UPSTREAM SIGNALIZED CROSSOVER INTERSECTION

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ABSTRACT

The impact of left turns on operation is probably the most significant factor in the performance of conventional intersections. As a result engineers have looked to alternative measures for dealing with left turns at intersections to improve performance, some of which have been unconventional schemes. The purpose of this thesis is to introduce a new unconventional intersection scheme, the Upstream Signalized Crossover (USC), which is a four-legged intersection designed to eliminate left turn opposing conflicts by crossing the left and through traffic to the left side of the road at all four approaches before the intersection. The crisscrossing of traffic upstream of the intersection results in four additional secondary signalized intersections. The operation of the USC intersection was analyzed, along with a typical conventional intersection, using the *Highway Capacity Manual* (HCM) *2000* methodology and simulation models that were developed using *VISSIM*. The HCM analysis employed the Syncro software to compute the signal timings and offsets for the multiple signals of the USC. The results from this analysis indicated that a significant reduction in average delays could be achieved by the USC intersection when directly compared to conventional intersections. The average delays computed for the USC was about 45 % to 60% less for the same traffic volumes that caused a sample conventional intersection to fail and was able to accommodate between 15% and 20% more traffic before the USC reached failure. The VISSIM simulation model revealed that the USC was not as sensitive to left turn volumes as compared to conventional intersections and has the potential for handling large left turn volumes while maintaining acceptable levels of performance for through traffic. However, when compared to a conventional intersection the analysis did not show significant improvements in left turn delays for the USC, which may indicate that progression between the primary and secondary signals for the left turn movements were not as favourable. Future analysis of the USC intersection should develop and utilize a more dynamic technique for optimizing the signals for both the through and left turn movements. The USC has 50% less crossing conflicts than a typical four-legged conventional intersection and as a result could offer a significant reduction in left turn opposing collisions. The potential for rear-end collisions could be higher for the USC given the additional signals; however, this should be mitigated by improving the coordination and progression between the signals.

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CHAPTER 1: INTRODUCTION

The major arterial roads of urban cities are congested with little immediate hope of relief. While traffic demand management strategies offer long-term hope for cities with increasing trends in traffic growth, there is little promise in the short-term. In an attempt to relieve congestion at intersections, transportation engineers have done what they can in optimizing signal timings and coordination, implementing turning lanes, and other conventional measures. Short of building overpasses or converting intersections to interchanges, conventional intersections will eventually fail, as they are limited in capacity and performance. The impact of left turns on operations is probably the most significant factor in assessing the level of performance of conventional intersections.

To improve the performance of intersections, an unconventional intersection scheme is proposed, which eliminates the left-turn-opposing conflict that conventional intersections are subject to. The new Upstream Signalized Crossover (USC) intersection is designed to switch traffic movements approaching an intersection to the opposite side of the road such that left turns can be made directly without opposing vehicle conflicts. This redirecting scheme involves four additional signalized intersections upstream of the main intersection, and by adequate phasing and coordination of the signals, the USC intersection can reduce delays and significantly improve intersection operation.

This thesis has ten chapters. **Chapter Two** will discuss conventional intersections, describing some of the key elements and characteristics, and provides an overview of a commonly used methodology for analysing and evaluating the operation of conventional intersections. Intersections create conflict points on roads that will depend on the number and direction of the approaches, number of lanes, signal control, traffic volumes and the percentage of right or left turns. Intersections are signalized as a means of improving safety and regulating the movements of the conflicting flows of traffic. Left-turning vehicles are often the controlling element of the operation of signalized intersections and will dictate the number, type and sequence of signal phases. Common measures by which signalized intersections are evaluated include capacity, level of service (LOS), delay and queue length. In determining these measures, the analysis must consider a wide variety of prevailing conditions, including the amount and distribution of traffic movements, traffic composition, geometric conditions and signalization conditions.

Chapter Three presents alternative schemes for dealing with left turns at intersections, which include both traditional applications and unconventional schemes. The impact of left turns on operations is probably the most significant factor considered in analyzing conventional signalized intersections and is often the principal factor in degraded performance. When protected left-turn phases are used, the time dedicated for the left turns is not available for the through movements, which often results in increased delays for through movements. In addition to the effect on intersection productivity, left-turn activity generally has an impact on accident experience. Unconventional measures for treating left turns have been developed in the past and offer an alternative to expanding the size of existing conventional intersections or upgrading intersections to interchanges. The focus of these alternative measures is to reduce the conflict points associated with left turns and maximize the throughput of vehicles. The unconventional schemes that will be discussed include the Median U-turn, Bowtie, Super-Street and the XDL intersection.

The focus of this thesis is to introduce this new unconventional scheme conceived by the author, which has been given the name "Upstream Signalized Crossover" (USC) intersection. **Chapter Four** describes the functional and operational characteristics of this new intersection concept, including a description of the signal phasing and sequencing. In the new USC intersection scheme both thru and left turn traffic cross the median to the left side of the road at a location upstream of the main intersection, while right turn traffic is maintained on the right side. When this is applied to all four directions of a four-legged intersection, the result is that left turns can be made directly without opposing conflict. The crossing over of vehicles on both sides of the primary intersection necessitates four secondary intersections upstream of the primary intersection. By eliminating turning conflicts, all five intersections could operate on a two-phase cycle and coordination between the primary and secondary signals would be feasible given the simple two-phase operation. A well-timed and efficient phasing plan would streamline the progression of traffic through the USC intersection.

In **Chapter Five**, the USC intersection is analyzed using the same analysis method described in Chapter two. In this chapter the performance of both the conventional and USC intersection schemes is analyzed over a range of traffic demand and a comparison is made. Traffic analysis software was used to analyze the USC intersection because of the convenience in obtaining quick results and also for its capabilities in modelling coordinated signals. The total delays experienced by vehicles driving through the entire USC intersection were estimated by combining and summing the individual delays at each of the secondary and primary intersections. The delays were calculated over a range of traffic volumes. When compared to the conventional intersection, the USC intersection had a higher threshold for handling traffic. A separate analysis was done for left turns to investigate the sensitivity of the USC intersection to left turn volumes. Three left turn scenarios were analyzed, which included the conventional-single left, conventional-dual left and the USC left turn. The results showed that the left turns of the USC operated more efficiently at the low to moderate volume range.

Chapter Six presents the results of another comparison in operation using a simulation program, VISSIM, which was used to simulate the operation of the USC intersection and estimate the delays. This program is more flexible in that it can model unconventional movements, such as those under the USC scheme, but apply the same principals used in traditional traffic operation modelling. Compared to conventional intersections the USC shows less sensitivity to the magnitude of left turn volumes and shows the potential for accommodating larger turning volumes. Despite the additional intersections required and the crisscrossing manoeuvres associated with the USC, the analysis performed seems to indicate that the operational performance of through vehicles would not be made any worse than the conditions experienced by conventional intersections. For delays experienced by left turn vehicles, the analysis did not show significant improvements for the USC when compared to the conventional intersections. The problem may lie in the progression of left turn vehicles after the turn has been made.

Chapter Seven discusses key safety concerns of the USC intersection. Safety is an important consideration when designing and implementing intersections. Although the USC intersection would be considered unconventional overall, some features and operating characteristics would still be considered conventional and the same safety measures and principals can be applied. The expected safety performance of the USC intersection could be assessed by examining collision experiences of conventional intersections and relating them to the USC. Significant improvements in safety could be expected due to the elimination of left turns at the primary intersection, although the increased frequency of stops expected for the USC may increase the potential for rear-end collisions. For those features that are truly unique to this new scheme the safety risks and mitigating measures to improve safety would need to rely on engineering judgement.

Chapter Eight describes the implementation and design issues surrounding the new scheme. The USC intersection would be best suited for intersections with heavy left turn traffic volumes in all directions given its symmetric operation. An ideal location for this unconventional intersection would be at the crossing of major urban arterials with low to moderate posted speed requirements. Compared to a conventional intersection, the overall area of the USC intersection would be slightly greater. The location of the secondary intersections relative to the primary intersection will effect the progression and throughput of the intersection. The optimum spacing of the secondary intersections could be achieved and would depend on the desired or optimum cycle length for the anticipated traffic volumes and the average speed that vehicles travel between secondary intersections. Given the unconventional left turn at the primary intersection, it would be necessary to create a design that will minimize driver error. The key element in the geometric design of the USC intersection is the geometry at the secondary intersections, at which the crossing movements should be made as smooth as possible to minimize driver hesitation and confusion.

Finally, **Chapter Nine** summarizes the conclusions made in each chapter and provides final remarks regarding this thesis.

CHAPTER 2: CONVENTIONAL INTERSECTIONS

2.1 INTRODUCTION

The efficiency of a road network greatly depends on the design and operation of intersections. An intersection can be defined as the area where two or more roads cross at-grade and include roadways and roadside facilities for vehicular, bicycle and pedestrian traffic. Intersections appear in various sizes and configuration from simple tee- intersections to complex multi-legged intersections. The size of intersections generally increases with traffic demand.

The design of intersections involves four elements according to the Transportation Association of Canada (TAC) *Geometric Design Guide for Canadian Roads*[1], which are:

- Traffic factors (e.g. safety, volumes and control devices);
- Physical factors (e.g. road classification, lanes and grades);
- Human factors (e.g. driver expectations and reaction time); and
- Economic factors (e.g. land cost, construction and maintenance costs).

The method of traffic control used at intersections will depend on factors such as the size and configuration of the intersection, traffic demand and the desired level of service. Generally, simple and low volume intersections will utilize stop signs, while intersections with higher traffic volumes and complex manoeuvres will warrant the installation of traffic signals. **Figure 2.1** shows a typical 4-legged signalized intersection with right turn lanes.

2.2 CONFLICTS

Intersections create conflict points on roads due to the crossing and turning manoeuvres of vehicles. A traffic conflict occurs when the paths of vehicles cross, merge or diverge. Conflicts can also occur between vehicles, cyclists and pedestrians. The number of conflicts at an intersection will depend on the number and direction of the approaches, number of lanes, signal control, traffic volumes and the percentage of right and left turns. **Figure 2.2** illustrates the 32 possible vehicle conflict points for a four-legged intersection.

Figure 2.1: Typical Intersection Layout

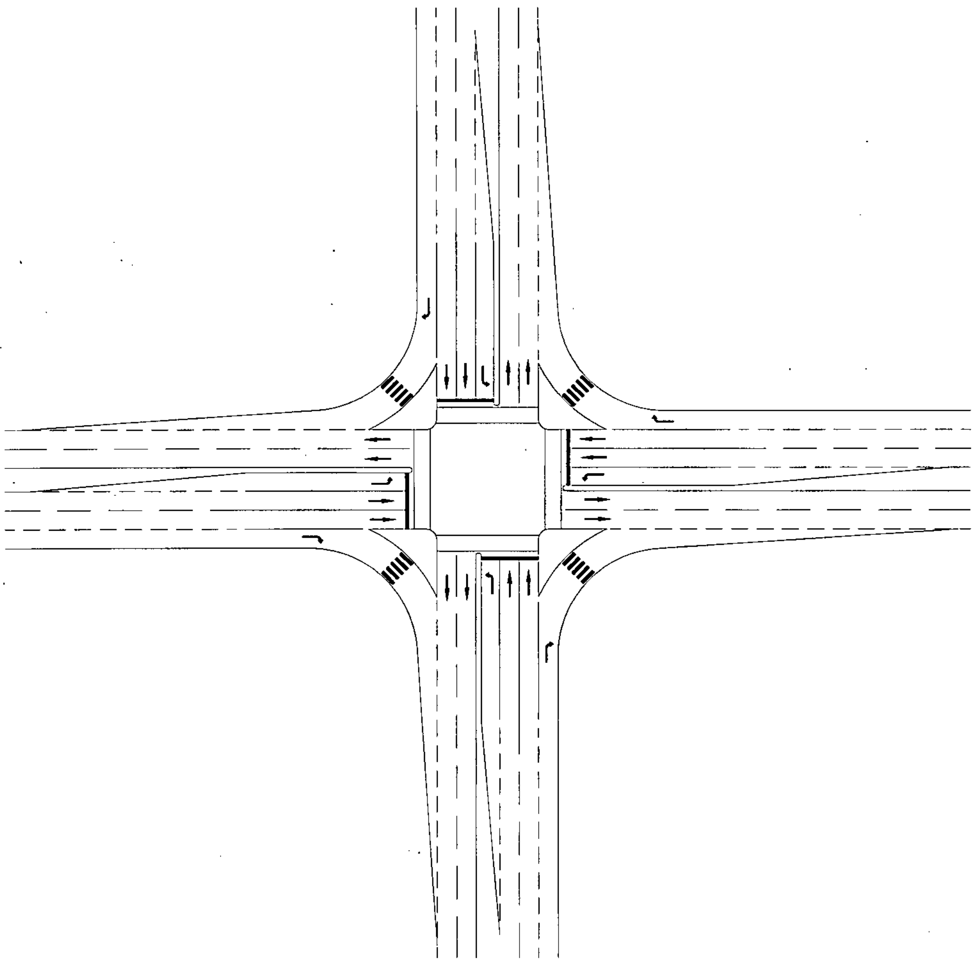
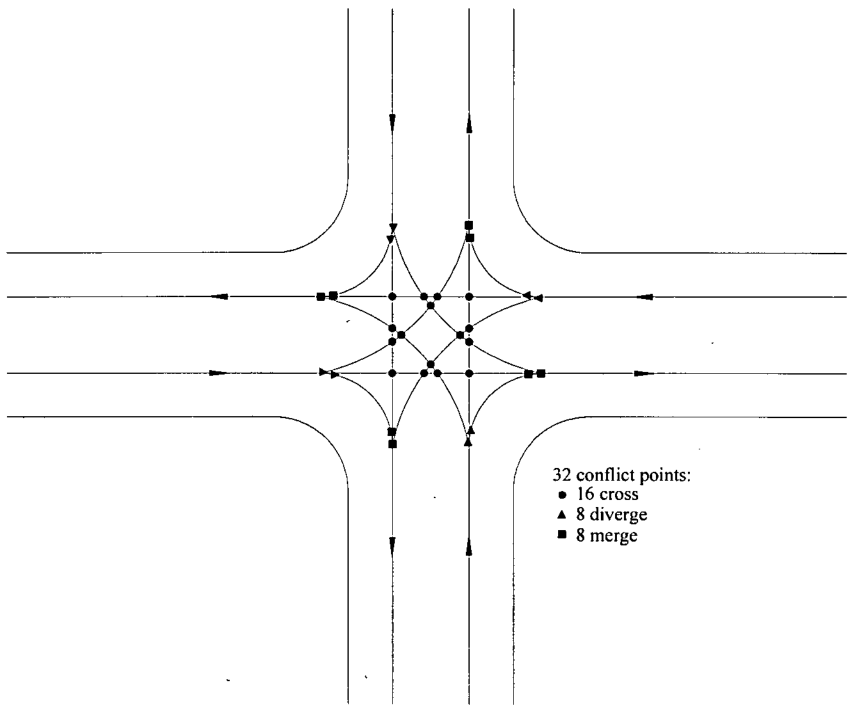


Figure 2.2 : Intersection Conflicts



2.3 SIGNALIZATION

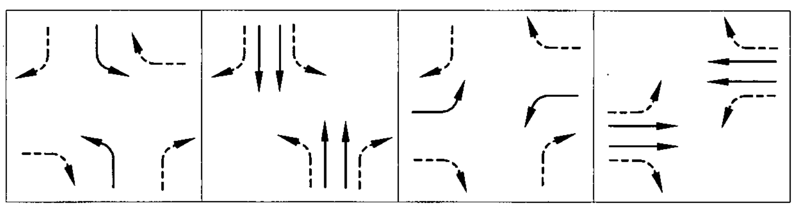
There are a number of reasons for signalizing an intersection, most of which relate to the safety and effective movement of conflicting flows through the intersection. There are also special warrants or formal justifications used by road authorities for signalizing an intersection. Some of the commonly used terms for describing traffic signal operation are defined below:

- *cycle*: one complete sequence of signal indications;
- *cycle length*: total time for the signal to complete one cycle;
- *phase*: part of a cycle allocated to any combination of traffic movements receiving the right of way simultaneously during one or more intervals;
- *interval*: period of time during which all signal indications remain constant;
- *green time*: time within a given phase during which the "green" indication is shown.

The basic principle of traffic signals is to regulate the approaching flows through the intersection while achieving some desired level of performance. Alternating the approach movements through separate phases eliminates crossing conflicts. However, a left turn conflict exists with the opposing through movement during each approach movement. If the left turn volume is minor, vehicles will wait for gaps in the opposing flow to turn left. This situation is commonly referred to as permitted left turns. If the left turn volume is significant and/or the opposing flow is large such that there are very few gaps for turning, the left turn movement is given its own separate phase, which is referred to as protected lefts. Left-turning vehicles are often the controlling element of the operation of signalized intersections and will dictate the number, type and sequence of signal phases.

For a given signalized intersection, each phase is allotted a specific green time, which is usually determined after optimizing all green times for the given cycle. **Figure 2.3** illustrates an example of a phasing scheme for a four-legged intersection. In this example protected lefts turn phases are provided for each approach followed by permitted lefts that run with through traffic.

Figure 2.3: Typical Phasing Scheme



PHASE 1 PHASE 2 PHASE 3 PHASE 4

2.4 OPERATIONAL ANALYSIS

Common measures by which signalized intersections are evaluated include capacity, level of service (LOS), delay and queue length. Each of these may be expressed as values that represent totals or averages for the entire intersection or for particular approaches or movements of intersection.

The methodology that is widely used in North America for analyzing signalized intersections is presented in the *Highway Capacity Manual 2000*[2]. This methodology addresses the capacity, LOS and other performance measures for lane groups and intersection approaches as well as the LOS for the intersection as a whole. In determining these measures, the analysis must consider a wide variety of prevailing conditions, including the amount and distribution of traffic movements, traffic composition, geometric conditions and signalization conditions.

Some of the key steps and features of the HCM methodology are discussed in this report to give the reader some background information. The reader is referred to the HCM manual for more detailed discussions and information.

2.4.1 INPUT PARAMETERS

The data required to conduct an operational analysis for signalized intersections fall into three main categories: geometric, traffic and signalization conditions. Geometric conditions include information on approach grades, the number and widths of lanes, storage length of exclusive turn lanes and parking conditions.

Traffic conditions include volumes of vehicles, pedestrians and cyclists, as well as vehicle type and distribution. An important traffic parameter that is used to analyze the performance of a signalized intersection is the arrival type. This parameter describes the quality of progression of a signalized intersection and will have a significant impact on the delay estimates and LOS. The HCM methodology uses six arrival types that range from type 1, which describes poor progression characteristics to type 6, which describes near-ideal progression characteristics.

For signal conditions, some of the information required includes phase and timing plans, cycle length, green times and change-and-clearance intervals. Other important features include the existence of actuated control and pedestrian-actuated phases.

2.4.2 LANE GROUPING AND DEMAND FLOW RATE

In analyzing the intersection, the methodology considers individual intersection approaches and individual lane groups within approaches. A lane group is defined as one or more lanes of an intersection approach serving one of more traffic movements. Examples of a lane group are exclusive left-turn and right-turn lanes. In the case of shared turn-through lanes, lane grouping will depend on the distribution of traffic volumes between the movements.

Demand flow rates are normally expressed as average hourly flow rates and are usually adjusted to depict the peak 15-minute period within the hour. Peak-hour factors (PHF) are used to adjust the rates to account for the peak 15-minute period within the peak hour. PHF's can be applied to each approach or each movement for the intersection.

2.4.3 SATURATION FLOW

The saturation flow represents the maximum flow in vehicles per hour that could be accommodated by the lane group that is assumed to have a continuous green phase. The methodology begins with an "ideal" or base saturation flow rate for each lane group, which is then adjusted using various adjustment factors. The factors used adjust for such things as lane width, turning volumes, heavy vehicles, grades and parking activity.

2.4.4 CAPACITY AND V/C RATIO

The results from the computations made from the previous sections of the methodology are used to compute capacity variables. The key variables include:

- Flow ratio for each lane group, *v/s*, which is the ratio of the demand flow rate to the saturation flow rate computed for the given lane group;
- Capacity, *c*, of each lane group, which is the saturation flow rate for the lane group multiplied by the effective green ratio for the lane group. The effective green ratio is the ratio between the effective green time and the cycle length;
- Volume to capacity ratio *v/c*, for each lane group; and
- Critical *v/c* ratio for the overall intersection, which indicates the proportion of available capacity that is being utilized by the critical lane groups. A critical lane groups are those that have the highest *v/s* ratio for a given signal phase.

A critical v/c ratio greater than 1.0 usually means that one or more of the critical lane groups will be oversaturated, which is an indication that the signal phases and timing are inadequate for the given traffic demand.

2.4.5 CONTROL DELAY

The principal measure used to describe the performance of signalized intersections is the average delay of vehicles passing through the intersection. The HCM method uses control delay, which is the portion of the total delay attributed to traffic signal operation, as the key performance measure. Control delay includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. The average control delay per vehicle for a lane group is computed using the following equation:

$$d = d_1(PF) + d_2 + d_3 \tag{2.1}$$

where

d = control delay per vehicle per lane group (s/veh);

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

PF = uniform delay progression adjustment factor;

d_2 = incremental delay for the effect of random arrivals and oversaturated queues;

d_3 = initial queue delay

The progression adjustment factor (PF) is applied to the uniform delay (d_1) to account for effects of signal progression and is applied to coordinated lane groups. A high proportion of vehicles arriving during the green phase indicate good signal progression, while vehicles arriving during the red light phase characterize poor progression.

The delay for each lane group is aggregated to provide a delay estimate for an approach and also for the intersection as a whole. This is done by computing weighted averages for the lane group delays, where by the delays are weighted by flows in the lane groups. The flows are computed by dividing the design flows by the PHF. Consequently, the control delay for an approach can be calculated using the following equation:

$$d_A = \frac{\sum d_i v_i}{\sum v_i} \tag{2.2}$$

where

d_A = delay for Approach A (s/veh);

d_i = delay for lane group i (on Approach A) (s/veh); and

v_i = adjusted flow for lane group i (veh/h).

Subsequently, the average control delay for the intersection can be computed by further aggregating the approach delays computed from Equation 2.2. This delay is calculated using the following equation:

$$d_I = \frac{\sum d_A v_A}{\sum v_A} \tag{2.3}$$

where

d_I = delay for intersection (s/veh);

d_A= delay for Approach A (s/veh); and

v_A= adjusted flow for Approach A (veh/h).

2.4.6 LEVEL OF SERVICE

The average control delay is directly related to intersection LOS. The HCM method employs criteria that assign a letter, from A to F, to control delay values based on the criteria listed in **Table 2.1**. LOS A to D is considered acceptable, LOS E tolerable, and LOS F considered to be poor and failing.

