DEVELOPMENT OF GUIDELINES TO COORDINATE TRAFFIC SIGNALS FOR NON-SIMILAR INTERSECTION CLUSTERS

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Degree of Doctor of Philosophy in Civil Engineering

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Abstract

Development of guidelines to coordinate traffic signals for non-similar intersection clusters

Despite the mega scale projects focusing on the long term benefits, proper traffic management initiatives should be introduced and implemented to reduce the unnecessary delays on roads resulting in road user frustration. Traffic signal coordination has been identified as one of the most sustainable solutions, if properly utilized. When it comes to traffic signal coordination, various techniques are available for coordinating similar type intersections. However, when non-similar intersection clusters are encountered, no proper guidelines have been developed for coordination.

The research sets out the preliminary requirements essential to be established for the progress of traffic signal coordination. As the first step, selection of an appropriate micro simulation model to support analysis and a procedure to identify and calibrate important input parameters are established. Second, guideline for signal phasing and timing design for individual intersections with fixed time signal timing is proposed. The guidelines are produced for intersections considering geometrical arrangements, traffic signal phasing and timing. Third, guidelines for real time traffic signal designs are produced where the guidelines address the extension of green split for different traffic situations.

Finally, the criteria for selecting intersections that should be clustered for traffic signal coordination is established. When clustering, importance of relying on travel time than distance is discussed when developing clustering guidelines. Further, seven basic categories of non-similar intersections based on the intersection geometry and priority directions for green platoon are identified. Based on the analysis, two matrixes are developed for peak period and off-peak period to be utilized by traffic and transportation engineers when non-similar intersections are encountered for traffic signal coordination. The developed guidelines are successfully verified using two case studies, for a selected Baseline road intersection cluster for fixed time traffic signal coordination and Kadawata intersection cluster for real time traffic signal coordination.

Keywords: traffic signals, traffic signal coordination, non-similar intersections

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Table of Contents

List of Figures

List of Tables

CHAPTER 1 : INTRODUCTION

1.1. Traffic Signal Definitions and Uses

Traffic signals is a traffic control mechanism which is installed in a particular conflicting location in order to facilitate smooth movement of road users, including vehicles and pedestrians. The traffic signals can either be implemented at an intersection, pedestrian crossing, railway crossing, parking area, bus lanes and other locations requiring force controls.

Further, Mcshane (1999) mentions that traffic signals are not just a physical system but rather a mechanism to impose a strong control over the natural fundamental human behaviours.

Traffic signals are introduced primarily for two reasons. One reason is for increasing the safety of the conflicting vehicular movements and the other reason is to reduce the unnecessary delays expected by the vehicles.

1.2. Research Gap Identification

1.2.1. Traffic Congestion

Traffic congestion has been a long-standing problem in developing countries as well as developed countries. Many signalized intersections in United States are currently operated at oversaturated conditions even in the morning as well in the evening peak hours with considerable traffic congestions (Chen, Xu, & Liu, 2013). Theories that have been produced are actually reliable and accurate only for certain conditions, where still no proper and accurate guidelines have been developed for accounting the complexities especially in oversaturated conditions (Chen, Xu, & Liu, 2013).

However, when it comes to reduction of delays several scenarios exist. When an intersection is under saturated, a traffic signal might not necessarily produce the expected delay reduction, if it is not properly timed. Even a traffic signal installation may increase delays when compared with the right of way rule control, due to stochastic variations of the vehicle movement, though the optimum signal timings are provided. This has been a good observation in many practical scenarios where in most of the locations amber blinking light is implemented during night times when the vehicular flow is significantly lower. However, this can increase the safety risks and also sometimes can result in deadlock situations due to not properly following the right of way rule.

On the other hand, when the intersections are operating at peak levels or at oversaturated conditions, the situation becomes adverse and significant delay reductions can be obtained with proper signal control, when compared with the right of way rule control situations, especially with the aggressive driving patterns observed.

1.2.2. Different Types of Traffic Signals

Intersection control traffic signals can be one of the most complicated traffic signal types when compared with the pedestrian signals and public transport control priority signals such as bus controls and train controls.

When complying with the international standards, traffic signals do need to adhere to certain guidelines in a general context. However, when it comes to varying patterns of driver behaviours, vehicles mix, peak off-peak variations, the countries have to face various difficulties in establishing proper guidelines which can still suit for international standards. As a result, there are different variety of traffic signals installed in different locations by different organizations and today there is no uniformity in signal indications, signal head placement and phasing selection. When considering the practical scenarios, even within the same geographical areas the placement of traffic signals differ significantly. The variations of locations, visibility, phasing arrangement, size, shapes of the signal heads and arrows significantly differ from one another.

This has contributed to confusion, safety concerns and sometimes unnecessary delays at intersections and resulted in user dissatisfaction in traffic management strategies used.

1.2.3. No proper mechanism for travel time savings valuations

Most of the new initiatives implemented may not produce intended results as no proper assessment is carried out on the feasibility of the initiatives. Though any initiatives become feasible in terms of technical terms, there should be a way to report that in monetary terms, so that even the non-technical decision makers can understand the gravity of the situations. One of the most important criteria for transportation project evaluation is the travel time, where a reliable figure for value of travel time estimation is lacking. It is also necessary to value travel time savings to justify project spending. Therefore, the research also focuses on valuing of travel time savings and determination of a proper and reliable method to estimate a monetary value for travel time savings, since the need has arisen to develop a strong case for value of time estimation

1.2.4. Unavailability of guidelines for traffic signal designs and signal coordination

There is a need to establish signal design standards taking in to account the latest trends in traffic signal development so that there will be consistency irrespective of the make of the traffic signal installed. Further, it is also vital to make provision to incorporate future coordination and other automation such as monitoring violations and gathering traffic flow information for real time updates. More importantly standardization will minimize user confusion and improve safety at traffic signals and help improving traffic flow and user satisfaction.

When it comes to traffic signal system installation, no proper guideline is followed. The only available and reliable guideline that will be used is the Webster's method (Webster, 1958), which give you a basic methodological way for the determination of key parameters and timings for the traffic signal installation.

However, when it comes to different geometrical arrangements, innovative phasing arrangements, effective timing shifts and smart pedestrians movements, Webster's method alone will not work effectively. Therefore it is needed to develop a set of guidelines to be used for traffic signal installation for individual intersections.

Further, in most developing countries, the traffic signals are rarely coordinated which results in significant delay increase (Kaczmarek, Cichocki, & Jabkowski, 2009). The performance of one traffic signal is significantly affecting the performance of the other traffic signal at another intersection in close proximity. The issues of the whole intersection cluster may have nothing to do with the individual intersections, but the combined effect of the intersection cluster. Therefore, a need has arisen to coordinate close by signals to achieve the effectiveness of signalization.

In coordinating traffic signals in similar type of intersections, models involving isolated intersections, grid systems, are developed and practiced in the Transportation Industry at present. However, when it comes to coordinating traffic signals in non-similar intersections the procedures become complex. No guidelines have been developed for the traffic signal coordination when it comes to different geometrical arrangements.

Further to this, it should be identified, to which direction priority should be given, and the variations of the vehicular flow in peak, off-peak hours. Further, the effects of pedestrians too need to be considered and can be crucial in some occasions.

According to the research carried out, though the American Highway Capacity Manual, considers the influence of the pedestrians and non-motorised users as an impedance factor, the German guidelines do not account the effect of them much (Brilon & Miltner, 2005). Therefore different countries would follow different guidelines when share of different modalities of operation is not same.

The traffic signal coordination can either be a fixed time system or can be a real time system. The fixed time traffic signals are the most common type of traffic signals that are currently installed. With the development of the technology, most of the developed countries have moved to real time traffic signals and traffic signal coordination. Most of the developing countries are yet to implement the real time traffic systems and no proper guidelines are produced to be followed for individual intersections as well as for intersection clusters when coordinating.

A proper signal coordination is of utmost importance, because the prevailing systems are not applicable to developing countries due to the varying driver behaviour patterns, geometrical arrangements of intersections, vehicle mix, peak off-peak variation and pedestrian behaviour. Therefore, the need for development of proper guidelines is vital for non-similar intersection groups of traffic signals and traffic signal coordination to increase the efficiency of vehicle circulation, reducing unnecessary delay for road users.

1.2.5. Non-existence of guidelines for intersection clustering in coordination

Further, it is needed to identify which intersections can benefit from traffic signal coordination. Some intersections might not benefit from traffic signal coordination, when considering the overall performance in every direction. Most of the critics about traffic signal coordination focus on the incremental increase of the delay for the non-coordinated directions. Therefore proper guidelines should be produced to identify which individual intersections can benefit from the traffic signal coordination in an overall perspective. Further the traffic signal coordination might not benefit for different traffic flow conditions. Therefore, it is required to determine under which conditions the traffic signal coordination would generate positive results.

Further, another important criteria to be considered is to check whether which intersections need to be clustered for traffic signal coordination purposes. No proper guideline is produced to adhere, when selecting which intersections are to be clustered for traffic signal coordination work. Therefore, a need arises to develop a criteria to select intersections for clustering when traffic signal coordination.

1.2.6. Data Requirements for software systems

When compared with the developed countries which have an efficient traffic, transportation and highway network, the developing countries are still struggling with the issues related to transportation. Lack of effective research that are carried out are actually worsening the situation of developing countries and very less efficient and sustainable solutions are implemented.

The research that are carried out are too mainstream and uses same sort of methodology which have been carried out over the past few years. Therefore, it highlights a research gap where novel methodologies are needed to be implemented in the research studies that are carried out.

One of the main techniques that are still new to developing countries are the traffic micro simulation software. The developed countries are continuously using traffic simulation software such as Transit-7F, Synchro, VISSIM etc. and successfully implementing the new initiatives. However, the developing countries are still way behind in practically using these in various application and research work. Therefore, it is very important to introduce these kinds of traffic simulation software to the industry and proper guidelines need to be produced when using these software for the traffic engineering research activities. Proper calibration is essential for the local conditions, when it comes to dealing with the respective traffic mix, peak off peak variations and driver behaviour patterns.

1.3. Objectives and Sub Objectives of the Research

1.3.1. Objective

The objective of the research is to develop set of guidelines for traffic signal designs and traffic signal coordination for non-similar intersection clusters.

1.3.2. Sub objectives

- To develop a procedure to calibrate and validate traffic micro simulation software
- To develop guidelines for design of traffic signals at individual intersections for fixed time systems
- To develop guidelines for design of traffic signals at individual intersections for real time systems
- To develop guidelines for traffic signal coordination at non-similar intersection clusters

1.4. Chapter Breakdown and summary

- **Chapter 1: Introduction**

Chapter 1 focuses on identifying the research background and gaps which has made the researcher perform the research study. It will describe the various issues that have been identified in traffic signal systems and traffic signal coordination in both fixed time and real time systems. The researcher would describe the importance of conducting the research in a wider and deeper perspectives and the value that it would bring about to the traffic engineering field and the economy. Further, based on the identified research gaps and questions, the objective and sub-objectives of the research would be developed.

- **Chapter 2: Literature Review**

This chapter reviews the current status of the traffic signals considering the world context especially in developed and developing countries. Starting with the history of traffic signals and the purpose of installing the traffic signals, the literature review would span to the present context with the modern dynamic and scientific technology involvement and software usage. It would critically analyse the existing literature and logical opinions would be produced and argued which can be used for the progress of the research. Further, useful insights and findings are obtained in formulating the methodology of the research from the literature review.

- **Chapter 3: Methodology, Analysis & Results**

Chapter 3 describes in detail the methodology that was followed for the research work. Each step of the methodology would be analysed in detail and the justifications for the selection of the steps of the methodology would be clearly provided. The methodology consists of a combination of analytical, experimental and simulation based approach with high focus on quantitative analysis techniques. Methodology does consist with a phased approach and a proper framework is developed in order to carry out the research work in a logical and methodical manner.

Further Chapter 3 discusses all the data collection, analysis work with respect to the various steps identified in the methodology. Using variety of tools, such as statistical tools, theoretical and simulation based frameworks, each step identified in the methodology would be critically analysed collecting respective data and information that is required.

With extensive analysis on the relevant steps identified in the methodology, the results and findings of each step in the framework would be produced with clear justifications.

- **Chapter 4: Case Study Applications and Verifications**

The developed guidelines need to be applied and verified using the case studies. Chapter 4 verifies the developed guidelines in the research using case studies, for traffic signal coordination on non-similar intersection clusters for fixed time and real time systems. The case studies would further address the variation of vehicle mix, driver behaviour and peak off-peak variations.

- **Chapter 5: Discussion**

The obtained results are further discussed in chapter 5, starting with the specific results and focussing further on the applicability and general context. The results are discussed identifying the limitations of each approach and also which can facilitate the research gaps still exist in the fields of traffic signal and traffic signal coordination.

- **Chapter 6: Conclusions and recommendations**

Chapter 6 sums up all the work that have been carried out in the research. Starting with the identification of the research gap and clearly highlighting the new knowledge and value that would bring about by the research, the conclusion would summarise the main findings of literature, the steps of the methodology and the framework that were followed.

The analysis techniques, results and findings obtained from the research would also be summarised in the conclusion chapter clearly highlighting the key findings. The conclusion chapter would further describe the limitations of the research. Limitations of the research address the issues that the research has faced while conducting the research and the assumptions the researcher has taken.

Further, to this the areas of further research are also recommended in the conclusion chapter for the benefit of the future researchers.

CHAPTER 2 : LITERATURE REVIEW

2.1. Introduction

In order to clearly formulate a well-rounded argument, the literature review is categorised into several sub sections as mentioned.

- History and development of traffic signals
- Fixed time traffic signals and traffic signal optimization
- Real time traffic and traffic signals
- Traffic Signal Coordination
- Benefits of Traffic Signal Coordination
- New Initiatives in traffic signals and traffic management
- Travel time savings valuations
- Modern Technology involvement
- Traffic Engineering Software usage

2.2. History and development of traffic signals

In developing set of guidelines for traffic signal design and coordination, it is important to review on the development of traffic signals over the past. The history can bring about various insights to the advancement of traffic signal and coordination aspects.

It is argued that the use of traffic control devices can go back to eras even before the beginning of the recorded history (Mueller, 1970) when analysing the recorded data and information. Muller (1970) reflects back on a form of traffic control device, which was used by the ancient road builders in Rome. This is one of the earliest recorded incident and this has provided clear directions and right of way for the travellers.

As for Ashvin (2015), the idea for traffic signal designing has arisen in the beginning of 1800's which is even before the automobiles were invented. The first recorded traffic signalling was developed on 10th of December 1868, outside the houses of parliament in London (Ashvin, 2015). This was a gas-lit traffic light which was again not for the vehicles, but to limit the conflicts of horse carriages with pedestrians when crossing the roads (Ashvin, 2015).

However, a signalling system had already been placed in rail roads in the 19th century (Gardner, 2017). The first recorded installation of the rail signalling system was introduced by a British Rail Road Engineer, John Peake Knight in 1868 (Gardner, 2017).

The traffic signal introduction initiative in United States was investigated by Gardner (2017), where the traffic congestion at New York was very significant, where it was mentioned that to travel from 57th to 34th street in New York took 40 minutes, though it was only 2.5km apart (Figure 2-1)

Figure 2-1: Traffic Congestion

Source: (Gardner, 2017)

This shows the significance of the traffic at New York in the late 19th century (Gardner, 2017). This is mainly due to the horses, pedestrians, street cars, bicycles and carriages competing for limited space in the Grand Boulevard area (Gardner, 2017), which is similar to the arguments brought by (Ashvin, 2015) referring to London.

A photograph of one of the first traffic signals installed at 57th street New York is shown in Figure 2-2.

Figure 2-2: Photograph of one of the first traffic signals installed

Source: (Gardner, 2017)

The main reason for having the traffic signals was not only to reduce the delays. However, the road users are typically getting frustrated over an installation of traffic signals at an intersection, because their primary concern was to expect a considerable delay reduction. This can be true in most of the occasions, but however what the road users need to keep in mind is that the traffic signals are not implemented only to reduce the delays, but also to increase the safety of the intersections reducing the number of conflicts.

The argument was proved even when referring to the history of traffic signals because installation of traffic signals at New York took place due to the traffic congestions caused by frequent collisions (Gardner, 2017). Gardner (2017) further elaborates that after the inventions of automobiles the cars started to rule the road which was one of the main reasons for the increase of collisions. This was further supported by the high end department factories and stores in place since bad traffic can create bad business (Gardner, 2017).

Muller (1970) argues that earlier forms of traffic signalling was done through the lanterns, semaphores and electric lights, which is somewhat similar to the argument brought by Ashvin (2015), where a gas lit traffic light was used which has to be controlled by a police officer using semaphore arms. Raising and the lowering of the semaphore arm has instructed the horse carriages to move or stop (Ashvin, 2015). With the need arising from the problems of congestion and hazardous nature, the development of more sophisticated electromechanical devices have come into existence (Mueller, 1970).

Ashvin (2015) tries to compare this situation to an occasion where a heated discussion is taking place in closed room. When there is no coordinator or a mediator, the conversation can turn up to a complete mess, which is the same thing that is happening if no control is placed for traffic at complicated situations such as intersections (Ashvin, 2015). Therefore, the need of more sophisticated devices need to be introduced.

"Recorded World's first electric traffic system" was introduced on 5th of August 1914 in Cleveland, Ohio, which is a system designed by Jamees Hoge and it was patented in 1918 (Gardner, 2017), which has paved way for more advanced traffic control systems.

When it comes to the history of modern day traffic signalling, the starting point of the traffic signalling was to display a single yellow light in order to alert the motorists on "caution" (Lamm, 2014). This can be similar to an amber blinking light, which is frequently used in the modern day traffic signals when the traffic flow is significantly lower especially at the night time. Lamm (2014) further argues that this single yellow light requires the motorists not to enter an intersection unless it is safer to proceed, which proves that the starting point could be the single yellow light.

With the time, two more colours were developed "Red ""for "Stop" and "Green" for "Go" (Lamm, 2014). However, Ashvin (2015) argues that the usage of these two colours red and green should date back to 1870 s where to control the traffic at night gas lit red and green lights were used instead of raising or lowering the semaphore arms due to poor visibility. However as for Lamm's (2014) argument, at the beginning, there were only three control leads in place; Red (Stop), Yellow (Be ready) and Green (Go), which were all single displays. Lamm (2014) argues that the main reason behind this system can be the simplicity of wiring and coding. However, this has created significant complications on the road users whether after the yellow light, the succeeding light is red or green (Lamm, 2014).

Due to these complications, when it is changing from red to green, with the yellow signal, the red signal was kept on. Even, when the light is changing from green to red, with the green indication the yellow light was kept on. Though the former system worked well, the latter system which is keeping the green signal on with the yellow signal, possessed significant risk to the road users, because when the signal is changing from yellow to red, the drivers tend to speed up the vehicles to somehow clear the intersection (Lamm, 2014).

Therefore, again the system has changed, and only when the signal is turning from green to red only the yellow signal was lit, which emphasises the drivers not to speed up the vehicles since the green light is off (Lamm, 2014).

Similarly, the traffic signal which was introduced to New York in 1920, was a simple system which only had two signal lights. However, opposed to the present day context a "Dark Green" indication was given for "Stop" and "White" or "Clear" indication was given for "Go" (Gardner, 2017). This is probably because in the early 19th century and even today, it was believed that "White" symbolizes purity, unspoilt and safety (Sherman & Gerald, 2009). This was further emphasized by Sherman & Gerald (2009) through their psychological research carried out about the colours and how they have emerged to the present day. Therefore at the beginning “White” was used for “Go” where the darker colours such as “Dark Green” was used to stop.

With these complications added to the existing traffic signal system finally it was agreed that the colours of spectrum would be used for traffic signal designing (Gardner, 2017). “Red” which had the highest wave length in the visible spectrum was used for “Stop”. “Amber” or “Yellow” which was next on the Spectrum was used for being Cautious for Stopping or Hurry up and “Green” which was next on the spectrum was used for Movement (Figure 2-3).

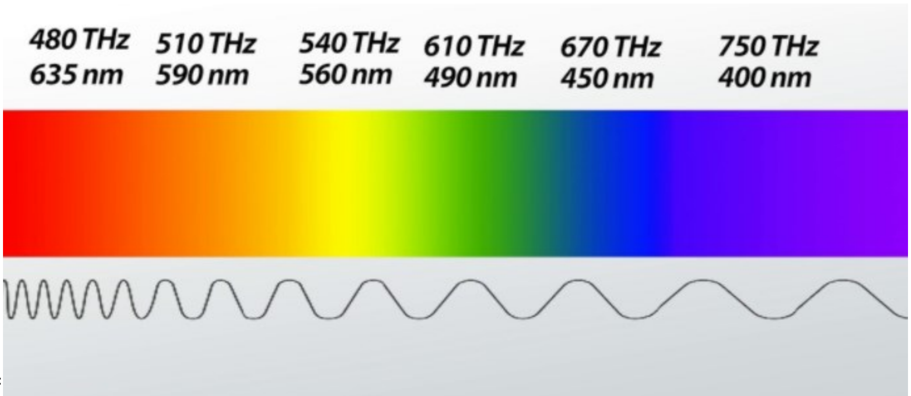

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Figure 2-3: Colours of the Spectrum

Source: (Gardner, 2017)

2.3. Fixed time traffic signals and traffic signal optimization

2.3.1. Fixed time Traffic Signals

With the development of the traffic signals, more complications had to be added to account for the varying behaviour of vehicles. In this background, traffic signal optimization was evolved to provide all the road users with the desired satisfaction.

Traffic congestion is common in all the major cities which cannot be avoided. However, what is important is to make sure at least a slow but smooth flow of traffic is obtained avoiding the intersection deadlock situations (Wu, Qi, & Yang, 2013).

In improving the efficiency of over saturated intersections, optimization of traffic signal timing place a major role. The optimization objectives of most of the research carried out are identified as maximum throughout, minimum delay and minimum average queue length (Li, Guo, Yang, Liu, & He, 2011) (Wu, Qi, & Yang, 2013).

In some researches, minimum total delay (Xinwu, Qiaohui, Huibin, & Xiaoyan, 2016) in the waiting queue is also considered as the optimization objective in developing traffic signal plan.

2.3.2. Traffic signal optimization

Traffic signal optimization should be carried out in any traffic signal system so that minimum average delay per vehicle is achieved. Different methodologies can be utilised for the optimization process.

The widely used and one of the oldest and well established method for traffic signal optimization is the use of Webster's method, which can be considered as a rational method for traffic signal design (Webster, 1958). The method incorporates various formulas developed by Webster and the optimum cycle time is calculated so that the least delay is expected (Webster, 1958).

Köhler & Strehler (2012) presents a cyclically time expanded network in order to provide with a realistic solution for traffic signal optimization and coordination. The developed model also account for the variations of travel time and utilise such to model the respective traffic signals (Köhler & Strehler, 2012). This model produced by Köhler & Strehler (2012) is different from the mathematical models developed by Serafini & Ukovich (1989) and Gartner, Little, & Gabbay (1975), since those models developed are static in nature and no new traffic assignment is simultaneously determined with the variations of traffic flows and mix. Reid & Hummer (1999) have also developed an optimization model for traffic signal timing using discrete time for urban traffic networks.

Further, arterial signal optimization models have been developed by Liu & Chang (2011) in order to cope up with the high demand and ineffective control methods experiencing. This model has the capability to either focus on maximizing the total throughout at an intersection or minimizing the travel time between two nodes (Liu & Chang, 2011).

2.3.3. Non-motorised transportation share

In some economies, non-motorised transportation consists with a considerable share in the traffic mix. In such situations, the traffic signalling and the intersection control strategies needs to be adjusted considering the difference in vehicle mix. Chen, Qian, & Shi (2011) address this scenario in a multi objective optimization method which further solved using genetic algorithms. Chen, Qian, & Shi, (2011)stresses the importance of changing the traffic signal timings for different share of non-motorised transportation in traffic mix and develops a model which can even give better timing than Webster Method (Webster, 1958), when it comes to non-motorised transportation dominant countries.

2.4. Real time traffic and traffic signals

It is required to identify that the traditional approach of predetermined cycle times, splits and offset optimizations are no longer valid when it comes to real time dynamic traffic management (Zhao-Meng et al., 2015). Therefore it is essential to incorporate the bandwidth optimization with the real time traffic actuated control to enhance the effectiveness of traffic operation (Zhao-Meng et al., 2015).

Real time Traffic management can be considered as one of the newest technology in traffic management. Though the developed countries have already implemented such real time traffic management systems, the developing countries are still in the process of implementing such. Since this is a new and evolving area, limited but effective research have been carried out.

Real time traffic signals can be very efficient in traffic engineering field where significant delay and waiting time reductions in the order of 25% can also be obtained when compared with the non-adoptive systems (Wiering, Vreeken, Veenen, & Koopman, 2004).

The real time traffic management research is generally done through simulations and field studies (Skabardonis & Christofa, 2011). The software such as VISSIM, transit 7-F, Synchro would be useful in carrying out and analysing the simulations. Further

Wahlstedt (2011) mentions difficulty in using analytical models for dynamic signal timing calculations and enhancements.

Skabardonis & Christofa (2011), in their research in determination of the impact of transit signal priorities, has proposed a methodology to understand the effect of transit signal priorities without going through the hassle of developing a simulation model for a particular practical application. In real time traffic management, the main two parameter which have been looked at are the extension of green time and truncation of red (Skabardonis & Christofa, 2011). This has been further recommended by Messe & Nageswara, (1996), with some additional parameters. However, the methodology developed may work for uniform vehicle behaviours, but the effects of non-similar intersection clusters may affect the accuracy of results.

Estimating queue length in a real time traffic system does create some problematic situation. In the traditional approach, which is based on input-output methods, the real time queue length estimation is possible only when the queues are shorter than the detector and stop line (Wu & Yang, 2013). Therefore some dynamic analysis of real time queuing is important where the theories such as, queuing models, shockwave theories may be beneficial.

Wu & Yang (2013) has developed model to solve the above issues using RFID detector data for the estimation of real time queue. This model addresses the issues with traditional input-output based method and the model has been validated using real time practical scenarios (Wu & Yang, 2013). The findings of the research can be beneficial for the research considered, when addressing real time data input source is very important. RFID detector data can easily provide the required solutions.

The level of real time traffic signal has also been expanded even to disaster management level, where in an urban evacuation, use of transit real time signal practices are also sought after (Parr, Kaisar, & Stevanovic, 2011) which would not let the police officers lives in danger in such a situation.

Further to the above methods, several important models such as "RHODES real time traffic-adaptive signal control system" (Mirchandani & Head, 2001), RT-TRACS Real time adaptive Control System (Gartner, Pooran, & Andrews, 2001) have also been developed which uses real time traffic flow data as input data to solve a particular traffic problems.

One of the newest ways of controlling the real time signals is to use multi-modal high-resolution data (Muralidharan, Coogan, Flores, & Varaiya, 2016) in the management of intersections.

Railway

Real time traffic management is not only applicable to the road industry. The same concepts are also applied in Railways and Railway traffic management. The concept brought forward is straight forward which involved with letting the trains wait at the stations, to avoid the modifications of the speed profiles while in open corridors (Corman, D'Ariano, Pacciarelli, & Pranzo, 2009), so that a continuous green wave is obtained. This is very important when the open corridors are the bottlenecks of the railway system, which is very common in developing countries and this can create more research areas to be explored especially considering the developing countries. Further, these real time railway traffic management systems may bring about new initiatives such as rerouting of trains for more efficient railway systems, flexible departure times, etc. (Corman, D'Ariano, Pacciarelli, & Pranzo, 2009).

2.5. Traffic signal coordination

Several of research papers and findings were also referred with respect to the traffic signal coordination and can date back to even half a century with the efforts of synchronizing the traffic signals in order to obtain the minimum delay (Hillier & Rothery, 1967). Traffic signal coordination is considered as one of the most cost-effective methods of improving the traffic situation in an urban area (Pillai, Rathi, & L. Cohen, 1998).

2.5.1. Need for traffic signal coordination

A variety of literature sources was referred about the traffic signal coordination and the need for establishing a properly coordinated system. Most of the previous research are focussing heavily on isolated intersections, which is further impossible to observe in the complex behaviour of vehicles and arrangement of intersections.

In order to cope up with the ever increasing traffic congestion, various methodologies are followed and implemented. One of the older methods developed was to use the semi-graphical methods to optimize the green splits in isolated intersections (Gazis, 1964).

Further, another method is to increase the capacity of the considered intersection which can result in an increase of the throughput in every approach (Yu, Sulijoadikusumo, & Prevedouros, 2012). This has been implemented by introducing intersection treatments like lane additions, restrictions of turning movements, grade separations, etc. (Yu, Sulijoadikusumo, & Prevedouros, 2012). These traditional treatments may limit the increase the capacity of the intersection and relieve the traffic congestion, due to complexity of traffic and vehicle behaviour. The traffic at downstream intersections may get deteriorated due to the increase of upstream arrivals (Yu, Sulijoadikusumo, & Prevedouros, 2012).

The situation can get worsened with the extended queues at downstream intersections negatively affecting the upstream intersections also, despite the treatments at upstream intersections (Yu, Sulijoadikusumo, & Prevedouros, 2012).

The above phenomena is heavily observed in urban areas of almost all the countries whether irrespective of developed or developing, and the situation can get worsened with the high traffic volumes, longer cycle lengths and close proximity of intersections (Yu, Sulijoadikusumo, & Prevedouros, 2012).

Jinpeng & Zhang (2012) have also determined that the cycle length can negatively affect the delays at an intersection as well as the emissions, whereas the impact on delay can be significantly greater.

2.5.2. History and the Need for clustering of intersections

The need of clustering the intersection for effective control can date back to 1960 s, where Hillier & Rothery (1967) marks the first few people to start researching on identification of intersections for coordination, though their primary concern was to reduce the delays expected in traffic signal coordination. However, with the gradual advancement of traffic signals, the need has become more crucial in having a proper method for identification of control region for traffic signal coordination. The first significant effort was carried out by Yagoda, Principe, Vick, & Leonard (1973) who identified that significant efficiency increase can be expected by interconnecting the isolated intersections. Further, it has been identified that the usage of combination of software such as TRANSYT-7F & PASSER II (Chang, 1985) and TRANSYT & SCOOT (Robertson & Hunt, 1982) for identification of interconnection of the intersection and benefits of coordination, should be the way forward.

With the efforts in traffic signal coordination, it is important to identify which intersections would benefit from coordination. Several parameters were used to identify the coordination potential of an intersection such as the reduction in queue index (Robertson & Hunt, 1982).

2.5.3. Development of an index for clustering

The first criteria for a clustering index was developed by Yagoda, Principe, Vick, & Leonard (1973), which assumed a basic index called "coupling index" which was the ratio of link volume to link distance. This has led to most of new research areas and number of research were conducted in identifying a criteria for the control region for coordination.

Wilshire, Black, Grochoske, & Higinbotham (1985), which stressed on considering the distance as the criteria for the identification control regions. The produced criteria was very basic where any two or more signalized intersections, which are in the radius of half a mile, should be coordinated for increase of efficiency of intersection clustering. However, having the distance alone as the criterion can produce inaccurate results when the traffic flows become complex, as well as when there is high accessibility within the intersections which can trouble the ongoing platoon of vehicles.

In the same time period, basic similar models such as "desirability index" (Chang & Messer, 1986) was developed, which incorporates a platoon dispersion model, for control region identification purposes. This has been one of the most suitable model that time, since it has accounted for various factors such as traffic flows and geographical considerations, when compare with the other models which were too simple and would not create reliable results with the effects of traffic mix, volume differences, etc. (Duan, Li, & Zhang, 2009).

However, even the desirability index can become invalid, when the situation becomes complicated with the addition of roads, intersections, traffic mix, peak off-peak variation etc (Duan, Li, & Zhang, 2009). Therefore, Duan, Li, & Zhang (2009) were more concerned on developing a model expressive and flexible model for control region identification purposes.

2.5.4. Existing widely used methods for clustering

Literature suggests limited but meaningful indexes that are produced for the traffic signal coordination specially in identifying which intersections are to be coupled. Though several but limited number of methods are developed for identification of the control regions for traffic signal coordination, still considerable research gap exists on a reliable and more accurate method predictions.

Hypergraph models

In generating the expected control region for traffic signal coordination, hypergraph models have been developed on when to coordinate the intersections as well as how to establish the control regions (Duan, Li, & Zhang, 2009).

The hypergraph model maps the given road network to a hypergraph, where the vertexes representing the intersections and hypergraph partitioning algorithms were used for identification of intersection control regions. Hypergraph model can be utilised for peak, off peak variation.

However, in order for hypergraph models also to be effective certain criteria need to be satisfied where significant emphasis is placed on the locations of the intersections. If the intersections are located similar to a grid, the hypergraphs models would be effective. Therefore the model may not be effective when the locations of the intersection are more complicated.

Coupling Index

With all the developments, Bonneson (2011) suggests to use the Coupling Index developed (Equation 2-1), which is a modified version of what was developed in early 1970 s by Yagoda, Principe, Vick, & Leonard (1973).

$$\text{Coupling Index}= \frac{Two\ Way\ Flow\ Rate\ (\frac{veh}{hour})}{Distance\ between\ the\ intersection\ (feet)} \qquad (2\text{-}1)$$

(Source: Bonneson (2011)

If the Coupling index is more than 0.5, Bonneson (2011) suggests to couple the two intersections for traffic signal coordination purposes. However, this can create number of issues where in countries that have higher number of access points and traffic disturbances within two intersections the above parameter of distance might not necessarily generate the expected results.

