Design strategies for priority infrastructure for minibus-taxis at signalised intersections

Mwenda, John Paul

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DISSERTATION SUMMARY

DESIGN STRATEGIES FOR PRIORITY INFRASTRUCTURE FOR MINIBUS-TAXIS AT SIGNALISED INTERSECTIONS

JOHN-PAUL MWENDA

Supervisor: Venter, Christoffel Jacobus

Department: Civil Engineering

University: University of Pretoria

Degree: Master of Engineering (Transportation engineering)

The paratransit industry in South Africa which mainly includes the minibus-taxis is growing at a fast pace. Thus, it has become the largest mobility supplier to the urban public. In Gauteng province, the economic hub of South Africa that includes Johannesburg, Tshwane and Ekurhuleni, minibus-taxis account for 46% of all peak-period passenger trips followed by private cars accounting for 44%, while buses and trains account for a combined total of 10% of peak-period. Unlike buses which have seen the provision of priority infrastructure at intersections in the form of bus rapid transit (BRT) with priority transit signals (PTS) to improve their efficiency, minibus-taxis currently do not enjoy the same benefits. However, any efforts of road authorities in South Africa to consider incorporating priority infrastructure for minibus-taxis would be constrained by the absence of literature suggesting the ideal choices and the design analytical procedures.

This research study aims to develop and evaluate design strategies for priority infrastructure for minibus-taxis at signalised intersections. Priority infrastructure at intersections can be in form of roadway facility infrastructure such as queue-jumping lanes, shared traffic lane, exclusive lanes or can be implemented via signal control. These infrastructure types are designed to provide efficiency benefits to road users mainly public transport such as buses. The first objective of this study is to develop an approach for identifying the design strategies for priority infrastructure for minibus-taxis at signalised intersections. A qualitative data method utilising document analysis technique is used to develop a framework matrix table to show the relationship between the geometric elements and the design treatments of priority infrastructure. Two categories of minibus-taxis (MBT) design strategies are then formed: 1) design strategies that only require repurposing of the existing intersection, 2) the design strategies that require major geometric improvements.

Secondly, an analytical approach is developed to evaluate the performance of two proposed design strategies using real world traffic data. To begin with, four isolated intersections in the city of Tshwane are evaluated for feasibility of the MBT design strategies. The framework matrix analysis developed earlier is utilised to select and evaluate the design strategies associated with the four intersections. In addition, the intersections are further assessed for safety, traffic operations and cost effectiveness. Eventually, the two most effective design strategies are selected for a detailed performance evaluation: 1) a shared MBT lane to be used by through movement minibus-taxis and left-turning vehicles (MBT+LT) and 2) a dedicated MBT lane for through minibus-taxis only. The approach uses modified analytical principles from the Highway Capacity Manual (HCM) to measure the performance of the selected design strategies using peak hour traffic data. The performance measures include volume to capacity ratio (v/c ratio), average vehicle delay, and adequacy of storage length of MBT priority lanes. The performances of existing intersections are compared with the performances of intersections after implementing the MBT design strategies. In general, the results show that the two proposed MBT design strategies significantly improved the performance of minibus-taxis at intersections while slightly reducing the performance of traffic in non-priority lanes.

Lastly, using the results from the two evaluated design strategies, a sensitivity analysis is performed on the modified HCM method to determine a range of traffic volumes for which the selected design strategies are feasible. Consequently, two models are set using a modified HCM method to evaluate two typical MBT design strategies involving a shared MBT lane and a dedicated MBT lane. The models are set to measure the v/c ratios of individual lanes on the approach as a measure of performances. The models are set to measure the highest v/c ratios while varying the traffic volumes at constant values of g/C ratios. The model outputs are in the form of graphs showing the relationship between left turning (LT) traffic, straight (MBT+T) traffic and v/c ratios at constant values of g/C ratios. These charts are developed as a planning and design guide when evaluating the feasibility of signalised intersections for the two evaluated MBT priority infrastructure types.

Overall, the study provides the first detailed results supporting the viability of priority infrastructure for minibus-taxis at signalised intersections. It also gives a detailed methodology and steps that could be used by traffic engineers and planners to design and evaluate the performance of priority infrastructure for minibus-taxis at signalised intersections. The matrix framework method and graphs for traffic volumes could provide planners with a structured way to identify feasible designs for the priority infrastructure for minibus-taxis at signalised intersections. The methodology used in this study can be adopted to evaluate other types of design strategies not evaluated in this study.

The study concludes that with well optimised design solutions, it is possible to use priority infrastructure to improve the performance of minibus-taxis at signalised intersections without adversely affecting the performance of traffic in the non-priority lanes.

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1 INTRODUCTION

1.1 BACKGROUND

Paratransit is defined as a flexible mode of public passenger transportation that does not follow fixed schedules and is typically available in the form of small to medium-sized buses (Behrens, McCormick and Mfinanga, 2016). Some of the commonly used names for paratransit vehicles in Africa include *Matatu* in Kenya, *Combi* in Botswana, *Taxis or Minibus-taxis* and A*maphela* in South Africa; *Daladala* in Tanzania, *Bush-Taxi* in West Africa and *Minibus* in Malawi. The paratransit industry in South Africa mainly includes minibus-taxis and is growing at a fast pace. Thus, it has become the largest mobility supplier to the urban public. In Gauteng province, the economic hub of South Africa which includes Johannesburg, Tshwane and Ekurhuleni, minibus-taxis are the ubiquitous mode of motorised transport. Minibus-taxis passenger trips account for 46% of all peak-period trips followed by private cars accounting for 44%, while buses and trains account for a combined total of 10% of peak-period trips (Gauteng Department of Roads and Transport (GDoRT), 2020). When compared to other modes of public transport, at least 66% of all peak-hour trips are made using minibus-taxis (van Ryneveld, 2018).

These high peak hour trips for minibus-taxis contribute to traffic congestion. Traffic congestion remains a major mobility problem in South African cities. Increased traffic congestion leads to a reduction in traffic speeds which consequently increases journey times, fuel consumption, operating cost and environmental pollution (Bull, 2003). According to the 2022 INRIX Global Traffic Scorecard report by Pishue (2023), the average annual delays for drivers commuting in Cape Town, Johannesburg and Pretoria are 80 hours, 61 hours, and 42 hours respectively. To address the congestion and equity issues, the Department of Transport (DoT) adopted the public transport strategy (DoT, 2007) to guide investment and upgrading of public transport. Integrated Rapid Public Transport Networks (IRPTN) was one of these strategies introduced which includes provision of the bus rapid transit (BRT) system. This system currently renders services to buses only by providing them with dedicated bus lanes and transit signal priority (TSP) at intersections within major cities of South Africa. These BRT systems include *A Re Yeng* in the city of Tshwane, *Rea Vaya* in the city of Johannesburg, *Harambee* in the city of Ekurhuleni, *MyCiti* in the city of Cape Town, *Go!Durban* in eThekweni and *Yarona* in Rustenburg.

In addition to BRT systems, McLachlan (2021) observes that over the past decade, the South African government has seen the construction of world-class depots and staging facilities for BRT buses that today serve a fraction of the demand that the minibus-taxis industry serves. Moreover, municipalities have invested in the construction of lay-bys and ranks (terminals) for minibus-taxis. However, other infrastructure-related strategies have not been explored to improve the operations and travel times for the mode that carries the bulk of public transport users. Given the strategic importance of minibus-taxis as a sector to the government of South Africa (GDoRT, 2020 & Jennings and Behrens, 2017), it becomes valuable to add to the existing literature pertaining to the design of priority infrastructure at intersections. Many studies around the world and some in South Africa (Chitauka & Vanderschuren,2014; Bulman & Van Ryneveld, 2015; Adewumi & Allopi, 2013) have examined the performance of priority infrastructure at intersections. However, the majority of them have focused on buses which account for very small proportions of peak-hour trips.

Intersections are a critical aspect of road design as a point where most traffic delays occur (Sampson, 2019). In most intersections with high volumes of traffic, traffic flow is ineffective during peak hours (Das and Keetse, 2015). This causes many intersections to have long queues. As a result, many minibus-taxi drivers tend to use illegal driving behaviour to bypass the long queues. A recent empirical study by De Beer and Venter (2021) on the potential benefits and impacts of priority infrastructure for minibus-taxis observed this phenomenon. Their study observed that minibus-taxi drivers in South Africa display a driving behaviour that simulates priority access. This behaviour includes queue-skipping and opposite-lane driving which are problems of safety. So, the question is, can we formalise this behaviour and provide benefits to minibus-taxi passengers and operators while reducing the safety and capacity impacts on other traffic?

Global evidence on priority infrastructure shows that this behaviour can however be formalised to ensure driving safety and effective traffic flows at intersections. De Beer and Venter (2021) proposed three interventions that would formalise this behaviour. These interventions include the provision of: (a) queue-jumping lanes, (b) single-lane pre-signal strategies, and (c) dedicated public transport lanes. Focussing on these three interventions, De Beer and Venter (2021) developed a simple analytical model to estimate the net economic impacts on taxi operators, passengers and private car users. The results showed a wide range of benefits such as reduced travel time, user cost, and operating cost. However, in as much as the model was a success in demonstrating the preliminary benefits of priority infrastructure for minibus-taxis, the study was exploratory. The study did not consider aspects of design, capacity or signalisation of entire intersections to ascertain the benefits using real traffic count data. In addition, the study was limited to undersaturated corridors with medium traffic volumes for two-directional traffic movements only.

1.2 SIGINIFICANCE OF THE STUDY

Current evidence in South Africa suggests that priority facilities are beneficial (De Beer and Venter, 2021; Oni 2018; Chitauka and Vanderschuren,2014). However, these studies did not evaluate other significant design questions which this study intends to address. Similarly, a study by Chitauka and Vanderschuren (2014) focussed on priority strategies for buses which does not have the same capacity demands as that of minibus-taxis. This study answers significant design question for minibus-taxi (MBT) priority infrastructure.

At a policy level, there is a general willingness from the South African government to consider incorporating transit priority facilities in the design of roads in South Africa. For instance, the Gauteng Department of Roads and Transport has recently updated the design guidelines for buses and minibus-taxis facilities on major provincial roads in Gauteng. In these guidelines, priority infrastructure has been added to improve the operations of public transport (GDoRT, 2021). Among the design considerations, designers are expected to provide supporting intersection analysis modelling "with" and "without" transit priority facilities with results showing vehicle movement and general intersection operational impacts. However, there are currently no design guidelines or analytical procedures for these priority treatments at an intersection to assist these transport system designers.

Similarly, the City of Johannesburg (CoJ) (2013) conducted a sustainability study where several new or improved infrastructure elements were identified for infrastructure implementation in the city. Some of the recommendations from the study included provision of public transport priority measures on the mixed traffic sections of the complementary routes at intersections, such as queue-jumping lanes and signal priority. In the City of Tshwane, Du Preez and Venter (2022) reported that according to A Re Yeng's Integrated Rapid Public Transport Network (IRPTN) Specialized Unit, the municipality has started investigating the use of queue-jumping infrastructure for the next phases of the BRT.

Even though there are these signs and efforts to accommodate transit priority infrastructure by the planning authorities and policy makers, there is still limited information on the methodology and guidelines when deciding on the choices of priority facilities and the procedures for evaluating intersections for successful results. In the same way, much focus of available literature has been on modelling the benefits and impacts of priority infrastructure, without providing a detailed design procedure or methodology particularly for minibus-taxis.

In addition, where information for priority facilities is available, most pertains to the design of other types of priority infrastructure for public transport such as exclusive bus lanes also known as bus rapid transit (BRT) (Lowe & Friesslaar, 2019; Bulman & Van Ryneveld, 2015). In support of this statement, most major metro cities in South Africa including the City of Tshwane, the City of Johannesburg and the City of Cape Town have started to implement BRTs in recent years. However, the realities of the slow and expensive roll-out of BRTs, coupled with the realisation that the minibus-taxi has a continuing role to play in a hybrid public transport system, has turned the attention of some authorities back towards priority infrastructure for informal transit (De Beer & Venter, 2021).

1.3 OBJECTIVES OF THE STUDY

The purpose of this study is to develop and evaluate selected design strategies for priority infrastructure for minibus- taxis at signalised intersections. The study output could potentially provide planners with a structured way to design priority infrastructure for minibus-taxis at signalised intersections.

The following key objectives formed part of the study:

- Examine a range of MBT priority infrastructure interventions at signalized intersections.
- Examine the impacts of MBT priority interventions on the performance of signalized intersections.
- Provide design guidance on feasibility of MBT priority interventions.

1.4 SCOPE OF THE STUDY

This study focuses on the evaluation of isolated signalised intersections in the City of Tshwane, South Africa. In the context of this study, we define design strategies as modifications of either the operations or the environment in which minibus-taxis operate that improve speeds, reduce delays or otherwise benefit minibus-taxis operations by improving reliability or attractiveness to patrons (National Capital Region Transportation Planning Board, 2011). The study focusses on evaluating two design strategies by first, providing a holistic approach on development of the strategies using existing geometric features. It is not a requirement in descriptive research to have a representative sample, therefore, two intersections are selected within the City of Tshwane to be representative of typical urban intersections for the purpose of performance evaluation of the selected design strategies. Collection and analysis of traffic data were based on video recording traffic counts conducted on all approaches of the two signalised intersections. The purpose of this analysis was to estimate and compute the traffic volume and evaluate the required performance of the intersections. Performance evaluation was carried out using the analytical Highway Capacity Manual (HCM) method. The findings from the performance evaluation formed the basis for the development of graphical design models for predicting feasibility of the two selected design strategies under a range of volume conditions. All analysis assumed minibus-taxis will not be stopping inside the intersection as this behaviour is considered illegal and unsafe. Due to low volumes of pedestrian traffic in the selected intersections, the analysis did not consider the impact of pedestrian on the performance of the intersections. This study was limited to signalised intersections on class 2 to 3 urban roads with speed limits of between 60km/hr and 80km/hr. The graphical design models developed in this study assumed the targeted approaching lanes to have MBT traffic volumes of not more than ten percent of the total through traffic volumes.

1.5 OVERVIEW OF METHODOLOGY

Due to the descriptive and empirical nature of this evaluation, a sequential mixed methods research design was used, combining both qualitative and quantitative methods. Sequential mixed methods research involves using more than one phase of data collection and analysis (Saunders, Lewis & Thornhill, 2012). When using this design, the researcher follows the use of one method with another in order to expand or elaborate on the initial set of findings. This study involved the use of two phases of data collection. The qualitative method focussed on the collection and analysis of data for developing design strategies for the priority infrastructure. On the other hand, the quantitative method focussed on data collection and analysis for performance evaluation and determination of graphical design models under a range of traffic volumes for priority infrastructure.

Figure 1-1 summarises the methodology which was employed. The evaluation was informed through multiple streams of information. Data was collected through desk document reviews, site observations, discussions and traffic count. Desk document reviews and site observations were mainly used in developing the proposed design strategies. An in-depth discussion with industry experts was done before selecting intersections for performance evaluation. Traffic data was collected through a ninety-minute traffic count using video cameras for the purpose of performance evaluation of the MBT priority infrastructure at signalised intersections.

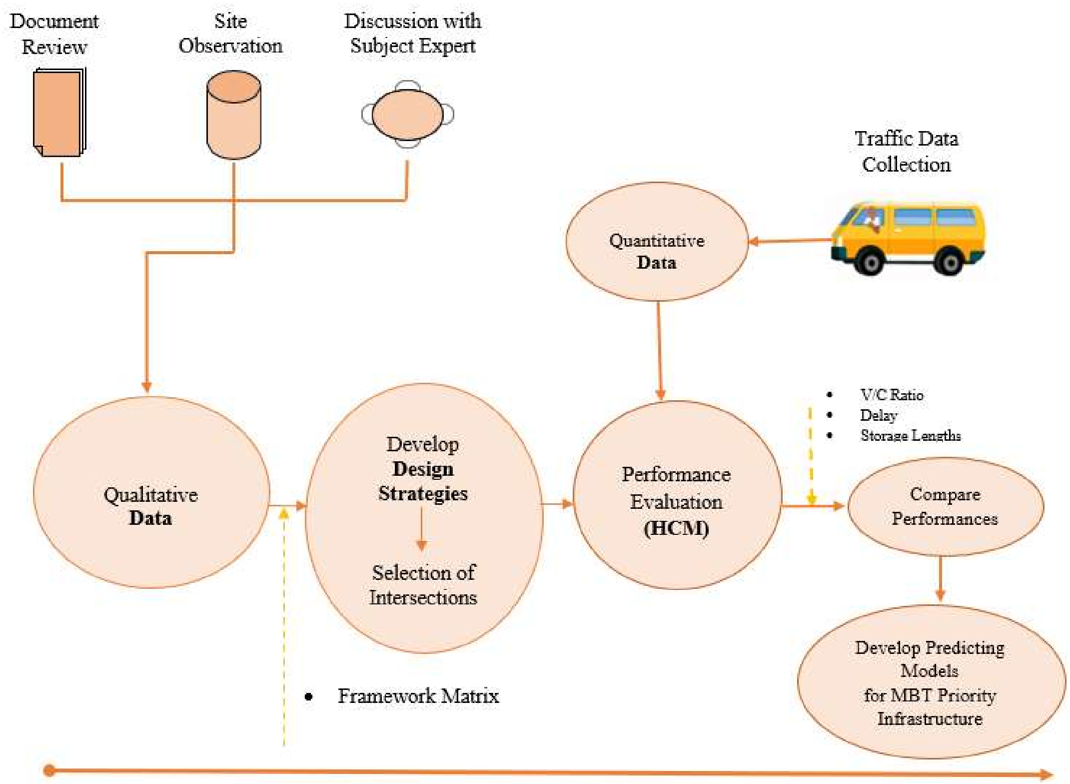


Figure 1-1 Summary of Research Methodology

1.5.1 Framework Matrix Analysis

The framework matrix analysis was used in summarising of qualitative data used for developing the design strategies for MBT priority infrastructure. The analysis was done in an Excel spreadsheet. The final objective was to show the relationship between the intersection geometry and the design treatments associated with priority infrastructure. The framework matrix table rows provide themes associated with geometric elements of urban intersections in South Africa. The table columns contain themes for design treatments associated with priority infrastructure from the best practices on priority infrastructure. The matrix table uses colour codes and numbering to show feasible design strategies for priority infrastructure at an intersection. Using the framework matrix evaluation, two categories of design strategies were developed for each choice of MBT priority infrastructure. First, design strategies that only requires repurposing of the existing intersection. Second, design strategies that require major geometric improvements. Figure 1-2 summarises the procedure taken for framework matrix analysis.

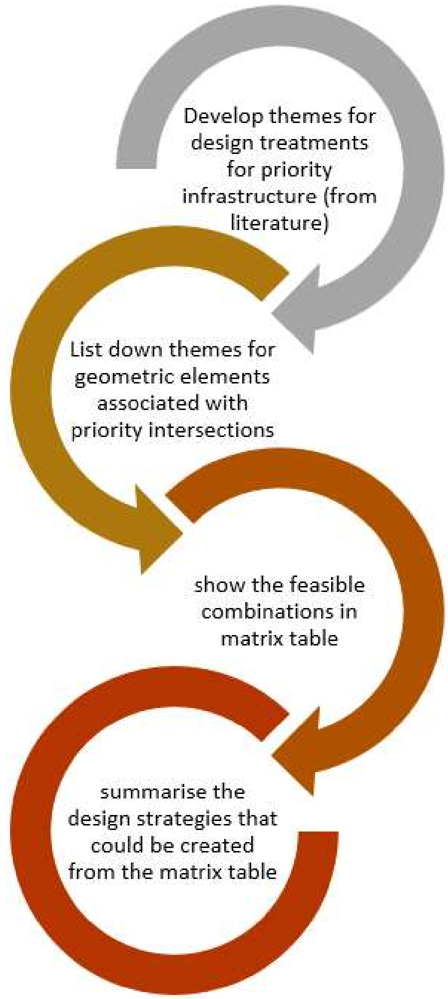


Figure 1-2: Procedure for the Framework Matrix Analysis

1.5.2 HCM Method

The Highway Capacity Manual, referred to further as HCM 2010, provides a systematic way for estimating the capacity as well as the level of service (LOS) for intersections (TRB, 2010). Figure 1-3 shows a high-level summary of the method from input parameters to performance measures. The method was used to evaluate two selected design strategies for the MBT priority infrastructure on two isolated intersections. The analysis used the peak hour traffic count data collected from the two intersections in City of Tshwane. Three performance measures which were used for the evaluation included the v/c ratios, the average vehicle delay, and the adequacy of existing storage length. For each intersection, three design scenarios were set up for performance analysis. The three design scenarios included the following: a) Option 1: 'Do Nothing' which provided the current performance of the intersection, b) Option 2: which evaluated the performance of design strategies without modification of traffic signals, and c) Option 3: which involved evaluation of the design strategies together with optimisation of traffic signals. The performance analysis provided results for traffic in both priority and non-priority lanes and how the MBT priority infrastructure affected the performance of the targeted approach and hence the entire intersection.

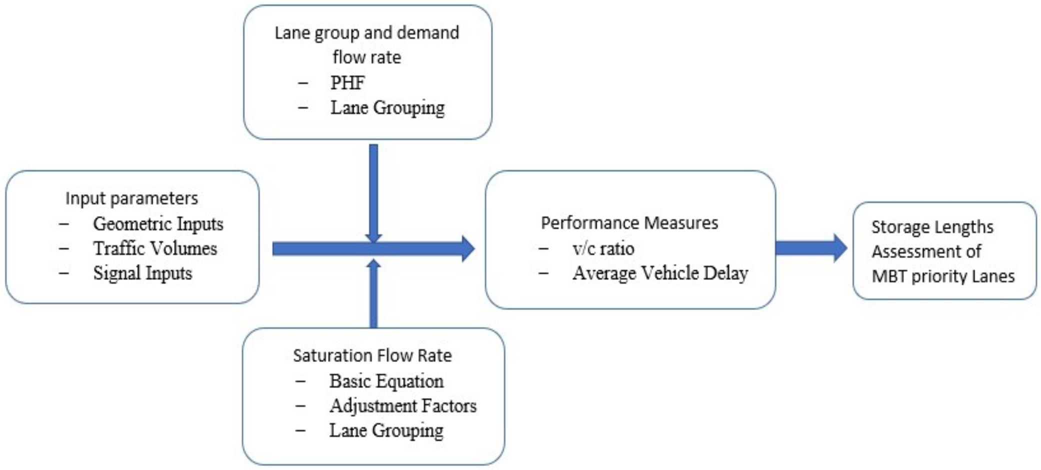


Figure 1-3: Modified HCM Analytical procedure (Adapted from: TRB,2010)

1.5.3 Sensitivity Analysis

This study used sensitivity analysis (SA) to develop graphical design models to understand the relationship between traffic volumes and capacity of MBT priority infrastructure. The analysis was performed on modified HCM analytical method used for performance evaluation. The effect of straight traffic (MBT+T) volumes and left turning (LT) volumes on capacity (v/c ratio) of the MBT priority lanes was evaluated. Traffic count data analysis on sampled selected intersections in city of Tshwane shows that through MBT traffic ranged from 3 to 7 percent of total through (T+MBT) traffic. Hence among several other assumptions, the priority lanes for both models were set to take a maximum MBT traffic of ten percent (10%) of through traffic (MBT+T) volumes to capture the worst-case scenarios. For each MBT priority lane, four different analyses were conducted, and these were set at constant g/c ratios of 0,2, 03, 0.4, and 05. For each g/C ratio, values of MBT+T and LT traffic volumes were varied. The MBT+T traffic was varied from 50 PCU/Hr to 1600 PCU/Hr at each constant LT traffic volume. Eight constant values of LT traffic were used between 50 PCU/Hr and 800 PCU/Hr. The model was set to select the maximum (critical) v/c ratio from approaching lanes including the MBT priority lanes. For each constant value of g/C ratio, graphs of constant LT were plotted to show changes in straight traffic and v/c ratios. Figure 1-4 provides a simplified setup of the sensitivity analysis.

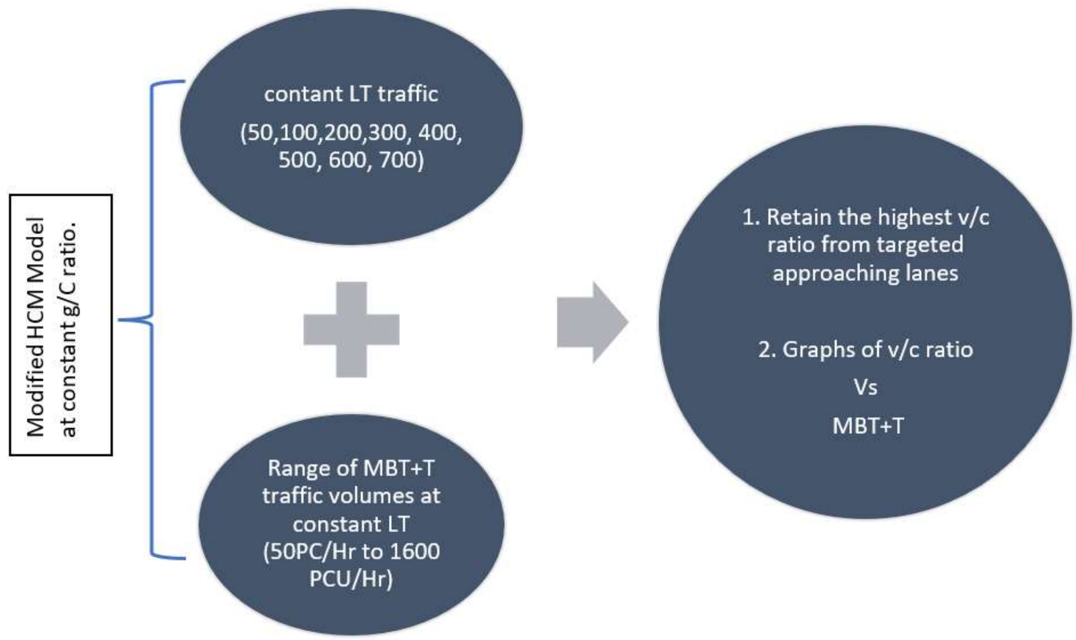


Figure 1-4: Simplified Setup of the Sensitivity Analysis

1.6 ORGANISATION OF THE REPORT

The rest of the dissertation is organised as follows: The second chapter provides a detailed review of relevant previous studies conducted on similar research and the theoretical framework. Chapter Three presents the development of design strategies for priority infrastructure and the selection of intersections for performance evaluation. In Chapter Four, the performance evaluation of signalised intersections 'with' and 'without' priority infrastructure is presented. Chapter Five provides a range of traffic conditions viable for the selected priority infrastructure. Finally, Chapter Six includes a summary of key findings, the dissertation's contribution, and future research contribution.

2 LITERATURE REVIEW

2.1 INTRODUCTION

This chapter reviews relevant literature related to the paratransit industry as well as theoretical knowledge regarding the designs and evaluation of priority infrastructure and signalised intersections.

2.2 INFORMAL PUBLIC TRANSPORT IN SOUTH AFRICA

Wilkinson et al. (2012) define informal public transport also known as paratransit as demand-driven, unscheduled public transport provided by small operators, typically in mini- to medium-sized buses, operating along quasi-fixed routes that may frequently change. Today, paratransit accounts for between 50-98% of passenger trips in Sub-Saharan cities (Jennings & Behrens, 2017).

Although the industry is not entirely regulated, it is the most used form of public transport in South Africa. In the cities of Johannesburg and Cape Town, minibus-taxis account for 66% of all public transport trips (van Ryneveld, 2018). Paratransit typically operates with little government oversight or regulation, which often results in poorly maintained vehicles, unsafe driver behaviour and fierce competition among operators for routes and passengers. In many parts of the world including South Africa, informal public transport is the only public transportation option for residents, providing transport where none may exist, as well as employment to poor or lower-skilled workers (Jennings & Behrens, 2017).

Oni (2018) observes that in recent years, the focus of the South African government has been to improve the public transport system throughout the country. This is evidenced through the implementation of the Integrated Rapid Public Transport Network (IRPTN) using BRT as the backbone of the system. In addition, municipalities across the country have implemented various approaches to upgrade and support MBT which include provision of infrastructure such as ranks and lay-bys. However, other related infrastructure related strategies have not been explored.

2.3 BENEFITS AND APPLICATION OF TRANSIT PRIORITY FACILITIES

A few studies have examined transit priority interventions, which are measures that would reduce delays for public transport on congested roads and advance the quality of services for public transport. Most of these studies are theoretical and have not been tested in practice. De Beer and Venter (2021) developed an analytical model to quantify the potential benefits and impacts of priority infrastructure for minibus-taxis in Pretoria, South Africa. The study looked at three priority infrastructure that have proven to be effective in the public transport sphere, particularly pertaining to buses. These included the single-lane pre-signal strategy, queue-jumping lane, and dedicated taxi lane. The first priority infrastructure, the single-lane pre-signal strategy, was modelled to provide priority to minibus-taxis at signalized intersections with single-lane approaches by using additional signals to stop cars on the opposing travel lane while allowing minibus-taxis to jump a portion of the car queue using the travel lane in the opposite direction. The second priority infrastructure, a queue-jumping lane, was modelled to allow a minibus-taxi to bypass queued traffic by allowing it to gain an advantage at a signalised intersection so that it leaves the queue and enters the queue-jumping lane. The third priority infrastructure, the dedicated taxi lane, was modelled to provide an exclusive lane that is restricted to be used by minibus-taxis only to increase travel times of minibus-taxis that would otherwise be held up by traffic congestion. The paper contends that substantial savings could be realised in terms of travel time, user cost, and operating cost to taxi passengers and drivers without additional costs being incurred by other road users. Quantitatively, the single-lane pre-signal strategy, the queue-jumping lane and the dedicated taxi lane saw a decrease in total hourly cost by 12%, 14% and 30% respectively. These costs include construction cost, user cost, and agency cost, indicating a net social benefit.

Oni (2018) evaluated warrants for road space prioritisation of paratransit vehicles on the road network along the Mitchell Plain interchange in Cape Town, South Africa. The study used a mathematical model to investigate various infrastructure developments to the road network using road space prioritisation that could enhance the operational efficiency of paratransit distributor services to bring about a more efficient and coordinated paratransit-schedule trunk service complementarity. Using an agent-based simulation modelling tool called Commuter, the study evaluated the impact of each implemented scenario on the efficiency of the paratransit feeder's service. The implemented scenarios included priority infrastructure such as traffic signals, queue-jumping lanes only, dedicated lanes and a combination of queue-jumping lanes and traffic signals. The results showed that the provision of the priority infrastructure improved efficiency for paratransit vehicles, with dedicated lanes for paratransit vehicles being the most efficient infrastructural improvement strategy, especially in a traffic-congested route.

In a study on the performance of full BRT and partial bus priority strategies at intersections in a South African context, Chitauka and Vanderschuren (2014) investigated the effect of priority strategies on delay and speed. The study sought to quantify the performance of dedicated bus lanes, bus queue jumps and bus mini-bus taxi (BMT) lanes. Through the application of micro-simulation software, Quadstone PARAMICS, several suitable transit priority schemes were modelled for a proposed transit corridor in Cape Town. The results showed that the alternative forms of public transport priority measures reduce travel delays and improve bus speeds without adversely affecting the operations of other general traffic. The study recommended the implementation of alternative bus priority schemes such as bus queue jumpers at intersections and BMT lanes along sections at appropriate locations or corridor segments.

At the international level, many studies have shown substantial benefits of providing transit-priority infrastructure. In the United States of America, studies by Cesme et al (2014), Zahid Reza (2012) and Ilgin Guler et al (2015) evaluated the potential benefits of different transit preferential treatments for buses at intersections. Such transit preferential treatments included queue-jumping lanes and transit signal priority on single-lane approaches. The results saw bus travel times decrease by 5-30 seconds per vehicle per intersection.

In Hungary, Desta and Tóth (2021) evaluated the performance of integrated bus priority setups using Vissim microscopic traffic simulation software. They studied three priority models which included queue jump lanes with signal priority for buses approaching an intersection, an exclusive bus lane or dedicated median lane for buses, and an exclusive bus lane followed by a special lane for turning buses at intersections. The results showed that the priority schemes reduce bus delays by a minimum of 21.32% at intersections with some of the strategies likely to require less investment.

Delay at intersections is one of the largest components of bus delay on arterial streets (Evan and Skiles, 1970). Bus delay at traffic signals comprises between 10 and 20 percent of overall bus trip times and nearly 50 percent of the delay experienced by a bus. Greater Empire Transit (2018) outlines that transit priority measures seek to improve bus service by reducing travel time. The components of this travel time include getting to and from bus stops, time waiting for the bus to arrive, and the time spent travelling on the bus. Transit priority measures therefore primarily seek to reduce the in-vehicle component of travel time by giving public transport priority over other types of vehicles on streets or at an intersection. To be successful, the study recommends that the transit priority measures must be coordinated with the local jurisdictions responsible for traffic control and roadway planning and operations.

In addition, Nelson and Bullock (2000) indicate that priority for public transport vehicles is only granted when specific conditions are satisfied and should be carefully investigated case by case. Priority for public transport (PT) vehicles is commonly applied for reasons of schedule adherence, for instance., punctuality for PT vehicles, or headway adherence (Hounsel & Shrestha, 2012).

Current evidence in South Africa shows that priority facilities are beneficial (De Beer and Venter, 2021; Oni 2018; Chitauka and Vanderschuren,2014). However, models by De Beer and Venter (2021); and Oni (2018) were exploratory. The two models did not provide a detailed analysis of the design considerations, capacity performance, signalisation, and proposed geometric layout with respect to transit treatments and other elements of the intersections. Similarly, a study by Chitauka and Vanderschuren (2014) focussed on priority strategies for buses which ideally may not have the same capacity demands as that of minibus-taxis. Lastly, the decisions for the choices of priority infrastructure that were evaluated in the three studies were not comprehensively and scientifically motivated.

Road authorities for city roads in South Africa such as Gauteng Department of Roads and Transport (GDoRT), City of Johannesburg (CoJ), City of Tshwane (CoT) and City of Cape Town have all recently considered incorporating transit-priority facilities in the design of roads in South Africa (CoJ, 2013; GDoRT, 2021, Du Preeze and Venter, 2022; De Beer & Venter, 2021). These recent design considerations demand that road designers should include priority facilities and provide supporting detailed analysis of intersections showing the impact of including priority facilities on overall intersection operation. However, there are no further design guidelines or analytical procedures to assist engineers during planning and detailed design stages of these priority treatments at an intersection. Moreover, most of the available literature (Lowe and Friesslaar (2019); Bulman and Van Ryneveld (2015)) pertains to the design of other types of priority infrastructure especially for buses.

It is against the backdrop of this research gap that this research study attempts to develop and evaluate design strategies for priority infrastructure for minibus-taxis at signalised intersections.

2.4 DESIGN THEORY OF TRANSIT TREATMENTS AT INTERSECTION

The design of transit treatments at intersections follows the use of both technologies as well as modification of intersection geometry. The National Association of City Transportation Officials (NACTO) (2016) in the United States observes that geometry guides street users through intersections, working in tandem with signals to sort out conflicts and establish priority among users. The study also indicates that intersections can be organized by designating turn and through lanes, setting clear vehicle and walking paths through the intersection, and providing transit vehicles with a way to avoid general traffic queues and make use of signal priority treatment.

SCAG (2022) conducted a study on transit priority best practices as part of the research to provide guidelines for effective and sustainable transit options for the California region. The study grouped transit treatments into two main groups which include design or infrastructure treatments, and operations and technology treatments. The study defines infrastructure treatments as facilities that make transit faster and more reliable. Typical examples of the design treatments that were evaluated included bus lanes, far-side bus stops, bus bulb-outs, level boarding, facilitate left turns, floating bus islands, and bus-bicycle treatments. Similarly, the study defined operational and technology treatments as strategies that complement design treatments to make service faster and more reliable. Typical examples included transit signal priority (TSP), queue jump/bypass, bus stop balancing, bus-bicycle treatments, and real-time information.

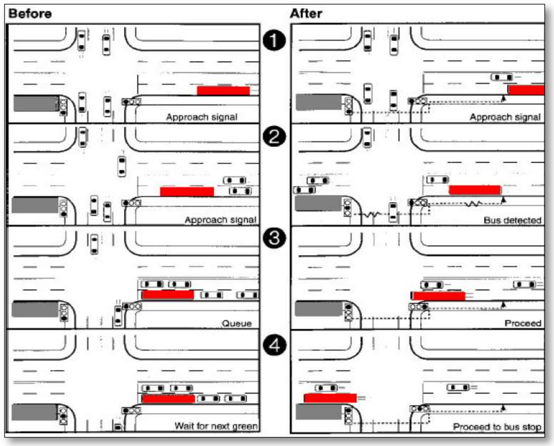
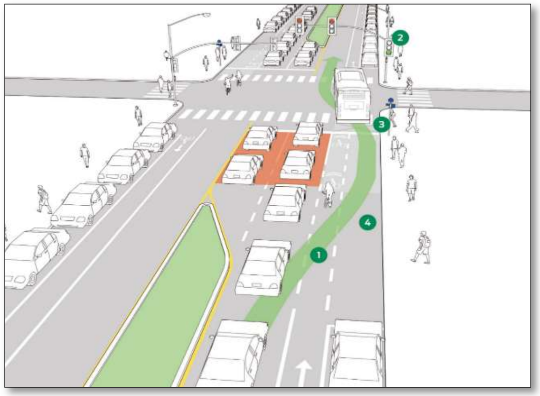
A study on the evaluation of transit signal priority strategies for small-medium cities by Ova and Smadi (2016) made a similar observation on the fundamental design theory of transit treatments. The study focussed on TSP strategies which among other strategies included phase splitting, progression/coordination to favour priority vehicle movements, increasing the priority phase split, and queue jumps. The study indicated that passive priority strategies mainly consist of signal timing modifications favouring the transit vehicle but also include geometric or infrastructure enhancements.

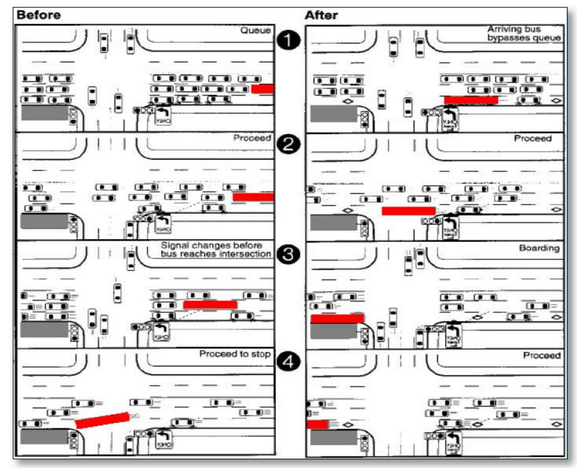
This study utilised these design principles in developing and evaluation of priority infrastructure for MBT at intersections.

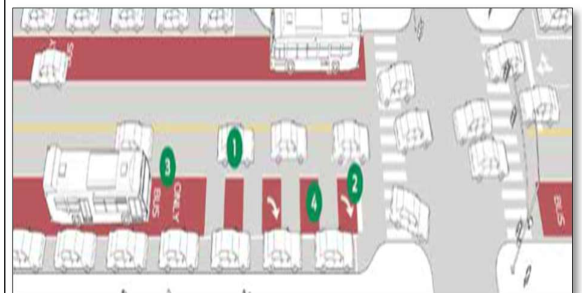
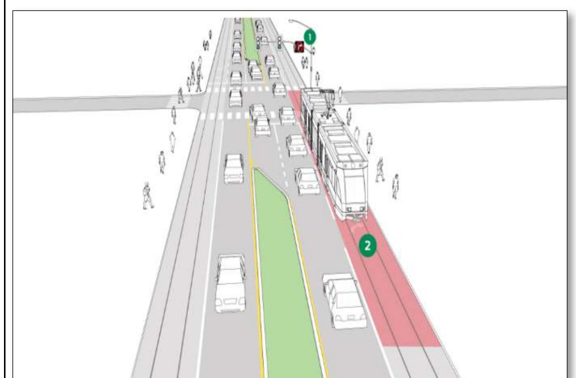
2.5 BEST PRACTICES ON TRANSIT TREATMENTS AT INTERSECTIONS

Studies done by NACTO (2016), SCAG (2022), FHWA (2008), CoJ (2013) and Cesme et al (2014) have presented several guidelines for designing priority facilities for transit vehicles. Table 2-1 summarises guidelines for some of these design strategies or treatments suitable at intersections. It provides a summary of descriptions and applications of selected transit treatments that have been adopted or proposed on intersections. The table also shows geometric layouts of these selected treatments. It is important to note that these priority infrastructure and guidelines outlined in the table are a starting point but do not apply as is to the MBT case in South Africa.

Table 2-1: Summary of Best Practices of Transit Treatment Applications

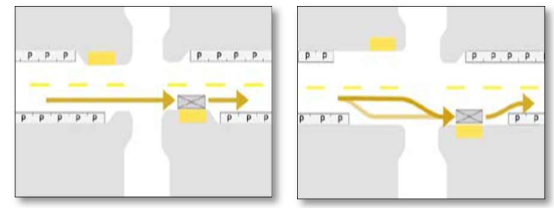
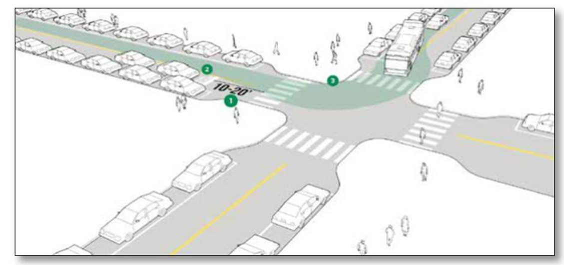
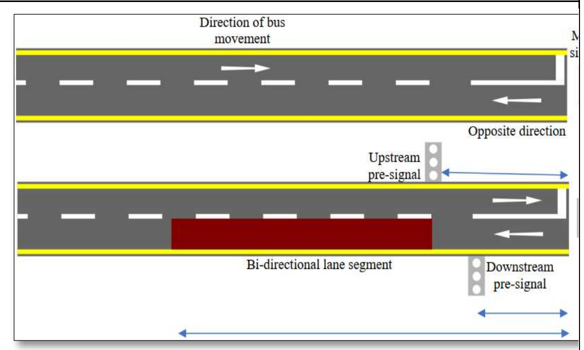
Transit Treatments	Description	Application & Context	Geometric/Design Layout
Transit Signal Priority (TSP) (SCAG, 2022; FHWA, 2008; CoJ, 2013)	Allows transit vehicles to communicate with signals to: • extend green lights, • end red lights early, • add a bus-only signal phase. The most common public TSP systems in use are: *Passive priority* (adjust cycle length, split phases, bypass metered signals). *Active priority* (early start, phase extension, special phase).	Signalized intersections with a far-side stop or no transit stop, allowing the bus to clear the intersection without waiting at a signal. Depends on both geometric and operational factors, including roadway facility type, general traffic volume and capacity, signal spacing, and cycle length, and signal detectors. TSP is most effective when designed in conjunction with measures such as queue bypass lanes.	
Queue Jump (SCAG, 2022; NACTO, 2016; Cesme et al, 2014; CoJ, 2013)	Designated spaces that allow buses to proceed through a signalized intersection ahead of general traffic. It allows buses to call for an early green phase that starts 2 – 3 seconds ahead of the normal green phase. Queue jump lanes combine short, dedicated transit facilities with either a leading bus interval or active signal priority to allow buses to easily enter traffic flow in a priority position.	Used along a corridor with existing lane geometry that supports installation, at spot locations with high delay or nearside bus stops. Can also be used in signalized streets with low or moderately frequent bus routes, especially where transit operates in a left lane with high peak hour volumes, but relatively low left turns.	

Transit Treatments	Description	Application & Context	Geometric/Design Layout
Queue Bypass or Transit Approach Lanes (Cesme et al, 2014; SCAG,2022; CoJ, 2013, NACTO, 2016)	Queue bypass or transit approach lanes are bus-only lanes to the nearside of left turn pocket. A queue bypass extends to the other end of a signalised intersection. It is a short lane used by public transport vehicles to bypass traffic queues at signalised intersections. It allows transit vehicles to bypass long queues that form at major cross streets.	Used where a dedicated left turn lane is present, and traffic volumes are high. These are particularly applicable to intersection approaches with high through lane queue delay and low left-turning volumes. They are also used at the approaches to signalized intersections where transit encounters lengthy delays.	
No Car Lanes (Mulley, 2011)	These are lanes used by buses, goods vehicles and some other modes of transport, but cars are prevented from using the designated lane.	These are specifically used in cities with mixed traffic road networks whereby the priority lanes, if restricted only to public transport vehicles, would have spare capacity.	

Transit Treatments	Description	Application & Context	Geometric/Design Layout
Shared Transit (NACTO 2016)	On streets with a right-side[1]dedicated transit lane that accommodates a moderate volume of right-turn[1]movements, the transit lane can permit right-turn[1] vehicles approaching an intersection.	Used at locations where right-turning[1] vehicles can typically clear through the intersection quickly. Can accommodate moderate right-turn[1] volumes at intersections where right turn on red is permitted and pedestrian volumes are low. Can be applied to streets with or without dedicated transit lanes.	
Virtual Transit Lane (NACTO 2016)	These lanes permit right-turns[1] only when a transit vehicle is not present. When a transit vehicle approaches, right turns[1] are prohibited. Transit signals are triggered to allow transit vehicles to pass through the intersection.	Usually used at streets with moderate transit service frequency, often with streetcar operating in a mixed-travel or shared turn lane. Also used at intersections where right-turning[1] vehicles are subject to delays while yielding to pedestrians and bicyclists. Note: Prohibiting turns when transit is present may be beneficial with or without a dedicated transit lane, especially for street cars.	

[1] NACTO is a US Manual which uses right-tuns or right sides to mean left-turn or left side in South Africa. In US, drivers drive on the right hand side.

Transit Treatments	**Description**	**Application & Context**	**Geometric/Design Layout**
Peak-Only Lanes (SCAG,2022)	Lanes which are reserved for transit at peak travel periods (such as the morning and evening commute) and are used for general traffic at other times of the day.	May require repurposing existing travel lane, parking lane, or additional right of way to support new construction.	 Peak-only bus lane signage.
Business Access and Transit (SCAG,2022)	These are dedicated bus lanes that allow intermittent access for vehicles turning at intersections and vehicular access to driveways to reduce travel times, improve reliability, and maintain business and community access.	Often deployed in urbanized areas that have an established roadway grid network with alternative routing options for existing auto traffic. Corridors where implementation of BRT or enhanced bus lines with high frequency service have been proposed. Support high ridership lines that experience high delay due to traffic congestion; or where increased capacity is warranted to meet demand or mitigate potential crowding at bus-stop locations.	

Transit Treatments	Description	Application & Context	Geometric/Design Layout
Far-side bus stops (SCAG,2022)	They are located after an intersection, allowing the bus to travel through the intersection before stopping to load and unload customers.	Beneficial in locations with long signal cycles or short green signal times.	
Facilitate left-turns. (SCAG,2022)	Formed by modifications to the existing lane striping and marking at intersections, as well as potential changes to on-street parking, curb or travel lane geometry to support buses making left-turns.	Where bus routes require a left turn. Intersections in urban environments where space is constrained (narrow lane widths and turning radii) and buses may be delayed when attempting to make left turns.	
Single lane pre-signal strategy (Ilgin Guler et al, 2015; De Beer & Venter, 2021)	Strategy that provides priority to buses at signalized intersections with single-lane approaches through the use of additional signals by allowing the bus to jump a portion of the car queue using the travel lane in the opposite direction.	Used at signalised intersections with single-lane approaches.	

2.6 OVERVIEW OF THEORIES ON INTERSECTION DESIGNS

NamGung et al (2020) define an intersection as a place where two or more roads intersect, allowing vehicles and pedestrians to gather, turn, and evacuate. Its main purpose is to give road users the chance to change their route direction. Different street markings, traffic signs, and traffic control lights are used to guide the lines of vehicles towards the intersection at applicable speeds and avoid vehicle crashes. Intersections are the elements of the road network at which most collisions occur in urban areas (COTO, 2012; Sampson, 2019). It is the place with the least capacity, the most delay and the highest crash rate.

2.6.1 Intersection Geometry

Literature (SCAG,2022; NACTO 2016; TRB 2010; CoJ,2013) on priority infrastructure has shown that auxiliary lanes are widely used to give priority to transit vehicles at intersections. In South Africa, COTO (2012) provides specifications for the design of auxiliary lanes. For example, it recommends that left-turn auxiliary lanes should be provided on all uncontrolled and traffic signal-controlled approaches to intersections and accesses on Class 1 to 3 roads. Left-turn lanes are not required on Class 4 and 5 roads (including service stations on such roads). Figure 2-1 shows the auxiliary lane at intersections:

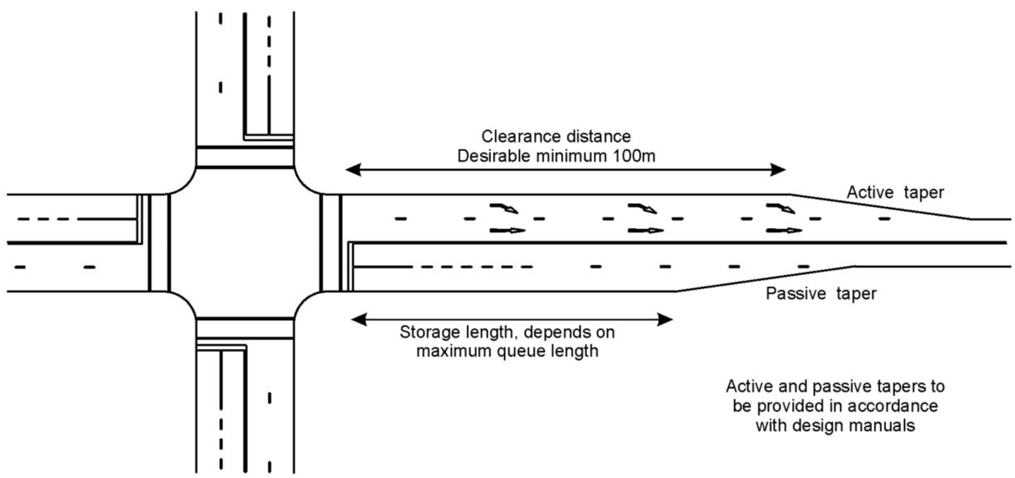


Figure 2-1: Auxiliary lanes at signalised intersection (Source: NDoT, 2012)

a) Auxiliary through-lanes

These are auxiliary lanes for through-traffic which are added outside the through-lanes to match the capacity of the intersection with that of the road between intersections (CSIR,2000). These lanes are normally only provided at signalised intersections. The length of the lane to be added is a matter of calculation. It is dependent on the traffic flow to be serviced and on the length of green time available for the approach leg in question.

b) Auxiliary turning-lanes

These are turning lanes provided for traffic turning either to the left or to the right which are added either outside the through-lanes or immediately adjacent to the centreline (CSIR,2000). The length of turning lanes has three components: the deceleration length, the storage length and the entering taper. CSIR (2000) provides an equation for deceleration lane length as

$$s = \frac{v}{38.9} v \qquad \text{(Equation 1)}$$

where:

s = deceleration lane length (m)

v = design speed (km/h)

c) Storage Length

Storage lengths of auxiliary lanes at signalised intersections are determined based upon the maximum number of vehicles that will accumulate at any one time. It is more desirable to provide storage length for at least five vehicles (about 30m) (SARTSM, 2012). The AASHTO (2018) provides the formular (Equation 2) used for calculating average storage lengths as a function of the probability of occurrence of events and is usually based on one and one-half to two (1.5 to 2) times the average number of vehicles that would store per cycle, which is predicated on the design volume or directly from traffic counts. This equation has also been adopted in this study to check the adequacy of the existing storage lengths for MBT priority infrastructure.

Average Storage Length (L_{avg})= $1.5*n_i*L_i$ (Equation 2)

Where:

1.5 = gap factor between queuing vehicles

n_I=number of vehicles by type that would store per cycle

L_i=Length of the type of vehicle

Note: In practice, storage lengths vary between 30 m and 60 m.

d) Tapers

Tapers are classified as either passive, allowing a lateral movement in the traffic stream, or active, forcing the lateral movement to take place (CSIR,2000). Thus, the addition of a lane to the cross-section is preceded by a passive taper, and a lane drop by an active taper. In general, an active taper should be long whereas a passive taper can be short. Table 2-2 provides details of values to use for tapers at different design speed of the roads.

Table 2-2: Taper rates (Source: CSIR, 2000)

DESIGN SPEED (km/h)	30	40	50	60	80	100
Passive tapers						
Taper rate (1 in)	5	8	10	15	20	25
Active tapers						
Taper rate (1 in) for painted line taper	20	23	25	35	40	45
Taper rate (1 in) for kerbed taper	10	13	15	20	25	30

e) Corner radii

Desirable corner radii at signalised intersections range from 6 m to 10 m but radii as small as 6 m and as large as 12 m can still be used (National Department of Transport, 2012)

2.6.2 Types of Intersection Control

Intersection control is classified as priority control (unsignalized intersection) and traffic signals (signalised intersection) (Sampson, 2019).

Unsignalised intersections also known as priority controls are all fixed once implemented and cannot be adjusted for any time of day or varying traffic conditions. Priority control implies that one of the intersecting roads always takes precedence over the other with control taking the form of either stop or yield control (CSIR,2000). An unsignalized intersection operates without being controlled by a signal device and that gives a few vehicles chances to disregard the movement directions to cross through the intersection as quickly as likely (Fan et al, 2014).

On the other hand, signalised intersections are intersections which are controlled by signals such as traffic lights and hence give drivers less freedom (Yao et. al, 2018). In terms of flexibility in operations, Sampson (2019) observes that traffic signals are highly flexible and have literally an infinite number of possible operational settings. Hence, a traffic signal is considered the most critical type of control (TRB,2010). In a study report on intersection traffic engineering, Sampson (2019) compares the performance of different intersection controls such as two-way stop, four-way stop, mini-circle, roundabout, and traffic signals. The study ranks the performances based on maximum capacity, minimum delay, shortest queues, maximum safety and minimum cost and maintenance. Among other observations, the study ranks traffic signal as the control that gives the maximum capacity.

2.7 CAPACITY ANALYSIS OF SIGNALISED INTERSECTIONS

A capacity analysis is undertaken to determine whether the transportation system has sufficient capacity to accommodate the expected traffic demand (COTO, 2014). Pretorius and van As (2004) observe that various studies undertaken in South Africa and Australia show that the operational analysis of an intersection is a complex exercise which often produces invalid results. They further highlight that other studies have shown that many factors need to be taken into account when modelling urban intersections, and that the models should be properly calibrated and validated. They conclude that unless these issues are properly addressed, the operational analysis of intersections serves little or no purpose. In conclusion, they propose that to measure the capacity of interactions, it is better to use simpler approaches or methods such as measuring volume/capacity ratios instead of using models that would end up providing invalid results.

2.7.1 Capacity Models/Methods for Signalised Intersection

Technical Methods for Highway (TMH16) Volume 2 provides models for evaluating the capacity and performance of intersections or accesses in South Africa (COTO, 2014). The manual stipulates that the capacity analysis of traffic signal-controlled intersections should be undertaken using the method and model parameters provided in the Highway Capacity Manual (HCM 2010). Where software is available, the manual emphasises that such sofware should be well-calibrated and validated for South African conditions. The two softwares which were recommended included SIMTRA Traffic Simulation and HTModel.

