Development of a Micro-simulation Model to Evaluate Shuttle-lane Roadwork Operations

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DECLARATION

Parts of the current PhD research have resulted in the following publications, conferences and presentations.

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Alterawi, M. (2012). A critical review of urban roadworks using shuttle-lane operations. *CSE Conference*. University of Salford, UK.

Alterawi, M. (2012). Utilising Paramics in testing traffic calming management scenarios. *Paramics workshop*. Manchester, UK.

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Yousif, S., Alterawi, M. and Henson, R. (2013). Effect of road narrowing on junction capacity using microsimulation. *ASCE, Journal of Transportation Engineering,* 139(6), pp.574-584.

Alterawi, M. (2013). Site observations of aggressive drivers' behaviour at urban roadworks. *SPARC Conference*. University of Salford, UK.

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DEDICATION

- ⇨ To **God,** who is indeed the Beneficent and the Merciful
- ⇨ To my **father** for his support
- ⇨ To my **wife** and my **children** for their help, support and courage
- ⇨ To my **sisters** and **brothers**
- ⇨ To my **friends,** who didn't forget me in their support and prayers

ABBREVIATIONS

ACR:	After Crossing Roadworks
AGV:	Automatic Guided Vehicles
AH:	Arrival Headway
BAR:	Before Approaching Roadworks
CCTV:	Closed Circuit Television
CORSIM:	CORnell microSIMulation Model
DfT:	Department for Transport
DZ:	Dilemma Zone
FHWA:	Federal Highway Administration (USA)
FORTRAN:	FORmula TRANslating System
FT:	Fixed Time signals operation
GEH:	Geoffrey E. Havers statistical test
HA:	Highways Agency
HCM:	Highway Capacity Manual
HGVs:	Heavy Goods Vehicles
ITS:	Intelligent Transport System
kph:	Kilometre per hour
K-S:	Kolmogorov-Smirnov
LOS:	Level Of Service
MOVA:	Microprocessor Optimised Vehicle Actuation
MUD:	Move Up Delay
MUT:	Move Up Time
MUTCD:	Manual on Uniform Traffic Control Devices
MVD:	Microwave Vehicle Detector
Paramics:	PARAllel MICroscopic Simulation
QUADRO:	QUeues and Delays at ROadworks
QUEWZ:	Queue and User cost Evaluation of Work Zone
RAND:	RANDom number generated by the program
RMSE:	Root Mean Square Error
RMSEP:	Root Mean Square Error Percentage
SCOOT:	Split Cycle and Offset Optimisation Technique

SIMSUR: SIMulation of Shuttle-lane Urban Roadworks

TAL: Transport Advisory Leaflet

TH: Time Headway in seconds

TS: Traffic Signals junction

TTI: Texas Transport Institute (USA)

VA: Vehicle Actuated signals operation

VISSIM: VlSual SIMulation

vph: Vehicle per hour

WZCAT: Work Zone Capacity Analysis Tool

SYMBOLS

μ:	The mean of the distribution
σ:	The standard deviation of the distribution
Δt:	Scanning time in seconds
α:	Level of significance
δ:	The perception reaction time of the driver (seconds)
τ :	The length of the clearing period (Amber plus all-red period) (seconds)
ACC:	Final calculated acceleration/deceleration rate of the vehicle (m/sec^2)
$\mathbf{ACC_1}$:	Maximum acceleration rate of the vehicle (m/sec^2)
$\mathbf{ACC_2}$:	Acceleration rate required to reach desired speed (m/sec^2)
$\mathbf{ACC_3}$:	Acceleration rate required to meet the non-collision criteria (m/sec^2)
$\mathbf{ACC_4}$:	Acceleration rate for slow moving vehicle (m/sec^2)
$\mathbf{ACC_5}$:	Acceleration rate for moving from stationary (m/sec^2)
$\mathbf{ACC_6}$:	Acceleration rate for stopping at traffic signals (m/sec^2)
Amb:	Amber time in seconds
$\mathbf{Amb_P}$:	Amber time for Primary stream in seconds
$\mathbf{Amb_S}$:	Amber time for Secondary stream in seconds
AR:	All-Red time in seconds
$\mathbf{AR_P}$:	All-Red time for Primary stream in seconds
$\mathbf{AR_S}$:	All-Red time for Secondary stream in seconds
Bs:	Buffer Space in metres
C:	Number of Cycles
CT:	Cycle Time in seconds
dc:	The maximum distance a vehicle can safely cross (metres)
$\mathbf{D_{cr}}$:	Critical difference in the cumulative distributions between two samples
$\mathbf{D_{max}}$:	Maximum difference in the cumulative distributions between two samples
DS:	Design Standards
$\mathbf{d_S}$**:**	The shortest distance a vehicle can safely stop (metres)
Ds(n):	The difference between the current position of vehicle *n* and the stop line in metres
$\mathbf{G_P}$:	Green time for Primary stream in seconds
$\mathbf{G_S}$:	Green time for Secondary stream in seconds

k: Traffic density in vehicle/km

L: Site Length in metres

L_V : Vehicle length in metres

m**:** The mean of the lognormal distribution

MaxDL: Maximum deceleration rate of the leading vehicle (m/sec^2)

MaxDF: Maximum deceleration rate of the following vehicle (m/sec^2)

N: Sample size

$NPos_n$: New position of vehicle (n) in metres

NSp_n: New speed of vehicle (n) in km/hr

P: Primary stream

PosF: Position of the follower vehicle

PosL: Position of the leader vehicle

Pos_n: Old position of vehicle (n) in metres

q: Traffic flow in vehicle/hr

r: Coefficient of correlation

RA: Red Amber time in seconds

R_{ACF}: Random number generated from Amber Crossing for Follower

R_{ACL}: Random number generated from Amber Crossing for Leader

$RAmb_P$: Red Amber time for Primary stream in seconds

$RAmb_S$: Red Amber time for Secondary stream in seconds

R_L: Random number generated for vehicle length

R_{RGC}: Random number generated from Red Group Crossing

R_{RSC}: Random number generated from Red Single Crossing

Rt: The reaction time of the vehicle in seconds

R_{type}: Random number generated for vehicle type

S: Secondary stream

s**:** Standard deviation of the lognormal distribution

SpF: Speed of the follower vehicle in km/hr

SpL: Speed of the leader vehicle in km/hr

Sp_n: Old speed of vehicle (n) in km/hr

StpL: Stop line location

T: Total simulation time in seconds

t: Time in seconds

U: Theil's Inequality Coefficient

$\mathbf{U_m}$: Bias proportion

$\mathbf{U_s}$: Variance proportion

V: Number of Vehicles

v: Traffic speed in km/hr

$\mathbf{v_{85}}$: The 85th percentile speed of vehicles or the speed limit (m/s)

$\mathbf{v_{15}}$: The 15th percentile speed of vehicles (m/s)

$\mathbf{x_i}$: The observed flow at time interval i

$\bar{x}$, σx: The mean and the standard deviation for the actual observed data

$\mathbf{y_i}$: The simulated flow at time interval i

$\bar{y}$, σy: The mean and the standard deviation for the modelled data

z: Arrival flow rate

ABSTRACT

This thesis focuses on the development of a micro-simulation model for urban shuttle-lane roadworks. The aim of this research is to study the effectiveness of shuttle-lane roadworks traffic management controls (i.e. operated by temporary traffic signals) on capacity, delays and safety.

SIMSUR (SIMulation of Shuttle-lane Urban Roadworks) micro-simulation model is based on car-following and shuttle-lane rules, considers the various decisions undertaken when approaching temporary traffic signals at urban shuttle-lane roadworks (i.e. tailgating, crossing through amber or even violating the red light). Data from six different sources were collected (from 23 different sites with over 54 hours of traffic data video recordings). This includes data from visited roadworks sites, Individual Vehicle Data (IVD) from UK motorways and data from typical signalised junctions.

Temporary traffic signals operation modes, including Fixed Time (FT) and Vehicle Actuated (VA) signals, have been integrated within the developed micro-simulation model. The developed model has been verified, calibrated and validated using real traffic data.

Various scenarios were tested using the developed simulation model such as the effect of various parameters on system capacity, delays and safety (i.e. site length, HGVs%, directional split, and drivers' non-compliance with temporary traffic signals). The results revealed that the maximum shuttle-lane roadworks capacity values which could be achieved (using existing temporary traffic signals settings) for two-way flow are 1,860 and 2,060 veh/hr for FT and VA signals, respectively. Regression analysis was also carried out using different factors and could be used in analytical models to provide a more accurate estimation of system capacity compared to existing equations. Using improved signals settings, capacity could be increased by about 3.5%. Making the assumption that Microwave Vehicle Detector (MVD) could be simulated within the model, various ranges were tested and the optimum range was found to be 80m (rather than the existing 40m) which could result in an increase in system capacity of 4.2%. Using speed reduction (i.e. speed hump) in advance of the stop line could reduce the effect of dilemma zone by reducing the number of vehicles crossing at the onset of amber or violating the red light by about 33%.

CHAPTER ONE: INTRODUCTION

1.1 Background

Roadworks have become an unavoidable aspect of the road network due to the continuous requirement for road surface maintenance and the need for utility companies to perform their tasks (i.e. water, gas and electricity companies). When roadworks take place in any urban road network, they cause an obstruction to traffic, which in turn increases the risk of accidents and delays/congestion and reduces capacity and vehicle speed, which could lead to extra costs for road users. These issues have led to an investigation into the main factors influencing roadworks operations, with the aim of reducing their effects as much as possible.

In the United States, the Federal Highway Administration (2004) estimated that work zones cause around 10% of overall congestion. It was reported by Tang (2008) that the Texas Transportation Institute report (2007) stated that the cost of congestion in the United States in 2005 alone was $78 billion. In the United Kingdom, it was estimated that the congestion caused by roadworks in London alone costs around £750 million/year (London First, 2012). Furthermore, in the United States in 2006, 1,010 people were killed and around 40,000 injuries were caused because of traffic accidents in work zones (Tang, 2008).

Transportation agencies are under pressure to reduce congestion and accident levels at roadworks. In the United States, the government has started applying incentive/disincentive (I/D) fees to roadworks contractors in an attempt to reduce the duration of the roadworks and therefore try reducing congestion and accident levels. It was indicated that 35 states in the USA are using I/D methods. Lane rental has also been introduced to add daily costs to contractors to speed up the roadworks to reduce duration (Herbsman *et al.*, 1995; Benekohal *et al.*, 2003 and Tang, 2008). In the United Kingdom, the lane rental scheme was introduced in 2011 and implemented in 2012 in the City of London.

1.2 Roadworks in an urban environment

Most urban road networks are built up from single carriageway roads, and when roadworks take place, it is usually carried out by closing one-lane and leaving the other lane for use in alternate one-way working. This is referred to as shuttle-lane operation (Summersgill, 1981; Department for Transport, 2009).

When applying shuttle-lane operations, appropriate types of control are required. These types of control should achieve the following goals:

1- Minimise delays for road users and disperse queues effectively;
2- Safety for road users (drivers, pedestrians and workers).

Designing shuttle-lane roadworks and the traffic control selection method are very complex issues as various factors need to be taken into account. These factors are (but not limited to) site length, presence of pedestrians and cyclists movements, junction proximity and type of operation, proximity to a railway crossing, public transport routes, level and directional split of existing flow, etc.

According to Mahoney *et al*. (2007), alternating one-way operation (shuttle-lane) has the following advantages and disadvantages:

Advantages:

- ⇨ Low agency cost compared with other methods;
- ⇨ Several variations available (various control methods can be implemented).

Disadvantages:

- ⇨ Requires the stopping of traffic;
- ⇨ Reduces capacity.

1.3 Problem statement

Shuttle-lane roadworks government design guidelines of several countries (i.e. United Kingdom, United States, France and Australia) have been collected and studied. Various differences were found within these guidelines such as recommended site lengths and maximum allowed flow levels for each method of traffic operation. Important factors were not taken into account when selecting or designing each traffic control method.

Mathematical models generally have various limitations, including the ability to replicate queues and delays for oversaturated conditions and a lack of comparison with observed field data. Mathematical models also cannot replicate the effect of various methods of control for shuttle-lane such as Vehicle Actuated (VA) or Intelligent Transportation System (ITS) and no effect of vehicles' acceleration and deceleration (Cassidy and Han, 1993; Son, 1999; Huang and Shi, 2008).

Analytical models use inadequate capacity models, because there are several roadworks features such as the presence of flaggers, ITSs and Vehicles Actuated (VA) traffic signals which are not reflected in these models (Edara and Cottrell, 2007; Tang, 2008; Ramezani *et al*., 2011). Previous studies showed that the accuracy of some software programs in estimating roadworks delay and queue length are inadequate (Schnell *et al*., 2002; Benekohal *et al*., 2003 and Lee *et al*., 2008).

Simulation models were generally designed for under-saturated conditions where traffic does not exceed the site capacity. Simulation packages have various limitations such as omitting vehicles, various parameters are imbedded within the program code that the users do not have access to and the required level of complicated steps to ensure correct behaviour of such a system as is the case from real traffic situations. The models also do not take into account the aggressive nature of drivers' behaviour (i.e. tailgating and red light violations).

Considering the limitations of the existing simulation, mathematical and analytical models in estimating site capacity, queue length and delay, a new micro-simulation model needed to be developed to take into account:

- ⇨ Accurate estimation of shuttle-lane roadworks capacity, delays and queues under various traffic control conditions;
- ⇨ The ability to replicate aggressive drivers' behaviour such as tailgating (close-following), amber crossing and red light violations.
- ⇨ The effect of various parameters that affect roadworks performance such as HGVs percentage, directional split, etc.;
- ⇨ The ability to test advanced traffic control techniques such as the latest Vehicle Actuated (VA) signals settings, microwave vehicle detectors with various detection lengths, improved control methods, etc.

1.4 Aims and objectives

The aim of this study is to develop a micro-simulation model, which will be used as a tool to investigate the factors that affect the operation of shuttle-lane roadworks on reducing travel time by reducing delays and maximising capacity (which might lead to a reduction in vehicle emissions) and reducing the aggressive nature of drivers' behaviour which might lead to a reduction in the risk of accidents (i.e. tailgating and amber crossing/red light violations).

The objectives of the study are to:

- ⇨ Determine the factors that affect the operation of shuttle-lane roadworks based on previous literature research.
- ⇨ Develop a traffic micro-simulation model (i.e. using S-Paramics and Compaq Visual Fortran) representing shuttle-lane operation. The model should be capable of taking into consideration the limitations of previous models using the existing rules and algorithms and applying the necessary modifications as required.
- ⇨ Use real observed traffic data to build, verify, calibrate and validate the developed model.
- ⇨ Use real traffic data to study the effect of various traffic signals operation methods such as Fixed Time signals (FT) and Vehicle Actuated signals (VA).
- ⇨ Utilise the model to study the effect of various traffic parameters such as traffic composition, flow levels and HGVs percentage on delays and site capacity.
- ⇨ Use the model to test roadway factors such as site length, which affect capacity and delays.
- ⇨ Carry out regression analysis to develop a more comprehensive relationship between those parameters and capacity which can be used in analytical models.
- ⇨ Utilise the model to test new techniques on the methods of operation which could lead to improvement of site operation (maximising capacity and reducing delays) and improving safety.
- ⇨ Utilise the model to test the effect of aggressive drivers' behaviour such as tailgating and amber crossing/red light violations and propose new techniques to reduce it accordingly.

1.5 Thesis outline

The thesis is divided into nine sections as described below:

- ⇨ **Chapter one** provides an introduction to the importance of roadworks and a brief description of roadworks in urban environment, problem statement and the study aim and objectives;

- ⇨ **Chapter two** presents the review of literature of shuttle-lane roadworks from previous studies and design manuals.
- ⇨ **Chapter three** presents the data collection methodology and description of the visited sites during the current and previous available studies.
- ⇨ **Chapter four** presents the analysis that was performed on the collected data.
- ⇨ **Chapter five** describes the developed S-Paramics simulation model. It also describes the calibration, validation and limitations of the developed model.
- ⇨ **Chapter six** describes the newly developed SIMSUR (SIMulation of Shuttle-lane Urban Roadworks) simulation model and explains the adopted car-following and shuttle-lane rules.
- ⇨ **Chapter seven** explains the verification, calibration and validation of the car-following and shuttle-lane rules and also for the whole of the simulation model using real data from the visited sites and from different sources.
- ⇨ **Chapter eight** presents the application of the developed model and the improvement achieved in terms of safety and capacity.
- ⇨ **Chapter nine** presents the conclusions and recommendations for future work.

The structure of the following chapters has been presented to correspond to the development process of the current research as illustrated in Figure 1.1. It can be seen from Figure 1.1 that two main rules (sub-models) have been developed for the current study (i.e. car-following and shuttle-lane rules). The car-following rule governs the longitudinal vehicle behaviour (i.e. the relationship between the leader and the follower) and shuttle-lane rule governs the vehicle behaviour and interaction at the shuttle-lane roadworks operated by temporary traffic signals.

The calibration of these rules was achieved using different data categories (i.e. trajectory data from Germany and observed real data from the UK). The results show reasonable behaviour when compared with other simulation models such as VISSIM and S-Paramics. Following the development of shuttle-lane rules, the calibration process has been achieved by utilising field data for both Fixed Time (FT) and Vehicle Actuated signals (VA). Finally, the whole developed simulation model was calibrated and validated with over 54 hours of real video recorded field data in addition to other sources as discussed in Chapters 3 and 4.

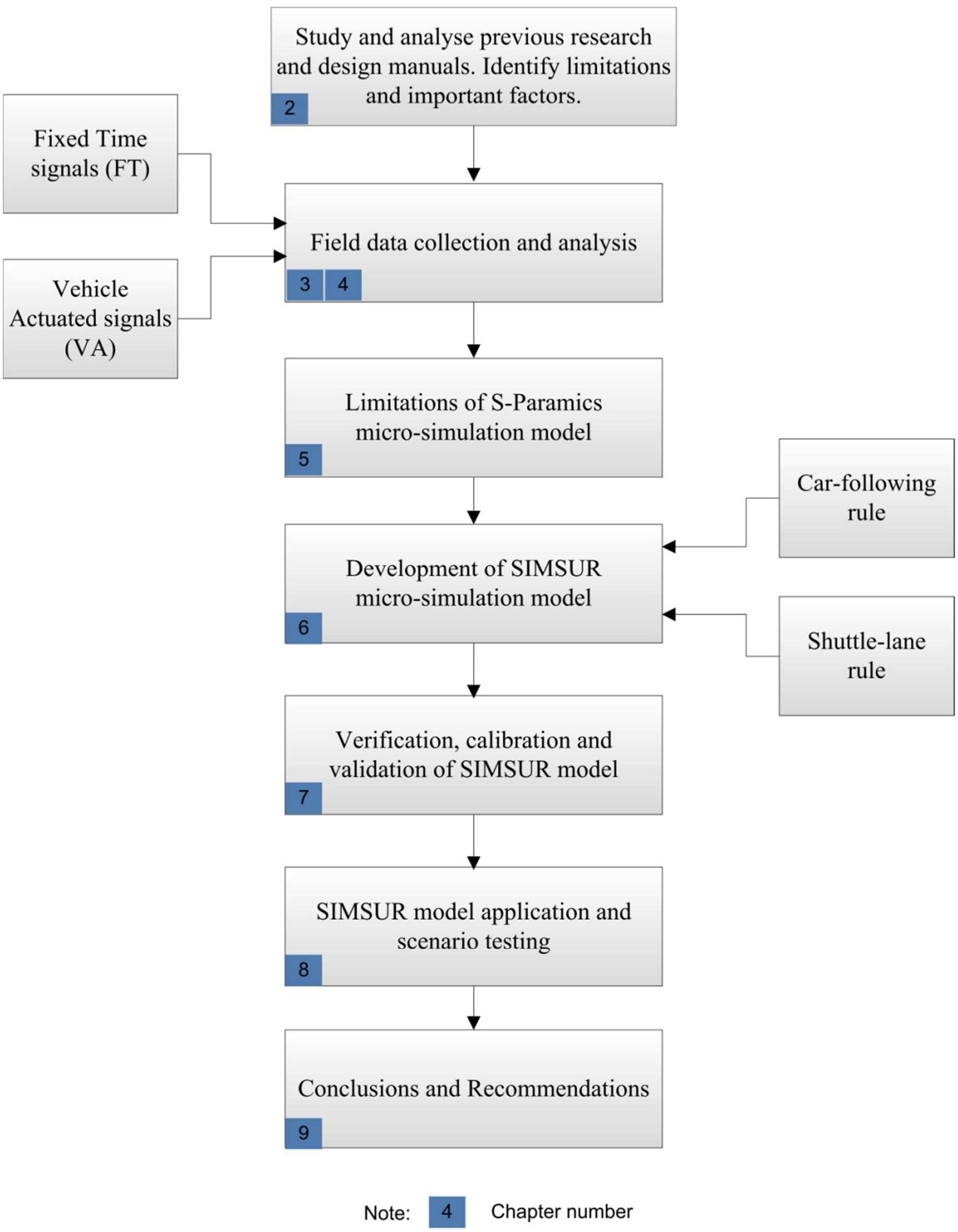


Figure 1.1: Flow chart of the current research study

CHAPTER TWO: LITERATURE REVIEW

2.1 Introduction

The literature review chapter summarises the various aspects and design standards of shuttle-lane operation and the types of traffic control methods employed. It also summarises studies that have been carried out to test different shuttle-lane roadworks components.

2.2 Shuttle-lane site layout

A typical site layout of shuttle-lane operation on single carriageway roads can be seen in Figure 2.1. According to the Department for Transport (2009), shuttle working with traffic control will be implemented if the unobstructed width (the distance between the edge of the cone and the carriageway curb) is within the limits shown in Table 2.1.

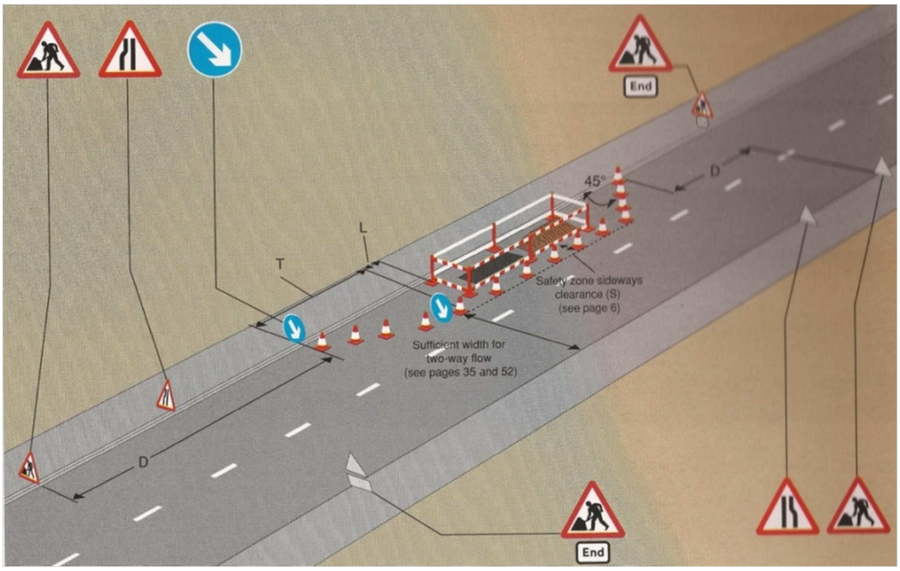


Figure 2.1: Typical site layout of shuttle-lane operation on single carriageway (Department for Transport, 2011)

Table 2.1: Unobstructed width for different single carriageway roadworks types (Department for Transport, 2009)

Method of operation	Normal traffic including buses and HGVs	Cars and light vehicles only
Two-way working	6.75 metres minimum	5.5 metres minimum
Shuttle-lane working with traffic control	3.7 metres maximum 3.25 metres desirable minimum 3.0 metres absolute minimum	3.7 metres maximum 2.75 metres desirable minimum 2.5 metres absolute minimum

2.3 Stream definition

In order to differentiate between the two traffic streams that use the shuttle-lane roadworks site, the following terms have been used (Summersgill, 1981), which are also illustrated in Figure 2.2:

- *Primary stream*: Is the traffic stream which is running in the obstructed path (by the works);
- *Secondary stream:* Is the traffic stream which is running in the unobstructed path;

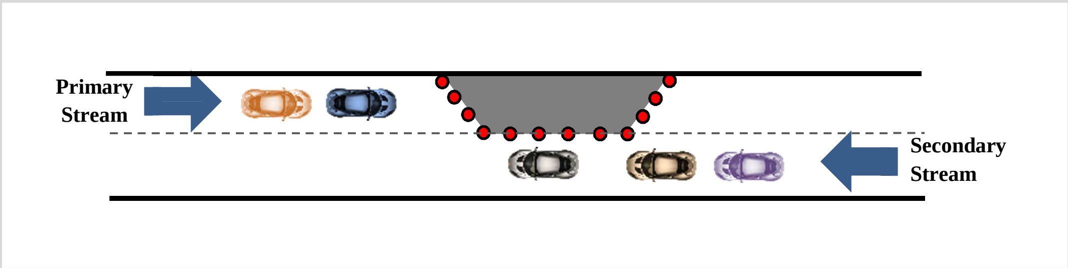


Figure 2.2: Illustration of primary and secondary streams

It is important to distinguish between both streams as the drivers in the primary stream (which is obstructed by the work) generally gives priority to the secondary stream regardless if a signed priority control is used or not (Summersgill, 1981). Primary stream vehicles also require extra time to negotiate the layout (enter the running lane), and also when leaving the site back to their original lane.

2.4 Types of traffic control

Shuttle-lane roadworks will create conflict points between both traffic streams (e.g. primary and secondary) which require some form of control. The functions of using traffic control devices in roadworks are as follows (Matson *et al*., 1955):

- To warn drivers about the hazards ahead;
- Alert drivers of traffic conditions ahead;
- Guide drivers by the right instructions in order to minimise the conflicts that could occur.

Traffic control methods that can be used to operate shuttle-lane roadworks, which are obtained from various design manuals are summarised below (Federal Highway Administration, 2009; Queensland Goverment, 2010; Department for Transport, 2009; Makhloufi and Certu, 2003):

1- No specified priority control (Give and Take);
2- Signed priority control;
3- Traffic signals control;
4- Control by manually operated Stop/Go signs;
5- Flag transfer method;
6- Pilot car method/convoy working.

Table 2.2 shows the available control methods for shuttle-lane roadworks in different countries, and each method is described in details in the following sections.

Table 2.2: Various shuttle-lane roadworks control methods in different countries

No.	Control Method	Australia	France	United Kingdom	United States
1	No specified priority	✓	✓	✓	✓
2	Signed priority	✓	✓	✓	✓
3	Traffic signal	✓	✓	✓	✓
4	Stop/Go sign		✓	✓	✓
5	Flag transfer				✓
6	Pilot car	✓		✓	✓

2.4.1 No specified priority (Give and Take)

The “Give and Take” is a control method where there is no specified priority for any traffic stream. Both directions (streams) have equal priority and the drivers have to take a decision on the suitability of gaps in order to cross the roadworks site safely. A typical site layout of shuttle-lane roadworks operated by “give and Take” method is shown in Figure 2.1 above.

According to the Department for Transport (2009), the “Give and Take” operation is the natural method for operating shuttle-lane roadworks. The visibility should be good where drivers from each approach should see 50 metres beyond the end of the works. It is also stated that if the work is to be carried out at night, then another alternative to this method should be considered such as temporary traffic signals.

2.4.2 Signed priority control

Signed priority is a control method where one stream, usually the one with the unobstructed lane by works (secondary stream) has priority over the obstructed direction (primary stream) as stated by the Department for Transport (2009). These priorities are backed up with the use of priority signs as shown in Figure 2.3.

Signed priority control requires a certain amount of visibility based on different speed limits, and it can be employed at night if certain conditions are met, such as street lighting and illuminated traffic signs as stated by the Department for Transport (2009).

According to the Federal Highway Administration (2009), the stop or yield sign can be used on low volume roads (less than 400 vehicles per day) and when visibility is good that the drivers can see the other end of the work zone and also the other direction of traffic.

Signed priority method operates in the same way as traffic calming using throttles. According to Yousif *et al*. (2013), if the sign is placed near a junction, special consideration should be given to various parameters such as the level of traffic, the distance from the junction and also the direction of the priority streams.

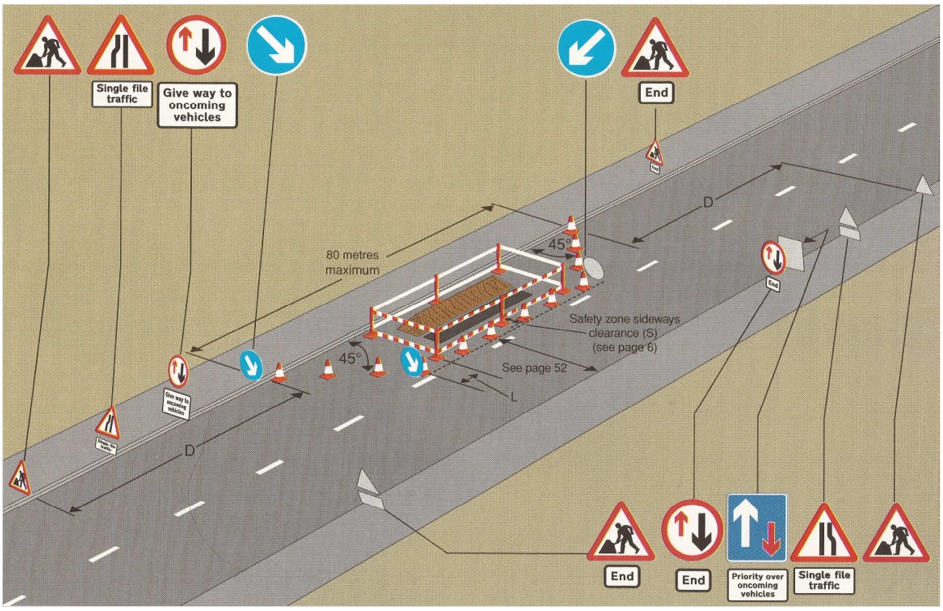


Figure 2.3: Typical site layout for shuttle-lane roadworks operated by priority signs (Department for Transport, 2011)

2.4.3 Traffic signals control

Traffic signals control method is where portable or fixed traffic signals are placed in certain locations at the roadworks site to control traffic movements of both streams. The operation of the signals can be based on either Fixed Time signals (FT) or on Vehicle Actuated signals (VA) with the aid of vehicle detection techniques.

According to the Queensland Goverment (2010), either a portable or temporary fixed traffic signals can be used in shuttle-lane roadworks. The signals should be operated primarily under

VA mode to satisfy certain conditions. If the VA operation is not possible, FT operation is the next preferred option. Manual operation should also be allowed (to override the signals settings) in certain circumstances.

According to the Federal Highway Administration (2009), if the temporary traffic signals are located within 0.5 miles of an adjacent signalised intersection, a connected operation should be considered. There is no reference to signals settings for flow groups or all-red period according to different site lengths but engineering judgment should be used to determine these settings.

The traffic signals controller can adjust the signals timings (all-red time and maximum green time) in order to suit the site length, which is measured between the ‘WAIT HERE’ signs as shown in Figure 2.4 (Department for Transport, 2009).

All-red timing has to be adjusted to the minimum in order to give the moving vehicles chance to clear the roadworks site as shown in Table 2.3 and illustrated in Figure 2.5. The maximum green timing has to be set to the maximum depending on the site length as shown in Table 2.4 (Department for Transport, 2009; ITE, 2010).

Table 2.3: All-Red timing for different site length (Department for Transport, 2009)

Distance (metres)	**0-50**	**50-100**	**100-150**	**150-200**	**200-250**	**250-300**
All-red timing (sec)	5	10	15	20	25	30

Table 2.4: Maximum green time for different site length
(Department for Transport, 2009)

Distance (metres)	**30-75**	**75-135**	**135-195**	**195-300**
Green time (sec)	35	40	45	50

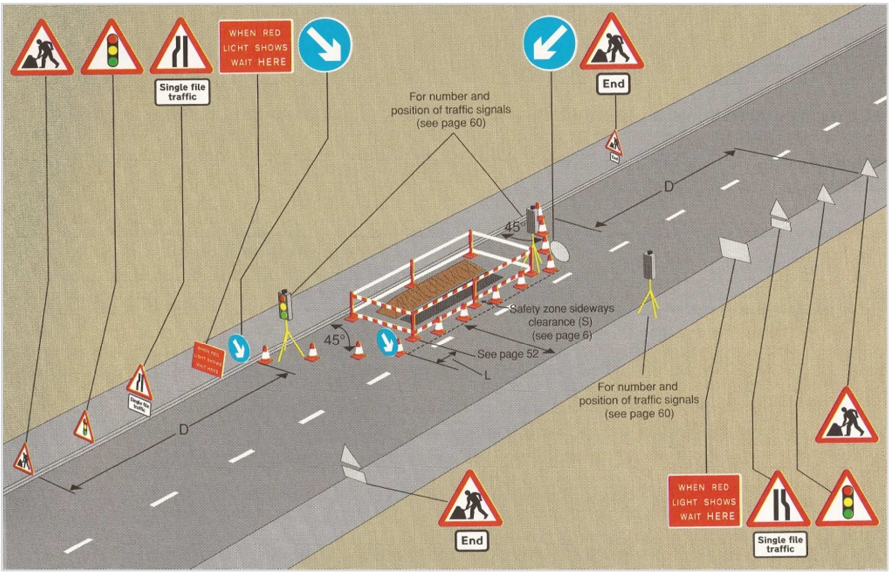


Figure 2.4: Typical site layout for shuttle-lane roadworks operated by portable traffic signals (Department for Transport, 2011)

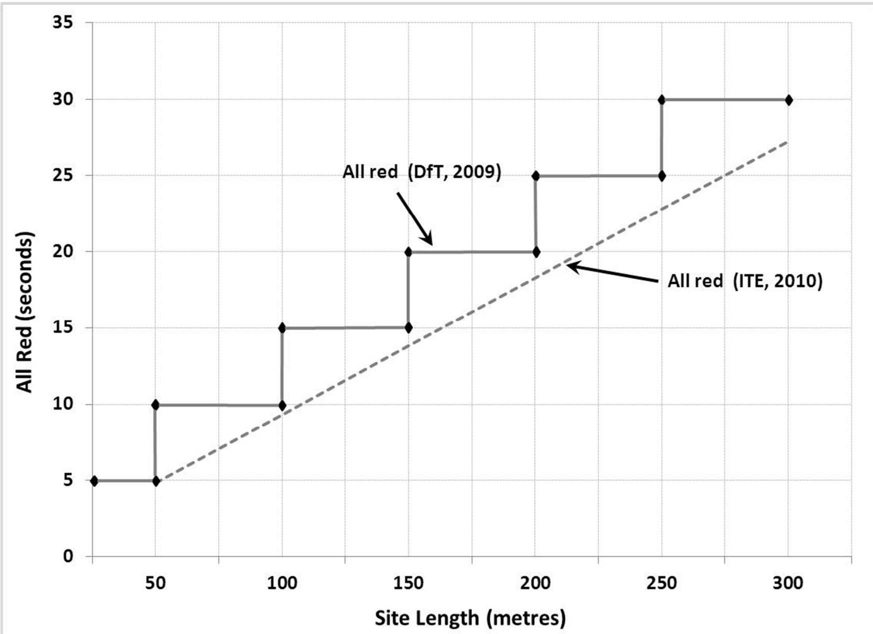


Figure 2.5: All-red timing for different site length

2.4.3.1 Vehicle-Actuated operation (VA)

The VA mode is the default preferred option when using the signals control method. The signals controller should be accompanied with detectors to respond to variable vehicle demand. The VA mode will be used if the traffic flow is not hindered by the operations at the work zone, and the roadworks site traffic control is required after working hours as stated by the Queensland Goverment (2010).

Microwave Vehicle Detector (MVD) is a detection unit placed on top of the signals head which uses microwave technology to detect the movement of vehicles. The MVD can detect most moving vehicles, including larger motorcycles with a maximum detection range of up to 40 metres (but with smaller motorcycles and cycles the range is reduced to 25 metres) assuming that vehicles are travelling towards the MVD with a speed greater than 10 mph and the detector is aligned correctly (Department for Transport, 2008).

Few limitations have been reported regarding the use of MVD detectors, such as it needs to be correctly aligned (correct angle), detection ability might be affected under harsh weather conditions (heavy rain or snow), the detection might be affected by parked vehicles or blocked view such as trees and might not detect approaching vehicle with speed under 10 mph (Department for Transport, 2008; Dickenson and Wan, 1990 and Medina *et al*., 2012).

According to the latest specification for portable traffic signals control and equipment to be used at roadworks (Highways Agency, 2005B), the minimum green time should be configured to either 7 or 12 seconds and will be extended following a passage of each vehicle (vehicles will be detected using the MVD unit) . The maximum green will be set to a value up to 50 seconds (depending on site length and as shown in Table 2.4).

There is no direct reference given by the Highways Agency (2005B) to the green time extension amount which should be given to each vehicle. It was stated by the Department for Transport (1999a) that green time should be extended by an increment of 0.5 seconds until the vehicle passes the stop line.

2.4.3.2 Fixed Time operation (FT)

According to the Queensland Goverment (2010), FT mode will be used where the VA mode is not possible, the traffic flow at the approaches is relatively constant and the traffic control operation is required at the roadworks site after working hours.

2.4.3.3 Manual operation

The manual operation of traffic signals by the traffic controllers can be carried out if traffic flow at the approaches is variable and is blocked from time to time by the roadworks, or the VA mode fails and the FT mode is not appropriate, or traffic must be kept outside the work zone for a period of time (e.g. blasting, priming, etc.) as stated by the Queensland Goverment (2010).

2.4.3.4 Signals phase sequence

The design procedure of traffic signals timings for shuttle-lane roadworks is not as complicated as in the case of typical signalised junctions in urban areas where the designer has to consider multiple conflicting movements from various directions and also the presence of pedestrians crossing at junctions (for the design procedure of traffic signals for normal junctions, see for example Salter and Hounsell, 1996). Temporary traffic lights at shuttle-lane roadworks are operated using the following phases (Highways Agency, 2005A):

- ⇨ Green period (G);
- ⇨ Stopping amber period (Amb), usually applied as 3 seconds;
- ⇨ Red and/or all-red period(AR), which is used to clear the shuttle-lane site;
- ⇨ Red-amber period (RAmb), usually applied as 2 seconds;

The phase sequence of temporary traffic signals at shuttle-lane roadworks is illustrated in Figure 2.6. Values for maximum green and all-red periods are calculated based on site length as shown in Tables 2.3 and 2.4. Cycle length for the temporary traffic signals can be calculated using Equation 2.1 as follows:

$$\mathrm{CT} = \mathrm{G}_P + \mathrm{Amb}_P + \mathrm{AR}_P + \mathrm{RAmb}_S + \mathrm{G}_S + \mathrm{Amb}_S + \mathrm{AR}_S + \mathrm{RAmb}_P \qquad \text{Equation 2.1}$$

Where,

CT is the Cycle Time;

P is the Primary stream;

S is the Secondary stream;

$\mathbf{Amb_P}$ and **$\mathbf{Amb_S}$** are the stopping amber time for primary and secondary streams, respectively (in seconds);

$\mathbf{AR_P}$ and **$\mathbf{AR_S}$** are the all-red period for primary and secondary streams, respectively (in seconds);

$\mathbf{RAmb_P}$ and **$\mathbf{RAmb_S}$** are the red amber time for primary and secondary streams, respectively (in seconds);

$\mathbf{G_P}$ and **$\mathbf{G_S}$** are the green time for primary and secondary streams, respectively (in seconds).

Figure 2.6: Temporary traffic signals phase sequence

2.4.3.5 Comparison with typical signalised junction

There are various differences between Temporary Traffic Signals (TTS) at shuttle-lane roadworks and Typical Signalised Junctions (TSJ). These differences could be summarised as follows:

- ⇨ The number of conflicting movements is less in TTS at shuttle-lane roadworks compared with TSJ due to the presence of left/right movements and minor/major arms in TSJ which could affect safety. Therefore, the calculation of the phase sequence and green time/all red period may be different.
- ⇨ There may be issues relating to visibility on shuttle-lane roadworks when compared with TSJ. These are affected by the presence of roadworks (including site lengths, especially for long site lengths and possible bends within the geometry of the road). Vehicles on the primary stream have to change their horizontal trajectory at shuttle-lane roadworks.
- ⇨ There may possibly be differences in detection methods used for TTS at shuttle-lane roadworks. Normally Microwave Vehicle Detectors (MVD) are used which influence the operation as explained in previous sections compared with TSJ which mostly use loop detectors.
- ⇨ Other differences may include factors such as the required time of installing the signals, connection between signal heads (radio connection for TTS) and method of operation (e.g. FT and VA for TTS while a more advanced operation is available for TSJ, such as MOVA and SCOOT). See for example Department for Transport (1997, 1999b).

2.4.4 Control by manually operated Stop/Go Signs

Stop/Go boards control method is controlled either manually or electronically (e.g. using radio device) by an operator, which give directions to each traffic stream to either go or stop as shown in Figure 2.7.

It is stated by the Department for Transport (2009) that the “Stop and Go” signs form a double sided sign placed outside the safety zone on a suitable stand. The sign will be operated remotely (e.g. using radio control device) unless for safety reasons, then manual operation will take place. If the sign could not be placed outside the safety zone, then temporary traffic signals are introduced. This method can be used at night if accompanied with appropriate illumination by operators.

According to the Federal Highway Administration (2009), for a one-lane two-way operation (shuttle-lane) traffic can be controlled by a flagger (holding Stop and Slow signs) at each end of the work space and they can communicate orally, electronically or by a manual signal. If the visibility is good and the site length is short, the site can be controlled by a single flagger.

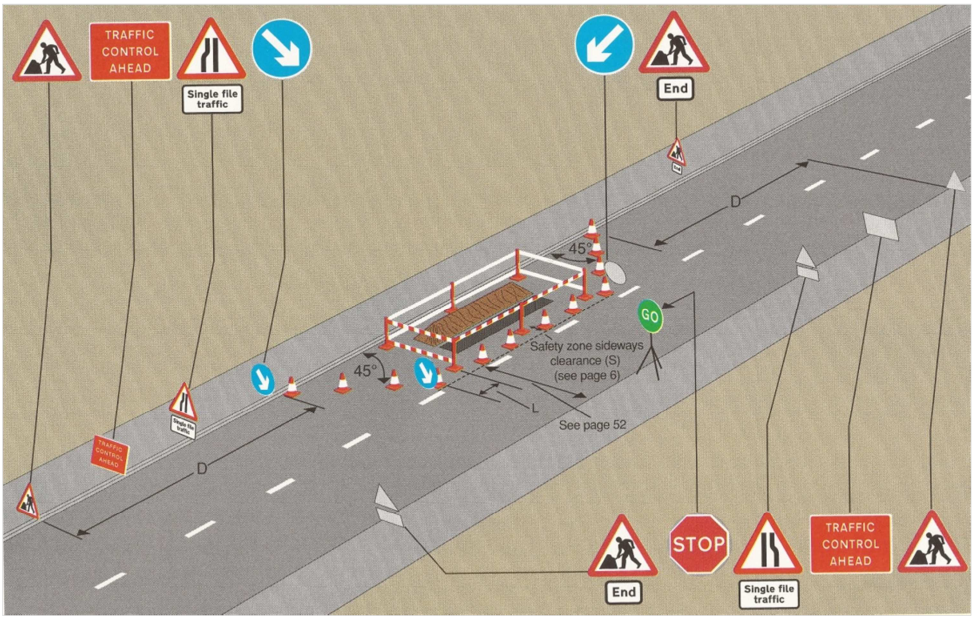


Figure 2.7: Typical site layout for shuttle-lane roadworks operated by Stop/Go boards (Department for Transport, 2011)

2.4.5 Flag transfer method

According to the Federal Highway Administration (2009), the flag transfer method can be carried out by asking the driver of the last vehicle entering the roadwork site to transfer the flag and deliver it to the flagger at the other end. The flagger at the other end will then know that traffic is permitted to move in the other direction. Usually, this method is carried out only on roadwork section of less than 1 mile in length. Typical shuttle-lane operation site layout operated by flag transfer is shown in Figure 2.8.

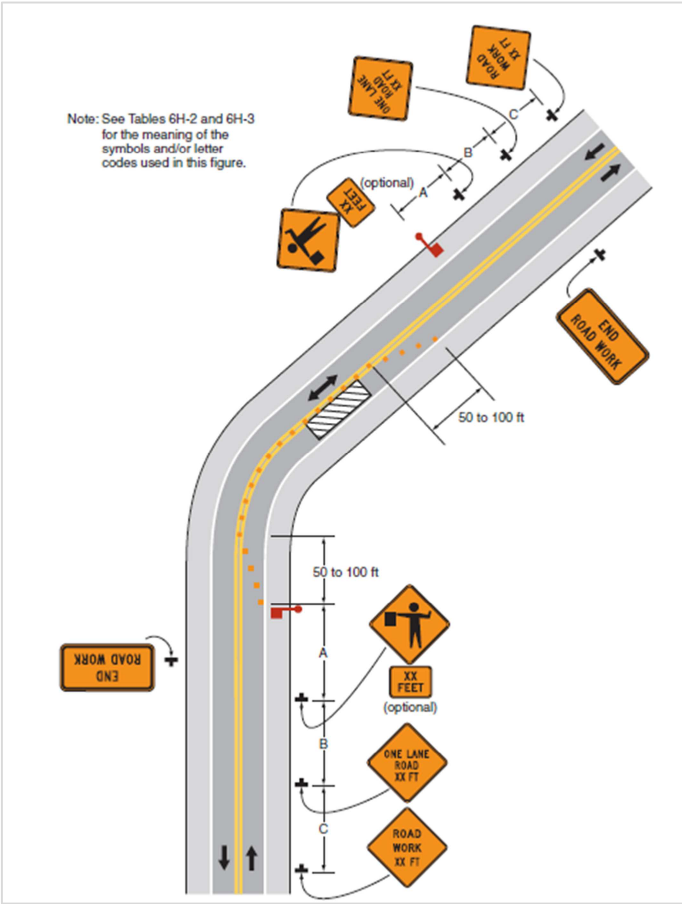


Figure 2.8: Typical site layout for shuttle-lane roadworks operated by flag transfer method (Federal Highway Administration, 2009)

2.4.6 Pilot car method

Based on the Federal Highway Administration (2009), the pilot car control method is carried out by the use of a car which will lead a queue of vehicles from each stream in alternating way to the other side of the roadworks site. A flagger should also be available at the end of the activity area controlling the traffic until the pilot car is available. The use of pilot car is usually associated with poor visibility at roadworks site.

The pilot car/convoy working is usually implemented if there is no or little safety zone for vehicles to pass the shuttle-lane roadworks site and the traffic should be brought to a stop before approaching the site. The traffic will be passing through the site at a reduced speed (10 mph or less). The method can be successfully implemented at sites with two-way traffic volume between 900-1000 vehicles/hour (Department for Transport, 2009).

Table 2.5 summarises the advantages and disadvantages of each of the seven shuttle-lane control methods listed previously.

2.5 Traffic control selection criteria

Selection criteria for the appropriate traffic control method require the following considerations (Federal Highway Administration, 2009; Department for Transport, 2009):

- ⇨ Traffic volumes;
- ⇨ Duration of work;
- ⇨ Site layout and conditions (e.g. visibility, site length, etc.);
- ⇨ Personnel available;
- ⇨ Proximity of a junction, railway crossing or pedestrian crossing.

Table 2.5: Advantages and disadvantages of shuttle-lane control methods

Control Method	Advantages	Disadvantages
No specified priority	⇨ No setup cost required. ⇨ Do not require personnel to operate or maintain. ⇨ Long work duration.	⇨ Cannot be applied at night (unless illuminated) or in poor visibility. ⇨ Only operates on low traffic volume.
Signed priority	⇨ Can be operated at night if illuminated and the street is lit. ⇨ Do not require personnel to operate or maintain. ⇨ Long work duration.	⇨ Special considerations need to be taken if placed near a junction.
Traffic signal	⇨ Operate on high volume of traffic. ⇨ Long work duration. ⇨ Can be synchronised/linked if near signalised junction.	⇨ Can cause high delays and queues if not setup correctly. ⇨ Regular maintenance of detectors is required to ensure adequate operation.
Stop/Go sign	⇨ Can be used at night with sufficient illumination. ⇨ Can be used on high speed roads. ⇨ Can be adapted to respond to traffic flow variability. ⇨ Can be used near intersection.	⇨ Requires personnel to operate. ⇨ Short duration of work. ⇨ Requires good visibility. ⇨ Not applicable to use during night or weekends (inactive time) without the presence of personnel.
Flag transfer	⇨ Can be used at sites up to 1 mile in length.	⇨ Inadequate for sites with high traffic volume. ⇨ Interrupt the drivers by asking to deliver a flag. ⇨ Requires personnel to operate.
Pilot car	⇨ Can be used with other control methods.	⇨ Requires personnel to operate. ⇨ Requires car with special signs to operate all the time.

Table 2.6 indicates the maximum desirable site length for each level of traffic volume in vehicles per hour as a selection method in Australia. The site can be operated by traffic controller, portable or temporary fixed traffic signals (Queensland Government, 2010).

Table 2.7 shows the maximum two-way traffic flow for each roadworks site length under the Stop and Go method (Department for Transport, 2009).

Table 2.8 summarises the different criteria available for selecting the appropriate traffic control method for shuttle-lane roadworks in different countries. The values shown in this table for the two-way flow, site length and speed indicate the maximum values.

The values shown in Table 2.8 may vary depending on local conditions and circumstances. It is also noticed in the design guidelines, that (in few cases) there is no specific value provided such as low/high for traffic volume and short/long for site length without providing guidance on those descriptions. Therefore, it is based on engineering judgment to decide which might cause poor design leading to higher delays and queues than expected.

Table 2.6: Desirable maximum length of single-lane operation (Queensland Goverment, 2010)

Traffic volume (both directions) in vehicle per hour	Length of single lane section (metres)
800	70
700	100
600	150
500	250
300	600
<300	800

Table 2.7: Critical site length vs. two-way flow for Stop/Go operation method (Department for Transport, 2009)

Method of control	Maximum speed limit (mph)	Length of coned area (metres)	Maximum two-way traffic flow (vehicle per hour)
Stop/Go signs	60	100	1,400
		200	1,260
		300	1,060
		400	940
		500	840

According to Schonfeld and Chien (1999) and Chien *et al*. (2002), both the Queensland Government (1988) and Highway Capacity Manual (HCM, 1985) cannot address the variety of problems, such as the optimal traffic control method selection at roadworks, although the responsible agencies have attempted to develop such guidelines.

Table 2.8 shows that the maximum two-way traffic flow for each control method varies between countries. It also shows that there is no reference to the heavy goods vehicles (HGVs) composition (except in one situation) or the directional split for both streams in selecting each control method which is believed to be an important factor that affects the traffic operations at shuttle-lane roadworks.

Table 2.8: Traffic control selection criteria for shuttle-lane roadworks in different countries

Method of control	Length of coned area (metres)	Maximum two-way traffic flow (vehicle per hour)	Maximum speed limit (kph)	Considerations
No specified priority (Give and Take)	**(1)** 60 (max) **(2)** 15 (max) **(3)** 50 (max) **(4)** N/A	**(1)** 40 **(2)** 100 **(3)** 400 and 20 HGVs/hr **(4)** 400	**(1)** 70 **(2)** 50 **(3)** 48 (30 mph) **(4)** N/A	**(1)** Good visibility **(2)** Good visibility **(3)** Good visibility **(4)** Low volume and good visibility
Signed Priority	**(1)** 100 **(2)** 50-100 **(3)** 80 (max) **(4)** N/A	**(1)** 150 (vehicle/day) **(2)** 400 **(3)** 840 **(4)** 400	**(1)** 60 **(2)** 50 **(3)** 96 (60 mph) **(4)** N/A	**(1)** Only in good visibility **(2)** N/A **(3)** N/A **(4)** Low volume and good visibility
Traffic signals	**(1)** N/A **(2)** 400 **(3)** 300 (max) **(4)** N/A	**(1)** No limit **(2)** 800 **(3)** No limit **(4)** N/A	**(1)** 60 **(2)** 50 **(3)** 96 (60 mph) **(4)** 96	**(1)** N/A **(2)** N/A **(3)** N/A **(4)** N/A
Stop/Go signs	**(1)** Not Used **(2)** 400 **(3)** 500 **(4)** Controller sight	**(1)** Not Used **(2)** 1,000 **(3)** 1,400 **(4)** N/A	**(1)** Not Used **(2)** 50 **(3)** 96 (60 mph) **(4)** 120	**(1)** Not Used **(2)** N/A **(3)** Check **(4)** Table 2.7
Flagg Transfer	**(1)** Not Used **(2)** Not Used **(3)** Not Used **(4)** 1,610 (1 mile)	**(1)** Not Used **(2)** Not Used **(3)** Not Used **(4)** N/A	**(1)** Not Used **(2)** Not Used **(3)** Not Used **(4)** N/A	**(1)** Not Used **(2)** Not Used **(3)** Not Used **(4)** N/A
Pilot Car	Usually available with other control methods if the visibility is poor or little safety clearance			

(1) Queensland Goverment (2010)-MUTCD **(2)** Makhloufi and Certu (2003) **(3)** Department for Transport (2009)

(4) Federal Highway Administration (2009)-MUTCD **N/A** No direct information Available

2.6 Effects of roadworks

Shuttle-lane roadworks have various effects on the road network and road users. Taking those impacts into consideration will determine the optimum management and operational methods. Detailed description of each type of the likely effects caused by shuttle-lane roadworks are provided in the following sections.

2.6.1 Restricted height or width

When applying shuttle-lane roadworks, the road width is subjected to a reduction (restriction) as mentioned in Table 2.1. Height restriction can also be imposed on drivers if the work is being carried out under a bridge. In both situations, warning signs should be provided at suitable distance to allow drivers to take extra care or to follow a diversion (Department for Transport, 2009).

2.6.2 Reduction in speed

Speed reduction at shuttle-lane roadworks sites are usually introduced for safety reasons. The speed limit can range between 30 mph to 60 mph. Department for Transport (2009) specified the speed limit for each type of hazard for high speed roads with speed limits of 50 mph or more (e.g. poor visibility, narrow lanes, etc.).

Traffic speed will inevitably be reduced in urban environment, because of the presence of busy roads and temporary speed limits might not be necessary except in certain circumstances.

The reduction in drivers' speed at roadworks is also caused by the presence of traffic control devices where drivers have to take extra caution to reduce their speed to respond to such a control. Furthermore, the presence of queues at roadworks site will force drivers to reduce their speed and therefore increase in journey time.

2.6.3 Reduction in visibility

As mentioned by the Department for Transport (2009) that because of the presence of temporary structures, stores of the materials, bends, etc., the forward visibility available to the drivers will be reduced. It is therefore essential to make sure that the reduction of visibility is kept to the minimum at all day times especially at bends. Some methods are introduced to the drivers such as warning signs and speed limits that could reduce the hazard of reduced visibility.

2.6.4 Interference with non-motorised road users

Shuttle-lane roadworks will usually affect the free movement of pedestrians, cyclists, vulnerable road users, which will force them to be diverted from their normal path to the carriageway. According to the Department for Transport (2009), this hazard should be minimised through the use of barriers or fences with the addition of lamps by night, which clearly warn the pedestrians of their presence and keep them away from the movement of the traffic providing safe a route/exit.

2.6.5 Interference with other junctions

In urban areas, junctions are usually located within a close proximity to each other. Shuttle-lane roadworks might cause enough queues to block back several junctions, which will cause delays to drivers which are not part of the roadworks traffic. Extra care should be taken when designing roadworks near junctions.

2.6.6 Reduction in capacity

Capacity is a very important measure which determines the maximum amount of traffic volume that can pass through the shuttle-lane roadworks in an hour. The appropriate identification of the system capacity will be used to calculate queues and delays resulted from capacity reduction.

Maze *et al*. (2000) stated that different methods of calculating capacity were noticed such as the Texas Transportation Institute (TTI), which defines capacity as the hourly traffic volume under congested traffic conditions. A Pennsylvania study defined work zone capacity as the hourly traffic volume converted from the maximum 5 minutes flow rate. A California study measured capacity by using 2-three minutes intervals separated by 1 minute. The value was then averaged and multiplied by 20 to convert to hourly value. It was also stated by Maze *et al*. (2000) that Dixon and Hummer (1995) defined capacity as the flow rate where traffic changes from an uncongested to a congested condition. Jiang (1999) defined capacity as the flow just before a sharp drop in speed followed by a steady period of low speed and fluctuating traffic flow.

Summersgill (1981) have calculated the capacity of shuttle-lane roadworks for different site lengths (30 to 280 metres) operated by traffic signals, no priority and priority specified as illustrated in Figure 2.9.

It can be seen from Figure 2.9 that the maximum site capacity for no priority or priority specified operation is around 1290 veh/hr for two-way flow. For shuttle-lane roadworks operated by temporary traffic signals, the maximum site capacity achieved is 1590 veh/hr for two-way flow.

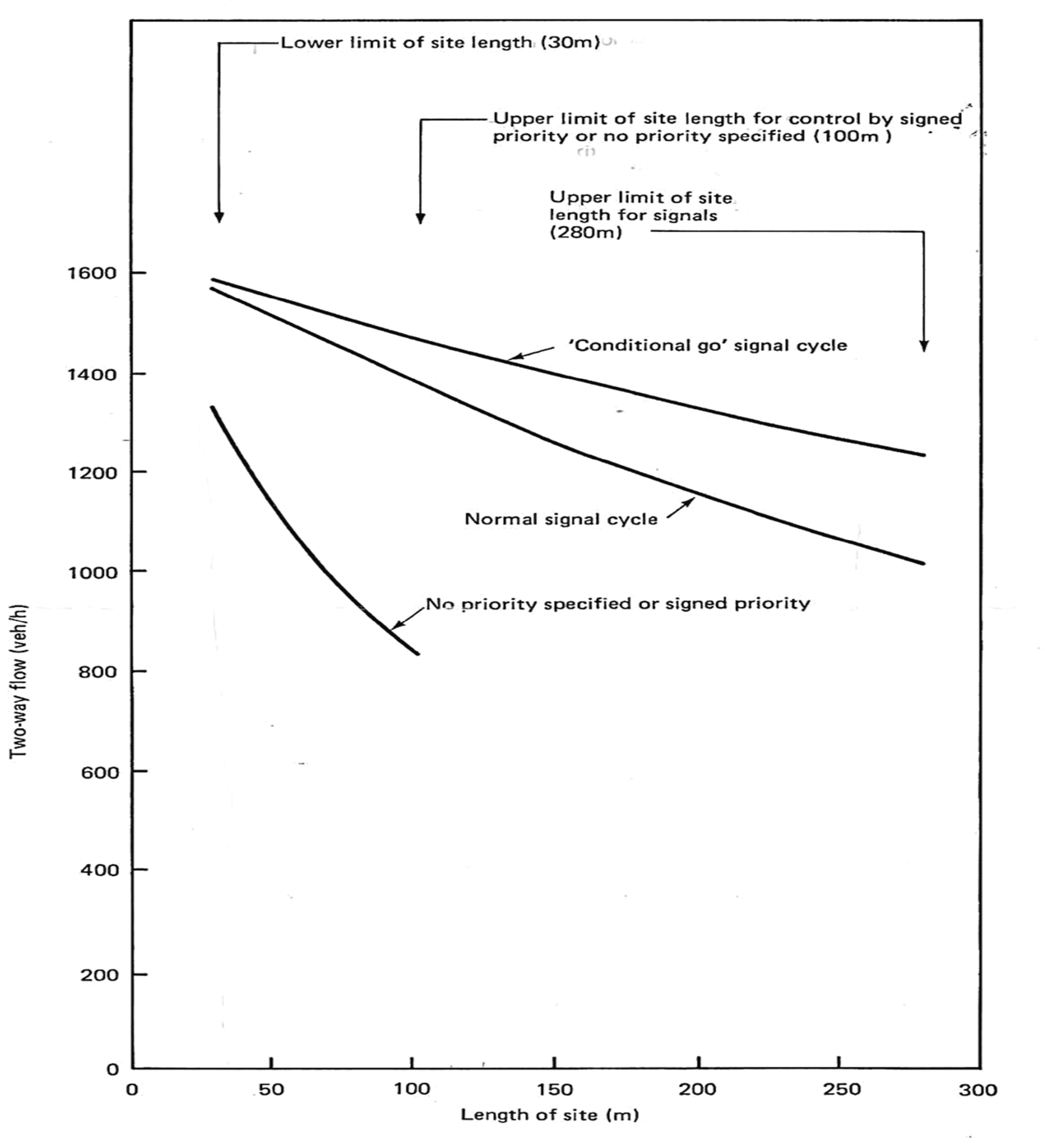


Fig. 9 The relationship between capacity and length of site

Figure 2.9: Relationship between site length and maximum capacity (Summersgill, 1981)

2.6.7 Increased delays

In Great Britain, delays at traffic signals are estimated to be 100 million vehicle-hours each year. If a saving of few percent was possible by using improved methods of operation, the financial savings each year would be considerable (Webster and Cobbe, 1966).

Delays and queues are the most important measures which determine the operational performance of roadworks. The overall delay due to shuttle-lane roadworks can be divided into two categories (Cassidy and Han, 1993):

1- Queuing delay;
2- Travel time delay.

The components of these delay categories are:

- ⇨ Delay according to the reduction in speed/capacity of shuttle-lane operation;
- ⇨ Waiting time delay according to red light;
- ⇨ Move-up delay following a stopping situation (e.g. traffic signals, queues);
- ⇨ Acceleration/deceleration delays.

Vehicle delay is the main parameter used to calculate the travel time cost of roadworks, which is then used for selecting optimum management and operational strategies.

2.6.8 Environmental effect

Traffic congestion (reduced speed and increased delays) contribute to a major part of the deteriorating urban air quality and pollution (Shefer, 1994). If urban roadworks are not designed appropriately, queues and delays are likely to form which could worsen air quality. More studies should therefore be carried out to show the effect of roadworks on urban air quality.

2.6.9 Safety effect

Safety is a very important aspect of the operational performance of shuttle-lane roadworks as it affects drivers, workers and pedestrians. The presence of roadworks and the associated traffic control devices, changes to road layout and congestion (which increase drivers' frustration leading to dangerous actions such as crossing the red light, unsafe gaps or close following "tailgating") are contributory factors to the high accident levels at roadworks sites.

A study carried out by the Highways Agency (2011) showed that around 83% of drivers change their behaviour through roadworks. Various reasons were given to the change in behaviour such as trying to avoid accidents, reading signs, narrow lanes, etc. The above changed behaviour is caused by the change in road layout especially to unfamiliar drivers, which could lead to accidents at roadworks. Based on SWOV (2010), it has been noticed that fewer accidents are recorded at longer duration and longer site length in the Netherlands.

Various accident reduction studies were carried out (Allpress and Leland Jr, 2010; Xing *et al*., 2010 and Elghamrawy, 2011) to study and analyse the factors influencing the cause of accidents at roadworks and suggesting methods and solutions to reduce the accident rates. SWOV (2010) mentioned that various measures could be taken to improve safety through roadworks such as demarcating the work area for traffic, guiding traffic through the roadworks site, making roadworks and workers visible to road users and simplifying the driving task.

Li and Bai (2009) carried out a study to test the effect of four shuttle-lane operation control methods on accident data using logistic regression analysis. It was found that flaggers/officers could considerably lower the odds of having severe accidents caused by human errors, while having stop signs/signals would dramatically increase the odds of having severe accidents. The study was based on limited data which was collected from the state of Kansas and will be unreliable to rule out those results on other states or countries.

Pilot study was carried out by the Isle of Man's Department of Infrastructure (2012) by installing mobile CCTV on urban roadworks to reduce the drivers' violation of crossing the red light on temporary traffic signals. If the pilot study is successful, it will be a requirement to install mobile CCTV cameras on all future roadworks.

2.6.9.1 Close following behaviour "tailgating"

Many studies, driving codes of practice and drivers training programmes (UK Highway Code, 2012; National Safety Council, 1992; Tennessee Department of Safety, 1991) state that a two second gap, referred to as the "two-seconds rule", is the minimum time gap for safe following on a dry road surface. On wet roads, the equivalent gap is increased to 4 seconds and could increase further for icy roads.

Various studies have reported that based on everyday driving experience in both urban and motorway environments, it has been noticed that many drivers attempt to follow with time headways significantly lower than two seconds. This is commonly referred to as “tailgating” (Michael *et al*., 2000; Brackstone *et al*., 2002 and Rajalin *et al*., 1997).

Tailgating is a very dangerous phenomenon of drivers’ behaviour and contributes to a high percentage of the road traffic accidents (mainly rear-end collisions). For example in China it contributes to nearly 16% of all road traffic accidents (Duan *et al*., 2013). According to Michael *et al*. (2000), tailgating contributed to 1,835 fatalities and 653,000 injuries in 1996 in the USA alone.

Considerable laboratory research using simulation techniques has investigated the factors associated with following distance and braking reaction time (e.g., Van Winsum and Heino, 1996; Van Winsum and Brouwer, 1997 and Evans *et al*., 2010). These studies examined how the drivers estimate time to collision and braking performance which are linked to the drivers' chosen headway.

According to Shrestha and Chang (2005), there are very few studies on close following “tailgating” with no standard criteria (clear definition) or effective system to observe and reduce it accordingly. The factors that influence tailgating behaviour can be grouped under three main categories: Driver’s Profile, Driver’s Behaviour and External Conditions. The parameters under driver’s profile include (but are not limited to) age, gender and intoxication. The parameters that are under driver’s behaviour include speeding, braking and maintaining minimum headway and the parameters under external conditions include traffic density, weather, speed limit, number of lanes, tyre and brake efficiency and enforcement.

2.6.9.2 Non-compliance with temporary traffic signals

Red light running violations at signalised junctions constitute a widespread and growing phenomenon which has a significant cost to society. In the USA, red light running contributes to around 260,000 crashes each year, of which about 750 are fatal. Red light running crashes were also found to be more severe than other types of crashes (Retting *et al*., 1998 and Retting *et al*., 1999).

A wide range of countermeasures has been studied and implemented to reduce red light running behaviour and its frequency. A study by Retting *et al*. (2007) has shown that both countermeasure categories (i.e. engineering and enforcement) are effective in reducing the frequency of red light violations. According to Bonneson and Zimmerman (2004), guidelines on which countermeasures (i.e. whether engineering or enforcement should be used) are scarce in identifying junctions with the potential for safety improvement.

Most of the available research is focused on the effect of implementing either engineering or enforcement countermeasures on signalised junctions using actual countermeasures on site or utilising micro-simulation approach (see for example Porter *et al*., 2013 and Bell *et al*., 2012). However, there is a clear lack of research on both drivers' behaviour and red light running violations on shuttle-lane urban roadworks operated by temporary traffic signals.

In the current study, driver compliance at temporary traffic signals with shuttle-lane operation has been observed to determine the factors that affect the drivers' decision to undertake such red light violations. There are some similarities between traffic signals at junctions and those temporary ones at shuttle-lane operations. One phenomenon that occurs at traffic signals junctions is what is referred to as the "dilemma zone".

According to previous research, the dilemma zone is defined as a physical zone (area) in advance of the junction's stop line where vehicles in the dilemma zone at the onset of amber indication can neither clear the junction during the amber interval nor safely stop before the stop line (Gazis *et al*., 1960). According to Puan and Ismail (2010), the dilemma zone is defined as an area near the stop line within which the driver finds him/herself too close to stop safely and yet too far away to pass completely through the intersection at a legal speed before the red phase starts. Any decision made by the driver may lead to an accident or near–accident. When traffic signals change from green to amber, approaching vehicles have two possible decisions from which to choose, either to stop or pass depending on various factors such as speed, distance from stop line, drivers' characteristics and other geometric layout (Hicks *et al*., 2005 and Papaioannou, 2007).