Table 2.1: LOS Criteria for Signalized Intersections (HCM)

LOS	Control Delay (s/veh)
A	< 10
B	10-20
C	20-35
D	35-55
E	55-80
F	> 80

2.4.7 SYNCHRO PROGRAM

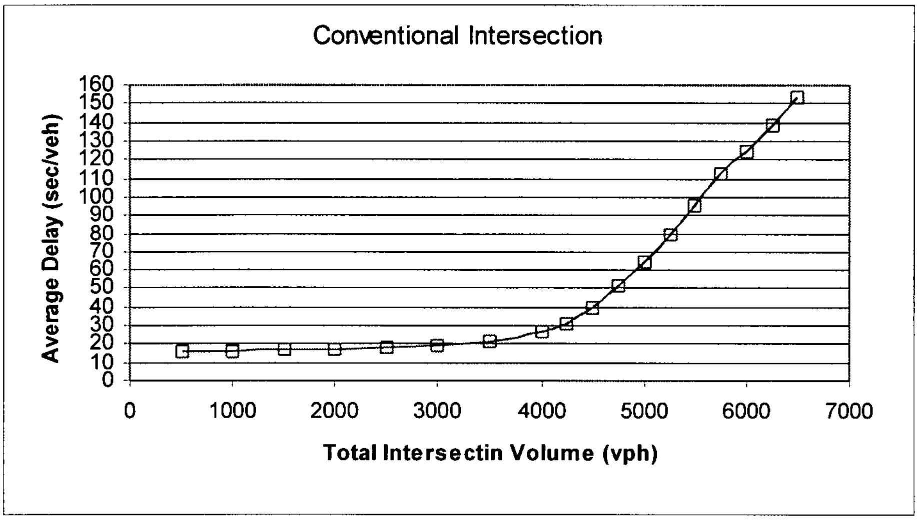
Synchro[3] is a widely used software program for modelling and optimizing traffic signal timings. The program implements the methods of the HCM 2000 for calculating intersection capacity and other performance measures. In addition, Synchro can also be used to optimize cycle lengths and splits, which eliminates the need to create multiple timing plans in search of the optimum. This program was used to analyze the intersections for this study.

2.4.8 DELAY VERSUS TRAFFIC DEMAND

Delays at signalized intersections generally increase with traffic demand. As higher volumes continue to compete for green time the less efficient the signal timing and operation becomes. With the growth of urban areas, it is common for urban intersections to experience continued growth in traffic demand.

To illustrate the effects of traffic demand on the intersection delay, a graph of traffic volumes versus average control delays can be plotted, such as the graph shown on **Figure 2.4**. For this particular exercise, the typical intersection layout shown in Figure 2.1 was used, which has exclusive left- and right- turn lanes and two through lanes for each approach. The average delays for the intersection were calculated using the *Synchro* program and for each trial the intersection's cycle length and timing splits were optimized. The process was repeated over a range of traffic demand. A sample printout from *Synchro* is included in Appendix A.

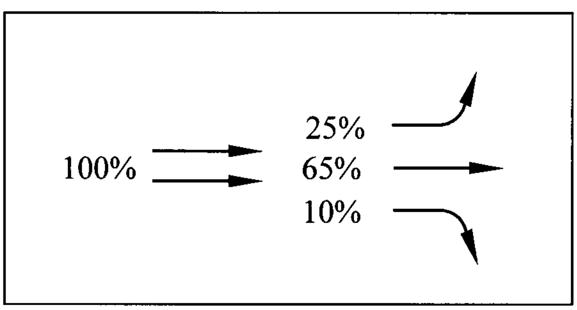
Figure 2.4: Control Delay versus Traffic Demand



The traffic for a single approach in the above example was distributed as shown in **Figure 2.5**; and for simplicity, each approach was given identical traffic volumes and distribution and analyzed as such in *Synchro*. For each analysis trial the total intersection volume was increased in increments of 500 veh/h.

As shown in Figure 2.4, the intersection is able to efficiently handle traffic demands up to a certain level and thereafter the delays increase progressively until the intersection eventually fails. For this particular intersection, a LOS F (> 80 s/veh) is reached for a traffic demand of about 5200 veh/h.

Figure 2.5: Sample Traffic Distribution



2.4.9 *INTERSECTION EXPANSION*

Intersections can be expanded in an attempt to increase capacity and reduce congestion. It is common practice to add lanes to the approaches of intersections as traffic demands increase. However, there are limits to the size that intersections can be expanded. Available right-of-way and economic impacts often restrict the expansion of intersections.

Furthermore, adding lanes and increasing the size of intersections may not be as effective in solving traffic congestion as it is often assumed. Mucsi and Khan[4] studied the effectiveness of increasing the size of signalized intersections, by adding through and turn lanes, over a typical lifespan of an intersection. Using a hypothetical intersection, the key performance measures of an intersection were computed over a range of traffic volumes. This is similar to the test described above (Figure 2.4) except that, when the intersection reached critical traffic demand levels the intersection was modified by adding more lanes. This was repeated for every subsequent size increase made to the intersection.

The study found that as intersections grow they become less effective in providing additional capacity, i.e. every new lane that was added provided less additional capacity than the previous lane added. The result is a diminishing marginal capacity benefit associated with additional lanes. Expanding intersections above a certain size, especially in locations that experience high traffic growth, may end up being an expensive, ineffective and short-lived solution to the problems of traffic congestion.

2.5 CONCLUSIONS

The efficiency of a road network greatly depends on the design and operation of intersections. The design of intersections involves four key elements: traffic factors, physical features, human factors and economical factors. Intersections create conflict points on roads, which will depend on the number and direction of the approaches, number of lanes, signal control, traffic volumes and the percentage of right or left turns. Intersections are signalized as a means of improving safety and regulating the movements of the conflicting flows of traffic. Left-turning vehicles are often the controlling element of the operation of signalized intersections and will dictate the number, type and sequence of signal phases.

Common measures by which signalized intersections are evaluated include capacity, level of service (LOS), delay and queue length. The HCM 2000 method for analyzing signalized intersections is a widely used methodology, which addresses the capacity, LOS and other key performance. In determining these measures, the analysis must consider a wide variety of prevailing conditions, including the amount and distribution of traffic movements, traffic composition, geometric conditions and signalization conditions.

Synchro is a widely used software program for modelling and optimizing traffic signal timings, which implements the methods of the HCM 2000. This program was used to analyze the intersections under this study due to its ability to optimize cycle lengths and phasing splits, and thus generate quick results. A sample intersection was analyzed over a range of traffic volumes and the results showed that the intersection was able to efficiently handle traffic demands up to a certain level and thereafter the delays increased progressively until the intersection eventually failed.

As traffic demands increase, it is common practice to add lanes to the approaches of intersections in an attempt to increase capacity and reduce congestion. However, a study found that as intersections grow they become less effective in providing additional capacity. The result is a "diminishing marginal capacity benefit" associated with additional lanes.

CHAPTER 3: LEFT TURN ALTERNATIVES

3.1 INTRODUCTION

The impact of left turns on operations is probably the most significant factor considered in analyzing conventional signalized intersections and is often the principal factor in degraded performance. When left turns occur against opposing flow during permitted phasing they must wait for gaps in that flow. When protected left-turn phases are used, the time dedicated for the left turns is not available for the through movements, which results in increased delays for through movements.

In addition to the effect on intersection productivity, left-turn activity generally has an impact on accident experience. Accidents involving left turns during permitted phases are often attributed to red light running of through vehicles, poor sight distance from the left turn and insufficient clearance intervals.

3.2 CONVENTIONAL LEFT TURN ALTERNATIVES

Table 3.1 lists alternatives for handling left turns for conventional intersections. While the last two alternatives in Table 3.1 eliminate the left-turn-opposing (LTO) conflict completely, the remaining alternatives merely manage the conflict by way of phasing and adding left turn lanes. However, higher traffic volumes, such as those found on urban arterials during peak-hour, demand longer cycle times and complex phasing. Increasing the cycle length and introducing multiple phases normally lead to increased intersection delays.

Dual left-turn lanes are commonly installed when the left-turn volumes reach high levels and/or the LOS of the left turns at intersections reaches undesired levels. While this treatment may increase the capacity of left-turn movements, it can also result in longer cycle times and increase delays overall. This is due to the fact that dual left-turns are restricted to protected phases, which takes time away from opposing through movements. Permitted phases for dual left-turns are normally avoided due to safety concerns. Consequently, there is a practical limit to the performance of conventional intersections when dealing with left turns.

Table 3.1: Left-turn Alternatives for Conventional Intersections

Option	Advantages	Disadvantages
Two-phase signalization	Minimizes lost times Efficient where LT's are few	Left turns are opposed Congestion can occur with many LT's
Two-phase signalization with LT lane	Removes waiting LT's from through lanes	Does not address limits of opposing flow on LT's
Multiphase signalization/protected LT's	Provides unopposed LT's Reduces congestion caused by LT's	Requires LT Lane Increases cycle length May increase total delay
Multiphase signalization/ protected + permitted LT's	Minimizes cycle required to provide for protected LT's Reduces delay to LT's by allowing permitted LT's throughout green	Complex phasing is difficult to convey May confuse drivers
Prohibition of LT's	Avoids all LT problems	Diverts traffic to other locations where problems may occur Causes driver inconvenience
One-way streets	Removes opposing flow for LT's	Requires compatible system geometry May increase average trip lengths, and VMT

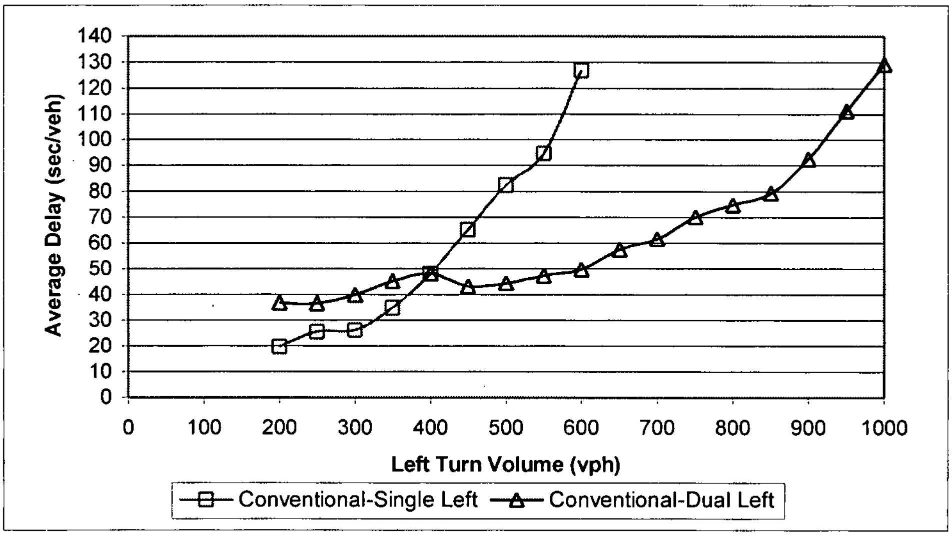
3.3 DELAY VERSUS TRAFFIC DEMAND

Figure 3.1 below illustrates the effects of left-turn traffic demands on intersection delays. The same intersection layout shown in Figure 2.1 was also used for this analysis. Again for simplicity, identical volumes were applied to each approach. For this exercise, the left turn volumes were increased for each trial while the volumes of the other movements (through and right-turn) were kept constant. The volumes for the through and right-turn movements were set at levels that would normally result in a LOS of B/C for the intersection. The average delays for the intersection were calculated using the Synchro program and for each trial the intersection's cycle length and timing splits were optimized, as in the previous analysis.

As can be seen from Figure 3.1, intersections with single left lanes are limited to handling a certain amount of left turns before the performance of the intersection begins to degrade. For this hypothetical intersection, a LOS F is reached when left-turn volumes are at about 500 veh/h.

Real-life urban intersections may handle more (600 veh/h) before failing given that heavy left turn volumes occur only for certain approaches (i.e. during peak hour periods) and are not symmetrical as the volumes used for the example intersection.

Figure 3.1: Delay Versus Left-Turn Demand



Intersections with dual-left turn lanes are able to accommodate greater turning traffic, as can also be seen from Figure 3.1. However, the dual lane scheme will be less efficient for lower volumes compared to single lane schemes given that intersection with single left-turn lanes can make use of permitted phases in facilitating turning traffic, which result in shorter cycle lengths and less delays.

The advantage of dual lanes is realized for higher volumes, where the throughput of turning vehicles is substantially better. However, the plot shown in Figure 3.1 is likely to be conservative when compared to actual intersections because the distribution of traffic in the real world would be more proportionate. In other words, the through movements would grow in accordance with the left turn volumes and as such, the intersection would experience greater delays.

3.4 UNCONVENTIONAL LEFT TURN ALTERNATIVES

Unconventional measures for treating left turns have been developed in the past. These measures offer an alternative to expanding the size of existing conventional intersections or upgrading intersections to interchanges, which have been the common practice for improving operations on road networks. The focus of these alternative measures is to reduce the conflict points associated with left turns and maximize the throughput of vehicles.

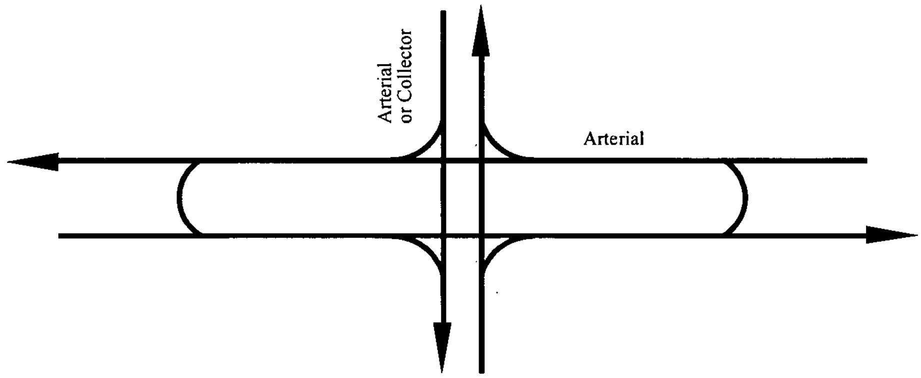
There have been several unconventional schemes developed, some of which have been implemented on existing road networks. Hummer[5] investigated seven schemes that focused on treating left turns to and from arterials. Three of the alternatives have been used successfully in at least one state in the US. The remaining four were considered newer and extensions of the older unconventional schemes. This study will discuss four unconventional alternatives, which include:

- Median U-Turn;
- Bowtie;
- Super-Street and;
- Crossover Displaced Left-turn.

3.4.1 Median U-turn

The median U-turn scheme eliminates left turns at the intersection and instead diverts the left turns downstream from the intersection to directional median crossovers. A schematic of the intersection is illustrated in **Figure 3.2**. Vehicles turning left from the arterial continue past the intersection, make u-turn manoeuvres across the median and then proceed to turn right at the intersection. Left turns from the cross street are completed by, first making a right turn at the intersection and then making a u-turn at the median crossovers. A variation to this scheme is to place directional median crossovers at the cross streets in addition to the crossovers placed on the main arterial. This would increase the left turn capacity of the intersection.

Figure 3.2: Median U-Turn Intersection



The restriction of left turns at the intersection means that signalization can be simplified to two phases. The result would be shorter delays for through traffic. Also, depending on the traffic conditions, the median crossovers may also be signalized and coordinated with the main intersection traffic signal, which would reduce delays for left turning (or u-turning) vehicles.

The width and configuration of the crossovers will depend on the design vehicle. The location of the crossovers relative to the intersection will be a trade-off between minimizing queues at the intersection and minimizing travel time for vehicles undergoing the u-turn manoeuvre.

The advantages and disadvantages of this unconventional scheme, as listed in the Hummer report, are presented in **Table 3.2**. The main advantage of this scheme is that through traffic will experience less delay as compared to conventional multiphase intersections. One downside is the increase in delay and travel distance for left turn vehicles. As such, this scheme should be considered for arterials with high through volumes and moderate to low left-turn volumes. High left-turn volumes would result in extra delay and longer queues, which would likely outweigh the savings for through traffic.

The median u-turns have been used by the Michigan Department of Transportation and other agencies in Michigan, USA for over 30 years[5].

Table 3.2: Median U-Turn Advantages/Disadvantages

Advantages	Disadvantages
• Reduced delay for through arterial traffic; • Easier progression for through arterial traffic; • Fewer stops for through traffic, particularly on approaches without signalized directional crossovers; • Fewer threats to crossing pedestrians; and • Fewer and more separated conflict points.	• Driver confusion; • Driver disregard of the left-turn prohibition at the main intersection; • Increased delay and travel distance for left-turning traffic; • Increased stops for left-turning traffic; • Larger rights-of-way along the arterial; • Higher operation costs for extra signals; and • Longer cross-street minimum green times or two-cycle pedestrian crossing.

3.4.2 Bowtie

The bowtie scheme, shown in **Figure 3.3**, is similar to the median u-turn except that it uses roundabouts to accommodate the left turns instead of the directional median crossovers. Vehicles intending to turn left from the main arterial will turn right at the intersection and use the roundabout to turn around.

The bowtie design was inspired by the "raindrop" interchange designs commonly found in Great Britain[5]. The roundabouts are placed on the cross street to minimize disruption on the main arterial road. The prohibition of left turns at the intersection means that only a two-phase signal is required.

The advantages and disadvantages of this unconventional scheme, as listed in the Hummer report, are presented in **Table 3.3**. As in the median u-turn scheme, the main advantage of this scheme is reduced delays for through traffic but has the downside of increasing delays and travel distances for left-turning vehicles. Also, if the cross street traffic is too high, additional delay would be experienced by the left-turn vehicles entering the roundabouts. Consequently, the bowtie alternative should be considered for intersections comprising of an arterial having high through traffic and a cross street having moderate to low through and left turn volumes.

Figure 3.3: Bowtie Intersection

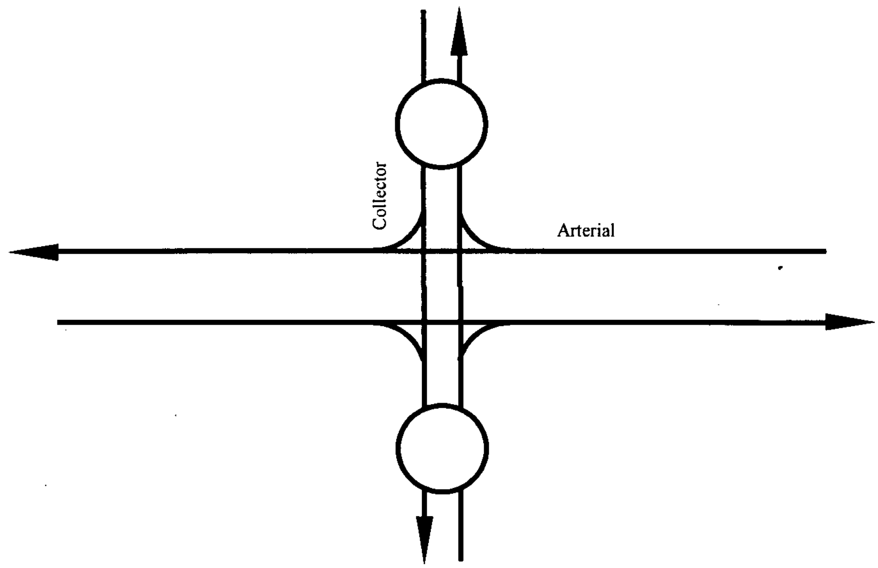


Table 3.3: Bowtie Advantages/Disadvantages

Advantages	Disadvantages
• Reduced delay for through arterial traffic; • Reduced stops for through traffic; • Easier progression for through arterial traffic; • Fewer threats to crossing pedestrians; and • Reduced and separated conflict points.	• Driver confusion; • Driver disregard of the left-turn prohibition at the main intersection; • Increased delay for left-turning traffic and possibly cross-street through traffic; • Increased travel distances for left-turning traffic; • Increased stops for left-turning and cross-street through traffic; • Larger rights-of-way along the arterial; • Additional right-of-way for the roundabouts; and • Difficult arterial u-turns.

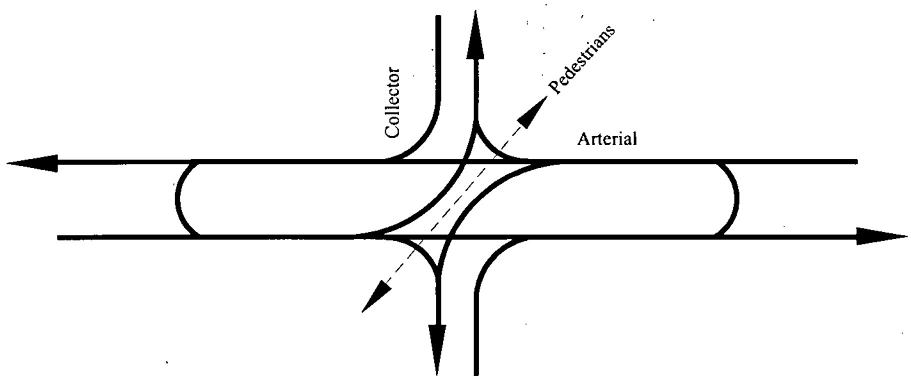
Although roundabouts are becoming more popular in North America, the author is unaware of any installations of a complete Bowtie intersection. There have been a few raindrop interchanges installed in the USA, which are similar to diamond interchanges except that roundabouts are used instead of signalized intersections.

3.4.3 Super-Street

The super-street alternative is also based on the median u-turn concept but as shown in **Figure 3.4**, the super-street scheme does not permit cross-street through traffic to use the main intersection. Instead, the cross street through movement is required to use the median directional u-turns along with the left turning movement. Another variation from the median u-turn scheme is that left turn movements from the arterial are permitted at the main intersection. This configuration results in two independent intersections with each having three approaches.

Each independent intersection can utilize a two-phase signal design. This independence allows each direction of the arterial to have its own signal timing pattern and cycle lengths. Consequently, ideal progression in both directions can be achieved at any time with any intersection spacing.

Figure 3.4: Super-Street Intersection



The advantages and disadvantages of the super-street alternative scheme are presented in **Table 3.4**. As in the previous two schemes, the key advantage offered by the super-street scheme is reduced delay for arterial through traffic. The added benefit is that travel time for left turns from the arterial is minimized given that these manoeuvres are permitted at the main intersection and not rerouted as in the other two schemes. However, the consequence of the intersection scheme is the increased delays and travel time for cross street through traffic. Thus, the super-street alternative should be considered where high arterial through volumes conflict with moderate to low cross street through volumes.

Table 3.4: Super-Street Advantages/Disadvantages

Advantages	Disadvantages
• Reduced delay for through arterial traffic and for one pair of left turns (usually left turns from the arterial); • Reduced stops for through arterial traffic; • "Perfect" two-way progression at all times with any signal spacing for through arterial traffic; • Fewer threats to crossing pedestrians; and • Reduced and separated conflict points.	• Driver and pedestrian confusion; • Increased delay for cross-street through traffic and for one pair of left turns (usually left turns to the arterial); • Increased travel distances for cross-street traffic and one pair of left turns; • Increased stops for cross-street traffic and one pair of left turns; • A slow two-stage crossing of the arterial for pedestrians; and • Additional right-of-way along the arterial.

3.4.4 Crossover Displaced Left-turn Intersection

The Crossover Displaced Left-turn (XDL) intersection is another unconventional intersection concept that involves, as its name implies, displacing left turns onto the other side of the opposing traffic lanes prior to the main intersection. **Figure 3.5** shows a schematic of the XDL intersection. The displacement results in left turn traffic crossing the opposing lanes, creating an at-grade intersection upstream of the main intersection. Once on the other side, left turns can be made at the main intersection simultaneously with through traffic without the opposing through traffic conflicts. Right-turn traffic under this scheme bypasses the main intersection, around the displaced left turns, and is merged back into mainstream traffic downstream, as shown in Figure 3.5.