Strength of attraction

Fan, Winkler, & Tian, (2011) identifies the factors that need to be considered for coupling index varies with number of factors and is given a name as strength of attraction. The most common factors would be the flow rate, distance, travel time and the speeds. Further, Fan, Winkler, & Tian (2011) suggest to modify and localize the factors for any differences in traffic mix, access points and peak off-peak variations.

Coordinatability Factor

This is also another techniques used for identifying which techniques are to be used for identifying coordination potential intersections (Fan, Winkler, & Tian, 2011). With opposed to the coupling index and the strength of attraction, the significance in this technique is to use the land use patterns for identification of intersections for coordination and coupling work.

2.5.5. Safety and clustering

It is important to note that not only the delay minimization is important in traffic signal designs and coordination the effect of safety should also need to be considered. Through research, it is concluded that the spacing of the intersections do create significant impact on the safety aspects (Xie, Wang, Huang, & Chen, 2013). As for the research conducted by Xie, Wang, Huang, & Chen (2013) mention that the intersections which are in close proximity have tended to show higher crash frequencies.

Not only the distance, but also the speed between the intersections also is also an important parameter for the safety aspects, where fewer crashes were associated with corridors with lower mean speeds when compared to higher mean speeds (Xie, Wang, Huang, & Chen, 2013).

2.5.6. Methods and Techniques for traffic signal coordination

Though optimizing the offsets is the centre of attraction in traffic signal coordination, most of the research (Lan, Messer, Chaudary, & Chang, 2004) (Hung, 2014), Koshi (1989) focus on individual cycle time optimization over the offset optimization in coordination methodologies. Koshi (1989) further concludes that through individual cycle time optimizations, higher delay and stop reductions can be obtained over the offset optimization. This is arguable since the individual cycle time optimization has a limit for the capacity of intersection cluster, especially when the non-similar intersections are incorporated.

Most common approaches when dealing with traffic signal coordination and optimization are minimization of delay and maximization of progression bandwidth (Lan, Messer, Chaudary, & Chang, 2004). At some instances, these two criteria can be conflicting to each other. In order to avoid such conflicting scenarios, a compromise approach is presented (Lan, Messer, Chaudary, & Chang, 2004), where it suggests to utilise a trade-off approach between the delay minimization and progression bandwidth maximization (Lan, Messer, Chaudary, & Chang, 2004) by using the MILP models and MAXBAND model. This has successfully produced better delay reductions with respect to the user defined level of service, by optimizing the cycle lengths, green splits and phase sequences bandwidth (Lan, Messer, Chaudary, & Chang, 2004).

2.5.7. Maximize bandwidth

Maximization of bandwidth is one of the main criteria considered in traffic signal coordination, where several techniques are used for maximization, such as mixed-integer linear program method, Land and Powell Branch and Bound Technique, etc. (Pillai, Rathi, & L. Cohen, 1998). Further to this, Little, Kelson, & Gartner (1981) also focus on maximization of bandwidth using MAXBAND and Messer, Haenel, & Koeppe, (1974) using PASSER. Zhang, Song, Tang, & Wang (2016) have also developed few models such as MaxBandLA and MaxBandGN for traffic signal coordination purposes with the knowledge of Little, Kelson, & Gartner (1981) on bandwidth maximization.

2.5.8. Minimization of delay

Opposed to the maximization of bandwidth, another group of researchers were focussing on minimization of delay in gaining the highest efficiency in traffic signal coordination such as Robertson (1983) using TRANSYT and Reid & Hummer (1999) using Synchro.

2.5.9. Offset optimization

Offset optimization can be considered as one of the key criteria for traffic signal optimization. Traditionally, offset optimization has been carried out using the travel time between the intersections and the related traffic volumes (Hu & Liu, 2013). One of the main limitations of this approach is that, the stochastic nature of vehicles are not accounted under such scenarios, which can lead to inefficient traffic signal coordination.

However, with the advancement of technology, new methodologies are produced to account for the dynamic and stochastic nature of vehicles, such as archived traffic signal data (Hu & Liu, 2013). Hu & Liu (2013) develops a data driven archived model for offset optimization, which can account for the vehicle actuation addressing the effects of real time traffic signal coordination.

Further, research have also been carried out in the areas of real time traffic signal coordination in a multi modal signal priority controls in order to increase the efficiency of the complete network (He, Head, & Ding, 2014).

2.6. Benefits of traffic signal coordination

2.6.1. Positive sides of Traffic Signal Coordination

- **Delay Reductions**

Various research have been conducted on traffic signal coordination as well as the different situations that traffic signal coordination might need special attention.

Since the delays at traffic signals can create a significant impact on the public transport, it is required to implement a system for public transport priority for traffic signals. According to the research (Wahlstedt, 2011), the use of traffic signal coordination for bus priority lanes, can bring about 9% travel time reduction for the bus passengers in an experiment conducted in Sweden, which is verified through the field simulations.

It is important to find out the sustainability of traffic signal coordination with the introduction and implementation of new systems such as bus priority lanes. Wahlstedt (2011) in his research tries to find out the impact of Bus Priority System on the established traffic signal coordination whereas Ma, Yang, & Liu (2010) are interested in generating the optimal combination of priority strategies in a coordinated network.

Hung (2014) has conducted a research on expected safety and operational benefits pertaining to the introduction of traffic signal coordination. On comparative terms significant travel time reductions in the range of 12.9% can be obtained in the morning peak hour and 26.6% in the evening peak hour (Hung, 2014) in an experiment conducted at Kuching, Malaysia.

- **Reductions of stops frequencies**

Not only the delay reductions, but traffic signal coordination has significantly reduced the average number of stops also in the order of 23% during the morning peak hours and 28% during the evening peak hours (Hung, 2014).

Rakha et al. (2000) also conclude that on average vehicle stops would be reduced by 3.6% through properly coordinated traffic signals, which shows that location and set up of the traffic signal coordination is important.

- **Benefits for pedestrians**

Traffic signal coordination not only provides benefits for the motorised and non-motorised vehicles but also a properly installed traffic signal coordination can bring significant benefits for pedestrians as well.

Virkler (1998) in his research on identifying the benefits for pedestrians in traffic signal coordination, emphasises on the importance of a proper trade-off between the vehicular delay and pedestrians delay. With the variations of the signal coordination plan, significant platoon of pedestrians are observed due to the timings of upstream intersections and can create issues with sustainability if not properly accounted for the impact on pedestrians (Virkler, 1998).

- **Safety increase**

Safety at an intersection or intersection cluster may depend on various factors and factors such as protected turns, channelization, and three-leg intersections have proved to lessen the crashes expected (Wang & Abdel-Aty, 2006).

Not only the operational benefits, but also significant safety benefits can also be obtained through properly installed traffic signal coordination system (Li & Tarko, 2010). According to Hung (2014), a significant reductions of crash frequencies were observed such as around 96% reduction during morning peak hour and around 65% during the evening peak hour with the installation of traffic signal coordination. Rakha et al. (2000) also mention that the crash risk reduction was around 6.7% with proper traffic signal coordination.

Guo, Wang, & Abdel-Aty (2010) mention that significant safety improvements can be obtained from traffic coordination of closely spaced intersections but however for obtaining a more favourable result in safety perspectives such as intersection geometry, environment and land use should also be considered.

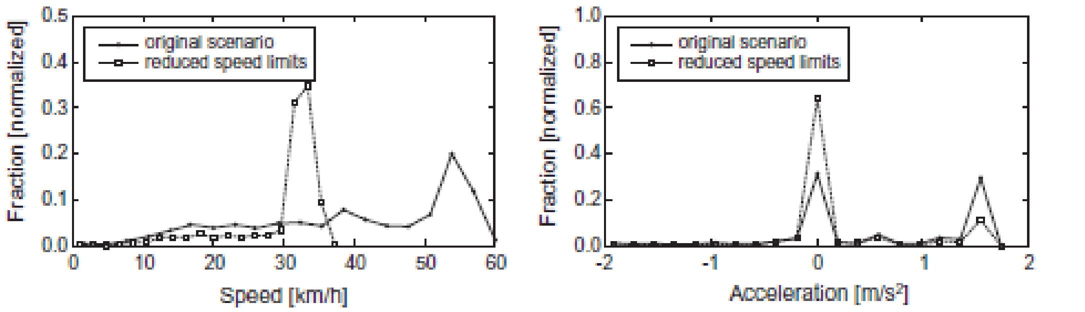
- **Vehicle emission**

Traditionally, the effectiveness of any signal control mechanism was obtained through the criteria such as efficiency and safety (Meneguzzer, Gastaldi, Rossi, Gecchele, & Prati, 2017). However, with the sustainable and green concepts being brought forward, the concern on the environment and environment pollution are key perspectives to be addressed. Number of research are conducted in order to come up with a proper solution addressing the sustainability and green perspectives (Li, Li, Pang, Yang, & Tian, 2004).

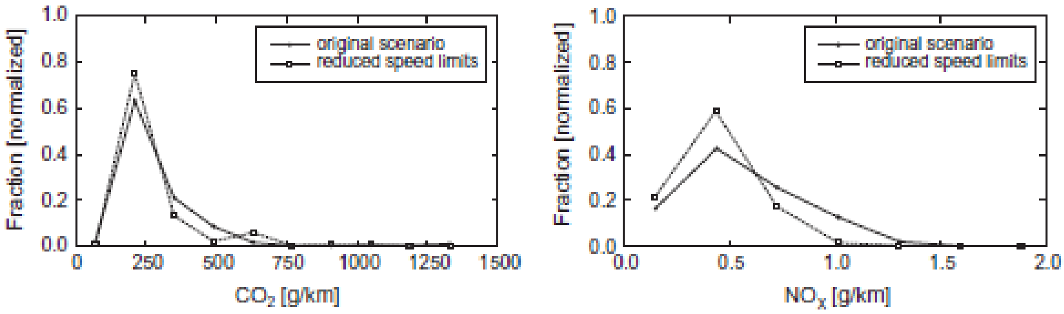
The traffic signal control system implemented has a strong relationship with the vehicle emission (Zhang et al., 2009). Zhang et al. (2009) discovers that significant emission reductions can be obtained by the use of alternative signal timing strategies with the use of a combined system using VISSIM traffic simulation model and VSP-based emission model.

A similar to research by Zhang et al. (2009), Madireddy et al. (2011) have carried out study on the effect of speed limit reduction and traffic signal coordination on the vehicle emissions based on Belgium. Madireddy et al. (2011) have also used an integrated model combining the micro simulation model Paramics with the Emission model VERSIT.

Few important findings were obtained which can be crucial for the progress of the research. When the speed limits were reduced from 50 to 30 km/h, significant reduction of CO_2 and NO_x emissions were observed in the residential areas of Antwerp, Belgium (Madireddy et al., 2011). Similarly, with the introduction of green wave signal in traffic signal coordination, an emission reduction of around 10% were also expected (Madireddy et al., 2011) (Figure 2-4).



3. Normalized distributions of instantaneous speed and acceleration, for vehicles driving within the residential part of the network



ig. 4. Normalized distributions of CO_2 and NO_X emissions per km, for vehicles driving within the residential part of the network

Figure 2-4: Impact from reduced speed

Source: (Madireddy et al., 2011)

Vehicle emission can be highly influenced by stops as well as the delay. Various research were carried out in that regard, where it was found out that the vehicle emission is correlated with the stops rather than the delays (Jinpeng & Zhang, 2012). Further, Jinpeng & Zhang (2012) elaborates that in traffic signal coordination, an early arrival situation can generate higher emissions than a late platoon arrival. This can be a good finding for the research, where it would emphasize on the importance of a sufficient bandwidth of a traffic signal coordination.

Li, Wu, & Zou (2011) also prove that the amount of CO and NOx emissions are considerably higher in the acceleration and deceleration phases when compared with the vehicle idling situation, which also proves the findings brought by Jinpeng & Zhang (2012).

In another research it has found that, by the introduction of green wave in traffic signal coordination, vehicle emissions can be reduced in the range of 10% to 40% under the most favourable situation (De Coensel, Can, Degraeuwe, De Vlieger, & Botteldooren, 2012).

Opposed to the above findings, Rakha et al. (2000) mentions that no significant emission reduction was observed in hydrocarbons and NO_X, and most importantly an increase in CO by 1.2% was observed in an experiment conducted in Virginia, United States, may be due to the differences in traffic mix in the research study area.

- **Sound Pollution Reduction**

Another benefit of proper traffic signal coordination is the reduction of unnecessary sound expected in accelerations and decelerations. In a research conducted in urban city area, the sound levels are expected to be reduced in the range of 1 - 1.5 dB, by the introduction of traffic signal coordination (De Coensel, Can, Degraeuwe, De Vlieger, & Botteldooren, 2012).

- **Fuel efficiency**

Through proper traffic signal coordination, fuel consumption can be reduced by on average 1.6% which can directly benefit the vehicle owners (Rakha et al., 2000).

2.6.2. Negative impacts

It is undoubtedly accepted that a properly timed efficient traffic signal coordination, improves the efficiency of the road way and thus reduces the delay and emissions. Despite, the benefits of traffic signal coordination, several research were carried out to analyse the negative impacts that the traffic signal coordination may bring about.

Tindale & Hsu (2005) mention that engaging in unsafe or speedy driving behaviour in order to stay in the "Green Platoon" can create safety concerns. The situation may be worsened in one way traffic signal coordination, because the movements are fewer compared to two way operation and thus allowing the motorists to engage in highly unsafe driver behaviour (Tindale & Hsu, 2005).

One of the interesting findings, depict that out of the total crashes in District 7 of USA, nearly 15% of the crashes are due to disregarded traffic signals happening due to unsafe driver behaviour in traffic signal coordination, where in some streets the value is touching 25% (Tindale & Hsu, 2005). This has resulted in carrying out various researches related to automated red light running cameras (Ahmed & Abdel-Aty, 2015).

Traffic signal coordination can create negative impacts in terms of delays as well. Wahlstedt (2011) in his research on the impacts of bus priority lanes on coordinated traffic signal systems mentions that having the bus priority lane in a coordinated traffic signal can result in a delay increase of other vehicles in the order of 6% for main streets and 13% for cross streets, though significant benefits can be achieved for the bus priority lanes.

Similar to the above arguments, one of the main critics of traffic signal coordination is the additional delay created for the minor direction (Zhou, Hawkins, & Zhang, 2017). This can create user frustration and dissatisfaction if not managed well. Concepts such as uneven double cycling can be utilised in such situations (Zhou, Hawkins, & Zhang, 2017). This will be useful in occasions where significantly larger cycle times need to be provided in maximizing the bandwidth at one or two intersections. Some intersections, might not need larger cycle lengths and the concept of uneven double cycling can be utilized to reduce the delays of minor directions (Zhou, Hawkins, & Zhang, 2017).

This suggests that careful attention needs to be placed on developing the guidelines for traffic signal coordination, so that the overall delay needs to be considered rather than only the coordinated direction delays, because significant negative impacts can be observed in minor directions.

Traffic signal coordination is a subject which should be dealt with proper understanding. It is frequently observed in practice that the traffic signal coordination might not necessarily produce the expected outputs in saturated as well as unsaturated conditions (Lan, Messer, Chaudary, & Chang, 2004). This can create negative and significant delay increase for the uncoordinated directions (Lan, Messer, Chaudary, & Chang, 2004)

2.7. New initiatives in traffic signals and traffic management

2.7.1. Speed limit reductions

Another common initiatives that is brought forward for establishing better roads is the reduction of speed limits. This will inevitably enhance the safety aspects of the road users. However, Yang, Wang, & Yin (2012) mention that rather than safety enhancements, several other benefits can also be obtained when considering the macro level system performance of the network. Speed limit reductions can relocate the existing traffic flow in an equilibrium manner, though it can violate the principles of travel time-flow relationships (Yang, Wang, & Yin, 2012). This can be considered as a different mechanism to regulate the traffic flows, where the authors compare the speed limit law to a toll charge scheme and can be a solid solution for network flow management (Yang, Wang, & Yin, 2012).

2.7.2. Separate right turn phase

In order to increase the capacity of an intersection, number of new and innovative initiatives are brought forward, through various research that have been carried out.

In any intersection, one of the main issues has been the right turns that are occurring and a separate turn phase is the most common methodology used which significantly reduce the capacity of an intersection (Xuan, Daganzo, & Cassidy, 2011). Various techniques are used to eliminate the impact of right turns such as eliminating the right turns at an intersection, allowing the vehicles to take a left turn followed up by U turns, using a right turn only phase in signal phasing, etc.

Xuan, Daganzo, & Cassidy (2011) have been quite instrumental in proposing a methodology to eliminate the effect of right turns at an intersection. Xuan, Daganzo, & Cassidy (2011) propose a mid-block opening with a pre signal (Figure 2-5) and if proper deterministic behaviour of the drivers are expected, it was found out that a similar capacity increase can be obtained just as the right turns have been banned in the intersection. However, the drivers should be well communicated and trained on the procedure to reduce the complexities that can arise, in order to harness the best results.

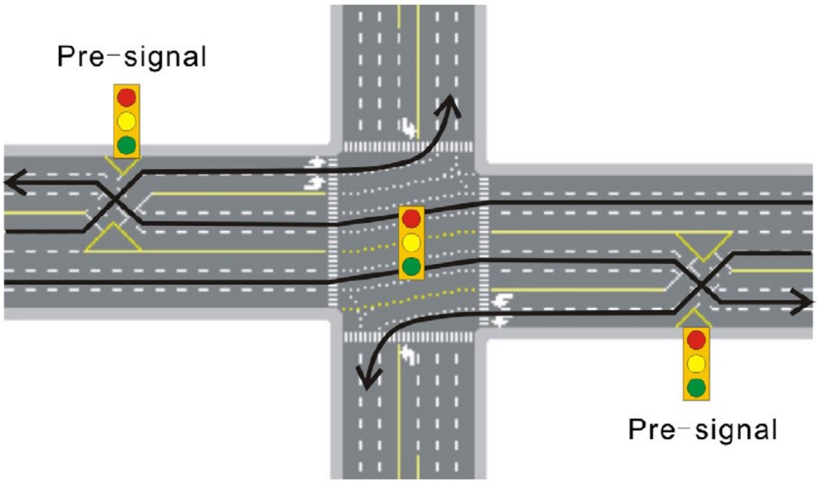


Figure 2-5: Pre-Signal

Source: (Xuan, Daganzo, & Cassidy, 2011)

2.7.3. Intersection control using cooperative vehicles

Traffic modelling becomes quite complex with the driver behaviours, intersection geometry, intersection clusters, etc. Therefore, most of the research that were conducted have considered isolated intersection, in order to reduce the effects of other intersections in close proximity (Ioslovich, Haddad, Gutman, & Mahalel, 2011).

Ahmane et al. (2013) proposes a new methodology to control the traffic at an intersection using vehicle equipped on-board units such as ITS stations (Figure 2-6) (Figure 2-7), which is similar to the research conducted by Sommer, German, & Dressler (2011) on Inter-Vehicle Communication. These devices would allow the driver of each vehicles to negotiate the "right of way" with the help of the measurements by the equipped ITS stations (Ahmane et al., 2013).

Figure 2-6: Isolated Intersection with cooperative vehicles

Source: (Ahmane et al., 2013)

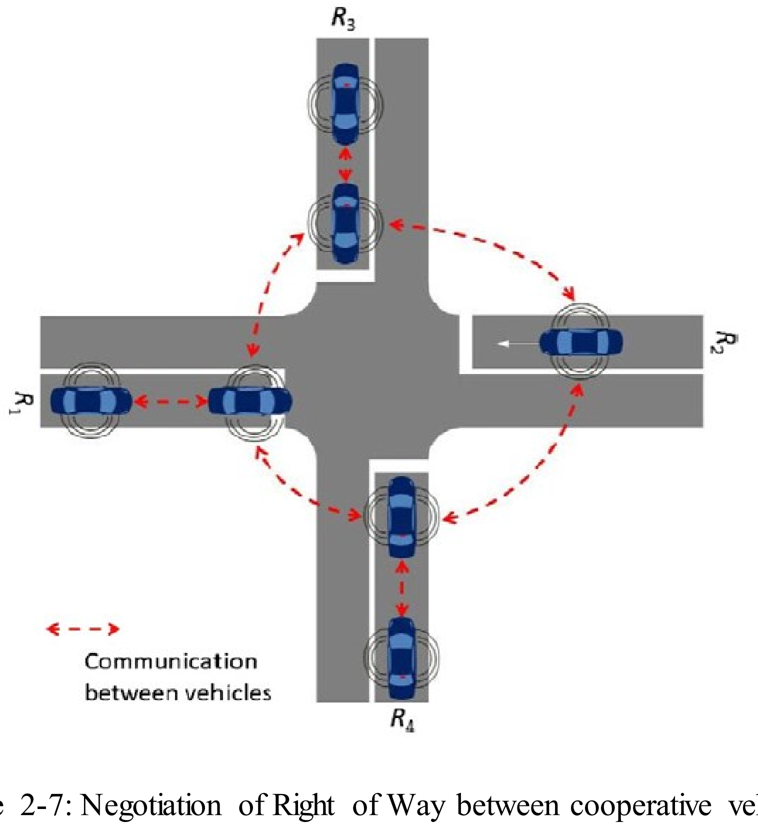


Figure 2-7: Negotiation of Right of Way between cooperative vehicles

Source: (Ahmane et al., 2013)

Similar research were also conducted on self-organized and self-control traffic control systems by Lämmer & Helbing (2008), Ferreira, Fernandes, Conceição, Viriyasitavat, & Tonguz (2010) and Sekiyama, Nakanishi, Takagawa, Higashi, & Fukuda (2001).

This would be quite important when considering the dynamic vehicle handling procedures and would provide a useful insight to the real time traffic management. However, the practicality of obtaining a similar isolated intersection would be questionable especially with the close proximity of intersections. Further, proper methodology should also be produced to deal with when the unequipped vehicles enter the intersection.

2.7.4. Separate midblock opening for U-turns

Midblock opening for U turns, is a newer addition in managing traffic congestion especially in order to address the negative impacts that U turns create at a signalized intersection.

Due to the U-turns, around 1.8% saturation flow rate decrease would be experienced by a 10% increase in U-turn percentage according to a research conducted at 14 signalized intersections (Carter, Hummer, Foyle, & Phillips, 2005).

Further, Al-Masaeid & Hashem (2008) mention that U-turns would increase the traffic congestion significantly since it involves a large gap with respect to the conflicting stream when it is performing a U-turn.

According to the Transportation Research Board (2014), 6 basic categories of mid-block openings were identified as indicated in Figure 2-8, Figure 2-9, Figure 2-10, Figure 2-11, Figure 2-12 and Figure 2-13.

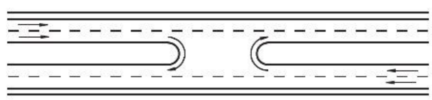


Figure 2-8: Conventional Mid-block opening without right turn lanes

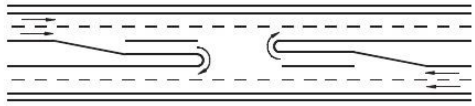


Figure 2-9: Conventional mid-block opening with right turn lanes

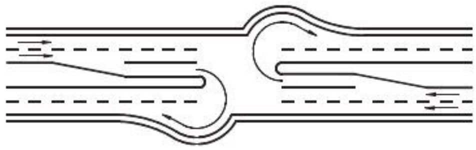


Figure 2-10: Conventional mid-block opening with right turn lanes and loons

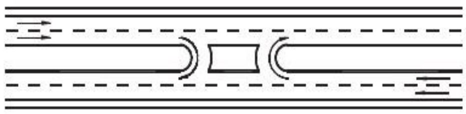


Figure 2-11: Directional mid-block opening without right turn lanes

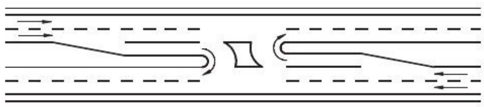


Figure 2-12: Directional mid-block opening with right turn lanes

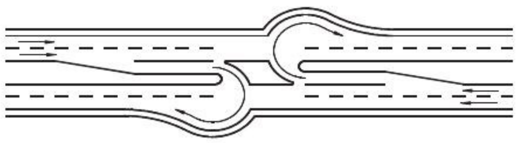


Figure 2-13: Directional mid-block opening with right turn lanes and loons

Estimation of capacity of mid-block openings is also an area where limited but useful research was carried out. According to Liu, Lu, & Chen (2008), in estimating capacity of mid-block openings in 6 lane streets, they have identified two important parameters, namely critical headway and follow up time. There they have identified for U-turns, the critical headway as 5.6s and follow up time was considered as 2.3s (Liu, Lu, & Chen, 2008).

Further in estimating capacity, it has been identified that the conflicting traffic volume for U-turn movements was equal to that of major direction traffic volume plus the major street right turn traffic volumes (Yu, Sulijoadikusumo, & Prevedouros, 2012).

Other than that they have also mentioned that the drivers taking U-turns in 6 lane streets, have more options to consider and also have higher turning speeds and thus it is more difficult to determine the conflicting traffic volumes (Yu, Sulijoadikusumo, & Prevedouros, 2012).

Further, the disturbance that a U-turn vehicle cause to the general traffic stream is analysed and a regression model was developed to identify the relationship between the percentage of vehicles taking U-turns and the average queue discharge time, which can be used for capacity estimations (Olarte, Bared, Sutherland, & Asokan, 2011).

Liu, Lu, & Cao (2009) has obtained that the weighted conflicting volume of traffic for U-turns on six-lane roads is equal to 2.2 times the average major road traffic volume in the opposing direction. It further mentions that the capacity estimation methods which are provided by the Highway Capacity Manual can be used for median openings in six lane roads.

When providing a separate opening for U turns the offset distance, which is the distance to the downstream intersection, is an important parameter.

Liu, Lu, Fan, Pernia, & Sokolow (2005) have identified that the minimum offset distance that needs to be provided for U-turn mid-block openings. For 4-lanes minimum offset distance was obtained as 400 feet (122 meters) and for 6 or more lanes that was obtained as 500 feet (152 meters). Further models were also developed incorporating the operational and safety issues in order to reduce the average delay for U-turns as well (Zhou, Hsu, Lu, & Wright, 2003).

With respect to the research conducted by Liu, Lu, & Chen (2008), with a 10% increase in the separation distance, 3.3% decrease in total crashes can be obtained when implementing the concept of right turns followed by U-turns as a substitute for the direct right turns.

Chen, Xu, & Liu (2013) mentioned that for effective operation the median nose width should be kept more than 6.4m, which would increase the potential capacity of the mid-term opening.

2.7.5. Right Turn Waiting areas

Another new initiative that is being brought forward is utilizing right turn waiting areas for capacity improvements at an intersection.

Yang, Liu, Chen, & Yu (2012) has researched on making use of the unutilized right turn waiting areas in order to improve the throughput of an intersection. Using calibrated VISSIM micro simulation software, by utilizing the right turn waiting areas a capacity increase of 17.82% can be obtained when compared with the unutilized situation, which is a significant achievement (Yang, Liu, Chen, & Yu, 2012).

As for the research conducted by Xi, ZhaoCheng, WenBo, ZhanQiu, & JunFeng (2013) waiting area concept which was only considered for right turns by Yang, Liu, Chen, & Yu (2012) has been extended even to through or straight vehicle waiting areas. Xi, ZhaoCheng, WenBo, ZhanQiu, & JunFeng (2013) conclude the research work with few interesting points after verification it in different traffic volumes.

By considering the queue lengths, link delays and link average speeds, Xi, ZhaoCheng, WenBo, ZhanQiu, & JunFeng (2013) conclude that Left Turn and through vehicle waiting areas can be highly effective in oversaturated condition, but will not necessarily generate the expected results in under saturated conditions due to driver psychological behaviour and the stochastic nature of vehicle arrivals.

2.8. Travel time savings valuations

It was also important to value the delay and travel time savings that has been generated through the implementation of a specific initiative. Existing studies on Value of Time (VOT) evaluation was reviewed in order to obtain the current value of time for Sri Lanka and other countries and also the tools and methodologies available to calculate the value of time based on different scenarios. Though number of literature have been reviewed, limited amount was identified which addresses the cases of Sri Lanka. The literature review would critically analyse the existing tools and methodologies available, considering the international context.

Number of Value of time studies were conducted considering the passenger trips for different geographical areas of the world using several models. In estimating commuters' value of time discrete choice models such as Disaggregate Binary choice models (Abdel-Aal, 2017), (Dhibi & Belkacem, 2013) and Nested Logit models (Lai & Bierlaire, 2015) were used relying on traveller survey data. Further, time-cost algorithms (Ding & Xu, 2015) and incident resolution time and value models (Domenichini, Fanfani, Bacchi, & Braccini, 2013) were also developed to quantify the value of time savings under different circumstances. Delay Fee models are also developed to quantify the costs resulting in a particular traffic congestion (Chimba, 2011).

At the same time, understanding the user behaviour, especially with mode or route choices, is an important aspect in value of time determination (Merkert & Beck, 2017) since this would govern the commercial viability of different initiatives also. Even when the tolled roads are available, understanding the cognitive behaviour of individuals does play a major role with the effects of willingness to pay for particular mode (Merkert & Beck, 2017).

Further, one of the dominant empirical approach when determining value of time is relying on experiments where the respondents are the major source of data extractions (Mouter & Chorus, 2016), where the travellers are expected to pay the travel costs from their own as an exchange to the expected travel time savings. Mouter & Chorus (2016) further argue that travel time savings obtaining from a government initiative can be valued more than that obtained from a person's own choice.

Value of travel time is also a function of time of the day, trip length, journey purpose, mode, Income and GDP (Wardman, Chintakayala, & de Jong, 2016). The variables such as income, trip length, work trips has a positive correlation with the value of travel time (Athira, Muneera, Krishnamurthy, & Anjaneyulu, 2016). Metz (2017) argues the need for assessing the effects of temporal variation of land use also in determining value of travel time, referring to the United Kingdom policy objectives in valuing transport investments.

However, a typical uncertainty is paramount with the estimation and appraisal of value of time and it is desirable to continuously update the value of time in order to avoid excessive temporal variations in VOT (Wheat & Batley, 2015).

2.8.1. Value of time for Sri Lanka

With the boost experienced in the construction industry after 2010, number of mega scale transportation projects have been evolved in Sri Lanka, where the value of time estimate was of utmost importance. However, limited number of studies has been conducted, focussing on calculating the value of time for the Sri Lankan context. The recent most study conducted by the Ministry of National Planning gives the following values given in Table 2-1 (Kumarage, Gunaruwan, Storm, Ranawana, & Mudannayake, 2000).

Table 2-1: Value of Time for Sri Lanka

User Group	Value of time in 1999 (Rs. /Hour)
Car	106.5
Van	48.44
Motor Cycle	19.22
Public Transport	11.62
Non-Motorized modes	7.39
All motorized modes	25.55

According to Kumarage, Gunaruwan, Storm, Ranawana, & Mudannayake (2000), the value of time of people for Sri Lanka considering the all motorized vehicle users is Rs.25 per hour in 1999. In order to estimate the value of time in the future years, the study has proposed to use the Colombo Consumer Price Index (CCPI). The above values cannot be used any more, even though they are adjusted with CCPI, where most of the values are under estimated for today's context.

2.8.2. Methods used to calculate the value of times

According to literature a number of methods are used in calculating value of time savings. The commonly used four methods are identified below.

- Wage rate or cost savings model
- Hensher model

- Willingness to pay analysis
- Gross Domestic Product

The methods mentioned would be discussed in detail.

- **Wage Rate Model or cost savings model**

Wage rate or cost savings model is mainly based on the theory of marginal productivity (Victoria Transport Policy Institute, 2013). It suggests that the value of time of work based trips is the wage rate plus the overheads associated with the work. Number of research have been conducted to obtain the value of time of the work based trips focussing on the wage rate or cost savings model.

Transfund, New Zealand (Victoria Transport Policy Institute, 2013) uses the following base values for transportation evaluations using 1998 as the base year, which are indicated in Table 2-2.

Table 2-2: Value of times for work based and non-work based trips (in NZ dollars per hour)

Mode	**Car users**	**Light goods vehicles**	**Medium Goods vehicles**	**Heavy Goods Vehicles**	**Bus passengers**	**Pedestrian/ cyclist**
Work Based	21.3	19.3	15.8	15.8	21.3	21.3
Non work based	7	7	5.3	5.3	10.6	10.6

Börjesson & Eliasson (2012) mention that the value of time of cyclists is estimated as €16 ($17.9) per hour for those who are using streets and €11($12.3) per hour for those who are using bike lanes. It is also essential to mention that the respondents have also valued the additional benefits of cycling as well, when determining the value of time.

Belosic (2015) found out the tools and the methodologies available to calculate the value of time. It further states that the value of time with respect to business can be calculated using the Equation (2-2).

$$\text{Value of Time} = \frac{(\text{Annual salary} + \text{monetary value of benefits})}{\text{Number of real hours worked in a year}} \qquad (2\text{-}2)$$

The value of time for France has been obtained using income based approach and it has been categorized with respect to the purpose of the trips. In that research, professional trips are valued at $17.5 per hour, home-work based trips at $10.0 per hour and other trips are valued at $6.8 per hour (Meunier & Quinet, 2015).

