博士論文

Further Improvements at Signalized Intersections: Solutions with Auxiliary Signals

(信号交差点性能の増強方策に関する研究 － 付加的信号機の適用 －)

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Abstract

This dissertation focuses on two approaches for improving the capacity and operational performance of a Regular Signalized Intersection (RSI): the Tandem Sorting Strategy (TSS) and Median U-Turn (MUT). Without requiring additional space, both approaches provide additional capacity by relaxing the traditional practice of assigning one vehicle movement to one lane in an intersection. In this study, both strategies are operated with the use of auxiliary signals which are placed upstream of the main intersection stop line. TSS uses auxiliary signals (or pre-signals) to separate the vehicles belonging to the through and turning phase upstream of the intersection so that they can approach the main intersection in bunches and utilizes as many possible lanes for discharging. Meanwhile, MUT uses auxiliary signals to manage conflicts between the merging streams at the U-turn crossover that want to approach the main signal stopline.

The operational performance of the two approaches were assessed with respect to delay and capacity. To this end, delay models and corresponding signal optimization problems were developed. For TSS, the effects of storage limitations and headway variations were incorporated into the signal settings.

The evaluation results showed the range of demand conditions for which TSS and MUT can provide improvements to RSI. It was shown that reverting to TSS (i.e.,turning on the pre-signal) while the intersection is undersaturated can be beneficial for avoiding oversaturation. Meanwhile, MUT was shown to have good performance when U-turn demands are low. However, simulation results showed that under certain cases of oversaturation, MUT can lead to a failed state where it gives a lower capacity than the regular signalized intersection.

Contents

Acknowledgements **iii**

Abstract **v**

List of Figures **x**

List of Tables **xi**

1 Introduction **1**
1.1 Research Objectives and Significance 2
1.2 Scope and Limitations 3
1.3 Research contributions 3
1.4 Outline of the Dissertation 4

2 Literature Review **5**
2.1 Conventional Treatments to Improve Regular Signalized Intersections . . . 5
2.1.1 Physical and Pre-set Signal Modifications 5
2.1.2 Improvements by Traffic Signal Modification 8
2.1.3 System-wide treatments 11
2.2 Limitations of Conventional Treatments 12
2.3 Unconventional Arterial Intersection Designs (UAIDs) 14
2.4 Applications of Auxiliary Signals 16
2.5 Considerations in Improving Intersections 20
2.6 Gaps in the Literature 21

3 Preliminaries **25**
3.1 Tandem Sorting Strategy 25
3.1.1 TSS Concept 25
3.1.2 TSS Capacity Without Constraints 26
3.1.3 Phase-Swap Strategy 27
3.1.4 Treatment of three turning movements 28
3.2 Median U-Turns 29
3.2.1 MUT Signal Setting 30

4 Tandem Sorting Strategy: Performance and Limitations **32**
4.1 Capacity Evaluation Considering Constraints 33
4.1.1 Split Constraint 33
4.1.2 Storage length constraint 34
4.1.3 Upstream link length constraint 35
4.2 Phase Swap Strategy 36

4.2.1 PSS Split Constraint . . . 36
4.2.2 PSS Storage Length Constraint . . . 37
4.2.3 Pre-signal operation algorithm . . . 40
4.3 Tandem Sorting Strategy for Undersaturated Conditions: A two-step optimization . . . 41
4.3.1 Step One: Delay Minimization Problem . . . 41
4.3.2 Step Two: Capacity Maximization Problem: TSS . . . 44
4.3.3 Phase Swap Strategy . . . 47
4.4 Stochastic Headways . . . 48
4.5 Results and Discussion . . . 49
4.5.1 Application domain in Undersaturated Demands . . . 49
4.5.2 Effect of discharge headway variations . . . 51
4.5.3 Effect of Storage Length . . . 53

5 Median U-Turn: Performance and Limitations 57
5.1 Safety and Efficiency Issues in Conventional MUTs . . . 57
5.1.1 Reduction of conflict points in MUTs . . . 57
5.1.2 Conventional Two-Phase Plan: Efficiency Issues . . . 58
5.2 Simulation-Based Evaluation . . . 59
5.2.1 Demand Scenarios . . . 60
5.2.2 Phasing Plan . . . 60
5.2.3 Signal Settings . . . 63
5.2.4 Offset setting comparison . . . 65
5.3 Simulation Results . . . 66
5.3.1 Average travel time trends . . . 67
5.3.2 Composition of Travel Times . . . 68
5.3.3 Effect of crossover distance . . . 69
5.3.4 Comparisons with Regular Signalized Intersection . . . 70
5.4 MUT optimized signal settings . . . 72
5.5 Verification of optimization results . . . 75
5.6 Results and Discussion . . . 76

6 Comparison of Attributes 79
6.1 Role of Auxiliary Signals . . . 79
6.2 Operational Performance . . . 80
6.3 Operational Flexibility . . . 82
6.4 Geometric adjustments . . . 82
6.5 Auxiliary Signal Distance . . . 83

7 Final Remarks 84
7.1 Summary and Conclusions . . . 84
7.2 Recommendations and Further Work . . . 85

A Derivation of ρ 95

B Operational Performance of RSI, TSS, and MUT 96

C Optimization Result: Cycle Lengths 98
C.1 RSI Cycle Lengths . . . 98

C.2 TSS Cycle Lengths 99
C.3 MUT Cycle Lengths 100

Contents

List of Figures

2.1 Relationship of Intersection type and traffic volume 14
2.2 Continuous Flow Intersection 16
2.3 Intersection Design Factors 19

3.1 Tandem Sorting Strategy (TSS) Concept 25
3.2 Four Phase Plan Assumed in the Study 27
3.3 Phase swap at the main signal 28
3.4 Lane configuration for Phase Swap Strategy 28
3.5 TSS with shared upstream lane 28
3.6 TSS with separate left-turn lane and no pre-signal 29
3.7 Typical 2-phase plan for a signalized MUT 30
3.8 A typical phase plan for a signalized MUT (with overlaps) 30
3.9 Typical Median U-turn design 30

4.1 Fundamental Diagram 33
4.2 Time-space diagram of queue evolution in the storage area 34
4.3 Queue evolution in Phase Swap Strategy 36
4.4 PSS: Storage length constraint violations 38
4.5 Pre-Signal Operation Algorithm 40
4.6 Arrival and departure curves in the storage area 42
4.7 Intersection configuration 50
4.8 TSS vs RSI performance comparison 50
4.9 TSS:RSI delay at 60% Major demand 51
4.10 TSS:RSI delay at 30% Right-turn rate 51
4.11 Sensitivity to coefficient of variation in discharge headways 52
4.12 Delay increase after green time adjustment, 60% Major demand 52
4.13 TSS:RSI delay under stochastic headways 53
4.14 Capacity imbalance caused by inadequate storage length 53
4.15 Capacities for typical link lengths 55
4.16 Storage Lengths Required to attain maximum capacity 55

5.1 Conflict points in an MUT (Total: 16) 58

5.2 Effect of Saturation Flow Rate on Travel Times 58
5.3 MUT Diagram 59
5.4 Intersection layouts used in the comparison 60
5.5 RSI phasing plan 60
5.6 Plan A 62
5.7 Plan B 62
5.8 Plan C 63
5.9 Through-priority offset setting 64
5.10 MUT-priority offset setting 65
5.11 Intersection break down due to overflows from the U-turn lane 66
5.12 Average Travel time trends 67
5.13 Low Demand: 3000 veh/hr 68
5.14 Near-Saturation to Oversaturated Demand: 6000 veh/hr 68
5.15 Analysis areas 68
5.16 Average delays plotted per area 68
5.17 Effect of crossover distance on travel times 69
5.18 Effect of crossover distance on each movement's travel time 69
5.19 Cycle Lengths Used in the Simulations 71
5.21 MUT and RSI capacities at Demand = 6000 veh/hr 71
5.20 MUT:RSI travel time ratios 72
5.22 Verification of estimated delays 76
5.23 Comparison of delays 77
5.24 Comparison of Cycle Lengths 77
5.25 MUT:RSI expected delays (60% Major demand) 78

6.1 Intersection layouts used in the comparison 80
6.2 Delay comparison 81
6.3 Cycle length comparison 81
6.4 Intersection type with least expected delay 82
6.5 Alternative MUT design with loons 83

A.1 Time-space diagram of queue evolution in the storage area 95

List of Tables

2.1 Intersection-level treatments for Pedestrians 7
2.2 Intersection-level treatments for Bicycles 7
2.3 Intersection-level treatments for Transit 7
2.4 Unconventional Arterial Intersection Designs 24

3.1 Main Signal Demands 31

5.1 Simulation Scenarios 61

5.2 Parameters used in the calculation of cycle lengths 61

Chapter 1

Introduction

Traffic congestion is a serious problem in many areas especially with the increase in motorization. According to the Asian Development Bank, motor vehicle fleets are observed to double every 5 to 7 years in Asian urban areas, costing around 2-5% of GDP due to wasted time and higher transport costs (Asian Development Bank, 2014). It was estimated that by 2030, around 1.1 billion more people will be living in urban areas than in 2005 (Asian Development Bank, 2008), clearly translating to a higher demand for roads.

To address these problems, the usual approach would be to increase the capacity of existing infrastructure by widening existing roads or building additional pathways. However, these approaches require high capital costs which may not be easily available. In the United States, the cost of adding lanes was estimated to cost between 2-10 million USD per lane-mile of freeway or 750,000 USD per lane-mile of surface street (Texas A & M Transportation Institute, 2013).

This dissertation deals with the issue of improving the performance of existing infrastructure. The intersection will be the focal point of the analyses that will be conducted as it is the core of a transportation network. This study will deal with improvements to Regular Signalized Intersections (RSIs). These are traditionally designed intersections where vehicles of each movement can directly cross the junction using their designated lanes and without the need for detours or any upstream intervention.

Two alternative strategies for increasing roadway capacity without necessarily providing additional space are studied: the Tandem Sorting Strategy(TSS) and the Median U-Turn (MUT). Both strategies improve intersection performance by dealing with turning movements such that the discharge lanes are fully utilized, albeit in different ways. The two approaches use upstream auxiliary signals, a feature which will be studied in this dissertation. TSS uses auxiliary signals (or pre-signals) to separate the vehicles belonging to different movements (i.e., through and right-turn) upstream of the intersection so that they can approach the main intersection in separate bunches and utilize as many possible lanes for discharging. For this reason, TSS is considered as suitable for high right-turn rates. The other strategy considered is the MUT. One of the most common alternative intersection types, MUT prohibits direct right turns at the main intersection and instead re-routes them to a downstream U-turn crossover. The auxiliary signals are used to manage conflicts between the merging streams at the U-turn crossover that want to approach the main signal stopline. MUT is more suitable for lower right-turning movements. The advantages, disadvantages, and limitations of both strategies are explored and explained in this dissertation.

1.1 Research Objectives and Significance

The general aim of this research is to gain a better understanding of the performance and limitations of alternative strategies that use auxiliary signals, particularly the Tandem Sorting Strategy and Median U-turn. This is for the purpose of having a clearer idea of their various functions, benefits, and limitations.

Firstly, this research aims to perform delay estimation and signal optimization of intersections with auxiliary signals in undersaturated conditions. So far, both TSS and MUT have been introduced as measures to increase intersection capacity. This means that they are often considered as measures to improve oversaturated intersections. However, operational benefits in undersaturated conditions can also be reaped in terms of green duration and cycle length reductions. An important performance measure to be considered here is the delay. Since an auxiliary signal requires additional stops and delay, there is a need to quantify the additional delay caused by the auxiliary signal and evaluate the overall intersection performance compared to a regular signalized intersection. When these information are known, a more accurate operation and application of auxiliary signals can be done. Apart from improving delays in oversaturated intersections, they may also be used to extend the undersaturated state for a longer time.

Furthermore, this research also aims to evaluate not just the performance benefits of both strategies, but also their limitations. As most literature focus on the advantages of alternative approaches over conventional ones, limitations are often not considered. For example, the performance of the alternative intersection in oversaturated conditions are often not studied. By conducting a thorough evaluation of the limitations of each approach, the vulnerabilities of a particular approach are determined and should be considered during decision making.

To these ends, the results of this dissertation augment the current knowledge on intersections with auxiliary signals and contributes to the promotion of practical and innovative strategies that improve regular signalized intersections.

1.2 Scope and Limitations

The analyses in this dissertation are focused on isolated intersections only. In evaluating the performance of both TSS and MUT, steady state traffic conditions were assumed so the signal control parameters correspond to a pre-timed control system. Various demand values were checked in order to account for the variability in traffic demands.

1.3 Research contributions

The main contributions of this dissertation are:

1. A detailed discussion of the spatial and temporal factors that can reduce the performance of Tandem Sorting Strategy. Limiting equations were formulated and a simple algorithm for computing the combined effect of such factors was developed. While some of these factors have been considered in other works, this work explicitly considered the effect of storage length on the capacity and proposed an operational procedure for controlling the pre-signal despite storage limitations. Moreover, adjustments to the pre-signal green times which accounted for variations in discharge headways was also introduced.
2. Development of a two-step optimization method for determining the pre-signal and main signal settings under Tandem Sorting Strategy as well as the expected delay and capacity for the entire intersection. Although signal optimization of TSS has been done in other works, the formulation maximized the pre-signal capacity despite the lack of storage space. This was found to improve the pre-signal capacity compared to other formulations.
3. The undersaturated demand boundary where TSS performs better than the regular signalized intersection can be found from the expected delay and capacities. This is useful for determining when to turn on the pre-signal. It is shown that TSS not only increases capacity, but also decreases delays in the entire intersection even if not all links have pre-signals.
4. A delay minimization program was formulated for Median U-Turns and was verified to give comparable delay estimated with that of a simulation.
5. A simulation-based evaluation was conducted to compare the performance of the Median U-turn and Regular Signalized Intersection. By testing a wide spectrum of demands, it was found that at near saturation demands the MUT performance declines to levels that are even much worse than RSI. Compared to other studies with the same intent, a clear methodology for signal setting was presented by explicitly defining the phasing plan and offset settings.

1.4 Outline of the Dissertation

The remaining parts of this dissertation are organized as follows:
Chapter 2 presents a review of traditional signal optimization research and provides a summary of unconventional alternative intersection designs. The existing researches on TSS and MUT are discussed and gaps in the literature are identified. Chapter 3 presents the basic concept of TSS and gives the capacity formulae under ideal conditions. It also introduces the MUT layout studied in this work and details the composition of vehicle flows. The different factors that affect TSS capacity are discussed in Chapter 4. Here, an optimization problem that maximizes TSS capacity despite storage constraints is formulated. Numerical experiments show that benefits from turning on the pre-signal can be reaped even before oversaturation occurs. Chapter 5 presents a simulation-based evaluation of the MUT. Based on the simulation results, a simplified delay estimation model is formulated. A comparison between TSS and MUT is provided in Chapter 6. Finally, Chapter 7 summarizes the findings of the dissertation and gives recommendations and identifies areas for further work.

Chapter 2

Literature Review

This chapter provides a review of different treatments used to improve regular signalized intersections (also called conventional intersections) and identifies related issues. The treatments are generally targeted to improve two aspects: safety and efficiency.

2.1 Conventional Treatments to Improve Regular Signalized Intersections

Various treatments are applied at an intersection level to improve safety and traffic flow efficiency for the different users. Varying problems arise due to the interactions between different users (i.e., pedestrians, cyclists, transit, vehicles) which require various measures. Some measures involve physical modifications to the intersection layout while some involve minor modifications to the signal control operation. Assuming that all users follow traffic signals, the overall traffic flow can also be significantly improved by changes in the signal control algorithm. Thus, a separate subsection is provided on this subject.

2.1.1 Physical and Pre-set Signal Modifications

These type of modifications mostly deal with physical or structural improvements or signal control decisions which have to be pre-set. Their applicable conditions or triggers are summarized in Tables 2.1, 2.2, and 2.3 [1]. Quantitative studies suggesting threshold values for each treatment were limited, indicating that engineering judgment is likely applied when deciding when each measure is implemented.

[1] Main Source: (Rodegerdts et al., 2004). Secondary sources: (FHWA, 2008), (National Association of City Transportation Officials, 2014), (Huang and Zegeer, 2000), (Smith et al., 2005),(Hughes et al., 2006b)

PEDESTRIAN TREATMENTS			
Treatment	Applicable Conditions	Benefits	Liabilities
► Reduction of Curb-Radius	► Crashes between right-turning vehicles and pedestrians ► High-speed turning movements ► Poor sight-distance between pedestrians and motorists	► Improved Safety ► Reduced speed of left-turning vehicles ► Reduced collision severity-Shorter pedestrian crossing distance, shorter green times required ► Reduced intersection size	► Capacity reduction for left-turn movements ► Possible risk of rear-end collisions involving left-turn and through movements in case of shared discharge lanes ► Turning difficulty for large vehicles
► Provision of Curb Extensions	► High incidence of the following aggressive vehicle behaviors in high pedestrian crosswalks : not yielding to pedestrians, high-speed turns, invasion of parking lanes ► High volume of pedestrian collisions	► Improved visibility of pedestrians by vehicles ► Reduced turning speeds leading to reduced collisions with pedestrians ► Shorter pedestrian crossing distance, shorter green times required	► Capacity reduction for left-turn movements ► Possible risk of rear-end collisions involving left-turn and through movements in case of shared discharge lanes ► Turning difficulty for large vehicles ► Possible traffic diversion to other roads without curb extensions
► Provision of Advanced Stop Line	► High incidence of right-turn-on-red collisions between vehicles and pedestrians	► Reduced risk of collisions between pedestrians and right-turning vehicles. ► Easier turning maneuver for large vehicles.	► Increased intersection clearance time. leading to higher loss time.
► Improvement of Pedestrian Signal Displays (i.e., WALK/DONT WALK signals, countdown displays, animated eyes display)	► Pedestrian-related collisions	► Assistance to visually impaired pedestrians. ► Improved pedestrian awareness.*Higher percentage of successful crossings.	► Increased accident risk when drivers immediately accelerate in anticipation of the countdown displaying zero. ► Increased accident risk when pedestrians start crossing during flashing "Dont Walk"
Protected Pedestrian Phase	Pedestrian-related collisions	Reduction in pedestrian collisions	Possible increased required green time.
Signal Phasing Modification (a-c):			
► a. Leading Pedestrian Interval	► Moderate to heavy pedestrian traffic. ► High pedestrian-vehicle collisions	► Reduction in conflict between pedestrians and vehicles due to early recognition of pedestrians	► Increased intersection delay.
► b. Lagging pedestrian Interval	► Moderate to heavy pedestrian traffic. ► High left-turn volume ► Exclusive left-turn lane ► One-way traffic in two intersecting links	► Improved safety: left-turn queue is cleared before pedestrians are allowed to cross	► Increased pedestrian delay.
► c. Exclusive Pedestrian Phase	► High number of pedestrians which affects vehicular delays ► High pedestrian-vehicle conflicts in all movements.	► No conflicts between vehicles and pedestrians. ► Decreased walking distance and pedestrian waiting time. ► Increased vehicle flow in each phase.	► Longer cycle length. ► Reduced overall vehicle flows due to long cycle lengths. ► Long waiting time for pedestrians if vehicle phases are given long green times.
Grade-Separation of Pedestrian Movements (i.e., Pedestrian overpasses or underpasses)	► Very high number of pedestrian-vehicle conflicts. ► High number of children using the crossing. ► High speed turning vehicles. ► Inadequate sight distance	► Removal of pedestrian-vehicle conflicts. ► Traffic flow improvement: faster vehicle speeds, shorter cycle length.	► Pedestrian avoidance if personal security risks are high. ► Not ideal for disabled pedestrians. ► Illegal pedestrian crossing behavior when grade-separated crossing is perceived to be inconvenient.

Table 2.1: Intersection-level treatments for Pedestrians

BICYCLE TREATMENTS			
Treatment	Applicable Conditions	Benefits	Liabilities
► Provision of Bicycle Box	► Presence of a bicycle lane. ► High bicycle-vehicle collisions. ► High bicyle and vehicle volumes where the dominant directions of each mode are not parallel. (Ex: high bicycle right-turns, high vehicle left-turns)	► Increased bicyclist visibility. ► Reduced bicyclist delays at the intersection. ► Facilitates transition between a left-side bike lane to a right-side bike lane (in case of bike lanes that extend across the intersection). ► Eases turning for right-turning bicyclists.	► May be incompatible under high left-turn movements.
► Provision of Bicycle Lanes	► High volume of bicyclists. ► Bicycle use is being promoted.	► Reduced bicycle-vehicle conflicts due to separation and increased predictability of movements	► Additional right-of-way required.

Table 2.2: Intersection-level treatments for Bicycles

TRANSIT TREATMENTS			
Treatment	Applicable Conditions	Benefits	Liabilities
► Relocation of Transit Stop(a-b):			
► a. Upstream of the intersection (near-side)	► Peak-hour congestion on the far side of the intersection.	► Bus has ample space for pulling away from the curb.	► Increased conflict between left-turning vehicles, through-vehicles, and buses. ► Bus can obstruct view of signal, intersection signages, and pedestrians
► b. Downstream of the intersection (far-side)	► Significant number of collisions involving left-turning vehicles, through vehicles, and stopped near-side buses (typically of the rear-end and side-swipe types). ► Significant number of collisions involving crossing pedestrians getting hit by vehicles after passing a stationary bus.	► Improved safety since pedestrians are more visible to drivers.	► Risk of rear-end collisions when drivers do not anticipate that the bus will stop after crossing the intersection.
► Transit priority signal	► Promotion of transit use. ► High bus delays and low bus reliability.	► Reduced travel time for buses. ► Improved bus reliability leading to increase in bus frequency	► Increased delays to non-priority traffic

Table 2.3: Intersection-level treatments for Transit

Modification of Intergreen Times

The intergreen time is the duration provided between phases which ensures that vehicles in the current phase can clear the intersection before vehicles in the succeeding phase are released. This is considered to be the sum of the yellow change interval and red clearance interval. When a driver encounters a yellow indication, he/she may face the "yellow interval dilemma" where he/she may be close enough to the intersection to stop yet far enough to clear the intersection in the given duration (Liu et al., 1996). A "dilemma zone" is defined along the intersection approach based on the speed and acceleration of discharging vehicles. Inadequate dilemma zone protection may result in right-angle crashes or rear-end collisions. The number of crashes is used as an indicator of the performance or need for dilemma zone protection which is in the form of vehicle detection system which extends the green time so that a vehicle can proceed safely through the intersection (Zhang et al., 2014).

2.1.2 Improvements by Traffic Signal Modification

Introduction to Traffic Signal Optimization

Traffic control signals are primarily installed to improve safety and efficiency at an intersection where several conflicting movements are present. The traffic signal assigns green times to a set of non-conflicting movements called a signal group. The set of signal groups are then arranged into signal stages. The arrangement of groups and stages coupled with the corresponding green durations form the signal timing plan.

In signal control optimization, the problem becomes that of finding the optimal signal parameters that will either minimize delay or maximize capacity. In general, the objective is to minimize delay given that there is sufficient reserve capacity (Bell, 1992).

Delay minimization problems were tackled in the works of (Webster, 1958), (Allsop, 1971a) and (Yagar, 1977). These works assumed that the signal groups and stages were known. Webster (Webster, 1958) derived an expression for delay which incorporated a uniform arrival term, another term accounting for random arrivals, and an adjustment term derived from simulation results. He then derived an expression for the optimum cycle length which minimized the total expected delay in an intersection. In (Yagar, 1977), the delay minimization problem was extended to consider multiple time slices.

In (Improta and Cantarella, 1984), the signal setting problem was extended to include the movements that comprise a signal group and stage sequences as decision variables. Formulated as a Binary-Mixed-Integer-Linear Programming model, this required as an additional input the crossing compatibility of different streams. This formulation allowed for more flexibility in signal setting especially for intersections with unbalanced flows.

More recently, lane-based optimization was introduced in (Wong and Wong, 2003b). This type of problem could optimize the allocation of movements to a fixed number of lanes. The optimization objective was to maximize a common flow multiplier, a factor that is uniformly multiplied to the initial OD matrix. Maximizing this common flow multiplier is interpreted as maximizing the reserve capacity. This problem was formulated as a Binary-Mixed-Integer-Linear-Program. In later works, the formulation was extended to consider a more complex delay minimization problem (Wong and Wong, 2003a) as well as time-varying demands (Wong et al., 2006).

This sub-section summarizes some well-known traffic control programs.

Fixed-Time Control

Fixed time control refer to traffic signal settings that are pre-determined and operated unchanged in a given time period. These are only applicable to undersaturated conditions with relatively time-stable traffic demands (Papageorgiou et al., 2003). One such example is SIGSET (Allsop, 1971b) which estimates the undersaturated delays in an intersection and calculates the signal timing parameters based on the minimization of delay. This problem is a linearly constrained nonlinear programming problem. In SIGSET, the stages are pre-determined (i.e., stage-based control).

Another fixed-time, stage-based control algorithm is SIGCAP (Allsop, 1976). Instead of delay, the practical traffic capacity is estimated and signal timings are computed based on a capacity maximizing objective function. This is done by multiplying the real demand by a factor μ. The optimization problem is linear and is more suitable for over saturated conditions.

Another signal control algorithm is SIGSIGN (Silcock and Sang, 1990). Compared to the previous ones mentioned, this is a group-based (or phase-based) algorithm which determines the composition of traffic stages and stage sequences.

Adaptive Control

Recently, innovations in signal control involved the use of modern vehicle detection systems which allowed the real-time measurement of traffic conditions. Thus, switching from fixed-time to adaptive signals became proved to be more effective and flexible means of setting traffic signals.

Adaptive traffic signals use real-time collected data from detectors to operate traffic signals. Compared to pre-timed signals, adaptive signals can readily adjust the signal settings depending on the detected demand, therefore maximizing the available green times. Adaptive control can be semi-adaptive or fully adaptive ((Rodegerdts et al., 2004)). In semi-adaptive signals, detectors are located on the minor approach as well as the right-turn movement of the major approach. Fully adaptive signals have detectors on all approaches.

