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THESIS

IMPROVEMENT OF THE MAJOR INTERSECTIONS IN YANGON

AUNG KYAW HAN

A Thesis Submitted in Partial Fulfillment of
the Requirements for the Degree of
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With the increase in demand on the current roadway systems, intersections in Yangon nowadays are found not fulfilling the standard requirements of the design period. When the performance of existing intersections reaches an unacceptable level, typically, the treatment is to transform the existing intersections into channelized and roundabout intersections to enhance both the capacity and level of service. This paper describes the key methodology in the improvement design plan of three major intersections (four and five-legged) in Yangon. Then it proposes the best improvement design which will minimize traffic congestion, vehicle delay and maximize level of service for each existing intersection. In this research, the set of variables that affect control delay at signalized and unsignalized intersections were firstly determined. Then, channelized intersections with traffic signal control designs were proposed for existing intersections using AASHTO (2001) "A Policy on Geometric Design of Highways and Streets" and Highway Capacity Manual (2000) guidelines and the results were computed. In addition, proposals for the roundabout intersection designs were also made using the gap acceptance approach by Federal Highway Administration (2000) "Roundabouts: An informational guide" and the outcomes were determined. Finally, comparison between before and after improvement using both plans was made according to the level of service (LOS) and average control delay. As a result, the appropriate design for each intersection was evaluated. From the results of this paper, it was found that roundabout design was the most suitable upgrade plan for all three existing major intersections.

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Aung Kyaw Han

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LIST OF ABBREVIATIONS

%G	-	% grade on a lane group volume
%HV	-	% heavy vehicle for lane group volume
AASHTO	-	American Association of State Highway and Transportation Officials
A_{pbT}	-	Permitted phase pedestrian-bicycle adjustment factor
C	-	Cycle length
CBD	-	Central Business District
c_i	-	Capacity of lane group i
$C_{m,x}$	-	Capacity of movement x
d	-	Average control delay
d_1	-	Uniform control delay assuming uniform arrivals
d_2	-	Incremental delay to account for effect of random arrivals
d_3	-	Initial queue delay
d_A	-	Average delay per vehicle for approach A
d_i	-	Average delay per vehicle for lane group i
d_I	-	Average delay per vehicle for the intersection
e	-	Extension of effective green time
E_{L1}	-	Through-car equivalent for permitted left turns
f_a	-	Adjustment factor for area type
f_{bb}	-	Adjustment factor for blocking effect of local buses
f_g	-	Adjustment factor for approach grade
f_{HV}	-	Adjustment factor for heavy vehicles in traffic stream
FHWA	-	Federal Highway Administration
f_{Lpb}	-	Pedestrian-bicycle adjustment factor for left-turn movements
f_{LT}	-	Left-turn adjustment factor applied to a total lane group from which left turns are made
f_{LU}	-	Adjustment factor for lane utilization
f_{LUo}	-	Lane utilization factor for opposing flow
f_m	-	left-turn adjustment factor applied only to lane from which left turns are made

LIST OF ABBREVIATIONS (Continued)

f_{min}	-	Minimum value of f_m
f_p	-	Adjustment factor for parking
f_{PA}	-	Supplemental adjustment factor for platoon arriving during green
f_{Rpb}	-	Pedestrian-bicycle adjustment factor for right-turn movements
f_{RT}	-	Adjustment factor for right-turns in lane group
f_w	-	Adjustment factor for lane width
$g_/C$	-	Proportion of green time available
g_f	-	Portion of effective green until the arrival of the first left-turning vehicle
G_i	-	Actual green time
g_i	-	Effective green time
g_o	-	Effective green for opposing flow
g_p	-	Effective pedestrian green time
G_p	-	Minimum pedestrian green time
g_q	-	Portion of effective green blocked by the clearance of an opposing queue of vehicle
g_u	-	Portion of the effective green during which left turns filter through the opposing flow
H	-	H test or Kruskal-Wallis test
HCM	-	Highway Capacity Manual
I	-	Upstream filtering/metering adjustment factor
I_1	-	Start-up lost time
I_2	-	Clearance lost time
K	-	Delay adjustment factor that is dependent on signal controller mode
L	-	Total lost time
LOS	-	Level-of-Service
LT	-	Left-turn
LTC	-	Left turns per cycle
N	-	Number of lanes in lane group
N_B	-	Number of buses stopping

LIST OF ABBREVIATIONS (Continued)

NCHRP	-	National Cooperative Highway Research Program
NL	-	Number of lanes
N_m	-	Number of parking maneuvers
N_o	-	Number of opposing lane
N_{ped}	-	Number of pedestrians crossing during an interval
N_{rec}	-	Number of receiving lanes
N_{turn}	-	Number of turning lanes
OCC_{bicg}	-	Bicycle conflict zone occupancy
OCC_{pedg}	-	Average pedestrian occupancy
OCC_{pedu}	-	Pedestrian occupancy
OCC_r	-	Relevant conflict zone occupancy
P	-	Proportion of vehicles arriving on green
PF	-	Uniform delay progression adjustment factor
PHF	-	Peak-hour factor
P_L	-	Proportion of left turns in shared lane
P_{LT}	-	Proportion of left turns in lane group
P_{LTA}	-	Proportion of Left-turn movements
P_{RT}	-	Proportion of RTs in lane group
qr_o	-	Opposing queue ratio
R_{po}	-	Platoon ratio for opposing flow
RT	-	Right-turn
S	-	Queue storage length per vehicle
s	-	Saturation flow rate for subject lane group
S_{HT}	-	Saturation flow of through traffic
S_{LT}	-	Filter saturation flow of permitted left turns
s_o	-	Base saturation flow rate per lane
S_p	-	Average walking speed of pedestrians
T	-	Analysis time period
t_c	-	Critical gap
t_f	-	Follow-up headway

LIST OF ABBREVIATIONS (Continued)

TH	-	Through
t_L	-	Lost time per phase
TRB	-	Transportation Research Board
V	-	Traffic volume per hour per lane
v_A	-	Analysis flow rate for approach A
v_{bic}	-	Bicycle volume
v_{bicg}	-	Bicycle flow rate
V_{Circ}	-	Circulation flow rates
v_i	-	Actual or projected demand flow rate for lane group i
v_{LT}	-	Adjusted left-turn flow rate
V_{LT}	-	Left-turn traffic volume
v_o	-	Adjusted opposing flow rate
v_{olc}	-	Adjusted opposing flow rate per lane per cycle
v_p	-	Flow rate during peak 15-min period
v_{ped}	-	Pedestrian volume
v_{pedg}	-	Pedestrian flow rate
V_{RT}	-	Right-turn traffic volume
V_{Thru}	-	Through traffic volume
V_x	-	Flow rate for movement x
W	-	Lane width
W_E	-	Effective crosswalk width
X_c	-	Critical v/c ratio for intersection
X_i	-	Volume to capacity ratio for lane group i
Y_i	-	Change and clearance interval

IMPROVEMENT OF THE MAJOR INTERSECTIONS IN YANGON

INTRODUCTION

General

Myanmar, with a total area of around 677,000 sq.km and with a growing population of approximately 58.846 million, has experienced an increase in vehicles estimated at 979,288, a rate of nearly 3% whereas registered motorcycles increased at around 7% per annum, according to the official statistics in 2008. Yangon, the former capital of Myanmar, which is the largest city and the most important commercial center and now the commercial hub, has found its communication passage mainly through road, rail and water facilities for domestic areas while air route is mostly responsible for overseas journeys.

As the road system was constructed by the British, streets in Yangon are all perpendicular retaining geometric symmetry. With the coverage of 598.76 sq km and 6.7 million population, Yangon is bordered on three sides by water with Hlaing River (Yangon River), the gulf of Martaban and Pazundaung Creek, a tributary to the Hlaing.

In this research, studies will be made on level of service at the three major intersections in Yangon namely (1) Hledan five-legged signalized intersection, (2) Myaynigone four-legged signalized intersection, and (3) Tamwe five-legged signalized intersection shown in Figure1.

Figure 1 Yangon City Map showing Three Major Intersection

Source: JICA (2009)

The development of road transportation facilities including intersections as public transportation in Yangon represents for the majority of urbanites (84%) whereas private car and rail transport take the shares of 2% and 6% respectively. Despite no permission on the use of truck, motorcycle and bicycle in some specific areas like downtown areas, the commercial hub city has been experiencing long queues of traffic congestion at most of its junctions and intersections especially during morning and evening rush hours. Traffic congestion in Yangon is shown in Figure 2.

Figure 2 Traffic congestion in Yangon

Statement of Problems

The previous town planning inevitably made the width of the road narrow and public transportation like bus is quite popular among Yangon citizens of all walks of life as railway cannot take the sufficient share of passengers like bus these days. Hledan intersection is situated near Yangon University and a supermarket, Myaynigone intersection is near the Dagon Center supermarket and Tamwe intersection is near a fuel station. All have bus stops which are situated near the intersections.

Three major intersections in Yangon are four and five leg intersections. All intersections are operated under signalized traffic control system and all of them are plain at grade intersections with wide area of conflicts. There are no high type transportation facilities such as overpass, underpass, bridges, etc. provided. Most especially in the downtown area, increasing traffic volumes will rapidly lead to deep congestion and significant delays. Most of the road users are waiting for a long time at the major intersection to pass through the intersection according to over capacity of the under designed intersections.

Accident rates at the major intersections are increasing because of the traffic congestion, drivers' behavior and existing conditions of the junctions which are inappropriate for most of the drivers and pedestrians. Traffic congestion usually occurrs during the peak hours of morning and evening, ranging from (8:00-10:00 AM) and (4:30-6:30 PM). During off-peak hour, traffic runs normally but traffic congestion at the major intersections is increasing from year to year.

OBJECTIVES

The main purposes of the present research are as follows,

1. To review the existing three major intersections with signal control in Yangon.

2. To describe the key methodology in the improvement design plan of four and five-legged major intersections in Yangon.

3. To propose the best improvement design which minimizes traffic congestion, vehicle delay and maximizes level of service for each existing intersection.

Scope of study

The scope of study will include,

1. The study of focus on the current traffic congestion at three major intersections in Yangon namely Hledan, Myaynigone and Tamwe intersections.

2. Only the channelized and roundabout designs are considered for the improvement of intersections.

3. The channelized intersection with traffic signal control designs will be proposed for the existing intersections using AASHTO (2001) "A Policy on Geometric Design of Highways and Streets" and Highway Capacity Manual (2000) guidelines and the results will be computed.

4. Besides, Roundabout intersection designs will be also proposed using the gap acceptance approach by Federal Highway Administration (2000) "Roundabouts: An informational guide" and the outcomes will be determined.

5. Finally, comparison between before and after improvement using both plans will be evaluated based upon level of service (LOS) and average control delay, and as a result, the appropriate design for each intersection will be evaluated.

6. Use Urban Intersection Design Guide (2005), FHWA/TX-05/0-4365-P2 Vol.2 to calculate acceleration and deceleration length of the channelized intersection design.

7. Use Right-In-Right-Out Channelization (1998), Oregon Department of Transportation to analyze the directional and divisional islands.

8. Use Roundabouts: An informational guide (2010), NCHRP Report 672, second edition to analyze splitter island and double-lane roundabout design.

LITERATURE REVIEW

1. Three Major Intersections in Yangon

Here in this chapter, three major intersections namely (1) Hledan intersection, (2) Myaynigone intersection, and (3) Tamwe intersection will be the focus.

1.1 Hledan intersection

Hledan intersection with five legs is a four phase signalized intersection and has the most traffic congestion in Yangon. It is located near Yangon University and a bus stop.

During peak hours, people need to go through the long queuing bus-line to get onto the bus they want to take and only just a small number of buses can pass through the intersection at a time. The intersection for Pyay Road (1) &Road (2) and Insein Road (1) & Road (2) provides 90-second-interval for traffic light. The individual queue of the traffic at four directions lengthens from 400 meter to over 500 meters. For regular period, the queuing length usually reduces to half. Mostly nearly twenty cars and buses from each lane can pass the traffic light during rush hours and approximately 10 minutes and above is needed to pass the intersection during rush hours.

As the bus-stop is located just near the intersection, it is all the time crowded with passengers and buses which try to stop in the middle of the road to pick up more and more passengers. Hledan intersection has the most traffic congesrion and sometimes vehicle-pedestrian accident takes place near the intersection. Figure 3 shows Hledan intersection.

Figure 3 Hledan intersection

1.2 Myaynigone intersection

Myaynigone intersection has four legs and it is the nearest intersection to downtown area. Crowded places such as a bus-stop and supermarkets are located near the intersection resulting in traffic congestion for the whole day, but if it is compared with Hledan intersection, the traffic volume and congestion is quite small. The traffic control system at the intersection for Pyay road is left-turn early control system and the three phase signalized intersection.

The main problems of traffic congestion at Myaynigone intersection during rush hours are the busy bus-stop where public buses and taxis line up haphazardly as well as less traffic steering measures by traffic police. Unlike Hledan intersection, Myaynigone intersection meets another intersection (Wisaya intersection) just after the short turn and that also is one of the points making both the two intersections overcrowded.

Though the intersection for the Pyay road offers 75-second-interval for traffic light, the individual traffic queue length of the road between these two intersections reaches over 100 meters. For regular period, the queuing length usually reaches half. In general, around twenty cars and buses from each lane can pass the traffic light during rush hours and it takes over 10 minutes and above to pass the intersection. Figure 4 shows Myaynigone intersection.

Figure 4 Myaynigone intersection

1.3 Tamwe intersection

Tamwe intersection with five legs is the four-phase signalized intersection and is not far from downtown area. It is also one of the most crowded intersections as just a very few number of buses or cars can pass through the traffic light at each interval.

The intersection for East Race Course Road, Ba Nyar Da La Road (1) & Road (2) and U Chit Maung Road offers 60-second-interval for traffic light but as the route is like uphill and busy with cars from different directions, sometimes just one or two vehicles can pass through the traffic light. For regular period, the queuing length usually reaches around 50 meters and it doubles to 100 meters and above during rush hours. Generally, it takes over 10 minutes and above to pass the intersection. Figure 5 shows Tamwe intersection.

Figure 5 Tamwe intersection

2. Intersection Types

An intersection is defined as the general area where two or more highways join or cross, including the roadway and roadside facilities for traffic movements within the area. The most common intersections at which two highways cross one another has four legs.

The intersection type is determined primarily by the number of intersecting legs, the topography, the character of the intersecting highways, the traffic volumes, patterns, and speeds, and the desired type of operation. Any of the basic intersection types can vary greatly in scope, shape, and degree of channelization.

Intersections are classified into three general categories: grade-separated without ramps, grade-separated with ramps (commonly known as interchanges), and at-grade intersection (AASHTO, 2001).

2.1 Grade-separated without ramps

There are many situations where grade separations are constructed without the provision of ramps as shown in Figure 6. Some major arterials intersecting the existing highway must be kept open for access but carry only low traffic volumes. Lacking a suitable relocation plan for the crossroad, a highway grade separation without ramps may be provided. All drivers desiring to turn to or from that road are required to use other existing routes and enter or leave the highway at other locations. These vehicles may have to travel a considerable extra distance (AASHTO, 2001).

Figure 6 Grade separated without ramps

Source: AASHTO (2001)

2.2 Grade-separated with ramps (interchanges)

There are several basic interchange configurations to accommodate turning movements at a grade separation as shown in Figure 7. The type of configuration used at a particular site is determined by the number of intersection legs, expected volumes of through and turning movements, type of truck traffic, topography, culture, design controls, and proper signing. The designer’s initiative also plays an important role (AASHTO, 2001).

Figure 7 Grade-separated with ramps (interchanges)

Source: AASHTO (2001)

2.3 At-grade intersection

The important design considerations for at-grade intersections fall into two major categories: the geometric design of the intersection (including capacity analysis) and the location and type of traffic control devices. The basic types of at-grade intersections are T or three-leg intersections, which consist of three approaches; four-leg or cross intersections, which consist of four approaches; and multi-leg intersections, which consist of five or more approaches (AASHTO, 2001).

3. Channelized intersection

Channelization is the separation or regulation of conflicting traffic movements into defined paths of travel by traffic islands or pavement markings to facilitate the safe and orderly movements of both vehicles and pedestrians. Design of a channelized intersection at an approach road usually involves the type of design vehicle, the cross section on major roadway, the projected traffic volumes in the relation to capacity, the number of pedestrians, the speed of vehicles, the location of any required bus stop, and the type and location of local traffic control devices.

In order to achieve the purposes of channelization, certain design principles or rules should be followed. The type of intersection control used, that is stop, yield, or traffic signal, has a large impact on many of the design rules. Raised traffic islands are typically required for right-in-right-out channelization. The curbed island that normally should be considered is one that has an area of approximately 60 ft^2 for urban streets. Adequate reflectorization and/or illumination should be used to make all islands clearly visible at night (Aksan, A., *et al*., 1998). The corner radii are important design elements in that they influence the operational characteristics, construction cost, and maintenance of the intersection. According to (AASHTO, 2001), the recommended corner radii are necessary for each intersection.

3.1 Four-leg intersections with simple channelization

The usual configurations of four leg intersections with simple channelization expect at minor intersections, right-turning roadways are often provided. The channelized intersection shall be either signalized or unsignalized depending on the warrants (AASHTO, 2001). Channelized at grade intersection is shown in Figure 8.

3.2 Upgrading System for Three Intersections

Four or five legs intersection with signal control like Hledan, Myaynigone and Tamwe intersections should be transformed channelized intersection with signal control.

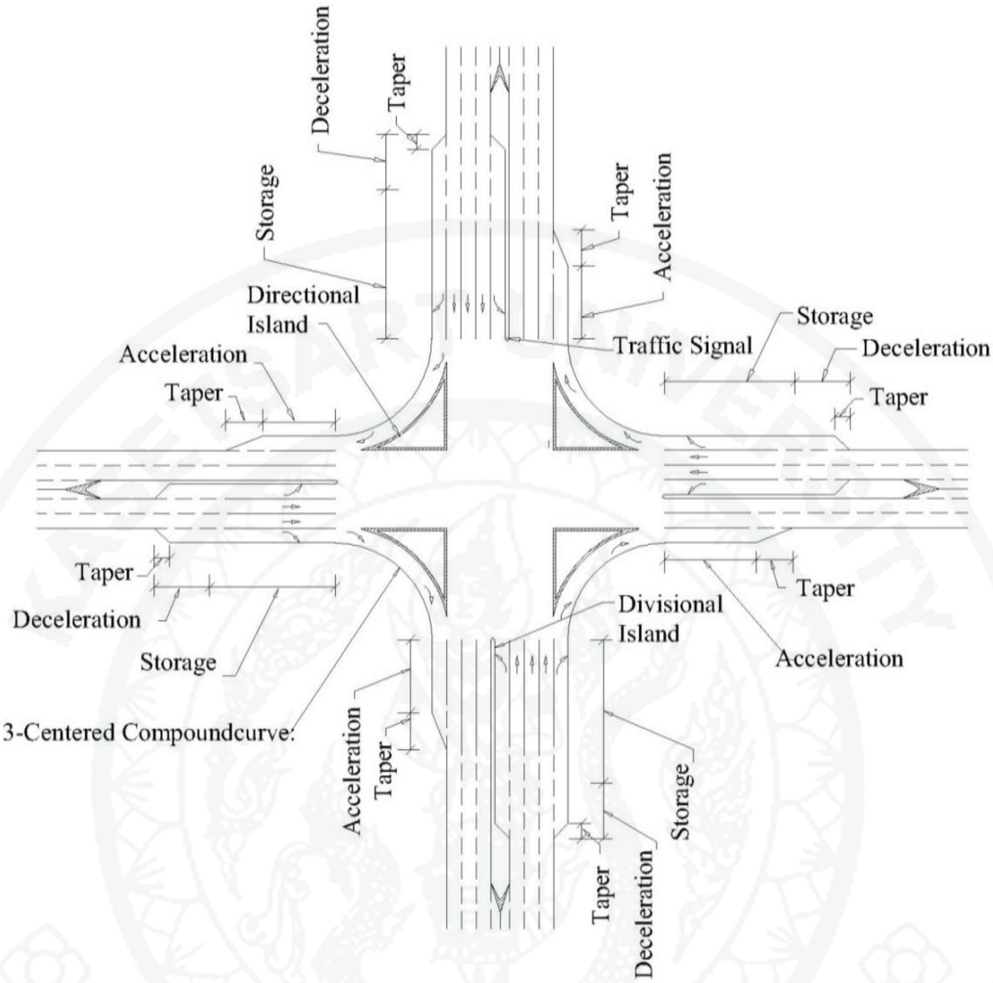


Figure 8 Channelized at grade intersection

Source: AASHTO (2001)

3.3 Turning Roadway with Corner Islands

Where the inner edges of the traveled way for right turns are designed to accommodate semitrailer combinations or where the design permits passenger vehicles to turn at speed of 15 km/h [10 mph] or more, the pavement area within the intersection may become excessively large and consequently does not provide for the proper control of traffic. To avoid this condition, a corner island can be provided to form a separate turning roadway between the two intersection legs (AASHTO, 2001). Table 1 shows typical design for turning roadways.

Table 1 Typical Design for Turning Roadways

US Customary					
Angle of Turn (degree)	Design classification	Three-centered compound curve		Width of Lane (ft)	Approx. island size (ft^2)
		Radii (ft)	Offset (ft)		
75	A	150-75-150	3.5	14	60
	B	150-75-150	5.0	18	50
	C	180-90-180	3.5	20	50
90	A	150-50-150	3.0	14	50
	B	150-50-150	5.0	18	80
	C	180-65-180	6.0	20	125
105	A	120-40-120	2.0	15	70
	B	100-35-100	5.0	22	50
	C	180-45-180	8.0	30	60
120	A	100-30-100	2.5	16	120
	B	100-30-100	5.0	24	90
	C	180-40-180	8.5	34	220
135	A	100-30-100	2.5	16	460
	B	100-30-100	5.0	26	370
	C	160-35-160	9.0	35	640
150	A	100-30-100	2.5	16	1400
	B	100-30-100	6.0	30	1170
	C	160-35-160	7.1	38	1720

Source: AASHTO (2001)

3.4 Design length of Right-Turn Acceleration lanes

Design length of right-turn acceleration lanes is shown in Table 2.

Table 2 Design length of right-turn acceleration lanes

Highway Design Speed (mph)	**Minimum Length of Taper T (ft)**	**Acceleration Length, A (ft) for Entrance Curve Design Speed (mph)**								
		Stop Condition	**15**	**20**	**25**	**30**	**35**	**1140**	**45**	**50**
		AND INITIAL SPEED (MPH)								
		0	**14**	**18**	**22**	**26**	**30**	**36**	**40**	**44**
30	150	180	140	-	-	-	-	-	-	-
35	165	280	220	160	-	-	-	-	-	-
40	180	360	300	270	210	120	-	-	-	-
45	200	560	490	440	380	280	160	-	-	-
50	230	720	660	610	550	450	350	130	-	-
55	250	960	900	810	780	670	550	320	150	-
60	265	1200	1140	1100	1020	910	800	550	420	180
65	285	1410	1350	1310	1220	1120	1000	770	600	370
70	300	1620	1560	1520	1420	1350	1230	1000	820	580

Table 2 (Continued)

Highway Design Speed (mph)	Minimum Length of Taper T (ft)	Acceleration Length, A (ft) for Entrance Curve Design Speed (mph)								
		Stop Condition	**15**	**20**	**25**	**30**	**35**	**1140**	**45**	**50**
		AND INITIAL SPEED (MPH)								
		0	**14**	**18**	**22**	**26**	**30**	**36**	**40**	**44**
75	330	1790	1730	1630	1580	1510	1420	1160	1040	780

Source: Fitzpatrick *et al*. (2005)

3.5 Design Lengths of Single Left-Turn Lanes

Design lengths of single left-turn lanes on urban streets are shown in Table 3.

Table 3 Design lengths of single left-turn lanes on urban streets

Speed (mph)	Deceleration Length (ft)	Taper Length (ft)	Storage Length			
			Signalized		Non-Signalized	
			Calculated[1]	Minimum[2]	Calculated[1]	Minimum[2]
30	160	50	Equation	100	Equation	100
35	215	50	Equation	100	Equation	100
40	275	50	Equation	100	Equation	100
45	345	100	Equation	100	Equation	100
50	425	100	Equation	100	Equation	100
55	510	100	Equation	100	Equation	100

Source: Fitzpatrick *et al*. (2005)

3.6 Storage Length of Vehicles

The storage length of vehicles, S, is determined by the percentage of trucks as shown in Table 4.

Table 4 Storage Length of Vehicles

% of Trucks	S, ft [m]
< 5	25 [7.6]
5 to 9	30 [9.1]
10 to 14	35 [10.7]
15 to 19	40 [12.2]

Source: Fitzpatrick *et al*. (2005)

3.7 Storage Length for Left-Turn Lanes

The required storage length for left-turn lanes can be estimated according to the Urban Intersection Design Guide or with the following storage length Equation (1).

$$L = (V/N)\ (2)\ (S) \quad (1)$$

Where, L = storage length, ft

V = left-turn volume per hour, vph

N = number of cycles/hour for the traffic signal,

2 = factor that provides for all left-turning vehicles on most cycles; a value of 1.8 may be acceptable on collector streets;

S = queue storage length per vehicle, ft.

3.8 Storage Length for Through Lanes

The required queue storage length per vehicle for through lanes (and the through/right-turn lane) can be estimated according to the Urban Intersection Design Guide’s Equation (2).

$$V = (V_{Thru} + V_{Rt})/NL \qquad (2)$$

Where, V = traffic volume per hour per lane, vphpl
V_{Thru} = through traffic volume, vphpl
V_{Rt} = right-turn traffic volume, vphpl
NL = number of lanes

4. Multi-leg Intersection

Multileg intersections including five or more legs should be avoided wherever practical. At locations where multileg intersections are used, it may be satisfactory to have all intersection legs intersect at a common paved area, where volumes are light and stop control is used (AASHTO, 2001).

4.1 Multi-leg Intersection with five approach legs

The simplest application of this principle for the intersection with five approach legs approach, the diagonal leg is realigned to join the upper road at sufficient distance from the main intersection to form two distinct intersections, each of which can be operated simply. The left-to-right highway is likely to be the more important route, and for this reason the diagonal leg is realigned to locate the new intersection on the less important road (AASHTO, 2001).

4.2 Upgrading System for Three Intersections

Four or five leg intersection with signal control like Hledan, Myaynigone, Tamwe intersections should be transformed into roundabout intersection.

4.3 Roundabout

Roundabouts commonly termed as “traffic circles” have been widely accepted as replacements or alternatives to conventional intersections in Europe and Australia (Troutbeck, 1993). In these countries, there is no specific way to distinguish roundabouts from traffic circles. Key features of roundabouts include yield control of entering traffic, channelized approaches and appropriate geometric curvature to slow speed typically less than 50 km/h (FHWA, 2000). In particular, the numbers of roundabouts constructed in the United States that are currently in operation have been reported to perform favorably (shorter delay, improved safety) when compared with conventional intersections (Flannery *et al*., 1998).

All over the world, roundabouts that have been properly designed and constructed at appropriate locations have demonstrated that they are superior to signalized intersections in safety, capacity, environmental considerations, economics and esthetics. The geometry of roundabouts eliminates most of the angles and traffic flows that create the potential for crashes at signalized intersections, while the continuous movement of vehicles, albeit at slower speeds, increases the capacity of an intersection (Isaacs and Barrett, 2003). Drawing of key roundabout dimensions are shown in Figure 9.

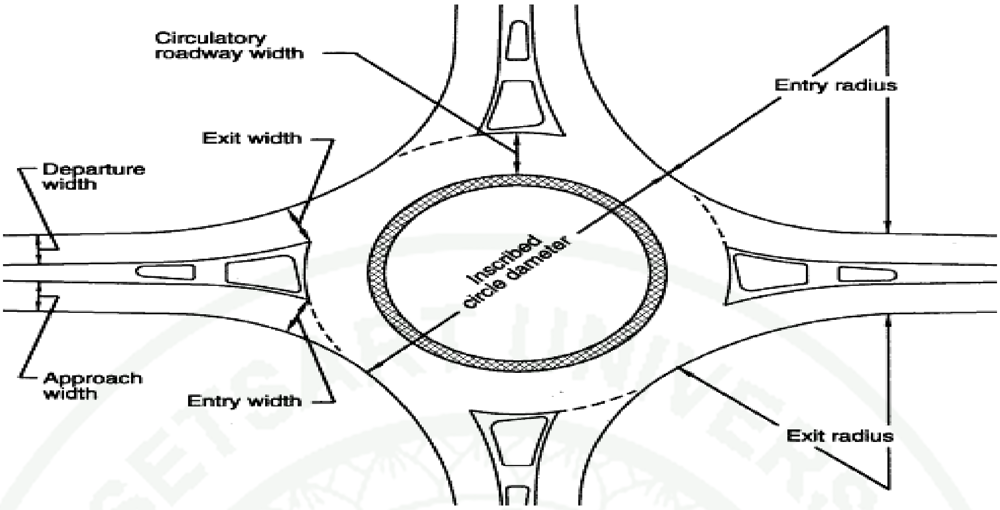


Figure 9 Drawing of key roundabout dimensions

Source: Robinson *et al.* (2000)

4.4 Concept of roundabout capacity

The capacity of each entry approach to a roundabout is the maximum rate at which vehicles can reasonably be expected to enter the roundabout from an approach during a given time period under prevailing traffic and roadway (geometric) conditions. An operational analysis considers a precise set of geometric conditions and traffic flow rates defined for a 15-minute analysis period for each roundabout entry (Robinson *et al*., 2000)**.**

4.5 Data Requirements

The analysis method described the specification of traffic volumes for each approach to the roundabout, including the flow rate for each directional movement. Volumes are typically expressed in passenger car vehicles per hour (vph), for a specified 15-minute analysis period. Entry flow is simply the sum of the through, left, and right turn movements on an approach. Circulating flow is the sum of the vehicles from different movements passing in front of the adjacent upstream splitter island. Right turns are included in approach volumes and require capacity, but are not included in the circulating volumes downstream because they exit before the next entrance (Robinson *et al*., 2000).

4.6 Determining circulating volumes as a function of turning movement volumes

For proposed or planned four-legged roundabouts, Equations (3), (4), (5) and (6) through can be applied to determine conflicting (circulating) flow rates, as shown graphically in Figure 10.

$$V_{EB,circ}= V_{WB,LT}+V_{SB,LT}+V_{SB,TH}+V_{NB,U\text{-}turn}+V_{WB,U\text{-}turn}+V_{SB,U\text{-}turn} \quad (3)$$

$$V_{WB,circ} = V_{EB,LT}+V_{NB,LT}+V_{NB,TH}+V_{SB,U\text{-}turn}+V_{EB,U\text{-}turn}+V_{NB,U\text{-}turn} \quad (4)$$

$$V_{NB,circ} = V_{EB,LT}+V_{EB,TH}+V_{SB,LT}+V_{WB,U\text{-}turn}+V_{SB,U\text{-}turn}+V_{EB,U\text{-}turn} \quad (5)$$

$$V_{SB,circ} = V_{WB,LT}+V_{WB,TH}+V_{NB,LT}+V_{EB,U\text{-}turn}+V_{NB,U\text{-}turn}+V_{WB,U\text{-}turn} \quad (6)$$

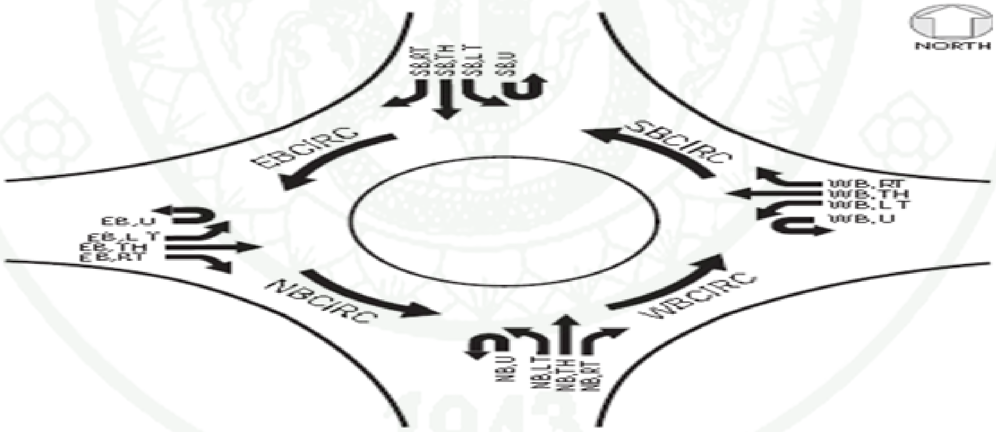


Figure 10 Traffic flow parameters

Source: Robinson *et al*. (2000)

4.7 Capacity

The maximum flow rate for roundabout that can be accommodated at a roundabout entry depends on two factors: the circulating flow on the roundabout that conflicts with the entry flow, and the geometric elements of the roundabout. Roundabouts should be designed to operate at no more than 85 percent of their estimated capacity (Robinson *et al*., 2000).

4.8 Double-lane roundabout capacity

The capacity of a double-lane roundabout that is based on the design templates for the urban/rural double-lane roundabouts inscribed circle diameters of 40 m to 60 m (130 ft to 200 ft). Larger inscribed diameter roundabouts are expected to have slightly higher capacities at moderate to high circulating flows. Figure 11 shows approach capacity of a double-lane roundabout (Robinson *et al.*, 2000).

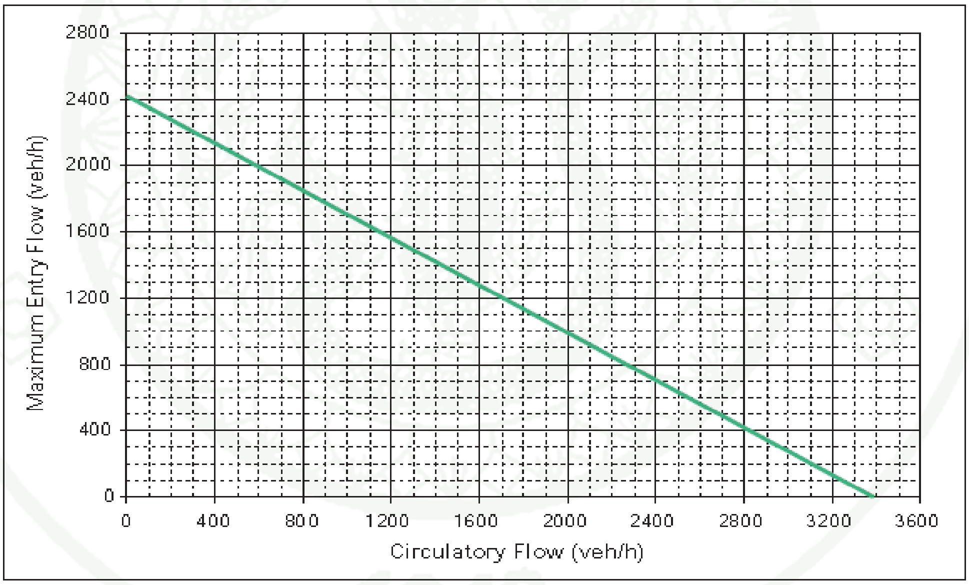


Figure 11 Approach capacity of a double-lane roundabout

Source: Robinson *et al*. (2000**)**

4.9 Pedestrian effects on entry capacity

Pedestrians crossing at a marked crosswalk given priority over entering motor vehicles can have a significant effect on the entry capacity. The vehicular capacity should be factored (multiply by M) according to the relationship shown in Figure 12 for double-lane roundabout assuming pedestrian priority.

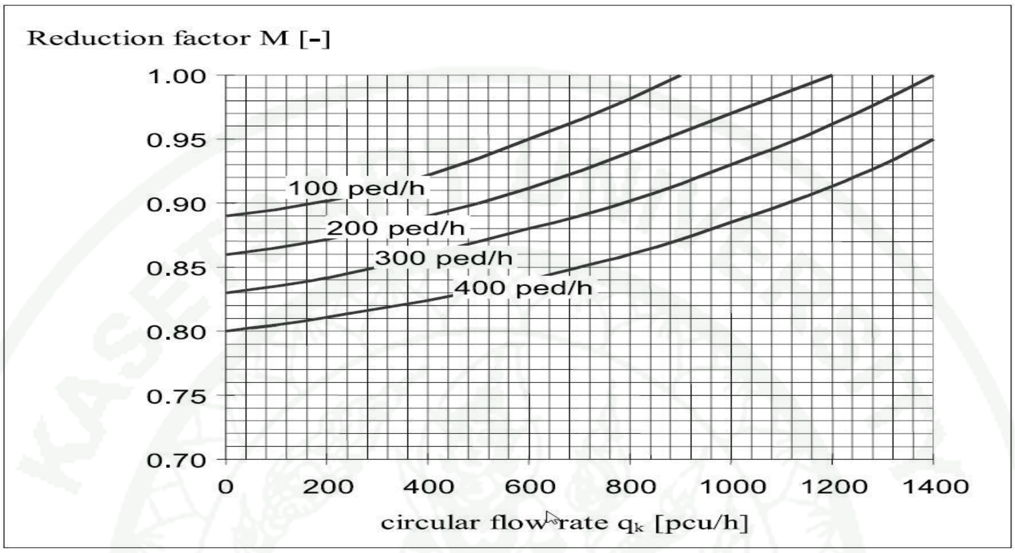


Figure 12 Capacity reduction factor M for a double-lane roundabout assuming pedestrian priority

Source: Robinson *et al*. (2000)

4.10 Exit capacity

An exit flow on a single - lane of more than 1,400 veh/h, even under good operating conditions for vehicles (i.e., tangential alignment, and no pedestrians and bicyclists) is difficult to achieve. Under normal urban conditions, the exit lane capacity is in the range of 1,200 to 1,300 veh/h. Therefore, exit flows exceeding 1,200 veh/h may indicate the need for a double-lane exit (Robinson *et al*., 2000).

4.11 Performance Analysis

The performance measures are typically used to estimate the performance of a given roundabout design: degree of saturation (volume to capacity ratio) and control delay **(**Robinson *et al.*, 2000**)**.

4.12 Degree of saturation

Degree of saturation is the ratio of the demand at the roundabout entry to the capacity of the entry. The Australian design procedure suggests that the degree of saturation for an entry lane should be less than 0.85 for satisfactory operation. When the degree of saturation exceeds this range, queues may form and delay begins to increase exponentially (Robinson *et al.*, 2000).

4.13 Delay

Delay is a standard parameter used to measure the performance of an intersection. The Highway Capacity Manual 2000 identifies delay as the primary measure of effectiveness for both signalized and unsignalized intersections, with level of service determined from the delay estimate. HCM 2000 only includes control delay, the delay attributable to the control device. Control delay is the time that a driver spends queuing and then waiting for an acceptable gap in the circulating flow while at the front of the queue. The formula for computing this delay is given in Equation (7) (Robinson *et al.*, 2000).

$$d = \frac{3600}{C_{m,x}} + 900T\left[\frac{V_x}{C_{m,x}} - 1 + \sqrt{\left(\frac{V_x}{C_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{C_{m,x}}\right)\left(\frac{V_x}{C_{m,x}}\right)}{450T}}\right] \quad (7)$$

Where, d = Average control delay, sec/veh;

V_x = Flow rate for movement x, veh/h;

$C_{m,x}$ = capacity of movement x, veh/h; and

T = Analysis time period, h (T= 0.25 for a 15-minute period)

4.14 Level-of-Service Criteria for Unsignalized Intersections.

Table 5 shows level-of-service Criteria for Unsignalized Intersections.

Table 5 Level-of-service Criteria for Unsignalized Intersections

LOS	Average Control Delay (s/veh)
A	0 – 10
B	> 10 – 15
C	> 15 – 25
D	> 25 – 35
E	> 35 – 50
F	> 50

Source: TRB (2000)

5. Double-Lane Roundabout Design

Double-lane roundabout design is significantly different from single-lane roundabout design, and many of the techniques used in single-lane roundabout design do not directly transfer to double-lane design (Rodegerdts *et al*., 2010).

5.1 Maximum Entry Design Speeds

Recommended maximum entry design speeds for roundabouts at various intersection site categories are provided in Table 6.

Table 6 Recommended maximum entry design speeds

Site Category	Recommended Maximum Entry Design Speed
Mini-Roundabout	25 km/h (15 mph)
Urban Compact	25 km/h (15 mph)
Urban Single-Lane	35 km/h (20 mph)
Urban Double-Lane	40 km/h (25mph)
Rural Single-Lane	40 km/h (25mph)

Table 6 (Continued)

Site Category	Recommended Maximum Entry Design Speed
Rural Double-Lane	50 km/h (30 mph)

Source: Robinson *et al*. (2000)

5.2 Inscribed Diameter Ranges

The size of the inscribed circle is largely dependent upon the turning requirements of the design vehicle. Recommended ranges of inscribed circle diameters for various site locations are provided in Table 7.

Table 7 Recommended inscribed circle diameter ranges

Site Category	Typical Design Vehicle	Inscribed Circle Diameter Range
Mini-Roundabout	Single-Unit Truck	13-25 m (45-80 ft)
Urban Compact	Single-Unit Truck/Bus	25-30 m (80-100 ft)
Urban Single Lane	WB-15 (WB-50)	30-40 m (100-130 ft)
Urban Double Lane	WB-15 (WB-50)	45-55 m (150-180 ft)
Rural Single Lane	WB-20 (WB-67)	35-40 m (115-130 ft)
Rural Double Lane	WB-20 (WB-67)	55-60 m (180-200 ft)

Source: Robinson *et al*. (2000)

5.3 Flare Length

Flaring is an effective means of increasing capacity without requiring as much capacity; it does not increase crash frequency. The crash frequency for two approaches with the same entry width will be essentially the same, whether they have parallel entry lanes or flared entry designs. Entry widths should be minimized and flare lengths maximized to achieve the desired capacity with minimal effect on crashes. Generally, flare lengths should be a minimum of 25 m (80 ft) in urban areas and 40 m (130 ft) in rural areas. If right-of-way is constrained, shorter lengths can be used with noticeable effects on capacity (Rodegerdts *et al*., 2010).

