MODELLING DRIVERS' BEHAVIOUR WITHIN THE DILEMMA ZONE AT TRAFFIC SIGNAL JUNCTIONS

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DECLARATION

Some parts of this PhD study have been presented in the following conferences and events:

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* This paper won the best presentation prize in the session of 'Speed' Vehicles & Managing their Responses.

DEDICATION

To my family and friends,

For making it all worthwhile ….

ABBREVIATIONS

AIMSUN:	Advanced Interactive Microscopic Simulator for Urban and Non-urban Networks.
ALR:	Amber Light Running
ARLS:	Amber and Red Light Stopping
AV:	Autonomous Vehicles
CARSIM:	CAR following SIMulation model
DRAC:	Deceleration Rate to Avoid a Collision
DZ:	Dilemma Zone
FORTRAN:	FORmula TRANslating System
FT:	Fixed-Time signals
GEH:	Geoffrey E. Havers statistical test
GHM:	Gazis-Herman-Maradudin model
GHR:	Gazis-Herman-Rothery model
GSCD:	Green Signal Countdown Display
GZ:	Go Zone
HGVs:	Heavy Goods Vehicles
ITE:	Institute of Transportation Engineering
ITS:	Intelligent Transport Systems
MITSIM:	MIcroscopic Traffic SIMulator
MITRAM:	MIcroscopic model for analysing TRAffic jaM
OZ:	Option Zone
PARAMICS:	PARAllel MICroscopic Simulation
PCs:	Passenger Cars
RLC:	Red Light Cameras
RLR:	Red Light Running
RMSE:	Root Mean Square Error
RMSPE:	Root Mean Square Percentage Error
RTAs:	Road Traffic Accidents
SSMs:	Surrogate Safety Measures
SZ:	Stop Zone
TTC:	Time To Collision
VA:	Vehicle-Actuated time signals
VISSIM:	VISual SIMulation

SYMBOLS

Δt :	Scanning time (0.5 sec)
μ :	Mean
σ :	Standard deviation
a :	Acceleration rate (i.e. positive value) or deceleration rate (i.e. negative value), measured in m/sec^2
a_1:	Maximum acceleration rate from mechanical capability of vehicle, measured in m/sec^2
a_2:	Acceleration/deceleration rate for vehicle moving at desired speed, measured in m/sec^2
a_3:	Acceleration/deceleration rate for non-collision situations, measured in m/sec^2
a_4:	Acceleration/deceleration rate for slow-moving situations, measured in m/sec^2
a_5:	Acceleration rate for moving from stationary situations, measured in m/sec^2
a_6:	Deceleration rate for stopping behind the stopline, measured in m/sec^2
amb:	Amber period (usually 3 seconds after the green phase)
BS :	Buffer spacing or gap distance between two successive vehicles, measured in metre
CD :	Clearance distance, measured in metre
CT:	Cycle time, measured in second
e:	Road gradient, measured in percent (such as 0.05 for 5%)
f:	Friction coefficient, measured in percent (such as 0.5 for 50%)
g:	Gravity acceleration measured in m/sec^2
GN:	Generated green time, measured in second
Green:	Actual green period, measured in second
HGVs%	Percentage of Heavy Goods Vehicles in the flow (such as 0.08 for 8%)
HW:	Time headway, measured in second
L_v :	Length of the vehicle, measured in metre
L_{vF} :	Length of the following vehicle, measured in metre
L_{vL} :	Length of the leading vehicle, measured in metre
$MADR_F$:	Maximum available deceleration rate of the following vehicle, measured in m/sec^2
$MADR_L$:	Maximum available deceleration rate of the leading vehicle, measured in m/sec^2
$MADR_n$:	Maximum available deceleration rate of the vehicle *n*, measured in m/sec^2

Max GN:	Maximum green period, measured in second
Min GN:	Minimum green period, measured in second
Pos_F :	Position of the following vehicle, measured in metre
Pos_L :	Position of the leading vehicle, measured in metre
Pos_n :	Position of the vehicle *n*, measured in metre
Pos_{nSL} :	Position of vehicle *n* from the stopline, measured in metre
Q:	Flow rate, measured in vph
R_t :	Driver reaction time, measured in second
RAND:	Random number (usually between 0 and 1)
RAmb:	Generated red-amber time, measured in second
RD:	Generated red time, measured in second
Red:	Red period, measured in second
Red-Amber:	Red-Amber period (usually 2 second before the Green phase)
Shift:	The additional time such as 0.25, 0.5 and 1.0, measured in second
SSD:	Safe Stopping Distance, measured in metre
T:	Total simulation time, measured in second
T_F	Time headway of the following vehicle, measured in second
T_L	Time headway of the leading vehicle, measured in second
U	Theil's inequality coefficient
U_m	Theil's mean difference
U_s	Theil's standard deviation difference
V_F :	Speed of the following vehicle, measured in m/sec
V_L :	Speed of the leading vehicle, measured in m/sec
V_n :	Speed of the vehicle *n*, measured in m/sec
w:	Intersection width, measured in metre
x_i:	Observed data at time interval i
y_i:	Simulated data at time interval i

ABSTRACT

The current study introduces a newly developed, calibrated, and validated micro-simulation model for predicting drivers' decisions following the onset of an amber traffic light signal under the effect of the dilemma zone where a driver can neither stop safely nor cross and clear the junction before the onset of red. The purpose of building this model is to investigate the effects of various parameters (such as heavy goods vehicles proportion HGVs%, intergreen length and installation of red light cameras) on drivers' compliance with the traffic light signal change and junction capacity as well as vehicles' delays.

Based on existing traffic simulation models such as CARSIM, the simulation methodology considered car-following algorithms with some modifications. These modified model includes the dilemma zone algorithms for predicting drivers' STOP/GO decisions after the onset of amber. Various parameters were modelled such as distances from the stopline, travelling speeds, drivers' responses to the signal change, junction width and the length of the amber period. The codes were written using FORTRAN-95 programming language.

Traffic data from five sites were collected and analysed to be used for the calibration and validation of the developed model. The collected data included information about traffic flow characteristics, drivers' compliance and junction details such as width, and the traffic lights periods and operation system (i.e. Fixed-Time (FT) or Vehicle-Actuated (VA) signals mode).

Finally, the results of the newly developed model revealed that the number of signal violations increase as the intergreen length increases. Vehicle delays at junctions operated by FT signals are higher by 20% than those for VA signals mode. Moreover, an increase in the HGVs% causes a reduction in the red light running events by 40% and 45% at VA and FT traffic signal junctions, respectively. When the HGVs% constitutes of 50% of traffic composition, junction capacity is reduced by 42% and 51% at VA and FT junctions, respectively. In addition, the installation of red light cameras in the model showed positive effects on the reduction of signal violations. The reduction percentages were 70% at junctions controlled by VA signals and about 20% at FT traffic signal junctions.

CHAPTER ONE: INTRODUCTION

1.1 Background

Research in the area of road safety has been developed as traffic flow and vehicle numbers have increased. This increase has been accompanied by a gradual increase in the accident frequencies that result in serious injuries and fatalities. Road traffic accidents (RTAs) reflect global health and safety issues. In addition to the suffering and psychological harm experienced by people injured and families of loved ones, traffic accidents cause an economic problem in most countries. According to the World Health Organization (2015), about 1.25 million people are killed on the roads and 50% of those are vulnerable road users. These accidents cost European countries between 2% and 5% of their gross domestic product. In the UK, it is estimated that the average value of preventing one injury accident on built-up roads is approximately £60,000 per year, based on statistical data published by the Department for Transport (2017).

Intersections are the most interesting places to study collision dynamics because of traffic interaction between road users, particularly relating to through and turning flows near or at the centre of intersection. A full definition of an RTA was given by Baguley (2001) as: "*a rare, random, multi-factor event always preceded by a situation in which one or more road users have failed to cope with their environment*", such as a red signal violation or a rear-end collision that should be reported by the police. As a result, a traffic accident is caused by one or more contributory factors (e.g. speeding, road design, vehicle defects or weather conditions) leading to death, disabilities from injuries, and/or property damage (Khisty and Lall, 1998). Identifying those factors depends on an effective traffic accident database.

With this in mind, it is worth investigating in depth signal compliance and car-following behaviour microscopically for the case of vehicles approaching traffic signals. Studying these concepts may lead to an understanding of drivers' responses to signal changes. This could change the possibility of accident occurrence particularly at signalised junctions (such as rear-end collisions and red light running phenomena).

1.2 Significance of this research

Despite the fact that signalisation has increased safety and reduced the number of collisions at signalised intersections by 15%-30%, it seems less likely to be effective in reducing other types of accidents such as rear shunts (Kennedy and Sexton, 2009). Drivers have to make STOP/GO decisions after the onset of amber. Based on their speeds and positions from the stopline, some drivers hesitate when deciding whether to proceed through the junction or start decelerating for the red phase. This phenomenon has been recognised as the influence of the dilemma zone which can be defined as a critical area on the carriageway where drivers can neither clear the junction nor stop safely before the stopline (Gazis et al., 1960). It may cause red light violations and possibly severe accidents at the intersection area if drivers decide to cross, or start tailgating with other vehicles when stopping or decelerating suddenly during the amber interval.

This study is an attempt to investigate the complex behaviour governing such traffic situations at signalised junctions. It may be of interest to traffic, economic, health, social and safety agencies and may improve knowledge of the risk factors associated with RTAs which may help in identifying appropriate solutions to reduce conflicts on the approach towards traffic light junctions.

1.3 Aims and objectives

On the basis of the above introduction and previous findings and to achieve the purpose of this research, the study aims to answer the following questions:

1. Why are traffic conflicts happening at traffic signal junctions, such as rear-end collisions and signal violations?
2. What are the relationships between drivers' reaction times, speed and acceleration/deceleration rate for leading and following vehicles?
3. How are the variables, such as distance from the stopline, intersection width and intergreen period, affect drivers' behaviour, and how can the models be adapted accordingly?
4. What output variables can be used to indicate safety performance at junctions?
5. What design interventions can be tested by the model?

Answering the above questions will help to identify how drivers tend to react in response to the amber/red signal.

The main contribution of this research is to introduce a new micro-simulation subroutine that represents drivers' behaviour under the influence of the dilemma zone based on empirical data collected from signalised intersections in and around Greater Manchester. Modelling such behaviour may be associated with understanding and reducing the problems of red light violations and tailgating behaviour. In order to develop this model, it is necessary to:

1. Conduct a comprehensive literature review that describes the problem of dilemma zone and modelling techniques in order to produce a real representation of drivers' behaviour and compliance at signal-controlled junctions.
2. Design an appropriate methodology to collect data from major/minor crossroad junctions operated by Fixed-Time (FT) and Vehicle-Actuated time (VA) traffic light signals.
3. Determine the factors that affect drivers' behaviour approaching traffic light junctions such as distance to the stopline, drivers' responses to the signal change, type of traffic light control, intergreen period and width of intersection.
4. Develop a micro-simulation model by using a visual programming language such as FORmula TRANslating System (FORTRAN-95) that provides a graphical user interface. The CARSIM model established by Benekohal (1986) has been adopted in this study. This model provides car-following algorithms that represent vehicles' approaching behaviour in the green phase. Then, a new sub-model has been developed based on the dilemma zone theory established by Gazis et al. (1960) to govern drivers' STOP/GO decisions after the onset of amber.
5. Verify and calibrate the developed model with real data collected from the selected survey sites for this study.
6. Conduct an appropriate validation of the developed model by using other datasets different to that used in the calibration process.
7. Utilise the developed model to test the effect of length of intergreen period and different traffic scenarios such as testing the junction performance and design intervention. In addition, evaluation of junction safety can be conducted by testing various safety measures such as Time To Collision (TTC).

The new micro-simulation model is capable of taking into consideration some limitations in the existing micro-simulation software and adding modifications as required. Such a model will reflect the circumstances of vehicles approaching traffic light signals and predict drivers'

STOP/GO decisions following the onset of amber based on empirical data collected from the survey sites.

1.4 Scope of the study

To achieve the objectives of this study, it was found necessary to conduct the study in a number of stages. The stages are described in eight chapters of this thesis. A brief description of the contents of each chapter is presented in Table 1.1. This Table shows the logic that is followed in the compilation of this study. In addition, a brief summary of the contents of each chapter is presented.

Table 1.1: Scope of the study

Chapter	Scope
One	Introduces a background to the RTAs at signalised intersections, the research significance, the key questions and the objectives of this research.
Two	Reviews previous studies regarding modelling drivers' behaviour near junctions controlled by signal settings, design standards of intersection and signal installations.
Three	Presents the data collection method using video cameras, selection of survey sites, and data abstraction and processing.
Four	Presents the analysis of the real observed data using different statistical approaches in order to determine the best data representation.
Five	Provides full details of the micro-simulation model development process and describes the adopted car-following rules and traffic light operation systems.
Six	Describes the verification, calibration and validation processes of the modified car-following model using different data sets from previous studies and the different visited sites.
Seven	Provides the applications of the newly developed model.
Eight	Presents the conclusions for this work and recommendations for future research.

Figure 1.1 shows the structure of the chapters which corresponds to the development of the current study. As illustrated in Figure 1.1, it can be seen that two main rules (i.e. normal car-following and dilemma zone) have been developed to govern drivers' responses to the signal change near junctions controlled by traffic light settings. The calibration of these rules was achieved by using different data sources (i.e. from previous studies and visited sites). Following the development process, the calibration and validation of the entire developed model have

been achieved by utilising different empirical data collected from the survey sites operated by FT and VA traffic light signal settings.

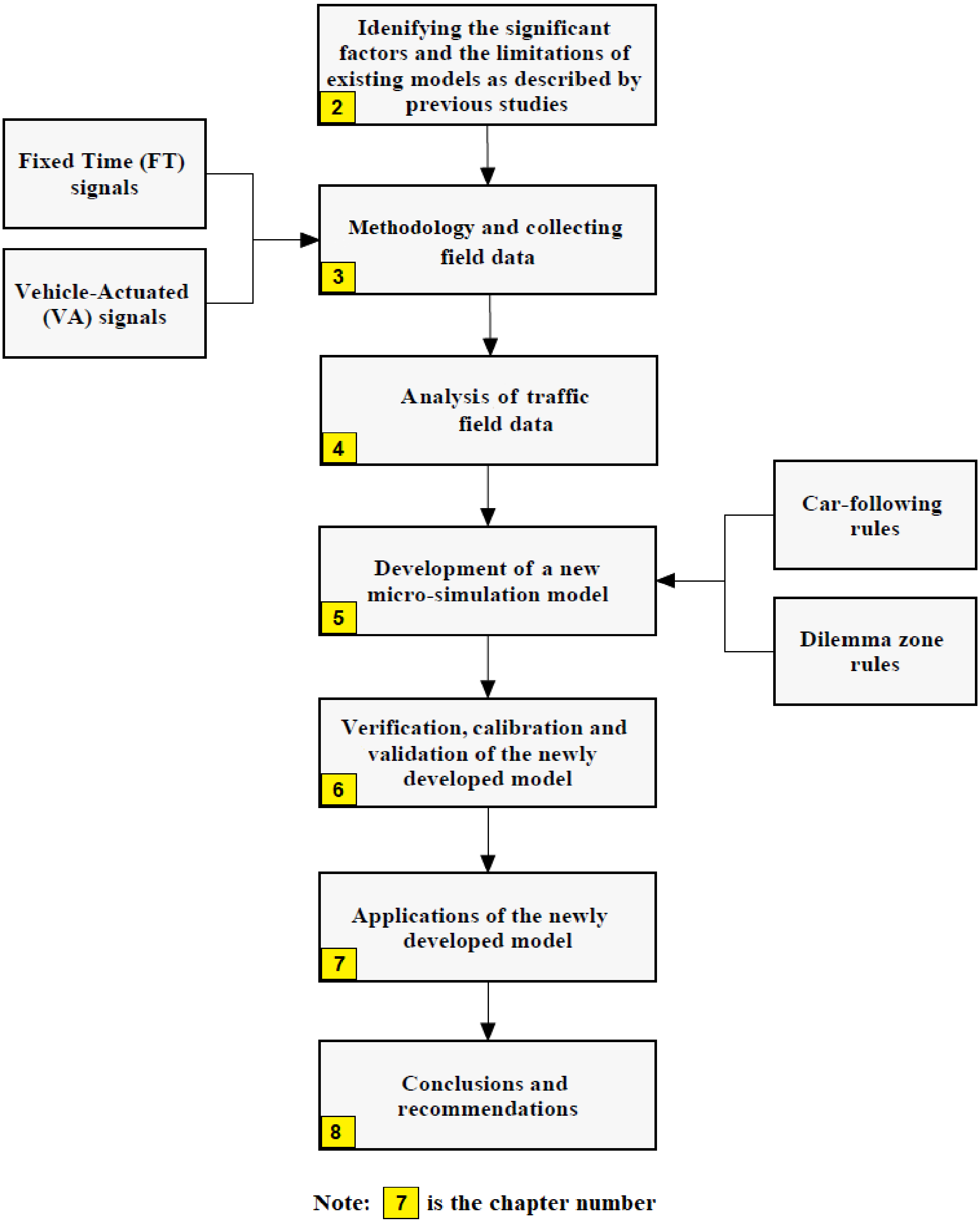


Figure 1.1: Flow chart of the current study

CHAPTER TWO: REVIEW OF LITERATURE

2.1 Introduction

This chapter summarises the literature review of traffic accidents and road safety at signalised intersections. In addition, it includes the description of installation of traffic signal and design aspects, as well as the driver response to the amber/red signals and the modelling techniques, which have been discussed.

2.2 The history of road accidents research

On the 17th August 1896, Mrs Bridget Driscoll (44 years old) was the first victim of a car accident in the UK. She was killed during the exhibition ride in London when a fast motor car swerved suddenly. After that, accidents carried on happening. According to Palutikof (2003), the first statistics of reported road accidents were collected in 1909 at national level in the UK including around 100,000 registered vehicles and more than 1000 fatal accidents. The accident data were published annually between 1909 and 1938 by the Ministry of Transport including the frequencies of fatalities and injuries. Until 1925, Smeed (1949) reported that the number of deaths can be predicted by multiplying the factor of 1.028 by the number of fatal accidents for future estimations. In addition, it was found that the ratio of the number of injuries to the number of injury accidents was equivalent to 1.17. Following that, curves were created to provide the relationships between the number of causalities, the number of population, and the number of registered cars in the UK. These figures were the beginning of studies of road accidents and enabled researchers and interested people in road safety to put forward their plans and strategies to reduce the number of fatal accidents and thereby the injuries as much as possible. It is worth mentioning here that the trend of the ratio of the number of fatal accidents to 1000 registered vehicles declined from 9.0 in 1909 to 2.1 in 1937, and then to 1.4 in 1947 (Smeed, 1949).

Because of the unavailability of the details of accident data, there was a necessity to establish a satisfactory system for reporting and analysing accidents in the UK. Therefore, Smeed (1949) suggested to create an accidents database in order to gain useful background information from the available records and then to find out the effective solutions that must be taken into consideration in the diagnosis of the major factors associated with these accidents to reduce the cost of road accidents as much as possible. The modern input data system was established in 1949 using STATS19 forms. These forms have been modified overtime and have been updated to include different information about accident details, such as accident severity, road type, junction details and weather and lighting conditions. More details regarding each vehicle involved in an accident have been addressed, for example vehicle type, vehicle location at the time of accident, vehicle registration, movement direction and vehicle manoeuvre. In addition, these forms include details of each casualty in the accident such as age, gender and type of casualties such as driver, passenger or pedestrian. Moreover, information about other contributory factors includes human errors such as violating traffic signal and driver impaired by alcohol or drugs. Additionally, vehicles defects include defective brakes, tyres or mirrors. Slippery roads and defective traffic signals are added as examples of environmental factors. Typical forms are as shown in Appendix A.

The STATS19 data are reported by the police who pass it to the Department for Transport every month after checking and validating, then it is added to the national recording system. Every three months, the Department for Transport publishes a bulletin describing the vehicle, pedestrian and cyclist accidents data. Currently, the annual report is published online on the website of the UK government including a comprehensive analysis of the number of accidents and causalities for the current and previous years (Department for Transport, 2017).

In recent years and according to the annual statistics of the World Health Organization (2015), about 1.25 million people are killed on the roads, nearly 50% of those are vulnerable road users. In addition, these accidents cost European countries between 2% and 5% of their gross domestic product (GDP). In the UK, it is estimated that the average value of preventing one injury accident on built-up roads is approximately £60,000 per year, based on statistical data published by the Department for Transport (2015).

According to recorded accidents by the Department for Transport, a significant decrease in fatal accidents was shown between 2003 and 2010, as shown in Figure 2.1. This is because of a number of strategies and measures which were adopted suggesting appropriate solutions and learning from the experiences of other countries such as France, Norway, Netherlands and Sweden in order to reduce causalities and save lives for road users (Department for Transport, 2007). For example, improving the safety of vehicle, providing Intelligent Transport Systems (ITS) as an enforcement and safety features to manage road networks, improving driving style by trainings for drivers/riders, improving public transport by buses and trains/trams to tackle congestion problems and reduce fuel consumptions and finally, considering a number of laws and legislations regarding drinking alcohol and wearing seatbelts for drivers, helmets for cyclists and child restraints in cars.

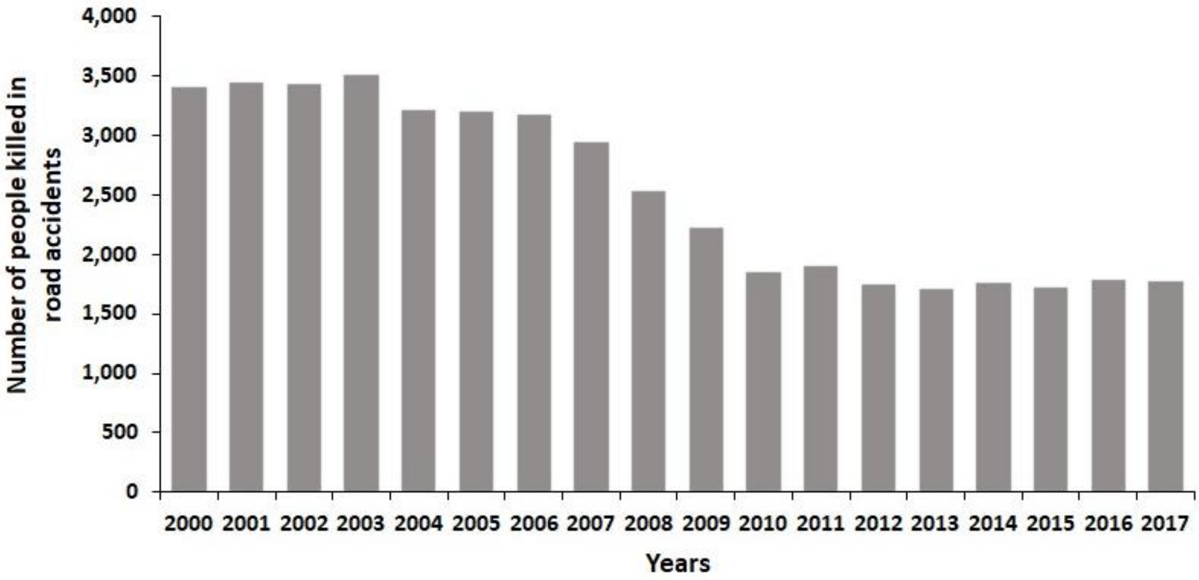


Figure 2.1: Deaths in reported accidents during the period from 2000 to 2017 in Great Britain according to the records of the Department for Transport

2.3 The geometric design and operation of traffic signal-controlled junctions

An intersection is a location where two or more roads cross or meet, the drivers' decisions may pose a big challenge because of crossing, merging and diverging manoeuvres as well as interacting with other road users movements such as pedestrians and cyclists. This location considers a high risk of accidents occurrence without taking into consideration the provision of traffic light signals and road traffic regulations or priority rules.

The main objectives of a signalised intersection design are to minimise or reduce the severity of potential conflicts, improve the intersection performance and give priority to specific road users. To achieve the above objectives, a properly designed approach should include a sufficient number of lanes (with the allowable speed limit) to serve the traffic flow.

2.3.1 Installation of traffic light signals

A traffic light signal is designed to minimise the delays and to control interaction between road users in the intersection area. According to Salter and Hounsell (1989), traffic lights were first designed in the UK in 1868 and it was manually lit by town gas and then was operated automatically in 1926. Traffic signals were established in the US in 1913 to control highway traffic by using the current format of red, amber, and green lamps, however it was believed to be installed and operated manually in 1918 (Salter and Hounsell, 1989). In the installation of traffic light signals, the following factors should be considered: vehicular and pedestrian volumes, design speed, delay, cost, and accident records (Department for Transport, 2004).

The capacity of an approach is affected by vehicles' types and sizes which have different road space requirements and different effects on the capacity of intersections. In addition, it is measured independently based on the traffic flow conditions and is expressed as the saturation flow. According to Salter and Hounsell (1989), the saturation flow is measured in equivalent passenger car units and represents the highest flow that can cross the stopline when there is a continuous green signal aspect and a continuous vehicles' queue on the approach. Also, the geometry of approach can affect the saturation flow such as lane position, lane width, radius of turning movements and gradient.

The traffic light system can be operated by Fixed Time signals (FT). Calculation details of the maximum length of cycle time for a fixed time signal sequence and the length of effective green for an intersection are presented by Salter and Hounsell (1989). On the other hand, Vehicle Actuated signals (VA) with the aid of vehicle detection methods such as loop detectors are used. These techniques provide an extension in the green time to reduce delay and increase intersection capacity. Design procedures of the VA signal are introduced by Mathew (2014). According to the UK Standards (2016), the sequence of traffic light signal or signal phase can be indicated as follows:

- Green interval applies for traffic flow crossing the stopline.
- Amber or Ready-to-Stop interval usually sets as 3 seconds.
- Red interval applies for stopped condition including all-red period, and
- Red-Amber or Ready-to-Go interval usually sets as 2 seconds.

Another significant interval is referred to as the Intergreen period that can be defined as the time between the end of the green time of a traffic flow and the beginning of the green time of another conflicting traffic as shown in Figure 2.2. This period has been achieved in order to ensure that the intersection area is cleared before the movement of conflicting vehicles for entering, turning and clearing (Salter and Hounsell, 1989). According to the Department for Transport (2006a), the minimum intergreen interval in the UK is 5 seconds (which is consisted of 3 seconds amber after green and 2 seconds before the next).

As illustrated in Figure 2.2, the intergreen interval includes all-red period that used to ensure the intersection area is clear before the conflict flows starting movement after the onset of green. In practice, the Department for Transport (2006c) proposed a method to estimate the length of intergreen period by assigning the maximum relative distance travelled to potential conflict points in the intersection area for each traffic flow path as illustrated in Figure 2.3. By comparing the distances of vehicles losing right of way and those gaining right of way, the intergreen period can be found from the table in Figure 2.3.

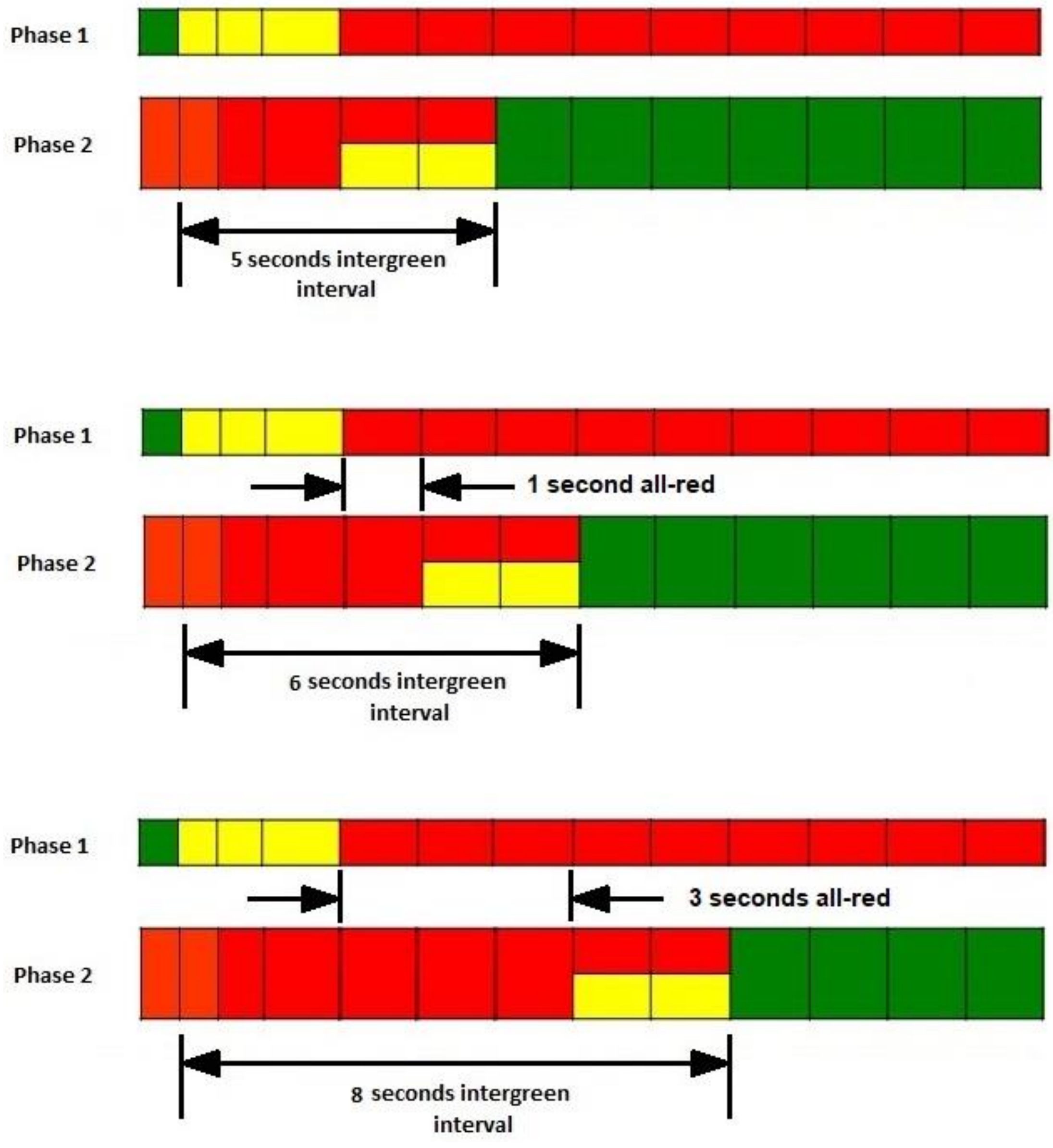
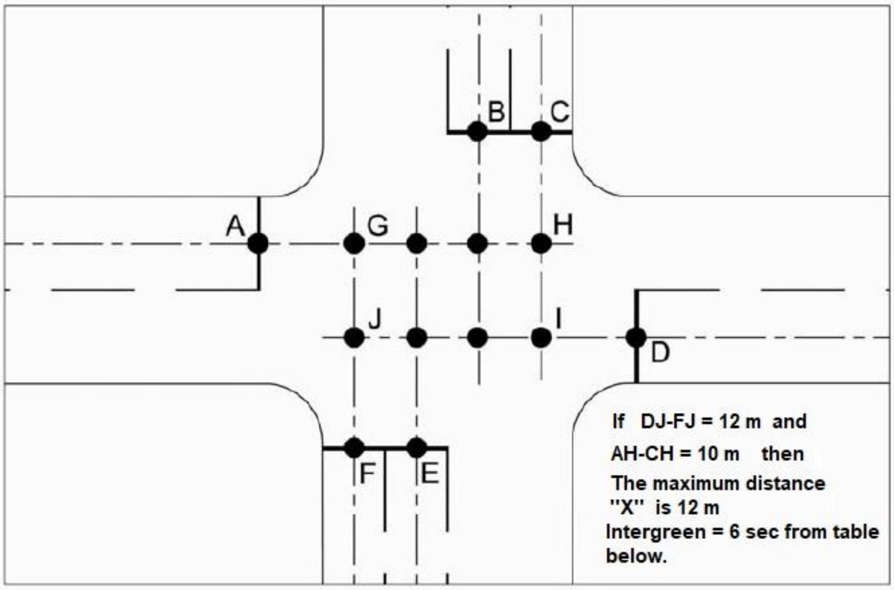


Figure 2.2: Examples of two-phase traffic signal indicating intergreen periods (Adapted from Salter and Hounsell (1989))



Distance "X" (m)	9	10-18	19-27	28-37	38-46	47-55	56-64	65-73
Intergreen (sec)	5	6	7	8	9	10	11	12

Figure 2.3: Estimation of the intergreen period for a typical 4-arms signalised junction (Adapted from Department for Transport (2006c))

2.3.2 The geometric design of a signalised junction

For any approach, the Department for Transport (2004) recommends to install at least 2 signals (primary and secondary) as illustrated in Figure 2.4. In addition, the distance between the secondary signal and the stopline should be 50m maximum. Moreover, the turning movement should be controlled by a separate signal. Another significant issue is providing pedestrian facilities such as puffin crossings and refuges to separate their movement and to avoid possible collisions. Additionally, the Department for Transport (2004) recommends to provide advanced stopline for cyclists to make them more visible to drivers.

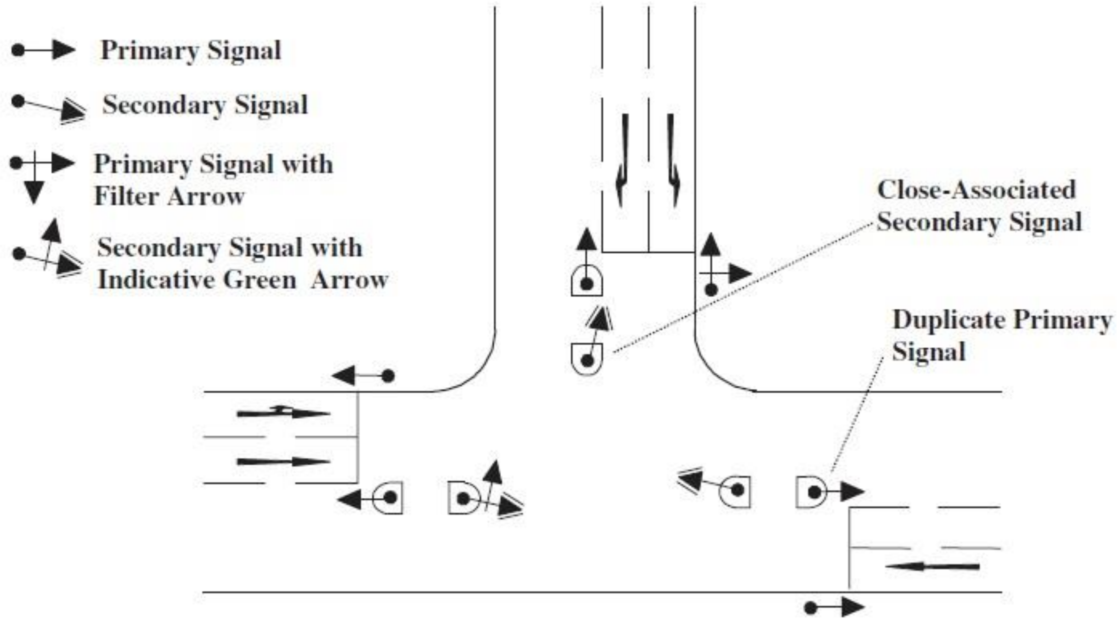


Figure 2.4: Typical design of a signalised junction (Department for Transport, 2004).

2.4 Reported accidents at signalised junctions

Intersections are the most researched places to study and identify the risky factors affecting traffic movement. This is because of the interaction between different road users, particularly near or at the centre of an intersection, which may pose hazards to all road users and reduce the safety level. Kennedy and Sexton (2009) revealed that 19% of all accidents in London occurred at signalised intersections. They argued that although the installation of traffic light reduces the number of right-angle crashes, the problem of rear shunts has worsened. Another investigation from the US related to junction-related accidents (787,236 events) was conducted by Choi (2010) who reported that approximately 53% of these accidents occurred at traffic signal junctions in comparison to other non-controlled junctions, as illustrated in Figure 2.5. These compare with about 37% and 24% of total accidents occurred at signalised junctions in Auckland and Melbourne, respectively (Turner et al., 2012).

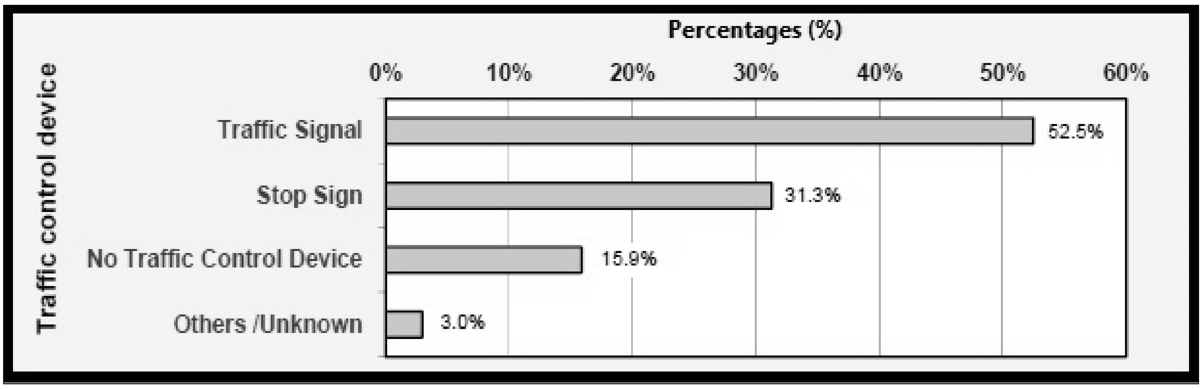


Figure 2.5: Percentages of reported intersection-related accidents (Choi, 2010).

More specifically, many researchers have investigated driver compliance with the traffic light signal in association with what is known as the dilemma zone. Most frequent accidents due to hesitating drivers can be limited to rear-end collisions and red light violations. According to Retting and Kyrychenko (2002), red light violations caused 260,000 crashes each year in the US including 750 fatalities and severe injuries as well as property damage. Porter and England (2000) highlighted that 35.2% of observed traffic light cycles (1,798 out of 5,112 observations collected from 6 traffic-controlled junctions at three cities in the US) included at least one red light runner prior to the onset of opposing traffic. In addition, around 33% of the total reported crashes were rear-end collisions (National Highway Traffic Safety Administration, 2015). Many researchers have confirmed that rear-end collisions at urban signal junctions occur by two successive drivers who make conflicting decisions when the amber light comes on (Abdel-Aty and Keller, 2005; Baguley and Ray, 1989; and Kennedy and Sexton, 2009).

In Greater Manchester, details of accidents occurred at signalised intersections are obtained from the police reports. A total of 5855 accidents were reported between 2009 and 2014 using the STATS19 forms. Rear-end collisions were the majority of all accident types (by 26.5%) followed by principle or right turning accidents (18%) and red light violations (9%), as shown in Figure 2.6. In addition, human error was a major factor in around 76% of these events such as disobeying traffic signals (33%), exceeding the speed limit (16%), failing to look properly (13%), following too close (9%) and failing to judge other vehicles' paths or speeds (5%). Whereas, only 24% of the reported accidents were caused by vehicle defects, traffic light faults, weather and road conditions.

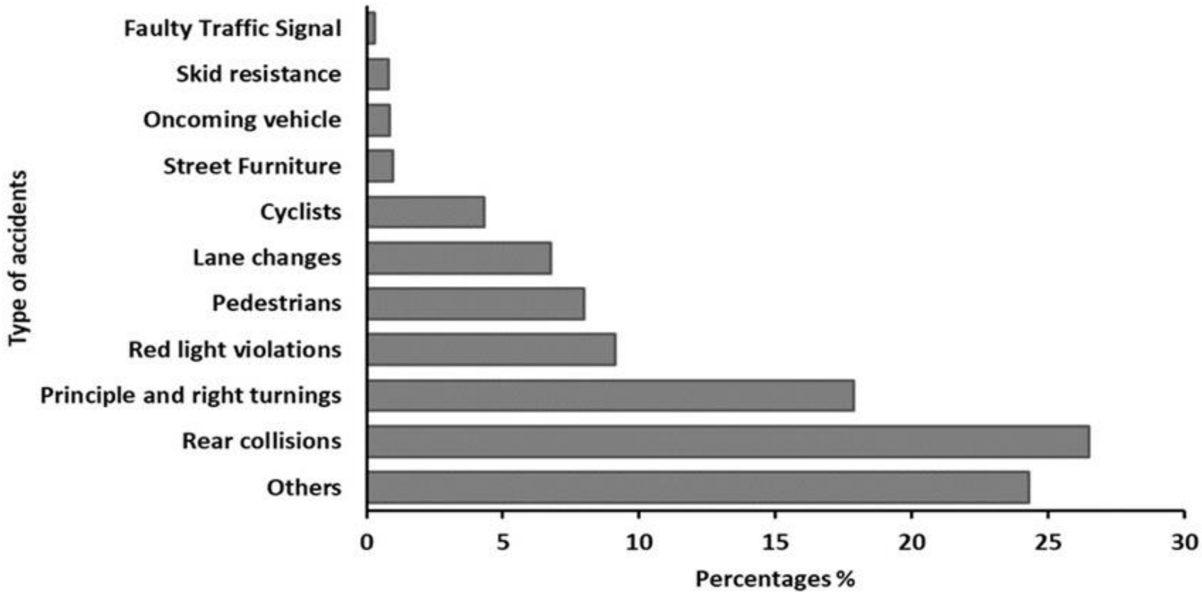


Figure 2.6: Types of traffic accidents at signalised intersections in Greater Manchester (reported by police over the period from 2009 to 2014).

Finally, it can be concluded that rear-end collisions and signal violations constitute about 35% of the total reported accidents in Greater Manchester. Such traffic conflicts remain critical issues not only due to vehicles' defects and faulty traffic signals but also because of human behaviour. The next section will discuss the psychology studies that are concerned with driver behaviour.

2.5 The problem of dilemma zone

2.5.1 Background

Amber aspect (which is often of a 3 seconds duration) is provided in order to help drivers clear the intersection before the conflicting traffic stream starts its movement. Many studies have focused on a critical area called the 'Dilemma Zone' upstream from the intersection approach during the amber period. A driver approaching the dilemma zone has to make a decision either to stop or proceed through an intersection area before the onset of red. His/her decision is made based on several factors including distance from the stopline, travelling speed, driver reaction time and intersection geometry. According to Kennedy and Sexton (2009), a driver's incorrect decision at the dilemma zone is a more risky behaviour because he/she might decelerate suddenly resulting in a rear-end collision with the close following vehicle or proceed through the red and cause a collision with the conflicting vehicle at the centre of the intersection.

The following sub-sections discuss some of the previous studies that define the dilemma zone boundaries and how this problem affects drivers' decisions. In addition, strategies that have been implemented to reduce its influence will be discussed later.

2.5.2 Definition of the dilemma zone boundaries

There are several methods of identifying the dilemma zone. The first method was established by Gazis et al. (1960) who was the first researcher that suggested the definition of the "amber problem". As shown in Figure 2.7, in the Stop Zone (SZ), drivers have enough time and distance to stop safely by starting to decelerate before the onset of red. In the Go Zone (GZ), drivers are able to cross the stopline because they are close enough to the stopline to cross before the red signal appears. However, if the Stop Zone does not meet the Go Zone, there is a critical area of the carriageway called the Dilemma Zone (DZ) where the driver can neither clear the intersection nor stop safely before the stopline (Gazis et al., 1960).

Just before the onset of red, some hesitating drivers do not have a sufficient distance for safe stopping without applying sudden braking and they might start tailgating other vehicles. In addition, they cannot clear the intersection resulting in signal violations and possibly severe right-angle accidents at the centre of the intersection. Conversely, if the Stop Zone overlaps the Go Zone, no Dilemma Zone is confronted and the driver has the option either to proceed through the intersection or stop safely. The critical zone, in this case, is called the Option Zone (OZ) as shown in Figure 2.7.

Since 1960, addressing the DZ has been one of the traffic safety challenges for researchers. The geometric boundaries of the SZ, GZ, and DZ were firstly drawn by Gazis et al. (1960) at the General Motor Corporation Laboratories. Based on Figure 2.7, they established the Gazis-Herman-Maradudin (GHM) model that represents definitions of the minimum required distance for comfortable stopping and the minimum required distance to clear the junction before the red light comes on, as follows:

$SSD = V_n(R_t) + \frac{V_n^2}{2g(e \mp f)}$, or it can be written as $V_n(R_t) + \frac{V_n^2}{2MADR_n}$ *Equation 2.1*

$CD = V_n(amb) - (w + L_v)$ *Equation 2.2*

$DZ = SSD - CD$ *Equation 2.3*

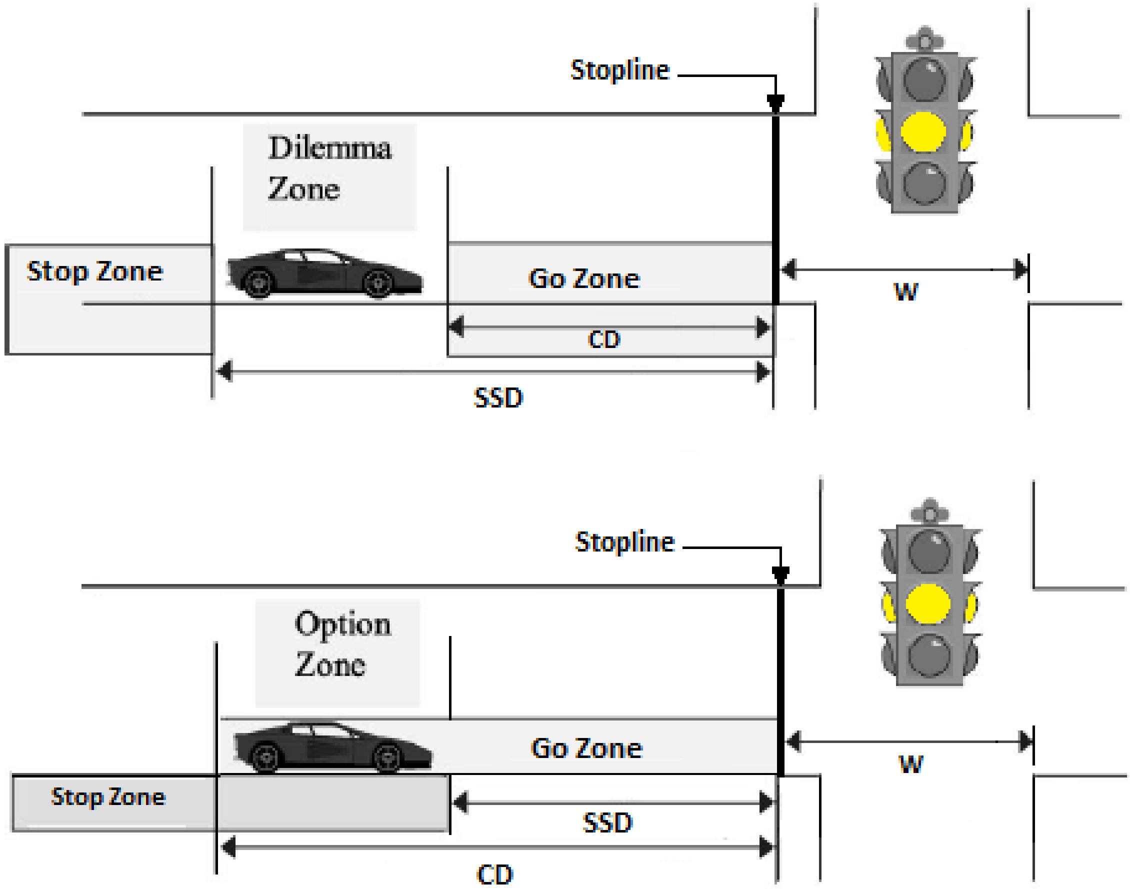


Figure 2.7: Illustration of Dilemma and Option Zones at a signalised junction (Adapted from Moon and Coleman III (2003))

where,

V_n is the speed of the vehicle *n* measured in m/sec.

$MADR_n$ is the maximum available deceleration rate of the vehicle *n* measured in m/sec^2.

R_t is the driver reaction time measured in sec.

amb is the amber traffic light period (usually 3 sec).

w is the width of an intersection measured in m.

L_v is the length of vehicle measured in m.

e is the road gradient in percent.

f is the coefficient of friction in percent.

g is the gravity acceleration measured in m/sec^2.

SSD and CD are the safe stopping distance and clearance distance to the stopline measured in m, respectively.

In addition, the deceleration rate required in order to bring the vehicle to a stop safely before the stopline as a function of distance and amber period can be found as follows:

$$Deceleartion\ rate\ for\ stopping = \frac{0.5\ V_n^2}{V_n(R_t) - Pos_{nSL}} \qquad \textit{Equation 2.4}$$

where, Pos_{nSL} is the position of vehicle *n* from the stopline at the start of amber.

If CD is less than SSD, the driver will be under the effect of the DZ at the onset of the amber and cannot stop comfortably or clear the junction safely. However, if CD is greater than SSD, the driver has an option either to proceed or stop before the onset of red. Hence, if the vehicle position from the stopline is greater than SSD, it will be in the SZ. Additionally, the vehicle will cross and clear the junction area if it is on a distance shorter than CD to the stopline. In their study, Li et al. (2010) investigated the length of DZ with various amber intervals. They found that the length of DZ decreases with the decrease of vehicle speed, and increases with the increase of the length of amber. This is evidence that lengthening the amber light period might increase drivers' indecision as discussed by York and Al-Katib (2000).

Moon and Coleman III (2003) introduced the term 'Dynamic Dilemma Zone'. They proposed that various values of vehicles' speeds, drivers' perception-reaction times and vehicle lengths can be applied in the GHM model established previously by Gazis et al. (1960). In addition, the rate of acceleration/deceleration will be different within each different input parameters of vehicle performance in the model. The new model reflects simplicity and reality in determining the dynamic DZ limits which have been agreed by many researchers. Moreover, Liu et al. (2007) investigated dynamic DZ based on empirical data collected from six signalised intersections in Maryland-USA. The findings were showing different dilemma zones with various driver behaviour groups and the longer amber period showed no significant benefits in reducing severities.

The second approach has been adopted by other researchers based on empirical data including a large sample of vehicles approaching the stopline at signalised intersections (Herman et al., 1963; Olson and Rothery, 1961; Parsonson et al., 1974; and Zegeer, 1977). They referred to the 'Indecision zone' for identifying the dilemma zone in their studies because they considered various drivers' decisions that determine their behaviour. The researchers analysed the proportion of stopping in relation to the distance from the stopline at the start of amber. They agreed that the DZ is located upstream from the stopline and typically represents the zone between the point where 90% of drivers stop comfortably and the distance covered by the 3 seconds amber (which is sufficient for vehicles to clear the junction before the onset of red). As illustrated in Figure 2.8, the researchers suggested alternative dilemma zone boundaries based on their own surveys (Baguley and Ray, 1989). For example, if a driver moves with a constant speed (for example 90 kph) on the approach and is at point X (74 m from the stopline) when the signal is showing amber, he/she will pass through the junction within no more than 3 seconds. If this driver is at point Y (115 m from the stopline), he/she will be able to stop safely before the onset of red. However between these limits, the driver will experience the dilemma zone and either he/she brakes heavily towards the stopline or continues crossing with accelerating and running the red light.

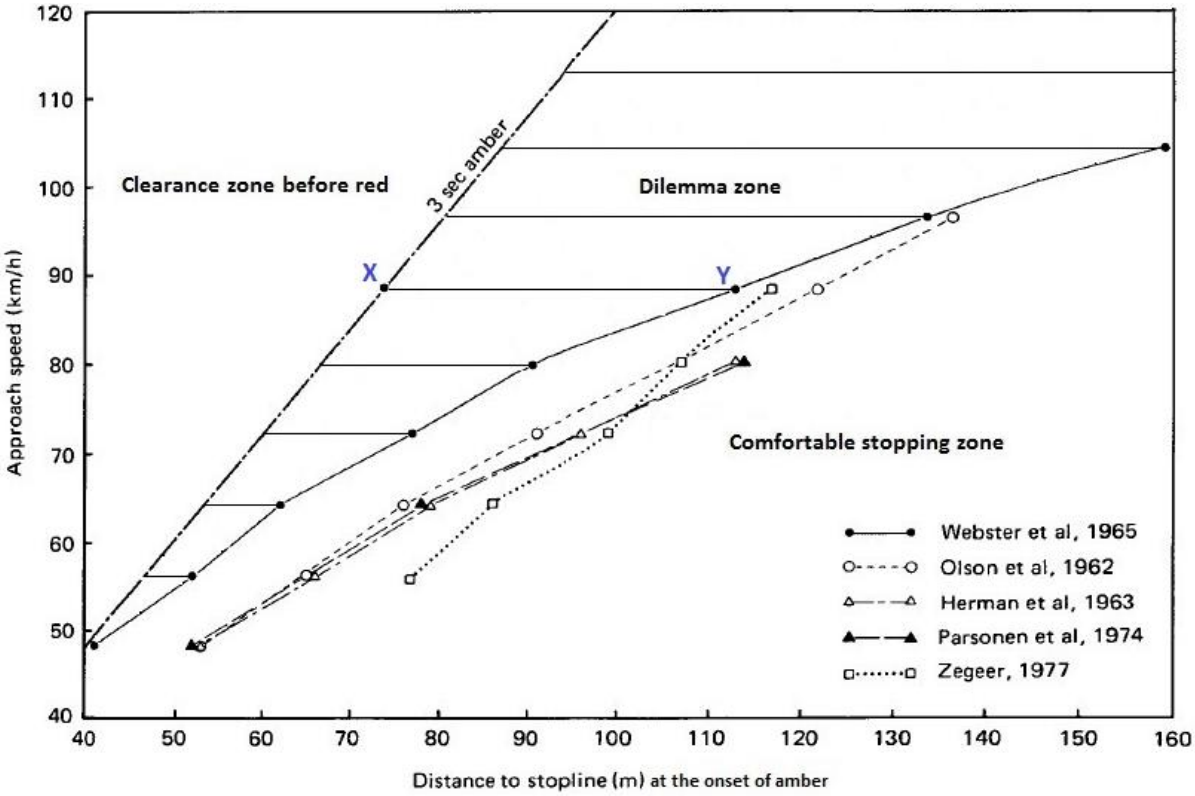


Figure 2.8: Dilemma zone boundaries investigated by different researchers (adapted from Baguley and Ray (1989)).

Other studies were carried out to develop the dilemma zone boundary over a wide range of approaching speeds. They defined the DZ as the distance from the stopline where 10% and 90% of drivers are able to stop, as shown in Figure 2.9. Table 2.1 presents the observed dilemma zone that has been reported by various researchers. It can be shown that the dilemma zone (between 10% and 90% of stopping) increases dramatically as the approaching speed increases.

Table 2.1: Observed dilemma zone boundaries in terms of distance from the stopline (measured in metre) based on the probability of stopping (Zhang et al., 2014).