Examples of Adaptive Control Programs:

1. VA (Vehicle Actuated) Control Strategy. One of the earliest forms of vehicle actuation only involved the extension or termination of green durations. The current green duration ends when the maximum green time is reached or when a critical headway value (measured by the detector) is exceeded. A signal stage that has no demand receives no green time. This control strategy does not account for the size of queues, thereby the traffic control is not optimal (Highways Agency Bristol, 2005).
2. MOVA (Microprocessor Optimised Vehicle Actuation). (Vincent and Peirce, 1988) was developed in the United Kingdom in the late 1980s. Data is collected from vehicle detectors and fed into an online microprocessor which adjusts the green durations based on an algorithm that minimizes delay and the number of stops. When an approach becomes oversaturated, a capacity-maximization algorithm is invoked.
3. LHOVRA. This Swedish control system (Harirforoush, 2012) is widely used in Europe. This control system is group-based, meaning it can determine the composition and sequencing of the signal phases. In addition, it has the following features:
   - Truck(or bus) Priority. In this function, large vehicles are detected via closely-spaced detectors at the approach entrance.
   - Major-road Priority. Green duration is extended based on detections at a detector placed 130 meters (D130) from the stop line. Green time is maintained for headways at D130 that are less than 6.2 seconds. This function enables the controller to specify priority for the major approach.
   - Incident Reduction. The purpose of this function is to reduce the number of rear-end collisions and red light violations (i.e., reduction of vehicles in the dilemma zone). The green duration is extended so that a vehicle estimated to be in the dilemma zone can clear it safely. In a link with 70 km/hr speed limit, the green duration will be extended by 3.5 seconds if a vehicle passed D130 and the end of the unextended green. The green extension is 2.5 seconds at D80.
   - Variable Yellow. Also for safety, the yellow (or amber) duration is divided into two: a fixed part and variable part. The yellow time is extended based on detections at D80. If a vehicle is detected at D80 after the change to amber has started, the yellow duration is extended to allow the vehicle to pass the stopline.
   - Reduction of Red Light Infringements. This function extends the red time in anticipation of intentional red light violations as well as handle secondary conflicts. Using vehicle detections at D10 and D80, the a variable red duration is invoked to avoid collision between the red light violating vehicle and vehicles in the succeeding phase.
   - Green-yellow-red-green sequences. Using detections at D200/D140, this function

reduces the number of instantaneous green-yellow-red-green sequences. This estimates the presence of tailing vehicles at the intersection when the "all red" occurs. It ensures that the tailing vehicle is far enough from the intersection during "all red". In case the vehicle is too near, the yellow is turned off immediately and the green duration is commenced.

When the traffic demand can no longer be served by the intersection capacity, a congested state is reached and the queues cease to be in steady state. One fundamental indicator of congestion is the queue length and it is important to manage queues such that they do not extend to upstream intersections and cause blockages. A comprehensive review of oversaturated control strategies can be found in (Gettman et al., 2012).

2.1.3 System-wide treatments

Though not part of the scope of this research, a brief description of system-wide treatments is provided here in order to give a preview of the bigger picture of conventional treatments being applied today. System-wide treatments are applied uniformly to links that form a corridor. According to Signalized Intersections: Informational Guide (**SIIG**) (Rodegerdts et al., 2004), these treatments are applied to address safety issues brought about by sudden acceleration or deceleration of vehicles, disruptions caused by vehicles entering the corridor from midblock access points, and issues related to vehicle progression between signalized intersections.

To address safety issues, one recommendation proposed in **SIIG** are median improvement in the form of structural median modifications. (Garner and Deen, 1972) and (Knuiman et al., 1993) reported accident reductions for wider medians. However, it was found that for median widths between 4.2-24 meters, the frequency of crashes increased with median width in urban and sub-urban intersections (Harwood, 1995).

In addition to median treatments, **SIIG** also discussed the benefits of access management strategies to improve safety and efficiency at intersections. Entry points or access points are sources of conflicts in the traffic stream. It has been reported that the accident potential increases with the number of driveways [2] and intersections (Gluck et al., 1999). An additional access point per mile in a four-lane roadway is estimated to increase crash rates by four percent (Rodegerdts et al., 2004). It is easy to see how an increase in vehicle volumes emanating from access points can affect vehicles on the main road. To reduce the negative impacts of entering vehicles from access points, the following access management treatments have been introduced.

(i) Non-traversable medians
(ii) Two-way left turn lanes

[2] A driveway is a short road leading from a public road to a house (Oxford Dictionary, 2016)

(iii) U-turns as alternatives to direct left-turns
(iv) Access separation at interchanges
(v) Frontage roads

Detailed discussions of their operational and safety benefits as well as application guidelines are provided in (Rodegerdts et al., 2004).

Another system-wide treatment is signal coordination. To improve vehicle progression between intersections, the traffic signals are coordinated so that vehicles traveling through a series of intersections experience minimum delays and stops. Coordination is beneficial especially when the distance between intersections is quite near, around 3/4 mile according to (Henry, 2005). Signal coordination is approached in different ways. The approaches used in MAXBAND (Little, 1966) and PASSER IV-96 (Chaudhary and Messer, 1993) are based on the principle of maximizing the bandwidth (or the proportion of a signal cycle for which a vehicle from one end of a link can reach the other end without being stopped by the red light. Meanwhile, other approaches determine the coordinated signal settings by minimizing delay [SYNCHRO,(Husch and Albeck, 2003)] or a weighted function of delay and number of stops [TRANSYT,(Robertson, 1969),(Binning, 2015)].

2.2 Limitations of Conventional Treatments

The traffic control measures mentioned in the previous section have been shown to provide improvements to both safety and efficiency of intersections in the existing literature. The benefits of such measures target the following major points:

(i) Safety improvement by reduction of potential conflicts between users.
(ii) Improvement of traffic flow efficiency by green time adjustment that matches actual traffic conditions
(iii) Increased control flexibility so that priority can be provided to critical links or users

With the rapid growth in motorization and increase in the number of road users, it becomes important to consider the "upper limit" or the maximum possible benefit that can be gained from proposed intersection improvements. For instance, when all links in an intersection are oversaturated, the most that an adaptive signal controller can do is perhaps to provide more green to the more critical links or implement control measures that avoid spillover. However, further capacity increases are no longer possible with the present infrastructure.

When frequent traffic jams are observed in a specific area, the problem will be addressed only after a long process that involves public consultations, budget requests, construction planning, and in many cases, right-of-way acquisitions. A traditional approach to addressing capacity problems is by physically increasing the road space via road widening or construction of elevated pathways such as flyovers. However, this capacity expansion entails massive construction costs and requires the government to acquire road right-of-way which is usually not an easy process especially in urban areas. Recent researches have identified innovative ways to increase intersection capacity that require small to almost no additional space. These approaches give additional capacities which may be good enough to address the capacity issue. Before these approaches are discussed, issues related to the existing treatments are discussed.

1. Treatment of turning movements. In intersections with significant number of turning movements, a protected turning phase can improve safety. However, the additional phase increases cycle length and delays due to the reduction of green time for other phases. Also, additional loss time is caused by the added change intervals, starting delays, etc. (Kell and Fullerton, 1991). In adaptive signal systems, limitations in the detection system can lead to inaccurate signal settings (Gaspay et al., 2013). Also, adaptive signal systems must have maximum values of green times and cycle length in order to prevent exceedingly long cycles. In these cases, turning movements often receive less green times compared to the more-prioritized through traffic.
2. Underutilized lanes. The conventional methods subscribe to the common practice of assigning one lane to a single or pair of movements. When a single approach is assigned two phases (e.g., one phase for through and left, the other phase for right-turners), not all lanes are utilized for discharging vehicles during the green time. In intersections experiencing inefficiencies due to high number of turning vehicles, additional turning lanes become necessary. When additional lanes cannot be provided, we return to the problem where turning vehicles are not given adequate green times.
3. Variability of traffic composition. Lane assignment becomes very significant in case of highly variant traffic compositions. In the design phase, lane assignments are based on average traffic demands during peak hours. This is reasonable but not entirely flexible. For example, in a four lane approach, one lane each is allocated to the left and right-turners while the rest to through traffic. In case of high right-turn demand, the adaptive control system gives a higher (but limited) green time to the right-turn phase. However, since only one lane is assigned to it, the number of discharged vehicles becomes limited by the number of lanes and depending on the maximum allowable green time, the right-turn queue may or may not be dissipated.

If traffic demands continue to increase, capacity problems will soon be encountered and the signal optimization methods will no longer be able to accommodate existing demands (El Esawey and Sayed, 2013). The issues identified led researchers and practitioners to devise alternative ways to improve RSI's.

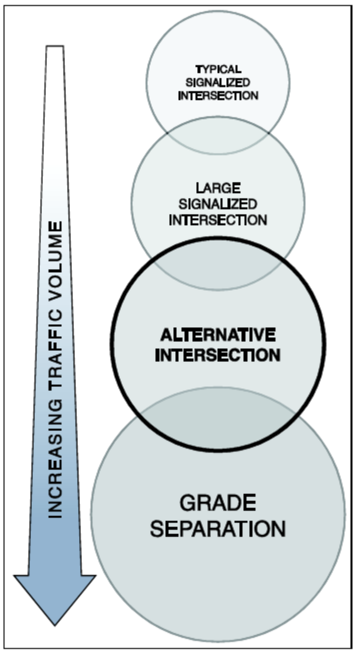


Figure 2.1. Relationship of Intersection type and traffic volume.

Source: (Reid et al., 2014).

2.3 Unconventional Arterial Intersection Designs (UAIDs)

An alternative approach to improving the performance of a regular signalized intersection involves the physical re-design of the intersection. A general term for such new designs is Unconventional Arterial Intersection Designs (UAIDs)(El Esawey and Sayed, 2013) though in many cases the term used is simply Alternative Intersection. Several studies have documented the benefits of UAIDs in terms of improving safety, capacity, and delays.

UAIDs introduced in the literature vary in two main aspects: A.)in terms of their treatment of Left, Right, and Through movements and B.) in terms of the way lanes are shared. A UAID improves safety by reducing the number of conflicts at the intersection. This is done by prohibiting specific movements from making usual turns. Instead, these vehicles are diverted to another location where they can make their desired maneuver with reduced conflict. Examples would be the Median U-turn and Superstreet Median where right-turning vehicles (in case of left-hand side traffic) are prohibited from making direct turns at the intersection. Instead, right-turners should go through the main intersection, turn right at a downstream U-turn crossover and go back to the main intersection this time as a left-turner. Other UAIDs such as the Jughandle and Quadrant Roadway divert turning vehicles upstream of the main intersection by providing dedicated pathways. The main advantages of diverting vehicles are to reduce conflicts between through and turning movements.

In terms of lane utilization, certain sections of the road are assigned as mixed-use lanes in some UAIDs. For example, in the Continuous Flow Intersection, the area near the main intersection signal is alternatively used as both approach and discharge lanes. Auxiliary signals are placed upstream of the main intersection to avoid conflicts. Such multiple usage of lanes allow for increased capacity.

In terms of traffic volume, UAIDs generally provide higher capacity than typical intersections. The applicability of UAIDs with respect to traffic volume is adeptly summarized in Figure 2.1. A summary of different UAIDs and their attributes is provided in Table 2.4. The detailed description of each UAID is skipped in this work. Instead, the reader is referred to related literature for the specific details.

Several studies such as (Reid and Hummer, 2001), (Autey et al., 2010), and (Kim et al., 2007a) have performed comparisons of different UAIDs by evaluating their performances (common performance indices used as network travel time, number of stops,etc.) under real-world demands. These studies are so far consistent in showing the advantage of each UAID over the regular signalized intersection but the advantage of each UAID over another depends on the demand scenarios. Thus, the engineer must select the most appropriate option based on a set of quantitative and qualitative criteria.

Existing studies give a good overview and general guidelines for selection of a specific design. One very comprehensive guidebook on alternative intersection design is given by FHWA (Hughes et al., 2010). This work gives guidelines on geometric design, signage, operational performance, construction, among others. A comprehensive review was also conducted by (El Esawey and Sayed, 2013) though this did not perform operational performance analysis and instead compared the different approaches used by many researchers in evaluating UAIDs.

Despite these, several issues still remain. Most studies employ a blackbox approach to the analysis of UAIDs. In most cases, signal setting parameters used in evaluating UAIDs are derived from a software optimizer and the intersection performance is evaluated using a traffic simulation tool. Although the same tools are used in all designs, no further details on relevant parameters such as offsets, minimum green times, phasing plans, etc. are provided in the studies. These parameters are important determinants of an intersection's performance. Dependence on simulation softwares without adequate knowledge of the fundamental assumptions of such software can lead engineers to use the tool incorrectly and make inaccurate conclusions. While these tools are practical, these make assessments difficult to replicate if others have no access to such software.

Figure 2.2. Continuous Flow Intersection.

Source: (Jeuniewic, 2016).

2.4 Applications of Auxiliary Signals

Auxiliary signals are defined here as traffic signals that facilitate the efficient flow of vehicles at the main signal. Some UAIDs such as the Continuous Flow Intersection 2.2 use auxiliary signals to function, in this case to manage vehicles going in opposing directions. In this case, auxiliary signals are necessary to avoid head-on collisions but in some cases, the auxiliary signals are optional.

In Median U-Turns

One way that auxiliary signals can be used to improve safety is to facilitate merging traffic flows, an example of which is the ramp meter. In the context of alternative intersections, one most common application of auxiliary signals is in Median U-Turns.

First, a short background on MUT safety is provided. Many empirical studies have shown that accidents are reduced in MUTs compared to making direct turns at the intersection [(Castronovo et al., 1995), (Xu, 2001), (Jagannathan, 2007)]. The most common type of accidents in junctions with direct left-turns (in case of right-hand side traffic) are head-on and angle crashes which are more likely to results to more severe injuries (Jagannathan, 2007). Though these are reduced in MUTs, the crash type also changed. MUTs reportedly have more side-swipe crashes due to the increase in weaving vehicles (Ruihua and Heng, 2009).

An analysis of crash data from 125 median openings from 7 states in the USA revealed that most accidents occur between the U-turning vehicle attempts to merge onto the main road (Potts, 2004). The nature of the conflict involved the following behavior: a.) multiple drivers use the same gap and b.) drivers make a discontinuous U-turn (otherwise known as a 3-Point turn due to lack of adequate turning radius).

Meanwhile, (Liu et al., 2007) and (Xu, 2001) surveyed numerous U-turn sections in America and found that the three most common types of collisions are of the rear-end, side-swipe, and angle type. Conflicts between vehicles not only have safety impacts but also have impacts on traffic flow. Through vehicles near the U-turn slot need to decrease speeds near the U-turn crossover in anticipation of oncoming U-turners. When the number of U-turning and opposing through vehicles are high, U-turners may find it difficult to find a suitable gap in order to merge into the through lane. This can create a long U-turn queue which may also affect the traffic flow from the section upstream of the queue. The performance of MUTs with unsignalized crossovers have been previously studied. The capacity and/or delays at the U-turn crossover are estimated using gap acceptance models like in (Al-Masaeid, 1999),(Liu et al., 2008) and (TRB, 2010). Another estimation method uses the output of traffic simulation software [(Pirdavani et al., 2011),(Liu et al., 2012)].

The crossover can be considered as a regular intersection where vehicles emanate from different links. Thus, it is easy to see that when the number of conflicting vehicles at the crossover increase, accidents and flow inefficiencies will rise due to the heightened interactions. Traffic signals at the crossover can be used. In the USA, warrants for providing signals in crossovers can be similar to warrants for protected left-turns. The criteria can be found in the Manual on Uniform Traffic Control Devises (FHWA, 2009). Works that considered signalized crossovers are (Bared and Kaisar, 2002), (Autey et al., 2010), and (Hughes et al., 2010). All these studies used simulation softwares in signal setting and travel time evaluation of MUT and the general findings were that MUT's can perform better than a conventional intersection. However, in these studies, the limitations of MUTs were not quantitatively discussed and the signal parameters were not detailed.

Bus Priority

Early applications of auxiliary signals outside of UAIDs are in the realm of bus priority research. Bus pre-signals were introduced by (Wu and Hounsell, 1998) in London. Basically, when the presence of an oncoming bus is detected, pre-signals located upstream of the main intersection turns red on the vehicle lanes in order to clear the space between the pre-signal and main signals and so that the bus will be allowed to reach the front of the main signal queue. This reduces delays for buses and ensures that the arriving bus can depart in the current cycle. An analytical and experimental study was conducted in Switzerland by (Guler and Menendez, 2014), (Guler and Menendez, 2013) for a pre-timed pre-signal system. More recently, an adaptive control algorithm was proposed in (He et al., 2016). An innovative application of the pre-signal was applied to single lane approaches in (Guler et al., 2016). Instead of providing a separate bus lane, a pre-signal was used to allow buses to utilize part of the lanes used by vehicles going in the opposite direction. These studies have shown the role of pre-signals in allowing multiple uses of the existing roadspace. These approaches, while useful, are only applicable in areas with separate bus lanes with relatively low frequency.

Increasing discharge lanes

Another application of auxiliary signals is to increase intersection throughput by increasing the number of discharge lanes. Similar to the concept of the Continuous flow intersection is the use of auxiliary signals to increase discharge capacity by "borrowing" exit lanes from the opposing direction. This follows the concept of displaced left-turns which aim to remove the left-through conflict at the main intersection and transferring it to an upstream location (Jagannathan and Bared, 2004),(Tabernero and Sayed, 2006), (Zhao et al., 2015a). (Zhao et al., 2013) introduced the EFL (Exit lane for Left-turn) design by assigning some opposing-through lanes as mixed-use lanes. These mixed-use lanes can be used by left-turning vehicles, thereby increasing the number of discharge lanes in situations with a high number of left-turning vehicles. The same author also conducted driving simulator experiments (Zhao et al., 2015b) to examine driver reactions to this design and it was found that drivers in the experiment showed some confusion when encountering this design for the first time but they eventually got accustomed to the design after several tries. While this design is theoretically more effective, driver confusion can significantly reduce the discharge rate at the main signal.

An idea that employs the concept of increasing discharge lanes but does not invole borrowing lanes from the opposing movement is the Tandem Sorting Strategy (TSS). In this strategy, auxiliary signals are used to "sort" or arrange queued vehicles at the main signal by phase. First introduced in (Xuan, 2011), TSS relaxed the practice of assigning a single lane to each traffic movement in an intersection. Xuan promoted TSS as a strategy for increasing intersection capacity under oversaturated conditions. This strategy was proposed to be a more practical alternative to UAIDs since it only required additional signals and did not need to increase road space or divert turning vehicles which requires some geometrical modifications. In this work, TSS capacity was formulated by solving a linear signal optimization problem for a single approach. In the formulation, the cycle length was a required input. The application domain of TSS was compared to RSI for various combinations of left-turn rates and demand proportion in the approach considered. This was done by plotting the capacity ratio between TSS and RSI.

Following this seminal work, (Yan et al., 2014) and (Zhou and Zhuang, 2014) used the lane-based paradigm to determine the lane configuration for tandem sorting strategy

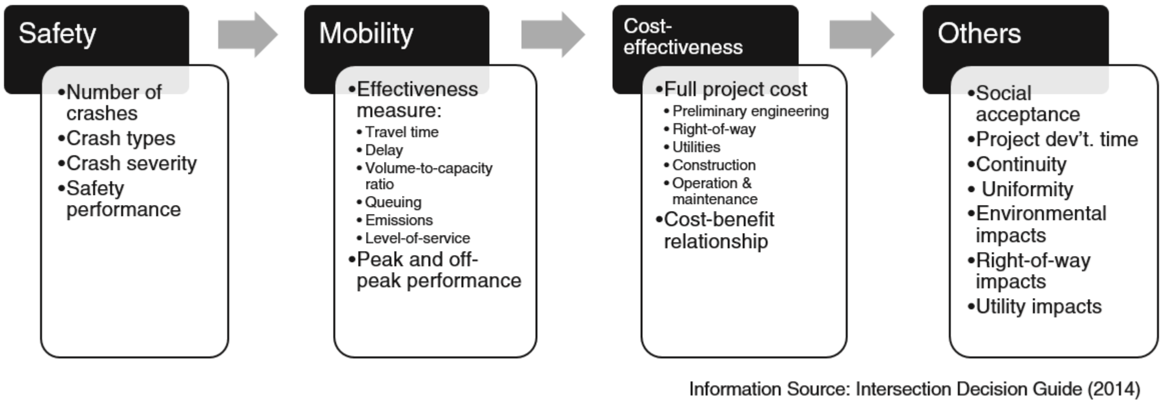


Figure 2.3. Intersection Design Factors.

and the phase swap strategy, respectively. Yan,et.al. formulated a Binary-Mixed-Integer-Linear-Programming model for the phase-swap strategy, a variation of TSS which is more suitable for shorter storage lengths. The formulation allowed for a group-based signal setting. This time, cycle length was a decision variable. Similar to the lane-based problem formulated by (Wong and Wong, 2003b), the problem aimed to maximize the flow multiplier μ, assuming that the demands in each of the OD pairs are proportional to each other. Meanwhile, Zhou formulated a mixed-integer non-linear program based on delay minimization. Here, cycle length and the phase sequence were also decision variables. Both approaches assumed that queues did not exceed the storage area. (Ma et al., 2013) conducted coordinated optimization of signal settings in TSS but only for a single approach with a pre-set cycle length. They formulated a two-level optimization problem where the lower level was a delay minimization problem and the high-level problem aimed to minimize the main signal green time. The aim of this formulation was to make the traffic flows as efficient as possible so that the extra green time can be given to other approaches. On a more microscopic level, (Li et al., 2014) used cellular automata theory to model the lane changing movements of drivers in the storage area, considering the slow probability, lane changing rules, and turning-deceleration rules. This is a good starting point for considering the relationship between driver behavior and allowable space in TSS. However, it still remains to be seen how actual vehicles will behave and react to the dynamic discharge lane allocation.

2.5 Considerations in Improving Intersections

Apart from operational performance, other factors must be considered when implementing improvements to existing intersections. Step-by-step guidelines in deciding the appropriate alternative intersection type are summarized in (Bowen et al., 2014). The factors below are briefly described here. Although these factors pertain to deciding on major intersection improvements (e.g., converting an RSI to an MUT), applicable factors must still be considered even for minor improvements such as those mentioned in section 2.1.1. It should be noted that "changing nothing" is also considered as an option.

1. Safety. Firstly, the candidate options must be able to satisfy safety requirements. Historic data on crashes must be gathered and the severity of each accident is given a weighting factor depending on whether the accident fell onto the following categories:
   - Fatal and Incapacitating Injury (FII)
   - Non-Incapacitating Injury (NII)
   - Property Damage (PD)

   In the state of Indiana, FII is given a factor of 58, NII is given 8, and zero for PD. Next, the Crash Reduction Factor is computed. CRF is the expected reduction in the number of crashes. An online Crash Modification Factors Clearinghouse[3] was established by FHWA to give the CRF of a given countermeasure. The CRFs for each proposed improvement measure is computed. If several improvement measures are combined, a composite CRF can be calculated. Finally, the Annual Expected Crash Reduction is computed. In an entirely new facility is added, then the evaluation is based on a qualitative assessment of its safety performance.
2. Mobility. This item refers to the effect of the proposed treatment on the mobility of the users. According to this source, mobility pertains to movements of traffic and considers not just the pace of movement but also accessibility and connectivity. Several measures of effectiveness (MOEs) are used. These are: travel time, delay, volume-to-capacity ratio, queuing , emissions, and level of service. The primary MOE suggested by the Technical Working Group in Indiana is the total or average travel time. Other MOEs and even multiple MOEs are also acceptable. The evaluation of alternatives must be carried out in the base and future years as well as in peak and off-peak periods.
3. Cost-effectiveness. The source recommends that cost calculations be computed separately for Mobility and Safety benefits. First, the total project cost must be converted to equivalent annual value. The costs must include all stages of the project, from preliminary engineering works, right-of-way acquisition, utilities, construction to operation

[3]http://www.cmfclearinghouse.org/

and maintenance costs. Next, the costs are related to the mobility benefits by computing the ratio between the total annual project cost divided by one or more mobility measures. For example: $500,000/100 sec average travel time. Safety measures are similarly calculated where a ratio between the annual cost and projected number of crashes is obtained. Example cost evaluations of alternative intersections can be found in (Gyawali, 2014) and (Hughes et al., 2010).

4. Others. Other performance measures that must be considered include the following:
   (a) Project development time. This is an important consideration especially if improvement measures have high urgency.
   (b) Right-of-way impacts. Apart from the costs involved in acquiring ROW, it should also be considered if all property owners, tenants, and lessees will be treated fairly and equitably. Also, impacts of ROW acquisition on existing residential, commercial areas, utility lines, rail roads, flood control infrastructure, etc. must be taken into account (URS, 2011).
   (c) Utility impacts. As mentioned in the previous item, impacts of the proposed alternative on existing utilities should be considered.
   (d) Continuity, Uniformity. The proposed project must be in harmony with existing infrastructure. If not, then measures to improve uniformity must be taken into account. In terms of driver expectation, non-uniform intersection types may cause confusion.
   (e) Environmental Impacts. These impacts are usually measured in terms of the vehicle emissions. The carbon footprint of the construction can also be considered.

   These considerations are summarized in Figure 2.3.

2.6 Gaps in the Literature

The literature review in the previous sections showed the limitations of conventional intersections and how alternative treatments can improve RSI performance. In this section, gaps in the literature are provided. Following the mentioned gaps, the sections that address such gaps are provided.

Measures to improve RSI's were categorized into two: a.) Physical and pre-set modifications and b.)Traffic Signal Modifications. These measures provide improvements in either safety, efficiency, or both. The efficiency improvements, while effective, measures fail to address issues caused by additional turning phases which include the underutilization of discharge lanes and the additional green time required for additional phases. As a result, the improvements that they bring to RSI's become limited when traffic demands reach oversaturated levels.