However this model assumes the full savings are transferred to work based trips which might not be reliable and accurate. Further, the method values the in-work time savings based on wage rate plus overhead costs which would make it more complicated for a country like Sri Lanka, with various overhead costs being available.

- **Hensher Model**

Hensher model is mainly focussed on modifying the value of times obtained through number of methodologies with the use of factors which might affect the value of time (Hensher & Goodwin, 2004). For example, the cost savings approach transfer the full time savings for money, but there can be occasions where the travel time may be utilized for leisure activities. The Hensher method focuses on such situations, in order to reduce the value of travel time for those instances (Hensher & Goodwin, 2004).

Further, Hensher method accounts for the travel time utilized for productive activities, where the cost savings approach does not account for that (Hensher & Goodwin, 2004).

The main limitation of Hensher Model is the lack of research available about the Hensher model, due to the unavailability of the required detailed data. For example, most of the questionnaire surveys and other types of surveys were mainly based on specific information for that particular survey, but very little research was there focusing on the traveling for leisure activities, since transportation is mainly considered as a negative attribute. Thus a more effective and practical methodology is desirable.

- **Willingness to pay analysis**

The main aim of conducting a willingness to pay analysis is to obtain the price a person is willing to pay for using a particular transportation route/mode and in turn obtain a value of time.

"Willingness to pay analysis" has also been used as a tool for calculating the value of time in other countries specially in calculating the value of time for work based and non-work based trips. However, in most research, it is argued that the estimates for value of times which are arrived trough willingness to pay analysis do not reflect the true value of time. Merkert & Beck (2017) argue that time is a scarce resource, but direct use of willingness to pay values for social appraisal is inappropriate. Therefore it can be argued that estimates obtained under willingness to pay analysis do not reflect the work based trips VOT but the non-work based trips VOT.

Victoria Transport Policy Institute (2013) states that that the value of time of people at New Jersey would be varying from US $ 10 -40 per hour in the morning peak hour, which is calculated based on the willingness to pay analysis data.

- **Gross Domestic Product**

Gross domestic product has been used as a tool in calculating value of time for other countries. Therefore, as part of the literature review the general values for GDP of Sri Lanka and the growth has been considered in Table 2-3.

Table 2-3: Gross Domestic Product of Sri Lanka

Source: (Central Bank of Sri Lanka, 2014) (Central Bank of Sri Lanka, 2012)

Year	**2010**	**2011**	**2012**	**2013**	**2014**
GDP (At current prices)	5,604,104	6,543,313	7,578,554	8,674,230	9,784,672
GDP (2002 Constant prices)	2,645,542	2,863,691	3,045,288	3,266,041	3,506,664
Growth(Based on constant prices)		8.25%	6.34%	7.25%	7.37%

It is argued that the above GDP based method can be used for a general understanding a country's value of time without a detailed investigation.

2.9. Modern Technology involvement

Use of ontological engineering systems for traffic light control (Keyarsalan & Ali Montazer, 2011), store and forward based methods (Wilkins, 2015), agent technology (Chen & Cheng, 2010) and second order traffic modelling (Ngoduy & Maher, 2012) are some of the key areas in the development of traffic signal designs where newest technology are involved.

Store and forward based methods are common in telecommunication industry, where the data is sent to an intermediate location and then it is stored there and disseminate once the information is available (Wilkins, 2015). These store and forward based methods can even be utilised for traffic designing work especially in congested large scale road networks (Aboudolas, Papageorgiou, & Kosmatopoulos, 2009). These models can even be used for comparative efficiency checks as well as for real time feasibility signal control methods (Aboudolas, Papageorgiou, & Kosmatopoulos, 2009).

2.9.1. Intelligent Transportation Systems

Further, Intelligent Transportation Systems (ITS) which is the latest addition to traffic engineering field, can produce well informed and safer road network and system by providing innovative services for different modes of transport.

Many countries are moving towards intelligent transportation systems, where China considers ITS as one area out of the 12 key areas for Research and Development in China (Wang, Tang, Sui, & Wang, 2003). Over the decade, the highway system of China has grown rapidly to 25,000 kilometres from virtually nothing at the beginning (Wang, Tang, Sui, & Wang, 2003). As a result a 15% annual vehicle growth is expected (Wang, Tang, Sui, & Wang, 2003) and proper ITS application is vital for the sustainability of transportation systems.

The traffic engineering field has been developing through various research that have been carried out in order to accurately model the traffic and vehicle behaviour. Most of the traffic models, that have been developed and used are first order traffic models (Lebacque, Lesort, & Giorgi, 1998), where an equilibrium speed density relationship is assumed, such as Lighthill, Whitham, Richards Traffic model (Ngoduy & Maher, 2012).

The newest research is moving towards developing second order traffic models, in order to account for the non-linear traffic, such as phantom traffic jams and traffic instabilities (Ngoduy & Maher, 2012) and predict more reliable and accurate results.

2.9.2. Agent Technology

The use of agent technology for traffic management and transportation systems has been another area the researchers are interested in, where the behaviour and the dynamic nature of traffic suits the application of agent technology (Chen & Cheng, 2010). Especially in traffic signal coordination, the separate instruments can be modelled as intelligent agents, allowing these modelled agents handle the criteria for coordination automatically (van Katwijk & van Koningsbruggen, 2002).

Further, for control of traffic signal and coordination, multi-agent reinforcement learnings (Bazzan, de Oliveira, & da Silva, 2010), can also be used either to operate individually or in a guided manner.

Autonomous intelligent agents which can adapt themselves intelligently are also used for urban traffic control systems (Roozemond, 2001).

2.9.3. Distributed approach

When considering the urban traffic chaos which are experienced in most of the developing and developed countries, it is required to introduce innovative strategies of traffic management (Bazzan, 2005).

The current practice in traffic signal coordination is to have a central traffic control and responsive system and do the adjustments as necessary. However, with the ever increasing technology, centralization would no longer effective (Porche, Sampath, Sengupta, Chen, & Lafortune, 1996), whereas the techniques involving functional as well as spatial decentralization (Bazzan, 2005), phase synchronization (Lämmer, Kori, Peters, & Helbing, 2006), fuzzy traffic controlling (Lee & Lee-Kwang, 1999) are vital. Techniques such as distributed artificial intelligence including the multi agent systems need to be developed for the sustainability of modern traffic signal installations. However, various difficulties may come across in a decentralized approach, since there is lack of decision support strategies and systems which are capable of dealing with decentralized management opposed to the centralized system of work (Bazzan, 2005).

2.9.4. Other methods

Putha & Quadrifoglio (2010) mentions that for traffic signal coordination purposes new probabilistic methods like Ant Colony Optimization techniques can also be used which can simplify the complex computational problems into simple steps using graphs. These techniques can create more reliable and accurate results which are also verified through field inspections (Putha & Quadrifoglio, 2010).

Wang, Cottrell, & Mu (2005) also propose new techniques for traffic signal designs such as K-means method. Time-of-day Breakpoints which is necessary for traffic signal timing plans can be determined using K-means method (Wang, Cottrell, & Mu, 2005). Further, this method is a non-hierarchical clustering method, which requires lower space for data storage opposed to the hierarchical clustering based techniques (Wang, Cottrell, & Mu, 2005).

2.10. Traffic Engineering Software Usage

Microscopic simulation has been quite instrumental in predicting reliable and accurate results in the domain of traffic and transportation engineering field including the evaluation of intelligent transportation system strategies (Chu, Liu, & Recker, 2004).

2.10.1. Usage of microscopic simulation software

Microscopic traffic simulation studies have been quite popular in predicting reliable and accurate results in the domain of traffic and transportation engineering, including the evaluation of intelligent transportation system strategies (Chu, Liu, & Recker, 2004).

Handful of microscopic simulation software have been used for various research and practical implementations.

Jinpeng & Zhang (2012) and Zhang et al. (2009) used VISSIM software for traffic simulations for emission testing, while Yang, Liu, Chen, & Yu (2012) utilized VISSIM for determining the effectiveness of left turn waiting areas. Usage of VISSIM has reached even for public transportation operations as well where Wahlstedt (2011) used VISSIM for bus priority signals.

Other micro simulation software, TRANSYT-7F and Synchro were also extensively used by CHEN, XU, & LIU (2013) and Liu & Chang (2011) for optimization of traffic signals. Sidra software (Hung, 2014) was used for traffic signal coordination and PARAMICS software (Yang, Liu, Chen, & Yu, 2012) for determination of effectiveness of waiting areas at signalized intersections.

2.10.2. Parameter Identifications

In VISSIM microscopic simulation software, there are around 40 parameters which can be changed for modelling the driver behavior patterns.

Park, Won, & Yun (2006) has identified 14 calibrated parameters based on the engineering judgement and the respective calibrated values were proposed.

For the context of India, the following VISSIM parameters were identified for calibration purposes (Siddharth & Ramadurai, 2013).

- Average Standstill Distance
- Additive Part of Safety Distance
- Multiplicative Part of Safety Distance
- Look Ahead Distance
- Look Back Distance
- Waiting time before diffusion
- Minimum headway (front/rear)
- Safety Distance Reduction Factor
- Distance Standing (at 0 km/h)
- Distance driving (at 50 km/h)
- Desired position at free flow

Referring to the previous literature, the main parameters used for VISSIM calibration were more or less the same (Zhizhou, Jian, & Xiaoguang, 2005), (Lownes & Machemehl, 2006), (Rrecaj & Bombol, 2015), as proposed by Siddharth & Ramadurai (2013) but the values might differ for the respective local conditions.

2.10.3. Objective functions and calibration measures

For calibration purposes of VISSIM traffic simulation software, literature identifies several key measures being used, consisting with single or multi-parameter calibrations. Park, Won, & Yun (2006) has used travel time as the calibration measure, where the difference with the obtained travel time and simulated travel time was compared.

Opposed to using travel time alone, travel time distribution was also used as objective function for optimizing calibrating parameters, where the closeness of the observed and simulated travel time variation was considered (Kim, Kim, & Rilett, 2005)

In another calibration study, saturation flow rate and queue discharge headway were used for the calibration measures in obtaining optimum values for calibrated parameters (Rrecaj & Bombol, 2015).

Further to this, minimizing the error between observed and simulated queue length has been considered as the objective functions, in most of the research due to the simplicity and practicality of obtaining data (Rrecaj & Bombol, 2015).

Further, maximum flow rate, capacity, speed and delay measurements are also used for calibration purposes as objective functions (Rrecaj & Bombol, 2015).

Calibration Processes

In most of the scenarios, calibration parameters are identified using sensitivity analysis techniques and sampling purposes Latin Hypercube sampling technique is used (Park, Won, & Yun, 2006). The optimum parameters were selected, so that the minimum error between the observed behavior and simulated behavior is reached ((Mathew & Radhakrishnan, 2010).

In calibration processes, most of the researches first target to obtain the discrepancy between the simulated results and the field results by using the default parameters in VISSIM (Manjunatha, Vortisch, & Mathew, 2012) to get an understanding about the level of calibration needed.

Statistical methods in terms of parametric and non-parametric techniques are used for calibration purposes (Rrecaj & Bombol, 2015).

Intelligence Decision Support System, is used for the model calibration purposes taking the calibration as a decision making process (Huang, Sun, Wang, & Li, 2008). Further, for pattern recognition purposes of a traffic simulation model, neural networks are used. Optimum values for the parameters are obtained through genetic algorithm (Huang, Sun, Wang, & Li, 2008) where Genetic algorithms are considered as a heuristic approach for VISSIM calibration purposes (Rrecaj & Bombol, 2015).

Further to these, Brockfeld, Kühne, & Wagner (2004) has tested the models with Differential Global Positioning Systems equipped cars, where the data of the leading car is used to determine the headway with respect to the following car. The deviation between the observed and the simulated headways are used for the purpose of calibration and validation. Calibration process was able to produce results with the errors of 12% to 17% and for validation process the errors were in the range of 17% to 22% (Brockfeld, Kühne, & Wagner, 2004).

Further for parameter consistency measurements, various tests such as F Test, Chi-square Test, ANOVA, Pattern recognition, Wilcoxon Test and T-test are used in various occasions (Rrecaj & Bombol, 2015)

2.10.4. Validation Process

With the obtained parameters validations need to be carried out to check the accuracy of the optimum parameters under different traffic conditions so that the calibrated parameters, represent actual condition when simulated. Most of the research have focused on validating based on signalized intersections with differing traffic flows and mix (Park, Won, & Yun, 2006). The main parameters used for validation purposes were queue length and travel time (Park, Won, & Yun, 2006).

Further, Highway Capacity Manual (HCM) procedure combined with field data is also used for validation purposes (Park & Qi, 2006).

2.10.5. Sample parameters used for other countries

In order to obtain a good understanding, the calibrated parameters used in India, Indonesia and USA are mentioned along with the default values in Table 2-4 (Loisiana Department of Transportation, 2010) (Siddharth & Ramadurai, 2013) (Soelistyopribadi, Munawar, & Sitimalkhamah, 2016).

Table 2-4: Calibrated VISSIM Parameter values for different countries

Parameter	Default Value	India	Indonesia	USA
Average Standstill distance	2	1	0.5	1 to 2
Additive Part of safety distance	2	0.2	0.5	2.1
Multiplicative part of safety distance	3	0.78	1	2.9
Waiting time before diffusion	60s	60s	60s	60s
Min. headway (front/rear)	0.6	0.11	-	-
Distance standing (0kmph)	1.25	0.62	-	-
Look Back Distance	150	14.31	120	-
Look ahead Distance	250	27.91	120	-

As mentioned in the literature review, several key areas of literature gaps are identified and the methodology of the research is formulated accordingly.

CHAPTER 3 : METHODS, ANALYSIS AND RESULTS

Based on the literature review which is carried out, important findings were observed which had been very crucial in formulating the methodology and for the progress of the research. The main outcomes from the literature review would be discussed in brief.

With reference to literature, since most of the research in the field of traffic signals and traffic signal coordination has used traffic micro simulation software such as VISSIM, Synchro, Transit 7-F, Sidra, etc. It is vital to utilize a properly calibrated software for the traffic conditions. This software needs to be calibrated and validated in order to account for differences in traffic mix, peak-off peak variations, pedestrian behaviour and driver behaviour.

Literature further suggests when developing guidelines for traffic signal coordination, it is necessary to start off with the individual intersections first. It is always better to first address the individual intersections and the differences in geometry, signal timings, and signal phasing and pedestrian movements.

According to the findings, it is also vital to address the effects in traffic signals in fixed time as well as in real time. Focussing only on fixed time would not be beneficial in the long run, because the world is now moving towards the real time information with advancement of the technology and the complexity of road, traffic and driver behaviour.

When considering the individual traffic signals, proper optimization objective needs to be considered. The literature suggests to use few objectives such as throughput maximization, delay minimization and queue length minimization, where achieving one of the above objectives would necessarily impact on the other objective also in a favourable manner.

According to literature, in real time traffic management, the main criteria that should be considered are the maximum and minimum green to be provided for a particular phase if continuous vehicles are detected.

New initiatives are always encouraged and proper guidelines should be established also to utilize the ever developing technology in the traffic engineering field so that a combined and an effective outcome is generated.

In traffic signal coordination, optimization objectives need to be considered such as maximization of bandwidth, minimization of delay and optimization of offsets. Most important consideration is the negative impacts of traffic signal coordination despite the various benefits that can be achieved through proper coordination. The negative impacts highlighted are the adverse effects for the minor roads due to increased delay and waiting time and the possibility of red light running crashes to stay in the green platoon when traffic signal coordination.

Having a good understanding on the existing knowledge with referring to the literature, the methodology of the research is structured into two main categories as mentioned below in order to achieve the objectives which are laid at the beginning of the research.

A. Establishing Preliminary Requirements

1. Calibration of the VISSIM traffic micro simulation software
2. Verification of the calibrated software for practical applications
3. Quantification of travel time savings and updating the Value of Time for evaluation of transportation projects

B. Traffic Signal Design and Coordination

Stand Alone Traffic Signals

4. Development of guidelines for Individual intersections for fixed time systems
5. Development of guidelines for Individual intersections for real time systems.

Traffic Signal Coordination

6. Identification of the traffic signal coordination potential for intersections
7. Development of guidelines for traffic signal coordination for non-similar intersection clusters

A. Establishing Preliminary Requirements

3.1. Calibration of the VISSIM traffic micro simulation software

As for the findings of the literature review, it is argued that the newest trend and the future for traffic modelling is through traffic micro simulation. With the development of the technology various software are brought forward with advance features to model the real traffic behaviour.

Modelling traffic behaviour itself is a very complicated task, due to the cognitive and dynamic behaviour of individual minds which is further complicated when it comes to the road with the thousands of vehicles experiencing. Therefore with the newest technology being available, it is necessary to produce reliable and accurate software for traffic simulation purposes.

Further, when the new initiatives and ideas are brought forward, number of practical difficulties arise when applying it directly to the practical scenarios being considered. User frustration is a major issue, that most of the developing countries are facing which halts them from progression as a nation due to number of trial and error procedures being introduced which do not show any signs of sustainability.

Further to this, the practical difficulties in obtaining the expected results such as delays and queue lengths in repetitive nature is another reason for moving into traffic simulation software.

Blindly using a software can create unreliable results leading to failures in decision making and implementation. Therefore, the first step of the methodology was developed to calibrate the VISSIM traffic micro simulation software for the local condition with the differences in traffic mix, driver behaviour patterns, peak-off peak variations and pedestrian behaviour.

In order to do so, the traffic data was collected at two signalized intersections, namely Katubedda Junction (Figure 3-1) and Kesbewa Junction (Figure 3-2), through video surveys. From the video surveys the traffic volumes along with the turning movements, vehicle mix and queue lengths were obtained analysing the records available. The observed features of the intersections and their behaviour has been modelled using “VISSIM”, which is a traffic simulation software, providing a total solution for traffic management.

As for the literature, few calibration parameters such as the priority rules, following distances, lane change behaviour, lane occupancy behaviour, reduced speed areas have been used for calibrating VISSIM in order to suit for the Sri Lankan condition.

The verification of the modelling was done using the “Queue Length”, because queue length is the most practical and accurate enough parameter for model calibration and validation purposes. The observed queue length in the existing condition has been compared with the queue lengths obtained from the VISSIM simulation for both the intersections. The results were obtained and the conclusions are provided along with the future potential research areas to be explored.

The methodology is structured into two main sub categories, calibration and validation.

3.1.1. Calibration

The first step of microscopic simulation modelling is to calibrate the software to identify the optimum parameters which would represent the actual condition.

3.1.1.1. Parameter Identification

As for the research conducted by (Brian) Park & [Maggie] Qi (2005), Siddharth & Ramadurai (2013), Loisiana Department of Transportation (2010) and (Soelistyopribadi, Munawar, & Sitimalkhamah (2016), 9 common parameters have been identified out of more than 30 parameters, to be used for the calibration purposes as mentioned.

1. Average Standstill Distance
2. Additive Part of Safety Distance
3. Multiplicative Part of Safety Distance
4. Waiting time before diffusion
5. Minimum headway (front/rear)

6. Safety Distance Reduction Factor
7. Distance Standing (at 0 kmph)
8. Distance driving (at 50 kmph)
9. Desired position at free flow

3.1.1.2. Parameter range

Since it was time consuming and complicated to do the simulations for all 9 parameters, a statistical analysis is conducted to identify the most sensitive parameters for the calibration processes.

Out of these 9 parameters, last parameter, which is "Desired position at free flow" was not used for simulations due to the obvious driver behavior pattern of Sri Lanka, which is not lane based.

Obtaining the minimum and maximum values for the remaining 8 parameters were done by referring to the calibration parameter values obtained in similar countries referring to Siddharth & Ramadurai (2013), Loisiana Department of Transportation (2010) and (Oelistyopribadi, Munawar, & Sitimalkhamah (2016), as mentioned in Table 3-1.

Table 3-1: Parameter Range

No.	Parameter	Default	Min	Max
1	Average Standstill distance	2	1	2.5
2	Additive Part of safety distance	2	1	2.5
3	Multiplicative part of safety distance	3	1	4
4	Waiting time before diffusion	60	30	75
5	Min. headway (front/rear)	0.5	0.3	0.6
6	Safety distance reduction factor	0.6	0.4	0.7
7	Distance standing (0kmph)	1m	0.5	1.25
8	Distance driving (50kmph)	1m	0.5	1.25

Since simulating all combinations for all the values is time consuming, randomly obtained 32 parameter sets were selected to be simulated, in order to have a considerable sample.

3.1.1.3. Developing an intersection in VISSIM

For simulation and calibration purposes, Katubedda Junction is considered.

A 30 minute video survey along with manual classified count were conducted in the morning peak hours during 7am to 7.30am and the data obtained are indicated. From the video surveys the traffic volumes along with the turning movements, vehicle mix and queue lengths were also obtained through the recordings and field measurements.

The observed features of the intersections, traffic data (Figure 3-1) and traffic signal timings (Table 3-2) were used to model the intersection using simulation software.

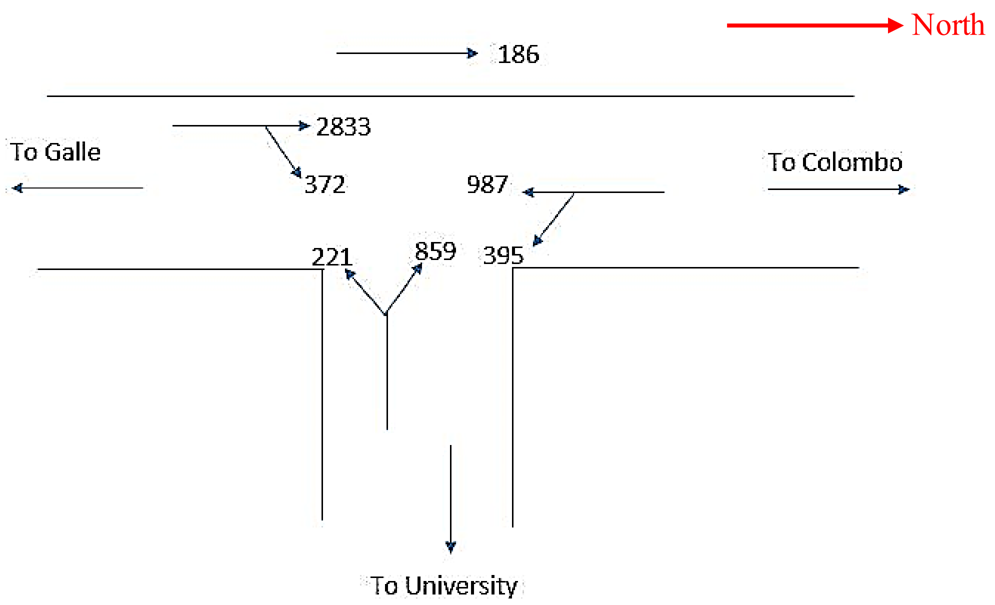
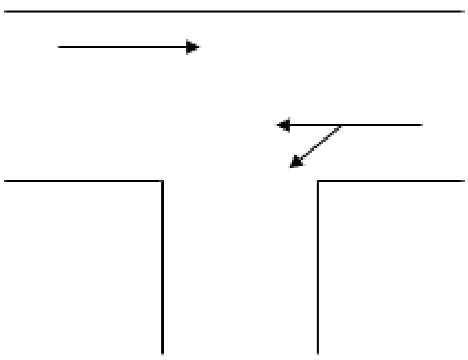
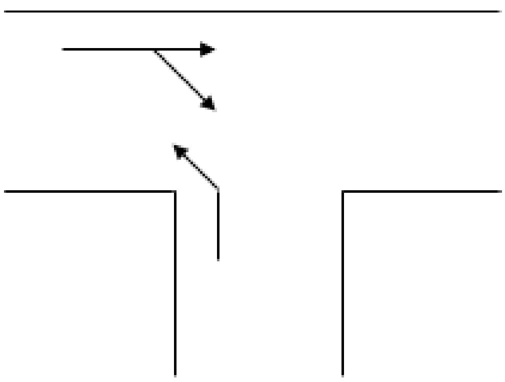
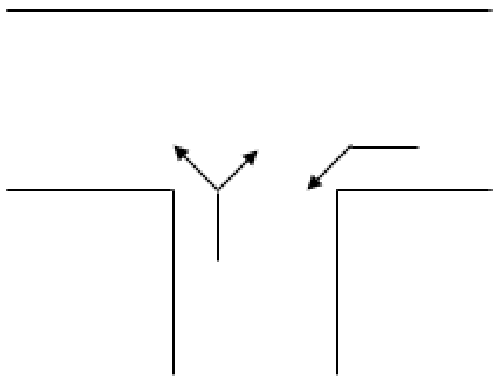


Figure 3-1: Flow at Katubedda Junction (Veh/Hour)

Table 3-2: Traffic signal phases and timings

Phase	Signal Timing of the Phase (In seconds)
Phase 1 	36
Phase 2 	16

Phase 3 	28

The desired speed for each vehicle type used is also mentioned in Table 3-3, which are similar to the default values suggested in VISSIM and research done by Siddharth & Ramadurai (2013), Loisiana Department of Transportation (2010) and (Soelistyopribadi, Munawar, & Sitimalkhamah (2016).

Table 3-3: Desired speeds for vehicles

Vehicle Type	Desired Speed (km/h)
Car	40
Motor Bike/Three Wheel	30
Heavy Goods Vehicle	25
Bus	30

3.1.1.4. Correlation identification

For calibration purposes the few parameters are identified as mentioned.

- Traffic flow
- Delay
- Travel Time
- Queue Length

Using traffic flow as a calibration measure will not work fundamentally, since VISSIM is for traffic micro simulation software and not a macro simulation software such as CUBE or GIS. Therefore, using traffic flow would not be accurate and can generate high errors.

Delay can be a good parameter for calibration purposes, but measuring delay in the practical real life context is rather impossible though that can be easily obtained in VISSIM software.

So, only viable parameters for calibration would be travel time and queue length. But then again, obtaining travel time in the practical real life context in a micro simulation environment would be rather impossible and subject to high level of errors.

Therefore, only reliable parameter for micro simulation software calibration is using observed real life queue length and VISSIM software queue length.

As described, the calibration of the modelling was done using the "Queue Length" as the objective function also referring to the inputs from ((Brian) Park & [Maggie] Qi, 2005) & (Rrecaj & Bombol, 2015). Further, the main justification for using the queue length as the objective function is the practicality and accuracy of obtaining data as opposed to obtaining vehicle or passenger delays. The observed queue length in the existing condition was compared with the queue lengths obtained from the VISSIM simulation for Katubedda Junction.

Based on the 32 sets of parameters, generated through judgmental sampling from the values in Table 3-1, the simulations were conducted and the difference in simulated queue length with actual queue length was compared and a percentage error is obtained.

These error percentage of observed and queue lengths along with the respective parameter values were considered in order to identify the most sensitive parameters for the queue lengths and Pearson correlation coefficient is obtained, where the results are mentioned (Table 3-4).

Table 3-4: Correlation and Significance Values

Parameter No.	Pearson Correlation	Significance (2-tailed)	Sample size
1. Average Standstill distance	0.891	0.000	32
2. Additive Part of safety distance	0.560	0.001	
3. Multiplicative part of safety distance	0.560	0.001	
4. Waiting time before diffusion	0.003	0.985	
5. Min. headway (front/rear)	-0.002	0.990	
6. Safety distance reduction factor	0.003	0.985	
7. Distance standing (0kmph)	0.891	0.000	
8. Distance driving (50kmph)	0.318	0.076	

Based on the results of both the Pearson correlation coefficient and 10% significant level, the following 5 parameters were identified as the most sensitive parameters for calibration purposes and they were selected for identifying the optimum calibration values.

- No.1: Average Standstill Distance
- No. 2: Additive Part of Safety Distance
- No. 3: Multiplicative Part of Safety Distance
- No. 7: Distance Standing (at 0 kmph)
- No.8: Distance driving (at 50 kmph)

3.1.1.5. Obtaining the optimum calibration parameters

Considering the identified 5 parameters and the previously obtained percentage errors, parameter values were changed and the simulation tests are conducted till the best parameter set with the least error values are obtained based on the conducted simulations (Table 3-5).

Further, GEH statistic (G_H) for turning movements are also developed for the optimum calibration values for calibration purposes as in Equation 3-1.

$$G_H = \sqrt{\frac{2(m-c)^2}{m+c}} \qquad (3\text{-}1)$$

where, m refers to the model traffic flow in veh/h

c refers to the real traffic flow in veh/h

Table 3-5: Simulated and Observed Queue Length

Direction		**Queue Length (m)**		**GEH Statistic**	**% Error**
		Observed	***VISSIM***		
From Colombo (North)	**To Galle (South)**	35	36	1.12	4%
	To University (East)	14	15	1.43	8%
From Galle (South)	**To Colombo (North)**	48	51	2.28	7%
	To University (East)	20	18	0.84	-6%
From University (East)	**To Colombo (North)**	75	78	1.22	4%
	To Galle (South)	25	26	1.66	5%

The above minimum error values were obtained for the following calibration parameter values (Table 3-6).

Table 3-6: Optimum Calibrated Parameters

Parameter	Value
Average Standstill distance	1.5
Additive Part of safety distance	1.5
Multiplicative part of safety distance	2
Distance standing (0kmph)	0.5
Distance driving (50kmph)	0.75

Further, the GEH values calculated for all the turning movements are less than 5, which suggests that the obtained calibration model parameters are valid.

3.1.2. Validation

The calibrated parameters were crucial to be validated for different traffic conditions.

Kesbewa Junction (Figure 3-2) (Figure 3-3), which is a signalized four leg junction, was considered as the initial intersection for validation

The profiling validation was done using one hour survey data obtained through the video surveys and manual classified counts and the data used are mentioned in Figure 3-2 & Figure 3-3.

For profiling validation purposes, the observed queue length in the site was compared with queue length obtained through VISSIM traffic micro simulation software.

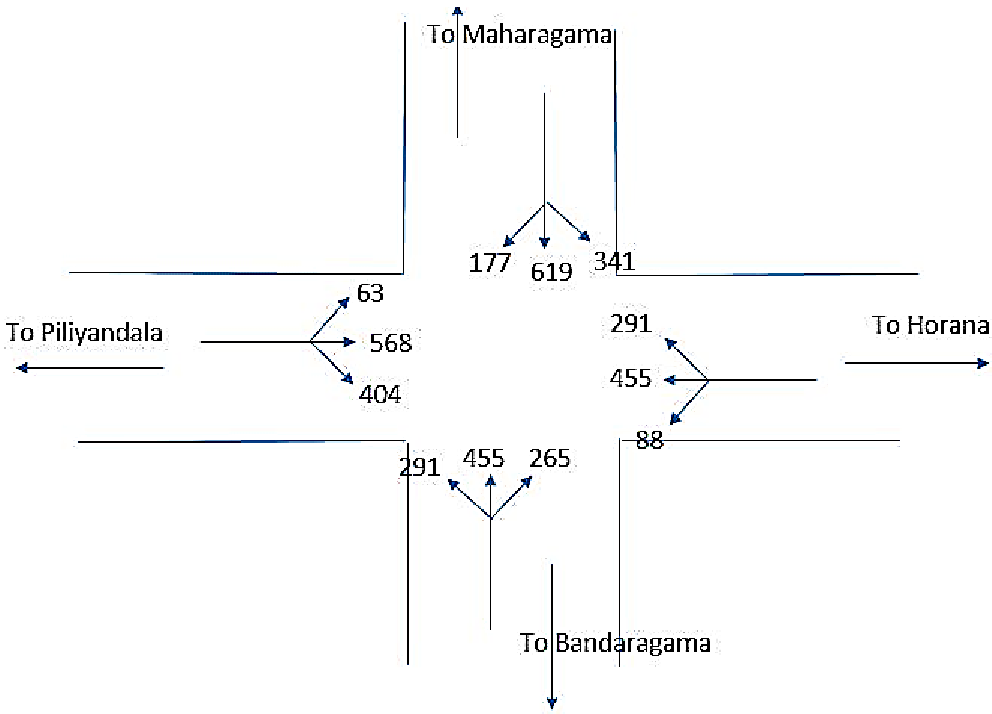


Figure 3-2: Flow at Kesbewa Intersection

Figure 3-3: Modelling Kesbewa Intersection

The following results were obtained from simulations for one hour time period at Kesbewa Intersection (Table 3-7).

Table 3-7: VISSIM and Observed Queue Length

Direction		**Queue Length (m)**		**% Error**
		Observed	**VISSIM**	
From Horana (3)	To Piliyandala (1)	25	27	7%
	To Maharagama (2)	20	22	10%
From Maharagama (2)	To Horana (3)	28	25	-12%
	To Bandaragama (4)	25	30	21%
From Piliyandala (1)	To Horana (3)	25	21	-15%
From Bandaragama (4)	To Maharagama (2)	95	82	-14%
	To Piliyandala (1)	5	4	-21%

The arithmetic mean percentage error obtained was 14%.

The validation was further conducted for 22 more intersections in the districts of Colombo and Gampaha (See Appendix A) to clearly identify the variation in traffic flow over the intersections and the mean percentage error values were obtained (Table 3-8).

