FINAL PROJECT

PERFORMANCE ANALYSIS OF MONJALI INTERSECTION AND ITS IMPACT ON FUEL CONSUMPTION

Submitted to Islamic University of Indonesia Yogyakarta to Meet the Requirements to Obtain Bachelor Degree in Civil Engineering

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I thus declare that my work is apparent in the Final Assignment report that has been completed in order to fulfill the requirements of the Civil Engineering Undergraduate Program at the Faculty of Civil Engineering and Planning, Universitas Islam Indonesia, for the recognition of a Bachelor's degree. The conventions, guidelines, and ethics of writing scientific papers are clearly followed in the source for certain sections of the Final Project report that I cited from other people's work. In compliance with all relevant rules and regulations, I agree to accept consequences, including the revocation of my academic title, if it turns out later that all or a portion of my Final Project report contains plagiarism.

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PREFACE

The author would like to thank Allah SWT for His guidance to complete the Final Project titled “Performance Analysis of Monjali Intersection and Its Impact on Fuel Consumption”. This Final Project is one of the academic requirements for completing the Civil Engineering Undergraduate Program, Faculty of Civil Engineering and Planning, Universitas Islam Indonesia, Yogyakarta.

Although there were many challenges the author had to overcome in order to prepare this Final Project, the author was able to overcome them with the help of many people's advice, critiques, and support. Regarding this, the author would like to extend their sincere gratitude to:

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Author,

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LIST OF NOTATION AND ABBREVIATION

pcu	= Passenger Car Unit
LV	= Light Vehicle
HV	= Heavy Vehicle
MC	= Motor Cycle
UM	= Unmotorized
IHCM 1997	= Indonesian Highway Capacity Manual 1997
pce	= Passenger Car Equivalent
F_{SF}	= Factor of Side Friction
F_{CS}	= Factor of City Size
C	= Capacity (pcu/hour)
C_0	= Base Capacity (pcu/hour)
F_{RT}	= Factor of Right Turn
F_{LT}	= Factor of Left Turn
DS	= Degree of Saturation
Q	= Arus total (pcu/hour)
V	= Velocity (km/hour)
L	= Length of segment (m)
kl	= Kilo-liter
l	= liter
ml	= milliliter
cc	= cubic centimeter
LOS	= Level of Service
PTV - AG	= Planning Transportasi Verkher AG
VISSIM	= Verkehr InStadten Simulations Model
m	= meter
km	= kilometer
GEH	= Validation value using formula of Geoffery E. Havers
MAPE	= Mean Absolute Percentage Error

ABSTRACT

Vehicle collisions often take place at intersections. Based on data from *Badan Pusat Statistik*, Yogyakarta's Special Region has seen a sharp increase in the number of automobiles. It is estimated that the overall number of passenger cars increased by 3.83%, trucks by 4.00%, and motorbikes by 3.30% between 2020 and 2021. The length of the vehicle queue grows as a result of this vehicle growth, lengthening the wait time. This has an impact on the amount of fuel oil wasted as a result of the lengthy delay. On 2021 up to 2023 *Dinas Lingkungan Hidup dan Kehutanan* Yogyakarta did air quality survey which shows the 70% increase of Yogyakarta pollution. The objectives of this study are to assess the Monjali signalized intersection's performance with the presence of alley, ascertain how queue length and delay relate to fuel oil, and compare the best condition for the intersection.

The 1997 Indonesian Road Capacity Manual (IHCM 1997) theory, PTV VISSIM software calibration, and LAPI-ITB theory (fuel consumption) were all used in this study. There are four arms at this crossroads, along with four-time phases and a short alley on the north arm. A direct survey is used in the field to obtain data. The signalized intersection's performance as well as the correlation between wait times and queue length and fuel oil consumption are examined. The degree of saturation (DS) value at the Monjali signalized intersection is known to be larger than 0.85 based on the analysis's findings especially with the existence of the alleyway, indicating that the intersection is already oversaturated.

Given that the average delay number is more than 60 seconds which is 386 seconds precisely, the Monjali intersection's level of service is rated as F. By analyzing the several conditions, each condition has its advantage and impact to the intersection. By closing the alleyway, resulting in the decrease of delay compared to existing condition and safest flow compared to other alternatives, by adding separated phase to the alleyway resulting in longer cycle time with the note that the alley still exist, and it is found that changing the protected phase into opposite phase has significant impact which the delay decreased by 56% with the note that the alleyway is still exist but the safety should be considered. It was discovered that a total of 8.653 cc/pcu of fuel oil were lost at the Monjali intersection as a result of the queue and the delay in time. Result of the condition to close the alleyway has a fuel consumption of 2.836 cc/pcu, 3.002 cc/pcu in five phase conditions, and 1.531 cc/pcu for opposite flow.

Keywords: Alleyway, Delay, Fuel Consumption, Intersection, Performance.

CHAPTER 1
INTRODUCTION

1.1 Background

Highways are one of the most important transportation infrastructures that supports reconstruction particularly in supporting people economic activities and region development. Highway needs transportation system which coordinates the movement process of passengers and objects so the transportation process can be obtained at the best condition while considering safety factor, convenience, fluency and efficiency of time and cost. In some highways there will be a point called intersection, where two highway networks meet, and it is also where the problem in traffic flow happen. The performance of an intersection is the primary factor in selecting the best course of action to take in order to maximize its functionality, particularly regarding to the issue of the amount of traffic that can travel through the intersection. One of the examples, of the increasing volume of vehicles that are affecting the road performance is the intersection of Monjali.

Various types of vehicles become unity so that resulting in delays, jam, and accident. Area of that intersection has many office buildings and markets that resulting in many people pass that road. Population growth and increased community needs for transportation facility in Yogyakarta will cause more crowded vehicles that pass the area. The north arm of 4-leg intersection of Monjali is a dense area that makes the traffic compact moreover at the peak hour. The north arm of this intersection is already compact meanwhile there is an alley on the left side of that road which makes the jam worse. The 4-leg intersection of Monjali has the sign of “Turn Left Directly” which intended to make more smooth traffic, but the existence of the alley has been interrupting this sign of north arm. Vehicles that have to turn left which can pass directly has to wait for the road to be clear from the vehicles from the left alley as could be seen in Figure 1.1.

Yogyakarta has several signalized intersections, one of those intersection is Monjali intersection. It is located in the east side of *Monumen Jogja Kembali* (Monjali). The classification of this intersection is 412 based on IHCM 1997, this 4-leg intersection has 1 lane 2 ways on the minor approach with the width of 11 meters on north arm and 10,5 meters on south arm. 2 lanes (slow and fast lanes) on the main approach with width of 3,5 meters on slow lanes and 7 meters on fast lanes as described in the road geometry of Monjali Intersection in Figure 1.2. Except for the slow lane on west arm (east direction) it was widened, so it has the width of 7 meters alone. The alley in the left side of north arm has width of 5 meters. The vehicles on Special Region of Yogyakarta have been increasing rapidly based on *Badan Pusat Statistik* data. It is known from 2020 to 2021, the total of passenger cars has increased around 3,83%, trucks 4,00%, and for motorcycles 3,30%. Saputri (2022) found according to the findings of the IHCM 1997 examination of the actual conditions, the DS values for the south and north arms do not satisfy the IHCM 1997 criteria (DS=0,85). The observation lines on the west, south, east, and north arms were 350 meters, 300 meters, 240 meters, and 220 meters, respectively. The interchange level of services falls into category F because to the 160 second delay. Road widening at the south and north arms of an intersection can reduce DS values on those arms, cut average intersection delays by 83 seconds, and drastically shorten line length in the intersection arms. It is found that the level of service (LOS) of Monjali Intersection is F.

The use of motorized vehicles has grown to be a significant aspect of people’s lives today, serving as both a method of transportation and a gauge of success. Higher rates of population motorization from year to year are evidence of this. In general, there are two categories of motorized vehicles: public and private. Private vehicle use is more prevalent that that of public transit. This is due to the fact that private vehicles typically offer a greater caliber of service compared to public transportation, which is run by both public and private operators. Additionally, Yogyakarta’s public transit still falls short in terms of both comfort and safety. Vehicle operating expenses and time values are included in the operational costs of road use. The speed of the vehicle is directly correlated with both cost considerations. Vehicle running expenses tend to rise at low speeds or in congested areas where there is fuel waste, component wear, and time waste.

All kinds of motor vehicles require fuel oil. Fuel usage increased as a result of the rise in the number of automobiles. In order to understand the factors that are related to fuel consumption and the reasons behind its yearly increase, special attention must be paid to the transportation sector, particularly fuel consumption. The loss will be significant. The length of the wait at the signalized intersection can have an impact on the amount of fuel consumed when the vehicle is stopped there.

Transportation continues to be the sector with the highest fuel consumption when compared to other sectors, such as industry and power plants, according to the report on the findings of the Energy Supply-Demand evaluation and analysis study doneby the Ministry of Energy and Mineral Resources in 2012. Out of the overall fuel demand in 2011, which was 70,89 million kilo-liter, fuel oil usage in the transportation sector accounted for 65%, power generation 16%, industry 10%, home 2%, commercial 1%, and other sectors 6%. From the previous 68,14 million KL, this number has climbed by 4,04% since 2010.

Sinambela et al., (2021) stated that analysis of the intersection is using the calculation method of Indonesian Highway Capacity Manual (IHCM) year 1997, modelling using software VISSIM 2022 version, and also fuel oil energy consumption analysis using the approachment method of LAPI-ITB 1996 that is converted in passenger car unit (pcu). The purpose of this analysis to observe the parameters: Degree of Saturation (DS); Queue length; Delay, to know the Level of Service (LOS), and provide alternative to optimize the performance of Monjali intersection, especially in the north arm of the intersection and also to analyse the fuel oil consumption that is affected by the delay of vehicles. To collect the primary data which are vehicle velocity, the method of traffic counting is used. Additionally, primary data in the form of the speed of passing vehicles were taken using the spot speed method using segment. In addition, this study might be needed to help increasing the performance of an intersection by adding suggestion after reviewing the analysis of the intersection performance.

Figure 1.1 Traffic Jam from Alley and North Arm of 4-leg Monjali Intersection

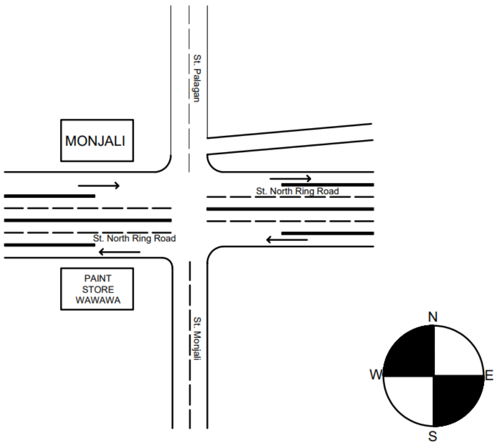

Figure 1.2 Road Geometry of Monjali Intersection

1.2 Problem Formulation

The formulation of the problem to be discussed in this study is as follows.

1. How does alley influence Monjali intersection?
2. How is the performance of Monjali intersection if the alley affects the north arm?
3. How does the performance of Monjali intersection affects the fuel oil consumption?

1.3 Purpose

The purposes of this study are:

1. Evaluate the existing performance of the intersection due to the influence of the alley on the north arm.
2. Knowing the solution for the intersection to improve performance.
3. Evaluate fuel oil demand in existing conditions and after the solution.

1.4 Benefits

The benefits of this study are:

1. It is hoped that the results of this study can be a reference for performance intersection by relevant parties to make decisions and take action to further optimize the performance of intersection/
2. It is hoped that new insight is found to minimalize the risk of accident that could happen in the intersection area.
3. It is hoped that this research could add source for the fuel oil energy references.
4. It is hoped that the results of this study can be a study material and reference for other students.

1.5 Limitations

This study of intersection modeling is a study which has a wide scope, then the limitations of the problem are set, including:

1. Geometrical condition, covering the width of the road of each intersection line, the number of lanes, and the type of intersection.
2. Primary data including traffic volume, phase, velocity and cycle time.
3. Traffic conditions, namely by recording all vehicles that passing intersections with the division of vehicle types, recording of traffic regulation conditions and traffic flow movements.
4. The performance parameters of the road sections used are the degree of saturation and average velocity using segment method.
5. Analysis method that is used refers to Indonesian Highway Capacity Manual (IHCM 1997) and modelling using software PTV VISSIM.
6. Grouping of vehicle types that is observed:
    a. Light Vehicle. Example: private vehicle and public car.

b. Heavy Vehicle. Example: truck and bus.
c. Motorcycle.
d. Unmotorized vehicles.

7. Analysis of fuel oil consumption based on delay that happens in the intersection using the approaches method of LAPI-ITB 1997 that is already converted to passenger car unit (pcu) by Isnaeini (2003).
8. Deliberations of the relationship of intersection performance (delay) with the consumption of fuel oil.
9. For reduction of vehicles, only light vehicles and motorcycle are reduced.

CHAPTER 2
LITERATURE REVIEW

2.1 Intersection Performance

Suryaningsih et al., (2020) conducted research that took place in Hasanuddin street – Kamboja street, Sumbawa Besar, which is a congested area because it is a center for business, government, and education. The goal of this study is to evaluate the performance of signalized intersection based on an examination of signal time, capacity, saturation level, and level of intersection services using IHCM 1997 technique. Specifically, the degree of saturation for the west, south, and east approaches is 0.53, 0.55, and 0.56 at level C (current is steady but speed is constrained). This shows that the signalized intersection is still functioning fairly well because of the saturation level is still below 0.75.

Sinambela et al., (2021) stated in the research that delay parameter that occurs on each approach in the peak hour, is an indicator determination of intersection performance through the level of service (LOS) for each approach. Level of service classification level of service of an intersection based on the delay according to IHCM 1997. the performance of the intersection in the morning peak hour with category D (less) with average intersection delay = 28.45 sec/pcu. The worst performance occurs on approach N with delay = 80.34 sec/pcu with a level of service of category F (very poor), as well as the best performance is on the NE approach with delay = 22.93 det/pcu category C (medium).

Saputro (2013) stated that traffic problems are generally caused by the irregular direction of vehicle flow at intersections. This occurs during peak hours, with traffic volumes reaching maximum levels. To overcome these problems, it is necessary to establish traffic control at road intersections, including traffic lights as a traffic flow regulator. The evaluation of the four-signalized intersection obtained a Degree of Saturation (DS) of 1.001 > 0.85, indicating that the the intersection is above the saturation limit with a cycle time of 105 seconds.

2.2 Analysis of Alley Road

Romadhona and Fauzi (2018) performed study about analysis of alley impact on u-turn towards the performance of Affandi street road section. This study focused analyzing the queue length, delay, and speed, spesifically examining the impact of a U-turn facility located in front of an access road. Data collection took place on Saturdays and Wednesdays and was analyzed dusing VISSIM software. The analysis of the existing conditions revealed a queue length of 67,03 meters, a delay of 22,61 seconds, a north-south speed of 23,04 kmph, and a south-north speed of 26,69 kmph.

Syahidan et al., (2016) conducted a study that focused on the performance evaluation and improvement of the traffic sign on Giwangan intersection in response to the increasing population and traffic volume in Yogyakarta. The evaluation of the existing intersection revealed a high average delay of 499,42 second per vehicle with a service levell rating of F, indicating poor performancc. To address this issue, three alternative solutions were proposed: implementing a new signal cycle plan with a delay of 92,42 seconds per vehicle, or a combination of both resulting in a delay of 58,56 seconds per vehicle.

Susanti (2015) conducted a study is the performance of Krian Five Intersection is in the LOS F category which means the intersection performance is poor. From the results of the traffic counting survey and identification of problems in the field, it can be made a handling strategy plan with Traffic Management which is divided into 3 periods, namely short term, medium term, and long term. For short-term strategies coupled with changes in cycle time settings to 70 seconds, green time of 30 seconds and red time of 40 seconds, the level of service in each road section which was originally at LOS F changed to LOS D and LOS C. On Jalan Gubernur Sunandar, the planned conditions are at LOS C during the day with a DS value of 0.66 and an afternoon of 0.66. 0.66 and 0.73 in the afternoon. On Jalan M.Yamin during the daytime is at LOS C with a DS value of 0.66. DS value of 0.66.

2.3 Delay Time Analysis

Lukita et al., (2022) carried out a research to ascertain the possibility of delays and lines associated with the crossing door of a plot at the Bekasi Station Crossing during its working hours. This study takes a quantitative approach, performing analysis with Vissim software and linear regression. The study and discussion's findings demonstrated a strong influence on the variables of delay, vehicle queue lengths, and the number of trains crossing a plot. The Road Geometry variable also has an impact on the length of lines and delays caused by moving cars. H. Juanda, IR, Bekasi. The study's findings should be taken into account while managing traffic and have the potential to reduce the likelihood of traffic congestion.

Novianka P et al., (2020) did a study of traffic delay time in signalized intersection that is located at the T-junction intersection of Brigjen Sudiarto street – Majapahit street in Semarang City. This area has high traffic growth and the traffic system is not functioning properly. In order to collect both primary and secondary data for intersection management processing, a field study was done. Planning processes traffic data using Excel software and IHCM 1997. In order to study the intersection’s behavior, traffic data is collected by counting the number of vehicles on the road for three days during peak hours. The value of the degree of saturation (DS) at the intersection of Brigjen Sudiarto street and Majapahit street was calculated using the analysis result which has the value of 0.991 and this value has exceeded the required value by the IHCM 1997, DS $\leq$ 0.85. According to the analysis, this intersection has Level of Service (LOS) F (>60) due to the average traffic delay, which is 141.320 sec/pcu.

Yunus et al., (2020) conducted a research which was aimed to offer logical alternatives as input to the related institute and also road users. According to the findings of the traffic volume analysis on Tegal City highway affected by shunting operations, peak hours were observed to happen three times a day, in the morning, afternoon, and evening. The density on Abimanyu street at 12.45 – 13.45 with a traffic volume of 2774 pcu/hour, on Menteri Supeno I street between 16.30 – 17.30 with a traffic volume of 1549 pcu/hour, and the last one which occurred on Menteri

Supeno II between 16.00 – 17.00 with a traffic volume of 899 pcu/hour. Results from the analysis, from Abimanyu street had the longest line, measuring 70.5 pcu with a 581.5 seconds delay per pcu. The traffic on Menteri Supeno I segment was reported to have reached 47.8 pcu with a delay of 441 seconds per pcu, for the queue at the Menteri Supeno II segment have reached a queue of 10.8 pcu with a delay of 368.5 seconds per pcu, meanwhile on Semeru street segment queue was reported to have reached 17 pcu with a delay of 395.6 seconds per pcu. According to the findings of alternative analysis, one of the keys to overcome the issues with traffic queues and delays was splitting the shunting time into two phases, where the traffic queue shrank to 35.3 pcu with a delay time of 290 seconds/pcu. Another key was switching the shunting schedule to an off-peak period.

2.5 Analysis Using PTV VISSIM Software

Tunggadewi (2022) did research on Condongcatur intersection which still have congestion problem. This research has a purpose to evaluate intersection and interchange performance. Analysis is done in every modelling in existing condition and two alternatives solution are obtained. Running VISSIM software for 3 modellings was done with calibration and the same randomseed. From the performance evaluation that was done the final decision for the alternative suggestion is to ream every arm and to eliminate median in the north arm and south arm. From the analysis of Alternative II, it shows that the average value of capacity increased 57,69% and decrease on delay value as big as 65,87%, queue length of 61,28%, and degree of saturation 43,82%.

Romadhona (2018) conducted research on the use of VISSIM PTV Software for comparison of road section performance before and after the implementation of one – way system and concluded that Prawirotaman Road section before the one – way system change, the degree of saturation was 0,46 and the condition after the one – way system change was 0,06, in other words it increased by 87,45%. The level of service of Prawirotaman Road section before and after the implementation of one direction has not changed, which remains at the F value even though the speed increased by 15,72% which was originally 23,87 kmph to 27,62 kmph. The impact due to the implementation of a one – way system on the Prawitotaman Road section on the surroundings road sections is not too significant, the speed on the Sisingamangaraja Road Section increased by 2,39% with an increase in the degree of saturation by 12,18%, the speed on the Menukan Road section increased by 14,47% with a decrease in the degree of saturation by -8,12% and the speed of the Parangtritis Road section increased by 11,02% with a decrease in the degree of saturation by -0,06%. All three road sections remain at level of service F.

Setiawan et al., (2021) did a performance of signalized crossings under current circumstances and planned for the ensuing ten years is to be examined, assessed, and modelled in this study. Peak traffic volume, geometric conditions, environmental conditions, cycle durations, queue and speed data, and population density statistics for Semarang City are among the many data that are required. Peak traffic volume (Qtot) at 16.00–18.00 intervals, or 3555 pcu/hour, queue length (Qlen) of 91.81 meters, delay (D) of 105.10 seconds/pcu, and degree of saturation (DS) of 1,071 in the north arm, 0.530 in the east arm, 0.880 in the south arm, and 0.637 in the west arm, are the results of analysis of the current conditions. The current state is classified as "F" (extremely bad) service level.

2.6 Fuel Oil Consumption

Hadis and Sumarno (2019) stated that fuel is a fairly finite natural resource, so as energy demand rises, particularly in the transportation sector, fuel availability will also rise. The number of vehicles on the road has led to an increase in fuel consumption for motor vehicles. Fuel is wasted when delays and long lines result in idle time brought on by a closed railroad crossing. The purpose of this study is to examine the relationship between fuel consumption brought on by closed railway crossings in Surakarta City and delays and long lines. analysis of long lines and delays based on survey results at each crossing. Analysis of fuel usage based on delay time using the passenger car unit-converted LAPI-ITB formula. the use of multiple linear regression analysis to examine the association between fuel use and railway crossing closures such as delays.

Romadhona and Suhanda (2019) performed a study was to evaluate the relationship between fuel consumption and the intersection performance of the current conditions. To obtain traffic flow, delay, and wait duration, primary data was collected. Bina Marga and VISSIM software were utilized in the performance analysis, together with the Lamsal (Indian-ATIS) fuel consumption equation. To ascertain the correlation between fuel usage and intersection performance, a basic linear regression analysis was conducted. Its V/C exceeded 0.85 and its delay exceeded 25 seconds as a result. A one-hour delay in the current conditions resulted in an average total fuel usage of 286,668 litters wasted, costing Rp 2,150,012.

Fadhil (2019) did a research on signalized intersection and the impact of delay time and queue length on fuel oil consumption. The UPN Yogyakarta intersection was the site of the research. The purpose of this study is to evaluate the funcionally of UPN signalized intersections and to establish a correlation between fuel oil queue length and delay. According to the analysis's findings, the UPN Yogyakarta signalized intersection is oversaturated because the saturation level there is larger than 0,85. The UPN intersection service level is at level F and its average delay value is greater than 60 seconds/pcu. Additionally, 444,653 liters of fuel oil were lost altogether at the UPN intersection as a result of the length of the line and the duration of the delay.

2.7 Comparison with Former Research

From the results of previous research that has been researched will be presented on table 2.1 Comparison of the author's research with the following previous studies.

Table 2. 1 Comparison of Author Research with Former Research

No.	Author	Title	Location	Method	Result	Current Study
1.	Anita Susanti (2021)	Studi Perencanaan Simpang Koordinasi Jl. Dr. Soetomo – Jl. RA. Kartini – Jl. Pandegiling di Kota Surabaya	Simpang Jl. Dr. Soetomo – Jl. RA. Kartini – Jl. Pandegiling di Kota Surabaya	IHCM 1997	The three intersections along the internode of highway have degree of saturation (DS) 1,104, queue length (QL) 832 meters, and delay 325 seconds. After the coordination is held, the average value of intersection for DS decreased to 0,857, QL decreased to 353 meters and delay decreased to 75 seconds.	The study of evaluating the performance of Monjali Intersection due to an alley that is being studied, has some similarities but also differences. Which is the location of the study, the geometry of the road, for this case an alley exists as an
2.	Fitria Purnayanti Cahyaningrum (2014)	Koordinasi Simpang Bersinyal Pada Simpang Kentungan – Simpang Monjali Yogyakarta	Simpang Kentungan – Simpang Monjali Yogyakarta	IHCM 1997	The results of the analysis are known to the two intersections not yet coordinated. From planning obtained 130 seconds with an offset time of 70,93 seconds for both directions. Coordination diagram gained 37 seconds of bandwidth for direction from east and 32 seconds for direction west.	
3.	Prayoga, Sulistyorini, Hadi (2017)	Analisis Koordinasi Sinyal Antar Simpang Pada Ruas Jalan Z. A. Pagar Alam	Persimpangan Jl. Z. A. Pagar Alam - Jalan Pramuka (Section I) and Simpang Jl. Z. A. Pagar Alam – Terminal Rajabasa	IHCM 1997	According to the analysis, DS in the first section is 0.73 with the queue of 70,23 m and delay of 18729 pcu/hour. The second section with DS of 0.70, queue of 146.71 m and total delay of 38181 pcu/hour, and third section with the	

Sources: Fitria Purnayanti Cahyaningrum (2014), Prayoga, Sulistyorini, Hadi (2017), Anita Susanti (2021)

			(Section II), Simpang Jl. Z. A. Pagar Alam – Jl. Sumantri Brojonegoro (Section III)		value of DS 0.83, queue length of 82.03 m and total delay of 30125 pcu/hour.	friction for the traffic. The current peak hour data obtained from Department of Transportation for the north arm of Monjali Intersection are as follows; 06.45 – 07.45 AM with the volume of 1284 pcu/hour, 12.00 – 01.00 PM with the volume of 1155 pcu/hour, and 4.30 – 5.30 PM with the volume of 1442
4.	Suryaningsih, Hermansyah, Kurniati (2020)	Analisis Kinerja Simpang Bersinyal (Studi Kasus Jalan Hasanuddin – Jalan Kamboja, Sumbawa Besar)	Jl. Hasanuddin – Jl. Kamboja, Sumbawa Besar	IHCM 1997	From the study can be concluded that the DS intersection Jl. Hasanuddin – Jl. Kamboja for the west approach, south approach, and east approach are 0.53, 0.55, and 0.56 so those intersections have stable flow with a medium traffic volume, the speed has started to be limited by the traffic condition and medium traffic density but the traffic friction has started to affect speed because the DS value is below 0.75.	
5.	Taufikkurrahman (2013)	Analisis Kinerja Simpang Bersinyal	Persimpangan Jl. Sudirman – Jl. Urip Sumohardjo Malang	IHCM 1997	Based on the study, the performance of the existing intersection: the longest queue in the southern approach is 361 m, largest capacity in the north approach is 686, biggest value of DS in the east approach with the value of 4,4 and delay with the value of 3102 second/pcu, so it resulted in the Level of Service (LOS) F that has forced traffic condition, relatively low speed traffic.	

Sources: Taufikkurrahman (2013), Suryaningsih, Hermansyah, Kurniati (2020)

6.	Pratama (2012)	Analisis Tundaan Pada Simpang Bersinyal	Simpang Dago, Bandung.	IHCM 1997	A potential solution to the Dago intersection's performance limitations is to reduce the side barriers, increase the width of the short, and reset the signal time on the short north.	pcu/hour. The degree of saturation (DS) for each peak hour in the north arm are 0,54, 0,5, and 0,66 so it is resulting in the DS max for 0,66. Based on the previous evaluation study, the fuel consumption has not been calculated so in this chance of research, the fuel energy consumption will also be
7.	Novianka, Hidayati, Supriyadi, Junaidi (2020)	Kajian Tundaan Lalu Lintas Pada Simpang Bersinyal	Simpang Jl. Brigjen Sudiarto – Jl. Majapahit – Jl. Fatmawati Kota Semarang	IHCM 1997	The volume of traffic (Q) at each intersection is almost close to the value of capacity (C), where this shows that the intersection of Brigjen Sudiarto street – Majapahit street - Fatmawati street is overcrowded. There is only one approach that meets the requirements and the approach does not experience congestion, namely the approach of Brigjen street meanwhile the rest have DS value above 0.85.	
8.	Yunus, et. al (2020)	The Analysis of Traffic Delay and Queue due to the Shunting Activities of Pertamina Trains of Tegal City	Jl. Abimanyu, Jl. Semeru, Jl. Menteri Supeno I, Jl. Menteri Supeno II	IHCM 1997	The worst traffic jams and lineups caused by Pertamina train shunting occurred on Jl. Abimanyu, where the average total queue was 70.5 pcu and the average queue length was 126 m. with 286 pcu of vehicles stopped every hour, the amount of delay that actually happened on the road was 582 seconds/pcu or 9.69 minutes/pcu.	