Tandem Sorting Strategy basically retains the original geometry of the intersection and changes the queuing order of vehicles in the storage area. In order to implement TSS in the field, some issues have yet to be considered. First, the capacity of the storage area is critical in determining the performance of TSS. Practically speaking, if TSS were to be implemented in an existing road section, the link length and the location of the pre-signal will constrain TSS capacity and this capacity is breached depending on the green durations and cycle length. Therefore, spatial factors should be embedded into the TSS signal setting algorithm. Existing TSS signal algorithms merely impose constraints that do not allow queues to extend out of the storage area which can under-utilize the available green time. Since the pre-signals can be easily turned on and off, the range of demands by which pre-signals should be activated has not been quantified in the literature. In addition, the TSS performance is severely affected by variations in discharge headways (Xuan et al., 2011). However, this has not been considered in the studies that considered signal control of TSS. Finally, most of the notable works on TSS/PSS focused on the formulation of lane assignment and signal control. These are very useful in the initial design of TSS but discussions on the benefits and drawbacks of using TSS on an operational perspective are still incomplete. For example, what is the demand boundary at which pre-signals should be activated? Or will TSS always work ideally under varying traffic compositions? If not, under which conditions will TSS not be beneficial? To answer these questions, a suitable estimation method must be established. These issues will be addressed in Chapter 4.

Many Unconventional Arterial Intersection Designs (UAIDs) have been proposed to improve the operational performance of RSI's. In the existing literature on the evaluation of UAIDs, the common evaluation method used often rely on a simulation software and a signal optimization software in order to ensure uniformity in the models used. Also, only the overall results are presented in comparison with RSI.

These kinds of analyses have some issues. Firstly, limitations in these softwares lead the users to run only a limited number of cases which may not reveal the full range of the applicability of each UAID. In most cases, UAIDs were not evaluated under oversaturated conditions which, when considered, may have revealed significant design vulnerabilities which may be significant in decision-making. On a practical note, relying on softwares may run the risk of the models being used inappropriately, leading to inaccurate conclusions as corroborated by (Abu-Lebdeh and Benekohal, 1997). Finally, although a Measure of Effectiveness (MOE) representing the overall performance of the intersection is important, a breakdown of the MOEs per movement have not been conducted, making it difficult to identify and compare weak points in the design and eventually propose countermeasures. These issues are addressed in Chapter 5.

Finally, a comparison between these alternative applications of auxiliary signals is essential in order to make better decisions regarding implementation. Moreover, these alternative applications should be compared to the performance of the Regular Signalized Intersection in order to determine the extent at which these applications can provide improvements. The comparison will be conducted in Chapter 6.

Table 2.4: Unconventional Arterial Intersection Designs

Intersection Type	Right-turn			Left-turn			Through			Lane Utilization	
	U	D	N	U	D	N	U	D	N	Multi-use	Single Use
Conventional Median U-Turn (Hummer, 1998a) (Hummer, 1998b)		o				o			o		o
Unconventional Median U-Turn (Shahi and Choupani, 2009) (El Esawey and Sayed, 2011a) (El Esawey and Sayed, 2011b)		o				o		o	o		o
Superstreet Median (Hughes et al., 2010) (Kim et al., 2007b) (Hummer, 1998a) (Haley et al., 2011)		o		o				o	o		o
Bowtie (Hummer, 1998a) (Reid and Hummer, 2001)		o				o			o		o
Jughandle (Hummer, 1998a) (Reid and Hummer, 2001)	o	o		o					o		o
Quadrant roadway (Reid, 2000) (Reid and Hummer, 2001)		o				o			o		o
Split intersections (Polus and Cohen, 1997) (Bared and Kaisar, 2000)			o			o			o		o
Parallel Flow Intersection (Dhatrak et al., 2010) (Parsons, 2009)	o					o			o	o	
Continuous Flow Intersection (Berkowitz et al., 1997) (Carroll and Lahusen, 2013)	o					o			o	o	
Upstream Signalized Crossover (Tabernero and Sayed, 2006) (El Esawey and Sayed, 2007)	o					o	o			o	

Legend:
U: Vehicles are diverted upstream of the main intersection.
D: Vehicles are diverted downstream of the main intersection.
N: Vehicles are not diverted and move in the same manner as in the regular signalized intersection.

Chapter 3

Preliminaries

3.1 Tandem Sorting Strategy

The Tandem Sorting Strategy (TSS) (Xuan et al., 2011) was proposed as an approach to increase the capacity of intersections with separate left turn phases. Left-turn phases, while necessary in many cases, compete for the limited green time during peak hours and contribute to longer cycle lengths. In this age of increasing car use rates, expanding road widths in order to provide larger capacity can be very costly. It is also impractical for roads that are only congested during certain times of the day. The TSS is a innovative approach that uses pre-signals to significantly increase intersection capacity without the need to widen road space. As of this writing, the author is not aware of any real-world implementation of the TSS. However, the concept of pre-signals are not new as they have already been implemented for providing bus priority [(Wu and Hounsell, 1998), (Guler and Menendez, 2014)]. The benefits from TSS were analytically shown to be significant and thus further studies that aim towards its applicability are warranted.

3.1.1 TSS Concept

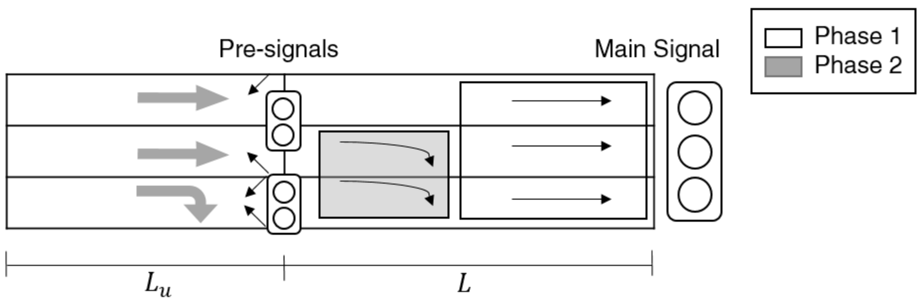


Figure 3.1. Tandem Sorting Strategy (TSS) Concept.

A diagram of the TSS mechanism is presented in Figure 3.1. In one approach, pre-signals are installed upstream of the intersection signal (or main signal). Upstream of the pre-signal, the turning vehicles (i.e., the right-turn and through vehicles) have their own lanes. The pre-signals are used to allow vehicles to enter the storage area by phase.

Using Figure 3.1 for reference, the procedure for vehicles entering the storage area is described here. Let us assume that the main signal red duration for the phases in the approach (i.e., phase 1 and 2) considered has just started.

The cycle lengths at the pre-signal and main signal are equal. However, the pre-signal only has 2 phases, allowing for longer green times. Let us denote the pre-signal green duration in phase i as g_i.

During the main signal red, the pre-signal green time for through vehicles will start and the through-vehicles will enter the storage area L using three lanes. At the end of the g_1 plus some clearance time, the pre-signal green duration for the right-turn vehicles will start. Similarly, the right-turn vehicles will enter the storage area and occupy two lanes. When the main signal green duration begins, the sorted vehicles discharge in their respective phases.

Note that the number of storage area-lanes that can be used by each turning movement is subject to the availability of exit lanes at the main signal. In this case, it is assumed that there are 3 exit lanes for through vehicles and only 2 for right-turn vehicles.

In the example shown, both right-turn and through traffic were able to occupy an additional lane in the storage area. The benefit of this increase in lanes over an approach without TSS can be seen in two ways: a. for the same green duration as a No TSS system, the discharge capacity per phase is larger or b. for the same discharge capacity as a No TSS system, the required discharge time is shorter which means decreased delay for vehicles in other phases. The lanes shared by vehicles from both phases are referred to as tandem lanes.

3.1.2 TSS Capacity Without Constraints

A four legged intersection with one congested approach is considered. TSS will be implemented on this approach in order to increase its capacity. The authors assume that the number of upstream lanes and tandem lanes for each phase are pre-determined, vehicles entering the storage area sort themselves evenly between the tandem lanes, and the number of tandem lanes are equal to the number of downstream lanes.

The pre-signal and main signal settings must be coordinated so they must have the same cycle length, C. An offset $\varphi = L/v_f$ is used so that the last vehicle discharged by the pre-signal can travel through the length of the storage area L at free flow speed v_f just as the green duration is about to end.

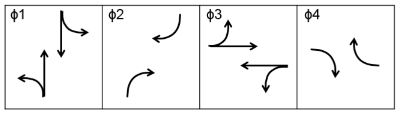


Figure 3.2. Four Phase Plan Assumed in the Study.

For any cycle length C, the main signal parameters are calculated based on the demand in each phase. A four-phase signal plan such as in Figure 3.2 is assumed and the green splits are calculated according to the flow ratios of each phase [(Webster, 1958)].

Now only one leg of the intersection is considered. TSS is applied to this link and Phases 1 and 2 are assigned to the two main movements. For simplicity, right-turn traffic is neglected and all vehicles are assumed to be passenger cars. The link discharge capacity Q at the approach is given by:

$$Q = G_1 S_1^m + G_2 S_2^m \tag{3.1}$$

where G_i is the green duration and S_i^m is the saturation flow rate per lane multiplied by number of discharge lanes. The indices are assigned as follows: the superscript refers to the main signal (m) or pre-signal (p) and the subscript i refers to the phase number.

Ideally, the previous equation should also be equal to the pre-signal discharge capacity Q^p, given by

$$Q^p = g_1 S^p + g_2 S^p \tag{3.2}$$

It is assumed that S^p is equal for both phases since vehicles execute the same movements as they enter the storage area. If $Q = Q^p$, it follows that:

$$g_i = \frac{G_i S_i^m}{S^p} \tag{3.3}$$

3.1.3 Phase-Swap Strategy

The Phase-Swap Strategy is a variation of TSS which can be applied when the storage area cannot accommodate vehicles from two-phases (Xuan, 2011). In this strategy, the vehicles from the pre-signal no longer occupy the storage area simultaneously. Instead, the TSS phasing plan is changed such that the storage area can be occupied by vehicles from only one phase.

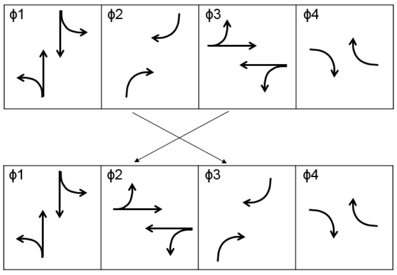


Figure 3.3. Phase swap at the main signal.

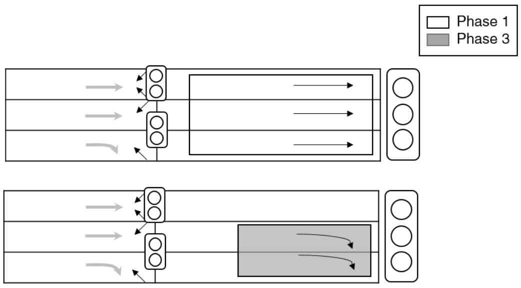


Figure 3.4. Lane configuration for Phase Swap Strategy.

Referring to Figure 3.3, phases 2 and 3 are swapped so that at phase 1, only the through vehicles need to discharge at the main intersection. Phase 2 is used for loading right-turners into the storage area, where they leave the main signal during phase 3 (see Figure 3.4).

3.1.4 Treatment of three turning movements

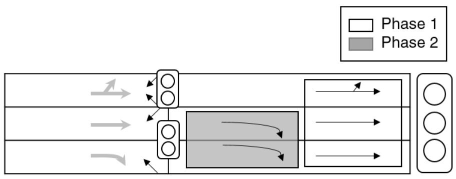


Figure 3.5. TSS with shared upstream lane.

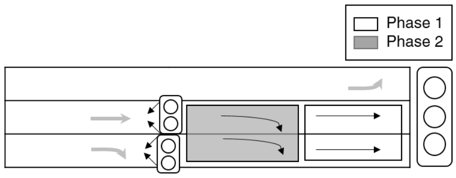


Figure 3.6. TSS with separate left-turn lane and no pre-signal.

There are two ways to consider all three turning movements. It is assumed that the left-turn vehicles discharge in the same cycle as the through vehicles following the phasing plan in 3.2. When the left-turner demand is low, the left most lane can be used as a shared lane. This design was mentioned in Xuan's seminal work on the TSS (Xuan, 2011). This is shown in Figure 3.5.

However, the use of shared lanes may run the risk of having left-turn vehicles in the middle lane and through vehicles in the left-most lane. This will cause conflicts at the discharge area which may be unsafe and inefficient. To prevent this from happening, an approach is to remove the pre-signal for left-turn vehicles and give them exclusive left-turn lanes. This was adapted in (Yan et al., 2014). In the interest of safety, this approach (Figure 3.6) will be used in this study.

3.2 Median U-Turns

The FHWA guideline on Alternative Intersections (Hughes et al., 2010) stated that much of the benefit from MUT is from the reduction of signal phases by removing left-turn vehicles and exclusive pedestrian phases. In its discussion, MUT had two signal phases, the first phase corresponding to the major approach and the second to the minor approach. To facilitate progression, the major street green phase starts n seconds before the main signal green starts and has the same duration. The typical phasing plans are shown in figures 3.7 and 3.8. Note that the diagram refers to drivers running on the right side of the road.

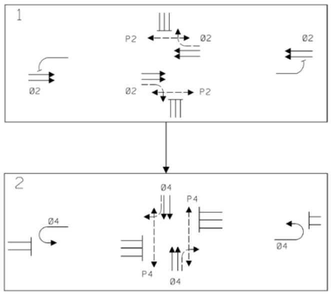


Figure 3.7. Typical 2-phase plan for a signalized MUT.

Source: (Hughes et al., 2010).

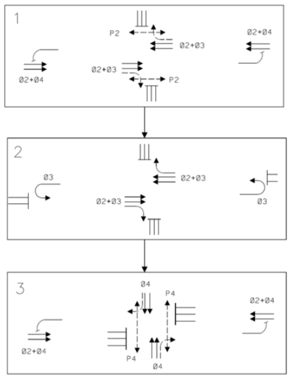


Figure 3.8. A typical phase plan for a signalized MUT (with overlaps).

Source: (Hughes et al., 2010).

3.2.1 MUT Signal Setting

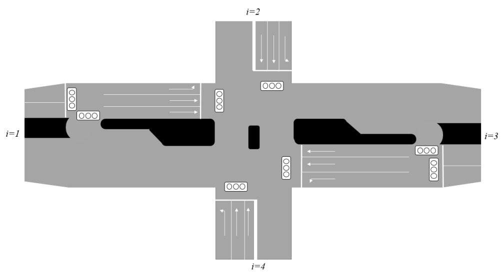


Figure 3.9. Typical Median U-turn design.

For signalized crossovers, recommendations for coordination between the crossover and main intersection signals are provided in FHWA's Median U-turn Intersection Informational Guide (Reid et al., 2014). It recommended that for the major approach, the red time at the crossover signal must several seconds before the red time at the main intersection. This is to minimize the number of queued vehicles between the crossover and the main intersection signals and also so that the U-turning vehicles can turn smoothly without being obstructed by queued vehicles.

The same guide also stated that for a two-phase MUT, the signal phasing and green durations at the crossover are to match that of the main intersection.

If these recommendations are followed, then it is reasonable to calculate the signal parameters based on the demand at the main intersection and simply apply the same green durations at the crossover, with the major crossover phase having the same duration as that of the main intersection's major approach. Meanwhile, the U-turn approach will have the same duration as that of the main intersection's minor approach. The green durations can be calculated using the same procedures as that of a regular signalized intersection with the difference that the link demands in the major approach now include the U-turning vehicles.

Say we want to set the green durations of a two-phase plan for the U-turn crossover in Figure 3.9. Suppose that the first phase is for the major direction (i=1,3) and the second phase is for the minor direction(i=2,4). The following demands q_{ij} from origin i to destination j must be considered for signal setting:

Table 3.1: Main Signal Demands

Phase	Movements	Demand	
		Left	Through
1-Major	Left, Through	$q_{12} + q_{32}$, $q_{34} + q_{14}$	$q_{13} + q_{43}$, $q_{31} + q_{21}$
2-Minor	Left, Through	q_{41},q_{23}	q_{42},q_{24}

The step-by-step procedure for computing the signal parameters is omitted here. Instead, the reader is referred to Chapter 5 of (Sigua, 2008) for examples.

Chapter 4

Tandem Sorting Strategy: Performance and Limitations

This chapter deals with the control of main and auxiliary signals in the Tandem Sorting Strategy, covering both undersaturated and oversaturated demands. Basically, signal setting under this strategy is similar to that of a regular signalized intersection except for the additional terms brought about by the presence of auxiliary-signals. A delay and capacity assessment of TSS is done by formulating signal setting optimization problems and performing analyses based on the results. In this chapter, auxiliary signals will be referred to as pre-signals to be consistent with previous studies. It is also assumed that vehicles drive on the left side of the road.

Before discussing the signal setting problems, a detailed discussion of the limiting factors affecting TSS performance is provided. The limiting factors affecting the performance of TSS come in two forms: Temporal and Spatial. Temporal constraints refer to limitations in the allowable green time at the pre-signal due to the limited cycle length. Spatial constraints refer to decrease in capacity due to limited storage. Both temporal and spatial constraints can be violated depending on the prevailing traffic demand compositions. The capacity formulation in (Xuan et al., 2011) is invoked and extended to consider storage constraints.

The following are the assumptions made in this study:

1. The densities and flows in the links can be described by a triangular fundamental diagram (Daganzo, 1997) shown in Figure 4.1. This means that vehicles travel at a uniform speed v_f and discharge at both the pre-signal and main signal at the saturation flow rate.
2. The vehicles sort themselves evenly among the tandem lanes upon inside the storage area.

The basic concepts and terminologies related to TSS are provided in Chapter 3.

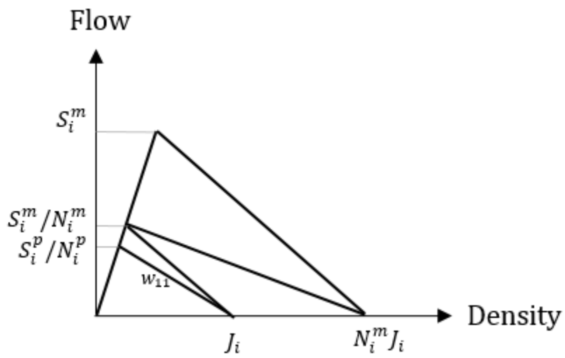


Figure 4.1. Fundamental Diagram.

4.1 Capacity Evaluation Considering Constraints

In TSS, the main signal capacity is limited by the pre-signal capacity. The capacity is fully utilized when the vehicles entering the storage area have enough time to occupy the tandem lanes before the main signal turns green. For example, if there is only one lane upstream of the pre-signal but there are three tandem lanes to be filled, more pre-signal green time is required to fill up all the tandem lanes. However, this green time is bounded by the cycle length or the maximum allowable green time. When this upper boundary is reached, the link discharge capacity becomes limited. This is why in some cases, fewer tandem lanes are more preferable (Gaspay et al., 2015). The temporal constraints in this section pertain to the availability of green time to load the storage area.

4.1.1 Split Constraint

The split constraint is violated when the required pre-signal green times (equation 3.3) exceed the cycle length. This was implicitly considered in the pre-signal capacity function [equation 4.1 in (Xuan et al., 2011)]. Assuming a minimum red time τ^p between phases gives us the following limiting equation:

$$g_1 + g_2 \leq C - 2\tau^p \tag{4.1}$$

This implies that when equation 4.1 is violated, the maximum possible main signal throughput is limited by the pre-signal capacity. Since the pre-signal green durations are computed based on those of the main signal, the split constraint is expressed as follows:

$$\lambda_1\alpha_1 + \lambda_2\alpha_2 \leq 1 - \frac{2\tau^p}{C} \tag{4.2}$$

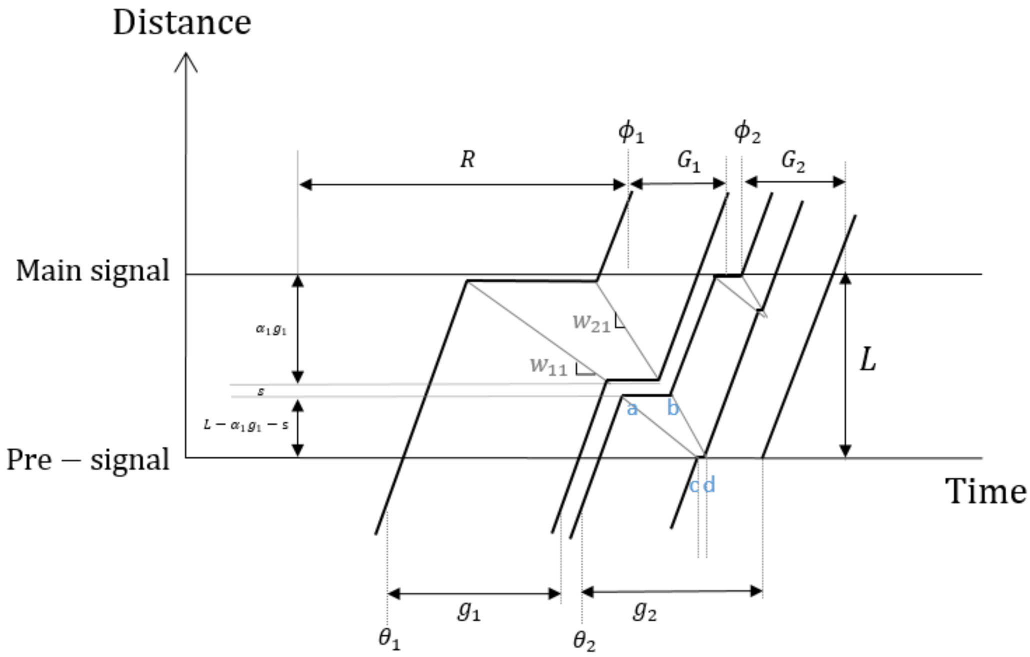


Figure 4.2. Time-space diagram of queue evolution in the storage area.

where λ_i is the green split of phase i and α_i is the ratio of saturation flow rates at the main signal to the pre-signal (i.e., $\alpha_i = \frac{S_i^m}{S_i^p}$).

4.1.2 Storage length constraint

If vehicles will distribute themselves evenly among the tandem lanes, the required storage length L_{ri} in phase i is given by a function of the number of discharging vehicles in each cycle divided by the product of the critical density per lane J and the number of tandem lanes in phase i, N_{mi}:

$$L_{ri} = \frac{g_i S_i^p}{N_i J}, i \in [1 : n] \tag{4.3}$$

When storage length is inadequate, the discharge capacity becomes less than that specified in equation 3.1. The storage length is considered to be adequate when the following equation is satisfied:

$$L \geq L_{r1} + L_{r2} \tag{4.4}$$

Equation 4.4 is called the storage length constraint. To represent the relationship between queues and storage length, the queue length is modeled using LWR theory (Lighthill and Whitham, 1955) (or shockwave theory). Figure A.1 shows the vehicle queues when the storage length is inadequate with respect to the phase 2 vehicles. The black lines show the vehicle trajectories at key durations in the cycle and the gray lines are the shockwaves. When the queue reaches the pre-signal entrance while the main traffic light is still red, the pre-signal continues to be green but no vehicles can enter. This green time is considered as "wasted" time and is marked by letters c and d in Figure A.1. Once the main signal light turns green, the vehicles are discharged and the waiting vehicles outside of the storage area can enter.

The useable green time in this case is given by $g_2 - \rho$, where ρ is the wasted green time (derivation is provided in Appendix A).

$$\rho = g_1\left(\gamma - \frac{\alpha_1}{v_f} + \frac{\alpha_1}{w_{22}}\right) - \theta_2 + R + h - \frac{s}{v_f} + (L-s)\left(\frac{1}{w_{22}} - \frac{1}{w_{12}} - \frac{1}{v_f}\right) \tag{4.5}$$

where:
$\gamma = \frac{n_2 S_2^p}{N_2 S_2^m}$
$\alpha = \frac{S_i^p n_i}{N_i J}$
s: spacing between vehicles (m)
h: discharge headway (sec)
w_{1i}: stopping shockwave speed for phase i (m/s)
w_{2i}: starting shockwave speed for phase i (m/s)

The pre-signal capacity is now reduced to:

$$Q^p = g_1 S^p + (g_2 - \rho) S^p \tag{4.6}$$

4.1.3 Upstream link length constraint

Another constraint appears when the link length upstream of the pre-signal, L_u, has a limited storage capacity. In this case, the maximum discharge capacity becomes a function of available storage space in L_u. (Lieberman et al., 2000) introduced the following function for the maximum cycle length that must be used to avoid spillovers:

$$C_{max} = \frac{L_u h}{l_v \frac{G_i - l}{C}}\left(1 - \frac{W}{L_u} - \frac{(SF)l_v}{L_u}\right) \tag{4.7}$$

where h is the discharge headway, l_v is the effective length of each vehicle, W is the upstream intersection's width, SF is a safety factor, G_a is the green time, and l is lost time. The last two terms inside the parentheses are allowances provided for safety purposes. This function is used in this work but the safety terms are neglected. Simplifying this equation yields the maximum green time which can be interpreted as the maximum number of vehicles that can be discharged by the pre-signal:

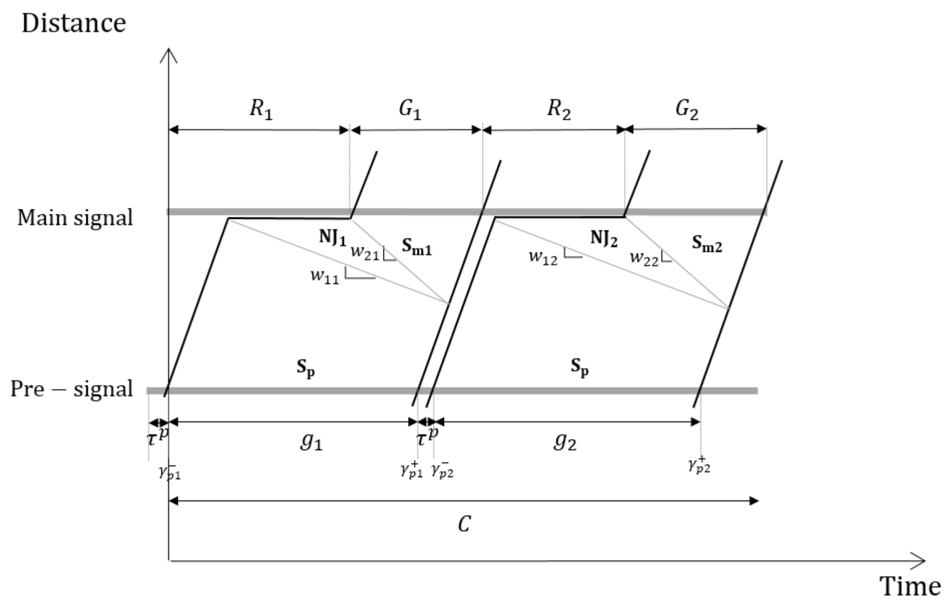


Figure 4.3. Queue evolution in Phase Swap Strategy.

$$g_{max} = min\left(\frac{L_u h}{l_v}\right), i \in [1:n] \tag{4.8}$$

Here, n is the number of phases in the link considered. This constraint gives a green duration that limits the number of vehicles that can be discharged at the pre-signal to that which can be accommodated by provided link length.

