Highway Efficiency Improvement: Thailand’s Route no. 4 Case Study

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Abstract

The economic growth in Thailand has spread from the metropolitan area to other region of the country. This has led to the development of regional highway to serve as the connection link between the main port city, Bangkok and other part of the country. However, the project did not foresee the population growth situated along the highway. New dwellings on roadside are common throughout the highway. This increase demand for accessibility to the highway results in disruption of the main traffic flow by the local traffic. Thus these communities have contributed to the traffic problem on the highway.

One of the most severe situations occurs on regional highway route no. 4. The link acts as a feeder from the metropolitan area to the southern region. The traffic composition on route 4 comprises of private vehicles such as cars and motorcycles and commercial vehicles such as buses and trucks. This study chosen a critical segment on route 4 where congestion problem has been escalating and potential crisis seems certain. The road section for investigation comprises of 3 lanes highway and 2 lanes frontage road (one way). This section has 2 traffic lights locate less than 2 kilometers apart and 2 hyper markets situated along the roadside. The congestion is mainly the result from the traffic light pile-up, mix traffic (local traffic) and at grade U-turns.

Two design alternatives are proposed; flyover and compact interchange scenario. New design alternatives are simulated in the micro traffic simulation software, S-Paramics. The future demand of the year 2014 and 2019 are simulated in the new scenarios as well as the existing one (do nothing scenario). The highway efficiency improvement is evaluated by comparing the following measure of performances: speed, travel time and queue length with the do nothing scenario.

Both alternatives solve the traffic situation by removing the traffic light from the main road. This result in an improvement in all MOPs considered. The compact scenario proves superior to the flyover scenario. This became more apparent in the simulation for 2019 where the speed efficiency increases by 59%, travel time decreases by 70% and the queue length lessen by 79%.
Preface

I would like to express my deepest gratitude to Professor Haris Koutsopoulos and Ms. Albania Nissan for all your counsel and guidance and for making remote research and presentation possible. I would like to thank all my professors, staffs and fellow students at KTH for all their help and support.

Thank you to Dr. Surasak Taweesilp, Khun Isaradatta Rasmidatta and all the members of TEAM Logistics and Transport Co, Ltd for accepting and guiding me during my time in Thailand. And last but not least, I would like to thank my loving wife, Minty, for your love and support.

I hope that my thesis would be useful and interesting to readers.

Thanarit Duke Charupa

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Chapter I  Introduction

1.1  Introduction

Thailand has seen a fair share of traffic congestion over that past 2 decades. Starting in Bangkok, the increase in the number vehicles with poor traffic management operations are the main causes for traffic congestion. The inadequate public transport induces the need for private vehicles (TEAM Consulting Engineering and Management, 2003). Now, Bangkok has become synonymous with traffic jam. Beginning of the new millennium, the country has seen the traffic problem spreading to other province and big cities and now the interstate highway has traffic congestion on a regular basis.

Act as a regional highway connecting between Bangkok and the southern region, traffic on Route no. 4 has increase in correspond with the country’s economic growth. The traffic travelling on route 4 are comprised of private vehicles such as cars and motorcycles to commercial vehicles such as buses and trucks. This study chosen a critical section on route 4 where congestion problem has been escalating and potential crisis looks certain.

Road section between KM 55 and KM 58 on route 4 has seen an increase in traffic congestion due to economic growth. The route was originally design to be a link between Nakorn Prathum and Ratchabui province and has grown to become the main stopover for the southern route. The road section for investigation comprises of 3 lanes highway and 2 lanes frontage road (one way). This road segment has 2 traffic lights locate less than 2 kilometers apart and 2 hyper markets along the roadside.

The congestion is mainly result of the traffic light and at grade U-turns. There are as many 6 at grade U-turns within the three kilometers section. The at grade U-turns cause the slow traffic to be on the fast lanes when in queue and cut through the fast approaching traffic. The traffic light disrupts traffic flow on the highway causing delays to the main traffic. The study surveys the road segment in 2009 as part of the nationwide highway efficiency improvement project contracted by the Highway Department of Thailand. The alternative proposal is carried out by utilizing the micro traffic simulation software, S-Paramics. The new network alternatives proposed for this study is entitle according to the layout of the new intersection designs. The first is the flyover bridge designs which adopt the elevated road over the junction road over the intersection allowing for a non-disruptive journey for the traffic on the highway. The second design is the compact interchange (partial clover leaf) which adopts loop that shapes similar to that of a clover leaf. Both designs are simulated with
the collected data to test speed, travel time and queue length form. The study also forecast the
future demand using Thailand’s AADT as reference. The year 2014 and 2019 are simulated and all
the results are compared to obtain the most efficient design.

1.2 Project background

The highway department of Thailand has expanded the highly congested regional highway from 2
lanes to a 4 lanes across all sectors to facilitate fast and safe travelling to various regions around the
country. The expansion includes 11 road network covering 4366 kilometers (TEAM Consulting
Engineering and Management et al., 2010). The regional highway that connects difference parts of
the country currently has a high traffic volume transporting both people and goods. Although the
projected has been completed in 1999, the minor roads connected to the main roads, interchange
and U-turn areas have not been upgraded and optimized and thus are operating below standard.
They remain the source of traffic congestion, road accidents and disturbance to the journeys which
reduce the productivity of the nation.

![Figure 1: At grade U-turn on route 4](image)

The network elements and below standard traffic management play a major role in contributing to
congestion. The new highway cannot perform to its full potential when it is counteracting parts are
outdated such as intersection type and U-turns designs and placement. In addition, traffic coming in
and out of a highly populated community along the corridor is the main obstacles to smooth traffic
flow resulting in longer travel time and higher accident rates (due to mix vehicles type with mix
speed). To travel with the convenience, reduce transportation costs and accident rate and enhancing
competitiveness of the country we need to study the ways to increase the efficiency of the highway
traffic.
The expansion of the community is increasing on both sides of the main highways due to the economic development of the country presents a potential traffic problem. The increase number of local residence induces the demand in accessing the main highway. This results in new and unstructured roads accessing to the main highway. These new roads are not part of the initial plan and often are built by a new contractor who is unfamiliar with the original plan and layout. This often solve the traffic situation at hand (local accessibility) but disruptive to the main traffic and has the potential to be the main cause of traffic congestion. Until now, there are no laws specifically against partitioning for a new road and thus new minor roads with direct access to the highway are seen throughout the country (TEAM Consulting Engineering and Management et al., 2010). In addition, the government operations did not include public involvement and thus many complaints and criticism arises after the project in completed. A public discussion is necessary to exchange information and receive feedbacks from the community. This is an essential process in implementing a strategy in order to solve both national and local problems.

![Figure 2: Minor road accessing the highway on route 4](image)

### 1.2.1 Objective and Scope of the Project

The highway efficiency improvement project is subsidized by Thailand Highway Department. The objectives have been adopted from the Highway Department main goals.

- Study the traffic situation on the highways and the limitations of the network
- Study and propose alternatives design and location to improve traffic congestion, traffic flow, accessibility and safety.
1.3 Highway Route No. 4

Highway route 4 is the main path that connects Bangkok and the southern provinces. It extends from intersection of Barom Ratchachonnani road (KM 40 + 000) to intersection Pranburee. The number of lanes varies from 2 to 4 (each direction). The highway passes through many local communities. The locations of these communities dictate the locations of intersections and U-turns. The Highway department has constructed frontage road in the area where the local communities are closely situated. The main function of the frontage road is to remove local traffic from the main traffic.
traveling with high speed on the highway. It connects these small communities and eases the mean of travel. The frontage road, mostly, comprise of 2 lanes, one way road. The U-turn locations are located near the entrance and exit of the local village and towns. U-turns on route 4 are, usually, at graded with some under the bridge U-turns and some U-turn bridges. Often, the various types of U-turns confuse unfamiliar drivers. In many areas, U-turns are closely situated due to the need to service the local residence. However, in some areas, U-turns are located far from each other causing illegal driving such as driving in the wrong direction to the closest U-turn point. The problem with under the bridge U-turns is the low clearance which negates high ceiling-vans and trucks from using the facility. Another issue on route 4 is the numerous signalized intersections. There are total of 14 traffic lights along the route. Some are closely place together within 100 meter interrupting the flow of the main traffic.

This study will focus on a segment of route 4, between the 55 and 58 Km mark. It contains multiple issues and most suited for this study. This highway segment has local development on both sides on the road. There are 2 intersections with traffic lights spreading less than 2 kilometer apart. This is the major interruption to the flow of the traffic on the highway. The traffic lights cause delay and spill back during peak hours. The initial purposes of the 2 traffic lights were to act as access point between the communities on each side. The problem escalates during peak hours. The local commuters lose valuable minutes in waiting for the green light for the minor road.
Another issue is the type and the placement of the U-turns in the selected road segment. The U-turns developed in this network are at grade U-turns which is inappropriate with heavy vehicles and local traffic mix in the traffic composition. The trucks and buses require a wider turn angle with lower speed which is in conflict with the fast moving vehicles in the right lane. Local traffic operates at a low speed and thus it is able to speed up to the highway average speed. In addition the dense residential area along the path sees the new development of hyper market and convenient stores along the path. The frontage road were not design to support high volume traffic and many of the entrance and exit of the frontage are situated closely to the U-turns resulting in a short weaving distance and increase in accidents. Other problems are the street side parking, in front of the department and convenient stores. This adds to the number of accidents cause by vehicles pulling in and out of the left lane.
Chapter II  Literature Review

This chapter covers the information gathered from the relevant researches regarding highway efficiency improvement. It includes books, journals, conference proceedings, research, technical reports, scientific papers, theses and information available on World Wide Web pages.

The common definition of highway is the main road that connects different cities and states and thus is able to support heavy load from truck carrying goods. It comprises of 2 or more lanes with a central separation in the form of land strip or water. Highway is often characterized by the grade separate intersection with limited access. The grade separation promotes the smooth and uninterrupted traffic movement by omitting the use traffic light and stop sign (Bangkok Highway Assessment Report: the follow up report, 2003). The Highway Capacity Manual 2000 has classified 2 categories of highways base on the flow: the uninterrupted and interrupted flow. The network with uninterrupted flow has no external component that could interrupt traffic flow such as traffic light and stop signs. The highway network with interrupted flow contains fixed elements such as traffic light, stop and yield signs. These components have the potential to interrupt traffic flow. The type of the highway network does not dictate the quality or the level of service of the highway (Transportation Research Board (2000).

In Thailand, the term highway may not fall in the exact definition set by the western standard and therefore for the purpose of this thesis the term highway is redefine to prevent misinterpretation. Highway, in Thailand, is the main road with dual carriageway that may be comprise of 2 or more lanes that may have roads connect or pass through. Many design of the intersection have been adopted over the years (mostly from the UK). At grade intersection with traffic light directing conflicting traffic are common throughout Thailand. This is the result from the discontinuous urban planning strategy from one government regime to another (Bangkok Highway Assessment Report: the follow up report, 2003 and Annual report: Eastern region, 2002).

Highway elements include the physical aspects such as road, lane, intersection, u –turns as well as the theoretical aspects such as demand, capacity, passenger car unit, etc.

2.1  Speed flow density relationship

The fundamental concept of traffic flow is the speed, flow and density relationship. The concept took in consideration how each driver has different behavior and reaction and thus a relationship between speed, flow and density is developed to accurately represent the traffic flow
The relationship of speed flow and density is represented by the following equation

\[ q = \frac{v}{k} \]

where \( q \) is flow (veh/hr), \( v \) is speed (km/hr) and \( k \) is density (veh/km).

One of the most common traffic flow diagrams was developed by Greenshields in 1935. It is based on the assumption that speed and density has a linear relationship. From this relationship, speed flow and flow density relationship is established (Transportation Engineering: Online Lab Manual, 2003).

\[ u = \frac{v}{k} \]

where \( u \) is mean speed, \( v \) is free flow speed, \( k \) is density and \( k_{jam} \) is jam density

As seen in the figure 5, when the density is zero, the speed is high on the other hand as the density approaches jam density, the traffic stops and the speed goes to zero (Transportation Engineering: Online Lab Manual, 2003).

The figures above show the relation of speed and flow and flow and density. Both of the relationships produce parabolic curves. The relationships yield essential traffic elements; free flow
speed ($u_f$), maximum flow ($q_{\text{max}}$) and jam density ($k_{\text{max}}$) \cite{TransportationEngineering2003}.

### 2.2 Capacity

The capacity of a network is “the maximum hourly rate at which persons or vehicles can reasonably be expected to transverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions” \cite{HighwayCapacityManual2000}. It is an indicator of traffic volume that a road can potentially service. The evaluation of the capacity is one of the critical criteria in assessing a network performance. The capacity is measured by computing the maximum number of vehicles that a road network can accommodate with sufficient performance in term of speed and safety while maintaining the intended level of service. \cite{MassachusettsDepartmentofTransportation2006}. The maximum capacity is stated as a planning guideline and is rarely reach in actual road facilities.

The most basic unit for capacity is vehicle per hour per lane but other units are widely used depending on the focus of the analysis. When planning transportation mode, person per hour per lane is utilized. It presents the number of persons travelling on each mode type. This is useful when planning the public transport, bus priority and high occupancy vehicle lane. The passenger car unit is informative when calculating the vehicle mix in the network. Person per hour is employed when computing the capacity of each vehicle type. The Highway Capacity Manual 2000 states that the capacity of a two-lane rural highway under ideal conditions is 3200 passenger car unit per hour for both directions combined \cite{HighwayCapacityManual2000}. The passenger car unit will be discussed in more detail in section 2.4.

The capacity analysis evaluates the road network in term of level of service. It is the guidelines in transportation design and planning process. However, this thesis investigates a network which comprises of 2 intersections. The intersection analysis is more complicated than that of a road capacity analysis. The intersection analysis involves not only the elements on the road but the minor roads that form the intersection as well as the vehicle turning movement \cite{ZainalAbidin2007}.