However, in recent years, several softwares have been developed to aid in intersection capacity analysis. Other software currently available in South Africa for intersection capacity analysis and design include:

AutoJ (Sampson, 2020)
Sidra Intersection (Akcelik 1990)
Vissim (PTV Group, 2018)

Table 2-3 summarises the key features of each of these models and software. The models and software that have been evaluated include: HCM Model, SIDRA, Vissim, HTM, SIMTRA, and Auto J

Table 2-3: Summary of common software/ models used for capacity analysis.

Model/Software	Key features
HCM 2010 (TRB, 2010)	Analytical model developed by American Transportation Research Board. Provides a methodology that analyses the capacity and level of service (LOS) of signalized intersections. The analysis considers a wide variety of prevailing conditions, including the amount and distribution of traffic movements, traffic composition, geometric characteristics, and details of intersection signalization. The methodology addresses the capacity, LOS, and other performance measures for lane groups and intersection approaches and the LOS for the intersection as a whole.
SIDRA Interaction (Akcelik 1990)	SIDRA (Signalised Intersection Design and Research Aid) is a macroscopic software developed by the Australian Road Research Board as an aid for capacity, timing and performance analysis of signalised intersections. It uses input and output facilities at individual turn, lane, lane group, approach road, movement grouping and interaction levels to provide flexible structure which allows multilevel analysis of very simple to very complex intersection conditions.
Vissim (PTV Group, 2018)	It was developed by PTV Group from Germany. Vissim is a microscopic, time step oriented, and behaviour-based simulation tool for modelling urban and rural traffic as well as pedestrian flows. It uses gap acceptance model and car following model plus lane changing model based on Wiedemann's model. It allows exact dimensioning of lanes and features due to its high spatial resolution. Its application includes the following: comparison of junction geometry, traffic development planning, capacity analysis, traffic control systems, signal systems operations and re-timing studies, public transit simulation.
AutoJ (Sampson, 2020)	AutoJ is a software program developed in South Africa by a renowned traffic engineer, Dr John Sampson. The software simulates and optimizes intersection control devices. The program has built-in defaults for all other input data at a typical urban intersection, but traffic engineers are advised to confirm the defaults and over-write if necessary. It is primarily used for Traffic Impact Assessments, signal designs and timing plans, and warrant Investigations.
SIMTRA Traffic Simulation (COTO, 2014)	Developed in South Africa in 1985 by Van As. It is used for intersection simulation including signalised intersection. (No further literature found regarding its operation)
HTModel Highway Traffic Model (COTO, 2014)	Developed by Van As in 2008. (No further literature found on its operation)

This study uses HCM Model to evaluate performance of the priority infrastructure against the existing performance conditions. The HMC Model is detailed in the subsequent section.

2.7.2 Application of the HCM Model

The Highway Capacity Manual, referred to further as HCM 2010, provides a method for estimating capacity as well as the level of service (LOS) for intersections (TRB, 2010). Since its inception in 1950, HCM methodologies of evaluating intersections have been widely used in estimating delay at signalised intersections (Alkaissi and Hussain, 2020). It is, therefore, no surprise to see that the model has been well accepted in many countries as a key guide in evaluating the capacity of intersections. In South Africa, the committee for transport officials (COTO) have recommended HCM as the acceptable analytical method to use for analysing the capacity of signalised intersections (COTO, 2014). On the other hand, there is a literature void regarding industrial experiences on the performance of locally available software such as SIMTRA Traffic Simulation and HTModel which were also recommended by COTO (2014).

A study by Pretorius et.al (2004) warns that research studies on operational analyses of intersections have shown that a large number of factors need to be taken into account when modelling urban intersections, and that the models should be properly calibrated and validated. For the purpose of intersection analyses, the paper proposes the use of simpler approaches like the volume/capacity ratio as stipulated in HCM.

Chaudhry and Ranjitkar (2009) made a comparison between analytical models and simulation software. They selected two analytical models (HCM 2010 and ARR) and one traffic simulation model (AIMSUN). The micro-simulation model was calibrated using realistic local parameters to represent the real-world situation. The capacity and performance results show that a correlation (R) of about 0.98 exists between the two analytical models and the simulation models. This further proves the effectiveness of the HCM model. In another study conducted by Dion, Rakha and Kang (2004), three capacity models (HCM, Australian capacity guide and Canadian capacity guide) and a microscopic simulation model (Integration) were compared in under-saturated and oversaturated traffic conditions to determine any available variances. The results indicated that all models produced similar results for signalised intersections with lower traffic demands.

Furthermore, HCM 2010 method is also used in the majority of traffic simulation packages, including SIDRA, PTV Vistro and Aimsun (Bruwer; Bester & Viljoen, 2019). Over the years, traffic simulation models have been enhanced to align with HCM principles of evaluating intersections which suggest the confidence the industry has with the method. Milam, Stanek and Breiland (2007) observed that level details used in performance reporting and input data for software like Vissim, SimTraffic, CORSIM, and Paramics are mostly in line with HCM methodologies. Akcelik (1990) conducted a study on features of SIDRA software to improve its performance by using HCM model principles as a base for software calibration, modification, and improvement.

In addition, the unrealistic results and complexity in the calibration and validation of traffic simulation software have led to continuous reliance on the HCM model. A typical example is a paper presented by Akcelik (2022) on the level of service and performance discrepancies experienced on SIDRA. The paper presents a number of issues that professionals have raised regarding the limitations of SIDRA software. Notable findings include low delay estimated at high a high degree of saturation (v/c ratio) leading to a LOS of F in HCM. Salgado et al (2016) performed an assessment of traffic microsimulation models which included the Vissim model. The study sought to evaluate the advantages and disadvantages of the models. Among the conclusions, the study observed that there is no script available if the modeller does not have solid programming knowledge. In addition, the study noted that it is time-consuming to properly validate the model in terms of wait times. It is difficult to calibrate lane change behaviour for heavily congested conditions to make it realistic. Also, VISSIM does not have a proper toolbox when it comes to lane closure management. Vissim was also found to be more complex in terms of operation (Tianzi, Shaochen & Hongxu, 2013).

Arnold and McGhee (1996) evaluated existing signalized intersection capacity analysis software to determine programs that provide acceptable results by evaluating the results from simulation models to determine when and how to use this output in the analysis of signalized intersections. The software included HCS (computer software for HCM), SIGNAL94, CINCH and HCM/Cinema. Based on the case study evaluations, all the software produced acceptable estimates of delay when compared to observed field measurements at isolated signalised intersections.

2.8 CAPACITY EVALUATION USING HCM 2010

Highway Capacity Manual provides the opportunity to analyse the capacity and level of service of the roads in urban or rural areas, by defining the volume/capacity (v/c) ratio and delay of the analysed facilities (Nedevska, Ognjenović & Gusakova, 2016). To obtain this information, one must have data for geometric, traffic and signalisation parameters, when the intersection is signalised. The HCM methodology follows the procedure in Figure 2-2 in evaluation capacity and performance of signalised intersection:

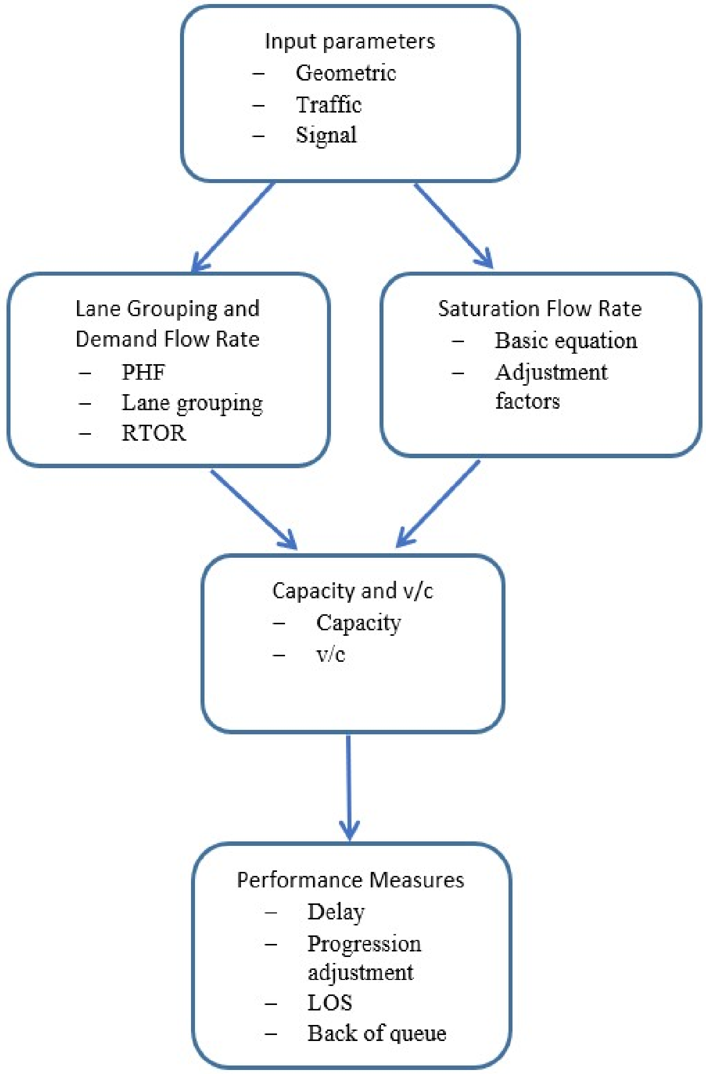


Figure 2-2: HCM Procedure for Signalised Intersection (Source: Adapted from TRB,2010)

2.8.1 Input Parameters

The key parameters that are considered when using the HCM method include geometric parameters, traffic and signal conditions. These input parameters under each category are listed as follows:

a) *Geometric parameters*

- Area type
- Number of lanes N,
- Average lane width W (m),
- Grade G (%),
- Existence of exclusive LT or RT lanes,
- Length of storage bay LT or RT lane, Ls (m),
- Parking

b) Traffic conditions

- Demand volume by movement V (veh/h)
- Base saturation flow rate s_0 (pc/h/ln),
- Peak-hour factor PHF,
- Percent heavy vehicles, HV (%),
- Approach pedestrian flow rate vped (p/h),
- Local buses stopping at intersection Na (buses/h),
- Parking activity Nm (manoeuvres/h),
- Arrival type AT,
- Portion of vehicles arriving on green P,
- Approach speed SA (km/h)

c) Traffic signal conditions

- Cycle length C (s),
- Green time G (s),
- Yellow – plus – all – red change – and – clearance interval (intergreen) Y (s),
- Actuated or pretimed operation,
- Pedestrian push button,
- Minimum pedestrian green Gp (s),
- Phase plan,
- Analysis period T (h)

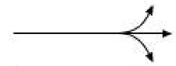
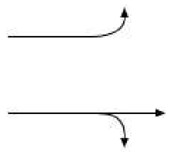
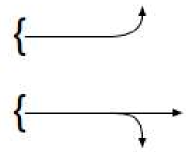
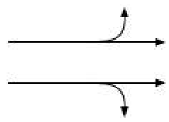
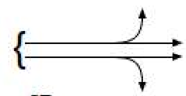
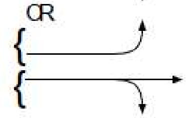
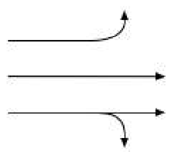
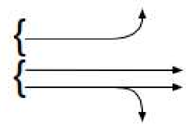
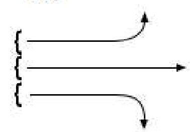
2.8.2 Lane Grouping

Lane grouping is dependent on geometric conditions and traffic movement distribution. The lane group describes the manoeuvres in the intersection area:

- Lanes for right turns should be considered separately unless there is a lane for a right-through shared lane. In this case, a portion of right turns should be computed.
- When there are exclusive left and right–turn lanes, the other lanes go in one lane group.
- If the lane is used for both left–turning and through vehicles, the portion of left turns will determine whether the lane will be considered an exclusive left lane.

Typical lane groups used in the analysis are shown in Table 2-4.

Table 2-4: Typical Lane groups for analysis (Source: TRB, 2010)

Number of Lanes	Movements by Lanes	Number of Possible Lane Groups
1	LT + TH + RT 	①  (Single-lane approach)
2	EXC LT TH + RT 	② 
2	LT + TH TH + RT 	①  OR ② 
3	EXC LT TH TH + RT 	②  OR ③ 

2.8.3 Demand Flow Rate

Traffic demand is calculated using the average flow in the analysed period (usually 15 minutes). The flow rate during the peak 15-minute period is computed by dividing hourly volume (veh/h) with the peak–hour factor using the equation (3):

$$v_p = \frac{V}{PHF} \qquad \text{(Equation 3)}$$

Where:

v_p = demand flow rate

v = Hourly traffic volumes in veh/hr

PHF = Peak hour factor (typically between 0.85 and 0.95 for urban roads (Sampson, 2019))

2.8.4 Saturation Flow Rate and Adjustment Factors

Saturation flow describes the number of passenger car units (pcu) in a dense flow of traffic for a specific intersection lane group (Bester and Meyers, 2007). The saturation flow rate is determined for each lane group, and it is considered to be the flow in veh/h in a lane group, assuming the green phase is displayed 100% of the time (TRB, 2010). The saturation flow rate is determined by applying several adjustment factors (f) to the base saturation rate as shown in Equation 4.

$$S = S_o N\ f_w\ f_{tv}\ f_i\ f_p\ f_{bb}\ f_a\ f_{LU}\ f_{LS} f_{RS} f_{Lpb}\ f_{Rpb} \quad \text{(Equation 4)}$$

Where:

S_o = Base Saturation rate

N = Number of lanes

f_w =Adjustment factor for Lane width

f_{HV} =Adjustment factor for heavy vehicles in traffic stream

f_g = Adjustment factor for approach grade

f_p = Adjustment factor for the existence of a parking lane and parking activity

f_{bb} = Adjustment factor for blocking effect of local buses in intersection area

f_a = Adjustment factor for area type

f_{LU} = Adjustment factor for lane utilization

f_{LT} = Adjustment factor for left turns in lane group

f_{RT}= Adjustment factor for right turns in lane group

f_{Lbp} and f_{Rbp} = adjustment factors for pedestrian and bicycle factor for left and right – turn movements

Determination of base saturation flow (S_o) has been a contentious topic over the years with values varying from country to country. In South Africa, a study by Bester and Meyers (2007) on based saturation rate for roads in Stellenbosch, Cape Town found that the base saturation rate ranges from 1553 to 2605 veh/hr/lane with 2076veh/hr/lane being the mean value in a speed zone of 60km/hr. In the same study, the saturation flow rates from other countries were compared as shown in Table 2-5.

Table 2-5: Previous studies' saturation flow rates (Source: Bester & Meyers, 2007)

Study	Country	Mean veh/hr/lane
C J Bester and W L Meyers	RSA	2076
J Sampson	RSA	2000
Kimber et al	UK	2080
H E L Athens	Greece	1972
Hussain	Malaysia	1945
Bonneson et al	USA (Texas)	1905
Webster & Cobbe	UK	1800
Branston	UK	1778
Miller	Australia	1710
De Andrade	Brazil	1660
Shoukry & Huizayyin	Egypt	1617
Coeyman & Meely	Chile	1603
Bhattacharya & Bhattacharya	India	1232

In agreement with the findings of Bester and Meyers (2007), Sampson (2019) observe that in most cities and metropolitan areas in South Africa, drivers are aggressive, follow closely, take small gaps, and maximize capacity. Henceforth, the saturation flow is expected to easily reach a flow rate of 2 000 vehicles per hour per lane in these conditions.

2.8.5 Capacity and v/c ratio

a) Capacity

Capacity at signalized intersection is largely dependent on the concept of saturation flow and saturation flow rate (TRB,2010). Equation 5 is used in calculating capacity for each lane group:

$$C_i = S_i \left(\frac{g_i}{C}\right) \qquad \text{(Equation 5)}$$

Where:

Ci = Lane/Lane group capacity

s_i =Saturation flow rate for lane group i,

g_i/C =Effective green ratio for lane group i.

b) v/c Ratio

The ratio of flow rate to capacity (v/c), often called the volume to capacity ratio, is given the symbol X in intersection analysis and represents the degree of saturation (TRB,2010).

$$X_i = \left(\frac{V}{C}\right)i = \frac{Vi}{Si\left(\frac{gi}{C}\right)} = \frac{V_i C}{S_i g_i} \qquad \text{(Equation 6)}$$

Where:

v_i = demand flow rate for lane group

C = Cycle length

Values of equal to or less than 1.0 are said to be sustainable whereas values of more than 1.0 are said to be of excess capacity. It is also important to note that rarely do all movements at an intersection become saturated at the same time of day.

c) Critical lane groups

This is a concept used to analyse the capacity of an entire intersection. It considers only the lane groups with the highest v/s ratio for a given signal phase. Put differently, these are lane groups that demand longer green light. Each phase has one critical lane group. The critical v/c ratio for the intersection is determined by Equation 7.

$$X_c = \Sigma\left(\frac{v}{s}\right)_{ci}\left(\frac{C}{C-L}\right) \qquad \text{(Equation 7)}$$

Where L is the total lost time per cycle.

A critical v/c ratio of less than 1.0, however, does indicate that all movements in the intersection can be accommodated within the defined cycle length and phase sequence by proportionally allocating green time (TRB, 2010).

2.8.6 Determining Delay at Intersection

Values provided by the calculated delay represent the average control delay of all vehicles that arrive in the analysed period (TRB, 2010). The average control delay per vehicle for a given lane group is calculated by Equation 8.

$$d = d_1(PF)+d_2+d_3 \qquad \text{(Equation 8)}$$

Where:

d_1= uniform control delay assuming uniform arrivals (s/veh),
PF=uniform delay progression adjustment factor,
d_2= incremental delay for random arrivals and oversaturation queues, adjusted for duration of analysis period and type of signal control. This delay assumes that there is no initial queue for the lane group at the start of the analysed period (s/veh),
d_3= initial queue delay for all vehicles in the analysis period due to the initial queue at the start of the analysis period(s/veh).

2.8.7 Determining Level of Service (LOS)

The level of service of an intersection is directly related to the average control delay per vehicle. By the value of the delay, LOS can be determined in accordance with table 2-6.

Table 2-6: LOS Versus v/c Ratios for signalised intersection

Level of Service (LOS)	Average Control Delay (Seconds)
A	≤10
B	>10-20
C	>20-35
D	>35-55
E	>55-80
F	>80

2.9 TRAFFIC SIGNAL DESIGN AND OPTIMISATION PARAMETERS

The South African National Department of Transport (2012) provides recommended values to be used in traffic signal design and optimisation at intersections in South Africa as described below:

- The optimum cycle length (C) will in most cases lie in the range of 50 to 100 seconds. The minimum safe green interval for a main signal phase shall not be less than 7 seconds, but preferably not less than 11 seconds.
- The optimum cycle length (that would minimise total delay) is given by:

$$C_o = \frac{1{,}5 \cdot L + 5}{1 - \sum Y_i}$$

(Equation 9)

Where:

Co = Optimum cycle length (s)

L= Total lost time per cycle (s)

Yi = Volume/ saturation flow ratio per critical movement

- A left- or right turn phase shall not be less than 4 seconds, but preferably not less than 7 seconds.
- The yellow and all red intervals are 3 seconds and 2 seconds respectively for 60km/hr roads for intersections with gradients between +3% and-3%.

2.10 CHOICE OF PERFORMANCE MEASURES AT INTERSECTIONS

Sampson (2019) observes that the performance of an intersection can be judged in different ways and the choice of performance measure depends on the analyst. Sampson (2019) cites the following as the commonly used measures:

- Volume / Capacity ratio (V/C)
- average Delay
- maximum Delay
- total Delay

- maximum Queue
- total Queue
- Level of Service (based on V/C or delay)

In addition, Sampson (2019) also notes that if a movement is operating under heavy traffic load, the volume to capacity ratio becomes important as no movement is supposed to exceed capacity. This also ultimately means that under lighter traffic loads, the delay is more important as capacity is unlikely to be of concern. Whereas if block lengths are short, the queue may be the most important factor.

Recently, Othayoth and Rao (2019) investigated the relationship between the level of service and volume to capacity ratio at Signalized Intersections under Heterogeneous traffic. The study found that there is no one-to-one correspondence between v/c ratio and delay values. However, based on the study results, some approximate thresholds of v/c ratios were proposed linking v/c ratios to LOS as shown in Table 2-7

Table 2-7: Approximate v/c ratios for various LOS (Source: Othayoth and Rao ,2019)

Level of Service (LOS)	Average Control Delay (Seconds)
A	≤0.60
B	>0.60-85
C	>0.85-0.95
D	>95-1.05
E	>1.05-1.10
F	>1.10

Layton (1996) performed a detailed comparison of delay and capacity as performance measures of a signalised intersection. The study observed that the critical volume to capacity ratio for the intersection (Xc), can be employed to indicate the adequacy of the intersection geometry and capacity as needed for planning. The study further notes that Xc is a good indication of whether the physical geometry design features and the signal design provide sufficient capacity for the intersection.

The HCM (TRB,2010) recommends average delay per vehicle as the preferred measure of the level of services. While in light flow conditions, this is best, other measures could also be utilised to define the level of services for the reasons given above.

2.11 SUMMARY OF THE CHAPTER

This chapter has reviewed several pieces of literature on priority infrastructure and signalised intersection to shed insights into the key arguments, theories and the identified gaps in the literature about the topic. The literature review provided the reasons why minibus-taxis require priority infrastructure. The review also showed the different types of priority measures used on intersections around the world for public transport especially buses that can be considered for minibus-taxis. It provided an in-depth understanding of their applications, impacts and benefits in areas where they have been studied. The chapter also reviewed several theoretical models for analysing signalised intersections. These theories provided an in-depth understanding on their applications and successes in general, and what is required to attain these successes. This chapter has also provided a detailed analysis of the HCM theory which has been adopted in this study for performance evaluation of the MBT priority infrastructure at intersections.

3 DEVELOPMENT OF THE DESIGN STRATEGIES

3.1 INTRODUCTION

Chapter Two showed the studies that have provided different design choices for priority infrastructure at intersections for transit vehicles. It was also revealed that there is a literature gap on the approach that is used to determine the viable options of MBT priority infrastructure at a particular intersection. This chapter attempts to close this literature gap by providing the approach for determining the choices of priority infrastructure at intersections. First, it discusses the methodology used to develop design strategies for priority infrastructure. It covers the research design, data collection techniques and analytical approach. The second part of the chapter provides the findings on the first objective of this study which was centred on developing an approach for determining design strategies for the MBT priority infrastructure.

3.2 METHODOLOGY

3.2.1 Research Design

The first part of this study is grounded on a qualitative research design which follows an inductive approach to develop a procedure for determining viable priority infrastructure at signalised intersections. Qualitative research is defined as a study of phenomena in a natural setting and includes data that is in the form of words (Busetto et al., 2020). The goal of qualitative research is to discover patterns which emerge after close observation, careful documentation, and thoughtful analysis of the research topic (Maykut and Morehouse ,1994; Cavana, et al., 2001).

In addition, qualitative research is exploratory and descriptive in focus, therefore, it has an emergent design and not a fixed one which gives the researcher greater flexibility. Exploratory studies seek to explore and investigate the phenomena that have not before been researched or adequately explained (Saunders, Lewis & Thornhill, 2012). Similarly, Singh (2007) explains that exploration is a method of research investigation through observation and description of events, and then developing basic models. This means that exploratory research is the initial investigation of a theoretical or hypothetical idea of the phenomena that one observes. As the name implies, in exploratory research, the primary idea is to explore. Exploratory research therefore tends to tackle new problems on which little or no previous research has been done (Brown, 2006).

Furthermore, exploratory research design deals with exploring into the phenomenon focusing on collecting either secondary or primary data using an unstructured formal or informal procedure to interpret them (Malhotra, 2010). Accordingly, exploratory studies are often conducted using interpretive research methods. These methods include document reviews, interviewing 'experts' on the subject, conducting in-depth interviews, using focus groups, and performing case studies.

Consequently, this part of the study uses secondary data derived from existing literature to develop a conceptual framework matrix to be used for determining choices of priority infrastructure for MBTs at intersections. It uses archival research as a research strategy for data collection. This part of the study also adopts the grounded theory for data analysis. Grounded theory is a methodological approach that begins with data observations and looks for patterns, themes, or common categories (Omona, 2013). On the other hand, an archival research strategy makes use of administrative records and documents as the principal source of data (Saunders, Lewis & Thornhill, 2012). Thus, through these strategies, a holistic understanding of the phenomenon was developed to bring to the surface answers to the first research question outlined in Chapter One.

3.2.2 Data collection

This part of the research study used secondary data collected through document reviews. Data from design guidelines and literature on the best practices were utilised to inform choices of potential design treatments at an intersection. This data was in the form of key design principles obtained from documents and literature mainly from the United States of America where similar design strategies have been successfully implemented. Data for design treatments was also collected from documents including local guidelines proposing design strategies for priority infrastructure at intersections. Priority infrastructure associated with transit vehicles were selected for design review through document analysis. These included queue-jumping lane, queue bypass lane, transit signal priority, shared transit lane and far side transit stop facilities. The purpose of the evaluation was to document key geometric, and traffic needs that inform the design of the selected priority infrastructure at an intersection.

This part of the study also used secondary data gathered through a document review of guidelines for designing existing intersections. This data for the existing intersection geometry was collected from design manuals of urban roads in South Africa. This data was in the form of typical details of intersection layout plans from major metropolitan road authorities and regulators. These authorities included the National Department of Transport (NDoT), City of Johannesburg (CoJ), Joburg Roads Authority (JRA), City of Tshwane (CoT), Gauteng Department of Roads and Transport (GDoRT) and the Western Cape Department of Transport and Public Works. The purpose of this evaluation was to assess the suitability of the existing intersections to accommodate the design strategies for priority infrastructure as determined from document analysis of the best practices.

3.2.3 Data Analysis

Data for this part of the research was analysed qualitatively and involved the following steps:

- Searching, filtering and selecting relevant documents for the research question. This phase involved selecting documents associated with best practices of design principles of priority infrastructure at intersections.
- Examining the context surrounding the documents to determine the key design elements.
- Listing down all relevant themes in the form of excepts selected from the documents. This is where relevant design principles for each priority infrastructure were examined and summarised in a table format.
- Sorting and creating relevant design themes. This involved evaluating and revising themes to ensure that each theme was distinct. Themes similar to each other were merged. This helped to ensure that themes accurately reflected what was evident in the data set as a whole (Braun & Clarke, 2006).
- After themes for design principles were created, document analysis was also performed on relevant documents regarding the existing prevailing geometry of intersections for urban roads in South Africa. The idea was to evaluate intersection layouts and identify the existing elements that could be utilised for the design of the MBT priority infrastructure.
- Themes for these prevailing geometric elements were identified and used as design principles of MBT priority intersections.
- A framework matrix was then used to determine the feasible combination of design treatments and geometric elements of existing layouts as determined from the document analyses.

Figure 3.1 provides a summary of the document analysis procedure that was utilised.

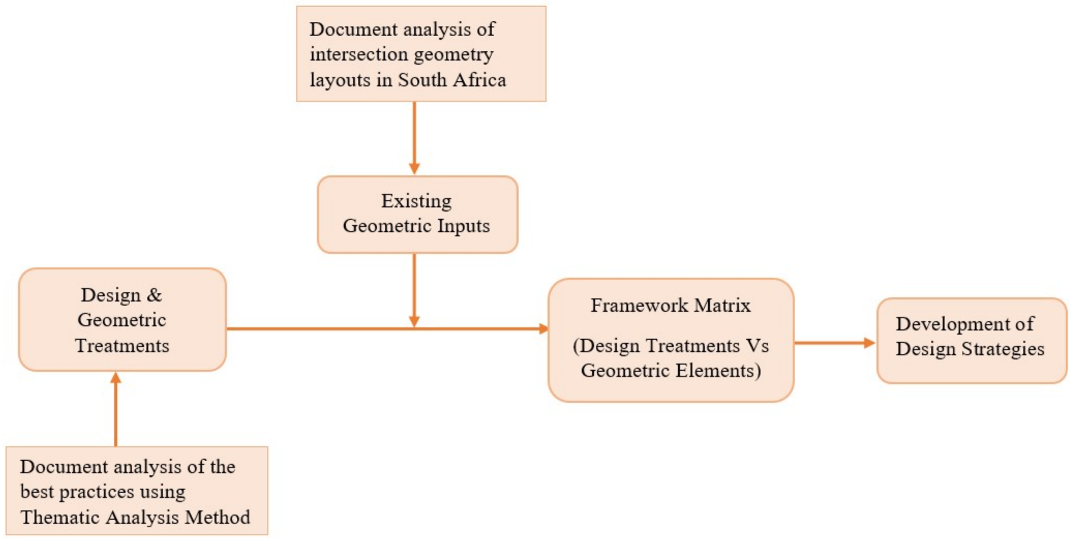


Figure 3-1: Data Analysis Procedure

3.2.4 Research questions

This chapter was designed to answer the following key research questions:

- What are some of the design treatments associated with priority infrastructure at intersections based on a literature review of the 'best' practices?
- Could the current geometric designs of intersections for urban roads in South Africa be utilised to provide choices of MBT priority infrastructure?

3.3 LIMITATIONS

This part of the study was limited to a review of design strategies that provide priority to through traffic movement of MBT at intersections other than the turning traffic movements of MBT. In addition, this part of the study was also limited to a review of four-legged at grade intersection layouts on a typical urban or city road because they are the most used intersections in city or urban roads but with the most crashes due to a large number of vehicle conflict points (Sampson, 2019).

3.4 DESIGN STRATEGIES

This section presents the findings from the data analysis conducted to answer the research objective which was to develop an approach for selecting choices of design strategies for priority infrastructure at intersections. A document review on priority infrastructure was conducted to determine the relationship between design treatments for priority infrastructure and the intersection geometry. This section is divided into three distinct sub-sections. It begins by presenting design treatment summarises identified from a document review of the priority infrastructure at an intersection. Thereafter, it presents different existing geometric elements for urban intersections in South Africa as reviewed from local design guidelines. Finally, this section used themes obtained from these two- subsections to create a framework matrix for use in developing design strategies for priority infrastructure.

3.4.1 Determination of Geometric and Traffic Treatments

This part of the study involved listing all design treatments which are associated with the common priority infrastructure reviewed from documents regarding the best design practices. These design treatments include geometric treatments and traffic related treatments. Geometric treatments include treatments that require additional space to be implemented such as the addition of new turning lanes, new receiving lanes, bus stops and short bypass lanes (SCAG, 2022 and NACTO, 2016). On the other hand, traffic related treatments do not require space but rather involve the conversion of existing road markings or the addition of traffic signals.

Table 3-1 provides summarised notes of key design treatments associated with priority infrastructure at signalised intersections for transit vehicles as reviewed from both local and international literature. This table contains three columns namely, the name of the priority infrastructure, a summary of key design requirements and a typical example of the place or country where the priority infrastructure has been implemented. The table was developed on the basis of information presented in Chapter 2 which includes Table 2-1.

Table 3-1: Summarised Notes of Design Treatments at Intersections.

Name of Priority Infrastructure	Summary of Key Design Treatments Requirements	Place or Country of application [2]
Queue Jumping Lane	• Designed by converting a dedicated left-turn lane (based on SA lane conversion) to also allow priority vehicles to proceed ahead of general traffic. • Uses Priority Traffic Signal (PTS) to call for an early green phase. <u>Comment on design needs</u>: Key design needs therefore entail presence or provision of an auxiliary left- turning lane (based on SA lane conversion) and priority traffic signals to allow queue bypass. Space availability is also considered where provision of new lane is required.	USA (West Valley City, State of Utah) Canada (Calgary, Alberta; M86, New York City)

[2] See Appendix A for photographs and sources of information.

Name of Priority Infrastructure	Summary of Key Design Treatments Requirements	Place or Country of application [2]
Queue Bypass Lane (Transit Approach Lane)	• Designed by converting one of the approach lanes into a short, dedicated lane for priority vehicles on the approach side. • A queue bypass extends to the other end of a signalised intersection hence does not use priority signals. <u>Key design needs:</u> The design needs therefore entail presence or provision of a short dedicated nearside lane with corresponding receiving lane. However, this study preferred to use internal lane as bypass lane to prevent potential traffic movement conflicts between through transits and left turning vehicles if a nearside lane is used as a bypass lane. Space availability is also considered where provision of new lane is required.	USA (Stockton St, San Francisco, USA; Washington Street in Chicago; SBS86, New York City)
Transit Signal Priority (TSP)	• Designed by allowing transit vehicles to communicate with signals to: – extend green lights, – end red lights early, – add a transit-only signal phase. • Transit vehicles use either a dedicated lane or bypass lane. • It acts as a complimentary design treatment for a bypass or dedicated lane. <u>Key design needs:</u> The design needs therefore entail presence or provision of extended green lights or transit only signal phase for transit vehicles on a dedicated or bypass lane. The design therefore requires an immediate receiving lane for the dedicated lane. Space availability is also considered where provision of new elements is involved.	USA (New York City; Seattle, Washington)
Shared Transit Lane	• Designed by converting left-turning lane to accommodate moderate volume of left turn movements and through movements for transit vehicles. It does not use priority signals unless preferred to do so. • Alternatively, also designed by converting through movement on a shared nearside lane to ONLY allow through movements for transit vehicles (Transit Approach Lane). • Used at locations where left-turning vehicles can typically clear through the intersection quickly. <u>Key design needs</u>: The design needs therefore entail presence or provision of a shared lane that can be converted to a shared transit lane. It also entails presence or provision of an auxiliary left turning lane that can be converted to a shared transit lane. However, the latter treatments require addition of a receiving lane. Space availability is also considered where provision of new elements is involved.	Spain (Barcelona) USA (West Valley City, State of Utah)

Name of Priority Infrastructure	Summary of Key Design Treatments Requirements	Place or Country of application [2]
Far-side bus or minibus-taxi stops.	• They are located after an intersection, allowing the bus to travel through the intersection before stopping to load and unload passengers. Key design needs: The design needs entail availability of space to accommodate addition of a bus-stop and availability of pedestrian traffic for pick-up. This treatment is therefore considered as complimentary to other design treatments especially those associated with the queue-jumping lanes.	• USA (Los Angeles, California)

Interpretation

The description of design treatment requirements in Table 3-1 is based on South African lane configurations. The results in the table have shown that traffic movements such as left- turning and straight movements could be converted to provide feasible priority infrastructure at intersections. These design treatments form part of traffic related treatments. For example, to design queue jumping lanes, the existing left-turn lane for all vehicles could be converted to a shared lane consisting of a combination of turning movement for all vehicles plus through movement for priority vehicles. Similarly, to develop shared transit lanes, the existing shared traffic movements for all vehicles could be converted to shared traffic movements of turning traffic movements for all traffic vehicles plus through movement for priority vehicles only.

For geometric treatments, the design treatments at intersections include the addition of new lanes, new transit priority signals and the addition of far-side stops where spaces for these geometric improvements are available. Typical examples of such design treatments include queue bypass lanes which require multiple lanes with a new dedicated lane for MBT, transit signal priority which requires the installation of new traffic, as well as the addition of far-side stops. All these geometric treatments require the availability of space.

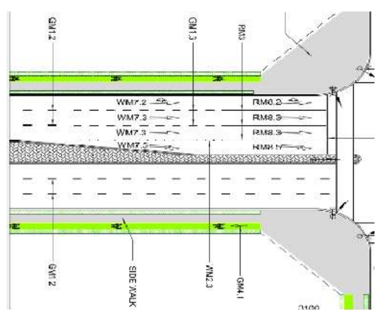
3.4.2 Determination of Existing Geometric Inputs

Literature review has revealed that priority designs at intersections are influenced by the existing geometric layout of the intersection (SCAG, 2022; NACTO 2016). These studies have shown that the availability of auxiliary lanes such as nearside (deceleration) lanes and receiving or acceleration lanes play a vital role in determining the choices of design treatments of the priority infrastructure. The document analysis for this sub-section involved evaluating the geometric elements of existing intersections that have the potential to influence the choice of priority infrastructure as reviewed from the previous sub-section. Such geometric elements include but are not limited to the availability of turning or auxiliary lanes, straight (through) lanes, receiving lanes, and bus/minibus-taxi stops. The evaluation also included presenting the type of traffic movements associated with the priority facilities. Traffic movements such as straight, left turns (LT), and a shared movements (through and turning movements) were included in the analysis.

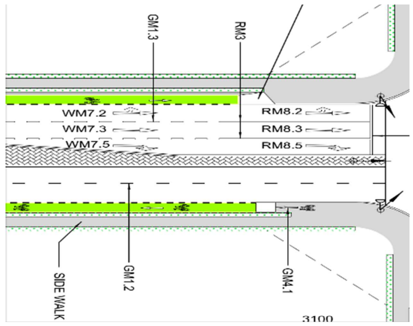
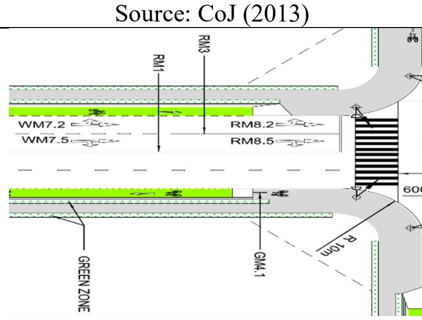
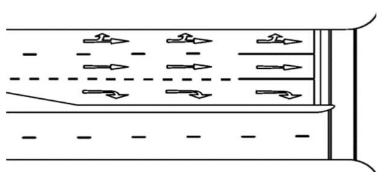
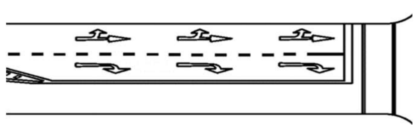
Excerpts of four-legged signalised intersection layouts were selected from South African design guidelines of the urban intersections. The existing geometric conditions of approaching lanes and exit lanes were evaluated for design suitability to accommodate the priority infrastructure.

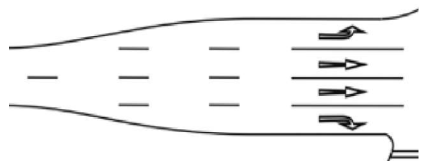
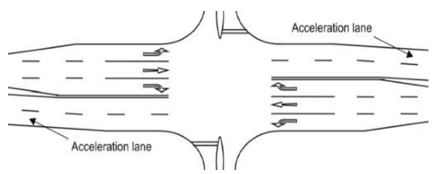
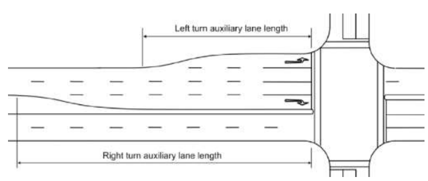
Table 3-2 presents the evaluation of existing geometric elements for signalised intersections for the purpose of identifying suitable design strategies for MBT priority infrastructure. The focus of this evaluation is to highlight geometric elements present at the existing intersections that could be utilised for the purpose of designing priority infrastructure. These geometric elements include the presence of nearside and immediate inside approaching lanes, the presence of receiving lanes, the availability of space for upgrades and availability of bus or minibus-taxi stops on near or far side end of the intersections.

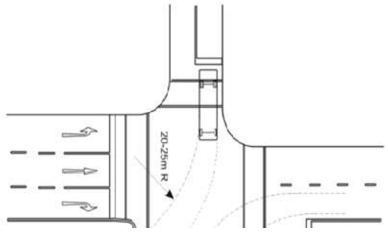
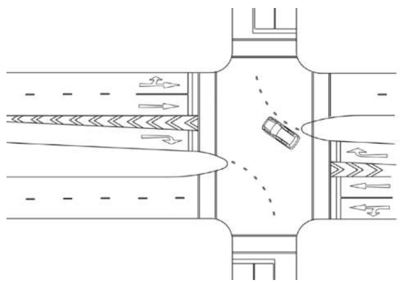
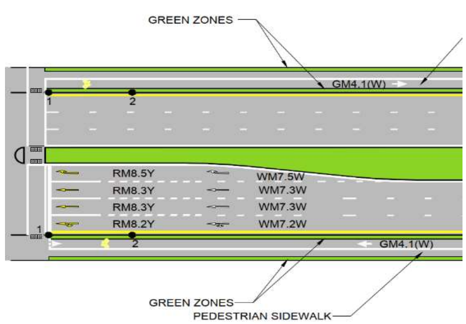
Table 3-2: Summary of Existing Geometric Inputs

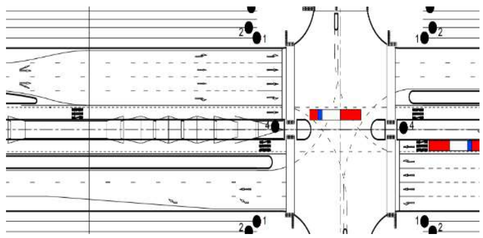
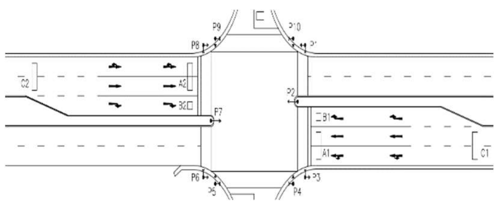
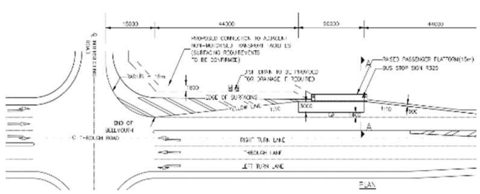
Approach/Exit Layout	**Key Geometric and Traffic Elements Present/Absent**
L1  Source: CoJ (2013)	L1 consists of four approaching lanes: two exclusive straight lanes, one shared traffic movement lane and one exclusive right turning lane. The geometric themes associated with this layout therefore include: • Presence of one full-length nearside lane with shared traffic movement. • Presence of two full length inside[3] lanes with straight traffic movement. • Absence of space for additional lanes as part of the upgrades on the approach.

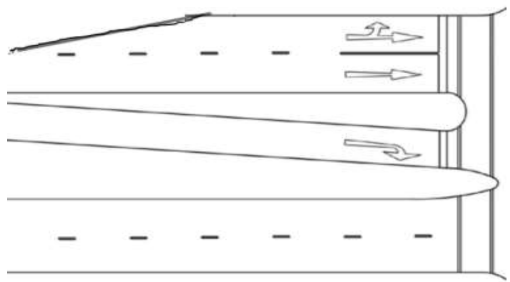
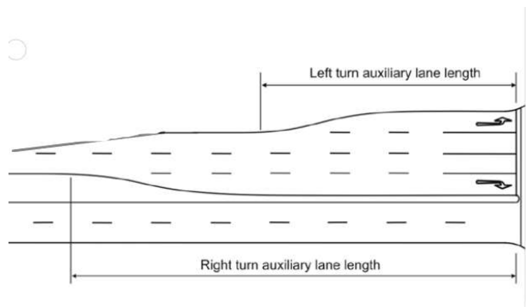
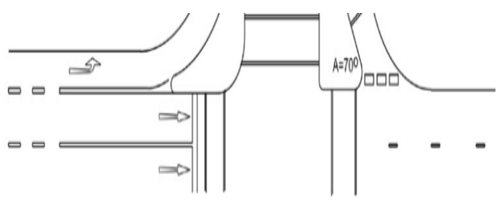
[3] means through traffic lane which is next to nearside lane at intersections where two or more through traffic movements are present.

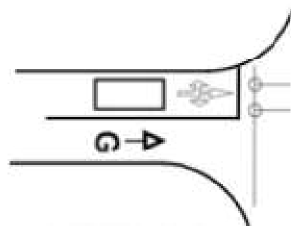
Approach/Exit Layout	**Key Geometric and Traffic Elements Present/Absent**
L2  Source: CoJ (2013)	L2 consists of three approaching lanes: one exclusive straight lane, one shared traffic lane and one exclusive right turning lane. The geometric themes associated with this layout therefore include: • Presence of one full-length nearside lane with shared traffic movement on the approach. • Absence of space for upgrades available on the approach.
L3  Source: CoJ (2013)	L3 consists of two full approaching lanes: two full lanes with shared traffic movements. Possible geometric themes associated with this layout therefore include: • Presence of one full length nearside lane with shared traffic movement • Absence of space for upgrades on the approach.
L4  Source: COTO (2014)	L4 consists of three approaching lanes: one full lanes with shared traffic movements, one full lane with exclusive straight traffic movement, and one short right turning lane. Possible geometric themes associated with this layout therefore include: • Presence of full-length nearside lane with shared traffic movement. • Presence of space for upgrades on the approach
L5  Source: COTO (2014)	L5 consists of two approaching lanes: one full lane with shared traffic movements, and one short lane with exclusive right-turning traffic movement. Possible geometric themes associated with this layout therefore include: • Presence of full-length nearside lane with shared traffic movement. • Presence of space for upgrades on the approach

Approach/Exit Layout	Key Geometric and Traffic Elements Present/Absent
<u>L6</u>  Source: NDoT (2015)	L6 consists of four approaching lanes: one short lane with exclusive left turning traffic, two full lanes with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of short auxiliary nearside lane with LT traffic movement. • Presence of full length inside lane with straight traffic movement on the approach • Presence of space for upgrades on the approach
<u>L7</u>  Source: NDoT (2015)	L7 consists of three approaching lanes and two receiving lanes: one short lane with exclusive left turning traffic, one full lane with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of auxiliary nearside lane with LT traffic movement. • Presence of receiving lane for nearside lane present • Presence of space for upgrades on the approach • Presence of space for upgrades on the exit
<u>L8</u>  Source: NDoT (2015)	L8 consists of four approaching lanes: one short lane with exclusive left turning traffic, two full lanes with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of auxiliary nearside lane with LT traffic movement. • Absence of receiving lane for nearside lane • Presence of full length inside lane with straight traffic movement on the approach • Presence of space for upgrades on the approach

Approach/Exit Layout	Key Geometric and Traffic Elements Present/Absent
L9  Source: COTO (2014)	L9 consists of three approaching lanes: one full lane with exclusive left turning traffic, one full lane with straight movement traffic, and one full lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of auxiliary nearside lane with LT traffic movement. • Absence of receiving lane present for the nearside lane • Presence of space for upgrades on the approach
L10  Source: COTO (2014)	L10 consists of two approaching lanes: one full lane with shared traffic, one full lane with straight movement traffic, and one full lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of full length shared nearside lane. • Presence of receiving lane present for nearside lane • Presence of space for upgrades on the approach
L11  Source: JRA (2015)	L11 consists of four approaching lanes: one full lane with shared traffic, two full lanes with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of full-length nearside lane with shared traffic movement. • Presence of full length inside lane with straight traffic movement. • Absence of space for upgrades on the approach

Approach/Exit Layout	Key Geometric and Traffic Elements Present/Absent
L12  Source: JRA (2015)	L12 consists of four approaching lanes: one short lane with exclusive left turning lane, two full lanes with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of auxiliary nearside LT lane. • Presence of receiving lane for nearside auxiliary lane. • Presence of space on both approach and exit for upgrades. • Presence of full length inside lane with straight traffic movement on the approach available • Absence of bus/minibus-taxi stop
L13  Source: DoTPW (2019)	L13 consists of three approaching lanes: one full lane with shared traffic movements, one full lane with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of full length nearside shared traffic lane. • Presence of receiving lane for nearside auxiliary lane. • Presence of space available on both approach and exit for upgrades. • Absence of bus/minibus-taxi stop
L14  Source: DoTPW (2019)	L14 consists of three approaching lanes: one short lane with left turning traffic movements, one full lane with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Absence of receiving lane for nearside auxiliary lane • Availability of space on the exit side for upgrades • Presence of bus/minibus-taxi stop on the far side exit end

Approach/Exit Layout	**Key Geometric and Traffic Elements Present/Absent**
L15  Source: COTO (2014)	L15 consists of three approaching lanes: one auxiliary lane with left and straight traffic movement, one full lane with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of auxiliary lane with combined left and straight traffic movements • Availability of space for upgrades on the approach and exit. • Absence of bus/minibus-taxi stop on the far side exit end
L16  Source: COTO (2014)	L16 consists of four approaching lanes: one auxiliary lane with left turning traffic movement, two full lane with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of auxiliary lane with left traffic movement • Presence of auxiliary lane with through traffic movement • Availability of space for upgrades on the approach and exit. • Absence of bus/minibus-taxi stop on the far side exit end
L17  Source: COTO (2014)	L17 consists of four approaching lanes: one auxiliary lane with left turning slip lane, two full lane with straight movement traffic, and one short lane with exclusive right-turning traffic. Possible geometric themes associated with this layout therefore include: • Presence of auxiliary lanes with straight (ALL) traffic movement. • Receiving lanes for through traffic • Availability of space for upgrades on the approach and exit. • Absence of bus/minibus-taxi stop on the far side exit end

Approach/Exit Layout	Key Geometric and Traffic Elements Present/Absent
L18  Source: COTO (2014)	L18 consists of single approaching lane left through and right turning movements. Possible geometric themes associated with this layout therefore include: • Presence of single approaching lane with LT/T/RT (ALL) traffic movement. • Receiving lanes for through traffic • Availability of space for upgrades on the approach and exit. • Absence of bus/minibus-taxi stop on the far side exit end

Interpretation

Table 3-2 has presented various layouts of four legged intersections found within the cities of South Africa as extracted from design guidelines of urban roads. The layouts show multiple approaching lanes ranging from two to four lanes. In addition, most of these intersection layouts have auxiliary lanes currently being used for both exclusive turning movements as well as shared traffic movements. Literature has shown that intersections with multiple number of approaching lanes has potential of providing geometric treatments for priority infrastructure at intersections. Therefore, the table has also presented potential themes associated with the geometric treatments for priority infrastructure at intersections created from the current layouts of urban roads. These themes are in form of geometric features available or absent at the existing intersections. These are elements that have an impact on priority vehicles and could be used for developing the choices of MBT priority infrastructure at intersection. The themes include the presence of nearside auxiliary lanes which could be converted to form a shared MBT lane, the presence of receiving lanes which could be used for priority vehicles, the presence of multiple approach and exit lanes to accommodate dedicated MBT lanes as well as availability of space to accommodate any geometric upgrades for MBT infrastructure.

3.4.3 Matrix Framework Analysis

The evaluation involved cross-classification of the design treatments with the existing layout elements to determine which design treatments could be applicable to which combination of geometric elements. These combinations formed possible design strategies for priority infrastructure. A framework matrix was used to conduct this evaluation. Table 3-3 provides a framework matrix that has been developed for determining the design strategies for priority infrastructure at a four-way signalised intersection. The table rows show geometric elements for existing intersection layouts which could be utilised for MBT priority designs. The columns under the header, 'Design Treatments' provide the design treatments associated with each geometric element. Colour codes and numbers were used to represent different combinations of geometric elements and design treatments. The green colour represents combinations that do not require geometric improvement, whereas the light-blue colour represents combinations that require geometric improvement. The yellow colour represents an additional or complimentary treatments that other studies have found to reduce congestion at intersections. Finally, the red colour represents non-feasible combinations. The feasible combinations were used to develop the design strategies for the priority infrastructure for minibus-taxis at intersections. These additional treatments include presence or absence of minibus-taxi or bus stops. A legend is also given providing the description of the colour codes and acronyms used in the framework matrix.

Table 3-3: The matrix table for identifying design strategies for priority infrastructure for Minibus-Taxis at four-way signalised intersections.

			Design Treatments															
	Groups of Geometric Elements	Full Description of the Elements	Convert LT (ALL) to shared LT (ALL) plus straight (MBT)	Convert LT (ALL) to straight (MBT)	Convert LT (ALL) to straight (ALL)	Convert LT/T (ALL) lane to shared straight (MBT) plus LT (ALL)	Convert LT/T (ALL) lane to straight (MBT) lane	Convert LT/T(ALL) lane to straight (ALL)	Convert straight (ALL) lane to a straight (MBT) lane	Convert LT/T/RT lane to T/RT	Use existing receiving lane for straight (MBT) traffic	Add TSP for MBT	Add new receiving lane for straight (MBT) traffic	Add new receiving lane for straight (ALL) traffic	Add new nearside auxiliary lane for straight MBT	Add new nearside shared lane for straight (MBT) and LT (ALL)	Add new nearside auxiliary lane for LT traffic	Add MBT stop on far side (Exit lanes)
Existing Geometric Elements Associated with the targeted MBT corridor	Auxiliary nearside lanes (approach)	Auxiliary nearside lane with LT traffic movement	1	2														
		Auxiliary nearside lane with LT/T traffic movement				3	4											
	Full length nearside lanes (approach)	Full length nearside lane with LT traffic movement	5	6	7													
		Full length nearside lane with LT/T traffic movement				8	9	10										
	Auxiliary inside lanes (approach)	Auxiliary inside lane with straight traffic movement							11									
	Full length inside lanes (approach)	Full length inside lane with straight traffic movement							12									
	Receiving lanes	Receiving lane present for LT/T nearside traffic									13							
		Receiving lane present for inside straight (ALL) traffic									14							
		No receiving lane present for nearside traffic										15	16	17				
	Space for upgrades	Space for upgrades available for receiving lanes											18	19				
		Space for upgrades available for approaching lanes													20	21	22	
	MBT Stops	MBT stop on far sides not available																23
	Full Length Single Lane (approach)	Single Lane approach with LT/T/RT movements								24								

COLOUR CODES	DESCRIPTION OF COLOUR CODES
	Combination Not Feasible
	Design Strategies which only require modification of road traffic markings Hence Not Dependent on Space Availability
	Design strategies which require modification of road traffic markings plus addition of geometric elements hence dependent on space availability
	Complimentary treatments that are slightly off the intersections but have been found to ease congestion at intersections
	Design strategies that require addition of geometric elements to the traffic modifications

ACRONYMS	MEANING
LT	Left Turning
L/LT	Shared or Mixed Traffic (Left and Through Turning)
LT/T/RT	Single Lane Approach
MBT	Minibus-Taxis
TSP	Transit Signal Priority

3.4.4 Interpretation of the Framework Matrix

The framework matrix provides different combinations that could be used to form design strategies for MBT priority infrastructure at signalised intersections. To use the matrix framework, geometric elements, or conditions for the existing intersections along the selected corridor for priority infrastructure are evaluated against the design treatments forming the numbered combinations of colour codes. Along each row these feasible combinations are not mutually exclusive. This means that the feasible design strategies for MBT priority infrastructure could be formed by combining multiple combinations that are not mutually exclusive. For instance, intersection layouts L1, L2, L4, and L13 had the following MBT geometric conditions: the auxiliary nearside lanes with LT/T traffic movements and receiving lanes for LT/T movements without MBT stops on the far sides of the intersections. To form the design strategy for a shared MBT lane, first, combination '3' could be used to change auxiliary LT lanes on the approach and allow straight MBT traffic use the lanes as well. In addition, the design strategy could also include combination '13' which allows the provision of the receiving lanes for the straight MBT traffic. This design strategy could be completed by combination '23' which requires the addition of a MBT bus stop on the far side of the intersections. The full design strategy for the shared MBT lane on these intersections therefore could be formed by combinations '3', '13' and '23' (3-13-23).

The matrix table also provides two main categories of combinations. First, combinations that do not require geometric improvements categorised as Design Strategy 1 (DS1) and those that require geometric improvements categorised as Design Strategy 2 (DS2). Table 3-4 summarises the combinations associated with these design strategies that could be used to develop different choices of priority infrastructure at a four-way intersection. The table should be read together with the matrix framework. The table also provides typical examples of intersections layout plan where the design strategies could apply.