4.2 Phase Swap Strategy

To remedy the loss in capacity due to inadequate storage length, (Xuan, 2011) proposed an alternative to TSS which requires a shorter storage length called the Phase Swap Strategy (PSS). The intersection can be set up to run only PSS as in (Yan et al., 2014) but ideally, it is also possible to switch between TSS and PSS depending on which option gives a better performance.

4.2.1 PSS Split Constraint

The evolution of queues in the storage area under PSS for two phases are shown in Figure 4.3. To derive the split constraints under PSS, the range of possible green times are first formulated. When the link is on PSS mode, the earliest time that a vehicle waiting at the pre-signal can enter the storage area is when the queue from the previous phase has cleared. This is denoted by γ_{pi}^{-} in Figure 4.3 for phase i. The upper bound of the duration of G_{pi} is given by γ_{pi}^{+} which is set so that the last vehicle in phase i can clear the storage area. Setting the beginning of each cycle as R_{m1}, the durations are given below:

$$\gamma_{p1}^{+} = R_1 + G_1 - \frac{L}{v_f}$$

$$\gamma_{p1}^{-} = max(\gamma_{p1}^{+} - g_1, \tau^p - \frac{L}{v_f})$$

The updated duration of g_1 is given by:

$$g_1 = min(g_1^0, \Delta\gamma_{p1}) \tag{4.9}$$

where $\Delta\gamma_{p1} = \gamma_{p1}^{+} - \gamma_{p1}^{-}$ for each phase i and g_1^0 is the unupdated pre-signal green time. The equations for phase 2 are similarly derived below:

$$\gamma_{p2}^{+} = C - \frac{L}{v_f}$$

$$\gamma_{p2}^{-} = max(\gamma_{p2}^{+} - g_2, \gamma_{p1}^{+} + \tau_p)$$

The duration of g_2 is then given by:

$$g_2 = max(\gamma_{p2}^{+} - g_2, \gamma_{p1}^{+} + \tau_p) \tag{4.10}$$

Simplifying equations 4.9 and 4.10 lead to the following general equation for the updating the pre-signal green times under PSS:

$$g_i = min(g_i^0, R_i + G_i - \tau^p) \tag{4.11}$$

Here, G_i refers to the main signal green time being served by pre-signal phase i and R_i refers to the red duration preceding G_i. We can now explicitly express the PSS split constraint:

$$g_i \leq R_i + G_i - \tau^p \tag{4.12}$$

4.2.2 PSS Storage Length Constraint

The queuing situation when the required storage length is insufficient (i.e, $L < L_{ri}$) is illustrated in the time-space diagram in Figure 4.4. The bold lines are trajectories of

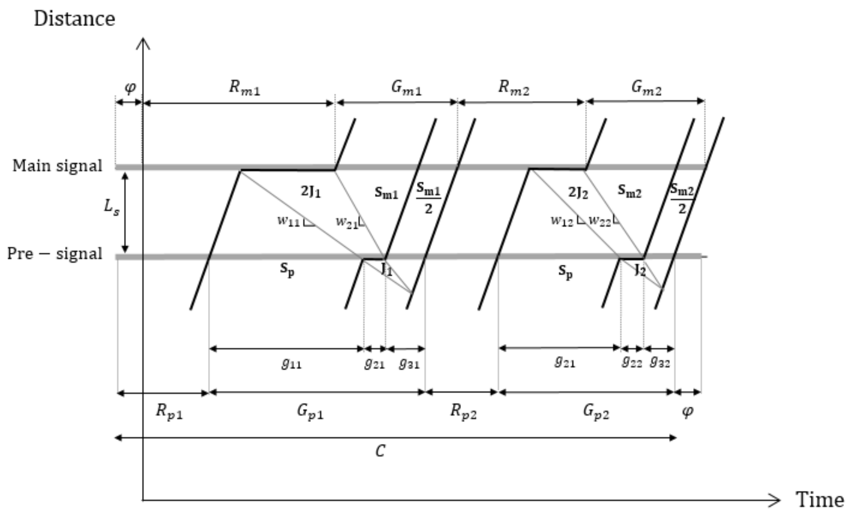


Figure 4.4. PSS: Storage length constraint violations.

vehicles at the shockwave boundaries. Note that the red times R_1 and R_2 are locally indexed meaning they are numbered with respect to the phases in the link considered.

The pre-signal green time for phase i, g_i, is divided into three parts denoted by ϵ_{1i}, ϵ_{2i}, and ϵ_{3i}.

The first part ϵ_{1i} is the extent of green time used to fill up the storage area. This is given by:

$$\epsilon_{1i} = \frac{L_s N_{mi} J}{S_p} \tag{4.13}$$

When the queue in the storage area exceeds the storage length, the rest of the oncoming vehicles have to wait until the space ahead of them is clear before they can enter the storage area. While waiting, the pre-signal is already green but no vehicles are entering the storage area. This is referred to as the wasted time, which is given by ϵ_{2i}:

$$\epsilon_{2i} = max(0, \Delta\omega_i) \tag{4.14}$$

where:

$$\Delta\omega_i = \omega_i^+ - \omega_i^-$$

$$\omega_i^- = \gamma_{pi}^- + g_{1i}$$

$$\omega_1^+ = min(R_1 + \frac{L}{w_{21}}, \gamma_{p1}^+)$$

$$\omega_2^+ = min(C - G_2 + \frac{L}{w_{22}}, \gamma_{p2}^+)$$

Finally, the third part is the green duration which overlaps (or not) with the main signal green. During this duration, the benefits from TSS are lost and for phase i, the number of tandem lanes used are equal to the number of lanes upstream of the pre-signal. This is given by the following equation:

$$\epsilon_{3i} = min(g_i - \epsilon_{1i} - \epsilon_{2i}, G_i - \frac{\epsilon_{1i} S_i^p}{S_i^m}) \tag{4.15}$$

Combining equations 4.13, 4.14, and 4.15 give us a general equation of discharge capacity Q_i for phase i which is a function of storage length:

$$Q_i = (\epsilon_{1i} + \epsilon_{3i}) S_{pi} \tag{4.16}$$

For any storage length l_s, the limit of equation 4.16 as l_s approaches zero gives us the following:

if $G_i > g_i$,

$$\lim_{l_s \to 0} Q_i(l_s) = g_i S_i^p$$

and if $G_i \leq g_i$,

$$\lim_{l_s \to 0} Q_i(l_s) = G_i S_i^p$$

The equations above imply that the PSS capacity approaches the No PSS capacity as the the storage length is decreased.

4.2.3 Pre-signal operation algorithm

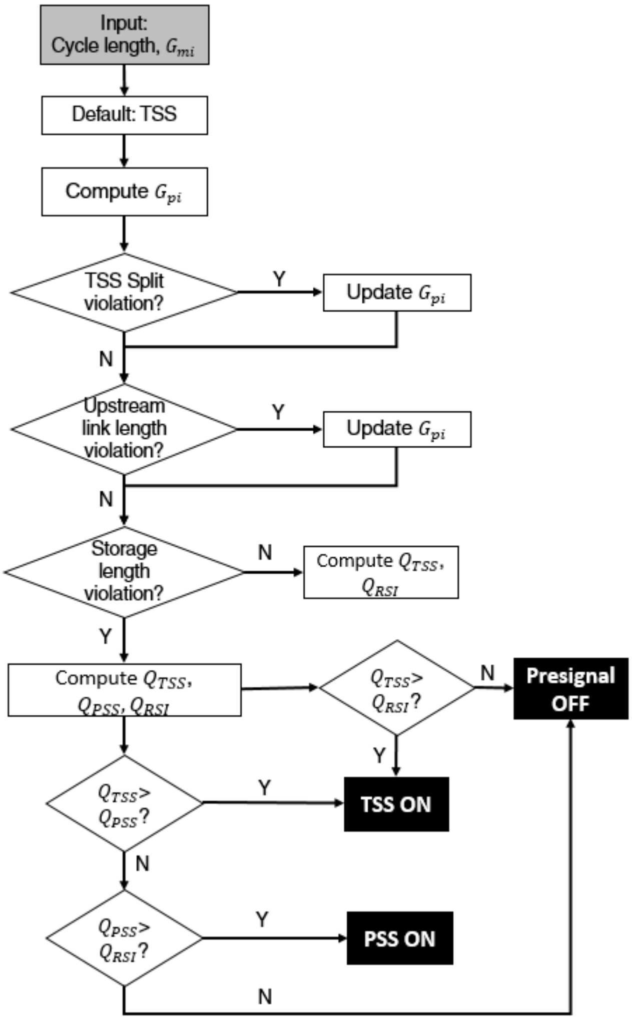


Figure 4.5. Pre-Signal Operation Algorithm.

When the demands increase, there is a higher chance that the storage length becomes inadequate and may decrease the applicability of TSS. At this point it would be useful to check if the Phase Swap Strategy can give a higher capacity. To control the pre-signal under the split and/or storage constraints, the algorithm shown in Figure 4.5 is used to adjust the pre-signal parameters. This is a simplified algorithm that yields optimal capacities. Note that in this algorithm, it is possible to switch between turning the pre-signal on or off or PSS.

The limiting equations in the previous sections are consolidated into the operational algorithm shown in Figure 4.5. This algorithm can be used when the pre-signal is turned on. A simpler approach suggested in (Xuan et al., 2011) would be to turn on the pre-signal when the queues have reached the pre-signal area. This can simply be done by installing detectors near the pre-signal.

When the intersection demands reach an oversaturated level, the cycle lengths calculated using the optimization problem introduced will be large enough to be impractical. In practice, a maximum cycle length is usually set by the traffic controller. Since the cycle length is shorter than what is ideally required, the intersection is in an oversaturated state and residual queues begin to form.

The signal control policy for oversaturated states may differ between traffic controllers as mentioned in (Gettman et al., 2012) and is another complex problem in itself. It should be noted, however, that the signal control algorithms presented here can be applied to any oversaturated intersection signal control policy. Cycle lengths are treated here as user inputs as they are also dependent on the policy selected. Several methods for detecting and managing oversaturation have been discussed in (Sims and Dobinson, 1980), (Wu et al., 2010), and (Liu et al., 2009).

In this study, the green times are allocated using an equisaturation policy (Webster, 1958) where the green times are proportional to the demand.

4.3 Tandem Sorting Strategy for Undersaturated Conditions: A two-step optimization

The discussion in (Xuan, 2011) asserts that the Tandem Sorting Strategy is better applied for oversaturated cases. Therefore, it skipped the evaluation for undersaturated conditions. Apart from an increase in capacity, another advantage of increasing discharge capacity is that the total discharge time can be shortened, leading to a decrease in the required cycle length which benefits even vehicles that are not in the TSS link. In this section a two-step optimization problem is presented which can calculate the optimal signal parameters.

4.3.1 Step One: Delay Minimization Problem

The optimization of cycle length is done by minimizing the total delay D for all movements x in each link k.

$$\min D = \sum_{k=1}^{K} \sum_{x=1}^{X} d_{xk} q_{xk} \tag{4.17}$$

In links without pre-signals, the expected delay is given by Webster's delay function:

$$d_{xk} = 0.9\left(\frac{C(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)}\right) \tag{4.18}$$

Note that the adjustment term used by Webster in (Webster, 1958) is replaced by the factor 0.9 (Cheng et al., 2016). In links with pre-signals, the expected delay has two parts, the delay upstream and downstream of the pre-signal (d_k^u and d_k^d, respectively).

$$d_{xk} = d_k^u + d_k^d \tag{4.19}$$

The delay upstream of the pre-signal is given by equation 4.18.

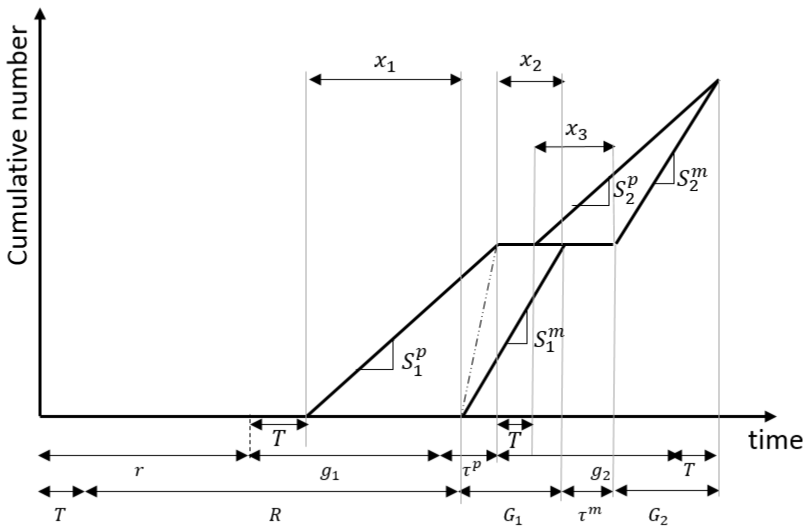


Figure 4.6. Arrival and departure curves in the storage area.

Meanwhile, the delay downstream of the main signal is derived from cumulative curve plot shown in Figure 4.6.
Derivation of the delay between pre-signals:
Phase 1:

$$= \frac{1}{2}\frac{x_1 g_1 S_1^p + x_2 g_1 S_1^p}{g_1 S_1^p}$$

$$= \frac{1}{2}x_1 + x_2$$

$$x_1 = (C + T - G_1 - G_2 - \tau^m) - (C - g_2 - g_1 - \tau^p + T)$$

$$x_2 = (C + T - G_2 - \tau^m) - (C - g_1 - g_2 - \tau^p + T + g_1)$$

$$x_1 + x_2 = -G_1 - 2G_2 + g_1 + 2g_2 + 2\tau^p - 2\tau^m$$

Since $\tau^m = \tau^p$, the equation is simplified to:

$$d_1^d = \frac{1}{2}(g_1 - G_1) + (g_2 + G_2) \tag{4.20}$$

Phase 2:

$$\begin{aligned} &= \frac{1}{2}\frac{x_3 g_2 S_2^p}{g_2 S_2^p} \\ &= \frac{1}{2}x_3 \\ x_3 &= (C + T - G_2) - (C - g_2 + T) \\ d_2^d &= \frac{1}{2}(g_2 - G_2) \end{aligned} \tag{4.21}$$

Equations 4.20 and 4.20 are the same as those in (Zhou and Zhuang, 2014). This delay is just an approximation because it assumes that G_i is fully utilized which is not the case when the split constraint is violated. However, the difference is negligible.

Constraints:
1. The sum of green times plus the total lost time (given by the lost time τ^m multiplied by the number of phases I) must be equal to the cycle length.

$$\sum G_i + I\tau^m = C \tag{4.22}$$

2. The pre-signal and main signal green times for phase i, g_i and G_i, respectively, must be greater than the prescribed minimum green time.

$$g_i, G_i \geq g_{min} \tag{4.23}$$

3. The main signal capacity must be greater than the pre-signal capacity.

$$g_i S_i^p \leq G_i S_i^m \tag{4.24}$$

4. The sum of the pre-signal green times should not exceed the cycle length minus the sum

of pre-signal green times (τ_p).

$$g_1 + g_2 \leq C - 2\tau_p \tag{4.25}$$

5.The degree of saturation for each movement upstream of the pre-signal or main signal without pre-signals must be less than an allowable value, x_{all}.

$$\frac{q_i C}{G_i S_i^m} \leq x_{all} \tag{4.26}$$

This problem is a nonlinear optimization problem with linear constraints. In (Gallivan, 1982), it was shown that Webster's delay function is convex with respect to $1/C$ and g/C. Since the additional delay terms are linear, the solution to the problem is the global optimum.

4.3.2 Step Two: Capacity Maximization Problem: TSS

After the main signal parameters have been obtained, the pre-signal parameters are updated by solving the lower-level problem whose objective is to maximize the pre-signal capacity (i.e. throughput). The constraints introduced in the previous section are integrated to form the following maximization problem. Compared to other formulations, this maximization problem explicitly considers queue lengths in the storage area as well as queue spillovers outside of the storage area.
Objective function:

$$\max_{\mathbf{g},\theta_2 \geq 0} g_1 S_1^p + \hat{g} S_2^p \tag{4.27}$$

Constraints:
1. The sum of pre-signal green times must not exceed the cycle length minus total lost time.

$$g_1 + g_2 \leq C - 2\tau^p \tag{4.28}$$

2. The pre-signal green duration for phase i must not exceed the minimum green time which are either a function of the upstream link length (Equation 4.8) or the main signal green duration corresponding to that phase.

$$g_i \leq \bar{g} \equiv \min\{G_i S_i^m / S_i^p, g_{max}\} \tag{4.29}$$

3. This constraint restricts the starting time of the first pre-signal phase, θ_1 so that it does not overlap with the previous cycle's phase and so that all vehicles discharging from the pre-signal can reach the main signal.

$$-T + \tau^p \leq \theta_1 \leq R \tag{4.30}$$

4. This constraint ensures that the second pre-signal phase must end early enough so that the last discharging vehicle can reach the main signal before the G_2 ends.

$$\theta_2 + g_2 = C - T \tag{4.31}$$

5. This constraint ensures that the pre-signal phases follow each other.

$$\theta_2 = \theta_1 + g_1 + \tau^p \tag{4.32}$$

6. This constraint ensures that a vehicle from the second pre-signal green phase reaches the main signal at the beginning of green.

$$\theta_2 \leq R + G_1 + \tau^p \tag{4.33}$$

7. This constraint accounts for queue spillover outside of the storage area. The evolution of queues in the storage area is plotted in Figure A.1. When the queue length exceeds the storage area, part of the pre-signal green duration (indicated by line segment cd) is not utilized for discharging. This "wasted" green time is given by ρ and the useable green time is given by:

$$\hat{g} = \min\{g_2, g_2 - \rho\} \tag{4.34}$$

The problem now becomes a non-linear problem due to equation 4.34. This constraint can be converted to a linear problem by using binary variables. Equation 4.34 is expressed in terms of binary variables z_1 and z_2:

$$\hat{g} \leq g_2 \tag{4.35}$$

$$\hat{g} \leq \hat{g} - \rho \tag{4.36}$$

$$\hat{g} \geq g_2 - M(1 - z_1) \tag{4.37}$$

$$\hat{g} \geq (g_2 - \rho) - M(1 - z_2) \tag{4.38}$$

$$z_1 + z_2 = 1 \tag{4.39}$$

where M is a reasonably large positive number.

This problem is now a Binary Mixed Integer Linear problem which is relaxed into the linear programming problem below:

$$\max_{g_1, \hat{g}_2, \theta_2 \geq 0} g_1 S_1^p + \hat{g} S_2^p \tag{4.40}$$

$$g_1 + g_2 \leq C - 2\tau^p \tag{4.41}$$

$$g_i \leq \bar{g} \equiv \min\{G_i S_i^m / S_i^p, g_{max}\} \tag{4.42}$$

$$-T + \tau^p \leq \theta_1 \leq R \tag{4.43}$$

$$\theta_2 + g_2 = C - T \tag{4.44}$$

$$\theta_2 = \theta_1 + g_1 + \tau^p \tag{4.45}$$

$$\theta_2 \leq R + G_1 + \tau^p \tag{4.46}$$

$$\hat{g} \leq g_2 \tag{4.47}$$

$$\hat{g} \leq g_2 - \rho \tag{4.48}$$

The proof of equivalence between the The Binary Mixed Integer Linear Problem (BMILP)and the Relaxed Linear Problem (RLP) is given below:

Proposition: The Binary Mixed Integer Linear Problem is equivalent to Relaxed Linear Problem

Proof: Theorem 1 of the *Basic Theorems of Linear Programming* (Lasdon, 1970) states that the objective function assumes its minimum (or maximum) at an extreme point of the constraint set . Thus, the RLP will be maximized at either of the extreme points a or b. If $a < b$, the RLP will be maximized at a and $\hat{g} \leq b$ holds. The same is true for $a \geq b$.

The delay minimization problem provides the main signal parameters and the upper limit of the pre-signal green times. Since the results in the first step do not consider the effect of link length, the pre-signal green times as well as the starting times of each phase are provided by the capacity maximization problem solution. The capacity maximization problem for the Phase Swap Strategy is thus formulated similar to the relaxed linear problem. The delays in the storage area for TSS and PSS have little difference so the results of the delay minimization problem (save for the arrangement of the phases) are also used for PSS. It should be noted that the result of the capacity maximization problem was found to be equal to the result of the equations in the previous sections for both TSS and PSS.

4.3.3 Phase Swap Strategy

The capacity maximization problem is also formulated similarly for the phase swap strategy.

$$\max_{\hat{g},\theta_2 \geq 0} \hat{g}_1 S_1^p + \hat{g}_2 S_2^p \tag{4.49}$$

$$g_1 + g_2 \leq C - 2\tau^p \tag{4.50}$$

$$g_i \leq \bar{g} \equiv \min\{G_i S_i^m / S_i^p, g_{max}\} \tag{4.51}$$

$$\theta_1 \geq -T + \tau^p \tag{4.52}$$

$$\theta_1 + g_1 \leq R_1 + G_1 - T \tag{4.53}$$

$$\theta_2 \geq \theta_1 + g_1 + \tau^p \tag{4.54}$$

$$\theta_2 + g_2 = C - T \tag{4.55}$$

$$\hat{g}_1 \leq g_1 \tag{4.56}$$

$$\hat{g}_1 \leq g_1 - \rho_1 \tag{4.57}$$

$$\hat{g}_2 \leq g_2 \tag{4.58}$$

$$\hat{g}_2 \leq g_2 - \rho_2 \tag{4.59}$$

where:

$$\rho_1 = R_1 + L\left(\frac{1}{w_{21}} - \frac{1}{v_f} - \frac{1}{w_{11}}\right) - \theta_1$$

$$\rho_2 = R_1 + G_1 + R_2 + L\left(\frac{1}{w_{22}} - \frac{1}{v_f} - \frac{1}{w_{12}}\right) - \theta_2$$

4.4 Stochastic Headways

The results and formulations presented so far only consider deterministic headways. In reality, however, the discharge headways vary. This variation in discharge headways can lead to what Xuan (Xuan, 2011) refers to as *lane failure*. Lane failure occurs when vehicles in the storage area fail to discharge in their assigned cycle, forming a residual queue in the storage area. Vehicles in the next cycle must then wait for the next cycle to be able to discharge. Meanwhile, queue builds up upstream of the pre-signal. The occurrence of lane failure will lead to a worse performance under the Tandem Sorting Strategy and must be avoided.

Equation 4.60 shows the probability of lane failure formulated by Xuan in (Xuan, 2011). It is probability that the sum of discharge headways H_i exceeds the mean sum of headways which is given by the deterministic discharge headway H multiplied by the expected number of discharging vehicles m_X per lane (the subscript X is for the movement type L,T, or R). This equation holds under the assumption that vehicles arrange themselves evenly among the tandem lanes.

$$p_X = Pr\left\{\sum_{i=1}^{m_X^s} H_i \geq m_X H\right\} \approx \Phi\left(-\frac{m_X - m_X^s}{\sqrt{m_X^s}\gamma'}\right) \tag{4.60}$$

By virtue of the Central Limit Theorem, the probability of lane failure can be said to follow a normal distribution with random variable m_X^s, mean m_X, and standard deviation $\sqrt{m_X^s}\gamma'$. γ' is the coefficient of variation of the vehicle headways.

The probability of lane failure can be reduced in two ways: first is to increase the main signal green duration in order to accommodate all vehicles and second is to decrease the pre-signal duration so that fewer vehicles can enter the storage area. Increasing the main signal duration will affect the vehicles in other approaches so the second option is chosen instead.

The green time reduction is calculated as the duration that will reduce the probability of lane failure to a specified threshold value, ν. The standardized value of the random variable m_X^s is given as $k_X = \frac{m_X - m_X^s}{\sqrt{m_X^s}\gamma'}$. For a given probability value ν, the corresponding value of k_X can be obtained by taking the inverse of the cumulative normal distribution function with mean=0 and standard deviation=1. Knowing k_X, the stochastic number of vehicles discharged m_X^s can be easily obtained. The green time required to discharge these number of vehicles in each tandem lane is expressed as:

$$g'_p = \frac{m_i^s n_i}{S_i^p} \tag{4.61}$$

where n_i is the number of tandem lanes in phase i. Finally, the green time reduction is then given by:

$$\delta_i = g_p - g'_p \tag{4.62}$$

4.5 Results and Discussion

A numerical experiment is conducted to highlight the features of TSS and the algorithms presented. A four-legged intersection where pre-signals are installed on two opposing links of a four-legged intersection (Figure 4.7) is considered. These links are referred to as the major links. The main intersection phasing plan is that in Figure 3.2. The free flow speed is assumed to be 60 km/hr and the jam density is 140 veh/km. A uniform saturation flow rate of 2000 veh/hr is used for all approaches.

The calculations were coded into the MATLAB version R2016a software. In MATLAB, several algorithms were recommended for solving small to medium-sized non-linear problems. At first, the default algorithm (interior point) was used but some infeasibility issues were encountered. Finally, the Sequential Quadratic Programming (SQP) Algorithm was used because of its speed and because the algorithm it was able to recover from infeasible results (MathWorks, 2016).Thus, the delay minimization (step 1) problem was solved using SQP Algorithm while the capacity maximization (step 2) problem was solved using the Simplex algorithm.

4.5.1 Application domain in Undersaturated Demands

From a link perspective, the pre-signal seems to increase delays in undersaturated intersections. This was noted by Xuan in (Xuan, 2011). However, an additional benefit to adding discharge lanes is that the green duration can be reduced, thereby decreasing the required green times and overall, the cycle length. Therefore, TSS can be beneficial even when the intersection is in an undersaturated state.