Table 3-8: Error Percentages in Validation

No.	Intersection	Arithmetic Mean Error
1	Orugodawatta Junction	7%
2	Rajagiriya Junction	10%
3	Nawala Road Junction	13%
4	Ayurvedic Junction	4%
5	Pamankada Junction	6%
6	Peliyagoda Junction	15%
7	A1 Connection of expressway entrance road near Peliyagoda	14%
8	A3 Connection of expressway entrance road near Peliyagoda	15%
9	Ingurukade Junction	12%
10	Bandaranaike Round about	12%
11	Nawaloka Roundabout	2%
12	Kadawata A1 Junction	3%
13	Shramadana Mw. Junction	5%
14	Kadawata Expressway Exit	6%
15	Kadawata Expressway Entry	7%
16	Bandarawatta Junction	8%
17	Eldeniya Junction	14%
18	AAT Junction	13%
19	Kirimandala Junction	3%
20	Narahenpita Junction	1%
21	Apollo Junction	2%
22	Park Road Junction	8%

Based on the mean percentage values obtained for the above 22 intersections considered (See Appendix A), the error obtained for all the intersections were less than 15%, where the acceptable values according to literature should be less than 22% as in Brockfeld, Kühne, & Wagner (2004). Therefore, this successfully validates the values for calibrated parameters and the parameters in Table 3-6 can be used for the progress of the research.

3.2. Verification of the calibrated software for practical applications

The calibrated software needs to be verified for the use of various practical applications. In developing guidelines for traffic signals and traffic signal coordination, it is important to have a properly calibrated and validated software in hand.

For the verification and application purposes of the calibrated VISSIM traffic micro simulation software, a study on assessing the suitability of providing separate openings for U turns was considered. Orugodawatta Junction, which is a signalized intersection consisting of 4 legs was considered as a case study for calibrated VISSIM software application. This was very crucial for the ongoing of the research to determine the obstacles for using the software in practical applications.

Therefore, traffic flow data for complete one day, along with the existing traffic signal system and the geometrical details were obtained at the Orugodawatta junction (Figure 3-4), where high amount of U-turns are observed from the vehicles coming from the Borella direction.

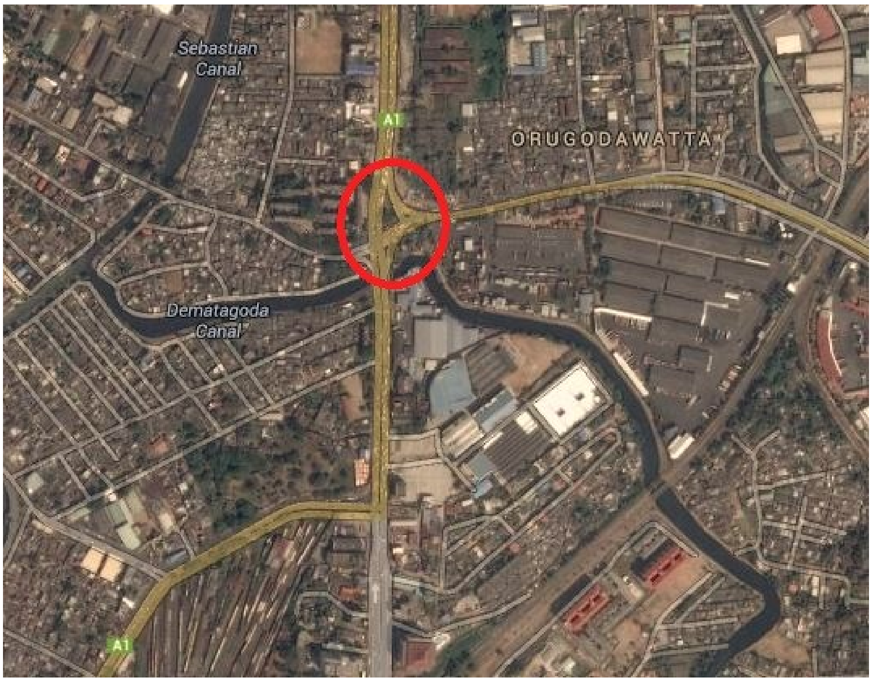


Figure 3-4: Orugodawatta Junction

Traffic flow data was then analysed in order to assess the appropriateness of providing a separate opening for U-turns.

BLINK 2005 software has been used to obtain the signal timings for different proportions of flows observed at the Orugodawatta junction. Further, the reduction of cycle times which can be obtained from eliminating the U-turns at the junction has also been calculated using the software.

For further analysis of different traffic conditions, VISSIM software already calibrated for the Sri Lankan condition & for different driver behaviour patterns, was used to simulate different traffic conditions, with and without the provision of separate opening. This was used for the estimation of delays and queue lengths.

Using the above analysis a guideline, which can be used for assessing the suitability of providing separate openings for U-turns at intersections is proposed so that the traffic engineers can use the values provided to minimize the delays in the intersection, by providing separate openings. Further, this can also be considered in coordinating the separate opening signal with the main intersection signals.

3.2.1. Data Collection

A full day traffic flow count was conducted at the Orugodawatta Junction. The flow count was based on a combination of video surveys and manual classified counts, because it was needed to simulate the existing traffic condition in the VISSIM traffic simulation software.

Further to this, information related to the existing, lane arrangement, traffic signal system and the phasing arrangements has also been obtained (Figure 3-5 & Figure 3-6).

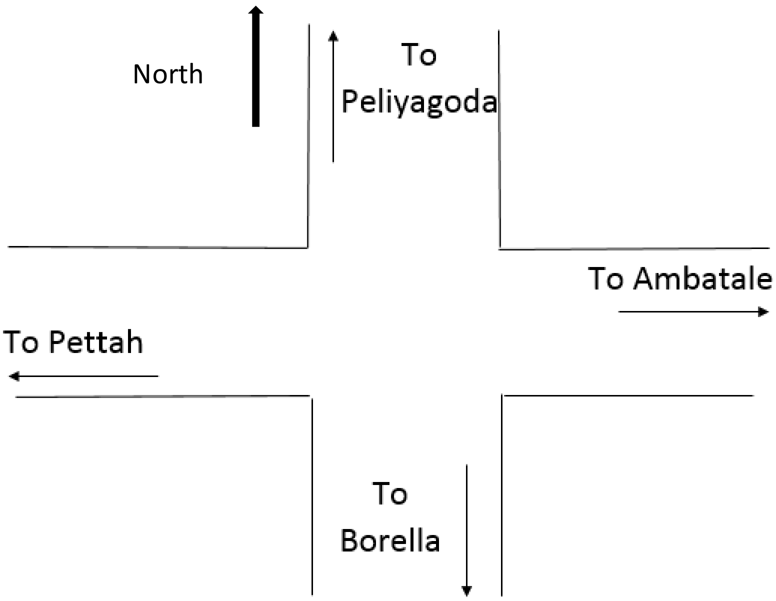


Figure 3-5: General arrangement of the intersection

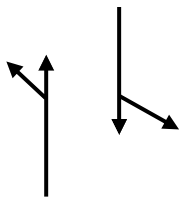
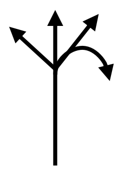
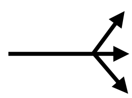
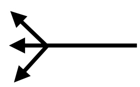
Phase 1	Phase 2
	
Phase 3	Phase 4
	

Figure 3-6: Phasing arrangement

The right turns are prohibited for the vehicles coming from the North Direction. The queue lengths obtained in most of the directions were more than 100m, where the cycle time was nearly 300 seconds which is very large due to the high amount of traffic flow.

The collected data with respect to the South Approach was summarized in the Table 3-9.

Table 3-9: Summary of Traffic Flow count at Orugodawatta Junction

Time	From Borella (South)		
	Through	**Right**	**U-turn**
06:00 - 07:00	2631	292	108
07:00 - 08:00	2400	277	120
08:00 - 09:00	2235	338	104
09:00 - 10:00	2431	307	89
10:00 - 11:00	2475	283	76
11:00 - 12:00	2577	306	76
12:00 - 13:00	2907	342	84
13:00 - 14:00	3149	347	66
14:00 - 15:00	2869	313	61
15:00 - 16:00	2956	414	89
16:00 - 17:00	4121	478	88
17:00 - 18:00	3715	514	80
Total	**34466**	**4211**	**1041**

The traffic flow data was obtained for a full day, in order to identify, if there is any significant variance of proportion of U-turns in the peak and off peak periods.

3.2.2. Analysis

The traffic flow data obtained was then analysed in order to assess the appropriateness of providing separate openings for U-turns at intersections.

From the basic analysis, results obtained are indicated in Table 3-10.

Table 3-10: Summary of traffic flow after basic analysis

Time	Out of all the vehicles			Out of all right + U turns
	% of right tur3s	% of U-turns	% of right+ U-turns	% of U-turns
06:00 - 07:00	10%	4%	13%	**27%**
07:00 - 08:00	10%	4%	14%	**30%**
08:00 - 09:00	13%	4%	17%	**24%**
09:00 - 10:00	11%	3%	14%	**22%**
10:00 - 11:00	10%	3%	13%	**21%**
11:00 - 12:00	10%	3%	13%	**20%**
12:00 - 13:00	10%	3%	13%	**20%**
13:00 - 14:00	10%	2%	12%	**16%**
14:00 - 15:00	10%	2%	12%	**16%**
15:00 - 16:00	12%	3%	15%	**18%**
16:00 - 17:00	10%	2%	12%	**16%**
17:00 - 18:00	12%	2%	14%	**13%**
Total	**11%**	**3%**	**13%**	**20%**

With respect to the basic analysis it has been observed that out of all the right turns nearly 20% - 25% of them are U-turns.

Further analysis has been done based on two steps, a theoretical analysis followed by a simulation analysis using VISSIM traffic simulation software, to arrive at a more concrete solution and a guideline.

3.2.2.1. Theoretical Analysis

The obtained data has been analysed using the Webster's Method to calculate the cycle times. The junction has been proven to be operating at a 4 phase system, and with the use of the vehicles flows and the mix, the cycle time is calculated as 291 seconds for the peak period.

Then the U-turns, which were around 25% of the total right turns have been eliminated from the intersections. The cycle time was again calculated and found to be 240 seconds, which is a 17.5% reduction (Table 3-11).

Table 3-11: Cycle time reduction

	Present Context	With Separate Opening
Cycle Time (Seconds)	291	240

- **Sensitivity Analysis**

A sensitivity analysis was conducted in order to arrive at a more reliable estimation which can be used for different scenarios.

The sensitivity analysis was conducted for 5 different cases, addressing 5 different intersections experiencing different proportions of U-turns and the results are tabulated in Table 3-12.

Table 3-12: Sensitivity Analysis

Proportion of eliminated U-turns	**Cycle Time (In seconds)**	**Reduction (%)**
Present Context	291	
5%	280	4.1
10%	269	7.8
15%	259	11.3
20%	249	14.5
25%	240	17.5

The cycle time reduction has then been converted to delay, in order to get a practical output to be considered by use of simulation.

3.2.2.2. Simulation based Analysis

It is decided to introduce a separate opening to the Orugodawatta Junction for the vehicles which are coming from the Borella direction to have their U-turns prior to the intersection (Figure 3-7).

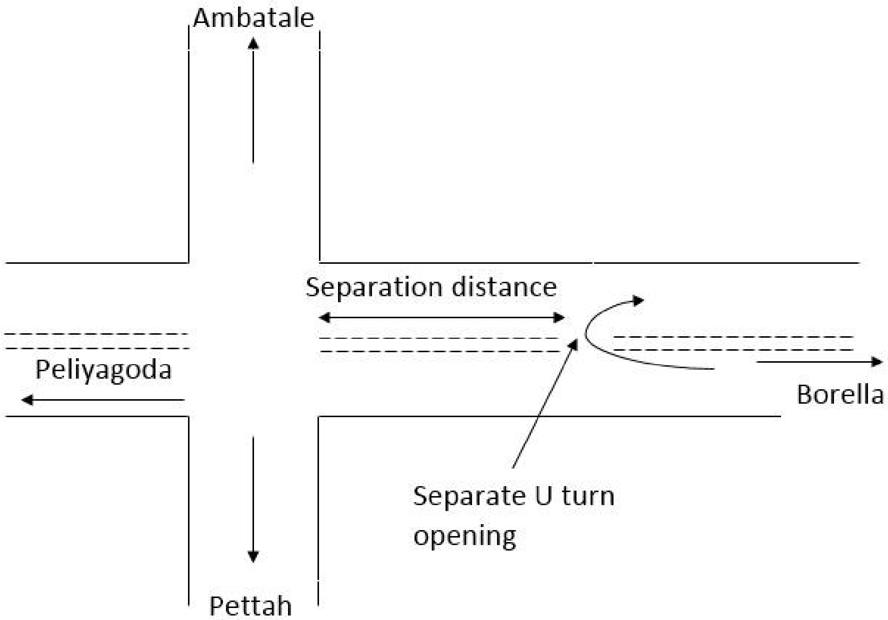


Figure 3-7: Orugodawatta Junction with separate opening

In order to analyse the effects of the elimination of the U-turns along with the delay reductions and capacity gains, the Orugodawatta junction was simulated using the VISSIM software (Figure 3-8).

Figure 3-8: Developing the network in VISSIM

The first step of simulation was to make sure the VISSIM software shows the existing traffic condition of the Orugodawatta intersection (Figure 3-9).

Figure 3-9: Simulating using VISSIM

In order to prove this, the queue length was observed in each direction of the existing condition and it was compared with the obtained queue length from the VISSIM software (Table 3-13). The queue counters were placed at each direction in the VISSIM software in order to obtain the queue lengths.

Table 3-13: Queue length observed vs Queue length obtained from VISSIM

	From Peliyagoda	**From Borella**		**From Ambatale**	**From Pettah**
	To Borella	**To Peliyagoda**	**To Ambatale**	**To Pettah**	**To Borella/ Ambatale**
Queue Length (Observed) - (m)	>150	80	97	130	73
Queue Length (VISSIM) - (m)	168	86	101	124	70

After properly calibrating the VISSIM software to show the existing traffic condition, the separate opening for U-turns was introduced.

Selection of the appropriate separation distance was also a challenge since there was no proper literature available to be used for the Sri Lankan context. Therefore, three separation distances were considered, 80m, 100m and 120m from the intersection, where the average queue in the Borella direction in the right turn lane was around 100m.

Different simulations have been conducted to obtain the most appropriate separation distance and the results obtained are tabulated in Table 3-14.

Table 3-14: Finding the appropriate separation distance

	Separation Distance		
	80m	**100m**	**120m**
Delay per vehicle in seconds	68.32	65.73	65.50

Separation distance of 80m was not considered because the delay obtained from the simulation was more than the other two scenarios.

Separation distance of 120m was not taken due to the site specific conditions prevailing at the Orugodawatta Junction, such as the issue with the school nearby and the adverse potential influence from the upstream intersection.

Therefore, with the above results, the appropriate separation distance was taken as 100m, where the delay per vehicle was reduced to 65.73 s, when compared with 68.32s.

With having the separation distance as 100m, the network has been simulated using VISSIM software (Figure 3-10) to understand the effects of the separate opening.

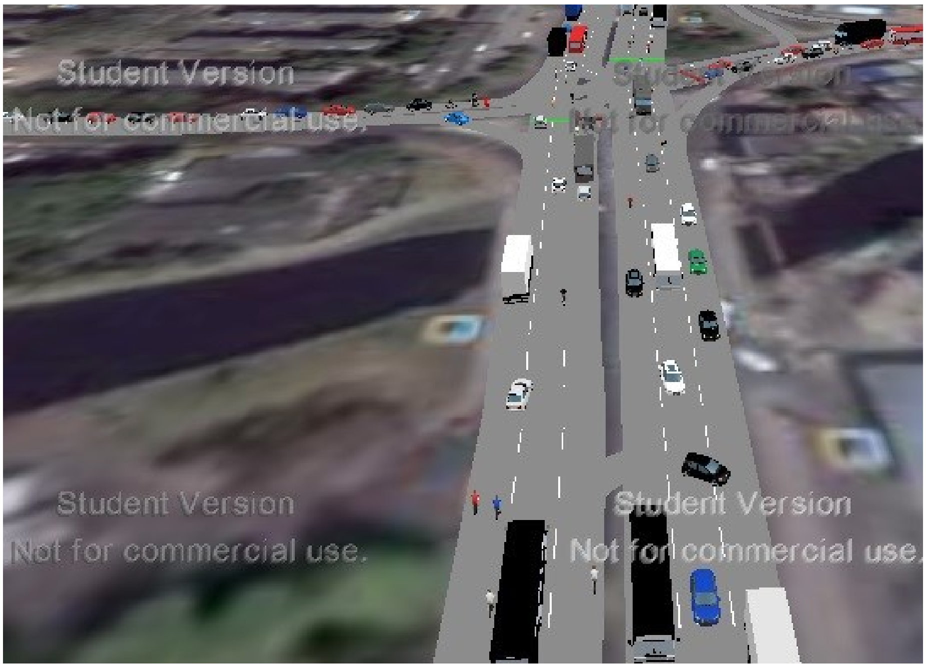


Figure 3-10: Simulation and Separate U-turn

The simulation time period was taken as two consecutive cycle lengths and the average number of vehicles which passed the intersection was obtained as 876 vehicles.

Based on the simulation, the queue lengths have been obtained and compared with the queue lengths without the opening (Table 3-15).

Table 3-15: Queue length comparison with and without the opening

	From Peliyagoda	From Borella		From Ambatale	From Pettah
	To Borella	To Peliyagoda	To Ambatale	To Pettah	To Borella/Ambatale
Queue Length (m) (Without opening)	168	86	101	124	70
Queue Length (m) With opening	152	60	52	110	58

Further to this, the average delays per vehicle have been obtained, for the considered 5 cases of simulation.

In calculating the delay, the vehicles which happened to be in the area of 150m radius from the centre of the intersection has been considered (Table 3-16).

Since the aim of any traffic management system is to improve the total travel time saving, savings that can be obtained were also separately calculated for all the scenarios considered.

The total travel time saving was calculated by multiplying the delay reduction per vehicle for different scenarios by number of vehicles for a period of one hour.

The results were obtained for the peak period and it is assumed that the approximately same values can be used for the off peak period because of the non-existence of the off peak time period in the considered road section.

Table 3-16: Analysis based on VISSIM simulation

Percentage of Eliminated U-turns	**Intersection With separate opening (delay per vehicle in seconds)**	**Total time saving per hour (in hours)**
Present	79.7	
5%	78.3	2.04
10%	76.3	5.06
15%	75.8	5.81
20%	72.3	11.10
25%	**70.3**	14.12

Another scenario is considered, where the U turns are allowed only when the right turns are given green. But it was obtained that the delay has been increased to 74.3s due to the additional waiting times expected for a 25% U turn proportion scenario, where earlier it was 70.3s (Table 3-16). However, if U-turns are allowed all the time there will be conflicts with through traffic. Idea of separate opening is to make more vehicle turn + U-turns during right turn phase

3.2.3. Results

Based on the analysis, the following results have been obtained from simulation approach, which can be used for intersections experiencing high proportion of U-turns (Table 3-17).

Table 3-17: Developed Guideline

Percentage of Eliminated U-turns	**Intersection With separate opening (delay per vehicle in seconds)**		**Total time saving per hour (in vehicle hours)**
	Delay	**Delay Reduction**	
Present	79.7		
5%	78.3	1.69%	2.04
10%	76.3	4.21%	5.06
15%	75.8	4.83%	5.81
20%	72.3	9.23%	11.10
25%	70.3	11.74%	14.12

For example, if an intersection is experiencing 15% of U-turns out of all the right turns, providing a separate opening, the average delay per vehicle can be reduced by 4.83%. Therefore based on the results, the above guideline can be used, to assess the effectiveness of eliminating U-turns at a signalized intersection and provide separate openings for U-turns.

3.3. Quantification of travel time savings and updating the Value of Time for evaluation of transportation projects

The delays that were obtained from the theoretical and simulation based analysis frameworks, need to be converted to a monetary value, which is very crucial in transportation project designs and implementations. This is very important for the research project also, since it can have a quantification of the benefits or the travel time savings that can present the gravity of the saving in terms of monetary figures when progressing in the research. For this purpose, the quantification of travel time savings were crucial.

The further analysis of value of time and updating the value of time is presented in Appendix B (See Appendix B).

B. Traffic Signal Design and Coordination

3.4. Development of guidelines for Individual intersections for fixed time systems

In development of guidelines for traffic signal designs and traffic signal coordination, the first step is to consider the individual intersections separately and to focus on fixed time traffic signals. If the guidelines are properly developed for individual fixed time signalized intersections, the same guidelines can be further improved for real time systems as well as for traffic signal coordination.

In developing guidelines for individual intersections, the following 2 categories of guidelines are expected to be developed.

I. Guidelines on phasing arrangement based on geometric details
II. Guidelines considering the traffic signal timings

3.4.1. Guidelines on phasing arrangement based on geometric details

Selection of phasing arrangement is very important not only for coordination but also for isolated intersections.

Each signalized intersection can be classified based on the availability of the separate turn lanes. The separate turn lanes can be either right turn or left turns, where depending on the availability the following category of classifications are identified.

- Intersections with no separate right turn lane available
- Intersections with separate right turn lane available
- Intersections with separate left turn lane available

Therefore, for the purposes of development of guidelines the above three categories of intersections are identified.

Due to non-availability of guidelines for traffic signal phasing, significant amount of delays are occurred mainly due to the complexities and non-uniformity resulting in road user frustrations. The main issue is if the optimum phasing arrangement is not selected there can be unnecessary increase in delays. At present it is not checked as there is no guideline available on selecting the best phasing arrangement based on the traffic flow and geometric conditions.

Therefore, the methodology is formulated so that respective guidelines are also developed for the identified phasing sequences for different geometrical arrangements.

With the findings of the literature review, the main criteria to be looked into when developing guidelines for phasing arrangements are the lagging green and leading green for different turning movements with availability of separate left and right turning lanes.

3.4.1.1. No Separate Right Turn Lane Available

The first situation is when the separate right turn lanes are not available. In this occasion, it has been argued that the leading green allows the right turning vehicles to take their right turns in advance. This would clear up the inner turning lane allowing the vehicles to use that lane even for their through movements. The issues encountered with lagging green such as lane blockage would no longer be there, increasing the capacity of the intersection as a whole. The decision should also be used in conjunction with proper phasing guidelines, when deciding on the phasing arrangement.

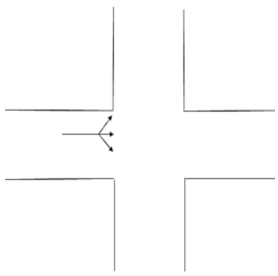
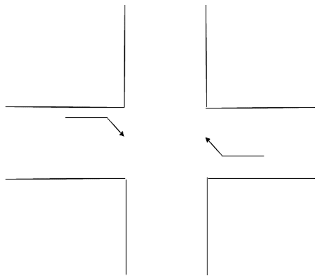
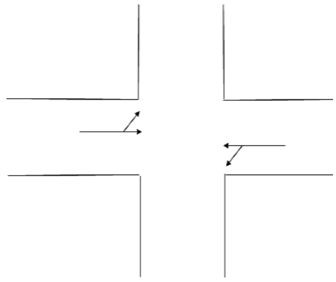
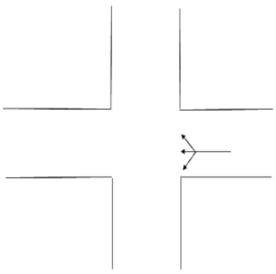
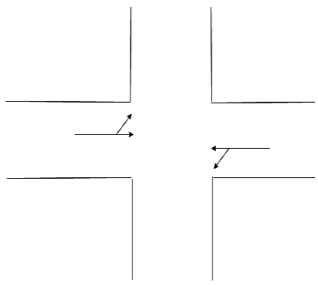
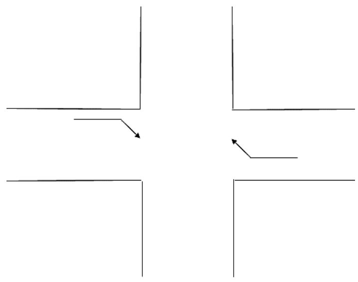
Development of Optimum Phasing Arrangements

According to literature (Xuan, Daganzo, & Cassidy, 2011), (Yang, Liu, Chen, & Yu, 2012), (Xi, ZhaoCheng, WenBo, ZhanQiu, & JunFeng, 2013), it is suggested to focus on theoretical and simulation based approaches in developing the research. Delay minimization was considered as optimization objective.

Eliminating the impact of right turns at the signalized intersections to minimize the delays is an important consideration. The research was formulated in a way that, impact on right turns are reduced through the introduction of optimum phasing arrangement.

Three main phasing arrangements were considered for the right turns as mentioned in Option A, Option B and Option C (Table 3-18). This is applicable only if two or more lanes per approach is available to facilitate the right turns only phase.

Table 3-18: Option A, Option B & Option C

	Option A	**Option B**	**Option C**
Phase 1			
Phase 2			

- Option A: Separate through plus right turn phase
- Option B: Advance Green for right turns only phase for peak approach
- Option C: Delayed green for right turns only phase for peak approach

Theoretical Analysis

In order to introduce theoretical aspects of traffic signal designs, Webster's method was used for the traffic signal design calculations. To obtain the effect on the traffic flow from Option A, Option B and Option C, a theoretical analysis is carried out considering 5 different scenarios.

Development of scenarios

In order to reflect the changes of traffic flow, 5 different cycle times were considered under each scenario. In all the cases, the right turn percentage was kept as 10% out of the total through plus right turns volume (Table 3-19). The volume/capacity ratios were varied, so that different traffic conditions are experienced, keeping the minor road traffic flows constant.

Table 3-19: Developed Scenarios

Scenario	**Volume/Capacity Ratios**
1	0.9
2	0.8
3	0.7
4	0.6
5	0.5

Theoretical Analysis Process and Results

As for the developed scenarios, the respective cycle times for Option A, Option B and Option C were separately calculated. The reduction of cycle time was needed to be converted to delay, which is obtained by using Webster's delay formula (Equation 3-5). The formula has been extensively used in research to convert the cycle time reductions to delay reduction.

$$d = \frac{C\,(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q\,(1-x)} - 0.65\left(\frac{C}{q^2}\right)^{\frac{1}{3}} \times x^{(2+5\lambda)} \qquad (3\text{-}5)$$

where, d represents the average delay per vehicle measured in seconds

C is the Cycle time of the signalized intersection

λ is the proportion of effective green on the cycle time

x refers to the degree of saturation

q refers to the arrival rate of vehicles measured in seconds

A cycle time reduction was also obtained in the order of 3.5% to 5% per vehicle, when shifting from Option A to Option B or Option C (Table 3-20). When considering the total traffic flows in an overall perspective, significant savings can be obtained.

Table 3-20: Results of Theoretical Analysis

% of right turns out of the total through + right volume	Scenario	Cycle time (in seconds)		Reduction of Cycle Time	Reduction of Delay per vehicle(in seconds) (From Equ.3-5)	Reduction of Delay (Percentage)
		Option A	Option B / Option C			
10%	1	150	126	16.0%	1.3	3.5%
	2	141	121	14.2%	1.5	3.9%
	3	122	106	13.1%	1.9	5.1%
	4	108	100	7.4%	1.6	4.4%
	5	99	92	7.1%	1.5	4.4%

Based on the preliminary theoretical analysis, it is concluded that significant cycle time reduction can be obtained, which in turn will reduce the overall delay when shifting from Option A to Option B or Option C.

Simulation Based Analysis

In addition to a theoretical based analysis, a simulation based analysis was also conducted using VISSIM traffic micro simulation software in order to determine whether the same delay reductions can be obtained irrespective of the percentage of right turns.

6 new scenarios were developed to analyse the impact of phasing arrangement with the differing percentage of right turns. The right turn percentage was varied from 5%

to 30% and the respective cycle time using Webster's method was calculated and used as inputs for the VISSIM micro simulation analysis.

The results for the 6 considered scenarios are mentioned in Table 3-21 and Figure 3-11.

Table 3-21: Results of the Simulation Analysis (No right turn lane available)

Scenario	% of right turns	Average Delay per Vehicle (Seconds per Vehicle)		
		Option A	Option B	Option C
1	5	40.6	38.2	40.2
2	10	41.1	40.0	40.8
3	15	41.5	41.4	42.2
4	20	42.5	42.8	43.6
5	25	43.4	44.6	45.5
6	30	44.5	44.9	45.8

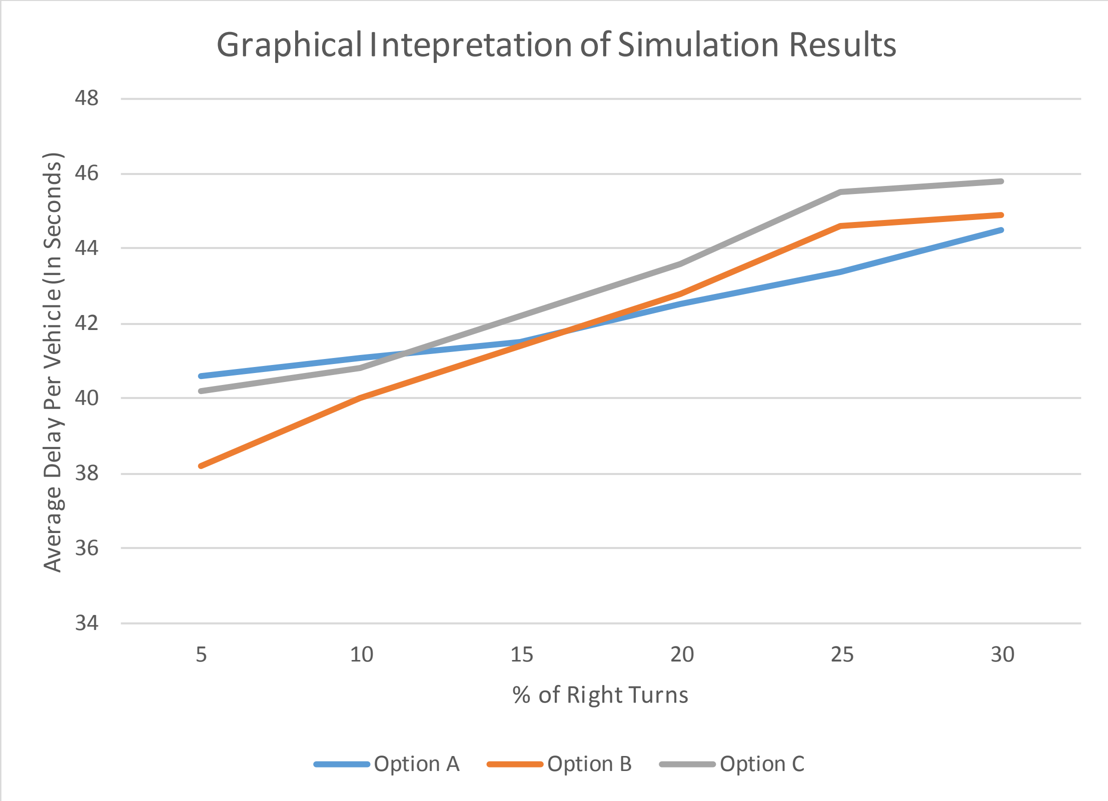


Figure 3-11: Graphical Interpretation of Simulation Results

From the analysis it can be concluded that Option B would be the optimum phasing arrangement, when the right turns are less than 15% of the total per direction volume, which results in minimum overall delay per vehicle. When the right turn percentage is more than 15%, Option A would produce lower delay values.

Discussion

The main criterion was to decide whether the green phase for right turns should be given with the through movements (Option A) or separately and if separately whether it is to be given leading (Option B) or lagging (Option C), when no separate right turn lane is available, for the situations where two or more lanes are available.

Right turns only phase either leading or lagging (Option B or Option C) would considerably reduce the cycle time of a signalized intersection, which would in turn reduce the overall delay per vehicle in an intersection, which is proved through the theoretical analysis conducted.

Based on the simulations and observations, it can be seen that the leading green allows the right turning vehicles to take their right turns in advance. This would clear up the inner turning lane allowing the vehicles to use that lane even for their through movements. The issues encountered with lagging green such as lane blockage would be less, increasing the capacity of the intersection as a whole.

Option B would be ideal when the right turns flow is less than 15% of the total through volume which is verified through simulations. This is important because if not, the right turns will block the ongoing though movements also increase the overall delay.

However, as for the simulation results, Option A would be ideal when the right turn flow is more than 15% of the total through volume, in order to minimize the delays. This is mainly due to additional lane blockage, due to the residual right turning vehicles, which did not have adequate signal timing for manoeuvring, creating additional blockage for the continuing through movements.

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3.4.1.2. Separate Right Turn Lane Available

With respect to geometrical details, one of the main considerations was to analyse the junctions having separate turning lanes, either separate right turn lanes or separate left turn lanes. Under that the intersections with separate right turn lanes were analysed.

The main criterion was to decide whether the green phase should be given lagging or leading. The lagging green would allow the vehicles to safely get themselves stored in the separate right turn bay without disturbing the through movements. When the right turn vehicles get green, they can proceed with their maneuverer without much issue. Further, it is essential to make sure that sufficient length is available for right turn bays, so that it would not get filled up. Leading green would also make sure that the vehicles in the right turn bay is cleared and would not disturb the through movements. Therefore, intersections having separate right turn lanes can be catered using both leading and lagging green for right turns but the phasing of the right turns should be carefully selected.

