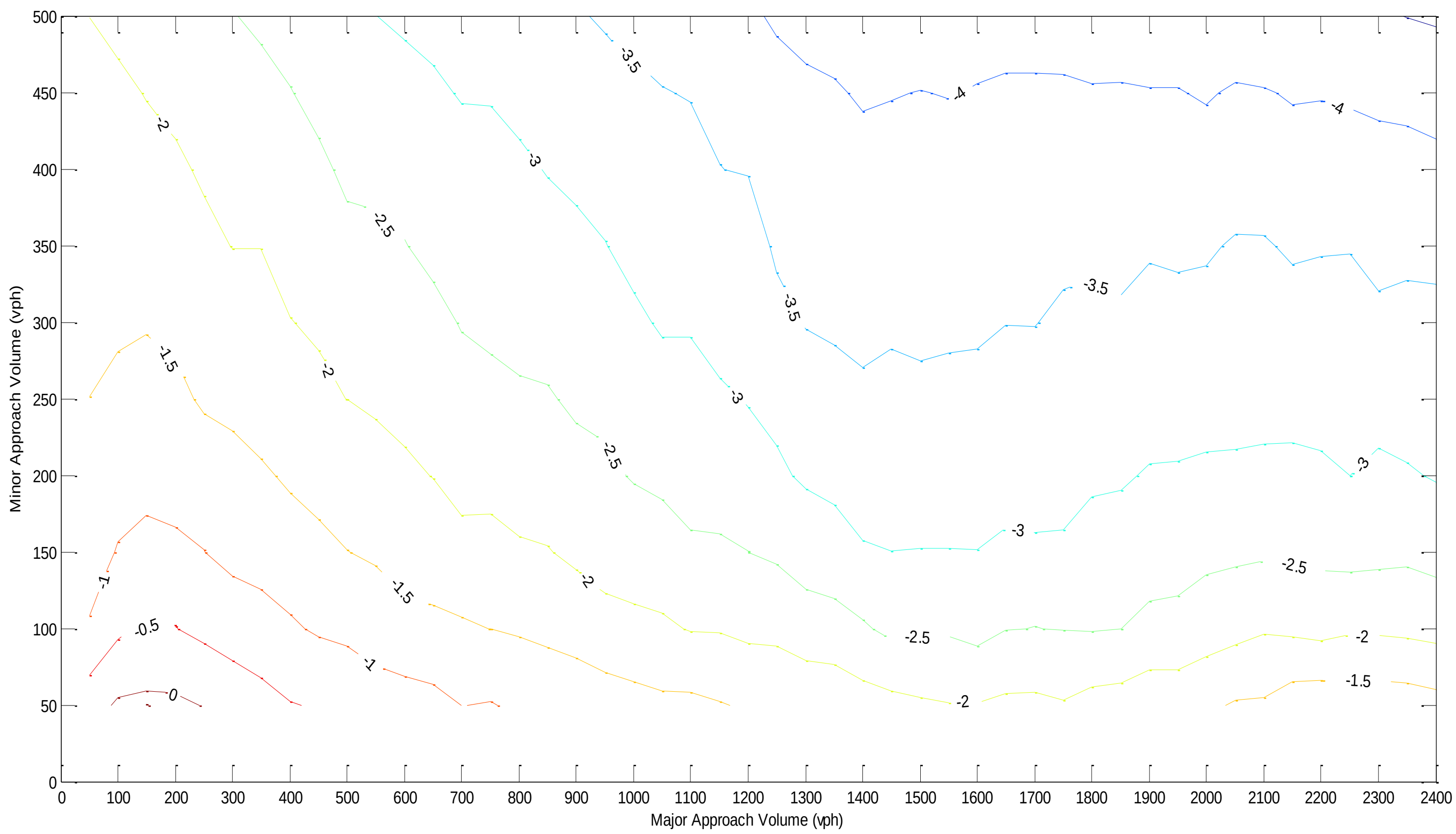


Figure B.106 Contour plot at BF=0.5, LTP=15 %, TP=10 %

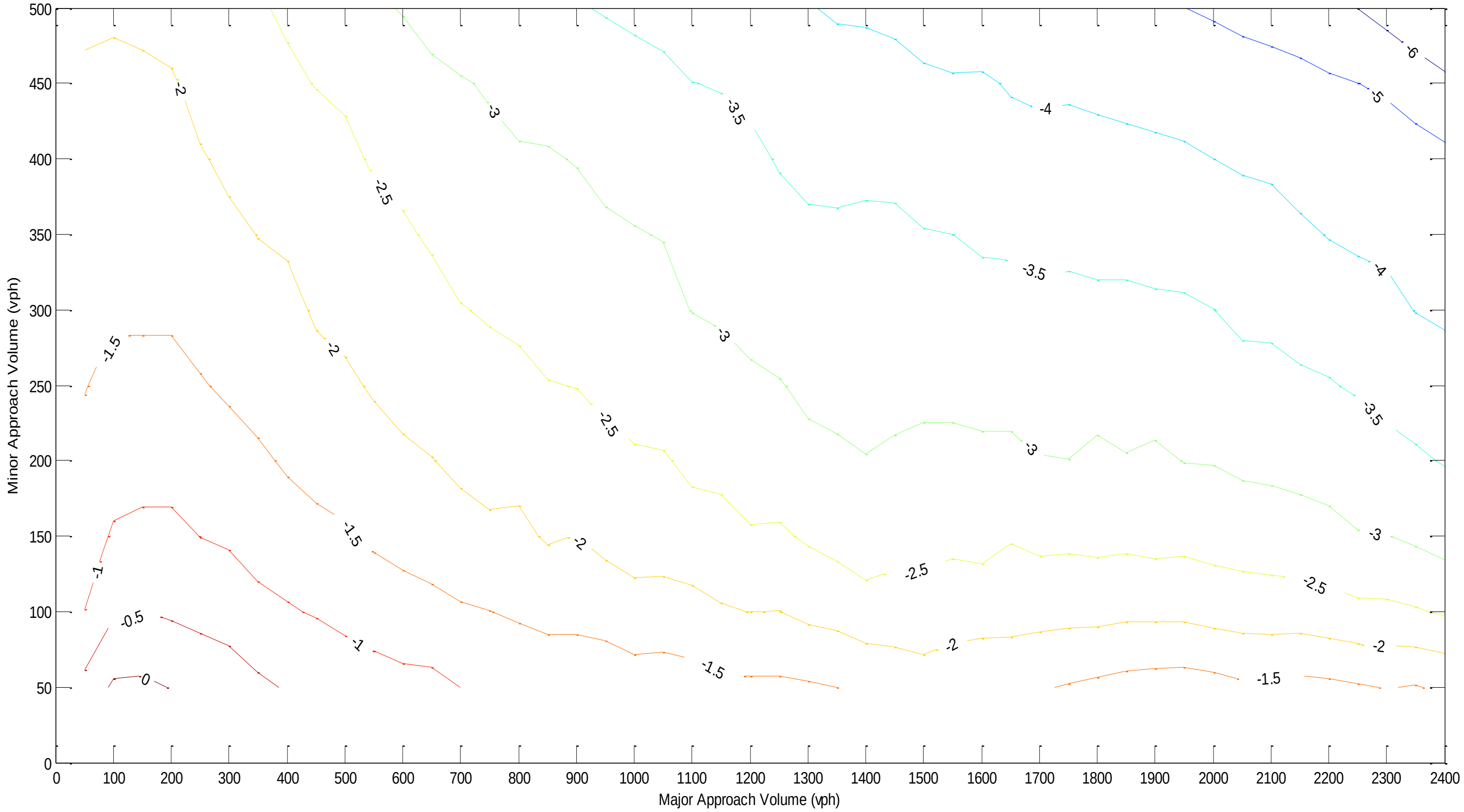


Figure B.107 Contour plot at BF=0.6, LTP=15 %, TP=10 %

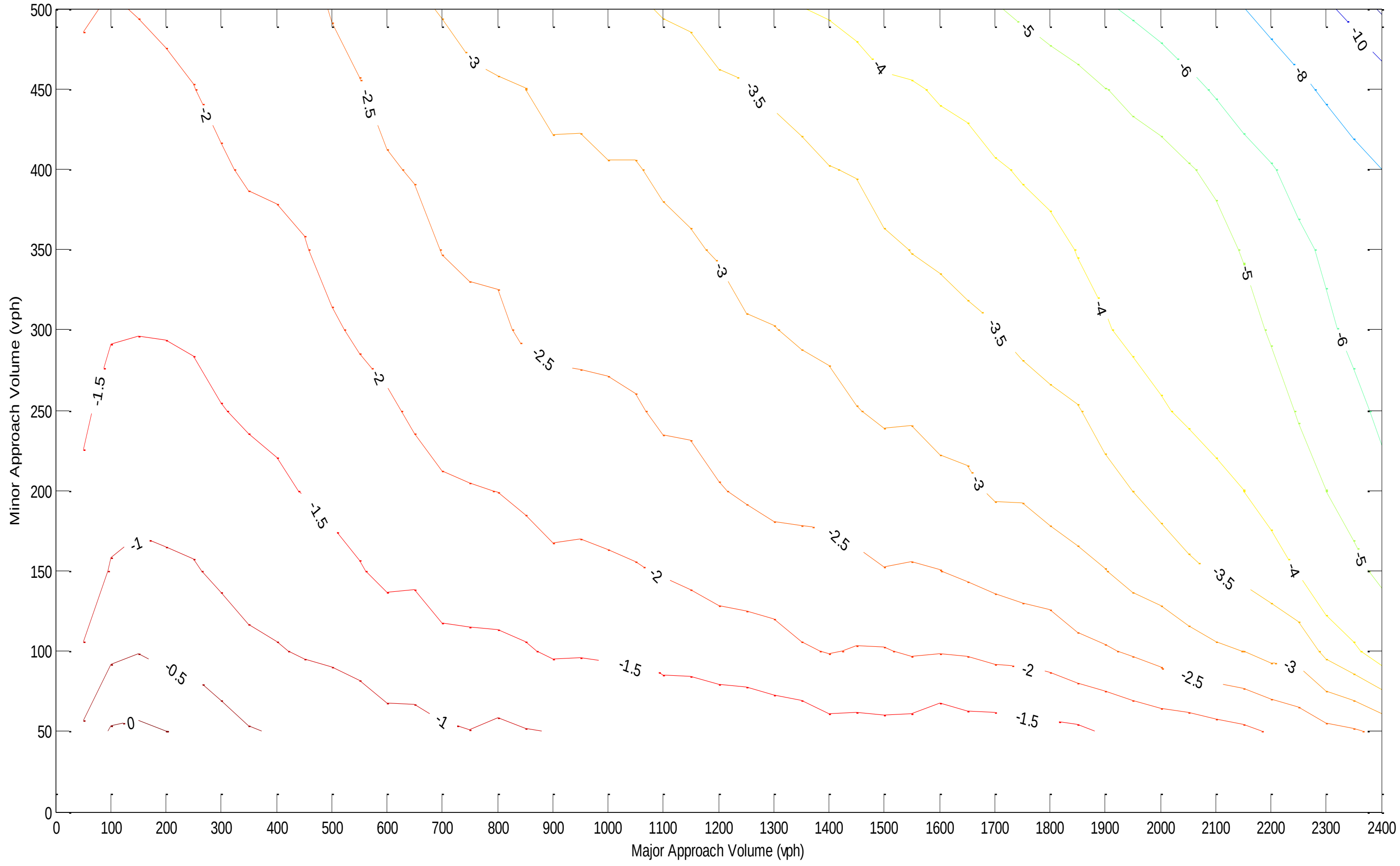


Figure B.108 Contour plot at BF=0.7, LTP=15 %, TP=10 %

B.3. Contour Plots Jughandle

B.3.1 Contour Plots for Critical Left Turn Delay Difference with Standatd Signalized Intersection