A performance comparison is provided in Figure 4.8 for 60 % Major demand and selected right-turn rates. TSS delays are much higher at low demands. This is due to the additional delay at pre-signals. For demands upto 1500 veh/hr, the main signal parameters and cycle lengths are operating at minimum green time. Due to the additional discharge lane, the required green time for TSS is shorter and thus the minimum cycle length is

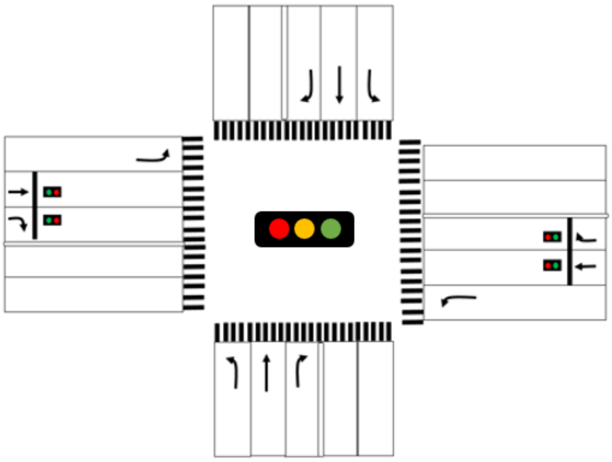


Figure 4.7. Intersection configuration.

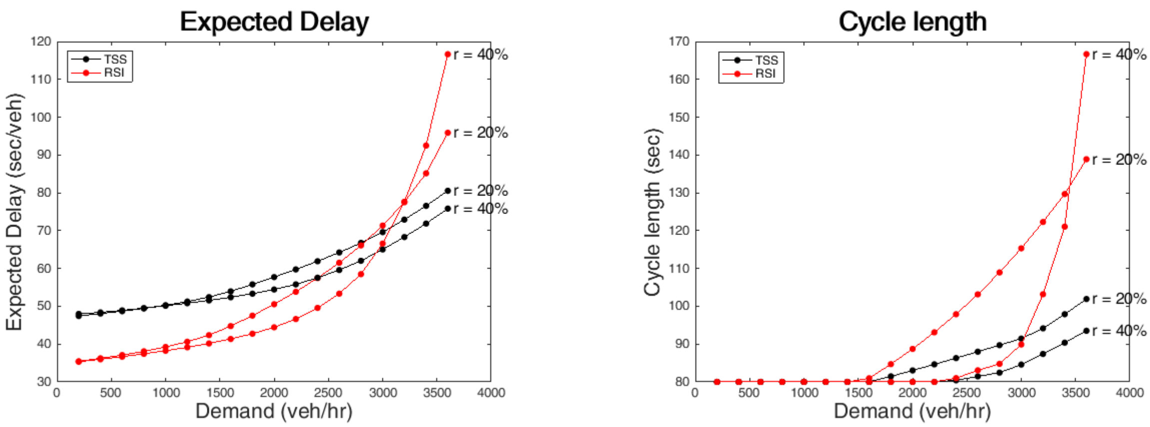


Figure 4.8. TSS vs RSI performance comparison.

applicable for a much larger array of demands. For RSI, the increase in right turn rates cause an abrupt increase in cycle length due to the additional green time needed for the right-turn phases. Notice how the expected delay shoots up at demands beyond 3000 veh/hr, indicating that the system is approaching saturation. Turning on the pre-signal slows down this approach to an oversaturated state at high right-turn rates.

By performing this kind of comparative analysis, the demand boundary between TSS and RSI can be determined. For pre-timed signals, knowledge of historical demand patterns can be used to determine when the pre-signals can be turned on. By activating TSS early on, reaching an oversaturated state may be avoided.

For the demands considered, the domain of application of TSS with respect to demand compositions are presented in Figures 4.9 and 4.10. The shading colors are based on the ratio of delay between TSS and RSI. In Figure 4.9, the blue shaded area refers to the demand scenarios where TSS yields lower delays than RSI. The TSS domain is wider at lower right-turn rates because the increased number of discharge lanes reduces the required green time necessary for each cycle.

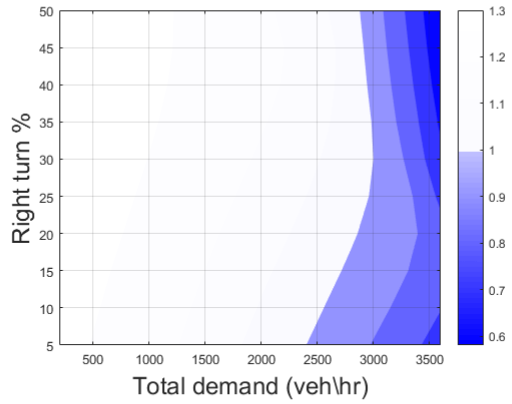


Figure 4.9. TSS:RSI delay at 60% Major demand.

The application domain of TSS with respect to the demand composition between the major and minor links is presented in Figure 4.10. As expected, the TSS domain is wider when there are more vehicles on the major link.

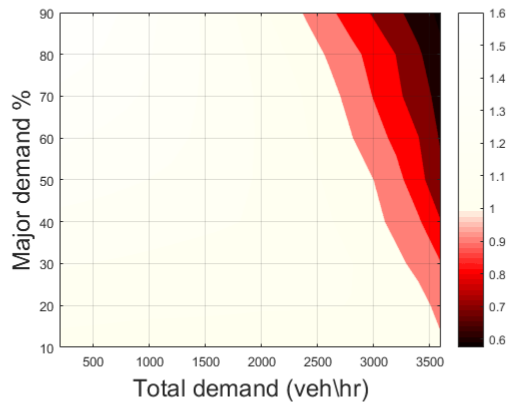


Figure 4.10. TSS:RSI delay at 30% Right-turn rate.

4.5.2 Effect of discharge headway variations

The applicability of TSS may also be affected by variations in discharge headways. As mentioned in Section 4.4, the pre-signal green durations are adjusted to reduce the probability of lane failure. One important parameter here is the coefficient of variation of discharge headways. These values must be observed from the field. Figure 4.11 shows the sensitivity of the delay values to the coefficient of variation. Notice that the expected delay shoots up high at a high coefficient of variance. In the field, efforts to minimize the headway variances must be employed.

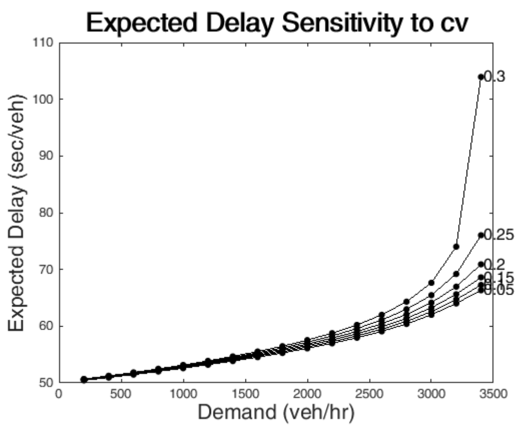


Figure 4.11. Sensitivity to coefficient of variation in discharge headways.

40% Right turn, 60% Major demand, ν=0.01%.

An example of the increase in delay due to the green time reduction is shown in Figure 4.12 as the line marked by asterisks. The domains of application with respect to the right-turn rate and major demands are then given in Figure 4.13. Compared to Figures 4.9and 4.10, the applicability range of TSS was reduced.

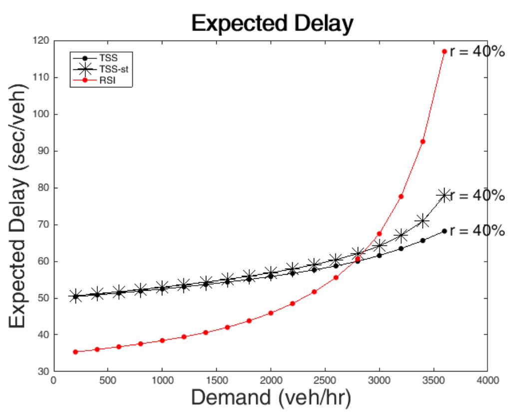


Figure 4.12. Delay increase after green time adjustment, 60% Major demand.

60% Major demand, ν=0.01%, γ′=0.2.

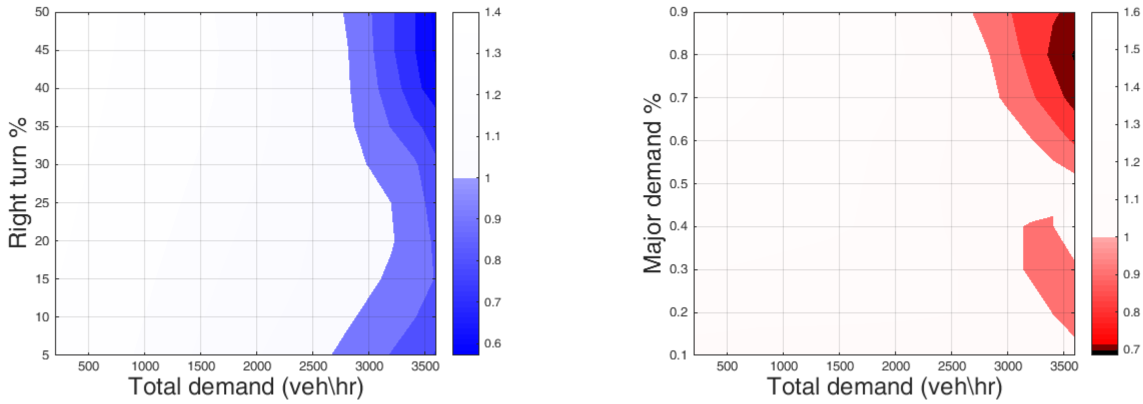


Figure 4.13. TSS:RSI delay under stochastic headways.

γ'=0.15, ν=0.01%.

4.5.3 Effect of Storage Length

Adequate storage length (or length of storage area) must be provided in order to attain the maximum possible capacity of TSS. Xuan gave recommendations on the length of the storage area in (Xuan et al., 2011) as a function of the average queue length. It is impractical to design a storage length that can accommodate all combinations of demands as well as putting the pre-signal too far from the main signal. Also, the presence of large vehicles may also pose limitations to the storage capacity. Therefore, inadequacies in the available storage area must be dealt with.

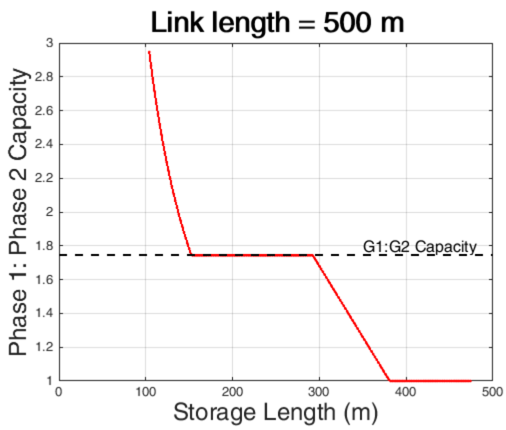


Figure 4.14. Capacity imbalance caused by inadequate storage length.

3600 veh/hr, 60% Major demand, 10 % right turn.

Queue spillovers at the storage area have two main consequences: first, it wastes pre-signal green time and leads to a capacity reduction proportional to ρ and second, it creates an imbalance of flows between the first and second phases. The leading phase is always sure to benefit from TSS but not the lagging phase. Figure 4.14 shows the capacity ratio between the first and second phases. The ideal ratio between the two phases is given by the black dashed line. When the storage length is too short, Phase 2 gets less capacity. At storage lengths between 154-292 meters, the storage length is adequate. Beyond these lengths, the upstream link becomes inadequate, thus affecting the allowable duration of the pre-signal green times.

To give proportional flows between the phases in case of inadequate storage, an additional constraint must be added to the lower level optimization problem in the form:

$$\frac{g_1 S^p}{g_2 S^p} = \frac{G_1 S_1^m}{G_2 S_2^m} \tag{4.63}$$

Another question worth asking is whether TSS can be applied to short links given that it requires ample storage lengths both upstream and downstream of the pre-signal. The lower level optimization problem can give the capacity of the targeted link length for different values of the storage area. An example is shown in Figure 4.15 for a given demand scenario and compared to its equivalent RSI capacity (shown as black dashed line). The blue lines are marked separately for the different link lengths considered. For completeness sake, a wide range of storage lengths are considered. Link lengths 200-300m fail to attain the maximum possible capacity but are still shown to have a larger capacity compared to RSI for a range of storage lengths.

The red dashed line in the figure shows the lower bound of the capacities when queues are not allowed to extend beyond the storage length as assumed in (Ma et al., 2013) and (Yan et al., 2014). If the queues are restricted, the 200 m link lengths will have a worse capacity than TSS and the

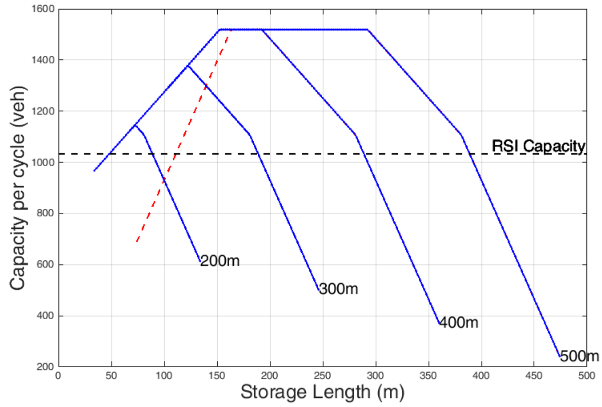


Figure 4.15. Capacities for typical link lengths.

3600 veh/hr, 60% Major demand, 10% right turn.

C_{RSI}=178 sec, C_{TSS}=110 sec.

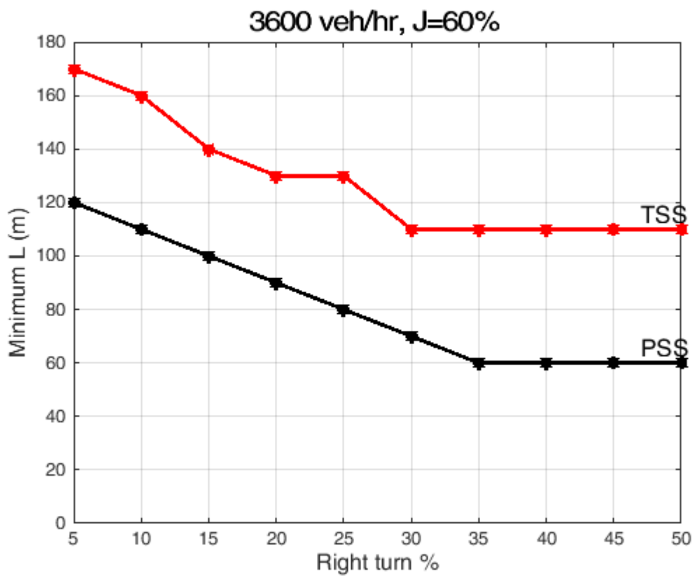


Figure 4.16. Storage Lengths Required to attain maximum capacity.

An alternative approach to increase TSS capacity under inadequate storage lengths is the phase swap strategy. In figure 4.16, the minimum storage lengths required to attain the maximum discharge capacity (i.e., remove the effect of storage length inadequacy) are shown. PSS clearly requires shorter storage lengths. It seems here that PSS is more attractive than TSS. However, one drawback to PSS is that the available time for vehicles in one pre-signal phase to enter the main signal depends on the available red times (i.e., the green durations of the minor phases). If the PSS phases are phase 1 and 3, respectively, the queuing time for Phase 1 depends on the duration of Phase 4 (or R_1 in Figure 4.4) and that of Phase 2 depends on Phase 3. This can limit the PSS capacity. Therefore, TSS is still the preferable option when sufficient length is available.

Chapter 5

Median U-Turn: Performance and Limitations

The Median U-Turn (MUT) is a very common alternative intersection type in both urban and suburban areas. By prohibiting vehicles from making right-turns at the main intersection and re-routing them to a U-turn crossover downstream of the main signal, the delays and number of stops in the through direction be decreased (El Esawey and Sayed, 2013). This intersection type also has fewer conflict points compared to RSIs. Thus, they are relatively safer.

Studies have mentioned that MUT's are applicable to areas with heavy through traffic coupled with low to moderate left-turn traffic. However, not much has been said about the performance of MUT's in case of high right-turners (or U-turners, assuming that drivers stay on the left side of the road). U-turn efficiency can be surmised to decrease but the actual impacts should be quantified because the designers cannot easily control the growth of demand and its composition after the MUT has been constructed. In the case of Epifanio delos Santos Avenue in Manila, the proliferation of U-turn slots initially provided improvements in speed and capacity but after a few years, the U-turn slots had to be closed because they became sources of traffic jams.

In summary, the traffic demand assumed during the design phase may no longer be applicable after the intersection has been built especially if the land use has been modified such that traffic attraction has increased. Thus, it would be reasonable to also analyze the performance of the MUT when the traffic conditions deviate from its intended design and get a closer look on how the system fails in order to apply suitable improvement strategies.

5.1 Safety and Efficiency Issues in Conventional MUTs

5.1.1 Reduction of conflict points in MUTs

The safety benefits of MUT's stem from the reduction of conflict points at the intersection. A typical four-legged conventional intersection has 32 conflict points (El Esawey and Sayed, 2013) whereas a four-legged MUT only has 16 (Figure 5.1). The reduction is due to

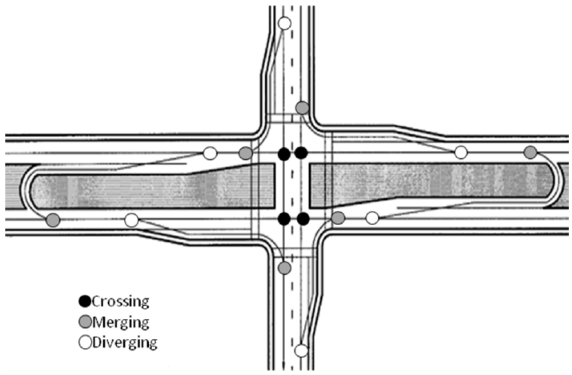


Figure 5.1. Conflict points in an MUT (Total: 16).

Source: (Hughes et al., 2010).

the removal of left-turn vehicles from the main signal. A diagram of the different conflict points in a four-legged MUT intersection is shown in Figure 5.1. Other safety issues related to MUTs have been discussed in Section 2.4.

5.1.2 Conventional Two-Phase Plan: Efficiency Issues

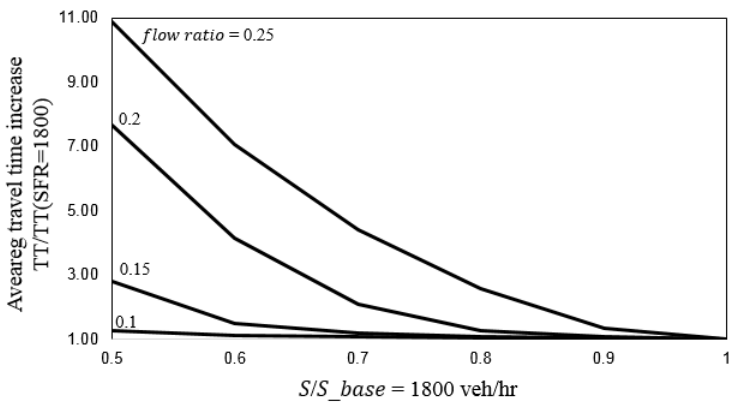


Figure 5.2. Effect of Saturation Flow Rate on Travel Times.

The MUT two-phase plan reported in Chapter 3 is suitable only for simple cases, i.e., when there is enough storage capacity, when the demands are not large enough, and when there are not so many pedestrians conflicting with turning traffic. However, when this is not the case, the number of pedestrians could impede the efficiency of vehicle flows and possibly cause accidents.

Previous studies such as (Milazzo et al., 1998) have quantified the effect of pedestrians on turning vehicle flows in case of a permissive phase. Essentially, an increase in the number of pedestrians translates to a decrease in the turning movement's saturation flow

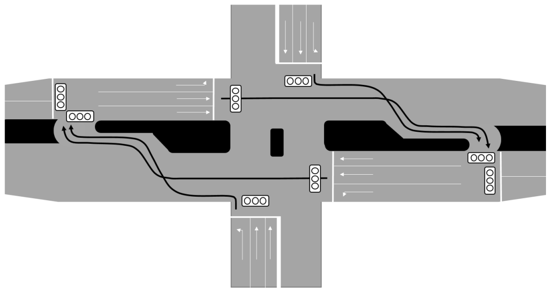


Figure 5.3. MUT Diagram.

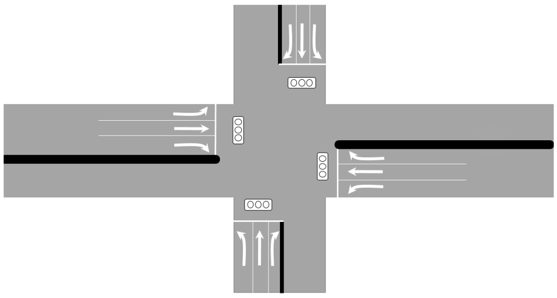
rate. On the pedestrian side, the number of turning vehicles has also been shown to affect the probability that a pedestrian is delayed or changes its path and speed (Hubbard et al., 2009).

Figure 5.2 shows the results of a simulation experiment in TRANSYT that illustrates the effect of pedestrians on turning vehicle travel time. The base saturation flow rate S_{base} was set to 1800 veh/hr and gradually decreased upto half of the base value. The figure shows that when the flow ratio is rather high, average travel times quickly increase (meaning efficiency decreases) even with just a small change in the saturation flow rate. More importantly, this figure shows that it does not take much to significantly increase the travel time of turning vehicles.

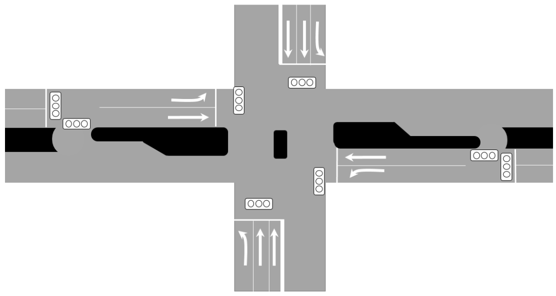
5.2 Simulation-Based Evaluation

To determine the failure mechanism of a U-turn, high vehicle and pedestrian volumes are assumed. Thus, an MUT with signalized crossovers is considered (Figure 5.3). The signal control parameters at the crossover, particularly the offsets, must be carefully set in order to ensure that the MUT does not breakdown due to faulty coordination alone.

Since the U-turning vehicles need to pass the main intersection twice, the main and crossover signal parameters from all sides cannot be treated independently of each other especially when turning flows are high. The signals in the MUT must then be treated in an integrated manner. For this reason, MUT signal setting is not as straightforward as that in a regular signalized intersection. At the U-turn crossover, the arrival pattern of vehicles do not follow a regular pattern. The problem thus becomes complex and multi-faceted. To ensure consistency in the results, only one model should be used in the assessments. Thus, a simulation tool is used. The TRANSYT-7F macroscopic simulator is used in this study. This simulator was primarily selected for its signal optimization tool. This is based on the Cell-Transmission model (Daganzo, 1994).



Regular signalized intersection.



Median U-Turn.

Figure 5.4. Intersection layouts used in the comparison.

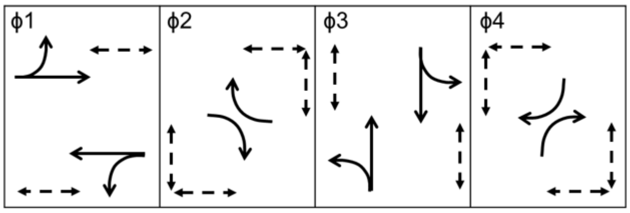


Figure 5.5. RSI phasing plan.

The U-turn performance is compared to that of a Regular Signalized Intersection. Unlike most studies that make comparisons between intersection types, the same total lane width is assumed for both intersection types. This is because the problem is seen from the context of bringing an improvement to an existing RSI. Therefore, if the RSI were to be converted to an MUT, the same area restrictions must be applied. The lane configurations considered are shown in Figure 5.4.

The phasing plan used for the RSI is given in Figure 5.5. The phasing plan used for MUT will be discussed in the succeeding sections.

5.2.1 Demand Scenarios

The simulation is run for a duration of one hour under the demand settings in Table 5.1. For simplicity, flows along the East-West direction and vice versa are assumed to be equal. The right-turn rates simulated were between 5-30% and the Through:Left ratio was kept at 80:20.

5.2.2 Phasing Plan

To avoid the effects of pedestrians on the efficiency of turning flows, a zero-conflict phasing plan is adapted where the pedestrian phases are protected instead of permissive phases.

Table 5.1: Simulation Scenarios

Scenario	Total Demand (veh/hr)	Flow Ratio Range			
		Major Direction Demand Proportion			
		50%	60%	70%	80%
A	3000	0.40-0.50	0.35-0.50	0.35-0.45	0.35-0.40
B	4000	0.50-0.70	0.50-0.60	0.45-0.60	0.45-0.55
C	5000	0.65-0.85	0.60-0.80	0.60-0.75	0.55-0.70
D	6000	0.80-1.0	0.75-0.95	0.70-0.85	0.65-0.80

	MUT-A	MUT-B	MUT-C	RSI
Minimum green time (seconds):				
Phase 1	15	18	40	15
Phase 2	15	18	30	15
Phase 3	40	18	15	15
Maximum Cycle Length	180			
Total lost time (seconds)	12	14	14	24
Saturation flow rate (veh/hr):				
Through	2000			
Left, Right-turn	1800			
U-turn	1440			

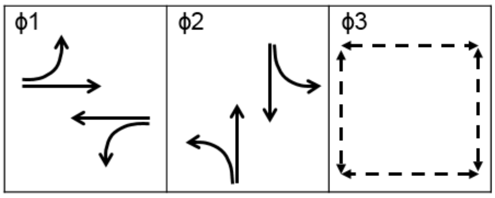
Table 5.2: Parameters used in the calculation of cycle lengths

This increases the minimum number of phases to three instead of two, similar to the analysis done in (Bared and Kaisar, 2002). Figures 5.6,5.7 and 5.8 (left figure) show variants of a three-phase, no-conflict phasing plan for MUT.