Guideline

If separate right turn is available, go for leading or lagging green for right turns only phase and if right turn % is significantly smaller, it can be accommodated at the end of Amber (Figure 3-12 & Figure 3-13).

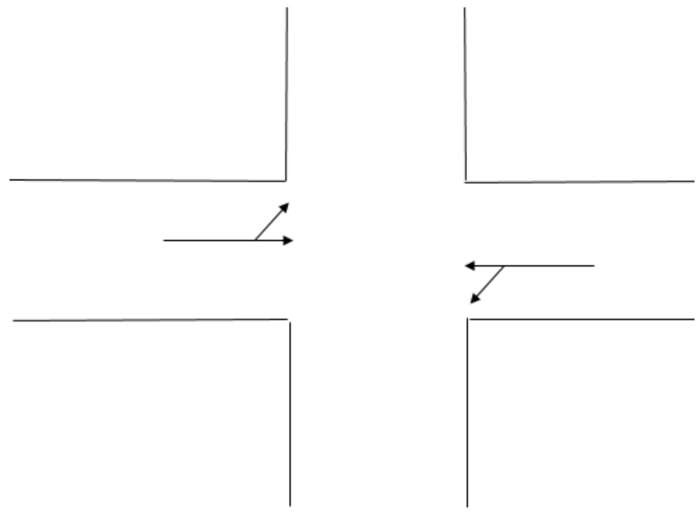


Figure 3-12: Phasing Arrangement – Phase 1

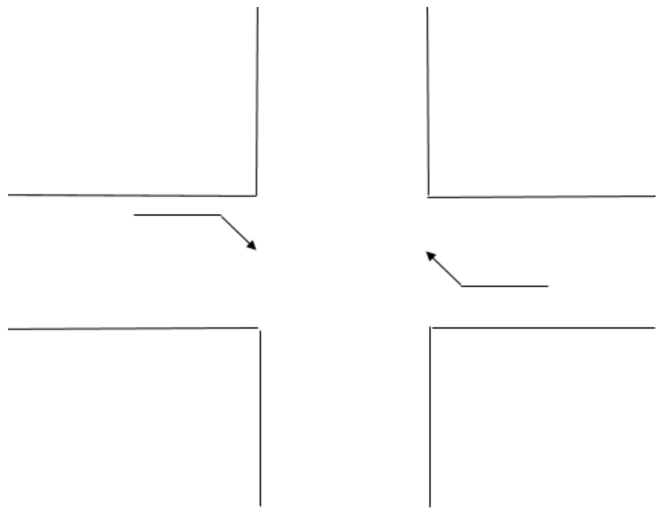


Figure 3-13: Phasing Arrangement – Phase 2

This would considerably reduce the cycle time of a signalized intersection, which would in turn reduce the overall delay per vehicle in an intersection.

3.4.1.3. Conditions for provision of Separate Left Turn Lanes

It is important to mention that provision of separate left turns are observed in most of the intersections but due to various issues, the functionality of such separate left turns were not observed as desired. Due to this a guideline was developed, based on the length allocation for the lanes.

Provision of separate left turns should be subject to the proportion of left turns out of other movements.

In order to analyse the intersections with separate left turns, a simulation was conducted, varying the % of left turns out of the through volume.

The following parameters were kept constant in all the simulation cases considered.

- Cycle time = 120 seconds
- Volume over capacity ratio is kept at 0.8 to simulate the peak condition
- Simulation time period = 20 mins
- 3 lanes per direction is considered

The following results were obtained for different scenarios being considered (Table 3-22).

Table 3-22: Results of simulation for separate left turn situation

% of left turns out of through volume	Average Delay per vehicle in the considered direction (in seconds)		Delay reduction with separate left turn situation
	Without separate left turn	With separate left turn	
0%	14.12	14.12	
5%	14.53	14.04	0.57%
10%	15.23	13.92	0.85%
15%	16.45	13.73	1.36%
20%	17.94	13.24	3.57%
25%	18.45	12.93	2.34%
30%	19.35	12.43	3.87%
35%	21.45	11.41	8.21%
40%	24.56	10.23	10.34%
45%	27.35	9.25	9.58%
50%	31.54	8.34	9.84%

According to the results obtained, a significant decrease in the average delay per vehicle is observed when the percentage of left turns are more than 35% of the through vehicles, where the equivalent per lane volume of the considered scenario is 33.33%. Therefore it can be concluded that, if the amount of left turns expected are more than the equivalent per lane amount of through movement, it is worthwhile to provide a separate left turn lane because it would facilitate the filtering of the left turns, increasing the capacity of the intersection.

Further, if it is decided to introduce a separate left turn lane, it is always desirable to use amber blinking light for left turns. However, this should actually work effectively if no adverse geometrical details and no heavy pedestrians' movements are observed.

Guideline

It is recommended to provide a separate left turn, if the amount of left turns expected are more than the equivalent per lane amount of through movement. If separate left turn lane is provided amber blinking light is recommended if no adverse pedestrian and geometric details are observed.

3.4.2. Guidelines considering the traffic signal timing

With properly sequenced phasing arrangements, the next most important step is to develop guidelines for traffic signal timings. As for the literature review findings, significant contribution for traffic signal delay is from improper and inappropriate traffic signal timings, such as unusual longer cycle lengths.

This can not only affect the considered intersection, but also the adverse impacts of longer cycle lengths can also significantly hinder the performance of the nearby intersections. Therefore, it is essential to develop set of guidelines for traffic signal timings also, which is another crucial step in the methodology.

3.4.2.1. Maximum Cycle time

With the recent observations in Sri Lanka, it has been identified that some signalized intersections do operate with very high cycle times, even more than 200 seconds. As a result, this can affect the efficiency of the intersection due to stochastic random nature of the vehicles. Therefore an experiment was considered in order to identify the most effective cycle time to be used for the context of Sri Lanka.

Four Phase Intersections

A simulation is conducted in order to determine the maximum cycle time to be used for 4 phase intersections.

The following parameters were kept constant in all the simulation cases considered.

- Volume over capacity ratio is kept as 0.8 to simulate the peak condition
- Simulation time period = 30 mins
- Phase timings were kept at the same proportions, with respect to the cycle times

Cycle time was varied at 8 different values keeping the others as constant and the following results were obtained for a 4 phase intersection (Figure 3-14).

Cycle Time (In seconds)	Average delay per vehicle (In seconds)
130	25.3
140	24.9
150	24.5
160	24.7
170	25.1
180	25.0
190	25.4
200	25.7

Figure 3-14: Cycle time for 4 phase intersections

Three Phase Intersections

A simulation is conducted in order to determine the maximum cycle time to be used for 3 phase intersections.

The following parameters were kept constant in all the simulation cases considered.

- Volume over capacity ratio is kept as 0.8 to simulate the peak condition
- Simulation time period = 30 mins
- Phase timings were kept at the same proportions, with respect to the cycle times

Cycle time was varied at 6 different values keeping the others as constant and the following results were obtained for a 3 phase intersection (Figure 3-15).

Cycle Time (In seconds)	delay per vehicle (In seconds)
100	18.9
110	18.6
120	18.6
130	18.8
140	19.3
150	19.6

Figure 3-15: Cycle time for 3 phase intersections

Therefore, maximum cycle time should be kept as 150 seconds for 4 phase intersections and 120 for three phase intersections, except under special circumstances, such as for signal coordination.

With the simulations being conducted, the least delay was expected as according to the following scenarios.

- For the scenario of 4 phase intersections, when the cycle time was kept as 150 seconds
- For the scenario of 3 phase intersections, when the cycle time was kept as 120 seconds

Even though the cycle time obtained through the Webster's method is greater than 150 seconds it is always recommended to keep a maximum cycle time of 150 seconds for 4 phase intersections and 120 seconds for 3 phase intersections

The stochastic nature of the vehicle movement and driver behaviour would further expect the cycle time to be reduced to a maximum limit as well. **Abbas (2000)** argues that cycle time should be kept **less than 140 seconds.**

3.5. Development of guidelines for individual intersections for real time systems

It is vital to utilize the modern technology available for the benefits of the man kind in the areas of transportation also. Therefore the latest technology should be made available and proper guidelines should be developed for the traffic signals and traffic signal coordination in real time systems, which is identified as one of the main priorities in the research.

In some occasions the fixed time traffic signal system will not yield the expected outcomes and developed real time systems will not necessarily generate the favourable results also in all the situations. Therefore, a proper criteria and guidelines should also be developed to identify under which circumstances the developed strategies to be used.

As for the literature review findings, it is noted that there can be number of real time applications that you can address for traffic signal designs, but the main criteria was to determine the extensions of green time for traffic signal platoon directions. This extended green time should also be limited, having a maximum green time to obtain the lowest possible overall delay.

Minimization of delay was considered as the optimization criteria in real time traffic management for the considered scenario. The negative impacts that can have on the cross traffic and safety concerns in traffic signal coordination were also given serious concerns in developing the methodology and in the analysis procedure. The methodology comprises of the following steps in development of guidelines for real time traffic signals.

3.5.1. Procedure

In order to ground the theoretical aspects, it is proposed to use a theoretical framework for the analysis purposes. Webster's delay estimation formula was used for the theoretical analysis as mentioned in Equation (3-5).

$$d = \frac{C\,(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q\,(1-x)} - 0.65\left(\frac{C}{q^2}\right)^{\frac{1}{3}} \times\ x^{(2+5\lambda)} \qquad (3\text{-}5)$$

where,

d	the average delay per vehicle measured in seconds
C	the Cycle time of the signalized intersection
λ	the proportion of effective green on the cycle time
x	the degree of saturation
q	the arrival rate of vehicles measured in seconds

For simulation based analysis, calibrated VISSIM micro traffic simulation software was utilized (Figure 3-16).

Figure 3-16: Calibration of VISSIM software

In order to develop a reliable guideline for the variation of traffic flows which is experienced in real time traffic and geometrical arrangements in different intersections, concept of Volume/Capacity (V/C) ratios were considered. Further, if traffic signal coordination is expected, minor road volume is also crucial for accurate result prediction. The following 4 scenarios and sub scenarios were considered for the study (Table 3-23).

Table 3-23: Scenarios developed

Scenario	V/C Ratio	
	Major Road (M)	Minor/Cross Road (C)
1	0.8	0.3
2		0.7
3	0.6	0.3
4	0.4	0.3

As per the surveys conducted, in high peak periods, where the major road V/C is around 0.8, 2 sub scenarios were considered for low minor road volumes (V/C = 0.3) and for high volumes (V/C = 0.7). For scenario 3 & 4, where the Major road V/C are 0.6 & 0.4, no sub scenarios were considered, assuming the minor road volumes would remain low with V/C around 0.3.

As identified in the literature review, the main two factors to be considered in real time traffic signals is the minimum and the maximum green time extensions, which would depend on the geometrical arrangements and the traffic flows.

Therefore, the procedure is developed so that each of the 4 considered scenarios are tested for the variation of vehicular flow differences. For each intersection and scenario considered, the green split provided for the main direction, obtained through Webster's method, is changed from -15% to +15%, with the splits as -15%,-10%-5%, 0%, 5%, 10% & 15% and the respective overall delay per vehicle was calculated using Webster's delay estimation formula.

The lowest overall delay per vehicle was considered as the limit for the respective minimum and maximum green time extensions for the scenario considered.

3.5.2. Analysis and Results

3.5.2.1. Calculation of delay for intersections with the variation of green split

As for the analysis for Scenario 1, where major road V/C = 0.8 and minor road V/C = 0.3, the following results were obtained (Figure 3-17) using Webster's delay estimation formula.

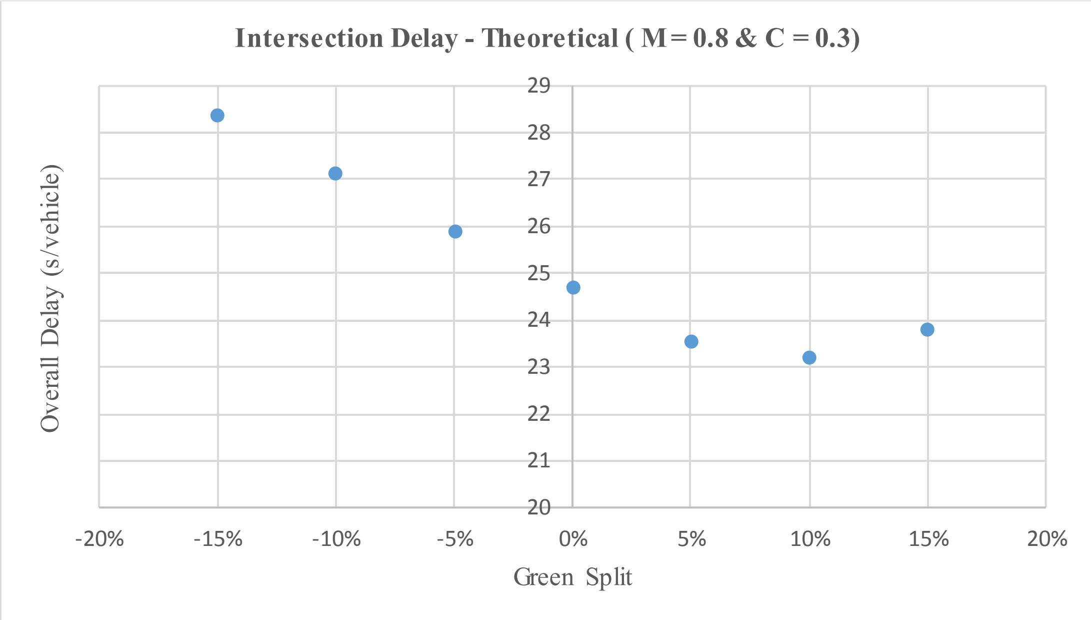


Figure 3-17: Theoretical Delay (Scenario 1)

In the situation where the major road v/c ratio is 0.8 and the minor road v/c is 0.3, the lowest delay was recorded when the green split is 10%. This suggests that intersections can be benefited from real time traffic signals, when the green extension is extended for a maximum 10% of the available green time.

The same calculation was performed for the scenario 2 as well, where the major road v/c is 0.8 and the minor road v/c is 0.7 and the respective results are obtained (Figure 3-18)

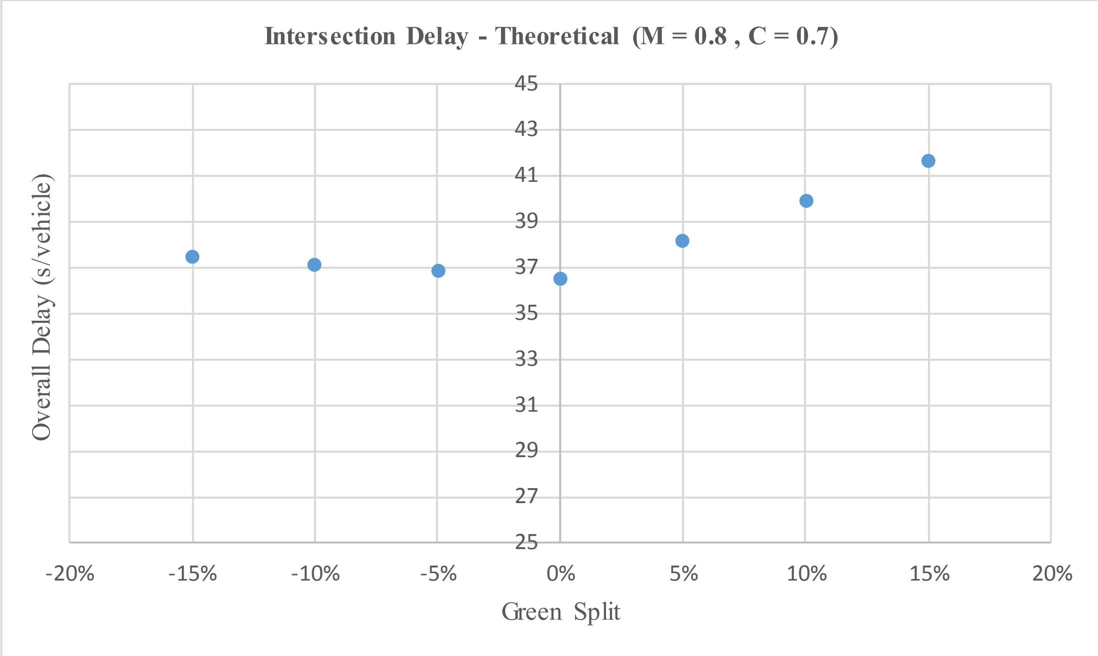


Figure 3-18: Theoretical Delay (Scenario 2)

Here, any extension of green time would additionally create negative impact and overall delay for the vehicles, therefore minimum green should be kept as the calculated cycle time. This is mainly due to the increase of cross minor road traffic volume which can increase the delay in an overall perspective.

The above procedure was performed for all the other considered scenarios to obtain the optimum green splits.

3.5.2.2. Delay estimation using simulation

In order to verify the obtained theoretical results, VISSIM micro simulations were also carried out for the same scenarios. Figure 3-19 presents the simulation results for the Scenario 1 and Figure 3-20 for the Scenario 2.

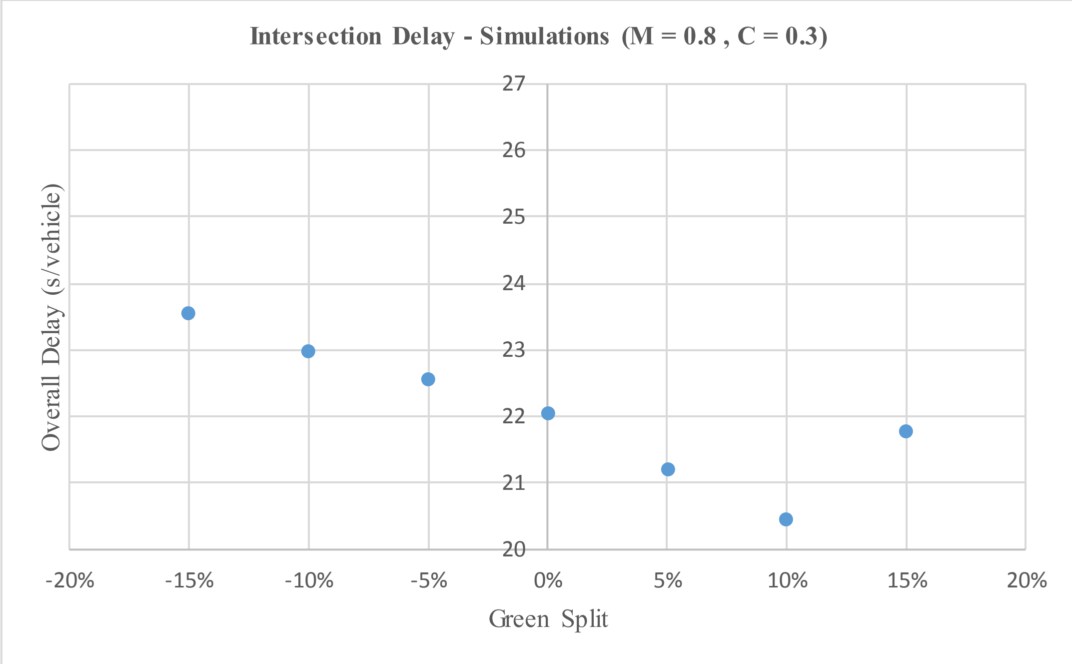


Figure 3-19: Simulations Delay (Scenario 1)

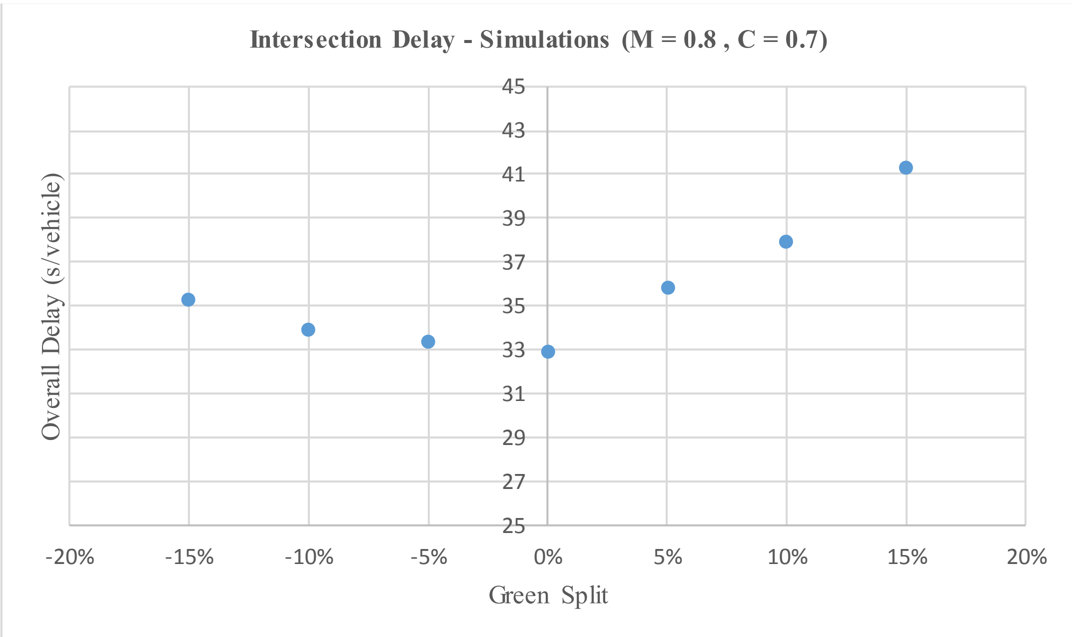


Figure 3-20: Simulation Delay (Scenario 2)

Though the delay values obtained from micro simulations were slightly underestimated when compared with theoretical delays based on Webster Equation, the optimum green split for minimum delay was the same with the theoretical calculations.

Therefore the same procedure both calculation and simulation was conducted for all the other scenarios considered and the respective optimum green split timings were obtained.

3.5.2.3. Determination of the optimum green time extension for all the considered scenarios

Based on the theoretical and simulation based scenarios considered, the respective green split values were obtained as mentioned in Table 3-24.

Therefore, according to the theoretical and simulation results, the green time extensions can be beneficial only with the following scenarios.

- Scenario 1: When Major road V/C = 0.8 and minor road V/C = 0.3
- Scenario 3: When Major road V/C = 0.6 and minor road V/C = 0.3

Overall delay reductions per vehicle in the range of 5% - 7% can be obtained through the extensions of green time for the above two scenarios, which is significant when considering the total vehicle volumes and total value of time savings.

Table 3-24: Results for all the scenarios

V/C Ratio		Theoretical analysis			Simulation Analysis		
Major Road	Cross Road	Recommended green time extension	Delay reduction per vehicle (in seconds)		Recommended green time extension	Delay Reduction per vehicle	
			In seconds	Percentage		In Seconds	Percentage
0.8	0.3	10%	1.51	6.11%	10%	1.59	7.21%
	0.7	No extension recommended	-		0%	-	-
0.6	0.3	5%	1.26	5.73%	5%	1.42	7.10%
0.4	0.3	No extension recommended	-	-	0%	-	-

3.6. Identification of the signal coordination potential for intersections

After the proper guidelines for traffic signal designs for individual intersections, it is vital to focus on traffic signal coordination. It is required to mention that the research focused on development of guidelines for non-similar intersection clusters in traffic signal coordination, since there is no such guidelines available with the differing geometrical arrangements, traffic mix and peak off-peak variations where most of the research only focussed on isolated intersections.

As for the first step, it is required to identify which intersections will be benefited from traffic signal coordination. Therefore, proper guidelines should be developed for identification of intersections which can benefit from traffic signal coordination.

The next step was to determine which intersections should be considered or clustered for traffic signal designs and coordination aspects because improper clustering of intersections can lead to significant delay overruns.

It is decided to use a combination of theoretical and simulation based approach for the development of a value for clustering index.

It is observed that for countries where high accessibility within arterial roads (many byroads in between) and aggressive vehicle behaviour pattern exists, it is not appropriate to use distance as a reliable parameter for the development of a coupling index.

Literature identifies two main parameters that should be used when developing an index for identification of control region, namely, traffic flow and distance separation of intersections. Further, peak flow direction for coordination was used as different intersection geometries will not allow coordinating both direction at the same time.

Further, though Bonneson (2011) suggests to use distance as a parameter, the high accessibility observed between intersections can imply inaccurate results if the distance is used as a parameter. Travel time is considered as a better indicator (parameter) over the distance separation as intermediate or intermittent disturbance from by roads can make travel time different for same distance separation Therefore, in order to address the effects of separation of intersection, travel time between the intersections is considered.

Therefore, the research tries to develop an index called "Clustering Index" which can be considered as the ratio of the coordinated direction flow measured in veh/hour to the vehicle travel time between the intersections, measured in seconds, where the higher the clustering index, higher efficiencies can be obtained (Equation 3-6).

$$\text{Clustering Index} = \frac{Coordinated\ Direction\ Flow\ \left(\frac{veh}{hour}\right)}{Travel\ Time\ (seconds)} \qquad (3\text{-}6)$$

3.6.1. Test Model for Clustering Index Development

In order to decide whether two intersections should be clustered for traffic signal coordination, it is essential to develop a criteria for clustering index. With the developed ratio, coordinated direction flow is proportional to clustering index and travel time is inversely proportional to clustering index. Therefore, in order to decide whether the two considered intersections are to be clustered, a minimum value for the clustering index should be established. Higher the clustering value, higher efficiency can also be obtained.

For the traffic signal coordination work, VISSIM traffic micro simulation software was used. The VISSIM software was calibrated for the local condition, with the inclusion of the calibrated parameters to account for the driver behaviour, traffic mix and peak off peak variations.

The two identified parameters, coordinated direction flow and travel time need to be varied, in order to decide on a minimum value for the clustering index, where a test model has been developed as mentioned in Figure 3-21 using VISSIM traffic simulation software.

The test model consists with the following characteristics.

- The distance between the selected two intersections were kept as 500m
- The two junctions were identical in geometry

- The east-west direction is considered as the coordinated direction and consist with 3 lanes in one direction
- The north south roads are the cross roads which consist with two lanes in one direction
- The cycle time was calculated using Webster's Method (1958)

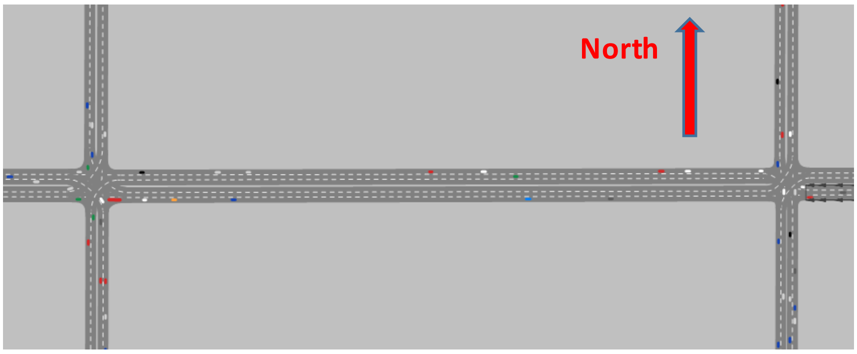


Figure 3-21: Snap shot of simulation

3.6.2. Varying the Vehicle Flow

The variation of the vehicle flow was obtained by changing the traffic flow, by keeping the traffic mix and other related parameters constant. For this purposes, volume over capacity (V/C) ratios were considered, since this can be made applicable for all the links irrespective of number of lanes.

The following five V/C ratios were considered for the analysis purposes to represent the actual traffic flow conditions (Table 3-25).

Table 3-25: 5 Scenarios considered

Scenario	Volume Over Capacity Ratios
1	0.5
2	0.6
3	0.7
4	0.8
5	0.9

3.6.3. Varying the travel time

The travel time could be varied either by changing the distance or by changing the access points within the considered two intersections. When considering varying the distance, it ends up in the same logic as proposed in the previous research (Bonneson, 2011), where the effect of accessibility might not be accounted, which is significant in the local context. Therefore, the latter option was considered, where the access points between the intersections were varied so that the resultant travel time would be changed, since number of disturbances were increased.

3.6.4. Analysis Procedure

The number of access points between the intersections was increased from one by one and the variations of the travel time between the intersections were recorded with the simulations conducted for the respective scenario considered.

Further, with the obtained travel time and the coordinated direction traffic flow, the clustering index was calculated which is the ratio of coordinated direction flow to vehicle travel time in that direction. For the identification of the minimum clustering index values, the HCM delay criteria was used.

As for the guidelines given by Highway Capacity Manual (2010), the intersection delay less than 35 seconds/ vehicle is acceptable. Therefore, the scenarios where the expected delay was less than 35 seconds/vehicle were considered as favourable and the corresponding clustering index value for the maximum accepted delay is selected as 35 seconds.

The same procedure was carried out for the other scenarios of different V/C ratios as well.

3.6.5. Analysis & Results

The results obtained for the considered scenarios are mentioned along with the sample calculation for V/C = 0.5 scenario (Table 3-26).

- **Scenario 1: V/C = 0.5**

Table 3-26: Scenario 1: V/C =0.5

Number of disturbances to through traffic between intersections	Travel Time (seconds)	Average Delay per vehicle (Seconds)	Clustering Index
1	25.2	16.1	79.37
2	26.4	16.3	75.76
3	28.3	16.7	70.67
4	29.6	17.4	67.57
5	31.5	18.7	63.49
6	33.4	19.6	59.88
7	34.7	21.2	57.64
8	38.5	23.3	51.95
9	41.2	36.1	48.54
10	46.5	28	43.01
11	49.5	30.2	40.40
12	51.6	31.2	38.76
13	57.5	33.4	34.78
14	**65.4**	**35.2**	**30.58**
15	68.4	37.4	29.24
16	71.3	39.2	28.05
17	78.4	42.3	25.51
18	81.2	46.1	24.63

For the scenario 1 considered, the average delay per vehicle was increasing when additional disturbances are introduced to the ongoing traffic and the respective clustering index values were reducing. As for the HCM criteria, the minimum value for clustering index was determined when the average delay value per vehicle is 35 seconds (Table 3-26).

Similarly same procedure was carried out for the remaining 4 scenarios and the results are as mentioned in Table 3-27.

Table 3-27: Respective Clustering Index Values

Scenario	V/C Ratio	Clustering Index (Minimum)
1	0.5	30.58
2	0.6	45.89
3	0.7	56.80
4	0.8	67.65
5	0.9	83.33

For a respective V/C ratio, above values are the minimum values that should be satisfied for traffic signal coordination. It is recommended to cluster the intersections if the clustering index values obtained are more than the given minimum clustering index values.

Therefore, before coordinating the traffic signals, it is required to establish the methods and decide on whether the clustering index criteria are satisfied to harness maximum benefits in traffic signal coordination.

3.7. Development of Guidelines for traffic signal coordination for non-similar intersections

After the identification of the potential intersections and intersection clusters for traffic signal coordination, the next step of the research is to develop guidelines for traffic signal coordination when non-similar intersections are encountered. The research focuses on the methodology to be followed when non-similar intersection are experienced.

The non-similar intersections can occur due to the different geometrical arrangements and also with the direction to which the priority should be given for the green coordinated platoon of vehicles.

3.7.1. Identification of the sub categories of intersections

The following sub categories of the intersections are identified based on the geometrical arrangements considering whether the intersections are 3 way or 4 way and also considering the direction of the green platoon (Figure 3-22)

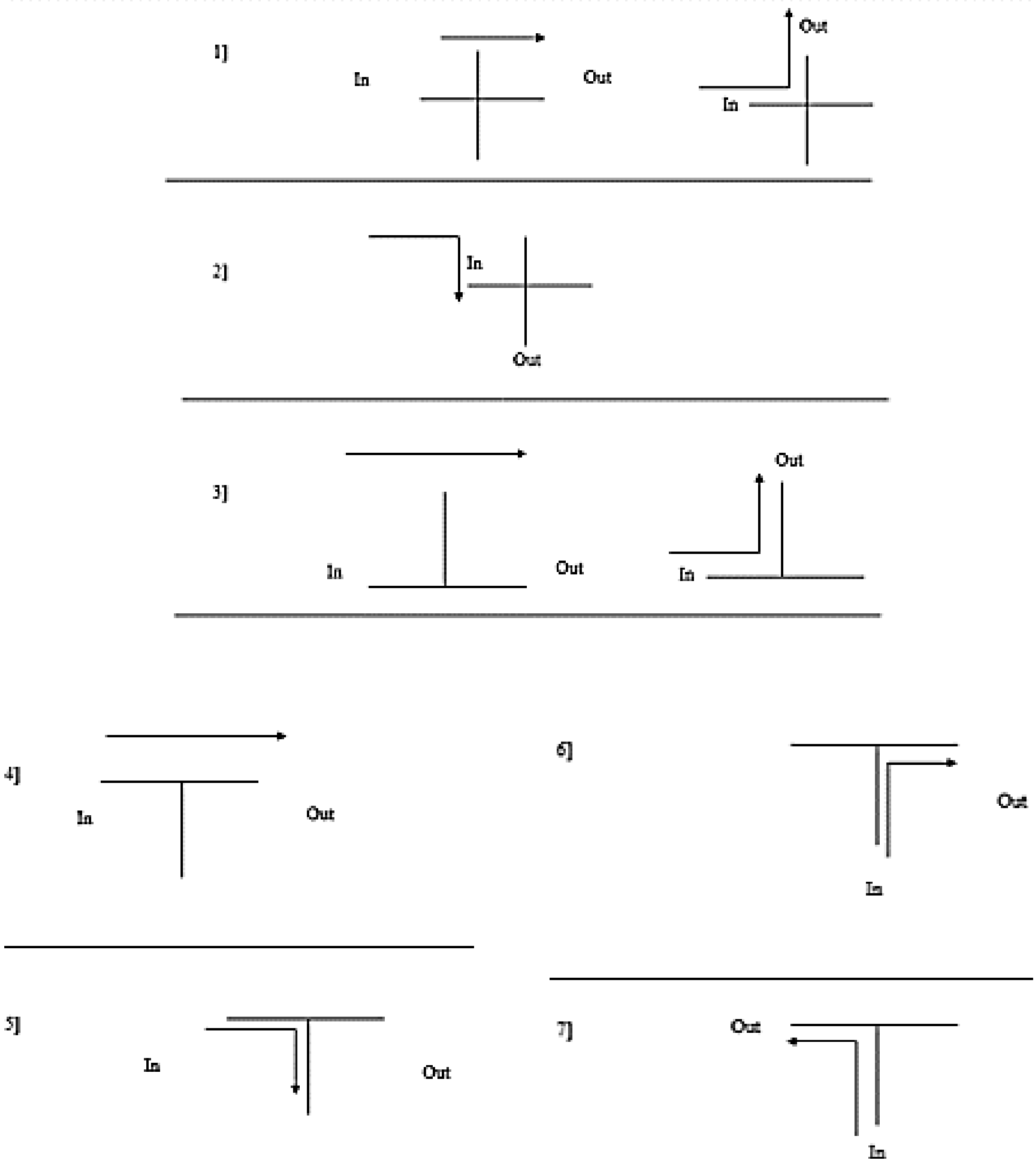


Figure 3-22: Intersection Types for Traffic Signal Coordination

Intersection types 1 & 2 are for the 4 leg intersections and the intersection types 3, 4, 5, 6 & 7 are for the 3 leg intersections. The arrows mentioned in the diagrams would guide on the priority direction of the green platoon of vehicles.

