IDENTIFICAION OF EFFECTIVE CONTROL STRATEGIES FOR SIGNALIZED INTERSECTIONS DURING PEAK HOURS

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Degree of Master of Science

Department of Civil Engineering

University of Moratuwa
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Thesis submitted in partial fulfilment of the requirements for the degree Master of Science in Civil Engineering

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DECLARATION

I declare that this is my own work and this thesis does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other University or institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

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A. Vajeeran

The above candidate has carried out research for the Masters under my supervision.

……………………………………… Date:……………………

Dr. GLDI De Silva

ABSTRACT

Traffic congestion during peak hours is one of the major issues in the Sri Lankan city centres. There are several identified reasons for the congestion. One of the major reasons for congestion is ineffective control strategy used at signalised intersections. In Sri Lanka during the peak hours most of the signalized intersections in city centres are controlled manually by traffic police officers. The objective of the study is to find out the effectiveness of the manual control by traffic police officers compare to traffic signals and the effectiveness of the traffic signal cycle time design for Sri Lankan conditions.

Four Junctions which are controlled by traffic police officers during the peak hours were selected for the analysis and data were collected. Traffic micro simulation software VISSIM has been used for the analysis and it has been calibrated and validated to local conditions before using it for the analysis. Junctions were modelled for three cases, existing posted signal time, updated traffic signal time, and traffic police phase arrangement and cycle time. For the updated signal case, signal times has been initially designed using Webster & Cobbe (1966) signal cycle time model and modified by simulating for several factorised cycle times to achieve optimum delay.

Out of the four junctions, three of them are not saturated and one junction is over saturated. When the junctions are not saturated, the signal times were updated and the updated signal times results lesser delays than the existing posted signal times. When the junctions controlled manually by traffic police the delays are lesser than the existing signal time case. But the updated signal times result lesser delay than the manual control when the junction is not saturated. When the junction is oversaturated, delays for manual control by traffic police result lesser delay than the existing signal times. Traffic police officers become effective when the junction is oversaturated as they allow risky merging movements to reduce the critical flows.

From the result it is evident that the delays for major road movements have no significant change for the two cases manual control and updated traffic signal design. But for minor road movements significant reduction of delay observed from manual control to traffic signal control. The maximum queue lengths on the minor roads also higher for the manual control than the updated signal control. Daily variation of traffic also affects the junction delays significantly as the fixed cycle time signals are used. Introducing vehicle actuated signals will be an effective solution for random arrivals of vehicles and daily variation of traffic than controlling the intersections manually.

Keywords: Cycle time; Phase; Delay; Queue Length.

DEDICATION

To my wife and Daughter

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A. Vajeeran
5th March 2020

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1. BACKGROUND INFORMATION

1.1. Introduction

Traffic congestion and delays during peak hours has become a serious issue in developing countries like Sri Lanka. Traffic congestion occurs when the traffic demand exceeds the capacity of the road network. Two types of congestion occur in the city centres. They are, recurrent congestion and non-recurrent congestion. Recurrent congestion occurs at the same time and place every day repeatedly. This kind of congestion normally occurs during the peak hours. Non recurrent congestion occurs due to unexpected events such as, accident, road repairs, road closures, etc.

In Sri Lanka, many road networks especially within the city centres are congested during the peak hours. The traffic congestion was largely limited to Colombo city in Sri Lanka previously and now it has been spread to all the major cities in Sri Lanka. Heavy traffic congestion occurs during school opening hours especially between 7:30 a.m. to 8:00a.m. and then followed by office traffic between 8:00a.m and 9.00a.m. In the afternoons and evenings, the same sort of traffic congestion occurs between 1:30 p.m. to 2:30 p.m. and 5:00 p.m. to 8:00p.m. [25].

This traffic congestion will cause several impacts to the road users such as,

- Traffic congestion creates more delays to the vehicles
- Traffic congestions increase the travel time and as a result people spend more time on the road and large amount of fuel is consumed
- Traffic congestion impacts the environment as, additional vehicle travel time and stop time leads to emission of harmful carbon emissions
- Traffic congestions make the drivers impatient and it can lead the people to drive aggressively. It also affects the psychology of the people and will affect the productivity at work places and personal relationships.
- Traffic congestion reduces the lifetime of the road surfaces as the number of stops and stop times are increased.
- One positive effect of traffic congestion is many people will shift from private vehicle to public transports. But this depends on the effectiveness of the public transport network.

So, need of finding an immediate short term and proper long-term solutions are essential.

There are several identified reasons for the congestion such as excessive traffic flow, insufficient road capacities, pavement conditions, poor road network, driving behaviours of the drivers, shortage of parking areas, poor infrastructure for pedestrians, parking of heavy vehicles on main roads, insufficient public transport services, poor intersection control strategies, and etc.

Vehicle growth rate in Sri Lanka has been increasing rapidly in the recent years and it result excessive traffic flows in the city centres. The number of vehicles on the road has been increasing without any improvements to the road network. To reduce the present road traffic the government has taken several actions. The recent statistics says there are 130 vehicles per 1,000 people. Among them 66% are motorbikes, three wheelers and cars [25]. If the government fails to implement proper control of vehicle growth it will create severe problems related to space sharing of vehicles on the road.

The data from Ministry of transport and civil aviation, the vehicle growth in Sri Lanka from 2012 to 2018 is shown in Table 1-1. The data clearly shows the growth rate of private vehicles such as motor cars, motor tricycle and motor cycles are very high. That means people prefer more the private transport instead of public transport. There should be long term solutions proposed to overcome this to attract more people to the public transport. There are several researches and proposals made to implement public transport systems such as Light rail transit, bus modernisation, inland water transport, electrification of existing rail lines and etc. But the transportation engineers are in an urge to find a short-term solution immediately for the issue as the traffic congestion has already went past an acceptable level.

There can be several short-term and long-term solutions proposed for the traffic congestion issue based on the traffic theories. Road widening to meet the increasing demand could be a solution for the traffic issue. But the congestion is largely limited to the Colombo and neighbour city and there is no place for the widening of roads as the cities are not developed to a proper planning. Land acquisition is highly impossible in a city like Colombo.

Table 1-1 Number of registered vehicles in Sri Lanka

Year	2012	2013	2014	2015	2016	2017
Motor Cars	499,714	528,094	566,874	672,502	717,674	756,856
Motor Tricycle	766,784	850,457	929,495	1,059,042	1,115,987	1,139,524
Motor Cycles	2,546,447	2,715,727	2,988,612	3,359,501	3,699,630	4,044,010
Buses	91,623	93,428	97,279	101,419	104,104	107,435
Dual purpose vehicles	280,143	304,746	325,545	365,001	391,888	408,630
Motor Lorries	323,776	329,648	334,769	341,911	349,474	352,275
Land Vehicles-Tractors	315,520	326,292	333,362	343,339	353,624	362,445
Land Vehicles-Trailers	53,020	55,286	57,298	59,426	63,088	75,947
Total	4,877,027	5,203,678	5,633,234	6,302,141	6,795,469	7,247,122

Source: Ministry of transport and civil aviation

Among the reasons for the traffic problem poor intersection control strategy has been identified as one of the main reasons for the traffic problem. Improper control on each individual intersection will be contributed to the congestion in a road network. Traffic management at intersections during peak hours when the maximum flows are experienced is becoming a challenge and some of the current methods adopted have become ineffective. A more efficient control strategy for an oversaturated network is required.

There are several types of intersections in roads such as Cross intersection, T-intersection, Y-intersection, multiple road intersection, skewed intersection, road and railway intersection are there. According to the amount of traffic these intersections are controlled by different levels of intersection control strategies. There are three levels of intersection control.

1. Active control
2. Passive control
3. Semi control.

Right hand rule and traffic signs comes under passive control. Roundabouts comes under semi control. Traffic signals, traffic police manual control and grade separation fall under active control. When it comes to the peak hours, in Sri Lanka most frequent control strategies used in major signalised intersections are traffic signals and manual control by traffic police and during some extreme cases grade separation is provided. When it comes to these three control strategies traffic signals and grade separation are properly analysed with the data and designed control. But the manual control by traffic police, the decision (phase arrangement, phase time and cycle time) will be taken on the field by them.

During the peak hours almost all the intersections in the Colombo city are congested. Even most of the local and collector roads also has reached the capacities as people started to use them as the alternative diversion routes. Because of that traffic police officers are present at most of the minor and major junctions to control the traffic during the peak hours. When it comes to unsignalized intersections traffic police officer’s presence is important during the peak hours. In most of the major signalised intersections also traffic police officers switch off the current traffic signals and they starts to control manually.

Queue clearing approach has been used by the traffic police officers to control the intersection. Because of that their priority can go in favour of the major road movements. The cycle time and phase arrangement traffic police maintain also not consistent. But with the existing congestion manual control by traffic police officers seems to be the only solution. Therefore, this research study tries to analyse the effectiveness of manual control and traffic signal control in the signalised intersections.

Traffic micro simulation software VISSIM has been used as the analysing tool in this research. VISSIM is a microscopic time step and behaviour-based traffic simulation software which has been widely used assessing traffic conditions. It is especially useful to evaluate different traffic management scenarios to choose the best and optimization measures before implementation [29]. The traffic analysing key parameters like Delay, Vehicle travel time, Queue length and junction flow values can be easily generated using the VISSIM software [22]. The benefit of using this computer software is that it is fast and cost effective. The software has to be calibrated to Sri Lankan conditions before using for the analysis [28].

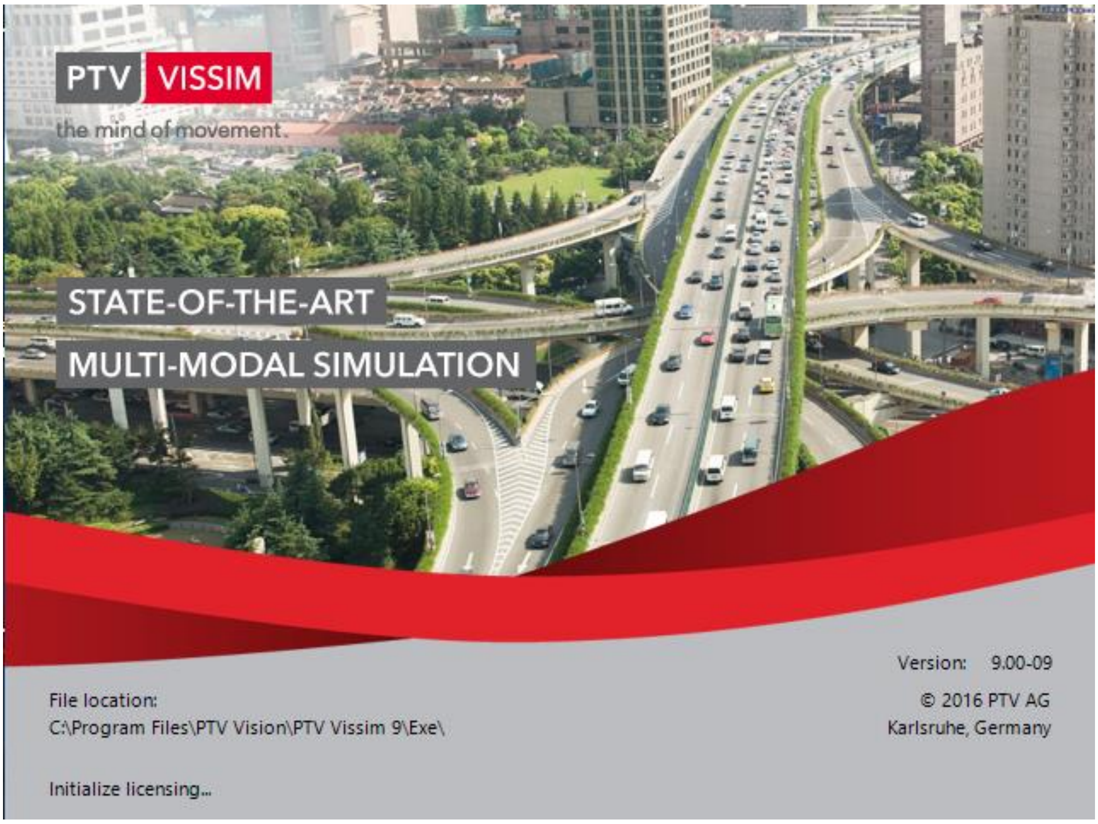


Figure 1-1 PTV VISSIM software startup menu

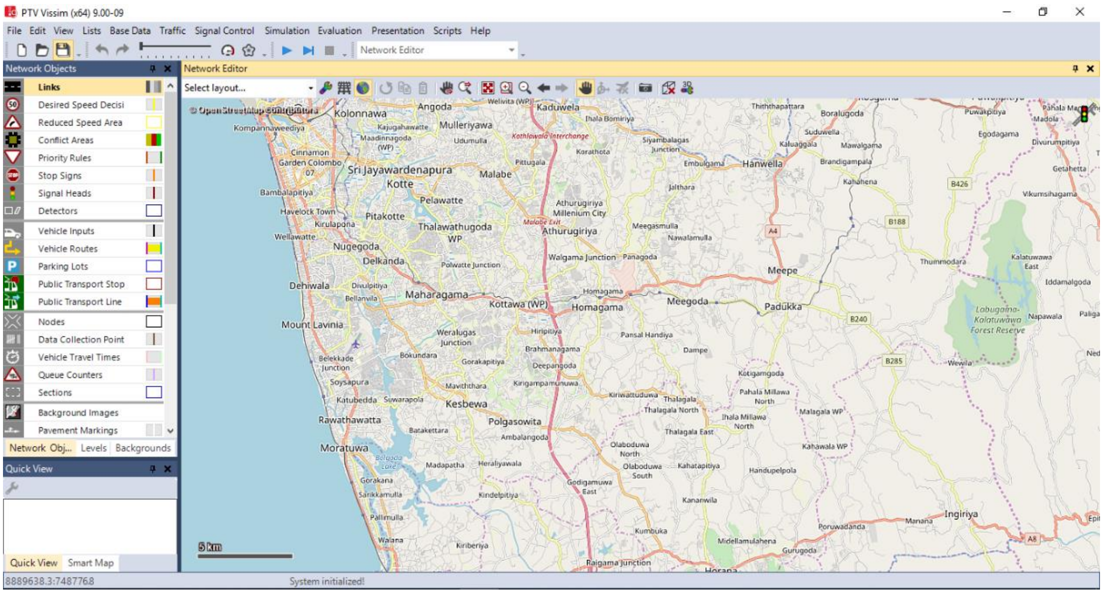


Figure 1-2 PTV VISSIM software working interface

1.2. Objectives

The main objective of the study is to find the effectiveness of the manual intersection control strategy compare to Traffic signals. The study will also find the effectiveness of the basic signal design concepts during peak hours for Sri Lankan conditions.

1.3. Methodology

The research methodology includes the following tasks to achieve the research objectives.

- Review of literature regarding different intersection control strategies and micro simulation.
- Selection of multiple intersections which are controlled by traffic police during peak hours
- Data collection
    - Geometrical details
    - Traffic flow, Vehicle classified count, turning movements, queue length
    - Timing and phasing arrangement of traffic police
- Calibrate the Vissim Software driving behavioural parameters to suit the Sri Lankan conditions and validate those parameters.
- Modelling the selected junctions with,
    - Existing traffic signal
    - Updated traffic signal
    - Manual control by traffic police
- Analyse the delay and queue results and find the effective control strategy during peak hours
- Also find out the improvements which can be made to the existing signal design theories to make them effective during the peak hours, especially for Sri Lankan conditions.

1.4. Outline of the thesis

The thesis consists of six chapters which includes the following contents.

Chapter 1: Includes the background of the research and objectives.

Chapter 2: Reviews traffic theories related to this study. Also reviews several intersection control strategies and design of intersection control. In addition to that the chapter discusses the use of micro simulation in traffic analysis and calibration of VSSIM software driving behavioural parameters to Sri Lankan conditions.

Chapter 3: Includes the data collection part. It discusses the selection of junctions for analysis and provides the data collected on the selected junctions.

Chapter 4: Presents the need of calibration of Vissim software model for the Sri Lankan conditions. It provides the calibration process and validation of the calibrated parameters. Calibrated driving behavioural parameters also provided in this chapter.

Chapter 5: Presents the analysis of the junctions and the results generated from the VISSIM software model for the junctions. The models were created for the selected three strategies for the comparison. The delay and queue results are presented as the control measures.

Chapter 6: Presents conclusions of the present study and future areas for consideration in relation to this study.

2. LITERATURE REVIEW

2.1. Introduction

Literature review regarding different intersection control strategies and micro simulation has been done to achieve the research objectives. literature review has been done on 5 areas related to the research. They are,

1. Types of intersections
2. Different intersection control strategies
3. Traffic signal design concepts
4. Delay models
5. Micro simulation using VISSIM

2.2.Types of Intersection

Intersection in road is defined as two or more roads meets in the same elevation. This intersection will be used by several road users such as motor vehicles, pedestrian, bicycles and public transports. Thus, the intersection includes not only the road pavement area, but the adjacent sidewalks for pedestrians. Basically, the intersections can be classified into four major types [3] [18]. They are priority intersection, space sharing intersection, time sharing intersection and uncontrolled intersection

- **Priority Intersection**

In this type of intersection more priority will be given to one particular major road over the other minor roads. The minor road will be controlled by stop sign and yield line. These types of intersections ensure that there is less or no delay for the vehicles travelling on the major street.

- **Uncontrolled intersection**

This type of intersections are common ones in the road networks where there is not much traffic to control and where the intersecting roads are equal importance.

- **Space sharing intersection**

Space sharing intersections gives equal priority to the vehicles approaching the intersections, thus allows continuous movements for all directions of traffic. An example of this type will be roundabouts.

- **Time Sharing Intersection**

In this type of intersections traffic is allowed for different directional approaches in different point in time. Traffic signal control and manual control by traffic police officer are example of time-sharing intersections.

2.3. Intersection Control strategies

At the intersection vehicles coming from different directions are moving towards different destinations. Those vehicles want to share the same space at the same time. When they reach the intersection, they have to take decision on their speed, direction and movement along the intersection. These decisions had to be taken very quickly and a minute error can lead to serious accidents. Therefore, some control on the vehicles should be applied externally to reduce the accident risks.

Controlled intersections cause delay as well according to the geometry of the intersection and control strategy applied on the vehicles. For a road network to be effective, each intersection on the network also needs to be perform effectively. The efficiency of the intersection also affects the capacity of the road.

The accident risks, road capacities and performance of a road network depends on the efficiency of the individual intersection performance. Therefore, the intersection control strategies study is important.

2.3.1. Three levels of intersection control

The intersection control strategies can be categorized into three according to the different levels of control. They are,

- Active control
- Passive control
- Semi control

To move from one level of control to the next level, there are several warrants to be satisfied [3] [18].

2.3.2. Passive control

In this case drivers are not undergoing any external control. This type of control can be applied when the traffic flows are relatively low. The road users have to follow the basic rules. Passive control can be classified into three categories such as no control, traffic signs only and traffic signs with road markings.

2.3.2.1. No control

When the traffic volumes are relatively very low intersection can be left with no control. Here the road users have to follow the basic road rules such as giving priority to the through movements.

2.3.2.2. Traffic signs

By erecting proper sign boards such as stop sign and give way sign some level of control can be achieved. This type of control can be classified into three types,

- Give way control
- Two-way stop control
- All-way stop control

In give way control the driver from the major road gets the priority over the vehicle coming from minor road. The driver from the minor road no need to stop the vehicle fully, but has to reduce the speed and check the through movement. In two-way stop control the driver coming from the minor road has to stop the vehicle and check for the availability to enter the major road. All way stop control is applicable when both the road crossing at the intersection are equally important and cannot differentiate the major road and minor road. Here vehicles coming from all the directions has to stop at the junction to check the availability to enter the junction. If two vehicles arrive at the same time, the vehicle on the right side will get the priority.

2.3.2.3. Traffic signs plus marking

In this type of control there will be lane markings such as stop line along with the sign boards.

2.3.3. Semi Control

Semi control can be termed as partial control also. Here the drivers are guided up to some extent to avoid conflicts. The two types of semi control are channelization and roundabouts.

2.3.3.1. Roundabouts

At roundabouts the through movement conflicts, through and right turn conflicts are eliminated by allowing the traffic to flow around a specific path around a centre island. The through and right turn conflicts are converted to merging conflicts which is relatively safer than through and right turn movements.

2.3.4. Active control

Active control applies full control on the traffic movements. In active control drivers cannot act for their own choice. Traffic signals, manual control by traffic police and grade separated intersections comes under active control.