According to Jagannathan[6], an XDL intersection has been built at the entrance to the Dowling College National Aviation and Transportation Centre, Oakdale, New York and a T-intersection version built in Prince Georges County in Maryland. There has been some interest elsewhere in the USA to install the XDL intersection.

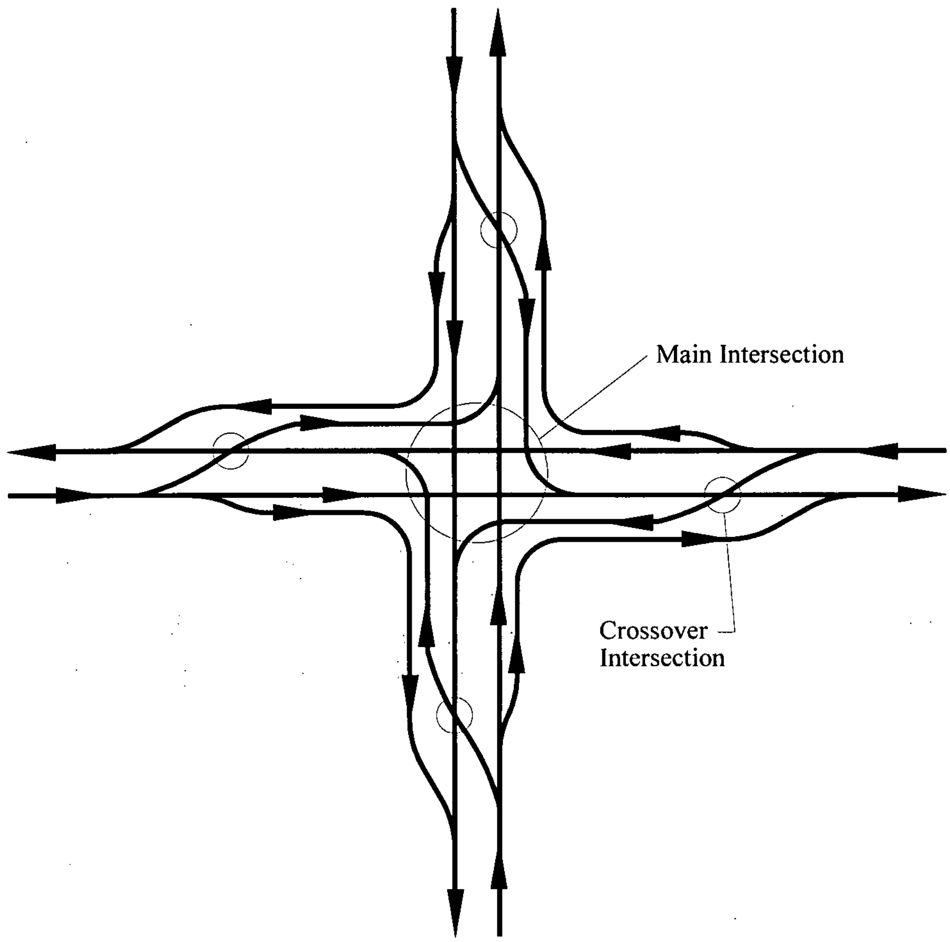
Besides eliminating the left-turn opposing conflict, the key advantage of the XDL intersection is that it simplifies the phasing at the main intersection, as there would be no need for protected left-turn phases. The left turns could run simultaneously with through traffic in both directions, and thus the main intersection could operate with two phases. Coordination with the upstream intersections would be possible given that it too would operate with two phases. In the same paper, Jagannathan analyzed the XDL intersection and found it to have significant savings in delays and considerably more capacity as compared to conventional intersections. **Table 3.5** lists some of the key advantages of the XDL intersection.

As noted in Table 3.5, one key disadvantage of the XDL intersection is the large area and right-of-way required to build the intersection compared to a conventional intersection. The additional lanes required for the left turns as well as for the right turn bypass require a much wider cross section at the intersection. The wider intersection would require longer clearance interval times which would result in longer inter-green time. Red light violators could be more prominent under such conditions.

Table 3.5: XDL Intersection Advantages/Disadvantages

Advantages	Disadvantages
• Elimination of left-turn opposing conflicts; • Simple two phase operation at both the main intersection and upstream intersections; • Good coordination between crossover signals and the signals at the main intersection; and • Reduced delays and queue lengths for through traffic.	• Additional right-of-way along the arterial; • Longer clearance interval given the wider intersection; • Driver and pedestrian confusion; • Increased stops for left turn traffic; and • Longer travel distance for pedestrians.

Figure 3.5: XDL Intersection



3.5 CONCLUSIONS

The impact of left turns on operations is probably the most significant factor considered in analyzing conventional signalized intersections and is often the principal factor in degraded performance. When protected left-turn phases are used, the time dedicated for the left turns is not available for the through movements, which results in increased delays for through movements. In addition to the effect on intersection productivity, left-turn activity generally has an impact on accident experience.

Conventional alternatives can manage left-turn-opposing (LTO) conflicts by way of phasing and adding left turn lanes or in some cases the left turns are eliminated at the intersection.

Intersections with high traffic volumes demand longer cycle times and complex phasing, which in turn lead to increased intersection delays. A sample intersection was analyzed using Synchro over a range of left turn volumes for both the single- and dual-left turn lane conditions. The results showed that in either case there are practical limits for the left turn volumes that intersections can accommodate.

Unconventional measures for treating left turns have been developed in the past and offer an alternative to expanding the size of existing conventional intersections or upgrading intersections to interchanges. The focus of these alternative measures is to reduce the conflict points associated with left turns and maximize the throughput of vehicles. The unconventional schemes that were discussed included the Median U-turn, Bowtie, Super-Street and the XDL intersection.

CHAPTER 4: UPSTREAM SIGNALIZED CROSSOVER INTERSECTION

4.1 INTRODUCTION

The focus of this thesis is to introduce a new unconventional scheme conceived by the author, which has been given the name "Upstream Signalized Crossover" (USC) intersection. This chapter will describe the functional and operational characteristics of this new intersection scheme, including a description of the signal phasing and sequencing.

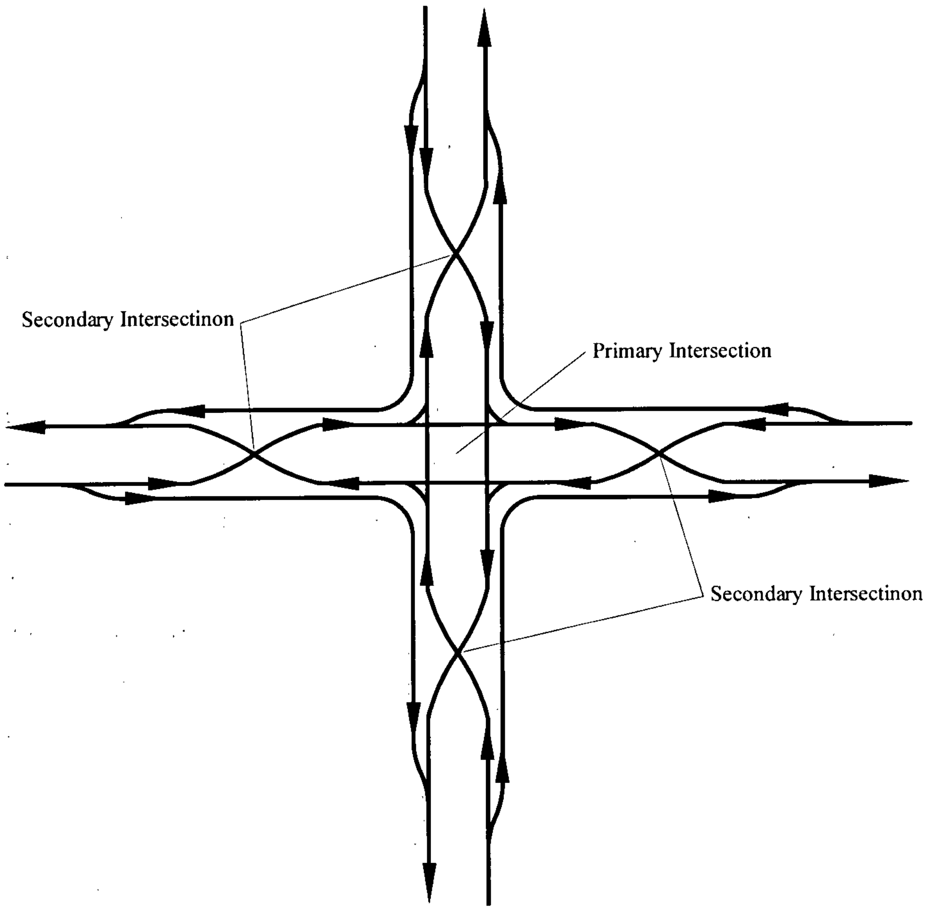
4.2 CONCEPT

This new scheme, shown in **Figure 4.1**, through and left turn traffic cross the median to the left side of the road at a location upstream of the main intersection, while right turn traffic is maintained on the right side. It is similar to the XDL concept in that traffic is crossed over to the other side of the road. The major difference, however, is that thru traffic is also displaced to the other side of the road. Left turns are made at the main intersection from the left side of the road. Thru traffic remains on the left side until it crosses back over to the right side of the road downstream of the intersection. The same movements are made for traffic in the opposite direction, creating a symmetrical crisscrossing movement of traffic. When this is applied to all four directions of a four-legged intersection, the result is that left turns can be made directly without opposing conflict. This condition for left turns would be similar to the right turn condition found at conventional intersections.

The crossing over of vehicles on both sides of the primary intersection necessitates secondary intersections upstream of the primary intersection. The secondary intersections would facilitate both the crossover movement of through and left-turn traffic approaching the primary intersection and departing traffic from the opposite direction. The departing traffic would be composed of through vehicles and vehicles that have turned left from the cross street.

Right-turn traffic for the USC intersection is separated from through and left-turn traffic and directed onto a right-turn bypass. A physical barrier of some kind would be required to separate vehicles travelling on the right-turn bypass from opposing vehicles at the intersection. After completing the turn, right-turn traffic would then be merged back into mainstream traffic beyond the secondary intersection of the cross street, as illustrated in Figure 4.1.

Figure 4.1: USC Intersection



4.3 SIGNALIZATION AND PHASING

By simplifying the traffic movements to only thru and direct left turns, the primary intersection could operate on a two-phase cycle since there would be no need for protected left-turn phases. All four secondary intersections would also need to be signalized to control and regulate the crossing of approaching and departing traffic; and since these intersections do not facilitate any turning movements they too can operate on a two-phase cycle. Consequently, coordination between the primary and secondary signals would be feasible given the simple two-phase operation.

In order to maximize throughput and minimize delays through the USC intersection, the signals would be coordinated such that a continuous green-band carries through from one secondary intersection to the other secondary intersection on the other side of the primary intersection. By using a symmetrical green-band in the opposite direction, opposing green-bands could cross the primary intersection concurrently during a single phase, as illustrated in the time-space diagram shown in **Figure 4.2**. The symmetry would also allow one band to clear a secondary intersection before the opposing band arrives at the same secondary intersection. This would streamline the progression of through traffic in both directions.

The same coordination plan would be employed for the cross street. However, the green-bands for the cross street would be offset from the main street green-bands such that the bands arrive at the primary intersection during the second phase, i.e. after the bands from the main street have crossed the primary intersection. By doing this, the cycle length at the primary intersection is minimized and the green times utilized more efficiently.

In order for the entire intersection to be coordinated all signals would need to have the same cycle length, which is a basic and fundamental requirement of coordinated signals. Cycle lengths that are an even multiple (e.g. two times) of the shortest cycle time for the system could also be used when coordinating signals with two phases. It is highly possible for the offsets of all signals of the USC intersection to be optimized to produce good progression.

Figure 4.3 illustrates the phasing plan for each signal and the ideal sequencing of traffic movements for the overall intersection. The first sequence (A) shown in Figure 5.3 shows eastbound traffic approaching the west secondary intersection (S_W) and westbound traffic approaching the east secondary intersection (S_E) having the green light. The primary intersection is also green for eastbound and westbound through/left-turn traffic during sequence A. The second sequence (B) has the east and west secondary intersections giving the green to the departing traffic in both directions. The signals can be coordinated such that sequence B will occur just as departing traffic reaches the downstream secondary intersection. Sequence C and D are identical to A and B, respectively, except that they are for the northbound and southbound movements.

Figure 4.2: USC Intersection Progression

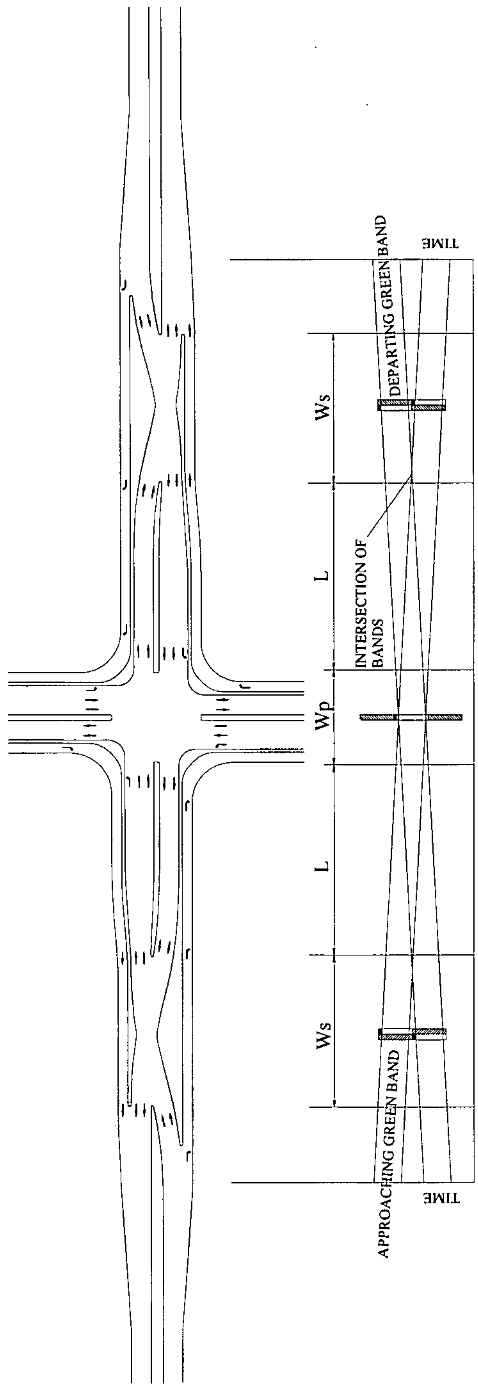
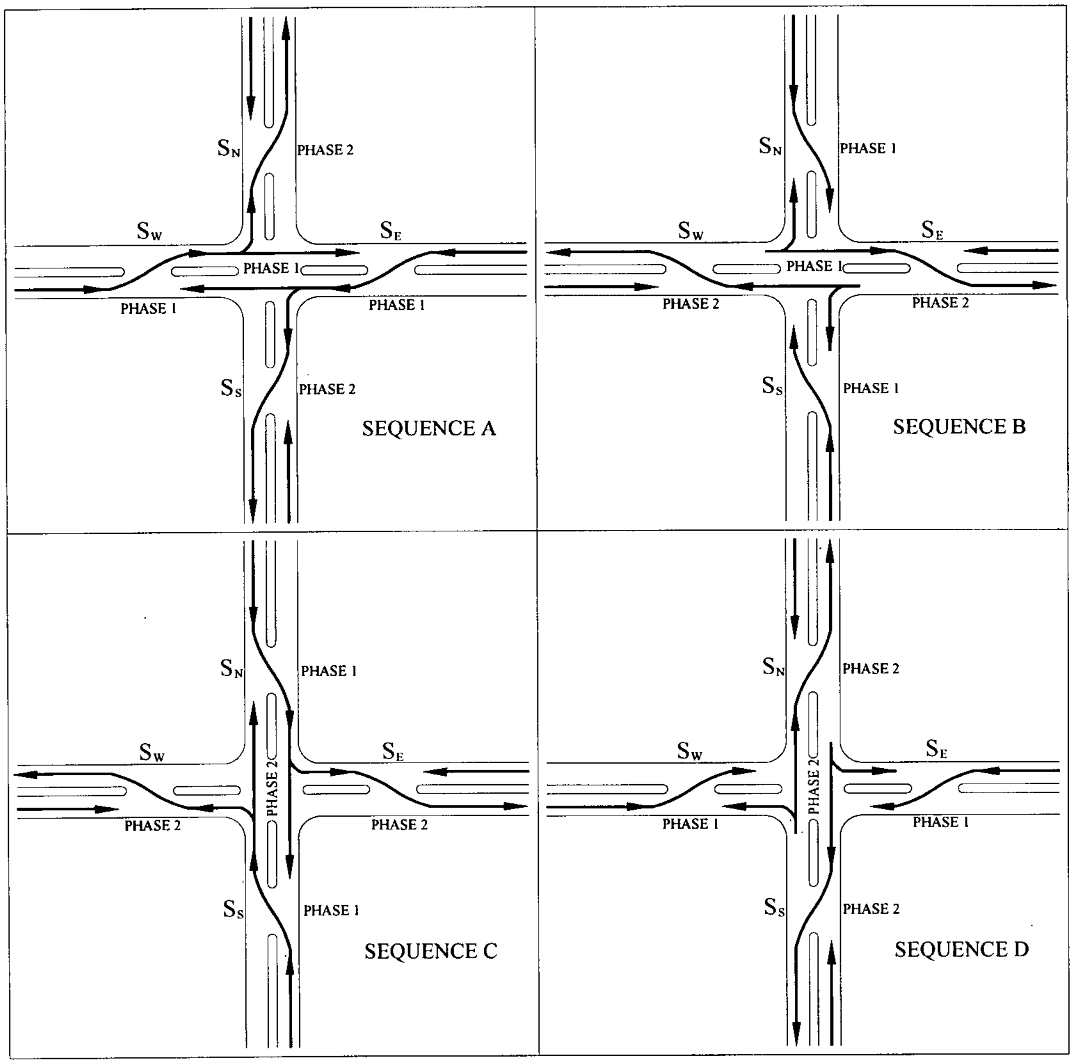


Figure 4.3: Phasing and Coordination



4.4 CONCLUSION

The new USC intersection scheme is similar to the XDL concept in that traffic is crossed over to the other side of the road. Thru and left turn traffic cross the median to the left side of the road at a location upstream of the main intersection, while right turn traffic is maintained on the right side. When this is applied to all four directions of a four-legged intersection, the result is that left turns can be made directly without opposing conflict. The crossing over of vehicles on both sides of the primary intersection necessitates secondary intersections upstream of the primary intersection.

By simplifying the traffic movements to only thru and direct left turns, the primary intersection could operate on a two-phase cycle since there would be no need for protected left-turn phases. All four secondary intersections could also operate on a two-phase cycle. Consequently, coordination between the primary and secondary signals would be feasible given the simple two-phase operation.

In order to maximize throughput and minimize delays through the USC intersection, the signals would be coordinated such that a symmetrical green-band carries through in opposite directions for both the main street and cross street. A well-timed and efficient phasing plan would streamline the progression of traffic through the USC intersection.

CHAPTER 5: OPERATIONAL ANALYSIS OF USC INTERSECTION

5.1 INTRODUCTION

The HCM methodology discussed previously was used to analyze the unconventional USC intersection to determine capacity, delays, LOS and other performance measures. The key steps of this methodology are discussed to demonstrate some of the differences in analyzing the USC intersection compared to a conventional intersection and to note the assumptions made.

The *Synchro* program was also used to analyze the USC intersection because of the convenience in obtaining quick results and also for its capabilities in modelling coordinated signals. As in the sample conventional intersection, the USC intersection was analyzed using hypothetical traffic volumes, which were then varied to determine the effects of traffic demand. The results were compared to a conventional intersection.

5.2 INTERSECTION GEOMETRY

An example of the geometry and lane configuration of the USC intersection is illustrated in **Figure 5.1**. The design of the sample intersection was achieved using principles and criteria found in the TAC design guidelines. The approach used in designing the unconventional intersection and details of the geometric design are discussed later in the report. The sample intersection in Figure 4.1 shows two through lanes and one left turn lane for each approach. A wide raised median divides opposing through traffic and a narrower raised median separates the right-turn bypass from opposing through traffic.

The geometry for the secondary intersections is essentially two one-way roads intersecting at an accute angle. Two through lanes have been provided in each direction at the secondary intersection for this sample design. The right-turn bypass is developed prior to the intersection, which means that it can operate independently of the secondary intersection and facilitate a free-flow movement.

In using *Synchro*, the program could not be set up to represent the reversed configuration of the USC intersection directly because the program has been designed and intended for conventional

"North American" intersections. It would have been ideal to use a "British-based" graphical program instead since the USC intersection functions much like a British intersection.

The author did try "tricking" *Synchro* by creating a small network of multiple one-way links to represent the reversed intersection instead of conventional two-way links. However, this created additional intersections or nodes that required complex phasing and coordination and was consequently abandoned.

Alternatively, the USC intersection was modelled as a conventional "North American" intersection with left turns prohibited entirely at the primary intersection. For all intents and purposes the operation at the primary intersection is the same except that the USC left turns are now considered as right turns for the conventional intersection. USC right turns (bypassed traffic) are now left turns, which are simply ignored at the primary intersection since they would run independently of the primary intersection. **Figure 5.2** shows the link and node configuration used to analyze the USC intersection in *Synchro*. In order to model the "crisscrossing" operation at the secondary intersections, two independent one-way links were used to join the two-way conventional link between the primary and secondary intersections.

5.3 TRAFFIC CONDITIONS

Hypothetical traffic volumes for the USC intersection were used for this analysis. **Figure 5.3** shows sample traffic volumes that have been applied to the USC intersection. The same volume distribution pattern used previously for the conventional intersection analysis was used for the USC intersection, the difference being that the right turn volumes in the Synchro analysis represented the left turn volumes. Also, as in the previous conventional exercise, pedestrians were excluded to simplify the analysis. As such, a direct comparison of the operating conditions can be made between the conventional and USC schemes. **Table 5.1** summarizes the key traffic parameters for the USC intersection. The traffic volumes shown in the table are for one trial analysis period.