Approach speed		Olson and Rothery (1961)		Herman et al. (1963)		Webster et al. (1965)		ITE (1974)		Zegeer and Deen (1978)		Chang et al. (1985)		Bonneson et al. (1994)		Maxwell and Wood (2006)	
kph	**mph**	**10%**	**90%**	**10%**	**90%**	**10%**	**90%**	**10%**	**90%**	**10%**	**90%**	**10%**	**90%**	**10%**	**90%**	**10%**	**90%**
48	**30**	---	---	---	---	---	---	---	---	---	---	---	---	---	---	37.0	73.0
56	**35**	31.40	64.63	30.48	66.46	31.40	51.83	32.01	64.63	31.40	77.44	39.02	87.80	44.21	94.69	43.0	74.0
64	**40**	33.53	77.74	33.53	79.27	38.11	62.50	33.53	76.22	36.89	86.28	44.82	93.59	54.27	89.33	47.0	79.0
72	**45**	50.30	96.04	50.30	96.04	47.25	76.83	50.30	91.46	46.34	99.08	50.61	99.39	64.44	104.5	50.0	9.03
80	**50**	67.07	114.3	67.07	112.8	56.40	91.46	67.07	106.7	51.83	106.7	56.40	105.2	76.83	120.7	61.0	102.0
88	**55**	---	---	---	---	70.12	112.8	73.17	121.9	70.73	117.1	62.19	110.9	89.63	137.8	62.0	106.0

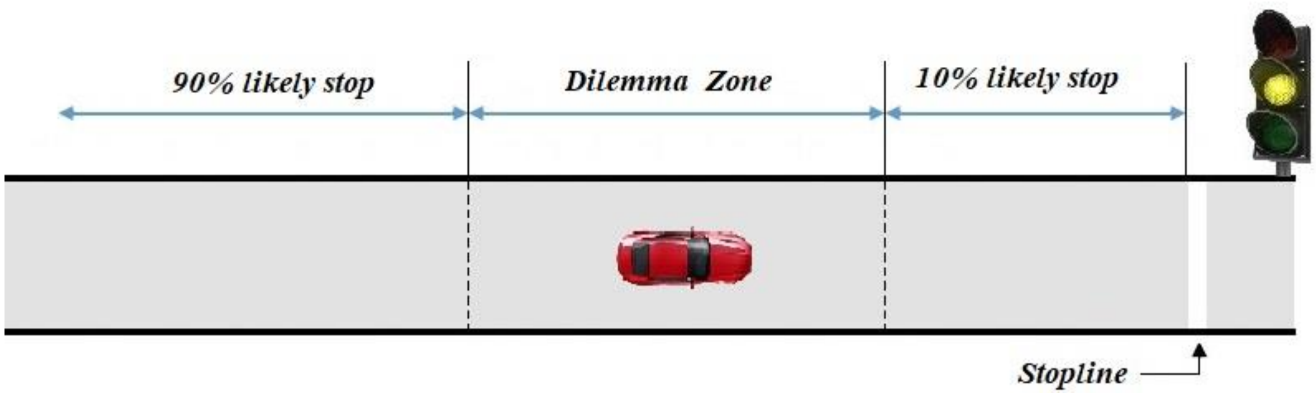


Figure 2.9: Illustration of dilemma zone ranging between 10% and 90% of stopping

On the other hand, many researchers conducted several studies to define the boundaries of DZ by measuring the time to the stopline as shown in Table 2.2. Bonneson et al. (2001) demonstrated that this is related to the proportion of vehicles that stopped after the onset of amber, travelling speed, traffic flow fluctuation and roadway conditions. In their research, they stated that a shorter amber period and high flow rate increases the number of traffic signal violations. As a result, this leads to an increase in the probability of right-angle accidents.

Table 2.2: Observed time to the stopline dilemma zone after the onset of amber.

Researcher's name	Time to the stopline in second after the onset of amber
Parsonson et al. (1974)	2.5 – 5.0
Zegeer (1977)	2.0 – 5.0
Chang et al. (1985)	3.0 – 6.0
Bonneson et al. (1994)	3.0 – 6.0
Bonneson et al. (2001)	2.5 – 5.5
Gates et al. (2007)	2.90 – 5.0 for passenger cars 3.7 – 5.7 for heavy good vehicles
Rakha et al. (2007) and El-Shawarby et al. (2007)	1.85 – 3.90 for ages < 40 years old 1.50 – 3.20 for ages > 70 years old
Rakha et al. (2008)	2.87 – 4.90 for younger ages 1.66 – 4.81 for ages > 65 years old

Finally, the GHM model can represent drivers' behaviour approaching the stopline following the onset of amber. Equations 2.1 to 2.3 can be used to allocate the related zone (i.e. SZ, GZ, OZ or DZ) including different values of vehicles' speeds and lengths, drivers' reaction times, intersection width and amber period. Then, a driver's decision whether to proceed or stop can be predicted. The deceleration rate for stopping condition following the onset of amber can be determined from Equation 2.4.

2.5.3 Strategies to reduce the effect of dilemma zone

Many researchers have indicated that the safety level of any intersection is increased by improving traffic signal design and the geometric design of road elements (Kennedy and Sexton, 2009; Khisty and Lall, 1998; Roess et al., 2004; and Salter and Hounsell, 1989). Basically, three main elements must be taken into consideration to improve the safety issue: the road, the driver and the vehicle. However, traffic engineers have direct control to tackle risks on one of these elements (which is the road) by providing routine maintenance to the installed signals in order to ensure that they remain working consistently within the design specifications. In addition, others can play an important role in improving driving skill by a series of educational methods and licensing procedures and develop safety equipment's to prevent road traffic accidents and reduce the number of injuries in future.

Although the safety level has been improved by signalisation, the problems of traffic conflicts and delay have increased. More specifically, red light violations and rear-end collisions are largely related to the problem of dilemma zone that leads to an increase in the level of severity. Different traffic control measures including traffic control devices and engineering strategies have been implemented on streets and highways in order to facilitate both vehicular and pedestrian movements, minimise the risk of dilemma zone, and improve the performance during rush hours particularly at controlled junctions. Examples of these strategies are listed as follows:

2.5.3.1 The MOVA strategy

The strategy of Microprocessor Optimised Vehicle Actuation (MOVA) was established in the 1980's by the Transport Research Laboratory (TRL) to control traffic flow fluctuations at any signalised junction. Figure 2.10 illustrates the design of MOVA strategy by installing inductive loop detectors in the pavement to detect the number of vehicles that are passing a certain point and extend the green phase at traffic congestion conditions (Department for Transport, 1997).

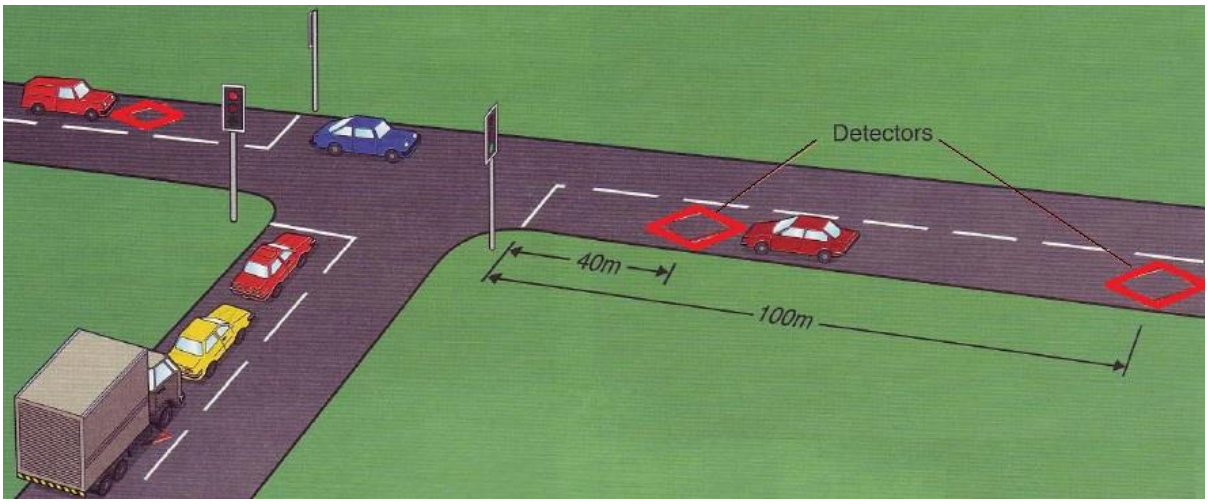


Figure 2.10: Detectors arrangement for MOVA at a traffic signal junction (Department for Transport, 1997)

The main benefits of MOVA as reported by the Department for Transport (1997) are the reductions in injury accidents on excessive speeding approaches, and reduction in the red light running violations up to 50,000 vehicles per junction per year. In addition, more outcomes are remarkable reductions in delays (by 13%) and maximising junction throughputs particularly at junctions controlled by alternative strategies with congestion on one or more approaches. Another evaluation study was carried out by Kennedy and Sexton (2009) to see the effect of the installation of MOVA on the safety issue. The study showed that junction collisions were reduced by 26% after the conversion to MOVA. Despite the fact that other traffic solutions have been widely used such as vehicle actuated signal control, MOVA proves to be the only strategy which is having a significant impact on traffic safety.

2.5.3.2 Enforcement strategy using red light cameras

Red Light Cameras (RLC) have a great impact in reducing the danger of red light violations at signal-controlled junctions. Red light cameras are connected with the traffic light signal system to record the traffic flow and detect any vehicle's number plate passing over the sensors after the onset of red. In California, these enforcement tools were found to be very effective and had resulted in a reduction in angle crashes between 17% and 32% as well as a 40% drop in the number of RLR, as reported by Retting and Kyrychenko (2002). Similar research was conducted by Retting et al. (2008) to investigate the impact of the amber length on drivers' behaviour at six approaches to two signalised junctions controlled by RLC in Philadelphia City. The results showed that the number of RLR drivers was reduced by 96% after installing RLC and extending the amber length 1 sec extra. Another study from the Netherlands was conducted by Goldenbeld and Schagen (2005) who investigated the effect of speed enforcement on speed levels and accidents events by using mobile radar. Over five years of this enforcement project, the results revealed a significant decrease in the frequency of serious and injury accidents by an average of 21% and a reduction in the percentage of speed limit violators by 12%.

2.5.3.3 The length of intergreen period

Another essential aspect was indicated to reduce the influence of dilemma zone is the length of intergreen time. Kennedy and Sexton (2009) highlighted that a longer intergreen time causes an increase in RLR events. It also reduces the right turning accidents by providing sufficient time to clear the intersection area before the conflict flows start their movements. Maxwell and Wood (2006) investigated the influence of various amber periods on safety and capacity of signalised junctions (A range of 30-55 mph speed limit) in the UK. They demonstrated that an increase in the length of amber by an extra 1 second increases the speed level as well as driver behaviour becomes more aggressive particularly at lower speed junctions. More specifically, York and Al-Katib (2000) stated that a shorter amber time (<3 sec) increases the RLR while longer amber (>3 sec) might increase the hesitancy of drivers then they will enter the intersection area too late. The researchers recommended keeping the amber interval at 3 seconds for all signalised intersections.

2.5.3.4 Usage of traffic signals countdown display

A number of studies have discussed alternative methods to improve safety level and reduce the number of conflicts such as the installation of Green Signal Countdown Display (GSCD) and flashing green devices (usually 3-5 second before the onset of amber) at signalised intersections. In both techniques, the GSCD can be set between the green and amber to warn drivers when the green will change to amber accurately. However, drivers have no idea when to change to amber in case the signal setting operated with flashing green. A comparative before and after study was conducted by Lum and Halim (2006) who reported that GSCD devices could help in reducing the frequency of red light running by 65% and the number of stops increased significantly by 6.2 times during the amber period after 45 days of installation. However, the number of RLR returned to be higher over time particularly under high flow conditions.

On the other hand, Wei and Jia-jun (2015) investigated the influence of flashing green on drivers' STOP/GO decisions at signalised junctions in China. The researchers revealed that flashing green affects positively the intersection safety. However, it works effectively with strict enforcement such as early warning signs or speed controls from a sufficient distance from the stopline. Moreover, Factor et al. (2012) studied driver knowledge, self-reported behaviour and attitudes towards the flashing green. They argued that flashing green does not help drivers in making STOP/GO decisions, also it does not increase the level of safety at signalised intersections. In addition, Köll et al. (2004) illustrated that the flashing green may contribute to an increase in the early stopping which perhaps leads to a rear-end collision. This happens when the following driver did not know whether the leading driver would decide to cross or stop. Further discussions of modelling drivers' behaviour research and making a STOP/GO decision in the dilemma zone will be explained in the following section.

2.6 Review of drivers' psychology and modelling research

As the human behaviour is such a significant feature in the majority of safety problems, it is necessary to present the history of traffic psychology studies and factors influencing the drivers' behaviour as well as modelling methods.

2.6.1 Background

In the earlier years of motorisation in the 20th century, more focus was considered to the drivers and the traffic law enforcement. However, earlier research focused on accidents circumstances and basic analysis of accident statistics (OECD, 1997). After the end of World War II in 1945, a lot of attention was paid to psychology research particularly where it related to road safety, because of the high trend of road accidents and the increasing number of casualties. After that, a considerable interest was given to describe the drivers' perceptive behaviour by many researchers because some drivers have difficulties in coping with the road environment.

In the 1970's and 1980's, drivers' research were becoming more established. Two groups of researchers were recognised at these periods: the first was psychologists groups, mainly coming from Europe, focused on human psychology that drivers are able to behave according to the traffic situation and hence they try to face the level of risk. The second was road safety researchers, many of them from the US, concerned with a certain group of drivers that could be mainly responsible to an expected number of accidents. Their perspectives shifted towards development of vehicles' industry and mechanical development (Hakkert and Gitelman, 2014).

Since the 1990's, several studies were also dealing with traffic psychology, however they focused on the experience of drivers and their ability to resume control in different driving conditions. Summaries and examples of several aforementioned studies are listed in Table 2.3.

In the traffic psychology field, drivers have to negotiate with other road users as well as with the road environment. Making a decision can be divided into three types depending upon understanding and information of drivers as follows (Wu et al., 2009):

- Certain decision: refers to a structured decision that has been made under certain conditions and its result can be expected.
- Risky decision: refers to a human decision that has been made on the basis of suggesting the probability that an event will occur in the future under certain conditions.
- Uncertain decision: refers to a decision that has been made without forecasting the probability that an event will occur in the future under various conditions.

Table 2.3 Development of traffic psychology research

Periods	Summary of research	Examples
Earlier years until the end of World War II in 1945	A lot of research were introduced based on the assumption that some drivers caused accidents by chance because of their personality. However, these studies did not take into consideration the effects of other road users or road environment that had contributed to these traffic accidents. The solutions were attempted in traffic legislation, punishment and re-education.	Forbes (1922), Farmer (1925), and Farmer and Chambers (1929)
1950's – 1960's	These studies considered drivers as victims. Theories were introduced based on driver perceptual behaviour as a measure of causing an accident. They discussed the inability of drivers to analyse and process traffic environment complexity, such as traffic signs.	Taylor (1964), Fitts and Posner (1967) and Rumar (1985)
1970's – 1980's	Researchers discussed that drivers have chosen their levels of subjective risk based on their adaptions to the traffic situations. They realised that some safety measures (such as wearing seat belts) might result in more risky behaviour (such as speeding), which may lead to an accident.	Fahner and Hane (1973) and Naatanen and Summala (1976)
1980's – 1990's	Studies were conducted and resulted in a classification of drivers' behaviour based on their skills and knowledge levels under different driving tasks. Researches developed forms of questionnaire including 50 items to discuss aberrant behaviour of drivers. They were designed to illustrate how this behaviour was 'bad' and 'dangerous' to other road users. The outcomes can be classified into 3 levels: violations, errors and lapses. These studies are widely described under the title of *The Driver Behaviour Questionnaire (DBQ).*	Rasmussen (1980), Reason et al. (1990), Parker et al. (1995), Lawton et al. (1997), and Sucha et al. (2014)
1990's and over	These studies have focused on the fact that drivers are affected by other road users and behave within a social context. In addition, these studies take into consideration peoples' opinion, evaluation, and perceived behavioural control regarding drivers' behaviour (negative or positive). A number of applications of this research can be found under the field of *The Theory of Planned Behaviour (TPB).*	Ajzen (1991), Elliott et al. (2004) Elliott et al. (2007) and Li et al. (2016)

Therefore, the decision made by a driver at signal-controlled junctions belongs to the risky decisions because he/she is unable to predict the traffic circumstances at the intersection, for instance the movement of the following and preceding vehicles particularly after the traffic light signal showing an amber indication. Drivers have a tendency to pass through the junction without stopping. However, incorrect decisions before assessing the pros and cons of crossing during the 3 seconds amber may increase the risks of accidents like rear-end collisions or right angle accidents with vehicles moving in the conflict flow (Wu et al., 2009).

According to Garber and Hoel (2009), the driver perception-reaction process to a stimulus can be summarised as follows: perception, identification, handling and reaction. At traffic light junctions, the driver perception happens when observing the amber indication. Following that, the driver will understand or identify the meaning of amber aspect which is '*Continue to cross only if unable to stop safely*'. Then, the handling process starts when a driver analyses the surrounding circumstances including junction layout and movement of other road users and decides what plan to take in response to the amber before executing it. For example, if a driver plans to cross the stopline, he/she has three choices or plans: either accelerating, maintaining speed with the preceding and following vehicles or decelerating. Based on which plan decided, the comparison of both merits and demerits of each plan can identify the implementation process to achieve the objectives. Finally, the driver will execute the plan decided in the handling stage (Wu et al., 2009). Figure 2.11 shows the process of driver's decision-making at signalised junctions at the onset of amber.

2.6.2 Factors affecting drivers' behaviour

Since drivers are the fundamental component affecting the road network systems, studies have shown that behavioural change of drivers has important impacts. Based on real accident statistics, Wetteland and Lundebye (1997) conducted a comparison study between the UK and US accidents to identify the most significant contributory factors. They found that human errors were a major factor in 73% of crashes in the UK. This percent was 6% higher than that of the US. Factors that influence drivers' behaviour can be categorised as in the following sub-sections.

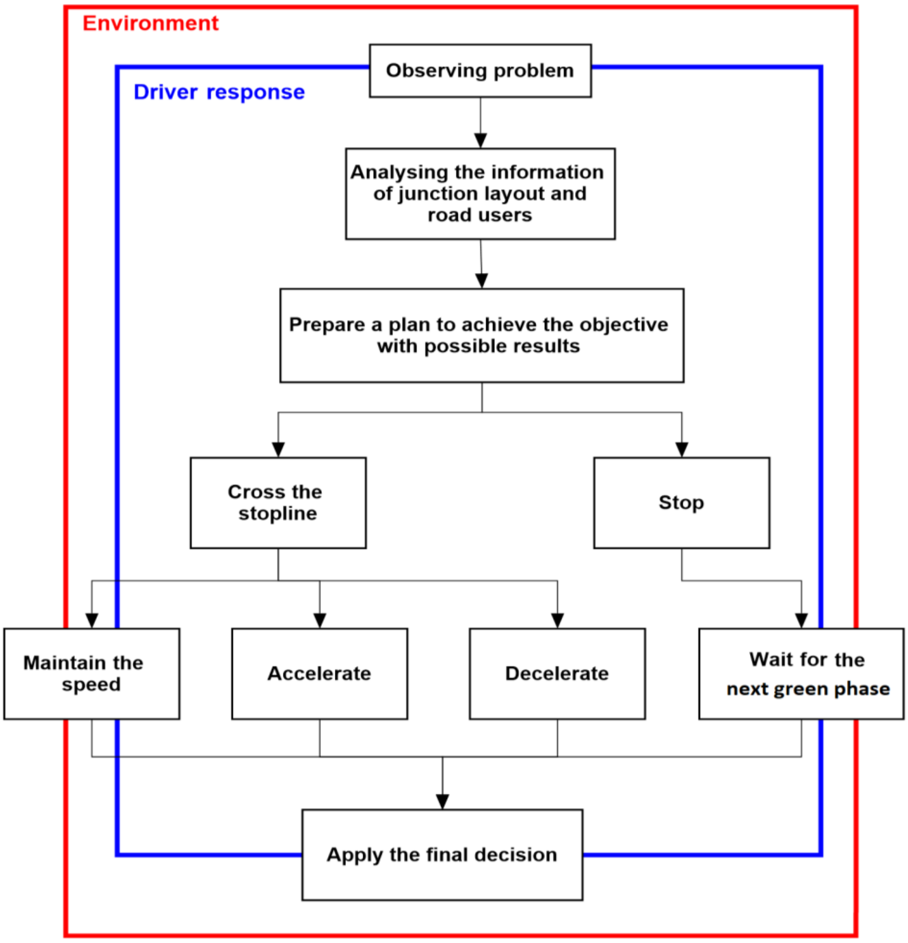


Figure 2.11: Strategy of driver's decision-making at a traffic signal-controlled junction (adapted from Wu et al. (2009)).

2.6.2.1 Human factors

According to the National Cooperative Highway Research Program (2003), aggressive drivers are more likely to accelerate near signalised junctions. The majority of accidents were caused by aggressive driving. Aggressive behaviour can be summarised as: following too close to others, braking instantly, overtaking other vehicles or lanes, preventing passage of other vehicles and violating speed limit or changing speed too suddenly for the conditions. In addition, traffic signal violation indicates the noncompliance behaviour of drivers to the traffic signal leading to an increase in the risk of accidents.

Perception-reaction time (R_t) is another significant factor affecting driver behaviour near signalised intersections. According to Gazis et al. (1960), it is a major factor that affects the DZ boundaries and it can be defined as the time travelled after the amber onset but before the driver applies the brakes. Different factors can influence driver R_t such as vehicle type, travelling speed, driver gender and age. In addition, the time to the stopline and road grade affect significantly on driver R_t (Rakha et al., 2007). Moreover, an increase in amber interval may lead to greater R_t values (York and Al-Katib, 2000). Finally, a high travelling speed was found to result in lower values of R_t (Gates et al., 2007, Rakha et al., 2008). This is reasonable since the mean value of R_t might be influenced by the vehicle platooning situations (i.e. the position of vehicle in the queue: leader, follower or drive alone) under different traffic flow conditions. More specifically, the leader's R_t was found to be shorter than other situations (Gates et al., 2007, Rakha et al., 2008).

Investigating the psychology of driver behaviour may lead to identifying possible factors associating with speeding. Wang et al. (2009) insisted that there is a positive relationship between the risk of accidents and driving with a speed more than the allowable speed limit in England. According to Silcock et al. (1999), one of these factors is related to drivers' culture because they consider exceeding the speed limit is an enjoyable and exciting experience. The second is drivers who have suffered from work pressure and stress. Moreover, some drivers have a tendency to increase their speed because of positive and negative emotions or because others start tailgating them. Additionally, the purpose of the trip and journey time have contributed positively in exceeding the speed limit. Silcock et al. (1999) stated that 85% of drivers found themselves exceeding the prevailing speed limit, and 98% of them drive with illegal speeds at least once during one hour of driving.

El-Shawarby et al. (2007) concluded that age and gender are important factors affecting driver's decision in the dilemma zone. Their investigation included the impact of distance to the stopline at the onset of amber on driving behaviour. They found that male drivers are more likely to run through the amber than females. Also, elderly people over the age of 65 are more careful to stop compared with other age groups because of driver-learning and experience (Retting and Williams, 1996). It was found that the performance and experience of novice young drivers have been improved in lane keeping, but difficulties have been noticed in tailgating with other vehicles (Summala et al., 1998).

Two surveys were conducted by Lawton et al. (1997), involving drivers between 17 and 70 years of age living in Manchester and Reading to define various types of violations and identify the contributory factors. The outcomes of the first survey, based on Driver Behaviour Questionnaires DBQ (Parker et al., 1995), found that left-turn manoeuvres and misjudging the speed of oncoming vehicles are the most common drivers' errors. Following that, overtaking movements and intentional red-running violations increase the risk of incidents. Finally, offensive driving due to aggressive behaviour and involving in race or chase were considered major reasons for collision occurrence. In the second survey, Lawton et al. (1997) suggested to increasing focus on motivational factors that affect drivers' decisions and result in traffic violations. The results of this survey highlighted that speeding, misjudging other drivers' manoeuvres and hostility are the most common reasons of making a decision of violation.

Another factor related to driver behaviour is the drink-driving which is a contributory cause of accidents. Statistics have shown that the numbers of injury crashes dropped by 5-16% after reducing the limit of alcohol concentration in the blood from 0.08% to 0.05% in Australia, France and Netherlands and from 0.05% to 0.02% in Sweden (Fell and Voas, 2006). Additionally, an average of 300 people are killed in 10% of total accidents on major roads every year because of driver fatigue and sleeping at the wheel in the UK (Ladyman et al., 2007). On the other hand, mobile communication during driving influences positively on the driver performance and increases the risk of involving in an accident. Brookhuis et al. (1991) claimed that the probability of accident might increase during communication when the driver of the following vehicle cannot avoid the collision with the preceding vehicle that decelerated for stopping during the amber period.

Finally, a view was revealed by Kimber (2005) who suggested to use the term 'driver' instead of 'road system' for most of the theories in the area of road safety and accident studies. In other words, the interactions between the following and preceding vehicles (the striking and struck vehicles) should be investigated more to find out the probability of road accidents and thereby to identify the level of safety on the road. Therefore, special knowledge is required and more investigations regarding driver behaviour and his/her decisions, particularly after the onset of amber.

2.6.2.2 Other contributory factors

A driver's decision and response cannot only be influenced by the driver him/herself but also there are other influential factors including vehicle characteristics and junction layout. The effect of vehicle characteristics (such as size and type) on driver's decision in the DZ was investigated by Sayer et al. (2003), Papaioannou (2007) and Gates and Noyce (2010). They realized that a driver tries to keep a greater following distance with the heavy good vehicles in order to avoid possible collision. This is because they seem to be as obstacle objects that reduce the forward views particularly at sudden changes in speed.

Junction layout such as junction width, number of lanes, lane position and grade are recognized as factors associated with making a decision. Papaioannou (2007) and Yan et al. (2007) stated that an increase in the number of lanes leads to an increase in the probability of signal violations. Finally, the existence of red camera enforcement, pavement marking, location of road signs and signals can improve the safety performance and driver visibility as well as avoiding possible conflicts. According to Goh and Wong (2004), driver response to the signal change seems to be similar at signalised intersections with and without red light camera. However, the frequency of RLR was lower at junctions supplied with the camera enforcement compared to others.

Additionally, driver behaviour can be also influenced by weather and climatic conditions. The results of a questionnaire survey carried out by Kilpelainen and Summala (2007) showed that drivers are likely to be more careful and show lower driving performance during the winter than other seasons particularly in rural areas. Also, the findings indicated that driver behaviour is mostly affected by the prevailing weather conditions instead of traffic-weather forecasts.

2.6.3 Modelling driver's decision when approaching a signalised junction

Various methods of modelling drivers' behaviour have focused on modelling his/her response to the signal change in the dilemma zone. Different mathematical models have been proposed to represent the probability of driver' choice (whether to stop or proceed through the junction) based on data collected from sites including influential factors.

One of the earlier studies was conducted by Olson and Rothery (1961). They attempted to model the probability of stopping as a function of distance to the stopline. Chang et al. (1985) examined the distance and time to the stopline as well as speed, as factors affecting the percentage of stops and passing vehicles. A modelling method, using data collected from a driving simulator was carried out by Caird et al. (2007) to investigate driver behaviour, including the influence of the amber period and the driver age. Their conclusion was that the driver's STOP/GO decision depends on the time to the stopline, taking into consideration his/her response time. However, the age classification showed no significant differences with respect to response times.

In addition, Gates et al. (2007) used logistic regression analysis to explore the effect of approaching speeds and applied decelerations on the probability of stopping. As a result, it was found that drivers have a tendency to cross with a short time to the stopline and with a longer amber period; other factors were noted, such as whether the vehicle was a bus or heavy truck, the absence of a bus or cycle lane and turning movement lane. Elmitiny et al. (2010) applied classification tree models to predict driver's STOP/GO decision under the following situations:

distance to the stopline, vehicle's speed and position in the flow (leading or following). They predicted a RLR model which is strongly dependent on the distance to the stopline (by 99%), position in the platoon (by 25%), speed (by 20%), vehicle type (by 10%), and finally lane position (by 1%) according to the parameter importance.

Wu et al. (2013) developed a driver's decision model by using a binary logistical regression method. For the purpose of their study, loop detectors were located at the stopline and several hundred feet upstream to collect high resolution and signal event data. They attempted to predict driver's STOP/GO decision and RLR events including the effect of the amber period and vehicles' details (e.g. speed, number of amber running and number of red running). The accuracy of the predicted model was 87%. Elhenawy et al. (2015) proposed a measure of the driver aggressiveness (varies from 0= not aggressive to 1= very aggressive) to be included in the model of driver STOP/GO behaviour. Generalized linear models were applied to model the driver response by using historical observations of drivers who decide to proceed when the time to the stopline is greater than the amber interval or travelling at speeds that exceed the speed limit. The authors demonstrated that the accuracy of the model increased significantly after adding the developed driver aggressiveness predictor.

A panel data random parameters probit model was developed by Savolainen et al. (2016) to focus on the influence of camera enforcement and warning flashers on a driver's STOP/GO decision. The result of their study showed that driver behaviour is more affected by the existence of warning flashers or camera enforcement. In addition, drivers were more likely to stop at road junctions with lower speed limits and longer intersection widths as well as if pedestrian facilities were present. Similar study was conducted by Wu et al. (2009) revealed that driver's decision is affected by the speed and position in the traffic when there is no countdown units at the junction.

On the other hand, fuzzy inference rules were developed by Kikuchi et al. (1993) for modelling driver's STOP/GO decision based on empirical data. They estimated the degree of anxiety for conservative and aggressive drivers. Another research related to the dilemma zone was carried out by Lin and Kuo (2001). They focused on developing a procedure to estimate the change of a traffic signal and clearance periods using a rule-based fuzzy logic system. Moreover, Tanga et al. (2016) applied a fuzzy approach and binary logic model to investigate the effect of a 3 seconds flashing green (before the 3 seconds amber) on the driver decision-making process, at high speed intersections (in this case the speed limit was 80 kph). They concluded that the results obtained from the fuzzy model were better and more consistent than those produced from the binary logic model. The researchers revealed that the findings can be used to improve the driver STOP/GO decision model in the microscopic simulation software, signal design and dilemma zone protection strategy.

Finally, the dilemma zone still remains a big challenge to all road safety researchers. Despite the fact that many studies have been carried out to find the best prediction model of driver response to the signal change. In addition, the statistical models cannot reflect the situation of interaction between successive vehicles and cannot indicate the effect of changes in the transport system after installing new technologies since this is very complex. More specifically, these models cannot include different drivers' and vehicles' characteristics as in the real world. Therefore, more investigation and micro-simulation research are needed particularly that define the factors affecting drivers' decisions in the dilemma zone. Then, a new microscopic model can be introduced that is more realistic and shows more logic and is capable of representing dynamics of driver behaviour.

2.7 Simulation approaches

2.7.1 Introduction

Broad studies have been carried out using traffic simulation techniques because of limitations of accident data with regard to quality and time required to record. In addition, elimination of cost and risk factors are required to help the simulation model users in the evaluation of different strategies that improve traffic safety and road network performance by choosing the best and effective solution.

Traffic simulation models have been developed coinciding with the computer software development in recent decades. According to the Institute of Transportation Engineering (2010), traffic simulation models can be divided, based on the levels of detail and fidelity, into the following common classes:

- **Macroscopic simulation models:** they give a description of interaction between various activities of traffic characteristics which follow the traffic rules. However, this type of simulation models presents low level of detail; it is used to investigate the fundamental relationships between flow, density and speed and analyse the travel demand in a certain zone in the road network.
- **Mesoscopic simulation models:** in these models, the traffic components develop from the analogy of flow to dynamic mode. The model deals with traffic demand as individual

packets located at a certain point in the road network and analyse their travel dynamics such as vehicle platoon. This type of simulation models presents mixed level of detail from macro and microscopic models, however higher than macroscopic model and lower than microscopic models since the mutual interaction between successive vehicles is not considered in these models.

- **Microscopic simulation models:** they study individual traffic components in the road network, such as road user behaviour and vehicle dynamics. These models are helpful to examine changes over a small area in the transportation system such as effect of using Intelligent Transport Systems (ITS), as well as to solve traffic problems in a particular road section, such as congestion and shockwaves. This type of simulation models presents high level of detail and requirements to calibrate and validate the model results.

The microscopic simulation technique has been adopted in the current study since the microscopic models include sub-models to represent the following behaviour of successive vehicles and other relevant parameters of traffic movement, such as, vehicle and driver characteristics. However, lane changing and overtaking behaviour have not been adopted in this study because of rare observations caused by these types of behaviour particularly near signal-controlled junctions. Finally, the microscopic simulation processes provide a better understanding of the factors influencing traffic light violations and tailgating conflicts, as they provide a closer representation of the reality.

2.7.2 Autonomous vehicle as an example of advanced simulation technologies

Since this work is arising from road safety, it is necessary to highlight the link between the development of micro-simulation models and how can these models be applied to alleviate traffic conflict as much as possible. Autonomous vehicle (AV) is one of the developed simulation tools and is commonly known as self-driving or driverless vehicle. It travels by a combination of video cameras, radar, sensors and global position systems (GPS) without any control from the drivers.

Because of the rapid progress in the Intelligent Transport Systems (ITS), these vehicles will be introduced in the nearest future after getting approval of AV driving regulations. Since 2010, partial automated vehicles are provided by advanced safety features such as advanced driver assistance capabilities, vehicle-to-vehicle communications and automatic emergency brakes to avoid collisions. However, fully automated vehicles are not legal and available on roads yet.

They are now under developing and testing its performance with other road users to be adapted over road network under different traffic circumstances.

Developers have worked on many challenges in the field of artificial intelligence in order to improve AV learning algorithms in order to be capable to make complex decisions (like a human) such as changing lane or stopping for emergency vehicles to pass and what to do in case of an accident ahead, roadworks or crossing pedestrian or animal. One of the biggest challenges is programing a vehicle to react to the traffic light changing particularly from green to amber. This could be referred to the problem of the Dilemma Zone (DZ). The developer has to program the vehicle either to STOP or GO after recognizing the amber light. Several details have to be defined such as distance to the stopline, distance and speeds of vehicles ahead and behind as well as what acceleration will be used to proceed or deceleration rate to stop?.

A recent study conducted by Brown et al. (2018) focused on patterns in drivers' performance when driving surrounding AV under the effect of DZ (where 10% to 90% of drivers would stop according to previous studies listed in Table 2.1) after showing the amber light. The experimental data are resulted from scenarios created on Unity 3D platform presented through a virtual reality environment headset. These data include information regarding human response and vehicle dynamics. The simulation outcomes showed that rear-end collisions are caused by incorrect driving behaviour following the onset of amber which highlights the risk of DZ. In addition, drivers have a tendency to increase their following distance with AV that indicating low comfort levels when driving behind AV. Finally, the researchers recommended to introduce accurate driving algorithms of AV before being adapted with other road users.

Under common circumstances, simulation machines are only the way to test all the potential conflicts using different scenarios as in the real world. Understanding the car-following rules and enhancing making-decision algorithms may lead to reduce the risk of accidents and causalities as well as assess the safety issues in their implementation. In addition, the roads occupied by autonomous vehicles would be less congestion. The next sections describe the car-following models and highlight the shortcomings of the existing micro-simulation softwares in order to develop a new micro-simulation model including junction geometric, humans' and vehicles' factors influencing driver's decision who approaches a traffic signal junction.

2.7.3 Car-following models

Car-following rules have been proposed to represent the longitudinal progress of vehicles in a traffic stream. For the purpose of road safety and performance, car-following models are frequently used to understand the driving task which is a result of mutual interaction between successive vehicles and different road features (Brackstone and McDonald, 1999).

Different car-following models have been applied in various micro-simulation software packages in order to analyse complex situations that cannot be studied accurately by other analytical measures such as traffic congestion problems, installation of new intelligent transport techniques and to simulate different accident conflicts. According to Olstam and Tapani (2004), car-following models can be divided into groups based on the utilized logic. A brief description of each group can be found in Table 2.4.

For the purpose of model development, the CAR-following SIMulation model (CARSIM); established by Benekohal (1986), has been used for simulating traffic flow in both free-flow and congestion conditions in the current study. Previous microscopic simulation works of Yousif (1993), Al-Jameel (2012), Alterawi (2014), and Nassrullah (2016) developed the CARSIM model to evaluate traffic characteristics (e.g. capacity and delay) in different road and highway features such as roadworks, weaving, and merge sections on the motorways. The main advantages of using CARSIM model can be recognised as it is capable of handling different types of vehicles and maintaining their speeds by applying a desired acceleration and deceleration rates as well as starting their movements to their desired speeds after stopping as in real world situations. In addition to that, it provides a safe following distance for each vehicle in the system by applying a safe deceleration rate to avoid collision with the leading vehicle if the latter reduces its speed (Benekohal, 1986).

However, Benekohal (1986) did not take into consideration the dilemma zone rules (i.e. the Gazis et al. (1960) model as described in Section 2.5.2) affecting driver's STOP/GO decisions near junctions controlled by Fixed-Time (FT) or Vehicle-Actuated (VA) traffic light signals. The model should include a special algorithm that is able to predict the driver's decision following the onset of amber. Therefore, several parameters are needed to add to the model to develop the braking and crossing behaviour before the driver is approaching the stopline at a signal-controlled junction. These parameters can be recognized as the length of amber and intergreen periods, distance from the stopline at the onset of amber, minimum safe stopping distance, clearance distance and intersection width.

Table 2.4: Classes of car-following models

Researcher's name and year	Established model	Basic idea	Advantages	Disadvantages	Example of simulation models
Established by General Motors research laboratories in 1958 and used and developed by other researchers (see Brackstone and McDonald (1999) for more details)	Stimulus-Response or GHR models	A driver acceleration has a proportional representation with the relative speed.	• The simulation models show a good correlation with the data collected from the field based on General Motors car-following models. • General Motors car-following model can be used as a starting point to derive other mathematical relationships, for example speed-flow and speed-density relationships in Greenberg's logarithmic models.	• No significant association between the models' parameters and the features of drivers and vehicles. • No response when two successive vehicles moving with the same speed and when the acceleration is equal to zero.	MITSIM
Kometani and Sasaki (1958) and then improved by Gipps (1981) and Benekohal (1986)	Collision avoidance models	To prevent crash occurrence by controlling a safe distance between two vehicles.	Widely used in simulation models since this model was able to mimic the traffic flow and driving behaviour in the real world.	There is a difficulty in examining safe headway and braking situations due to drivers' STOP/GO decisions at signalised intersections.	CARSIM and AIMSUN
Used by several researchers (see Brackstone and McDonald (1999) for more details)	Fuzzy logic-base model	It is based on the GHR models and introduces a desire following distance.	The model was developed to mimic and predict the instantaneous reaction of a Driver-Vehicle-Unit (DVU).	Its difficulty is the lack of confidence in specifying the 'membership' of the functions.	MITRAM and MISSION

Table 2.4: Classes of car-following models (continued)

Researcher's name and year	Established model	Basic idea	Advantages	Disadvantages	Example of simulation models
Michaels (1963) and then developed by Evans and Rothery (1977)	Psychophysical spacing or action point models	A driver of the following vehicle accelerates or decelerates depending on relative speed of the leading vehicle and the changes in the visual angle.	A driver is able to assess with the variation in distance or relative speed only when this threshold limit is reached.	• There is a negative response bias in the threshold limits which may cause an increase in the spacings between vehicles. • There are problems in calibration and calculation of the individual thresholds.	VISSIM and PARAMICS
Helly (1959) and then developed by other researchers (see Brackstone and McDonald (1999) for more details)	Linear model	A desired following distance was produced in this model depending on the assumption whether the leading vehicle was suddenly braking.	It could be applied when a specific acceleration/deceleration for a situation has been determined.	The main difficulty is in the calibration of the coefficients of its parameters.	SITRA-B

2.7.4 Limitations of current micro-simulation software

Different micro-simulation software have been developed to analyse and evaluate road network performance such as PARAMICS and AIMSUN. They provide full understanding of the differences in individual vehicles behaviour travelling in the road networks depending on three fundamental concepts of dynamic traffic assignment: gap acceptance, lane changing and car-following rules. In addition, different vehicles' and drivers' characteristics such as vehicle types, speed and driver reaction time as well as various road features and layouts such as signalised junction can be applied in order to replicate observed events in the real world.

Despite the fact that all aforementioned micro-simulation packages provide good two- and three-dimensional animation, however different limitations were reported in the software and these can be summarised as follows:

1. The users cannot sometimes access the code of these software packages particularly in the case of checking the complicated steps of corrected behaviour in the system as in a real life, for example traffic incidents.
2. In PARAMICS, the values of maximum acceleration/deceleration rates are the same for all speed limits. The acceleration/deceleration model seems better represented in VISSIM than PARAMICS. On the other hand, both models fail in predicting of the number of conflicts and conflict mechanism at signalised junctions despite the good correlation between the observed and simulated data (Essa and Sayed, 2016).
3. Driver aggressiveness has been considered in PARAMICS only. This factor is very important in assigning the distance headway between vehicles. That means a higher level of aggressiveness will lead to a reduced following distance and then the start of tailgating with the lead vehicle. Lower time headways can be provided in AIMSUN and PARAMICS to replicate the tailgating phenomena (Laagland, 2005).
4. None of the previous industry standard software provides a real representation of the traffic signal violation after the red light comes on. Apparently, there is a lack of published details considering vehicle's position from the stopline, speed, and acceleration before the onset of red light. For example, in PARAMICS two possible cases of signal violation can happen: either if a very aggressive driver does not apply any deceleration rate during the amber period and goes through the red, or if a driver decides to stop but he/she has to wait longer which increases the aggressiveness particularly in congested conditions. In general, running the red signal happens by

increasing the level of aggressiveness to the threshold and that makes the following vehicle go through the red even the first vehicle does not comply with the red light (Laagland, 2005).

Finally, it can be concluded that there are missing or no details on some default parameters, such as driver response to the amber, percentage of driver alertness after the onset of amber, and how can the users modify them under different traffic conditions. Therefore, because of the above shortcomings in the existing micro-simulation models (such as CARSIM), a new contribution will add to the knowledge by developing the car-following rules that will highlight the problem of the dilemma zone. Improvements will include applying the GHM model (developed by Gazis et al. (1960)) with various data of drivers' reaction times, vehicles' characteristics and traffic light operation systems in order to make the new model capable of giving a prediction of driver's STOP/GO decision after the onset of amber at signalised junctions. Hence, the newly developed model will give a prediction of the number of RLR events, as well as give an indication about the tailgating behaviour. Then, appropriate solutions can be suggested to reduce the number of these conflicts and increase the safety level at signalised intersections.

2.7.5 Other related simulation sub-models

2.7.5.2 Importance of time headway

The time headway is one of the surrogate safety measures that plays a significant role in traffic safety and performance. It can be defined as the time gap between the passage of two successive vehicles over a certain point or a reference line on a road section. The mathematical expression can be written as:

$$Time\ headway = T_F - T_L \qquad \textit{Equation 2.5}$$

where T_L and T_F are the time headways (measured in second) of the leading and following vehicles, respectively.

Fairclough et al. (1997) referred to the Driving Standards Agency (1992) recommendation that the allowable following headway with the leading vehicle should not be less than 2 seconds distance. The '2-seconds rule' is necessary for all drivers since it gives an indication of the probability of two or more vehicles involved in a tailgating collision (SWOV, 2012). Tailgating or close-following is risky driving behaviour and can be defined as a time gap equal to or less than 1 second. Previous work conducted by Michael et al. (2000) who used hand-held signs to warn drivers to keep safe distances with others. They revealed that about 57% of drivers were in compliance with the 2-second rule and only 3% followed with a time headway lower than 1 seconds which is generally recognised as ‘tailgating or close-following behaviour’. These figures were lower when the sign was absent and found to be around 50% of drivers had a tendency to comply with the 2-second rule and 7% of them followed too close with other vehicles.

Few studies have been carried out to investigate the most significant factors contributing to rear-end collisions due to close-following behaviour. According to Postans and Wilson (1983), about 23% of drivers have a tendency to maintain a 0.5 second headway particularly in traffic congestion situations. Also, Tlhabano et al. (2013) demonstrated that female drivers showed less tailgating behaviour compared to male drivers.

On the other hand, Hurwitz et al. (2013) investigated driver behaviour approaching signalised intersections in the US and studied the effect of distraction and several factors on the vehicle’s headway using a linear regression method. The study’s finding was that the headway increased by around 5% and 19% particularly in the start-up lost time because of various distractions. Yousif et al. (2014) examined the tailgating behaviour at urban shuttle-lane roadworks operated by portable traffic signals. Their conclusion was that 24% of drivers follow too close and violate the 2-second rule before approaching the roadwork zone, while 38% of drivers start tailgating after crossing the roadwork zone.

Brackstone et al. (2009) mentioned that headway analysis in numerous studies has either not measured or not considered associated independent factors, or has been conducted using simulation techniques where only a simple representation of traffic flow can be provided. Therefore, they suggested four assumptions which may have an essential role in the development of car-following models as follows:

1. The driver behaviour is influenced by traffic flow and density. The higher the flow and density on the road become, the higher the probability of sudden deceleration and the occurrence of rear-end collisions due to short time headways and distance gaps.

2. The following behaviour of a driver differs with road features. For example, the presence of traffic signal control and interaction with other road users, such as pedestrians, may lead to an increase in headways between successive vehicles.

3. The type and physical size of the leading vehicle affects the driver behaviour under the following conditions. A driver follows a truck with a greater time headway and gap than

following a car. However, inverse findings of the research of Parker (1996) and Sayer et al. (2003) presented that trucks were being followed more closely than cars if they travel at the same speed and acceleration.

4. The headway is chosen inconsistently by drivers. The headway may be affected by driver aggressiveness and motivation, as well as having influence as a 'background noise' or hampering the prediction of the model for a driver or group of drivers. Finally, Brackstone et al. (2009) stated that ;*"No previous attempts have been made to establish the magnitude of such effects, which could mask, or at worst be taken as a surrogate for, the effect of other variables".*

The main outcomes of the Brackstone et al. (2009) study were that the time headway can be affected by the vehicle type and size and the variations in traffic flow throughout the day and the week. Another evidence was highlighted by SWOV (2012) and Tlhabano et al. (2013) that the 2-second rule is reduced at peak hours compared to off-peak hours.

Finally, the consistent tailgating behaviour is an interesting evidence of the fact that shorter gaps between successive vehicles increases the likelihood of rear-end collisions. The analysis of headways at signalised intersections needs further investigation in order to find out its impact on the following behaviour and the relationship with possible conflicts, such as, red light violations and rear-end collisions.

2.7.5.2 Surrogate Safety Measures (SSMs)

Various methods have been used to assess traffic safety based on computing the accidents rate in terms of frequencies in relation to different environmental, geometric, demographic and traffic information. For example, the severity index has been used to describe traffic safety in terms of the number of killed or seriously injured per accident. An earlier perspective considered that speed is one of the safety indicators because it plays a key role in the occurrence of collisions (Solomon, 1964). However, Tarko et al. (2009) argued that it is difficult to consider the speed as a measure of traffic safety because crashes are typically represented by number of accidents or frequencies.

Despite the fact that the analysis of traffic safety for a single road or intersection based on historical data (i.e. number of accidents) is not reliable because of the low frequency of such incidents, therefore different surrogate safety measures have been proposed in order to indicate the risk of accidents. The safety indicators are briefly summarised in Table 2.5 with threshold values and mathematical expressions from previous works.

Surrogate Safety Measures (SSMs) are most widely used in traffic conflict analysis. These measures have been built based on the motion characteristics of vehicles. For example, Time To Collision (TTC) is a promising indicator for rating the severity of conflicts particularly the rear-end collisions. TTC was initially proposed by Hayward (1972) and can be defined as the time required for two successive vehicles (on the same path) to collide if they continue at their present speeds. More specifically, Hoffmann and Mortimer (1994) investigated the accuracy of estimation TTC. They explained that TTC was influenced by three variables: relative speed, viewing time and headways between two successive vehicles. The simulated TTC data were introduced in terms of the changes in the spacing between two successive vehicles and the perception of angular velocity of the leading vehicle. The study indicated high possibility of rear-end collision due to poor estimation of TTC, in particular when the times for vehicle deceleration and control action are required.

Vogel (2003) reported that both TTC and headway are independent parameters of each other. The lower values of time headway and TTC indicate an imminent collision between the successive vehicles for deceleration conditions and vice versa. On the other hand, there is another possible case that higher TTC values and lower headways may reflect the probability of potential conflict particularly for vehicles travelling along the approach towards a traffic light signal junction.

Nowadays, Advanced Driver Assistance Systems (ADAS) have been developed by vehicle manufacturers in order to eliminate drivers' errors as much as possible. Effective collision avoidance systems considers one of ADAS that are built to collisions with sufficient time to alert the driver for an instant reaction to avoid the collision (Van der Horst and Hogema, 1993). Different safety measures such as Time To Collision (TTC) and Deceleration Rate to Avoid Collision (DRAC) are suitable parameters for defining collision avoidance and detecting risky driving behaviour. For the purpose of model development in the current work, safety sub-model will include estimations of a number of safety indicators such as measuring the TTC between the successive vehicles following the onset of amber. In conclusion, the sub-model outputs will provide an indication regarding the possibility of conflict occurrence such as RLR event or rear-end collision at traffic signal junctions.

Table 2.5: Traffic safety measures

Indicator	Definition	Mathematical expression	Critical value (sec)	Source
TTC	Time To Collision: is the time that remains until the two vehicles would collide if they remain on the same path and speed difference	$TTC = \frac{Pos_L - Pos_F - L_{vF}}{V_F - V_L}$	5	Vogel (2003)
			4	Hirst and Graham (1997)
			3 – 6	Cavallo and Laurent (1988), Hogema and Janssen (1996)
			1.5 – 5	Van der Horst and Hogema (1993)
HW	Time Headway: is the time gap between the passage of two vehicles over a fixed point or a reference line on a road section on the basis of front wheels	$HW = T_F - T_L$	2	Michael et al. (2000)
DRAC	Deceleration Rate to Avoid Collision: is the rate at which crossing vehicle must decelerate to avoid collision.	$DRAC = \frac{0.5\,(V_F - V_L)^2}{Pos_L - Pos_F - L_{vL}}$	N.A.	Cunto and Saccomanno (2008)

where:

Pos_L and Pos_F are the positions of leading and following vehicles in (m), respectively.
V_L and V_F are the speeds of leading and following vehicles in (m/sec), respectively.
L_{vL} and L_{vF} are the lengths of leading and following vehicles in (m), respectively.
T_L and T_F are the time headways in (sec) of the leading and following vehicles, respectively.

2.8 Summary

The main goal of this chapter was to introduce the literature review of driver response to the traffic light change at traffic signal junctions in order to acquire a better understanding regarding the red light running and close-following behaviour. First of all, this chapter presents the history of road accidents and establishment of the STATS19 forms in the UK. Then, the design standard for traffic signal and intersection safety were discussed. Numerous studies have indicated that the intersection safety and performance have been improved by the installation of traffic signals. However, many perspectives have highlighted that the problem of dilemma zone can affect drivers' STOP/GO decision following the onset of amber. In addition, appropriate engineering measures can be implemented for eliminating the effect of dilemma zone such as managing excessive speeds and recommendations were drawn to keep the amber interval at 3 seconds.

Next, a comprehensive literature review was conducted to introduce driving behaviour near the signalised intersections. Many researchers have agreed that the driver's decision is a risky decision since it can be considered as a result of perception, identification, handling and reaction processes. Different factors can affect driver's choice, for example travelling speed, perception-reaction time, age and gender, driving experience and other vehicles' sizes and position in the traffic platooning. Many researchers highlighted that excessive speeding and short perception-reaction times are a reflection of aggressive behaviour in most traffic conflicts such as rear-end accidents and signal violations.

Development of the learning and making-decision algorithms are continuous to achieve accurate driving algorithms of automatous vehicle before being approved to use on the roads. It necessary to investigate the car-following rules in depth in order to ensure that the simulation model become able to replicate such traffic conflicts. In addition, some reported limitations in the existing car-following models should be considered in the newly developed model particularly the dilemma zone rules. Drivers' responses to the signal change can be replicated by using the GHM model that was established by Gazis et al. (1960). This model will be included in a combination with the original CARSIM model (developed by Benekohal (1986)) for representing driving behaviour and response to the signal changes near signal-controlled junctions. In addition, safety indicators such as Time-To-Collision (TTC) can be computed within the newly developed algorithms in order to test the junction performance and evaluate the safety issue after the onset of amber.

Finally, there is a need to collect data to observe traffic and driver behaviour following the onset of amber in order to take into account the extra impact of a driver's decision when the traffic signal changes aspect. Such data includes distances from the stopline, speeds of vehicles, traffic signal aspects and intersection width. The following chapters in this study show how such data can be combined with the car-following algorithms to develop a micro-simulation model for representing drivers' behaviour under the effect of the dilemma zone.

CHAPTER THREE: METHODOLOGY AND DATA COLLECTION

3.1 Introduction

This chapter presents the methodology that was used to collect field data in order to gain a better understanding of driver behaviour under the effect of dilemma zone (i.e. following the onset of amber) at signalised junctions. The collected data included information about traffic flow, junction layout and traffic light settings that are needed for providing necessary input parameters in the model development, calibration and validation processes.

3.2 Methodology

To study driver behaviour at signalised junctions and the effect of dilemma zone, it is necessary to select sites where vehicular traffic moves towards a traffic signal-controlled junction. Typical 3-arm or 4-arm signalised (crossroads) junctions were chosen for the current study to collect data from. Data are observed on one approach since it is difficult to observe and record traffic movements on all approaches that are leading to the junction area. The collected data are analysed and used in the model development stage that contains the relevant dependent and independent parameters. With this in mind, several considerations were taken into account prior to selecting the sites and collecting data:

- Finding a good vantage point located on a high place (such as a pedestrian footbridge or a building) to cover a sufficient approach section to the stopline and allow observation of the change of traffic light signals for the approach from the vantage point.
- The method of data collection should provide permanent records (such as video recordings). In addition, drivers should not be influenced, nor distracted which may result in changes to their decisions or speeds.
- Specifying the type of data collected that would be used for developing, calibrating and validating the simulation model. Table 3.1 summarises the parameters collected from the survey sites using the video recording method.
- Some of the collected data are analysed (as described in Chapter Four) in order to find out data distribution and curve fitting for using in the model development.

Further details regarding the data collection method and site selection are discussed in Sections 3.3 and 3.4, respectively. In addition, a description of each parameter collected in this study can be found in Section 3.5.

Table 3.1: Summary of the parameters collected in this study

Type of data	Observed parameters	Purposes
Vehicle characteristics	Vehicle length	- To be used as an input parameter to differentiate between passenger cars and heavy goods vehicles.
	Percentage of heavy goods vehicles (HGVs%)	- To find out the HGVs% in the flow. - To be used as an input parameter.
	Desired speed	- To be used as an input parameter.
Traffic flow characteristics	Traffic flow rate	- To be used as an input parameter.
	Time headway	- To find out the headway distribution of the observed arrival flow. - To be used for model calibration and validation purposes.
	Buffer space	- To find out the distribution of buffer spaces between successive vehicles in stopping conditions after the onset of red. -To be used for model development purpose.
	Queuing	- To examine the position of vehicle in the queue before the onset of green. - To be used for model development purpose.
	Move-up time (MUT)	- To find out the vehicles' departure headways following the onset of green. - To be used for the model validation purpose.
	Move-up delay (MUD)	- To find out the time needed by the first driver in the queue to start movement from the stationary condition. - To be used as an input parameter.
Driver behaviour in the dilemma zone	STOP/GO decisions	- To be used for comparing with the model outputs in the calibration and validation processes.
	Drivers' compliance (i.e. the numbers of amber and red light running as well as drivers who decided to stop after seeing the amber indication)	- To observe driver compliance and response to the signal change following the onset of amber. - To be used for comparing with the model outputs in the calibration and validation stages.
Junction characteristics	Intersection width	- To be used as an input parameter.
	Traffic light timings (i.e. the lengths of Green, Amber, Red, Red-Amber, Intergreen and all-red periods)	- To simulate the traffic light signals. - To be used as input parameters.
	Signal control (i.e. Fixed Time (FT) or Vehicle Actuated time (VA) signals)	- To be used as an input parameter.