According to previous research (see for example Martin *et al*., 2003 and Elmitiny *et al*., 2010), it is suggested that a driver in a dilemma zone may run through red lights, come to an abrupt stop or accelerate through amber. This decision is considered the main risk of causing rear-end collisions (from vehicles following closely behind) and right angle collisions (from conflicting vehicles) at signalised junctions (Puan and Ismail, 2010; Gates *et al*., 2006 and Wei *et al*., 2010).

According to previous literature, there are two types of dilemma zone. The Type I dilemma zone (classic yellow time dilemma zone) was first explained by Gazis *et al*. (1960). This type is when the driver is unable to perform a safe and comfortable manoeuvre. In this case, a driver will either proceed through the intersection before the start of the red phase or stops in advance of the stop line. As originally described by Gazis *et al*. (1960), the yellow indication (yellow plus all red) is needed to be long enough for a vehicle to either stop or clear the intersection safely.

The Type II dilemma zone (also called option or indecision zone), which was first described by ITE Technical Committee 18 (1974), is associated with driver behaviour. Drivers within a few seconds travel time (usually between 2.5 and 5 seconds) from the intersection tend to be indecisive about their ability to stop at the onset of the yellow (amber) indication. According to Wei *et al.* (2010), option zone is defined as a zone within which at the onset of amber, the driver can either come to a safe stop or proceed through the intersection (before the end of amber interval). The word "option" means that the driver's final decision of whether to stop or to pass is optional and could be completed safely (without an abrupt stop or accelerating). This behaviour creates an indecision/option zone in advance of the stop line where some drivers may proceed and others may stop safely. Different definitions of Type II dilemma zone have been observed (see for example Zegeer and Deen, 1978 and Chang *et al*., 1985).

For the purpose of the current study, the term "dilemma zone" refers to both Type I and Type II dilemma zones since there is not enough data to allow the authors to identify the exact type of dilemma zone accurately. The detailed definitions, equations and all aspects of dilemma zone have been adapted from a typical signalised junction and applied to shuttle-lane roadworks using temporary traffic signals as illustrated in Figure 2.10.

According to Gazis *et al*. (1960), the dilemma zone is the difference between the distance from the stop line to the front of the nearest vehicle that can safely and completely stop (Pds/Sds) and the maximum distance from the stop line to the front of the nearest vehicle that can safely cross (Pdc/Sdc). Equations 2.2 to 2.4 show the calculation of the dilemma zone (DZ) as stated by many researchers (see for example Gazis *et al*., 1960; Puan and Ismail, 2010 and Wei *et al*., 2010).

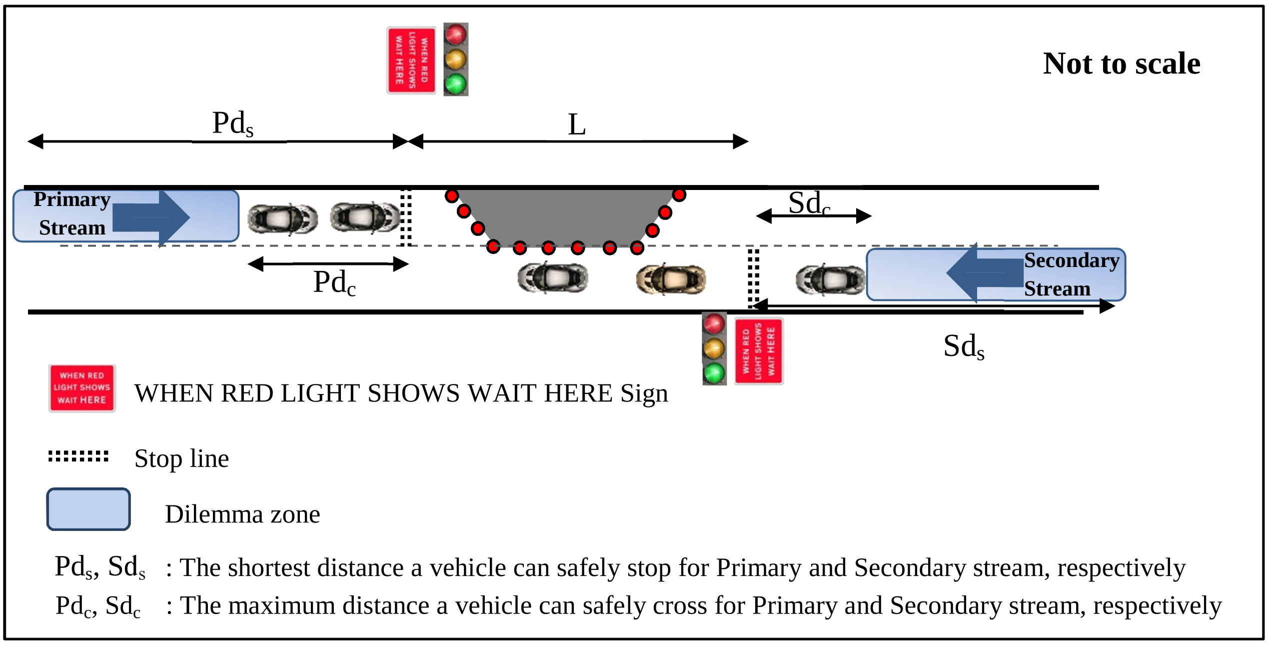


Figure 2.10: Dilemma zone at shuttle-lane roadworks

$$\text{Dilemma Zone (DZ)} = \text{ds} - \text{dc} \qquad \text{Equation 2.2}$$

$$\text{Stopping distance(ds)} = v\delta + \frac{v^2}{2\text{MaxDL}} \qquad \text{Equation 2.3}$$

$$\text{Clearing/crossing distance (dc)} = v\tau - (Lv + L) \qquad \text{Equation 2.4}$$

Where,

d_S is the shortest distance a vehicle can safely stop (metres)

d_C is the maximum distance a vehicle can safely cross (metres)

v is the speed of the vehicle (m/s)

δ is the perception reaction time of the driver (seconds)

MaxDL is the maximum comfortable deceleration rate (m/s^2)

τ is the length of the clearing period (amber plus all-red period) (seconds)

L_v is the length of the vehicle (metres)

L is the length of the roadworks site (metres)

According to ITE (1999), the method of calculating the amber and all-red period is shown in Equations 2.5 to 2.7.

$$\text{Length of clearing period } (\tau) = \text{Amb} + \text{AR} \qquad \text{Equation 2.5}$$

$$\text{Amb} = \delta + \frac{v_{85}}{2\text{MaxDL}} \qquad \text{Equation 2.6}$$

$$\text{AR} = \frac{L_v + L}{v_{15}} \qquad \text{Equation 2.7}$$

Where,

v_{85} is the 85th percentile speed of vehicles or the speed limit (m/s)

v_{15} is the 15th percentile speed of vehicles (m/s)

Amb is the stopping amber time (in seconds)

AR is the all red period (in seconds)

2.7 Traffic flow modelling

Various mathematical, simulation models and software packages were developed to estimate roadworks site capacity, queue length and delays to provide the following:

- ⇨ Comparison between different methods of operations;
- ⇨ Cost estimate for different roadworks management scenarios.

The following sections will provide an overview of the fundamentals of traffic flow theory, definitions of the different types of models and also an overview of the fundamentals of micro-simulation modelling.

2.7.1 Fundamentals of traffic flow theory

When studying traffic flow characteristics, there are two main types of traffic stream models, namely microscopic (micro-simulation) and macroscopic (mathematical and analytical) models. The microscopic traffic models deal with traffic characteristics of individual vehicles (i.e. headways, gaps and speed/position of each vehicle). On the other hand, macroscopic models are based on the fundamental diagrams of traffic flow that represents the dependent relationship between average values of flow (q), speed (v) and density (k), and their importance for the use in many practical fields of traffic engineering studies. Equation 2.8 shows the relationship between the main elements of traffic characteristics of speed, flow and density (Mannering and Washburn, 2012).

$$q = vk \qquad \text{Equation 2.8}$$

Where,

q is the traffic flow (vehicle/hour);

v is the traffic speed (km/hour);

k is the traffic density (vehicle/km);

The first study to model the relationship between these parameters was carried out by Greenshields (1935). This was followed by several studies proposing modifications and adapting various versions of some of the main proposed models (see for example Heydecker and Addison, 2011; Zhang and Jin, 2002). Figure 2.11 shows a simple general layout of the relationship between speed-density, flow-density and speed-flow elements of the traffic assuming linear speed-density model.

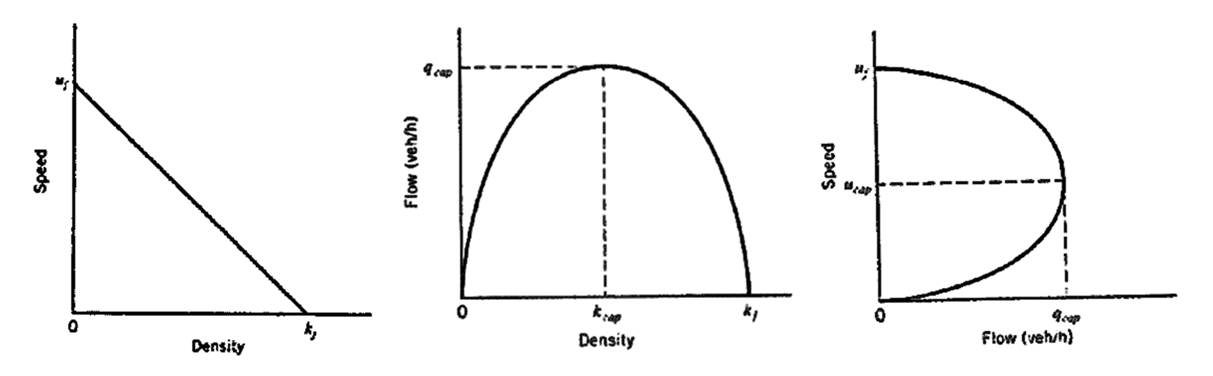


Figure 2.11: General layout of speed-density, flow-density and speed-flow relationships (Mannering and Washburn, 2012)

The relationships between these main elements of traffic flow (speed, flow and density) provide the basis for the measurement and calculation of traffic stream parameters. It is also important to understand the interaction between those measures to fully analyse the operational performance of a traffic stream.

2.7.2 Definitions of different types of models

According to Perlman (2008), mathematical models are models that apply concepts or theoretical principles to represent the behaviour of a system (to solve/explain the targeted problems/phenomena). Mathematical model deals with the system as a whole (i.e. average value of the traffic stream in hourly basis) which is often based on fluid flow analogy. Mathematical models also are not dynamic (i.e. do not take into account live interaction between drivers and the system components such as dilemma zone, sudden change in acceleration/deceleration, vehicle actuated signals, effect of traffic management safety devices at shuttle-lane roadworks, change in visibility of drivers, etc.).

For example, an input to a mathematical model could assume that vehicles arriving on the primary stream (e.g. 500 vehicle/hour) will have a fixed total green of 1,200 seconds in an hour for example. The equations will then calculate the throughput based on those assumptions without taking into account the interaction of the signal setting, VA, dilemma zone effect, etc. Therefore, mathematical models were not deemed to be fit for purpose when studying the operational effects of shuttle-lane roadworks under varying conditions such as, for example, vehicle actuated signals and differing layout conditions. A summary of the existing mathematical models for shuttle-lane roadworks are provided in the following sections.

According to Akcelik (2007), analytical models are software packages that were developed (by companies or institutions) using simplified mathematical equations and have a closed form solution (i.e. the solution to the equations used to describe changes in a system can be expressed as a mathematical analytic function). These models utilise various techniques such as regression analysis and simplified mathematical equations.

The end-user may not have access to the software code and therefore the majority of assumptions were kept constant. Analytical models also provide average hourly values of the system (e.g. queues and delays) with no option for measuring the effect of the dynamic interaction between drivers (e.g. signal setting, VA, dilemma zone effect, etc.). Therefore, analytical models were also deemed to be not fit for purpose when studying the detailed operational effect of, for example, dilemma zone on shuttle-lane roadworks. A summary of the existing shuttle-lane analytical software (models) are shown in the following sections.

Microscopic (micro-simulation) models describe the traffic at a detailed level where specific rules (sub-models) are used and applied to represent the interaction between individual vehicles such as:

- ⇨ Car-following rule;
- ⇨ Lane-changing rule;
- ⇨ Gap acceptance rule.

Car-following sub-model calculates the acceleration/deceleration rates used in updating the longitudinal positions of vehicles in correspondence to the leader. Lane- changing sub-model describes the lateral movements of vehicles based on traffic conditions in the current and the target lanes. The gap acceptance sub-model is used to check the feasibility of executing a lane change (Al-Obaedi, 2012). In the case of shuttle lane roadworks operated by temporary traffic signals, no lane-changing is allowed and consequently there is no need for a gap acceptance rule to be applied. For every scanning time (e.g. 0.5 seconds), each vehicle in the system checks the feasibility of executing all these rules based on the vehicles' situation and each rule (sub-model) consists of detailed decision tree. Detailed description of the listed rules (i.e. decision tree for each rule) which are used in the development of the micro-simulation model is explained in Chapter 6.

The randomness used in micro-simulation models may help in replicating real traffic conditions. The calibration process is not as straightforward as in other high level models (i.e. macroscopic models). Micro-simulation models are more efficient in studying complicated situations such as merge sections, inclusion of ITS and vehicle actuated traffic lights (Burghout, 2004; Akcelik, 2007). Micro-simulation models could also replicate the geometry of the road even if the traffic management used on site is complex.

Based on the above mentioned limitations of mathematical and analytical models, micro-simulation models were found to be the most appropriate type of models that are fit for purpose when studying shuttle-lane operations at roadworks with various traffic control settings and other varying factors on site (including drivers' behavior, dilemma zone effect and road geometry). Micro-simulation models are capable of dealing with individual vehicles' movements and drivers' decisions in crossing dilemma zone, interaction with changes to traffic signals, response to loop detectors, etc. A summary of the existing shuttle-lane roadworks mathematical, analytical and simulation models are shown in the following sections.

2.7.3 Existing shuttle-lane mathematical models and their limitations

Various mathematical models have been developed to study the effects of shuttle-lane roadworks capacity on user delays, queues and cost. Table 2.9 summarises the main mathematical models, the parameters used and their limitations. These models are described in the following section.

2.7.3.1 Summary of existing mathematical models

Cassidy and Han (1993) developed a mathematical model to calculate traffic delay and queue length per cycle on a one-lane road with a two-way control work zone (shuttle-lane roadworks) under temporary traffic signals. Delays were divided into two components, namely queuing delays and travel time delays. The model was then compared with observed data and showed a relatively close fit to observed data under steady state conditions. Schonfeld and Chien (1999) developed a mathematical user delay model to calculate the optimum work zone length and cycle time taking into account different variables such as traffic volume, speeds, maximum discharge rate and setup costs.

Chien *et al*. (2002) improved the model by adding maintenance cost idling to the overall cost and adding maintenance breaks to avoid peak traffic which would help in optimising work zone length and cycle time. According to Huang and Shi (2008), the model does not address the practicality of maintenance breaks (how to withdraw workers and materials during or before rush hour).

Huang and Shi (2008) developed a mathematical model to calculate users delay and total roadworks cost using objective function to optimise the work zone length and maintenance schedule. They also added delays caused by the presence of non-motorised users (e.g. vehicles travelling behind cyclists).

Table 2.9: List of the main shuttle-lane roadworks mathematical models

Model	Purpose	Comments
Cassidy and Han (1993)	Delay model	Only applicable for under-saturated conditions.
Schonfeld and Chien (1999)	Optimisation model. Work zone length and traffic control	Only applicable for under-saturated conditions; Does not include the effect of acceleration and deceleration; No observed data, calibration or validation.
Chien *et al*. (2002)	Optimisation model. Work zone length and cycle time	Only applicable for under-saturated conditions; Does not include the effect of acceleration and deceleration; No observed data, calibration or validation.
Huang and Shi (2008)	Optimisation model. Work zone length and starting time	Does not include the effect of acceleration and deceleration; No observed data, calibration or validation.

2.7.3.2 Limitations of existing mathematical models

The main limitations of the existing mathematical models are as follows:

- ⇨ The determination of the adjustment parameters could be complicated, especially with complicated control methods (Edara and Cottrell, 2007);
- ⇨ Using deterministic queuing analysis by simplified assumptions, to estimate the user delays may neglect important details (Yang *et al*., 2009).
- ⇨ Cassidy and Han (1993) model is not valid for oversaturated conditions which are the current situation in most roadworks sites. The equations for estimating delays sometimes are not equivalent to the real observed delays (Son *et al*., 1999).

- ⇨ Schonfeld and Chien (1999) model is also not applicable to oversaturated conditions (Huang and Shi, 2008). Also there was no model calibration and validation using real observed data.
- ⇨ Chien *et al*. (2002) model did not take into account the effect of drivers' acceleration and deceleration when approaching and passing through the work zone. There was no model calibration and validation of real data. Also the model is only capable of calculating the delay for under-saturated situation where traffic flow level does not exceed the site capacity.
- ⇨ Huang and Shi (2008) model has several limitations as the assumptions used do not reflect the real situation (i.e.no effect of acceleration or deceleration) and traffic have to use two constant speed values only. There is no observed real data and therefore, no calibration or validation process which does not provide the goodness of fit of the calculated values.

2.7.4 Existing analytical software packages and their limitations

Various computer software packages were developed using analytical methods which utilise an equation or a series of equations to calculate the roadworks site capacity, user delays, queue length and cost (Ramezani *et al*., 2011). The most popular software packages are summarised in the following section and listed in Table 2.10.

2.7.4.1 Summary of existing analytical software packages

QUADRO (QUeues and Delays at ROadworks) is a sponsored Department for Transport (United Kingdom) software package, which is used to evaluate road maintenance work by calculating user delay costs, vehicle operating costs and accident costs. The shuttle-lane roadworks section in QUADRO (part 5, volume 14, section 1 and chapter 10) provides equations used to calculate site capacity and therefore associated user delay and queues. QUADRO assumes that shuttle-lane is operating under temporary signals with fixed settings and uses the site length as the main parameter.

QUEWZ (Queue and User cost Evaluation of Work Zone) was developed by the Texas Transportation Institute in the late 1990s. QUEWZ is a menu-driven program which can be run under DOS. The latest QUEWZ model uses HCM 2000 equations to calculate site capacity, user delays, queues and vehicle operating cost. The model does not provide accurate delays and queues estimation compared with real data (Ramezani *et al*., 2011).

QUICKZONE model was developed by the Federal Highways Authority. The model is a spreadsheet-based tool which uses the deterministic queuing theory to calculate queues and delays using input parameters such as traffic flow, site capacity and site length (Tang, 2008). The model also takes into account travelers responses to the roadworks traffic conditions such as mode shift, peak spreading, etc. (Edara and Cottrell, 2007).

WZCAT (Work Zone Capacity Analysis Tool) model was developed recently by the Wisconsin Department of Transportation. The model is a spreadsheet-based software package used to estimate delays and queues in short term roadworks (e.g. daily). The capacity is estimated using HCM 2000 values.

Table 2.10: List of shuttle-lane roadworks software packages

Model	Purpose	Comments
QUADRO (2004)	Analytical model-Provides total cost for road maintenance work and accident cost	Uses text file as input; Shuttle-lane roadworks queues and delays are based on study carried out by Summersgill (1981); Uses regression analysis; Does not take into account VA or other control methods; Very simple representation of SF curves.
QUEWZ (1998)	Analytical model-Calculates user cost including user delay costs and vehicle operating costs	No work zone optimisation capability; Capacity and queues does not match field data; Very simple representation of SF curves.
QUICKZONE (1998)	Analytical model-Spreadsheet-based tool	No work zone optimisation capability; Does not take into account delays due to lower speeds.
WZCAT and WZCAT-R (2008)	Analytical model-spreadsheet-based tool – to calculate queue length and delays	Short-term work zone (daily); Capacity model is not adequate; Does not take into account traffic control methods such as flaggers or ITS.
INTELLIZONE, (2004)	Analytical model	Capacity model is not adequate; Does not take into account traffic control methods such as flaggers or ITS.

2.7.4.2 Limitations of existing analytical software packages

The main limitations of the existing analytical models are:

- ⇨ The assumptions in QUADRO are very simple and do not take into account very important factors which affect shuttle-lane operation (i.e. HGV%, directional split);

- ⇨ QUEWZ model does not accurately estimate queues and delays when compared with real data (Ramezani *et al*., 2011). Determining adjustment factors could be a complicated task (Edara and Cottrell, 2007).
- ⇨ QUICKZONE, WZCAT and INTELLIZONE models do not accurately estimate queues and delays when compared with real data (Ramezani *et al*., 2011). QUICKZONE requires detailed coding of the road network by the user for both the roadwork and alternative routes (Edara and Cottrell, 2007).

2.7.5 Existing simulation models and their limitations

Various micro-simulation models have been developed to study the effects of shuttle-lane roadworks and to optimise various parameters to achieve minimum delay costs. Table 2.11 summarises the main simulation models, the parameters used and their limitations. These models are described in the following section.

2.7.5.1 Summary of existing simulation models

Summersgill (1981) developed a micro-simulation model to calculate shuttle-lane roadworks site capacity, delays and queues for three traffic control methods and various site lengths (up to 300 metres). Summersgill also studied an improved method of traffic control by the use of all-red period when there is no traffic crossing the site. The study sets the basics for estimating delays and queues, and the outcomes were used by the Department for Transport software QUADRO (which is discussed in the previous sections) to estimate vehicle delays at shuttle-lane roadworks.

Cassidy *et al*. (1994) employed Monte-Carlo simulation technique and approximate analysis techniques to calculate delay distribution. Deficiencies were found with the use of Monte-Carlo simulation techniques in dealing with the use of random vehicle arrival and variability in headway discharge (Cassidy *et al*., 1994).

Son *et al*. (1995) evaluated the appropriateness of the steady-state assumptions in calculating delays at shuttle-lane roadworks through the development of a simulation model. The study suggests that this method does not always provide reasonable results especially when demand increases.

Son (1999) developed a simulation model to evaluate queuing delay models for shuttle-lane roadworks. The models were derived from Newell's (1969) delay equation for intersections of one-way streets operated by VA signals. The model shows a close representation when compared with real data. The delay models were only tested with a flow of up to 600 vehicles per hour for both directions.

Ebben *et al*. (2004) developed a simulation model to study the Automatic Guided Vehicles (AGVs) system which uses single lane traffic for both directions to reduce infrastructure cost. The system is used to carry underground freight in Schiphol Airport in the Netherlands. The study tested an advanced traffic responsive system to reduce the waiting times. The study shows that when using the advanced control methods, the waiting times can be reduced by 10-25% compared with the standard operation methods.

Al-Kaisy and Kerestes (2006) carried out a study to evaluate four shuttle-lane roadworks control strategies including fixed-time control, fixed-queue control, convoy rule and adaptive control. The model consists of two parts, a spreadsheet deterministic approach which feeds into a simulation model (using Synchro and SimTraffic). Various variables were taken into account such as site length, speed through site, lost time and travel time. The study shows that significant delay reductions can be achieved through advanced traffic control techniques and appropriate flaggers training can result in highly efficient traffic control operations.

CORSIM simulation model was developed by the FHWA (United States Federal Highway Authority) and has two components (NETSIM and FRESIM). Road network can be coded in CORSIM as nodes and links. Roadworks can be coded as incidents as there is no direct option of modelling roadworks (Benekohal *et al*., 2003; Ramezani and Benekohal, 2011 and Bloomberg and Dale, 2000).

VISSIM is a microscopic, stochastic, discrete time-step based simulation where individual vehicles represent the most basic elements of the simulation. VISSIM is based on the Wiedemann “psycho-physical” car-following and lane changing model. The characteristics and behaviour of individual vehicles affect the performance measures such as queue length, speed and throughput (Edara and Cottrell, 2007). Roadworks can be coded in VISSIM as incidents or by the use of complicated sets of signals and detectors as there is no direct option of modelling shuttle-lane roadworks.

Table 2.11: List of the main shuttle-lane roadworks simulation models

Model	Purpose	Comments
Summersgill (1981)	Control methods, operation, capacity and delays	Uses steady state delay equations; Does not include the effect of different HGVs percentage or directional split; Site length only up to 300 metres; Old VA signals specifications; No information about model calibration or validation; Only 3 control methods were tested.
Cassidy *et al*. (1994)	Monte-Carlo simulation to calculate average delay	Only applicable for under-saturated conditions; Does not include the effect of different variables; Deficiencies associated with the model.
Son *et al*. (1995)	Micro-simulation model	Only testing the appropriateness of the steady state assumption for the shuttle-lane operation with no operation scenarios tested.
Son (1999)	Simulation model to calculate queuing delay	Long site length (starts from 0.75 miles); Low traffic levels were tested (up to 600 vehicles per hour).
Ebben *et al*. (2004)	Simulation model to calculate waiting time savings	Acceleration and deceleration are instant.
Al-Kaisy and Kerestes (2006)	Simulation model to calculate delay savings	Only applicable for undersaturated conditions; Long site length (starts from 1km); Vehicle Actuated (VA) signals was not tested; No information about model calibration or validation; Four control methods were tested.
VISSIM, S-Paramics	Micro-simulation package	No direct option of coding roadworks; The replication of shuttle-lane operation is complicated.
CORSIM	Micro-simulation package	FRESIM-Do not have the capability of directly returning queue length; May not readily adapt work zone; Capacity and queues does not match field data.

The S-Paramics is a micro-simulation software package which is capable of representing the behaviour and interaction between individual drivers on a road network. Different road layouts and features could be simulated and some drivers' behaviour characteristics (parameters) can be changed relatively easily as part of the calibration and validation of the model to replicate actual site observations. Roadworks can be coded in S-Paramics as incidents or by the use of complicated sets of signals and detectors as there is no direct option of modelling shuttle-lane roadworks.

2.7.5.2 Limitations of existing simulation models

The main limitations of the existing simulation models are:

- The model developed by Summersgill (1981) is only applicable to under-saturated situations where traffic is less than the site capacity. Summersgill used the VA specification that was available at that time and recent specifications are being developed which could affect the modelling results. Site lengths of up to 300 metres only were tested (Summersgill, 1981).
- Monte-Carlo simulation was utilised by Cassidy *et al*. (1994) and various deficiencies were found (Cassidy *et al*., 1994).
- Son *et al*. (1995) model only evaluates the steady state assumption (for under-saturated situation) without reference to the evaluation method for different traffic control techniques.
- Son's (1999) model only tested long site lengths (starting from 0.75 mile) and low traffic volume (up to 600 vehicles for both directions).
- Ebben *et al*. (2004) simulation model tested the AGVs system. The system has major differences to actual vehicles characteristics such as constant speed, instant acceleration and decelerations.
- Al-Kaisy and Kerestes' (2006) model is applicable to long site length (starts from 1 km) and for under-saturated situations. Only four control methods were tested (not including VA signals) and there was no calibration or validation for the modelling output.
- CORSIM model requires the calibration of several variables that are difficult to measure (Crowther, 2001). CORSIM was not designed to model the effects of work zones on maintenance of traffic where the model shows a significant underestimation or overestimation of the queue length (Schnell *et al*., 2002).
- VISSIM and S-Paramics do not have the ability of modelling roadworks directly. The operation of shuttle-lane needs to be replicated by the use of incidents or by the use of a complicated set of signals and the use of detectors which require highly skilled users. Another limitation in VISSIM is the cancellation of vehicles that reach their maximum waiting time and failed to reach their destination (PTV Vision, 2011).

2.8 Summary

The current chapter can be summarised by the following main points in relation to urban shuttle-lane roadworks based on previous limitations from the literature::

- ⇨ Various discrepancies and limitations were found from the design guidelines such as site length, maximum allowed flow levels for each method of traffic operation. Important factors were not taken into account when selecting or designing each traffic control method (i.e. HGVs %, directional split and other parameters).
- ⇨ Mathematical models are inadequate in accurately modelling shuttle-lane roadworks with the limitation of correctly replicating queues and delays. They also lack the comparison with real observed data and the inability to model the effect of any advanced traffic control techniques.
- ⇨ Analytical models were proven to be inadequate in estimating shuttle-lane roadworks capacity, delays and queues, because of their inability to model several roadworks features such as the presence of flaggers, ITSs, Vehicle Actuated traffic signals.
- ⇨ Simulation models are designed for under-saturated conditions. The models also have various limitations such as omitting vehicles, various parameters are imbedded within the program code that the user does not have access to, and the required level of complicated steps to ensure correct behaviour of such a system.
- ⇨ In addition to the previously described limitations from available literature, none of the existing models takes into account certain aspects of aggressive drivers' behaviour (i.e. close following "tailgating" and red light running) which may have an impact on site safety and capacity.

Therefore, there is a need to carry out data collection on shuttle-lane roadworks sites to observe aggressive drivers' behaviour (i.e. close following "tailgating" and red light running). The data will then be used to develop a micro-simulation model to cover the previously described limitations in existing models and design manuals. Detailed description of the data collection and model development are explained in the following chapters.

CHAPTER THREE: DATA COLLECTION AND DESCRIPTION

3.1 Introduction

The aim of this chapter is to give a brief description of the available data collection methods and the difficult issues related to data collection. It also summarises the details of the visited sites such as location maps, layouts, collected data, etc. Data was collected and analysed for various shuttle-lane roadworks sites to understand and observe traffic and drivers' behaviour at shuttle-lane roadworks. The data was also used to provide the necessary input for developing, calibrating and validating the micro-simulation model.

3.2 Data collection techniques

Video recording was the main type of data collection technique used to capture traffic information at urban shuttle-lane roadworks sites. Various difficulties were experienced during the data collection stage, which are summarised below:

- ⇨ Unavailability of shuttle-lane roadworks sites due to their temporary nature;
- ⇨ Lack of proposed/current roadworks information from relevant government agencies;
- ⇨ Short duration of planning time for the available sites;
- ⇨ Lack of safe/appropriate position for recording;
- ⇨ Difficulties associated with adverse weather conditions;
- ⇨ Short roadworks duration or late knowledge of roadworks site;
- ⇨ Unavailability of cameras and other personnel.

During recent decades, several methods of data collection have been developed. However, collecting traffic data using video cameras is still the main method for academic research purposes in the UK because of the reasonably low cost involved. Furthermore, video recording systems are the best tools for investigating certain traffic characteristics such as drivers' behaviour, drivers' compliance with temporary traffic signals at roadworks, gap acceptance, temporary signals settings, etc. In addition, other advantages have been reported by Yousif (1993) such as:

- ⇨ One person is capable of collecting information (low personnel requirements);
- ⇨ Any comments on events outside the field of the camera can be reported through the recording system;

- The relatively simple and quick setup of the video recording system;
- The ability to view the recorded data continuously to extract the required information.

As a result of the previously mentioned reasons, the most effective technique for capturing traffic information for this research was the use of video recordings. These took place using two video cameras (Sony HDD Handycam DCR-SR57) at each side of the selected shuttle-lane roadworks as illustrated in Figure 3.1. Synchronisation between the cameras was carried out using stop watches and mobile phone communications to ensure accuracy.

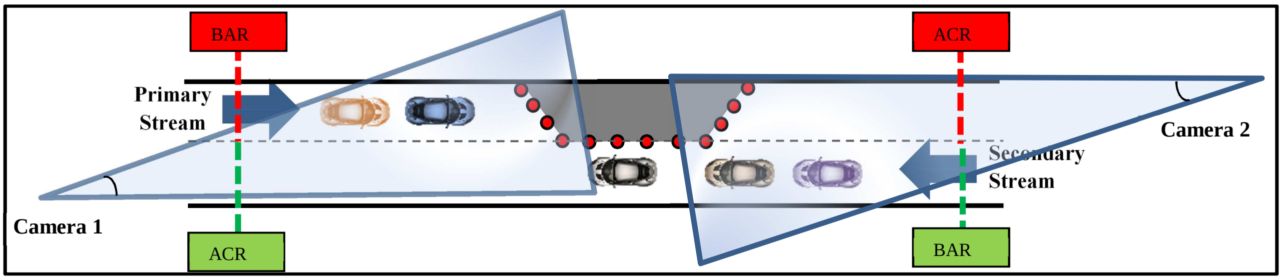


Figure 3.1: Schematic of the location of video cameras

In order to ensure that drivers' behaviours were not influenced by the presence of video cameras, the positioning and locations of video cameras were carefully considered for each site. For example, one camera was located inside the observer's parked vehicle at one side of the shuttle (e.g. one observer filming the primary stream) and the other camera was located in a safe hiding place such as a shop frontage, inside a building or unobstructed footpath, etc. on the other side of a shuttle-lane (e.g. a second observer capturing information from the secondary stream).

Some sites were visited more than once, at different times and over several days (especially for those roadworks sites which were carried out over several days and to ensure that weather conditions were not that adverse to affect the surveys) to capture different types of information and to increase the available sample size for drivers' behaviours at different flow levels. Both primary and secondary streams were captured on both cameras as illustrated in Figure 3.1and explained below:

- *Primary* (or *Secondary*) stream (BAR): primary (or secondary) stream vehicles are captured by camera Before Approaching Roadworks (BAR).
- *Primary* (or *Secondary*) stream (ACR): primary (or secondary) stream vehicles are captured by camera After Crossing Roadworks (ACR).

3.3 Site selection

Due to the temporary nature of shuttle-lane roadworks in urban areas and the experienced difficulties that were previously mentioned (i.e. in Section 3.2), it was decided to visit any available shuttle-lane roadworks sites and is accessible for surveying. Six surveys/data collection categories were used in the current study and are described in the following sections.

3.3.1 Category 1: Historical shuttle-lane roadworks sites

Historical sites were previously surveyed between the years of 1996 and 2002. The data was collected by MSc students at the University of Salford, who investigated the operation of shuttle-lane roadworks. The data was very useful for the current study, it covers temporary traffic signals using both FT and VA modes and it also covers standard signalised junction. The data covers one stream only (i.e. primary or secondary) for seven sites in Greater Manchester as shown in Table 3.1 (with 20 hours of video recording) and a location map is shown in Figure 3.2. Please note that sites 8, 9 and 10 are traffic calming sites (using throttle) and were not used in the current study.

Table 3.1: Summary of site list and collected data (historical sites)

Site No.	Site	Date (Duration)	Duration	Surveyed Direction	Site Length (metres)	Type
1	Great Cheetham St. East, Salford	(a) 09.07.02	2 hrs (AM)	NA	NA	TS
2	Agecroft Rd, Salford	(a) 12.07.02 (b) 21.07.02	3 hr (AM)	P	114	Rd/FT
3	Europa Boulevard Road, Warrington	(a) 01.09.02	2.5 hrs (AM)	S	147	Rd/VA
4	Wilmslow Rd, Cheadle	(a) 24.04.97	2.5 hrs (AM)	P	46	Rd/VA
5	Cromwell Grove, Levenshulme	(a) 16.09.96	3 hrs (PM)	P	137	Rd/FT
6	Wilmslow Rd, Whithington	(a) 26.07.96	3 hrs (PM)	P	34	Rd/VA
7	Liverpool Rd., Irlam	(a) 28.02.96 (b) 10.03.96	4 hrs (PM)	P	72	Rd/VA

P: Primary stream **S:** Secondary stream **Rd:** Roadworks **VA:** Vehicle Actuated signals
FT: Fixed Time signals **TS:** Traffic signals junction

3.3.2 Category 2: Current shuttle-lane roadworks sites (full surveys)

Current sites were surveyed in 2012 (during the period of the current research study). The data covers different types of shuttle-lane operations (i.e. temporary traffic signals using both FT and VA modes, priority operation and Give/Take operation) and it also covers standard signalised junction. The data covers both streams (i.e. primary and secondary) for nine sites as shown in Table 3.2 (with 34 hours of video recording). Please see Figure 3.2 for sites locations and Appendix A for site plans, pictures, dimensions, etc.

Table 3.2: Summary of site list and collected data (current sites)

Site No.	Site	Date (Duration)	Duration	Surveyed Direction	Site Length (metres)	Type
11	Broughton Ln., Salford	(a) 09.07.12 (b) 10.07.12 (c) 13.09.12	7 hrs (AM &PM)	P, S	42	Rd/VA
12	Burton Rd., Chorlton	(a) 11.07.12	2 hrs (AM)	P, S	107	Rd/VA
13	Brunswick St., Manchester	(a) 12.07.12	2 hrs (AM)	P	8	Pri
14	Liverpool St., Salford	(a) 17.07.12 (b)17.09.12	4.5 hrs (AM &PM)	S	17	G/T
15	Langworthy Rd., Salford	(a) 18.07.12	2 hrs (AM)	NA	NA	TS
16	High Ln, Manchester	(a) 04.09.12 (b) 05.09.12 (c) 08.11.12	4.5 hrs (AM &PM)	P, S	52	Rd/FT
17	New Blackley Rd., Manchester	(a) 06.09.12	2 hrs (AM)	P, S	39	Rd/VA
18	New Blackley Rd., Manchester	(a) 17.09.12	3 hrs (PM)	P, S	73	Rd/VA
19	Frederick Rd., Salford	(a) 27.09.12 (b) 02.10.12 (c) 05.11.12	7 hrs (AM)	P, S	38	Rd/FT

P: Primary stream **S**: Secondary stream **Rd**: Roadworks
VA: Vehicle Actuated signals **FT**: Fixed Time signals **Pri**: Priority operated shuttle-lane
G/T: Give and Take operated shuttle-lane **TS**: Traffic signals junction **NA**: Not Applicable (not a roadworks site)
Note: Any reference at later sections/chapters to any site without letters (e.g. a, b or c) means all observations are taken across all visits (i.e. representing the combined total of a, b and c)

3.3.3 Category 3: Current shuttle-lane roadworks sites (partial surveys)

Partial shuttle-lane roadworks sites surveys were undertaken on two sites in Greater Manchester as summarised in Table 3.3 and the location map shown in Figure 3.2. The sites dimensions, layout and pictures were collected and are shown in full details in Appendix A. No videos were recorded due to the various reasons/difficulties mentioned earlier (i.e. no suitable/safe locations to take video recording from, difficulties with adverse weather or lack of available equipment/personnel) and also due to very low traffic flows at certain site (less than 50 veh/hr). However, data from these sites were used to study signage, traffic management setup, etc.

3.3.4 Category 4: Current post-removal of shuttle-lane roadworks sites

Surveys were also carried out on some of the previously surveyed sites (sites listed in Table 3.2) but post-removal of the roadworks (without the effect of roadworks) to obtain the arrival distribution of vehicles without the impact of vehicles re-routing or stopping due to the presence of shuttle-lane roadworks. The data covers both directions for Sites 11, 16 and 19.

Table 3.3: Summary of partially surveyed sites

Site No.	Site	Date (Duration)	Surveyed Direction	Site Length (metres)	Type
20	Manchester Road, Swinton	(a) 12.12.11	P, S	67	Pri
21	Silk street, Salford	(a) 17.07.12	P, S	19	G/T
22	University Road, Salford	(a) 21.12.12	P, S	79	Rd/FT
23	University Road West, Salford	(a) 21.12.12	P, S	68	Rd/FT

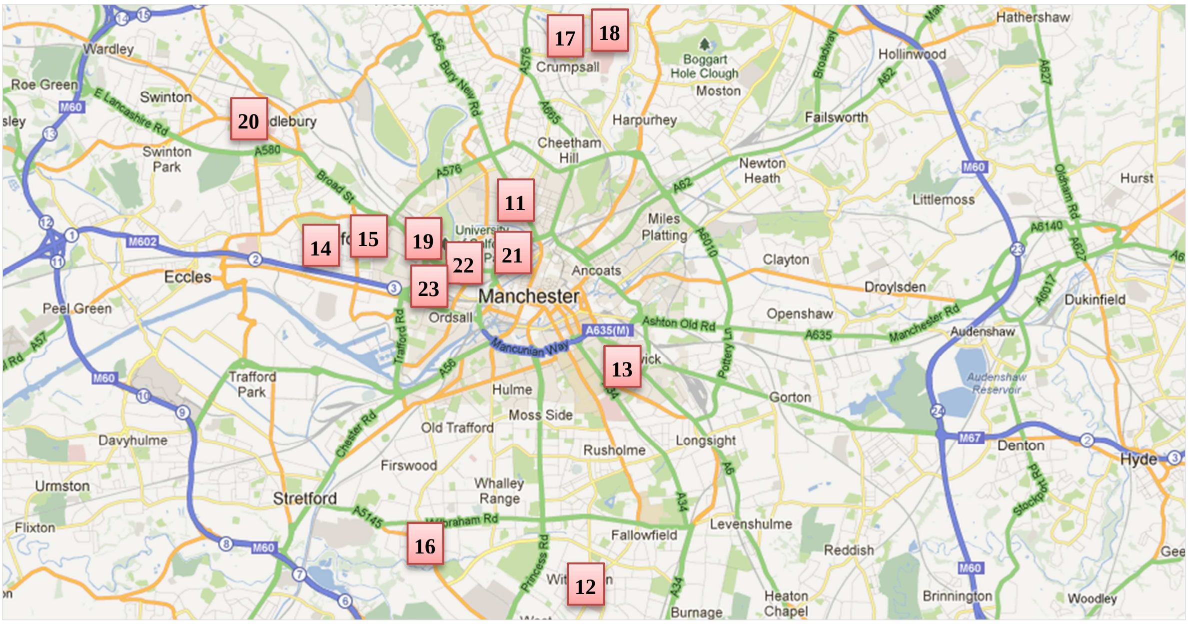


Figure 3.2: Map of site locations (categories 2 to 5)

3.3.5 Category 5: Typical signalised junctions sites

Two typical signalised junctions were surveyed (with one-lane approach on the surveyed arms). The surveys were obtained from historical data as mentioned in Section 3.3.1 and also during the current research study as mentioned in Section 3.3.2. The data covers four hours of video recording in the area of Greater Manchester as summarised in Tables 3.1 and 3.2.

The data analysed from typical signalised junctions is described in Section 3.4 and will be compared with the same data collected from temporary traffic lights at shuttle-lane roadworks.

3.3.6 Category 6: other data (Individual Vehicle Data-IVD)

Individual Vehicle Data (IVD) was obtained from the Highways Agency, which contains headway data between vehicles, the length and speed for each individual vehicle. The data collected over several days in March and May in 2002 for the M25 and in August and September for the M42 in 2002 and covers over 5.3 million vehicles records.

The lengths of cars obtained from IVD were used to build the distribution of car length which will be used as an input into the developed simulation model. It is assumed that the distribution of car length on motorways will be the same distribution as urban environment (cars will use the urban road network as part of their daily travel pattern). Full details of the data and the detailed analysis are presented in Chapter 4.

3.4 Description of collected data

Various types of data were collected from the surveyed sites. The data is summarised under four main headings as shown below and also summarised for each site in Table 3.4. A detailed description of the methods used for data extraction and the definition of each type is provided below and a full analysis is presented in Chapter 4.

- ⇨ **Traffic characteristics**: arrival traffic flow, traffic composition, time headway, site throughput, directional split, MUT (Move-up Time), MUD (Move-up Delay) and queues.
- ⇨ **Roadworks site characteristics**: site length, operation type and signage.
- ⇨ **Drivers’ behaviours**: close following “tailgating”, amber crossing and red light violations.
- ⇨ **Signals settings**: signals timing (i.e. green time and all-red period), signals type (i.e. FT and VA).

3.4.1 Flow level, profile and composition

Traffic flow information (i.e. flow by vehicle type, directional split and HGVs percentage) was collected for each site and each direction separately for each 5-minutes interval in order to create accurate flow profiles. Vehicles in each traffic stream were classified into two vehicle types (i.e. cars and HGVs). Bicycles and motorcycles were ignored, because their number was negligible. Observed flow levels, profiles and composition will be used as input into the developed micro-simulation model.

Table 3.4: Summary of studied parameters through site categories

Site Selection				
Historical shuttle-lane roadworks sites	**Current shuttle-lane roadworks sites (full surveys)**	**Current shuttle-lane roadworks sites (partial surveys)**	**Post-removal of shuttle-lane roadworks sites**	**Normal signalised intersections**
⇨ MUT ⇨ Signals compliance ⇨ Signage	⇨ Flow levels ⇨ Following time headway ⇨ MUT ⇨ MUD ⇨ Signals Timings ⇨ Queuing ⇨ Drivers behaviour ⇨ Signals compliance ⇨ Signage	⇨ Site dimensions ⇨ Signage	⇨ Arrival headway distribution	⇨ MUT ⇨ Signals compliance
(Sites 2 to 7)	**(Sites 11 to 19)**	**(Sites 20 to 23)**	**(Sites 11,16, 19)**	**(Sites 1, 15)**

3.4.2 Throughput

The number of vehicles passing the shuttle-lane site was counted every cycle (for sites operated by temporary traffic signals) and also for every 5-minutes interval and by vehicle type and for each direction separately. Observed system throughput will be used to compare with the micro-simulation model output for validation purposes.

3.4.3 Time headway

Time Headway (TH) is defined as the time elapsed between the front of the leading vehicle (n-1) passing an imaginary datum line (x) on the road (or on the playback screen) and the front of the following vehicle (n) passing the same point, as illustrated in Figure 3.3 and shown in Equation 3.1 (Evans and Wasielewski, 1982; Evans, 1991). Observed TH (before approaching the roadworks) will be used to compare with the micro-simulation model output for validation purposes.

$$\mathrm{TH}\,(n,d) = [\mathrm{t}\,(n) - \mathrm{t}\,(n-1)] \qquad \text{Equation 3.1}$$

Where,

t is the time when the vehicle (n-1, n) crosses the datum line

d is the distance from stop line where the time headway is observed

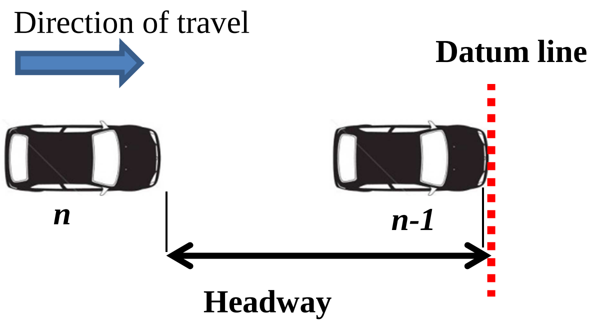


Figure 3.3: Illustration of time headway

3.4.4 Move-up Time (MUT)

Move-up Time (departure headway or discharge headway) at traffic signals is a fundamental parameter used to measure the capacity of an intersection and timing the traffic signals. MUT is usually defined as the time elapsed between successive vehicles (in a queue) when they start to cross the stop line at a signalised junction, after the traffic lights turn green (Jin *et al*., 2009; Briggs, 1977 and Michael *et al*., 2000). The MUT can be calculated using Equation 3.2. Observed MUT for each site and direction will be used to compare with micro-simulation model output for validation purposes.

$$\mathrm{MUT}\ (n) = \ [\mathrm{t}\ (n) - \ \mathrm{t}\ (n-1)] \qquad \text{Equation 3.2}$$

3.4.5 Move-up Delay (MUD)

Following a stopping situation at traffic signals the driver (queue leader) will spend time preparing to move when the light shows green (move-up delay or start-up time). The move-up delay was captured from the time the signals sequence shows red-amber until the first vehicle in the queue starts to move. Observed MUD for each stream will be used as an input into the developed micro-simulation model.

3.4.6 Drivers' behaviour

Drivers' behaviour was also observed on site and from video playback to capture the statistics of vehicles following too closely "tailgating". Also drivers' behaviour in terms of number of vehicles (cycles) crossing the stop line at the onset of amber and red light violations.

3.4.7 Roadworks sites characteristics

Various measurements such as site length and the location of roadworks signs were collected on site using a measuring wheel.

3.4.8 Signals settings

Detailed signals settings such as green time (minimum and maximum for VA sites), all-red period, amber and red-amber were collected on site and from video playbacks. Also the type of signals operation was identified on site and from video playbacks.

3.5 Summary

This chapter described the data collection stages, available methods and the difficult issues related to data collection. In total, data from six different categories have been collected (data from 23 different sites with over 54 hours of video recording). Furthermore, description of each type of the collected data was also provided.

The collected data will be analysed in details as shown in the next chapter and will be used as inputs/outputs for the developed S-Paramics simulation model as explained in the following chapters.

CHAPTER FOUR: DATA ANALYSIS

4.1 Introduction

The aim of this chapter is to present the work that was undertaken in analysing the field data which will be used in developing, calibrating and validating the micro-simulation model. Data collection and analysis is a critical part of the current research in capturing the characteristics of various types of shuttle-lane roadworks and also in understanding drivers' behaviour through urban roadworks.

Data collected on roadworks sites were used to capture vehicles following time headways, departure headways at temporary traffic signals, move-up delays, queuing information, signing and drivers' compliance with temporary traffic signals. Data collected from the same sites (post the removal of roadworks) were used to capture the arrival headway without the effect of the roadworks. Other data were collected on normal signalised junctions, which were used to compare drivers' behaviour with temporary traffic signals at roadworks.

4.2 Data extraction

Data extraction was carried out by displaying the video playback on a computer monitor and a datum line was drawn to extract the data manually (i.e. using event recorder program which was designed for this purpose and works as a stop watch and called "Traffic Logger"). Every time a vehicle crosses the introduced datum line, the observer has to press a button on the keyboard (i.e. 1 for cars and 2 for HGVs) and the program will output a text file with the exact relative time (with accuracy of 1/100 of a second). Following the completion of the playback time (video survey time), the observer has to export the text file into excel and start the analysis work.

Data such as queues was observed on site (using both a video camera microphone and a paper copy). Site length, road width and distance between signs were collected on site using a measuring wheel and a site layout was drawn on site with the respective dimensions.

4.3 Accuracy of observed data

It was mentioned in the previous section that the data extraction technique for the event time data (i.e. move-up delay, arrival time headway) was carried out using video playback and the

"Traffic Logger" program which acts as a stop watch. This method will result in two types of errors:

1- Video playback error (depends on the playback frame rate per second);
2- Human error (i.e. time taken to manually press a button when vehicle passes the datum line).

The video rate in the current study is fixed to the standard 25 frames per second, which produces an error of 0.04s for the event-time data. According to Bonneson and Fitts (1995), the total possible error for the event-time data could be up to 0.1s.

To measure accuracy (the amount of possible errors), two types of event time data (arrival time headway and move-up delay) were repeated by different trained observers (both used for the rest of the data analysis).

Figure 4.1 shows the MUD measurements obtained by different observers with the R^2 (coefficient of determination) = 0.9479, which shows a close fit between both observers and the high reliability of the observed data. Table 4.1 shows the mean (µ) and standard deviation (σ) for the tested sample.

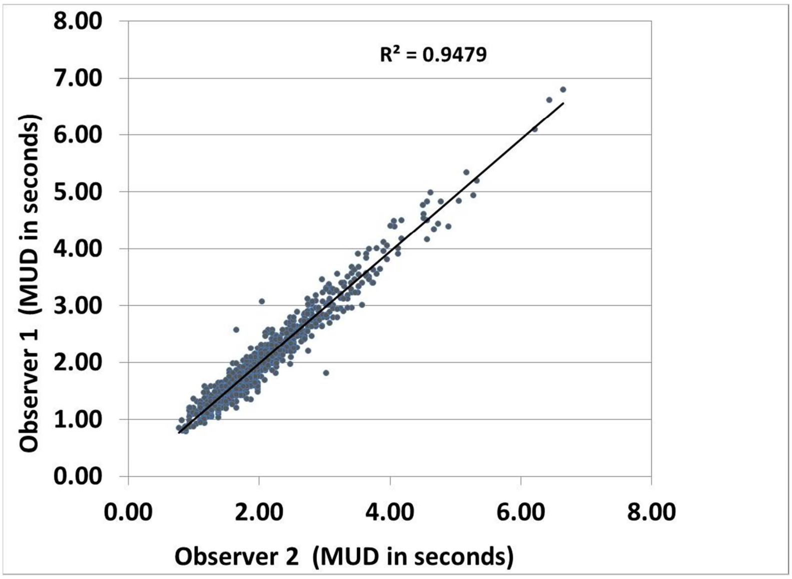


Figure 4.1: Correlation between different observers in measuring MUD

Table 4.1: MUD mean (µ) and sd (σ) by different observers (in seconds)

Observer	µ	Σ
1	2.13	0.65
2	2.08	0.67

Figure 4.2 shows the arrival time headway measurements obtained by observer 1 and observer 2 with the R2 = 0.999, which shows a close fit between both observers and the high reliability of the observed data. Table 4.2 shows the mean (μ) and standard deviation (σ) for the tested sample. It can be seen from Table 4.2 that the difference in the sample mean between observer 1 and observer 2 is only 0.04 seconds, which can be assumed to be acceptable.

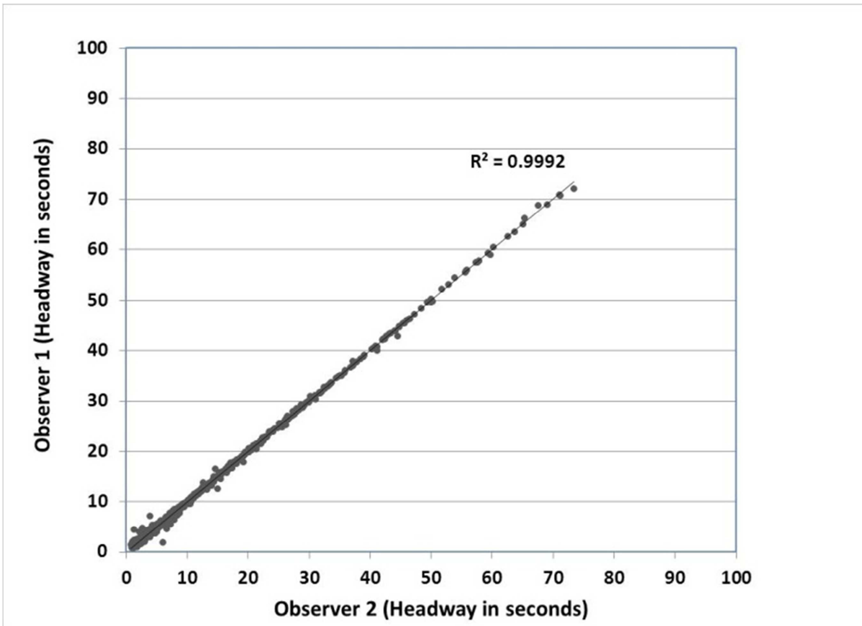


Figure 4.2: Correlation between different observers in measuring arrival headway

Table 4.2: Arrival headway mean (μ) and sd (σ) by different observers (in seconds)

Observer	Arrival headway	
	μ	σ
1	8.25	10.74
2	8.21	10.75

4.4 Flow level and profile

Traffic flow information (i.e. flow by vehicle type, directional split and HGVs percentage) was collected for each site and for each direction separately and will be used as input into the developed micro-simulation model. Flow profile (flow per 5-minutes interval) was also collected for each site and direction. Table 4.3 summarises the arrival flow, directional split and HGVs percentage for each site and each stream separately. The flow for each 5-minutes interval was multiplied by 12 to represent an average hourly flow in veh/hr.

It can be seen from Table 4.3 that the flow level varies between sites with an equivalent hourly rate ranging between 72-888 veh/hr. The directional split also varies between sites and between different hours for the same site (between 70/30 and 50/50). HGVs percentage also varies between sites, streams and different hours on the same site with HGVs percentage varies between 1% (minimum) and 10% (maximum). Flow profiles for each 5-minutes interval (for each site and each stream) are presented graphically in Appendix B.

Table 4.3: Summary of arrival flow for each site

Flow (veh/hr)		**Directional Split range P/S (%)**		**HGVs Percentage (%)**	
Minimum	**Maximum**	**Minimum**	**Maximum**	**Minimum**	**Maximum**
(P) 72 (S) 84	(P) 888 (S) 636	50/50	71/29	(P) 1 (S) 1	(P) 10 (S) 10

4.5 Arrival headway

4.5.1 Arrival headway distribution

As discussed in Chapter 3, arrival time headway represents the time interval between the passages of successive vehicles passing a reference point on the road (Salter and Hounsell, 1996). Arrival headway is used to generate vehicles at the start of the micro-simulation model.

Different single and composite distribution models have been used by previous researchers to represent headway distribution such as exponential distribution, negative exponential distribution (with and without the shift), lognormal distribution (with and without the shift) and gamma distribution (Salter and Hounsell, 1996; Luttinen, 1996). Detailed description of the tested headway distribution models are presented in the following sections.

4.5.1.1 The shifted negative exponential

The shifted negative exponential distribution by a minimum headway (Shift) is able to represent the vehicle arrival rate for moderate flow (Sultan, 2000; Al-Obaedi, 2012). The headway for each vehicle can be represented by Equation 4.1 (Benekohal, 1986);

$$\mathrm{TH} = \mathrm{shift} - [\mathrm{l}/\mathrm{z} - \mathrm{shift}]\ \ln(\mathrm{RAND}) \qquad \text{Equation 4.1}$$

Where,

Shift is the additional time such as 0.25, 0.5 and 1 in seconds
RAND is the random number generated by the program
z is the arrival flow rate

4.5.1.2 Lognormal distribution

Yin *et al*. (2009) reported that the lognormal distribution is suitable to fit headway distribution data under low traffic level situations in urban areas. Below is the probability density function as shown in Equation 4.2 to Equation 4.4 (Walck, 1996; Sultan, 2000)

$$F(x) = \frac{1}{\sigma x\sqrt{2\pi}} e^{-\frac{(\ln(x) - \mu)^2}{2\sigma^2}} \qquad \text{Equation 4.2}$$

$$\mu = \ln(m) - \frac{\sigma^2}{2} \qquad \text{Equation 4.3}$$

$$\sigma^2 = \ln(1 + \frac{s^2}{m^2}) \qquad \text{Equation 4.4}$$

Where,

μ and **σ** are the mean and the standard deviation of the normal distribution

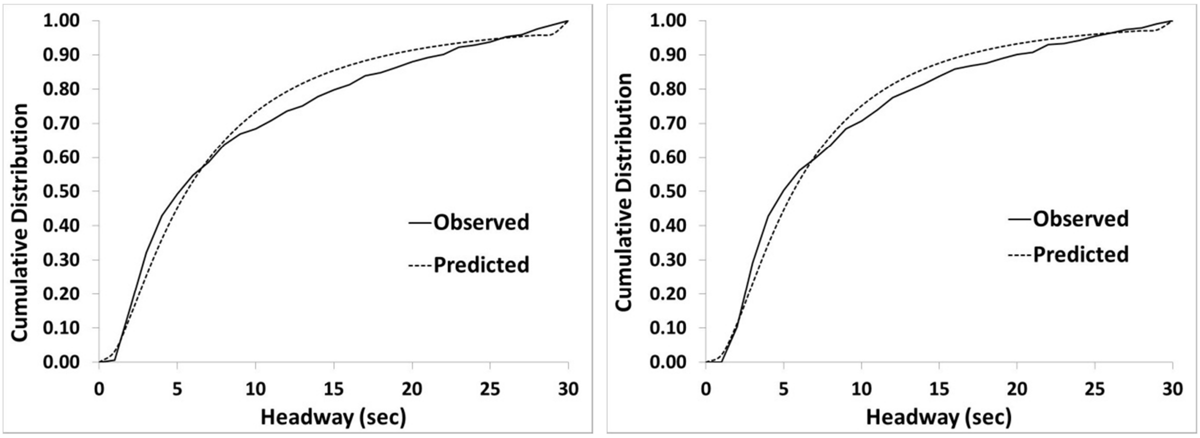
m and ***s*** are the mean and the standard deviation of the lognormal distribution

The simplest way of achieving random numbers from a lognormal distribution is to take the exponential of the generated random numbers u (RAND=e^u) from a normal distribution with mean μ and standard deviation σ (Walck, 1996).

4.5.2 Headway models using real data

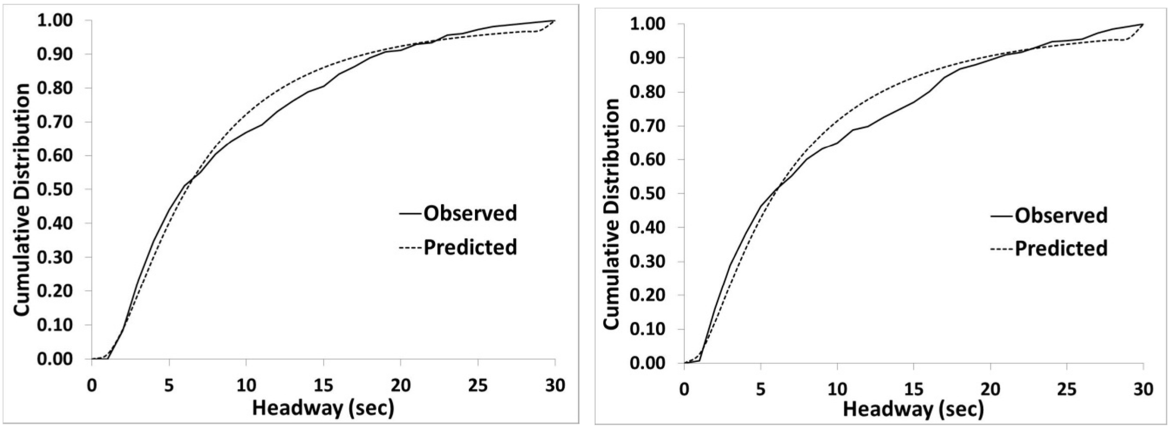
Data was collected using the video recording technique as explained earlier for three sites (Sites 11, 16 and 19) without the presence of roadworks (post-completion of roadworks on separate days). The data was collected to test the goodness of fit with the headway distribution models mentioned earlier. The equivalent flow range was between 230-676 veh/hr. For each site, data was collected for 60 minutes period. The tested models are the lognormal and the shifted negative exponential distributions.

Using the lognormal distribution and based on sites 11 and 16 data, Figures 4.3 and 4.4 (for primary and secondary streams) show good agreement between the actual and the predicted cumulative headway distribution for a flow between 230-300 veh/hr (low level of flow) for both streams. The results also show that the shifted negative exponential distribution is not applicable for sites 11 and 16 where low flow levels exist. These results are in agreement with the findings of Yin *et al*. (2009), which recommended using the lognormal distribution for low flow level on urban roads.



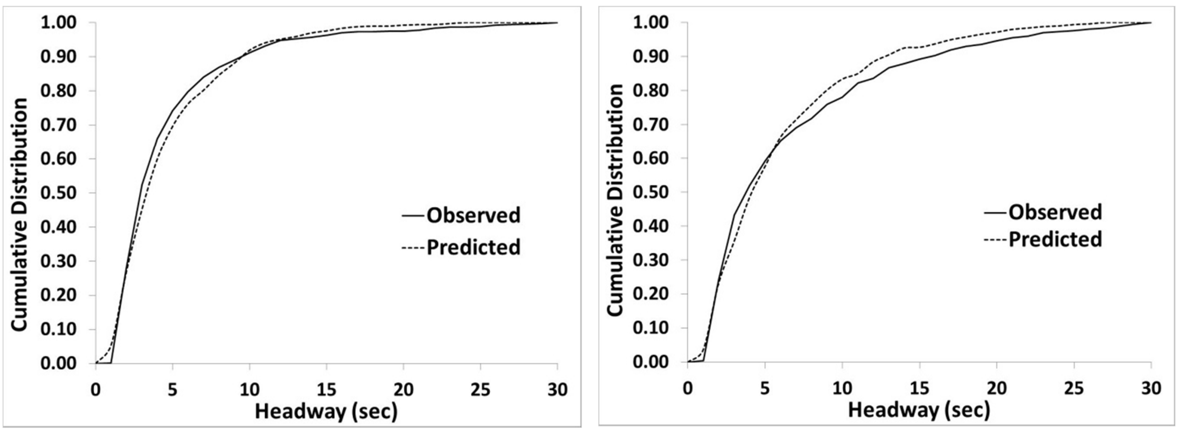
(a) Primary Stream (b) Secondary Stream

Figure 4.3: Observed and predicted arrival headway cumulative distribution for Site 11 using lognormal distribution



(a) Primary Stream (b) Secondary Stream

Figure 4.4: Observed and predicted arrival headway cumulative distribution for Site 16 using lognormal distribution



(a) Primary Stream (b) Secondary Stream

Figure 4.5: Observed and predicted arrival headway cumulative distribution for Site 19 using shifted negative exponential distribution

For Site 19 where moderate/high flow level is observed (500-700 veh/hr), the shifted negative exponential headway distribution shows good agreement between the actual and predicted cumulative headway distribution for both streams with the best shift value of 0.8 as shown in Figure 4.5.

The non-parametric Kolmogorov-Smirnov hypothesis statistical test (K-S test) was used at 5% level of significance (α = 0.05). The test compares the maximum difference (D_{max}) between two observed and fitted distribution functions with the critical value (D_{cr}) which can be obtained from tables or as shown in Equation 4.5 (Hayter, 2002).

$$D_{cr} = 1.36\sqrt{\frac{N1+N2}{N1N2}} \qquad \text{Equation 4.5}$$

(for 95% confidence level and sample size over 35 for each N1 or N2)
Where,

$\mathbf{N_1}$ and $\mathbf{N_2}$ are the sample sizes

The test results are shown in Table 4.4 which reflects the above results and findings. The table also suggests that no single model is capable of representing the arrival distribution of traffic for the different tested flow rates. Therefore, when generating traffic in the simulation model, it was suggested to use the lognormal distribution for sites with low flow levels (up to 500 veh/hr) and use the shifted negative exponential distribution for sites with moderate to high flow levels (over 500 veh/hr).

Table 4.4: Summary of statistics for arrival headway distribution fitting

Site	**Site 11**		**Site 16**		**Site 19**	
Stream	**P**	**S**	**P**	**S**	**P**	**S**
Average Flow (veh/hr)	254	235	263	297	676	509
Lognormal (D_{max})	0.069	0.081	0.067	0.078	0.081	0.091
Shifted negative exponential (D_{max})	0.149	0.174	0.189	0.139	0.072	0.077
K-S critical value (D_{cr})	0.121	0.126	0.119	0.112	0.074	0.085
Accept-Lognormal	Yes	Yes	Yes	Yes	No	No
Accept-Shifted neg. exp.	No	No	No	No	Yes	Yes

4.6 Following headway

Time headway is a fundamental and important parameter which has been used in traffic flow theory to determine system capacity, level of service (LOS) and safety aspects

(Jang *et al*., 2011). In the current study, time headway will be used to study both following behaviour and also for calibrating/validating the micro-simulation model.

In an urban environment, the vehicle can be in two states; a "free vehicle" state, where the vehicle's desired speed is not influenced by the speed of the leader or in a platoon state, where the vehicle is influenced by the speed of the leader vehicle and is forced to change its desired speed (Vogel, 2002).

4.6.1 Following headway distribution

Many studies have examined the criteria for defining the platoon threshold. These criteria are expressed by time, gap or following distance as summarised in Table 4.5. The suggested threshold of 6 seconds by Vogel (2002) has been adopted for the current study because of it being based on a similar urban environment whilst other studies were carried out on high speed roads. Therefore, a vehicle is assumed to be travelling at free flow conditions when the time headway between the following vehicle and the leading vehicle is > 6 seconds and vehicles are in a platoon when time headways are ≤ 6 seconds.

Table 4.5: Summary of some previous studies examined following behaviour

Study	Type/Data source	Number of Observations	Following threshold	Comments
Parker (1996)	Empirical data from video	7,199	5 seconds	Motorway roadworks
Vogel (2002)	Empirical data from loops	100,000	6 seconds	Urban areas/signalised intersection
HCM (2000)	TWOPAS simulation program	NA	3 seconds (5 previously)	based on assessment by Rouphail (2000) and Vogel (2002)
Wasielewski (1979)	Empirical data from site	42,000	4 seconds	Expressway
Al-Kaisy and Karjala (2010)	Pneumatic tubes	50,854	6 seconds	Highway (8 sites)

Data was collected using the video recording technique as explained in Section 3.4 for all sites and for the primary and secondary streams separately. Only time headway data for vehicles ≤ 6 seconds were analysed. The data was also recorded for two separate situations (as shown in Figure 3.1) to determine the effect of the presence of roadworks on the following behaviour:

1- Before Approaching the Roadworks sites (referred to as BAR);
2- After Crossing the Roadworks sites (referred to as ACR).