5.4 Splitter Islands

Splitter islands should be provided on all roundabouts. Their purpose is to provide shelter for pedestrians (including wheelchairs, bicycles, and baby strollers), assist in controlling speeds, guide traffic into the roundabout, physically separate entering and exiting traffic streams, and deter wrong-way movements.

The splitter island is formed by the entry and exit curves on a leg. The total length of the island should be at least 15 m (50 ft), although 30m (100 ft) is desirable. The splitter island width should be a minimum of 1.8 m (6 ft) at the crosswalk to adequately provide refuge for pedestrians (Rodegerdts *et al*., 2010).

5.5 Entry Width

The entry width is dependent upon the number of lanes and design vehicle. Individual lanes range from 12 to 15 ft (3.7 to 4.6 m) (Rodegerdts *et al*., 2010).

5.6 Circulatory Roadway Width

The circulatory roadway width is usually governed by the design criteria relating to the type of vehicles that may need to be accommodated adjacent to one another through a multilane roundabout. Individual multilane circulatory roadway lane widths typically range from 14 to 16 ft (4.3 to 4.9 m). Use of these values results in a total circulating width of 28 to 32 ft (8.5 to 9.8 m) for a two-lane circulatory roadway (Rodegerdts *et al*., 2010). Table 8 provides minimum recommended circulatory roadway widths for two lane roundabouts where semi-trailer traffic is relatively infrequent.

Table 8 Minimum circulatory lane widths for two-lane roundabouts

Inscribed Circle Diameter	Minimum Circulatory Lane Width	Central Island Diameter
45.m (1540 ft)	9.8 m (32 ft)	25.4 m (86 ft)
50 m (165 ft)	9.3 m (31 ft)	31.4 m (103 ft)
55 m (180 ft)	9.1 m (30 ft)	36.8 m (120 ft)
60 m (200 ft)	9.1 m (30 ft)	41.8 m (140 ft)
65 m (215 ft)	8.7 m (29 ft)	47.6 m (157 ft)
70 m (230 ft)	8.7 m (29 ft)	52.6 m (172 ft)

Source: Robinson *et al*. (2000)

5.7 Exit Width

The exit width is the width of the carriageway and is measured in a similar manner to the entry width. It is the distance between the nearside kerb and the exit median where it intersects with the outer edge of the circulatory carriageway. Values are typically similar to or slightly less than entry widths (Rodegerdts *et al*., 2010).

5.8 Entry Radius

The entry radius is an important factor in determining the operation of a roundabout as it has significant impacts on both capacity and safety. The entry radius, in conjunction with the entry width, the circulatory roadway width, and the central island geometry, controls the amount of deflection imposed on a vehicle's entry path. Larger entry radii produce faster entry speeds and generally result in higher crash rates between entering and circulating vehicles. Entry radii for multilane roundabouts should typically exceed 65 ft (20 m) to encourage adequate natural paths and avoid sideswipe collisions on entry. Entry radii at urban roundabouts typically range from 50 to 100 ft (15 to 30 m) (Robinson *et al*., 2000).

5.9 Exit Radius

The exit curb radii are usually larger than the entry curb radii in order to minimize the likelihood of congestion and crashes at the exits. It should be no less than 50 ft (15 m), with the values of 100 to 200 ft (30 to 60 m) being more common. The exit curve should produce an exit path radius smaller than the circulating path radius (Robinson *et al*., 2000).

5.10 Stopping Sight Distance

Stopping sight distance is the distance along a roadway required for a driver and should be provided at every point within a roundabout and on each entering and exiting approach. Rrecommended design values for stopping sight distances are shown in Table 9.

Table 9 Design values for stopping sight distances

Speed (mph)	Computed Distance (ft)
10	46.4
15	77.0
20	112.4
25	152.7
30	197.8
35	247.8
40	302.7
45	362
50	427.2
55	496.7

Source: Robinson *et al*. (2000)

5.11 Superelevation

As a general practice, a cross slope of 2 percent away from the central island [Negative superelevation (-2 %)] is used. This technique of sloping outward is recommended for four main reasons (Robinson *et al*., 2000).

1. It promotes safety by raising the elevation of the central island and improving its visibility;
2. It promotes lower circulating speeds;
3. It minimizes breaks in the cross slopes of the entrance and exit lanes; and
4. It helps drain surface water to the outside of the roundabout.

6. Analysis of traffic at signalized intersections

The analysis of traffic at signalized intersections determines two key values. The first one is the v/c ratio of each lane group and of the intersection, and the second one the delays and level of services of the lane groups and of the intersection as a whole (TRB, 2000).

6.1 Lost Time

Equation 8 (TRB, 2000) giving lost time is a combination of start-up lost time and clearance lost time. In actual practice the lost due to starting and end lost times is often taken as 2 seconds. Clearance lost time is the difference between change and clearance interval, and extension of effective green time.

$$t_L = I_1+I_2 = I_1+Y_i - e \tag{8}$$

Where, t_L = lost time per phase in seconds;
I_1 = start-up lost time in seconds;
I_2 = clearance lost time in seconds;

Y_i = change and clearance interval; and

e = extension of effective green time.

6.2 Total Lost Time

Total lost time can be computed by using Equation 9 (TRB, 2000).

$$L = \text{lost time per phase} \times \text{number of phase} \quad (9)$$

Where, L = total lost time in second.

6.3 Signalization Conditions

Complete information regarding signalization intersection is needed to perform an analysis. This information for the signalized intersection includes a phase diagram illustrating the phase plan, cycle length, green times, and change-and-clearance intervals. Lane groups operating under actuated control must be identified, including the existence of push-button pedestrian-actuated phases. If pedestrian timing requirements exist, the minimum green time for the phase is indicated and provided for in the signal timing. The minimum green time for a phase is estimated by using Equations 10 and 11(TRB, 2000).

$$G_p = 3.2 + \frac{L}{S_p} + (0.27N_{ped}) \quad \text{for } W_E \leq 10 \text{ ft} \quad (10)$$

$$G_p = 3.2 + \frac{L}{S_p} + \frac{2.7N_{ped}}{W_E} \quad \text{for } W_E > 10 \text{ ft} \quad (11)$$

Where, G_p = minimum pedestrian green time in seconds,

3.2 = pedestrian start-up time in seconds,

L = crosswalk length in ft,

S_p = average walking speed of pedestrians, usually taken as 4.0 ft/s,

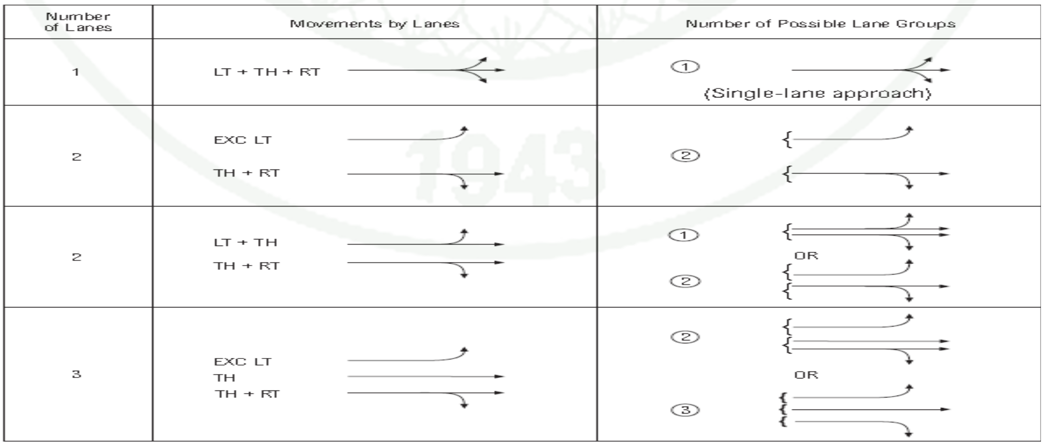
N_{ped} = number of pedestrians crossing during an interval, and
W_E = effective crosswalk width in ft.

6.4 Lane Grouping

The methodology for signalized intersections is disaggregated, that is, it is designed to consider individual intersection approaches and individual lane groups within approaches. Segmenting the intersection into lane groups is a relatively simple process that considers both the geometry of the intersection and the distribution of traffic movements. In general, the smallest number of lane groups is used that adequately describes the operation of the intersection.

An exclusive left-turn lane or lanes should normally be designated as a separate lane group unless there is also a share left-through lane present in which the proper lane grouping will depend on the distribution of traffic volume.

When two or more lanes are included in a lane group for analysis purposes, all subsequent computations treat these lanes as a single entity (TRB, 2000). Figure 13 shows some common lane group used for analysis.



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

Figure 13 Typical lane groups for analysis

Source: TRB (2000)

6.5 Determining Flow Rate

Demand volumes are best provided as average flow rates (in vehicles per hour) for the analysis period. Although analysis periods are usually 15 min long and demand volume, volume may also be stated for a time that encompasses more than one analysis period, such as an hourly volume. A peak 15-min flow rate is derived from an hourly volume by dividing the movement volumes by an appropriate PHF, which may be defined for the intersection as a whole, for each approach, or for each movement. The flow rate is computed by using Equation 12 (TRB, 2000).

$$v_p = \frac{V}{PHF} \tag{12}$$

Where, v_p = flow rate during peak 15-min period (veh/h),
V = hourly volume (veh/h), and
PHF = peak-hour factor.

6.6 Determining Saturation Flow Rate

A saturation flow rate for each lane group is computed according to Equation 13 (TRB, 2000). The saturation flow rate is the flow in vehicles per hour that can be accommodated by the lane group assuming that the green phase were displayed 100 percent of the time (i.e., g/C = 1.0).

$$s = s_o \, N \, f_w \, f_{HV} \, f_g \, f_p \, f_{bb} \, f_a \, f_{LU} \, f_{LT} \, f_{RT} \, f_{Lpb} \, f_{Rpb} \tag{13}$$

Where, s = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group (veh/h);
s_o = base saturation flow rate per lane (pc/h/ln);
N = number of lanes in lane group;
f_w = adjustment factor for lane width;
f_{HV} = adjustment factor for heavy vehicles in traffic stream;

f_g = adjustment factor for approach grade;

f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group;

f_{bb} = adjustment factor for blocking effect of local buses that stop within intersection area;

f_a = adjustment factor for area type;

f_{LU} = adjustment factor for lane utilization;

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

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

f_{Lpb} = pedestrian-bicycle adjustment factor for left-turn movements; and

f_{Rpb} = pedestrian-bicycle adjustment factor for right-turn movements.

6.7 Base Saturation Flow Rate

Computations begin with the selection of a base saturation flow rate, usually 1,900 passenger cars per hour per lane (pc/h/ln) (TRB, 2000).

6.8 Adjustment factor for Lane width

Equation 14 (TRB, 2000) gives the lane width adjustment factor, f_w, which accounts for the negative impact of arrow lanes on saturation flow rate and allows for an increased flow rate on wide lanes. Standard lane widths are 12 ft. In no case should the lane width factor be calculated for widths less than 8.0 ft.

$$f_w = 1 + \frac{(W-12)}{30} \qquad (14)$$

Where, f_w = adjustment factor for lane width,

W = lane width (ft). ($W \geq 8$, if $W > 16$, a two-lane analysis may be considered)

6.9 Adjustment for Heavy vehicles and Grade

The effects of heavy vehicles and approach grades are treated by separate factors, f_{HV} and f_g which are determined by using Equations 15 and 16 (TRB, 2000). Their separate treatment recognizes that passenger cars are affected by approach grades, as are heavy vehicles. The effect of grades on the operation of all vehicles.

$$f_{HV} = \frac{100}{100+\%HV\,(E_T-1)} \qquad (15)$$

Where, f_{HV} = adjustment factor for heavy vehicle,
% HV = % heavy vehicle for lane group volume,
E_T = 2.0 pc/HV

$$f_g = 1 - \frac{\%G}{200} \qquad (16)$$

Where, f_g = adjustment factor for grade,
% G = % grade on a lane group volume, ($-6 \leq \%\ G \leq +10$).

6.10 Adjustment for parking

The parking adjustment factor, f_p, which accounts for the frictional effect of a parking lane on flow in an adjacent lane group as well as for the occasional blocking of an adjacent lane by vehicles moving into and out of parking spaces, is computed by using Equation 17 (TRB, 2000).

$$f_p = \frac{N-0.1-\frac{18N_m}{3600}}{N} \qquad (17)$$

Where, f_p = adjustment factor for parking, ($f_p \geq 0.05$, f_p = 1.00 for no parking);
N_m = number of parking maneuvers/h; ($0 \leq N_m \leq 180$).

6.11 Adjustment for Bus Blockage

The bus blockage adjustment factor, f_{bb}, which accounts for the impact of local transit buses that stop to discharge or pick up passengers at a near-side or far-side bus stop within 250 ft of the stop line (upstream or downstream), is determined by using Equation 18. The factor used an average blockage time of 14.4 s during a green indication (TRB, 2000).

$$f_{bb} = \frac{N - \frac{14.4 N_B}{3600}}{N} \tag{18}$$

Where, f_{bb} = adjustment factor for bus blockage, ($f_{bb} \geq 0.05$);

N_B = number of buses stopping/h; ($0 \leq N_B \leq 250$).

6.12 Adjustment factor Area Type

Equations 19 and 20 (TRB, 2000) give the area type adjustment factor, f_a, which accounts for the relative inefficiency of intersections in business districts in comparison with those in other locations.

f_a = 0.9 in CBD, (19)

f_a = 1.0 in all other areas. (20)

Where, f_a = adjustment factor area type,

6.13 Adjustment for Lane Utilization

The lane utilization adjustment factor, f_{LU}, provides an adjustment to the base saturation flow rate. The adjustment factor is based on the flow in the lane with the highest volume (TRB, 2000). Table 10 summarizes lane utilization adjustment factors for different lane group movements and number of lanes.

Table 10 Default Lane Utilization Adjustment Factors

Lane Group Movement	No. of Lanes in Lane Group	Traffic in Most Heavily Traveled Lane (%)	Lane Utilization Adjustment Factor (f_{LU})
Through or shared	1	100.0	1.000
	2	52.5	0.952
	3[a]	36.7	0.908
Exclusive left turn	1	100.0	1.000
	2[a]	51.5	0.971
Exclusive right turn	1	100.0	1.000
	2[a]	56.5	0.885

Source: TRB (2000)

6.14 Adjustment for Right Turns

The right-turn adjustment factors, f_{RT}, which are primarily intended to reflect the effect of geometry, are given by Equations 21 to 23 (TRB, 2000).

Exclusive lane:

$$f_{RT} = 0.85 \quad (21)$$

Shared Lane:

$$f_{RT} = 1.0 - (0.15)P_{RT} \quad (22)$$

Single lane:

$$f_{RT} = 1.0 - (0.135)P_{RT} \quad (23)$$

Where, f_{RT} = right-turn adjustment factor,

P_{RT} = proportion of RTs in lane group.

The right-turn factor is 1.0 if the lane group does not include any right turns.

6.15 Adjustment for Left Turns

The left-turn adjustment factor, f_{LT}, is determined by using Equations 24 and 25 (TRB, 2000).

Protected phasing:

Exclusive lane

$$f_{LT} = 0.95 \quad (24)$$

Shared lane:

$$f_{LT} = \frac{1}{1.0+0.05P_{LT}} \quad (25)$$

Where, f_{LT} = left-turn adjustment factor,

P_{LT} = proportion of LTs in lane group.

7. Multilane Approach with Opposing Multilane Approaches

For multilane approaches, the impact of left turns on a shared lane must be extended to include their impact on the entire lane group (TRB, 2000).

7.1 Effective Green Time

The effective green time for each phase can be computed by using Equation (26) (TRB, 2000).

$$g_i = G_i + Y_i - t_L \quad (26)$$

Where, g_i = effective green time in second;

G_i = actual green time in second.

7.2 Division of the Effective Green Phase

The portion of effective green until the arrival of the first left-turning vehicle is designated g_f by using Equation 27 (TRB, 2000).

$$g_f = Ge^{-0.882\,LTC^{0.717}} - t_L \qquad 0 \leq g_f \leq g \qquad (27)$$

(shared permitted left-turn lanes)

Where, g_f = the portion of effective green until the arrival of the first left-turning vehicle

G = actual green time for permitted phase (s);

LTC = left turns per cycle, computed as $v_{LT}C/3600$;

v_{LT} = adjusted left-turn flow rate (veh/h);

C = cycle length (s); and

t_L = lost time for subject left-turn lane group (s).

The portion of effective green blocked by the clearance of an opposing queue of vehicle is designated g_q by using Equation 28 (TRB, 2000).

$$g_q = \frac{v_{olc}\, qr_0}{0.5 - \frac{v_{olc}\,(1-qr_0)}{g_0}} - t_l \qquad (28)$$

$$\frac{v_{olc}\,(1-qr_0)}{g_0} \leq 0.49 \text{ (note case specific parameter)}$$

Where, g_q = the portion of effective green blocked by the clearance of an opposing queue of vehicle;

v_{olc} = adjusted opposing flow rate per lane per cycle, computed as $v_oC/(3600N_of_{LUo})$;

v_o = adjusted opposing flow rate (veh/h);

f_{LUo} = lane utilization factor for opposing flow;

N_o = number of opposing lane;

qr_o = opposing queue ratio computed as $(1 - R_{po}(g_o/C)$; $qr_o \geq 0)$;

R_{po} = platoon ratio for opposing flow;
g_o = effective green for opposing flow (s).

The portion of the effective green during which left turns filter through the opposing flow is designated g_u by using Equation 29 and 30.

$$g_u = g - g_q \text{ when } g_q \geq g_f \quad (29)$$
$$g_u = g - g_f \text{ when } g_q \geq g_f \quad (30)$$

Where, g_u = the portion of the effective green during which left turns filter through the opposing flow;
g = effective green time for subject permitted left-turn (s).

7.3 Opposing Platoon Ratio.

The approximate ranges of R_{po} are related to arrival type as shown in Table 11, and default values are suggested for use in subsequent computations in Table 13.

Table 11 Relation Between Arrival Type and Opposing Platoon Ratio (R_{po})

Arrival Type	Range of Platoon (R_{po})	Default Value (R_{po})	Progression Quality
1	≤ 0.50	0.333	Very poor
2	> 0.50 – 0.85	0.667	Unfavorable
3	> 0.85 – 1.15	1.000	Random arrivals
4	> 1.15 – 1.50	1.333	Favorable
5	> 1.50 – 2.00	1.667	Highly favorable
6	> 2.00	2.000	Exceptional

Source: TRB (2000)

7.4 Through-Car Equivalents

Table 11 is used to assign E_{L1} through-car equivalents for each left-turning vehicle.

Table 12 Through-Car Equivalents, E_{L1}, for permitted left turns

Type of Left-Turn Lane	Effective Opposing Flow, $v_{oe} = v_o/f_{LUo}$						
	1	200	400	600	800	1000	1200[a]
Shared	1.4	1.7	2.1	2.5	3.1	3.7	4.5
Exclusive	1.3	1.6	1.9	2.3	2.8	3.3	4.0

Notes: "a" Use Equation 31, 32 and 33 for effective opposing flow more than 1200; v_{oe} must be > 0

Source: TRB (2000)

$$E_{L1} = S_{HT}/S_{LT} - 1 \text{ (shared)} \tag{31}$$

$$E_{L1} = S_{HT}/S_{LT} \text{ (exclusive)} \tag{32}$$

$$S_{LT} = \frac{v_{oe}\, e^{\left(\frac{-v_{oe}\, t_c}{3600}\right)}}{1 - e^{\left(\frac{-v_{oe}\, t_f}{3600}\right)}} \tag{33}$$

Where, E_{L1} = through-car equivalent for permitted left turns

S_{HT} = saturation flow of through traffic = 1900 veh/h/ln

S_{LT} = filter saturation flow of permitted left turns (veh/h/ln)

t_c = critical gap = 4.5 s

t_f = follow-up headway = 4.5 s (shared), 2.5 s (exclusive)

7.5 Proportion of left turns in shared lane

Equation 34 (TRB, 2000) gives the proportion of left turn in shared lane.

$$P_L = P_{LT}\left[1 + \frac{(N-1)g}{g_f + \frac{g_u}{E_{L1}} + 4.24}\right] \quad (34)$$

Where, P_L = proportion of left turns in shared lane;

N = number of lanes in lane group.

7.6 Left-Turn Adjustment Factor

The left-turn adjustment factor for the lane from which permitted left turns are made can be computed by using Equation 35 (TRB, 2000).

$$f_m = \left(\frac{g_f}{g}\right) + \left(\frac{g_u}{g}\right)\left[\frac{1}{1 + P_L(E_{L1} - 1)}\right] \quad (35)$$

Where, f_m = left-turn adjustment factor applied only to lane.

7.7 Minimum Value of Left-Turn Adjustment Factor

Equation 36 gives the value of left-turn adjustment factor (TRB, 2000).

$$f_{min} = 2(1 + P_L)/g \quad (36)$$

Where, f_{min} = minimum value of f_m.

7.8 Left-Turn Adjustment Factor

Equation 37 (TRB, 2000) gives the left-turn adjustment factor.

$$f_{LT} = \frac{f_m + 0.91(N-1)}{N} \quad (37)$$

Where, f_{LT} = left-turn adjustment factor applied to a total lane group from which left turns are made.

8. Pedestrian and Bicycle Adjustment Factors

The procedure used to calculate pedestrian and bicycle adjustment factors.

8.1 Pedestrian Flow Rate

Pedestrian flow rate first has to be converted from pedestrian volume by using Equation 38 (TRB, 2000).

$$v_{pedg} = v_{ped} * (C/g_p) \qquad (v_{pedg} \leq 5000) \qquad (38)$$

Where, v_{pedg} = pedestrian flow rate;
v_{ped} = pedestrian volume.

8.2 Bicycle Flow Rate

Bicycle flow rate first has to be converted from bicycle volume by using Equation 39 (TRB, 2000).

$$v_{bicg} = v_{bic} * (C/g) \qquad (v_{bicg} \leq 1900) \qquad (39)$$

Where, v_{bicg} = bicycle flow rate;
v_{bic} = bicycle volume.

8.3 Average Pedestrian Occupancy

Equation 40 and 41 (TRB, 2000) giving the average pedestrian occupancy.

$$OCC_{pedg} = 0.4+v_{pedg}/10,000 \qquad (1000 < v_{pedg} < 5,000) \qquad (41)$$

Where, OCC_{pedg} = average pedestrian occupancy.

8.4 Pedestrian Occupancy

For left-turn movements from a two-way street, opposing queue clearing time, g_q, is first compared with pedestrian green, g_p. Pedestrian occupancy after the opposing queue clears is determined by using Equation 42 (TRB, 2000).

$$OCC_{pedu} = OCC_{pedg} [1 - 0.5(g_q/g_p)] \quad (42)$$

Where, OCC_{pedu} = pedestrian occupancy.

8.5 Bicycle Conflict Zone Occupancy

Equation 43 (TRB, 2000) gives the bicycle conflict zone occupancy.

$$OCC_{bicg} = 0.02 + v_{bicg}/2700 \quad (v_{bicg} \leq 1900 \text{ and } OCC_{bicg} \leq 0.72) \quad (43)$$

Where, OCC_{bicg} = bicycle conflict zone occupancy.

8.6 Relevant Conflict Zone Occupancy

Relevant conflict zone occupancy is a function of the probability of acceptable gap availability and pedestrian occupancy and is computed by using Equation 44 (TRB, 2000).

$$OCC_r = OCC_{pedu} \left[e^{-(5/3600)v_o}\right] \quad (44)$$

Equation 45 (TRB, 2000) is determined from combined pedestrian occupancy and bicycle conflict zone occupancy.

$$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg}) \quad (45)$$

Where, OCC_r = relevant conflict zone occupancy.

8.7 Permitted Phase Pedestrian-Bicycle Adjustment Factor

Permitted phase pedestrian-bicycle adjustment factor can be calculated by using Equation 46 and 47 (TRB, 2000).

$$A_{pbT} = 1 - OCC_r \qquad (N_{rec} = N_{turn}) \qquad (46)$$

$$A_{pbT} = 1 - 0.6(OCC_r) \qquad (N_{rec} > N_{turn}) \qquad (47)$$

Where, A_{pbT} = permitted phase pedestrian-bicycle adjustment factor;
N_{rec} = number of receiving lanes; and
N_{turn} = number of turning lanes.

8.8 Proportion of Left-Turn Movements

The proportion of left-turn movements can be calculated by using Equation 48 (TRB, 2000).

$$P_{LTA} = (1 - \text{permitted phase } f_{LT})/0.95 \qquad (48)$$

Where, P_{LTA} = proportion of LT protected green over total LT green,

8.9 Pedestrian-Bicycle Adjustment Factor for Left-Turn Movements

Saturation flow adjustment factors account for pedestrian-bicycle effects on saturation flow for turning vehicles and the factors are dependent on the proportion of turning traffic using protected phases. For left-turn movements, the pedestrian-bicycle adjustment factor can be calculated by using Equation 49 (TRB, 2000).

$$f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA}) \qquad (49)$$

Where, f_{Lpb} = pedestrian-bicycle adjustment factor for left-turn movements, 8.10 Pedestrian-Bicycle Adjustment Factor for Right-Turn Movements

For right-turn movements, the pedestrian-bicycle adjustment factor can be calculated by using Equation 50 (TRB, 2000).

$$f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})\,(1 - P_{RTA}) \qquad (50)$$

Where, f_{Rpb} = pedestrian-bicycle adjustment factor for right-turn movements,

P_{RTA} = proportion of RT protected green over total RT green

9. Determining Capacity and v/c Ratio

9.1 Capacity

Capacity at signalized intersections is based on the concept of saturation flow and saturation flow rate. The flow ratio for a given lane group is defined as the ratio of the actual or projected demand flow rate for the lane group (v_i) and the saturation flow rate (s_i). The flow ratio is given the symbol $(v/s)_i$ for lane group i. the capacity of a given lane group may be stated as shown in Equation 51 (TRB,2000).

$$c_i = s_i \times g_i/C \qquad (51)$$

Where, c_i = capacity of lane group i (veh/h),

s_i = saturation flow rate for lane group i (veh/h), and

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

9.2 v/c Ratio

The ratio of flow rate to capacity (v/c), often called the volume to capacity ratio, is typically referred to as degree of saturation (X). For a given lane group i, X_i is computed by using Equation 52 (TRB, 2000).

$$X_i = \left(\frac{v}{c}\right)_i = \frac{v_i}{s_i\left(\frac{g_i}{C}\right)} = \frac{v_i C}{s_i g_i} \quad (52)$$

Where, $X_i = (v/c)_i$ = volume to capacity ratio for lane group i,
v_i = actual demand flow rate for lane group (veh/h),
s_i = saturation flow rate for lane group i (veh/h),
g_i = effective green time for lane group i (s), and
C = cycle length (s).

9.3 v/c ratio for critical lane groups

The critical v/c ratio, X_c. is the v/c ratio for the intersection as a whole, considering only the lane groups that have the highest flow ratio (v/s) for a given signal phase and is determined by using Equation 53 (TRB, 2000).

$$X_c = \sum \left(\frac{v}{s}\right)_{ci} \left(\frac{C}{C-L}\right) \quad (53)$$

Where, X_c = critical v/c ratio for intersection;
$\sum \left(\frac{v}{s}\right)_{ci}$ = summation of flow ratios for all critical lane groups i;

10. Determining Delay

Equation 54 (TRB, 2000) gives the average control delay per vehicle for a given lane group.

$$d = d_1 \times PF + d_2 + d_3 \quad (54)$$

Where, d = control delay per vehicle (s/veh);
d_1 = uniform control delay assuming uniform arrivals (s/veh);
PF = uniform delay progression adjustment factor, which accounts for effects of signal progression;

d_2 = incremental delay to account for effect of random arrivals (s/veh); and

d_3 = initial queue delay (s/veh).

10.1 Progression Adjustment Factor

The progression adjustment factor, PF, applies to all coordinated lane groups. PF affects uniform delay, and for this reason, the adjustment is applied only to d_1. The value of PF may be determined by using Equation 55. Table 13 gives Progression adjustment factor for uniform delay calculation (TRB, 2000).

$$PF = \frac{(1-P)f_{PA}}{1-\left(\frac{g}{C}\right)} \tag{55}$$

Where, PF = progression adjustment factor,

P = proportion of vehicles arriving on green,

g/C = proportion of green time available, and

f_{PA} = supplemental adjustment factor for platoon arriving during green.

Table 13 Progression Adjustment Factor (PF) for Uniform Delay Calculation

Green Ratio (g/C)	Arrival Type (AT)					
	AT 1	AT 2	AT 3	AT 4	AT 5	AT 6
0.20	1.167	1.007	1.000	1.000	0.833	0.750
0.30	1.286	1.063	1.000	0.986	0.714	0.571
0.40	1.445	1.136	1.000	0.895	0.555	0.333
0.50	1.667	1.240	1.000	0.767	0.333	0.000
0.60	2.001	1.395	1.000	0.576	0.000	0.000
0.70	2.556	1.653	1.000	0.256	0.000	0.000

Table 13 (Continued)

Green Ratio (g/C)	Arrival Type (AT)					
	AT 1	AT 2	AT 3	AT 4	AT 5	AT 6
f_{PA}	1.00	0.93	1.00	1.15	1.00	1.00
Default, R_p	0.333	0.667	1.000	1.333	1.667	2.000

Source: TRB (2000)

10.2 Uniform Delay

Equation 56 (TRB, 2000) gives an estimate of delay assuming uniform arrivals, stable flow, and no initial queue. It is based on the first term of Webster's delay formulation and is widely accepted as an accurate depiction of delay for the idealized case of uniform arrivals (7). Note that values of X beyond 1.0 are not used in the computation of d_1.

$$d_1 = \frac{0.5C\left(1-\frac{g}{C}\right)^2}{1-\left[\min(1,X)\frac{g}{C}\right]} \quad (56)$$

Where, d_1 = average delay per vehicle due to uniform arrivals in seconds,
C = cycle length in seconds,
g = effective green time for lane group in seconds, and
X = v/c ratio for lane group.

10.3 Incremental Delay

Equation 57 (TRB, 2000) is used to estimate the incremental delay due to nonuniform arrivals and temporary cycle failures (random delay) as well as delay caused by sustained periods of oversaturation. The incremental delay term is valid for all values of X, including highly oversaturated lane groups.There is no unmet demand that causes initial queues at the start of the analysis period (T).

$$d_2 = 900T\left[(X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}\right] \tag{57}$$

Where, d_2 = incremental delay per vehicle in seconds,

T = duration of analysis period in h,

I = upstream filtering/metering adjustment factor, and

K = incremental delay adjustment factor

10.4 Initial Queue Delay

When a residual queue from a previous times period causes an initial queue to occur at the start of the analysis period (T), additional delay is experienced by vehicles arriving in the period since the initial queue must first clear the intersection. If this is the case, a value of zero is used for d_3 (TRB, 2000).

10.5 Aggregated Delay Estimates

The procedure for delay estimation yields the control delay per vehicle for each lane group. It is often desirable to aggregate these values to provide delay for an intersection approach and for the intersection as a whole. This aggregation is done by computing weighted averages, where the lane group delays are weighted by the adjusted flows in the lane groups. The delay for an approach is computed by using Equation 58. Control delays on the approaches can be further aggregated using Equation 59 to provide the average control delay for the intersection (TRB, 2000).

$$d_A = \frac{\sum_i d_i v_i}{\sum_i v_i} \tag{58}$$

Where, d_A = average delay per vehicle for approach A in seconds,

d_i = average delay per vehicle for lane group i in seconds, and

v_i = analysis flow rate for lane group i in veh/h.

$$d_I = \frac{\sum_A d_A v_A}{\sum_A v_A} \qquad (59)$$

Where, d_I = average delay per vehicle for the intersection in seconds,
v_A = analysis flow rate for approach A in veh/h.

11. Determining Level of Service

Intersection LOS is directly related to the average control delay per vehicle. Table 14 is consulted, and the appropriate LOS is determined (TRB, 2000).

Table 14 Level-of-service Criteria for Signalized Intersections

LOS	Average Control Delay (s/veh)
A	≤ 10
B	> 10-20
C	> 20-35
D	> 35-55
E	> 55-80
F	> 80

Source: TRB (2000)

12. Nonparametric Hypothesis Test

The H test, or Kruskal-Wallis test, is a generation of the U test (also called the Wilcoxon test or the Mann-Whitney test) in that it enables to test the null hypothesis that k independent random samples come from identical populations. The H test or Kruskal-Wallis test is determined by using Equation 60 (Miller *et al*., 1990).

$$H = \frac{12}{n(n+1)} \sum_{i=1}^{k} \frac{R_i^2}{n_i} - 3(n+1) \quad (60)$$

Where, H = H test, or Kruskal-Wallis test;

K = independent random samples come from identical population;

R_i = sum of the rank;

n_i = observations of the i^{th} sample;

$n = n_1 + n_2 + \ldots + n_k$ = sum of observations of the k^{th} sample.

MATERIALS AND METHODS

Materials

1. PC-Computer with CPU speed 2.0 GHz and 1.99 GB of RAMS
2. Microsoft Excel (2007)

Methods

1. Literature Review

The past literature related to the behavior of three major intersections in Yangon was reviewed for the existing signalized intersection and the existing intersections were transformed into channelized intersection with signal control and roundabout intersection to be used in this study. Highway Capacity Manual (2000) and Federal Highway Administration (2000) methods for control delay calculation of signalized, channelized and roundabout intersections were studied to compute average control delay provided by the level of service (LOS) and also by the improvement of the intersections. The channelized intersections with signal control designs were based on existing intersections using American Association of State Highway and Transportation Officials (2001) “A Policy on Geometric Design of Highways and Streets” and Federal Highway Administration (2005) “Urban Intersection Design Guide”. The roundabout intersection designs were also made using the gap acceptance approach by Federal Highway Administration (2000) “Roundabouts: An informational guide” and National Cooperative Highway Research Program (2010) “Roundabouts: An informational guide”.

2. Data collection for three major intersections

Before analyzing for this research, traffic volumes data collection are needed for three major intersections. Traffic volume data for 4 days was 15 minute interval during the morning peak hours (7:00 a.m-10:00 a.m) and evening peak hours (4:00

p.m-7:00 p.m) on Monday and Friday, (6:00 a.m-21:00 p.m) on Wednesday and Sunday, and also pedestrians volume data during the morning and evening peak hours on each intersection, existing geometry, signal timing, and phasing.

3. Traffic Analysis at Exiting Signalized Intersections According to HCM 2000

Three intersections were operating in three-phase for Myaynigone (four-legged) and four-phase for Hledan and Tamwe (five-legged) intersections. Cycle length for Myaynigone four legs was (162) seconds, for Hledan five legs (271) seconds, and Tamwe five legs (196) seconds. The traffic control system at existing signalized intersection for Myaynigone (Pyay Road), for Hledan (Insein Road (1) & Road (2)) and Tamwe (East Race Course Road & Ba Nyar Da La Road (1)) were left-turn early control system, and pretimed traffic signal control. Average control delay (intersection overall delay) was calculated by using Highway Capacity Manual formula. Level of service (LOS) was determined by average control delay.

3.1 Existing Myaynigone Signalized Intersection

Figure 14 shows Myaynigone signalized intersection geometry and peak-hour traffic volumes. Peak hour traffic volumes for Myaynigone signalized intersection is shown in Table 15. All the general data and calculations concerning Myaynigone signalized intersection are in appendix A.

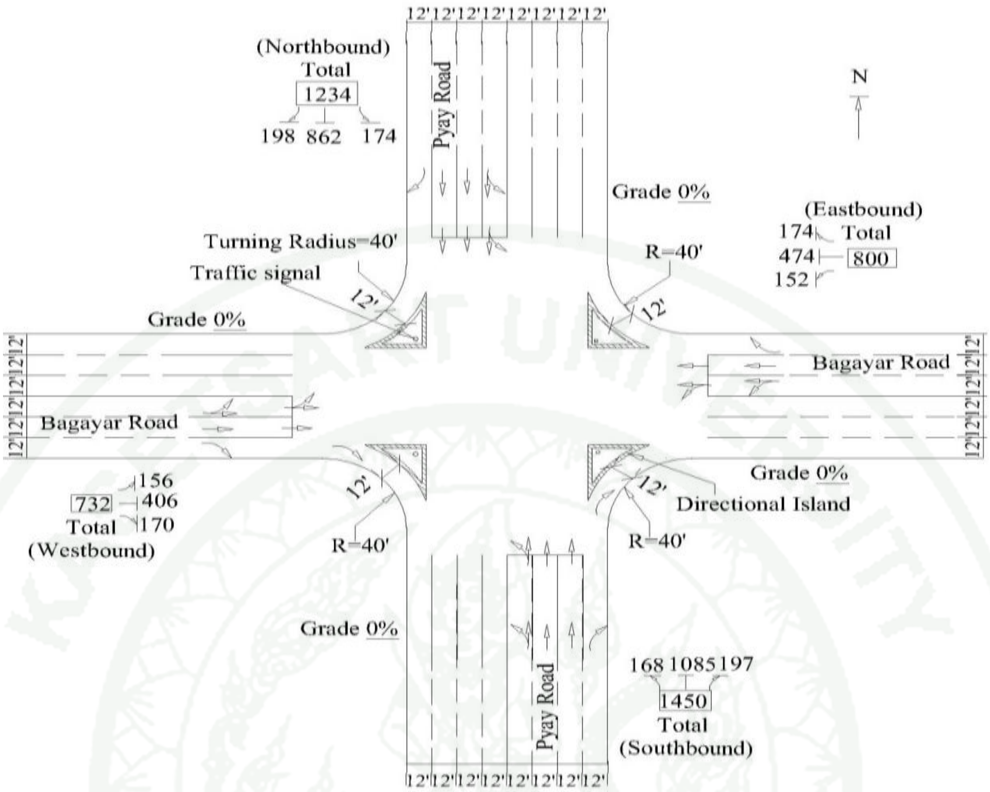


Figure 14 Myaynigone signalized intersection geometry and peak-hour traffic volumes

Table 15 Peak-Hour Traffic Volumes for Myaynigone Signalized Intersection

Time	Eastbound			Westbound			Northbound			Southbound		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
8:00-8:15 A.M	31	114	47	30	99	40	36	209	40	45	264	52
8:15-8:30 A.M	37	119	43	36	94	43	44	219	52	39	276	48
8:30-8:45 A.M	40	128	46	47	108	46	48	227	55	43	274	53
8:45-9:00 A.M	44	113	38	43	105	41	46	207	51	41	271	44
Total	152	474	174	156	406	170	174	862	198	168	1085	197

Table 16 shows peak-hour factor, pedestrian volume and effective cross walk width for Myaynigone signalized intersection.

Table 16 Peak-Hour Factor, Pedestrian Volumes and Effective Cross Walk Width for Myaynigone Signalized Intersection

	Eastbound	Westbound	Northbound	Southbound
Peak-hour factor (PHF)	0.93	0.91	0.93	0.98
Pedestrian volume (P/h)	80	80	100	100
Effective cross walk width, W_E (ft)	6	6	8	8

Table 17 shows aggregated delay and level-of-service (LOS) for Myaynigone signalized intersection.

Table 17 Aggregated Delay and Level-of-Service for Myaynigone Signalized Intersection

	Eastbound	Westbound	Northbound	Southbound
Approach delay, d_A (s/veh)	44.5	43.1	31.7	34.4
LOS for approach	D	D	C	C
Intersection delay, d_I (s/veh)	37.1			
LOS for intersection	D			

3.2 Existing Hledan Signalized Intersection

Hledan signalized intersection geometry and peak-hour traffic volumes are shown in Figure 15. Table 18 shows peak-hour traffic volumes for Hledan signalized intersection. All the general data and calculations concerning Hledan signalized intersection are in appendix D.