3.3 Data collection method

A video recording technique was used in the current study as a data collection method. Most studies of similar nature have considered video recording techniques for collecting sufficient data (see Bonneson et al. (2001), Yang and Najm (2007), Elmitiny et al. (2010) and Hurwitz et al. (2012) as examples of using this technique for recording traffic and drivers' behaviour near signalised intersections). However, different methods have also been used and developed to collect traffic data. For example, loop detectors have been installed to enhance intersection capacity and safety as well as provide real information about the signal operation and traffic flow. However, this equipment does not provide sufficient information regarding drivers' STOP/GO decisions, in addition to the high costs needed for the installation and maintenance processes and the quality of the material from which the detectors were made of. Moreover, it is worth mentioning here that the unmanned aerial vehicle which is commonly known as drone may not be considered for similar studies due to the following reasons: it is expensive, needs to a special permission and training to be used in a popular area, time of record is limited to the battery life which is usually 10-15 min only, may cause distraction to drivers and difficulty in seeing the changes in the traffic light signals.

Therefore, video recording was used for this study. Over all, this method remains the main technique used by most researchers in the UK. It has advantages in that it is reasonably cheap and provides a complete record of the behaviour and traffic activities at any site. Also, it eliminates observer bias and most importantly, it gives a permanent record. The disadvantage of this technique is that it is relatively inflexible in terms of finding a suitable camera position for filming on footways or from a lower level building due to poor visibility either because of trees, an insufficient road section to the stopline or because the traffic light signal cannot be seen by the observer. In addition, working in bad weather conditions can disturb the process of recording. Finally, teamwork (three people) was needed to protect cameras and collect the field data, in addition to the set of apparatus used in this study was comprised of the following:

1. More than one camera was needed with tripods (Sony HDD Handycam DCR-SR57 were used since they were made available). The cameras were set on high buildings or footbridges and were hidden between trees to observe traffic light aspect, traffic flow and drivers' compliance to the signal change on one approach to the signalised junction without causing any distraction to them.

2. Measuring wheel was used to measure the length of the recorded section of the road and mark it into 10 m intervals from the position of the stopline in order to assign the position of each vehicle at the onset of amber.
3. Radar speed meter was used to collect a sample of actual speed data on the approach to a traffic signal junction.

3.4 Site selection and description

As shown in Figure 3.1, ten survey sites with different road widths and allowable speed limits as well as different signal control settings (i.e. fixed-time or vehicle-actuated time settings) were visited to collect data and check the possibility of a continuous recording process from a good vantage point where a pedestrian footbridge or high-level building existed. However, the researcher encountered several difficulties at some sites as described in Table 3.2. More details of each selected site are shown in Table 3.3 and Appendix B. These sites were visited a couple of times to collect real data in good weather conditions so that unbiased results could be obtained. Days affected by special events were disregarded.

As mentioned earlier in Section 3.2, the survey was carried out along one approach to the traffic light junction where a footbridge or high-level building exists. Only through and left traffic flow were chosen in this study because the right turning flow was controlled by a different signal setting (Sites #1, 2, 3 and 4). Additionally, data from the bus lane (Site #4) were not collected because of the existence of two bus stops near the traffic signal junction. Also, vehicles with a lane changing manoeuvre at the onset of amber were not included in this study because of rare events.

In addition, each section length was measured and marked into 10-metre intervals from the stopline at the beginning of filming process. The reference lines were drawn with the aid of the Coral Video Studio X8 program (after identifying each point using a measuring wheel during the filming process) in order to get an approximate distance of each vehicle from the stopline position at the onset of amber as shown in Figure 3.2 as an example. More details regarding the collected data are described in the following section.

Figure 3.1: Survey locations taken from Google Maps

Table 3.2: List of the visited sites for collecting data in and around Greater Manchester

Site #	Description	Speed limit (mph)	Site selection	Remarks
1	Junction of A34 Kingsway Road with B5095 Wilmslow Road in Manchester.	40	Yes	It provides a good view of traffic movement towards the junctions from a 5.7 m-high footbridge.
2	Junction of A57 Sankey Way Road with Cromwell Ave Road in Warrington.	40	Yes	A 7.6 m-high footbridge located at 142 m from the junction provides a good view of drivers’ behaviour with the traffic light changes.
3	Junction of A580 East Lancashire Road with Eccles Road in Salford.	40	Yes	The existence of a 5.7 m-high footbridge was useful for recording drivers’ responses to the signal change before the stopline.
4	Junction of A6 Broad Street with B6186 Frederick Road in Salford.	30	Yes	Video recording from the 8^{th} floor of a neighbouring block of flats provides a good view of the traffic movement toward the junction.
5	Junction of A5186 Langworthy Road with Liverpool Street in Salford.	30	Yes	The camera was set up 6 m above the road surface in a hiding place between trees in order to record drivers’ responses to the signal change.
6	Junction of A6010 Alan Turing Way with A662 Ashton New Road in Manchester.	30	No	Despite the availability of pedestrian footbridge near the Etihad Campus, the recording process did not complete because some trees obscured the view of traffic movements and signal aspects.
7	Junction of Trafford Road with Merchants Quay Road in Salford.	30	No	Although a hotel administration team agreed to record for the study purposes, the road section was short in length and the signal aspect could not be seen nor could the real positions of vehicles from the stopline at the onset of amber.
8	Junction of Oxford Road with Grafton Street in Manchester.	30	No	Positive permission was given by the NHS security team to record from a three-level car park building; however the building was very close to the junction and the signal aspect change could not be seen during the video recording process.
9	Junction of Upper Brook Road with Hathersage Road in Manchester.	30	No	Manchester children’s hospital car park building was visited for video recording purposes. However, it was not selected because it cannot see the road section.
10	Junction of A5181 Park Road with Derbyshire Lane in Trafford.	30	No	Video recording from the 2^{nd} floor of a neighbouring flat did not provide a good road section because the view was covered partially with trees and the signal aspects could not be seen.

(a) Camera location at Site #4 (Screenshot from Google Maps)

(b) Screenshot technique

Figure 3.2: Video recording method (Site #4 as an example)

Table 3.3: Details of the survey sites in and around Greater Manchester

Site #	Recording date	Duration (min)	Time	Site length (m)	No. of lanes	Speed limit (mph)	Signal setting
1	(a) 16 Nov 2015 (b) 19 Nov 2015	100 120	10:00-11:40 AM 13:00-15:00 PM	70 m	3	40	FT
2	17 Jun 2016	120	11:30-13:30 PM	80 m	2	40	FT
3	(a) 13 Jul 2016 (b) 25 Jul 2016	105 120	12:00-14:00 PM 10:00-12:00 AM	70 m	3	40	VA
4	(a) 18 Apr 2016 (b) 19 Apr 2016 (c) 22 Apr 2016	120 120 120	10:30-12:30 AM 14:00-16:00 PM 15:00-17:00 PM	70 m	2	30	VA
5	(a) 28 Oct 2015 (b) 29 Oct 2015	120 60	13:00-15:00 PM 10:30-11:30 AM	50 m	2	30	FT

FT: Fixed-Time signal
VA: Vehicle-Actuated signal

3.5 Data description

3.5.1 Vehicle characteristics

Vehicles of different types have different road space requirements and different effects on the capacity of the intersection because of variations in size and performance. Traffic composition was classified into heavy goods vehicles (HGVs) and passenger cars (PCs) including light goods vehicles. Other vehicle types such as motorcycles were ignored because of rare observations. Following previous studies conducted by Al-Obaedi (2011) and Alterawi (2014), the value of 5.6 m vehicle length was used to distinguish between passenger cars and heavy goods vehicles and was adopted for this study as presented in Table 3.4.

Table 3.4: Summary of vehicle types and lengths based on previous studies

Researcher's name	Vehicle type	Minimum (m)	Maximum (m)	Mean (m)	Standard deviation (m)
Alterawi (2014)	PC	2.30	5.60	4.20	0.50
	HGV	5.60	16.50	9.50	1.90
Al-Obaedi (2011)	PC	2.30	5.60	4.20	0.45
	HGV	5.60	25.50	11.40	4.30

3.5.2 Traffic flow characteristics

3.5.2.1 Time Headway

Time headway can be defined as the time which elapses between successive vehicles over a certain point or reference line on a road section according to Garber and Hoel (2009), as illustrated in Figure 3.3. Yousif (1993) revealed that time headway distribution is influenced by several factors, for instance traffic composition, traffic flow, reaction time of the driver, and braking distance. Traffic flow for each site was collected in 5-minute intervals to be used as inputs into the newly developed model. Then, time headways can be generated from the observed arrival flow profile in order to compare with the outputs of the developed model.

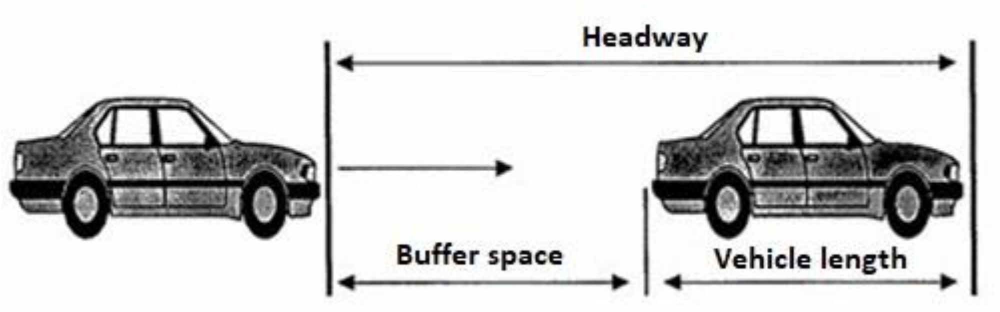


Figure 3.3: Illustration of time headway and buffer space (Federal Highway Administration, 2005)

Several statistical formulas of time headway distributions can be applied to represent the vehicles' arrival depending on the traffic flow rates. According to Salter and Hounsell (1989), the negative exponential distribution can be used to represent the random arrival of vehicles. It assumes that random arrivals occur within a specific time interval *t* and follow the Poisson distribution. Time headways can be generated randomly from the following equation:

$$Time\ headway = \frac{\ln(RAND)}{Q} \qquad \textit{Equation 3.1}$$

Where Q is the mean rate of vehicle arrival per unit time, and *RAND* is a random number from 0 to 1 (for example, generated by a FORTRAN program). Panichpapiboon (2014) reported that the negative exponential distribution can be used where the traffic flow is extremely low (vehicular flow < 250 vph) particularly during the earliest hours (e.g. 12:00 – 03:00 am).

Furthermore, the shifted negative exponential distribution is suitable to fit headway distributions on roads where the traffic flow is moderate (400-600 vph) (Al-Obaedi, 2011, Alterawi, 2014, Nassrullah, 2016, and Yousif, 1993). However, Panichpapiboon (2014) reported that the shifted negative exponential distribution can fit headway data for traffic flows between (250-750 vph). The headways can be generated randomly from the following equation:

$$Time\ headway = shift - \left(\frac{1}{Q} - shift\right)\ln(RAND) \qquad \textit{Equation 3.2}$$

Where *Shift* is an additional time such as 0.25, 0.50, or 1 sec., which recognises that in practice there is a minimum time headway value between vehicles.

On the other hand, Nassrullah (2016) and Alterawi (2014) examined distribution of vehicle arrival with the lognormal distributions for heavy flow rates. They agreed that there are significant differences to in replication of observed data with the lognormal distributions. The probability density function of lognormal distribution can be found from Equations 3.3 to 3.5 as shown below:

$$f(t) = \frac{1}{\sigma . t . \sqrt{2\pi}}\, e^{\frac{-(\ln(t)-\mu)^2}{2\sigma^2}} \qquad \textit{Equation 3.3}$$

$$\mu = \ln(m) - \frac{\sigma^2}{2} \qquad \textit{Equation 3.4}$$

$$\sigma = \sqrt{\ln(1 + \frac{s^2}{m^2})} \qquad \textit{Equation 3.5}$$

Where m and *s* are the mean and standard deviation of the lognormal distribution respectively. The symbols μ and σ are the mean and standard deviation of the normal distribution respectively.

Additionally, time headway is a very important factor as a safety indicator since it gives a better understanding of driver response following the onset of amber. Because of the difficulty in measuring Time-To-Collision (TTC) between successive vehicles at the survey sites, it is interesting to investigate the effect of time headways for successive vehicles approaching the stopline at the onset of amber in terms of drivers' STOP/GO decisions. Time headways were collected by using a video playback method at the onset of amber for data analysis as shown in Figure 3.4. The position of vehicle *n* will be used as a reference line to compute the time headways with the preceding and following vehicles. The mathematical expressions can be written as:

$$\begin{matrix} Time\ headway\ (for\ vehicle\ n) \\ with\ preceding\ vehicle, (n, Pos_{nSL})_L \end{matrix} = T_n - T_L \qquad \textit{Equation 3.6}$$

$$\begin{matrix} Time\ headway\ (for\ vehicle\ n) \\ with\ following\ vehicle\ (n, Pos_{nSL})_F \end{matrix} = T_F - T_n \qquad \textit{Equation 3.7}$$

where T_F and T_L are the time headways of following and preceding vehicles, respectively, and Pos_{nSL} is the position of vehicle *n* from the stopline.

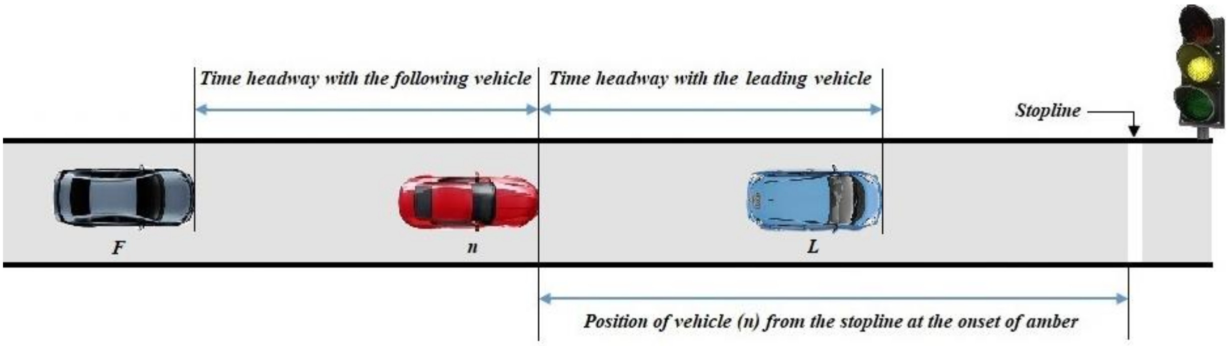


Figure 3.4: Time headway between three successive vehicles at the onset of amber

3.5.2.2 Buffer space

Buffer space can be measured from the front bumper of the following vehicle to the rear of the front vehicle as shown previously in Figure 3.3. The safe buffer space between stopped vehicles was measured by the measuring wheel for analysis purposes in order to be used as input data in the model development.

3.5.2.3 Move-up time (MUT)

This is known as discharge headway or vehicle departure headway and it was used to measure the capacity of a junction and signal time setting. It can be defined as the interval between the passage of successive vehicles over the stopline of a signalised junction after the onset of green (Roess et al., 2004). The headway will decline gradually to a stable value (which is recognised as saturation headway) after the fourth or fifth vehicle in the queue, as illustrated in Figure 3.5 (Michael et al., 2000; Niittymaki and Pursula, 1996; and Roess et al., 2004). The observed MUT will be analysed and compared with the outputs of the developed model as part of the model validation process.

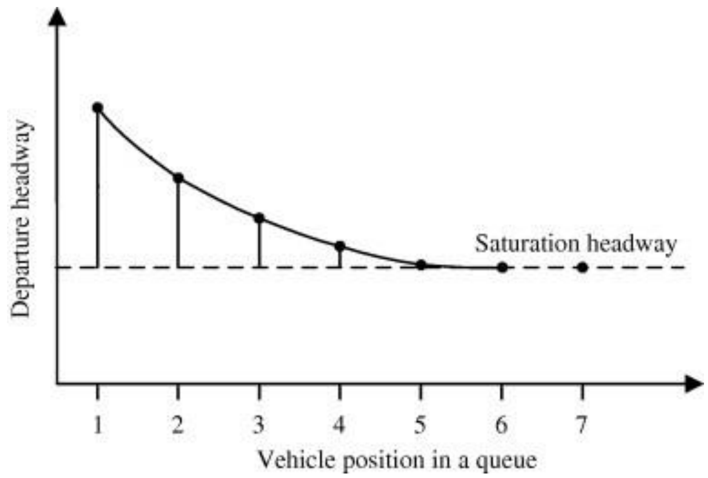


Figure 3.5: Departure headway according to a vehicle position in a queue (adapted from Roess et al. (2004))

3.5.2.4 Move-up delay (MUD)

When a traffic light signal shows green, the first driver in the queue will take a few seconds to accelerate in response to the green indication. This is known as move-up delay or start-up time. The MUD was observed and collected throughout the survey by capturing the difference between the time that the red-amber signal came on and the time that the first driver in the queue started to move. The observed MUD data will be analysed in order to be used as input parameter for the developed model.

3.5.3 Drivers' compliance and STOP/GO decisions

The video playback technique was used to observe drivers' behaviour approaching the stopline following the onset of amber. Drivers' STOP/GO decisions under the effect of the dilemma zone were detected from the videos and reported in this survey. The number of amber crossing and red light runnings were also observed from the records and reported. Three categories of driver compliance were reported: Amber Light Running (ALR), Red Light Running (RLR), and Amber/Red Light Stopping (ARLS). Full details of drivers' categories will be explained in Section 4.4.1. Furthermore, the distance and time to the stopline for each approaching vehicle at the onset of amber was detected in terms of whether the driver's decision was to stop or continue crossing the stopline. These data will be compared with the model results in the calibration and validation processes.

3.5.4 Geometric data and signal settings

Other relevant data were collected from the sites by using the measuring wheel, such as the length of the approach being surveyed, number of lanes, and lane position. It is important to be reminded that the traffic data in each lane have been collected separately in order to see if there are any significant differences between the behaviours of drivers in each lane. As described previously, the bus lane and right turning lane controlled by separate traffic signals were not considered in the data collection process. Figure 3.6 depicts the lane position for each survey site. Moreover, the intersection width was measured with the aid of the Google Maps measuring tool and used as an input parameter for the model development. In addition, traffic signal control type (i.e. fixed-time or vehicle-actuated time), cycle length (i.e. green, amber, red and red-amber timings) and all-red period were collected from video recordings for each survey site.

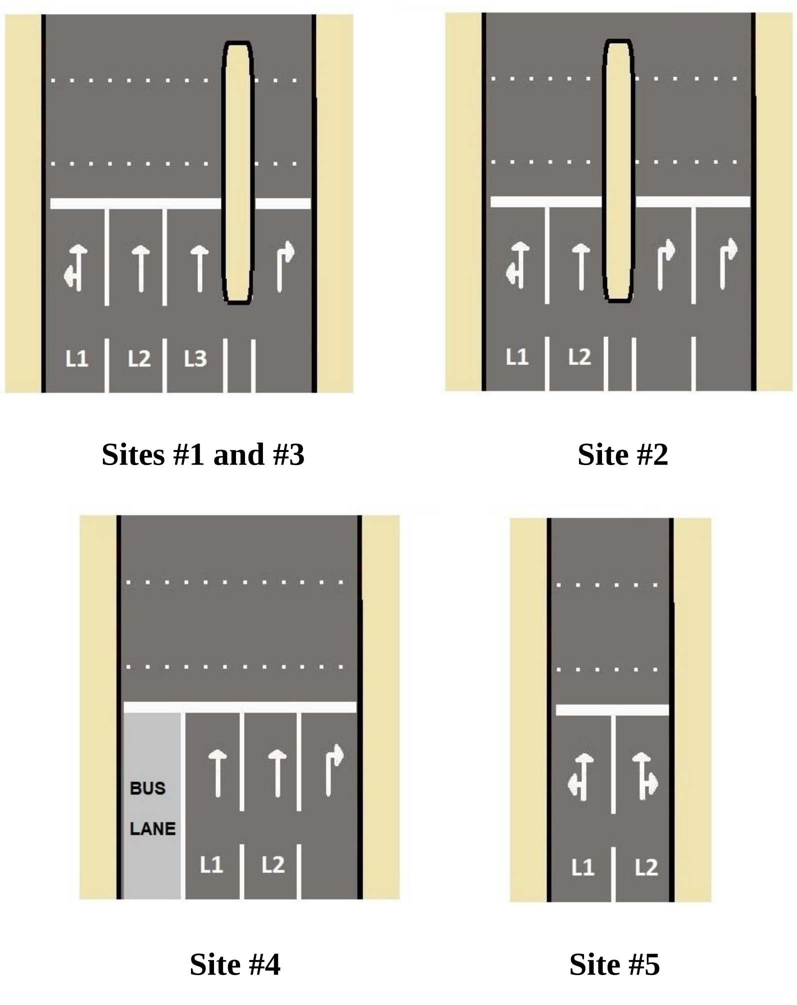


Figure 3.6: Lane position and nomination for each survey site

3.6 Data extraction method

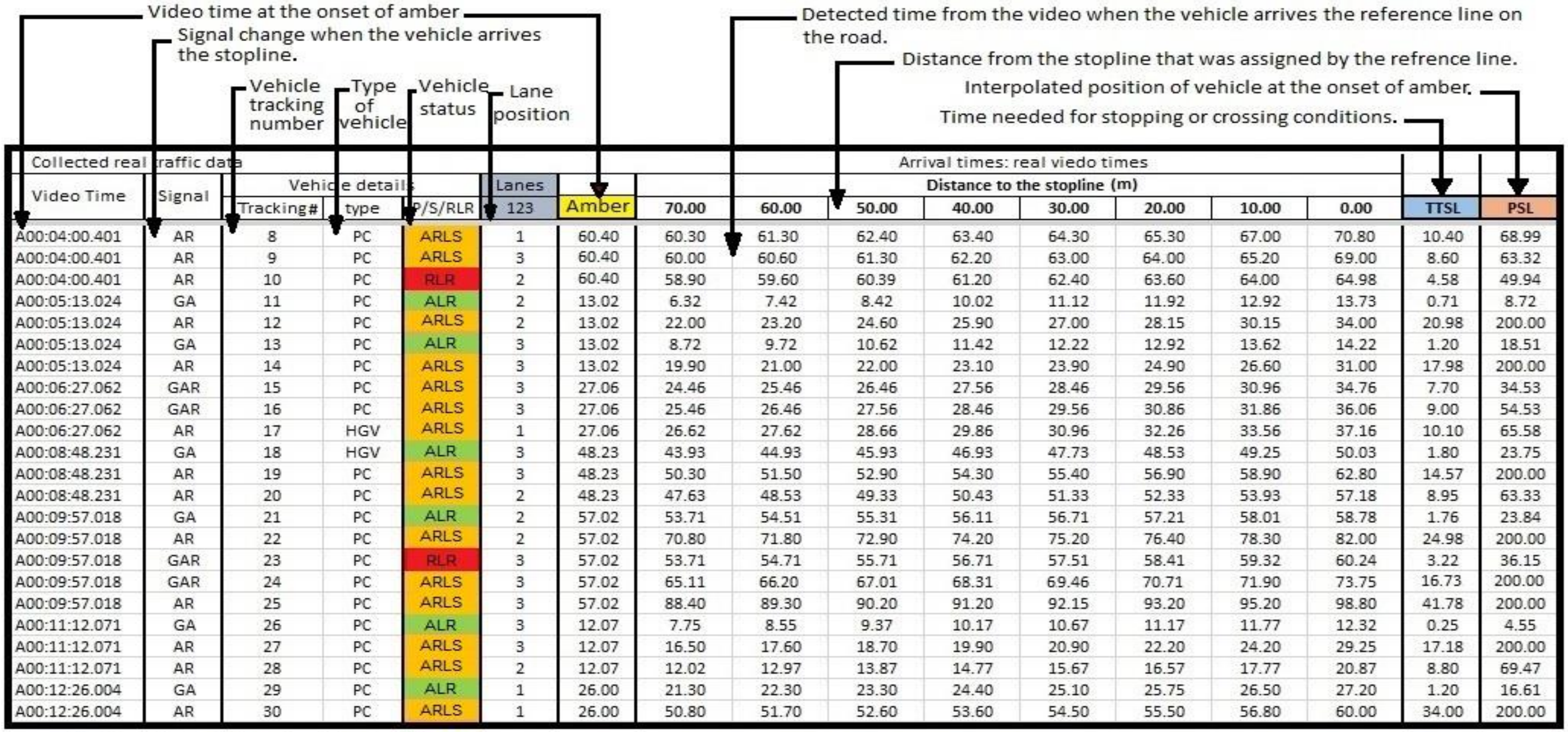
The collected data in this study were extracted using a video playback method in order to record the passage of time of vehicles at 10-metre intervals and their positions from the stopline when the signal was showing amber. The collected data were saved on a data entry sheet in an Excel file for later processing, as illustrated in Figure 3.7. It was decided that a suitable data entry form for each site within an Excel sheet should be designed for the following reasons:

1. It is simple to use and enter the data.
2. Each sheet can accept a large amount of data.
3. It is easy to develop, modify and create new sheets as well as frequency tables by using statistical functions.

3.7 Accuracy of measurements

To minimise possible errors and data bias from the video recording method, three different data sets were used to check accuracy. The first was the measurement of the 10-metre successive distances along each site length as shown previously in Figure 3.2b. Two people measured these distances twice by using a measuring wheel to assign a reference point for the datum line marking. Both of them repeated the measurement process twice for double checking. The findings showed that the distances between the reference points gave consistent values at (10m±3cm). It can be suggested that the difference (±3 cm) is very small and that it is considered acceptable.

Two samples of vehicles' speeds were collected from two sites (Sites #2 and #4) to measure the approaching speed on the road. A vehicle speed was measured by considering two datum lines covering a 30-metre road section (40 m from the stopline position). The speed was calculated manually by dividing the measured distance (30 m) by the time required for a vehicle to pass this distance. To eliminate errors in measuring the vehicle passing time which might affect the accuracy of speed measurements, an alternative method of speed measurement was carried out using a radar speed meter. The speeds of vehicles were detected using a radar meter from a sufficient distance (70 m) from the stopline without causing distraction to drivers. The results from the radar speed meter were compared with those obtained from the calculation method. There were no significant differences between both methods, as shown in Table 3.5, and the results were therefore accepted.



Collected real traffic data							Arrival times: real viedo times									
Video Time	Signal	Vehicle details			Lanes		Distance to the stopline (m)									
		Tracking#	type	P/S/RLR	123	Amber	70.00	60.00	50.00	40.00	30.00	20.00	10.00	0.00	TTSL	PSL
A00:04:00.401	AR	8	PC	ARLS	1	60.40	60.30	61.30	62.40	63.40	64.30	65.30	67.00	70.80	10.40	68.99
A00:04:00.401	AR	9	PC	ARLS	3	60.40	60.00	60.60	61.30	62.20	63.00	64.00	65.20	69.00	8.60	63.32
A00:04:00.401	AR	10	PC	RLR	2	60.40	58.90	59.60	60.39	61.20	62.40	63.60	64.00	64.98	4.58	49.94
A00:05:13.024	GA	11	PC	ALR	2	13.02	6.32	7.42	8.42	10.02	11.12	11.92	12.92	13.73	0.71	8.72
A00:05:13.024	AR	12	PC	ARLS	2	13.02	22.00	23.20	24.60	25.90	27.00	28.15	30.15	34.00	20.98	200.00
A00:05:13.024	GA	13	PC	ALR	3	13.02	8.72	9.72	10.62	11.42	12.22	12.92	13.62	14.22	1.20	18.51
A00:05:13.024	AR	14	PC	ARLS	3	13.02	19.90	21.00	22.00	23.10	23.90	24.90	26.60	31.00	17.98	200.00
A00:06:27.062	GAR	15	PC	ARLS	3	27.06	24.46	25.46	26.46	27.56	28.46	29.56	30.96	34.76	7.70	34.53
A00:06:27.062	GAR	16	PC	ARLS	3	27.06	25.46	26.46	27.56	28.46	29.56	30.86	31.86	36.06	9.00	54.53
A00:06:27.062	AR	17	HGV	ARLS	1	27.06	26.62	27.62	28.66	29.86	30.96	32.26	33.56	37.16	10.10	65.58
A00:08:48.231	GA	18	HGV	ALR	3	48.23	43.93	44.93	45.93	46.93	47.73	48.53	49.25	50.03	1.80	23.75
A00:08:48.231	AR	19	PC	ARLS	3	48.23	50.30	51.50	52.90	54.30	55.40	56.90	58.90	62.80	14.57	200.00
A00:08:48.231	AR	20	PC	ARLS	2	48.23	47.63	48.53	49.33	50.43	51.33	52.33	53.93	57.18	8.95	63.33
A00:09:57.018	GA	21	PC	ALR	2	57.02	53.71	54.51	55.31	56.11	56.71	57.21	58.01	58.78	1.76	23.84
A00:09:57.018	AR	22	PC	ARLS	2	57.02	70.80	71.80	72.90	74.20	75.20	76.40	78.30	82.00	24.98	200.00
A00:09:57.018	GAR	23	PC	RLR	3	57.02	53.71	54.71	55.71	56.71	57.51	58.41	59.32	60.24	3.22	36.15
A00:09:57.018	GAR	24	PC	ARLS	3	57.02	65.11	66.20	67.01	68.31	69.46	70.71	71.90	73.75	16.73	200.00
A00:09:57.018	AR	25	PC	ARLS	3	57.02	88.40	89.30	90.20	91.20	92.15	93.20	95.20	98.80	41.78	200.00
A00:11:12.071	GA	26	PC	ALR	3	12.07	7.75	8.55	9.37	10.17	10.67	11.17	11.77	12.32	0.25	4.55
A00:11:12.071	AR	27	PC	ARLS	3	12.07	16.50	17.60	18.70	19.90	20.90	22.20	24.20	29.25	17.18	200.00
A00:11:12.071	AR	28	PC	ARLS	2	12.07	12.02	12.97	13.87	14.77	15.67	16.57	17.77	20.87	8.80	69.47
A00:12:26.004	GA	29	PC	ALR	1	26.00	21.30	22.30	23.30	24.40	25.10	25.75	26.50	27.20	1.20	16.61
A00:12:26.004	AR	30	PC	ARLS	1	26.00	50.80	51.70	52.60	53.60	54.50	55.50	56.80	60.00	34.00	200.00

TTSL : Time to the stopline (sec)
PSL : Position from the stopline (m)
AR: Amber-Red
GA: Green-Amber
GAR : Green-Amber-Red
PC: Passenger car
HGV: Heavy Goods Vehicle

ALR : Amber Light Running driver passes or crosses the stopline during the amber period.
ARLS: Amber/Red Light Stopping driver stops after the onset of amber.
RLR : Red Light Running driver crosses the stopline and violates the red light.
Lane 1 : Left lane
Lane 2 : Middle lane
Lane 3 : Right lane

Figure 3.7: Screenshot of data extraction and storage in the Excel sheet

Table 3.5: Collecting vehicles' speeds in (kph) using two different methods

Site	Methods	Manual	Radar meter
#2	Sample size	100	100
	Mean	60.67	60.59
	Standard deviation	5.48	5.45
#4	Sample size	100	100
	Mean	47.83	47.91
	Standard deviation	5.33	5.41

Finally, a sample of vehicle arrival time headways was collected twice from Site #1 to check data accuracy. Firstly, they were collected from the video using a video playback method, and then repeated using the EVENT program developed by Al-Neami (2000). This program was written using C++ computer language. The program introduces time accuracy value for the recorded data. This accuracy is about up to 0.055 of a second which can be considered acceptable. Table 3.6 shows that the differences between both methods were very small which can be considered acceptable. Additionally, Figure 3.8 shows a good fit and high reliability of the collected data by finding the correlation between both methods.

Table 3.6: Collecting arrival time headway in (sec) using two different methods

Methods	Playback	EVENT	Sample size
Mean	9.05	9.00	147 vehicles
Standard deviation	8.40	8.37	

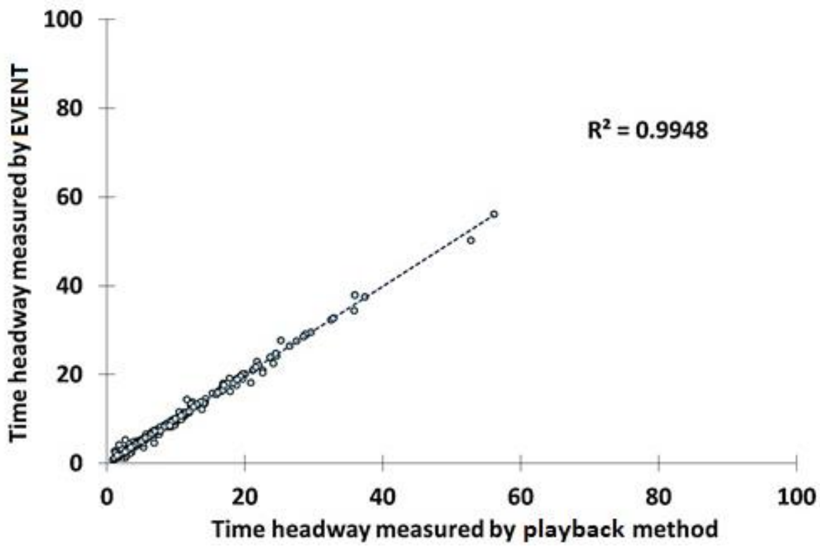


Figure 3.8: Correlation between the two different data extraction methods

3.8 Summary

The methodology and data collection have been defined in this chapter. The video recording method is a common method used by many traffic engineering researchers worldwide. Details of five different approaches incorporating three and two lanes controlled by traffic signals have been described including site length and the allowable speed limit. Red light violations due to lane changing as well as right turning movement controlled by a different traffic signal have been disregarded in this study due to rare observations. Distances and times to the stopline during the amber/red period have been collected along with driver compliance with the amber/red light signal. Junction widths and signal timings were collected from all survey sites. Finally, this data will be used in the model development, calibration and validation processes. Analysis of the collected data is described in the next chapter.

CHAPTER FOUR: DATA PRESENTATION AND ANALYSIS

4.1 Introduction

The main aim of this chapter is to analyse the observed data that will be used to develop, calibrate and validate the proposed micro-simulation model. Analysis of the observed data is a crucial element of developing the model assumptions regarding the traffic arrival patterns and drivers' behaviour approaching traffic light signal junctions. In addition, this chapter presents factors that might significantly affect drivers' STOP/GO decisions after the onset of amber.

4.2 Traffic composition and flow level

It is necessary to obtain details of vehicles' composition and traffic flow moves towards a signalised junction, so that an initial set-up can be modelled effectively. A traffic stream comprises different types of vehicles which vary in size and performance. As discussed in Section 3.5.1, traffic composition for this research is divided into two main categories: passenger cars (including light goods vehicles) and heavy goods vehicles based on previous studies conducted by Alterawi (2014) and Al-Obaedi (2011).

The traffic flow profile was surveyed at each site for 5-minute intervals and then multiplied by 12 to obtain the hourly flow rate, as presented in Table 4.1. In addition, the percentages of HGVs in the traffic flow were observed and recorded.

Table 4.1: Minimum and maximum observed hourly flow and HGV percentage

Description	Minimum	Maximum
Flow rate (vph)	248	724
HGVs %	0.3	13.2

4.3 Vehicle characteristics

4.3.1 Vehicle type and length

On urban roads, there are different types of vehicles which vary in size. Vehicle length is one of the components of car-following rules particularly in calculating a vehicle's acceleration/deceleration rate and spacing between vehicles.

To investigate the length of vehicles, El-Hanna (1974) classified vehicles into two main categories: passenger cars (PCs) and heavy goods vehicles (HGVs). El-Hanna (1974) highlighted that for both types of vehicles, length was normally distributed with the statistics presented in Table 4.2. This classification has been adopted in other research works such as Zia (1992) and Wang (2006).

Table 4.2: Vehicle lengths in (m) based on El-Hanna (1974) study

Vehicle type	Mean (μ)	Standard deviation (σ)
Passenger cars (PCs)	4.2	0.4
Heavy goods vehicles (HGVs)	11.2	2.4

Other classifications of vehicle lengths have been suggested by many researchers. Al-Jameel (2012), Al-Obaedi (2011), Alterawi (2014) and Nassrullah (2016) investigated the lengths of passenger cars based on UK motorway data. They found that the lengths were normally distributed for PCs and close to those values suggested by El-Hanna (1974) as shown in Table 4.3. In addition, the width of PCs was assumed to be equal to 1.8 m following previous work by Al-Obaedi (2011).

Table 4.3: Summary of PCs' lengths in (m) based on real UK motorway data

Researcher's name	Minimum	Maximum	Mean (μ)	Standard deviation (σ)
Al-Jameel (2012)	2.25	5.99	4.40	0.42
Al-Obaedi (2011)	2.30	5.60	4.20	0.45
Alterawi (2014)	2.30	5.60	4.20	0.50
Nassrullah (2016)	2.52	5.59	4.31	0.44

Following the previous works of Nassrullah (2016), Alterawi (2014) and Al-Obaedi (2011), the value of 5.6 m was used in the current study to differentiate between passenger cars and heavy goods vehicles. The normal distribution of PCs' lengths was obtained from Alterawi (2014) and adopted for this study as shown in Figure 4.1; it was based on a sample size of more than 5 million cars. The relative cumulative frequencies will be used to generate random numbers and predict car lengths in the simulation model based on the hypothesis of the normal distribution.

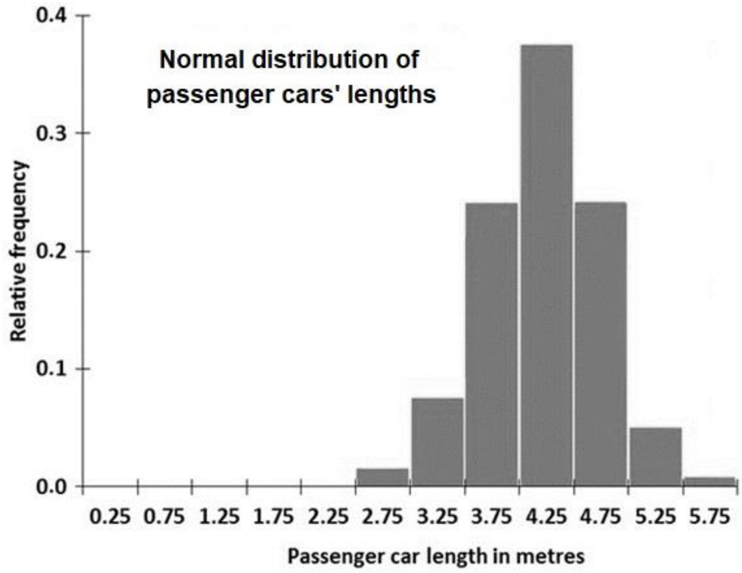
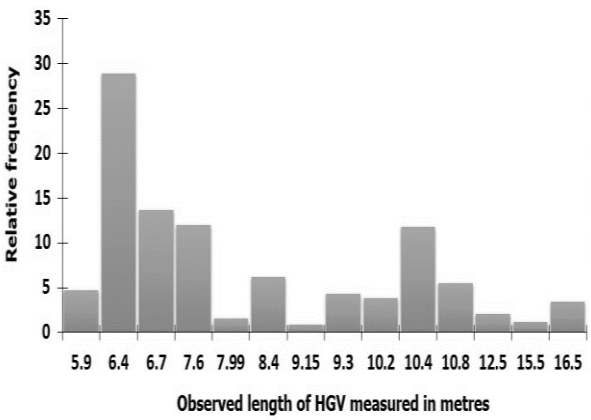
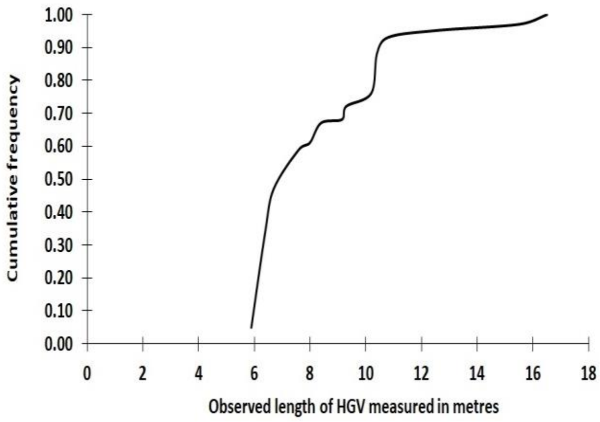


Figure 4.1: Passenger cars' lengths normal distribution based on the UK motorway dataset (Alterawi, 2014)

To identify the lengths and widths of HGVs, a sample of data was collected from the survey sites taking into considerration the overall length, width, and classification of HGVs according to the number of axles as shown in Table 4.4. Some of this information was taken from the manufacturers' websites published online, while others were documented as presented in Table 4.4. As with previous studies (Al-Jameel, 2012; Al-Obaedi, 2011; and Alterawi, 2014), the distribution of HGVs does not follow a normal distribution as illustrated in Figure 4.2a. For the purpose of the micro-simulation model, the length of HGVs will be generated randomly from the cumulative frequency curve shown in Figure 4.2b, while the width of HGVs will be assigned as shown in Table 4.4.





(a) **(b)**

Figure 4.2: HGV lengths' distribution based on real data collected from survey sites

Table 4.4: Observed HGVs based on the Department for Transport (2006a) classification

HGV type	Illustration	Vehicle Length* (m)	Vehicle width* (m)	Observed frequency**	Source
2-axle		5.9 – 10.2	max 2.0	409	Ford Transit website Volvo Trucks website
3-axle		7.6 – 9.3	max 2.0	30	Ford Transit website Volvo Trucks website
4-axle		9.15 – 10.8	max 2.0	32	Volvo Trucks website
5-axle		max 15.5	max 2.3	7	Butcher (2009)
6-axle		max 16.5	max 2.3	20	Butcher (2009)
Single deck bus		12.0 – 12.5	max 2.55	12	Department for Transport (2013)
Double deck bus		9.9 – 11.4	max 2.55	68	Department for Transport (2013)

* The length and width of vehicle were taken from the source mentioned in the table.
** The data were obtained from the survey sites.

4.3.2 Vehicle arrival distributions using real data

Under conditions of random traffic flow, vehicles arrive with different time gaps passing a given point or a datum line on the approach towards a traffic light signal. Three statistical formulas of time headway distribution, described previously in Section 3.5.2.1, were used to test the random arrival of traffic flow near signalised junctions. Time headways were observed for a 1-hour period and extracted with the aid of the EVENT program (Al-Neami, 2000). Figure 4.3 illustrates that the shifted negative exponential with a shifted value of 0.5 shows a good fit between the observed and the predicted cumulative arrival headway for a flow level of less than 400 vph. For moderate to high flow (i.e. over 400 vph), the negative exponential distribution shows good agreement between the cumulative observed and the predicted headways, as shown in Figure 4.4.

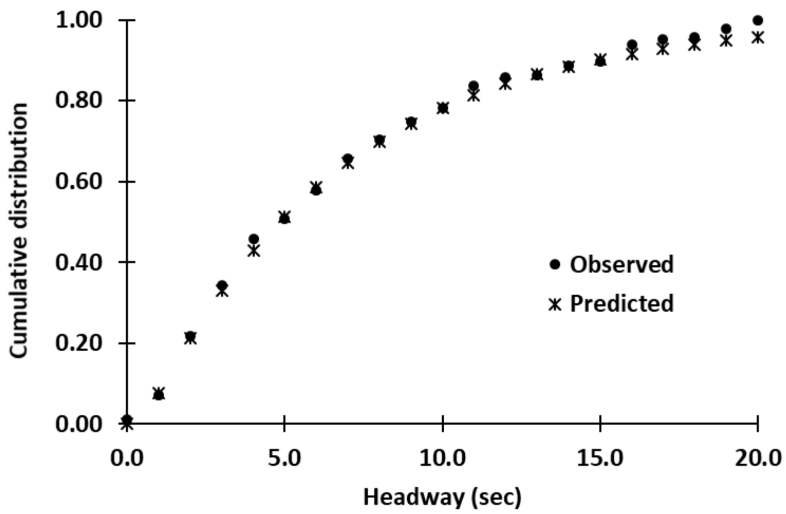


Figure 4.3: Observed and predicted cumulative time headway distributions for a flow up to 400 vph using the shifted negative exponential distribution

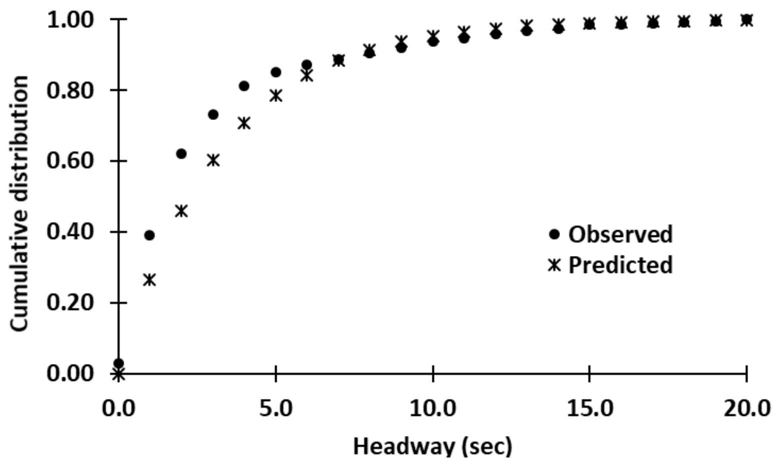


Figure 4.4: Observed and predicted cumulative time headway distributions for a flow over 400 vph using the negative exponential distribution

The differences between the predicted and observed cumulative distributions were tested statistically using the non-parametric Kolmogorv-Smirnov hypothesis test (K-S test) at a 95% level of confidence. The critical difference value can be obtained from the following formula:

$$D_{Critical} = 1.36\sqrt{\frac{N_1+N_2}{N_1(N_2)}} \qquad \textit{Equation 4.1}$$

where, N_1 and N_2 are two different sample sizes over 35 observations for each sample.

Table 4.5 shows the K-S test results of cumulative headway distribution for two sites (Sites #1 and #4). Both shifted and negative exponential distributions are capable of representing the random arrival of vehicles. However, the test shows lower diffrences between the observed and predicted data for moderate to high flow lanes (over 400 vph) represented by the negative exponential distribution. Also, very small differences were shown in representing the vehicles' arrival by the shifted negative exponential distribution on lanes of a low flow level (less than 400 vph). For simulation purposes, it can be concluded that time headway can be generated from the shifted negative exponential distribution on lanes with free-flow conditions (up to 400 vph) and from the negative exponential on lanes with a flow level greater than 400 vph.

Table 4.5: The K-S test summary for testing vehicles' headways distribution

Sites	**Site #1 (40 mph)**			**Site #4 (30 mph)**	
Lane position	**1**	**2**	**3**	**1**	**2**
Average flow (vph)	235	352	445	737	617
Shifted negative exponential D_{Max}	0.006	0.028	0.037	0.035	0.034
Negative exponential D_{Max}	0.067	0.049	0.024	0.019	0.022
Lognormal distribution D_{Max}	0.145*	0.155*	0.181*	0.160*	0.172*
K-S critical value $D_{Critical}$	0.125	0.103	0.091	0.071	0.077
Curve fitting (Shifted Neg. Exp.)	Accepted	Accepted	Accepted	Accepted	Accepted
Curve fitting (Neg. Exp. Dist.)	Accepted	Accepted	Accepted	Accepted	Accepted
Curve fitting (Lognormal Dist.)	Rejected	Rejected	Rejected	Rejected	Rejected

* $D_{Max}>D_{Critical}$

4.3.3 Buffer space

Buffer space is the safety gap between the front bumper and rear of two successive vehicles in stopping conditions, as illustrated in Figure 3.3. Previous studies have assumed different values to assign buffer space, as presented in Table 4.6 based on empirical data. For the purpose of the current study, a sample size of 120 gaps between vehicles was measured. The obtained data were tested using the K-S test as shown in Table 4.7. The distribution of observed data showed good agreement with the lognormal distribution as illustrated in Figure 4.5. Therefore, random numbers of spacings between vehicles will be generated from the lognormal distribution for the model development process.

Table 4.6: Buffer space values adopted in previous works

Researcher's name	Buffer space (m)	Study site
Benekohal (1986)	3.00	Roads and bottleneck sections
Zia (1992)	3.00	Motorway merges and dual carriageways
Yousif (1993)	1.80	Dual-carriageway motorway with roadworks
Al-Obaedi (2011)	1.50 – 3.00	Motorway merges with ramps
Al-Jameel (2012)	2.00	Weaving sections
Alterawi (2014)	1.50	Shuttle-lane roadworks
Nassrullah (2016)	1.80	Motorway roadworks

Table 4.7: Summary of buffer space statistics measured in (m) and the K-S test

Sample size	Mean (μ)	Standard deviation (σ)	Min	Max	D_{Max}	$D_{Critical}$	Curve fitting
120	2.32	0.77	1.00	4.50	0.026	0.176	Accepted

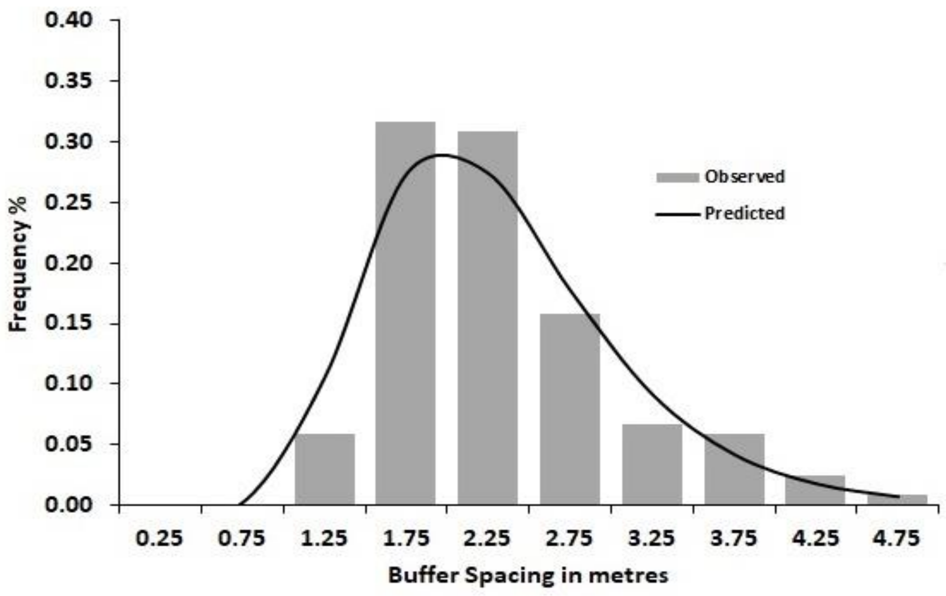


Figure 4.5: Observed and predicted buffer spacing between stopped vehicles

4.3.4 Move-up time (MUT)

A sample of vehicles' MUTs was investigated at 30 and 40 mph signalised junctions in this study. Table 4.8 provides the observed MUT data including the position of vehicles in the queue on both signalised junctions.

According to previous studies (Michael et al., 2000; Niittymaki and Pursula, 1996; and Roess et al., 2004), vehicles' headways in the queue decrease slightly until they reach a steady value called the 'saturation headway' after movement of the 4th or 5th vehicle in the queue. This is reasonable because some drivers, particularly the few vehicles at the beginning of the queue, tend to have a longer reaction time to accelerate to their desired speeds compared with other drivers who have sufficient times and distance gaps. The saturation headway value was approximately 2 seconds after the 6th vehicle starts to accelerate at both survey sites. Finally, the results of this study showed consistency with previous studies, as shown in Figure 4.6.

Table 4.8: Summary of drivers' MUT at 30 mph and 40 mph signalised junctions

Vehicle position in the queue	30 mph signalised junction					40 mph signalised junction				
	Sample size	μ (sec)	σ (sec)	Min (sec)	Max (sec)	Sample size	μ (sec)	σ (sec)	Min (sec)	Max (sec)
2	284	2.70	0.65	1.38	6.18	657	2.82	0.83	1.46	5.14
3	262	2.38	0.40	1.25	4.83	604	2.47	0.75	1.15	6.49
4	226	2.21	0.32	1.23	4.62	552	2.38	0.52	1.41	4.17
5	188	2.06	0.26	1.21	3.33	461	2.12	0.52	1.05	3.37
6	159	2.01	0.37	1.40	4.00	304	1.98	0.39	1.30	3.22
7	124	2.01	0.53	1.30	4.64	226	2.02	0.45	0.94	2.98
8	61	1.91	0.14	1.13	2.79	152	1.97	0.63	0.12	2.63
9	32	1.80	0.61	1.33	3.83	73	1.87	0.43	1.38	2.61
10	17	1.78	0.13	1.42	2.14	24	1.78	0.29	1.53	2.17

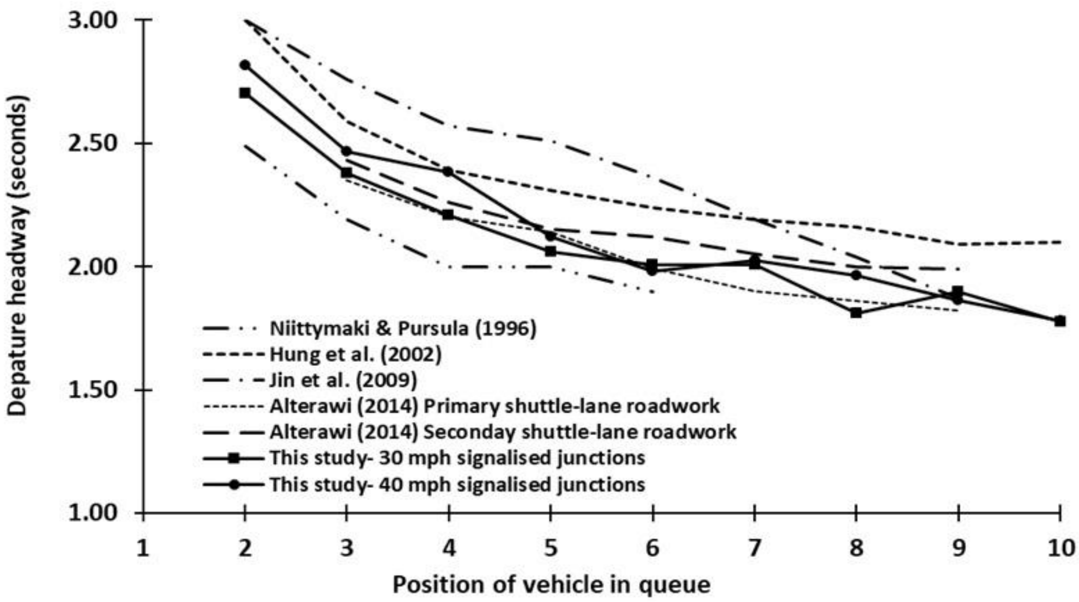
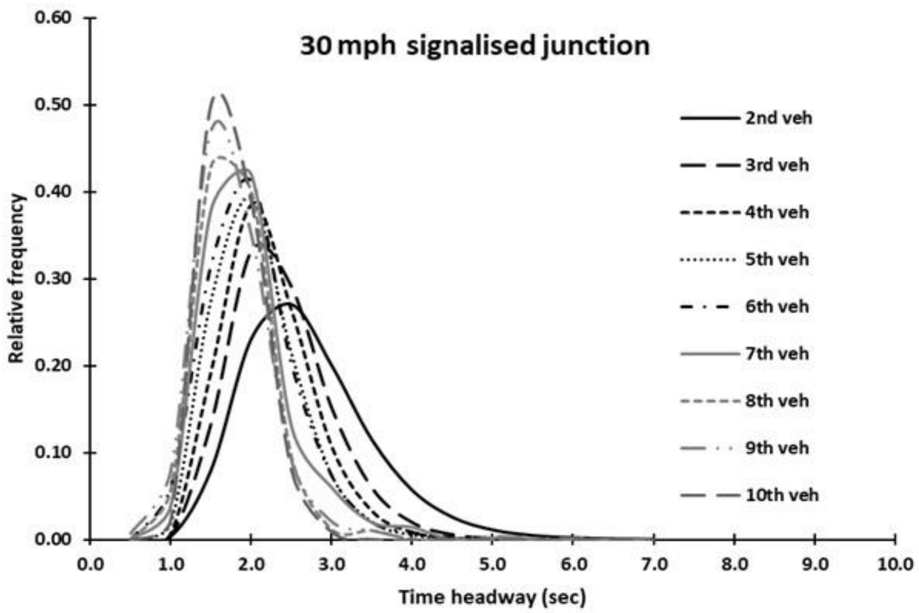


Figure 4.6: Observed drivers' MUT at 30 mph and 40 mph signalised junctions compared with previous studies

As with previous studies (Alterawi, 2014 and Jin et al., 2009), the distribution of drivers' MUT follows the lognormal distribution as illustrated in Figure 4.7 and Table 4.9. The non-parametric statistical K-S test was used to compare the maximum differences between the cumulative distribution functions of observed and predicted data at a 5% level of confidence.