### 2.3 Level of Service

The level of service or LOS is a qualitative measurement of the performance of the road network. It describes different range of performances by the effectiveness of traffic flow or the level of congestion. LOS can be measured by the speed, travel time, number of stops of each vehicles, the level of comfort and convenience perceived by the passenger. These types of assessment are known
collectively as the measure of effectiveness or MOP. There are 6 levels of LOS which are designated by letters from A to F with LOS A being the best performance possible and F being the worst case scenario. However, LOS does not considered safety measurement. The LOS is determined by volume per lane which is which is express by the following formula (Highway Capacity Manual, 2000 and Maerivoet et. al, 2005):

\[ v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p} \]

- Where \( v_p \) is actual volume per lane, \( V \) is hourly volume, \( PHF \) is peak hour factor, \( N \) is number of lanes, \( f_{HV} \) is heavy vehicle factor and \( f_p \) is the driver familiarity (Fundamentals of Transportation/Traffic Flow, 2009).

- Peak hour factor take into consideration of the irregularity in the traffic flow during the peak hour period. It quantifies the variation in the traffic flow. It assists traffic planners in assessing the network performance. Peak hour factor (PHF) is expressed by (Maerivoet et. al, 2005 and Oregon state University et. al, 2003)

\[ PHF = \frac{\text{hourly volume}}{\text{peak flow in an hour}} = \frac{V}{4 \times V_{15}} \]

- where \( V \) is volume (veh/hr) and \( V_{15} \) is volume during the peak 15 min (veh/15min)

The Highway Capacity Manual 2000 computes LOS using delay as one of the parameters. This delay is based on the adjusted flow using the average control delay of the peak 15. The control delay is the time delay result from the traffic signal which affects all vehicles at the intersection. In Microsimulation programs, total delay is calculated and therefore the delay must be converted before the simulation output can be used to compute LOS. Another point of concern is the queue length. The highway capacity manual 2000 defines by the length of the vehicles waiting to be served by the network. This includes vehicles joining the back of the queue or slowing moving vehicles. Microsimulation software cannot measure queue beyond virtual network drawn in the program or exceed the storage capability. The end of the queue that is outside the network is not accounted for (Wikipedia.org: traffic simulation).
2.4 Passenger car unit

Initially, all vehicles in network are considered to have the same attributes when calculating the flow and density where flow is expressed by vehicle per hour and density is expressed by vehicle per kilometer (Baykal-Gursoy et. al, 2009). However, when developing a microscopic traffic simulation, the heterogeneous traffic is utilized. It takes into account the distinctive characteristics each type of vehicles and drivers possess and therefore it is impractical to express the traffic flow by the number of vehicles. Type of vehicles can be categorized by the engine size and the weight of the load it can carry (Highway Capacity Manual, 2000). Mix traffic affects both the number of vehicles in the network and its average speed of the network. Trucks occupy more road space than others and have lower acceleration and speed. Vehicles with inferior capabilities contribute to the overall network performance inefficiency (Maerivoet et. al, 2005).

A generic unit is developed in order to compare flow at different location where each location has a distinctive traffic composition. The passenger car unit is utilized instead of the number of vehicles. Passenger car unit or PCU is defined in the by the Highway Capacity Manual 2000 as “the number of passenger cars displaced in the traffic flow by a truck or a bus, under prevailing roadway and traffic conditions”. PCU captures the difference characteristics of each vehicle types in the heterogeneous traffic by comparing the road space a vehicle occupy to that of a passenger vehicle (Turner et. al, 1993).

Many of the studies on PCU have been developed according to the Western norm. These have not included smaller vehicles which have a unique characteristic that is different from passenger cars. These vehicles are motor cycle and three-wheeler. In South East Asia, motorcycle is a major mode of transport that makes up a very unique traffic composition (Minh et. al, 2003). In Thailand, motorcycle makes up 25% of the traffic composition on the national highway (Traffic accident on National Highway, 2008). Motorcycles have distinctive behaviors that can be described by the lack of queuing priority and lane discipline. They do not follow the first come first serve queuing principle and queue in front of the pack, often on the pedestrian crossings. And because of its size, they are able to operate in small spaces between larger vehicles and maneuver in between lanes (Minh et. al, 2003 and Turner et. al, 1993). At the traffic light intersection, it is common to see groups of motorcycle piling up in front of the first passenger car in queue, in between lanes and sometimes on the pedestrian crossing. This affects the speed and the ease of driving of other modes due to the extra attention given to these small vehicles (Turner et. al, 1993). These driving behaviors are unseen in the West and very little studies have been done to see the effects they have on the traffic flow thus no existing micro simulation program has tackle this issue.
In capturing the motorcycle driving behavior and its effects in microscopic traffic simulation, the representation of the PCU has become the crucial. Previous research reveals that the PCU depends largely on the position of the vehicle relative to the car. The motorcycle PCU in Bangkok is reported to be 0.63 (Nakatsuji et. al, 2001). However the research only take into consideration the passenger car and motorcycle and neglecting other modes of transport especially, pickup trucks and vans which make up 30% of the traffic composition in Thailand (Bangkok Highway Assessment Report: the follow up report, 2003).

Another research which based on the information gather in Bangkok suggested that motorcycle crossing the intersection in the first 5 seconds of the effective green should be given 0 PCU value and those crossing the stop line after 5 seconds has PCU value between 0.53 and 0.65 depending on the lateral position of the motorcycle in relation to its turning movement (May et. al, 1986).

A detailed investigation has been carried out to analyze the lane changing behavior of motorcycle and how it affects the traffic. One research found that the maximum car flow for the network decreases as a result of the motorcycle lane changing behavior (Schadschneider, 2008). The density of the motorcycle has an inverse relationship with the maximum flow. The maximum flow increases and then decreases as the motorcycle flow density increases. The increase of car density results in a reduction of motorcycle lane changing rate. Furthermore, with the increase of motorcycle density, the lane changing rate first raises then decline. The studied found that the lane changing is not advantageous in increasing the flow rate of the network when the motorcycle density is small but increase the traffic flow when the density is sufficiently large (Nakatsuji et. al, 2001).

The most comprehensive study of the passenger car unit for motorcycle in Thailand was carried out by Minh and Sano in 2003. They consider motorcycle as the major mode of transport unlike other research. The research focuses on the interaction between motorcycle and passenger car and the percentage of motorcycle on the road. Others researches cover important issues and characteristics but this study investigate the characteristics of the Thai motorcycle and incorporate it in the analysis. A table of the researches done on PCU in East Asia is presented below. Motorcycle PCU of 0.18 will be adopted for this study.
<table>
<thead>
<tr>
<th>Research/ Organization</th>
<th>Motorcycle</th>
<th>Passenger car</th>
<th>Light van</th>
<th>Medium lorry</th>
<th>Heavy lorry</th>
<th>Bus</th>
<th>Trailer</th>
</tr>
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<tr>
<td>ASEAN highway standard</td>
<td>0.5</td>
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<td>1.75</td>
<td>2.25</td>
<td>2.25</td>
<td>2.25</td>
</tr>
<tr>
<td>Transport research laboratory</td>
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<td>1</td>
<td>1.65</td>
<td>2.23</td>
<td>2.23</td>
<td>2.18</td>
<td></td>
</tr>
<tr>
<td>Webster: PCE and saturation flow (1996)</td>
<td>0.33</td>
<td>1</td>
<td>1.75</td>
<td>2.25</td>
<td>2.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Analysis of motor cycle effects to saturation flow rate at signalized intersection in developing countries</td>
<td>0.18</td>
<td>1</td>
<td>1</td>
<td>2.18</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Indonesian HCM 1996</td>
<td>0.2</td>
<td>1</td>
<td>1.3</td>
<td>1.3</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Malaysian HCM 2006</td>
<td>0.22</td>
<td>1</td>
<td>1.19</td>
<td>2.27</td>
<td>2.08</td>
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<tr>
<td>Determination of Highway Capacity on Uninterrupted Flow: Asian Institute of Technology (1997)</td>
<td>0.25</td>
<td>1</td>
<td>1.75</td>
<td>3</td>
<td>2</td>
<td></td>
<td></td>
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<tr>
<td>Ministry of work, Malaysia</td>
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<td>1</td>
<td>1.75</td>
<td>1.75</td>
<td>2.25</td>
<td>2.25</td>
<td></td>
</tr>
</tbody>
</table>

Table 1: Passenger Car Unit summary

The different PCU in these studies could be explained by the spacing of the motorcycles. The motorcycles in Thailand drive closely to each other and consume every available space to stay in front of the queue. This is the reason the motorcycle PCU is lower in Thailand. This situation is similar to traffic situation in Indonesia where motorcycle is given 0.2 PCU. The study carried out in Thailand also compared the interaction between the motorcycles and passenger vehicles. This includes not only the spaces from the vehicle in front and back, but also the space on the side of the vehicles where there is sufficient space for motorcycles to drive through or park. Hence there are almost 5 motorcycles to 1 passenger vehicle; 3 in the lane and 1 on each side.

2.5 **Microscopic Simulation**

Microsimulation traffic program is a tool for modeling traffic situation in the real world. It represents in a microscopic view of the road, driver, and vehicle where each vehicle is simulated base on the
driver’s behavior. With the improvement in the technology, Microsimulation program has become increasingly popular in many engineering fields. Its application includes testing and validating new traffic models (Chu et al., 2004). The concept of microscopic traffic was developed from the car following model which is base on the driver’s ability to reaction to the vehicle in front by breaking or accelerating (Baykal-Gursoy et al., 2009). The driver decelerates when the front gap falls below safety gap and the driver accelerate to reach the desire speed when there is available front gap (Rakha et al., 2008). Traffic Microsimulation associate each vehicle with a driving behavior, destination and route choice where each vehicle must follow lane changing, car following and gap acceptance principles.

The gap acceptance is the minimum distance that a vehicle can safely change lane. The basic concept is that the driver determines the acceptable distance between the adjacent vehicles in the desire lane. An important parameter in calculating gap acceptance is critical gap. Critical gap is the minimum distance that the vehicle (driver) takes (accepted gap) to accomplish the lane changing maneuver. The rejected gap is the distance that the driver considers as unsafe distance and does not take. The critical gap is more than the rejected gap and less than or equal to the accepted gap (Ahmed et al., 1996).

**Accepted gap < Critical gap > Rejected gap**

![Figure 6: Gap acceptance (Ahmed et al., 1996)](image)

Traffic simulation has many advantages compare to the more traditional aggregated simulation – macroscopic model. (Bertini et al., 2002)

- Represents the changes in demand over time
- More appropriate and accurate than the analytical approaches
- Ability to implementation new scenarios and conditions
- Test the potentially risky and unsafe scenarios
- And study and fine tune the degree of changes of the road network
- Save resources in term of time and money
Currently there are numerous Microsimulation software packages used for research and planning worldwide. Different countries and regions adopt different programs according to its capabilities, availabilities and preferences. Three packages are widely use in Asia and are compare in the table below (Park et. al, 2004).

<table>
<thead>
<tr>
<th>Model Comparison</th>
<th>CORSIM</th>
<th>VISSIM</th>
<th>PARAMICS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Price ($)</td>
<td>500</td>
<td>500 – 15,000</td>
<td>13,310</td>
</tr>
<tr>
<td>User interface</td>
<td>Text editor, graphical user interface (TRAFED), Network can be imported from Synchro and TEAPAC</td>
<td>Text editor, graphical user interface, (main option), TEAPAC can export signal to VISSIM</td>
<td>Text editor, graphical user interface, (main option)</td>
</tr>
<tr>
<td>Network limitation</td>
<td>900 nodes, unlimited links and vehicles, 7000 detectors, 1000 actuated signals, 1000 pedestrian phases, 9999 feet link length</td>
<td>None, except for memory limit on computer</td>
<td>None, except for memory limit on computer</td>
</tr>
<tr>
<td>Traffic control</td>
<td>Yield sign, stop sign, pre-timed actuated signal, ramp metering control roundabout</td>
<td>Priority rules, stop sign, pre-timed signal, actuated signal, roundabout</td>
<td>Priority junction, stop sign, pre-timed signal, actuated signal, roundabout</td>
</tr>
<tr>
<td>Multi-model Transportation</td>
<td>Car, trucks, pedestrian and user friendly modification</td>
<td>Car, trucks, bus, rail, tram, bike, pedestrian and user friendly modification</td>
<td>Car, trucks, bus, pedestrian and user friendly modification</td>
</tr>
<tr>
<td>Traffic assignment</td>
<td>Static traffic assignment with equilibrium and optimization</td>
<td>Static traffic assignment, dynamic traffic assignment</td>
<td>Static traffic assignment, dynamic traffic assignment</td>
</tr>
<tr>
<td>Measure of performance</td>
<td>Traffic volume, delay time, travel time, control delay by turn movement, stopped delay, queue time, queue length, vehicle speed, vehicle fuel consumption, vehicle emission by link</td>
<td>Traffic volume, vehicle speed, mean speed, travel time, total delay, stopped delay, average queue length, maximum queue length, vehicle stops within the queue, bus/ tram wait time, vehicle emission</td>
<td>Point/ link flow, point/ link speed, headway, occupancy, acceleration, density, link/ bus/ total delay, turn/ queue/ link counts</td>
</tr>
<tr>
<td>Graphic output</td>
<td>2D animation</td>
<td>2D &amp; 3D animation</td>
<td>2D &amp; 3D animation</td>
</tr>
<tr>
<td>Multi-Run</td>
<td>Corsim driver interface window, command line, scripts</td>
<td>Multi interface, command line</td>
<td>Command line</td>
</tr>
</tbody>
</table>

Table 2: Microsimulation software comparison

For this thesis, S-Paramics is chosen because of its 3D animation ability. S-Paramics can render the traffic flow, road components as well as the surrounding environment. The detail level of the road components and city design is superior to other Microsimulation package. VISSIM also has the 3D animation but this only extend to the road network.
2.6 S-Paramics