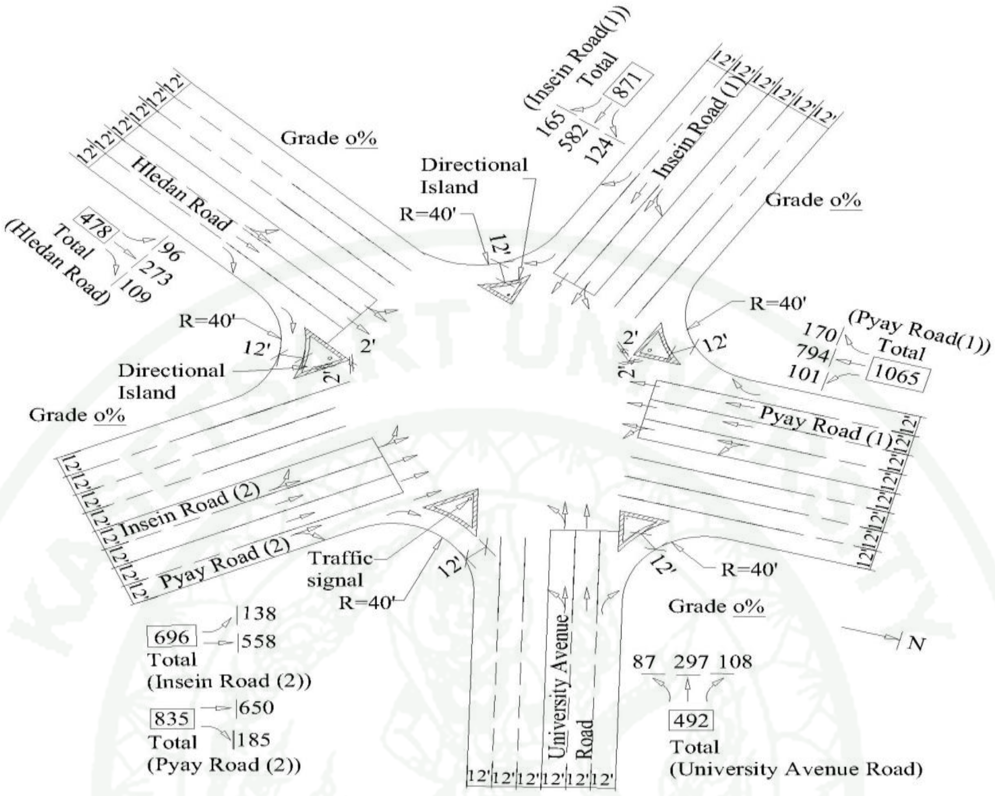


Figure 15 Hledan signalized intersection geometry and peak-hour traffic volumes

Table 18 Peak-Hour Traffic Volumes for Hledan Signalized Intersection

Time	Pyay Road (1)			Pyay Road (2)			Insein Road (1)			Insein Road (2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
8:00-8:15 A.M	22	194	38	-	161	50	28	138	37	32	136	-	21	65	22	18	75	26
8:15-8:30 A.M	26	196	45	-	157	48	30	143	40	30	142	-	23	66	26	21	72	31
8:30-8:45 A.M	28	200	41	-	164	42	34	149	46	35	141	-	27	72	31	25	74	24
8:45-9:00 A.M	25	204	46	-	168	45	32	152	42	41	139	-	25	70	30	23	76	27
Total	101	794	170	-	650	185	124	582	165	138	558	-	96	273	109	87	297	108

Table 19 shows Peak-hour factor, pedestrian volume, and effective cross walk width for Hledan signalized intersection. Aggregated delay and level-of-service (LOS) for Hledan signalized intersection are shown in Table 20.

Table 19 Peak-Hour Factor, Pedestrian Volumes and Effective Cross Walk Width for Hledan Signalized Intersection

	Pyay Road (1)	Pyay Road (2)	Insein Road (1)	Insein Road (2)	Hledan Road	University Avenue Road
Peak-hour factor (PHF)	0.97	0.98	0.95	0.97	0.92	0.98
Pedestrian volume (P/h)	120	100	100	100	80	80
Effective cross walk width, W_E (ft)	8	8	8	8	6	6

Table 20 Aggregated Delay and Level-of-Service for Hledan Signalized Intersection

	Pyay Road (1)	Pyay Road (2)	Insein Road (1)	Insein Road (2)	Hledan Road	University Avenue Road
Approach delay, d_A (s/veh)	83.6	85.0	88.2	104.3	101.6	101.1
LOS for approach	F	F	F	F	F	F
Intersection delay, d_I (s/veh)	91.9					
LOS for intersection	F					

3.3 Existing Tamwe Signalized Intersection

Figure 16 shows Tamwe signalized intersection geometry and peak-hour traffic volumes. Peak-hour traffic volumes for Tamwe signalized intersection is shown in Table 21. All the general data and calculations concerning Tamwe signalized intersection are in appendix G.

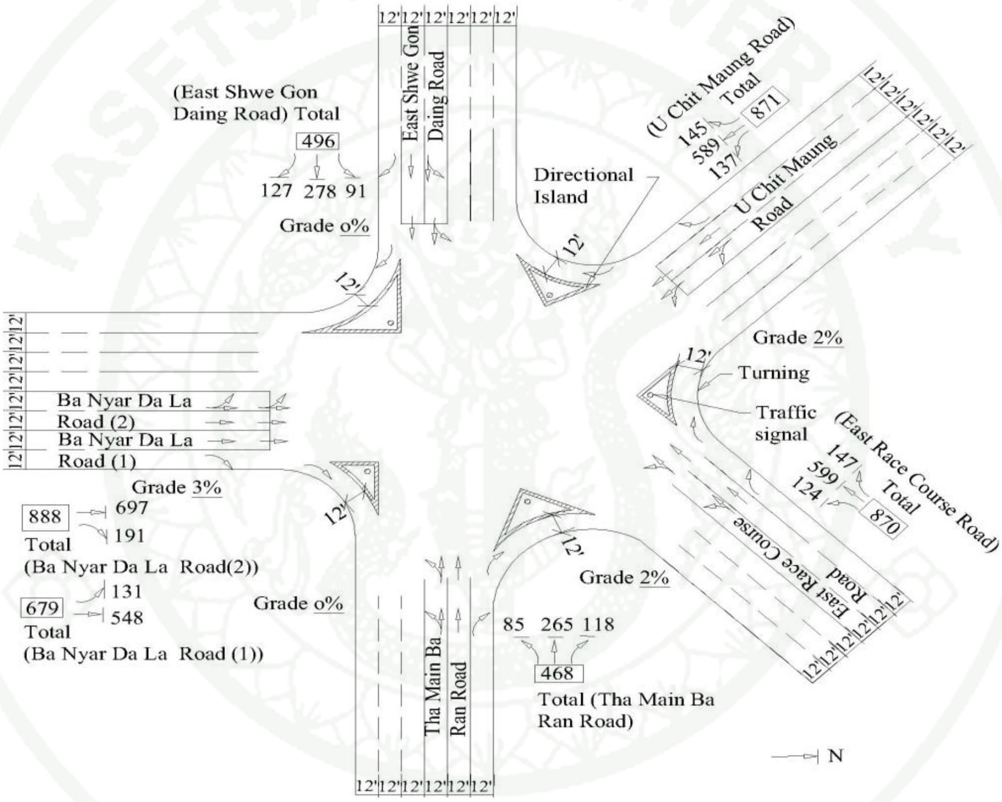


Figure 16 Tamwe signalized intersection geometry and peak-hour traffic volumes

Table 21 Peak-Hour Traffic Volumes for Tamwe Signalized Intersection

Time	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gon Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
5:00-5:15 P.M	28	144	205	33	131	-	31	145	32	-	170	46	22	71	36	19	66	34
5:15-5:30 P.M	30	148	216	30	137	-	35	157	40	-	173	52	25	65	30	20	63	31
5:30-5:45 P.M	34	150	226	36	142	-	33	148	34	-	178	49	24	73	33	25	71	29
5:45-6:00 P.M	32	157	223	32	138	-	38	139	39	-	176	44	20	69	28	21	65	24
Total	124	599	870	131	548	-	137	589	145	-	697	191	91	278	127	85	265	118

Peak-hour factor, pedestrian volume, and effective cross walk width for Tamwe signalized intersection are shown in Table 22. Aggregated delay and level-of-service (LOS) for Tamwe signalized intersection are shown in Table 23.

Table 22 Peak-Hour Factor, Pedestrian Volumes and Effective Cross Walk Width for Tamwe Signalized Intersection

	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Peak-hour factor, (PHF)	0.96	0.95	0.94	0.98	0.95	0.94
Pedestrian volume, (P/h)	80	80	60	80	40	40
Effective cross walk width, W_E (ft)	8	8	8	8	6	6

Table 23 Aggregated Delay and Level-of-Service for Tamwe Signalized Intersection

	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Approach delay, d_A (s/veh)	74.4	107.2	189.8	74.1	78	76.2
LOS for approach	E	F	F	E	E	E
Intersection delay, d_I (s/veh)	104.1					
LOS for intersection	F					

4. Traffic Analysis at Improving Channelized Intersections with Signal Control According to HCM 2000.

The traffic control system at improving channelized (Myaynigone, Hledan, & Tamwe) intersections are the same as existing signalized intersections. Average control delay (intersection overall delay) was calculated by using Highway Capacity Manual formula. Level of service (LOS) was determined by average control.

4.1 Improving Myaynigone channelized intersection with signal control

Figure 17 shows Myaynigone channelized intersection with signal control geometry and peak-hour traffic volumes. Peak-hour traffic volumes, peak-hour factor, pedestrian volume and effective cross walk width are the same as exiting Myaynigone signalized intersection. All the general data and calculations concerning Myaynigone channelized intersection with signal control are in appendix B. Aggregated delay and level-of-service (LOS) for Myaynigone channelized intersection with signal control are shown in Table 24.

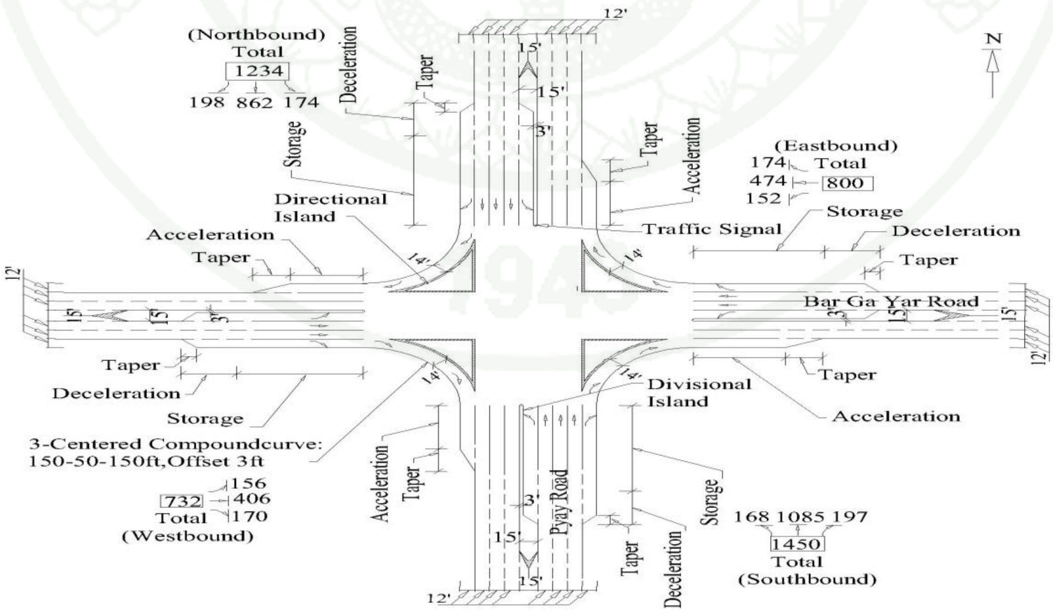


Figure 17 Myaynigone channelized intersection with signal control geometry and peak-hour traffic volumes

Table 24 Aggregated Delay and Level-of-Service for Myaynigone Channelized Intersection with signal control

	Eastbound	Westbound	Northbound	Southbound
Approach delay, d_A (s/veh)	41.6	42.6	30.1	30.8
LOS for approach	D	D	C	C
Intersection delay, d_I (s/veh)	34.8			
LOS for intersection	C			

4.2 Improving Hledan channelized intersection with signal control

Hledan channelized intersection with signal control geometry and peak-hour traffic volumes are illustrated in Figure 18. Peak-hour traffic volumes, peak-hour factor, pedestrian volume and effective cross walk width are the same as existing Hledan signalized intersection. All the general data and calculations concerning Hledan channelized intersection with signal control are in appendix E. Table 25 shows aggregated delay and level-of-service (LOS) for Hledan channelized intersection with signal control.

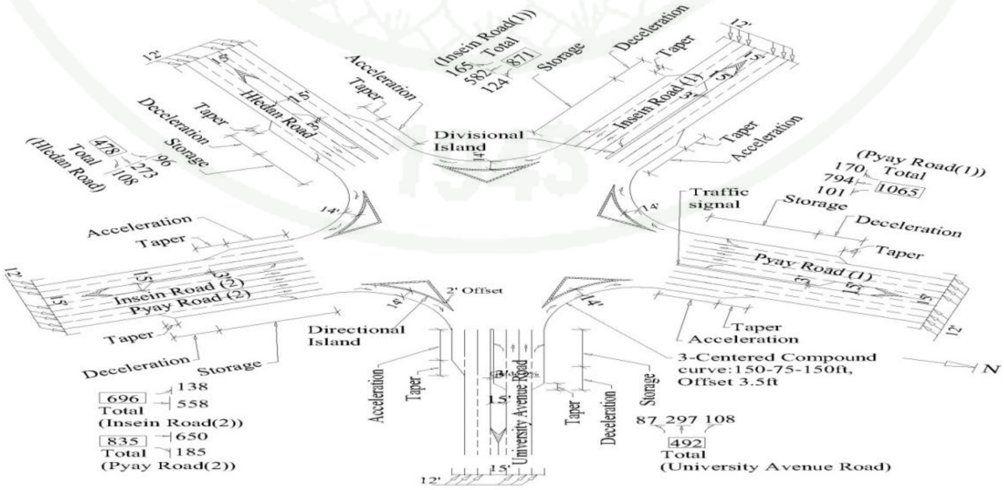


Figure 18 Hledan channelized intersection with signal control geometry and peak-hour traffic volumes

Table 25 Aggregated Delay and Level-of-Service (LOS) for Hledan Channelized Intersection with signal control

	Pyay Road (1)	Pyay Road (2)	Insein Road (1)	Insein Road (2)	Hledan Road	University Avenue Road
Approach delay, d_A (s/veh)	77.1	78.4	73.6	71.5	96.0	94.6
LOS for approach	E	E	E	E	F	F
Intersection delay, d_I (s/veh)	79.8					
LOS for intersection	E					

4.3 Improving Tamwe channelized intersection with signal control

Tamwe channelized intersection with signal control geometry and peak-hour traffic volumes are shown in Figure 19. Peak-hour traffic volumes, peak-hour factor, pedestrian volumes and effective cross walk width are the same as existing Tamwe signalized intersection. All the general data and calculations concerning Tamwe channelized intersection with signal control are in appendix H. Aggregated delay and level-of-service (LOS) for Tamwe channelized intersection with signal control are shown in Table 26.

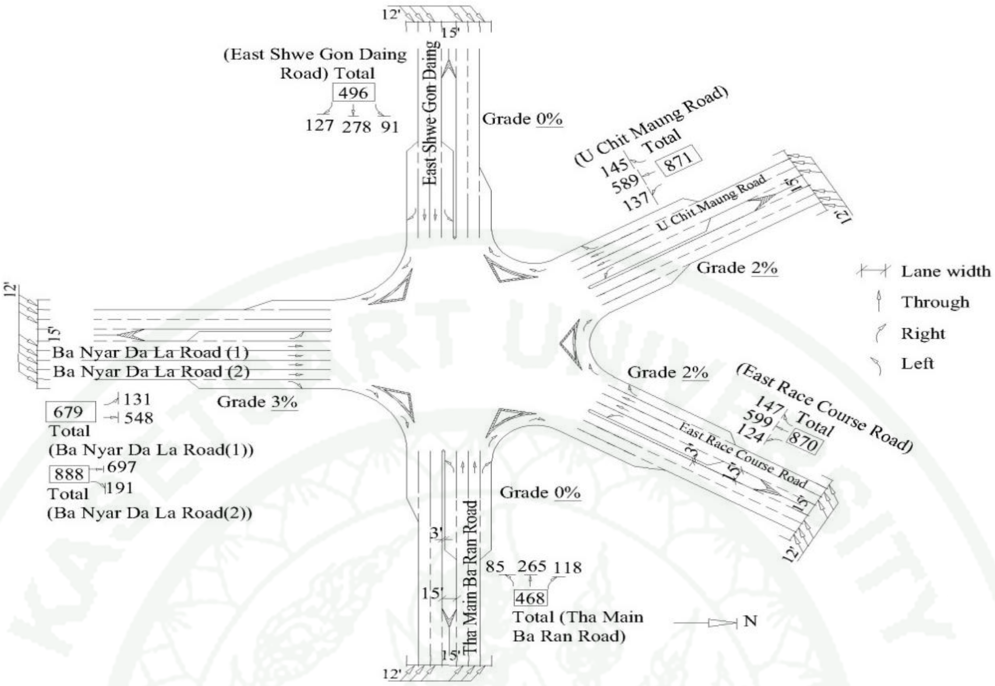


Figure 19 Tamwe channelized intersection with signal control geometry and peak-hour traffic volumes

Table 26 Aggregated Delay and Level-of-Service (LOS) for Tamwe Channelized Intersection with signal control

	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Approach delay, d_A (s/veh)	57.7	56.0	69.1	63.6	67.1	65
LOS for approach	E	E	E	E	E	E
Intersection delay, d_I (s/veh)	62.9					
LOS for intersection	E					

5. Traffic Analysis at Improving Roundabout Intersection According to FHWA 2000.

The traffic control system at improving roundabout (Myaynigone, Hledan, & Tamwe) intersections for all vehicles circulate counterclockwise (in countries with a drive right policy), passing to the right of the central island. Average control delay (intersection overall delay) was calculated by using Federal Highway Administration's formula. Level of service (LOS) was determined by average control delay.

5.1 Improving Myaynigone roundabout intersection

Myaynigone roundabout intersection geometry and peak-hour traffic volumes are shown in Figure 20. Peak-hour traffic volumes, pedestrian volumes and effective cross walk width are the same as existing Myaynigone signalized intersection. All the general data and calculations concerning Myaynigone roundabout intersection are in appendix C. Aggregated delay and level-of-service (LOS) for Myaynigone roundabout intersection are shown in Table 27.

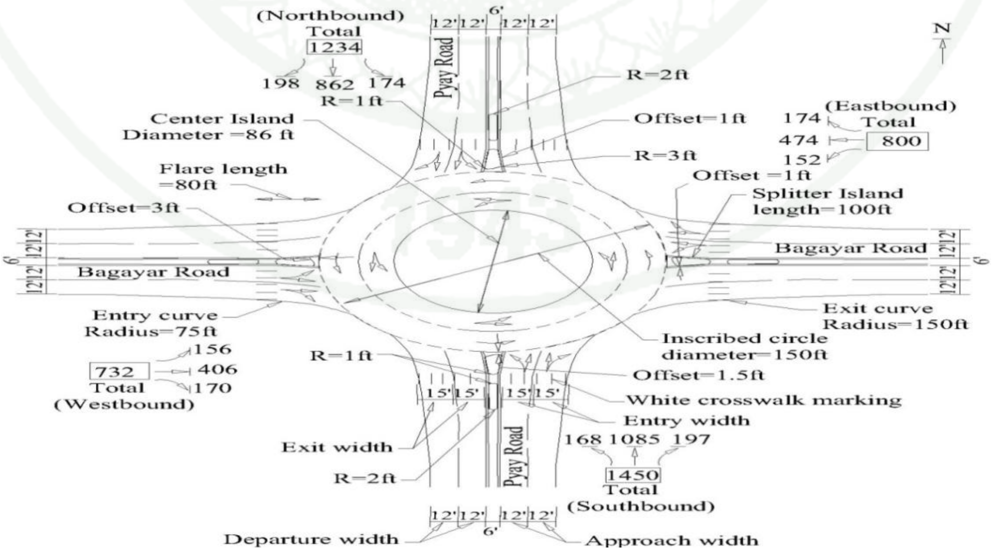


Figure 20 Myaynigone roundabout intersection geometry and peak-hour traffic volumes

Table 27 Aggregated Delay and Level-of-Service (LOS) for Myaynigone Roundabout Intersection

	Eastbound	Westbound	Northbound	Southbound
Approach delay, d_A (s/veh)	6.815	5.074	7.362	10.126
LOS for approach	A	A	A	B
Intersection delay, d_I (s/veh)		7.760		
LOS for intersection		A		

5.2 Improving Hledan roundabout intersection

Figure 21 shows Hledan roundabout intersection geometry and peak-hour traffic volumes. Peak-hour traffic volumes, pedestrian volumes and effective cross walk width are the same as existing Hledan signalized intersection. All the general data and calculations concerning Hledan roundabout intersection are in appendix F. Aggregated delay and level-of-service (LOS) for Hledan roundabout intersection are shown in Table 28.

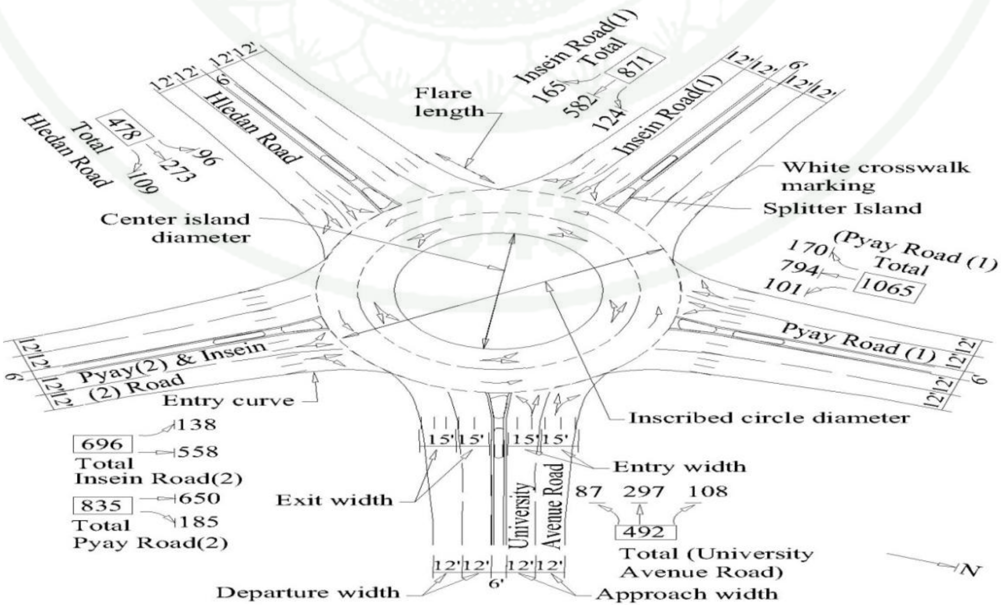


Figure 21 Hledan roundabout intersection geometry and peak-hour traffic volumes

Table 28 Aggregated Delay and Level-of-Service (LOS) for Hledan Roundabout Intersection

	Pyay Road (1)	Pyay Road (2) & Insein Road (2)	Insein Road (1)	Hledan Road	University Avenue Road
Approach delay, d_A (s/veh)	7.526	10.341	7.421	5.468	4.647
LOS for approach	A	B	A	A	A
Intersection delay, d_I (s/veh)			7.918		
LOS for intersection			A		

5.3 Improving Tamwe roundabout intersection

Tamwe roundabout intersection geometry and peak-hour traffic volumes are shown in Figure 22. Peak-hour traffic volumes, pedestrian volumes and effective cross walk width are the same as existing Tamwe signalized intersection. All the general data and calculations concerning Tamwe roundabout intersection are in appendix I. Aggregated delay and level-of-service (LOS) for Tamwe roundabout intersection are shown in Table 29.

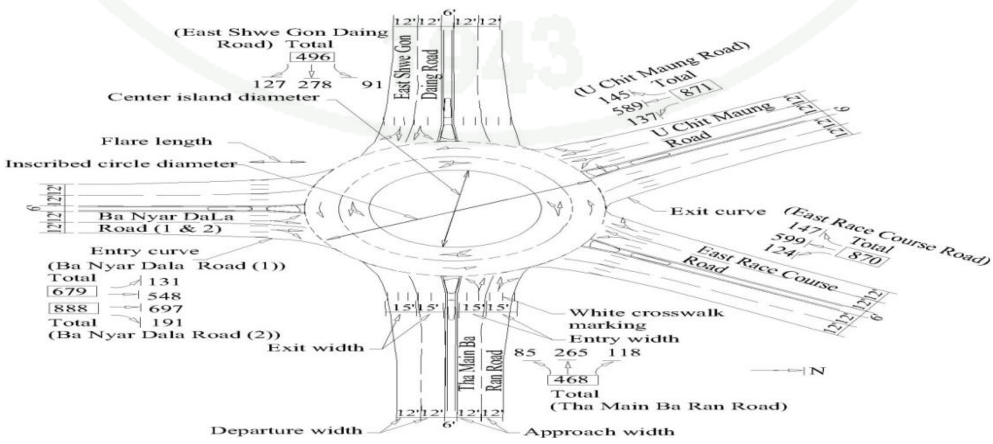


Figure 22 Tamwe roundabout intersection geometry and peak-hour traffic volumes

Table 29 Aggregated Delay and Level-of-Service (LOS) for Tamwe Roundabout Intersection

	East Race Course Road	Ba Nyar Da La Road (1) & Road (2)	U Chit Maung Road	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Approach delay, d_A (s/veh)	5.349	12.179	6.047	4.865	4.945
LOS for approach	A	B	A	A	A
Intersection delay, d_I (s/veh)			7.866		
LOS for intersection			A		

6. Comparison between intersection delay and level of service.

Finally, comparison between before and after improvement using both plans (channelized intersection with signal control and roundabout intersection) of three major intersections (Myaynigone four-legged, Hledan five-legged and Tamwe fivelegged) signalized traffic control system in Yangon were determined according to the average control delay (intersection delay) and level of service (LOS). Table 30 shows comparison between average control delay (intersection delay) and level-of-service for three intersections.

Table 30 Comparison between average control delay and level-of-service for three intersections

	Existing signalized intersection		Channelized intersection with signal control		Roundabout intersection without signal	
	Average control delay	Level of service (LOS)	Average control delay	Level of service (LOS)	Average control delay	Level of service (LOS)
Myaynigone intersection	37.1	D	34.8	C	7.760	A
Hledan intersection	91.9	F	79.8	E	7.918	A
Tamwe intersection	104.1	F	62.9	E	7.866	A

7. Measurement of Effectiveness (MOEs)

The performances of existing signalized intersections and corresponding channelized intersections with signal control and roundabouts are compared on two MOEs: control delay and v/c ratio. Calculated values for signalized intersections and channelized intersections are obtained from Highway Capacity Manual (HCM) 2000 and those for roundabouts are obtained from Federal Highway Administration (FHWA) 2000. A nonparametric hypothesis test is proposed to investigate the performance of candidate signalized intersection, channelized intersection with signal control and roundabouts for different MOEs. The Kruskal-Wallis test provides ranks for comparison of explanatory factors (Miller, I., *et al*., 1990). Microsoft Excel (2007) is simply used for conducting the Kruskal-Wallis test. The MOE for roundabouts, signalized intersections and channelized intersections with signal control are assessed at 95 percent level of confidence.

8. Channelized Intersection Design for Three Intersections

8.1 Myaynigone Channelized Intersection Design

8.1.1 Calculation for acceleration and deceleration length of right-turn and left-turn lanes

Proposed Design: The following are the steps used to generate the proposed design for adding through lanes.

Step 1: Calculate taper length

From Table 2 a taper length of (180 ft [55 m]) is used with a design speed of (40 mph [60 kmph]).

Step 2: Calculate acceleration length

The acceleration distance from a stop condition with a design speed of (40 mph [60 kmph]) is provided also in Table 2, acceleration length is (360 ft [110 m]).

Step 3: Calculate storage length for left-turn lanes.

The required storage length can be estimated according to Table 4 or with the following storage length Equation (1).

$L = (V/N)\ (2)\ (S)$

$S = 25$ ft [7.6 m]

$N = (3600/162) = 22$ cycles

Eastbound Bagayar Road through lanes,

$V = (V_{Thru} + V_{Rt})/NL = (510 + 187)/3 = 232$ vphpl

$L = (232/22) \times 2 \times 25 = 527$ ft [161 m]

Westbound Bagayar Road through lanes,

$V = (446 + 187)/3 = 211$ vphpl

$L = (211/22) \times 2 \times 25 = 480$ ft [146 m]

Northbound Pyay Road through lanes,

V = (927 + 213)/4 = 285 vphpl

L = (285/22) × 2 × 25 = 648 ft [198 m]

Southbound Pyay Road through lanes,

V = (1107 + 201)/4 = 327 vphpl

L = (327/22) × 2 × 25 = 743 ft [226 m]

Step 4: Calculate storage length for left-turn lane

Eastbound Bagayar Road left-turn lane,

L = (163/22) × 2 × 25 = 370 ft [113 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (527 ft [161 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Westbound Bagayar Road left-turn lane,

L = (171/22) × 2 × 25 = 389 ft [119 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (480 ft [146 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Northbound Pyay Road left-turn lane,

L = (187/22) × 2 × 25 = 425 ft [130 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (648 ft [198 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Southbound Pyay Road left-turn lane

L = (171/22) × 2 × 25 = 389 ft [119 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (743 ft [226 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Step 5: Determine taper and deceleration length for left-turn lane

The taper length for the left-turn lanes on Bagayar and Pyay Road are shown in Table 3 as (50 ft [15 m]) using the design speed of (40 mph [60 kmph]). The deceleration length is also provided in Table 3; the given length is (275 ft [85 m]).

Step 6: Provide dimensions for lane lengths

The proposed design for the addition of the through lane on each of the Bagayar and Pyay Road approach along with the left-turn lane are shown in Figure 23.

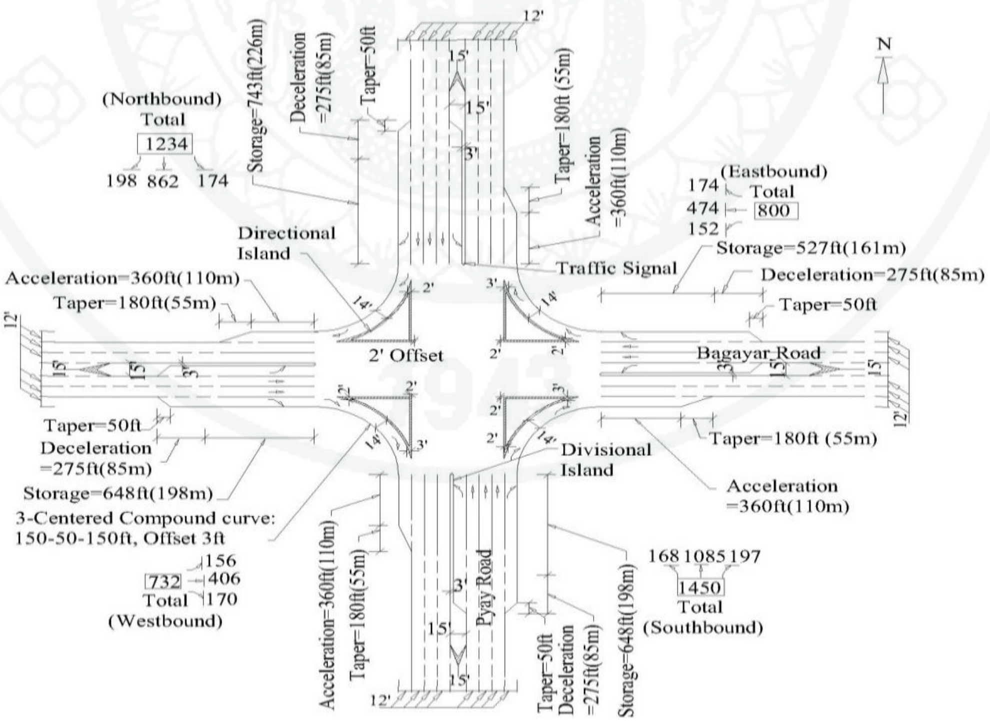


Figure 23 Proposed design for Myaynigone channelized intersection with signal control

Step 7: Select right-turn radius and median-turn radius

The radii used in the intersection design were selected with regard to the location and traffic mix found at the site. A Single Unit Truck design vehicle was used to select the radii for both the right-turn and median turns.

8.1.2 Right-Turn Radius

From Table 1 turning roadway for a 90° right turn to fit these controls. Minimum size island (50 ft^2) and minimum width of channel (14 ft) result in a three-centered compound curve with radii (150-50-150 ft) with the curve offset (3 ft) from the tangent edges extended.

8.1.3 Median-Turn Radius

A (60 ft [18 m]) median-turn radius was selected to accommodate the Single Unit Truck without increasing the size of the intersection more than necessary.

8.2 Hledan Channelized Intersection Design

8.2.1 Calculation for acceleration and deceleration length of right-turn and left-turn lanes

Proposed Design: The following are the steps used to generate the proposed design for adding through lanes.

Step 1: Calculate taper length

From Table 2 a taper length of (180 ft [55 m]) is used with a design speed of (40 mph [60 kmph]).

Step 2: Calculate acceleration length

The acceleration distance from a stop condition with a design speed of 40 mph [60 kmph] is provided also in Table 2, acceleration length is (360 ft [110 m]).

Step 3: Calculate storage length for left-turn lane

The required storage length can be estimated according to Table 4 or with the following storage length Equation (1).

L = (V/N) (2) (S)

S = 25 ft [7.6 m]

N = (3600/271) = 13 cycles

Pyay Road (1) through lanes,

$V = (V_{Thru} + V_{Rt})/NL$ = (819 + 175)/4 = 249 vphpl

L = (249/13) × 2 × 25 = 958 ft [292 m]

Insein Road (1) through lanes,

V = (613 + 174)/3 = 262 vphpl

L = (262/13) × 2 × 25 = 1008 ft [307 m]

Pyay Road (2) and Insein Road (2) through lanes,

V = (1238 + 189)/5 = 285 vphpl

L = (285/13) × 2 × 25 = 1096 ft [334 m]

Hledan Road through lanes,

V = (281 + 108)/3 = 130 vphpl

L = (130/13) × 2 × 25 = 500 ft [152 m]

University Avenue Road through lanes,

V = (303 + 110)/3 = 138 vphpl

L = (138/13) × 2 × 25 = 531 ft [162 m]

Step 4: Calculate storage length for left-turn lane

Pyay Road (1) left-turn lane,

L = (104/13) × 2 × 25 = 400 ft [122 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (958 ft [292 m]).

Therefore, the left-turn storage length should be increased to match that of the through lanes.

Insein Road (1) left-turn lane,

L = (131/13) × 2 × 25 = 504 ft [154 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (1008 ft [307 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Pyay Road (2) and Insein Road (2) left-turn lane,

L = (142/13) × 2 × 25 = 546 ft [166 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (1096 ft [334 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Hledan Road left-turn lane,

L = (99/13) × 2 × 25 = 381 ft [116 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (500 ft [152 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

University Avenue Road left-turn lane,

L = (89/13) × 2 × 25 = 342 ft [104 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (531 ft [162 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Step 5: Determine taper and deceleration length for left-turn lane

The taper length for the left-turn lanes on Pyay Road (1), Insein Road (1), Pyay Road (2) and Insein Road (2),Hledan Road and University Avenue Road are shown in Table 3 as (50 ft [15 m]) using the design speed of (40 mph [60 kmph]). The deceleration length is also provided in Table 3; the given length is (275 ft [85 m]).

Step 6: Provide dimensions for lane lengths

The proposed design for the addition of the through lane on each of the Pyay Road (1), Insein Road (1), Pyay Road (2) and Insein Road (2), Hledan Road and University Avenue Road approach along with the left-turn lane are shown in Figure 24.

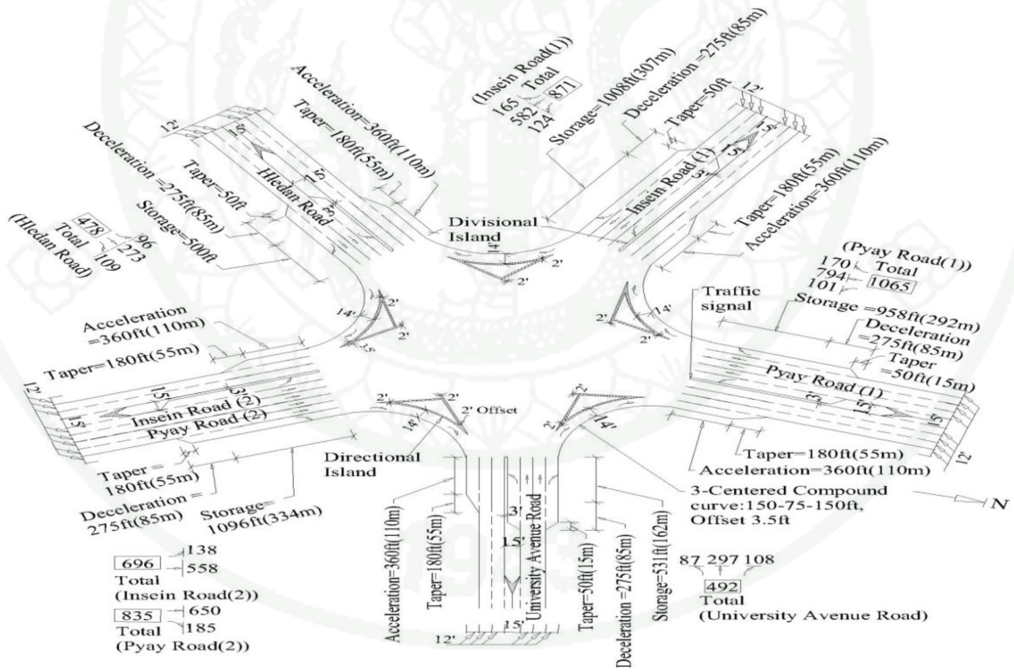


Figure 24 Proposed design for Hledan channelized intersection with signal control

Step 7: Select right-turn radius and median-turn radius

The radii used in the intersection design were selected with regard to the location and traffic mix found at the site. A Single Unit Truck design vehicle was used to select the radii for both the right-turn and median turns.

8.2.2 Right-Turn Radius

From Table 1 turning roadway for a 75° right turn to fit these controls. Minimum size is land (60 ft^2) and minimum width of channel (14 ft) result in a three-centered compound curve with radii (150-75-150 ft) with the curve offset (3.5 ft) from the tangent edges extended.

8.2.3 Median-Turn Radius

A (60 ft [18 m]) median-turn radius was selected to accommodate the Single Unit Truck without increasing the size of the intersection more than necessary.

8.3 Tamwe Channelized Intersection Design

8.3.1 Calculation for acceleration and deceleration length of right-turn and left-turn lanes

Proposed Design: The following are the steps used to generate the proposed design for adding through lanes.

Step 1: Calculate taper length

From Table 2 a taper length of (180 ft [55 km]) is used with a design speed of (40 mph [60 kmph]).

Step 2: Calculate acceleration length

The acceleration distance from a stop condition with a design speed of (40 mph [60 kmph]) is provided also in Table 2, acceleration length is (360 ft [110 m]).

Step 3: Calculate storage length for left-turn lane

From Equation (1) the require storage length can be estimated according to Table 4 or with the following storage length.

L = (V/N) (2) (S)

S = 25 ft [7.6 m]

N = (3600/196) = 18 cycles

East Race Course Road through lanes,

V = (V_{Thru} + V_{Rt})/NL = (624 + 153)/3 = 259 vphpl

L = (259/18) × 2 × 25 = 719 ft [219 m]

Ba Nyar Da La Road (1) and Ba Nyar Da La Road (2) through lanes,

V = (1288 + 195)/5 = 297 vphpl

L = (297/18) × 2 × 25 = 825 ft [251 m]

U Chit Maung Road through lanes,

V = (627 + 154)/3 = 260 vphpl

L = (260/18) × 2 × 25 = 722 ft [220 m]

East Shwe Gon Daing Road through lanes,

V = (293 + 134)/3 = 142 vphpl

L = (142/18) × 2 × 25 = 394 ft [120 m]

Tha Main Ba Ran Road through lanes,

V = (282 + 126)/3 = 136 vphpl

L = (136/18) ×2 × 25 = 378 ft [115 m]

Step 4: Calculate storage length for left-turn lane

East Race Course Road left-turn lane,

L = (129/18) × 2 × 25 = 358 ft [109 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (719 ft [219 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Ba Nyar Da La Road (1) and (2) left-turn lane,

L = (138/18) × 2 × 25 = 383 ft [117 m]

This length is greater than the minimum queue length (100 ft

[30 m]), but less than the storage length for the through lanes (825 ft [251 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

U Chit Maung Road left-turn lane,

L = (146/18) × 2 × 25 = 406 ft [124 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (722 ft [220 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

East Shwe Gone Daing Road left-turn lane,

L = (96/18) × 2 × 25 = 267 ft [81 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (394 ft [120 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Tha Main Ba Ran Road left-turn lane,

L = (90/18) × 2 × 25 = 250 ft [76 m]

This length is greater than the minimum queue length (100 ft [30 m]), but less than the storage length for the through lanes (378 ft [115 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

Step 5: Determine taper and deceleration length for left-turn lane

The taper length for the left-turn lanes on East Race Course Road, Ba Nyar Da La Road (1) and (2), U Chit Maung Road, East Shwe Gon Daing Road and Tha Main Ba Ran Road are shown in Table 3 as (50 ft [15 m]) using the design speed of (40 mph [60 kmph]). The deceleration length is also provided in Table 3; the given length is (275 ft [85 m]).

Step 6: Provide dimensions for lane lengths

The proposed design for the addition of the through lane on each of the East Race Course Road, Ba Nyar Da La Road (1) and (2), U Chit Maung Road, East Shwe Gon Daing Road and Tha Main Ba Ran Road approach along with the left-turn lane are shown in Figure 25.