Finally, the observed MUT data will be compared with the micro-simulation model outputs for the purposes of model calibration and validation techniques.



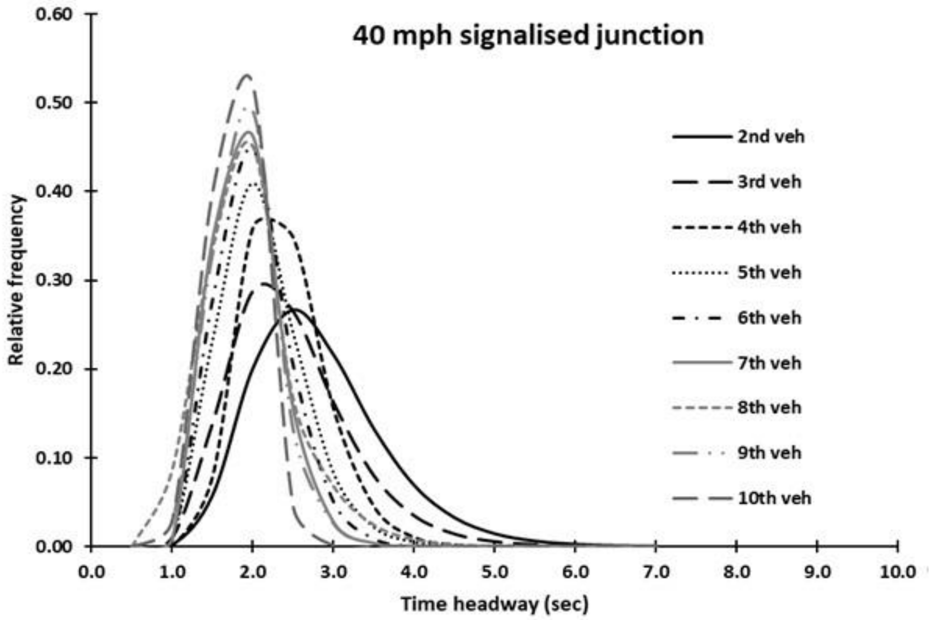


Figure 4.7: Relative distribution of observed drivers' MUT at 30 mph and 40 mph signalised junctions

Table 4.9: The K-S test summary for testing drivers' MUT distribution with lognormal distribution at 30 mph and 40 mph signalised junctions

Vehicle position in the queue	30 mph signalised junctions				40 mph signalised junctions			
	Sample size	D_{Max}	$D_{Critical}$	Curve fitting	Sample size	D_{Max}	$D_{Critical}$	Curve fitting
2	284	0.001	0.114	Accepted	657	0.004	0.075	Accepted
3	262	0.000	0.119	Accepted	604	0.009	0.078	Accepted
4	226	0.000	0.128	Accepted	552	0.001	0.082	Accepted
5	188	0.000	0.140	Accepted	461	0.001	0.090	Accepted
6	159	0.012	0.153	Accepted	304	0.000	0.110	Accepted
7	124	0.016	0.173	Accepted	226	0.000	0.128	Accepted
8	61	0.000	0.246	Accepted	152	0.031	0.156	Accepted
9	32	0.058	0.340	Accepted	69	0.000	0.232	Accepted
10	17	0.000	0.466	Accepted	24	0.000	0.393	Accepted

4.3.5 Move-up delay (MUD)

As mentioned in Section 3.5.2.4, the first vehicle in the queue (i.e. leading vehicle) takes a few more seconds to accelerate after the onset of a red-amber traffic signal. Data on MUD were observed and collected for both 30 mph and 40 mph signalised junctions. The MUD parameter will be used as an input value in the developed model. Table 4.10 provides a comparison of MUD values from previous works and the current study including different investigated factors such as vehicle types and road surface conditions.

As shown in Table 4.10, the range of MUD values was between 1.0 and 6.12 seconds on 30 mph signalised junctions while it was between 1.03 and 5.33 seconds on 40 mph junctions. The data were collected in dry weather conditions (sunny and cloudy conditions). The results also showed consistent mean MUD values for both junctions. It can be indicated that the reason for these figures may be due to the fact that some drivers have longer reaction times for starting to move since the junction area is not clear yet. Another reason, recognised from the site observations, was that some vehicles take a few seconds to restart their engines because of the stop/start system.

Table 4.10: Summary of drivers' MUD for current and previous works

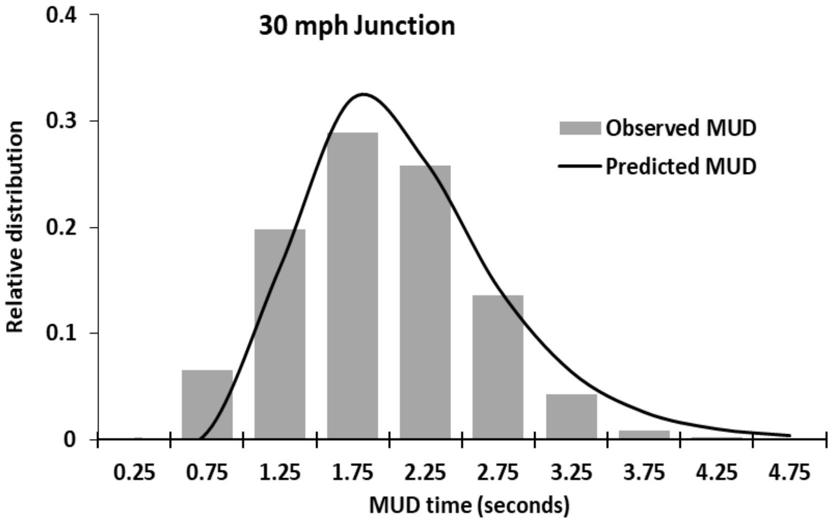
Researcher's name	Study site	Investigated factor	Sample size	μ (sec)	σ (sec)	Min (sec)	Max (sec)
Yousif (1993)	Motorway	Veh. Type (PC)	437	1.8	NA	0.6	6.0
		Veh. Type (HGV)		2.0			
Al-Obaedi (2011)	Motorway	NA	NA	1.8	NA	0.5	6.5
Alterawi (2014)	Shuttle-lane Roadworks (P)	Road Surface (W)	48	2.7	0.8	1.0	4.9
		Road Surface (D)	510	2.0	0.7	0.8	6.2
	Shuttle-lane Roadworks (S)	Road Surface (W)	71	2.6	0.9	1.0	6.4
		Road Surface (D)	411	2.0	0.7	0.8	6.7
Current study	Sig. Junctions (30 mph)	Road Surface (D)	353	2.13	0.70	1.03	5.33
	Sig. Junctions (40 mph)		669	2.04	0.69	1.00	6.12

P: Primary direction D: Dry surface NA: Not available
S: Secondary direction W: Wet surface

As shown in Figures 4.8 for 30 mph and 40 mph signalised junctions, the MUD data were analysed to find out the curve fitting. Table 4.11 includes the K-S statistical summary of goodness of fit. It can be concluded that both fields' data distributions are shown to conform with the lognormal distribution.

Table 4.11: The K-S test summary for testing MUD distribution with lognormal distribution at 30 mph and 40 mph signalised junctions

Approach speed limit	Sample size	D_{Max}	$D_{Critical}$	Curve fitting
30 mph	353	0.094	0.102	Accepted
40 mph	669	0.006	0.074	Accepted



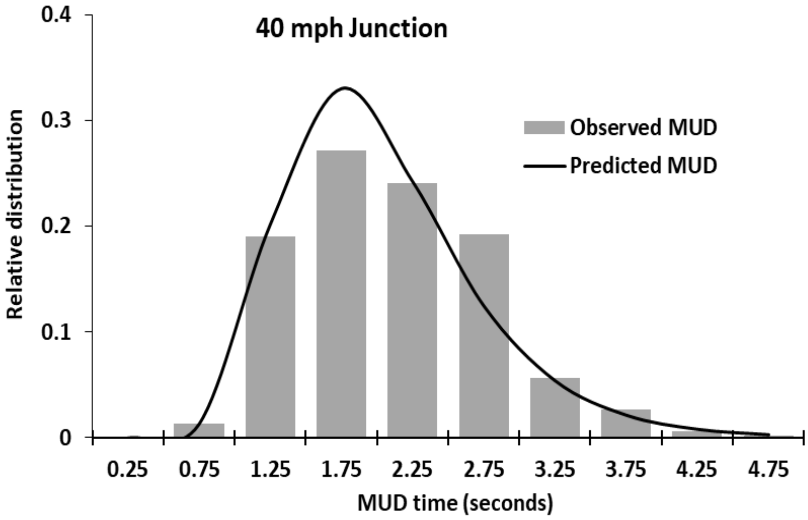


Figure 4.8: Observed and predicted MUD distributions at 30 mph and 40 mph signalised junctions

4.4 Drivers' behaviour in the dilemma zone

4.4.1 Drivers' compliance

The amber signal is designed to warn approaching drivers before the signal indication changes to red. Under the effect of a dilemma zone, a driver has to choose either to stop comfortably in response to the amber/red light or proceed and cross the stopline. Different observed drivers' compliance near signalised junctions can be summarised as presented in Table 4.12.

A total sample size of 1089 vehicles was observed under the effect of the dilemma zone at the sites. The survey sites were classified into two cases, one for a 30 mph signalised junction and the other representing a 40 mph signalised junction. In order to see the significant differences between them, Table 4.13 summarises the observed drivers' compliance after the onset of amber at all survey sites including information on the number of lanes and lane position as described previously in Section 3.6. The findings showed that the percentage of amber crossings was higher on the 40 mph junctions than those on the 30 mph sites. It can be noticed that approximately 65% of drivers (i.e. 443 out of 687) have a tendency to cross the stopline after the onset of amber including running the red signal on 40 mph junctions. In contrast, the overall percentage of crossing vehicles was lower on 30 mph signalised junctions at 44.5% (i.e. 179 out of 402). However, the findings for 30 mph junctions do not replicate the real percentages because there is a large number of 30 mph signalised junctions (145 out of 235 reported by the police with traffic conflicts as discussed in Section 2.4) in Greater Manchester compared to other junctions. In addition, some drivers either arrived late to the stopline after the onset of red or they crossed the stopline at the onset on amber. Therefore, an in-depth investigation is needed to achieve a better understanding of drivers' crossing/stopping behaviour under the effect of various factors such as different traffic flow rates, distance from the stopline, intersection width and all-red period.

Table 4.12: Groups of observed driver behaviour near the signalised junctions

Category	Description	Examples of observed driver behaviour
Amber Light Running (ALR)	Drivers decided to pass through the junction with no more than 3 seconds from the onset of amber.	• Driving at the desired speed close or equal to the speed limit in free-flow conditions. • Unable to stop within 40 m from the stopline in free-flow conditions. • Following the preceding vehicle from the green light in free-flow conditions.
Red Light Running (RLR)	Drivers decided to continue crossing and violate the red light within no more than 3 seconds from the start of red based on field observations.	• Driving at a high speed close to or greater than the speed limit in free-flow conditions. • Increasing his/her speed because of the large gap or space to the preceding vehicle in free-flow conditions. • Follow the preceding vehicle from the amber light in free-flow conditions. • Deciding to violate the red light to avoid waiting and delay.
Amber/Red Light Stopping (ARLS)	Drivers decided to stop comfortably after the onset of amber.	• Complying with the traffic signal, driving at his/her desired speed in free flow conditions and able to stop for amber/red indication.

Table 4.13: Summary of observed ALR, RLR and ARLS near signalised junctions

Approach speed (mph)	Site #	Lane #	ALR	RLR	ARLS	Total observed	Overall crossing (ALR+RLR)
40	**1**	**1**	35	17	21	73	52
		2	80	23	32	135	103
		3	73	28	45	146	101
	2	**1**	43	7	32	82	50
		2	35	7	31	73	42
	3	**1**	19	7	24	50	26
		2	24	7	26	57	31
		3	29	9	33	71	38
	Total		**338**	**105**	**244**	**687**	**443**
30	**4**	**1**	44	13	83	140	57
		2	29	9	53	91	38
	5	**1**	43	2	46	91	45
		2	33	6	41	80	39
	Total		**149**	**30**	**223**	**402**	**179**
Total observed			**487**	**135**	**467**	**1089**	**622**

4.4.2 Factors affecting drivers' compliance during amber and red light signals

4.4.2.1 The effect of distance from the stopline

Driver behaviour has been observed for amber and red light crossings as vehicles move towards a signal junction. At all sites, the distances from the stopline were detected using a video recording method. Figures 4.9 and 4.10 provide the distributions of observed ALR and RLR respectively over different distances from the stopline for both 30 mph and 40 mph junctions.

As shown in Figure 4.9, it can be indicated that most ALR drivers tend to make a GO decision for crossing if they are at a distance 40 m or less from the stopline after the onset of amber at all junctions. Also, there was a remarkable decrease in the percentages of ALR drivers where they were at a distance 40 m or over from the stopline after the signal showing the amber light.

On the other hand, no red signal violation events were seen at distances less than 40 m from the stopline at all survey sites as shown in Figure 4.10. The majority of red light incidents (approximately 45%) was seen at distances from 50-60 m from the stopline at both 30 mph and 40 mph signalised junctions.

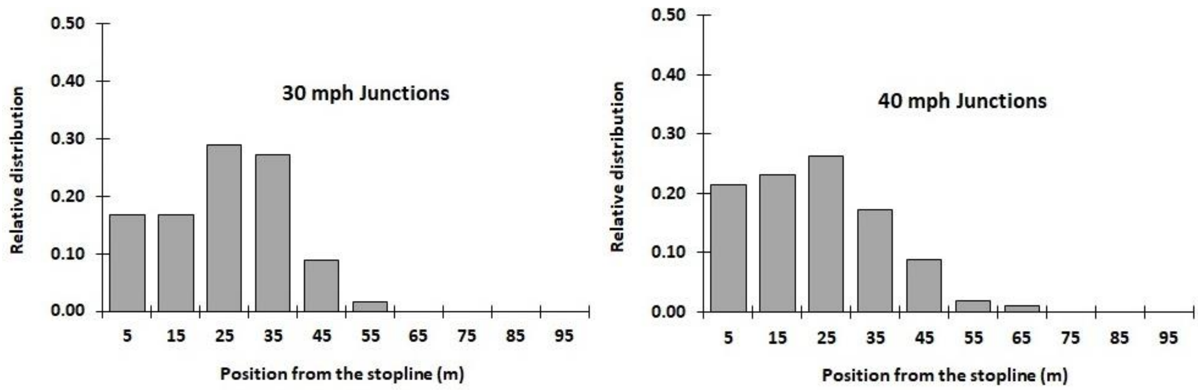


Figure 4.9: Histogram distribution of observed ALR drivers approaching 30 mph and 40 mph signalised junctions by distance from the stopline

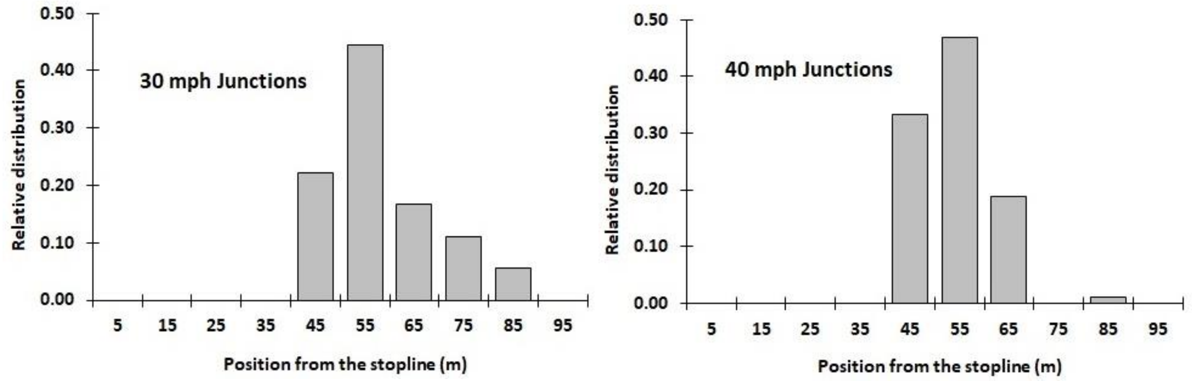


Figure 4.10: Histogram distribution of observed RLR drivers approaching 30 mph and 40 mph signalised junctions by distance from the stopline

Previously, the cumulative distribution of stopping was modelled as a function of distance from the stopline (Baguley and Ray, 1989; Olson and Rothery, 1961; Zegeer and Deen, 1978; Parsonson et al., 1974) and as a function of time to the stopline (Bonneson et al., 2001; Chang et al., 1985; Gates and Noyce, 2010; Rakha et al., 2008; Rakha et al., 2007; and Zegeer, 1977) assuming a constant approach speed. The cumulative percentiles of all crossing (i.e. ALR+RLR) and stopping drivers (ARLS) versus the distances to the stopline position at the onset of amber are shown in Figures 4.11 and 4.12 for 30 mph and 40 mph junctions, respectively. Corresponding to the 10th and 90th percentiles of stops after the onset of amber, the dilemma zone from the stopline lies approximately between 40 and 81 m for 30 mph approaches and from 48 to 85 m for 40 mph approaches. These findings are close to the field data analysed by Maxwell and Wood (2006), as shown previously in Table 2.1.

However, analysis of this observed data will not be undertaken for model development purposes since the dilemma zone is a function of speed, driver reaction time, junction width and amber period. Therefore, the GHM model developed by Gazis et al. (1960) will be used to develop the model assumptions because it is capable of giving a real representation of various speeds and drivers' responses to the signal change particularly after the start of amber.

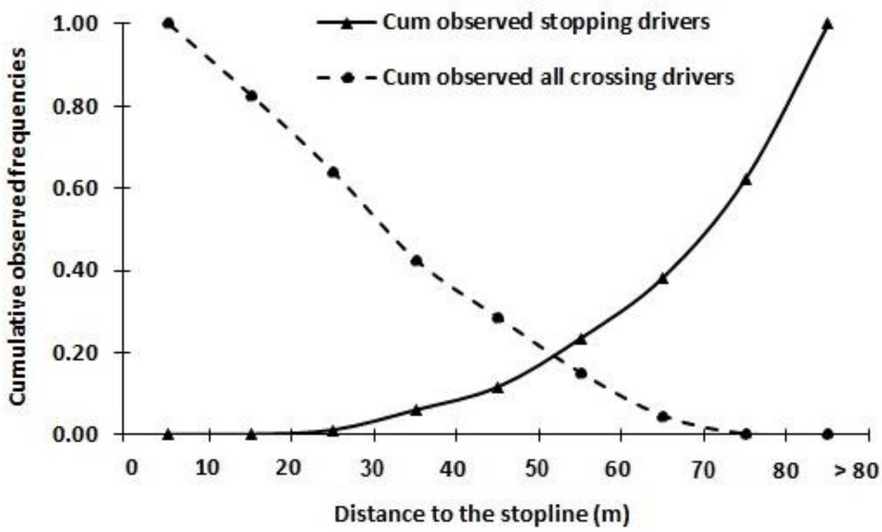


Figure 4.11: Cumulative distribution of observed crossing and stopping drivers approaching 30 mph signalised intersections

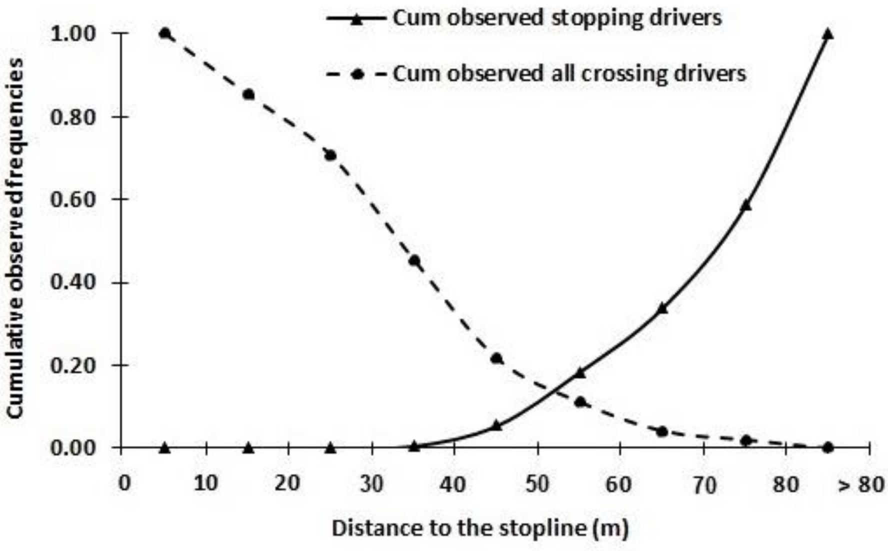


Figure 4.12: Cumulative distribution of observed crossing and stopping drivers approaching 40 mph signalised intersections

4.4.2.2 The effect of speed

Drivers were observed as they moved towards a signal junction from different distances from the stopline when the signal was showing the amber indication. For each vehicle, the distance from the stopline, speed and driver's STOP/GO decision were detected from the video recordings.

According to Baguley and Ray (1989), the 3 seconds amber line distinguishes between the clearance area (i.e. Go Zone to the left) and the comfortable stopping area (i.e. Stop Zone to the right), as shown previously in Figure 2.8. Three categories of drivers (i.e. ALR, RLR, and ARLS) were allocated based on speeds and positions from the stopline as illustrated in Figures 4.13 and 4.14 for data collected from 30 mph and 40 mph junctions, respectively. It can be seen that the observed ALR drivers took 3 seconds or less to cross and clear the junction area safely without running the red signal. Those drivers can be considered under the effect of the clearance zone (i.e. Go Zone) because they were close to the stopline. Other drivers' groups (i.e. RLR and ARLS) were seen at distances 40 m and over from the stopline position at all sites and they were either influenced by the Dilemma Zone or the Option Zone rules.

Due to various types and sizes of vehicles, a wide range of travelling speeds was seen on the approach. Therefore, the GHM model will be used to develop and modify the CARSIM model in order to replicate real drivers' behaviour while approaching the stopline. In order to differentiate between the RLR and ARLS groups, the standard Equations from 2.1 to 2.4, as described previously in Section 2.5.2, with various values for speeds and reaction times will help to predict drivers' STOP/GO decisions after the onset of amber.

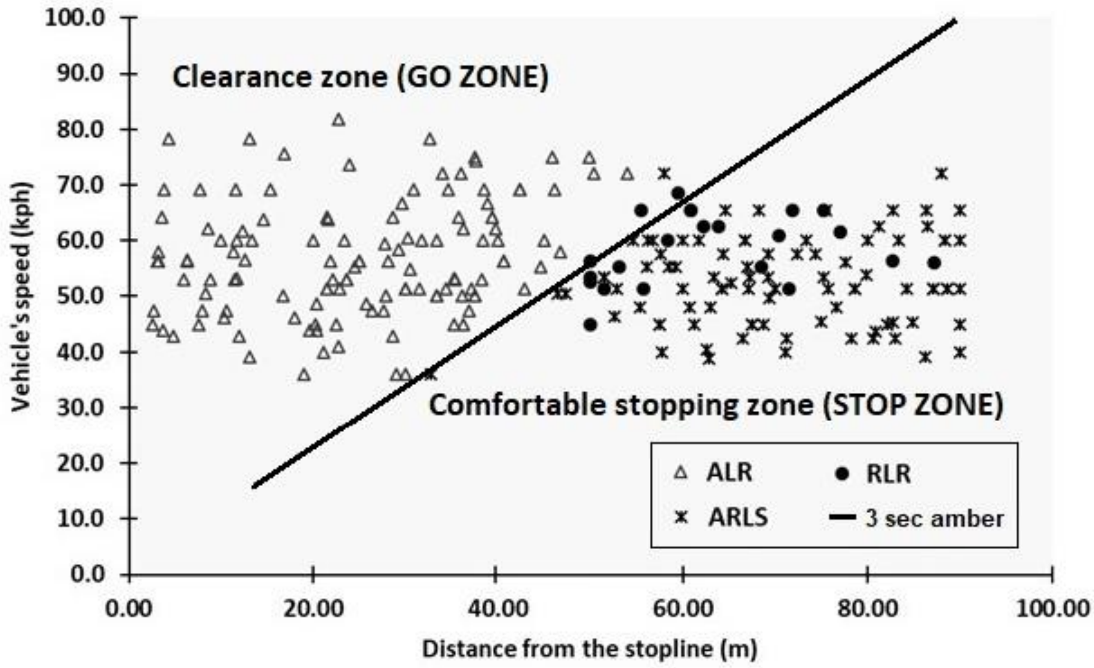


Figure 4.13: Vehicle speed versus distance from the stopline at the onset of amber at 30 mph signalised junctions

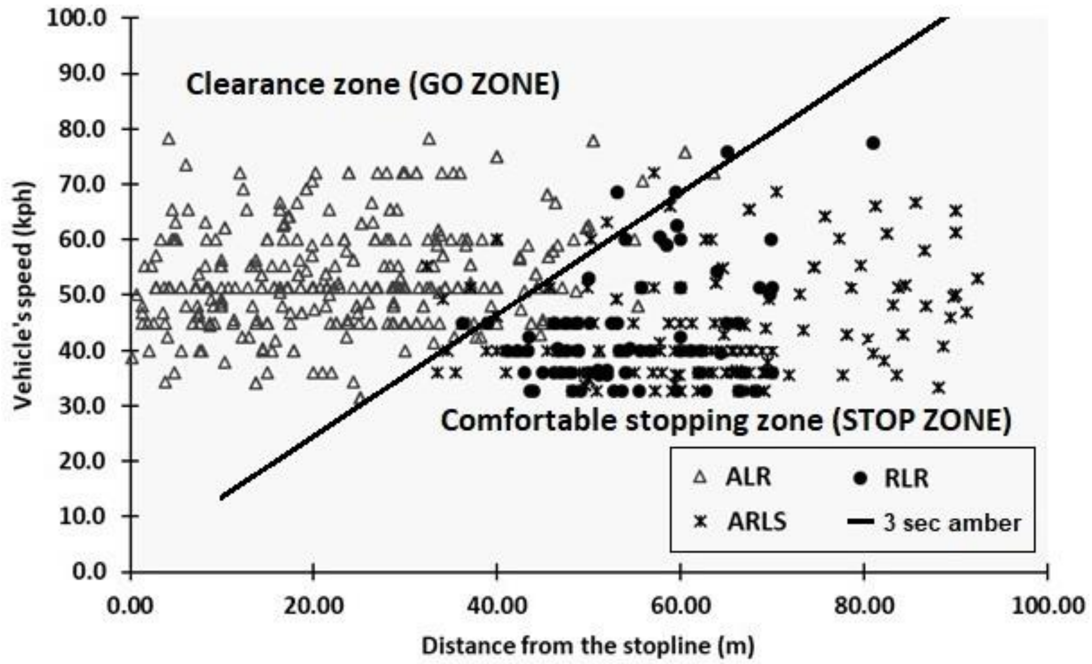


Figure 4.14: Vehicle speed versus distance from the stopline at the onset of amber at 40 mph signalised junctions

4.4.2.3 The effect of time headway

Because of the difficulty of measuring time to collision and spaces between successive vehicles at the onset of amber from video records, time headways were measured for data analysis. Time headway is one of the surrogate safety measures that plays a significant role in traffic safety and performance. Shorter headways between two successive vehicles may increase the risk of tailgating or rear-end collisions.

The Department for Transport (2015) stated that a driver has to maintain at least a 2-second gap for following the preceding vehicle at any travelling speed which is known as the '2-second rule' or the vehicle is too close to the preceding one (i.e. the time headway is equal to or less than 1 second). Vogel (2002) suggested that a 6-seconds threshold can be used to recognise vehicles travelling in a platoon. More specifically, vehicles were assumed to be in free-flow conditions if the threshold was greater than 6 seconds and, while they travelled in a platoon, if the time headway was equal to or less than 6 seconds. This value was adopted in this study to investigate how the following headway might affect a driver's STOP/GO decision at the onset of amber.

Time headways with the preceding and following vehicles were detected after assigning the vehicle's position at the onset of amber using the video playback method. Only vehicle time headways up to the 6 seconds threshold were chosen for this analysis. The next step was to find the relative distribution of time headways for each driver category (i.e. ARL, RLR and ARLS). Finally, the results are summarised in Tables 4.14 and 4.15 for 30 and 40 mph approaches, respectively. The results also include drivers' tailgating behaviour and how this influences their STOP/GO decisions.

On 30 mph approaches, it can be indicated that more than 75% of ALR drivers have headway times of 2 sec or more with the following vehicles. In addition, around 54% of ALR drivers contravened the '2 second rule' and maintained following headways < 2 sec with the preceding vehicles which is considered unsafe and might increase the probability of rear-end collisions. This can be highlighted as aggressive driving behaviour. This evidence was found by the National Cooperative Highway Research Program (2003) which demonstrated that aggressive drivers are more likely to accelerate near signalised junctions. On the other hand, ARLS drivers tried to keep a safe following distance with the leading and following vehicles. Other results showed the effect of following and preceding vehicles on RLR drivers' behaviour. For instance, only 10% of observed RLR maintained short headways < 2 sec with the leading vehicles and no close-following behaviour with the following vehicles. However, more than 90% of RLR drivers have large headways ($\geq$ 2 sec) with the following and preceding vehicles. This figure indicates that RLR drivers seem to be more aggressive than ALR drivers and have a tendency to violate the red light when they have large headways with the preceding vehicles.

Table 4.14: Effect of time headway on drivers' decisions on 30 mph approaches at the onset of amber

Driver compliance	ALR	RLR	ARLS
Percentage of drivers who had a headway of less than 2 sec with the preceding vehicle.	54.4%	10.0%	1.5%
Percentage of drivers who had a headway of 2 sec or more with the preceding vehicle.	45.6%	90.0%	98.5%
Percentage of drivers who had a headway of less than 2 sec with the following vehicle.	24.4%	0.0%	0.0%
Percentage of drivers who had a headway of 2 sec or more with the following vehicle.	75.6%	100%	100%

On 40 mph approaches, it can be highlighted that more than 70% of all driver categories have headway times of 2 sec or more with the following vehicles, as shown in Table 4.15. Moreover, approximately 31% of ALR and 36% of RLR drivers maintained short headways < 2 sec with the preceding vehicle at the onset of amber which is considered to be a risky behaviour that might result in rear-end collisions. This gives an indication of drivers' aggressiveness arriving at the intersection area. This outcome was found to be in agreement with the findings of the National Cooperative Highway Research Program (2003) mentioned previously regarding aggressive drivers. The findings with the following vehicles showed that 29% of ALR and 22% of RLR drivers were affected by headways < 2 sec.

Table 4.15: Effect of time headway on drivers' decisions on 40 mph approaches at the onset of amber

Driver compliance	ALR	RLR	ARLS
Percentage of drivers who had a headway of less than 2 sec with the preceding vehicle.	31.2%	36.2%	19.5%
Percentage of drivers who had a headway of 2 sec or more with the preceding vehicle.	71.8%	63.8%	80.5%
Percentage of drivers who had a headway of less than 2 sec with the following vehicle.	29.0%	22.2%	15.3%
Percentage of drivers who had a headway of 2 sec or more with the following vehicle.	71.0%	77.8%	84.7%

Overall, it can be summarised that drivers are mostly affected by time headways with the preceding vehicles. In addition, it can be indicated that most of ARLS drivers (> 85%) were not affected by the close-following behaviour with the vehicles following them at 30 and 40 mph junctions. The ALR and RLR drivers continue crossing the stopline after seeing the amber signal because they may not be able decelerate in the Go Zone or the Dilemma Zone based on their positions. According to Garber and Hoel (2009), time headways can be defined as the reciprocal of flow. Therefore, it is necessary to investigate the effect of flow rates on drivers' decisions as shown in the next section.

4.4.2.4 The effect of traffic flow rate and vehicles' composition

To get a better understanding of the drivers' STOP/GO decisions, the effect of flow rates and traffic composition were investigated in the current study. Traffic flow was reported from the video recordings in this study in order to recognise the variation in flow level for each lane over a 1-hour period. Additionally, the percentages of HGVs and RLR events were detected for data analysis. It was found that 90% of RLR were passenger cars including light goods vehicles (such as vans) after the signal showed an amber indication. The correlation test was conducted to find out the strength of the linear association between the aforementioned variables (i.e. RLR% with flow rate and RLR% with HGVs%) in terms of coefficient ±1 (where +1 means a positive relationship, -1 means a negative relationship, and 0 means no significant association between variables). As shown in Figure 4.15, a value of -0.52 was accepted as a negative association between RLR% and average flow. This is perhaps true because of fewer safe spacings on the road throughout the higher flow rates which might eliminate the RLR incidents.

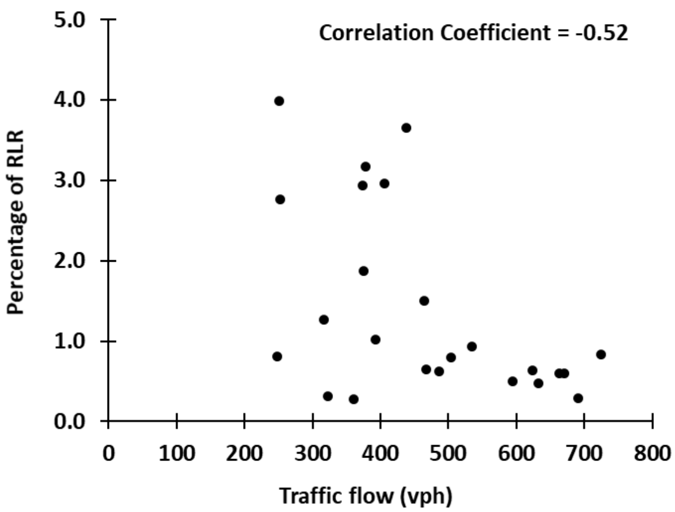


Figure 4.15: Relationship between the observed RLR% and average flow rate

In Figure 4.16, a weak association between RLR% and HGVs% was seen at a value of +0.18. It can be suggested that the RLR are random events and might be affected by the HGVs% after the onset of amber. However, the number of RLR reduces as the HGVs% increases. This is due to the fact that heavy goods vehicles have longer lengths and occupy more space than cars. In addition, these vehicles decelerate faster than cars after the onset of amber because they are moving at lower speeds and have lower acceleration rates.

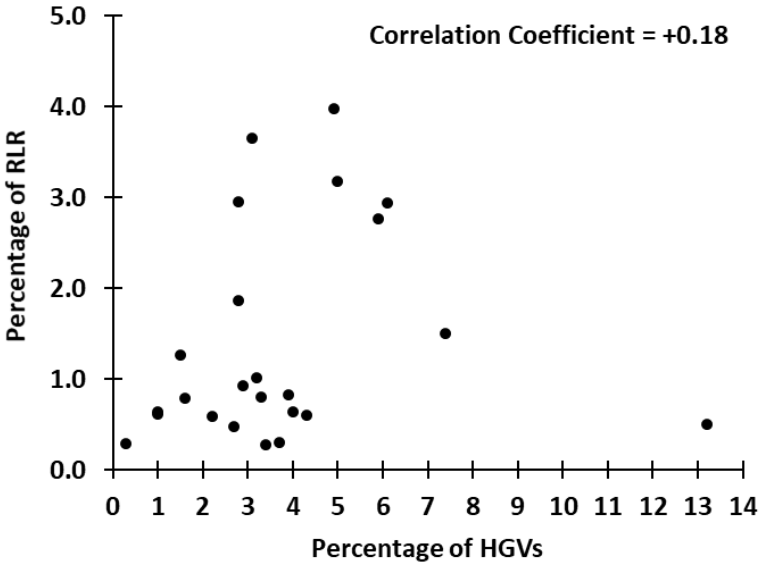


Figure 4.16: Relationship between the observed RLR% and HGVs%

4.4.2.5 The effect of intersection geometry

Five different signalised intersection widths were used to investigate the effect of intersection width on a driver's decision after seeing the amber lights. According to Gazis et al. (1960), the intersection width (w) can be defined as the distance between the stopline and the far edge of the conflicting traffic lane as shown in Figure 4.17. In this study, the width was measured with the aid of the Google Maps distance measuring tool because of the difficulty of measurement in the field due to safety related issues.

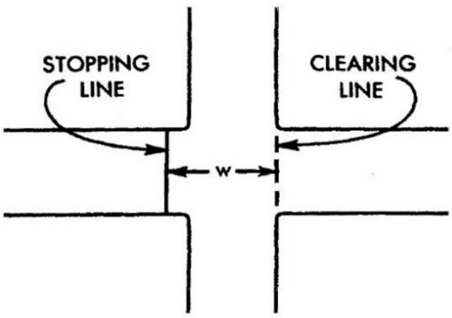


Figure 4.17: Intersection width (adapted from Gazis et al. (1960))

Figure 4.18 illustrates the proportions of ALR, RLR and ARLS versus intersection widths from all site observations. It is clear that the ALR and RLR events increase as the intersection width decreases (i.e. as shown at Site #1). More specifically, a driver tends to cross the stopline after the onset of amber if the intersection width is short. However, the number of RLR crossings on other junctions (Sites #2 to #5) were lower than those observed at Site #1. This result might be affected by the traffic flow rate and the type of signal settings (i.e. FT or VA signals) as well as the posted speed limit on the road.

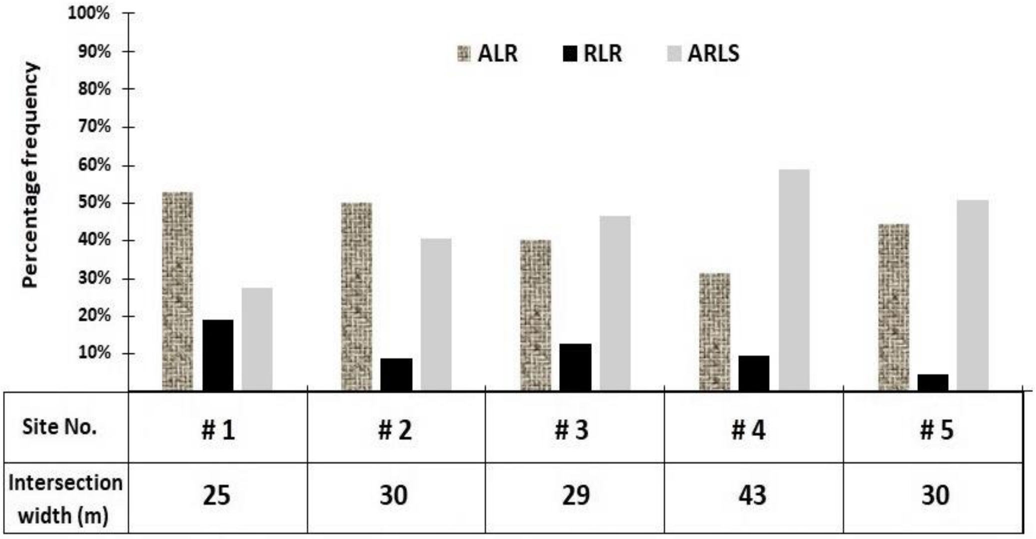


Figure 4.18: Observed driver compliance versus intersection width

Finally, the number of lanes was surveyed in the current study and it was found that an increase in the number of lanes may increase the probability of signal violations. The frequencies of RLR on 3-lanes approaches (Sites #1 and #3) was higher than for other approaches as presented previously in Table 4.13. This is reasonable since more lanes means more vehicles approaching an intersection during amber and red aspects. This fact has been recognised before by Papaioannou (2007) and Yan et al. (2007).

4.4.2.6 The effect of traffic light setting

The frequency of RLR incidents might be increased by inappropriate signal timing (Federal Highway Administration, 2005). The effect of type of traffic signal on the occurrence of red signal violation was examined by Mohamedshah et al. (2000). The researchers found that intersections controlled by Vehicle-Actuated signals (VA) have an increased incidence of RLR by 35-39%. As shown in Table 4.16, it can be seen that the proportion of RLR at Site #4 was higher than that at Site #5, given that both sites have a 30 mph legal speed limit and have the same number of lanes. This gives an indication that VA signals may perhaps surprise drivers and they consequently fail to comply with the amber followed by the red. In contrast to 30 mph approaches, the percentages of RLR on 40 mph approaches was different. Site #1 showed higher RLR% compared to Site #2. This can be explained by an understanding of the fact that a long red phase may increase RLR frequency at FT signal junctions. This finding is also in agreement with the investigation of the Federal Highway Administration (2000). However, Site #3 (which is controlled by VA signal) showed higher RLR% than Site #2. This is because it has 3-lanes which may mean that it has higher traffic flows.

Table 4.16: Summary of traffic signal timing at the survey sites measured in (sec)

Site #	Signal type	Cycle time		Green period		Red period		Amber period*	Red-amber period*	RLR %
		Min	Max	Min	Max	Min	Max			
1	FT	72	72	33	33	34	34	3	2	19.3
2	FT	93	93	42	42	46	46	3	2	9.0
3	VA	94	124	63	87	26	32	3	2	12.9
4	VA	77	114	46	78	24	31	3	2	9.5
5	FT	68	68	35	35	28	28	3	2	4.7

FT: Fixed-Time signal
VA: Vehicle-Actuated time signal
* The observed 3-sec amber and 2-sec red-amber are the recommended periods advised by the UK standards (2016) in all cases.

Furthermore, the traffic signal cycle has an intergreen time of not less than 5 seconds as stated by the Department for Transport (2006b). A practical method has been proposed by the Department for Transport (2006c) (see Section 2.3.1) to estimate intergreen time theoretically for two conflicting flows at a typical cross junction. However, two data observers used stopwatches to estimate the intergreen time at the site. The observed intergreen was computed as the difference between the time that a traffic light showed amber to a traffic stream on one approach and the time that a signal showed a green light to conflicting traffic. Both observed and computed intergreen times for each survey site are presented in Table 4.17. Following the design procedures established by the Department for Transport (2006c), the observed intergreen times were longer than the computed values at all sites. According to Kennedy and Sexton (2009), longer intergreen time results in an increase in the frequency of RLR. This figure was recognised at Site #1 which reported higher RLR% than other locations.

Table 4.17: Summary of observed and computed intergreen and all-red periods

Site No.	Signal type	Observed intergreen time (sec)	Observed all-red period (sec)	Min intergreen time computed from DS (DfT, 2006c)	Min all-red period computed from DS (ITE, 2015)	RLR %
1	FT	13 ✓	8 ✓	7	1.86	19.3
2	FT	11 ✓	6 ✓	8	2.31	9.0
3	VA	10 ✓	5 ✓	6	2.85	12.9
4	VA	6 ✓	1 ×	6	3.66	9.5
5	FT	9 ✓	4 ✓	5	3.52	4.7

FT: Fixed-Time signal
VA: Vehicle-Actuated time signal
DS: Design standards
RLR: Red-light-running
DfT: Department for Transport (2006c)
ITE: Institute of Transportation Engineering (2015)
(×) : not following the DS
(✓): following the DS

On the other hand, it is necessary to examine the effect of all-red period on the frequency of RLR incidents. Previous work carried out by Retting and Greene (1997) showed that the all-red period is a function of intersection width and desired speed. The Institute of Transportation Engineering (2015) introduced design standards to calculate the length of all-red period. The mathematical equations include aforementioned parameters (i.e. intersection width and speed) and a typical length of vehicle (20 ft or around 6 m) to calculate the length of the all-red period for a signal-controlled intersection as follows:

$$\textit{All-red period} = \frac{w+L_v}{v} \qquad \textit{Equation 4.2}$$

$$\textit{Or} = \frac{P+L_v}{v} \qquad \textit{Equation 4.3}$$

where

v is the speed limit or approach speed (ft/sec).

L_v is the typical length of vehicle.

w is the junction width measured in ft from the upstream stopline to the downstream extended edge of the pavement.

P is the junction width measured in ft from the near-side stopline to the far-side of the farthest conflicting pedestrian crosswalk along an actual vehicle path.

According to the Institute of Transportation Engineering (2015), Equation 4.3 has been used to calculate the all-red interval if there is a pedestrian crosswalk in the approach such as at Site#5 used in the current study. This additional interval provides safe crossing time to vehicles and to clear the intersection area as well as the pedestrian crosswalk before the conflicting vehicles start their movements. The observed and recommended all-red periods given by the Institute of Transportation Engineering (2015) for each site are reported in Table 4.17. It can be realised that the number of signal violations is largely affected by the length of the all-red and intergreen periods as illustrated in Figures 4.19 and 4.20, respectively. Positive correlations (+0.81) were shown between the RLR% and the all-red times as well as intergreen periods. Finally, this can be explained by the fact that some drivers tend to use the intergreen period and continue crossing the stopline before the green phase is shown to the conflicting traffic as recognised by Kennedy and Sexton (2009).

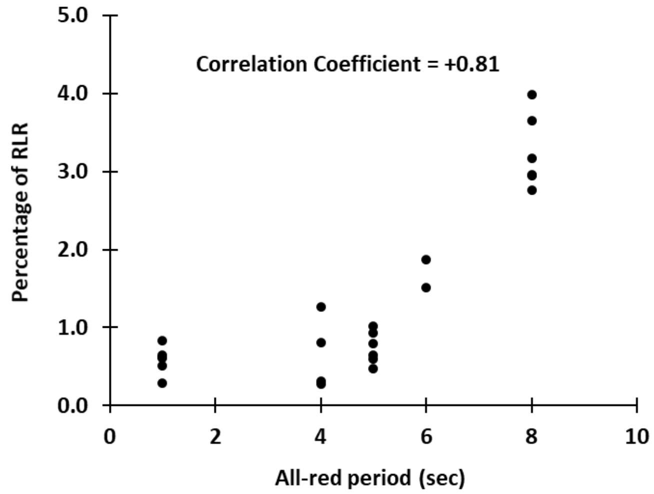


Figure 4.19: Relationship between the observed RLR% and all-red period

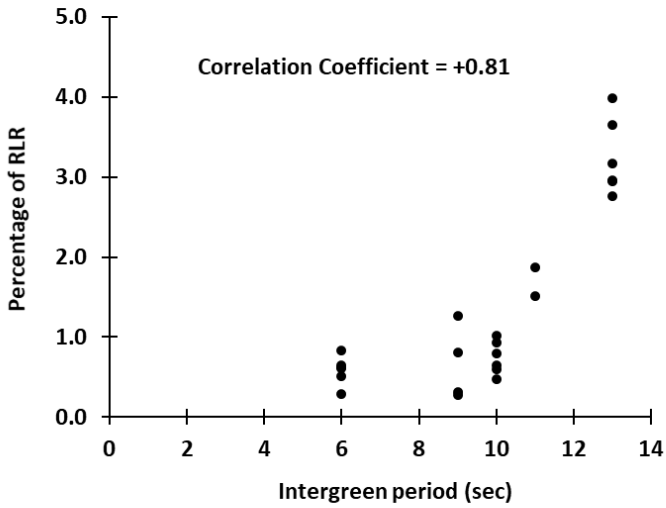


Figure 4.20: Relationship between the observed RLR% and intergreen period

4.5 Summary

This chapter presents the analysis of the observed traffic data at the survey sites. Because of some limitations in the existing micro-simulation models such as PARAMICS and AIMSUN (see Chapter Two, Section 2.7.4), data regarding drivers' STOP/GO decisions, when they are facing amber and red near a signalised intersection, was collected. The collected and analysed data will be used to develop, calibrate and validate the new micro-simulation model for the current work. The analysed data can be listed below:

1. Different data sets were collected from five signalised intersections and analysed based on over 18 hours of video recordings including details of traffic flow profiles, arrival headways, move-up time and move-up delay.
2. Vehicle characteristics were analysed such as types and lengths. The passenger car data were adopted from previous work of Alterawi (2014) to calculate the cumulative probability distribution of car length for the developed model. The HGVs types and lengths were collected from the survey sites and accompanied with the standard lengths presented by the UK government and manufacturing companies' websites.
3. Driver compliance with the amber and red signals was observed along the approach to the stopline from video recordings. The observations also included distances from the stopline, speeds, flow rate, traffic signal timings and headways with the preceding and following vehicles.

Different factors that may affect RLR frequency were investigated such as traffic flow level, HGVs% and traffic signal type and timing settings. The collected and analysed data will be used as inputs to develop the model assumptions regarding driver behaviour approaching a signalised junction. The next chapter describes the model specification and development.

CHAPTER FIVE: MODEL SPECIFICATION AND DEVELOPMENT

5.1 Introduction

Since standard micro-simulation models (for example, PARAMICS) cannot accurately model the traffic conflicts at signalised intersections arising from behaviour such as red light violations and close following, an alternative tool for studying these conflicts is required. The microscopic simulation technique was adopted in the current work because it is able to represent the interactions between individual vehicles.

The new micro-simulation model needs to be programed using an appropriate programing language. The FORTRAN-95 language was chosen because it is commonly used in many applications by engineers and transport researchers. It includes a number of logical and statistical statements that could help in executing modelling tasks and provides a graphical user interface of driving behaviour in the real world. In addition, it presents the changes arising from the drivers' decisions before and after any design interventions.

The structure of the proposed new micro-simulation model is a combination of the CAR-following SIMulation (CARSIM) model and the Gazis-Herman-Maradudin (GHM) model. In this case, details of drivers' reaction times and vehicle characteristics such as types and physical dimensions are necessary for developing the model. In addition, the proposed model requires information about the approach length and the traffic signal settings that will control the traffic movement on the approach. These data were collected from the survey sites and analysed in order to build the model.

5.2 The original CARSIM model structure

The original car-following rules in the CARSIM model were developed by Benekohal (1986) and have been used by many researchers (see for example: Yousif (1993), Al-Obaedi (2011), Al-Jameel (2012), Alterawi (2014) and Nassrullah (2016)) to develop micro-simulation models. The car-following rules were developed based on the assumption that a vehicle travels at an acceleration/deceleration rate and a safe distance from the leading vehicle. Figure 5.1 illustrates the structure of the original car-following sub-model. The car-following rules represent free-following and congestion situations on normal roads and highways that are not controlled by traffic signals. Firstly, vehicles will be generated in the simulation by giving random values of speeds and lengths in addition to drivers' reaction times based on a specific data distribution. Following that, vehicles will be entered into the simulated road section and the longitudinal positions and speeds of those vehicles in correspondence with the leading vehicle will be updated after determining the acceleration/deceleration rates at every scanning time (Δt).

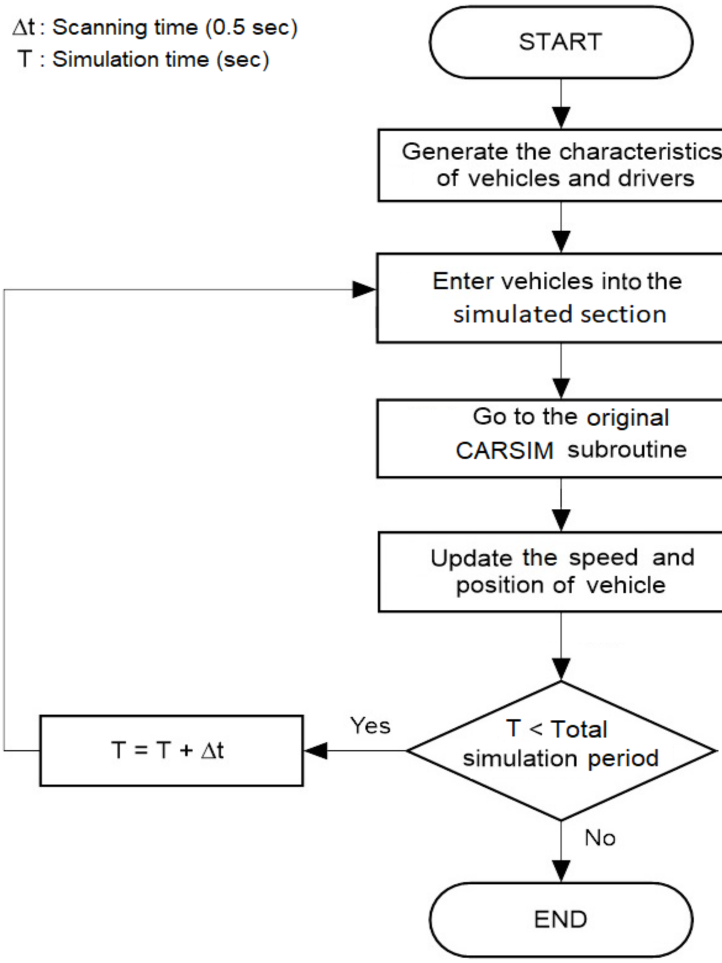


Figure 5.1: The structure of the original CARSIM model (adapted from Benekohal (1986))

Since drivers' decisions and responses to signal changes are affected by the dilemma zone rules that take effect on the approach to a signalised junction after the onset of amber, the car-following rules (developed by Benekohal (1986)) have been adopted in the current work. However, Benekohal (1986) did not take into consideration the dilemma zone rules in the CARSIM model. Therefore, it is necessary to modify the original car-following rules by adding the GHM model that was developed by Gazis et al. (1960). The next section describes the structure of the newly developed model that is suggested in the current study to replicate drivers' behaviour following the onset of amber.

5.3 The structure of the newly developed micro-simulation model

Since the current work is concerned with the arrival of vehicles at a traffic signal junction, the structure of the newly developed micro-simulation model to represent drivers' responses following the onset of amber is shown in Figure 5.2. The proposed model consists of three sub-models:

- The Green-CARSIM sub-model: to model the interaction between successive vehicles travelling in the same lane during the green phase in normal and congested flow conditions based on the original CARSIM model.
- The STOP-GO sub-model: to model drivers' responses to signal changes and STOP/GO decisions during the amber period (usually 3 sec) and after the onset of red using the algorithms of the GHM and original CARSIM models.
- The Ready-to-GO sub-model: to model drivers' behaviour within the 2-sec red-amber period before the onset of green using the original CARSIM model.

Full details of the structure of each sub-model can be found in Sections 5.6 and 5.7. As shown in Figure 5.2, vehicles types and lengths as well as drivers' reaction times and desired speeds will be defined at the beginning of the simulation process. Next, the generation of a complete traffic cycle is a very important aspect when representing the stop/go situations at a traffic signal junction. Based on the traffic light signal (i.e. green, amber or red), the arrival of vehicles and traffic signal timing will be updated for each scanning interval (Δt) within the simulation.