Table 4.6 summarises the statistical values for following headway for each site by direction and situation (time headway for vehicles ≤ 6 seconds). It can be seen from Table 4.6 that the number of observations (amount of vehicles with time headway ≤ 6 seconds) have increased when comparing BAR and ACR for both streams (primary and secondary) and for all sites which shows that more vehicles have joined a platoon in the ACR situation as a result of introducing temporary traffic signals at shuttle-lane roadworks. This increase in the number of observations (N) and the decrease in mean time headway (μ) relates to various parameters including arrival rates, cycle time (and each direction green time), site length, the behaviour of drivers at roadworks, traffic composition and weather conditions.

Table 4.6: Summary of statistics for following headway for each site

Site No.	Site Length (m)	Location	Number of observations (N)	μ (sec)	Min (sec)	σ (sec)
Site 11	42	BAR	(P) 174 (S) 239	(P) 3.18 (S) 2.52	(P) 0.93 (S) 0.70	(P) 0.97 (S) 0.88
		ACR	(P) 195 (S) 283	(P) 2.43 (S) 2.51	(P) 1.16 (S) 0.99	(P) 0.92 (S) 1.07
Site 12	107	BAR	(P) 86 (S) 70	(P) 2.49 (S) 2.70	(P) 0.88 (S) 0.77	(P) 1.09 (S) 1.01
		ACR	(P) 147 (S) 155	(P) 2.46 (S) 2.48	(P) 1.32 (S) 1.37	(P) 0.90 (S) 0.86
Site 16	52	BAR	(P) 383 (S) 438	(P) 3.30 (S) 3.13	(P) 1.04 (S) 0.93	(P) 0.57 (S) 0.59
		ACR	(P) 545 (S) 639	(P) 2.48 (S) 2.52	(P) 0.93 (S) 1.10	(P) 0.59 (S) 0.47
Site 17	39	BAR	(P) 58 (S) 78	(P) 3.41 (S) 3.49	(P) 1.17 (S) 1.20	(P) 0.66 (S) 0.60
		ACR	(P) 108 (S) 123	(P) 2.52 (S) 2.49	(P) 1.20 (S) 0.94	(P) 0.99 (S) 0.88
Site 18	73	BAR	(P) 160 (S) 358	(P) 3.26 (S) 2.97	(P) 0.89 (S) 0.76	(P) 1.16 (S) 1.07
		ACR	(P) 240 (S) 475	(P) 2.44 (S) 2.41	(P) 0.72 (S) 0.83	(P) 1.15 (S) 0.96
Site 19	38	BAR	(P) 932 (S) 571	(P) 2.88 (S) 2.75	(P) 0.82 (S) 0.82	(P) 0.83 (S) 0.96
		ACR	(P) 958 (S) 707	(P) 2.33 (S) 2.33	(P) 0.72 (S) 0.93	(P) 0.58 (S) 0.85
All sites		**BAR**	**(P) 1,793 (S) 1,754**	**(P) 2.84 (S) 3.01**	**(P) 0.82 (S) 0.72**	**(P) 0.81 (S) 0.86**
		ACR	**(P) 2,192 (S) 2,382**	**(P) 2.47 (S) 2.51**	**(P) 0.70 (S) 0.83**	**(P) 0.71 (S) 0.79**
All sites		**BAR ACR**	**3,547 4,574**	**2.93 2.49**	**0.72 0.70**	**0.97 1.08**

The distributions of following time headways in platoons for both primary and secondary streams and for both situations (BAR and ACR) fit the lognormal distribution and the statistical results are summarised in Table 4.7 and shown in details in Appendix B. This finding confirms studies carried out by Chin *et al*. (2010) and Dey and Chandra (2009). The non-parametric Kolmogorov-Smirnov (K-S) hypothesis statistical test was used at the 5% level of significance ($\alpha = 0.05$). The K-S test compares the maximum difference (D_{max}) between the two cumulative distributions and the critical value (D_{cr}) which is obtained from the K-S tables. The following headway distribution results will be used as part of the micro-simulation model calibration/validation stage.

Table 4.7: Summary of statistics for time headway with lognormal distribution fitting

Site	Location	Sample Size	D_{cr}	D_{max} (lognormal)	Accept
11	BAR	(P) 174 (S) 239	(P) 0.15 (S) 0.12	(P) 0.10 (S) 0.09	(P) Yes (S) Yes
	ACR	(P) 195 (S) 283	(P) 0.14 (S) 0.11	(P) 0.07 (S) 0.08	(P) Yes (S) Yes
12	BAR	(P) 86 (S) 70	(P) 0.21 (S) 0.23	(P) 0.13 (S) 0.09	(P) Yes (S) Yes
	ACR	(P) 147 (S) 155	(P) 0.16 (S) 0.15	(P) 0.05 (S) 0.05	(P) Yes (S) Yes
16	BAR	(P) 383 (S) 438	(P) 0.09 (S) 0.09	(P) 0.08 (S) 0.06	(P) Yes (S) Yes
	ACR	(P) 544 (S) 639	(P) 0.08 (S) 0.07	(P) 0.05 (S) 0.04	(P) Yes (S) Yes
17	BAR	(P) 58 (S) 78	(P) 0.25 (S) 0.22	(P) 0.08 (S) 0.16	(P) Yes (S) Yes
	ACR	(P) 108 (S) 123	(P) 0.19 (S) 0.17	(P) 0.05 (S) 0.05	(P) Yes (S) Yes
18	BAR	(P) 160 (S) 358	(P) 0.15 (S) 0.10	(P) 0.07 (S) 0.05	(P) Yes (S) Yes
	ACR	(P) 240 (S) 475	(P) 0.12 (S) 0.09	(P) 0.06 (S) 0.03	(P) Yes (S) Yes
19	BAR	(P) 932 (S) 571	(P) 0.06 (S) 0.08	(P) 0.04 (S) 0.04	(P) Yes (S) Yes
	ACR	(P) 958 (S) 707	(P) 0.06 (S) 0.07	(P) 0.05 (S) 0.04	(P) Yes (S) Yes

Following headway (vehicle with time headway $\leq$ 6 seconds) comparisons were also carried out for both BAR and ACR to compare if the roadworks site length had any significant impact on the following behaviour. A cumulative distribution function was plotted for both BAR and ACR situations and for the primary and secondary streams as shown in Appendix B. The non-parametric Kolmogorov-Smirnov hypothesis statistical test (K-S test) was used at 5% level of significance ($\alpha = 0.05$) to compare both situations for each stream. The results are summarised in Table 4.8.

It can be seen from Table 4.8 that there is a significant difference in following time headway for vehicles when comparing BAR and ACR except for Site 12 using the K-S test. The reasons for Site 12 being the exception could be attributed to the fact that it has a length of

107 metres, which could be considered as long enough length for drivers to regulate to normal following behaviour after stopping at temporary traffic signals. While for relatively shorter site lengths (all other sites) less than 100 metres, this will not give the drivers enough time to regulate back to their normal following driving behaviour. It can also be seen that the increase in the number of vehicles with headway ≤6 seconds when comparing BAR and ACR is a result of both the introduction of temporary traffic signals and also roadworks sites with different lengths.

Table 4.8: Summary of statistics for time headway comparison between BAR and ACR

Site	Direction	Sample Size	D_{cr}	D_{max}	Accept
11	Primary	(BAR) 174 (ACR) 195	0.14	0.15	No
	Secondary	(BAR) 239 (ACR) 283	0.12	0.17	No
12	Primary	(BAR) 86 (ACR) 147	0.18	0.14	Yes
	Secondary	(BAR) 70 (ACR) 155	0.20	0.17	Yes
16	Primary	(BAR) 383 (ACR) 544	0.09	0.31	No
	Secondary	(BAR) 438 (ACR) 639	0.08	0.23	No
17	Primary	(BAR) 58 (ACR) 108	0.22	0.30	No
	Secondary	(BAR) 78 (ACR) 123	0.20	0.33	No
18	Primary	(BAR) 160 (ACR) 240	0.14	0.30	No
	Secondary	(BAR) 358 (ACR) 475	0.10	0.18	No
19	Primary	(BAR) 932 (ACR) 958	0.06	0.09	No
	Secondary	(BAR) 571 (ACR) 707	0.08	0.18	No

Analysis of platoon size was also carried out for each site, each direction (i.e. primary and secondary streams) and for each situation (i.e. BAR and ACR). The size of each platoon (the number of vehicles following with time headway ≤ 6 seconds) was recorded and is shown in Figure 4.6. The x-axis shows the platoon size in vehicles and the y-axis shows the frequency of each platoon size.

Figure 4.6 shows that the relative frequency of longer platoon sizes (from platoon size of 4 vehicles or more) increased for ACR compared with BAR. This could be due to various factors such as site length, the presence of temporary traffic signals, arrival headway distribution, etc. Platoon size could be investigated further in a separate study to determine the platoon characteristics (i.e. platoon speed, platoon average headway and inter-arrival between consecutive platoons). Platoon-based signals algorithm was developed by Jiang *et al*. (2006), which could reduce vehicle delays, increase site capacity and also help in identifying the need for signals coordination between the upstream intersection and temporary traffic signals (to allow for the effective release of the queue and to avoid blocking back especially for longer platoon sizes).

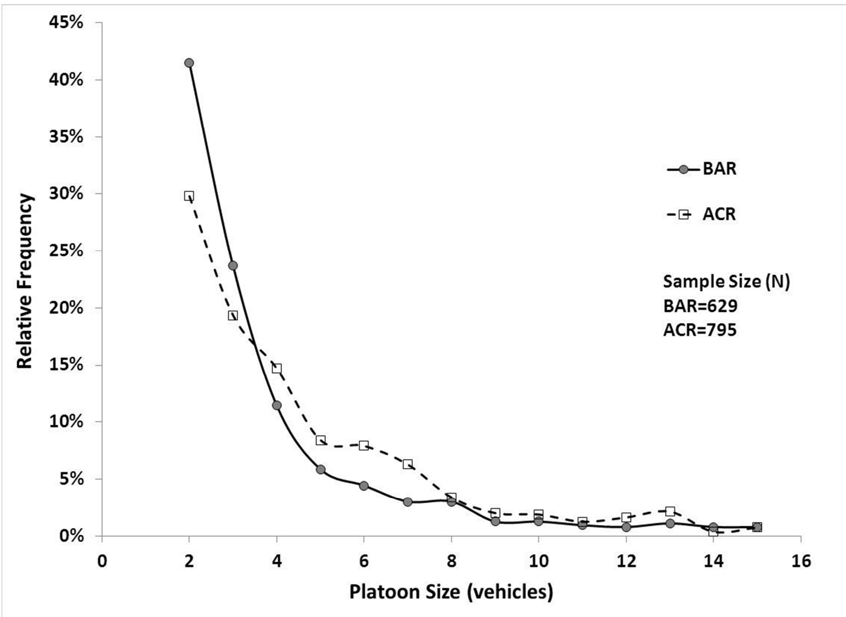


Figure 4.6: Platoon size frequency for shuttle-lane roadworks

4.6.2 Close following behaviour "tailgating"

In the current study, vehicles with time headway less than two seconds for each site, stream and location (BAR and ACR) are summarised in Table 4.9 and illustrated in Figure 4.7. The number of observations (n) shown in column 4 is the number of vehicles that are following in platoons with time headway ≤ 6 seconds.

Table 4.9 shows that a high proportion of drivers do not comply with the two-seconds rule (i.e. travelling with time headways < 2 seconds). The figures suggest a range between 14% and 49% (in each site and for both streams) which is deemed to be high. Vehicles following with a time headways less than 1.5 seconds are also summarised (ranging between 3% to 22%). Vehicles with time headways less than 1 second were also noticed (up to 4% of the total).

Figure 4.7 shows that the percentage of vehicles "tailgating", which as a behaviour is considered dangerous and aggressive, is higher for ACR (After Crossing Roadworks) compared with BAR (Before Approaching Roadworks) for every site and for every stream. Moreover, the percentage of non-complying drivers is higher in the primary stream (27% and 39% for BAR and ACR, respectively) compared with the secondary stream (22% and 37% for BAR and ACR, respectively) as shown in Table 4.9. This might be related to the primary horizontal deflection due to site obstruction and/or the manoeuvre to return to the original lane which could affect drivers' behaviour and increase tailgating (e.g. primary stream drivers' speed may be slightly lower due to horizontal deflection which may result in driving closer without compromising safety). However, the horizontal deflection could be one of many factors that could affect the increased behaviour of tailgating for few drivers.

The increase in tailgating behaviour when comparing BAR with ACR for both streams could also be attributed to the fact that drivers experienced some delay when stopping at the temporary traffic signals. Drivers may perceive that clearing the site as quickly as possible by speeding could save them time and as a result, they may not follow a safe following headway (i.e. less than 2 seconds). This higher percentage of tailgating could result in higher risks of rear-end collision or sudden/sharp braking. This in turn could have an adverse impact on safety and possibly cause capacity reduction. Further work may be needed to investigate the relationship between relative speeds and close following for BAR and ACR.

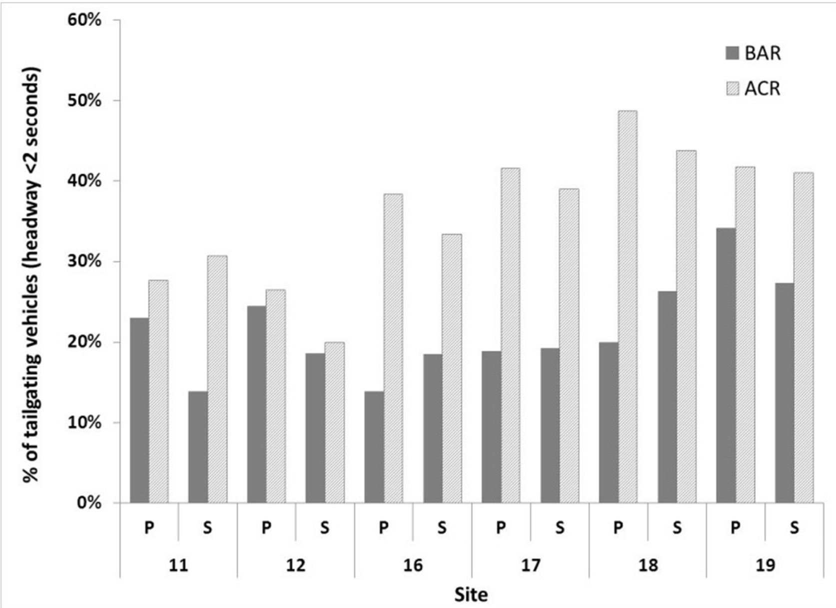


Figure 4.7: Percentage of vehicles with tailgating behaviour for each site, stream and location

Table 4.9: Summary of close following behaviour for all sites, streams and location

Site	Direction	Location	n	Vehicles with TH < 2 sec (%)	Vehicles with TH ≤ 1.5 sec (%)	Vehicles with TH ≤ 1 sec (%)
11	Primary	(BAR) (ACR)	174 195	40 (23%) 54 (28%)	13 (7%) 10 (5%)	2 (1%) 0 (0%)
	Secondary	(BAR) (ACR)	239 283	33 (14%) 87 (31%)	7 (3%) 23 (8%)	1 (0.4%) 1 (0.4%)
12	Primary	(BAR) (ACR)	86 147	21 (24%) 39 (27%)	7 (8%) 7 (5%)	1 (1%) 0 (0%)
	Secondary	(BAR) (ACR)	70 155	13 (19%) 31 (20%)	4 (6%) 11 (7%)	2 (3%) 0 (0%)
16	Primary	(BAR) (ACR)	383 544	53 (14%) 209 (38%)	16 (4%) 58 (11%)	0 (0%) 1 (0.2%)
	Secondary	(BAR) (ACR)	438 639	81 (18%) 214 (33%)	15 (3%) 45 (7%)	1 (0.2%) 0 (0%)
17	Primary	(BAR) (ACR)	58 108	10 (17%) 45 (42%)	5 (9%) 16 (15%)	1 (2%) 2 (2%)
	Secondary	(BAR) (ACR)	78 123	15 (19%) 48 (39%)	7 (9%) 27 (22%)	3 (4%) 3 (2%)
18	Primary	(BAR) (ACR)	160 240	32 (20%) 117 (49%)	19 (12%) 51 (21%)	2 (1%) 10 (4%)
	Secondary	(BAR) (ACR)	358 475	94 (26%) 208 (44%)	40 (11%) 93 (20%)	4 (1%) 8 (2%)
19	Primary	(BAR) (ACR)	932 958	319 (34%) 400 (42%)	108 (12%) 161 (17%)	10 (1%) 21 (2%)
	Secondary	(BAR) (ACR)	571 707	156 (27%) 290 (41%)	52 (9%) 109 (15%)	5 (1%) 12 (2%)
Total	**Primary**	**(BAR) (ACR)**	**1,793 2,192**	**475 (27%) 864 (39%)**	**168 (9%) 303 (14%)**	**16 (1%) 34 (2%)**
	Secondary	**(BAR) (ACR)**	**1,754 2,382**	**392 (22%) 878 (37%)**	**125 (7%) 308 (13%)**	**16 (1%) 24 (1%)**
Total		**(BAR) (ACR)**	**3,547 4,575**	**867 (24%) 1,742 (38%)**	**293 (8%) 611 (13%)**	**32 (1%) 58 (1%)**

N: Sample size **TH**: time headway

4.7 Move-up time (MUT)

4.7.1 Existing shuttle-lane roadworks data

MUT data was collected for primary and secondary streams separately for micro-simulation model purposes. The observed drivers MUT will be compared with the micro-simulation model output as part of the model calibration/validation process.

Table 4.10 summaries the move-up time statistical data (i.e. mean (μ), standard deviation (σ), minimum and maximum) for each vehicle position in the queue for both primary and secondary streams. Several studies (Jin *et al*., 2009; Briggs, 1977; Michael *et al*., 2000; Gerlough and Wagner, 1967; Hung *et. al*, 2002 and Niittymaki and Pursula, 1996) have indicated that a key feature of vehicles headway in a queue is that it often gradually decreases from the first vehicle in the queue (queue head) to its end (last vehicle in the queue) and steady headway will be reached by the fourth or the fifth vehicle.

According to Su *et al*. (2009), this occurs because the first few vehicles in the queue require longer reaction time and space to start while the rest have enough time and space to keep a distance. These findings were confirmed by the current study using temporary traffic signals at shuttle-lane roadworks, as shown in Figure 4.8. Various parameters (external factors) such as number of lanes, vehicle types, etc. were considered in the studies mentioned above but none of these were carried out on a single lane traffic signals or shuttle-lane roadworks.

Table 4.10: Move-up time for shuttle-lane roadworks

Vehicle position in a queue	**Sample Size (N)**	**μ (sec)**	**σ (sec)**	**Minimum (sec)**	**Maximum (sec)**
2	(P) 405 (S) 382	(P) 2.35 (S) 2.43	(P) 0.84 (S) 0.66	(P) 1.15 (S) 1.32	(P) 5.66 (S) 5.16
3	(P) 316 (S) 288	(P) 2.20 (S) 2.26	(P) 0.57 (S) 0.48	(P) 1.04 (S) 1.16	(P) 4.23 (S) 4.07
4	(P) 228 (S) 205	(P) 2.14 (S) 2.15	(P) 0.61 (S) 0.49	(P) 0.99 (S) 1.02	(P) 4.34 (S) 4.28
5	(P) 159 (S) 125	(P) 1.99 (S) 2.12	(P) 0.49 (S) 0.39	(P) 1.04 (S) 1.26	(P) 4.84 (S) 3.63
6	(P) 110 (S) 67	(P) 1.90 (S) 2.05	(P) 0.41 (S) 0.27	(P) 1.05 (S) 1.32	(P) 3.24 (S) 3.57
7	(P) 73 (S) 34	(P) 1.86 (S) 2.00	(P) 0.20 (S) 0.42	(P) 1.15 (S) 1.17	(P) 2.80 (S) 2.69
8	(P) 44 (S) 16	(P) 1.82 (S) 1.99	(P) 0.43 (S) 0.37	(P)1.27 (S) 1.37	(P) 2.58 (S) 2.80

The distribution of move-up time for both streams fit the lognormal distribution as illustrated in Appendix B and for which the statistical results are summarised in Table 4.11. This finding is confirmed by a study that was carried out by Jin *et al*. (2009). The non-parametric Kolmogorov-Smirnov hypothesis statistical test (K-S test) was used at the 5% level of significance (α = 0.05). The test compares the maximum difference (D_{max}) between two observed and fitted cumulative distribution functions with the critical value (D_{cr}) which can be obtained from K-S tables or as shown in Equation 4.5 (Hayter, 2002).

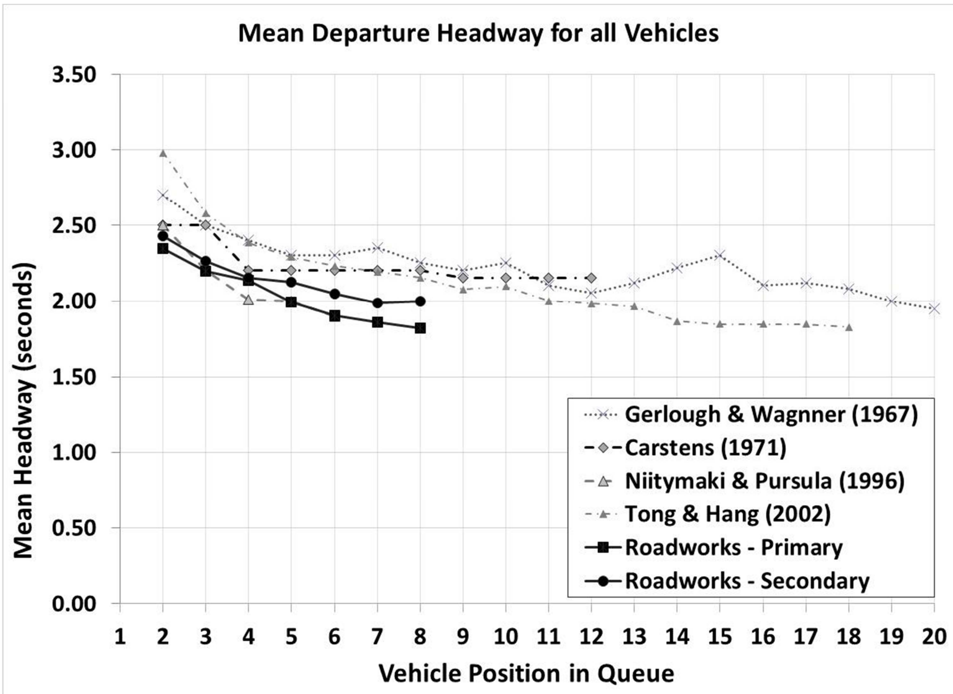


Figure 4.8: Move-up time for various studies compared with shuttle-lane roadworks

Table 4.11: Summary of statistics for MUT with lognormal distribution fitting

Vehicle position in a queue	Sample Size	D_{cr}	D_{max} (lognormal)	Accept
2	(P) 405 (S) 382	(P) 0.095 (S) 0.098	(P) 0.036 (S) 0.021	(P) Yes (S) Yes
3	(P) 316 (S) 288	(P) 0.108 (S) 0.113	(P) 0.076 (S) 0.019	(P) Yes (S) Yes
4	(P) 228 (S) 205	(P) 0.127 (S) 0.134	(P) 0.071 (S) 0.127	(P) Yes (S) Yes
5	(P) 159 (S) 125	(P) 0.152 (S) 0.172	(P) 0.102 (S) 0.036	(P) Yes (S) Yes
6	(P) 110 (S) 67	(P) 0.183 (S) 0.235	(P) 0.093 (S) 0.096	(P) Yes (S) Yes
7	(P) 73 (S) 34	(P) 0.225 (S) 0.329	(P) 0.065 (S) 0.076	(P) Yes (S) Yes
8	(P) 44 (S) 16	(P) 0.290 (S) 0.481	(P) 0.058 (S) 0.282	(P) Yes (S) Yes

4.7.2 Comparison with historical data

Move-up time for the historical sites (Site 1 to Site7 and 15) were observed and analysed on the same basis that was carried out on the current roadworks sites. MUT data for both roadworks (historical and current) sites and traffic signals sites are summarised in Table 4.12.

Data for the primary and secondary streams were combined for the current roadworks sites for comparison purposes as there was no significant difference between both streams.

Table 4.12: Summary of MUT mean value (**μ**) in seconds for current and historical sites

Vehicle position in a queue	Current roadworks sites	Historical roadworks sites	Traffic signals
2	2.39	2.47	1.95
3	2.23	2.46	2.02
4	2.15	2.35	2.13
5	2.05	2.25	2.21
6	1.96	2.23	1.97
7	1.90	NA	NA
8	1.87	NA	NA
Total *	**10.78**	**11.76**	**10.28**

*****: Total is up to the 6th vehicle **NA**: Not Available

It can be seen from Table 4.12 that the MUT results for roadworks sites (current and historic) are not significantly different from typical signalised junctions. The difference is negligible which could be due to different factors (i.e. percentage of HGVs, drivers' reaction time, acceleration capabilities, flow levels and frustration, weather conditions, etc.).

4.8 Move-up delay (MUD)

MUD was captured from the time that the signals sequence shows red-amber until the first vehicle in the queue starts to move. The data was collected for primary and secondary streams separately and will be used as an input in the micro-simulation model.

Table 4.13 summaries the MUD for shuttle-lane roadworks compared with previous studies. It was noticed that for Site 12 (heavy rain conditions), the move-up delay increased by 35% and 30% for primary and secondary streams, respectively compared with sunny/cloudy with dry surface which could be attributed to poor visibility caused by adverse weather condition (i.e. heavy rain).

It can be seen from Table 4.13 that move-up delay ranges between 0.8 and 6.2 seconds with a sample mean (μ) of 2.0 seconds for the primary stream. For the secondary stream, move-up delay ranges between 0.8 and 6.7 seconds with a sample mean (μ) of 2.0 seconds. These statistical values are for dry road surface condition (cloudy or sunny situation).

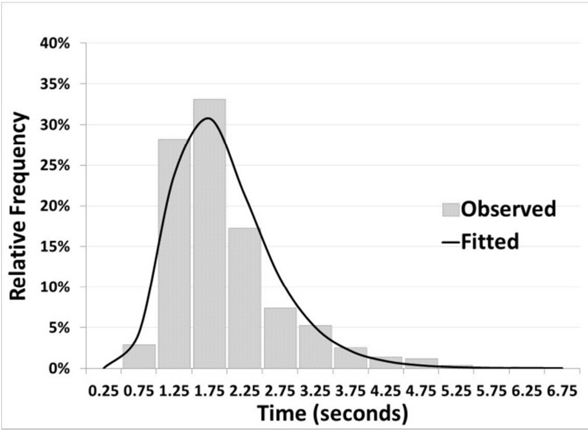
In heavy rain situations with wet road surface, the move-up delay ranges between 1.0 and 4.9 seconds with a sample mean (μ) of 2.7 seconds for the primary stream. For the secondary stream, move-up delay ranges between 1.0 and 6.4 seconds with a sample mean (μ) of 2.6 seconds. These means are higher than all averages reported by previous studies at signals control operations. The reason may be drivers' slow reactions when they anticipate that congestion still exists (Yousif, 1993). It is reasonable to assume that drivers with longer reaction times will have a longer move-up delay than those having shorter reaction times (Benekohal, 1986; Yousif, 1993; Al-Obaedi, 2012).

Table 4.13: Move-up delay for various studies

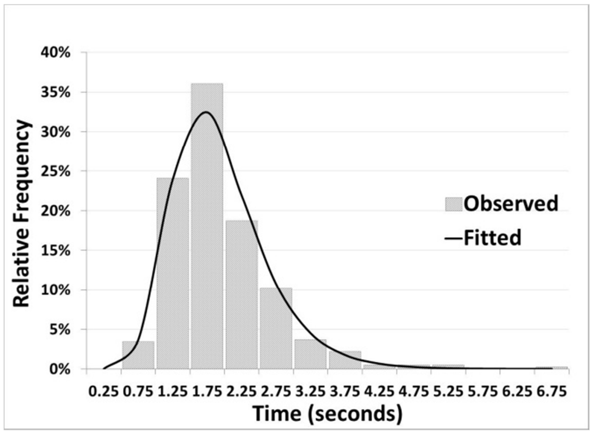
Study	**Road Type**	**Road Condition**	**Sample Size (N)**	**μ (sec)**	**σ (sec)**	**Min (sec)**	**Max (sec)**
Yousif, 1993	Motorway	NA	437	Cars 1.8 HGVs 2.0	NA	0.6	6.0
Al-Obaedi, 2012	Motorway	NA	NA	1.8	NA	0.5	6.5
Current Study	Urban-Roadworks	Dry	(P) 510 (S) 411	(P) 2.0 (S) 2.0	(P) 0.7 (S) 0.7	(P) 0.8 (S) 0.8	(P) 6.2 (S) 6.7
Current Study	Urban-Roadworks	Heavy rain/Wet	(P) 48 (S) 71	(P) 2.7 (S) 2.6	(P) 0.8 (S) 0.9	(P) 1.0 (S) 1.0	(P) 4.9 (S) 6.4

NA: Not Available

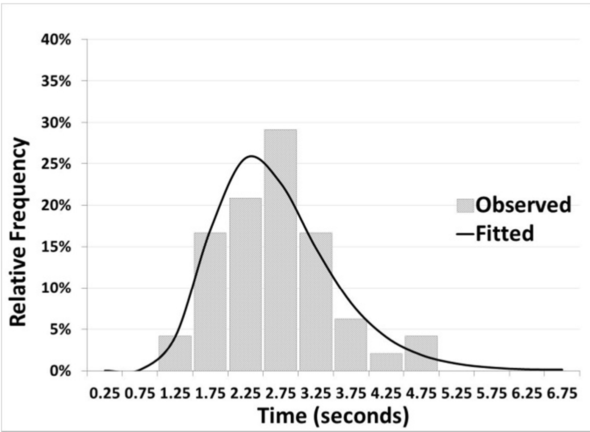
The distribution of move-up delay for both dry and wet situations and for both primary and secondary streams fit the lognormal distribution as shown in Figure 4.9 and the statistical results are summarised in Table 4.14. In the current study, drivers will be assigned a move-up delay as part of their characteristics before entering the model based on the lognormal distribution. Figure 4.10 show the cumulative frequency for the MUD for both dry and wet surface conditions and for both the primary and secondary streams.



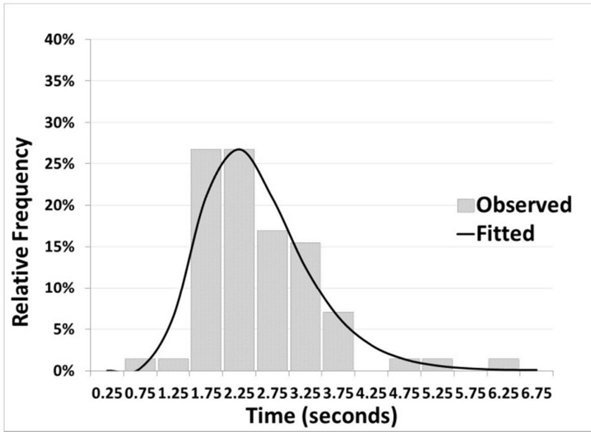
(a) Dry Surface – Primary



(b) Dry Surface – Secondary



(c) Wet Surface-Primary



(d) Wet Surface-Secondary

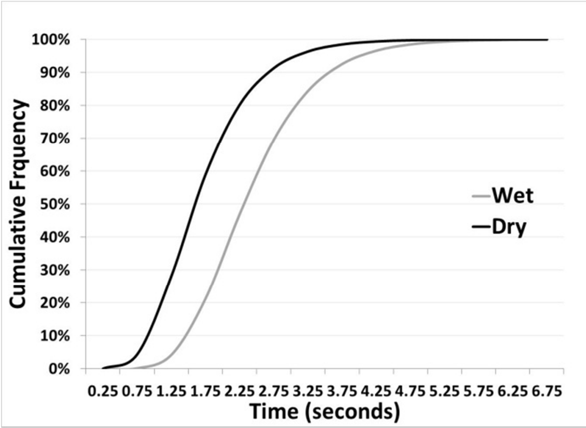
Figure 4.9: Distribution of move-up delay for shuttle-lane roadworks

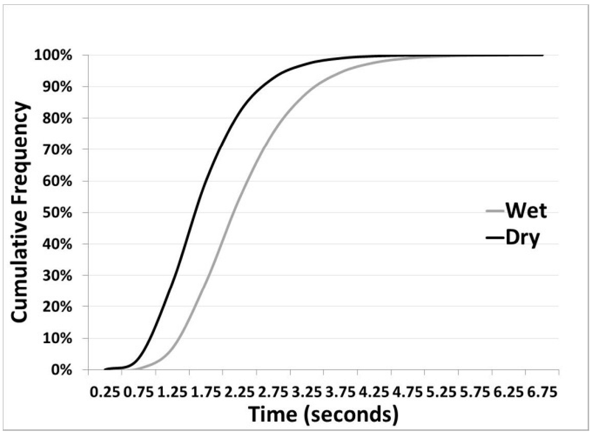
Table 4.14: Summary of statistics for move-up delay distribution fitting

Road Type	Road Condition	Distribution	Sample Size	D_{cr}	D_{max} (lognormal)	Accept
Urban-Roadworks	Dry	Lognormal	(P) 510	0.085	0.053	Yes
			(S) 411	0.095	0.033	Yes
	Wet		(P) 48	0.278	0.052	Yes
			(S) 71	0.228	0.038	Yes

4.9 Vehicle length

In urban areas, there are various types of vehicles ranging from motorcycles, to heavy goods vehicles (HGVs). The dimensions and mechanical abilities between HGVs types and also over the years are different. Vehicle length is one of the factors that is considered in the calculation of acceleration/deceleration rates of the car following rule and also the estimation of gaps required, etc.





(a) Primary Stream (b) Secondary Stream

Figure 4.10: Cumulative frequency MUD for shuttle-lane roadworks

A study was carried out by El-Hanna (1974) on UK motorways which classified the vehicles into two types, namely passenger cars and HGVs based on empirical data. El-Hanna reported that vehicle length is normally distributed with mean and standard deviation as reported in Table 4.15. Based on the assumption of normality, these results will produce unrealistically short length of HGVs. Chin (1983) found different results with HGVs mean length of 6.8m.

Table 4.15: Vehicle classification (Source: El-Hanna, 1974)

Vehicle Type	Cars	HGVs
Mean (μ)	4.2	11.2
Standard deviation (σ)	0.4	2.4

The classification of vehicles into two types (cars and HGVs) was carried out because of the difficulty in obtaining standard values for both vehicle length and acceleration/deceleration for each vehicle type. In the current study the distribution of cars' length were obtained from the M25 and M42 motorways IVD field data because of the availability and accuracy and large sample size of the obtained data. The data consisted of 5,338,769 vehicles that were analysed using a database. It was found that cars range from 2.3m to 5.6m. Table 4.16 shows statistical summary of the car data which shows a good agreement with El-Hanna (1974).

Table 4.16: Vehicle classification based on UK motorway data (M25 and M42)

Vehicle Type	Mean (μ)	SD (σ)	Min	Max	Sample
Cars	4.2	0.5	2.3	5.6	5,338,769

Car length distribution fits a truncated (bounded) normal distribution as shown in Figure 4.11 and the car length cumulative distribution is shown in Figure 4.12. The truncated normal distribution has been used by many researchers and studies of various types of highway such as motorways, merging sections, parking (Zarean, 1987; Purnawan, 2005; Zheng, 2003; Wang, 2006 and Al-Jameel, 2012). Therefore, for the distribution of cars, a truncated normal distribution will be used with the statistical values shown in Table 4.16.

For HGVs, the data was collected from all surveyed sites for various HGVs types as classified by the Department for Transport (2006) and shown in Table 4.17, with the sample size (which were obtained from urban roadworks sites) and average length for each type (which was obtained from HGVs manufacturers that follows the EU length standards). Al-Obaedi (2012) and Westhaven Worldwide Logistics (2012) have reported that typical manufacturer's data sources have been investigated and the minimum length of HGVs is 5.6m. Therefore, this value was used to distinguish between cars and HGVs. The distribution of HGVs does not follow any distribution as shown in Figure 4.13. Therefore, in the current study, the value of HGVs length will be generated from a cumulative frequency curve as shown in Figure 4.14 with the statistical values shown in Table 4.18 and will be used as an input into the micro-simulation model.

Table 4.17: HGVs classification based on urban roads

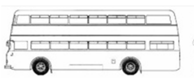
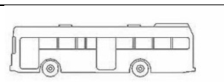
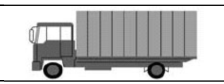
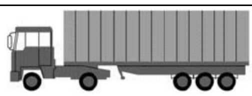
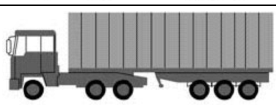
Vehicle Type	Illustration	Overall length (m)	Sample size	Source
Bus-double decker		10	39	TAN BP1/06
Bus-single decker		12	55	TAN BP1/06
2-axle rigid		5.6-10.0	217	www.volvotrucks.com
3-axle (rigid and articulated)		7.6-11.0	42	
4-axle (rigid and articulated)		9.5-11.5	17	
5-axle articulated		15.5 (max)	19	SN/BT/654
6 (or more) -axle articulated		16.5 (max)	5	

Table 4.18: HGVs length statistical summary based on UK urban roadworks sites

Vehicle Type	Mean (μ)	SD (σ)	Min	Max	Sample
HGVs	9.5	1.9	5.6	16.5	394

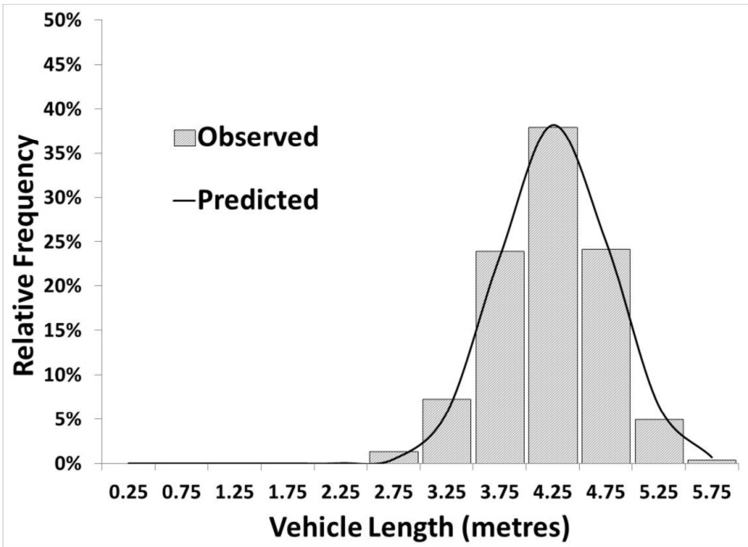


Figure 4.11: Frequency car length distribution based on UK motorway (M25 and M42)

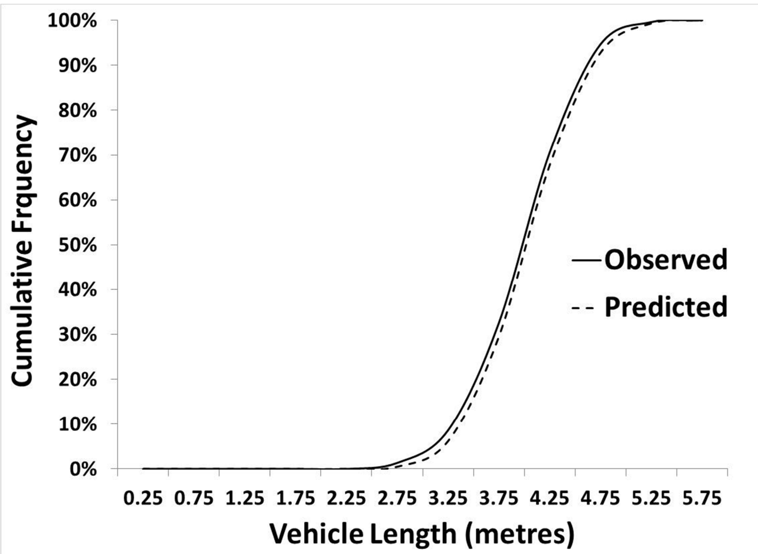


Figure 4.12: Cumulative car length distribution based on UK motorway (M25 and M42)

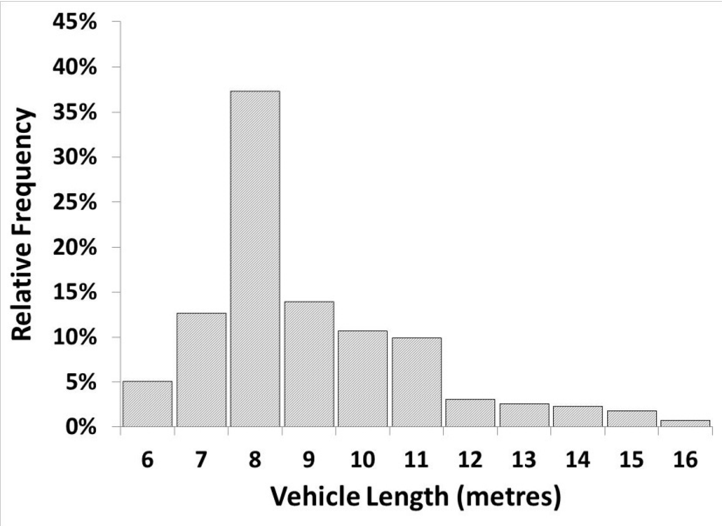


Figure 4.13: Frequency distribution for HGVs length based on urban roadworks sites

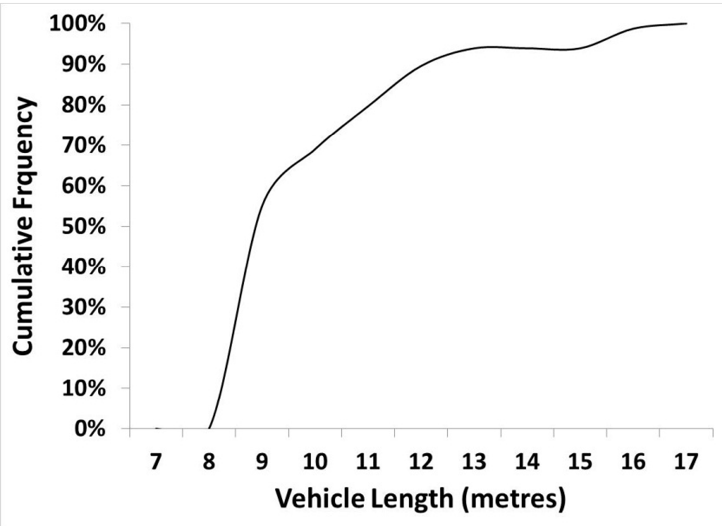


Figure 4.14: Cumulative distribution for HGVs length based on urban roadworks sites

4.10 Signals settings

Traffic signals information and settings have been collected on site and also from video playbacks using an event time recorder to record various phases and times. These signals settings are summarised in Table 4.19 along with the recommended design standard values.

4.10.1 Signals settings

It can be seen from Table 4.19 that FT traffic signals were used at sites 16 and 19. The Department for Transport (2008) clearly states that the signals control should always be VA unless agreed otherwise by the traffic authority to use other modes (i.e. FT or manual control) to relive short term difficulties. It was also observed on the FT sites that the system was not used for short term as the sites were surveyed over various days and different periods. It was also noticed that for the primary stream of Site 19, the green time was not used according to the recommended maximum green time as specified by the Department for Transport (2008) and also different green for each stream were used.

For the VA sites, it was observed that none of the sites followed the maximum green time recommended by the Department for Transport (2008) which is also summarised in Table 4.19. Green time was extended to reach a value of 88 seconds (Site 12) which is clearly higher than what is recommended by the Department for Transport (2008) design guidelines. This extension occurred when the maximum recommended green time was reached for a certain stream (i.e. primary stream) without the presence of any detected vehicle on the opposite stream (i.e. secondary stream), and it continued to be extended until a vehicle was detected on the other stream (i.e. secondary stream).

No reference was made to such a high green time in any of the design standards for shuttle-lane roadworks (i.e. such as the Department for Transport (2008, 2009, 2011); Highways Agency (2005A, 2005B)). Temporary traffic signals (signals controllers) have a built-in maximum green up to 90 seconds. This information has been provided by temporary traffic signals manufacturers (see for example A-Plant LUX, 2013 and Pike Signals, 2013).

It was also noticed that minimum green time used at all sites are 12 seconds (which will be used regardless if a vehicle is detected or not). Also, when there are 1 or 2 vehicles queuing at the traffic light on one stream (i.e. primary stream) with no further vehicles approaching the site from the same stream, the phase will run for a minimum of 12 seconds before it terminates. This will increase the amount of lost time and reduce site capacity for the opposite stream (in the case of vehicles already queuing in the opposite stream). It can be improved by using a minimum green time of 7 seconds as stated by the Highways Agency (2005A) and the Highways Agency (2005B) that the minimum green could be configured to either 7 or 12 seconds.

The recommended all-red period by the Department for Transport (2008) was only implemented on sites 12 and 19. The observed values on site are higher than the design standards, although there is a safety margin accounted for in the design standard values. This increase in all-red period might improve safety of the shuttle-lane site by ensuring that all amber crossing/red light runners cleared the site safely, but it will reduce site capacity by increasing the cycle time. This could possibly lead to an increase in queues and drivers' frustration resulting in more violations (i.e. red light running).

Table 4.19: Summary of signals settings for each site

Site	Direction	L (m)	Type	CT (sec)		GT (sec)		GT-DS (sec)	AR (sec)	AR-DS (sec)	Amb, RA (sec)
				min	max	min	max	max		min	
11	P	42	VA	40	152	12	*60	35 (×)	*3	5 (×)	3,2
	S					12	*48				
12	P	107	VA	74	154	12	*88	40 (×)	20	15 (✓)	3,2
	S					12	*60				
16	P	52	FT	60	60	20	20	35 (✓)	*5	10 (×)	3,2
	S					20	20				
17	P	39	VA	40	118	12	*54	35 (×)	*3	5 (×)	3,2
	S					12	*72				
18	P	73	VA	44	128	12	*76	35 (×)	*3	10 (×)	3,2
	S					12	*78				
19	P	38	FT	115	115	*50	*50	35 (× for S and ✓ for P)	10	5 (✓)	3,2
	S					35	35				

L: site Length VA: Actuated signals FT: Fixed Time signals CT: Cycle Time
GT: Green Time AR: All-Red period Amb: Amber period RA: Red Amber
DS: Design Standards (×) and * not following design standards (✓) following design standards

4.10.2 VA detection failure

Vehicle detection errors were observed on all sites operated by VA signals on 10 occasions (10 out of 956 cycles operated by VA signals). The detection failure is caused by failure in the MVD unit (Microwave Vehicle Detector). This error in detection was observed and reported in Appendix B. In few occasions, the detection error caused drivers to violate the temporary traffic signals, which is explained in details in the next section.

4.11 Drivers' compliance

Various types/categories of amber crossing/red light running violations were observed on shuttle-lane roadworks sites operated by temporary traffic signals. These were categorised under four main headings as explained in the following sections and as shown in Table 4.20.

Table 4.20: Categories of observed amber crossing/red light violations at temporary traffic signals

<table>
<tr><th colspan="4">Violation type</th></tr>
<tr><th>Category 1:
Dilemma Zone (DZ)</th><th>Category 2:
DZ follower</th><th>Category 3:
Group violations</th><th>Category 4:
Single violation</th></tr>
<tr><th colspan="4">Definition</th></tr>
<tr><td>Drivers choose to cross on amber/red light due to the presence of dilemma zone (DZ) as explained in Section 2.6.9.2 and illustrated in Figure 2.10.</td><td>Drivers choose to follow a leader that crossed on amber/red light due to the presence of DZ as explained in Section 2.6.9.2 and illustrated in Figure 2.10.</td><td>Drivers choose to violate the red light because of the frustration / long waiting time due to vehicle detection error.</td><td>Drivers choose to violate the red light because of the available opportunity / gap.</td></tr>
<tr><th colspan="4">Factors affecting the decision</th></tr>
<tr><td>⇨ Vehicles approaching the temporary signals at the time that the light changes from green to amber.
⇨ Crossing on amber/red light is violated (could either deliberately or not deliberately).
⇨ Occurs on both saturated and unsaturated cycles (could happen in both FT and VA signals and for both good and bad visibility).</td><td>⇨ Vehicles are following amber crossing/red light violator (crossed because of the presence of dilemma zone).
⇨ Occurs on both saturated and unsaturated cycles (could happen in both FT and VA signals and for both good and bad visibility).
⇨ Occur in both single and group violations.
⇨ Crosses the amber/red light deliberately.</td><td>⇨ Detection failure: the drivers in the violated stream suffer long delays due to long red phase caused by green phase extension in the opposite stream (happens in VA signals, good visibility and with no vehicles in opposite stream at the time of the violation).
⇨ Usually occurs in group violations (3 cars or more).</td><td>⇨ Occurs in a single violation were a vehicle stops at the stop line for a very short time (less than 10 seconds) and once the opposite stream is clear, the driver violates the red light and crosses the site (usually happens in good visibility and with no vehicles in opposite direction).
⇨ Run the red light deliberately.</td></tr>
</table>

4.11.1 Category 1: Dilemma Zone (DZ)

Observations from several shuttle-lane sites suggest that some drivers choose to cross on amber/red lights caused by the presence of dilemma zone. This behaviour is referred to here as Category 1. Table 4.21 summarises the amber crossing/red light violations that were observed from each site and each stream separately for this category.

Table 4.21 shows that out of the 1,484 signal cycles, vehicles passed the stop line on 232 (15.6%) cycles when the lights show amber while 97 (6.5%) drivers violated the cycles by crossing within red phase. These percentages (in the last column) underestimate non-compliance behaviour as the total number of cycles includes cycles with no approaching vehicles at the onset of amber.

Although the total observed number of cycles operating under VA (956 cycles) is higher than for FT (528 cycles), it can be seen that 100 vehicles (10.5%) crossed the stop line on the onset of amber/red on VA signals while the number of vehicles that crossed the stop line on the onset of amber/red was 229 vehicles (43.4%) for FT signals. This indicates that the VA system performs better in reducing the number of vehicles that crosses on both amber/red in the presence of the dilemma zone. These results are in agreement with Puan and Ismail (2010). Over all, amber crossing for the primary stream is almost equal to the secondary stream with a total of 167 (22.5%) and 162 (22.8%), respectively.

It is believed that the percentage of the total vehicles crossing on amber and red (22.2%) should be higher as the total number of cycles (1,484) includes empty cycles. Drivers in empty cycles do not have a decision to make whether to cross on amber or stop (i.e. because they arrive late after the other stream approached or no vehicles arrived at the stop line on the onset of amber).

Table 4.21: Amber crossing/red light violation for Category 1 (DZ)

Site	Direction	Type	No of cycles	Amber crossing	Red light violation	Overall crossing (Amber + Red)
11	P	VA	260	38 (14.6%)	11 (4.2%)	49 (18.8%)
	S		260	29 (11.2%)	9 (3.5%)	38 (14.6%)
12	P	VA	80	3 (3.8%)	0 (0%)	3 (3.8%)
	S		80	1 (1.3%)	0 (0%)	1 (1.3%)
16	P	FT	163	36 (22.1%)	18 (11%)	54 (33.1%)
	S		163	49 (30.1%)	16 (9.8%)	65 (39.9%)
17	P	VA	61	0 (0%)	1 (1.6%)	1 (1.6%)
	S		61	0 (0%)	1 (1.6%)	1 (1.6%)
18	P	VA	77	2 (2.6%)	0 (0%)	2 (2.6%)
	S		77	4 (5.2%)	1 (1.3%)	5 (6.5%)
19	P	FT	101	41 (40.6%)	17 (16.8%)	58 (57.4%)
	S		101	29 (28.7%)	23 (22.8%)	52 (51.5%)
Total	**P**		**742**	**120 (16.2%)**	**47 (6.3%)**	**167 (22.5%)**
	S		**742**	**112 (15.1%)**	**50 (6.7%)**	**162 (22.8%)**
Total	**VA**		**956**	**77 (8.1%)**	**23 (2.4%)**	**100 (10.5%)**
	FT		**528**	**155 (29.4%)**	**74 (14%)**	**229 (43.4%)**
Total			**1,484**	**232 (15.6%)**	**97 (6.6%)**	**329 (22.2%)**

4.11.2 Category 2: Dilemma zone follower

According to previous research on dilemma zone, researchers focused on the effect of dilemma zone on the leading vehicle (if the driver will stop or cross and the ways to reduce the impact of dilemma zone) without paying much attention to vehicles that are following a leading vehicle in the dilemma zone.

In urban shuttle-lane roadworks operated by temporary traffic signals, it was noticed that when the leading vehicle (leader) decided to cross the stop line (amber crossing or red light violation) due to the presence of dilemma zone (DZ), the following vehicles may also make a decision to stop or cross the stop line following the leader. This will increase the risk of rear-end collisions or near accidents.

The observed dilemma zone follower violations (Category 2) in urban roadworks were clearly noticed during over saturated cycles with long queues, which possibly triggered drivers' frustration. According to Bonneson and Zimmerman (2004), drivers' frustration due to congestion and delays are the main factors that influence the decision of deliberate red light running (RLR) for drivers which is directly related to volume/capacity ratio. Porter and Berry (2001) found that being in a hurry was the most important factor affecting RLR.

Table 4.22 summarises the amber crossing/red light violations that were observed from each site and for each stream separately for Category 2 (DZ followers). It can be seen from Table 4.22 that that during the 329 cycles representing Category 2 violations, 95 following drivers decided to violate (cross the stop line) on either amber or red lights in 82 (24.9%) cycles. It can also be seen that sites operated by VA signals have DZ followers in 8 (8%) of the violated cycles while sites with FT signal have DZ followers in 74 (32.3%) cycles. This also indicates the effectiveness of the VA signal control in reducing the DZ followers.

Table 4.22 also shows that the primary stream has DZ followers in 48 (28.7%) of the violated cycles while the secondary stream has 34 (21%) cycles with DZ followers. This difference may be attributed to the fact that there are slightly fewer observed cycles for the secondary stream compared with the primary. However, it could be argued that for vehicles in the primary stream, they have to change their horizontal trajectory (to the opposite lane) to negotiate the roadwork site layout and the decision of stopping on the stop line becomes more difficult while they are deciding whether to cross rather than stop.

It was also observed from site that the number of DZ followers varies between one and five vehicles in each cycle. The distribution of the number of vehicles involved in Category 1 and Category 2 violations are summarised in Table 4.23 and shown in Figure 4.15.

Table 4.22: Amber crossing/red light violation for Category 2 (DZ followers)

Site	Direction	Type	No of cycles in Category 1	Amber crossing		Red light violation		Overall crossing (Amber + Red)	
				V	C (%)	V	C (%)	V	C (%)
11	P	VA	49	1	1 (2)	2	2 (4.1)	3	3 (6.1)
	S		38	0	0 (0)	2	2 (5.3)	2	2 (5.3)
12	P	VA	3	0	0 (0)	0	0 (0)	0	0 (0)
	S		1	0	0 (0)	0	0 (0)	0	0 (0)
16	P	FT	54	6	4 (7.4)	3	3 (5.6)	9	7 (13)
	S		65	1	1 (1.5)	0	0 (0)	1	1 (1.5)
17	P	VA	1	0	0 (0)	0	0 (0)	0	0 (0)
	S		1	0	0 (0)	0	0 (0)	0	0 (0)
18	P	VA	2	2	2 (100)	0	0 (0)	2	2 (100)
	S		5	1	1 (20)	0	0 (0)	1	1 (20)
19	P	FT	58	19	19 (32.8)	28	17 (29.3)	47	36 (62.1)
	S		52	18	18 (34.6)	12	12 (23.1)	30	30 (57.7)
Total	**P**		**167**	**28**	**26 (15.6)**	**33**	**22 (13.2)**	**61**	**48 (28.7)**
	S		**162**	**20**	**20 (12.3)**	**14**	**14 (8.6)**	**34**	**34 (21)**
Total	**VA**		**100**	**4**	**4 (4)**	**4**	**4 (4)**	**8**	**8 (8)**
	FT		**229**	**44**	**42 (18.3)**	**43**	**32 (14)**	**87**	**74 (32.3)**
Total			**329**	**48**	**46 (14)**	**47**	**36 (10.9)**	**95**	**82 (24.9)**

V: number of Vehicles C: number of Cycles

Table 4.23: Frequency of DZ and DZ follower (Category 1 and Category 2)

Category	Number of vehicles crossing (Amber/Red)	Frequency	Number of vehicles	Relative Frequency
1	1	329	329	78%
2	2	30	60	14%
	3	6	18	4%
	4	3	12	3%
	5	1	5	1%
Total			**424**	**100%**

4.11.3 Category 3: Group violations

In this category, the observed deliberate red light violations in urban roadworks were due to drivers' frustration and were mainly attributed to a long red phase in the stopped stream due to failure in the MVD equipment (Microwave Vehicle Detector) and the continuous green phase from the opposite stream (exceeding the maximum due to a double or triple green phase). This usually happens in multiple violations, VA signals, good visibility and when there are no vehicles in the opposite stream.

Vehicle detection error was observed from sites operated by VA signals on 10 occasions (10 out of 956 cycles operated by VA signals) and when the red phase exceeded 75 seconds. In two out of the 10 cases in which MVD failures occurred, drivers decided to violate the red lights in group violations due to the presence of long queues and delays (violations occurred in groups of 3 and 4 vehicles).

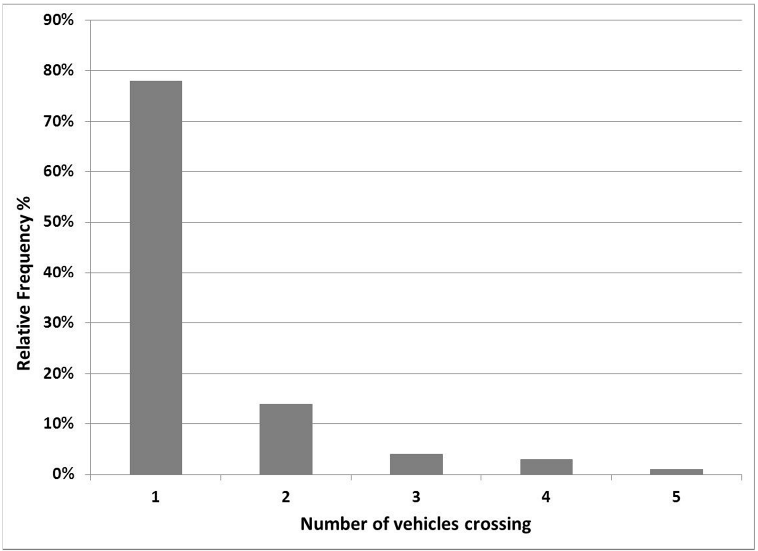


Figure 4.15: Distribution of DZ and DZ follower (Category 1 and Category 2)

4.11.4 Category 4: Single violation

The observed single violations usually occur when a single vehicle briefly waits at the stop line at the temporary traffic lights (vehicle waiting time is less than 10 seconds) and deliberately decides to violate the red lights before the green phase starts. This type of violation usually occurs when the visibility is good (i.e. a driver can see the opposite stream) and there are no vehicles approaching from the opposite stream. Table 4.24 summarises the observed red light violations for each site and for each stream separately for Category 4 (single violation).

It can be seen from Table 4.24 that 21 cycles (1.4%) out of 1,484 cycles were violated by a single crossing and all violations occurred on sites with VA signals and when visibility was good. Also, the single violation for the primary stream is almost equal to the secondary stream with total violations of 12 (1.6%) and 9 (1.2%), respectively.

Table 4.24: Red light violation for Category 4 (single violation)

Site	Direction	Type	Visibility	No of cycles	Red crossing
11	P	VA	Good	260	8 (3.1)
	S			260	4 (1.5)
12	P	VA	Bad	80	0 (0)
	S			80	0 (0)
16	P	FT	Bad	163	0 (0)
	S			163	0 (0)
17	P	VA	Good	61	3 (4.9)
	S			61	2 (3.3)
18	P	VA	Good	77	1 (1.3)
	S			77	3 (3.9)
19	P	FT	Bad	101	0 (0)
	S			101	0 (0)
Total	**P**			**742**	**12 (1.6)**
	S			**742**	**9 (1.2)**
Total	**VA**			**956**	**21 (2.2)**
	FT			**528**	**0 (0)**
Total				**1,484**	**21 (1.4)**

Good visibility: drivers from one stream can see the vehicles from the opposite stream
Bad visibility: drivers from one stream cannot see the vehicles from the opposite stream

4.11.5 Summary of observed red light violations

This section summarises the overall observed amber crossing and red light violations and also compares violations at roadworks to signalised junction from previous studies. Table 4.25 summarises the amber crossing and red light violations by category and their frequency are shown graphically in Figure 4.16.

It can be seen from Table 4.25 and Figure 4.16 that the dilemma zone (DZ) has the highest impact on drivers' decision to cross on amber or red lights in shuttle-lane roadworks with a relative frequency of 82.9% and 58.1% for amber crossing and red light violations, respectively. Vehicles following the DZ vehicles that decided to cross are the second factor/category with a relative frequency of 17.1% and 28.1% for amber crossing and red light violations respectively. The third highest category is the single violation which contributes to 4.7% of the overall violations while the group violations caused by MVD error consist of only 0.4% of the red light violations.

Table 4.25: Amber crossing/red light violation by category

Category	Amber crossing		Red light violation		Overall crossing (Amber + Red)	
	Frequency (cycles)	Relative Frequency (%)	Frequency (cycles)	Relative Frequency (%)	Frequency (cycles)	Relative Frequency (%)
Category 1 (DZ)	232	82.9	97	58.1	329	73.6
Category 2 (DZ follower)	48	17.1	47	28.1	95	21.3
Category 3 (Group violations)	NA	NA	2	1.2	2	0.4
Category 4 (Single violations)	NA	NA	21	12.6	21	4.7
Total	**280**	**100**	**167**	**100**	**447**	**100**

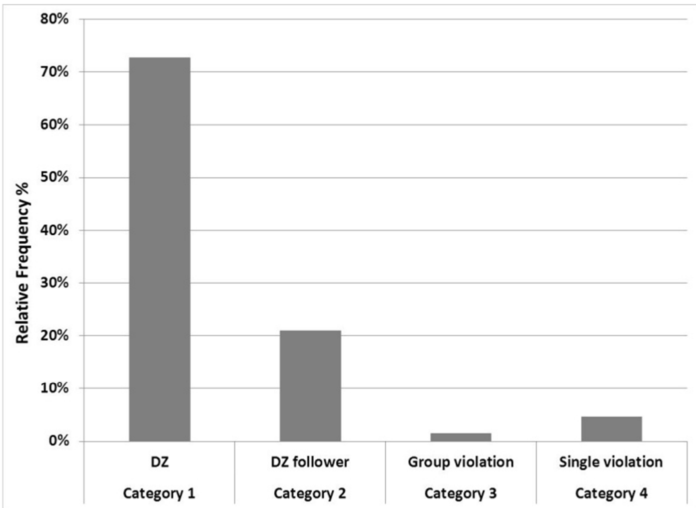


Figure 4.16: Amber crossing/red light violation by category

Red light violations at roadworks are also compared to typical signalised junctions from previous available research and summarised in Table 4.26. It is important to clarify that red light violations are site specific in signalised junctions and therefore, the numbers shown in Table 4.26 are averages sampled from previous available research and could vary from site to site. It is also important to clarify that signalised junctions consist of various arms and vehicles are driving in different directions (left, straight and right) while in shuttle-lane roadworks, vehicles are driving in one direction with one arm for each stream.

It is clearly seen from Table 4.26 that red light violation in signalised junctions are lower (between 2.7% and 5.3%) than at shuttle-lane roadworks (11.3% on average for the current study and ranges between 12% and 30% from a previous study that was carried out by

Samoail and Yousif (1998)). This is deemed to be relatively high and could cause a risk of road accident involving vehicles, pedestrians and workers at the site.

Table 4.26: Comparison of red light violations (typical signals vs. roadworks)

Study	Country	Type	Total Cycles	Red light violations
Wei *et al*. (2010)	USA	Traffic signals	1,601	77 (4.8%)
Bonneson and Zimmerman (2004)	USA		11,266	595 (5.3%)
Koll *et al*. (2004)	Europe		4,997	133 (2.7)
Samoail and Yousif (1998)	United Kingdom	Roadworks	12% – 30%	
Current study			1,484	167 (11.3%)

4.12 Signage

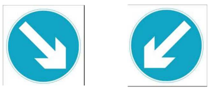
Signage is another important element of urban shuttle-lane roadworks as it provides drivers with warnings about oncoming hazards or change of road layout. The Department for Transport (2011) sets the required signs and their setting distances for each type of shuttle-lane roadworks based on previous research. Failure to comply with the design standards will create unnecessary risks to drivers. Table 4.27 lists the required signs at each type of shuttle-lane roadworks.

According to the Department for Transport (2011), the minimum and normal maximum setting distance for the first sign in advance of the lead-in taper (first cone) should be between 20 and 45 metres. It also states that all signs should be visible to approaching drivers with a minimum clear visibility of the first sign at 60 metres.

For shuttle-lane roadworks controlled by temporary traffic signals, signs 1 to 6 should be used. For Give/Take operation, signs 1, 3 and 4 only should be used. For priority operation, signs 1, 3, 4, 6 and 7 should be used. All signs should be placed according to the Department for Transport (2011).

Tables 4.28 and 4.29 summarise the observed signage for each site and each stream with their setting distances and comparing each direction to the design standards. Sites 1, 8, 9, 10 and 15 are not included in the table as they are not shuttle-lane roadworks sites (i.e. signalised junction or traffic calming sites).

Table 4.27: List of sign for shuttle-lane roadworks

Sign Number	Sign	Description
1		Roadworks ahead
2		Traffic signals ahead (if site is operated by temporary traffic light)
3		Road narrows on left hand side (used in primary stream) Road narrows on right hand side (used in secondary stream)
4		Where vehicles should stop at temporary traffic signals
4		Keep right, keep left
6		End of roadworks
7		Give way to oncoming vehicles Priority over oncoming vehicles

It can be seen from Tables 4.28 and 4.29 that placing signs at roadworks were not carried out correctly according to the design standards at most of the sites. At some sites (i.e. sites 7, 12, 13 and 14), there were missing signs which can cause confusion to drivers approaching the roadworks site and may result in an increase in the risk of collision due to driver hesitation or sudden braking.

It was also observed at most sites (i.e. sites 2 to 7, 11, 14, 15, 78, 18 and19) that signs were not placed according to the recommended design standards (signs should be placed at a distance between 25-50 metres from the first cone). It was also observed that not all signs were visible to drivers at most of the sites (columns 7 in Tables 4.28 and 4.29), were signs were either covered by parked vehicles or had been knocked down.

Table 4.28: Signs and distances for historical roadworks sites

Site	Dir.	Are all signs available	Missing signs	Distance to first sign (m)	According to standards	Are all signs clear to oncoming traffic
2	P	✓	-	260	×	✓
	S	✓	-	285	×	✓
3	P	✓	-	89	×	✓
	S	✓	-	102	×	✓
4	P	✓	-	187	×	✓
	S	✓	-	142	×	×
5	P	✓	-	95	×	✓
	S	✓	-	110	×	✓
6	P	✓	-	70	×	✓
	S	✓	-	92	×	✓
7	P	×	1,2,3	21	×	×
	S	×	all	26	×	×

Table 4.29: Signs and distances for current roadworks sites

Site	Dir.	Are all signs available	Missing signs	Distance to first sign (m)	According to standards	Are all signs clear to oncoming traffic
11	P	✓	-	55	×	✓
	S	✓	-	68	×	✓
12	P	×	1,3	40	✓	✓
	S	✓	-	44	✓	✓
13	P	×	1,7	25	✓	×
	S	×	1,3	20	✓	×
14	P	✓	-	30	✓	×
	S	×	all	0	×	×
16	P	✓	-	54	×	✓
	S	✓	-	42	✓	✓
17	P	✓	-	89	×	×
	S	✓	-	114	×	×
18	P	✓	-	93	×	×
	S	✓	-	99	×	×
19	P	✓	-	54	×	✓
	S	✓	-	42	✓	✓
20	P	✓	-	65	×	×
	S	✓	-	52	×	×
21	P	✓	-	32	✓	✓
	S	✓	-	28	✓	✓
22	P	✓	-	59	×	✓
	S	✓	-	47	✓	✓
23	P	✓	-	48	✓	✓
	S	✓	-	30	✓	✓

4.13 Summary

This chapter presented the data analysis stage which is performed on the data collected from shuttle-lane roadworks sites and other various sources. The analysed data was used in developing, calibrating and validating the micro-simulation model as described in Chapter 5 and 6.