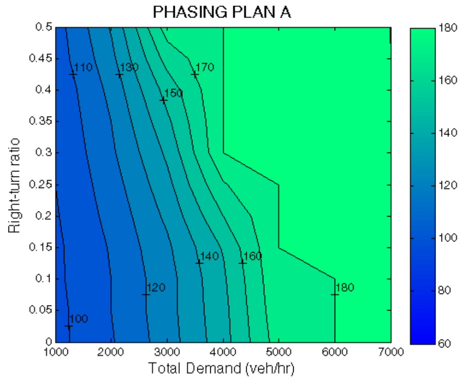
The choice of phasing plan significantly affects the intersection performance because it influences the minimum required green times and cycle length. Also, some phasing plans require physical re-alignments in the intersection or additional pedestrian refuge islands (in the case of two-stage pedestrian crossings).

To determine the most suitable phasing plan in the absence of an optimization program, a sensitivity analysis was conducted for three possible combinations of pedestrian and vehicle movements. These combinations are reasonable since the demands between opposing directions (i.e. North-South, South-North bound) are assumed to be balanced. Following the geometric guidelines for MUT's in (Hughes et al., 2010), the minimum green times are calculated by setting a minimum walking time of 7 seconds plus the time it takes to cross the width of the pedestrian lane at a walking speed of 1 meter/second. In the said guideline, the saturation flow rate at the MUT is said to be 80% of the turning saturation flow rate. The three phasing plans considered and their attributes are shown in Table 5.2.

The sensitivity analysis aims to check which phasing plan will result to the lowest cycle length and thus, the lowest delay. The cycle lengths are approximated using Webster's

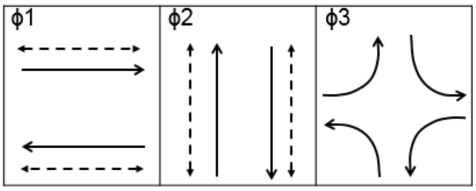


Phasing Plan.

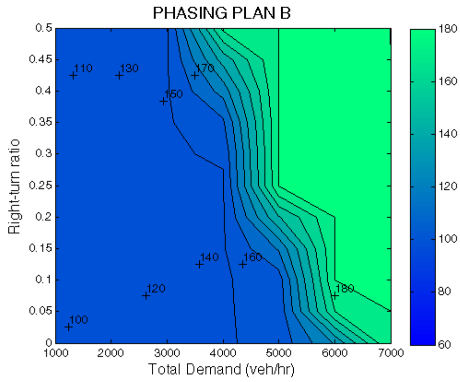


Cycle lengths (sec).

Figure 5.6. Plan A.



Phasing Plan.

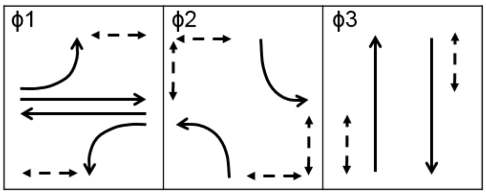


Cycle lengths (sec).

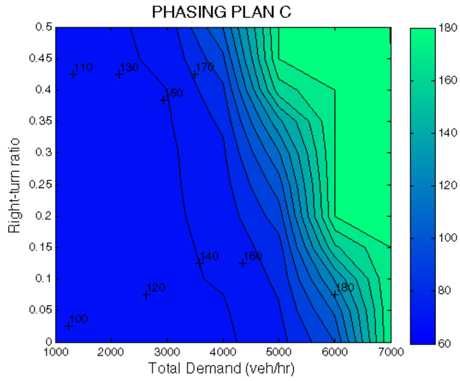
Figure 5.7. Plan B.

method using the demands specified in section 3.2.1. Although the cycle lengths in MUT's are higher than this value, the relative difference between the three phases should be similar.

Phasing plan A is a variation of the two-phase plan but with a separate pedestrian phase (Figure 5.6). This was shown to have reached the maximum cycle length at even relatively low demands (Figure 5.6). Phasing Plan B has pedestrian phases parallel to the through movements (Figures 5.7 and 5.6). Phase Plan C resulted to the lowest cycle lengths at much lower demands because of the two-stage crossings. It was also found to reach maximum cycle time later (Figures 5.8 and 5.8). Thus, it was selected for the simulation.



Phasing Plan.



Cycle lengths (sec).

Figure 5.8. Plan C.

5.2.3 Signal Settings

To ensure fair comparison between MUT and RSI, the signal control policy used in assigning green times should be the same for both intersection types. For the simulation experiments in this work, the equisaturation policy is used. Under this policy, the green times are set so that the degree of saturation for all phases are equal. This policy has been shown in (Webster, 1958) to reduce average delays for undersaturated flows.

For RSI, the cycle length was calculated using the following equation:

$$C = \frac{1.5\tau + 5}{1 - \sum\left(\frac{q_i}{s_i}\right)} \tag{5.1}$$

where:
C: cycle length (sec)
τ: total lost time (sec)
q_i: demand from critical movement in phase i (veh/hr)
s_i: saturation flow rate of critical movement in phase i (veh/hr)

For MUT, the initial value of the cycle length was also computed using equation 5.1. The TRANSYT cycle length optimizer tool was used to search for the cycle length that resulted to the minimum delay. Initial attempts to simultaneously optimize the offsets and green splits in TRANSYT resulted to solutions that were not globally optimal. A similar study (Maher, 2011) using TRANSYT also reported similar problems with the optimal offset results. This problem is avoided by setting constant offsets.

The offset setting used for MUT is patterned after the MUT two-phase plan where the green time for through traffic at the crossover is coordinated with the main signal through

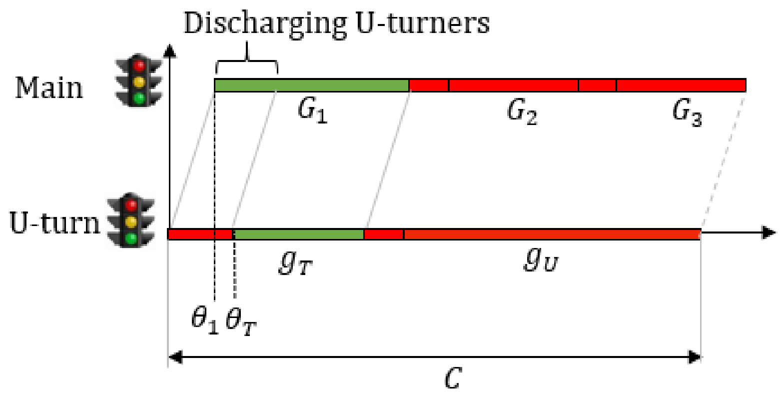


Figure 5.9. Through-priority offset setting.

direction (Figure 5.9). Two offset settings were considered. In both offset settings, the the storage area between the main signal and the crossover must be cleared of vehicles to avoid queue buildup.

Offset Setting 1: Through priority

The offset is set such that the end of the green time of the upstream through direction coincides with the end of the main signal green time for the through phase. This is illustrated in Figure 5.9. In the figure, the main signal phases are number from 1 to 3 corresponding to the phases in Figure 5.8 and the crossover phases are indexed as phase T for the main link through phase and U for the U-turn phase. The offset setting satisfies the following equation:

$$\theta_T + g_T + t = \theta_1 + G_1 \tag{5.2}$$

where:
θ_i: start of green time of phases i
G_1: duration of phase 1
g_T: duration of phase T
t: free flow travel time from the crossover to the main signal

The green duration allotted to U-turners is estimated from the known U-turn demand multiplied by the saturation headway.

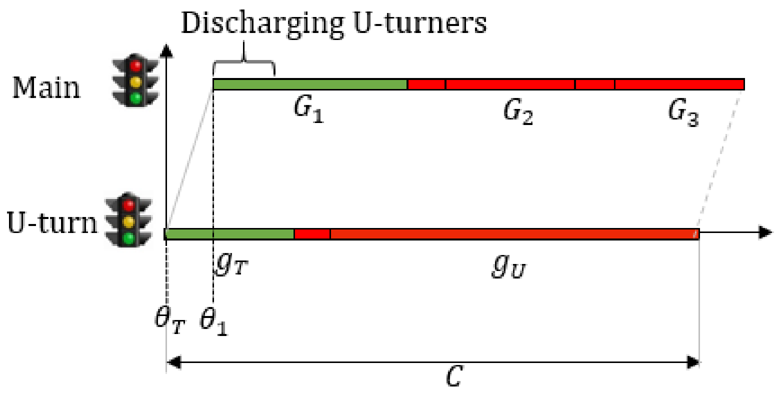


Figure 5.10. MUT-priority offset setting.

Offset Setting 2: U-turn priority

In this offset setting, priority is given to U-turn traffic by ending g_T earlier. This is similar to the two-phase offset setting in (Hughes et al., 2010). Assuming that the storage area is long enough to accommodate all queued vehicles, the vehicles released at the crossover will simply join the U-turn queue at the main signal stop line. This setting satisfies the following equation:

$$\theta_1 = \theta_T + t \tag{5.3}$$

5.2.4 Offset setting comparison

Between the two offset settings considered, setting 1 appears to give lower delay to through vehicles coming from crossover signal. However, the existing U-turn queue at the main signal is only estimated and cannot be known exactly. If the existing U-turn queue length is overestimated, some parts of the duration G_1 is not fully utilized. If the queue is underestimated, the vehicles discharged from the upstream signal will still meet with the U-turn queue.

Another consequence of setting 1 is that it allows U-turn queues to build up at the crossover signal. At very high U-turn demands, this leads to a spillover of queues in the U-turn lanes and leads to the breakdown of the simulator. The vehicle interactions were observed using the microsimulation tool VISSIM. It was observed that at high U-turn demands, the accumulated vehicles at the U-turn crossover spilled over to the upstream adjacent lane (marked in red in Figure 5.11). This in turn lead to U-turning vehicles from phase 2 (marked in yellow) to utilize the adjacent lane. While these queues are waiting to be cleared, they block the passage of the upstream through vehicles (marked in orange),

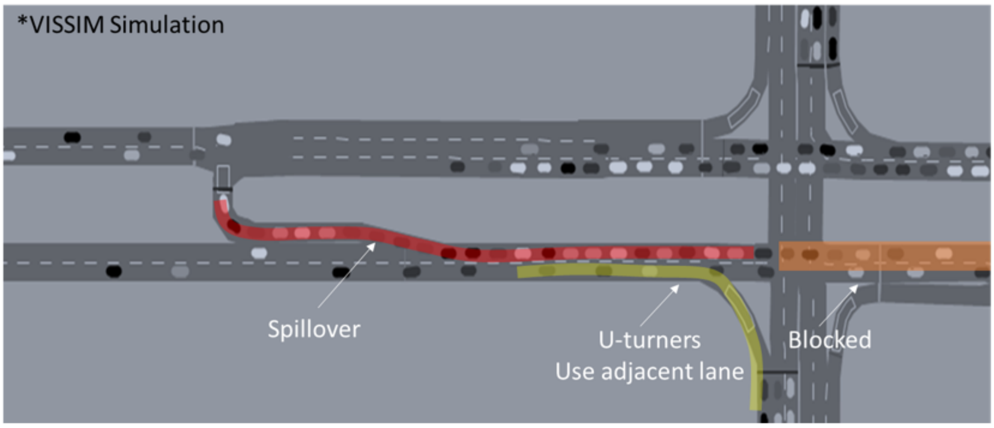


Figure 5.11. Intersection break down due to overflows from the U-turn lane.

eventually leading to a gridlocked state.This situation was also reported in (Autey et al., 2010) which expressed the difficulty of using MUT in high volume situations. In that study, the authors stopped the simulation upto the demand level which did not result to the gridlock.

Under offset setting 2, the build up of queues was reduced and the gridlock state was avoided. Another advantage of this offset setting is that the main signal approach is saturated thereby maximizing G_1. The drawback is that priority to through vehicles is not guaranteed but compared to the possibility of intersection breakdown, the increase in delay is acceptable. From here on, the results presented are those where offset 2 is used.

5.3 Simulation Results

The results after running 96 demand scenarios are presented in this section. Each scenario was run for a one hour time period. The arrival patterns in TRANSYT followed a uniform distribution and adjustments to reflect random and oversaturation effects were added. Therefore, 1 hour simulation is adequate for undersaturated conditions.

5.3.1 Average travel time trends

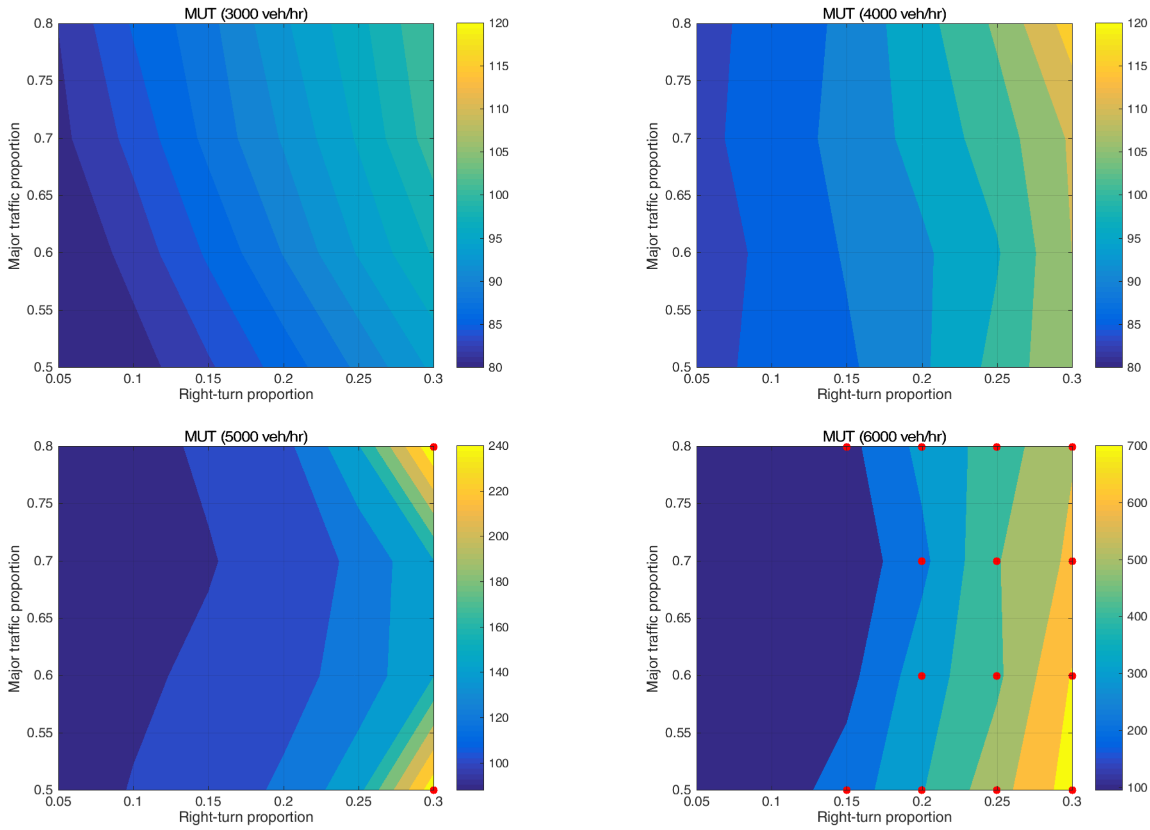


Figure 5.12. Average Travel time trends.

The travel times under saturated demands are also presented here for comparison purposes but the total demand served is observed as it is an indication of the capacity of the intersection.

Contour plots of the average travel times are presented in Figure 5.12. Although the simulation only yielded discrete points (24 points per figure), plotting a contour map shows the general trends in travel times. In general, the travel times increase towards the direction of increasing right-turn proportion. In terms of major demand proportion, the pattern is not so clear as the contour lines are interpolated values.

The figures with the red-colored points indicate scenarios where the demand is higher than the capacity. At the high demand case, the U-turn is seen to perform well at low right-turn proportions. So, how does the increase in right-turn rate affect the overall MUT performance?

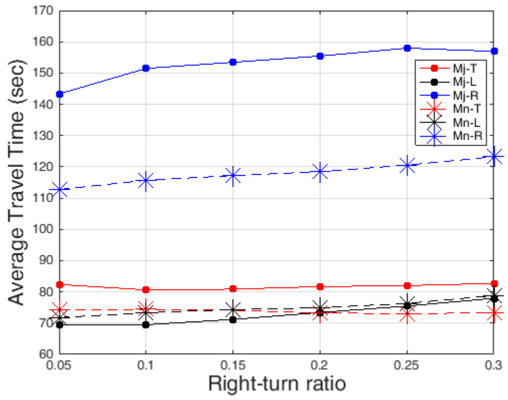


Figure 5.13. Low Demand: 3000 veh/hr.

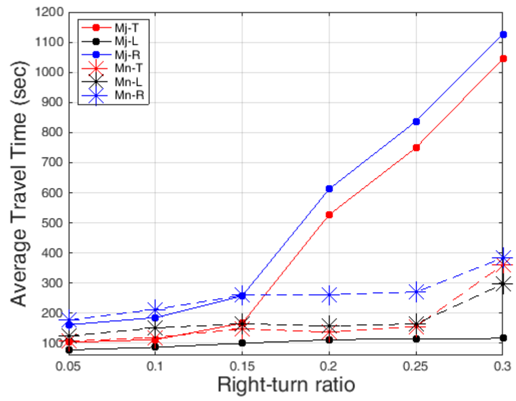


Figure 5.14. Near-Saturation to Oversaturated Demand: 6000 veh/hr.

5.3.2 Composition of Travel Times

Next, the average travel times of each movement are investigated to see which movements experience gains in travel times when the demand and right-turn rates increase. The plot for 60 % Major traffic demand is shown in Figure 5.13. At low demand, the highest average travel times are experienced by the Right-turners in both the Major and Minor directions (Note:"Mj-T" in the legend refers to Major direction, Through traffic). This high travel time is attributed to the effect of the U-turners having to make a "detour".

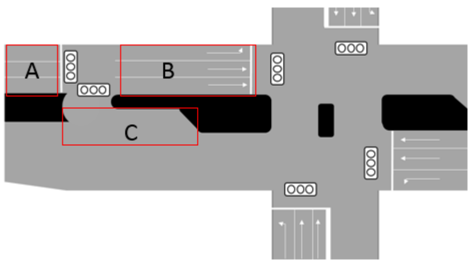


Figure 5.15. Analysis areas.

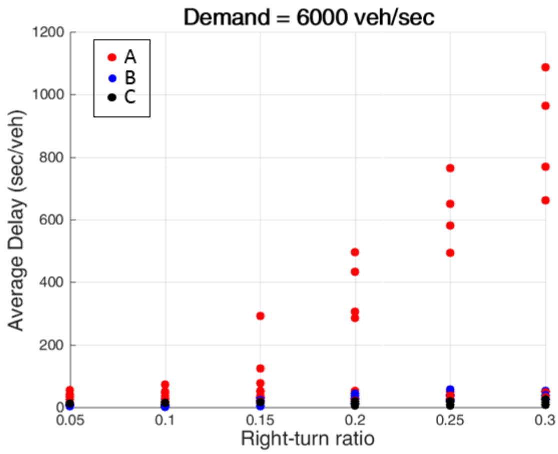


Figure 5.16. Average delays plotted per area.

For the high demand case, exceedingly high travel times can be observed but the highest travel times belonges to the Major-Through and Major-Right traffic. Looking at delays incurred in three areas (Figure 5.15): A: upstream of the crossover (through-vehicle side), B:downstream of the crossover, and C:upstream of the crossover (U-turn side) indicates that the highest delays are experienced upstream of the crossover (Area

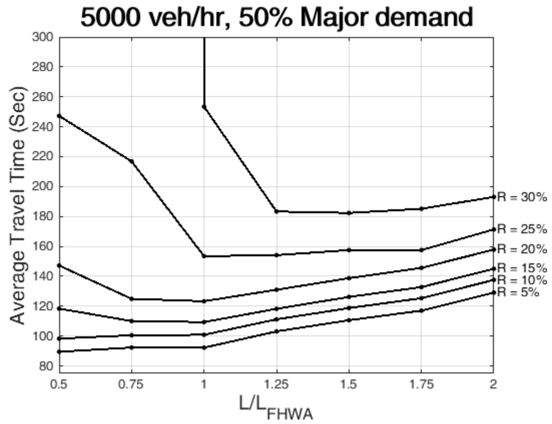
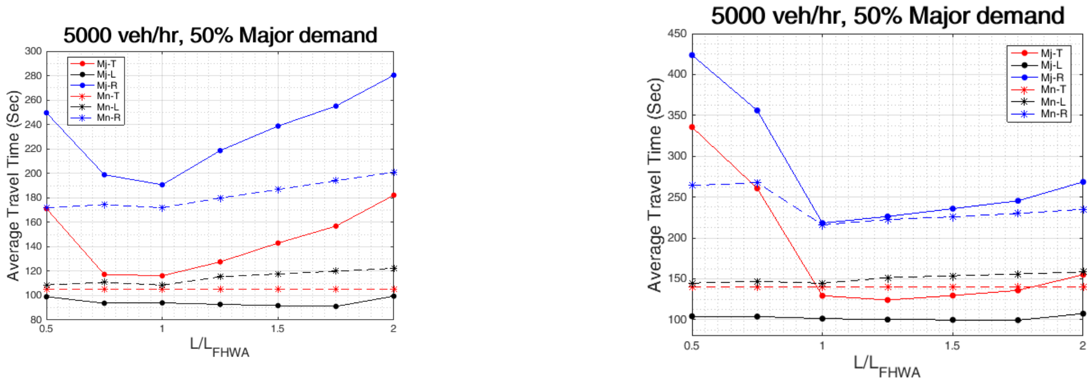


Figure 5.17. Effect of crossover distance on travel times.



R=20%.

R=25%.

Figure 5.18. Effect of crossover distance on each movement's travel time.

A, Figure 5.16) . The average delay values greater than half of the red time (indicating oversaturated delays) are only present in Area A. Therefore, the offset setting reducing queue buildup in the U-turn approach was effective.

5.3.3 Effect of crossover distance

With regards to the impact of crossover distance, a "short" distance leads to inadequate storage space especially for U-turning vehicles. On the other hand, a "long" crossover distance increases the U-turner travel times. In the simulations conducted in this chapter, the FHWA standard length of 400 feet (122 meters) (Hughes et al., 2010) was used. To determine the impact of crossover distance, a demand condition that is high enough for the queue lengths to be significant must be chosen. For this reason, a total demand of 5000 veh/hr, 50 % major demand was considered. Seven crossover length values ranging from 0.5L-2L were tested, where L is the crossover length recommended by FHWA. The results are summarized in Figure 5.17. As expected, increasing average travel times can be observed for shorter L's. These even become significantly high (indicating oversaturated levels are reached) for r=20% to r=30%.

Figure 5.18 shows the effect of crossover distance on each movement's average travel times. In most cases, the standard FHWA crossover distance was adequate. The through and right-turn vehicles from the major direction as well as the right-turn vehicles from the minor direction are significantly affected by the crossover distance. When the right-turn rates are high enough, many U-turning vehicles occupy the downstream area of the crossover signal. In this case, the vehicles upstream of the crossover can only advance once the existing queues have dissipated. Under high right-turn volumes, the vehicles released upstream of the crossover may not pass the main intersection in the same cycle. This is why major direction-through vehicles experience oversaturated average travel times at low L values. The same can be said for major direction-right turn vehicles but these have much higher travel times because they also have to stop at the U-turn signal. Finally, the short crossover distance reduces the available storage space for minor direction-right turn vehicles which is why they also experience high average travel times.

5.3.4 Comparisons with Regular Signalized Intersection

Figure 5.19 shows the cycle lengths used in the simulations. For all demands, the MUT has a lower cycle length due to the reduced number of phases. Therefore, RSI reaches oversaturation at lower flow ratios compared to MUT.In RSI, an increasing linear trend can be observed between flow ratio and cycle length but in MUT, a clear trend between cycle length and flow ratio cannot be observed. This is because the cycle length is significantly affected by the right-turn rates.

The average travel time ratios between MUT and RSI for all scenarios are summarized in Figure 5.20. Note that at low demands (i.e. 3000 veh/hr), the RSI has a better performance due to the effects of detouring. As expected, MUT becomes better than RSI at higher demands (4000-5000 veh/hr). At demand scenarios close to saturation (i.e. flow ratio =1), MUT performance declines. The flow ratios in the graph are estimated at the main signal so although the flow ratios are less than 1, in some cases the condition at the crossover signal is already oversaturated.