- **Intersection Type 1**

This considers a 4 leg signalized intersection, where the priority direction for the green platoon is given for the through traffic as well as the left turns. Priority directions for both through traffic and left turns are considered as similar in the analysis procedures. Depending on the arrangement of the practical intersections, it is necessary to decide whether which intersections would suit this category 1.

- **Intersection Type 2**

Type 2 intersections are also 4 leg intersections, but the priority direction for the green platoon is different in this occasion. Type 2 intersections can be identified when the priority direction is given for the right turns, opposed to the through and left turns. Regardless of the entry direction, if the priority should be given for the right turns, such intersections can be classified as Type 2.

- **Intersection Type 3**

Type 3 intersections are 3 leg intersections, where the 3rd leg in the intersection is positioned to the left of the entry direction and the priority direction is either the through or the left turns. It is also considered that both situations where the through traffic and left turns priority directions would be categorised as under the Intersection Type 3.

- **Intersection Type 4**

Type 4 Intersections are also 3 leg intersections but the arrangement of the 3 leg intersection is different from type 3. The 3rd leg in the intersection is positioned in the right side of the entry direction of the priority vehicles. Further, the direction of the priority is also given to the through direction only under this intersection type 4.

- **Intersection Type 5**

Type 5 intersections are also 3 way intersections and the 3rd leg is position right side to the entry direction. This is also similar to the Intersection Type 4, but the priority direction of the green platoon is given for the right turns opposed to the type 4 direction.

- **Intersection Type 6**

This is also a 3 leg signalized intersection as mentioned in intersection types 3, 4 & 5. However, the priority vehicles enter the intersection from the 3rd leg direction which makes it different from the other intersection types. The entry vehicles having the priority need to take the right turns to be categorised under this category.

- **Intersection Type 7**

This is also a 3 leg intersection and the geometrical arrangement of the intersection and vehicle entry direction is similar to type 6. However, the exit direction of the priority vehicles should be left turns from the entry direction, to be categorised under the intersection type 7.

Based on the above identified intersections, it is necessary to develop guidelines when the above non-similar intersections and priority directions are encountered in traffic signal coordination.

3.7.2. Calculation of cycle time for the considered intersections

Individual intersections were first considered in order to proceed with the respective coordination analysis. The first step was to calculate the cycle time of each individual intersections. For the calculation of cycle time, Webster's method was used, addressing the modifications proposed in the Section 3.4.

It is also essential to consider similar cycle lengths or multiples of smaller cycle lengths for all the intersections considered in order to maintain the consistency throughout the intersections. The maximum cycle length for a four leg intersection is kept as 150 seconds, where the same for 3 leg intersection was 120 seconds, based on the guidelines in Section 3.4.

3.7.3. Optimizing the offsets of timing between intersections

The offsets were optimized so that, a continuous green platoon is observed throughout the intersection cluster considered. For determination of offsets, average travel time between the intersections were considered.

3.7.4. Developing Two Matrixes (7x7) with the coordination sub categories

Two matrixes were developed using the seven sub categories identified to account for the effects of peak and off peak time periods. The following traffic and geometrical considerations were considered.

- To obtain peak traffic in the coordinated direction, a volume/capacity ratio of 0.8 was used and to obtain the off peak traffic 0.4 was used
- The non-coordinated direction V/C ratio is kept as 0.4 for peak traffic and 0.2 for off-peak traffic
- The offsets between the two intersections were taken as 20 seconds with an optimum bandwidth ranging from 30-40 seconds for the coordinated direction

Three different delay categories were identified

- Heavy Delay (H) – The mean delay of vehicle is more than 35 seconds per vehicle
- Medium Delay (M) - The mean delay of vehicle is between 20 seconds to 35 seconds
- Low Delay (L) – The mean delay of vehicle is less than 20 seconds

3.7.5. Matrix – For Peak Periods

The matrix was developed considering 2 intersections at a time and simulating the conditions, to obtain the overall delay per vehicle.

For example, if in an intersection a vehicle enters to a type 1 intersection and enters again to a type 1 intersection, the average delay per vehicle for the peak period would be 38s based on the simulations.

In the same way, all the intersections are considered as couples and under the provided conditions the average delay values per vehicle is obtained. The results obtained were tabulated in Table 3-28.

Table 3-28: Matrix for delay values for coordination of non-similar intersection clusters - for peak period

Out / In	1	2	3	4	5	6	7
1	H (38s)	H (50s)	M (21s)	M (28s)	H (51s)	H (39s)	M (35s)
2	M (24s)	M (31s)	M (23s)	M (31s)	H (54s)	H (47s)	H (45s)
3	M (30s)	M (28s)	M (20s)	M (25s)	H (49s)	H (41s)	M (31s)
4	H (36s)	H (49s)	M (28s)	M (29s)	H (48s)	M (26s)	M (30s)
5	M (25s)	H (36s)	M (25s)	M (33s)	H (51s)	H (37s)	H (37s)
6	M (25s)	H (41s)	M (25s)	M (26s)	H (43s)	H (42s)	H (36s)
7	M (22s)	H (37s)	M (22s)	M (29s)	H (46s)	H (37s)	M (33s)

3.7.6. Matrix – For Non-Peak Periods

In the same manner, matrix for non-peak period is also developed and the results are given in Table 3-29.

Table 3-29: Matrix for delay values for coordination of non-similar intersection clusters - for non-peak period

Out / In	1	2	3	4	5	6	7
1	M (26s)	H (40s)	L (17s)	L (18s)	H (38s)	M (28s)	M (26s)
2	L (18s)	M (24s)	L (18s)	M (24s)	H (40s)	M (32s)	M (31s)
3	M (23s)	M (22s)	L (17s)	M (22s)	H (36s)	M (27s)	M (23s)
4	M (24s)	H (36s)	M (26s)	M (22s)	M (33s)	M (21s)	M (22s)
5	L (19s)	M (26s)	M (20s)	M (25s)	M (34s)	M (26s)	M (25s)
6	M (20s)	M (28s)	L (19s)	M (21s)	M (29s)	M (28s)	M (25s)
7	L (18s)	M (26s)	L (18s)	M (23s)	M (31s)	M (26s)	M (25s)

CHAPTER 4 : CASE STUDY APPLICATIONS AND VERIFICATIONS

4.1. Case Study 1: Base Line Road Selected Intersection Cluster

In order to verify the developed guidelines, a case study was carried out on the Baseline Road, which is a main arterial road that runs between Peliyagoda and Kirulapone, for a selected intersection cluster consist of 5 signalized intersections (Figure 4-1). As at present this intersection cluster is operating under fixed time individual intersections, the guidelines related to real time systems are not applied to this case study.

Figure 4-1: Baseline Road Intersection Cluster

The VISSIM software has been calibrated using the calibration parameter values proposed previously in the study. It was assumed that the actual behaviour of road users are shown in the software addressing the differences in vehicle mix, peak-off peak variations, pedestrian behaviour and driver behaviour.

The guidelines developed for the individual intersections under fixed time systems are used for providing the respective traffic and signal conditions. The phasing arrangement, traffic signal timings and the geometrical arrangement were considered and modified according to the guidelines developed in Section 3.4.

The guideline for the selection of intersection clusters was used and all the intersections considered were placed within the clustering index criteria that was developed.

The case study is conducted for verification of traffic signal coordination in non-similar intersection clusters subject to satisfying the guidelines developed.

The analysis of the case study was structured into two parts, existing condition and the simulated condition.

4.1.1. Existing Condition

Through a 1 ½ hour video recording at the 4 different intersections at one particular time and based on a manual classified count for 2 hours (Figure 4-2), the traffic signal and flow data were obtained.

Figure 4-2: Existing Condition of the Baseline Road

The descriptions of the junctions and existing traffic signal timings of the intersections are as mentioned (Table 4-1) (Table 4-2).

Table 4-1: Description of the intersections – Baseline Road Intersection cluster

Intersection	**Description**
AAT Junction	This is a 4 leg junction, where the major road is the baseline road, with no separate right turn lanes. The peak flow is towards Kirulapone in the evening time. The junction currently operates as a 4 phase junction with leading green for right turn phases for major road direction.
Kirimandala Junction	Kirimandala Mawatha junction is a 3 leg junction and the major road direction is the baseline road. The junction currently operates as a 3 phase signalized intersection. In the evening peak period, the downstream queue at the Narahenpita junction affects the performance of the Kirimandala junction
Narahenpita Junction	This is a 4 leg junction, which can be considered as the major junction in the selected intersection cluster, where vehicles from Nawala and Jawatta area would utilize this junction. The Narahenpita junction currently operates as a 5 phase junction, where the major road (Baseline road) right turns are split into two phases along with the respective through movements.
Apollo Junction	Apollo junction is a 3 leg junction. However, this is a minor junction where the major impact is from the vehicles coming from Kirulapone direction. Due to this reason, when coordinated direction is towards Kirulapone, this junction may be neglected for analysis purposes. The exiting vehicles from Apollo Hospital are not allowed to turn right where as a signal is placed for vehicles turning right into the hospital. This arrangement makes it operate as a 2 phase signalized intersection.

Park Road Junction	Park road junction is a 4 leg junction and operates as a 4 phase signalized intersection. The minor roads are operated in a single phase due to the lower amount of vehicles coming from the minor road.

Table 4-2: Existing traffic signal timings

	AAT Junction	**Kirimandala Mw**	**Narahenpita**	**Park Road**
Phase 1	90	45	45	43
Phase 2	23	20	21	10
Phase 3	23	20	39	36
Phase 4	20		30	31
Phase 5			30	
Cycle Time	156	85	165	120

The respective phasing arrangements were also as mentioned (Figure 4-3) (Figure 4-4) (Figure 4-5) (Figure 4-6) & (Figure 4-7).

AAT junction

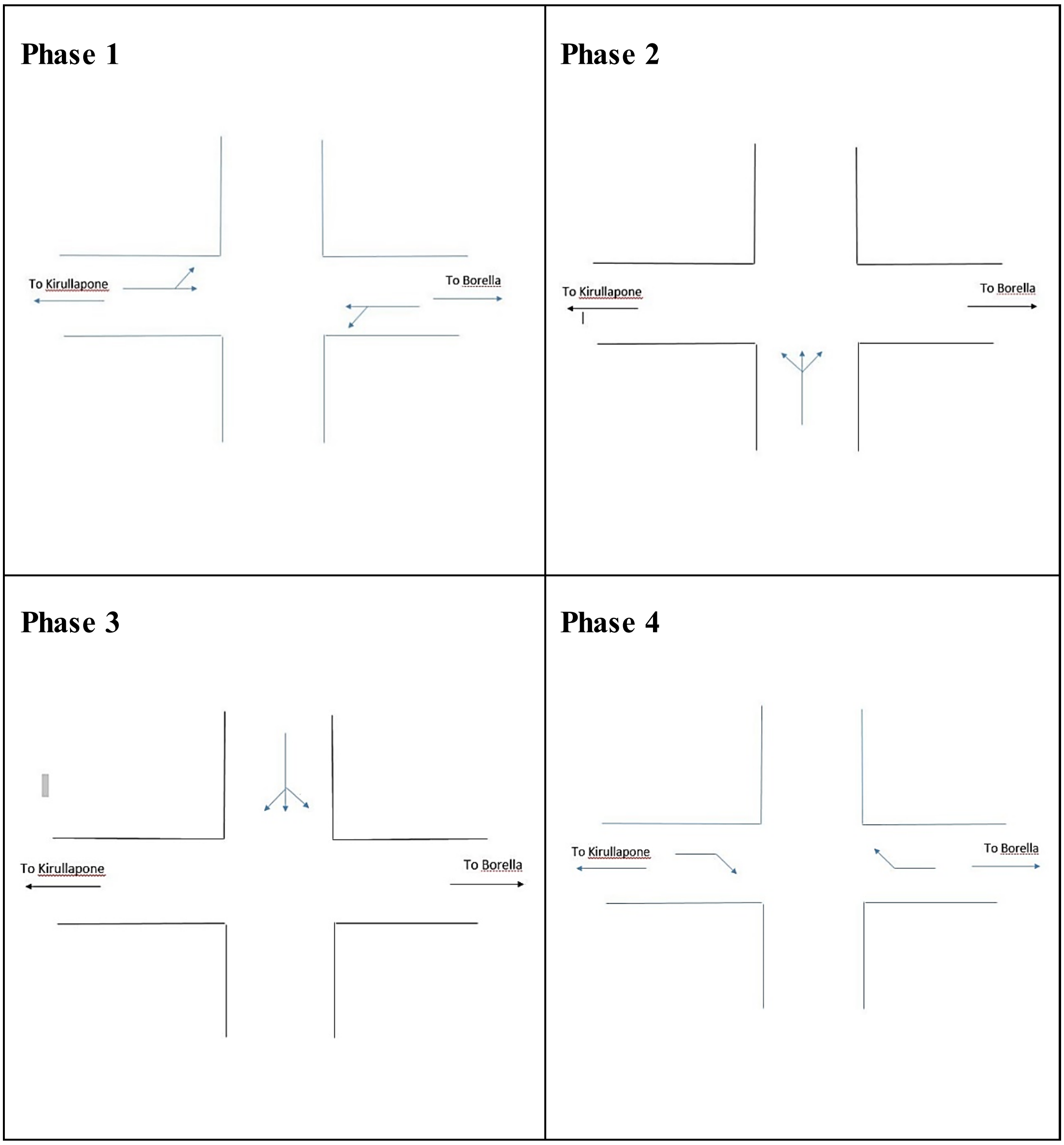


Figure 4-3: Phasing – AAT Junction

Kirimandala Junction

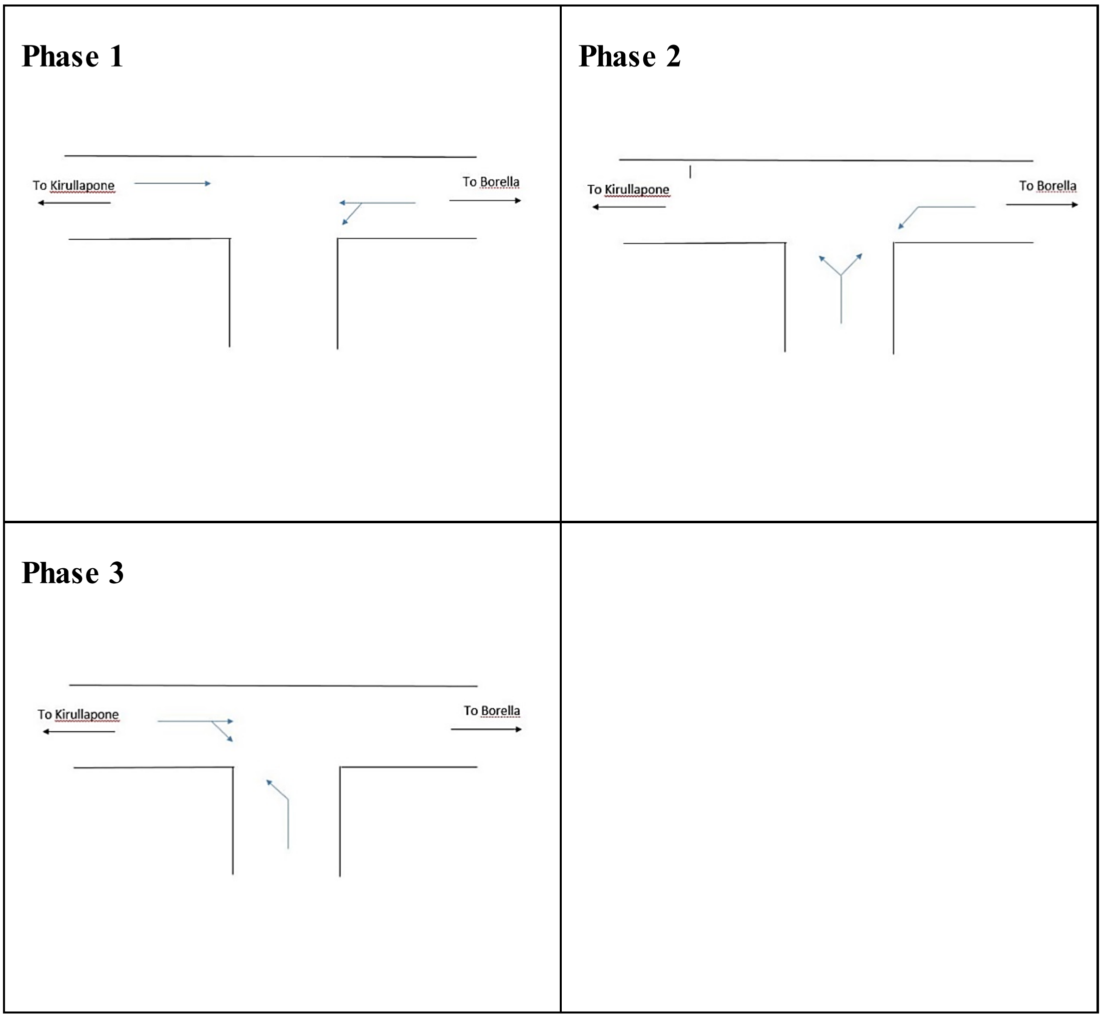
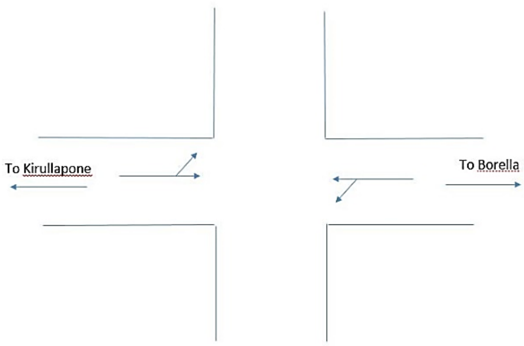


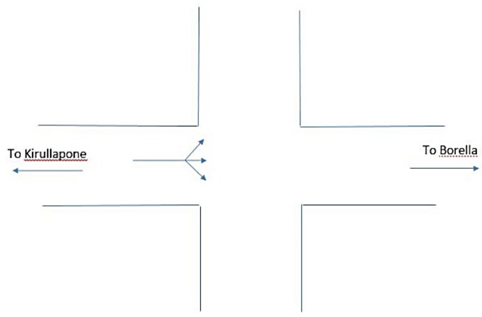
Figure 4-4: Phasing - Kirimandala Mawatha Junction

Narahenpita Junction

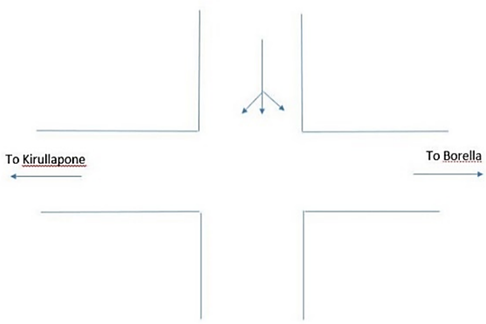
Phase 1



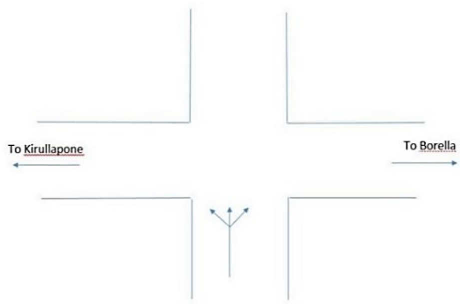
Phase 2



Phase 3



Phase 4



Phase 5

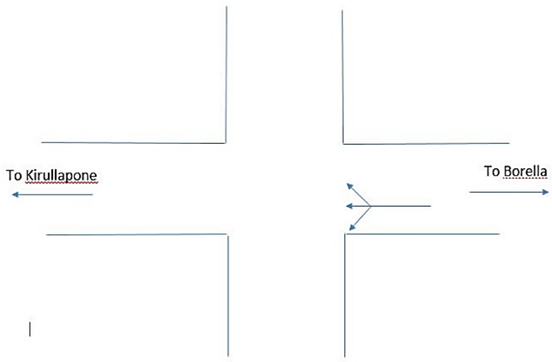


Figure 4-5: Phasing - Narahenpita Junction

Apollo Junction

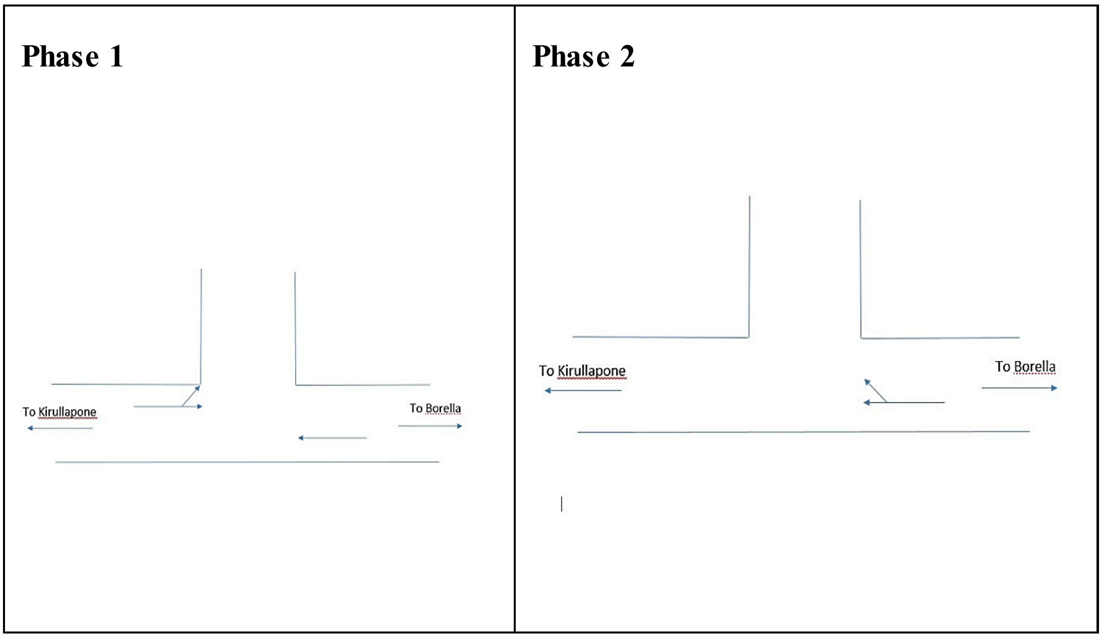


Figure 4-6: Phasing - Apollo Junction

Park Road Junction

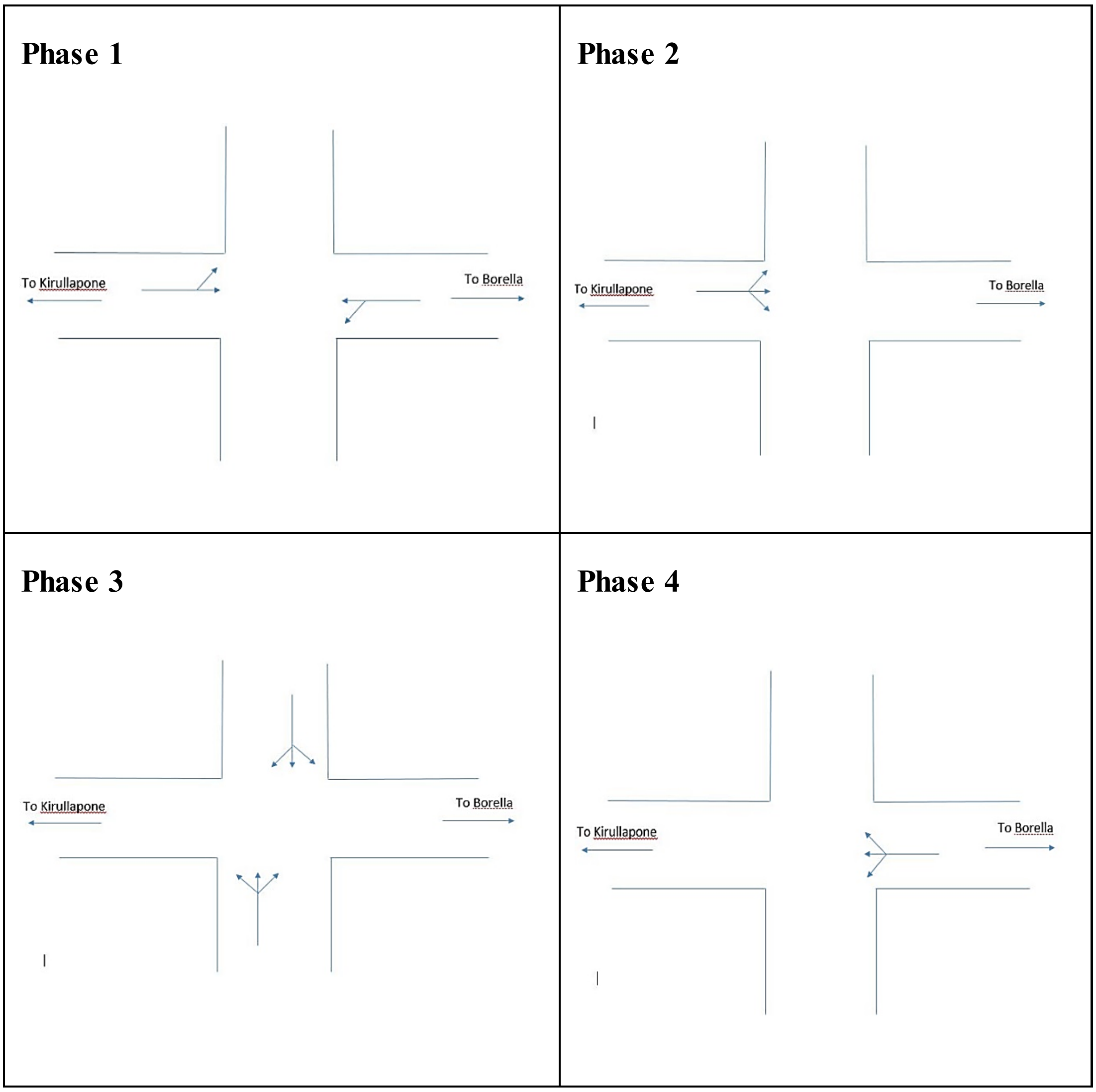


Figure 4-7: Phasing - Park Road Junction

The flow towards Kirulapone at AAT is 2180 vehicles/ hour and the flow towards Borella at Park road junction is 2050 vehicles/ hour and there was no coordination between the intersections at present.

With the obtained information, the existing system was modelled using calibrated VISSIM software (Figure 4-8) (Figure 4-9) (Figure 4-10).

Figure 4-8: Modelling the existing condition

Queue counters, delay detectors, travel time measurements were placed at each intersections and directions in all the approaches.

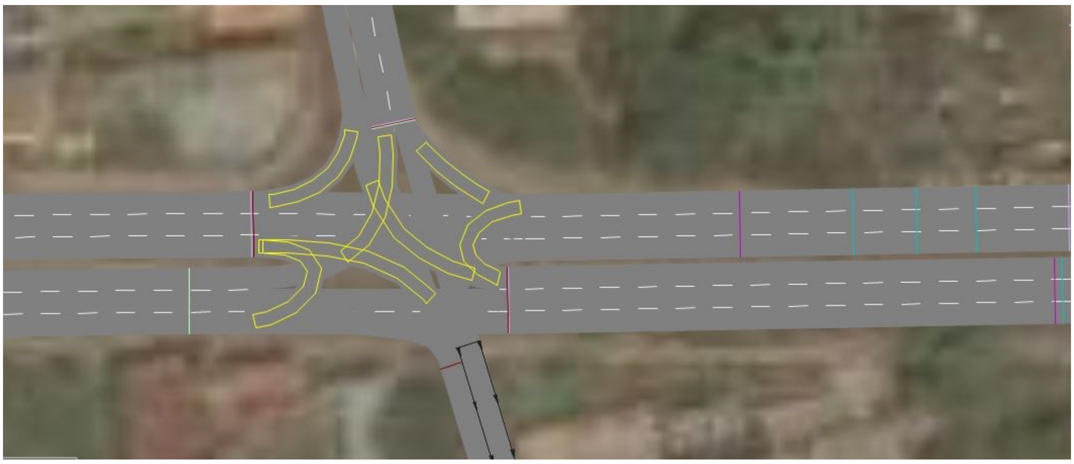


Figure 4-9: Queue Counters Placed

Figure 4-10: Snap shot of the simulation of existing condition

The following results are obtained for the existing condition as in Table 4-3.

Table 4-3: Results for Existing Condition

Delay Measurement	Stop delay(s)	Vehicle delay(s)
1: Park Road to AAT	123	163
2: AAT to Park Road	88	150
3: Park road to Park road intersection	56	70
4: Thimbirigasyaya to AAT intersection	73	83
5: Nawala road to Narahenpita intersection	70	81
6: Kirimandala Mw. to Kirimandala intersection	23	29
7: AAT Kirimandala Mawatha to intersection	150	167

4.1.2. Traffic signal coordinated condition

For coordination purposes, the following criteria were used.

The Approach speed is taken as 35kmph based on the survey results and the offsets are calculated based on the distance between the intersections and approach speed.

The following offsets are considered as in Table 4-4.

Table 4-4: Offsets used

Intersection	Offsets calculated
AAT	0
Kirimandala Mw	31
Narahenpita	51
Lanka Hospitals	71
Park road	88

Proposed signal timings for traffic signal coordination are also given in Table 4-5, calculated based on Webster's method, which is modified based on the guidelines produced in Section 3.4.

Table 4-5: Proposed signal timings

	AAT	Kirimandala Mw	Narahenpita	Park Road
Phase 1	66	32	35	45
Phase 2	18	14	16	15
Phase 3	18	14	25	30
Phase 4	18		22	30
Phase 5			22	
Cycle time	120	60	120	120

With the simulations carried out, the following results were obtained for the coordinated condition (Table 4-6).

Table 4-6: Results for coordinated condition

Delay Measurement	**Stop delay(s)**	**Vehicle delay(s)**
1: Park Road to AAT	72	125
2: AAT to Park Road	63	112
3: Park road to Park road intersection	89	75
4: Thimbirigasyaya to AAT intersection	91	85
5: Nawala road to Narahenpita intersection	52	61
6: Kirimandala Mw. to Kirimandala intersection	14	20
7: AAT Kirimandala Mawatha to intersection	132	152

When the Mean delays are compared significant delay reductions were obtained in most of the directions as mentioned in Table 4-7.

Table 4-7: Comparison of mean delays

	Mean Vehicle delay (s)		
Delay Measurement	**Existing**	**With signal coordination**	**Delay reduction**
1: Park Road to AAT	163	125	23%
2: AAT to Park Road	150	112	25%
3: Park road to Park road intersection	70	75	-7%
4: Thimbirigasyaya to AAT intersection	83	85	-2%
5: Nawala road to Narahenpita intersection	81	61	25%
6: Kirimandala Mw. to Kirimandala intersection	29	20	31%
7: AAT Kirimandala Mawatha to intersection	167	152	9%

Further, to this to get a good understanding about the effectiveness of traffic signal coordination, the following queue length reduction values were also obtained (Table 4-8).