S-Paramics is a microscopic traffic simulator that has the capability to model individual vehicles in a road network for both urban and highway networks. It adopted the car following, gap acceptance and lane changing so each vehicle is designated a driving behavior, vehicle type and route choice. The simulation output includes speed, travel time and pollution emission (Park et. al, 2004). It also has the ability to interface between the driver decision and Intelligent Transportation System in the form of traffic updates (Boxill et. al, 2000). The car following model in S-Paramics follows Fritzche model where the driver reacts when the speed or distance of the vehicle in front drops below a threshold. Below diagram depicts the Fritsch’s model threshold. $\Delta u$ represents the difference in speed (km/hr) between the lead car and the car following and $\Delta x$ is the difference in distance of respective vehicles (meter) (Rakha et. al, 2008).
The diagram illustrates four scenarios in the Fritsch’s car following model (Rakha et. al, 2008):

- **Following**: the driver maintains the current speed when the one of the following conditions is true:
  - Speed is between PTN and PTP (km/hr)
  - Speed different is greater than PTP (km/hr)
  - Space headway is between AR and AD (km/hr)
  When the conditions are false, the driver reacts due to his/her inability to maintain speed and is represented by parameter $b_{null}$. When the speed surpass the threshold of PTN, the driver decelerate at a rate of $-b_{null}$ on the other hand when the speed falls below PTP or AD, the driver accelerate at the rate of $b_{null}$.
- **Free acceleration**: the driver can accelerate at the desire rate when
  - Speed difference is greater than PT and the space headway is greater than AS or
  - Speed difference is less than PTN and the space headway is larger than AD
- **Braking**: the distance from the vehicle in front is less than risky distance AR, the driver applies the maximum deceleration rate, $b$, to increase the safety distance.
- **Closing in**: the vehicle decelerate to adopt the speed of the vehicle in front in order to attain at minimum the space headway of risky distance, AR. The execution occurs when
  - Speed difference is greater than PTN and
  - Space headway is between AB and AR or
There are four essential threshold parameters in Fritzsche model. There are express mathematically below:

- **Desire distance** (meter) – the gap between the vehicle and the preceding vehicle that the driver wants to maintain (comfortable distance)

  \[ AD = A_0 + T_D \times u_n \]

- **Risky distance** (meter) – the distance below the comfort level perceive by the driver (unsafe distance). The driver decelerate when the spacing is equal or less than the risky time gap \((T_r)\)

  \[ AR = A_0 + T_r \times u_{n-1} \]

- **Safe distance** (meter) – is the smallest gap (headway) where the driver can maintain a constant speed without having to decelerate

  \[ AS = A_0 + T_s \times v_n \]

- **Breaking distance** (meter) – the smallest gap that the vehicle can execute the maximum deceleration to avoid collision given that the speed of the two vehicles are different

  \[ AB = AR + \frac{\Delta v^2}{\Delta b_m} \]

Where \(A_0\) is vehicle spacing at jam density and is expressed by

\[ A_0 = \frac{1000}{k_j} \]

where \(k_j\) is jam density,

\(T_D\) is the desire time gap in seconds and is expressed by

\[ T_D = 3600 \left( \frac{1}{q_c} - \frac{1}{k_j u_f} \right) \]

where \(q_c\) is maximum flow and

\(T_r\) is the risky time gap in seconds and is expressed by

\[ T_r = 3600 \left( \frac{1}{q_{c_{\text{max}}} - \frac{1}{k_j u_f}} \right) \]

Where \(u_f\) is free flow

\(T_s\) is the simulated parameter
\( u_{n-1} \) - speed of vehicle in front (n-1)

\( u_n \) - speed of the following vehicle (n)

\( \Delta b_m \) is the speed controlling parameters which is expressed by

\[ \Delta b_m = |b_{min}| + a_{n-1} \]

S-Paramics is applicable for design planning, policy and operational planning. Its ability not only to model congested area at a microscopic level but also the ability to render 3D visual image of real time traffic operation. This is practical when presenting the results to a non specialist and more importantly the decision makers. The simulation package consists of modeler, processor, and analyzer. The modeler is the core simulation tool. Its main purpose is to perform the network building process by using graphic user interface to construct, simulate and visualize the road network (Park et. al, 2004). It has a stochastic behavior which allows it to produce different results for every simulation run by utilizing only one set of data. The mean value can then be computed. This is advantage quality over deterministic models where the same input data generate identical outcomes every time (Velez, 2006).

The processor has a similar function as the modeler that it can run the simulation but with a command line execution which run the model without the visualization steps. This is referred to as Batch Simulation model or Bath Run. It speeds up the simulation time for when multiple replications is necessary and the model calibration has been completed. However it does not allow the opportunity to edit the network or view the vehicle movement during the simulation (S-Paramics Reference Manual, 2007). The analyzer, also known as Data Analysis Tool, is an analysis tool that presents and manages the output data from the in the form of statistical and graphical analysis (Bowill et. al, 2000 and Bertini et. al, 2002).
S-Paramics ability to model the movement of each vehicle type comes from the extensive vehicle type data available whether they are embedded in the program or inserted as additional information. The vehicle type parameters include engine size, width, length, height, and axis length. In addition to the physical elements, vehicle driving abilities and other parameters are also included; speed, acceleration, deceleration, drag, inertia, PCU, trip purpose, and pollution emission rate. The level of details for vehicle type allows S-Paramics to produce a wide range of statistical output. In addition, it simulates how the network geometry influences the vehicle and the output. The road geometry, angle and layout have a direct affect to the vehicle movement and thus speed (Chu et. al, 2004). S-Paramics also take into consideration the effects the new road design or policy has on the pedestrian. It has the capability to model pedestrian and the interaction they have with the vehicle travelling through the road. Bicycle is another additional parameter that can be include in the road network. However, this only limits to a dedicated bicycle lane (Bertini et. al, 2002).

The analyzer in S-Paramics provides many statistical tools in recording, measuring and analyzing the outputs. The results can be categorized into 5 groups (Park et. al, 2004):

1. General network data - This includes turn count, queue count, release vehicle count, vehicle in the network, network mean speed, total distance travelled.
2. By point data (loop detector) - assign a loop detector to a specific location to collect information on speed, flow, density, headway, and vehicle mix
3. By link (loop detector) - collect data at a specified link (road). The data includes link speed, link flow, link density, lane change, link delay and count
4. By path/trip – specify trip or path by joining connected links to form a path/route. The data collected are trip information, travel time, trip count and delay.
5. Other – saturation flow, incident, trace, cost, and car park information

The network building in S-Paramics, like many traffic simulation programs, is based on node-link structure where a link is connected between two nodes and zone for assigning travel demand. The network coding can be accomplished in many methods from using the graphical user interface freehand drawing to overlaying an image from aerial photo in BMP and OS/NTF format or CAD drawing in DFX format (Park et. al, 2004). The geometry of the network is vital for the accurate outcome as the vehicles behave according to the width and curve of the road. For example, acute turning angle or narrow width will cause the vehicle to travel at a lower speed. Each link can have a unique attribute such as design speed, width, and number of lanes, road type and restrictions. Other parameters also influence the change in speed (Bertini et. al, 2002):

- Relative position to the bus stop and curb lines
- Signal timing: actuated time control and fix time control
- Relative position to pedestrian crossings
- Lane control and access restrictions
- Relative position to on-street parking affect

In S-Paramics, traffic demand assignment is presented in an origin-destination matrix that offers multiple demand dispatch periods. It assigns the destination and time period to each vehicle except for fixed route vehicles such as public bus (Park et. al, 2004). The simulator follows 3 route choice principles; all or nothing assignment, stochastic assignment and dynamic assignment (S-Paramics Reference Manual, 2007)

- All or nothing assignment (shortest path) – base on the assumption that when there are no congestion effects, all drivers consider the same attributes for each route choice and they perceived and weigh them the same. The shortest path or shortest travel time is determined and all vehicles are assigned. It is the default trip assignment principle in S-Paramics and which is used by this research (Transportation Engineering: Online Lab Manual, 2003)
- Stochastic – takes into account the variability of the cost by assuming the travel cost for each driver are not identical and no alternative route can reduce the journey cost. The route choices are assigned among the cheapest routes
- Dynamic assignment – assume that familiar drivers will determine his/her shortest path and are able to reroute with an update traffic information. This traffic assignment is implemented in association with ITS. It synchronizes the simulation data with the real time routing decision.
The travel cost in S-Paramics is calculated at each time step by using the generalized function cost:

\[
    Cost = aT + bD + cP
\]

Where:
- \( a \) - time coefficient (min)
- \( T \) – travel time at free flow speed (min)
- \( b \) – distance coefficient (min per km)
- \( D \) – distance (length of the link) (km)
- \( c \) - road tool coefficient (min per monetary unit)
- \( P \) - toll fee (monetary unit)

S-Paramics provides an extensive vehicle type. The database covers different types of passenger cars to different types of trucks. Further modification can also be made such as length, width, height, axle length, weight, drag, inertia, speed, acceleration, or deceleration. This is advantageous where vehicles are of different design. Vehicles in South East Asia do not have the same size and dimension as those in the West, and thus the default characteristics used on other Microsimulation packages are not appropriate.

The visualization of traffic movement is beneficial in reviewing the unorthodox behavior and inefficiency in the network performance. Intersection spillback, illegal and irregular weaving and U-turn problems, or inadequate signal time may not be evident in the numerical output but clearly observable during the simulation run. Visualization is also practical during the calibration process. One can observe if the vehicles stop directly on the stop line or travel in the wrong path (Bertini et al., 2002).
Chapter III Methodology

3.1 Data collection

This section describes in detail the data collection and preparation procedures. The performance of the model and the validity of the results depend highly on the accuracy of the data collection and handling process (Chu et. al, 2004). The data collection process begins with the on-sight examination. The road network inspections focus on the locations with the most severe congestion problem and the area with the most complaints of the local communities. The criteria for selecting sight for investigation are:

- Intersection and U-turn: functionality and suitability of each design
- Side friction from the pavement: from street vendors and street parking
- Entrance, exit and end point of the shoulder lane
- Signage locations
- High accidents rate
- Complaints from residence and authorities

After the sight is selected, in this case the section between KM 53 to KM 58, the data collection process can begin. The main focus of the traffic simulation is to analyze and improve on the current traffic situation. It is vital to simulate the network when the congestion is most severe. The most traffic congestion on route occurs during the morning peak hour from 07:00 – 09:00 AM on the weekdays. The survey records traffic data for every 15 minutes interval. The AADT is supplied by the Department of Highway. The field data will be compared to the AADT and the percentage of change is applied in the demand forecasting.

<table>
<thead>
<tr>
<th>Route number 4</th>
<th>KM</th>
<th>AADT</th>
<th>Heavy vehicles ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Route number 4</td>
<td>65 + 300</td>
<td>75,457</td>
<td>34.8%</td>
</tr>
</tbody>
</table>

Table 3: Thailand AADT on route 4

- Turning movement count

The turning movement count (TMC) is the conventional method in deriving the Origin-Destination Matrix. The TMC is collected by visual observation. Surveyors are employed at the strategic points of the intersection. The surveyors count the number of vehicles, the length of the queue and the vehicle composition. The flow collected is presented in the section 3.2.2: O–D Matrix. The vehicle mix collected from the survey is shown below.
The PCU for the motorcycle of 0.18 is applied to transfer the raw data to the vehicle ratio. The vehicle composition ratio and average maximum queue length are presented below.

The vehicle composition can be directly imported to s-Paramics without further modification.
Data collected by turning movement count are prone to human errors. The lack of understanding and different standard from one surveyor to another can caused inaccuracy in the data. The data is collected on one day which may not be accurate and true to the current situation. Data collected from several days would be more precise and accurate to the real nature of the situation. However, due to low budget, only data collected from one day is gathered.

- **Speed circulation**

The methods for collecting speed survey are speed circulation and free flow speed survey. The speed circulation is done by employing the floating car in the main stream traffic. The traffic simulation duplicates the behavior of the vehicle by using the car following model. The assigned vehicle will travel in the middle lane of the main road and follow the vehicle in front without over taking (spot...
speed). The speedometer is recorded every 30 seconds with the distance traveled and the current kilometer. This process is done on both inbound and outbound traffic. By recording both the KM and the speed, the travel time can also be computed from the speed data. The figures contain speed and distance travelled is presented below.

![Outbound speed circulation](image1)

**Figure 13: Outbound speed circulation weekday AM between KM 40 – KM 70 (km/hr)**

![Inbound speed circulation](image2)

**Figure 14: Inbound speed circulation weekday AM between KM 70 – KM 40 (km/hr)**

As expected, the speed decreases due to the congestion near the intersection for both inbound and outbound. The speed of the inbound traffic (West to East) is lower than outbound traffic (East to West). One explanation is the traffic from people commuting to work in the morning as there s a
small district area in the East of the network. To increase the precision in the simulation, the study calculates the average for average speed for only the KM 55 – KM 58 section. The result is 55.2 km/hr for outbound flow and 52 km/hr for inbound flow.

- **Free flow speed survey**

  The free flow speed will incorporate into the simulation as the design speed of the links. Free flow speed describes the average speed when there is no congestion under fair weather condition. In this study, the free flow is measured by calculating the 85\(^{th}\) percentile from the speed survey.

![Free flow survey](image)

*Figure 15: Free flow speed curve for passenger vehicle (km/hr)*

The 85\(^{th}\) percentile for passenger car on Highway route 4 is 113 km per hour.

- **Flow (u-turn and enter & exit)**

  Another method for measuring flow is by recording a video. This method was employed at the u-turns and entrance of the major street along the highway. The footage is then used to count the number of vehicles entering and exiting the minor road along the highway and u-turning. This method is time consuming but highly accurate with reliable reference.