2.3.4.1. Traffic signals

The traffic signals control the intersection by time-sharing approach. During a particular time, traffic signals allows some movements and restrict some other movements. Depending on the traffic two or more phases can be provided.

There are two types of signals commonly available. They are fixed cycle time signals and vehicle actuated signals. The signal cycle time, phase arrangements and phase times will not change in a fixed time traffic signal. The phase arrangement and phase times will be constant for every successive cycle in fixed time traffic signals. Because of the randomness of the vehicle arriving the fixed time traffic signals cannot accommodate the fluctuating traffic.

Vehicle actuated signals are a better option for a fluctuating traffic. The vehicle detectors/sensors will observe the incoming traffic the controller will automatically adjust the cycle time, phase arrangement and phase time according to the incoming traffic.

2.3.4.2. Grade separated intersections

According to the space sharing approach intersections are classified into two. They are at-grade intersections and grade-separated intersections. In at-grade intersections all legs are meeting at the same elevation. But in grade separated intersections some road movements are crossing at several elevation levels to separate the space. In some cases, the topography at the intersection naturally creates a grade separated intersection. But on a flat terrain providing grade separated intersections are costlier. Normally grade separated intersections are built on expressways and high-volume urban junctions. Grade separated intersections will increase the capacity of the roads.

2.3.4.3. Classification of Grade Separated Intersection

The major grade separated intersection types are,

1. Underpass
2. Overpass
3. Trumpet Interchange
4. Diamond Interchange
5. Cloverleaf Interchange
6. Partial Cloverleaf Interchange
7. Directional Interchange

2.4. Traffic Signal Design

2.4.1. Overview

Traffic signal is one of the frequently used control strategies of traffic in intersections worldwide. By the traffic signals the conflicts from movements of traffic in different directions is controlled by time sharing approach. Some of the key words related to traffic signal design are,

- **Cycle:** A signal cycle is one complete rotation of all the phases.
- **Cycle time:** It is the time difference between successive starting times of one particular phase. For example, time difference between start of green for a particular phase to again green.
- **Interval:** It is the change from one phase to another. There are change interval and clearance interval. Change interval is the amber time between green and red. Clearance interval is the all red time given for the clearance of vehicles from the intersection.
- **Green time:** During this time particular phases are allowed to move through the junction. It is denoted by G_i.
- **Phase:** It is the green interval plus the change and clearance intervals

- **Lost time:** The time which the intersection is not effectively used for any movement. Lost time consist of starting delay of the vehicles in the queue and the all red time.

The design of traffic signals has 6 major steps. They are,

1. Phase design
2. Calculating the amber time and clearance time
3. Calculation of cycle time
4. Calculation of green time for each phase
5. Checks for pedestrian crossing signal time
6. Performance evaluation.

2.4.2. Phase design

At the signalised junction conflicting movements can be separated by different phases. To eliminate all the conflicting movements more phases may be required. So signal design is done with less severe conflicts.

For example, let’s consider a cross junction with through traffic and right turns. Left turn is ignored. The first step in phase design is to find how many phases are required. We can have any number of phases. The basic phase arrangement starts with two phases.

2.4.2.1. Two phase signals

Two phase signals can be accommodated when the through movement volumes are relatively very higher compare to the right turn movements. Figure 2-3 shows a basic 2-phase signal system for a four-way junction. The through movements 1 and 2 are grouped in phase 1 and through movement 3 and 4 are grouped in the second phase. But the conflicting right turn 5 and 6 also allowed during phase 1 and conflicting right turn movement 7 and 8 allowed in phase 2. This kind of phase can only be provided when the turning movements are relatively low. If the turning movements are relatively high there is a high risk of accidents. So, a four-phase signal has to be provided.

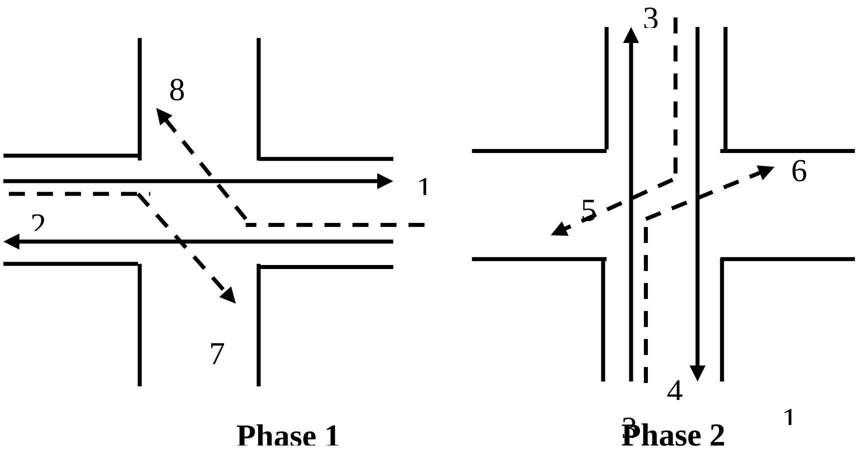


Figure 2-1 Example of a 2 phase system

2.4.2.2. Four phase signals

There are several phasing arrangements which we can have on a four-phase signal system. Figure 2-4 shows basic four phase signal system. In this type of phase arrangement all 4 direction approaches are given separate phases. So, there is no conflict. This phase type can be accommodated when the right turns are very high. If the right turning volume are relatively low this is ineffective.

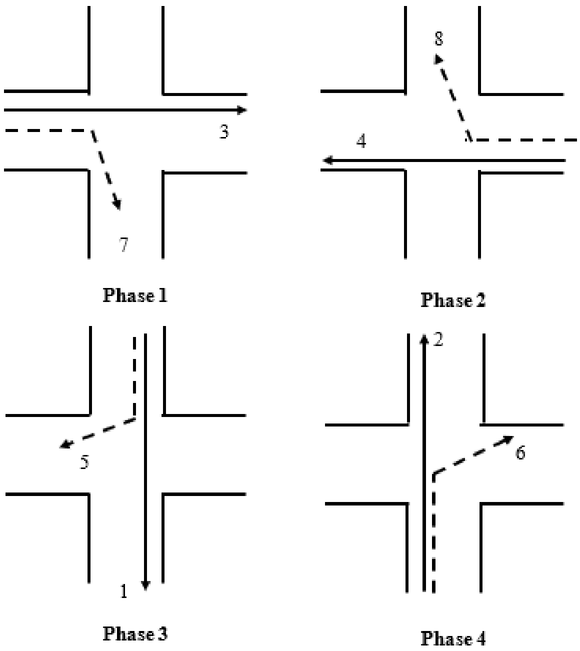


Figure 2-2 Example of a four-phase traffic signal arrangement -1

Figure 2-5 shows another type of a four-phase signal system. In this system opposite through movements are combined together and opposite right turns are combined together into different phases. This kind of phasing can be used when there is a separate lane for each movement.

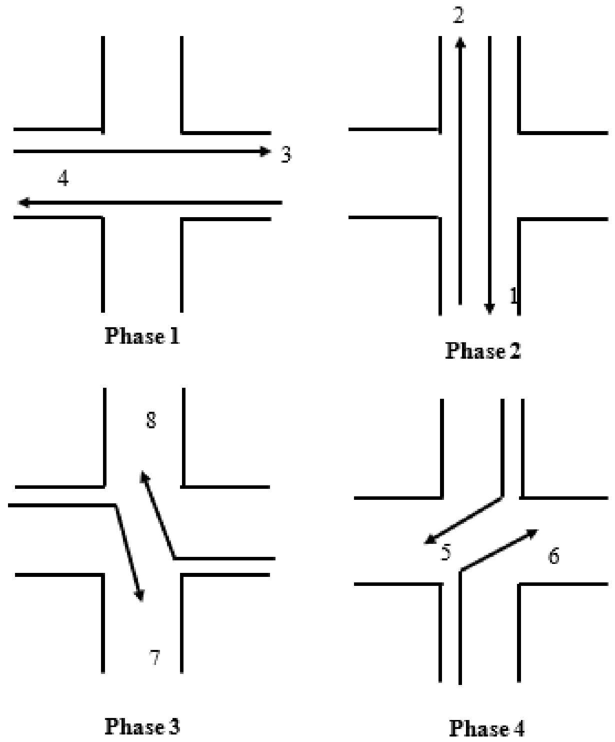


Figure 2-3 Example of a four phase traffic signal arrangement -2

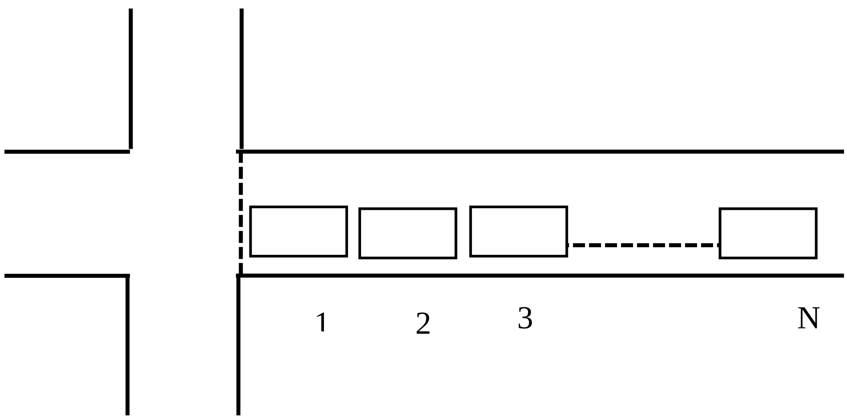


Figure 2-6 Group of vehicles at a signalized intersection waiting for green signal

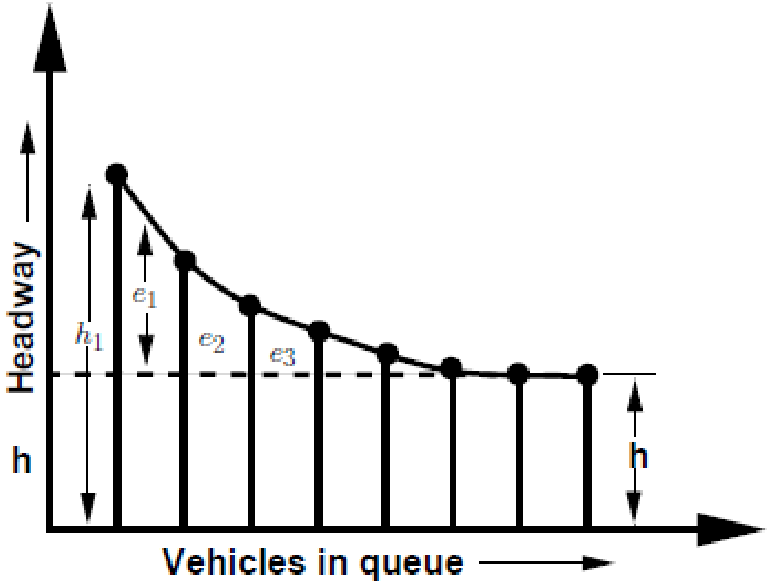


Figure 2-7 Headways departing signal [18]

2.4.2.3. Lost time

Figure 2-6 shows a group of vehicles queued and waiting for the green signal. When the green signal starts the time interval between two successive vehicles passing the stop line (head way) is taken. Figure 2-7 shows the headway vs vehicle graph. The headway h_1 includes the reaction time and the acceleration time. The second headway h_2 will be lesser than the first one as the reaction time is overlapped. Likewise, it reduces and there will be some constant headway h will be achieved. That constant headway h is defined as the saturation headway

If each vehicle requires h seconds to pass the stop line during the green time, and if the signal is always green, then the no of vehicles which can pass the intersection can be computed as S. where S is,

$$S = \frac{3600}{h} \tag{2.1}$$

The unit of S is veh/h/lane and saturation headway h measured in seconds.

For the first few vehicles the headway is slightly higher than the saturation headway. If the difference in saturation headway and actual headway for an i^{th} vehicle is e_i, additions of all e_i will give the starting lost time l_1.

$$l_1 = \sum_{i=1}^{n} e_i \tag{2.2}$$

The total time required to clear N number of vehicles from the intersection queue can be computed by,

$$T = l_1 + h.N \tag{2.3}$$

2.4.2.4. Effective green time

It is the effective time available for the vehicles to cross the signalised junction.

$$g_i = G_i + Y_i - l_1 - l_2 \tag{2.4}$$

Where,

G_i - Actual green time

l_1 - Start-up lost time

l_2 - Clearance lost time

Y_i - Amber time

2.4.2.5. Lane capacity

The ratio between the effective green time to the cycle time $\frac{g_i}{C}$ is called as the green ratio. If the signal is always green, saturation flow rate is the number of vehicles which can move in one lane in one hour. The capacity C_i of the lane is,

$$c_i = S_i \frac{g_i}{C} \tag{2.5}$$

2.4.2.6. Critical lane

When the phase design is done there can be several lanes with different flows combined into one phase. During green time the combined flows will be allowed to cross the intersection. Among the combined flows one of the directional flows will need more time to clear all the vehicles. Enough green time has to be given that high volume movement at the same time the low volume movements in that phase also can pass the intersection. The higher volume movement lane is the critical lane and the vehicle volume for the movement is the critical lane volume.

2.4.3. Cycle time

Signal Cycle time is defined by the time taken for a signal to complete one full cycle of indications. If t_{Li} is denoted by the starting lost time for a particular phase i, then the total starting lost time (L) for a signal cycle is,

$$L = \sum_{i=1}^{N} t_{Li} \tag{2.6}$$

If assumed the starting lost time for all the phases are equal then total starting lost time can be computed as $L = Nt_L$. If C is the cycle time in seconds then,

$$\text{Number of cycles within one hour} = \frac{3600}{C}$$

$$\text{Total lost time in hour is} = \frac{3600}{C} L$$

$$= \frac{3600\, N\, t_l}{C} \tag{2.7}$$

The effective green time (T_g) available for all vehicle movements will be,

$$T_g = 3600 - \frac{3600\, N\, t_L}{C}$$

$$= 3600 \left[1 - \frac{N\, t_L}{C}\right] \tag{2.8}$$

The total number of critical lane volume which can be accommodated per hour (V_c) can be computed as,

$$V_c = \frac{T_g}{h}. \tag{2.9}$$

from Equation-2.1 and Equation-2.8

$$V_c = \frac{3600}{h} \left[1 - \frac{Nt_L}{C}\right]$$

$$= s_i \left[1 - \frac{Nt_L}{C}\right]$$

$$\therefore\ \boldsymbol{C = \frac{Nt_L}{1 - \frac{V_c}{s}}} \tag{2.10}$$

The equation for cycle derived above is a basic equation. It doesn't consider the uniformity of the vehicle flow. The peak hour factor (PHF) can be accommodated to represent the peak hour variation. The v/c ratio also needs to be included to represent the quality of the service. By considering these two factors equation can be modified as,

$$C = \frac{Nt_L}{1 - \frac{V_c}{s_i \times PHF \times \frac{V}{C}}} \tag{2.11}$$

Webster & Cobbe (1966) has modified the cycle time model for optimum delays at the junction as follows [4].

$$C = \frac{1.5L + 5}{1 - \frac{\sum_{i=1}^{n} q_i}{S}} \tag{2.12}$$

2.4.4. Green splitting

The total effective green time has to be distributed among the phases. The green splitting is calculated by,

$$g_i = \left[\frac{V_{c_i}}{\sum_{i=1}^{N} V_{c_i}}\right] \times t_g \tag{2.13}$$

Where V_{c_i} - Critical lane volume

t_g - Total effective green time available in a cycle

t_g can be calculated by, $t_g = C - Nt_L$ (2.14)

The actual green time (G_i) on the posted signal is given by,

$$G_i = g_i - y_i + t_{L_i} \tag{2.16}$$

where y_i is the amber time, and t_{L_i} is the lost time for phase i.

2.4.5. Pedestrian crossing

Pedestrian crossing requirements has to be considered while designing signals. The pedestrian requirement can be satisfied by suitable phase design or by providing a separate pedestrian phase. To provide separate phase for pedestrians the phase time needed for the pedestrians has to be calculated. Green time for pedestrian crossing $G_p$can be calculated by,

$$G_p = t_s + \frac{dx}{up} \tag{2.17}$$

Where

t_s is the starting lost time

dx – width of the road

u_p – Average walking speed

2.4.6. Interval design

In traffic signal design there are two intervals called the change interval and clearance interval are considered.

Change interval

The change interval is the amber time provided after green time. This yellow time gives warning to the driver that the green time is going to end and the red signal is about to come. Amber time will be normally in the range of 3 to 4 seconds.

Clearance interval

The clearance interval is the all-red time. It is provided for a vehicle just crossed the stop line at the end of amber time to clear the intersection safely before the next phase starts.

$$R_{AR} = \begin{cases} \dfrac{w+L}{v} \;\; if\ no\ pedestrians \\ max\left(\dfrac{w+L}{v}, \dfrac{P}{v}\right) \; if\ pedestrian\ crossing \\ \dfrac{P+L}{v} \; if\ protected \end{cases} \tag{2.18}$$

Where

w - Width of the intersection

L - Length of the vehicle

v – Average Speed of the vehicle

P - Width of the intersection between the STOP line and pedestrian cross walk.

2.5. Signalized Intersection Delay Models

2.5.1. Overview

There are several parameters used in the analysis and simulation in traffic engineering such as average delay per vehicle, average queue length, and number of stops, etc. All of them are some indication of drivers experience while going through the intersection. The most important measurement is delay. The delay at intersection means, the additional time spent at the junction by the vehicle.

Queue lengths also an important parameter to consider at the junctions. When junctions are located too close, then queue length is a critical parameter to measure. Number of stops and stop time also an important parameter to measure at the junctions. Determination of delay is complex because of the random arrival of vehicles.

2.5.2. Types of delay

The major parameter to measure the quality of the traffic operation is delay. Queue length is also considered as a secondary parameter to assess the quality of the operation. Measuring the delay on the field is possible but it is a difficult process. Therefore, there should be a predictive model to calculate the delay. There were several people who had done research's on different delay models. During their analysis they made several assumptions for the arrival of vehicles for their models because of the randomness in the vehicle arriving. [3][4].

Delay can be quantified in many different ways. The most widely used types of delay forms are,

- Total vehicle delay
- Approaching delay

- Stop delay
- Time-in-queue delay
- Control delay

Travel Time Delay

The travel time delay is the difference in the actual travel time taken to pass the intersection and the expected time to pass the intersection when there is no control at the intersection.

Approach Delay

It is the combination of stop delay and the lost time due to the deceleration to stop the vehicle and acceleration to go back to the normal speed.

Stop Delay

It is the total time the vehicle has been stopped at the intersection to pass through. The stop delay starts when the vehicle is stopped completely at the intersection and ends when the vehicle stars to accelerate.

Time-in-queue Delay

This delay is the time interval which starts when the vehicle is joining a queue at an intersection and ends when the vehicle crosses the stop line.

Control Delay

It is the delay caused by any control devices at the intersection such as traffic signals, traffic police control or stop sign control.

Delay can be expressed in two ways. Delays for all the vehicles can be averaged and expressed as delay per vehicle (s/veh). Or the delay can be expressed as the total delay undergone by all the vehicles (vehh).

2.6. Vehicle actuated signals

Traffic signals operate by fixed cycle time or vehicle actuated times or some combination of the two. Fixed cycle time control consists of a series of intervals that are fixed in duration. In vehicle actuated signals cycle time and phases will be adjusted according to the presence of vehicles or pedestrians at the intersection. Vehicle detectors are set at the intersection to track the vehicle arrivals. The actuated controllers are capable of not only varying the cycle length & green times in response to detector actuation, but of altering the order and sequence of phases.

Adaptive or area traffic control systems (ATCS) belong to the latest generation of signalized intersection control. ATCS continuously detect vehicular traffic volume, compute optimal signal timings based on this detected volume and simultaneously implement them. There are three basic types of actuated control, each using signal controllers that are somewhat different in their design:

1. Semi-Actuated Control
2. Full-Actuated Control
3. Volume-Density Control

2.7. Micro Simulation

2.7.1. Traffic simulation

The behaviour of traffic stream is complex and doing experiments in the real world is difficult. So, computer-based traffic simulations become effective in analysing various experiments. Traffic simulation models are classified into three according to the accuracy level needed. They are,

1. Macroscopic models
2. Mesoscopic models
3. Microscopic models

In macroscopic model, traffic stream is represented in an aggregated level. Measurements are made in terms of speed, flow, capacity, density, etc. vehicle activities and interactions are considered at a lower level. A mesoscopic model generally represents most entities at a high level of detail but describes their activities and interactions at a much lower level of detail. A microscopic model describes both the system entities and their interactions at a high level of detail. Car following models and lane changing models are some significant examples. The considered area and type of analysis defines which level of modelling has to be done.