- ⇨ Video recordings for over 54 hours (23 sites) were used to analyse the various information of shuttle-lane roadworks.
- ⇨ Video recordings were used to extract various sets of information such as flow level and profile, following headway (close following "tailgating"), Move-up time (MUT) and Move-up delay (MUD).
- ⇨ Data taken from over 5.3 million IVD data records were used to calculate car length and model the probability distribution of car length. HGVs data collected from both visited sites and manufacturer catalogues of vehicles and were used to calculate the HGVs length and distribution.
- ⇨ Signals type and settings were collected and analysed using video recordings and onsite observations.
- ⇨ Drivers' compliance with temporary traffic signals was collected using video recordings. Information regarding drivers' compliance included drivers crossing through amber and red light violations per cycle.
- ⇨ Site observations using a measuring wheel were used to collect signage information and distances at all shuttle-lane roadworks sites.

CHAPTER FIVE: LIMITATIONS OF THE S-PARAMICS MICRO-SIMULATION MODEL

5.1 Introduction

The current chapter describes the development process of the S-Paramics micro-simulation model for studying urban roadworks and specifically, shuttle-lane roadworks operated by temporary traffic signals. The chapter also describes the calibration, validation and limitations of the S-Paramics micro-simulation model.

S-Paramics is a micro-simulation software package capable of representing the behaviour and interaction between individual vehicles on the road network. Different road layouts and features may be simulated and drivers' behaviour characteristics can be changed relatively easily as part of the calibration and validation of the model to replicate actual site observations.

The S-Paramics also provides outputs and presents real-time visual displays for various traffic management and road network designs. Vehicle dynamics (i.e. acceleration and deceleration rates and vehicle dimensions) can also be changed to represent real observations (SIAS Limited, 2007). In the current study, the S-Paramics 2010.1 was used to develop, calibrate and validate shuttle-lane urban roadworks micro-simulation model operated by FT signals as discussed in the following sub-sections.

5.2 Development of the S-Paramics model

Geometric layout using AutoCAD drawing was established using aerial photographs before the model building stage commenced. A model for Site 16 was built and a model screen shot layout is shown in Figure 5.1.

The S-Paramics model consists of nodes and links, with each node carrying different characteristics, such as the type of traffic control and sight distance. Links carry the characteristics of traffic and geometric design (e.g. speed, visibility, number of lanes, directional movement, etc.).

Below is a description of the various assumptions made in the S-Paramics model:

- ⇨ The model covers a 2 hours period (AM period);

- ⇨ The first and last 30 minutes in the model are used as the warming up and cooling down periods, respectively;
- ⇨ Traffic is profiled for the whole of the 2-hour period with 5-minute profile details (representing the observed arrival rates of vehicles every 5 minutes);
- ⇨ Two vehicle classes were modelled namely, cars and HGVs with their corresponding proportions.

Temporary traffic signals were coded in the S-Paramics model based on observed phases and values. All the model default values were used at the start of the verification and calibration processes (e.g. minimum gap, headway, visibility, acceleration and deceleration rates, awareness and aggression level). As part of the model calibration, these values can be amended to represent any observed surveyed values/behaviour on site as described in details in the following section.

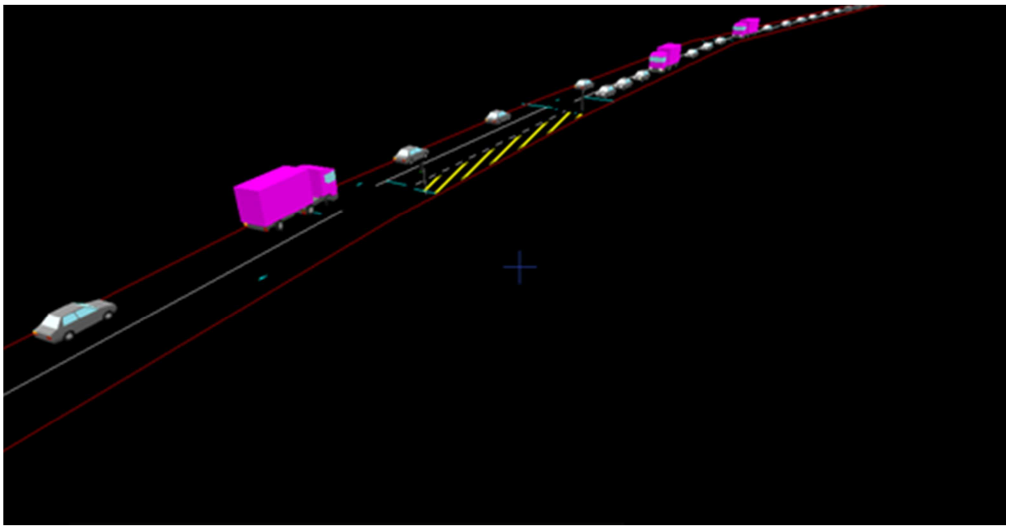


Figure 5.1: Shuttle-lane roadworks using S-Paramics

5.3 Statistical tests

Statistical goodness-of-fit measurements (tests) were carried out between the observed and simulation output data for calibration and validation purposes. In addition, graphical representation was also produced. Two goodness-of-fit measures were introduced, explained below and were used in the model calibration and validation for comparison between observed and modelled results.

5.3.1 Root Mean Square Error Percentage (RMSEP)

This test is considered to be a good initial test to make a comparison between empirical and simulated data because it penalises high errors at higher rates than small errors (Toledo, 2003). This test has been used in many simulation studies (see for example Wu *et al*. (2003), Panwai and Dia (2005), Wang (2006), Al-Jameel, (2012) and Al-Obaedi (2012)) and is represented in Equation 5.1.

$$\text{RMSEP} = \sqrt{\frac{1}{\text{n}}\sum_{\text{i}=1}^{\text{n}}\left(\frac{\text{xi}-\text{yi}}{\text{xi}}\right)^2} \qquad \text{Equation 5.1}$$

Where,

n is the number of time intervals
xi is the observed flow at time interval i;
yi is the simulated flow at time interval i.

5.3.2 Geoffrey E. Havers (GEH)

The GEH statistical test was developed by the Department for Transport in 1996. The test is used to compare two sets of readings (modelled and observed) in order to test the validity of the model. The test (which is similar to the Chi-squared statistic) is widely used and recommended by the Department for Transport (1996) and is represented in Equation 5.2.

$$\text{GEH} = \sqrt{\frac{2(\text{xi}-\text{yi})^2}{\text{xi}+\text{yi}}} \qquad \text{Equation 5.2}$$

According to Hourdakis *et al*. (2003), satisfactory model results will be achieved if RMSEP is less than 15%. According to the Design Manual for Roads and Bridges (1996), the GEH should be ≤ 5 for the link flow to be satisfactory. These thresholds are monitored throughout the calibration/validation process to ensure acceptable model quality along with other measures.

5.4 Calibration and validation of the S-Paramics model

Following the S-Paramics model building stage, various calibration parameters were used to obtain the best results (i.e. headway factor, mean time headway and minimum space) and default S-Paramics values were used for other unavailable values.

Although S-Paramics can model complicated control operations such as road narrowing using throttle and could also replicate complicated drivers' behaviour such as cooperative behaviour as was carried out in a study by Yousif *et al*. (2013). S-Paramics cannot correctly replicate shuttle-lane rules such as drivers' behaviour in dilemma zone or amber crossing/red light violations and also no Move-up time (MUT) information could be extracted. Therefore, only flow, throughput, headway and queues will be provided in this section and compared with observed data with no amber crossing/red light violations results.

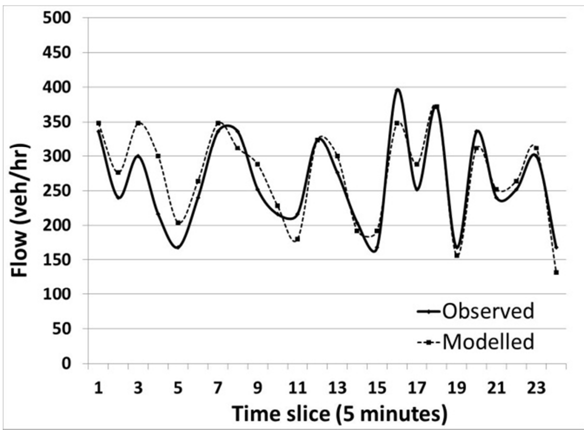
The statistical results are presented in Table 5.1 for flow and throughput for each 5-minutes interval and shown graphically in Figure 5.2. Table 5.2 shows the output results for following headway values and Table 5.3 shows the queue results.

Table 5.1: S-Paramics model statistics - flow and throughput (Site 16a)

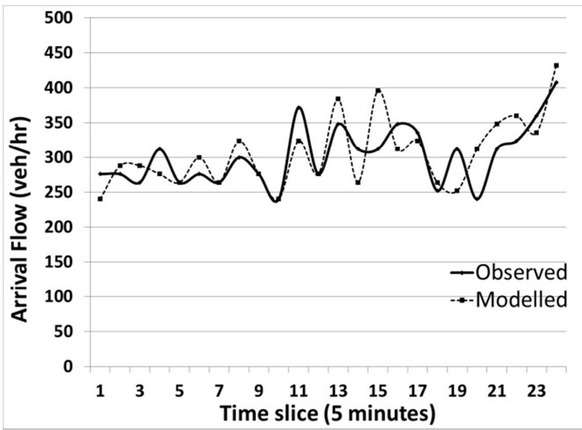
Statistical Test	Primary Stream		Secondary Stream	
	Flow	Throughput	Flow	Throughput
RMSEP %	15.9	18.4	14.5	18.9
GEH	2.64	3.26	2.72	3.44

Table 5.2: S-Paramics model statistics - headway (Site 16a)

Headway criteria	Location	Primary Stream			Secondary Stream		
		Observed	Modelled	Diff.	Observed	Modelled	Diff.
Avg. Headway (sec)	(BAR)	3.46	3.20	-0.26	3.15	3.08	-0.07
Avg. Headway (sec)	(ACR)	2.49	2.77	0.28	2.56	2.42	-0.14
< 2.0 (%)	(BAR)	13%	10%	-3%	17%	9%	-8%
< 2.0 (%)	(ACR)	38%	27%	-11%	33%	34%	1%
≤ 6.0 (veh)	(BAR)	242	220	-22	284	321	37
≤ 6.0 (veh)	(ACR)	355	398	43	423	459	36



(a) Primary Stream



(b) Secondary Stream

Figure 5.2: S-Paramics model vs. observed flow (Site 16a)

It can be seen from Table 5.1 that the S-Paramics model fails statistically to replicate both flow and throughput for the primary stream and throughput for the secondary stream according to RMSEP % results. This indicates that a great deal of care should be taken in selecting the default values when using S-Paramics to represent shuttle-lane roadworks behaviour. Although the S-Paramics GEH results are within the acceptable limits, throughput per cycle could not be represented correctly due to the amber crossing/red light violations (drivers' decisions) observed on site which could not be replicated accurately by S-Paramics.

Results of the average time headway for vehicles in platoons (following headway ≤ 6 seconds) were compared between observed and modelled data for vehicles in both situations (i.e. BAR and ACR) as shown in Table 5.2. It can be seen from Table 5.2 that the average time headway between real observed data and simulation model output for both streams and for both situations (BAR and ACR) are in good agreement with a maximum difference of less than 0.28 seconds for all situations.

It can also be seen from Table 5.2 that the percentage of drivers violating the two-seconds rule are also in good agreement between modelled and observed data for both streams and all situations with a maximum difference of -11%. The final comparison which is the total number of vehicles in platoons is also in good agreement between modelled and observed on all situations (BAR and ACR) with a maximum difference of 13% (37 vehicles).

Queues are reported and a comparison between the observed and simulation model output is shown in Table 5.3 for each stream separately. According to Dowling *et al*., (2002), maximum queue (in vehicles) is the maximum observed queue in any 5-minutes interval (over the simulation period). It is a useful measure that needs to be observed and compared between real data and the simulation model to indicate if the queues will spill back to the next junction. Average queue in any 5-minutes interval and total queued vehicles over the simulation period are also useful measures to report and compare between observed and modelled data.

It can be seen from Table 5.3 that the differences in maximum queues are -40% and 50% for primary and secondary stream, respectively; while the differences in average queues are -30% and -43.1% for primary and secondary stream, respectively. The differences in the total reported queues over the simulation period are -31.4% and -20.1% for primary and secondary stream, respectively. According to Lee (2008), the validation criterion for the simulated maximum and average queue is to be within ±20% of the observed value. Therefore, it can be concluded that reported simulation queues are not in good agreement with the real observed queues. Therefore, the S-Paramics model fails to replicate observed queues and that the queues statistics obtained from the S-Paramics model constantly under-estimates the queues for both streams.

Table 5.3: S-Paramics model statistics - queues (Site 16a)

Queue Measure	Primary Stream			Secondary Stream		
	Observed	Modelled	Diff.	Observed	Modelled	Diff.
Maximum queue (veh)	15	9	-40.0%	14	7	-50.0%
Average queue (veh)	4.0	2.8	-30.0%	5.1	2.9	-43.1%
Total queued vehicles (veh)	357	245	-31.4%	483	386	-20.1%

5.5 Summary

The current chapter presented the building, calibration and validation of the S-Paramics simulation model using real observed traffic data from surveyed shuttle-lane roadworks site. The results showed that the S-Paramics model fails statistically to replicate both flow and throughput for the primary stream and throughput for the secondary stream according to RMSEP % results. For all time headway statistics, it can be seen that the observed data and S-Paramics simulation model output for both streams and for both situations (BAR and ACR) are in good agreement. For queue statistics, the S-Paramics model fails to replicate observed queues and constantly under-estimates the queues for both streams. In addition, the S-Paramics model could not replicate the aggressive drivers' behaviour of amber crossing/red light violation observed on site and the presence of dilemma zone which has an effect on both safety and capacity.

Therefore, a new micro-simulation model needs to be developed as part of the current study to provide more accurate results. The model should also have the ability to cover S-Paramics limitations (the ability to model aggressive drivers' behaviour and the effect of dilemma zone).

CHAPTER SIX: SIMSUR MODEL SPECIFICATION AND DEVELOPMENT

6.1 Introduction

The current chapter describes the specification and the structure of SIMSUR (SIMulation of Shuttle-lane Urban Roadworks) simulation model for studying urban roadworks and specifically, shuttle-lane roadworks operated by temporary traffic signals. SIMSUR simulation model consists of two sub-models: car-following and shuttle-lane sub-models. Each of these sub-models is discussed in details in this chapter. Micro-simulation technique has been selected in the current study because of its ability to represent the interaction between individual vehicles.

The micro-simulation model development requires information about vehicles, drivers and shuttle-lane roadworks characteristics. It also requires selection and development of suitable algorithms for car following sub-model. These characteristics and rules need to be programmed using a suitable programming language to test the performance of such a model before it could be applied.

Compaq Visual FORTRAN (6.5) programming language was used in the current study to develop the SIMSUR simulation model. FORTRAN language was selected for this purpose because it has been widely used in engineering applications and the current version of visual FORTRAN could provide a visual representation of vehicles' movements and interactions. SIMSUR model was built from scratch for the current study using over 4,700 lines of coding and took around 18 months to be developed.

The determination of scanning time for SIMSUR model was selected based on previous research that was carried out by Yousif (1993). Small scanning time will result in more time and cost, whereas long scanning time may affect the results. Therefore, the default scanning time of 0.5 sec was adopted for this study, which is the typical minimum value of drivers' reaction time (Gipps, 1981).

6.2 SIMSUR model structure

SIMSUR micro-simulation model consists of various sub-models such as car following and shuttle-lane rules. The simplified micro-simulation model structure is shown in Figure 6.1.

The first process in SIMSUR micro-simulation model is to define each driver's and vehicle's characteristics (i.e. assigned speed, desired speed, driver's reaction time, vehicle type and length, etc.). Vehicles will then be generated and assigned into the road network based on their arrival headway. Vehicle information (i.e. current position and speed) will be updated every scanning time (Δt) for the whole simulated road length including the warm-up and cool-off sections. Data will be collected and sent to an output file. The process is carried out for each stream (i.e. primary and secondary) separately. The model will be terminated once the simulation period has been reached which is equal to the total simulation time (T).

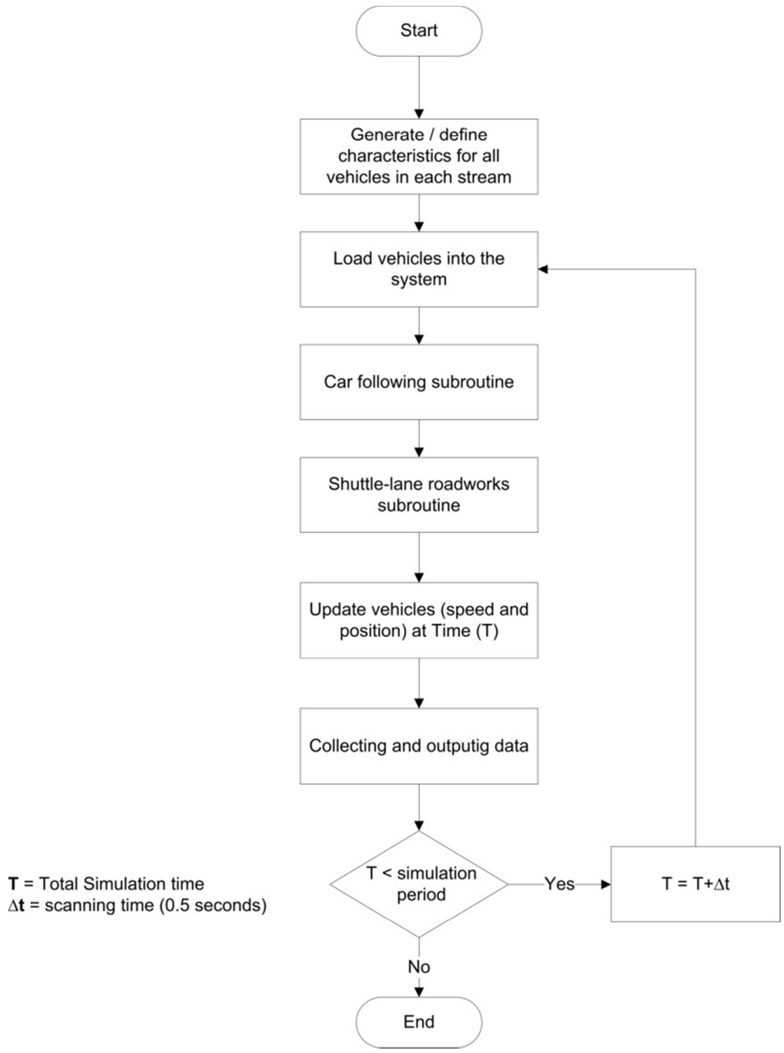


Figure 6.1: General structure of SIMSUR simulation model

6.3 Drivers’ and vehicles’ characteristics

Various drivers’ and vehicles’ characteristics are generated before the vehicle entry into the system. The generations of these characteristics are described in the following sections.

6.3.1 Perception reaction time

Driver reaction time is a very important factor that contributes to the headway value between vehicles. The perception reaction time consists of two components: the time of seeing or receiving the stimulus and the application of the response (O'Flaherty, 1986). According to Green (2000), there are various factors affecting reaction time values (i.e. expectation, urgency, age and gender, cognitive load and testing conditions) and it is impossible to derive a single all-purpose value.

According to Green (2000), participants in the controlled environment drove on either public or private roads (i.e. test tracks) while a researcher sat in the passenger seat. In most occasions, the participants knew that he/she is being tested (but without knowing the real purpose of the study). The natural environment is where the researchers set up digital recording equipment and the drivers' response was measured (the drivers' were unaware of being monitored). The typical natural reaction time study recorded the interval between a yellow traffic signal, brake lights of a leading vehicle (could possibly be driven by a researcher) and onset of the naive driver's brake lights.

Various researchers have attempted to study driver reaction time under various conditions. Table 6.1 summarises the main studies that were carried out in natural and controlled environment.

Table 6.1: Summary of previous studies on drivers' reaction time

Study	Sample size	Min-Max	RT mean and sd	Situation
Chang *et al*. (1985)	1,614	NA	1.30 (0.74)	Surprised
Sivak *et al*. (1982)	1,644	0.65-2.40	1.21 (0.63)	Surprised
Lerner (1994)	56	0.7-2.5	1.51 (0.39)	Surprised
Johansson and Rumer (1971)	321	0.73-2.2	0.90	Surprised
		0.54-1.70	0.69	Alerted

Johansson and Rumer (1971) defined the brake reaction time as representing the perception reaction time. They studied reaction times in both alerted and surprised situations using a sample of 321 drivers and the results (cumulative distribution) are shown in Figure 6.2. Those values are adopted and used in SIMSUR model (minimum of 0.73 and maximum of 2.2 for surprised situation). The conversion factor from surprised to alerted situation is 1.35.

Congested conditions (i.e. density of more than 37 veh/km) are considered to be for the alerted situations as implemented by many previous researchers (Benekohal, 1986; Yousif, 1993; Al-Jameel, 2012; Al-Obaedi, 2012). According to the Department for Transport (2011), the roadworks signs warn (alert) drivers of the oncoming hazard. Therefore, it is assumed that the driver is in an alert situation if he/she approaches the “roadworks start” sign until he/she approaches the “roadworks end” sign.

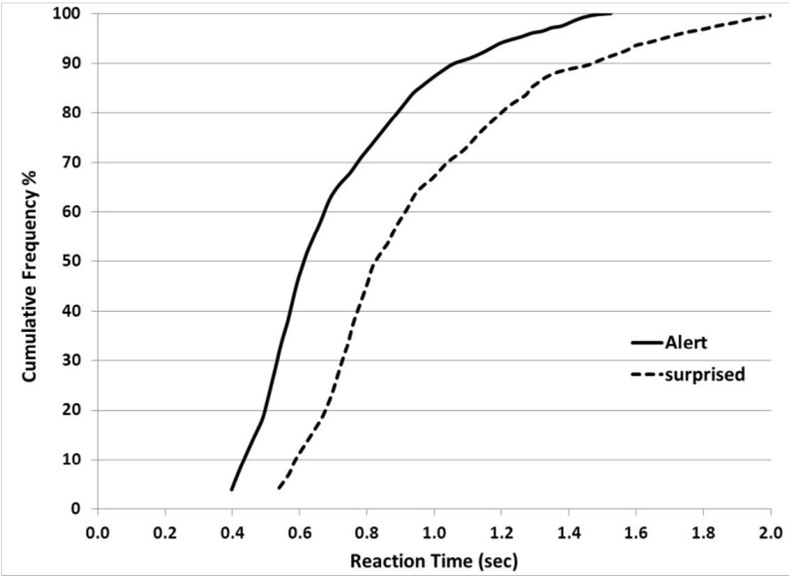


Figure 6.2: Distribution of drivers’ reaction time for alerted and surprised conditions (Johansson and Rumer, 1971)

In the current study, drivers' reaction times were obtained from Figure 6.2 cumulative distribution by generating random numbers from a uniform distribution. The random numbers were set to be equal to the cumulative distribution as was modelled by others (e.g. Al-Jameel, (2012) and Al-Obaedi (2012)).

6.3.2 Move-up delay

Move-up delay is the time spent by the driver preparing to move when the lights show green following a stopping situation at traffic signals. It is obtained for each driver by generating a random number from a lognormal distribution based on site observations as discussed in Chapter 4, Section 4.8.

6.3.3 Vehicle type and length

In SIMSUR model, vehicle type is assigned to each vehicle by generating a random number from a uniform distribution (R_{type}). The vehicle will be regarded as a HGV if the random number is lower than the percentage of observed HGVs for each site and each stream (i.e. primary or secondary stream). A random number (R_L) will be also generated after the assignment of vehicle type to obtain vehicle length. The distribution for vehicle length is obtained using truncated normal distribution for cars and cumulative distribution for HGVs as explained in Chapter 4, Section 4.9. The steps of assigning vehicle type and length are illustrated in Figure 6.3.

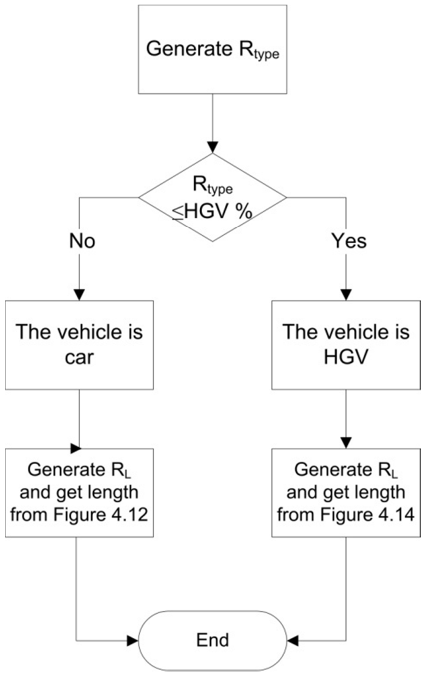


Figure 6.3: Method of obtaining vehicle type and length

6.3.4 Desired Speed

Desired speed is the maximum speed at which the driver may wish to travel in a road section without the influence of any other road users (Yousif, 1993). The desired speed is assigned to each driver using normal distribution as reported by previous studies (Al-Jameel, 2012; Al-Obaedi, 2012) using a random number generator. The mean desired speed used in the current study is assumed to be the road speed limit of 30 mph (48 km/h) because of the unavailable observed speed data and the standard deviation (σ) of 2 mph.

6.3.5 Buffer space

Buffer space is the space between stopped vehicles under congested conditions (from the front bumper follower to the back of the leader) as shown in Figure 6.4. Buffer space is assumed to be 1.5 metres which is within the reported limits (Benekohal, 1986; Yousif, 1993; Al-Jameel, 2012; Al-Obaedi, 2012).

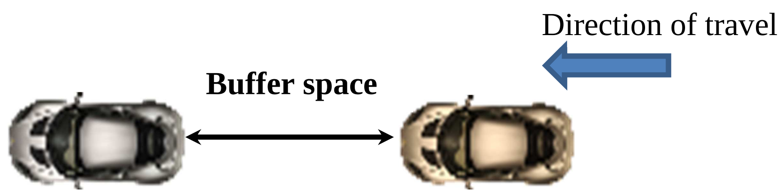


Figure 6.4: Definition of buffer space

6.3.6 Arrival headway distribution

Arrival time headway represents the time interval between the arrivals of two successive vehicles at a given point (datum line), which is used to generate vehicles arriving into the system. According to O'Flaherty (1986), the distribution of the time headway depends on various parameters such as driver reaction time, braking distance, vehicle composition, and other factors. The headway distribution that is used in SIMSUR model is the lognormal distribution for sites with low flow levels (up to 500 veh/hr) and the shifted negative exponential distribution for sites with moderate to high flow levels (over 500 veh/hr) which were obtained from observed data as explained in Chapter 4, Section 4.5.

6.3.7 Acceleration and deceleration rates

The normal and maximum acceleration rates were obtained from the ITE (1999) and were also in the updated ITE (2010) as there is an absence of such data from the UK. The normal acceleration rate (comfortable acceleration) is used by the driver to reach his/her desired speed or when exceeding the desired speed. The values for the normal acceleration rates are suggested to be 1.1 m/sec^2 for cars and 0.37 m/sec^2 for HGVs. For normal deceleration rates, the values are 3.0 m/sec^2 and 1.8 m/sec^2 for cars and HGVs, respectively. The maximum acceleration rates (which represents the vehicle's mechanical ability) rates are shown in Table 6.2 for cars and HGVs for each speed group. The maximum deceleration rate is assumed as 4.9 m/sec^2. These values were factored down by 75% as suggested by previous research studies because of the relatively higher vehicle capabilities in the USA compared with Europe and the UK (Yousif, 1993; Wang, 2006; Al-Jameel, 2012).

Table 6.2: Maximum acceleration rates (m/sec^2) for cars and HGVs (ITE, 1999)

Speed (km/hr)	0-32	32-48	48-64	64-80	> 80
Cars	2.4	2.0	1.8	1.6	1.4
HGVs	0.5	0.4	0.2	0.2	0.1

6.4 Car-following model structure

6.4.1 Introduction

The rule governing the relationship between the leader and the follower is the most important rule (car-following rule) that governs the microscopic model. The car-following rule that was developed by Benekohal (1986) CARSIM was adopted for SIMSUR model because it is realistic and represents the free following as well as stop/go conditions which is the situation in an urban environment and particularly in shuttle-lane roadworks.

The car-following sub-model structure which is based on CARSIM is shown in Figure 6.5 as used by many previous researchers (see for example Benekohal, 1986; Yousif, 1993; Al-Jameel, 2012; Al-Obaedi, 2012) with a small modification to account for shuttle-lane rules, drivers' compliance with temporary traffic signals and dilemma zone. The subroutine is called in SIMSUR every scanning time Δt (0.5 seconds) to determine the acceleration/deceleration value in order to determine the new vehicle speed and the new position based on the safety rule. The different acceleration/deceleration rates used are explained in the following sections.

Vehicles new speeds and positions were updated at the end of each scanning time using Equations 6.1 and 6.2.

$$\mathrm{NSp_n} = \mathrm{Sp_n} + \mathrm{ACC}(\Delta t) \qquad \text{Equation 6.1}$$

$$\mathrm{NPos_n} = \mathrm{Pos_n} + \mathrm{SP_n}(\Delta t) + 0.5\,\mathrm{ACC}\,(\Delta t)^2 \qquad \text{Equation 6.2}$$

Where,

ACC is the acceleration/deceleration rate of the vehicle n (m/sec^2).

Δt is the scanning time and it is equal to 0.5 seconds.

$\mathbf{NSp_n}$ and $\mathbf{NPos_n}$ are the updated speed (m/sec) and position (m) of vehicle *n* (at the end of the current scan time interval).

$\mathbf{Sp_n}$ and $\mathbf{Pos_n}$ are the current speed (m/sec) and position (m) of the vehicle *n*, respectively.

6.4.2 Acceleration from vehicle capability (ACC_1)

Based on the vehicle type, the mechanical capability of cars will be different from HGVs and each vehicle will be assigned an acceleration (positive value)/deceleration (negative value) accordingly. These values are assigned based on the vehicle's current speed according to Table 6.2.

6.4.3 Acceleration from comfortable conditions (ACC_2)

The desired speed is assigned to each vehicle before the entry to the system and the driver tries to reach his desired speed using normal acceleration/deceleration rates if there is no constraint by the leader or by roadway conditions such as speed limits, weather conditions, etc. The acceleration obtained by this condition is represented by the symbol ACC_2.

6.4.4 Acceleration from stopping distance conditions (ACC_3)

The spacing between the leader and the follower at every time scan is calculated to ensure that the follower can stop safely even in the situation of a sudden stop by the leader. The acceleration/deceleration rates (ACC_3) that satisfy this situation can be calculated according to the following equations (Equation 6.3 to Equation 6.5):

$$\mathrm{PosL} - (\mathrm{PosF} + \mathrm{SpF}\,(\Delta t) + 0.5\,(\mathrm{ACC}_3)\,\Delta t^2) - \mathrm{L_v} - \mathrm{Bs} \geq \qquad \text{Equation 6.3}$$

Maximum of Equation 6.4 and Equation 6.5

$$(\mathrm{SpF} + (\mathrm{ACC}_3)\,\Delta t\,)\,\mathrm{Rt} \qquad \text{Equation 6.4}$$

OR

$$\big(\mathrm{SpF} + (\mathrm{Acc}_3)(\Delta t)\big)\mathrm{Rt} + \frac{\left(\mathrm{SpF} + (\mathrm{ACC3})(\Delta t)\right)^2}{2\mathrm{MaxDF}} - \frac{(\mathrm{SpL})^2}{2\mathrm{MaxDL}} \qquad \text{Equation 6.5}$$

Where,

ACC_3 is the acceleration due to safe stopping conditions (m/sec^2).

Rt is the reaction time (sec).

Δt is the scanning time (sec).

PosF, PosL is the position of the follower and leader, respectively (m).

SpF, SpL is the speed of the follower and leader, respectively (m/sec).

Bs is the buffer spacing (m).

L_V is the vehicle length (m).

MaxDL and **MaxDF** are the maximum deceleration for the leader and the follower, respectively (m/sec^2).

6.4.5 Acceleration from slow moving conditions (ACC_4)

As the vehicle is moving in a platoon of very slow moving traffic, the distance between the follower and the leader is governed by the buffer space. The acceleration/deceleration used in this situation is determined according to the following equations (Equation 6.6 and 6.7):

$$\mathrm{PosL} - \mathrm{PosF} \geq \mathrm{L_v} + \mathrm{B_s} \qquad \text{Equation 6.6}$$

$$\mathrm{PosL} - (\mathrm{PosF} + \mathrm{SpF}\,(\Delta t) + 0.5\,(\mathrm{ACC_4})\,\Delta t^2) - \mathrm{L_v} - \mathrm{B_s} \geq 0.0 \qquad \text{Equation 6.7}$$

Where:

ACC_4 is the acceleration/deceleration due to slow conditions (m/sec^2).

6.4.6 Acceleration from stationary conditions (ACC_5)

Moving from a stationary situation occurs when the leader stops due to roadway conditions (e.g. red signals, awaiting a gap to cross the shuttle-lane site, etc.) which forces the follower to stop. When the vehicle starts to accelerate due to the absence of that condition (e.g. the signals turn into green, safe gap is available, etc.), it will spend some time to start moving again (move-up delay). The acceleration values that the driver will use (ACC_5) to accelerate when moving from stationary conditions are 0.42 m/sec^2 and 0.21 m/sec^2 for cars and HGVs, respectively (Benekohal, 1986; Al-Jameel, 2012).

6.4.7 Acceleration for stopping at traffic signals (ACC_6)

When the driver approaches temporary traffic signals and the traffic lights show amber or red, the driver starts to calculate the required deceleration to stop at the stop line (ACC_6). The deceleration rate (ACC_6) that satisfies this situation can be calculated according to Equation 6.8 and 6.9, which were calculated using the standard stopping distance equation (Gazis *et al*., 1960):

$$\mathrm{Ds(n)} = \mathrm{Pos_n} - \mathrm{StpL} \qquad \text{Equation 6.8}$$

$$\mathrm{Acc_6} = \frac{0.5\,(\mathrm{Spn}^2)}{(\mathrm{Spn})(\mathrm{Rt}) - \mathrm{Ds(n)}} \qquad \text{Equation 6.9}$$

Where,

Ds (n) is the difference between the current position of vehicle *n* and the stop line

StpL is the stop line location

6.4.8 Selection criteria of the final acceleration rate (ACC)

At every Δt (scanning time), a unique value for the acceleration/deceleration rate (ACC) is calculated, selected and used in updating speeds and positions for each vehicle using Equations 6.1 and 6.2 shown above. The criteria for selecting this value is shown in the flowchart as illustrated in Figure 6.5.

The value of ACC_3 in Equations 6.3 to 6.5 is developed to enable the follower to stop safely even if the leader makes a sudden stop by applying a maximum deceleration. The value of ACC_4 (in Equations 6.6 and 6.7) is developed for vehicles moving in a very slow moving platoon (the distance between the follower and the leader is governed by the buffer space). The ACC_3 and ACC_4 values are calculated using an iterative process (starting from a maximum acceleration to a maximum deceleration with an increment of -0.05 m/sec^2).

When the driver approaches the temporary traffic signals and the traffic lights shows amber or red, the driver starts to calculate the required deceleration to stop at the stop line (ACC_6). The deceleration rate (ACC_6) that satisfies this situation can be calculated according to Equations 6.8 and 6.9 using the standard stopping distance equation.

At every Δt, a unique value of acceleration/deceleration rate (ACC) is selected and is used in updating speed and position of each vehicle using Equations 6.1 and 6.2. The selection criterion of this value is shown in Figure 6.5. This selected rate should not exceed the maximum acceleration rate (i.e. ACC_1). In addition, if the speed of the leader (SpL) is higher than the speed of the follower (SpF) by a certain value (i.e. 5 km/h) and if the distance headway between the two vehicles is available (>Bs+LV), the follower will not apply any deceleration rate. In all cases, the absolute value of the deceleration rate should not exceed the absolute value of the maximum deceleration rate (MaxDL), as shown in Figure 6.5.

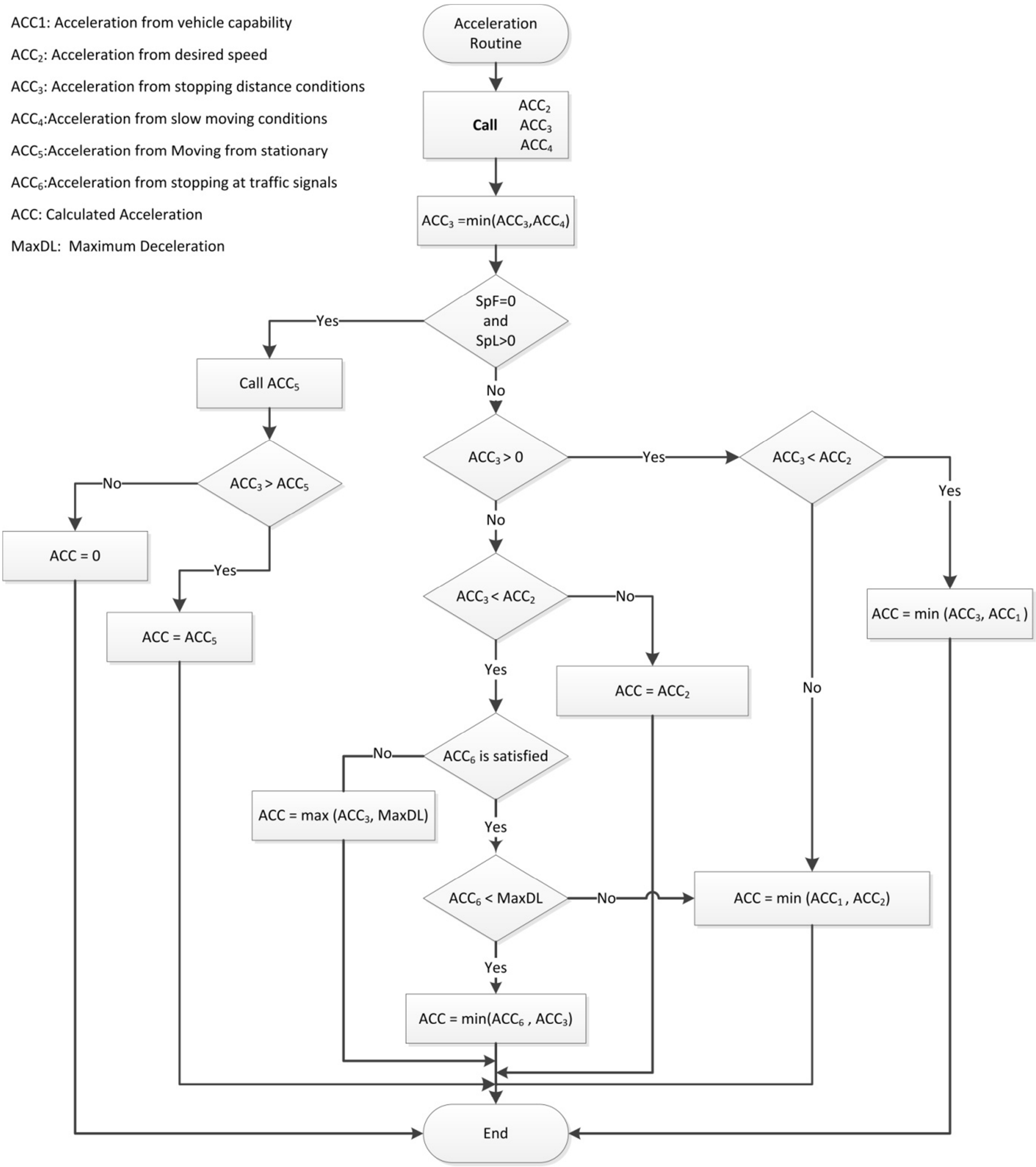


Figure 6.5: Car-following sub-model structure

6.5 Modelling of shuttle-lane roadworks

6.5.1 Introduction

Modelling the correct drivers' behaviour when vehicles are in the influence zone of the shuttle-lane roadworks (before approaching the temporary traffic signals and while crossing the roadworks site) is a very important aspect of SIMSUR simulation model.

Shuttle-lane roadworks subroutine is called every scanning time (Δt) to determine all possible decisions that can be taken by the driver based on site observations. These decisions are either to stop at the traffic signals (when the lights show amber or red) or to cross (when lights show green or amber) or to violate the traffic signals (when the lights show red). Detailed description of the shuttle-lane roadworks subroutine is shown in the following section and illustrated in Figures 6.6 and 6.7.

6.5.2 Drivers' compliance with temporary traffic signals

Shuttle-lane roadworks subroutine is explained in Figure 6.6. It can be seen from Figure 6.6 that if the vehicle is inside the roadworks influence zone, the value of Ds (using Equation 6.8 above), will be calculated for each vehicle for every time scan (Δt). The next step will be to check if the signal lights show green. In this case the vehicle will continue using the car following rule. If the signals does not show green and shows amber, then deceleration rate (ACC_6) will be calculated based on Ds.

If the vehicle is the first to approach the temporary traffic signals, then a check will be carried out to compare the calculated deceleration rate (ACC_6) and the maximum deceleration rate (-4.9 m/sec^2). If ACC_6 is lower than the maximum deceleration, then the driver could not possibly stop on time at the stop line. In this case, he/she will cross the site on amber (unintentionally).

If ACC_6 is higher than the maximum deceleration and the driver has the capability to stop the vehicle, a random number generator (R_{ACL}) is called (representing Random number generated from Amber Crossing for Leaders). If this number is lower than the percentage of amber crossing for leading vehicles (Category 1 - DZ) based on observed data, then the driver will cross the site on amber. Alternatively, the driver will stop at the stop line using the calculated deceleration rate.

The observed percentage of R_{ACL} is based on three types of amber crossing and red violations. These are summarised below:

1- Drivers crossed unintentionally because they cannot stop due to their speed and distance to the stop line (crossed the stop line at the onset of amber);
2- Drivers crossed intentionally (crossed the stop line at the onset of amber);
3- Drivers decided to cross on amber but the lights turned to red at the time of crossing.

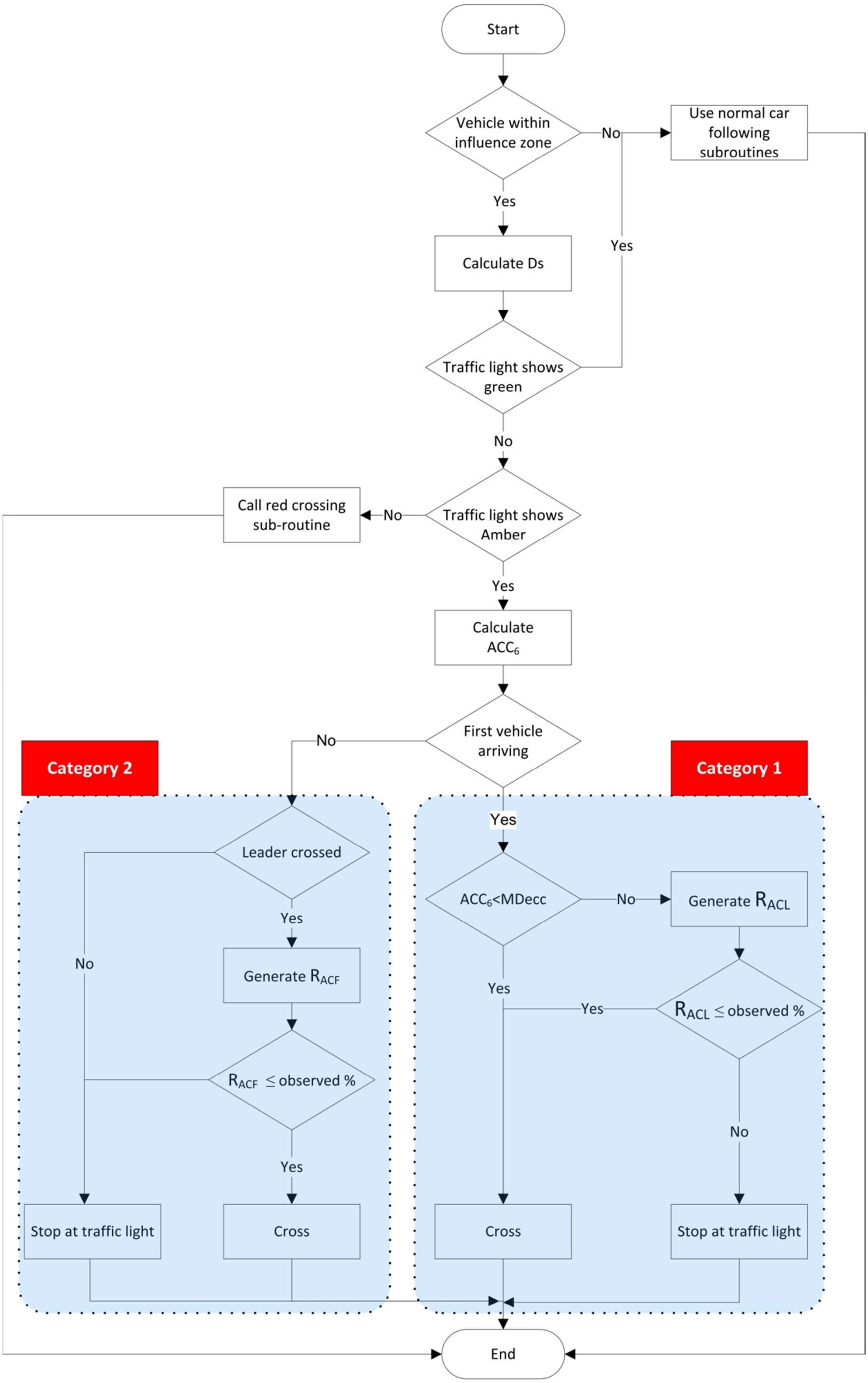


Figure 6.6: Shuttle-lane roadworks sub-model structure (green/amber crossing)

The split between first two types cannot be identified from the observed data. Therefore, the R_{ACL} value used in the model will account for intentional crossing only while the unintentional crossing due driver inability to stop will be accounted using the comparison between ACC_6 and maximum deceleration rate (as discussed earlier).

If the vehicle approaching the temporary traffic signals is not the leading vehicle, the subroutine will check the status of the leading vehicle (i.e. whether stopped at the stop line or crossed on amber). If the leading vehicle stopped at the stop line, then the following vehicle must also stop. If the leading vehicle crossed the stop line on amber, then a random number generator will be called (R_{ACF}) and if the random number is lower than the observed percentage of amber crossing for following vehicles (as shown in Table 4.22 for Category 2 – DZ follower), then the driver will cross the site on amber. Otherwise, the driver will stop at the stop line.

If the approaching vehicle arrived at the roadworks influence zone and the temporary traffic signals show red, then the red crossing (violation) subroutine will be called. The detailed structure for this case is illustrated in Figure 6.7. The subroutine illustrates Category 2 (DZ follower crossing on the onset of red light as shown in Table 4.22), Category 3 (group violations) and Category 4 (single violation as shown in Table 4.24), as observed at the surveyed sites and explained in Chapter 4, Section 4.11.

If the vehicle is the first vehicle approaching the temporary traffic signals on the onset of red, then the visibility will be checked and if the driver cannot see the first vehicle from the opposite stream, then the driver will stop at the stop line and comply with the traffic signals. Also, if the visibility is good, but there are vehicles approaching from the opposite stream, the driver will also stop at the stop line and will not violate the red light (as was the case from site observations).

Alternatively, if visibility is good and there are no drivers approaching from the opposite stream and the vehicle is already stopped at the stop line (speed equal to zero), then Category 3 violation criteria are checked.

Category 3 (group violations) occurs on VA signals and when vehicle waiting time exceeds 75 seconds (as observed from sites due to MVD detection failures). If all these conditions are met, then a random number generator will be called (R_{RGC}). If this random number is lower than the percentage of group violations, then a group of 3 to 4 drivers, as observed from sites, may cross the site while on red signals, but if the number is higher, then the drivers will stop at the stop line.

Category 4 (single violations) occurs at both FT and VA signals. A random number generator is called (R_{RSC}) and if it is lowers than the observed percentage of single violations (as shown in Table 4.24), then the driver will cross the site on red. Otherwise, the driver will stop at the stop line.

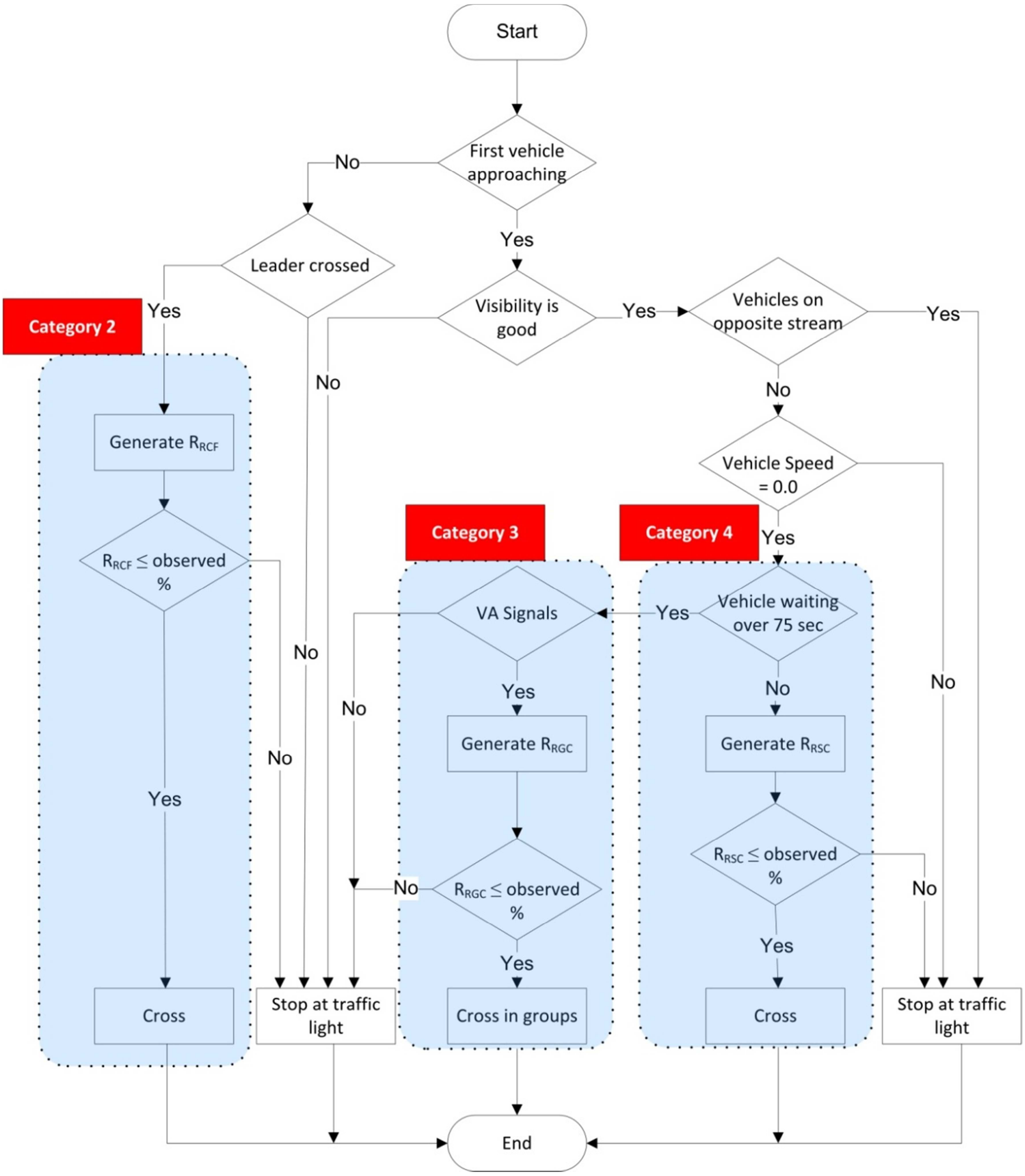


Figure 6.7: Shuttle-lane roadworks sub-model structure (red violations)

If the vehicle approaching the temporary traffic signal is not the leading vehicle, the model will check if the leading vehicle has crossed on amber/red. If not, then the following vehicle will also stop. Alternatively, if the leading vehicle crossed the stop line on amber, then a random number generator will be called (R_{RCF}). If this number is lower than the observed percentage of red crossing for following vehicles (as shown in Table 4.22 for Category 2 – DZ follower), then the driver is assumed to cross the site on amber. Otherwise, the driver is assumed to stop at the stop line. Category 3 and category 4 violations are built in to the model but has not been activated as they are contributing to negligible amount of violations as shown in Table 4.25.

6.6 Modelling of shuttle-lane traffic signals

6.6.1 Introduction

Modelling temporary traffic signals requires applying the correct system and sequences as observed on visited sites. Two types of signals settings were observed on site namely Fixed Time (FT) and Vehicle Actuated (VA). Detailed description of the method of modelling these systems is explained in the following sections.

6.6.2 Fixed time signals (FT)

Although it is stated by the Department for Transport (2008) that temporary traffic signals should always be operating under Vehicle Actuated (VA) settings and Fixed Time (FT) needs to be authorised in writing, it was noticed that two out of the six visited roadworks sites were operating under FT settings. Using fixed signals settings, green time will be fixed to the maximum recommended green time (which depends on site length) without taking into account the flow level or tidality.

6.6.3 Vehicle actuated signals (VA)

There is no direct reference to the amount of green time extension which will be given to each vehicle in the Highways Agency (2005B). It was stated by the Department for Transport (1999a) that vehicles will be extended by an increment of 0.5 seconds until the vehicle reaches the stop line. It was also noticed on site that when a stream has the active green stage running and no vehicles are detected on the opposite stream, the green stage can be extended for up to 90 seconds before it is terminated. The simplified operation of VA signals is illustrated in Figure 6.8.

6.7 Other model characteristics

Other SIMSUR model characteristics were added, such as each vehicle is assigned a unique vehicle code (serial number). The serial number does not change once vehicles exit the system and it holds all the characteristics of that vehicle for the output analysis.

Warm-up and cooling-down sections were introduced (500 metres at each end of the model) representing the generation and exiting points of the model and all data on the traffic behaviour on these sections have been ignored in the output data analysis (road length excluding both warm-up and cooling-down sections is 2 kms in length).

Warm-up and cooling down periods were also introduced (5 minutes each) at the start and the end of the simulation period and data on the traffic behaviour was ignored in the output data analysis.

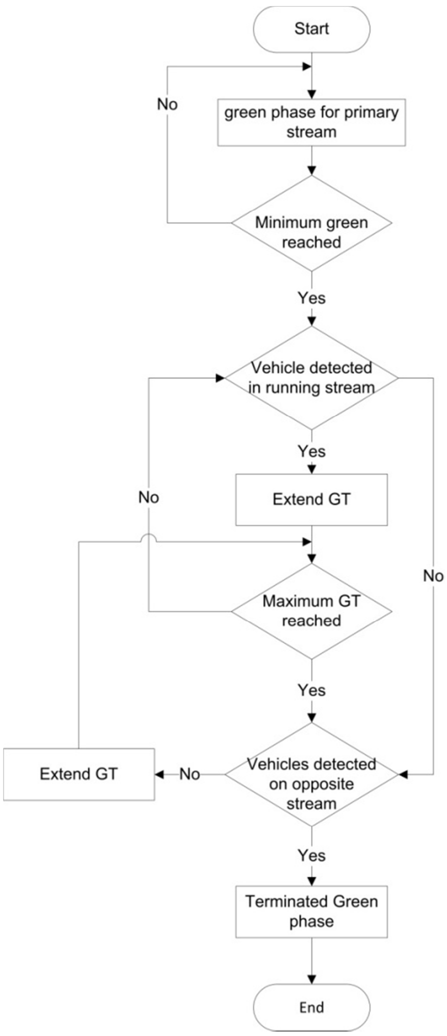


Figure 6.8: Vehicle Actuated (VA) sequence for temporary traffic signals

6.8 Model output

There are various types of SIMSUR model output that were used for different tasks such as model verification, calibration, validation, traffic management testing, vehicle interaction and signals extension (i.e. VA signals and safety in shuttle-lane site). The model output files can be grouped under four main headings:

1- Micro reporting:
   - ⇨ Vehicle position, speed and acceleration (every scanning time)

2- Cyclical reporting:
   - ⇨ Cycle throughput;
   - ⇨ Compliance with traffic signals;
   - ⇨ Queues;
   - ⇨ Move-up time.

3- Macro reporting:
   - ⇨ Waiting time (due to stopping at traffic signals);
   - ⇨ Overall travel time;
   - ⇨ Hourly throughput;
   - ⇨ Average vehicle speed.

4- Detector and interactive reporting:
   - ⇨ Time headway for BAR and ACR;
   - ⇨ Arrival flow and headway (when vehicles enter the system);
   - ⇨ Vehicle detection to alter signals settings (VA signals);
   - ⇨ Vehicle detection for shuttle-lane site safety.

6.9 Model capabilities

SIMSUR model was designed in order to test the effect of different traffic management controls and layouts (i.e. speed limits, signals settings, site length) on travel time, system capacity and drivers' behaviour. Furthermore, all related parameters are easily changed in the input file in order to assess the effect of applying different values.

SIMSUR model takes into account the limitations of previously reported simulation models (Chapter 2, Section 2.7) such as:

1- The ability to take into account the effect of various parameters such as HGVs percentage, tidal flow, new VA signals specification;
2- The ability to take into account the effect of dynamic acceleration/deceleration changes for every vehicle;
3- The ability to replicate the actual correct behaviour of dilemma zone and red light violations observed on site and test their effect on system capacity;

6.10 Summary

This chapter described the development of SIMSUR model for shuttle-lane roadworks operated by temporary traffic signals. The car following and shuttle-lane rules (sub models) were also discussed in details. The rules used in SIMSUR model were based on real data from sites observations as well as related previous studies. FORTAN programming language was used with over 4,700 lines of coding. The next chapter will describe SIMSUR model verification, calibration and validation stages using real data taken from different shuttle-lane roadworks sites and other sources.

CHAPTER SEVEN: SIMSUR MODEL VERIFICATION, CALIBRATION AND VALIDATION

7.1 Introduction

The reliability of any traffic micro-simulation model depends on the model's ability to produce system's behaviour that is close enough to real traffic situations (FHWA, 2004). According to Al-Obaedi (2012), exact replication of traffic parameters might not be achieved as it mainly depends on human behaviour which is subject to change (randomness) because of various reasons and that simulation errors should not exceed the permitted limits.

In the previous chapter, SIMSUR simulation model and the associated sub-models (rules) for car-following and shuttle-lane were explained in details. The current chapter presents the verification, calibration and validation stages of these rules and also for the whole micro-simulation model.

The model verification process is determining the computer code which implements the modelling logic and produces the desired output for various sets of input data, observing the animation of the simulation outputs under a variety of input parameters (Olstam and Tapani, 2011; Wang, 2006). Model calibration is the adjustment of model parameters (from real observations) using optimisation to determine the best match of the simulated outputs with real observations from sites (May, 1990). According to Liu and Wang (2007), model validation is the testing of different sets of data (from different time periods or different sites) using the calibrated model parameters, in which statistical measures (goodness-of-fit) are used to quantify the similarities between the simulated model output and the observed data.

May (1990) described the typical structure of any simulation model as shown in Figure 7.1. The figure shows that the verification, calibration and validation processes are dependent and repetitive since any discovered error may require adjusting the model's assumptions and parameters.

In this chapter, the statistical tests and the three model stages (verification, calibration and validation) are discussed in details in the following sections.

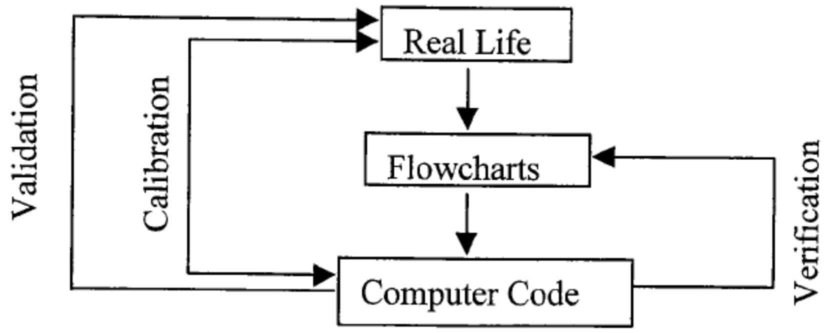


Figure 7.1: Micro-simulation model verification, calibration and validation stages (May, 1990)

Various sites were used for the model calibration and model validation stages as shown in Table 7.1. Different information (both observed data and assumed values) was used either as input for, or output from, the SIMSUR model. The information used as input/output were also utilised for either model calibration or validation stage. Table 7.2 provides a summary of the parameters/measurements which were used in the SIMSUR model either as an input or obtained as an output for both calibration/validation stages.

Table 7.1: List of sites used for model calibration / validation stage

Process	FT sites	VA sites
Calibration	⇨ Site 16a	⇨ Site 12
Validation	⇨ Site 16b ⇨ Site 19	⇨ Site 17 ⇨ Site 18

Table 7.2: Data used in model input/output or calibration/validation stage

<table>
<tr><th>Parameters/Measurements</th><th>Observed/Assumed</th><th>Input</th><th>Output</th><th>Calibration</th><th>Validation</th></tr>
<tr><td>Arrival flow</td><td>Observed</td><td>✓</td><td></td><td colspan="2">NA</td></tr>
<tr><td>Throughput</td><td>Observed</td><td></td><td>✓</td><td></td><td>✓</td></tr>
<tr><td>HGV %</td><td>Observed</td><td>✓</td><td></td><td colspan="2">NA</td></tr>
<tr><td>Site geometric details</td><td>Observed</td><td>✓</td><td></td><td colspan="2">NA</td></tr>
<tr><td>Signal timings</td><td>Observed</td><td>✓</td><td></td><td colspan="2">NA</td></tr>
<tr><td>Arrival time headway</td><td>Observed</td><td>✓</td><td></td><td>✓</td><td></td></tr>
<tr><td>Following time headway</td><td>Observed</td><td></td><td>✓</td><td></td><td>✓</td></tr>
<tr><td>MUT</td><td>Observed</td><td></td><td>✓</td><td></td><td>✓</td></tr>
<tr><td>MUD</td><td>Observed</td><td>✓</td><td></td><td colspan="2">NA</td></tr>
<tr><td>Drivers’ signals compliance %</td><td>Observed</td><td>✓</td><td></td><td>✓</td><td></td></tr>
<tr><td>Close following</td><td>Observed</td><td></td><td>✓</td><td></td><td>✓</td></tr>
<tr><td>Queues</td><td>Observed</td><td></td><td>✓</td><td></td><td>✓</td></tr>
<tr><td>Buffer space</td><td>Assumed</td><td>✓</td><td></td><td>✓</td><td></td></tr>
<tr><td>Reaction time</td><td>Assumed</td><td>✓</td><td></td><td>✓</td><td></td></tr>
<tr><td>Shift value</td><td>Assumed</td><td>✓</td><td></td><td>✓</td><td></td></tr>
<tr><td>Speed</td><td>Assumed</td><td>✓</td><td></td><td colspan="2">NA</td></tr>
<tr><td>Car length</td><td>Assumed</td><td>✓</td><td></td><td colspan="2">NA</td></tr>
<tr><td>HGV length</td><td>Observed</td><td>✓</td><td></td><td colspan="2">NA</td></tr>
<tr><td>Acceleration</td><td>Assumed</td><td>✓</td><td></td><td colspan="2">NA</td></tr>
</table>

Observed: Values taken from site visits   Assumed: Values taken from previous literature
NA: Not applicable (neither used in calibration nor validation)
Site geometric details: Such as site length, location of signs, etc.
Arrival time headway: Time headway between successive vehicles (used to generate traffic to enter the system)
Following time headway: Time headway between successive vehicles in a platoon (i.e. with headway 6 ≤ seconds)
Drivers’ signals compliance %: Percentage of vehicles involved in amber crossing/red light violations (R_{ACL}, R_{ACF} R_{RCF})
Close following: Vehicles not complying with the 2 seconds rule (i.e. with time headway < 2 seconds)

7.2 Statistical tests

Statistical goodness-of-fit measurements (tests) were carried out between the observed and simulation output data for calibration and validation purposes. In addition, graphical representation was also produced. Five new goodness-of-fit measures have been introduced, explained and used in the model calibration and validations for comparison between observed and modelled results.

7.2.1 Root Mean Square Error (RMSE)

This test is considered to be good initial test to make a comparison between empirical and simulated data because it penalises high errors at higher rates than small errors (Toledo, 2003). The test has been used in many simulation studies (see for example Wu *et al*., 2003; Panwai and Dia, 2005; Wang, 2006; Al-Jameel, 2012 and Al-Obaedi, 2012) and is shown in Equation 7.1.

$$\text{RMSE} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(xi - yi)^2} \qquad \text{Equation 7.1}$$

Where,

n is the number of time intervals

xi is the observed flow at time interval i;

yi is the simulated flow at time interval i.

7.2.2 Coefficient of correlation (r)

The coefficient of correlation is considered a popular goodness-of-fit measure for testing the strength of the linear relationship between modelled and observed data and is shown in Equation 7.2 (Hourdakis *et al*., 2003).

$$r = \frac{1}{n-1}\sum_{i=1}^{n}\frac{(xi-\bar{x})(yi-\bar{y})}{\sigma x \sigma y} \qquad \text{Equation 7.2}$$

Where,

$\bar{x}$ and σx are the mean and the standard deviation for the actual observed data

$\bar{y}$ and σy are the mean and the standard deviation for the simulation output

7.2.3 Theil's Inequality Coefficient (U)

Theil's Inequality Coefficient is considered to be more sensitive and accurate than the RMSEP or (r) and it is widely used in calibration and validation of traffic simulation models (see for example Wang (2006), Al-Jameel, (2012) and Al-Obaedi (2012)). The U value is between 0 and 1 with a value of 0 represent a perfect fit. It can be determined by the following equation (Hourdakis *et al*., 2003).

$$U = \frac{\sqrt{\frac{1}{n}\sum_{i=1}^{n}(xi-yi)^2}}{\sqrt{\frac{1}{n}\sum_{i=1}^{n}(xi)^2}+\sqrt{\frac{1}{n}\sum_{i=1}^{n}(yi)^2}} \qquad \text{Equation 7.3}$$

7.2.4 Bias proportion (Um)

Bias proportion (U_m) measures the error that can be used to determine consistent over-counting or undercounting of vehicles by comparing mean values and is represented in equation 7.4 (Hourdakis *et al*., 2003).

$$U_m = \frac{n(\bar{x}-\bar{y})^2}{\sum_{i=1}^{n}(xi-yi)^2} \qquad \text{Equation 7.4}$$

7.2.5 Variance proportion (Us)

Variance proportion (Us) can measure the degree of variability of the simulated measurements compared with actual observed measurements (Hourdakis *et al*., 2003).

$$U_s = \frac{n(\sigma x-\sigma y)^2}{\sum_{i=1}^{n}(xi-yi)^2} \qquad \text{Equation 7.5}$$

According to Hourdakis *et al*. (2003), the coefficient of correlation (r) is considered to be a good measure but it does not provide any additional information on the nature of the error (difference) between real measurements and simulation. “Theil’s Inequality Coefficient” is more accurate and sensitive than RMSEP or r and it can also be decomposed into three other coefficients that provide more specific information about the nature of the error.

According to Hourdakis *et al*. (2003), satisfactory model results will be achieved if (r) is above 0.8 and (U) is lower than 0.3. According to the Design Manual for Roads and Bridges (1996), the GEH should be ≤ 5 for the link flow to be satisfactory. These thresholds, along with other measures, are monitored throughout the calibration/validation process to ensure acceptable model quality.