Figure 5.1: USCntersection Geometry

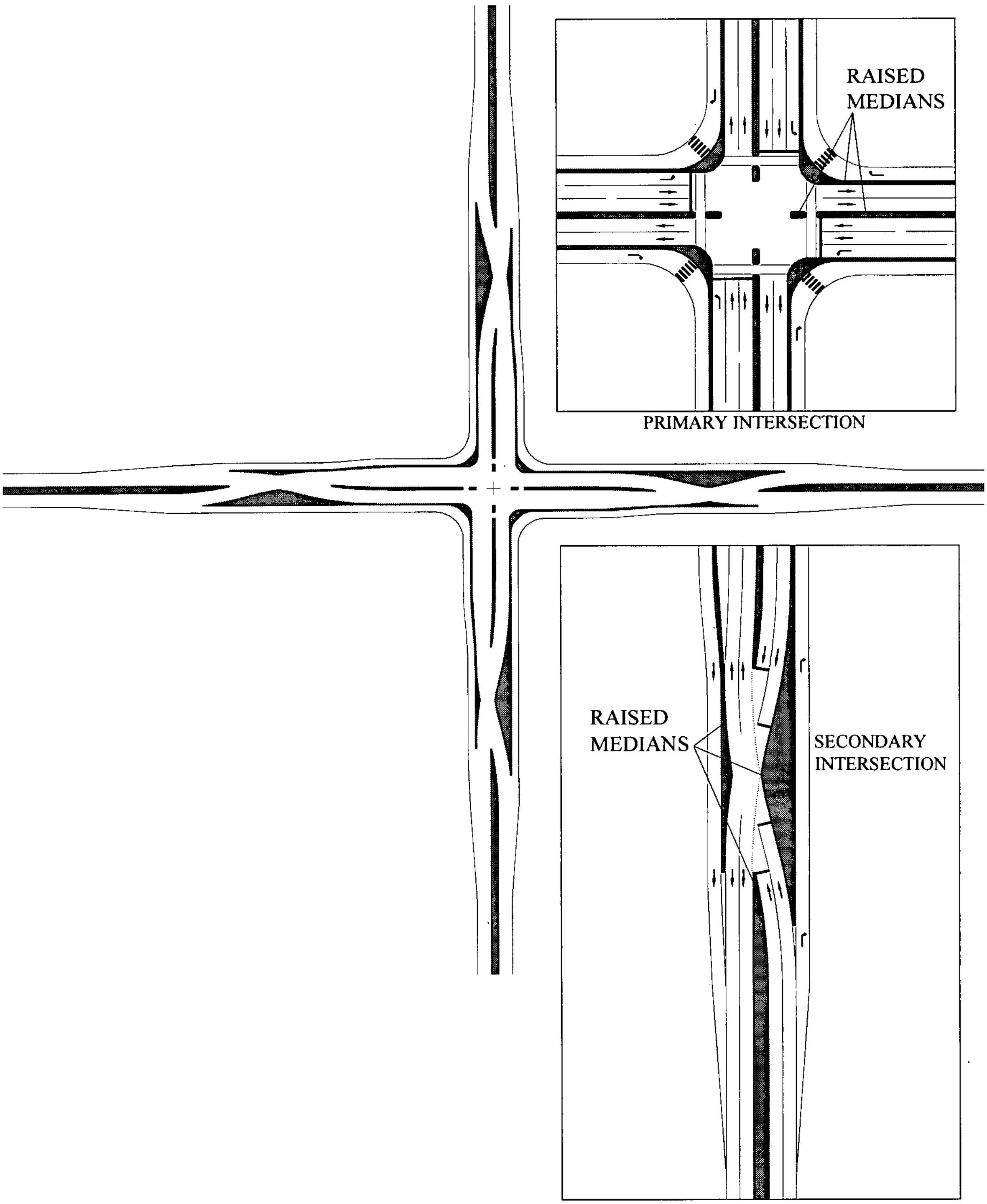


Figure 5.2: Synchro Configuration

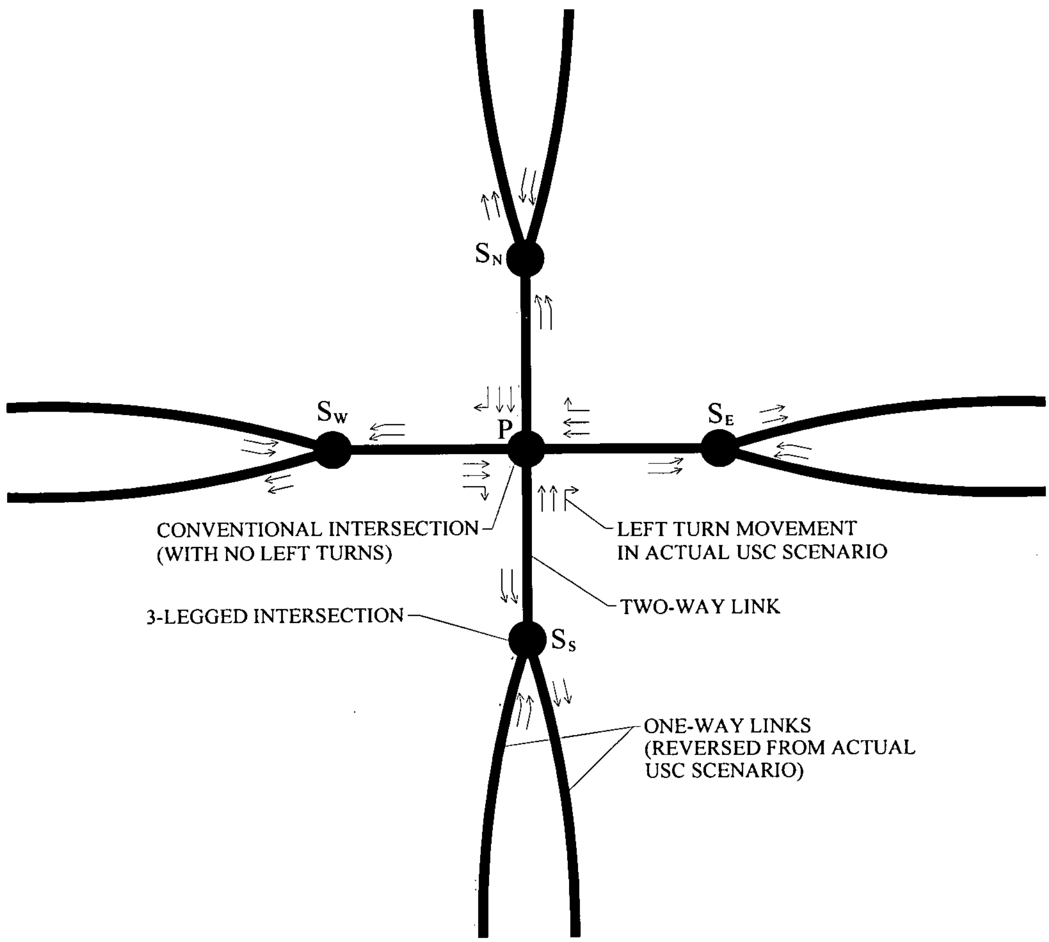
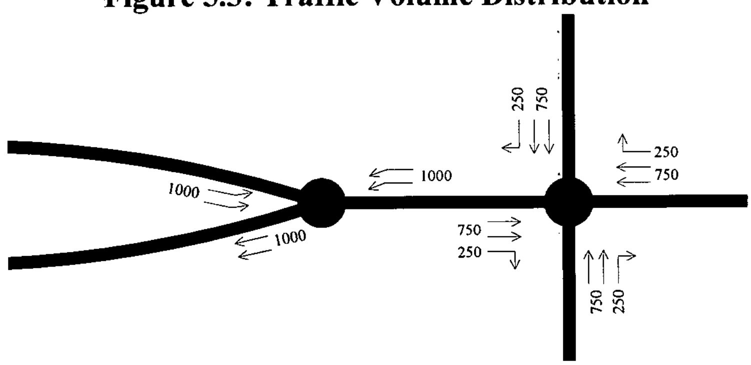


Figure 5.3: Traffic Volume Distribution



As discussed earlier, the key to the efficient operation of the USC intersection would be proper coordination of the multiple signals, such that good progression would be achieved within the secondary and primary intersections. Near-ideal progression could be possible through the USC intersection given the close proximity of the primary signal to the secondary signals and the fact that inbound traffic would be metered by the secondary signals. This condition would create dense platoons approaching the primary signals. Consequently, the arrival type, as shown in Table 5.1, for all approaches to the primary intersection was considered to be a 6, which represents exceptional progression.

Table 5.1: Traffic Parameters

Traffic Parameter	Primary Intersection		Secondary Intersections	
	LT (RT in Synchro)	TH	Inbound	Outbound
Volume, V (veh/h)	313	938	1250	1250
% heavy vehicles, % HV	5%	5%	5%	5%
Peak-hour factor, PHF	0.92	0.92	0.92	0.92
Arrival type, AT	6	6	3	5
Parking maneuvers, N_m (maneuvers/h)	0	0	0	0
Bus stopping, N_b (buses/h)	0	0	0	0
Min. green time, G_p (s)	19	19	n/a	n/a

Outbound traffic approaching a secondary intersection would also be considered to have a favourable arrival type because the same platoon passing through the primary intersection would continue through to the secondary intersections. However, given that the outbound traffic would also be made up of traffic that has turned from the cross street, the platooning of traffic would be less dense. Thus, the arrival type for the outbound traffic approaching the secondary intersections was assumed to be 5.

The arrival type for the inbound traffic approaching a secondary intersection may not be as favourable depending on the traffic and signal conditions upstream of the intersection. For the purpose of this study, an arrival type of 3 was assumed for this approach movement, which represents a condition whereby the arrivals are random with minor platooning.

5.4 SIGNALIZATION CONDITIONS

The signals of the USC intersection could operate with relatively short cycle lengths with two phases, as previously discussed. The phasing plan shown in Figure 4.3 was used as the basis for setting up the signal timing and phasing in *Synchro*. Left turns (or right turns in *Synchro*) were permitted during red light phases.

Determining the cycle length for all five intersections involved two steps: first, the program was used to optimize the cycle length for each secondary intersection and the primary intersection; second, the longest cycle length was then selected as the master cycle length. After applying this cycle length to all five signals, the signal timing splits and offsets were optimized for each of the five intersections. This process was repeated over the same range of traffic volume as the analysis conducted for the conventional intersection.

Theoretically, each secondary intersection should have the same split given the symmetry of both the geometry and the traffic volumes assigned at each intersection. As well, the signal offsets should also be identical for all four secondary signals with respect to the primary signals. The cycle lengths ranged from 40 seconds to 100 seconds.

5.5 LANE GROUPING AND VOLUME ADJUSTMENT

Determining the lane groups for the USC intersection was straightforward. At the primary intersection, the left movement was considered a separate lane group from the through movement.

The default peak-hour factor (PHF) of 0.92 in *Synchro* was used to adjust the volumes. The same PHF was applied for all movements.

5.6 SATURATION FLOW

Table 5.2 summarizes the adjustment factors used to compute the saturation flow rates. The base saturation flow rate for all lane groups was 1900 veh/h. Note that the factors associated with pedestrians, bicycles, buses and parking are all set to 1.0 given their exclusion from the intersection analysis.

Table 5.2: Saturation Flow Parameters

Saturation Flow Rate Parameters	Primary Intersection		Secondary Intersections	
	LT (RT in Synchro)	TH	Inbound	Outbound
Base saturation flow, s_o (pc/h/lane)	1900	1900	1900	1900
Number of lanes, N	1	2	2	2
Lane width adjustment factor, f_w	0.97	1.00	1.00	1.00
Heavy vehicle adjuctment factor, f_{hv}	0.95	0.95	0.95	0.95
Grade adjustment factor, f_g	1.00	1.00	1.00	1.00
Parking adjustment factor, f_p	1.00	1.00	1.00	1.00
Bus blockage adjustment factor, f_{bb}	1.00	1.00	1.00	1.00
Lane utilization adjustment factor, f_{LU}	1.00	1.00	1.00	1.00
Left -turn adjustment factor, f_{LT}	0.85	1.00	1.00	1.00
Right-turn adjustment factor, f_{RT}	n/a	n/a	n/a	n/a
Left -turn ped/bike adjust. factor, f_{Lbb}	1.00	1.00	1.00	1.00
Right-turn ped/bike adjust. factor, f_{Rbb}	n/a	n/a	n/a	n/a

Also, a notable difference between the USC intersection and a conventional intersection is the way the left turn factor is determined. The left turn factor as defined in the HCM methodology would not apply to the USC intersection given that this factor was derived for conventional left turns, which unlike the USC left turns are made with opposing traffic conflict. It would be more appropriate to use the right turn factor, as defined in the HCM, for the left turns made at the USC intersection since the operating principle is virtually the same as a conventional right turn. The fact that left turns were reversed in *Synchro*, i.e. treated as right turns, made the analysis simple because there was no need to make special adjustments to the left turn factors

5.7 CAPACITY AND V/C RATIO

The capacity for the primary intersection and for each of the secondary intersections could be computed using the HCM method, since they operate individually as conventional intersections. Each intersection would have its own critical lane group for which the critical v/c ratio, X_c, is computed. The computations would be straight forward given the two-phase operation at each intersection.

5.8 CONTROL DELAY

The delays at the primary intersection and the secondary intersections can be estimated using the HCM method for calculating control delay, since they would function as regular signalized intersections. The delay values are easily obtained using the *Synchro* program, and since it is able to account for the synchronization characteristics of the multiple intersection schemes, the delay calculations would be more realistic. **Table 5.3** shows an example of delays calculated by *Synchro* at each of the five intersections. Note that due to the symmetry of both the design volumes and the intersection geometry the delays at each of the secondary intersections are identical and the delays for each approach of the primary intersection are also identical.

Table 5.3: USC Delay Results

Intersection		Eastbound		Westbound		Northbound		Southbound	
		Thru	Left	Thru	Left	Thru	Left	Thru	Left
Primary	Volume	750	250	750	250	750	250	750	250
	App. Delay	3.9	2.6	3.9	2.6	3.9	2.6	3.9	2.6
East Secondary (Se)	Volume	1000		1000					
	App. Delay	4.4		11.2					
West Secondary (Sw)	Volume	1000		1000					
	App. Delay	11.2		4.4					
North Secondary (Sn)	Volume					1000		1000	
	App. Delay					4.4		11.2	
South Secondary (Ss)	Volume					1000		1000	
	App. Delay					11.2		4.40	

As shown in Table 5.3, at each secondary intersection the delays are not the same for both approaches even though the volumes are equal. The outbound approach (heading away from the primary intersection) experience less delay than the inbound approach, due to the favourable progression and synchronization between the primary and secondary intersections. As discussed earlier, the inbound approaches (heading towards the primary intersection) would be more susceptible to random arrivals and its operation would depend somewhat on upstream conditions.

Combining and summing the individual delays experienced at each of the intersections estimated the total delays experienced by vehicles driving through the entire USC intersection. **Figure 5.4** illustrates the various delays encountered for each manoeuvre. As an example, the total delay for the through traffic from the west approach *(dTw)* would be computed by summing the delays calculated at the west secondary intersection (dTw_1), at the primary intersection (dTw_2) and at the east secondary intersection (dTw_3). The total delay for the left turn vehicles from the same approach would also have three delay components, as shown in Figure 5.4, with the third component at the north secondary intersection. Note, that the delay at the west secondary intersection for the left turn-bound vehicles (dLw_1) would be equal to the delay calculated for the real through movement (dTw_1) since there would be no distinction between the two movements.

Delays for the right turn (bypass) movements are not taken into account as it is not controlled by any of the intersection signals and is considered to be free flow. In reality, however, pedestrians crossing at the primary intersection would affect the movement and also queues that may form at the secondary intersections, which could block right turning vehicles. Also, the merge condition at the end of the right turn could also affect the operation of the right turns especially if through traffic is heavy.

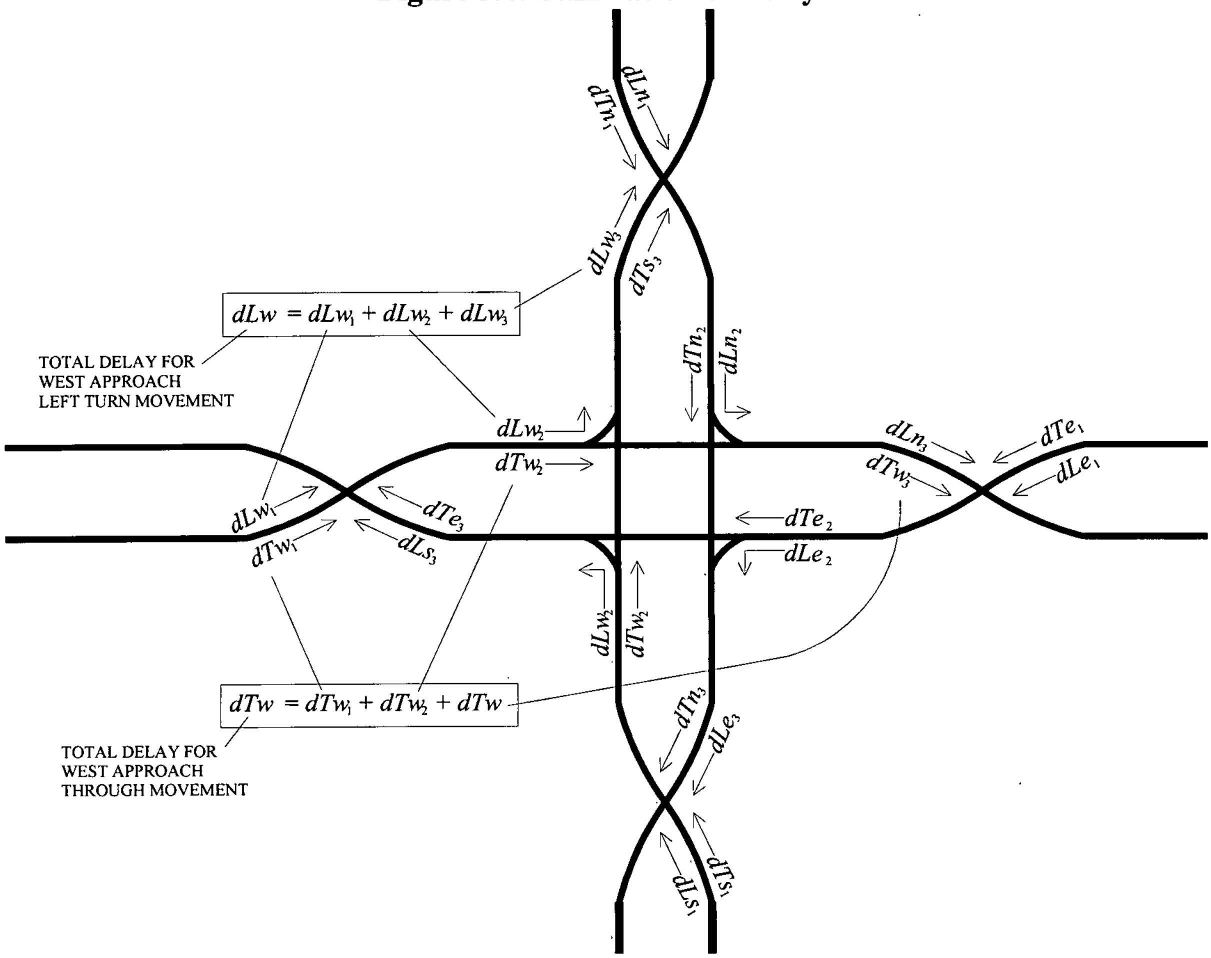
Using the same HCM approach for calculating the delay for each approach, the total delay calculated for both the left and through movements is aggregated to provide a delay estimate for the entire approach. This is done by computing weighted averages for the delays of both movements using Equation 2.2 such that:

$$d_i = \frac{dL_i \cdot vL_i + dT_i \cdot vT_i}{vL_i + vT_i} \qquad (5.1)$$

where

d_i = delay for approach i (s/veh);
dL_i = delay for left turn traffic from approach i (s/veh);
dT_i = delay for through traffic from approach i (s/veh);
vL_i = adjusted flow for left turn traffic from approach i (veh/h);
vT_i = adjusted flow for through traffic from approach i (veh/h);

Figure 5.4: Summation of Delays



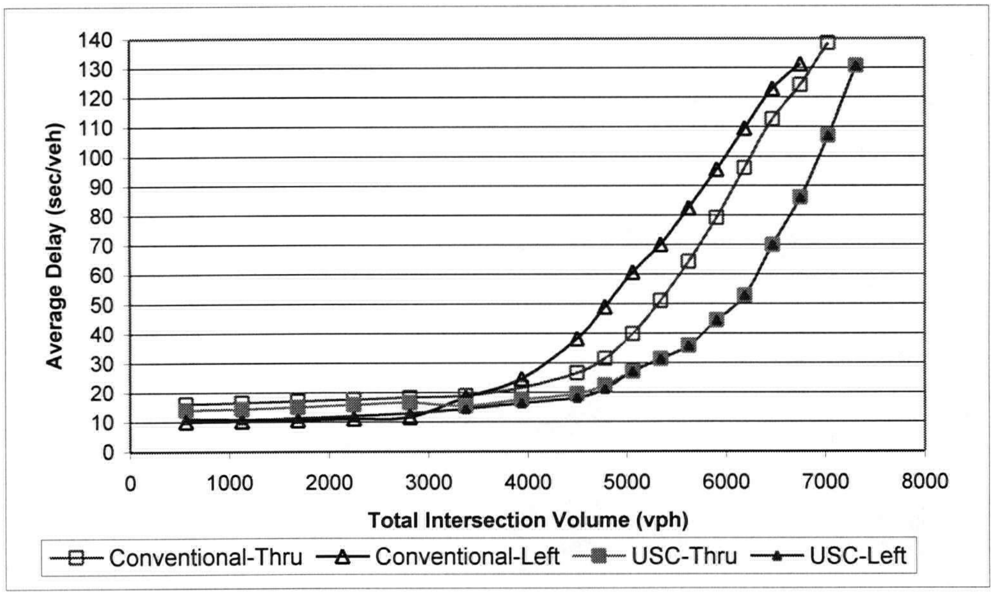
Similarly, the delays for each approach calculated from Equation 5.1 can be aggregated to estimate the delay for the overall intersection as in Equation 2.3. However, for this example the approach volumes and delays are identical and thus a weighted average of the delays by volume would work out to be the same as the delay for a single approach.

Note, the delay values for the USC left turn movements were obtained from *Synchro* as the delay values for the right turn movements.

5.9 DELAY VERSUS TRAFFIC DEMAND

The USC intersection was analyzed over a range of traffic volumes as was done with the conventional intersection. **Figure 5.5** shows the results for the left turn and through movements for the USC intersection along with the results for the conventional intersection for comparison. The delays plotted are for one approach and since they are identical for all four approaches for the conventional and USC intersection analysis, the delays represent the average delays for the intersection.

Figure 5.5: Control Delay Comparison



In order to make a direct comparison between the two intersections, the delays shown in Figure 5.5 for the USC intersection were plotted against the total intersection volume, which included the right turn (bypass) volumes. The same split between the through and left/right turn volumes shown in Figure 2.5 for the conventional intersection analysis was used to arrive at the right turn volumes for the USC intersection.

As can be seen from Figure 5.5, the delays for the USC left and through movements are quite similar. This is due to the fact that the volumes, timings and delays for this exercise were identical at each of the four secondary intersections. Had they not been balanced as such, a variance would have been observed between the two movements.

When compared to the conventional intersection, the USC intersection has a higher threshold for handling traffic. From Figure 5.5, where the left turn movement for the conventional intersection begins to fail (LOS F, 80 sec/veh) the left turns for the USC intersection operates at about LOS C (32 sec/veh), which is a reduction of about 60% in terms of average delays. For the through movements, the USC operates at an average delay of 43 sec/veh, or LOS D, when the conventional intersection has reached 80 sec/veh. Also, in terms of traffic demand, the USC intersection will handle 15%-20% more traffic before either of the through and left turn movements reach 80 sec/veh. **Table 5.4** summarizes the key findings.