Table 3-4: **Summary of design strategies developed using the framework matrix analysis.**

Proposed Names of MBT Priority Infrastructure	**Design Strategy 1**		**Design Strategy 2**	
	Design Strategies (Without Geometric Improvements)	**Examples of Typical Layout Application**	**Design Strategies (With Geometric Improvements)**	**Examples of Typical Layout Application**
Shared MBT Lane	• 3-13-23	• L15	• 1-16-18-23	• L6, L8
	• 8-13-23	• L1, L2, L4, L10, L11, L13	• 5-16-18-23 • 7-17-19-21-16-18-23	• L9
	• 3-23	• L7, L14	• 10-21-16-18	• L5
	•	•	• 24-16-18-21	• L18
Dedicated MBT Lane	• 11-14-23	• L16	• 2-16-18-22-23	• L15
	• 12-14-23	• L6, L7, L8, L11, L12	• 6-16-18-22-23	• L1, L2, L3, L4, L10, L11, L13
Transit Signal Priority with MBT dedicated lane	• 11-14-15-23	• L15	• 2-16-18-22-13-23	• L7, L14
	• 12-14-15-23	• L1, L2, L4, L10, L11, L13	• 6-16-18-22-13-23	• L1, L6, L11, L13
MBT Queue Jumping Lane	• 1-15-23	• L6, L8, L16	15-24-21	• L9, L18
	• 5-15-23	• L9	• 7-17-19-22-23	• L9

3.5 SUMMARY

This chapter has developed design strategies for the priority infrastructure for minibus-taxis at signalised intersections. A qualitative data method was used utilising document analysis techniques to develop a framework matrix. The framework matrix was used to show relationship between the geometric elements and the design treatments of priority infrastructure. To do this, themes from the best practices on priority infrastructure were combined with themes for geometric elements of existing intersections in South Africa. The matrix table used colour codes and numbering to show feasible design strategies for priority infrastructure at an intersection. The study developed two categories of design strategies for each of the priority infrastructure using the framework matrix. First, design strategies that only require repurposing of the existing intersection. Second, design strategies that require major geometric improvements. The findings of this chapter have provided key knowledge that could be used for determining choices of priority infrastructure at intersections.

4 PERFORMANCE EVALUATION

4.1 INTRODUCTION

This chapter discusses the methodology and results for the second research question provided in chapter one namely: examine the impacts of priority interventions on the performance of signalized intersections. The first part of the chapter presents the research methodology and involves the research design, data collection methods, data analysis, limitations and sampling and data analysis procedure. The second part presents the results of the matrix evaluation and performance evaluation of two design strategies. The chapter ends with a section for results discussions and a summary of the chapter.

4.2 METHODOLOGY

The subsequent sections provide details of the methodologies employed in this study.

4.2.1 Research approach

Both qualitative and quantitative analysis methods were utilised to answer the research question on performance evaluation. To evaluate the intersections, first, a qualitative approach was adopted utilising the framework matrix evaluation method developed in Chapter Three. The analysis began with selection of four isolated intersections for this evaluation. Unlike intersections along one corridor, isolated intersections provide opportunity to utilise and compare a variety of geometric and traffic conditions from different corridors. For this study, the isolated intersections were also used to identify prevailing ranges of MBT traffic volumes on different corridors. Overall, the purpose of this intersection evaluation was to determine the feasible design strategies associated with the sampled intersections. It was also done to demonstrate how the matrix framework developed in Chapter Three could be applied. The evaluation also involved providing a high-level assessment on impacts of the design strategies on safety, traffic operation, and costs. After the evaluation of four intersections, two most feasible design strategies were selected for performance evaluation on two isolated intersections. The performance evaluation aimed at evaluating the impacts of the two selected design strategies on capacity and delay using real world traffic count data.

The traffic count data on the final two intersections was collected and analysed quantitatively to determine peak hour volumes. The Highway Capacity Manual (HCM) (2010) method was the preferred method of this performance analysis. HCM method is an analytical method developed by the American Transportation Research Board which provides a methodology that analyses the performance of intersections in terms of the capacity, delay, and level of service

(LOS) (TRB, 2010). In South Africa, the HCM method remains the most preferred and recommended analytical method for analysing the performance of intersections (COTO, 2014). This method is effective for this analysis results because it does not require the complex exercise of using large factors required in operational simulation analyses associated with calibration and validation processes (Pretoria et al, 2004). Essentially, this means the method is not prone to systematic errors common with simulation software.

The HCM method used the peak hour traffic volumes to compare the performance of intersections ‘with’ and ‘without’ priority infrastructure. This means the performance of the existing (without priority) intersections was compared to the performance of intersections after implementing the proposed design strategies. The performance measures used for this evaluation included v/c ratio, average vehicle delay and storage lengths. The purpose of this analysis was to determine the impact of the selected design strategies on the overall performance of the intersections. Figure 4-1 shows the data analysis procedure that was used to perform capacity evaluation.

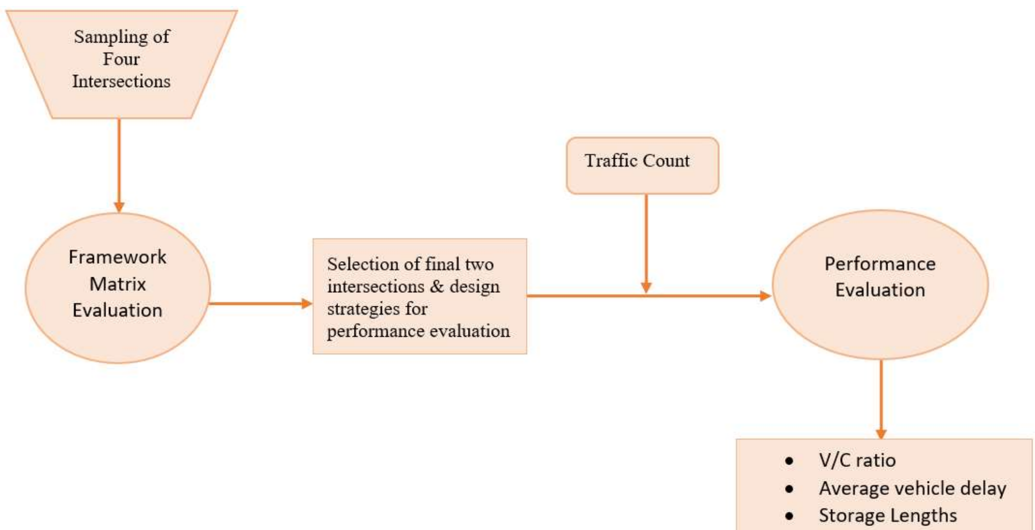


Figure 4-1: Data Analysis Procedure for the Performance Evaluation

4.2.2 Site Selection

Four intersections were purposively selected in the City of Tshwane (CoT) to use in evaluating and selecting final design strategies for the MBT priority infrastructure for the purpose of performance analysis. The CoT municipality is in the northern part of Gauteng province, and it is the largest of the three metropolitan municipalities in this province. Gauteng province, where CoT is located, is the province with the highest levels (45.7%) of minibus-taxis use (Stats SA, 2021). The city registered a population of about three million two hundred seventy-five thousand people (3,275,000) people in the year 2016 and is also the fourth biggest municipality in size out of the eight metropolitan municipalities in South Africa (Cooperative Governance and Traditional Affairs, 2016). The city covers an area of up to 6345 square kilometres. According to Stats SA (2022), about 40% of households who owned a minibus-taxis were from Gauteng (19.7%) and Limpopo (20.6%) provinces. Subsequently, Limpopo province is one of the three provinces that borders the CoT. All these factors make the CoT an ideal area for this study. Figure 4-2 shows the locality plan of the city of Tshwane metropolitan municipality. The figure shows the location of CoT in Gauteng province and its location in relation to the bordering provinces.

Figure 4-2: Locality Plan of the City of Tshwane (Source: www.tshwane.gov.za)

Like in many South African cities, the CoT has a high demand for public transport which includes minibus-taxis. The city is therefore not spared from problems of traffic congestion which becomes worse during peak periods. For instance, between 2019 and 2020, about 50% of peak-period trips for educational related purposes were done by minibus-taxis (GDoRT, 2020). Figure 4-3 shows a road network available in the city of Tshwane which was targeted for operational improvements. The figure shows different classes of roads available in the city of Tshwane (CoT). The road categories range from Class 1 to Class 6 and the classes are mainly subdivided into mobility and access roads (COTO, 2012). Class 1 roads are high mobility roads which connect provinces and predominantly owned by the South African National Roads Agency (SANRAL). On the other hand, Class 2 and Class 3 roads are also mobility roads which usually connect metropolitan cities within a province and are predominantly owned by provincial governments and in this case, the Gauteng province. Some Class 4, Class 5 and sometimes Class 6 roads are predominantly access roads connecting local municipalities within a metropolitan city or local municipality. These roads are predominantly owned by local or metropolitan municipalities and in this case, the CoT metropolitan municipality. This study targeted roads with high MBTs trips which happened to be of the categories ranging from Class 2, Class 3 and Class 4a urban roads with intersection spacing between 150m and 600m.

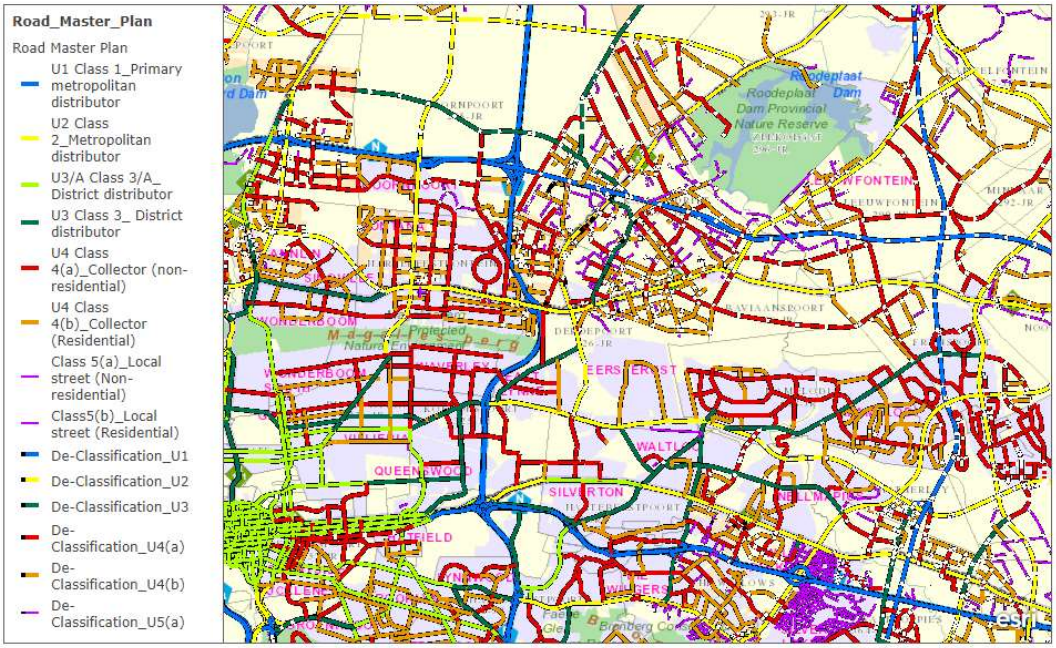


Figure 4-3: Road Network in the City of Tshwane (Source: www.tshwane.gov.za)

4.2.3 Sampling of Intersections

Non-probability convenience and purposive sampling were used to sample the intersections. The purposive sampling is a sampling technique used when it is not possible to select participants randomly. In purposive sampling, the sample is chosen or selected based on characteristics, the aims of the research or the researcher's knowledge of the population (Babbie and Mouton, 2006). On the other hand, the non-probability-based nature of the sampling method has a tendency to introduce bias into the evaluation sample. This potential bias has been mitigated as much as possible by using a set of criteria that was established as described below:

a) Type of intersection: From the start, this study only targeted signalised intersections because of the flexibility of signalised intersections to provide the highest capacity when compared with other intersection controls (Sampson, 2019). In addition, traffic signals can provide priority to the targeted traffic without major geometric upgrades or modifications at intersections. Such priority infrastructure includes a queue-jumping lane that requires the installation of a priority traffic signal at an intersection.

b) Fully traffic signal-controlled corridors: The study selected intersections with target corridors that were fully controlled by traffic signals. This means the targeted corridors where all approaching lanes are fully controlled by the existing traffic signals.

c) Straight (through) MBT traffic movements: the study targeted intersections with a higher traffic volume of through movements than the turning movements for both minibus taxis and other types of vehicles. This is because all the priority design strategies that were considered in this study are designed to improve traffic operations mainly for through or straight movement traffic at intersections.

d) Viability of existing intersections to be repurposed: The study targeted intersections that could easily be repurposed to accommodate priority treatments. To achieve this criterion, the study selected intersections with more than one approaching lane in the city of Tshwane as well as intersections with available space to accommodate geometric upgrades.

e) Type of Roads and range of MBT traffic trip counts

   The study targeted urban roads (Class 2, Class 3 and Class 4a) of design speed of between 70km/hr and 80km/hr with medium to high volume of through (straight) movement MBTs. These were roads which were found to have recorded a high number of daily trip counts following analysis of minibus-taxi trip counts from GPS tracking data (De Beer, 2023). These are corridors which registered daily trip counts of between 340 trips and 2440 trips. Figure 4-4 shows the minibus-taxi trip count classification following a survey conducted on trips made by minibus-taxis in the city of Tshwane using GPS tracking data (De Beer, 2023). The minibus-taxi GPS data used for analysis was acquired from iSAHA, a company that specialises in transportation data collection and analysis in South Africa.

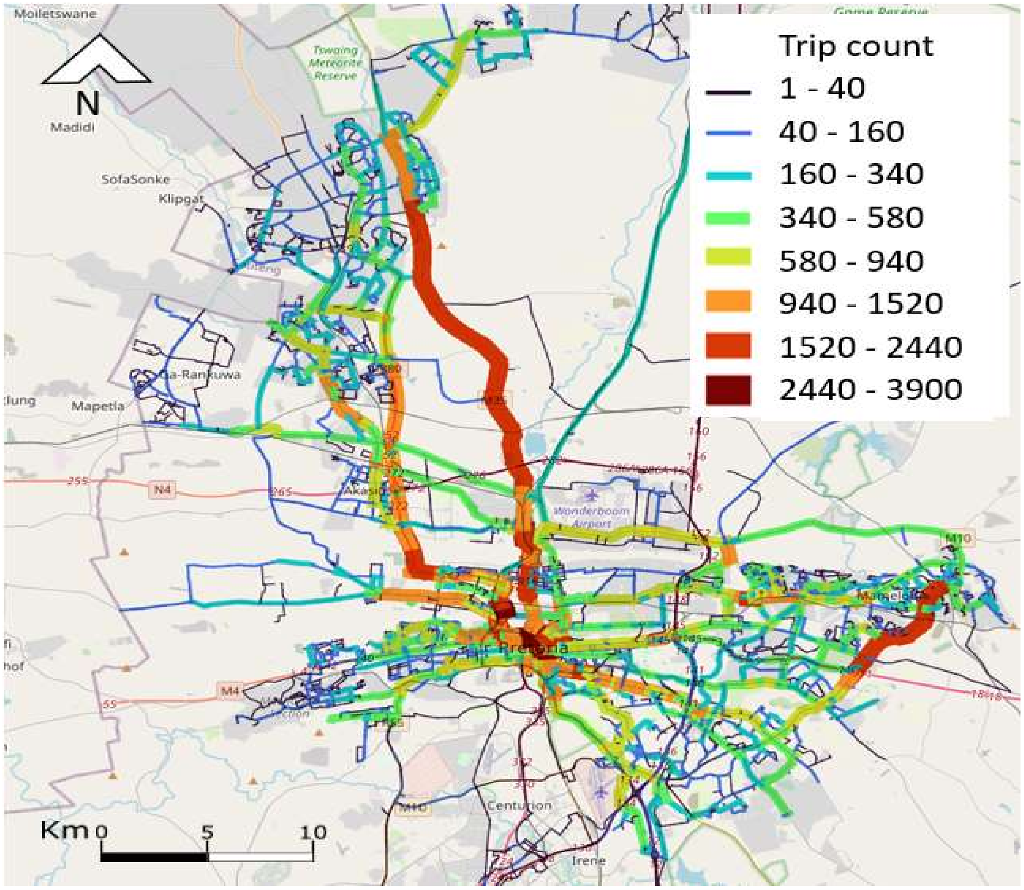


Figure 4-4: Minibus-taxi daily trip count classification for the Tshwane Metropolitan Municipality (Source: De Beer, 2023).

4.2.4 Data Collection Methods

Both primary and secondary data were collected to achieve the objectives of this chapter. Primary data was collected through field observation and traffic counts, whereas secondary data was collected through document review.

<u>Document Review</u>

Several documents were reviewed prior to site visits to get an in-depth understanding of prevailing geometric layouts of intersections in the city of Tshwane. Document review also helped in understanding trends of peak traffic flows in the city. Examples of these reviews included a review and analysis of the minibus-taxi daily trip count from the city of Tshwane municipality, previous traffic studies in the city of Tshwane, legislation, policies, and strategies. The review also included design manuals for intersections to understand the recommended design procedures for intersections.

Observation

Site visits were conducted to examine the functioning of the intersections in their normal day-to-day setting. This was done to also gain an in-depth understanding of the operations of the MBTs on the sampled sites. Site visits were also conducted to collect geometric data on the study intersections. These visits also provided an opportunity to observe and record the nature of the traffic flow and turning movements for the study intersections. The pre-traffic count study visits also helped to ascertain and determine the peak morning and afternoon hours as determined from the document reviews.

Traffic Counts

The purpose of traffic counts was to collect peak-hour traffic volumes of MBTs and other vehicles to use as input in evaluating performance of the sampled intersections. Data was collected through classified traffic counts at all four sampled intersections. In terms of traffic flows, the most critical periods occur during the weekdays AM and PM hours when Home to School, Home to Work, School to Home and Work to Home trips are at peak. Subsequently, ninety-minute video traffic counts were conducted on Tuesdays and Wednesdays for both morning and afternoon peak hours when the traffic flows were at a maximum. Video recording cameras were used to conduct traffic counts due to their flexibility in recording and keeping raw visuals for offsite in-depth processing and analysis. Physical dimensions for features such as lane width, lengths of turning lanes, turning radii and queue lengths were also recorded and verified during the period of traffic count. Traffic signal timing settings for all phases at the study intersections were also collected.

4.2.5 Data Collection Challenges and Limitations

To improve the validity, credibility and reliability of the data, efforts were made to address the challenges faced during data collection and processing. Table 4-1 illustrates the challenges and limitations that were encountered during the data collection phase and the mitigation strategies employed to counter these challenges. The biggest challenge during data collection was finding the correct position to mount the video camera so that it shows the entire intersection. Much as there were efforts to get the best angle, however, in some situations where this could not be achieved, the manual count was adopted to record traffic queues beyond camera visibility

Table 4-1: Data collection challenges and limitations

Challenge/ Limitation	Mitigation Strategy
• Poor camera visibility: Difficulties in finding a proper angle to view the entire intersection including queue lengths in all four approaches of an intersection	• The video Camera was mounted on the roof of the car to get a good camera elevation and angle. • Queues not visible to camera were counted manually.
• Skipping count during data recording due to blind spots created by heavy vehicles when they are turning inside the intersection.	• Video camera data provided opportunity to count the data repeatedly.
• Some traffic vehicles were using shoulders or spaces available on the intersections to skip the queues	• Vehicles using these spaces were added to their respective traffic movements during data compilation and analysis.

4.2.6 Current Road Network Conditions

This section provides detailed road conditions, road classes as well as road ownership of the sampled sites. The purpose is to describe the key characteristics and differences of the four selected intersections for evaluation for priority infrastructure. The section begins with the locality plan of the sampled intersections as depicted in Figure 4-5. The figure shows the location of the four sampled intersections with reference to other streets. The four intersections sampled are signalised and fall under the jurisdictions of the City of Tshwane municipality and the Gauteng Department of Roads and Transport. These roads under this study fall within Class 2, Class 3 to Class 4a categories with road reserve widths of between 20m to 40m. The roads also fall within the speed limits of between 60km/hr and 80km/hr. The figure shows the locality plan of the four intersections sampled and these include the following:

- J1: Garsfontein Road & Solomon Mahlangu Drive
- J2: Lynwood Road & Jan Shoba Street
- J3: Paul Kruger Street & Green Street
- J4: Solomon Mahlangu Drive & Bronkhorstspruit Road

In addition, Table 4-2 provides the description of the current condition of each of the four intersections. The table gives brief summarises in terms of intersection geometric layout, traffic conditions as well as road classes. At the end of each summary, a recommendation is provided on the ideal corridor for the MBT priority infrastructure.

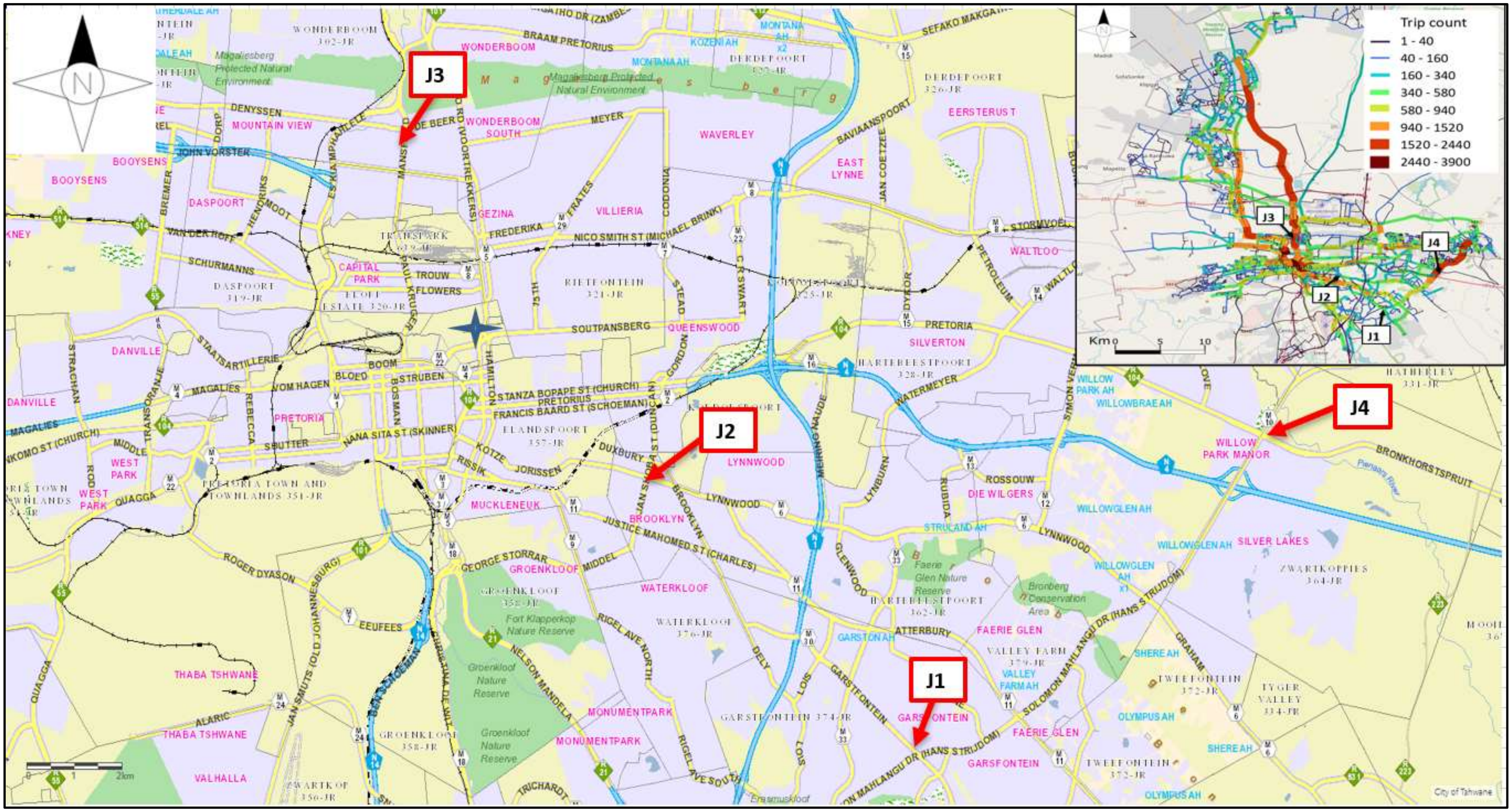


Figure 4-5: The Locality Plan of the Four Intersections in the City of Tshwane

Table 4-2 Current Road Network Conditions

Existing Intersection	Description of Geometric and Traffic Conditions
J1: Garsfontein Road & Solomon Mahlangu Drive	
 	• **Geometry:** ✓ 4-Way Signalised intersection. ✓ Both are urban Class 2 roads owned by Gauteng Department of Roads and Transport (GDoRT) ✓ All approaching lanes on Garsfontein road (North/ South bound) are fully controlled by signals. • **Traffic Volumes:** ✓ Both roads fall within minibus taxi daily trip count classification of between 340 trip counts and 580 trip counts. ✓ There is higher volume of peak hour minibus-taxi traffic on Garsfontein compared to Solomon Mahlangu (source: observation) ✓ Higher volumes of **through** minibus-taxi traffic compared to turning minibus-taxi traffic on Garsfontein (source: observation) ✓ Longer Q-Length on Garsfontein than on Solomon Mahlangu (source: observation) • **Recommendation:** The minibus-Taxi Priority facility should be located on Garsfontein corridor
J2: Lynwood Road & Jan Shoba Street	
 	• **Geometry:** ✓ 4-Way Signalised intersection. ✓ Both are urban Class 2 roads owned by City of Tshwane (CoT) ✓ All approaching lanes are fully controlled by signals. • **Traffic Volumes:** ✓ Both roads fall within minibus-taxi daily trip count classification of between 340 trip counts and 580 trips counts. ✓ There is higher volume of peak hour minibus-taxi traffic on Lynwood compared to Jan Shoba (source: observation) ✓ There is higher volume of through minibus-taxi traffic compared to turning minibus-taxi traffic on Lynwood (source: observation). ✓ Longer Q-Length on Lynwood than Jan Shoba (source: observation). **Recommendation:** The minibus-taxi Priority facility should be located on Lynwood corridor

Existing Intersection	Description of Geometric and Traffic Conditions
J3: Paul Kruger Street & Green Street	
	• **Geometry:** ✓ 4-Way Signalised intersection. ✓ Urban Class 3 (Kruger st) and Class 4a (Green st). Both roads owned by CoT ✓ All approaching lanes are fully controlled by signals. • **Traffic Volumes:** ✓ Kruger street falls within the minibus-taxi daily trip count classification of between has average daily MBT trip counts of between 940 trip counts and 1520 trips counts. ✓ Green street falls within the minibus-taxi daily trip count classification of between 340 trip counts and 580 trip counts. ✓ There is higher volume of peak hour MBT traffic on Paul Kruger compared to Green St (Observation) ✓ There is a higher volume of through minibus-taxi traffic as compared to turning minibus- taxi traffic on Paul Kruger (Observation). ✓ Longer Q-Length on Paul Kruger than Green during peak hour (Observation) **Recommendation:** The minibus-taxi priority facility should be located on Paul Kruger corridor
J4: Solomon Mahlangu Drive & Bronkhorstspruit Road	
	• **Geometry:** ✓ 4-Way Signalised intersection. ✓ Both are class 2 roads. ✓ Solomon Mahlangu is owned GDoRT while Bronkhorspruit is owned by CoT ✓ All approaches have exclusive slip lanes but through movements are fully controlled by the traffic lights. • **Traffic Volumes:** ✓ Both roads fall within the minibus-taxi daily trip count classification of between 940 trip counts and 1520 trip counts. ✓ There is a higher volumes of peak hour minibus-taxi traffic on Solomon Malhangu as compared to Bronkhorstspruit (Field observation) ✓ There is a higher volume of through minibus-taxi traffic compared to turning minibus taxi traffic on Solomon Malhangu (field observation) ✓ Longer Q-Length on Solomon Malhangu is longer than Bronkhorstspruit during peak hour (field observation) **Recommendation:** The minibus-taxi priority facility should be located on Solomon Malhangu

4.3 FRAMEWORK MATRIX EVALUATION OF THE SAMPLED INTERSECTION

This section provides the evaluation outcome of all the four intersections. For each of the four sampled intersections, at least two design strategies were identified and evaluated using the framework matrix evaluation technique developed in Chapter Three. One of the two strategies involved repurposing or reconfiguration of the existing intersections, whereas the other involved geometric improvements such as the addition or construction of new geometric elements. In addition, a high-level evaluation was performed related to other design considerations that have an influence on feasibility such as safety, operations, and cost of the priority infrastructure based on engineering practical experience. In addition, road markings and signages were also included in the proposed design strategies. The detailed drawings of these design strategies have been provided in Appendix B. Results from this evaluation were used in determining the choices of the final intersections and design strategies selected for further performance evaluation.

Table 4-3 provides a summary of the proposed design strategies for the four intersections using the framework matrix criteria established in Chapter Three. All existing geometric and traffic elements for each intersection along the approach of the targeted MBT corridors have been listed. For each intersection approach, the two most viable design strategies were proposed as design option one and design option two. The Design Option One comprises of design strategies that only require modification of existing geometric and traffic elements without the addition of new geometric features such as new lanes. On the other hand, the Design Option Two consists of design strategies that require the addition of new geometric elements such as new lanes. In this regard, the table shows that the design strategies for queue-jumping lanes were proposed on north bound approach for J1 intersection as Design Option One while design strategies for shared MBT lanes were proposed as Design Option Two on south bound approach. On the other hand, for both approaches of J2, J3 and J4 intersections, design strategies for shared MBT lanes and dedicated MBT lanes were proposed as option one and option two respectively.

The table has also provided a high-level summary of safety, traffic operation and costs of the proposed design strategies. At the end of the analysis of each design strategy, feasible design strategies are indicated with a GREEN colour. Design strategies that are not feasible strategies are indicated with RED colour whereas partially feasible design strategies are indicated with DARK GREEN colour.

The safety and operational analysis of the design strategies relied on views from the subject matter expert. The subject matter expert suggested that queue jumping lanes may not be a feasible design strategy considering the aggressive behaviour of MBT drivers. To expand on this point, there are concerns of MBT traffic build-up at the end of the GREEN interval that would block other traffic types, and this could easily render the whole intersection not operational and unsafe. It was also noted that the absence of receiving lanes on queue jumping lanes strategies could also further render the intersection more dangerous especially at night when traffic volumes are low while vehicles travel at higher speeds. In addition, the changing of lanes for traffic approaching the intersection was also considered to partially compromise the safety and traffic flow at the intersection for all the design strategies.

In terms of traffic operation, it was found that the provision of shared MBT lanes may not be feasible in situations where there are only two approaching lanes on the target MBT corridor. To emphasise this point, the subject matter expert suggested that if the nearside lane is converted to a shared MBT lane, the remaining lane would need to take both through and right turning traffic. This could worsen the levels of services of through traffic in situations where right turning traffic do not have the right of way concurrently with the through traffic. In addition, it was also suggested that converting a slip lane into a shared MBT lane would not be a good option. This is so because most slip lanes by design are already associated with high volumes of peak hour traffic hence converting to a shared MBT lane would worsen levels of service with the additional MBT traffic.

With regards to cost analysis, the analysis assumed that all design strategies that require the addition of geometric and traffic elements (option two) were considered to have high initial implementation costs. However, for these design strategies to be considered, there is a need for the existing intersections to have enough space available to accommodate the upgrades. On the other hand, design strategies that do not require new geometric elements like new lanes were considered to be less costly. These options therefore were found to be viable regardless of the availability of space.

The table has also provided proposals with respect to road markings and road signs for the priority lanes. For instance, a proposal was made to paint all priority lanes with RED colour plus GREEN road markings (arrows). In addition, a proposal has also been put forward to provide a road sign associated with priority lanes at a recommended sight distance before the start of the priority lanes. Lastly, other studies have shown that far-side stops could improve operation of public transport at intersections hence a proposal has been made in the evaluation to include MBT stops at the far side (exit) of each intersection along the targeted MBT corridor.

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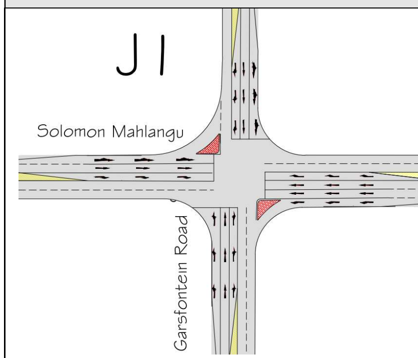
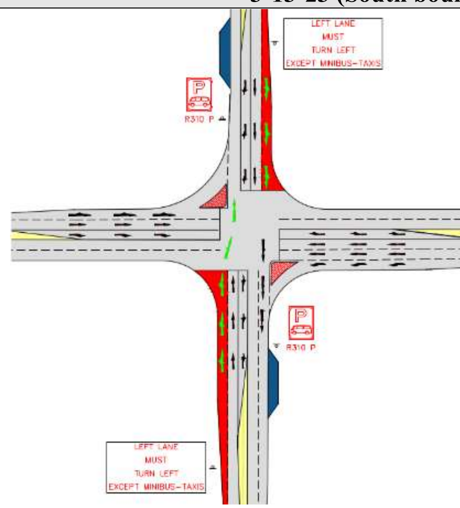
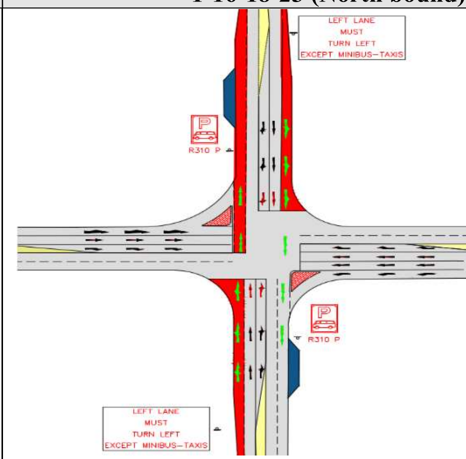
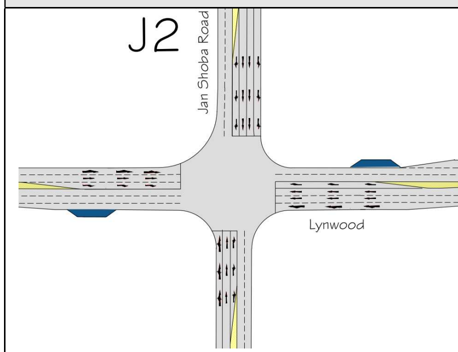
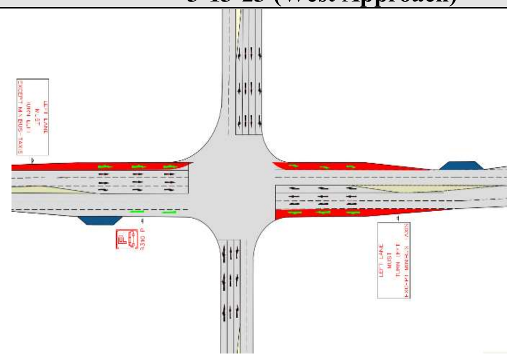
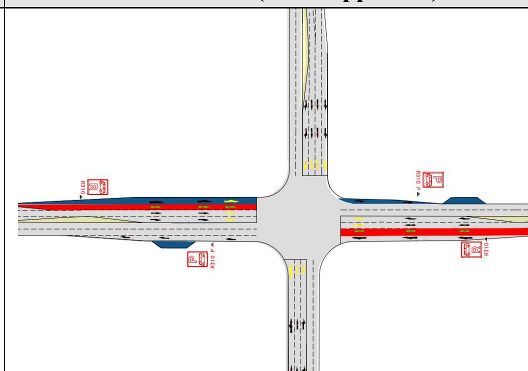
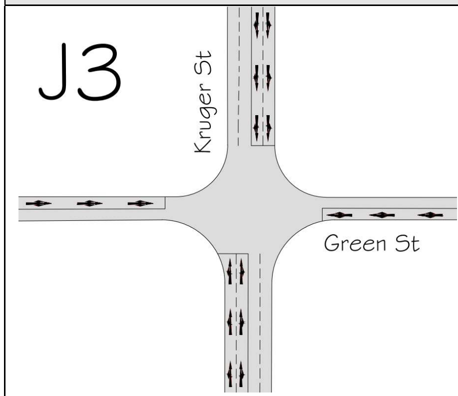
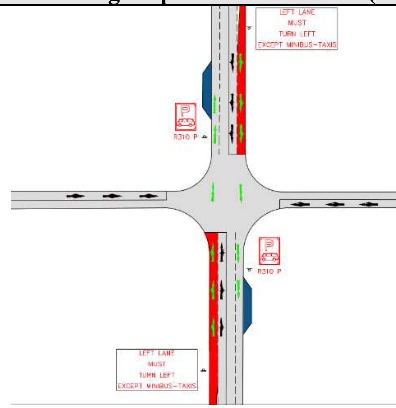
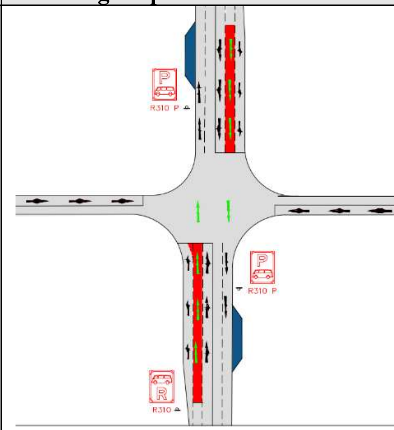
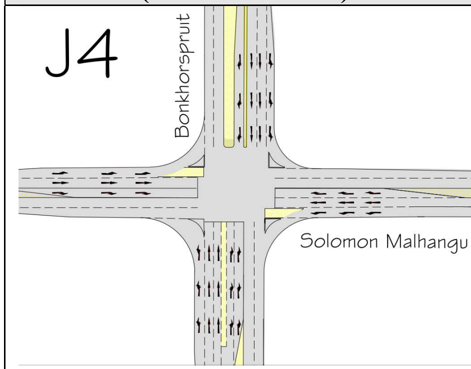
Table 4-3: Evaluation of the proposed design strategies for the priority infrastructure

Existing Layout & Geometric Conditions (See also Table 4-2)	**Evaluation of Design Strategies for Garsfontein & Solomon Mahlangu (J1) (See also Appendix B)**	
	Design Option One: 1-15-23 (North Bound); 3-13-23 (South bound)	**Design Option Two: 3-13-23 (south bound); 1-16-18-23 (North bound)**
 **Name of Junction:** • Garsfontein & Solomon Mahlangu (J1) **Target corridor for priority facilities**: • Garsfontein (N-S/S-N Corridor) **Geometric & Traffic Conditions on targeted corridor** • Exclusive auxiliary left turning (LT) lane on north bound approach. • Shared or mixed (turning and through) auxiliary lane on south bound approach. • Receiving lane for south bound approach present • Unlimited space available for geometric improvement • No MBT stops available on both ends of intersections	 **Comments on Design Option One:** • <u>NB Approach</u>: queue jump lane with early GREEN priority traffic signal (PTS) • <u>SB Approach:</u> a shared MBT lane. **<u>Proposed Road Signs & Markings:</u>** • Paint priority lanes with a different colour, preferably RED or yellow. • Use GREEN arrows for traffic movement on priority lanes. • Provide a signage for turning traffic EXCEPT minibus-taxis at recommended sight distance before the start of the auxiliary left turning lanes. **<u>Design Concerns</u>**: • Queue jump could lead to high chance of traffic build up in cases of minibus-taxis arriving at the end of early GREEN interval. • Queue jump lane without receiving or accelerating lane on the exit end could be unsafe at night for minibus-taxis considering aggressive behaviour of minibus-taxi drivers. **COMMENT: Partially feasible due to highlighted safety concerns related to aggressive driving behaviours of MBT drivers.**	 **Comments on Design Option Two:** • <u>NB Approach</u>: shared MBT lane • <u>SB Approach:</u> Shared MBT Lane. **<u>Road Signs & Markings:</u>** • Paint priority lanes with a different colour, preferably RED or yellow. • Use GREEN arrows for traffic movement on priority lanes. • Provide a signage for turning traffic EXCEPT minibus-taxis at recommended sight distance before the start of the priority lanes. **<u>Design Concerns</u>**: • Higher initial implementation cost than Option One due to additional receiving lane and physical reconfiguration of traffic signal poles. **COMMENT: Highly feasible**

Existing Layout & Geometric Conditions (See Table 4-2)	**Evaluation of Design Strategies for Lynwood & Jan Shoba (J2) (See also Appendix B)**	
	Design Option One: 7-17-19-21-16-18-23 (East Approach); 3-13-23 (West Approach)	**Design Option Two: 6-16-18-22-23 (East Approach); 12-14-23 (West Approach)**
 **Name of Junction:** • Lynwood & Jan Shoba (J2) **Target corridor for priority facilities:** • Lynwood (W-E/E-W corridor) **Geometric & Traffic Conditions on targeted corridor** • Shared nearside auxiliary lane present on West bound approach. • Shared nearside auxiliary lane present on East bound approach. • Straight inside lane present on the West bound approach. • Receiving lanes present for both approaches. • Space available for geometric improvements on both approach and exit sides. • Minibus-taxi stops present on both far sides of exit lanes	 **Comments on Design Option One:** • <u>EB Approach</u>: shared MBT lane on a new LT auxiliary lane • <u>WB Approach:</u> shared MBT lane on existing turning lane **<u>Road Signs & Markings:</u>** • Paint priority lanes with a different colour, preferably RED or yellow. • Use GREEN arrows for traffic movement on priority lanes. • Provide a signage for reserved minibus-taxi lanes at recommended sight distance before the start of the priority lanes. **<u>Design Concern</u>:** • Higher initial cost due to additional lane on EB approach. • Requires space for additional lane. **<u>Alternative Design 1b:</u>** 8-13-23 (EB Approach); 3-13-23 (WB Approach) • Shared MBT lane on both approaches using existing nearside lanes (without geometric improvements) <u>Design Concern</u>: On EB approach, with one remaining through lane, the level of service could be tremendously reduced. **COMMENT: This option is more feasible than the alternative option. The alternative option is partially feasible due to capacity constraints described above.**	 **Comments on Design Option Two:** • <u>EB Approach</u>: dedicated MBT lane with new addition of auxiliary LT lane. <u>WB Approach:</u> • MBT dedicated lane. **<u>Road Signs & Markings:</u>** • Paint priority lanes with a different colour, preferably RED or yellow. • Use GREEN arrows for traffic movement on priority lanes. • Provide a signage for reserved minibus-taxi lanes at recommended sight distance before the start of the priority lanes. **<u>Design Concern:</u>** • Higher initial cost due to new additional lane. • Requires space for upgrades. **COMMENT: Feasible**

Existing Layout & Geometric Conditions (See also Table 4-2)	Description of Design Strategies for Paul Kruger & Green (J3) (See also Appendix B)	
	Design Option One: 8-13-23 (Both Approaches)	**Design Option Two: 2-16-18-22-23 (Both Approaches)**
 **Name of Junction:** • Paul Kruger & Green (J3) **Target corridor for priority facilities:** • Kruger street (N-S/S-N corridor) **Geometric & Traffic Conditions on targeted corridor** • Full length shared nearside lanes present on both approaches. • No space available for upgrades • Two lanes on both approaches • Right turns shared with through lanes. • No auxiliary lanes	 **Comments on the Proposed Design Option One:** • <u>NB approach</u>: a shared MBT lane. • <u>SB</u>: a shared MBT lane. **<u>Proposed Road Signs & Markings:</u>** • Paint priority lanes with a different colour, preferably RED or yellow. • Use GREEN arrows for traffic movement on priority lanes. • Provide a signage for the priority lanes at recommended sight distance before the start of the priority lanes. **<u>Design Concerns</u>**: • Possibility of queue build-up on both approaches due to blockages by right turning traffic. **<u>Alternative Design:</u>** 7-17-19-21-16-18-23 (Both Approaches) • Add new auxiliary shared MBT by lane with receiving lane on both approaches allowing through MBTs and turning traffic on both approaches. However, this approach requires space which is not available on this intersection. **COMMENT: Not feasible due to potential traffic blockages that could arise as a result of right turning traffic. The alternative approach also not feasible due to lack of space for geometric upgrade**	 **Comments on the Proposed Design Option Two:** • <u>NB approach:</u> provision of a dedicated MBT lanes and addition of a new LT lane • <u>SB approach:</u> provision of a dedicated MBT lanes and addition of a new LT lane • **<u>Road Signs & Markings:</u>** • Paint priority lanes with a different colour, preferably RED or yellow. • Use GREEN arrows for traffic movement on priority lanes. • Provide a signage for the priority lanes at recommended sight distance before the start of the priority lanes. **<u>Design Concern</u>**: • More costly than Option One due to new LT lanes and physical reconfiguration of signal poles. • Ideally this can only work where there is enough space to accommodate extra lanes. • Could be a viable option on roads with wider road reserves. **COMMENT: Not feasible because the space is currently not available to accommodate these upgrades.**

<table>
<tr><th rowspan="2">Existing Layout & Geometric Conditions (See also Table 4-2)</th><th colspan="2">Description of Design Strategies for Solomon Mahlangu & Bronkhorstspruit (J4) (See also Appendix B)</th></tr>
<tr><th>Design Option One: 3-13-23 (Both Approaches)</th><th>Design Option Two: 2-16-18-22-23 (Both Approaches)</th></tr>
<tr>
<td>


Name of Junction:
Solomon Mahlangu & Bronkhorstspruit (J4)
Target corridor for priority facilities:
Solomon Mahlangu (W-E/E-W Corridor)
Geometric & Traffic Conditions on targeted corridor

One approaching through lane available.
Additional space available for upgrades
No minibus-taxis stops on both far sides ends of the intersection.
Slip lane present on both approaches.
Islands with no kerbs present on both approaches separating slip lanes from other approaching lanes.

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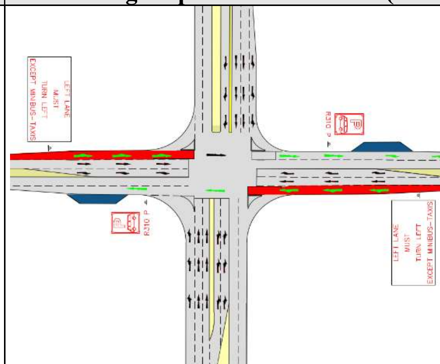
<td>


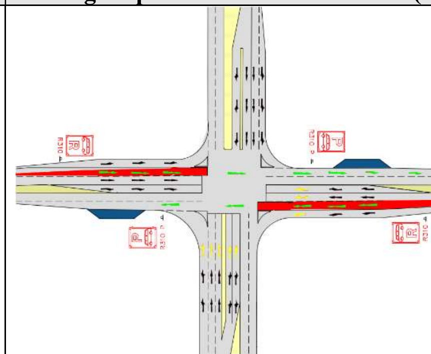
Comments Proposed Design Option One:

<u>EB approach</u>: provision of a shared MBT lane
<u>WB approach</u>: provision of a shared MBT lane

<u>Proposed Road Signs & Markings:</u>
Paint priority lanes with a different colour, preferably RED or yellow.
Use GREEN arrows for traffic movement on priority lanes.
Provide a signage for the priority lanes at recommended sight distance before the start of the priority lanes.

Design Concerns:

Slip lanes are associated with heavy traffic hence converting to shared traffic with minibus-taxis could worsen levels of service.
Could be feasible in situation where traffic volumes are low.

COMMENT: Not feasible due to a high volume of traffic associated with slip lanes on this intersection
</td>
<td>


Comments on the Proposed Design Option Two:

<u>EB approach:</u> Provision of dedicated MBT lane with addition of a new slip lane.
<u>WB approach:</u>

Provision of dedicated MBT lane with addition of a new slip lane
<u>Proposed Road Signs & Markings:</u>

Paint priority lanes with a different colour, preferably RED or yellow.
Use GREEN arrows for traffic movement on priority lanes.
Provide a signage for the priority lanes at recommended sight distance before the start of the priority lanes.

Design Concerns:

High initial costs due to new slip lanes.
Requires more space.

COMMENT: This option could be more feasible than option 1
</td>
</tr>
</table>

4.4 FINAL SELECTION OF DESIGN STRATEGIES AND INTERSECTIONS FOR PERFORMANCE EVALUATION

Table 4-3 has provided a detailed evaluation of intersections to determine choices of priority infrastructure at intersections using the framework matrix developed in Chapter Three. At least two design strategies were proposed for each of the four intersections. The table also shows that not all design strategies were feasible. This is because some of the design strategies are associated with a reduced level of services and compromised safety.

The evaluation also established that design strategies that were considered not feasible included J1 (option one), J3 (options one and two) and J4 (option one). Design strategies for intersections J3 were considered not feasible due to safety concerns and lack of space on the existing intersections to accommodate geometric improvements. Option one (queue jumping lanes) for intersection J1 was considered partially feasible because of safety concerns that could rise due to the aggressive driving behaviours associated with MBT drivers. For instance, the analysis showed that there are concerns that the queue-jumping lanes without receiving lanes could lead to a high chance of traffic blockages in cases of MBTs arriving just at the end of the early GREEN interval. This could render the intersection unsafe, and the conditions could worsen at night considering the aggressive behaviours of many MBT drivers. Lastly, Option One for intersection J4 was considered not feasible because of the possibility of poor traffic operation that could arise due to a high volume of traffic associated with slip lanes at this intersection.

On the other hand, the evaluation found that feasible design strategies included J1 (Option Two), J2 (Options One and Two) and J4 (option two). J1 (option two) design involved the provision of a shared MBT lane on existing auxiliary lanes. J2 (Option One) involved the provision of shared MBT lane on the new auxiliary lane while J2 (Option Two) involved the provision of a short dedicated MBT lane. And finally, J4 (Option Two) also involved provision of short dedicated MBT lane.

Further analysis on the feasible design options showed that geometric layouts for J2 (Option one) and J4 (option one) were similar. In the same way, layouts for J2 (Option Two) and J4 (Option two) were also found to be identical. For the purpose of performance evaluation, J2 was therefore selected ahead of J4 since J2 was associated with two feasible design strategies unlike J4.

In conclusion, two intersections are selected for further performance evaluation, and these include J1 and J2. The study has also selected two choices of feasible design strategies for performance analysis. The two design strategies included a shared MBT lane on J1 and a dedicated MBT lane on J2.

Table 4-4 provides a summary of the final selected choices of intersections and design strategies for performance evaluation. The analysis has shown that J1 was found to be more feasible for the implementation of shared MBT lanes (Option Two) on both approaches. On the other hand, the J2 intersection was recommended for dedicated MBT lanes (Option Two) on both approaches.

Table 4-4: Final Selected Intersections and Design Strategies

Name of Intersection	Proposed Design Strategies	Name of the priority infrastructure
J1	– 3-13-23 (SB Approach). – 1-16-18-23 (NB Approach)	Shared MBT Lanes
J2	– 6-16-18-22-23 (EB Approach) – 12-14-23 (WB Approach)	Dedicated MBT Lanes

4.5 PEAK HOUR TRAFFIC DETERMINATION

The subsequent sections provide the steps and analyses that were conducted to determine the peak hour traffic volumes for performance evaluation.

4.5.1 Traffic Count Analysis

Traffic data was collected through video recording cameras on the two selected intersections. An initial observation on a day prior to the traffic count and a review of previous traffic studies within the city helped to establish the peak hour period for the study sites. Most often traffic pattern in the morning is different from the pattern in the afternoon hours due to Home to Work/School-based trips and School/Work to Home based trips. To capture these trends, traffic counts were conducted in the morning and afternoon for both intersections. After identifying the critical AM and PM peak hours, a ninety-minute video traffic count was therefore conducted on Tuesday and Wednesday for both J1 and J2 respectively. The timeline of the fieldwork is depicted in the Figure 4-6.

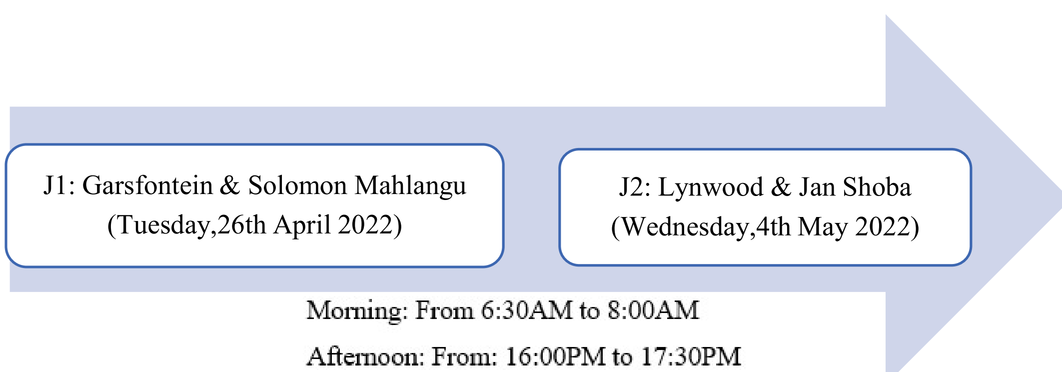


Figure 4-6: Timeline for Traffic Count

Traffic count data was organised, compiled, and analysed in a Microsoft Excel sheet before performing capacity analysis. Traffic was divided into three categories comprising heavy vehicles, light vehicles and minibus-taxis. First, the heavy vehicles were made up of at least one heavy axle and/or any vehicle which is principally designed or adapted for the conveyance of persons exceeding sixteen (DoT, 2006). The second category comprised of small cars including passenger cars, pickups but excluding minibus-taxis. The third category comprised of minibus-taxis (Figure 4.7) as targeted vehicles of this study. Through observation, pedestrian volumes were very low and found not to greatly interfere with the intersection operation hence not considered in this research study. The peak hour volume was determined empirically by an iterative process. This involved selecting the highest traffic volume from the sum of any four adjacent fifteen-minute traffic volumes. The highest 15-minute flow within period of consideration (90 minutes) was multiplied by four to determine the peak demand flow. To determine peak hour factor, peak demand flow was divided by peak hour volume as shown in Equation 3. Appendix C provides detailed results of peak hour traffic data for J1 and J2. The subsequent sections provide a detailed analysis of the traffic count data for the two intersections.

Figure 4-7: Photograph of a minibus-taxi in a stream of mixed traffic on J2

4.5.2 AM and PM Peak Hour Traffic Data for J1

Figure 4-8 is an extract of data from Appendix C and presents the AM and PM peak hour traffic volumes which were used for the performance analysis of the J1 intersection. The morning peak hour was from 06:30 AM to 7:30 AM, whereas the afternoon peak hour was from 16:30 PM to 17:30 PM. Traffic volumes for three categories of vehicles were counted separately and these include light vehicles, MBTs and heavy vehicles. All traffic movements for the approaches of the intersection are shown by numbers from 1 to 12. Data for MBTs and heavy vehicles is presented in total volumes as well as in percentages of the total for each traffic movement. The Garsfontein road was the targeted corridor because of the high traffic volume of straight MBT traffic. Specifically, straight MBTs on the Garsfontein road accounted for traffic volume of between 5% and 7% of the total through traffic for both AM and PM traffic volumes. All straight traffic movements on the approaches of the Garsfontein road were represented by the numerical symbols '5' and '11'. The figure also shows the calculated morning and afternoon peak traffic volumes for all the twelve movements approaching the intersection.