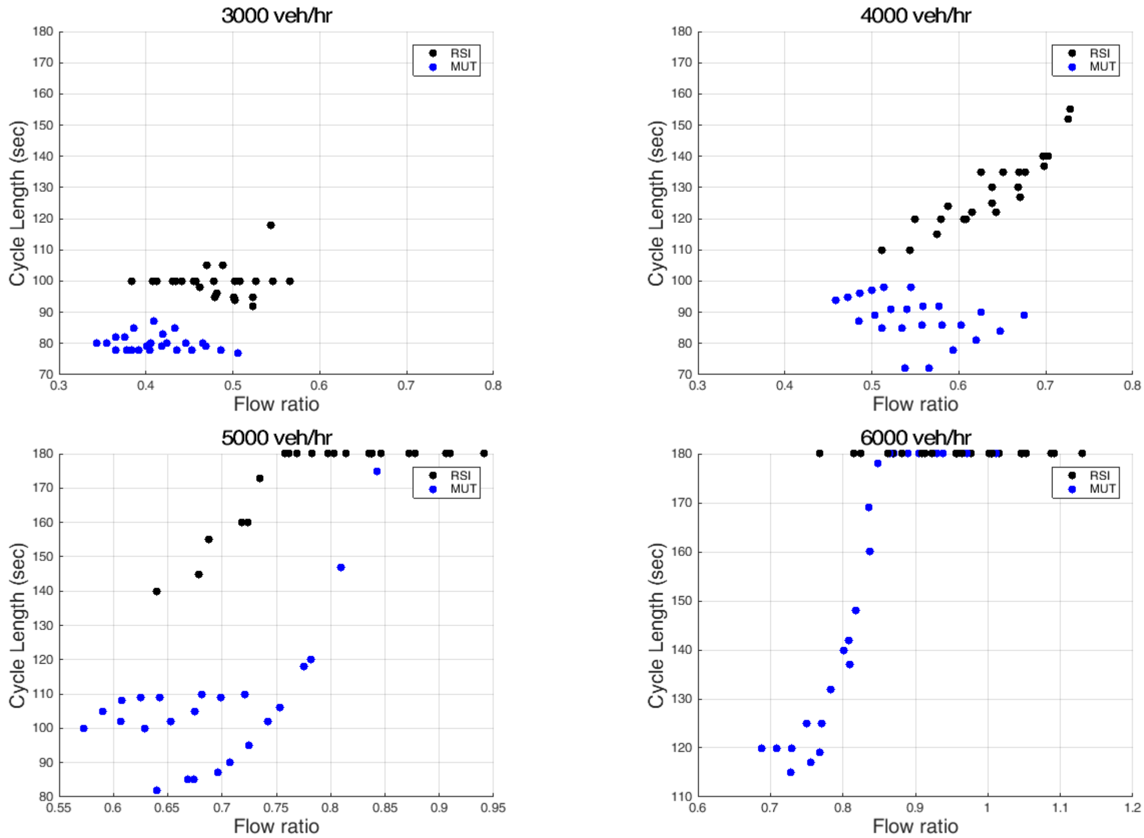


Figure 5.19. Cycle Lengths Used in the Simulations.

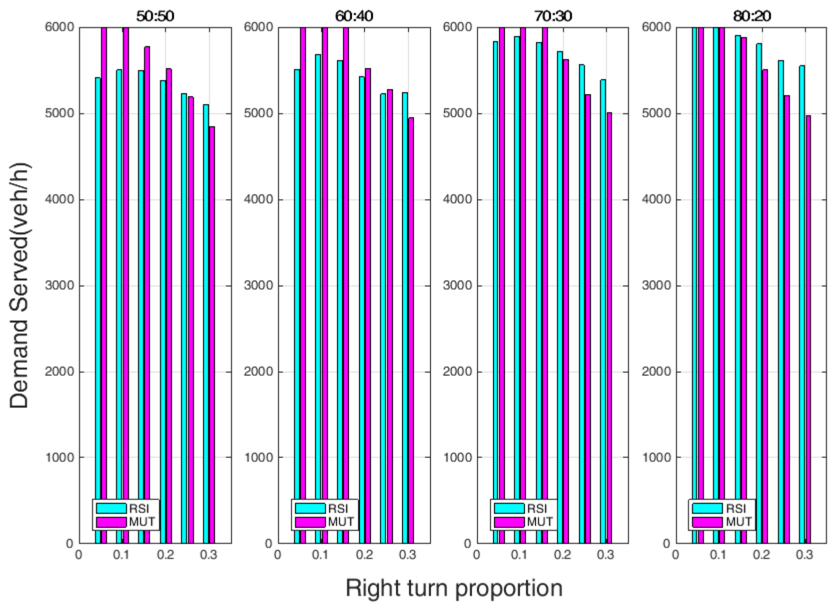


Figure 5.21. MUT and RSI capacities at Demand = 6000 veh/hr.

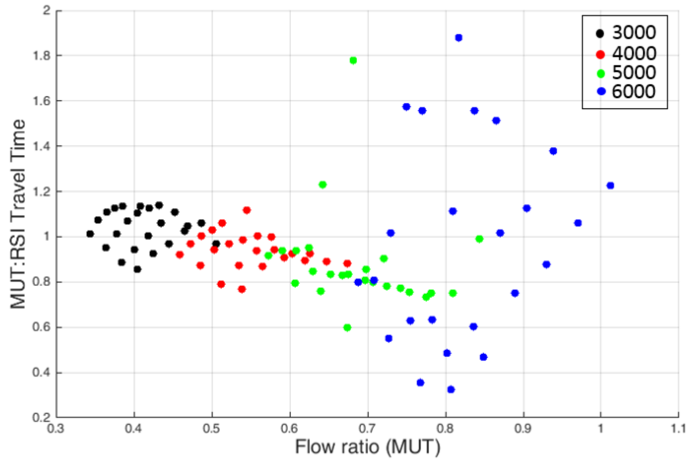


Figure 5.20. MUT:RSI travel time ratios.

A look at the capacity of the intersection at the 6000 veh/hr demand scenarios (Figure 5.21) shows that the MUT capacity decreases to levels that are even worse than the RSI when right-turns are high.

5.4 MUT optimized signal settings

An optimization problem is formulated for signal setting of an isolated MUT. Similar to the upper-level optimization problem in TSS, the objective function is to minimize the total delay, D.

Objective function:

$$\min D = \sum_{k=1}^{K} \sum_{x=1}^{X} d_{xk} q_{xk} \tag{5.4}$$

where:
k, K: link number and total number of links, respectively
$x = 1, 2, 3$: for Left, Through, Right movements, respectively
X: total number of movements
d_{xk}: expected delay of movement x,link k (in sec)
q_{xk}: demand of movement x, link k (in veh/hr)

In links without pre-signals, the delay equation is given by Webster's two-term equation. Following the link notations in section 3.2.1, the delay in links 2 and 4 is given by:

$$d_{xk} = 0.9\left(\frac{C(1-(G_i/C))^2}{2(1-(q_x/s))} + \frac{(q_x C/G_i s_x)^2}{2q_x(1-(q_x C/G_i s_x))}\right) \forall x = 1,2; k = 2,4 \tag{5.5}$$

In links with the crossover signal, the delay is divided into upstream d^u_{xk} and downstream d^d_{xk} terms:

$$d_{xk} = d^u_{xk} + d^d_{xk} \tag{5.6}$$

Upstream delay terms:
The delay term in the upstream through movement is the same as equation 5.5. Since priority is given to U-turn vehicles at the crossover, the delay experienced by U-turners at the crossover is neglected.

Downstream delay terms:
The delays downstream of the crossover are formulated separately for the through and U-turning vehicles coming from the upstream signal. Since U-turning vehicles emanate from shared lanes, their arrival at the U-turn crossover may not be uniform. The arrivals are assumed to follow a Poisson process and thus their delay is given as:

$$d^{du}_{xk} = 0.9\left(\frac{C(1-(G_i/C))^2}{2(1-(q_x/s))} + \frac{(q_x C/G_i s_x)^2}{2q_x(1-(q_x C/G_i s_x))}\right) \forall x = 1,2; k = 1,3 \tag{5.7}$$

where:

$$q_x = G_x S^m_x r_x \quad \forall x = 1,2$$

The green time for through traffic coming from then crossover starts T seconds earlier than the main signal. This is to allow as many vehicles as possible to pass through the main signal. The through traffic from the crossover will encounter a queue of U-turners at the main signal so the average delay that these vehicles experience in the area between the crossover and main signal is equal to the discharge time of the U-turn queue (which is estimated to be a fraction of the discharge flow from the upstream signals). This is given by:

$$d^{dt}_{xk} = \frac{G_x S^m_x r_x}{w_x J} \quad \forall x = 1,2 \tag{5.8}$$

The proportion of U-turners are calculated as follows:

$$r_1 = \frac{r}{t + r + r'}$$

$$r_2 = \frac{r'}{r' + l'}$$

where:
t, l, r: demand for movements (l,r,t) (major link)
t', l', r': demand for movements (l,r,t) (minor link)

The final delay term to be added is the additional travel time incurred by U-turners due to the detouring. This is given by:

$$d_3 = \frac{2L + W}{v_f} \tag{5.9}$$

where:
L: the distance between the crossover and the main signal W: the turning distance at the crossover v_f: the free flow travel speed

Constraints:
In this optimization problem, it is assumed that the OD demands are known and that the distance between the main signal and crossover is sufficiently long for there not to be storage problems. The constraints are discussed here:

1. The first two constraints ensure that the sum of the green times plus the total lost times at the main signal (τ^m) and at the crossover signal (τ^c) are equal to the cycle length, C.

$$C = G_1 + G_2 + G_3 + 3\tau^m \tag{5.10}$$

$$C = g_1 + g_2 + 2\tau^c \tag{5.11}$$

2. The main and crossover signal green durations must not be shorter than the minimum green time g_{min}.

$$g_{min} \leq G_i, \quad \forall i \tag{5.12}$$

$$g_{min} \leq g_i, \quad \forall i \tag{5.13}$$

3. The degree of saturation must not exceed an allowable value, x_{all}.

$$\frac{q_i C}{G_i S_i^m} \leq x_{all} \tag{5.14}$$

4. To avoid spillover, the capacity at the crossover should at least be larger than the estimated U-turn demands discharged from the main signal.

$$G_1 S_t^m r_1 + G_2 S_l^m r_2 \leq g_2 S_u^p \tag{5.15}$$

5. The main signal green duration for phase 1 (Through phase) must be greater than the green duration for through vehicles at the crossover plus the time it takes to discharge the initial U-turn queue at the main signal. This is estimated as the discharge time of the smaller queue (through lane queue).

$$g_1 + \frac{G_2 S_2^m r_2}{S_1^m} \leq G_1 \tag{5.16}$$

This optimization problem is a nonlinear programming problem with linear constraints. Since this problem is convex (Gallivan, 1982), an globally optimal solution can be obtained using standard optimization algorithms.

5.5 Verification of optimization results

To verify if the assumptions used in the optimization are valid, the results are compared to that of a simulation tool to see if the increase in delays resulting from increasing demands and right-turn rates can be accurately captured. At this point, no attempts are made to verify the accuracy of the estimated delays compared to real-world data.

The optimization results are compared to the output of TRANSYT simulation for verification. The signal settings were optimized for demands ranging from 3000 veh/hr to 6000 veh/hr and right-turn rates from 5-30%. Basically, these scenarios cover the spectrum of flow ratios from low saturation to near saturation. The signal parameters obtained from the optimization were inputted to TRANSYT. Finally, the average delays from TRANSYT and the optimization were compared.

The results are summarized in Figure 5.22. It indicates a good agreement between the expected delays calculated by the optimization and by TRANSYT. A systematic error was found to be caused by the difference in the random delay equation used by TRANSYT (Binning, 2013), shown in equation 5.17. Consistent with the observation made in (Wong

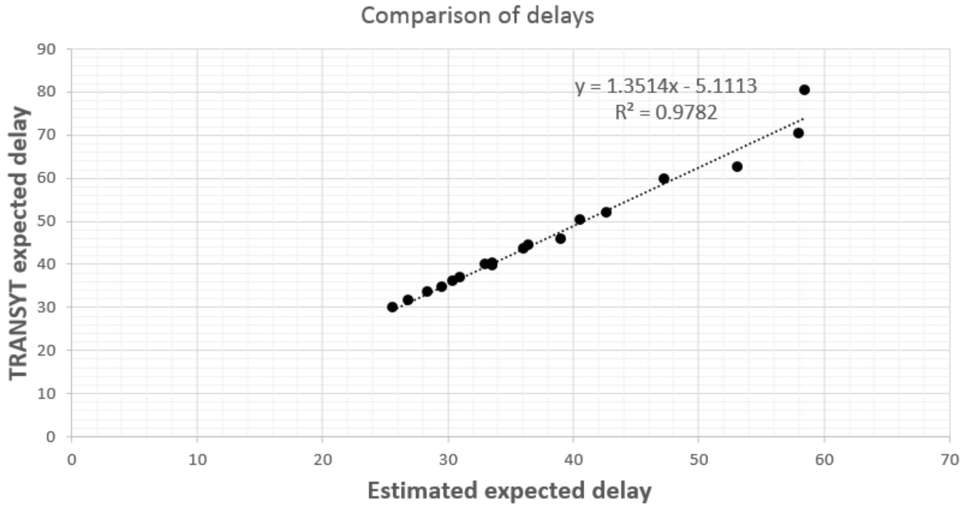


Figure 5.22. Verification of estimated delays.

et al., 2003), TRANSYT gave higher estimates compared to Webster's delay.

$$D = \frac{T}{4}\left\{\left[(f+F)^2 + \frac{4f}{T}\right]^{1/2} + (f+F)\right\} \tag{5.17}$$

where:
D= Random + oversaturation delay in pcu-hours/hour
f= average arrival rate on the link (PCU/hour)
F= maximum flow that can discharge from the link (PCU/hour)
T= the duration of the flow condition for which signal timings are being considered (hours)

A closer check of the delays incurred in each link showed little to no delays at the U-turn signal. However, this is only true for demands way below saturation. When the demands approach saturation, (in this case, flow ratios 0.75-0.8), residual queues begin to form at the U-turn and main signal. This is difficult to account for in the formulation with the steady state equations used. At this moment, this formulation is limited to demands that are not near saturation.

5.6 Results and Discussion

The optimization results now allow us to perform an evaluation of MUT without relying on the simulator. The delay minimization problem was solved using Sequential Quadratic

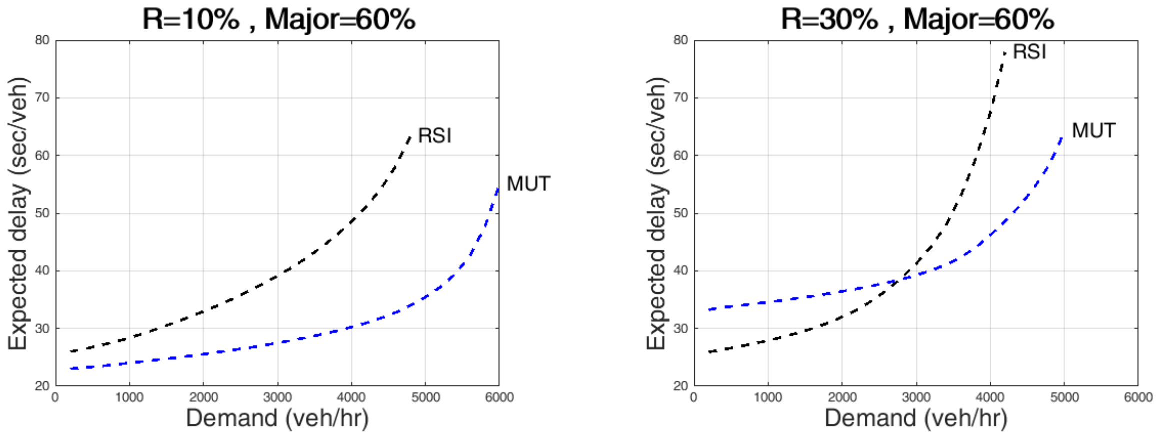


Figure 5.23. Comparison of delays.

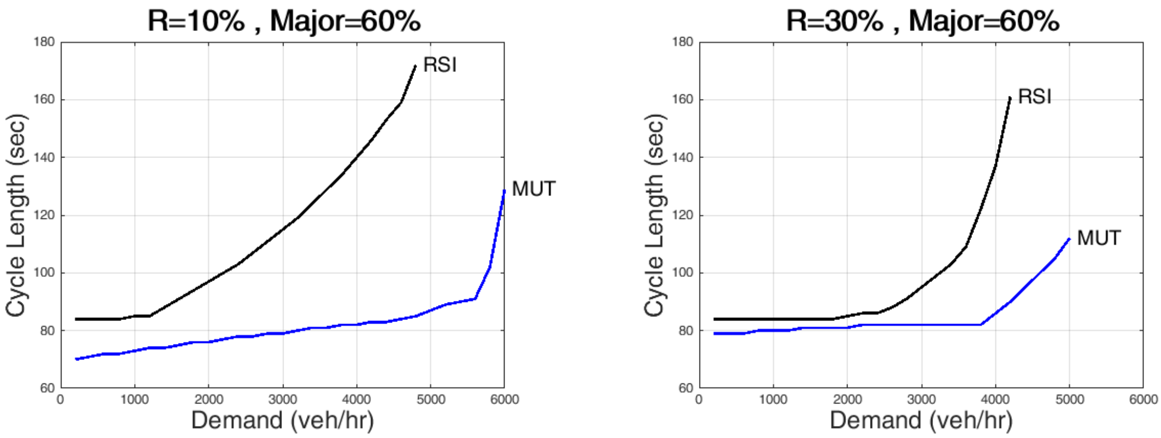


Figure 5.24. Comparison of Cycle Lengths.

Programming algorithm in MATLAB. A comparison between delays and cycle lengths is shown in Figure 5.23. The lane geometries used are the same as those in the previous sections.

Consistent with the simulation results, MUT has better performance at low right-turn rate (in this case, 10%) and its performance begins to decline at higher right turn rate even though it still has a lower cycle length in all cases (see Figure 5.24). Despite the decline, MUT can still hold much larger demands compared to RSI.

The application domain of MUT is summarized in Figure 5.25 where the ratio of expected delays between MUT and RSI are summarized. The blue shaded areas show the demands where MUT In the figure, the benefits of MUT can bring lower delays. The MUT's "sweet spot" is at low right-turn rates but since it reaches oversaturation slower than RSI, it has a higher performance at much higher demands.

The optimization program formulated can be used to calculate the demand thresholds for which installing an MUT becomes beneficial. The formulation can also be modified to

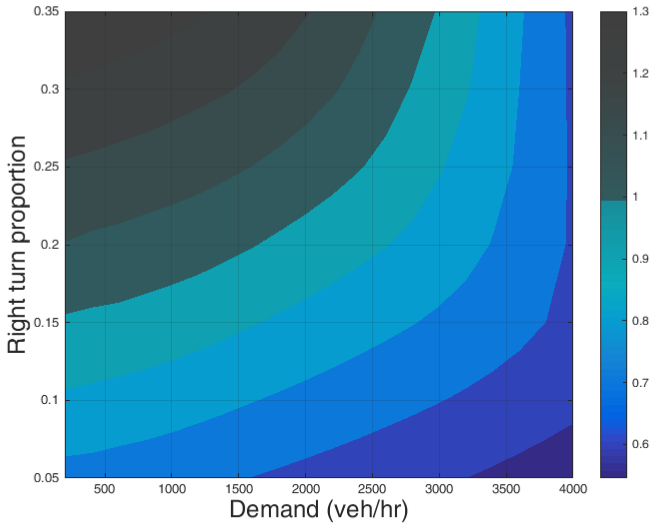


Figure 5.25. MUT:RSI expected delays (60% Major demand).

assess the viability of alternative MUT designs such as a half U-turn.

From the results presented, the following are apparent:

- Despite the reduced number of lanes and increased number of phases, the signalized MUT was found to improve RSI delays especially under low right-turn rates. This is consistent with findings of related researches. The analysis conducted here clearly showed the effect of varying demands on MUT performance and was able to determine the boundary between RSI and MUT.
- When MUT demands become too high to the point of oversaturation, the MUT performance declined to levels even lower than that of RSI. This oversaturated state could have critical consequences on the entire intersection if offsets are not properly set. In an oversaturated state, queue buildup on the U-turn approach that leads to spillovers onto the adjacent lane can cause the intersection to reach a locked state. In this case, it is better to adjust the offset in favor of U-turning vehicles.

Chapter 6

Comparison of Attributes

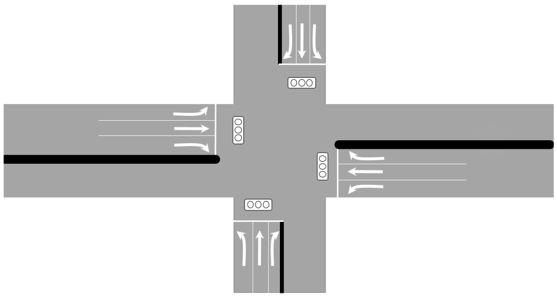
In this dissertation, an analysis of the performance of two alternative approaches that can improve the performance of a regular signalized intersection was provided. These alternative approaches improve intersection performance by allowing for the maximized utilization of discharge lanes. The benefits of each intersection type over the regular signalized intersection have been discussed in previous chapters. Here, a comparison is provided between the two.

6.1 Role of Auxiliary Signals

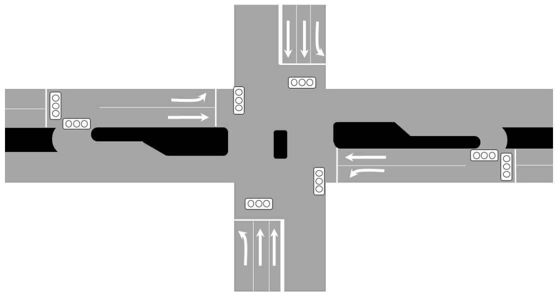
Auxiliary Signals have different purposes for TSS and MUT. Essentially, the auxiliary signals ensure efficient passage of vehicles from the upstream intersections onto the main signal stop line. Here, the movements being controlled do not conflict with each other and are assumed to have chosen their respective lanes upon reaching the pre-signal. Here, the auxiliary signals aim to stop vehicles at the pre-signal area so that only vehicles belonging to one signal phase can enter the storage area at a time. The main objective when controlling the pre-signal should be to maximize capacity subject to the main signal and storage area constraints. The queues in the storage area belonging to each phase are dependent on the pre-signal durations only. The pre-signal green times must be kept long as possible so that the main signal green times are maximized but not too long that vehicles run the risk of not being able to discharge at the main signal.

In an MUT, the auxiliary signals' main role is to manage the flows between conflicting movements at the crossover, where they approach their respective pre-assigned lanes at the main signal stopline. The auxiliary signal also reduces the travel time of U-turners by stopping the main link through vehicles at the crossover stopline.

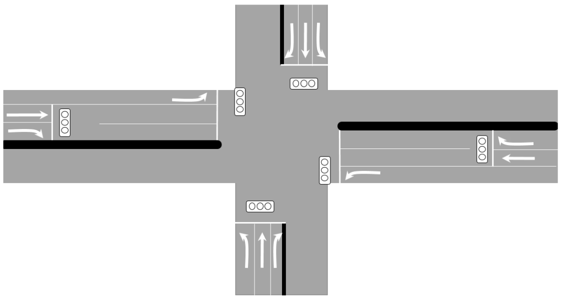
Similar to the TSS pre-signal, green time durations must be set such that the main signal green times are fully utilized. Ideally, the queues at the main signal must be cleared at the end of each cycle but short residual queues do not pose any serious consequences compared to TSS. Unless detectors are installed, it is difficult to accurately predict the



Regular signalized intersection.



Median U-Turn.



Tandem Sorting Strategy.

Figure 6.1. Intersection layouts used in the comparison.

volume of U-turners who wish to reach the main signal so to minimize waiting times for U-turners,

6.2 Operational Performance

A comparison of the operational performance between each intersection type is conducted. To ensure fair comparison, the three intersections have the same total lane widths. For RSI and TSS, three lanes are provided on the major link and three lanes on the minor direction. For MUT, however, the median lane consumed space so only two lanes are available on both links. TSS had two tandem lanes.

In the comparison, the effect of inadequate storage was neglected and adjustments considering stochastic headways were neglected. The objective of the comparison is to determine trends in the applicability of TSS and MUT and not to give specific demand values where each intersection type should is applicable. The same zero-conflict phasing plans as in the previous chapters were used and the following saturation flow rates were assumed: 2000 veh/hr for through movements (including those at pre-signals/crossover signal), 1800 veh/hr for left and right-turns, and 1440 veh/hr for U-turn. The allowable degree of saturation set was 0.90.

The delay and cycle lengths for a major demand of 60 % are presented for 10 % and 30

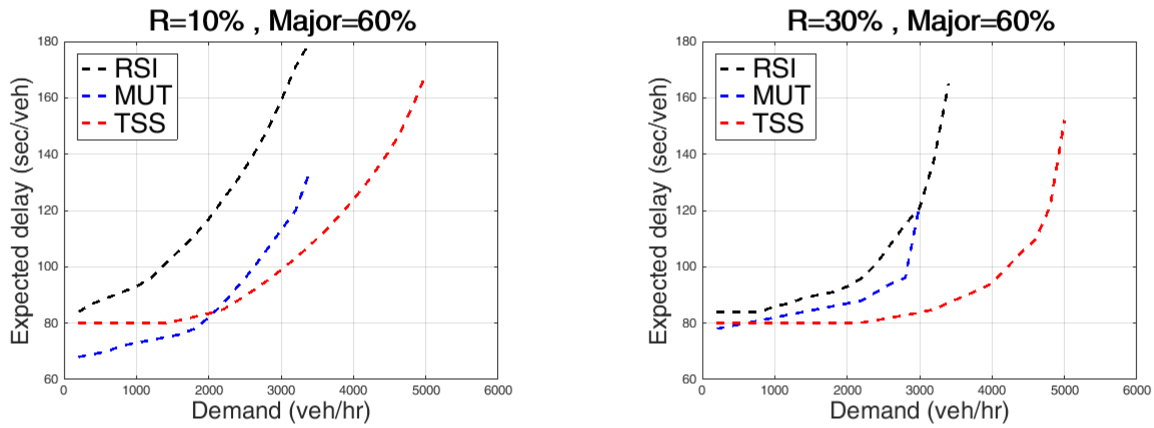


Figure 6.2. Delay comparison.

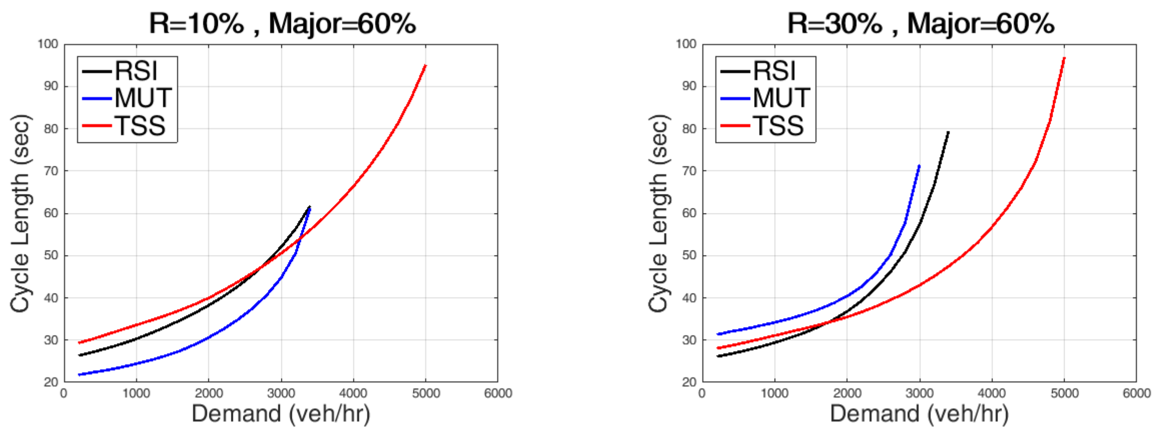


Figure 6.3. Cycle length comparison.