Sources: Pratama (2012), Novianka, Hidayati, Supriyadi, Junaidi (2020), Yunus, et. al (2020)

						considered in the research as well as the solution from the economy point of view.
9.	Fadhil (2019)	Analisis Simpang Bersinyal dan Hubungan Panjang Antrian dan Waktu Tundaan terhadap Konsumsi Bahan Bakar Minyak	Simpang Bersinyal UPN Yogyakarta	IHCM 1997, LAPI-ITB	The four arms intersection has four times phases. Direct surveys are used to obtain data in the field. According to the analysis's findings, the UPN signalized intersection's degree of saturation (DS) value is larger than 0.85, which means that the intersection is already oversaturated. Since the average delay value is higher than 60, the service level is also considered to be F. A total of 444,653 liters of fuel oil were lost, costing Rp. 3,173,725 in total, as a result of the UPN intersection's long line-up and delay.	
10.	Yogama,Yudha Dwi (2015)	Hubungan Antara Tundaan dan Panjang Antrian dengan Konsumsi Bahan Bakar Minyak pada Pendekat Simpang di Surakarta	Simpang Surakarta	IHCM 1997, LAPI-ITB	The study and discussion lead to the conclusion that the average approach intersection delay, queue length, and fuel consumption in Surakarta is 14,50 seconds/pcu; 43,17 meters; and 0,091 liters/pcu. The dependent variable, fuel usage	

Sources: Yogama,Yudha Dwi (2015), Fadhil (2019)

					in liters per pcu, is influenced by the independent variables, delay in seconds per pcu and queue length in meters. If the length of the line and the delay both grow, then the amount of fuel consumed will likewise rise.	
11.	Putra (2016)	Analisis Kinerja Simpang Bersinyal Terhadap Konsumsi Bahan Bakar di Kota Surakarta	Kota Surakarta	IHCM 1997, LAPI-ITB	It was found that the degree of saturation value was bigger than 0,85 and the lost fuel consumption amount with details at point intersection was 0,12 liter/pcu with a total delay of 307,80 sec/pcu. For Ngemplak intersection is 0,13 liter/pcu with a total delay of 330,98 sec/pcu, and for the Gemblegan intersection it has the value of 269,20 sec/pcu. According to the analysis research, it shows that delay has a major influence on fuel consumption at signalized intersections, it means that the higher the delay value is, the greater the lost fuel oil.	

Sources: Putra (2016)

CHAPTER 3
THEORETICAL BACKGROUND

3.1 Traffic Characteristics

Basic characteristics of traffic flow according to Khisty and Lall (2005) classified into 2 categories, namely:

1. Macroscopic

   Macroscopically, there are 3 basic traffic characteristics, namely:

   a. Volume and flow

      Volume is the actual number of vehicles that are observed or estimated to pass a point over a certain period of time, usually expressed in unit of vehicle/hour. meanwhile flow is the number of vehicles that pass a point in less than an hour but is equivalent to an hourly average rate, usually expressed in unit of vehicles/time or pcu/time (hour).

   b. Speed

      Speed is the rate of movement of a vehicle calculated in distanced per unit time, usually expressed in unit of km/hour.

   c. Density

      Density is the number of vehicles occupying a certain length of road or lane in vehicles per km or vehicles per km per lane, usually expressed in unit of vehicles/km.

The three elements of the basic traffic characteristics above are elements that form the flow of traffic flow which will get the following relationship pattern:

   a. Flow with Density

      The maximum flow occurs when the density reaches its maximum point (roadway capacity has been reached). After reaching this point the flow will decrease even though the density increases until congestion occurs.

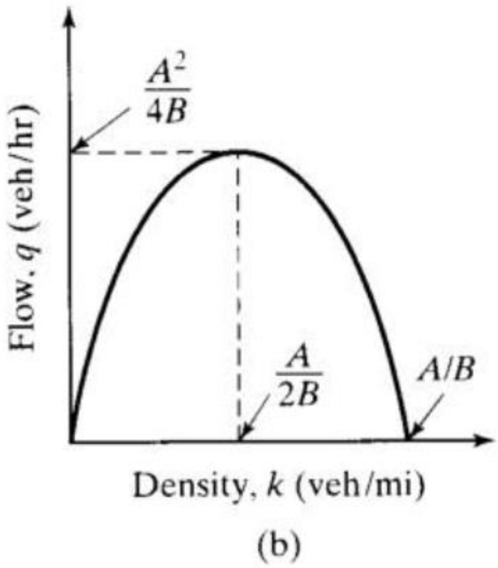

Figure 3.1 Relation of Flow and Density

(Source: Khisty and Lall, 2005)

b. Speed and Density

Speed will decrease if the density increases. Free flow velocity will occur if the density is equal to zero, and when the speed is equal to zero then there will be congestion (density jam).

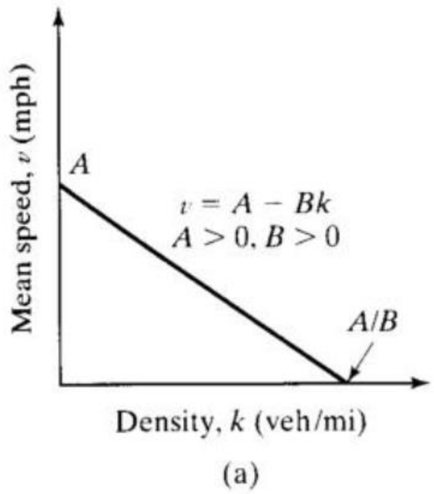

Figure 3. 2 Relation of Speed and Density

(Source: Khisty and Lall, 2005)

c. Flow and Speed

The fundamental relationship between flow and speed is that as traffic flow increases, the average spatial velocity decreases until a critical density is reached. After the critical density is reached, the velocity of space and volume will decrease.

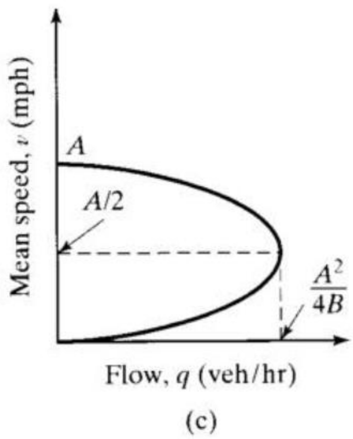

Figure 3.3 Relation between Flow and Speed

(Source: Khisty and Lall, 2005)

Relation between those three characteristics could be seen in the figure below.

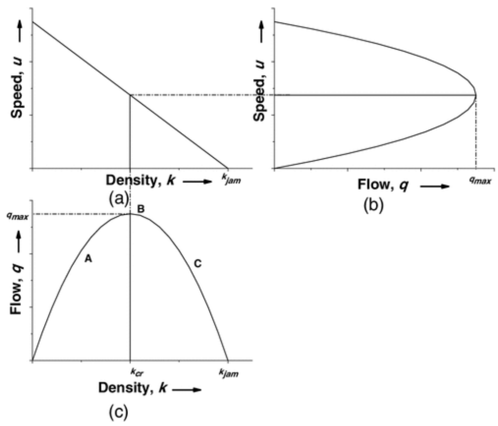

Figure 3.4 Relation of Flow, Speed and Density

(Source: Khisty and Lall, 2005)

2. Microscopic

Microscopically, the fundamental of traffic characteristics are namely:

a. Headway

Headway is time interval between two vehicles when passing through an observation point on the highway sequentially in traffic flow, the unit is seconds.

b. Spacing

Spacing is the distance between two consecutive vehicles in the traffic flow, measured from the front bumper of one vehicle to the bumper of the vehicle behind it, usually expressed in meters. Spacing data was obtained by surveying photographs from the air.

3.1.1 Types of Movement

There are several types of movement that can cause traffic conflict points at an intersection. Conflicts are caused by the need for road space at the same time from other road users. According to Harianto (2004) the movement of conflicts that occur at intersections, as follows:

1. Diverging Movement

   Diverging movement is the event of separation of vehicles from the same stream to another lane. According to Bina Marga (1992) diverging is the spread of vehicle flows from one traffic lane to several directions.

2. Merging Movement

   Merging is the event of merging vehicles from one lane to the same lane. According to Bina Marga (1992) merging is the combining movement of vehicle flows from several traffic lanes in one direction.

3. Crossing Movement

   This movement is an intersection event between the flow of vehicles from one lane to another at an intersection where such circumstances will cause a point of conflict at the intersection. According to Bina Marga (1992) crossing is the intersecting of two traffic lanes perpendicularly.

4. Weaving Movement

   This movement is a confluence of two or more traffic streams that run in the same direction along a lane on the highway without the aid of traffic signs. This movement often occurs in a vehicle that moves from one lane to another, for example when the vehicle enters a highway from the entrance, then moves to another lane to take the exit from the highway. This situation will also cause conflict points at intersection.

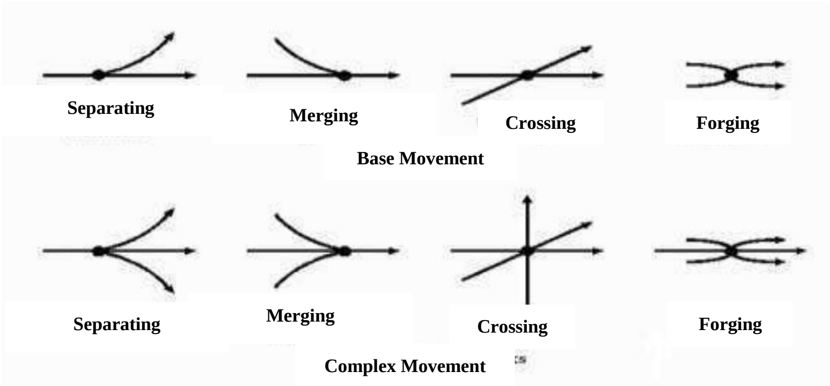

Figure 3.5 Maneuver of Vehicles

(Source: Tamin, 2008, in Nuryadin, 2012)

3.1.2 Conflict Points of Road Intersection

According to Hobbs (1995), traffic flow from various directions will meet at an intersection point, this condition causes conflicts between drivers from different directions. Conflicts between drivers are divided into two points of conflict which include several things as follow:

1. Primary conflict, is a conflict that happens between crossing traffic flow.
2. Secondary conflict, is a conflict that happens between right traffic flow with traffic flow from other direction and traffic flow from left with pedestrians.

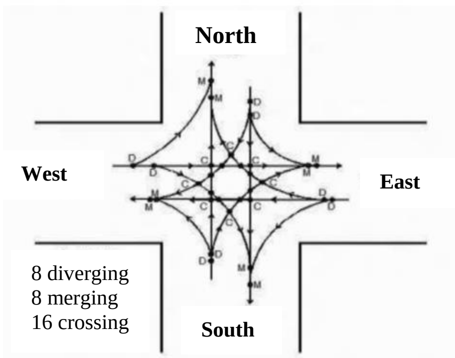

Figure 3.6 Number and Types of Conflict Points at 4-Legged-Intersection

(Source: Tamin, 2008, in Nuryadin, 2012)

3.1.3 Traffic Signal

Traffic signals are all traffic control equipment that uses electricity, road signs, and markings to direct or warn motorized vehicle drivers, cyclist or pedestrians, Oglesby and Hick (1982).

1. Function of Traffic Signals

   According to Oglesby and Hick (1982) every traffic signal installation works as follows:

   a. To get regular traffic movement.
   b. Increase traffic capacity at the crossroads.
   c. Reducing the frequency of accidents.
   d. Regulates the use of traffic lanes.
   e. As meeting controller at the entrance to the freeway barriers.
   f. Coordinate traffic under conditions of good signal spacing, so that the flow of traffic flows continuously at a certain speed.
   g. Breaking high traffic flow to make it possible for crossing other vehicles or pedestrians.
   h. Decide the flow of traffic for emergency vehicles (ambulance) or on new bridge.

2. Physical Characteristics of Traffic Signal

   a. Modern electrically controlled signals.
   b. Modern signals are equipped with regulatory signals for pedestrians.
   c. Each unit consists of red, amber, and green colored lights separated with diameter of 0,203-0,305 m.
   d. Traffic lights are installed outside the road boundaries or suspended above road junctions. The height of the traffifc lights is installed outside 2,438-4,572 m.
   e. Traffic lights are required to use poles with arms or suspended by cable and spaced 12,912-36,576 m stop line.
   f. The traffic light is angled no more than 20° which is formed by the driver's normal line of sight.

3. Settings of Traffic Signals
   a. Fixed time settings

      Generally chosen when the intersection is part of a coordinated traffic signal system.
   b. Semi actuation signal settings

      Generally selected when the intersection is isolated and consists of a minor road or pedestrian crossing and intersects a major arterial road (detectors are only installed on minor roads or pedestrian crossings).
   c. Full actuation signal settings

      The most efficient arrangement for isolated intersections between streets with the same or nearly the same traffic interests and requirements.
4. Traffic Light Operation Parameters

   The parameter commonly used in planning the traffic light includes:
   a. Signal Phase

      Phases are chosen based on the number of main conflicts, namely conflicts that occur in a fairly large volume of vehicles. If the signal phase is not known, then a two-phase setip should use the base case.
   b. Intergreen Period

      The intergreen time is the time required ro change berween the green time of an initial phase to the next phase, which is the period of yellow (amber) and all red between two successive signal phases. The minimume time for intergreen is 4-6 seconds.

      The intergreen period is also the sum of all the yellow (amber) time, which is generally 3 seconds, and the all-red period, which is generally 2 seconds. Clearance time is all red time used to clear the intersection area from vehicles that are stuck while crossing the intersection.

Table 3.1 Intergreen Normal Time Value

Junction Size	Average road width (m)	Lost time value (sec/phase)
Small	6-9	4

Continuation of Table 3.1 Intergreen Normal Time Value

Medium	10-14	5
Big	≥ 15	≥ 6

(Source: Bina Marga, 1997)

3.2 Signalized Intersections

3.2.1 Traffic Flow (Q)

Traffic flow (Q) for each movement of light vehicles, heavy vehicles, and motorcycle (QLV, QHV, and QMC) are converted from hourly vehicles to hourly passenger car unit (PCU) using passenger vehicle equivalents (PCE) for each protected and resisted vehicle. The passenger vehicle equivalent figures are shown in Table 3.2 below.

Table 3.2 Passenger Car Equivalent Number

Vehicle type	PCE for approachment type	
	Protected	Resisted
Light vehicle (LV)	1	1
Heavy vehicle (HV)	1,3	1,3
Motorcycle (MC)	0,2	0,4

(Source: Bina Marga, 1997)

To calculate traffic flow, formula 3.1 below can be used.

$$\mathbf{Q = QLV + QHV \times pceHV + QMC \times pceMC} \quad (3.1)$$

with:

Q = traffic flow (pcu/hour),

QLV = light vehicle traffic flow (vehicle/hour),

QHV = heavy vehicle traffic flow (vehicle/hour),

QMC = motorcycle traffic flow (vehicle/hour),

pceHV = pce for heavy vehicle, and

pceMC = pce for motorcycle.

3.2.2 Base Saturated Flow (S_0)

Base saturated flow is the maximum traffic flow that can pass through intersection with traffic lights. According to Indonesian Highway Capacity Manual (IHCM, 1997), the base saturated traffic flow can be calculated using equation 3.2.

$S_0 = 850 \times We^{0.95}$ (3.2)

With:

S_0 = base saturated traffic flow (pcu/hour), and

We = effective width (meter).

From several studies in several cities in Indonesia from Munawar et. al (2003), the value of saturated current in the fiels is greater than that value, which is about 1,3 so that the empirircal formula from IHCM 1997is recommended to be corrected as equation 3.3 below.

$S_0 = 780 \times We$ (3.3)

Basic saturated flow has two types: type approach O and type approach P, for P approach type, how to use it is using equation 3.2 or using the graphic on figure 3.7 as shown below.

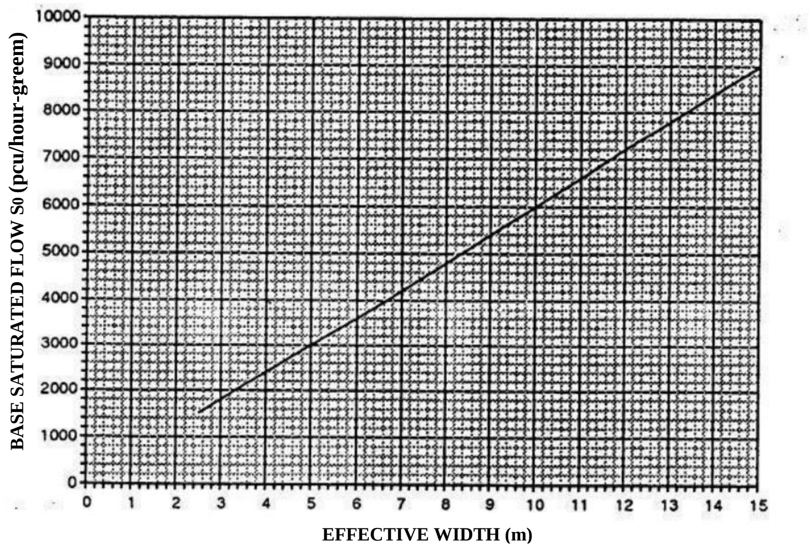

Figure 3.7 Base Saturated Flow for P Type Approach

(Source: Bina Marga, 1997)

3.2.3 Capacity of Intersection

Capacity is the ability of an intersection to accommodate traffic flow, the maximum per unit time is expressed in pcu/green time. The capacity at an intersection is calculated on each approach or group of lanes in an approach. The intersection capacity is calculated by the following equation 3.4.

$$\mathbf{C} = \mathbf{S} \times \frac{\mathbf{g}}{\mathbf{c}} \qquad (3.4)$$

With:

C = capacity (pcu/green time),

S = saturated flow (pcu/green time),

g = green time (second), and

c = cycle time (second).

3.2.4 Saturated Flow

Saturated flow based on Indonesian Road Capacity Manual (1997) is defined as the average departure of the queue in an intersection approach during a green signal. This time period is measured in pcu per green hour (pcu/green hour). Equation 3.5 can be used to obtain the saturated current for signalized intersections.

$$\mathbf{S} = \mathbf{So} \times \mathbf{Fcs} \times \mathbf{Fsf} \times \mathbf{Fg} \times \mathbf{Fp} \times \mathbf{Flt} \times \mathbf{Frt} \qquad (3.5)$$

With:

S = saturated flow (pcu/effective green time),

So = base saturated flow (pcu/effective green time),

Fcs = city size correction number for saturated flow (population number),

Fsf = side friction correction number for saturated flow,

Fg = gradient correction number for saturated flow,

Fp = parking area correction number for saturated flow,

Flt = left turn correction number for saturated flow, and

Frt = right turn correction number for saturated flow.

In determining the correction number of city size (F_{CS}), it could be seen in Table 3.3 below.

Table 3.3 City Size Correction Value

City Population (million)	Factor adjustment for city size (F_{CS})
>3,0	1,05
1,0 – 3,0	1,00

Continuation of Table 3.3 City Size Correction Value

0,5 – 1,0	0,94
0,1 – 0,5	0,83
<0,1	0,82

(Source: Bina Marga, 1997)

To determine the correction factor gradient (F_G) can be seen in figure 3.8 below.

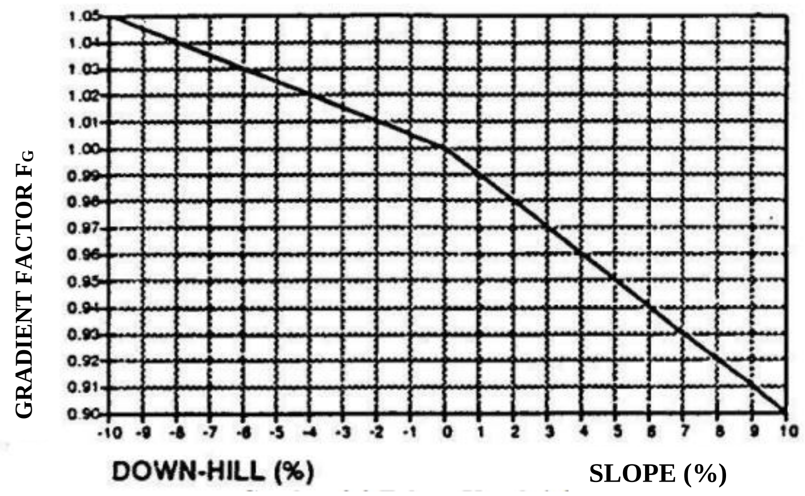

Figure 3.8 Correction Factor for Gradient (F_G)

(Source: Bina Marga, 1997)

Meanwhile for parking correction factor (F_P), is a distance from stop line to vehicle that is first parked and the width of approach, could be determined from the formula below and also using figure 3.9, how to use the graphic is by determining the width of approach (W_A) then determine line stop for parking area and then drag the line as the width of approach and drag to the left to obtain the value (F_P) that can be seen in equation 3.6.

$$F_P = (L_P / 3 – (W_A – 2) \times (L_P / 3 – g) / W_A) g \quad (3.6)$$

With:

L_P = distance between stop line and the first vehicle parked,

W_A = width approach (m), and

g = green time in the approach (second).

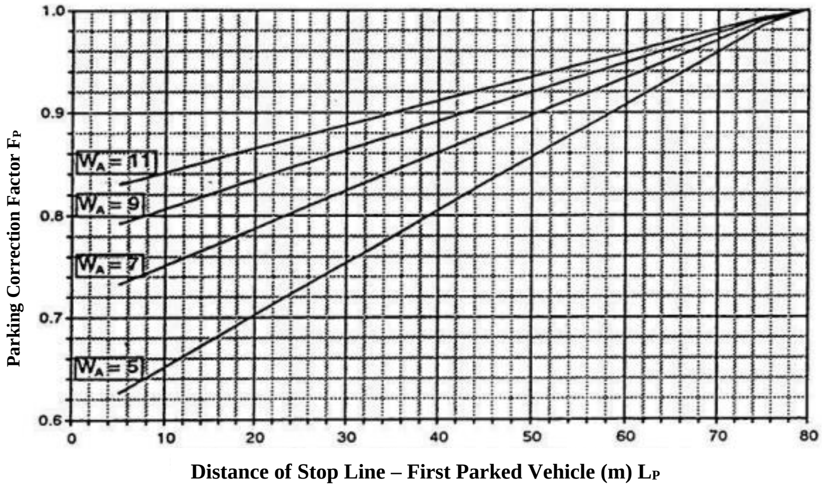

Figure 3.9 Correction Factor for Parking Area (F_P)

(Source: Bina Marga, 1997)

The determination of the correction factor for the next basic saturated current value is only for the P type, which is as follows.

1. Right turn correction factor (F_{RT}), determined as a comparison fucntion vehicles that turn right (P_{RT}). This factor is only for the approachment type P, two-way roads without median, vehicles turning right from protected departing flow (type P approach) has a tendency to cut lines middle of the road before crossing the sto line when completing the turn, this leads to an increase in the ratio of high right turns on saturated flow, as can be seen in Figure 3.10.

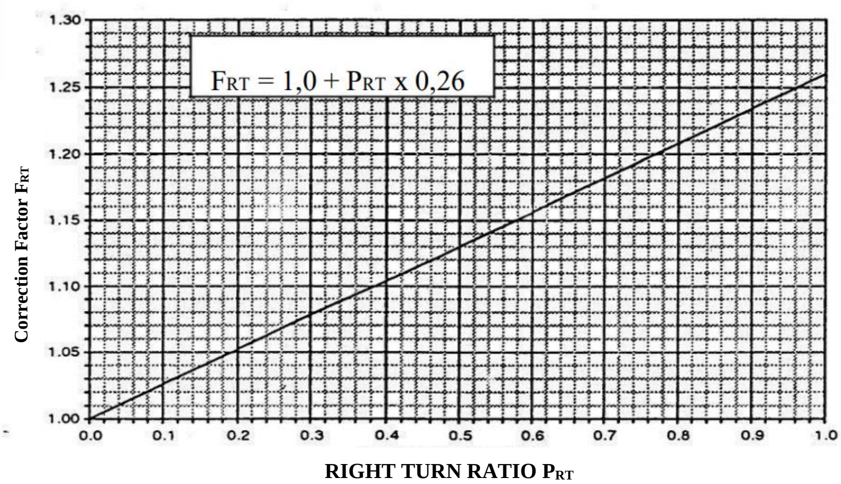

Figure 3.10 Right Turn Correction Factor (F_{RT})

(Source: Bina Marga, 1997)

2. Left turn correction factor (F_{LT}), determined as a function of turn comparion left (P_{LT}). This factor is only for approach type without effective width LTOR determined by the width of the entrance. On protected approaches without the provision of a direct left turn, left-turning vehicles tend to slow down and reduce the saturation flow of the approachment.

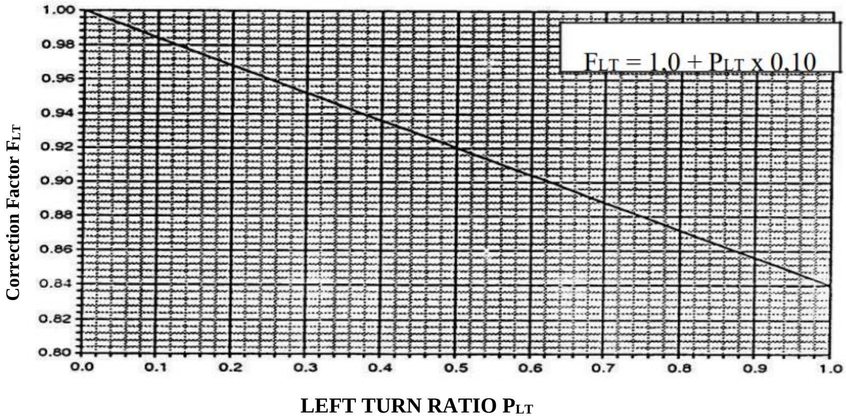

Figure 3.11 Left Turn Correction Factor (F_{LT})

(Source: Bina Marga, 1997)

3.2.5 Flow Ratio to Saturated Flow

The calculation of the ratio of flow (Q) to saturated flow (S) for each approach can be formulated with the equation 3.7.

$$FR = Q/S \tag{3.7}$$

With:

FR = Flow Ratio,

Q = Flow or Volume (pcu/hour), and

S = Saturated flow (pcu/effective green time).

Critical flow comparison (FR_{CRIT}) is the highest comparison flow value in each phase. If the ratio values of critical flow for each phase are added together, the following ratio of intersection flow will be obtained in equation 3.8.

$$IFR = \sum(FR_{CRIT}) \tag{3.8}$$

Phase ratio (PR) for each phase is a function of comparison between FR_{CRIT} and IFR, can be calculated using equation 3.9.

$$PR = FR_{CRIT}/IFR \tag{3.9}$$

3.2.6 Degree of Saturation (DS)

Degree of saturation (DS) is defined as ratio of volume (Q) towards capacity (C). Degree of saturation can be obtained using the equation 3.10 as written below.

$$DS = Q/C \tag{3.10}$$

With:

DS = degree of saturation,

Q = volume or traffic flow (pcu/hour), and

C = capacity (pcu/hour).

3.2.6 Number of Queue

Number of queues is the number of vehicles at each intersection lane at red light (Department of Public Work, 1997). Here is the equation 3.11 to determine the average queue length based on IHCM 1997.

For degree of saturation (DS) > 0,5:

$$\mathbf{NQ1 = 0,25 \times C \times \left[(DS - 1) + \sqrt{(DS-1)^2 + \frac{8(DS-0,5)}{C}}\right]} \tag{3.11}$$

With:

NQ1 = number of pcu left from the green phase before,

DS = degree of saturation, and

C = capacity (pcu/hour).

For DS < 0,5; NQ1 = 0

Length of queue during red phase (NQ2)

$$\mathbf{NQ2 = c \times \frac{1-GR}{1-GR \times DS} \times \frac{Qentry}{3600}} \tag{3.12}$$

With:

NQ2 = number of pcu that comes when red phase occurs,

GR = green ratio, and

c = cycle time (second).

Qentry = traffic flow enters out of LTOR (pcu/hour)

Number of queues becomes:

$$NQ = NQ1+NQ2 \tag{3.13}$$

NQ = total number of queues,

NQ1 = number of pcu left from the green phase before, and

NQ2 = number of pcu that comes when red phase occurs.

The queue length (QL) is obtained grom multiplication (Nqmax) with the average used per pcu (20 m^2) and division by the entry width (Wentry). NQmax is obtained by adjusting the value of NQ in terms of the desired chance of overloading POL (%) using a graph as shown in figure 3.12 for planning and design with recommended POL ≤ 5%, meanwhile for operational it is recommended POL = 5-10%. Using the equation 3.14 below.

$$\mathbf{QL = NQmax} \times \frac{\mathbf{20}}{\mathbf{Wentry}} \tag{3.14}$$

With:

QL = queue length,

Nqmax= maximum number of queues, and

Wentry= width of entrance.