2.7.2. Introduction to micro simulation

Micro simulations are now widely used in several transportation analysis and management as it is safe, less cost and faster than implementing on the field and check the suitability. Through micro simulations new improvement plans can be effectively analysed with less cost. For example, new signal designs can be checked with micro simulation before implementing on the field.

2.7.3. Introduction to VISSIM

VISSIM software will be used in this study for the analysis of delay and queue lengths. Vissim is a microscopic time step and behaviour-based simulation model. In VISSIM the vehicle flow and pedestrian flow both can be analysed. We can analyse the public transport services also through VISSIM. Through VISSIM 2D and 3D visualizations are possible. In VISSIM we can accommodate several lanes behavioural such as vehicles can move anywhere on the link without restrictions. We can have several number of vehicle categories to analyse. Overtaking also can be modelled on both left and right. These facilities are helpful to model heterogeneous traffic like in Sri Lanka.

There are many advantages of VISSIM over other micro-simulation software. VISSIM has the ability to model the interaction between the various modes of transit with automobile traffic, ability to generate vehicles randomly and flexibility in modelling complex geometries. VISSIM is better in terms of ease of use and does not require coding.

There are several researches done using the traffic simulation software VISSIM. Z. Yang, P. Liu, Y. Chen, and H. Yu has done a survey on “Can Left-turn Waiting Areas Improve the Capacity of Left-turn Lanes at Signalized Intersections” by the micro simulation approach using VISSIM. J. Wahlstedt, has done a research on “Impacts of Bus Priority in Coordinated Traffic Signals, using VISSIM [20]. In Sri Lanka also, there are several people doing researches using the VISSIM software.

There are some other micro simulation software’s such as TRANSYT-7F, Synchro, Sidra, PARAMICS and etc. also available in the industry. There are several people done researches using these software’s as well [8][19][23].

2.7.4. Calibration of vissim software

Before using the software for analysis any model created in VISSIM has to be calibrated to represent the local conditions. The model should represent the field conditions to generate accurate results. Calibration is the process in which the input parameters are refined so that the model accurately replicates observed traffic conditions. In calibration, the parameters are adjusted so that the model outputs are similar to observed data. In VISSIM simulation software, there are more than 30 parameters which can be changed for modelling the driver behaviour patterns.

S. M. P. Siddharth and G. Ramadurai, has done calibration of vissim driving behavioural parameters for Indian heterogeneous traffic conditions [9]. They found that the parameters that can be calibrated in VISSIM for their conditions are acceleration, desired speed, and clearance distance, emergency stopping distance, waiting time before diffusion, lane change distance, standstill distance, minimum headway, Lateral standstill distance and lateral distance while driving. Some other researchers also used more or less the same parameters for the calibration for their local conditions [5] [11] [27].

Several researchers have used several control parameters for calibration such as travel time, queue length, headway, flow rates and delays [6] [27]. There are several ways to calculate the error related with the control parameters during the calibration. One of the common measures of error is root mean square error (RMSE). RMSE is defined as,

$$RMSE = \sqrt{\frac{\sum_{i=1}^{N}(x_i - y_i)^2}{N}}$$

Where,

x_i – Simulation value

y_i – Actual value

N – No of samples considered

3. DATA COLLECTION

3.1. Introduction

For the analysis appropriate intersections has to be identified. As per the research requirement intersections had to be congested during the peak hours and they should be controlled manually by traffic police officers during the peak hours. Such 4 intersections were identified and selected for the analysis. As the traffic police control phase arrangement and time wasn't constant, Kesbewa junction which was controlled by traffic signals during the peak hour was selected for the calibration. The selected intersections are,

1. Kohuwala junction
2. Katubedda junction
3. Maliban junction
4. Golumadama junction
5. Kesbewa Junction

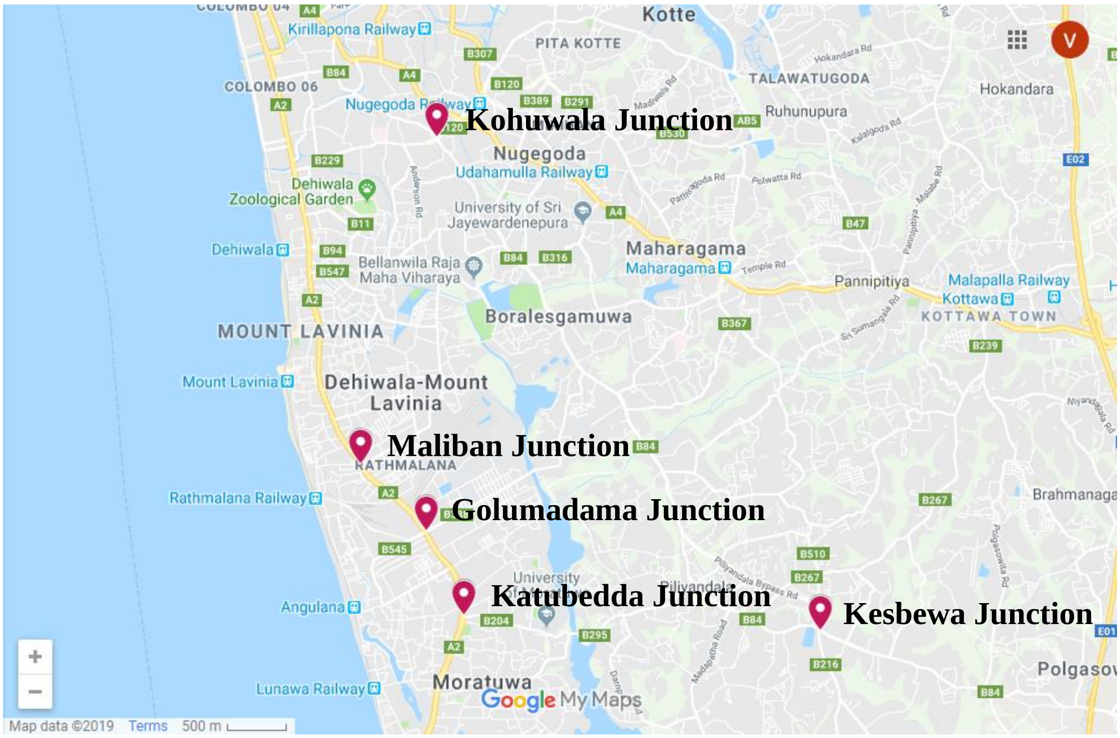


Figure 3-1 Selected Junctions for the analysis

3.1.1. Kesbewa Junction

This junction was specially selected for the calibration of the vissim model. It has proper lane markings and controlled by traffic signals during the morning peak hour. It is a 4-way intersection consist of Colombo-Horana road and Piliyandala by pass road.

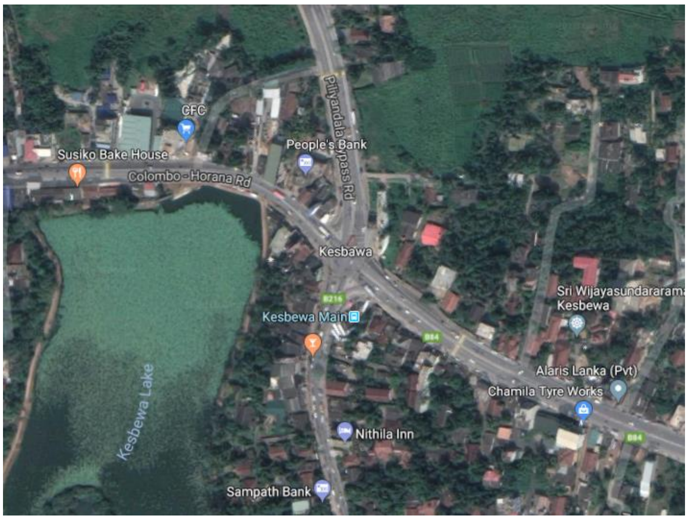


Figure 3-2 Kesbewa junction – map

3.1.2. Maliban Junction

This is one of the three selected junctions for the analysis along the Galle– Colombo road. It is a T junction where Ratmalana-Attidiya road meets the Colombo-Galle road. Colombo-Galle road is one of the most congested roads during the peak hours as the Maliban junction too congested during the peak hours. This junction is controlled manually by traffic police officers during the evening peak hours.

Figure 3-3 Maliban Junction - map

3.1.3. Golumadama Junction

This junction is a 4-way junction where kaldemulla road and Ratmalana-Borupana road meets the Colombo-Galle road. This junction also congested during the peak hours and controlled manually by traffic police officers during the peak hours.

Figure 3-4 Golumadama junction - map

3.1.4. Katubedda Junction

Katubedda junction is one of the three selected junctions for the analysis along the Galle– Colombo road. This is a T junction where Bandaranayke Mawatha meets the Colombo-Galle road. This Bandaranayke Mawatha is also a major traffic moving road as most of the vehicles from Kottawa, Piliyandala, Katubedda and university of Moratuwa are using those roads to come to the Colombo-Galle road. This junction also controlled by traffic police officers during the evening peak hours.

Figure 3-5 Katubedda junction – map

3.1.5. Kohuwala Junction

It is one of the most congested junctions during morning and evening peak hours inside the Colombo city. It located in-between the Nugegoda junction, Kalubowila teaching hospital and Pepiliyana Junction. Kohuwala Junction is a four-way junction and controlled by traffic police officers during the peak hours.

Figure 3-6 Kohuwala Junction – map

Reconnaissance survey has been done on these junctions to collect information about the traffic and geometric conditions of these junctions before the start of data collection. Among the 5 intersection the Kesbewa Junction has been selected for the Calibration of the VISSIM model and the other four junctions were selected for the analysis part as they were heavily congested junctions during the peak hours and they were controlled manually by the traffic police officers during the peak hours. Among them Kohuwala junction and Golumadama junction are four-way junctions and maliban junction and Katubedda junction are T junctions.

For the modelling and analysis several data need to be collected. The Geometric data, Traffic flow, turning movements, Vehicle Classified count, Queue Lengths, timing and phasing arrangement of existing traffic signals and traffic police were collected in all the junctions.

3.2. Geometric Data

The geometric arrangement of the intersection is essential for the vissim modelling to ensure the results. The important geometric parameters like number of lanes, lane widths, length of the turning lanes, pedestrian crossings were collected in the junctions. Figure 3-7, Figure 3-8, Figure 3-9 and Figure 3-10 shows the geometric arrangement of the selected junctions.

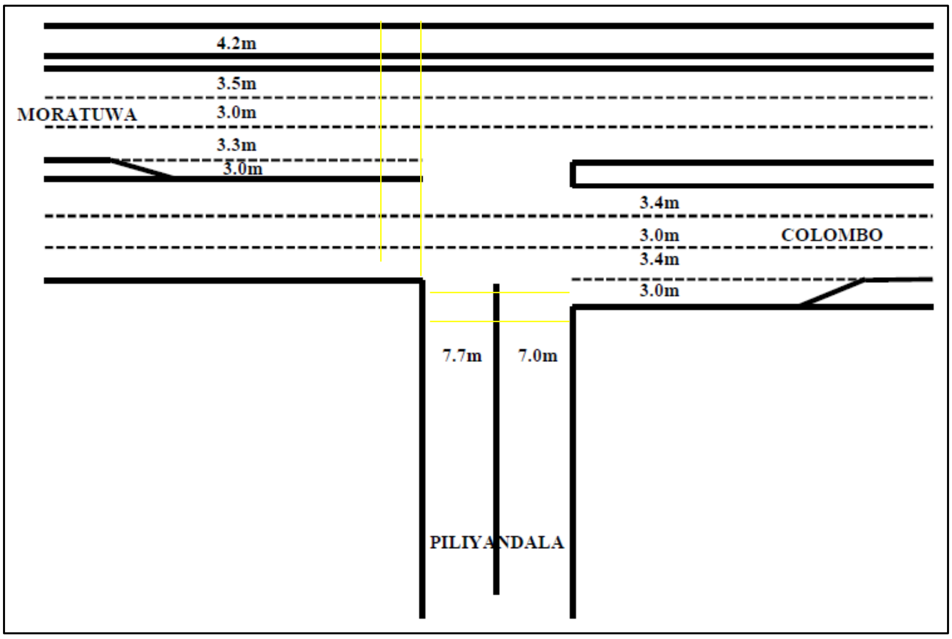


Figure 3-7 Geometric Arrangement at Katubedda Junction

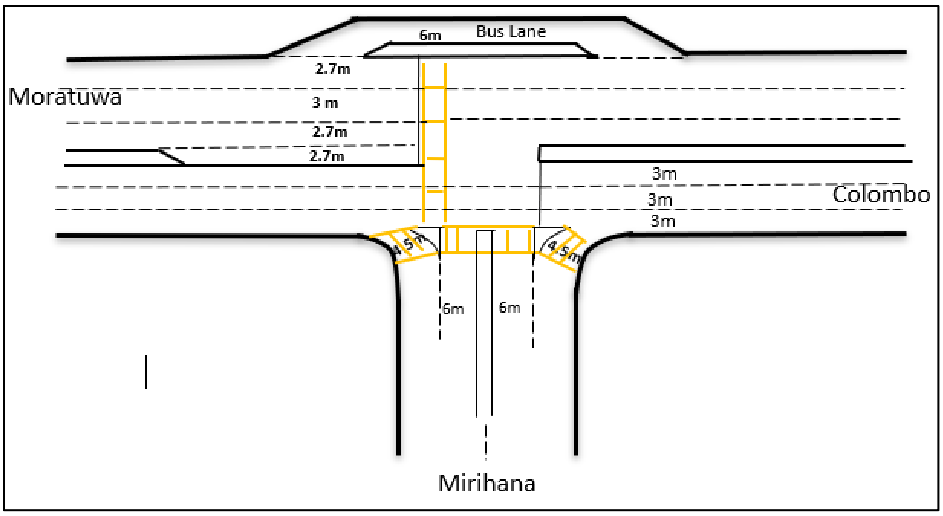


Figure 3-8 Geometric Arrangement at Maliban Junction

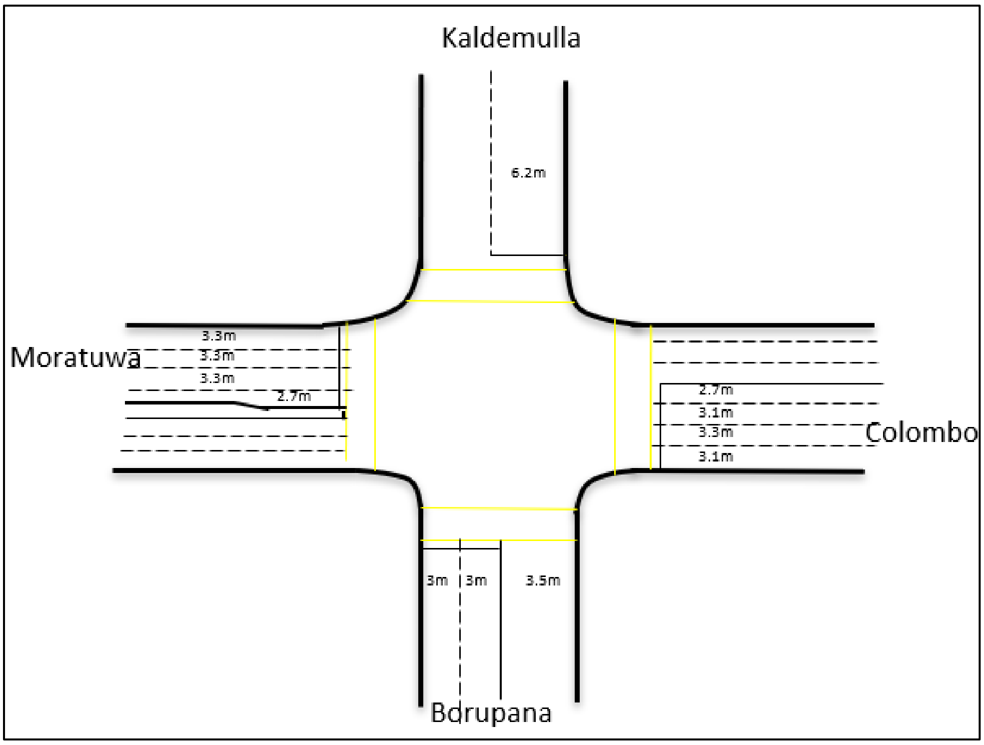


Figure 3-9 Geometric Arrangement at Golumadama Junction

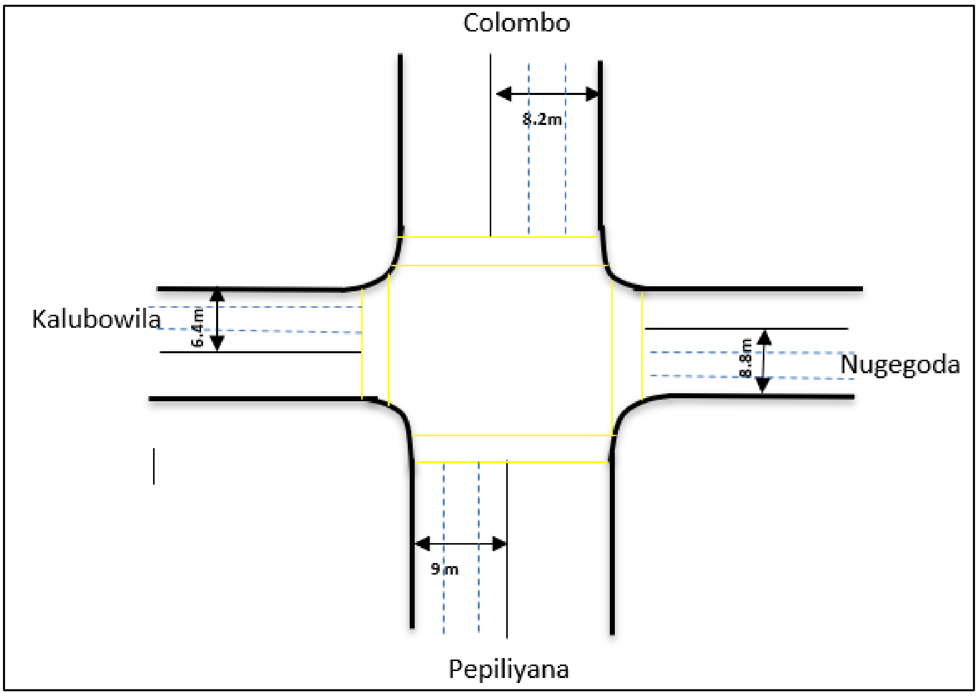


Figure 3-10 Geometric Arrangement Kohuwala Junction

3.3. Traffic flow, turning movements, Vehicle Classified count and Queue Lengths

From the previous studies it was identified that the evening peak at these junctions occurs from 5:00 p.m. to 7:00 p.m. So, the traffic surveys were conducted from 5:00 p.m. to 7:00 p.m. The turning movements and vehicle classified counts were taken through traffic video survey at the junction and the traffic flow at the junctions were counted manually on the field and verified with the Video count. The camera positions were selected during the reconnaissance survey by considering the safety and coverage of the junction area. Vehicle classified count has been taken in all junctions for 8 different vehicle categories (Motor Bike, Three-wheeler, Car, Van, Medium good Vehicle, Large Good Vehicle, Container and Bus).

At the Kesbewa junction, which is selected for the calibration of vissim model, the maximum queue length per each signal cycle for all 4 directions has been gathered for the calibration process. For the calibration minimum of one-hour flow data needed and that calibrated parameters needs to be validated. So total of two-hour data has been collected. The survey there had been conducted from 7:00 a.m. to 9:00 a.m. where the first hour data had been used for the calibration process and second hour data used for the validation of the calibrated parameters.

Traffic data (traffic flow, vehicle composition and turning movements) collected at the selected junctions are presented below.

3.3.1. Kohuwala Junction

Peak hourly junction flow (5839 veh) at the Kohuwala junction is identified from 5:15 pm to 6:15 pm. The data has been collected for three continuous days (Wednesday, Thursday and Friday) and maximum flow occurs on Wednesday and Least flow occurs on Friday. Higher number of vehicles were approaching from the Colombo direction and it was the most critical direction in the junction during the evening peak hours.