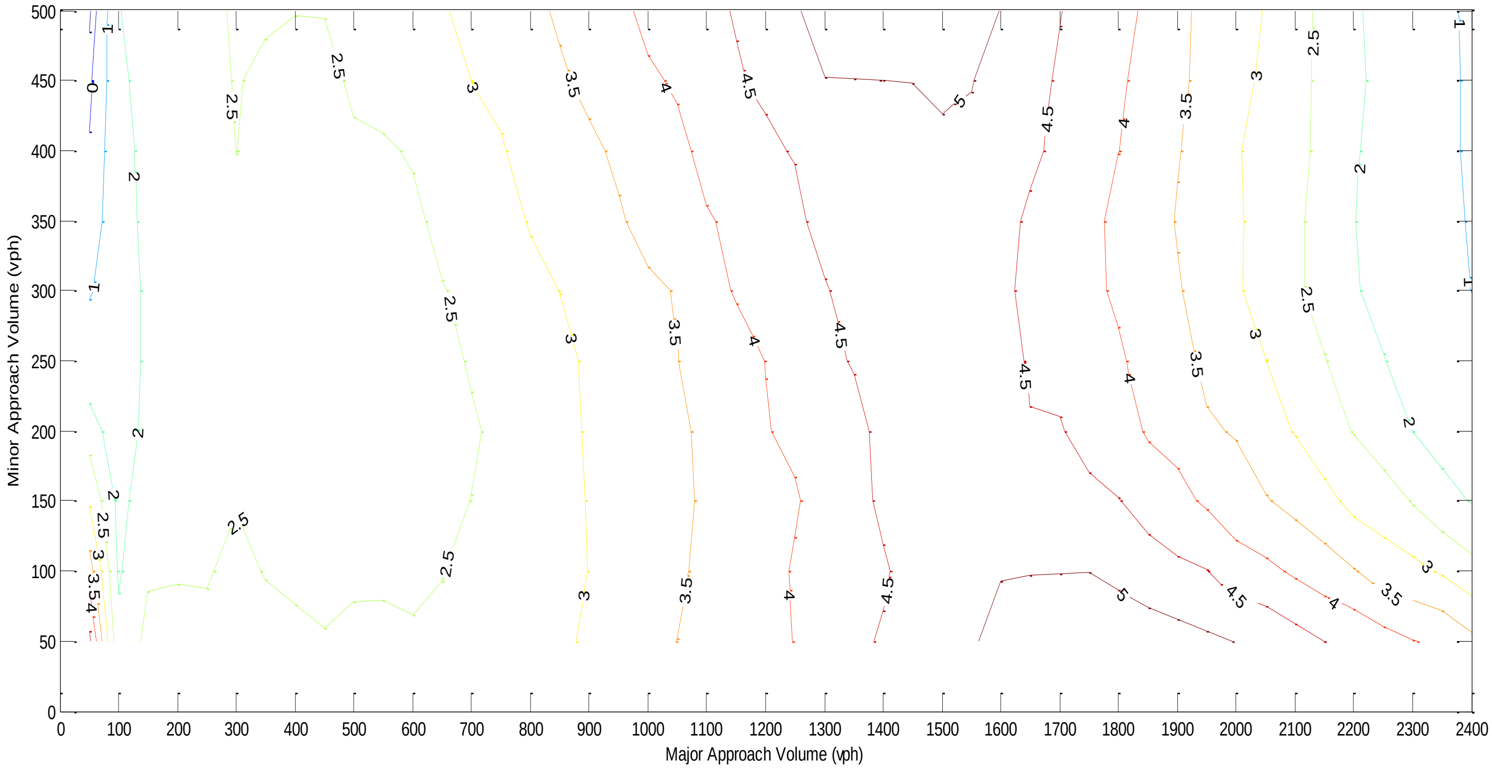


Figure B.109 Contour plot at BF=0.5, LTP=5%, TP=2%

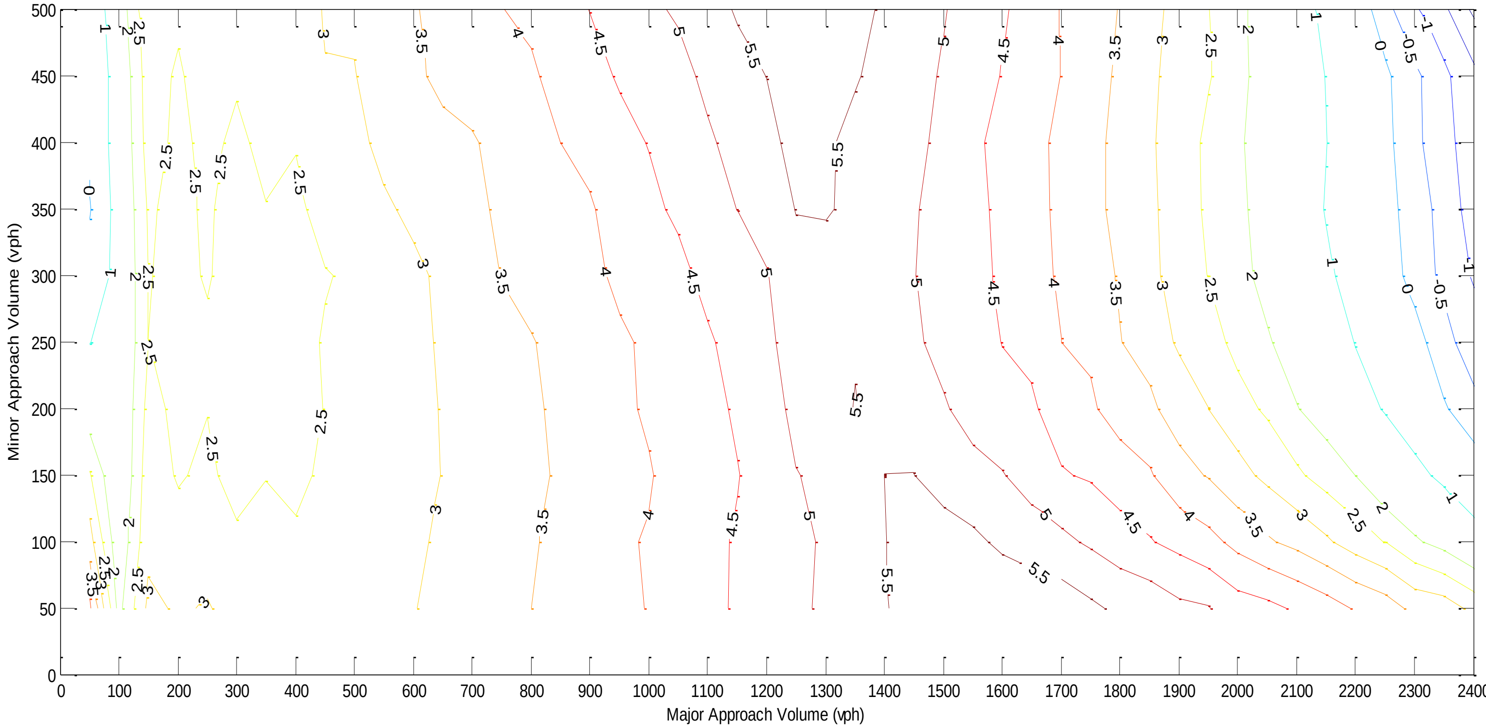


Figure B.110 Contour plot at BF=0.6, LTP=5%, TP=2%

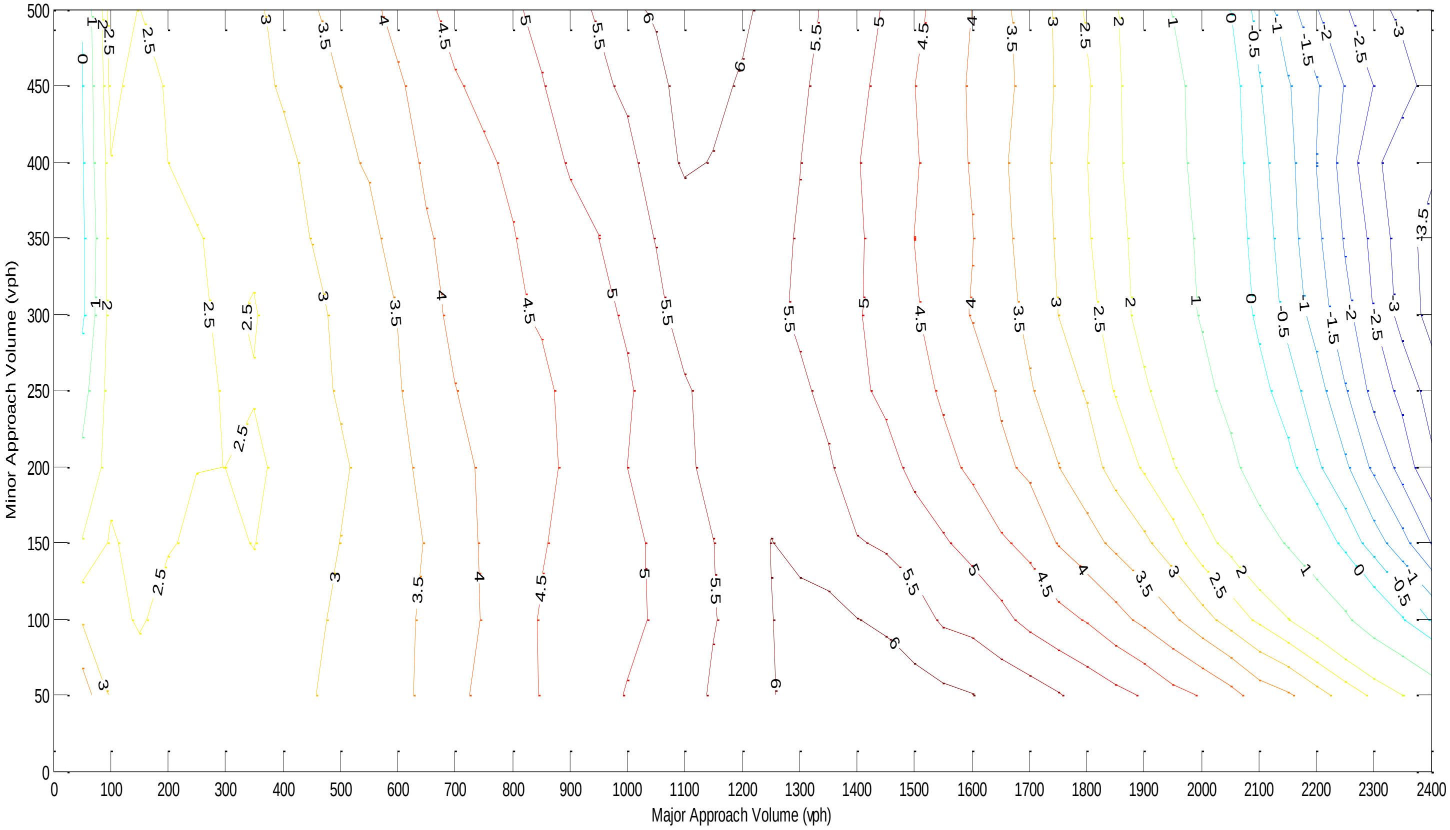


Figure B.111 Contour plot at BF=0.7, LTP=5%, TP=2%

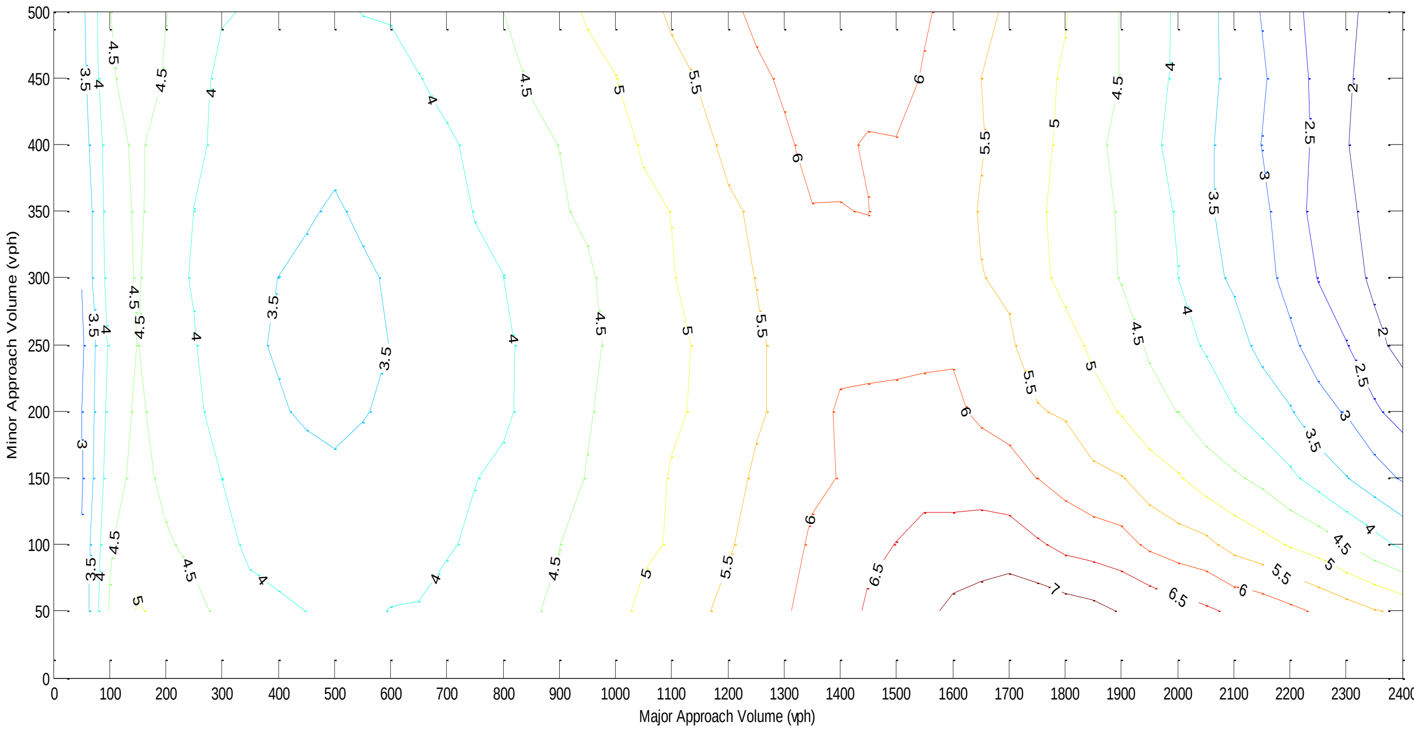


Figure B.112 Contour plot at BF=0.5, LTP=10%, TP=2%

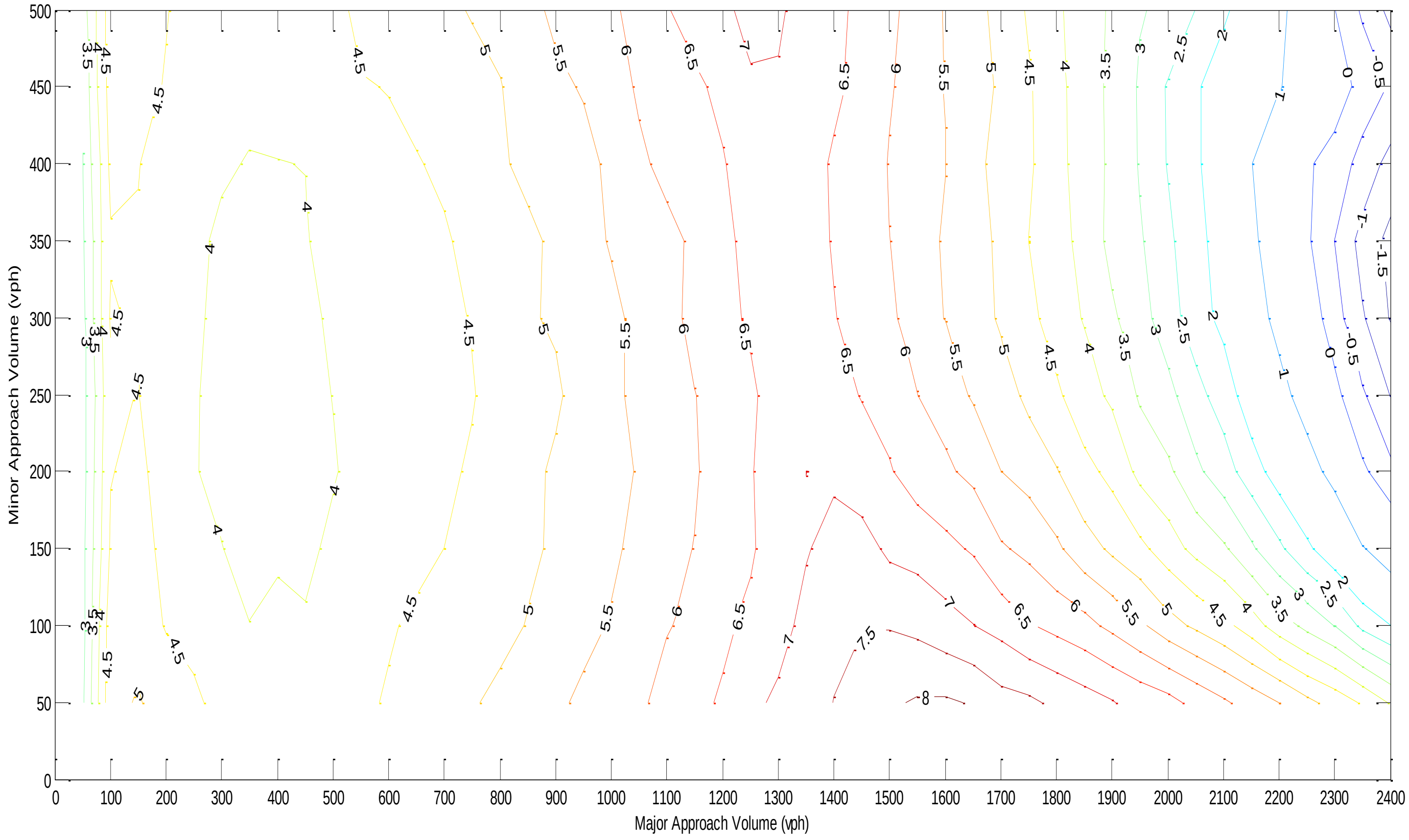


Figure B.113 Contour plot at BF=0.6, LTP=10%, TP=2%

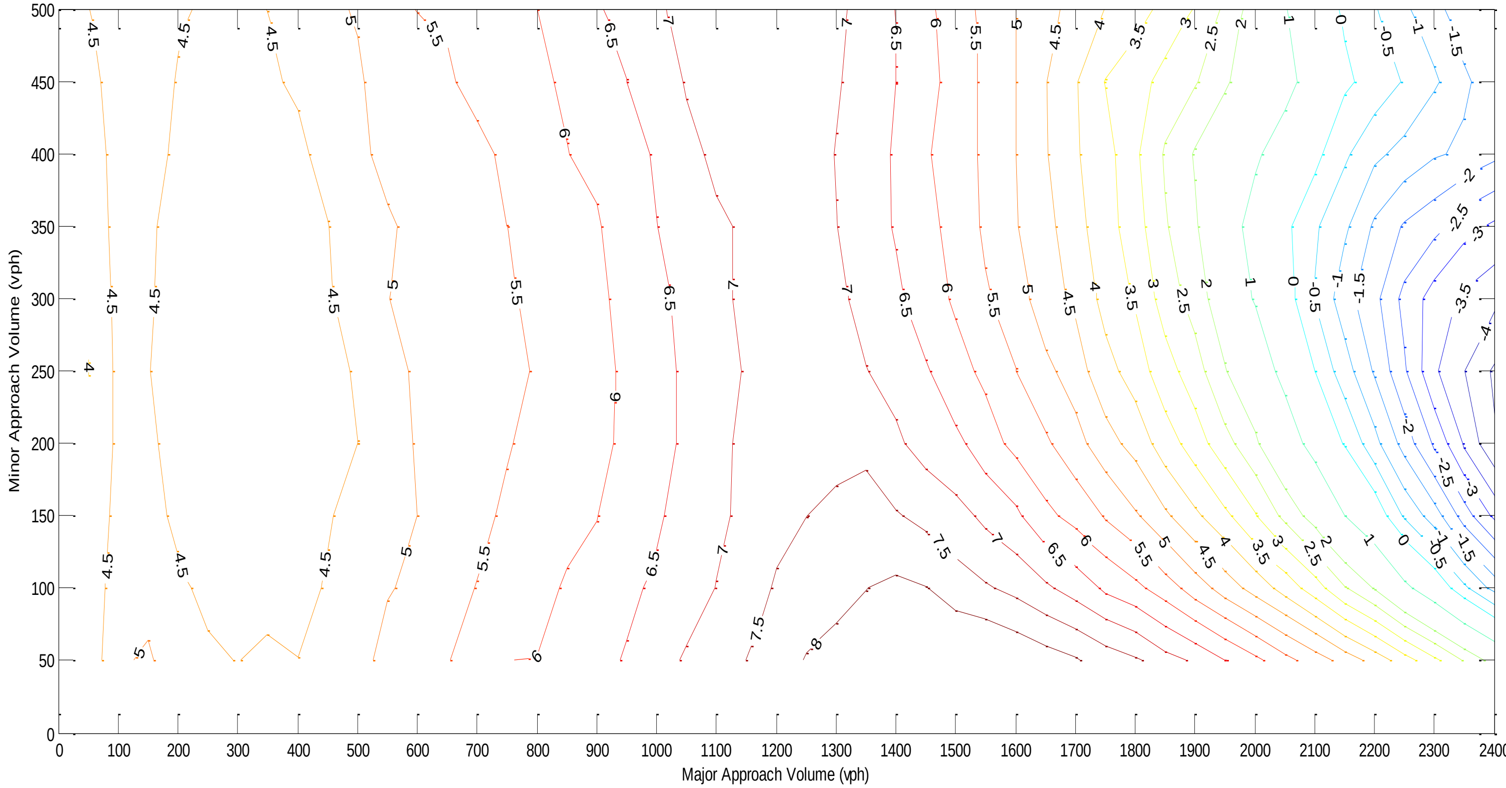


Figure B.114 Contour plot at BF=0.7, LTP=10%, TP=2%

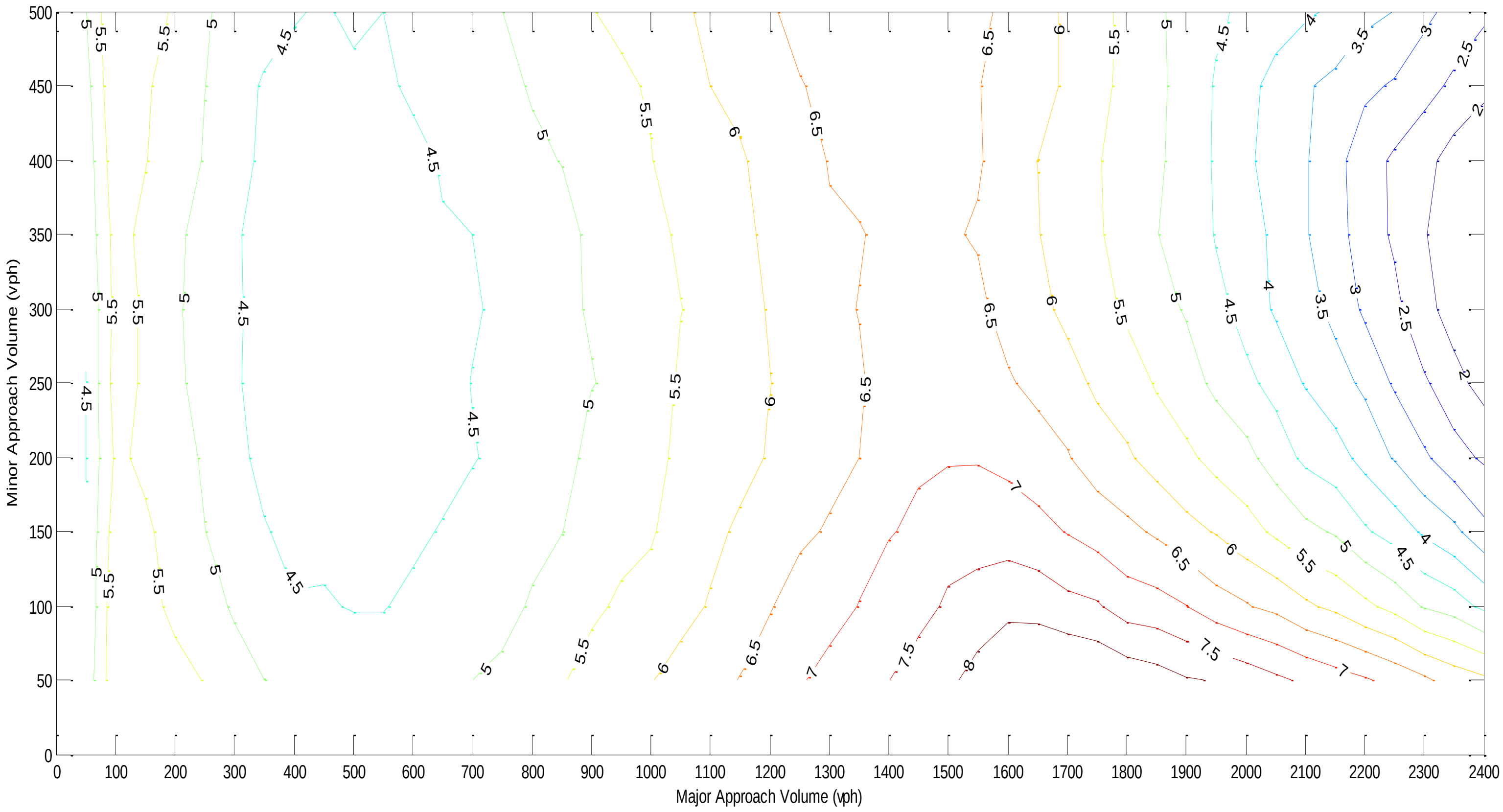


Figure B.115 Contour plot at BF=0.5, LTP=15%, TP=2%

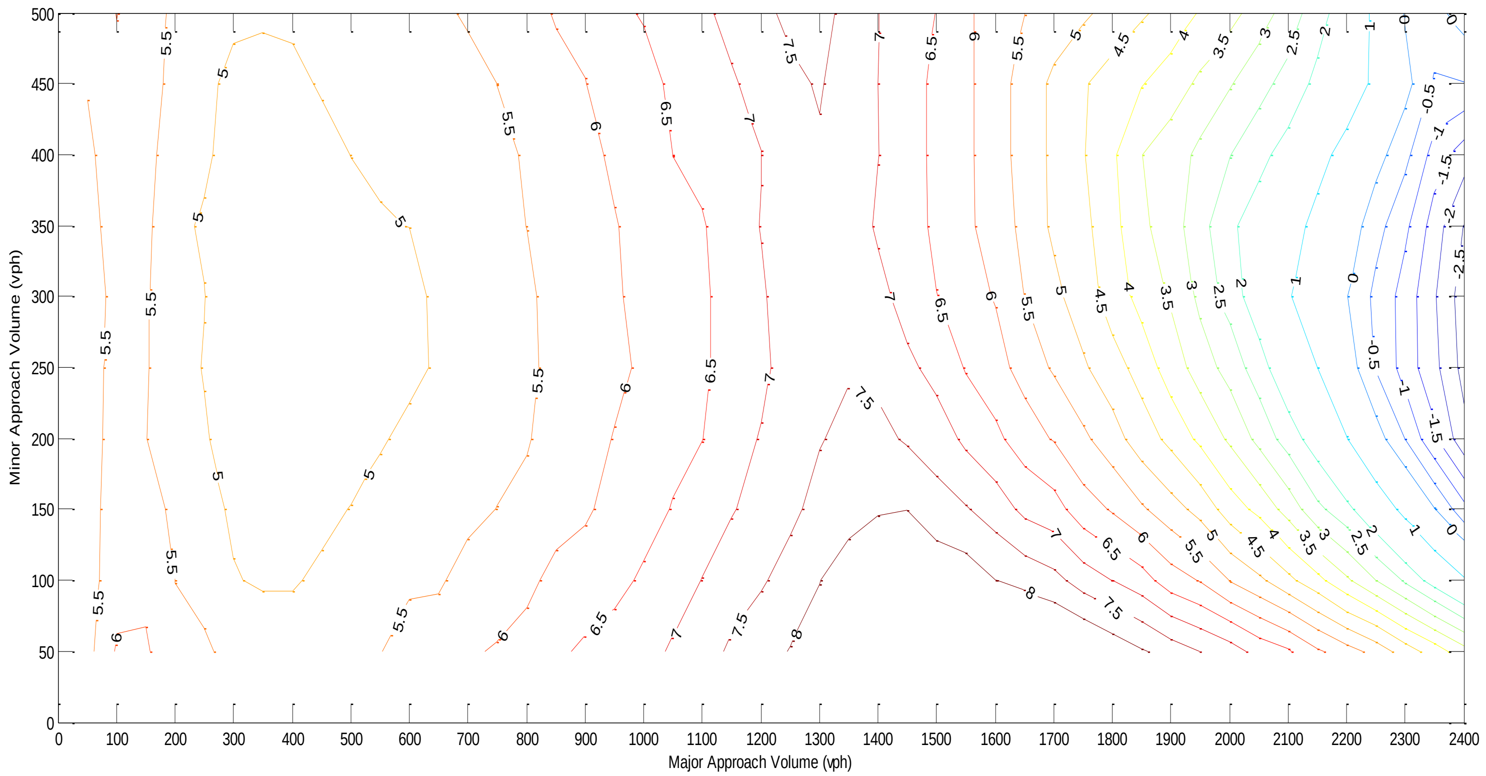


Figure B.116 Contour plot at BF=0.6, LTP=15%, TP=2%

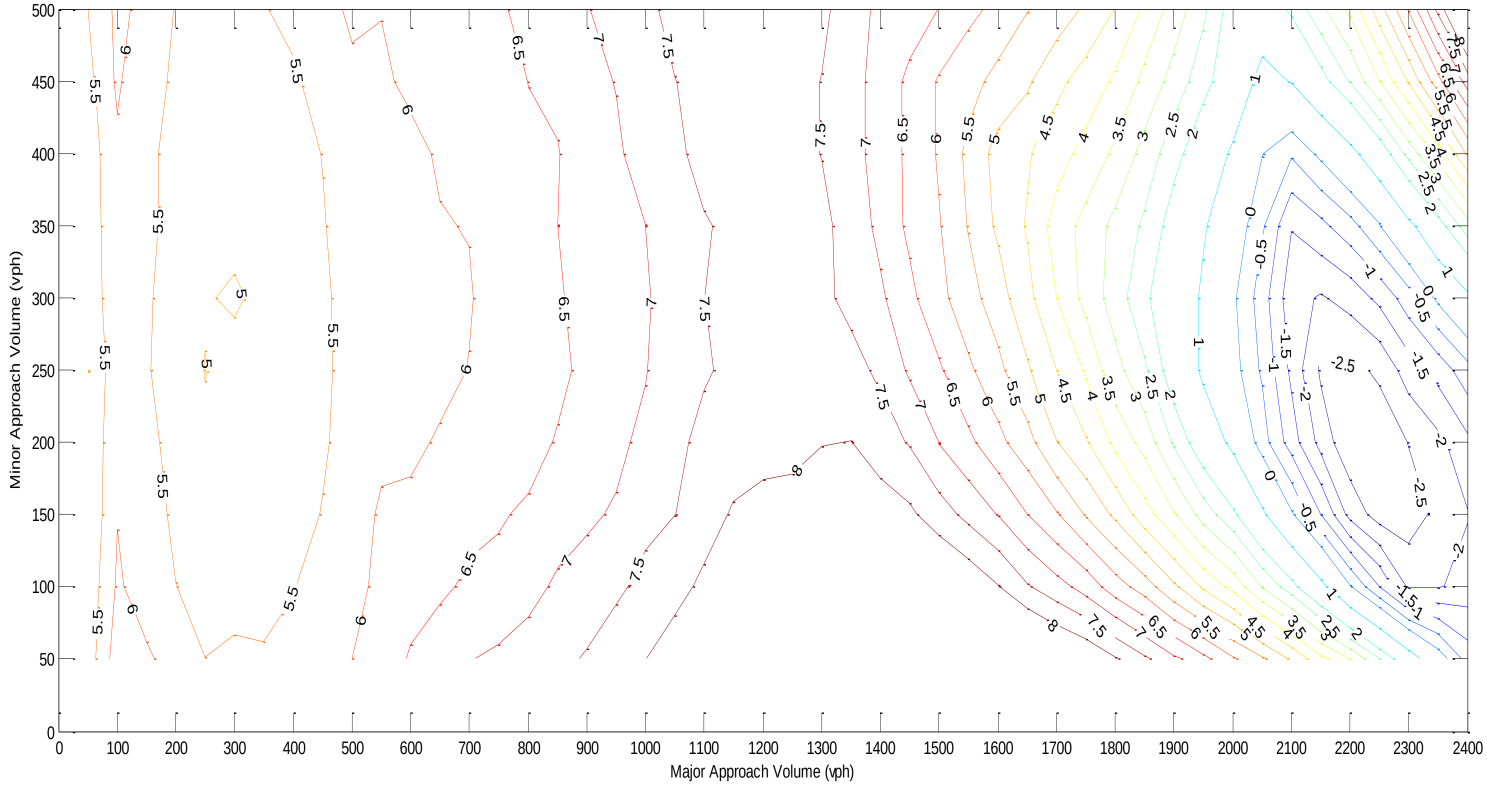


Figure B.117 Contour plot at BF=0.7, LTP=15%, TP=2%

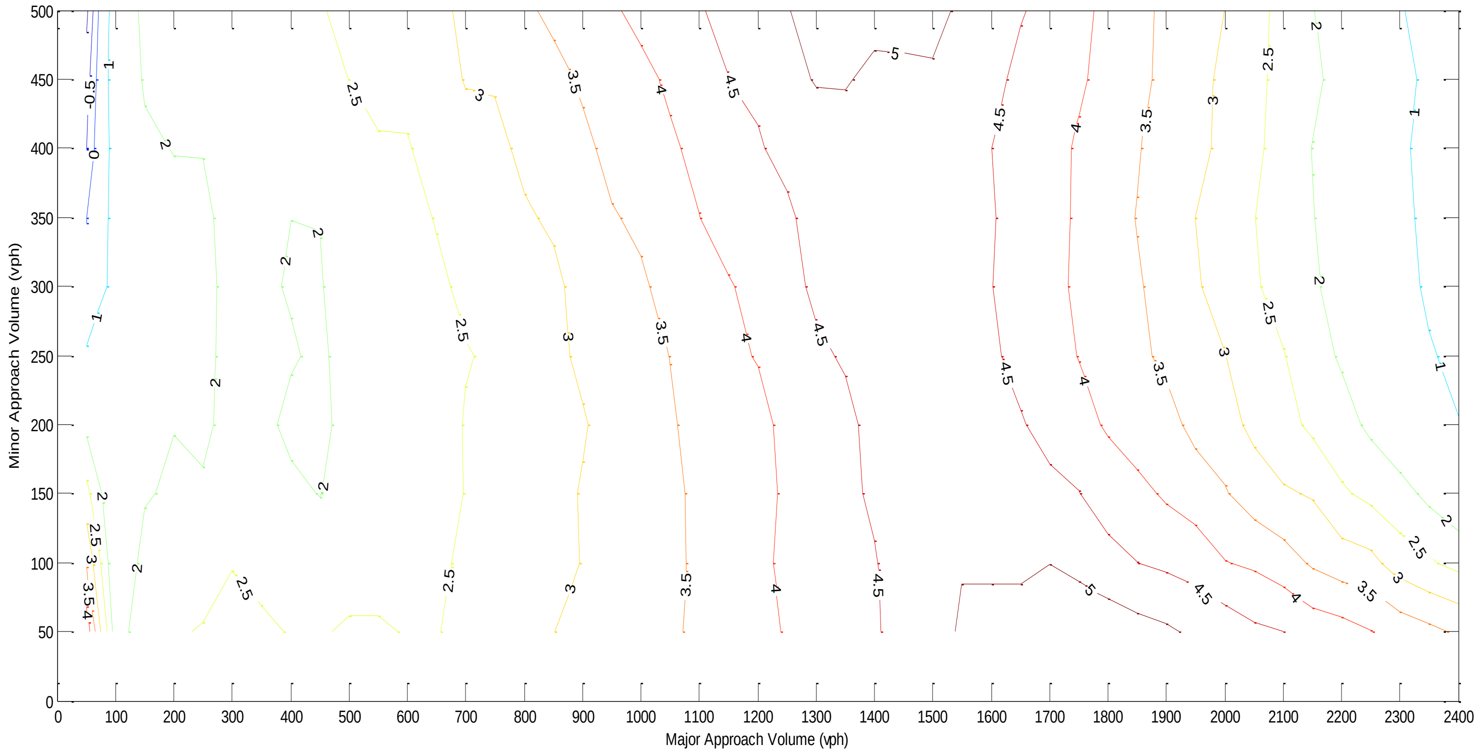