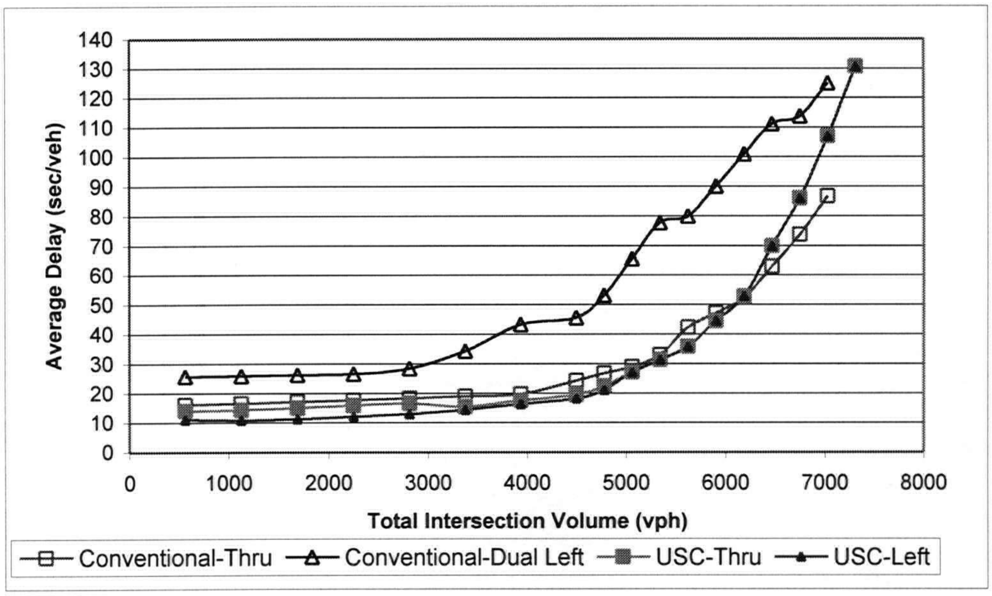
Although the delays (y axis) computed for the USC intersection were not influenced by the right turn volumes, these volumes were included under the total intersection volumes (x axis) in order to make a direct comparison with the conventional intersection. As such, the right turn volumes could in fact be increased from what was used in the above analysis, thereby increasing the overall intersection volume, without affecting the delays computed at each of the USC intersections. This would further increase the gap between the two intersection schemes in the "x" axis making the benefits of the USC intersection more apparent.

Figure 5.6 shows a similar comparison, except this time the conventional intersection was equipped with dual left turn lanes. Again, the USC showed better results for left turns. Part of the reason was that intersections with dual left turn lanes require protected turn phases, which generally increases the overall cycle length and consequently the delays as well. For left turns the USC achieved 59% less delay at the intersection volume that caused the dual left turns to fail, as shown in Table 5.4. The delays for the conventional through movement were now similar to those of the USC through delays. An important observation that can be made here is that the USC intersection still out performs the conventional intersection with respect to left turn operation while maintaining the level of performance for the through movement (i.e. the through movement is not made worse under the USC configuration). Beyond a volume of about 6100 veh/hr the conventional intersection outperforms the USC.

Table 5.4: Summary of Average Delay Comparison

	Single Left-Turn Lane		Dual Left-Turn Lanes		USC	
	Left	Thru	Left	Thru	Left	Thru
Volume at LOS F (veh/hr)	5600	5850	5700	6900	6600	6600
Equivalent USC Delay (sec/veh)	32	43	33	93	-	-
Reduction in Delay by USC (%)	-60%	-46%	-59%	16%	-	-

Figure 5.6: Control Delay Comparison, with Dual Left Turns



A separate analysis was done for left turns to investigate the effects of increasing left turn volumes. For this analysis the through and right turn volumes were held constant while the left turn volumes were progressively increased. The through/right volumes were set at typical values that would normally yield a level of service of around C (moderate) for the intersection. Again for simplicity the volumes and distribution of traffic used were identical for all approaches. **Figure 5.7** below shows the results for the three left turn scenarios, namely conventional-single left, conventional-dual left and the USC left turn.

As shown in Figure 5.7, compared to the other two schemes the left turns of the USC operate more efficiently at the low to moderate volume range. The conventional single-left turn experiences a greater rate of increase in delays, reaching 80 sec/veh (LOS F) at a turning volume of just over 500 veh/hr. For the same volume the USC left turn is operating at about LOS C or 22 sec/veh. Moreover, the USC left turn will handle approximately 50% more turning traffic than the single left before it reaches a delay of 80 sec/veh. However, the USC left turn will fail much sooner than the dual left turns. **Table 5.5** summarizes the delay comparisons between the three scenarios.

Figure 5.7: Control Delay Comparison, Left Turns

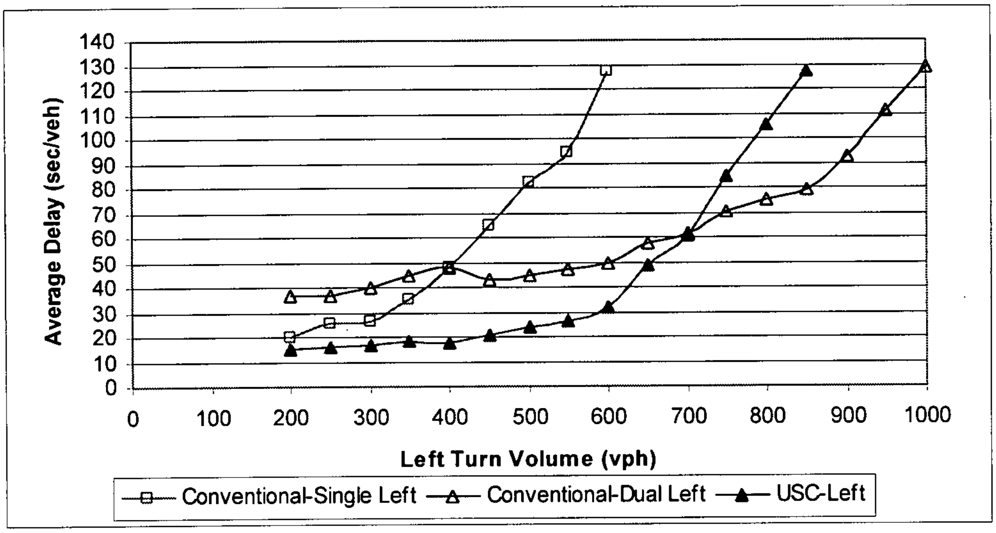


Table 5.5: Summary of Average Delay Comparison for Left Turns

	Single Left-Turn Lane Left	Dual Left-Turn Lanes Left	USC Left
Vol at LOS F (veh/hr)	490	860	740
Equivalent USC Delay (sec/veh)	44	128	-
Reduction in Delay by USC (%)	-45%	60%	-

When compared with the dual lanes of a conventional intersection the USC left turn performs better up to a certain level of traffic demand. In this example, the two plots intersect at about 700 veh/hr, which represents the demand volume for which adding a second left turn would equal the benefits of converting an intersection to a USC intersection. It is suspected that adding a second left turn lane to the USC intersection would greatly improve the turning capacity; and unlike the conventional intersection, opposing dual left turns could operate simultaneously in the same phase and would not require protected split phasing, since there would be no conflicts. However, the consequence of adding a second turning lane is increased road width and higher construction costs.

5.10 CONCLUSION

Since *Synchro* could not model the USC intersection as its true form and configuration, the intersection was instead modeled as a conventional "North American" intersection with left turns prohibited entirely at the primary intersection. For all intents and purposes the operation at the primary intersection is the same except that the USC left turns are now considered as right turns for the conventional intersection. In order to model the "crisscrossing" operation at the secondary intersections, two independent one-way links were used to join the two-way conventional link between the primary and secondary intersections.

The total delays experienced by vehicles driving through the entire USC intersection were estimated by combining and summing the individual delays, computed by *Synchro*, at each of the secondary and primary intersections. The delays were calculated over a range of traffic volumes. When compared to the conventional intersection, the USC intersection had a higher threshold for handling traffic. In terms of total intersection volumes the USC intersection was able to handle 15% to 20% more traffic, compared to the conventional intersection, before the average delays reached failure. Furthermore, for the same intersection volume at which the conventional intersection reached the failing level of service, the USC intersection was operating at a moderate level of service.

A separate analysis was done for left turns to investigate the effects of increasing left turn volumes. The three left turn scenarios included the conventional-single left, conventional-dual left and the USC left turn. The results showed that the left turns of the USC operated more efficiently at the low to moderate volume range. The USC left turn will handle approximately 50% more turning traffic than the single left scenario before it reaches failure. When compared with the dual lanes of a conventional intersection the USC left turn performs better up to a certain level of traffic demand. However, it is suspected that adding a second left turn lane to the USC intersection would greatly improve the turning capacity and reduce delays.

CHAPTER 6: SIMULATION MODEL ANALYSIS

6.1 INTRODUCTION

In addition to the HCM/Synchro analysis carried out in the previous chapters, a separate analysis has been carried out using a simulation-modeling program called VISSIM. This program is more flexible in that it can model unconventional movements, such as those under the USC scheme, but apply the same principals used in traditional traffic operation modelling.

PTV-Vision, a distributor of VISSIM in Canada, describes the program as a "microscopic, time step and behaviour based simulation model...[that is] a useful tool for the evaluation of various alternatives based on transportation engineering and planning measures of effectiveness".[7] Driver behaviour is incorporated in VISSIM, is based on the Wiedemann Traffic Model[8].

6.2 ANALYSIS METHODOLOGY

As in the previous chapters, both a 4-way conventional intersection and the USC intersection was analyzed and compared. Two scenarios were looked at for the conventional intersection, one with just a single left turn and the second with dual left turn lanes. The intersections consisted of two through lanes, a 50m left turn bay(s) and a 70m right turn bay on all approaches. A 70m acceleration lane for right turning vehicles was also used. The USC primary intersection that was modelled also had two through lanes and a 50m left turn bay on all approaches. The secondary intersections were placed at approximately 120m from the primary intersection and the crossover distance for the secondary intersections was approximately 50m.

The traffic conditions used in the analysis are summarized in **Table 6.1,** which shows the two different cases that were used in the HCM/Synchro analysis. For the first case (Case 1), the left turn volumes were held constant at 25% of the total left and through volumes. The volumes for all movements were then increased at the same rate. The second case (Case 2) held the through and right turn volumes constant while the left turn volumes were varied. The volumes selected for the through and right turn movements for Case 2 were considered moderate. To simplify the analysis, the same volumes were assigned to each approach and pedestrian traffic was ignored at the intersections for both Case 1 and Case 2.

Cycle lengths and signal timings were determined using the Sychro program. The volumes and lane configurations for each scenario were input into Synchro. The program was then used to optimize the cycle length and timing splits. The cycle lengths ranged from 60 sec to 120 sec for the conventional intersections and 40 to 60 for the USC. Cycle lengths and green times for the chosen traffic conditions are also shown in Table 6.1.

Table 6.1: Traffic Conditions Modeled

APPROACH VOLUMES				CYCLE LENGTH	
LEFT (veh/hr)	**THRU (veh/hr)**	**RIGHT (veh/hr)**	**(LEFT + THRU)x4**	**USC (sec)**	**Conventional (sec)**
CASE 1					
125	375	63	2000	40	60
188	562	94	3000	46	60
250	750	125	4000	46	65
313	937	156	5000	56	90
CASE 2					
100	500	100	2400	40	60
200	500	100	2800	40	60
300	500	100	3200	40	76
400	500	100	3600	46	90
500	500	100	4000	46	100
600	500	100	4400	46	100
700	500	100	4800	56	120
800	500	100	5200	60	-

For each scenario, ten runs were performed in VISSIM to determine the delay values. The program computes both stop delay and average delay. Stop delay is defined as the time each vehicle is in a standstill position. Average delay is defined as the total delay, which includes the delay due to acceleration and deceleration, as well as the stop delay. A summary of the VISSIM results is included in Appendix C.

The average delays were computed using start and end delay markers for the various movements. **Figure 6.1** shows the placement and description of each delay marker used for the conventional intersection example. The markers were placed as far upstream and downstream as possible to accurately capture delay data. The average delays computed between the delay markers will include all stop delays, acceleration and deceleration delays. A sensitivity analysis for the placement of the delay markers was conducted, which found that the distance of the delay markers from the intersection made minor difference in the delay results. Similarly, for the USC intersection the delay markers were placed as far upstream and downstream as possible from the intersection. **Figure 6.2** shows the locations of delay markers used in VISSIM for the USC intersection.

Figure 6.1: Delay Marker Placement for Conventional Intersection

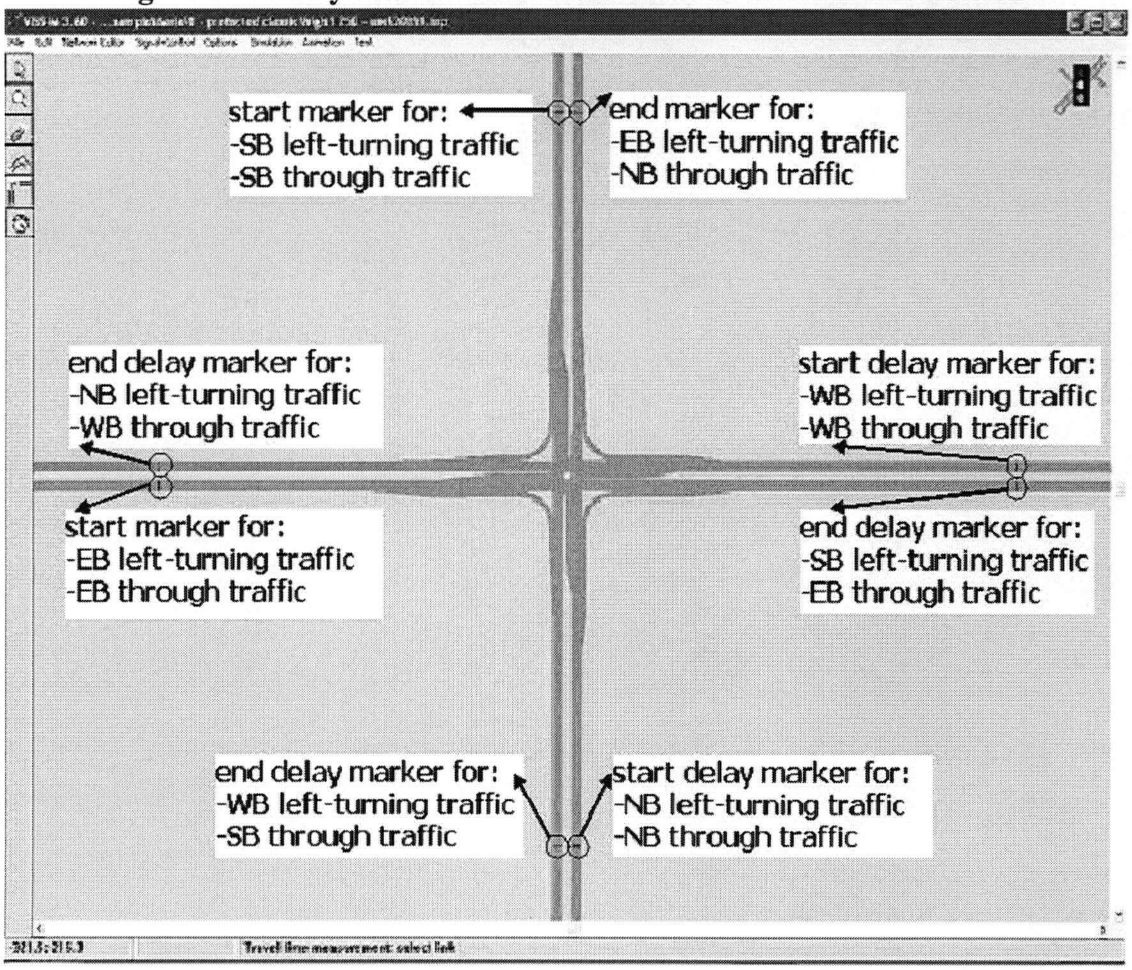
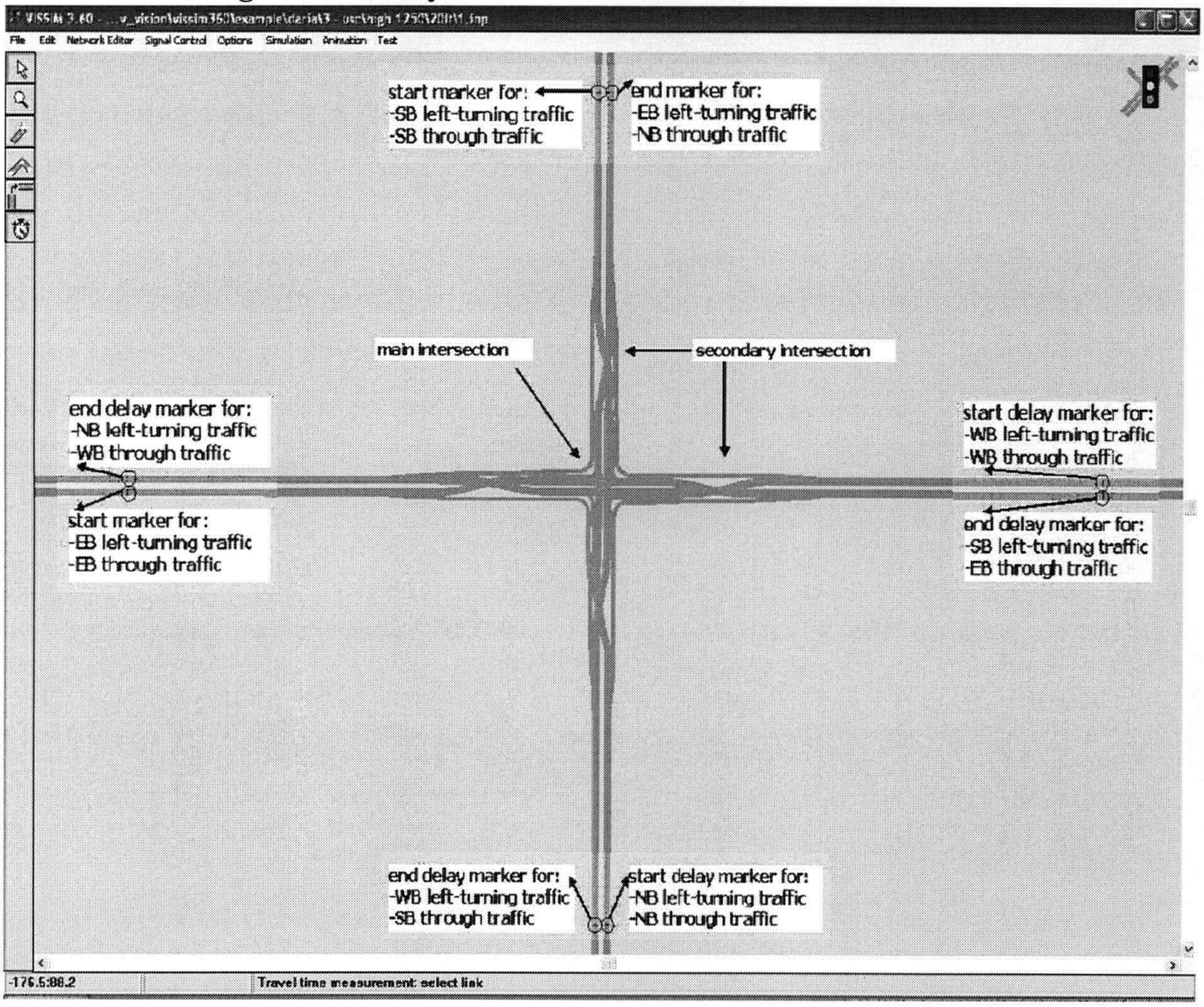


Figure 6.2: Delay Marker Placement for USC Intersection



6.3 RESULTS

6.3.1 Case 1

Figures 6.3 and 6.4 shows the results from the VISSIM analysis for Case 1, where the volumes for all movements were increased at the same rate. The delay results for left turn vehicles from Figure 6.3 shows that the conventional single-left intersection experienced less delay than either of the conventional dual-left and USC intersections for volumes (left + through) of up to 4000 veh/hr. At 5000 veh/hr the USC outperformed both the conventional intersections. Noteworthy is the significant increase in delay for the conventional single-left intersection from 4000 veh/hr to 5000 veh/hr, while the USC experienced little change over the same increment. This may indicate that the USC has a higher capacity for left turn vehicles and that it has the potential of handling more vehicles before it fails.

For the average delays for through traffic shown in Figure 6.4, the variances between the three types of intersections were insignificant. One might expect the USC to experience more delays for through traffic given the two additional intersections upstream and downstream of the main intersection. However, the results indicate that the operational performance of through vehicles would not be made worse in the USC scheme if the signals are adequately coordinated. That is, if the opposing green bands could be made to pass through all three intersections continuously, then the operation would be similar in principle to opposing green bands that pass through a single conventional intersection.

6.3.2 Case 2

The results for Case 2 are shown in Figures 6.5 and 6.6, where the delays are plotted against the various left turn volume increments. Note that the left turn volumes on the x axis are for one approach and that all four approaches were identical in volume and signal timing. For the left turn delays shown in Figure 6.5, the USC delays increased gradually over the range of left turn volumes, indicating that it could be less sensitive to the magnitude of left turns as compared to conventional intersections. On the other hand, the two conventional intersections experienced significant increases beyond the 600 veh/hr level. In fact, the VISSIM program experienced problems when modeling the conventional single-left intersection at the 800 veh/hr level and thus no data was obtained for this volume. The results for the conventional intersections are perhaps indicative of the significant stopped delays that can be experienced for higher volumes as a result of using multiple phases.