Table 3-1 Kohuwala Junction Vehicle Flow Day 1 (Wednesday)

Vehicle Flow - Kohuwala Junction Day1							
Vehicle approaching from		Nugegoda	Pepiliyana	Kalubowila Hospital	Colombo	Junction Flow	Hourly flow
Start time	End Time						
17:00	17:15	338	301	228	504	1371	
17:15	17:30	348	304	252	629	1533	
17:30	17:45	445	283	219	491	1438	
17:45	18:00	402	331	156	532	1421	5763
18:00	18:15	443	276	197	531	1447	**5839**
18:15	18:30	363	315	`210	402	1290	5596
18:30	18:45	317	321	210	445	1293	5451
18:45	19:00	312	306	201	421	1240	5270
Peak Hour Flow		**1638**	**1194**	**824**	**2183**	**5839**	

Table 3-2 Kohuwala Junction Vehicle Flow Day 2 (Thursday)

Vehicle Flow - Kohuwala Junction Day2							
Vehicle approaching from		Nugegoda	Pepiliyana	Kalubowila Hospital	Colombo	Junction Flow	Hourly flow
Start time	End Time						
17:00	17:15	305	276	207	489	1277	
17:15	17:30	348	304	252	629	1533	
17:30	17:45	374	301	236	504	1415	
17:45	18:00	321	295	187	578	1381	5606
18:00	18:15	306	285	224	486	1301	5630
18:15	18:30	375	283	212	392	1262	5359
18:30	18:45	302	325	206	432	1265	5209
18:45	19:00	317	323	196	421	1257	5085
Peak Hour Flow		**1349**	**1185**	**899**	**2197**	**5630**	

Table 3-3 Kohuwala Junction -Manual classified count percentage

Vehicles approaching from	**Car**	**Motor bike**	**Three -wheel**	**Van / jeep**	**Bus**	**LGV**	**HGV**	**Cont ainer**
Nugegoda	31.8%	27.2%	30.5%	4.6%	2.4%	1.8%	1.8%	0.0%
Pepiliyana	34.6%	19.8%	39.3%	1.6%	2.3%	1.9%	0.4%	0.0%
Kalubowila	33.5%	20.4%	37.6%	3.6%	2.7%	1.4%	0.9%	0.0%
Colombo	33.7%	37.6%	16.8%	5.2%	3.0%	2.6%	1.1%	0.0%

Table 3-4 Kohuwala Junction Vehicle Flow Day 3 (Friday)

Vehicle Flow - Kohuwala Junction Day3							
Vehicle approaching from		Colombo	Pepiliyana	Nugegoda	Kalubowila Hospital	Junction Flow	Hourly flow
Start time	End Time						
17:00	17:15	338	301	228	504	1371	
17:15	17:30	380	286	161	530	1357	
17:30	17:45	338	278	217	551	1384	
17:45	18:00	348	325	214	460	1347	5459
18:00	18:15	443	276	197	531	1447	5535
18:15	18:30	363	315	210	402	1290	5468
18:30	18:45	317	321	210	445	1293	5377
18:45	19:00	312	306	201	421	1240	5270
Peak Hour Flow		**1537**	**1509**	**1165**	**789**	**5535**	

Table 3-5 Kohuwala Junction Peak hour Flows - Daily Variation

Peak Hour flow						Percentage change from Wed flow
Day	Pepiliyana	Kalubowila Hospital	Nugegoda	Colombo	Junction Flow	
Wed	1194	824	1638	2183	5839	0
Thu	1168	836	1536	2071	5630	-3.91%
Fri	1165	789	1537	1537	5535	-5.21%

Table 3-6 Kohuwala Junction Turning Movements Day 1

Turning Movements at Kohuwala Junction Day 1							
From Nugegoda				**From Pepiliyana**			
Time	Through	Right	Left	Time	Through	Right	Left
0-15	134	38	176	0-15	177	78	49
15-30	197	32	216	15-30	167	83	33
30-45	157	34	211	30-45	199	101	31
45-60	188	38	217	45-60	175	75	26
60-75	183	34	146	60-75	196	92	27
75-90	148	16	153	75-90	220	75	26
From Kalubowila Hospital				**From Colombo**			
Time	Through	Right	Left	Time	Through	Right	Left
0-15	181	56	15	0-15	553	5	71
15-30	181	28	10	15-30	406	15	70
30-45	122	26	8	30-45	458	7	67
45-60	152	37	8	45-60	450	6	75
60-75	154	47	9	60-75	356	6	40

Table 3-7 Kohuwala Junction Turning Movements Day 2

Turning Movements at Kohuwala Junction Day 2								
From Nugegoda					**From Pepiliyana**			
Time	Through	Right	Left		Time	Through	Right	Left
0-15	134	38	176		0-15	177	78	49
15-30	197	32	216		15-30	167	83	33
30-45	157	34	211		30-45	199	101	31
45-60	188	38	217		45-60	175	75	26
60-75	183	34	146		60-75	196	92	27
75-90	148	16	153		75-90	220	75	26
From Kalubowila Hospital					**From Colombo**			
Time	Through	Right	Left		Time	Through	Right	Left
0-15	181	56	15		0-15	553	5	71
15-30	181	28	10		15-30	406	15	70
30-45	122	26	8		30-45	458	7	67
45-60	152	37	8		45-60	450	6	75
60-75	154	47	9		60-75	356	6	40
75-90	158	45	7		75-90	401	4	40

Table 3-8 Kohuwala Junction Turning Movements Day 3

Turning Movements at Kohuwala Junction Day 3								
From Nugegoda					**From Pepiliyana**			
Time	Through	Right	Left		Time	Through	Right	Left
0-15	134	38	176		0-15	177	78	49
15-30	197	32	216		15-30	167	83	33
30-45	157	34	211		30-45	199	101	31
45-60	188	38	217		45-60	175	75	26
60-75	183	34	146		60-75	196	92	27
75-90	148	16	153		75-90	220	75	26
From Kalubowila Hospital					**From Colombo**			
Time	Through	Right	Left		Time	Through	Right	Left
0-15	181	56	15		0-15	553	5	71
15-30	181	28	10		15-30	406	15	70
30-45	122	26	8		30-45	458	7	67
45-60	152	37	8		45-60	450	6	75
60-75	154	47	9		60-75	356	6	40
75-90	158	45	7		75-90	401	4	40

3.3.2. Katubedda Junction

Table 3-9 Katubedda junction - Vehicle flow

Vehicle approaching from		Colombo	Moratuwa	Piliyandala	Junction Flow	Hourly flow
Start time	End Time					
17:00	17:15	734	437	162	1333	
17:15	17:30	786	480	177	1443	
17:30	17:45	830	509	225	1564	
17:45	18:00	840	472	228	1540	5880
18:00	18:15	764	461	205	1430	5977
18:15	18:30	773	460	225	1458	**5992**
18:30	18:45	780	401	171	1352	5780
18:45	19:00	783	395	167	1345	5585
Peak Hour Flow		**3207**	**1902**	**783**	**5977**	

Table 3-10 Katubedda junction – Turning movements

From Colombo			
Time	Through	Right	Left
0-15	635	-	201
15-30	653	-	195
30-45	558	-	209
45-60	585	-	183
60-75	587	-	198
From Moratuwa			
Time	Through	Right	Left
0-15	391	113	-
15-30	377	100	-
30-45	355	98	-
45-60	368	93	-
60-75	320	84	-
From Piliyandala			
Time	Through	Right	Left
0-15	-	112	80
15-30	-	116	94
30-45	-	104	87
45-60	-	109	101
60-75	-	77	73

Table 3-11 Katubedda junction- Manual classified count

Katubedda junction				
From Colombo				
Time	Motor Bikes	Three wheel	Car/van/ LGV	Bus/HG V
0-15	271	181	345	39
15-30	273	170	374	31
30-45	233	157	351	26
45-60	248	151	344	25
60-75	232	165	367	21
From Piliyandala				
0-15	63	57	111	11
15-30	61	55	127	7
30-45	59	36	131	5
45-60	68	45	133	4
60-75	47	31	106	6
From Moratuwa				
0-15	140	132	208	24
15-30	129	115	201	32
30-45	116	90	219	28
45-60	104	96	225	36
60-75	98	85	196	25

At Katubedda junction peak junction flow of 5977veh/h occurred from 5:30 pm to 6:30 pm. The highest peak directional flow of 3207veh/h had approached from the Colombo direction during the peak hour. At Katubedda junction 23.62% vehicles were motorbikes, 23.67% vehicles were three wheelers. These two vehicle categories motorbikes and three wheelers were 47.3% of the total number of vehicles.

3.3.3. Maliban Junction

At Maliban junction peak junction flow of 5986veh/h occurred from 5:30 pm to 6:30 pm. The highest peak directional flow of 2475veh/h had approached from the Moratuwa direction during the peak hour. At the maliban junction 28.69% vehicles were motorbikes, 20.97% vehicles were three wheelers. These two vehicle categories motorbikes and three wheelers were 49.66% of the total number of vehicles.

Table 3-12 Maliban junction vehicle flow

Vehicle approaching from		Colombo	Moratuwa	Mirihana	Junction Flow	Hourly flow
Start time	End Time					
17:00	17:15	481	594	291	1366	
17:15	17:30	490	597	287	1374	
17:30	17:45	540	615	304	1459	
17:45	18:00	571	653	309	1533	5732
18:00	18:15	544	611	340	1495	5861
18:15	18:30	552	596	352	1500	5986
18:30	18:45	521	567	344	1432	5960
18:45	19:00	508	543	330	1381	5808
	Peak Hour Flow	**2207**	**2475**	**1305**	**5986**	

Table 3-13 Maliban junction – Turning movements

From Colombo			
Time	Through	Right	Left
0-15	524	-	19
15-30	555	-	21
30-45	523	-	20
45-60	527	-	17
60-75	500	-	18
75-90	495	-	16
From Moratuwa			
Time	Through	Right	Left
0-15	456	152	-
15-30	489	156	-
30-45	437	169	-
45-60	427	162	-
60-75	414	153	-
75-90	398	145	-
From Mirihana			
Time	Through	Right	Left
0-15	-	37	267
15-30	-	41	273
30-45	-	31	314
45-60	-	46	306
60-75	-	39	301
75-90	-	36	296

Table 3-14 Maliban junction – Manual classified count

Maliban junction				
From Moratuwa				
Time	Motor Bikes	Three - wheel	Car/van/ LGV	Bus/HGV
0-15	128	157	262	61
15-30	136	165	274	70
30-45	124	152	267	63
45-60	109	166	263	51
60-75	117	143	259	48
75-90	121	138	243	41
From Colombo				
0-15	137	98	260	48
15-30	145	112	278	41
30-45	131	106	253	53
45-60	138	132	236	38
60-75	124	127	238	29
75-90	148	104	225	34
From Mirihana				
0-15	83	70	140	11
15-30	79	76	151	8
30-45	86	83	164	12
45-60	94	86	157	15
60-75	90	80	160	10
75-90	84	84	151	13

3.3.4. Golumadama Junction

Table 3-15 Golumadama junction – Vehicle flow

Vehicle Flow – Golumadama Junction							
Vehicle approaching from		Colombo	Moratuwa	Borupana	Kaldemulla	Junction Flow	Hourly flow
Start time	End Time						
17:00	17:15	722	501	79	68	1370	
17:15	17:30	756	530	107	63	1456	
17:30	17:45	800	559	137	68	1564	
17:45	18:00	936	495	104	87	1622	6012
18:00	18:15	891	566	98	73	1628	6270
18:15	18:30	881	556	97	81	1615	6429
18:30	18:45	870	525	89	72	1556	6421
18:45	19:00	873	519	94	64	1550	6349
	Peak Hour Flow	**3508**	**2176**	**436**	**309**	**6429**	

Table 3-16 Golumadama junction - Turning Movements

Golumadama Turning Movements					
From Colombo					
Time Interval		Through	Left	Right	U turn
17:00	17:15	650	38	27	7
17:15	17:30	680	37	31	8
17:30	17:45	716	43	32	9
17:45	18:00	856	40	31	9
18:00	18:15	799	45	39	8
18:15	18:30	795	41	38	7
From Moratuwa					
Time Interval		Through	Left	Right	U turn
17:00	17:15	443	19	35	4
17:15	17:30	457	18	47	8
17:30	17:45	477	23	52	7
17:45	18:00	422	32	33	8
18:00	18:15	485	44	31	6
18:15	18:30	474	40	35	7
From Borupana					
Time Interval		Through	Left	Right	U turn
17:00	17:15	15	45	39	0
17:15	17:30	16	48	46	0
17:30	17:45	26	56	55	0
17:45	18:00	14	55	35	0
18:00	18:15	19	45	34	0
18:15	18:30	17	50	30	0
From Kaldemulla					
Time Interval		Through	Left	Right	U turn
17:00	17:15	19	21	23	0
17:15	17:30	21	18	18	0
17:30	17:45	24	25	24	0
17:45	18:00	19	53	19	0
18:00	18:15	33	18	33	0
18:15	18:30	30	31	30	0

Table 3-17 Golumadama Junction – Manual classified count

Golumadama Junction MCC									
Time	**Motor bike**	**Car**	**Three wheel**	**Van/ Jeep**	**Freight**			**Bus**	**Total**
					Light	**Medi**	**Large**		
From Colombo									
0-15	221	263	167	36	51	17	4	35	794
15-30	290	322	179	55	27	12	2	46	933
30-45	240	322	190	58	36	11	3	31	891
45-60	235	310	185	58	35	12	4	35	874
From Moratuwa									
0-15	112	175	152	43	39	14	3	21	559
15-30	109	161	116	52	18	15	1	20	492
30-45	142	175	151	34	20	11	2	26	561
45-60	140	168	147	38	17	18	1	24	553
From Kaldemulla									
0-15	22	12	22	2	3	5	0	0	66
15-30	30	21	27	6	1	2	0	0	87
30-45	24	16	19	7	3	2	0	0	71
45-60	28	17	19	6	4	3	1	0	78
From Borupana									
0-15	40	45	33	5	5	2	1	1	132
15-30	36	32	23	6	2	0	0	1	100
30-45	29	33	30	3	2	1	0	0	98
45-60	27	30	23	4	4	2	0	0	90

3.3.5. Kesbewa Junction

Table 3-18 Kesbewa Junction – Vehicle flow

Vehicle Flow – Kesbewa Junction				
Vehicles From	**Kahathuduwa**	**Makandana**	**Piliyandala**	**Piliyandala By pass**
0-15	479	782	125	237
15-30	542	591	157	225
30-45	468	663	186	232
45-60	421	599	183	254
Total	**1910**	**2635**	**651**	**948**
60-75	404	430	167	251
75-90	355	470	188	221
90-105	305	385	189	235
105-120	245	458	210	198
Total	**1309**	**1743**	**754**	**905**

Table 3-19 Kesbewa junction - Turning movement

Turning Movement - Kesbewa Junction								
Vehicles from Kahathuduwa					**Vehicles from Makandana**			
Time	Left turn	Right turn	through		Time	Left turn	Right turn	through
0-15	82	273	124		0-15	160	65	557
15-30	96	330	116		15-30	158	67	366
30-45	89	313	66		30-45	144	75	444
45-60	78	244	99		45-60	144	58	397
Total	**345**	**1160**	**405**		**Total**	**606**	**265**	**1764**
60-75	80	252	72		60-75	113	40	277
75-90	65	200	90		75-90	155	57	258
90-105	48	133	124		90-105	145	42	198
105-120	46	118	82		105-120	160	45	253
Total	**239**	**703**	**368**		**Total**	**573**	**184**	**986**
Vehicles from Piliyandala					**Vehicles from Piliyandala By pass**			
Time	Left turn	Right turn	through		Time	Left turn	Right turn	through
0-15	7	55	63		0-15	91	8	138
15-30	9	67	81		15-30	101	15	109
30-45	14	93	79		30-45	99	13	120
45-60	18	87	78		45-60	98	14	142
Total	**48**	**302**	**301**		**Total**	**389**	**50**	**509**
60-75	17	75	75		60-75	84	22	145
75-90	17	76	95		75-90	84	14	123
90-105	19	82	88		90-105	83	20	132
105-120	18	113	80		105-120	100	17.5	80
Total	**71**	**346**	**338**		**Total**	**351**	**74**	**480**

Table 3-20 Kesbewa Junction - Average of Maximum queue length per cycle

Direction	Average of Maximum queue length per cycle (m)	
	7:00 am to 8:00 am	8:00 am to 9:00 am
Kahathuduwa	78	62
Makandana	123	69
Piliyandala	66	76
Piliyandala By pass	103	55

s

Table 3-21 Kesbewa Junction – Manual classified count

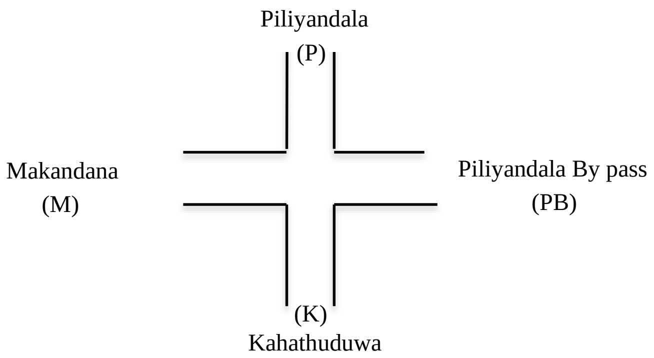
Manual Classified Count - Vehicles from Kahathuduwa								
Time	**Motor bike**	**Car**	**Three wheels**	**Van/ Jeep**	**Freight**			**Bus**
					Light	**Medium**	**Large**	
0-15	249	75	104	20	12	4	3	12
15-30	290	84	106	34	11	4	2	11
30-45	218	82	112	36	11	2	0	7
45-60	189	71	95	33	12	5	5	11
60-75	157	75	83	37	18	16	4	14
75-90	118	80	90	25	19	6	5	12
90-105	108	40	82	32	16	13	3	11
105-120	83	55	50	23	8	8	3	18
Manual Classified Count - Vehicles from Makandana								
0-15	445	107	150	43	18	10	3	6
15-30	316	100	131	16	9	9	5	5
30-45	326	111	143	38	24	11	5	5
45-60	296	105	108	56	15	11	3	5
60-75	180	72	101	35	21	12	2	7
75-90	177	91	113	46	24	15	2	2
90-105	146	64	106	39	15	8	4	3
105-120	183	63	105	33	38	25	8	5
Manual Classified Count - Vehicles from Piliyandala								
0-15	39	19	33	13	4	2	1	14
15-30	54	24	41	12	6	3	2	15
30-45	51	30	57	15	9	3	1	20
45-60	55	30	55	12	10	2	3	16
60-75	49	27	60	6	2	4	2	17
75-90	49	26	68	13	7	8	1	16
90-105	51	25	57	17	12	9	2	16
105-120	48	25	70	28	8	15	0	18
Manual Classified Count - Vehicles from Piliyandala By pass								
0-15	82	65	52	20	5	8	4	1
15-30	40	65	57	32	15	7	9	0
30-45	95	59	43	19	6	5	5	0
45-60	98	69	41	25	10	7	4	0
60-75	87	49	69	26	9	9	2	0
75-90	70	49	47	32	8	6	7	2
90-105	81	43	53	28	12	11	6	1
105-120	68	25	65	25	5	8	0	3

3.4. Intersection Control Data

All the four junctions selected for the analysis were controlled manually by traffic police officers during the evening peak hours which is essential for the analysis. At Kesbewa junction which was selected for the calibration process, during the morning peak hour the junction was controlled by traffic signals.

To simulate the manual control in vissim software, the cycle time, phase time and phase arrangement used by the manual controllers has been taken for each cycle of control during the considered peak hour. The average of that phase time and arrangement was modelled as a traffic signal with some constrained conflicting movement in the VISSIM software to simulate what was done manually on the field. Existing posted signal time on the intersection has also been taken to check the suitability. But, currently posted signal time could be outdated.