7.3 Model verification process

According to Al-Jameel (2012), the model verification process can be described as the process of checking if the modelling assumptions have been correctly translated into a computer code (i.e. debugging the program code). In the simulation model development, the model verification can also be achieved by the observation of the model animation and the simulation output to check if they are reasonable under various input parameters without comparing with the real observed input data (Wang, 2006).

Therefore, the model verification process was carried out at the model development stage by observing the animation, analysing the model output and debugging the program code for any warnings and errors. Model animation screenshot is as shown in Figure 7.2. The verification was carried out for all input parameters (i.e. desired speed, arrival headway distribution, vehicle length distributions, car-following rules and shuttle-lane rules, etc.).

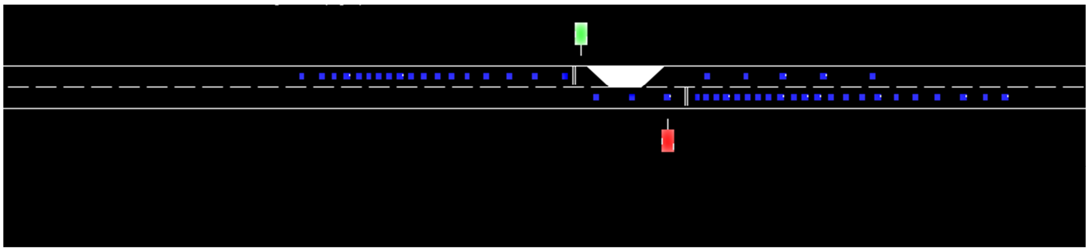
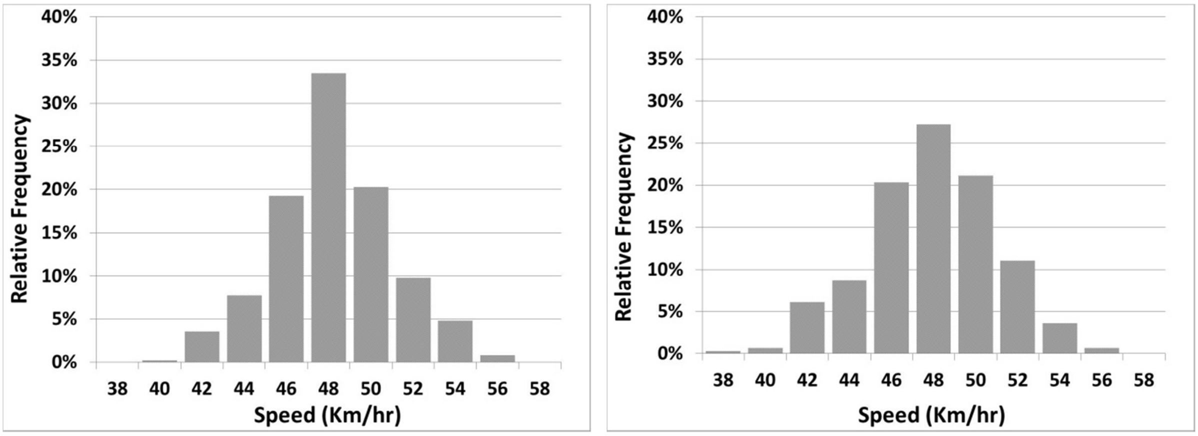


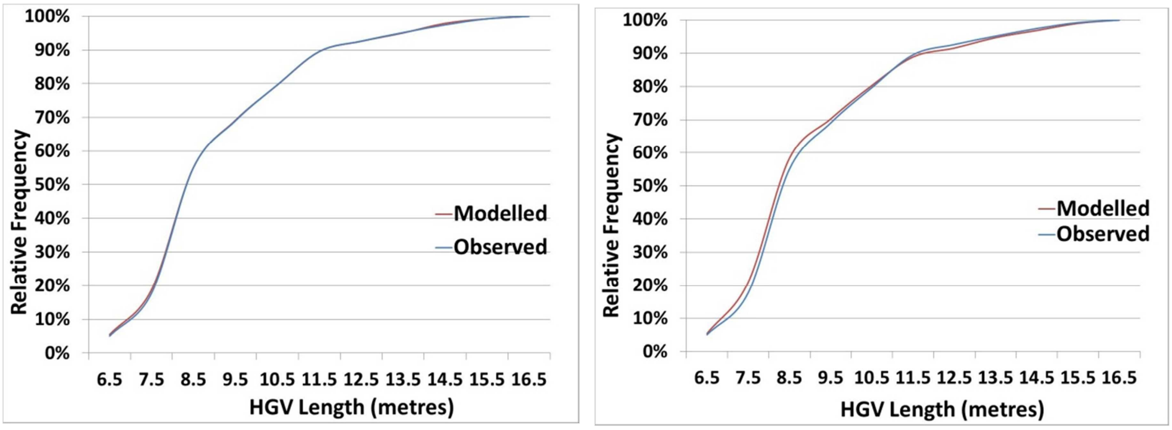
Figure 7.2: Typical screenshot from the simulation model

The normal distribution of the desired speed and for example, the cumulative distribution of HGVs length were found to be similar to what was expected (input into the model) as shown in Figure 7.3 and 7.4. The same tests were carried out for arrival headway distribution, reactions time, move-up delay.



(a) Primary Stream (b) Secondary Stream

Figure 7.3: An example of model verification for desired speed distribution



(a) Primary Stream (b) Secondary Stream

Figure 7.4: An example of model verification for vehicle length distribution (HGVs)

Vehicle trajectory diagram is the most commonly used parameter to reveal the capability of vehicle movements in simulation models and is used as part of the model verification process of vehicle movements and response to traffic signals. Figure 7.5 shows the trajectories for a sample of 20 vehicles on the primary stream. Vehicles arriving at the shuttle-lane roadworks site operated by temporary traffic signals will experience both interrupted and uninterrupted flow conditions at the traffic signals as illustrated in Figure 7.5.

Figure 7.5 shows an example of the different arrival headways and following headways between vehicles during the simulation process. It also shows that interrupted vehicles will start decelerating before arriving at the temporary traffic signals and stopping during the red light. Vehicles will start accelerating when the lights show green. On the other hand, uninterrupted vehicles arrived at the site when the traffic lights show green will continue without stopping.

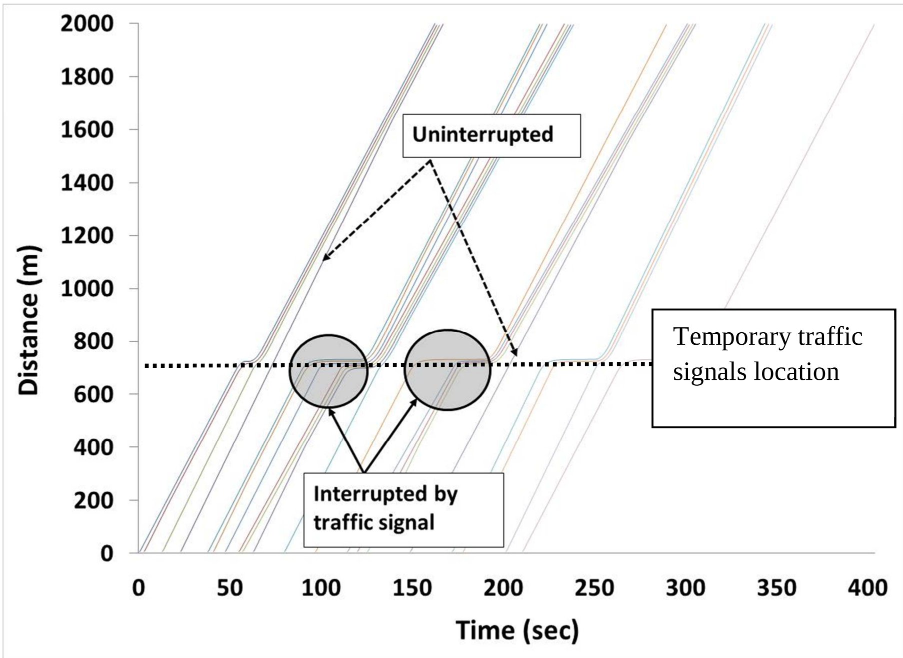


Figure 7.5: Sample of vehicle trajectories and the effect of traffic signals

7.4 Model calibration process

This section describes the calibration process of the car-following sub-model, shuttle-lane sub-model and the overall simulation model. It should be noted that the model calibration results were achieved after repetitive iterations of the model's verification and parameter calibration stages, as mentioned in section 7.1 and illustrated in Figure 7.1. During the iterative processes, the parameters were modified in order to achieve a closer fit between real observed data and simulation output.

7.4.1 Calibration of the car-following model

The calibration of the car-following sub-model is an important step to ensure the correct use of the car-following rule. Due to the very limited or unavailable trajectory data in the UK, the simulation model results were compared with real observed trajectory data that was collected by Robert Bosch GmbH Research Group using instrumented vehicles to collect relative speed and space headway between the leader and the follower (Panwai and Dia, 2005). The trajectory dataset can be summarised as:

- Speed ranging between 0 and 60 kph;
- The duration of the test is 300 seconds;
- Three stopping situations.

Panwai and Dia (2005) compared the trajectory data with simulation models such as S-Paramics, VISSIM and AIMSUM. The comparison used both RMSE in metres and EM (Error Metric) statistical tests to show the goodness-of-fit between the modelled and observed value of the spacing between the leader and the follower. The results of the tests with the developed SIMSUR simulation model are summarised in Table 7.3 and the graphical representation is shown in Figure 7.6.

The calibration of the car-following model is a complicated and very sensitive step in determining the appropriate behaviour of the leader-follower relationship. As the developed model is based on the safety criteria, the assumed values of the reaction time and the buffer space are important and critical factors that were calibrated during this process. Iterative processes were carried out to select the optimum reaction time Rt (starting with Rt of 0.5 seconds and ending with 2.2 seconds) and buffer space Bs (assuming Bs of 0.5 to 2.5 metres) which helped to achieve the best fit between simulation and observed values as shown in Table 7.3 and presented in Figure 7.6.

It can be seen from Table 7.3 and Figure 7.6 that using the RMSE statistics, SIMSUR simulation model produced the best results (when compared with observed data) in terms of the representation of the car-following behaviour between the follower and the leader under the current test conditions. Using other statistics, such as the EM, the model is considered the second best after AIMSUM with a very small difference. These results were obtained using an optimum reaction time value of 1.2 seconds and an optimum buffer space value of 1.5 metres following an iterative process.

Table 7.3: Performance of the car-following model in the selected traffic micro-simulation models (Panwai and Dia, 2005)

Statistical measure	**AIMSUN (v4.15)**	**VISSIM (v3.70)**		**Paramics (v4.1)**	**SIMSUR Model**
		Wiedemann 74	**Wiedemann 99**		
RMSE	4.99	5.72	5.05	10.43	3.96
EM	2.55	4.78	4.50	4.68	3.77

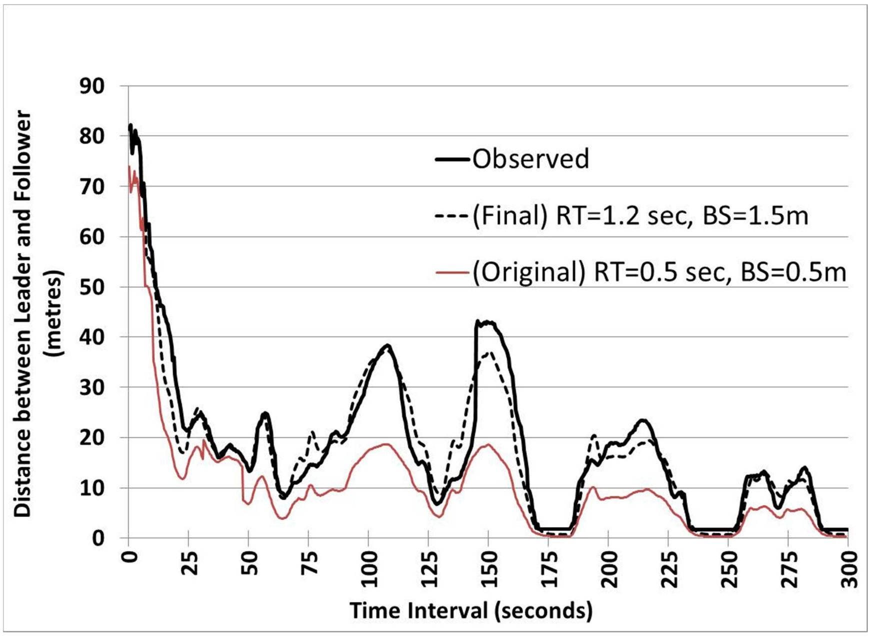


Figure 7.6: Comparison between SIMSUR simulation model and observed distance between the follower and the leader

Following the above results which shows a reliable fit of the car-following behaviour between the leader and the follower (which is known to be the core of the simulation model), the model can be used in building the anticipated shuttle-lane simulation model.

7.4.2 Calibration of the shuttle-lane model

The calibration of the shuttle-lane rule was carried out by ensuring that the model could replicate the correct drivers' behaviour as accurately as possible (i.e. amber crossing and red light violations). This step was carried out by implementing the drivers' behaviour rules and decisions when approaching temporary traffic signals as discussed in Section 6.5 (illustrated in Figure 6.6 and 6.7) and also by assuming that drivers are alert when they approach the site (which affects the reaction time and following headway).

The calibration of temporary traffic signals violations was carried out for Category 1 and Category 2 violations only (as explained in Section 6.5) using a random number generator in an iterative process (starting with the observed values until it reached the final modelled value as shown in Table 7.4), which compares the generated number with the observed percentage (i.e. R_{ACL}, R_{ACF}, R_{RCF}, etc.) in order to achieve as accurate results as possible for each site. These percentages (used in SIMSUR simulation model as an input) were modified slightly to take into account the random arrival and position of vehicles at the onset of amber as shown in Table 7.4. Calibration results for R_{ACL} are shown in details in Appendix C (Tables C.13 and C.14).

For example, the observed percentage of R_{ACL} (Random number generated from Amber Crossing for Leaders) is made up of three types of amber crossing and red violations, which are summarised below:

1- Drivers crossed the stop line because they could not stop due to their speed and distance to stop line (crossed the stop line at the onset of amber);
2- Drivers crossed the stop line intentionally (crossed the stop line at the onset of amber);
3- Drivers decided to cross the stop line on amber but the lights changed to red at the time of crossing.

The split between the first two categories cannot be identified from observed data. Therefore, the R_{ACL} value used in the model will account for intentional crossing only while the unintentional crossing due to driver's inability to stop will be accounted for using the comparison between ACC_6 and the maximum deceleration rule as discussed in Section 6.5. The calibration and validation results for the whole of the simulation model are shown in the following section.

Table 7.4: Summary of observed and calibrated R_{ACL} for all sites

Site	Observed R_{ACL} (%)		Final Modelled R_{ACL} (%)	
	Primary	Secondary	Primary	Secondary
Site 12	3.8	1.3	3.0	0.5
Site 16a	22.5	14.8	18.0	12.0
Site 16b	10.6	25.1	8.0	23.0
Site 17	1.6	1.6	1.0	1.0
Site 18	0.3	1.9	0.5	1.5
Site 19b	23.5	27.2	21.0	25.0

7.4.3 Calibration of the simulation model

The calibration for the whole of the simulation model was carried out by iterating different sets of various parameters such as buffer space, different reaction time assumptions, different shift value (for headway distribution of vehicle arrivals) and different R_{ACL} values.

Regarding reaction time assumptions, two cases were assumed on drivers' alertness and were used in model calibration as explained in Section 6.3 and summarised below. Results of following time headway were used as a primary indicator of the validity of the assumptions and are summarised in Table 7.5.

- ⇨ **Case 1**: drivers are alert when density reaches or exceeds 37 veh/km as implemented by many previous researchers (Benekohal, 1986; Yousif, 1993; Da Silva and Stosie, 2010; Al-Jameel, 2012; Al-Obaedi, 2012);
- ⇨ **Case 2**: drivers are alert if he/she approaches the "roadworks start" sign until he/she approaches the "roadworks end" sign.

Table 7.5: Summary of time headway results for different cases for Site 16a

Direction	Location	Headway	Case 1	Cases 1 & 2 (combined)
			% difference between observed and modelled time headway	
Primary	BAR	< 2 sec	-7	-2
	ACR		-16	3
	BAR	≤ 6 sec	-9	-9
	ACR		15	10
Secondary	BAR	< 2 sec	-7	2
	ACR		-13	4
	BAR	≤ 6 sec	3	1
	ACR		-14	6

BAR: Before Approaching Roadworks **ACR**: After Crossing Roadworks

It can be seen from Table 7.5 that the combined cases (i.e. cases 1 and 2) of drivers' alertness provided closer values (smaller differences) to observed following headway data for both streams (i.e. primary and secondary streams), both situations (i.e. BAR and ACR) and for time headway (for both headway ≤ 6 seconds and < 2 seconds) for Site 16a. Therefore, both cases were used in all models and the results are shown in the following sections.

Different shift values (in the shifted negative exponential distribution) were calibrated using various sets of shift value (i.e. 0.2 to 1.0 with an increment of 0.1 seconds). Table 7.6 shows the arrival flow statistical tests for both a selected shift value, the final calibrated shift value (i.e. primary and secondary streams) and the arrival flow profile is presented in Figure 7.7. The final selected shift value is 0.5 seconds for primary and secondary streams.

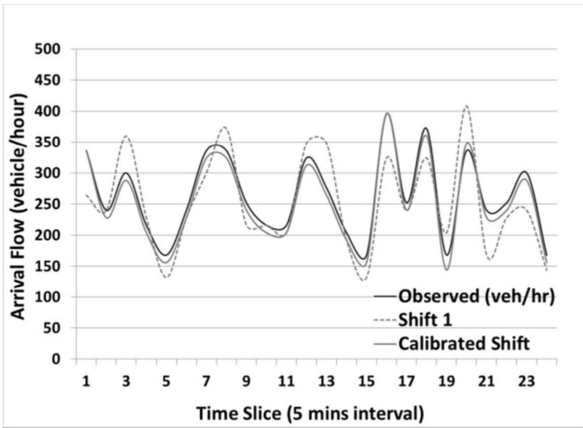
Various model outputs were also tested to determine that the model calibration is adequate. The model outputs are shown below for each site and each stream separately:

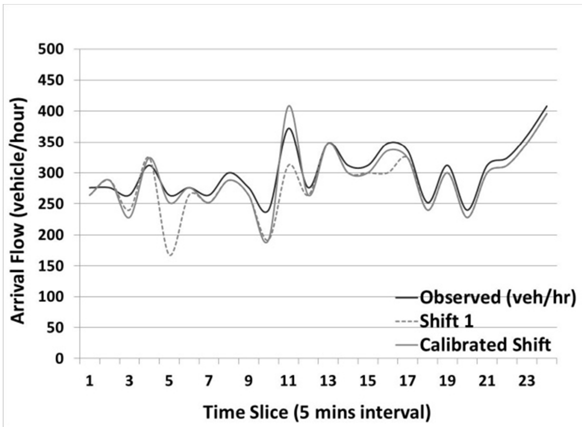
- ⇨ Hourly directional flow and throughput (i.e. the number of vehicles passing through TSS at every cycle) for each 5-minutes interval;
- ⇨ Average time headway (for both BAR and ACR);
- ⇨ Percentage of vehicles in platoons (time headway ≤ 6 seconds);
- ⇨ Drivers' non-compliance with the two-seconds rule (time headway < 2 seconds);
- ⇨ Drivers' non-compliance with traffic signals (amber crossing and red light violations);
- ⇨ Move-up time;
- ⇨ Queues.

Table 7.6: Model calibration statistics-flow for various shift values (Site 16a)

Statistical Test	Primary Stream		Secondary Stream	
	Shift 1	Calibrated shift value	Shift 1	Calibrated shift value
RMSEP %	15.9	5.6	12.2	11.5
r	0.836	0.997	0.92	0.89
U	0.08	0.02	0.05	0.07
U_m	0.00	0.57	0.01	0.00
U_s	0.00	0.20	0.00	0.00
GEH	3.18	1.07	1.8	0.91

Shift 1: is the worst results from a selected shift value of 0.8 seconds





(a) Primary Stream (b) Secondary Stream

Figure 7.7: Arrival flow profile for different shift values (Site 16a)

The calibration was divided into two categories, FT signals sites and VA signals sites. The sites used for the calibration process (as shown in Table 7.1) were divided into two categories, as follows:

- ⇨ Fixed Time signals (FT):
    - ▪ Site 16a

⇨ Vehicle Actuated signals (VA):
- Site 12

These were based on using different ranges of site lengths to cover a wider range (i.e. 52 metres for Site 16a and 107 metres for Site 12). The results of the calibrated model are summarised in the following sections.

7.4.3.1 Fixed time signals (FT)

a. Input Parameters

Input data (as was observed and explained in Chapter 4) for Site 16a is summarised in Table 7.7.

Table 7.7: Model input parameters (Site 16a)

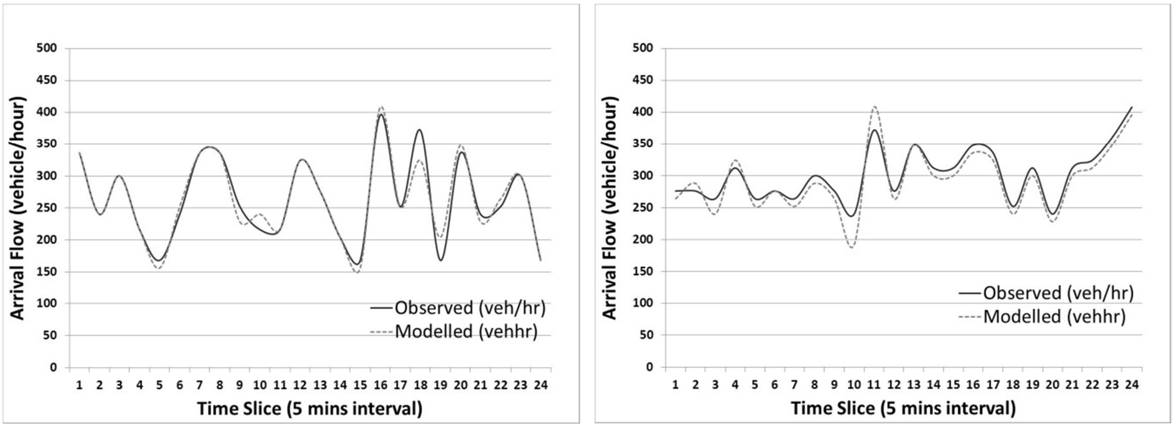
Observed Characteristics	Parameter	Primary Stream	Secondary Stream
Flow	**Arrival Flow (veh/hr)**	263	303
	HGVs (%)	5.5	3.5
Site	**Site Length (m)**	52	
	RDW sign (m)	59	52
Signal	**GT (sec)**	20	20
	All-Red (sec)	5	
Safety	***R_{ACL} (%)**	18.0	12.0
	R_{ACF} (%)	8.0	1.5
	R_{RCF} (%)	1.6	0.0

RDW sign: the distance between the “Roadworks Start Sign” and the stop line
* Calibrated value obtained from Table 7.4

b. Flow and throughput

Site 16a traffic flow data (observed data for 2 hours period) were compared with simulation model output for each stream and for each 5-minutes interval. Figure 7.8 shows the observed and modelled traffic flow data for each 5-minutes interval and for each stream separately. The statistical goodness-of-fit measures for the assigned flow and signals throughput are reported in Table 7.8.

It can be seen from Figure 7.8 that the arrival flow in the simulation model is in good agreement with the real observed data for each 5-minutes interval and for both primary and secondary streams. It can also be seen from Table 7.8 that all the six statistical goodness-of-fit results for both flow and throughput for each 5-minutes interval are satisfactory.



(a) Primary Stream (b) Secondary Stream

Figure 7.8: Model calibration – observed vs. modelled flow data (Site 16a)

Table 7.8: Model calibration statistics-flow and throughput (Site 16a)

Statistical Test	Primary Stream		Secondary Stream	
	Flow	Throughput	Flow	Throughput
RMSEP %	5.6	6.4	11.5	12.3
r	0.997	0.96	0.89	0.87
U	0.02	0.03	0.07	0.09
U_m	0.57	0.04	0.00	0.00
U_s	0.20	0.04	0.00	0.00
GEH	1.07	1.22	0.91	0.50

Throughput per cycle between observed and modelled data at cycle level was also compared to ensure that the model will not have a lower/higher maximum throughput which will affect the system capacity when the model application stage is carried out. The maximum throughput per cycle is observed to be 10 vehicles at oversaturated cycles which were also equal to the modelled throughput per cycle from the simulation model.

c. ***Following headway***

Results of the average time headway for vehicles in platoons (following headway ≤ 6 seconds) between observed and modelled data were compared for the vehicles in both situations (i.e. BAR and ACR) as shown in Table 7.9. It can be seen from Table 7.9 that the average time headway between real observed data and simulation model output for both streams and for both situations (BAR and ACR) are in good agreement with a maximum difference of less than 0.25 seconds in all situations.

It can also be seen from Table 7.9 that the percentage of drivers violating the two-seconds rule are also in good agreement between modelled and observed data for both streams and all situations with a maximum difference of -4%. The final comparison is the total number of vehicles in platoons is also in good agreement between modelled and observed data for all situations with a maximum difference of 9% (34 vehicles).

Table 7.9: Model calibration results – time headway (Site 16a)

Headway criteria	Location	Primary Stream			Secondary Stream		
		Observed	Modelled	Diff.	Observed	Modelled	Diff.
Avg. Headway (sec)	**(BAR)**	3.46	3.40	-0.06	3.15	3.02	-0.13
Avg. Headway (sec)	**(ACR)**	2.49	2.36	-0.13	2.56	2.31	-0.25
< 2.0 (%)	**(BAR)**	13%	11%	-2%	17%	19%	2%
< 2.0 (%)	**(ACR)**	38%	41%	3%	33%	29%	-4%
≤ 6.0 (veh)	**(BAR)**	242	221	-21	284	286	2
≤ 6.0 (veh)	**(ACR)**	355	389	34	423	448	25

d. Drivers' compliance

Drivers' non-compliance with traffic signals were also reported in details (amber crossing and red light violations) and were compared between observed and simulation model output as shown in Table 7.10 (for each stream separately and for both default and calibrated R_{ACL} value). Drivers' non-compliance was calibrated using various sets of R_{ACL} percentage as explained earlier. It can be seen from Table 7.10 that the total number of amber crossing and red light violations for both streams is in good agreement between the real observed and model simulation data. For violation breakdown for both categories (i.e. vehicles and cycles), more details are shown in Appendix C.

Table 7.10: Model calibration statistics – signals compliance (Site 16a)

R_{ACL}	Violation	Primary Stream		Secondary Stream	
		Obs.	Mod.	Obs.	Mod.
Default/observed R_{ACL}	**Total crossing on Amber & Red (veh)**	34	39	49	60
	Total crossing on Amber & Red (%)	6%	7%	8%	10%
Calibrated R_{ACL}	**Total crossing on Amber & Red (veh)**	34	33	49	49
	Total crossing on Amber & Red (%)	6%	6%	8%	8%

Obs. is the observed value from site Mod. is the simulation model output

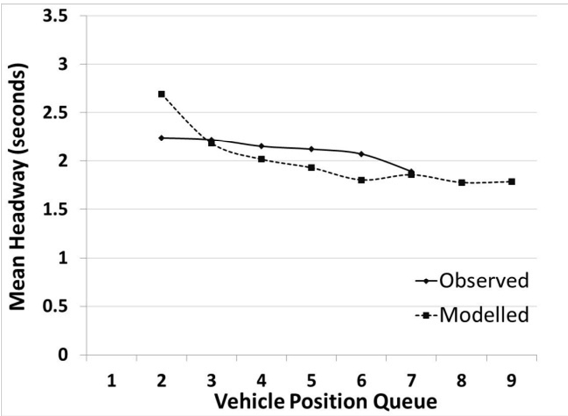
e. Move-up time (MUT)

Move-up time (MUT) is also reported and a comparison between observed and simulation output was conducted as shown in Table 7.11 (total MUT for all vehicles) and presented graphically in Figure 7.9 (for each vehicle) for both streams. It can be seen from both Table 7.11 and Figure 7.9 that the MUT results between observed and modelled data for both streams are in good agreement with a difference of -2% and -6% for primary and secondary streams, respectively. For complete MUT comparison for each vehicle position in the queue, see Appendix C.

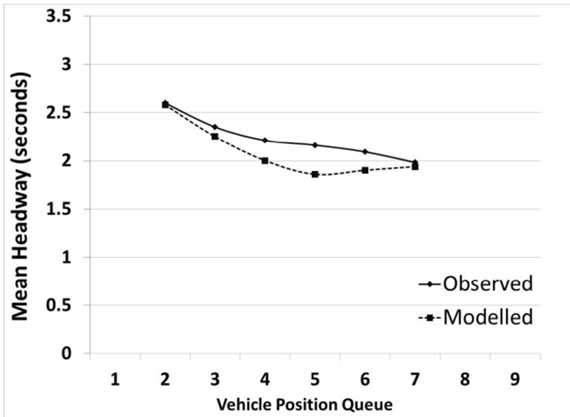
Table 7.11: Model calibration statistics – total MUT (Site 16a)

MUT	MUT (sec)	
	Observed	**Modelled**
Total	(P) 12.69 (S) 13.39	(P) 12.48 (S) 12.53

P is Primary stream S is Secondary stream



(a) Primary Stream



(b) Secondary Stream

Figure 7.9: Model calibration – MUT for each vehicle (Site 16a)

f. ***<u>Queues</u>***

Queues are reported and a comparison between the observed and simulation model output is shown in Table 7.12 for each stream separately. According to Dowling *et al.*, (2002), maximum queue (in vehicles) is the maximum observed queue in any 5-minutes interval (over the simulation period). It is a useful measure that needs to be observed and compared between real data and simulation model to indicate if the queues will spill back to the next junction. Average queue in any 5-minutes interval and total queued vehicles over the simulation period are also useful measures to report on and compare with observed and modelled data.

It can be seen from Table 7.12 that the differences in maximum queues are -6.7% and 7.1% for primary and secondary stream, respectively; while the difference in average queues is -5.0% and -3.9% for primary and secondary stream, respectively. The difference in the total reported queues over the simulation period is 3.9% and -3.5% for primary and secondary stream, respectively. According to Lee (2008), the validation criterion for the simulated maximum and average queue is to be within ±20% of the observed value. Therefore, it can be concluded that reported simulation queues are in good agreement with the real observed queues.

Table 7.12: Model calibration statistics-queues (Site 16a)

Queue Measure	Primary Stream			Secondary Stream		
	Observed	Modelled	Diff.	Observed	Modelled	Diff.
Maximum queue (veh)	15	14	-6.7%	14	15	7.1%
Average queue (veh)	4.0	3.8	-5.0%	5.1	4.9	-3.9%
Total queued vehicles (veh)	357	371	3.9%	483	466	-3.5%

7.4.3.2 Vehicle Actuated signals (VA)

a. Input Parameters

Input data (as was observed and explained in Chapter 4) for Site 12 is summarised in Table 7.13.

Table 7.13: Model input parameters (Site 12)

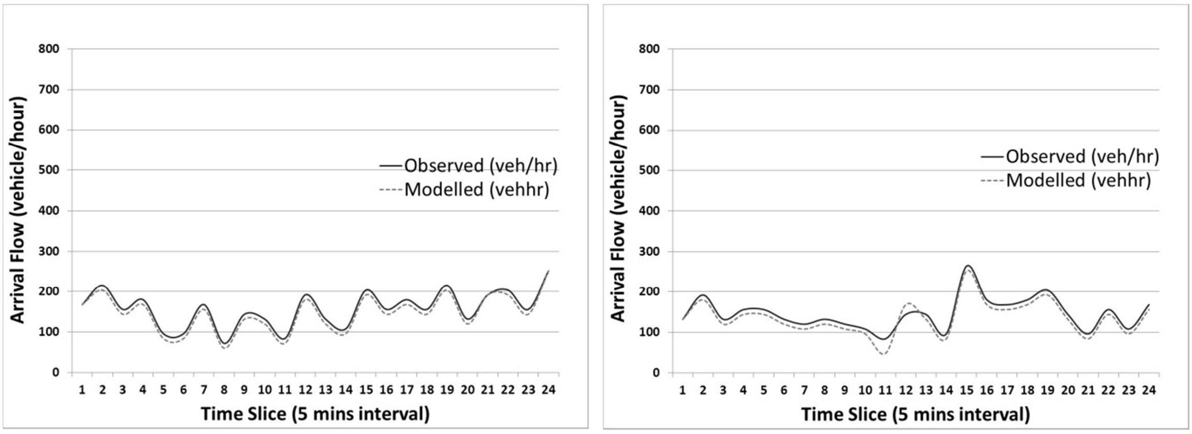
Observed Characteristics	Parameter	Primary Stream	Secondary Stream
Flow	**Arrival Flow (veh/hr)**	158	147
	HGVs (%)	8.5	9.5
Site	**Site Length (m)**	107	
	RDW sign (m)	40	42
Signal	**GT (min, max) (sec)**	(12,88)	(12,60)
	All-Red (sec)	20	
Safety	***R_{ACL} (%)**	3.0	0.5
	R_{ACF} (%)	0.0	0.0
	R_{RCF} (%)	0.0	0.0

RDW sign: the distance between the "Roadworks Start Sign" and the stop line
* Calibrated value obtained from Table 7.4

b. Flow and throughput

Site 12 traffic flow data (observed data for 2 hours period) was compared with simulation model output for each stream and for each 5-minutes interval. Figure 7.10 shows the observed and modelled traffic flow data for each 5-minutes interval and for each stream separately. The statistical goodness-of-fit measures for the assigned flow and signals throughput for the 5-minutes interval are reported in Table 7.14. These results were obtained after calibrating the shift value (in the shifted-negative exponential distribution for vehicle arrival).

It can be seen from Figure 7.10 that the simulation model is in good agreement with the real observed data for each 5-minutes interval and for both primary and secondary streams. It can also be seen from Table 7.14 that all the six statistical goodness-of-fit results for both flow and throughput for each 5-minutes interval are satisfactory on all tests as they are within the limits as suggested by Hourdakis *et al*. (2003).



(b) Primary Stream (b) Secondary Stream

Figure 7.10: Model calibration – observed vs. modelled flow data (Site 12)

Table 7.14: Model calibration statistics-flow and throughput (Site 12)

Statistical Test	Primary Stream		Secondary Stream	
	Flow	Throughput	Flow	Throughput
RMSEP %	8.6	12.5	11.3	11.6
r	1.00	0.95	0.90	0.96
U	0.04	0.05	0.10	0.10
U_m	0.68	0.14	0.00	0.00
U_s	0.16	0.01	0.01	0.00
GEH	1.28	1.74	2.83	2.69

c. ***Following headway***

Results of the average time headway for vehicles in platoons (following headway ≤ 6 seconds) were compared between observed and modelled data for vehicles before approaching roadworks (BAR) and after crossing roadworks (ACR) as shown in Table 7.15. It can be seen from Table 7.15 that average time headway between real observed data and simulation model output for both stream and for both situations (BAR and ACR) are in good agreement with a maximum difference of less than 0.48 seconds in all situations.

It can also be seen from Table 7.15 that the percentage of drivers violating the two-seconds rule are also in good agreement between modelled and observed data for both streams and all situations with a difference of less than 7%. The final comparison of the total number of vehicles in platoons is also in good agreement between the modelled and observed data for all situations with a maximum difference of 9% (13 vehicles).

Table 7.15: Model calibration results-headway (Site 12)

Headway criteria	Location	Primary Stream			Secondary Stream		
		Observed	Modelled	Diff.	Observed	Modelled	Diff.
Avg. Headway (sec)	**(BAR)**	3.19	3.31	0.12	3.17	3.02	-0.15
Avg. Headway (sec)	**(ACR)**	2.90	2.42	-0.48	2.85	2.44	-0.41
< 2.0 (%)	**(BAR)**	24%	17%	-7%	19%	20%	1%
< 2.0 (%)	**(ACR)**	27%	27%	0%	20%	22%	2%
≤ 6.0 (veh)	**(BAR)**	86	93	7	70	75	5
≤ 6.0 (veh)	**(ACR)**	147	160	13	155	160	5

d. Drivers' compliance

Drivers' non-compliance with traffic signals was also reported in details (amber crossing and red light violations) and were compared between observed and simulation output as shown in Table 7.16 for each stream separately (for both the default and calibrated R_{ACL} value). Drivers' non-compliance was calibrated using various sets of R_{ACL} percentage as explained earlier .It can be seen from Table 7.16 that the total number of amber crossing and red violations for both streams is in good agreement between the real observed and model simulation data. For violation breakdown for both categories (i.e. vehicles and cycles), more details are shown in Appendix C.

Table 7.16: Model calibration statistics-compliance (Site 12)

R_{ACL}	Violation	Primary Stream		Secondary Stream	
		Obs.	Mod.	Obs.	Mod.
Default/observed R_{ACL}	**Total crossing on Amber & Red (veh)**	3	6	1	4
	Total crossing on Amber & Red (%)	1%	2%	0.2%	1%
Calibrated R_{ACL}	**Total crossing on Amber & Red (veh)**	3	3	1	2
	Total crossing on Amber & Red (%)	1%	1%	0.2%	1%

Obs. is the observed value from site Mod. is the simulation model output

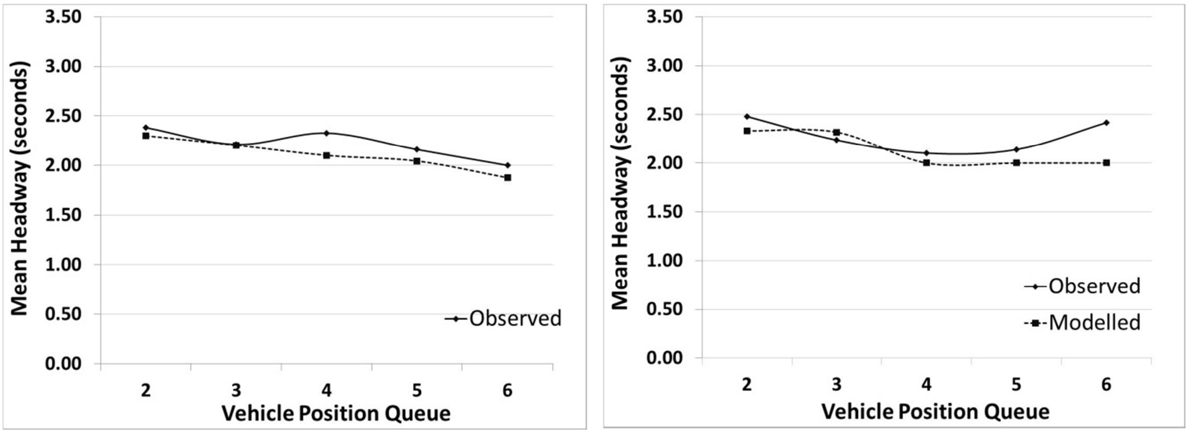
e. ***Move-up time (MUT)***

Move-up time (MUT) is also reported and a comparison between observed and simulation output was conducted as shown in Table 7.17 and presented graphically in Figure 7.11 for both streams. It can be seen from both Table 7.17 and Figure 7.11 that the MUT results between observed and modelled data for both streams are in good agreement with a difference of -5% and -6% for the primary and secondary streams, respectively. For complete MUT comparison for each vehicle position in the queue, see Appendix C.

Table 7.17: Model calibration statistics – total MUT (Site 12)

MUT	**MUT (sec)**	
	Observed	**Modelled**
Total	(P) 11.08 (S) 11.37	(P) 10.52 (S) 10.65

P is Primary stream S is Secondary stream



(b) Primary Stream (b) Secondary Stream

Figure 7.11: Model calibration – MUT for each vehicle (Site 12)

f. ***Queues***

Queues are reported and comparison between observed and simulation model output is as shown in Table 7.18 for each stream separately. It can be seen from Table 7.18 that the differences in maximum queues are 14.3% and 0% for primary and secondary stream, respectively; while the difference in average queues is -12.1% and -16.1% for primary and secondary stream, respectively. The difference in the total reported queues over the simulation period is 6.0% and -9.1% for the primary and secondary stream, respectively.

Therefore, it can be concluded that reported simulation queues are in good agreement with the real observed queues.

Table 7.18: Model calibration statistics-queues (Site 12)

Queue Measure	Primary Stream			Secondary Stream		
	Observed	Modelled	Diff.	Observed	Modelled	Diff.
Maximum queue (veh)	7	8.0	14.3%	7	7	0.0%
Average queue (veh)	3.3	2.9	-12.1%	3.1	2.6	-16.1%
Total queued vehicles (veh)	248	263	6.0%	263	239	-9.1%

7.5 Model validation process

In the previous section, the main two parts of the developed micro-simulation model (i.e. car following and shuttle-lane rules) were calibrated and tested using various data from real traffic information that was observed on sites. However, there is a need to check the overall model performance against independent sets of data from the same site or from different sites. The model validation was also divided into two groups (categories), Fixed Time signals sites (FT) and Vehicle Actuated signals sites (VA). Sites that were used in the validation for each category are shown below.

⇨ Fixed Time signals (FT):
- Site 16b
- Site 19

⇨ Vehicle Actuated signals (VA):
- Site 17
- Site 18

7.5.1 Fixed Time signals (FT)

a. <u>Input Parameters</u>

Input data (as was observed and explained in Chapter 4) for Site 16b and Site 19 is summarised in Tables 7.19 and 7.20, respectively.

Table 7.19: Model input parameters (Site 16b)

Observed Characteristics	Parameter	Primary Stream	Secondary Stream
Flow	Arrival Flow (veh/hr)	383	379
	HGVs (%)	2	1
Site	Site Length (m)	52	
	RDW sign (m)	59	52
Signal	GT (sec)	20	20
	All-Red (sec)	5	
Safety	*R_{ACL} (%)	8.0	23.0
	R_{ACF} (%)	3.1	0.0
	R_{RCF} (%)	4.0	0.0

RDW sign: the distance between the "Roadworks Start Sign" and the stop line
* Calibrated value
NA surveyed only for one hour

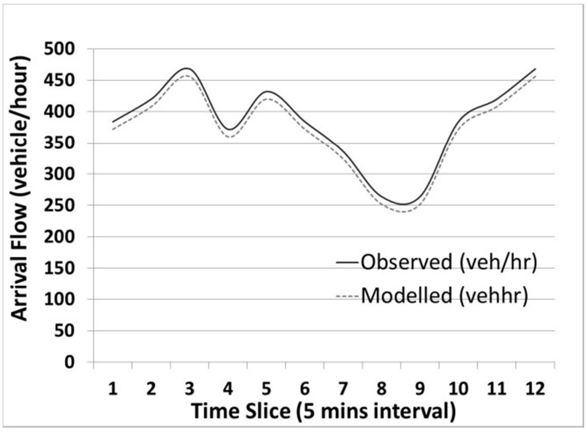
Table 7.20: Model input parameters (Site 19)

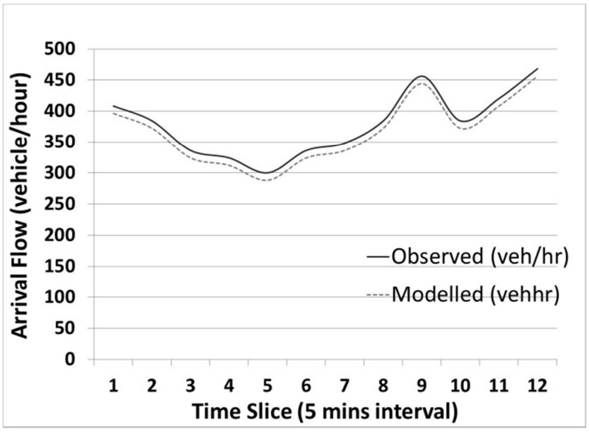
Observed Characteristics	Parameter	Primary Stream	Secondary Stream
Flow	Arrival Flow (veh/hr)	553	408
	HGVs (%)	3	4
Site	Site Length (m)	38	
	RDW sign (m)	54	42
Signal	GT (sec)	50	35
	All-Red (sec)	10	
Safety	*R_{ACL} (%)	21.0	25.0
	R_{ACF} (%)	16.5	18.2
	R_{RCF} (%)	26.1	12.4

RDW sign: the distance between the "Roadworks Start Sign" and the stop line
* Calibrated value obtained from Table 7.4

b. Flow and throughput

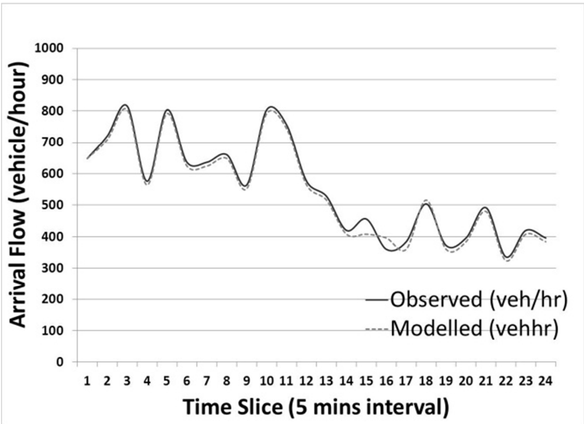
Traffic flow data for Site 16b (observed for 1 hour period) and Site 19 (observed for 2 hours period) were compared with the simulation model output for each stream and for each 5-minutes interval. Figures 7.12 and 7.13 show the observed and modelled traffic flow data for each 5-minutes interval and for each stream separately. The statistical goodness-of-fit measures for the assigned flow and signals throughput for the 5-minutes interval are reported in Tables 7.21 and 7.22. These results were obtained after calibrating the shift value (in the shifted-negative exponential distribution for vehicle arrival).

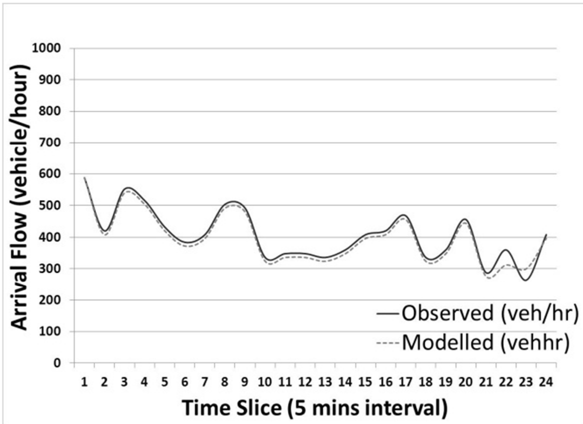




(c) Primary Stream (b) Secondary Stream

Figure 7.12: Model validation – observed vs. modelled flow data (Site 16b)





(a) Primary Stream (b) Secondary Stream

Figure 7.13: Model validation – observed vs. modelled flow data (Site 19)

Table 7.21: Model validation statistics-flow and throughput (Site 16b)

Statistical Test	Primary Stream		Secondary Stream	
	Flow	Throughput	Flow	Throughput
RMSEP %	7.1	11.1	3.3	3.2
R	0.989	0.998	0.97	0.97
U	0.04	0.03	0.08	0.00
U_m	0.02	0.19	0.00	0.00
U_s	0.00	0.01	0.00	0.00
GEH	2.06	1.37	2.18	2.46

Table 7.22: Model validation statistics-flow and throughput (Site 19)

Statistical Test	Primary Stream		Secondary Stream	
	Flow	Throughput	Flow	Throughput
RMSEP %	3.9	4.8	13.0	12.5
r	0.99	0.99	0.95	0.93
U	0.02	0.02	0.12	0.11
U_m	0.06	0.06	0.00	0.00
U_s	0.00	0.00	0.00	0.00
GEH	0.93	1.03	0.49	1.08

It can be seen from Figures 7.12 and 7.13 that the simulation model for both sites (Site 16b and Site 19) is in good agreement with the real observed data for each 5-minutes interval and for both the primary and secondary streams. It can also be seen from Tables 7.21 and 7.22 that all six statistical goodness-of-fit results for both flow and throughput for each 5-minutes interval and for both sites are satisfactory on all tests as they are within the acceptable limits.

Throughput per cycle was also compared between observed and modelled data at cycle level to ensure that the model will not have a lower/higher maximum throughput which will affect the system capacity when the model application stage is carried out. The observed maximum throughput per cycle is also equal to the modelled throughput per cycle from the simulation model for each site and each stream.

c. Following headway

Results of the average time headway for vehicles in platoons (following headway $\leq$ 6 seconds) were compared between observed and modelled data for vehicles before approaching roadworks (BAR) and after crossing roadworks (ACR) as shown in Tables 7.23 and 7.24 for Site 16b and 19, respectively.

It can be seen from Table 7.23 (Site 16b) that average time headway between real observed data and simulation model output for both stream and for both situations (BAR and ACR) are in good agreement with a maximum difference of less than 0.35 seconds in all situations. It can also be seen that the percentage of drivers who violate the two-seconds rule are also in good agreement between modelled and observed data for both streams and all situations with a difference of less than 6%. The final comparison of the total number of vehicles in platoons is also in good agreement between modelled and observed data for all situations with a difference of a maximum of 7% (13 vehicles).

Table 7.23: Model validation results-headway (Site 16b)

Headway criteria	**Location**	**Primary Stream**			**Secondary Stream**		
		Observed	**Modelled**	**Diff.**	**Observed**	**Modelled**	**Diff.**
Avg. Headway (sec)	**(BAR)**	3.46	3.27	-0.19	3.15	2.96	-0.19
Avg. Headway (sec)	**(ACR)**	2.49	2.15	-0.34	2.56	2.21	-0.35
< 2.0 (%)	**(BAR)**	16%	19%	3%	21%	27%	6%
< 2.0 (%)	**(ACR)**	39%	41%	2%	35%	41%	6%
≤ 6.0 (veh)	**(BAR)**	141	134	-7	154	162	8
≤ 6.0 (veh)	**(ACR)**	189	202	13	216	229	13

It can be seen from Table 7.24 (Site 19) that the average time headway between real observed data and simulation model output for both streams and for both situations (BAR and ACR) are in good agreement with a maximum difference of less than 0.20 seconds in all situations. It can also be seen that the percentage of drivers who violate the two-seconds rule are also in good agreement between modelled and observed data for both streams and all situations with a difference of less than 9%. The final comparison of the total number of vehicles in platoons is also in good agreement between modelled and observed for all situations with a difference of a maximum of 6% (57 vehicles).

Table 7.24: Model validation results-headway (Site 19)

Headway criteria	**Location**	**Primary Stream**			**Secondary Stream**		
		Observed	**Modelled**	**Diff.**	**Observed**	**Modelled**	**Diff.**
Avg. Headway (sec)	**(BAR)**	2.48	2.66	0.18	2.75	2.79	0.04
Avg. Headway (sec)	**(ACR)**	2.33	2.15	-0.18	2.33	2.13	-0.20
< 2.0 (%)	**(BAR)**	38%	32%	-6%	30%	21%	-9%
< 2.0 (%)	**(ACR)**	47%	52%	5%	45%	48%	3%
≤ 6.0 (veh)	**(BAR)**	932	875	-57	571	555	-16
≤ 6.0 (veh)	**(ACR)**	958	974	16	707	698	-9

d. __Drivers' compliance__

Drivers' non-compliance with traffic signals was also reported in details (amber crossing and red light violations) and was compared between observed and simulation output for both sites (Site 16b and Site 19) as shown in Tables 7.25 and 7.26 for each stream separately. Drivers' non-compliance was calibrated using various sets of R_{ACL} percentage as explained earlier. It can be seen from Tables 7.25 and 7.26 that the total number of amber crossing and red light violations for both streams is in good agreement between the real observed and model simulation data. For violation breakdown for both categories (i.e. vehicles and cycles), see Appendix C.

Table 7.25: Model validation statistics-compliance (Site 16b)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Total crossing on Amber & Red (veh)	34	34	25	26
Total crossing on Amber & Red (%)	9%	9%	7%	7%

Table 7.26: Model validation statistics-compliance (Site 19)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Total crossing on Amber & Red (veh)	59	57	49	53
Total crossing on Amber & Red (%)	5%	5%	6%	7%

e. Move-up time (MUT)

Move-up time (MUT) is also reported and a comparison between observed and simulation output was conducted as shown in Tables 7.27 and 7.28 and presented graphically in Figures 7.14 and 7.15 for Site 17 and Site 18, respectively. For Site 16b, it can be seen from both Table 7.27 and Figure 7.14 that the MUT results between observed and modelled data for both streams are in good agreement with a difference of 3% and 0% for primary and secondary streams, respectively. For Site 19, it can be seen from both Table 7.28 and Figure 7.15 that the MUT results between observed and modelled data for both streams are in good agreement with a difference of 3% and 5% for the primary and secondary streams, respectively. For complete MUT comparison for each vehicle position in the queue, see Appendix C.

Table 7.27: Model validation statistics – total MUT (Site 16b)

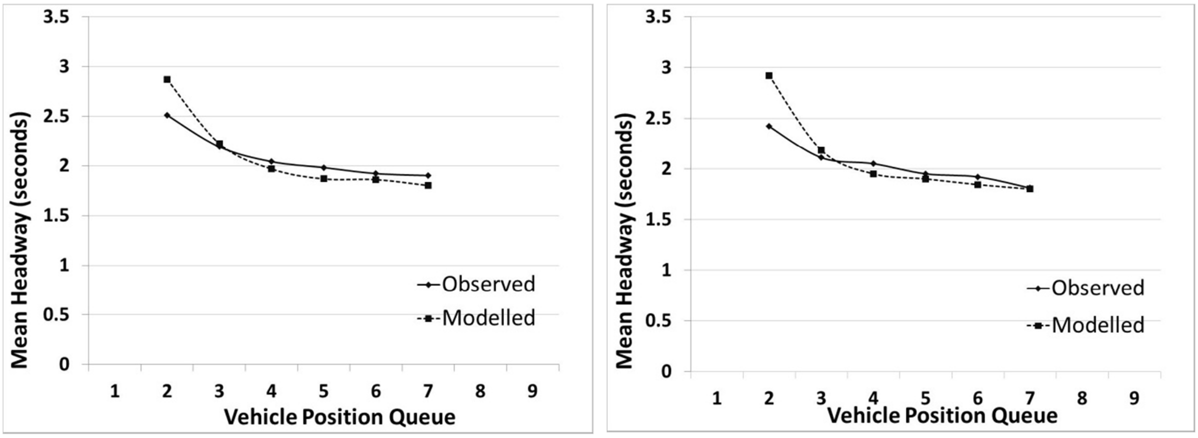
MUT	MUT (sec)	
	Observed	Modelled
Total	(P) 12.26 (S) 12.54	(P) 12.59 (S) 12.60

P is Primary stream S is Secondary stream

Table 7.28: Model validation statistics – total MUT (Site 19)

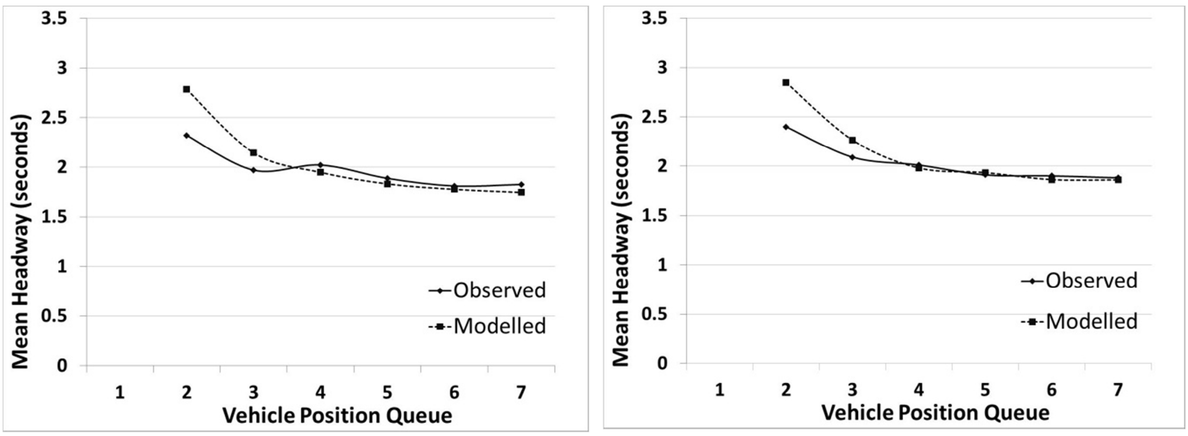
MUT	MUT (sec)	
	Observed	Modelled
Total	(P) 11.83 (S) 12.19	(P) 12.23 (S) 12.75

P is Primary stream S is Secondary stream



(a) Primary Stream (b) Secondary Stream

Figure 7.14: Model validation – MUT for each vehicle (Site 16b)



(a) Primary Stream (b) Secondary Stream

Figure 7.15: Model validation – MUT for each vehicle (Site 19)

f. ***Queues***

Queues are reported and comparison made between observed and simulation model output as shown in Tables 7.29 and 7.30 for Site 16b and Site 19, respectively. It can be seen from Table 7.29 (Site 16b) that the differences in maximum queues are -11.1% and 5.6% for primary and secondary streams, respectively; while the difference in average queues is -7.9%

and 13.7% for the primary and secondary streams, respectively. The difference in the total reported queues over the simulation period is 1.3% and 3.1% for primary and secondary streams, respectively.

It can be seen from Table 7.30 (Site 19) that the differences in maximum queues are 3.6% and 4.5% for the primary and secondary streams, respectively; while the difference in average queues is 7.0% and 2.7% for the primary and secondary streams, respectively. The difference in the total reported queues over the simulation period is 6.8% and 8.3% for the primary and secondary streams, respectively. It can be concluded that reported simulation queues for both sites (Site 16b and Site 19) are in good agreement with the real observed queues as all queue measures for both sites are within acceptable limits (within ±20% of the observed value).

Table 7.29: Model validation statistics-queues (Site 16b)

Queue Measure	Primary Stream			Secondary Stream		
	Observed	Modelled	Diff.	Observed	Modelled	Diff.
Maximum queue (veh)	18	16	-11.1%	18	19	5.6%
Average queue (veh)	6.3	5.8	-7.9%	5.1	5.8	13.7%
Total queued vehicles (veh)	303	307	1.3%	294	303	3.1%

Table 7.30: Model validation statistics-queues (Site 19)

Queue Measure	Primary Stream			Secondary Stream		
	Observed	Modelled	Diff.	Observed	Modelled	Diff.
Maximum queue (veh)	28	29	3.6%	22	23	4.5%
Average queue (veh)	12.9	13.8	7.0%	11.1	11.4	2.7%
Total queued vehicles (veh)	751	802	6.8%	569	616	8.3%

7.5.2 Vehicle Actuated signals (VA)

a. <u>Input Parameters</u>

Input data (as was observed and explained in Chapter 4) for Site 17 and Site 18 are summarised in Tables 7.31 and 7.32, respectively.

Table 7.31: Model input parameters (Site 17)

Observed Characteristics	Parameter	Primary Stream	Secondary Stream
Flow	Arrival Flow (veh/hr)	204	223
	HGVs (%)	5	5
Site	Site Length (m)	39	
	RDW sign (m)	89	114
Signal	GT (min, max) (sec)	(12,54)	(12,72)
	All-Red (sec)	3	
Safety	*R_{ACL} (%)	1.0	1.0
	R_{ACF} (%)	0.0	0.0
	R_{RCF} (%)	0.0	0.0

RDW sign: the distance between the "Roadworks Start Sign" and the stop line
* Calibrated value obtained from Table 7.4
NA surveyed only for one hour

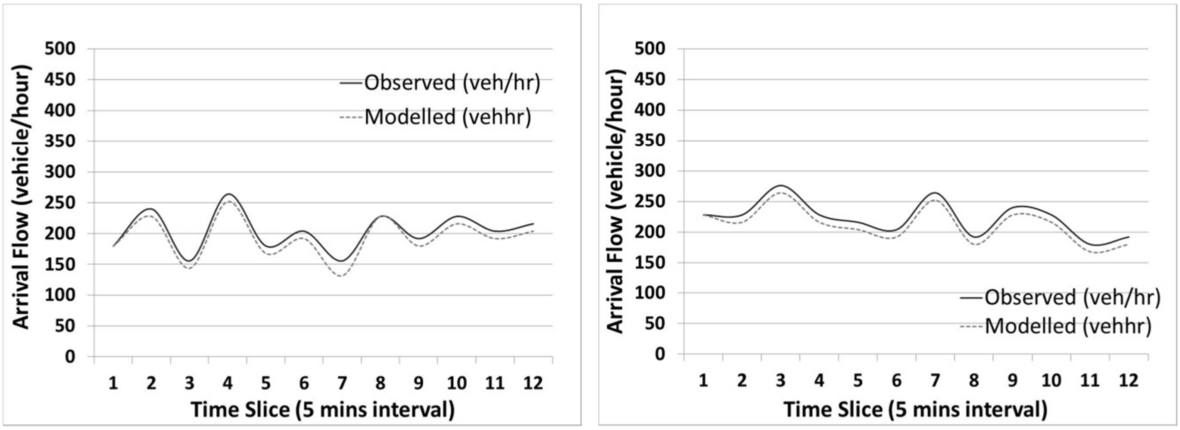
Table 7.32: Model input parameters (Site 18)

Observed Characteristics	Parameter	Primary Stream	Secondary Stream
Flow	Arrival Flow (veh/hr)	179	313
	HGVs (%)	3	3
Site	Site Length (m)	73	
	RDW sign (m)	91	99
Signal	GT (min, max) (sec)	(12,76)	(12,78)
	All-Red (sec)	3	
Safety	*R_{ACL} (%)	0.5	1.5
	R_{ACF} (%)	0.0	0.0
	R_{RCF} (%)	0.0	0.0

RDW sign: the distance between the "Roadworks Start Sign" and the stop line
* Calibrated value
** observed for 30 minutes (not full hour)

b. Flow and throughput

Traffic flow data for Site 17 (observed for 1 hour period) and Site 18 (observed for 1.5 hours period) was compared with simulation model output for each stream and for each 5-minutes interval. Figures 7.16 and 7.17 show the observed and modelled traffic flow data for each 5-minutes interval and for each stream separately. The statistical goodness-of-fit measures for the assigned flow and signals throughput for the 5-minutes interval are reported in Tables 7.33 and 7.34 These results were obtained after calibrating the shift value (in the shifted-negative exponential distribution for vehicle arrival).



(a) Primary Stream (b) Secondary Stream

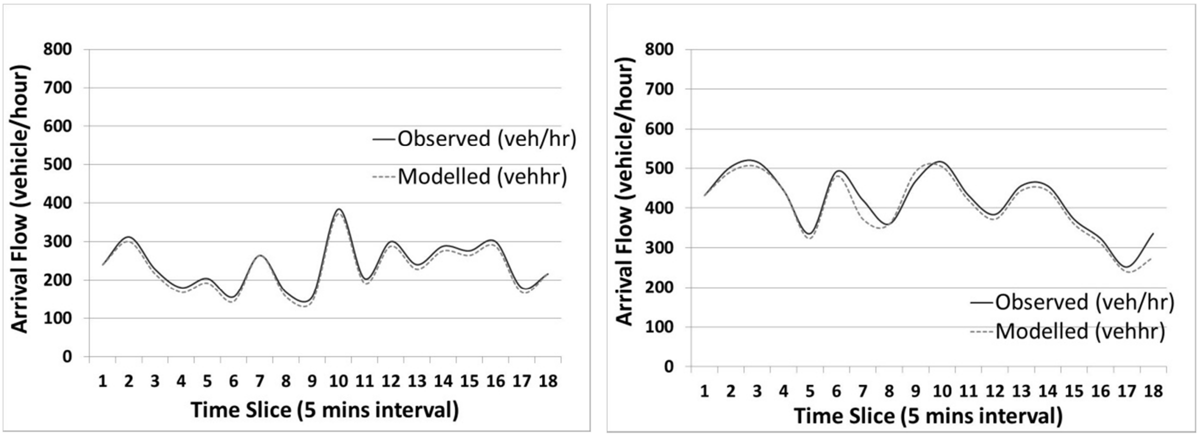
Figure 7.16: Model validation – observed vs. modelled flow data (Site 17)

Table 7.33: Model validation statistics-flow and throughput (Site 17)

Statistical Test	Primary Stream		Secondary Stream	
	Flow	Throughput	Flow	Throughput
RMSEP %	11.4	13.3	9.4	8.3
R	0.95	0.87	0.90	0.88
U	0.04	0.04	0.05	0.06
U_m	0.06	0.04	0.00	0.00
U_s	0.04	0.01	0.03	0.01
GEH	1.45	1.40	1.38	2.10

Table 7.34: Model validation statistics-flow and throughput (Site 18)

Statistical Test	Primary Stream		Secondary Stream	
	Flow	Throughput	Flow	Throughput
RMSEP %	5.0	6.8	11.3	11.4
r	0.99	0.96	0.87	0.96
U	0.02	0.03	0.10	0.10
U_m	0.02	0.02	0.00	0.00
U_s	0.04	0.01	0.00	0.00
GEH	0.95	1.33	2.94	2.17



(a) Primary Stream (b) Secondary Stream

Figure 7.17: Model validation-observed vs. modelled flow data (Site 18)

It can be seen from Figures 7.16 and 7.17 that the simulation model for both sites (Site 17 and Site 18) is in good agreement with the real observed data for each 5-minutes interval and for both the primary and secondary streams. It can also be seen from Tables 7.33 and 7.34 that all six statistical goodness-of-fit results for both flow and throughput for each 5-minutes interval and for both sites are satisfactory on all tests as they are within the limits.

c. Following headway

Results of the average time headway for the vehicles in platoons (following headway ≤ 6 seconds) were compared between observed and modelled data for vehicles before approaching roadworks (BAR) and after crossing roadworks (ACR) for both sites as shown in Tables 7.35 and 7.36 for Site 17 and 18, respectively.

It can be seen from Table 7.35 (Site 17) that the average time headway between real observed data and simulation model output for both streams and for both situations (BAR and ACR) is in good agreement with a maximum difference of less than 0.35 seconds in all situations.

It can also be seen from Table 7.35 that the percentage of drivers who violate the two-seconds rule are also in good agreement between the modelled and observed data for both streams and all situations with a difference of less than 9%. The final comparison of the total number of vehicles in platoons is also in good agreement between the modelled and observed data for all situations with a difference of a maximum of 19% (21 vehicles).

It can be seen from Table 7.36 (Site 18) that average time headway between real observed data and simulation model output for both streams and for both situations (BAR and ACR) is in good agreement with a maximum difference of less than 0.18 seconds in all situations. It can also be seen that the percentage of drivers who violate the two-seconds rule are also in good agreement between modelled and observed data for both streams and all situations with a difference less than 7%. The final comparison of the total number of vehicles in platoons is also in good agreement between modelled and observed data for all situations with a difference of a maximum of 11% (18 vehicles).

Table 7.35: Model validation results-headway (Site 17)

Headway criteria	Location	Primary Stream			Secondary Stream		
		Observed	Modelled	Diff.	Observed	Modelled	Diff.
Avg. Headway (sec)	**(BAR)**	3.41	3.10	-0.31	3.49	3.14	-0.35
Avg. Headway (sec)	**(ACR)**	2.52	2.34	-0.18	2.49	2.52	0.03
< 2.0 (%)	**(BAR)**	17%	13%	-4%	19%	17%	-2%
< 2.0 (%)	**(ACR)**	42%	43%	1%	39%	30%	-9%
≤ 6.0 (veh)	**(BAR)**	58	68	10	78	79	1
≤ 6.0 (veh)	**(ACR)**	108	129	21	123	138	15

Table 7.36: Model validation results-headway (Site 18)

Headway criteria	Location	Primary Stream			Secondary Stream		
		Observed	Modelled	Diff.	Observed	Modelled	Diff.
Avg. Headway (sec)	**(BAR)**	3.26	3.27	0.01	2.97	2.79	-0.18
Avg. Headway (sec)	**(ACR)**	2.44	2.26	-0.18	2.41	2.32	-0.09
< 2.0 (%)	**(BAR)**	20%	13%	-7%	26%	25%	-1%
< 2.0 (%)	**(ACR)**	49%	52%	3%	44%	37%	-7%
≤ 6.0 (veh)	**(BAR)**	160	142	-18	358	363	5
≤ 6.0 (veh)	**(ACR)**	240	263	23	475	489	14

d. Drivers' compliance

Drivers' non-compliance with traffic signals was also reported in details (amber crossing and red light violations) and was compared between observed and simulation output for both sites (Site 17 and Site 18) as shown in Tables 7.37 and 7.38 for each stream separately. Drivers' non-compliance was calibrated using various sets of R_{ACL} percentage as explained earlier. It can be seen from Tables 7.37 and 7.38 that the total number of amber crossing and red light violations for both streams are in good agreement between the real observed and model simulation data. For violation breakdown for both categories (i.e. vehicles and cycles), see Appendix C.