Since the accuracy of simulation models is a crucial issue and depends upon the accurate values of input parameters, using a short scanning interval (Δt) is more useful for updating the simulation (Al-Obaedi, 2011). According to Gipps (1981), a scanning time of 0.5 sec can be considered as the minimum value for the driver's reaction time. Based on previous works carried out by Al-Jameel (2012), Al-Obaedi (2011) and Yousif et al. (2014), a 0.5 sec scanning time has been adopted in the current study.

The next sections describe in detail the generation of traffic light signals, vehicle and driver characteristics as well as the sub-models' structures.

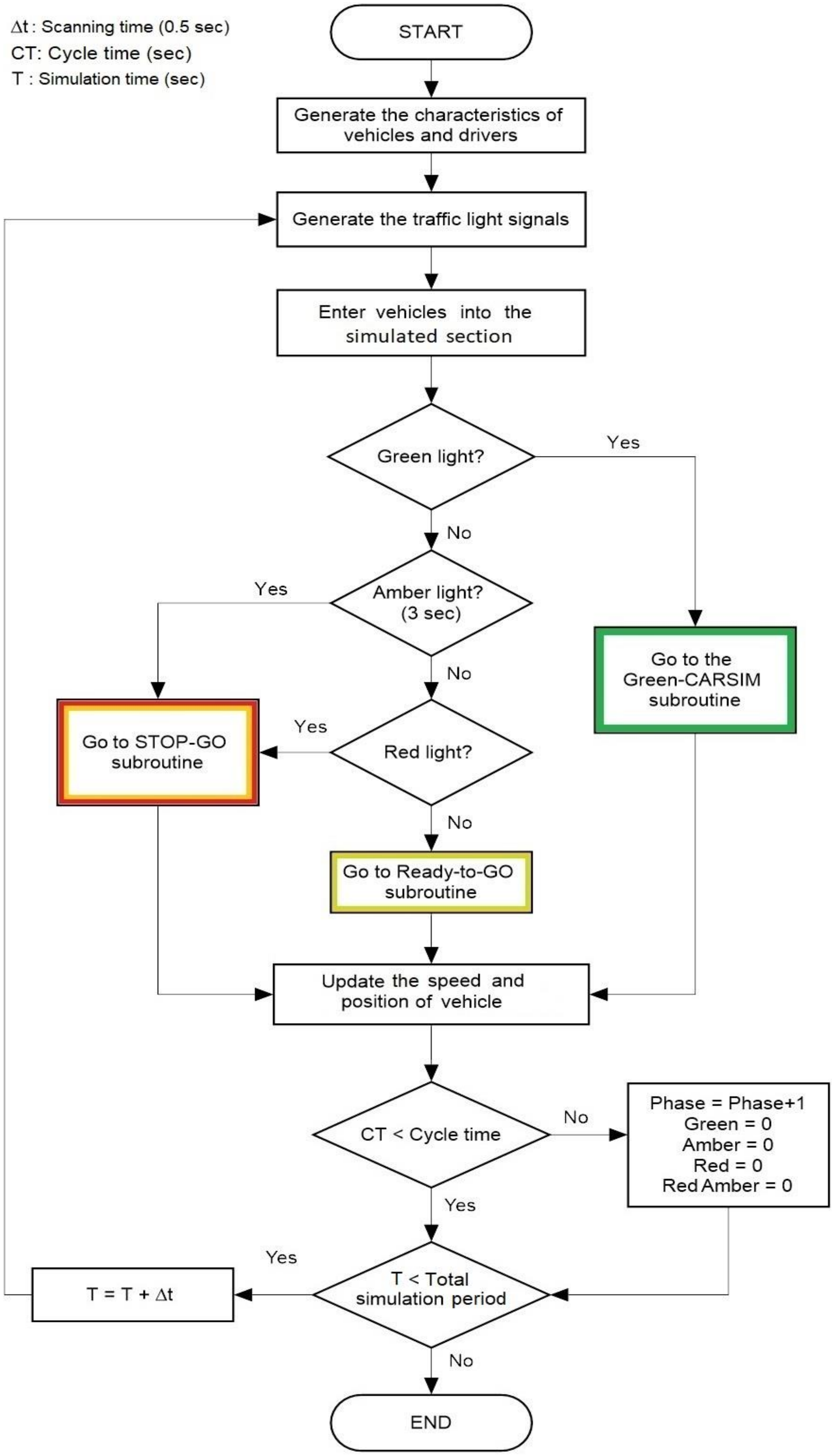


Figure 5.2: The structure of the newly developed micro-simulation model

5.4 Definitions of vehicle and driver characteristics

5.4.1 Vehicle type, length and desired speed

First of all, it is necessary to generate different vehicle types before entering them into the simulated approach. A distance of 1500 m from the stopline was considered sufficient for this purpose. Random numbers will be generated from a uniform distribution to assign vehicle types and speed. As explained previously in Section 4.3.1, a value of 5.6 m was used to differentiate between heavy goods vehicles (HGVs) and passenger cars (PCs) including light goods vehicles. A vehicle type of HGV will be assigned if the generated random number ($R_{v.type}$) is equal to or less than the proportion of HGVs in the traffic flow, otherwise it can be considered as a PC vehicle. Figure 5.3 illustrates the step of random generation for vehicle type and length.

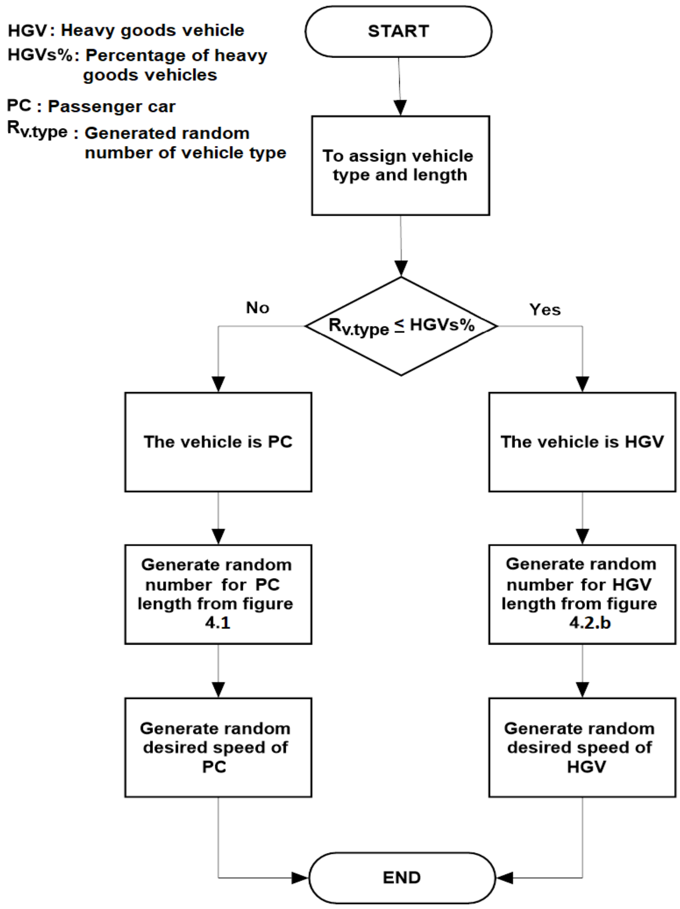


Figure 5.3: Generation of vehicle characteristics subroutine

On the other hand, the desired speed can be defined as the speed chosen by a driver travelling on the carriageway under free-flow conditions without constraints or delays from other road users. According to Roess et al. (2004), the desired speed might be influenced by driver characteristics such as age and gender, vehicle design and performance and finally the road characteristics such as lane position and existence of vertical and horizontal curves. For the purpose of the model development, the desired speed for each vehicle should be generated randomly from the cumulative normal distribution after assigning the vehicle type.

5.4.2 Vehicle arrival time headway

Vehicles arrive with different time gaps passing over a reference line on the carriageway approaching a traffic signal. Time headway distribution will be used to generate random numbers (based on the traffic flow rate) to represent the arrival of vehicles into the simulated road. As explained previously in Section 4.3.2, the shifted negative exponential distribution was used to generate vehicle time headways for free-flow conditions up to 400 vph, and the negative exponential distribution for generating time headways from vehicular flows greater than 400 vph.

5.4.3 Buffer space

The safe buffer space between stopped vehicles was measured and collected from the survey sites as illustrated previously in Figure 3.3. It can be assigned to each vehicle by using a random generating number from the lognormal distribution as described previously in Section 4.3.3 and as shown in Figure 4.5.

5.4.4 The Move-up delay (MUD)

The MUD can be defined as the time taken by the first vehicle in the queue to move after the signal shows the green light (Michael et al., 2000). The required data for MUD were collected and analysed in order to be used as input data in the model development stage. The results showed that the MUD data can be generated randomly from the lognormal distribution and assigned for each driver as described previously in Section 4.3.5.

5.4.5 Drivers' reaction times

Driver reaction time is a major component of the car-following model and is affected by the stopping sight distance. It can also be defined as the brake reaction time (Johansson and Rumar, 1971). Many researchers have investigated the driver perception-reaction time under different weather and light conditions. On the other hand, the driver brake time might be influenced by the driver experience, age, gender, distance to the obstacle object, and the physical and psychological conditions of that a driver.

Previous works have investigated the values for reaction time by collecting data from video recordings. More recently, many drivers (of various ages) have participated in driving tests using driving simulators and digital recording equipment to measure the driver response to any changes in the road environment, for example crossing pedestrians, traffic light changes and the presence of red light cameras. Table 5.1 and Figure 5.4 provide a summary of studies that have been conducted to measure drivers' brake reaction times for surprised and alerted conditions.

Table 5.1: Range of drivers' reaction times in seconds from previous works

Researcher's name	Sample size	Mean μ and standard deviation (σ)	Type of conditions
Gazis et al. (1960)	87	1.14 (0.32)	Surprised
Johansson and Rumar (1971)	321	0.90	Surprised
		0.69	Alerted
Sivak et al. (1982)	1644	1.21 (0.63)	Surprised
Wortman and Matthias (1983)	692	1.30 (0.60)	Surprised
Chang et al. (1985)	1614	1.30 (0.74)	Surprised
Olson and Sivak (1986)	64	1.60	Surprised
		1.15	Alerted
Lerner et al. (1995)	56	1.51 (0.39)	Surprised
Goh and Wong (2004)	222	0.84 (0.23)*	Alerted
	142	0.87 (0.22)**	Alerted
Rakha et al. (2007)	351	0.74 (0.19)	Alerted
Gates et al. (2007)	898	1.0	Alerted

* Data were taken from cross signalised junctions.
** Data were taken from T signalised junctions.

In this study, reaction times during the green period were represented by the values in the surprise conditions. However, drivers' reaction times in the alert situations were adopted for when the amber light comes on since the drivers would be alerted to stop for the red light or continue crossing the stopline on junctions controlled by fixed-time (FT) settings. For simulation purposes, it was assumed that 50% of drivers had surprised reaction times at the onset of amber on junctions controlled by the vehicle-actuated (VA) settings.

Following previous works of Al-Obaedi (2011), Al-Jameel (2012), Alterawi (2014) and Nassrullah (2016), the drivers' reaction times defined by Johansson and Rumar (1971) will be used in this study because they were measured in both alerted and surprised conditions in contrast to other research shown in Table 5.1. Random numbers for reaction times will be obtained from Figure 5.4 (consistent with the cumulative distribution) to represent alerted and surprised responses in the simulation model. It is worth mentioning that the data presented by Olson and Sivak's (1986) study were not considered since it replicates the drivers' reaction times when travelling on a two-lane rural road located on a vertical crest curve which may not be applicable for traffic signals.

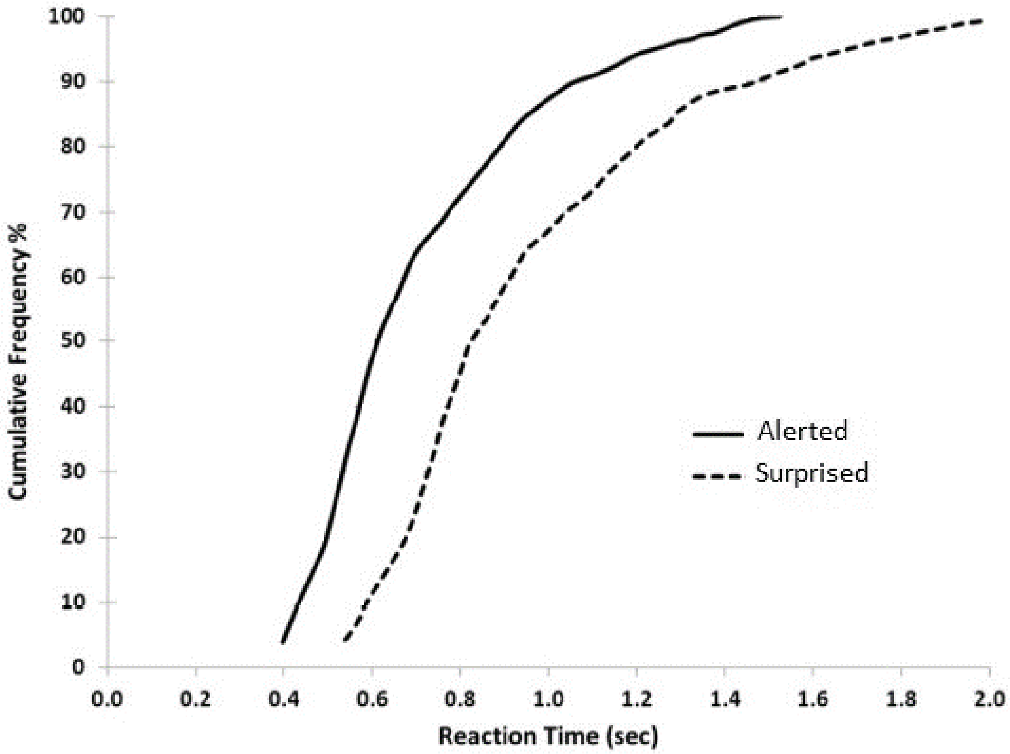


Figure 5.4: Cumulative distribution of drivers' reaction times for alerted and surprised conditions (Johansson and Rumar, 1971)

On the other hand, researchers reported that a factor of 1.35 was adopted to convert from surprised to alerted condition. Moreover, it has been considered that alerted conditions occur in congested flow conditions where traffic density exceeds the value of 37 veh/km. With this in mind, the driver is assumed to be surprised if the traffic density is equal to or lower than 37 veh/km following the previous studies (Al-Jameel, 2012; Al-Obaedi, 2011; Alterawi, 2014; Benekohal, 1986; Nassrullah, 2016 and Yousif, 1993).

5.4.6 Drivers' responses to the signal change

Driver response to the amber/red signal at the onset of amber is a very important factor that might reflect signal violation behaviour. Based on drivers' reaction times in the surprised conditions shown in Figure 5.4, drivers' responses were divided into two groups. The first set of responses represents drivers who take 1 second or less (equivalent to 70% of drivers) to comply with the signal change and be able to stop after seeing the amber indication. The second group represents drivers with a higher reaction time (greater than 1 second), who would fail to comply with the signal change. Those drivers (i.e. group 2) will be able to run either the amber light or both the amber and red signals depending on their speeds and distances from the stopline if they satisfy the following conditions:

1. The driver of vehicle ahead does not decide to stop.
2. Drivers (i.e. group 2) are able to cross and clear the junction area within the intergreen period.

This classification was used to distinguish between drivers' compliance with the amber/red signal and will be described later in this chapter.

5.5 Modelling traffic light signals

One of the main sections in the micro-simulation program is the traffic signal lights. The cycle time can be defined as the sum of the green, amber (3 sec), red, and red-amber (2 sec) times. Modelling traffic lights signals is an important aspect in the simulation process. Three out of the five visited sites were FT signal-controlled junctions (i.e. constant cycle time length). The other two junctions were controlled by VA signal settings. The maximum and minimum green times for VA signals will be added into the simulated road section. The green time will be extended if there are many vehicles passing over the detection area before crossing the stopline. However, the traffic lights will change from green to amber if there are no vehicles on the detectors and the green time exceeds the maximum green period. On the other hand, the all-red period was collected from the site and added as input data into the simulation model. This period provides a safe crossing for the ALR and RLR drivers and clears an intersection area before the conflicting flows start their movements. Finally, the intergreen period will be computed within the simulation. This parameter (i.e. intergreen period) is a significant factor in determining a driver's decision at the onset of amber. Figures 5.5 illustrates the algorithm of operation for traffic lights for FT and VA signals.

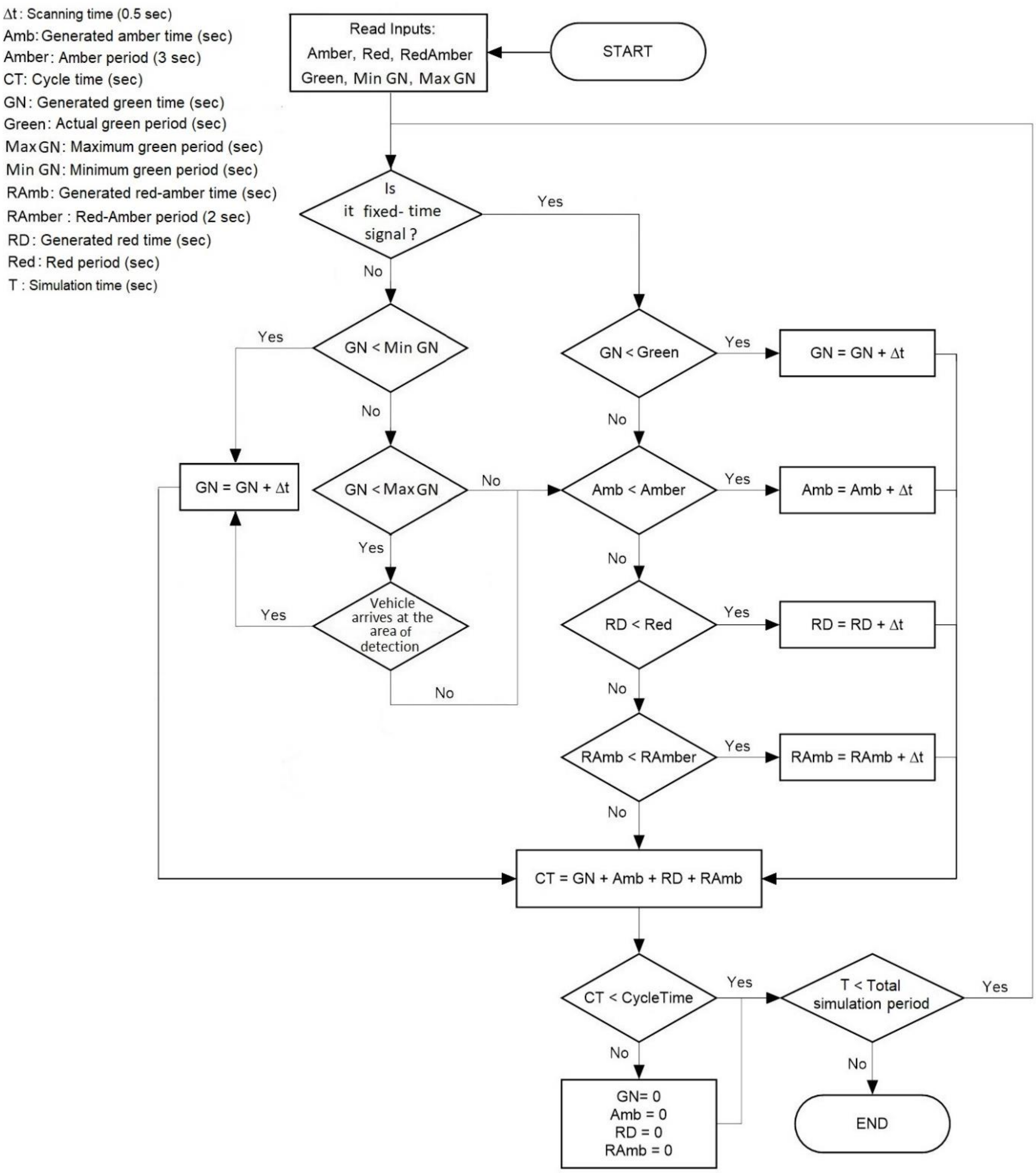


Figure 5.5: Traffic light operation system subroutine

5.6 The car-following rules

Benekohal (1986) developed car-following rules based on the interaction between the following and preceding vehicles travelling in the same lane. The model has been modified in order to be used in other micro-simulation sub-models because of its realism and ability to mimic free-flow and congestion situations in an urban environment and highways. Different acceleration/deceleration values are computed in the CARSIM using different parameters.

Benekohal (1986) described the determination of acceleration/deceleration rate in different traffic situations as follows:

1. **Acceleration rate from mechanical capability of vehicle (a_1):** based on a vehicle's current speed and type, the maximum acceleration rate will be assigned in the simulation. Table 5.2 provides different values of (a_1). These values were factored by 0.75 following the previous works of Yousif (1993), Al-Jameel (2012), Alterawi (2014), and Nassrullah (2016). This is because of higher mechanical capabilities of vehicles in the USA in comparison with vehicles in the UK and European countries. The maximum deceleration rate (MADR) was assumed to be equal to -4.9 m/sec^2 for all vehicle types.

Table 5.2 Maximum acceleration rate of PCs and HGVs measured in (m/sec^2) taken from Institute of Transportation Engineering (2010)

Speeds (kph)	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

2. **Acceleration rate for vehicle moving at desired speed (a_2):** if the vehicle is not constrained by the preceding vehicle or road conditions, a_2 will be assigned to each generated vehicle. The driver will try to reach the desired speed using the comfortable acceleration rate, or apply the comfortable deceleration rate if he/she exceeds the speed limit. The values of comfortable acceleration are (1.1 m/sec^2) and (0.37 m/sec^2) for PCs and HGVs, respectively. The comfortable deceleration rate will be (-3.0 m/sec^2) for PCs and (-1.8 m/sec^2) for HGVs (Institute of Transportation Engineering, 2010).

3. **Acceleration rate for non-collision situations (a_3):** to avoid collision between the successive vehicles under congested conditions particularly when the preceding vehicle stops suddenly, a_3 can be applied by the follower in order to stop safely. For this purpose, the distance between the vehicle will be computed and checked within the simulation at every scanning time to satisfy that situation as follows:

$$Pos_L - \left[Pos_F + V_F(\Delta t) + \frac{1}{2}a_3(\Delta t^2)\right] - L_{vL} - BS \qquad Equation\ 5.1$$

Equation 5.1 should be equal to or greater than the maximum of the following equations:

$$R_t(V_F + a_3(\Delta t))\ , or \qquad Equation\ 5.2$$

$$R_t(V_F + a_3(\Delta t)) + \frac{(V_F+a_3(\Delta t))^2}{2MADR_F} - \frac{V_L^2}{2MADR_L} \qquad Equation\ 5.3$$

where:

R_t is the driver reaction time measured in (sec).

L_{vL} is the length of leading vehicle measured in (m).

BS is the buffer space between following and leading vehicles measured in (m).

Δt is the scanning time (which is equal to 0.5 sec in the current study).

V_F and V_L are the speeds of following and leading vehicles, respectively, measured in (m/sec).

Pos_F and Pos_L are the positions of following and leading vehicles, respectively, measured in (m).

$MADR_F$ and $MADR_L$ are the maximum deceleration rates of following and leading vehicles, respectively, measured in (m/sec^2).

4. **Acceleration rate for slow-moving situations (*a_4*):** under congested or forced flow conditions, the vehicles will move slowly in a platoon with closed space headways. The a_4 for this situation can be determined from the following equation:

$$Pos_L - \left[Pos_F + V_F(\Delta t) + \frac{1}{2}a_4(\Delta t^2)\right] - L_{vL} - BS\ \geq 0 \qquad Equation\ 5.4$$

The distance between the vehicles will be checked and will not be less than the buffer space to avoid collision. The buffer space can be generated randomly in the simulation. The random number will be set to be equal to the lognormal distribution as discussed previously in Section 4.3.3.

5. **Acceleration rate for moving from stationary situations (*a_5*):** this situation happens when the stopped vehicle prepares to move from its position with a_5 after forced stopping because of the red signal. In addition, the driver will spend a few seconds starting his/her vehicle movement, particularly when the signal is showing a green light; this is called Move-Up-Delay (MUD). Based on the vehicle type, the a_5 values for moving from stationary and after MUD situations are (0.42 m/sec^2) for PCs and (0.21 m/sec^2) for HGVs as stated by Benekohal (1986) and following the previous research studies of Al-Jameel (2012), Alterawi (2014) and Nassrullah (2016).

The developed model will be used to determine the required acceleration/deceleration rate to update the new speed and position of a vehicle at every scanning time (Δt = 0.5 sec). The mathematical expression of the n^{th} vehicle speed and position at a certain time t and updating to time Δt according to Newton's laws of motion can be determined as follows:

$$V_n^{t+\Delta t} = V_n^t + a_n^t(\Delta t) \qquad Equation\ 5.5$$

$$Pos_n^{t+\Delta t} = Pos_n^t + V_n^t(\Delta t) + \frac{1}{2}a_n^t(\Delta t^2) \qquad Equation\ 5.6$$

where:

Δt is the scanning time (which is equal to 0.5 sec in this study),

a_n^t is the acceleration/deceleration rate of vehicle n at time t measured in (m/sec^2),

$Pos_n^t\ and\ V_n^t$ are the current position (m) and current speed (m/sec) of vehicle n at time t, respectively, and

$Pos_n^{t+\Delta t}\ and\ V_n^{t+\Delta t}$ are the updated position (m) and updated speed (m/sec) of vehicle n at time $t+\Delta t$, respectively.

The acceleration subroutine can be summarised in the flowchart shown in Figure 5.6 based on the traffic movements and road conditions. The selected acceleration/deceleration of the vehicle is influenced by the speed difference of the preceding and following vehicles, and the headway between the vehicles.

5.7 Modelling drivers' compliance at signalised intersections

First of all, it is assumed that there is no lane changing based on the site observations. The vehicles will enter the simulated section successively based on flow arrival distribution. The speeds and positions of all vehicles will be updated at the end of every scanning time. Drivers will face three traffic lights conditions (i.e. Green, Amber and Red lights) and have to make decisions depending on their speeds and distances from the stopline position. Their decisions are either STOP when the signal shows amber or red indication, or GO when the signal shows green or amber light. Additionally, some drivers fail to comply with the red light signal which is recognised as a signal violation or red light running event.

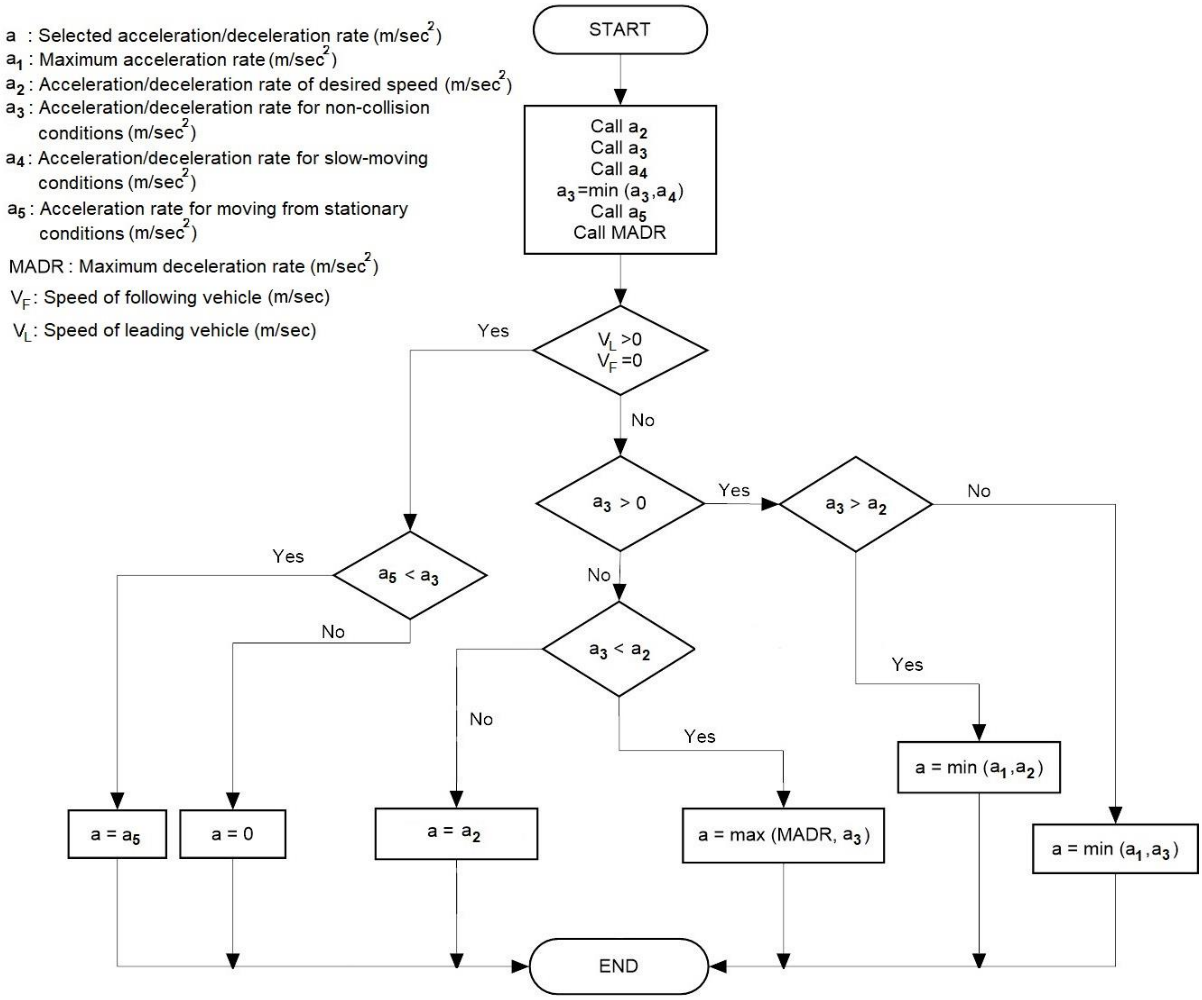


Figure 5.6: The structure of the acceleration subroutine in the CARSIM model

As discussed in Section 5.3, the model consists of three main components: the Green-CARSIM, the STOP-GO (which is the core of the newly developed micro-simulation model) and the Ready-to-GO sub-models. More details regarding driver compliance can be found in the following sections.

5.7.1 The Green-CARSIM sub-model structure

The movement of traffic under the effect of the green phase before the onset of red will be governed by the original CARSIM algorithms as depicted in Figure 5.6. At every scanning time, an appropriate acceleration/deceleration value will be selected to update the speed and position of the vehicle. At the onset of green, the vehicle starts to move from rest with a_5 until it reaches its desired speed. The CARSIM model will choose the minimum positive value of a_1, a_2, a_3 and a_4 as the acceleration rate that the following vehicle has to maintain its speed with the preceding vehicle without causing any collision. On the other hand, the model introduces a safe deceleration rate (minimum of a_3 *or* a_4) for slow moving and sudden stop conditions by governing a safe spacing (buffer space) between vehicles in a platoon (Benekohal, 1986).

In all cases, the selected acceleration rate should not exceed the maximum acceleration rate (i.e. a_1) in addition to not being less than the maximum deceleration rate (*MADR*). Also, the following vehicle will not apply any deceleration rate in two cases: either if the following headway distance between two vehicles is greater than the buffer space and length of vehicle, or if the speed of the lead vehicle is greater than the follower's speed by a certain value which is equal to 5 kph (Benekohal, 1986).

5.7.2 The STOP-GO sub-model structure

At the onset of amber, a driver has to predict what decision should be made based on the position from the stopline, speed and time required to cross the stopline. The prediction STOP-GO sub-model was built based on the CARSIM model and GHM model. The number of vehicles approaching the stopline will be counted at the onset of amber as shown in Figure 5.7. It is worth mentioning here that the vehicle's zone and driver's decision will be assigned only once at the beginning of amber and will not be repeated again for the rest of the amber and red periods.

Figure 5.7: Counting of vehicles approaching the stopline at the onset of amber

To assign the position of the vehicle from the stopline (i.e. in the Stop Zone, Dilemma Zone, Go Zone, or Option Zone), a safe stopping distance (SSD) and a clearance distance (CD) will be computed for each vehicle on the simulated section as described previously in Section 2.5.2. Various parameters were used to calculate the SSD and CD, such as the amber interval (usually 3 sec), vehicle speed, driver perception-reaction time, and the maximum deceleration rate (-4.9 m/sec^2) as expressed in Equations 2.1, 2.2 and 2.3. Then, the position of vehicles from the stopline (Pos_{nSL}) will be assigned in the model as follows (see Figure 2.7 for explanation):

- *If* $Pos_{nSL} \leq CD \leq SSD$ *or* $Pos_{nSL} \leq SSD \leq CD$, then the vehicle is in Go Zone.
- *If* $Pos_{nSL} \geq SSD \geq CD$ *or* $Pos_{nSL} \geq CD \geq SSD$, then the vehicle is in Stop Zone.

- *If* $CD < Pos_{nSL} < SSD$, then the vehicle is in Dilemma Zone.
- *If* $SSD < Pos_{nSL} < CD$, then the vehicle is in Option Zone.

The next step will be assigning driver's decision at the onset of amber. GO decisions will be made by drivers who are able to cross the stopline within no more than the 3 sec on amber if the preceding vehicle does not stop. Those drivers will be defined as Amber Light Running (ALR) drivers. The ALR drivers could be either in the Go Zone, Dilemma Zone or Option Zone as shown previously in Figure 2.7.

On the other hand, other drivers may hesitate to decide whether to STOP or GO. It is necessary to predict if some drivers are able to cross and clear the intersection before green is shown to the conflict flow (i.e. within the intergreen period). They were recognised as Red Light Running (RLR) drivers. Based on site observations, the RLR drivers decided to cross the stopline and violate the red signal. As illustrated previously in Figure 2.7, there were two groups of RLR drivers. The first group crossed with normal acceleration and desired speeds because they had enough distance to stop, but they were able to clear the junction before the end of the intergreen period (this is the case for the Option Zone). The second group decided to accelerate and pass through the junction because they neither had the distance nor time to stop safely (this is the case of the Dilemma Zone). In addition, it was assumed that drivers who had surprised reaction times greater than 1 second would fail to comply with the red at the onset of amber and cross the intersection area before the conflicting flows start their movements (as described previously in Section 5.4.6). This is actually the reality of RLR behaviour that was observed at the surveyed sites. The GO decision made by RLR drivers in the Dilemma Zone, or Option Zone will be implemented if the leading vehicle was not stopped after the onset of amber as shown in Figure 5.8.

However, the Amber/Red Light Stopping (ARLS) drivers were unable to cross and clear the intersection area before the green phase began. The ARLS drivers could be either under the effect of the Stop Zone or Option Zone if they were far away from the stopline and unable to clear the junction area during the intergreen period, or they were under the effect of the Dilemma Zone and complied with the signal change (see Figure 2.7 for more detailed explanation).

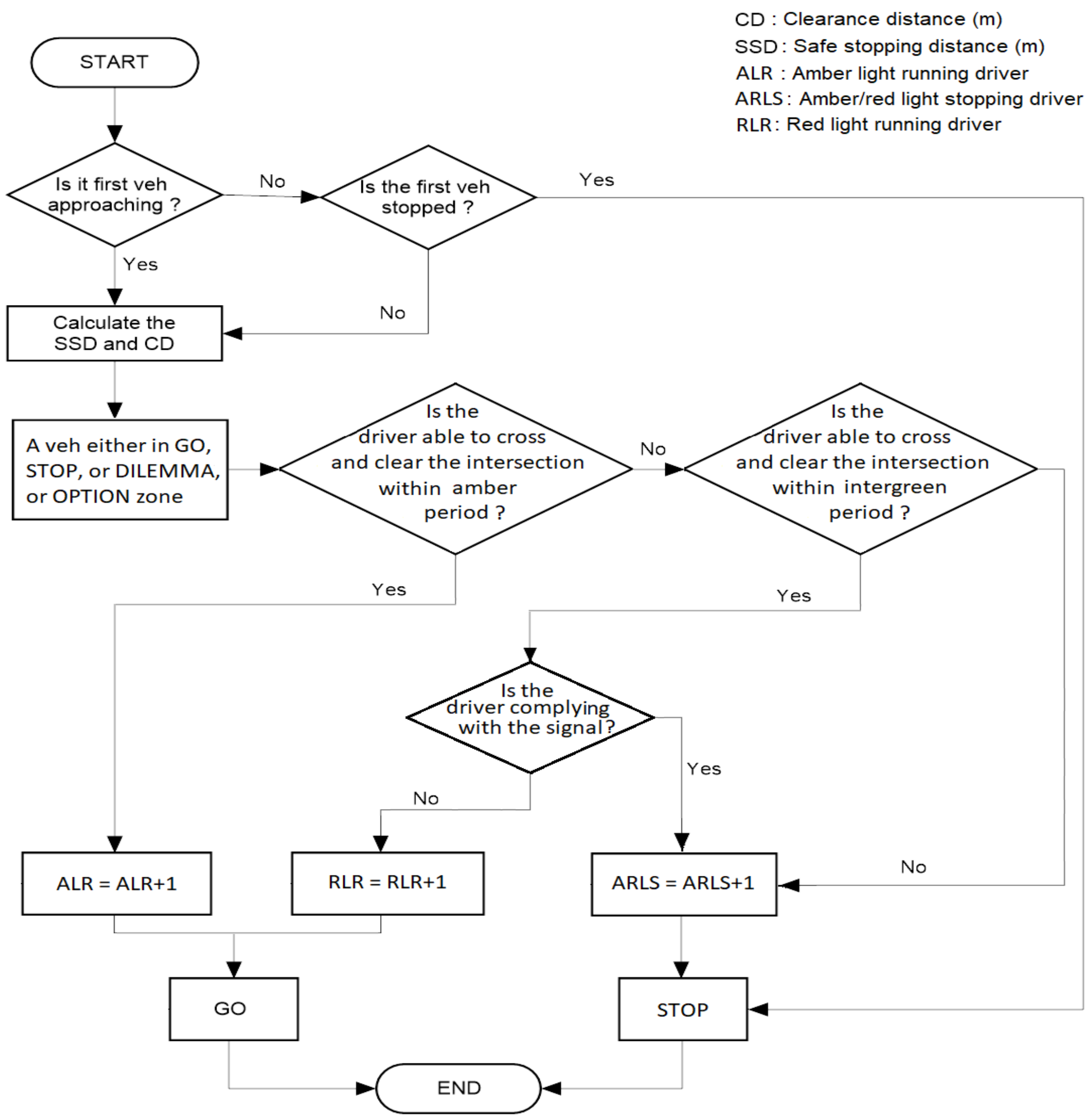


Figure 5.8: The structure of the prediction STOP-GO sub-model at the onset of amber/red

5.7.3 The acceleration subroutine in the STOP-GO sub-model structure

Modelling drivers' behaviour and the response to the signal change is an important aspect of the micro-simulation model. The CARSIM algorithms can be applied to model driver behaviour along the approach to the signal junction when the signal is showing the green light. However, a different process is required for modelling driver behaviour when the signal light changes from green to amber and then to red. When a driver decides to stop after the onset of amber or red, Gazis et al. (1960) stated that the required deceleration rate a_6 can be calculated from the following equation:

$$a_6 = \frac{V_n^2}{2(V_n(R_t) - Pos_{nSL})} \qquad \text{Equation 5.7}$$

where,

Pos_{nSL} represents the position of the vehicle n from the stopline in (m).

V_n is the speed of the vehicle in (m/sec).

R_t is the driver alerted reaction time (sec) in the amber and red periods.

The calculated a_6 should not exceed the maximum deceleration rate value (i.e. $MADR$= -4.9 m/sec^2). The algorithm for the selected acceleration/deceleration rate during amber and red periods can be shown in Figure 5.9. If a driver decides to GO, the acceleration rate will be generated from the original CARSIM subroutine by keeping a safe following spacing ($>L_v+BS$) with the leading vehicle. However, if the first driver decides to STOP, then the acceleration will be calculated from Equation 5.7 until the vehicle stops completely. Subsequently, a driver who follows the preceding vehicle will maintain his/her speed and decelerate comfortably using a deceleration rate generated at sudden stopping conditions from the original CARSIM model. Finally, the updated speed and position of vehicle will be determined from Equations 5.4 and 5.5.

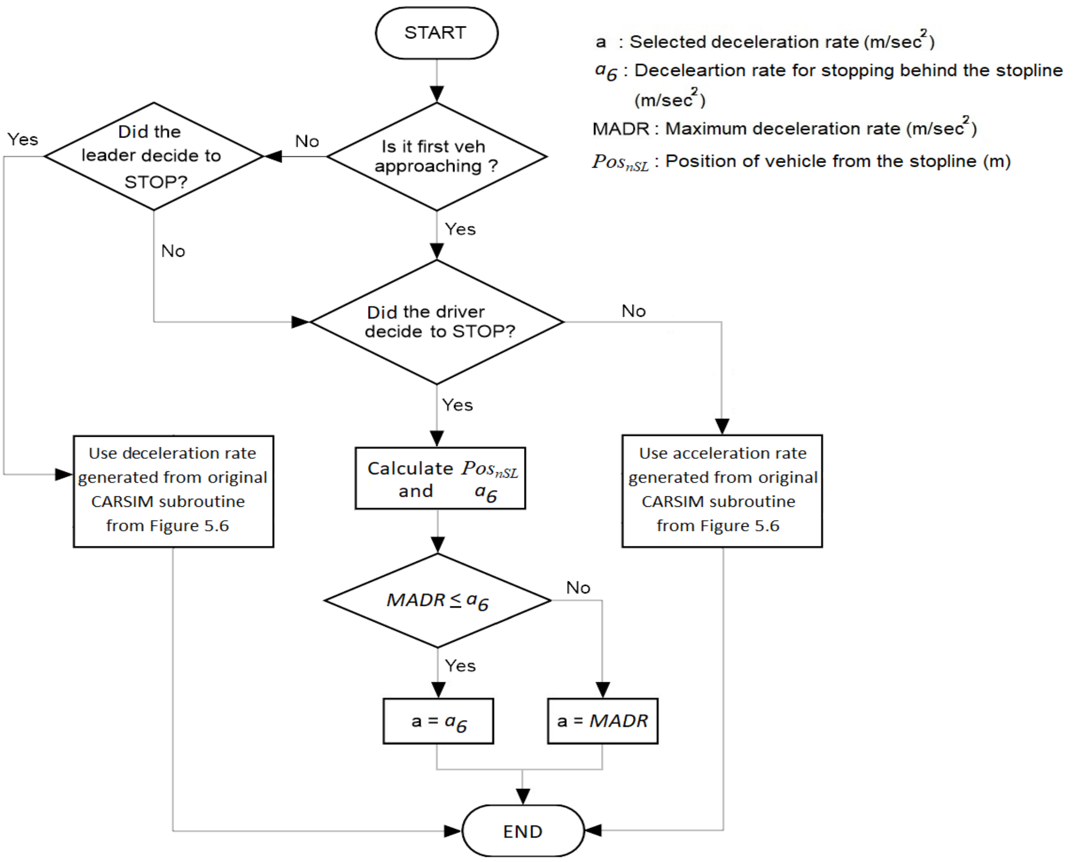


Figure 5.9: The structure of the acceleration subroutine in the STOP-GO model

5.7.4 The Ready-to-GO sub-model structure

In this part of the developed model, the driver behaviour can be governed by the STOP-GO algorithm for modelling the arrival of a vehicle's approach to the traffic signal junction after the end of red (i.e. at the onset of the red-amber period which is usually 2 seconds before the green comes on) as illustrated in Figure 5.9. Based on site observations, there were two groups of drivers. The first was drivers in full stop conditions who waited for the green indication to accelerate. Those drivers had taken their decisions to stop previously after seeing the amber or red traffic light. When the green phase is beginning, they will start to move with *a5* from rest according to the original Green-CARSIM algorithm as illustrated in Figure 5.6.

The second group was drivers who arrived too late and their behaviours were assumed to follow the STOP-GO algorithm (see Figure 5.9). The assumption was that a driver starts to decelerate before the onset of the red-amber signal by applying either a deceleration rate a_6 from Equation 5.7 (if it is the first vehicle arriving at the stopline), or a deceleration rate calculated from the original Green-CARSIM model (if there was a stopped vehicle ahead). After the onset of green, those drivers will continue their crossing behaviour using the acceleration/deceleration rate generated from the original Green-CARSIM sub-model based on the desired speed and available safe spacing with the preceding vehicles. Then, the updated speed and position of vehicle will be determined from Equations 5.4 and 5.5.

5.8 Other model characteristics

Other fundamental traffic characteristics were needed in the newly developed simulation model to use as input data such as road length, signal timing and type. When the simulation model starts running, the simulation will be in unsteady conditions. These could be eliminated by introducing the warm-up period to mimic the actual situations of traffic movement before collecting the results. Similarly, the cooling-down period will be added at the end of the simulation process in order to prevent any changes in the traffic movement particularly after vehicles exit from the simulated approach. Following previous works (Alterawi, 2014 and Nassrullah, 2016), each period was assumed to be equal to 5 minutes only. Also, the lengths of the warm-up and cooling-down sections were introduced to be 500 metre at each end of the simulated approach. The purpose is to exclude traffic data generated in these sections from the outputs of the newly developed model. In addition, several surrogated safety measures (that listed previously in Table 2.5) have been calculated through the simulation in order to investigate possible tailgating behaviour.

5.9 Outputs of the developed model

The model outputs were used for model verification, calibration, validation and application processes. They can be categorised as follows:

1. Micro output data: including details of vehicle speed, position and acceleration/deceleration rate at every scanning time.
2. Macro output data: including information about average vehicle speed and traffic flow.
3. Other output data: including driver compliance with the traffic signal, speeds and acceleration/deceleration rates at a certain section, distance and time to the stopline at the onset of amber, move-up time, and some safety measures at the onset of amber such as time-to-collision and time headways.

5.10 Capabilities of the newly developed model

This model has been developed to predicted drivers' responses to the signal change following the onset of amber. In addition, this model can test the effects of lengths of amber and all-red periods as well as giving an evaluation of safety issues at traffic signal junctions. The related parameters can easily be changed in the input files to replicate traffic and driver behaviour on the approach.

Despite the shortcomings of simulation models which have been previously mentioned in Section 2.7.4, the developed model takes into consideration the following effects:

1. It is possible to include different lengths and types of vehicles as well as the proportion of heavy goods vehicles in the flow.
2. It is possible to set different signal timings such as fixed-time and vehicle-actuated signals.
3. It is possible to replicate the variation in the acceleration/deceleration rates for every vehicle generated in the model.
4. It is possible to replicate driver compliance when the traffic signal is showing amber. In addition, it is possible to model traffic conflicts such as red light violations.
5. This model provides an evaluation of traffic safety at signalised junctions and gives an indication of accident risk by measuring time-to-collision and headways between vehicles at the onset of amber, as will be discussed in the next chapter.

5.11 Summary

This chapter has described the development of the newly developed microscopic simulation sub-models (i.e. the CARSIM and GHM models) which are to be used in combination for modelling drivers' behaviour approaching traffic signal junctions. The simulation process is based on the analysis of real data observations including driver compliance with the signal change in the dilemma zone. The FORTRAN programing language F95 was used to write the codes for the simulation model. The model verification, calibration and validation processes will be explained in the next chapter.

CHAPTER SIX: MODEL VERIFICATION, CALIBRATION AND VALIDATION PROCESSES

6.1 Introduction

As mentioned in the previous chapter, it is necessary to assess and test the newly developed model by comparing its results with the observed data before using it for evaluating junction performance and safety issue. The most important stages of building the simulation model are the verification, calibration and validation processes. This is to ensure that the developed sub-models are effectively representing the problems in the real world (May, 1990; Olstam and Tapani, 2004; and Young et al., 2014). Figure 6.1 illustrates the verification, calibration and validation processes for the developed model.

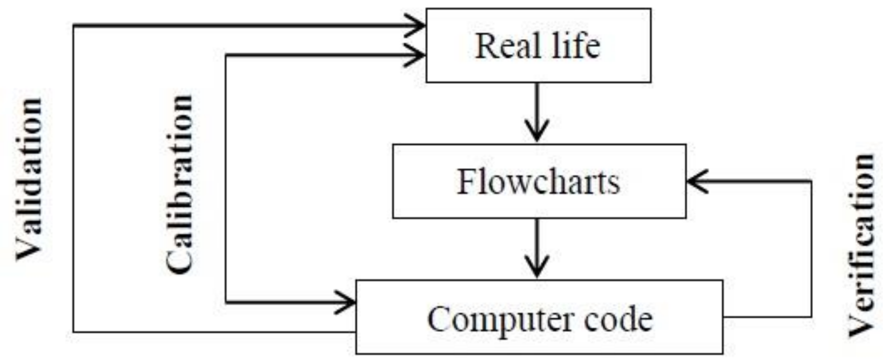


Figure 6.1: The verification, calibration and validation steps for the developed micro-simulation model (May, 1990)

According to Olstam and Tapani (2004), the verification process aims to check if the model assumptions have been correctly translated into codes and given reasonable outputs using different input parameters (without comparing them with the observed data). The calibration process means checking if the model is working correctly and it gives accurate results by comparing the observed data with the model outputs. Finally, model validation can be achieved by testing other data sets obtained from other sites. May (1990) demonstrated that each step shown in Figure 6.1 is dependent and repetitive in order to eliminate errors by adjusting the model assumptions and/or input parameters. Different sites were used for the calibration and validation processes as shown in Table 6.1. These sites have differences in traffic flow rates, geometric details, traffic signal settings, and numbers of traffic signal compliance (i.e. ALR, RLR and ARLS events). Details of parameters which were used in the developed model are listed in Table 6.2. Some of these parameters were used as inputs that obtained from the field observations or previous literature, whereas the others were the model outputs used for comparing with the site observations in the calibration and validation processes. More information regarding the three model steps and statistical tests are described in the next sections.

Table 6.1: List of sites used in the calibration and validation processes

Model Stage	FT signal junctions	VA signal junctions
Calibration	Site #1a Site #5a	Site #3a Site #4a and b
Validation	Site #1b Site #2 Site #5b	Site #3b Site #4c

Table 6.2: List of parameters used in the model development, calibration and validation processes

Parameter	Assumed or Observed	Input	output	Calibration	Validation
Arrival flow	Observed	✓			
Headway shift value	Assumed	✓		✓	
Arrival headway	Observed	✓	✓		
Intersection width	Observed	✓			
Signal setting (FT or VA)	Observed	✓			
Traffic light timings*	Observed	✓			
Speed	Observed	✓	✓		
PC length	Assumed	✓			
HGV length	Observed	✓			
HGVs%	Observed	✓			
Buffer space	Observed	✓			
Acc/deceleration rate	Assumed	✓	✓		
Driver reaction time	Assumed	✓			
Driver response	Assumed	✓		✓	
Driver alertness %	Assumed	✓		✓	
Signal compliance	Observed		✓	✓	✓
MUD	Observed	✓			
MUT	Observed	✓	✓		✓

* Traffic light timings: all-red, intergreen and cycle time lengths (i.e. Green, Amber, Red and Red-Amber periods).

6.2 Statistical tests

Several statistical tests were carried out in order to assess the difference between the observed and the simulated data for the purposes of calibration and validation. These tests can be listed as follows:

1. **Root Mean Square Error (*RMSE*):** this test was used in previous works (Al-Jameel, 2012; Al-Obaedi, 2011; Alterawi, 2014; Nassrullah, 2016 and Panwai and Dia, 2005) to check the goodness of fit between the empirical and modelled data. In addition, Root Mean Square Percentage Error (*RMSPE*) has been used by the aforementioned researchers. Both tests *RMSE* and *RMSPE* were used to test the system error in the developed model. A lower value indicates good representation of the simulated data to the empirical data. A *RMSPE* =15% is adopted as a maximum to give an indication that the model outputs satisfy the simulation process (Hourdakis et al., 2003). The mathematical expressions of both tests can be written as:

$$RMSE = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(x_i - y_i)^2} \qquad \text{Equation 6.1}$$

$$RMSPE = \sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(\frac{x_i - y_i}{x_i}\right)^2} \qquad \text{Equation 6.2}$$

where, n is the number of time intervals and x_i and y_i are the observed and simulated data at time i, respectively.

2. **Geoffrey E. Havers (*GEH*):** this statistical test is recommended by the Department for Transport (1996). It is similar to the Chi-Square statistical which can provide a comparison between the observed and simulated data. The model can be considered acceptable and the simulation results match the observed data if 85% of the calculated *GEH* values are lower than 5. The *GEH* formula can be written as:

$$GEH = \sqrt{\frac{2\,(y_i - x_i)^2}{y_i + x_i}} \qquad \text{Equation 6.3}$$

3. **Theil's Inequality Coefficient (*U*):** this test has been used in the simulation research since it is more accurate and efficient than the *RMSE* and *RMSPE* and gives an indication that how close the model outputs are to the real data (Al-Obaedi, 2011; Alterawi, 2014; Hourdakis et al., 2003 and Nassrullah, 2016). The mathematical equation can be defined as shown in Equation 6.4. The estimated value of *U* is also between 0 and 1. A lower

value of U (less than 0.3) gives good representation of the observed data in the simulation model (Hourdakis et al., 2003).

$$U = \frac{\sqrt{\frac{1}{n}\sum_{i=1}^{n}(x_i - y_i)^2}}{\sqrt{\frac{1}{n}\sum_{i=1}^{n}(x_i)^2} + \sqrt{\frac{1}{n}\sum_{i=1}^{n}(y_i)^2}} \qquad Equation\ 6.4$$

4. **Bias proportion (U_m):** this is also known as the Theil's mean difference. It measures the differences between the mean values of the real and modelled data (Hourdakis et al., 2003). The U_m value is also between 0 and 1. This measure can be represented by the following equation:

$$U_m = \frac{n\,(\mu_x - \mu_y)^2}{\sum_{i=1}^{n}(x_i - y_i)^2} \qquad Equation\ 6.5$$

where, μ_x and μ_y are the means of the observed and simulated data, respectively.

5. **Variance proportion (U_s):** this is also called the Theil's standard deviation difference, which measures the degree of variability in the standard deviation value of the simulated data compared with that of site observations. The value of U_s is also between 0 and 1 according to Hourdakis et al. (2003). This measure can be determined as follows:

$$U_s = \frac{n\,(\sigma_x - \sigma_y)^2}{\sum_{i=1}^{n}(x_i - y_i)^2} \qquad Equation\ 6.6$$

where, σ_x and σ_y are the standard deviation values of the observed and simulated data, respectively.

Finally, it is worth mentioning that an acceptable simulation model can be achieved if the aforementioned statistical measures satisfy the threshold values (i.e. $U < 0.3$ and/or $GEH \leq 5$). The units of U and *RMSPE* are given in percentages, while the unit of *GEH* is a scalar quantity. Moreover, the unit of *RMSE* measure is the unit of the parameters, for example (vph) when testing the goodness of fit of the observed flow with the modelled data. The value statistical measures are discussed in the calibration and validation processes sections.

6.3 Model verification process

According to Olstam and Tapani (2004), the verification process aims to check if the model assumptions and suggested flowcharts have been correctly translated into programing language codes and given acceptable results. For this purpose, observing the animation environment of the developed model, as shown in Figure 6.2, could be useful. In addition, analysing the simulated data (such as the distributions of vehicle lengths, desired speed, driver reaction time, vehicle arrival headways, percentage of heavy goods vehicles, … etc.) were carried out at earlier stages of the model building by debugging the written codes to eliminate illogical errors without using the real observations.