Below is the graphic calculation of number of queue (Nqmax) in pcu:

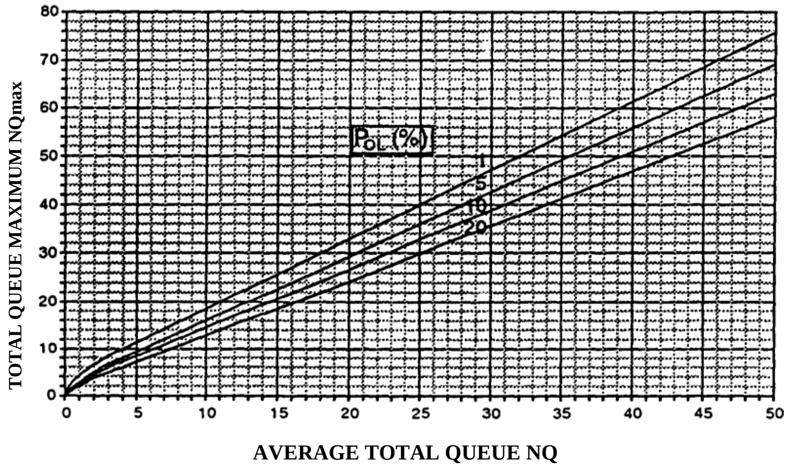

Figure 3.12 Number of Queue Calculation Graphic (Nqmax) in pcu

(Source: Bina Marga, 1997)

3.2.6 Number of Stops

Number of stop (NS) in each approach is total average number of stopped vehicles per pcu, it is counted as repeated stops before passing the stop line of intersection. Number of stops equation can be seen in equation 3.15.

$$\mathbf{NS = 0,9 \times \frac{NQ}{Q \times c} \times 3600} \quad (3.15)$$

With:

NS = number of stops,

NQ = total of queue,

Q = traffic flow (pcu/bour), and

c = cycle time (second).

3.2.7 Delay

Delay, there are two kinds of delay in an intersection: geometry delay (DG) and traffic delay (DT). So, delay can be calculated using equation 3.16, 3.17, 3.18 as follows.

D = DT+DG (3.16)

With:

$$\mathbf{DT = c \times 0,5 \times (1 - GR)^2 \times (1 - GR \times DS) + NQ1 + 3600 \times C} \quad (3.17)$$

$$\mathbf{DG = (1 - Psv) \times Pt \times 6 + (Psv \times 4)} \quad (3.18)$$

With:

DT = traffic delay (sec/pcu),

DG = geometry delay (sec/pcu),

c = adjusted cycle time (sec),

GR = green ratio (g/c),

DS = degree of saturation,

NQ1 = number of pcu left from the green phase before,

C = capacity (pcu/hour),

Pt = turning vehicle ratio in an approach, and

Psv = stoped vehicle ratio in an approach.

3.3 Side Friction

Side friction according to the Indonesian Highway Capacity Manual (1997) are the impact on traffic behavior due to activities on the side of the road segment as follows.

1. A walking pedestrian or the one who crosses along the road segment,
2. Stopped and parked vehicles,
3. Motorized vehicles entering and exiting from/to land beside roads and side roads,
4. The flow of slow-moving vehicles, and
5. Commercial activities that use the shoulder of the road.

To simplify its role in the calculation procedure, the level of side resistance has been grouped into five classes from very low to very high as a function of the frequency of side resistance along the observed road segment. The classes of side barriers for urban roads can be seen in Table 3.4.

Table 3.4 Side Friction Class

Side Friction Class (SFC)	Code	Number of weighted events per 200 m per hour (two sides)	Special condition
Very Low	VL	<100	Residential areas; a road with a side road.
Low	L	100-299	Residential areas; some public transportation etc.
Medium	M	300-499	Industrial area, a few shops on the side of the road.
High	H	500-899	Commercial area, high roadside activity.
Very High	VH	>900	Commercial areas with market activity beside the road.

(Source: Bina Marga, 1997)

If detailed data on side frictions are not available, the class of side frictions can be specified as follows:

1. Check the description of ‘special conditions’ from Table 3.4 and choose the most appropriate one for the state of the analyzed road segment.

2. Observe the photo in Figure 3.13 until Figure 3.17 which shows the special average visual impression of each class of side friction, and choose the one that best suits the contitions of actual averages at locations for the observed period.
3. Select a side friction class based on considerations from the combined steps 1 and 2 in the above.

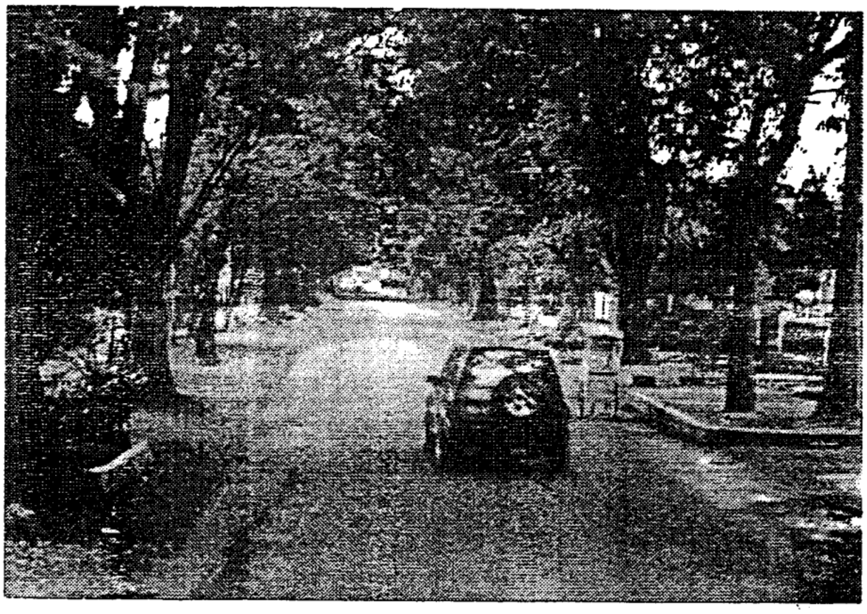

Figure 3.13 Very Low Side Friction on Urban Roads

(Source: Bina Marga, 1997)

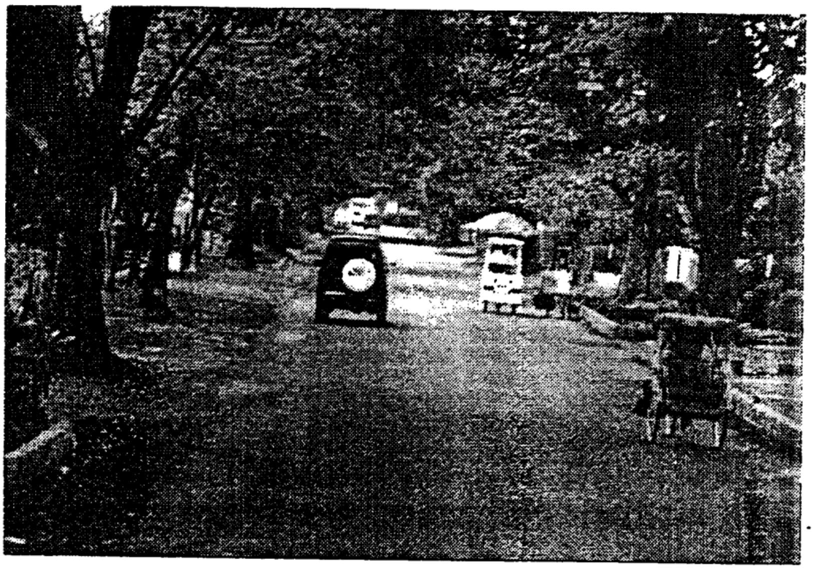

Figure 3.14 Low Side Friction on Urban Roads

(Source: Bina Marga, 1997)

Figure 3.15 Medium Side Friction on Urban Roads

(Source: Bina Marga, 1997)

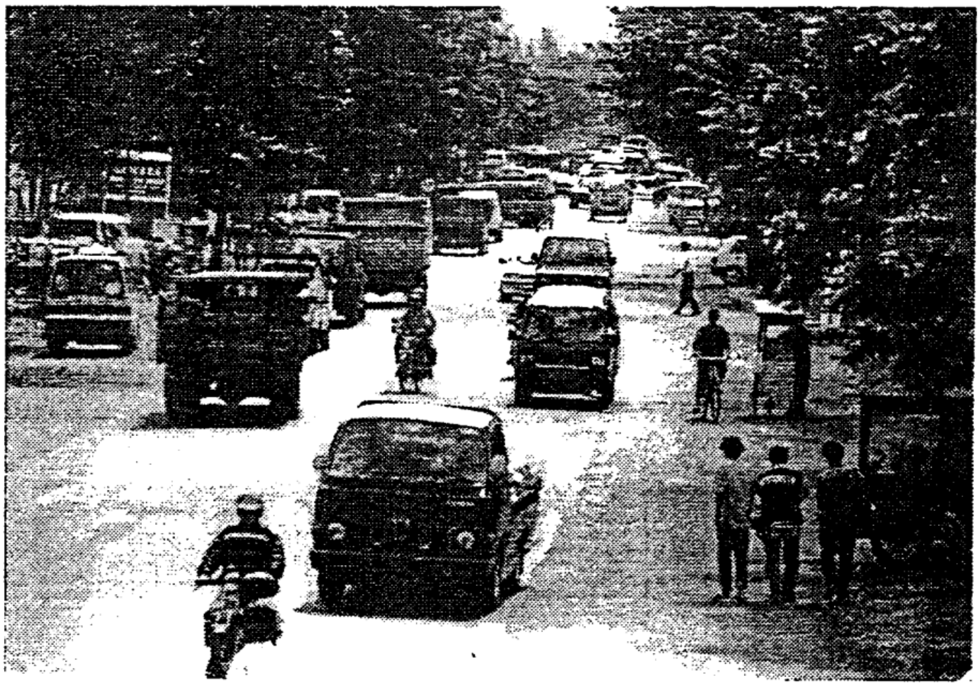

Figure 3.16 High Side Friction on Urban Roads

(Source: Bina Marga, 1997)

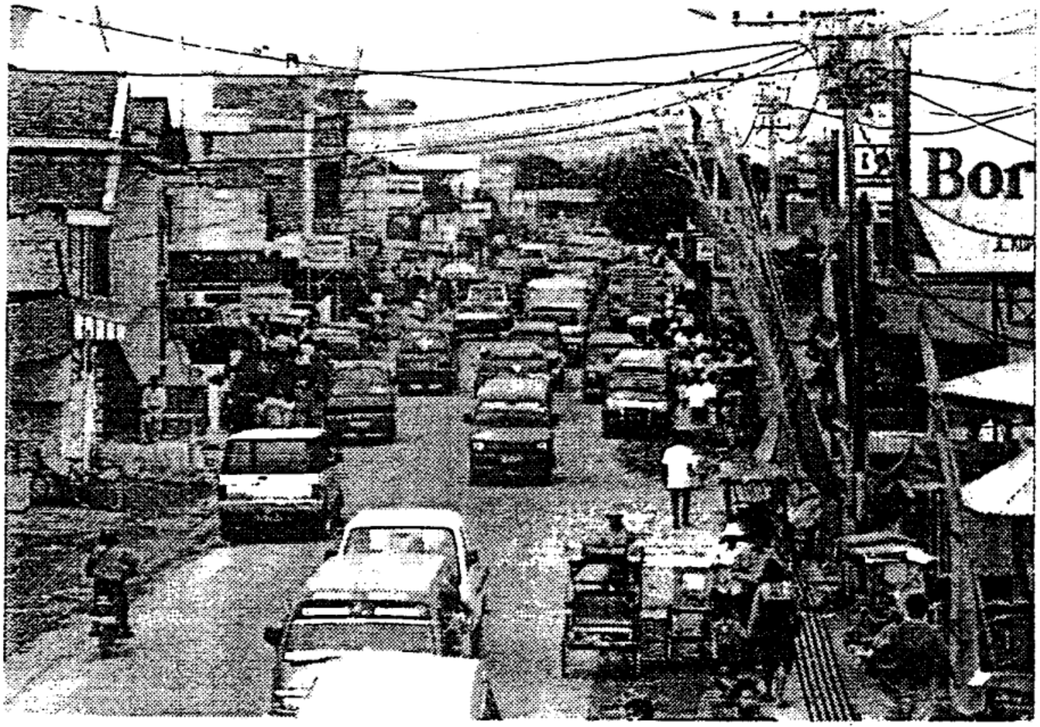

Figure 3.17 Very High Side Friction on Urban Roads

(Source: Bina Marga, 1997)

3.4 Determination of Cycle Time and Green Time

3.4.1 Cycle Time Before Adjustment (C_{UA})

Cycle time for phase, can be calcuared using equation or in Figure 3.18. Cycle time as a result of this calculation is an optimum cycle time, that will be resulting small delay, using equation 3.19

$$\mathbf{C_{UA}} = \frac{\mathbf{1{,}5 \times LTI + 5}}{\mathbf{(1 - IFR)}} \tag{3.19}$$

With:

C_{UA} = signal cycle time (second),

LTI = total of lost green time per cycle (second), and

IFR = comparison flow intersection $\sum FR_{CRIT.}$

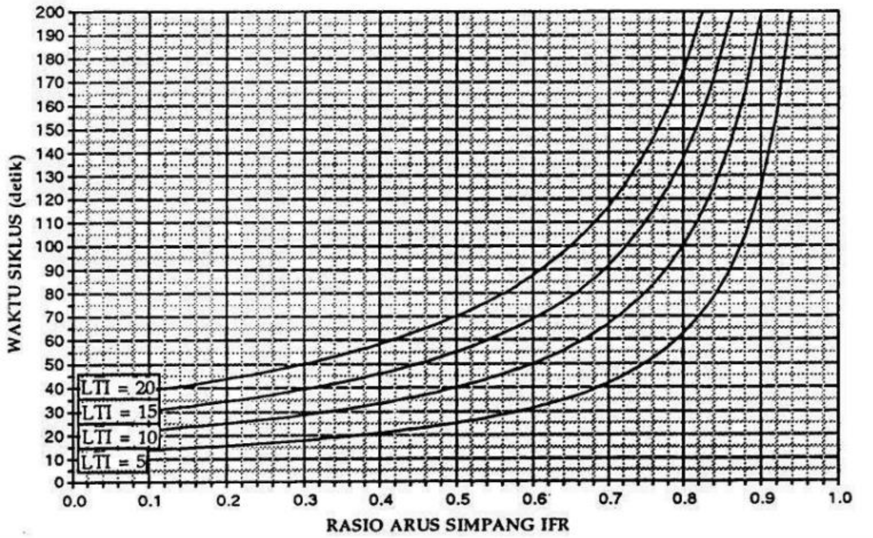

Figure 3.18 Determination of Cycle Time

(Source: Bina Marga, 1997)

This outcome will be more effective if the evaluated planned signal alternative yields a low value for (IFR = LT/c). Using Figure 3.18, the cycle time can be calculated by calculating the IFR ratio and drawing a line up in accordance with the green time loss. To obtain the cycle time, drag the line to the left for each value. The result of cycle time is supposed to be as the limit that is suggested by IHCM 1997, as consideration of traffic engineering that is explained in table 3.5.

Table 3.5 Suggested Cycle Time

Control Type	Decent Cycle Time (second)
2 phases	40 – 80
3 phases	50 – 100
4 phases	80 – 130

(Source: Bina Marga, 1997)

Lower values are used for intersections with a road width of < 10m, higher value for bigger roads. Cycle times exceeding the recommended value of more than 130 seconds should be avoided except in very special cases (very large intersections) as is often leads to a loss in overall capacity. If the calculation results in a cycle time that is much higher than the recommended limit, then it indicates that the capacity of the intersection plan is insufficient.

3.4.2 Green Time (g)

Calculation of green time for each phase is explained using equation that is provided in equation 3.20 below.

$$gi = (cua - LTI) \times Pri \tag{3.20}$$

with:

gi = green time in phase – i (second),

cua = cycle time that is decided (second),

LTI = lost time per cycle, and

Pri = comparison of phase Frcrit : (Frcrit).

Green time shorter than 10 seconds should be avoided, as it can resulting in excessive redlight violations and difficulties for pedestrians when crossing the road.

3.5 Level of Service

Determination of service level aims to establish the level services on a road and/or intersection.

Level of service must fulfill these indicators:

1. ratio between volume and road capacity;
2. speed which is the above limit and below limit that is set based on area conditon.
3. travel time;
4. freedom of movement;
5. security;
6. safety;
7. order;
8. smoothness, and;
9. driver’s assesment of traffic flow conditions.

3.5.1 Intersection Level of Service

Based on ministry of Transportation (2015) number 96, it states that the service level of intersection is classified into:

1. Level of service A, with a delay condition less than 5 seconds per vehicle;

2. Level of service B, with a delay condition between more than 15 seconds up to 25 seconds per vehicle;
3. Level of service C, with a delay condition more than 15 seconds up to 25 seconds per vehicle;
4. Level of service D, with a delay condition more than 25 seconds up to 40 seconds per vehicle;
5. Level of service E, with a delay condition more than 40 seconds up to 60 seconds per vehicle;
6. Level of service F, with a delay condition more than 60 seconds per vehicle.

3.5.2 Determination Service Level on Intersection

The desired level of service on road sections in the primary road network system according to their fucntions, including:

1. primary arteries road, level of service minimum B;
2. primary collector road, level of service minimum B;
3. primary local road, level of service minimum C;
4. highway road, level of service minimum B.

The desired level of service on road sections in the secondary road network system according to their fucntions, including:

1. secondary arteries road, level of service minimum C;
2. secondary collector road, level of service minimum C;
3. secondary local road, level of service minimum D;
4. environment road, level of service minimum D.

3.6 Traffic Management

According to Law No. 22 of 2009 traffic management on Road Traffic and Transport is defined as a series of businesses and activities that include planning, procurement, installation, arrangement, and maintenance of road equipment facilities in order to realize, support and maintain security, safety, order, and smooth traffic.

According to Wells (1993), in order for roads dto function optimally and to reduce thr growing problem, traffic techniques are needed. Traffic engineering is a relatively new discipline within the field of civil engineering that includes traffic plannig, traffic design, and road development, the front of the building bordering the road, parking facilities, traffic control to be safe and comfortable and affordable for pedestrians and vehicles.

3.6.1 Purpose of Traffic Management

The objectives of traffic management are as follows.

1. Gain efficiency from overall traffic movement with a high level of accessibility (comfort measure) by balancing movement demand with existing supporting facilities.
2. Increase the level of user safety acceptable to all parties and improve the level of safety as best as possible.
3. Protect and improve the state of environmental conditions where the traffic flow is located.
4. Promoting efficient use of energy.

3.6.2 Target of Traffic Management

The target of traffic management in accordance with the purposes as written in the above are as follows.

1. Managing and simplifying traffic flow by managing different road types, speeds and users ro minimize disruption to fasten traffic flow.
2. Reducing traffic congestion levels by increasing capacity or reducing the volume of traffic on a road. Optimizing road sections by determining the function of the road and controlling activities that are not compatible with the function of the road.

3.6.3 Alternative and Scenarios of Traffic Management

In solving traffic problems based on Bina Marga (1997), engineering and traffic management are needed to improve road performance. The following are the workarounds that can be applied to intersection according to the guidelines of the Bina Marga 1997.

1. Resetting Cycle Time

Cycle time is the time of a period of a traffic light, for example when a current in the north arm start to change into green until that approach becomes green again. Cycle time is one of the easiest ways to increase the capacity of the intersection. The higher the cycle time, the higher interchange capacity, but also higher queues and delays that will happen. Meanwhile, cycle time that are too low will make the capacity low, resulting in high queues and delays as well. Then an optimum cycle time analysis is needed.

2. Adding traffic signal in the alley arm

   The use of signals with three-color lights (green, amber, red) is applied to separate trajectories of conflicting traffic movements in the dimension of time. It is absolutely necessary for traffic movements coming from intersecting roads (major conflicts). By adding new traffic signal in the alley, it will separate the delay and capacity between its road and the main north arm.

3.7 Traffic Prediction

Traffic growth is the increase or development of traffic from year to year over the life of the plan. Factors affecting its magnitude is the growth of vehicles. The growth of vehicles as a major factor in planning is part of social factors that are always changing both in number and condition and tend to experience an increase. In urban transportation network planning cannot be separated from the influence every activity of city residents will directly cause traffic movement. The growth of vehicles in Sleman regency can be seen in Table 3.6 below.

Table 3.6 Data of Motorized Vehicles in Sleman Regency 2018 – 2022

Vehicles Types	2018	2019	2020	2021	2022
Passenger Cars	11560	9924	7615	8322	9255
Bus	168	145	99	54	60
Items Cars	1601	1581	1242	1373	1458
Special Vehicle	6	8	9	19	16
Motorcycle	40740	44844	31471	32974	36985
Total	54075	56502	40436	42742	47774

(Source: Badan Pusat Statistik Sleman, 2023)

The method to predict traffic growth is to calculate traffic growth factors and subsequently the amount of future traffic flow can be calculated using equation 3.21 according to Supranto (2004).

$$Qn = Q_0 (1+i)^n \tag{3.21}$$

With:

Qn = traffic flow n years ahead (pcu/hour),

Q_0 = current traffic flow (pcu/hour),

i = factor of traffic growth (%/year), and

n = total of planning years (year).

The magnitude of traffic growth factor (i%) is obtained through analysis based on average vehicle growth.

3.8 PTV VISSIM Software

PTV VISSIM (Verkehr in Stadten Simulations Model) according to PTV-AG (2011) is a multi-modal microscopic traffic flow simulation software that can analyze the operation of private vehicles and public transportation with problems such as lane configuration, vehicle composition, traffic signals, and others. PTV VISSIM was developed by PTV (Planung Transport Verkher AG) in Karlsruhe, Germany. PTV VISSIM is used for the evaluation of various alternative steps based on transportation engineering steps and effectiveness planning. Some of the uses of PTV VISSIM in modelling are as follows.

1. Arterial Simulation
   a. Road network model
   b. Simulated intersections against all vehicle modes
   c. Analysis of queue characteristics
   d. Signal timing design
2. Public Transportation Simulation
   a. All model details for bus, BRT, Tram, LRT, and MRT
   b. Analysis of improvements in the public operation of certain transportation
   c. Test and standardize public transport signaling times according to planning priorities.

3. Pedestrians Simulation
   a. Pedestrians model in multimodal environment
   b. Planning of evacuation form building and special event
4. Motorway Simulation
   a. Active traffic management simulation and smart transportation system
   b. Test and analyzing strategy of working zone.

3.8.1 PTV VISSIM Software Calibration and Validation

Calibration in PTV VISSIM Software is a process of forming appropriate parameter values so that the model can represent traffic conditions as closely as possible. The calibration process can be carried out based on the driver’s behaviour at the observed location. The method used is trial and error by referring to prebious studies on calibration and validation using PTV VISSIM Software. Validation of PTV VISSIM Software is the process of testing the correctness of calibration by comparing survey results with simulation results.

The validation process is carried out based on the amount of traffic flow volume. The method used is to use the basic Chi-squared formula in the form of the statistical formula Geoffery E. Havers (GEH) (Gustavsson, 2007). GEH is a modified statistical formula of the T test by combining the difference berween relative and absolute values. The GEH formula can be seen in Equation 3.5 as follows.

$$GEH = \sqrt{\frac{2 \text{ x } (\boldsymbol{qsimulated} - \boldsymbol{qobserved})^2}{(\boldsymbol{qsimulated} + \boldsymbol{qobserved})}} \tag{3.5}$$

with:

$q_{simulated}$ = data on the volume of traffic flow simulated results (vehicles/hour), and

$q_{observed}$ = data on the volume of traffic flow from observations (vehicle/hour).

The GEH formula has specific conditions of the resulting error value as shown in Table 3.8 as follows.

Table 3.7 Error Value Terms of Geoffery E. Havers Statistical Formula

GEH < 5,0	Accepted
5,0 ≤ GEH ≤ 10,0	Warning: Possible model errors or bad data
GEH > 10,0	Rejected

(Source: PTV-AG, 2016)

3.8.4 MAPE Formula

The accuracy of a forecasting system is measured by the mean absolute percentage error (MAPE). This accuracy is expressed as a percentage, which may be computed by dividing the actual values divided by the average absolute percent inaccuracy for each time period. Because the variable’s units are scaled to percentage units, making it easier to interpret, the MAPE is the most widely used measure to forecast error.

$$\mathbf{MAPE} = \frac{\mathbf{1}}{\mathbf{n}}\sum_{\mathbf{i=1}}^{\mathbf{n}}\left|\frac{\mathbf{Ai-Fi}}{\mathbf{Ai}}\right| \times \mathbf{100}\% \qquad (3.6)$$

Where:

n = sample size,

Ai = actual data value, and

Fi = forecast data value.

The interpretation of MAPE formula result could be seen in these interval values as follows.

Table 3.8 MAPE Interpretation Intervals

MAPE Value	Interpretation
≤ 10	Very accurate forecast result
10 – 20	Good forecast result
20 – 50	Feasible forecast result
> 50	Inaccurate forecast result

3.9 Fuel Energy Consumption

According to Watanadata et al., (1987), regional considerations, road characteristics, and vehicle characteristics all have a significant impact on each type of transport mode’s fuel usage. Based on the methodology from Taylor and Young

(1996) that is used for data collection and analysis of fuel consumption models can be divided into four categories: immediate, elemental, running speed, and average trip speed. The average travel speed model, when model variables can be predicted consistently throughout the review year, is the most straightforward and practical approach for planning. Based on Khristy and Lall (1990), the following equations, which use the average travel speed model approach, describe the consumption rate of materials fuel (F) per unit distance for a spesific type of vehicle or mode of transportation.

$$\mathbf{F = (kl + k2) \times T} \tag{3.7}$$

Where k1 and k2 are parameters for the vehicle type and the coefficient of distance or journey time, respectively. For example, Pacific Consultant International/PCI (1979), HDM-World Bank (1987), RUCM-Bina Marga and Hoff & Overgaard (1992), and LAPI-ITB (1996) conducted research to influence Indonesia's fuel consumption model. LAPI-ITB suggested the following fuel consumption formulation derived from PCI:

$$\mathbf{Fuel\ Consumption = basic\ fuel\ (1 \pm (kk + kl + kr))} \tag{3.8}$$

With:

Basic fuel = basic fuel consumption in liter (liter/1000 km),

kk = correction due to agility,

kl = correction due to traffic condition, and

kr = correction due to road roughness.

Basic fuel each vehicle class as follows:

$$\mathbf{Basic\ fuel\ vehicle\ type\ I = 0,0284V^2 - 2,0644V + 141,68} \tag{3.9}$$

$$\mathbf{Basic\ fuel\ vehicle\ type\ IIA = 2,26533 \times Basic\ fuel\ type\ I} \tag{3.10}$$

$$\mathbf{Basic\ fuel\ vehicle\ type\ IIB = 2,90805 \times Basic\ fuel\ type\ I} \tag{3.11}$$

With:

V = vehicle speed (km/h),

Vehicle type I = sedan, jeep, pick up, small bus, truck (3/4), and medium bus,

Vehicle type IIA = big truck and big bus, with 2 axles, meanwhile

Vehicle type IIB = big truck and big bus with 3 axles or more.

Table 3.9 Vehicle Base Fuel Consumption Correction Factor

Correction Factor	Description	Condition Limitation	Correction
Correction of negative agility (kk)	g = gradient	0% < g < 5%	-0,337
		g > 5%	-0,158
Correction of positive agility (kk)	g = gradient	0% < g < 5%	0,400
		g > 5%	0,820
Correction of traffic (kl)	v/c = volume per capacity ratio	0 < v/c < 0,6	0,050
		0,6 < v/c < 0,8	0,185
		v/c > 0,8	0,253
Correction of roughness (kr)	r = roughness	r < 3 m/km	0,035
		r > 3 m/km	0,085

(Source: LAPI-ITB, 1996)

Isnaeni (2003) looked at traffic indicators from an environmental perspective, specifically fuel consumption and exhaust emissions. The fuel consumption formulation proposed by LAPI-ITB was converted into passenger car units for the study and the following equation was used to estimate fuel consumption:

$$\mathbf{F_1 = A + BV + CV^2} \quad (3.12)$$

$$\mathbf{F_2 = EV^2} \quad (3.13)$$

$$\mathbf{F_3 = D} \quad (3.14)$$

With:

F_1 = Fuel consumption on constant speed (liter/100 pcu-km),

F_2 = Fuel consumption on acceleration/decelaration (liter/pcu),

F_3 = Fuel consumption on idle (liter/pcu-hour),

V = Vehicle velocity (km/h), and

A = 170.10^{-1} B = -455.10^{-3} C = 490.10^{-5} D = 140.10^{-2} E = 770.10^{-8} (3.15)

Total consumption of fuel in signallized intersection uses the equation of F_3 = fuel consumption on idle, based on the delay time in red light condition.

CHAPTER 4
METHODOLOGY

4.1 Location of Research

This research was conducted in the intersection of Monjali, Sleman, Yogyakarta, Indonesia. This area is one of compact intersection in Yogyakarta because it is near with hotels, ring road, and other public facilities. Based on *Badan Pusat Statistik* of Sleman regency, the total population of each group of age and gender on 2022 is 1.147.562 populations. Each arm of the intersection has different width from north, east, south, west and alley, respectively, 11 meters, 21 meters, 10.5 meters, 21 meters, and 5 meters. This area has counter clockwise cycle traffic and has the most conflict in north arm and the alley. The alley is shown in white area as can be seen in Figure 4.1 below.