Table 4-8: Queue Length Comparison

Queue Counter	Queue Length(m)		Reduction
	Existing Condition	**Signal Coordination**	
Park junction to Borella	150	82	45%
AAT to Kirulapone	130	75	42%
Park by road	58	63	-9%
Thimbirigasyaya by road	67	72	-7%
Nawala by road	47	34	28%
Kirimandala by road	6	4	33%
AAT Kirimandala by road	120	121	-1%
AAT Thimbirigasyaya by road	83	79	5%

Queue length reductions observed through the simulation further support the reduction in delay.

4.1.3. Reference to the Matrix developed

According to the Matrix developed in Table 3-28, the following sub categories were identified for the peak period, where the green platoon was provided for the major road direction from Borella to Narahenpita.

- 1→3→1→4→1

The results obtained are mentioned in Table 4-9, which were similar to Matrix Results when compared in Table 3-28.

Table 4-9: Delay Results – Baseline Road Intersection Cluster

Intersection	Delay From Matrix	Delay From Simulations
AAT – Kirimandala	M (21s)	21s
Kirimandala – Narahenpita	M (30s)	31s
Narahenpita – Apollo	L (18s)	18s
Apollo- Park Road	H (36s)	35s

The verification was done using a non-parametric test, Wilcoxon Signed Rank Test, where the results are significant at a significant level of 5%.

Therefore, the developed guidelines in the research study are verified from the 1st case study conducted referring to the Baseline Road Traffic Signal Coordination.

4.2. Case Study 2: Kadawata Expressway Entrance Intersection Cluster

Another case study was conducted at the Kadawata Expressway Entrance Intersection Cluster, where six signalized intersection were observed within a span of 2 km distance (Figure 4-11). The description of the intersections are provided in Table 4-10. This intersection cluster was important to consider because this was to operate under real time systems, under semi actuated conditions.

4.2.1. Development of the Intersection cluster

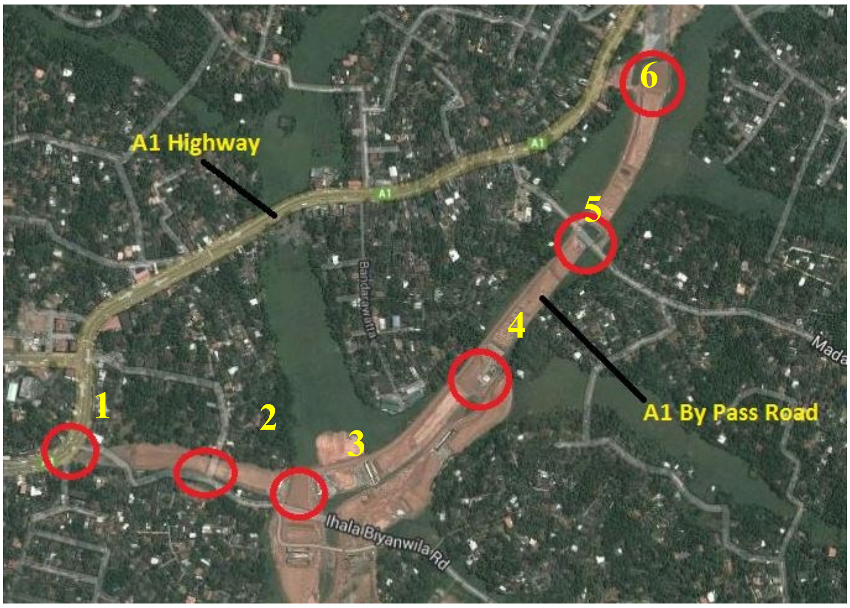


Figure 4-11: Kadawata Intersection Cluster

With referencing to guidelines developed in study for VISSIM calibration in Section 3.1, individual intersection for fixed time systems in Section 3.4, for real time systems in Section 3.5 and for clustering of intersection in Section 3.6, the case study was conducted.

Table 4-10: Description of Intersections – Kadawata Intersection Cluster

Intersection	**Description**
Intersection 1 (Kadawata A1 intersection)	This is a 3 leg junction, with no separate right turn lanes, where the major road considered is the Kadawata by pass road. The peak flow is towards Kirillawala in the evening time. The junction currently operates as a 3 phase junction, where pedestrians are provided in all three directions.
Intersection 2 (Shramadana Mw Intersection)	This junction is a 4 leg junction and the major road direction is the Kadawata bypass road. The minor road (Temple road) is only given an amber blinking signal, due to lower traffic volumes The junction currently operates as a 3 phase signalized intersection. Significant number of vehicles are entering and exiting from Shramadana Mawatha. Pedestrian phase is provided only to cross the bypass road.
Intersection 3 (Expressway exit)	This is again 3 leg junction, where the expressway exit vehicles can turn towards Kadawata or Kirillawala. The junction is currently operated as a 2 phase signalized intersections, since no vehicles are allowed into the expressway from this junction.
Intersection 4 (Expressway entry)	This is a 3 leg junction, where the expressway entry vehicles coming from Kirillawala and Kadawata directions should enter the expressway from the junction. This junction is also operating as a 2 phase junction, since no vehicles from expressway are exiting from here.
Intersection 5 (Bandarawatta Junction)	This junction is also a 4 leg junction, and operates as a 3 phase signalized intersection. In the major road, leading green for right turns only phase is provided to reduce cycle time and delays. The

	minor road both directions are provided in a single phase due to the lower vehicular volumes expected.
Intersection 6 (Eldeniya Junction)	This is a 3 leg junction and operates as a 3 phase junction. Pedestrians are allowed only in two phases across the by-pass road and A1 road (towards Kadawata direction).

Introducing real time system through loop detectors

In order to introduce the real time aspects, loop detectors are used for minor road movements and some major road movements as mentioned in Figure 4-12, Figure 4-13, Figure 4-14, Figure 4-15 and Figure 4-16, along with the direction where the actuation is taking place.

Figure 4-12: Intersection 1: towards Kandy Direction

Figure 4-13: Intersection 2: Shramadana Mawatha

Figure 4-14: Intersection 3: Expressway Exit

Figure 4-15: Intersection 4: Expressway Entry

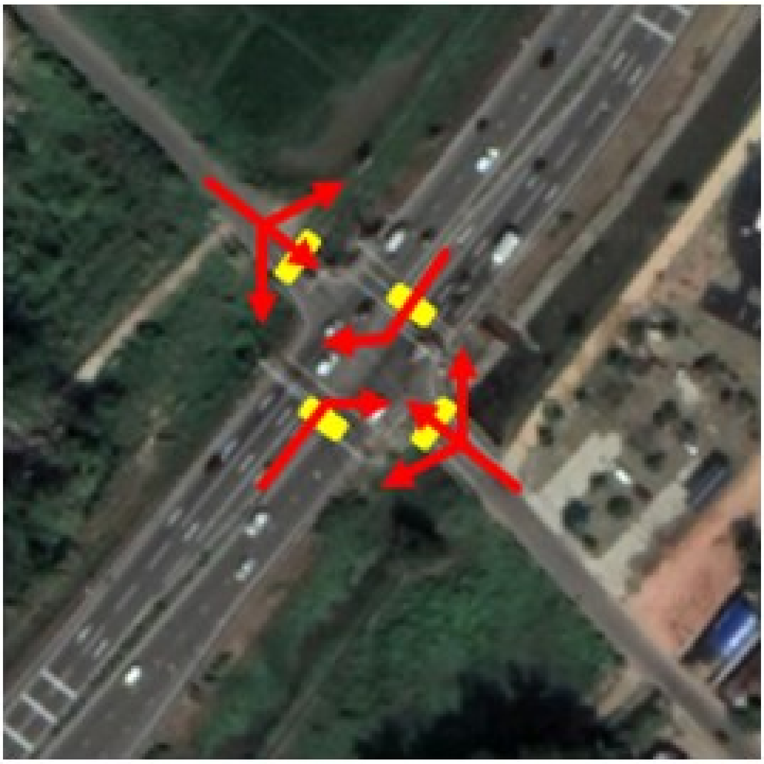


Figure 4-16: Intersection 5: Major Road right turns and Minor Road

With the real time detectors and using the developed guidelines in Section 3.4 and Webster's method, the signal phases were developed (Figure 4-17), (Figure 4-18), (Figure 4-19), (Figure 4-20), (Figure 4-21) & (Figure 4-22) and signal timings (Table 4-11) were obtained, for the 6 intersections considered at this Semi Actuated Intersection Cluster, where not all approaches are controlled but only selective approaches.

For Coordination purposes, the respective offsets and coordinated phases are also calculated and identified using the guidelines proposed in Section 3 (Table 4-12)

In addition to this, important criteria are also developed for real time traffic management which is given in Table 4-13.

Phasing Arrangement for Intersection 1

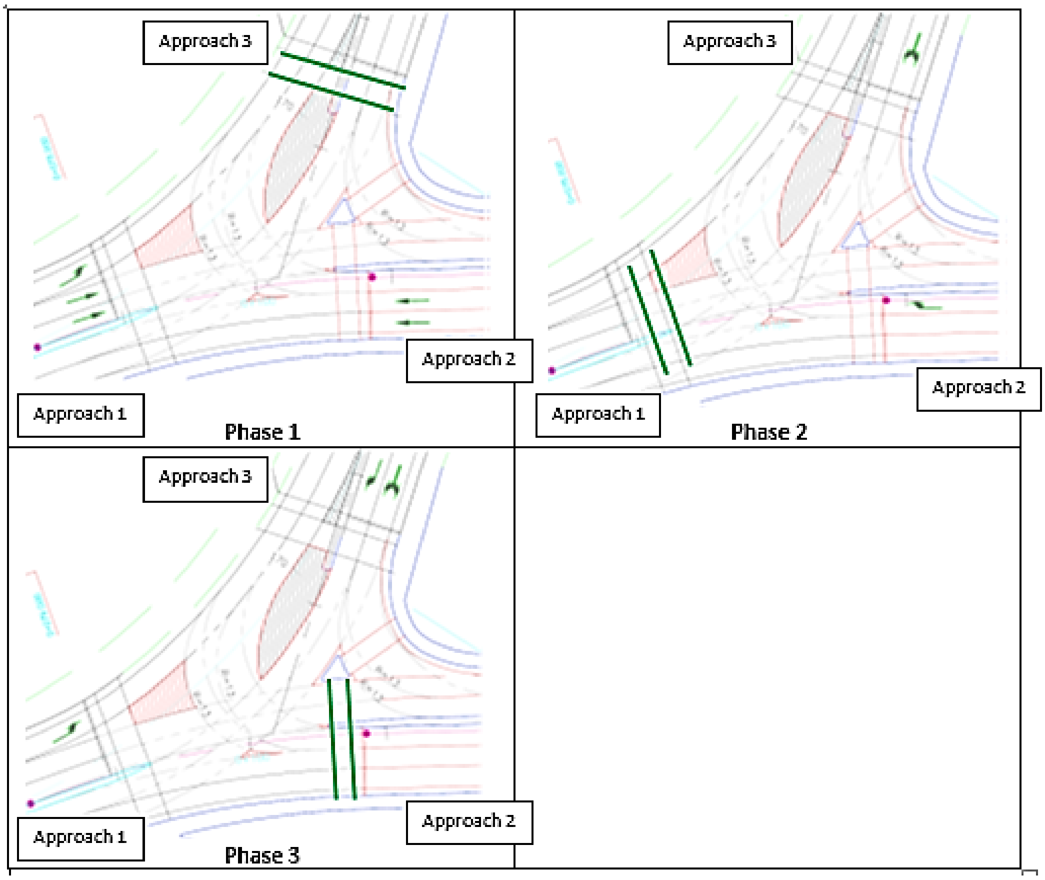


Figure 4-17: Phasing Arrangement for Intersection 1

Phasing Arrangement for Intersection 2

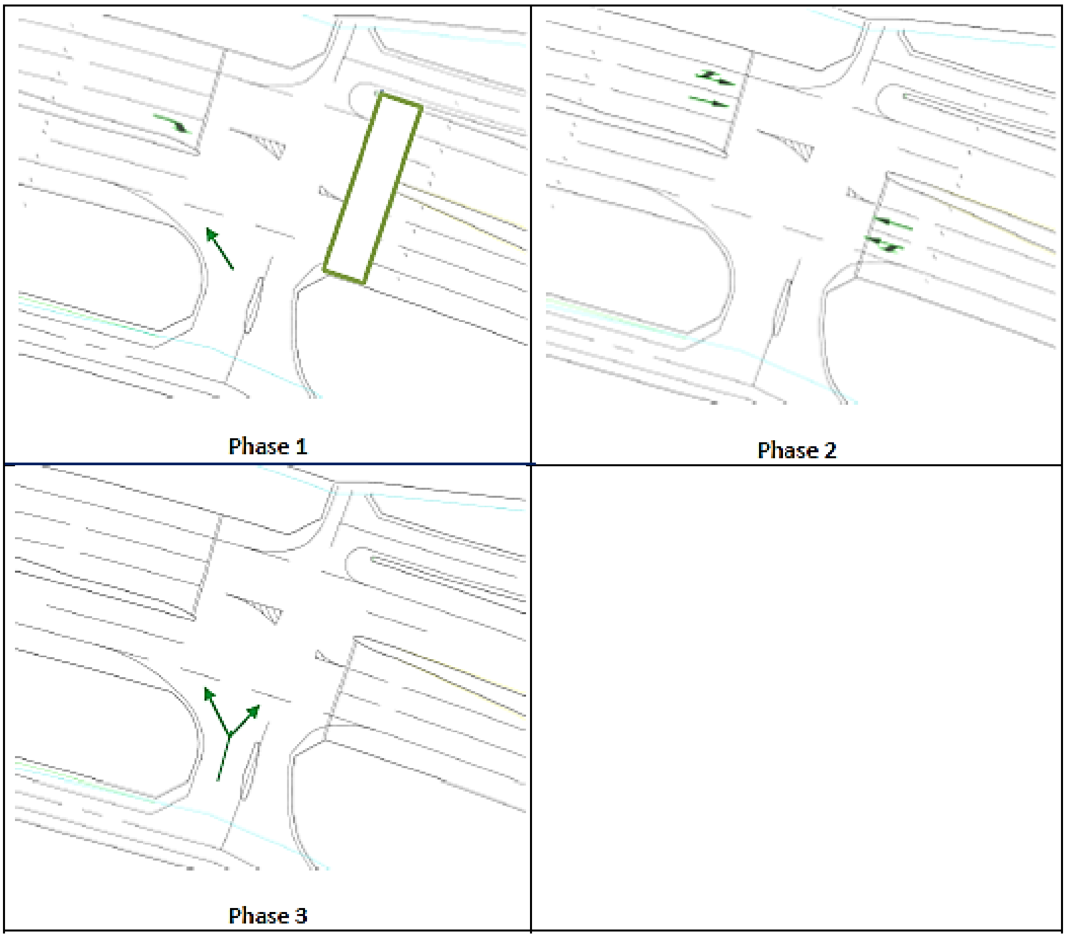


Figure 4-18: Phasing Arrangement for Intersection 2

Phasing Arrangement for Intersection 3

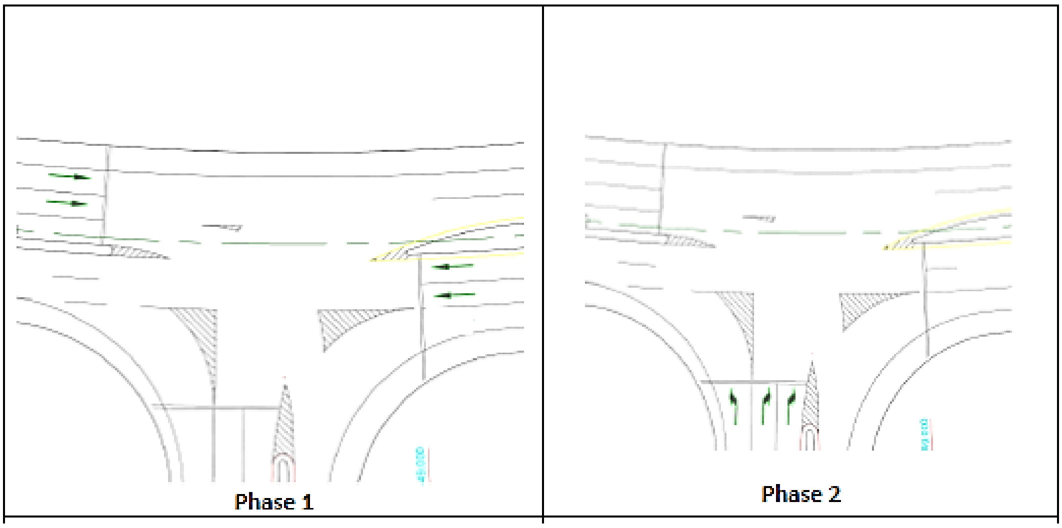


Figure 4-19: Phasing Arrangement for Intersection 3

Phasing Arrangement for Intersection 4

Figure 4-20: Phasing Arrangement for Intersection 4

Phasing Arrangement for Intersection 5

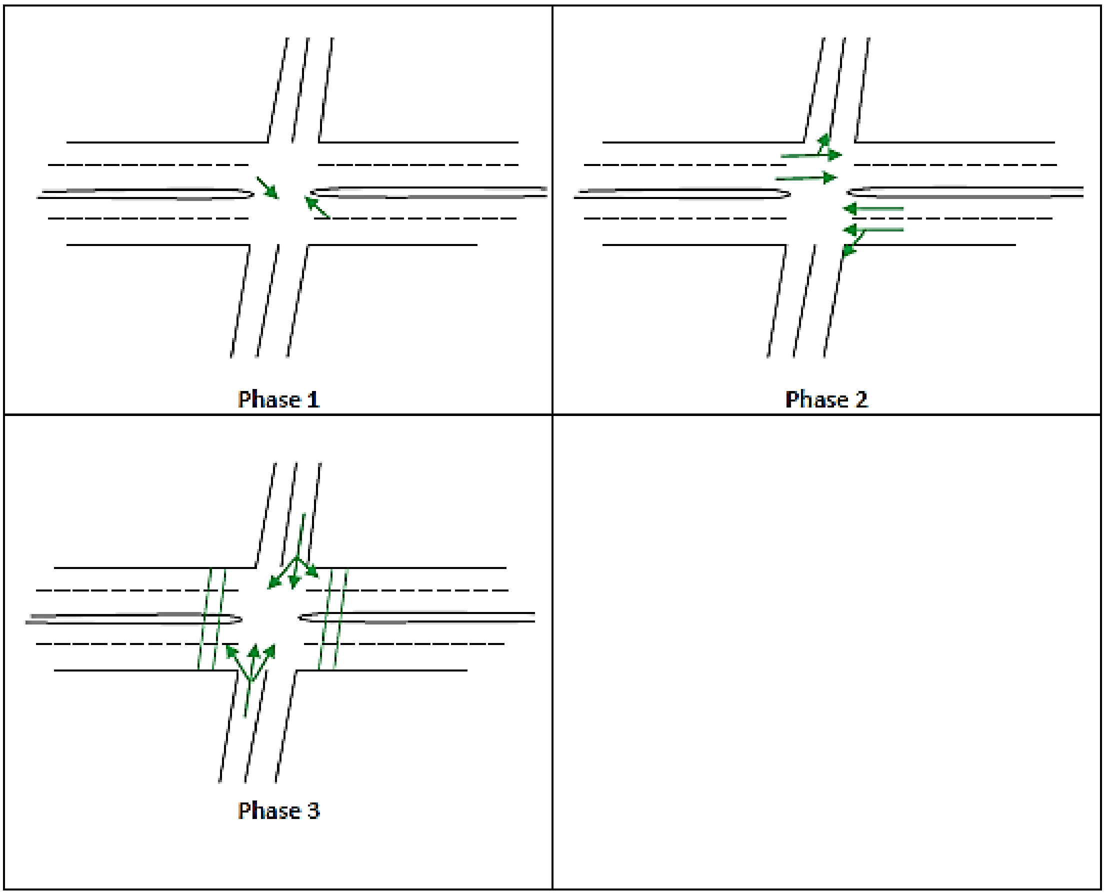


Figure 4-21: Phasing Arrangement for Intersection 5

Phasing Arrangement for Intersection 6

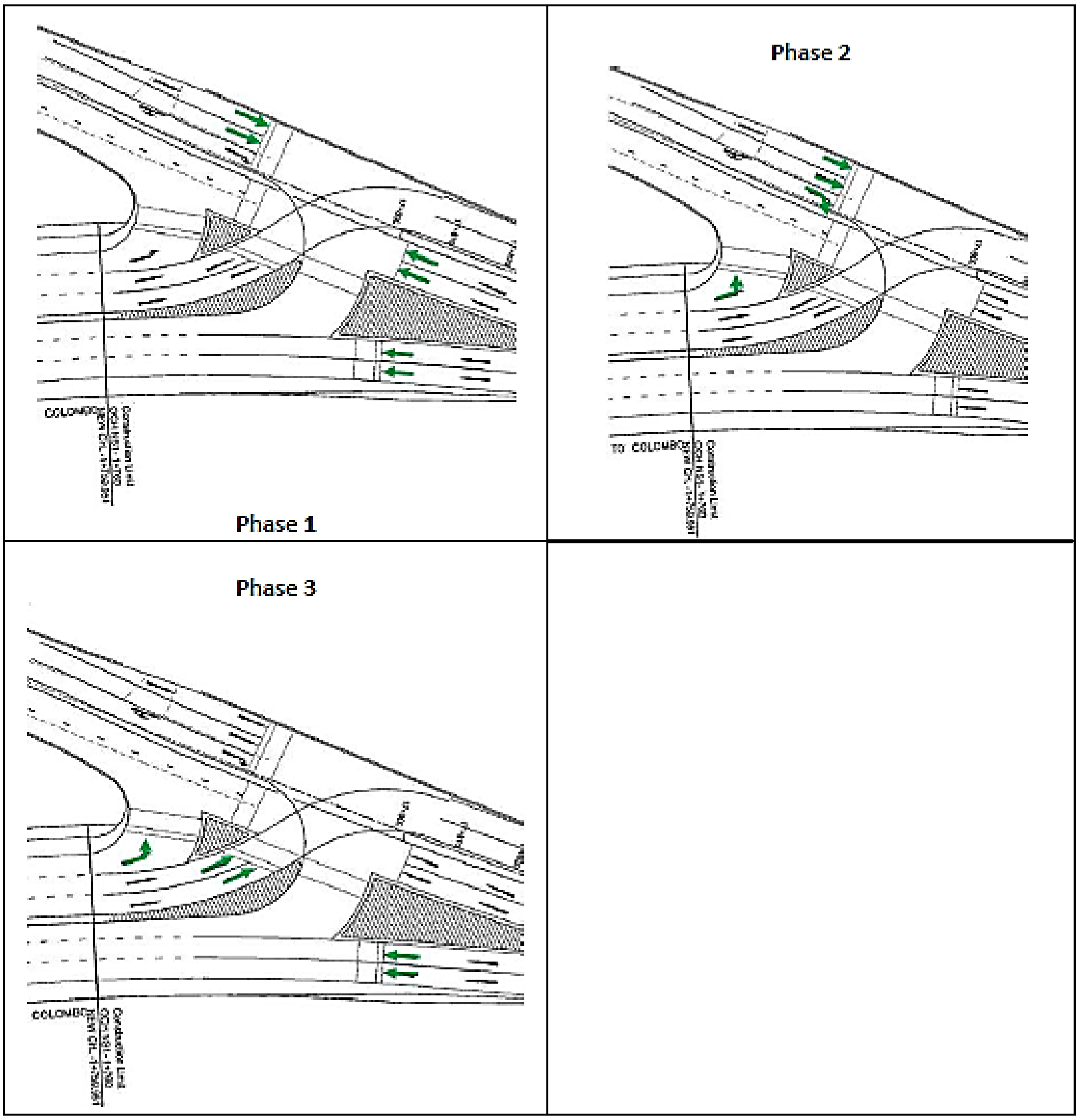


Figure 4-22: Phasing Arrangement for Intersection 6

Table 4-11: Signal Timings for the Intersections

	Phase 1				Phase 2				Phase 3				Cycle time
	Red/Amber	**Actual Green**	**Amber**	**All red**	**Red/ Amber**	**Actual Green**	**Amber**	**All red**	**Red/Amber**	**Actual Green**	**Amber**	**All red**	
Intersection 1	1	37	3	2	1	8	3	2	1	13	3	2	**76**
Intersection 2	1	18	3	1	1	34	3	1	1	9	3	1	**76**
Intersection 3	1	16	3	2	1	10	3	2	-	-	-	-	**38**
Intersection 4	1	16	3	2	1	10	3	2	-	-	-	-	**38**
Intersection 5	1	10	3	1	1	31	3	1	1	20	3	1	**76**
Intersection 6	1	20	3	2	1	9	3	2	1	29	3	2	**76**

Table 4-12: Offsets and Coordinated phases

	Evening Peak		**Morning Peak**	
	Coordinated phase	**Start time of the coordinated phase in the master clock**	**Coordinated phase**	**Start time of the coordinated phase in the master clock**
Intersection 1	Phase 1	0	Phase 1	141
Intersection 2	Phase 2	30	Phase 2	101
Intersection 3	Phase 1	43	Phase 1	88
Intersection 4	Phase 1	79	Phase 1	52
Intersection 5	Phase 2	108	Phase 2	33
Intersection 6	Phase 3	141	Phase 3	0

Table 4-13: Criteria for Real time traffic signal coordination

Intersection	Decision Criteria	Impacts to other intersections	Remarks
1	If loops in approach 1 is detecting vehicles, the maximum additional extension time of green phase (Phase 1) is 3 seconds according to the developed guideline for off peak periods, which is 5% green time extension.	Intersections 2, 5 and 6 should be given the same extensions for the coordinated phase. Intersection 3 and 4 should be given half of that value.	Approach 2 and 3 loops are not considered, because only coordinated direction is considered, Maximum extension is only 3 seconds because otherwise the overall intersection cluster delay is increasing according to the developed guidelines
2	If no vehicles are detected in right turn to Shramadana Mw, or from Shramadana Mw or pedestrian push buttons, give the timings of phase 1 and 3 to the Phase 2 as well and after that start the phase 2 again	No need to change the other intersection timings	This would facilitate the vehicles from South to North direction, who missed the coordination and also who have turned from approach 2 in intersection 1

3,4	If no vehicles are detected in both the loops entering or exiting expressway, give the phase 2 timings to phase 1 and continue the cycle	No need to change the other intersection timings	This would increase the bandwidth to around 25-30 vehicles which is now around 12-15 vehicles
5	If no vehicles or passengers are detected on the minor road, for right turns ,give the timings of phase 3 and 1 to the Phase 2 as well and start the phase 2 again	No need to change the other intersection timings	
6	Not applicable for the evening period		

4.2.2. Reference to the off-peak Matrix developed

For the considered traffic signal timings and phases along with the coordinated criteria, the intersection cluster was simulated for the off-peak period after identifying the following sub category of signalized intersections.

- 3→1→4→4→1→6

The obtained results were compared with the delays obtained from the Matrix for Off-Peak Period in Table 3-29, as mentioned in Table 4-14.

Table 4-14: Delay Results – Kadawata Interchange Intersection Cluster

Intersection	**Delay From Matrix**	**Delay From Simulations**
1→ 2	M (23s)	22s
2→ 3	L (18s)	21s
3→ 4	M (22s)	23s
4→ 5	L (17s)	18s
5→ 6	M (28s)	29s

The verification was done using a non-parametric test, Wilcoxon Signed Rank Test, where the results are significant at a significant level of 5%.

Therefore based on case study at Kadawata Intersection Cluster, it has been concluded that the developed guidelines are verified and the same delay values were obtained from the simulations and the matrix, which is also verified using non-parametric test, Wilcoxon Signed Rank Test.

CHAPTER 5 : DISCUSSION

With the ever increasing traffic congestion, the traffic management becomes difficult and complex day by day. Traffic signals in particular is of utmost importance, when managing the traffic in terms of reduction of delays as well to increase the safety of road users. The development of technology has produced different types of vehicles with different characteristics making the roads and behaviour more complex in nature.

In such a background, to control the traffic and behaviour, traffic signals also need to be carefully modified and updated. With the varying driver behaviour patterns experiencing, peak-off peak variation, vehicle mix differences, pedestrian behaviour and geometrical differences, a theory which is applicable to one situation may not be directly applicable to another situation. Further, no proper guidelines are produced when it comes to complexities, so the adverse effects such as delays and congestion are increasing day by day.

The research identifies the above as the research gap and aims to develop guidelines for traffic signals and traffic signal coordination in the fixed time as well as real time systems. The developed guidelines should address the effects of driver behaviour, vehicle mix, peak off peak variations and geometrical differences also.

One of the main limitations observed through literature is the isolation and idealistic nature of vehicles and infrastructure expected in implementing the theories developed. However, in a practical context, such perfect systems are quite rare and thus more practical approach is paramount.

Having such an understanding, the methodology of the research is divided to few key phases as mentioned and the respective results are generated.

A. Establishing Preliminary Requirements

1. Calibration of the VISSIM traffic micro simulation software
2. Verification of the calibrated software for practical applications
3. Quantification of travel time savings and updating the Value of Time for evaluation of transportation projects

B. Traffic Signal Design and Coordination

Stand Alone Traffic Signals

4. Development of guidelines for Individual intersections for fixed time systems
5. Development of guidelines for Individual intersections for real time systems.

Traffic Signal Coordination

6. Identification of the traffic signal coordination potential for intersections
7. Development of guidelines for traffic signal coordination for non-similar intersection clusters

5.1. Calibration of the VISSIM traffic micro simulation software

As for the first step, the research focussed on calibration and validation of VISSIM traffic micro simulation software for the context of Sri Lanka in order to practically apply the software for various kinds of situations. For calibration purposes, Katubedda Junction, which is a 3 leg signalized junction is used and for validation purposes Kesbewa Junction and 22 other junctions were used.

This has been one of the preliminary requirements for the research, and the findings of the research can be used in most of the urban areas in Sri Lanka, where urban driver behaviour patterns are expected. Since there were no proper calibration for the micro simulations for the practical context, the future researchers and practitioners can directly use the calibrated parameter values presented for their micro simulation studies.

Limitations

The parameters developed in this research are only considering the Colombo and Gampaha Districts of Sri Lanka, where the driver behaviour may change when it comes to rural areas. Therefore, additional site specific calibration should be carried out for VISSIM microscopic simulation software when it is applied for rural areas, where different driver behaviour patterns are expected.

Further to this, the research focuses on queue length as the main calibration measurements. Future research can be conducted using delay as a calibration parameter, since the main output that is expected is the delay. However, proper instruments and methodology should be available for conducting the research when observing the delays.

Another parameter that can be used for calibration is the vehicle travel time and speed data that can be extracted from google traffic. The interested researches can use these data and carry out furthermore calibration in the future.

Further, the 9 parameters obtained was based on literature and engineering judgement where, VISSIM does have various other parameters, which can be used for complex situations involving with police control and dynamic traffic.

5.2. Verification of the calibrated software for practical applications

The calibrated and the validated VISSIM traffic micro simulations software, was used for practical applications in a signalized intersection, considering Orugodawatta Junction. Since the Orugodawatta junction is experiencing high amount of U turns, a separate opening for U turns is proposed before the junction and the VISSIM traffic simulation software was successfully used to test the applicability of the separate openings.

The specific results of the application of micro simulation software can be used for the intersections which have high amount of U turns creating unnecessary delays. The results will guide the researchers and practitioners on the amount of delay reduction that can be obtained depending on the amount of U turns to be eliminated.

More importantly, this has successfully applied the calibrated VISSIM micro simulation software for the local context paving way for more advance framework and guidelines development. This has also proved that properly calibrated VISSIM micro simulation software can be used for furthermore applications in future researchers.

Limitations

The above research has addressed the delay reductions at the Intersection itself that can be obtained by eliminating the U-turns and providing separate openings for U-turns. It has accounted the complications arising at the separate openings away from the intersection as well through simulations.

However, methodologies to calculate the optimum distance for providing such separate openings were not extensively addressed in the research. For example, Orugodawatta junction did not have a separate right turn bay. If the intersections do have a separate right turn bay, the optimum distance would be changed due to the other complications arising.

Further, geometric details that should be considered in providing such separate openings for U-turns should also be addressed in the future studies.

5.3. Quantification of travel time savings and updating the Value of Time for evaluation of transportation projects

As for the above analysis of eliminating U turns in a signalized intersection using VISSIM software, it was necessary to quantify the travel time into a monitory value to understand the gravity of the savings. Further to this, the currently used value of time for the Sri Lankan context was obtained as out dated and a reliable estimate for transportation projects was essential.

The value of time estimated in the research are of utmost importance for the transportation engineering field especially in the occasions where transportation meets economy science. The importance of the estimates are ranging from simple traffic management initiatives to complex mega scale projects. The results are of utmost importance to the entire traffic engineering field since a true and reliable estimate for value of time can save millions of money.

However, this section is analysed separately in the Appendix B, since this guideline is to be provided as a standalone guideline, which can be utilized in any research and practical application when necessary.

5.4. Development of guidelines for Individual intersections for fixed time systems

Having a quantification parameter for vehicle delays such as Value of Time as well as a properly calibrated and validated VISSIM traffic micro simulation software, which comes under preliminary requirements, the next step was to develop guidelines for signalized intersections and traffic signal coordination using theoretical and simulation framework.

As for the first step of traffic signal coordination, individual intersections are considered for fixed time signal systems. The guidelines to be produced are divided into 3 types such as guidelines for Geometry & Phasing Arrangement, for traffic signal timings and for pedestrians.