Table 7.37: Model validation statistics-compliance (Site 17)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Total crossing in Amber & Red (veh)	1	0	1	3
Total crossing in Amber & Red (%)	0%	0%	0%	1%

Table 7.38: Model validation statistics-compliance (Site 18)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Total crossing in Amber & Red (veh)	4	5	6	8
Total crossing in Amber & Red (%)	1%	1%	1%	1%

e. <u>Move-up time (MUT)</u>

Move-up time (MUT) is also reported and a comparison between observed and simulation output was conducted as shown in Tables 7.39 and 7.40 and presented graphically Figures 7.18 and 7.19 for Site 17 and Site 18, respectively. For Site 17, it can be seen from both Table 7.39 and Figure 7.18 that the MUT results between the observed and modelled data for both streams are in good agreement with a difference of -6% and -3% for the primary and secondary streams, respectively.

For Site 18, it can be seen from both Table 7.40 and Figure 7.19 that the MUT results between the observed and modelled data for both streams are in good agreement with a difference of 9% and -4% for the primary and secondary streams, respectively. For complete MUT comparison for each vehicle position in the queue, see Appendix C.

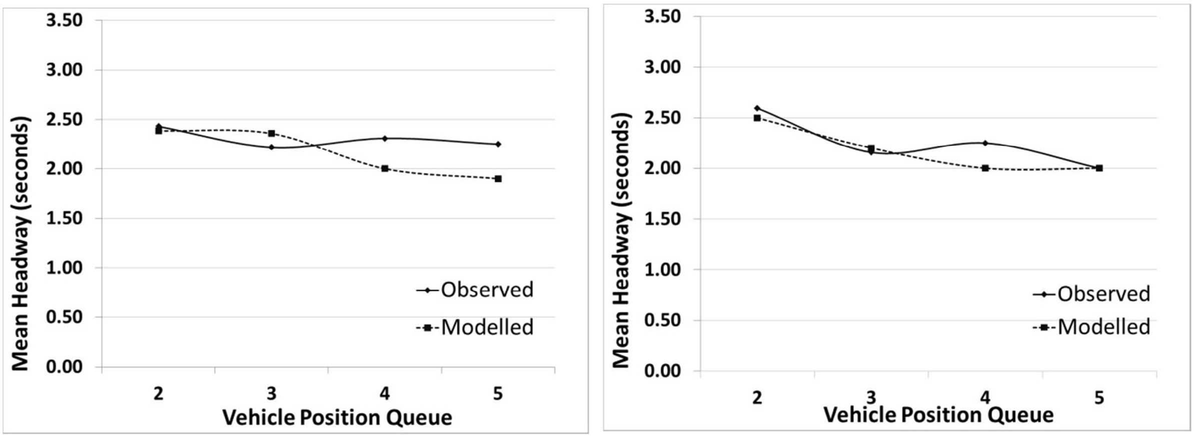
Table 7.39: Model validation statistics – total MUT (Site 17)

MUT	MUT (sec)	
	Observed	Modelled
Total	(P) 9.21 (S) 9.00	(P) 8.64 (S) 8.70

P is Primary stream S is Secondary stream

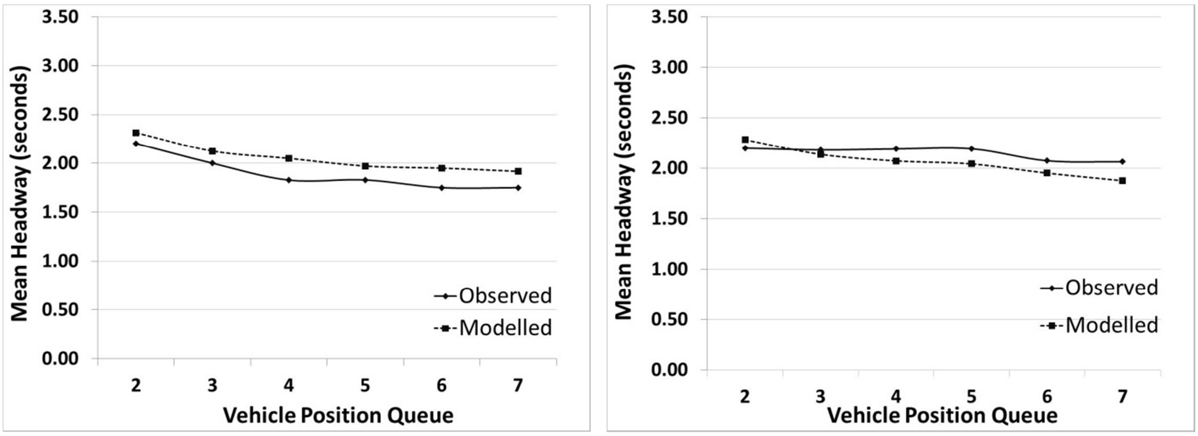
Table 7.40: Model validation statistics – total MUT (Site 18)

MUT	MUT (sec)	
	Observed	Modelled
Total	(P) 11.36 (S) 12.90	(P) 12.33 (S) 12.36



(b) Primary Stream (b) Secondary Stream

Figure 7.18: Model validation – MUT for each vehicle (Site 17)



(b) Primary Stream (b) Secondary Stream

Figure 7.19: Model validation – MUT for each vehicle (Site 18)

f. ***Queues***

Queues are reported and compared between observed and simulation output as shown in Tables 7.41 and 7.42 for Site 17 and Site 18, respectively. It can be seen from Table 7.41 (Site 17) that the difference in maximum queues are 16.7% and 0% for the primary and secondary streams, respectively; while the difference in average queues is -8.7% and -17.2% for primary and secondary streams, respectively. The difference in the total reported queues over the simulation period is 11.2% and 3.3% for primary and secondary stream, respectively.

It can be seen from Table 7.42 (Site 18) that the difference in maximum queues is 10% and -7.7% for primary and secondary streams, respectively; while the difference in average queues is -10% and -17.3% for the primary and secondary streams, respectively. The difference in the total reported queues over the simulation period is 6.2% and13% for the primary and secondary stream, respectively. Therefore, it can be concluded that reported simulation queues are in good agreement with the real observed queues.

Table 7.41: Model validation statistics-queues (Site 17)

Queue Measure	Primary Stream			Secondary Stream		
	Observed	Modelled	Diff.	Observed	Modelled	Diff.
Maximum queue (veh)	6	7	16.7%	9	9	0.0%
Average queue (veh)	2.3	2.1	-8.7%	2.9	2.4	-17.2%
Total queued vehicles (veh)	232	258	11.2%	275	284	3.3%

Table 7.42: Model validation statistics-queues (Site 18)

Queue Measure	Primary Stream			Secondary Stream		
	Observed	Modelled	Diff.	Observed	Modelled	Diff.
Maximum queue (veh)	10	11	10.0%	13	12.0	-7.7%
Average queue (veh)	4.0	3.6	-10.0%	5.2	4.3	-17.3%
Total queued vehicles (veh)	325	345	6.2%	345	390	13.0%

7.6 Summary

The current chapter presented the verification, calibration and validation of the car-following and shuttle-lane rules as well as the calibration/validation of the whole SIMSUR simulation model using real observed traffic data from various surveyed shuttle-lane roadworks sites. The results showed the validity of SIMSUR model results for various (short to long) site length (38m-107m), various flow (uncongested and congested) levels (134 veh/hr – 888 veh/hr) and for both FT and VA signals.

Therefore, SIMSUR simulation model provides a sound basis for testing the effect of different scenarios on the traffic conditions at shuttle-lane roadworks. The next chapter shows the model applications that have been carried out.

CHAPTER EIGHT: SIMSUR MODEL APPLICATION

8.1 Introduction

The current chapter presents a few of the possible applications of the newly developed SIMSUR micro-simulation model for testing different management scenarios of shuttle-lane roadworks operated by temporary traffic signals. Various parameters were also tested using the developed micro-simulation model to study their effect on capacity and delays (i.e. site length, signals operation type, HGVs % and directional split).

The developed simulation model was also used to test improved shuttle-lane roadworks scenarios and their effect on capacity, delays and in reducing aggressive drivers' behaviour.

8.2 The effect of site length on system capacity and delays

Site length has an effect on shuttle-lane roadworks site capacity. However, previous studies did not take into account various factors (i.e. up-to-date signals specifications, amber crossing and close following behaviour), which are believed to have an impact on shuttle-lane system capacity and delays.

For the current test, various site lengths were tested (site length of 25m, 50m to 300m with an increment of 50m). Various flow levels for two-way flows (veh/hr) were also tested (flow level of 250-3000 with 250 veh increment). Table 8.1 lists the main parameters that have been used to test the effect of site length on shuttle-lane roadworks capacity and delays. All other vehicle and drivers' characteristics were kept fixed. Over 1,600 simulation runs were used to test this parameter.

Table 8.1: Input data for testing the effect of site length on system capacity and delays

Operation type	Site length (m)	two-way flow (veh/hr)	HGVs %	Direction split (P/S)
FT	25, 50-300	250-3000	5	50/50
VA				

8.2.1 Effect on system capacity

According to Summersgill (1981), the maximum site capacity achieved for shuttle-lane roadworks operated by temporary traffic signals is around 1590 veh/hr for two-way flow (as described in Section 2.6.6).

Figure 8.1 shows the relationship (effect) between site length and site capacity for shuttle-lane roadworks operated by temporary traffic signals for both FT and VA modes. In general, as the site length increases, the capacity decreases from 1,860 veh/hr to 1,240 veh/hr (for two-way flow) for FT mode. For VA mode, as the site length increases, the capacity decreases from 2,060 veh/hr to 1,350 veh/hr (for two-way flow).

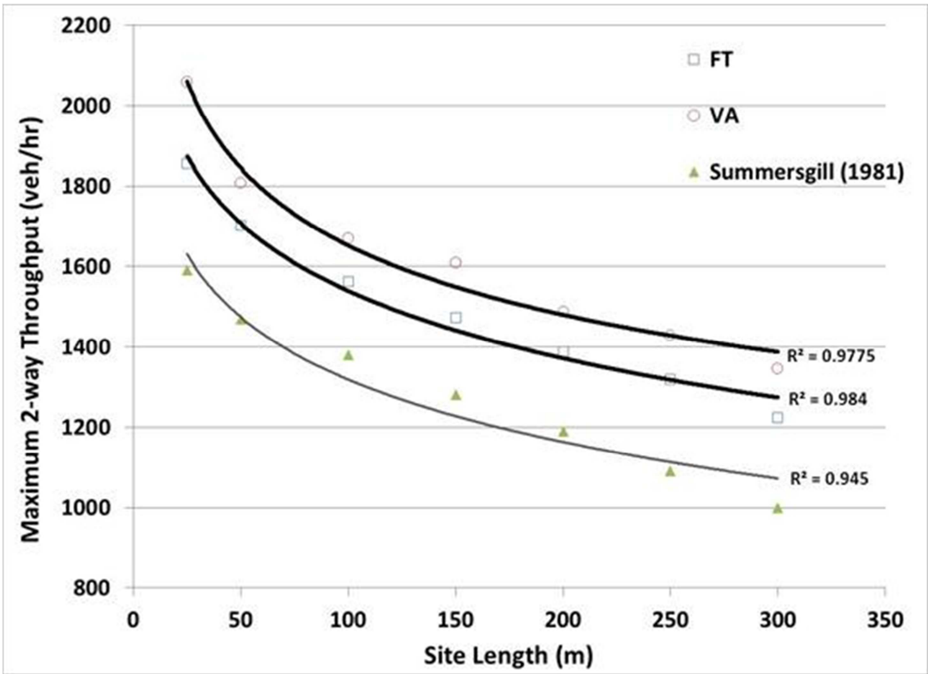


Figure 8.1: Effect of site length on shuttle-lane roadworks throughput

The reduction in capacity is around 23 and 26 veh/hr for every 10 metres of site length for FT and VA modes, respectively. Maximum system throughput are higher than the results achieved by Summersgill (1981) because of the latest signals settings and specifications as well as various different assumptions (i.e. amber crossing and close following that were observed on site and the assumption of directional split). The amount of vehicles crossing on amber and red due to the presence of dilemma zone are 186 and 103 veh/hr (for 25 metres site length) for FT and VA modes, respectively.

8.2.2 Effect on delays

Testing the effect of introducing shuttle-lane roadworks operated by temporary traffic signals on drivers' delays was also tested. The test was carried out by comparing the same input data with and without the presence of roadworks as shown in Figure 8.2 for FT and VA modes.

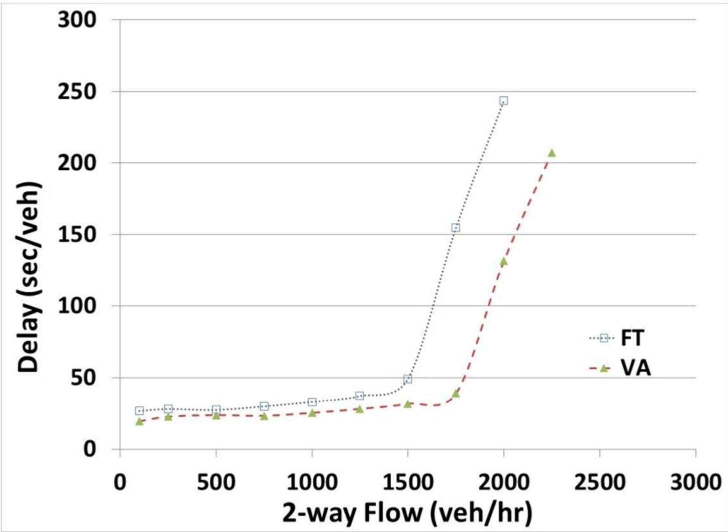


Figure 8.2: Effect of shuttle-lane roadworks site length on drivers' delays

It can be seen from Figure 8.2 that the delay curves follow the typical delay curves for signalised junctions as presented by HCM (1985) and Rouphail *et al*. (1996). It can also be seen that the VA mode reduced average vehicle delays by 14% for flow levels of 1,500 veh/hr for two-way flow. The VA mode also has lower delays per vehicle at maximum capacity. Drivers' delays could be higher when directional split is slightly imbalanced (i.e. directional split of 40/60 or 70/30).

8.3 The effect of drivers' non-compliance on system capacity

Testing the effect of drivers' non-compliance (red light violations) with temporary traffic signals on shuttle-lane roadworks capacity was carried out using various non-compliance percentages (0-40% with 10% increment) as shown in Table 8.2.

Table 8.2: Input data for testing the effect of drivers' non-compliance on system capacity

Operation type	Site length (m)	two-way flow (veh/hr)	HGVs %	Direction split (P/S)	Non-Compliance %
VA	100	250-3000	5	50/50	0-40 (10% increment)

Figure 8.3 illustrates the effect of different percentages of drivers' non-compliance with temporary traffic signals at shuttle-lane roadworks system capacity. The results show that throughput could increase when drivers' non-compliance increases up to 20% (maximum throughput of approximately 1,740 veh/hr (two-way)). The throughput might decrease when drivers' non-compliance increases over 20%. This could be attributed to the fact that drivers violating the temporary traffic signals on one stream (e.g. primary stream) will restrict the movement of the other stream (e.g. secondary stream) when the lights show green (occupying the shuttle-lane roadworks site).

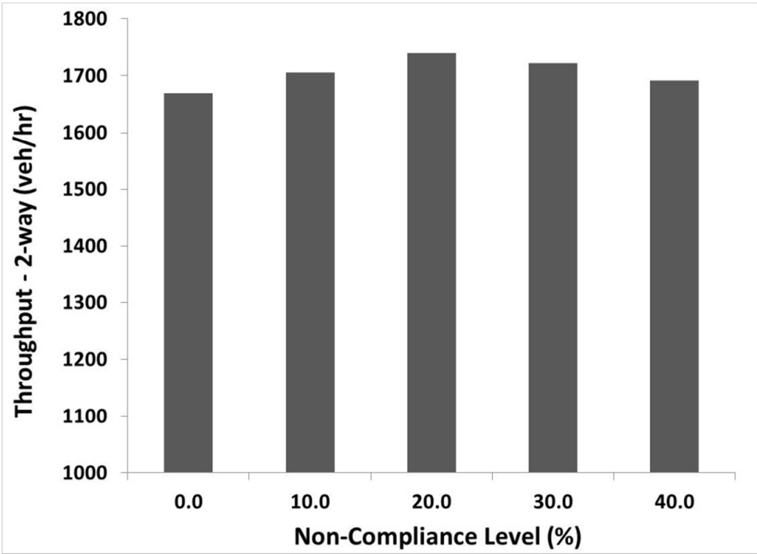


Figure 8.3: Effect of drivers' non-compliance on shuttle-lane roadworks site capacity

8.4 The effect of HGVs percentage on system capacity

Previous studies have suggested that the percentage of Heavy Goods Vehicles (HGVs) has a negative impact on the system capacity and could be related to the following reasons:

- ⇨ HGVs have longer lengths than cars which will increase headways and hence reduce capacity.
- ⇨ HGVs have lower acceleration rate abilities (ITE, 2010).
- ⇨ HGVs have lower desired speeds than those of small cars (Yousif, 1993).

To test the effect of HGVs percentage on shuttle-lane roadworks capacity, various HGVs percentages between 0 and 30 (with 5 percent interval) were used as shown in Table 8.3.

Table 8.3: Input data for testing the effect of HGVs percentage on system capacity

Operation type	Site length (m)	two-way flow (veh/hr)	HGVs %	Direction split (P/S)
VA	100	250-3000	0-30 (5% interval)	50/50

Figure 8.4 illustrates the effect of different HGVs percentage on shuttle-lane roadworks capacity. As the percentage of HGVs increases, the capacity decreases from approximately 1,740 veh/hr (two-way) at 0.0% HGVs to approximately 1,340 veh/hr (two-way) at 30.0% (approximate reduction of 23% in system throughput). This shows the great effect of HGVs presence on system capacity.

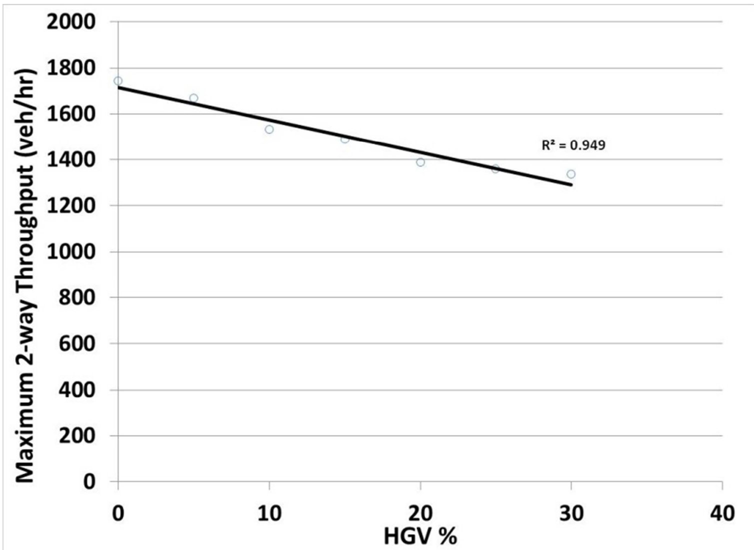


Figure 8.4: Effect of HGVs percentage on shuttle-lane roadworks capacity

8.5 The effect of directional split on system capacity

Testing the effect of directional split on shuttle-lane roadworks capacity was carried out using various directional split proportions for the primary and secondary streams as shown in Table 8.4. The tested directional splits are 50/50, 60/40, 70/30 and 80/20.

Table 8.4: Input data for testing the effect of HGVs percentage on system capacity

Operation type	Site length (m)	two-way flow (veh/hr)	HGVs %	Direction split (P/S)
VA	100	250-3000	5	50/50, 60/40, 70/30, 80/20
FT				

Figure 8.5 illustrates the effect of different directional splits on maximum system throughput for both FT and VA signals operation. The results show that for FT signals operation, the more the imbalanced directional flow (i.e. 80/20) will have a more adverse impact on system throughput and capacity reduction. The system throughput decreased from approximately 1,560 veh/hr (two-way) for a directional split of 50/50 to approximately 1,220 veh/hr (two-way) for a directional split of 80/20 (approximately a reduction of 21.8% in system throughput).

For VA signals, the throughput is higher than those of FT signals (VA signals can adapt to different directional splits and the throughput reduction only starts with directional split of 70/30 or higher). The system throughput decreased from approximately 1,670 veh/hr (two-way) for a directional split of 50/50 to approximately 1,400 veh/hr (two-way) for a directional split of 80/20 (approximately a reduction of 16.2% in system throughput).

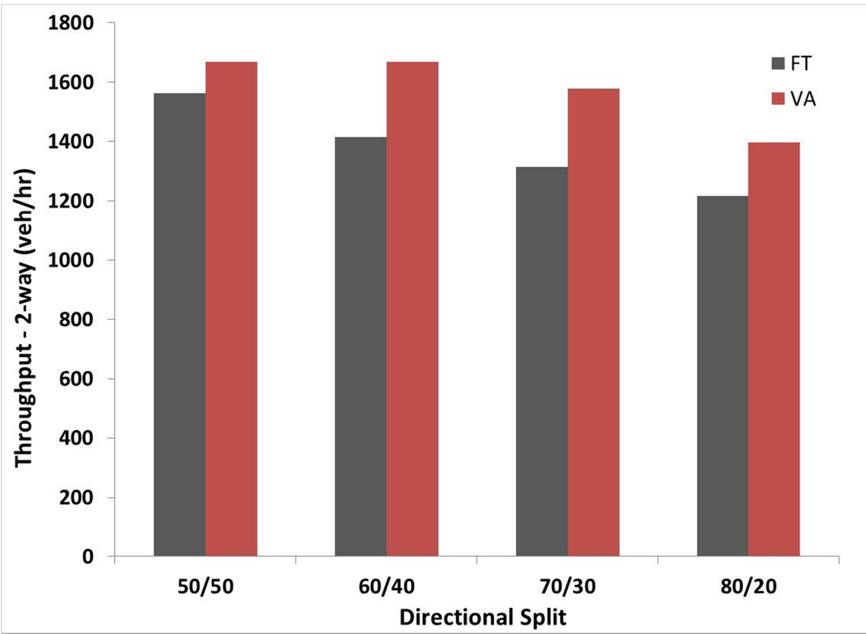


Figure 8.5: Effect of HGVs percentage on shuttle-lane roadworks capacity

8.6 Estimation of maximum system throughput

Following the testing of different factors that affect the maximum throughput of shuttle-lane roadworks in previous sections using the simulation model, several proposed relationships were identified between maximum throughput and different factors (i.e. site length, HGVs%, directional split and type of signals operation). Multiple regression analysis was carried out between the identified factors and the maximum throughput using statistical program (STATISTICA, version 10).

Equation 8.1 shows the results from the regression analysis, which describes the relationship between the maximum throughput and the tested factors.

$$\mathrm{Qm} = 4120 - 2.08(\mathrm{L}) - 18.11(\mathrm{HGV}) - 1140.5(\mathrm{DSp}) - 1708(\mathrm{FT}) - 1578(\mathrm{VA}) \qquad \text{Equation 8.1}$$

Where,

Qm is the maximum two-way throughput of the shuttle-lane roadworks (veh/hr).

L is the site length (metres).

HGVs% is the percentage of Heavy Goods Vehicles.

DSp is the directional split of the primary stream (decimal, i.e. 0.5 for 50%).

FT is the type of signals operation-Fixed Time (1 if true and 0 if false).

VA is the type of signals operation-Vehicle Actuated (1 if true and 0 if false).

The coefficient of determination (R^2) value of the regression analysis is 0.98 which indicates strong relationship between the dependent variable (throughput) with the other independent variables. According to Equation 8.1, as the values of site length (L), HGVs% and DSp increase, the throughput decreases. Whereas, if the signals type is FT signals, the throughput decreases and if it is VA, the throughput increases. This effect seems to be consistent with the description of how these variables affect the throughput of shuttle-lane roadworks as discussed in sections 8.2 to 8.5.

Equation 8.1 could be used as a more accurate representation of the maximum two-way throughput of shuttle-lane roadworks compared to existing equation used in analytical models (i.e. QUADRO), which only depends on one factor (i.e. site length).

8.7 Improved shuttle-lane roadworks operation

The micro-simulation modelling is a very useful tool for testing various types of traffic management scenarios without a real disruption to traffic and with little cost. As described in Section 2.7.3, various limitations have been listed in previous studies (i.e. very low flow, very long site length, no effect of drivers' behaviour, outdated signals specifications) to test an improved operation of shuttle-lane roadworks operated by temporary traffic signals.

Therefore, this step has been taken by this study to improve capacity, reduce delays and improve safety by studying effective factors that influence the shuttle-lane roadworks' capacity and safety, such as type of improved new signals technology and introducing new interventions (i.e. speed reduction techniques).

8.7.1 Signals settings

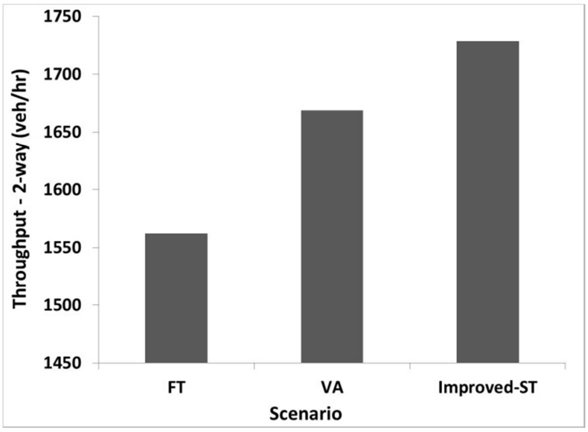
New signals settings have been proposed to improve the operation of shuttle-lane roadworks. The new signals settings are presented in Table 8.5 and the maximum green time has been increased from 90 to 120 seconds. The minimum green time is proposed to be 7 seconds (instead of 12 seconds based on the Highways Agency (2005B) and the all-red period has been modified from 10 seconds (based on the Department for Transport, 2009) to 8 seconds (based on ITE, 2010).

Table 8.5: Input data for testing the effect of new signals settings

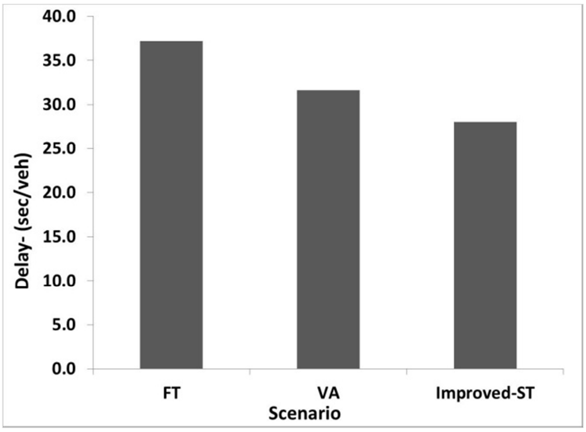
Parameters	Default value	New value
Maximum green time (sec)	90	120
Minimum green time (sec)	12	7
All-red time (sec)	10*	8**
Site length (m)	100	
two-way flow (veh/hr)	250-3000	
HGVs %	5	
Direction split (P/S)	50/50	

* based on the Department of Transport (2009) ** based on ITE (2010)

It can be seen from Figure 8.6 that the new signals settings can increase the site capacity from two-way flow of 1,670 veh/hr to 1,730 veh/hr (an increase of 3.5%) compared with the original VA settings and an increase of about 11% if FT signals were used. Figure 8.6 shows that delays in sec/veh can be reduced from 32 sec/veh to 28 sec/veh (reduction of 11% for VA signals and about 25% when using FT signals) using the new signals settings.



(a) Throughput



(b) Delay

Figure 8.6: Effect of new signals settings on site capacity and delays

8.7.2 Detection range

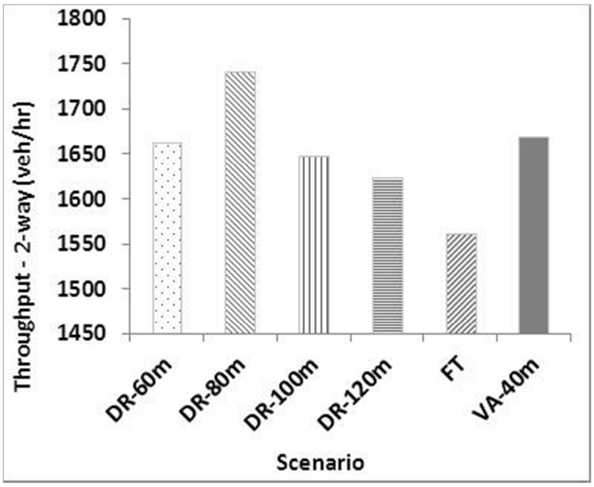
According to the Department for Transport (2008), the MVD detector has a detection range up to 40m. New technology has been adopted by various MVD manufacturers to increase the detection length to 100 metres. There are no previous studies about the effect of such a technology on shuttle-lane roadworks site capacity, delays and dilemma zone. Therefore, various MVD detector ranges (60 to 120 with 20m increment) have been tested using the developed simulation model with the input data as presented in Table 8.6.

Table 8.6: Input data for testing the effect of MVD detection range

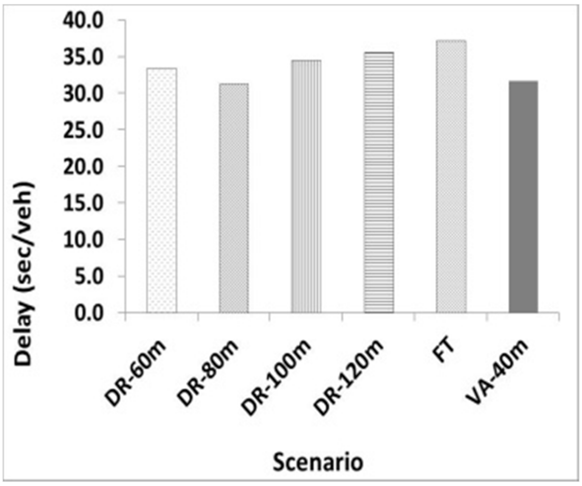
Operation type	Detection range (m)	Site length (m)	two-way flow (veh/hr)	HGVs %	Direction split (P/S)
VA	60-120	100	250-3000	5	50/50

It can be seen from Figure 8.7 that the optimum MVD detection range that provides the maximum site capacity (throughput) is at 80 metres. The capacity could be increased for two-way flow from 1,670 veh/hr to 1,740 veh/hr (by a further 4.2%) when the range increased from 40m to 80m for VA signals and a 12% where FT signals are used.

Figure 8.7 also shows that delays in sec/veh remains the same for a detection range of 80m compared with the existing 40m MVD for VA signals, there is a slight reduction of about 1%. However, this reduction is about 14% when compared with the use of FT signals.



(a) Throughput



(b) Delay

Figure 8.7: Effect of MVD detection range on site capacity and delays

8.7.3 Reduction of dilemma zone effect

As discussed in Section 2.6.9.2, the main factor affecting the length of dilemma zone is vehicle approach speed. Reducing the length of dilemma zone will reduce the risk of accident or near accident (Amer *et al*., 2011 and Medina *et al*., 2012). Practical speed reduction on site could be achieved by either installing temporary 20mph sign, Speed Indicator Devices (SID) or by installing temporary speed hump in advance of shuttle-lane roadworks site stop line.

A temporary 20mph sign might be an effective speed reduction measure if accompanied by other extensive features Department for Transport (2004). The extensive speed reduction features will be inappropriate for temporary installation at shuttle-lane roadworks. On the other hand, Speed Indicator Devices (SID) can have a little effect on speed reduction of up to 1.4 mph, which is not significant as indicated by Walter and Broughton (2011). Therefore, temporary speed hump is being chosen as an effective measure to test using the developed SIMSUR simulation model.

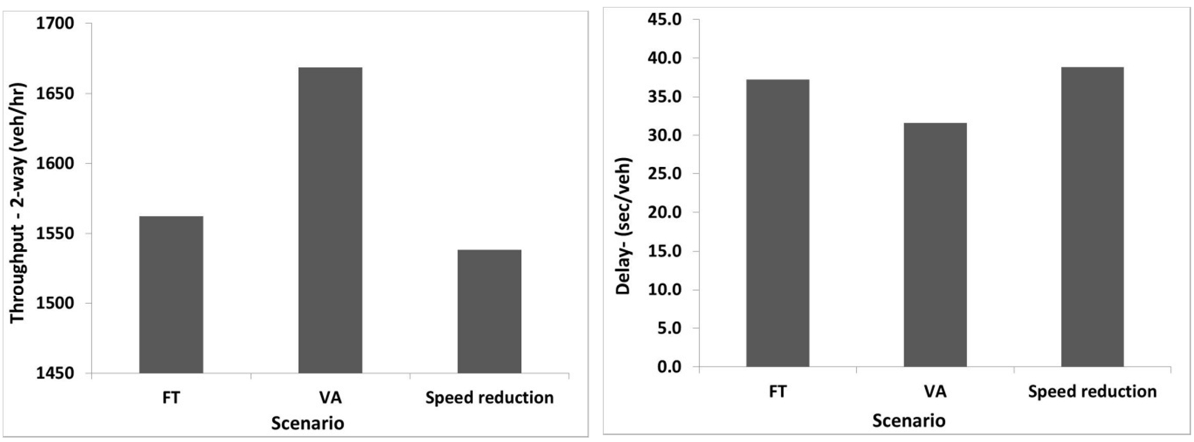
Temporary speed hump (i.e. Pittman, easy rider speed hump) can be installed on the road in less than an hour using 16inch spikes (bolts). According to the Department for Transport (2007) (LTN 1/07), speed humps are more effective in reducing vehicles speed on 30mph roads. The mean vehicle's speed will be 20mph (depending on hump dimensions). Speed reduction were tested in the developed simulation model with input data as shown in Table 8.7 with the position of the speed hump at 30 metres from the stop line.

Table 8.7: Input data for testing the effect of speed reduction

Operation type	Position of speed reduction from stop line (m)	Site length (m)	two-way flow (veh/hr)	HGVs %	Direction split (P/S)
VA	30	100	250-3000	5	50/50

It can be seen from Figure 8.8 that there is a negative impact between the introduction of speed reduction and the increase in site capacity. Site capacity could be reduced for two-way flow from 1,670 veh/hr to 1,540 veh/hr (a reduction of 7.8%) for VA signals and a reduction of 1.3% if FT signals are used.

Figure 8.8 also shows that delays in sec/veh can be increased from 32 sec/veh to 38 sec/veh (increase of 23% for VA signals) and by an increase of about 4.6% if FT signals is used. Figure 8.9 shows that the number of vehicles crossing due to the presence of dilemma zone could be reduced from 102 veh/hr to 69 veh/hr (with a reduction of about 33%) for VA signals and about 64% if FT signals are used. This reduction could decrease the effect of dilemma zone and possible reductions of the risk of rear-end collisions. On the other hand it might reduce site capacity and increase average delay per vehicle.



(a) Throughput (b) Delay

Figure 8.8: Effect of speed reduction on site capacity and delays

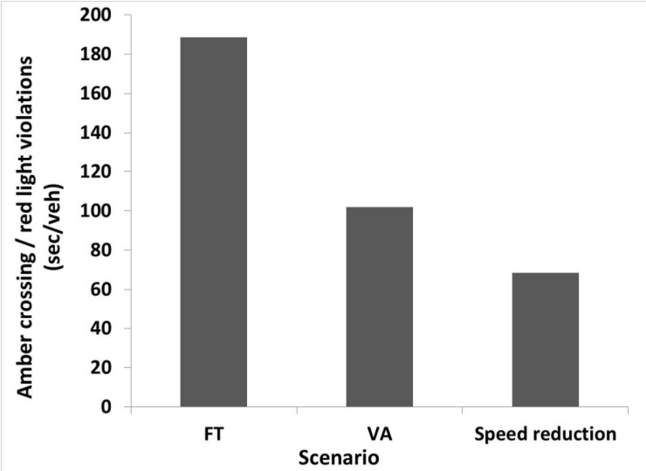


Figure 8.9: Effect of speed reduction on dilemma zone vehicles

8.7.4 Reduction of tailgating behaviour

As discussed in Section 2.6.9.1, tailgating is a dangerous drivers' behaviour and attributes to a high percentage of road traffic accidents. According to Michael *et al*. (2000), the percentage of drivers that are involved in tailgating could be reduced by 8% using active message signs (the study was carried out on urban environment in the United States). These signs warn drivers who follow with headway less than 2 seconds by displaying variable signs (i.e. "*Please Don't Tailgate*" or "*Help Prevent Crashes, Please don't Tailgate*"). Message sign could have a different effect on shuttle-lane roadworks compared with normal urban roads in different countries.

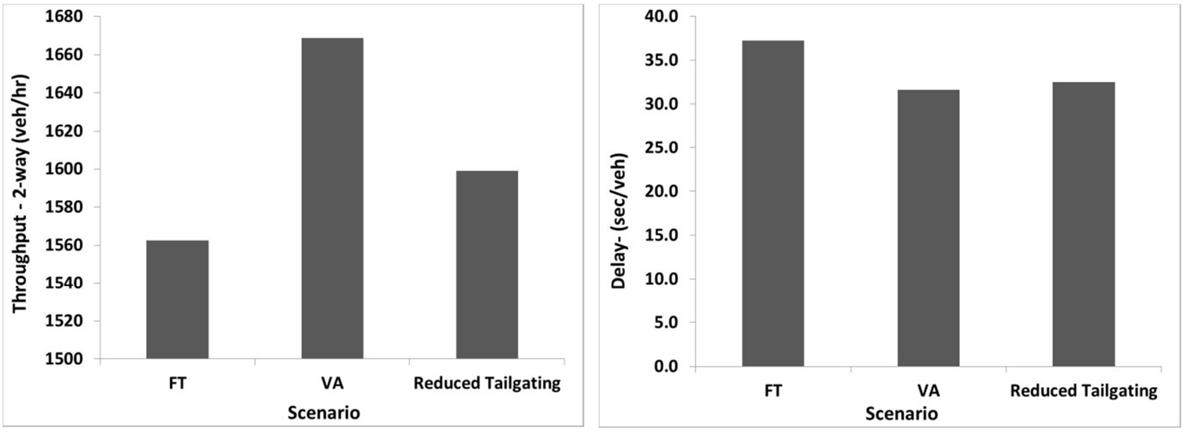
To implement the effect of the above sign, drivers' buffer space was increased from 1.5 metres to 3.0 metres (which will increase time headway between drivers) and test the effect on system capacity, delay and vehicles crossing on amber/red violations. A message sign is being tested in the developed simulation model with input data as shown in Table 8.8.

Table 8.8: Input data for testing the combined improvements

Operation type	Site length (m)	two-way flow (veh/hr)	HGVs %	Direction split (P/S)
VA	100	250-3000	5	50/50

It can be seen from Figure 8.10 that there is a negative impact between the introduction of message signs (assuming 8% reduction on tailgating will be achieved) and the increase in site capacity. Site capacity could be reduced for two-way flow from 1,670 veh/hr to 1,600 veh/hr (reduction of 4.2%) for VA signals and capacity increase by about 2.6% if FT signals are used.

Figure 8.10 also shows that delays in sec/veh could increase from 32 sec/veh to 33 sec/veh (increase of 3%) for VA signals and delay reduction by about 8.2% if FT signals are used as a result of reduced tailgating behaviour. Figure 8.11 shows that the number of vehicles crossing due to the presence of dilemma zone could be reduced from 102 veh/hr to 96 veh/hr (a reduction of about 4%) for VA signals and a possible reduction of about 48.5% if FT signals are used. This reduction could reduce the effect of dilemma zone and the possibilities of reducing the risk of rear-end collisions. On the other hand it might slightly reduce site capacity and increase average delay per vehicle.



(a) Throughput (b) Delay

Figure 8.10: Effect of tailgating reduction on site capacity and delays

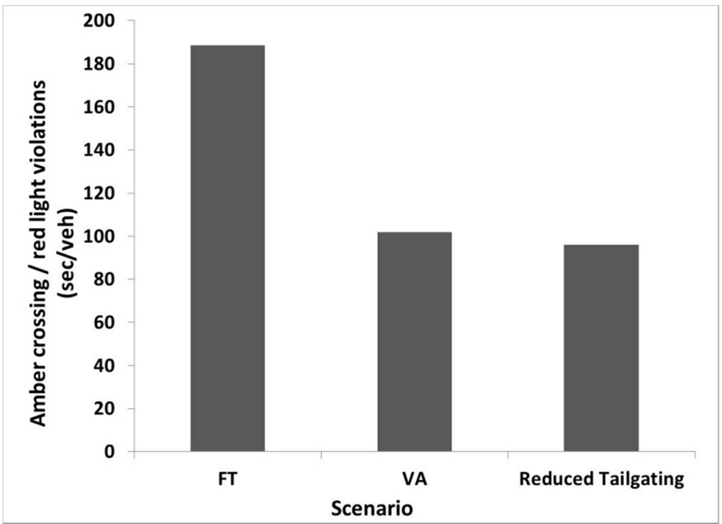


Figure 8.11: Effect of tailgating reduction on dilemma zone vehicles

8.8 Summary

The current chapter presented the application of the developed simulation model in testing various shuttle-lane roadworks scenarios. The main points can be summarised as follows:

- ⇨ The effect of various parameters on shuttle-lane roadworks site capacity and delays were tested (i.e. site length, drivers' non-compliance, HGVs percentage, directional split and signals type). See Sections 8.2 to 8.5.

- ⇨ Regression analysis was carried out using different factors (i.e. site length, HGVs %, directional split and signals operation type). The regression analysis can be used in analytical models to produce more accurate representation of system capacity compared with previous models.

- ⇨ New signals settings were proposed and tested. The new signals settings could improve site capacity and reduce the level of delays (see Section 8.7.1).

- ⇨ Various MVD detection ranges were tested using the developed model and the optimum range was found to be 80m. The new range could increase site capacity and reduce delays (see Section 8.7.2).

- ⇨ Speed reduction scenario was proposed to reduce the effect of dilemma zone on drivers crossing on amber and violating the red light. It was found that speed reduction scenario could reduce the effect of vehicles crossing due to dilemma zone by up to 33% (see Section 8.7.3).

- ⇨ Reduction of tailgating behaviour was tested using the developed simulation model based on previous research by Michael *et al*. (2000). The tailgating behaviour was reduced by 8%. This reduction has little impact on system capacity (could be reduced by reduced by 4%) and delay could be increased by only 3%. The reduction in tailgating behaviour could reduce the amount of drivers crossing on amber/violating the red light by 4% (see Section 8.7.4).

CHAPTER NINE: CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

The main conclusions and findings from this study can be summarised as follows:

- ⇨ Various discrepancies were found in the design guidelines such as site length, maximum allowed flow levels for each method of traffic operation (see Section 2.5). Important factors were not taken into account when selecting or designing each traffic control method (i.e. HGV %, directional split, junction proximity, etc.).

- ⇨ Mathematical and analytical models are inadequate for accurately modelling shuttle-lane roadworks with the limitation of correctly replicating queues and delays. They also lack the comparison with real observed data and the inability to model the effect of any advanced traffic control techniques (see Section 2.7).

- ⇨ Simulation models are designed for under-saturated conditions. Different models also have various limitations, such as omitting vehicles, various parameters are imbedded within the program code that the users do not have access to and the required level of complicated steps to ensure correct behaviour of such a system (see Section 2.7).

- ⇨ None of the mathematical, analytical or simulation models took into account the effect of aggressive drivers' behaviour (i.e. close following "tailgating" and red light running) which may have an impact on site safety and capacity (see Section 2.7).

- ⇨ Large amount of data under six different categories were collected and analysed. Video survey site visits were carried out on 23 different sites with over 54 hours of video recording of traffic data in the area of Greater Manchester (see Chapter 3).

- ⇨ Various factors were analysed from the survey data including traffic flow (5-minutes profile), directional split, HGVs %, vehicle length, arrival headway, following headway (BAR and ACR), move-up time and move-up delay (see Chapter 4).

- ⇨ Two new extra drivers' behaviours were studied at temporary traffic signals which are, close following "tailgating" and drivers' compliance with traffic signals. It is found that the percentage of drivers tailgating increased after crossing the roadworks compared with before approaching the roadworks (see Sections 4.6.2 and 4.11).

- ⇨ Drivers' non-compliance with temporary traffic signals was much higher compared with normal signalised junctions as reported by previous researchers. Drivers' non-compliance was broken down into four categories and full statistics were provided (see Section 4.11).

- ⇨ The developed S-Paramics model fails statistically to replicate flow, throughput and queues. In addition, S-Paramics model could not replicate the aggressive drivers' behaviour of amber crossing/red light violation observed on site and the presence of dilemma zone which has an effect on both safety and capacity. Therefore, it was recommended that a new micro-simulation model needs to be developed as part of the current study to provide more accurate results with the ability to cover S-Paramics limitations (see Chapter 5).

- ⇨ The developed SIMSUR car-following model was calibrated with the real field data. The results showed that the developed simulation model could represent both free following and "stop and go" conditions (which are similar to the conditions in shuttle-lane roadworks operated by temporary traffic signals) with more accurate representation compared with S-Paramics, VISSIM and AIMSUN (see Section 7.4.1).

- ⇨ The developed SIMSUR micro-simulation model shows satisfactory results when compared with the collected field data using various statistical measures and tests. The model was calibrated and validated for both FT and VA traffic signals. Moreover, the calibration and validation results of the developed simulation model show better compliance than the results obtained from the S-Paramics simulation model using the same set of field data (see Section 7.4, 7.5 and 7.6).

- ⇨ The developed SIMSUR simulation model was then applied to test the effect of various factors and new parameters on shuttle-lane roadworks capacity, delays and compliance with traffic signals.

- ⇨ The effect of site length was investigated using the developed simulation model. The relationship shows that when the site length increases, the maximum throughput decreases. The results for capacity are higher than those obtained by Summersgill (1981), see Section 8.2.

- ⇨ The type of signals operation was also investigated. The results show that Vehicle Actuated (VA) signals provided higher throughput compared with Fixed Time (FT) signals for the same flow levels and site lengths. Delays (sec/veh) were also decreased using VA signals, see Section 8.2.

- ⇨ The effect of drivers' non-compliance with temporary traffic signals was investigated using the developed simulation model. The relationship shows that when the percentage of drivers not complying with temporary traffic signals increased up to a maximum of 20%, the maximum throughput increased. When the percentage of drivers' not complying with temporary traffic signals increased (beyond 20%), the maximum throughput decreased because of the blocking of the opposite stream (see Section 8.3).

- ⇨ The effect of HGVs percentage was investigated using the developed simulation model. The relationship shows that when the percentage of HGVs increased, the maximum throughput decreased, see Section 8.4.

- ⇨ The effect of directional split was investigated using the developed simulation model. The relationship shows that for FT signals, when the directional split is imbalanced (i.e. 60/40 or higher), the maximum throughput decreased. For VA signals, when the directional split is (i.e. 50/50 or higher up to 60/40), the maximum throughput is the same and the signals can cope with the imbalanced flow. For imbalanced directional split beyond 70/30, the maximum throughput decreased (see Section 8.5).

- ⇨ Regression analysis was carried out to test the effect of different factors (i.e. site length, HGVs %, directional split and signals operation type). The regression equation may be recommended for use in analytical models (i.e. QUADRO) to provide more accurate representation of system capacity compared with existing equations which only take site length into consideration (see Section 8.6).

- ⇨ Modified signals settings (timings) were proposed and tested using the developed simulation model. The new timings show that the system capacity could be increased by 3.5% and delays were reduced by 11% (for VA signals). If FT signals were used, the system capacity could be increased by 11% and delays could be reduced by 25% (see Section 8.7.1).

- ⇨ Various settings of MVD detection ranges were tested to study the effect on system capacity, delays and dilemma zone crossings. It is found that the optimum detection length would be 80m. The optimum length increased system capacity by an extra 4.2% and delays were slightly increased by 1% (compared with VA signals). If FT signals were used, system capacity could be increased by an extra 12% and delays could be reduced by 14%. See Section 8.7.2.

- ⇨ A speed reduction scenario was proposed and tested to introduce a speed limit of 20 mph at a distance of 30 metres from the stop line using a speed hump. Speed reduction could reduce system capacity by 8%, increased delays by 23% and possibly reduce the amount of drivers' crossing on amber or red (due to the presence of dilemma zone) by 33% (for VA signals). If FT signals were used, system capacity could be reduced by 1.3% and delays could be increased by 4.6% and the amount of vehicles crossing on amber or red could be reduced by 64% (see Section 8.7.3).

- ⇨ Reduction of tailgating behaviour was tested using the developed simulation model based on previous research. The tailgating behaviour was reduced by 8%. This reduction could reduce system capacity by 4.2%, increased delay by only 3% and the amount of drivers crossing on amber/violating the red light could be reduced by 4% (for VA signals). If FT signals were used, system capacity could be reduced by 2.6% and delays could be reduced by 8.2% and the amount of vehicles crossing on amber or red could be reduced by 48.5% (see Section 8.7.4).

9.2 Recommendations and further research

- ⇨ Another type of shuttle-lane operation (i.e. priority operation or stop/go signs) could be modelled to test various parameters and compared with other operation methods by modifying SIMSUR model to account for gap acceptance rules. This could help in identifying the maximum system capacity using other operational methods compared to temporary traffic signals.

- ⇨ The effect of junction proximity on system capacity and delays needs to be studied using various junction proximity lengths and flow levels by carrying out extensive site visits.

This could help in identifying various parameters (e.g. appropriate operational type, signal coordination, signal settings)

- ⇨ Due to the lack of research in studying shuttle-lane roadworks on junctions, it could be studied (using more than 2 sets of traffic signals) to test the effect of other various parameters (i.e. the effect of right turners, minor arm location from the main road, signals settings and timings).
- ⇨ Speed hump (to reduce the effect of dilemma zone) and message signs (to reduce tailgating behaviour) could be tried and tested on real shuttle-lane roadworks sites to have a real estimate of their impact on system capacity and on drivers' behaviour.
- ⇨ More studies on shuttle-lane roadworks sites in different cities and countries could be carried out to test for similarities and provide a more accurate representation.
- ⇨ Studying the platoon characteristics in shuttle-lane roadworks and develop a platoon-based algorithm with the aim to increase site capacity and reduce delays.

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APPENDIX A: Shuttle-lane roadworks sites plans

A.1. Site 11

Site Number: 11

Location: Broughton Lane, Salford (between Arrow Street and Milton Street)

Roadwork operation method: Temporary traffic signal

Site Length: 42 metres

Surveyed: Primary and secondary streams

Date: Monday and Tuesday 09 and 10 July 2012

Time: 09:45 to 11:45 and 15:30 to 17:30

Duration: 6 hours

Weather: cloudy

Comments: the site was filmed on 3 separate occasions. The weather was cloudy on the first occasion so filming from the road was achieved. The second time it was raining so each stream was filmed separately on 2 different days to cover the peak periods and filmed from inside parked vehicle.

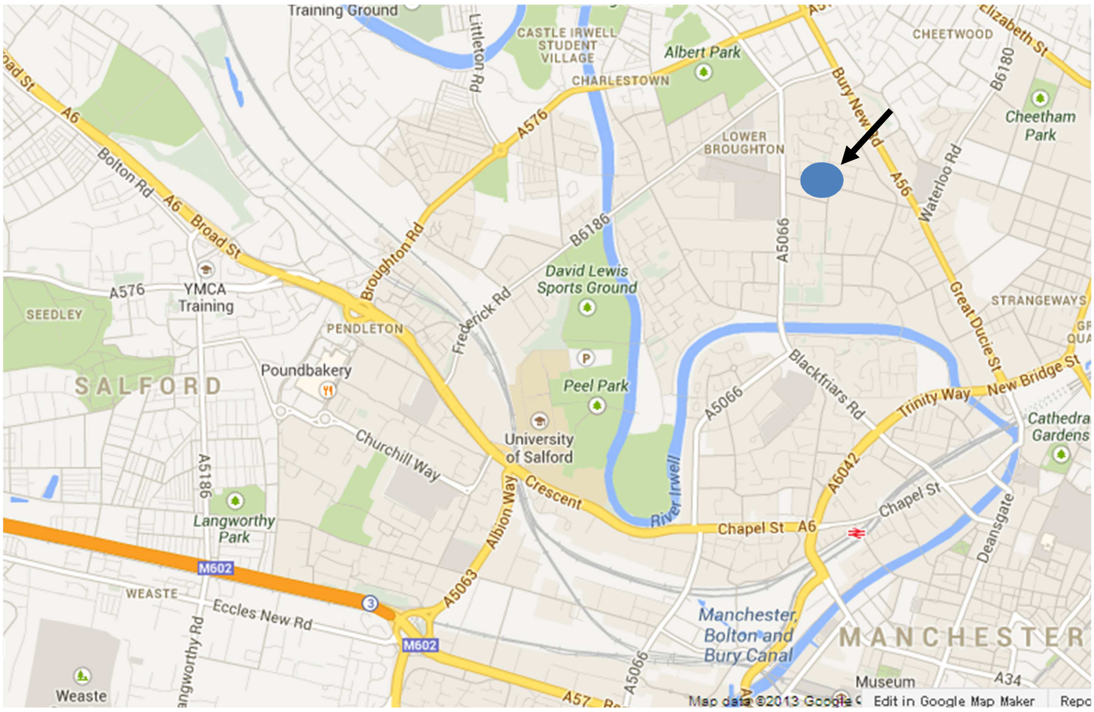


Figure A.1: Location map of Site 11

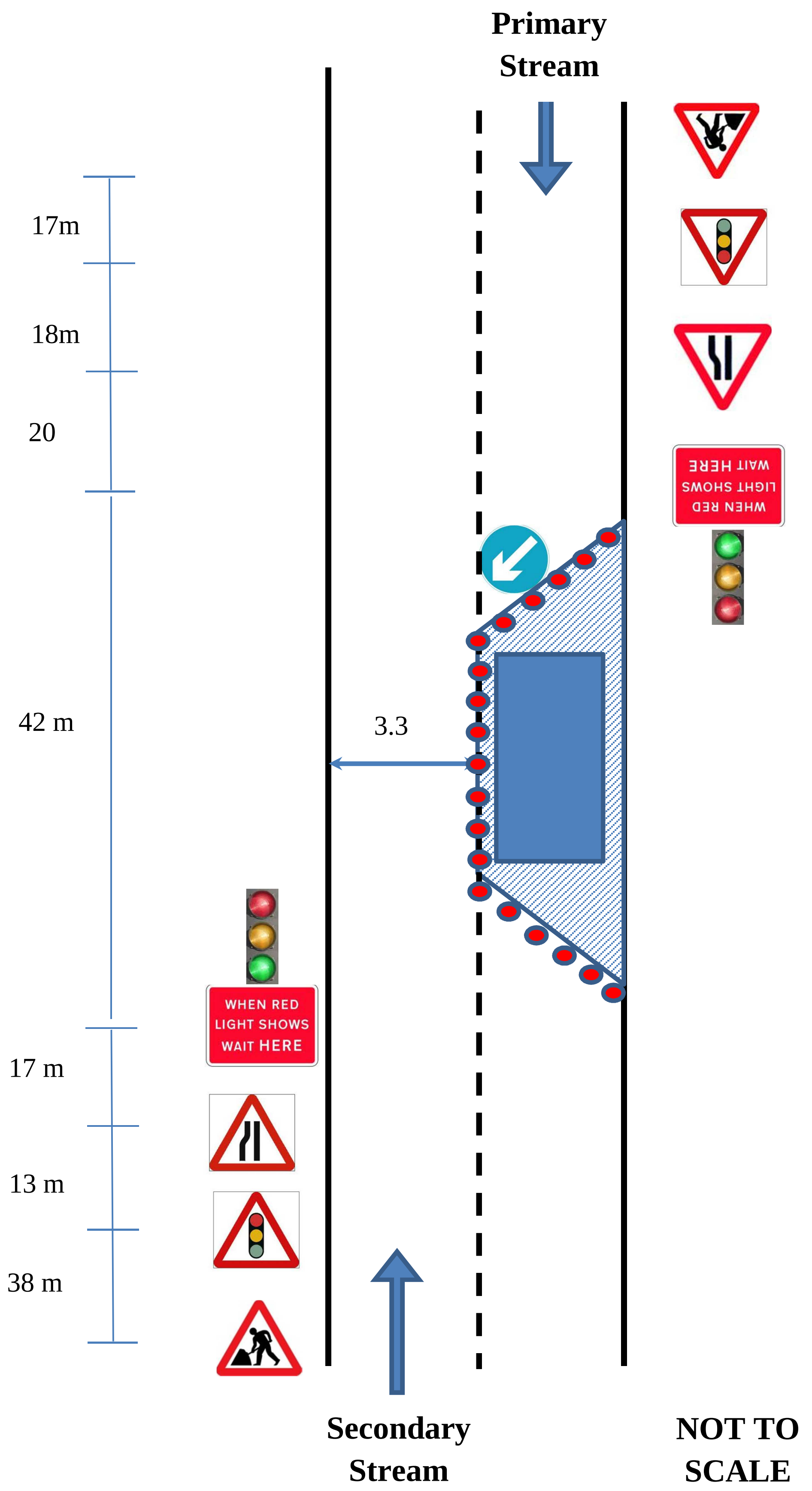


Figure A.2: Site 11 layout

A.2. Site 12

Site Number: 12

Location: Burton Road, Chorlton (between Everett Road/Ridsdale Avenue and Darlington Road)

Roadwork operation method: Temporary traffic signal

Site Length: 107 metres

Surveyed: Primary and secondary streams

Date: Tuesday 10.07.2012

Time: 09:45 to 11:45

Duration: 2 hours

Weather: Rainy

Comments: the site was filmed on once. The first camera was placed in parked vehicle and the second was in a barber shop. Was not able to film again as the site was movable. It can be seen from Figure A.4 that only the temporary traffic signal sign was present on site in the direction of primary stream and also illustrated in Figure A.5 which is not according to the standards.

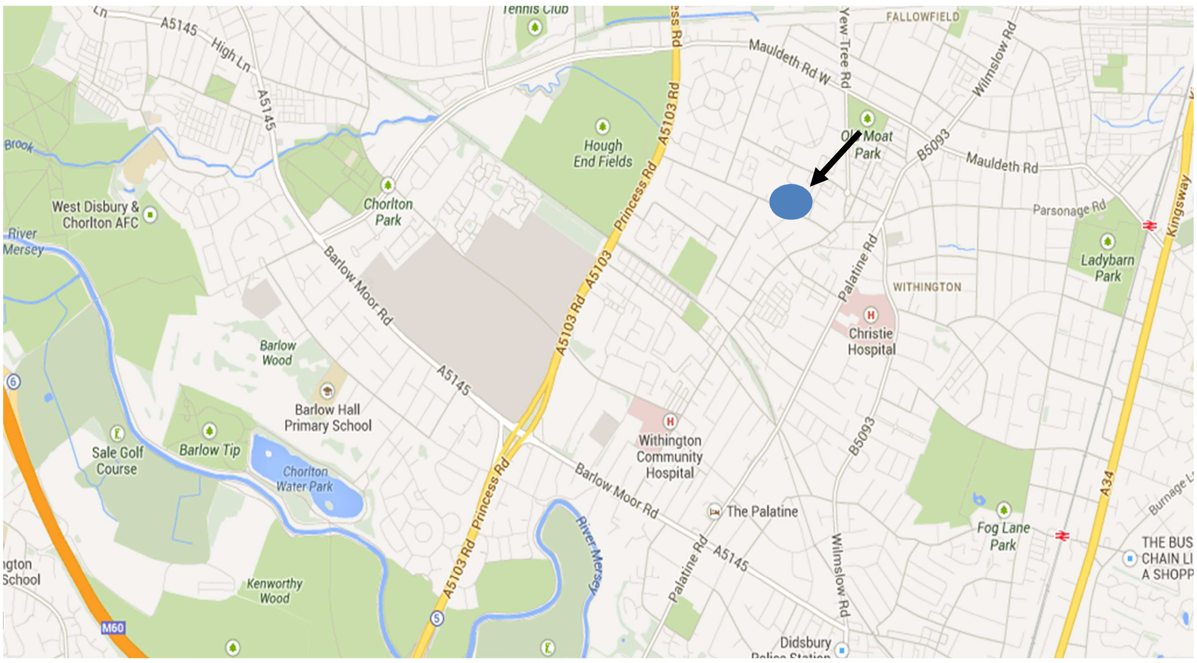


Figure A.3: Location map of Site 12

Figure A.4: Pictures showing Site 12

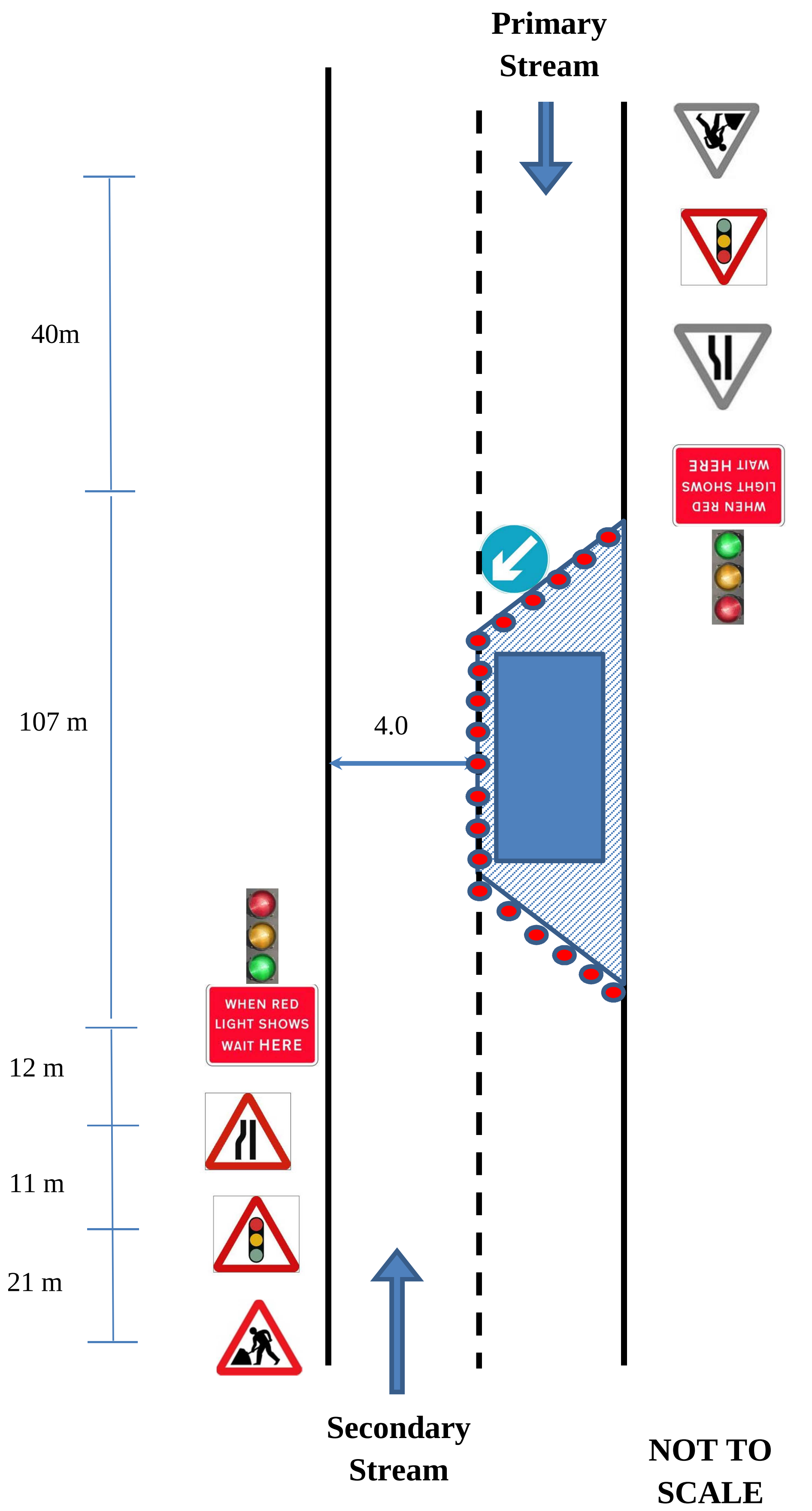


Figure A.5: Site 12 layout

A.3. Site 13

Site Number: 13

Location: Brunswick Street, Manchester (between Bramwell Drive and Beamish close/Wadeson Road)

Roadwork operation method: Priority Signs

Site Length: 8 metres

Surveyed: Primary and secondary streams

Date: Thursday 12 July 2012

Time: 09:45 to 11:45

Duration: 2 hours

Weather: cloudy and sunny

The area was rough and mainly council estate. Filmed the primary stream and went back to film the secondary stream but the work was completed. It can be seen from Figures A.7 to A.9 the bad practise signage (i.e. knocked down signs, missing signs, etc.) at the roadworks site which is not according to the standards.