Table 3-20 Kesbewa junction - Current traffic signal phase and time



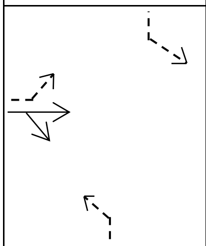
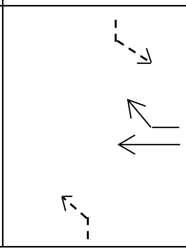
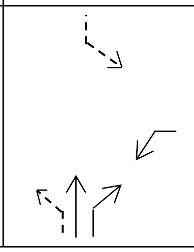
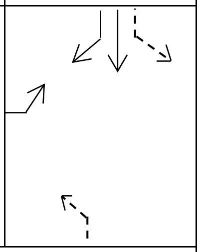
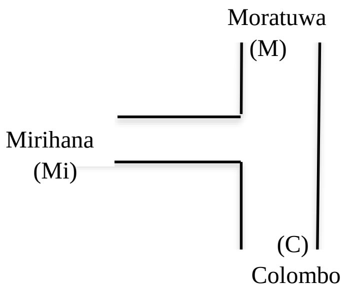
Phase	1	2	3	5
				
Cycle Time(s)	160 s			
Green Time	40	20	65	20

Table 3-21 Maliban Junction – Existing taffic signal phase and time



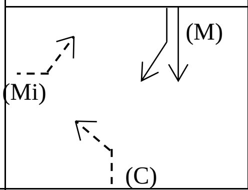
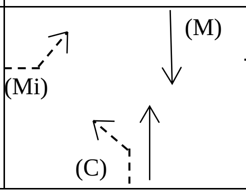
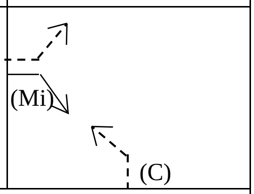
	1	2	3
Phase			
Cycle Time(s)	165		
Green Time (s)	55	70	25

Table 3-22 Maliban Junction -Manual control - Phase dirrection, time and cycle time

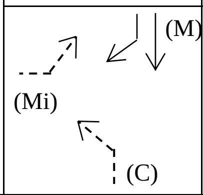
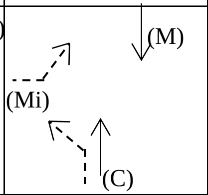
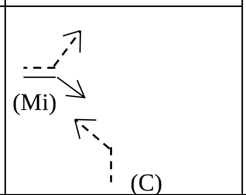
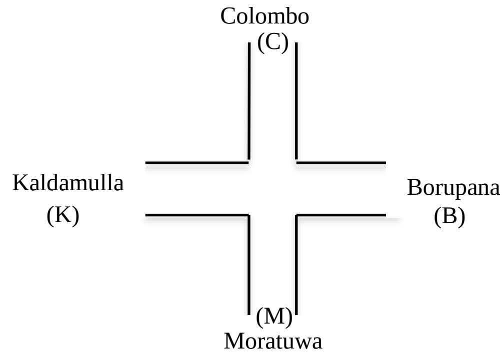
Cycle No	Phase			Cycle time (s)
	1	2	3	
				
1	77	66	32	175
2	50	107	18	175
3	59	109	20	188
4	60	150	20	230
5	60	80	20	160
6	53	63	20	136
7	70	86	23	179
8	57	101	20	178
9	62	149	25	236
10	66	120	18	204
11	78	120	20	218
12	47	95	23	165
13	65	108	20	193
14	58	115	20	193
15	61	110	17	188
Average Time	**62**	**105**	**21**	**188**

Table 3-23 Golumadama Junction – Existing taffic signal phase and time

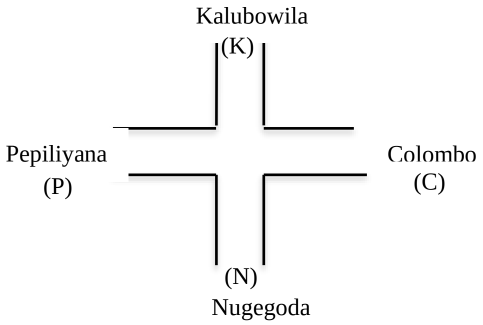


	1	2	3	4	5
Phase	(B) (M)	(C) (M)	(C) (K)	(K) (M)	(C) (B)
Cycle Time(s)	215				
Green Time (s)	35	86	24	24	38

Table 3-24 Golumadama Junction -Manual control - Phase dirrection, time and cycle time

Cycle No	Phase					Cycle time
	1	2	3	4	5	
	(B) (M)	(C) (M)	(C) (K)	(K) (M)	(C) (B)	
1	30	55	10	12	17	124
2	18	49	28	17	16	128
3	33	50	18	12	23	136
4	20	67	20	20	23	150
5	26	82	20	20	22	170
6	21	49	24	14	15	123
7	14	34	8	8	16	80
8	0	55	14	10	18	97
9	8	60	24	10	17	119
10	10	61	18	10	14	113
Average	**18**	**56**	**19**	**13**	**18**	**125**

Table 3-25 Kohuwala Junction – Existing taffic signal phase and time

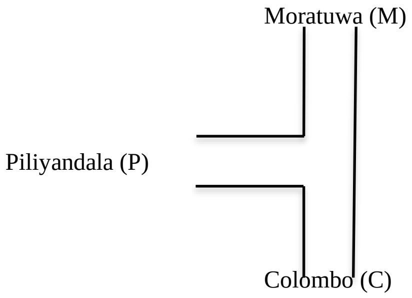


Phase	1	2	3	4	5	6
Direction	(K)	(K) (N)	(N)	(P)	(P) (C)	(C)
Cycle time(s)	200					
Green Time(s)	22	30	33	40	58	16

Table 3-26 Kohuwala Junction -Manual control - Phase dirrection, time and cycle time

Cycle No	Phase					Cycle time (s)
	1	2	3	4	5	
Direction	(K)	(K) (N)	(P)	(P) (C)	(P)	
1	28	74	30	118	10	**260**
2	30	81	21	122	21	**275**
4	60	85	35	95	20	**295**
5	20	65	25	90	15	**215**
7	20	95	45	110	15	**285**
8	20	80	15	95	20	**230**
10	22	60	25	80	10	**197**
11	40	80	40	70	10	**240**
13	35	85	25	85	10	**240**
Average Time	**31**	**78**	**29**	**96**	**15**	**249**

Table 3-27 Katubedda junction – Existing taffic signal phase and time



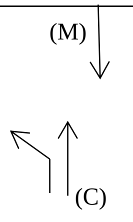
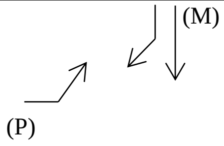
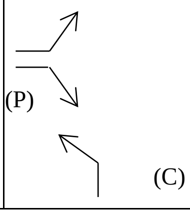
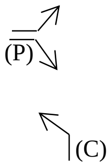
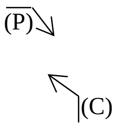
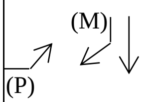
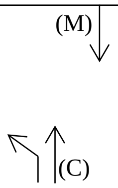
Phase	1	2	3
			
Cycle Time(s)	225		
Green Time (s)	130	30	65

Table 3-28 Katubedda junction -Manual control - Phase dirrection, time and cycle time

Cycle No	Phase				Cycle time (s)
	1	2	3	4	
					
1	43	21	35	109	208
2	42	20	43	153	258
3	40	21	40	162	263
4	47	20	31	206	304
5	53	21	20	151	245
6	45	20	35	148	248
7	40	25	34	120	219
8	54	19	54	125	252
9	55	20	27	125	227
10	60	22	40	120	242
11	70	24	42	135	271
12	39	20	36	130	225
13	50	22	40	137	249
14	40	22	50	146	258
Average Time	**46**	**20**	**35**	**138**	**247**

4. CALIBRATION OF VISSIM SOFTWARE

VISSIM is a microscopic time step and behaviour-based traffic simulation software which has been widely used assessing traffic conditions. It is especially useful to evaluate different traffic management scenarios to choose the best and optimization measures before implementation. The traffic analysing key parameters like delay, vehicle travel time, queue length and junction flow values can be easily generated using the VISSIM software.

To generate accurate results the model needs to be calibrated to represent the Sri Lankan conditions. Traffic in developing countries like Sri Lanka are heterogeneous in nature. Heterogeneous traffic is characterized by a wide mix of vehicles having diverse static and dynamic characteristics. In Sri Lanka, from small vehicles like motorbike, three wheelers to large vehicles like containers are moving in the same road.

“Driving behaviour” parameters are the key parameters to calibrate the model to Sri Lankan conditions. The VISSIM software itself has some default driving behavioural parameters to suit many conditions such as urban and rural areas. But none of them represents the Sri Lankan conditions because most of the Sri Lankan drivers have aggressive mind set of driving especially during peak hours. The lateral spacing and standstill distance they maintain are very low compare to a developed country. Most of the Sri Lankan drivers are not maintaining the lane behaviours during the peak hours and they overtake on both sides. So, a driving behaviour parameter template has been created in VISSIM software to match the driving behaviour of the Sri Lankan drivers.

In this study the calibration has been done in a trial and error process by changing the driving behavioural parameters (look ahead distance, look back distance, minimum headway, average standstill distance, additive part of safety distance, multiplicative part of safety distance, lateral distances) in the software model [6]. The propagated actual queue length in field (average of maximum queue length propagated per each signal cycle) is compared with the queue length observed in the field. This process is continued until these two set of values become nearly equal [28] (% error < 15% and RMSE < 15)

For the calibration the Kesbewa Junction has been selected. It is a four-leg intersection with proper lane markings and controlled by traffic signals during the peak hours. All four directions of traffic were considered during the calibration process. The traffic data at this intersection has been taken for 2 Hours (7:00 a.m. to 9:00 a.m.). The first one-hour data was used for the calibration of the junction and the second hour data used for the validation of the calibrated driving behavioural parameters.

The Kesbewa junction has been modelled in the VISSM for the 7:00 am to 8:00 am traffic data and calibrated. The driving behavioural parameters were adjusted during the calibration to match the average of maximum queue length per traffic signal cycle from the survey and output of VISSM model. The final queue length comparison for calibration is shown in Table 4-1. The root mean square error (RMSE) for the queue length comparison for calibration is 12.4 and it is inside the acceptable level (<15). To validate the calibrated driving behavioral parameters the model has been created for the 8:00 am to 9:00 am traffic flow and the queue lengths generated. The error between the surveyed queue lengths and model queue lengths were in an acceptable level and RMSE is 10.4 (<15). The final queue length comparison for validation is shown in Table 4-2. The calibrated Driving Behaviour parameters are Shown in Figure 4-1, Figure 4-2, Figure 4-3 and Figure 4-4.

Table 4-1 Calibration of Vissim Software parameters - Queue length comparison

Calibration			
Direction	Average of Maximum queue length per cycle (m)		% Error from Actual Value
	Actual	Model	
1	78	90	15.30%
2	123	138	12.55%
3	66	73	11.66%
4	103	117	13.45%

Table 4-2 Validation of the calibrated parameters - Queue length comparison

Validation			
Direction	Average of Maximum queue length per cycle (m)		% Error from Actual Value
	Actual	Model	
1	62	71	14.62%
2	69	81	17.32%
3	76	89	16.64%
4	55	61	9.36%

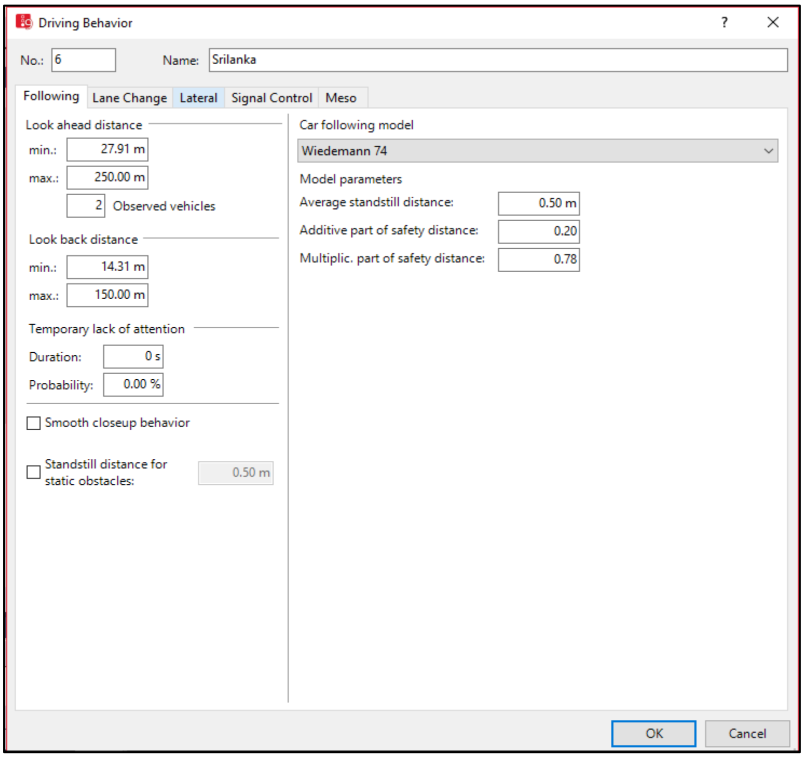


Figure 4-1 Driving behaviour parameters 1

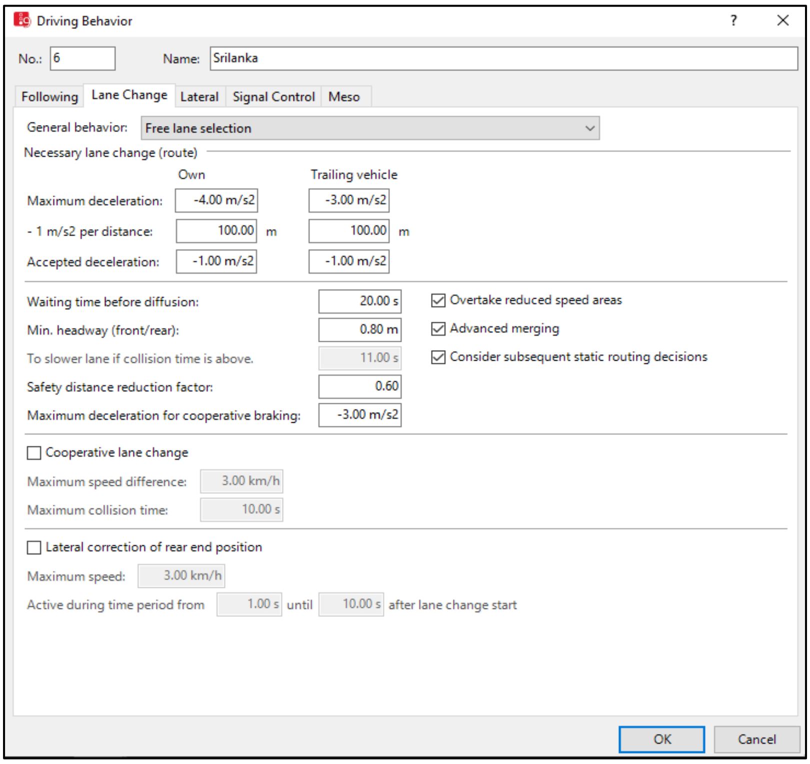


Figure 4-2 Driving behaviour parameters 2

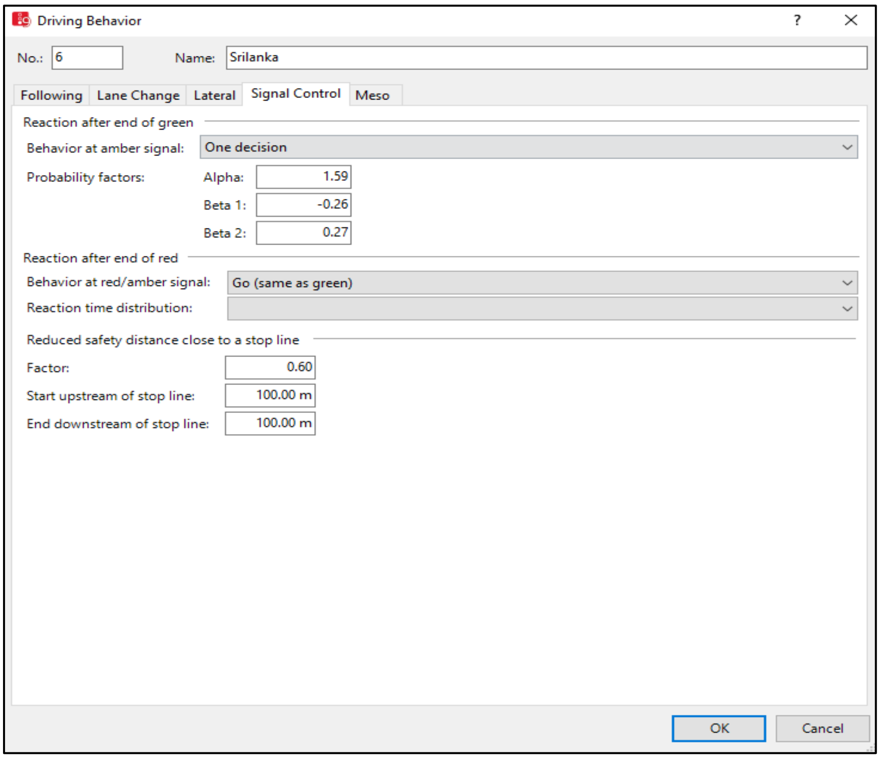


Figure 4-3 Driving behaviour parameters 3

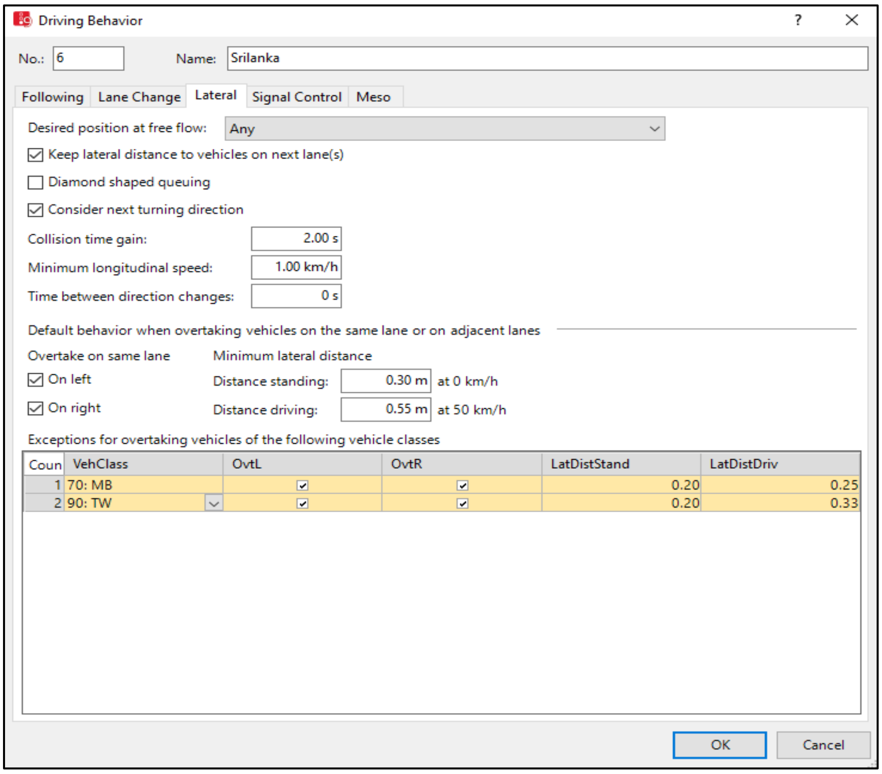


Figure 4-4 Driving behaviour parameters 4

While calibrating using the queue lengths the lateral distances and the standstill distances were the key to calibrate. The motorbikes and three wheelers are maintaining a lateral distance of around 0.2m when they wait in the queue. While driving also Motorbikes maintain a lateral clearance of 0.25m to overtake and 3 wheelers maintain 0.33m to overtake

These calibrated parameters can be applicable to the urban areas in Sri Lanka such as Colombo, Gampaha and Kandy city centres. In rural areas these parameters may not be valid. Table 4-3 and Table 4-4 shows the calibrated driving behavioural parameters for Sri Lankan conditions.

Table 4-3 Calibrated Driving behavioural Parameters 1

Average Standstill distance	0.5m
Additive part of safety distance	0.2m
Multiplicative part of safety distance	0.78m
Waiting time before diffusion	20s
Keeping lateral clearance to the vehicles on next lane	yes
Consider next turning direction	yes

Table 4-4 Calibrated Driving behavioural Parameters 2

Veh class	Overtaking Left	Overtaking Right	Lateral distance standing	Lateral distance Driving
Motor bike	allowed	allowed	0.2m	0.25m
Three wheelers	allowed	allowed	0.2m	0.33m
Other vehicles	Not allowed	allowed	0.3m	0.55m

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5. RESULTS

5.1. Overview

In Sri Lanka, especially in the Colombo city, during the peak hour's intersections are controlled manually or by traffic signals. For the analysis four junctions which were controlled manually by traffic police officers during the peak hours have been selected. Where two of them are T junctions and two are four-way junctions. The geometric arrangement of the junctions, lane widths, vehicle flow, turning movements, vehicle classification count, traffic signal and manual control time and phase details were taken through traffic surveys. For the analysis three different intersection control cases were considered for each junction. They are,

1. Existing posted traffic signal phase and time
2. Updated traffic signal phase and time
3. Traffic police control phase and time

To update the signal, cycle time has been designed using Webster's signal cycle time model and modified to achieve least delay for local conditions. The total delay at the junction, average delay per vehicle for each movements and maximum queue lengths were considered as the key performance measurements for each case.