Figure 6.3: Case 1 - Left Turn Average Delay Results

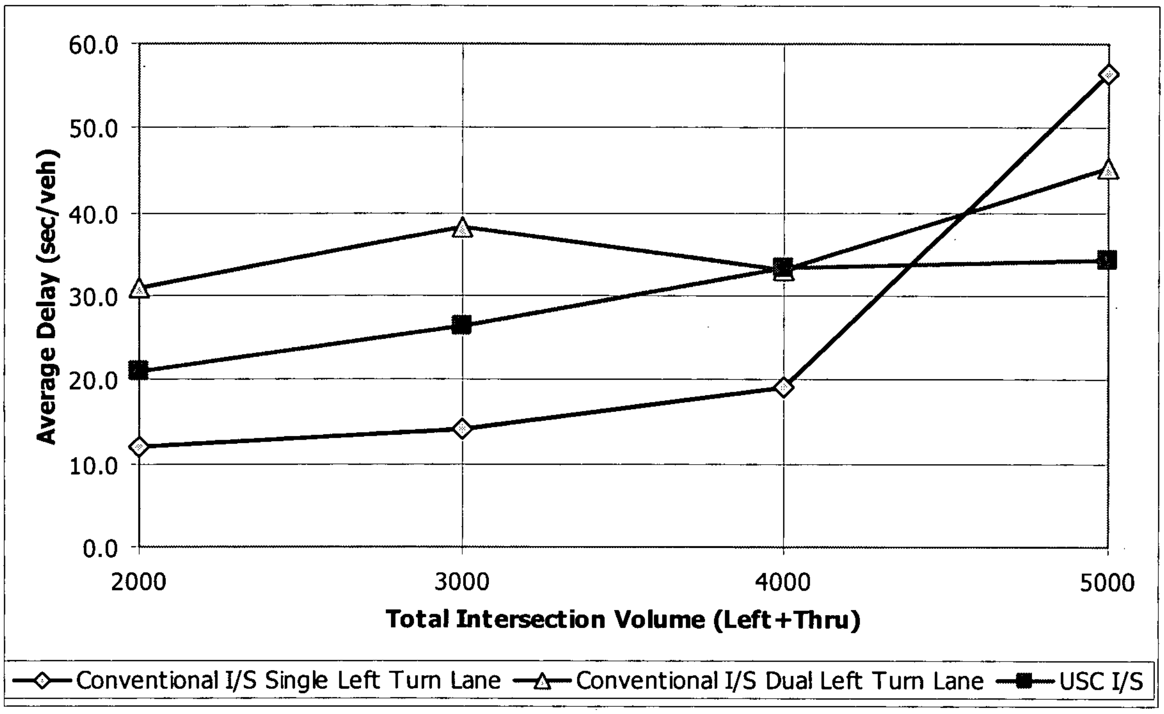


Figure 6.4: Case 1 - Through Traffic Average Delay Results

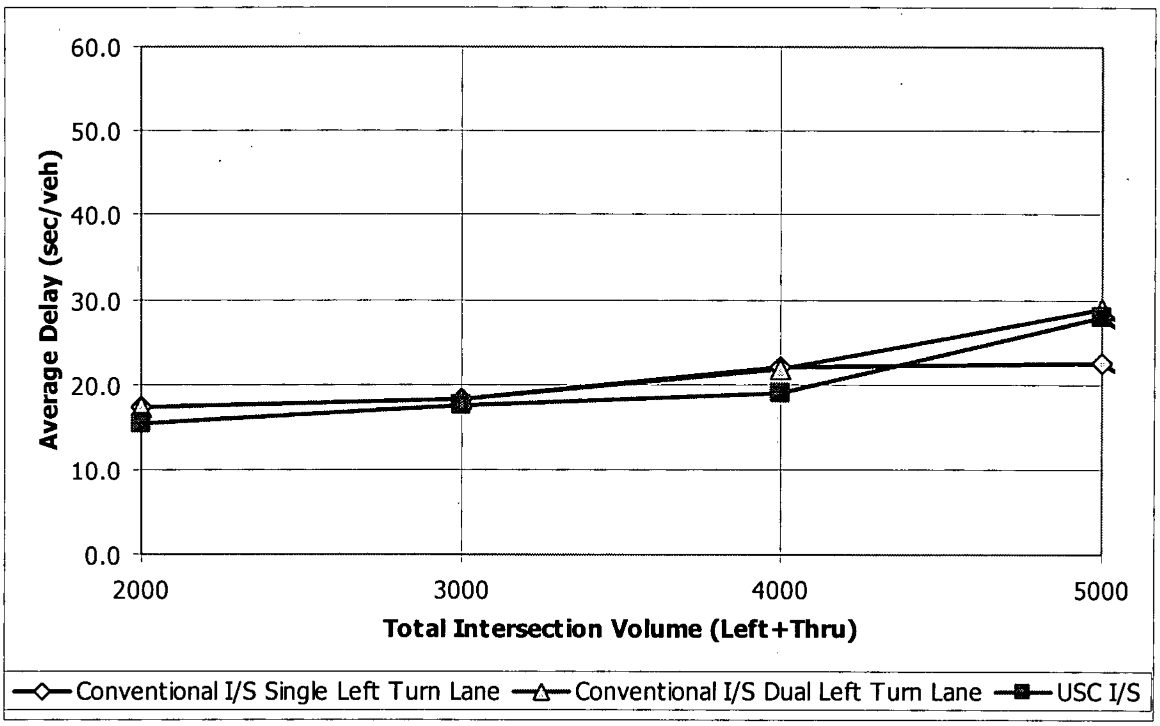


Figure 6.5: Case 2 - Left Turn Average Delay Results

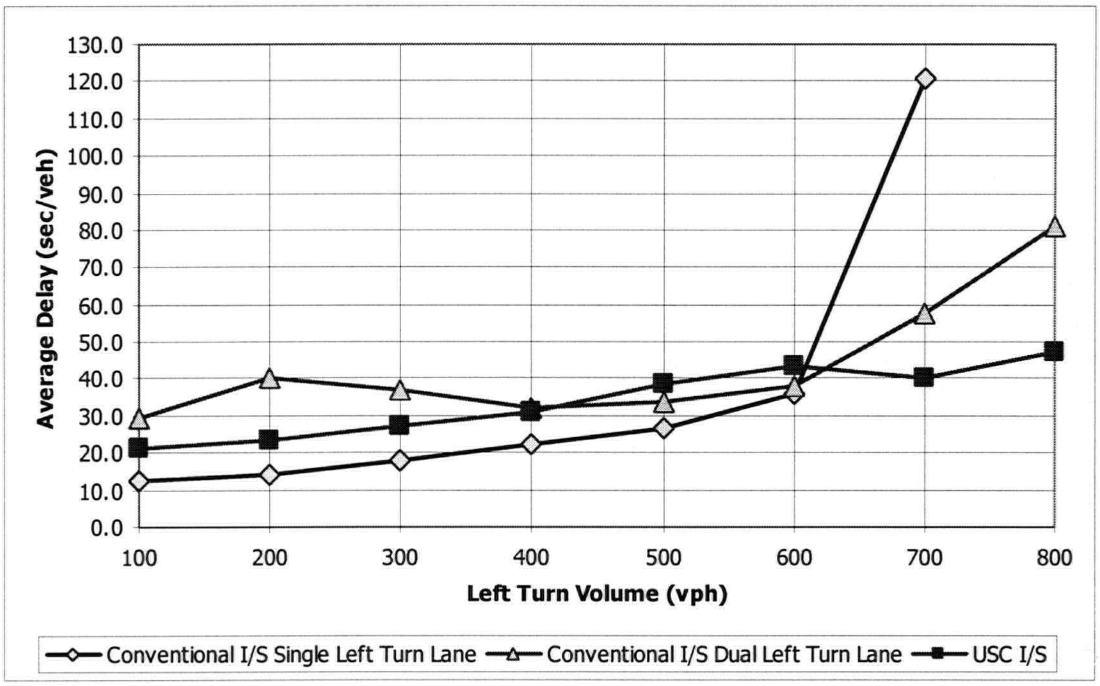
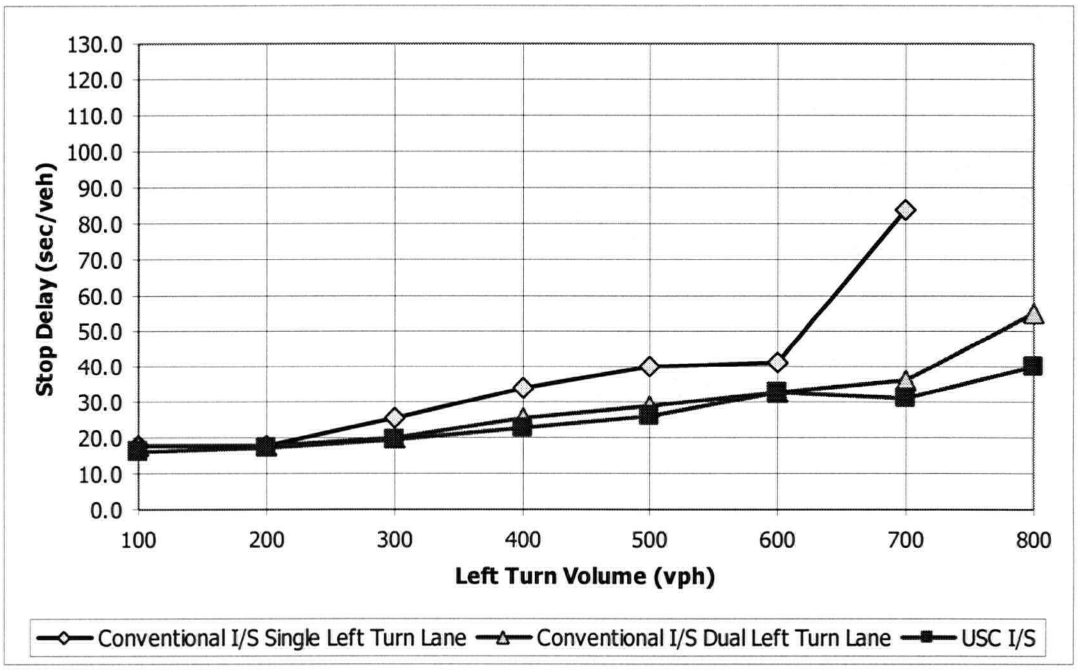


Figure 6.6: Case 2 - Through Traffic Average Delay Results



For the through movement delays, Figure 6.6 shows that there were no appreciable differences between the three intersection schemes over the same range of left turn volumes. The exception was for the conventional single-left, which experienced a significant increase in through delays when the left turn volumes were increased from 600 veh/hr to 700 veh/hr. Even for the conventional dual-left the rate at which the delays increased beyond the 600 veh/hr level was slightly greater as compared to the USC. The phasing used for the dual-left intersection employed simultaneous opposing left turn phases, which would typically require a larger area in order to accommodate the dual left turns. However, where right-of-way is a constraint the phasing for such intersections would not allow simultaneous turns, resulting in additional phases to stagger the dual left turns, which in turn would increase the overall delays for the intersection.

6.4 CONCLUSIONS

Unlike *Synchro*, VISSIM was able to model the USC intersection in its true form and configuration. This program was described as a microscopic, time step and behaviour based simulation model. As in the previous chapters, the three intersection scenarios (2 conventional and the USC) were analyzed and compared.

When analyzed using VISSIM, the USC shows promise in reducing average vehicle delays for intersections that experience higher traffic volumes. Compared to conventional intersections the USC shows less sensitivity to the magnitude of left turn volumes and shows the potential for accommodating larger turning volumes. Despite the additional intersections required and the crisscrossing manoeuvres associated with the USC, the analysis performed seems to indicate that the operational performance of through vehicles would not be made any worse than the conditions experienced by conventional intersections. In fact, for higher traffic volumes through traffic may experience less delay under this new scheme, which can be attributed to the simplified phasing and coordination between the primary and secondary signals.

For delays experienced by left turn vehicles, the analysis did not show significant improvements for the USC when compared to the conventional intersections. One might expect that left turns made without opposing traffic, such as those made at the primary intersection of the USC, would yield less delays. However, the problem may lie in the progression of left turn vehicles after the turn has been made. That is, signal coordination between the primary intersection and the secondary intersection that left turn vehicles approach after making the turn may not have been fully optimized. As a result, under the analysis settings the left turn vehicles may have experience additional delays associated with stopping and decelerating/accelerating. Subsequent analysis of the USC intersection should develop and utilize a more dynamic technique for optimizing the signals for both the through and left turn movements.

CHAPTER 7: SAFETY CONSIDERATIONS FOR USC INTERSECTION

7.1 INTRODUCTION

Safety is an important consideration when designing and implementing intersections. Although the USC intersection would be considered unconventional overall, some features and operating characteristics would still be considered conventional to which the same safety measures and principals can be applied. The expected safety performance of the USC intersection could be assessed by looking to collision experiences of conventional intersections and applying trends that would be applicable. For those features that are truly unique to this new scheme the safety risks and mitigating measures to improve safety would need to rely on engineering judgement. The following discusses the overall safety performance and the key safety issues associated with the USC intersection.

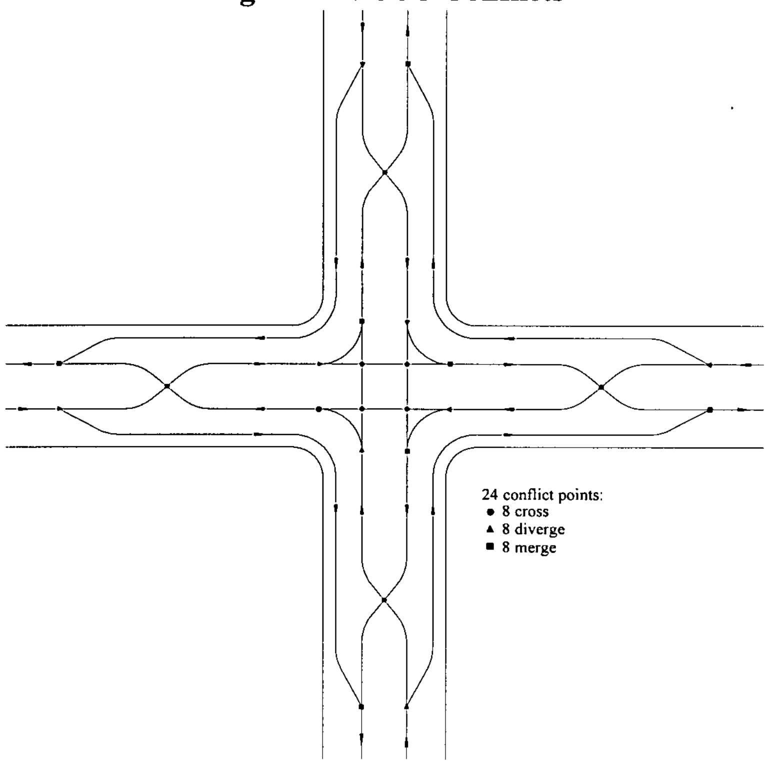
7.2 CONFLICTS

A major safety improvement offered by the USC intersection is the elimination of the left turn opposing (LTO) conflict. As illustrated in **Figure 7.1**, there are eight crossing conflicts associated with the USC intersection, which is a 50% reduction from the number of conflicts for the typical conventional intersection shown in Figure 2.2. The number of merging and diverging conflicts remains the same.

7.3 DRIVER WORKLOAD

The major drawback of the USC intersection would be that it could conceivably amplify driver confusion. The feeling of being on the wrong side of the road may make unfamiliar drivers uncomfortable and increase driver workload. Such a perception could lead to hesitation, sudden stops or sudden lane changes. Driving through the secondary intersection could also cause confusion especially if the experience makes drivers feel that they are crossing into the wrong side of the road. Therefore, it is critical that the geometry approaching the secondary intersection follows a continuous smooth alignment so that drivers will not feel they are deviating from their path.

Figure 7.1: USC Conflicts



7.4 COLLISION EXPERIENCE

The reduction in crossing conflicts should result in an overall reduction in collisions for the USC intersection. The elimination of the LTO conflict would be a significant safety improvement as the consequences of LTO collisions are often severe and sometimes fatal. In the province of British Columbia, LTO collisions accounted for about 20% of collisions at signalized intersections.[9] Signalized conventional intersections with permitted left turn operations are subjected to LTO collisions as drivers of left turning vehicles prematurely turn during the yellow phase in anticipating that the opposing vehicles will come to a stop. Conventional intersections that experience high LTO collisions are often restricted to protected left-turn phasing only or in some cases are prohibited entirely. A study of existing intersections by the Southeast Michigan Council of Governors[10] found that prohibiting left turns at an intersection resulted in a 40% reduction in collisions. A similar reduction could be assumed for the USC given the elimination of the LTO conflict and the displacement of the right turn.

One disadvantage of the USC intersection is that it introduces more stops given the additional secondary intersections. An increase in the frequency of stops may result in an increase in rear-end collisions. According to Ogden[11] the introduction of a traffic signal at a low speed intersection generally reduce collisions but has the tendency of increasing rear-end collisions by as much as 30% to 50%. It would be more difficult to estimate the incremental increase in rear-end collisions associated with a USC intersection over a conventional intersection. It can be argued that for through vehicles the much simpler phasing at the primary intersection would produce fewer stops than a conventional intersection with multiple phasing, and the reduction could be further amplified if good progression is achieved through the primary intersection. As such, the majority of the additional stops experienced at a USC intersection would likely be attributed to left turn vehicles, which was shown to have less favourable progression in the simulation discussed in Chapter Six. However, developing and implementing proper signal coordination is the key to reducing the number of stops through the USC intersection and thus minimize the occurrence of rear-end collisions.

Driver confusion and increased driver workload could further contribute to the expected collisions for the USC. Driver hesitation could lead to unnecessary vehicle deceleration, which in turn could lead to more rear-end collisions.

At the secondary intersections, the resulting angle between the opposing alignments would likely be acute (less than 70 degrees) in order to minimize the road cross section. This acute angle creates longer crossing distances for vehicles to clear the intersection. Intersections with long clearance times are often subject to red light violations, which in turn may lead to crossing collisions.

7.5 COLLISION MITIGATION

Given the potential for driver confusion, visual cues should be provided for the USC such that drivers would not feel they are on the wrong side of the road. A physical obstruction such as a fence or shrubbery could be placed in the median approaching the primary intersection, which would prevent drivers from seeing vehicles on the other side of the road. The objective would be to make drivers feel they are driving on an independent roadway, which is similar to a one-way road.

Another cause for confusion at the primary intersection would be for drivers turning left who are accustomed to making a conventional left turn around the central median. In order to prevent such an error, it would be necessary to extend the central median and restrict left-turning vehicles from driving past the median where they may turn left onto oncoming traffic. It would be prudent to provide positive signing and visual cues that give the sense that the driver is in the correct path.

Pedestrian safety would also have to be addressed given the unconventional nature of the USC intersection. Pedestrians have certain expectations of vehicle operation at intersections and where hazards normally take place. Pedestrians could instinctively look in the wrong directions when crossing the primary intersection since the overall orientation and the pedestrian manoeuvres of the unconventional intersection would be very similar to those of a conventional intersection. Thus, in order to minimize pedestrian confusion and prevent errors, informational and warning signs should be put into place.

7.5 CONCLUSIONS

A major safety improvement offered by the USC intersection is the elimination of the left turn opposing (LTO) conflict, which results in a 50% reduction of conflicts from a typical conventional intersection. The major drawback of the USC intersection would be driver confusion and increased driver workload due to the perception of being on the wrong side of the road. It is critical that the geometry approaching the secondary intersection follows a continuous smooth alignment so that drivers will not feel they are deviating from the correct path.

The elimination of the LTO conflict would be a significant safety improvement as the consequences of LTO collisions are often severe and sometimes fatal. A 40% reduction in collisions could be expected for the USC, compared to a conventional intersection, given the elimination of left turns at the primary intersection.

One disadvantage of the USC intersection is that it introduces more stops given the additional secondary intersections, which could increase rear-end collisions. Although it may be difficult to estimate the increment increase in rear-end collisions, over and above those normally experienced at conventional intersections, it is likely that the increase in rear-end collisions would be attributed more to left turn vehicles due to unfavourable progression expected for this movement. However, developing and implementing proper signal coordination is the key to reducing the number of stops through the USC intersection and thus minimize the occurrence of rear-end collisions.

In mitigating the effects of driver confusion, a physical obstruction such as a fence or shrubbery could be placed in the median approaching the primary intersection, which would prevent drivers from seeing vehicles on the other side of the road. The objective would be to make drivers feel they are driving on an independent roadway, which is similar to a one-way road. At the primary intersection, it would be necessary to extend the central median and restrict left-turning vehicles from driving past the median where they may turn left onto oncoming traffic. To minimize pedestrian confusion and prevent errors, informational and warning signs should be put in place specifically for pedestrians.

CHAPTER 8: DESIGN AND IMPLEMENTATION

8.1 INTRODUCTION

The previous chapters have shown that there are some advantages of the USC intersection over the conventional designs. The next step is to develop a design and a plan to implement the intersection. This chapter discusses some of the key considerations for designing and implementing the USC intersection.

8.2 TRAFFIC CONDITIONS

The USC intersection would be best suited for intersections with heavy left turn traffic volumes. As discussed in the previous delay analysis, the delays for left turn traffic could be reduced significantly while at the very least maintaining the level of service for through traffic at acceptable levels. The intersection would ideally handle heavy left turn volumes in all directions given its symmetric operation. Such conditions may exist at intersections in the heart of a central business district where there may not be a distinct peak direction of traffic.

Existing intersections that are already experiencing heavy volumes and are on the verge of failing could be converted to an USC intersection. This would be an alternative to increasing the number of lanes through the intersection to increase capacity.

8.3 ROADWAY CONDITIONS

An ideal location for this unconventional intersection would be at the crossing of major urban arterials with low to moderate posted speed requirements. Travelling on these types of roadways is characterized by regular stops at intersections, which is the main reason for enforcing low/moderate speed regimes. Given the greater frequency of stoppage expected for the USC intersection, the experience would not be out of the norm for drivers who are already accustomed to such conditions. The scenario would be similar to those at a roundabout intersection where drivers are not permitted to drive directly through the intersection and therefore must reduce speeds when approaching the intersection.

Placing this type of intersection on high speed arterials would create safety issues given that the USC intersection does not allow through traffic to run continuously for long periods at a time, which is characteristic of intersections on high-speed roadways. Giving this movement signal priority and more green time minimizes stoppages of through traffic on these roadways. Thus drivers expect to stop less often while travelling on these roads, which is an expectation that the USC intersection would not be conducive to.

One disadvantage of providing the right turn bypass is it limits access to any property adjacent to the intersection. Any vehicles accessing a corner property would do so through the right turn bypass only, which means it, would only be accessed from one direction. Egress from the property would also be limited to only one direction. Consequently, adjacent properties that require no vehicle access (driveways) would be a good candidate for the USC intersection. An example of such an intersection would be in the central business zones where vehicle access is often provided in the back alleys instead of directly off the main streets.

8.4 LOCATING SECONDARY INTERSECTIONS

The location of the secondary intersections relative to the primary intersection will effect the progression and throughput of the intersection. The closer they are to the primary intersection the easier it would be to coordinate the signals. However, there would be fewer throughputs with the shorter distance between the signals since this would limit the length of the platoon that travels through the entire USC intersection. The governing principal is that a platoon travelling in one direction should reach the downstream secondary intersection when the tail end of the opposing platoon has just crossed the same secondary intersection (see Figure 4.2). Therefore, the amount of green time that would be allocated at each secondary intersection for one direction would be more or less equal to the time it takes for the opposing platoon to travel between two secondary intersections. The shorter the travel time, as a result of a shorter distance between the primary and secondary intersection, the less green time could be provided.

On the other hand, the farther the intersections are spaced the greater the chances of queues developing at the intersections since it would be more difficult to coordinate the intersections and maintain good progression. To maximize throughput with the greater spacing, longer green times would have to be provided to correspond with the longer travel time between secondary intersections. However, longer green times would mean longer cycle lengths and, as discussed earlier, intersections with excessive cycle lengths result in increased delays and so cycle lengths should be minimized as much as possible. If shorter cycle lengths are used in combination with a long spacing between the primary and secondary intersections, not all of the platoon that leaves one secondary intersection would be able to clear the downstream secondary intersection. This would result in the formation of queues at the downstream secondary intersection.