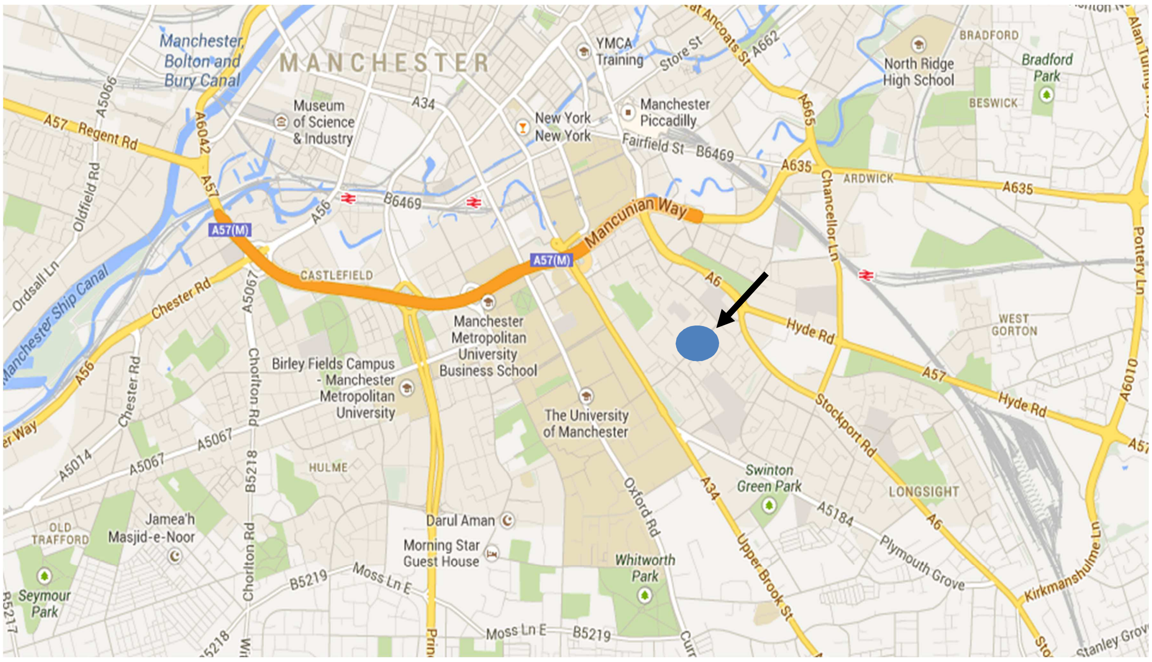


Figure A.6: Location map of Site 13

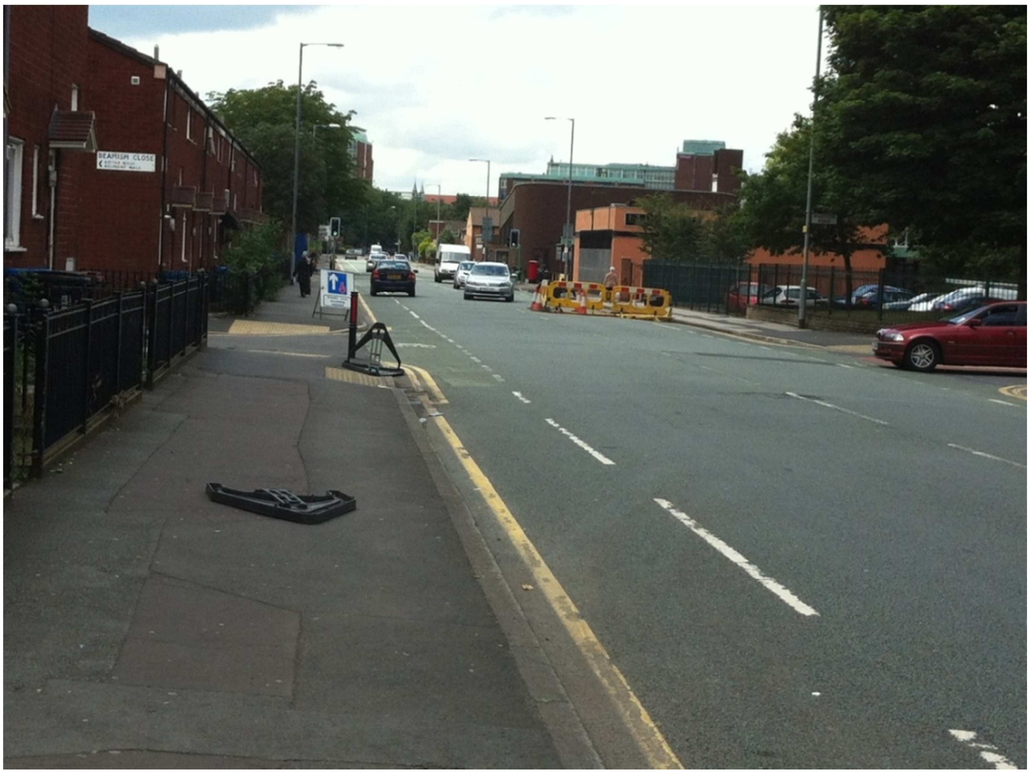


Figure A.7: Pictures showing Site 13 (secondary stream)

Figure A.8: Pictures showing Site 13 (primary stream)

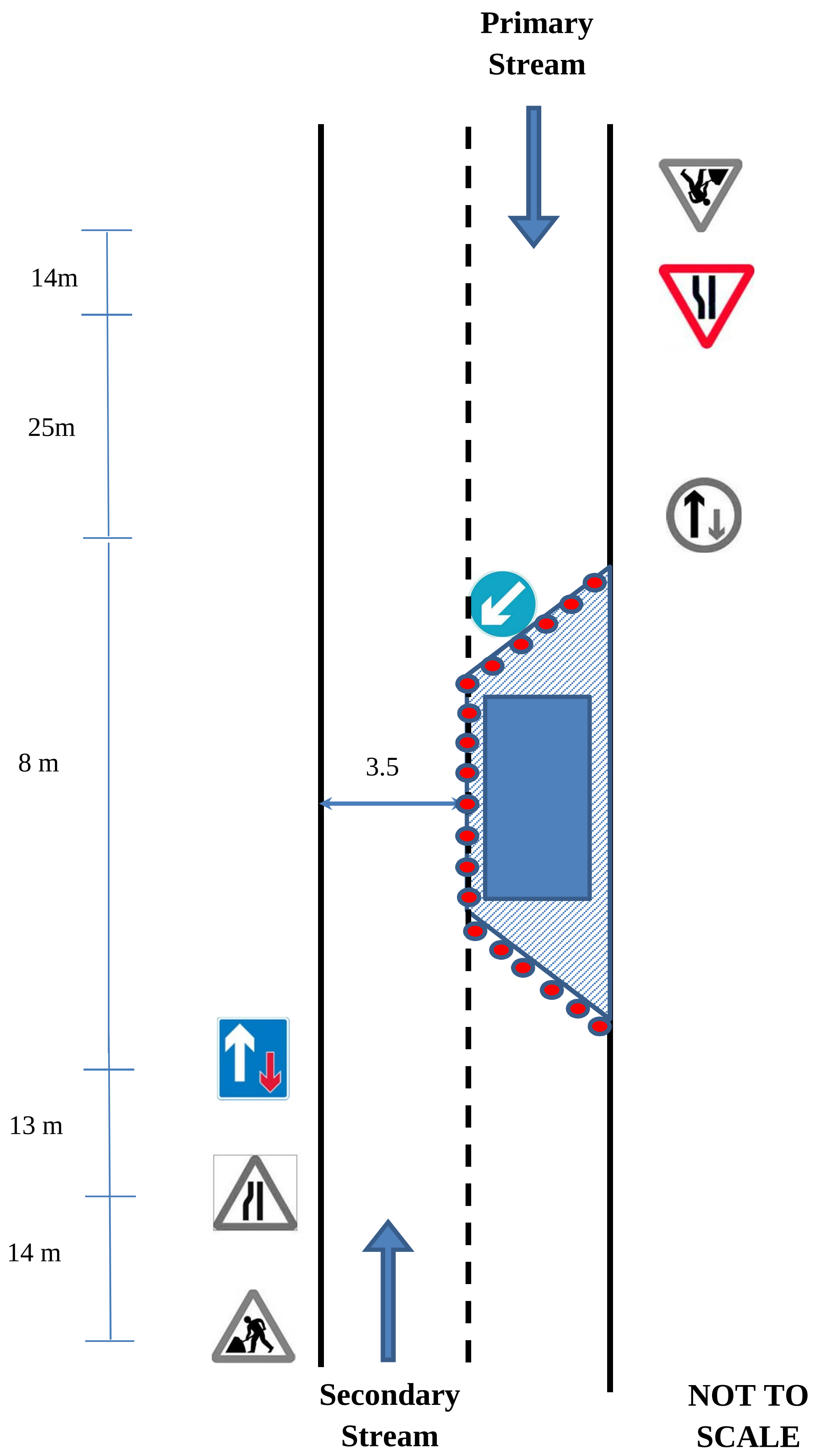


Figure A.9: Site 13 layout

A.4. Site 14

Site Number: 14

Location: Liverpool Street, Salford (between Fitzwarren Street/Athole Street and Bradden close)

Roadwork operation method: Give or Take

Site Length: 17 metres

Surveyed: Secondary stream (no space to film the primary stream)

Date: Tuesday 17 July 2012

Time: 09:15 to 10:45 and 15:30 to 17:30

Duration: 3.5 hours

Weather: cloudy and rainy

Comments: There was no safe position to film the primary stream and the weather was rainy. The area was not safe to film again due to the local circumstances (e.g. youth passing by, vandalism). It can be seen from Figures A.11 and A.12 that there are no signs in the secondary stream direction which is not according to the standards.

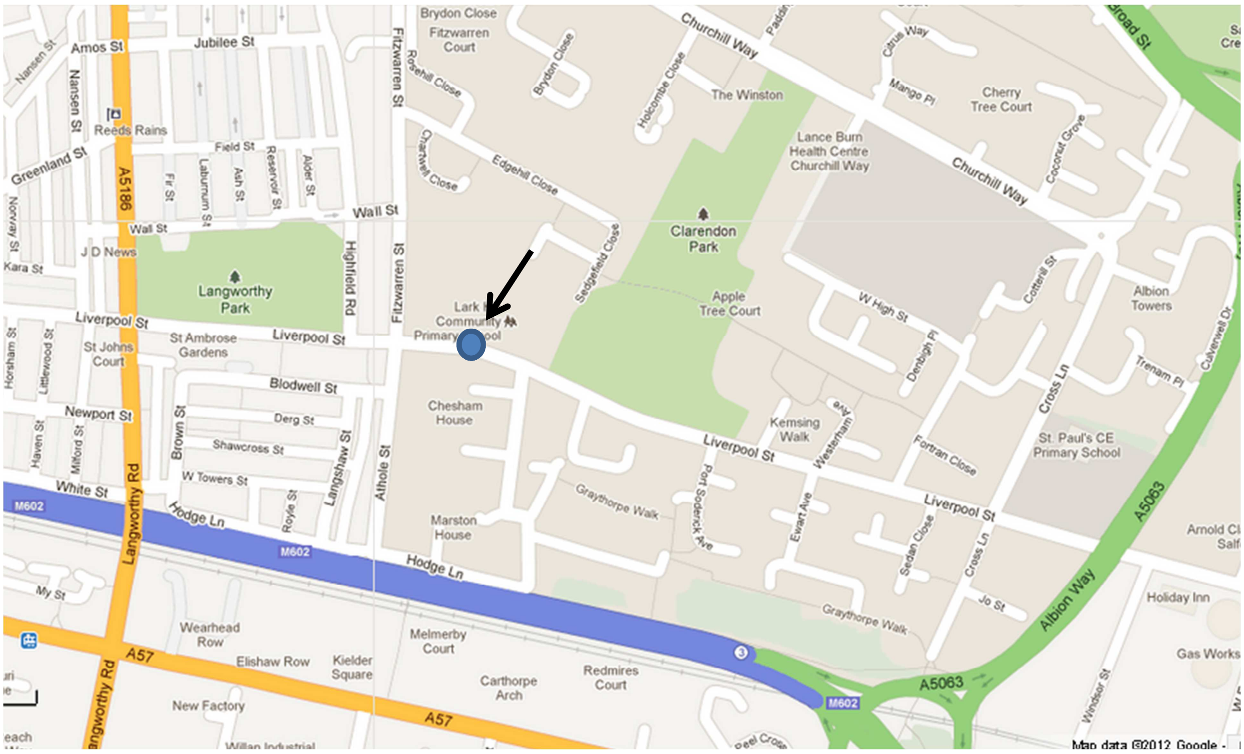


Figure A.10: Location map of Site 14

Figure A.11: Pictures showing Site 14

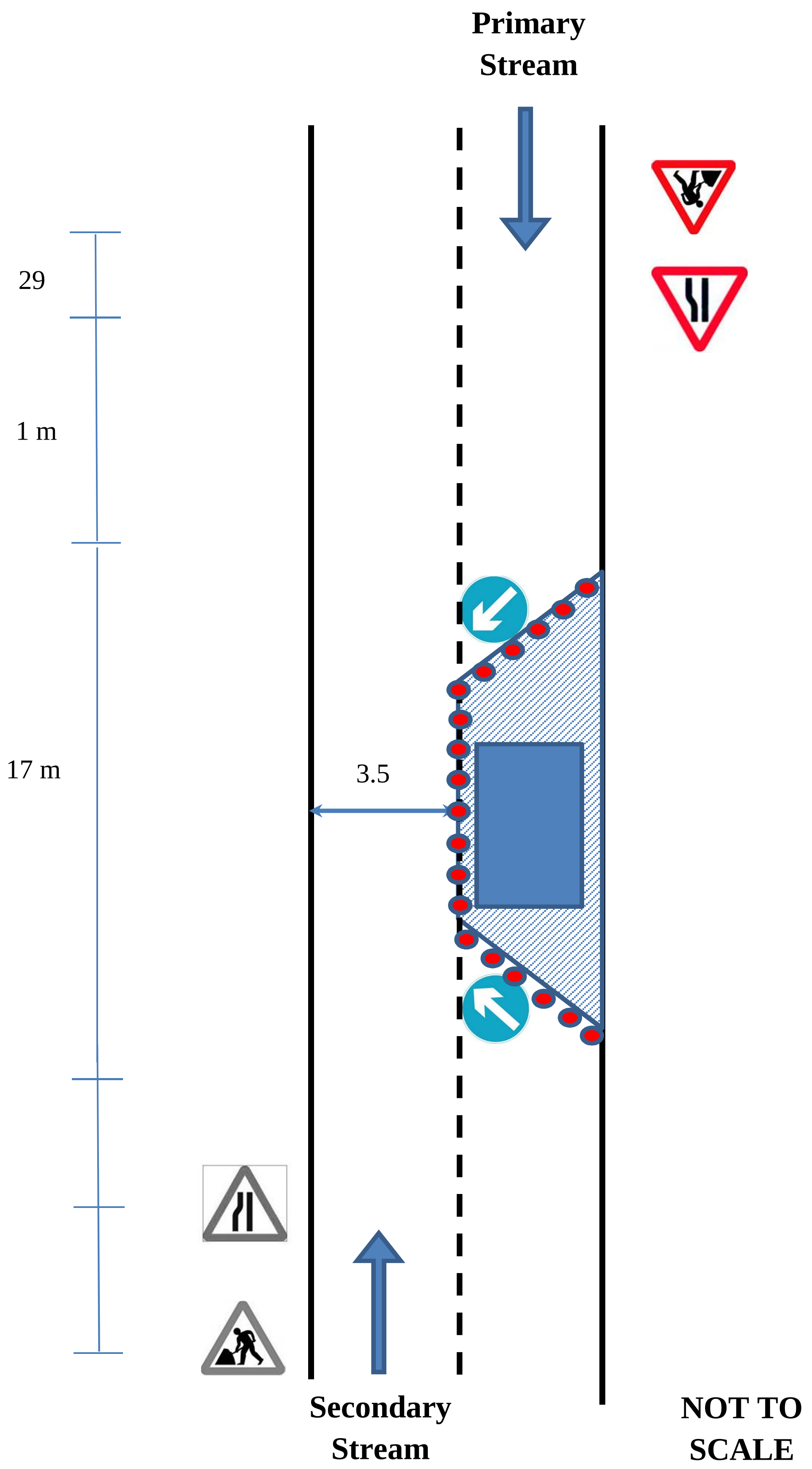


Figure A.12: Site 14 layout

A.5. Site 15

Site Number: 15

Location: Langworthy Road, Salford (junction Langworthy Road/Seedley Road/Sandy Lane)

Signalised junction

Date: Wednesday 18.07.2012

Time: 09:25 to 11:25

Duration: 2 hours

Weather: cloudy

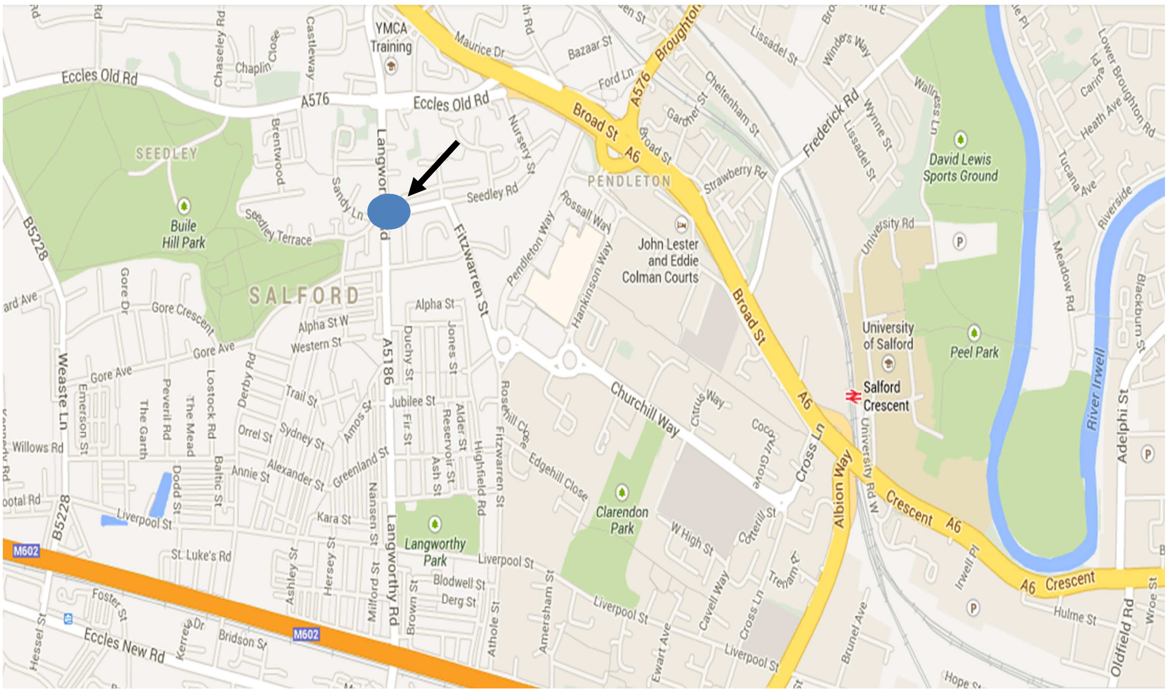


Figure A.13: Location map of Site 15

A.6. Site 16

Site Number: 16

Location: High Lane, Manchester (between Acres Road and Chequers Road)

Roadwork operation method: Temporary traffic signal

Site Length: 42 metres

Surveyed: Primary and secondary streams

Date: Monday 04 September 2012

Time: 11:30 to 13:30

Duration: 3.5 hours

Weather: Sunny

Comments: the site was filmed for both directions. The weather was sunny.

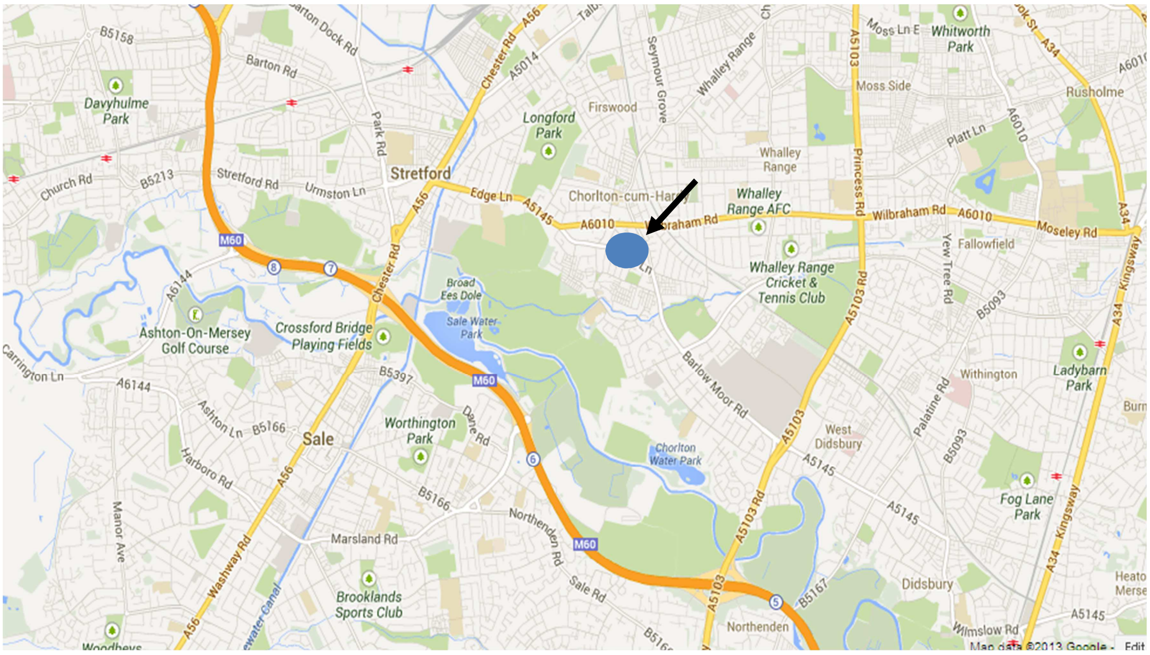


Figure A.14: Location map of Site 16

Figure A.15: Pictures showing Site 16

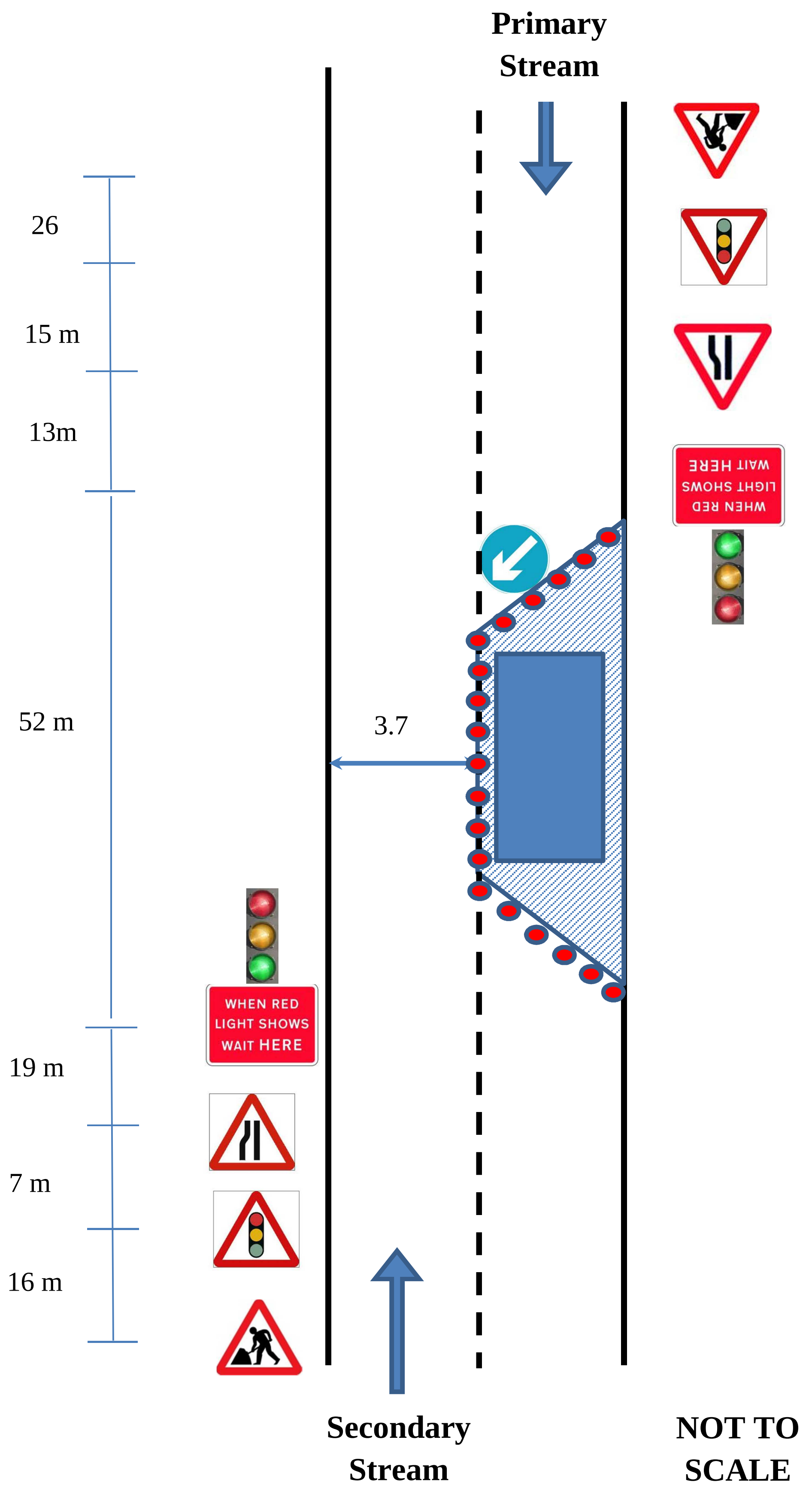


Figure A.16: Site 16 layout

A.7. Site 17

Site Number: 17

Location: New Blackley Road, Manchester (between Ellbourne Rd and Roch Bank)

Roadwork operation method: Temporary traffic signal

Site Length: 39 metres

Surveyed: Primary and secondary streams

Date: Thursday 06 September 2012

Time: 11:00 to 12:00

Duration: 1 hours

Weather: Sunny

Comments: the site was filmed for both directions. The weather was sunny. It can be seen from Figure A.18 the on-street parking in close proximity to the shuttle-lane roadworks site which makes the signs unclear to the drivers (in the secondary stream direction) causing traffic to stop in advance of the stop line.

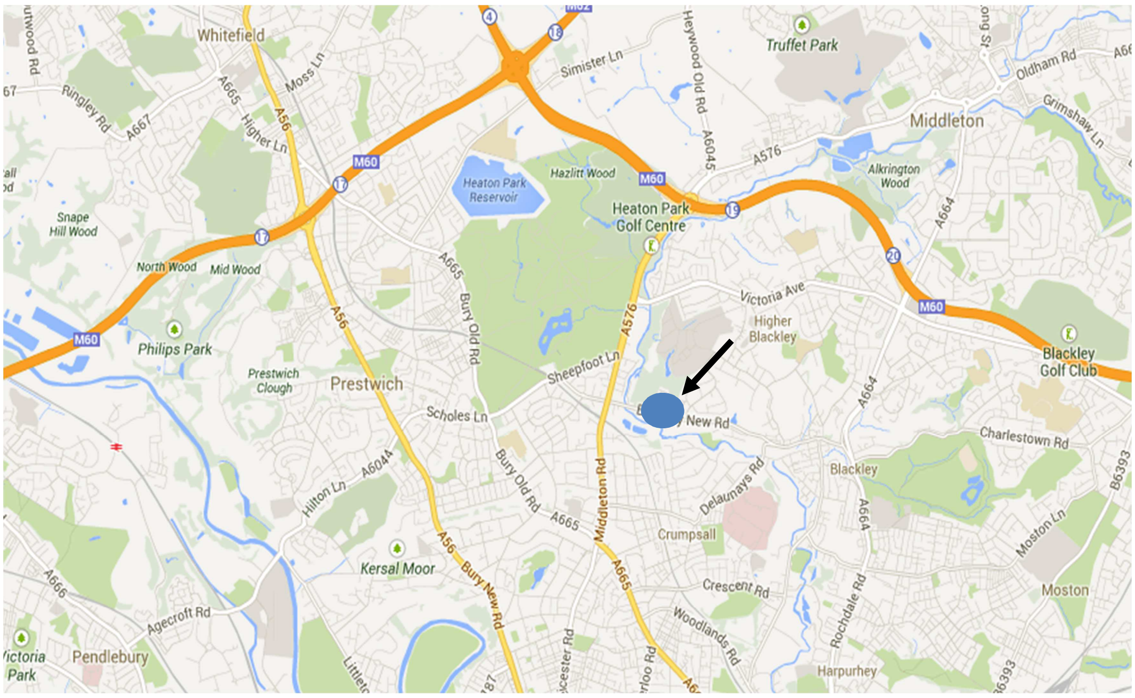


Figure A.17: Location map of Site 17

Figure A.18: Pictures showing Site 17

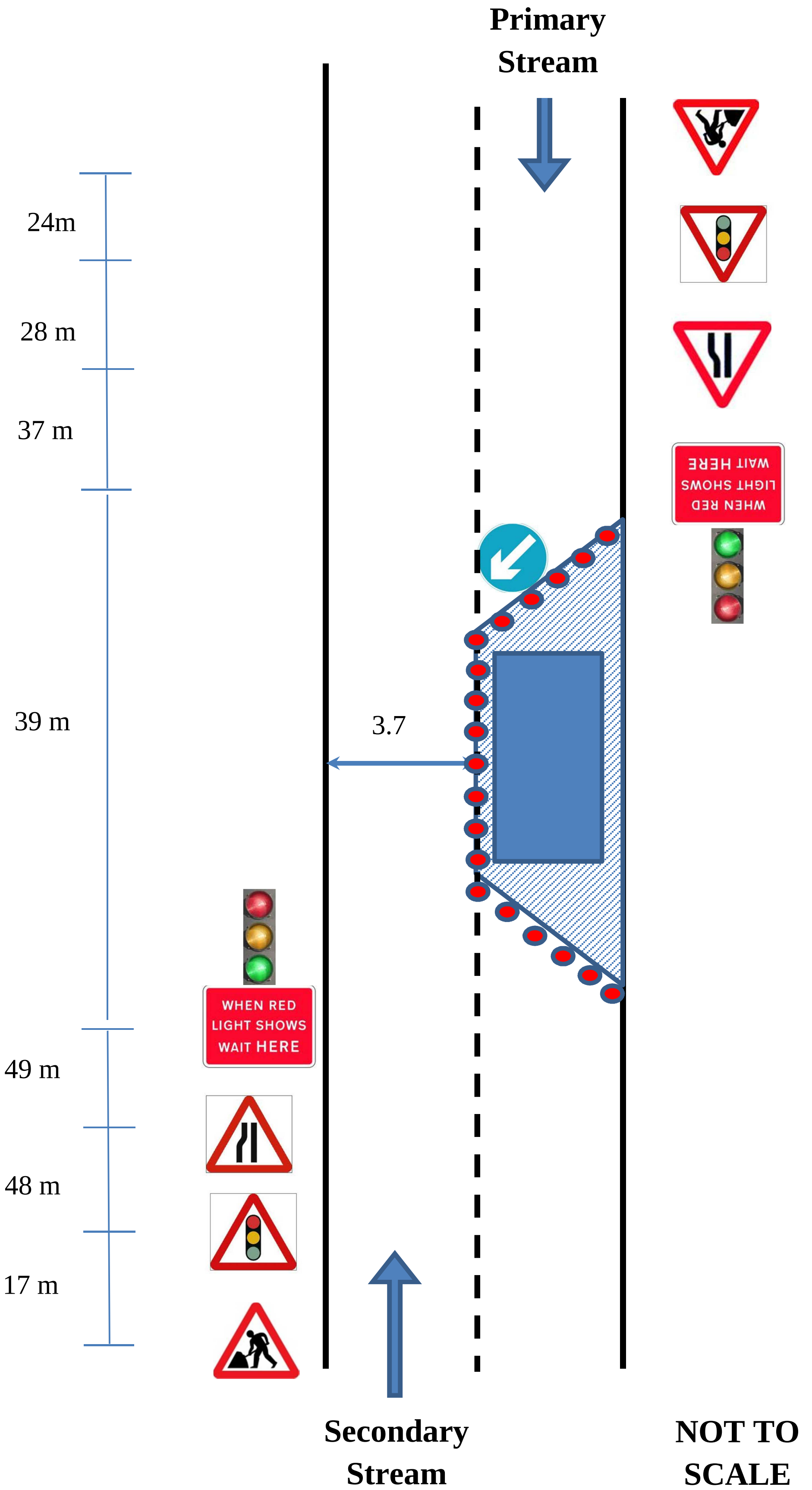


Figure A.19: Site 17 layout

A.8. Site 18

Site Number: 18

Location: New Blackley Road, Manchester (between Ellbourne Rd and Roch Bank)

Roadwork operation method: Temporary traffic signal

Site Length: 73 metres

Surveyed: Primary and secondary streams

Date: Monday 17 September 2012

Time: 16:30 to 18:00

Duration: 1.5 hours

Weather: Sunny

Comments: the site was filmed for both directions. The weather was sunny. It can be seen from Figure A.21 the on-street parking in close proximity to the shuttle-lane roadworks site which makes the signs unclear to the drivers (in the primary stream direction) causing traffic to stop in advance of the stop line.

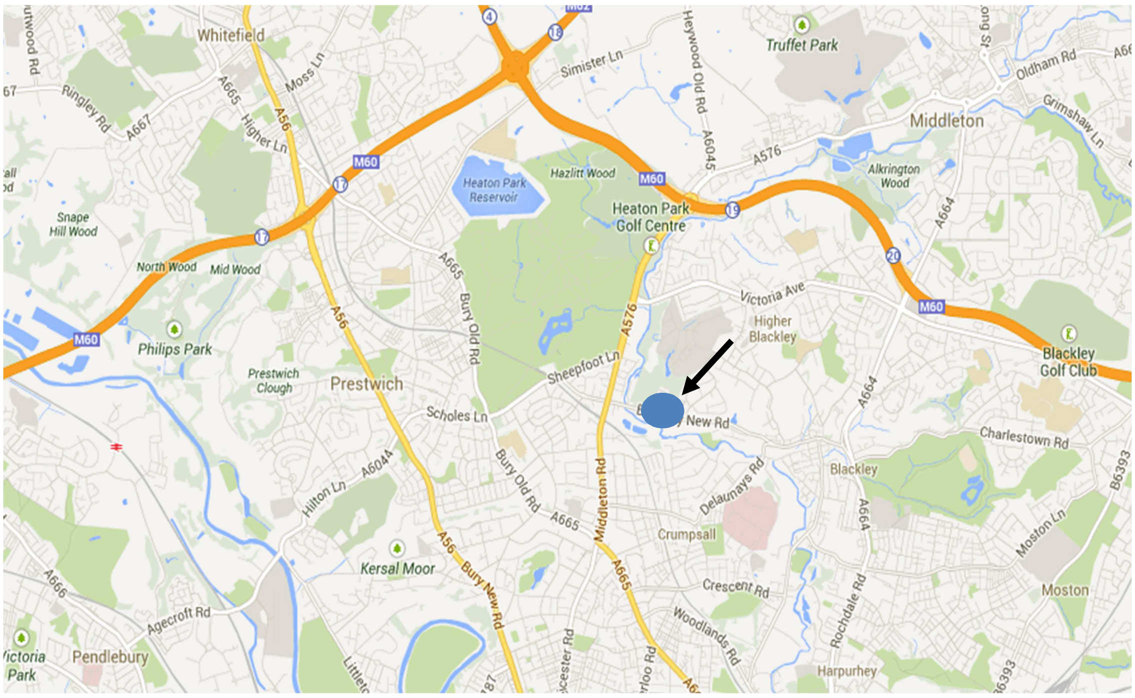


Figure A.20: Location map of Site 18

Figure A.21: Pictures showing Site 18

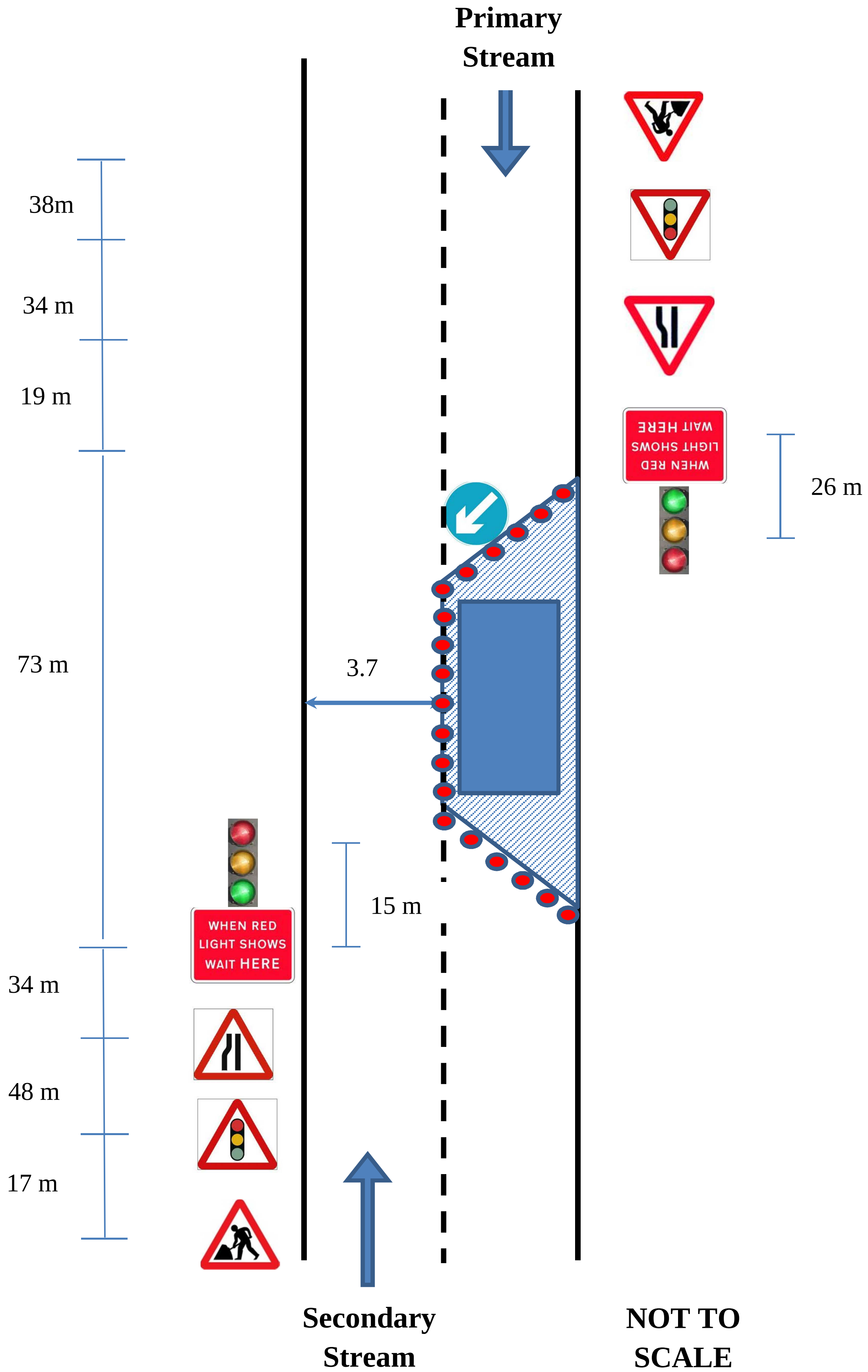


Figure A.22: Site 18 layout

A.9. Site 19

Site Number: 19

Location: Frederick Road, Manchester (between Cheltenham Street and Lissadel Street)

Roadwork operation method: Temporary traffic signal

Site Length: 38 metres

Surveyed: Primary and secondary streams

Date: Thursday 27 September 2012 and Tuesday 02 October 2012

Time: 09:15 to 11:15

Duration: 3.5 hours

Weather: Sunny

Comments: the site was filmed for both directions. The weather was sunny.

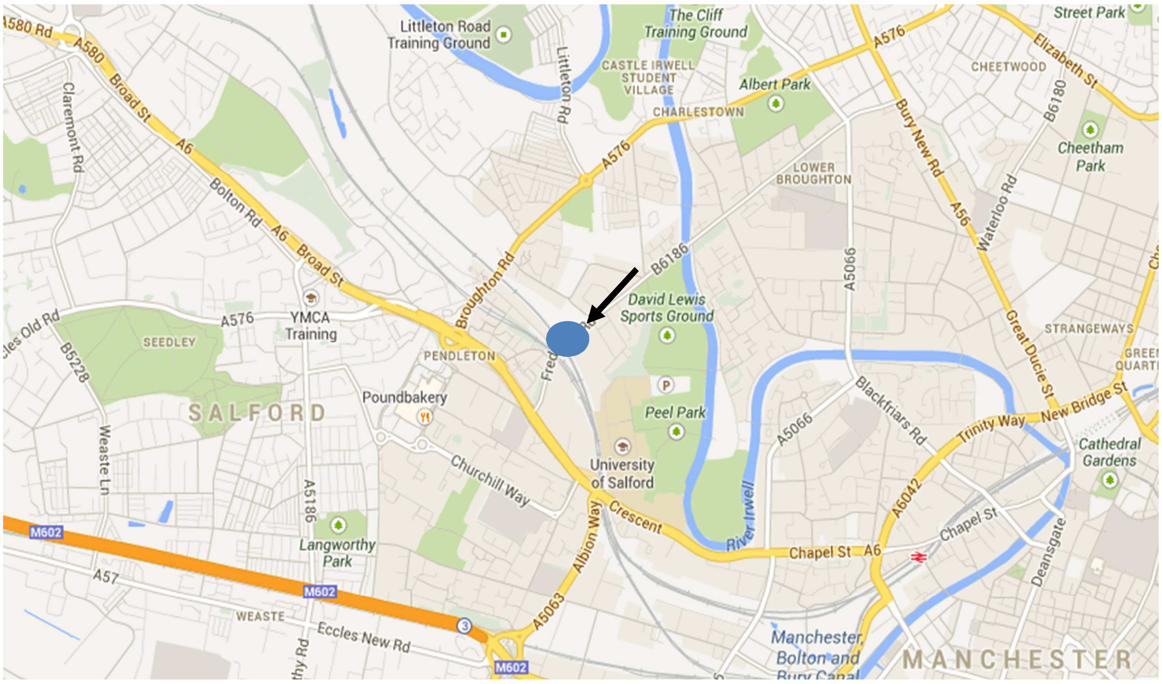


Figure A.23: Location map of Site 19

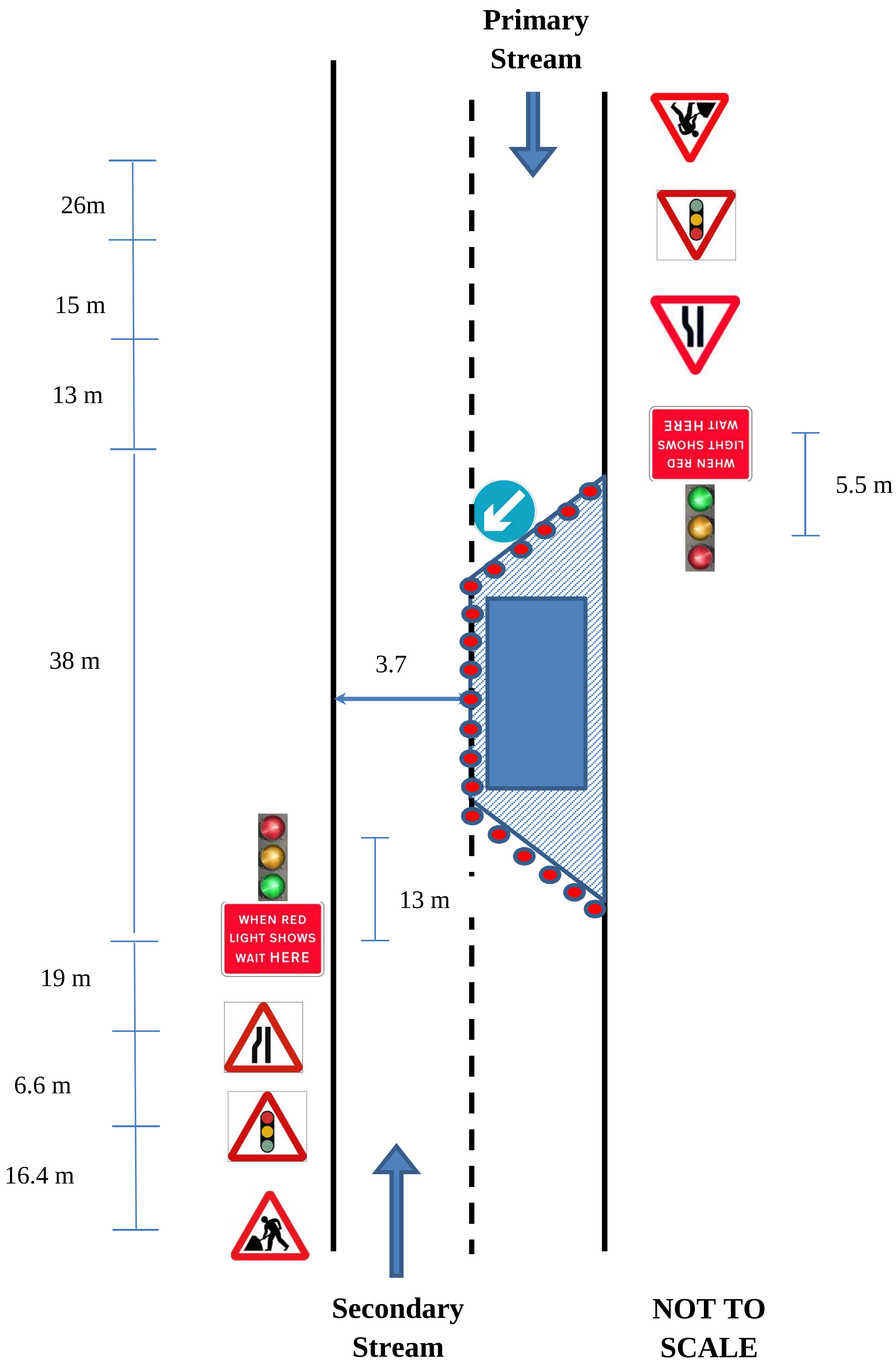


Figure A.24: Site 19 layout

A.10. Site 20

Site Number: 20

Date: 12.12.2011

Location: Manchester Road, Swinton (between East Drive and Hospital Road)

Roadwork operation method: Priority Rules

Site Length: 67 metres

Comments: The signage at site was very poorly designed and causing confusion to drivers. The signs were also covered behind parked vehicles as shown in Figures A.26 and A.27.

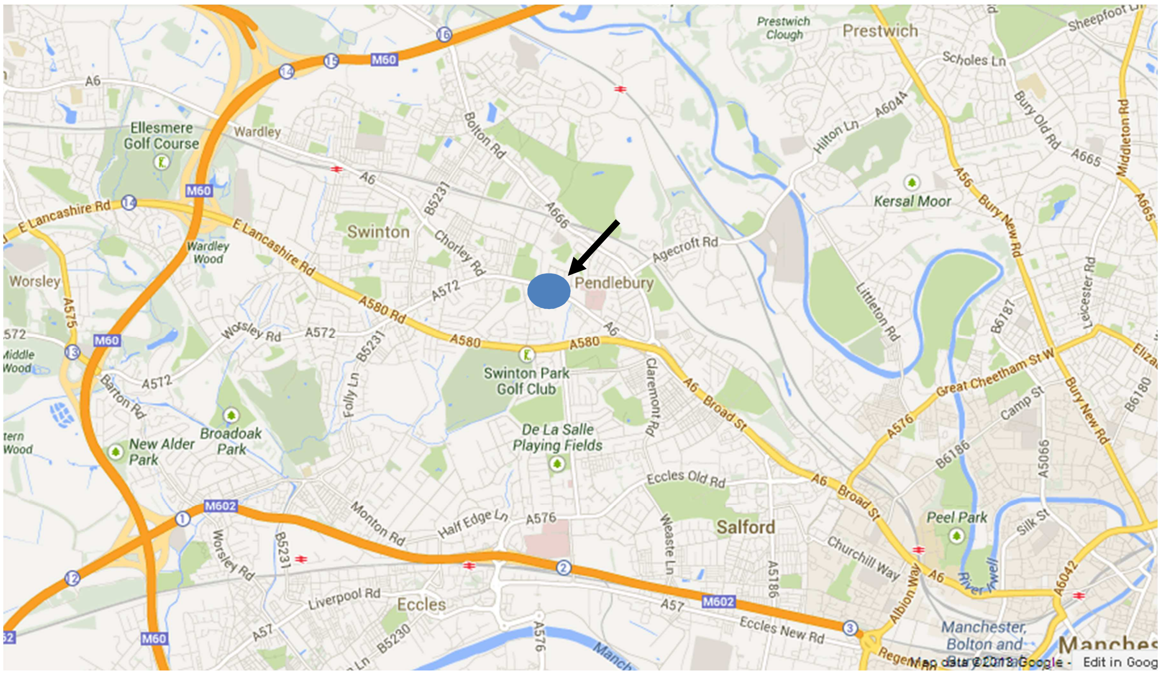


Figure A.25: Location map of Site 20

Figure A.26: Pictures showing Site 20 (primary stream)

Figure A.27: Pictures showing Site 20 (secondary stream)

A.11. Site 21

Site Number: 21

Date: 17.07.2012

Location: Silk Street, Salford (between North George Street and Cannon Street)

Roadwork operation method: Give or Take

Site Length: 19 metres

Comments: The visibility was very poor because of the bend and the site could not be filmed as there is no safe location to park a vehicle or stand as shown in Figures A.29 and A.30. Roadworks duration was for 2 days.

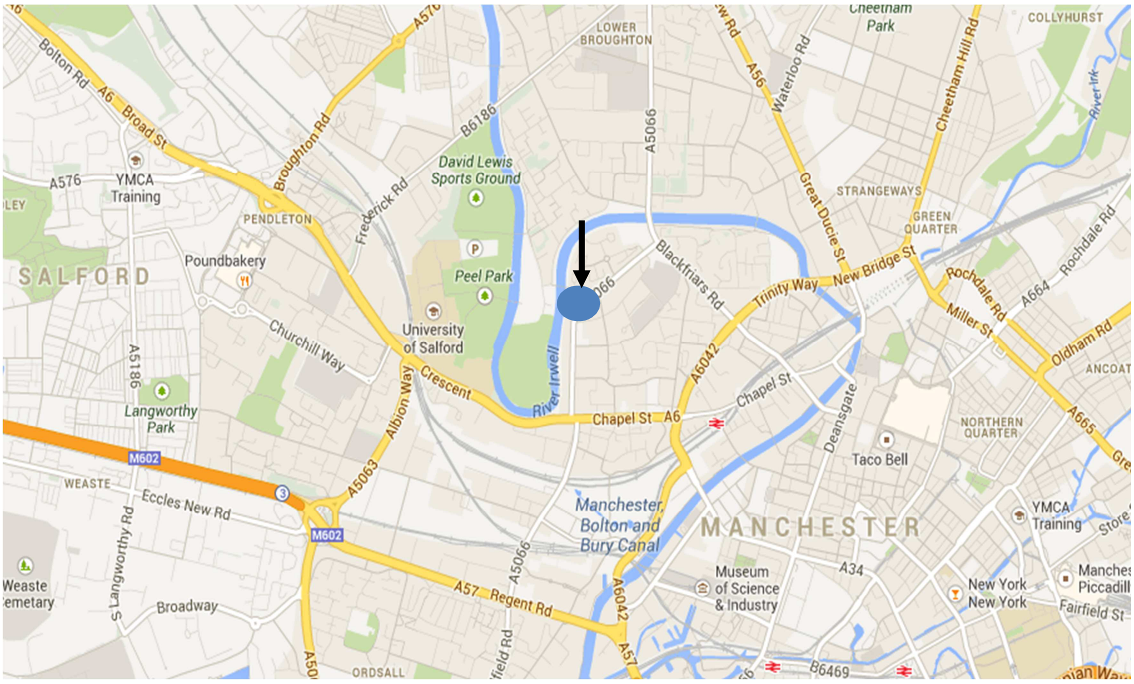


Figure A.28: Location map of Site 21

Figure A.29: Pictures showing Site 21 (secondary stream)

Figure A.30: Pictures showing Site 21 (primary stream)

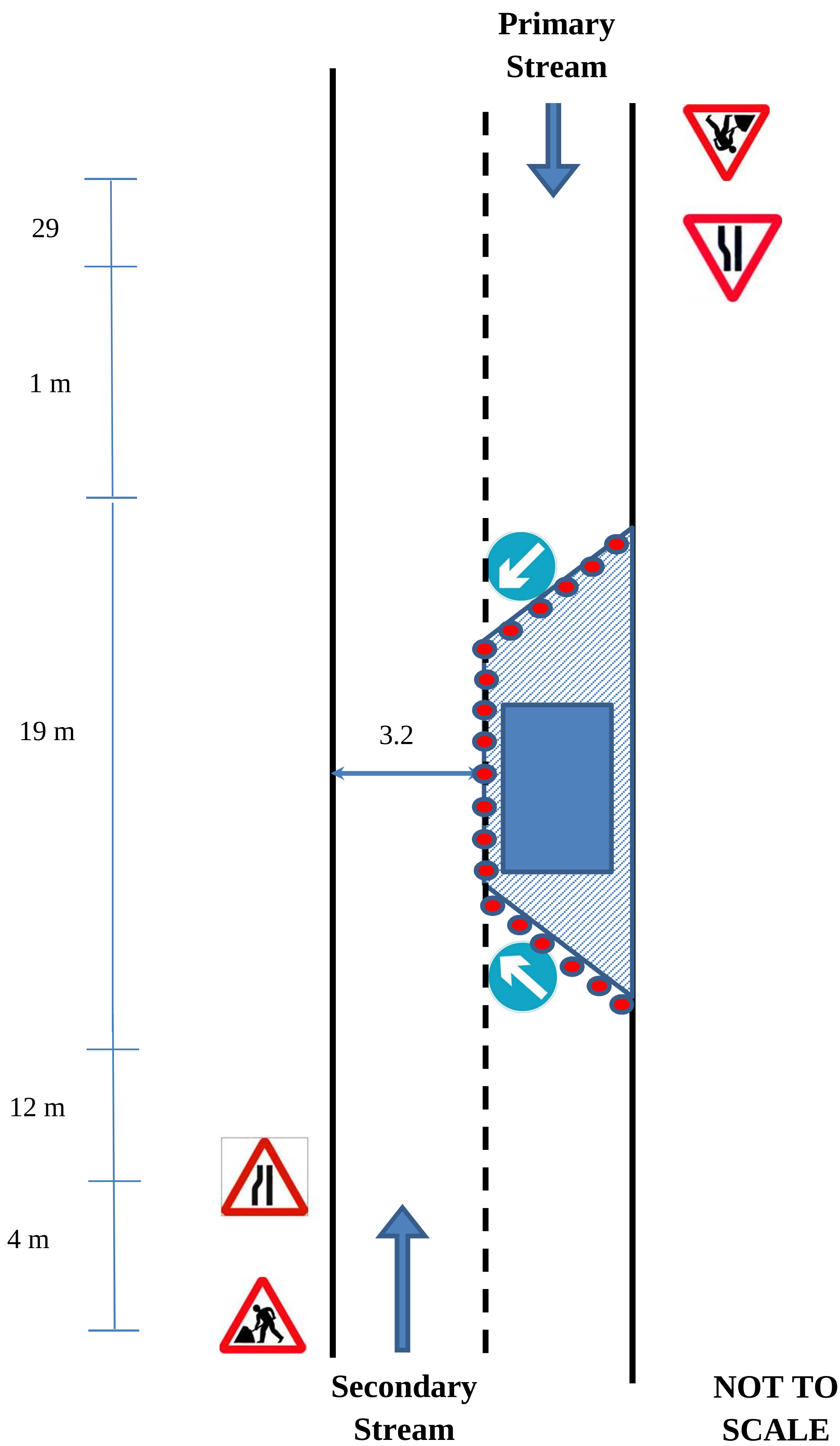


Figure A.31: Site 21 layout

A.12. Site 22

Site Number: 22

Date: 21.12.2012

Location: University Road, Salford

Roadwork operation method: FT temporary traffic signal

Site Length: 79 metres

Comments: Traffic flow was too low. No access provided to pedestrians which is not according to the design standards as shown in Figure A.33 and A.34.

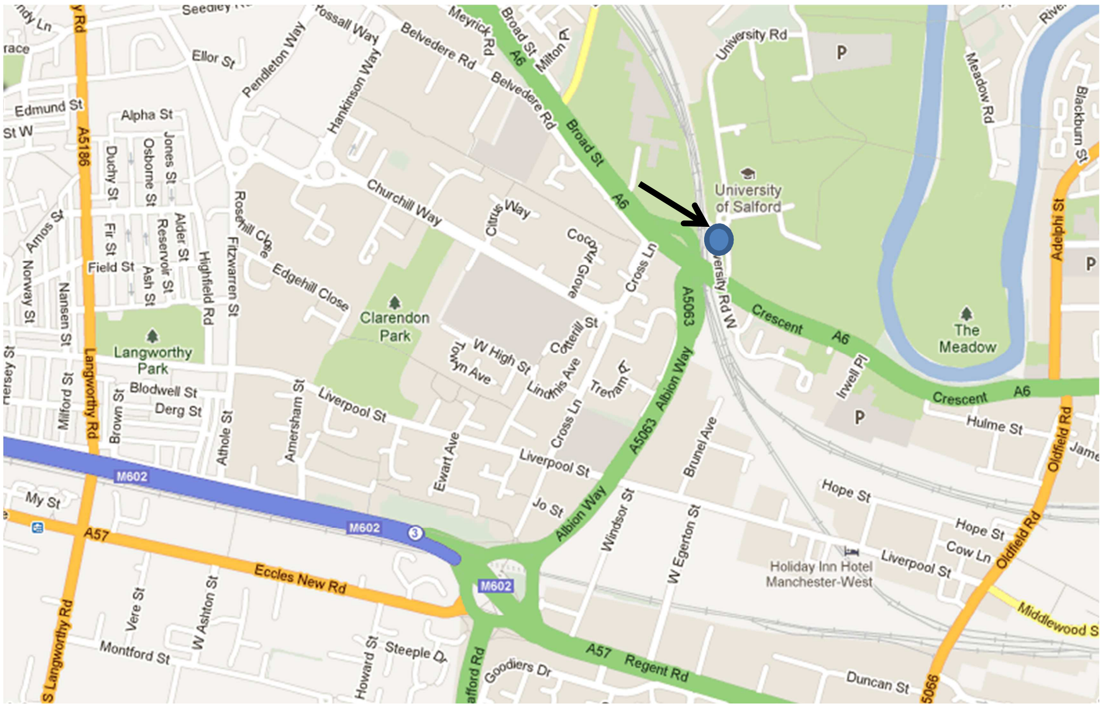


Figure A.32: Location map of Site 22

Figure A.33: Pictures showing Site 22 (secondary stream)

Figure A.34: Pictures showing Site 22 (primary stream)

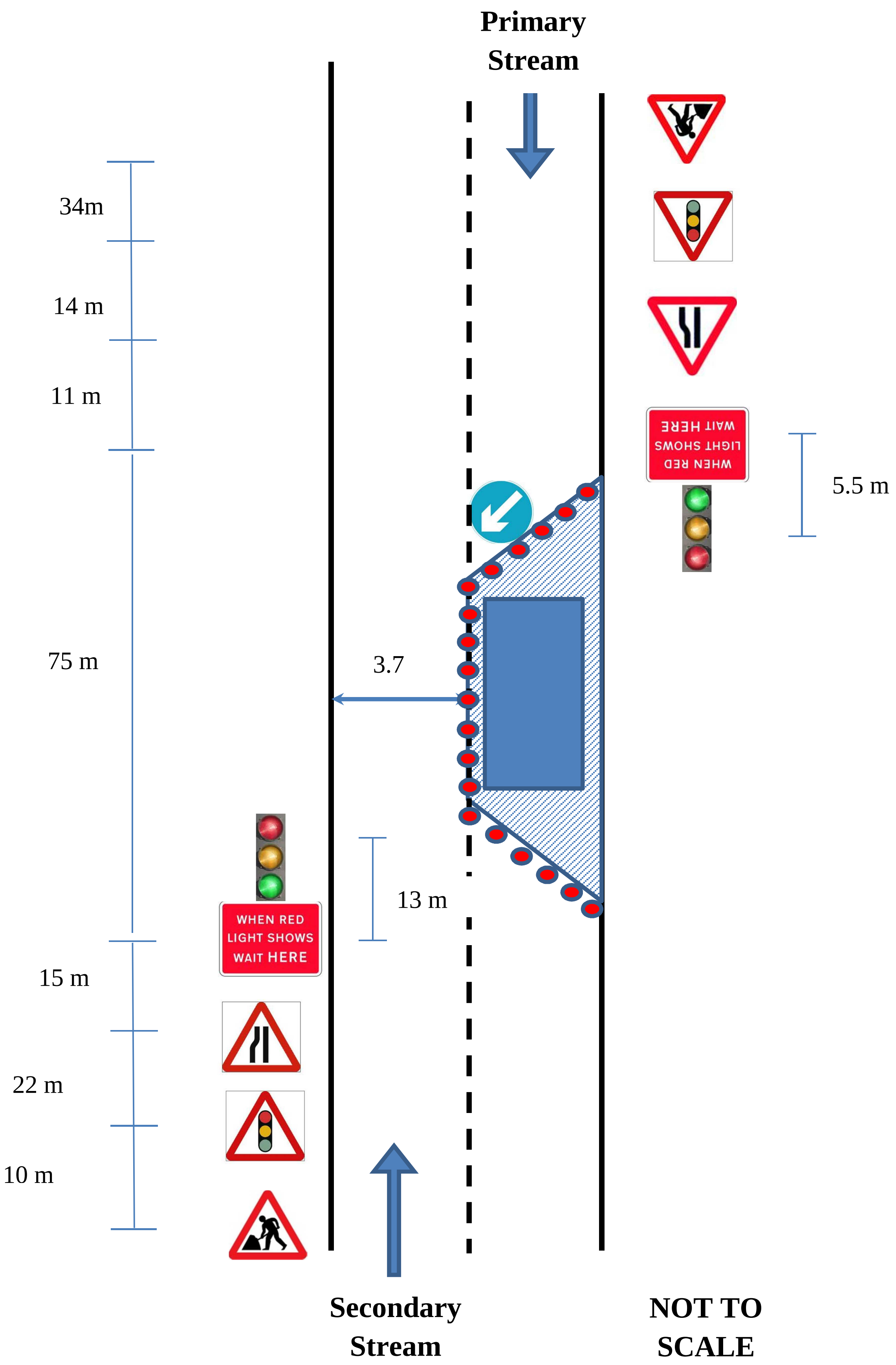


Figure A.35: Site 22 layout

A.13. Site 23

Site Number: 23

Date: 21.12.2012

Location: University Road West, Salford

Roadwork operation method: FT temporary traffic signal

Site Length: 68 metres

Comments: Traffic flow was too low. Roadworks took place under a bridge.

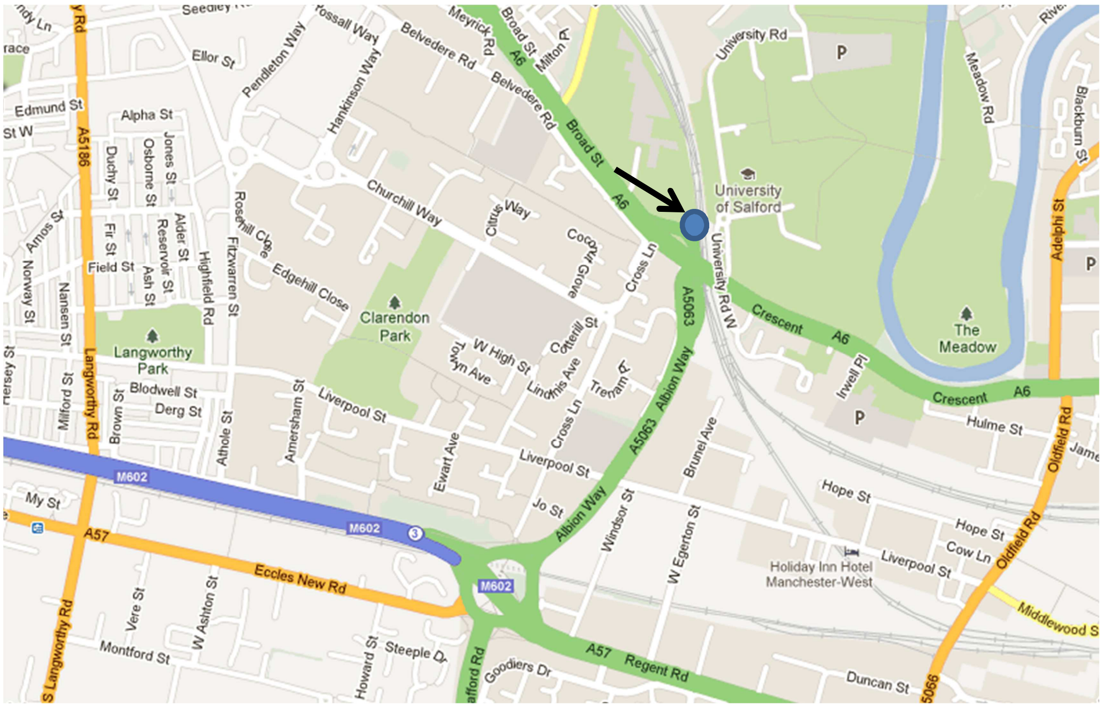


Figure A.36: Location map of Site 23

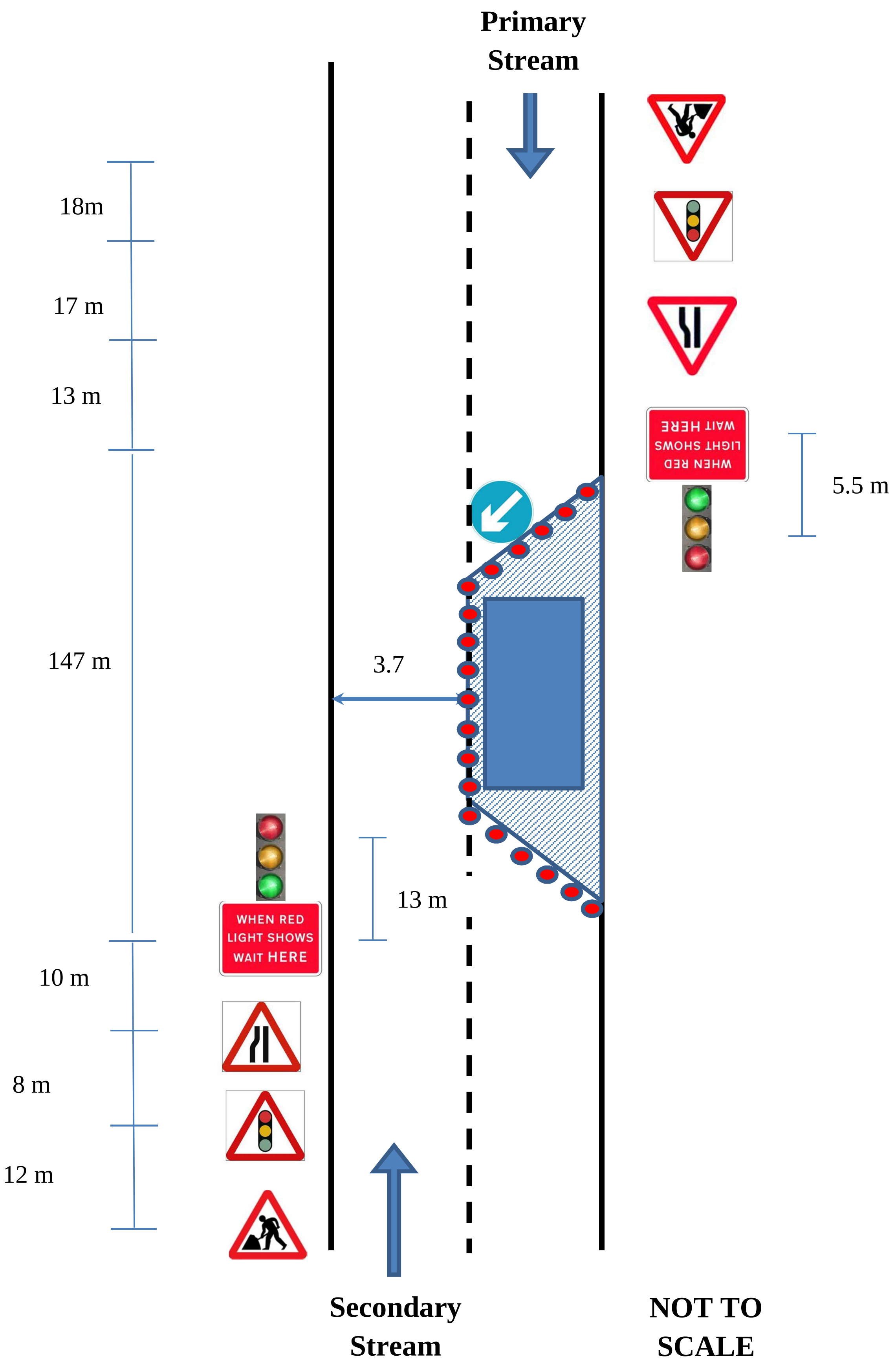


Figure A.37: Site 23 layout

A.14. Temporary traffic signal control equipment

Figure A.38 shown the inside of the temporary traffic signal head. The buttons can be changed easily to set the site leng.th and the controller will change the all-red period and maximum green time accordingly. The controller is set to a TA 47/85 which is out of date (superseded) and the new settings should be used. Figure A.39 shows the full signal head and the battery.