Total delay (vehh) = $\sum_{i=1}^{N}[\,average\ delay\ per\ vehicle\ (s/veh) * No\ of\ vehicles\,]/3600$

Average delay (s/veh) = $\dfrac{\sum_{i=1}^{N}[\,average\ delay\ per\ vehicle\ (s/veh) * No\ of\ vehicles\,]}{\sum No\ of\ vehicles}$

Here, N=number of turning movements at the junction

For the analysis traffic micro simulation approach has been used. Traffic simulation software VISSIM has been used as the analysing tool. The software has been calibrated and validated to suit the Sri Lankan driver's behaviour before using it for the analysis. In the VISSIM model vehicle travel time measurements were defined to obtain the delays and travel times. To cover the maximum queue length (vehicle in queue delay) vehicle travel time measurements were defined in the model 300m upstream (greater than the maximum queue lengths) and 100m downstream for each movements of traffic. Queue counters were placed at each stop lines to get the maximum queue lengths per each signal cycle. In the data collection measurement definition, queue counter results interval was set as the corresponding cycle time for each case during the analysis.

5.2.VISSIM outputs for each junction

5.2.1. Katubedda Junction

At the Katubedda junction, during the evening peak hour (5:45 p.m. to 6:45 p.m.) the total junction flow was 5992veh/h. Existing traffic signal cycle time is 225s and the traffic police were maintaining an average cycle time of 257s. The traffic flows and turning movements at Katubedda junction during the evening peak hour are shown in Figure 5-1. The average saturation headway measured at the junction was 1.9s and the saturation flow rate is 1900 pcu/h. The total Critical lane flow at the junction is 1722 pcu/h. So, with the existing geometry of the junction it can be controlled by traffic signals. The updated signal cycle time is 165s.The movements for each phase, Critical lane flow and effective green time for the updated signal time are listed in Table 5-1.

The Figure5-2 shows the model of the Katubedda junction. The junction has been modelled using the calibrated driving behaviour parameters. The delay results obtained from Vissim software analysis for the three scenarios are given in Table5-3, Table 5-4 and Table 5-5. The queue results are listed in Table5-2.

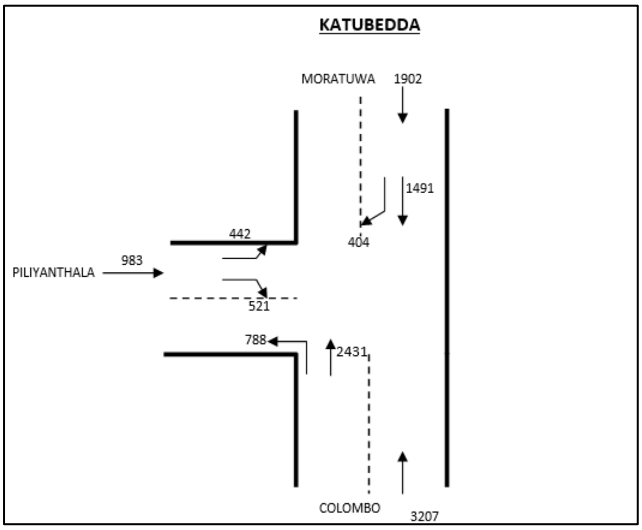


Figure 5-1 Turning movements at Katubedda junction during the peak hour

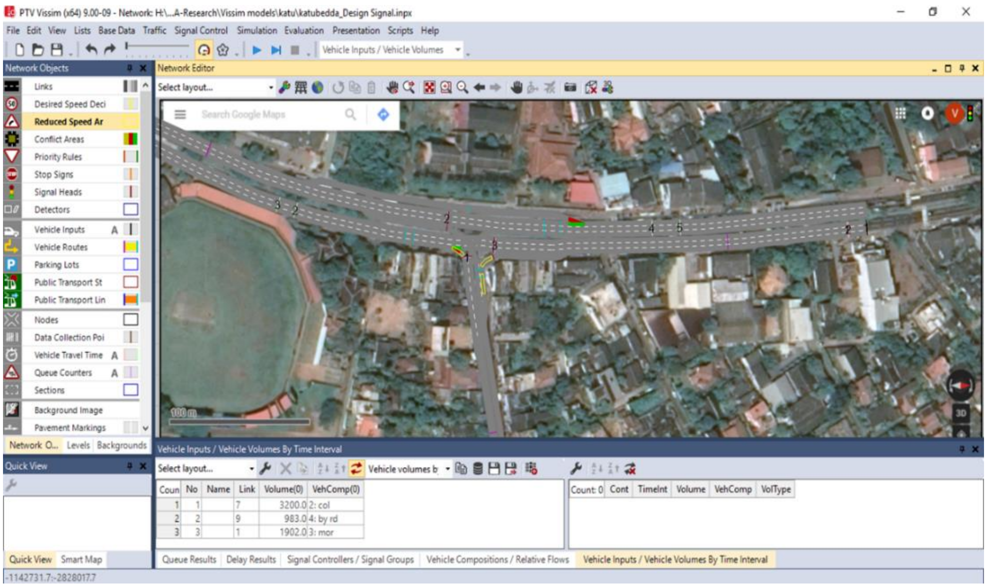


Figure 5-2 Katubedda Junction Vissim Model

Table 5-1 Katubedda Junction - Designed signal time for peak hour

Moratuwa (M) Piliyandala (P) Colombo (C)			
Phase	1	2	3
	(M) (C)	(P) (M)	(P) (C)
Critical lane flow (ULi)	806	411	505
Cycle Time(s)	165		
Effective Green Time (s)	78	39	49

The total delay during the peak hour when the junction was controlled by the existing signal time is 121.8vehh and the average delay for the vehicles passing the junction from all directions is 81.3s/veh. When the traffic signal was updated the total delay during the peak hour reduced to 83.8 vehh and the average delay for the vehicles passing the junction from all directions reduced to 57.7 s/veh. When the junction was controlled manually by traffic police, total delay during the peak hour was 109.1vehh and the average delay for the vehicles passing the junction from all directions is 74.1s/veh. The delays for existing traffic signals were higher than the manual control, but by updating the signal time the delays became less than the manual control. The maximum queue lengths also lesser when the junction is controlled by updated traffic signals.

Table 5-2 Katubedda Junction Delay results during peak hour- Existing traffic signal

Existing Traffic Signal			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Colombo to Piliyandala	96.4	684	18.3
2: Colombo to Moratuwa	99.3	2022	57.1
3: Piliyandala to Moratuwa	136.5	516	19.6
4: Piliyandala to Colombo	97.4	382	10.3
5: Moratuwa to Colombo	16.4	1444	6.6
6: Moratuwa to Piliyandala	102.6	346	9.9
Total junction delay during the peak hour (Vehh)			**121.8**
Average delay per vehicle (s/veh)			**81.3**

Table 5-3 Katubedda Junction Delay results during peak hour- Updated traffic signal

Updated Traffic signal			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Colombo to Piliyandala	67.2	615	11.5
2: Colombo to Moratuwa	89.3	1916	47.5
3: Piliyandala to Moratuwa	60.7	528	8.9
4: Piliyandala to Colombo	47	376	4.9
5: Moratuwa to Colombo	12.9	1441	5.2
6: Moratuwa to Piliyandala	59.9	349	5.8
Total junction delay during the peak hour (Vehh)			**83.8**
Average delay per vehicle (s/veh)			**57.7**

Table 5-4 Katubedda Junction Delay results during peak hour- Manual control

Manual control			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Colombo to Piliyandala	94.8	646	17.0
2: Colombo to Moratuwa	95.2	1975	52.2
3: Piliyandala to Moratuwa	76.2	530	11.2
4: Piliyandala to Colombo	126.1	384	13.4
5: Moratuwa to Colombo	15.2	1444	6.1
6: Moratuwa to Piliyandala	100.7	325	9.1
Total junction delay during the peak hour (Vehh)			**109.1**
Average delay per vehicle (s/veh)			**74.0**

Table 5-5 Katubedda junction Maximum Queue lengths during peak hour

Vehicles approaching from	Max Queue Length(m)		
	Existing traffic signal	Updated traffic signal	Manual control
Piliyandala	124	115	189
Moratuwa	53	57	74
Colombo	280	254	280

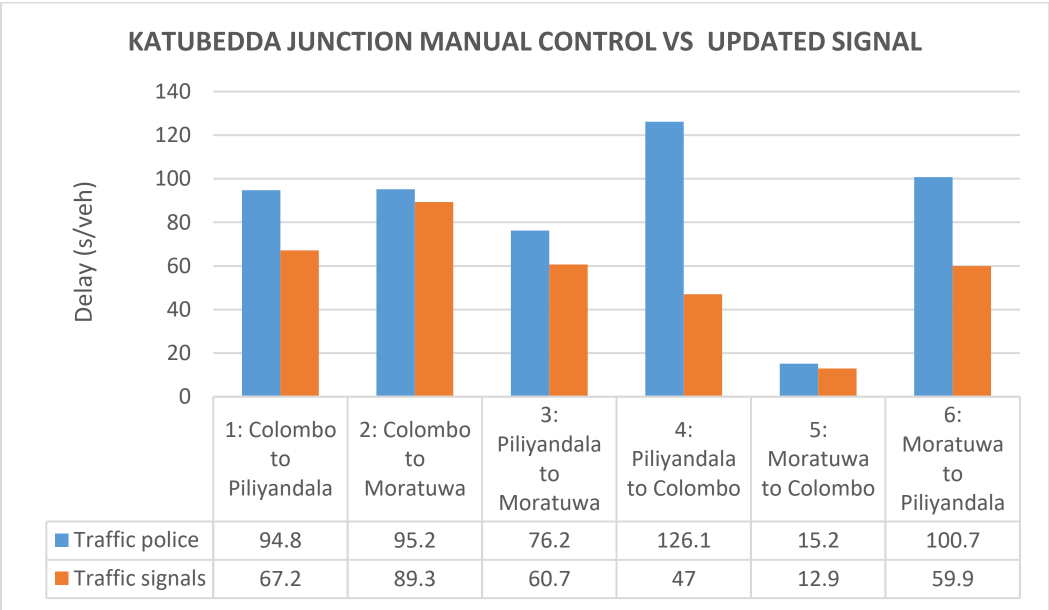


Figure 5-3 Comparison of delay - manual control vs updated signal time

To check the efficiency of manual control and traffic signals the two scenarios manual control and updated traffic signals were considered for the comparison. In this junction Colombo to Moratuwa and Moratuwa to Colombo are the two major movements in the Colombo-Galle road. When it comes to the major road movements Colombo to Moratuwa and Moratuwa to Colombo there is no significant difference in delay between both cases. Where for Colombo to Moratuwa delays are 95.16 s/veh for manual control and 89.3 s/veh for traffic signal control. In Moratuwa to Colombo movement delays are 15.23 s/veh for manual control and 12.9 s/veh for traffic signal control.

When it comes to minor road movement Piliyandala to Moratuwa, Piliyandala to Colombo and Moratuwa to Piliyandala the delay from manual control is significantly higher from traffic signal control. In Piliyandala to Moratuwa movement delays are 76.2 s/veh for manual control and 60.7 s/veh for traffic signal control, which is 20% lesser than the manual control. Likewise, for Piliyandala to Colombo the reduction is 63% and for Moratuwa to Piliyandala the reduction is 41%. The maximum queue length on the Piliyandala direction is 189m for manual control and 115m for updated traffic signals. This also shows that less preference has been given to minor roads.

5.2.2. Maliban Junction

At the maliban junction (T Junction) during the peak hour (5:30 p.m. to 6:30 p.m.) the total junction flow was 5987veh/h. Existing traffic signal cycle time is 170s and the traffic police officer was maintaining an average cycle time of 188s. The traffic flow and turning movements at Katubedda junction during the evening peak hour are shown in Figure 5-4. The average saturation headway measured at the junction is 1.9s and the saturation flow is 1900 pcu/h. The total Critical lane flow at the junction is 1522 pcu/h. So, with the existing geometry of the junction it can be controlled by traffic signals. The updated signal cycle time is 120s.The movements for each phase, Critical lane flow and effective green time for the updated signal are listed in Table 5-6.

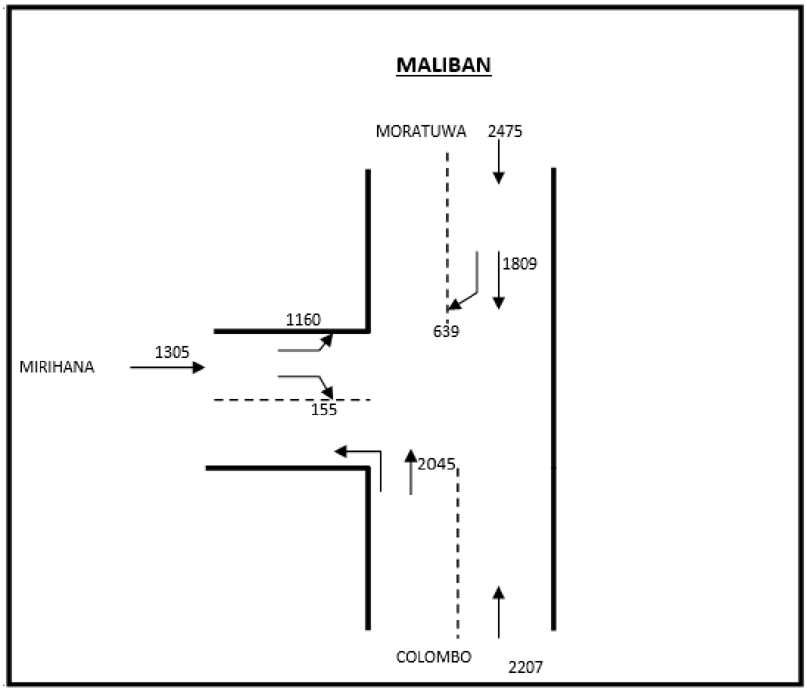
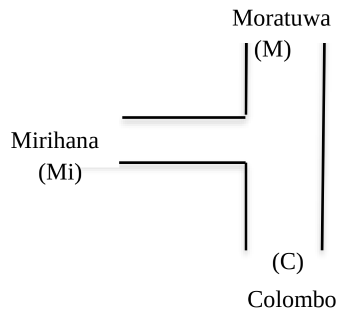
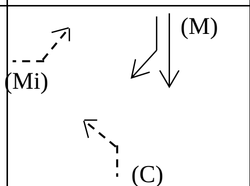
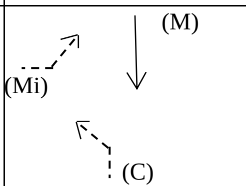
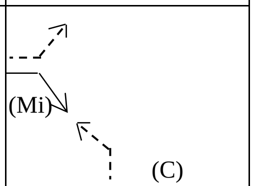


Figure 5-4 Traffic flow and turning movements at Maliban junction during the peak hour

Table 5-6 Maliban Junction - Designed signal time for peak hour



Phase	1	2	3
Direction			
Critical lane flow (Uli)	639	795	155
Cycle Time(s)	120		
Effective Green Time (s)	49	54	11

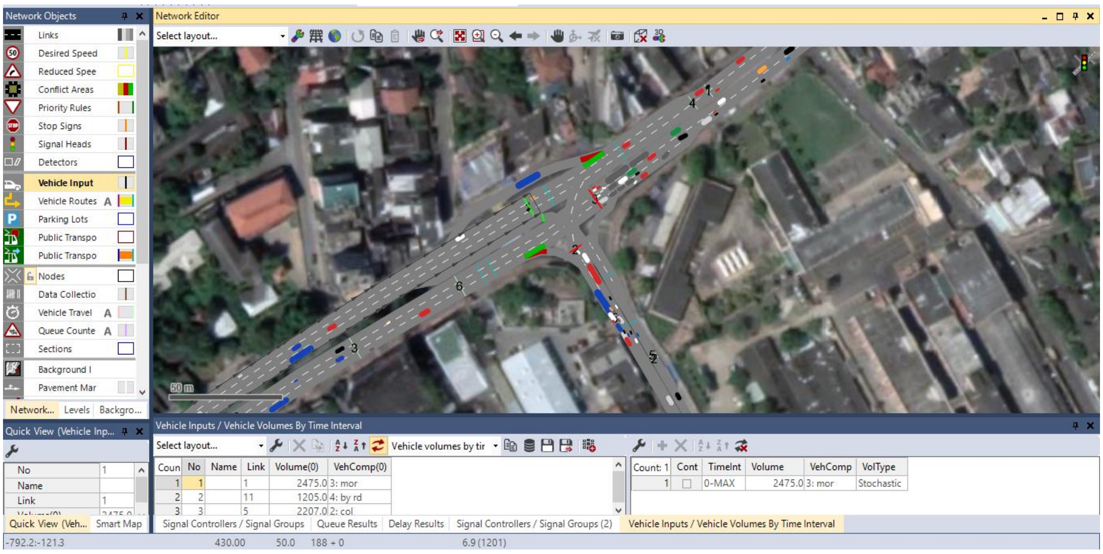


Figure 5-5 Maliban Junction VISSIM model

Figure 5-5 shows the VISSIM model of the Maliban Junction and the delay results obtained from maliban junction VISSIM software analysis for the three scenarios are given in Table5-8, Table 5-9 and Table 5-10.

Table 5-7 Maliban junction Maximum Queue lengths during peak hour

Vehicles Approaching from	Max Queue Length (m)		
	Existing Traffic Signal	Updated Traffic signal	Manual Control
Moratuwa	47	35	38
Mirihana	182	187	275
Colombo	213	208	170

Table 5-8 Maliban Junction Delay results during peak hour- Existing traffic signal

Existing Traffic Signal			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Moratuwa to Colombo	7.5	1737	3.6
2: Moratuwa to Mirihana	45.6	564	7.1
3: Mirihana to Moratuwa	147.8	967	39.7
4: Mirihana to Colombo	189.0	121	6.4
5: Colombo to Mirihana	28.3	72	0.6
6: Colombo to Moratuwa	43.7	2002	24.3
Total delay during the peak hour (Vehh)			**81.7**
Average delay per vehicle (s/veh)			**53.8**

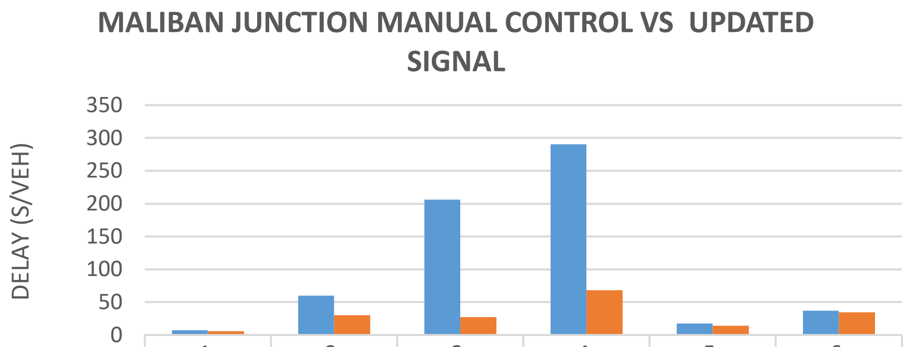
Table 5-9 Maliban Junction Delay results during peak hour- Updated traffic signal

Updated Traffic signal			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Moratuwa to Colombo	5.8	1743	2.8
2: Moratuwa to Mirihana	30	540	4.5
3: Mirihana to Moratuwa	27.2	862	6.5
4: Mirihana to Colombo	68.3	95	1.8
5: Colombo to Mirihana	14.1	73	0.3
6: Colombo to Moratuwa	34.5	2035	19.5
Total delay during the peak hour (Vehh)			35.4
Average delay per vehicle (s/veh)			23.8

Table 5-10 Maliban Junction Delay results during peak hour- Manual control

Manual control			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Moratuwa to Colombo	7.2	1741	3.5
2: Moratuwa to Mirihana	59.7	564	9.4
3: Mirihana to Moratuwa	206.3	887	50.8
4: Mirihana to Colombo	290.3	105	8.5
5: Colombo to Mirihana	17.7	71	0.4
6: Colombo to Moratuwa	36.9	1991	20.4
Total delay during the peak hour (Vehh)			**92.9**
Average delay per vehicle (s/veh)			**62.4**

The total delay during the peak hour when the junction was controlled by the existing signal time was 81.7 vehh and the average delay for the vehicles passing the junction from all directions was 53.8 s/veh. When the traffic signal was updated the total delay during the peak hour reduced to 35.4 vehh and the average delay for the vehicles passing the junction from all directions reduced to 23.8 s/veh. When the junction is controlled manually by traffic police, total delay during the peak hour was 92.9 vehh and the average delay for the vehicles passing the junction from all directions was 62.4 s/veh. The delays for manual control were higher than the existing signal time, and the delays were further reduced by updating the traffic signals. The maximum queue lengths also lesser when the junction is controlled by proper traffic signals.