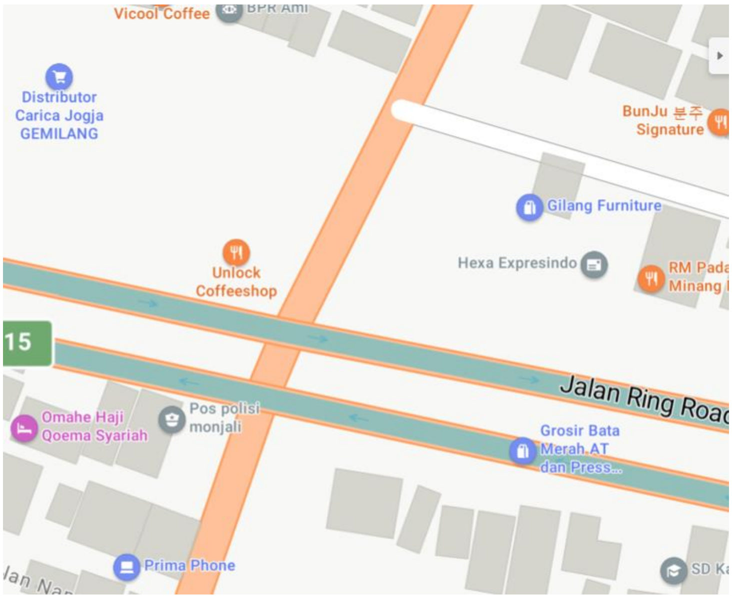

Figure 4.1 Map Location of Research

(Source: Google Maps, 2023)

4.2 Data Collection

Data is a source that needs to be controlled and managed to become a functional form and beneficial. Data collection chosen for this study is observation. Data that could be collected there 2 types, primary data and secondary data that could be seen in Table 4.1.

Table 4.1 Research Data

Primary Data	**Secondary Data**
a. Traffic volume on peak hours b. Queue length on peak hours c. Delay time on peak hours d. Vehicle types (HV, LV, MC, UV) e. Geometry data (road width, lanes, median) f. Traffic signal (cycle time, all red, a mber, intergreen, phase, travel behaviour)	a. Survey location map, Monjali Intersection, Yogyakarta. b. City size of research area.

4.3 Data Collection Method

The method of collecting data is divided into two. Primary data collection is conducted by field survey, meanwhile secondary data is obtained using google maps. See figure 4.2 to see the position of cameras that will be used for traffic counting.

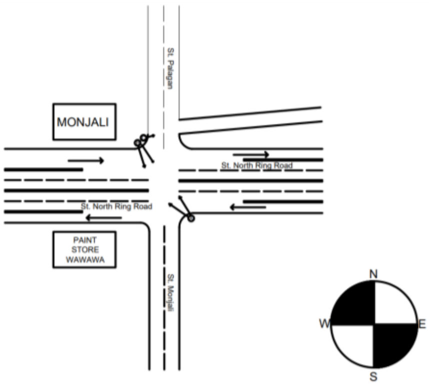

Figure 4.2 Cameras Position for Survey

With:

HV = Heavy Vehicles

LV = Light Vehicles

MC = Motorcyle

UV = Unmotorized Vehicles

A = HV, LV, MC, UV (west and north main arm)

B = HV, LV, MC, UV (south and east)

C = HV, LV, MC, UV (alley in north arm)

Site data survey should consider these following conditions:

1. The lane division must be clear to make it easier for observers to determine whether vehicles have entered or exited the intersection,
2. Divisons of reference points. Usually, the vehicle stops at this point, when the vehicle passes this point, meaning that the vehicle has entered the intersection.

4.2.1 Tools

Tools that are used in this survey or data collection are as follows.

1. Form for traffic counting
2. Camera Go Pro
3. Mobile Phone
4. Walking Measurement

4.2.2 Time for Data Survey

In this survey, since cameras are used to take the traffic counting, videos could be replayed multiple times to count the vehicles passing the area. Cameras were installed in the planned position to take video from that angle. Survey was done in two days, on Wednesday, August 16th 2023 and Saturday, August 19th 2023 based on the peak hours data from Department of Transportation Sleman Regency that is shown in attachment 1. The survey was done at:

1. Morning between 06.45 – 07.45 WIB
2. Afternoon between 12.00 – 13.00 WIB
3. Evening between 16.30 – 17.30 WIB

Determination of the timing of the survey based on the consideration from data that is given by Department of Transportation Yogyakarta that represents the peak day of activity in the region in one week.

4.2.3 Data Collection Information

1. Survey of Intersection Geometry

   Calculations are done in a separate for each approach. One intersection arm can consist of more than one approaches, that is separated into two or more sub-approach. Road geometry data that is observed is consisted of width of road, total of lanes, and road direction.

2. Traffic Light Survey

   The purpose of traffic light survey is to know the length of time of green light, yellow light, and red-light cycle. Survey on (date) at (range time) using stationery and stopwatch.

3. Traffic Volume Survey

   This survey has the purpose to count the volume of vehicles passing the observation point. The vehicles that are observed are all kinds of vehicles. Survey is done an hour each for one or more period, on peak hours in the morning, afternoon, or evening. This will be done using method written in IHCM 1997.

4. Traffic Signal and Intersection Phase Survey

   Traffic signal is done by direct observation in each arm by using stopwatch to get the green time, amber, red, and all red.

5. Spot Speed Survey

   This survey has a purpose to obtain speed data to be input into the VISSIM before calibration.

4.2.4 Traffic Volume Data Survey

The method used in this survey is digital method where cameras were used to record the traffic in the decided time and location. The camera used were CCTV camera and Go Pro cameras. Cameras are installed to record from all directions in the intersection including the alley. Subsequently, in the recorded traffic videos could be done the traffic counting for each arm of the intersections. Traffic counting data are compiled per 15 minutes for each arm and direction.

4.2.5 Number of Queue Data Survey

The existing data was taken directly in the field by surveyors. Surveyors were divided in each arm, by using the written sign on the side road it could be seen the number of queues of the vehicle when the red light on. The number written on the side of the road was in the range of 0 – 200 meters for the bigger road and 0 – 30 meters for the alley.

4.2.6 Delay Time Data Survey

Data of delay time was taken by the same surveyor with the number of queues. Using stopwatch, surveyor starts the stopwatch from the very first queue enters the area, the timer keeps going until the green time and after the queue in the area already in the position of other area.

4.2.7 Spot Speed Segment Method

By determining distance of observation about 25 meters, stopwatch was used to track one chosen vehicle that passes through the length of observation. By calculating the result of distance divided by the time, value of speed is obtained.

4.3 Method of Data Analysis

Data analysis is done using quantitative approach using Indonesian Highway Capacity Manual (IHCM 1997) to calculate the capacity and degree of saturation. The result of traffic survey from the intersection of Monjali and the alley will be analyzed to get the peak hour that is obtained from volume for each 15 minutes for three hours. The result will be used in operating VISSIM software. For the fuel consumption data analysis, LAPI-ITB equations are used to determine the fuel energy consumption using the delay time data.

4.4 Research Flowchart

The flowchart of the research can be seen in Figure 4.2 below.

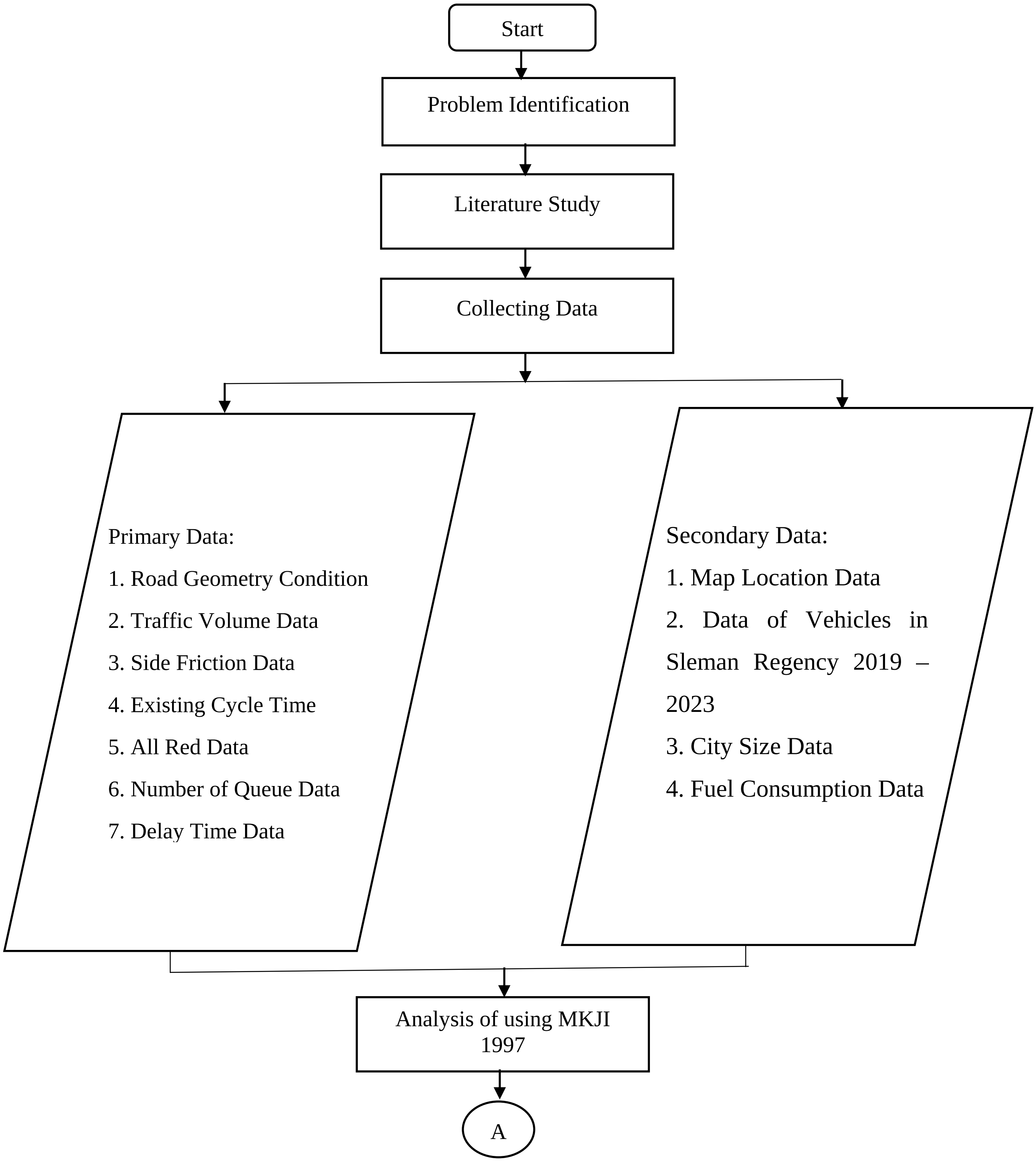

Figure 4.3 Flowchart of Research Step Process (1 of 2)

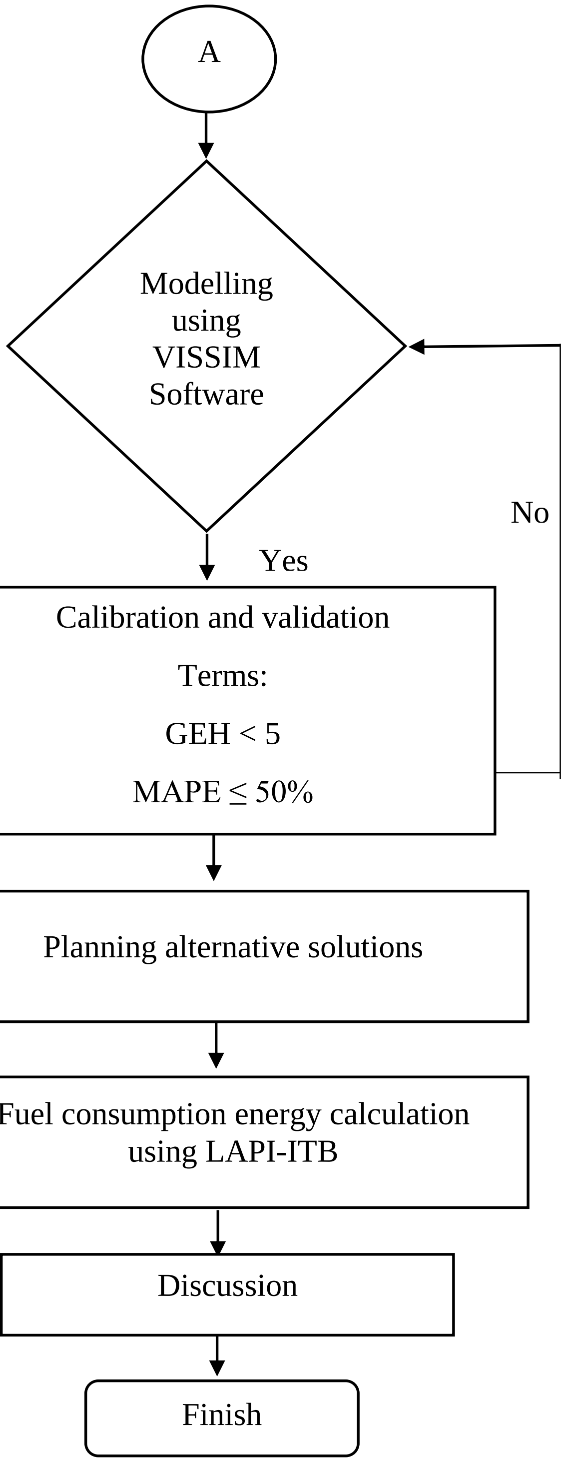

Figure 4.4 Flowchart of Research Step Process (2 of 2)

CHAPTER 5
DATA ANALYSIS AND DISCUSSION

5.1 Collecting Data Results

Data that is needed in the analysis are primary data and secondary data. Primary data is data that is obtained directly from the field observation in purpose to gain the research main target meanwhile secondary data is data that is gained from other sources that is connected to current research. Sources of secondary data could be gotten from government also private institution, that are usually in the form of survey result, census, mapping, etc.

5.1.1 Data of Intersection Geometry

Data of intersection geometry is geometry condition of the road that is being observed. This data could be obtained from both primary data that is gained from existing condition and secondary data that could be obtained from Public Works Department of Sub Bina Marga Special Region of Yogyakarta and Transportation Department of Special Region of Yogyakarta. In this research, the geometry of the intersection is obtained from direct observation, because the information and inventory that is given by Public Works Department of Sub Bina Marga Special Region of Yogyakarta and Transportation Department of Special Region of Yogyakarta very minimal. Therefore, the geometry data could be seen in Table 5.1.

Table 5.1 Geometrical Data and Environmental Road Type in Monjali Intersection

Approachment	North	West	South	East
Road Environment Type	COM	COM	COM	COM
Side Friction	High	Med	High	Med
Median	No	Yes	No	Yes

Continuation Table 5.1 Geometrical Data and Environmental Road Type in Monjali Intersection

LTOR	Yes	Yes	Yes	Yes
Entry Approach Width (m) LTOR Approach Width (m) Exit Approach Width (m)	5.5 2.1 5.5	10.5 7 13.8	5.25 1.6 5.25	10.4 2.3 10.4
Traffic Island	No	No	No	No

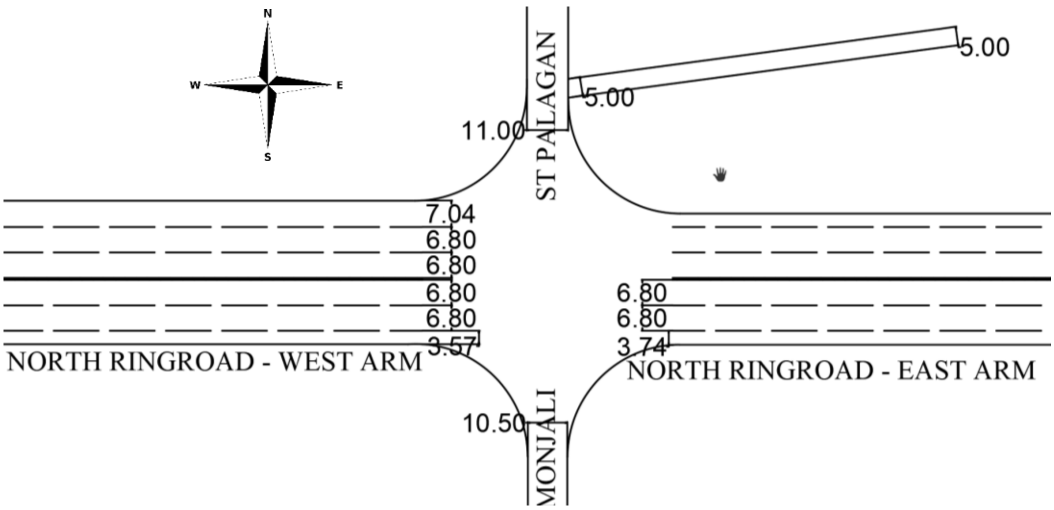

Figure 5.1 Road Geometry of Monjali Intersection

5.1.2 Flow Data and Traffic Composition

The traffic data required is data regarding traffic flow and composition. Both types of data are obtained by conducting surveys directly to the field. Data collection time is carried out on Wednesday and Saturday. As for peak traffic flow hours, it is estimated to be influenced by activities, such as work, school, campus activities and others. For morning peak hours are estimated between 06.00 to 08.00 WIB. The peak afternoon hours are expected between 11.00 to 13.00 WIB. For the evening peak hour estimated at 16.00 to 18.00 WIB. For more details can be seen in Table 5.2.

Table 5.2 Peak Hour Determination Based on Survey Data

Periode	Time	Number of Vehicles (pcu/hour)				Total Number of Vehicles (pcu/hour)
		Utara	Barat	Selatan	Timur	Total
1	2	3	4	5	6	7
Wednesday, morning	06.00-07.00	1026	1981	761	1406	5173
	06.15-07.15	1060	2102	852	1459	5474
	06.30-07.30	1222	2149	888	1522	**5780**
	06.45-07.45	968	2199	872	1540	5580
	07.00-08.00	703	1502	834	1551	4589
	07.15-08.15	609	1688	619	1184	4101
Wednesday, afternoon	11.00-12.00	585	1703	770	1368	4426
	11.15-12.15	612	1757	775	1395	4540
	11.30-12.30	717	1802	781	1361	**4660**
	11.45-12.45	698	1799	769	1332	4598
	12.00-13.00	661	1336	766	1276	4039
	12.15-13.15	547	1341	579	956	3424
Wednesday, evening	16.00-17.00	887	1825	1108	1876	**5696**
	16.15-17.15	859	1818	1096	1880	5653
	16.30-17.30	882	1807	1081	1799	5569
	16.45-17.45	855	1807	1083	1785	5530
	17.00-18.00	745	1527	1061	1784	5117
	17.15-18.15	654	1345	782	1333	4114
Saturday, morning	06.00-07.00	491	1129	376	952	2948
	06.15-07.15	517	1217	443	1043	3220
	06.30-07.30	559	1303	522	1169	3552
	06.45-07.45	621	1428	597	1293	**3940**
	07.00-08.00	574	1197	636	1342	3749
	07.15-08.15	514	1203	494	1020	3231
Saturday, afternoon	11.00-12.00	698	1458	749	1697	4602
	11.15-12.15	691	1482	755	1704	4630
	11.30-12.30	705	1467	769	1721	**4662**
	11.45-12.45	691	1455	773	1696	4615
	12.00-13.00	610	1056	755	1681	4101
	12.15-13.15	505	1111	565	1251	3432
Saturday, evening	16.00-17.00	812	1805	898	1747	**5262**
	16.15-17.15	785	1806	874	1743	5207
	16.30-17.30	732	1825	852	1736	5145

Continuation of Table 5.2 Peak Hour Determination Based on Survey Data

	16.45-17.45	711	1793	823	1719	5046
	17.00-18.00	616	1470	829	1699	4614
	17.15-18.15	495	1314	621	1269	3699

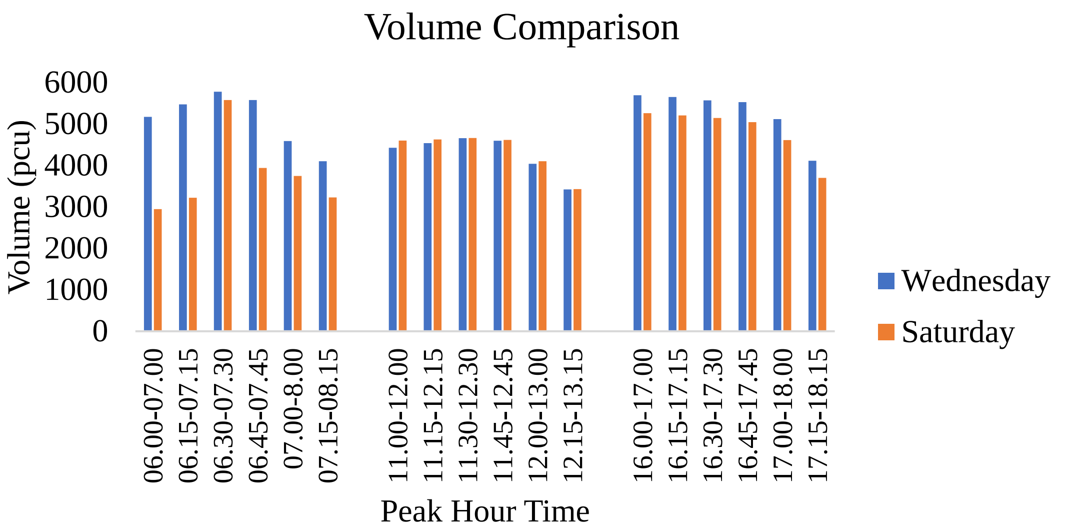

Figure 5.2 Bar Chart of Volume Comparison of Weekday and Weekend

5.1.3 Signal Data, Phase, and Traffic Cycle Time

Signal data, phase and traffic cycle time at Monjali intersection which covers green time, amber, and red time, is obtained from existing survey data in the field by counting using stopwatch. The data could be seen on Table 5.3 as well as the description.

Table 5.3 Signal Data, Phase, and Existing Traffic Time

Approach	**Time (second)**													
	Green			**Yellow**			**Red**			**All Red**	**Cycle Time**		
	M	**A**	**E**	**M**	**A**	**E**	**M**	**A**	**E**		**M**	**A**	**E**
North	31	31	31	3	3	3	128	128	128	3	165	165	165
West	42	42	42	3	3	3	117	117	117	3	165	165	165
South	31	31	31	3	3	3	128	128	128	3	165	165	165
East	37	37	37	3	3	3	122	122	122	3	165	165	165

Description: M = Morning ; A = Afternoon ; E = Evening

Table 5.4 Conversion Result of Morning Peak Hour in the First Day of Passenger Car Unit at Monjali Intersection

Traffic Composition		**LV**		**HV**		**MC**		**PCU-factor**		**K-factor**	**Unmotorized Vehicle (veh/hour)**
Traffic Flow	**Direction**	**Light Vehucle**		**Heavy Vehicle**		**Motorcycle**		**Total of Motorized Vehicle**			
Approach		**Veh/hour**	**pce 1 Pcu/hour**	**Veh/hour**	**pce 1.3 Pcu/hour**	**Veh/hour**	**pce 0.2 Pcu/hour**	**Veh/hour**	**Pcu/hour**	**Turning Ratio**	
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
Minor (North)/A	LT	340	340	0	0	277	55.4	617	396	0.39	2
	ST	138	138	0	0	1240	248	1378	386		11
	RT	142	142	0	0	484	96.8	626	239	0.23	0
	Total	620	620	0	0	2001	400.2	2621	1021		13
Minor (South)/C	LT	227	227	0	0	598	119.6	825	347	0.45	0
	ST	143	143	0	0	848	169.6	991	313		3
	RT	25	25	0	0	423	84.6	448	110	0.14	2
	Total	395	395	0	0	1869	373.8	2264	770		5
Total of Minor Road		1015	1015	0	0	3870	774	4885	1791		18
Major (West)/B	LT	132	132	0	0	395	79	527	211	0.10	2
	ST	877	877	21	27.3	3718	743.6	4616	1648		1
	RT	155	155	0	0	732	146.4	887	302	0.14	0
	Total	1164	1164	21	27.3	4845	969	6030	2161		3
Major (East)/D	LT	123	123	0	0	393	78.6	516	202	0.13	17
	ST	649	649	21	27.3	2279	455.8	2949	1133		8
	RT	130	130	0	0	309	61.8	439	192	0.13	1
	Total	902	902	21	27.3	2981	596.2	3904	1527		26
Total of Major Road		902	902	21	27.3	2981	596.2	3904	1527		29
Major+Minor Road	LT	822	822	0	0	1663	332.6	2485	1156	0.21	21
	ST	1807	1807	42	54.6	8085	1617	9934	3480		23
	RT	452	452	0	0	1948	389.6	2400	843	0.15	3
	Total	3081	3081	42	54.6	11696	2339.2	14819	5479		47
					MINOR ROAD RATIO				0.279	Unmotorized Ratio	0.617

5.2 Performance Analysis of Monjali Signalized Intersection Existing Condition

5.2.1 Monjali Intersection Performance in the Peak Hour Data (06.30 – 07.30) using IHCM 1997

Calculation of road capacity and level of service at Monjali Intersection was completed using the IHCM 1997 method, namely by entering the survey data into the worksheet of IHCM 1997 with the following data sequence as follows:

1. Form GIS-1 : geometry, traffic setting and environment.
2. Form GIS-II : traffic flow,
3. Form SIG-III : green time and lost time.
4. Form GIS-IV : signal time determination and capacity.
5. Form GIS-V : queue length, number of stopped vehicle and delay.

All input data for the calculations below are based on GIS-I to GIS-V forms and the order in which the data is entered into the worksheets is as follows:

1. Form GIS-1 : geometry, traffic setting and environment

   City : Yogyakarta

   City Size : 1.157.642 people

   Day/date : Wednesday, February 23rd 2022

   Total traffic phase : 4

   a. Phase 1 : green time (g) : 31 seconds, time between green = 6 seconds
   b. Phase 2 : green time (g) : 42 seconds, time between green = 6 seconds
   c. Phase 3 : green time (g) : 31 seconds, time between green = 6 seconds
   d. Phase 4 : green time (g) : 37 seconds, time between green = 6 seconds

   Geometry data and environment condition at Monjali Intersection could be seen on Table 5.5:

Table 5.5 Geometry Data and Environment Condition in Monjali Intersection

Approachment	North	West	South	East
Road Environment Type	COM	COM	COM	COM

Continuation of Table 5.5 Geometry Data and Environment Condition in Monjali Intersection

Side Friction	High	Med	High	Med
Median	No	Yes	No	Yes
LTOR	Yes	Yes	Yes	Yes
Entry Approach Width (m) LTOR Approach Width (m) Exit Approach Width (m)	5.5 2.1 5.5	10.5 7 13.8	5.25 1.6 5.25	10.4 2.3 10.4
Traffic Island	No	No	No	No

2. Form GIS-II : Traffic Flow

   Form GIS-II is filled with traffic flow data and turning ratio at Monjali Intersection that could be seen on Table 5.6 as follows:

Table 5.6 Traffic Flow Data and Turning Ratio on the Peak Hour in Monjali Intersection

Approach Flow	North (pcu)			West (pcu)			South (pcu)			East (pcu)		
	LT	ST	RT	LT	ST	RT	LT	ST	RT	LT	ST	RT
MC	55.4	248	96.8	79	743.6	969	119.6	169.6	84.6	78.6	455.8	61.8
LV	340	138	142	132	877	155	227	143	25	123	649	130
HV	0	0	0	0	27.3	0	0	0	0	0	27.3	0
Left Turn Ratio	0.39			0.10			0.45			0.13		
Right Turn Ratio	0.23			0.14			0.14			0.13		

3. Form GIS-IV : signal time determination and capacity

   Calculation example of signal time and capacity:

   Overview of NORTH approach

   $S = S0 \times F_{CS} \times F_{SF} \times F_g \times F_p \times Frt \times F_{lt}$

   (1) Saturated Flow Calculation

   a. Base saturated flow (S0), for:

      Approach type : protected (P)

Width effective : 5.50 m

From the attachment graph VI or with the equation of,

$S0 = 850 \times We^{0.95} = 850 \times 5{,}50^{0.95} = 4293$ pcu/hour

b. Adjustment factor of city size (Fcs), from Table 3.3

Total population = 1157642 people so the Fcs = 1

c. Adjustment factor of side friction (F_{SF}), from Table 3.4 for:

Road environment	: commercial (COM)
Side friction class	: high
Phase type	: protected (P)
Unmotorized vehicle ratio	: 0,617
$F_{SF\ value}$	: 0,81

d. Adjustment factor of gradient (F_G), the result for factor of gradient is 1 based on Figure 3.8

e. Adjustment factor of parking (Fp)

From the first 80 m on the north arm, there is no parked vehicle. Based on Figure 3.9 the result is 1.

f. Factor of turning right adjustment (F_{RT}), from the calculation using the formula is obtained $F_{RT} = 1{,}061$

g. Factor of turning left adjustment (F_{LT}), from the calculation using formula is obtained $F_{LT} = 0{,}938$

h. Value of saturated flow that is adjusted

$S = S0 \times F_{CS} \times F_{SF} \times F_G \times F_p \times F_{RT} \times F_{LT}$

$S = 4293 \times 1 \times 0{,}81 \times 1 \times 1 \times 1{,}061 \times 0{,}938$

$S = 3460$ pcu/hour

(2) Traffic Flow Calculation

Based on the convertion calculation PCU (Passenger Car Unit), it is gained the traffic flow as big as = 1021 pcu/hour

(3) Flow Ratio Calculation (FR)

Equation: $FR = Q/S$

$FR = 1021/3460$

$FR = 0{,}295$

(4) Capacity Calculation (C)

Equation: $C = S \times g/c$

g = green time = 31 seconds

c = cycle time = 165 seconds

$$C = \mathbf{3460}\,\frac{\mathbf{pcu}}{\mathbf{hour}}\ \mathbf{x}\ \frac{\mathbf{31\ seconds}}{\mathbf{165\ seconds}}$$

= 650 pcu/hour

(5) Degree of Saturation (DS)

Equation: DS = Q/C

DS = 1021/650

DS = 1,571

From the calculation above, the traffic flow, capacity and degree of saturation values are obtained. For the recapitulation could be seen in Table 5.7.