Figure A.38: Photo of the signal control box for temporary traffic signal

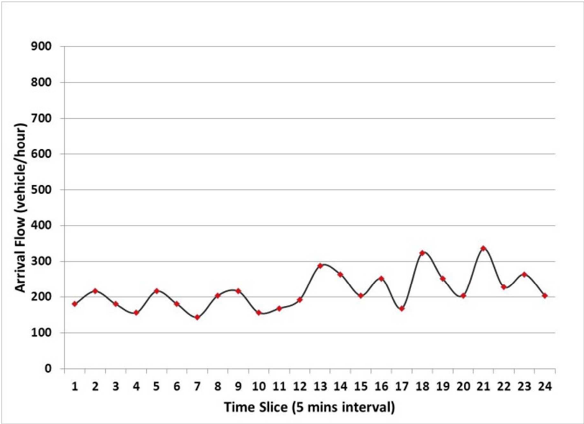
Figure A.39: Photo of the signal control equipment at temporary traffic signal

APPENDIX B: DATA ANALYSIS

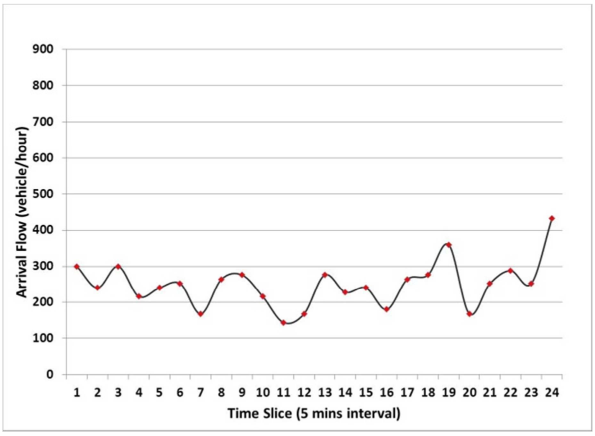
B.1 Flow levels and profile

Table B.1: Flow statistics for each site

Site No.	Hour	Minimum Flow (veh/hr)	Maximum Flow (veh/hr)	Average Flow (veh/hr)	Directional Split (%)	HGVs Percentage (%)
Site 11a	1	(P) 144 (S) 144	(P) 216 (S) 300	(P) 184 (S) 232	(P) 44 (S) 56	(P) 6 (S) 4
	2	(P) 168 (S) 168	(P) 336 (S) 432	(P) 249 (S) 268	(P) 48 (S) 52	(P) 4 (S) 3
Site 11b	1	(P) 168 (S) 192	(P) 504 (S) 360	(P) 322 (S) 279	(P) 53 (S) 47	(P) 2 (S) 5
	2	(P) 264 (S) 204	(P) 468 (S) 408	(P) 373 (S) 313	(P) 54 (S) 46	(P) 1 (S) 3
Site 12	1	(P) 72 (S) 84	(P) 216 (S) 192	(P) 142 (S) 134	(P) 51 (S) 49	(P) 10 (S) 10
	2	(P) 108 (S) 96	(P) 252 (S) 264	(P) 174 (S) 159	(P) 52 (S) 48	(P) 7 (S) 9
Site 13	1	(P) 228 (S) 168	(P) 372 (S) 444	(P) 307 (S) 273	(P) 53 (S) 47	(P) 7 (S) 5
	2	(P) 192 (S) 156	(P) 348 (S) 324	(P) 265 (S) 245	(P) 52 (S) 48	(P) 4 (S) 3
Site 14	1	(P) 96 (S) 96	(P) 444 (S) 312	(P) 258 (S) 198	(P) 57 (S) 43	(P) 2 (S) 4
	30 mins	(P) 216 (S) 120	(P) 312 (S) 324	(P) 276 (S) 178	(P) 61 (S) 39	(P) 2 (S) 1
Site 16a	1	(P) 168 (S) 240	(P) 336 (S) 372	(P) 265 (S) 283	(P) 48 (S) 52	(P) 7 (S) 2
	2	(P) 168 (S) 240	(P) 396 (S) 408	(P) 261 (S) 322	(P) 45 (S) 55	(P) 4 (S) 5
Site 16b	1	(P) 264 (S) 300	(P) 468 (S) 468	(P) 383 (S) 379	(P) 50 (S) 50	(P) 2 (S) 1
Site 17	1	(P) 156 (S) 180	(P) 264 (S) 276	(P) 204 (S) 223	(P) 48 (S) 52	(P) 5 (S) 5
Site 18	1	(P) 156 (S) 336	(P) 384 (S) 516	(P) 233 (S) 442	(P) 35 (S) 65	(P) 4 (S) 2
	30 mins	(P) 180 (S) 252	(P) 300 (S) 456	(P) 250 (S) 366	(P) 41 (S) 59	(P) 2 (S) 3
Site 19a	1	(P) 816 (S) 216	(P) 888 (S) 636	(P) 854 (S) 450	(P) 65 (S) 35	(P) 2 (S) 2
	2	(P) 816 (S) 216	(P) 888 (S) 480	(P) 832 (S) 334	(P) 71 (S) 29	(P) 1 (S) 2
Site 19b	1	(P) 564 (S) 336	(P) 816 (S) 588	(P) 683 (S) 444	(P) 61 (S) 39	(P) 2 (S) 5
	2	(P) 336 (S) 264	(P) 528 (S) 468	(P) 422 (S) 372	(P) 53 (S) 47	(P) 4 (S) 2

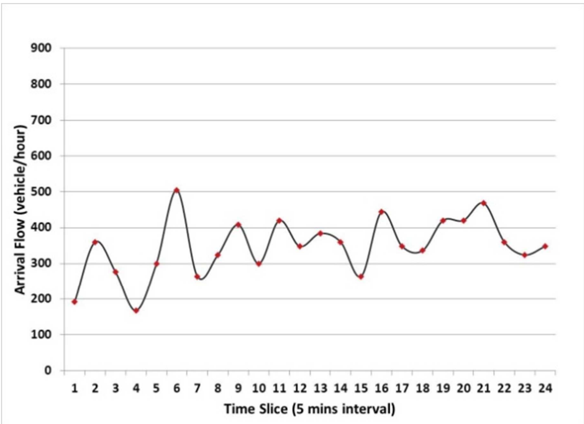


(a) Primary Stream

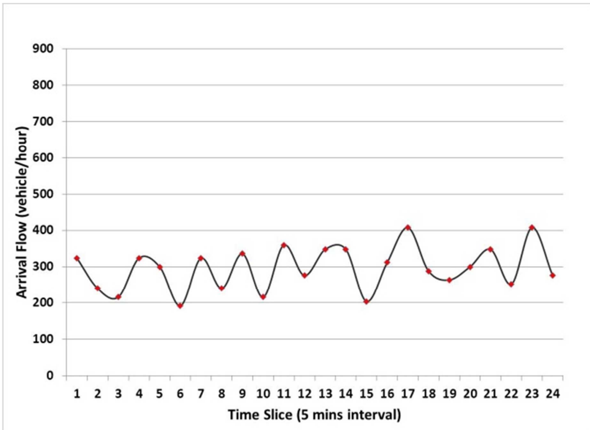


(b) Secondary Stream

Figure B.1: Vehicle arrival profile for Site 11a

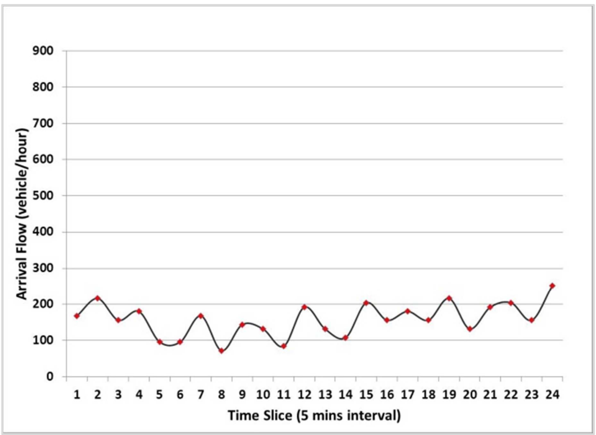


(a) Primary Stream

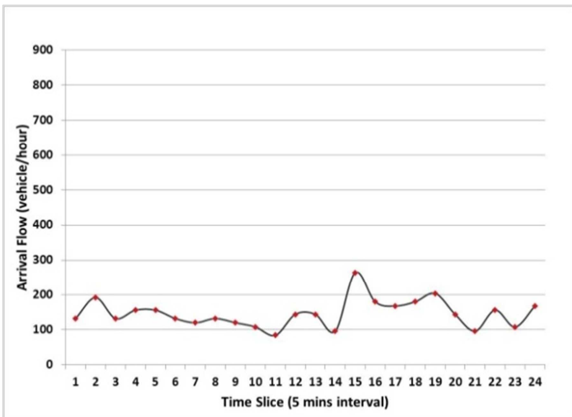


(b) Secondary Stream

Figure B.2: Vehicle arrival profile for Site 11b

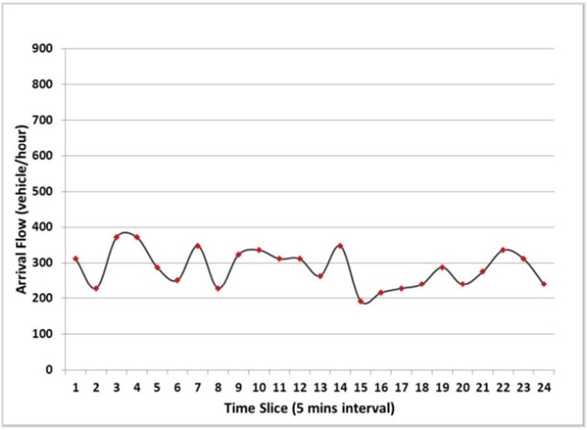


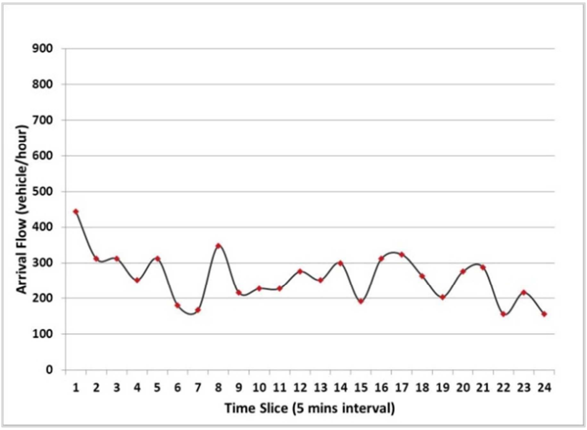
(a) Primary Stream



(b) Secondary Stream

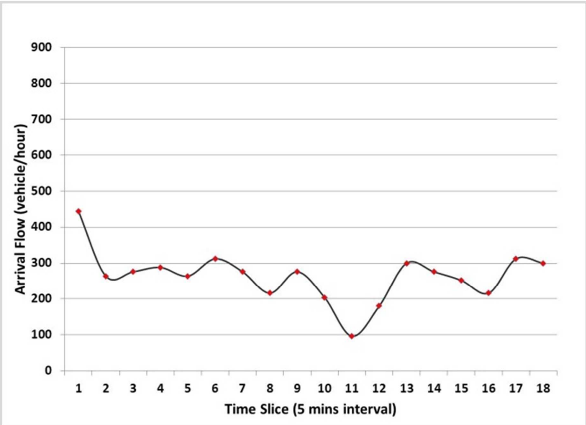
Figure B.3: Vehicle arrival profile for Site 12

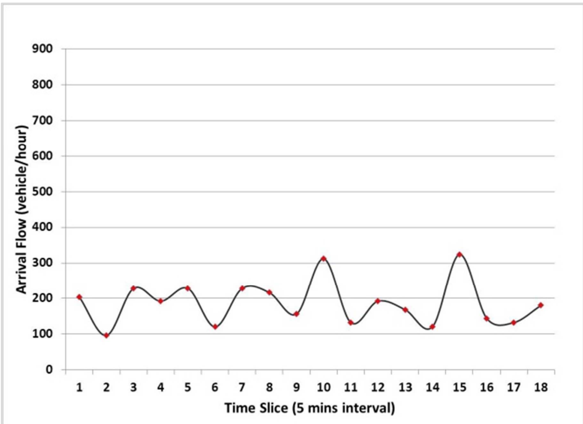




(a) Primary Stream (b) Secondary Stream

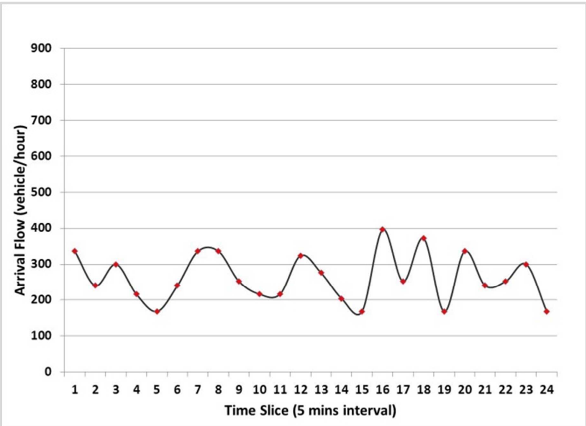
Figure B.4: Vehicle arrival profile for Site 13

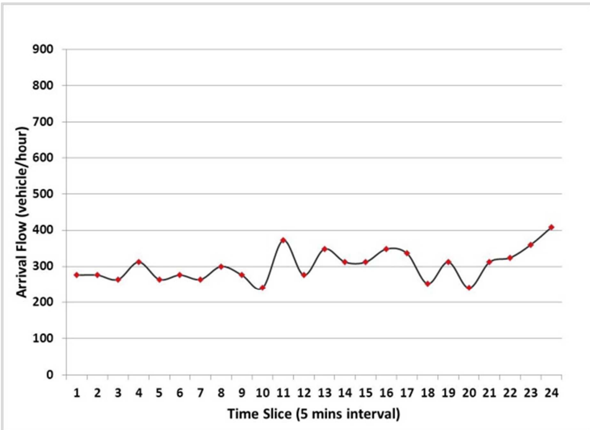




(a) Primary Stream (b) Secondary Stream

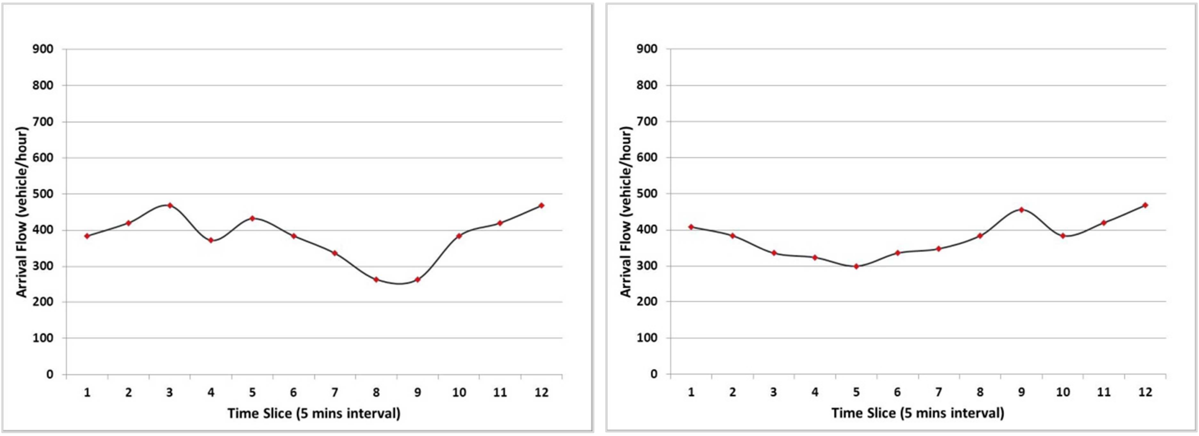
Figure B.5: Vehicle arrival profile for Site 14





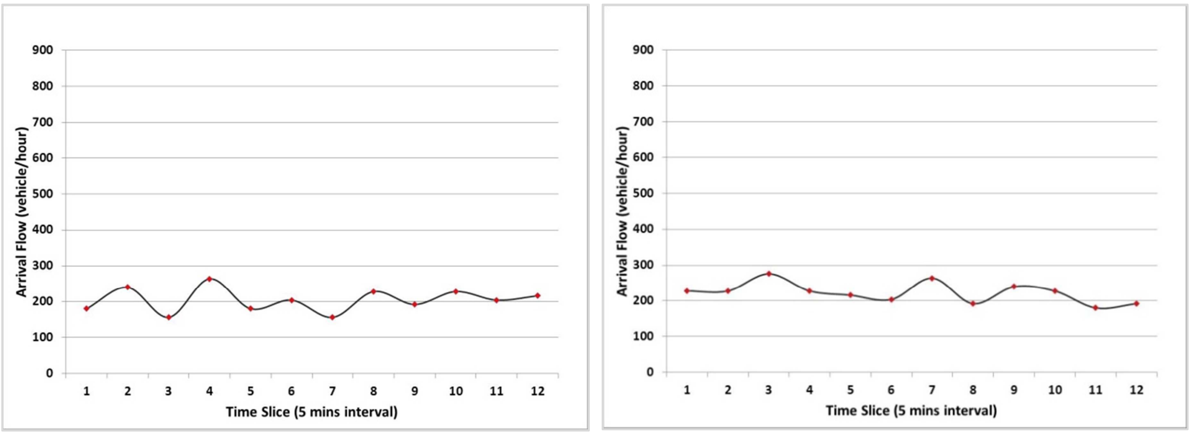
(a) Primary Stream (b) Secondary Stream

Figure B.6: Vehicle arrival profile for Site 16a



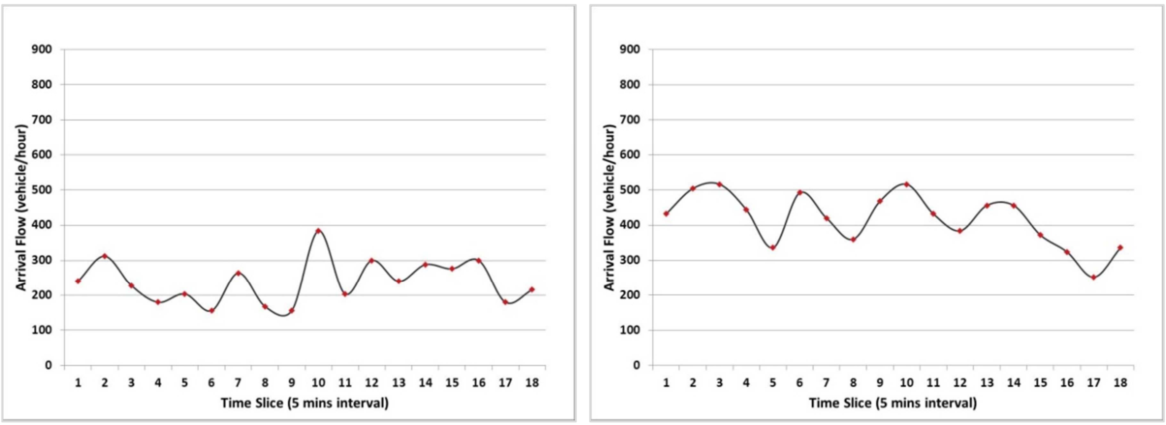
(a) Primary Stream (b) Secondary Stream

Figure B.7: Vehicle arrival profile for Site 16b



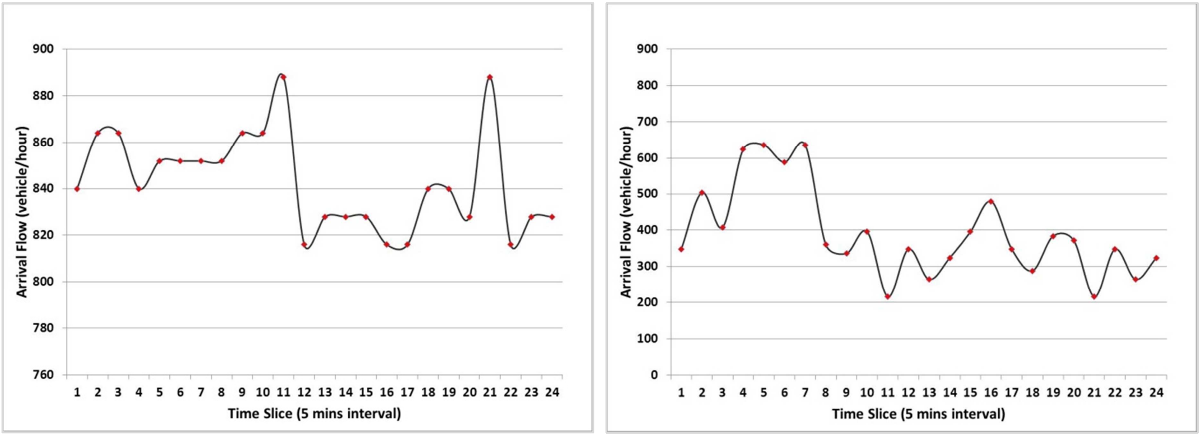
(a) Primary Stream (b) Secondary Stream

Figure B.8: Vehicle arrival profile for Site 17



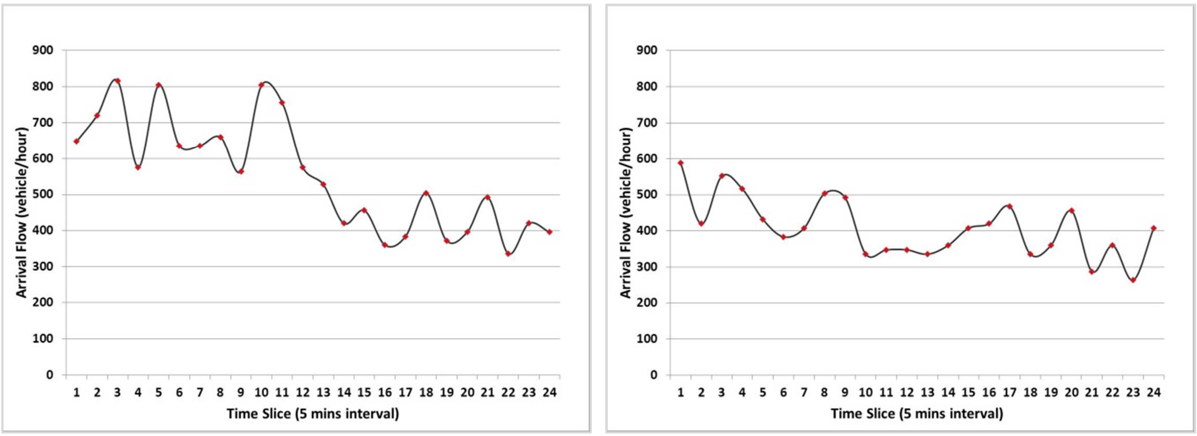
(a) Primary Stream (b) Secondary Stream

Figure B.9: Vehicle arrival profile for Site 18



(a) Primary Stream (b) Secondary Stream

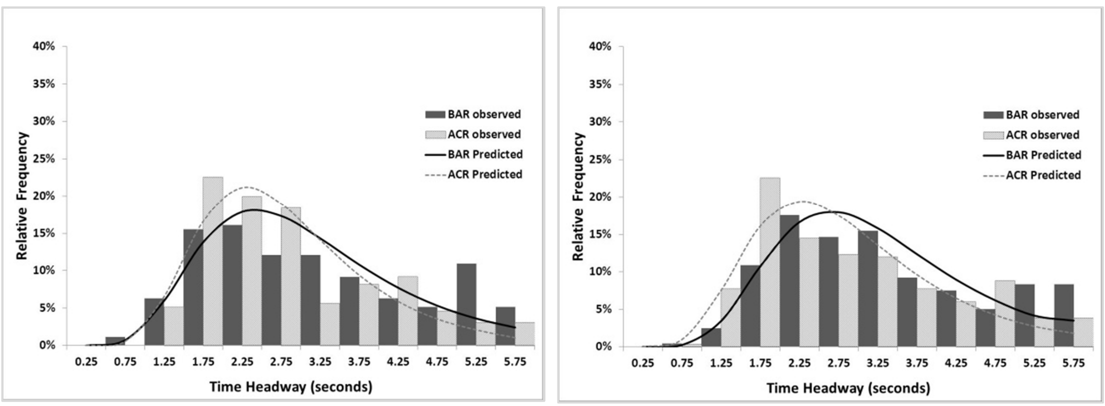
Figure B.10: Vehicle arrival profile for Site 19a



(b) Primary Stream (b) Secondary Stream

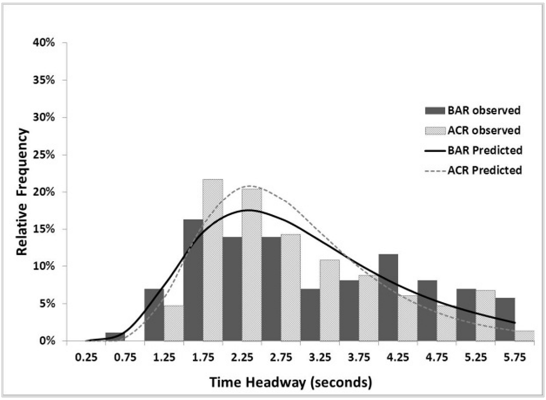
Figure B.11: Vehicle arrival profile for Site 19b

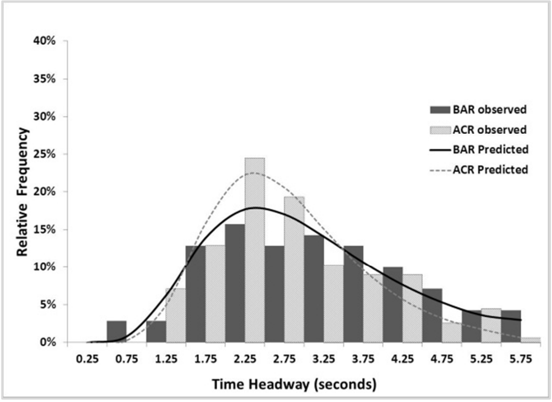
B.2 Time headway distribution



(c) Primary Stream (b) Secondary Stream

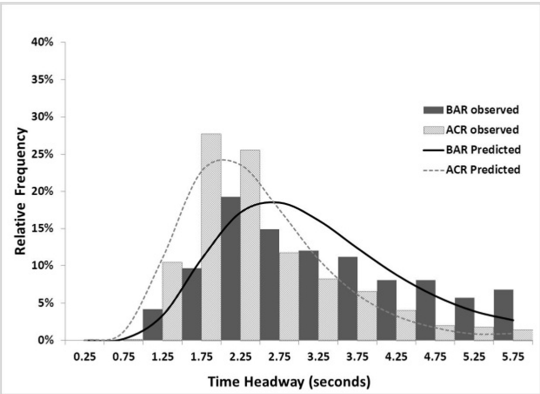
Figure B.12: Time headway distribution for Site 11

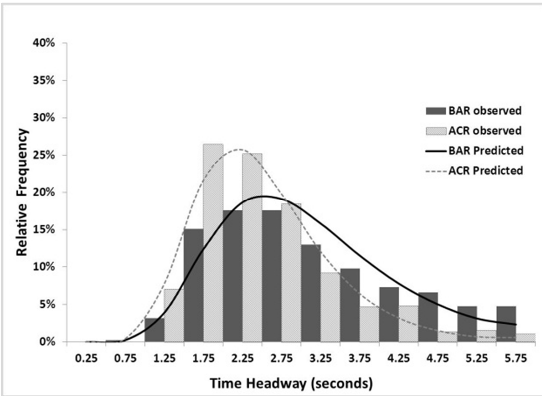




(a) Primary Stream (b) Secondary Stream

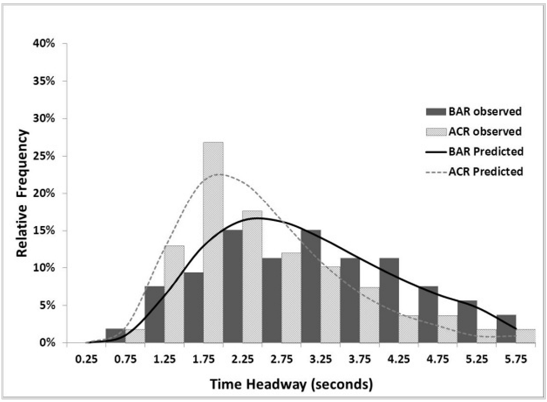
Figure B.13: Time headway distribution for Site 12

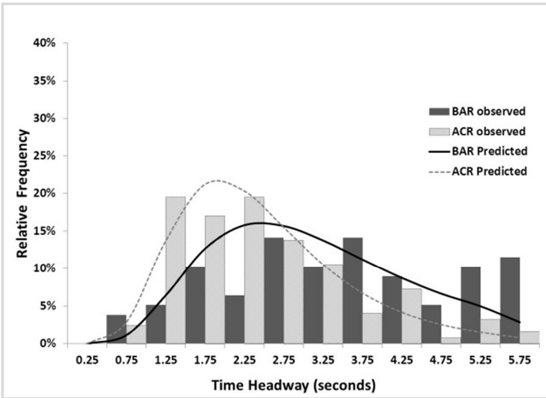




(a) Primary Stream (b) Secondary Stream

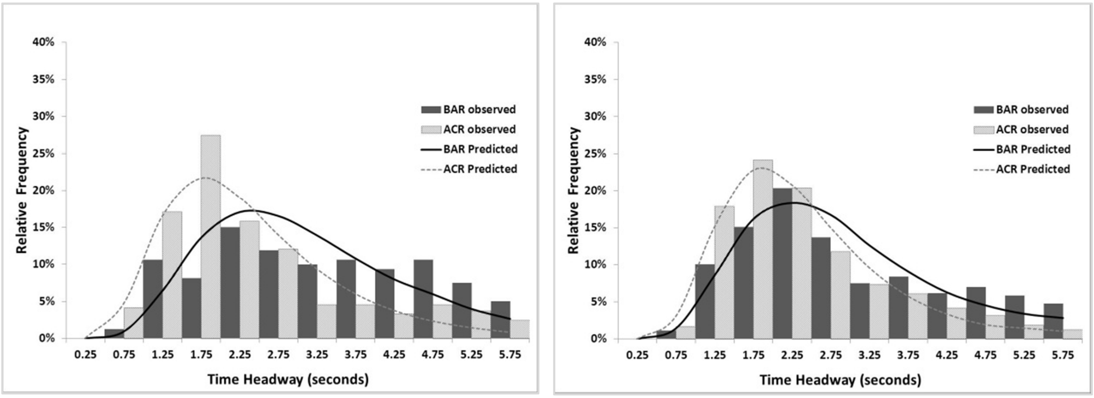
Figure B.14: Time headway distribution for Site 16





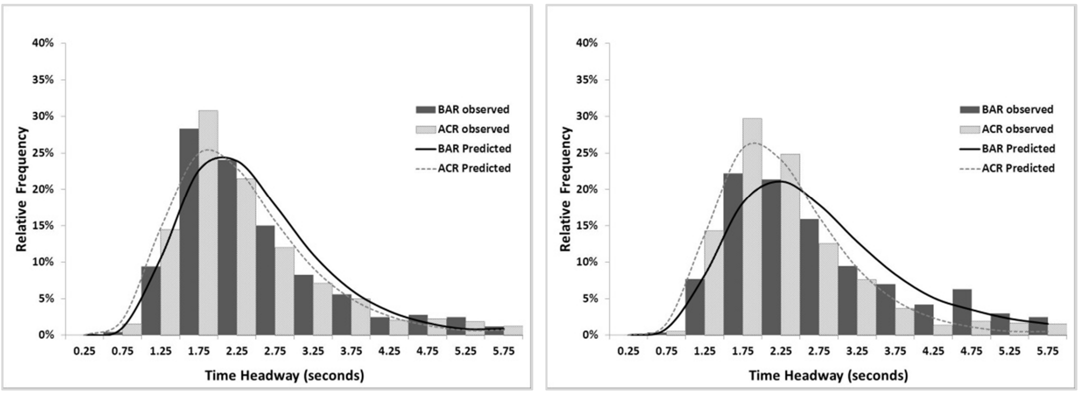
(a) Primary Stream (b) Secondary Stream

Figure B.15: Time headway distribution for Site 17



(a) Primary Stream (b) Secondary Stream

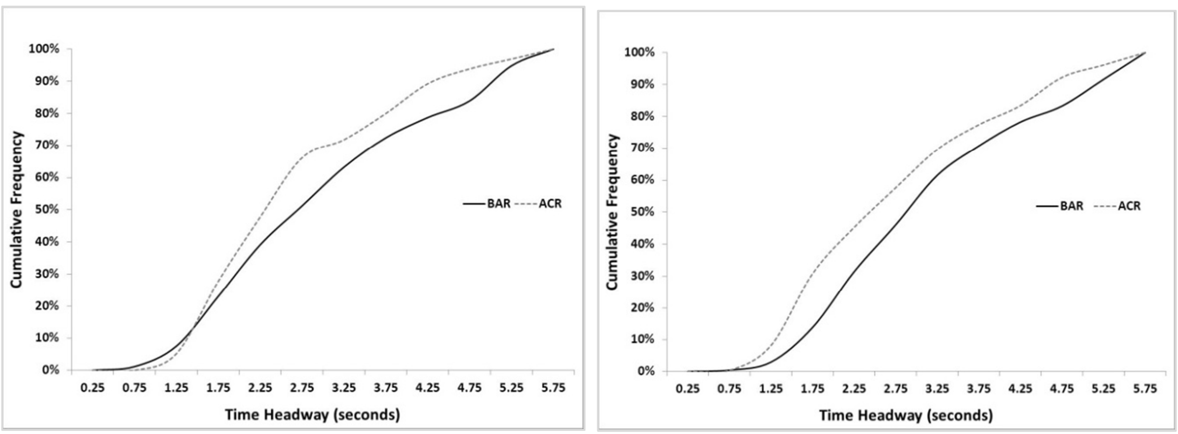
Figure B.16: Time headway distribution for Site 18



(a) Primary Stream (b) Secondary Stream

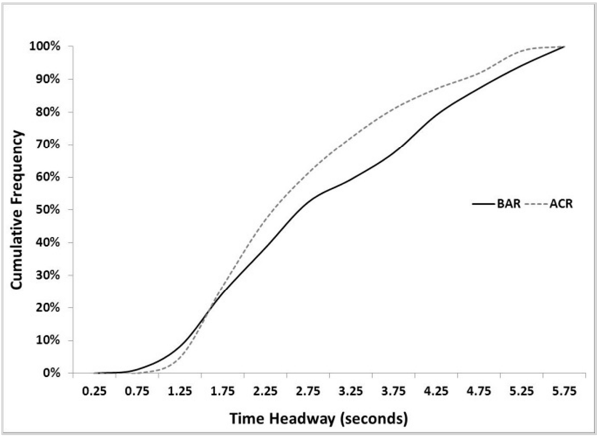
Figure B.17: Time headway distribution for Site 19

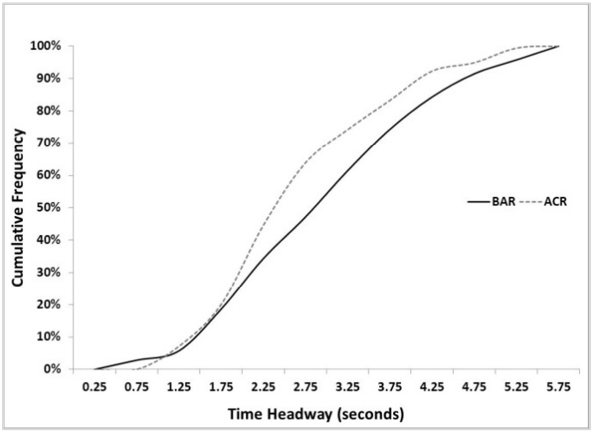
B.3 Time headway cumulative distribution



(a) Primary Stream (b) Secondary Stream

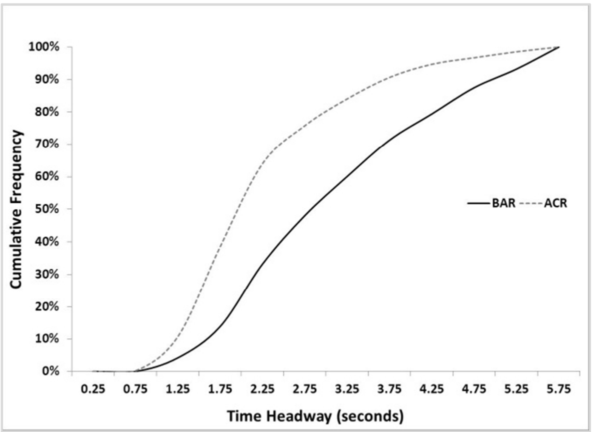
Figure B.18: Cumulative distribution for time headway for Site 11

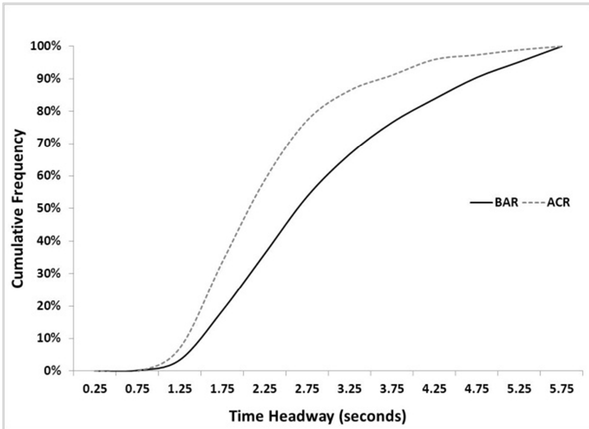




(a) Primary Stream (b) Secondary Stream

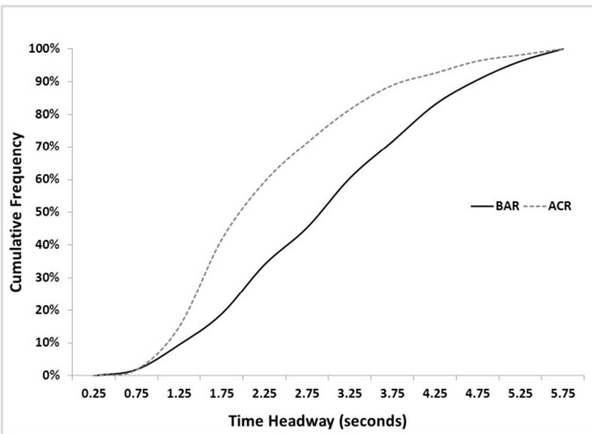
Figure B.19: Cumulative distribution for time headway for Site 12

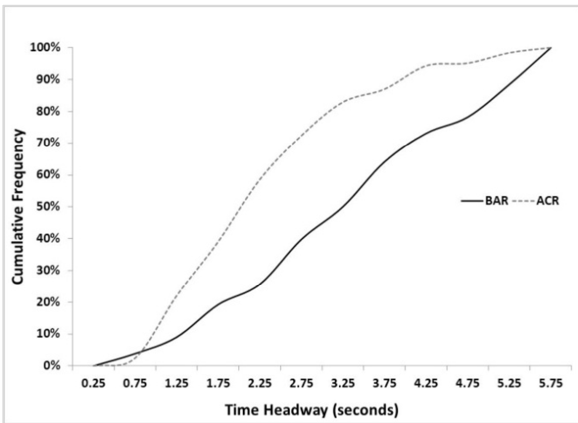




(a) Primary Stream (b) Secondary Stream

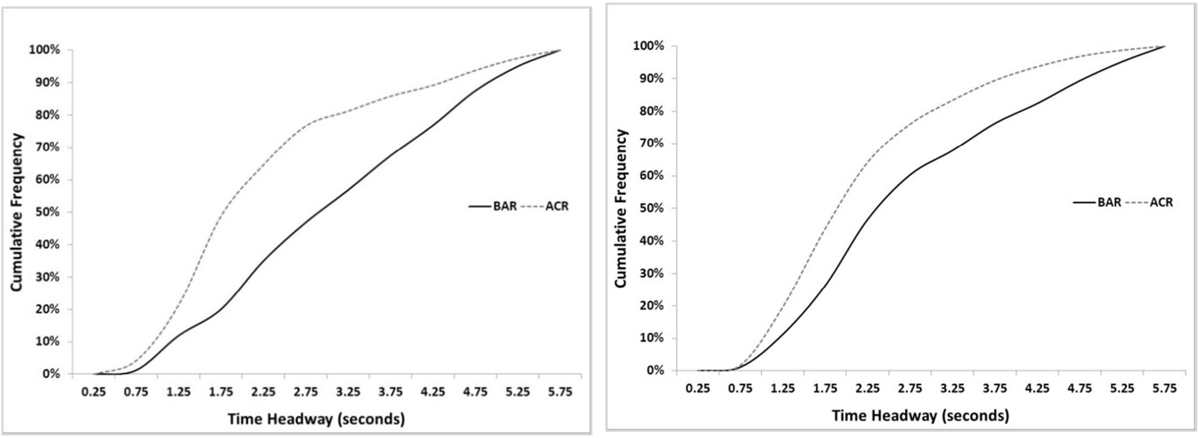
Figure B.20: Cumulative distribution for time headway for Site 16





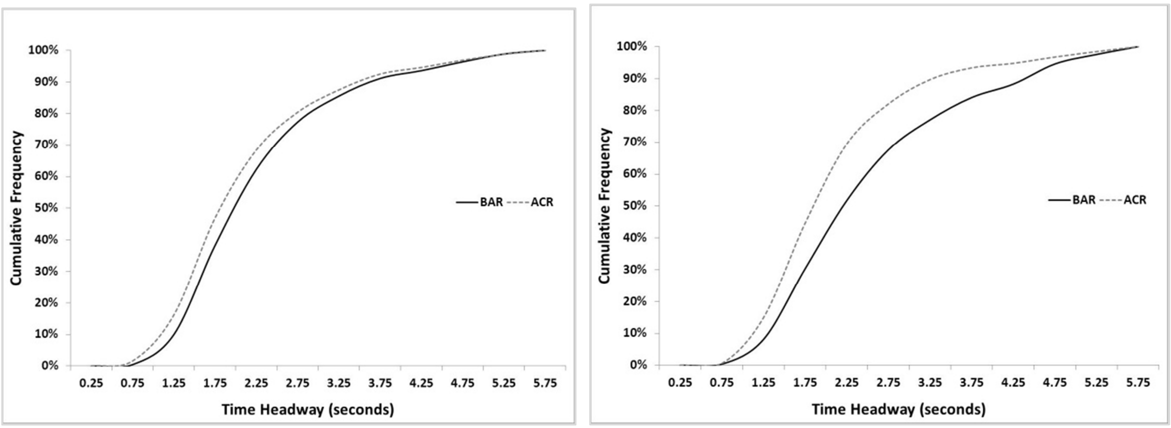
(a) Primary Stream (b) Secondary Stream

Figure B.21: Cumulative distribution for time headway for Site 17



(a) Primary Stream (b) Secondary Stream

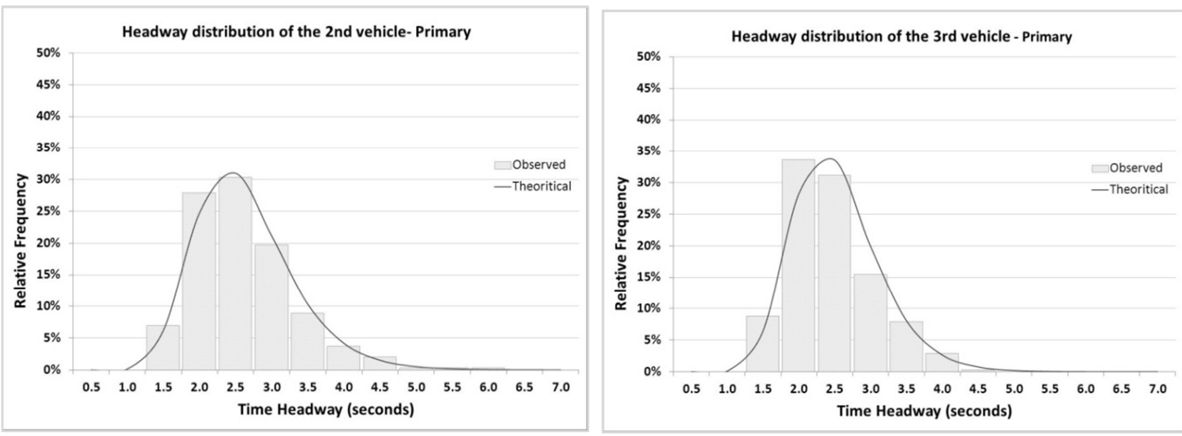
Figure B.22: Cumulative distribution for time headway for Site 18



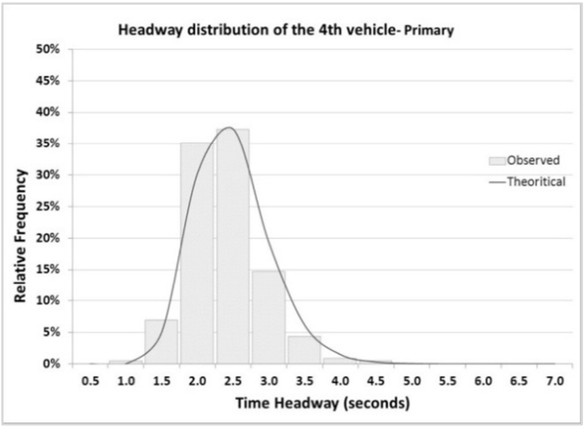
(a) Primary Stream (b) Secondary Stream

Figure B.23: Cumulative distribution for time headway for Site 19

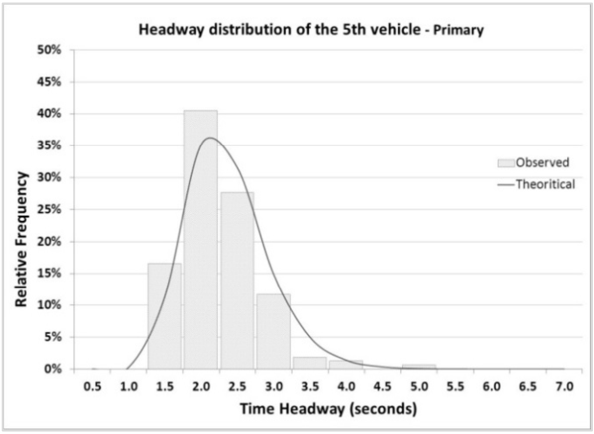
B.4 Move-up time (MUT)



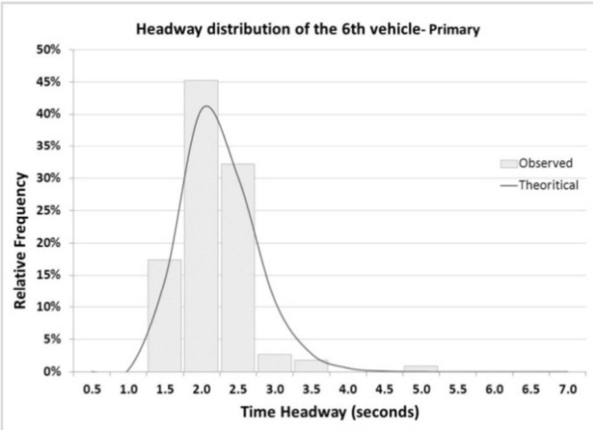
(d) Headway of the 2nd Vehicle (b) Headway of the 3rd Vehicle



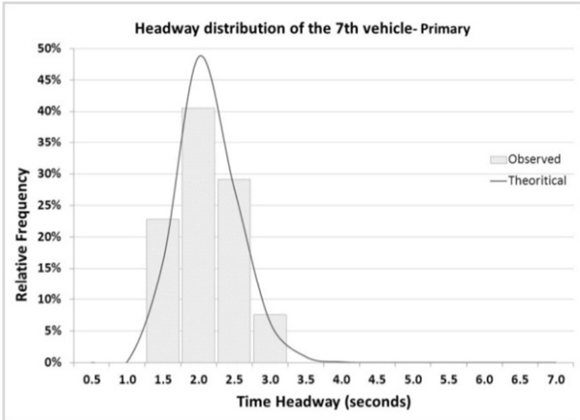
(c) Headway of the 4th Vehicle



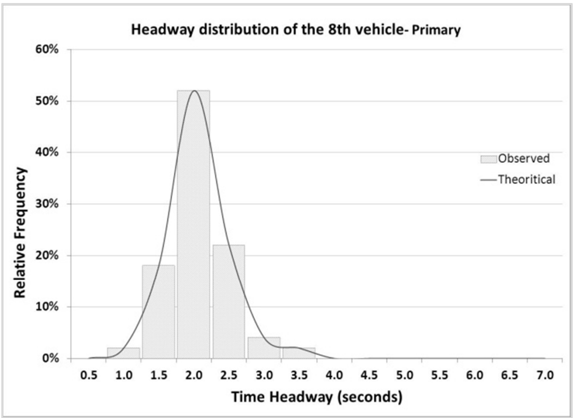
(d) Headway of the 5th Vehicle



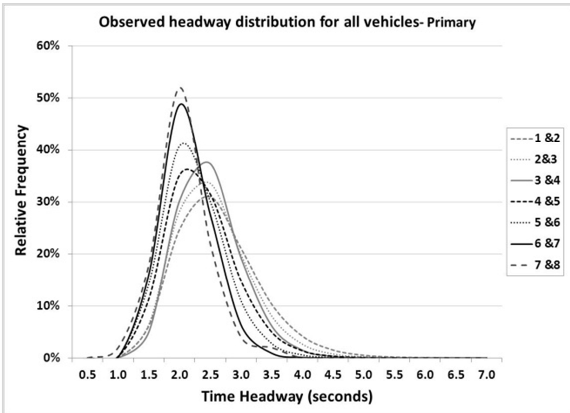
(e) Headway of the 6th Vehicle



(f) Headway of the 7th Vehicle

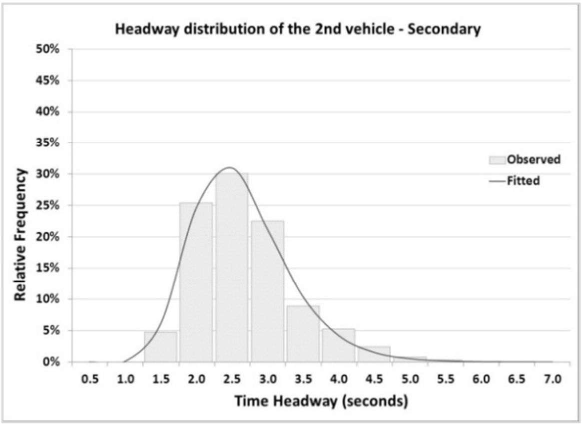


(g) Headway of the 8th Vehicle

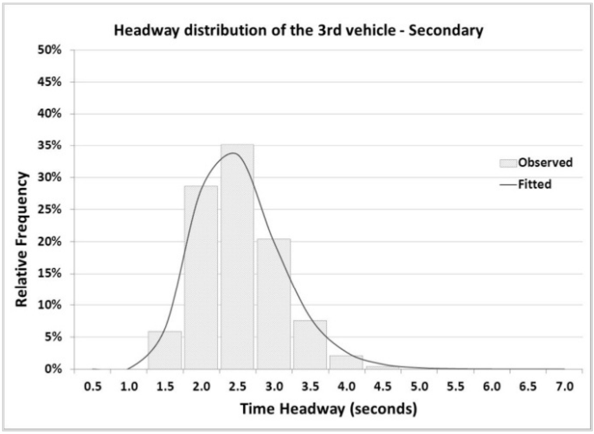


(h) Headway Distribution of All Vehicles

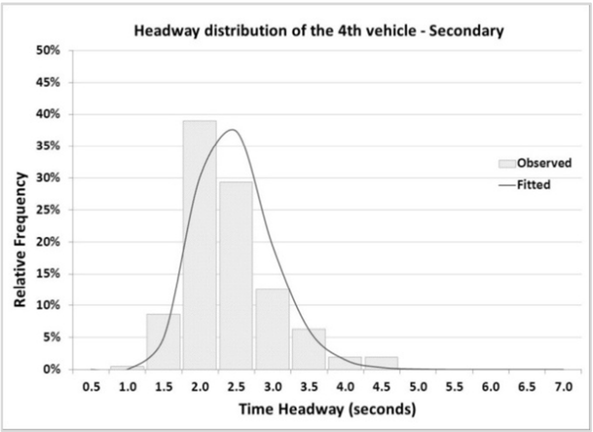
Figure B.24: Distribution of MUT for Primary Stream in Shuttle-lane Roadworks



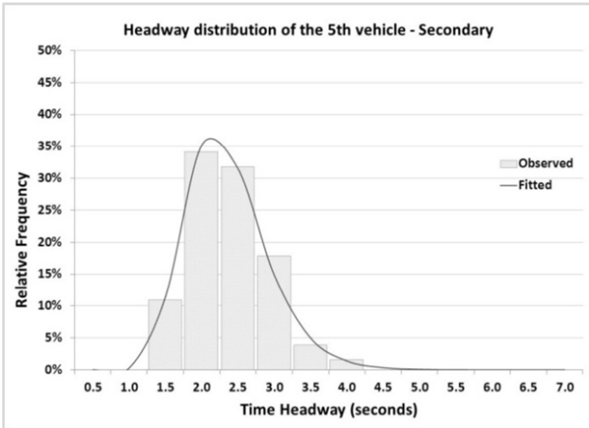
(a) Headway of the 2nd Vehicle



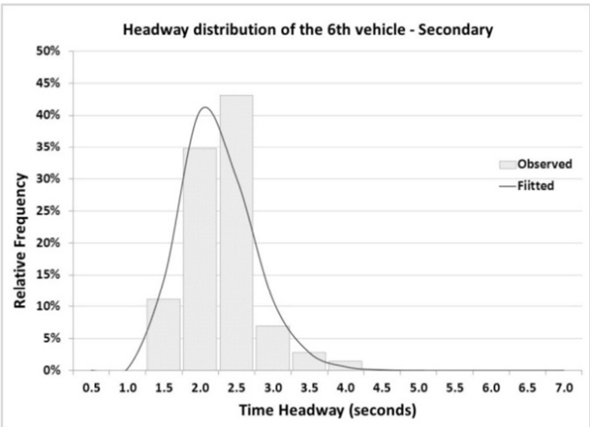
(b) Headway of the 3rd Vehicle



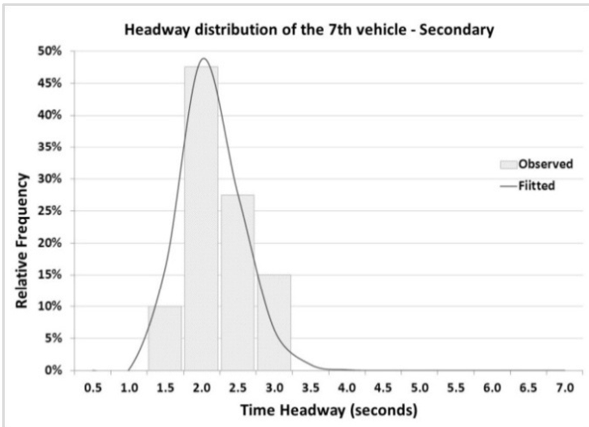
(c) Headway of the 4th Vehicle



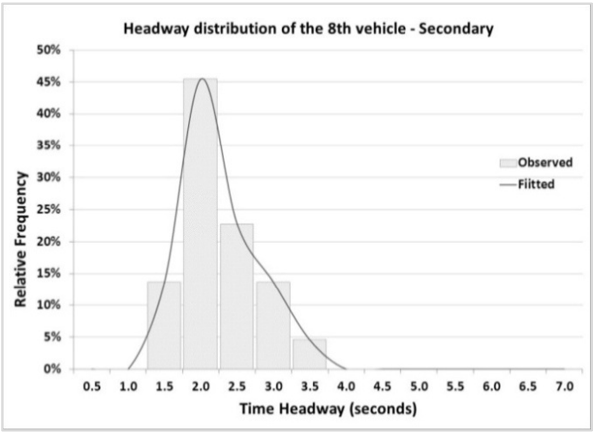
(d) Headway of the 5th Vehicle

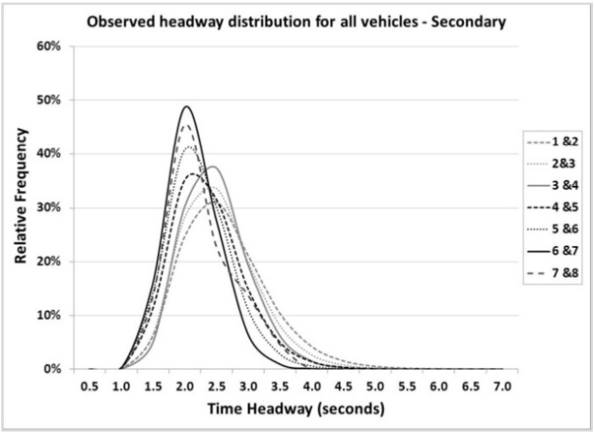


(e) Headway of the 6th Vehicle



(f) Headway of the 7th Vehicle





(g) Headway of the 8th Vehicle (h) Headway Distribution of All Vehicles

Figure B.25: Distribution of MUT for Secondary Stream in Shuttle-lane Roadworks

B.5 VA detection failure

Table B.2: VA signals detection failure

Site	Direction	Time	Comments
Site 11a	S	00:03:39 00:30:20	1. The green phase was terminated and only 8 seconds later, the green phase was resumed again with vehicles already queuing on the primary stream. 2. The green phase terminated and only 8 seconds later, the green phase resumed again with vehicles already queuing on the primary stream.
Site 12	P	00:59:35	The green phase was terminated and only 30 seconds later, the green phase was resumed again with vehicles already queuing on the secondary stream.
	S	00:18:01	The green phase was terminated and only 30 seconds later, the green phase resumed again with vehicles already queuing on the primary stream.
Site 17	S	00:53:26	The green phase was terminated and only 8 seconds later, the green phase was resumed again twice with no vehicles queuing on the primary stream.
Site 18	P	00:04:45 00:57:11 01:12:01	1. Vehicles stopped 45 m away from the stop line and have not been detected by the MVD as an effect of invisible WAIT HERE SIGN. 2. Green time stayed for 75 seconds with no continuous arrival of vehicles forcing vehicles in the secondary stream to violate the red light 3. The green phase was terminated and only 12 seconds later, the green phase was resumed again with vehicles already queuing on the secondary stream. This happened twice until 01:13:30 which forced the queued traffic on the secondary stream to violate the traffic light
	S	00:04:41 00:40:51	1. The green phase was terminated and only 10 seconds later, the green phase was resumed again with vehicles already queuing on the primary stream. This happens twice until 00:06:40. 2. The green phase was terminated and only 10 seconds later, the green phase was resumed again.

APPENDIX C: MODEL CALIBRATION AND VALIDATION

Table C.1: Model calibration statistics – signal compliance (Site 16a)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Number of Amber crossing (veh)	22	21	33	34
Number of Amber crossing (%)	4%	4%	5%	6%
Number of Red violations (veh)	12	12	16	15
Number of Red violations (%)	2%	2%	3%	3%
Total violations on Amber & Red (veh)	**34**	**33**	**49**	**49**
Total violations on Amber & Red (%)	**6%**	**6%**	**8%**	**8%**
Crossed cycles on Amber (cycles)	16	14	32	30
Crossed cycles on Amber (%)	13%	12%	27%	19%
Violated cycles on Red (cycles)	9	9	16	15
Violated cycles on Red (%)	8%	8%	13%	13%

Table C.2: Model calibration statistics - MUT (Site 16a)

Vehicle Position	MUT (sec)	
	Observed	Modelled
2	(P) 2.24 (S) 2.60	(P) 2.69 (S) 2.58
3	(P) 2.22 (S) 2.35	(P) 2.18 (S) 2.25
4	(P) 2.15 (S) 2.21	(P) 2.02 (S) 2.00
5	(P) 2.12 (S) 2.16	(P) 1.99 (S) 1.86
6	(P) 2.07 (S) 2.09	(P) 1.93 (S) 1.94
7	(P) 1.89 (S) 1.98	(P) 1.86 (S) 1.90
Total	**(P) 12.69 (S) 13.39**	**(P) 12.48 (S) 12.53**

P is Primary stream S is Secondary stream

Table C.3: Model calibration statistics - compliance (Site 12)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Number of Amber crossing (veh)	3	2	1	1
Number of Amber crossing (%)	1%	1%	0%	0%
Number of Red violations (veh)	0	1	0	1
Number of Red violations (%)	0%	0%	0%	0%
Total violations on Amber & Red (veh)	**3**	**3**	**1**	**2**
Total violations on Amber & Red (%)	**1%**	**1%**	**0%**	**1%**
Crossed cycles on Amber (cycles)	3	2	1	1
Crossed cycles on Amber (%)	4%	3%	1%	1%
Violated cycles on Red (cycles)	0	1	0	1
Violated cycles on Red (%)	0%	1%	0%	1%

Table C.4: Model calibration statistics - MUT (Site 12)

Vehicle Position	MUT (sec)	
	Observed	Modelled
2	(P) 2.38 (S) 2.48	(P) 2.30 (S) 2.33
3	(P) 2.21 (S) 2.24	(P) 2.20 (S) 2.32
4	(P) 2.33 (S) 2.10	(P) 2.10 (S) 2.00
5	(P) 2.16 (S) 2.14	(P) 2.04 (S) 2.00
6	(P) 2.02 (S) 2.42	(P) 1.88 (S) 2.00
Total	**(P) 11.08 (S) 11.37**	**(P) 10.52 (S) 10.65**

P is Primary stream S is Secondary stream

Table C.5: Model validation statistics - compliance (Site 16b)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Number of Amber crossing (veh)	25	24	22	23
Number of Amber crossing (%)	7%	6%	6%	6%
Number of Red violations (veh)	9	10	3	3
Number of Red violations (%)	2%	3%	1%	1%
Total violations on Amber & Red (veh)	**34**	**34**	**25**	**26**
Total violations on Amber & Red (%)	**9%**	**9%**	**7%**	**7%**
Crossed cycles on Amber (cycles)	15	13	17	18
Crossed cycles on Amber (%)	25%	22%	28%	30%
Violated cycles on Red (cycles)	8	8	3	3
Violated cycles on Red (%)	13%	13%	5%	5%

Table C.6: Model validation statistics - MUT (Site 16b)

Vehicle Position	MUT (sec)	
	Observed	Modelled
2	(P) 2.42 (S) 2.51	(P) 2.87 (S) 2.92
3	(P) 2.11 (S) 2.19	(P) 2.22 (S) 2.18
4	(P) 2.05 (S) 2.04	(P) 1.97 (S) 1.95
5	(P) 1.95 (S) 1.98	(P) 1.87 (S) 1.90
6	(P) 1.92 (S) 1.92	(P) 1.86 (S) 1.84
7	(P) 1.81 (S) 1.90	(P) 1.80 (S) 1.80
Total	**(P) 12.26** **(S) 12.54**	**(P) 12.59** **(S) 12.60**

P is Primary stream S is Secondary stream

Table C.7: Model validation statistics - compliance (Site 19)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Number of Amber crossing (veh)	38	36	31	34
Number of Amber crossing (%)	3%	3%	4%	4%
Number of Red violations (veh)	21	21	18	19
Number of Red violations (%)	2%	2%	2%	2%
Total violations on Amber & Red (veh)	**59**	**57**	**49**	**53**
Total violations on Amber & Red (%)	**5%**	**5%**	**6%**	**7%**
Crossed cycles on Amber (cycles)	19	23	20	22
Crossed cycles on Amber (%)	30%	37%	32%	35%
Violated cycles on Red (cycles)	17	18	19	19
Violated cycles on Red (%)	27%	29%	30%	30%

Table C.8: Model validation statistics - MUT (Site 19)

Vehicle Position	MUT (sec)	
	Observed	Modelled
2	(P) 2.32 (S) 2.40	(P) 2.79 (S) 2.85
3	(P) 2.02 (S) 2.09	(P) 2.14 (S) 2.26
4	(P) 1.97 (S) 2.01	(P) 1.95 (S) 1.98
5	(P) 1.89 (S) 1.91	(P) 1.83 (S) 1.93
6	(P) 1.83 (S) 1.90	(P) 1.78 (S) 1.86
7	(P) 1.81 (S) 1.88	(P) 1.74 (S) 1.86
Total	**(P) 11.83** **(S) 12.19**	**(P) 12.23** **(S) 12.75**

P is Primary stream S is Secondary stream

Table C.9: Model validation statistics - compliance (Site 17)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Number of Amber crossing (veh)	0	0	0	2
Number of Amber crossing (%)	0%	0%	0%	0%
Number of Red violations (veh)	1	0	1	1
Number of Red violations (%)	0%	0%	0%	0%
Total violations on Amber & Red (veh)	**1**	**0**	**1**	**3**
Total violations on Amber & Red (%)	**0%**	**0%**	**0%**	**1%**
Crossed cycles on Amber (cycles)	0	5	0	2
Crossed cycles on Amber (%)	0%	4%	0%	2%
Violated cycles on Red (cycles)	1	0	1	1
Violated cycles on Red (%)	1%	0%	1%	1%

Table C.10: Model validation statistics - MUT (Site 17)

Vehicle Position	MUT (sec)	
	Observed	Modelled
2	(P) 2.43 (S) 2.59	(P) 2.39 (S) 2.50
3	(P) 2.22 (S) 2.16	(P) 2.36 (S) 2.20
4	(P) 2.31 (S) 2.25	(P) 2.00 (S) 2.00
5	(P) 2.25 (S) 2.00	(P) 1.90 (S) 2.00
Total	**(P) 9.21** **(S) 9.00**	**(P) 8.64** **(S) 8.70**

P is Primary stream S is Secondary stream

Table C.11: Model validation statistics - compliance (Site 18)

Violation	Primary Stream		Secondary Stream	
	Observed	Modelled	Observed	Modelled
Number of Amber crossing (veh)	4	5	5	6
Number of Amber crossing (%)	1%	1%	1%	1%
Number of Red violations (veh)	0	0	1	2
Number of Red violations (%)	0%	0%	0%	0%
Total violations on Amber & Red (veh)	**4**	**5**	**6**	**8**
Total violations on Amber & Red (%)	**1%**	**1%**	**1%**	**1%**
Crossed cycles on Amber (cycles)	4	5	5	6
Crossed cycles on Amber (%)	4%	5%	5%	6%
Violated cycles on Red (cycles)	0	0	1	2
Violated cycles on Red (%)	0%	0%	1%	2%

Table C.12: Model validation statistics - MUT (Site 18)

Vehicle Position	MUT (sec)	
	Observed	Modelled
2	(P) 2.20 (S) 2.20	(P) 2.31 (S) 2.28
3	(P) 2.00 (S) 2.18	(P) 2.13 (S) 2.14
4	(P) 1.83 (S) 2.19	(P) 2.05 (S) 2.07
5	(P) 1.83 (S) 2.19	(P) 1.97 (S) 2.04
6	(P) 1.75 (S) 2.07	(P) 1.95 (S) 1.95
7	(P) 1.75 (S) 2.06	(P) 1.92 (S) 1.88
Total	**(P) 11.36 (S) 12.90**	**(P) 12.33 (S) 12.36**

P is Primary stream S is Secondary stream

Table C.13: R_{ACL} calibration values for all sites (primary stream)

Site	Obs. %	Obs. value	Iter. 1 %	Iter. 1 Value	Iter. 2 %	Iter. 2 Value	Final Iteration %	Final Iteration value
Site 12	3.8	3	3.8	6	3.5	5	3.0	3
Site 16a	22.5	34	22.5	39	21.0	37	18.0	33
Site 16b	10.6	34	10.6	39	10.0	37	8.0	34
Site 17	1.6	1	1.6	3	1.4	3	1.0	0
Site 18	0.3	4	0.3	6	0.4	6	0.5	5
Site 19b	23.5	59	23.5	65	23.0	63	21.0	57

Obs.: is the Observed value

Iter.: is the Iteration, which is the average of 10 simulation runs (each simulation run required approximately 2 hours of running time)

Table C.14: R_{ACL} calibration values for all sites (secondary stream)

Site	Obs. %	Obs. value	Iter. 1 %	Iter. 1 Value	Iter. 2 %	Iter. 2 Value	Final Iteration %	Final Iteration value
Site 12	1.3	1	1.3	4	1.0	3	0.5	2
Site 16a	14.8	49	14.8	60	14.0	58	12.0	49
Site 16b	25.1	25	25.1	32	24.5	30	23.0	26
Site 17	1.6	1	1.6	5	1.4	4	1.0	3
Site 18	1.9	6	1.9	10	1.6	8	1.5	6
Site 19b	27.2	49	27.2	61	26.5	58	25.0	53

Obs.: is the Observed value

Iter.: is the Iteration, which is the average of 10 simulation runs (each simulation run required approximately 2 hours of running time)