Figure B.118 Contour plot at BF=0.5, LTP=5 %, TP=5 %

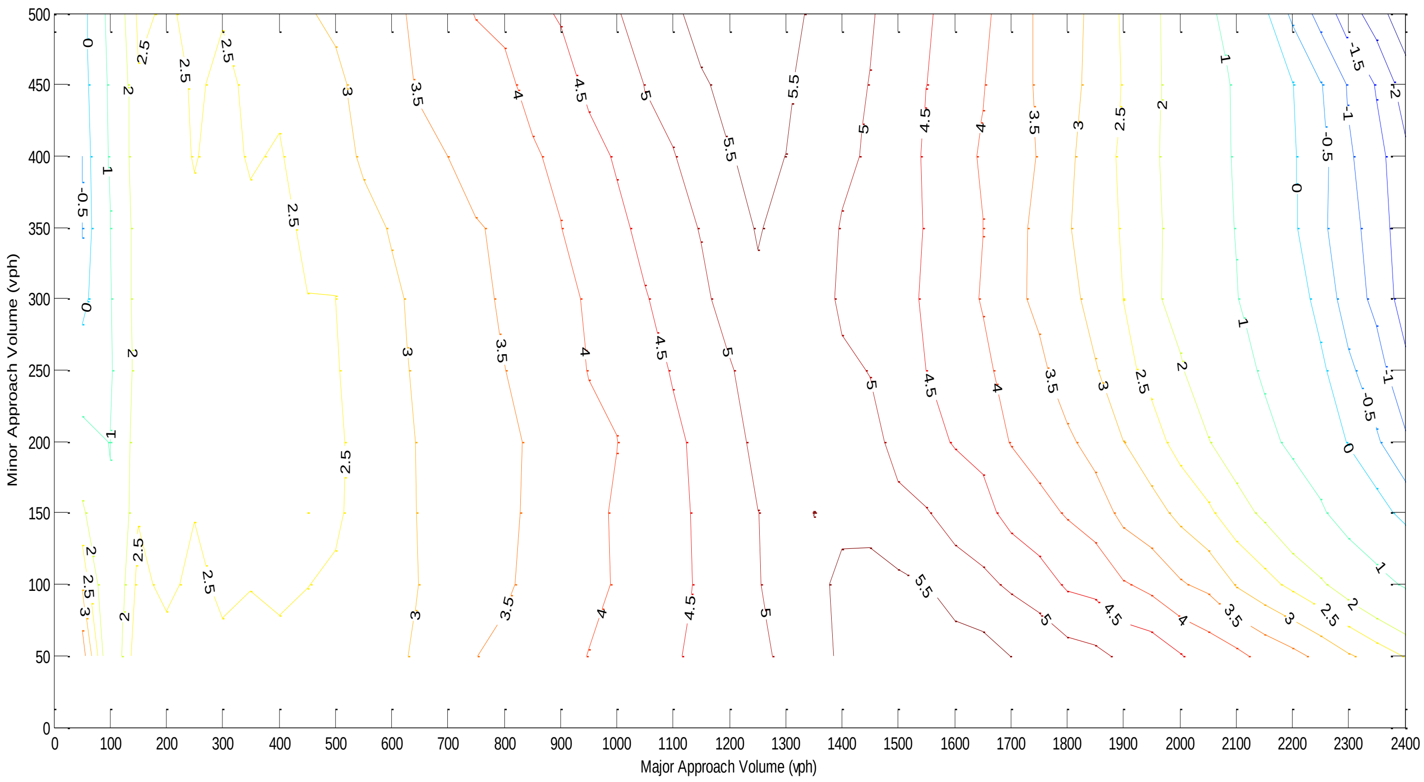


Figure B.119 Contour plot at BF=0.6, LTP=5 %, TP=5 %

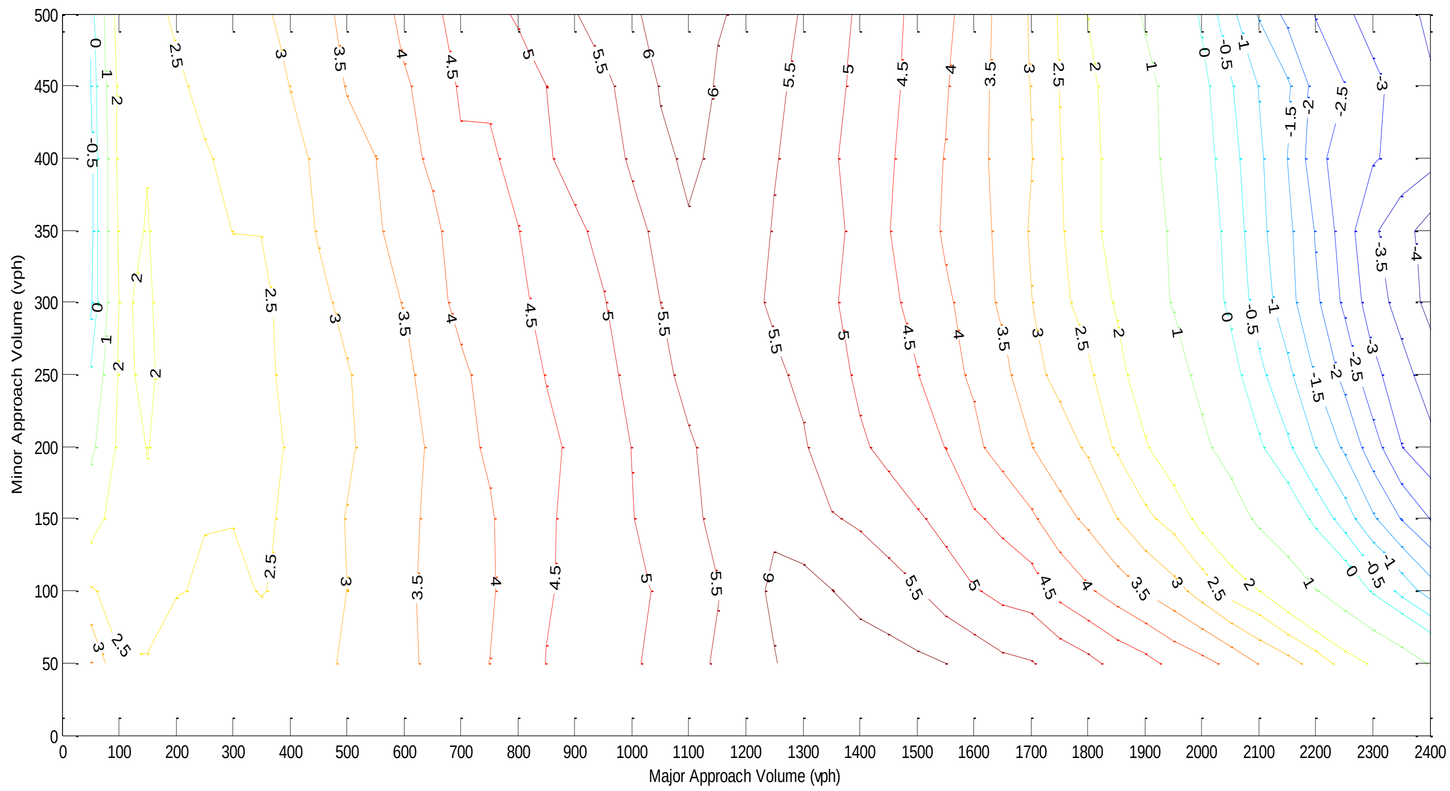


Figure B.120 Contour plot at BF=0.7, LTP=5 %, TP=5 %

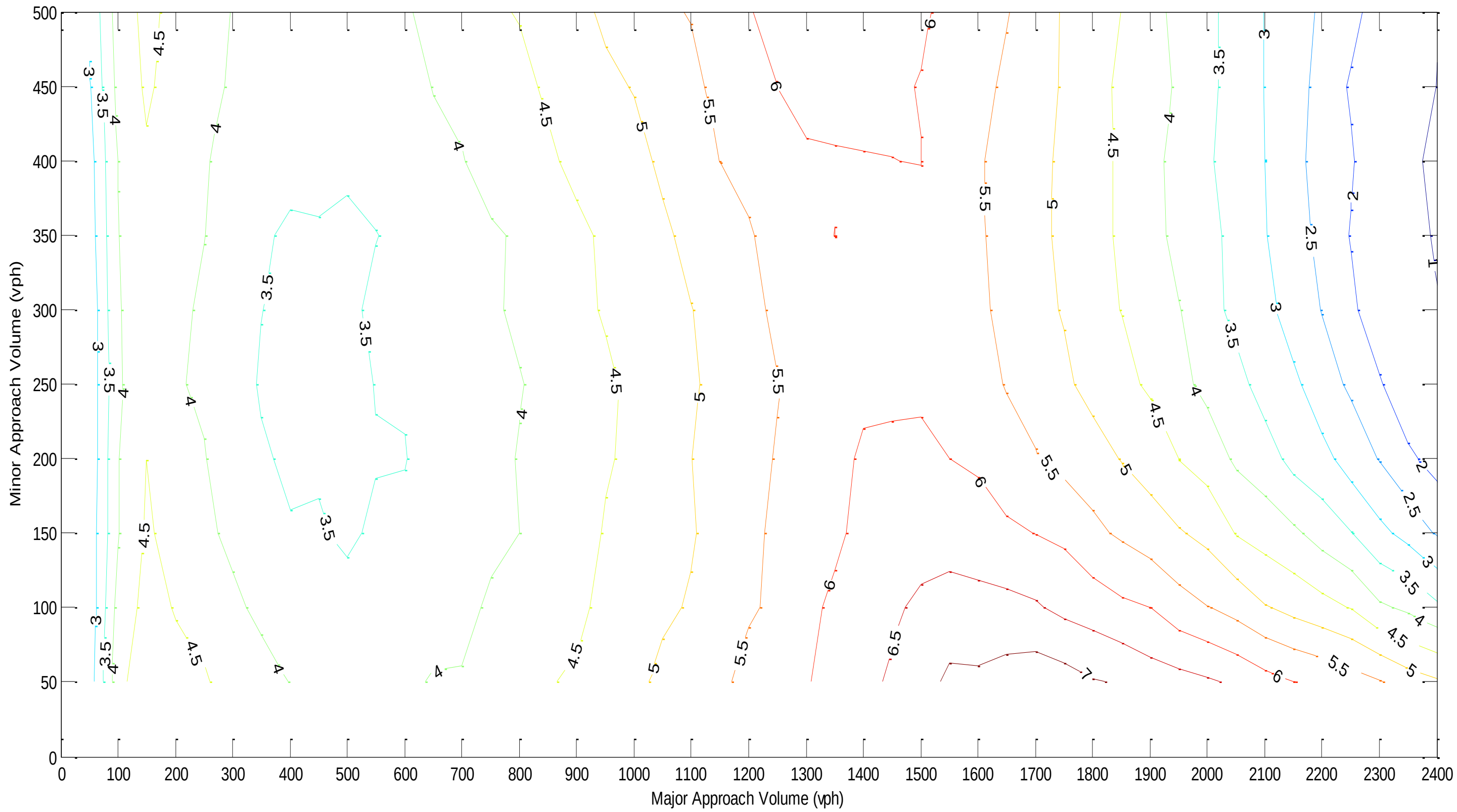


Figure B.121 Contour plot at BF=0.5, LTP=10 %, TP=5 %

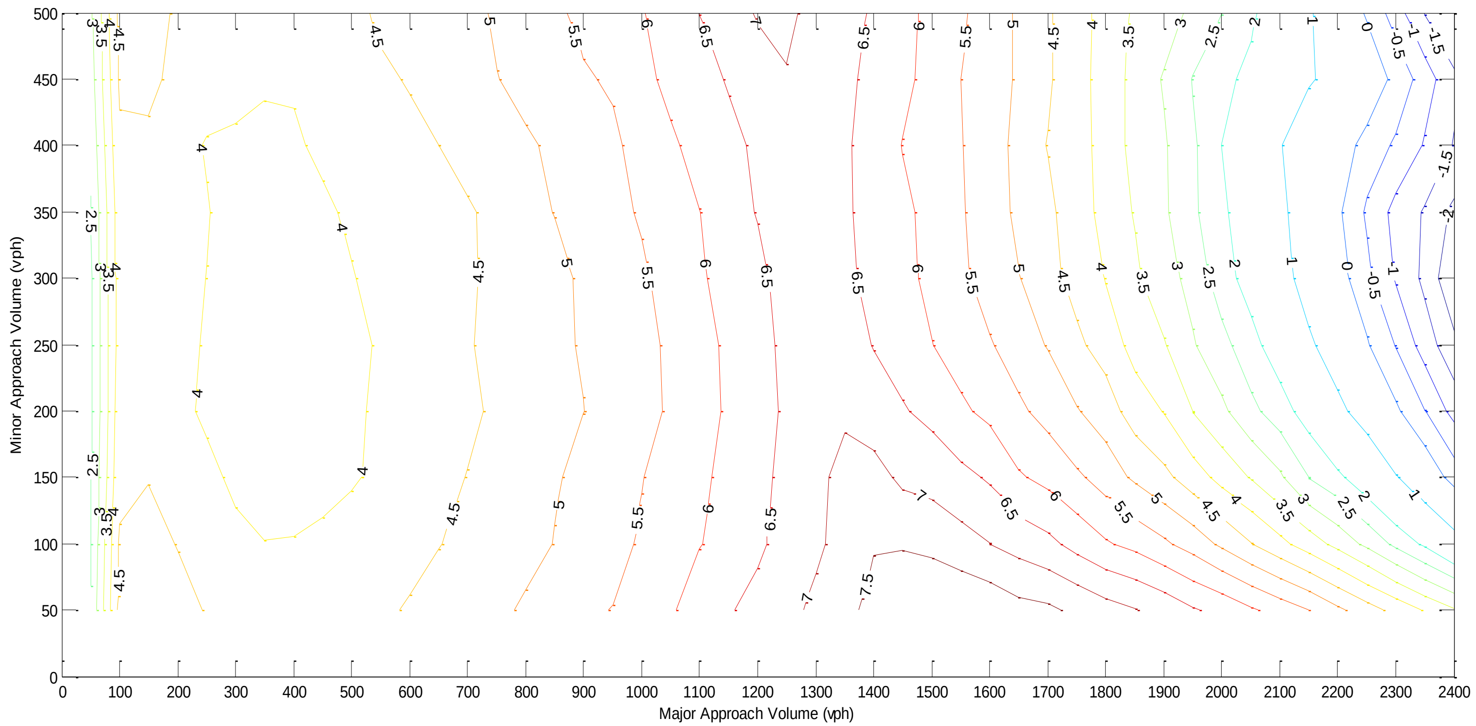


Figure B.122 Contour plot at BF=0.6, LTP=10 %, TP=5 %

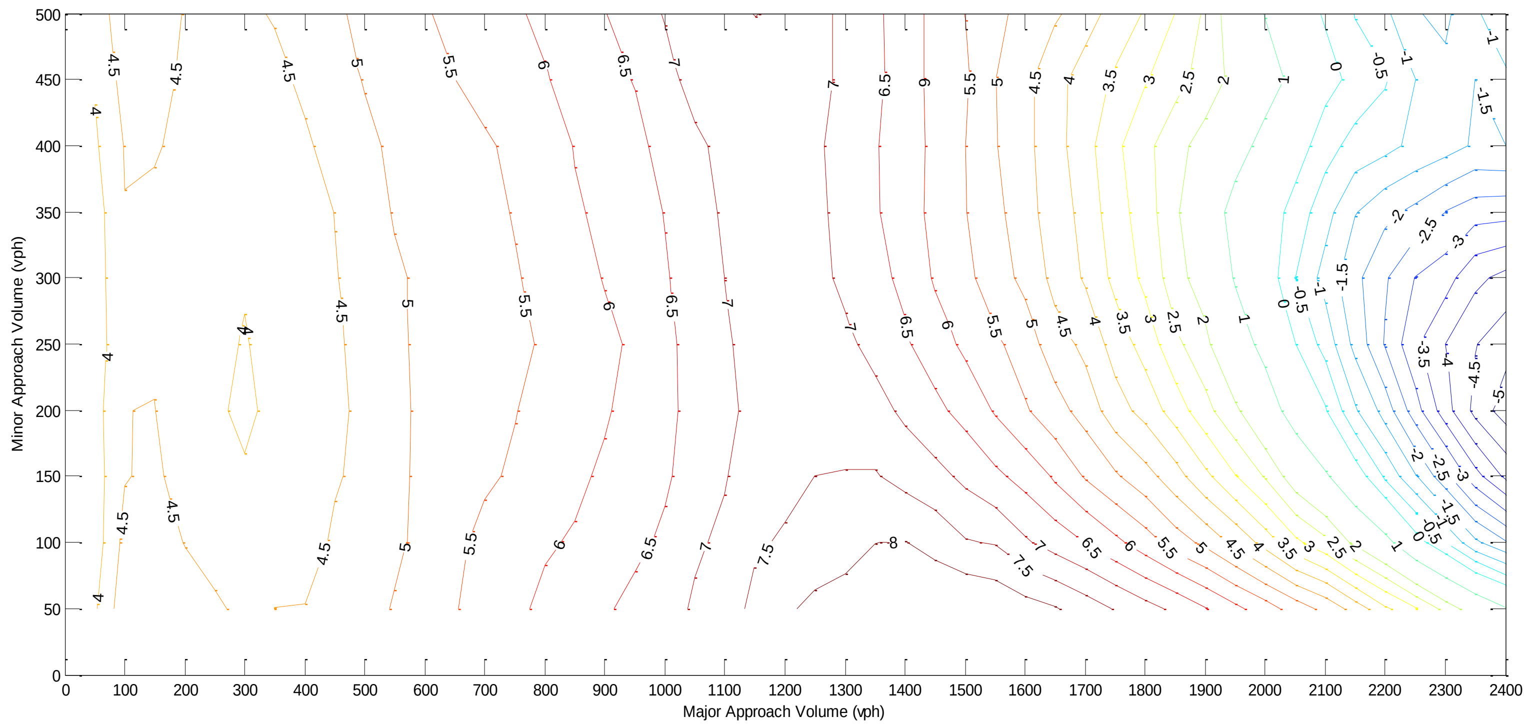


Figure B.123 Contour plot at BF=0.7, LTP=10 %, TP=5 %

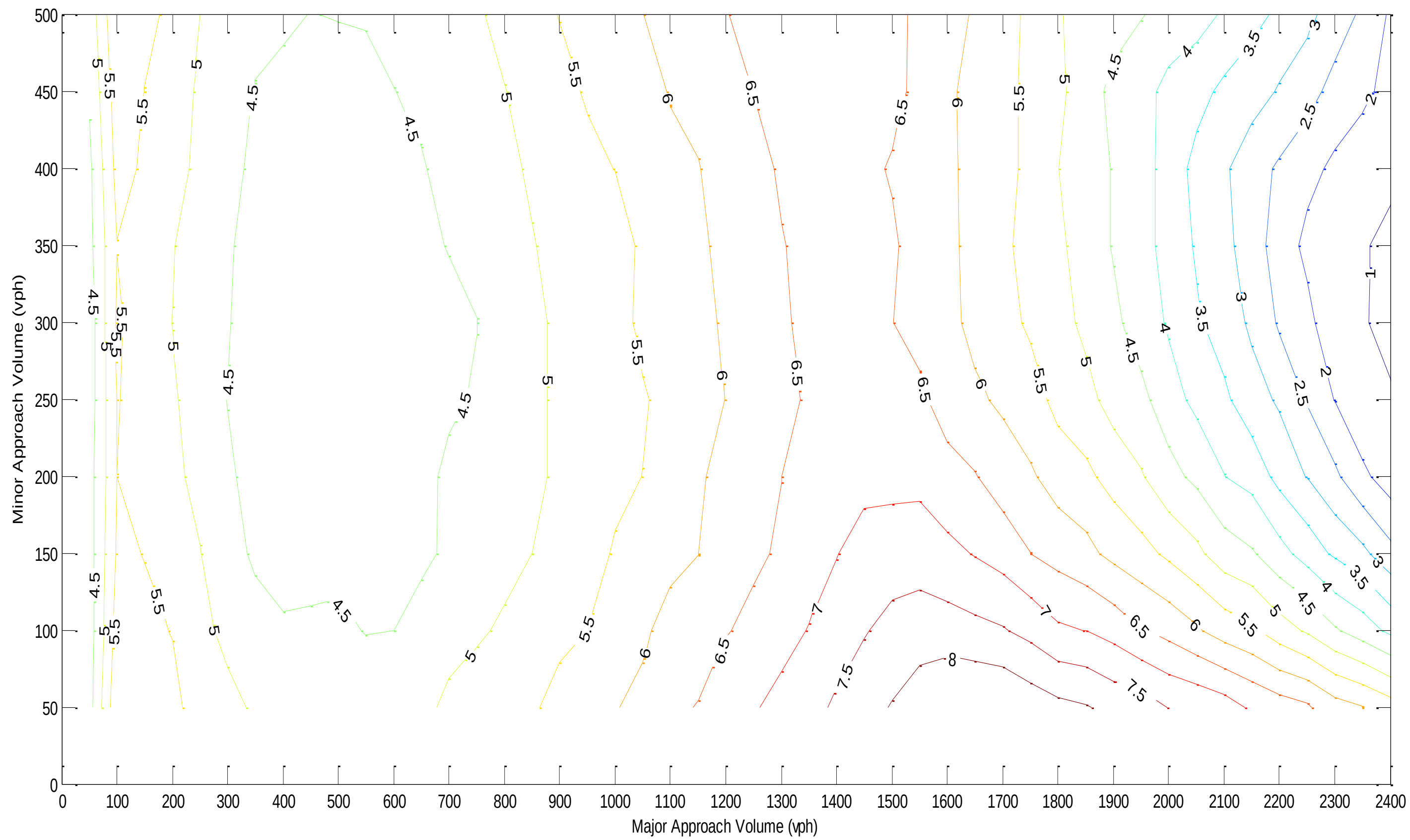


Figure B.124 Contour plot at BF=0.5, LTP=15 %, TP=5 %

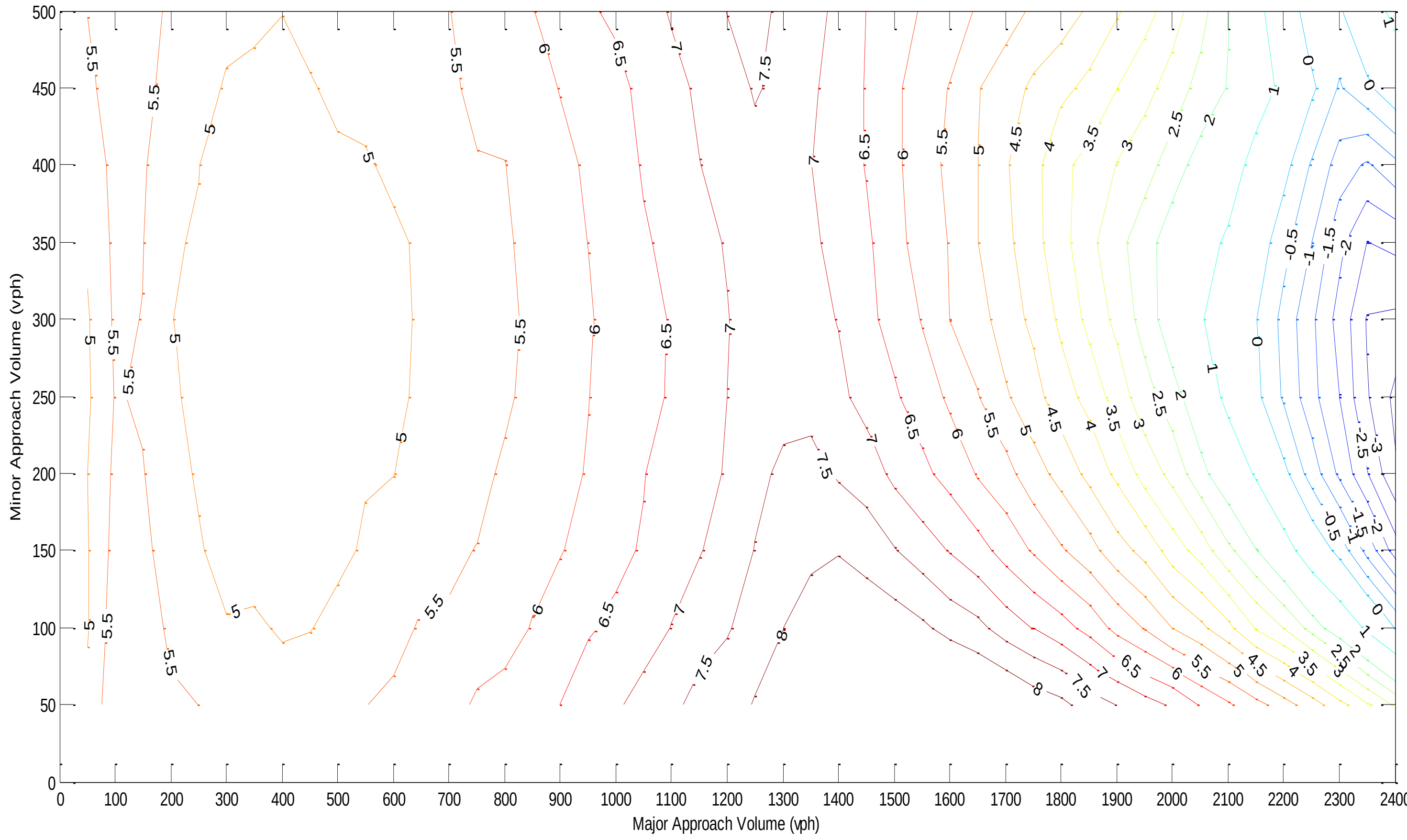


Figure B.125 Contour plot at BF=0.6, LTP=15 %, TP=5 %

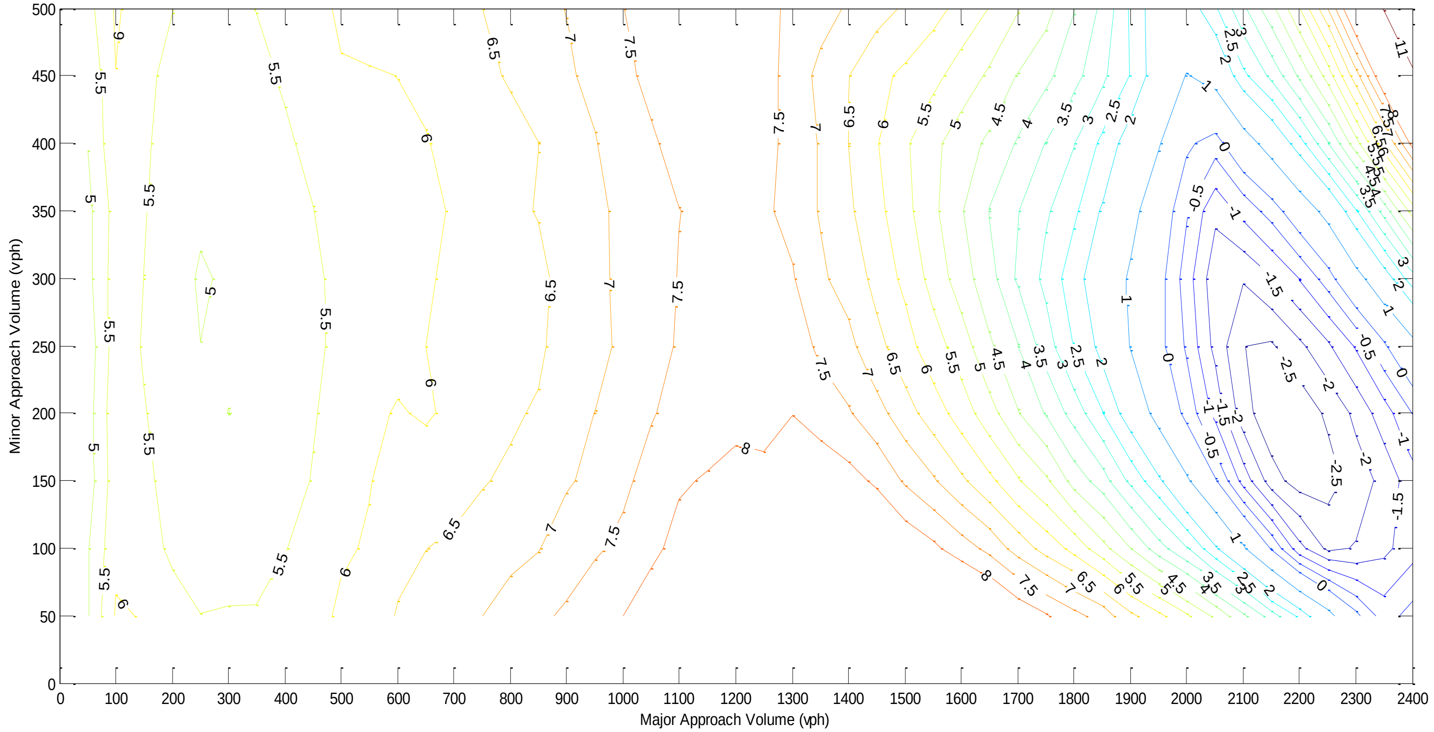


Figure B.126 Contour plot at BF=0.7, LTP=15 %, TP=5 %

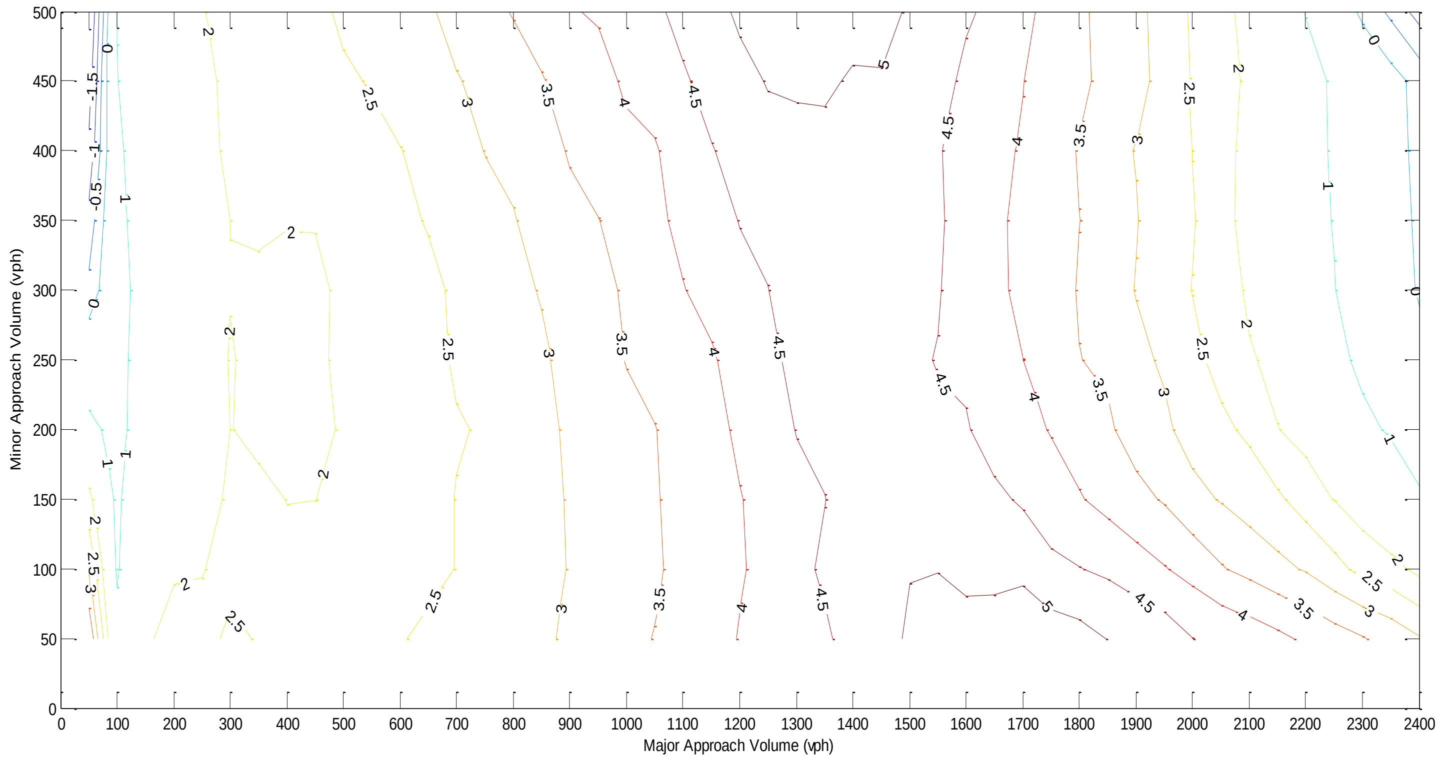


Figure B.127 Contour plot at BF=0.5, LTP=5 %, TP=10 %