Table 5.7 Recapitulation of Operational Calculation in Monjali Intersection on Peak Hour Time

	Approach			
	North	West	South	East
Base Saturated Flow (S0)	4293	10429	3921	7935
City Size Adjustment Factor (Fcs)	1	1	1	1
Gradient Adjustment Factor (Fg)	1	1	1	1
Parking Adjusment Factor (Fp)	1	1	1	1
Right Turn Adjusment Factor (FRT)	1.061	1	1.037	1
Left Turn Adjusment Factor (FLT)	0.938	1	0.928	1
Saturated Flow (S)	3460	8656	3057	6586
Traffic Flow (Q)	1021	2161	770	1527
Flow Ratio (FR)	0.295	0.25	0.252	0.232
Capacity (C)	650	2203	574	1477
Degree of Saturation (DS)	1.571	0.981	1.341	1.034

4. Form GIS-V : Number of queues, number of stopped vehicles, and delay. Calculation example of number of queues, number of stopped vehicles, and delay are as follows:

(1) Calculation of number of queues

a. Number of vehicles that are left behind from the previous green phase

From the equation, it is obtained NQ1 = 187,322 pcu

b. Number of vehicles arriving during the red phase NQ2

From the equation, it is obtained NQ2 = 53.912 pcu

c. Number of queue

NQ = NQ1+NQ2 = 187.322 + 53.912 = 241.234 pcu

d. Number of maximum queued vehicles NQ_{max} = 241.234 pcu

(2) Calculation of number of queues QL

From the equation, it is obtained QL = 877.215 m

(3) Calculation of stop vehicles ratio NS

From the equation, it is obtained NS = 4.640 stop/pcu

(4) Calculation of number of stopped vehicles Nsv

From the equation, it is obtained Nsv = 4737

(5) Calculation of delay

a. Average traffic delay

From the equation, it is obtained DT = 1114.547 sec/pcu

b. Average geometrical delay

From the equation, it is obtained DG = 4 sec/pcu

c. Average delay

D = DT + DG = 1114.547 + 4 = 1118.547 sec/pcu

d. Total delay = D x Q = 1118,547 x (1021/3600) = 317.232 seconds

From the calculation above, it is obtained queue calculation, queue length, number of stops, and total delay. For more details could be seen in Table 5.9.

Table 5.8 Analysis Result of Intersection Performance on Monjali Intersection in the Peak Hour

	Approach			
	North	West	South	East
NQ1, pcu	187.322	14.745	100.257	36.01
NQ2, pcu	53.912	98.4	38.312	70.681
NQ, pcu	241.234	113.145	138.568	106.691
NQmax, pcu	241.234	113.145	138.568	106.691
QL, meter	877.215	161.636	554.274	203.221

Continuation of Table 5.8 Analysis Result of Intersection Performance on Monjali Intersection in the Peak Hour

NS, stop/pcu	4.640	1.028	3.534	1.372
Nsv, smp/pcu	4737	2222	2721	2096
DT, second/pcu	1114.55	85.191	701.189	152.411
DG, second/pcu	4	4	4	4
D, second/pcu	1118.55	89.191	705.189	156.411
Total Delay, second	317.232	53.539	150.832	66.344

So, the average delay for one intersection = sum of total delay / total flow

$$= \frac{\mathbf{2116615}}{(\mathbf{5479}\,\frac{\mathbf{pcu}}{\mathbf{3600\ seconds}})}$$

= 386.314 seconds/pcu

Saputri (2022) stated that in the previous research that was done, the delay of the Monjali intersection was 160 seconds which resulting LOS of F. Meanwhile, in current research that was also considering the alley existence, the delay result is 386 seconds with LOS of F which means the condition become worse.

Table 5.9 Recapitulation Performance Analysis of Existing Condition

Arm Type	Phase	Flow (Q) (pcu/hour)	We	S0	Fcs	Fsf	Fg	Fp	Flt	Frt
A / North	1	1021	5.5	4293	1	0.81	1	1	0.938	1.061
B / West	2	2161	14	10428	1	0.83	1	1	1.000	1.000
C / South	3	770	5	3921	1	0.81	1	1	0.928	1.037
D / East	4	1527	10.5	7935	1	0.83	1	1	1.000	1.000

S	FR	FRcr	PR	green time (g)	c cycle adjusment	Capacity C	DS
3460	0.295	0.295	0.370	31	165	650.072	1.571
8656	0.250	0.250	0.313	42	165	2203.353	0.981
3056	0.252	0.252	0.316	31	165	574.303	1.341
6586	0.232	0.232	0.291	37	165	1476.878	1.034
IFR	0.797						

GR	NQ1	NQ2	NQ	QL	RNS	NS	Nsv
0.188	187.322	53.912	241.234	877.215	4.640	4737	4836477
0.255	14.745	98.400	113.145	161.636	1.028	2222	4801742
0.188	100.257	38.312	138.568	554.274	3.534	2721	2095170
0.224	36.010	70.681	106.691	203.221	1.372	2096	3200592
			241.234				14933981

Continuation of Table 5.9 Recapitulation Performance Analysis of Existing Condition

A	DT	Psv	PT	DG	D	D x Q	D Intersection	LOS
0.468	1114.547	1	0.234	4	1118.547	1142037	386.314	F
0.370	85.191	1	0.140	4	89.191	192741		
0.441	701.189	1	0.143	4	705.189	542996		
0.392	152.411	1	0.126	4	156.411	238841		
						2116615		

Table 5.10 Traffic Signal Timing on Monjali Intersection Peak Hour based on Existing Data

Arm	Time (second)				
	Red	Green	Amber	All Red	Cycle
North	128	31	3	3	165
West	117	42	3	3	
South	128	31	3	3	
East	122	37	3	3	

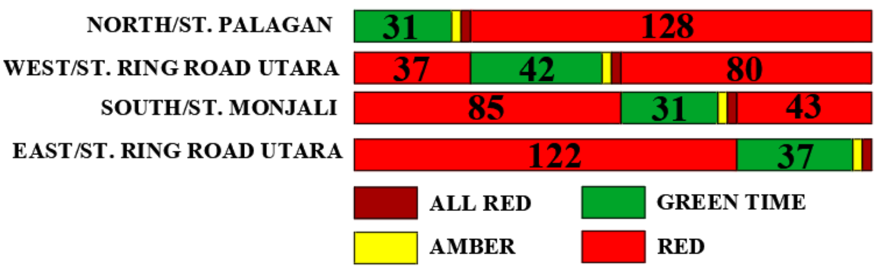

Figure 5.3 Signal Cycle Time Diagram of Monjali Intersection Peak Hour Exisitng Data

5.3.2 Modelling using PTV VISSIM Software

1. Parameter Input VISSIM

   a. Road network

      The existing width of each approach is insert into the road network of VISSIM, following the existing design of using the background road as the base from google maps. The detail of the data could be seen in Table 5.11.

Table 5.11 Monjali Intersection Geometry

Road Name	Approach		
	LTOR Width (m)	Entry Width (m)	Exit Width (m)
St. Palagan	1.76	5	5.5
St. Ring Road Utara West Arm	7	13.6	13.7
St. Monjali	1.5	5.5	5
St. Ring Road Utara East Arm	3.7	13.6	13.6

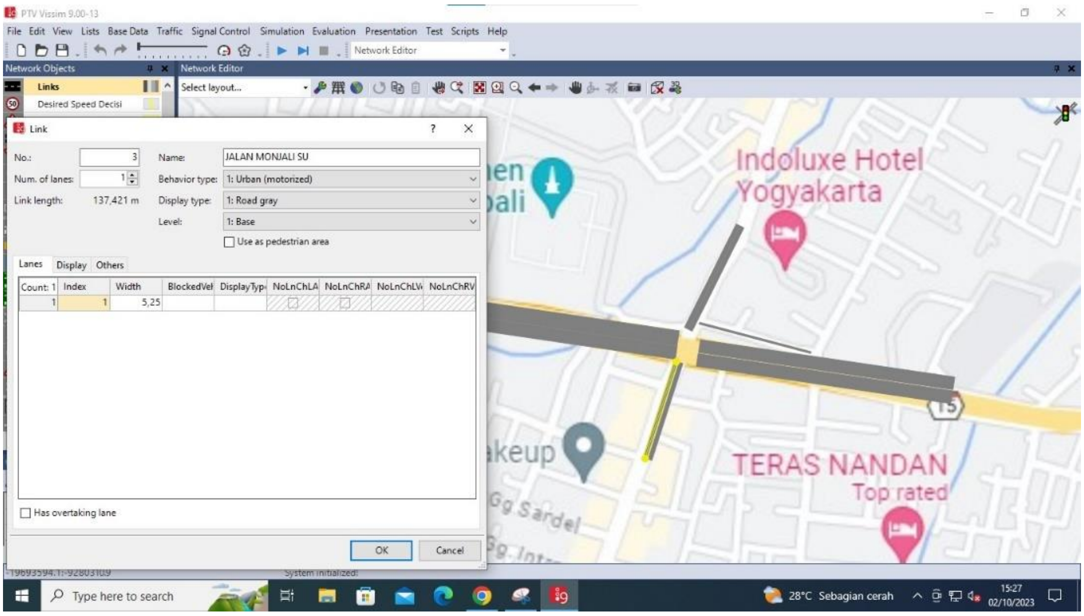

Figure 5.4 Road Network on Monjali Intersection

b. Vehicle Input

Volume of traffic is input in the vehicle input section. For the north arm data is the combination of the main north arm and the alley volume since the traffic phase for that area becomes one, but it is input separately in VISSIM since the road is separated. For east arm case, the total volume input is divided by two since the road link in the VISSIM is also separated.

Table 5.12 Total Vehicle Input for Each Arm of Monjali Intersection in VISSIM

Arm	Vehicle Input
North	2490
West	6021
South	2383
East	3901

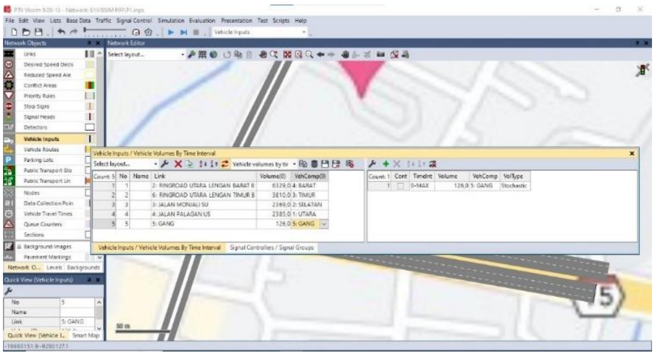

Figure 5.5 Vehicle Input on VISSIM 9

c. Vehicle Composition

Desired speed design is input for each kind of vehicles that were counted in the survey. As it could be seen in the Figure 5.5 RelFLow value is obtained from the number of desired vehicle divided by the total number of vehicle in one arm. Type of vehicles in this modelling is classified into 4 parts:

1) HV is heavy vehicle like bus and big truck that has more than 2 axles.
2) LV is light vehicle like car and mini bus.
3) MC is motorized vehicle with two wheels like motorcycle.

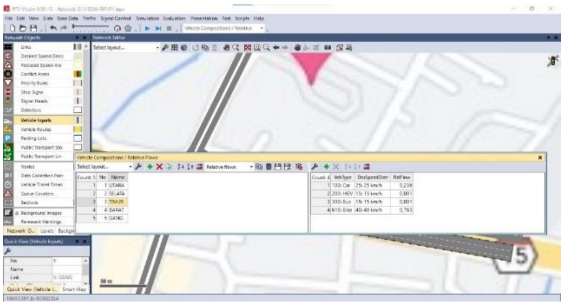

Figure 5.6 Vehicle Composition on VISSIM 9

d. Conflict Area

This part shows the areas of the intersection that will mostly have conflict from one and another arm that happens from the vehicles that pass through the intersection and vehicles that move from one lane to another lane.

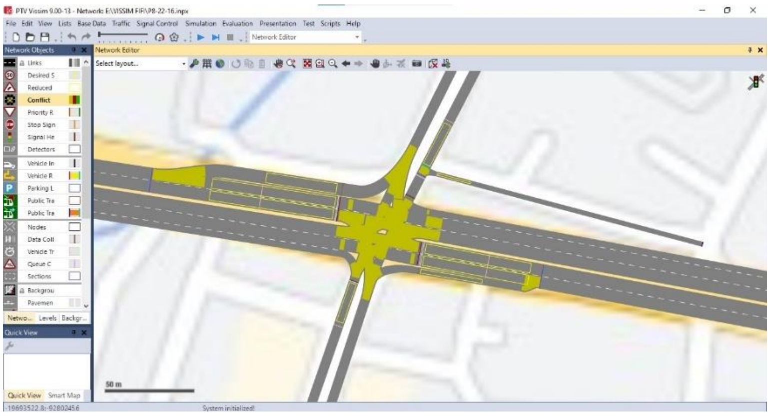

Figure 5.7 Conflict Area on Existing Model of VISSIM

e. Vehicle Routing

This part manages the route of each arm to others arm, the RelFlow value here is gained from the volume of vehicles that turn to one direction divided by the total of volume of each arms.

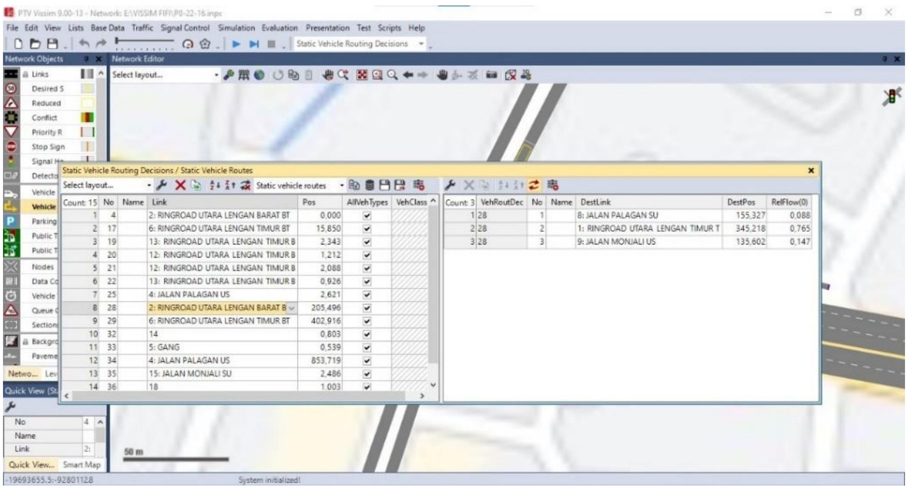

Figure 5.8 Vehicle Routing of Existing Modeling of VISSIM

f. Reduce Speed Area Input

Trial and error were multiple times applied in this section to make sure all vehicles can go pass the road section to obtain lower value of GEH.

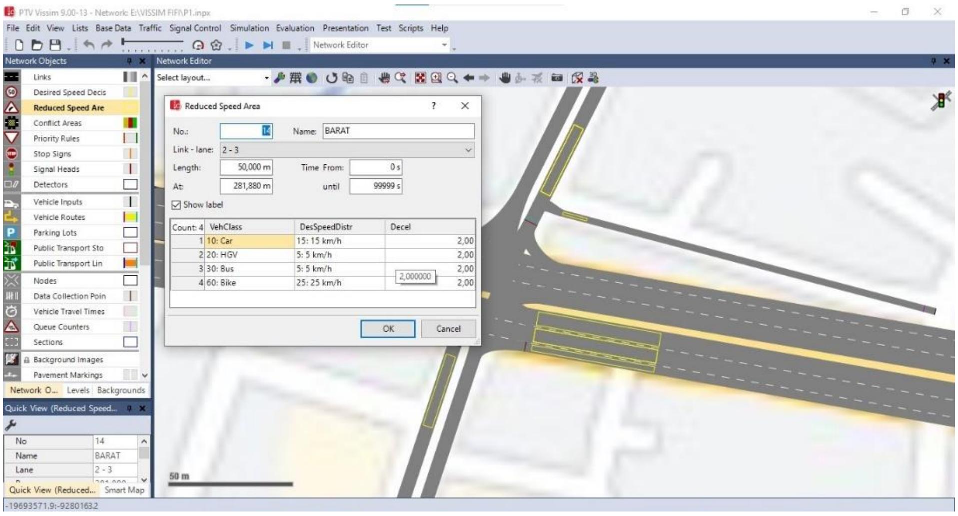

Figure 5.9 Reduce Speed Area of Each Arm of the Existing Model

g. Running Configuration

The data collected from VISSIM are volume, queue counter, and delay.

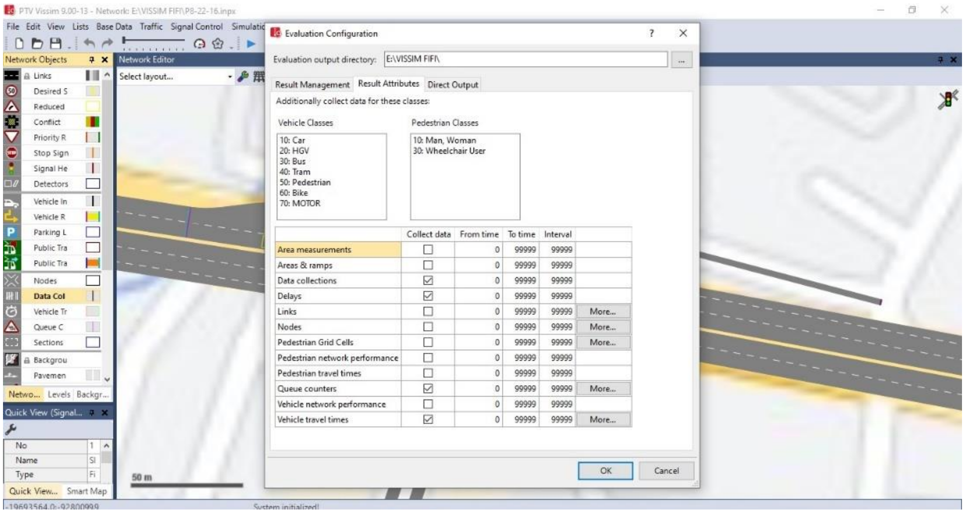

Figure 5.10 Configuration on VISSIM

h. Driving Behavior

In this section driving behavior is set based on existing driver behavior. The original value from VISSIM was changed due to the result of calibration vehicle input could not be finished in some roads, especially in north and south arms of the intersection. Below in Table 5.13 could be seen the adjustment values for the parameter.

Table 5.13 Parameter on Driving Behavior Tab Adjustment

Parameter	Calibration Value	
	Before	After
Desired position at free flow	Middle of lane	Any
Overtake on same lane: on left & on right	off	on
Distance standing (at 0 kmph)(m)	1	0.15
Distance standing (at 50 kmph)(m)	1	0.15
Look ahead distance	400	200
Look back distance	400	200
Average standstill distance	2	0.35
Additive part of safety distance	2	0.35
Multiplicative part of safety distance	3	0.80
Waiting time before diffusion (s)	60	20
Min. headway (front/rear)(m)	0.5	0.15
Safety distance reduction factor	0.6	0.15

2. Result of Existing Modelling

Modelling for existing condition is done using data in accordance with the field data obtained from the survey. Calibration was done multiple times until the GEH value is below 5%. Below is the average result after running five times for calibration, it could be seen in Table 5.14.

Table 5.14 Running Result of Existing Condition

Road	Qlen (m)	Vehs (All)	VehDelay (sec/pcu)
North	52.11	2271	71.08
West	44.09	5979	1.56

Continuation of Table 5.14 Running Result of Existing Condition

South	40.69	2290	3.87
East	50.13	3841	0.55
Alley	26.20	125	2.69

Based on the running result, the vehicle delay of each arm is calculated to obtain the delay value for the intersection.

Total delay = D x Q

= 71.08 x (1021/3600 seconds)

= 20.159

Table 5.15 Calculation result of VISSIM Total Delay Existing Condition

Arm	Dvissim	Q	D x Q
North	71.08	0.284	20.159
West	1.56	0.600	0.936
South	3.87	0.214	0.828
East	0.55	0.424	0.233
Alley	2.69	0.036	0.100
Total		1.558	22.254

So, the average delay for one intersection = sum of total delay / total flow

$$= \frac{\mathbf{22.254}}{\left(\mathbf{1.558}\,\frac{\mathbf{pcu}}{\mathbf{second}}\right)}$$

= 14.281 seconds/pcu

5.3.3 Validation Data using GEH Statistics Formula

In validating using the total traffic flow volume according to Gustavsson (2007), the best method to compare the input and output data of the simulation is to use the Geoffrey E. Havers statistical formula (GEH). The GEH formula has specific provisions for the resulting error values as shown in Table 5.16 and the results of the simulation as in Table 3.18.

GEH of north arm:

$$\mathbf{GEH} = \sqrt{\frac{\mathbf{2(2396-2621)^2}}{\mathbf{(2396+2621)}}} = \mathbf{4.492 < 5.0\ accepted}$$

Table 5.16 Validation Result of GEH Statistical Trial Existing Condition

Arm	$q_{observed}$ (veh/hour)	$Q_{simulated}$ (veh/hour)	GEH	Description
North	2621	2396	4.492	Accepted
West	6021	5979	0.542	Accepted
South	2383	2290	1.924	Accepted
East	3901	3841	0.964	Accepted

Based on Table 5.16 above, it can be concluded that the existing simulation modelling of VISSIM simulation is acceptable after seeing the results of the validation test using the GEH statistical formula.

5.3.4 Queue Validation using MAPE Formula

In the existing data survey, queue length is also observed. In this section using the MAPE formula, the comparison percentage between existing data and VISSIM data after calibration is calculated. Below is the queue length comparison from the existing data and VISSIM calibration result as well as the MAPE result in Table 5.17.

North Arm MAPE Calculation:

$$\mathbf{MAPE} = \frac{\mathbf{1}}{\mathbf{5}}\sum_{\mathbf{i=1}}^{\mathbf{5}}\left|\frac{\mathbf{121.67-59.93}}{\mathbf{121.67}}\right| \times \mathbf{100\%} = \mathbf{10\%}\ \mathbf{, good\ forecasting\ result}$$

Table 5.17 MAPE Result of QLength from Existing and VISSIM

Arm	Data		MAPE
	Existing	VISSIM	
North	121.67	59.93	10%
West	80	44.75	9%
South	68.00	40.69	8%
East	62.00	50.13	4%
Alley	15.25	26.25	14%

5.4 Alternative Scenarios Analysis using VISSIM Modelling 1

Based on the existing data calculation it is required to find solution. The first solution is the scenario of closing the alley next to north arm by using the same cycle time with the existing condition. Below is the calculation of the alternative 1, by reducing the volume on the north arm since the scenario is to block the alley, the calculation could be seen in the Table 5.18.

5.4.1 Signal Phase 1 Calculation using IHCM 1997

The calculation was done exactly like the example in existing condition and the result could be seen in the Table 5.18 as recapitulation.

Table 5.18 Recapitulation of Analysis Calculation Alternatives 1

Arm Type	Phase	Flow (Q) (pcu/hour)	We	S0	Fcs	Fsf	Fg	Fp	Flt	Frt
A / North	1	970	5.5	4293	1	0.81	1	1	0.938	1.061
B / West	2	2161	14	10428	1	0.83	1	1	1.000	1.000
C / South	3	770	5	3921	1	0.81	1	1	0.928	1.037
D / East	4	1527	10.5	7935	1	0.83	1	1	1.000	1.000

S	FR	FRcr	PR	green time (g)	c cycle adjusment	Capacity C	DS
3460	0.280	0.280	0.277	31	165	650	1.492
8656	0.250	0.250	0.246	42	165	2203	0.981
3056	0.252	0.252	0.248	31	165	574	1.341
6586	0.232	0.232	0.229	37	165	1477	1.034
IFR	1.014						

GR	NQ1	NQ2	NQ	QL	RNS	NS	Nsv
0.188	161.948	50.170	212.118	771.337	4.294	4166	4041020
0.255	14.745	98.400	113.145	161.636	1.028	2222	4801742
0.188	100.257	38.312	138.568	554.274	3.534	2721	2095170
0.224	36.010	70.681	106.691	203.221	1.372	2096	3200592
			154.610				14138524

A	DT	Psv	PT	DG	D	D x Q	D Intersection	LOS
0.458	972.428	1.000	0.234	4.000	976.428	947136	354.03721	F
0.370	85.191	1.000	0.140	4.000	89.191	192741		
0.441	701.189	1.000	0.143	4.000	705.189	542996		
0.392	152.411	1.000	0.126	4.000	156.411	238841		
						192714		

Table 5.19 Conversion Result of Alternative 1 Data

Traffic Composition		LV		HV		MC		PCU-factor		K-factor	Unmotorized Vehicle (veh/hour)
Traffic Flow	Direction	Light Vehicle		Heavy Vehicle		Motorcycle		Total of Motorized Vehicle			
Approach		Veh/hour	pce 1 Pcu/hour	Veh/hour	pce 1.3 Pcu/hour	Veh/hour	pce 0.2 Pcu/hour	Veh/hour	Pcu/hour	Turning Ratio	
1	2	3	4	5	6	7	8	9	10	11	12
Minor (North)/A	LT	323	323	0	0	263	52.63	586.15	376	0.39	2
	ST	131	131.1	0	0	1178	235.6	1309.1	367		10
	RT	135	134.9	0	0	460	91.96	594.7	227	0.23	1
	Total	589	589	0	0	1901	380	2490	970		13
Minor (South)/C	LT	227	227	0	0	598	119.6	825	347	0.45	0
	ST	143	143	0	0	848	169.6	991	313		3
	RT	25	25	0	0	423	84.6	448	110	0.14	2
	Total	395	395	0	0	1869	373.8	2264	770		5
Total of Minor Road		984	984	0	0	3769.95	753.8	4753.95	1740		18
Major (West)/B	LT	132	132	0	0	395	79	527	211	0.10	2
	ST	877	877	21	27.3	3718	743.6	4616	1648		1
	RT	155	155	0	0	732	146.4	887	302	0.14	0
	Total	1164	1164	21	27.3	4845	969	6030	2161		3
Major (East)/D	LT	123	123	0	0	393	78.6	516	202	0.13	17
	ST	649	649	21	27.3	2279	455.8	2949	1133		8
	RT	130	130	0	0	309	61.8	439	192	0.13	1
	Total	902	902	21	27.3	2981	596.2	3904	1527		26
Total of Major Road		902	902	21	27.3	2981	596.2	3904	1527		29
Major+Minor Road	LT	805	805	0	0	1649.15	329.83	2454.15	1136	0.21	21
	ST	1800.1	1800.1	42	54.6	8023	1604.6	9865.1	3461		23
	RT	444.9	444.9	0	0	1923.8	384.76	2368.7	831	0.15	3
	Total	3050	3050	42	54.6	11595.95	2319.19	14687.95	5428		47
					MINOR ROAD RATIO				0.281	Unmotorized Ratio	0.613

The signal phase design is shown in chart below in Figure 5.11. In the signal phase figure, the amber and all red are not written since those will not be visible but it is already stated in the Table 5.20.

Table 5.20 Traffic Signal Timing on Monjali Intersection Peak Hour Alternative 1

Arm	Time (second)				
	Red	Green	Amber	All Red	Cycle
North	128	31	3	3	165
West	117	42	3	3	
South	128	31	3	3	
East	122	37	3	3	

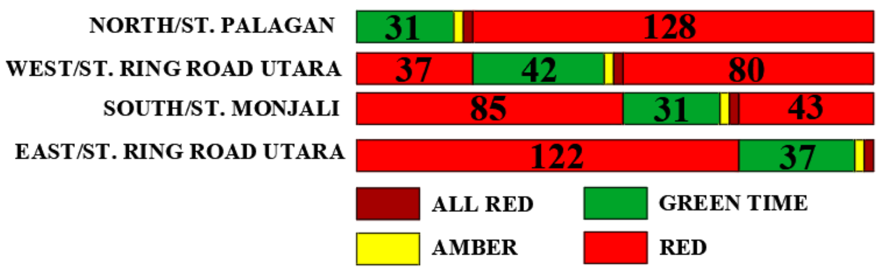

Figure 5.11 Signal Cycle Time Diagram of Monjali Intersection Alternative 1

5.4.2 Alternative 1 Modelling Using PTV VISSIM Software

Using the previous calculation of IHCM 1997, the analysis is continued by inputting the cycle time into signal control of PTV VISSIM Model and removing the road network of the alley as it follows the design of alternative.