	Light Vehicles											
	Movement											
	1	2	3	4	5	6	7	8	9	10	11	12
Am Peak 1hr vol:	6	974	203	146	649	95	141	660	124	58	811	3
Am Peak 1hr vol:	6	988	203	157	709	98	146	701	128	62	866	3
PM Peak 1hr vol:	23	880	164	202	904	85	81	767	138	103	707	46
PM Peak 1hr vol:	22	849	150	198	856	81	81	761	136	98	655	38

	Minibus Taxis											
	Movement											
	1	2	3	4	5	6	7	8	9	10	11	12
Am Peak 1hr vol:	0	6	0	9	51	3	5	29	0	2	54	0
Am Peak 1hr vol:		1%		6%	7%	3%	3%	4%		3%	6%	
PM Peak 1hr vol:	4%	2%	5%	1%	5%	4%		1%	1%	1%	6%	2%
PM Peak 1hr vol:	1	16	9	3	41	3	0	6	2	1	40	1

	Heavy Vehicles											
	Movement											
	1	2	3	4	5	6	7	8	9	10	11	12
Am Peak 1hr vol:	0	8	0	2	9	0	0	12	4	2	1	0
Am Peak 1hr vol:		1%		1%	1%			2%	3%	3%	0%	
PM Peak 1hr vol:		2%	3%	0%	1%	1%				4%	2%	15%
PM Peak 1hr vol:	0	15	5	1	7	1	0	0	0	4	12	7

Figure 4-8: Peak Hr Traffic Volume for J1

Figure 4-9 presents the distribution of 15-minute traffic volumes during the traffic count in the morning. The maximum AM traffic volume of was recorded during the first 15 minutes of traffic count between 6:30 AM and 6:45 AM. This is in consistence with the expected critical peak period when Home to Work/School trips are high. The lowest volume was recorded during the last 15 minutes between 7:45 AM and 8:00 AM, just before the normal start time for work/school.

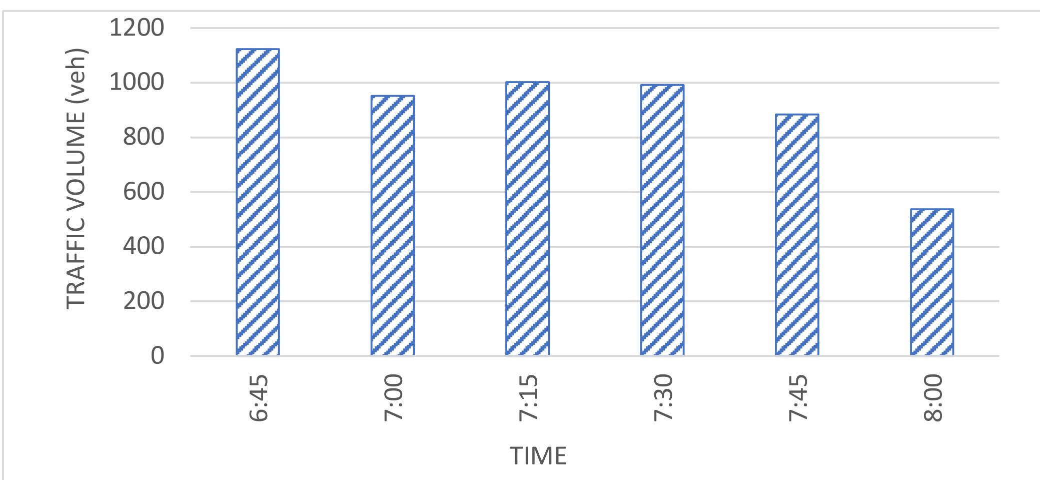


Figure 4-9: Distribution of AM 15-Minutes Traffic Volumes for J1

Figure 4-10 presents the distribution of 15-minutes traffic volumes during the traffic count in the afternoon. The maximum PM traffic volume of was recorded during the first 15 minutes of traffic count between 16:00 PM and 16:15 PM. This is in consistence with the expected critical peak period when Work/School to Home trips are high. In addition, there were almost constant volumes of traffic after the first 15 minutes between 16:15 PM and 17:30 PM.

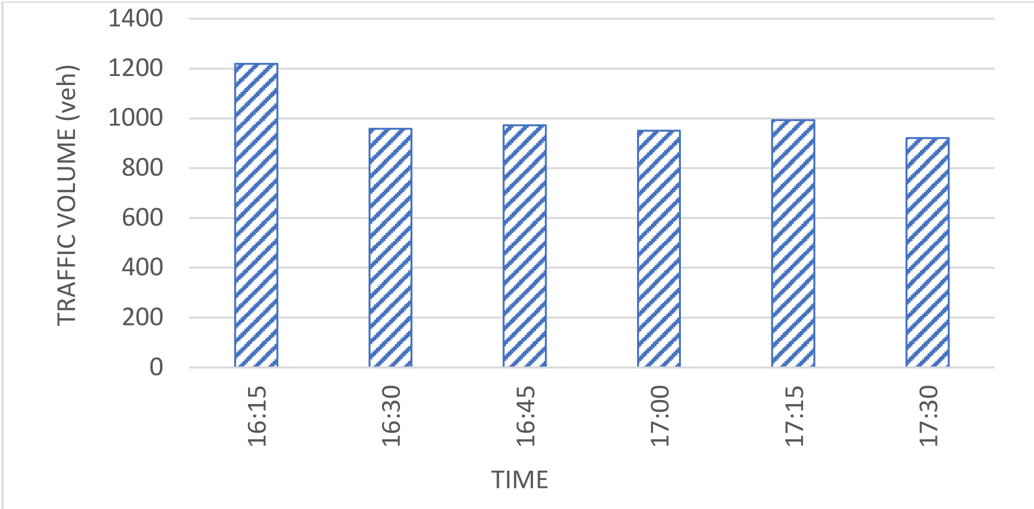


Figure 4-10: Distribution of PM 15-Minutes Traffic Volumes for J1

Figure 4-11 presents AM and PM traffic volumes for each movement. The critical peak period for through (T) traffic movement 5 was in the afternoon because PM peak trips were higher than AM peak trips. Therefore, it was recommended to use PM peak volumes when conducting performance analysis for design strategies on the NB approach. On the other hand, the critical peak period for traffic movement 11 was in the morning because AM trips were higher than PM trips. Therefore, it was recommended to use AM peak volumes when conducting performance analysis for design strategies on the SB approach.

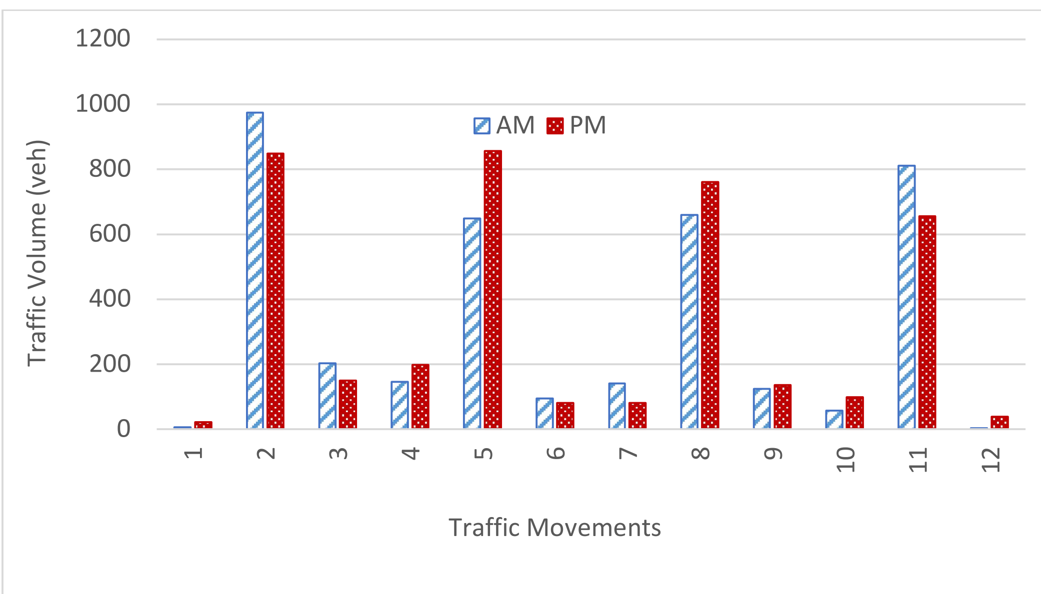
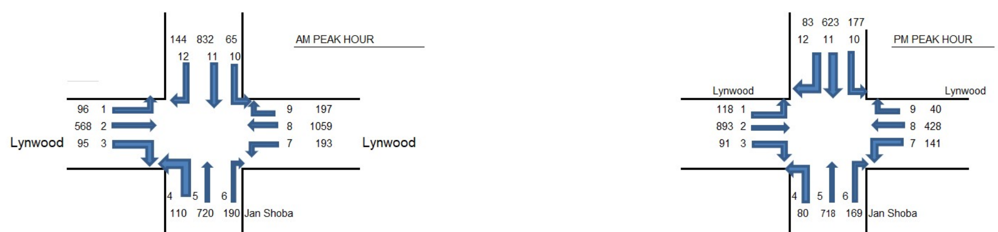


Figure 4-11: AM and PM Peak volumes per traffic movement for J1

4.5.3 AM and PM Peak Hour Traffic Data for J2

Figure 4-12 is an extract of data in Appendix C and presents the AM and PM peak hour traffic volumes which were used for the capacity analysis of J2 intersection. The morning peak hour was from 06:45 AM to 7:45 AM, whereas the afternoon peak hour was from 16:30 PM to 17:30 PM. Traffic counts were done for light vehicles, MBTs and heavy vehicles. The counts considered all traffic movements for the approaches of the intersection denoted by numbers from 1 to 12. The figure also shows traffic volumes of MBTs and heavy vehicles presented in total volumes as well as in percentages of the totals for each movement. The Lynwood Road was the targeted corridor because of the high traffic volume of through MBT. Specifically, through MBTs on the Lynwood Road accounted for traffic volumes of between 3% and 6% of the total AM/PM through traffic. The through traffic movements on the approaches of the Lynwood Road were represented by the numerical symbols ‘2’ and ‘8’. The figure also shows the calculated morning and afternoon peak traffic volumes for all the twelve movements approaching the intersection.



	Light Vehicles											
	Movement											
	1	2	3	4	5	6	7	8	9	10	11	12
Am Peak 1hr vol:	96	524	92	104	690	186	185	1018	152	46	773	143
Am Peak 1hr vol:	96	568	95	110	720	190	193	1059	197	65	832	144
PM Peak 1hr vol:	118	893	91	80	718	169	141	428	40	177	623	83
PM Peak 1hr vol:	117	862	86	76	680	163	136	393	25	138	597	81

	Minibus Taxis											
	Movement											
	1	2	3	4	5	6	7	8	9	10	11	12
Am Peak 1hr vol:	0	32	0	1	20	2	2	28	38	5	40	1
Am Peak 1hr vol:		6%		1%	3%	1%	1%	3%	19%	8%	5%	1%
PM Peak 1hr vol:	1%	3%	2%	1%	3%	1%	2%	6%	15%	19%	2%	1%
PM Peak 1hr vol:	1	25	2	1	25	1	3	24	6	33	15	1

	Heavy Vehicles											
	Movement											
	1	2	3	4	5	6	7	8	9	10	11	12
Am Peak 1hr vol:	0	12	3	5	10	2	6	13	7	14	19	0
Am Peak 1hr vol:		2%	3%	5%	1%	1%	3%	1%	4%	22%	2%	
PM Peak 1hr vol:		1%	3%	4%	2%	3%	1%	3%	23%	3%	2%	1%
PM Peak 1hr vol:	0	6	3	3	13	5	2	11	9	6	11	1

Figure 4-12 : AM and PM Peak Hr Traffic Volume for J2

Figure 4-13 presents the distribution of 15-minute traffic volumes during the traffic count in the morning. The maximum AM traffic volume of was recorded during the third 15 minutes of traffic count between 7:00 AM and 7:15 AM. This is in consistence with the expected critical peak period when Home to Work/School trips are high. The lowest volume was recorded during the first 15-minutes between 6:30AM and 6:45AM.

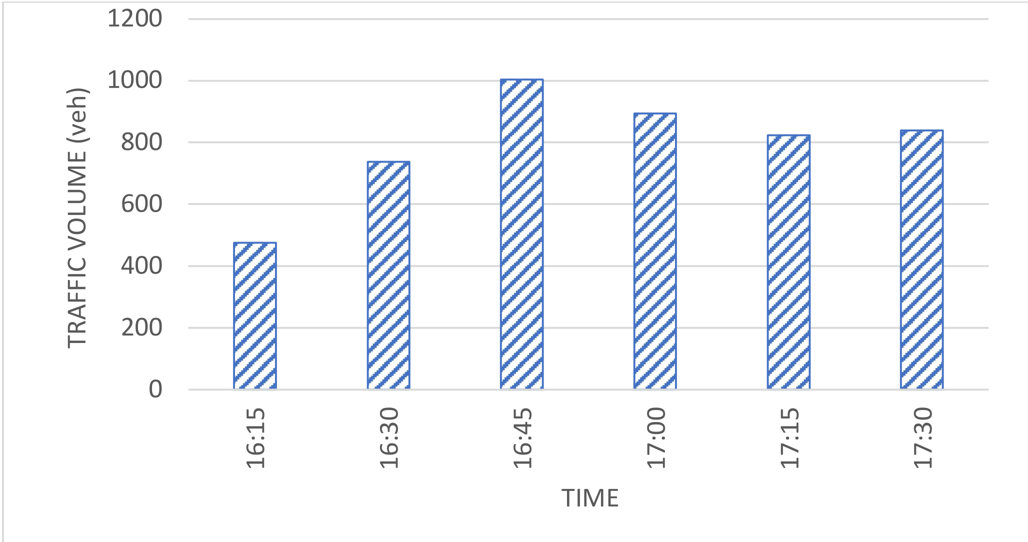


Figure 4-13: AM Peak Hr Volumes per 15-minute intervals for J2

Figure 4-14 presents the distribution of 15- minute traffic volumes during the traffic count in the afternoon. The maximum PM traffic volume of was recorded during the third 15 minutes of traffic count between 16:30 PM and 16:45 PM. This is in consistence with the expected critical peak period when Work/School to Home trips are high. In addition, the lowest PM traffic volume was recorded during the first 15-minutes between 16:00 PM and 16:15 PM.

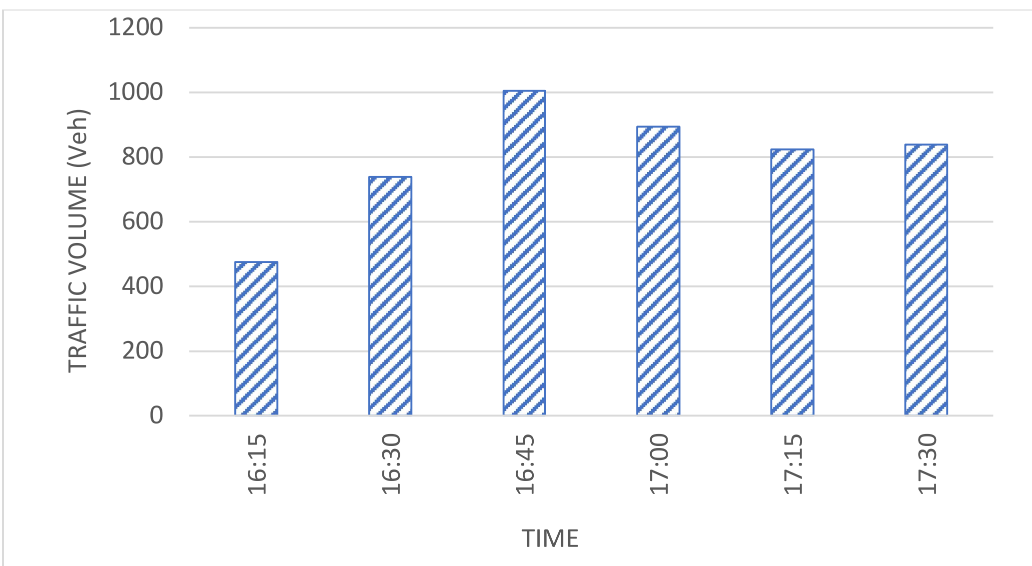


Figure 4-14: Distribution of PM 15-Minutes Traffic Volumes for J2

Figure 4-15 presents the AM and PM traffic volumes for each movement. The critical peak period for through (T) traffic movement '2' (for targeted MBTs) was in the afternoon because PM peak trips were higher than AM peak trips. Therefore, it was recommended to use PM peak volumes when conducting performance analysis for design strategies on the eastbound approach. On the other hand, the critical peak period for traffic movement 8 was in the morning because AM trips were higher than PM trips. Therefore, it was recommended to use AM peak volumes when conducting performance analysis for design strategies on the westbound approach.

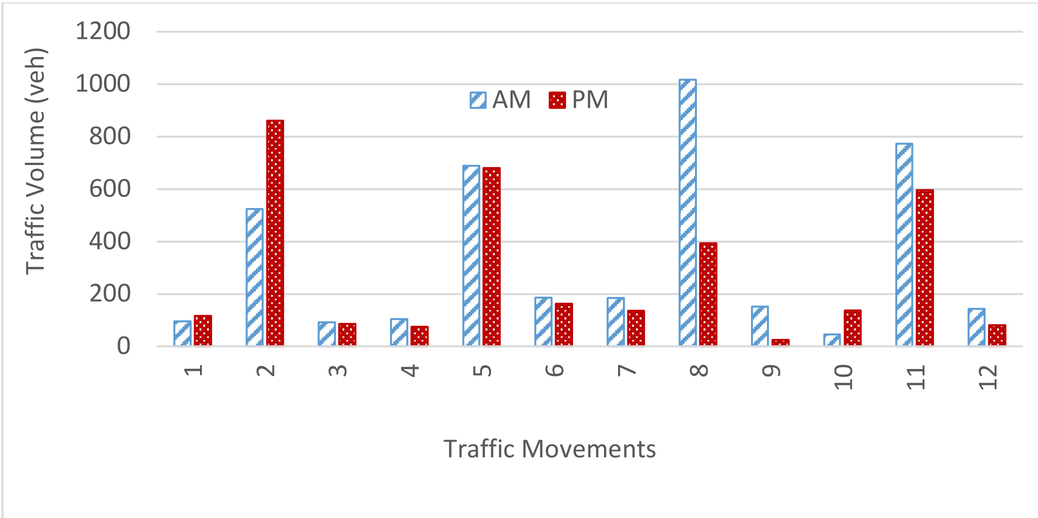


Figure 4-15 AM and PM Peak volumes per movement.

4.6 DATA ANALYSIS

Data analysis was done using the HCM method. Three performance scenarios were considered under each design strategy. First, the performance of existing intersections was evaluated using peak-hour traffic volumes. The performance of the existing intersections acted as the base scenario with no modifications to both geometric and signal conditions. Second, the performance of intersections with the proposed design strategies was evaluated without modifications to existing signal settings. The third scenario involved the assessment of proposed design strategies by modifying the traffic signal settings.

Two proposed MBT priority infrastructure were evaluated in this study. These included a shared MBT lane and dedicated MBT lane. The performance analysis for the shared MBT lane referred to as 'DS1' was evaluated on J1 intersection. On the other hand, performance analysis for the dedicated MBT lane referred to as 'DS2' was evaluated on J2 intersection. It has been shown in previous sections that most often traffic patterns in the morning are different from the patterns in the afternoon hours due to Home to School/Work-based trips and School/Work to Home based trips. To capture these trends, all capacity assessments were done using both AM and PM peak-hour traffic volumes. The traffic volumes for heavy vehicles were converted to passenger car units (PCU) before performing the capacity analysis. The performances of all intersections were evaluated quantitatively using the HCM analysis by using volume to capacity ratio and average vehicle delay as performance measures. Appendices D and E provides summarised calculation sheets of performance analysis of J1 and J2 intersections using HCM method. The theoretical nature of this method was detailed thoroughly in chapter two which is based on the following procedure:

Step 1: Geometric input: The intersection layouts were developed showing traffic movements on all approaches.

Step 2: Traffic input: The AM and PM peak hour traffic volumes determined from traffic counts were converted to passenger car units (PCU) and then allocated to their respective traffic movements. The PCUs were then converted to adjusted flow rates by multiplying with peak hour factors.

Step 3: Input for signal settings: The green times (g), yellow times (Y) and cycle times (C) for all traffic movements were allocated as determined from traffic counts for each intersection.

Step 4: Adjusted saturation flow rate (s): The adjusted saturation flow rate for each movement was determined by applying adjustment factors to the base saturation flow rates. Typical adjustment factors that were used include the number of lanes (N), lane width (f_w) of 3.5m, percentage of heavy vehicles in traffic stream (f_{HV}); approach grade (fg) of 0%; an area type adjustment factor (f) of 0.9 for urban area, a bus and MBT blockage factor (f_{bb}) of 1.0 representing no blockage to traffic flow.

Step 5: Determination of lane capacity (c): For each individual lane, capacity was calculated by multiplying adjusted saturation flow rates (s) by green ratio (g/C) i.e capacity, c = $S_i\ (\frac{g_i}{C})$ (see also Equation 5).

Step 6: v/c ratio (X): The v/c ratio for traffic in each individual lane was determined by dividing the adjusted flow rates of the individual lane by lanes capacity i.e X =$\frac{Vi}{Si(\frac{gi}{C})}$ (see also Equation 6).

Step 7: Lane group geometry: In order to determine average vehicle delay, lane groups were first created. The lane groups which were largely used for analysis included shared lane group for a through and turning traffic, left turning lane group for left turning traffic only, straight movement traffic for all through traffic, right turning lane group for right turning traffic and MBT lane group for traffic occupying MBT priority lane.

Step 8: Adjusted saturation flow rates (s) based on lane groups were determined by adding together all adjusted saturation flow rates of individual lanes forming a lane group.

Step 9: Lane group capacity (c): This was determined by adding capacities of individual lanes forming a lane group.

Step 10: Lane group flow ratio: This was determined by adding flow ratios on individual lanes forming a lane group.

Step 11: Critical flow rate to capacity ratio (X_c). The purpose of this evaluation was to determine if there were available capacities within the existing intersection designs to be utilised for MBT priority infrastructure. This critical X_c ratio was calculated by the formula $Xc= (Y_c)(C)/ (C – L)$ (Equation 7) where Y_c represents the sum of the highest adjusted flow rates of lane groups for each signal phase, while L represents total lost time per signal cycle, and C represents signal cycle time. An analysis duration period of fifteen minutes (0.25hr) was assumed with an upstream filtering metering adjustment facto (I) of 1.0 for all lane groups.

Step 12: Average vehicle delay: This was determined by adding uniform delay, incremental delay, and initial queue delay using Equation 7 i.e total delay (d) = $d_1(PF)+d_2+d_3$. A random arrival progression quality was assumed for all uniform delays hence a progression adjustment factor of 1.0 was used. The average vehicle delay was calculated for the lane group, and approach as well as for the overall intersection in all design scenarios.

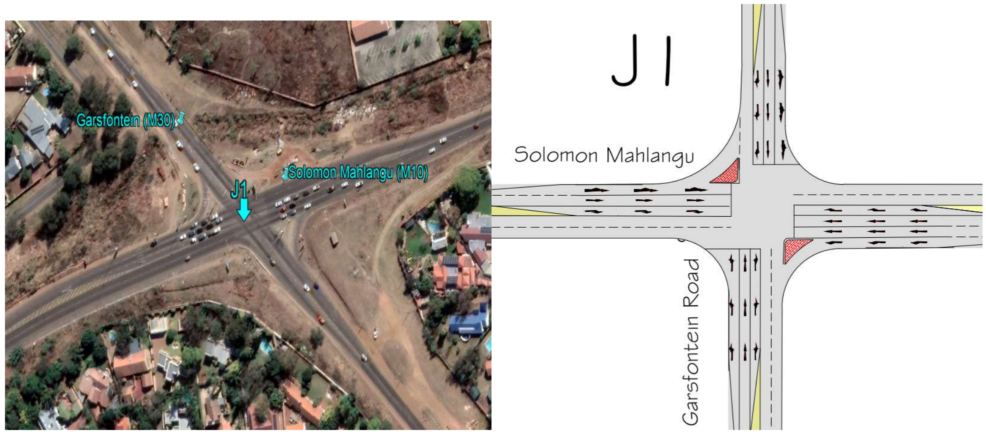
In addition to the above steps, the limits for acceptable values of the v/c ratio and delay used in the analysis were based on scientific conclusions from the studies that drivers in South Africa are aggressive, follow closely and take small gaps which maximize capacity (Bester & Meyers, 2007, Sampson, 2019). This behaviour is therefore related to traffic conditions under a heterogeneous mix of vehicles where drivers do not follow any lane discipline. The analysis for this study therefore adopted a v/c ratio of less than 1.10 and an average vehicle delay of 80 seconds as acceptable limits of performance. These are limits that would give the level of services (LOS) of E (Othayoth & Rao, 2019; and TRB, 2010).

4.6.1 Data Analysis and Interpretation for J1 Intersection

<u>Overview of Analysis Design and Assumptions</u>

J1 was evaluated for shared MBT lane which is also referred to as design strategy 1 (DS1). Appendix D provides details for all the calculations using the HCM method. The targeted corridor for J1 was the Garsfontein road carrying north and south bound through traffic. Three design scenarios were considered for the analysis.

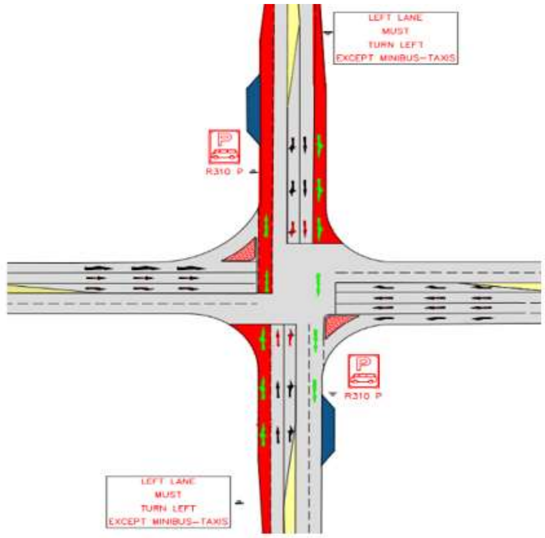
- The first scenario was a 'do nothing' (Existing) which involved the evaluation of existing intersections without modifications to the geometric layout and traffic signals. The idea was to use the results of this scenario to compare with the results for the other two design scenarios. The intersection was designed for two-phased signals. The g/C ratio for the target corridor (N-S) was 0.47. On the other hand, 0.39 was the g/C ratio for the alternate corridor (E-W). Figure 4-16 shows the layout of the existing intersection. It also provides the breakdown of signal timings for both signal phases (stages). The traffic signal was designed with a cycle time of 75 seconds.



Stage 1 (N-S)			Stage 2 (E-W)			Cycle Time
Green phase 1	Yellow phase 1	All-red	Red phase 1			
Red phase 2			Green phase 2	Yellow phase 2	All-red	
34	4	2	35			75
41			28	4	2	75

Figure 4-16: Geometric Layout and Signalisation Existing Scenario for J1

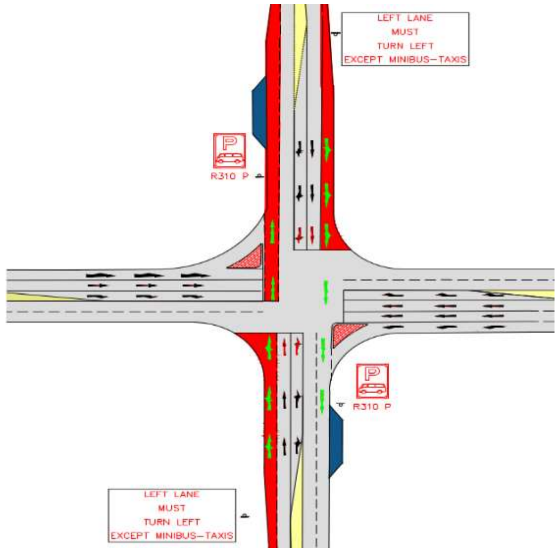
- The second design scenario involved modifications to geometric conditions (DS1a) without changing traffic signals. Under this scenario, the design strategy on the south bound approach involved converting of an auxiliary shared lane to a shared MBT lane. On the other hand, the design strategy on the north bound approach involved converting of a left turning lane to a shared MBT lane plus the addition of a receiving lane on the exit side for the newly converted lane. The g/C ratios of 0.47 and 0.39 were maintained for N-S and E-W signal phases respectively while also maintaining the cycle time of 75s. Figure 4-17 shows the geometric layout and signalisation of DS1a. The shared MBT lanes are shown in RED colour and GREEN road markings. The figure also provides the breakdown of signal timings for both phases with a cycle time of 75 seconds.



Stage 1 (N-S)			Stage 2 (E-W)			Cycle Time
Green phase 1	Yellow phase 1	All-red	Red phase 1			
Red phase 2			Green phase 2	Yellow phase 2	All-red	
34	4	2	35			75
41			28	4	2	75

Figure 4-17: Geometric Layout and Signalisation of DS1a for J1

- The third design scenario involved modifying both geometric conditions and traffic signals (DS1b). The geometric layout changes for this scenario were similar to those defined under DS1a. On the other hand, the optimal g/C ratios were determined through an iterative process. The existing g/C ratios were changed to 0.56 and 0.31 for N-S and E-W signal phases respectively while maintaining the cycle time of 75 seconds. Figure 4-18 provides the geometric layout and signalisation of DS1b. The priority lanes are shown in red colour with green arrows for traffic movements, similar to the layout for DS1a. Similarly, the figure provides the breakdown of signal timings for both signal phases with a cycle time of 75 seconds.



Stage 1 (N-S)			Stage 2 (E-W)			Cycle Time
Green phase 1	Yellow phase 1	All-red	Red phase 1			
Red phase 2			Green phase 2	Yellow phase 2	All-red	
41	4	2	28			75
47			22	4	2	75

Figure 4-18: Geometric Layout and Signalisation of DS1b for J1

<u>Overview of J1 Peak Traffic for South Bound and North Bound Approaches.</u>

The traffic count analysis (Figure 4-7) shows that the south bound approach had peak traffic in the morning (AM) period while the north bound approach had the highest (peak) traffic during the afternoon (PM) period. Subsequently, the PM traffic data was used for the evaluation of south bound approach and AM traffic data was used for the evaluation of north bound approach. Performance evaluation results have been divided into two main categories which include v/c ratio and average vehicle delay. The v/c ratios for the vehicles on the existing lanes were compared to the v/c ratios of vehicles on individual lanes for DS1a and DS1b scenarios. Existing average vehicle delays were calculated for each lane group, approach and overall intersection and then compared to respective delays for DS1a and DS1b scenarios to determine the performance of the design strategies.

a) South Bound (SB) Approach

The geometric changes for DS1a (g/C=0.47) and DS1b (g/C=0.56) design strategies on the SB approach involved converting the shared (mixed) traffic movement lane for all vehicle types to a shared MBT lane (See Fig 4-15, Fig 4-16, and Fig 4-17). The PM peak volumes were used for this evaluation. The subsequent sections provide detailed analysis of the results on the SB approach.

The v/c Ratio Evaluation

Figure 4-19 shows the impact of the three design strategies on v/c ratios of the individual lanes using the PM peak traffic volume. The analysis considers traffic for the lanes on the SB approach. The two types of lanes targeted were MBT lane and the straight movement traffic lane. For each lane, v/c ratios for existing conditions were compared to v/c ratios for DS1a and DS1b.

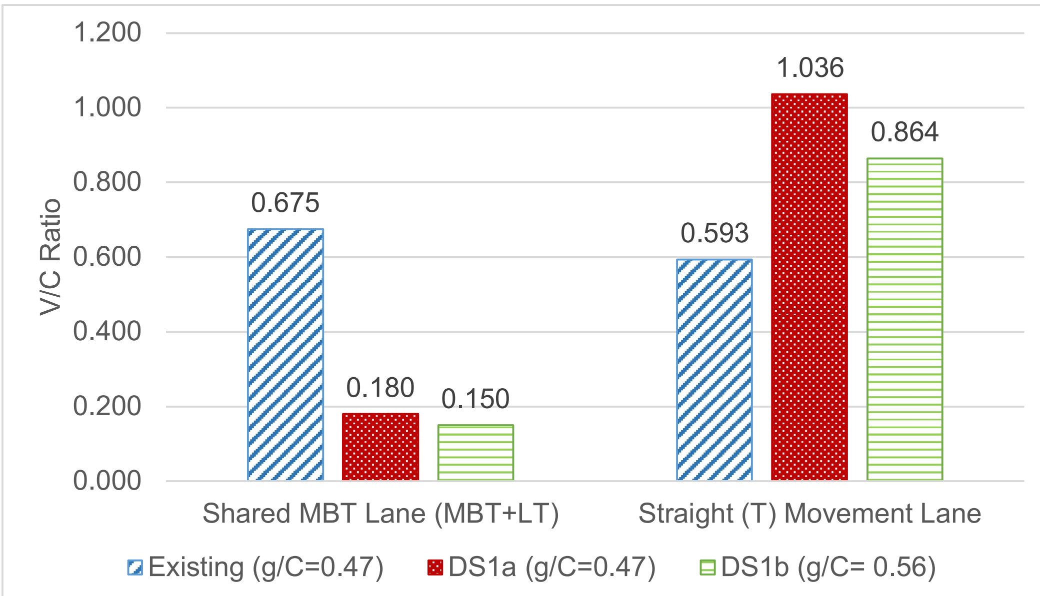


Figure 4-19: Impact of DS1 on v/c Ratio (PM) for J1 SB Approach

The results indicate that DS1 has improved the capacity of the shared lane from v/c ratio of 0.675 to a v/c ratio of 0.180 for DS1a and to a v/c of 0.150 for DS1b. This represents an improvement of 73% and 78% respectively. This improvement is mainly attributed to removal of through traffic of other cars from the shared MBT lane. On the other hand, the v/c ratio for the adjacent straight movement lane increased from v/c ratio of 0.593 to v/c ratios of 1.036 and 0.864 for DS1a and DS1b respectively. This represents a v/c ratio increase of 75% and 46% for DS1a and DS1b respectively. The increase in v/c ratio was due to through traffic from the MBT shared lane. In addition, all results are falling within acceptable limits defined in this study.

Lane Delay Evaluation

Figure 4-20 summarises delay associated with lane groups that were affected by the design strategies using the PM peak traffic volumes. Two groups of traffic movements were targeted, and these included a shared MBT lane group and a straight movement lane group. The shared MBT lane group comprised of straight traffic movement of MBTs and turning traffic movement of all other vehicles. On the other hand, the straight movement lane group consisted of straight traffic movement of all other vehicles. For each lane group, average vehicle delays for existing scenario were compared to average vehicle delays for DS1a and DS2b scenarios.

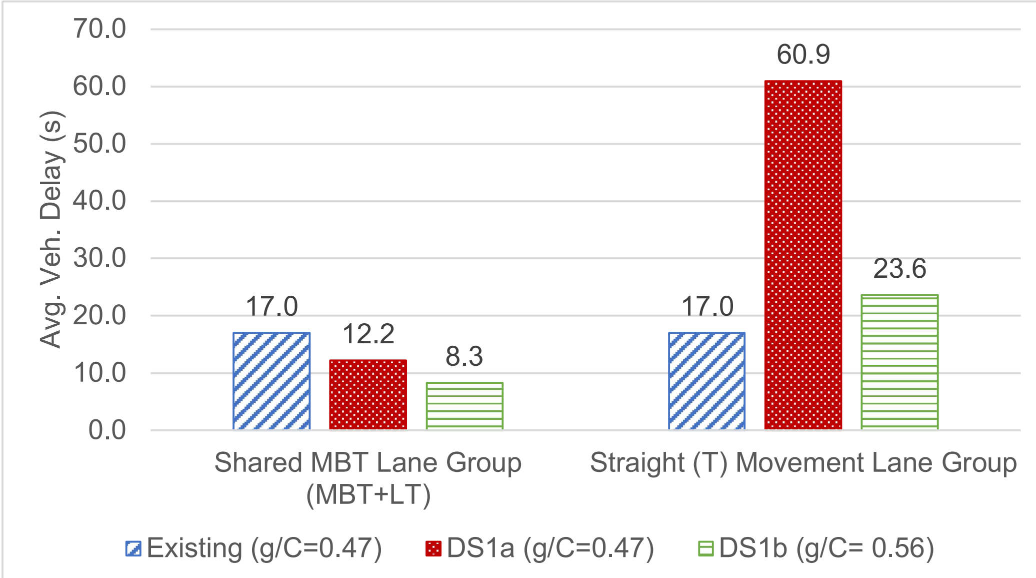


Figure 4-20: Impact of DS1 on Lane Group Delay (AM) for J1 SB Approach

The figure shows that design strategy DS1 has led to a reduced vehicle delay on the shared MBT lane while increasing the delay for traffic on straight movement lane. Vehicle delay for shared MBT lane reduced from 17 seconds to 12.2 seconds and 8.3 seconds for DS1a and DS1b respectively representing 28% and 51% improvements. On the contrary, there was an increase on the average vehicle delay for vehicles on the straight movement lane. The results show an average vehicle delay for vehicles on the straight movement lane increased from 17 seconds to 60.9 seconds and 23.6 seconds for DS1b and DS1b respectively representing delay increase of 250% and 39%.

<u>Delay Evaluation on SB Approach and Overall Intersection</u>

Figure 4-21 summarises average vehicle delays on the SB approach and overall J1 intersection using the PM peak traffic volumes. In both situations, the analysis compares average vehicle delays for the existing scenario to the delays for the DS1a and DS1b scenarios.

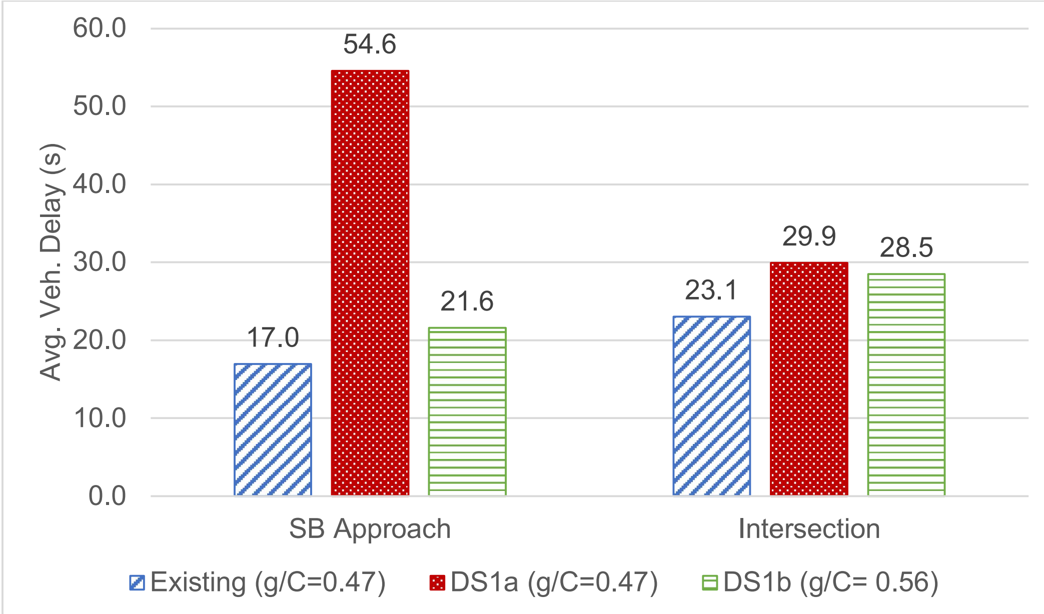


Figure 4-21: Impact of DS1 on Delay (PM) for J1 SB Approach and Overall Intersection

Results show that DS1 design strategies increased the average vehicle delay of the approach from 17 seconds to 54.6 seconds and 21.6 seconds for DS1a and DS1b respectively representing 221% and 27% increase. On the other hand, the average delay for the entire intersection also increased but did not significantly change for both design strategies. The average delay increased from 23.1 seconds to 29.9 seconds and 28.5 seconds for DS1b and DS1b respectively. This represents the delay increase of 29% and 23% for DS1a and DS1b respectively.

b) <u>North Bound (NB) Approach</u>

The design strategies on north bound approach involved changing a short left turning lane to a shared MBT lane and adding a receiving lane for the MBT traffic (See layouts in Figures 4-15, 4-16, 4-17). The AM peak volumes were used for this evaluation. The subsequent sections provide detailed analysis of the results on the NB approach.

<u>The v/c Ratio Evaluation</u>

Figure 4-22 provides v/c ratios of different vehicles on individual lanes that are affected by the introduction of the design strategies. The figure compares v/c ratios of the existing traffic and geometric conditions to the v/c ratios for traffic under DS1a and DS1b scenarios. The AM peak hour traffic volumes on the shared MBT lane and straight traffic movement lane were used to perform this evaluation.

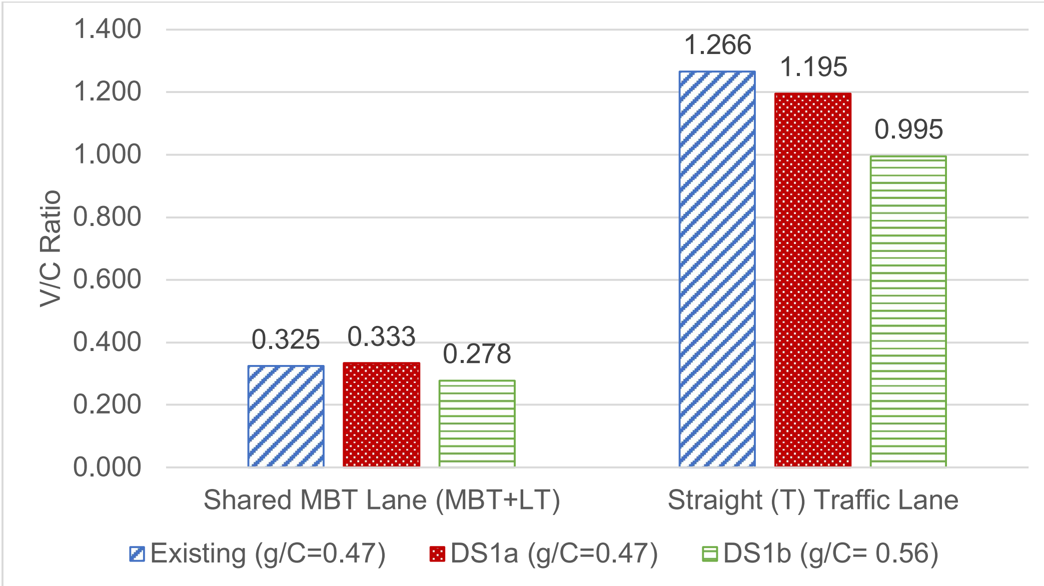


Figure 4-22 : Impact of DS1 on v/c Ratio (AM) for J1 NB Approach

The results indicate minor changes to v/c ratios on both the shared MBT lane and the straight (through) lane. For the shared lane, the v/c ratio changed from 0.325 to a v/c ratio of 0.333 for DS1a and to a v/c ratio of 0.278 for DS1b. This represents a capacity decrease of 2% for DS1a and a capacity increase of 14% for DS1b. The decrease in capacity for DS1a is mainly attributed to additional through MBT traffic volumes coming from exclusive through lanes. On the other hand, the capacity of the straight movement lane improved due to the removal of MBT traffic from the stream. The results show v/c ratios improving from 1.266 to v/c ratios of 1.195 and 0.995 for DS1a and DS1b respectively. This represents a capacity improvement of 6% and 21% for DS1a and DS1b respectively.

Lane Delay Evaluation

Figure 4-23 summarises delay associated with lane groups and vehicle type on shared MBT lane and straight movement lanes that were affected by the design strategy DS1. The shared MBT lane group was used for analysis of delays for MBT traffic and left turning traffic. On the other hand, the straight movement lane group was used for the analysis of delays for straight movement traffic. In both situations, AM peak hour traffic volumes for the NB approach were used.

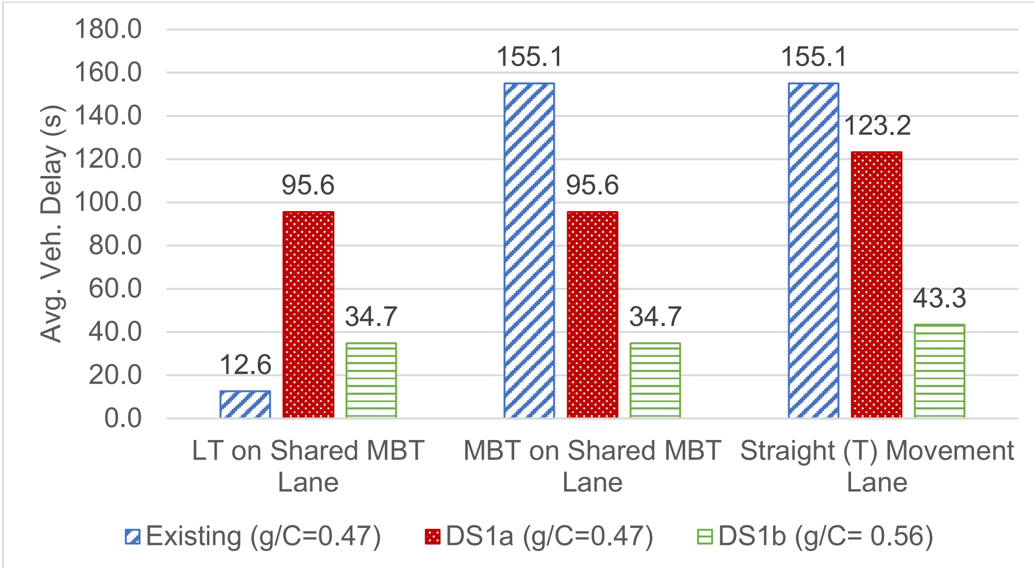


Figure 4-23 : Impact of DS1 on Delay (AM) for J1 NB Approach

The results indicate that DS1 had multiple impacts of delay on shared and straight traffic. Delay for turning traffic on the shared MBT lane increased from 12.6 seconds to 95.6 seconds and 34.7 seconds for DS1a and DS1b respectively. This represents a delay increase of 658% and 175% for DS1a and DS1b respectively.

On the contrary, delay for MBT traffic and straight vehicles improved for both DS1a and DS1b. For MBT traffic, the delay decreased from 155.1 seconds to 95.6 seconds and 34.7 seconds for DS1a and DS1b respectively. This represents an improvement of 38% and 78% respectively. Similarly, the delay for straight vehicles decreased from 155.1 seconds to 123 seconds and 43.3 seconds for DS1a and DS1b respectively. This represents an improvement in the delay of 21% and 72% for DS1a and DS1b respectively.

<u>Delay Evaluation for the NB Approach and Overall Intersection</u>

Figure 4-24 summarises average vehicle delays for the NB approach and overall intersection. In both situations, the analysis uses AM peak hour traffic to compare average vehicle delays for the existing scenario to the delays for the DS1a and DS1b scenarios.

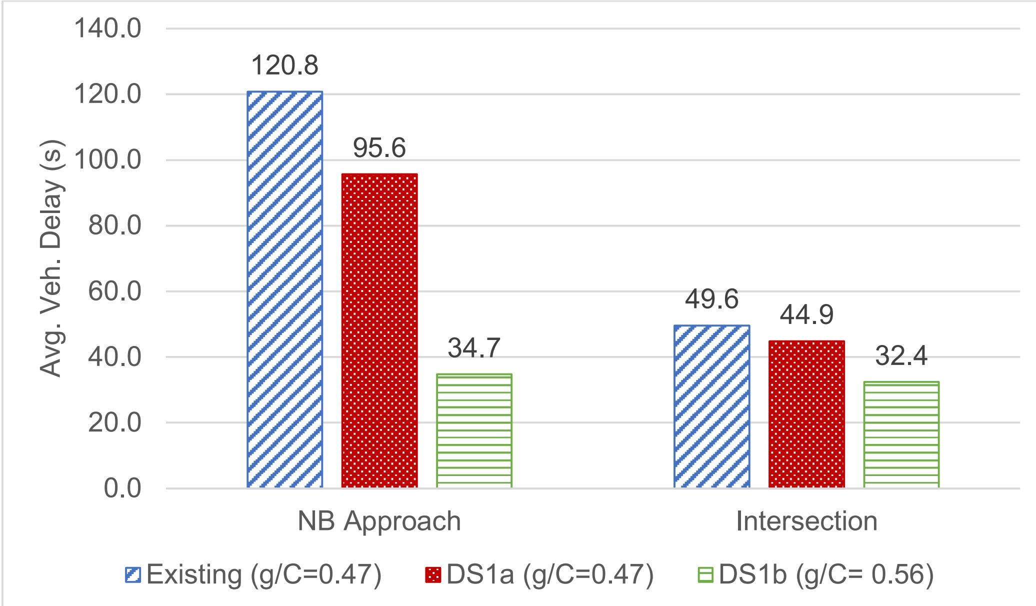


Figure 4-24: Impact of DS1 on Delay (AM) for J1 NB Approach and Overall Intersection

The results show that DS1 design strategies improved the overall vehicle delay for the approach and intersection. The delay for the north bound approach decreased from 120.8 seconds to 95.6 seconds and 34.7 seconds for DS1a and DS1b respectively. This represents an improvement of 23% and 71% for DS1a and DS1b respectively. On the other hand, the average delay for the entire intersection also improved for both design strategies. The average delay increased from 49.6 seconds to 44.9 seconds and 32.4 seconds for DS1b and DS1b respectively. This represents an improvement of 9% and 35% for DS1a and DS1b respectively.

c) <u>East Bound (EB) and West Bound (WB) Capacity and Delay Analysis for J1</u>

The capacity results further indicate that traffic on EB and WB approaches was greatly impacted by DS1b (modification of geometric conditions plus traffic signals). This is because of the decrease in green time on the signal phase associated with traffic on the EB and WB approaches during the DS1b scenario. The g/c ratios for EB and WB approaches were reduced from 0.39 for existing condition to 0.31 for DS1b. As a result, the analysis has included a summary of this evaluation to determine the impact of DS1b on lane capacity and average vehicle delay. The evaluation was conducted on the worst impacted lane on both approaches. This ideally means the lane with the worst (highest) values of v/c ratio and delay under existing conditions. The AM peak traffic volume was used for analysing the EB approach while PM peak traffic volume was used to determine the impact on the WB approach. The subsequent sections provide a high-level analysis of the results on the EB and WB approaches.

<u>The v/C Ratio Evaluation</u>

Figure 4-25 shows changes in v/c ratios for the worst affected lane on EB and WB approaches using both AM and PM peak traffic. The v/c ratios for the existing conditions were compared to the respective v/c ratios for the DS1b design scenario. The AM peak traffic volumes were used for determining changes in v/c ratio for the EB approach whereas the PM peak traffic volumes were used for evaluating v/c ratios for the WB approach.

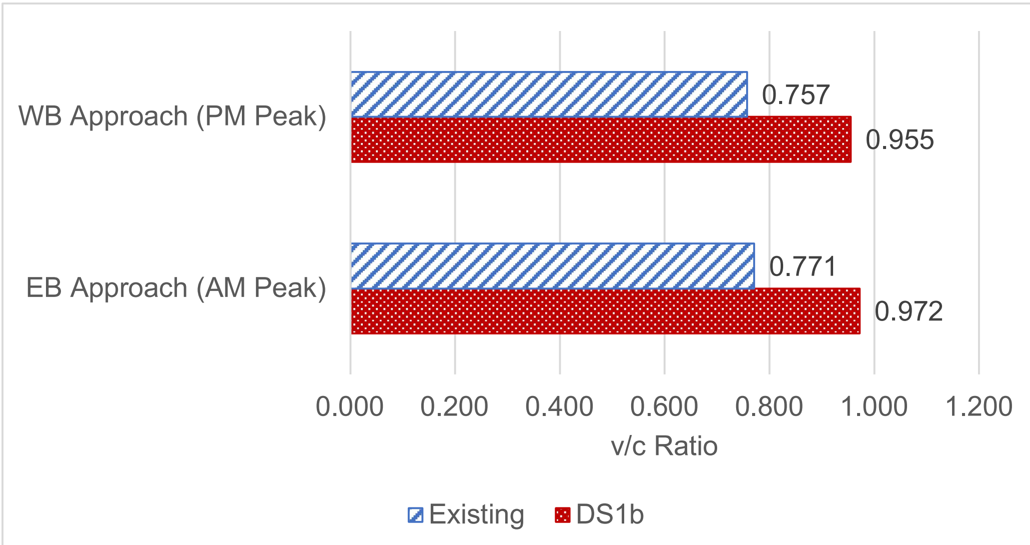


Figure 4-25: Impact of DS1b on v/C ratio (AM&PM) for J1 EB and WB Approaches

The results show an increase in the v/c ratio for the worst affected lane under both AM and PM conditions. The v/c ratio increased from 0.771 to a v/c ratio of 0.972 for the AM peak and from 0.757 to 0.955 for the PM peak. This represents a capacity decrease of 26% for both AM and PM peak conditions. The reduction in capacity is a result of a reduction in green time allocated to both east and west bound approaches during the DS1b design scenario. However, all values of v/c ratios were falling within the acceptable level of services defined in this study.

<u>Delay Evaluation</u>

Figure 4-26 shows changes in delay for the worst affected lane on EB and WB approaches for both AM and PM peak traffic. The worst lane group delays for the existing conditions were compared to respective lane group delays under the DS1b scenario. The AM peak traffic volumes were used for determining changes in delays for the EB approach, whereas the PM peak traffic volumes were used for evaluating delays for the WB approach.

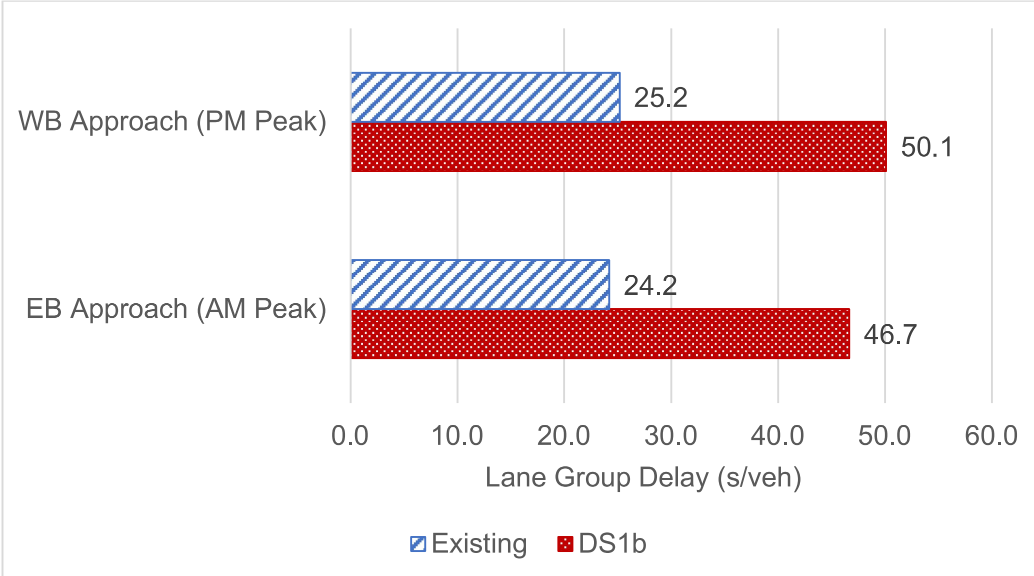


Figure 4-26: Impact of DS1b on Delay (AM&PM) for J1 EB and WB Approaches

The results indicate an increase in delay on the worst impacted lane under both AM and PM peak conditions. The average vehicle delay increased from 24.2 seconds to 46.7 seconds for the AM peak traffic conditions and from 25.2 seconds to 50.1 seconds for the PM peak traffic conditions. This represents a delay increase of 97% and 99% for the AM and PM peak conditions respectively. The increase in delay is a result of a decrease in signal green time for traffic on both EB and WB approaches under the DS1b design scenario. On the other hand, all delay times were falling within acceptable levels of services defined in this study.