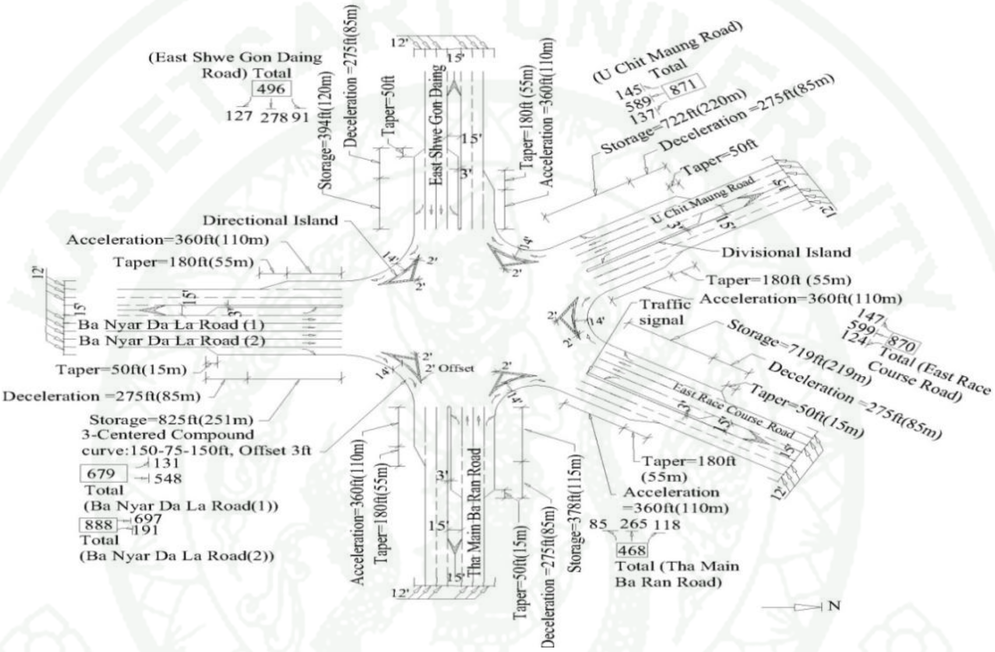


Figure 25 Proposed design for Tamwe channelized intersection with signal control

Step 7: Select right-turn radius and median-turn radius

The radii used in the intersection design were selected with regard to the location and traffic mix found at the site. A Single Unit Truck design vehicle was used to select the radii for both the right-turn and median turns.

8.3.1 Right-Turn Radius

From Table 1 turning roadway for a 75° right turn to fit these controls. Minimum size island (60 ft^2) and minimum width of channel (14 ft) result in a three- centered compound curve with radii (150-75-150 ft) with the curve offset (3.5 ft) from the tangent edges extended.

8.3.2 Median-Turn Radius

A (60 ft [18 m]) median-turn radius was selected to accommodate the Single Unit Truck without increasing the size of the intersection more than necessary.

9. Roundabout Intersection Design for Three Intersections

9.1 Myaynigone Roundabout Intersection Design

9.1.1 Conversion of Turning-Movement Volumes to Roundabout Volumes

Turning-Movement Data -

Percent heavy vehicles for all movements = 0%

Peak-Hour Factor (PHF) (Eastbound) = 0.93

Peak-Hour Factor (PHF) (Westbound) = 0.91

Peak-Hour Factor (PHF) (Northbound) = 0.93

Peak-Hour Factor (PHF) (Southbound) = 0.98

Step 1: Convert Movement Demand Volumes to Flow Rates

Each turning-movement volume given in Table 15 is converted to a demand flow rate by dividing by the peak-hour factor.

$v_{EBL} = V_{EBL}/PHF = 152/0.93 = 163$ pc/h

$v_{EBT} = V_{EBT}/PHF = 474/0.93 = 510$ pc/h

$v_{EBR} = V_{EBR}/PHF = 174/0.93 = 187$ pc/h

$v_{WBL} = V_{WBL}/PHF = 156/0.91 = 171$ pc/h

$v_{WBT} = V_{WBT}/PHF = 406/0.91 = 446$ pc/h

$v_{WBR} = V_{WBR}/PHF = 170/0.91 = 187$ pc/h

$v_{NBL} = V_{NBL}/PHF = 174/0.93 = 187$ pc/h

$v_{NBT} = V_{NBT}/PHF = 862/0.93 = 927$ pc/h

$v_{NBR} = V_{NBR}/PHF = 198/0.93 = 213$ pc/h

$v_{SBL} = V_{SBL}/PHF = 168/0.98 = 171$ pc/h

$v_{SBT} = V_{SBT}/PHF = 1085/0.98 = 1107$ pc/h

$v_{SBR} = V_{SBR}/PHF = 197/0.98 = 201$ pc/h

Step 2: Adjust Flow Rates for Heavy Vehicles

The flow rate for each movement may be adjusted to account for vehicles stream characteristics as follows:

$$f_{HV} = \frac{1}{1+P_T(E_T-1)} = \frac{1}{1+0(2-1)} = 1$$

$v_{EBL} = v_{EBL}/f_{HV} = 152/1 = 152$ pc/h

$v_{EBT} = v_{EBT}/f_{HV} = 510/1 = 510$ pc/h

$v_{EBR} = v_{EBR}/f_{HV} = 187/1 = 187$ pc/h

$v_{WBL} = v_{WBL}/f_{HV} = 171/1 = 171$ pc/h

$v_{WBT} = v_{WBT}/f_{HV} = 446/1 = 446$ pc/h

$v_{WBR} = v_{WBR}/f_{HV} = 187/1 = 187$ pc/h

$v_{NBL} = v_{NBL}/f_{HV} = 187/1 = 187$ pc/h

$v_{NBT} = v_{NBT}/f_{HV} = 927/1 = 927$ pc/h

$v_{NBR} = v_{NBR}/f_{HV} = 213/1 = 213$ pc/h

$v_{SBL} = v_{SBL}/f_{HV} = 171/1 = 171$ pc/h

$v_{SBT} = v_{SBT}/f_{HV} = 1107/1 = 1107$ pc/h

$v_{SBR} = v_{SBR}/f_{HV} = 201/1 = 201$ pc/h

9.1.2 Conversion of Turning-Movement Volumes to Roundabout Volumes

Step 3: Determine Entry Flow Rates by Lane

The entry flow rate is calculated by summing up the movement flow rates that enter the roundabout. Additional lane-use calculations are required for multilane roundabouts.

The entry flow rates are calculated as follows:

$v_{EB} = 163+510+187 = 860$ pc/h

$v_{WB} = 171+446+187 = 804$ pc/h

v_{NB} = 187+927+213 = 1327 pc/h

v_{SB} = 171+1107+201 = 1479 pc/h

Step 4: Determine Circulating Flow Rates

The circulating flow is calculated for each leg. The circulating volumes are the sum of all volumes that will conflict with entering vehicles on the subject approach. The circulating flow is calculating flow is calculated as follows:

$V_{EB\ CIRC}$ = 171+171+1107 = 1449 pc/h

$V_{WB\ CIRC}$ = 163+187+927 = 1277 pc/h

$V_{NB\ CIRC}$ = 171+163+510 = 844 pc/h

$V_{SB\ CIRC}$ = 187+171+446 = 804 pc/h

Step 5: The following Figure 26 illustrates the final volumes converted into Myaynigone roundabout entering and circulation flow rates.

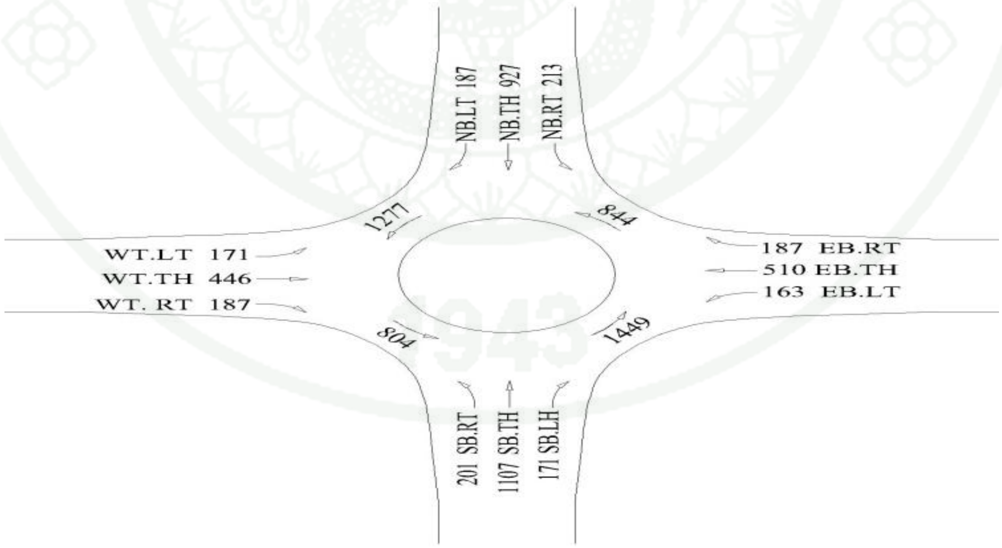


Figure 26 Myaynigone roundabout entering and circulation flow rates

9.1.3 Myaynigone Geometric Design Elements for Double-Lane Roundabout

9.1.3.1 Entry Width

Entry width for a two-lane entry = 30 ft (9.1 m)

Entry width for individual lane = 15 ft (4.6 m)

9.1.3.2 Approach Width

Approach width for a two-lane entry = 24 ft (7.3 m)

Approach width for individual lane = 12 ft (3.7 m)

9.1.3.3 Circulatory Roadway Width

Circulatory roadway width for a two-lane circulatory = 32 ft (9.8 m) (From Table-8)

Circulatory roadway width for individual inner lane = 15 ft (4.6 m)

Circulatory roadway width for individual outer lane = 17 ft (5.2 m)

9.1.3.4 Exit Width

Exit width for a two-lane exit = 30 ft (9.1 m)

Exit width for individual lane = 15 ft (4.6 m)

9.1.3.5 Departure Width

Departure width for individual lane = 12 ft (3.7m)

9.1.3.6 Design Speed

Maximum design speed = 25 mph (40 km/h) (From Table-6)

9.1.3.7 Design Vehicle

Design vehicle = WB-15 (WB-50) Intermediata Semitrailer
Truck (From Table-7)

9.1.3.8 Entry and Exit Radius

Entry radius for double-lane roundabout = 75 ft (22.86 m)
Exit radius for double-lane roundabout = 150 ft (45.72 m)

9.1.3.9 Inscribed Circle Diameter

Inscribed circle diameter for double-lane roundabout = 150 ft (45.72 m) (From Table 8)

9.1.3.10 Central Island Diameter

Central island diameter for double-lane roundabout = 86 ft (26.21 m)

9.1.3.11 Flare Length

Minimum flare length = 80 ft (25 m)

9.1.3.12 Splitter Island

Splitter island width = 6 ft (1.8 m)
Splitter island length = 100 ft (30 m)

9.1.3.13 Stopping Sight Distance, d (ft)

Stopping sight distance (SSD) for double-lane roundabout = 152.7 ft (46.2 m) (From Table-9)

9.1.4 Provide Geometric Design for Double-Lane Roundabout

The proposed Myaynigone intersection geometric design for double-lane roundabout on each of the Pyay Road and Bagayar Road are shown in Figure 27.

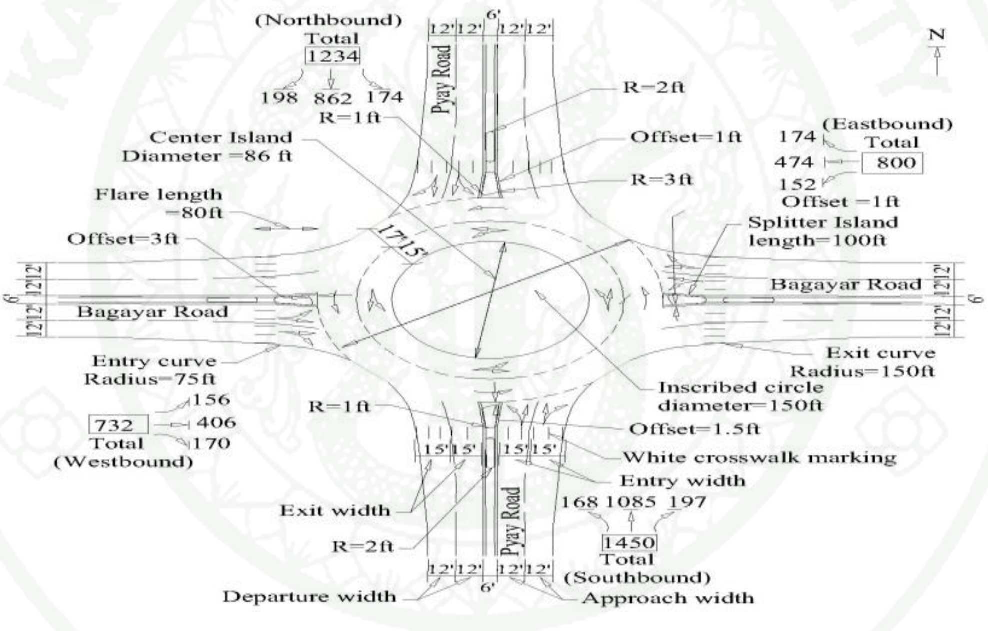


Figure 27 Myaynigone intersection geometric design for double-lane roundabout

9.2 Hledan Roundabout Intersection Design

9.2.1 Conversion of Turning-Movement Volumes to Roundabout Volumes

Turning-Movement Data

Percent heavy vehicles for all movements = 0%

Peak-Hour Factor (PHF) (Pyay Road (1)) = 0.97

Peak-Hour Factor (PHF) (Pyay Road (2)) = 0.98

Peak-Hour Factor (PHF) (Insein Road (1)) = 0.95

Peak-Hour Factor (PHF) (Insein Road (2)) = 0.97

Peak-Hour Factor (PHF) (Hledan Road) = 0.92

Peak-Hour Factor (PHF) (University Avenue Road) = 0.98

Step 1: Convert Movement Demand Volumes to Flow Rates

Each turning-movement volume given in Table 18 is converted to a demand flow rate by dividing by the peak-hour factor.

$v_{\text{Pyay Road (1) L}}$ = 101/0.97 = 104 pc/h

$v_{\text{Pyay Road (1) T}}$ = 794/0.97 = 819 pc/h

$v_{\text{Pyay Road (1) R}}$ = 170/0.97 = 175 pc/h

$v_{\text{Pyay Road (2) T}}$ = 650/0.98 = 663 pc/h

$v_{\text{Pyay Road (2) R}}$ = 185/0.98 = 189 pc/h

$v_{\text{Insein Road (1) L}}$ = 124/0.95 = 131 pc/h

$v_{\text{Insein Road (1) T}}$ = 582/0.95 = 613 pc/h

$v_{\text{Insein Road (1) R}}$ = 165/0.95 = 174 pc/h

$v_{\text{Insein Road (2) L}}$ = 138/0.97 = 142 pc/h

$v_{\text{Insein Road (2) T}}$ = 558/0.97 = 575 pc/h

$v_{\text{Hledan Road L}}$ = 96/0.92 = 104 pc/h

$v_{\text{Hledan Road T}}$ = 273/0.92 = 297 pc/h

$v_{\text{Hledan Road R}}$ = 109/0.92 =118 pc/h

$v_{\text{University Avenue Road L}}$ = 87/0.98 = 89 pc/h

$v_{\text{University Avenue Road T}}$ = 297/0.98 = 303 pc/h

$v_{\text{University Avenue Road R}}$ = 108/0.98 = 110 pc/h

Step 2: Adjust Flow Rates for Heavy Vehicles

The flow rate for each movement may be adjusted to account for vehicles stream characteristics as follows:

$$f_{HV} = \frac{1}{1+P_T(E_T-1)} = \frac{1}{1+0(2-1)} = 1$$

$v_{\text{Pyay Road (1) L}}$ = 104/1 = 104 pc/h

$v_{\text{Pyay Road (1) T}}$ = 819/1 = 819 pc/h

$v_{\text{Pyay Road (1) R}}$ = 175/1 = 175 pc/h

$v_{\text{Pyay Road (2) T}}$ = 663/1 = 663 pc/h

$v_{\text{Pyay Road (2) R}}$ = 189/1 = 189 pc/h

$v_{\text{Insein Road (1) L}}$ = 131/1 = 131 pc/h

$v_{\text{Insein Road (1) T}}$ = 613/1 = 613 pc/h

$v_{\text{Insein Road (1) R}}$ = 174/1 = 174 pc/h

$v_{\text{Insein Road (2) L}}$ = 142/1 = 142 pc/h

$v_{\text{Insein Road (2) T}}$ = 575/1 = 575 pc/h

$v_{\text{Hledan Road L}}$ = 104/1 = 104 pc/h

$v_{\text{Hledan Road T}}$ = 297/1 = 297 pc/h

$v_{\text{Hledan Road R}}$ = 118/1 = 118 pc/h

$v_{\text{University Avenue Road L}}$ = 89/1 = 89 pc/h

$v_{\text{University Avenue Road T}}$ = 303/1 = 303 pc/h

$v_{\text{University Avenue Road R}}$ = 110/1 = 110 pc/h

9.2.2 Conversion of Turning-Movement Volumes to Roundabout Volumes

Step 3: Determine Entry Flow Rates by Lane

The entry flow rate is calculated by summing up the movement flow rates that enter the roundabout. Additional lane-use calculations are required for multilane roundabouts.

The entry flow rates are calculated as follows:

$v_{\text{Pyay Road (1)}}$ = 104+819+175 = 1098 pc/h

$v_{\text{Pyay Road (2)}}$ = 663+189 = 852 pc/h

$v_{\text{Insein Road (1)}}$ = 131+613+174 = 918 pc/h

$v_{\text{Insein Road (2)}}$ = 142+575 = 717 pc/h

$v_{\text{Hledan Road}}$ = 104+297+118 = 519 pc/h

$v_{\text{University Avenue Road}}$ = 89+303+110 = 502 pc/h

Step 4: Determine Circulating Flow Rates

The circulating flow is calculated for each leg. The circulating volumes are the sum of all volumes that will conflict with entering vehicles on the subject approach. The circulating flow is calculated as follows:

$V_{\text{Pyay Road (1) CIRC}}$ = 575+142+89+303+104 = 1213 pc/h

$V_{\text{Insein Road (1) CIRC}}$ = 142+89+303+104+819 = 1457 pc/h

$V_{\text{Hledan Road CIRC}}$ = 104+819+131+613+89 = 1756 pc/h

$V_{\text{Pyay Road (2) \& Insein Road (2) CIRC}}$ = 131+104+297+104 = 636 pc/h

$V_{\text{University Avenue Road CIRC}}$ = 104+142+575+663+131
= 1615 pc/h

Step 5: The following Figure 28 illustrates the final volumes converted into Hledan roundabout entering and circulation flow rates.

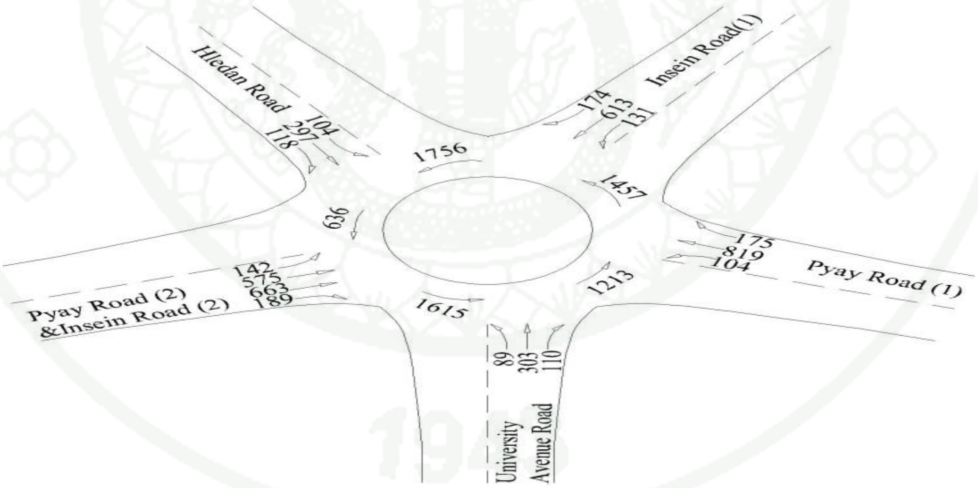


Figure 28 Hledan roundabout entering and circulation flow rates

9.2.3 Hledan Intersection Geometric Design Elements for Double-Lane Roundabout

Hledan intersection geometric design elements for double-lane roundabout are the same as Myaynigone intersection double-lane roundabout.

9.2.4 Provide Geometric Design for Double-Lane Roundabout

The proposed Hledan intersection geometric design for double-lane roundabout on each of the Pyay Road (1), Pyay Road (2) & Insein Road (2), Insein Road (1), Hledan Road and University Avenue Road are shown in Figure 29.

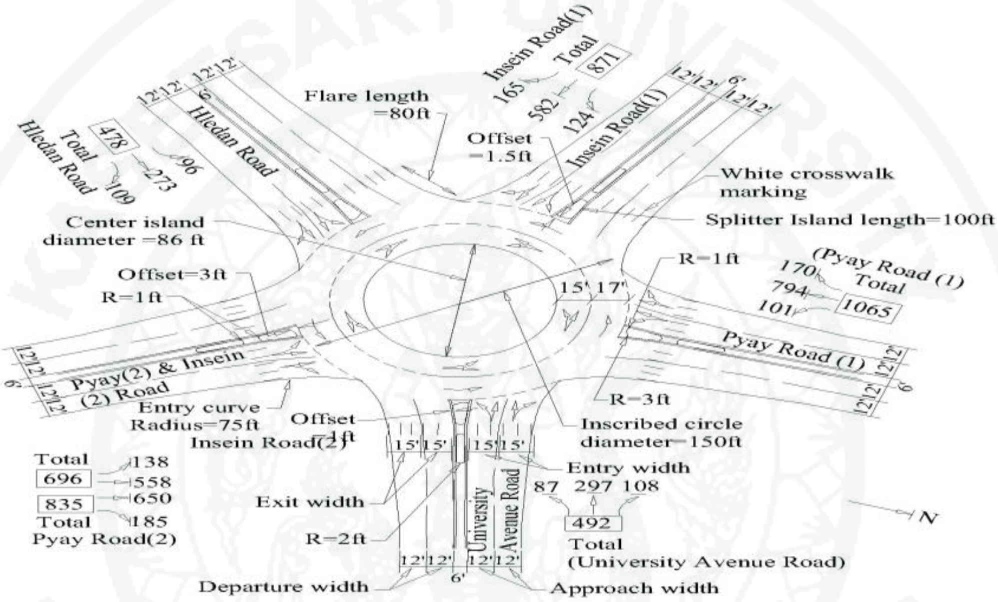


Figure 29 Hledan intersection geometric design for double-lane roundabout

9.3 Tamwe Roundabout Intersection Design

9.3.1 Conversion of Turning-Movement Volumes to Roundabout Volumes

Turning-Movement Data

Percent heavy vehicles for all movements = 0%

Peak-Hour Factor (PHF) (East Race Course Road) = 0.96

Peak-Hour Factor (PHF) (Ba Nyar Da La Road (1)) = 0.95

Peak-Hour Factor (PHF) (U Chit Maung Road) = 0.94

Peak-Hour Factor (PHF) (Ba Nyar Da La Road (2)) = 0.98

Peak-Hour Factor (PHF) (East Shwe Gon Daing Road) = 0.95

Peak-Hour Factor (PHF) (Tha Main Ba Ran Road) = 0.94

Step 1: Convert Movement Demand Volumes to Flow Rates

Each turning-movement volume given in Table 21 is converted to a demand flow rate by dividing by the peak-hour factor.

$v_{\text{East Race Course Road L}}$ = 124/0.96 = 129 pc/h

$v_{\text{East Race Course Road T}}$ = 599/0.96 = 624 pc/h

$v_{\text{East Race Course Road R}}$ = 147/0.96 = 153 pc/h

$v_{\text{Ba Nyar Da La Road (1) L}}$ = 131/0.95 = 138 pc/h

$v_{\text{Ba Nyar Da La Road (1) T}}$ = 548/0.95 = 577 pc/h

$v_{\text{U Chit Maung Road L}}$ = 137/0.94 = 146 pc/h

$v_{\text{U Chit Maung Road T}}$ = 589/0.94 = 627 pc/h

$v_{\text{U Chit Maung Road R}}$ = 145/0.94 = 154 pc/h

$v_{\text{Ba Nyar Da La Road (2) T}}$ = 697/0.98 = 711 pc/h

$v_{\text{Ba Nyar Da La Road (2) R}}$ = 191/0.98 = 195 pc/h

$v_{\text{East Shwe Gon Daing Road L}}$ = 91/0.95 = 96 pc/h

$v_{\text{East Shwe Gon Daing Road T}}$ = 278/0.95 = 293 pc/h

$v_{\text{East Shwe Gon Daing Road R}}$ = 127/0.95 = 134 pc/h

$v_{\text{Tha Main Ba Ran Road L}}$ = 85/0.94 = 90 pc/h

$v_{\text{Tha Main Ba Ran Road T}}$ = 265/0.94 = 282 pc/h

$v_{\text{Tha Main Ba Ran Road R}}$ = 118/0.94 = 126 pc/h

Step 2: Adjust Flow Rates for Heavy Vehicles

The flow rate for each movement may be adjusted to account for vehicles stream characteristics as follows:

$$f_{HV} = \frac{1}{1+P_T(E_T-1)} = \frac{1}{1+0(2-1)} = 1$$

$v_{\text{East Race Course Road L}}$ = 129/1 = 129 pc/h

$v_{\text{East Race Course Road T}}$ = 624/1 = 624 pc/h

$v_{\text{East Race Course Road R}}$ = 153/1 = 153 pc/h

$v_{\text{Ba Nyar Da La Road (1) L}}$ = 138/1 = 138 pc/h

$v_{\text{Ba Nyar Da La Road (1) T}}$ = 577/1 = 577 pc/h

$v_{\text{U Chit Maung Road L}}$ = 146/1 = 146 pc/h

$v_{\text{U Chit Maung Road T}}$ = 627/1 = 627 pc/h

$v_{\text{U Chit Maung Road R}}$ = 154/1 = 154 pc/h

$v_{\text{Ba Nyar Da La Road (2) T}}$ = 711/1 = 711 pc/h

$v_{\text{Ba Nyar Da La Road (2) R}}$ = 195/1 = 195 pc/h

$v_{\text{East Shwe Gon Daing Road L}}$ = 96/1 = 96 pc/h

$v_{\text{East Shwe Gon Daing Road T}}$ = 293/1 = 293 pc/h

$v_{\text{East Shwe Gon Daing Road R}}$ = 134/1 = 134 pc/h

$v_{\text{Tha Main Ba Ran Road L}}$ = 90/1 = 90 pc/h

$v_{\text{Tha Main Ba Ran Road T}}$ = 282/1 = 282 pc/h

$v_{\text{Tha Main Ba Ran Road R}}$ = 126/1 =126 pc/h

9.3.2 Conversion of Turning-Movement Volumes to Roundabout Volumes

Step 3: Determine Entry Flow Rates by Lane

The entry flow rate is calculated by summing up the movement flow rates that enter the roundabout. Additional lane-use calculations are required for multilane roundabouts.

The entry flow rates are calculated as follows:

$v_{\text{East Race Course Road}}$ = 129+624+153 = 906 pc/h

$v_{\text{Ba Nyar Da La Road (1)}}$ = 138+577 = 715 pc/h

$v_{\text{U Chit Maung Road}}$ = 146+627+154 = 927 pc/h

$v_{\text{Ba Nyar Da La Road (2)}}$ = 711+195 = 906 pc/h

$v_{\text{East Shwe Gon Daing Road}}$ = 96+293+134 = 523 pc/h

$v_{\text{Tha Main Ba Ran Road}}$ = 90+282+126 = 498 pc/h

Step 4: Determine Circulating Flow Rates

The circulating flow is calculated for each leg. The circulating volumes are the sum of all volumes that will conflict with entering vehicles on the subject approach. The circulating flow is calculated as follows:

$V_{\text{East Race Course Road CIRC}}$ = 90+282+96+138+577 = 1183 pc/h

$V_{\text{U Chit Maung Road CIRC}}$ = 138+90+282+129+624 = 1263 pc/h

$V_{\text{East Shwe Gon Daing Road CIRC}}$ = 90+129+624+146+627

= 1616 pc/h

$V_{\text{Ba Nyar Da La Road (1) \& (2) CIRC}}$ = 129+146+96+293 = 664 pc/h

$V_{\text{Tha Main Ba Ran Road CIRC}}$ = 138+577+711+146+96 = 1668 pc/h

Step 5: The following Figure 30 illustrates the final volumes converted into Tamwe roundabout entering and circulation flow rates.

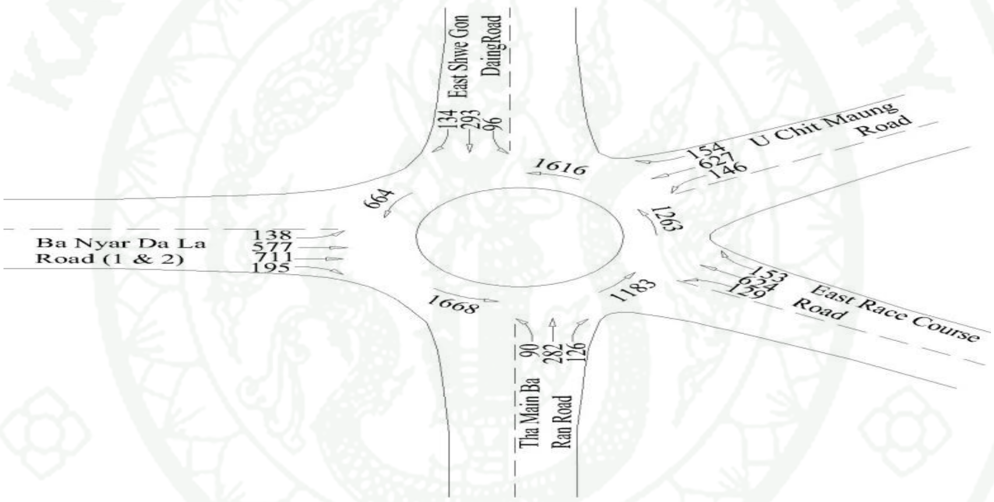


Figure 30 Tamwe roundabout entering and circulation flow rates

9.3.3 Tamwe Intersection Geometric Design Elements for Double-Lane Roundabout

Tamwe intersection geometric design elements for double-lane roundabout are the same as Myaynigone intersection double-lane roundabout.

9.3.4 Provide Geometric Design for Double-Lane Roundabout

The proposed Tamwe intersection geometric design for double-lane roundabout on each of the East Race Course Road, U Chit Maung Road, East Shwe

Gon Daing Road, Ba Nyar Da La Road (1) & (2), and Tha Main Ba Ran Road are shown in Figure 31.

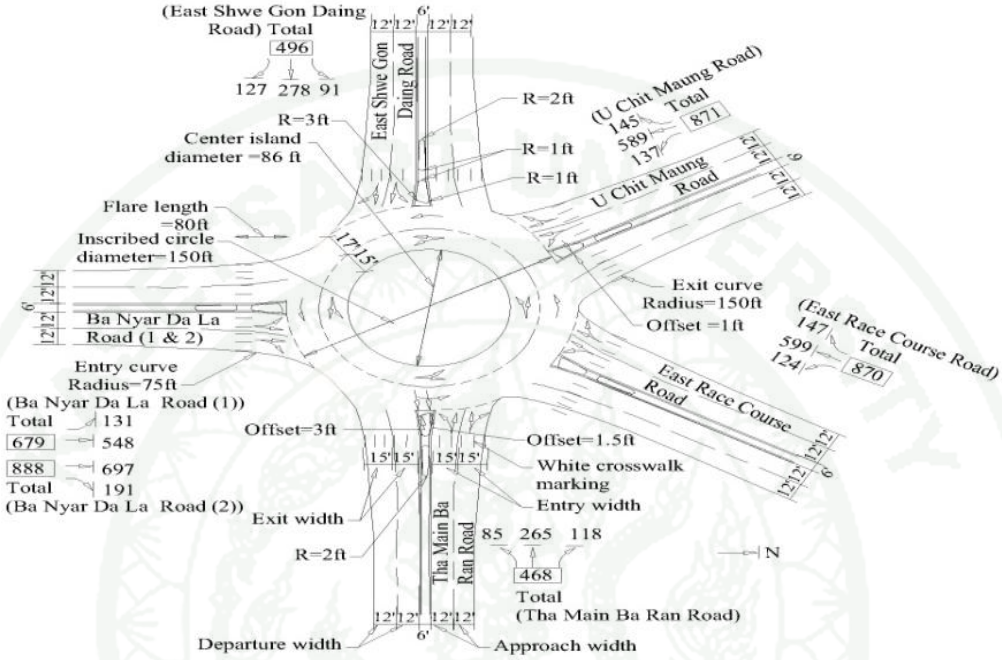


Figure 31 Tamwe intersection geometric design for double-lane roundabout

RESULTS AND DISCUSSIONS

1. Analysis of Three Intersections

For three intersections, the analytical results are presented in term of measure of effectiveness. Each MOE (Control delay and v/c ratio) consist of two sets (types of intersections). These are four-legged intersection and five-legged intersections. For each intersection, channelized intersections with signal control for all entry lane width of 12 ft (3.7 m) and roundabouts of entry lane width of 15 ft (4.6 m) are proposed. Then a total of three comparison groups, existing intersection, channelized intersection with signal control and roundabout are made. A measure of effectiveness and comparison tree is presented in Figure 32.

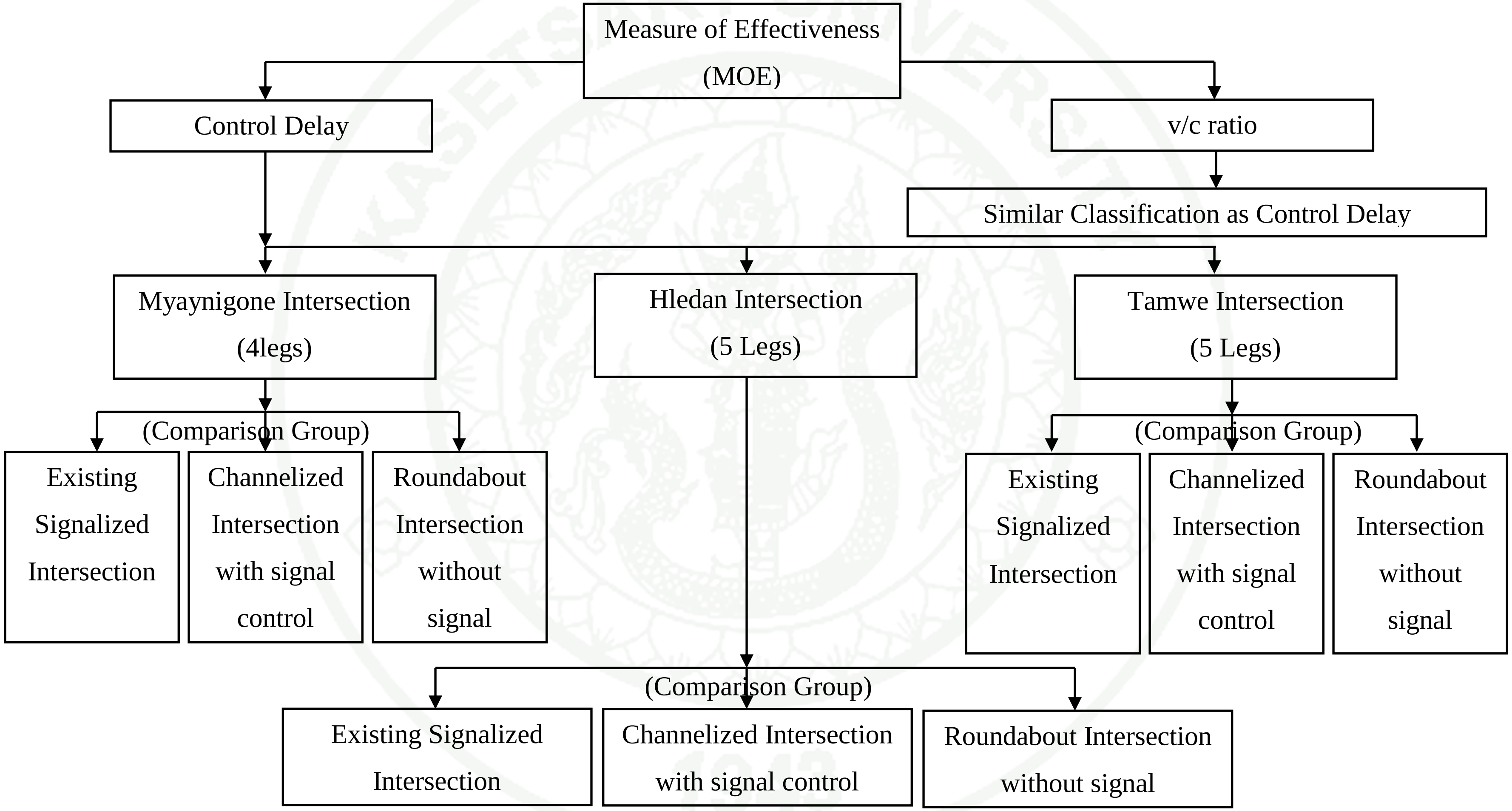


Figure 32 Measure of Effectiveness and Comparison Tree

1.1 Analysis of Myaynigone Intersection

Data collected for this analysis include: traffic volume data during the morning peak hours (7:00 a.m-10:00 a.m) and evening peak hours (4:00 p.m-7:00 p.m) on Monday and Friday, (6:00 a.m-21:00 p.m) on Wednesday and Sunday. Data calculations for programs are writing by using Microsoft Excel (2007). The analysis results for control delays and v/c ratio for existing signalized intersections, channelized intersection with signal control, and roundabouts (Myaynigone intersection) are presented in Table 31. Control delay and v/c ratio for existing signalized intersection, channelized intersection with signal control, and roundabout (Myaynigone intersection) are presented in Figure 33 and Figure 34.