1. Parameter Input VISSIM
   a. Vehicle Input

      In the vehicle input area, the volume of traffic is entered. The vehicle intake for the alley is likewise eliminated because the alley road network is blocked. Given that the road link in the VISSIM is similarly divided, the east arm volume included into the current model is divided by two.

Table 5.21 Total Vehicle Input for Alternatice 1 in VISSIM

Arm	Vehicle Input
North	2490
West	6021
South	2383
East	3901

b. Driving Behavior

This section sets driving behavior is also adjusted before calibrating the model. Due to incomplete vehicle input during calibration on some routes, particularly in the intersection's north and south arms, the initial value from VISSIM was altered. The parameter adjustment values are shown below in Table 5.22.

Table 5.22 Parameter on Driving Behavior Tab Adjustment Alternative 1

Parameter	Calibration Value	
	Before	After
Desired position at free flow	Middle of lane	Any
Overtake on same lane: on left & on right	off	on
Distance standing (at 0 kmph)(m)	1	0.15
Distance standing (at 50 kmph)(m)	1	0.25
Look ahead distance	400	200
Look back distance	400	200
Average standstill distance	2	0.4
Additive part of safety distance	2	0.4
Multiplicative part of safety distance	3	0.80
Waiting time before diffusion (s)	60	20
Min. headway (front/rear)(m)	0.5	0.15
Safety distance reduction factor	0.6	0.15

2. Result of Alternative 1 Modelling

From the calibration of alternative 1 which the model is already adjusted, calibration was done several times until the value of GEH of each arm is no more than 5% as it can be seen in Table 5.23.

Table 5.23 Running Result of Alternative 1 Condition

Road	Qlen (m)	Vehs (All)	VehDelay (sec/pcu)
North	58.93	2274	21.06
West	44.15	5987	1.76
South	19.53	2357	1.44
East	50.76	3840	0.6

Based on the running result, the vehicle delay of each arm is calculated to obtain the delay value for the intersection.

Total delay = D x Q

= 21.06 x (970/3600 seconds)

= 5.675

Table 5.24 Calculation result of VISSIM Total Delay Alternative 1

Arm	Dvissim	Q	D x Q
North	21.06	0.269	5.675
West	1.76	0.600	1.056
South	1.44	0.214	0.308
East	0.6	0.424	0.255
Total		1.508	7.293

So, the average delay for one intersection = sum of total delay / total flow

$$= \frac{\mathbf{7.293}}{(\mathbf{1.508}\ \frac{\mathbf{pcu}}{\mathbf{second}})}$$

= 4.837 seconds/pcu

5.4.3 Impact Analysis of Alternative 1

For alternative 1 which the design is to block the current alleyway and move the flow, the impact would be a higher conflict on the three-legged intersection of Bawal alley – Palagan street since the volume coming in and out from that area will increase. As it could be seen in Figure 5.12 from the Sumberan alley (yellow triangle) to Bawal alley (green triangle), the flow would be transferred and based on the existing condition, the volume of vehicles on the Bawal alley has reached 10% of the north arm’s total volume.

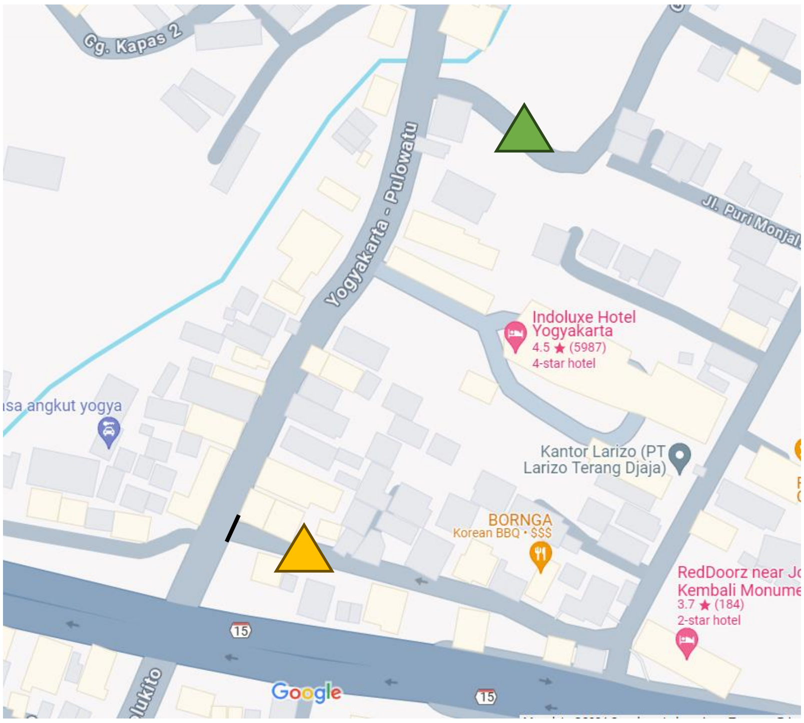

Figure 5.12 Location of Volume Transfers to Bawal Alley

5.5 Alternative Scenarios Analysis using VISSIM Modelling 2

The second alternative for Monjali intersection is to design the intersection using 5 phases cycle time with additional traffic signal specifically for the alley. By separating the alley from the north arm traffic signal, the north arm traffic light should be put behind of the current position as well as the stop line of the north arm. The plan is to give two signals on north arm, when the alleyway traffic light is on green the LTOR on north arm will be red and it will turn green when the traffic light on north arm turns green until the green time of east arm. Table 5.25 displays the IHCM 1997 analysis., whereas Table 5.27 displays the conversion of the volume into PCU unit.

5.5.1 Signal Phase 2 Calculation using IHCM 1997

The computation was carried out precisely as the example in the existing comdition and the outcome was summarized in Table 5.25.

Table 5.25 Recapitulation of Analysis Calculation Alternative 2

Arm Type	Phase	Flow (Q) (pcu/hour)	We	S0	Fcs	Fsf	Fg	Fp	Flt	Frt
A / North	1	970	5.5	4293	1	0.81	1	1	0.938	1.061
B / West	2	2161	14	10428	1	0.83	1	1	1.000	1.000
C / South	3	770	5	3921	1	0.81	1	1	0.928	1.037
D / East	4	1527	10.5	7935	1	0.83	1	1	1.000	1.000
E/ Alley	5	55	2.5	2030	1	0.83	1	1	1.000	1.006

S	FR	FRcr	PR	green time (g)	c cycle adjusment	Capacity C	DS
3460	0.280	0.280	0.277	31	149	697	1.392
8656	0.250	0.250	0.246	42	149	1569	1.378
3056	0.252	0.252	0.248	31	149	552	1.396
6586	0.232	0.232	0.229	37	149	1105	1.382
1696	0.032	0.032	0.031	10	149	114	0.483
IFR	1.047						

GR	NQ1	NQ2	NQ	QL	RNS	NS	Nsv
0.201	22.700	44.554	67.254	244.560	1.508	1463	1419110
0.181	43.910	97.600	141.510	202.157	1.424	3078	6651558
0.181	24.730	34.932	59.662	238.647	1.685	1298	999460
0.168	40.700	68.472	109.172	207.947	1.555	2374	3625098
0.067	24.480	2.195	26.675	213.398	10.546	581	31955
			80.854				12727181

A	DT	Psv	PT	DG	D	D x Q	D Intersection	LOS
0.443	183.331	1.000	0.234	4.000	187.331	181712		
0.447	167.343	1.000	0.140	4.000	171.343	370273		
0.449	228.280	1.000	0.143	4.000	232.280	178856	198.520	F
0.451	199.763	1.000	0.126	4.000	203.763	311146		
0.450	841.427	1.000	0.025	4.000	845.427	46499		
						1088486		

Table 5.26 Conversion Result of Alternatives 2 Data

Traffic Composition		LV		HV		MC		PCU-factor		K-factor	Unmotorized Vehicle (veh/hour)
Traffic Flow		Light Vehicle		Heavy Vehicle		Motorcycle		Total of Motorized Vehicle			
Approach	Direction	Veh/hour	pce 1 Pcu/hour	Veh/hour	pce 1.3 Pcu/hour	Veh/hour	pce 0.2 Pcu/hour	Veh/hour	Pcu/hour	Turning Ratio	
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
Minor (North)/A	LT	323	323	0	0	263	52.63	586.15	376	0.39	2
	ST	131	131.1	0	0	1178	235.6	1309.1	367		10
	RT	135	134.9	0	0	460	91.96	594.7	227	0.23	1
	Total	589	589	0	0	1901	380	2490	970		13
Minor (South)/C	LT	227	227	0	0	598	119.6	825	347	0.45	0
	ST	143	143	0	0	848	169.6	991	313		3
	RT	25	25	0	0	423	84.6	448	110	0.14	2
	Total	395	395	0	0	1869	373.8	2264	770		5
Minor (Alley)/E	LT	0	0	0	0	0	0	0	0	0.00	0
	ST	18	18	0	0	62	12.4	80	31		1
	RT	16	16	0	0	38	7.6	54	24	0.02	0
	Total	34	34	0	0	100	20	134	55		1
Total of Minor Road		1015	1015	0	0	3870	774	4885	1791		18
Major (West)/B	LT	132	132	0	0	395	79	527	211	0.10	2
	ST	877	877	21	27.3	3718	743.6	4616	1648		1
	RT	155	155	0	0	732	146.4	887	302	0.14	0
	Total	1164	1164	21	27.3	4845	969	6030	2161		3
Major (East)/D	LT	123	123	0	0	393	78.6	516	202	0.13	17
	ST	649	649	21	27.3	2279	455.8	2949	1133		8
	RT	130	130	0	0	309	61.8	439	192	0.13	1
	Total	902	902	21	27.3	2981	596.2	3904	1527		26
Total of Major Road		902	902	21	27.3	2981	596.2	3904	1527		29
Major+Minor Road	LT	805	805	0	0	1649	330	2454	1136	0.21	21
	ST	1818	1818	42	54.6	8085	1617	9945	3492		23
	RT	461	461	0	0	1962	392	2423	855	0.16	3
	Total	3050	3050	42	54.6	11695	2339	14822	5483		48
					MINOR ROAD RATIO				0.278	Unmotorized Ratio	0.604

Table 5.26 displays the timing and Figure 5.12 shows the cycle time. The amber and all red are not shown but it is stated clearly in the table. For the traffic order design is pretty similar with the existing condition, which starts from the north and ends in the alleyway.

Table 5.27 Traffic Signal Timing on Monjali Intersection Alternative 2

Arm	Time (second)				
	Red	Green	Amber	All Red	Cycle
North	113	30	3	3	149
West	116	27	3	3	
South	116	27	3	3	
East	118	25	3	3	
Alley	133	10	3	3	

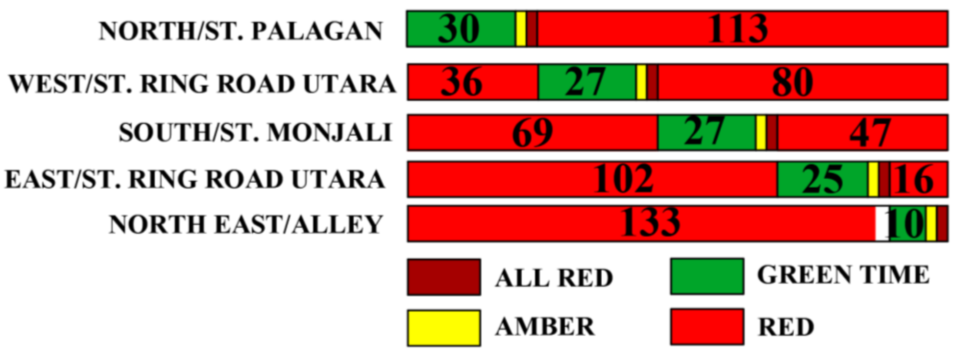

Figure 5.13 Signal Cycle Time Diagram of Monjali Intersection Alternative 2

5.5.2 Alternative 2 Modelling Using PTV VISSIM Software

Continuing the analysis, the cycle time of alternative 2 was calculated using IHCM 1997 formula. The road network of the alley is available in accordance with the alternative's design and the cycle time result which is 5 phases cycle time is entered into the PTV VISSIM Model's signal control.

1. Parameter Input VISSIM

   a. Vehicle Input

   In the vehicle input area, the volume of traffic is entered. The vehicle intake for the alley is separated since the design is to give the alley its own traffic signal. Given that the road link in the VISSIM is similarly divided, the east arm volume included into the current model is divided by two.

Table 5.28 Total Vehicle Input for Alternative 2 in VISSIM

Arm	Vehicle Input
North	2490
West	6021
South	2383
East	3901
Alley	131

a. Driving Behavior

Prior to calibrating the model, driving behavior is also changed in this stage. The initial value from VISSIM was changed because partial vehicle input occurred during calibration on various routes, especially in the north and south arms of the intersection. Table 5.29 below displays the values for the parameter adjustments. The value used in this calibration of alternative 2 is pretty similar with alternative 1.

Table 5.29 Parameter on Driving Behavior Tab Adjustment Alternative 2

Parameter	Calibration Value	
	Before	After
Desired position at free flow	Middle of lane	Any
Overtake on same lane: on left & on right	off	on
Distance standing (at 0 kmph)(m)	1	0.15
Distance standing (at 50 kmph)(m)	1	0.25
Look ahead distance	400	200
Look back distance	400	200
Average standstill distance	2	0.35
Additive part of safety distance	2	0.35
Multiplicative part of safety distance	3	0.80
Waiting time before diffusion (s)	60	20
Min. headway (front/rear)(m)	0.5	0.15
Safety distance reduction factor	0.6	0.15

2. Result of Alternative 2 Modelling

As shown in Table 5.32 below, calibration was carried out multiple times starting with Alternative's 2 calibration, in which the model had previously been changed. This process was completed until the GEH value of each arm was no higher than 5%.

Table 5.30 Running Result of Alternative 2 Condition

Road	Qlen (m)	Vehs (All)	VehDelay (sec/pcu)
North	29.60	2406	22.19
West	45.91	5979	1.58
South	26.93	2355	2.51
East	50.76	3840	0.6
Alley	26.65	130	36.65

Based on the running result, the vehicle delay of each arm is calculated to obtain the delay value for the intersection.

Total delay = D x Q

= 22.19 x (970/3600 seconds)

= 5.979

Table 5.31 Calculation result of VISSIM Total Delay Alternative 2

Arm	Dvissim	Q	D x Q
North	22.19	0.269	5.979
West	1.56	0.600	0.948
South	2.51	0.214	0.537
East	0.6	0.424	0.255
Alley	36.65	0.015	0.560
Total		1.523	8.279

So, the average delay for one intersection = sum of total delay / total flow

$$= \frac{\mathbf{8.279}}{(\mathbf{1.523}\ \frac{\mathbf{pcu}}{\mathbf{second}})}$$

= 5.436 seconds/pcu

5.5.3 Impact Analysis of Alternative 2

The second alternative which adding traffic signal on the alleyway, based on the calculation the cycle time will be much longer compared to the existing condition. Additional conflict will occupy in the LTOR system of north arm and alleyway if the vehicles do not follow the traffic signal for the turn left signal. As it could be seen on Figure 5.14 the blue arrow which shows the turn left flow from alley and orange arrow from the north arm will meet conflict (black star). As it is stated in the previous part where the turn left signal for both north arm and alleyway will follow the time of green phases, LTOR on north arm might off when the alleyway is on green time and vice versa.

Figure 5.14 Conflict Occurrence on LTOR System of North Arm and Alleyway

5.6 Alternative Scenarios Analysis using VISSIM Modelling 3

Welendo and Syamsul (2017) conducted research where the results of the level of service by using protected phase is E, increased into B by using the opposite phase. By using the original design of the existing condition, the turn right ratio of north arm and south arm are small which could be designed into the opposite phase.

5.6.1 Signal Phase 3 Calculation using IHCM 1997

By adding traffic signal on the alleyway and combining the traffic signal of north and south arm, the flow shown is a combined data from south and north arm. Pce used for the conversion is changed from 0.2 to 0.4. The final result of delay intersection could be seen in Table 5.32.

Table 5.32 Recapitulation of Analysis Calculation Alternative 3

Arm Type	Phase	Flow (Q) (pcu/hour)	We	S0	Fcs	Fsf	Fg	Fp	Flt	Frt
A / West	1	2161	14	10429	1	0.83	1	1	1.000	1.000
B / South+North	2	1740	5	4107	1	0.81	1	1	0.928	1.033
C / East	3	1527	10.5	7935	1	0.83	1	1	1.000	1.000
D / Alley	4	55	2.5	2030	1	0.83	1	1	1.000	1.006

S	FR	FRcr	PR	green time (g)	c cycle adjusment	Capacity C	DS
8656	0.250	0.250	0.236	28	148	1638	1.320
3056	0.546	0.546	0.515	60	148	1292	1.346
6586	0.232	0.232	0.219	26	148	1157	1.320
1696	0.032	0.032	0.031	10	148	115	0.480
IFR	1.060						

GR	NQ1	NQ2	NQ	QL	RNS	NS	Nsv
0.189	43.910	96.000	139.910	199.871	1.417	3063	6619143
0.405	24.730	93.642	118.372	450.941	1.489	2592	4510080
0.176	40.700	67.368	108.068	205.843	1.549	2366	3612882
0.068	24.480	2.179	26.659	213.272	10.611	584	32120
			98.252				14774225

A	DT	Psv	PT	DG	D	D x Q	D Intersection	LOS
0.438	161.362	1.000	0.140	4.000	165.362	357348	169.618	F
0.389	126.482	1.000	0.143	4.000	130.482	227039		
0.442	192.097	1.000	0.126	4.000	196.097	299441		
0.449	835.715	1.000	0.025	4.000	839.715	46185		
						930013		

Table 5.33 Conversion Result of Alternative 3 Data

Traffic Composition		LV		HV		MC		PCU-factor		K-factor	Unmotorized Vehicle (veh/hour)
Traffic Flow		Light Vehicle		Heavy Vehicle		Motorcycle		Total of Motorized Vehicle			
Approach	Direction	Veh/hour	pce 1 Pcu/hour	Veh/hour	pce 1.3 Pcu/hour	Veh/hour	pce 0.4 Pcu/hour	Veh/hour	Pcu/hour	Turning Ratio	
1	2	3	4	5	6	7	8	9	10	11	12
Minor (North)/A	LT	323	323	0	0	263	105.26	586.15	429	0.39	2
	ST	131	131.1	0	0	1178	471.2	1309.1	603		10
	RT	135	134.9	0	0	460	183.92	594.7	319	0.23	1
	Total	589	589	0	0	1901	760	2490	1351		13
Minor (South)/C	LT	227	227	0	0	598	239.2	825	467	0.45	0
	ST	143	143	0	0	848	339.2	991	313		3
	RT	25	25	0	0	423	169.2	448	110	0.14	2
	Total	395	395	0	0	1869	747.6	2264	770		5
Minor (Alley)/E	LT	0	0	0	0	0	0	0	0	0.00	0
	ST	18	18	0	0	62	24.8	80	31		1
	RT	16	16	0	0	38	15.2	54	24	0.02	0
	Total	34	34	0	0	100	40	134	55		1
Total of Minor Road		1015	1015	0	0	3870	1548	4885	2571		18
Major (West)/B	LT	132	132	0	0	395	158	527	211	0.10	2
	ST	877	877	21	27.3	3718	1487.2	4616	1648		1
	RT	155	155	0	0	732	292.8	887	302	0.14	0
	Total	1164	1164	21	27.3	4845	1938	6030	2161		3
Major (East)/D	LT	123	123	0	0	393	157.2	516	202	0.13	17
	ST	649	649	21	27.3	2279	911.6	2949	1133		8
	RT	130	130	0	0	309	123.6	439	192	0.13	1
	Total	902	902	21	27.3	2981	1192.4	3904	1527		26
Total of Major Road		902	902	21	27.3	2981	1192.4	3904	1527		29
Major+Minor Road	LT	805	805	0	0	1649	660	2454	1136	0.21	21
	ST	1818	1818	42	54.6	8085	3234	9945	3492		23
	RT	461	461	0	0	1962	785	2423	855	0.16	3
	Total	3050	3050	42	54.6	11695	4687	14822	5483		48
					MINOR ROAD RATIO				0.271	Unmotorized Ratio	0.604

Table 5.34 Traffic Signal Timing on Monjali Intersection Alternative 3

Arm	Time (second)				
	Red	Green	Amber	All Red	Cycle
West	114	28	3	3	148
South+North	82	60	3	3	
East	116	26	3	3	
Alley	132	10	3	3	

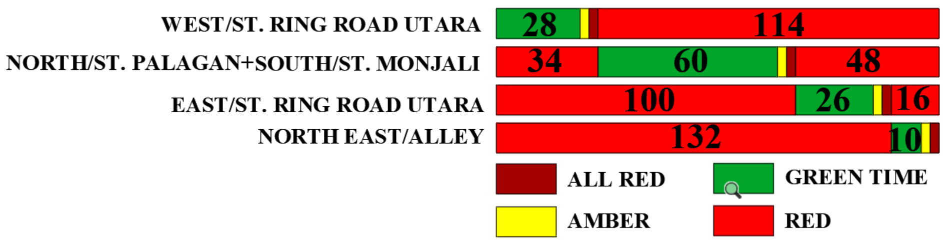

Figure 5.15 Signal Cycle Time Diagram of Monjali Intersection Alternative 3

5.6.2 Alternative 3 Modelling Using PTV VISSIM Software

Continuing the analysis, the cycle time of option 3 was determined using the earlier IHCM 1997 calculation. The original intersection road geometry design was used, and the cycle time was entered into the PTV VISSIM Model's signal control.

1. Parameter Input VISSIM
   a. Vehicle Input

      Using the original road network and the vehicle input is still separated, only the traffic signal is different so the south and north arm could passes through at green together.

Table 5.35 Total Vehicle Input for Alternative 3 in VISSIM

Arm	Vehicle Input
North	2490
West	6021
South	2383
East	3901
Alley	131

b. Driving Behavior

Prior to calibrating the model, driving behavior is also changed in this stage. The initial value from VISSIM was changed because partial vehicle input occurred during calibration on various routes, especially in the north and south arms of the intersection. Table 5.37 below displays the values for the parameter adjustments.

Table 5.36 Parameter on Driving Behavior Tab Adjustment Alternative 3

Parameter	Calibration Value	
	Before	After
Desired position at free flow	Middle of lane	Any
Overtake on same lane: on left & on right	off	on
Distance standing (at 0 kmph)(m)	1	0.4
Distance standing (at 50 kmph)(m)	1	0.4
Look ahead distance	400	250
Look back distance	400	250
Average standstill distance	2	0.45
Additive part of safety distance	2	0.45
Multiplicative part of safety distance	3	0.80
Waiting time before diffusion (s)	60	20
Min. headway (front/rear)(m)	0.5	0.3
Safety distance reduction factor	0.6	0.3

2. Result of Alternative 3 Modelling

As shown in Table 5.38 below, calibration was carried out multiple times based on the calibration of option 1, in which the model has already been updated. This process was completed until the value of GEH for each arm was no greater than 5%.

Table 5.37 Running Result of Alternative 3 Condition

Road	Qlen (m)	Vehs (All)	VehDelay (sec/pcu)
North	29.92	2441	4.39
West	45.58	5978	1.65

Continuation of Table 5.37 Running Result of Alternative 3 Condition

South	0.00	2358	0.01
East	49.03	3842	0.67
Alley	27.45	129	35.09

Based on the running result, the vehicle delay of each arm is calculated to obtain the delay value for the intersection.

Total delay = D x Q

= 4.39 x (1740/3600 seconds)

= 2.122

Table 5.38 Calculation result of VISSIM Total Delay Alternative 3

Arm	Dvissim	Q	D x Q
North	4.39	0.483	2.122
West	1.65	0.600	0.990
South	0.01	0.483	0.005
East	0.67	0.424	0.284
Alley	35.09	0.015	0.536
Total		2.006	3.937

So, the average delay for one intersection = sum of total delay / total flow

$$= \frac{\mathbf{3.937}}{\left(\mathbf{2.006}\ \frac{\mathbf{pcu}}{\mathbf{second}}\right)}$$

= 1.962 seconds/pcu

5.6.3 Impact Analysis of Alternative 3

The plan to change the protected flow into opposite flow for the north arm and south arm, based on the IHCM 1997 calculation only might decrease the volume of delay mathematically. Somehow, in real condition with the volume of south and north arms which both are not small, the safety will be at risk. Conflict occurs since the flow from north and south arms move at the same time. Despite of the best result of delay value, safety should be considered as well. This alternative phase is similar with the alternative 2 which the Sumberan alleyway has its own traffic signal and phase, as well as the LTOR system establishment.

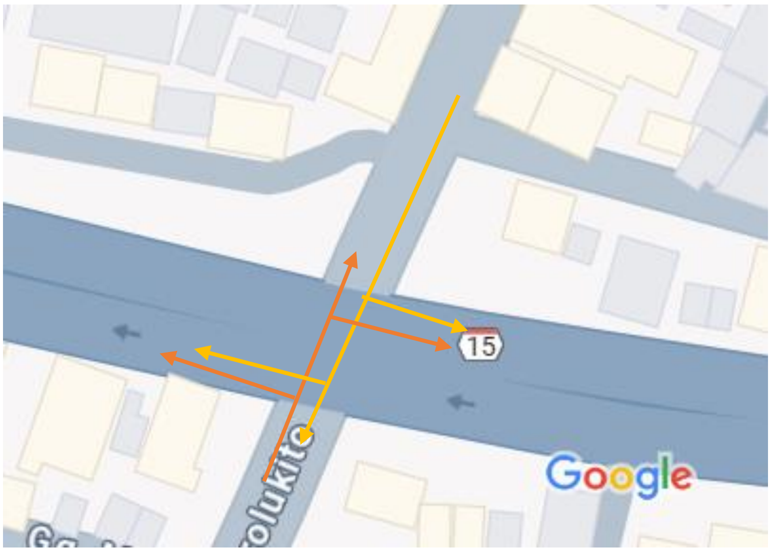

Figure 5.16 Conflict Occurrence Caused by Opposite Flow on Monjali Intersection

5.7 Recapitulation of Alternatives and Discussion

After calculating each condition from existing to alternative 3 using IHCM 1997 as well as calibrating using PTV VISSIM, the data of intersection delay (sec/pcu) is obtained and it can be seen in Table 5.39.

Table 5.39 Delay of Intersection Recapitulation of Each Condition from VISSIM

Intersection Delay		
Condition	IHCM 1997	VISSIM
	Delay (sec/pcu)	
Existing	386.314	14.558
Alternative 1	354.037	4.837
Alternative 2	198.520	5.436
Alternative 3	169.618	1.962

From the comparison result of IHCM 1997 and VISSIM above, the data of alternative 1 and 2 are inversely proportional. The influence of cycle time in alternative 1, the longer the cycle time the longer the delay will be. In alternative 1, the scenario of the existing condition without the alley causes the calculation of the

IHCM 1997 using the existing cycle time resulting in higher delay than the 5 phases cycle time (alternative 2). Meanwhile, the cycle time in alternative 2 is default from the IHCM 1997 calculation which resulting in smaller value of delay. Somehow, in the VISSIM modeling the five phases resulting in bigger value of delay since the alley has the biggest red time compared to other arms. By comparing the existing and all three alternatives, the smallest delay value from IHCM 1997 calculation and VISSIM modeling is the alternative 3 which uses opposite flow phase with delay intersection from IHCM 1997 is 169.918 seconds and from VISSIM is 1.962 seconds.

5.8 Fuel Consumption Analysis

5.8.1 Delay and Queue Length from Each Conditions from VISSIM Calibration

It is possible to obtain delays and queue lengths by firsthand observations made in the field. The amount of time the car waits in line from the moment the red light turns green (when it is stationary) until the very last car in the line begins to move again is how long the delay is measured. The front and back vehicles in the line, which are determined for each lane, show the delays that happen. On each lane, the last car in the queue is measured from the leading vehicle's stop line to determine the length of the queue. The last car to halt in a stationary vehicle is the definition of the last vehicle in the queue. Delay values and queue length values used in order to compare the fuel consumption are obtained from the results of VISSIM calibrations. In the previous calculation for each alternative, the total delay of intersection from each alternative was obtained and will be used for the fuel consumption comparison.

1. Total Delay of Intersection from VISSIM Calibration

   One of the outcomes of the VISSIM calibration is the vehicle delay result, which is computed to determine the intersection's overall delay under four different scenarios. Every condition has a vehicle delay result value however, in order to require the intersection's total delay value, the delay value from the VISSIM must be multiplied by flow (Q). The final total delay value will then be determined averaging the delay values. Calculation details from existing condition is shown in Table 5.15, alternative 1 in Table 5.24, alternative 2 in Table 5.31, and alternative 3 in Table 5.38. For the recapitulation of approach total delay in every condition is displayed in Table 5.40.