Figure 6.2: Typical screenshot from the new micro-simulation model illustrating vehicles approaching a traffic signal junction

6.3.1 Verification process for vehicles' characteristics

The results of the simulated data of the cumulative distribution of heavy goods vehicle lengths and the normal distribution of passenger cars can be shown in Figure 6.3. In addition, Figure 6.4 illustrates the distribution of vehicles' arrival headways. Figures 6.3 and 6.4 show good fit between the input data and that which was simulated by using the statistical distributions as described previously in Sections 4.3.1 and 4.3.2. The same process was carried out for checking the distributions of driver move-up delay, reaction time and desired speed.

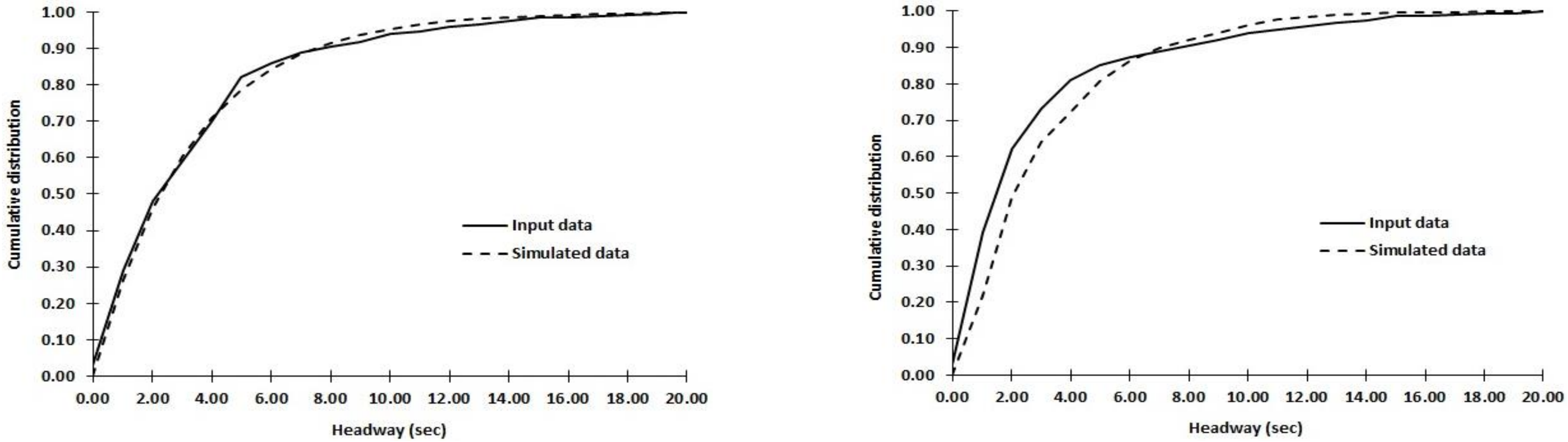
6.3.2 Vehicle trajectories along the approach to a traffic signal junction

The trajectories of a sample of 50 vehicles travelling towards a traffic signal junction can be shown in Figure 6.5 as an example of the simulation results. The stopline position and traffic light signals were set at 1500 m for simulation purposes. In addition, the driver compliance with the traffic signal change was replicated throughout the new simulation model. As illustrated in Figure 6.5, a group of drivers complies with the signal change and has to decelerate for stopping conditions after the onset of amber and/or red. Whereas other drivers continue crossing the stopline either during the green period or after the onset of amber.

(a) Lengths of PCs

(b) Lengths of HGVs

Figure 6.3: An example of model verification for simulated vehicle lengths



(a) Shifted Negative Exponential Distribution

(b) Negative Exponential Distribution

Figure 6.4: An example of model verification for simulated arrival headways

Figure 6.6 presents a sample of the output data for two successive vehicles including time, distance, vehicle speed and acceleration/deceleration rate. As shown in Figure 6.6, the following vehicle enters the simulated approach section at a speed of 50 kph and it is faster than the vehicle ahead by 10 kph until it reaches the leading vehicle speed by a decelerating action. Both vehicles showed car-following rules by using different acceleration/deceleration rates and keeping safe spacings between them to avoid collision. A decision to stop or cross the stopline will be made by the leading vehicle at the onset of amber, then the following vehicle will decide subsequently either to stop or follow the leader and cross the stopline.

In addition, a separate output file includes information about drivers' behaviour under the effect of the dilemma zone with regard to vehicle speed, distance from the stopline, and zone (i.e. whether the vehicle in the Stop Zone, Go Zone, Dilemma Zone Or Option Zone). As described previously in Section 5.7, a driver's STOP/GO decision was assigned in the new model (i.e. STOP-GO subroutine) based on a number of factors such as distance from the stopline, driver reaction time, his/her response to the amber, intersection width and the length of intergreen period.

A speed-distance profile of two passing vehicles travelling on the 150 m approach to the stopline can be illustrated in Figure 6.7. The driver of the leading vehicle was an ALR driver, whereas the follower was an RLR driver. Based on the adopted car-following rules in the current study, it can be seen that drivers tend to accelerate and cross the stopline before the onset of red using acceleration/deceleration rates during the amber period as shown in Figure 6.6. In addition, both vehicles showed a reduction in their speeds particularly before entering the intersection area because they have to use the original car-following rules after crossing the stopline (i.e. they have to use an acceleration/deceleration rate generated from the Green-CARSIM subroutine as described in Section 5.6).

Finally, it can be concluded that drivers who decided to proceed through the junction have a tendency to increase their speeds after seeing the amber phase. This driving behaviour reflects the aggression of drivers who might fail to comply with the signal change under the effect of the dilemma zone.

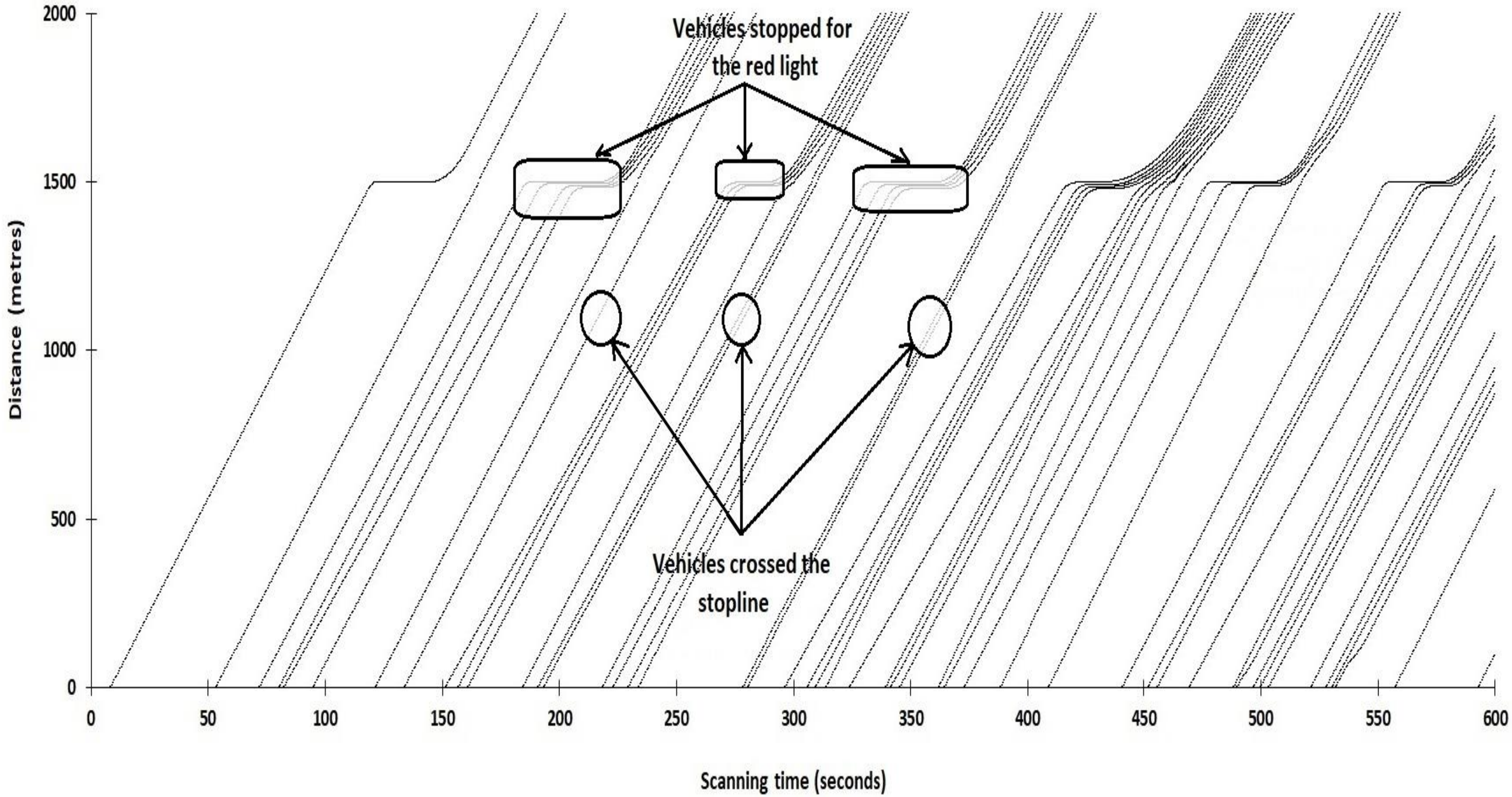


Figure 6.5: Sample of vehicle trajectories under the effect of traffic light signals

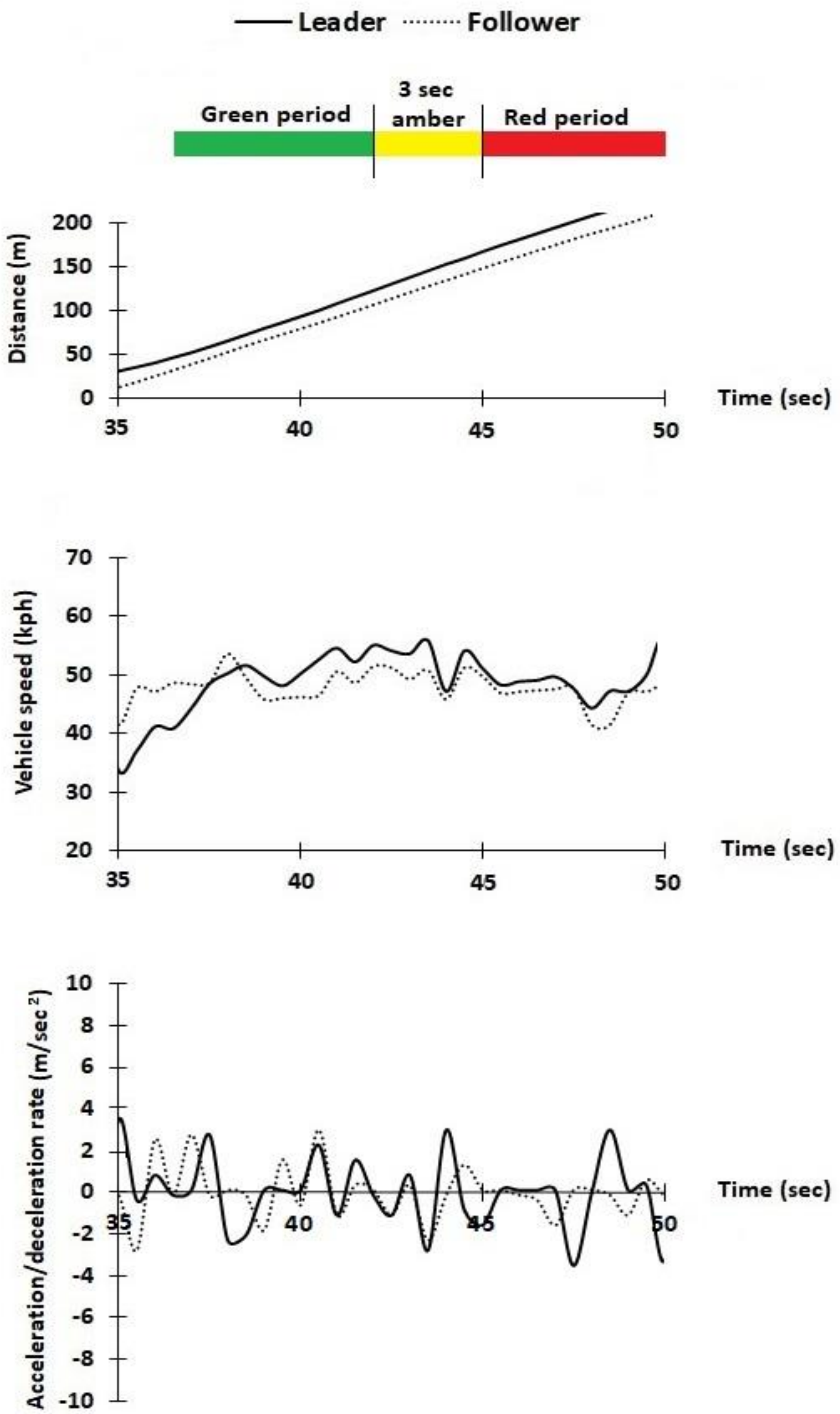


Figure 6.6: Trajectories of two successive vehicles approaching a traffic signal junction

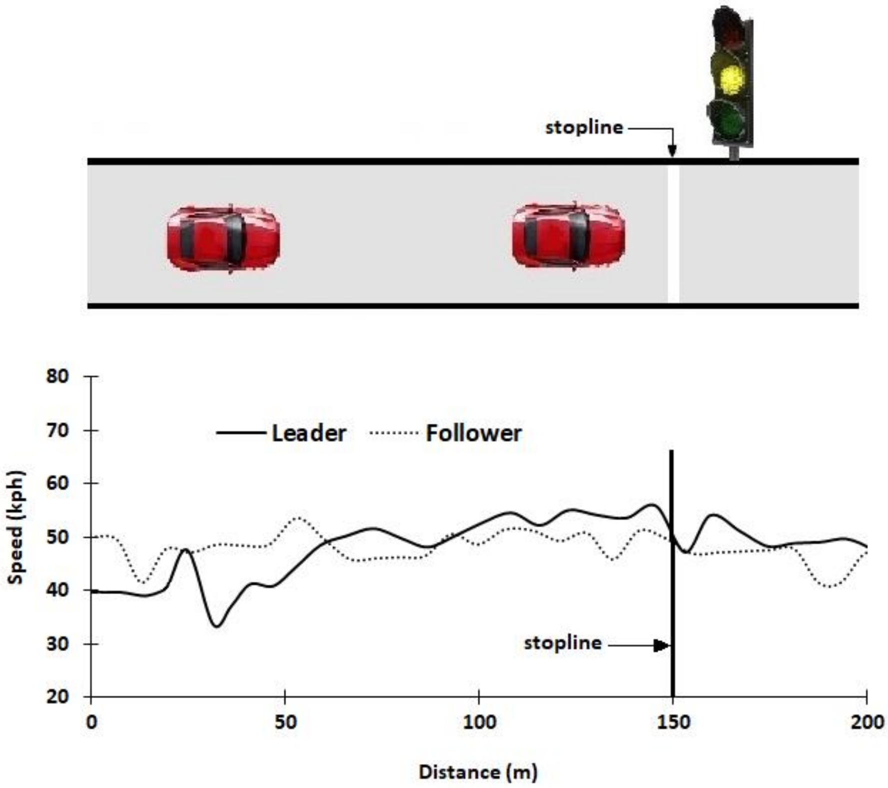


Figure 6.7: Speed-distance profile for drivers under the effect of the dilemma zone

6.4 Model calibration process

Since all the traffic information (i.e. flow rate every 5-minutes interval, type of vehicle, percentage of HGVs, number of amber crossings, number of red light runnings,… etc.) were collected separately from each lane at the survey site, it was suggested that the calibration of the developed model should be made on the single-lane model.

Because of the unavailability or limited trajectory data near traffic light junctions in the UK, several sensitive steps were implemented to determine adjusted car-following behaviour near signalised intersections. The calibration process for the whole model was carried out using an iterative processes including different input parameters such as driver reaction time, buffer space, vehicle length and maximum acceleration/deceleration rates. During these processes, the original CARSIM model was modified (as described previously in Section 5.7) in order get good representation of driver behaviour before and after the onset of amber as observed in the survey sites. The simulation model was run for thousand times during calibration process. Following previous researchers (Alterawi, 2014; Benekohal, 1986; Nassrullah, 2016 and Yousif, 1993), it was assumed that a driver uses a maximum acceleration/deceleration rate of (-3.9 m/sec^2) if the density exceeds 37 veh/km. Otherwise, he/she uses (-4.9 m/sec^2).

Finally, the model outputs will be compared with the observed data using the statistical tests (see Section 6.2) to check the goodness of fit. The model outputs can be listed below:

1. Hourly traffic flow for each 5-minutes interval.
2. The number of ALR and RLR drivers.
3. The move-up time for drivers.

6.4.1 Calibration of arrival flow profile

As part of the calibration process, the arrival flow profile has been generated from the shifted negative exponential distribution. Different values of shift between 0.1 and 1.0 (with an increment of 0.05 second) were tested for calibration purposes. Hundred times of simulation runs were executed including different flow rates up to 400 vph, as described previously in Section 4.3.2. The final value selected from this process was equal to 0.5 that gives better representation of the observed flow. No enhancement was obtained for shift values lower than 0.5. Figure 6.8 illustrates the calibration process for various shift values and the relevant statistical tests are presented in Table 6.3.

Table 6.3 Calibrated shift values of arrival flow

Shift values (sec)	Statistical tests					
	RMSE	***RMSPE%***	***GEH***	***U***	U_m	U_s
0.25	2.74	11.13	1.28	0.05	0.00	0.06
0.50*	2.45	9.90	1.08	0.04	0.00	0.08
1.00	3.54	12.80	1.63	0.06	0.00	0.12

* Calibrated value

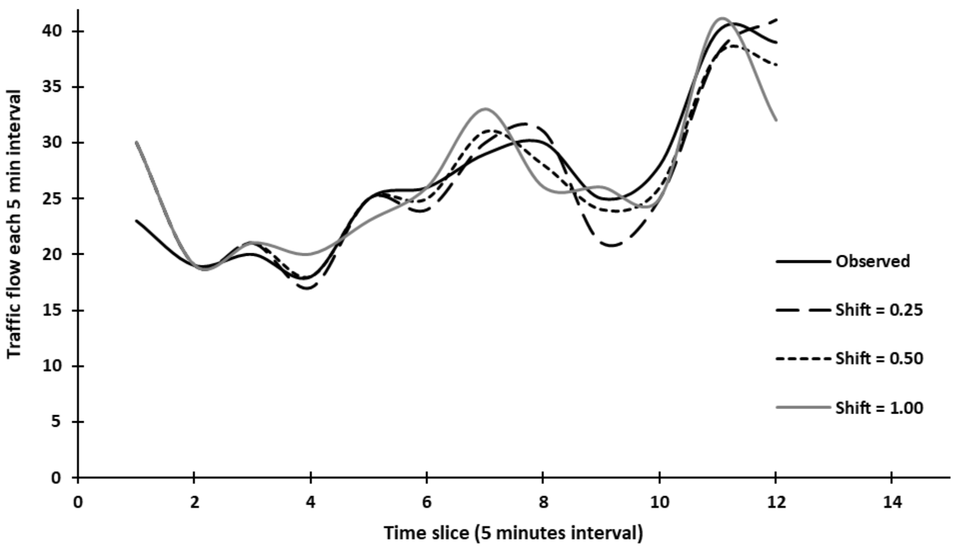


Figure 6.8: Arrival flow profile for different shift values

6.4.2 Calibration of driver response to traffic lights

Basically, the reaction time is a critical parameter that affects driver behaviour approaching a signal-controlled junction. This is because a driver must make a STOP/GO decision at a critical moment when he/she sees the amber light taking into consideration his/her distance from the stopline and speed. As mentioned earlier in Section 5.4.5, drivers will be alerted under the following circumstances:

1. If traffic density is equal to or greater than 37 veh/km as recommended by many researchers such as Benekohal (1986), Yousif (1993), Al-Obaedi (2011), Al-Jameel (2012), Alterawi (2014), and Nassrullah (2016).
2. If a driver decides to stop after the onset of amber, or a driver is ready to stop following the stopping vehicle ahead.

For the simulation purposes, thousand times of runs were executed in order to get good replication of driver response to the signals change at FT and VA traffic signal junctions. As mentioned earlier in Section 5.4.6, a certain driver reaction time was assumed to give an indication about driver compliance particularly after the amber light comes on. This value can be assigned from the generated random numbers for drivers' reaction times in the simulation. Hundred times of simulation runs were conducted using different values (from 0.5 to 1.5 sec with an increment of 0.1 sec) to replicate driver response to the amber. Finally, it was found that drivers with surprise reaction times equal to 1 sec or less are able to comply with the signal change and start to use alert reaction times if they decided to STOP at FT signalised intersections. It can be suggested that driver compliance with the signal change is equivalent to 70% (i.e. 30% of drivers contravened the signal when their reaction times are greater than 1 sec, as depicted in Figure 5.4). This result is similar to that reported by the police between 2009 and 2014 who demonstrated that 33.2% of drivers failed to comply with the signal change (see Section 2.4).

Similar procedures were used to find out drivers' responses to the signal change at VA signalised junctions. It was found that the (1 sec) driver response is not enough to replicate ALR and RLR events. As discussed in Section 4.4.2.6, the amber signal may surprise the drivers and they will not be under alert conditions. Therefore, it is necessary to assume another percentage to represent this type of behaviour (i.e. driver alertness). It was assumed that this percentage is equal to 50%. Finally, drivers' GO decisions would be made if they do not comply with the signal change and are capable of clearing the junction within the intergreen period as well as if the leading vehicle does not stop (as described previously in Section 5.7.2).

Figure 6.9 shows the modelling process for drivers' responses to the amber indication at FT and VA signal junctions. Details of sites used in the calibration and validation processes are summarised in the following sections and classified according to the control settings into FT and VA signal-controlled junctions, as described previously in Table 6.1.

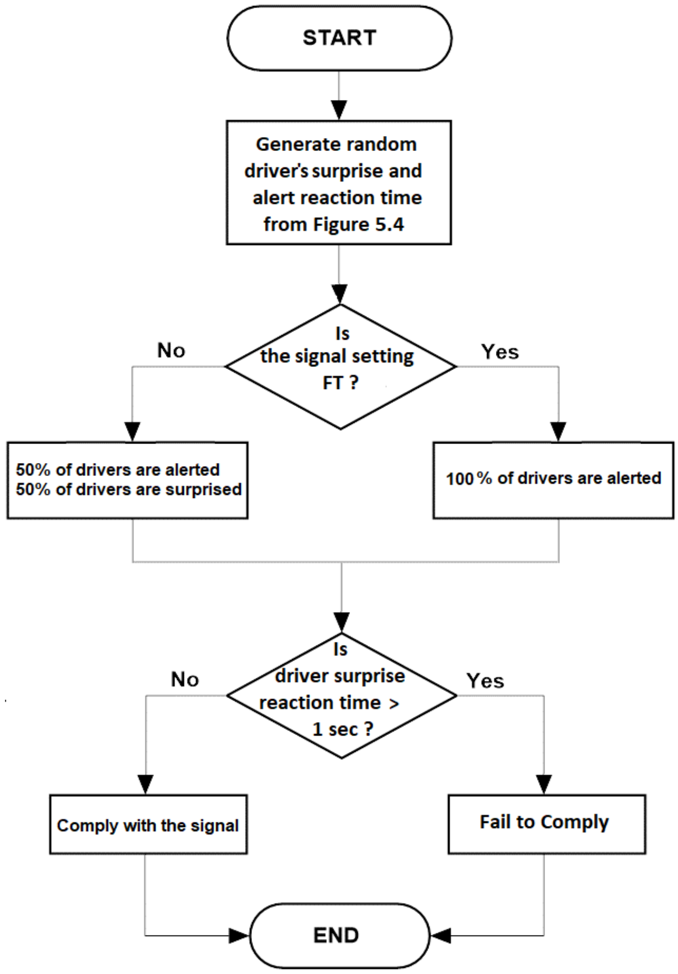


Figure 6.9: Modelling drivers' responses to the amber light at junctions controlled by FT and VA signals

6.4.2.1 Fixed Time signal (FT) sites

6.4.2.1.1 Site #1a (40 mph signalised junction)

Data collected from 40 mph signalised junction (Site #1a observed for 1 hr and 40 minutes) were used as input data in the developed model for the purposes of the calibration process. Details of this site can be seen in Table 6.4. The amber and red-amber periods in all visited sites were reported as 3 and 2 seconds, respectively. The intersection width was measured as the distance between the stopline and the far edge of the conflicting traffic lane, as shown previously in Figure 4.17.

Table 6.4: Model input parameters for Site #1a

Intersection details	**Intersection width in m**	25		
	Speed limit in kph	64		
	Number of lanes	3		
		Lane 1	Lane 2	Lane 3
Flow details	**Arrival flow in vph**	251	378	406
	HGVs (%)	4.9	5.0	2.8
	PC speed (μ,σ) in kph	(53, 5.6)	(56, 5.8)	(60, 8.87)
	HGV speed (μ,σ) in kph	(53, 4.0)	(53, 4.0)	(58, 4.0)
Traffic light details	**Green period in sec**	33		
	Red period in sec	34		
	All-red period in sec	8		

The arrival flow profile (each 5-minutes interval) for each lane was obtained from the simulation model and was compared with the observed data, as shown in Figure 6.10. The statistical tests have been carried out to test the goodness of fit as presented in Table 6.5. The results showed good agreement between the observed and simulated data.

Table 6.5: Statistical tests for arrival flow data calibration (Site #1a)

Lane #	**Statistical tests**					
	RMSE	***RMSPE%***	***GEH***	***U***	U_m	U_s
1	2.66	11.95	1.72	0.06	0.00	0.01
2	3.73	11.89	1.92	0.06	0.00	0.01
3	3.35	8.60	1.63	0.05	0.05	0.01

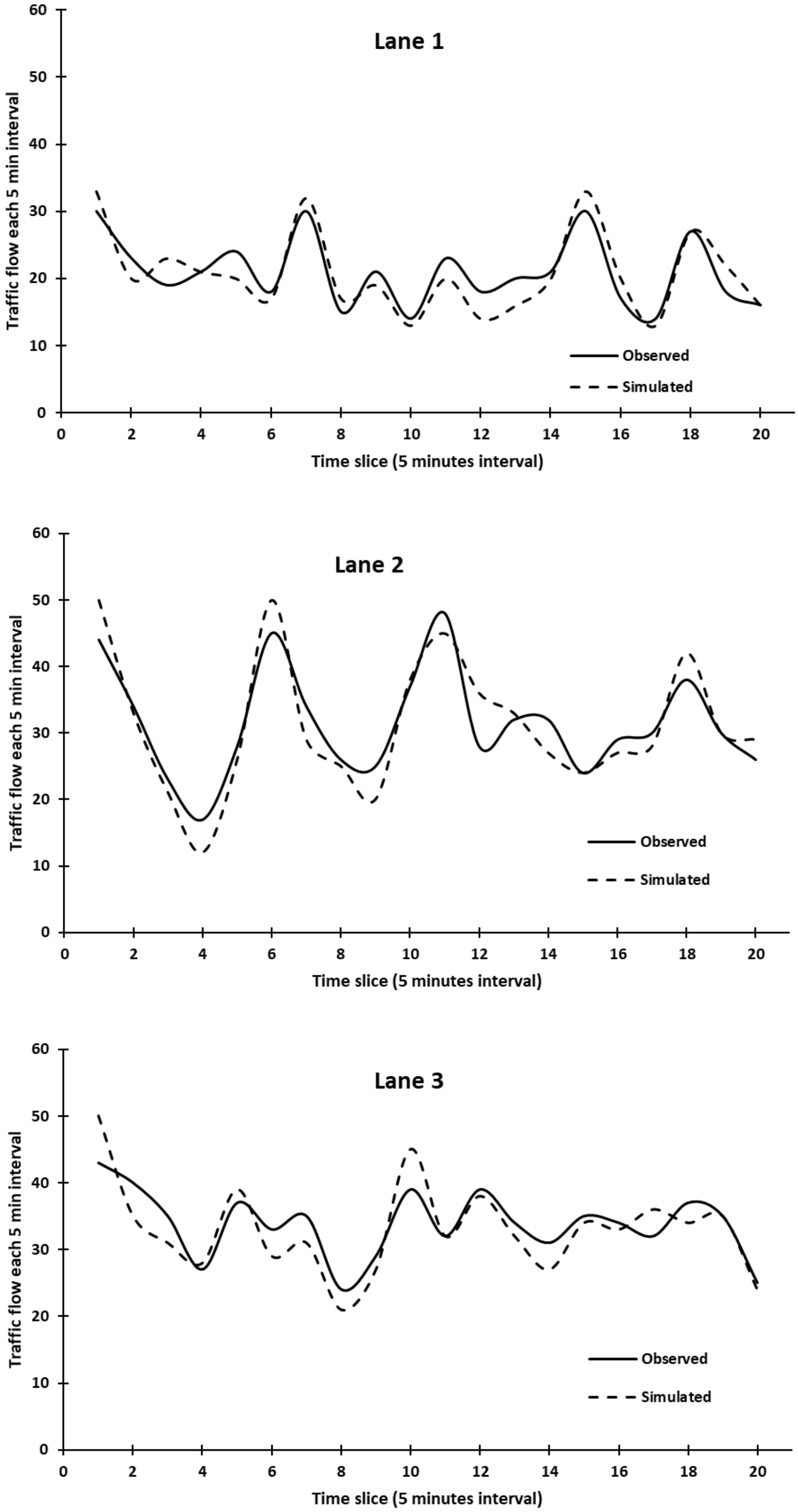


Figure 6.10: Arrival flow profiles for 3-lanes (Site #1a)

Drivers' non-compliance behaviour with the signal change (i.e. ALR and RLR) were reported for each lane separately. As mentioned in Section 6.4.2, different drivers' reaction times were tested to investigate drivers' responses to the amber/red signal. Comparison between the simulated results and the observed data are shown in Figure 6.11. It can be concluded that drivers with a reaction time equal to or less than 1 sec gives the best fit between the observed and the simulated data. The calibrated driver response gives an indication that 70% of drivers may comply with the signal change and be able to stop comfortably if they have a sufficient distance from the stopline.

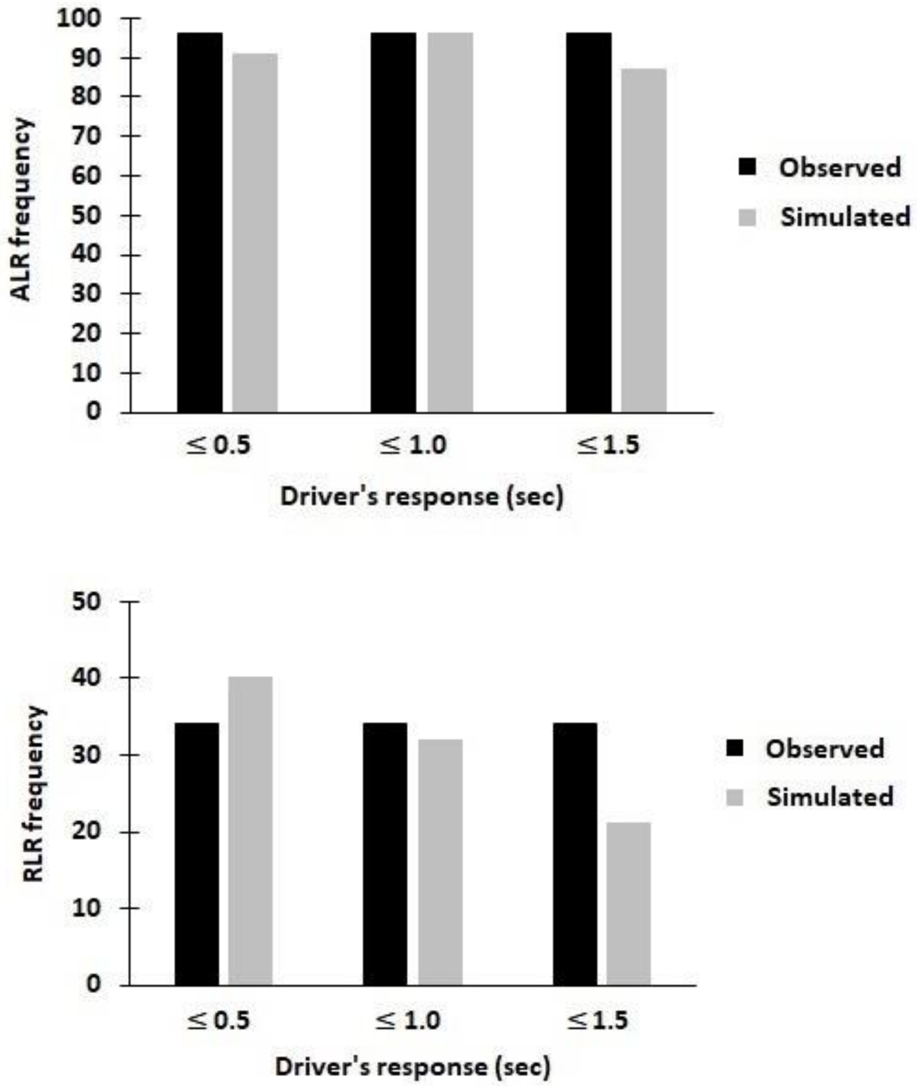


Figure 6.11: Calibrated values for drivers' responses to the signal change at Site #1a

6.4.2.1.2 Site #5a (30 mph signalised junction)

Data collected from 30 mph junction (Site #5a observed for 2 hrs) were used as input data for the purposes of simulation and the calibration process. Details of this site can be seen in Table 6.6. The amber and red-amber periods were reported as 3 and 2 seconds respectively.

Table 6.6: Model input parameters for Site #5a

Intersection details	**Intersection width in m**	30	
	Speed limit in kph	48	
	Number of lanes	2	
		Lane 1	Lane 2
Flow details	**Arrival flow in vph**	357	314
	HGVs (%)	3.4	1.5
	PC speed (μ,σ) in kph	(48, 5.0)	
	HGV speed (μ,σ) in kph	(45, 4.0)	
Traffic light details	**Green period in sec**	35	
	Red period in sec	28	
	All-red period in sec	4	

The 5-minutes interval arrival flow profile for each lane was obtained from the simulation model and compared with the observed data as shown in Figure 6.12. The statistical tests were carried out to test the goodness of fit as presented in Table 6.7. The results showed good agreement between the observed and simulated data.

Table 6.7: Statistical tests for arrival flow data calibration (Site #5a)

Lane #	**Statistical tests**					
	RMSE	***RMSPE%***	***GEH***	***U***	U_m	U_s
1	3.08	10.50	1.60	0.05	0.03	0.06
2	3.19	12.72	1.99	0.06	0.01	0.08

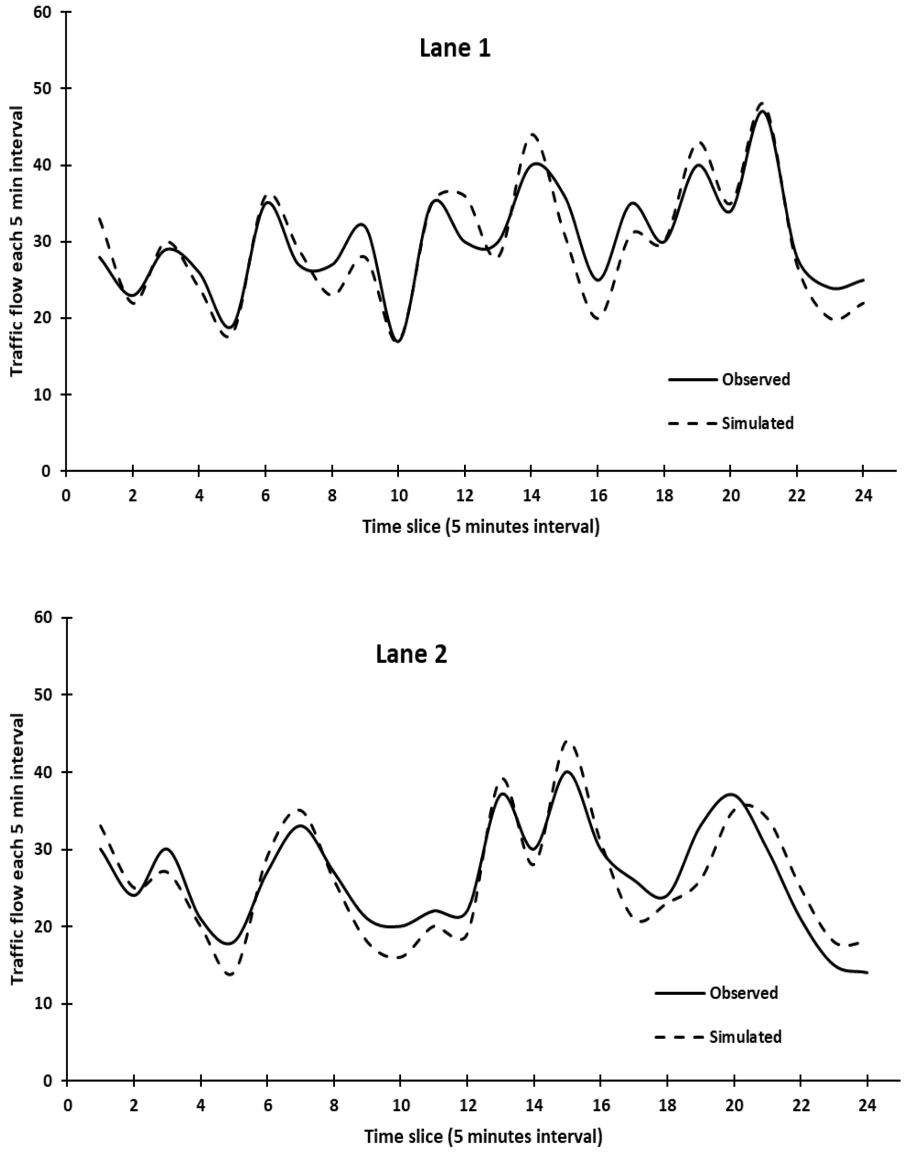


Figure 6.12: Arrival flow profiles for 2-lanes (Site #5a)

The number of ALR and RLR drivers were reported on each lane separately. The simulated results and the observed data are shown in Figure 6.13. Similar to Site #1a, it was concluded that drivers with reaction times equal to or less than 1 sec give the best fit between the observed and the simulated data. The calibrated driver response gives an indication that 70% of drivers may comply with the signal after seeing the amber indication.

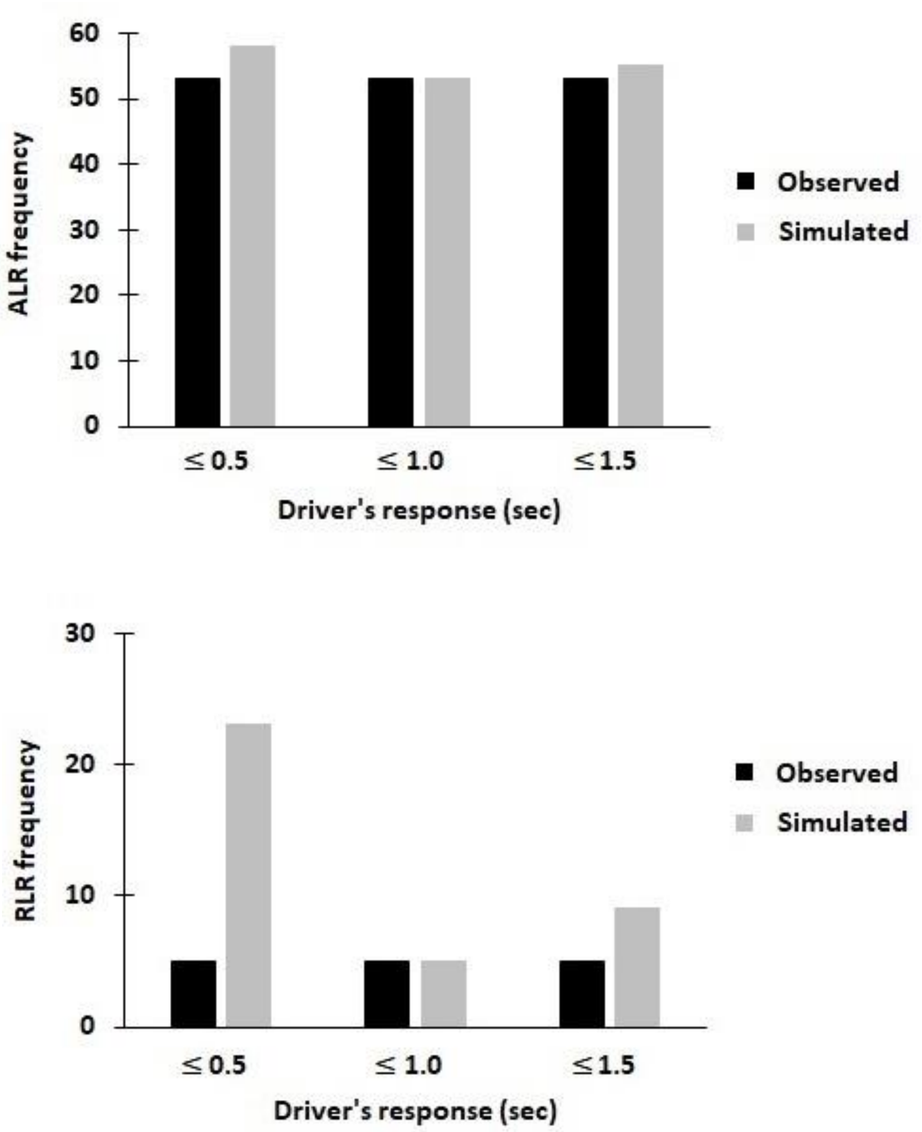


Figure 6.13: Calibrated values for drivers' responses to the signal change at Site #5a

6.4.2.2 Vehicle Actuated time signal (VA) sites

6.4.2.2.1 Site #3a (40 mph signalised junction)

Data from three lanes were collected from a 40 mph signalised junction (Site #3a observed for 1 hr and 45 min) and used as input data for the calibration process. Details of this site can be seen in Table 6.8.

Table 6.8: Model input parameters for Site #3a

Intersection details	**Intersection width in m**	29		
	Speed limit in kph	64		
	Number of lanes	3		
		Lane 1	Lane 2	Lane 3
Flow details	**Arrival flow in vph**	467	643	670
	HGVs (%)	4.0	2.7	2.2
	PC speed (μ,σ) in kph	(53, 5.88)	(55, 6.73)	(57, 5.77)
	HGV speed (μ,σ) in kph	(50, 4.0)	(52, 3.0)	(54, 5.0)
Traffic light details	**Min green period in sec**	75		
	Max green period in sec	87		
	Red period in sec	32		
	All-red period in sec	5		

The 5-minutes interval observed flow profile for each lane was compared with that obtained from the simulation model as shown in Figure 6.14. The goodness of fit was measured using different statistical tests as presented in Table 6.9. The results showed good agreement between the observed and simulated data.

Table 6.9: Statistical tests for arrival flow data calibration (Site #3a)

Lane #	**Statistical tests**					
	RMSE	***RMSPE%***	***GEH***	U	U_m	U_s
1	4.77	12.35	2.29	0.10	0.03	0.15
2	6.28	10.49	2.43	0.10	0.12	0.00
3	4.24	7.46	1.65	0.06	0.08	0.05

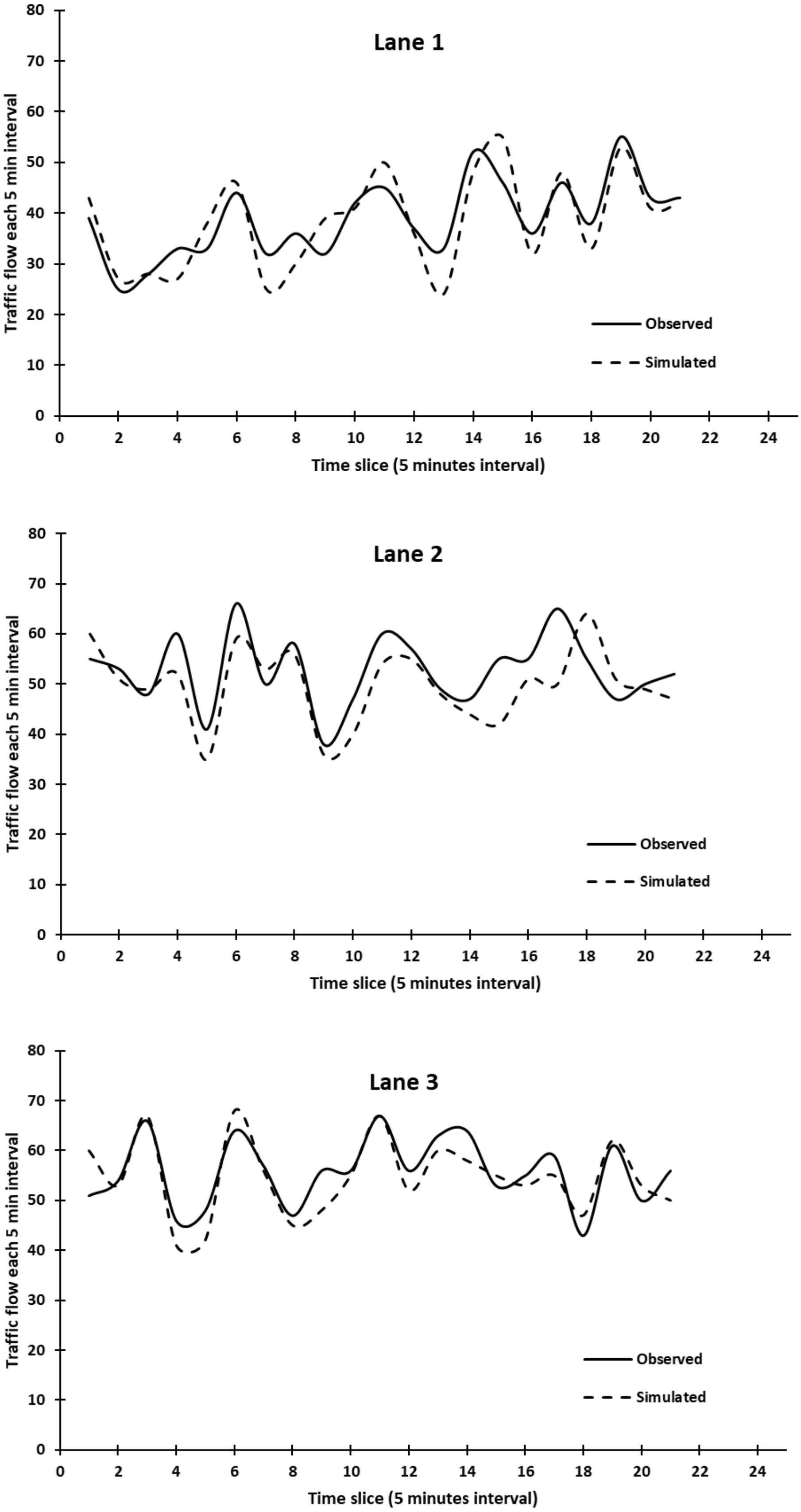
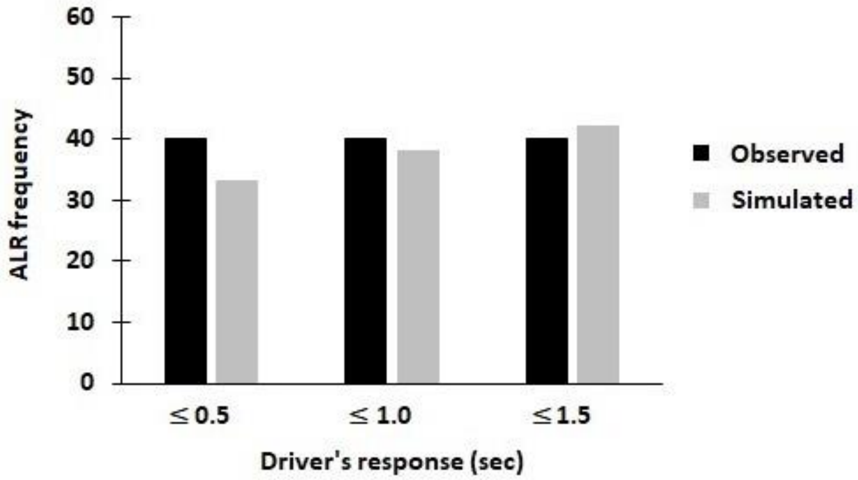


Figure 6.14: Arrival flow profiles for 3-lanes (Site #3a)

The number of ALR and RLR events were counted in the system in order to compare them with the observed data. As mentioned earlier in Section 6.4.2, different values were tested to investigate driver response to the amber/red signal. For calibration purposes, it was assumed that a group of non-compliance drivers who have a surprised reaction time of more than 1 sec are able to run the amber and red signal. This can be considered as a significant factor in signal violation. However, there was another percentage (50% of drivers) which need to be added to give better results regarding driver non-compliance at the VA site. This percentage was added based on the hypothesis that some drivers make a GO decision when they are surprised by the amber/red light aspect. Fitting the simulated results with the observed data is shown in Figure 6.15.



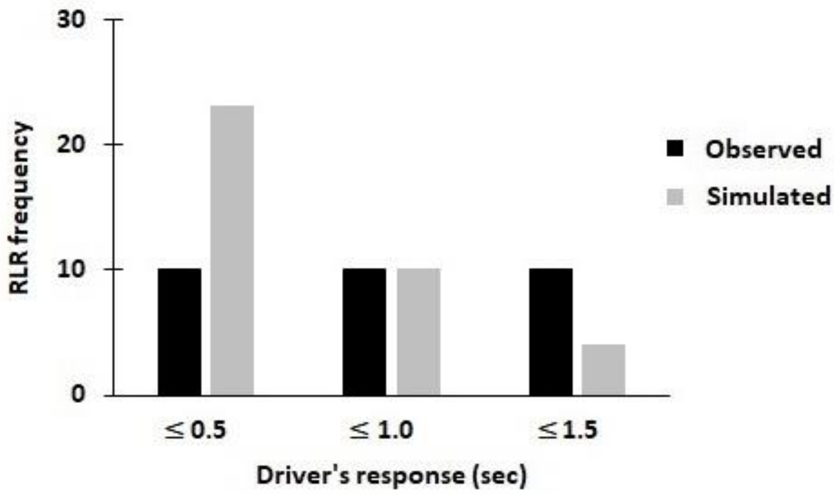


Figure 6.15: Calibrated values for drivers' responses to the signal change at Site #3a

6.4.2.2.2 Sites #4a and b (30 mph signalised junctions)

Two data sets were collected from 30 mph junctions (Site #4a and #4b observed for 2 hrs each) and used as input data for the calibration process. The input details for this site can be seen in Table 6.10.

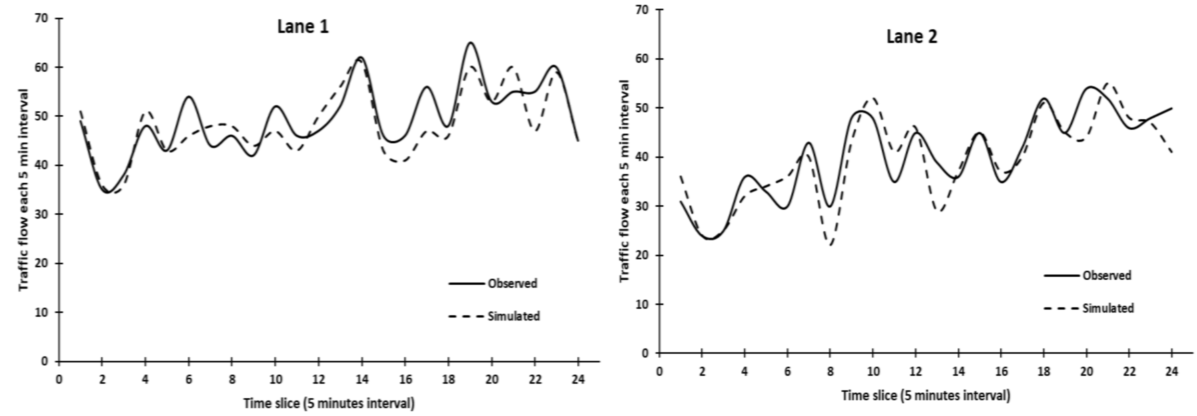
Table 6.10: Model input parameters for Site #4a and b

Data set from		Site #4a		Site #4b	
Intersection details	Intersection width in m	43		43	
	Speed limit in kph	48		48	
	Number of lanes	2		2	
		Lane 1	Lane 2	Lane 1	Lane 2
Flow details	Arrival flow in vph	595	486	724	691
	HGVs (%)	13.2	1.0	3.9	0.3
	PC speed (μ,σ) in kph	(48, 5.3)	(48, 5.6)	(56, 5.09)	(56, 5.21)
	HGV speed (μ,σ) in kph	(45, 4.5)	(45, 4.5)	(53, 3.0)	(54, 3.0)
Traffic light details	Min green period in sec	46		64	
	Max green period in sec	60		78	
	Red period in sec	28		26	
	All-red period in sec	1		1	

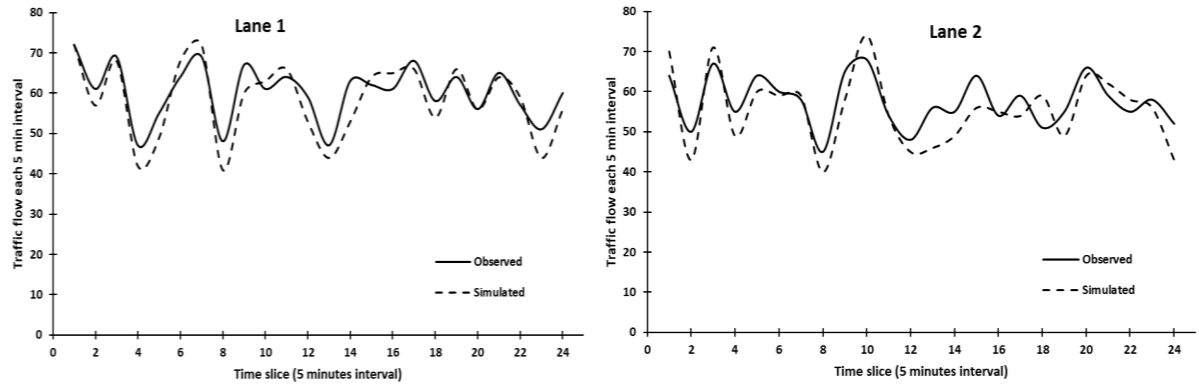
The simulated 5-minutes interval flow profile for each lane was drawn with the observed data as shown in Figure 6.16. Table 6.11 provides statistical tests that showed good fit between the observed and modelled data for both data sets.

Table 6.11: Statistical tests for arrival flow data calibration (Site #4a and b)

Statistical tests	Data set from Site #4a		Data set from Site #4b	
	Lane 1	Lane 2	Lane 1	Lane 2
RMSE	3.08	3.64	4.42	5.43
RMSPE%	10.50	12.43	7.75	9.68
GEH	1.60	1.95	1.70	2.20
U	0.05	0.07	0.04	0.05
U_m	0.03	0.01	0.09	0.04
U_s	0.02	0.03	0.09	0.06



(a) Observed and simulated data from Site #4a



(b) Observed and simulated data from Site #4b

Figure 6.16: Arrival flow profiles for 2-lanes (Site #4a and b)

Similar procedures used for calibrating Site #3a were applied to calibrate this site (i.e. 30 mph VA signal junction). Firstly, it was assumed that drivers with 1 sec reaction times or less are able to comply with the signal change. Then, it was assumed that 50% of drivers are non-alerted and have a tendency to violate the amber/red signal. Figures 6.17 and 6.18 show the simulated and the observed data for Site #4a and #4b, respectively.