3.2 **Model development**

This section will describe in detail the network coding and demand preparation process.
3.2.1 Network coding

The foundation of the road network in S-Paramics is called overlay or annotation. They are the blue print for creating a new traffic network. They can be imported from aerial photograph or the Computer Aided Design (CAD). Aerial image can be acquired from a virtual map tool such as Google earth. They are imported as a bitmap image in the form of PNG or JPG format. CAD is the technical drawing of the road network therefore it contains accurate detail of the road geometry. It is imported in DFX format and is the recommended by S-Paramics. DFX can contain multiple layers of graphical and geographical data. It is recommended that DFX file should have 100 x 100 meter grid to ensure that the scaling to compatible with those in S-Paramics (S-Paramics Reference Manual, 2007).

This thesis employs the use of Google earth to capture the aerial image of the studied area. Before importing the bitmap image to S-Paramics, coordinates must be assigned. S-Paramics follow OSGR coordinate system. This serve as a scaling compatibility assurance but it may cause some display abnormally resulting in a low resolution annotation. Coordination of the aerial photograph can be accomplished in CAD. It contains a scaling tool and the image is exported in a DFX format. Once the overlay is in place, the road network construction can be implemented by following the conventional node, link and zone structure (S-Paramics Reference Manual, 2007). The nodes and links are presented below in the S-Paramics network. The nodes are the yellow dot and the links are the lines connecting each dot. The zones configuration is presented in the below diagram.
### 3.2.2 Origin-Destination matrix

Traffic demands in S-Paramics are assigned in an origin-destination matrix (O-D matrix). The demand editor in S-Paramics support OD matrix, profile & assignment, vehicle proportion, and path routing. Demand profile & assignment function is to model different time period, each with different traffic distribution (*S-Paramics Reference Manual, 2007*). This is convenient when modeling different season or weekdays and weekends. This thesis only focus on the workday peak hours and the morning peak profile is selected and presented below. Vehicle proportion editor allows S-Paramics to capture the vehicle type mix and incorporate into vehicle distribution and allocation of the simulator. Path routing increases the detail of traffic distribution by assigning each path with a release profile, demand rate and route choice (*S-Paramics Reference Manual, 2007*). Due to the size of the study area, a flat profile is adequate for simulation (*Bertini et. al, 2002*).
In this study, the turning count was collected as the measure of flow. The flow count was position at 2 intersections, 4 U-turns and 2 main entrance and exit points from the minor road. As the nature of field survey, the traffic flows on each location are not consistent and the values do not correspond to each other. A conversion from turning movement to OD matrix is required (Bertini et. al, 2002).

The conversion can be completed in 4 steps procedure in transforming the turning movement count into a well balanced OD matrix.

1. Balance traffic flow between all links
2. Create Origin-Destination matrix (unbalanced)
3. Balance overall production/ attraction zones
4. Iterative Proportional fitting (Furness procedure)

Since, the TMC collected at different locations are from the same time period and therefore, some vehicles were present inside the network between the two main intersections and some remain in

<table>
<thead>
<tr>
<th>Time interval</th>
<th>Vehicle distribution (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7:00-7:15</td>
<td>9%</td>
</tr>
<tr>
<td>7:15-7:30</td>
<td>12%</td>
</tr>
<tr>
<td>7:30-7:45</td>
<td>13%</td>
</tr>
<tr>
<td>7:45-8:00</td>
<td>14%</td>
</tr>
<tr>
<td>8:00-8:15</td>
<td>13%</td>
</tr>
<tr>
<td>8:15-8:30</td>
<td>16%</td>
</tr>
<tr>
<td>8:30-8:45</td>
<td>13%</td>
</tr>
<tr>
<td>8:45-9:00</td>
<td>10%</td>
</tr>
</tbody>
</table>

Table 5: Vehicle distribution ratio

Figure 19: Vehicle distribution
the network and did not pass through the second intersection at the end of the survey period. The first step is to balance the raw TMC data. The TMC data at the 2 intersections are done manually and thus most likely to contain human errors. The balancing of traffic volume is carried out over the 2 intersections. The table below associates the terms and the directions to prevent any misinterpretation. Intersection 55 refers to the East and intersection 57 is on the West.

<table>
<thead>
<tr>
<th>Origin</th>
<th>Destination</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Outbound</strong></td>
<td>Bangkok (East)</td>
</tr>
<tr>
<td><strong>Inbound</strong></td>
<td>Ratchaburi (West)</td>
</tr>
</tbody>
</table>

*Table 6: Direction clarification*
Figure 20: Unbalance flow distribution – raw data (veh/hr)
An assumption is made that the flow recorded from video camera, in and out of major street and U-turns, are correct and will remain constant for the balancing process. The study focuses on a highway segment that contains frontage road, 2 lanes on each direction. From the aerial photograph and the visual observation, the flow on these local roads is 9% of the flow on the highway.

For outbound traffic towards Ratchaburi, the traffic volume from the origin (Bangkok) is more than the destination zone (Ratchaburi) and thus the vehicles will be added to the destination flow at intersection 57 to balance the matrix. The different in flow between the outbound flow from intersection 57 and the flow entering intersection 55 is 16 vehicles. The difference of 16 vehicles is distributed based on the proportion of the inbound traffic flow from intersection 57.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Unbalance</th>
<th>Balance</th>
</tr>
</thead>
<tbody>
<tr>
<td>North-&gt;East</td>
<td>148</td>
<td>148</td>
</tr>
<tr>
<td>West through</td>
<td>2071</td>
<td>2078</td>
</tr>
<tr>
<td>South-&gt;East</td>
<td>187</td>
<td>188</td>
</tr>
</tbody>
</table>

Table 7: Balance outbound flow to intersection 57 (veh/hr)

For inbound traffic, the origin, intersection 57 produces less flow than the destination intersection and so the vehicles will be added to the origin intersection. The difference in flow for the inbound traffic is 8 vehicles. The same method is repeated to balance the flow.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Unbalance</th>
<th>Balance</th>
</tr>
</thead>
<tbody>
<tr>
<td>North-&gt;East</td>
<td>148</td>
<td>148</td>
</tr>
<tr>
<td>West through</td>
<td>2071</td>
<td>2078</td>
</tr>
<tr>
<td>South-&gt;East</td>
<td>187</td>
<td>188</td>
</tr>
</tbody>
</table>

Table 8: Balance inbound flow from intersection 57 (veh/hr)

The diagram below illustrates the balance flow.
Figure 21: Balanced flow distribution (veh/hr)
The balanced flow can now be put in an O-D Matrix by distributing the flow proportionally according to the total flow movement at that location. The sum of the total flow going into the destination zones must be equal to the sum of the total flow from the origin zones. Similar to the previous step, an adjustment is made by distributing the different of the total flow to each destination zone according to its contribution to the total flow.

Once the total flow for both origin and destination are balance, the Furness procedure is applied to the adjusted flow from each O-D pair. The Furness procedure is an iterative proportional fitting process. It adjusts each O-D pair by balancing with the total zone flow. For this study, the maximum is error is 3 vehicles. The number of iterations required was 15 (Miller et. al, 2000).

\[ T_{ij}^{new} = \frac{T_{ij}^{old}O_i}{\sum_j T_{ij}^{old}} \]

\[ T_{ij}^{new} = \frac{T_{ij}^{old}D_j}{\sum_i T_{ij}^{old}} \]

where \( T_{ij}^{old} \) is the old flow matrix

\( T_{ij}^{new} \) is the new travel matrix

\( O_i \) is the origin flow

\( D_j \) is the destination flow

The flow of the frontage can be completed by assigning 9% of the flow on the main road to its respective frontage road. The completed O-D matrix is shown below:

<table>
<thead>
<tr>
<th>Zone</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>1560</td>
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<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td>2128</td>
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<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td>2066</td>
</tr>
<tr>
<td>4</td>
<td>1191</td>
<td>132</td>
<td></td>
<td>232</td>
<td>198</td>
<td>195</td>
<td>26</td>
<td>16</td>
<td>76</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>236</td>
</tr>
<tr>
<td>5</td>
<td>173</td>
<td></td>
<td>19</td>
<td>7</td>
<td>19</td>
<td>9</td>
<td>2</td>
<td>2</td>
<td>5</td>
<td></td>
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<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td>557</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>519</td>
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<td>500</td>
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<tr>
<td>9</td>
<td>146</td>
<td>129</td>
<td>16</td>
<td>14</td>
<td>237</td>
<td>7</td>
<td>1</td>
<td>3</td>
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<td>10</td>
<td>55</td>
<td>159</td>
<td>6</td>
<td>18</td>
<td>262</td>
<td>9</td>
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<td>4</td>
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<td>15</td>
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<tr>
<td>11</td>
<td>124</td>
<td>151</td>
<td>14</td>
<td>17</td>
<td>27</td>
<td>23</td>
<td>132</td>
<td>4</td>
<td>8</td>
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<td></td>
<td></td>
<td>406</td>
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<tr>
<td>12</td>
<td>74</td>
<td>101</td>
<td>8</td>
<td>11</td>
<td>16</td>
<td>14</td>
<td>172</td>
<td>2</td>
<td>6</td>
<td></td>
<td></td>
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<td>404</td>
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<td>13</td>
<td>12</td>
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<td>15</td>
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<tr>
<td>14</td>
<td>256</td>
<td>128</td>
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<td></td>
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<td>22</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>406</td>
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<tr>
<td>Total</td>
<td>1590</td>
<td>2541</td>
<td>176</td>
<td>381</td>
<td>637</td>
<td>684</td>
<td>517</td>
<td>181</td>
<td>49</td>
<td>158</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6914</td>
</tr>
</tbody>
</table>

Table 9: Balanced O-D Matrix (veh/hr)
• **Traffic assignment technique selection**

From the data collected, a complete O-D matrix can be developed. Each O-D pair offer one route choice and therefore the all or nothing traffic assignment is used (default technique).

• **Random seeds**

Random seed in S-Paramics is used to determine the release time, the release link, the randomization in the route perturbation and random assignment of the attributes of each individual vehicle. Each simulation runs with identical set ups but with different random seeds will produce slightly different results. The random seed will associated each vehicle with predefined parameters. This ensures that no two runs of models will be identical. The seed value of zero, by default, will generate a random seed value according to the PC clock time. The actual value can be retrieve from the log file of each run (*S-Paramics Reference Manual, 2007*).

3.3 **Model calibration**

Applying the adjusted demand in the network, the calibration of the model simulated. The main objective of calibration is to adjust the parameters of the model so that it is able to duplicate the reality accurately. The calibration process is divided into 2 steps:

• **Visual adjustment**

The initial step in calibrating a model in S-Paramics is to examine the traffic movement visually. The graphic animation replicates the movement of the vehicles and by viewing the traffic one can observe the unrealistic behavior. These observations can be compared to the traffic survey video or the general behavior of the driver in the area. S-Paramics allows the adjustment to be made during the simulation. Adjustment such as position of the curve, stop line, and lane priorities help control the irregular movements.
After the visual examination is a mathematical evaluation is applied to measure the goodness of fit. One method in measuring the goodness of fit is the root mean square relative error or RMSE. RMSE measures the deviation of the simulated variables from the observed values. This study will employ the root mean square percentage error (RMSPE) for comparison because its scale is the same for the different set of data and the. The RMSPE is calculated base on the following formula (Vaze, 2007):

$$RMSPE = \sqrt{\frac{1}{N} \sum_{n=1}^{N} \left( \frac{Y_n^S - Y_n^O}{Y_n^O} \right)^2}$$

where $Y_n^S$ and $Y_n^O$ are the simulated and the observed values, respectively.

The minimum threshold for the discrepancy is the 10%. The traffic variables considered for the calibration and validation is the flow, speed and queue length. Delay is often used in comparison of the accuracy of the traffic simulation. However, the study did not have the equipment to collect the delay of each vehicle and thus travel time is used an alternative.

This study also incorporates two other goodness of fits. GEH will be used in assisting in the flow evaluation and percentage error is included in the speed evaluation along with RMSPE. GEH is an empirical evaluation rather than a true statistical test. When comparing the baseline scenario for the traffic hourly volume, a GEH of less than 5.0 is acceptable. According to DMRB, 85% of the volumes in a traffic model should have a GEH less than 5.0. GEHs and in the range of 5.0 to 10.0 may warrant an investigation (Chu, et. al, 2004). The GEH is presented as
\[
GEH = \sqrt{\frac{2(S - O)^2}{S + O}}
\]

where S is the simulated value and O is the observed value.

The relative error (percentage error) is applied to assist in evaluating the MOPs. The relative error is defined by the absolute error divided by the observed value. The absolute value is the discrepancy between the observed value and the simulated value. The relative error will be expressed as the percentage error in the comparison. The percentage error is expressed by (Weisstein, 2002):

\[
\delta = \left| \frac{x_{obs} - x_{sim}}{x_{obs}} \right| \times 100
\]

The threshold for the percentage error is 10% error. After calibrating criteria is satisfied, the model can be validated.

### 3.4 Model validation

Validation is the process in determining whether the model inherent the characteristic of the real situation. Model validation is considered as the final step in model construction before the results can be analyzed and interpreted. Ideally, model validation is carried out with a different set of data. The data may be collected at the same location on a different day. The validation function is to test the liability of the calibrated model and so the model is not over-fitted for just one scenario (data) (Chu et. al, 2004). The same evaluation procedure and MOPs as the calibration process is carried out the validation process.

This study has acquired the data from another day using the same period which will be applied to the model validation process. The same RMSPE method is employed on the identical variables.

### 3.5 Evaluation

Once the model is verified, the alternatives are coded into the network. Often these alternatives propose new designs or new traffic management policy and strategy. The evaluation of the alternatives is done via evaluating the measure of performance in each case. The network performances that will be assessed are the average speed, total travel time, and queue length. The network performances when compare with the base scenario will represent the effectiveness of changes. The scenario with the most efficient results will be proposed to the Department of highway. The process in implementing a traffic simulation is summarized in the flow chart below.
Figure 23: Traffic simulation flow chart
Chapter IV Model

The following chapter describes the current situation and the alternative design proposed. The study site offers a unique problem comprising of the road geometry, growing traffic demand, and the road components. The current network situation and two proposed alternatives are described. The study will forecast the traffic demand according Thailand’s Average Annual Daily Traffic on Highway reports. The 3 models are base scenario, compact scenario and flyover scenario.