<u>Checking Adequacy of Storage Length of the Shared MBT Lane (Auxiliary LT Lane)</u>

This analysis was carried out on the approach with the highest peak hour traffic volumes in the shared MBT lane. Consequently, the analysis was done using the PM traffic volume in the shared MBT lane for the north bound approach. The AASHTO (2004) provides the formula (Equation 2) which is used for determining the storage lengths for auxiliary lanes as follows.

Length (L)= $1.5*n_i*L_i$ (Equation 2)

Table 4-5 uses Equation 2 to check the adequacy of existing storage length of the auxiliary lane to accommodate traffic for the shared MBT lane. The analysis uses the expected PM peak hour traffic volumes in the MBT shared lane for the north bound with a cycle length of 75 seconds.

Table 4-5: Storage Length Calculation for the Shared MBT Lane for J1

Peak Hr Traffic Vol (LT+MBT), $V_{sharedMBT}$	244 PCU/Hr
Cycle Time,C = Seconds	75 seconds
Average. Traffic per cycle (n)=($V_{sharedMBT}$/(3600/C))	5.0625 = approx. 6 PCU/Cycle
Storage Length in PCU=(L)=(1.5*n)= (PCU)	9 PCUs
Calculated Storage Length in m = $L_{(PCU)}$X Average size of passenger car (4.8m)	43.2m
Existing storage length (L)=(m)	30m

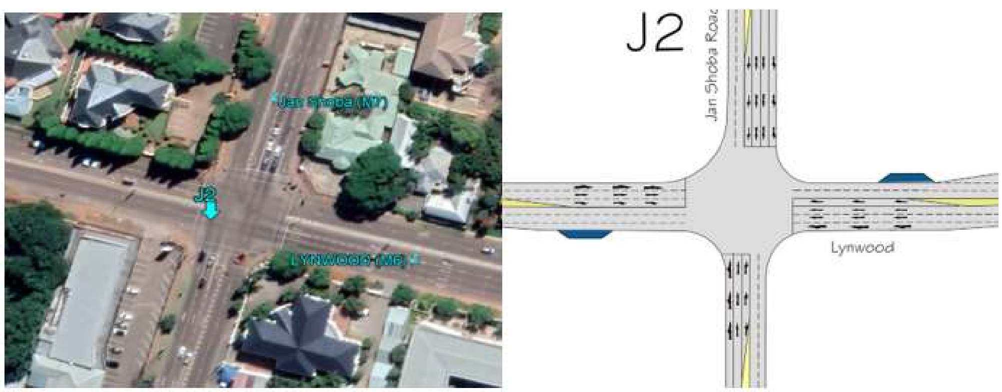
The results show that about 45m of the storage length would be required to accommodate the peak hour traffic. The current existing storage length is 30m which is not adequate to accommodate the shared MBT lane hence for the purpose of the shared MBT lane, a recommendation would be made to extend the existing storage length by a minimum of 15m.

4.6.2 Data Analysis and Interpretation for J2 Intersection

<u>Overview of Design Scenarios and Assumptions</u>

J2 evaluation involved adding dedicated short MBT lanes which is also referred to as design strategy 2 (DS2). Appendix E provides details for all the calculations using the HCM method. The targeted corridor for the analysis was the east/west corridor (Lynwood Rd). Three design scenarios were considered for the analysis and include the following:

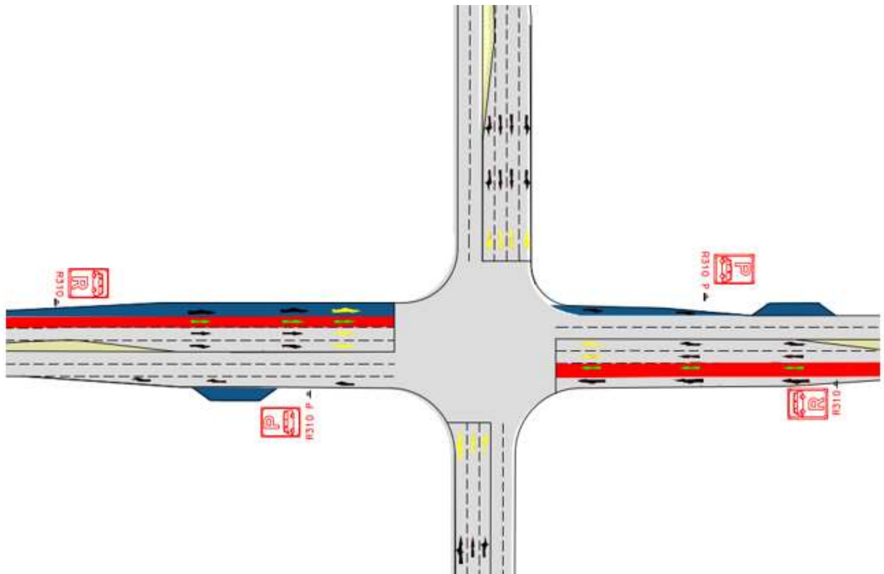
- A 'do nothing' scenario. This involved evaluating the 'existing' intersection with no modifications to the intersection geometry or signalisation. The idea was to use the results of this scenario to compare with the results for the other two design scenarios. The existing g/C ratios of 0.31 and 0.42 for east bound and west bound respectively were utilised. Figure 4-27 summarises the existing geometric and signal conditions for the J2 intersection. The intersection has three lanes on the east bound approach and four lanes on the west bound approach. In terms of signalisation, the figure shows the allocation of green, yellow and red times for both signal phases in line with the direction of traffic.



Approach	Stage 1 (N-S)				Stage 2 (E-W)				Cycle Time
	Green phase 1		Yellow phase 1	All-red	Red phase 1			All-red	
	Red phase 2				Green phase 2		Yellow phase 2	All-red	
NB	36		4	2	39				81
SB	12	24	4	2	39				81
WB	42				33		4	2	81
EB	51					24	4	2	81

Figure 4-27: Geometric Layout and Signalisation Existing Scenario of J2

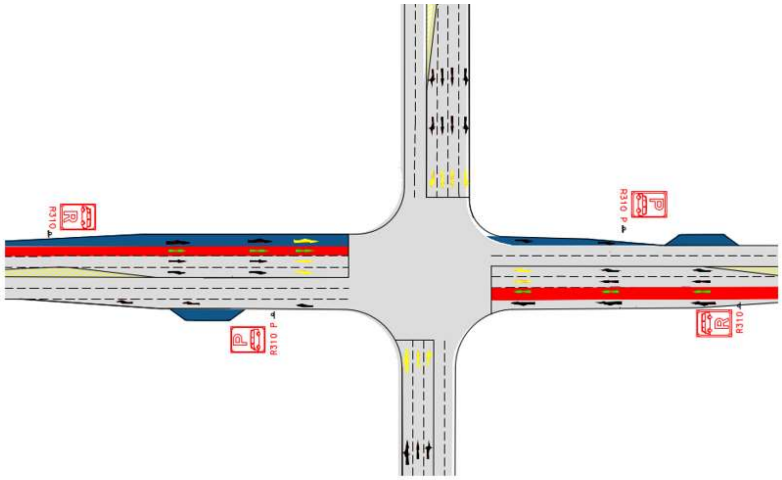
- DS2a scenario: This design scenario involved modification of geometric conditions without modifying traffic signal settings (Figure 4-28). Specifically, the design strategy on the west bound approach involved converting one of the exclusive 'through' lanes to a dedicated short MBT lane. The design strategy on the east bound approach involved, first, converting a nearside mixed or shared traffic movement lane to a dedicated short MBT lane and then adding a new auxiliary mixed lane for turning and through traffic. In terms of signalisation, the existing g/C ratios of 0.31 and 0.42 for traffic on east bound and west bound respectively were maintained. The dedicated MBT lanes are shown in red colour, whereas the newly added auxiliary lanes are shown in dark blue colour.



Approach	Stage 1 (N-S)				Stage 2 (E-W)				Cycle Time
	Green phase 1		Yellow phase 1	All-red	Red phase 1			All-red	
	Red phase 2				Green phase 2		Yellow phase 2	All-red	
NB	36		4	2	39				81
SB	12	24	4	2	39				81
WB	42				33		4	2	81
EB	51					24	4	2	81

Figure 4-28: Geometric Layout and Signalisation of DS2a for J2

- DS2b scenario: This involved modifying both geometric conditions and traffic signals (Figure 4-29). The geometric layout changes for DS2b scenario were similar to those defined under DS2a. On the other hand, the optimum g/C ratios for this scenario were developed through an iterative process. The g/C ratios were modified from 0.31 to 0.47 for the east bound approach; and from 0.42 to 0.58 for the west bound approach. The dedicated MBT lanes are shown in red colour, whereas the newly added auxiliary lanes are shown in dark blue colour.



Approach	Stage 1 (N-S)				Stage 2 (E-W)				Cycle Time
	Green phase 1		Yellow phase 1	All-red	Red phase 1			All-red	
	Red phase 2				Green phase 2		Yellow phase 2	All-red	
NB	25		3	2	51				81
SB	4	21	3	2	51				81
WB	30				46		3	2	81
EB	30				9	37	3	2	81

Figure 4-29: Geometric Layout and Signalisation of DS2b for J2

<u>Overview of J2 Peak Traffic for East Bound and West Bound Approaches.</u>

Traffic count analysis (Figure 4-11) showed that the west bound approach had the highest (peak) traffic during the morning (AM) period while the east bound approach had peak traffic in the afternoon (PM) period. Subsequently, AM traffic data was used for the evaluation of west bound approach and PM traffic data was used for the evaluation of the east bound approach.

The subsequent sections evaluate the impact of the DS2 design strategies on v/c ratios and average vehicle delay.

d) West Bound (WB) Approach

The geometric changes for DS2a (g/C=0.42) and DS2b (g/C=0.58) design strategies on the WB approach involved converting a straight traffic movement lane for all vehicle types to a dedicated MBT lane (See Fig 4-26, Fig 4-27, and Fig 4-28). The AM peak volumes were used for this evaluation.

<u>The v/c Ratio Evaluation</u>

Figure 4-30 provides a v/c ratio analysis of traffic in the individual lanes that are affected by the introduction of the DS2 design scenarios on the WB approach using AM peak traffic volume. These lanes include a shared auxiliary lane (LT+T), a dedicated MBT lane (MBT) and a straight movement lane (T). For each of these lanes, the v/c ratios under existing conditions (g/C =0.42) are compared to v/c ratios for DS2a (g/C=0.42) and DS2b (g/C=0.58) design scenarios.

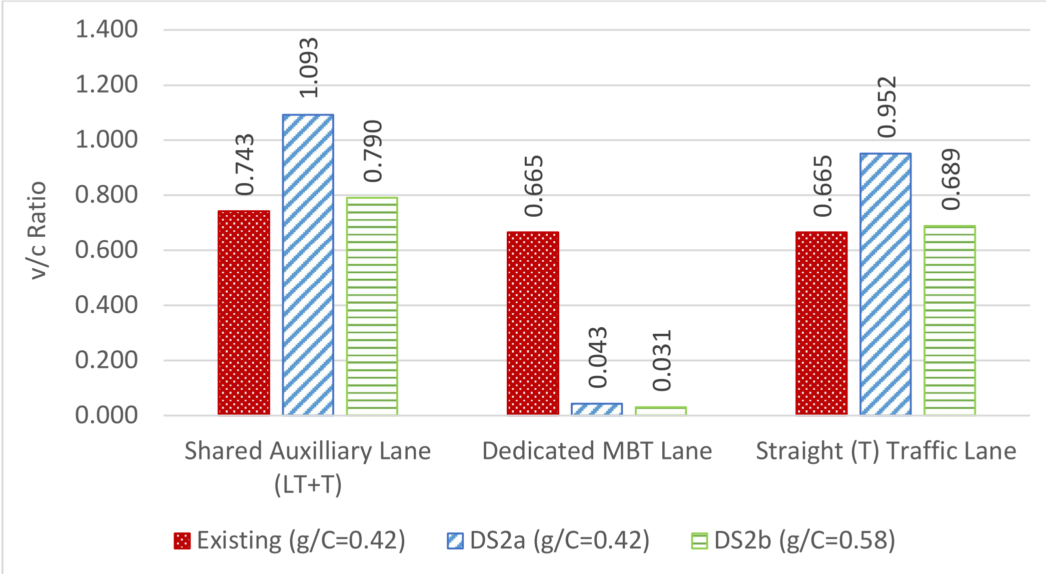


Figure 4-30: Impact of DS2 on v/c Ratio (AM) for J2 WB Approach

Results show that DS2 had an impact on traffic flow in the dedicated MBT Lane, auxiliary lane and straight traffic lane. The v/c ratio for vehicles in the dedicated MBT lane changed from 0.665 to 0.043 for DS2a, and to a v/c ratio of 0.031 for DS2b. This represents a capacity improvement of 94% and 95% for DS2a and DS2b respectively. This is mainly attributed to a shift in through traffic from the MBT dedicated lane to both the shared auxiliary lane and the straight traffic lane.

On the other hand, the v/c ratio for vehicles flowing in straight traffic lanes and auxiliary mixed traffic lane increased. The results show the v/c ratio of vehicles in the shared auxiliary lane increasing from a v/c ratio of 0.743 to v/c ratios of 1.093 and 0.790 for DS1a and DS1b respectively. This represents a lane capacity decrease of 35% and 6% for DS1a and DS1b respectively. Similarly, the capacity of vehicles in the straight movement lane decreased. The v/c ratio increased from a v/c ratio of 0.665 to v/c ratios of 0.952 and 0.689 for DS2a and DS2b respectively. This represents a capacity decrease of 43% and 4% for DS2a and DS2b respectively. The increase in v/c ratios for traffic on straight movement and shared auxiliary lanes is a result of additional through traffic (other than MBT) from the dedicated MBT lane.

The results also show all values of v/c ratios falling within acceptable limits as defined in this study.

<u>Lane Delay Evaluation</u>

Figure 4-31 summarises the average vehicle delay associated with lane groups that were affected by the DS2 design strategies using the AM peak traffic volumes. The analysis uses changes in delay for the shared movement lane group and dedicated MBT lane group on the

WB approach. The shared movement lane group comprised of a combination of straight movement traffic and left turning traffic. On the other hand, the dedicated MBT lane group contained MBT traffic making straight traffic movement. For each lane group, the average vehicle delays for existing conditions were compared to average vehicle delay under DS2a and DS2b design scenarios.

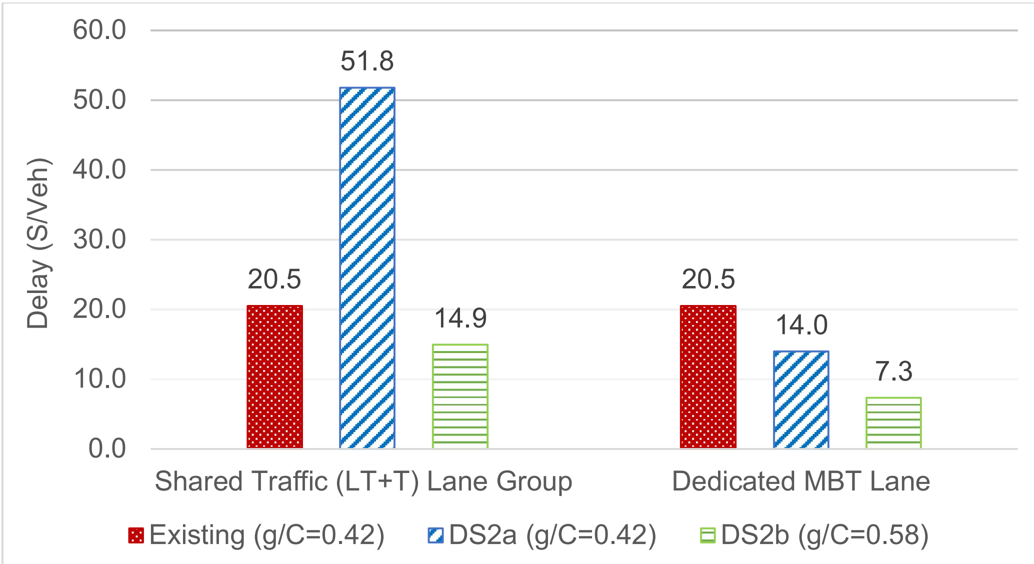


Figure 4-31:Impact of DS2 on Delay (AM) for J2 WB Approach

The results show that design strategy DS2 has reduced vehicle delay on the dedicated MBT lane while increasing delays for vehicles on the other lanes. Vehicle delay for the dedicated MBT lane was reduced from 20.5 seconds to 14.0 seconds for DS2a; and to 7.3 seconds for DS2b. This represents a decrease in average vehicle delay of 32% and 64% for DS2a and DS2b respectively. The improvement in delay is attributed to a reduced traffic volume of MBT vehicles flowing in the dedicated MBT lane.

On the other hand, DS2 produced mixed results of delay for vehicles in the LT+T auxiliary lane and T lane. The DS2a increased average vehicle delay for the left turning and straight movement traffic. The average vehicle delay increased from 20.5 seconds to 51.8 seconds in the shared lane group. This represents a delay increase of 154% in the shared traffic lane group. The increase in average vehicle delay is mainly as a result of additional traffic from the MBT lane. On the contrary, DS2b produced the least delays of all design scenarios. The results show the average vehicle delay decreased from 20.5 seconds to 14.9 seconds for the shared traffic lane group. This represents a delay decrease of 27%. The reduced delay is attributed to the higher g/C ratio on the WB approach during the DS2b scenario.

Delay Evaluation for WB Approach and Overall Intersection

Figure 4-32 summarises average vehicle delays for the west bound approach and overall intersection using AM peak hour traffic volume. In both situations, the analysis uses AM peak hour traffic to compare average vehicle delays for the existing scenario to the delays for the DS2a and DS2b scenarios.

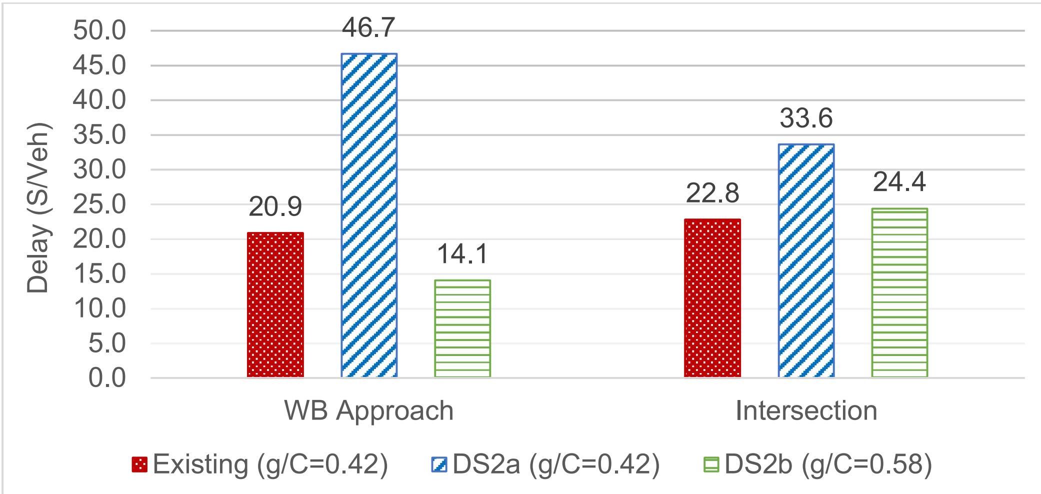


Figure 4-32: Impact of DS2 on Delay (AM) for J2 WB Approach and Overall Intersection

Results indicate that DS2a increased the average vehicle delay of the approach from 20.9 seconds to 46.7 seconds on the WB approach, and from 22.8 seconds to 33.6 seconds on the overall average intersection delay. This represents an average vehicle delay increase of 123% and 47% on west bound approach and overall intersection respectively. The increase in delay was mainly attributed to delays for cars in the shared lane and straight movement lane.

On the other hand, results show that DS2b decreased average vehicle delay for both the west bound approach and overall intersection when compared to DS2a. The average approach delay decreased from 20.9 seconds to 14.1 seconds on west bound approach representing a delay decrease of 33%. Whereas the delay for the entire intersection changed from 33.6 for DS2a to 24.4 seconds for DS2b representing a delay decrease of 27%. The decrease in delay for DS2b is a result of increased g/C ratio for traffic on both WB approach which also had a positive impact on the overall performance of the intersection.

e) <u>East Bound (EB) Approach</u>

The geometric design strategy on the east bound approach involved converting existing nearside shared traffic lane to a dedicated MBT and then adding a new shared movement auxiliary lane (See Fig 4-26, Fig 4-27, and Fig 4-28). The PM peak volumes were used for this evaluation.

<u>The v/c Ratio Evaluation</u>

Figure 4-33 provides a v/c ratio analysis of individual lanes that are affected by DS2 on east bound using PM peak hour traffic. These lanes include a shared auxiliary lane, a dedicated MBT lane and a straight traffic lane. For each of these lanes, the v/c ratios under existing conditions (g/c =0.31) are compared to v/c ratios for DS2a (g/c=0.31) and DS2b (g/c=0.47) design scenarios.

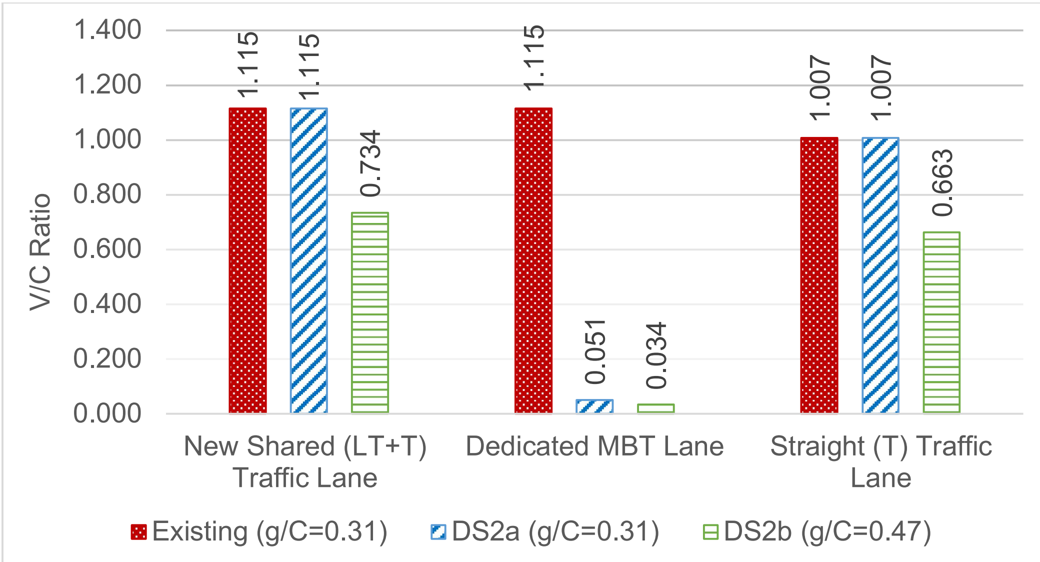


Figure 4-33: Impact of DS2 on v/c Ratio (PM) for J2 EB Approach

Results show that DS2 had an impact on the v/c ratio for traffic flow in the dedicated MBT lane, auxiliary shared lane and straight (T) lane. The v/c ratio of MBT traffic for the dedicated MBT lane reduced from a v/c ratio of 1.115 to a v/c ratio of 0.051 for DS2a, and to a v/c ratio of 0.034 for DS2b. This represents a capacity improvement of 95% and 97% for DS2a and DS2b respectively. The improvement is mainly attributed to the removal of vehicles from the dedicated MBT lane.

The v/c ratio of vehicles in the auxiliary shared movement lane remained constant for DS2a and decreased for DS2b. The results show a constant v/c ratio of 1.115 for DS2a. This constant capacity is due to the addition of a new auxiliary lane for shared traffic movement (through plus turning traffic) which performed almost similar role as the existing shared movement lane which was changed to a dedicated MBT short lane. On the other hand, a v/c ratio of 0.734 was recorded for DS2b. This represents a capacity improvement of 34% for DS2b. The improvement in capacity for DS2b is because of the increased g/C ratio (0.47) which provided more green time to the east bound traffic.

Similarly, the v/c ratio of vehicles in the straight movement lane was constant for DS2a and decreased for DS2b. The results show a constant v/c ratio of 1.005 for DS2a. This constant capacity for DS2a is due to the through traffic (which previously used the MBT lane) shifting to the new auxiliary lane hence no changes in traffic flow for the straight movement lane. On the other hand, a v/c ratio of 0.663 was recorded for DS2b. This represents a capacity improvement of 34% for DS2b. The improvement in capacity for DS2b is because of the increased g/C ratio which is providing more green time to the east bound traffic.

Lane Delay Evaluation

Figure 4-34 summarises delay associated with lane groups that were affected by the design strategy DS2 using PM peak hour traffic volume. The analysis uses changes in delay for the shared movement lane group and dedicated MBT lane group on the WB approach. The shared movement lane group comprised of a combination of straight movement traffic and left turning traffic. On the other hand, the dedicated MBT lane group contained MBT traffic making straight traffic movement. For each lane group, the average vehicle delay of existing conditions was compared to the average vehicle delay under DS2a and DS2b design scenarios.

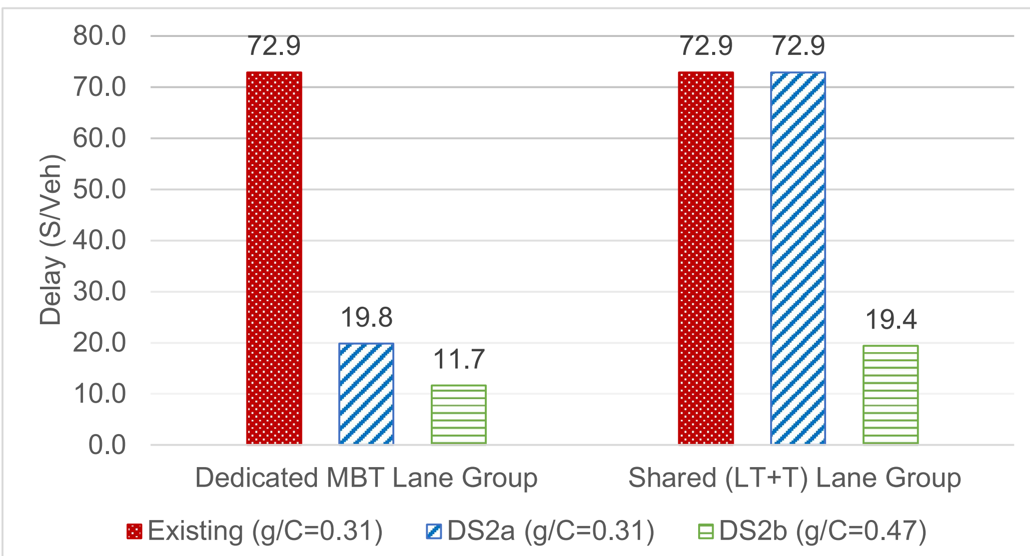


Figure 4-34: Impact of DS2 on Delay (PM) for J2 EB Approach

The results indicate that DS2a improved the average delay for MBT traffic with no changes to the delay for traffic in the shared lane group. The delay for MBT traffic improved from 72.9 seconds to 19.8 seconds for DS2a representing a delay improvement of 73%. The improvement is attributed to the removal of all other vehicles to the new shared traffic lane which is part of the shared lane group. Delay for the traffic in the shared lane group was constant even after the removal of MBT vehicles because of the small percentage of MBT which did not have a significant impact.

On the contrary, DS2b improved the average delay for traffic in both the dedicated MBT lane group and the shared traffic lane group. Specifically, the average delay improved from 72.9 seconds to 11.7 seconds for MBT lane group and from 72.9 seconds to 19.4 seconds for shared traffic lane group. This represents 84% and 73% delay improvement for traffic on the MBT lane group and share traffic lane group respectively. The increase in g/C ratio for DS2b enabled more green light for EB traffic hence improvement in the average vehicle delay for dedicated MBT lane group and shared lane group.

Delay Evaluation for EB Approach and Overall Intersection

Figure 4-35 summarises average vehicle delays for the east bound approach and overall intersection. In both situations, the analysis uses PM peak hour traffic to compare average vehicle delays for the existing scenario to the delays for the DS2a and DS2b scenarios.

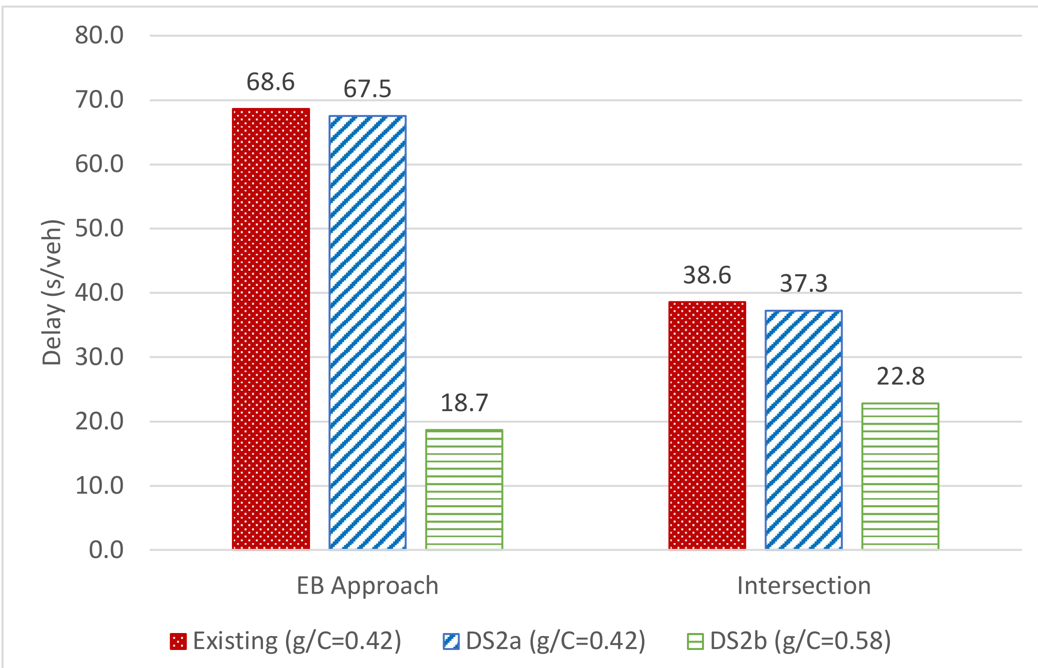


Figure 4-35: Impact of DS2 on Delay (PM) for J2 EB Approach and Overall Intersection

The results show that DS2 design strategies improved the overall vehicle delay for the approach and intersection. The delay for the east bound approach improved from 68.6 seconds to 67.5 seconds for DS2a and to 18.7 seconds for DS2b. This represents an average delay improvement of 2% and 73% for DS2a and DS2b respectively.

Similarly, the average delay for the entire intersection improved for both design strategies. The average delay improved from 38.6 seconds to 37.3 seconds and 22.8 seconds for DS2a and DS2b respectively. This represents an improvement of 3% and 41% for DS2a and DS2b respectively.

f) The v/c Ratio and Delay Analysis for the North Bound (NB) and South Bound (SB) Approaches

To determine the impact of design strategy DS2 on lane capacity and delay associated with the north-south corridor, evaluation was conducted using the worst impacted lane on both NB and SB approaches. The assessment for this corridor was done using the optimised design scenario DS2b where both geometric and signal settings were modified. In addition, traffic in the lane with the worst values of v/c ratio and delay under existing conditions were compared to respective traffic in the lanes or lane groups under the DS2b scenario.

The v/c Ratio Evaluation

Figure 4-36 shows changes in capacity for the worst affected lane by the DS2b on NB and SB approaches using both AM and PM peak hour traffic volumes. The v/c ratios for the existing conditions were compared to the respective v/c ratios for the DS2b design scenario. The AM peak traffic volumes were used for determining changes in the v/c ratio for the SB approach, whereas the PM peak traffic volumes were used for evaluating v/c ratios for the NB approach.

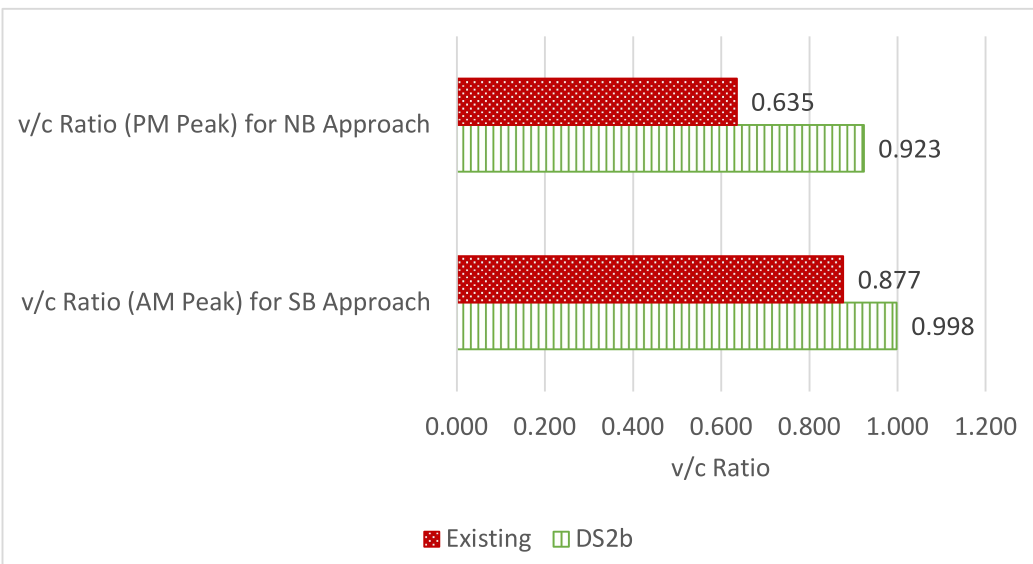


Figure 4-36: Impact of DS2b on v/c Ratio (AM&PM) for J2 NB and SB Approaches

The results indicate an increase in the v/c ratio for both AM and PM traffic conditions. The v/c ratio increased from 0.877 to a v/c ratio of 0.998 for the AM peak traffic and from 0.635 to 0.923 for the PM peak. This represents a capacity decrease of 14% and 45% for both AM and PM peak conditions. The increase in v/c ratios for the DS2b is a result of a shorter signal green time for traffic on the NB and SB approaches under DS2b design scenario. However, all v/c ratios were falling within acceptable levels of services defined for this study.

Delay Evaluation

Figure 4-37 shows changes in capacity for the worst affected lane groups on NB and SB approaches. The worst lane group delays for the existing conditions were compared to respective lane group delays under the DS2b scenario. The AM peak traffic volumes were used for determining changes in delays for the SB approach, whereas the PM peak traffic volumes were used for evaluating delays for the NB approach.

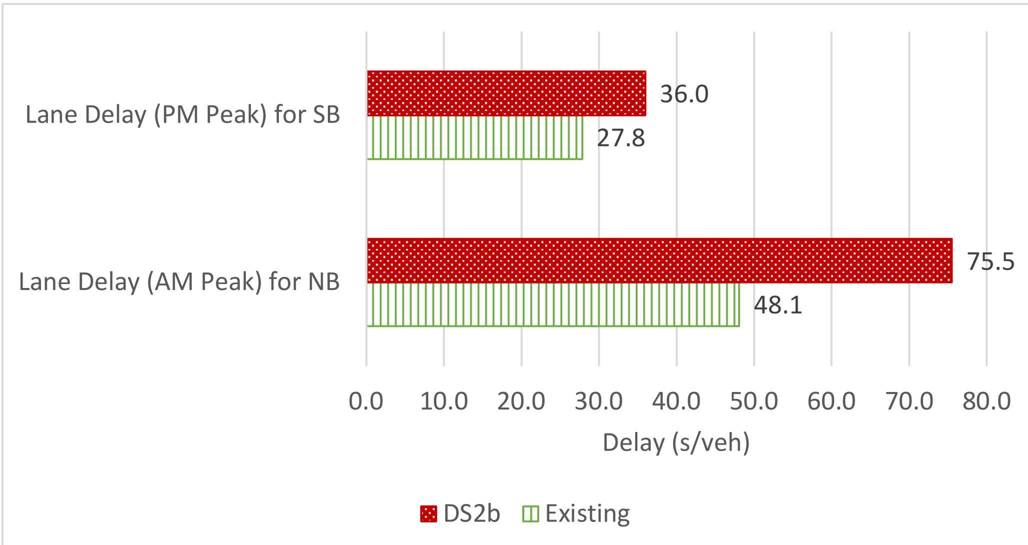


Figure 4-37: Impact of DS2b on Delay (AM&PM) for NB and SB Approaches

The results indicate an increase in delay on the worst impacted lane under both AM and PM peak conditions. The average vehicle delay increased from 48.1 seconds to 75.5 seconds for the AM peak traffic conditions and from 27.8 seconds to 36.0 seconds for the PM peak traffic conditions. This represents a delay increase of 57% and 29% for AM and PM peak conditions respectively. The increase in delay is a result of a decrease in signal green time for traffic on the NB and SB approaches under the DS2b design scenario. On the other hand, all delay times were falling within acceptable levels of services defined for this study.

Checking Adequacy of Storage Length of a dedicated MBT Lane

Like for the shared MBT lane, this analysis was carried out on the approach of the target corridor with the highest peak hour traffic volumes in the dedicated MBT lane. Consequently, the analysis was done using the PM traffic volume in the shared MBT lane for the north bound approach. The AASHTO (2004) provides the formula (Equation 2) which is used for determining the storage lengths for auxiliary lanes as follows:

Length (L)= 1.5 x average number of vehicles that would store per cycle……..(Equation 2)

Table 4-6 uses Equation 2 to check the adequacy of the existing storage length of auxiliary lane to accommodate traffic for the shared MBT lane. The analysis uses the west bound AM peak hour traffic volumes in the MBT shared lane for a traffic signal with a cycle length of 81seconds.

Table 4-6: Storage Length Calculation for shared MBT Lane for J1

Peak Hr Traffic Vol (MBT), V_{MBT}	28 PCU/Hr
Cycle Time,C = Seconds	81 seconds
Average. Traffic per cycle (n)=(V_{MBT}/(3600/C))	0.64 PCU/Cycle = approx. 1 PCU/Cycle
Storage Length in PCU=(L)=(1.5*n)= (PCU)	1.5 PCU
Calculated Storage Length in m = $L_{(PCU)}$X Average size of passenger car (4.8)	7.2m
Recommendation	Provide a minimum of 30m storage length (CSIR,2000)

The results show that a storage length of approximately two vehicles (7.2m) would be required to accommodate the peak hour traffic. This suggests that there is enough storage space available to MBT traffic in the dedicated lane. However, to prevent traffic in the adjacent lanes from blocking the proposed dedicated MBT lane, a longer dedicated MBT lane would be recommended. A minimum storage length of 30m was recommended as provided in the human settlement planning and design guidelines (CSIR,2000).

4.7 DISCUSSION OF RESULTS

4.7.1 `Introduction

The previous sections have presented some important insights into the impacts of MBT design strategies on capacity and average vehicle delay for the targeted corridor. The sections also shed light on the performance of the design strategies in relation to all other intersection approaches and the entire intersections. The purpose of this section is to synthesize and interpret the findings to capture the importance and relevance of this research. It further discusses the implications of such results to the design and planning of priority infrastructure for MBT at intersections.

This part of the research as previously highlighted presented two main design strategies for MBT priority infrastructure that were evaluated in detail. The two priority infrastructure types included the provision of a shared MBT lane and the provision of a dedicated MBT lane. For each of these priority infrastructure types, three design scenarios were evaluated using peak hour traffic volumes. The analysis was done using the HCM methodology which gave results in the form of v/c ratio and average vehicle delays. The results were presented in graphical form for easy interpretation and capturing of any interrelationships.

4.7.2 Impact of design strategies on the performance of the intersection

The results confirmed the proposition that MBT priority infrastructure have the potential to improve the operation of MBT traffic. The results showed that both shared and dedicated MBT lanes greatly improved the performance of MBT vehicles by increasing lane capacity and reducing average vehicle delays by up to 28% and 32% for the shared MBT lane and dedicated MBT lane respectively. These reduction percentages in delay nearly doubled when the signal timings were optimised.

Two design strategies for the shared MBT lanes were evaluated. The first strategy involved converting the existing shared traffic lane (left turn and straight movements) into a shared MBT lane (left turning movement and through MBT movement). The results suggest that by removing the straight (T) traffic from the existing shared (LT+T) lanes, more capacity was created which subsequently helped to reduce the overall average vehicle delay for the turning (LT) traffic and the straight MBT traffic. The second strategy involved converting the left turning (LT) lane into a shared MBT lane (LT+MBT) under DS1. The results suggest that the average MBT delay was greatly improved due to less volume of left turning (LT) traffic which subsequently left more capacity for MBT traffic. This option greatly depends on the existing performance of the traffic on the left turning (LT)lane and straight (T) lane. Specifically, the results suggest that the design strategy may not be ideal in a situation where the existing v/c ratio and delays on the left turning (LT) traffic lanes are higher than existing performance on the through (T) lanes as MBT traffic would prefer to use the existing straight (T) lanes in such situations.

Similarly, the results for dedicated MBT lanes (DS2) suggest a significant improvement in the performance of MBT traffic. The v/c ratios and average vehicle delay for MBT traffic were greatly improved with the provision of dedicated MBT lanes due to the removal of straight (T) from using the lanes. This created more capacity and hence reduced average vehicle delays for MBT traffic.

In addition, results for shared (DS1a) and dedicated (DS2a) MBT lanes respectively suggest that there is available capacity within the existing traffic signal designs that can be utilised for MBT priority infrastructure. The results for the design strategies show that most of the delay and v/c ratio values for other traffic were falling within acceptable limits as defined in this study. However, these results also suggest that the performance of other traffic other than MBTs were greatly impacted by both DS1a and DS2a design strategies. In this regard, the results show that the v/c ratio and delay for other traffic other than MBT was greatly increased during DS1a and DS2a design strategies. This suggests a need for optimised design to achieve maximum benefits from these MBT priority infrastructure. This was achieved through design strategies DS1b and DS2b.

The results for DS1b and DS2b suggest that optimal performance of the priority infrastructure could be achieved through both geometric modifications as well as traffic signal optimisation. The results show a great improvement in performance for both MBT traffic and other cars. This optimal performance is achieved not only on traffic for the targeted corridor but also on other corridors crossing the MBT corridors. These results also suggest that for the best performance outcome, the performance analysis should therefore not only focus on the traffic for the targeted intersection approaches but also on all other approaches of an intersection.

On the other hand, results for storage lengths suggest that the storage lengths are also a critical design parameter that needs to be adequately considered when designing MBT priority lanes. The sizes of storage lengths largely depend on traffic volumes for the shared MBT lanes because shared MBT lanes normally take higher traffic volumes (MBT+LT) than dedicated MBT lanes which are only designed for MBT traffic. Due to less traffic volumes of MBT for dedicated MBT lanes, it has been suggested that the overall lengths of dedicated lanes should therefore be based on prevention of blockage by traffic from adjacent lanes.

In general, the results have also shown that both the shared MBT and dedicated MBT priority infrastructure improve the operation of minibus-taxis at intersections. The results have also shown that some design strategies for these choices of MBT priority facilities can be implemented without geometric modifications while other design strategies would require geometric upgrades. The design strategies that would require geometric improvements such as the addition of new lanes would also need extra space to accommodate the upgrades. To achieve the best traffic operation results for these priority infrastructure at intersection for all traffic vehicles on all approaches. To maximise the benefits of both MBT priority infrastructure as observed in this analysis, this study recommends using the third design scenarios which demands the modification of the intersection geometry as well as the optimisation of the existing traffic signals.

4.8 SUMMARY OF THE CHAPTER

This chapter provided a performance analysis of design strategies for the MBT priority infrastructure. Two intersections and two priority infrastructure types were selected from an initial sample of four intersections. The two priority infrastructure types that were evaluated included a shared MBT lane and a dedicated MBT lane. For each priority infrastructure, three design scenarios were considered. First, the ‘do nothing’ scenario which included evaluation of existing intersections without modification to the geometric and traffic signal conditions.

Second, evaluation of intersection after modification of geometric conditions. Third, evaluation of the modified intersections with optimised signalisation. The HCM method was utilised to perform capacity analysis. Capacity performance analysis provided results for traffic in both priority and non-priority lanes and how they affected the performance of the targeted approach and hence the entire intersection. In general, the results showed great improvement in v/c ratio (capacity) and delay performance of MBTs traffic which were given priority. On the other hand, traffic performance decreased for the traffic which operated on the lanes without priority. The study has found that for both priority infrastructure, the third design scenario provides the best performance results. This is an option which demands modification of intersection geometry together with optimisation of the traffic signals. The results for this design scenario showed a great improvement for traffic in priority and non-priority lanes, as well as for the entire intersections. An evaluation on storage length was also conducted to determine the adequacy of the existing lanes. The results show that the existing storage lengths for dedicated MBT lanes were adequate to accommodate MBT traffic while storage lengths for the shared MBT lanes were slightly insufficient. Consequently, the study recommended to increase the existing storage lengths of shared MBT lane to accommodate the peak traffic volumes.

Overall, the results for this section provide further evidence that the MBT priority infrastructure could indeed greatly improve the operation of MBT traffic by reducing average delays. However, these facilities come with a proportional reduction in performance for the non-prioritised traffic which could be improved through signal optimisation.

5 GRAPHICAL DESIGN MODELS FOR MBT PRIORITY INFRASTRUCTURE

5.1 INTRODUCTION

Results from Chapter Four have shown that the two evaluated MBT priority infrastructure could be beneficial in reducing delays for MBT traffic. This chapter develops graphical design models for the MBT priority infrastructure discussed in Chapter Four. These models define the ranges of traffic volumes where the implementation of the MBT priority infrastructure could become feasible. A sensitivity analysis is performed on the HCM models used in Chapter Four to show the relationship between the traffic volumes and the capacity of the approaching lanes to accommodate MBT priority infrastructure. These sensitivity analyses are set to measure the highest value of the v/c ratios by varying the straight traffic (T+MBT) and the left-turning (LT) traffic. The first part of the chapter covers the methodology, the key assumptions, and the setup of the models. The second part of the chapter provides the output (results) of the models. The outputs are in the form of the graphs or charts developed for predicting viable ranges of traffic volumes for MBT priority infrastructure. The last part of the chapter uses the volume ranges from the graphical models to determine the maximum storage lengths associated with the evaluated MBT priority lanes.

5.2 METHODOLOGY

This section presents the research design adopted for developing the models, the assumptions used, and the analytical procedure that was followed.

5.2.1 Research Design

This part of the study used the quantitative method to develop graphical models for predicting the range of viable traffic conditions for MBT priority infrastructure. The model setup adopted the HCM-based evaluation method already discussed in Chapter Four. A sensitivity analysis was used to vary the input variables of traffic volumes to determine the changes in capacity in the form of v/c ratios. The method is graphical in the sense that the outputs were presented in graphical form. The traffic volume limits from the graphical models were used to determine maximum feasible storage lengths for the MBT priority facilities.

5.2.2 Model Design

From the performance evaluation in Chapter Four, several key points and factors which affect the v/c ratio were identified. These factors played a key part in formulating the assumptions used for setting up the predicting models. The factors are summarised below:

- g/C ratio: It was shown that the v/c ratios for both design strategies were largely influenced by the g/C ratio. Specifically, the modified design strategies with optimised traffic signals (DS1b and DS2b), significantly improved the capacity for both priority and non-priority lanes. The increase in v/c ratio was due to the high g/C ratios associated with these DS1b and DS2b which led to the increase in the green time for traffic in the targeted corridor.
- Left turning traffic: The v/c ratio for the left-turning (LT) vehicles in the shared MBT lane (MBT+LT) under DS1a decreased significantly after the removal of straight (T) vehicles (J1 south bound approach). On the other hand, the v/c ratio for left turning (LT) traffic increased for DS2a due to the additional straight (T) traffic which was moved from the dedicated MBT lane on the west bound of J2. This suggests the need for further analysis that could estimate the ranges of the LT traffic volumes that would give the acceptable v/c ratios in both design scenarios.
- Straight Traffic: The analysis found that the v/c ratio for traffic in the straight (T) lane and shared (T+LT) lane significantly increased on both approaches of J2 for the DS2a scenario. This increase in v/c ratio was due to the straight (T) from the MBT dedicated lane. Similarly, the v/c ratio for straight (T) vehicles under DS1a increased due to the straight (T) traffic which came from the auxiliary shared MBT lane. This suggests that with very high straight traffic volumes, the v/c ratios for traffic in the straight (T) lanes and shared (LT+T) lanes could likely go beyond the acceptable limits. This finding therefore calls for a further analysis which could define ranges of the straight (T+MBT) traffic volumes that would give acceptable v/c ratios.

Analysis Assumptions

In addition to the HCM assumptions made in Chapter Four, the following key assumptions were considered and used for the data analysis of the MBT priority infrastructure:

- Maximum v/c ratio of 1.0: The v/c ratio should not exceed a value of 1.0 for the traffic in any of the approaching lanes on the targeted corridor of the MBT priority infrastructure. The v/c ratio of 1.0 represents the maximum v/c ratio associated with level of service (LOS) of E (COTO, 2014). The findings in Chapter Four have shown that that the MBT priority infrastructure comes with a sharp increase in the v/c ratio for traffic in the non-priority lanes hence a need to set this limit. This ideally means any combination of traffic volumes which could produce critical v/c ratios of equal to or less than 1.0 were considered to fall within the acceptable range for the MBT priority infrastructure. On the other hand, the traffic volumes for the targeted MBT corridors producing the v/c ratios of greater than 1.0 were considered not feasible for the MBT priority infrastructure.

- A minimum lane v/c ratio of 0.6: A lower limit of v/c ratio equal to 0.6 was assumed for any traffic in the approaching lanes of the targeted corridor. This means that where the lane v/c ratios of less than 0.6 for either straight (T or MBT) traffic or turning (LT) traffic were considered to operate already at an acceptable level of service hence no need to provide MBT priority lanes. A v/c ratio of less than 0.6 is associated with the level of service (LOS) of A or B (Othayoth & Rao, 2019; COTO 2019) which are considered as acceptable levels of service for MBT traffic hence no need for further improvements.
- Percentage of the MBT Traffic: The traffic count results in Chapter Four also showed that straight MBT traffic on all approaches were ranging from 3% to 7% of total straight traffic (MBT+T). The percentages of the MBT traffic could easily exceed this range if intersections with higher minibus-taxi volumes were sampled. For this reason, a 10% of the straight (MBT+T) traffic was assumed to estimate the MBT traffic volumes.
- Exclusive right turning (RT) traffic: The analysis also assumed that all targeted approaches should have the exclusive right turning lanes to eliminate any possibility of traffic operational conflict between the straight (T+MBT) and right turning (RT) traffic.
- Lane Geometry for the DS1 (Model Design 1): The following was the assumed geometry on the target approach for the shared MBT lanes (Model Design 1): one straight (T) lane, an auxiliary shared MBT lane (MBT+LT), and one exclusive right turning (RT) lane. Figure 5-1 shows the layout plan representing model design 1 for the shared MBT lane. The shared MBT lanes are on the approaches of the North-South corridor. On both approaches, the three targeted lanes include the shared MBT lane (MBT+LT), straight traffic lane (T) and the auxiliary right turning (RT) lane.

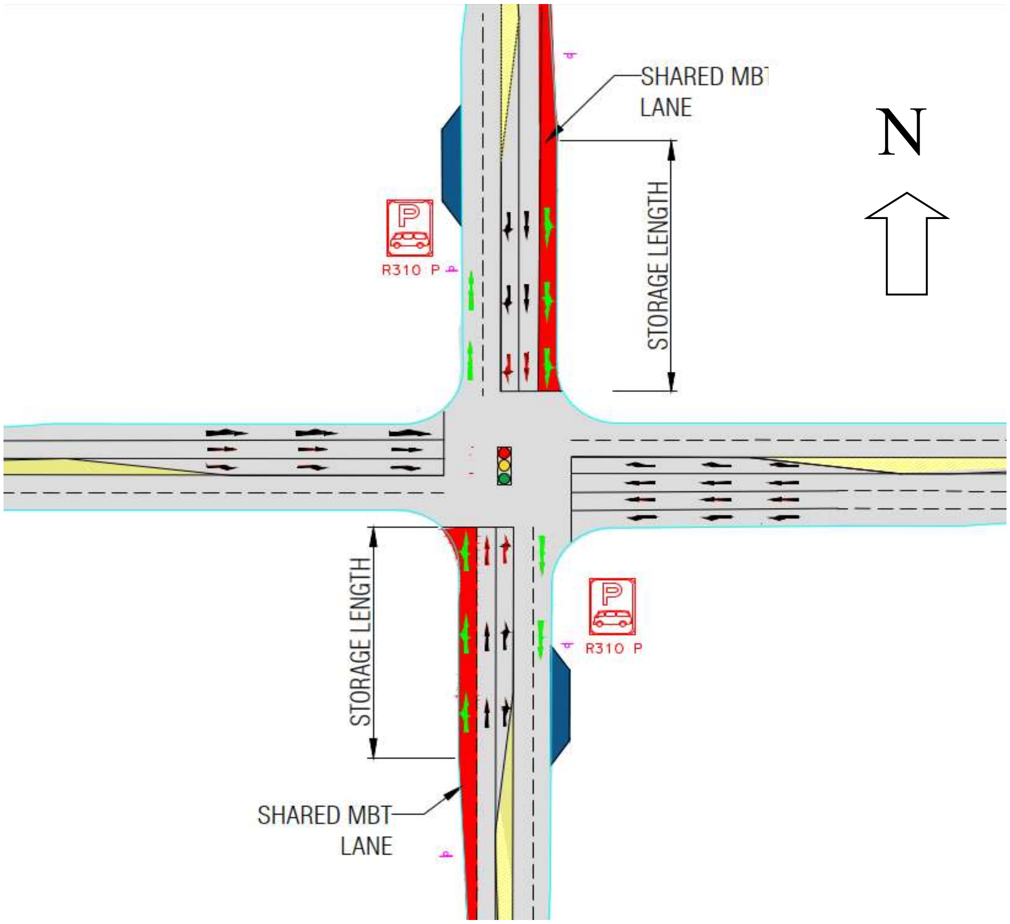


Figure 5-1: Layout Plan of Model Design 1 for Shared MBT Lane on N-S Corridor

- Lane Geometry for DS2 (Model Design 2): The following was the assumed geometry on the target approach for the dedicated MBT lanes (Model Design 2): one auxiliary shared lane (T+LT), one dedicated MBT lane, one straight traffic (T) lane, and one exclusive right turning (RT) lane. Figure 5-2 shows the layout plan representing the model design 2 for the dedicated MBT lane. The dedicated MBT lanes are on the approaches of the East-West corridor. Both approaches have four lanes which include the following: shared auxiliary lane (T+LT), dedicated MBT lane (painted RED), through lane (T) and exclusive right turning lane (RT).