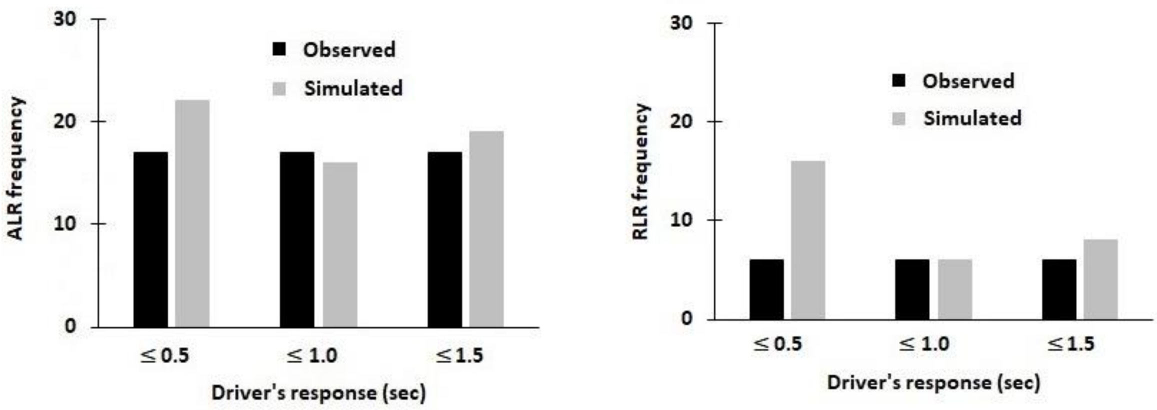


Figure 6.17: Calibrated values for drivers' responses to the signal change at Site #4a

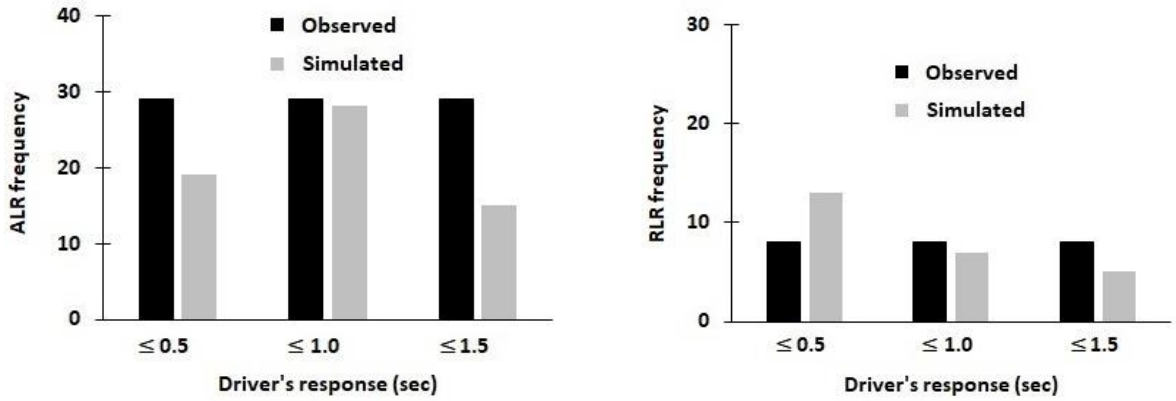


Figure 6.18: Calibrated values for drivers' responses to the signal change at Site #4b

6.5 Model validation process

After calibrating the developed model using real data sets collected from different junctions controlled by FT and VA signal settings, it is necessary to check that the model has achieved the goals of this study by replicating the observed driver behaviour and his/her response to the signal change. Details of sites used for the model validation process were summarised previously in Table 6.1 and are described in the following sections.

6.5.1 Fixed Time signal (FT) sites

6.5.1.1 Sites #1b and #2 (40 mph signalised junctions)

Two data sets were collected from 40 mph signalised junctions (Site #1b and Site #2 observed for 2 hrs each) and used as inputs for the purposes of the model validation process. Table 6.12 provides a summary of the input parameters for both sites.

Table 6.12: Model input parameters for FT signal-controlled junctions

<table>
<tr><th colspan="2">Data set from</th><th colspan="3">Site #1b</th><th colspan="2">Site #2</th></tr>
<tr><td rowspan="4">Intersection details</td><td>Intersection width in m</td><td colspan="3">25</td><td colspan="2">30</td></tr>
<tr><td>Speed limit in kph</td><td colspan="3">64</td><td colspan="2">64</td></tr>
<tr><td rowspan="2">Number of lanes</td><td colspan="3">3</td><td colspan="2">2</td></tr>
<tr><td>Lane 1</td><td>Lane 2</td><td>Lane 3</td><td>Lane 1</td><td>Lane 2</td></tr>
<tr><td rowspan="4">Flow details</td><td>Arrival flow in vph</td><td>253</td><td>374</td><td>438</td><td>465</td><td>375</td></tr>
<tr><td>HGVs (%)</td><td>5.9</td><td>6.1</td><td>3.1</td><td>7.4</td><td>2.8</td></tr>
<tr><td>PC speed (μ,σ) in kph</td><td>(54, 5.22)</td><td>(56, 5.74)</td><td>(56, 5.75)</td><td>(60, 5.49)</td><td>(62, 5.60)</td></tr>
<tr><td>HGV speed (μ,σ) in kph</td><td>(53, 4.0)</td><td>(54, 4. 0)</td><td>(56, 4.0)</td><td>(58, 4.0)</td><td>(58, 4.0)</td></tr>
<tr><td rowspan="3">Traffic light details</td><td>Green period in sec</td><td colspan="3">33</td><td colspan="2">42</td></tr>
<tr><td>Red period in sec</td><td colspan="3">34</td><td colspan="2">46</td></tr>
<tr><td>All-red period in sec</td><td colspan="3">8</td><td colspan="2">6</td></tr>
</table>

The arrival flow profile (for each 5-minutes interval) was observed for each lane separately and was compared statistically with the simulated data for Site #1b and Site #2 as shown in Figures 6.19 and 6.20, respectively. The results showed good agreement between the observed and modelled flows as presented in Table 6.13. These results were obtained after calibrating the shift value of (0.5 sec), as described in Section 6.4.1.

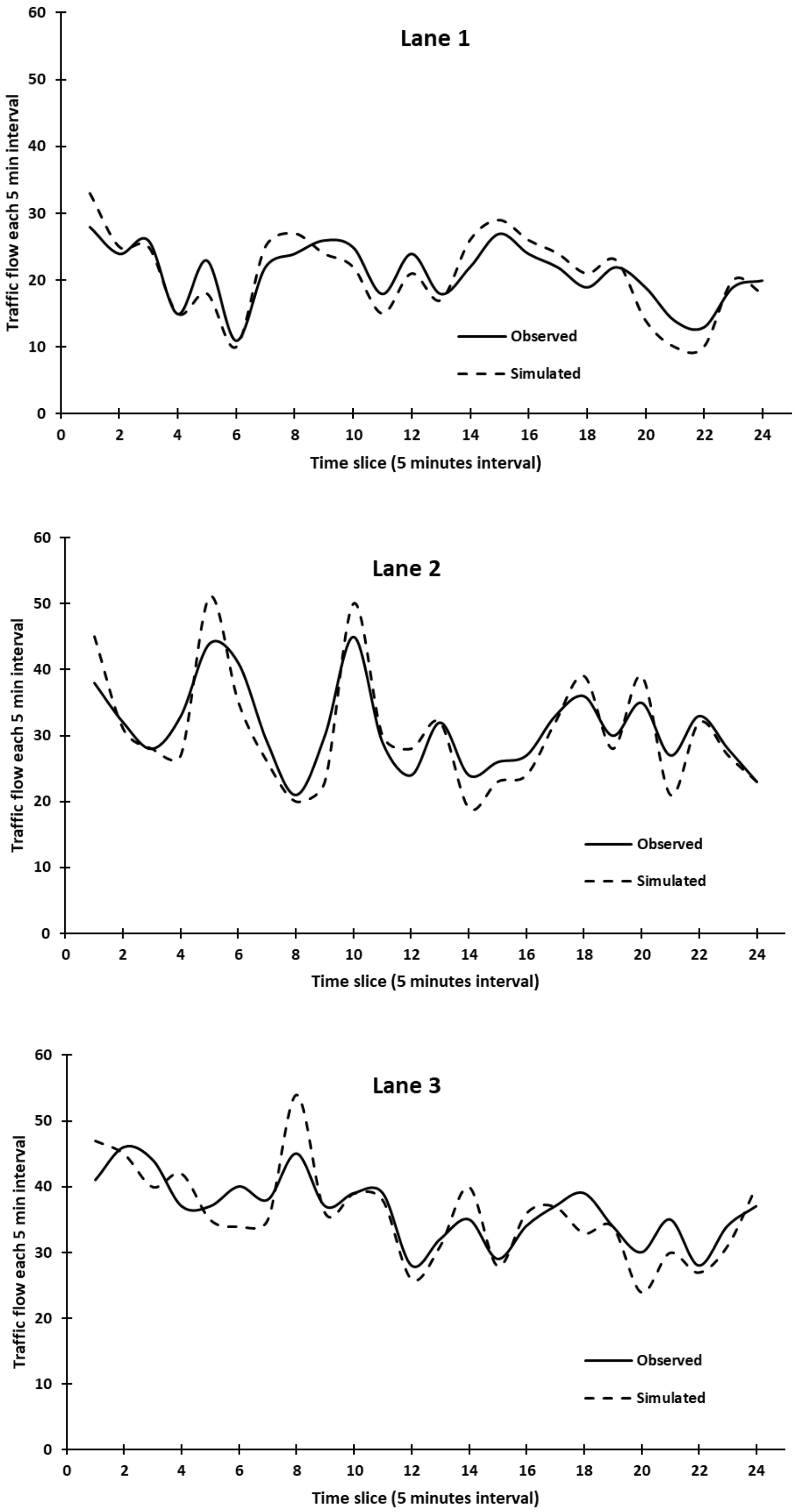
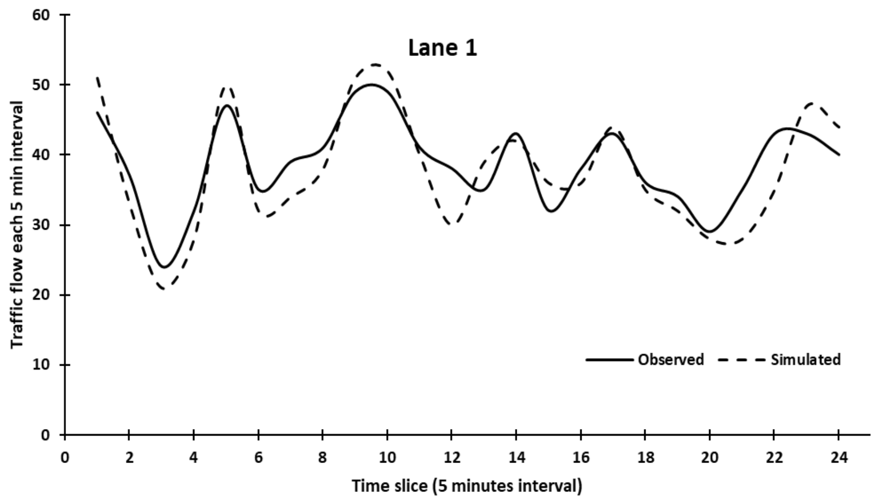


Figure 6.19: Arrival flow profiles for 3-lanes (Site #1b)



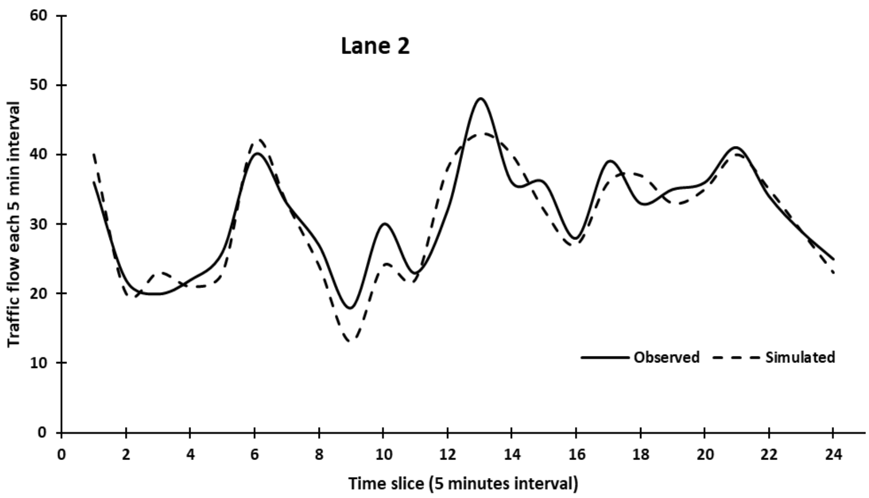


Figure 6.20: Arrival flow profiles for 2-lanes (Site #2)

Table 6.13: Statistical tests for arrival flow data validation (Sites #1b and #2)

Statistical tests	Data set from Site #1b			Data set from Site #2	
	Lane 1	Lane 2	Lane 3	Lane 1	Lane 2
RMSE	2.82	3.99	3.88	4.01	3.19
RMSPE%	12.08	12.41	10.25	10.60	11.12
GEH	1.90	1.99	1.74	1.97	1.70
U	0.07	0.06	0.05	0.05	0.05
U_m	0.01	0.02	0.02	0.06	0.04
U_s	0.04	0.05	0.08	0.03	0.07

The ALR and RLR events were reported for each lane and compared with the simulation outputs for the purpose of the validation process. As explained earlier in Section 6.4.2.1.1, driver response to the amber/red signal was calibrated. The calibrated value was 1.0 sec. Figures 6.21 and 6.22 show a good fit between the real and simulated data for Site #1b and Site #2, respectively.

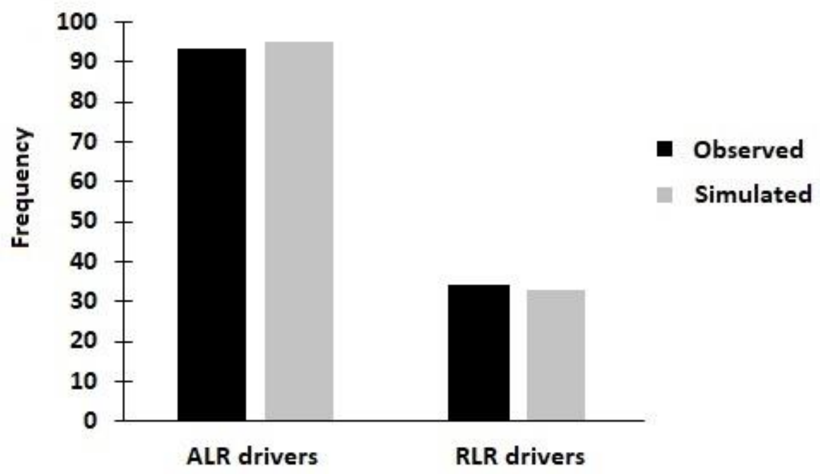


Figure 6.21: Model validation for drivers' non-compliant behaviour at Site #1b

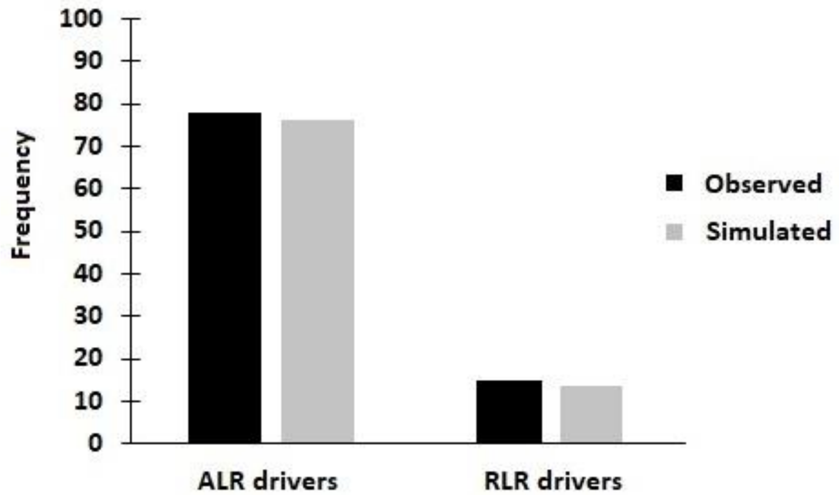


Figure 6.22: Model validation for drivers' non-compliant behaviour at Site #2

6.5.1.2 Site #5b (30 mph signalised junction)

Site #5b was selected for the model validation purpose. The required data for inputs were collected and listed in Table 6.14. In addition, the recommended 3 sec amber and 2 red-amber as well as the intersection width were also added as input parameters.

Table 6.14: Model input parameters for Site #5b

Intersection details	**Intersection width in m**	30	
	Speed limit in kph	48	
	Number of lanes	2	
		Lane 1	Lane 2
Flow details	**Arrival flow in vph**	322	248
	HGVs (%)	3.7	3.3
	PC speed (μ,σ) in kph	(48, 4.0)	
	HGV speed (μ,σ) in kph	(45, 4.0)	
Traffic light details	**Green period in sec**	35	
	Red period in sec	28	
	All-red period in sec	4	

The profile of the 5-minutes arrival flow was observed for each lane separately and was compared statistically with the modelled data as shown in Figure 6.23. The results showed good agreement between the observed and simulated flows as presented in Table 6.15.

Table 6.15: Statistical tests for arrival flow data validation (Site #5b)

Lane #	**Statistical tests**					
	RMSE	***RMSPE%***	***GEH***	***U***	U_m	U_s
1	2.45	9.90	1.08	0.04	0.00	0.08
2	2.57	8.95	1.74	0.06	0.03	0.05

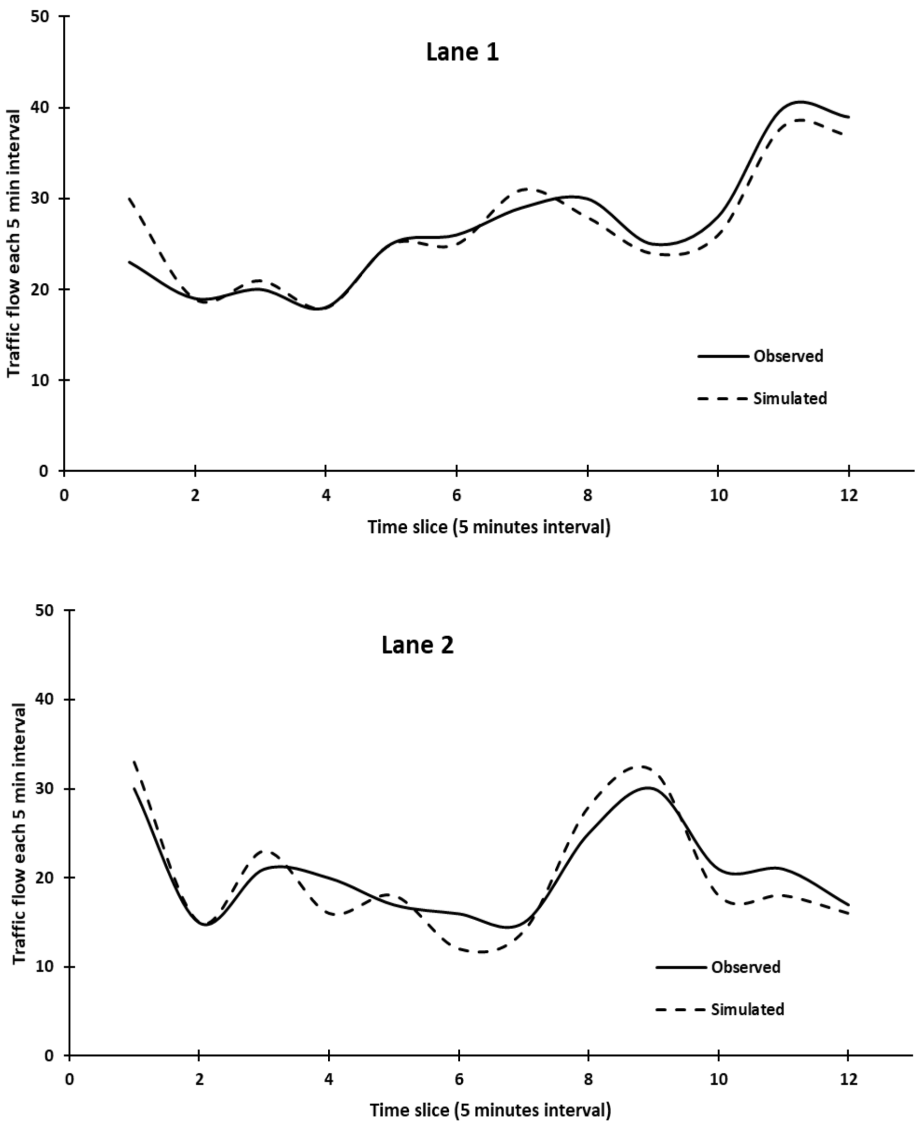


Figure 6.23: Arrival flow profiles for 2-lanes (Site #5b)

Finally, the reported non-complying drivers (i.e. ALR and RLR drivers) for the site were compared with those obtained from the simulation model, as presented in Figure 6.24. It can be seen that the simulated results are consistent with the observed data after calibrating driver response to the signal change.

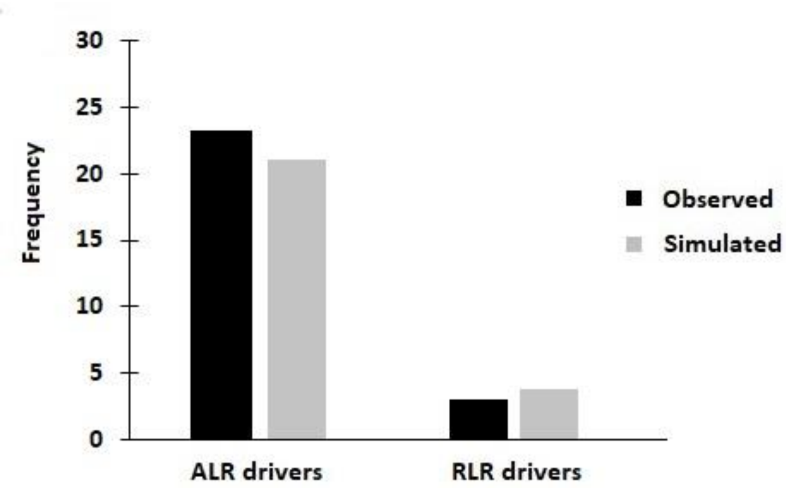


Figure 6.24: Model validation for drivers' non-compliant behaviour at Site #5b

6.5.2 Vehicle Actuated time signal (VA) sites

6.5.2.1 Site #3b (40 mph signalised junction)

A data set collected from Site #3b was used for the model validation purpose. The input parameters can be summarised in Table 6.16. Figure 6.25 shows the profile for the arrival flow for each lane.

Table 6.16: Model input parameters for Site #3b

Intersection details	**Intersection width in m**	29		
	Speed limit in kph	64		
	Number of lanes	3		
		Lane 1	Lane 2	Lane 3
Flow details	**Arrival flow in vph**	393	504	535
	HGVs (%)	3.2	1.6	2.9
	PC speed (μ,σ) in kph	(52, 6.3)	(54, 5.73)	(56, 5.76)
	HGV speed (μ,σ) in kph	(51, 3.0)	(51, 4.0)	(56, 4.0)
Traffic light details	**Min green period in sec**	63		
	Max green period in sec	75		
	Red period in sec	26		
	All-red period in sec	5		

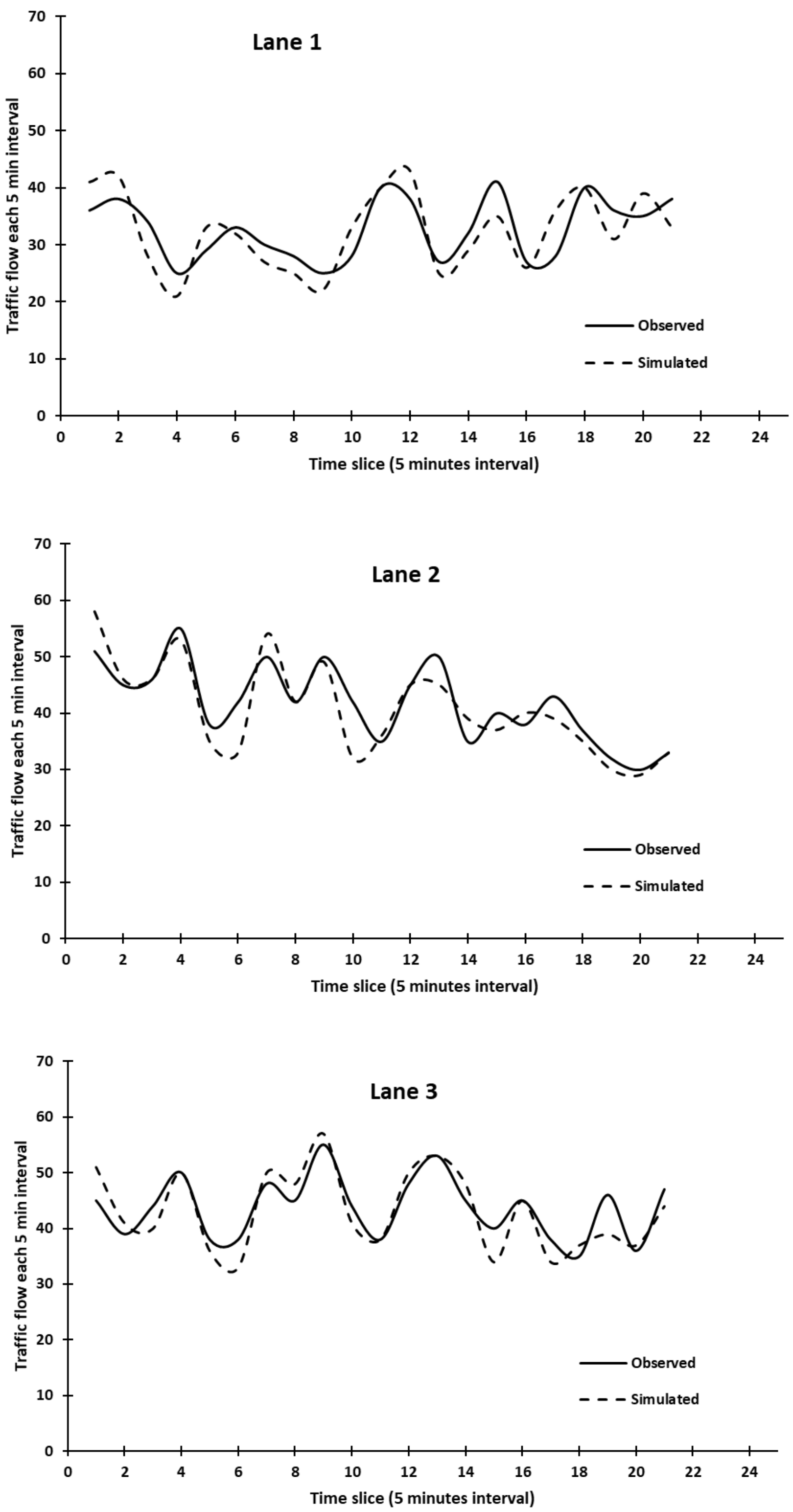


Figure 6.25: Arrival flow profiles for 3-lanes (Site #3b)

Table 6.17 presents the statistical test for measuring the goodness of curve fitting. The observed arrival flow showed a good representation with the simulation model after calibrating the shift value of (0.5 sec).

Table 6.17: Statistical tests for arrival flow data validation (Site #3b)

Lane #	Statistical tests					
	RMSE	***RMSPE%***	***GEH***	***U***	U_m	U_s
1	4.14	11.88	2.22	0.10	0.03	0.10
2	4.22	9.85	1.74	0.08	0.09	0.08
3	3.75	8.46	1.57	0.07	0.07	0.12

In addition, signal violation events (i.e. ALR and RLR drivers) were observed and reported in this study in order to compare them with the simulated results after calibrating for driver response at junctions controlled by VA signals. Figure 6.26 shows that the observed data are very close to the simulation outputs after calibrating for driver response to the traffic signal change.

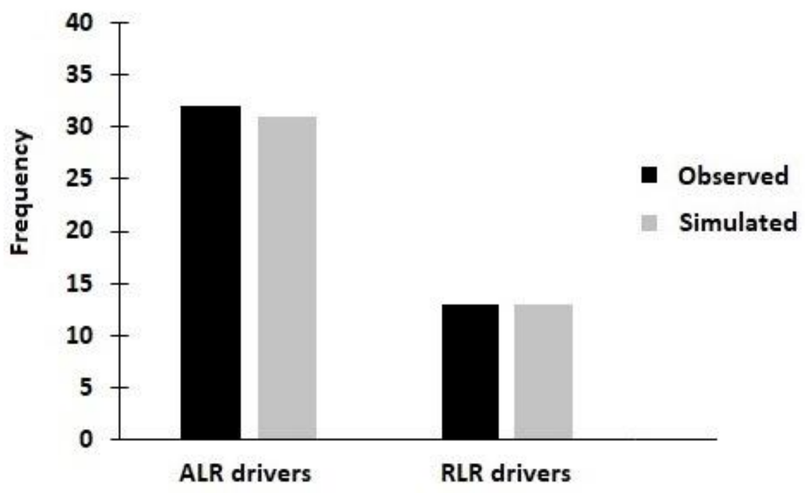


Figure 6.26: Model validation for drivers' non-compliant behaviour at Site #3b

6.5.2.2 Site #4c (30 mph signalised junction)

A data set collected from Site #4c was used for the model validation process. The input parameters are listed in Table 6.18. The profile of arrival flow for each lane is illustrated in Figure 6.27. The difference between the modelled and observed arrival flow was tested statistically by different measures as presented in Table 6.19. The results show that there are no significant differences between both data.

Table 6.18: Model input parameters for Site #4c

Intersection details	**Intersection width in m**	43	
	Speed limit in kph	48	
	Number of lanes	2	
		Lane 1	Lane 2
Flow details	**Arrival flow in vph**	663	624
	HGVs (%)	4.3	1.0
	PC speed (μ,σ) in kph	(56, 6.40)	(57, 5.95)
	HGV speed (μ,σ) in kph	(53, 3.00)	(54, 3.00)
Traffic light details	**Min green period in sec**	64	
	Max green period in sec	78	
	Red period in sec	26	
	All-red period in sec	1	

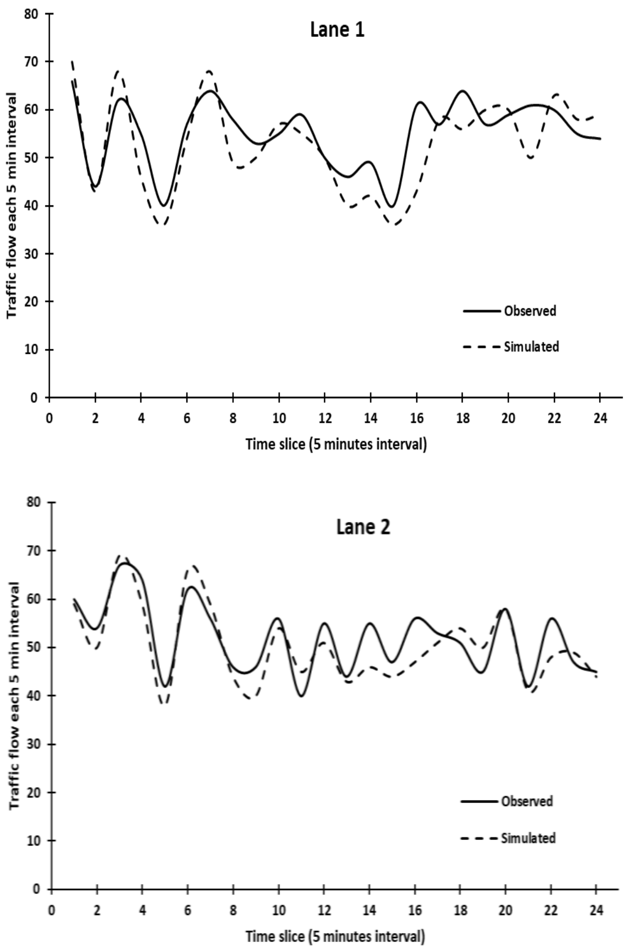


Figure 6.27: Arrival flow profiles for 2-lanes (Site #4c)

Table 6.19: Statistical tests for arrival flow data validation (Site #4c)

Lane #	Statistical tests					
	RMSE	*RMSPE%*	*GEH*	*U*	U_m	U_s
1	5.92	10.00	2.36	0.06	0.09	0.06
2	4.34	8.36	1.75	0.04	0.08	0.02

Finally, driver non-compliance (i.e. ALR and RLR drivers) was observed and reported in this study in order to compare it with the simulated results. Figure 6.28 shows small differences between the observed data and the simulation outputs after obtaining the calibrated driver response of (1.0 sec).

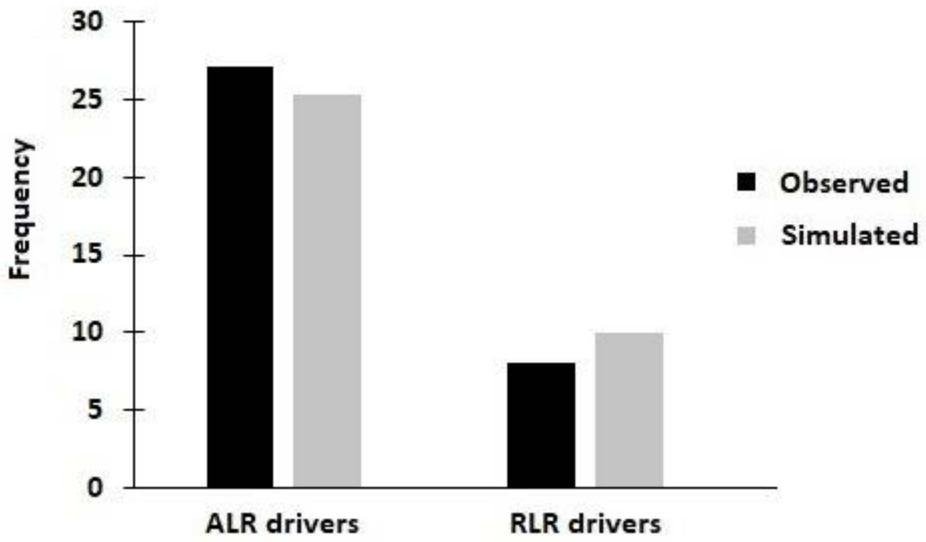
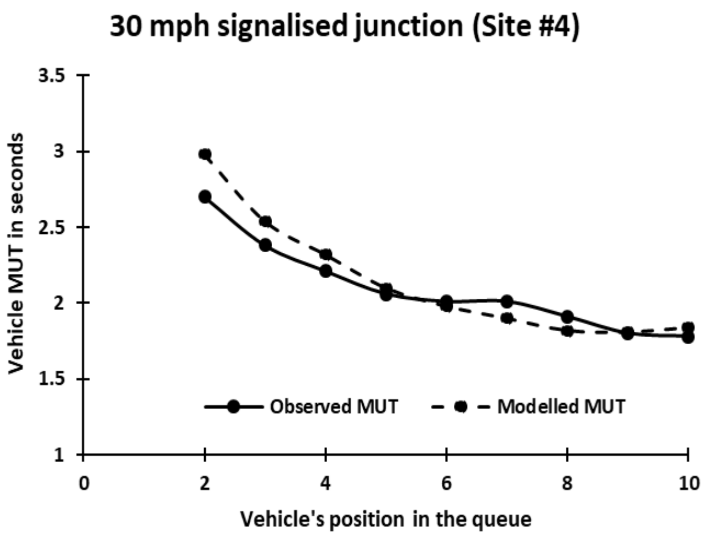


Figure 6.28: Model validation for drivers' non-compliant behaviour Site #4c

6.5.3 Validation of Move-Up-Time (MUT)

Vehicles' MUTs were reported at the survey sites (Sites #1 and #4) for comparison with those from the developed model. As illustrated in Figure 6.29, it can be suggested that the simulated data are consistent with the observed data can be considered acceptable. The differences between field data and simulation outputs were tested statistically using different measures, as shown in Table 6.20.



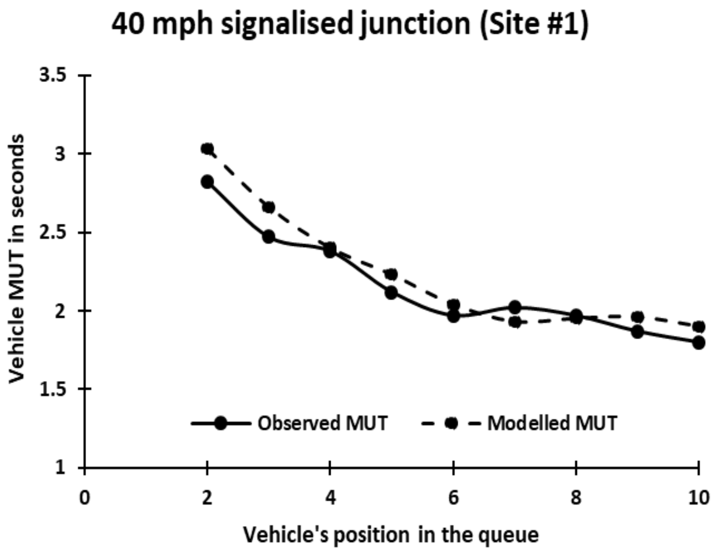


Figure 6.29: Model validation of observed and modelled vehicles' MUTs at 30 mph and 40 mph traffic signal junctions

Table 6.20: Statistical tests for vehicles' MUT data at 30 mph and 40 mph approaches to the traffic signal junctions

Approach speed limit (mph)	**Statistical tests**					
	RMSE	***RMSPE%***	***GEH***	***U***	U_m	U_s
30	0.13	5.24	0.08	0.03	0.14	0.17
40	0.12	5.05	0.07	0.03	0.13	0.15

6.6 Summary

This chapter presents the model verification, calibration and validation stages for replication of driver behaviour at signalised intersections. Real data were collected from 5 sites which are different in flow level, signal control setting and junction geometry. The observed data were used for comparison with the model outputs after calibrating and validating the developed model. Table 6.21 presents a comparison between the observed and simulated data. The main points can be listed as follows:

1. The verification stage was achieved for the developed model by observing the model outputs. The results showed that the micro-simulation model performs logically after making some modifications to the coded statements and debugging any errors.
2. Behaviour of drivers approaching a signalised intersection was investigated by introducing the profile of speed, distance and acceleration/deceleration rate with time. It was found that RLR drivers have a tendency to increase their speeds when they are under the effect of the dilemma zone after seeing the amber phase. In addition, this has been confirmed by presenting the profile for speed-distance to the stopline, as discussed in Section 6.3.2.

Table 6.21: Comparison between the observed data and the simulation results after calibrating and validating the newly developed model

<table>
<tr><th rowspan="2">Approach speed (mph)</th><th rowspan="2">Site #</th><th rowspan="2">Lane #</th><th colspan="2">ALR</th><th colspan="2">RLR</th><th colspan="2">Overall crossing (ALR+RLR)</th></tr>
<tr><th>Obs.</th><th>Sim.</th><th>Obs.</th><th>Sim.</th><th>Obs.</th><th>Sim.</th></tr>
<tr><td rowspan="9">40</td><td rowspan="3">1</td><td>1</td><td>35</td><td>37</td><td>17</td><td>17</td><td>52</td><td>54</td></tr>
<tr><td>2</td><td>80</td><td>79</td><td>23</td><td>21</td><td>103</td><td>100</td></tr>
<tr><td>3</td><td>73</td><td>74</td><td>28</td><td>27</td><td>101</td><td>101</td></tr>
<tr><td rowspan="2">2</td><td>1</td><td>43</td><td>43</td><td>7</td><td>7</td><td>50</td><td>50</td></tr>
<tr><td>2</td><td>35</td><td>34</td><td>7</td><td>6</td><td>42</td><td>40</td></tr>
<tr><td rowspan="3">3</td><td>1</td><td>19</td><td>18</td><td>7</td><td>6</td><td>26</td><td>24</td></tr>
<tr><td>2</td><td>24</td><td>24</td><td>7</td><td>6</td><td>31</td><td>30</td></tr>
<tr><td>3</td><td>29</td><td>26</td><td>9</td><td>11</td><td>38</td><td>37</td></tr>
<tr><td colspan="2">Total</td><td>338</td><td>335</td><td>105</td><td>101</td><td>443</td><td>436</td></tr>
<tr><td rowspan="5">30</td><td rowspan="2">4</td><td>1</td><td>44</td><td>42</td><td>13</td><td>14</td><td>57</td><td>56</td></tr>
<tr><td>2</td><td>29</td><td>27</td><td>9</td><td>9</td><td>38</td><td>36</td></tr>
<tr><td rowspan="2">5</td><td>1</td><td>43</td><td>43</td><td>2</td><td>3</td><td>45</td><td>46</td></tr>
<tr><td>2</td><td>33</td><td>31</td><td>6</td><td>6</td><td>39</td><td>37</td></tr>
<tr><td colspan="2">Total</td><td>149</td><td>143</td><td>30</td><td>32</td><td>179</td><td>175</td></tr>
<tr><td colspan="3">Total</td><td>487</td><td>478</td><td>135</td><td>133</td><td>622</td><td>611</td></tr>
</table>

3. The calibration stage involves the main components of the developed micro-simulation model particularly in the STOP/GO sub-model. The main improvement was adding the dilemma zone rules to replicate driver behaviour when arriving at a traffic signal junction. Driver response to the signal change can be replicated by calibrating the reaction time value that may affect his/her STOP/GO decision based on the distance from the stopline and the travelling speed.
4. The simulation program was run thousands of times using different sets of filed data for the calibration stage. Then, the simulation outputs were tested and compared statistically with the real observations until the results obtained showed the best fit. In general, data resulting from the developed model showed good consistency with the real data after achieving the calibrated values (i.e. the shift value, driver's response and percentage of driver alertness at junction controlled by VA signals).
5. The validation stage was achieved for the newly developed model using real data sets that are different to those used in the calibration stage. The validation outputs showed acceptable representation with the field data which means that the validity of the developed model has been confirmed. The output data included driver compliance, traffic flow and MUT data.

The next chapter will discuss the model's applications. Several scenarios for traffic conditions at signal controlled junctions will be implemented, such as testing the length of the amber period and how that might affect driver compliance behaviour. Different HGVs% will be tested to investigate their impact on traffic capacity, vehicle delays and the number of RLR. To minimise the effect of the dilemma zone, red light cameras (RLC) will be introduced in the new micro-simulation model to test their impact on signal violations.

CHAPTER SEVEN: MODEL APPLICATIONS

7.1 Introduction

This chapter introduces some possible applications of the newly developed micro-simulation model. Different scenarios were implemented using various parameters such as the length of amber and all-red periods in order to investigate the effects of these periods on drivers' compliance. Using the new algorithms of the developed model, junction capacity and delays were tested under the effect of various percentages of HGVs at junctions controlled by FT and VA traffic light signals. In order to evaluate traffic safety after the onset of amber, time to collision (TTC) between successive vehicles was estimated at junctions controlled by FT and VA modes. Finally, the effect of the dilemma zone may be reduced by introducing red-light cameras in the developed model to improve intersection safety and performance. Full details of the above are discussed in the following sections.

7.2 Testing the length of the intergreen period

In this section, the effect of the length of the intergreen interval on driver compliance was investigated. This was conducted by increasing the length of amber and changing the all-red period so that it is close to that recommended by the Institute of Transportation Engineering (2015) as described in the following sub-sections.

7.2.1 The length of the amber period

The fixed 3 seconds of amber have been recommended by the UK Standards (2016) and Department for Transport (2006a). The impact of the length of amber on driver compliance was investigated in this study by changing the length of amber. Other input parameters used in the developed model are summarised in Table 7.1. It is worth mentioning that all drivers' and vehicles' characteristics remaining the same.

According to previous research carried out by Kennedy and Sexton (2009) and York and Al-Katib (2000), increasing the length of the amber period might increase the number of red light runnings. For the current test, different signalised junctions operated by FT and VA signals were tested. Table 7.2 lists the observed ALR and RLR events and the simulation results before and after increasing the amber period by an extra 1 second (i.e. a total of 4 seconds) on the numbers of ALR and RLR drivers. It is shown that changing the amber length to 4 seconds increased the number of RLR at all sites. This figure is found to be consistent with the findings of the aforementioned researchers which suggests that increasing the amber length leads to late exit from the intersection area.

Table 7.1: Input parameters for testing an increase in amber length (by an extra 1 sec)

Approach speed limit	Signal mode	Intersection width (m)	Cycle length (sec)	Average flow per lane (vph)	HGV%
30 mph	FT	30	69	300-350	2-4
	VA	43	78-115	500-700	1-4
40 mph	FT	30	94	350-450	3-7
	VA	29	95-125	400-600	3-6

However, this change shows a reduction (about 15%) in the number of ALR at junctions operated by VA signal settings only. In general, the intergreen period increases by increasing the amber period which causes an increase in the number of RLR drivers. In addition, it is possible that the difference in the numbers of ALR and RLR after increasing the amber period by an extra 1 second might be affected by several factors such as the vehicle position from the stopline, travelling speed, driver reaction time and driver response to the signal change.

Table 7.2: The effect of increasing the amber length (by an extra 1 sec)

Approach speed limit	Signal mode	The number of ALR				The number of RLR			
		Obs. (3 sec) amber	Sim. (3 sec) amber	Sim. (4 sec) amber	Diff (%)	Obs. (3 sec) amber	Sim. (3 sec) amber	Sim. (4 sec) amber	Diff (%)
30 mph	FT	76	74	95	+28.38	8	9	12	+33.33
	VA	73	69	59	-14.49	22	23	37	+60.87
40 mph	FT	78	77	85	+10.39	14	13	22	+69.23
	VA	72	68	42	-38.24	23	23	39	+69.56

Diff: can be defined as the measurement of percentage change in the simulation results after extending the amber length by an extra 1 second. A positive percentage refers to an increase in the simulated RLR or ALR events and vice versa.

Obs.: Observed data and Sim.: Simulated data

7.2.2 The length of all-red period

Similar procedures to that used in Section 7.2.1 were used to change the length of the all-red period. The minimum all-red period was calculated in the current study based on the design standards of the Institute of Transportation Engineering (2015) and compared with the observed period, as discussed previously in Section 4.4.2.6. The input data used for the current test is listed in Table 7.3. The recommended all-red values were set close to those calculated as shown in Table 7.3. Table 7.4 presents the influence of changing the all-red periods on the number of ALR and RLR drivers.

Table 7.3: Input parameters after changing the all-red period to the recommended values

Approach speed limit	Signal mode	Observed all-red	Calculated all-red*	Recommend -ed all-red	Cycle length (sec)	Average flow per lane (vph)	HGV %
30 mph	FT	4.0	3.52	4.0	68	300-350	2-4
	VA	1.0	3.66	4.0	80-117	500-700	1-4
40 mph	FT	6.0	2.31	3.0	90	350-450	3-7
	VA	5.0	2.85	3.0	92-122	400-600	3-6

* The minimum all-red calculated from the design standards using either Equation 4.2 or 4.3 (ITE, 2015).

Table 7.4: The effect of changing the all-red interval on ALR and RLR frequencies

Approach speed limit	Signal mode	The number of ALR				The number of RLR			
		Obs.	Sim.	Sim. recom.	Diff (%)	Obs.	Sim.	Sim. recom.	Diff (%)
30 mph	FT	76	74	74	0.00	8	9	9	0.00
	VA	73	69	61	-11.59	22	23	32	+39.13
40 mph	FT	78	77	77	0.00	14	13	13	0.00
	VA	72	68	68	0.00	23	23	22	-4.35

Diff: can be defined as the measurement of percentage change in the simulation results after changing the all-red period. A positive percentage refers to an increase in the simulated RLR or ALR events and vice versa.

Obs.: Observed data, Sim.: Simulated data, Sim. recom.: Simulation results after changing the all-red period to the recommended value by the ITE (2015)

It can be concluded that there are no significant effects from the reduction of the all-red period from the observed value to that recommended by the Institute of Transportation Engineering (2015) in terms of the number of ALR and RLR events at junctions controlled by FT signals settings. On the other hand, a reduction in ALR frequency (by approximately 12%) with an increase in the number of RLR (by about 40%) were reported on 30 mph approaches for VA traffic light junctions. This may be due to increases in the observed all-red period from 1 second to 4 seconds as recommended by the Institute of Transportation Engineering (2015) (see Table 7.3). A small reduction in RLR events was also seen on 40 mph approaches operated by VA traffic signals because the all red-period was reduced to the recommended values (from 5 to 3 seconds as illustrated in Table 7.3). In general, an increase in the all-red period leads to an increase in the intergreen time which results in an increase in the number of RLR drivers. This finding is consistent with the recommendations of Kennedy and Sexton (2009), Maxwell and Wood (2006) and York and Al-Katib (2000).

7.3 Testing the effect of traffic light signals on vehicle delays

This test was carried out to introduce the effect of FT and VA signal modes on vehicle delays. The overall delay at signalised intersections can be estimated during the effective green period only. Table 7.5 presents the input data for testing the effect of two traffic signal controls on vehicle delays. Different arrival flows were tested to estimate vehicle delays as shown in Figure 7.1. Different factors were taken into consideration in estimating vehicle delays including traffic demand, saturating flow, cycle length and effective green period. It can be indicated that vehicle delays varied from 8 to 15 sec/veh for a traffic flow range between 200 and 550 vph. This increased rapidly as the flows exceeded 550 vph for FT signalised junction and 650 vph for VA traffic signal junctions. For a junction controlled by VA mode, the overall delay is less by around 20% compared with FT signalised junctions. Finally, the delay curves follow the typical curves as presented by the Highway Capacity Manual (2000) and Rouphail et al. (1996).

Table 7.5: Input data for testing vehicle delays at junctions controlled by FT and VA traffic light signals

Traffic light signal control	Approach speed (mph)	Junction width (m)	Flow (vph)	HGVs%
FT signals	40	30	100 - 800	5.0
VA signals				

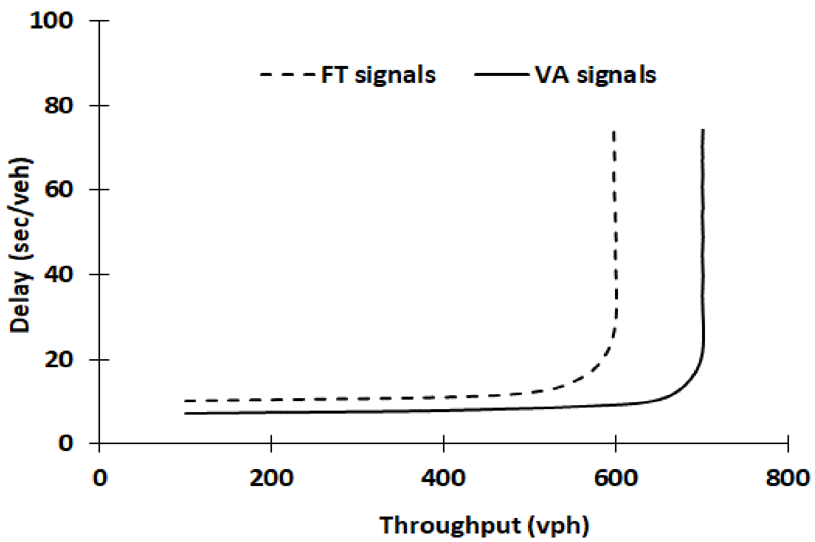


Figure 7.1: Influence of FT and VA signals on vehicle delays

7.4 Testing the effect of various HGVs%

This section discusses the effect of HGVs on signal violations, capacity and vehicle delays because they represent a traffic flow component and they occupy more space on the road than cars. Table 7.6 shows the input data for testing the effect of HGVs% on the number of RLR, capacity and delay at junctions controlled by FT and VA traffic light signals. Several tests were carried out by increasing HGVs% from 0% to 50% at increments of 10%, as described in the following sub-sections.

Table 7.6: Input data for testing the effect of various HGVs% on the number of RLR and junction capacity

Traffic light signal control	Approach speed (mph)	Junction width (m)	Flow (vph)	Observed RLR
FT signals	40	30	100 - 800	14
VA signals				

7.4.1 The effect of various HGVs% on signal violations

Previous research (such as Sayer et al. (2003) and Gates and Noyce (2010)) discussed the effect of vehicle size and weight on drivers' STOP/GO decisions near signalised intersections. The authors reported that drivers have greater headway if the leading vehicle is a HGV, in order to avoid possible collision particularly at sudden stopping conditions. In the current study, driver compliance with the signal change was tested in the newly developed model against various HGV proportions.

Figure 7.2 illustrates the reduction in percentages of RLR vehicles after the onset of amber. It can be seen that as the HGVs% increases in the traffic composition, the reduction in simulated signal violations increases for both FT and VA traffic signal junctions. For example, at 0% HGVs, the reductions are -6.79% and -10.43% for VA and FT signalised junctions, respectively (the negative sign refers to an increase in the number of simulated RLR vehicles after seeing the amber phase when there are no HGVs in the traffic flow). The reduction in RLR numbers increase gradually with an increase in HGVs% until they reach around 40% and 45% at VA and FT traffic signal junctions, respectively. This is due to the fact that following vehicles might try to keep longer headways with HGVs. In addition, HGVs decelerate faster than PCs after the onset of amber because they are moving at lower speeds and acceleration rates that increase the probability of stopping for the red phase.

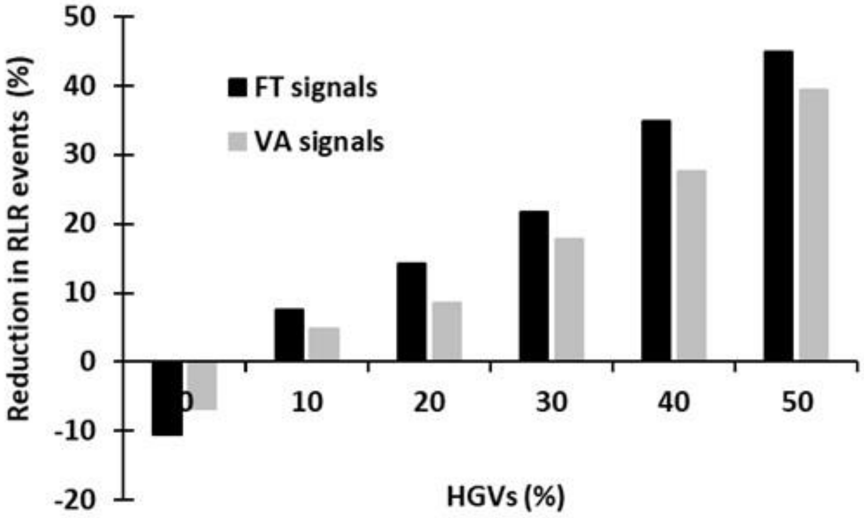


Figure 7.2: Simulation results that shows the reduction in RLR vehicles versus different HGVs% after the onset of amber at VA and FT traffic signal junctions

7.4.2 The effect of various HGVs% on junction capacity

Since capacity can be represented by the number of vehicles crossing the stopline (i.e. throughput), the effect of different HGV proportions on capacity was tested for after the traffic signal shows the green phase. An increase in HGVs% in the traffic composition causes reductions in the throughputs for FT and VA traffic signal junctions as shown in Figure 7.3. Figure 7.4 illustrates that when the HGV proportion was 50% of the traffic composition, the capacities at FT and VA traffic signal junctions were reduced by 51% and 42%, respectively.