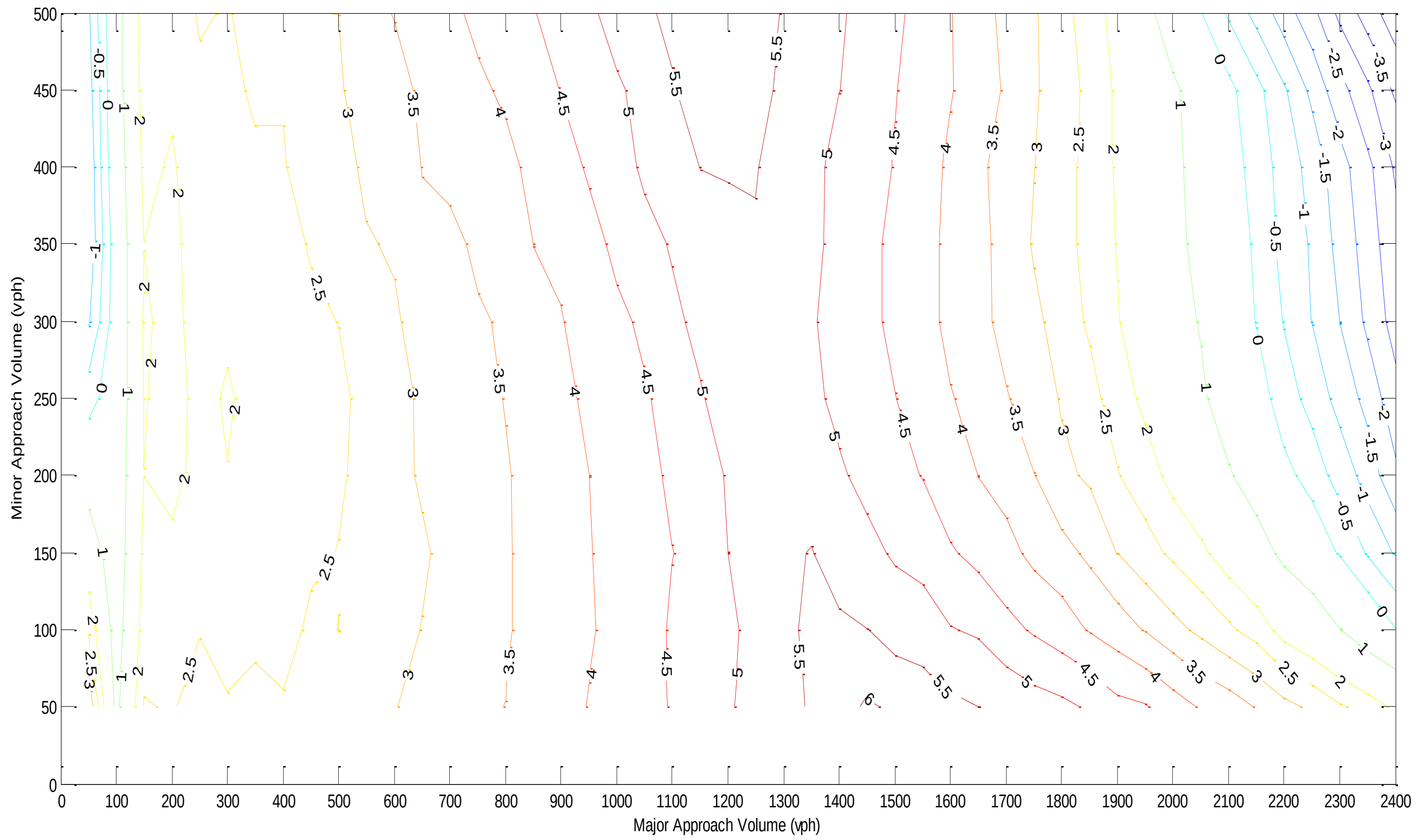


Figure B.128 Contour plot at BF=0.6, LTP=5 %, TP=10 %

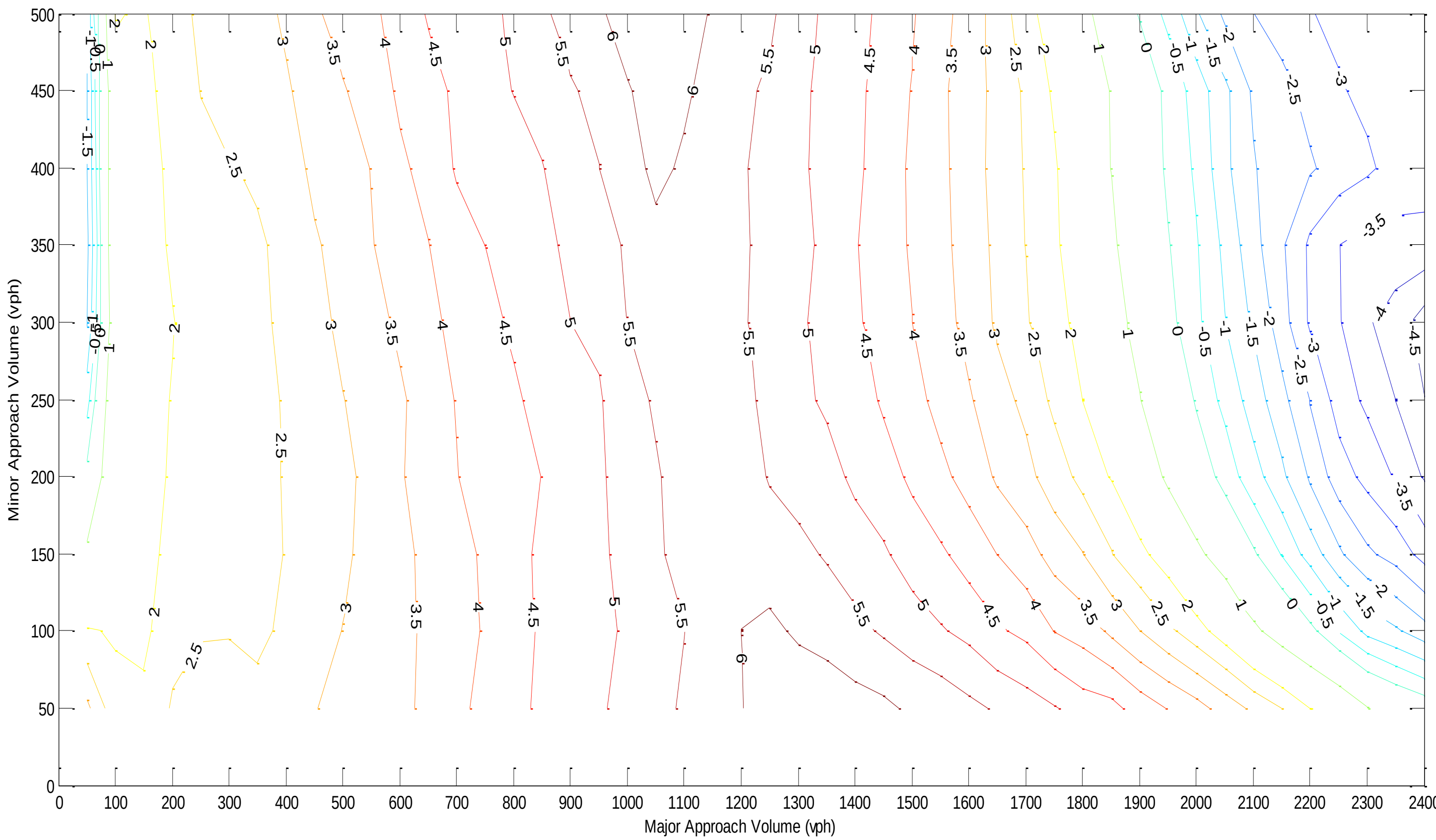


Figure B.129 Contour plot at BF=0.7, LTP=5 %, TP=10 %

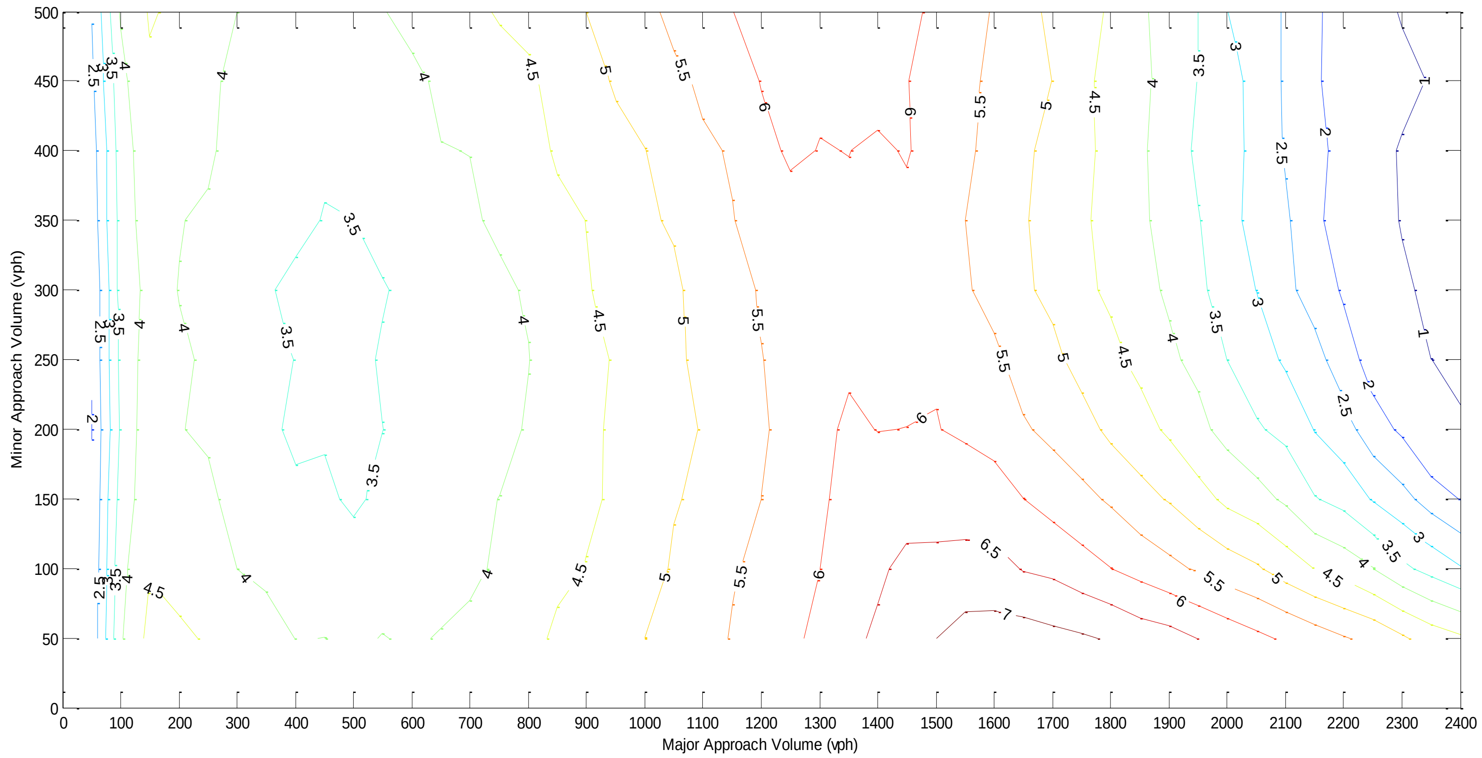


Figure B.130 Contour plot at BF=0.5, LTP=10%, TP=10 %

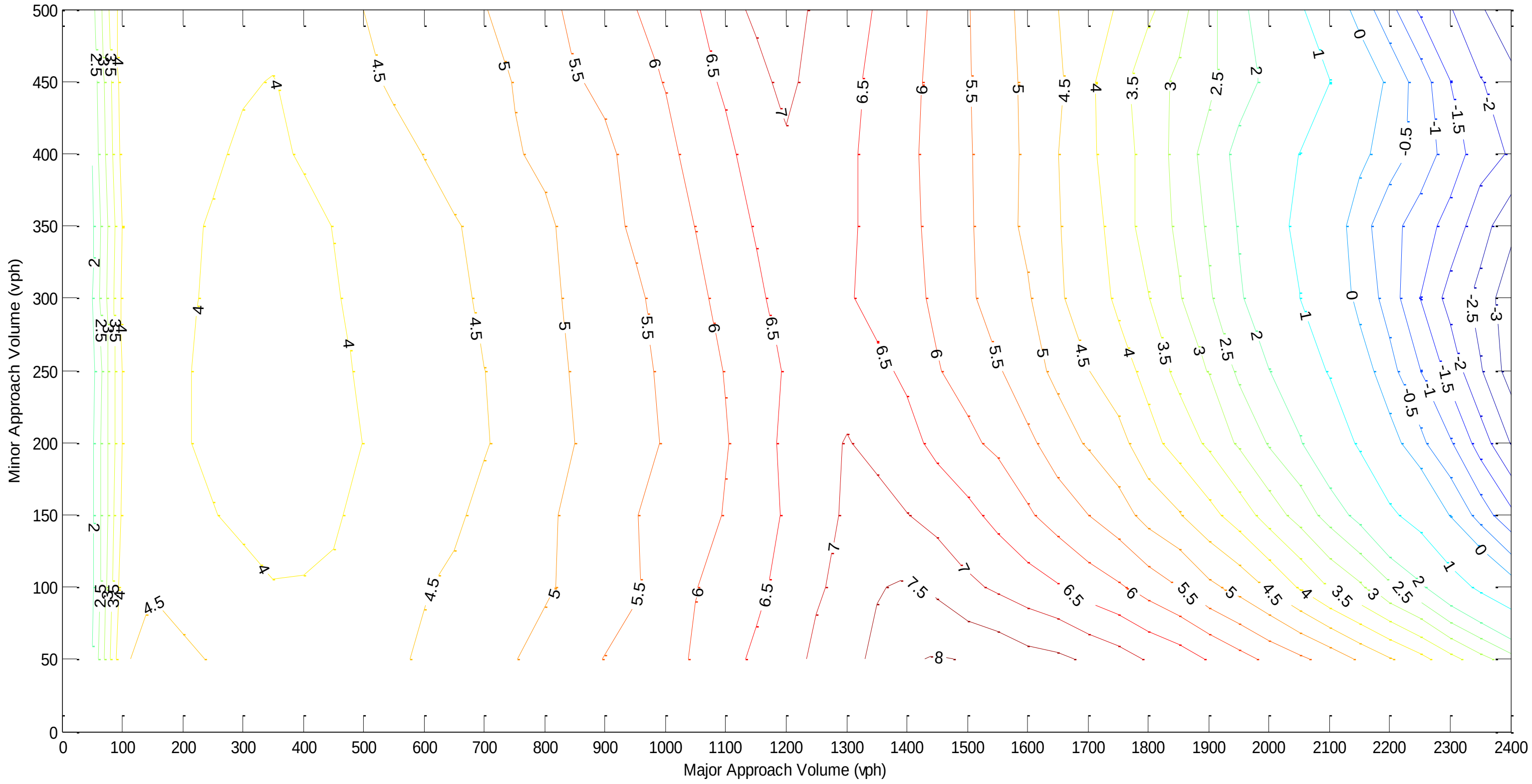


Figure B.131 Contour plot at BF=0.6, LTP=10 %, TP=10 %

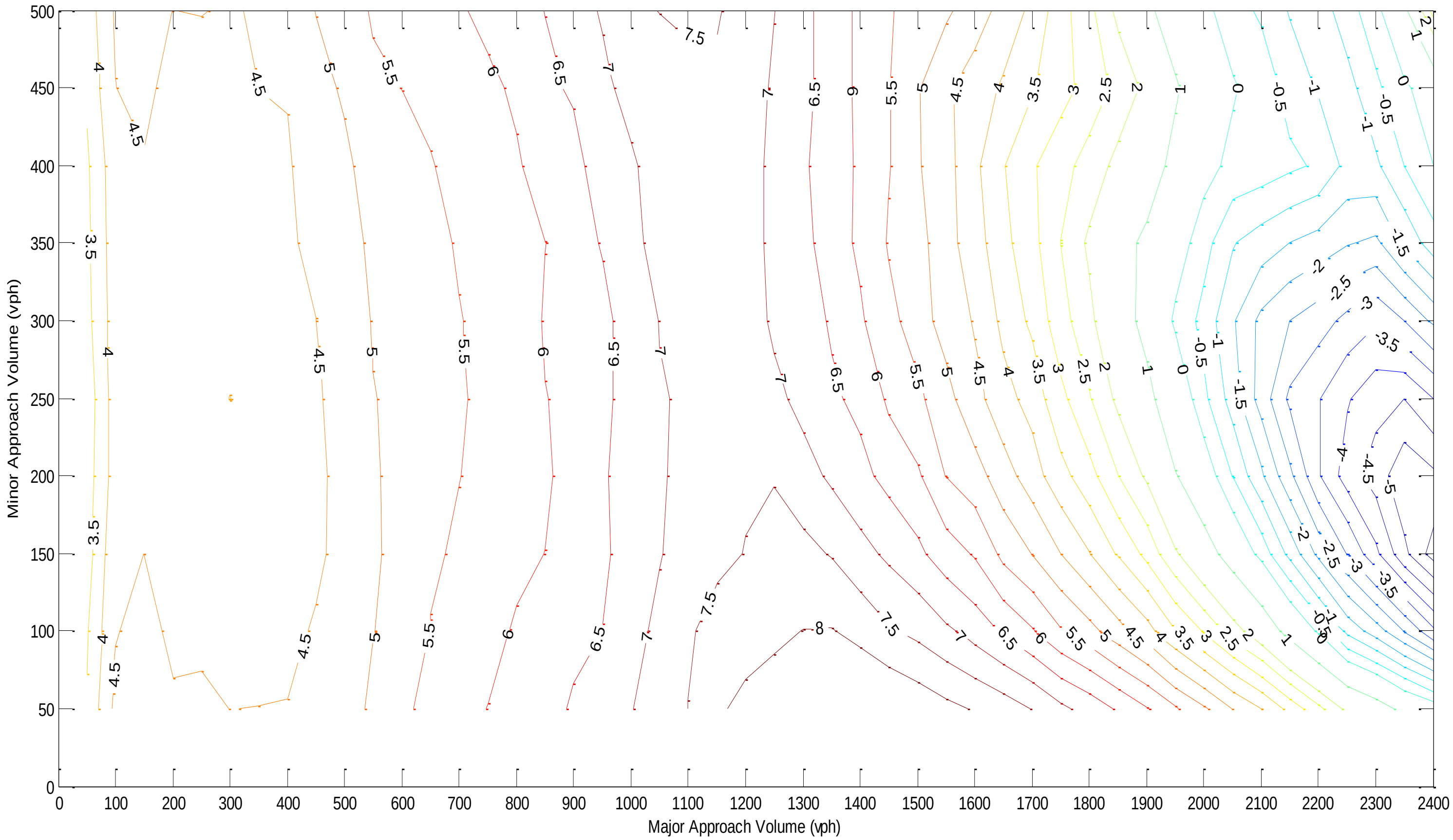


Figure B.132 Contour plot at BF=0.7, LTP=10 %, TP=10 %

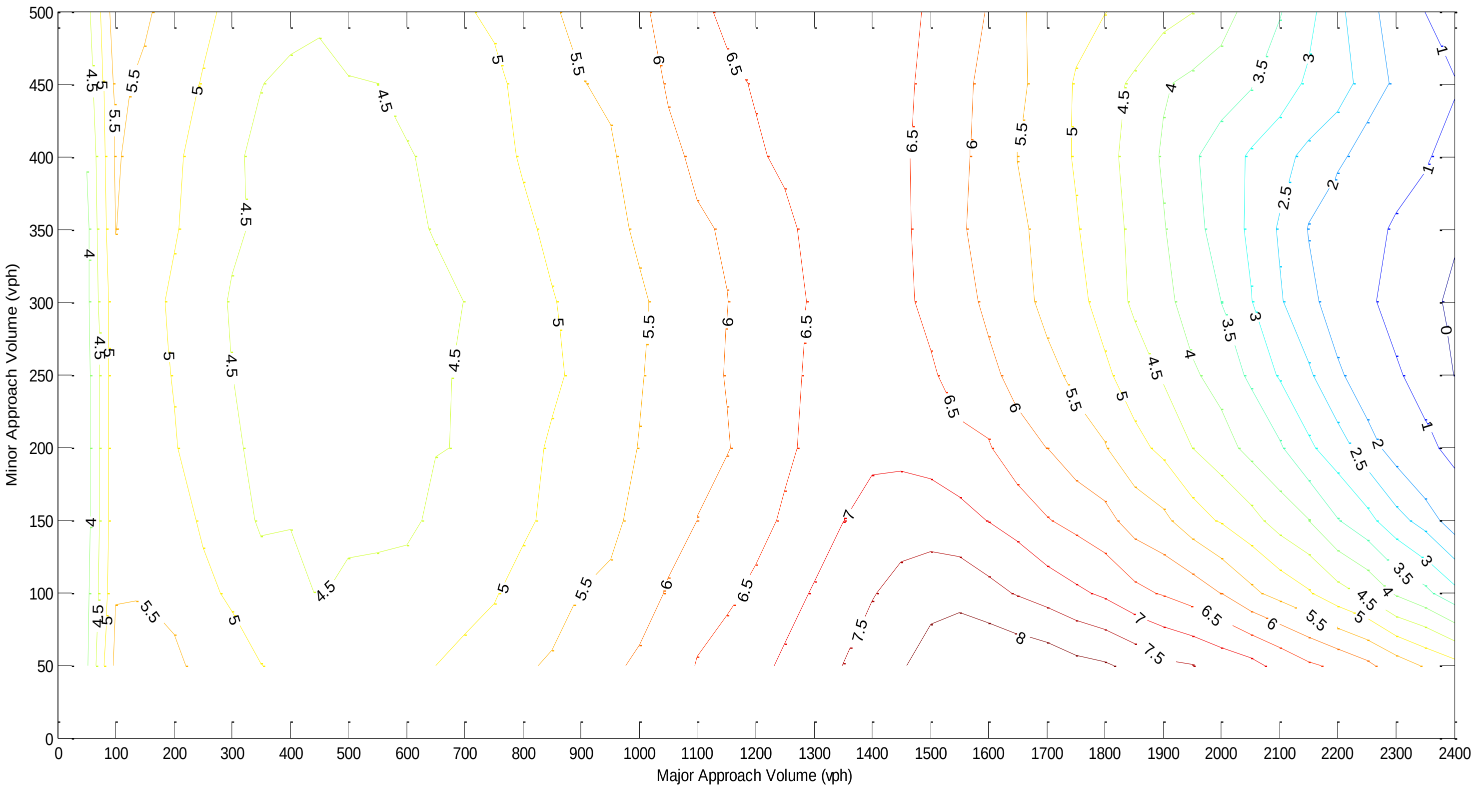


Figure B.133 Contour plot at BF=0.5, LTP=15 %, TP=10 %

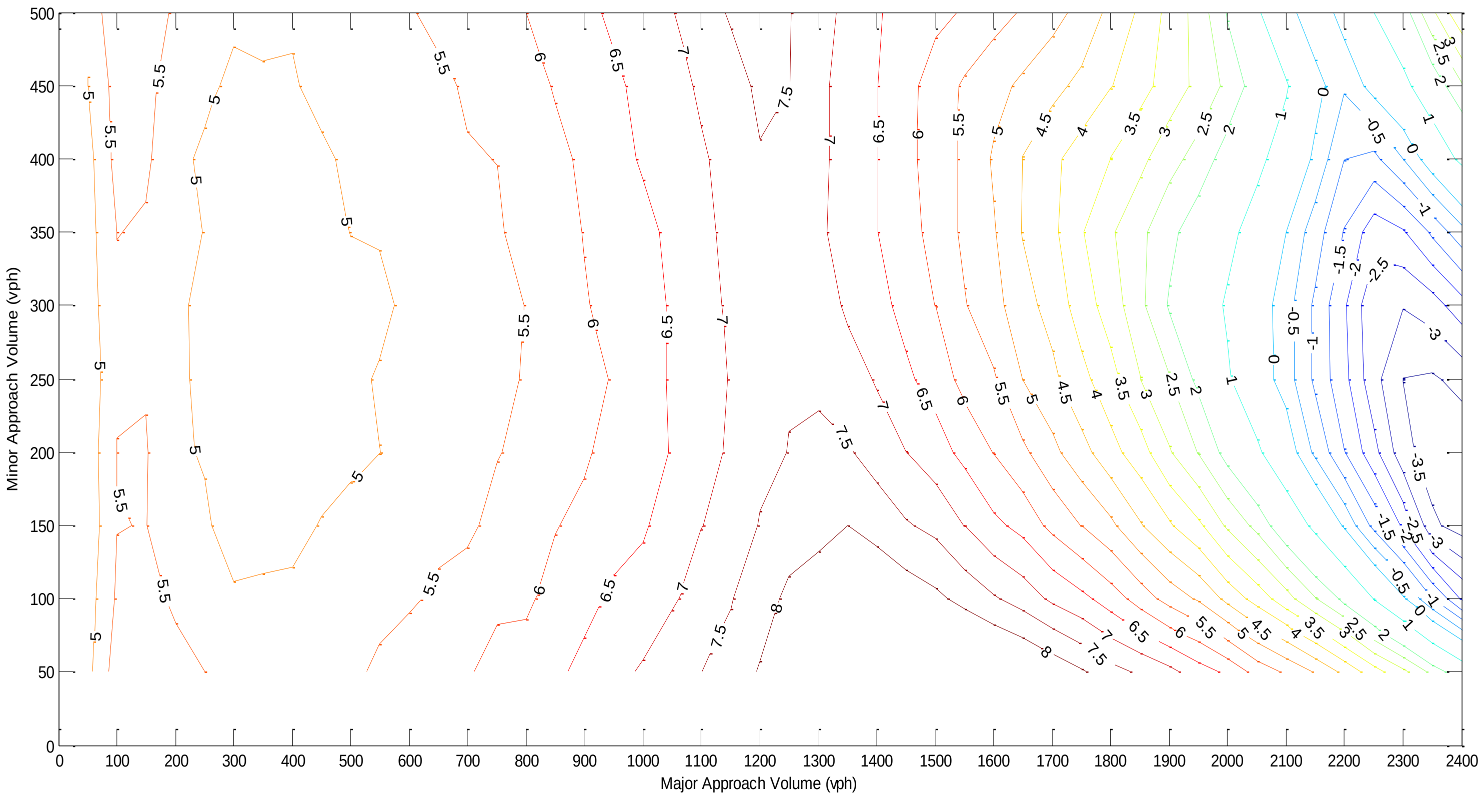


Figure B.134 Contour plot at BF=0.6, LTP=15 %, TP=10 %

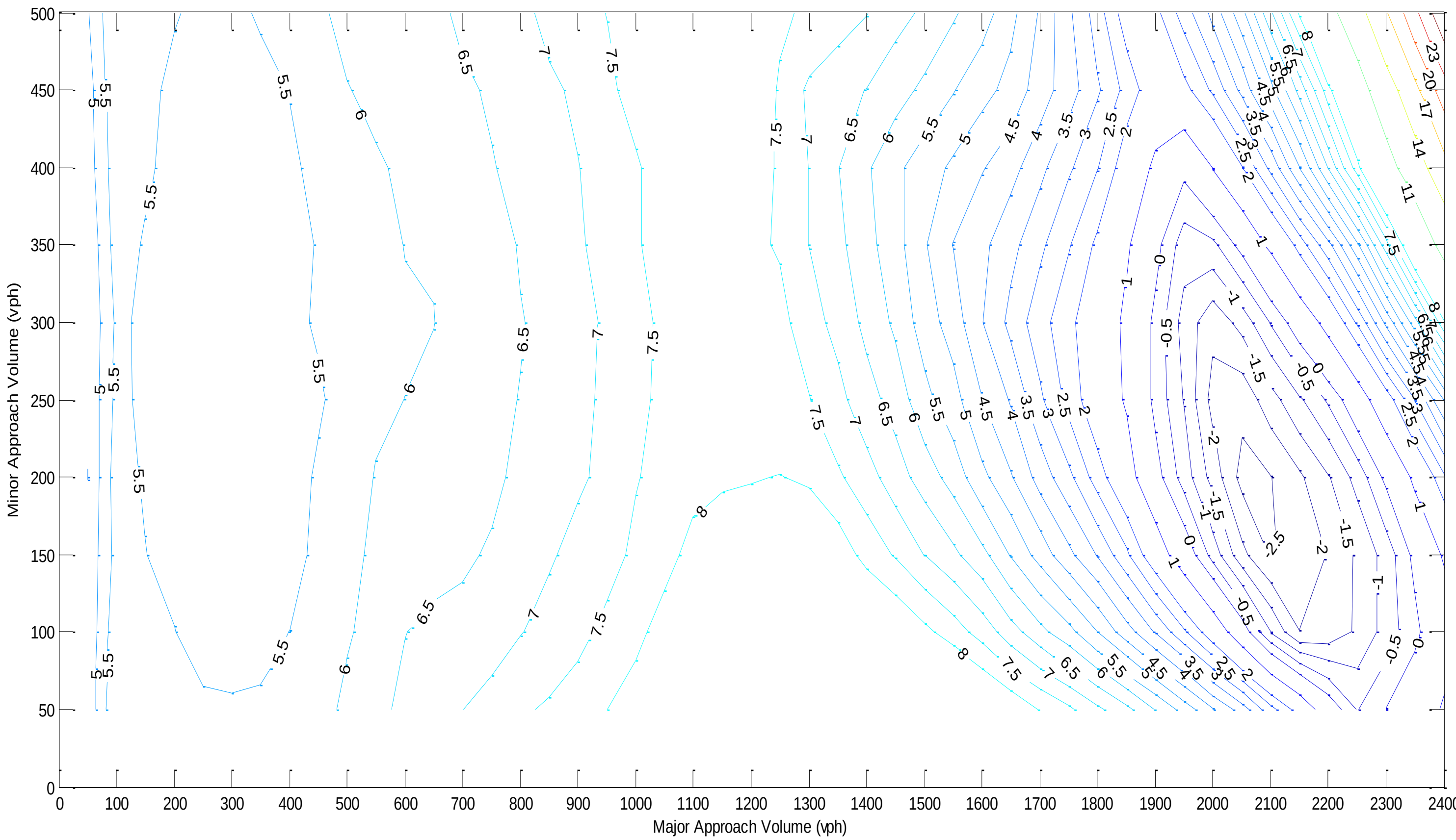


Figure B.135 Contour plot at BF=0.7, LTP=15 %, TP=10 %

B.3.2. Contour Plot for Total Intersection Delay Difference between Jugahndle and Standard Signalized Intersection