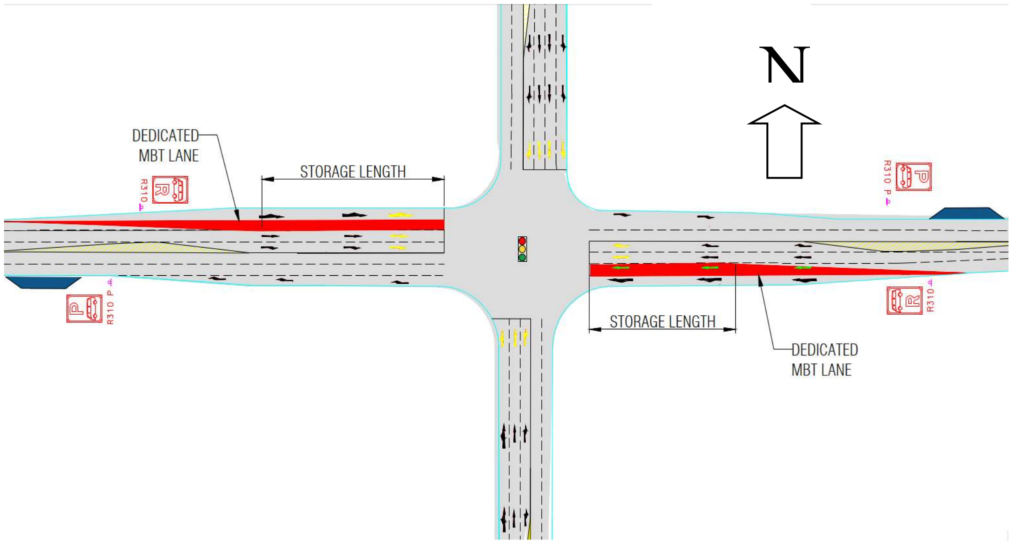


Figure 5-2: Layout Plan of Model Design 2 for Dedicated MBT Lane on E-W Corridor

- Summary of assumptions and adjustment factors: Table 3.5 summarises the assumptions and adjustment factors which were used for the evaluation. This is a summary of the assumptions and the adjustment factors with fixed values that were used when setting up the modified HCM model.

Table 5-1: Key Assumptions and adjustment factors for the analysis

Description of Key Assumptions	Fixed values used
Upper limit for v/c ratio	1.0
Lower limit for v/c ratio	0.6
Percentage of MBT traffic	10% of total straight (T+MBT) traffic on the targeted approach
Lane width for all lanes	3.5m
Intersection approach gradient	0%
Intersection left turning radius	12m
MBT Traffic	10% of total straight traffic (T+MBT)
Peak Hour Factor (PHF)	0.85
Base Saturation Flow (So) for LT traffic	1900 PCU/Hr/Lane
Base Saturation Flow Rate (So) for Straight (T+MBT) traffic	2000 PCU/Hr/Lane
Adjustment factor for bus or MBT blockage (f_{bb})	1.0
Adjustment factor for area type (f_a)	0.9 for urban area

5.2.3 Sensitivity Analysis Procedure

The analysis began by setting up the modified HCM model using the assumptions and the adjustment values in Table 5-1. Microsoft Excel was used to set up the entire spreadsheet which would permit varying three input variables (g/C ratio, LT traffic and T+MBT) while measuring the highest v/c ratio from the traffic in the approaching lanes. The subsequent sections explain the set-up in detail.

a) The g/C ratios

The g/C ratio of 0.2 is considered the most practical minimum g/C ratio that can be used for signal designs. The most prevailing g/C ratios range between the g/C ratio of 0.2 and 0.6. To determine the impact of g/C ratios, each model was setup using four g/C values of 0.2, 0.3, 0.4 and 0.5.

b) Traffic Volumes

The models were set up based on the type of MBT priority infrastructure under consideration. For a shared MBT lane, the traffic on the priority lane included the turning traffic and the straight MBT traffic (LT + MBT). For each constant left-turning traffic on the shared MBT lane, a range of straight traffic volumes was varied to determine the impact on the v/c ratios. The output was in the form of the highest v/c ratio associated with traffic in the approaching lanes. Graphs were then plotted to show the relationship between traffic volumes and v/c ratios at constant g/C ratios.

Traffic volumes of less than 50 PCU/Hr for both straight and turning traffic were considered too low for consideration of MBT priority infrastructure. Hence, a minimum value of 50PCU/Hr for both left turning (LT) and straight (MBT+T) traffic. For each constant volume of the turning traffic (LT), the straight (MBT+T) traffic volumes were varied between 50PCU/Hr and 1600PCU/Hr. For each g/C ratio, analysis was carried out over eight constant values of LT traffic (between 50PCU/Hr and 700PCU/Hr). In a situation where the measured v/c ratios were less than v/c =1.0, these traffic volume ranges were extended until the v/c ratio of 1.0 was exceeded.

Model Design 1

Table 5-3 and Figure 5-3 summarise the inputs variables for the Model Design 1 which were used for developing graphical models for the shared MBT lane. The table shows the four constant g/C ratios which were used. For each constant g/C ratio, the table also shows the corresponding constant LT values that were tested and a range of straight traffic volumes that were varied for each LT value.

Table 5-2: Inputs for Model Design 1 used for Shared MBT Lane (DS1) Analysis

Design Strategy	Constant values of g/C Ratios	Constant values of LT (PCU/Hr)	Range of Straight Traffic (T+MBT) (PCU/Hr)
DS1	0.2, 0.3, 0.40, 0.50	50,100,200,300,400, 500, 600, 700 for each constant value of g/C Ratio	From 50 to 1600 for each constant value of LT

Figure 5-3 provides the schematic set-up of the Model Design 1 (Shared MBT lane). Column 'A' represents constant g/C ratio under consideration. For each constant g/C ratio, the left-turning (LT) traffic (column 'B') were varied from 50 PCU/Hr to 700 PCU/Hr. Column 'C' represents ranges of straight traffic which were varied for each constant value of left- turning traffic. The model output (column 'D') was set up to measure the highest value of v/c ratio of traffic in the targeted individual approaching lanes. For each constant g/C ratio, graphs of the v/c ratio versus straight (MBT+T) traffic were plotted at constant left running (LT).

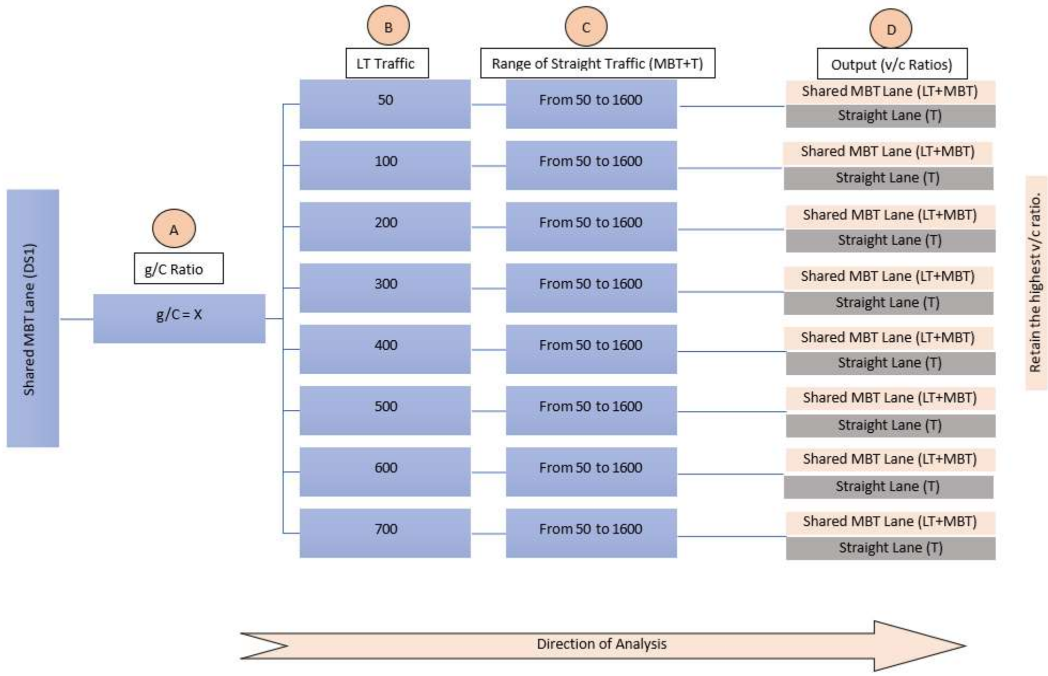


Figure 5-3: Model Design 1 for DS1 Capacity Analysis

<u>Model Design 2</u>

The Model Design 2 was set up for the analysis of the dedicated MBT lane. Like the Model Design 1, a range of straight (MBT+T) traffic volumes were varied to determine the v/c ratios at LT traffic and constant g/C ratio. The model was also setup to measure the highest v/c ratio of traffic from the individual approaching lanes. Eight constant values of left turning (LT) traffic between 50 PCU/Hr to 700 PCU/Hr we used in the analysis. On the other hand, the straight traffic volumes (T+MBT) were varied from 50 PCU/Hr to 1600 PCU/Hr. These variables are shown in Table 5-3. The table shows ranges of the constant values of the g/C ratios and the left turning traffic used in the analysis while varying traffic volumes of the straight traffic (MBT+T).

Table 5-3: Inputs for Model Design 2 used for Dedicated MBT Lane (DS2) Analysis

Design Strategy	Constant values of g/C Ratios	Constant values of LT (PCU/Hr)	Range of Straight Traffic (PCU/Hr)
DS2	0.2, 0.3, 0.42, 0.58	50,100,200,300,400, 500, 600, 700 for each constant value of g/C Ratio	From 50 to 1600 for each constant value of LT

Figure 5-4 provides the set-up of the Model Design 2 (Dedicated MBT lane). The analysis procedure was similar to the one provided for Model Design 1. Column ‘A’ represents the constant g/C ratios used. For each g/C ratio, the left turning (LT) traffic volumes (column ‘B’) were varied from 50 PCU/Hr to 700 PCU/Hr. Column ‘C’ shows ranges of straight (MBT+T) traffic which were varied for each constant value of left turning traffic. The model output (column 'D') was set up to measure the highest v/c ratio of traffic from the targeted individual approaching lanes. Graphs of the v/c ratio versus straight (MBT+T) traffic were plotted for each constant LT traffic at constant g/C ratio.

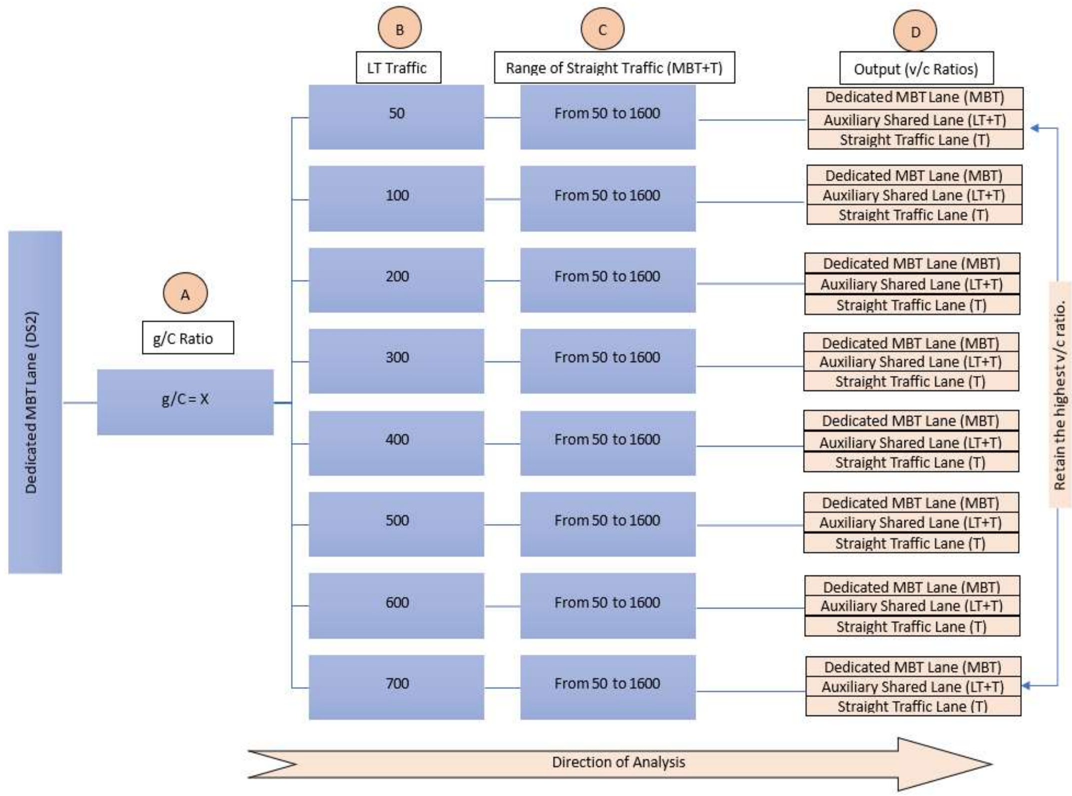


Figure 5-4: Model Design 2 for DS2 Capacity Analysis

After all models were set-up and input variables were defined, data was then analysed in an Excel spreadsheet to produce graphs of v/c ratio versus straight (MBT+T) traffic. These graphs were used to define the feasible region i.e volume combinations which produced v/c ratios of between 0.6 and 1.0. Results from this analysis were used to plot graphs (charts) showing the relationship of the turning traffic (LT) and the straight (MBT+T) traffic at constant values of v/c and g/c ratios. Finally, these ranges of feasible traffic volumes were used to estimate maximum storage lengths associated with the MBT priority lanes. Figure 5-5 summarises the procedure used to develop the graphical models for predicting the feasible traffic volumes for the MBT priority infrastructure.

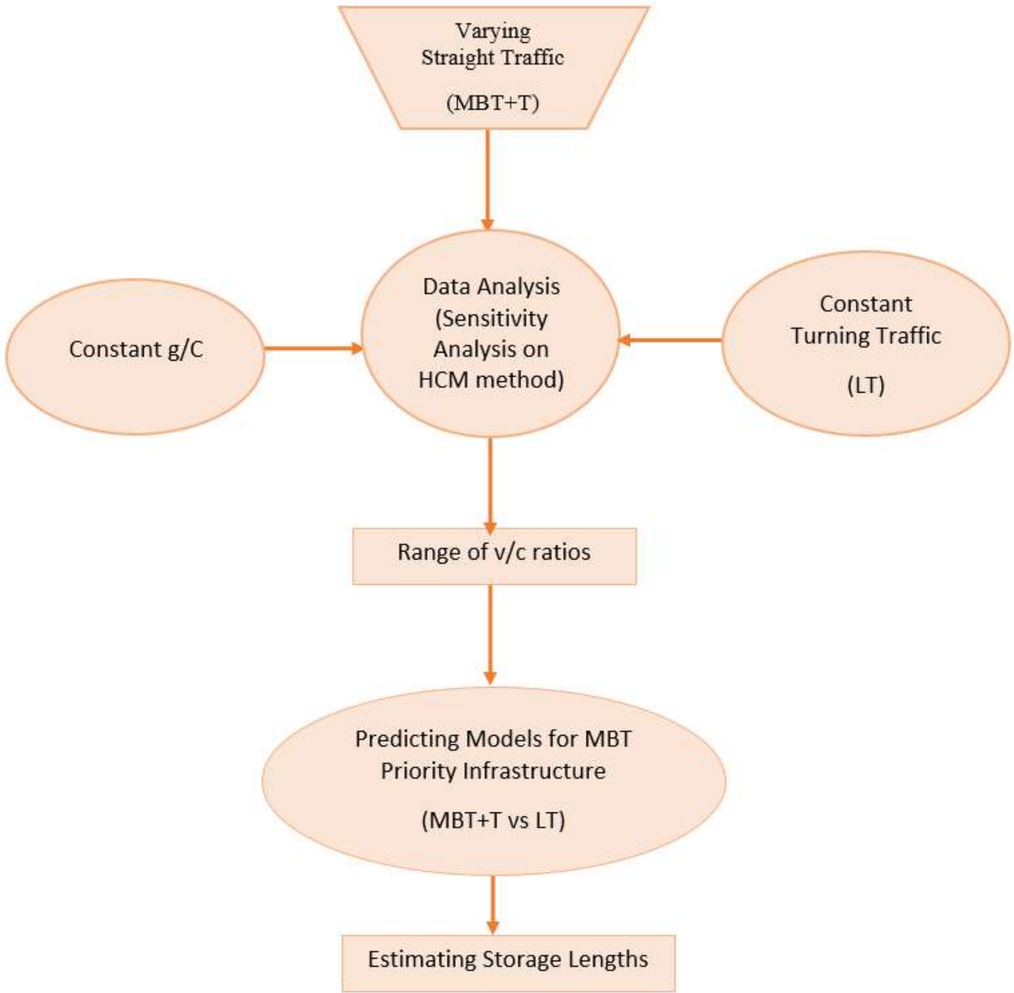


Figure 5-5: Procedure for Developing Graphical Models for Predicting Feasible Traffic Volumes for MBT Priority Infrastructure

5.3 OUTPUT FOR MODEL DESIGN 1

Model Design 1 involved the provision of the graphical models for predicting feasible traffic volumes of the shared MBT lane to allow MBT traffic to share the lane with left turning traffic while the other straight traffic (T) use the remaining straight (T) lane. Overall, traffic in two approaching lanes were targeted for the analysis and these included the shared MBT lane (MBT+LT) and the straight (T) lane. The model output has provided the relationship between the v/c ratios, the g/C ratios, the straight (T+MBT) traffic and the left turning (LT) traffic. The subsequent sections give the summary of the results at constant values of the g/C ratios.

a) Charts for v/c ratios and traffic volumes at g/C = 0.50

Two different charts were used to show the relationship between traffic volumes and v/c ratios. Figure 5-6 shows graphs of the relationship between the v/c ratio and traffic volumes at a constant g/C ratio of 0.50. Each of the lines was plotted at a constant volume of left-turning traffic and shows the changes in v/c ratio as the volume of straight traffic (MBT+T) varied from 50 PCU/Hr to 1600 PCU/Hr. The output v/c ratios were obtained by measuring the highest v/c ratio of traffic from individual approaching lanes. The purpose of these graphs was to identify the feasible region associated with Model Design 1. The feasible region lied between v/c = 1.0 and v/c = 0.6 where volumes of straight (MBT+T) traffic were higher than volumes of left turning (LT) traffic.

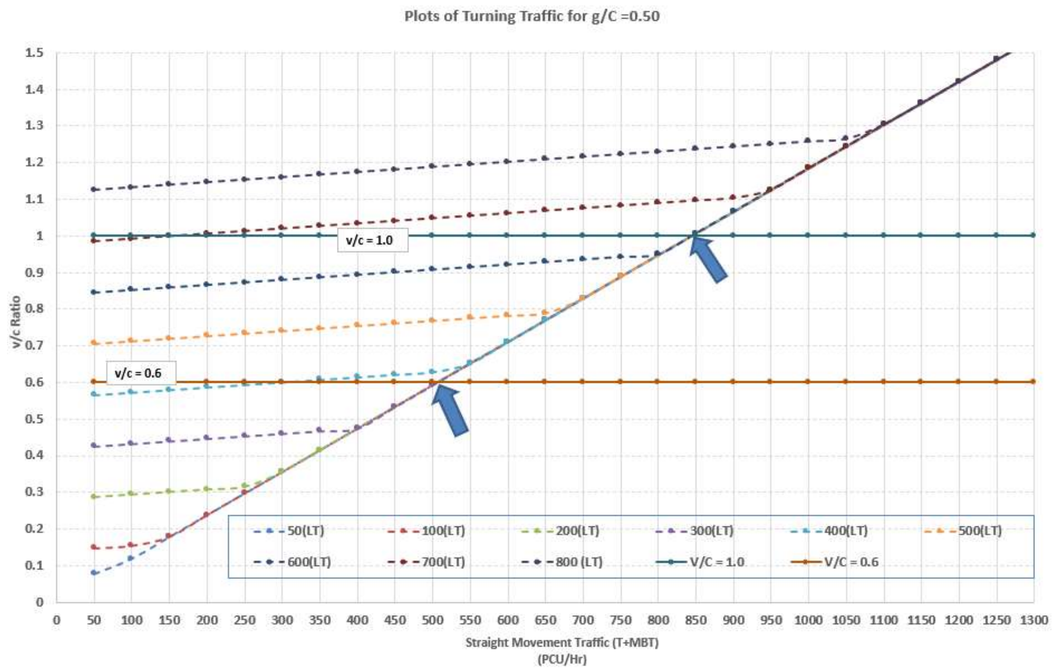


Figure 5-6: Changes in v/c ratios at g/C = 0.50 for Model Design 1

The results show that at a constant g/C ratio and for each graph of constant volume of the left turning traffic (LT), the v/c ratios increased proportionally with an increase in the volume of the straight traffic. The changes in v/c ratios were in two parts. The first part of each line was associated with v/c ratios for the shared MBT lane for which LT traffic was higher than the MBT+T traffic. For this part, the results show that the v/c ratios increased at a gradual rate and this gradual increase was a result of the MBT traffic volumes joining the LT traffic in the MBT+LT lane. The second part of each graph shows a rapid increase in v/c ratios. The v/c ratios for this part of the analysis were due to traffic in the straight (T) lane. Additionally, this second part represents situations where straight (T+MBT) traffic is higher than LT traffic. Specifically, the results indicate that at the v/c ratio=0.6, a T+MBT traffic volume of 520 PCU/Hr was feasible for the LT traffic ranging from 50 PCU/Hr to 380PCU/Hr. Similarly, at v/c ratio=1.0, the maximum straight (MBT+T) traffic of 845 PCU/Hr was feasible for a range of LT traffic between 50 PCU/Hr and 640 PCU/Hr.

Consequently, Figure 5-7 provides a simplified chart for predicting the feasibility of the MBT infrastructure for Model Design 1 by traffic volume ranges. The chart was developed to give a range of feasible traffic volumes at the constant values of the v/c ratios where the MBT+T is higher than the LT traffic. All combinations of traffic volumes falling between the graphs of v/c=0.6 and v/c=1.0 are deemed to be feasible whereas the traffic volume combinations falling outside these two lines were considered not feasible. The horizontal lines graphs suggest that the capacity of the shared MBT lane is influenced by volume of the straight (MBT+T) traffic. In addition, the graphs shows that a constant value of straight traffic could work for a wide range of LT traffic.

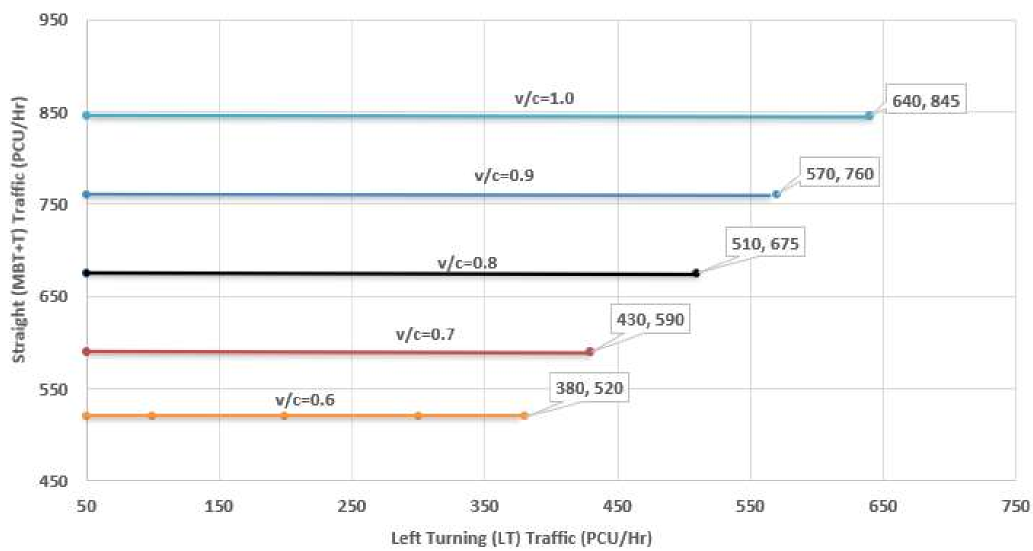


Figure 5-7: Chart of MBT+T Versus LT at g/C Ratio=0.5 for Model Design 1

b) Charts for v/c ratios and traffic volumes at g/C = 0.40

Figure 5-8 shows the graph of the relationship between the v/c ratio and the traffic volumes at a constant g/C ratio of 0.40. The graph was plotted in exact the same way as a).

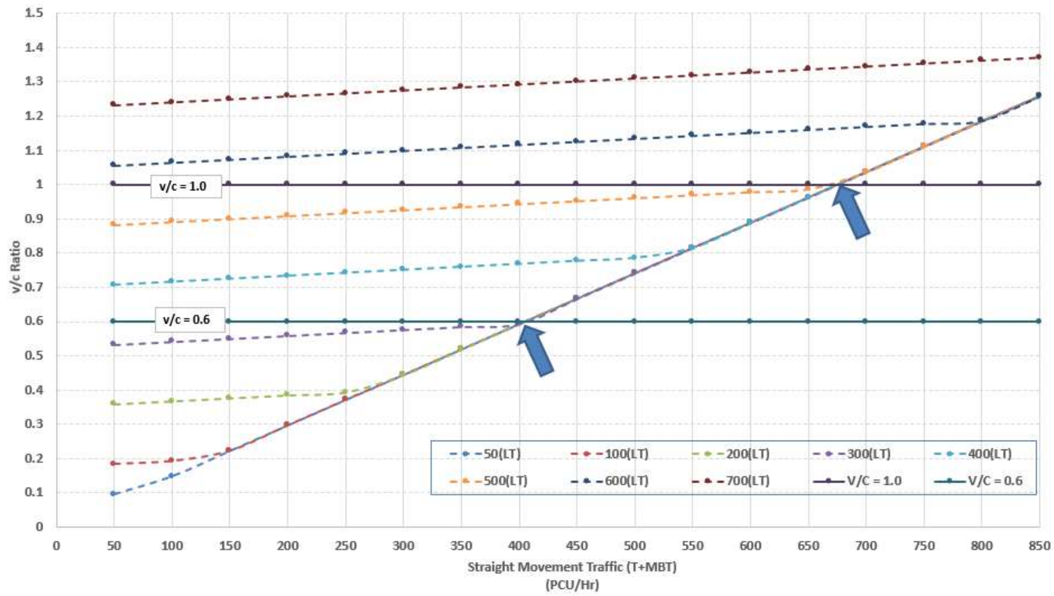


Figure 5-8: Changes in v/c ratios at g/C = 0.40 for Model Design 1

The results indicate that limits for traffic volumes at constant v/c ratios have reduced with decrease in g/C ratio. For example, at v/c ratio=0.6, a T+MBT traffic volume of 410 PCU/Hr was feasible for the LT traffic ranging from 50 PCU/Hr to 310 PCU/Hr. Similarly, at v/c ratio=1.0, the maximum straight (T+MBT) traffic of 675 PCU/Hr was feasible for a range of LT traffic between 50 PCU/Hr and 500 PCU/Hr.

Additionally, at constant v/c ratios, the decrease in limits of traffic volumes is illustrated using Figure 5-9.

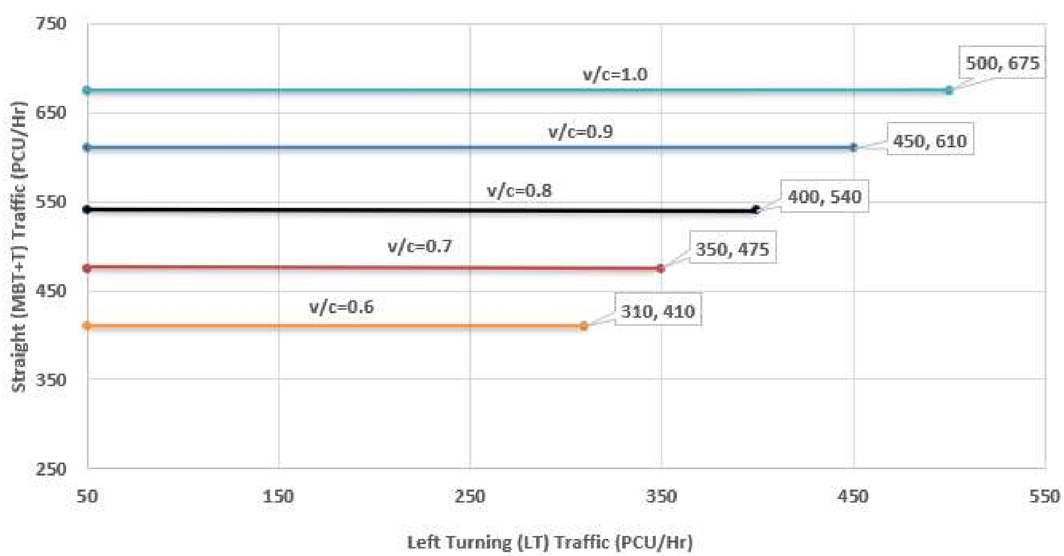


Figure 5-9: Chart of MBT+T Versus LT at g/C Ratio=0.4 for Model Design 1

c) <u>Charts for v/c ratios and traffic volumes at g/C = 0.30</u>

Figure 5-10 shows the graph of the relationship between the v/c ratios and the traffic volumes at a constant g/C ratio of 0.30. The graph was plotted in exact the same way as a) and b).

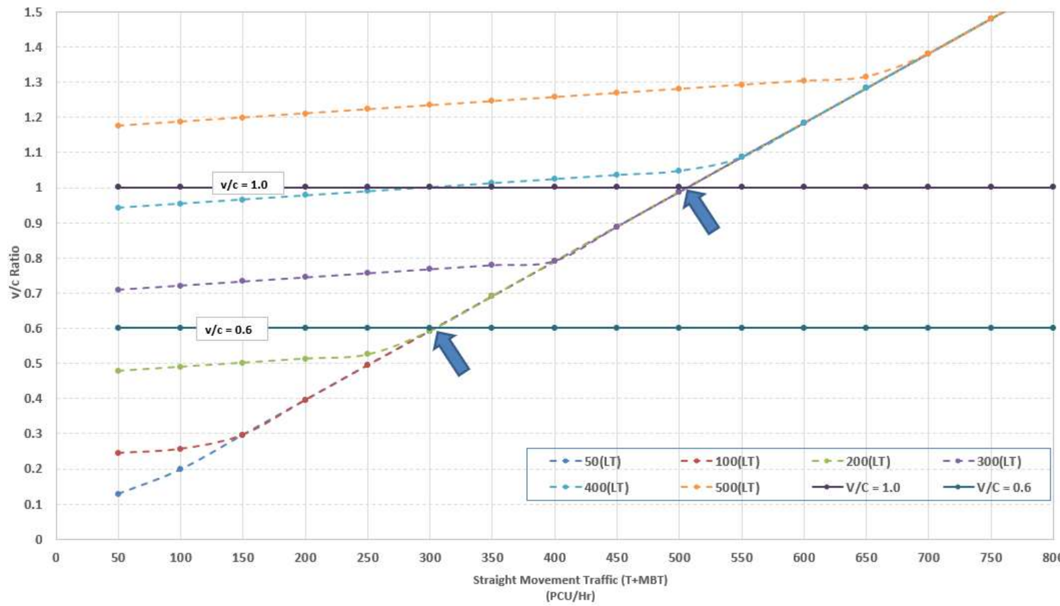


Figure 5-10: Changes in v/c ratios at g/C = 0.30 for Model Design 1

The results show that the traffic volume limits have further decreased with the decrease in the g/c ratio. For example, at a constant v/c ratio=0.6, a T+MBT traffic volume of 310 PCU/Hr was feasible for LT traffic ranging from 50 PCU/Hr to 230 PCU/Hr. Similarly, at a constant v/c ratio=1.0, the maximum straight (T+MBT) traffic of 510 PCU/Hr was feasible for a range of the LT traffic between 50 PCU/Hr and 375 PCU/Hr.

Additionally, at constant v/c ratios, the decrease in limits of traffic volumes is illustrated using Figure 5-11.

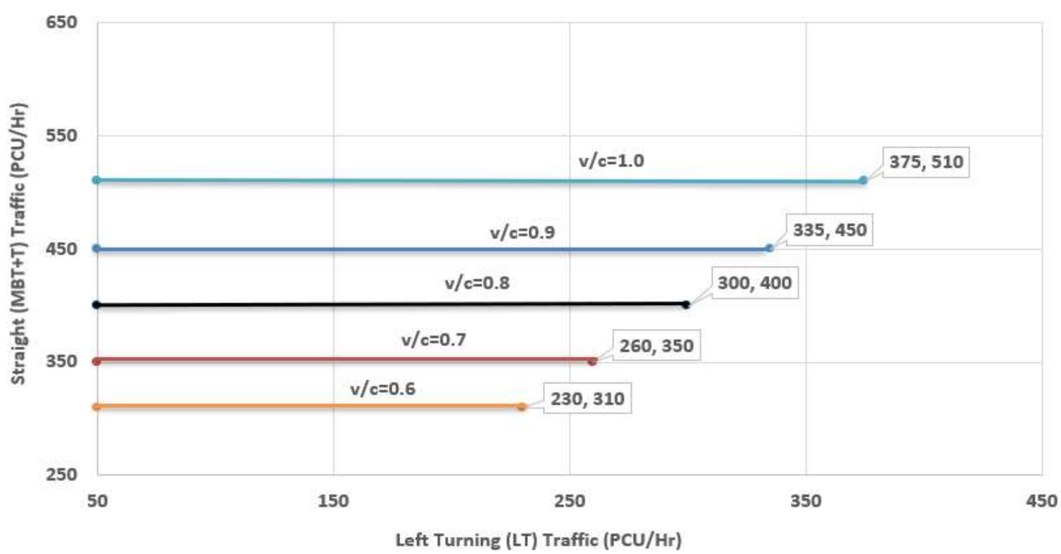


Figure 5-11: Chart of MBT+T Versus LT at g/C Ratio=0.3 for Model Design 1

d) Charts for v/c ratios and traffic volumes at g/C = 0.20

Figure 5-12 shows the graph of the relationship between the v/c ratios and the traffic volumes at a constant g/C ratio of 0.20. The graph was plotted in exact the same way as a), b) and c).

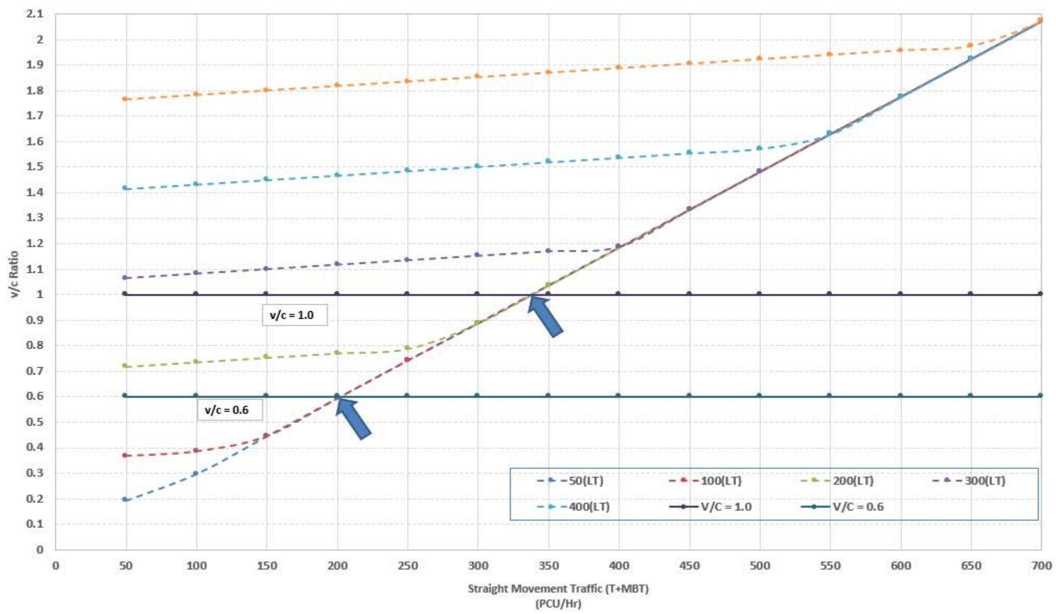


Figure 5-12: Changes in v/c ratios at g/C = 0.20 for Model Design 1

The results indicate that at the v/c ratio=0.6, a T+MBT traffic volume of 200 PCU/Hr was feasible for the LT traffic ranging from 50 PCU/Hr to 150 PCU/Hr. Similarly, at the v/c ratio=1.0, the maximum straight (T+MBT) traffic of 335 PCU/Hr was feasible for range of LT traffic between 50 PCU/Hr and 250 PCU/Hr.

Additionally, at constant v/c ratios, the decrease in limits of traffic volumes is illustrated using 5-13.

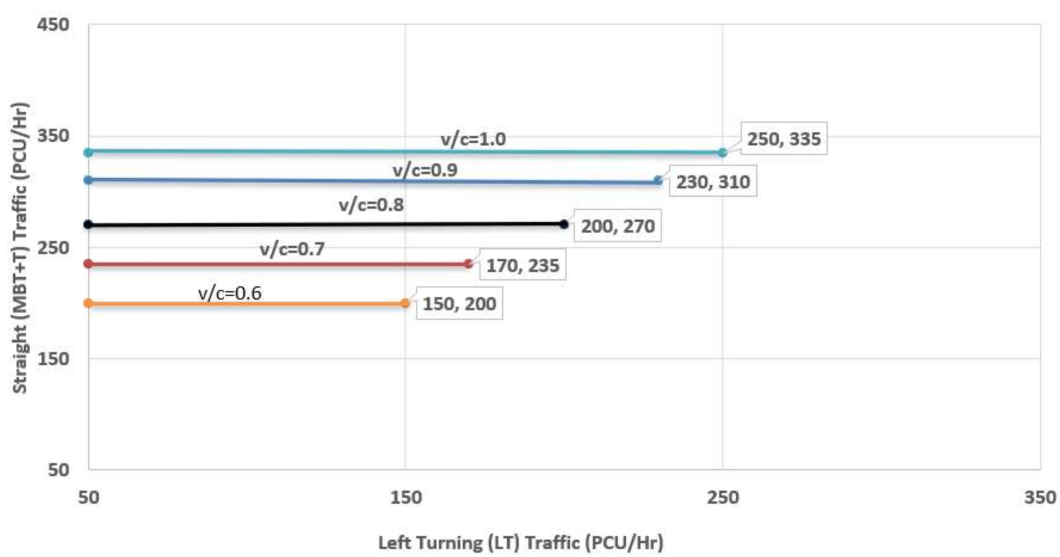


Figure 5-13: Chart of MBT+T Versus LT at g/C Ratio=0.3 for Model Design 1

5.3.1 Summary of Results for Model Design 1

The previous section presented the results associated with the Model Design 1 (shared MBT lane). The results have demonstrated that at a constant g/C ratio, the v/c ratio increases with an increase in the straight traffic (T+MBT) and left turning traffic (LT). In addition, it has been found that at a constant left turning traffic, the lines show that the v/c ratio tends to take a sudden rapid increase when the straight (T+MBT) traffic is higher than the left turning (LT) traffic. The results have also shown that the MBT traffic has a minimal impact on the overall v/c ratio of the shared MBT lane. This is shown by the small and the gradual change in the v/c ratios for situation when the LT traffic volumes are higher than the straight traffic volumes. Alternatively, the results suggest that the v/c ratio for the traffic in the straight lane is a critical parameter for predicting feasibility of the shared MBT lane. This is graphically shown by the large change, rapid and proportional increase in v/c ratios when straight (MBT+T) traffic volumes are higher than left turning (LT) traffic (a traffic condition which is prevalent for most intersections). Consequently, the v/c ratio for traffic in the lane was found to be critical in establishing the initial relationship between the volumes for the left turning (LT) traffic and straight (T+MBT) traffic. Graphs of v/c ratios and straight (T+MBT) traffic at constant volumes of the LT traffic were plotted for all the four g/C ratios considered in the analysis.

Using the relationship established from the initial graphs, simpler charts were developed to show the relationship between the left turning (LT) traffic and the straight (MBT+T) traffic at a constant v/c ratio and a constant g/C ratio. These were charts showing the feasible ranges of volumes for the left turning (LT) traffic and straight (MBT+T) traffic at constant g/C ratios between v/c ratio=0.6 and v/c ratio=1.0 in situations where the MBT+T traffic volumes were higher than LT traffic volumes.

Across these charts (between g/C ratios), it was also observed that the ranges of left turning (LT) and the straight traffic (MBT+T) volumes increased with the increase in the g/C ratios. The increase in the ranges of the traffic volume is a result of the higher green time interval as the g/C ratios increased. Overall, this section has developed graphs which could be used as design references for the shared MBT lanes. Specifically, these graphs could be used in predicting the feasibility of the shared MBT lanes when the ranges of traffic volumes at intersections are known. These charts could therefore be used for the planning and design of the shared MBT priority infrastructure.

5.4 OUTPUT FOR MODEL DESIGN 2

Model Design 2 involved the provision of the graphical models for the dedicated MBT lane to allow the straight MBT traffic to use at intersections while the other straight (T) traffic uses the auxiliary mixed (LT+T) lane and the remaining straight (T) lane. Overall, the model is set to accommodate four approaching lanes include: an auxiliary mixed traffic (LT+T) lane, a dedicated MBT lane, a straight (T) lane and an exclusive right turning (RT) lane. The model output provides the relationship between the v/c ratios, the g/C ratios, the straight traffic (T+MBT) and left-turning (LT) traffic. This relationship is shown using graphs at constant g/C ratios.

a) Charts for v/c ratios and traffic volumes at g/C = 0.50

Two different charts are used to provide the relationship between the traffic volumes and the v/c ratios at constant g/C ratio. Figure 5-14 shows graphs of the v/c ratio versus traffic volumes at constant g/C ratio of 0.50. Each of these lines was plotted at a constant volume of the left turning (LT) traffic and shows changes in v/c ratio as the volume of the straight traffic (MBT+T) increased from 50 PCU/Hr to 1600 PCU/Hr. The model output was set to measure the highest v/c ratios of traffic from the individual approaching lanes. Three approaching lanes were targeted, and they included an auxiliary mixed traffic lane (LT+T), a dedicated MBT lane and a straight (T) traffic lane. The purpose of these graphs was to identify the feasible region (shaded region) associated with the Model Design 2. The feasible region was defined between the v/c = 1.0 and v/c = 0.6 in situations where traffic volumes of straight (MBT+T) were higher than left turning (LT).

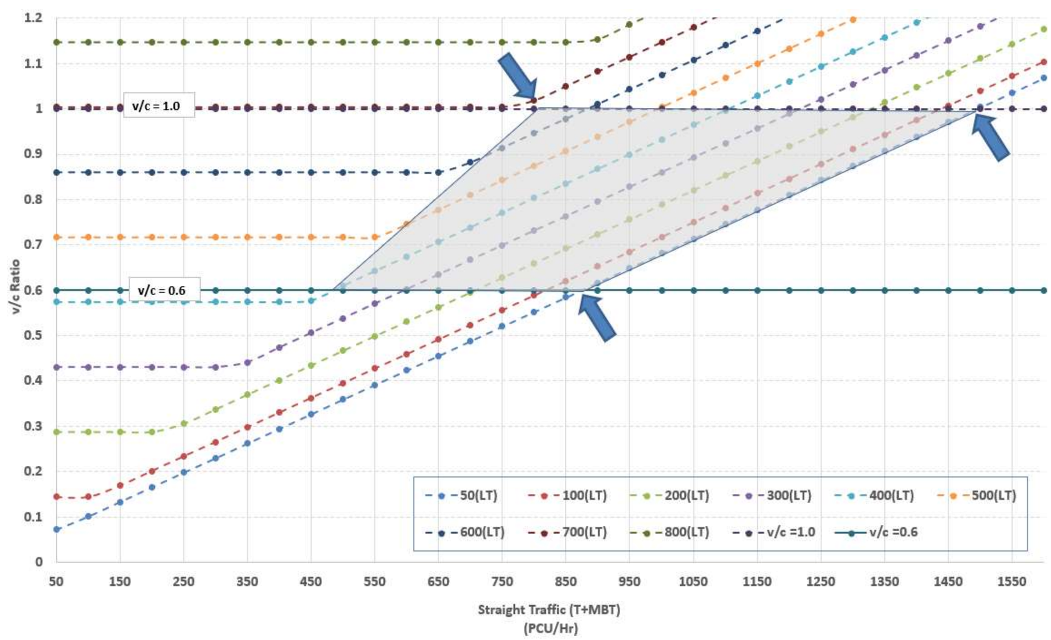


Figure 5-14: The feasible region and changes in v/c ratio at g/C=0.5 for Model Design 2

The shaded areas indicate the region for which the volumes of the LT traffic are less than the T+MBT traffic. The results show that for each graph of a constant volume of (LT) traffic, the v/c ratios increase proportionally with increase in volume of straight traffic (T+MBT). The rate of increase in the v/c ratios is shown in two main parts. The first part shows that the v/c ratio is constant for situations when volume of LT traffic is higher than MBT+T traffic. This suggests that LT+T had the highest traffic volumes. The second part shows a rapid rate of increase in the v/c ratios when the straight (MBT+T) traffic volumes were higher than LT traffic. This ideally means that the straight (T) lane had the highest traffic volumes. The MBT traffic volumes were always low (10% of T+MBT) which suggests that the v/c ratio for traffic in the dedicated lane was not critical. In addition, the results also show that for the second part, all lines were parallel to each other unlike for the Model Design 1 where graphs converged to a single straight line. This is because, for higher volumes of T+MBT traffic, the traffic volumes tend to distribute equally between the remaining straight traffic (T) lane and the LT+T lane. This suggests that the critical v/c ratio of MBT lane is highly dependent on both the LT traffic and the straight (T) traffic volumes as shown in Figure 5-15.

Figure 5-15 also shows that for constant values of the v/c ratios where the LT traffic is less than the MBT+T traffic, the volume of LT traffic is inversely proportional to the volume of MBT+T traffic. This is because the critical v/c ratios depend on traffic volumes of both lanes. Subsequently, at a constant v/c ratio, an increase in traffic volume in one lane requires a decrease in traffic in the other lane since v/c ratio depends on both variables. Specifically, it was found that at v/c=0.6, when MBT+T traffic was 880 PCU/Hr/, the LT required was 50 PCU/Hr. In the same way, 490 PCU/Hr traffic volume of the MBT+T traffic required an LT traffic volume of 400 PCU/Hr at v/c=0.6. Using the same logic, at v/c ratio=1.0, the maximum MBT+T traffic of 1495 PCU/Hr required 50 PCU/Hr of LT Traffic whereas 775 PCU/Hr traffic volume of MBT+T required 700 PCU/Hr volume of LT traffic

Consequently, Figure 5-15 provides a simplified chart for the Model Design 2 at g/C=0.5. The chart is developed from values of traffic volumes at constant values of v/c ratios along the feasible region. The chart summarises the ranges of volumes for the left-turning (LT) traffic volumes which are associated with the volumes for the straight traffic. As discussed in the previous section, the chart clearly shows the inverse relationship that exists between volumes of the straight (MBT+T) traffic and the left turning (LT) traffic. This suggests that the v/c ratios for this model depend on volumes of both LT traffic and straight traffic. This is so because traffic in the auxiliary mixed lane (LT+T) and remaining through (T) lane form a single lane group of (LT+T) during capacity (v/c ratio) analysis. This means for a constant v/c ratio of the lane group, a change in one variable affect the other variable. However, the feasible region was defined to fall between the graphs of v/c=0.6 and v/c=1.0 whereas traffic volume combinations falling outside these two lines were considered not feasible. In addition, in all analyses, the feasible region only considered the situations where the volumes of the straight (MBT+T) traffic were higher than volumes of the left turning (LT) traffic.

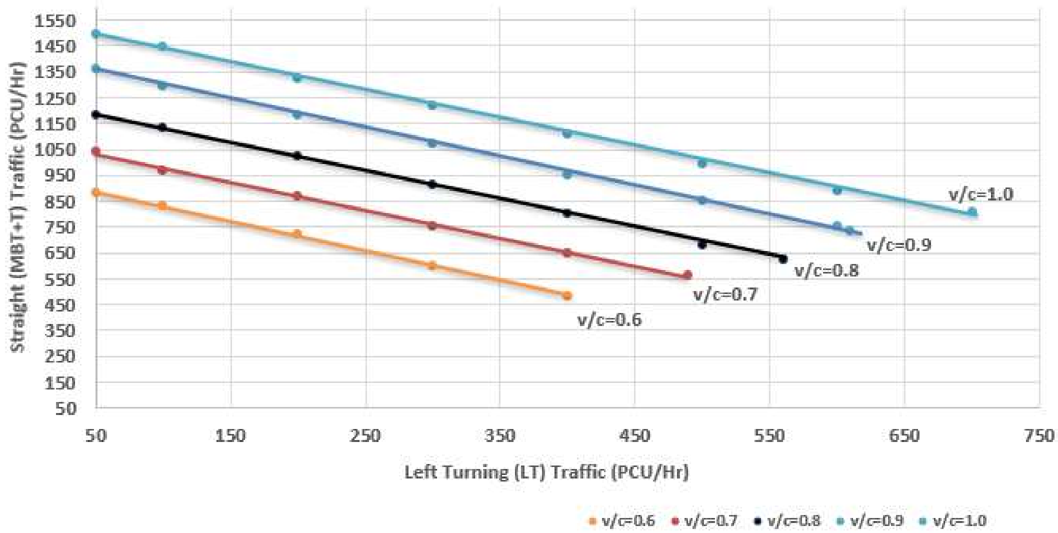


Figure 5-15: Chart of MBT+T Versus LT at g/C Ratio=0.5 for Model Design 2

b) <u>Charts for v/c ratios and traffic volumes at g/C = 0.40</u>

Figure 5-16 shows graphs of the relationship between the v/c ratios and the traffic volumes at constant g/C ratio of 0.40. The graph was plotted in exact the same way as a).

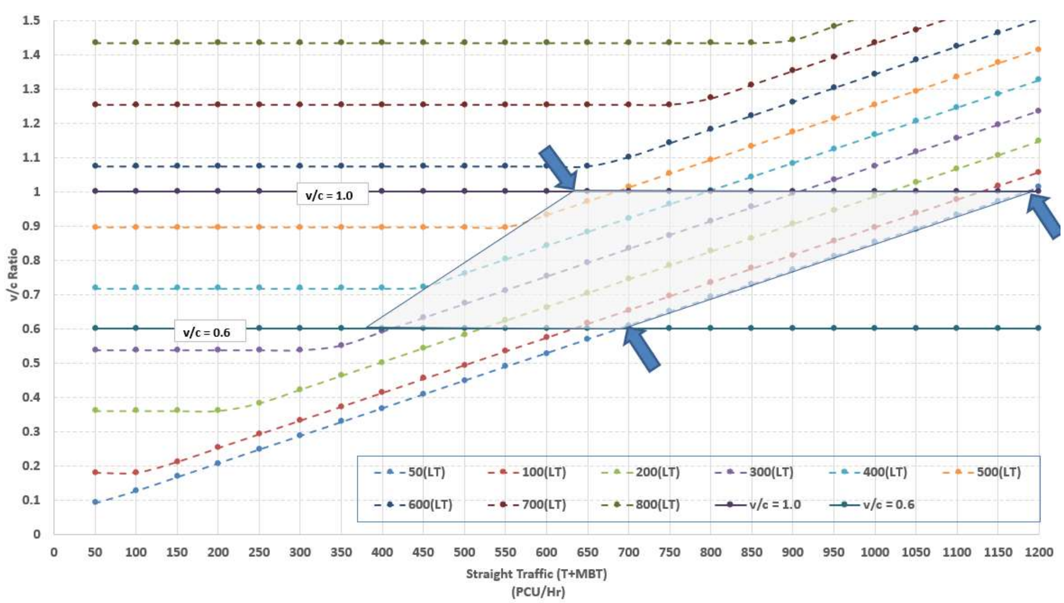


Figure 5-16: The feasible region and changes in v/c ratio at g/C=0.40 for Model Design 2

The results show that limits for traffic volumes decreased with decrease in g/C ratio. For example, at v/c=0.6, when MBT+T traffic was 680 PCU/Hr/, the LT required was 50 PCU/Hr. In the same way, 380 PCU/Hr traffic volume of the MBT+T traffic required an LT traffic volume of 330 PCU/Hr at v/c=0.6. Using the same logic, at v/c ratio=1.0, the maximum straight traffic of 1180 PCU/Hr required 50 PCU/Hr of the LT Traffic whereas 675 PCU/Hr traffic volume of MBT+T required 560 PCU/Hr volume of the LT traffic

Additionally, at constant v/c ratios, the changes in limits of traffic volumes are shown using Figure 5-17.

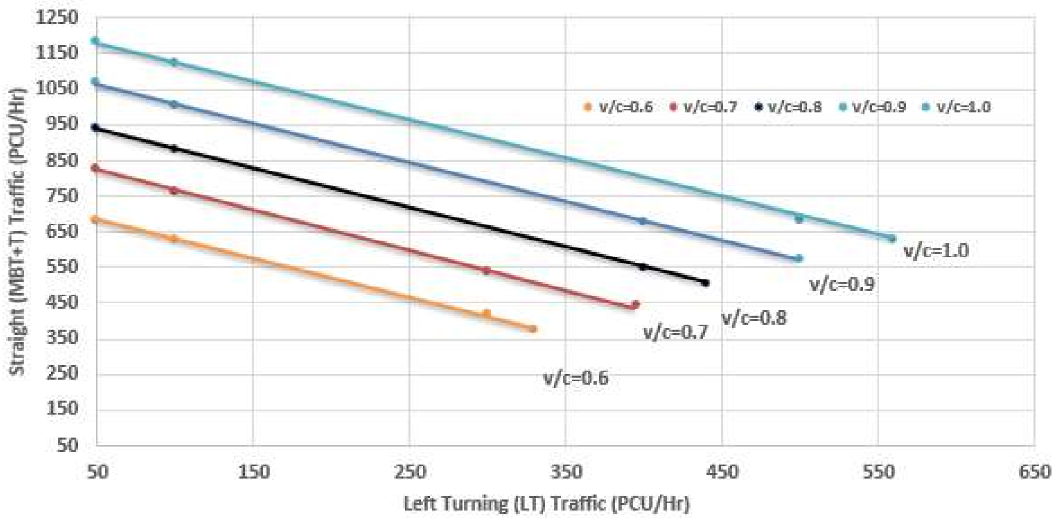


Figure 5-17: Chart of MBT+T Versus LT at g/C Ratio=0.4 for Model Design 2

c) <u>Charts for v/c ratios and traffic volumes at g/C = 0.30</u>

Figure 5-18 shows the graph of the relationship between the v/c ratios and the traffic volumes at constant g/C ratio of 0.30. The graph was plotted in the exact same way as a) and b).