It can be highlighted that HGVs have longer lengths and occupy more space on the road than cars. In addition, HGVs have lower acceleration and travelling speeds than PCs. Hence, they need more time to reach desired speeds and clear the junction area after the green signal comes on. Moreover, drivers are affected by longer vehicles such as buses or trucks more than other vehicles and they have a tendency to increase headways with these vehicles to avoid collisions, as discussed previously in Section 7.4.1.

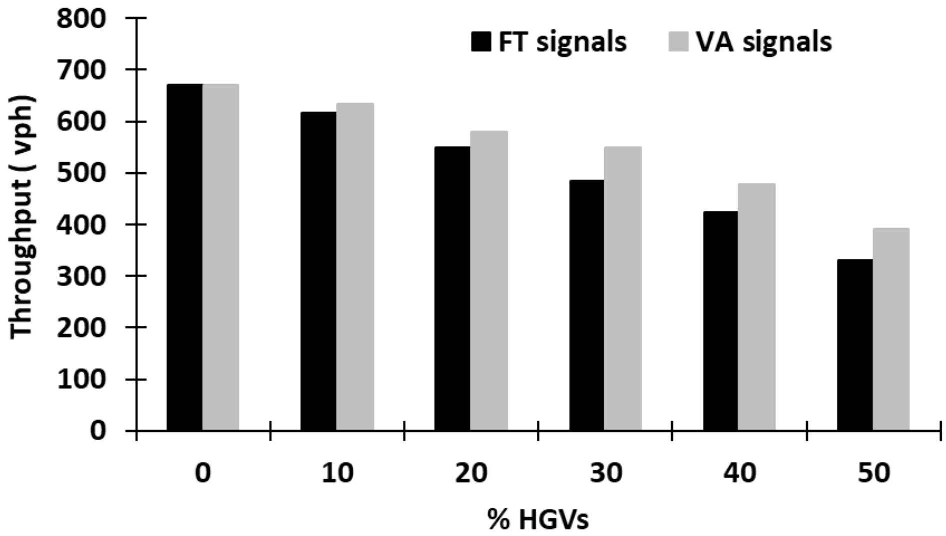


Figure 7.3: Effect of various HGVs% on junction throughput

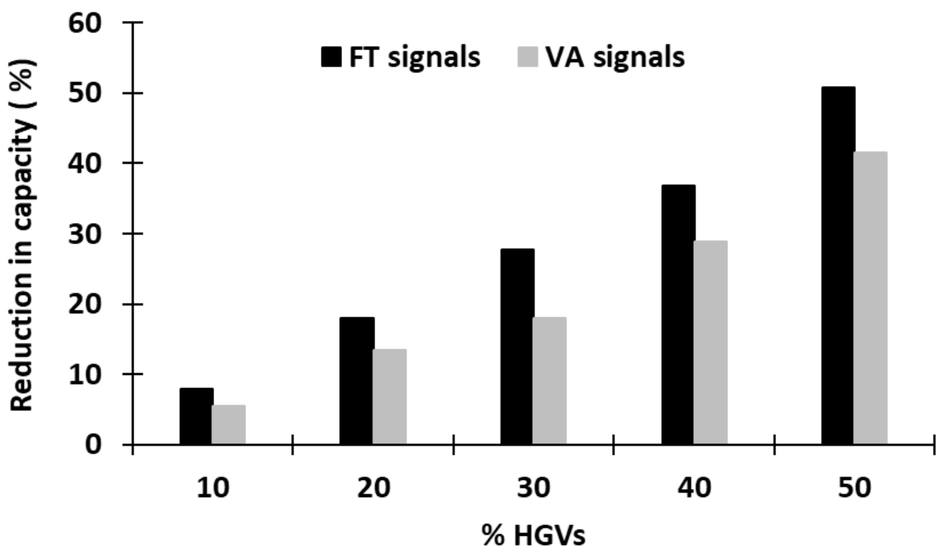


Figure 7.4: Effect of various HGVs% on capacity reduction

7.4.3 The effect of various HGVs% on vehicle delays

When the number of vehicles increases, vehicle delays increase and vehicles start to move in a platoon with lower speeds until they stop completely after the onset of the amber/red phase. Then, vehicles start their movements to clear the junction area after the onset of green. However, higher proportions of HGVs in the traffic composition cause additional delays which can be represented in acceleration delays, MUD and vehicle stop/start system after the onset of green.

The simulation results that illustrate the effect of different HGVs% on vehicle delays during the effective green at junctions controlled by either FT or VA traffic light signals are shown in Figure 7.5. It can be indicated that as HGVs% increases, the vehicle delays increase as well. In addition, delays on FT signal junctions were higher than those on VA signalised junctions. The difference between estimated delays at FT and VA signalised junctions is 18% when HGVs% is 0%. Then, the difference increases gradually to 27% when HGVs% is at 50%.

Vehicle delays under different HGVs% and traffic flow rates are shown in Figure 7.6. As HGVs increase in the traffic composition, other vehicles reduce their speeds and start to move in a platoon because of the slower acceleration of HGVs that affects throughputs, as illustrated in Figures 7.3 previously. As explained in Section 7.4.2, HGVs have longer lengths than cars; therefore, more space is occupied on the road and this causes a delay for other vehicles' inter-arrival times and the throughputs because of longer headways involving HGVs.

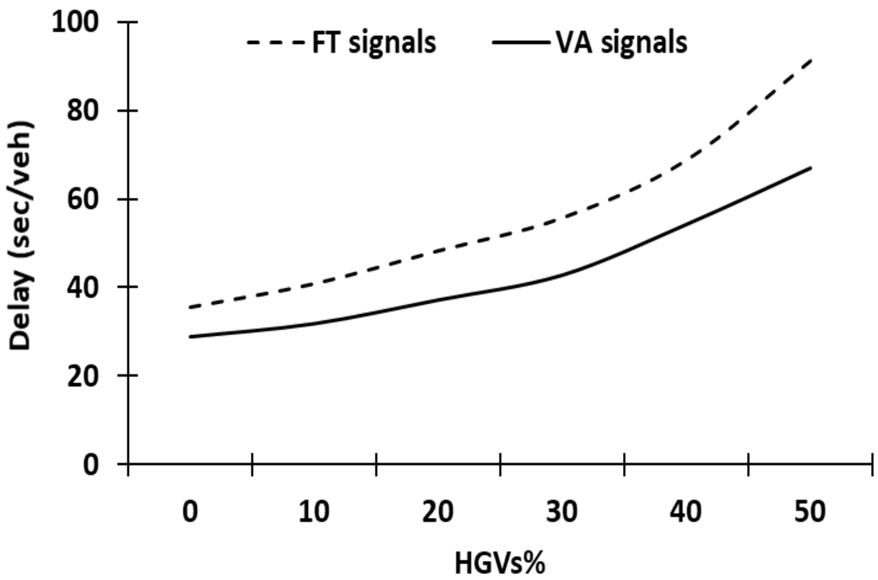


Figure 7.5: Effect of various HGVs% on vehicle delays at FT and VA traffic signal junctions

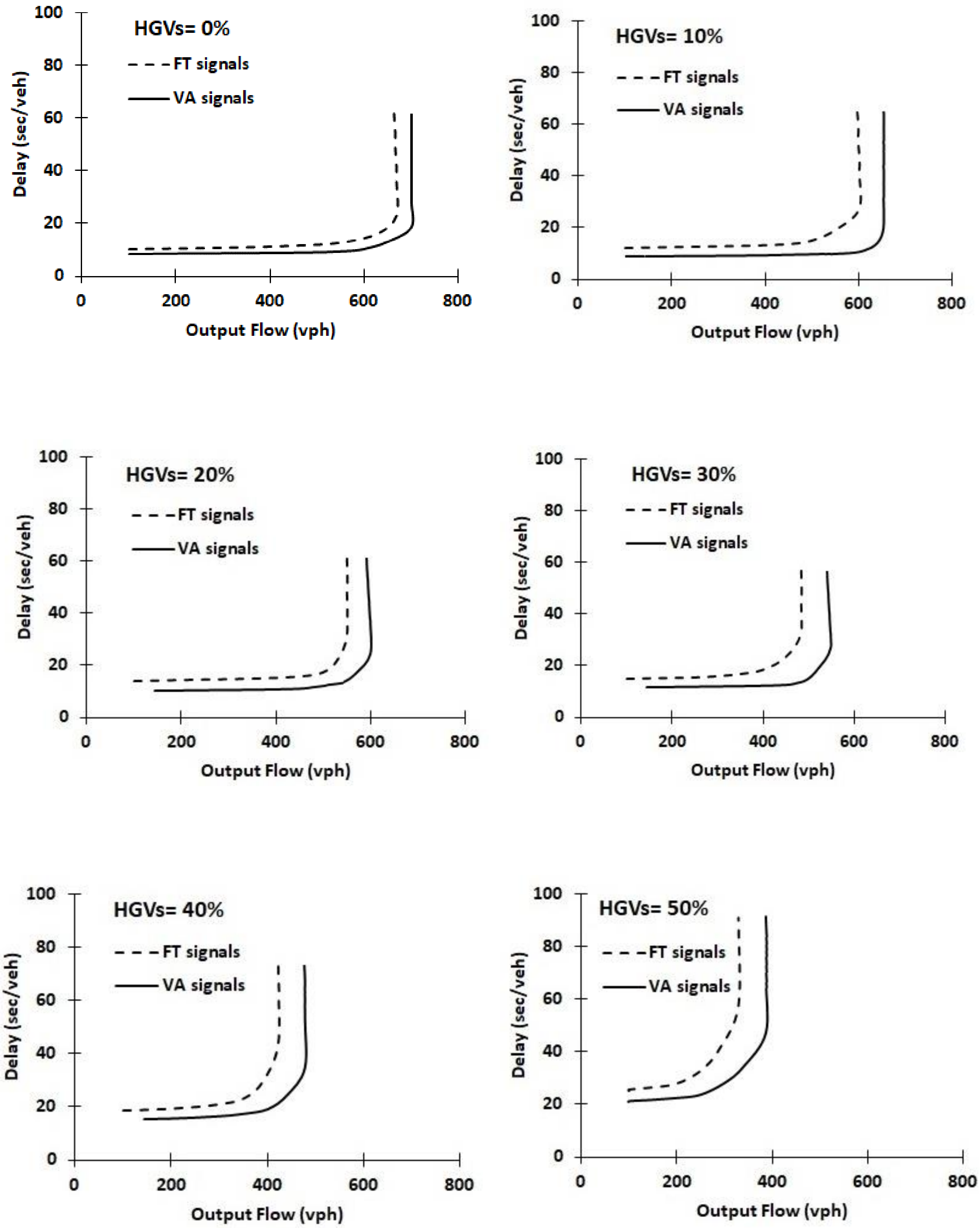


Figure 7.6: Vehicle delays at FT and VA signalised junctions under various HGVs% and traffic flow rates

7.5 Evaluation of traffic signal junction safety

Different Surrogate Safety Measures (SSMs) have been used for evaluating intersection safety. For example, Deceleration Rate to Avoid Collision (DRAC) is a promising indicator to detect braking behaviour. However, Cunto and Saccomanno (2008) have revealed that DRAC cannot replicate the occurrence of traffic conflict accurately because it does not take into consideration traffic flow and road surface conditions (i.e. dry or wet pavement) to estimate this parameter between successive vehicles.

As discussed previously in Section 2.7.5.2, the critical value of Time To Collision (TTC) was estimated to be between 1.5 and 6 seconds according to previous studies (Cavallo and Laurent, 1988; Hayward, 1972; Hirst and Graham, 1997; Hogema and Janssen, 1996; and Vogel, 2003). For the same trajectory sample of leading and following vehicles described previously in Figures 6.6 and 6.7, the developed model computes the TTC between two successive vehicles approaching a traffic light junction at every scanning time. As shown in Figure 7.7, (x) values represent the severe TTC values (that are lower than critical threshold as listed previously in Table 2.5) can be taken by the following vehicle at braking conditions if the vehicle ahead is slower. These situations can give an indication of the possibility of potential conflict occurrence, such as rear collisions. Furthermore, TTC values with different approaching speeds at the onset of amber on the approach towards a signalised junction were counted for all generated vehicles in the simulation model. About 8% of the total simulated ALR and RLR (49 out of 611) have TTC within the critical threshold values (i.e. the simulated TTC values were within the critical values as shown previously in Table 2.5). This means that the probability of a potential conflict would be higher particularly after showing the amber indication.

On the other hand, the probability of a potential conflict might be represented by higher TTC and lower headway between successive vehicles that decide to cross after the onset of amber. According to the Driving Standards Agency (1992), it is necessary for all drivers to keep a following distance of at least 2 seconds with the preceding vehicle to avoid collisions particularly in sudden braking conditions. Risky tailgating behaviour can be shown when the headways between successive vehicles are less than 2 seconds. By taking into consideration this concept, the model outputs showed an increase in risky driving behaviour in up to 10% of the overall simulated crossing vehicles (i.e. 61 out of 611 ALR and RLR drivers may be involved in a traffic conflict). This percentage is very close to that obtained from the reported accidents in Greater Manchester due to close following behaviour (i.e. 8.9% as discussed previously in Section 2.4).

Finally, the newly developed model shows capability to give a prediction of drivers' aggressiveness in terms of TTC and close following behaviour leading to involvement in potential conflicts (such as rear collisions and red light running after the onset of amber) near signalised junctions.

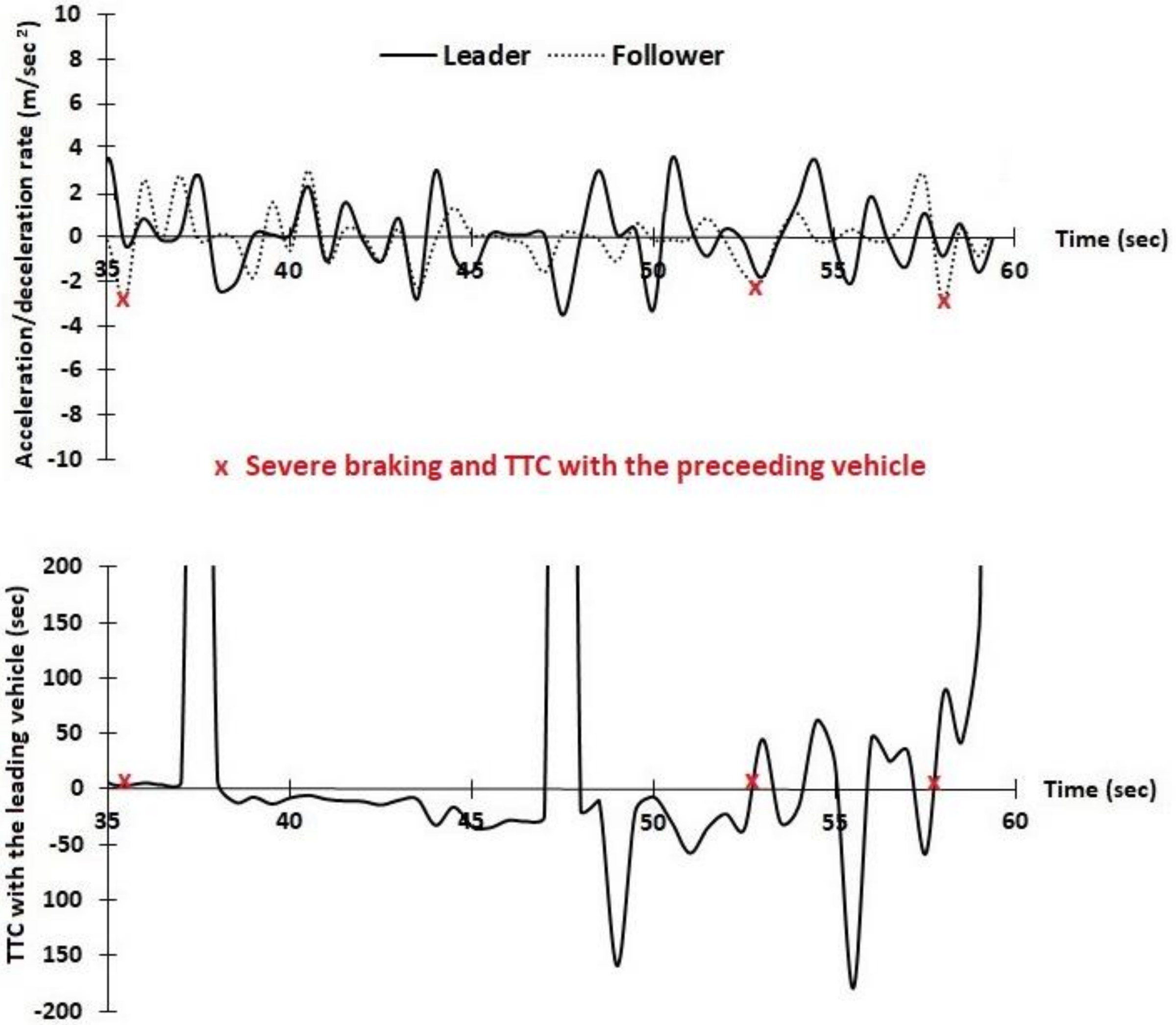


Figure 7.7: TTC values and braking action for a following vehicle approaching a traffic light signal junction

7.6 Reduction in drivers' non-compliant behaviour

As discussed previously in Section 2.5.3, different targeted enforcement techniques have been implemented to improve intersection safety and performance. For example, Red Light Cameras (RLC) are used and found to be an effective enforcement tools (see Section 2.5.3.2) causing a reduction in the number of signal violations by 40% in California, USA (Retting and Kyrychenko, 2002). This tool was modelled by introducing a parameter that given a value of (0) for junction without RLR and (1) for junction with RLC. The RLC was introduced in the simulation model in order to examine its efficiency in reducing RLR events at 40 mph signalised junctions operated by FT and VA modes. The input data are shown in Table 7.6. The number of RLR was tested in the developed model by assuming wide percentages of driver compliance to the RLC devices, between 50% and 90%, since lower percentages than 50% did not show any significant effect. These percentages were chosen in line with driver compliance with the amber indication.

Figure 7.8 illustrates that a reduction in RLR frequencies increases gradually with increasing driver compliance with the RLC. The reductions in RLR numbers at 50% compliance were 22% and 29% at FT and VA traffic signal junctions, respectively. At a junction controlled by VA signals, the reduction in RLR number increased gradually with driver compliance with the RLC until it reached 70% when driver compliance was at 90%. This is because of the area of detection that extends the green phase to reduce the effect of the dilemma zone. However, there was no remarkable decrease in the number of RLR at FT signal junctions after increasing the driver compliance factor up to 90%.

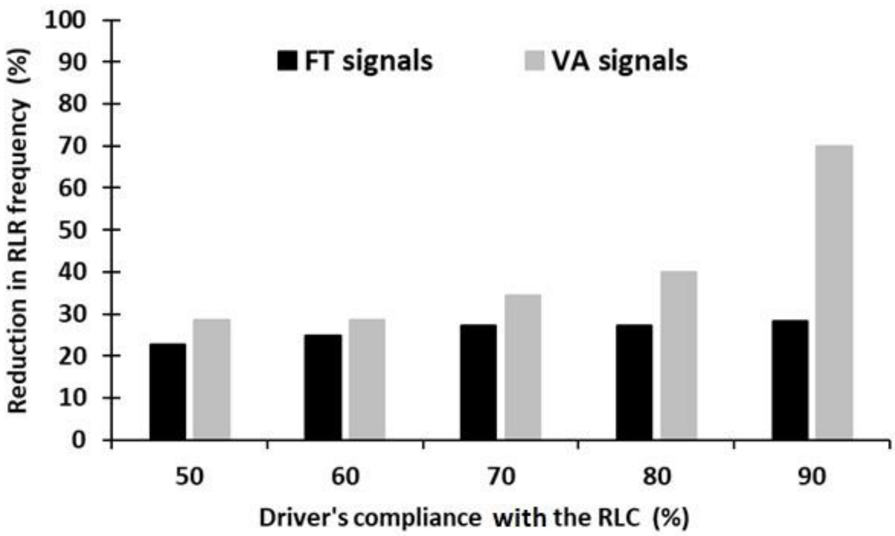


Figure 7.8: Simulation results for driver compliance with the RLC at 40 mph approaches to FT and VA traffic signal junctions

Overall, it can be indicated that the reduction in the percentages of RLR frequencies at VA signal junctions is higher than the reduction at FT signal junctions. This is because of the fact that VA signals extend the green phase if there are more vehicles in the detection area coming up to the junction, thereby reducing the number of late exits. In addition, some drivers might be surprised by the RLC at the end of amber. Finally, it can be stated that RLC does not show any significant reductions in the number of RLR at junctions controlled by FT mode.

7.7 Summary

This chapter presents several applications of the newly developed model. The main goal of developing this micro-simulation model was to represent driver behaviour approaching a traffic light junction controlled by FT and VA modes and his/her response following the onset of amber particularly in the dilemma zone. The following points can be summarised:

1. The lengths of amber and all-red periods were tested. The results show that increasing the amber length by an extra 1 second may lead to an increase in red light signal violations at signal junctions controlled by FT and VA modes. Changing the all-red period to that recommended by the Institute of Transportation Engineering (2015) caused a reduction in ALR and RLR events at all sites controlled by VA signal settings, while, there was an increase in RLR on 30 mph approaches controlled by FT signals.
2. The effect of FT and VA signals on vehicle delays was tested. The results show that there was a 20% reduction in the overall vehicle delay at junctions controlled by a VA mode in comparison with delays at FT signalised junctions.
3. The effect of increasing HGVs% on the number of RLR was tested within the newly developed model. The simulation results showed that in cases of HGVs% up to 50% in the traffic flow, there was a reduction in RLR events by 40% and 45% for VA and FT traffic signal junctions, respectively.
4. An increase in HGVs% up to 50% causes a reduction in junction capacity by approximately 42% at VA traffic signal junctions and 51% at junctions controlled by FT traffic signals.
5. Vehicle delays under various HGVs% and traffic flow rates were estimated. As discussed in Section 7.4.3, vehicle delays at VA signalised intersections are lower than those at junctions controlled by FT signals. In addition, as HGV% increases in the traffic composition, vehicle delays due to traffic signals increase as well.
6. Intersection safety was investigated in the current study by using TTC as a safety measure. The simulation results indicated that 10% of drivers who decided to proceed through the junction after seeing the amber signal are more likely to start tailgating with

the leading vehicles and 8% of those drivers have TTC values within the critical thresholds (i.e. between 1.5 and 6 secs as shown previously in Table 2.5).

7. To reduce the effect of the dilemma zone, red light cameras (RLC) were introduced in the simulation model to investigate their effect on the number of RLR at FT and VA signalised junctions. It was found that the reduction in RLR events was 26% on average at junctions controlled by FT signals, whereas, the reduction in the number of RLR at junctions controlled by VA signals increased from approximately 30% to 70% when the driver compliance with the RLC was at 50% and 90%, respectively.

CHAPTER EIGHT: CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

As explained in Section 1.2, the main purpose of this research is to investigate and evaluate possible reduction of traffic conflicts at signalised junctions. A micro-simulation model was developed in order to predict drivers' STOP/GO decisions when the signal changes from green to amber. The new micro-simulation model was developed based on the CARSIM model (established by Benekohal (1986)), which represents drivers' behaviour at the green phase, and the GHM model (developed by Gazis et al. (1960)), that represents drivers' behaviour and compliance following the onset of amber including the effect of the critical area on the approach, namely the 'Dilemma Zone'. The newly developed model was used to investigate various factors such as the effect of the length of the intergreen period on RLR frequencies in addition to the effect of various HGVs% on the number of RLR, junction capacity and vehicle delays. Model applications also included an investigation of junction safety measures (such as TTC and tailgating headway) and performance of enforcement techniques (such as RLC) on the number of RLR.

The most important findings arising from this study are listed as follows:

- **Objective 1:** The literature shows that traffic signals can help to control conflicts but there are still a significant number of conflicts at such locations. In addition, research has focused on a critical area called the "Dilemma Zone". In this area the risk of rear-end collisions and red signal offences might be increased because drivers neither have sufficient time to cross and clear the junction area, nor stop safely particularly after the onset of amber. Various simulation models (such as CARSIM) have been established to mimic the interaction between road users and the road environment under different conditions based on car-following rules. However, these models have some limitations, for example they do not consider the effect of the dilemma zone, in terms of replicating the correct behaviour as in the real world such as driver compliance following the onset of amber at signal-controlled junctions. Therefore, there is a need to understand the characteristics of car-following and dilemma zone algorithms in order to develop a tool for testing different traffic performance and design interventions.

- **Objectives 2 and 3:** Real site observations were collected from five junctions controlled by FT and VA signal settings. The data included investigations of traffic flow profile, vehicle characteristics (i.e. types, lengths and HGVs%), driver compliance, junction geometry and traffic signal timings as described in Chapter 3. The data were analysed from a total of 18 hrs of video recordings including three groups of drivers (i.e. ALR, RLR and ARLS drivers) in terms of several factors such as speeds, distances from the stopline at the onset of amber, headways, junction width and intergreen period (see Chapter Four). The analysed data were then used in the development, calibration and validation of the new micro-simulation model.
- **Objective 4:** The new micro-simulation model was developed based on the CARSIM model and included some modifications regarding the dilemma zone problem. The prediction STOPGO sub-model was introduced and developed based on the GHM model in order to replicate driver behaviour in the dilemma zone following the onset of amber (see Section 5.7) at junctions operated by FT or VA traffic light settings. Several factors were added to the developed model such as driver response to amber and the percentage of driver alertness at VA signal controlled junctions in order to predict the numbers of RLR and ALR drivers as closely as possible in line with the site observations, as explained in Section 6.4.2.
- **Objectives 5 and 6:** The developed model was calibrated and validated using different real datasets. The model outputs showed that the new model could represent vehicular flow in stop and go conditions at traffic signal junctions controlled by FT and VA modes. In addition, the simulation results, in terms of the numbers of RLR and ALR as well as drivers' MUT, were very close to those observed at survey sites. Comparisons between field and simulated data were carried out using different statistical tests as discussed and presented in Sections 6.4 and 6.5.
- **Objective 7:** The new micro-simulation model was applied to test different traffic scenarios, in order to alleviate the problem of the dilemma zone, such as the effect of intergreen length and introduction of enforcement tools, for example red light cameras (RLC). In addition, junction performance in terms of delay, capacity and safety issues were investigated within the new micro-simulation model. The key findings from this study objective are listed as follows:

1. It was concluded that increasing the amber length by an extra 1 second may lead to an increase in the number of RLR at FT and VA signal junctions. Similarly, an increase in the all red-period may cause an increase in RLR events and vice versa. Generally, increasing the length of intergreen results in an increase in RLR events (see Section 7.2).
2. The simulation results revealed that average vehicle delays at VA traffic signal junctions are lower by 20% than those at signalised junction operated by FT signals (as discussed in Section 7.3).
3. The number of RLR and junction capacity were investigated throughout the newly developed model in terms of HGVs% varying from 0% to 50%. It can be indicated that there is an inverse relationship between RLR frequencies and HGVs%. In addition, the results showed that junction capacity deceases as HGVs% increases. This can be explained by the fact that HGVs generally have lower speeds and less acceleration which might affect the movement of other vehicles. Consequently, this leads to an increase in time headways resulting in a decrease in junction throughput and capacity, as discussed in Sections 7.4.1 and 7.4.2.
4. Testing vehicle delays was carried out for different HGVs%. The simulation outputs revealed that vehicle delays increase as HGVs% increases. This may be due to the fact that HGVs have longer lengths in addition to lower speeds and less acceleration than PCs. Hence, more space on the road is occupied and additionally, drivers tend to maintain longer headways with HGVs (see Section 7.4.3).
5. The newly developed model was capable of giving an indication of junction safety by calculating safety measures such as TTC and headways between successive vehicles, particularly after the onset of amber. In addition, the model can show severe braking conditions when the TTC and following headway are lower than the critical limits. This can indicate the number of drivers who are tailgating with the leading vehicles following the onset of amber (as discussed in Section 7.5).
6. Finally, the effect of using enforcement red light cameras (RLC) on the number of signal violations was tested in the newly developed model. The simulation results showed a higher reduction in the number of RLR (about 70%) at VA signalised junctions compared with those junctions operated under FT signals (about 30%), particularly when the driver compliance to RLCs was at high level (i.e. 90%), as explained in Section 7.6.

8.2 Recommendations and future research

Several recommendations can be made based on this work as follows:

1. Some difficulties were encountered in the methodology of data collection particularly in finding a good vantage point for the filming and recording process. The length of the clearly covered road section was not more than 80 m. This was due to the limitations in the angle of view for the camera used in this study (i.e. Sony HDD DCR-SR57). Therefore, it is necessary to recommend using a more sophisticated digital camera that covers a longer section of the road such as between 100 and 200 m.
   For future research, the use of a particular traffic drone with high specifications of long life battery (up to 4 hours), with good image quality and stability covering an intersection area from a sufficient height in a good resolution is recommended. This advanced tool is provided with analysis software that gives a tracking number for each component (i.e. HGV, car, pedestrian, motorcycle … etc.) on the approach to the junction area including details of vehicle types and speeds. This high specification equipment could be helpful in improving data collection and analysis methods as well as minimising the time needed for data processing with enhanced accuracy. Such advanced technology is provided by many commercial companies interested in traffic data collection and analysis. The website of 'DataFromSky' (which is developed by robotics, camera vision and embedded systems team) provides advanced traffic analysis of aerial video data as an example of such commercial softwares. As mentioned previously in Section 3.3, this method was not considered in this work because of the cost and safety issues as well as it needs to training and permission to be used.
2. It is recommended that the new model should be modified to include the effects of lane changing and gap acceptance rules in order to investigate RLR due to overtaking behaviour, RLR in the left/right turning flows and RLR before the onset of green (i.e. during the red/amber period which is usually 2 seconds after the red phase). The current work has not considered the aforementioned effects because of rare observations of such situations after the onset of amber. This may be the case in future research. So, selection of site is importance to cover lane changing.
3. Other types of signalised junctions operated by either FT or VA signals such as junctions with 50 mph speed limits and more isolated intersections and roundabouts need to be investigated. This is reasonable in order to modify the car-following rules

in the developed model to include additional factors such as road gradient, curvatures, effect of skid resistance, driver gender and age.

4. Use of speed humps, flashing green signals and GSCD techniques with/without warning signs (to reduce the effect of the dilemma zone) could be tested by modifying the newly developed model and comparing the results with real data for predicting drivers' STOP/GO decisions after the onset of amber, in addition to studying their impacts on delays and capacity.
5. Safety issue at traffic signalised junctions could be tested by modifying the new micro-simulation model in order to estimate safety measures such as TTC and headways in terms of visual angle and changes in the spacing between successive vehicles after the onset of amber.
6. Because of the rapid development in the intelligent transport field, particularly autonomous cars, it is worth mentioning here that such factors should be taken into consideration as a proportion in the traffic composition for future research. Once there is approval for the use of autonomous cars by governments, these intelligent vehicles will influence the behaviour of other drivers to comply with the road and traffic regulations in terms of legal speed limit, safe headways with other vehicles, enforcement technology, and a sufficient stopping distance from the stopline at traffic signal-controlled junctions. Hence, a reduction in the number of red light runnings and tailgating behaviour will be achieved.

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Appendix A: STATS19 forms of accident details reported by police

A.1 Form of accident record

STATS19 (2005) — **Accident Record Attendant Circumstances** — **(For completion by Police)**

1.1 Record Type [1]
- 11 New accident record
- 15 Amended accident record

1.2 Police Force

1.3 Accident Reference

1.5 Number of Vehicle Records

1.6 Number of Casualty Records

1.7 Date of Accident (Day Month Year)

1.9 Time of Day (Hours Mins, 24 hour)

1.10 Local Authority

1.11 Location
10 digit OS Grid Reference number
Easting Northing

1.12 1st Road Class
- 1 Motorway
- 2 A(M)
- 3 A
- 4 B
- 5 C
- 6 Unclassified

1.13 1st Road Number

1.14 Road Type
- 1 Roundabout
- 2 One way street
- 3 Dual carriageway
- 6 Single carriageway
- 7 Slip road
- 9 Unknown

1.15 Speed Limit (mph) [0]

1.16 Junction Detail [0]
- 00 Not at or within 20 metres of junction
- 01 Roundabout
- 02 Mini roundabout
- 03 T or staggered junction
- 05 Slip road
- 06 Crossroads
- 07 Multiple junction
- 08 Using private drive or entrance
- 09 Other junction

Junction Accidents Only

1.17 Junction Control
- 1 Authorised person
- 2 Automatic traffic signal
- 3 Stop sign
- 4 Give way or uncontrolled

1.18 2nd Road Class
- 1 Motorway
- 2 A(M)
- 3 A
- 4 B
- 5 C
- 6 Unclassified

1.19 2nd Road Number

1.20a Pedestrian Crossing – Human Control
- 0 None within 50 metres
- 1 Control by school crossing patrol
- 2 Control by other authorised person

1.20b Pedestrian Crossing - Physical Facilities
- 0 No physical crossing facility within 50 metres
- 1 Zebra crossing
- 4 Pelican, puffin, toucan or similar non-junction pedestrian light crossing
- 5 Pedestrian phase at traffic signal junction
- 7 Footbridge or subway
- 8 Central refuge – no other controls

1.21 Light Conditions
- 1 Daylight: street lights present
- 2 Daylight: no street lighting
- 3 Daylight: street lighting unknown
- 4 Darkness: street lights present and lit
- 5 Darkness: street lights present but unlit
- 6 Darkness: no street lighting
- 7 Darkness: street lighting unknown

1.22 Weather
- 1 Fine without high winds
- 2 Raining without high winds
- 3 Snowing without high winds
- 4 Fine with high winds
- 5 Raining with high winds
- 6 Snowing with high winds
- 7 Fog or mist – if hazard
- 8 Other
- 9 Unknown

1.23 Road Surface Condition
- 1 Dry
- 2 Wet / Damp
- 3 Snow
- 4 Frost / Ice
- 5 Flood (surface water over 3cm deep)

1.24 Special Conditions at Site
- 0 None
- 1 Automatic traffic signal out
- 2 Automatic traffic signal partially defective
- 3 Permanent road signing or marking defective or obscured
- 4 Roadworks
- 5 Road surface defective
- 6 Oil or diesel
- 7 Mud

1.25 Carriageway Hazards
- 0 None
- 1 Dislodged vehicle load in carriageway
- 2 Other object in carriageway
- 3 Involvement with previous accident
- 6 Pedestrian in carriageway – not injured
- 7 Any animal in carriageway (except ridden horse)

1.26 Did A Police Officer Attend Accident and Complete Record?
- 1 Yes
- 2 No – accident was reported 'over the counter'

1.27 DfT Special Projects

Appendix A: STATS19 forms of accident details reported by police

A.2 Form of vehicle record

STATS19 (2005)

Vehicle Record

(For completion by Police)

2.1 Record Type [2]
- 21 New vehicle record
- 25 Amended vehicle record

2.2 Police Force

2.3 Accident Reference

2.4 Vehicle Reference Number

2.5 Type of Vehicle
- 01 Pedal cycle
- 02 M/cycle 50cc and under
- 03 Motorcycle over 50cc and up to 125cc
- 04 Motorcycle over 125cc and up to 500cc
- 05 Motorcycle over 500cc
- 08 Taxi/Private hire car
- 09 Car
- 10 Minibus (8 – 16 passenger seats)
- 11 Bus or coach (17 or more passenger seats)
- 14 Other motor vehicle
- 15 Other non-motor vehicle
- 16 Ridden horse
- 17 Agricultural vehicle (includes diggers etc.)
- 18 Tram / Light rail
- 19 Goods vehicle 3.5 tonnes mgw and under
- 20 Goods vehicle over 3.5 tonnes and under 7.5 tonnes mgw
- 21 Goods vehicle 7.5 tonnes mgw and over

2.6 Towing and Articulation
- 0 No tow or articulation
- 1 Articulated vehicle
- 2 Double or multiple trailer
- 3 Caravan
- 4 Single trailer
- 5 Other tow

2.7 Manoeuvres
- 01 Reversing
- 02 Parked
- 03 Waiting to go ahead but held up
- 04 Slowing or stopping
- 05 Moving off
- 06 U turn
- 07 Turning left
- 08 Waiting to turn left
- 09 Turning right
- 10 Waiting to turn right
- 11 Changing lane to left
- 12 Changing lane to right
- 13 Overtaking moving vehicle on its offside
- 14 Overtaking stationary vehicle on its offside
- 15 Overtaking on nearside
- 16 Going ahead left hand bend
- 17 Going ahead right hand bend
- 18 Going ahead other

2.8 Vehicle Movement Compass Point (From / To)
- 1 N
- 2 NE
- 3 E
- 4 SE
- 5 S
- 6 SW
- 7 W
- 8 NW
- Parked [0][0]

2.9 Vehicle Location at Time of Accident - Restricted Lane/ Away from Main Carriageway
- 00 On main c'way – not in restricted lane
- 01 Tram / Light rail track
- 02 Bus lane
- 03 Busway (including guided busway)
- 04 Cycle lane (on main carriageway)
- 05 Cycleway or shared use footway (not part of main carriageway)
- 06 On lay-by or hard shoulder
- 07 Entering lay-by or hard shoulder
- 08 Leaving lay-by or hard shoulder
- 09 Footway (pavement)

2.10 Junction Location of Vehicle
- 0 Not at, or within 20 metres of, junction
- 1 Approaching junction or waiting/parked at junction approach
- 2 Cleared junction or waiting/parked at junction exit
- 3 Leaving roundabout
- 4 Entering roundabout
- 5 Leaving main road
- 6 Entering main road
- 7 Entering from slip road
- 8 Mid junction – on roundabout or on main road

2.11 Skidding and Overturning
- 0 No skidding, jack-knifing or overturning
- 1 Skidded
- 2 Skidded and overturned
- 3 Jack-knifed
- 4 Jack-knifed and overturned
- 5 Overturned

2.12 Hit Object in Carriageway
- 00 None
- 01 Previous accident
- 02 Roadworks
- 04 Parked vehicle
- 05 Bridge – roof
- 06 Bridge – side
- 07 Bollard / Refuge
- 08 Open door of vehicle
- 09 Central island of roundabout
- 10 Kerb
- 11 Other object
- 12 Any animal (except ridden horse)

2.13 Vehicle Leaving Carriageway
- 0 Did not leave carriageway
- 1 Left carriageway nearside
- 2 Left carriageway nearside and rebounded
- 3 Left carriageway straight ahead at junction
- 4 Left carriageway offside onto central reservation
- 5 Left carriageway offside onto central reservation and rebounded
- 6 Left carriageway offside and crossed central reservation
- 7 Left carriageway offside
- 8 Left carriageway offside and rebounded

2.14 Hit Object Off Carriageway
- 00 None
- 01 Road sign / Traffic signal
- 02 Lamp post
- 03 Telegraph pole / Electricity pole
- 04 Tree
- 05 Bus stop / Bus shelter
- 06 Central crash barrier
- 07 Nearside or offside crash barrier
- 08 Submerged in water (completely)
- 09 Entered ditch
- 10 Other permanent object

2.16 First Point of Impact
- 0 Did not impact
- 1 Front
- 2 Back
- 3 Offside
- 4 Nearside

2.17 Other Vehicle Hit
Ref no. of other vehicle hit (or hit by)
Special code: 000 No other vehicle hit

2.21 Sex of Driver
- 1 Male
- 2 Female
- 3 Not traced

2.22 Age of Driver (Years)
Estimated if necessary

2.23 Breath Test
- 0 Not applicable
- 1 Positive
- 2 Negative
- 3 Not requested
- 4 Refused to provide
- 5 Driver not contacted at time of accident
- 6 Not provided (medical reasons)

2.24 Hit and Run
- 0 Other
- 1 Hit and Run
- 2 Non-stop vehicle, not hit

2.25 DfT Special Projects

2.26 Vehicle Registration Mark (VRM)

2.28 Foreign Registered Vehicle
- 0 Not foreign registered vehicle
- 1 Foreign registered vehicle – left hand drive
- 2 Foreign registered vehicle – right hand
- 3 Foreign registered vehicle – two wheeler

2.27 Driver Postcode
Special codes:
- 1 Unknown
- 2 Non-UK resident
- 3 Parked and unattended

2.29 Journey Purpose of Driver/Rider
- 1 Journey as part of work
- 2 Commuting to/from work
- 3 Taking pupil to/from school
- 4 Pupil riding to/from school
- 5 Other/Not known

Appendix A: STATS19 forms of accident details reported by police

A.3 Form of casualty record

STATS19 (2005)

Casualty Record

(For completion by Police)

3.1 Record Type 3

31 New casualty record
35 Amended casualty record

3.2 Police Force

3.3 Accident Reference

3.4 Vehicle Reference Number

3.5 Casualty Reference Number

3.6 Casualty Class

1 Driver or rider
2 Vehicle or pillion passenger
3 Pedestrian

3.7 Sex of Casualty

1 Male
2 Female

3.8 Age of Casualty
Estimated if necessary
Years

3.9 Severity of Casualty

1 Fatal
2 Serious
3 Slight

Pedestrian Casualties Only

3.10 Pedestrian Location

01 In carriageway, crossing on pedestrian crossing facility
02 In carriageway, crossing within zig-zag lines at crossing approach
03 In carriageway, crossing within zig-zag lines at crossing exit
04 In carriageway, crossing elsewhere within 50 metres of pedestrian crossing
05 In carriageway, crossing elsewhere
06 On footway or verge
07 On refuge, central island or central reservation
08 In centre of carriageway, not on refuge, central island or central reservation
09 In carriageway, not crossing
10 Unknown or other

3.11 Pedestrian Movement

1 Crossing from driver's nearside
2 Crossing from driver's nearside – masked by parked or stationary vehicle
3 Crossing from driver's offside
4 Crossing from driver's offside – masked by parked or stationary vehicle
5 In carriageway, stationary – not crossing (standing or playing)
6 In carriageway, stationary – not crossing (standing or playing), masked by parked or stationary vehicle
7 Walking along in carriageway – facing traffic
8 Walking along in carriageway – back to traffic
9 Unknown or other

Pedestrian Casualties Only

3.12 Pedestrian Direction

Compass point bound

1 N
2 NE
3 E
4 SE
5 S
6 SW
7 W
8 NW
9 Unknown
0 Standing still

3.19 Pedestrian Injured in the Course of 'On the Road' Work
Work activity carried out on public road (eg delivery services, road maintenance, traffic control etc.)

0 No
1 Yes
2 Not known

3.13 School Pupil Casualty

1 School pupil on journey to or from school
0 Other

3.15 Car Passenger

0 Not a car passenger
1 Front seat passenger
2 Rear seat passenger

3.16 Bus or Coach Passenger

0 Not a bus or coach passenger
1 Boarding
2 Alighting
3 Standing passenger
4 Seated passenger

3.17 DfT Special Projects

3.18 Casualty Postcode

Special codes:
1 Unknown
2 Non-UK resident

Appendix A: STATS19 forms of accident details reported by police

A.4 Form of factors contributed to the accident

STATS19 (2005)

What Factors Contributed To The Accident?

Select **up** to **six** Factors from the grid, relevant to the accident.

Factors may be shown in any order, but an indication must be given of whether each Factor is **very likely (A)** or **possible (B)**.

Only include factors which have contributed to the accident. (i.e. do NOT include "Poor road surface" unless it was relevant to the accident)

More than one factor may be related to the same road user

The same factor may be related to more than one road user, if appropriate

The participant should be identified by the STATS19 vehicle or casualty reference number, preceded by "V" if factor applies to a vehicle, driver/rider or the road environment (eg V002), or "C" for a pedestrian or passenger casualty (eg C001). Enter "U000" if an uninjured pedestrian contributed

	1st	2nd	3rd	4th	5th	6th
Factor in the accident						
Which participant? (eg V001, C001, U000)						
Very likely (A) or possible (B)						

Road Environment Contributed	**Vehicle Defects**	**Driver/Rider Only** (Includes Pedal Cyclists and Horse Riders)					**Pedestrian Only** (Casualty or Uninjured)	**Special Codes**
		Injudicious Action	**Driver/Rider Error or Reaction**	**Impairment or Distraction**	**Behaviour or Inexperience**	**Vision Affected by**		
Poor or defective road surface 101	Tyres illegal, defective or under inflated 201	Disobeyed automatic traffic signal 301	Junction overshoot 401	Impaired by alcohol 501	Aggressive driving 601	Stationary or parked vehicle(s) 701	Crossed road masked by stationary or parked vehicle 801	Stolen vehicle 901
Deposit on road (eg. oil, mud, chippings) 102	Defective lights or indicators 202	Disobeyed Give Way or Stop sign or markings 302	Junction restart 402	Impaired by drugs (illicit or medicinal) 502	Careless/Reckless/In a hurry 602	Vegetation 702	Failed to look properly 802	Vehicle in course of crime 902
Slippery road (due to weather) 103	Defective brakes 203	Disobeyed double white line 303	Poor turn or manoeuvre 403	Fatigue 503	Nervous/Uncertain/ Panic 603	Road layout (eg. bend, winding road, hill crest) 703	Failed to judge vehicle's path or speed 803	Emergency vehicle on call 903
Inadequate/Masked signs or road markings 104	Defective steering or suspension 204	Disobeyed pedestrian crossing facility 304	Failed to signal/ Misleading signal 404	Uncorrected, defective eyesight 504	Driving too slow for conditions or slow veh (eg tractor) 604	Buildings, road signs, street furniture 704	Wrong use of pedestrian crossing facility 804	Vehicle door opened or closed negligently 904
Defective traffic signals 105	Defective or missing mirrors 205	Illegal turn or direction of travel 305	Failed to look properly 405	Illness or disability, mental or physical 505	Inexperienced or learner driver/rider 605	Dazzling headlights 705	Dangerous action in carriageway (eg playing) 805	
Traffic calming (eg speed cushions, road humps, chicanes) 106	Overloaded or poorly loaded vehicle or trailer 206	Exceeding speed limit 306	Failed to judge other person's path or speed 406	Not displaying lights at night or in poor visibility 506	Inexperience of driving on the left 606	Dazzling sun 706	Impaired by alcohol 806	
Temporary road layout (eg contraflow) 107		Travelling too fast for conditions 307	Passing too close to cyclist, horse rider or pedestrian 407	Cyclist wearing dark clothing at night 507	Inexperience with type of vehicle 607	Rain, sleet, snow, or fog 707	Impaired by drugs (illicit or medicinal) 807	
Road layout (eg bend, hill, narrow carriageway) 108		Following too close 308	Sudden braking 408	Driver using mobile phone 508		Spray from other vehicles 708	Careless/Reckless/In a hurry 808	
Animal or object in carriageway 109		Vehicle travelling along pavement 309	Swerved 409	Distraction in vehicle 509		Visor or windscreen dirty or scratched 709	Pedestrian wearing dark clothing at night 809	
		Cyclist entering road from pavement 310	Loss of control 410	Distraction outside vehicle 510		Vehicle blind spot 710	Disability or illness, mental or physical 810	Other – Please specify below 999

If 999 Other: give brief details ....

Note: Only use if "Other" Factor contributed to the accident. Also include in text description of how accident happened

Note: These factors reflect the Reporting Officer's opinion at the time of the accident and are not necessarily the result of extensive investigation

Appendix-B: Survey sites

B.1. Junction of A34 Kingsway Road with B5095 Wilmslow Road in Manchester

Figure B.1: Site plan of Site #1

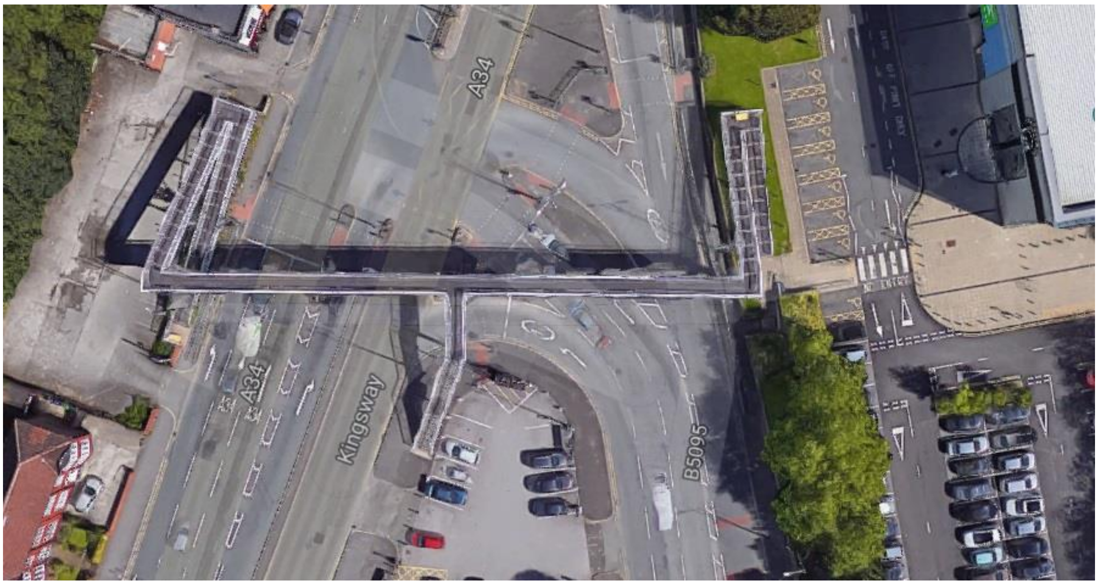

Figure B.2: Screenshot of Site #1 (Google Map)

Appendix B: Survey sites

B.2. Junction of A57 Sankey Way Road with Cromwell Ave Road in Warrington

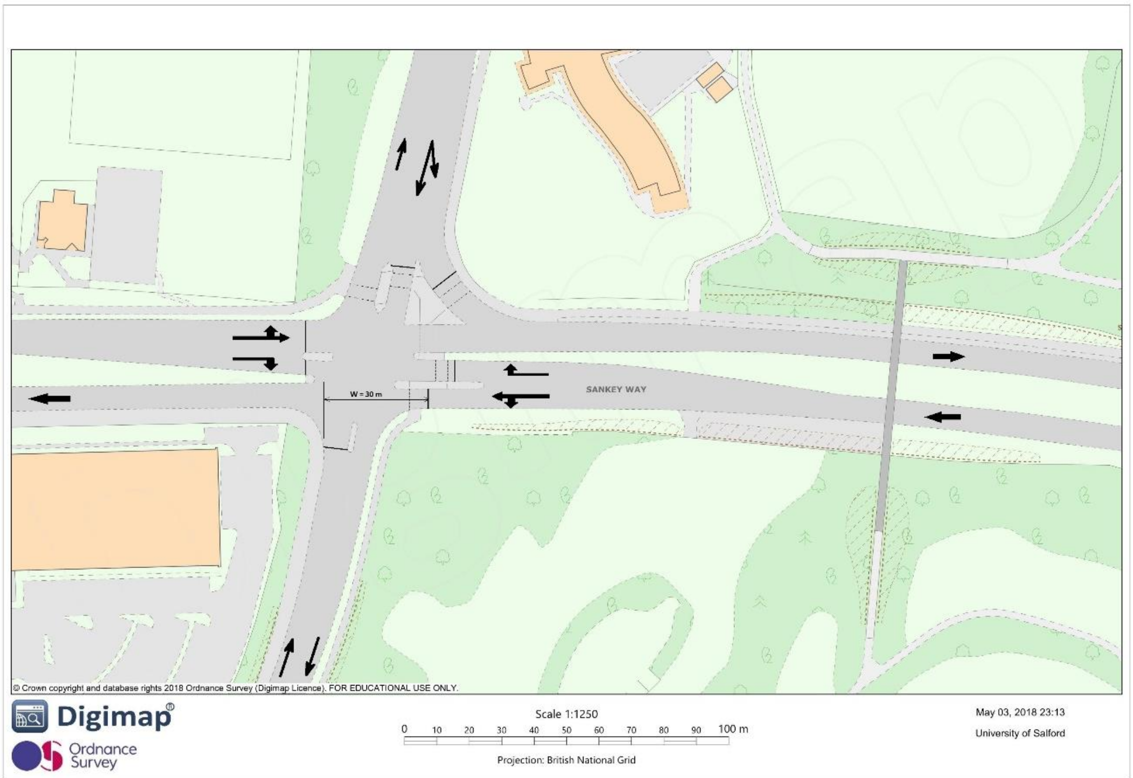

Figure B.3: Site plan of Site #2

Figure B.4: Screenshot of Site #2 (Google Map)

Appendix B: Survey sites

B.3. Junction of A580 East Lancashire Road with Eccles Road in Salford

Figure B.5: Site plan of Site #3

Figure B.6: Screenshot of Site #3 (Google Map)

Appendix B: Survey sites

B.4. Junction of A6 Broad Street with B6186 Frederick Road in Salford

Figure B.7: Site plan of Site #4

Figure B.8: Screenshot of Site #4 (Google Map)

Appendix B: Survey sites

B.5. Junction of A5186 Langworthy Road with Liverpool Street in Salford

Figure B.9: Site plan of Site #5

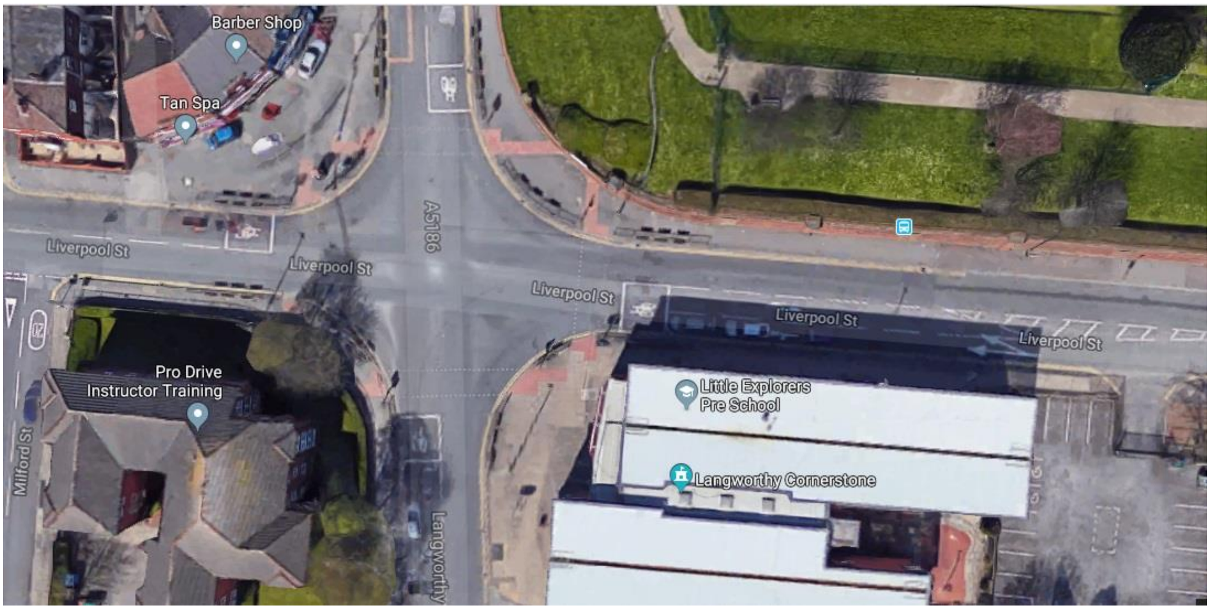

Figure B.10: Screenshot of Site #5 (Google Map)