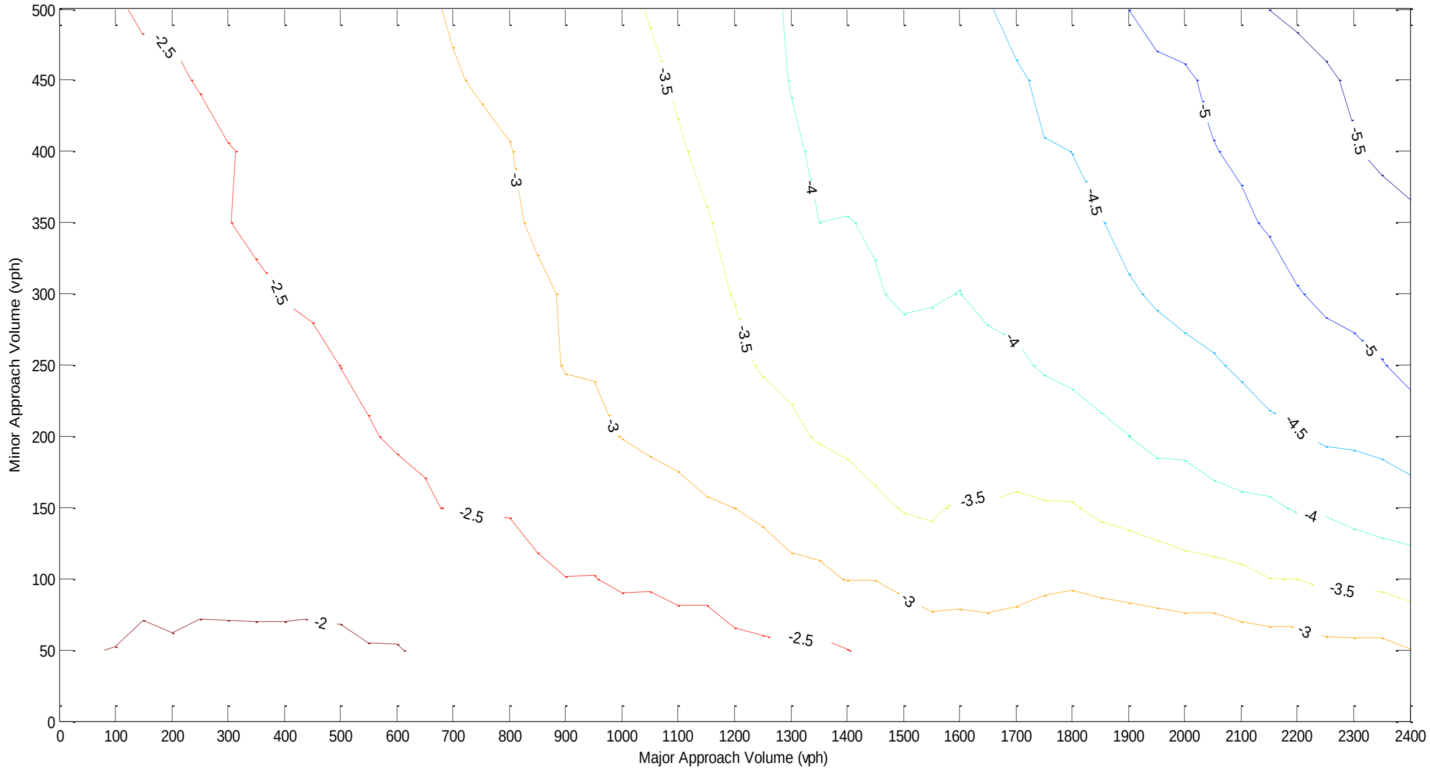


Figure B.136 Contour plot at BF=0.5, LTP=5%, TP=2%

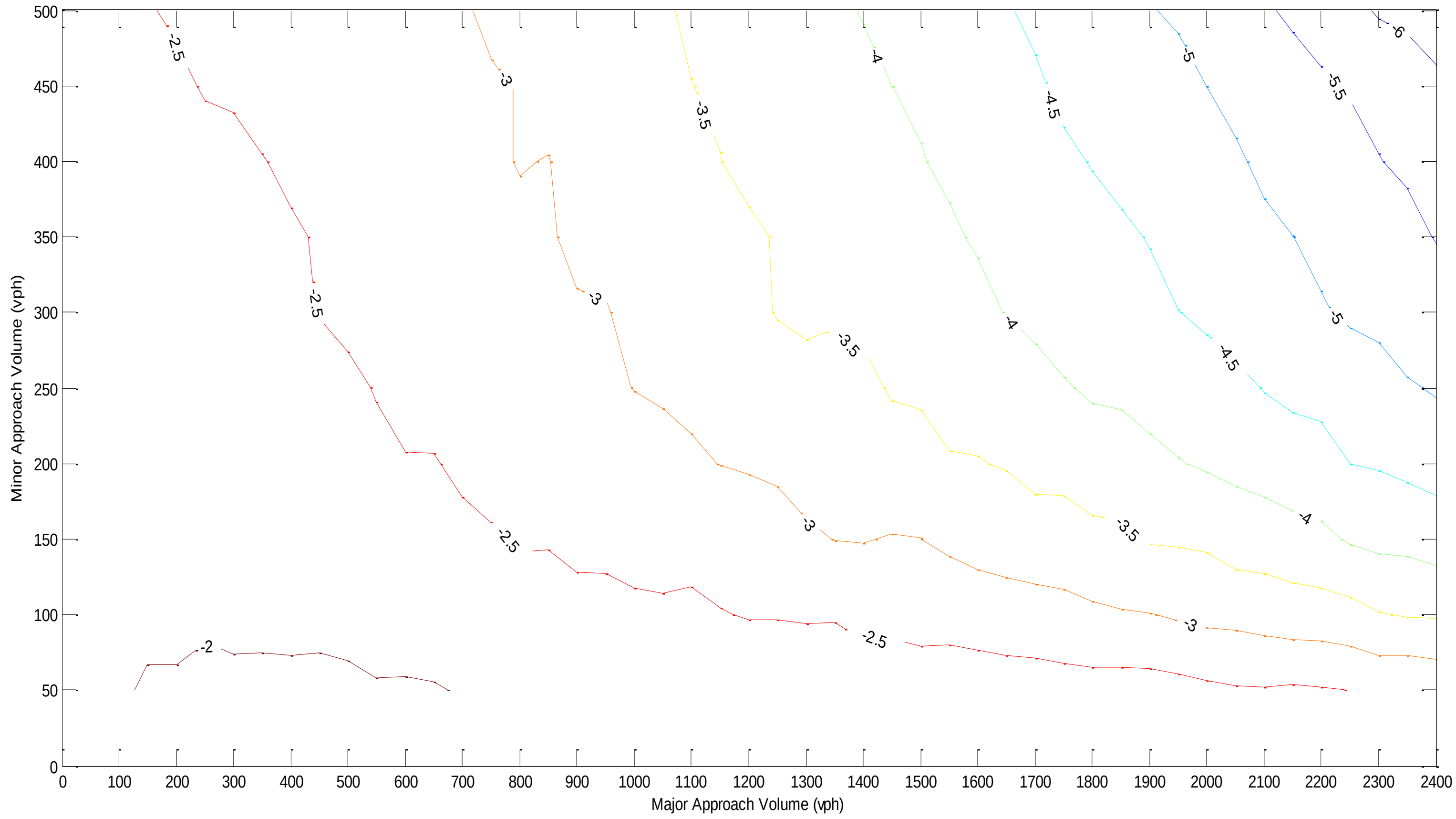


Figure B.137 Contour plot at BF=0.6, LTP=5%, TP=2%

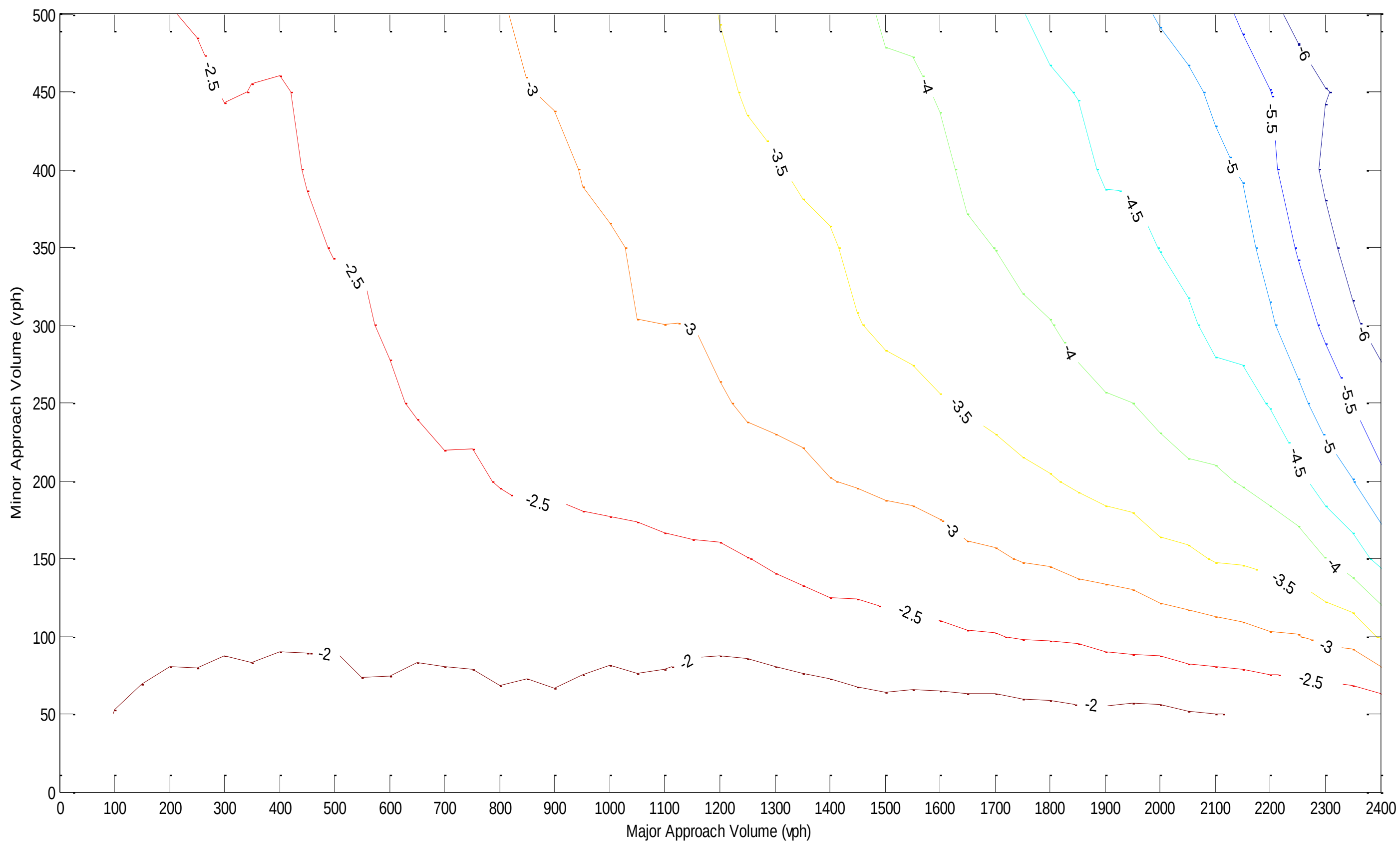


Figure B.138 Contour plot at BF=0.7, LTP=5%, TP=2%

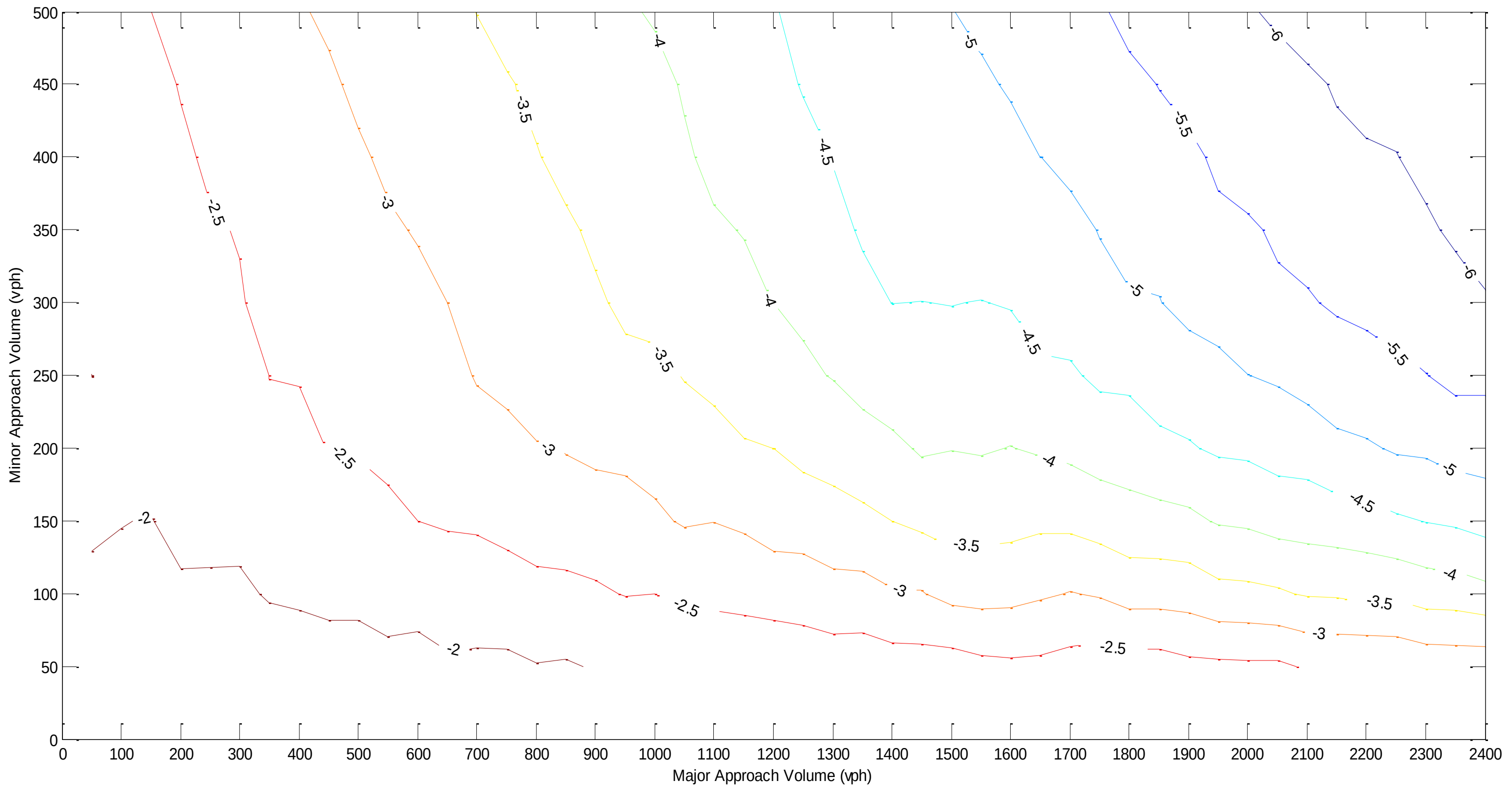


Figure B.139 Contour plot at BF=0.5, LTP=10%, TP=2%

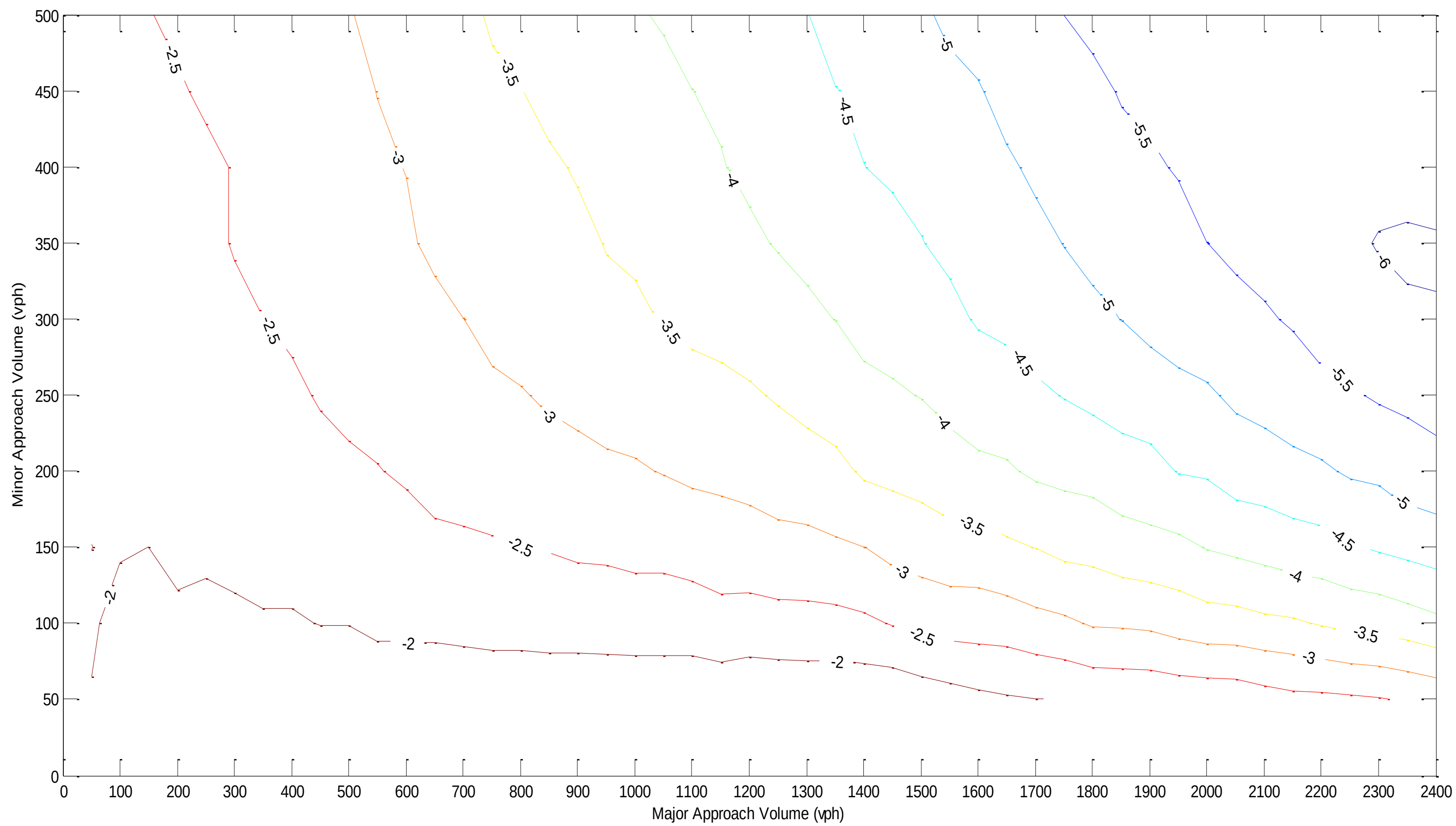


Figure B.140 Contour plot at BF=0.6, LTP=10%, TP=2%

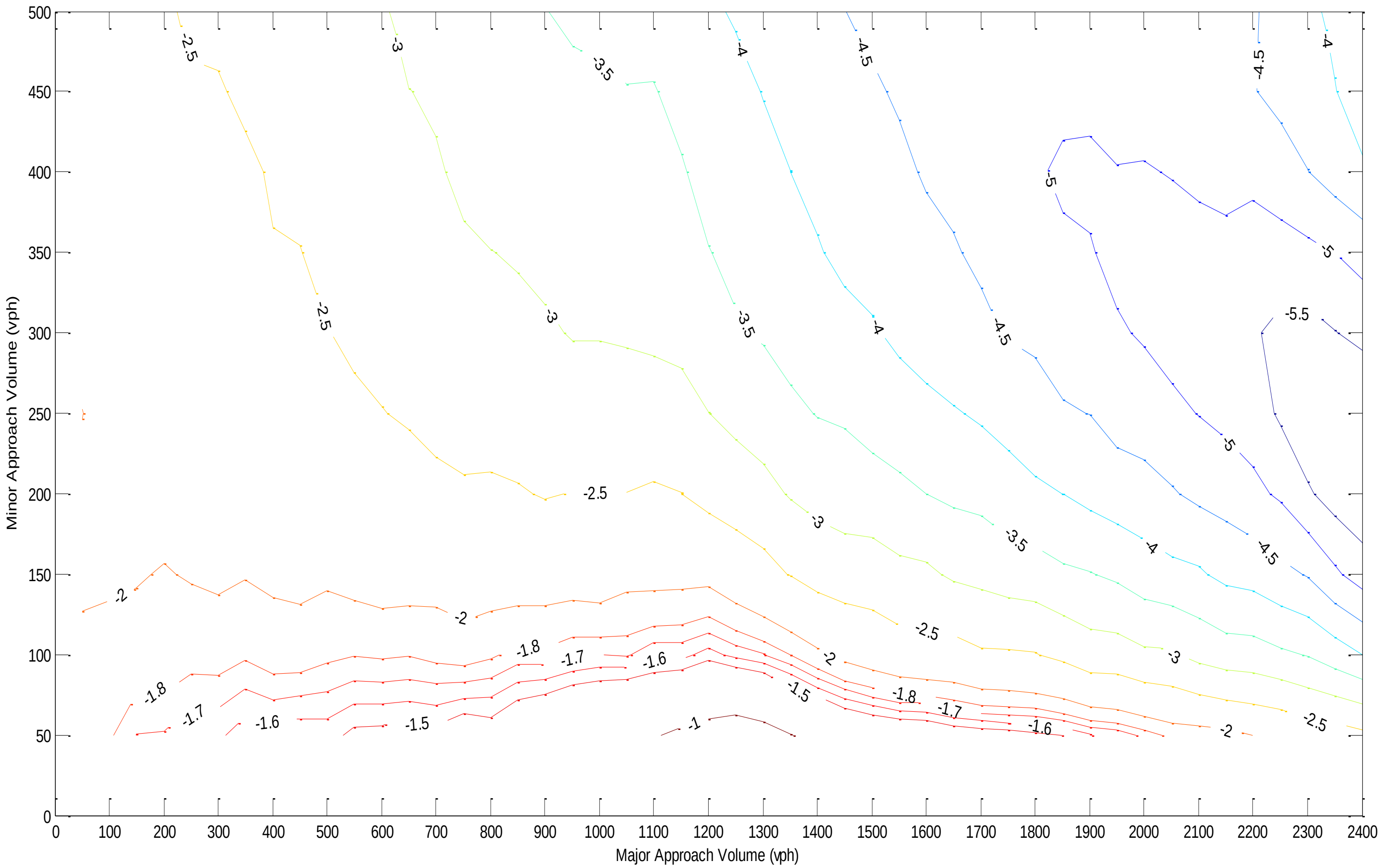


Figure B.141 Contour plot at BF=0.7, LTP=10%, TP=2%

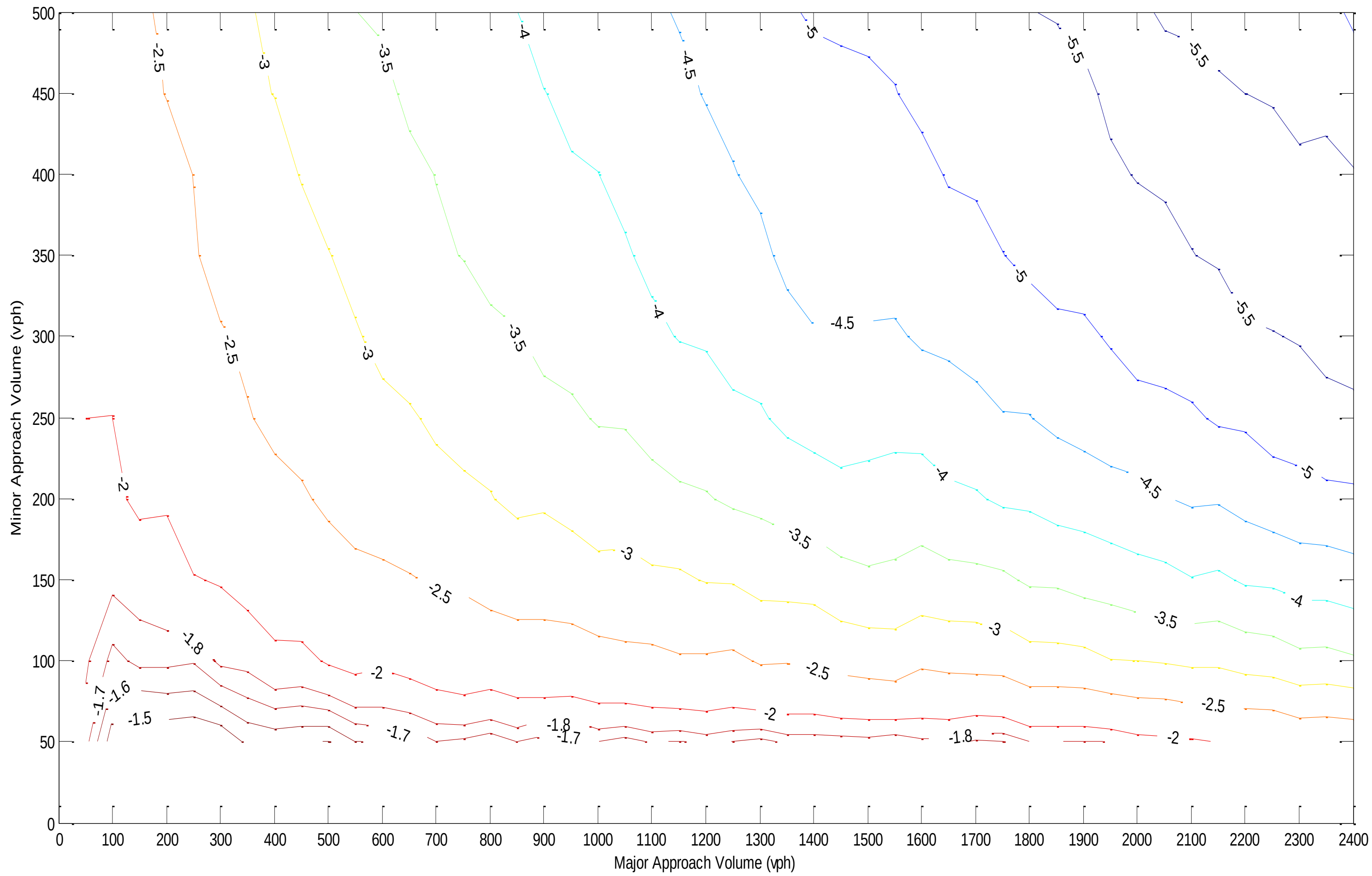


Figure B.142 Contour plot at BF=0.5, LTP=15%, TP=2%

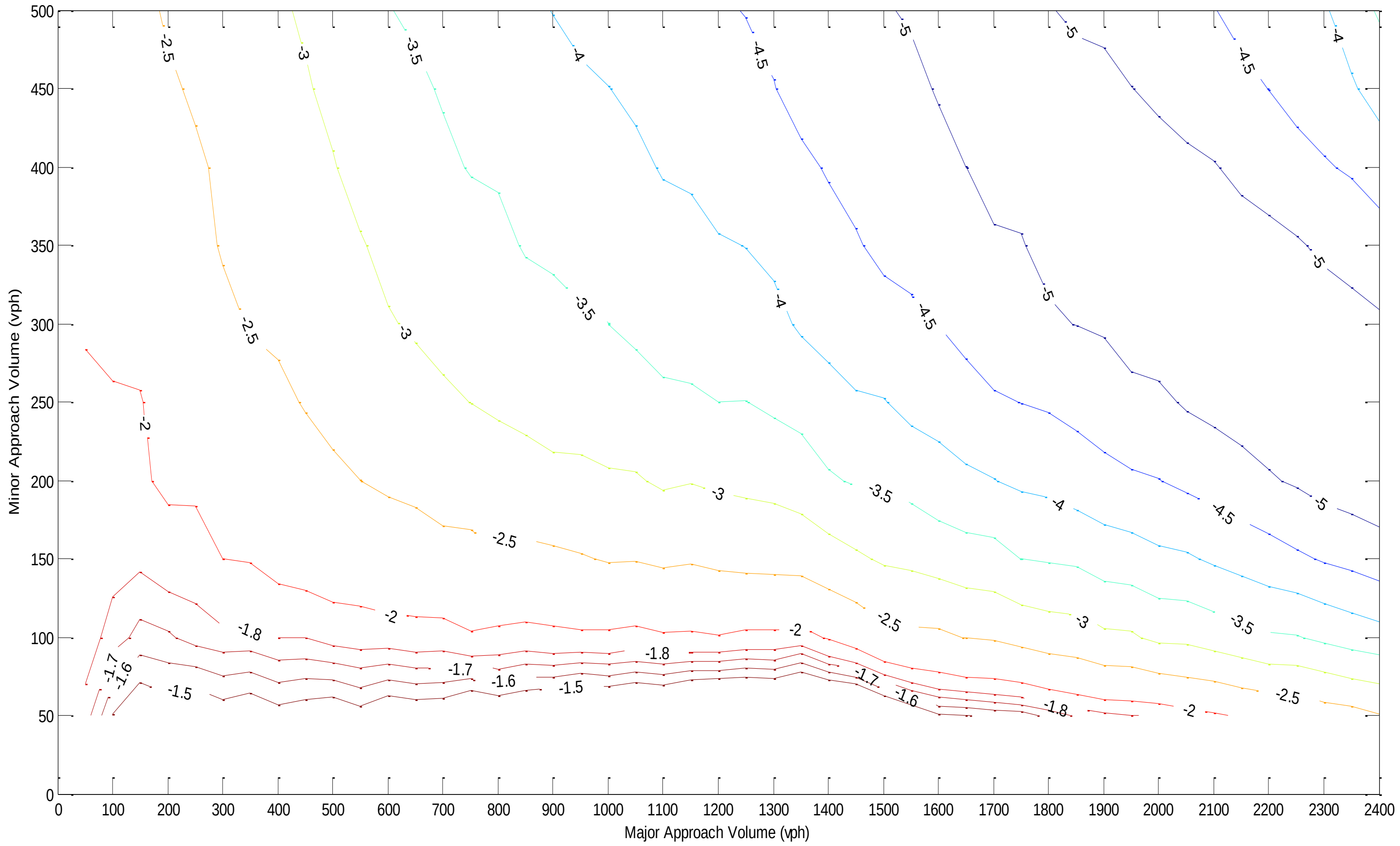


Figure B.143 Contour plot at BF=0.6, LTP=15%, TP=2%

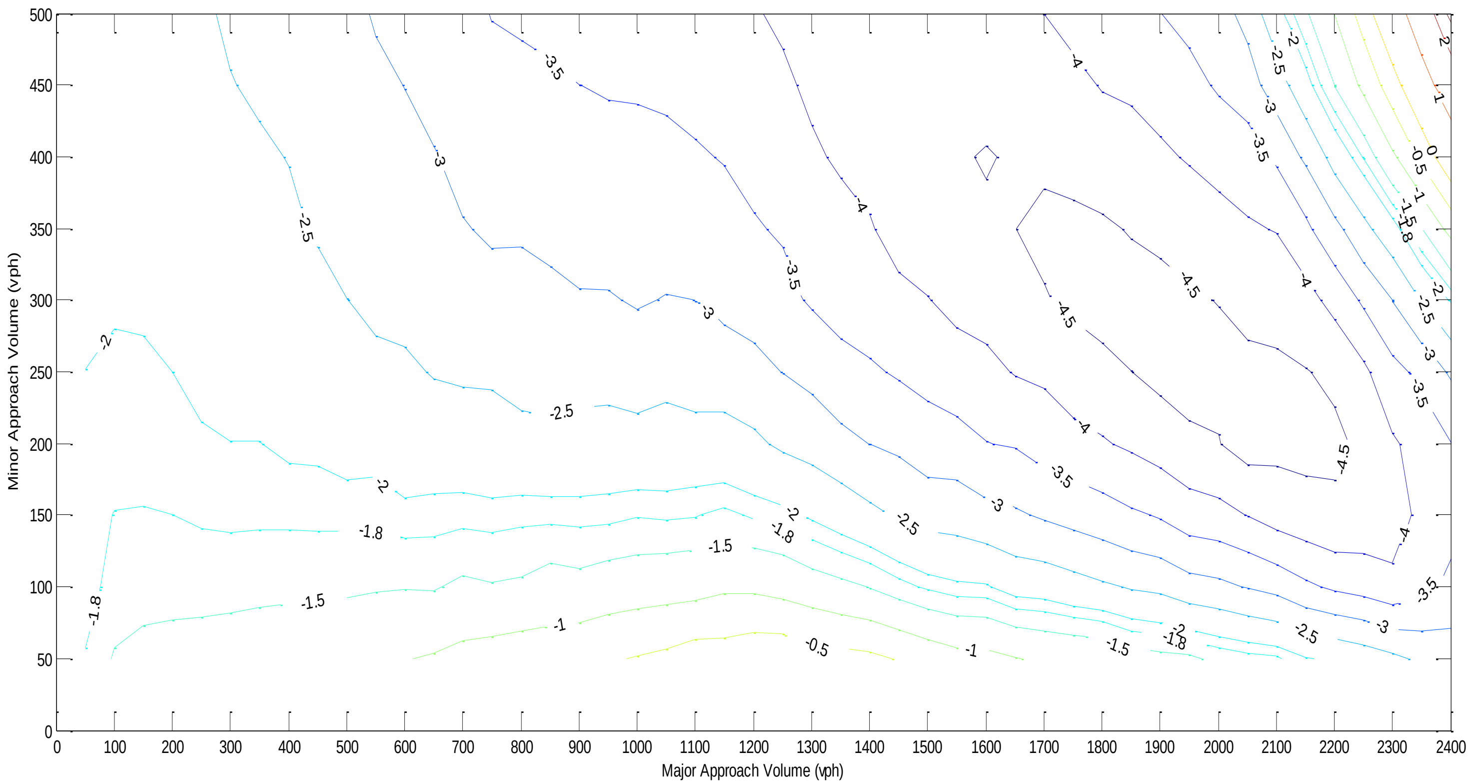


Figure B.144 Contour plot at BF=0.7, LTP=15%, TP=2%

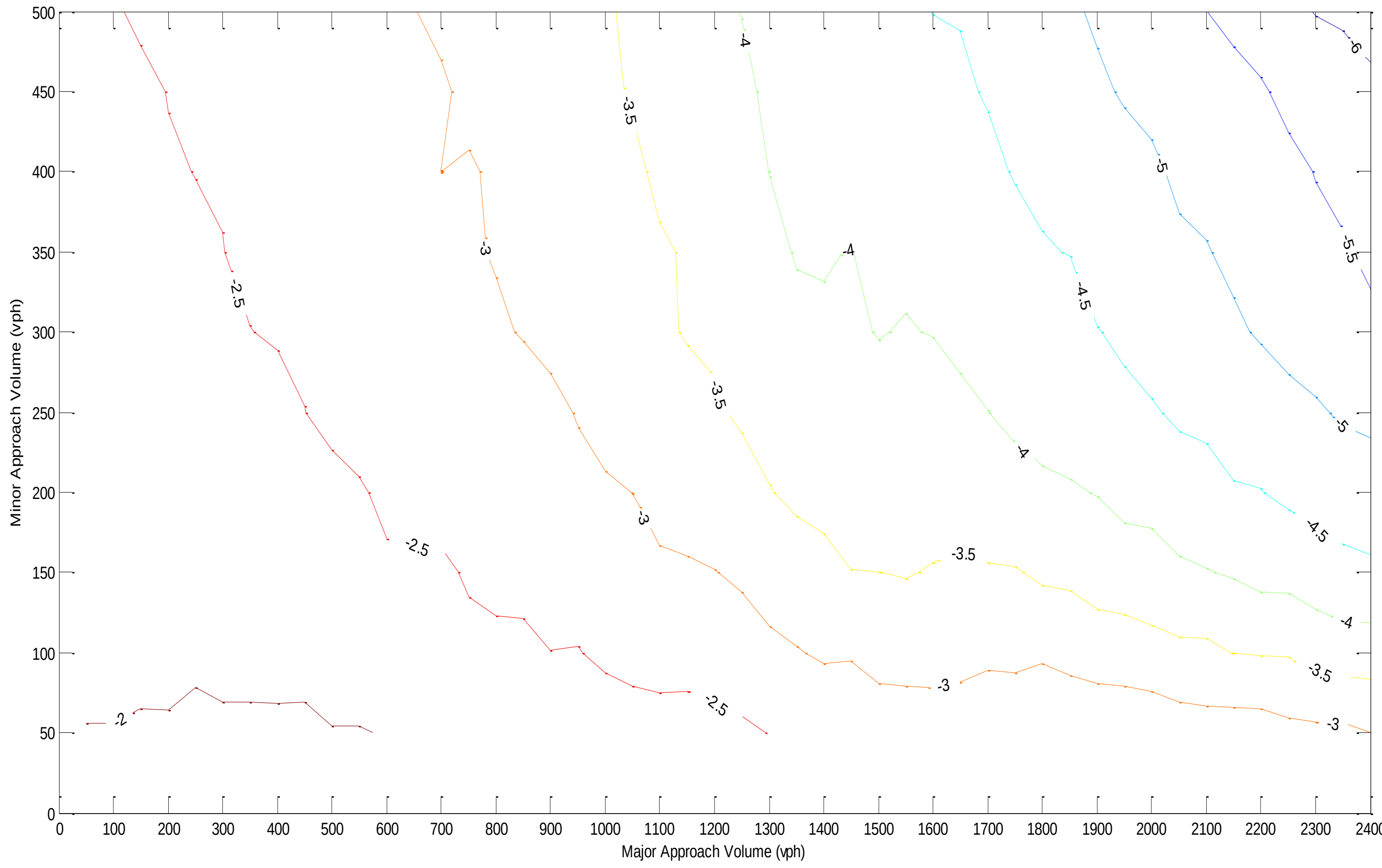


Figure B.145 Contour plot at BF=0.5, LTP=5 %, TP=5 %

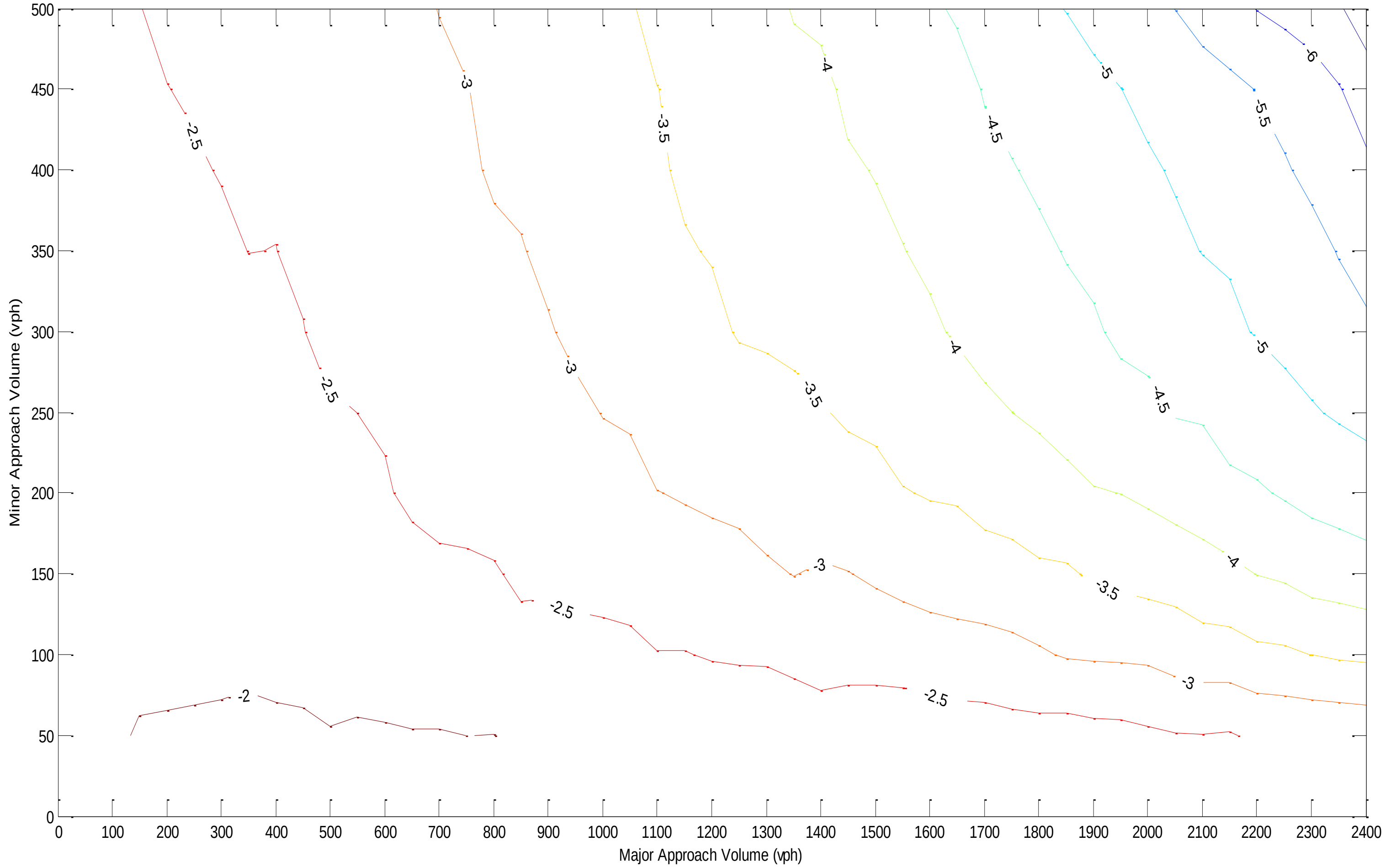


Figure B.146 Contour plot at BF=0.6, LTP=5 %, TP=5 %

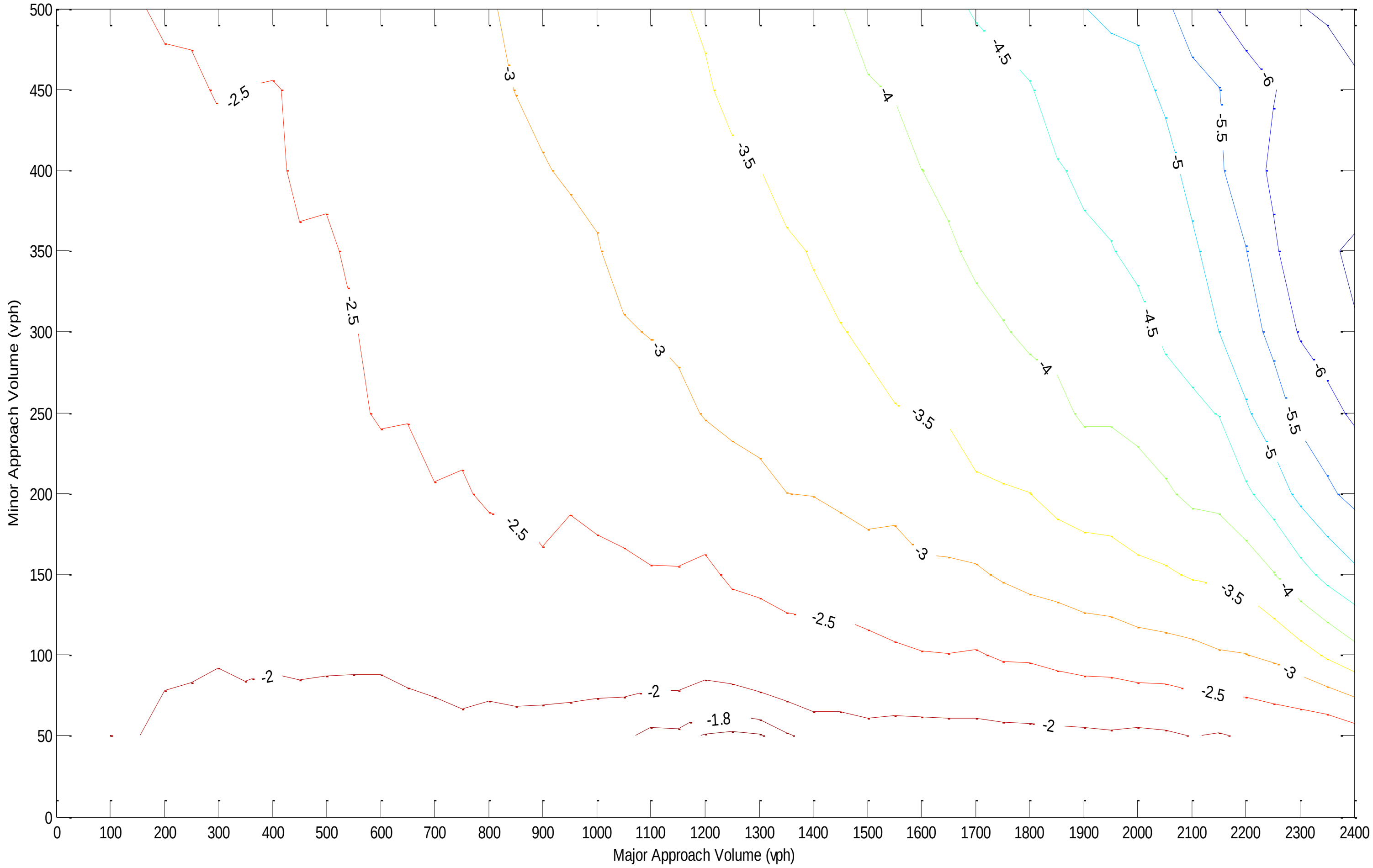