4.1 Existing problem

The field survey and local communities report (complaints) on the existing problems on route 4 are summarized and categorized in the table below.

<table>
<thead>
<tr>
<th>Problem</th>
<th>Description</th>
<th>Solution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>The minor road and the U-turns are closely situated resulting in short weaving distance. The layout resembles a small intersection where the entrance and exit of the minor and the at-grade U-turns forms the intersection. Another problem is illegal driving in the wrong direction especially from motorcycles to the closer U-turn. This compound to the number accidents</td>
<td>- U-turns relocation</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- new U-turns design</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- traffic law enforcement</td>
</tr>
<tr>
<td></td>
<td>Densely populated residence area along the highway resulting in high number of slow moving vehicles entering and exiting these main road This greatly reduces the speed of traffic on the highway.</td>
<td>- reduce main traffic interruption</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- improve connection between communities.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- flyover bridge</td>
</tr>
<tr>
<td>Problem Description</td>
<td>Solution</td>
<td></td>
</tr>
<tr>
<td>---------------------</td>
<td>----------</td>
<td></td>
</tr>
<tr>
<td>3 types of U-turns:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- at grade</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- graded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- under the bridge: many cases have low clearance distance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Many types of U-turns confuse unfamiliar drivers</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- remove unnecessary U-turns</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- develop a common U-turn design throughout the route: graded U-turn bridge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Entrance and exit of the local residence area are directly at the U-turns causing multiple lane changing in short distance and driving in the wrong direction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- U-turn location</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- connection between communities</td>
<td></td>
<td></td>
</tr>
<tr>
<td>The intersection on the highway reduces speed of traffic on the highway</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Replace the traffic light on the highway with the flyover bridge</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 10: Description of existing problems and solutions

The highway segment between 55 and 58 Km contains nearly all the type of problems present on Route 4. Two models were designed to solve the problems on route 4: flyover and compact interchange.

This report will investigate four scenarios:

- The base scenario: the current traffic situation and travelling pattern today
The ‘Do Nothing’ scenario: the current traffic network with the future traffic demand according to the growth pattern provided by the Thailand Highway Department

The ‘Flyover’ scenario: current and future traffic demand with the implementation of flyover bridge

The ‘Compact’ scenario: current and future traffic demand with the implementation of compact interchange (partial clover leaf design)

4.2 Base Scenario

The base scenario represents the current situation of the network. The model attempts to replicate the today traffic demand with the current geometry and problems as accurate as possible.
4.2.1 Signal plan

The signal plan for both intersections are collected via video and are presents the table below. Both intersections have the same stages and signal group.

<table>
<thead>
<tr>
<th>Node</th>
<th>Stage</th>
<th>55</th>
<th>57</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>![Diagram 1]</td>
<td>![Diagram 2]</td>
</tr>
<tr>
<td>2</td>
<td>![Diagram 3]</td>
<td>![Diagram 4]</td>
<td></td>
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<tr>
<td>3</td>
<td>![Diagram 5]</td>
<td>![Diagram 6]</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>![Diagram 7]</td>
<td>![Diagram 8]</td>
<td></td>
</tr>
</tbody>
</table>

The traffic signals timing are the replica of the video footage of the traffic light at both intersections. This signal timing is only applicable for base scenario 2009. The modification is made on the green time and thus the other values are set as default. The time unit is in seconds.
The two signal plans are not coordinated. The cycle time for the node 57 is higher than node 55, with 206 seconds and 139 seconds, respectively. The high different is due to the amount of traffic volume of between the two intersections. In Thailand, the traffic police are in charge of the day to day traffic management operation including traffic light management. It is common that a police officer will manually operate the traffic light during peak hour.

4.3 Do Nothing Scenario

In the Do nothing scenario, the report forecasts the future demand and apply it to the current traffic network without any modification or alterations to the road geometry. The notion is to reflect the future traffic situation if no solution is taken to solve the potential problems. The traffic demand forecast derives from the Thailand’s Average Annual Daily Traffic on Highways report from 2004 – 2008. The Average increase in traffic volume for the 5 years period is 7.19%. The do nothing scenario will be the reference scenario for which the results from the new alternatives are compared.

<table>
<thead>
<tr>
<th>Year</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVG</td>
<td>25496</td>
<td>27206</td>
<td>29408</td>
<td>29813</td>
<td>33564</td>
</tr>
<tr>
<td></td>
<td>6.71%</td>
<td>8.09%</td>
<td>1.38%</td>
<td>12.58%</td>
<td></td>
</tr>
</tbody>
</table>

Table 12: Thailand’s AADT on Southern route (Highway Department)

The high number of increase may be due to the growth of economy in the East and Southern part of the country. Route 4 plays a major role in connecting these regions to Bangkok and greater Bangkok where ports and airports are located.
4.4 Flyover Scenario

This first alternative proposed is the implementation of the flyover bridge for the highway. This allows the main traffic on the main road to travel over the congested intersection and thus avoid delay and traffic light. The flyover scenario directly tackles the delay and traffic problem caused by traffic light on the highway. The ground traffic operates in the same manner with traffic light.

The flyover bridge has a similar design as the following analytical designs (These designs are the prototype approved by the Department of Highway commission for this project). Although the legal height of any vehicles in Thailand is 4.0 meters, there are numerous trucks and buses with illegal modifications and sometimes overload the products so the height of these vehicles exceed the limit. The vertical clearance adopted for this project is 4.5 meters with the maximum of 5 meters (Transport and tourism issues: Improving road safety on the Asian highway, 2007 and Traffic Accidents on National Highways, 2008 and Dehnert et. al, 2003). However, here are some flaws and limitation to these designs. The area under the bridge may create blind spots that need to be lighted and clean. These can become hazardous areas for motorcycles and pedestrian especially at nighttime. Also, there might be opposition from business owners of shops near the intersection. The flyover would place their establishments under the bridge and it is likely to reduce the number of customers. And lastly, building a flyover for 3 lane highway is costly.
Figure 28: Flyover Scenario network in S-Paramics

Figure 29: Flyover interchange pneumatic design - Top view (The interstate highway study between Nakorn Prathom and Cha Um: const benefit, technical design and economical, 2007)
4.5 Compact Interchange Scenario (Partial clover leaf)

The compact interchange designs are common in the UK where many of Thailand’s designs replicate its traffic layout. Similarly to the flyover scenario, the compact design has an elevated road separating main and local traffic. The design suggests building and bridge connecting the two communities while the main traffic operates at ground level. It also considers the drawback of the signalize intersection and adopt a priority system. This reduces the cost of maintaining a traffic light and delay from red traffic light.
The compact design is advantageous in the 4 areas. Firstly, it reduces the severity and the number of accidents compared to signalized intersections. According to the Department of Highway’s Traffic Accidents on National Highways 2008, severe accidents in the suburban area occur at the signalized intersection. Most cases happened at the off peak hour when the traffic is low. Vehicles/drivers are impatient and attempt to pass at the red light when the opposing roadway seems clear. This results in a high speed collision with traffic from the green light direction. Secondly, the compact design provides roadway for vehicles to reduce speed in a turn right and U-turn maneuvers. Thirdly, the cost of construction is lower than other full intersection design such as diamond interchange. Also, the cost of constructing an overpass between 2 local communities is lower than a flyover bridge on the highway. The overpass only supports 2 lanes (one lane for each direction) traffic from 2 local communities compared to 6 lanes of the highway. The main traffic can travel at ground level thus eliminate the need to climb the ramp. This is beneficial for area with heavy vehicle because it does not have to slow down because of the elevation.

Figure 32: Compact Scenario network in S-Paramics
The compact interchange has two main drawbacks that need to be considered. The first is the traffic volume. The design is best suited for low to medium traffic volume which currently matches with current traffic situation. The study is unable to predict the growth of the residency in the area as a result from the new design. The second issue is the low design speed of the design. This can be solved by constructing appropriate acceleration and deceleration lanes. These can provide sufficient space for vehicle to slow down or accelerate to an adequate speed prior to merging with the main traffic.

Other designs for improvements are diamond interchange and dumbbell interchange. Diamond interchange is a very common at grade intersection where the minor and major roads are grade separated. An adaptation of diamond interchange is the dumbbell interchange. The name comes from the shape of the roundabout place at the ramp intersection at both ends. This design can help reduce cost due to the roundabout capacity to handle few lanes of approach traffic which prohibit the construction of a wide ramp bridge. (Wikipedia: diamond interchange)
Figure 34: Dumbbell interchange pneumatic design (The interstate highway study between Nakorn Prathom and Cha Um: const benefit, technical design and economical, 2007)

Figure 35: Diamond interchange pneumatic design (The interstate highway study between Nakorn Prathom and Cha Um: const benefit, technical design and economical, 2007)
4.6 U-Turn Bridge

In addition to the alteration to the intersection geometry, U-turns design and locations are also modified. The design concept is to create a uniformity to familiarize the road users to the design. The locations consider the access point of the communities along the highway. There must be sufficient weaving space for these vehicles to maneuver on to the U-turn ramp. The location of the U-turns is placed roughly 1 kilometer apart. This is a standard distance set by the Department of Highways. The U-turn bridge design also deviate decelerating traffic from the right lane which helps reduce the rear end accidents (Accidents on National Highway report, 2008).

![U-turn bridge pneumatic design - Top view](The interstate highway study between Nakorn Prathom and Cha Um: const benefit, technical design and economical, 2007)

4.7 Design Specification

S-Paramics provide default values for the design specification such as width and turn radius. The majority of these values are consistent with the Thai design specification. The parameters that are different to the default values are defined below.

- **Ramp & Design Speed**

  The ramp is an essential component of the graded interchange. It can be classified into 4 types:

  - Diagonal ramp for left turn
  - Loop ramp for right turn movement which circumnavigate the shape of the loop
  - Semi-directional ramp is a semi direct lane for the traffic turning right and for the cars wanting to turn right must divert to the left lane to enter the turning lane
  - Directional ramp a direct lane for turn right traffic where vehicles must be in the right lane
The design ramp speed should have the speed close to the design speed of the road that connects to the ramp especially for the direct and semi direct right turn lane. For the loop ramp, design speed is lower than the main road depending on the radius and the safety issue. It must provide appropriate speed change lane.

- **Design Speed Loop Ramp**

The design speed of the loop ramp depends on the steepness of the ramp and sight distance to the coming traffic. The average design speed is 60 kilometer per hour however, this result in expropriating larger piece of land. The reduction of design speed decrease the size of the land expropriation but at the same time lessens the capacity of the ramp and may need to increase the number of lanes. The acceleration and deceleration lane are required to increase safety of this alteration.

- Design speed 30 km/hr ➔ loop radius 30 meter
- Design speed 40 km/hr ➔ loop radius 55 meter
• **Design Speed Directional Ramp**

The design speed of directional ramp follows the same principles as the loop ramp design but with a direct line of sight which allows the sight distance to dictate the design speed. Generally, directional speed is higher than the loop ramp which is set at 90 kilometer per hour. The minimum speed is 70 kilometer per hour. The main criterion for adjusting the speed are safety, conveniency, and cost saving.

• **Radius Distance - Loop Ramp**

The radius distance is the most important factor. It has a high correlation with the design speed and generally has the distance of 130 meter with the lowest boundary of 50 meter (Engineering Consultant Guidelines, 2000).

• **Radius Distance - Directional Ramp**

The radius distance also has the direct relationship with the design speed. The universal distance is 355 meter with the lowest distance is 155 meter (Engineering Consultant Guidelines, 2000).

• **Maximum elevation**

The curvature of the ramp generally elevates 46% and with the maximum of 5%. Thailand adopted the 5% elevation by evaluating the rain fall and the side friction affects (Engineering Consultant Guidelines, 2000).

• **Down Grade Ramp Elevation – Loop & Directional Ramp**

The elevation of the ramp should be as minimal as possible to ease the convenient of the traffic flow during the lane changing and merging. For the down grade directional ramp, AASHTO suggests slope to be -4% to – 6% for the design speed between 60 to 80 km/hr. The 4% decline slope is considered as the desirable elevation while the 6% is the absolute minimum. For the loop ramp with the design speed between 40 and 60 km/hr, AASHTO recommends the down grade at 5% with the maximum of 7% (Engineering Consultant Guidelines, 2000 and Dehnert et. al, 2003).

• **Up Grade Ramp Elevation – Loop & Directional Ramp**

In the same way, AASHTO recommends an Up Grade Ramp from 4% to 5% for the design speed of 60 to 80 km/hr and for 4% to 6% with a Design Speed of 40 to 60 km/hr. Therefore the maximum up
grade directional ramp for general use is 4% and at the lowest is 5% and for the loop ramp is also 4% with the lowest of 6% (Dehnert et al., 2003)

- **Minimum Stop Sight Distance - Loop Ramp**

With what was explained before about the Design Speed and the lowest curve radius of the loop ramp is controlled by the stopping sight distance. So when the design speed is set to 60 km/hr the stopping sight distance would be 90 meters and if the design speed was set to 40 km/hr the stopping sight distance would be 50 meters (Engineering Consultant Guidelines, 2000).

- **Minimum Stop Sight Distance - Directional Ramp**

Similarly, the Directional Ramp follows the same concept as the loop ramp. With the design speed of 90 km/hr, the stopping sight distance would be 170 meters and with the lowest design speed set at 70 km/hr, the stopping sight distance will be at 110 meters (Engineering Consultant Guidelines, 2000).

- **Other roadway specification (AASHTO, 2005)**
  a) Entrance – entrance and exit - exit must be at least 240 meter apart.
  b) Exit – entrance must be at least 120 meter apart.
  c) Entrance – exit must be 300 meter apart.
  d) Minor street must be at least 50 meter from the intersection.
  e) Minor street must be at least 50 meter from the ramp of the bridge.