	1: Moratuwa to Colombo	2: Moratuwa to Mirihana	3: Mirihana to Moratuwa	4: Mirihana to Colombo	5: Colombo to Mirihana	6: Colombo to Moratuwa
Traffic police	7.2	59.7	206.3	290.3	17.7	36.9
Traffic signals	5.8	30	27.2	68.3	14.1	34.5

Figure 5-6 Comparison of delay -manual control vs updated signal time

To check the efficiency of manual control and traffic signals for the junction, the two scenarios manual control and updated traffic signals were considered. In the Maliban junction Colombo to Moratuwa and Moratuwa to Colombo are the two major movements in the Colombo-Galle road. When it comes to the major road movements Colombo to Moratuwa and Moratuwa to Colombo there is no significant difference in delay between both cases. Where for Colombo to Moratuwa delays are 36.9 s/veh for manual control and 34.5 s/veh for traffic signal control. In Moratuwa to Colombo movement delays are 7.2 s/veh for manual control and 5.8 s/veh for traffic signal control.

But when it comes to minor road movement Mirihana to Moratuwa, Mirihana to Colombo and Moratuwa to Mirihana the delay from manual control is significantly higher from traffic signal control. In Mirihana to Moratuwa movement delays are 206.3 s/veh for manual control and 27.2 s/veh for traffic signal control, which is 80% lesser from the manual control. Likewise, for Mirihana to Colombo the reduction is 76% and for Moratuwa to Mirihana the reduction is 50%. The maximum queue length on the Mirihana direction is 275m for manual control and 187m for updated traffic signals. This also shows that less preference has been given to minor roads.

5.2.3. Golumadama Junction

Golumadama junction is a four-way junction located in-between Maliban junction and Katubedda junction along the Galle-Colombo road. During the peak hour (5:30 p.m. to 6:30 p.m.) the total junction flow was 6429veh/h. Existing traffic signal cycle time is 215s and the traffic police was maintaining an average cycle time of 125s. The traffic flow and turning movements at Golumadama junction during the peak hour is shown in Figure 5-5. The total Critical lane flow at the junction is 1462 pcu/h. which is less than the saturation flow rate. So, with the existing geometry of the junction it can be controlled by traffic signals. The updated signal cycle time is 120s.The designed phase, Critical lane flow and effective green time are shown in Table 5-11.

Figure 5-6 shows the Vissim model of the Golumadama junction. The delay results during the peak hour for Existing posted signal time, updated signal time and manual control are given in Table 5-12 and Table 5-13 and Table 5-14. The maximum queue lengths for each case are listed in Table 5-15.

The total delay during the peak hour when the junction was controlled by the existing signal time was 82.2 vehh and the average delay for the vehicles passing the junction from all directions was 49.6 s/veh. When the traffic signal was updated the total delay during the peak hour reduced to 36.7 vehh and the average delay for the vehicles passing the junction from all directions reduced to 22.4 s/veh. When the junction was controlled manually by traffic police, total delay during the peak hour was 64 vehh and the average delay for the vehicles passing the junction from all directions was 38.6 s/veh. The delays for existing traffic signals were higher than the manual control, but by updating the signal time the delays became less than the manual control.

At the Golumadama junction the maximum queue lengths for each direction were lesser for manual control than the existing traffic signal. For updated signals maximum queue lengths for each direction were even lesser than the manual control.

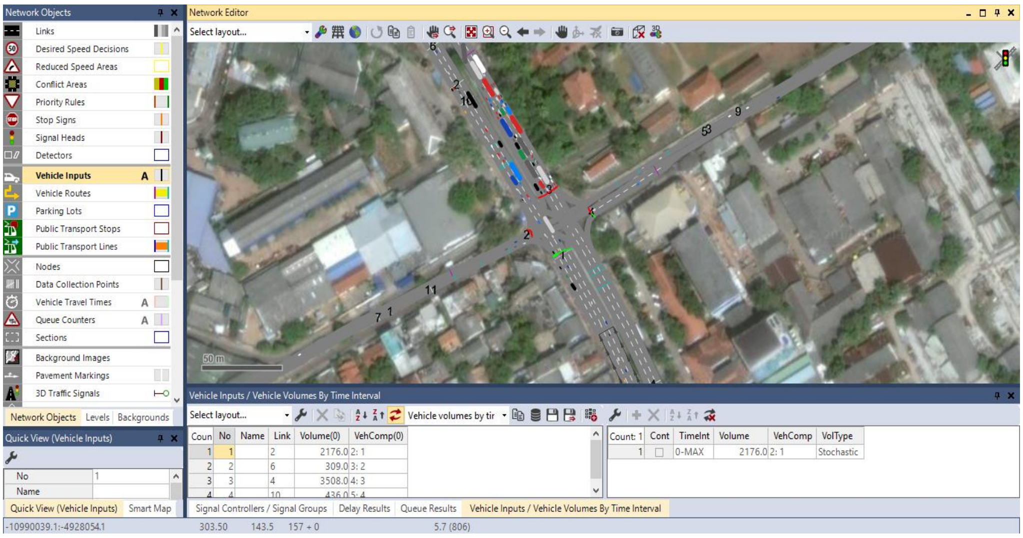


Figure 5-7 Golumadama Junction – VISSIM model

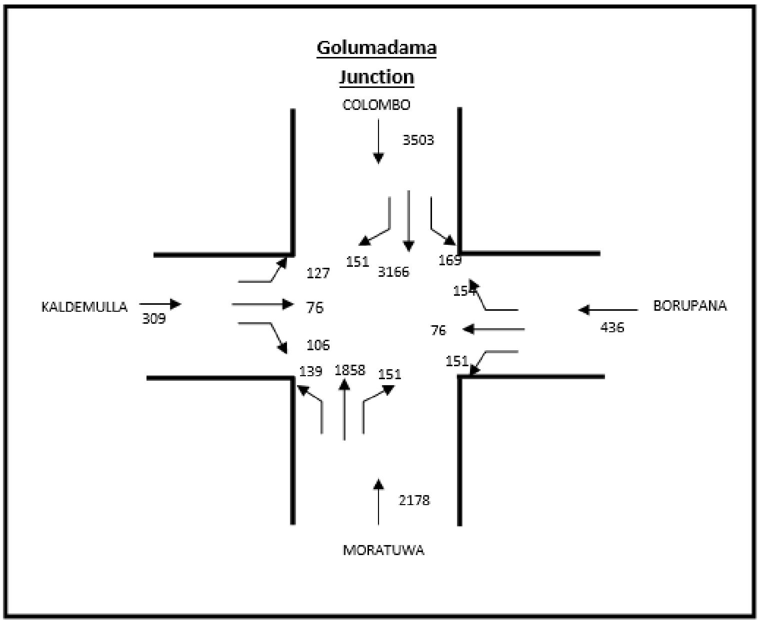
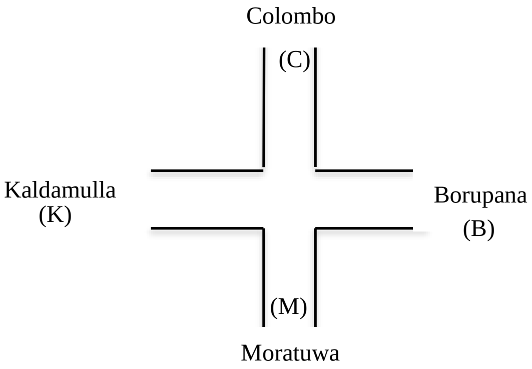


Figure 5-8 Turning movements at Golumadama junction during the peak hour

Table 5-11 Golumadama Junction - Designed signal time for peak hour



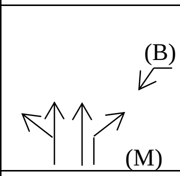
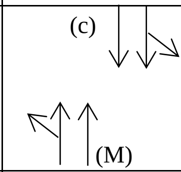
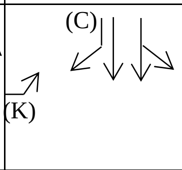
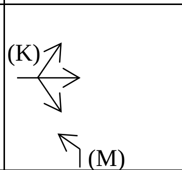
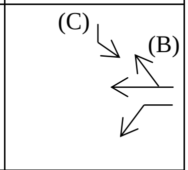
Phase	1	2	3	4	5
Direction					
Critical lane flow (ULi)	151	900	151	106	154
Cycle Time(s)	120				
Effective Green Time	12	72	12	10	12

Table 5-12 Golumadama Junction Delay results during peak hour- Existing traffic signal

Existing Traffic Signal			
Direction	vehicle delay- s/veh	No of veh	Tot Veh delay- vehh
1: Moratuwa to kaldemulla	30.7	125	1.1
2: Moratuwa to Colombo	33.9	1756	16.5
3: Moratuwa to Borupana	95.3	155	4.1
4: Kaldemulla to Moratuwa	81.2	88	2.0
5: Kaldemulla to Borupana	101.3	71	2.0
6: Kaldemulla to Colombo	69.9	120	2.3
7: Colombo to Kaldemulla	94.8	144	3.8
8: Colombo to Moratuwa	48.9	2948	40.0
9: Colombo to Borupana	49.8	153	2.1
10: Borupana to Colombo	82.3	133	3.0
11: Borupana to Kaldemulla	92.4	67	1.7
12: Borupana to Moratuwa	62.0	203	3.5
Total delay during the peak hour (Vehh)			**82.2**
Average delay per vehicle (s/veh)			**49.6**

Table 5-13 Golumadama Junction Delay results during peak hour- Updated traffic signal

Updated Traffic signal			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Moratuwa to kaldemulla	9.0	123	0.3
2: Moratuwa to Colombo	10.2	1725	4.9
3: Moratuwa to Borupana	65.0	153	2.8
4: Kaldemulla to Moratuwa	55.5	91	1.4
5: Kaldemulla to Borupana	72.1	76	1.5
6: Kaldemulla to Colombo	58.5	127	2.1
7: Colombo to Kaldemulla	58.2	141	2.3
8: Colombo to Moratuwa	17.7	2932	14.4
9: Colombo to Borupana	15.4	151	0.6
10: Borupana to Colombo	75.0	126	2.6
11: Borupana to Kaldemulla	63.7	64	1.1
12: Borupana to Moratuwa	48.9	195	2.7
Total delay during the peak hour (Vehh)			**36.7**
Average delay per vehicle (s/veh)			**22.4**

Table 5-14 Golumadama Junction Delay results during peak hour- Manual control

Manual Control			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Moratuwa to kaldemulla	20.4	125	0.7
2: Moratuwa to Colombo	22.6	1752	11
3: Moratuwa to Borupana	60.6	154	2.6
4: Kaldemulla to Moratuwa	60.1	90	1.5
5: Kaldemulla to Borupana	60.2	71	1.2
6: Kaldemulla to Colombo	53	122	1.8
7: Colombo to Kaldemulla	73.3	139	2.8
8: Colombo to Moratuwa	42.9	2964	35.3
9: Colombo to Borupana	39.9	156	1.7
10: Borupana to Colombo	55.2	129	2
11: Borupana to Kaldemulla	64.6	67	1.2
12: Borupana to Moratuwa	39.6	201	2.2
Total delay during the peak hour (Vehh)			**64.0**
Average delay per vehicle (s/veh)			**38.6**

Table 5-15 Golumadama junction Maximum Queue lengths during peak hour

Vehicles Approaching from	Max Queue Length(m)		
	Existing Traffic Signal	Updated Traffic signal	Manual Control
Moratuwa	107	65	69
Kaldemulla	61	42	44
Colombo	234	137	156
Borupana	111	67	88

5.2.4. Kohuwala Junction

It is a 4-way Junction and one of the heavily congested junctions in Colombo city during the morning and evening peak hours. The maximum evening peak junction flow recorded in day1 of the data collection and it has been used for the analysis. During the evening peak hour (5:15 p.m. to 6:15 p.m.) the junction flow was 5839veh/h. Existing traffic signal cycle time is 200s and the traffic police officer was maintaining an average cycle time of 249s. The traffic flow and turning movements at Kohuwala junction during the peak hour are shown in Figure 5-10. The total Critical lane flow at the junction is 2209 pcu/h which is higher than the saturation flow is 1900 pcu/h/lane. The junction is over saturated. So, with the existing geometry of the junction we cannot accommodate traffic signals there as we cannot further reduce the number of phases. Critical lane flows for each phase are listed in Table 5-16.

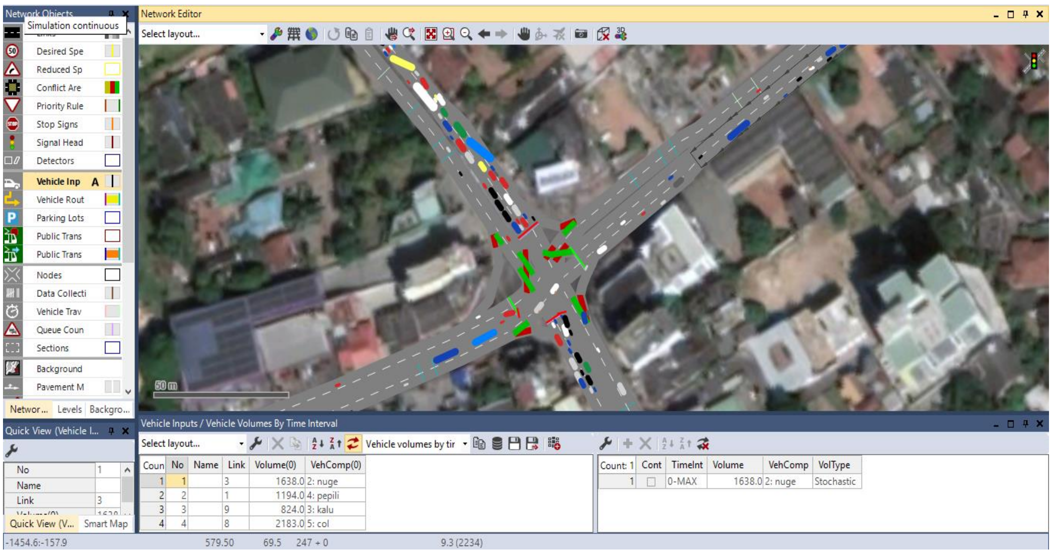
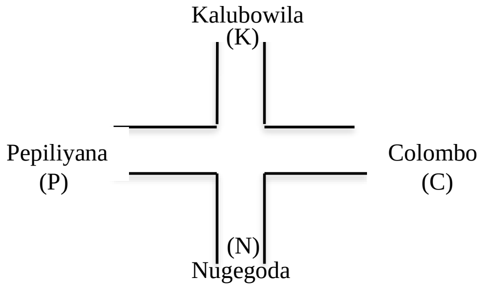


Figure 5-9 Kohuwala Junction Vissim Model

Table 5-16 Kohuwala Junction Critical flows



Phase	1	2	3	4	5	6
Phase	(K)	(K) (N)	(N)	(P)	(P) (C)	(C)
Critical lane flow (Uli)	147	489	391	337	800	45

The Figure5-9 shows the Vissim model of the Kohuwala junction. The junction has been modelled using the calibrated driving behaviour parameters. The delay results obtained from VISSIM software analysis for existing postal signal time and manual control are given in Table5-17 and Table 5-18.

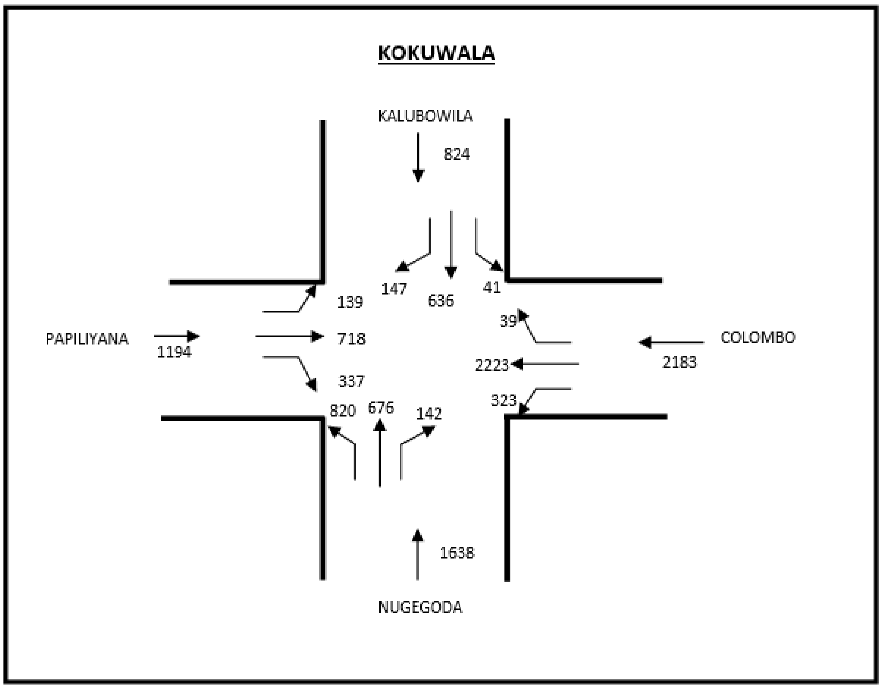


Figure 5-10 Traffic flow and turning movements at Kohuwala Junction

The total delay during the peak hour when the junction was controlled by the existing signal time was 150.5 vehh and the average delay for the vehicles passing the junction from all directions was 108.9 s/veh. When the junction was controlled manually by traffic police, total delay during the peak hour was 147.8 vehh and the average delay for the vehicles passing the junction from all directions was 104.7 s/veh. The delays for existing traffic signals were higher than the manual control. The traffic police officers control the intersection effectively than the existing traffic signal. Traffic police officers were allowing the merging traffic movements to reduce the critical lane flows. But with the aggressive nature of driving of Sri Lankan drivers it is not safe to allow merging a relatively high left turns to through movements. The phase arrangement used by the traffic police officers were also complicated and not consistent.

Table 5-17 Golumadama junction Maximum Queue lengths during peak hour

Vehicles Approaching from	Max Queue Length(m)	
	Existing Traffic Signal	Manual Control
Pepiliyana	199	175
Nugegoda	103	97
Kalubowila	92	82
Colombo	273	235

Table 5-18 Kohuwala Junction Delay results during peak hour- Existing traffic signal

Existing Signal			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Nugegoda to Colombo	72.4	118	2.4
2: Nugegoda to Kalubowila	52.7	610	8.9
3: Nugegoda to pepiliyana	66.6	711	13.1
4: Pepiliyana to Nugegoda	75.2	290	6.1
5: Pepiliyana to Colombo	39.0	665	7.2
6: Pepiliyana to Kalubowila	39.7	126	1.4
7: Kalubowila to Pepiliyana	102.7	135	3.9
8: Kalubowila to Nugegoda	61.8	596	10.2
9: Kalubowila to Colombo	60.6	38	0.6
10: Colombo to Kalubowila	236.7	31	2.0
11: Colombo to pepiliyana	205.9	1455	83.2
12: Colombo to Nugegoda	204.3	202	11.5
Total delay during the peak hour (Vehh)			**150.5**
Average delay per vehicle (s/veh)			**108.9**

Table 5-19 Kohuwala Junction Delay results during peak hour- Manual control

Manual Control			
Direction	vehicle delay-s/veh	No of veh	Tot Veh delay-vehh
1: Nugegoda to Colombo	70.0	128	2.5
2: Nugegoda to Kalubowila	75.9	634	13.4
3: Nugegoda to pepiliyana	66.4	736	13.6
4: Pepiliyana to Nugegoda	72.9	287	5.8
5: Pepiliyana to Colombo	56.6	653	10.3
6: Pepiliyana to Kalubowila	36.8	126	1.3
7: Kalubowila to Pepiliyana	121.7	133	4.5
8: Kalubowila to Nugegoda	62.0	591	10.2
9: Kalubowila to Colombo	33.4	37	0.3
10: Colombo to Kalubowila	190.7	31	1.6
11: Colombo to pepiliyana	179.8	1507	75.2
12: Colombo to Nugegoda	149.4	220	9.1
Total delay during the peak hour (Vehh)			**147.8**
Average delay per vehicle (s/veh)			**104.7**

5.3. Effect of daily variation of traffic

Daily variation of traffic flow makes significant effect on delays at traffic signals. At Katubedda junction and maliban junction analysis has been done for different percentage of peak hour junction flow with the updated signal time for the identified peak hour flow.