Figure B.147 Contour plot at BF=0.7, LTP=5 %, TP=5 %

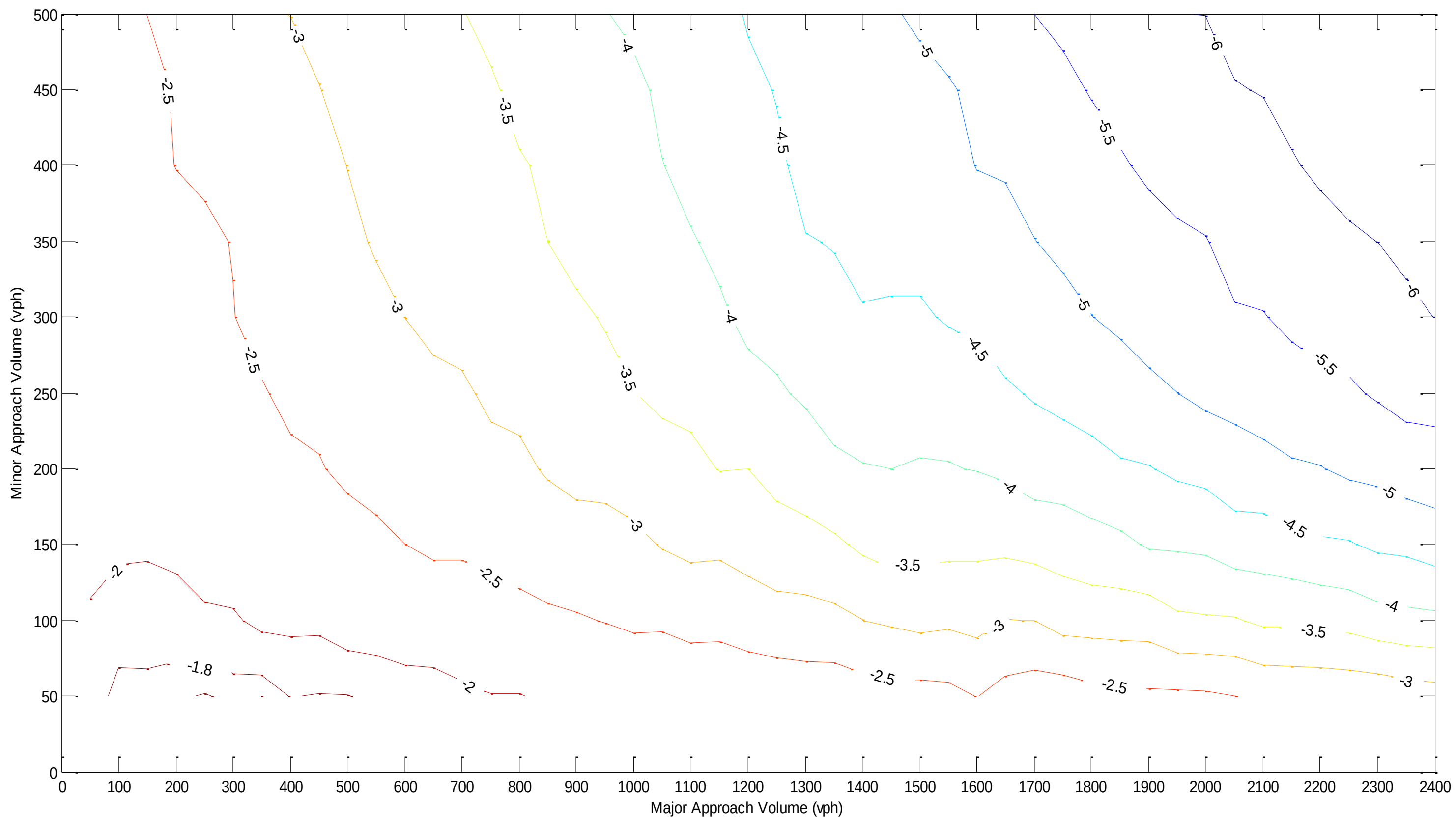


Figure B.148 Contour plot at BF=0.5, LTP=10 %, TP=5 %

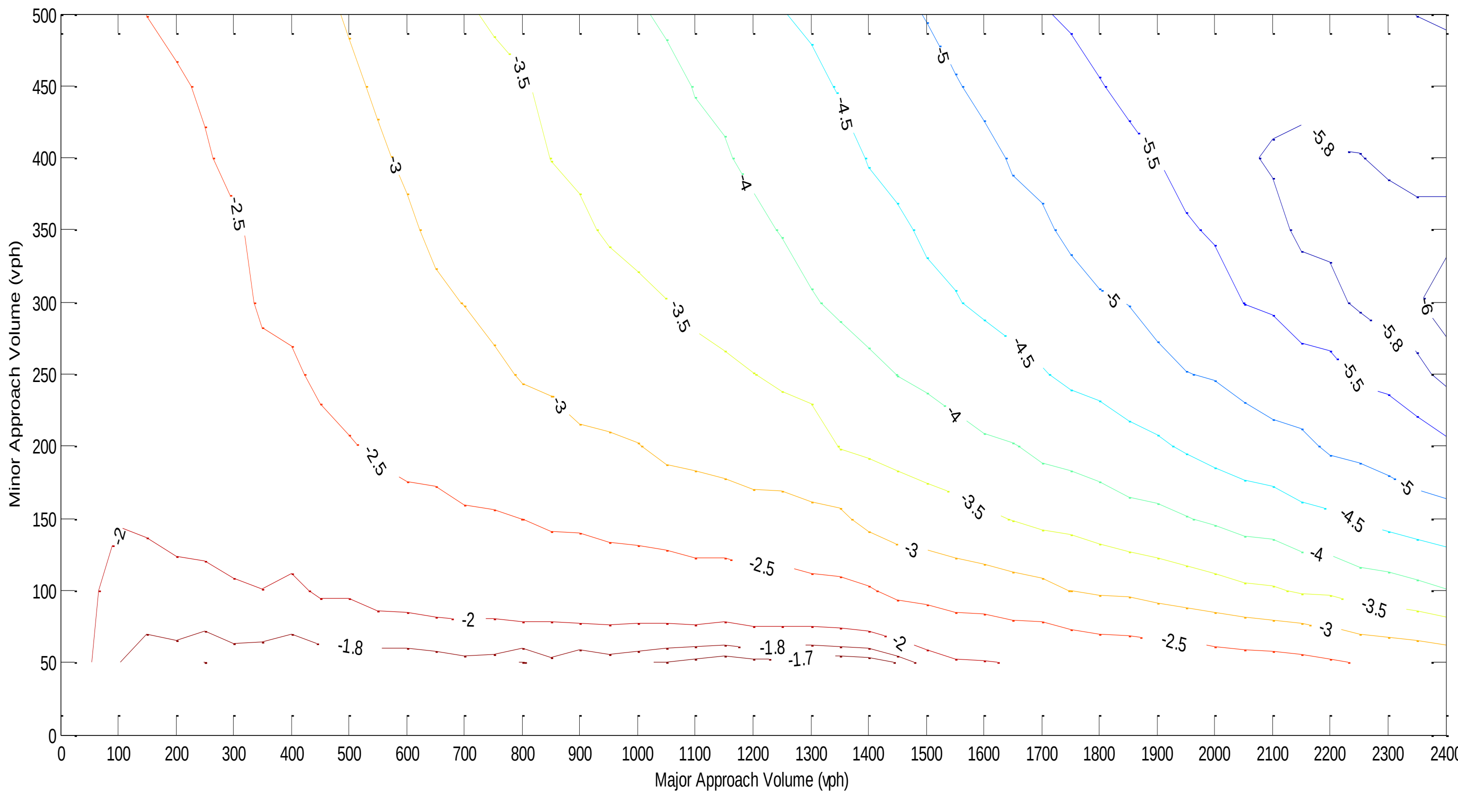


Figure B.149 Contour plot at BF=0.6, LTP=10 %, TP=5 %

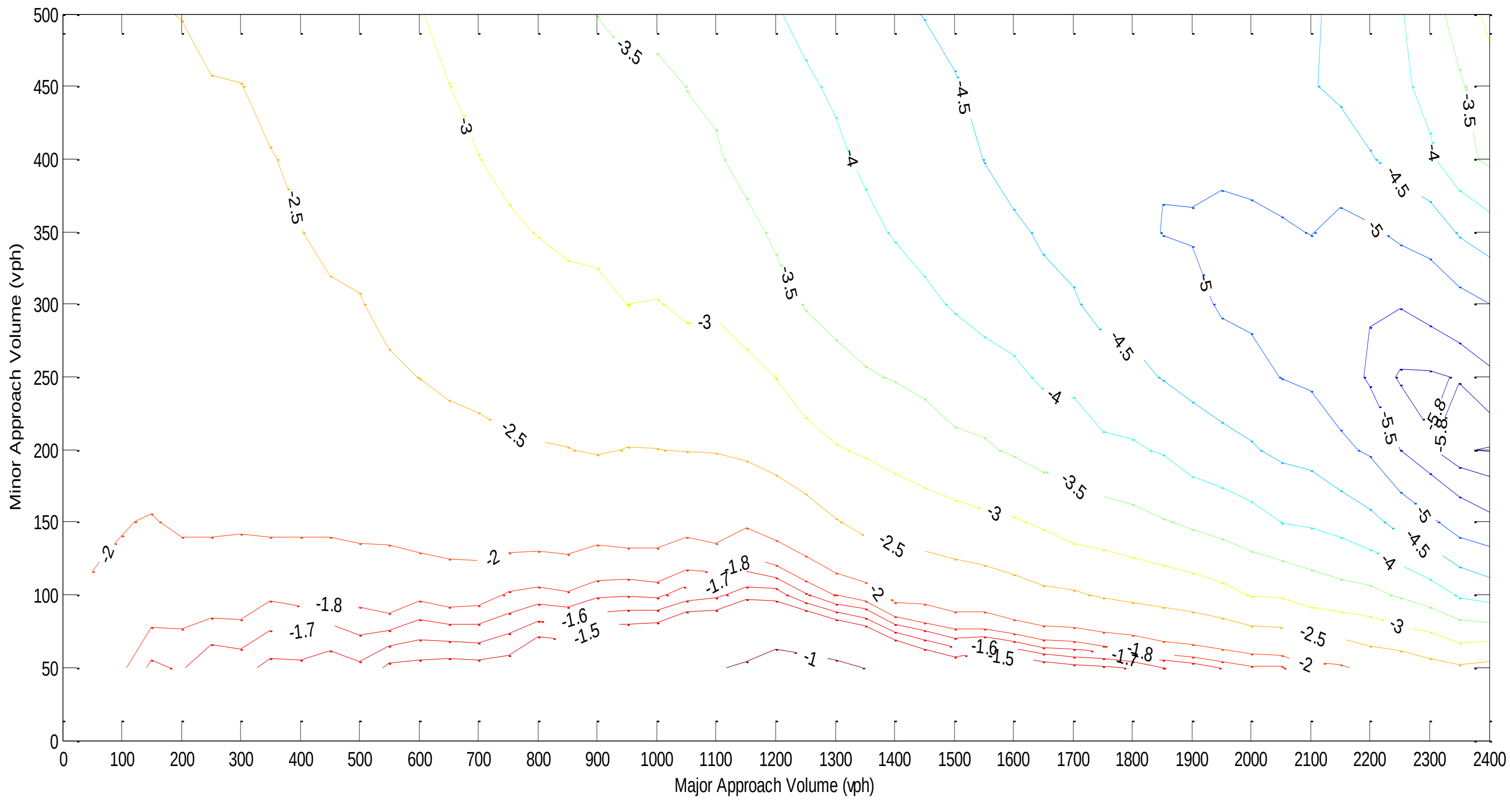


Figure B.150 Contour plot at BF=0.7, LTP=10 %, TP=5 %

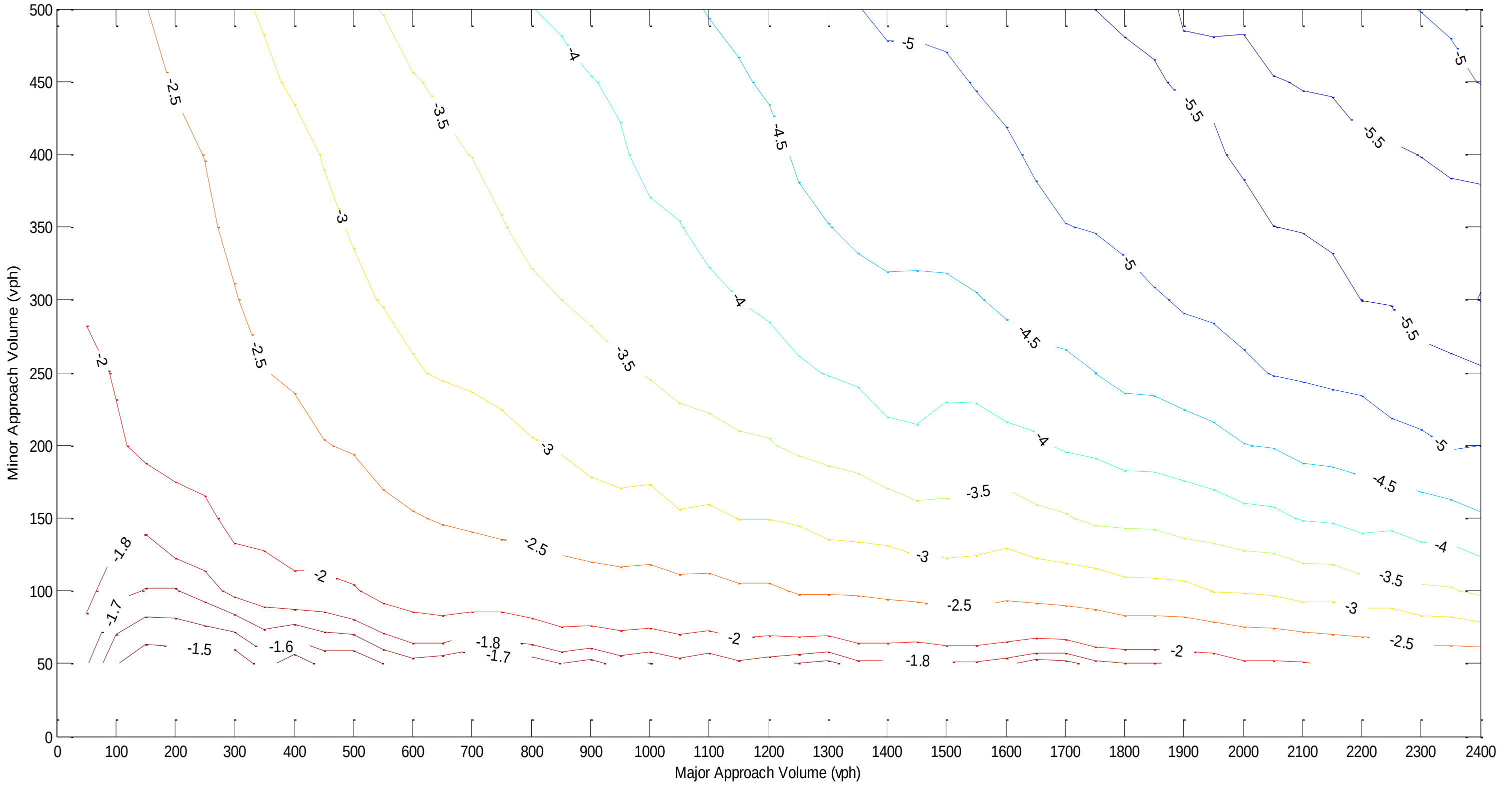


Figure B.151 Contour plot at BF=0.5, LTP=15 %, TP=5 %

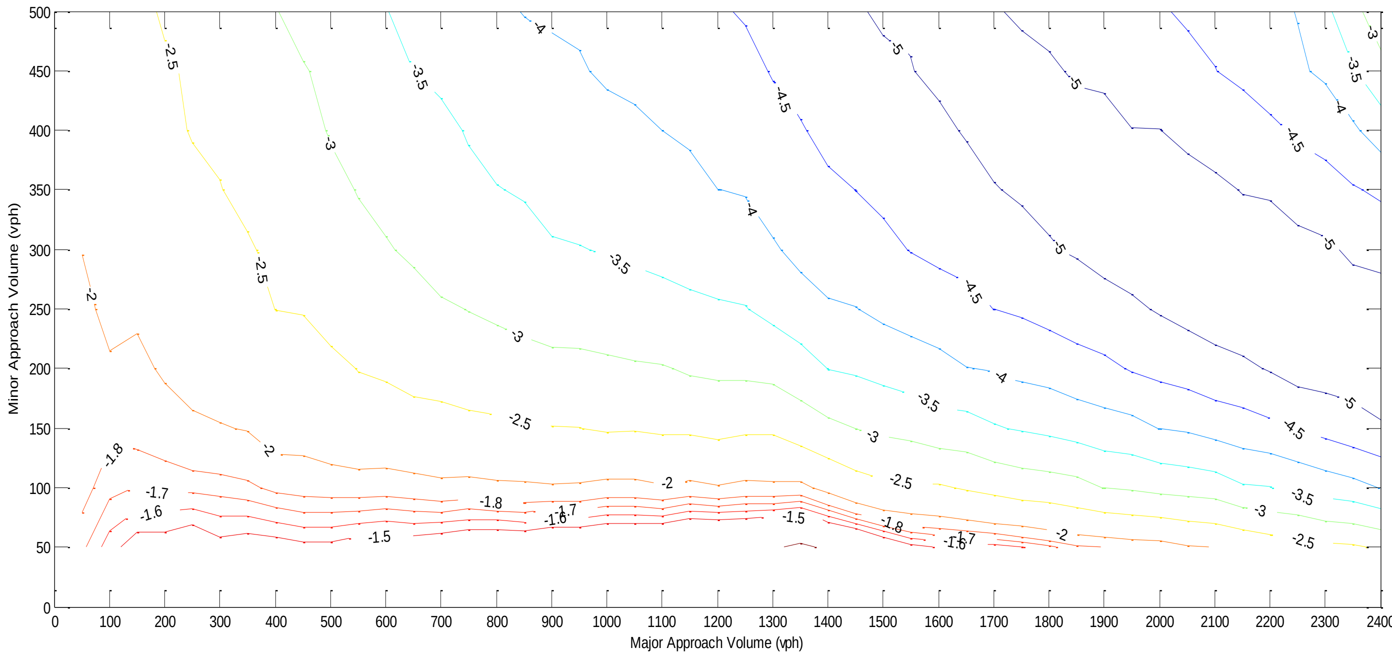


Figure B.152 Contour plot at BF=0.6, LTP=15 %, TP=5 %

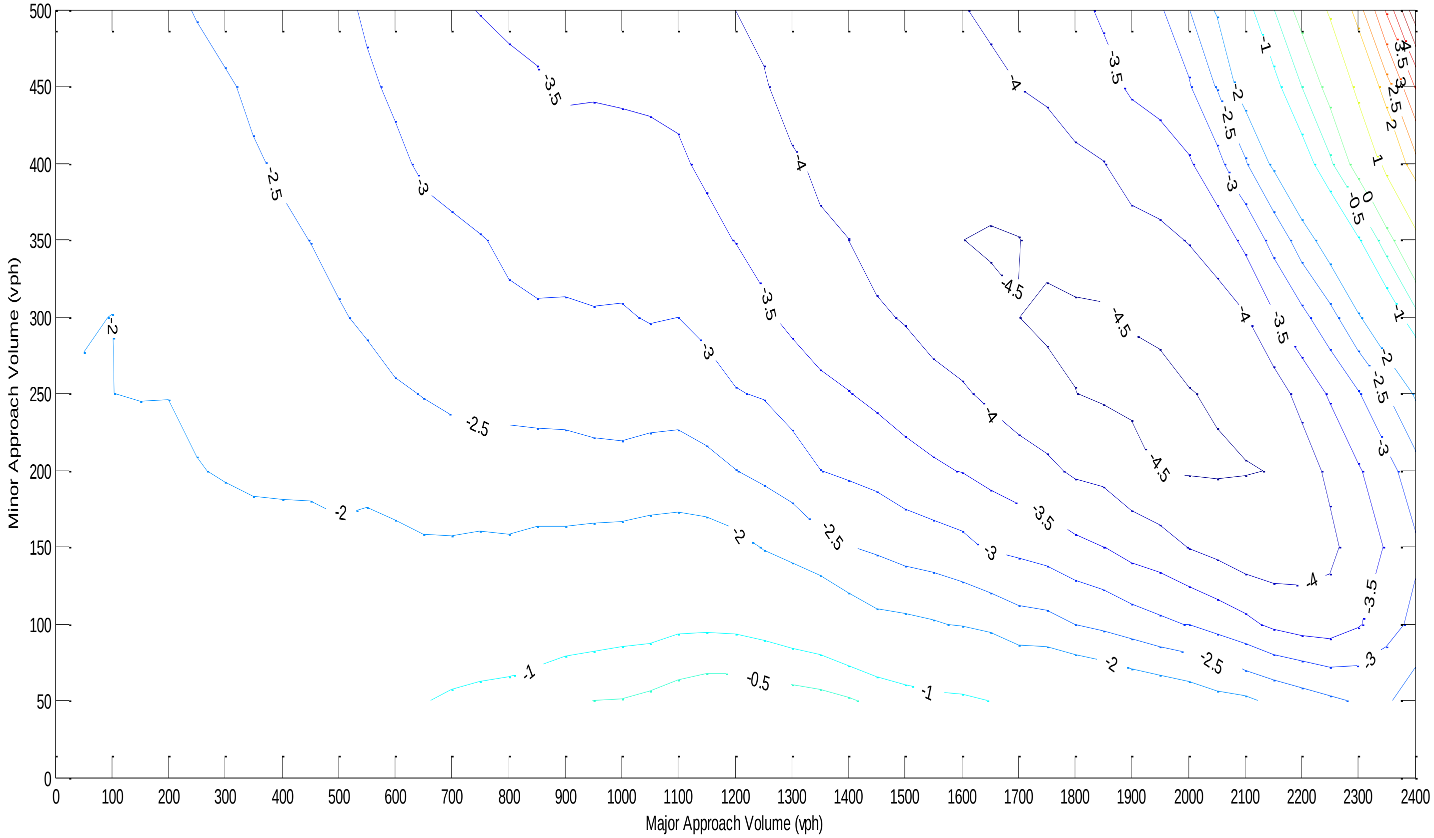


Figure B.153 Contour plot at BF=0.7, LTP=15 %, TP=5 %

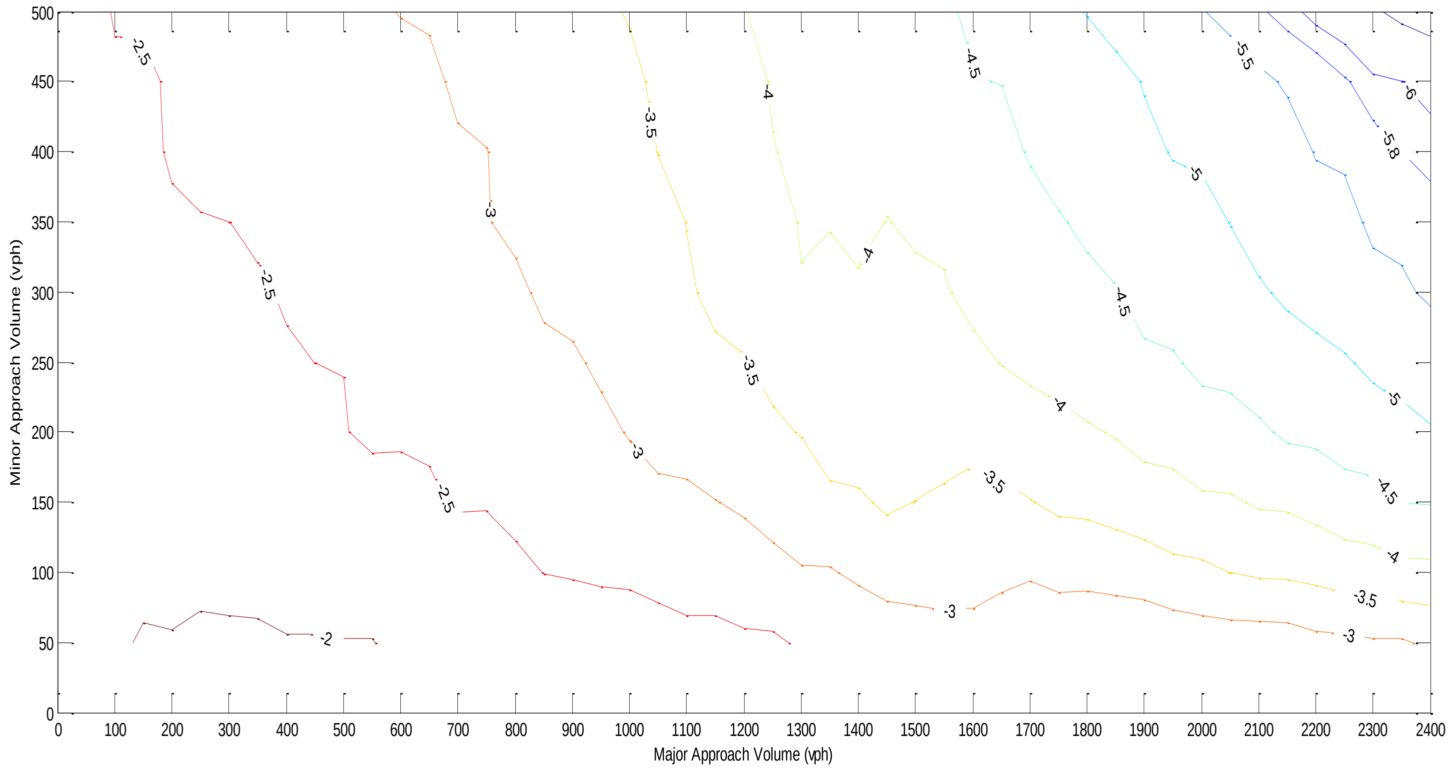


Figure B.154 Contour plot at BF=0.5, LTP=5 %, TP=10 %

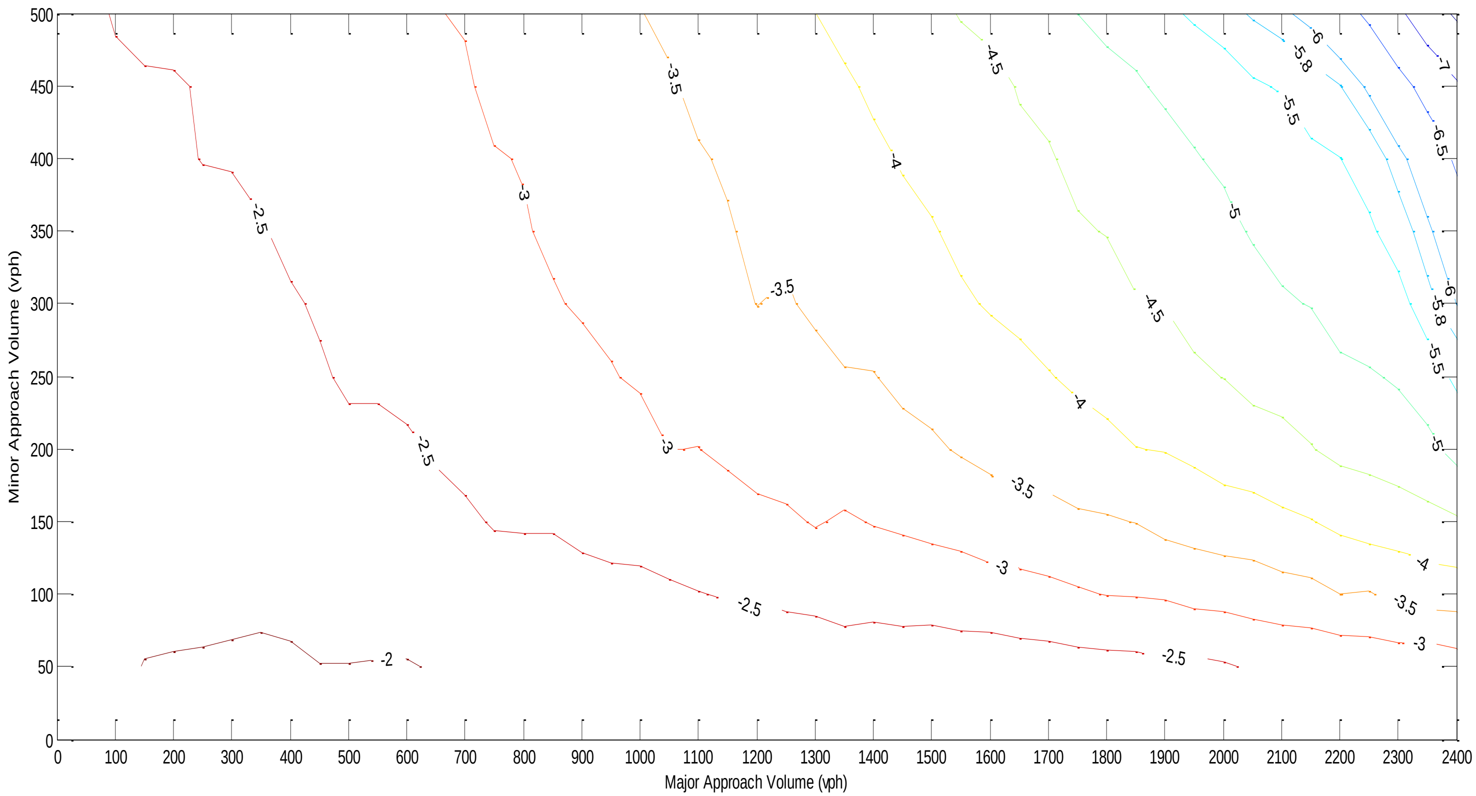


Figure B.155 Contour plot at BF=0.6, LTP=5 %, TP=10 %

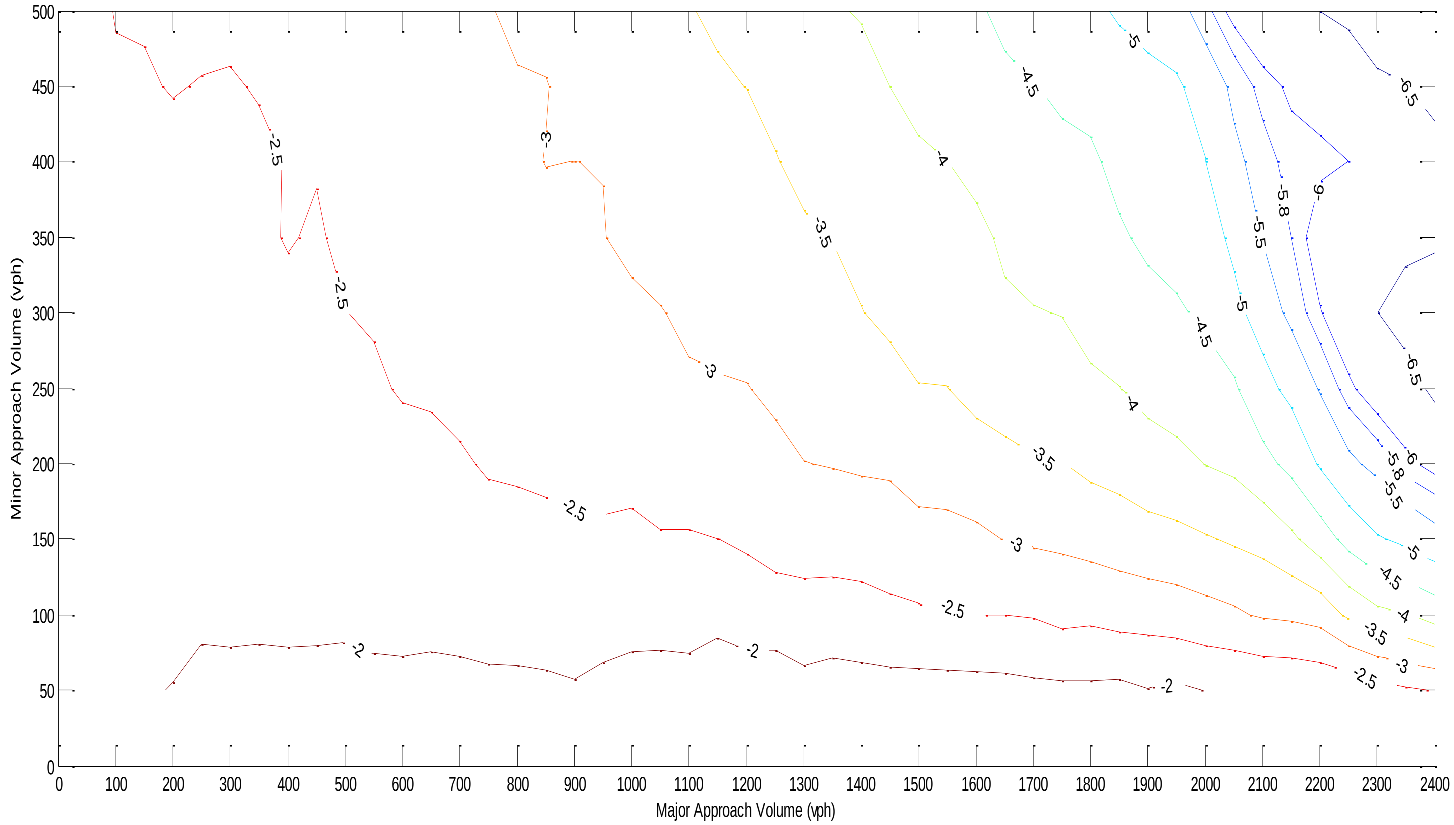


Figure B.156 Contour plot at BF=0.7, LTP=5 %, TP=10 %

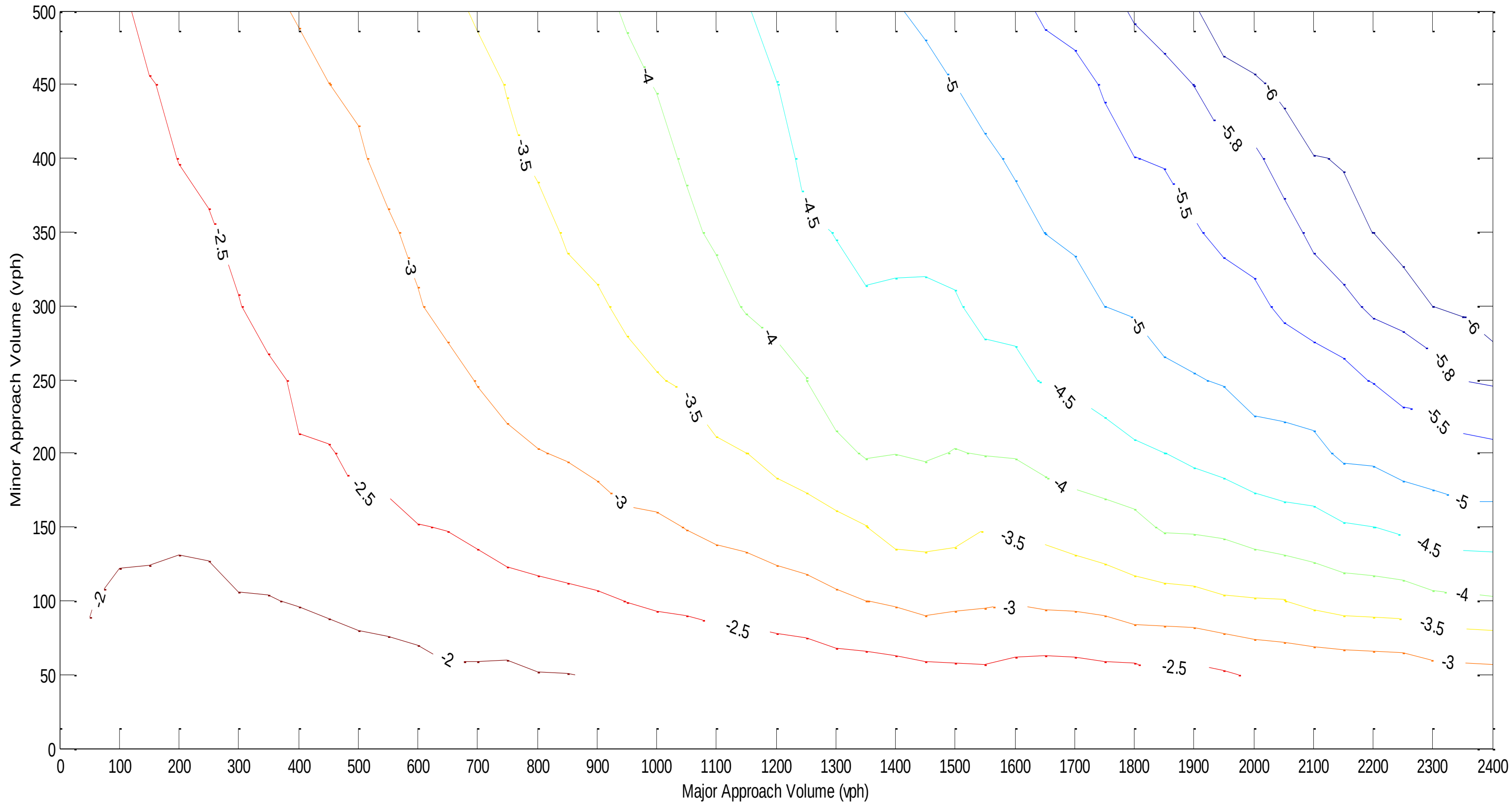


Figure B.157 Contour plot at BF=0.5, LTP=10%, TP=10 %

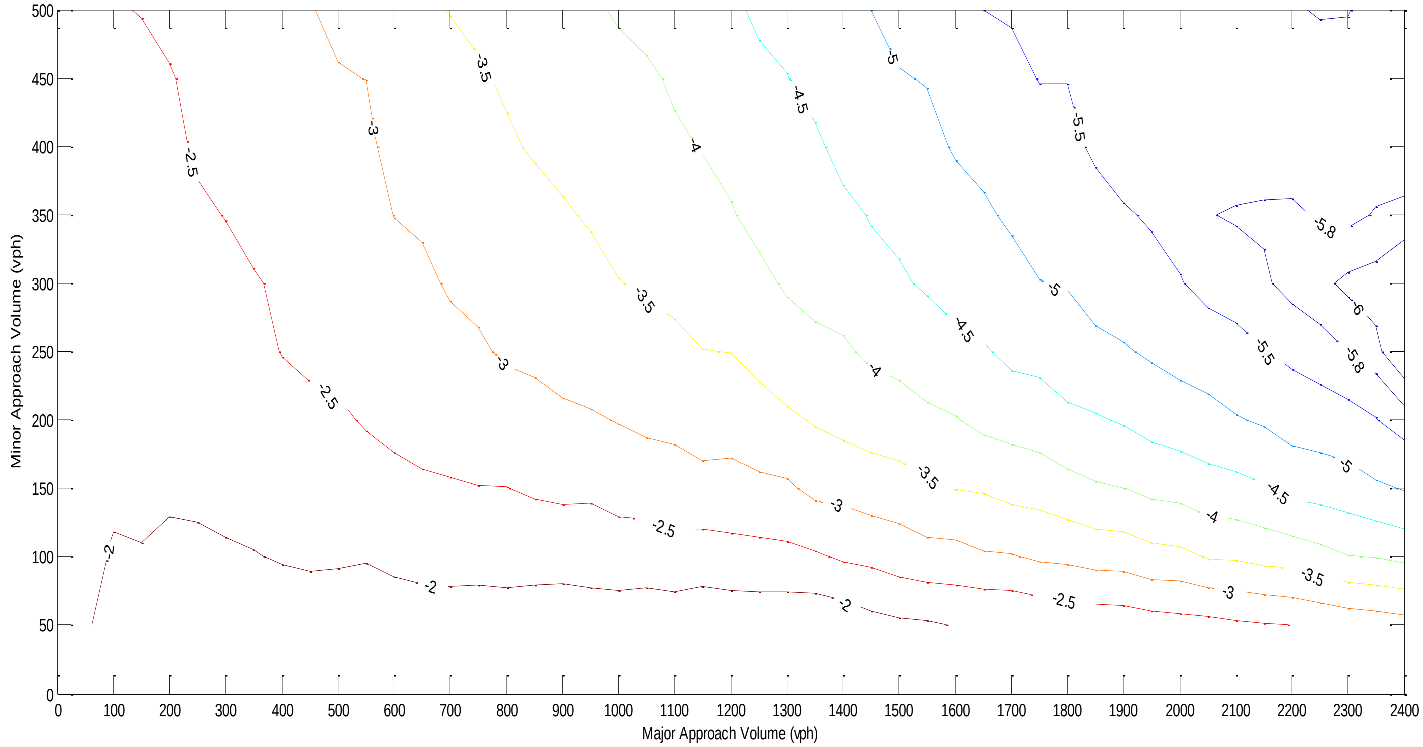