### 4.8 Calibration

Applying the prepared demand with the base scenario network, the model is now ready for calibration. The traffic model is adjusted to replicate the actual traffic movement. After the visual adjustment, the mathematical comparison can take place. From the ‘trip all’, ‘path starts’ and ‘trip queue summaries’ in the S-Paramics, the speed, flow and queue length can be derived. These results are compared with the observed data in order to calibrate the model. The first examination is done by comparing the turn counts or flow of the simulated with the observed data. The comparison for flow is done by using RMSPE with 90% accuracy threshold. The flow examination was conducted on the main road over the 2 main intersections and the U-turns. Each movement is calculated by collecting the count/flow passing the reference nodes. The data was collected over the morning peak hours between 7:00 – 9:00 AM with the 15 minute interval and therefore the RMSPE
comparison treat each time interval as an observation. The GEH calculation is also presented in the table below. The limit for GEH value is 5.

<table>
<thead>
<tr>
<th>Time interval</th>
<th>Simulated</th>
<th>Observed</th>
<th>GEH</th>
</tr>
</thead>
<tbody>
<tr>
<td>7:00-7:15</td>
<td>1135</td>
<td>1275</td>
<td>4.05</td>
</tr>
<tr>
<td>7:15-7:30</td>
<td>1605</td>
<td>1705</td>
<td>2.46</td>
</tr>
<tr>
<td>7:30-7:45</td>
<td>1752</td>
<td>1926</td>
<td>4.06</td>
</tr>
<tr>
<td>7:45-8:00</td>
<td>2175</td>
<td>1979</td>
<td>4.31</td>
</tr>
<tr>
<td>8:00-8:15</td>
<td>1852</td>
<td>1967</td>
<td>2.63</td>
</tr>
<tr>
<td>8:15-8:30</td>
<td>2022</td>
<td>2208</td>
<td>4.04</td>
</tr>
<tr>
<td>8:30-8:45</td>
<td>1829</td>
<td>1779</td>
<td>1.17</td>
</tr>
<tr>
<td>8:45-9:00</td>
<td>1358</td>
<td>1208</td>
<td>4.19</td>
</tr>
</tbody>
</table>

The unit is the number of vehicles per time interval. The average flow between simulation periods 7:00 – 9:00 AM is collected and compared. The RMSPE is 4.38% which is less than the maximum RMSPE value of 10%. The GEH evaluation confirms the model validation. The highest GEH value occurs during 7:45 – 8:00 AM with the value of 4.31. Nonetheless, this is below the threshold of 5. Both RMSPE and GEH verify the flow simulation and the model is ready to for the validation process.
A linear graph is plotted to assist in analyzing the result. The linear graph is shown below. The ideal output is to have the speed distribution along the 45 degree line. As observed, the plotted flow (blue dots) resembles the 45 degree line. This concludes that the model can accurately simulate flow distribution.

![Flow Comparison - Calibration](image)

Figure 40: Flow comparison – calibration (number of vehicles per 15 min interval)

Another indicator in the model ability to captures the current situation is the speed parameter. The average speed was collected for the main road in both directions and thus the comparison will be carried out for these directions. The replication of the start and end points is done on Google earth and then duplicate on the overlay of the network. The methods of comparison for speed and queue length are RMSPE and percentage error.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Observed</th>
<th>Simulated</th>
<th>RMS</th>
<th>Percentage error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outbound</td>
<td>55</td>
<td>50</td>
<td>6.43%</td>
<td>9.09%</td>
</tr>
<tr>
<td>Inbound</td>
<td>52</td>
<td>55</td>
<td>4.08%</td>
<td>5.77%</td>
</tr>
</tbody>
</table>

Table 15: Speed comparison – calibration (km/hr)

The accuracy of speed required for this research is 10% for both the RMSPE and the percentage error. The result from the simulation is shown in the table above. The result is collected from 7:00 – 9:00 AM with 15 minutes interval. The first speed evaluation is the RMSPE. The value for the RMSPE is 6.43% for outbound traffic and 4.08% for inbound traffic. The result justifies the criteria for the study. The speed evaluation also utilizes the percentage error. The calculation yields 9.09% and 5.77% for outbound and inbound traffic respectively. The percentage is less than the maximum value require for the validation process. Thus both methods of goodness of fit satisfy the criterion for this research.
The last MOP selected for the calibration process is queue length. Similar to the average speed, the queue length data available is for the major road and the method of comparison is RMSPE and percentage error. The maximum queue for every 15 minutes interval between 7:00 and 9:00 AM was collected (estimated).

<table>
<thead>
<tr>
<th>Direction</th>
<th>Observed</th>
<th>Simulated</th>
<th>RMSPE</th>
<th>Percentage error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intersection 55</td>
<td>122</td>
<td>133</td>
<td>3.72%</td>
<td>9.09%</td>
</tr>
<tr>
<td>Intersection 57</td>
<td>177</td>
<td>195</td>
<td>3.30%</td>
<td>10.32%</td>
</tr>
</tbody>
</table>

Table 16: Maximum queue length comparison - calibration (meter)

The threshold for maximum queue length comparison is also 10% for both RMSPE and percentage error. The data collected was from visual inspection and the length is estimate by using reference point. Both intersections satisfy RMSPE evaluation. The RMSPE value for intersection 55 is 3.72% and for intersection 57 is 3.30%. The other goodness of fit is the percentage of error. The result for percentage error is relatively high. The highest percentage error is 10.32% occur at intersection 57. The intersection 55 yields 9.09%. The high value of percentage errors may result from the human error during the data collection process. The end of the queue is observed visually and marked by reference to the on sight landmarks. The visual estimation may not be consistent from one collector to another and thus the 10.32% error at intersection 57 is accepted.

All three MOPs have been calibrated by using 3 methods of measuring goodness of fit. RMSPE, GEH and percentage error were utilized in the calibration process. The RMSPE was selected as the key factor in measuring the validity of the model. The result of RMSPE for flow, speed and queue length satisfy the criterion. The GEH and percentage error were the secondary evaluation methods. All MOPs evaluation proves that the model is calibrated and thus the model can ready to for validation process.

4.9 Validation

After the calibration process, the model validation is the final assessment for the validity of the model. The validation is carried out with a different data at the same location that was not used in the calibration process. This is vital to avoid over-fitting the model with a specific set of data. This study has obtained the turn count data and the average speed from another day during the same morning peak hour; from 7:00 – 9:00 AM. The research does not have the queue length information and thus it will be omitted in the validation process. The turn count data is transform into Origin-Destination matrix in the same way as shown in the Methodology section. The comparison methods are the same as the calibration process.
Flow validation analysis

<table>
<thead>
<tr>
<th>Time interval</th>
<th>Simulated</th>
<th>Observed</th>
<th>GEH</th>
</tr>
</thead>
<tbody>
<tr>
<td>7:00-7:15</td>
<td>997</td>
<td>1111</td>
<td>3.51</td>
</tr>
<tr>
<td>7:15-7:30</td>
<td>1485</td>
<td>1586</td>
<td>2.58</td>
</tr>
<tr>
<td>7:30-7:45</td>
<td>1363</td>
<td>1478</td>
<td>3.05</td>
</tr>
<tr>
<td>7:45-8:00</td>
<td>1319</td>
<td>1206</td>
<td>3.18</td>
</tr>
<tr>
<td>8:00-8:15</td>
<td>1485</td>
<td>1599</td>
<td>2.90</td>
</tr>
<tr>
<td>8:15-8:30</td>
<td>2295</td>
<td>2092</td>
<td>4.33</td>
</tr>
<tr>
<td>8:30-8:45</td>
<td>1607</td>
<td>1780</td>
<td>4.20</td>
</tr>
<tr>
<td>8:45-9:00</td>
<td>1341</td>
<td>1188</td>
<td>4.30</td>
</tr>
</tbody>
</table>

Table 17: RMSPE flow validation

Table 18: GEH flow analysis – validation (number of vehicles)

The first MOP utilize for the validation process is the flow. The calculation is the same as the calibration process. The results are presented in the above tables. The RMSPE yields 4.04% which is below the maximum threshold. The GEH confirms that the validation of the flow simulation. The highest GEH value is 4.33 which occur during 8:15 – 8:30. This result is lower than the threshold GEH value of 5.

The linear graph comparison illustrates that the model could accurately simulate the actual network. The results depict a 45 degree line which signifies its validity.
The final verification is the average speed on the major road. The average speed is illustrates in the above table. The speed is compare by RMSE and all the results are below the 10% limit. The inbound direction has the highest discrepancy with 4.08% RMSE value while the outbound direction yields 3.42% error. The validation result also satisfies the percentage evaluation. The inbound error is 5.77% and the outbound value is 4.84%. Both are below the 5% percentage error.

The validation process is completed. The selected measure of effectiveness have been tested and validated. The model is ready to be simulated and compare with the new scenarios.
Chapter V Results and Analysis

5.1 Replications - absolute error

One of the considerations of each simulation investigation is the number of replications. Two methods were employed to examine an acceptable level of consistency in the results; incremental calculation and the graph of the mean of the average against all the replications.

The incremental approach can be accomplished in 7 steps. The evaluation begins with an initial set of runs where the mean and the standard deviation are calculated. These estimates are evaluated and the required additional simulation runs is calculated (Burghout, 2004 and Quinzi, 2004).

1. Simulate an initial set of runs, \( m \geq 2 \) (\( m = 5 \) runs for this study)
2. Calculate the relative error, \( \varepsilon = \left| \frac{X(m) - \mu}{\mu} \right| \)
3. Adjust the relative error, \( \varepsilon' = \frac{\varepsilon}{1 + \varepsilon} \)
4. Calculate the required number of runs, \( n = \left( \frac{z_{\alpha/2}}{\varepsilon} \right)^2 \)
5. Calculate the new mean \( X(n) \) and variance \( S^2(n) \)
6. Calculate the half length of the confidence interval, \( \delta(n, \alpha) = t_{n-1,1-\alpha/2} \sqrt{\frac{S^2(n)}{n}} \)
7. If \( \frac{\delta(n, \alpha)}{|X(n)|} \leq \varepsilon' \) then \( X(n) \) is the required number of runs, else perform additional runs and repeat step 4

where \( X(m) \) - mean of \( m \) number of runs

\( \mu \) - observed mean

\( \sigma \) - standard deviation

\( z_{\alpha/2} \) - desired confidence level

\( \varepsilon \) - desired accuracy (5 for this study)

\( t_{m-1,1-\alpha/2} \) - critical value of the two-tailed t-distribution at a level \( \alpha \) of significance, given \( m-1 \) degrees of freedom.

The visual comparison also employed to assist in the accuracy of the replications. The outputs are put to the graphical format to compare how well the model fits with the observe data. The graph
illustrates 2 trend lines; the average and the mean of the average values. It can be said that the results will be consistent and stable when the mean of the average graph is leading to a smooth straight horizontal line. In an ideal situation, only one replication would simulate the situation. However, in this project, the inputs for the travel demands are derived from flow (vehicle/hour) and turn count movement, not from Origin-Destination matrix. Therefore, many replications needed to be performed to capture the true values from the variations.

The traffic parameters employed for the replication analysis are the MOPs that will be used in the network evaluation which are speed, travel time and maximum queue length. The flow is omitted in the network evaluation due to the demand forecast will automatically increase demand and thus made flow comparison irrelevant.

5.1.1 Speed

The analysis of the number of replication required is base on the network performance measure by S-Paramics, more specifically, the average speed of the network (km/hr), the travel time on the main highway (seconds) and the maximum queue length maximum (meter).

The speed analysis yields low variation in the test simulation runs with the standard deviation of 1.95. Only one additional iteration was required. The new result yields the mean value of 54.67 and the variance of 3.067. The speed parameter is tested against the adjusted error and satisfies the condition.

<table>
<thead>
<tr>
<th>Initial runs</th>
<th>New results</th>
<th>Evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean STDEV</td>
<td>Required runs</td>
<td>Mean Variance</td>
</tr>
<tr>
<td>54.60 1.95</td>
<td>6</td>
<td>54.67 3.07</td>
</tr>
</tbody>
</table>

Table 20: Number of iterations calculation - speed

The mean of the average graph is then drawn to support the number of iteration calculation. The average speed over the 6 iterations is 55 km/hr. The third simulation run yields the highest speed of 56 KM/hr and the lowest average speed of 53 km/hr which occurred during the first replication.

<table>
<thead>
<tr>
<th>Iterations</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVG speed</td>
<td>53</td>
<td>56</td>
<td>52</td>
<td>56</td>
<td>56</td>
<td>55</td>
</tr>
<tr>
<td>Mean of AVG speed</td>
<td>53</td>
<td>55</td>
<td>54</td>
<td>54</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

Table 21: Average speed and mean of the average speed (km/hr)
The figure above indicates low speed variations throughout the iterations. The mean of the average speed centralize to the true value. The mean of the average speed is above 54 km/hr as shown in the figure. This indicates that the simulation is able to produce true results for speed with minimal replications. The different in the mean of average speed is less than 3 km/hr which imply that 6 replications can conclusively produce an accurate speed simulation.

5.1.2 Travel time

The travel time is another MOP in evaluating traffic network. The initial travel time evaluation produces the mean value of 295.2 and the standard deviation of 14.46. The high variation required the large number of iterations, in this case, 18 iterations was performed. The new results are 291.78 for mean and 128.3 for the new variance. The evaluation is completed with the comparison against the adjusted error of 0.03.

<table>
<thead>
<tr>
<th>Initial runs</th>
<th>New results</th>
<th>Evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean STDEV</td>
<td>Required runs</td>
<td>Mean Variance</td>
</tr>
<tr>
<td>295.20 14.46</td>
<td>18</td>
<td>291.78 128.30</td>
</tr>
</tbody>
</table>

The mean of the average calculation yields the travel time value of 223 seconds after 18 iterations. The lowest travel time is 283 seconds which occurred in the 4$^{th}$ iteration and the longest average travel time is 321 seconds which occur in the 11$^{th}$ simulation run. The variation is in the simulation is trivial with less than 1 minute. The mean of the average graph lies just above 290 seconds for the
The majority of runs. The third iteration demonstrates the highest variation in the mean of the average graph. The mean of the average graph gain stability after the 7 iterations.