Table 31 Control Delay and v/c Ratio for Myaynigone Intersection

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
6:00 – 7:00	(12.1.2011)	2421	31.244	30.061	2.991	0.428	0.358	0.352
7:00 – 8:00		3411	34.368	32.601	4.362	0.601	0.521	0.513
8:00 – 9:00		4111	36.283	33.881	6.874	0.693	0.607	0.582
9:00 – 10:00		3959	35.828	33.440	6.394	0.663	0.576	0.554
10:00 – 11:00		3783	34.917	32.715	5.405	0.625	0.533	0.516
11:00 – 12:00		3321	33.275	31.376	4.520	0.557	0.481	0.462
12:00 – 13:00		2739	31.510	29.980	3.459	0.455	0.390	0.379
13:00 – 14:00		2541	30.598	29.243	3.197	0.418	0.360	0.338
14:00 – 15:00		2512	30.986	29.629	3.092	0.443	0.344	0.328
15:00 – 16:00		3525	34.577	32.601	4.816	0.604	0.520	0.502
16:00 – 17:00		3787	34.895	32.889	5.388	0.620	0.552	0.526
17:00 – 18:00		3622	34.325	32.445	4.678	0.593	0.521	0.515

Table 31 (Continued)

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
18:00 – 19:00	Wednesday	3290	33.260	31.580	4.231	0.546	0.483	0.467
19:00 – 20:00		2294	29.451	28.450	2.953	0.379	0.331	0.311
20:00 – 21:00		1865	29.302	28.207	2.763	0.339	0.295	0.279
7:00 – 8:00	(14.1.2011)	3091	33.235	31.598	3.588	0.542	0.450	0.434
8:00 – 9:00		4216	36.836	34.462	7.586	0.711	0.638	0.616
9:00 – 10:00		4086	36.280	33.995	6.598	0.678	0.607	0.564
16:00 – 17:00	Friday	3917	35.256	33.320	5.642	0.635	0.578	0.542
17:00 – 18:00		3850	35.230	33.333	5.560	0.640	0.581	0.546
18:00 – 19:00		3002	32.140	30.629	3.616	0.489	0.421	0.412
6:00 – 7:00	(16.1.2011)	1665	29.425	28.710	2.397	0.294	0.251	0.237
7:00 – 8:00		2352	31.519	30.340	2.842	0.416	0.337	0.331
8:00 – 9:00		3109	33.671	32.251	3.837	1.543	0.505	0.472

Table 31 (Continued)

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
9:00 – 10:00	Sunday	3093	33.872	32.392	3.835	0.553	0.483	0.457
10:00 – 11:00		2904	32.830	31.510	3.487	0.500	0.442	0.428
11:00 – 12:00		2377	30.747	29.714	2.867	0.389	0.337	0.329
12:00 – 13:00		2021	29.848	28.884	2.647	0.339	0.285	0.282
13:00 – 14:00		2093	29.664	28.720	2.685	0.343	0.296	0.285
14:00 – 15:00		2515	30.799	29.522	3.145	0.446	0.350	0.345
15:00 – 16:00		2992	32.971	31.462	3.629	0.513	0.434	0.397
16:00 – 17:00		3333	33.977	32.282	4.337	0.567	0.500	0.481
17:00 – 18:00		3295	33.281	31.673	4.083	0.548	0.477	0.462
18:00 – 19:00		2995	32.502	31.000	3.657	0.504	0.437	0.415
19:00 – 20:00		2132	29.380	28.392	2.788	0.345	0.308	0.285
20:00 – 21:00		1666	28.567	27.784	2.444	0.288	0.258	0.225

Table 31 (Continued)

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
7:00 – 8:00		2459	32.410	31.099	2.816	0.460	0.381	0.328
8:00 – 9:00	(17.1.2011)	4043	36.333	33.989	6.825	0.693	0.616	0.578
9:00 – 10:00		3895	35.485	33.248	5.883	0.652	0.564	0.537
16:00 – 17:00		3760	34.791	32.813	5.279	0.626	0.551	0.531
17:00 – 18:00	Monday	3700	34.736	32.868	4.702	0.613	0.556	0.530
18:00 – 19:00		3171	32.946	31.368	3.879	0.533	0.469	0.461

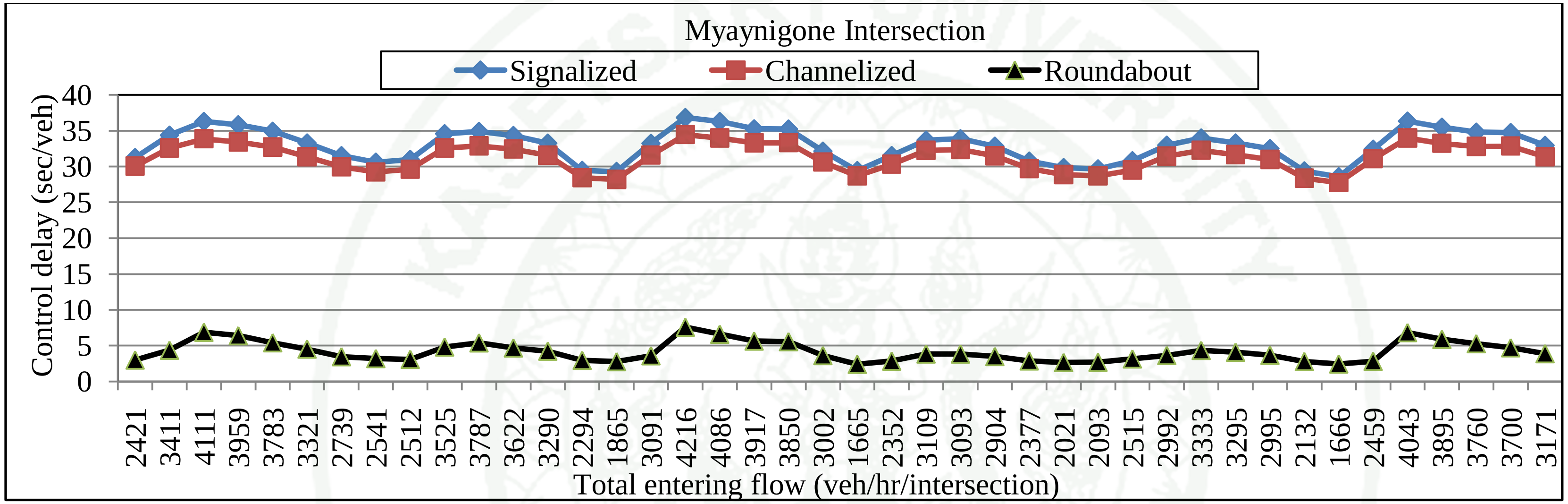


Figure 33 Control delays (Myaynigone intersection) for existing signalized intersection, channelized intersection with signal control, and roundabout intersection

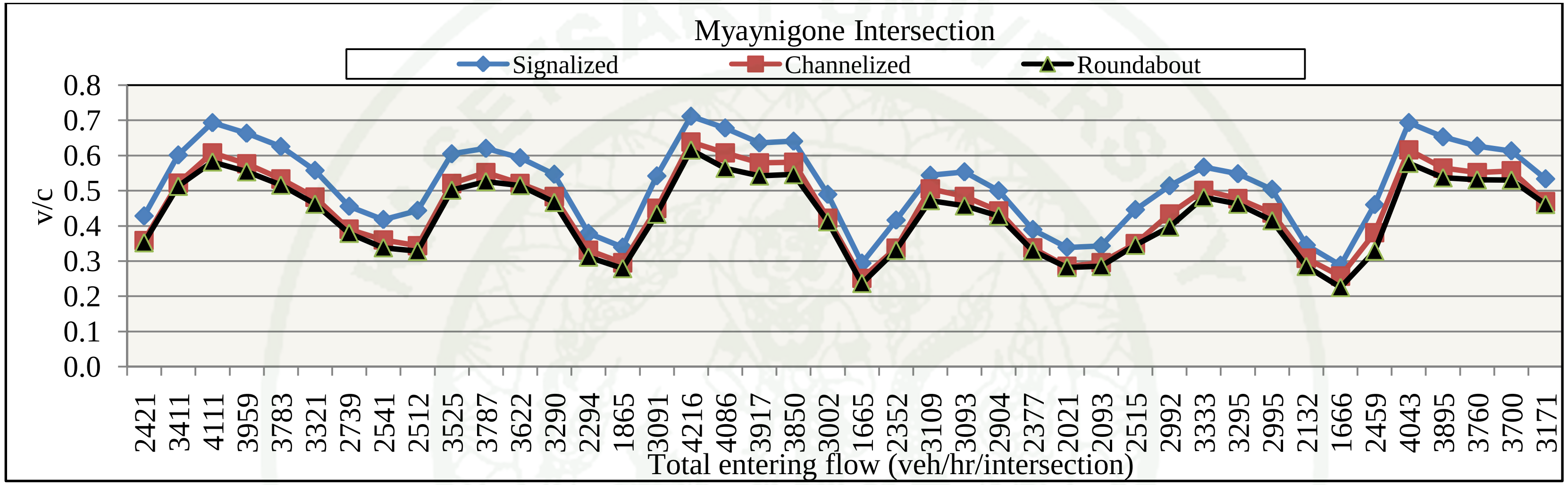


Figure 34 v/c ratios (Myaynigone intersection) for existing signalized intersection, channelized intersection with signal control, and roundabout intersection

1.2 Analysis of Hledan Intersection

Data collected for this analysis include: traffic volume data during the morning peak hours (7:00 a.m-10:00 a.m) and evening peak hours (4:00 p.m-7:00 p.m) on Monday and Friday, (6:00 a.m-21:00 p.m) on Wednesday and Sunday. Data calculations for programs are writing by using Microsoft Excel (2007). The analysis results for control delays and v/c ratio for existing signalized intersections, channelized intersection with signal control, and roundabouts (Hledan intersection) are presented in Table 32. Control delay and v/c ratio for existing signalized intersection, channelized intersection with signal control, and roundabout (Hledan intersection) are presented in Figure 35 and Figure 36.

Table 32 Control Delay and v/c Ratio for Hledan Intersection

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
6:00 – 7:00		2394	76.555	72.958	2.884	0.499	0.359	0.286
7:00 – 8:00		3625	85.236	77.116	5.137	0.759	0.559	0.480
8:00 – 9:00		4437	92.019	79.338	7.527	0.851	0.622	0.598
9:00 – 10:00		4056	90.386	78.995	7.077	0.840	0.609	0.569
10:00 – 11:00		3599	84.590	77.185	4.630	0.736	0.524	0.464
11:00 – 12:00		2843	78.516	74.409	3.259	0.554	0.413	0.342
12:00 – 13:00	(12.1.2011)	2608	77.286	73.492	3.062	0.519	0.385	0.315
13:00 – 14:00		2393	76.417	73.035	2.824	0.490	0.350	0.282
14:00 – 15:00		3363	81.924	75.700	4.048	0.678	0.490	0.410
15:00 – 16:00		3522	84.266	76.492	4.589	0.738	0.531	0.445
16:00 – 17:00		3846	87.725	78.169	6.035	0.795	0.572	0.537
17:00 – 18:00		4124	88.882	78.893	6.943	0.809	0.591	0.572

Table 32 (Continued)

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
18:00 – 19:00		3325	83.081	76.071	4.345	0.709	0.494	0.438
19:00 – 20:00	Wednesday	2333	76.593	73.223	2.862	0.478	0.359	0.289
20:00 – 21:00		2048	74.504	71.756	2.607	0.410	0.298	0.242
7:00 – 8:00		3502	83.430	76.371	4.601	0.721	0.540	0.449
8:00 – 9:00	(14.1.2011)	4174	89.433	78.808	6.968	0.815	0.591	0.572
9:00 – 10:00		3758	86.103	77.639	5.665	0.772	0.557	0.516
16:00 – 17:00		3575	83.764	76.458	4.710	0.715	0.509	0.466
17:00 – 18:00	Friday	3895	86.927	77.983	5.899	0.768	0.558	0.529
18:00 – 19:00		3046	80.619	74.865	3.730	0.649	0.453	0.383
6:00 – 7:00		2110	75.127	72.093	2.650	0.441	0.316	0.250
7:00 – 8:00	(15.1.2011)	3282	81.766	75.326	4.136	0.694	0.492	0.405
8:00 – 9:00		4022	90.237	78.535	7.085	0.834	0.601	0.554

Table 32 (Continued)

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
9:00 – 10:00	Sunday	3551	83.882	76.455	5.692	0.723	0.522	0.494
10:00 – 11:00		3185	81.425	75.183	4.463	0.670	0.474	0.421
11:00 – 12:00		2536	76.842	73.241	2.971	0.500	0.368	0.300
12:00 – 13:00		2237	75.648	72.595	2.696	0.444	0.325	0.263
13:00 – 14:00		2102	74.795	71.987	2.620	0.426	0.301	0.245
14:00 – 15:00		2938	78.858	73.765	3.793	0.597	0.424	0.359
15:00 – 16:00		3178	81.581	75.024	4.252	0.674	0.483	0.403
16:00 – 17:00		3460	83.344	76.288	4.464	0.698	0.506	0.452
17:00 – 18:00		3825	86.208	77.538	5.638	0.770	0.559	0.510
18:00 – 19:00		2935	79.621	74.359	3.519	0.622	0.433	0.362
19:00 – 20:00		2154	74.967	71.993	2.688	0.437	0.321	0.258
20:00 – 21:00		1800	73.077	70.680	2.436	0.365	0.264	0.209

Table 32 (Continued)

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
7:00 – 8:00	(17.1.2011)	3212	80.816	74.795	3.879	0.653	0.485	0.388
8:00 – 9:00		3919	88.343	77.789	6.031	0.791	0.564	0.526
9:00 – 10:00		3544	84.091	76.502	4.751	0.721	0.521	0.459
16:00 – 17:00		3390	82.634	75.846	4.405	0.681	0.490	0.441
17:00 – 18:00	Monday	3705	85.188	76.991	5.284	0.741	0.525	0.492
18:00 – 19:00		2905	79.114	73.980	3.516	0.603	0.422	0.362

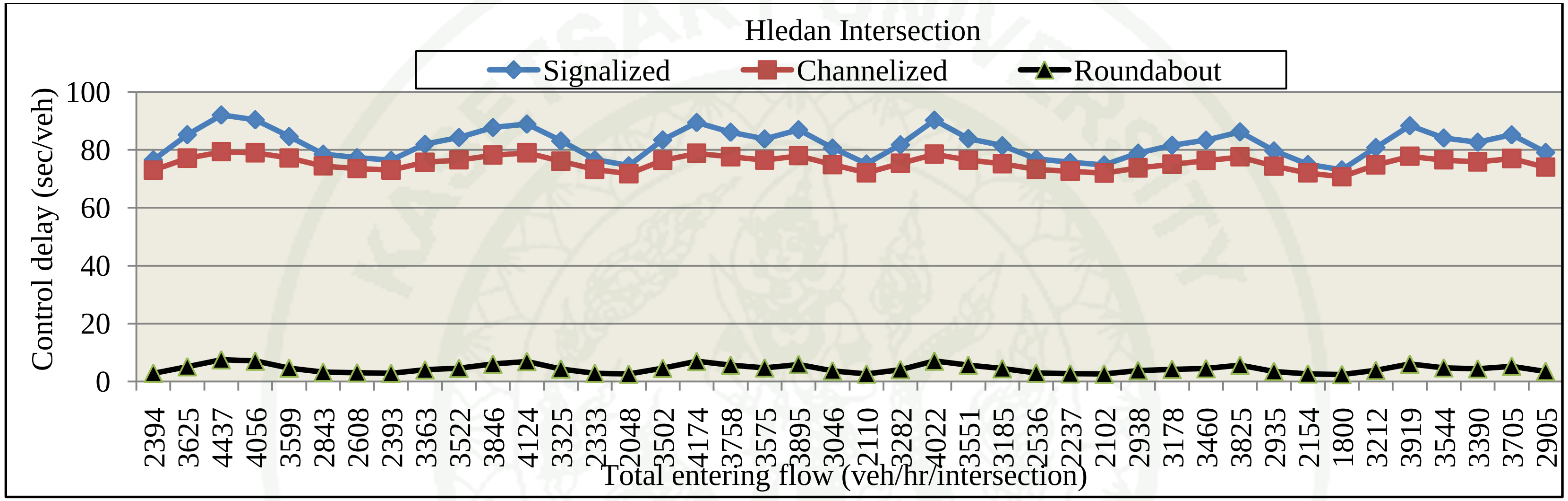


Figure 35 Control delays (Hledan intersection) for existing signalized intersection, channelized intersection with signal control, and roundabout intersection

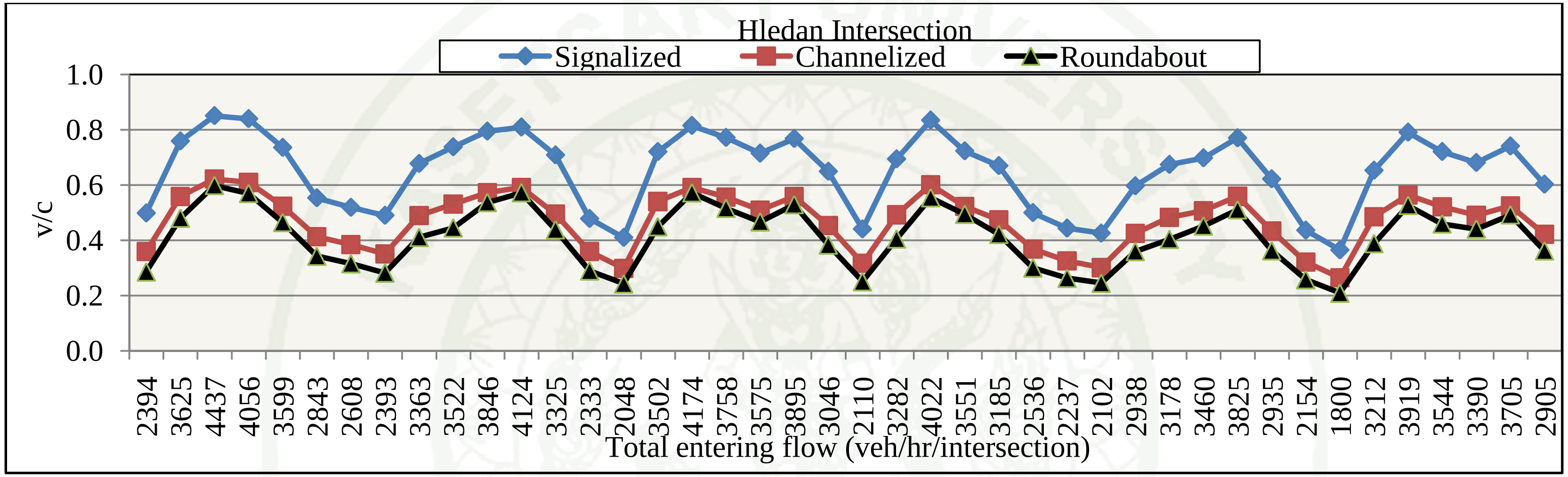


Figure 36 v/c ratios (Hledan intersection) for existing signalized intersection, channelized intersection with signal control, and roundabout intersection

1.3 Analysis of Tamwe Intersection

Data collected for this analysis include: traffic volume data during the morning peak hours (7:00 a.m-10:00 a.m) and evening peak hours (4:00 p.m-7:00 p.m) on Monday and Friday, (6:00 a.m-21:00 p.m) on Wednesday and Sunday. Data calculations for programs are writing by using Microsoft Excel (2007). The analysis results for control delays and v/c ratio (Tamwe intersection) for existing signalized intersections, channelized intersection with signal control, and roundabouts are presented in Table 33. Control delay (Tamwe intersection) is presented in Figure 37, and v/c ratio (Tamwe intersection) is presented in Figure 38 for existing signalized intersection, channelized intersection with signal control, and roundabout.

Table 33 Control Delay and v/c Ratio for Tamwe Intersection

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
6:00 – 7:00				55.088	2.626	0.548	0.375	0.244
7:00 – 8:00				58.455	3.979	0.797	0.539	0.398
8:00 – 9:00				62.032	6.581	0.953	0.677	0.536
9:00 – 10:00		3631	76.551	60.255	4.928	0.860	0.603	0.481
10:00 – 11:00		3296	71.301	59.234	4.268	0.813	0.547	0.438
11:00 – 12:00	(12.1.2011)	2874	63.105	57.293	3.305	0.654	0.453	0.349
12:00 – 13:00		2492	60.632	56.368	2.926	0.586	0.400	0.298
13:00 – 14:00		2201	58.705	55.497	2.673	0.529	0.363	0.257
14:00 – 15:00		3308	70.032	58.880	4.506	0.529	0.528	0.424
15:00 – 16:00		3525	75.134	59.435	4.531	0.857	0.569	0.438
16:00 – 17:00		3755	80.087	60.455	5.441	0.895	0.605	0.488
17:00 – 18:00		4042	93.754	62.184	6.687	0.959	0.676	0.545

Table 33 (Continued)

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
18:00 – 19:00		3538	77.082	59.806	5.210	0.868	0.578	0.470
19:00 – 20:00	Wednesday	2782	62.753	56.940	3.370	0.650	0.449	0.343
20:00 – 21:00		2403	59.754	55.872	2.976	0.558	0.395	0.292
7:00 – 8:00		3294	72.184	59.214	4.124	0.840	0.571	0.402
8:00 – 9:00	(12.1.2011)	4046	103.029	63.279	7.965	1.012	0.717	0.580
9:00 – 10:00		3669	76.329	60.343	5.119	0.866	0.590	0.487
16:00 – 17:00		3753	81.108	60.670	5.105	0.881	0.590	0.469
17:00 – 18:00	Friday	4163	101.504	63.211	6.252	0.991	0.709	0.573
18:00 – 19:00		3504	74.859	59.595	4.683	0.844	0.562	0.449
6:00 – 7:00		2036	58.138	54.980	2.545	0.532	0.365	0.229
7:00 – 8:00	(16.1.2011)	3130	67.853	58.134	3.628	0.779	0.520	0.365
8:00 – 9:00		3654	79.578	60.569	5.269	0.888	0.622	0.486

Table 33 (Continued)

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
9:00 – 10:00		3254	67.264	58.594	4.066	0.752	0.529	0.416
10:00 – 11:00		2981	65.419	57.833	3.636	0.720	0.481	0.381
11:00 – 12:00		2583	60.831	56.269	3.019	0.582	0.393	0.305
12:00 – 13:00		2288	59.157	55.622	2.763	0.522	0.350	0.269
13:00 – 14:00		2135	58.173	55.187	2.630	0.498	0.331	0.248
14:00 – 15:00	(Sunday)	2956	64.646	57.189	3.361	0.708	0.472	0.347
15:00 – 16:00		3371	71.226	58.908	4.141	0.819	0.542	0.429
16:00 – 17:00		3635	76.280	59.928	4.860	0.876	0.586	0.450
17:00 – 18:00		3870	86.569	61.190	6.046	0.929	0.637	0.521
18:00 – 19:00		3312	70.934	58.815	4.450	0.798	0.526	0.428
19:00 – 20:00		2637	61.868	56.474	3.216	0.624	0.419	0.323
20:00 – 21:00		2296	58.786	55.441	2.835	0.524	0.368	0.271

Table 33 (Continued)

Time (hr)	Date	Traffic volume, v (veh/h)	Control Delay (sec/veh)			v/c ratio		
			Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal	Signalized Intersection (Existing)	Channelized Intersection with signal	Roundabout Intersection without signal
7:00 – 8:00	(17.1.2011) Monday	3446	77.243	59.787	4.161	0.878	0.595	0.418
8:00 – 9:00		4128	105.820	63.441	8.310	1.022	0.717	0.592
9:00 – 10:00		3829	84.774	61.210	5.748	0.921	0.630	0.520
16:00 – 17:00		3944	90.932	61.724	5.853	0.953	0.652	0.502
17:00 – 18:00		4272	104.094	63.485	7.721	1.002	0.712	0.573
18:00 – 19:00		3711	79.510	60.284	5.504	0.886	0.601	0.492

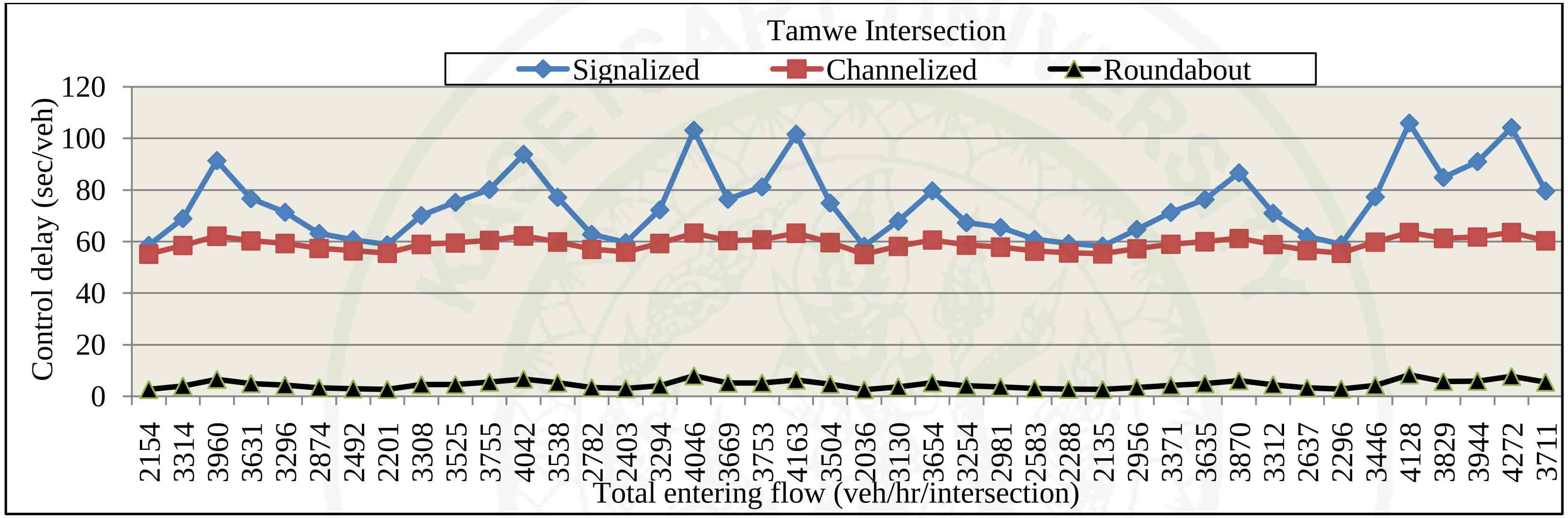


Figure 37 Control delays (Tamwe intersection) for existing signalized intersection, channelized intersection with signal control, and roundabout intersection

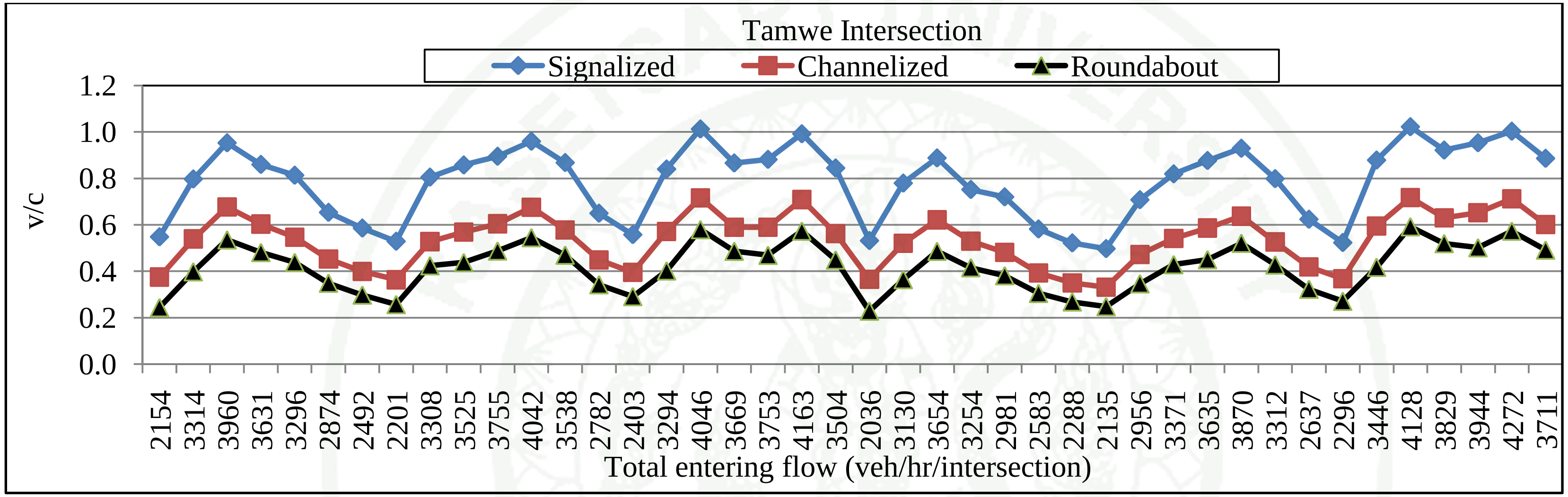


Figure 38 v/c ratios (Tamwe intersection) for existing signalized intersection, channelized intersection with signal control, and roundabout intersection

2. Effect of control delay and v/c ratio for three intersections

Three intersections for roundabouts appear to provide better level of service (Lower control delay) than existing signalized intersections and channelized intersections with signal control (Figure 33, 35 and 37). Roundabout intersections produce better level of service (LOS) while the existing signalized intersections and channelized intersections with signal control produce lowest level of service (LOS).

The performance of signalized, channelized, and roundabout cannot be judged only through control delay. v/c ratio is another independent measure that can be considered to evaluate the overall operation of a facility. Results of v/c ratio comparison are present in Figure 34, 36 and 38. Because of increased space availability, roundabouts are expected to provide lower v/c ratios. Roundabouts with two-lane (both four and five legged) approaches show lower v/c ratio (better performance).

3. Nonparametric Hypothesis Test

Nonparametric Hypothesis test results are presented in Table 34. The Kruskal-Wallis non parametric test is conducted for signalized, channelized, and roundabout, for a total of two factors. The null hypothesis tested is stated as “There is no significant difference between the means MOEs among comparison groups.

Table 34 Nonparametric test results for performance of signalized, channelized and roundabout

Intersection Type	Comparison Categories	Control Delay			v/c ratio		
		H-value[1]	Hypothesis[2]	Rank	H-value[1]	Hypothesis[2]	Rank
Myaynigone Four-Legged Intersection	Signalized (Existing)	87.801	Reject	3	80.334	Reject	3
	Channelized with signal			2			2
	Roundabout without signal			1			1
Hledan Five-Legged Intersection	Signalized (Existing)	98.356	Reject	3	85.755	Reject	3
	Channelized with signal			2			2
	Roundabout without signal			1			1
Tamwe Five-Legged Intersection	Signalized (Existing)	99.802	Reject	3	87.165	Reject	3
	Channelized with signal			2			2
	Roundabout without signal			1			1

Note: - [1]Significant-level: $\alpha = 0.05$

Rejection Region: Reject Null Hypothesis if H-value > 5.991, the value of $\chi^2_{0.05}$ for 2 degrees of freedom

[2]Hypothesis: H_0: Mean MOE for three comparison categories is equal

H_1: Mean MOE for three comparison categories is not equal

Reject Null Hypothesis: At $\alpha = 0.05$, there exits significant difference in the mean of three comparison categories

A confidence level of 95% is selected as the critical limit and the null hypothesis is rejected, for H-value greater than the value of $\chi^2_{0.05}$ for 2 degrees of freedom. If the null hypothesis is rejected, the comparison groups are ranked in order of performance. For case of failure to reject null hypothesis, no conclusive judgment was made. For control delay, roundabouts appeared to be the best alternative for corresponding Myaynigone, Hledan, and Tamwe intersections. There was significant difference in control delay performance for existing signalized intersection and improving channelized intersection with signal control. Roundabouts also produced better performance for v/c ratio.

4. Future year's improvement for three intersections

4.1 To convey more convenience in traffic communication for future year's improvement, overpass or underpass should be built on the busiest main traffic road at Myaynigone intersection as shown in Figure 39.

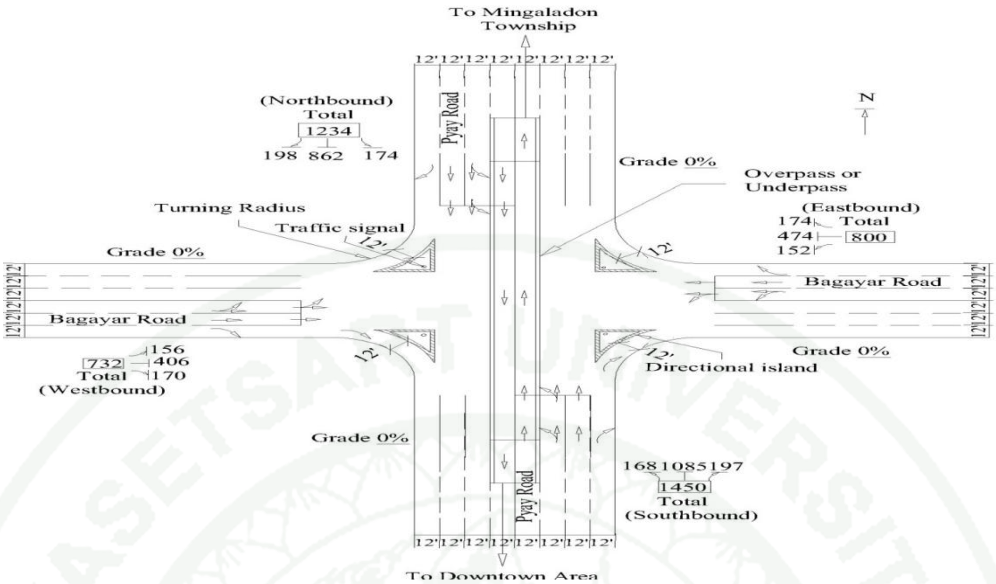


Figure 39 Myaynigone intersection with overpass or underpass

4.2 To convey more convenience in traffic communication for future year's improvement, overpass or underpass should be built on the busiest main traffic road at Hledan intersection as shown in Figure 40.

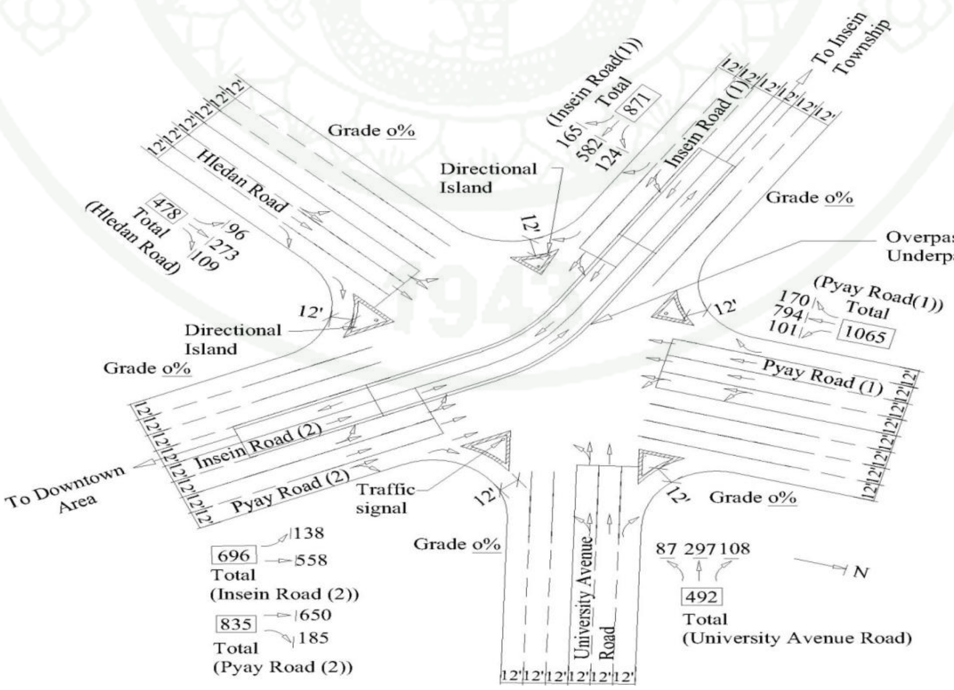


Figure 40 Hledan intersection with overpass or underpass

4.3 To convey more convenience in traffic communication for future year's improvement, overpass or underpass should be built on the busiest main traffic road at Tamwe intersection as shown in Figure 41.

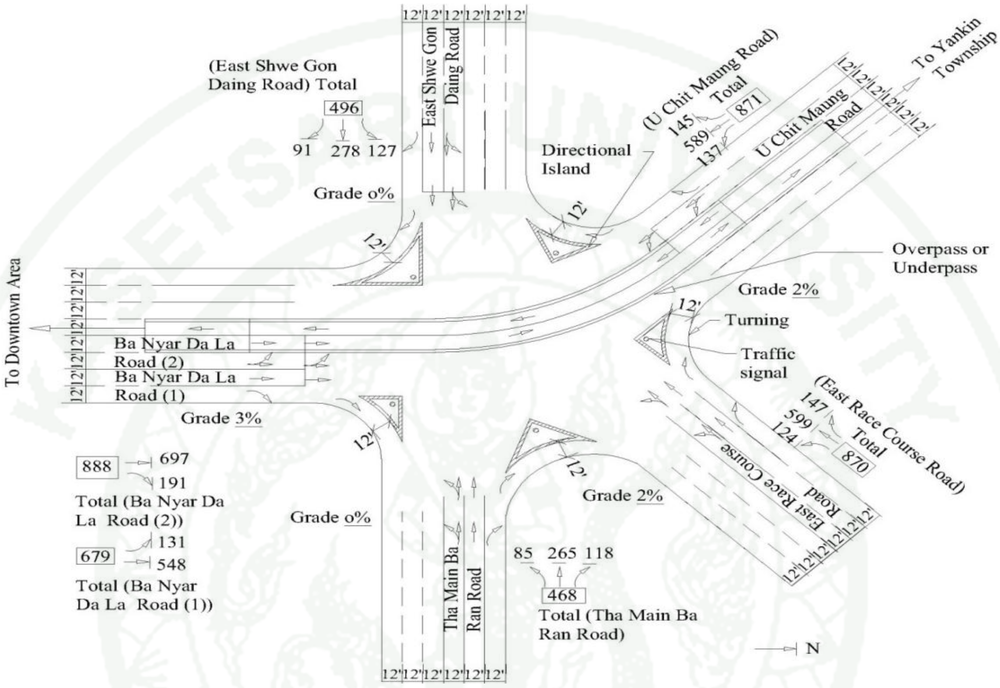


Figure 41 Tamwe intersection with overpass or underpass

CONCLUSION AND RECOMMENDATIONS

Conclusion

The performance of roundabout is compared to signalized intersection, and channelized intersection with signal control in two measures: control delay and volume to capacity ratio. Two types of signalized intersections are considered and corresponding highway geometry is evaluated for channelized intersection with signal control and roundabout of entry lane width of 12 ft (3.7 m) and 15 ft (4.6 m). In this study, baseline information on the selected intersections was gathered, the level of service at the intersections was determined and design requirements were identified for further improvement.

For the first MOE, control delays for three intersections of roundabouts are viable alternatives compared with signalized and channelized intersections at both high and low volume conditions. For the second MOE (v/c ratio), roundabouts show better performance under all conditions. This is because of increased space of roundabouts and the provision of relatively easier traffic movement in the central island. Flare lane width appears to be another factor contributing to lower v/c ratio. The nonparametric test revealed the significance and rank of improvement among categories.

Recommendations

In the long run, building of overpass or underpass on the busiest main road is the best way to improve and avoid traffic congestion particularly to reduce the travel time of the main traffic flow direction as a free flow with little additional widening of the road. The channelization and widening of the roadway at first stage of improvement should be designed for future stage of overpass or underpass facilities as much as possible. The implementation of each type of intersection will depend on the budget, the right of way, the upgrading of the control system devices, human factors, traffic regulations and law enforcement. In this thesis the traffic count was done only for 4 days due to limited time and workers, and also the limited right of way will result in short length between each leg of intersection, which effects the weaving problem of the roundabout, so these issues must be investigated and rechecked for the capacity analysis done in this thesis for future implementation. This thesis probably will be proposed to the concerned agencies in Myanmar for consideration as a guide line for future improvement.

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APPENDICES

Appendix A

Myaynigone Signalized Intersection

Appendix Table A1 Minimum pedestrian green time and adjustment factors for base saturation flow rate

Mininum pedestrian green time and adjustment factor for base saturation flow rate	Computation
1 Pedestrians / cycle	$100\frac{p}{h} x \frac{1h}{3600s} x\ 162 s = 4.500 p$ (Pyay Road) $80\frac{p}{h} x \frac{1h}{3600s} x\ 162\ s = 3.600\ p$ (Bagayar Road)
2. Minimum effective green time required for pedestrians (Use Equation-10).	$G_p = 3.2 + \frac{L}{Sp} + (0.27N_{ped})$ for $W_E \leq 10$ ft G_p (Pyay Road) = $3.2 + \frac{36}{4} + (0.27 \times 4.500) = 13.4$ s G_p (Bagayar Road) = $3.2 + \frac{48}{4} + (0.27 \times 3.600) = 16.2$ s
3. Compare minimum effective green time required for pedestrians with actual effective green.	G_p (Pyay Road) = 75 s, which is > 13.4 s G_p (Bagayar Road) = 60 s, which is > 16.2 s
4. Proportion of left turns and right turns.	P_{LT} (EB) = $\frac{152}{152+474} = 0.243$, P_{LT} (WB) = $\frac{156}{156+406} = 0.278$ P_{LT} (NB) = $\frac{174}{174+862} = 0.168$, P_{LT} (SB) = $\frac{168}{168+1085} = 0.134$ P_{RT} for exclusive RT Lane is 1.000.

Appendix Table A1 (Continued)

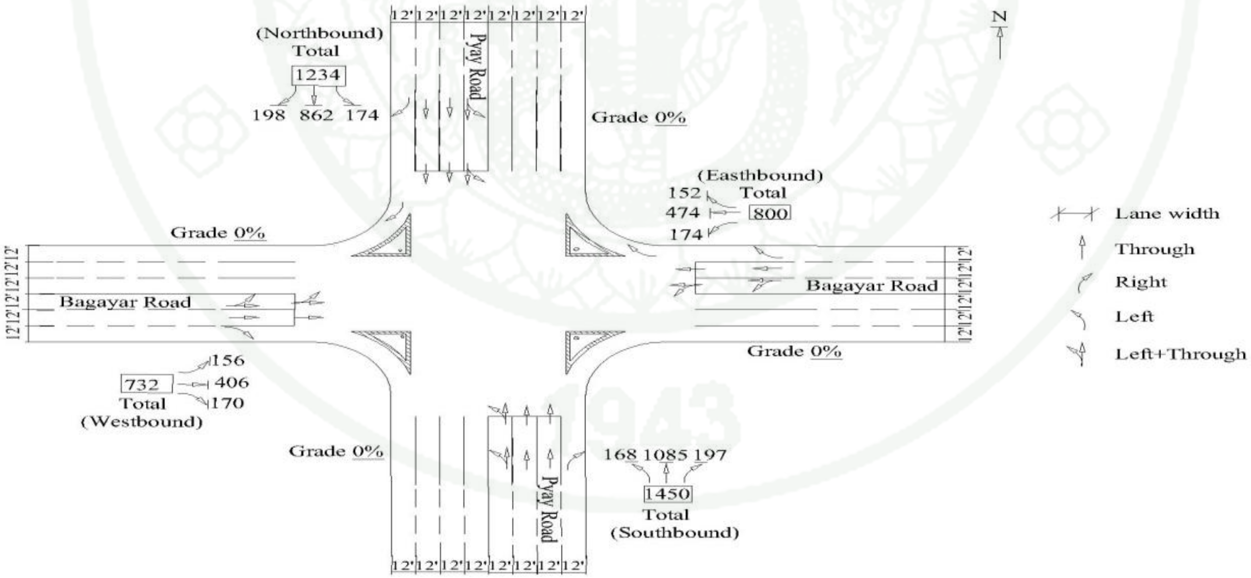
Mininum pedestrian green time and adjustment factor for base saturation flow rate	Computation
5. Lane width adjustment factor (Use Equation-14)	$f_w = 1 + \frac{(W-12)}{30}$, f_w (EB/WB/NB/SB) = $1 + \frac{(12-12)}{30} = 1.000$
6. Heavy-vehicle adjustment factor (Use Equation-15)	$f_{HV} = \frac{100}{100+\%HV(E_T-1)}$ f_{HV} (EB/WB/NB/SB) = $\frac{100}{100+0(2-1)} = 1.000$
7. Percent grade adjustment factor (Use Equation-16).	$f_g = 1 - \frac{\%G}{200}$, f_g (EB/WB/NB/SB) = $1 - \frac{0\%}{200} = 1.000$
8. Parking adjustment factor Use (Equation-17).	$f_p = \frac{N-0.1-\frac{18Nm}{3600}}{N}$, No Parking maneuvers, $f_p = 1.000$
9. Bus blockage adjustment factor Use (Equation-18).	$f_{bb} = \frac{N-\frac{14.4N_B}{3600}}{N}$, No Bus Stopping, $f_{bb} = 1.000$
10. Area type adjustment factor Use (Equation-20).	For all other areas, $f_a = 1.000$
11. Lane utilization adjustment factor (From Table-10).	No specific data are given. Use default of $f_{LU} = 1.000$ for exclusive LT. Use 0.908 for all shared LT.

Appendix Table A1 (Continued)

Mininum pedestrian green time and adjustment factor for base saturation flow rate	Computation
12. Left-turn adjustment factor	The Left-turn is permitted. A special Procedure is used. All approaches are opposed by multilane approaches. The supplemental worksheet for multilane approaches is used to determine the factor. The left turn is Protected, Use Equation (25), $f_{LT} = \frac{1}{1+0.05P_{LT}}$, f_{LT} (Northbound) = $\frac{1}{1+0.05x0.168}$ = 0.992 f_{LT} (Southbound) = $\frac{1}{1+0.05x0.134}$ = 0.993
13. Right-turn adjustment factor (Use Equation-21).	For all exclusive – lane approaches, f_{RT} (EB/WB/NB/SB) = 0.85
14. Left-turn pedestrian/bicycle adjustment factor	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.
15. Right-turn pedestrian/bicycle adjustment factor	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.