Table 5.40 Approach Total Delay Recapitulation from Each Condition

Arm/Condition	Total Delay (sec/pcu)			
	Existing (1)	Alternative 1 (2)	Alternative 2 (3)	Alternative 3 (4)
North	20.159	5.675	5.979	2.122
West	0.936	1.056	0.948	0.990
South	0.828	0.308	0.537	0.005
East	0.233	0.255	0.255	0.284
Alley	0.095		0.560	0.536
Average Delay	**14.281**	**4.837**	**5.436**	**1.962**

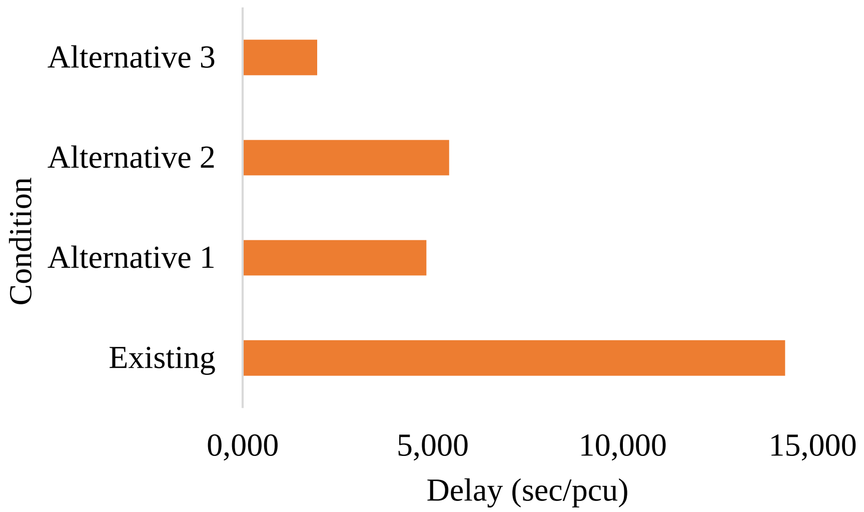

Figure 5.17 VISSIM Delay Comparison

2. From the calibration result of VISSIM from four condition of the intersection, the queue length (m) from each arm of four conditions design is displayed in the Table 5.41 below. All conditions do provide the result of queue length from the alley except alternative 1.

Table 5.41 Queue Length Results from VISSIM of Each Condition

Arm/Condition	Queue Length (m)			
	Existing (1)	Alternative 1 (2)	Alternative 2 (3)	Alternative 3 (4)
North	52.11	58.93	28.75	29.6
West	44.09	44.15	45.91	44.75
South	40.69	19.53	26.93	24.63
East	50.13	50.76	48.79	42.49
Alley	26.2	[redacted]	26.65	1.13

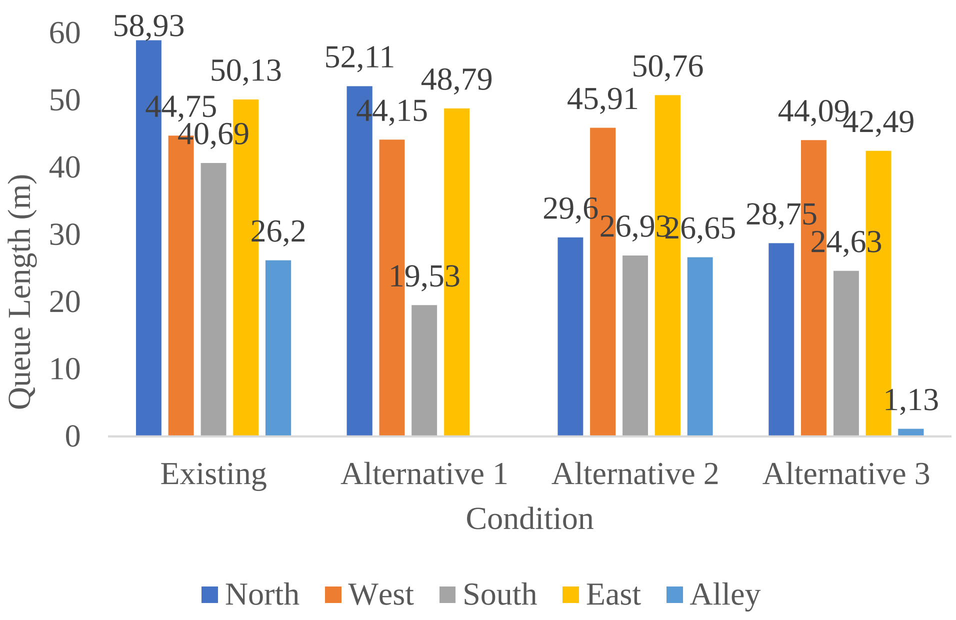

Figure 5.18 Queue Length Comparison of Each Condition

5.8.2 Fuel Consumption Calculation

The amount of fuel used is determined by measuring the duration of the vehicle's delay, or "stopped delay," in seconds which the condition of the vehicles is on idle. This information is then compared to the formula derived from LAPI-ITB, particularly in the middle of the trip, to determine the total amount of fuel needed as long as the vehicle experiences delays because of red lights. The more time the car is delayed, the more fuel it uses less efficiently.

1. Existing Condition Fuel Consumption Calculation Analysis

Example of calculations for fuel oil consumption at the Monjali north arm approach in existing condition. Find the total delay of each arm by multiplying the delay value and flow.

Total delay = D x Q

= 71.08 x (1021/3600 seconds)

= 20.159

After obtaining each arm total delay value, find the sum of the total delay as well as the total flow. So, the average delay for one intersection = sum of total delay / total flow

$$= \frac{\mathbf{22.254}}{\left(\mathbf{1.558}\,\frac{\mathbf{pcu}}{\mathbf{second}}\right)}$$

= 14.281 seconds/pcu

Using the value of the sum of total delay, input the value into the formula as follows.

$$\mathbf{F = 140x10^{-2}\left(\frac{liter}{pcu \times hour}\right) \times sum\ of\ total\ delay\ (second)}$$

$$\mathbf{F = 140x10^{-2}\left(\frac{liter}{pcu \times 3600}\right) \times 22.254\ seconds}$$

$$\mathbf{F = 0.00865\ liter/pcu}$$

With:

F = Fuel consumption on idle condition (liter/pcu).

Delay = Sum of total delay (second)

Table 5.42 Recapitulation of Fuel Consumption in Existing Condition

<table>
<tr><th>Arm</th><th>Total Delay (sec/pcu)</th><th>F (liter/pcu)</th></tr>
<tr><td>North</td><td>20.159</td><td rowspan="3">0.00865</td></tr>
<tr><td>West</td><td>0.936</td></tr>
<tr><td>South</td><td>0.828</td></tr>
<tr><td>East</td><td>0.233</td><td rowspan="3">0.00865</td></tr>
<tr><td>Alley</td><td>0.095</td></tr>
<tr><td>Total</td><td>22.252</td></tr>
<tr><td>D average</td><td>14.279</td><td></td></tr>
</table>

2. Alternative 1 Fuel Consumption Calculation Analysis

   The calculation is done with the same step and formula as written in the calculation example of existing condition. Below will be shown the recapitulation of fuel consumption in Alternative 1 in Table 5.43.

Table 5.43 Recapitulation of Fuel Consumption in Alternative 1

Arm	Total Delay (sec/pcu)	F (liter/pcu)
North	5.675	0.002836
West	1.056	
South	0.308	
East	0.255	
Total	7.293	
D average	4.837	

3. Alternative 2 Fuel Consumption Calculation Analysis

   The computation is carried out using the identical procedure and formula as stated in the calculation example for the current situation. Since the alley is included as the fifth phase, it is also computed, particularly for alternative 2. The summary of fuel use for alternative 2 in Table 5.44.

Table 5.44 Recapitulation of Fuel Consumption in Alternative 2

Arm	Total Delay (sec/pcu)	F (liter/pcu)
North	5.979	0.003002
West	0.948	
South	0.537	
East	0.255	0.003002
Alley	0.560	
Total	8.279	
D average	5.436	

4. Alternative 3 Fuel Consumption Calculation Analysis

For the alternative scenario of opposite flow phase, the calculation is performed using the same steps and formula as described in the calculation example. Below is a summary of alternative 3's fuel consumption from Table 5.45.

Table 5.45 Recapitulation of Fuel Consumption in Alternative 3

Arm	Total Delay (sec/pcu)	F (liter/pcu)
North	2.122	0.00153
West	0.990	
South	0.005	
East	0.284	
Alley	0.536	
Total	3.937	
D average	1.962	

5.8.3 Recapitulation of Delay and Fuel Consumption

Based on the calculation that was carried out from every condition, the result of delay and fuel were obtained in order to evaluate and compare the best scenario for the intersection. The comparison is displayed in Table 5.46 and Figure 5.15.

Table 5.46 Recapitulation of Delay Intersection and Fuel Consumption

Recapitulation of Delay Intersection and Fuel Consumption			
Condition	Delay (sec/pcu)	Fuel Consumption (liter/pcu)	Fuel Consumption (cc/pcu)
Exisitng	14.279	0.00865	8.653
Alternative1	4.837	0.00284	2.836
Alternative 2	5.436	0.00300	3.002
Alternative 3	1.962	0.00153	1.531

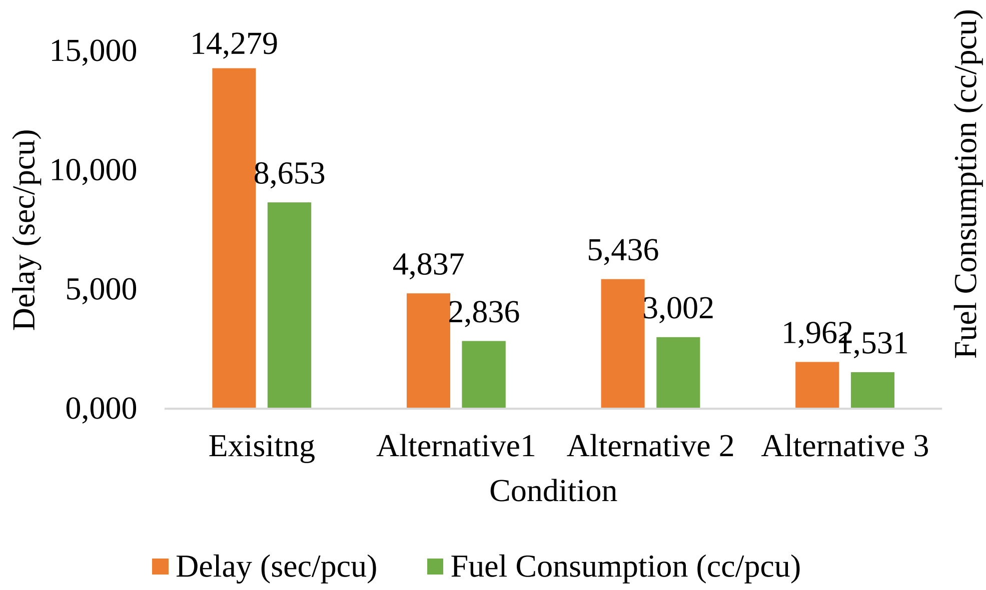

Figure 5.19 Comparison of Delay Intersection and Fuel Consumption

According to the calculation of the fuel consumption, it could be seen in the Figure 5.16 which shows the comparison of each condition's delay and fuel consumption. Such as the existing bar chart which shows with the delay of 14.279 second it has fuel consumption of 8.653 cc/pcu. The smallest delay value based on the analysis is the alternative 3 with delay of 1.962 second and fuel consumption of 1.531 cc/pcu which also becomes the best alternative for the intersection.

5.8 Discussion

In this study, a discussion is carried out to see the results of the theories that have been presented in the previous chapter.

1. From the first morning's peak hour analysis of the two field survey days that were carried out using the IHCM 1997 approach. The average delay time of each arm namely, north 1142037 sec/pcu, east 542996 sec/pcu, south 192741 sec/pcu, west 192741 sec/pcu. Saputri (2022), there was a 160-second delay at the Monjali crossing in the prior investigation, resulting in a LOS of F. In the meantime, the delay result in the current study, which also took the alley's

existence into account, is 386 seconds with a LOS of F, indicating that the situation has gotten worse.

2. Based on the findings of the IHCM 1997 method's calculation analysis and an intersection LOS value of F, the intersection requires an improvement solution. Closing the north arm alley, which is the primary source of traffic on the arm, is the first step in solving the problem (alternative 1). Adding a traffic signal to the alley to make the intersection have five phases is the second solution (alternative 2). and changing to opposite flow phase for the south and north arms (alternative 3).
3. After analyzing all alternatives by using IHCM 1997, it is found the value of intersection delay of the alternative consecutively from the alternative 1 until alternative 3 are 354 seconds, 198 seconds, and 169 seconds. Meanwhile, the intersection delay from VISSIM calibration are 4.837 seconds for alternative 1, 5.436 seconds for alternative 2, and 1.962 second for alternative 3.
4. Keeping with the fuel consumption of the peak hour data, the fuel consumption result in cc/pcu is obtained using the LAPI-ITB formula. As compared to the current state of 8.653 cc/pcu, alternative 1 has a fuel consumption of 2.836 cc/pcu, alternative 2 has 3.002 cc/pcu, and alternative 3 has 1.531 cc/pcu. The computation of alternative 3, which represents the scenario of opposite flow phase has the smallest delay value resulting in the most efficient fuel consumption compared to other alternatives.

CHAPTER 6
CONCLUSION AND SUGGESTION

6.1 Conclusion

Based on the data analysis and discussion that has been carried out, the following conclusions can be drawn conclusions are drawn as follows:

1. Saputri (2022), the previous investigation's Monjali crossing saw a 160-second delay, giving rise to a LOS of F. The current performance of the Monjali signalized intersection during peak hours in this study indicates that the intersection's capacity is saturated and unable to handle the current volume of traffic. This is evident from the average delay for the intersection with the presence of alleyway is 386 seconds and the degree of saturation value obtained larger than 0.85. The level of service value is F (extremely bad) > 60, since the state of the Monjali signalized intersection is over saturated.
2. The alternatives provided are closing the alleyway since that is the cause of the jam in the north arm by blocking the LTOR. The second alternative is by considering additional traffic signal for the alleyway so the intersection has five phases. The third alternative is to change the protected flow into opposite flow phase, precisely for the north arm and south arm since these arms has small right turn ratio. According to the analysis that was done, the best alternative is the third one which is changing the protected flow into opposite flow phase with the smallest delay intersection value, 1.962 second.
3. In the existing condition, the fuel consumption is less efficient with the delay of 14.279 second and fuel consumption of 8.653 cc/pcu. After the analysis was conducted, the third alternative with delay value of 1.962 second has the most efficient fuel consumption with a value of 1.531 cc/pcu.

6.2 Suggestion

Many suggestions can be made in light of the survey, data analysis, and debate. Some suggestions include:

1. Traffic regulation of vehicles, especially in the alley that covers the access of left-turning vehicles that will pass through the Monjali signalized intersection on the north arm. It is intended that the flow through these intersections is slightly reduced so as to reduce the degree of saturation and of course can reduce fuel oil consumption. reduce the degree of saturation and of course can reduce fuel oil consumption.
2. Cycle timing is no longer effective to do, because the intersection conditions are over saturated. Traffic in the city of Yogyakarta, especially the Monjali signalized intersection, requires additional regulations regarding the use of private vehicles that create congestion, by changing the use of private vehicles to public transportation, it can reduce fuel consumption as well.
3. For future research, it is expected to use more varied equations and other factors that affect fuel consumption, so that it is not limited to time delay. Using other methods in analyzing the effect of performance of the intersection on fuel consumption, can be developed by adding survey hours to get closer to real conditions and the number of intersections so that the results are more accurate and thorough. And also expected to continue this final project by develop a solution to change the geometric shape of the intersection from a level intersection into a non-intersection. With the availability of PTV VISSIM software, it is expected to help the modelling for intersection and other researches, with consideration of real conditions and modeling results that are still reasonable.

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ATTACHMENTS

Attachment 1

Primary Data of Number of Queue and Delay

Attachment 1 Session 1 of First Day Survey Number of Queue and Delay

SOUTH-1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	33	163
06.15 - 06.30	60	165
06.30 - 06.45	81	167
06.45 - 07.00	88	168
07.00 - 07.15	89	168
07.15 - 07.30	81	167
07.30 - 07.45	71	166
07.45 - 08.00	75	166

NORTH-1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	67	159
06.15 - 06.30	95	162
06.30 - 06.45	116	163
06.45 - 07.00	168	168
07.00 - 07.15	168	167
07.15 - 07.30	161	164
07.30 - 07.45	159	162
07.45 - 08.00	152	160

ALLEY-1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	5	155
06.15 - 06.30	15	158
06.30 - 06.45	18	159
06.45 - 07.00	22	160
07.00 - 07.15	22	160
07.15 - 07.30	16	157
07.30 - 07.45	14	157
07.45 - 08.00	12	156

EAST-1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	70	166
06.15 - 06.30	63	164
06.30 - 06.45	65	165
06.45 - 07.00	90	168
07.00 - 07.15	89	169
07.15 - 07.30	92	169
07.30 - 07.45	94	169
07.45 - 08.00	93	169

WEST-1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	68	167
06.15 - 06.30	85	168
06.30 - 06.45	93	169
06.45 - 07.00	91	169
07.00 - 07.15	89	169
07.15 - 07.30	90	169
07.30 - 07.45	89	169
07.45 - 08.00	91	169

Attachment 1 Session 2 of First Day Survey Number of Queue and Delay

SOUTH-2		
Time	NQ (m)	Delay (s)
11.00-11.15	82.5	153
11.15-11.30	65	151
11.30-11.45	88	153
11.45-12.00	74	152
12.00-12.15	88	154
12.15-12.30	88	154
12.30-12.45	78	152
12.45-13.00	86	153

NORTH-2		
Time	NQ (m)	Delay (s)
11.00-11.15	163	166
11.15-11.30	140	162
11.30-11.45	120	160
11.45-12.00	143	162
12.00-12.15	149	164
12.15-12.30	118	160
12.30-12.45	170	168
12.45-13.00	161	166

ALLEY-2		
Time	NQ (m)	Delay (s)
11.00-11.15	6	152
11.15-11.30	3	151
11.30-11.45	6	152
11.45-12.00	4	151
12.00-12.15	6	152
12.15-12.30	6	152
12.30-12.45	9	152
12.45-13.00	9	153

EAST-2		
Time	NQ (m)	Delay (s)
11.00-11.15	83	152
11.15-11.30	92	155
11.30-11.45	90	155
11.45-12.00	92	155
12.00-12.15	93	155
12.15-12.30	87	152
12.30-12.45	95	156
12.45-13.00	93	155

WEST-2		
Time	NQ (m)	Delay (s)
11.00-11.15	91	153
11.15-11.30	101	154
11.30-11.45	106	155
11.45-12.00	99	153
12.00-12.15	89	153
12.15-12.30	95	153
12.30-12.45	110	156
12.45-13.00	93	153

Attachment 1 Session 3 of First Day Survey Number of Queue and Delay

NORTH - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	173	169
16.15-16.30	177.0	170
16.30-16.45	154	165
16.45-17.00	162	166
17.00-17.15	150	166
17.15-17.30	159	166
17.30-17.45	127.5	164
17.45-18.00	80.0	162

ALLEY - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	6.2	160
16.15-16.30	7.6	161
16.30-16.45	6	160
16.45-17.00	7	161
17.00-17.15	15	164
17.15-17.30	10	162
17.30-17.45	10	162
17.45-18.00	16	164

SOUTH - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	98	163
16.15-16.30	93.3	162
16.30-16.45	96.0	163
16.45-17.00	93	162
17.00-17.15	99	163
17.15-17.30	96	163
17.30-17.45	100.0	163
17.45-18.00	110.0	164

WEST - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	98	165
16.15-16.30	100	165
16.30-16.45	115	166
16.45-17.00	91	165
17.00-17.15	99	165
17.15-17.30	104	165
17.30-17.45	83	164
17.45-18.00	81	164

EAST - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	102.5	165
16.15-16.30	93.0	165
16.30-16.45	97.5	165
16.45-17.00	109	166
17.00-17.15	101	165
17.15-17.30	102	165
17.30-17.45	91.7	164
17.45-18.00	87.5	164

Attachment 1 Session 1 of Second Day Survey Number of Queue and Delay

NORTH - 1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	30	155
06.15 - 06.30	36	155
06.30 - 06.45	50	163
06.45 - 07.00	54	157
07.00 - 07.15	54	158
07.15 - 07.30	58.3	167
07.30 - 07.45	60	168
07.45 - 08.00	74	169

ALLEY - 1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	4	157
06.15 - 06.30	5	160
06.30 - 06.45	7.2	160
06.45 - 07.00	8.2	160
07.00 - 07.15	11	163
07.15 - 07.30	9.2	162
07.30 - 07.45	13.8	164
07.45 - 08.00	8	160

SOUTH - 1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	25	163
06.15 - 06.30	22.5	163
06.30 - 06.45	37	164
06.45 - 07.00	35	165
07.00 - 07.15	64	167
07.15 - 07.30	58	167
07.30 - 07.45	58.3	167
07.45 - 08.00	74	168

WEST - 1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	60	166
06.15 - 06.30	65	166
06.30 - 06.45	74	167
06.45 - 07.00	69.2	167
07.00 - 07.15	67	167
07.15 - 07.30	58	165
07.30 - 07.45	80	168
07.45 - 08.00	110	170

EAST - 1		
Time	NQ (m)	Delay (s)
06.00 - 06.15	66.3	166
06.15 - 06.30	56	164
06.30 - 06.45	54	163
06.45 - 07.00	60	166
07.00 - 07.15	66	166
07.15 - 07.30	84	167
07.30 - 07.45	83	167
07.45 - 08.00	96	170

Attachment 1 Session 2 of Second Day Survey Number of Queue and Delay

NORTH - 2		
Time	NQ (m)	Delay (s)
11.00-11.15	201	158
11.15-11.30	228.6	162
11.30-11.45	245	162
11.45-12.00	266	164
12.00-12.15	284	167
12.15-12.30	255	163
12.30-12.45	262.5	164
12.45-13.00	269.2	164

ALLEY - 2		
Time	NQ (m)	Delay (s)
11.00-11.15	4	149
11.15-11.30	6	150
11.30-11.45	6.6	150
11.45-12.00	5	149
12.00-12.15	5.3	150
12.15-12.30	5.3	149
12.30-12.45	6.5	150
12.45-13.00	9.6	151

SOUTH - 2		
Time	NQ (m)	Delay (s)
11.00-11.15	124	151
11.15-11.30	125	151
11.30-11.45	126.7	152
11.45-12.00	119.2	151
12.00-12.15	121.7	152
12.15-12.30	118.3	151
12.30-12.45	120.8	152
12.45-13.00	111	148

WEST - 2		
Time	NQ (m)	Delay (s)
11.00-11.15	132.5	151
11.15-11.30	137.5	153
11.30-11.45	134.2	151
11.45-12.00	131.7	151
12.00-12.15	131.7	151
12.15-12.30	135	152
12.30-12.45	130.8	150
12.45-13.00	138.3	154

EAST - 2		
Time	NQ (m)	Delay (s)
11.00-11.15	124	151
11.15-11.30	130.8	152
11.30-11.45	131.7	152
11.45-12.00	123.3	151
12.00-12.15	126	151
12.15-12.30	135	154
12.30-12.45	129	152
12.45-13.00	129.2	152

Attachment 1 Session 3 of Second Day Survey Number of Queue and Delay

NORTH - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	218	167
16.15-16.30	244.2	168
16.30-16.45	225	168
16.45-17.00	187	168
17.00-17.15	128	166
17.15-17.30	128	166
17.30-17.45	126.7	165
17.45-18.00	77.5	160

ALLEY - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	22.5	165
16.15-16.30	24.7	166
16.30-16.45	23	165
16.45-17.00	14	162
17.00-17.15	17	162
17.15-17.30	20	165
17.30-17.45	15.5	162
17.45-18.00	13.2	161

SOUTH - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	137.5	168
16.15-16.30	123.0	168
16.30-16.45	110.8	164
16.45-17.00	109	164
17.00-17.15	134	168
17.15-17.30	130	168
17.30-17.45	64.0	162
17.45-18.00	82.0	163

WEST - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	139	168
16.15-16.30	138	168
16.30-16.45	140	168
16.45-17.00	140	168
17.00-17.15	138	168
17.15-17.30	143	168
17.30-17.45	137	167
17.45-18.00	142	169

EAST - 3		
Time	NQ (m)	Delay (s)
16.00-16.15	149	169
16.15-16.30	151.0	169
16.30-16.45	149	168
16.45-17.00	151	169
17.00-17.15	150	169
17.15-17.30	147	167
17.30-17.45	149.2	168
17.45-18.00	139.2	167

Attachment of Spot Speed Segment Method Peak Hour

Surveyor : Dika Kurniawan
Waktu : 06.30-06.45
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Timur-Barat

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	9	10.00	25	8	11.25	25	29	3.10
2	25	12	7.50	25	10	9.00	25	33	2.73
3	25	17	5.29	25	11	8.18	25	22	4.09
4	25	9	10.00	25	8	11.25	25		
5	25	16	5.63	25	17	5.29	25		
6	25	10	9.00	25	16	5.63	25		
7	25	12	7.50	25	5	18.00	25		
8	25	9	10.00	25	5	18.00	25		
9	25	17	5.29	25	18	5.00	25		
10	25	25	3.60	25	14	6.43	25		
11	25	22	4.09	25	20	4.50	25		
12	25	18	5.00	25	23	3.91	25		
13	25	10	9.00	25	14	6.43	25		
14	25	16	5.63	25	18	5.00	25		
15	25	5	18.00	25	19	4.74	25		
16	25	7	12.86	25	17	5.29	25		
17	25	17	5.29	25	10	9.00	25		
18	25	5	18.00	25	19	4.74	25		
19	25	8	11.25	25	12	7.50	25		
20	25	5	18.00	25	17	5.29	25		

Surveyor : Dika Kurniawan
Waktu : 06.45-07.00
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Timur-Barat

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	8	11.25	25	7	12.86	25	29	3.10
2	25	10	9.00	25	9	10.00	25	26	3.46
3	25	12	7.50	25	15	6.00	25	31	2.90
4	25	18	5.00	25	14	6.43	25	24	3.75
5	25	10	9.00	25	18	5.00	25		
6	25	15	6.00	25	12	7.50	25		
7	25	25	3.60	25	19	4.74	25		
8	25	22	4.09	25	22	4.09	25		
9	25	8	11.25	25	25	3.60	25		
10	25	10	9.00	25	24	3.75	25		
11	25	16	5.63	25	28	3.21	25		
12	25	6	15.00	25	19	4.74	25		
13	25	8	11.25	25	17	5.29	25		
14	25	12	7.50	25	12	7.50	25		
15	25	11	8.18	25	16	5.63	25		
16	25	18	5.00	25	21	4.29	25		
17	25	19	4.74	25	17	5.29	25		
18	25	12	7.50	25	13	6.92	25		
19	25	16	5.63	25	11	8.18	25		
20	25	8	11.25	25	21	4.29	25		

Surveyor : Dika Kurniawan
Waktu : 07.00-07.15
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Timur-Barat

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	10	9.00	25	11	8.18	25	19	4.74
2	25	11	8.18	25	12	7.50	25	26	3.46
3	25	18	5.00	25	18	5.00	25	29	3.10
4	25	17	5.29	25	17	5.29	25	23	3.91
5	25	13	6.92	25	20	4.50	25	22	4.09
6	25	9	10.00	25	22	4.09	25	28	3.21
7	25	11	8.18	25	16	5.63	25		
8	25	20	4.50	25	10	9.00	25		
9	25	24	3.75	25	7	12.86	25		
10	25	7	12.86	25	12	7.50	25		
11	25	19	4.74	25	19	4.74	25		
12	25	5	18.00	25	8	11.25	25		
13	25	10	9.00	25	9	10.00	25		
14	25	4	22.50	25	23	3.91	25		
15	25	6	15.00	25	22	4.09	25		
16	25	9	10.00	25	17	5.29	25		
17	25	7	12.86	25	16	5.63	25		
18	25	8	11.25	25	10	9.00	25		
19	25	11	8.18	25	19	4.74	25		
20	25	15	6.00	25	13	6.92	25		

Surveyor : Dika Kurniawan
Waktu : 07.15-07.30
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Timur-Barat

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	9	10.00	25	12	7.50	25	21	4.29
2	25	12	7.50	25	16	5.63	25	18	5.00
3	25	7	12.86	25	19	4.74	25	19	4.74
4	25	8	11.25	25	10	9.00	25	29	3.10
5	25	4	22.50	25	11	8.18	25	26	3.46
6	25	11	8.18	25	9	10.00	25		
7	25	6	15.00	25	12	7.50	25		
8	25	14	6.43	25	21	4.29	25		
9	25	10	9.00	25	17	5.29	25		
10	25	11	8.18	25	13	6.92	25		
11	25	4	22.50	25	12	7.50	25		
12	25	16	5.63	25	10	9.00	25		
13	25	7	12.86	25	15	6.00	25		
14	25	6	15.00	25	9	10.00	25		
15	25	9	10.00	25	7	12.86	25		
16	25	12	7.50	25	12	7.50	25		
17	25	17	5.29	25	13	6.92	25		
18	25	15	6.00	25	9	10.00	25		
19	25	9	10.00	25	8	11.25	25		
20	25	12	7.5	25	11	8.18	25		