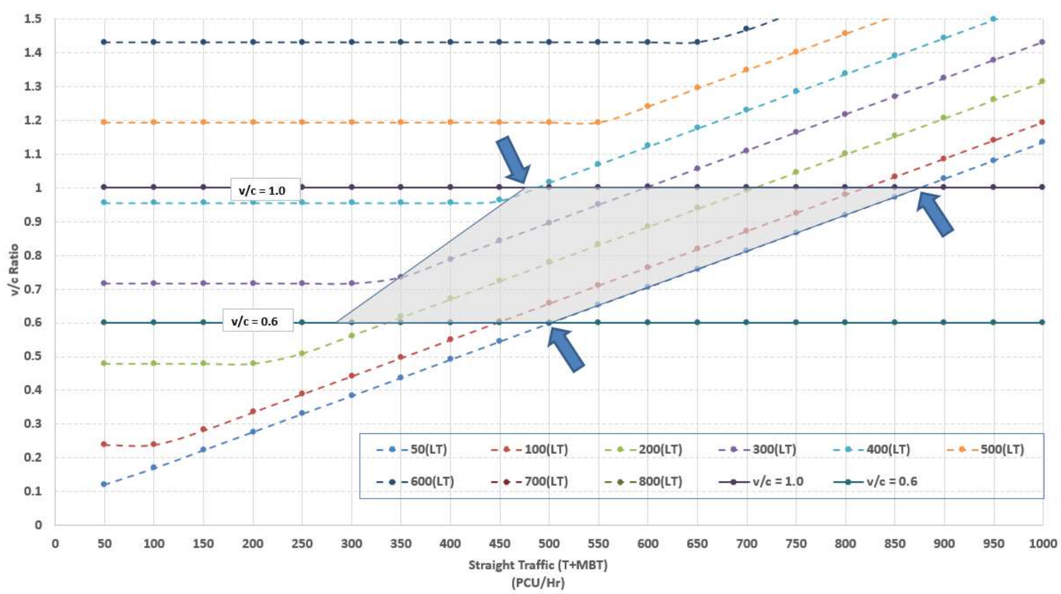


Figure 5-18: The feasible region and changes in v/c ratio at g/C=0.30 for Model Design 2

The results show that limits for traffic volumes decreased further with the further decrease in g/C ratio. The results show at v/c=0.6, when the MBT+T traffic was 500 PCU/Hr/, the LT required was 50PCU/Hr. In the same way, 280 PCU/Hr traffic volume of the MBT+T traffic required an LT traffic volume of 250 PCU/Hr at v/c=0.6. Using the same logic, at v/c ratio=1.0, the maximum MBT+T traffic of 875 PCU/Hr required 50 PCU/Hr of the LT Traffic whereas 475 PCU/Hr traffic volume of the MBT+T required 420 PCU/Hr volume of the LT traffic.

Additionally, at constant v/c ratios, the changes in limits of traffic volumes are shown using Figure 5-19.

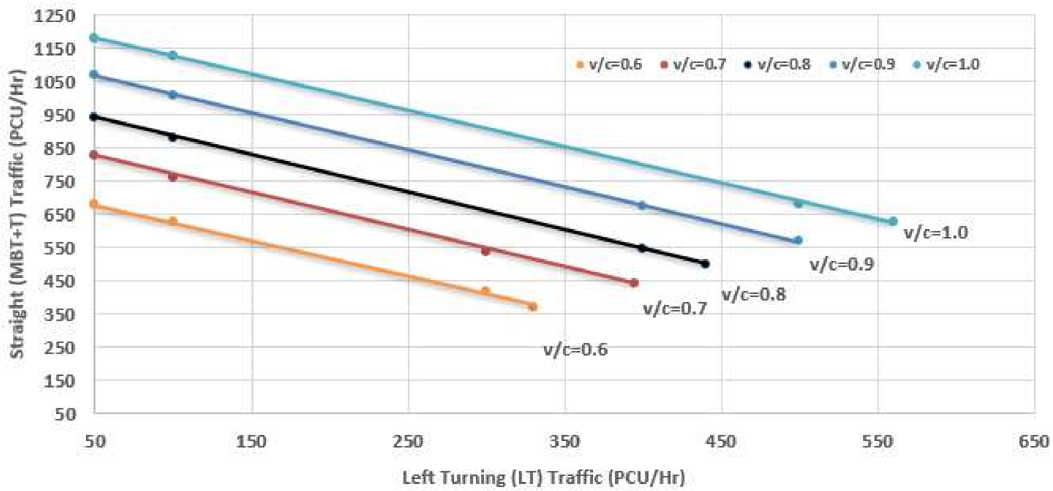


Figure 5-19: Chart of MBT+T Versus LT at g/C Ratio=0.3 for Model Design 2

d) Charts for v/c ratios and traffic volumes at g/C = 0.20

Figure 5-20 shows graphs of relationship between v/c ratios and traffic volumes at constant g/C ratio of 0.20. The graph was plotted in the exact same way as a), b) and c).

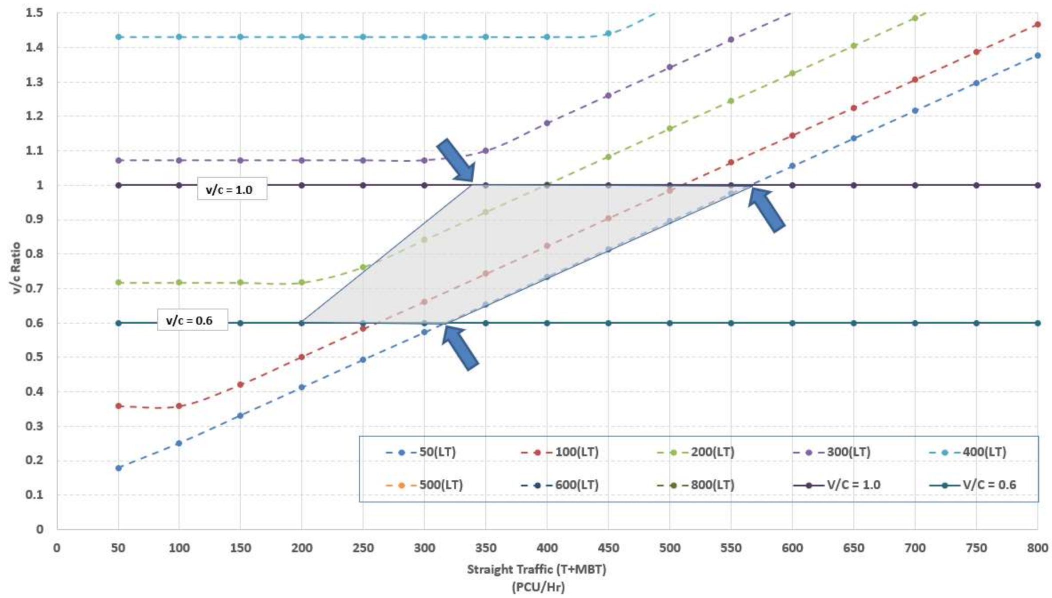


Figure 5-20: The feasible region and changes in v/c ratio at g/C=0.20 for Model Design 2

The results show that at v/c=0.6, when MBT+T traffic was 320 PCU/Hr/, the LT required was 50 PCU/Hr. In the same way, a 200 PCU/Hr traffic volume of the MBT+T traffic required an LT traffic volume of 170 PCU/Hr at v/c=0.6. Using the same logic, at v/c ratio=1.0, the maximum straight traffic of 570 PCU/Hr required 50 PCU/Hr of the LT Traffic whereas 335 PCU/Hr traffic volume of the MBT+T required 280 PCU/Hr volume of the LT traffic

Additionally, at constant v/c ratios, the changes in limits of traffic volumes are shown using Figure 5-21.

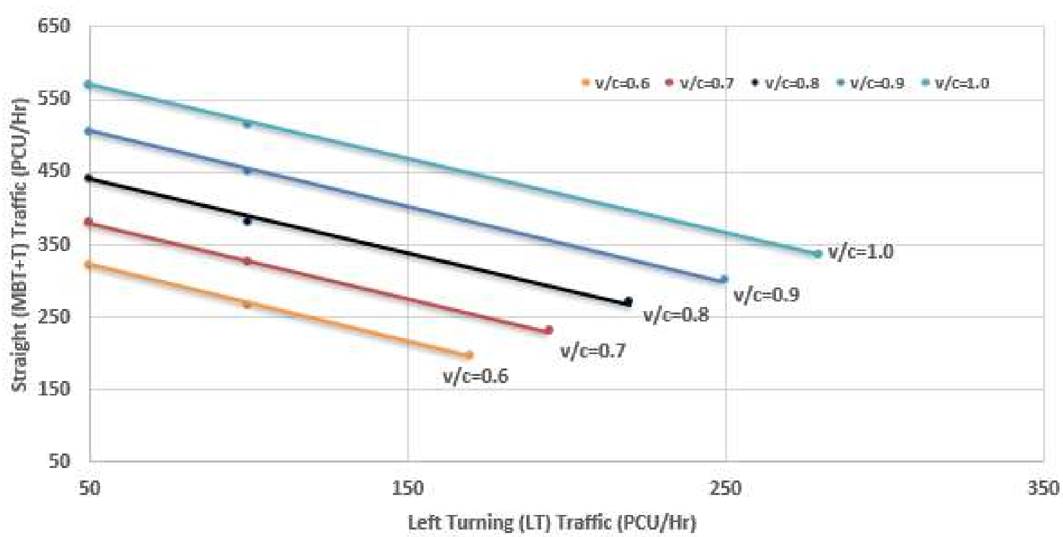


Figure 5-21: Chart of MBT+T Versus LT at g/C Ratio=0.2 for Model Design 2

5.4.1 Summary of Results for Model Design 2

The previous section has presented models in form of design charts for Model Design 2 to show the relationship between the v/c ratios and the traffic volumes at a constant g/C ratio. Overall, the results have shown that there is a general an increase in v/c ratios with increase in traffic volumes. Specifically, the results have shown that the v/c ratios for Model Design 2 are directly proportional to straight traffic (T+MBT) where T+MBT traffic volumes are higher than LT traffic volumes. The results have also shown that at constant values of v/c ratios, the LT traffic volumes are inversely proportional to the T+MBT traffic volumes. These findings suggest that there is significant relationship between all the input variables of the model. Consequently, to establish this relationship, four additional charts were developed at constant values of the g/C ratios to show the relationship between the v/c ratios and the traffic volumes. Specifically, each of these charts was developed to show the relationship between v/c ratios, turning traffic (LT) and straight traffic (T+MBT) at constant g/C ratios. For all the charts, the feasible region was taken to lie between the lines of v/c=0.6 and v/c=1.0. This suggests that combinations of variables outside these lines were therefore considered not feasible. These charts could be used to evaluate the feasible ranges of traffic volumes for a dedicated MBT lane. The simplified charts could therefore be used for the planning and design purposes of dedicated MBT lanes.

5.5 DETERMINATION OF STORAGE LENGTHS OF MBT PRIORITY LANES

The design of storage lanes demands that lanes should be of sufficient length to store the vehicles queued in the lane at urban signalized intersections (Yekhshaty & Schnell, 2008). However, at signalised intersections, the required storage length depends on the signal cycle length, the phasing arrangement, and the rate of arrivals of vehicles (CSIR, 2000). This section estimates the storage lengths associated with the critical traffic volume determined in the previous section. The study used maximum peak hour volumes for the MBT priority lanes to estimate average traffic volumes per cycle length. For both MBT priority models, the analysis assumed a traffic signal of a cycle length of 80 seconds. In addition, an average passenger car length of 4.8m was used in the analytical evaluation.

5.5.1 Maximum Traffic Volumes

This section of the chapter provides estimates for the maximum volumes required for MBT priority infrastructure. The analysis used the maximum values of traffic in the MBT priority lanes to calculate the required maximum traffic volumes. The previous section has shown that the maximum volumes of traffic were recorded at v/c ratios = 1.0. Tables 5-4 and 5-5 summarise volumes of traffic in the MBT priority lanes at v/c=1.0 and at constant g/C ratios. For both models, the volumes of the MBT traffic were assumed to be ten percent of total volumes of straight (10% of MBT+T) traffic. The following formulae were used in conducting this evaluation:

Shared MBT Lane= Left-Turning (LT) Traffic + 10% of Straight (MBT+T) Traffic (Equation 10)

Dedicated MBT Lane = 10% of Straight (MBT+T) Traffic (Equation 11)

Table 5-4: Maximum traffic volumes in the Shared MBT Lane

g/C ratio	Maximum Traffic Volumes at v/c=1.0		
	LT Traffic	MBT+T	Shared MBT Lane
0.2	250	335	**284**
0.3	375	250	**400**
0.4	500	675	**568**
0.5	640	845	**725**

Table 5-5: Maximum traffic volumes in the Dedicated MBT Lane

g/C ratio	Maximum Traffic Volumes at v/c=1.0		
	LT Traffic	MBT+T	Dedicated MBT Lane
0.2	50	570	**57**
0.3	50	875	**88**
0.4	50	1180	**118**
0.5	50	1495	**150**

5.5.2 Estimating Maximum Storage Lengths

To estimate the maximum storage lengths for the MBT priority lanes, the analysis utilised values of peak hour traffic volumes in the MBT priority lanes at v/c=1.0 as determined in the previous section. The SARTSM (2012) design manual recommends storage spaces of at least five vehicles (about 30m) and typically these storage lengths vary between 30m and 60m. The AASHTO (2004) provides the formular (Equation 2) which is used for calculating storage lengths which has also been adopted in this study, as follows:

Length (L)= 1.5* avg. number of vehicles that would store per cycle (Equation 2)

Tables 5-6 and 5-7 have used an Equation 2 to estimate the expected storage lengths of the MBT priority lanes that would accommodate the peak hour traffic volumes in the MBT priority lanes. For each of the four g/C scenarios, the tables estimate the storage lengths of the shared MBT lanes and the dedicated MBT lanes at v/c=1.0 for traffic signal of cycle length of 80 seconds.

Table 5-6: Maximum Storage Lengths for shared MBT lane at v/c=1.0

	g/C Ratio			
	0.2	0.3	0.4	0.5
Peak Hr Traffic Vol, V_{shared} = (PCU/Hr)	284	400	568	725
Cycle Time,C = Seconds	80	80	80	80
Average. Traffic per cycle (n)=(V_{shared}/(3600/C))	7	9	13	17
Storage Length in PCU=(L)=(1.5*n)= (PCU)	10.5	13.5	19.5	25.5
Calculated Storage Length in m = $L_{(PCU)}$X4.8	51	65	94	123
Recommended Storage Length (L)=(m)	55	65	95	125

Table 5-7: Maximum Storage Lengths for a dedicated MBT lane at v/c=1.0

	g/C Ratio			
	0.2	0.3	0.4	0.5
Peak Hr Traffic Vol, V_{shared} = (PCU/Hr)	57	88	118	150
Cycle Time,C = Seconds	80	80	80	80
Average. Traffic per cycle (n)=(V_{shared}/(3600/C))	2	2	3	3
Storage Length in PCU=(L)=(1.5*n)= (PCU)	3	3	4.5	4.5
Calculated Storage Length in m = $L_{(PCU)}$X4.8	15	15	22	22
Recommended Storage Length (L)=(m)	30	30	30	30

The relationship between maximum storage lengths for each specified g/C ratio has also been shown using Figure 5-22. The figure uses values of storage lengths at v/c =1.0 to show the relationship between maximum storage lengths that could be attained at a constant g/C ratio. Both graphs have the coefficient of determination (R^2) of more than 0.85 which shows existence of a strong relationship between the g/C ratio and the dependent variable (storage length).

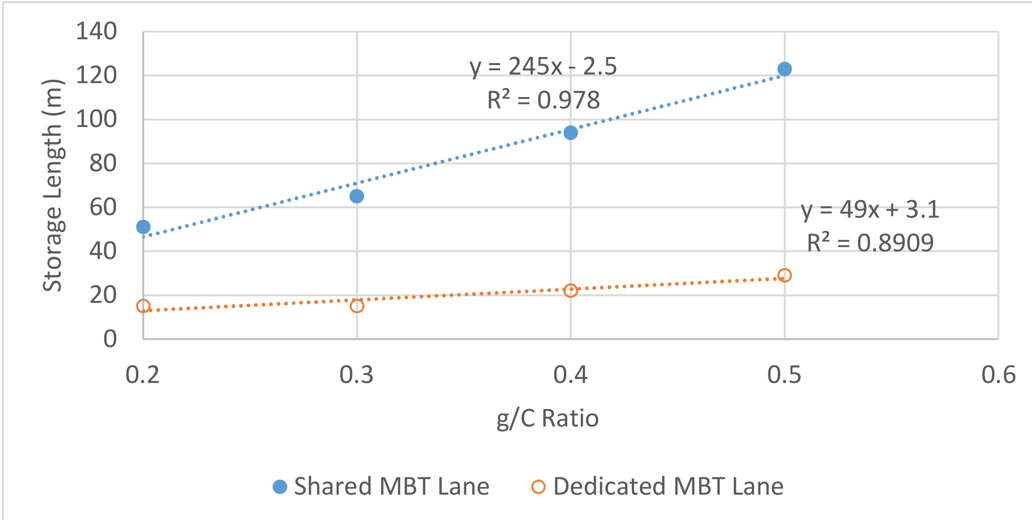


Figure 5-22: Relationship between storage lengths and g/C ratios of priority lanes at v/c=1.0

These results suggest that for the same g/C ratio and v/c ratio, a higher storage length would be recommended for a shared MBT lane than for the dedicated lane. This is because the size of the shared MBT lane (MBT+LT) is determined by the total volume of the MBT and the LT traffic which usually gives larger values. On the other hand, the storage lengths of dedicated MBT traffic lane are determined by MBT traffic volumes only which are usually very low (10% of MBT+T). It is therefore recommended to use a minimum storage length of 30m (SARTSM, 2012) in cases where storage lengths are less than 30m. The short storage lengths in dedicated MBT lanes also suggest that the dedicated MBT lanes could be prone to traffic blockages in situations where queue length for adjacent lanes is too long. To prevent traffic blockages of MBT dedicated lanes, it is therefore recommended to also consider the storage lengths of adjacent lanes when deciding the full length of the MBT dedicated lane.

5.6 CHAPTER CONCLUSION

This chapter provided design models in the form of charts for the shared MBT lanes and dedicated MBT lanes. A sensitivity analysis was used to analyse the modified HCM models used to develop the charts. Several assumptions were made prior to the analysis. Four main design variables were used for both models. These were variables which were found to have a significant impact on the performance of MBT infrastructure as observed from Chapter Four and they included the g/C ratios, v/c ratios, left turning (LT) traffic and straight (MBT+T) traffic. The models were set up to measure the highest v/c ratios in the approaching lanes as traffic volumes were varied.

Two model designs were set up for the purpose of developing the graphs to show the relationships between the input variables. These models included the Model Design 1 and the Model Design 2. The Model Design 1 involved the provision of the shared MBT lane design to allow the MBT traffic to share the lane with the left turning traffic while allowing all other straight traffic (T) to use the remaining straight (T) lane. On the other hand, Model Design 2 was set up to analyse the dedicated MBT lane which would allow straight MBT traffic to use the dedicated lane at intersection while other straight traffic occupy the remaining straight (T) traffic and the auxiliary mixed (LT+T) lane. The graphical analysis for each model was carried out at constant g/C ratio. The set up of the analysis at constant g/C ratio involved varying volumes of the straight traffic from 50 PCU/Hr to 1600 PCU/Hr for each constant value of left-turning traffic. The volume of LT traffic ranged from 50 PCU/Hr to 700 PCU/Hr.

The modified HCM calculation spreadsheets were set up using the Microsoft Excel designed to measure the highest value of v/c ratios of traffic in the individual approaching lanes at constant g/C ratio. The values of v/c ratios changed as volume of straight traffic (MBT+T) were varied at constant LT traffic for each g/C ratios. The first set of graphs were plotted showing relationship between the critical v/c ratios and straight (MBT+T) traffic for each constant value of LT traffic. All volume combinations between lines of v/c ratio = 0.6 and v/c ratio= 1.0 were considered feasible for which the volumes of straight (MBT+T) traffic were higher than LT traffic volumes. This first set of graphs was used to establish the relationship between the LT traffic and straight (MBT+T) traffic. Using this relationship, a second set of simplified charts were prepared at constant v/c ratios.

The results from the simplified charts for both models show that the v/c ratios generally increase with the increase in traffic volumes. Results showed significant difference in terms of feasible traffic volumes for the two models. The charts for Model Design 1 showed that v/c ratios were greatly influenced by straight (MBT+T) traffic volumes. For example, each constant value of straight (MBT+T) traffic required a range of the LT traffic volumes to give a constant value of the v/c ratio. This is so because for Model Design 1, there is no redistribution of straight traffic other than MBT traffic occupying the MBT shared lane. In this case, the critical v/c ratios directly depend on the values of the straight (T) traffic.

On the other hand, charts for Model Design 2 showed an inverse relationship between the LT traffic and the straight traffic. This suggests that dedicated MBT lane performance depends on both the left turning and the straight traffic volumes. This is because of the redistribution that happened by the straight (T) traffic between the straight (T) lane and the mixed (LT+T) lane. During the traffic redistribution (after dedicated MBT lane has been introduced), the straight (T) traffic and left turning traffic would form a single lane movement group (LT+T) used for the v/c ratio analysis purpose (HCM, 2010). Once the single lane group has been created, a change in one input variable automatically affects the other. Put differently, at a fixed value of v/c ratio, a higher volume of the straight (T) would require a lower value of LT traffic and vice versa.

These charts developed from both analyses could be used to evaluate the feasible ranges of traffic volumes for MBT priority lanes. The simplified charts could therefore be used for the planning and the design purposes of MBT priority lanes.

The study has also estimated the storage lengths associated with the established range of critical volumes. The analysis showed that the shared MBT lanes are associated with a higher value of the storage lengths than the dedicated lanes. This is because the shared MBT lanes area associated with higher traffic (MBT+LT) than dedicated MBT lanes which were designed to accommodate only ten percent of the straight traffic (10% of MBT+T). To prevent other traffic from blocking the dedicated MBT lane, a recommendation was made to consider evaluating the storage lengths of adjacent lanes when designing full length of dedicated MBT lanes.

In conclusion, this chapter has developed charts showing the relationships between the g/C ratios, traffic volumes and v/c ratios. These charts could be used to predict the feasibility of intersections for the MBT priority infrastructure. These results provide the first thoughts regarding the design considerations and feasibility of the MBT priority infrastructure at intersections.

6 CONCLUSIONS

This final chapter presents the principal conclusions of the study together with a description of its theoretical contribution. In addition, the chapter outlines the limitations of the research, and suggestions for future research. The objectives of this study were threefold: 1) to examine a range of MBT priority infrastructure interventions at signalized intersections 2) to examine the impacts of MBT priority interventions on the performance of signalized intersections, and 3) to provide design guidance on feasibility of MBT priority interventions at intersections.

6.1 SUMMARY OF FINDINGS

6.1.1 The Evaluation of Design Strategies for the MBT Priority Infrastructure

The first part of this study (Chapter 3) explored an approach for evaluating the design strategies for the MBT priority infrastructure at signalised intersections. The evidence of this part of the study suggested that there is a relationship between the intersection geometry and the type of design strategies for the priority infrastructure provided at intersections. Data for the design strategies was collected through a literature review to show that the existing geometric features of intersections could be used to determine which combination of design strategies are feasible for MBT priority infrastructure at intersections. This analysis included the evaluation of design treatments and geometric elements that influence design strategies for priority infrastructure at intersections from best practices. The results of this analysis therefore included the formulation of themes in the form of design treatments and geometric elements for priority infrastructure at intersections. Thereafter, using themes developed from this evaluation, a further analysis was conducted on the layout designs of intersections in South African cities including the City of Tshwane. The aim of this analysis was to evaluate the intersection geometry in South Africa for the possibility of providing the MBT priority infrastructure.

The results of this chapter included the development of a framework matrix that was used for analysing two sets of data. The rows of the matrix table contained the geometric data for the existing intersections while the columns of the matrix table contained the design treatments from best practices on priority infrastructure. The matrix table used colour codes and numbers to show feasible combinations. This evaluation of feasible combinations was followed by the development of two categories of design strategies. The first category comprised of design strategies evaluated by repurposing the existing intersection layouts without conducting major geometric upgrades or additions. Typical examples of these upgrades included changing of the existing road markings (traffic movements), repurposing the existing lanes, and adding the priority traffic signals. The second category compromised of the design strategies that require major geometric improvements such as the addition of new lanes.

Using the framework matrix analysis, four different MBT priority interventions were developed, and these included the shared MBT lane, the dedicated MBT lane, the MBT queue jumping lane, and the transit signal priority with MBT dedicated or MBT shared lane.

This part of the study concludes that the framework matrix evaluation that was developed could be used to assist in making the preliminary decisions about the choices of the interventions of MBT priority infrastructure at signalised intersections under various existing geometric conditions. Specifically, this part of the study has provided four interventions for the MBT priority infrastructure using the framework matrix analysis.

6.1.2 Performance Evaluation

The second part of this study (Chapter Four) presented the performance evaluation of MBT priority infrastructure. The purpose of this analysis was to determine the impacts of MBT priority interventions using real traffic data. Four intersections were initially sampled in the City of Tshwane to determine the most feasible interventions that could be used for the performance evaluation. In addition, these four sampled intersections were also used to show the application of the framework matrix analysis as a method for evaluating existing intersections for MBT design strategies as established in Chapter Three. A framework matrix evaluation was conducted on all four intersections and at least two design strategies for MBT priority infrastructure were proposed at each intersection. The Design Strategy 1(DS1) involved the repurposing of an existing intersection without the geometric modifications whereas the Design Strategy 2 (DS2) involved the modification to geometric layout of an existing intersection. An additional high-level evaluation was done to show the impact of the proposed MBT design strategies on design safety and traffic operations on the four evaluated intersections.

After the evaluation of the four intersections with their associated design strategies, two final intersections and two design strategies were selected for performance evaluation. A ninety-minute traffic count was then conducted during AM and PM peak periods to collect the peak hour traffic volumes for the purpose of performance analysis. The performance analysis utilised the HCM methodology by measuring changes in the v/c ratios and average vehicle delays. To determine the impact of each MBT priority infrastructure, three design scenarios were considered. First, a 'do-nothing' scenario which involved the evaluation of the existing intersections without the modification to the layout geometry and traffic signals. The second design scenario involved the analysis of the design strategy without modification to traffic signal settings (DSa). The last design scenario involved analysis of the design strategies after optimising the traffic signal settings (DSb).

The performance results of the existing intersection conditions were compared to the performance results after the implementation of the MBT design strategies (DSa and DSb). The results have shown that both MBT priority lanes and non-priority lanes were affected by the MBT interventions. Specifically, both design strategies improved the performance of traffic in the priority lanes while performance in the non-priority lanes was reduced. For a shared MBT lane (MBT+LT), the v/c ratio of traffic in the shared MBT lane reduced by up to 73% providing a decrease in the average vehicle delay of up to 28%. On the other hand, the v/c ratio for the straight traffic increased by up to 75%, providing an increase in the average vehicle delay of up to 250%. Similarly, where a short dedicated MBT lane was provided, the results have shown that the v/c ratio for the MBT traffic improved by up to 94% providing a delay decrease of up to 32%. On the contrary, the v/c ratio for traffic in the non-priority lanes increased by up to 43% providing a delay increase of up to 154%. These results suggest that MBT priority infrastructure could indeed become beneficial especially to the straight MBT traffic. However, the results also suggest that these priority infrastructure types could also negatively impact the performance of traffic in the non-priority lanes. To improve the operation of all traffic, an optimised design scenario was used which involve the modification of the existing traffic signals.

The optimised design solution was used to give benefits to the MBT traffic without adversely affecting the performance of traffic in the non-priority lanes. For the shared MBT lane, the optimised design option (DS1b) saw the v/c ratio of the traffic in the MBT priority lanes improve by up to 75% (compared to 73% for DS1a) while the v/c ratio of traffic in non-priority lanes increased by only 46% (compared to 75% for DS1a). These changes in the v/c ratios represent a decrease in the delay for traffic in priority lane of up to 51% (compared to 28% for DS1a) while the delay for traffic in the non-priority lanes increased by up to 39% only (compared to 250% for DS1a). Similar results were obtained for the dedicated MBT lanes. The signal optimisation design option (DS2b) for the MBT dedicated lanes saw the v/c ratio of traffic in MBT priority lanes improve by up to 95% (compared to 94% for DS2a) while the v/c ratio of traffic in non-priority lanes increased by only 4% (compared to 43% for DS2a). These changes in v/c ratios represented a decrease in delay for traffic in priority lanes of up to 64% (compared 32% for DS2a) while the delay for traffic in non-priority lanes increased by up to 27% only (compared to 154% for DS2a). In general, these results suggest that by optimising the existing signals, the MBT priority interventions could indeed greatly improve the performance of MBT traffic without adversely affecting the performance of traffic in non-priority lanes.

Finally, a comparative analysis of storage lengths has also shown that the existing lanes have adequate storage lengths to accommodate traffic in priority lanes. However, to prevent blockages of these priority lanes by traffic in non-priority lanes, the study has recommended to consider critical storages lengths of adjacent lanes when designing the full lengths of MBT priority lanes.

Overall, the results for this part of the study have shown that MBT priority infrastructure could greatly improve the operation of MBT traffic at intersections. While the results have proven the benefits of MBT priority infrastructure to MBT traffic, it has also been shown that these strategies could also reduce the performance of the traffic in non-priority lanes. However, the study has shown that with well optimised design solutions, the benefits of MBT priority lanes to MBT traffic could be enhanced without adversely affecting the performance of traffic in non-priority lanes.

6.1.3 The Determination of Viable Range of Traffic Volumes

This part of the study (Chapter Five) developed graphical design models of traffic volumes to use for predicting the feasibility of MBT priority infrastructure. The analysis was set up using principles of the HCM method for capacity evaluation by measuring the v/c ratios while varying traffic volumes. To set-up up the models, first, typical assumptions were made for two model designs representing design strategies for the shared MBT lanes and dedicated MBT lanes. Among other assumptions, the models assumed a maximum of 10% of straight volumes to be allocated to straight MBT traffic volumes. The feasible region was assumed to be fall between the v/c ratios of 0.6 and the v/c ratios of 1.0. Using the sensitivity analysis for two variables in Microsoft excel, these models were set up to measure the critical (highest) v/c ratios of traffic from the MBT priority or non-priority lanes by varying the left turning (LT) traffic and straight (MBT+T) traffic. Both models were tested over four different constant values of the g/C ratios which included the g/C ratios of 0.2, 0.3, 0.4 and 0.5 while varying volumes of straight (MBT+T) traffic from 50 PCU/Hr to 1600 PCU/Hr. Thereafter, graphs were plotted showing the changes in the v/c ratios and straight (MBT+T) traffic at fixed volumes of left turning (LT) traffic volumes.

The results have shown that there is a significant relationship between v/c ratios, straight traffic, LT traffic, and g/C ratios. Consequently, the graphs developed have provided ranges of feasible volumes for MBT priority infrastructure at constant g/C ratios. For Model Design 1 (shared MBT lane) and for the conditions where the LT traffic volumes were less than the MBT+T traffic volumes, the results show that the model was feasible for the volume of the straight (MBT+T) traffic ranging from 200 PCU/Hr (at g/C ratio=0.2, v/c ratio=0.6) to 845 PCU/Hr (at g/C ratio=0.5, v/c ratio=1.0). Similarly, the model was also feasible for the LT traffic ranging from 50 PCU/Hr (at g/C ratio=0.2; v/c ratio=0.6) to 640 PCU/Hr (at g/C ratio = 0.5; v/c ratio=1.0).

On the other hand, results for Model Design 2 (dedicated MBT lane) have shown that there is an inverse relationship between volumes for straight traffic and LT traffic at fixed v/c ratios and constant g/C ratio. This suggests that the critical v/c ratios for this model is largely dependent on both LT traffic and straight (MBT+T) traffic. This is because the critical traffic (in non-priority lanes) for this model is distributed equally between auxiliary (LT+T) lane and through (T) lane from which the critical v/c ratios are obtained. Consequently, graphs at constant g/C ratios have been developed to show ranges of traffic volumes for which the dedicated MBT priority infrastructure could be feasible. For example, the results have shown that the Model Design 2 could be feasible for volumes of straight traffic ranging from 320 PCU/Hr (at g/C ratio=0.2; v/c ratio=0.6) to 1495 PCU/Hr (at g/C=0.5; v/c ratio=1.0). Similarly, the results have shown that this model could also become feasible for volumes of LT traffic ranging from 50 PCU/Hr (at g/C=0.2; v/c ratio=0.6) to 700 PCU/Hr (at g/C ratio=0.5; v/c ratio= 1.0).

Following the establishment of the relationships between the v/c ratios and traffic volumes, more simplified design charts were created at constant values of the g/C ratios showing relationships between the LT traffic and the straight (MBT+T) traffic at constant v/c ratios. These charts were developed as a planning and design guidance when evaluating the feasibility of signalised intersections for MBT priority infrastructure.

Finally, this part of study also evaluated the adequacy of the storage lengths associated with the evaluated models (shared MBT lane for Model Design 1 and dedicated MBT lane for Model Design 2). The purpose of this evaluation was to recommend the maximum storage lengths for MBT priority lanes which could accommodate ranges of traffic volumes established by the two design models. This study has therefore provided estimates for the maximum storage length of MBT priority lanes which would be required by using the maximum traffic volumes obtained from the graphical design models. The study has also found that the shared MBT lanes could be associated with higher values of the storage lengths compared to the dedicated MBT lanes. This is because the shared MBT lanes tend to have higher traffic volumes (MBT+LT) as compared to the dedicated MBT lanes which were designed to exclusively give priority to MBT traffic volumes. The study has found that the maximum storage lengths for the shared MBT lanes would range from about 50m to 125m for the g/C ratios of 0.2 to 0.5 respectively. On the other hand, the required storage lengths for the dedicated MBT lanes were found to be less than 30m for all design scenarios. Due to the short storage lengths associated with the dedicated MBT lanes which could easily be blocked by traffic in non-priority lanes, the study recommends considering adjacent storage lengths when designing full lengths of the dedicated MBT lanes.

6.2 RESEARCH CONTRIBUTIONS

The contributions of this research study include the following:

- Development of the framework matrix for evaluating the design strategies for the MBT priority infrastructure: Previous studies did not fully investigate the relationships between the choices of the MBT priority infrastructure and the existing geometric features at intersections. However, this study has provided the analytical approach in the form of framework matrix to assist designers and planners when evaluating existing intersections for choices of MBT priority facilities.
- Capacity evaluation of the shared and the dedicated MBT lanes. Previous research studies did not attempt to evaluate the choices of MBT priority infrastructure using the conventional methods of analysing the intersections including the use of real time traffic count data. This is important as doing so would eliminate the doubt or design concerns regarding the feasibility of the MBT priority infrastructure within a wider public transport network system. This study has therefore provided a procedure on how to carry out a performance evaluation analysis for the MBT priority infrastructure. It has also highlighted key parameters which designers and planners should be aware of when evaluating the MBT priority lanes. Above all, the study has highlighted the limitations and design concerns which could affect the overall performance of an intersection and then provided methods of addressing them.
- Development of the graphical design models for feasible traffic volumes of the MBT priority infrastructure. Traffic volume is a significant design input that determines the performance of intersections. Any attempt to introduce the priority facilities to a particular type of traffic at intersections could also mean compromising the performance of the other types of traffic. Previous studies on MBT priority facilities did not provide insights into what could be the appropriate ranges of traffic volumes that could be considered as feasible for the MBT priority infrastructure. This study has developed a series of design charts that show the relationship between traffic volumes and the v/c ratios at constant g/C ratios. These charts could provide the basis of designs of intersections for the MBT priority infrastructure.
- The study has also highlighted the expected ranges of storage lengths for the two MBT priority lanes investigated in this study. No previous studies have attempted to show impacts

of the MBT priority interventions on the storage lengths. This study has shown that the storage length of the MBT priority lane is a significant parameter to be considered during designs of MBT priority lanes. The study has also provided estimates for the maximum storage lengths associated with the critical traffic volumes at constant values of g/C ratios.

6.3 LIMITATION OF THE RESEARCH

This section presents some the limitations of the study apart from the limitations discussed in the previous chapters.

6.3.1 Limited number of MBT priority infrastructure evaluated.

The performance evaluation and graphical models only considered a limited range of priority infrastructure namely, a shared MBT lane and a dedicated MBT lane. The results of this study should therefore apply to the evaluated priority infrastructure types within the assumed conditions.

6.3.2 Evaluation based on peak hour traffic volumes.

Both the performance evaluation and graphical model designs were based on peak hour traffic volumes only. The study has therefore not covered the conditions for the off-peak periods which could help to determine the optimal times for operating these MBT priority intersections.

6.3.3 Limited number of sampled intersections

The capacity evaluation of this study was based on traffic volumes from two intersections (one intersection for each MBT design strategy). Hence, the performance results obtained in terms of the percentage benefits or improvements were based on the traffic count from these two intersections. These results may not cover other intersections of different traffic conditions. It is therefore recommended that the performance results for this study should be used under the assumed traffic conditions.

6.4 SUGGESTIONS FOR FUTURE RESEARCH

Below are a few suggestions for future research:

6.4.1 Improving the design of conceptual framework matrix

Future research should focus on improving the conceptual design of the framework matrix analysis (Chapter Three) developed in this study. Other researchers should test its relevance and the ease of applicability by using different types of existing intersection geometry to show the understanding of the matrix. Other intersections where the method could be tested could include the intersections of multiple straight lanes, or multiple turning lanes. Alternatively, a quick survey on the users' perspectives of the framework matrix can be conducted for the purpose of improving any possible gaps.

6.4.2 Improving capacity evaluation analysis

Another avenue for future research could investigate the capacity evaluation through micro-simulation and compare the results with those from this study. In addition, future analysis should include other types of traffic such as the pedestrian and its impact on performance. Additionally, future analysis should also consider the impact on the performance when MBT traffic stop inside the intersections or block the lanes. Furthermore, an additional work should extend to evaluating the impact of location of the dedicated MBT lane. For example, by comparing the changes in lane capacities between cases where dedicated MBT lanes are located between two lanes to cases where dedicated MBT lanes are located on the nearside lane. Future analysis should also consider conducting the performance evaluation over the entire day for the purpose of determining optimal times for operating these MBT infrastructure. As part of the feasibility studies, the results from this study can also be used in the future to perform a detailed cost benefit analysis of the MBT priority infrastructure. Finally, other researchers should also look into conducting the capacity evaluation of the other choices of the MBT infrastructure on various types of intersection geometry and traffic conditions.

6.4.3 Testing the validity and applicability of the graphical design models for predicting the feasibility of MBT priority lanes.

Future research work should investigate on the validity of the conceptual graphical models (Chapter Five) developed in this study for use in predicting the feasibility of the MBT priority infrastructure using the traffic volumes and the v/c ratios. Other researchers should test its effectiveness and applicability using approved simulation software.

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8 APPENDICES

APPENDIX A: PHOTOGRAPHS OF PRIORITY INFRASTRUCTURE

<table>
<tr><th colspan="2">Examples of Priority Infrastructure Designs</th></tr>
<tr>
<td>

New York MTA Select Bus Service Queue Jump Lane
(Source: https://www.translink.ca)</td>
<td>

Queue jump allowing buses to take through movement (SCAG, 2021)</td>
</tr>
</table>

Examples of Priority Infrastructure Designs

Queue Jump Lanes in Calgary, AB2, Canada

(Source: https://www.translink.ca)

San Francisco Muni Transit Approach Lane–Stockton St

(Source: https://www.translink.ca)

Examples of Priority Infrastructure Designs	
 MTA Select Bus Service (SBS86) Approach Lane (Source: https://www.translink.ca)	 Barcelona Shared Bus Taxi Lane, Barcelona, Spain (Source: https://www.translink.ca)

Examples of Priority Infrastructure Designs	
 End of Transit Approach Lane with Separated Turning Traffic – Washington Street in Chicago (Source: https://www.translink.ca)	 Coordinated Signal Timing, Longview, WA (Source: https://www.translink.ca)
 Far-side bus stop in downtown Los Angeles, California. (SCAG 2021)	

APPENDIX B: DETAILED DESIGN STRATEGIES

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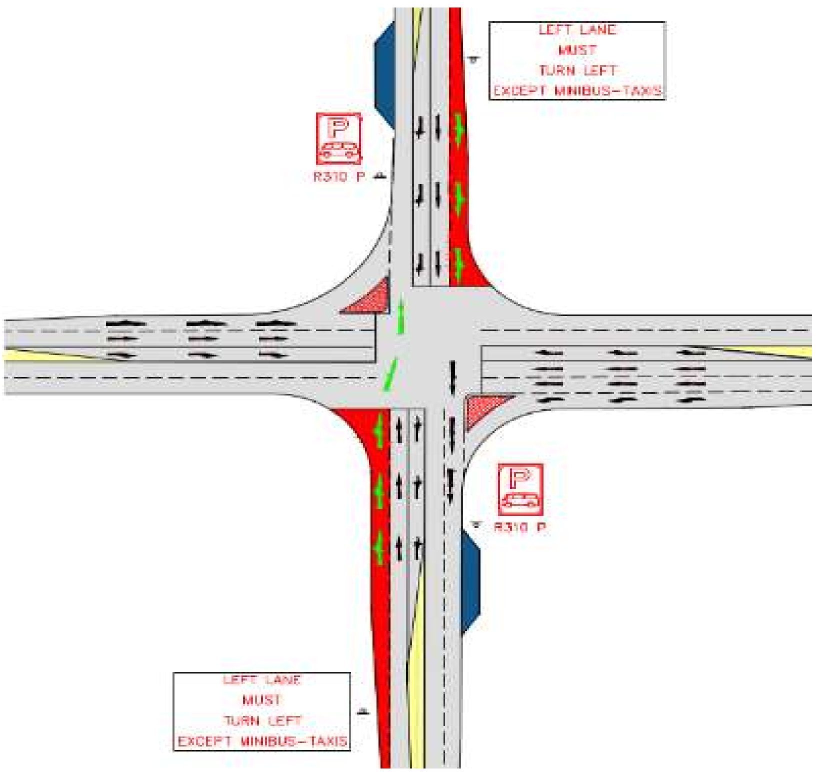
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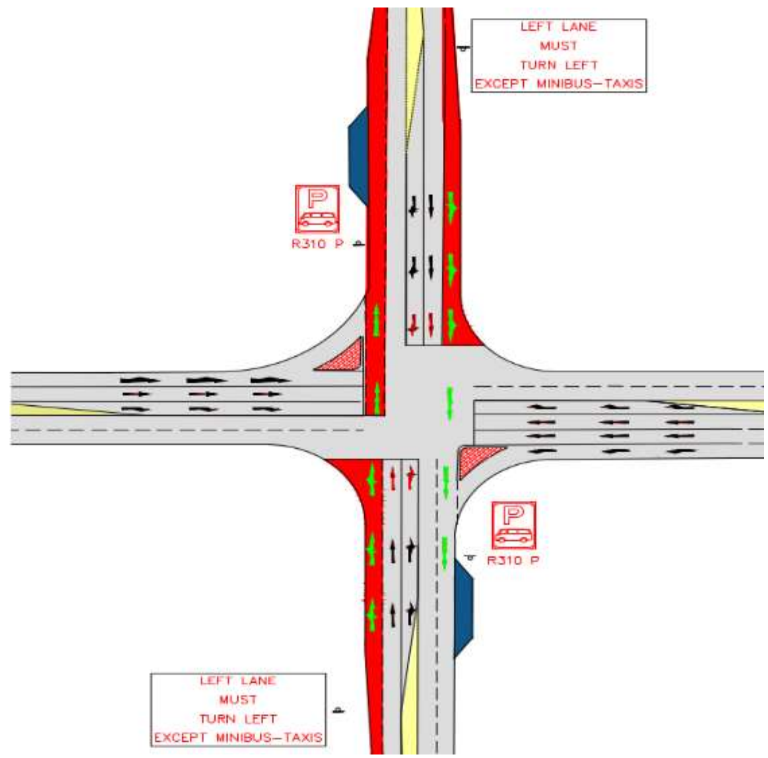
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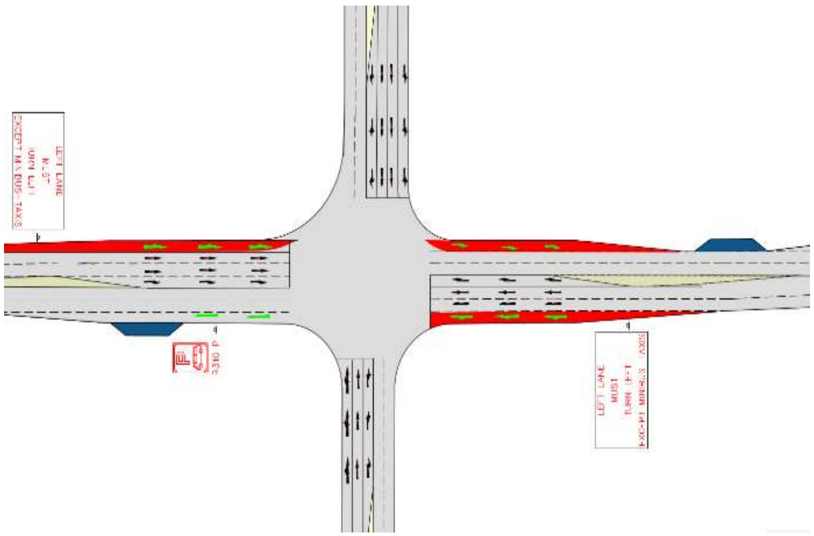
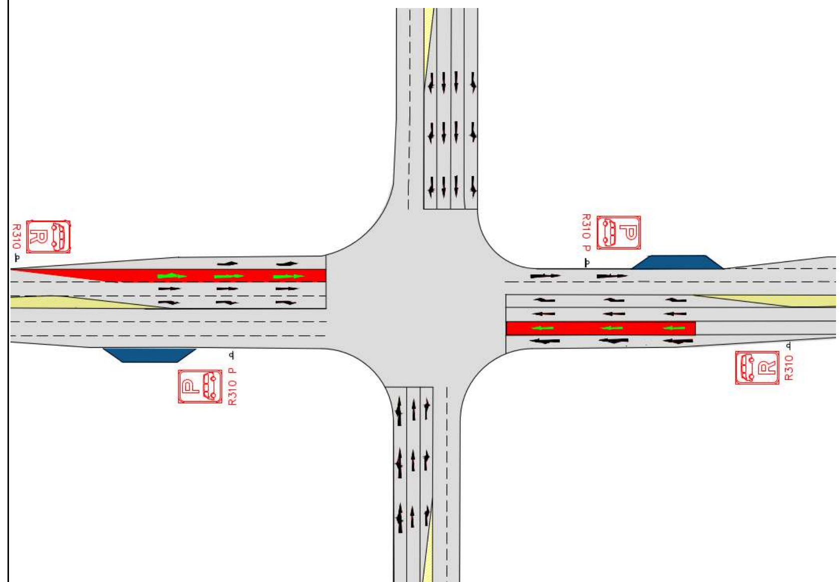
DESIGN OPTION ONE	DESIGN OPTION TWO
`J2 	J2 

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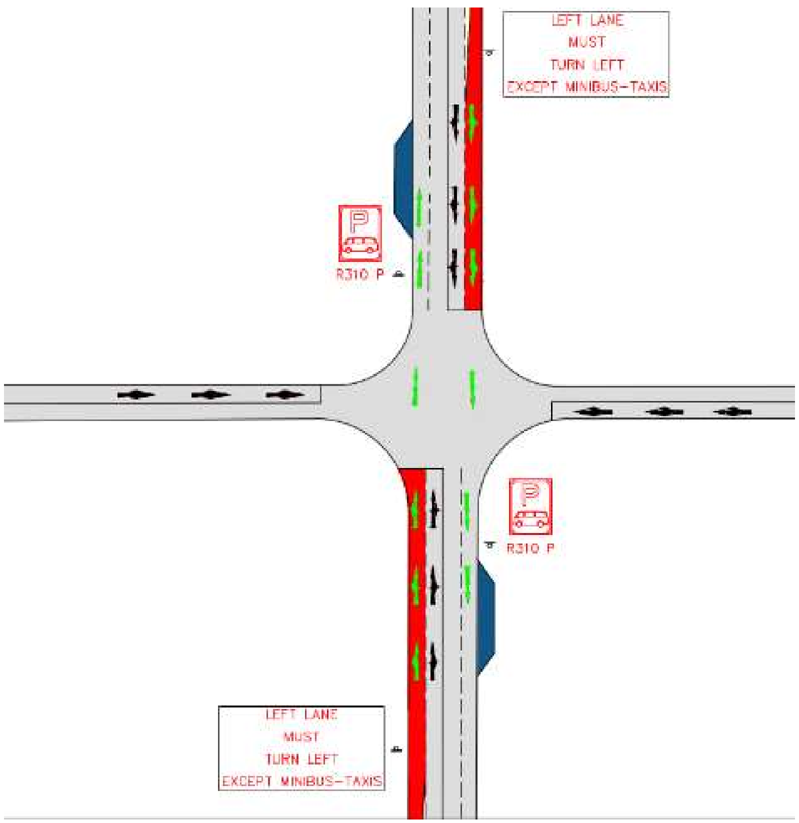
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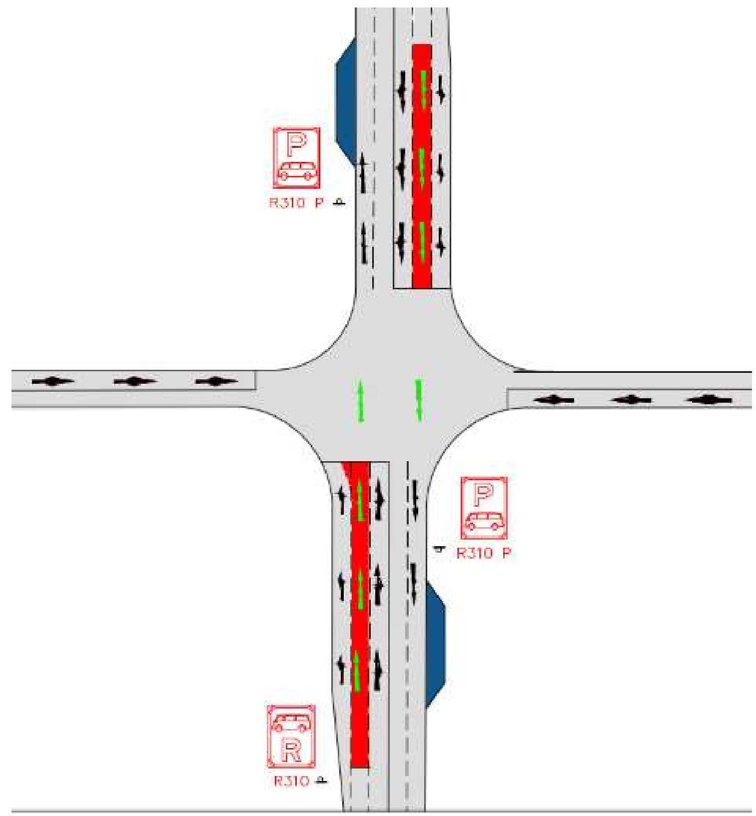
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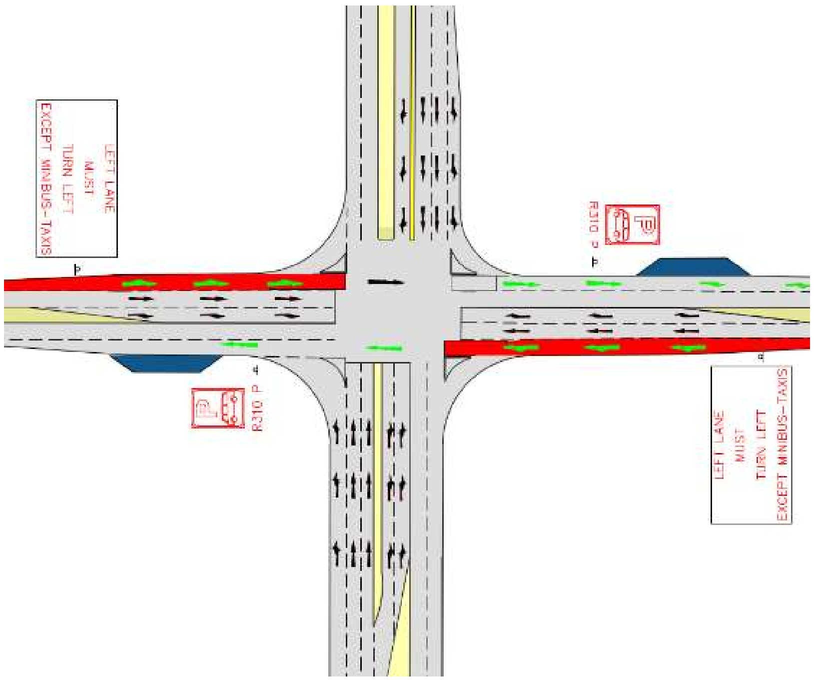
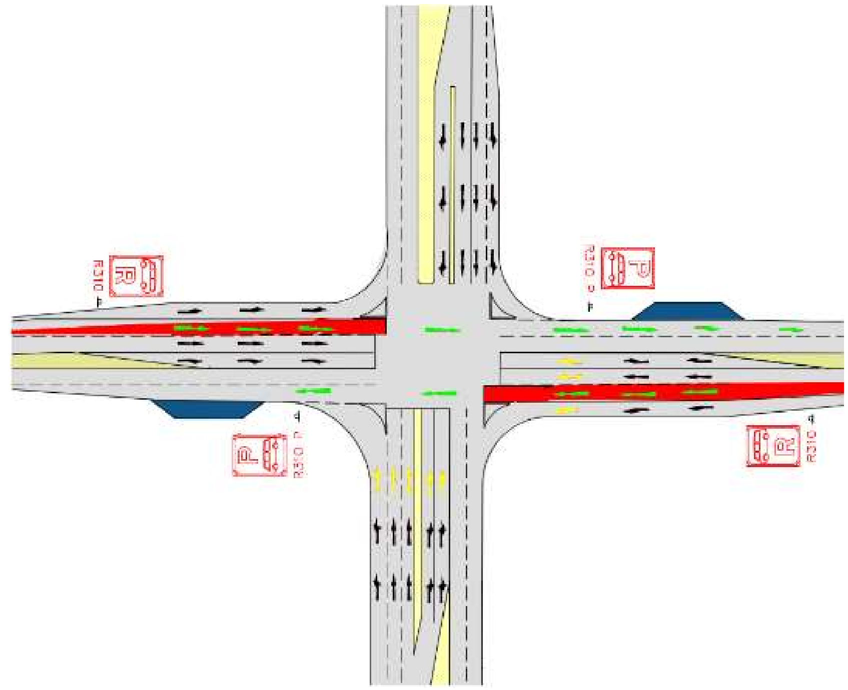
DESIGN OPTION ONE	DESIGN OPTION TWO
`J4 	J4 

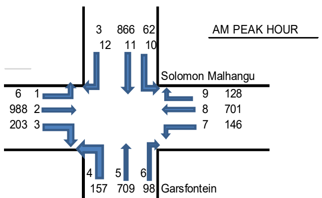
APPENDIX C: PEAK HR TRAFFIC VOLUMES

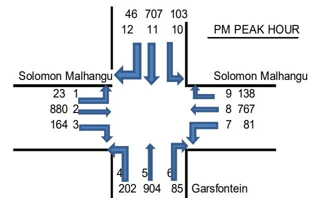
90 MINUTES TRAFFIC COUNTS

Analyst: John-Paul Mwenda
University University of Pretoria
Research Title Development and Evaluation of Design Strategies of Priority Infrastructure for Minibus-Taxis at Signalised Intersections
Intersection: J1 (Solomon Malhangu & Garsfontein)
Jurisdiction: City of Tshwane