<table>
<thead>
<tr>
<th>Iteration</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVG travel time</td>
<td>303</td>
<td>287</td>
<td>317</td>
<td>283</td>
<td>286</td>
<td>292</td>
<td>286</td>
<td>285</td>
<td>285</td>
</tr>
<tr>
<td>Mean of AVG travel time</td>
<td>303</td>
<td>295</td>
<td>302</td>
<td>298</td>
<td>295</td>
<td>295</td>
<td>293</td>
<td>292</td>
<td>292</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Iteration</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>15</th>
<th>16</th>
<th>17</th>
<th>18</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVG travel time</td>
<td>295</td>
<td>321</td>
<td>285</td>
<td>286</td>
<td>288</td>
<td>297</td>
<td>286</td>
<td>290</td>
<td>280</td>
</tr>
<tr>
<td>Mean of AVG travel time</td>
<td>228</td>
<td>228</td>
<td>228</td>
<td>226</td>
<td>225</td>
<td>225</td>
<td>224</td>
<td>224</td>
<td>223</td>
</tr>
</tbody>
</table>

Table 23 : Average travel time and the mean of the average travel time (seconds)

![Average travel time and the mean of the average travel time](image)

Figure 43: Average travel time and the mean of the average travel time (seconds)

### 5.1.3 Maximum queue length

The last MOP chosen for this thesis is the queue length. The maximum queue at every 5 minutes interval is collected and the average of all the queues form in each interval is calculated. The maximum queue length encapsulates longest delay and waiting time.

<table>
<thead>
<tr>
<th>Initial runs</th>
<th>New results</th>
<th>Evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean STDEV</td>
<td>Required runs Mean Variance $\delta(n, \alpha)$ $\leq \varepsilon'$</td>
<td></td>
</tr>
<tr>
<td>128.00 9.03</td>
<td>7 126.57 60.29 0.03 OK</td>
<td></td>
</tr>
</tbody>
</table>

Table 24: Number of iteration calculation – Maximum queue length
The initial 5 runs yield the mean value of 128 and the standard deviation of 9.03. An additional 2 runs was required to the total of 7 iterations. The new results are 126.57 for the mean the variance is 60.29. The new result is tested against the adjusted error of 0.123 and the parameter is satisfied.

<table>
<thead>
<tr>
<th>Iteration</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVG maximum queue length</td>
<td>144</td>
<td>123</td>
<td>126</td>
<td>123</td>
<td>124</td>
<td>123</td>
<td>123</td>
</tr>
<tr>
<td>Mean of AVG maximum queue length</td>
<td>144</td>
<td>134</td>
<td>131</td>
<td>129</td>
<td>128</td>
<td>127</td>
<td>127</td>
</tr>
</tbody>
</table>

Table 25: Average maximum queue length and the mean of the average of the maximum queue length (meter)

The maximum queue length presented in the table above is shown in meters. The longest queue occurred in the 1st iteration with the average maximum length of 144 meter. The shortest queue length was 123 meter. The average of maximum queue length over 7 iterations is 127 meter. This is one of the main concerns that will be analyzed and solved. The variations in the results, both the high and lows, are coherent with those of the travel time. This show the correlation between the travel time and the queue length as expected.

![Figure 44: Average maximum queue length and the mean of the average of the maximum queue length (meter)](image)

The mean of the average graph presents a clearer view of the variation from each run. The simulations contain some discrepancy in the first three runs. The trend graph reveals that the simulations centralize to the true value after the fifth run. The mean of the average values contain less discrepancy and more consistent to the true average of 127 meter.

In conclusion, the number of replication needed depends largely on the most effective MOP which in this case is the speed, travel time and queue length. The combinations of the MOPs suggest 7 iterations are sufficient to derive consistency in the results. The number of replications can increase
if more precise values are required. The increase in the number of replications will yield more accurate results but like any model simulation, it is time and resource consuming.

### 5.2 New Scenario Evaluation

This research simulated two alternatives for traffic simulation from wide alternatives adopted by Department of Highway; Compact interchange and Flyover. The new designs aim at solving the traffic jam on the highway due to traffic light, U-turns at the same time reduce travel time for minor road traffic.

The simulation will focus on two most important traffic parameters; average speed, average and maximum travel time. The total travel time may not reflect the severity of the situation and thus this investigation selects the average travel time and the maximum travel time for comparison. The travel time parameter is used in collaboration with other MOP to evaluate the network. When solving the traffic situation (instead of planning a network development), investigating the maximum travel time proves more sensible. The maximum travel time is calculated from the average of the maximum travel time of every 5 minutes interval. The average of maximum journey time represents the severity of the traffic situation which is the main focus of any traffic research.

#### 5.2.1 Flyover scenario

The initial approach is to solve the main problem on the existing highway which is the congestion on the highway that is caused by traffic light. The flyover interchange main objective is to remove the traffic light from the highway traffic to reduce the cause of any interruption to the traffic flow. It replaces the traffic light on the highway with the flyover bridge over the junction. This design demand little road geometry alteration. This allows the free movement of vehicles on the highway while the junction under the flyover continues the use of traffic light however with less traffic volume. The flyover interchange design is adopted for both intersections. The results are compared with the base scenarios in term of the 3 MOPs.

<table>
<thead>
<tr>
<th></th>
<th>Base Scenario</th>
<th>Flyover Scenario</th>
<th>Percentage difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVG speed (km/hr)</td>
<td>41.14</td>
<td>69.81</td>
<td>70%</td>
</tr>
<tr>
<td>AVG travel time (sec)</td>
<td>295</td>
<td>130</td>
<td>-56%</td>
</tr>
<tr>
<td>Max travel time (sec)</td>
<td>403</td>
<td>172</td>
<td>-57%</td>
</tr>
</tbody>
</table>

Table 26: MOPs comparison between base and flyover scenario

The average speed of the network improves significantly with the flyover bridge over the intersection. The removal of traffic light on the highway has increase the flow of traffic and the average speed by 70% from 41.14 to 69.81 km/hr. The travel time has also improved.
travel time lessens from 295 to 130 seconds and the maximum travel time reduces drastically from 403 to 172 seconds. The queue length has completely disappeared because the flyover bridge crosses over the junction thus no queue is formed. The condition for measuring queue length is when the speed drops below 7.19 km/hr and the front gap drops below 10 meter. The queue is considered disperse when either the gap is more than 15 meter or the speed is above 10.8 km/hr (S-Paramics Reference Manual, 2007). Flyover scenario proves to be an effective solution to the current traffic problem with solving the congestion problem by eliminating the queue and improve speed and reduce of maximum travel time of 70% and 57% respectively.

5.2.2 Compact Interchange scenario (Partial Clover leaf)

Another alternative considered is the compact interchange scenario which focuses on removing the traffic light from the entire network. The traffic light has many advantages in managing traffic flow and movement but it proves to be inappropriate on the highway roads which desire a smooth and non-disrupted movement. In Thailand, there have been many cases of severe accident on urban road (Consultant report on traffic movement on regional highway, 2010). In the suburb area where traffic are minimal during the off peak period, many accidents occur when vehicles have to wait for the green light while there are no traffic in any other directions. This entices driver to run the red light and the accidents are severe because the vehicle from the opposite direction is driving at full speed, due to the green light.

The compact interchange alternative removes the traffic light from both the highway and the minor road by implementing a loop over the minor road section or what is commonly known as the clover leaf design. This design manages approaching conflict traffic similar to those of a roundabout. The compact interchange is the adaptation of the clover leaf interchange which employs 4 loops as ramp access. The compact interchange implements 2 way roads on 2 loops which reduce the land expropriate required when compare to the more traditional diamond interchange or full clover leaf interchange. The design is implemented on both intersections. The results are compared with the base scenarios in term of the MOPs.

<table>
<thead>
<tr>
<th></th>
<th>Base Scenario</th>
<th>Compact Scenario</th>
<th>Percentage difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVG speed (km/hr)</td>
<td>41.14</td>
<td>76.57</td>
<td>86%</td>
</tr>
<tr>
<td>AVG travel time (sec)</td>
<td>295</td>
<td>100</td>
<td>-66%</td>
</tr>
<tr>
<td>Max travel time (sec)</td>
<td>403</td>
<td>118</td>
<td>-71%</td>
</tr>
</tbody>
</table>

Table 27: MOPs comparison between base and compact scenario

The compact scenario is also compared with the base scenario. The average speed of the network increases considerably with the new partial clover leaf modification of the network. The removal of
the traffic intersection has considerably improves flow and speed. The average speed has increase by 86% from 41.14 km/hr to 76.57 km/hr. The travel time has drastically reduces also. The average journey time decreases from 295 seconds to 100 seconds travel time on average. And the maximum travel time also drops by 71% from 403 seconds to 118 seconds. The queue on the high has been dissolved.

<table>
<thead>
<tr>
<th></th>
<th>Compare to base scenario</th>
<th>Flyover scenario</th>
<th>Compact scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td>AVG speed (km/hr)</td>
<td></td>
<td>70%</td>
<td>86%</td>
</tr>
<tr>
<td>AVG travel time (sec)</td>
<td>-56%</td>
<td>-66%</td>
<td>-66%</td>
</tr>
<tr>
<td>Max travel time (sec)</td>
<td>-57%</td>
<td>-71%</td>
<td>-71%</td>
</tr>
</tbody>
</table>

Table 28: MOPs comparison of flyover and compact scenario to base scenario

The results from both the flyover and compact scenario show an improvement from the current situation. The compact interchange scenario produces more significant improvement from the two alternatives. The propose solutions not only target at solving the current situation but should also prepare for the future problems that may arise.

5.3 Future Demand

To further analyze the situation and the potential problems, the future demand is simulated in the existing scenario as well as the flyover, and compact interchange scenario. The AADT data from the Highway department reveals a 7% increase on average over the past 5 years. The analysis for the future demand will be simulated for the year 2014 and 2019. The new demand for the simulation uses the collected demand from the year 2009 and estimate the new demand for the next 5 and 10 years. The future demand will be simulated for 5 iterations for each scenario. The new demand for the simulation is shown in the table below.

<table>
<thead>
<tr>
<th>Year</th>
<th>2009</th>
<th>2014</th>
<th>2019</th>
</tr>
</thead>
<tbody>
<tr>
<td>Demand</td>
<td>6914</td>
<td>9784</td>
<td>13845</td>
</tr>
</tbody>
</table>

Table 29: Forecast demand

The S-Paramics has the capabilities to increase the demand proportionally for all routes thus ease the future demand development process. The simulation for the future demand is collected for the same morning peak hours between 7:00 – 9:00 AM. The MOPs for comparison are average speed for the entire network and journey time.
The current scenario configuration was simulated for the future demand for the year 2014 and 2019. This will be labeled as ‘Do Nothing’ scenario. The results show a very drastic traffic situation for all MOPs considered especially for queue length. The average of the maximum queue length has 363 meters. This proves that a new solution is necessary to be implemented in order to avoid such crisis. The future demand is simulated in both new scenarios and the results are presented below.

### Table 30: MOPs on future demand of do nothing scenario

<table>
<thead>
<tr>
<th></th>
<th>AVG speed (km/hr)</th>
<th>AVG travel time (sec)</th>
<th>Max travel time (sec)</th>
<th>Max queue length (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>2009</strong></td>
<td>41.31</td>
<td>295</td>
<td>403</td>
<td>167</td>
</tr>
<tr>
<td><strong>2014</strong></td>
<td>10.74</td>
<td>717</td>
<td>975</td>
<td>275</td>
</tr>
<tr>
<td><strong>2014</strong></td>
<td>6.97</td>
<td>736</td>
<td>1066</td>
<td>363</td>
</tr>
</tbody>
</table>

### Table 31: MOPs on future demand of flyover scenario

<table>
<thead>
<tr>
<th></th>
<th>AVG speed (km/hr)</th>
<th>AVG travel time (sec)</th>
<th>Max travel time (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>2009</strong></td>
<td>69.81</td>
<td>130</td>
<td>172</td>
</tr>
<tr>
<td><strong>2014</strong></td>
<td>58.39</td>
<td>153</td>
<td>213</td>
</tr>
<tr>
<td><strong>2019</strong></td>
<td>10.13</td>
<td>458</td>
<td>840</td>
</tr>
</tbody>
</table>

### Table 32: MOPs on future demand of compact scenario

<table>
<thead>
<tr>
<th></th>
<th>AVG speed (km/hr)</th>
<th>AVG travel time (sec)</th>
<th>Max travel time (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>2009</strong></td>
<td>76.57</td>
<td>100</td>
<td>118</td>
</tr>
<tr>
<td><strong>2014</strong></td>
<td>72.73</td>
<td>104</td>
<td>131</td>
</tr>
<tr>
<td><strong>2019</strong></td>
<td>30.01</td>
<td>176</td>
<td>268</td>
</tr>
</tbody>
</table>

### 5.3.1 Speed

Each of the MOPs will be analyze separately. First, we investigate the average of the network. The speed will be the most influential factor in deciding the best scenario.