Table 5-17 Effect of daily variation of junction flow

Variation from Peak hour junction flow	**Katubedda junction**			**Maliban Junction**		
	Junction Flow (veh/h)	Total delay (vehh)	% Tot delay differs from 0% flow	Junction Flow (veh/h)	Total delay (vehh)	% Tot delay differs from 0% flow
-10%	5379	68.5	-27.8%	5387	28.7	-21.8%
-5%	5678	80.9	-14.7%	5687	32.4	-11.8%
0%	5977	94.7	0	5986	36.7	0
2.5%	6126	110.7	16.9%	6136	44.4	31.9%
5%	6276	126.6	34.8%	6285	51.9	41.6%

5.4. Optimum Signal cycle time

To update the signal time, initially signal cycle time has been calculated using Webster & Cobbe (1966) model [4].

$$C = \frac{1.5L + 5}{1 - \frac{\sum_{i=1}^{n} q_i}{S}}$$

But when it comes to Sri Lankan Driver's behaviour the cycle time from that model may not be an optimum solution. In order to find out the optimum delay, the junctions were modelled for different percentages of the cycle time from the Webster & Cobbe (1966) model for the three Junctions (Katubedda, Maliban and Golumadama)

$C^{/}$ **= A × C**

Where

$C^{/}$ **-** Modified cycle time

A - Modification Factor (0.75, 0.8, 0.85, 0.9, 0.95, 0.98, 1.02, 1.05, 1.1, 1.15, 1.2)

Table 5-20 Modified Signal Cycle times

Junction	Katubedda	Maliban	Golumadama
Designed Cycle time (C)	191s	148s	125s
$C^{/}$			
0.75C	143s	111s	94s
0.8C	153s	118s	100s
0.85C	163s	126s	106s
0.9C	172s	133s	113s
0.95C	182s	141s	119s
0.98C	187s	145s	123s
C	191s	148s	125s
1.02C	195s	151s	128s
1.05C	201s	155s	131s
1.1C	210s	163s	138s
1.15C	220s	170s	144s
1.2C	230s	178s	150s

The results for the analysis of each junction for the cycle time $C^{/}$ are given below in Table-5-19, Table-5-20 and Table-5-21.

Table 5-19 Delay results at maliban junction for different signal cycle times

Maliban Junction - C = 148s												
Delay (s/veh)												
***C**	**0.7**	**0.75**	**0.80**	**0.85**	**0.90**	**0.95**	**0.98**	**1.00**	**1.02**	**1.05**	**1.10**	**1.15**
1: m-c	5.1	5.3	5.8	5.6	5.7	5.6	5.2	5.6	5.2	5.2	5.6	5.6
2: m-mi	23.8	25.5	30.0	28.0	31.1	36.4	32.9	35.5	33.4	37.1	33.4	37.1
3: mi-m	46.6	41.2	27.2	29.9	50.9	58.4	68.3	111.7	118.9	151.1	156.9	165.4
4: mi-c	81.0	76.7	68.3	68.8	105.6	99.6	223.5	181.1	133.8	203.2	223.8	218.3
5: c-mi	24.0	22.1	24.1	28.5	21.4	24.0	33.4	29.6	36.8	32.4	30.7	39.7
6: c-m	37.9	34.7	34.5	40.9	36.1	38.5	40.8	42.0	49.9	42.3	46.1	50.1
Tot delay Vehh	**43.0**	**40.0**	**36.3**	**40.2**	**44.5**	**49.0**	**55.7**	**63.9**	**69.6**	**74.1**	**80.1**	**83.0**

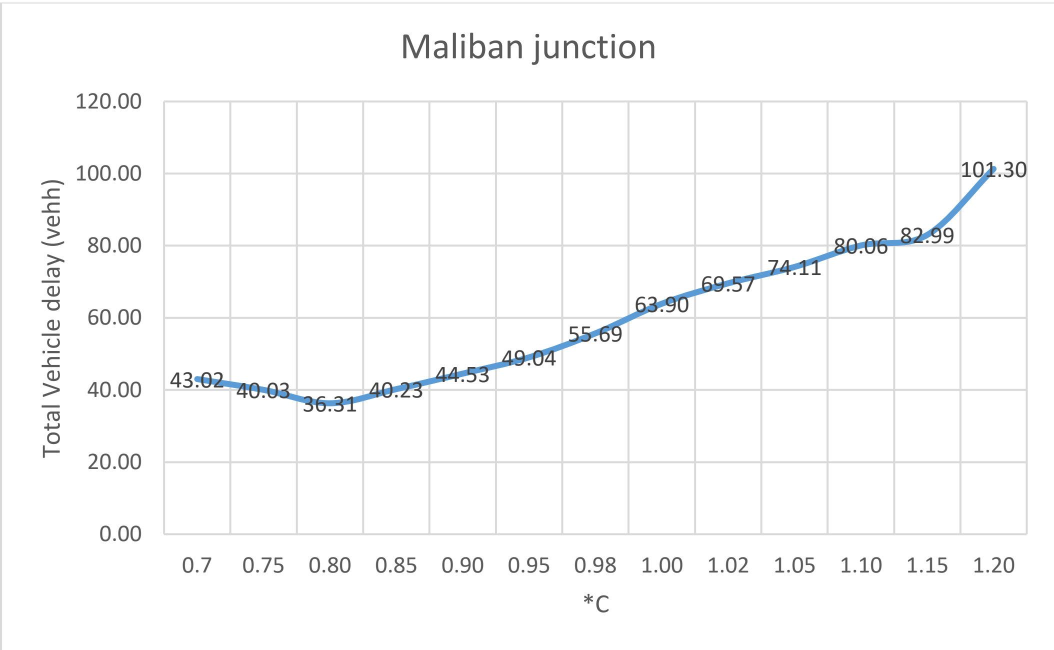


Figure 5-11 Variation - Total delay Vs Cycle time – Maliban Junction

At Maliban junction least total delay of 36.31vehh recorded when C $^{/}$ = 0.8C (120s).

Table 5-210 Delay results at Katubedda junction for different signal cycle times

Katubedda Junction , C = 191s												
Delay (s/veh)												
***C**	**0.75**	**0.80**	**0.85**	**0.90**	**0.95**	**0.98**	**1.00**	**1.02**	**1.05**	**1.10**	**1.15**	**1.20**
1: c-p	94.2	85.0	67.2	77.2	114.9	125.3	128.7	122.8	132.2	108.6	120.7	142.1
2: c-m	98.2	93.3	79.3	89.2	123.4	134.7	134.3	132.3	138.7	120.8	138.0	155.6
3: p-m	51.7	57.9	60.7	62.9	62.1	56.7	81.4	84.0	74.1	75.0	97.3	95.5
4: p-c	40.6	48.1	47.0	51.6	59.2	48.8	60.5	58.8	64.6	61.8	66.2	76.9
5: m-c	10.0	11.4	12.9	11.8	13.3	12.0	11.8	13.8	13.2	14.0	13.6	16.8
6: m-p	49.6	55.5	59.9	60.2	62.8	56.8	65.2	66.1	68.8	70.8	78.2	72.6
Tot delay (Vehh)	**94.7**	**92.9**	**83.8**	**88.2**	**112.0**	**116.8**	**123.7**	**125.8**	**130.2**	**123.8**	**132.4**	**147.6**

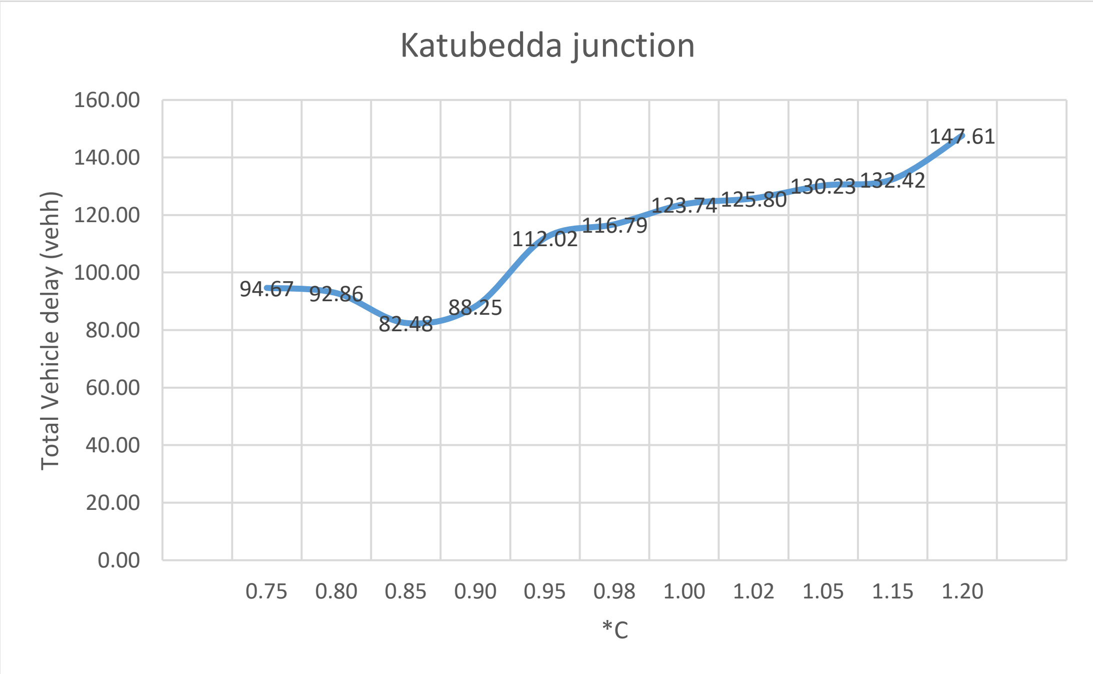


Figure 5-12 Variation - Total delay Vs Cycle time- Katubedda Junction

At Katubedda junction least total delay of 82.48vehh recorded when $C^{/} = 0.85C$ (165s).

Table 5-21 Delay results at Golumadama junction for different signal cycle times

Golumadama Junction , C = 125s												
	Delay (s/veh)											
***C**	**0.70**	**0.75**	**0.80**	**0.85**	**0.90**	**0.95**	**0.98**	**1.00**	**1.02**	**1.05**	**1.10**	**1.15**
1: m-k	7.6	6.2	9.4	8.1	8.7	7.4	10.0	9.0	9.0	10.9	11.9	10.5
2: m-c	9.0	8.5	8.7	8.8	9.6	10.0	10.6	10.2	10.5	10.2	9.9	11.0
3: m-b	46.8	44.9	46.2	55.8	51.4	59.0	70.1	65.0	57.0	71.3	64.8	66.0
4: k-m	46.0	45.0	47.7	52.0	51.6	64.2	61.4	55.5	59.2	56.1	65.3	71.1
5: k-b	46.3	49.3	56.9	56.9	50.6	59.2	68.2	72.1	66.9	57.1	68.4	65.1
6: k-c	43.2	44.7	46.0	51.5	48.3	53.9	64.3	58.5	59.4	52.8	61.1	59.3
7: c-k	50.4	48.2	51.0	52.1	58.0	59.0	63.1	58.2	60.8	59.7	76.3	70.9
8: c-m	18.9	15.3	13.8	15.9	15.6	18.7	16.7	17.7	17.4	17.4	18.4	19.3
9: c-b	16.7	12.6	12.7	15.2	15.7	16.9	16.1	15.4	15.6	15.0	14.5	16.2
10: b-c	72.4	93.5	62.5	69.2	95.7	69.1	86.1	75.0	78.3	93.9	82.8	84.2
11: b-k	64.7	92.0	53.6	72.8	101.6	66.4	84.7	63.7	79.5	89.8	85.6	73.2
12: b-m	43.3	58.2	36.5	42.9	58.5	47.2	55.4	48.9	59.8	56.2	57.0	58.1
Tot delay (Vehh)	**34.2**	**32.6**	**29.3**	**33.1**	**35.1**	**36.8**	**38.2**	**36.7**	**37.5**	**37.7**	**39.2**	**40.2**

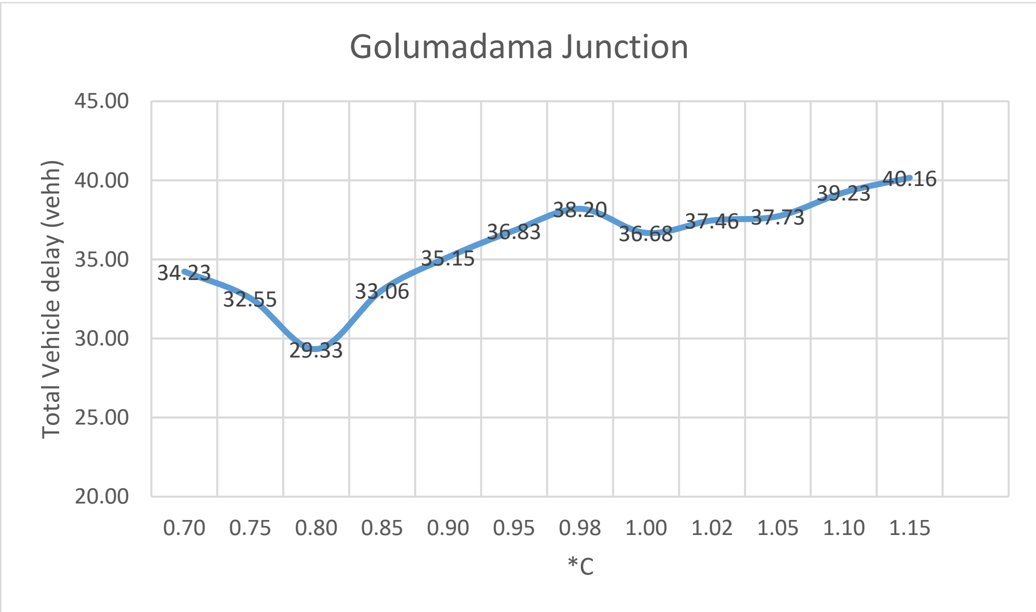


Figure 5-13 Variation - Total delay Vs Cycle time- Kolumadama Junction

At Golumadama junction least total delay of 29.33vehh recorded when C $^{/}$ = 0.8C (100s).

6. CONCLUSIONS

To meet the research objectives five junctions were selected. Kesbewa junctions was used to calibrate the VISSIM model and other four has been used for the analysis of manual control and traffic signal control. The analysis has been done by micro simulation approach using VISSIM software. Table 6-1 shows the summary of total delay during the peak hour and Table 6-2 shows the summary of average delay per vehicle for the three cases.

Table 6-1 Total delay during the peak hour for each junctions

Junction	Total delay during the peak hour (vehh)		
	Existing Traffic Signal	Updated Traffic signal	Manual Control
Katubedda	121.8	83.8	109.1
Maliban	81.7	35.4	92.9
Golumadama	82.2	36.7	64
Kohuwala	150.6	-	147.8

Table 6-2 Average delay per vehicle for each junctions

Junction	Average delay per vehicle (s/veh)		
	Existing Traffic Signal	Updated Traffic signal	Manual Control
Katubedda	81.3	57.7	74
Maliban	53.8	42.2	62.4
Golumadama	49.6	22.4	38.6
Kohuwala	108.9	-	104.7

At the three junctions Katubedda, Maliban and Golumadama the total critical lane flows were less than the saturation flow rate. But in the Kohuwala junction total critical lane flows were higher than the saturation flow rate. At the Katubedda, maliban and Golumadama junction traffic signals were updated and they result lesser total junction delay and lesser average delay per vehicle than the current posted signal time.

At the maliban junction total delays during the peak hour for existing posted signal (81.69vehh) is lesser than of the manual control by traffic police officers (92.87vehh).

But at the other three junctions Katubedda, Golumadama and Kohuwala total delays during the peak hour for existing posted signal (121.79vehh, 82.22vehh,150.56vehh) is higher than of the manual control by traffic police officers (109.9vehh, 64.03vehh, 147.84vehh). It shows that there was an inefficiency in the existing traffic signals and traffic police officers present there. Traffic police officers were able to reduce 10.43% (12.7vehh) of the total delay at Katubedda junction, 22.12% (18.19vehh) of the total delay at Golumadama junction and 1.8% (2.72vehh) of the total delay at the Kohuwala junction. But at same time at maliban junction their control increased the total delay by 13.69% (11.18vehh) than the current posted signal design.

When the signal time was updated for each junction, update signal times gave lesser total delay during the peak hours (83.8vehh, 35.4vehh, 36.7vehh) than of the manual control by traffic police officers (109.1vehh, 92.9vehh, 64.0vehh) for Katubedda, maliban and Golumadama junctions. Updated traffic signals were able to reduce 23% (25.3vehh) of the total delay at Katubedda junction, 62% (57.5vehh) of the total delay at Maliban junction and 54% (34.6vehh) of the total delay at the Golumadama junction during the evening peak hour.

Even though the manual control looks better than the current traffic signals, by upgrading the traffic signals, delays during the peak hour were even lesser than the manual control. The traffic flows will not be the same all the days. But in Sri Lanka fixed cycle time signals are used. From the analysis for daily variation (Table 5-17) at Katubedda junction when the junction flow increased by 5% of the designed traffic flow, delay increased by 34.76% (31.88vehh) than of the designed flow. At maliban junction when the junction flow increased by 5% of the designed traffic flow, delay increased by 41.58% (15.25vehh) than of the designed flow.

The analysis shows properly upgraded traffic signals are efficient than the manual control by traffic police officers. But with the daily variation of the traffic fixed cycle time signals cannot be an efficient option. Introducing vehicle actuated signals could be a better solution.

The other problem identified in the manual control was the minor road movement delays. From the results (Figure 5-3, Figure 5-6) from both Maliban junction and

Katubedda junction it is evident that the total delay for Major road movements Moratuwa to Colombo and Colombo to Moratuwa have no significant change for both cases (manual control and updated traffic signal control) of control. But the minor road movements have a significant reduction in delay from manual control to traffic signal control. This is because when the traffic police officers control the intersection, they use mostly queue clearing approach. Because of that their consideration has been mostly on the major road which will have large queue compared minor road movements. The traffic police officers cannot be blamed for this as on the field they currently have no other indication of traffic like vehicle flows or delays and only parameter they can visualise is the queue.

To upgrade the traffic signals, initially they were designed using Webster & Cobbe (1966) signal cycle time model. But for Sri Lankan Conditions it was not the optimum cycle time. From the analysis (Refer Figure 5-11, Figure 5-12, Figure 5-13) at Maliban Junction and Golumadama Junction 0.8C gives the least total delay and at Katubedda Junction 0.85C gives the least total delay.

There can be two reasons for the reduced Cycle time. This should have been due to the reduction of lost time due to the aggressive driving behaviour (lesser fore and lateral spacing while standstill and driving) of Sri Lankan Drivers. Or because of the motorbikes and Three Wheelers maintaining a very low lateral clearance (up to 0.2m) the effective critical Flow rates becomes lesser. So, For Sri Lankan Conditions to design Traffic signal cycle time 0.8 to 0.85 times the cycle time from Webster & Cobbe (1966) signal cycle time model could be an optimum solution.

To use the VISSIM software for the analysis, the driving behaviour parameters in the VISSIM software was calibrated and validated to Sri Lankan conditions. From the calibration results (Table 4-4) Sri Lankan drivers are maintain a very narrow lateral clearance at the junctions, especially three wheels and motorbikes (Standstill distance=0.2m). When overtaking also motor bikes and three wheelers maintain 0.25m and 0.33m lateral clearance. Other category vehicle also maintains 0.3m standstill distance at the junctions and 0.55m lateral distance while driving. And from the observation from the field during the data collection vehicles are overtaking on both sides at the junctions. All the calibrated driving behavioural parameters are presented in chapter 4 (Figure 4-1, Figure 4-2, Figure 4-3 and Figure 4-4). These calibrated parameters can be applicable to the urban areas in Sri Lanka such as Colombo, Gampaha and Kandy city centres. In rural areas these parameters may not be valid.

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