Figure B.158 Contour plot at BF=0.6, LTP=10 %, TP=10 %

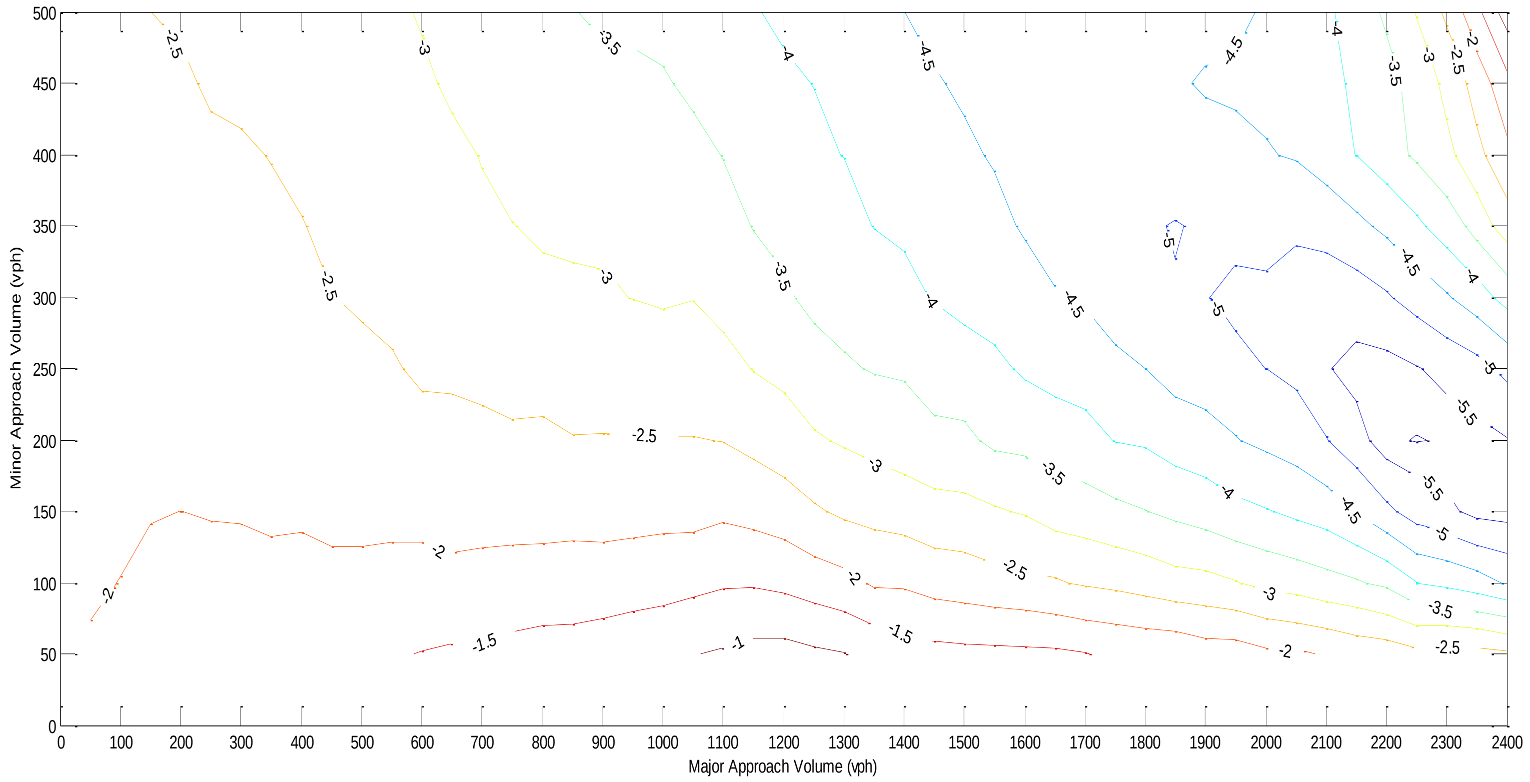


Figure B.159 Contour plot at BF=0.7, LTP=10 %, TP=10 %

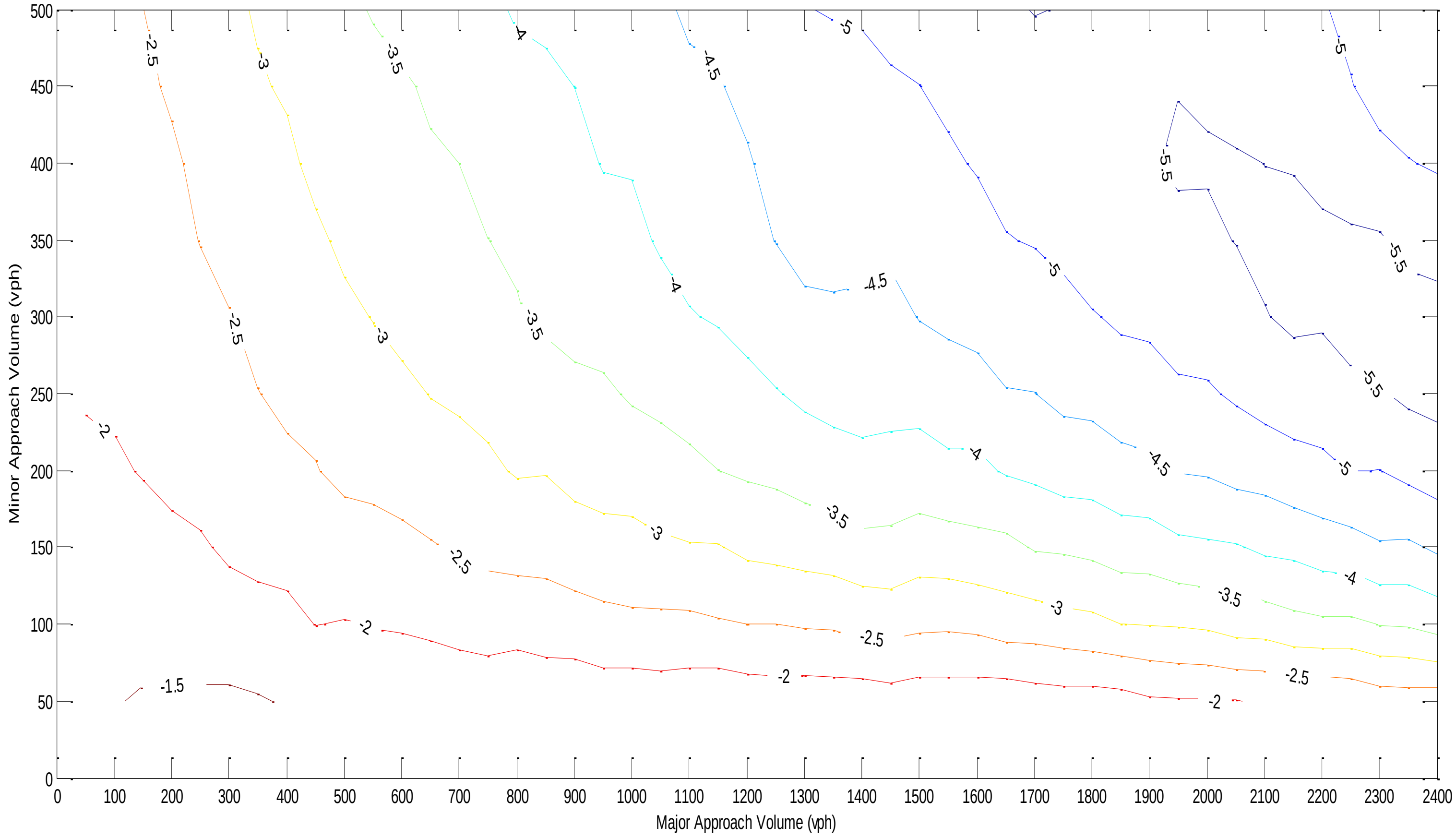


Figure B.160 Contour plot at BF=0.5, LTP=15 %, TP=10 %

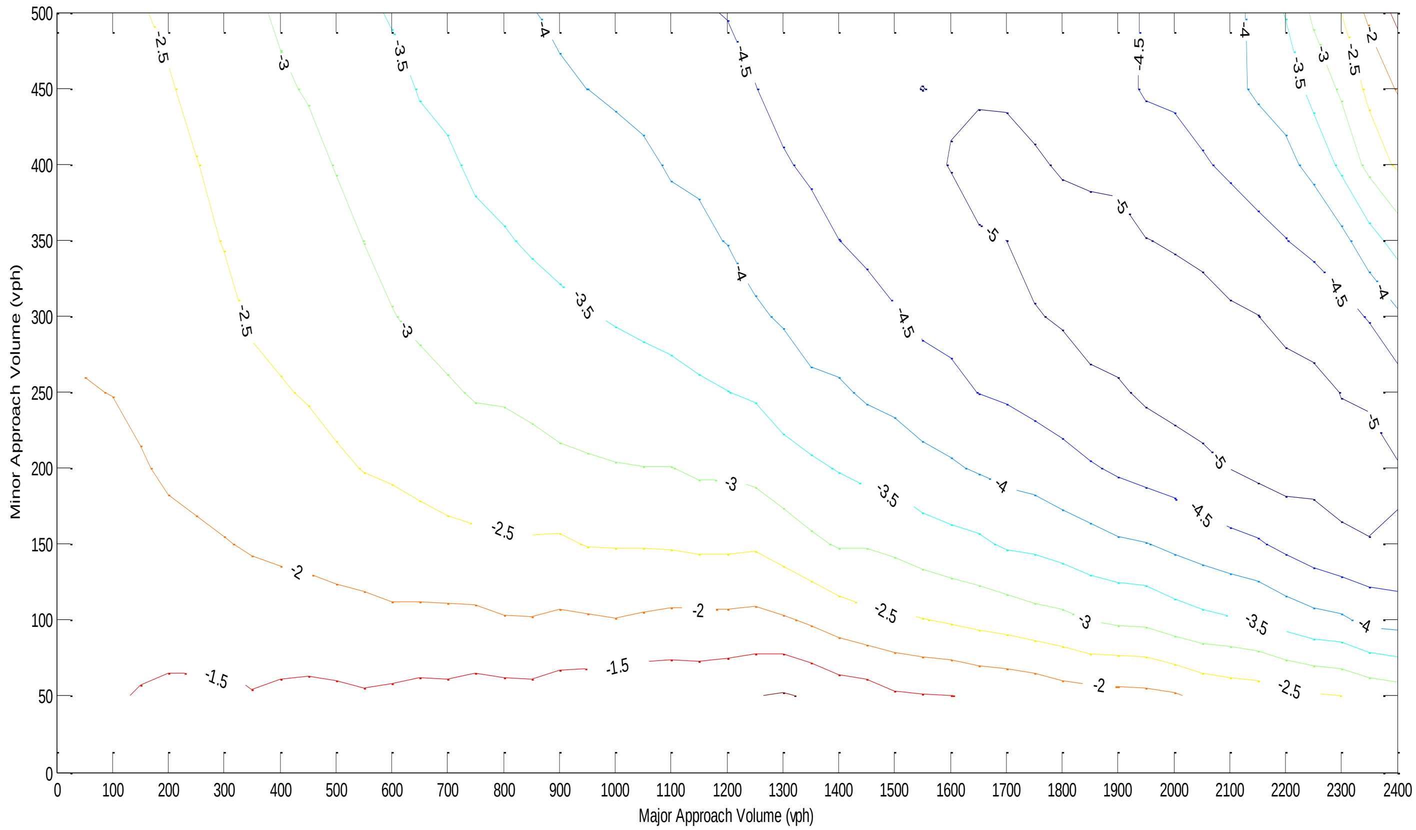


Figure B.161 Contour plot at BF=0.6, LTP=15 %, TP=10 %

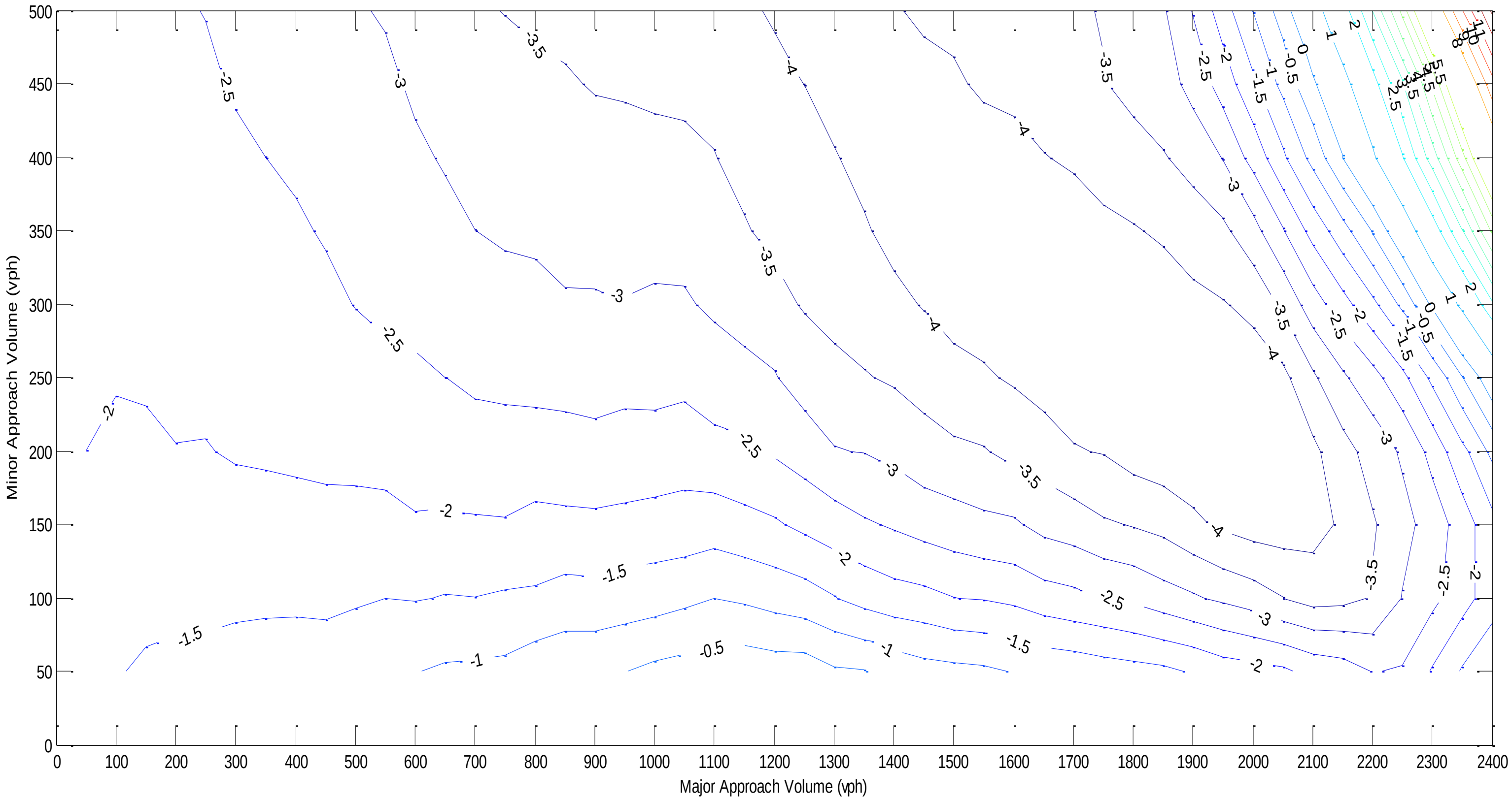


Figure B.162 Contour plot at BF=0.7, LTP=15 %, TP=10 %

B.4. The Growth Curves

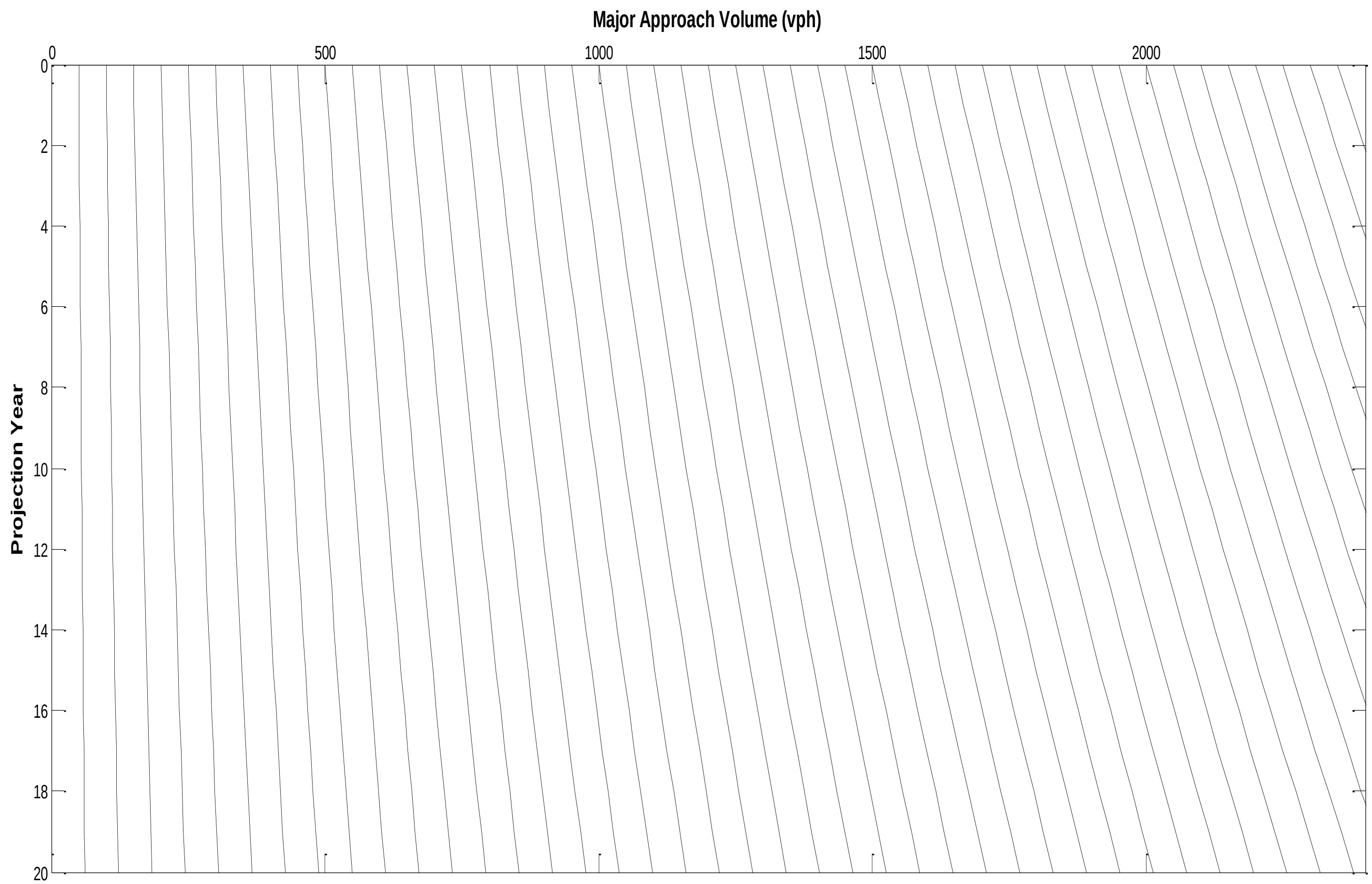


Figure B.163 Growth curves for growth rate of 1 %

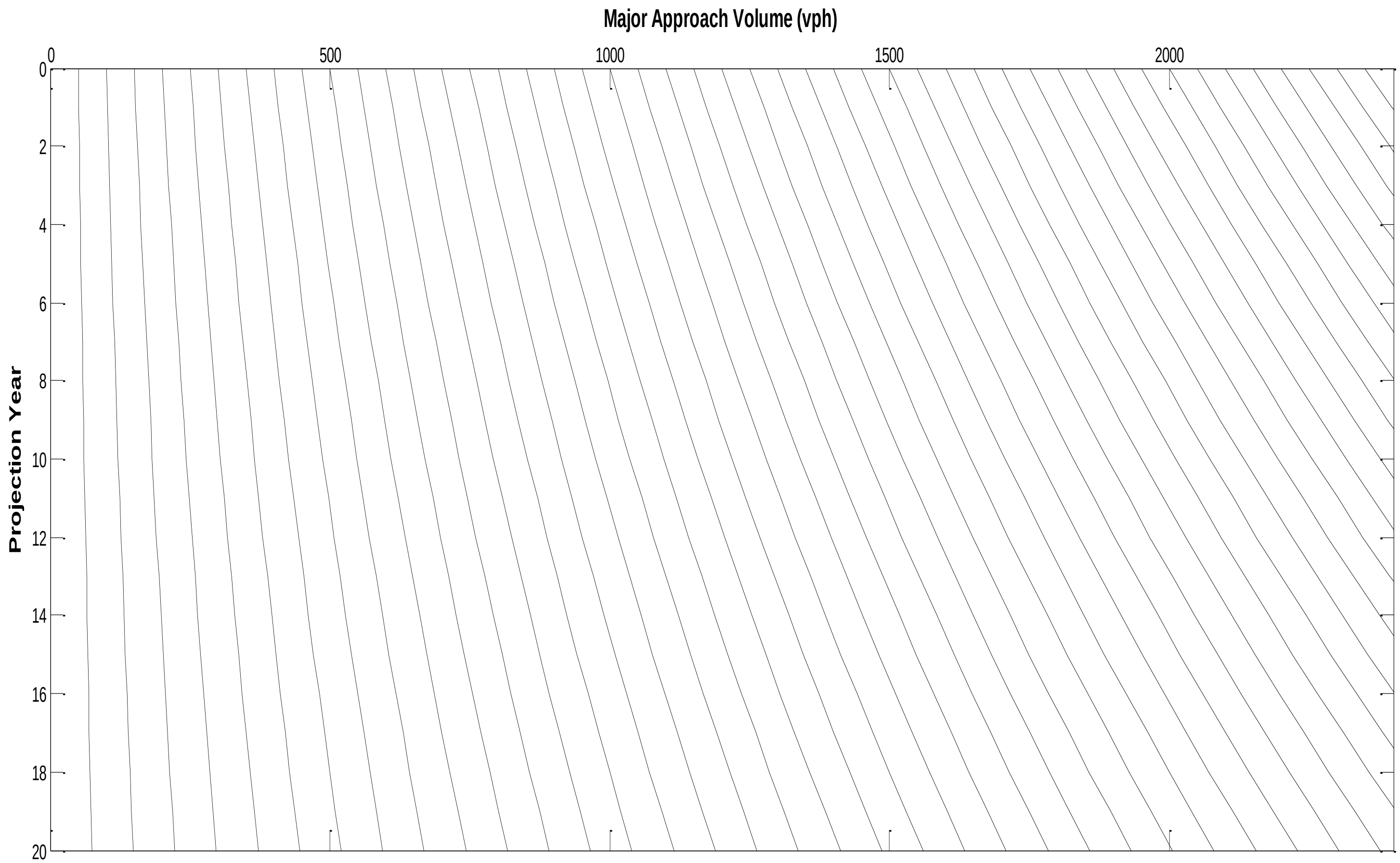


Figure B.164 Growth curves for growth rate of 2 %

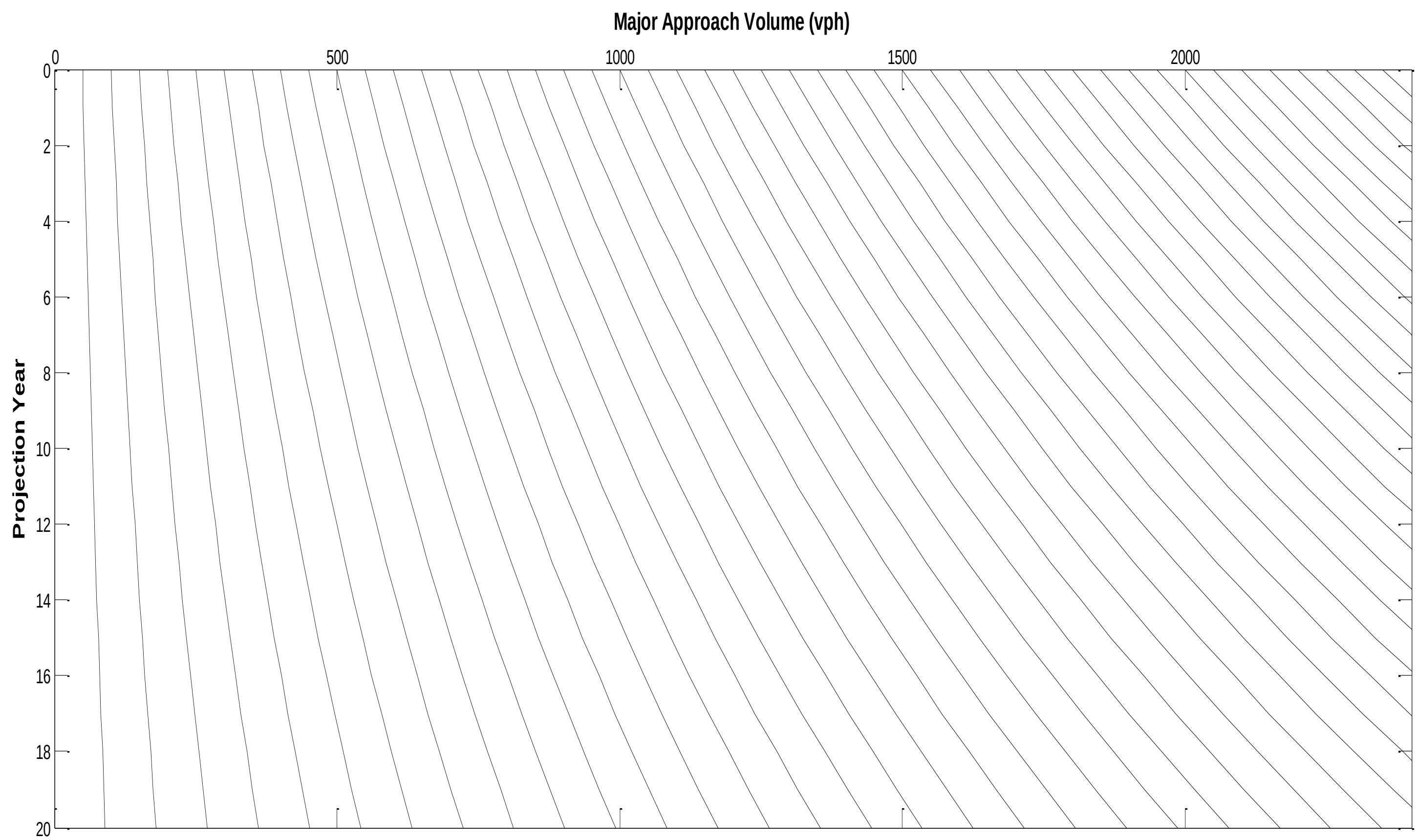


Figure B.165 Growth curves for growth rate of 3 %

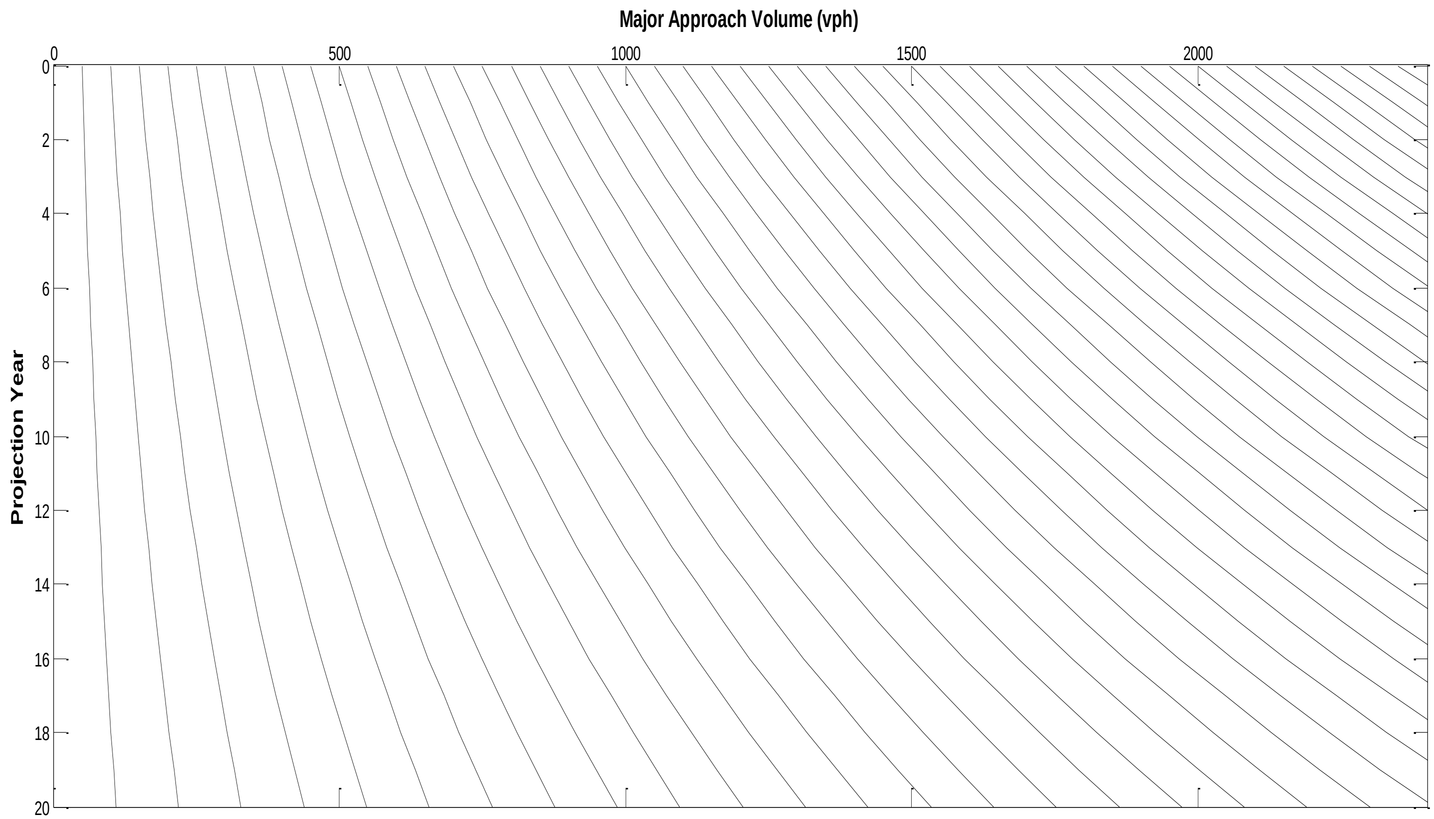


Figure B.166 Growth curves for growth rate of 4 %

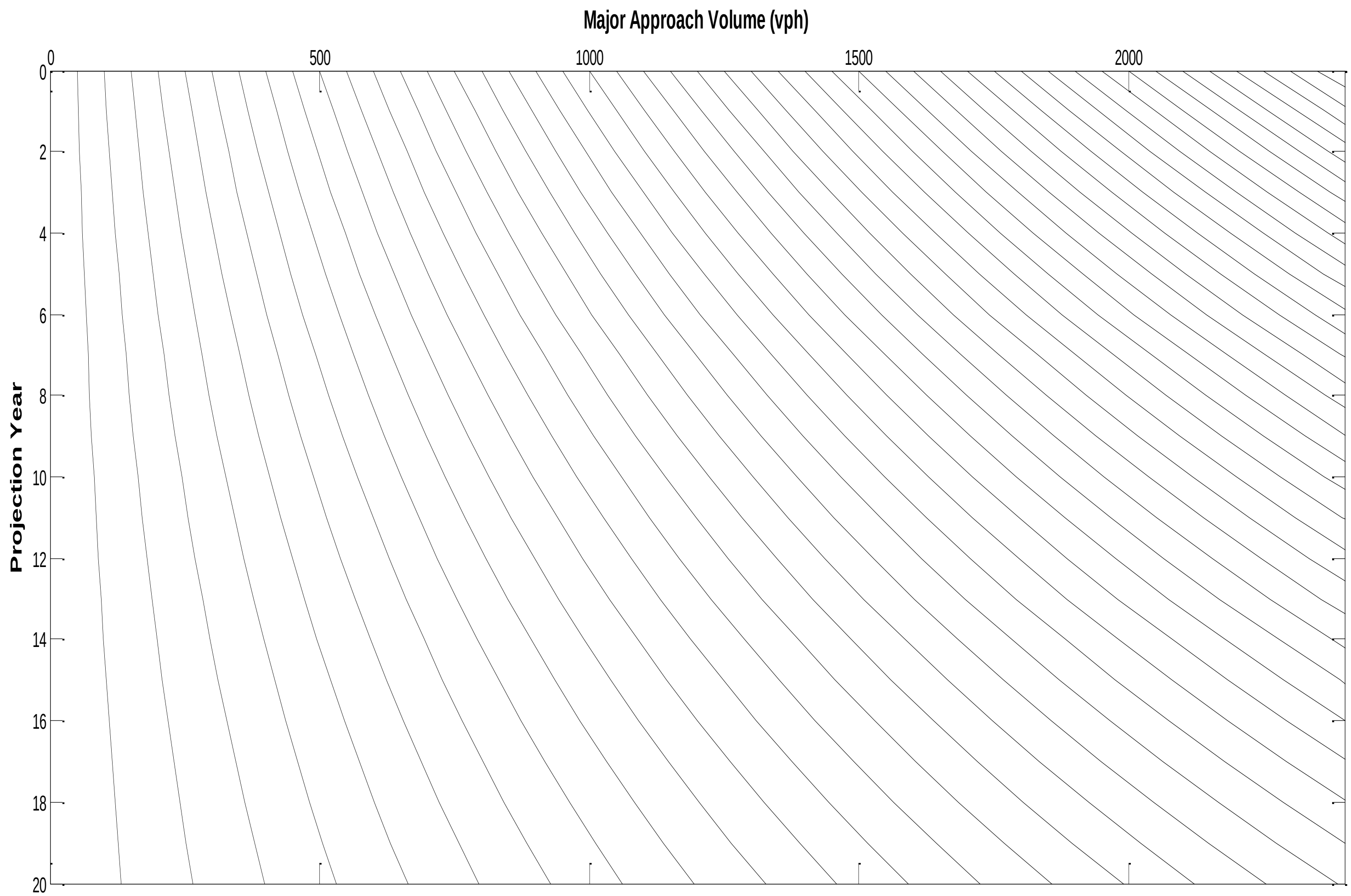


Figure B.167 Growth curves for growth rate of 5 %

USER'S MANUAL OF SILCC

A SUPPLEMENT

TO REPORT

"INVESTIGATING OPERATION AT GEOMETRICALLY UNCONVENTIONAL INTERSECTIONS"