The guidelines for Geometry and Phasing Arrangements are further sub divided to three scenarios such as no separate right turn lane available scenario, separate right turn lane available scenario and separate left turn lane available scenario. For each of the scenarios, specific guidelines are developed which can be used for different traffic flow scenarios. This will minimize the confusions that can arise in setting up the phasing arrangements and also will minimize the user frustration also since an established guidelines are produced to be utilized.

The guidelines for traffic signal timings would also facilitate smooth traffic flow operations and can save unnecessary delays that is experienced in the present context. The guidelines for pedestrians would also increase the safety of the intersections making roads a better place for all the road users.

5.5. Development of guidelines for Individual intersections for real time systems.

With the development of technology, it was desirable to move into dynamic real time traffic systems, therefore guidelines were produced for real time traffic engineering systems also.

The research results highlight the importance of providing maximum green extensions for real time traffic signals based on the different traffic flows. The methodology comprised with a combination of theoretical and simulation based approach. In order to ground the theoretical aspects of traffic signal designs, Webster's method was used for the traffic signal design calculations and Webster's delay estimation formula was used for estimating the vehicular delays.

4 scenarios were considered for the study, with the volume/capacity ratios of major and minor volumes as 0.8 & 0.3, 0.8 & 0.7, 0.6 & 0.3 and 0.4 & 0.3 respectively to account for the traffic flow differences in major and minor road directions. Selection of these 4 scenarios were crucial for the analysis, which is entirely based on the surveys conducted and similar traffic flow patterns observed.

In order to clearly identify the effects of vehicular flow variations which is accounted under real time traffic situation, a green time split was introduced ranging from -15% to +15% at an interval of 5% within the range.

With the obtained 4 categories of traffic flows, theoretical and simulation based analysis is conducted, with the varying green split percentages, keeping vehicle delay minimization as the optimization objective. For each of the scenario considered, the green split where the lowest overall delay per vehicle observed, has been selected as the maximum green extension time to be provided for the respective traffic flow situation considering both the major road direction and minor road directions.

Limitations

The results can directly be applied if any real time traffic management systems are planning to be implemented. However, it is important to note that real time is not only about extension of maximum green but you can do various changes to a traffic signalized system, whereas the research only focussed on extension of green time which is the most crucial out of all.

5.6. Identification of the traffic signal coordination potential for intersections

The research tries to develop a parameter called "Clustering Index" which is defined as the ratio between the coordinated direction flow and travel time between the intersections. Most of the previous research used distance as one of the parameters, but it is continuously argued in the research that distance might not be a reliable parameter for intersection clustering. Some of the methods practically available are also for isolated intersections and ideal scenario, where the development countries do not have such planned road networks. Therefore the research results are of utmost importance as the first step of identification of potential for intersection clustering.

As per the results it is recommended to cluster the intersection which have more than the minimum values that is recommended. The research identifies volume over capacity Ratios ranging from 0.5 to 0.9 and obtains the minimum values for clustering index ranging from 30 to 83.

The results are also referenced and addressed in the case studies conducted in Chapter 4, which also satisfy the requirements for intersection clustering. It is important to note that results for clustering index values obtained account for both flow and the travel time, where travel time can be a function of various parameters such as distance, access point in between.

Limitations and further research areas

The research considers the coordinated direction flow and travel time as the two main parameters when determining clustering index for non-similar intersections are encountered in traffic signal coordination, where different guidelines were developed for other situations. The variation of travel time can also be considered as a major parameter in intersection clustering for future researches also.

5.7. Development of guidelines for traffic signal coordination

Delays at intersections can be a result of lack of coordination when the signalized intersections are closely spaced. Although few methodologies and techniques are developed for the coordination of similar types of intersections, no proper guideline is developed when non-similar intersections are to be coordinated. The research focuses on development of a guideline for traffic signal coordination when non-similar intersections are encountered. Further, the guideline is modified for traffic mix, driver behaviour and peak-off peak variations to establish a unique guideline irrespective of the traffic flow and its characteristics.

The results of the research obtained are entirely based on the 7 sub categories that were identified. The identification of the 7 basic sub categories was very crucial in the study and has utilized considerable amount of time, since it is very important to make sure that all the possible scenarios are shown in the sub categories selected.

For each intersection, the optimum cycle time was calculated and modified to suit for the coordination purposes with similar cycle time or multiples of smaller cycle length. The simulation was conducted for two situations, for Peak period and for off peak period. After identification of 3 delay categories, showing heavy, medium and low delays, the intersections were simulated considering the intersections one by one. With the obtained results, two 7x7 matrixes were developed for the peak and off peak situations.

The developed matrixes were verified considering real life intersection clusters, using Baseline Road and Kadawata signalized intersection clusters for peak and off peak periods which is discussed in Chapter 4.

In traffic signal coordination in non-similar intersection clusters, it is always advised to start from the individual intersections. If the phasing arrangements and timings are properly established based on the guidelines produced in the research, the base is set up for traffic signal coordination.

The properly developed traffic signal designs for individual intersections can further minimize the confusion and frustration of road users. When it comes to traffic signal coordination, it is always advised to identify the intersections which should benefit from clustering based on the guidelines produced for clustering index. The properly identified intersection clusters can be referenced to the peak and off-peak matrixes developed to identifying the intersection sub-categories and priority directions to harness the maximum benefits of traffic signal coordination in fixed and real time systems.

CHAPTER 6 : CONCLUSIONS AND RECOMMENDATIONS

The research was conducted to develop a set of framework for traffic signals and traffic signal coordination, in a background where traffic signal coordination becomes complex with the involvement of non-similar intersection clusters, varying driver behaviour patterns, vehicle mix, peak – off peak variation and pedestrian behaviours. The research first sets out the preliminary requirements to be established for the conduct of the detailed research.

6.1. Establishing preliminary requirements

The research calibrates and validates the VISSIM micro simulation software for the local context so that the properly calibrated software could be utilized for further analysis using the calibrated parameters.

- Average Standstill distance - 1.5
- Additive Part of safety distance - 1.5
- Multiplicative part of safety distance - 2
- Distance standing (0kmph) - 0.5
- Distance driving (50kmph) - 0.75

When the traffic flow conditions are considerably different, it is recommended to first simulate the existing traffic flow condition for the respective intersections.

The research further verifies the calibrated model conducting an application for a scenario, in implementing a separate entrance for U turns, for the junctions which has considerable amount of U turns.

As per preliminary requirements, it was further required to establish a reliable estimate for value of time. This was placed as a separate standalone guideline to be utilized in the research and practical application when necessary (See Appendix B).

Having the proper preliminary requirements for traffic signal and traffic signal coordination being established, proper guidelines are developed using a theoretical approach and also verified using case studies.

6.2. Guidelines for fixed time traffic signal systems

The following guidelines are developed based on the following sub categories

Guidelines on phasing arrangement based on geometric details

- When no separate right turn lane is available, for the cases where right turn percentage is less than 15%, phasing arrangement, which is providing leading green for right turns only phase is recommended and for right turn percentages more than 15%, phasing arrangement, which is providing complete through plus right turn phase is recommended.
- If separate right turn is available, go for leading or lagging green for right turns only phase and if right turn % is significantly smaller, it can be accommodated at the end of Amber
- It is recommended to provide a separate left turn, if the amount of left turns expected are more than the equivalent per lane amount of through movement. If separate left turn lane is provided amber blinking light is recommended if no adverse pedestrian and geometric details are observed.

Guidelines for traffic signal timings

- Maximum cycle time should be kept as 150 seconds for 4 phase intersections except under very special circumstances
- Maximum cycle time should be kept as 120 for three phase intersections except under very special circumstances

6.3. Guidelines for real time traffic signal systems

- In a real time system, it can be concluded that maximum green extension ranging from 5% to 10% can be provided when V/C ratios of major road directions are at higher values such as 0.6 and 0.8 and the minor road V/C values are around 0.3, which can yield around 5% - 7% overall delay reduction per vehicle.

6.4. Guidelines for intersection clustering in traffic signal coordination

- For Volume over capacity ratios of 0.5 to 0.9, the minimum values of clustering index values were obtained ranging from 30 to 83. The signalized intersections are recommended to be clustered for traffic signal coordination, given the intersections satisfy the minimum values of clustering index produced.

6.5. Guidelines for traffic signal coordination for non-similar intersections

7 intersection types were developed based on the geometric details and priority directions for green platoon and two matrixes were developed for peak time period and off peak time period. The developed guidelines were successfully verified using 2 case studies conducted at Baseline Road Intersection cluster and Kadawata Intersection Cluster.

6.6. Further Research Areas

For further research work, based on the conclusions it is identified to explore the possibility of using other major parameters such as speed to be considered when determining the relevant clustering index values for the identification of potential for intersection clustering.

Further, the guidelines for traffic signal coordination identify 7 sub categories and priority directions for green platoons. However, for further research it is recommended to introduce additional sub categories of intersections for further complex situations involving with different geometrical condition and priority directions if in the future such becomes crucial.

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APPENDIX A: DESCRIPTION OF INTERSECTIONS USED FOR MODEL CALIBRATION

Intersection	Description
Katubedda Junction	This is a 3 leg junction and operates as a three phase signalized intersection. Separate turning lanes for right turns and left turns are available. A filtering bus lane is also available for buses towards Colombo. This junction is used in the study for calibration of the model.
Kesbewa Junction	This is a 4 leg junction and currently operates as a 4 phase signalized intersection. This junction is used in the study for validation purposes. Major traffic and queue us obtained in the Bandaragama leg due to single lane being available per direction effectively.
Orugodawatta Junction	This junction is used for practical application of the calibrated software. This is a major junction in the baseline road, where significant amount of U –turns are experienced from Borella Direction. Right turns from Peliyagoda direction is prohibited.
Rajagiriya Junction	This is a 3 leg junction, where nearby performance of nearby junction significantly affect this junction. This was analysed before the construction of the Rajagiriya flyover, where significant queues are observed. Police manual control is observed in most of the peak periods.
Nawala Road Junction	This is a 3 leg junction. Since few more junctions are located in close proximity, performance of close by junctions have also affected this junction. Significant queues are also observed

Ayurvedic Junction	This is a non-similar junction with a different geometrical arrangement, where police manual control is seen in most of the peal time periods
Pamankada Junction	This is a junction with a roundabout, where 3 legs are intersected. Police manual control is observed in most of the morning and evening peak times.
Peliyagoda Junction	The analysed section is the merging section of the vehicles towards Colombo from Expressway and A1 road. Significant queues are observed in the morning peak hour in both the expressway and A1 road. Police manual control is observed in the morning peak hours.
A1 Connection of expressway entrance road near Peliyagoda	This is a 3 leg junction, where proper channelization is observed. Left turn filter lane available for vehicles towards Peliyagoda. Currently operates as a 3 phase signalized intersection.
A3 Connection of expressway entrance road near Peliyagoda	This is a 3 leg junction, where significant queues are observed in the Negombo Road in the morning peak time towards Colombo. Currently operates as a 3 phase signalized intersection.
Ingurukade Junction	This is another major junction involving 5 legs. However, this is currently operating as a 4 phase signalized intersection and considerable heavy vehicles movement is expected in the junction.

Bandaranaike Round about	This is a 3 leg junction, which is currently operated as a 3 phase signalized intersection. Significant queues are observed in the morning and evening peak times.
Nawaloka Roundabout	This is a roundabout connecting 3 legs. Peak periods, police manual control is observed.
Kadawata A1 Junction	This is a 3 leg junction, with no separate right turn lanes, where the major road considered is the Kadawata by pass road. The peak flow is towards Kirillawala in the evening time. The junction currently operates as a 3 phase junction, where pedestrians are provided in all three directions.
Shramadana Mw. Junction	This junction is a 4 leg junction and the major road direction is the Kadawata bypass road. The minor road (Temple road) is only given an amber blinking signal, due to lower traffic volumes The junction currently operates as a 3 phase signalized intersection. Significant number of vehicles are entering and exiting from Shramadana Mawatha. Pedestrian phase is provided only to cross the bypass road.
Kadawata Expressway Exit	This is again 3 leg junction, where the expressway exit vehicles can turn towards Kadawata or Kirillawala. The junction is currently operated as a 2 phase signalized intersections, since no vehicles are allowed into the expressway from this junction.
Kadawata Expressway Entry	This is a 3 leg junction, where the expressway entry vehicles coming from Kirillawala and Kadawata directions should enter the expressway from the junction. This junction is also operating as a 2 phase junction, since no vehicles from expressway are exiting from here.

Bandarawatta Junction	This junction is also a 4 leg junction, and operates as a 3 phase signalized intersection. In the major road, leading green for right turns only phase is provided to reduce cycle time and delays. The minor road both directions are provided in a single phase due to the lower vehicular volumes expected.
Eldeniya Junction	This is a 3 leg junction and operates as a 3 phase junction. Pedestrians are allowed only in two phases across the by-pass road and A1 road (towards Kadawata direction).
AAT Junction	This is a 4 leg junction, where the major road is the baseline road, with no separate right turn lanes. The peak flow is towards Kirulapone in the evening time. The junction currently operates as a 4 phase junction with leading green for right turn phases for major road direction.
Kirimandala Junction	Kirimandala Mawatha junction is a 3 leg junction and the major road direction is the baseline road. The junction currently operates as a 3 phase signalized intersection. In the evening peak period, the downstream queue at the Narahenpita junction affects the performance of the Kirimandala junction
Narahenpita Junction	This is a 4 leg junction, which can be considered as the major junction in the selected intersection cluster, where vehicles from Nawala and Jawatta area would utilize this junction. The Narahenpita junction currently operates as a 5 phase junction, where the major road (Baseline road) right turns are split into two phases along with the respective through movements.
Apollo Junction	Apollo junction is a 3 leg junction. However, this is a minor junction where the major impact is from the vehicles coming from Kirulapone direction. Due to this reason, when coordinated

	direction is towards Kirulapone, this junction may be neglected for analysis purposes. The exiting vehicles from Apollo Hospital are not allowed to turn right where as a signal is placed for vehicles turning right into the hospital. This arrangement makes it operate as a 2 phase signalized intersection.
Park Road Junction	Park road junction is a 4 leg junction and operates as a 4 phase signalized intersection. The minor roads are operated in a single phase due to the lower amount of vehicles coming from the minor road.

APPENDIX B: QUANTIFICATION OF TRAVEL TIME SAVINGS AND UPDATING THE VALUE OF TIME FOR EVALUATION OF TRANSPORTATION PROJECTS

As for the literature review, the existing values that are used in transportation project evaluations has been developed around 20 years ago and significant changes in transport services and infrastructure has occurred during this period. Therefore, it is essential to update the value of time for accurate forecasting and estimation procedures.

In order to achieve the objectives, a reliable estimate is required for value of time for transportation projects and also a methodology to update the value of time over the years. This step would also be very crucial for the further aspects of the research when the need for quantification of travel time and delay savings becomes crucial.

Methodology

The study framework used for the research is presented in Figure A-1.

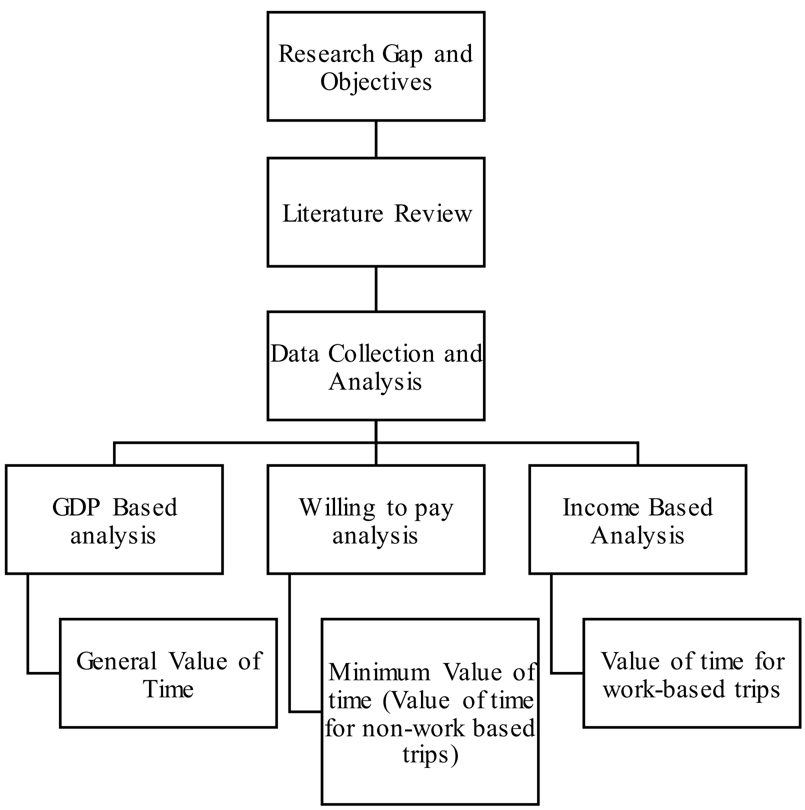

Figure A-1: Study Framework

Method 1: GDP based technique

In order to have a general understanding on the value of time for a region, it is essential to identify the value of time based on the Gross Domestic Product (GDP), which is available in literature for a given country or a region. The GDP based technique is specifically used in the research in order to approach and proceed with the research to have a general understanding of value of time.

The value of time of a person can be calculated using the Equation A-2 using published GDP values. When calculating the value of time of a person, it has been assumed that an average person would work for 40 hours per week over a period of 50 weeks per year.

$$Value\ of\ time = \frac{Gross\ Domestic\ Product}{Mid\ year\ Population*(40*50)} \quad \text{(A-2)}$$

The value that derived from this method can be used to obtain a general value of time of a person considering the total population. The advantage of this method is that it gives a general picture of VOT, since the GDP is calculated based on the productive work that has been generated throughout a year all over the country or region. However, it is argued that the estimate which is calculated through GDP is of limited usage, since it is not categorized to transport modes, income levels or trip purpose. Therefore, the GDP based method can only be used for general understanding in the research.

Method 2: Willingness to pay analysis based technique (for non-work based trips)

"Willingness to pay analysis" has been identified as one of the widely used methods to determine the toll to be charged for using a particular transportation mode or route as well as in estimating the value of time of a person. The value of time that is generated in this methods can be considered as the value of time of a person for non-work based trips (Victoria Transport Policy Institute, 2013). The rationale behind this is the travel time savings of the non-work based trips would be used by a road user for his leisure activities. Therefore, the price that would be getting would reflect his opportunity cost of that leisure activity, which would be the value of one's time for non-work based trips. However, this would heavily depend upon the social and cultural factors of a particular society.

What is expected from a willingness to pay analysis is that a question is asked about a fee that a person is willing to pay for using a particular transport initiative. Here, an unbiased sample needs to be identified first, where most of the times random sampling techniques are being used. From the interviewees, few questions are asked in order to arrive at a judgemental response based on their own logic. In this research a question asked from the users how much they are willing to pay for a 30 km long trip which is generally taking 1 hour, to get it reduced to 20 minutes was used.

It can be argued that the willingness to pay analysis would reflect the minimum estimate of an individual value of time. When individuals express the price they are willing to pay as a toll to save time from their journey, they will always factor in a net saving for them after paying the toll for the particular trip. Therefore, it is prudent to assume that the values given in such surveys are at the lower end of the actual VOT of a person and can be considered as the minimum value of time for non-work based trips.

Method 3: Wage rate based method (for work based trips)

Wage rate model which is based on the income was used to estimate the value of time of people. In calculating the benefits arising from transportation projects, the main concern is to identify the value of the total time savings.

It is important to categorize the value of time using the mode of transportation, such as the private transportation and public transportation. This can even be categorized further, for the type of vehicles as well. In terms of data collection and the availability of data, the easiest way is to obtain data on the transport mode which is mainly focused on this research.

If the average monthly income of people using different types of transport modes can be obtained, the estimates of the value of time can be obtained from the Equation (A-3) for respective transport mode assuming a person is working on average 40 hours per week into 50 weeks per year. The assumptions based on the methodology are clearly justified by the field data when considering the case study of Sri Lanka.

$$Value\ of\ time\ for\ work\ based\ trips = \frac{Average\ Monthly\ Income * 12}{(40 * 50)} \qquad \text{(A-3)}$$

Estimating value of time for future years

One of the main shortcomings of the existing methodologies is that no proper technique is available which will account the effects of interest rates, consumer behaviours and inflation. Therefore, the estimate for value of time obtained needs to be updated over the years and there should be reliable methodology to follow. In this research mainly two indices have been considered to update the value of time.

- Wage Rate indices
- Colombo Consumers Price Index (CCPI)

Case Study for Sri Lanka

Almost all the transport projects in Sri Lanka are currently using a single value and in order to have a more realistic and reliable value, the value of time needs revision. Therefore, a case study was conducted for the case of Sri Lanka, which can be considered as a developing country. The methodology produced in this research could be used to obtain the present value of time for Sri Lanka showing the value of time that is currently being used is giving an underestimated value in benefit calculations when evaluating projects.

Method 1: GDP based technique

GDP based technique was used to calculate the value of time in order to obtain a more general estimate for the value of time.

The GDP values for Sri Lanka along with the Mid-Year population were obtained using the Annual Reports of Central Bank of Sri Lanka over the past 20 years, as indicated in Figure A-2 and Figure A-3.

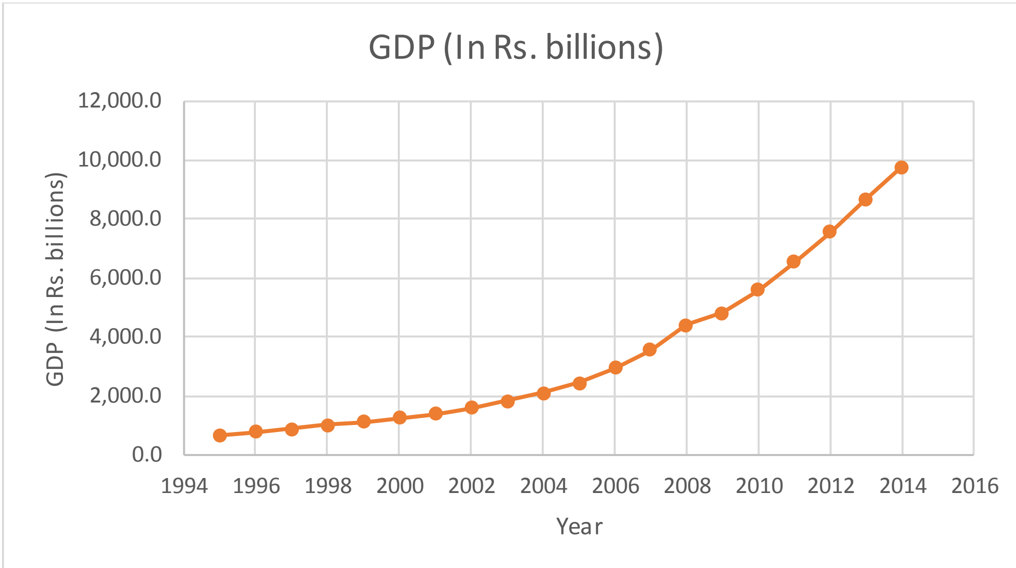

Figure A-2: Gross Domestic Product from 1995-2014

Source: Central Bank of Sri Lanka annual reports

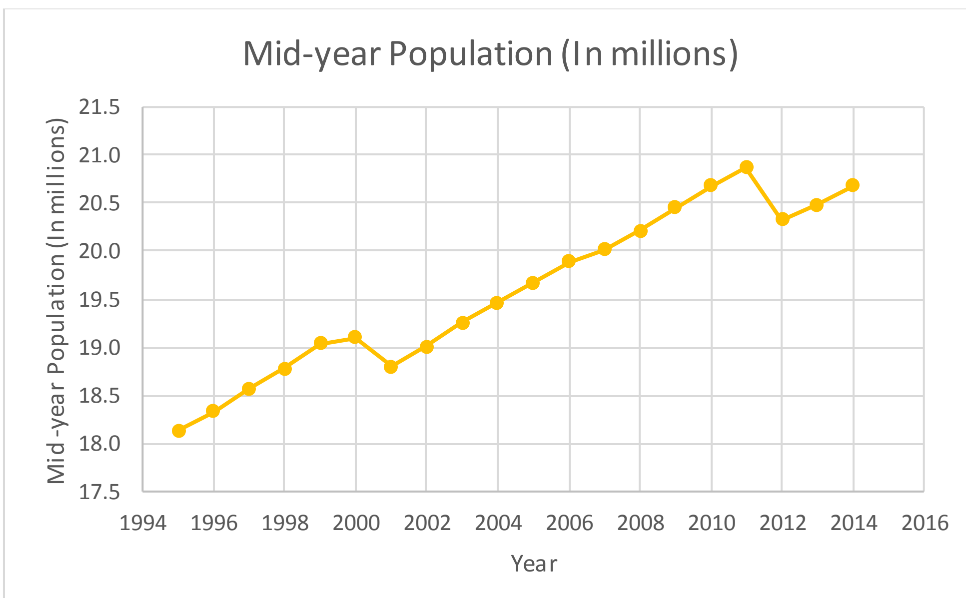

Figure A-3: Mid-year population from 1995-2014

Source: Central Bank of Sri Lanka annual reports

By considering the above data, the general value of time, which is the GDP per hour, derived for the past 20 years are mentioned in Figure A-4.

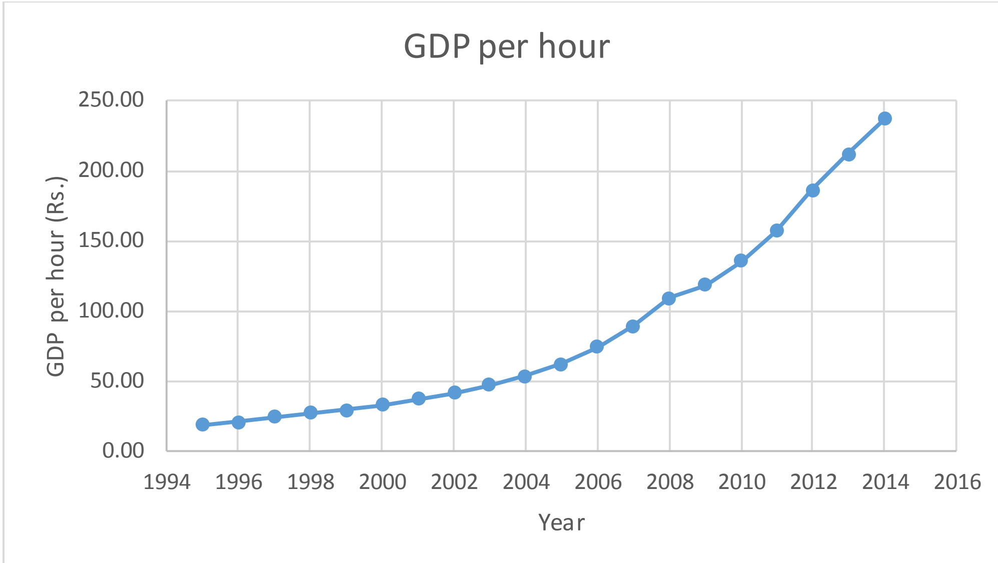

Figure A-4: GDP per hour (General Value of time)

The general value of time obtained through the GDP is Rs. 211.74/hour and Rs. 236.63/hour for the years 2013 and 2014 respectively. Based on the above calculations, the value of time for 1999 would be Rs.29/ hour. This is even higher than the Rs.25/hour value used as the base value for 1999 for motorized transport and the same under estimation happens, when adjusted with CCPI for the present context. This also suggests that an update is required for value of time estimations for Sri Lanka.

Further, value of time which is obtained through the Gross Domestic Product of a country would reflect the average value considering the total population, which would consist of people who will not even benefit from transportation infrastructure projects especially in rural areas. Thus, an underestimated value would be generally obtained using GDP based techniques.

Therefore, it can be concluded that the general value of time, obtained through GDP would be less than the value of times that are obtained through other mechanisms most of the times.

Method 2: Willingness to pay analysis based technique (for non-work based trips)

As for the Method 2, willingness to pay analysis has been conducted where the first step was to obtain an unbiased sample using random sample techniques. The random vehicles were stopped and were given a questionnaire form where the total sample size was 3,317 vehicle users. The respondents were asked to give a price that they are willing to pay for a 30km long trip which is generally taking 1 hour to get it reduced to 20 minutes (Figure A-5)

Vehicle Type		Category		Occupancy		Origin	Type of Origin		Destination	Type of Destination	
Car	1	Private	1	1	1	Village/GN Division	Home	1	Village/GN Division	Home	1
Jeep/Pickup	2	Official	2	2	2		Work	2		Work	2
Van	3	Hired	3	3	3	DS Division	School/Educ.	3	DS Division	School/Educ.	3
Special Bus	4	Chartered	4	4	4		Business	4		Business	4
Approx Travel Time		Other	5	5	5	District/Province	Social	5	District/Province	Social	5
H H : M M				6or more	6		Recreational	6		Recreational	6
Monthly Income Range Rs.		<25,000	1	50,000-75,000	3	100,000-200,000	5		Willingness to pay (1 hr trip -30km, in 20 min)		
		25,000 -50,000	2	75,000-100,000	4	>200,000	6				

Figure A-5: Questionnaire form used

The values that were obtained was considered as the minimum value of a person for saving 40 minutes of one's time. It was then converted to 1 hour to reflect the minimum value of time of a person per hour.

In deriving the results under willingness to pay analysis, five income categories and the respective transport modes were first identified. The survey data was first categorised into the above five income categories and the average values of the respective categories were identified. The results were as mentioned in Table A-1.

Table A-1: Willingness to Pay Analysis Results

Income Category	**Monthly Income Range (LKR)**	**Dominant Transport Mode**	**No. of respondents**	**Minimum Value of time (for 1 hour in LKR)**
1	< 25,000	Public Transport	223	174
2	25,000 - 50,000	Motor Cycle / Three Wheel	598	222
3	50,000 - 75,000	Van	1,456	268
4	75,000 - 100,000	Car/SUV	626	295
5	100,000 – 200,000	Car/SUV	200	297
6	> 200,000	Car/SUV	138	405

It can be concluded from this result that even the minimum value of time, which is the value of time for non-work based trips, that is obtained through the willingness to pay analysis is more than the value of time used currently for economic feasibility assessment in Sri Lanka.

Therefore the research could be further progressed to obtain a more detailed calculation of value of time of a person based on the wage rate or cost savings model.

Method 3: Wage rate based method (for work based trips)

It is important to obtain a more reliable number for value of time, as it was proved from the earlier methods, the value of times that are currently been used are far more outdated. The wage rate model has been used to obtain a more reliable estimate of the value of time for the Sri Lankan context.

The same sample and the survey questionnaire forms were used for the wage rate based method as well and when the additional data was required reliable literature from the department of census and statistics was also referred. For determination of the value of time, five main vehicle categories were identified as mentioned below.

- Public Transport
- Motor Cycles and Three wheelers
- Van
- Car
- SUV

Next step was to calculate the average monthly income of users in each category. For this purpose, data obtained through willingness to pay analysis and the data obtained through the surveys done by the Department of Census and Statistics were used. The average incomes obtained through the above sources are mentioned in Table A-2 .

Table A-2: Average Monthly Income

Source: (Central Bank of Sri Lanka, 2012)

Vehicle Category	Average Monthly Income (Rs.)
Public Transport	31,254
Motor Cycle and 3 wheel	39,345
Van	59,907
Car	85,325
SUV	87,993

The above average monthly incomes obtained, were then converted to hourly income, which was considered as the Value of time of a person per hour for work based trips using the Equation A-4.

$$Value\ of\ time\ for\ work\ based\ trips = \frac{Average\ Monthly\ Income * 12}{(40*50)} \tag{A-4}$$

When calculating this value of time per hour, as earlier it was assumed that a person would work 40 hours per week for 50 weeks for one year. The results obtained through the analysis is mentioned in the Table A-3.

Table A-3: Results from income based approach

Vehicle Category	Average Monthly Income (LKR	Average Hourly Income (VOT for work based trips) (LKR)
Public Transport	31,254	188
Motor Cycle/ 3 wheel	39,345	236
Van	59,907	359
Car	85,325	512
SUV	87,933	528

Estimating value of time for future years

The established methodology to update the value of time over the years is to use Colombo Consumer's Price Index (CCPI) (Kumarage, et. al. 2000). CCPI is calculated each year in Sri Lanka, having a base year as 2006 currently.

Wage rate indices can even be used, but due to the limitations such as not accounting for the income generated through the businesses, income of some private sector categories, and income from self-employment, CCPI has been recommended as the reliable updating method over the years to account for effects of inflation and growth. This method can be used as long as CCPI is calculated for Sri Lanka.

Results

The general value of time which is obtained through the GDP is Rs.237/hour. The minimum value of time obtained through the willingness to pay analysis for non-work based trips was varying from Rs.174/hour to Rs.405/hour (Table A-4) and that of work based trips was varying from Rs.188/hour to Rs.528/hour.

Table A-4: Summary of the Results

GDP Based Value of time	Transport Mode	Value of time of a person (Rs. Per hour)	
		For non-work based trips	For work based trips
Rs.236 per hour for all the vehicle categories	Public Transport	174	188
	Motor Cycle/ 3 wheel	222	236
	Van	268	359
	Car	296	512
	SUV	405	528