Consequently, an optimum spacing of the secondary intersections could be achieved which would be dependent upon the desired or optimum cycle length for the anticipated traffic volumes and the average speed that vehicles travel between secondary intersections. From Figure 4.2, L is the distance from the primary intersection to the location where the tail end of the approaching (towards the primary intersection) green band intersects with the front of the opposing departing green band (away from the primary intersection). Using the time-space diagram in Figure 4.2, an equation can be derived for L by solving two linear equations that represent the leading and trailing edges of the opposing bands. The resulting equation for L is in terms of average travel speed, desired green time and the width of the primary intersection. The equation is as follows:

$$L = 0.5 \left[\frac{g_a v}{3.6} - W_p \right] \qquad (8.1)$$

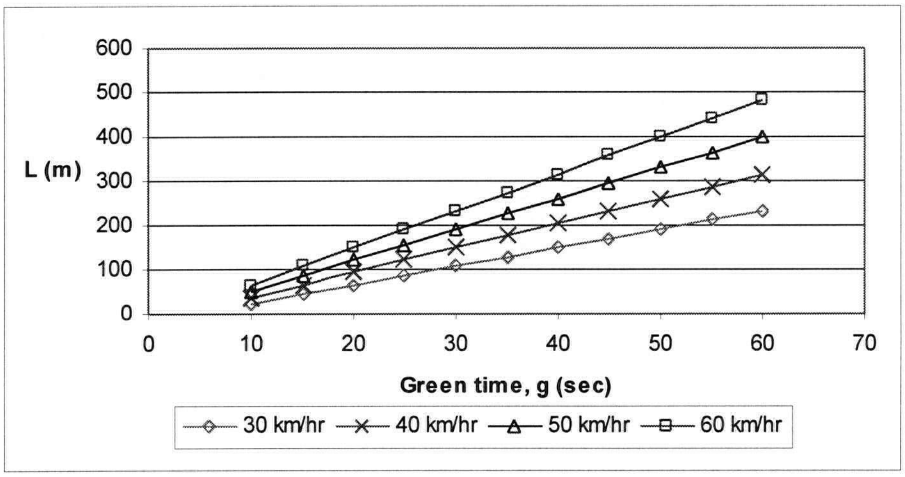
where

L = distance of secondary intersection (m);
g_a = green time/band (s);
v = average travel speed (km/hr);
W_p = width of primary intersection (m).

Values for L can be calculated over a range of green times and speeds as illustrated in **Figure 8.1**. The width of the primary intersection, W_p, was measured at about 35 m for the sample intersection and held constant for this exercise. The values obtained from Equation 8.1 are oversimplified and should only be used as a guide when locating the secondary intersections. Other factors, such as vehicle acceleration and headways should be taken into account when deriving more accurate travel times between intersections.

The location of secondary intersections would be affected by upstream conditions and whether or not there are conventional intersections upstream of the USC intersection. Furthermore, converting a conventional intersection to a USC intersection may not be possible if adjacent intersections are spaced too close together, such as the case in some urban downtown areas. If space is limited, then shorter green times should be selected to maintain good progression through the intersections. However, short green times would likely reduce the capacity of the secondary intersections with larger traffic volumes and could produce lengthy queues. Therefore, in such cases where secondary intersections must be placed fairly close to the primary intersection (due to limiting upstream conditions) progression through the USC intersection would likely be compromised for the sake of upstream conditions. The consequence would be the queuing of some vehicles at the primary intersection as not all vehicles in the platoon would make it through.

Figure 8.1: Optimum Secondary Intersection Offsets



8.5 GEOMETRIC DESIGN FEATURES

8.5.1 Cross-Section

Standard lane widths, 3.5 m to 3.7 m, would be used for the lanes of the USC intersection. Slightly narrower lanes could be used for the left turn lanes as commonly found in conventional intersections. Additional lane widening would be required along curved sections, such as along the right turn bypass lane at the primary intersection, to handle larger design vehicles.

A width of 2.0m could be used for the central raised median at the approach to the primary intersection. A narrower width could be used at this location given that there would be no need to develop a turning lane in the median as in conventional intersections, where the raised median at its widest point can be 4.0 m or wider. The central median should be wide enough to accommodate any landscaping or fencing required for visual obstruction as discussed earlier. A width of 1.0 m could be used as a minimum for the secondary raised medians that separate the right turn bypass lanes from the main traffic.

Upstream of the secondary intersection, the central median should be wider to provide enough separation between the two opposing alignments such that the alignments would cross at an appropriate angle. The greater the angle between the two alignments the better the design will be in terms of sight distance and safety. A width of 4.0 m or wider would be appropriate for the central raised median at this location.

Compared to a conventional intersection, the overall area of the USC intersection would be slightly greater due to the three sets of raised medians at each approach. **Figure 8.2** shows the additional area that would be required for the USC intersection to achieve the same number of lanes as a conventional intersection. The typical layouts shown in Figure 2.1 and Figure 5.1 above were superimposed to illustrate the difference in areas. The additional width required is equivalent to about a lane width (3.6 m), which is attributed to the presence of the right turn bypass lane on the left side of the roadway (lane leading away from the intersection). This lane also provides an opportunity for vehicles to accelerate before merging back into traffic. An acceleration/merge lane is commonly found in conventional intersections, although the example shown in Figure 2.1 does not show these lanes. The use of such lanes are usually dictated by the speed of the through vehicles and available sight distance for the turning vehicles. As such, the difference in areas would be negligible if a comparison is made to a conventional intersection with an acceleration lane.

On the right side of the roadway, additional area would be required in order to develop the right turn bypass lane (approaching the intersection) further back, prior to the secondary intersection. The development of the right turn lane for a conventional intersection is normally achieved within 50 to 100 m of the intersection. For roads with higher design speeds, a long parallel lane and taper is required for proper deceleration of vehicles that are turning right at the intersection, which could have a combined length of up to 200 m. Again, given such scenarios the additional area shown in Figure 8.2 would be within the realm of intersection design and would not be considered significant.

8.5.2 Primary intersection Geometry

Given the unconventional left turn at the primary intersection, it would be necessary to create a design that will minimize driver error. As previously proposed, the central median could be configured (extended) to prevent left turning vehicles from entering the wrong side of the road. Proper radii at the corners should be used to accommodate the turning paths of various types of design vehicles. In addition to this feature, the left turn lane could be deflected away from the through lanes, as shown in **Figure 8.3**, to improve the approach to the turn and provide positive guidance for drivers.

8.5.3 Secondary Intersection

The key element in the geometric design of the USC intersection is the geometry at the secondary intersections. As previously discussed, the crossing of vehicles should be made as smooth as possible to minimize driver hesitation and confusion. To achieve this, proper horizontal alignments with appropriate design elements should be employed, such as those illustrated in **Figure 8.4**. The centerline alignments shown in this figure consists of reverse curves. The radius of the curves would reflect the desired design speed. Ideally, the sections of the alignments through the intersection would consist of a straight tangent; however, to achieve this would require a wider cross section to accommodate the geometry. By placing curves through the intersection pulls the alignments closer to the center of the intersection and thus minimizes the width of the roadway allowance required.

Figure 8.2: Additional Widening

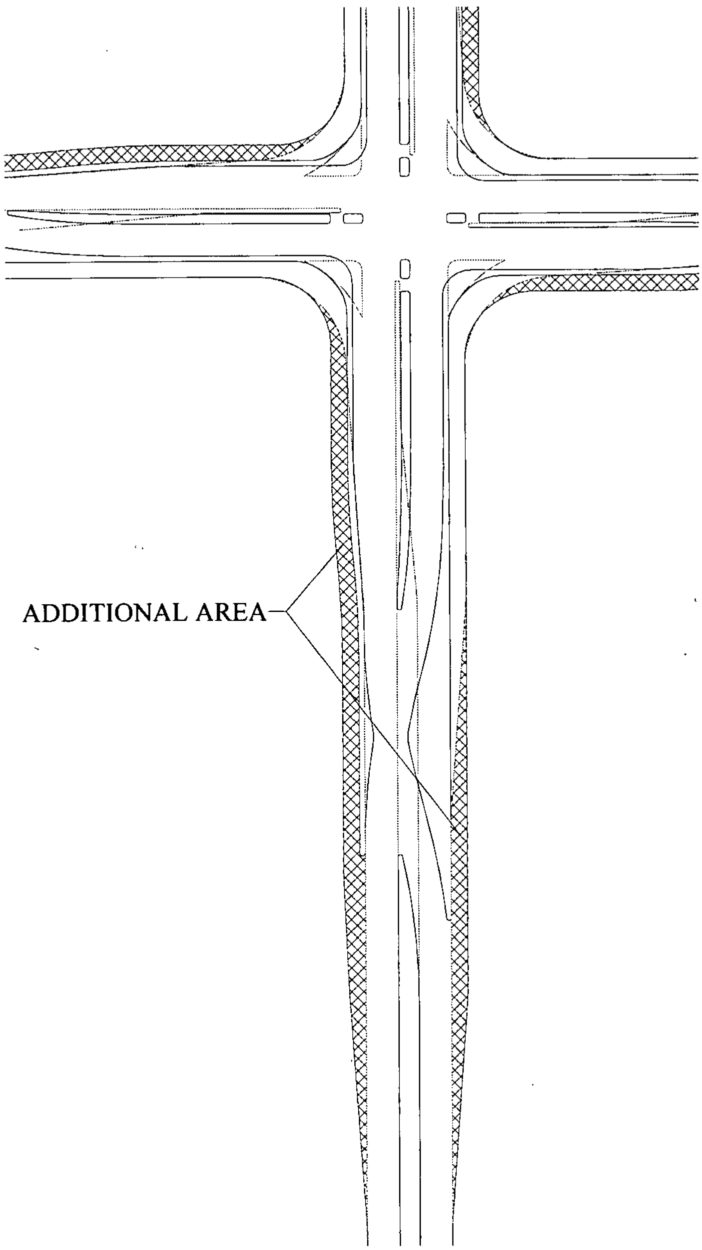
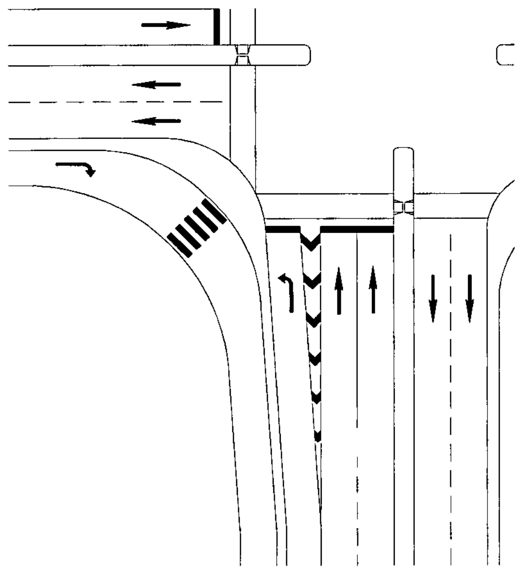


Figure 8.3: Primary Intersection Geometry



The approach alignment, upstream of the secondary intersection, should be deflected away from the center of the road so that it is not parallel with the opposing departure alignment. This would reduce head light glare between opposing vehicles as they approach the intersection. It also improves the orientation of vehicles crossing the intersection from the approach alignment, giving drivers better sight lines and judgement towards crossing vehicles.

Due to the desired curved alignment through the intersection, it would be prudent to install road delineation to guide drivers along the curved alignment. As shown in **Figure 8.5**, dashed pavement markings could be used for this purpose. This is commonly done at intersections with approaches that are offset to one another and also for dual left turn lanes to aid drivers in staying on the correct path.

Figure 8.4: Secondary Intersection Geometry

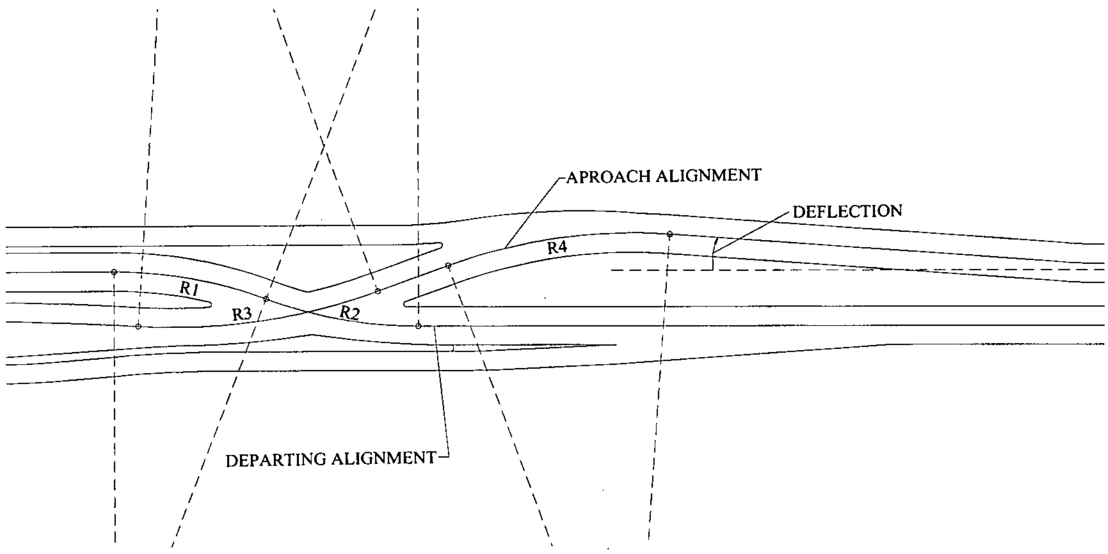
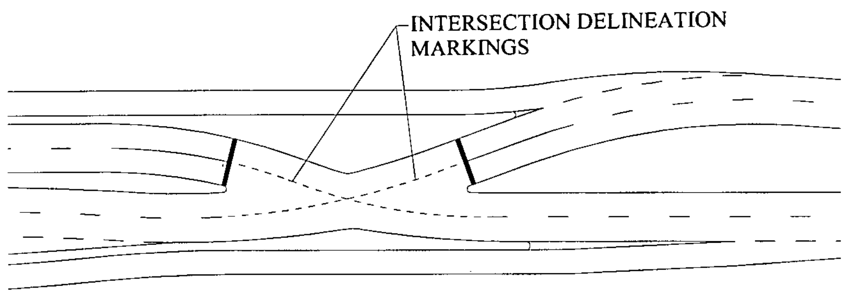


Figure 8.5: Secondary Intersection Pavement Markings



Normally, horizontal curves are super-elevated as per design guidelines to achieve vehicle stability and reduce the occurrence of vehicles driving off the road. However, when curves are located at the intersection, it is not always possible to achieve positive super-elevation in one direction as it would conflict with the super-elevation requirements of the alignment of the intersecting road. Also, drainage of the intersection, and the need to drain water away from the intersection, often dictate the cross-fall and grades at the intersection. Consequently, it would not be uncommon to have negative or adverse super-elevation through intersections.

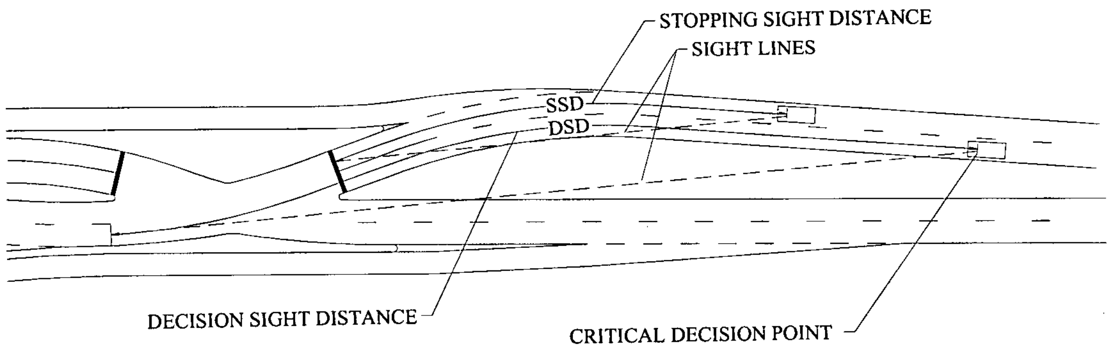
The important criteria to follow when designing the cross-fall through the intersection would be the maximum adverse (or negative) super-elevation that is acceptable for a given radius. Design guidelines, such as TAC, recommend maximum adverse super-elevation based on vehicle speed, radius and pavement friction. For the alignment shown in Figure 8.4, the cross-fall through curves R2 and R3 would likely be adverse for which a maximum cross-fall rate would be used.

Another important criterion would be the rate of change of the cross-fall through the intersection. Design guidelines specify maximum rates given the design speed and width of the travel lane(s). At intersections, however, higher rates (i.e. cross-fall changes more rapidly) are normally accepted due to the expectations that drivers have driving through intersections. That is, drivers are normally accustomed to rapid changes in cross-fall when crossing an intersection.

Sight distance is another important criterion when designing intersections. At intersections, sight distance must be considered for vehicles approaching the intersection and departing from the stopped position. Intersection sight distance is adequate when it permits vehicles to safely make all the manoeuvres at an intersection without significantly affecting vehicles travelling on the main roadway. For approaching vehicles, stopping sight distance (SSD) is usually the minimum criterion used, which is normally based on design speed and design vehicle. However, it is often desirable to use decision sight distance (DSD), which is a greater distance than stopping sight distance, due to the complex situations that are typically encountered by drivers at intersections. Drivers must make instantaneous decisions before entering an intersection and sufficient sight distance should be provided to minimize conflicts.

The unconventional crossing of vehicles at the secondary intersection could cause drivers to hesitate and thus adequate sight distance should be provided for vehicles approaching the intersection. By the time a vehicle enters the intersection, the driver should have a clear indication of the correct manoeuvre to exit the intersection. The minimum DSD would be measured from the far side of the intersection, as shown in **Figure 8.6**, since it would be the location the driver of the approaching vehicle would need to see in advance to correctly exit the intersection. Overhead traffic signals placed at the far side of the intersection would aid drivers in recognizing the correct path and would increase the available sight distance. Given the curved alignment of the approaches, the median should be free of obstructions within the critical sight lines.

Figure 8.6: Approach Sight Distance



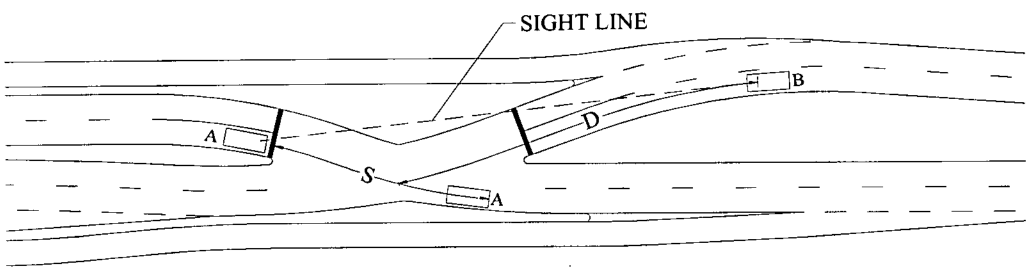
For vehicles stopped at the secondary intersection waiting to depart, the applicable sight distance criterion is crossing sight distance. That is, drivers of these stopped vehicles must be able to see oncoming vehicles from the cross street such that the driver would have sufficient time to make the decision and cross the intersection without impeding the oncoming vehicle. Although these situations exist primarily at stop-controlled intersections, it is also used for signal-controlled intersections since a malfunction of the signals could occur or operate in a flashing mode for which the intersection would function as stop control. **Figure 8.7** illustrates the departure sight distance for a vehicle at the stop line crossing the intersection. The sight distance is measured from the conflict point between the two vehicles to the approaching vehicle. The distance D represents the distance travelled by the approaching vehicle that is equal in duration to the time that the crossing vehicle takes to clear the intersection, represented by distance S. The driver of the departing vehicle would need an unobstructed sight line to the approaching vehicle at distance D. This is often referred to as the sight triangle. Equation 8.2 shows the relationship between the two distances.

$$D = \frac{V\,(P+t)}{3.6} \qquad (8.2)$$

where

D = departure sight distance for vehicle A (m);
S = distance travelled to cross intersection (m);
V = design speed (km/hr);
P = perception reaction time, 2 seconds;
t = time to cross intersection, distance S (s);

Figure 8.7: Departure Sight Distance



8.6 CONCLUSIONS

The USC intersection would be best suited for intersections with heavy left turn traffic volumes. The intersection would ideally handle heavy left turn volumes in all directions given its symmetric operation. Existing intersections that are already experiencing heavy volumes and are on the verge of failing could be converted to an USC intersection. An ideal location for this unconventional intersection would be at the crossing of major urban arterials with low to moderate posted speed requirements. Travelling on these types of roadways is characterized by regular stops at intersections and the experience would not be out of the norm for drivers who are already accustomed to such conditions. One disadvantage of providing the right turn bypass is it limits access to any property adjacent to the intersection.

The location of the secondary intersections relative to the primary intersection will effect the progression and throughput of the intersection. The closer they are to the primary intersection the easier it would be to coordinate the signals. The farther the intersections are spaced the greater the chances of queues developing at the intersections since it would be more difficult to coordinate the intersections and maintain good progression. The optimum spacing of the secondary intersections could be achieved which would be dependent on the desired or optimum cycle length for the anticipated traffic volumes and the average speed that vehicles travel between secondary intersections.

Compared to a conventional intersection, the overall area of the USC intersection would be slightly greater due to the three sets of raised medians at each approach. The additional width required is equivalent to about a lane width, which is attributed to the presence of the right turn bypass lane on the left side of the roadway. In reality, the difference in areas would be negligible if a comparison is made to a conventional intersection with acceleration lanes.

Given the unconventional left turn at the primary intersection, it would be necessary to create a design that will minimize driver error. The central median could be configured (extended) to prevent left turning vehicles from entering the wrong side of the road and the left turn lane could be deflected away from the through lanes.

The key element in the geometric design of the USC intersection is the geometry at the secondary intersections. The crossing movements should be made as smooth as possible to minimize driver hesitation and confusion. The approach alignment to the secondary intersection should be designed such that head light glare between opposing vehicles is minimized and that adequate sight distance for all critical movements be achieved.

CHAPTER 9: CONCLUSIONS

In an attempt to improve performance and reduce congestion problems at intersections, transportation engineers in the past have turned to conventional measures such as optimizing signal timings and coordination, adding lanes or restricting movements at intersections. The impact of left turns on operation is probably the most significant factor in assessing the level of performance of conventional intersections. As a result, engineers have looked to alternative measures for dealing with left turns at intersections. This thesis discussed some of the more notable unconventional schemes that have already been developed. The focus of this thesis was to introduce a new unconventional intersection scheme entitled Upstream Signalized Crossover (USC) intersection, which has been conceived by the author. The functional and operating characteristics of this new intersection scheme were discussed. Some of the key features of the USC intersection include:

- The left and through movements are symmetrically crossed over to the left side of the road for both cross roads. The result is the elimination of the left turn opposing conflict;
- The resulting configuration has introduced four additional secondary intersections that are also signalized;
- Right turns are bypassed and operate independently of the signalized intersections; and
- All five signals can operate under a two-phase cycle and coordinated.

The USC intersection was analyzed using both the Highway Capacity Manual (HCM) 2000 methodology through the Synchro program and a simulation model called VISSIM. A sample conventional intersection was also analyzed so that a direct comparison could be made with the USC intersection. Some of the key findings and conclusions from the analysis were:

HCM Analysis:

- A reversed configuration of the USC intersection had to be used in Synchro in order to obtain meaningful results;
- Good progression between the secondary and primary intersections was achievable for the through movements;

- For the analysis performed over a range of Total Intersection Volume (left turns held as constant percentage), the average delay for left turns through the USC was about 60% less for the same volume that caused the conventional intersection with single left turn lanes to fail. The reduction in average delays was about 45% for the through movements. The USC was able to accommodate between 15% and 20% more traffic before either of the left and through movements failed;
- When compared to the conventional intersection with dual-left turn lanes the reduction in average delay for left turns achieved by the USC was about the same (59%) at the failing point for the conventional intersection. However, for the through movements the USC performed equally well up to a certain volume and thereafter the delays increased more rapidly causing the USC through movement to fail before the through movements of the conventional intersection;
- For the analysis performed over a range of Left Turn Volume (through traffic held constant), a 45% reduction in average delay for left turns was achieved by the USC at the same volume that caused the conventional single left turn to fail. The USC was able to accommodate about 50% more traffic before the left movement failed;
- When compared to the dual-left scenario the USC performed better up to a certain left turn volume (700 veh/hr) and thereafter experienced a more rapid increasing rate in delays. It was noted that adding a second left turn lane for the USC would likely improve performance.