### Table 33: Speed comparison of the 3 scenarios on the future demand (km/hr)

<table>
<thead>
<tr>
<th></th>
<th>2009</th>
<th>2014</th>
<th>2019</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Do Nothing scenario</strong></td>
<td>41.14</td>
<td>10.74</td>
<td>6.97</td>
</tr>
<tr>
<td><strong>Flyover scenario</strong></td>
<td>69.81</td>
<td>58.39</td>
<td>10.13</td>
</tr>
<tr>
<td><strong>Compact scenario</strong></td>
<td>76.57</td>
<td>72.73</td>
<td>30.01</td>
</tr>
</tbody>
</table>

The mean of the average speed for the do nothing scenario has drastically reduced in the year 2014 to below 11 km/hr and it become almost stagnant in the 2019. The average speed is below 7 km/hr. The figure below gives a clear different of each scenario. The two propose scenarios solve the existing problem in the do nothing scenario with the compact scenario being the more efficient solution. In 2014, the mean of the average speed of compact scenario is 72.73 km/hr decreases from the base demand by 5% while the mean of the average of flyover scenario decreases by 16% from
69.81 to 58.39 km/hr. Both scenarios are able to manage the traffic problem in the 2019. The last period of simulation which see the demand more than double from 6914 to 13845 vehicles, is the most critical period. The flyover scenario cannot cope with the amount of vehicles in the network which cause the speed to plunge to 10.13 km/hr. The compact scenario also heavily effect by the increase of demand but it still manage to produce the average speed of 30.01 km/hr which is consider acceptable for the morning rush hour period (The interstate highway study between Nakorn Prathom and Cha Um: const benefit, technical design and economical, 2007).

![Figure 45: Speed comparison of the 3 scenarios on the future demand (km/hr)](image)

The mean of the average speed graph above shows that the speed for both the base scenario and the flyover scenario in the 2019 decreases to 6.97 and 10.13 respectively. The compact scenario is the most efficient at maintaining the average speed at an acceptable level in the simulation year 2019 although a significant decrease can be observed with the steep curve in the last section of the compact scenario. The compact scenario proves the best solution in improving the average speed of the network.

5.3.2 Travel time
The other MOP crucial in deciding the network efficiency is the travel time. This research will present both average travel time and the maximum travel time but the maximum travel time will be the main focus of discussion while the average travel time is presented as reference.
The trend for the average travel time is consistent of that of speed. Both the flyover and compact scenario are more efficient in improving the travel time than the do nothing scenario. Do nothing scenario produce the travel time of 295 seconds while the flyover and compact scenarios simulated 130 and 100 seconds, respectively. Simulation in 2014 yields a similar result. The compact scenario reduce travel time greater than that of flyover scenario when compare to do nothing scenario, by 86% and 79% respectively. In 2019, the compact scenario continues to improve travel time, lessen by 76% from 736 seconds in do nothing scenario to 104 seconds while the flyover scenario reduces travel time by 38% to 458 seconds.

The maximum travel time shows a similar trend to those of average travel time; both mean of the average graphs are presented below. They produce an almost identical trend graphs. Both flyover and compact scenario are able to significantly decrease travel time from the do nothing scenario in 2009 and 2014. The flyover scenario decreases maximum journey time by 57% and 78% while the compact scenario lessens maximum travel time by 71% and 87%. 2019 demand proves to be difficult for flyover scenario to manage. It was able to decrease maximum travel time by only 38%. The compact scenario continues to improve the journey time in 2019. It lessens travel time by 76% from 1066 in the do nothing scenario to 268 seconds.
As mentioned above, the maximum journey time and the average travel time have the same trend lines. The maximum travel time is more appropriate in indicating the severity of the situation. Although flyover and compact scenario can adequately solve the current and 2014 traffic problem, compact scenario proves superior in handling the increased demand of 2019. The travel time and maximum travel time suggest the compact scenario is the best solution in solving traffic problem in the long run.

### 5.3.3 Queue length

The queue length was dispersing after the traffic light on the main road was removed in 2009 for flyover and compact scenarios. The same was true for demand in 2014. However, in 2019, the high
volume of vehicles cause the queue to reemerges, therefore the study will compare and analyze only for this period.

<table>
<thead>
<tr>
<th>Average queue length (meter)</th>
<th>2009</th>
<th>2019</th>
</tr>
</thead>
<tbody>
<tr>
<td>Do Nothing Scenario</td>
<td>99</td>
<td>173</td>
</tr>
<tr>
<td>Flyover scenario</td>
<td>94</td>
<td></td>
</tr>
<tr>
<td>Compact scenario</td>
<td>36</td>
<td></td>
</tr>
</tbody>
</table>

Table 36: Average queue length comparison of the 3 scenarios on the future demand (meter)

The reference queue length is 173 meter from the do nothing scenario with 2019 traffic demand. The average queue length is reduced by 46% with the flyover scenario to 94 meter. This is considered a significant percentage decline but 94 meters average length is relatively high. This is similar length to the existing problem in the base scenario. The compact scenario demonstrates a reduction of 79%. The average queue was reduced to 36 meters.

<table>
<thead>
<tr>
<th>Maximum queue length (meter)</th>
<th>2009</th>
<th>2019</th>
</tr>
</thead>
<tbody>
<tr>
<td>Do Nothing Scenario</td>
<td>167</td>
<td>363</td>
</tr>
<tr>
<td>Flyover scenario</td>
<td>236</td>
<td></td>
</tr>
<tr>
<td>Compact scenario</td>
<td>88</td>
<td></td>
</tr>
</tbody>
</table>

Table 37: Maximum queue length comparison of the 3 scenarios on the future demand (meter)

The maximum queue length demonstrates a severe situation in the do nothing scenario as the maximum queue length reaches 363 meters in 2019. The flyover alternative was able to reduce the queue length by 35% to 236 meters. In flyover scenario produced satisfying percentage decrease in both average and maximum queue length but the actual queue length form remains extensive. Compact scenario lessens the maximum queue length in 2019 to 88 meters. A reduction by 76% from the base scenario confirms that the compact scenario is the most efficient alternative to the existing traffic problem.

5.4 Discussion & Recommendation

The compact scenario proves a superior alternative than the flyover scenario in all 3 MOPs. In 2014, compact scenario has a slight edge over the flyover scenario but in 2019 the flyover scenario cannot cope with the high volume of vehicle thus signifying the higher efficiency of the compact design to manage large traffic volume. The compact scenario was able to produce the average travel time less than 3 minutes with the average speed of 30 km/hrs. However, the results also indicate that the change in the road geometry and design alone will not be able to manage the increase in traffic volume in the long run. For example, the 88 meters maximum queue form is unacceptable. Other traffic management methods need to incorporate with the network alteration. Car pooling and HOV
lanes which have been tested in Bangkok in the past decades, may be practical (Bangkok Highway Assessment Report: the follow up report, 2003). Traffic law enforcement must be improved to prevent or lessen the violators. Road designs and other components must follow the same standard and design to avoid confusion from unfamiliar road users. In the study, the at grade u turns have been replace with u turns bridge which also improve the overall network efficiency. Its placement has also been strategized by positioning the U-turns at every 1 kilometer.

The compact scenario proves superior not only in the intersection design but also on the highway section. The analysis took place only on the highway in both directions. The results, speed, travel time and queue length were all collected on the main road where both design are straight road on the highway. However, the compact scenario is the flat road while the flyover scenario composes of a 6 lanes pass over bridges over 2 intersections. The bridges prevent vehicles from operating at a smooth flow by having to climb the bridge. This is essential when the traffic compose of large and heavy vehicles that are difficult in traveling at an incline angle. These low performance vehicles lower the performance of the network as a result of the pass over bridges. As the simulation results reveal, the compact scenario produces a higher efficiency network than the flyover over scenario because there are no bridges.

The affect the new designs have on the minor road could be beneficial. The only available data is collected from the highway, however, the simulator has the ability to produce the overall network performances (highway and minor road). In the flyover over scenario, the traffic condition on the minor road should improve due to the reduced traffic passing through the traffic light. The highway traffic is diverted from the intersection to the overpass bridge. This helps improve the performance of the intersection below and thus the minor road. The lower traffic volume at the traffic light allows the minor road traffic to perform at a higher level. Similarly to the compact scenario, the traffic light is completely remove thus reduce delay for both traffic on the highway and minor road. The minor would benefit greatly from the new designs.

The projected future demand is base on the static calculating of the average annual increase. The demand may fluctuate and disperse depending on the economy of the country and the new designs. The future demand calculated is only a prediction from the pass behavior. The new design can change the development pattern and thus alter the demand. For example, for the flyover scenario, the dwellings could develop more densely after the foot of the new bridge. This is because the potential customers would bypass the stores under the bridge, near the intersection and attract to the new stores with more accessibility. Similarly, the compact scenario could disperse the communities on the minor road near the intersection to further down the minor road because the
dwellings may be want to be near the highway. And in its place, new commercial buildings could take its place. The study of future developments and urban planning require more research and is beyond the scope of the thesis.

This research only consider the efficiency measure by 3 MOPs; speed, travel time and queue length. Additional factors are required when analyzing alternatives for implementation. Two other factors were adopted as part of this research for the Department of highway. Cost and benefit analysis is the most common analysis when proposing a new project. It compares that total cost with the return or benefit in momentary and non momentary value. Another important consideration is the land expropriation. This is highly political (in Thailand). The size and property value depends greatly on political connection. The power of persuasion for land expropriation also has deep political connection. The cost analysis is another important factor in selecting a project. This thesis did not cover the cost aspect in altering the intersection as well as the new U-turn bridge design. The cost may disagree with the results from the simulator. These two factors are beyond the scope of this thesis and thus more study is required.

A shortcoming of this thesis is the inability to fully capture the unique behavior of the motorcycle. Although, this thesis tried to capture the motorcycle characteristics by adopting 0.18 PCU from a research done in Thailand. This does not cover the unique driving pattern in this part of the world. As of this moment, no micro-simulator in the market can simulate such situation. One method that may be useful in representing motorcycle in the traffic flow is increase the lane changing behavior to represents the lane changing behavior of motorcycle. A portion of passenger vehicle which represent motorcycle can be assigned to the increase of changing lane behavior. This can be done in S-Paramics in between nodes where the merge traffic command can be inserted. This method would, in theory, have the equivalent number of lane changing maneuver; however, the location at which the lane changing is performed must be fixed. In reality, motorcycles change lane to move to the unoccupied space. This behavior is unpredictable of when and where it might occurred. Another unique driving behavior is the queuing in front at the traffic light. This behavior is seen throughout South East Asia. Motorcycles queue in front of the vehicles stopping at the light. This causes the vehicles to delay their start when the light green because they have to wait for all the motorcycles to discharge. A method that can replicate this situation is to delay the discharge of vehicles waiting for the traffic light. An increase yellow light of the previous signal plan would delay the traffic in the current traffic. This will not discharge additional vehicles from the previous and at the same time slow the discharge of vehicles at the current signal plan.
Also, the illegal driving cannot be replicate in the simulation. This type of behavior is common in South East Asia where drivers lack discipline and law enforcement is lenient. Illegal maneuver such as driving in the wrong direction and turning in and from the wrong lane cannot be simulate by a Microsimulation software, however, these maneuvers are most often done by avoiding the main traffic and get caught by the law enforcement. Although the behavior is common most do not disrupt the main traffic. The ones that affect the main traffic do cause serious accidents. This cause delays and effect the main traffic flow. The additional delays can be simulated by assigning the node to be a loading area which in turn would delay the traffic. The loading area can be assigned near the u-turns where the driving in the wrong direction would occur, however, the randomness in the behavior cannot be predicted and thus simulate. This illegal behavior can be counter in reality by stricken the laws.

Another weakness in this thesis is the accuracy and validity of the data. The data is collected manually and most of the surveyors are inexperienced. Another point is that the data is collected over one day period. The average derive from these data may not represent the true value. More data would improve the accuracy of the simulation and thesis. This inaccurate representation of the network may cause high sensitivity in the result. Because the TMC is collected manually, this could affect the number of vehicles represented in the network. The speed data collection process was also completed in one day. Data was collected once per direction and once for the morning and once for the off peak period. The morning peak yields the average speed while the speed collected during the off peak period represents the network design speed. The floating car method does not offer a conclusive speed pattern. It captures the speed of a specific run in a specific location and thus may not be a true representative of the average speed. However, with most of field data collection, time and resource are scarce. Data from several days period with more accurate method and technology would improve the accuracy and the validity of the thesis. This thesis offer the most accurate solution from the available data and resources and a more accurate data may be required for a higher level accuracy in the results.
Chapter VI Conclusion

The main objective of this study was to adopt an alternative to solve congestion problem on highway number 4 between km 53 to km 58. The traffic evaluation and analysis is carried out by traffic simulation program, S-Paramics. The majority of the data was collected in 2009 and thus it will be the base year for comparison; base scenario.

From the sight survey, it was evident that the traffic congestion is caused by traffic light which disturbs the traffic movement on the main highway. Many alternatives that modify intersection with various junction designs were considered and two were adopted for the simulation; flyover bridge design and compact interchange design. To evaluate and improve the efficiency of the road network, 3 MOPs are adopted for this study. The speed and travel time are the most important measure of performance in evaluating the performance of a road network while decreasing the queue is a decisive decision factor.

By removing the traffic light intersection, the performance of the network improves significantly as expected. Both scenarios increase efficiency of the network in all aspects with compact scenario being the more efficient. This was more apparent in the 2019 simulation where it steadily improves the efficiency with 59% increase in speed, 70% decrease in travel time and 79% decrease in the queue length. The flyover scenario could not keep a consistent performance and the mean of the average trend graphs show steep decline in all of the MOPs.

The draw backs of the compact interchange design are land expropriation and public disagreement. The compact interchange requires more land to build the loop that acts as the roundabout. An accurate cost of land expropriation must be included in the project budget. The resistant from the public especially those who owns shops and store near the junction is a sensitive issue. The opposing problem is that the overpass bridge will deter potential customers which would otherwise pass the stores at the ground level. This affects the business and the well being of the inhabitants. The flyover bridge also blocks the landscape of the housing alongside the bridge. The local government must inform and educate the potential benefits and include public participation from the early stage of the development. Also, the simulation program, S Paramics, was design for traffic operation and thus the traffic light in the simulation may not be an optimal signal plan. This may cause in longer waiting and ultimately higher journey time.

With the proposed designs, traffic management must also improve. The law enforcement must be stricter to eliminate illegal driving behaviors which reduce network safety. This must be implemented by the national government. This thesis tested the two proposed alternatives. A detail
pneumatic design using more accurate sight dimension and specification is required for a more
detail and accurate results. This is part of the planning process and may lack in depth and more
accurate results.
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