% right-turn rates (Figures 6.2 and 6.3). The data points for some intersections that do not reach 6000 veh/hr indicate that the degree of saturation constraint was not met thus the system is near saturated levels.

Since MUT only has three phases, it has the lowest cycle length at low demands (1000-3000 veh/hr) and also the lowest expected delay. At the 1000 veh/hr demand level (R=10%), the green times set to the minimum value of 15 seconds. Since MUT has the least number of phases, it will naturally have the shortest cycle length. However, MUT's performance declines at high right-turn rates.

At 10% right-turn rate, MUT reached the saturated state the earliest due to the reduced number of lanes. Consequently, TSS was shown to have the best performance at higher demands, as both RSI and MUT have already reached oversaturated state.

Figure 6.4 shows a plot of the demands considered with increasing right-turn rates. The areas are shaded according to the intersection type which yields the minimum expected delay. The boundaries between RSI, TSS, and MUT can be observed.

Here, it is apparent that the MUT performs best under low right-turn rates while

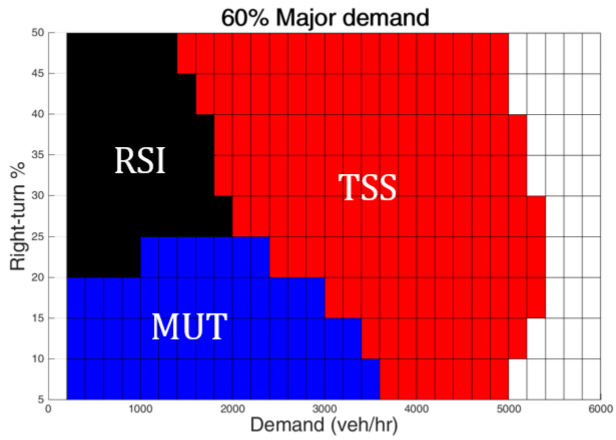


Figure 6.4. Intersection type with least expected delay.

TSS and RSI perform well at high right-turn rates. These results are consistent even for unbalanced demands. The results for other cases are summarized in Appendix B and C.

6.3 Operational Flexibility

In both TSS and MUT, the auxiliary signals can be turned off when demands are low in order to decrease the additional delays at these signals. Turning off the pre-signals in TSS reverts the intersection into an RSI and any additional delay caused by TSS is removed. Thus, a link with a pre-signal can enjoy both the benefits of RSI and TSS.

In case of MUT, operational flexibility comes in the form of the possibility of turning off the crossover signals when the demands are far from saturated levels. When the conflicting demands at the crossover are not significantly high, an unsignalized MUT has better overall performance as presented in (Autey et al., 2010). However, since turning off the crossover signals does not revert the intersection back into RSI, the Right-turners continue to experience the additional delay due to detour.

From the point of view of safety, turning the TSS pre-signals on or off may have some effect on driver expectations. Turning off the MUT signals may cause collisions at the crossover as reported in (Potts, 2004).

6.4 Geometric adjustments

When converting an existing RSI to TSS, the width of the existing RSI need not be changed to experience a capacity increase. However, enhancement of the intersection area may be required in the event that it cannot provide sufficient turning radius for the vehicles in

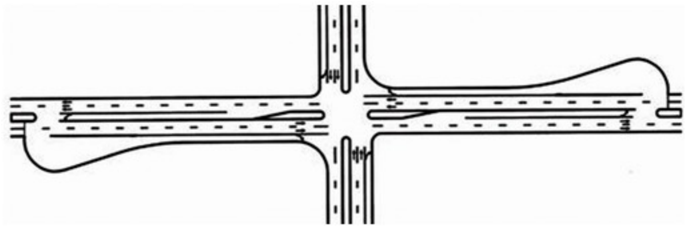


Figure 6.5. Alternative MUT design with loons.

Source:(Hughes et al., 2006a).

additional turning discharge lane. In the event that it is difficult to increase the intersection area, capacity enhancement can still be experienced even if only through vehicles utilize the tandem lanes.

For MUT, a wide median is required to allow vehicles to safely turn at the intersection. An alternative is to provide extra turning space on the opposite side of the crossover, called a "loon" (Figure 6.5). In any case, additional space is required so that safe turns can be executed.

6.5 Auxiliary Signal Distance

Apart from the limitations in link length, the distance between the main and auxiliary signal has a significant effect on the performance of the intersection.

In TSS, the auxiliary signal must be far enough to provide adequate storage space. The impacts of available storage space can be assessed using the equations in Chapter 4. It was also shown that when the storage area is inadequate, other option such as reverting to the Phase Swap Strategy are available to reduce capacity loss.

For MUT, storage problems may pose more serious impacts on the intersection performance. As was discussed in Chapter 5, inadequate storage upstream of the crossover can lead to queues spilling over from the U-turn approach onto the adjacent lane which can block upstream vehicles and cause the intersection to be in a locked state. Moreover, inadequate storage between the crossover and main signal on the main link can lead to an increase in residual queues upstream of the crossover which will exacerbate the storage problem. More careful consideration of the auxiliary signal distance must thus be carried out for MUT's to balance user costs and ensure efficient operation.

Chapter 7

Final Remarks

This dissertation evaluated two approaches that can improve the performance of a regular signalized intersection: the Tandem Sorting Strategy (TSS) and Median U-turn (MUT). Both approaches, operated with auxiliary signals, reduce the inefficiency in Regular Signalized Intersections due to additional turning phases. TSS accomplishes this by increasing the number of discharge lanes for each phase by controlling the vehicles that can enter the storage area. On the other hand, MUT maximizes the discharge lanes by prohibiting right-turns at the intersection and diverting right-turners to a downstream crossover.

This dissertation provided methodologies for operating systems with auxiliary signals. Expected delay estimation models were introduced for both TSS and MUT. Consequently, signal optimization programs for the entire system were formulated. In addition, capacity evaluations were also conducted using an analytical method for TSS and a simulation method for MUT. All these are discussed in Chapters 4 and 5 of the dissertation. This chapter provides a summary of the main contributions, followed by conclusions.

7.1 Summary and Conclusions

In Chapter 4, a discussion of the spatial and temporal factors that can affect the performance of the Tandem Sorting Strategy was made and the corresponding limiting equations were formulated. These allowed for a clearer understanding of the effect of each constraint violation on the performance of TSS. These constraints were combined to formulate a two-step optimization program that minimized intersection delays but maximized throughput at the pre-signal. A special feature of the formulation is that it permits vehicles to queue outside the storage area in the event that there is inadequate storage area length. This was found to improve TSS capacity especially in short links. The main advantages of TSS comes in two forms: a.) it increases discharge capacity, and b.) it decreases intersection delay by reducing the required green times in undersaturated conditions. Therefore, contrary to the recommendations made in (Xuan et al., 2011), the pre-signal can be turned on even before the oversaturated state is reached. In pre-timed signals, historical data can be used to plan when the pre-signal should be turned on or off. To deal with variations in discharge headways, reductions to the pre-signal green durations were made in order to decrease the probability of lane failure. The performance of TSS was significantly reduced when the headway variation increased.

In Chapter 5, performance evaluation was conducted for an MUT with signalized intersection using both an analytical and simulation-based method. The comparison was made under the assumption that an existing RSI is converted to an MUT given the same road width. Therefore, the MUT had fewer available lanes due to the space consumed by the median. Despite this, the results showed that MUT can still perform better than RSI especially under low right-turn demands. At high demands and high right-turn rates, a decline in then MUT performance was observed due to oversaturation at the main link upstream of the crossover signal, causing it to perform even worse than RSI. Although similar evaluations have been conducted in other studies, the analytical approach introduced here gives a better view of the extent at which MUT is beneficial. It can be concluded that installing MUT is worthwhile if low right-turn rates prevail.

Finally, a comparison between RSI, TSS, and MUT was conducted in Chapter 6. In terms of expected delay, MUT was found to be the better option under low right-turn demands while TSS is more appropriate for higher right-turn demands.

7.2 Recommendations and Further Work

To provide further improvements on the performance of TSS and MUT, future directions of this research are discussed in this section.

Towards a more comprehensive assessment of alternative intersections, the safety, cost, and other aspects of each alternative should also be studied, especially for Tandem Sorting Strategy which is a relatively new concept. On an analytical aspect, it will be useful to consider other transport modes such as buses, bicycles, and motorbikes in estimating the intersection performance. Although the presence of buses was used in the numerical experiment in (Xuan, 2011), a more generalized discussion on the treatment of other modes is still lacking.

The TSS analysis can be extended to consider multiple intersections and even to a network. One problem is: which pre-signals in the network should be turned on and how will this affect the route choice of vehicles? Principles of dynamic traffic assignment can be invoked. While the TSS analysis considered variations in vehicle headways, additional stochastic analyses can be conducted considering behavioral parameters such as lane changing and lane selection. These parameters can be tested under varying storage lengths or available space for queuing. Also, the assumption that the vehicles sort themselves evenly among the lanes may not always be followed. This can lead to lane failures so appropriate buffer times must be provided.

In this work, MUT spillovers were avoided by giving more green time to the U-turning vehicles, thereby shifting the delays to the main link. Alternative solutions to reducing U-turn demand may be to implement a half U-turn, meaning the vehicles from the minor direction are allowed to make direct right-turns. Similar to TSS where the pre-signal can be turned on or off, right-turn prohibitions may be activated or de-activated depending on the traffic conditions. Such "flexible" operation of MUTs can be studied. The problem that time-varying intersection rules will confuse drivers may soon be addressed by advances in driver assistance technology.

The delay optimization formulations assumed steady state traffic flows. This assumption no longer holds for demands at or near saturation. Under relatively short link lengths, it is important to consider queue lengths and avoid spillovers. One possible extension of this work is to perform more dynamic analyses that take into account physical queues. The literature on oversaturated signal control can be used as a starting point for this development. In this work, the two offset settings considered in the MUT analysis were pre-set and not included in the optimization to reduce complexity. Initial attempts to optimize offsets and green splits using the TRANSYT optimization tool sometimes yielded solutions that were not globally optimal. A methodology that provides the optimal offset, green times, and cycle length will improve the operation of MUT's.

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Appendix A

Derivation of ρ

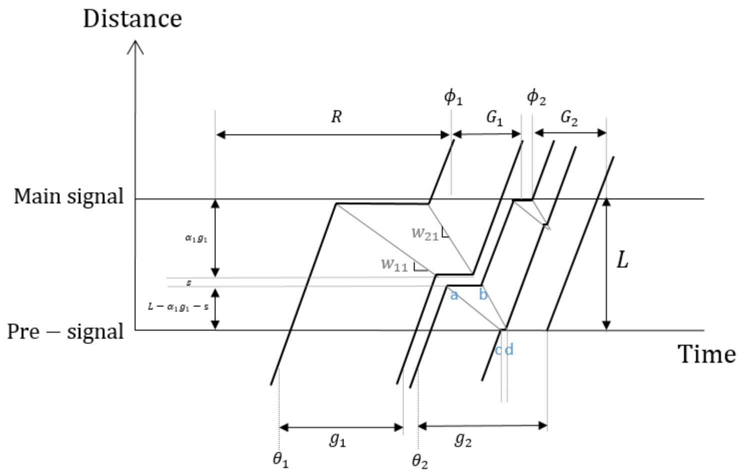


Figure A.1. Time-space diagram of queue evolution in the storage area.

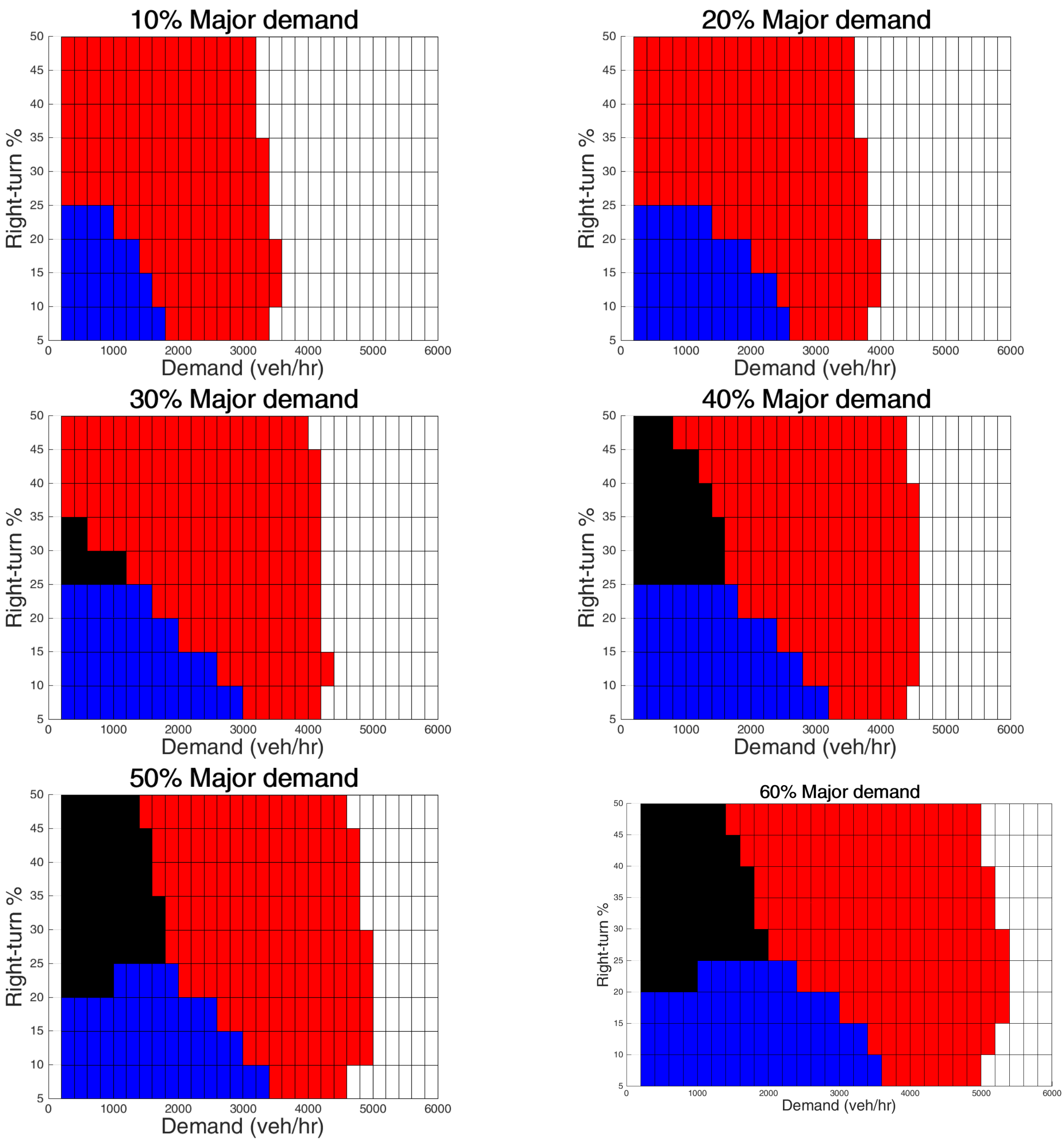
Derivation of ρ:
In the figure, ρ is the difference between time points marked by c and d. These time points are measured from points a and b, where a is the time when vehicles stop at the back of phase 1's queue i.e. $a = \theta_2 + (L - \alpha_1 g_1 + s)/v_f)$. Point b is measured from the time when the last queued vehicle in phase 1 starts moving. $b = R + G_1^f + h - (\alpha_1 g_1 + s/v_f)$ where G_1^f is the main signal green time used, $G_1^f = g_1\gamma$. The wasted green time is give by:

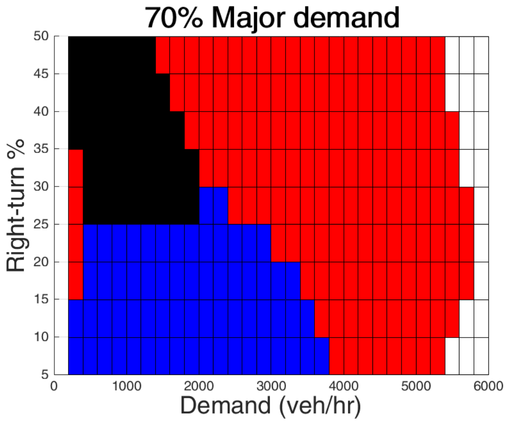
$$\rho = g_1\left(\gamma - \frac{\alpha_1}{v_f} + \frac{\alpha_1}{w_{22}}\right) - \theta_2 + R + h - \frac{s}{v_f} + (L - s)\left(\frac{1}{w_{22}} - \frac{1}{w_{12}} - \frac{1}{v_f}\right)$$

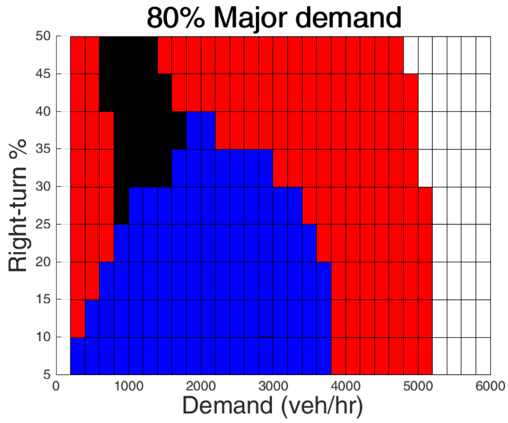
where:
$\gamma = \frac{n_2 S_2^p}{N_2 S_2^m}$
$\alpha = \frac{S_i^p n_i}{N_i J}$
s: spacing between vehicles (m)
h: discharge headway (sec)
w_{1i}: stopping shockwave speed for phase i (m/s)
w_{2i}: starting shockwave speed for phase i (m/s)

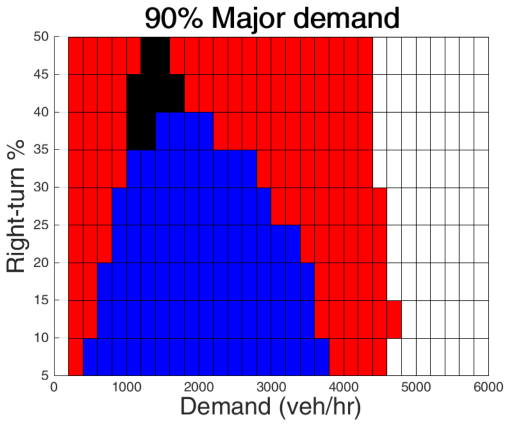
Appendix B

Operational Performance of RSI, TSS, and MUT









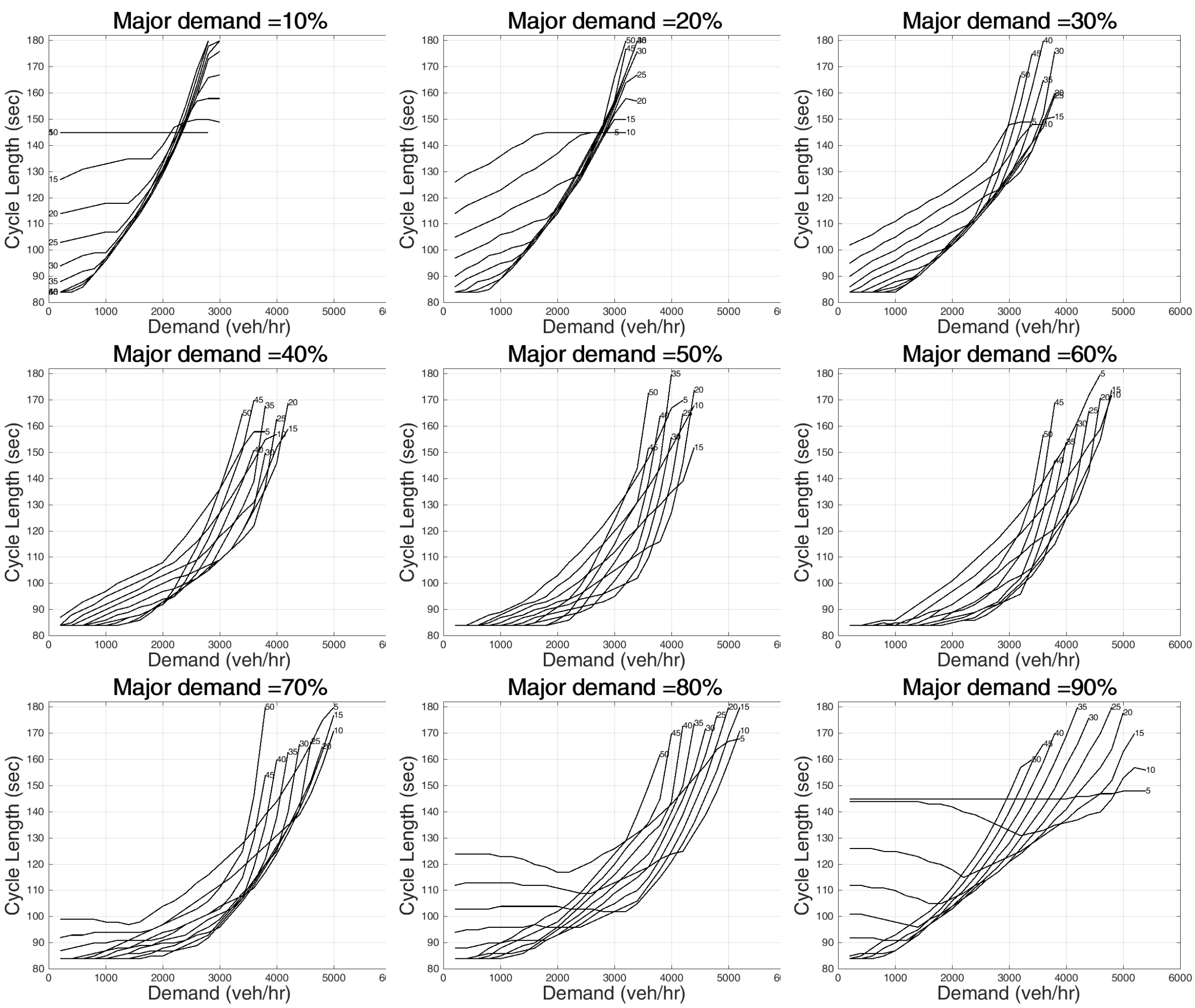
Legend:
Black: Regular Signalized Intersection
Blue: Median U-Turn
Red: Tandem Sorting Strategy

Appendix C

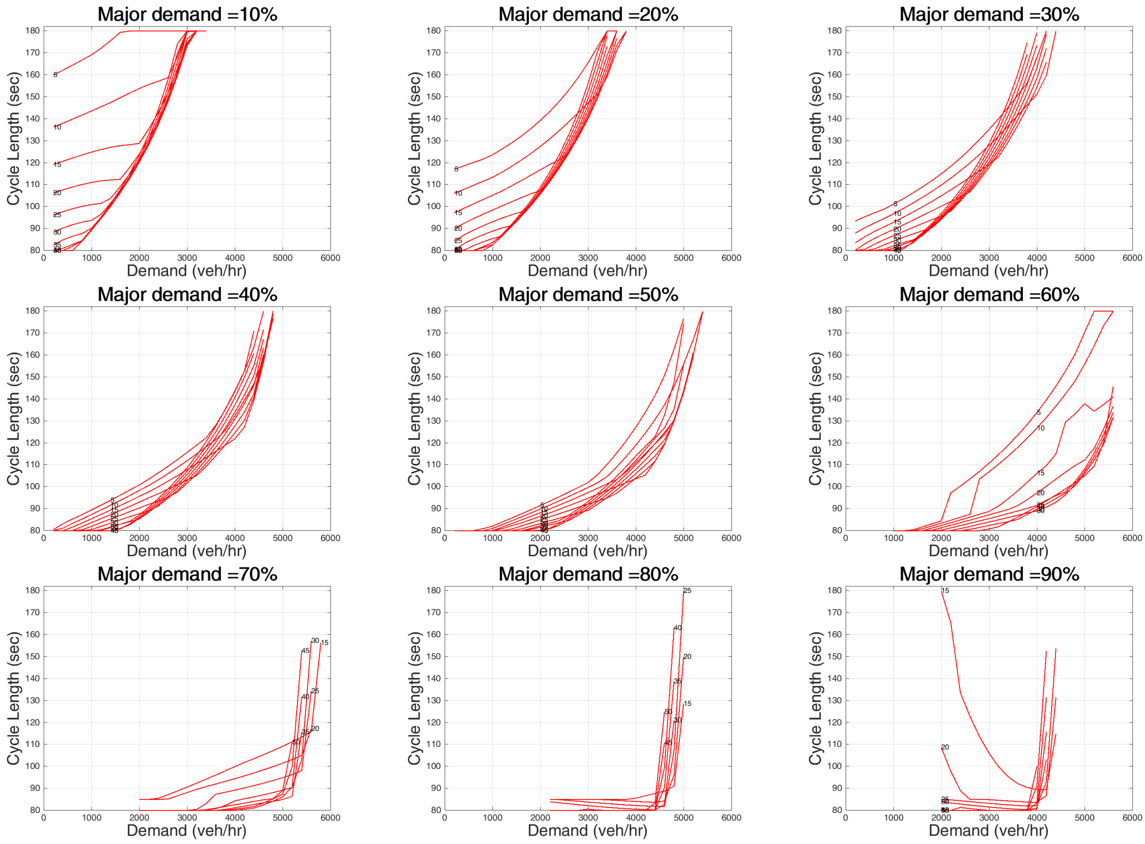
Optimization Result: Cycle Lengths

The figures are the results of the optimization conducted in Chapter 6. The labels indicate the right-turn rates.

C.1 RSI Cycle Lengths



C.2 TSS Cycle Lengths



C.3 MUT Cycle Lengths