Appendix Table A2 Input Worksheet

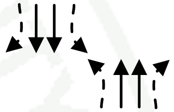
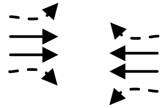
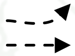
General Information		Site Information	
Analyst		Intersection	Myaynigone Intersection
Agency or Company		Area Type	□ CBD ■ Other
Date performed	14.1.2011	Jurisdiction	
Analysis Time Period	8:00 – 9:00 A.M	Analysis Year	2011
Intersection Geometry			



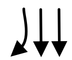
Appendix Table A2 (Continued)

Volume and Timing Input	Site Information											
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	152	474	174	156	406	170	174	862	198	168	1085	197
% heavy vehicles, %HV		0			0			0			0	
Peak-hour factor, PHF		0.93			0.91			0.93			0.98	
Pretimed (P) or actuated (A)		P			P			P			P	
Start-up lost time, I_1 (s)		-			-			-			-	
Extension of effective green time, e (s)		-			-			-			-	
Arrival type, AT		3			3			3			3	
Approach pedestrian volume, v_{ped} (p/h)		80			80			100			100	
Approach bicycle volume, v_{bic} (bicycles/h)		0			0			0			0	
Parking (Y or N)		N			N			N			N	
Parking maneuvers, N_m (maneuvers/h)		0			0			0			0	
Bus stopping, N_B (buses/h)		0			0			0			0	
Min. timing for pedestrians, G_p (s)		16.2			16.2			13.4			13.4	

Appendix Table A2 (Continued)

Signal Phasing Plan	Site Information				
Diagram	01 	02 	03 	04 	05 
Timing	G = 15 Y = 4	G = 0 Y = 0	G = 0 Y = 0	G = 75 Y = 4	G = 60 Y = 4
 Protected turns		Protected turns, Pedestrian		Cycle Length, C = 162 s	

Appendix Table A3 Volume Adjustment and Saturation Flow Rate Worksheet

Volume Adjustment	Computation											
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	152	474	174	156	406	170	174	862	198	168	1085	197
Peak-hour factor, PHF		0.93			0.91			0.93			0.98	
Adjusted flow rate, v_P = V/PHF (veh/h) (Use Equation-12)	163	510	187	171	446	187	187	927	213	171	1107	201
Lane group												
Adjusted flow rate in lane group (veh/h)	163	697		171	633		187		1140	171		1308
Proportion of LT or RT (P_{LT} or P_{RT})	0.243		1.00	0.278		1.00	0.168		1.00	0.134		1.00
Saturation Flow Rate	Computation											
Base saturation flow, s_o (pc/h/lane)	1900	1900		1900	1900		1900	1900	1900	1900	1900	1900
Number of Lanes, N	1	2		1	2		1	1	3	1	1	3
Lane width adjustment factor, f_w	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Heavy-vehicle adjustment factor, f_{HV}	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Grade adjustment factor, f_g	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00

Appendix Table A3 (Continued)

Saturation Flow Rate	Computation											
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Parking adjustment factor, f_p	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Bus Blockage adjustment factor, f_{bb}	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Area type adjustment factor, f_a	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Lane utilization adjustment factor, f_{LU}	1.00	0.952		1.00	0.952		1.00	1.00	0.908	1.00	1.00	0.908
Left-turn adjustment factor, f_{LT}	0.626	1.00		0.593	1.00		0.992	0.640	1.00	0.993	0.663	1.00
Right-turn adjustment factor, f_{RT}	1.00	0.850		1.00	0.850		1.00	1.00	0.850	1.00	1.00	0.850
Left-turn ped/bike adjustment factor, f_{Lpb}	0.988	1.00		0.997	1.00		1.000	0.999	1.00	1.00	0.999	1.00
Right-turn ped/bike adjustment factor, f_{Rpb}	1.00	0.935		1.00	0.935		1.00	1.00	0.935	1.00	1.00	0.935
Adjustment saturation flow, s (veh/h) $s = s_o N f_w f_{HV} f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$	1185	2875		1123	2875		1885	1215	4113	1887	1258	4113

Appendix Table A4 Supplemental Worksheet for Permitted Left Turns Opposed by Multilane Approach

Permitted Left Turn Adjustment Factor	Computation			
	EB	WB	NB	SB
Cycle Length, C(s)	162			
Total actual green time for LT Lane group, G(s)	60	60	94	94
Effective Permitted green time for LT lane group, G(s)	60	60	79	79
Opposing effective green time, g_o (s)	60	60	75	75
Number of Lanes in LT Lane group, N	2	2	3	3
Number of Lanes in opposing approach, N_o	3	3	4	4
Adjusted LT flow rate, v_{LT} (veh/h)	163	171	187	171
Proportion of LT volume in LT lane group, P_{LT}	0.243	0.278	0.168	0.134
Adjusted flow rate, opposing approach, v_o (veh/h)	617	673	1287	1114
Loft time for LT lane group, t_L	0	0	0	0
LT volume per cycle LTC = v_{LT} C/3600	7.335	7.695	8.415	7.695
Opposing lane utilization factor , f_{LUo}	0.952	0.952	0.908	0.908
Opposing flow per lane, per cycle $v_{olc} = \frac{v_o C}{3600 N_o f_{LUo}}$ (veh/C/Ln)	9.722	10.604	15.946	13.802
$g_f = G[e^{-0.882\,(LTC^{0.717})}] - t_L$, $g_f < g$(except for exclusive Left-turn lanes) (Use Equation-27)	1.512	1.329	1.618	2.083

Appendix Table A4 (Continued)

Permitted Left Turn Adjustment Factor	Computation			
	EB	WB	NB	SB
Opposing Platoon ratio, R_{po} (Use Table-11)	1.000	1.000	1.000	1.000
Opposing queue ratio, $qr_o = \max [1 - R_{po}(g_o/C),0]$	0.630	0.630	0.537	0.537
$g_q = \dfrac{v_{olc}qr_o}{0.5-[v_{olc}(1-qr_o)/g_o]}$ = t_L, v_{olc} $(1-qr_o)/g_o \leq 0.49$ (note case - specific parameters) (Use Equation-28)	13.919	15.371	21.324	17.868
$g_u = g - g_q$ if $g_q \geq g_f$ (Use Equation-29) or $g_u = g - g_f$ if $g_q < g_f$ (Use Equation-30)	46.081	44.629	57.676	61.132
E_{L1} (Refer to Table-13) and (Use Equation-31,32,33)	2.644	2.821	5.541	4.630
$P_L = P_{LT}\left[1+\dfrac{(N-1)g}{(g_f+\dfrac{g_u}{E_{L1}}+4.24)}\right]$ (except with multilane subject approach) (Use Equation-34)	0.872	1.058	1.800	1.218
$f_{min} = 2\,(1 + P_L)/g$ (Use Equation-35)	0.062	0.069	0.071	0.056
$f_m = [g_f/g] + [g_u/g]\left[1+\dfrac{1}{1+P_L(E_{L1}-1)}\right]$ $(f_{min} \leq f_m \leq 1.000)$ (Use Equation-36)	0.341	0.276	0.100	0.169
$f_{LT} = [f_m + 0.91\,(N-1)]/N$ (except for permitted left turn) (Use Equation-37)	0.626	0.593	0.640	0.663

Appendix Table A5 Supplemental Worksheet for Pedestrian – Bicycle Effects on Permitted Left Turns and Right Turns

Pedestrian – Bicycle Adjustment Factor				
Permitted Left Turns	Computation			
	EB	WB	NB	SB
Effective pedestrian green time, g_p (s)	60	60	75	75
Conflicting Pedestrian Volume, v_{ped} (p/h)	80	80	100	100
$v_{pedg} = v_{ped}$ (C/g_p) $(v_{pedg} \leq 5000)$ (Use Equation-38)	216	216	216	216
$OCC_{pedg} = v_{pedg}/2000$ if $(v_{pedg} \leq 1000)$ (Use Equation-40) or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if $(1000 < v_{pedg} \leq 5000)$ (Use Equation-41)	0.108	0.108	0.108	0.108
Opposing queue clearing green, g_q (s)	13.919	15.371	21.324	17.868
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p if $g_q \geq g_p$ then $f_{Lpb} = 1.0$	0.232	0.256	0.284	0.238
$OCC_{pedu} = OCC_{pedg}$ $[1-0.5$ $(g_q/g_p)]$ (Use Equation-42)	0.095	0.094	0.093	0.095
Opposing flow rate, v_o(veh/h)	617	673	1287	1114
$OCC_r = OCC_{pedu}$ $[e^{-(5/3600)Vo}]$ (Use Equation-44)	0.040	0.037	0.016	0.020
Number of cross-street receiving lanes, N_{rec}	4	4	3	3

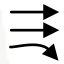
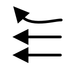
Appendix Table A5 (Continued)

Pedestrian – Bicycle Adjustment Factor				
Permitted Left Turns	Computation			
	EB	WB	NB	SB
Number of turning lanes, N_{turn}	1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$ (Use Equation-46) or $A_{pbT} = 1 - 0.6\ (OCC_r)$ if $N_{rec} > N_{turn}$ (Use Equation-47)	0.976	0.978	0.990	0.988
Proportion of left turn, P_{LT}	0.243	0.278	0.168	0.134
Proportion of left turns using protected phase, P_{LTA} = (1- permitted phase f_{LT})/0.95 (Use Equation-48)	0.394	0.428	0.379	0.355
$f_{Lpb} = 1.0 - P_{LT}\ (1- A_{pbT})\ (1 - P_{LTA})$ (Use Equation-49)	0.996	0.997	0.999	0.999
Permitted Right Turns	Computation			
Effective pedestrian green time, g_p (s)	60	60	75	75
Conflicting pedestrian volume, v_{ped}(p/h)	80	80	100	100

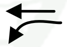
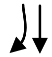
Appendix Table A5 (Continued)

Pedestrian – Bicycle Adjustment Factor				
Permitted Right Turns	Computation			
	EB	WB	NB	SB
				
Conflicting bicycle volume, V_{bic} (bicycles/h)	0	0	0	0
$v_{pedg} = v_{pedg}\ (C/g_p)$	216	216	216	216
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \leq 1000$) or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if ($1000 < v_{pedg} \leq 5000$)	0.108	0.108	0.108	0.108
Effective green, g(s)	60	60	75	75
$v_{bicg} = v_{bic}\ (C/g)$ ($v_{bicg} \leq 1900$) (Use Equation-39))	0	0	0	0
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})\ (OCC_{bicg})$ (Use Equation-45)	0.108	0.108	0.108	0.108
Number of cross-street receiving lanes, N_{rec}	4	4	3	3
Number of turning lanes, N_{turn}	1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, or $A_{pbT} = 1 - 0.6\ (OCC_r)$ if $N_{rec} > N_{turn}$	0.935	0.935	0.935	0.935
Proportion of right turn, P_{RT}	1.000	1.000	1.000	1.000
Proportion of right turns using protected phase, P_{RTA}	0	0	0	0
$f_{Rpb} = 1.0 - P_{RT}\ (1 - A_{pbT})\ (1 - P_{RTA})$ (Use Equation-50)	0.935	0.935	0.935	0.935

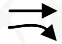
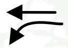
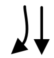
Appendix Table A6 Capacity and LOS Worksheet

Capacity Analysis	Computation									
Phase Number	1	1	2	2	2	2	3	3	3	3
Phase Type	P	P	P	P	P	P	P	P	P	P
Lane group										
Adjusted flow rate, v(veh/h)	187	171	0	1140	0	1308	163	697	171	633
Saturation flow rate, s(veh/h)	1885	1887	1215	4113	1258	4113	1185	2875	1123	2875
Lost time, t_L (s), $t_L = I_1 - Y - e$ (Use Equation-8)	4	4	4	4	4	4	4	4	4	4
Effective green time, g (s), $g = G + Y - t_L$ (Use Equation-26)	15	15	79	75	79	75	60	60	60	60
Green ratio, g/C	0.09	0.09	0.49	0.46	0.49	0.46	0.37	0.37	0.37	0.37
Lane group capacity, c = s (g/C) (veh/h) (Use Equation-51)	170	170	595	1892	616	1892	438	1064	416	1064
v/c ratio X (Use Equation-52)	1.100	1.006	0	0.603	0	0.691	0.372	0.655	0.411	0.595
Flow ratio, v/s	0.099					0.318			0.242	
Critical lane group/phase (✓)	✓					✓			✓	

Appendix Table A6 (Continued)

Capacity Analysis	Computation							
Sum of flow ratio for critical lane group, $Y_c = \sum$ (Critical lane groups v/s)	0.659							
Total Lost time per cycle, L(s) (Use Equation-9)	12.0							
Critical flow rate to capacity ratio, $X_c = (Y_c)(C)/(C\text{-}L)$ (Use Equation-53)	0.712							
Lane Group Capacity, Control Delay, and LOS	Computation							
	EB		WB		NB		SB	
Lane group								
Adjusted flow rate, v (veh/h)	163	697	171	633	187	1140	171	1308
Lane group capacity, c (veh/h)	438	1064	416	1064	765	1892	786	1892
v/c ratio, X = v/c	0.372	0.655	0.411	0.595	0.244	0.603	0.218	0.691
Total green ratio, g/C	0.37	0.37	0.37	0.37	0.58	0.46	0.58	0.46
Uniform delay, d_1 (Use Equation-56) $d_1 = \frac{0.5C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)	37.280	42.432	37.915	41.224	16.644	32.686	16.357	34.626

Appendix Table A6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation							
	EB		WB		NB		SB	
Lane group								
Incremental delay calibration, k	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
Upstream filtering / metering adjustment factor, I	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Duration of analysis period (h),T	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250
Incremental delay, d_2 (Use Equation-57) $d_2 = 900T (X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}$ (s/veh)	2.414	3.148	2.986	2.452	0.758	1.434	0.637	2.096
Initial queue delay, d_3 (s/veh)	0	0	0	0	0	0	0	0
Progression adjustment factor, PF (From Table-13)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Delay, $d = d_1 (PF) + d_2 + d_3$ (s/veh) (Use Equation-54)	39.7	45.6	40.9	43.7	17.4	34.1	17.0	36.7
LOS by lane group	D	D	D	D	B	C	B	D

Appendix Table A6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation			
	EB	WB	NB	SB
Delay by approach, d_A (Use Equation-58) $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	44.5	43.1	31.7	34.4
LOS by approach	D	D	C	C
Approach flow rate, v_A (veh/h)	860	804	1327	1479
Intersection delay, d_I (Use Equation-59) $d_I = \frac{\sum(d_A)(v_A)}{\sum(v_A)}$ (s/veh)	37.1	Intersection LOS Table-14		D

Appendix B

Myaynigone Channelized Intersection with Signal Control

Appendix Table B1 Minimum pedestrian green time and adjustment factors for base saturation flow rate

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
1. Pedestrians / cycle	$100\frac{p}{h} x \frac{1h}{3600s} x\ 162s = 4.500p$ (Pyay Road) $80\frac{p}{h} x \frac{1h}{3600s} x\ 162s = 3.600p$ (Bagayar Road)
2. Minimum effective green time required for pedestrians (Use Equation-10).	$G_p = 3.2 + \frac{L}{Sp} + (0.27N_{ped})$ for $W_E \leq 10$ ft G_p (Pyay Road) $= 3.2 + \frac{48}{4} + (0.27 x4.500) = 16.4$ s G_p (Bagayar Road) $= 3.2 + \frac{60}{4} + (0.27 x3.600) = 19.2$ s
3. Compare minimum effective green time required for pedestrians with actual effective green.	G_p (Pyay Road) = 75 s, which is > 16.4 s G_p (Bagayar Road) = 60 s, which is > 19.2 s
4. Proportion of left turns and right turns.	P_{LT} for exclusive LT lane is 1.000. P_{RT} for exclusive RT lane is 1.000.
5. Lane width adjustment factor (Use Equation-14).	$f_w = 1 + \frac{(W-12)}{30}$, f_w (EB/WB/NB/SB) $= 1 + \frac{(12-12)}{30} = 1.000$

Appendix Table B1 (Continued)

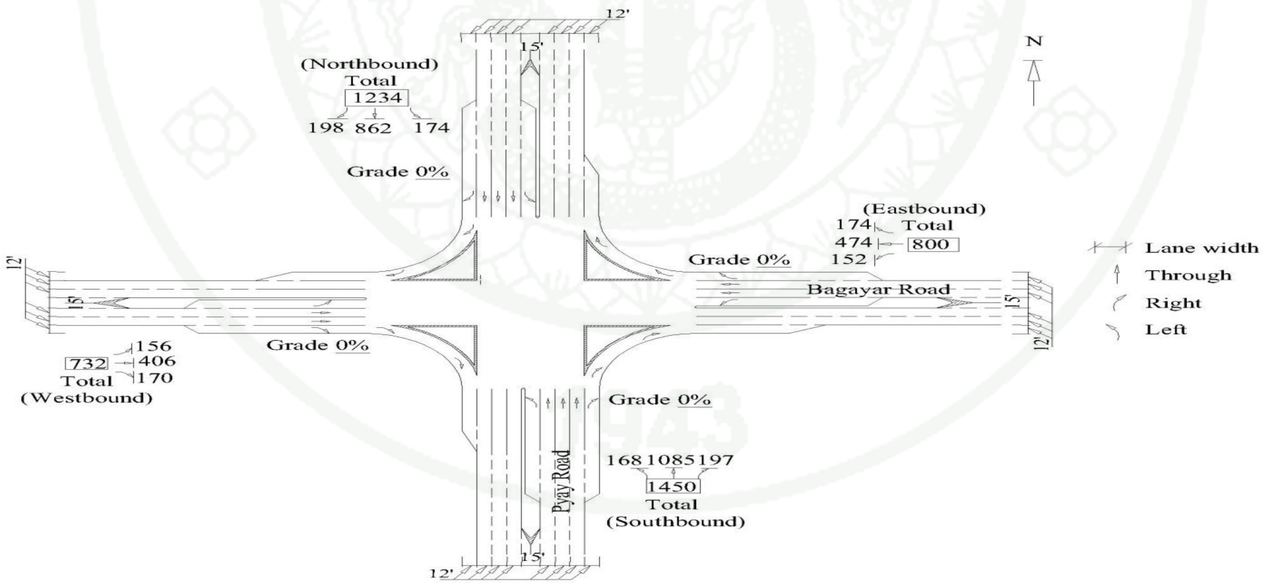
Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
6. Heavy-vehicle adjustment factor (Use Equation-15).	$f_{HV} = \frac{100}{100 + \%HV(E_T - 1)}$ f_{HV} (EB/WB/NB/SB) = $\frac{100}{100 + 0(2-1)}$ =1.000
7. Percent grade adjustment factor (Use Equation-16).	$f_g = 1 - \frac{\%G}{200}$, f_g (EB/WB/NB/SB) = $1 - \frac{0\%}{200}$ = 1.000
8. Parking adjustment factor (Use Equation-17).	$f_p = \frac{N - 0.1 - \frac{18Nm}{3600}}{N}$, No Parking maneuvers, f_p = 1.000
9. Bus blockage adjustment factor (Use Equation-18).	$f_{bb} = \frac{N - \frac{14.4N_B}{3600}}{N}$, No Bus Stopping, f_{bb} = 1.000
10. Area type adjustment factor (Use Equation-20).	For all other areas, fa = 1.000
11. Lane utilization adjustment factor (From Table-10).	No Specific data are given. Use default of f_{LU} = 1.000 for exclusive LT. Use 0.908 for all shared LT

Appendix Table B1 (Continued)

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
12. Left-turn adjustment factor.	The Left-turn is permitted. A special Procedure is used. All approaches are opposed by multilane approaches. The supplemental worksheet for multilane approaches is used to determine the factor. The left turn is protected Use Equation (24). For exclusive left-turn Lane, f_{LT} = 0.950
13. Right-turn adjustment factor (Use Equation-21).	For all exclusive – lane approaches, f_{RT} (EB/WB/NB/SB) = 0.85
14. Left-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.
15. Right-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.

Appendix Table B2 Input Worksheet

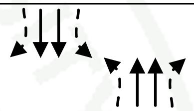
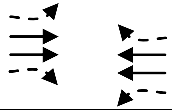
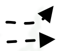
General Information		Site Information	
Analyst		Intersection	Hledan Intersection
Agency or Company		Area Type	□ CBD ■ Other
Date performed	14.1.2011	Jurisdiction	
Analysis Time Period	8:00 – 9:00 A.M	Analysis Year	2011
Intersection Geometry			



Appendix Table B2 (continued)

Volume and Timing Input	Site Information											
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V(veh/h)	152	474	174	156	406	170	174	862	198	168	1085	197
% heavy vehicles, %HV												
Peak-hour factor, PHF		0.93			0.91			0.93			0.98	
Pretimed (P) or actuated (A)		P			P			P			P	
Start-up lost time, I_1 (s)		-			-			-			-	
Extension of effective green time, e(s)		-			-			-			-	
Arrival type, AT		3			3			3			3	
Approach pedestrian volume, v_{ped} (p/h)		80			80			100			100	
Approach bicycle volume, v_{bic} (bicycles/h)		0			0			0			0	
Approach bicycle volume, v_{bic} (bicycles/h)		0			0			0			0	
Parking (Y or N)		N			N			N			N	
Parking maneuvers, N_m (maneuvers/h)		0			0			0			0	
Bus stopping, N_b (buses/h)		0			0			0			0	
Min. timing for pedestrians, G_p(s)		19.2			19.2			16.4			16.4	

Appendix Table B2 (Continued)

Signal Phasing Plan	Site Information				
Diagram	01 	02 	03 	04 	05 
Timing	G = 15 Y = 4	G = 0 Y = 0	G = 0 Y = 0	G = 75 Y = 4	G = 60 Y = 4
 Protected turns	 Protected turns Pedestrian			Cycle Length, C = 162 s	

Appendix Table B3 Volume Adjustment and Saturation Flow Rate Worksheet

Volume Adjustment	Computation											
	EB			WB			NB			SB		
	LT	TH	RT	TH	TH	RT	LT	TH	RT	LT	TH	RT
Volume, v(veh/h)	152	474	174	156	406	170	174	862	198	168	1085	197
Peak-hour factor, PHF		0.93			0.91			0.93			0.98	
Adjusted flow rate, v_P = V/PHF (veh/h) (Use Equation-12)	163	510	187	171	446	187	187	927	213	171	1107	201
Lane group												
Adjusted flow rate in lane group (veh/h)	163	697		171	633		187		1140	171		1308
Proportion of LT or RT (P_{LT} or P_{RT})	1.00	-	1.00	1.00		1.00	1.00		1.00	1.00		1.00
Saturation Flow Rate	Computation											
Base saturation flow, s_o (pc/h/lane)	1900	1900		1900	1900		1900	1900	1900	1900	1900	1900
Number of Lanes, N	1	3		1	3		1	1	4	1	1	4
Lane width adjustment factor, f_w	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Heavy-vehicle adjustment factor, f_{HV}	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Grade adjustment factor, f_g	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00

Appendix Table B3 (Continued)

Saturation Flow Rate	Computation											
	EB			WB			NB			SB		
	LT	TH	RT	TH	TH	RT	LT	TH	RT	LT	TH	RT
Parking adjustment factor, f_p	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Bus Blockage adjustment factor, f_{bb}	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Area type adjustment factor, f_a	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
Lane utilization adjustment factor, f_{LU}	1.00	0.908		1.00	0.908		1.00	1.00	0.908	1.00	1.00	0.908
Left-turn adjustment factor, f_{LT}	0.399	1.00		0.361	1.00		0.950	0.177	1.00	0.950	0.227	1.00
Right-turn adjustment factor, f_{RT}	1.00	0.850		1.00	0.850		1.00	1.00	0.850	1.00	1.00	0.850
Left-turn ped/bike adjustment factor, f_{Lpb}	0.988	1.00		0.991	1.00		1.000	0.998	1.00	1.00	0.997	1.00
Right-turn ped/bike adjustment factor, f_{Rpb}	1.00	0.935		1.00	0.935		1.00	1.00	0.935	1.00	1.00	0.935
Adjustment saturation flow, s (veh/h) $s = s_o N f_w f_{HV} f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$	749	4113		680	4113		1805	336	5484	1805	430	5484

Appendix Table B4 Supplemental Worksheet for Permitted Left Turns Opposed by Multilane Approach

Permitted Left Turn Adjustment Factor	Computation			
	EB	WB	NB	SB
Cycle Length, C(s)	162			
Total actual green time for LT Lane group, G(s)	60	60	94	94
Effective Permitted green time for LT lane group, G(s)	60	60	79	79
Opposing effective green time, g_o (s)	60	60	75	75
Number of Lanes in LT Lane group, N	1	1	1	1
Number of Lanes in opposing approach, N_o	3	3	4	4
Adjusted LT flow rate, v_{LT} (veh/h)	163	171	187	171
Adjusted flow rate, opposing approach, v_o (veh/h)	446	510	1107	927
Lost time for LT lane group, t_L	0	0	0	0
Computation				
LT volume per cycle LTC = v_{LT} C/3600	7.335	7.695	8.415	7.695
Opposing lane utilization factor, f_{LUo}	0.908	0.908	0.908	0.908
Opposing flow per lane, per cycle, $v_{olc} = \frac{v_o C}{3600 N_o f_{LUo}}$ (veh/C/Ln)	7.368	8.425	13.716	11.485
$g_f = G[e^{-0.882(LTC^{0.717})}] - t_L$, $g_f < g$ (except for exclusive Left-turn lanes) (Use Equation-27)	0	0	0	0

Appendix Table B4 (Continued)

Permitted Left Turn Adjustment Factor	Computation			
	EB	WB	NB	SB
Opposing Platoon ratio, R_{po}	1.000	1.000	1.000	1.000
Opposing queue ratio , qr_o = max [1-R_{po}(g_o/C),0]	0.630	0.630	0.537	0.537
$g_q = \frac{v_{olc}qr_o}{0.5-[v_{olc}(1-qr_o)/g_o]}$ = t_L, v_{olc} (1-qr_o)/$g_o \le$ 0.49 (note case-specific parameters) (Use Equation-28)	10.212	11.846	17.734	14.373
g_u = g - g_q if $g_q \ge g_f$, (Use Equation-29) or g_u = g - g_f if $g_q < g_f$ (Use Equation-30)	49.788	48.154	57.266	60.627
E_{L1} (Refer to Table-13) and (Use Equation-31,32,33)	2.082	2.224	4.085	3.374
$P_L = P_{LT}\left[1+\frac{(N-1)g}{(g_f+\frac{g_u}{E_{L1}}+4.24)}\right]$ (except with multilane subject approach) (Use Equation-34)	1.00	1.00	1.00	1.00
f_{min} = 2 (1 + P_L)/g (Use Equation-36)	0.667	0.667	0.051	0.051
$f_m = [g_f/g] + [g_u/g]\left[1+\frac{1}{1+P_L(E_{L1}-1)}\right]$ ($f_{min} \le f_m \le$ 1.000) (Use Equation-35)	0.399	0.361	0.177	0.227
f_{LT} = [f_m + 0.91 (N-1)]/N (except for permitted left turn) (Use Equation-37)	0.399	0.361	0.177	0.227

Appendix Table B5 Supplemental Worksheet for Pedestrian – Bicycle Effects on Permitted Left Turns and Right Turns

Pedestrian – Bicycle Adjustment Factor				
Permitted Left Turns	Computation			
	EB	WB	NB	SB
Effective pedestrian green time, g_p (s)	60	60	75	75
Conflicting Pedestrian Volume, v_{ped} (p/h)	80	80	100	100
$v_{pedg} = v_{ped}$ (C/g_p) ($v_{pedg} \leq 5000$) (Use Equation-38)	216	216	216	216
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \leq 1000$) (Use Equation-40) or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if ($1000 < v_{pedg} \leq 5000$) (Use Equation-41)	0.108	0.108	0.108	0.108
Opposing queue clearing green, g_q (s)	10.212	11.846	17.734	14.373
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p if $g_q \geq g_p$ then $f_{lpb} = 1.0$	0.170	0.197	0.236	0.192
$OCC_{pedu} = OCC_{pedg}$ [$1-0.5$ (g_q/g_p)] (Use Equation-42)	0.099	0.097	0.095	0.098
Opposing flow rate, v_o(veh/h)	446	510	1107	927
$OCC_r = OCC_{pedu}$ [$e^{-(5/3600)Vo}$] (Use Equation-44)	0.053	0.048	0.020	0.027
Number of cross-street receiving lanes, N_{rec}	5	5	4	4

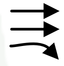
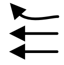
Appendix Table B5 (Continued)

Pedestrian – Bicycle Adjustment Factor				
Permitted Left Turns	Computation			
	EB	WB	NB	SB
Number of turning lanes, N_{turn}	1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, (Use Equation-46) or $A_{pbt} = 1 - 0.6\ (OCC_r)$ if $N_{rec} > N_{turn}$ (Use Equation-47)	0.968	0.971	0.988	0.984
Proportion of left turn, P_{LT}	1.00	1.00	1.00	1.00
Proportion of left turns using protected phase, $P_{LTA} = (1 –$ permitted phase $f_{LT})/0.95$ (Use Equation-48)	0.633	0.673	0.866	0.814
$f_{Lpb} = 1.0 - P_{LT}\ (1 - A_{pbT})\ (1 - P_{LTA})$ (Use Equation-49)	0.988	0.991	0.998	0.997
Permitted Right Turns	Computation			
Effective pedestrian green time, g_p (s)	60	60	75	75
Conflicting pedestrian volume, v_{ped} (p/h)	80	80	100	100

Appendix Table B5 (Continued)

Permitted Right Turns	Computation			
	EB	WB	NB	SB
				
Conflicting bicycle volume, v_{bic} (bicycles/h)	0	0	0	0
$v_{pedg} = v_{pedg}$ (C/g_p)	216	216	216	216
$OCC_{pedg} = v_{pedg}/2000$ if $(v_{pedg} \leq 1000)$ or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if $(1000 < v_{pedg} \leq 5000)$	0.108	0.108	0.108	0.108
Effective green, g(s)	60	60	75	75
$v_{bicg} = v_{bic}$ (C/g) $(v_{bicg} \leq 1900)$ (Use Equation-39)	0	0	0	0
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg}) (OCC_{bicg})$ (Use Equation-45)	0.108	0.108	0.108	0.108
Number of cross-street receiving lanes, N_{rec}	5	5	4	4
Number of turning lanes, N_{turn}	1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, or $A_{pbT} = 1 - 0.6 (OCC_r)$ if $N_{rec} > N_{turn}$	0.935	0.935	0.935	0.935
Proportion of right turns, P_{RT}	1.00	1.00	1.00	1.00
Proportion of right turns using protected phase, P_{RTA}	0	0	0	0
$f_{Rpb} = 1.0 - P_{RT} (1 - A_{pbT}) (1 - P_{RTA})$ (Use Equation-50)	0.935	0.935	0.935	0.935

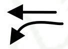
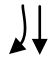
Appendix Table B6 Capacity and LOS Worksheet

Capacity Analysis	Computation									
Phase Number	1	1	2	2	2	2	3	3	3	3
Phase Type		P	P	P	P	P	P	P	P	P
Lane group										
Adjusted flow rate, v (veh/h)	187	171	0	1140	0	1308	163	697	171	633
Saturation flow rate, s (veh/h)	1805	1805	336	5484	430	5484	749	4113	680	4113
Lost time, t_L (s), $t_L = I_1 - Y - e$ (Use Equation-8)	4	4	4	4	4	4	4	4	4	4
Effective green time, g (s), $g = G + Y - t_L$ (Use Equation-26)	15	15	79	75	79	75	60	60	60	60
Green ratio, g/C	0.09	0.09	0.49	0.46	0.49	0.46	0.37	0.37	0.37	0.37
Lane group capacity, c = s (g/C) (veh/h) (Use Equation-51)	162	162	165	2523	211	2523	277	1522	252	1522
v/c ratio, X (Use Equation-52)	1.154	1.056	0	0.452	0	0.518	0.588	0.458	0.679	0.416
Flow ratio, v/s	0.104					0.239			0.251	
Critical lane group/phase (✓)	✓					✓			✓	

Appendix Table B6 (Continued)

Capacity Analysis	Computation							
Sum of flow ratio for critical lane group, $Y_c = \sum$ (Critical lane groups v/s)	0.594							
Total Lost time per cycle, L(s) (Use Equation-9)	12							
Critical flow rate to capacity ratio, $X_c = (Y_c)(C)/(C\text{-}L)$ (Use Equation-53)	0.642							
Lane Group Capacity, Control Delay, and LOS	Computation							
	EB		WB		NB		SB	
Lane group								
Adjusted flow rate, v (veh/h)	163	697	171	633	187	1140	171	1308
Lane group capacity, c (veh/h)	277	1522	252	1522	327	2523	373	2523
v/c ratio, X = v/c	0.588	0.458	0.679	0.416	0.572	0.452	0.458	0.518
Total green ratio, g/C	0.37	0.37	0.37	0.37	0.58	0.46	0.58	0.46
Uniform delay, d_1 (Use Equation-56) $d_1 = \frac{0.5C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)	41.088	38.708	42.936	37.997	21.382	29.820	19.457	31.008

Appendix Table B6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation							
	EB		WB		NB		SB	
Lane group								
Incremental delay calibration, k	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
Upstream filtering / metering adjustment factor, I	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Duration of analysis period (h),T	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250
Incremental delay, d_2 (Use Equation-57) $d_2 = 900T (X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}$ (s/veh)	8.852	0.995	13.792	0.840	7.095	0.587	4.012	0.764
Initial queue delay, d_3 (s/veh)	0	0	0	0	0	0	0	0
Progression adjustment factor, PF (From Table-13)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Delay, $d = d_1 (PF) + d_2 + d_3$ (s/veh) (Use Equation-54)	49.9	39.7	56.7	38.8	28.5	30.4	23.5	31.8
LOS by lane group	D	D	E	D	C	C	C	C

Appendix Table B6 (Continued)

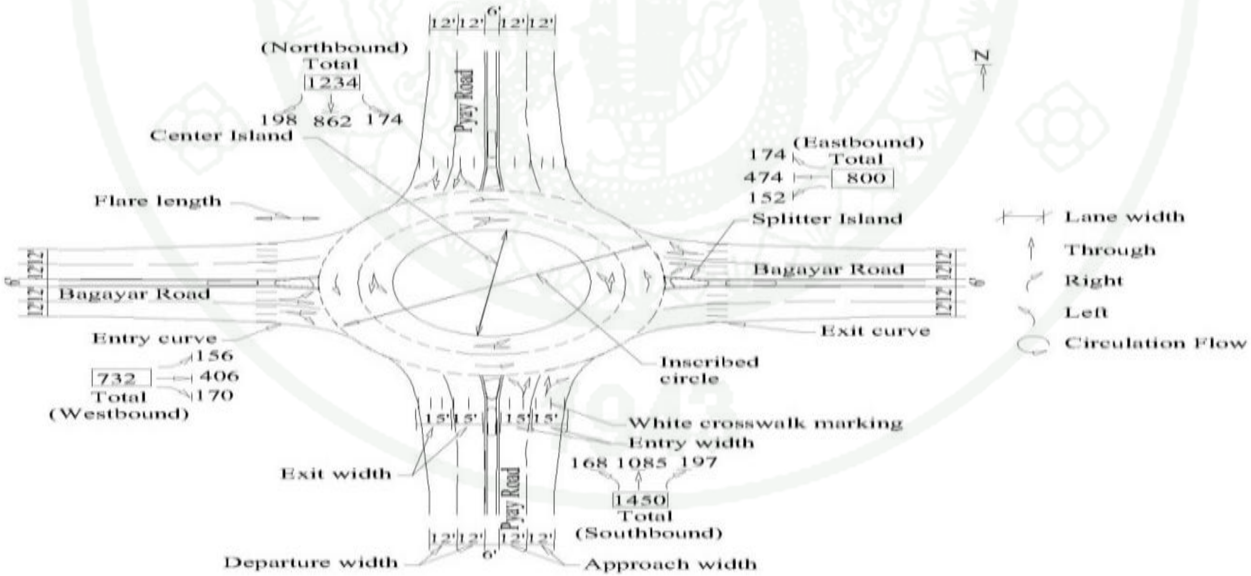
Lane Group Capacity, Control Delay, and LOS	Computation			
	EB	WB	NB	SB
Delay by approach, (Use Equation-58) $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	41.6	42.6	30.1	30.8
LOS by approach	D	D	C	C
Approach flow rate, v_A (veh/h)	860	804	1327	1479
Intersection delay, d_I (Use Equation-59) $d_I = \frac{\sum(d_A)(v_A)}{\sum(v_A)}$ (s/veh)	34.8	Intersection LOS Table-14		C

Appendix C

Myaynigone Roundabout Intersection

Appendix Table C1 General data and geometric design elements of Myaynigone roundabout intersection

General Information		Site Information	
Analyst		Intersection	Myaynigone Intersection
Agency or Company		Area Type	□ CBD ■ Other
Date performed	14.1.2011	Jurisdiction	
Analysis Time Period	8:00 – 9:00 A.M	Analysis Year	2011
Intersection Geometry			



Appendix Table C1 (Continued)

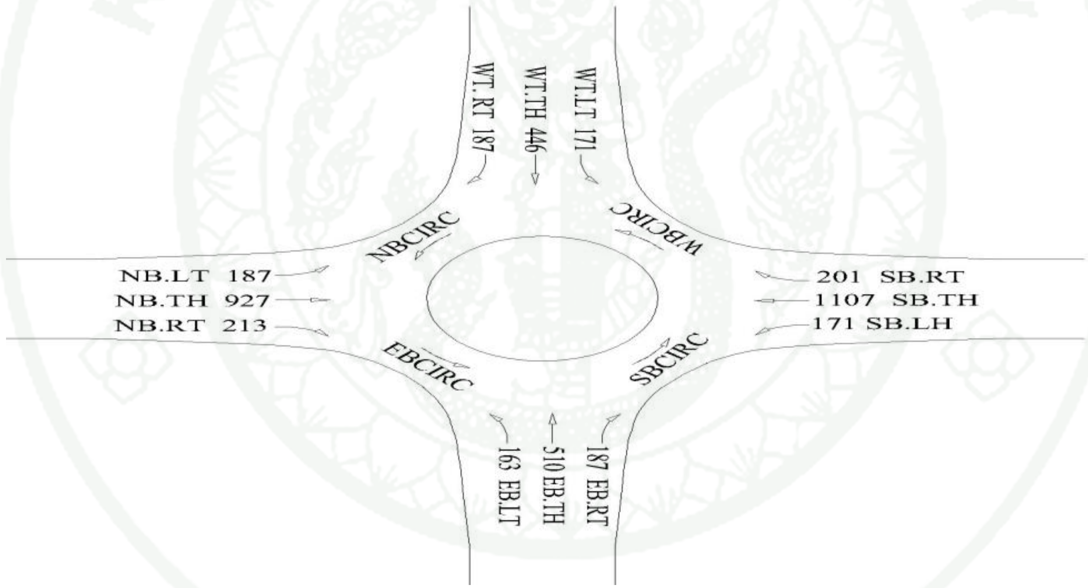
Entry Flow Rate and Geometric Design Elements	Computation											
	Eastbound			Westbound			Northbound			Southbound		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Entry flow rate, v (veh/h)	152	474	174	156	406	170	174	862	198	168	1085	197
Adjusted entry flow rate, v (veh/h)	163	510	187	171	446	187	187	927	213	171	1107	201
Entry width (ft)	30			30			30			30		
Approach width (ft)	12			12			12			12		
Circulatory roadway width (ft) (From Table-8)	32			32			32			32		
Exit width (ft)	30			30			30			30		
Departure width (ft)	12			12			12			12		
Design speed (mph) (From Table-6)	25			25			25			25		
Design vehicle (From Table-7)	WB-15(WB-50)			WB-15(WB-50)			WB-15(WB-50)			WB-15(WB-50)		
Entry radius (ft)	75			75			75			75		
Exit radius (ft)	150			150			150			150		
Inscribed circle diameter (ft) (From Table-8)	150			150			150			150		
Center island diameter (ft) (From Table-8)	86			86			86			86		

Appendix Table C1 (Continued)

Entry Flow Rate and Geometric Design Elements	Computation			
	Eastbound	Westbound	Northbound	Southbound
Flare length (ft)	80	80	80	80
Splitter island width (ft)	6	6	6	6
Total length of splitter islands (ft)	100	100	100	100
Stopping sight distance, d (ft) (From Table-9)	152.7	152.7	152.7	152.7

Appendix Table C2 Myaynigone Roundabout Intersection Calculation Worksheet

Double-Lane Roundabout Intersection	
Circulation Flow	Computation



Eastbound Circulation Flow (Use Equation-3)	$V_{EB\ CIRC}$ = 171 + 171 + 1107 = 1449 veh/ h
Westbound Circulation Flow (Use Equation-4)	$V_{WB\ CIRC}$ =163 + 187 + 927 = 1277 veh/ h
Northbound Circulation Flow (Use Equation-5)	$V_{NB\ CIRC}$ =171 + 163 + 510 = 844 veh/ h
Southbound Circulation Flow (Use Equation-6)	$V_{SB\ CIRC}$ = 187 + 171 + 446 = 804 veh/ h

Appendix Table C2 (Continued)