PEAK HOUR SUMMARY: Light vehicle, Minibus Taxis & Heavy (veh/hr)
AM Peak Hour: 6:30 to 7:30
PM Peak Hour: 16:00 to 17:00

Solomon Malhangu



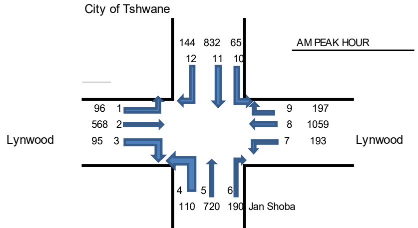


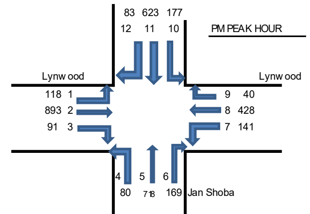
Time		Light Vehicles – Movement												Minibus Taxis – Movement												Heavy Vehicles – Movement													Total vehicles per 15 mins	Total vehicles per hr
		1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12			
05:00:00	05:15:00																																						0	0
05:15:00	05:30:00																																						0	0
05:30:00	05:45:00																																						0	0
05:45:00	06:00:00																																						0	0
06:00:00	06:15:00																																						0	0
06:15:00	06:30:00																																						0	0
06:30:00	06:45:00	0	296	63	35	132	23	28	159	35	20	263	0	0	0	0	7	18	2	2	10	0	0	16	0	0	3	0	0	2	0	0	4	2	1	1	0		1122	1122
06:45:00	07:00:00	2	228	51	23	156	17	39	170	25	7	189	0	0	1	0	1	13	0	3	6	0	2	12	0	0	2	0	0	1	0	0	2	1	1	0	0		952	2074
07:00:00	07:15:00	2	242	43	39	184	27	47	162	31	10	179	1	0	2	0	0	11	0	0	6	0	0	8	0	0	3	0	0	3	0	0	2	0	0	0	0		1002	3076
07:15:00	07:30:00	2	208	46	49	177	28	27	169	33	21	180	2	0	3	0	1	9	1	0	7	0	0	18	0	0	0	0	2	3	0	0	4	1	0	0	0		991	4067
07:30:00	07:45:00	12	195	42	37	139	17	13	156	38	13	167	3	0	1	0	0	14	3	0	5	0	1	12	0	0	1	0	2	6	0	0	4	0	1	1	1		884	3829
07:45:00	08:00:00	1	133	23	16	68	12	5	98	14	19	119	1	0	1	0	1	8	0	0	2	0	1	8	0	0	1	0	0	0	0	0	0	0	1	1	3		536	3413
08:00:00	08:15:00																																						0	2411
08:15:00	08:30:00																																						0	1420
08:30:00	08:45:00																																						0	536
08:45:00	09:00:00																																						0	0
09:00:00	09:15:00																																						0	0
09:15:00	09:30:00																																						0	0
09:30:00	09:45:00																																						0	0
09:45:00	10:00:00																																						0	0
10:00:00	10:15:00																																						0	0
10:15:00	10:30:00																																						0	0
10:30:00	10:45:00																																						0	0
10:45:00	11:00:00																																						0	0
11:00:00	11:15:00																																						0	0
11:15:00	11:30:00																																						0	0
11:30:00	11:45:00																																						0	0
11:45:00	12:00:00																																						0	0
12:00:00	12:15:00																																						0	0
12:15:00	12:30:00																																						0	0
12:30:00	12:45:00																																						0	0
12:45:00	13:00:00																																						0	0
13:00:00	13:15:00																																						0	0
13:15:00	13:30:00																																						0	0
13:30:00	13:45:00																																						0	0
13:45:00	14:00:00																																						0	0
14:00:00	14:15:00																																						0	0
14:15:00	14:30:00																																						0	0
14:30:00	14:45:00																																						0	0
14:45:00	15:00:00																																						0	0
15:00:00	15:15:00																																						0	0
15:15:00	15:30:00																																						0	0
15:30:00	15:45:00																																						0	0
15:45:00	16:00:00																																						0	0
16:00:00	16:15:00	7	249	44	56	237	22	22	252	46	24	184	14	0	3	3	1	14	0	0	2	1	1	11	0	0	6	3	0	2	1	0	0	0	3	6	5		1219	1219
16:15:00	16:30:00	8	203	35	41	204	21	20	161	35	21	164	10	1	2	0	0	7	0	0	4	0	0	9	0	0	5	2	0	2	0	0	0	0	1	3	0		959	2178
16:30:00	16:45:00	7	205	37	55	205	17	19	181	23	27	152	6	0	6	2	1	10	0	0	0	1	0	11	1	0	2	0	0	2	0	0	0	0	0	2	0		972	3150
16:45:00	17:00:00	0	192	34	46	210	21	20	167	32	26	155	8	0	5	4	1	10	3	0	0	0	0	9	0	0	2	0	1	1	0	0	0	0	0	1	2		950	4100
17:00:00	17:15:00	5	191	37	54	207	23	25	221	28	23	136	9	0	3	5	0	7	0	0	1	0	0	8	0	0	0	2	0	2	0	0	0	0	1	5	1		994	3875
17:15:00	17:30:00	3	156	26	40	191	14	25	205	27	30	149	11	0	3	3	0	11	1	0	3	0	0	9	0	0	6	5	0	0	0	0	0	0	0	3	0		921	3837
17:30:00	17:45:00	1	95	13	18	104	9	16	118	19	13	73	1	0	1	1	0	4	0	0	0	0	0	6	0	0	1	1	0	1	0	0	0	0	0	2	0		497	3362
17:45:00	18:00:00																																						0	2412
18:00:00	18:15:00																																						0	1418
18:15:00	18:30:00																																						0	497
18:30:00	18:45:00																																						0	0
18:45:00	19:00:00																																						0	0
19:00:00	19:15:00																																						0	0
19:15:00	19:30:00																																						0	0
19:30:00	19:45:00																																						0	0
19:45:00	20:15:00																																						0	0
20:00:00	20:30:00																																						0	0
20:15:00	20:30:00																																						0	0
20:30:00	00:00:00																																						0	0
Am Peak 1hr vol:		6	974	203	146	649	95	141	660	124	58	811	3	0	6	0	9	51	3	5	29	0	2	54	0	0	8	0	2	9	0	0	12	4	2	1	0	0	**1122**	**4067**
Am Peak 1hr vol:		6	988	203	157	709	98	146	701	128	62	866	3		1%		6%	7%	3%	3%	4%		3%	6%			1%		1%	1%			2%	3%	3%	0%			**0.91**	**4067**
																																							PHF	
PM Peak 1hr vol:		23	880	164	202	904	85	81	767	138	103	707	46	4%	2%	5%	1%	5%	4%		1%	1%	1%	6%	2%		2%	3%	0%	1%	1%				4%	2%	15%		**0.84**	**4100**
PM Peak 1hr vol:		22	849	150	198	856	81	81	761	136	98	655	38	1	16	9	3	41	3	0	6	2	1	40	1	0	15	5	1	7	1	0	0	0	4	12	7	0	**1219**	**4100**

90 MINUTES TRAFFIC COUNTS

Analyst: John-Paul Mwenda
University University of Pretoria
Research Title Development and Evaluation of Design Strategies of Priority Infrastructure for Minibus-Taxis at Signalised Intersections
Intersection: Lynwood & Jan Shoba
Jurisdiction: City of Tshwane

PEAK HOUR SUMMARY: Light, Minibus Taxis & Heavy veh/hr
AM Peak Hour: 6:45 to 7:45
PM Peak Hour: 16:30 to 17:30





Time		Light Vehicles Movement													Minibus Taxis Movement												Heavy Taxis Movement													Total vehicles per 15 mins	Total vehicles per hr
		1	2	3	4	5	6	7	8	9	10	11	12		1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12			
05:00:00	05:15:00																																							0	0
05:15:00	05:30:00																																							0	0
05:30:00	05:45:00																																							0	0
05:45:00	06:00:00																																							0	0
06:00:00	06:15:00																																							0	0
06:15:00	06:30:00																																							0	0
06:30:00	06:45:00	17	89	3	10	78	11	10	173	12	5	121	25		0	6	0	0	6	1	1	5	6	0	9	0	1	1	0	0	1	0	1	4	1	2	6	0		605	605
06:45:00	07:00:00	26	114	14	21	132	34	29	248	23	1	195	45		0	6	0	0	5	0	1	3	6	1	10	0	0	1	1	1	0	0	0	8	2	4	4	0		935	1540
07:00:00	07:15:00	27	139	28	30	207	57	58	326	35	15	196	34		0	7	0	1	7	1	1	11	7	1	12	0	0	3	1	2	4	0	1	3	3	3	2	0		1222	2762
07:15:00	07:30:00	25	140	26	34	167	48	47	230	55	22	209	31		0	5	0	0	4	1	0	6	13	1	7	0	0	3	1	2	3	2	4	2	1	2	5	0		1096	3858
07:30:00	07:45:00	18	131	24	19	184	47	51	214	39	8	173	33		0	14	0	0	4	0	0	8	12	2	11	1	0	5	0	0	3	0	1	0	1	5	8	0		1016	4269
07:45:00	08:00:00	23	108	25	18	155	37	52	193	31	7	152	25		0	10	0	1	2	1	0	7	9	2	6	2	0	5	1	0	3	0	2	1	2	4	3	0		887	4221
08:00:00	08:15:00																																							0	2999
08:15:00	08:30:00																																							0	1903
08:30:00	08:45:00																																							0	887
08:45:00	09:00:00																																							0	0
09:00:00	09:15:00																																							0	0
09:15:00	09:30:00																																							0	0
09:30:00	09:45:00																																							0	0
09:45:00	10:00:00																																							0	0
10:00:00	10:15:00																																							0	0
10:15:00	10:30:00																																							0	0
10:30:00	10:45:00																																							0	0
10:45:00	11:00:00																																							0	0
11:00:00	11:15:00																																							0	0
11:15:00	11:30:00																																							0	0
11:30:00	11:45:00																																							0	0
11:45:00	12:00:00																																							0	0
12:00:00	12:15:00																																							0	0
12:15:00	12:30:00																																							0	0
12:30:00	12:45:00																																							0	0
12:45:00	13:00:00																																							0	0
13:00:00	13:15:00																																							0	0
13:15:00	13:30:00																																							0	0
13:30:00	13:45:00																																							0	0
13:45:00	14:00:00																																							0	0
14:00:00	14:15:00																																							0	0
14:15:00	14:30:00																																							0	0
14:30:00	14:45:00																																							0	0
14:45:00	15:00:00																																							0	0
15:00:00	15:15:00																																							0	0
15:15:00	15:30:00																																							0	0
15:30:00	15:45:00																																							0	0
15:45:00	16:00:00																																							0	0
16:00:00	16:15:00	16	134	7	1	105	8	7	71	4	9	61	14		1	4	0	0	7	0	1	3	0	6	3	0	0	1	1	1	4	1	0	2	1	1	1	0		475	475
16:15:00	16:30:00	40	191	13	7	185	19	22	67	4	16	110	23		0	3	0	0	7	1	0	6	1	6	5	0	0	5	0	0	2	1	0	1	2	1	0	0		738	1213
16:30:00	16:45:00	30	285	24	18	179	46	37	85	8	32	191	19		0	6	1	1	6	0	1	7	1	7	5	1	0	2	0	0	1	1	0	2	2	2	4	0		1004	2217
16:45:00	17:00:00	25	195	30	19	188	35	41	116	12	41	117	19		0	6	0	0	7	0	1	5	1	9	4	0	0	2	2	1	5	2	2	2	2	1	3	1		894	3111
17:00:00	17:15:00	33	190	17	18	163	33	33	98	3	34	134	18		1	8	1	0	7	1	0	6	2	9	3	0	0	0	0	0	4	1	0	2	2	1	2	0		824	3460
17:15:00	17:30:00	29	192	15	21	150	49	25	94	2	31	155	25		0	5	0	0	5	0	1	6	2	8	3	0	0	2	1	2	3	1	0	5	3	2	2	0		839	3561
17:30:00	17:45:00																																							0	2557
17:45:00	18:00:00																																							0	1663
18:00:00	18:15:00																																							0	839
18:15:00	18:30:00																																							0	0
18:30:00	18:45:00																																							0	0
18:45:00	19:00:00																																							0	0
19:00:00	19:15:00																																							0	0
19:15:00	19:30:00																																							0	0
19:30:00	19:45:00																																							0	0
19:45:00	20:15:00																																							0	0
20:00:00	20:30:00																																							0	0
20:15:00	20:30:00																																							0	0
20:30:00	00:00:00																																							0	0
																																								0	0
Am Peak l hr vol:		96	524	92	104	690	186	185	1018	152	46	773	143	0	0	32	0	1	20	2	2	28	38	5	40	1	0	12	3	5	10	2	6	13	7	14	19	0	0	1222	4269
Am Peak l hr vol:		96	568	95	110	720	190	193	1059	197	65	832	144	0		6%		1%	3%	1%	1%	3%	19%	8%	5%	1%		2%	3%	5%	1%	1%	3%	1%	4%	22%	2%			0.87	4269
																																								PHF	
PM Peak 1hr vol:		118	893	91	80	718	169	141	428	40	177	623	83	0	1%	3%	2%	1%	3%	1%	2%	6%	15%	19%	2%	1%		1%	3%	4%	2%	3%	1%	3%	23%	3%	2%	1%		0.89	3561
PM Peak 1hr vol:		117	862	86	76	680	163	136	393	25	138	597	81	0	1	25	2	1	25	1	3	24	6	33	15	1	0	6	3	3	13	5	2	11	9	6	11	1	0	1004	3561

APPENDIX D: DETAILED CAPACITY ANALYSIS OF J1

J1_AM Existing Conditions

	EB				WB				NB			SB		
	LT		TH	RT	LT	TH		RT	LT	TH	RT	LT	TH	RT
	1		2	3	7	8		9	4	5	6	10	11	12
Volume Input														
Passenger Car Units, V (veh/h)	6		996	203	146	713		132	159	718	98	64	867	3
Peak-Hour Factor, PHF	0.91		0.91	0.91	0.91	0.91		0.91	0.91	0.91	0.91	0.91	0.91	0.91
Adjusted Flow Rate, v_p, (veh/h)	7		1095	224	161	784		146	175	790	108	71	953	4
Individual Lane Capacity Analysis														
Approach Lane Geometry														
Adjusted flow rate per individual lane, v_p'	7	547.5	547.5	224	161	392	392	146	175	790	108	512	512	4
Adj Saturated Flow Rate, s, (veh/hr)	1518	1835	1835	1031	1518	1819	1819	757	1499	1827	887	1626	1848	902
Green Ratio (g/C)	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.47	0.47	0.47	0.47	0.47	0.47
Lane Capacity, c = s(g/C), (veh/hr)	587	710	710	399	587	704	704	293	700	853	414	759	863	421
v/c Ratio, X=v_p/c	**0.01**	**0.77**	**0.77**	**0.56**	**0.27**	**0.56**	**0.56**	**0.50**	**0.25**	**0.93**	**0.26**	**0.67**	**0.59**	**0.01**
LOS by v/c ratio (Othayoth & Rao, 2019)	A	B	B	A	A	A	A	A	A	C	A	B	A	A
Lane Group Capacity Analysis														
Lane Group Geometry														
Adjusted Lane Group Saturation Rate (s)	1518	3670		1031	1518	3638		757	1499	1827	887	3474		902
Lane Group Capacity, c = s(g/C), (veh/hr)	587	1420		399	587	1408		293	700	853	414	1622		421
Adjusted Lane Group Flow Rate, v_p, (veh/h)	7	1095		224	161	784		146	175	790	108	1024		4
Lane Group v/c Ratio, X=v_p/c	0.01	0.77		0.56	0.27	0.56		0.50	0.25	0.93	0.26	0.63		0.01
Lane Group Flow Ratio=v_p/s	0.00	0.30		0.22	0.11	0.22		0.19	0.12	0.43	0.12	0.29		0.00
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	**0.794**													
Uniform Delay, d_1 (s/veh)	14.17	20.10		18.02	15.78	17.98		17.47	12.08	18.79	12.15	15.12		10.71
Incremental Delay, d_2, (s/veh)	0.04	4.11		5.61	1.15	1.59		5.95	0.85	17.38	1.53	1.88		0.04
Delay, d = d_1(PF) + d_2 + d_3 (s/veh)	**14.21**	**24.21**		**23.63**	**16.94**	**19.57**		**23.42**	**12.93**	**36.16**	**13.67**	**17.00**		**10.76**
LOS by Lane Group (TRB, 2010)	B	C		C	B	B		C	B	D	B	B		B
Delay by Approach, d_A, (s/veh)	24.1				19.70				30.11			16.98		
Intersection Delay, d_I, (s/veh)	23.1													

J1_PM Existing Conditions

	EB				WB				NB			SB		
	LT		TH	RT	LT	TH		RT	LT	TH	RT	LT	TH	RT
	1		2	3	7	8		9	4	5	6	10	11	12
Volume Input														
Passenger Car Units, V (veh/h)	23		895	169	81	767		138	203	911	86	107	719	53
Peak-Hour Factor, PHF	0.84		0.84	0.84	0.84	0.84		0.84	0.84	0.84	0.84	0.84	0.84	0.84
Adjusted Flow Rate, v_p, (veh/h)	28		1066	202	97	914		165	242	1085	103	128	856	64
Individual Lane Capacity Analysis														
Approach Lane Geometry														
Adjusted flow rate per individual lane, v_p'	28	533	533	202	97	457	457	165	242	1085	103	492	492	64
Adj Saturated Flow Rate, s, (veh/hr)	1518	1819	1819	897	1518	1850	1850	794	1510	1836	896	1592	1819	577
Green Ratio (g/C)	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.49	0.47	0.47	0.47	0.47	0.47
Lane Capacity, c = s(g/C), (veh/hr)	587	704	704	347	587	716	716	308	745	857	419	743	849	270
v/c Ratio, $X=v_p/c$	**0.05**	**0.76**	**0.76**	**0.58**	**0.17**	**0.64**	**0.64**	**0.54**	**0.32**	**1.27**	**0.25**	**0.66**	**0.58**	**0.24**
LOS by v/c ratio (Othayoth & Rao, 2019)	A	B	B	A	A	B	B	A	A	F	A	B	A	A
Lane Group Capacity Analysis														
Lane Group Geometry														
Adjusted Lane Group Saturation Rate (s)	1518	3638		897	1518	3700		794	1510	1836	896	3411		577
Lane Group Capacity, c = s(g/C), (veh/hr)	587	1408		347	587	1432		308	745	857	419	1592		270
Adjusted Lane Group Flow Rate, v_p, (veh/h)	28	1066		202	97	914		165	242	1085	103	984		64
Lane Group v/c Ratio, $X=v_p/c$	0.05	0.76		0.58	0.17	0.64		0.54	0.32	1.27	0.25	0.62		0.24
Lane Group Flow Ratio=v_p/s	0.02	0.29		0.23	0.06	0.25		0.21	0.16	0.59	0.11	0.29		0.11
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	**0.96**													
Uniform Delay, d_1 (s/veh)	14.37	19.95		18.20	15.07	18.73		17.79	11.46	26.07	12.05	14.99		11.99
Incremental Delay, d_2, (s/veh)	0.15	3.85		6.97	0.61	2.19		6.54	1.16	129.00	1.39	1.81		2.06
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	**14.53**	**23.79**		**25.17**	**15.68**	**20.92**		**24.33**	**12.62**	**155.06**	**13.44**	**16.80**		**14.05**
LOS by Lane Group (TRB, 2010)	B	C		C	B	C		B	B	F	B	C		C
Delay by Approach, d_A, (s/veh)	23.8				20.96				120.76			16.63		
Intersection Delay, d_I, (s/veh)	49.6													

J1_AM Design Strategy 1a (DS1a) (Modification to Geometric but Without Modification to Traffic Signals)

	EB				WB				NB			SB		
	LT		TH	RT	LT	TH		RT	LT	TH	RT	LT	TH	RT
	1		2	3	7	8		9	4	5	6	10	11	12
Volume Input														
Straight Movement of MBT			6			29				51			54	
Passenger Car Units, V (veh/h)	6		996	203	146	713		132	210	667	98	118	813	3
Peak-Hour Factor, PHF	0.91		0.91	0.91	0.91	0.91		0.91	0.91	0.91	0.91	0.91	0.91	0.91
Adjusted Flow Rate, v_p, (veh/h)	7		1095	224	161	784		146	231	733	108	130	894	4
Individual Lane Capacity Analysis														
Approach Lane Geometry														
Description of Traffic Movements	Left (ALL)	Straight (ALL)	Straight (ALL)	Right (ALL)	Left (ALL)	Straight (ALL)	Straight (ALL)	Right (ALL)	Left (All)+ Straight (MBT)	Straight (ALL)	Right (ALL)	Left (All)+ Straight(MBT)	Straight (ALL)	Right Turn (ALL)
Adjusted flow rate per individual lane, v_p'	7	547.5	547.5	224	161	392	392	146	231	733	108	130	894	4
Adj Saturated Flow Rate, s, (veh/hr)	1518	1835	1835	1031	1518	1819	1819	757	1540	1827	872	1549	1848	913
Green Ratio (g/C)	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.39	0.47	0.47	0.47	0.47	0.47	0.47
Lane Capacity, c = s(g/C), (veh/hr)	587	710	710	399	587	704	704	293	719	853	407	723	863	427
v/c Ratio, $X=v_p/c$	**0.012**	**0.771**	**0.771**	**0.561**	**0.274**	**0.557**	**0.557**	**0.498**	**0.321**	**0.859**	**0.265**	**0.180**	**1.036**	**0.009**
LOS by v/c ratio (Othayoth & Rao, 2019)	A	B	B	A	A	A	A	A	A	C	A	A	D	A
Lane Group Capacity Analysis														
Lane Group Geometry														
Adjusted Lane Group Saturation Rate (s)	1518		3670	1031	1518	3638		757	1540	1827	872	1549	1848	913
Lane Group Capacity, c = s(g/C), (veh/hr)	587		1420	399	587	1408		293	719	853	407	723	863	427
Adjusted Lane Group Flow Rate, v_p, (veh/h)	7		1095	224	161	784		146	231	733	108	130	894	4
Lane Group v/c Ratio, $X=v_p/c$	0.01		0.77	0.56	0.27	0.56		0.50	0.32	0.86	0.27	0.18	1.04	0.01
Lane Group Flow Ratio=v_p/s	0.00		0.30	0.22	0.11	0.22		0.19	0.15	0.40	0.12	0.08	0.48	0.00
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	**0.850**													
Uniform Delay, d_1 (s/veh)	14.17		20.10	18.02	15.78	17.98		17.47	12.55	17.81	12.17	11.64	20.65	10.71
Incremental Delay, d_2, (s/veh)	0.04		4.11	5.61	1.15	1.59		5.95	1.18	10.98	1.59	0.54	40.29	0.04
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	**14.21**		**24.21**	**23.63**	**16.94**	**19.57**		**23.42**	**13.73**	**28.79**	**13.76**	**12.19**	**60.94**	**10.75**
LOS by Lane Group (TRB, 2010)	B		C	C	B	B		B	B	C	B	B	E	B
Delay by Approach, d_A, (s/veh)	24.1				19.70				24.03			54.58		
Intersection Delay, d_I, (s/veh)	29.9													

J1_PM Design Strategy 1a (Modification to Geometric but Without Modification to Traffic Signals)

	EB				WB				NB			SB		
	LT		TH	RT	LT	TH		RT	LT	TH	RT	LT	TH	RT
	1		2	3	7	8		9	4	5	6	10	11	12
Volume Input														
Straight Movement of MBT			16			6				51			54	
Passenger Car Units, V (veh/h)	23		895	169	81	767		138	203	860	86	161	665	53
Peak-Hour Factor, PHF	0.84		0.84	0.84	0.84	0.84		0.84	0.84	0.84	0.84	0.84	0.84	0.84
Adjusted Flow Rate, v_p, (veh/h)	28		1066	202	97	914		165	242	1024	103	192	792	64
Individual Lane Capacity Analysis														
Approach Lane Geometry														
Description of Traffic Movements	Left (ALL)	Straight (ALL)	Straight (ALL)	Right (ALL)	Left (ALL)	Straight (ALL)	Straight (ALL)	Right (ALL)	Left (All)+ Straight (MBT)	Straight (ALL)	Right (ALL)	Left (All)+ Straight (MBT)	Straight (ALL)	Right (ALL)
Adjusted flow rate per individual lane, v_p'	28	533	533	202	97	457	457	165	242	1024	103	192	792	64
Adj Saturated Flow Rate, s, (veh/hr)	1518	1819	1819	897	1518	1850	1850	794	1554	1836	879	1518	1819	614
Green Ratio (g/C)	**0.39**	**0.39**	**0.39**	**0.39**	**0.39**	**0.39**	**0.39**	**0.39**	**0.47**	**0.47**	**0.47**	**0.47**	**0.47**	**0.47**
Lane Capacity, c = s(g/C), (veh/hr)	587	704	704	347	587	716	716	308	726	857	411	709	849	287
v/c Ratio, X=v_p/c	**0.048**	**0.757**	**0.757**	**0.582**	**0.165**	**0.638**	**0.638**	**0.536**	**0.333**	**1.195**	**0.251**	**0.271**	**0.933**	**0.223**
LOS by v/c ratio (Othayoth & Rao, 2019)	A	B	B	A	A	B	B	A	A	F	A	A	C	A
Lane Group Capacity Analysis														
Lane Group Geometry														
Adjusted Lane Group Saturation Rate (s)	1518		3638	897	1518	3700		794	1554	1836	879	1518	1819	614
Lane Group Capacity, c = s(g/C), (veh/hr)	587		1408	347	587	1432		308	726	857	411	709	849	287
Adjusted Lane Group Flow Rate, v_p, (veh/h)	28		1066	202	97	914		165	242	1024	103	192	792	64
Lane Group v/c Ratio, X=v_p/c	0.048		0.757	0.582	0.165	0.638		0.536	0.333	1.195	0.251	0.271	0.933	0.223
Lane Group Flow Ratio=v_p/s	0.018		0.293	0.225	0.064	0.247		0.208	0.156	0.558	0.117	0.126	0.435	0.104
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	0.925													
Uniform Delay, d_1 (s/veh)	14.37		19.95	18.20	15.07	18.73		17.79	12.63	24.11	12.08	12.21	18.89	11.91
Incremental Delay, d_2, (s/veh)	0.15		3.85	6.97	0.61	2.19		6.54	1.23	99.09	1.46	0.94	18.33	1.79
Delay, d = d_1(PF) + d_2 + d_3 (s/veh)	**14.53**		**23.79**	**25.17**	**15.68**	**20.92**		**24.33**	**13.87**	**123.20**	**13.54**	**13.15**	**37.22**	**13.70**
LOS by Lane Group (TRB, 2010)	B		C	C	B	C		C	B	F	B	B	D	B
Delay by Approach, d_A, (s/veh)	23.8				20.96				95.62			31.38		
Intersection Delay, d_I, (s/veh)	44.9													

J1_AM Design Strategy 1b (Modification to both Geometric and Traffic Signals)

	EB				WB				NB			SB		
	LT		TH	RT	LT	TH		RT	LT	TH	RT	LT	TH	RT
	1		2	3	7	8		9	4	5	6	10	11	12
Volume Input														
Straight Movement of MBT			6			29				51			54	
Passenger Car Units, V (veh/h)	6		996	203	146	713		132	210	667	98	118	813	3
Peak-Hour Factor, PHF	0.91		0.91	0.91	0.91	0.91		0.91	0.91	0.91	0.91	0.91	0.91	0.91
Adjusted Flow Rate, v_p, (veh/h)	7		1095	224	161	784		146	231	733	108	130	894	4
Individual Lane Capacity Analysis														
Approach Lane Geometry														
Description of Traffic Movements	Left (ALL)	Straight (ALL)	Straight (ALL)	Right (ALL)	Left (ALL)	Straight (ALL)	Straight (ALL)	Right (ALL)	Left (All)+ Straight (MBT)	Straight (ALL)	Right (ALL)	Left (All)+ Straight(MBT)	Straight (ALL)	Right Turn (ALL)
Adjusted flow rate per individual lane, v_p'	7	547.5	547.5	224	161	392	392	146	231	733	108	130	894	4
Adj Saturated Flow Rate, s, (veh/hr)	1518	1835	1835	921	1518	1819	1819	677	1540	1827	982	1549	1848	1022
Green Ratio (g/C)	**0.31**	**0.31**	**0.31**	**0.31**	**0.31**	**0.31**	**0.31**	**0.31**	**0.56**	**0.56**	**0.56**	**0.56**	**0.56**	**0.56**
Lane Capacity, c = s(g/C), (veh/hr)	466	563	563	283	466	558	558	208	863	1024	550	868	1035	573
v/c Ratio, $X=v_p/c$	**0.015**	**0.972**	**0.972**	**0.792**	**0.345**	**0.703**	**0.703**	**0.702**	**0.268**	**0.716**	**0.196**	**0.150**	**0.864**	**0.007**
LOS by v/c ratio (Othayoth & Rao, 2019)	A	D	D	B	A	B	B	B	A	B	A	A	C	A
Lane Group Capacity Analysis														
Lane Group Geometry														
Adjusted Lane Group Saturation Rate (s)	1518		3670	921	1518	3638		677	1540	1827	982	1549	1848	1022
Lane Group Capacity, c = s(g/C), (veh/hr)	466		1126	283	466	1116		208	863	1024	550	868	1035	573
Adjusted Lane Group Flow Rate, v_p, (veh/h)	7		1095	224	161	784		146	231	733	108	130	894	4
Lane Group v/c Ratio, $X=v_p/c$	0.015		0.972	0.792	0.345	0.703		0.702	0.268	0.716	0.196	0.150	0.864	0.007
Lane Group Flow Ratio=v_p/s	0.005		0.298	0.243	0.106	0.216		0.216	0.150	0.401	0.110	0.084	0.484	0.004
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	0.850													
Uniform Delay, d_1 (s/veh)	18.11		25.69	23.80	20.16	22.98		22.97	8.54	12.12	8.16	7.92	14.06	7.29
Incremental Delay, d_2, (s/veh)	0.06		20.97	19.92	2.03	3.71		17.97	0.76	4.28	0.80	0.36	9.54	0.02
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	**18.17**		**46.66**	**43.72**	**22.19**	**26.68**		**40.94**	**9.30**	**16.40**	**8.95**	**8.29**	**23.60**	**7.31**
LOS by Lane Group (TRB, 2010)	B		D	D	C	C		D	A	B	A	A	C	A
Delay by Approach, d_A, (s/veh)	46.0				27.93				14.12			21.60		
Intersection Delay, d_I, (s/veh)	28.5													

J1_PM Design Strategy 1b (Modification to both Geometric and Traffic Signals)

	EB				WB				NB			SB		
	LT		TH	RT	LT	TH		RT	LT	TH	RT	LT	TH	RT
	1		2	3	7	8		9	4	5	6	10	11	12
Volume Input														
Straight Movement of MBT			16			6				51			54	
Passenger Car Units, V (veh/h)	23		895	169	81	767		138	203	860	86	161	665	53
Peak-Hour Factor, PHF	0.84		0.84	0.84	0.84	0.84		0.84	0.84	0.84	0.84	0.84	0.84	0.84
Adjusted Flow Rate, v_p, (veh/h)	28		1066	202	97	914		165	242	1024	103	192	792	64
Individual Lane Capacity Analysis														
Approach Lane Geometry														
Description of Traffic Movements	Left (ALL)	Straight (ALL)	Straight (ALL)	Right (ALL)	Left (ALL)	Straight (ALL)	Straight (ALL)	Right (ALL)	Left (All)+ Straight (MBT)	Straight (ALL)	Right (ALL)	Left (All)+ Straight (MBT)	Straight (ALL)	Right (ALL)
Adjusted flow rate per individual lane, v_p'	28	533	533	202	97	457	457	165	242	1024	103	192	792	64
Adj Saturated Flow Rate, s, (veh/hr)	1518	1819	1819	797	1518	1850	1850	710	1554	1836	988	1518	1819	705
Green Ratio (g/C)	**0.31**	**0.31**	**0.31**	**0.31**	**0.31**	**0.31**	**0.31**	**0.31**	**0.56**	**0.56**	**0.56**	**0.56**	**0.56**	**0.56**
Lane Capacity, c = s(g/C), (veh/hr)	466	558	558	245	466	568	568	218	871	1029	554	851	1019	395
v/c Ratio, $X=v_p/c$	**0.060**	**0.955**	**0.955**	**0.824**	**0.208**	**0.805**	**0.805**	**0.757**	**0.278**	**0.995**	**0.186**	**0.226**	**0.777**	**0.162**
LOS by v/c ratio (Othayoth & Rao, 2019)	A	D	D	B	A	B	B	B	A	D	A	A	B	A
Lane Group Capacity Analysis														
Lane Group Geometry														
Adjusted Lane Group Saturation Rate (s)	1518		3638	797	1518	3700		710	1554	1836	988	1518	1819	705
Lane Group Capacity, c = s(g/C), (veh/hr)	466		1116	245	466	1136		218	871	1029	554	851	1019	395
Adjusted Lane Group Flow Rate, v_p, (veh/h)	28		1066	202	97	914		165	242	1024	103	192	792	64
Lane Group v/c Ratio, $X=v_p/c$	0.060		0.955	0.824	0.208	0.805		0.757	0.278	0.995	0.186	0.226	0.777	0.162
Lane Group Flow Ratio=v_p/s	0.018		0.293	0.253	0.064	0.247		0.232	0.156	0.558	0.104	0.126	0.435	0.091
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	0.925													
Uniform Delay, d_1 (s/veh)	18.37		25.49	24.13	19.26	23.93		23.48	8.60	16.40	8.10	8.31	12.86	7.98
Incremental Delay, d_2, (s/veh)	0.25		18.11	25.97	1.01	6.10		21.49	0.79	26.92	0.74	0.62	5.82	0.88
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	**18.61**		**43.61**	**50.10**	**20.27**	**30.03**		**44.96**	**9.39**	**43.31**	**8.84**	**8.93**	**18.68**	**8.86**
LOS by Lane Group (TRB, 2010)	B		D	D	C	C		D	A	D	A	A	B	A
Delay by Approach, d_A, (s/veh)	44.1				31.32				34.72			16.29		
Intersection Delay, d_I, (s/veh)	32.4													

APPENDIX E: DETAILED CAPACITY ANALYSIS OF J2

J2_AM Existing Conditions

	EB			WB				NB			SB			
	LT	TH	RT	LT	TH		RT	LT	TH	RT	LT	TH		RT
	1	2	3	7	8		9	4	5	6	10	11		12
Volume Input														
Passenger Car Units, V (veh/h)	96	580	98	199	1072		204	115	730	192	79	851		144
Peak-Hour Factor, PHF	0.87	0.87	0.87	0.87	0.87		0.87	0.87	0.87	0.87	0.87	0.87		0.87
Adjusted Flow Rate, v_p, (veh/h)	111	667	113	229	1233		235	133	840	221	91	979		166
Individual Lane Capacity Analysis														
Approach Lane Geometry														
Adjusted flow rate per individual lane, v_p'	389	389	113	488	488	488	235	487	487	221	91	490	490	166
Adj Saturated Flow Rate, s, (veh/hr)	1649	1812	980	1564	1828	1850	991	1579	1825	1341	1249	1809	1850	699
Green Ratio (g/C)	0.31	0.31	0.31	0.42	0.42	0.42	0.42	0.46	0.46	0.46	0.31	0.31	0.31	0.31
Lane Capacity, c = s(g/C), (veh/hr)	509	560	303	657	768	777	416	722	834	613	386	559	571	216
v/c Ratio, $X=v_p/c$	**0.764**	**0.695**	**0.373**	**0.743**	**0.635**	**0.628**	**0.565**	**0.675**	**0.584**	**0.361**	**0.236**	**0.877**	**0.858**	**0.769**
LOS by v/c ratio (Othayoth & Rao, 2019)	B	B	A	B	B	B	A	B	A	A	A	C	C	B
Flow Ratio (vp/s)	0.236	0.215	0.115	0.312	0.267	0.264	0.237	0.308	0.267	0.165	0.073	0.271	0.265	0.237
Lane Group Capacity Analysis														
Lane Group Geometry														
Adjusted Lane Group Saturation Rate (s)	3461		980	5242			991	3404		1341	1249	3659		699
Lane Group Capacity, c = s(g/C), (veh/hr)	1069		303	2202			416	1556		613	386	1130		216
Adjusted Lane Group Flow Rate, v_p, (veh/h)	778		113	1464			235	974		221	91	980		166
Lane Group v/c Ratio, $X=v_p/c$	0.728		0.373	0.665			0.565	0.626		0.361	0.236	0.867		0.769
Lane Group Flow Ratio=v_p/s	0.225		0.115	0.279			0.237	0.286		0.165	0.073	0.268		0.237
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	0.611													
Uniform Delay, d_1 (s/veh)	24.97		21.88	18.91			17.87	16.74		14.31	20.88	26.43		25.38
Incremental Delay, d_2, (s/veh)	4.35		3.49	1.60			5.47	1.91		1.65	1.43	9.04		22.71
Delay, d = d_1(PF) + d_2 + d_3 (s/veh)	**29.31**		**25.37**	**20.52**			**23.34**	**18.65**		**15.95**	**22.31**	**35.47**		**48.09**
LOS by Lane Group	C		C	C			C	B		B	C	C		D
Delay by Approach, d_A, (s/veh)	28.8			20.91				18.15			36.20			
Intersection Delay, d_I, (s/veh)	22.8													

J2_PM Existing Conditions

	EB			WB				NB			SB			
	LT	TH	RT	LT	TH		RT	LT	TH	RT	LT	TH		RT
	1	2	3	7	8		9	4	5	6	10	11		12
Volume Input														
Passenger Car Units, V (veh/h)	118	899	94	143	439		49	83	731	174	183	634		84
Peak-Hour Factor, PHF	0.89	0.89	0.89	0.89	0.89		0.89	0.89	0.89	0.89	0.89	0.89		0.89
Adjusted Flow Rate, v_p, (veh/h)	133	1011	106	161	494		56	94	822	196	206	713		95
Individual Lane Capacity Analysis														
Approach Lane Geometry														
Adjusted flow rate per individual lane, v_p'	572	572	106	219	219	219	56	458	458	196	206	357	357	95
Adj Saturated Flow Rate, s, (veh/hr)	1659	1838	1463	1541	1804	1850	599	1605	1817	1403	1468	1818	1850	734
Green Ratio (g/C)	0.31	0.31	0.31	0.42	0.42	0.42	0.42	0.46	0.46	0.46	0.31	0.31	0.31	0.31
Lane Capacity, c = s(g/C), (veh/hr)	513	568	452	647	758	777	252	734	830	641	454	562	571	227
v/c Ratio, $X=v_p/c$	1.115	1.007	0.235	**0.338**	**0.289**	**0.282**	**0.222**	**0.624**	**0.552**	**0.306**	**0.454**	**0.635**	**0.625**	**0.419**
LOS by v/c ratio (Othayoth & Rao, 2019)	F	E	A	A	A	A	A	B	A	A	A	B	B	A
Flow Ratio (vp/s)	0.345	0.311	0.072	0.142	0.121	0.118	0.093	0.285	0.252	0.140	0.140	0.196	0.193	0.129
Lane Group Capacity Analysis														
Lane Group Geometry														
Adjusted Lane Group Saturation Rate (s)	3497		1463	5195			599	3422		1403	1468	3668		734
Lane Group Capacity, c = s(g/C), (veh/hr)	1081		452	2182			252	1564		641	454	1133		227
Adjusted Lane Group Flow Rate, v_p, (veh/h)	1144		106	657			56	916		196	206	714		95
Lane Group v/c Ratio, $X=v_p/c$	1.058279		0.235	0.301			0.222	0.586		0.306	0.454	0.630		0.419
Lane Group Flow Ratio=v_p/s	0.327		0.072	0.126			0.093	0.268		0.140	0.140	0.195		0.129
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	0.426													
Uniform Delay, d_1 (s/veh)	28.75		20.87	24.06			15.04	16.32		13.89	22.51	24.03		22.23
Incremental Delay, d_2, (s/veh)	44.18		1.22	0.35			2.03	1.61		1.23	3.25	2.66		5.59
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	**72.92**		**22.08**	**24.42**			**17.07**	**17.93**		**15.12**	**25.76**	**26.70**		**27.82**
LOS by Lane Group	E		C	C			B	B		B	C	C		C
Delay by Approach, d_A, (s/veh)	68.6			23.84				17.43			26.61			
Intersection Delay, d_I, (s/veh)	38.6													

J2_AM Design Strategy 1a (Modification to Geometric but Without Modification to Traffic Signals)

	EB				WB				NB			SB			
	LT	TH	TH	RT	LT	TH	TH	RT	LT	TH	RT	LT	TH		RT
	1		2	3	7	8		9	4	5	6	10	11		12
Volume Input															
Straight Movement of MBT			32			28				20			40		
Passenger Car Units, V (veh/h)	96	32	580	98	199	1072	28	204	115	730	192	79	851		144
Peak-Hour Factor, PHF	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87		0.87
Adjusted Flow Rate, v_p, (veh/h)	111	37	667	113	229	1233	33	235	133	840	221	91	979		166
Individual Lane Capacity Analysis															
Approach Lane Geometry															
Description of Traffic Movements	Left Turn (All)+ Straight (All)	Straight (MBT)	Straight (ALL)	Right Turn (ALL)	Left Turn (All)+ Straight (All)	Straight (MBT)	Straight (ALL)	Right Turn (ALL)	Left Turn (All)+ Straight	Straight (ALL)	Right (ALL)	Left Turn(ALL)	Straight (ALL)	Straight (ALL)	Right Turn (ALL)
Adjusted flow rate per individual lane, v_p'	389	37	389	113	731	33	731	235	487	487	221	91	490	490	166
Green Ratio (g/C)	0.31	0.31	0.31	0.31	0.42	0.42	0.42	0.42	0.46	0.46	0.46	0.31	0.31	0.31	0.31
Lane Capacity, c = s(g/C), (veh/hr)	509	560	560	298	669	768	768	523	722	834	613	386	559	571	216
v/c Ratio, $X=v_p/c$	**0.764**	**0.066**	**0.695**	**0.379**	**1.093**	**0.043**	**0.952**	**0.449**	**0.675**	**0.584**	**0.361**	**0.236**	**0.877**	**0.858**	**0.769**
LOS by v/c ratio (Othayoth & Rao, 2019)	B	A	B	A	E	A	D	A	B	A	A	A	C	C	B
Lane Group Capacity Analysis															
Lane Group Geometry															
Adjusted Lane Group Saturation Rate (s)	3461	1812		965	3421	1828		1245	3404		1341	1249	3659		699
Lane Group Capacity, c = s(g/C), (veh/hr)	1069	560		298	1437	768		523	1556		613	386	1130		216
Adjusted Lane Group Flow Rate, v_p, (veh/h)	778	37		113	1462	33		235	974		221	91	980		166
Lane Group v/c Ratio, $X=v_p/c$	0.728	0.066		0.379	1.017	0.043		0.449	0.626		0.361	0.236	0.867		0.769
Lane Group Flow Ratio=v_p/s	0.225	0.020		0.117	0.427	0.018		0.189	0.286		0.165	0.073	0.268		0.237
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	0.771														
Uniform Delay, d_1 (s/veh)	24.97	19.76		21.92	23.80	13.89		16.81	16.74		14.31	20.88	26.43		25.38
Incremental Delay, d_2, (s/veh)	4.35	0.23		3.64	28.18	0.11		2.78	1.91		1.65	1.43	9.04		22.71
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	**29.31**	**19.99**		**25.57**	**51.98**	**13.99**		**19.58**	**18.65**		**15.95**	**22.31**	**35.47**		**48.09**
LOS by Lane Group	C	B		C	D	B		B	B		B	C	D		D
Delay by Approach, d_A, (s/veh)	28.5				46.85				18.15			36.20			
Intersection Delay, d_I, (s/veh)	33.7														

J2_ PM Design Strategy 1a (Modification to Geometric but Without Modification to Traffic Signals)

	EB				WB				NB			SB			
	LT	TH	TH	RT	LT	TH		RT	LT	TH	RT	LT	TH		RT
	1		2	3	7	8		9	4	5	6	10	11		12
Volume Input															
Straight Movement of MBT			25			24				25			33		
Passenger Car Units, V (veh/h)	118	25	899	94	143	439	24	49	83	731	174	183	634		84
Peak-Hour Factor, PHF	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89		0.89
Adjusted Flow Rate, v_p, (veh/h)	133	29	1011	106	161	494	27	56	94	822	196	206	713		95
Individual Lane Capacity Analysis															
Approach Lane Geometry															
Description of Traffic Movements	Left Turn (All)+ Straight (All)	Straight (MBT)	Straight (ALL)	Right Turn (ALL)	Left Turn (All)+ Straight (All)	Straight (MBT)	Straight (ALL)	Right Turn (ALL)	Left Turn (All)+ Straight (All)	Straight (ALL)	Right (ALL)	Left Turn(ALL)	Straight (ALL)	Straight (ALL)	Right Turn (ALL)
Adjusted flow rate per individual lane, v_p'	572	29	572	106	165.05	27	419.9	56	458	458	196	206	357	357	95
Adj Saturated Flow Rate, s, (veh/hr)	1659	1838	1838	1484	1500	1804	1804	851	1605	1817	1403	1468	1818	1850	734
Green Ratio (g/C)	**0.31**	**0.31**	**0.31**	**0.31**	**0.42**	**0.42**	**0.42**	**0.42**	**0.46**	**0.46**	**0.46**	**0.31**	**0.31**	**0.31**	**0.31**
Lane Capacity, c = s(g/C), (veh/hr)	513	568	568	459	630	758	758	358	734	830	641	454	562	571	227
v/c Ratio, $X=v_p/c$	**1.115**	**0.051**	**1.007**	**0.231**	**0.262**	**0.036**	**0.554**	**0.156**	**0.624**	**0.552**	**0.306**	**0.454**	**0.635**	**0.625**	**0.419**
LOS by v/c ratio (Othayoth & Rao, 2019)	F	A	E	A	A	A	A	A	B	A	A	A	B	B	A
Lane Group Capacity Analysis															
Lane Group Geometry															
Adjusted Lane Group Saturation Rate (s)	3497	1838		1484	3304	1804		851	3422		1403	1468	3668		734
Lane Group Capacity, c = s(g/C), (veh/hr)	1081	568		459	1388	758		358	1564		641	454	1133		227
Adjusted Lane Group Flow Rate, v_p, (veh/h)	1144	29		106	585	27		56	916		196	206	714		95
Lane Group v/c Ratio, $X=v_p/c$	1.058	0.051		0.231	0.421	0.036		0.156	0.586		0.306	0.454	0.630		0.419
Lane Group Flow Ratio=v_p/s	0.327	0.016		0.071	0.177	0.015		0.066	0.268		0.140	0.140	0.195		0.129
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	0.642														
Uniform Delay, d_1 (s/veh)	28.75	19.67		20.84	16.57	13.84		14.59	16.32		13.89	22.51	24.03		22.23
Incremental Delay, d_2, (s/veh)	44.18	0.17		1.17	0.94	0.09		0.93	1.61		1.23	3.25	2.66		5.59
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	**72.92**	**19.84**		**22.02**	**17.51**	**13.93**		**15.52**	**17.93**		**15.12**	**25.76**	**26.70**		**27.82**
LOS by Lane Group (TRB, 2010)	E	B		C	B	B		B	B		B	C	C		C
Delay by Approach, d_A, (s/veh)	67.5				17.20				17.43			26.61			
Intersection Delay, d_I, (s/veh)	37.3														

J2_ AM Design Strategy 1b (Modification to both Geometric and Traffic Signals)

	EB				WB				NB			SB			
	LT	TH	TH	RT	LT	TH	TH	RT	LT	TH	RT	LT	TH		RT
	1		2	3	7	8		9	4	5	6	10	11		12
Volume Input															
Straight Movement of MBT			32			28				20			40		
Passenger Car Units, V (veh/h)	96	32	580	98	199	1072	28	204	115	730	192	79	851		144
Peak-Hour Factor, PHF	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87		0.87
Adjusted Flow Rate, v_p, (veh/h)	111	37	667	113	229	1233	33	235	133	840	221	91	979		166
Individual Lane Capacity Analysis															
Approach Lane Geometry															
Description of Traffic Movements	Left Turn (All)+ Straight (All)	Straight (MBT)	Straight (ALL)	Right Turn (ALL)	Left Turn (All)+ Straight (All)	Straight (MBT)	Straight (ALL)	Right Turn (ALL)	Left Turn (All)+ Straight (All)	Straight (ALL)	Right (ALL)	Left Turn(ALL)	Straight (ALL)	Straight (ALL)	Right Turn (ALL)
Adjusted flow rate per individual lane, v_p'	389	37	389	113	731	33	731	235	487	487	221	91	490	490	166
Adj Saturated Flow Rate, s, (veh/hr)	1649	1812	1812	1171	1593	1828	1828	1352	1579	1825	1184	1249	1809	1850	670
Green Ratio (g/C)	**0.47**	**0.47**	**0.47**	**0.47**	**0.58**	**0.58**	**0.58**	**0.58**	**0.31**	**0.31**	**0.31**	**0.27**	**0.27**	**0.27**	**0.27**
Lane Capacity, c = s(g/C), (veh/hr)	774	851	851	550	925	1061	1061	785	488	564	366	340	492	503	182
v/c Ratio, $X=v_p/c$	**0.503**	**0.043**	**0.457**	**0.205**	**0.790**	**0.031**	**0.689**	**0.299**	**0.998**	**0.863**	**0.604**	**0.268**	**0.996**	**0.974**	**0.912**
LOS by v/c ratio (Othayoth & Rao, 2019)	A	A	A	A	B	A	C	A	D	C	B	A	D	D	C
Lane Group Capacity Analysis															
Lane Group Geometry															
Adjusted Lane Group Saturation Rate (s)	3461	1812		1171	3421	1828		1352	3404		1184	1249	3659		670
Lane Group Capacity, c = s(g/C), (veh/hr)	1625	851		550	1986	1061		785	1052		366	340	995		182
Adjusted Lane Group Flow Rate, v_p, (veh/h)	778	37		113	1462	33		235	974		221	91	980		166
Lane Group v/c Ratio, $X=v_p/c$	0.479	0.043		0.205	0.736	0.031		0.299	0.926		0.604	0.268	0.985		0.912
Lane Group Flow Ratio=v_p/s	0.225	0.020		0.096	0.427	0.018		0.174	0.286		0.187	0.073	0.268		0.248
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	0.751														
Uniform Delay, d_1 (s/veh)	14.72	11.65		12.63	12.46	7.27		8.64	27.10		23.79	23.17	29.34		28.56
Incremental Delay, d_2, (s/veh)	1.01	0.10		0.84	2.48	0.05		0.98	14.80		7.20	1.92	25.13		46.93
Delay, d = d_1(PF) + d_2 + d_3 (s/veh)	**15.73**	**11.75**		**13.48**	**14.93**	**7.32**		**9.61**	**41.90**		**31.00**	**25.10**	**54.46**		**75.50**
LOS by Lane Group	B	B		B	A	A	D	A	D		C	C	D		E
Delay by Approach, d_A, (s/veh)	15.3				14.07				39.89			55.12			
Intersection Delay, d_I, (s/veh)	24.4														

J2_PM Design Strategy 1b (Modification to both Geometric and Traffic Signals)

	EB				WB				NB			SB			
	LT	TH	TH	RT	LT	TH		RT	LT	TH	RT	LT	TH		RT
	1		2	3	7	8		9	4	5	6	10	11		12
Volume Input															
Straight Movement of MBT			25			24				25			33		
Passenger Car Units, V (veh/h)	118	25	899	94	143	439	24	49	83	731	174	183	634		84
Peak-Hour Factor, PHF	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.89		0.89
Adjusted Flow Rate, v_p, (veh/h)	133	29	1011	106	161	494	27	56	94	822	196	206	713		95
Individual Lane Capacity Analysis															
Approach Lane Geometry															
Description of Traffic Movements	Left Turn (All)+ Straight (All)	Straight (MBT)	Straight (ALL)	Right Turn (ALL)	Left Turn (All)+ Straight	Straight (MBT)	Straight (ALL)	Right Turn (ALL)	Left Turn (All)+ Straight (All)	Straight (ALL)	Right (ALL)	Left Turn(ALL)	Straight (ALL)	Straight (ALL)	Right Turn (ALL)
Adjusted flow rate per individual lane, v_p'	572	29	572	106	327.5	27	327.5	56	458	458	196	206	357	357	95
Adj Saturated Flow Rate, s, (veh/hr)	1659	1838	1838	1493	1586	1804	1804	994	1605	1817	1292	1468	1818	1850	700
Green Ratio (g/C)	**0.47**	**0.47**	**0.47**	**0.47**	**0.58**	**0.58**	**0.58**	**0.58**	**0.31**	**0.31**	**0.31**	**0.27**	**0.27**	**0.27**	**0.27**
Lane Capacity, c = s(g/C), (veh/hr)	779	863	863	701	921	1047	1047	577	496	561	399	399	494	503	191
v/c Ratio, X=v_p/c	**0.734**	**0.034**	**0.663**	**0.151**	**0.356**	**0.026**	**0.3128**	**0.097**	**0.923**	**0.816**	**0.491**	**0.516**	**0.723**	**0.710**	**0.497**
LOS by v/c ratio (Othayoth & Rao, 2019)	B	A	B	A	A	A	A	A	C	B	A	A	B	B	A
Lane Group Capacity Analysis															
Lane Group Geometry															
Adjusted Lane Group Saturation Rate (s)	3497	1838		1493	3390	1804		994	3422		1292	1468	3668		700
Lane Group Capacity, c = s(g/C), (veh/hr)	1642	863		701	1968	1047		577	1057		399	399	997		191
Adjusted Lane Group Flow Rate, v_p, (veh/h)	1144	29		106	655	27		56	916		196	206	714		95
Lane Group v/c Ratio, X=v_p/c	0.697	0.034		0.151	0.333	0.026		0.097	0.867		0.491	0.516	0.716		0.497
Lane Group Flow Ratio=v_p/s	0.327	0.016		0.071	0.193	0.015		0.056	0.268		0.152	0.140	0.195		0.136
Critical Flow Rate to Capacity Ratio, Xc = (Yc)(C)/(C – L)	0.626														
Uniform Delay, d_1 (s/veh)	16.96	11.60		12.29	8.84	7.24		7.56	26.43		22.82	24.99	26.68		24.84
Incremental Delay, d_2, (s/veh)	2.47	0.07		0.46	0.46	0.05		0.34	9.55		4.28	4.71	4.40		8.97
Delay, d = d_1(PF) + d_2 + d_3 (s/veh)	**19.43**	**11.67**		**12.74**	**9.30**	**7.29**		**7.90**	**35.97**		**27.09**	**29.71**	**31.08**		**33.81**
LOS by Lane Group	B	B		B	B	A		A	C		C	C	C		C
Delay by Approach, d_A, (s/veh)	18.7				9.12				34.41			31.06			
Intersection Delay, d_I, (s/veh)	22.8														