Surveyor :
Waktu : 06.30-06.45
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Barat-Timur

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	6	15.00	25	9	10.00	25	19	4.74
2	25	7	12.86	25	5	18.00	25		
3	25	12	7.50	25	8	11.25	25		
4	25	7	12.86	25	12	7.50	25		
5	25	5	18.00	25	16	5.63	25		
6	25	9	10.00	25	18	5.00	25		
7	25	7	12.86	25	12	7.50	25		
8	25	4	22.50	25	19	4.74	25		
9	25	9	10.00	25	22	4.09	25		
10	25	10	9.00	25	26	3.46	25		
11	25	16	5.63	25	13	6.92	25		
12	25	14	6.43	25	16	5.63	25		
13	25	12	7.50	25	21	4.29	25		
14	25	18	5.00	25	26	3.46	25		
15	25	8	11.25	25	12	7.50	25		
16	25	9	10.00	25	8	11.25	25		
17	25	10	9.00	25	9	10.00	25		
18	25	14	6.43	25	6	15.00	25		
19	25	12	7.50	25	13	6.92	25		
20	25	8	11.25	25	12	7.50	25		

Surveyor :
Waktu : 06.45-07.00
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Barat-Timur

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	12	7.50	25	7	12.86	25	20	4.50
2	25	18	5.00	25	7	12.86	25	17	5.29
3	25	12	7.50	25	12	7.50	25		
4	25	4	22.50	25	19	4.74	25		
5	25	5	18.00	25	8	11.25	25		
6	25	6	15.00	25	9	10.00	25		
7	25	12	7.50	25	9	10.00	25		
8	25	15	6.00	25	8	11.25	25		
9	25	19	4.74	25	10	9.00	25		
10	25	12	7.50	25	12	7.50	25		
11	25	9	10.00	25	10	9.00	25		
12	25	13	6.92	25	9	10.00	25		
13	25	16	5.63	25	13	6.92	25		
14	25	18	5.00	25	12	7.50	25		
15	25	12	7.50	25	9	10.00	25		
16	25	16	5.63	25	9	10.00	25		
17	25	10	9.00	25	10	9.00	25		
18	25	12	7.50	25	8	11.25	25		
19	25	16	5.63	25	8	11.25	25		
20	25	9	10.00	25	8	11.25	25		

Surveyor :
Waktu : 07.00-07.15
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Barat-Timur

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	5	18.00	25	17	5.29	25	17	5.29
2	25	7	12.86	25	9	10.00	25	19	4.74
3	25	15	6.00	25	25	3.60	25	23	3.91
4	25	8	11.25	25	11	8.18	25		
5	25	9	10.00	25	8	11.25	25		
6	25	6	15.00	25	18	5.00	25		
7	25	10	9.00	25	15	6.00	25		
8	25	12	7.50	25	22	4.09	25		
9	25	3	30.00	25	8	11.25	25		
10	25	11	8.18	25	16	5.63	25		
11	25	10	9.00	25	19	4.74	25		
12	25	14	6.43	25	20	4.50	25		
13	25	9	10.00	25	24	3.75	25		
14	25	15	6.00	25	15	6.00	25		
15	25	6	15.00	25	21	4.29	25		
16	25	7	12.86	25	14	6.43	25		
17	25	12	7.50	25	17	5.29	25		
18	25	5	18.00	25	13	6.92	25		
19	25	13	6.92	25	25	3.60	25		
20	25	7	12.86	25	19	4.74	25		

Surveyor :
Waktu : 07.15-07.30
Hari/Tanggal : Rabu 16 Agustus 2023
Arah : Barat-Timur

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	5	18.00	25	9	10.00	25	17	5.29
2	25	7	12.86	25	9	10.00	25	19	4.74
3	25	9	10.00	25	8	11.25	25	23	3.91
4	25	6	15.00	25	9	10.00	25	17	5.29
5	25	10	9.00	25	9	10.00	25	20	4.50
6	25	16	5.63	25	10	9.00	25		
7	25	12	7.50	25	10	9.00	25		
8	25	4	22.50	25	11	8.18	25		
9	25	15	6.00	25	9	10.00	25		
10	25	11	8.18	25	9	10.00	25		
11	25	12	7.50	25	8	11.25	25		
12	25	16	5.63	25	9	10.00	25		
13	25	19	4.74	25	9	10.00	25		
14	25	17	5.29	25	10	9.00	25		
15	25	11	8.18	25	9	10.00	25		
16	25	12	7.50	25	9	10.00	25		
17	25	10	9.00	25	7	12.86	25		
18	25	9	10.00	25	9	10.00	25		
19	25	11	8.18	25	11	8.18	25		
20	25	5	18.00	25	13	6.92	25		

Surveyor :
Waktu : 06.30-06.45
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Utara-Selatan

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	5	18,00	25	7	12,86	25		
2	25	7	12,86	25	6	15,00	25		
3	25	13	6,92	25	9	10,00	25		
4	25	7	12,86	25	11	8,18	25		
5	25	8	11,25	25	14	6,43	25		
6	25	4	22,50	25	8	11,25	25		
7	25	10	9,00	25	6	15,00	25		
8	25	19	4,74	25	15	6,00	25		
9	25	8	11,25	25	24	3,75	25		
10	25	22	4,09	25	21	4,29	25		
11	25	4	22,50	25	30	3,00	25		
12	25	6	15,00	25	23	3,91	25		
13	25	8	11,25	25	15	6,00	25		
14	25	11	8,18	25	18	5,00	25		
15	25	4	22,50	25	9	10,00	25		
16	25	9	10,00	25	17	5,29	25		
17	25	13	6,92	25	8	11,25	25		
18	25	6	15,00	25	7	12,86	25		
19	25	4	22,50	25			25		

20	25	9	10,00	25			25		

Surveyor :
Waktu : 06.45-07.00
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Utara-Selatan

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	5	18,00	25	6	15,00	25		
2	25	7	12,86	25	7	12,86	25		
3	25	5	18,00	25	6	15,00	25		
4	25	8	11,25	25	8	11,25	25		
5	25	11	8,18	25	18	5,00	25		
6	25	12	7,50	25	25	3,60	25		
7	25	6	15,00	25	12	7,50	25		
8	25	7	12,86	25	14	6,43	25		
9	25	7	12,86	25	10	9,00	25		
10	25	9	10,00	25	9	10,00	25		
11	25	6	15,00	25	17	5,29	25		
12	25	7	12,86	25	20	4,50	25		
13	25	9	10,00	25			25		
14	25	9	10,00	25			25		
15	25	14	6,43	25			25		
16	25	6	15,00	25			25		
17	25	6	15,00	25			25		
18	25	11	8,18	25			25		
19	25	18	5,00	25			25		

20	25	9	10,00	25			25		

Surveyor :
Waktu : 07.00-07.15
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Utara-Selatan

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	6	15,00	25	9	10,00	25		
2	25	5	18,00	25	6	15,00	25		
3	25	5	18,00	25	12	7,50	25		
4	25	8	11,25	25	8	11,25	25		
5	25	8	11,25	25	28	3,21	25		
6	25	7	12,86	25	18	5,00	25		
7	25	9	10,00	25	9	10,00	25		
8	25	12	7,50	25	9	10,00	25		
9	25	11	8,18	25	13	6,92	25		
10	25	6	15,00	25	11	8,18	25		
11	25	8	11,25	25	10	9,00	25		
12	25	7	12,86	25	8	11,25	25		
13	25	8	11,25	25	9	10,00	25		
14	25	15	6,00	25	9	10,00	25		
15	25	7	12,86	25	8	11,25	25		
16	25	10	9,00	25	14	6,43	25		
17	25	12	7,50	25	16	5,63	25		
18	25	9	10,00	25	11	8,18	25		

19	25	18	5,00	25	30	3,00	25		
20	25	7	12,86	25	21	4,29	25		

Surveyor :
Waktu : 07.15-07.30
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Utara-Selatan

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	4	22,50	25	6	15,00	25		
2	25	6	15,00	25	6	15,00	25		
3	25	6	15,00	25	7	12,86	25		
4	25	7	12,86	25	9	10,00	25		
5	25	11	8,18	25	11	8,18	25		
6	25	10	9,00	25	16	5,63	25		
7	25	9	10,00	25	15	6,00	25		
8	25	9	10,00	25	9	10,00	25		
9	25	9	10,00	25	11	8,18	25		
10	25	8	11,25	25	14	6,43	25		
11	25	14	6,43	25	8	11,25	25		
12	25	10	9,00	25	8	11,25	25		
13	25	9	10,00	25	17	5,29	25		
14	25	13	6,92	25	21	4,29	25		
15	25	15	6,00	25	13	6,92	25		
16	25	8	11,25	25			25		
17	25	6	15,00	25			25		

18	25			25			25		
19	25			25			25		
20	25			25			25		

Surveyor :
Waktu : 06.30-06.45
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Selatan-Utara

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	4	22,50	25	8	11,25	25		
2	25	4	22,50	25	9	10,00	25		
3	25	4	22,50	25	9	10,00	25		
4	25	7	12,86	25	9	10,00	25		
5	25	7	12,86	25	12	7,50	25		
6	25	5	18,00	25	14	6,43	25		
7	25	6	15,00	25	15	6,00	25		
8	25	9	10,00	25	9	10,00	25		
9	25	9	10,00	25	9	10,00	25		
10	25	12	7,50	25	10	9,00	25		
11	25	15	6,00	25	10	9,00	25		
12	25	9	10,00	25	21	4,29	25		
13	25	11	8,18	25	20	4,50	25		

14	25	16	5,63	25	12	7,50	25		
15	25	7	12,86	25	18	5,00	25		
16	25	9	10,00	25	9	10,00	25		
17	25	9	10,00	25	11	8,18	25		
18	25	8	11,25	25	8	11,25	25		
19	25	10	9,00	25	12	7,50	25		
20	25	8	11,25	25	12	7,50	25		

Surveyor :
Waktu : 06.45-07.00
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Selatan-Utara

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	8	11,25	25	7	12,86	25		
2	25	11	8,18	25	7	12,86	25		
3	25	8	11,25	25	12	7,50	25		
4	25	8	11,25	25	21	4,29	25		
5	25	8	11,25	25	8	11,25	25		
6	25	9	10,00	25	9	10,00	25		
7	25	11	8,18	25	9	10,00	25		
8	25	9	10,00	25	8	11,25	25		
9	25	9	10,00	25	10	9,00	25		
10	25	9	10,00	25	12	7,50	25		
11	25	9	10,00	25	10	9,00	25		
12	25	8	11,25	25	9	10,00	25		

13	25	10	9,00	25	13	6,92	25		
14	25	9	10,00	25	12	7,50	25		
15	25	11	8,18	25	9	10,00	25		
16	25	8	11,25	25	9	10,00	25		
17	25	7	12,86	25	10	9,00	25		
18	25	7	12,86	25	8	11,25	25		
19	25	7	12,86	25	8	11,25	25		
20	25	9	10,00	25	8	11,25	25		

Surveyor :
Waktu : 07.00-07.15
Hari/Tanggal : Rabu/16 Agustus 2023
Arah : Selatan-Utara

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	5	18,00	25	10	9,00	25		
2	25	6	15,00	25	9	10,00	25		
3	25	6	15,00	25	9	10,00	25		
4	25	6	15,00	25	10	9,00	25		
5	25	5	18,00	25	11	8,18	25		
6	25	5	18,00	25	12	7,50	25		
7	25	8	11,25	25	9	10,00	25		
8	25	8	11,25	25	9	10,00	25		
9	25	8	11,25	25	13	6,92	25		
10	25	9	10,00	25	13	6,92	25		
11	25	10	9,00	25	14	6,43	25		

12	25	11	8,18	25	16	5,63	25		
13	25	10	9,00	25	21	4,29	25		
14	25	9	10,00	25	9	10,00	25		
15	25	9	10,00	25	15	6,00	25		
16	25	8	11,25	25	9	10,00	25		
17	25	9	10,00	25	9	10,00	25		
18	25	6	15,00	25	8	11,25	25		
19	25	7	12,86	25	8	11,25	25		
20	25	7	12,86	25	11	8,18	25		

Surveyor :
Waktu : 07.15-07.30
Hari/Tanggal : Rabu 16 Agustus 2023
Arah : Selatan-Utara

Jenis Kendaraan	MC			LV			HV		
	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)	Panjang Segmen (m)	Waktu Tempuh (det)	Kecepatan (m/d)
1	25	4	22,50	25	9	10,00	25		
2	25	4	22,50	25	9	10,00	25		
3	25	5	18,00	25	8	11,25	25		
4	25	5	18,00	25	9	10,00	25		
5	25	7	12,86	25	9	10,00	25		
6	25	6	15,00	25	10	9,00	25		
7	25	6	15,00	25	10	9,00	25		
8	25	6	15,00	25	11	8,18	25		
9	25	5	18,00	25	9	10,00	25		
10	25	7	12,86	25	9	10,00	25		

11	25	9	10,00	25	8	11,25	25		
12	25	9	10,00	25	9	10,00	25		
13	25	7	12,86	25	9	10,00	25		
14	25	7	12,86	25	10	9,00	25		
15	25	8	11,25	25	9	10,00	25		
16	25	8	11,25	25	9	10,00	25		
17	25	5	18,00	25	7	12,86	25		
18	25	5	18,00	25	9	10,00	25		
19	25	6	15,00	25	11	8,18	25		
20	25	5	18,00	25	13	6,92	25		

Attachment 2

Secondary Data of Traffic Volume of Sleman District Transportation Office

Attachment 2 Analysis Data of Sleman Regency Road Segment

No Ruas	Ruas Jalan	Jam Puncak			Volume Jam Puncak			Kecepatan Arus Bebas	Kapasitas			Derajat Kejenuhan			V	DS MAX	Rank
		Pagi	Siang	Sore	Pagi	Siang	Sore		Pagi	Siang	Sore	Pagi	Siang	Sore			
					(smp/jam)	(smp/jam)	(smp/jam)	(km/jam)	(smp/jam)	(smp/jam)	(smp/jam)				(km/jam)		
Kabupaten Sleman																	
83	Yogyakarta - Pulowatu	06.45 - 07.45	12.00 - 13.00	16.30 - 17.30	1284	1155	1442	52,47	2401	2329	2185	0,54	0,5	0,66	24,00	0,66	22
84	Yogyakarta - Kaliurang (Gardu PLN)	06.30 - 07.30	11.15 - 12.15	16.45 - 17.45	1524	1287	1954	49,01	2043	2177	2177	0,76	0,59	0,9	20,00	0,90	1
84	Yogyakarta - Kaliurang (RSJ Grasia)	07.30 - 08.30	12.00 - 13.00	16.15 - 17.15	781	747	786	49,01	2170	2170	2105	0,38	0,34	0,38	45,00	0,38	80
85	Yogyakarta -Kebonagung 1 (DPN Giant Swalayan)	07.00 - 08.00	11.00 - 12.00	16.45 - 17.45	1018	1005	1806	55,35	2089	2021	2021	0,49	0,50	0,90	25,00	0,90	1
85	Yogyakarta -Kebonagung 1 (Ruko Bantulan)	07.00 - 08.00	11.00 - 12.00	16.30 - 17.30	1651	1481	1833	55,01	2185	2401	2113	0,77	0,62	0,87	23,00	0,87	5
85	Yogyakarta -Kebonagung 1 (Sentra Genteng)	06.15 - 07.15	12.00 - 13.00	16.30 - 17.30	1136	773	1171	55,01	2159	2380	2159	0,55	0,34	0,55	29,00	0,55	41
86	Prambanan - Piyungan (Dpn UD Kembar)	06.45 - 07.45	11.00 - 12.00	16.15 - 17.15	1598	880	1480	52,47	2519	2680	2600	0,64	0,33	0,57	38,00	0,64	25
87	Klangon - Tempel (Kecamatan Moyudan)	06.45 - 07.45	11.15 - 12.15	16.45 - 17.45	818	594	783	45,70	1888	1888	1888	0,44	0,32	0,42	41,00	0,44	65
87	Klangon - Tempel (mang engking)	07.30 - 08.30	11.00 - 12.00	16.15 - 17.15	960	425	483	48,21	2053	1992	1992	0,47	0,21	0,24	39,00	0,47	59
88	Mlati - Cebongan (Dpn Sego Pecel Ndoble)	06.00 - 07.00	11.00 - 12.00	17.00 - 18.00	1539	877	1156	45,70	2413	2490	2413	0,65	0,36	0,49	28,00	0,65	23
89	Cebongan - Seyegan (Dpn Martabak & Terang Bulan	07.30 - 08.30	11.00 - 12.00	16.30 - 17.30	1311	482	882	51,38	2184	2184	2044	0,61	0,22	0,43	35,00	0,61	31
90	Seyegan - Balangan (Dpn Nita Busana)	08.00 - 09.00	11.30 - 12.30	15.00 - 16.00	1043	970	1060	52,47	2567	2567	2567	0,41	0,38	0,41	32,00	0,41	72
91	Balangan – Kebonagung 2 (Sugeng Motor)	07.45 - 08.45	11.30 - 12.30	16.30 - 17.30	1097	741	844	55,01	2821	2821	2736	0,39	0,26	0,31	37,00	0,39	76
92	Tangisan - Blaburan (Dpn Balai Desa Bligo)	06.30 - 07.30	11.15 - 12.15	15.15 - 16.15	587	519	571	55,35	2624	2624	2624	0,23	0,2	0,22	41,00	0,23	95

Attachment 2 Analysis Data of Sleman Regency Road Segment

Nama Ruas

TIME SLICE		KENDARAAN BERMOTOR			KENDARAAN TIDAK BERMOTOR
Jam	Menit	Sepeda Motor	Mobil	Truk Besar	
06.00 - 07.00	06.00 - 06.15	267	192	0	0
	06.15 - 06.30	298	203	1	0
	06.30 - 06.45	352	269	0	0
	06.45 - 07.00	399	298	1	0
07.00 - 08.00	07.00 - 07.15	493	321	0	0
	07.15 - 07.30	573	442	0	0
	07.30 - 07.45	632	389	0	0
	07.45 - 08.00	693	353	0	0
08.00 - 09.00	08.00 - 08.15	632	301	0	0
	08.15 - 08.30	593	295	0	0
	08.30 - 08.45	532	283	0	0
	08.45 - 09.00	432	253	0	0
09.00 - 10.00	09.00 - 09.15	455	219	1	0
	09.15 - 09.30	421	226	1	0
	09.30 - 09.45	400	178	0	0
	09.45 - 10.00	364	194	0	0
10.00 - 11.00	10.00 - 10.15	321	200	0	0
	10.15 - 10.30	282	192	0	0
	10.30 - 10.45	277	189	0	0
	10.45 - 11.00	269	192	0	0
11.00 - 12.00	11.00 - 11.15	254	194	1	0
	11.15 - 11.30	248	201	0	0
	11.30 - 11.45	203	191	0	0
	11.45 - 12.00	231	215	0	0
12.00 - 13.00	12.00 - 12.15	156	212	0	0
	12.15 - 12.30	122	202	0	0
	12.30 - 12.45	142	187	0	0
	12.45 - 13.00	110	191	0	0
13.00 - 14.00	13.00 - 13.15	104	227	1	0
	13.15 - 13.30	67	182	0	0
	13.30 - 13.45	55	189	0	0
	13.45 - 14.00	56	200	0	0
14.00 - 15.00	14.00 - 14.15	122	197	0	0
	14.15 - 14.30	176	195	0	0
	14.30 - 14.45	145	215	0	0
	14.45 - 15.00	183	199	0	0
15.00 - 16.00	15.00 - 15.15	266	103	0	0
	15.15 - 15.30	290	231	0	0
	15.30 - 15.45	327	200	0	0
	15.45 - 16.00	380	290	0	0

Attachment 2 Analysis Data of Sleman Regency Road Segment

16.00 - 17.00	16.00 - 16.15	421	278	0	0
	16.15 - 16.30	502	247	0	0
	16.30 - 16.45	483	264	0	0
	16.45 - 17.00	506	314	0	0
17.00 - 18.00	17.00 - 17.15	477	327	0	0
	17.15 - 17.30	431	287	1	0
	17.30 - 17.45	458	196	0	0
	17.45 - 18.00	383	174	0	0
TOTAL (Kendaraan)		**15,983**	**11,297**	**7**	**-**

TIME SLICE		KENDARAAN BERMOTOR			KENDARAAN TIDAK BERMOTOR
		ANGKUTAN PRIBADI			
Jam	Menit	Sepeda Motor	Mobil	Truk Besar	
06.00 - 07.00	06.00 - 06.15	67	192	-	-
	06.15 - 06.30	75	203	1	-
	06.30 - 06.45	88	269	-	-
	06.45 - 07.00	100	298	1	-
07.00 - 08.00	07.00 - 07.15	123	321	-	-
	07.15 - 07.30	143	442	-	-
	07.30 - 07.45	158	389	-	-
	07.45 - 08.00	173	353	-	-
08.00 - 09.00	08.00 - 08.15	158	301	-	-
	08.15 - 08.30	148	295	-	-
	08.30 - 08.45	133	283	-	-
	08.45 - 09.00	108	253	-	-
09.00 - 10.00	09.00 - 09.15	114	219	1	-
	09.15 - 09.30	105	226	1	-
	09.30 - 09.45	100	178	-	-
	09.45 - 10.00	91	194	-	-
10.00 - 11.00	10.00 - 10.15	80	200	-	-
	10.15 - 10.30	71	192	-	-
	10.30 - 10.45	69	189	-	-
	10.45 - 11.00	67	192	-	-
11.00 - 12.00	11.00 - 11.15	64	194	1	-
	11.15 - 11.30	62	201	-	-
	11.30 - 11.45	51	191	-	-
	11.45 - 12.00	58	215	-	-
12.00 - 13.00	12.00 - 12.15	39	212	-	-
	12.15 - 12.30	31	202	-	-
	12.30 - 12.45	36	187	-	-
	12.45 - 13.00	28	191	-	-
13.00 - 14.00	13.00 - 13.15	26	227	1	-
	13.15 - 13.30	17	182	-	-
	13.30 - 13.45	14	189	-	-

Attachment 2 Analysis Data of Sleman Regency Road Segment

	13.45 - 14.00	14	200	-	-
14.00 - 15.00	14.00 - 14.15	31	197	-	-
	14.15 - 14.30	44	195	-	-
	14.30 - 14.45	36	215	-	-
	14.45 - 15.00	46	199	-	-
15.00 - 16.00	15.00 - 15.15	67	103	-	-
	15.15 - 15.30	73	231	-	-
	15.30 - 15.45	82	200	-	-
	15.45 - 16.00	95	290	-	-
16.00 - 17.00	16.00 - 16.15	105	278	-	-
	16.15 - 16.30	126	247	-	-
	16.30 - 16.45	121	264	-	-
	16.45 - 17.00	127	314	-	-
17.00 - 18.00	17.00 - 17.15	119	327	-	-
	17.15 - 17.30	108	287	1	-
	17.30 - 17.45	115	196	-	-
	17.45 - 18.00	96	174	-	-
TOTAL (smp)		**3,996**	**11,297**	**8**	**-**

Attachment 2 Analysis Data of Sleman Regency Road Segment

U

Nama Ruas

TIME SLICE		KENDARAAN BERMOTOR			KENDARAAN TIDAK BERMOTOR
Jam	Menit	Sepeda Motor	Mobil	Truk Besar	
06.00 - 07.00	06.00 - 06.15	383	136	0	0
	06.15 - 06.30	403	148	0	0
	06.30 - 06.45	520	255	0	0
	06.45 - 07.00	616	259	0	0
07.00 - 08.00	07.00 - 07.15	562	279	0	0
	07.15 - 07.30	583	264	0	0
	07.30 - 07.45	621	255	0	0
	07.45 - 08.00	694	145	0	0
08.00 - 09.00	08.00 - 08.15	544	147	0	0
	08.15 - 08.30	531	102	0	0
	08.30 - 08.45	504	111	0	0
	08.45 - 09.00	486	114	0	0
09.00 - 10.00	09.00 - 09.15	479	123	0	0
	09.15-09.30	422	112	0	0
	09.30 - 09.45	366	103	0	0
	09.45 - 10.00	289	98	0	0
10.00 - 11.00	10.00 - 10.15	267	115	0	0
	10.15 - 10.30	311	109	0	0
	10.30 - 10.45	283	97	0	0
	10.45 - 11.00	314	128	0	0
11.00 - 12.00	11.00 - 11.15	387	137	0	0
	11.15 - 11.30	301	133	0	0
	11.30 - 11.45	334	123	0	0
	11.45 - 12.00	356	127	0	0
12.00 - 13.00	12.00 - 12.15	367	135	0	0
	12.15 - 12.30	332	131	0	0
	12.30 - 12.45	223	125	0	0
	12.45 - 13.00	411	133	0	0
13.00 - 14.00	13.00 - 13.15	362	124	0	0
	13.15 - 13.30	376	110	0	0
	13.30 - 13.45	384	125	0	0
	13.45 - 14.00	265	113	0	0
14.00 - 15.00	14.00 - 14.15	276	102	0	0
	14.15 - 14.30	254	116	0	0
	14.30 - 14.45	267	124	0	0
	14.45 - 15.00	278	122	0	0
	15.00 - 15.15	328	111	0	0

Attachment 2 Analysis Data of Sleman Regency Road Segment

15.00 - 16.00	15.15-15.30	353	122	0	0
	15.30-15.45	437	123	0	0
	15.45-16.00	483	133	0	0
16.00 - 17.00	16.00-16.15	504	145	0	0
	16.15 - 16.30	589	177	0	0
	16.30 - 16.45	544	211	0	0
	16.45 - 17.00	483	246	0	0
17.00 - 18.00	17.00 - 17.15	579	221	0	0
	17.15 - 17.30	432	224	0	0
	17.30 - 17.45	421	223	0	0
	17.45 - 18.00	431	234	0	0
TOTAL (Kendaraan)		**19,935**	**7,250**	**-**	**-**

TIME SLICE		KENDARAAN BERMOTOR			KENDARAAN TIDAK BERMOTOR
		ANGKUTAN PRIBADI			
Jam	Menit	Sepeda Motor	Mobil	Truk Besar	
06.00 - 07.00	06.00 - 06.15	96	136	-	-
	06.15 - 06.30	101	148	-	-
	06.30 - 06.45	130	255	-	-
	06.45 - 07.00	154	259	-	-
07.00 - 08.00	07.00 - 07.15	141	279	-	-
	07.15 - 07.30	146	264	-	-
	07.30 - 07.45	155	255	-	-
	07.45 - 08.00	174	145	-	-
08.00 - 09.00	08.00 - 08.15	136	147	-	-
	08.15 - 08.30	133	102	-	-
	08.30 - 08.45	126	111	-	-
	08.45 - 09.00	122	114	-	-
09.00 - 10.00	09.00 - 09.15	120	123	-	-
	09.15 - 09.30	106	112	-	-
	09.30 - 09.45	92	103	-	-
	09.45 - 10.00	72	98	-	-
10.00 - 11.00	10.00 - 10.15	67	115	-	-
	10.15 - 10.30	78	109	-	-
	10.30 - 10.45	71	97	-	-
	10.45 - 11.00	79	128	-	-
11.00 - 12.00	11.00 - 11.15	97	137	-	-
	11.15 - 11.30	75	133	-	-
	11.30 - 11.45	84	123	-	-
	11.45 - 12.00	89	127	-	-
12.00 - 13.00	12.00 - 12.15	92	135	-	-
	12.15 - 12.30	83	131	-	-
	12.30 - 12.45	56	125	-	-

Attachment 2 Analysis Data of Sleman Regency Road Segment

	12.45 - 13.00	103	133	-	-
13.00 - 14.00	13.00 - 13.15	91	124	-	-
	13.15 - 13.30	94	110	-	-
	13.30 - 13.45	96	125	-	-
	13.45 - 14.00	66	113	-	-
14.00 - 15.00	14.00 - 14.15	69	102	-	-
	14.15 - 14.30	64	116	-	-
	14.30 - 14.45	67	124	-	-
	14.45 - 15.00	70	122	-	-
15.00 - 16.00	15.00 - 15.15	82	111	-	-
	15.15 - 15.30	88	122	-	-
	15.30 - 15.45	109	123	-	-
	15.45 - 16.00	121	133	-	-
16.00 - 17.00	16.00 - 16.15	126	145	-	-
	16.15 - 16.30	147	177	-	-
	16.30 - 16.45	136	211	-	-
	16.45 - 17.00	121	246	-	-
17.00 - 18.00	17.00 - 17.15	145	221	-	-
	17.15 - 17.30	108	224	-	-
	17.30 - 17.45	105	223	-	-
	17.45 - 18.00	108	234	-	-
TOTAL (smp)		**4,984**	**7,250**	**-**	**-**