VISSIM Analysis:

- This program was able to model the USC in its true form and configuration;
- The cycle lengths required were less than 60 seconds and did not vary significantly between the different levels of turning volumes;
- The USC shows promise in reducing average vehicle delays for intersections that experience higher traffic volumes. Compared to conventional intersections the USC shows less sensitivity to the magnitude of left turn volumes and shows the potential for accommodating larger turning volumes.
- Despite the additional intersections required and the crisscrossing manoeuvres associated with the USC, the analysis performed seems to indicate that the operational

performance of through vehicles would not be made any worse than the conditions experienced by conventional intersections;

- For delays experienced by left turn vehicles, the analysis did not show significant improvements for the USC when compared to the conventional intersections. The problem may lie in the progression of left turn vehicles after the turn has been made;
- Subsequent analysis of the USC intersection should develop and utilize a more dynamic technique for optimizing the signals for both the through and left turn movements.

Safety considerations and implications of the USC intersection were discussed in this thesis. Some of the key points were:

- A major safety improvement offered by the new scheme is the elimination of the left turn opposing conflict and as a result a 40% reduction in collisions could be expected when compared to a conventional intersection with left turns;
- Due to the increased number of stoppages, rear-end collisions are likely to increase for the USC intersection. The increase would be attributed more to left turning vehicles;
- One major challenge of the USC intersection is driver confusion given the unconventional manoeuvres. The geometry, physical features and visual cues, signing and pavement markings will all play an important role in minimizing driver confusion and improving safety.

The USC would be best suited for intersections with heavy left turn volumes and moderate to heavy through volumes. An ideal location would be at the crossing of major urban arterials with moderate operating speeds. The USC could be installed as an alternative to roundabouts given the lower approach speed requirements and also since it would require less area to construct. The USC would not be recommended for high-speed roads and expressways nor would it be suited at intersections requiring full access to the corner properties.

The additional area required to construct the USC intersection compared to a conventional intersection would not be significant. However, the cost of installing four additional signals and other physical features associated with the intersection would be a major factor in the implementation of the intersection. The costs of such items were not addressed in this thesis nor were the benefits of any savings in delays estimated. It would be prudent to carryout subsequent work and perform a cost-benefit analysis for this intersection in order to realize the true benefits of this new scheme.

BIBLIOGRAPHY

1. *Geometric Design Guide for Canadian Roads,* Transportation Association of Canada, 1999.
2. *Highway Capacity Manual,* 2000.
3. Synchro 5 Traffic Signal Coordination Software, Trafficware, 1993-2001
4. Mucsi, K.; Khan, M., *Effectiveness of Additional Lanes at Signalized Intersections*, Institute of Transportation Engineers Journal, January 2003 Issue.
5. Hummer, J. E., *Unconventional Left-Turn Alternative for Urban and Suburban Arterials – Part One*. Institute of Transportation of Engineers Journal, September 1998 Issue.
6. Jagannathan, R.; Bared, J. G., *Design and Operational Performance of Crossover Displaced Lefr-Turn (XDL) Intersections (Also Called Continuous Flow Intersection (CFI))*, TRB 2004 Annual Meeting CD-ROM.
7. VISSIM 3.60, PTV America Inc., User Manual, page 1-2 and page 5-4.
8. VISSIM 3.60, PTV America Inc., User Manual, Section 5.2 - Driving Behaviour, The Wiedemann Traffic Model in VISSIM, page 5-4.
9. Insurance Corporation of British Columbia, *Traffic Collision Statistics: Police-Attended Injury and Fatal Collision*, 2002.
10. Southeast Michigan Council of Governors (SEMCOG), *SEMCOG Traffic Safety Manual*, Second Edition, September 1997, pare B-6.
11. Ogden, K. W., *Safer Roads A Guide to Road Safety Engineering*, 1996, page 143.

APPENDIX A: SAMPLE SYNCHRO PRINTOUTS

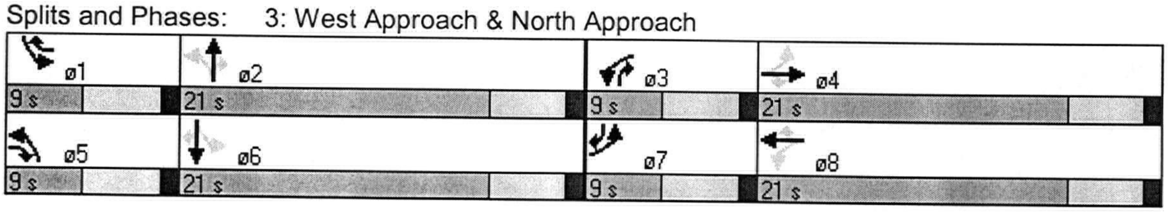
Conventional Intersection — Single Left - 2000 veh/hr

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑↑	↱	↰	↑↑	↱	↰	↑↑	↱	↰	↑↑	↱
Volume (vph)	125	375	63	125	375	63	125	375	63	125	375	63
Turn Type	pm+pt		pm+ov	pm+pt		pm+ov	pm+pt		pm+ov	pm+pt		pm+ov
Protected Phases	7	4	5	3	8	1	5	2	3	1	6	7
Permitted Phases	4		4	8		8	2		2	6		6
Detector Phases	7	4	5	3	8	1	5	2	3	1	6	7
Minimum Initial (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Minimum Split (s)	9.0	21.0	9.0	9.0	21.0	9.0	9.0	21.0	9.0	9.0	21.0	9.0
Total Split (s)	9.0	21.0	9.0	9.0	21.0	9.0	9.0	21.0	9.0	9.0	21.0	9.0
Total Split (%)	15%	35%	15%	15%	35%	15%	15%	35%	15%	15%	35%	15%
Yellow Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
All-Red Time (s)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Lead/Lag	Lead	Lag	Lead	Lead	Lag	Lead	Lead	Lag	Lead	Lead	Lag	Lead
Lead-Lag Optimize?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Recall Mode	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max
Act Effct Green (s)	20.0	16.0	25.0	20.0	16.0	25.0	20.0	16.0	25.0	20.0	16.0	25.0
Actuated g/C Ratio	0.33	0.27	0.42	0.33	0.27	0.42	0.33	0.27	0.42	0.33	0.27	0.42
v/c Ratio	0.38	0.43	0.10	0.38	0.43	0.10	0.38	0.43	0.10	0.38	0.43	0.10
Uniform Delay, d1	11.3	18.2	0.0	11.3	18.2	0.0	11.3	18.2	0.0	11.3	18.2	0.0
Delay	12.2	18.5	3.5	12.2	18.5	3.5	12.2	18.5	3.5	12.2	18.5	3.5
LOS	B	B	A	B	B	A	B	B	A	B	B	A
Approach Delay		15.5			15.5			15.5			15.5	
Approach LOS		B			B			B			B	

Intersection Summary

Cycle Length: 60
Actuated Cycle Length: 60
Offset: 39 (65%), Referenced to phase 4:EBTL, Start of Green
Natural Cycle: 60
Control Type: Pretimed
Maximum v/c Ratio: 0.43
Intersection Signal Delay: 15.5 — Intersection LOS: B
Intersection Capacity Utilization 54.3% — ICU Level of Service A

Splits and Phases: 3: West Approach & North Approach



Conventional Intersection

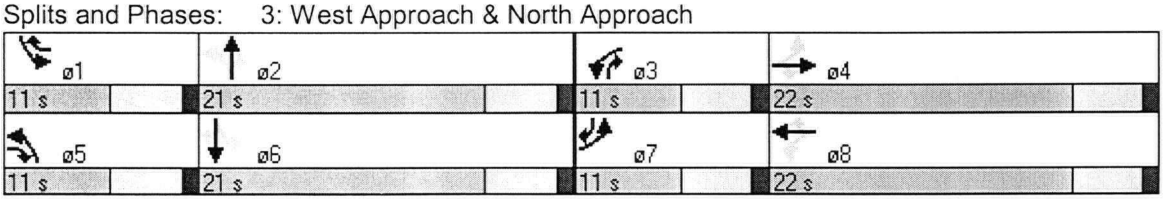
Single Left - 4000 veh/hr

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑↑	↱	↰	↑↑	↱	↰	↑↑	↱	↰	↑↑	↱
Volume (vph)	250	750	125	250	750	125	250	750	125	250	750	125
Turn Type	pm+pt		pm+ov	pm+pt		pm+ov	pm+pt		pm+ov	pm+pt		pm+ov
Protected Phases	7	4	5	3	8	1	5	2	3	1	6	7
Permitted Phases	4		4	8		8	2		2	6		6
Detector Phases	7	4	5	3	8	1	5	2	3	1	6	7
Minimum Initial (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Minimum Split (s)	9.0	21.0	9.0	9.0	21.0	9.0	9.0	21.0	9.0	9.0	21.0	9.0
Total Split (s)	11.0	22.0	11.0	11.0	22.0	11.0	11.0	21.0	11.0	11.0	21.0	11.0
Total Split (%)	17%	34%	17%	17%	34%	17%	17%	32%	17%	17%	32%	17%
Yellow Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
All-Red Time (s)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Lead/Lag	Lead	Lag	Lead	Lead	Lag	Lead	Lead	Lag	Lead	Lead	Lag	Lead
Lead-Lag Optimize?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Recall Mode	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max	Max
Act Effct Green (s)	23.0	17.0	28.0	23.0	17.0	28.0	22.0	16.0	27.0	22.0	16.0	27.0
Actuated g/C Ratio	0.35	0.26	0.43	0.35	0.26	0.43	0.34	0.25	0.42	0.34	0.25	0.42
v/c Ratio	0.98	0.88	0.19	0.98	0.88	0.19	0.98	0.94	0.20	0.98	0.94	0.20
Uniform Delay, d1	13.9	23.0	8.0	13.9	23.0	8.0	14.7	24.0	8.4	14.7	24.0	8.4
Delay	55.2	29.5	8.6	55.2	29.5	8.6	55.8	36.8	9.0	55.8	36.8	9.0
LOS	E	C	A	E	C	A	E	D	A	E	D	A
Approach Delay		32.9			32.9			38.0			38.0	
Approach LOS		C			C			D			D	

Intersection Summary

Cycle Length: 65
Actuated Cycle Length: 65
Offset: 0 (0%), Referenced to phase 4:EBTL, Start of Green
Natural Cycle: 65
Control Type: Pretimed
Maximum v/c Ratio: 0.98
Intersection Signal Delay: 35.4 Intersection LOS: D
Intersection Capacity Utilization 91.8% ICU Level of Service E

Splits and Phases: 3: West Approach & North Approach



APPENDIX B: USC SAMPLE DELAY CALCULATIONS

DELAY CALCULATIONS FOR USC INTERSECTION (VARYING LEFT AND THRU VOLUMES)

		SECONDARY INTERSECTIONS				PRIMARY INTERSECTION																TOTAL DELAYS BY MOVEMENT							
		Se/Sn/Sw/Ss				East Thru		East Left		North Thru		North Left		West Thru		West Left		South Thru		South Left		East Thru	East Left	North Thru	North Left	West Thru	West Left	South Thru	South Left
	Cycle	Inbound		Outbound		EB Thru		EB Left		WB Thru		WB Left		NB Thru		NB Left		SB Thru		SB Left									
Vol	Length	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	delay	delay	delay	delay	delay	delay	delay	delay
500	40	125	7.6	125	2.3	94	4.2	31	1.2	94	4.2	31	1.2	94	4.2	31	1.2	94	4.2	31	1.2	14.1	11.1	14.1	11.1	14.1	11.1	14.1	11.1
1000	40	250	8.0	250	2.2	188	4.2	63	0.7	188	4.2	63	0.7	188	4.2	63	0.7	188	4.2	63	0.7	14.4	10.9	14.4	10.9	14.4	10.9	14.4	10.9
1500	40	375	8.3	375	2.6	281	4.2	94	0.5	281	4.2	94	0.5	281	4.2	94	0.5	281	4.2	94	0.5	15.1	11.4	15.1	11.4	15.1	11.4	15.1	11.4
2000	40	500	8.7	500	3.0	375	4.2	125	0.4	375	4.2	125	0.4	375	4.2	125	0.4	375	4.2	125	0.4	15.9	12.1	15.9	12.1	15.9	12.1	15.9	12.1
2500	40	625	9.2	625	3.4	469	4.0	156	0.4	469	4.0	156	0.4	469	4.0	156	0.4	469	4.0	156	0.4	16.6	13.0	16.6	13.0	16.6	13.0	16.6	13.0
3000	40	750	9.6	750	4.7	563	1.2	188	0.1	563	1.2	188	0.1	563	1.2	188	0.1	563	1.2	188	0.1	15.5	14.4	15.5	14.4	15.5	14.4	15.5	14.4
3500	40	875	10.2	875	6.1	656	1.3	219	0.1	656	1.3	219	0.1	656	1.3	219	0.1	656	1.3	219	0.1	17.6	16.4	17.6	16.4	17.6	16.4	17.6	16.4
4000	45	1000	11.2	1000	4.4	750	3.9	250	2.6	750	3.9	250	2.6	750	3.9	250	2.6	750	3.9	250	2.6	19.5	18.2	19.5	18.2	19.5	18.2	19.5	18.2
4250	45	1063	12.4	1062.5	6.1	797	4.0	266	2.7	797	4.0	266	2.7	797	4.0	266	2.7	797	4.0	266	2.7	22.5	21.2	22.5	21.2	22.5	21.2	22.5	21.2
4500	50	1125	15.3	1125	11.8	844	0.1	281	0.1	844	0.1	281	0.1	844	0.1	281	0.1	844	0.1	281	0.1	27.2	27.2	27.2	27.2	27.2	27.2	27.2	27.2
4750	55	1188	16.1	1187.5	15.2	891	0.1	297	0.1	891	0.1	297	0.1	891	0.1	297	0.1	891	0.1	297	0.1	31.4	31.4	31.4	31.4	31.4	31.4	31.4	31.4
5000	55	1250	18.2	1250	17.6	938	0.1	313	0.1	938	0.1	313	0.1	938	0.1	313	0.1	938	0.1	313	0.1	35.9	35.9	35.9	35.9	35.9	35.9	35.9	35.9
5250	55	1313	21.8	1312.5	22.6	984	0.1	328	0.1	984	0.1	328	0.1	984	0.1	328	0.1	984	0.1	328	0.1	44.5	44.5	44.5	44.5	44.5	44.5	44.5	44.5
5500	70	1375	28.6	1375	24.2	1031	0.0	344	0.0	1031	0.0	344	0.0	1031	0.0	344	0.0	1031	0.0	344	0.0	52.8	52.8	52.8	52.8	52.8	52.8	52.8	52.8
5750	70	1438	38.3	1437.5	31.7	1078	0.0	359	0.0	1078	0.0	359	0.0	1078	0.0	359	0.0	1078	0.0	359	0.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0
6000	100	1500	39.2	1500	46.8	1125	0.1	375	0.0	1125	0.1	375	0.0	1125	0.1	375	0.0	1125	0.1	375	0.0	86.1	86.0	86.1	86.0	86.1	86.0	86.1	86.0
6250	100	1563	49.1	1562.5	57.8	1172	0.1	391	0.0	1172	0.1	391	0.0	1172	0.1	391	0.0	1172	0.1	391	0.0	107.0	106.9	107.0	106.9	107.0	106.9	107.0	106.9
6500	100	1625	60.4	1625	70.2	1219	0.1	406	0.0	1219	0.1	406	0.0	1219	0.1	406	0.0	1219	0.1	406	0.0	130.7	130.6	130.7	130.6	130.7	130.6	130.7	130.6

DELAY CALCULATIONS FOR USC INTERSECTION (VARYING LEFT VOLUMES)

	SECONDARY INTERSECTIONS				PRIMARY INTERSECTION																TOTAL DELAYS BY MOVEMENT							
	Se/Sn/Sw/Ss				East Thru		East Left		North Thru		North Left		West Thru		West Left		South Thru		South Left		East Thru	East Left	North Thru	North Left	West Thru	West Left	South Thru	South Left
Cycle	Inbound		Outbound		EB Thru		EB Left		WB Thru		WB Left		NB Thru		NB Left		SB Thru		SB Left									
Length	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	vol	delay	delay	delay	delay	delay	delay	delay	delay	delay
40	800	9.9	800	4.9	600	0.3	200	0.1	600	0.3	200	0.1	600	0.3	200	0.1	600	0.3	200	0.1	15.1	14.9	15.1	14.9	15.1	14.9	15.1	14.9
40	850	10.1	850	5.9	600	0.3	250	0.1	600	0.3	250	0.1	600	0.3	250	0.1	600	0.3	250	0.1	16.3	16.1	16.3	16.1	16.3	16.1	16.3	16.1
40	900	10.3	900	6.7	600	0.3	300	0.1	600	0.3	300	0.1	600	0.3	300	0.1	600	0.3	300	0.1	17.3	17.1	17.3	17.1	17.3	17.1	17.3	17.1
40	950	10.7	950	7.3	600	0.4	350	0.1	600	0.4	350	0.1	600	0.4	350	0.1	600	0.4	350	0.1	18.4	18.1	18.4	18.1	18.4	18.1	18.4	18.1
45	1000	11.2	1000	5.9	600	1.1	400	0.4	600	1.1	400	0.4	600	1.1	400	0.4	600	1.1	400	0.4	18.2	17.5	18.2	17.5	18.2	17.5	18.2	17.5
45	1050	12.0	1050	7.8	600	1.0	450	1.3	600	1.0	450	1.3	600	1.0	450	1.3	600	1.0	450	1.3	20.8	21.1	20.8	21.1	20.8	21.1	20.8	21.1
50	1100	14.5	1100	7.6	600	1.0	500	2.1	600	1.0	500	2.1	600	1.0	500	2.1	600	1.0	500	2.1	23.1	24.2	23.1	24.2	23.1	24.2	23.1	24.2
55	1150	15.1	1150	8.0	600	1.0	550	3.4	600	1.0	550	3.4	600	1.0	550	3.4	600	1.0	550	3.4	24.1	26.5	24.1	26.5	24.1	26.5	24.1	26.5
55	1200	16.5	1200	9.1	600	1.0	600	6.2	600	1.0	600	6.2	600	1.0	600	6.2	600	1.0	600	6.2	26.6	31.8	26.6	31.8	26.6	31.8	26.6	31.8
60	1250	17.5	1250	21.0	600	0.4	650	10.3	600	0.4	650	10.3	600	0.4	650	10.3	600	0.4	650	10.3	38.9	48.8	38.9	48.8	38.9	48.8	38.9	48.8
60	1300	19.3	1300	22.2	600	0.4	700	19.7	600	0.4	700	19.7	600	0.4	700	19.7	600	0.4	700	19.7	41.9	61.2	41.9	61.2	41.9	61.2	41.9	61.2
60	1350	21.9	1350	25.4	600	0.5	750	37.3	600	0.5	750	37.3	600	0.5	750	37.3	600	0.5	750	37.3	47.8	84.6	47.8	84.6	47.8	84.6	47.8	84.6
80	1400	26.7	1400	24.9	600	0.4	800	54.0	600	0.4	800	54.0	600	0.4	800	54.0	600	0.4	800	54.0	52.0	105.6	52.0	105.6	52.0	105.6	52.0	105.6
100	1450	32.3	1450	28.0	600	0.4	850	67.3	600	0.4	850	67.3	600	0.4	850	67.3	600	0.4	850	67.3	60.7	127.6	60.7	127.6	60.7	127.6	60.7	127.6

APPENDIX C: VISSIM SAMPLE OUTPUT

VISSIM OUTPUT SUMMARY:

USC vs Conventional(Single Left)

USC				
	Delay	Stopd	Stops	#Veh
LEFT	33.66	20.225	1.81125	738.525
THRU	18.305	6.5825	1.14725	1227.025
RIGHT	2.6225	0.5525	0.1725	499.55
CONVENTIONAL-SINGLE LEFT				
	Delay	Stopd	Stops	#Veh
LEFT	25.24	16.82	1.1705	743.825
THRU	27.2775	20.4275	0.88225	1233.55
RIGHT	1.8325	0.345	0.109	502.125
DIFFERENCE				
	Delay	Stopd	Stops	#Veh
LEFT	8.42	3.405	0.64075	-5.3
THRU	-8.9725	-13.845	0.265	-6.525
RIGHT	0.79	0.2075	0.0635	-2.575
% DIFF				
	Delay	Stopd	Stops	#Veh
LEFT	33%	20%	55%	-1%
THRU	-33%	-68%	30%	-1%
RIGHT	43%	60%	58%	-1%

USC				
	Delay	Stopd	Stops	#Veh
LEFT & THRU	25.9825	13.40375	1.47925	982.775
ALL	18.32689	9.192437	1.051092	824.437
CONVENTIONAL-SINGLE LEFT				
	Delay	Stopd	Stops	#Veh
LEFT & THRU	23.13375	16.3825	0.885125	960.7
ALL	18.11667	12.53083	0.720583	826.5
DIFFERENCE				
	Delay	Stopd	Stops	#Veh
LEFT & THRU	2.84875	-2.97875	0.594125	22.075
ALL	0.210224	-3.338396	0.330509	-2.063025
% DIFF				
	Delay	Stopd	Stops	#Veh
LEFT & THRU	12%	-18%	67%	2%
ALL	1%	-27%	46%	0%

VISSIM OUTPUT SUMMARY:

USC vs Conventional(Dual Left)

USC				
	Delay	Stopd	Stops	#Veh
LEFT	33.66	20.225	1.81125	738.525
THRU	18.305	6.5825	1.14725	1227.025
RIGHT	2.6225	0.5525	0.1725	499.55
CONVENTIONAL-DUALE LEFT				
	Delay	Stopd	Stops	#Veh
LEFT	35.6	26.7475	1.0745	743.825
THRU	24.6225	18.0275	0.86325	1238.25
RIGHT	2.3425	0.4075	0.15375	496.025
DIFFERENCE				
	Delay	Stopd	Stops	#Veh
LEFT	-1.94	-6.5225	0.73675	-5.3
THRU	-6.3175	-11.445	0.284	-11.225
RIGHT	0.28	0.145	0.01875	3.525
% DIFF				
	Delay	Stopd	Stops	#Veh
LEFT	-5%	-24%	69%	-1%
THRU	-26%	-63%	33%	-1%
RIGHT	12%	36%	12%	1%

USC				
	Delay	Stopd	Stops	#Veh
LEFT & THRU	25.9825	13.40375	1.47925	982.775
ALL	18.32689	9.192437	1.051092	824.437
CONVENTIONAL-DUALE LEFT				
	Delay	Stopd	Stops	#Veh
LEFT & THRU	25.80875	19.0175	0.8505	959.4125
ALL	20.855	15.06083	0.697167	826.0333
DIFFERENCE				
	Delay	Stopd	Stops	#Veh
LEFT & THRU	0.17375	-5.61375	0.62875	23.3625
ALL	-2.528109	-5.868396	0.353926	-1.596359
% DIFF				
	Delay	Stopd	Stops	#Veh
LEFT & THRU	1%	-30%	74%	2%
ALL	-12%	-39%	51%	0%