Double-Lane Roundabout Intersection	
Double-Lane Roundabout Capacity	Computation
Eastbound approach capacity of a double - lane roundabout	1380 veh/h
Westbound approach capacity of a double-lane roundabout	1509 veh/h
Northbound approach capacity of a double-lane roundabout	1818 veh/h
Southbound approach capacity of a double-lane roundabout	1846 veh/h
Pedestrian crossing volume	
Pedestrian volume (Eastbound, Westbound)	80 p/h
Pedestrian volume (Northbound, Southbound)	100 p/h
Capacity Reduction Factor, M (From Figure-9)	
Capacity Reduction Factor, M (Eastbound)	1.00
Capacity Reduction Factor, M (Westbound)	1.00
Capacity Reduction Factor, M (Northbound)	0.99
Capacity Reduction Factor, M (Southbound)	0.98

Appendix Table C2 (Continued)

Double-Lane Roundabout Intersection	
Double-Lane Roundabout Capacity	Computation
Pedestrian effects on entry capacity	
Eastbound approach capacity of a double-lane roundabout	1380 x 1.00 = 1380 veh/h
Westbound approach capacity of a double-lane roundabout	1509 x 1.00 = 1509 veh/h
Northbound approach capacity of a double-lane roundabout	1818 x 0.99 = 1800 veh/h
Southbound approach capacity of a double-lane roundabout	1846 x 0.98 = 1809 veh/h

Appendix Table C3 Capacity and LOS Worksheet

Capacity, Control Delay, and LOS	Computation											
	Eastbound			Westbound			Northbound			Southbound		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjusted entry flow rate, v (veh/h)	163	510	187	171	446	187	187	927	213	171	1107	201
Entry flow rate, v_x (veh/h)		860			804			1327			1479	
Circulatory flow rate, v_{CIRC} (veh/h)		1449			1277			844			804	
Double-lane roundabout capacity, $C_{m,x}$ (veh/h)		1380			1509			1800			1809	
v/c ratio, X= v/c	0.623	0.533	0.737	0.818								
Duration of analysis period (h), T	0.250	0.250	0.250	0.250								
Delay by approach, d_x (Use Equation-7)	6.815	5.074	7.362	10.126								
LOS by approach	A			A			A			A		
Intersection delay, d_I (Use Equation-59) $d_I = \frac{\sum(d_x)(V_x)}{\sum(V_x)}$	7.760		Intersection LOS (Table-5)							A		

Appendix D

Hledan Signalized Intersection

Appendix Table D1 Minimum pedestrian green time and adjustment factors for base saturation flow rate

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
1. Pedestrians / cycle	Pyay Road (1) = $120\frac{p}{h} \times \frac{1h}{3600s} \times\ 271s = 9.033p$ Insein Road (1), Insein Road (2) & Pyay Road (2) = $100\frac{p}{h} \times \frac{1h}{3600s} \times\ 271s = 7.528\,p$ Hledan Road, University Avenue Road = $80\frac{p}{h} \times \frac{1h}{3600s} \times\ 271s = 6.022\,p$
2. Minimum effective green time required for Pedestrians (Use Equation-10).	$G_p = 3.2 + \frac{L}{Sp} + (0.27\ N_{ped})$ for $W_E \leq 10$ ft G_p (Pyay Road (1)) = $3.2 + \frac{36}{4} + (0.27 \times 9.033) = 14.6$ s G_p (Insein Road (1), Insein Road (2) & Pyay Road (2)) = $3.2 + \frac{36}{4} + (0.27 \times 7.528)$ = 14.2 s G_p (Hledan Road, University Avenue Road) = $3.2 + \frac{48}{4} + (0.27 \times 6.022) = 16.8$ s

Appendix Table D1 (Continued)

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
3. Compare minimum effective green time required for pedestrians with actual effective green.	G_p (Pyay Road (1)) = 90.0 s, which is > 14.6 s G_p (Insein Road (1), Insein Road (2) & Pyay Road (2)) = 90.0 s, which is > 14.2 s G_p (Hledan Road, University Avenue Road) = 60.0 s, which is > 16.8 s
4. Proportion of left turns and right turns.	P_{LT} (Pyay Road (1)) = $\frac{101}{101+794}$ = 0.113, P_{LT} (Hledan Road) = $\frac{96}{96+273}$ = 0.260, P_{LT} (Pyay Road (2)) = 0, P_{LT} (Insein Road (1)) = $\frac{124}{124+582}$ = 0.176 P_{LT} (University Avenue Road) = $\frac{87}{87+297`}$ = 0.227 P_{LT} (Insein Road (2)) = $\frac{138}{138+558}$ = 0.198 P_{RT} for exclusive RT Lane is 1.000. P_{RT} (Insein Road (2)) =0 P_{RT} (Pyay Road (2)) = $\frac{185}{185+650}$ = 0.222
5. Lane width adjustment factor (Use Equation-14).	$f_w = 1 + \frac{(W-12)}{30}$, W = 12 ft, $f_w = 1 + \frac{(12-12)}{30}$ = 1.000

Appendix Table D1 (Continued)

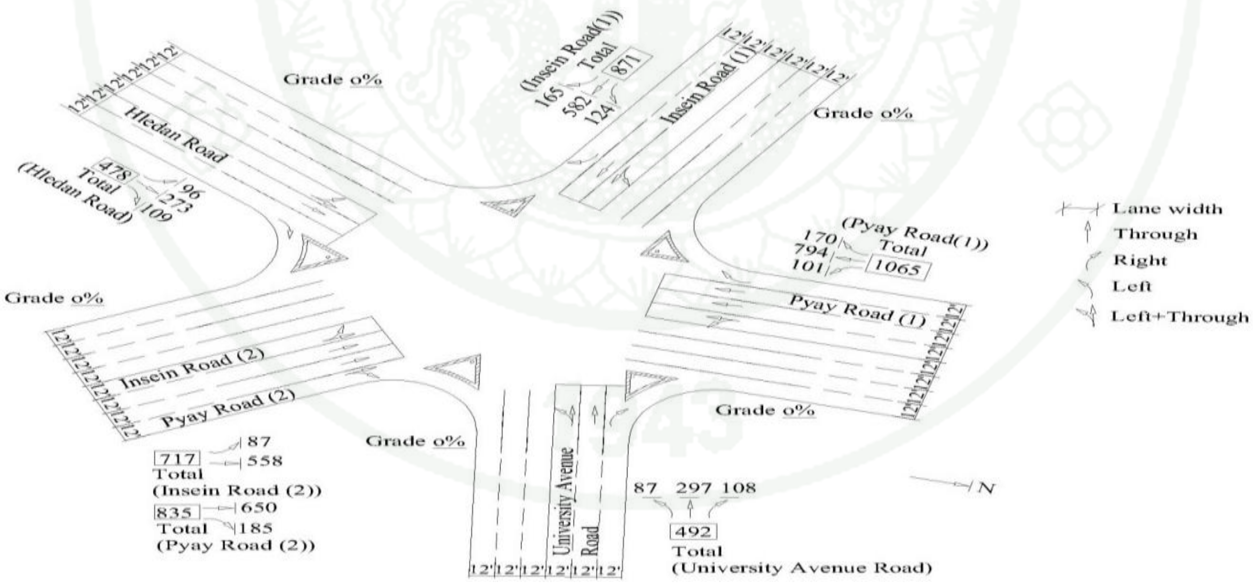
Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
6. Heavy-vehicle adjustment factor (Use Equation-15).	$f_{HV} = \frac{100}{100 + \%HV(E_T - 1)}$, $\%\ HV = 0$, $f_{HV} = \frac{100}{100 + 0(2-1)} = 1.000$
7. Percent grade adjustment factor (Use Equation-16).	$f_g = 1 - \frac{\%G}{200} = 1.000$, $\%G = 0$, $f_g = 1 - \frac{0\%}{200} = 1.000$
8. Parking adjustment factor (Use Equation-17).	$f_p = \frac{N - 0.1 - \frac{18Nm}{3600}}{N}$, No parking maneuvers, $f_p = 1.000$
9. Bus blockage adjustment factor (Use Equation-18).	$f_{bb} = \frac{N - \frac{14.4N_B}{3600}}{N}$, No buses stopping, $f_{bb} = 1.000$
10. Area type adjustment factor (Use Equation-20).	For all other areas, $f_a = 1.000$
11. Lane utilization adjustment factor (From Table-10).	No Specific data are Given. Use default of $f_{LU} = 1.000$ for exclusive LT. Use 0.908 for all shared LT.

Appendix Table D1 (Continued)

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
12. Left-turn adjustment factor	The Left-turn is permitted. A special Procedure is used. All approaches are opposed by multilane approaches. The supplemental worksheet for multilane approaches is used to determine the factor. The left turn is Protected. Use Equation (25), f_{LT} (Insein Road (1)) = $\frac{1}{1+0.05x0.176}$ = 0.991 f_{LT} (Insein Road (2)) = $\frac{1}{1+0.05x0.198}$ = 0.990
13. Right-turn adjustment factor (Use Equation-21).	For all exclusive - lane approaches, f_{RT} = 0.85 Share lane: (Pyay Road (2)), f_{RT} = 1.0 - (0.135) P_{RT} = 1.0 - 0.135 x 0.222 = 0.970
14. Left-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.
15. Right-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.

Appendix Table D2 Input Worksheet

General Information		Site Information	
Analyst		Intersection	Hledan Intersection
Agency or Company		Area Type	□ CBD ■ Other
Date performed	12.1.2011	Jurisdiction	
Analysis Time Period	8:00 – 9:00 A.M	Analysis Year	2011
Intersection Geometry			



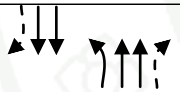
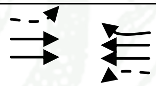
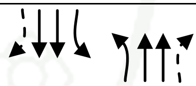
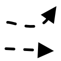
Appendix Table D2 (Continued)

Volume and Timing Input	Site Information																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V(veh/h)	101	794	170	-	650	185	124	582	165	138	558	-	96	273	109	89	297	108
% heavy vehicles, %HV		0			0			0			0			0			0	
Peak-hour factor, PHF		0.97			0.98			0.95			0.97			0.92			0.98	
Pretimed (P) or actuated (A)		P			P			P			P			P			P	
Start-up lost time, I_1 (s)		-			-			-			-			-			-	
Extension of effective green time, e (s)		-			-			-			-			-			-	

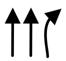
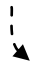
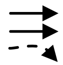
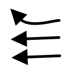
Appendix Table D2 (Continued)

Volume and Timing Input	Site Information																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Arrival type , AT		3			3			3			3			3			3	
Approach pedestrian volume, v_{ped}(p/h)		120			100			100			100			80			80	
Approach bicycle volume, v_{bic} (bicycle/h)		0			0			0			0			0			0	
Parking (Y or N)		N			N			N			N			N			N	
Parking maneuvers, N_m (maneuvers/h)		0			0			0			0			0			0	
Bus stopping, N_B (buses/h)		0			0			0			0			0			0	

Appendix Table D2 (Continued)

Volume and Timing Input	Site Information																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Min. timing for pedestrians, G_p(s)		14.6			14.2			14.2			14.2			16.8			16.8	
Signal Phasing Plan																		
Diagram	01 				02 			03 				04 						
Timing	G = 90 s Y = 4 s				G = 15 s Y = 4 s			G = 90 s Y = 4 s				G = 60 s Y = 4 s						
 Protected turns		 Protected turns Pedestrian										Cycle Length , C = 271 s						

Appendix Table D3 Volume Adjustment and Saturation Flow Rate Worksheet

Volume Adjustment	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	101	794	170	-	650	185	124	582	165	138	558	-	96	273	109	89	297	108
Peak-hour factor, PHF		0.97			0.98			0.95			0.97			0.92			0.98	
Adjusted flow rate, v_P =V/PHF (veh/h)	104	819	175	-	663	189	131	613	174	142	575	-	104	297	118	89	303	110
Lane group																		
Adjusted flow rate in lane group (veh/h)	104	994			852		131		787	142		575	104	415		89	413	

Appendix Table D3 (Continued)

Volume Adjustment	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Proportion of LT or RT (P_{LT} or P_{RT})	.113		1.00	0		.222	.176		1.00	.198		0	.260		1.00	.227		1.00
Saturation Flow Rate	Computation																	
Base saturation flow, s_o (pc/h/lane)	1900	1900			1900		1900	1900	1900	1900	1900	1900	1900	1900		1900	1900	
Number of Lanes, N	1	3			2		1	1	2	1	1	1	1	2		1	2	

Appendix Table D3 (Continued)

Saturation Flow Rate	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Lane width adjustment factor, f_w	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Heavy-vehicle adjustment factor, f_{HV}	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Grade adjustment factor, f_g	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Parking adjustment factor, f_p	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	

Appendix Table D3 (Continued)

Saturation Flow Rate	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Bus Blockage adjustment factor, f_{bb}	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Area type adjustment factor, f_a	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Lane utilization adjustment factor, f_{LU}	1.00	.908			.952		1.00	1.00	.952	1.00	1.00	.952	1.00	.952		1.00	.952	
Left-turn adjustment factor, f_{LT}	.665	1.00			-		.991	.597	1.00	.990	.577	1.00	.594	1.00		.605	1.00	

Appendix Table D3 (Continued)

Saturation Flow Rate	**Computation**																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Right-turn adjustment factor, f_{RT}	1.00	.850			.970		1.00	1.00	.850	1.00	1.00	-	1.00	.850		1.00	.850	
Left-turn ped/bike adjustment factor, f_{Lpb}	.998	1.00			-		1.00	.997	1.00	1.00	.997	1.00	.993	1.00		.994	1.00	
Right-turn ped/bike adjustment factor, f_{Rpb}	1.00	.891			.976		1.00	1.00	.909	1.00	1.00	-	1.00	.891		1.00	.891	

Appendix Table D3 (Continued)

Saturation Flow Rate	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjustment saturation flow, s (veh/h) (Use Equation - 13) $s = s_o N f_w f_{HV} f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$	1261	3920			3425		1883	1131	2795	1881	1093	1809	1121	2740		1143	2740	

Appendix Table D4 Supplemental Worksheet for Permitted Left Turns Opposed by Multilane Approach

Permitted Left Turn Adjustment Factor	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
Cycle Length, C (s)	271					
Total actual green time for LT Lane group, G (s)	90		109	109	60	60
Effective Permitted green time for LT lane group, G (s)	90		94	94	60	60
Opposing effective green time, g_o (s)	90		90	90	60	60
Number of Lanes in LT Lane group, N	3		2	2	2	2
Number of Lanes in opposing approach, N_o	3		3	3	2	2
Adjusted LT flow rate, v_{LT} (veh/h)	104		131	142	104	89
Proportion of LT volume in LT lane group, P_{LT}	0.113		0.176	0.198	0.260	0.227
Adjusted LT flow rate for opposing approach, v_o (veh/h)	852		747	744	392	401
Loft time for LT lane group, t_L	0		0	0	0	0
LT volume per cycle LTC = v_{LT} C/3600	7.829		9.861	10.689	7.829	6.700
Opposing lane utilization factor, f_{LUo}	0.908		0.952	0.952	0.952	0.952
Opposing flow per lane, per cycle $f_{bb} = \frac{v_o C}{3600 N_o f_{LUo}}$ (veh/C/Ln)	23.545		18.899	19.610	15.498	15.854

Appendix Table D4 (Continued)

Permitted Left Turn Adjustment Factor	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
$g_f = G[e^{-0.882(LTC^{0.717})}] - t_L$, $g_f < g$ (except for exclusive Left-turn lanes) (Use Equation-27)	1.902		1.151	0.878	1.268	1.906
Opposing Platoon ratio, R_{po}	1.000		1.000	1.000	1.000	1.000
Opposing queue ratio, $qr_o = \max[1-R_{po}(g_o/C),0]$	0.668		0.668	0.668	0.779	0.779
$g_q = \frac{v_{olc}qr_o}{0.5-[v_{olc}(1-qr_o)/g_o]} = t_L$, $v_{olc}(1-qr_o)/g_o \leq 0.49$ (note case-specific parameters) (Use Equation-28)	38.069		29.340	30.631	27.258	27.967
$g_u = g - g_q$ if $g_q \geq g_f$, (Use Equation-29) or $g_u = g - g_f$ if $g_q < g_f$ (Use Equation-30)	51.931		64.660	63.369	32.742	32.033
E_{L1} (Refer to Table-13) and (Use Equation-31,32,and 33)	3.514		2.959	3.046	2.124	2.142
$P_L = P_{LT}\left[1+\frac{(N-1)g}{(g_f+\frac{g_u}{E_{L1}}+4.24)}\right]$ (except with multilane subject approach) (Use Equation-34)	1.085		0.783	0.916	1.006	0.872

Appendix Table D4 (Continued)

Permitted Left Turn Adjustment Factor	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
$f_{min} = 2\,(1 + P_L)/g$ (Use Equation-36)	0.046		0.038	0.041	0.067	0.062
$f_m = [g_f/g] + [g_u/g]\left[1+\frac{1}{1+P_L(E_{L1}-1)}\right]$ ($f_{min} \leq f_m \leq 1.000$) (Use Equation-35)	0.176		0.284	0.244	0.277	0.299
$f_{LT} = [f_m + 0.91\,(N-1)]/N$ (except for permitted left turn) (Use Equation-37)	0.665		0.597	0.577	0.594	0.605

Appendix Table D5 Supplemental Worksheet for Pedestrian – Bicycle Effects on Permitted Left Turns and Right Turns

Pedestrian – Bicycle Adjustment Factor						
Permitted Left Turns	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
Effective pedestrian green time, g_p (s)	90		90	90	60	60
Conflicting Pedestrian Volume, vped (p/h)	120		100	100	80	80
$v_{pedg} = v_{ped} (C/g_p)$ (Use Equation-38)	361		301	301	361	361
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \leq 1000$) (Use Equation-40) or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if ($1000 < v_{pedg} \leq 5000$) (Use Equation-41)	0.181		0.151	0.151	0.181	0.181
Opposing queue clearing green, g_q (s)	38.069		29.340	30.631	27.258	27.967
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p, if $g_q \geq g_p$ then $f_{Lpb} = 1.0$	0.423		0.326	0.340	0.454	0.466
$OCC_{pedu} = OCC_{pedg} [1 - 0.5 (g_q/g_p)]$ (Use Equation-42)	0.143		0.126	0.125	0.140	0.139

Appendix Table D5 (Continued)

Pedestrian – Bicycle Adjustment Factor						
Permitted Left Turns	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
						
Opposing flow rate, v_o(veh/h)	852		717	744	392	401
$OCC_r = OCC_{pedu}\ [e^{-(5/3600)Vo}]$ (Use Equation-44)	0.044		0.047	0.044	0.081	0.080
Number of cross - street receiving lanes, N_{rec}	3		3	3	4	4
Number of turning lanes, N_{turn}	1		1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, (Use Equation-46) or $A_{pbT} = 1 - 0.6\ (OCC_r)$ if $N_{rec} > N_{turn}$ (Use Equation-47)	0.974		0.972	0.974	0.951	0.952
Proportion of left turn, P_{LT}	0.113		0.176	0.198	0.260	0.227
Proportion of left turns using protected phase, P_{LTA} = (1 - permitted phase f_{LT})/0.95 (Use Equation-48)	0.353		0.424	0.445	0.427	0.416
$f_{Lpb} = 1.0 - P_{LT}\ (1 - A_{pbT})\ (1 - P_{LTA})$ (Use Equation-49)	0.998		0.997	0.997	0.993	0.994

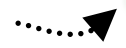
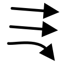
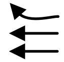
Appendix Table D5 (Continued)

Pedestrian – Bicycle Adjustment Factor						
Permitted Right Turns	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
Effective pedestrian green time, g_p (s)	90	90	90		60	60
Conflicting pedestrian volume, v_{ped} (p/h)	120	100	100		80	80
Conflicting pedestrian volume, v_{bic} (bicycle/h)	0	0	0		0	0
$v_{pedg} = v_{pedg}$ (C/g_p)	361	361	301		361	361
$OCC_{pedg} = v_{pedg}/2000$ if $(v_{pedg} \leq 1000)$ or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if $(1000 < v_{pedg} \leq 5000)$	0.181	0.181	0.151		0.181	0.181
Effective green, g (s)	90	90	90		60	60
$v_{bicg} = v_{bic}$ (C/g) (Use Equation-39)	0	0	0		0	0
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg})$ (Use Equation-45)	0.181	0.181	0.151		0.181	0.181

Appendix Table D5 (Continued)

Pedestrian – Bicycle Adjustment Factor						
Permitted Right Turns	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
Number of cross - street receiving lanes, N_{rec}	3	3	3		4	4
Number of turning lanes, N_{turn}	1	1	1		1	1
ApbT = 1 - OCCr if Nrec = Nturn, ApbT = 1 - 0.6 (OCCr) if Nrec > Nturn	0.891	0.891	0.909		0.891	0.891
Proportion of right turns, P_{RT}	1.000	0.222	1.000		1.000	1.000
Proportion of left turns using protected phase, P_{RTA}	0	0	0		0	0
$f_{Rpb} = 1.0 - P_{RT} (1- A_{pbT}) (1 - P_{RTA})$ (Use Equation-50)	0.891	0.976	0.909		0.891	0.891

Appendix Table D6 Capacity and LOS Worksheet

Capacity Analysis	Computation												
Phase Number	1	1	2	2	2	2	3	3	3	4	4	4	4
Phase Type	P	P	P	P	P	P	P	P	P	P	P	P	P
Lane group													
Adjusted flow rate, v (veh/h)	131	142	0	787	0	575	104	994	852	104	415	89	413
Saturation flow rate, s (veh/h)	1883	1881	1131	2795	1093	1809	1261	3920	3425	1121	2740	1143	2740
Lost time, t_L (s), $t_L = I_1 - Y - e$ (Use Equation-8)	4	4	4	4	4	4	4	4	4	4	4	4	4
Effective green time, g (s), $g = G + Y - t_L$ (Use Equation-26)	15	15	94	90	94	90	90	90	90	60	60	60	60
Green ratio, g/C	0.055	0.055	0.347	0.332	0.347	0.332	0.332	0.332	0.332	0.221	0.221	0.221	0.221
Lane group capacity, c = s (g/C) (veh/h) (Use Equation-51)	104	103	392	928	379	601	419	1301	1137	248	606	253	606
v/c ratio, X (Use Equation-52)	1.260	1.379	0	0.848	0	0.957	0.248	0.764	0.749	0.419	0.685	0.352	0.682

Appendix Table D6 (Continued)

Capacity Analysis	Computation												
Flow ratio, v/s		0.075				0.318		0.254			0.151		
Critical lane group/phase (✓)		✓				✓		✓			✓		
Sum of flow ratio for critical lane group, $Y_c = \sum$ (Critical lane groups v/s)	0.798												
Total Lost time per cycle, L (s) (Use Equation-9)	16												
Critical flow rate to capacity ratio, $X_c = (Y_c)$ (C) / (C-L) (Use Equation-53)	0.848												

Lane Group Capacity, Control Delay, and LOS	Computation										
	Pyay Road (1)		Pyay Road (2)	Insein Road (1)		Insein Road (2)		Hledan Road		University Avenue Road	
Lane group	↰	↑↑↱	↲↓↓	↰	↑↑↱	↳	↓↓	↗	⇉↘	↙	↖⇇
Adjusted flow rate, v (veh/h)	104	994	852	131	787	142	575	104	415	89	413

Appendix Table D6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation										
	Pyay Road (1)		Pyay Road (2)	Insein Road (1)		Insein Road (2)		Hledan Road		University Avenue Road	
Lane group capacity, c (veh/h)	419	1301	1137	496	928	482	601	248	606	253	606
v/c ratio, X = v/c	0.248	0.764	0.749	0.264	0.848	0.295	0.957	0.419	0.685	0.352	0.682
Total green ratio, g/C	0.332	0.332	0.332	0.402	0.332	0.402	0.332	0.221	0.221	0.221	0.221
Uniform delay, d_1 (Use Equation-56) $d_1 = \frac{0.5C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)	65.89	81.01	80.48	54.21	84.16	54.98	88.62	90.62	96.90	89.16	96.82
Incremental delay calibration, k	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
Upstream filtering / metering adjustment factor, I	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Duration of analysis period (h), T	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250

Appendix Table D6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation										
	Pyay Road (1)		Pyay Road (2)	Insein Road (1)		Insein Road (2)		Hledan Road		University Avenue Road	
Incremental delay, d_2 (Use Equation-57) $d_2 = 900T\ (X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}$ (s/veh)	1.411	4.304	4.542	1.297	9.501	1.555	27.519	5.134	6.189	3.815	6.109
Initial queue delay, d_3 (s/veh)	0	0	0	0	0	0	0	0	0	0	0
Progression adjustment factor, PF (From Table-13)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Delay, d (Use Equation-54) $d = d_1\,(PF) + d_2 + d_3$ (s/veh)	67.3	85.3	85.0	55.5	93.7	56.5	116.1	95.8	103.1	93.0	102.9
LOS by lane group	E	F	F	E	F	E	F	F	F	F	F
Delay by approach, d_A (Use Equation-58) $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	83.6	85.0	88.2	104.3	101.6	101.1					

Appendix Table D6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road (1)	Insein Road (2)	Hledan Road	University Avenue Road
LOS by approach	F	F	F	F	F	F
Approach flow rate, v_A (veh/h)	1098	852	918	717	519	502
Intersection delay, d_I (Use Equation-59) $d_I = \frac{\sum(d_A)(v_A)}{\sum(v_A)}$ (s/veh)	91.9	Intersection LOS Table-14			F	

Appendix E

Hledan Channelized Intersection with Signal Control

Appendix Table E1 Minimum pedestrian green time and adjustment factors for base saturation flow rate

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
1. Pedestrians / cycle	Pyay Road (1) = $120\frac{p}{h} \text{ x } \frac{1h}{3600s} \text{ x } 271s = 9.033p$ Insein Road (1), Insein Road (2) & Pyay Road (2) = $100\frac{p}{h} \text{ x } \frac{1h}{3600s} \text{ x } 271s = 7.528p$ Hledan Road, University Avenue Road = $80\frac{p}{h} \text{ x } \frac{1h}{3600s} \text{ x } 271s = 6.022p$
2. Minimum effective green time required for pedestrians (Use Equation-10).	$G_p = 3.2 + \frac{L}{Sp} + (0.27\, N_{ped})$ for $W_E \leq 10$ ft G_p (Pyay Road (1)) = $3.2 + \frac{48}{4} + (0.27 \text{ x } 9.033) = 17.6$ s G_p (Insein Road (1), Insein Road (2) & Pyay Road (2)) = $3.2 + \frac{48}{4} + (0.27 \text{ x } 7.528)$ = 17.2 s G_p (Hledan Road , University Avenue Road) = $3.2 + \frac{60}{4} + (0.27 \text{ x } 6.022)$ = 19.8 s

Appendix Table E1 (Continued)

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
3. Compare minimum effective green time required for pedestrians with actual effective green.	G_p (Pyay Road (1)) = 90.0 s, which is > 17.6 s G_p (Insein Road (1), Insein Road (2) & Pyay Road (2)) = 90.0 s, which is > 17.2 s G_p (Hledan Road, University Avenue Road) = 60.0 s, which is > 19.8 s
4. Proportion of left turns and right turns.	Proportions of left-and right-turn traffic are found by dividing the appropriate turning flow rates by the total lane group flow rate. P_{LT} for exclusive LT Lane is 1.000, P_{RT} for exclusive RT Lane is 1.000.
5. Lane width adjustment factor (Use Equation-14).	$f_w = 1 + \frac{(W-12)}{30}$, f_w = 12 ft (for all Lanes), $f_w = 1 + \frac{(12-12)}{30} = 1.000$
6. Heavy-vehicle adjustment factor (Use Equation-15).	$f_{HV} = \frac{100}{100+\%HV(E_T-1)}$, % HV = 0, $f_{HV} = \frac{100}{100+0(2-1)} = 1.000$
7. Percent grade adjustment factor (Use Equation-16).	%G = 0, $f_g = 1 - \frac{\%G}{200} = 1.000$
8. Parking adjustment factor (Use Equation-17).	N_m = 0 (No parking maneuvers), $f_p = \frac{N - 0.1 - \frac{18Nm}{3600}}{N} = 1.00$

Appendix Table E1 (Continued)

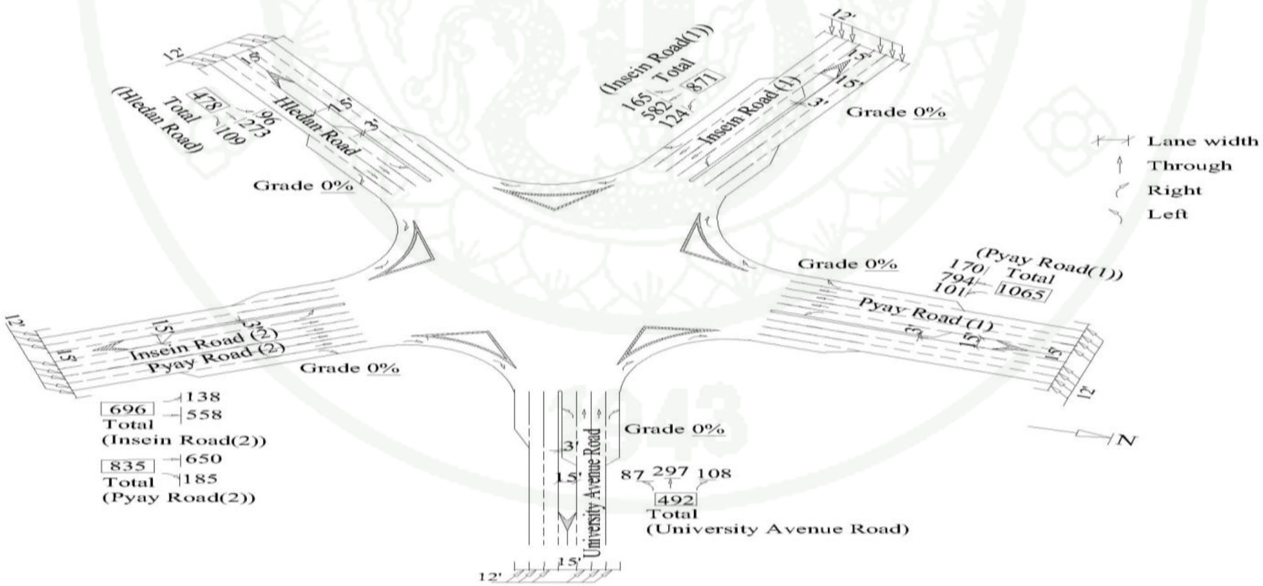
Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
9. Bus blockage adjustment factor (Use Equation-18).	N_B = 0 (No buses Stopping), $f_{bb} = \frac{N - \frac{14.4N_B}{3600}}{N} = 1.00$
10. Area type adjustment factor (Use Equation-20).	For all other areas, f_a = 1.000
11. Lane utilization adjustment factor (From Table-10).	f_{LU} (Pyay Road (1)) = 0.908 f_{LU} (Insein Road (1), Pyay Road (2), Insein Road (2) Hledan Road , University Avenue Road) = 0.952
12. Left-turn adjustment factor	The Left-turn is permitted. A special Procedure is used. All approaches are opposed by multilane approaches. The supplemental worksheet for multilane approaches is used to determine the factor. The left turn is protected. Use Equation (23), For exclusive left-turn Lane, f_{LT} = 0.950
13. Right-turn adjustment factor (Use Equation-21).	For all exclusive – lane approaches, f_{RT} = 0.85

Appendix Table E1 (Continued)

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
14. Left-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.
15. Right-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.

Appendix Table E2 Input Worksheet

General Information		Site Information	
Analyst		Intersection	Hledan Intersection
Agency or Company		Area Type	□ CBD ■ Other
Date performed	12.1.2011	Jurisdiction	
Analysis Time Period	8:00 – 9:00 A.M	Analysis Year	2011
Intersection Geometry			



Appendix Table E2 (Continued)

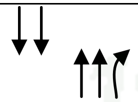
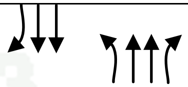
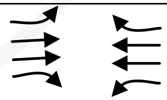
Volume and Timing Input	Site Information																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	101	794	170	-	650	185	124	582	165	138	558	-	96	273	109	89	297	108
% heavy vehicles, %HV		0			0			0			0			0			0	
Peak-hour factor, PHF		0.97			0.98			0.95			0.97			0.92			0.98	
Pretimed (P) or Actuated (A)		P			P			P			P			P			P	
Start-up lost time, I_1 (s)		-			-			-			-			-			-	

Appendix Table E2 (Continued)

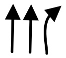
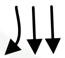
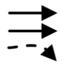
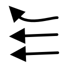
Volume and Timing Input	Site Information																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Extension of effective green time, e(s)		-			-			-			-			-			-	
Arrival type, AT		3			3			3			3			3			3	
Approach pedestrian volume, v_{ped} (p/h)		120			100			100			100			80			80	
Approach bicycle volume, v_{bic} (bicycles/h)		0			0			0			0			0			0	
Parking (Y or N)		N			N			N			N			N			N	

Appendix Tables E2 (Continued)

Volume and Timing Input	Site Information																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Parking maneuvers, N_m (maneuvers/h)		0			0			0			0			0			0	
Bus stopping, N_B (buses/h)		0			0			0			0			0			0	
Min. timing for pedestrians, G_p ,s		17.6			17.2			17.2			17.2			19.8			19.8	

Signal Phasing Plan					
Diagram	01 	02 	03 	04 	
Timing	G = 15 Y = 4	G = 90 Y = 4	G = 90 Y = 4	G = 60 Y = 4	
Protected turns	Protected turns Pedestrian		Cycle Length , C = 271 s		

Appendix Table E3 Volume Adjustment and Saturation Flow Rate Worksheet

Volume Adjustment	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, v (veh/h)	101	794	170	-	650	185	124	582	165	138	558	-	96	273	109	89	297	108
Peak-hour factor, PHF		0.97			0.98			0.95			0.97			0.92			0.98	
Adjusted flow rate, v_P = V/PHF (veh/h)	104	819	175	-	663	189	131	613	174	142	575	-	104	297	118	89	303	110
Lane group																		
Adjusted flow rate in lane group (veh/h)	104	994			852		131		787	142		575	104	415		89	413	

Appendix Table E3 (Continued)

Volume Adjustment	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Proportion of LT or RT (P_{LT} or P_{RT})	1.00		1.00	0		1.00	1.00		1.00	1.00		0	1.00		1.00	1.00		
Saturation Flow Rate	Computation																	
Base saturation flow, s_o (pc/h/ln)	1900	1900			1900		1900	1900	1900	1900	1900	1900	1900	1900		1900	1900	
Number of Lanes, N	1	4			3		1	1	3	1	1	2	1	3		1	3	

Appendix Table E3 (Continued)

Saturation Flow Rate	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Lane width adjustment factor, f_w	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Heavy-vehicle adjustment factor, f_{HV}	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Grade adjustment factor, f_g	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Parking adjustment factor, f_p	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	

Appendix Table E3 (Continued)

Saturation Flow Rate	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Bus Blockage adjustment factor, f_{bb}	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Area type adjustment factor, f_a	1.00	1.00			1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Lane utilization adjustment factor, f_{LU}	1.00	.908			.952		1.00	1.00	.952	1.00	1.00	.952	1.00	.952		1.00	.952	
Left-turn adjustment factor, f_{LT}	.294	1.00			-		.950	.357	1.00	.950	.336	1.00	.439	1.00		.444	1.00	

Appendix Table E3 (Continued)

Saturation Flow Rate	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Right-turn adjustment factor, f_{RT}	1.00	.850			.850		1.00	1.00	.850	1.00	1.00	-	1.00	.850		1.00	.850	
Left-turn ped/bike adjustment factor, f_{Lpb}	.990	1.00			-		1.00	.988	1.00	1.00	.989	1.00	.974	1.00		.973	1.00	
Right-turn ped/bike adjustment factor, f_{Rpb}	1.00	.891			.891		1.00	1.00	.909	1.00	1.00	-	1.00	.891		1.00	.891	

Appendix Table E3 (Continued)

Saturation Flow Rate	Computation																	
	Pyay Road (1)			Pyay Road (2)			Insein Road(1)			Insein Road(2)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjustment saturation flow, s (veh/h)	553	5226			4110		1805	670	4193	1805	631	3618	812	4110		821	4110	

Appendix Table E4 Supplemental Worksheet for Permitted Left Turns Opposed Multilane Approach

Permitted Left Turn Adjustment Factor	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
Cycle Length, C(s)	271					
Total actual green time for LT Lane group, G (s)	90		109	109	60	60
Effective Permitted green time for LT lane group, G (s)	90		94	94	60	60
Opposing effective green time, g_o (s)	90		90	90	90	90
Number of Lanes in LT Lane group, N	1		1	1	1	1
Number of Lanes in opposing approach, N_o	4		4	4	3	3
Adjusted LT flow rate, v_{LT} (veh/h)	104		131	142	104	89
Proportion of LT volume in LT lane group, P_{LT}	1.000		1.000	1.000	1.000	1.000
Adjusted LT flow rate, opposing approach, v_o (veh/h)	663		575	613	303	297
Lost time for LT lane group, t_L	0		0	0	0	0
LT volume per cycle LTC = v_{LT} C/3600	7.829		9.861	10.689	7.829	6.700
Opposing lane utilization factor, f_{LUo}	0.908		0.952	0.952	0.952	0.952
Opposing flow per lane, per cycle $v_{olc} = \frac{v_o C}{3600 N_o f_{LUo}}$ (veh/C/Ln)	13.742		11.367	12.118	7.986	7.828

Appendix Table E4 (Continued)

Permitted Left Turn Adjustment Factor	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
$g_f = G[e^{-0.882(LTC^{0.717})}] - t_L$, $g_f < g$ (except for exclusive Left-turn lanes) (Use Equation-27)	0		0	0	0	0
Opposing Platoon ratio, R_{po}	1.000		1.000	1.000	1.000	1.000
Opposing queue ratio, $qr_o = \max[1-R_{po}(g_o/C),0]$	0.668		0.668	0.668	0.779	0.779
$g_q = \frac{v_{olc}qr_o}{0.5-[v_{olc}(1-qr_o)/g_o]} = t_L$, $v_{olc}(1-qr_o)/g_o \leq 0.49$ (note case-specific parameters) (Use Equation-28)	20.431		16.576	17.779	13.220	12.942
$g_u = g - g_q$ if $g_q \geq g_f$, (Use Equation-29) or $g_u = g - g_f$ if $g_q < g_f$ (Use Equation-30)	69.569		77.424	76.221	46.780	47.058
E_{L1} (Refer to Table-13) and (Use Equation-31,32,33)	2.625		2.310	2.410	1.777	1.768
$P_L = P_{LT}\left[1+\frac{(N-1)g}{(g_f+\frac{g_u}{E_{L1}}+4.24)}\right]$ (except with multilane subject approach) (Use Equation-32)	1.000		1.000	1.000	1.000	1.000

Appendix Table E4 (Continued)

Permitted Left Turn Adjustment Factor	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
$f_{min} = 2\,(1 + P_L)/g$ (Use Equation-36)	0.044		0.043	0.043	0.067	0.067
$f_m = [g_f/g] + [g_u/g]\left[1+\frac{1}{1+P_L(E_{L1}-1)}\right]$ $(f_{min} \le f_m \le 1.000)$ (Use Equation-35)	0.294		0.357	0.336	0.439	0.444
$f_{LT} = [f_m + 0.91\,(N-1)]/N$ (except for permitted left turn) (Use Equation-37)	0.294		0.357	0.336	0.439	0.444

Appendix Table E5 Supplemental Worksheet for Pedestrian – Bicycle Effects on Permitted Left Turns and Right Turns

Pedestrian – Bicycle Adjustment Factor						
Permitted Left Turns	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
Effective pedestrian green time, g_p (s)	90		90	90	60	60
Conflicting Pedestrian Volume, v_{ped} (p/h)	120		100	100	80	80
$v_{pedg} = v_{ped} (C/g_p)$ ($v_{pedg} \leq 5000$) (Use equation-38)	361		301	301	361	361
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \leq 1000$) (Use Equation-40) $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if ($1000 < v_{pedg} \leq 5000$) (Use Equation-41)	0.181		0.151	0.151	0.181	0.181
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, (Use Equation-46) $A_{pbT} = 1 - 0.6 (OCC_r)$ if $N_{rec} > N_{turn}$ (Use Equation-47)	0.962		0.963	0.965	0.936	0.936
Proportion of left turn, P_{LT}	1.000		1.000	1.000	1.000	1.000

Appendix Table E5 (Continued)

Pedestrian – Bicycle Adjustment Factor						
Permitted Left Turns	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
						
Proportion of left turns using protected phase, P_{LTA} = (1 - permitted phase f_{LT})/0.95 (Use Equation-48)	0.743		0.677	0.699	0.591	0.585
$f_{Lpb} = 1.0 - P_{LT} (1- A_{pbT}) (1 - P_{LTA})$ (Use Equation-49)	0.990		0.988	0.989	0.974	0.973
Permitted Right Turns	Computation					
						
Effective pedestrian green time, g_p (s)	90	90	90		60	60
Conflicting Pedestrian Volume, v_{ped} (p/h)	120	100	100		80	80
Conflicting Pedestrian Volume, v_{bic} (bicycle/h)	0	0	0		0	0
$v_{pedg} = v_{pedg} (C/g_p)$	361	361	301		361	361
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \leq 1000$) or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if ($1000 < v_{pedg} \leq 5000$)	0.181	0.181	0.151		0.181	0.181

Appendix Table E5 (Continued)