INPUTS:

1. Please enter your values for each parameter in **WHITE CELLS WITH BLACK COLOURED FONTS ONLY** as shown in Figure 1.

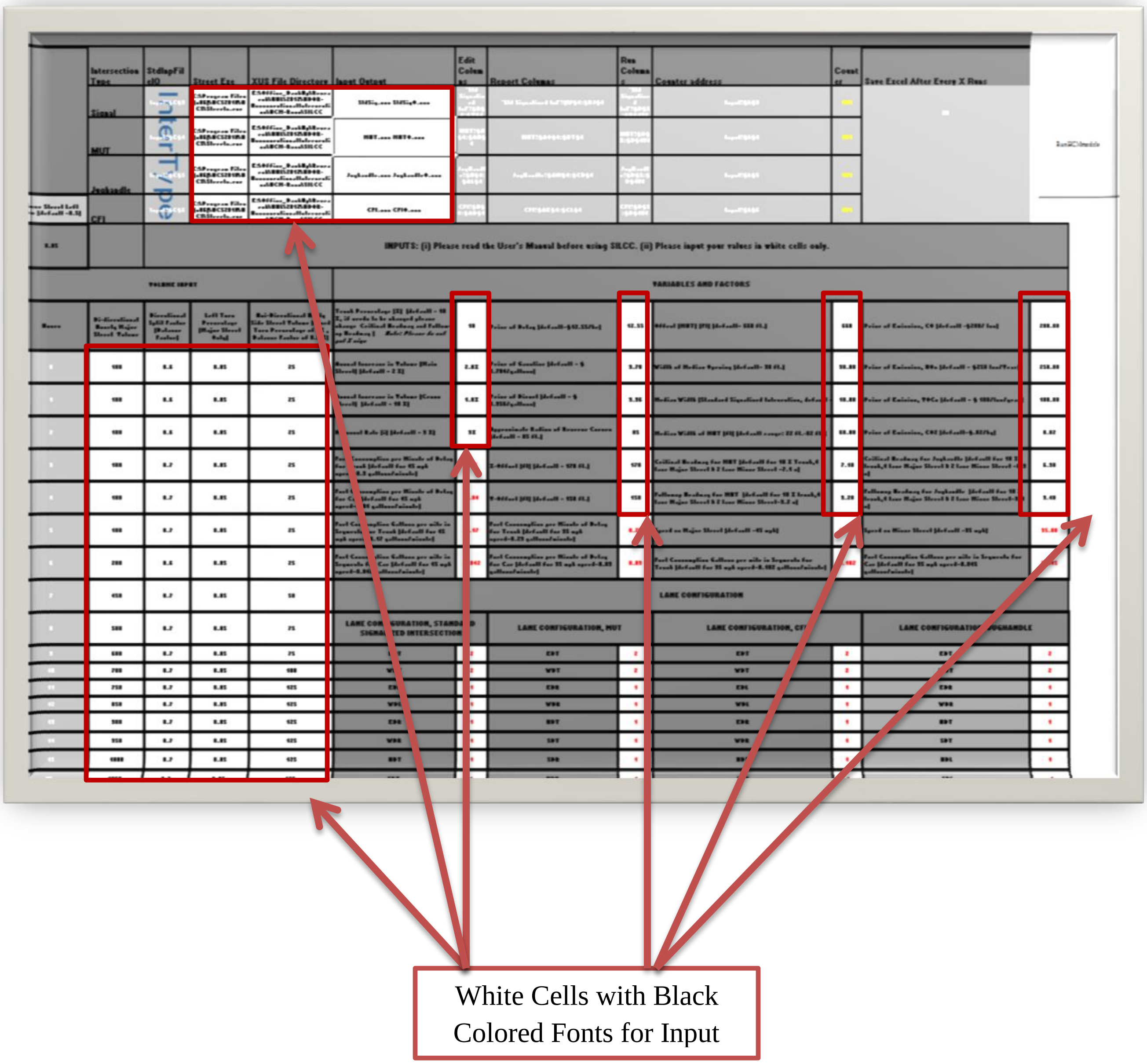


Figure 1 Input

A. HIGHWAY CAPACITY SOFTWARE (HCS) BATCH PROCESSING RELATED INPUTS

- The input under “Street. Exe.” is the path of HCS program files. Users are required to provide the path of program files of HCS.
- The input under “XUS File Directory” is the path of location of HCS files. Users are required to provide the path of the location of HCS files for four study intersections.
- The input under “Input Output” is the name of input and output files. Users are recommended to keep the same name of HCS files as provided as default in input. If they want to change the name of files, corresponding change has to be made on “Input Output”.
- Users are not allowed change other cells except mentioned above.

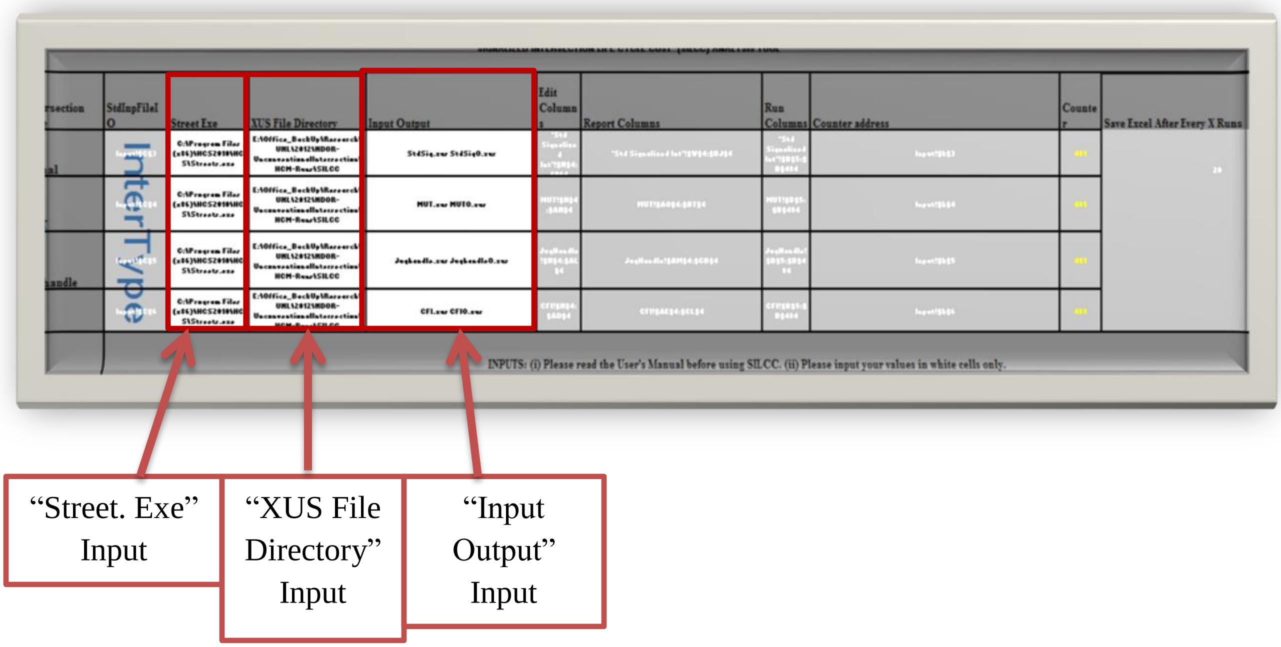


Figure 2 HCS Batch processing related input in the input sheet

B. VOLUME INPUTS

- The input volumes should be hourly volumes for 24 hr period.
- For major street volumes, please input bi-directional volume, balance factor and left turn percentage in the respective cells.
- For minor street volumes, please input uni-directional volume only. The software will automatically apply balance factor (0.5) and left turn percentage (5%) itself.

C. VARIABLES AND FACTORS

- If users want to change the input value of truck percentage, corresponding changes need to be made on the input values of critical headway and follow up headway for MUT and Jughandle. The procedure for the calculation of critical headway and follow-up headway is provided in the report in Chapter 4. This can also be referenced from Chapter 19 of HCM 2010. **Truck percentage should not be entered with % sign.**
- Default price information for delay, fuel consumption and emissions are based on literatures, websites and rate analysis. Users are flexible to change according to the local prices.
- Geometry related factors are based on different literatures. Users are flexible to change them according to requirement.
- Default discount and annual increment in traffic can also be changed according to the requirement.

D. CONSTRUCTION RELATED

- Unit Prices of all the construction related items except Land Price are taken from Average Unit Price Summaries from NDOR website for Item History and Info. Users are flexible to change them according to requirement.

- PE Cost and Contingency are also based on literatures. Users are flexible to change them according to requirement.
- Users can choose the treatment type whether it is **RETROFITS** or **NEW CONSTRUCTION**. Choose **0** for **NEW CONSTRUCTION** and **1** for **RETROFITS**.

E. VALUES IN CELLS WITH RED COLORED FONTS

It is not recommended to change the values in red cells unless it is required to change the speed and lane configurations. In case of need, users can contact developer for assistance.

Speeds: Default speeds on major and minor streets represent rural roads. If users want to change the speed, corresponding changes need to be made on HCS file as well. The speed will

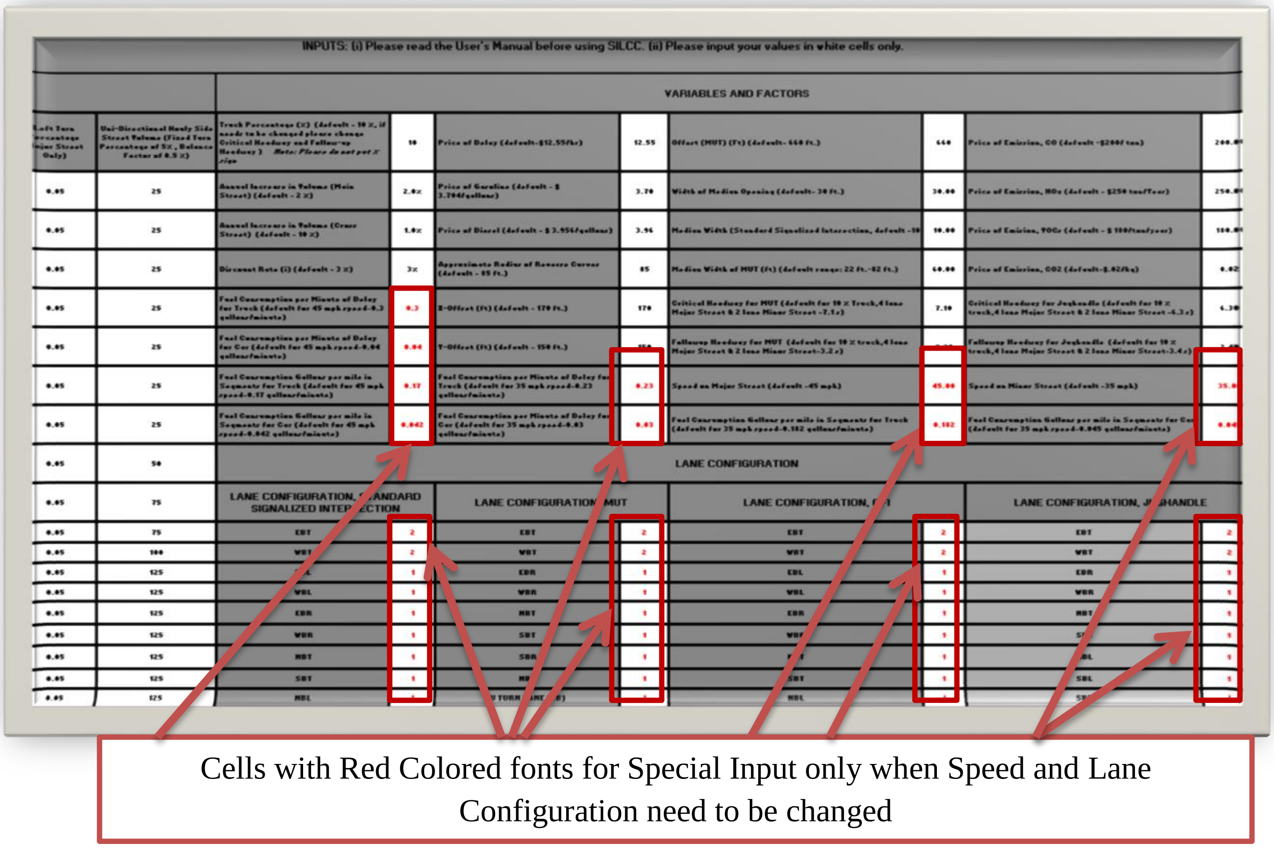


Figure 3 Special input at cells with red colored fonts

change the Passage Time. Therefore, if speed is changed, it is also needed to recalculate the Passage Time using the equation provided in Appendix A of the report and input it on HCS files.

Fuel Consumption Factors: The default fuel consumption factors are related to the default speeds of major street as 45 mph and minor street as 35 mph. It is necessary to change these values according to the speed by referencing Table 4.3 and Table 4.4 of the report.

Lane Configuration: The default lane configurations are provided for intersections with 4 lanes major street and 2 lanes minor street. If users need to change lane configuration, corresponding changes need to be made on HCS files as well.

2. After the input of all the values, please press the Button "RUNHCSMODELS" in the upper right corner of input sheet as shown in the following figure.

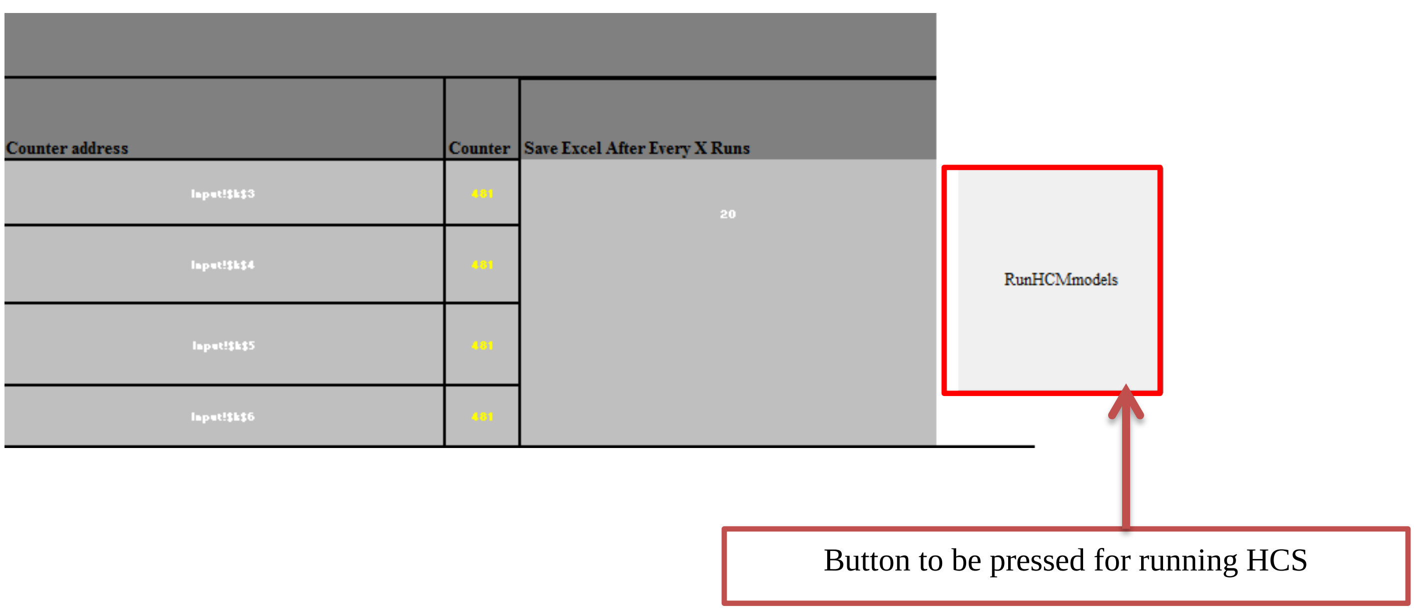


Figure 4 Button to run HCS

OPERATIONAL ANALYSIS SHEETS:

There are four different sheets for operational analysis of intersections:

(a) Standard Signalized intersection

(b) Median U-Turn (MUT)

(c) Continuous Flow Intersection (CFI)

(d) Jughandle

These sheets consist of tables for delay, fuel consumption and emission estimation. A summary for annual amount is shown in a right most table in each sheet as shown in the figure below. **These sheets will operate automatically. Users are not allowed to change anything on these sheets.** However, users can look at the information on these sheets.

Amount in 24 hrs period (day)						
elay	Petrol	Diesel	CO (ton)	Nox (ton)	VOCs (ton)	CO_2 (kg)
.3922	76.9306	91.9074	0.0118	0.0022962	0.00273517	3.40918

1

Yearly Amount							
Years	Delay	Petrol	Diesel	CO (ton)	Nox (ton)	VOCs	CO_2
Year 1	16568.2	28079.7	33546.2	4.30765	0.83811	0.99834	1244.3
Year 2	16911.6	28680	34265	4.39985	0.85605	1.01971	1270.9
Year 3	17286	29335.4	35050	4.50054	0.87564	1.04304	1300.0
Year 4	17699.4	30060.7	35918.7	4.61196	0.89732	1.06887	1332.2
Year 5	18225.3	30955.6	36988.2	4.74927	0.92404	1.10069	1371.9
Year 6	18655.3	31704.4	37884.6	4.86427	0.94641	1.12734	1405.1
Year 7	19095.1	32472.5	38804.2	4.98224	0.96936	1.15468	1439.2
Year 8	19502.8	33184.7	39657	5.09163	0.99065	1.18003	1470.8
Year 9	20039.3	34106.6	40759.6	5.23315	1.01818	1.21283	1511.7
Year 10	20502.9	34911.3	41722.7	5.35672	1.04222	1.24147	1547.4
Year 11	21000.1	35779	42761.5	5.48998	1.06815	1.27236	1585.9
Year 12	21465.7	36589	43731.1	5.61437	1.09235	1.30118	1621.8
Year 13	22049.9	37586.9	44924	5.76751	1.12215	1.33668	1666.1
Year 14	22537.6	38440	45945.6	5.89855	1.14764	1.36705	1703.9
Year 15	23146.3	39485	47195.3	6.05895	1.17885	1.40422	1750.3
Year 16	23679.7	40406.2	48297.3	6.20037	1.20637	1.437	1791.1
Year 17	24262.5	41412.2	49500.9	6.35483	1.23642	1.47279	1835.7
Year 18	24868.2	42461.2	50756.2	6.5159	1.26776	1.51012	1882.3
Year 19	25447	43461.8	51953.4	6.66952	1.29765	1.54573	1926.6
Year 20	26319.7	44933.3	53710.7	6.89522	1.34156	1.59803	1991.8

Figure 5 Yearly amount table in operational analysis sheet for standard signalized intersection

USER’S AND NON-USER’S COST SHEET:

This sheet operates automatically. Users are not allowed to make changes in this sheet. This sheet provides the information about marginal annual amount of delay, fuel and emissions with respect to standard signalized intersection and their monetization. Figure 5 depicts the screenshot of this sheet.

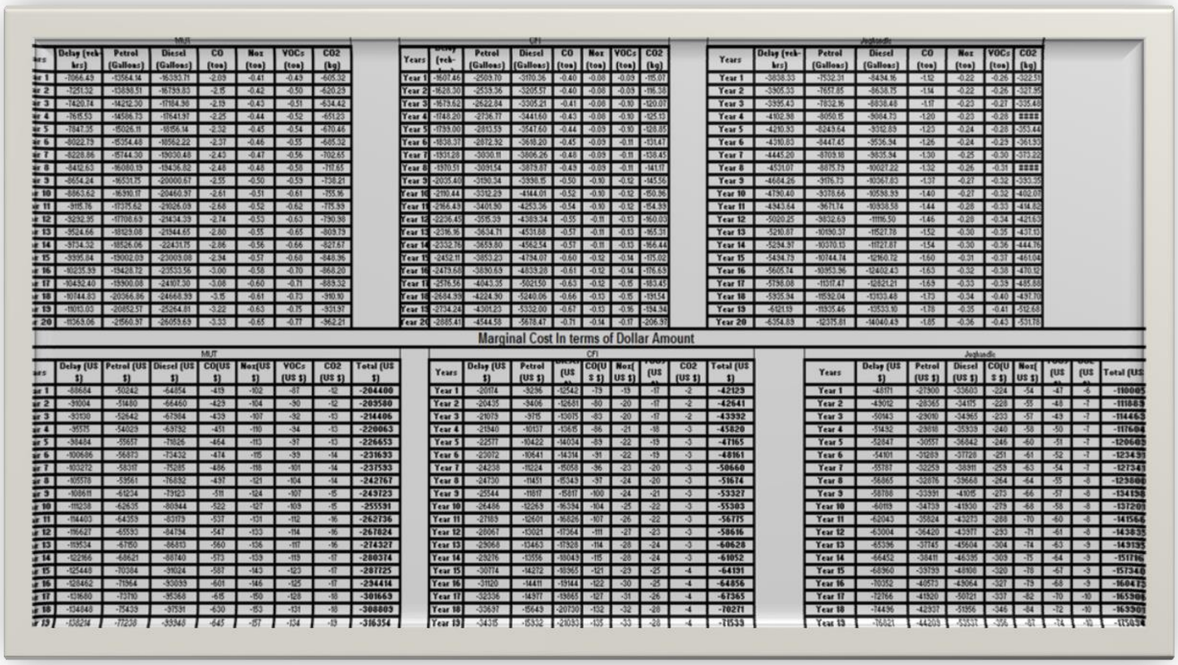


Figure 6 Screenshot of user’s and non-user’s cost sheet

CONSTRUCTION COST ESTIMATE SHEET:

There are two sheets each for (i) New Construction and (ii) Retrofits. Each sheet consists of estimate of marginal construction cost as well as operation and maintenance cost for (i) MUT (ii) CFI (iii) Jughandle. Following figures depict their screenshots. **These sheets operate automatically based on inputs. Users do not need to change these sheets**. However, if users feel to adjust some quantities, he may modify the calculation changing the excel formulas.

MUT RETROFIT

		Item		Qty	Unit	Rate	Amount in U Dollars
Construction Cost	1101	Concrete Pavement Removal		10778.10	SY	6.81	73398.86
	3075.6	Median Opening)		11910.30	SY	30.59	364336.08
	1010	Excavation (1.5')		5955.15	CY	3.26	19413.79
	1122	Remove Concrete Median Surfacing		2220.00	SY	5.90	13098.00
	A724.01	Relocate Traffic Signal		6.00	Each	470.00	2820.00
	A703.00	Relocate Mast Arm		4.00	Each	1470.00	5880.00
	A006.98	Vehicle Detector, Type TD-A Performed		24.00	Each	263.69	6328.56
	A001.7	Pull Box, Type PB-7		12.00	Each	1010.00	12120.00
	A070.22	4 Inch Conduit in Trench		160.00	LF	5.91	945.60
	A072.20	4 Inch Conduit in Roadway		432.00	LF	7.83	3382.56
	A077.26	16/C 14 AWG Traffic Signal Cable (lumpsump)		500.00	Lf	3.98	1990.00
	A079.01	2/C #14 AWG Detector Lead-In Cable (lumpsump)		250.00	LF	1.46	365.00
	7320.3	Traffic Sign and Post (STOP Sign, No RTOR and No Left Turn)		10.00	Each	295.85	2958.50
	7500.2	Right Arrow Performed Pavement Marking, Type 3		2.00	Each	420.00	840.00
	7500.3	Through and Left Arrow Performed Pavement Markings		6.00	Each	483.00	2898.00
	7495.6	5" White Permanent Pavement Markings		4000.00	LF	0.40	1600.00
	7496.1	5" Yellow Permanent Pavement Markings		8000.00	LF	0.19	1520.00
		Additional Land		2.30	Acre	4142.50	9509.87
		Subtotal					*523404.82*
Soft Cost		Preliminary Engineering (PE)	10% of Subtotal				52340.48
		Contingency	20% of Subtotal				104680.96
		Total Cost in US Dollar					*680426.26*
al O & M Cost		Landscaping of median	Lumpsump	2	Per year	1,000	2000.00

CFI RETROFIT

		Item		Qty	Unit	Rate	Amount in U Dollars
Construction Cost	1122	Remove Concrete Median Surfacing (6' Median)		2109	SY	5.90	12443.10
	3075.6	10" Doweled Concrete Pavement, Class 47 B-3500	LT- Lane + RT-Lane	3196.8	SY	30.59	97790.11
	1010	Excavation (1.5')		1598.4	CY	3.26	5210.78
	3017.4	Concrete Class 47B-3000 Median Surfacing		2386.5	SY	31.05	74100.83
	A016.8	Mast Arm Signal Pole, Type MP-60		8	Each	8500.00	68000.00
	A504.81	Install Mast Arm Signal Pole		8	Each	3700.00	29600.00
	A003.10	Traffic Signal, Type TS-1		10	Each	493.89	4938.90
	A501.00	Install Traffic Signal, Type TS-1		10	Each	242.00	2420.00
	A001.7	Pull Box, Type PB-7		12	Each	1010.00	12120.00
	A070.22	4 Inch Conduit in Trench (lumpsump)		300	LF	5.91	1773.00
	A072.20	4 Inch Conduit in Roadway		432	LF	7.83	3382.56
	A077.26	16/C 14 AWG Traffic Signal Cable (lumpsump)		1800	Lf	3.98	7164.00
	A006.98	Vehicle Detector, Type TD-A Performed		40	Each	263.69	10547.60
	A079.01	2/C #14 AWG Detector Lead-In Cable (lumpsump)		500	LF	1.46	730.00
	7320.3	Traffic Sign and Post (No Left Turn)		4	Each	295.85	1183.40
	7500.3	Left Arrow Performed Pavement Markings		4	Each	483.00	1932.00
	7495.6	5" White Permanent Pavement Markings		3800	LF	0.40	1520.00
	7496.1	5" Yellow Permanent Pavement Markings		5600	LF	0.19	1064.00
		Additional Land		0.58	Acre	4142.50	2386.98
		Subtotal					*338307.26*

Figure 7 Screenshot of construction cost estimate sheet for retrofit

MUT

		Item		Qty	Unit	Rate	Amount in US Dollars
...n Cost	3075.6	(Including Median Opening)		399.6	ST	30.59	12223.76
	1010	Excavation (1.5')		199.8	CY	3.26	651.35
	A070.22	4 Inch Conduit in Trench		160	LF	5.91	945.60
	A077.26	16/C 14 AWG Traffic Signal Cable (lumpsump)		500	LF	3.98	1990.00
	7320.3	Traffic Sign and Post (STOP Sign, No RTOR and No Left Turn)		10	Each	295.85	2958.50
		Additional Land		2.296	Acre	4142.50	9509.87
		Subtotal					28279.08
Cost		Preliminary Engineering (PE)	10% of Subtotal				2827.91
		Contingency	20% of Subtotal				5655.82
		Total Cost in US Dollar					36762.81
...O & M Cost		Landscaping of median	Lumpsump	2	Per year	1,000	2000.00

CFI

		Item		Qty	Unit	Rate	Amount in US Dollars
Construction Cost	3075.6	10" Doweled Concrete Pavement, Class 47 B-3500	LT-Lane + RT-Lane	2064.60	ST	30.59	63156.11
	1010	Excavation (1.5')		1032.30	CY	3.26	3365.30
	3017.4	Concrete Class 47B-3000 Median Surfacing		0	ST	31.05	0.00
	A016.8	Mast Arm Signal Pole, Type MP-60		8	Each	8500.00	68000.00
	A504.81	Install Mast Arm Signal Pole		8	Each	3700.00	29600.00
	A003.10	Traffic Signal, Type TS-1		10	Each	493.89	4938.90
	A501.00	Install Traffic Signal, Type TS-1		10	Each	242.00	2420.00
	A001.7	Pull Box, Type PB-7		12	Each	1010.00	12120.00
	A070.22	4 Inch Conduit in Trench (lumpsump)		300	LF	5.91	1773.00
	A072.10	4 Inch Conduit in Roadway		432	LF	7.83	3382.56
	A077.26	16/C 14 AWG Traffic Signal Cable (lumpsump)		1800	LF	3.98	7164.00
	A007.08	Vehicle Detector, Type TD-A Performed		40	Each	263.69	10547.60
	A079.1	2/C #14 AWG Detector Lead-In Cable (lumpsump)		500	LF	1.46	730.00
	7320.3	Traffic Sign and Post (No RTOR and NO Left Turn)		6	Each	295.85	1775.10
	7500.3	Left Arrow Performed Pavement Markings		6	Each	483.00	2898.00
	7496.1	5" Yellow Permanent Pavement Markings		2800	LF	0.19	532.00
		Additional Land		0.58	Acre	4142.50	2386.98
		Subtotal					*214789.55*
...Cost		Preliminary Engineering (PE)	10% of Subtotal				21478.95
		Contingency	20% of Subtotal				42957.91
		Total Cost in US Dollar					*279226.41*
Annual O & M		Additional Signal O and M (signal head, detectors, signal retiming, power supply, etc)		4	Per year	6,000	24000.00

JUGHANDLE

		Item		Qty	Unit		Amount in US Do...
Construction C...	1010	Excavation (1.5')		652.68	CY	3.26	2127.74
	3075.6	10" Doweled Concrete Pavement, Class 47 B-3500	Ramp	1305.36	ST	30.59	39930.96
	7320.3	Traffic Sign and Post (Yield Sign, No RTOR and No Left Turn)		8.00	Each	295.85	2366.80
	7495.6	5" White Permanent Pavement Markings		340.00	LF	0.40	136.00
	7496.1	5" Yellow Permanent Pavement Markings		1280.00	LF	0.19	243.20
		Additional Land		1.17	Acre	4142.50	4850.03

Figure 8 Screenshot of construction cost estimate sheet for new construction

LCC SHEET:

It is the output Sheet of SILCC which provides the results of Life Cycle Cost Analysis of intersections in the form of Net Present Value (NPV) of benefits and Benefit to Cost Ratio (B/C). This sheet also provides the total serving year of intersections till their life cycle period of 20 years without failure. These values are provided for all three intersections (i) MUT (ii) CFI and (ii) Jughandle. The screenshot of this sheet is depicted in the following figure. **This sheet operates automatically and users are not allowed to modify on this sheet**.

MUT	CFI	JUGHANDLE	
3%	3%	3%	
20	20	20	A
20	20	20	B
MUT NPV Value	CFI NPV Value	JUGHANDLE NPV Value	
36762.81	279226.41	64551.15	
29754.9	357059.40	29754.95	
3786813.43	817236.77	2046578.67	
3720295.68	180950.96	1952272.57	C
56.93	1.28	21.70	D

A: Design Service period of Intersections = 20 yrs

B: Actual Service period without failure

C: Net Present Value of Total

D: B/C ratio of each intersection

Note: Users are not allowed to modify any values in this table.

Figure 9 LCC sheet (Output Sheet)