Pedestrian – Bicycle Adjustment Factor						
Permitted Right Turns	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road(1)	Insein Road(2)	Hledan Road	University Avenue Road
Effective green, g(s)	90	90	90		60	60
$v_{bicg} = v_{bic}$ (C/g) ($v_{bicg} \leq 1900$) (Use Equation-37)	0	0	0		0	0
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg})$ (Use Equation-45)	0.181	0.181	0.151		0.181	0.181
Number of cross-street receiving lanes, N_{rec}	4	4	4		5	5
Number of turning lanes, N_{turn}	1	1	1		1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, $A_{pbT} = 1 - 0.6\,(OCC_r)$ if $N_{rec} > N_{turn}$	0.891	0.891	0.909		0.891	0.891
Proportion of right turns, P_{RT}	1.000	1.000	1.000		1.000	1.000
Proportion of right turns using protected phase, P_{RTA}	0	0	0		0	0
$f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$ (Use Equation-50)	0.891	0.891	0.909		0.891	0.891

Appendix Table E6 Capacity and LOS Worksheet

Capacity Analysis	Computation												
Phase Number	1	1	2	2	2	2	3	3	3	4	4	4	4
Phase Type	P	P	P	P	P	P	P	P	P	P	P	P	P
Lane group													
Adjusted flow rate, v (veh/h)	131	142	0	787	0	575	104	994	852	104	415	89	413
Saturation flow rate, (veh/h)	1805	1085	670	4193	610	3618	553	5226	4110	812	4110	821	4110
Lost time, t_L (s), $t_L = I_1 - Y - e$ (Use Equation-8)	4	4	4	4	4	4	4	4	4	4	4	4	4
Lane group													
Effective green time, g (s), $g = G + Y - t_L$ (Use Equation-26)	15	15	94	90	94	90	90	90	90	60	60	60	60
Green ratio, g/C	0.055	0.055	0.347	0.332	0.347	0.332	0.332	0.332	0.332	0.221	0.221	0.221	0.221

Appendix Table E6 (Continued)

Capacity Analysis	Computation												
Lane group capacity, c = s (g/C) (veh/h) (Use Equation-51)	99	99	232	1392	212	1201	184	1735	1365	179	908	181	908
v/c ratio, X (Use Equation52)	1.323	1.434	0	0.565	0	0.479	0.565	0.573	0.624	0.581	0.457	0.492	0.455
Flow ratio, v/s		0.079		0.188					0.207	0.128			
Critical lane group/phase (✓)		✓		✓					✓	✓			
Sum of flow ratio for critical lane group, $Y_c = \sum$ (Critical lane groups v/s)	0.602												
Total Lost time per cycle, L(s)	16												
Critical flow rate to capacity ratio, $X_c = (Y_c)(C)/(C-L)$ (Use Equation-53)	0.640												

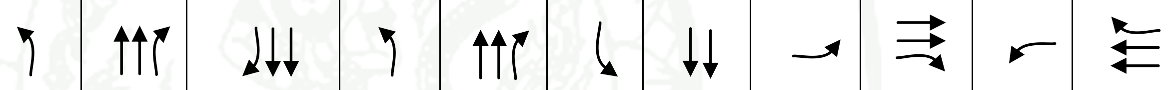
Appendix Table E6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation										
	Pyay Road (1)		Pyay Road (2)	Insein Road (1)		Insein Road (2)		Hledan Road		University Avenue Road	
Lane group											
Adjusted flow rate, v (veh/h)	104	994	852	131	787	142	575	104	415	89	413
Lane group capacity, c (veh/h)	184	1735	1365	331	1392	311	1201	179	908	181	908
v/c ratio, X = v/c	0.565	0.573	0.624	0.396	0.565	0.457	0.479	0.581	0.457	0.492	0.455
Total green ratio, g/C	0.332	0.332	0.332	0.402	0.332	0.402	0.332	0.221	0.221	0.221	0.221
Uniform delay, d_1 (Use Equation-56) $d_1 = \frac{0.5C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)	74.42	74.67	76.26	57.63	74.42	59.36	71.89	94.34	91.47	92.26	91.42
Incremental delay calibration, k	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
Upstream filtering / metering adjustment factor, I	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000

Appendix Table E6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation										
	Pyay Road (1)		Pyay Road (2)	Insein Road (1)		Insein Road (2)		Hledan Road		University Avenue Road	
Lane group											
Duration of analysis period (h), T	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250
Incremental delay, d_2 (Use Equation-57) $d_2 = 900T\,(X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}$ (s/veh)	11.974	1.382	2.161	0.862	1.665	4.778	1.370	13.042	1.657	9.257	1.644
Initial queue delay, d_3 (s/veh)	0	0	0	0	0	0	0	0	0	0	0
Progression adjustment factor, PF (From Table-13)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Delay, $d = d_1\,(PF) + d_2 + d_3$ (s/veh) (Use Equation-54)	86.4	76.1	78.4	58.5	76.1	64.1	73.3	107.4	93.1	101.5	93.1
LOS by lane group	F	E	E	E	E	E	E	F	F	F	F

Appendix Table E6 (Continued)

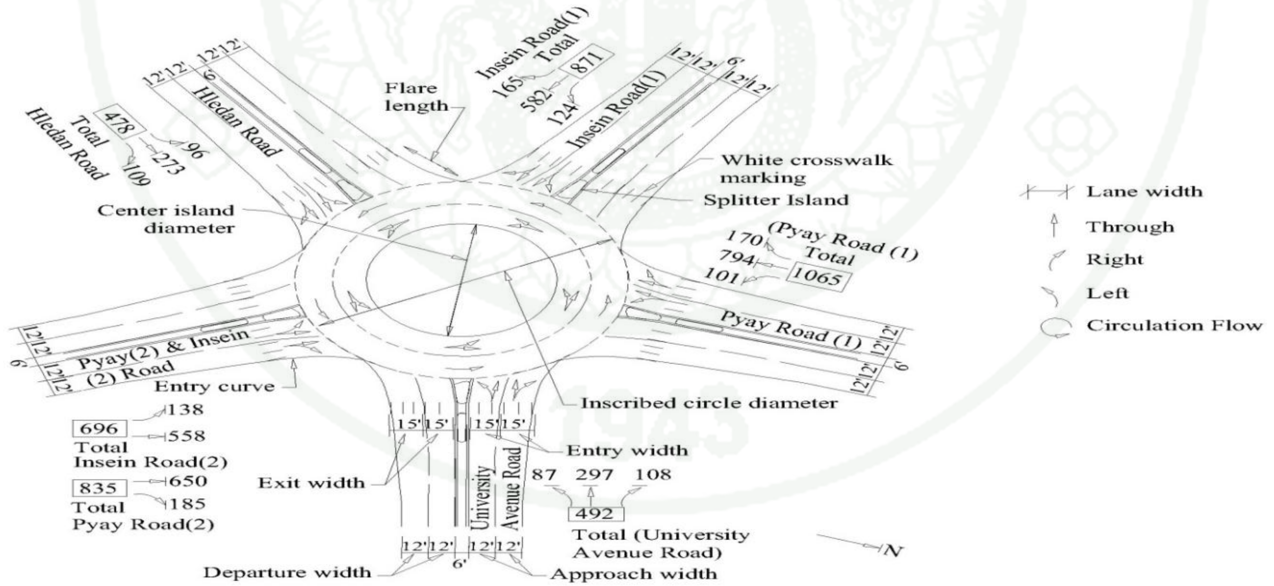
Lane Group Capacity, Control Delay, and LOS	Computation					
	Pyay Road (1)	Pyay Road (2)	Insein Road (1)	Insein Road (2)	Hledan Road	University Avenue Road
Lane group						
Delay by approach,(Use Equation-58) $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	77.1	78.4	73.6	71.5	96.0	94.6
LOS by approach	E	E	E	E	F	F
Approach flow rate, v_A (veh/h)	1098	852	918	717	519	502
Intersection delay, d_I (Use Equation-59) $d_I = \frac{\sum(d_A)(v_A)}{\sum(v_A)}$ (s/veh)	79.8	Intersection LOS Table-14			E	

Appendix F

Hledan Roundabout Intersection

Appendix Table F1 General data and geometric design elements of Hledan roundabout intersection

General Information		Site Information	
Analyst		Intersection	Hledan Intersection
Agency or Company		Area Type	□ CBD ■ Other
Date performed	12.1.2011	Jurisdiction	
Analysis Time Period	8:00 – 9:00 A.M	Analysis Year	2011
Intersection Geometry			



Appendix Table F1 (Continued)

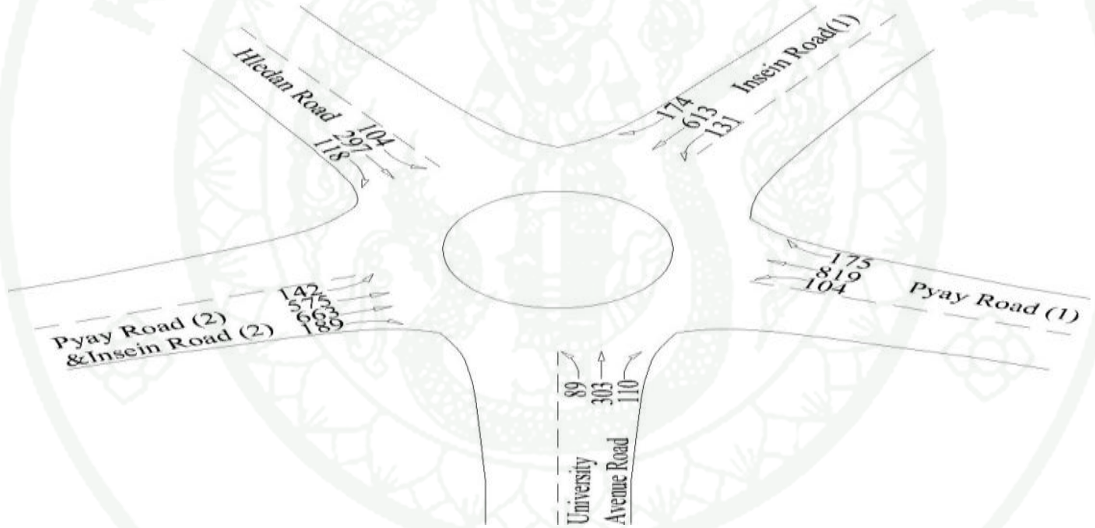
Entry Flow Rate and Geometric Design Elements	Computation														
	Pyay Road (1)			Pyay Road (2) & Insein Road (2)			Insein Road (1)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Entry flow rate, v (veh/h)	101	794	170	138	1208	185	124	582	165	96	273	109	87	297	108
Adjusted entry flow rate, v (veh/h)	104	819	175	142	1238	189	131	613	174	104	297	118	89	303	110
Entry width (ft)	30			30			30			30			30		
Approach width (ft)	12			12			12			12			12		
Circulatory roadway width (ft)	32			32			32			32			32		
Exit width (ft)	30			30			30			30			30		
Departure width (ft)	12			12			12			12			12		
Design Speed (mph) (From Table-6)	25			25			25			25			25		
Design Vehicle (From Table-7)	WB-15(WB-50)			WB-15(WB-50)			WB-15(WB-50)			WB-15(WB-50)			WB-15 (WB-50)		
Entry radius (ft)	75			75			75			75			75		
Exit radius (ft)	150			150			150			150			150		
Inscribed circle diameter (ft) (From Table-8)	150			150			150			150			150		

Appendix Table F1 (Continued)

Entry Flow Rate and Geometric Design Elements	Computation														
	Pyay Road (1)			Pyay Road (2) & Insein Road (2)			Insein Road (1)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Center island diameter (ft) (From Table-8)	86			86			86			86			86		
Flare Length (ft)	80			80			80			80			80		
Splitter island width (ft)	6			6			6			6			6		
Total length of splitter islands (ft)	100			100			100			100			100		
Stopping sight distance, d (ft)	152.7			152.7			152.7			152.7			152.7		

Appendix Table F2 Hledan Roundabout Intersection Calculation Worksheet

Double-Lane Roundabout Intersection	
Circulation Flow	Computation



Pyay Road (1) Circulation Flow	$V_{\text{Pyay Road (1) CIRC}}$ = 1213 veh/h
Insein Road (1) Circulation Flow	$V_{\text{Insein Road (1) CIRC}}$ = 1457 veh/h
Hledan Road Circulation Flow	$V_{\text{Hledan Road CIRC}}$ = 1756 veh/h
Pyay Road (2) & Insein Road (2) Circulation Flow	$V_{\text{Pyay Road (2) \& Insein Road (2) CIRC}}$ = 636 veh/h
University Avenue Road Circulation Flow	$V_{\text{University Avenue Road CIRC}}$ = 1615 veh/h

Appendix Table F2 (Continued)

Double-Lane Roundabout Intersection	
Double-Lane Roundabout Capacity	Computation
Pyay Road (1) approach capacity of a double-lane roundabout	1563 veh/h
Insein Road (1) approach capacity of a double-lane roundabout	1394 veh/h
Hledan Road approach capacity of a double-lane roundabout	1175 veh/h
Pyay Road (2) & Insein Road (2) approach capacity of a double-lane roundabout	1968 veh/h
University Avenue Road approach capacity of a double-lane roundabout	1275 veh/h
Pedestrian crossing volume	
Pedestrian volume (Pyay Road (1))	120 p/h
Pedestrian volume (Insein Road (1), Pyay Road (2) & Insein Road (2))	100 p/h
Pedestrian volume (Hledan Road, University Avenue Road)	80 p/h
Capacity Reduction Factor, M (From Figure-9)	
Capacity Reduction Factor, M (Pyay Road (1))	1.0
Capacity Reduction Factor, M (Insein Road (1))	1.0
Capacity Reduction Factor, M (Hledan Road)	1.0
Capacity Reduction Factor, M (Pyay Road (2) & Insein Road (2))	0.95
Capacity Reduction Factor, M (University Avenue Road)	1.0

Appendix Table F2 (Continued)

Double-Lane Roundabout Intersection	
Double-Lane Roundabout Capacity	Computation
Pedestrian effects on entry capacity	
Pyay Road (1) approach capacity of a double-lane roundabout	1563 x 1.0 = 1563 veh/h
Insein Road (1) approach capacity of a double-lane roundabout	1394 x 1.0 = 1394 veh/h
Hledan Road approach capacity of a double-lane roundabout	1175 x 1.0 = 1175 veh/h
Pyay Road (2) & Insein Road (2) approach capacity of a double-lane roundabout	1968 x 0.96 = 1889 veh/h
University Avenue Road approach capacity of a double-lane roundabout	1275 x 1.0 = 1275 veh/h

Appendix Table F3 Capacity and LOS Worksheet

Capacity, Control Delay, and LOS	Computation														
	Pyay Road (1)			Pyay Road (2) & Insein Road (2)			Insein Road (1)			Hledan Road			University Avenue Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjusted entry flow rate, v (veh/h)	104	819	175	142	1238	189	131	613	174	104	297	118	89	303	110
Entry flow rate, v_x (veh/h)		1098			1569			918			519			502	
Circulating flow rate, V_{CIRC}		1213			636			1457			1756			1615	
Double-lane roundabout capacity, $C_{m,x}$ (veh/h)		1563			1889			1394			1175			1275	
v/c ratio, X = v/c	0.702			0.831			0.659			0.442			0.394		
Duration of analysis period (h), T	0.250			0.250			0.250			0.250			0.250		
Delay by approach, d_x (Use Equation-7)	7.526			10.341			7.421			5.468			4.647		
LOS by approach	A			A			A			A			A		
Intersection delay, d_I (Use Equation-59) $d_I = \frac{\sum(d_x)(v_x)}{\sum(V_x)}$	7.918			Intersection LOS Table-5									A		

Appendix G

Tamwe Signalized Intersection

Appendix Table G1 Minimum pedestrian green time and adjustment factors for base saturation flow rate

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
1. Pedestrians / cycle	East Race Course Road, Ba Nyar Da La Road (1) & (2) = $80\frac{p}{h} x \frac{1h}{3600s} x$ 196 s = 4.356 p U Chit Maung Road = $60\frac{p}{h} x \frac{1h}{3600s} x$ 196 s = 3.267 p East Shwe Gone Daing Road, Tha main Ba Ran Road = $40\frac{p}{h} x \frac{1h}{3600s}$ x 196 s = 2.178 p
2. Minimum effective green time required for pedestrians (Use Equation-10).	$G_p = 3.2 + \frac{L}{Sp} + (0.27\ N_{ped})$ for $W_E \leq 10$ ft G_p (East Race Course Road, Ba Nyar Da La Road (1)&(2)) = $3.2+\frac{36}{4}$+(0.27x4.356) = 13.4 s G_p (U Chit Maung Road) = $3.2+\frac{36}{4}$+(0.27 x 3.267) = 13.1 s G_p (East Shwe Gone Daing Road, Tha main Ba Ran Road) = $3.2+\frac{48}{4}$+(0.27x2.178) = 15.8 s
3. Compare minimum effective green time required for pedestrians with actual effective green.	G_p (East Race Course Road, Ba Nyar Da La Road (1) & (2)) = 30.0 S, which is > 13.4 s G_p (U Chit Maung Road) = 45.0 S, which is > 13.1 s G_p (East Shwe Gone Daing Road, Tha main Ba Ran Road) = 45.0 S, which is > 15.8 s

Appendix Table G1 (Continued)

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
4. Proportion of left turns and right turns.	P_{LT} (East Race Course Road) = $\frac{124}{124+599}$ = 0.172, P_{LT} (Ba Nyar Da La Road (1)) = $\frac{131}{131+548}$ = 0.193, P_{LT} (East Shwe Gone Daing Road) = $\frac{91}{91+278}$ = 0.247, P_{LT} (Tha Main Ba Ran Road) = $\frac{85}{85+265}$ = 0.243, P_{LT} (U Chit Maung Road) = $\frac{137}{137+589`}$ = 0.189, P_{RT} (Ba Nyar Da La Road (2)) = $\frac{191}{191+697}$ = 0.215, P_{RT} (Ba Nyar Da La Road (1)) = 0, P_{RT} for exclusive RT Lane, P_{RT} = 1.000
5. Lane width adjustment factor (Use Equation-14).	$f_w = 1 + \frac{(W-12)}{30}$, f_w = 12 ft (for all Lanes), $f_w = 1 + \frac{(12-12)}{30}$ = 1.000
6. Heavy-vehicle adjustment factor (Use Equation15).	$f_{HV} = \frac{100}{100+\%HV(E_T-1)}$, % HV = 0, $f_{HV} = \frac{100}{100+0(2-1)}$ = 1.000

Appendix Table G1 (Continued)

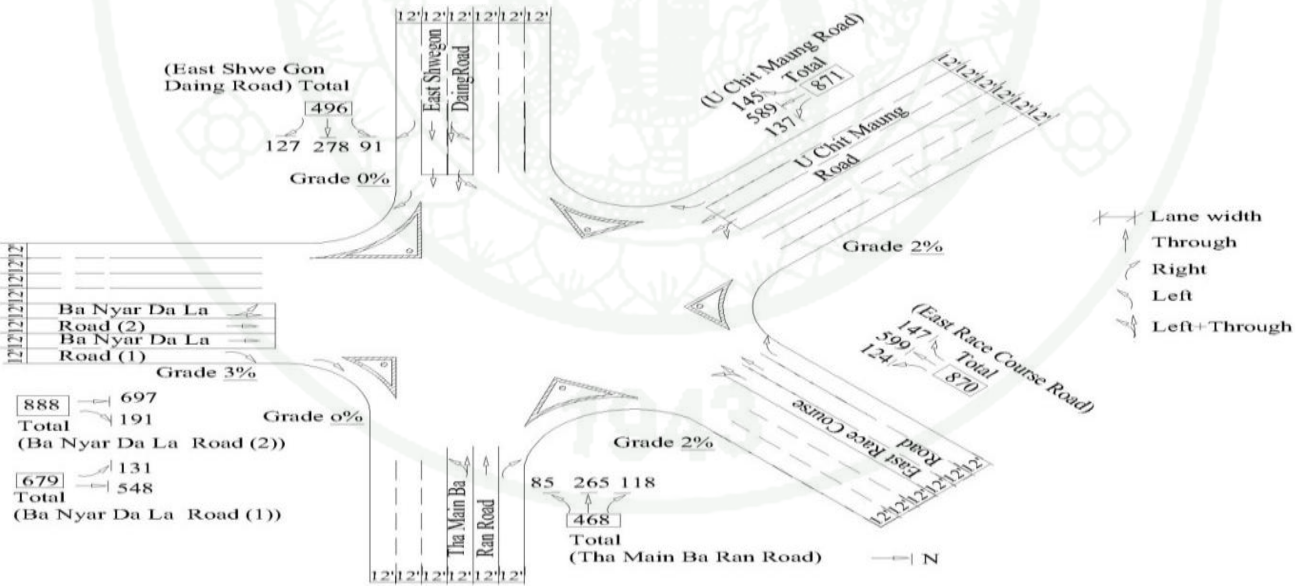
Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
7. Percent grade adjustment factor (Use Equation-16).	%G (Ba Nyar Da La Road (1) & (2)) = 3 %, $f_g = 1 - \frac{3}{200} = 0.985$ %G (East Race Course Road, U Chit Maung Road = 2 %, $f_g = 1 - \frac{2}{200} = 0.990$ %G (East Shwe Gone Daing Road, Tha Main Ba Ran Road) = 0 %, $f_g = 1.000$
8. Parking adjustment factor (Use Equation17)	$N_m = 0$ (No parking maneuvers), $f_p = \frac{N - 0.1 - \frac{18Nm}{3600}}{N} = 1.000$
9. Bus blockage adjustment factor (Use Equation-18).	$N_B = 0$ (No buses stopping), $f_{bb} = \frac{N - \frac{14.4N_B}{3600}}{N} = 1.000$
10. Area type adjustment factor (Use Equation-20).	For all other areas, $f_a = 1.000$
11. Lane utilization adjustment factor, (From Table-10).	Use $f_{LU} = 0.952$ for all through movements

Appendix Table G1 (Continued)

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
12. Left-turn adjustment factor	The Left-turn is permitted. A special Procedure is used. All approaches are opposed by multilane approaches. The supplemental worksheet for multilane approaches is used to determine the factor. The left turn is protected. Use Equation (25), $f_{LT} = \frac{1}{1+0.05xP_{LT}}$, f_{LT} (East Race Course Road) = $\frac{1}{1+0.05x0.172}$ = 0.991 f_{LT} (Ba Nyar Da La Road (1)) = $\frac{1}{1+0.05x0.193}$ = 0.990
13. Right-turn adjustment factor (Use Equation-21).	For share right turns lane, f_{RT} = 1.0 - (0.15) P_{RT} f_{RT} (Ba Nyar Da La Road (2)) = 1.0 - (0.15) x 0.215 = 0.968 For all exclusive right turn lane, f_{RT} = 0.850
14. Left-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.
15. Right-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.

Appendix Table G2 Input Worksheet

General Information		Site Information	
Analyst		Intersection	Tamwe Intersection
Agency or Company		Area Type	□ CBD ■ Other
Date performed	17.1.2011	Jurisdiction	
Analysis Time Period	5:00 – 6:00 P.M	Analysis Year	2011
Intersection Geometry			



Appendix Table G2 (Continued)

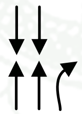
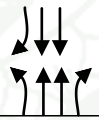
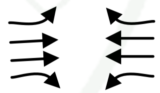
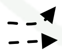
Volume and Timing Input	Site Information																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V(veh/h)	124	599	147	131	548	-	137	589	145		697	191	91	278	127	85	265	118
% heavy vehicles, %HV		0			0			0			0			0			0	
Peak-hour factor, PHF		0.96			0.95			0.94			0.98			0.95			0.94	
Pretimed (P) or Actuated (A)		P			P			P			P			P			P	
Start-up lost time, I_1 (s)		-			-			-			-			-			-	
Extension of effective green time, e (s)		-			-			-			-			-			-	

Appendix Table G2 (Continued)

Volume and Timing Input	Site Information																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Arrival type, AT		3			3			3			3			3			3	
Approach pedestrian volume, v_{ped} (p/h)		80			80			60			80			40			40	
Approach bicycle volume, v_{bic} (bicycle/h)		0			0			0			0			0			0	
Parking (Y or N)		N			N			N			N			N			N	
Parking maneuvers, N_m (maneuvers/h)		0			0			0			0			0			0	
Bus stopping, N_b (buses/h)		0			0			0			0			0			0	

Appendix Table G2 (Continued)

Volume and Timing Input	Site Information																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Min. timing for pedestrians, G_p(s)		13.4			13.4			13.1			13.4			15.8			15.8	

Signal Phasing Plan					
Diagram	01 	02 	03 	04 	
Timing	G = 15 Y = 4	G = 60 Y = 4	G = 60 Y = 4	G = 45 Y = 4	
 Protected turns	 Protected turns Pedestrian			Cycle Length, C = 196 s	

Appendix Table G3 Volume Adjustment and Saturation Flow Rate Worksheet

Volume Adjustment	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	124	599	147	131	548	-	137	589	145	-	697	191	91	278	127	85	265	118
Peak-hour factor, PHF		0.96			0.95			0.94			0.98			0.95			0.94	
Adjusted flow rate, v_P=V/PHF (veh/h)	129	624	153	138	577	-	146	627	154		711	195	96	293	134	90	282	126
Adjusted flow rate in lane group (veh/h)	129		777	138		577		927			906			523			498	
Proportion of LT or RT (P_{LT} or P_{RT})	.172		1.00	.193		-	.189		1.00	-		.215	.247		1.00	.243	1.00	

Appendix Table G3 (Continued)

Saturation Flow Rate	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Base saturation flow, s_o (pc/h/lane)	1900	1900	1900	1900	1900	1900		1900			1900			1900			1900	
Number of Lanes, N	1	1	2	1	1	1		3			2			3			3	
Lane width adjustment factor, f_w	1.00	1.00	1.00	1.00	1.00	1.00		1.00			1.00			1.00			1.00	
Heavy-vehicle adjustment factor, f_{HV}	1.00	1.00	1.00	1.00	1.00	1.00		1.00			1.00			1.00			1.00	

Appendix Table G3 (Continued)

Saturation Flow Rate	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Grade adjustment factor, f_g	.990	.990	.990	.985	.985	.985		.990			.985			1.00			1.00	
Parking adjustment factor, f_p	1.00	1.00	1.00	1.00	1.00	1.00		1.00			1.00			1.00			1.00	
Bus Blockage adjustment factor, f_{bb}	1.00	1.00	1.00	1.00	1.00	1.00		1.00			1.00			1.00			1.00	
Area type adjustment factor, f_a	1.00	1.00	1.00	1.00	1.00	1.00		1.00			1.00			1.00			1.00	

Appendix Table G3 (Continued)

Saturation Flow Rate	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Lane utilization adjustment factor, f_{LU}	1.00	1.00	.952	1.00	1.00	.952		.952			.952			.952			.952	
Left-turn adjustment factor, f_{LT}	.991	.636	1.00	.990	.586	1.00		.566			1.00			.689			.687	
Right-turn adjustment factor, f_{RT}	1.00	1.00	.850	1.00	1.00	1.00		.850			.968			.850			.850	
Left-turn ped/bike adjustment factor f_{Lpb}	1.00	.997	1.00	1.00	.998	1.00		.999			1.00			.996			.996	

Appendix Table G3 (Continued)

Saturation Flow Rate	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Right-turn ped/bike adjustment factor, f_{Rpb}	1.00	1.00	.921	1.00	1.00	1.00		.941			.987			.948			.948	
Adjustment saturation flow, s (veh/h) (Use Equation-13), $s = s_o N f_w f_{HV} f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$	1864	1193	2804	1853	1095	1782		2430			3404			3001			2992	

Appendix Table G4 Supplemental Worksheet for Permitted Left Turns Opposed by Multilane Approach

Permitted Left Adjustment Factor	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gone Daing Road	Tha main Ba Ran Road
Cycle Length, C (s)	196					
Total actual green time for LT Lane group, G (s)	79	79	60		45	45
Effective Permitted green time for LT lane group, G (s)	64	64	60		45	45
Opposing effective green time, g_o (s)	60	60	60		45	45
Number of Lanes in LT Lane group, N	2	2	2		2	2
Number of Lanes in opposing approach, No	4	3	4		3	3
Adjusted LT flow rate, v_{LT} (veh/h)	129	138	146		96	90
Proportion of LT volume in LT lane group, P_{LT}	0.172	0.193	0.189		0.247	0.243
Adjusted LT flow rate, opposing approach, v_o (veh/h)	715	753	906		372	389
Loft time for LT lane group, t_L	0	0	0		0	0
LT volume per cycle LTC = v_{LT} C/3600	7.023	7.513	7.949		5.227	4.900
Opposing lane utilization factor, f_{LUo}	0.952	0.952	0.952		0.952	0.952

Appendix Table G4 (Continued)

Permitted Left Adjustment Factor	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gone Daing Road	Tha main Ba Ran Road
Opposing flow per lane, per cycle , $v_{olc} = \frac{v_o C}{3600 N_o f_{LUo}}$ (veh/C/Ln)	10.223	14.355	12.953		7.092	7.416
$g_f = G\,[e^{-0.882(LTC^{0.717})}] - t_L$, $g_f < g$ (except for exclusive Left-turn lanes) (Use Equation-27)	2.229	1.868	1.215		2.508	2.858
Opposing Platoon ratio, R_{po} (From Table-12)	1.000	1.000	1.000		1.000	1.000
Opposing queue ratio, $qr_o = \max\,[1-R_{po}(g_o/C),0]$	0.694	0.694	0.694		0.770	0.770
$g_q = \frac{v_{olc} qr_o}{0.5-[v_{olc}(1-qr_o)/g_o]} - t_L$, $v_{olc}(1-qr_o)/g_o \leq 0.49$ (note case-specific parameters) (Use Equation-28)	15.841	23.343	20.716		11.775	12.357
$g_u = g - g_q$ if $g_q \geq g_f$, (Use Equation-29) or $g_u = g - g_f$ if $g_q < g_f$ (Use Equation-30)	48.159	40.657	39.284		33.225	32.643
E_{L1} (Refer to Table-13) and (Use Equation-31,32,33)	2.953	3.073	3.556		2.082	2.118

Appendix Table G4 (Continued)

Permitted Left Adjustment Factor	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gone Daing Road	Tha main Ba Ran Road
$P_L = P_{LT}\left[1+\frac{(N-1)g}{(g_f+\frac{g_u}{E_{L1}}+4.24)}\right]$ (except with multilane subject approach) (Use Equation-34)	0.665	0.832	0.876		0.737	0.729
$f_{min} = 2(1 + P_L)/g$ (Use Equation-36)	0.052	0.057	0.063		0.077	0.077
$f_m = [g_f/g] + [g_u/g]\left[1+\frac{1}{1+P_L(E_{L1}-1)}\right]$ ($f_{min} \le f_m \le 1.000$) (Use Equation-35)	0.362	0.262	0.222		0.467	0.463
$f_{LT} = [f_m + 0.91(N-1)]/N$ (except for permitted left turn) (Use Equation-37)	0.636	0.586	0.566		0.689	0.687

Appendix Table G5 Supplemental Worksheet for Pedestrian-Bicycle Effects on Permitted Left Turns and Right Turns

Pedestrian-Bicycle Adjustment Factor						
Permitted Left Turns	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
						
Effective pedestrian green time, g_p (s)	60	60	60		45	45
Conflicting Pedestrian Volume, v_{ped} (p/h)	80	80	60		40	40
$v_{pedg} = v_{ped} (C/g_p)$ ($v_{pedg} \leq 5000$) (Use Equation-38)	261	261	196		174	174
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \leq 1000$) (Use Equation-40) or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if ($1000 < v_{pedg} \leq 5000$) (Use Equation-41)	0.131	0.131	0.098		0.087	0.087
Opposing queue clearing green, g_q (s)	15.841	23.343	20.716		11.775	12.357
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p if $g_q \geq g_p$ then $f_{lpb} = 1.0$	0.264	0.389	0.345		0.262	0.275
$OCC_{pedu} = OCC_{pedg} [1 - 0.5 (g_q/g_p)]$ (Use Equation-42)	0.114	0.106	0.081		0.076	0.075

Appendix Table G5 (Continued)

Pedestrian-Bicycle Adjustment Factor						
Permitted Left Turns	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Opposing flow rate, v_o (veh/h)	715	753	906		372	389
$OCC_r = OCC_{pedu}\ [e^{-(5/3600)Vo}]$ (Use Equation-44)	0.042	0.037	0.023		0.045	0.044
Number of cross-street receiving lanes, N_{rec}	3	3	3		3	3
Number of turning lanes, N_{turn}	1	1	1		1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, (Use Equation-46) or $A_{pbT} = 1 - 0.6\ (OCC_r)$ if $N_{rec} > N_{turn}$ (Use Equation-47)	0.975	0.978	0.986		0.973	0.974
Proportion of left turn, P_{LT}	0.172	0.193	0.189		0.247	0.243
Proportion of left turns using protected phase, $P_{LTA} = (1-$ permitted phase $f_{LT})/0.95$ (Use Equation-48)	0.383	0.436	0.457		0.327	0.329
$f_{Lpb} = 1.0 - P_{LT}\ (1- A_{pbt})\ (1 - P_{LTA})$ (Use Equation-49)	0.997	0.998	0.999		0.996	0.996

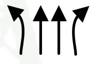
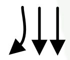
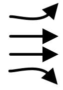
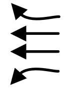
Appendix Table G5 (Continued)

Pedestrian-Bicycle Adjustment Factor						
Permitted Right Turns	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Effective pedestrian green time, g_p (s)	60		60	60	45	45
Conflicting Pedestrian Volume, v_{ped} (p/h)	80		60	60	40	40
Conflicting Pedestrian Volume, v_{bic} (bicycle/h)	0		0	0	0	0
$v_{pedg} = v_{pedg} (C/g_p)$	261		196	196	174	174
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \leq 1000$) or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if ($1000 < v_{pedg} \leq 5000$)	0.131		0.098	0.098	0.087	0.087
Effective green, g(s)	60		60	60	45	45
$v_{bicg} = v_{bic} (C/g)$ ($v_{bicg} \leq 1900$) (Use Equation-39)	0		0	0	0	0
$OCC_{bicg} = 0.02 + v_{bicg}/2700$	0	0	0	0	0	0

Appendix Table G5 (Continued)

Pedestrian-Bicycle Adjustment Factor						
Permitted Right Turns	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
						
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg})$ (Use Equation-45)	0.131		0.098	0.098	0.087	0.087
Number of cross-street receiving lanes, N_{rec}	3		3	3	3	3
Number of turning lanes, N_{turn}	1		1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, $A_{pbt} = 1 - 0.6\,(OCC_r)$ if $N_{rec} > N_{turn}$	0.921		0.941	0.941	0.948	0.948
Proportion of left turn, P_{RT}	1.000		1.000	0.215	1.000	1.000
Proportion of right turns using protected phase, P_{RTA}	0		0	0	0	0
$f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$ (Use Equation-50)	0.921		0.941	0.987	0.948	0.948

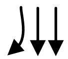
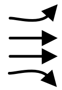
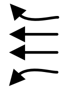
Appendix Table G6 Capacity and LOS Worksheet

Capacity Analysis	Computation									
Phase Number	1	1	2	2	2	2	3	3	3	4
Phase Type										
Lane group										
Adjusted flow rate, v (veh/h)	129	138	0	777	0	577	927	906	523	498
Saturation flow rate, s (veh/h)	1864	1853	1193	2804	1095	1782	2430	3404	3001	2992
Lost time, t_L (s), $t_L = I_1 - Y - e$ (Use Equation-8)	4	4	4	4	4	4	4	4	4	4
Effective green time, g(s), $g = G + Y - t_L$ (Use Equation-26)	15	15	64	60	64	60	60	60	45	45
Green ratio, g/C	0.077	0.077	0.327	0.306	0.327	0.306	0.306	0.306	0.230	0.230
Lane group capacity, c = s (g/C) (veh/h) (Use Equation-51)	144	143	390	858	358	545	744	1042	690	688
v/c ratio, X (Use Equation-52)	0.896	0.965	0	0.906	0	1.059	1.246	0.869	0.758	0.724
Flow ratio, v/s		0.074				0.324	0.381		0.174	

Appendix Table G6 (Continued)

Capacity Analysis	Computation							
Critical lane group/phase (✓)		✓			✓	✓	✓	
Sum of flow ratio for critical lane group, $Y_c = \sum$ (Critical lane groups v/s)	0.953							
Total Lost time per cycle, L (s) (Use Equation-9)	16.0							
Critical flow rate to capacity ratio, $X_c = (Y_c)(C)/(C-L)$ (Use Equation-53)	1.038							
Lane Group Capacity, Control Delay, and LOS	Computation							
	East Race Course Road		Ba Nyar Da La Road (2)		U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Lane group								
Adjusted flow rate, v (veh/h)	129	777	138	577	927	906	523	498

Appendix Table G6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation							
	East Race Course Road		Ba Nyar Da La Road (2)		U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Lane group								
Lane group capacity, c(veh/h)	534	858	501	545	744	1042	690	688
v/c ratio, X = v/c	0.242	0.906	0.275	1.059	1.246	0.869	0.758	0.724
Total green ratio, g/C	0.404	0.306	0.404	0.306	0.306	0.306	0.230	0.230
Uniform delay, d_1 (Use Equation-56) $d_1 = \frac{0.5C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)	38.583	65.305	39.162	68.012	68.012	64.298	70.373	69.713
Incremental delay calibration, k	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
Upstream filtering / metering adjustment factor, I	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Duration of analysis period (h),T	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250

Appendix Table G6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation							
	East Race Course Road		Ba Nyar Da La Road (2)		U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Lane group	↰	↑↑↱	↲	↓↓	↑↑↱	↲↓↓	⇉	⇇
Incremental delay, d_2, (Use Equation-57) $d_2 = 900T\,(X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}$ (s/veh)	1.073	14.942	1.357	55.110	121.834	9.823	7.636	6.521
Initial queue delay, d_3 (s/veh)	0	0	0	0	0	0	0	0
Progression adjustment factor, PF (From Table-13)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Delay, $d=d_1$ (PF) + d_2 + d_3 (s/veh) (Use Equation-54)	39.7	80.2	40.5	123.1	189.8	74.1	78.0	76.2
LOS by lane group	D	F	D	F	F	E	F	F
Delay by approach, d_A (Use Equation-58) $d_A = \frac{\Sigma(d)(v)}{\Sigma\,v}$ (s/veh)	74.4		107.2		189.8	74.1	78.0	76.2

Appendix Table G6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation					
	East Race Course Road	Ba Nyar Da La Road (2)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
LOS by approach	E	F	F	E	E	E
Approach flow rate, v_A (veh/h)	906	715	927	906	523	498
Intersection delay, d_I (Use Equation-59)	104.1	Intersection LOS Table-14			F	

Appendix H

Tamwe Channelized Intersection with Signal Control

Appendix Table H1 Minimum pedestrian green time and adjustment factors for base saturation flow rate

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
1. Pedestrians / cycle	East Race Course Road , Ba Nyar Da La Road (1) & (2) = $80\frac{p}{h} x \frac{1h}{3600s}$ x 196 s = 4.356 p U Chit Maung Road = $60\frac{p}{h} x \frac{1h}{3600s}$ x 196 s = 3.267 p East Shwe Gone Daing Road, Tha main Ba Ran Road, $40\frac{p}{h} x \frac{1h}{3600s}$ x 196 s = 2.178 p
2. Minimum effective green time required for pedestrians (Use Equatio-10).	$G_p = 3.2 + \frac{L}{Sp} + (0.27\ N_{ped})$ for $W_E \leq 10$ ft G_p (East Race Course Road, Ba Nyar Da La Road (1) & (2)) = $3.2 + \frac{48}{4} + (0.27 \times 4.356)$ = 16.4 s G_p (U Chit Maung Road) = $3.2 + \frac{48}{4} + (0.27 \times 3.267)$ = 16.1 s G_p (East Shwe Gone Daing Road, Tha Main Ba Ran Road) = $3.2 + \frac{72}{4} + (0.27 \times 2.178)$ = 21.8 s
3. Compare minimum effective green time required for pedestrians with actual effective green.	G_p (East Race Course Road, Ba Nyar Da La Road (1) & (2)) = 30 s, which is > 16.4 s G_p (U Chit Maung Road) = 45 s, which is > 16.1 s G_p (East Shwe Gone Daing Road, Tha main Ba Ran Road) = 45 s, which is > 21.8 s

Appendix Table H1 (Continued)

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
4. Proportion of left turns and right turns.	Proportions of left-and right-turn traffic are found by dividing the appropriate turning flow rates by the total lane group flow rate. P_{LT} for exclusive LT Lane is 1.000. P_{RT} for exclusive RT Lane is 1.000.
5. Lane width adjustment factor (Use Equation-14).	$f_w = 1 + \frac{(W-12)}{30}$, f_w = 12 ft (for all Lanes), $f_w = 1 + \frac{(12-12)}{30} = 1.000$
6. Heavy-vehicle adjustment factor (Use Equation-15).	$f_{HV} = \frac{100}{100+\%HV(E_T-1)}$, % HV = 0, $f_{HV} = \frac{100}{100+0(2-1)} = 1.000$
7. Percent grade adjustment factor (Use Equation-16).	%G (Ba Nyar Da La Road (1) and Ba Nyar Da La Road (2)) = 3 %, $f_g = 1 - \frac{3}{200} =$ 0.985 %G (East Race Course Road, U Chit Maung Road) = 2 %, $f_g = 1 - \frac{2}{200} = 0.990$ %G (East Shwe Gone Daing Road, Tha main Ba Ran Road) = 0 %, f_g = 1.000
8. Parking adjustment factor (Use Equation-17).	N_m = 0 (No buses stopping), $f_p = \frac{N - 0.1 - \frac{18Nm}{3600}}{N} = 1.00$

Appendix Table H1 (Continued)

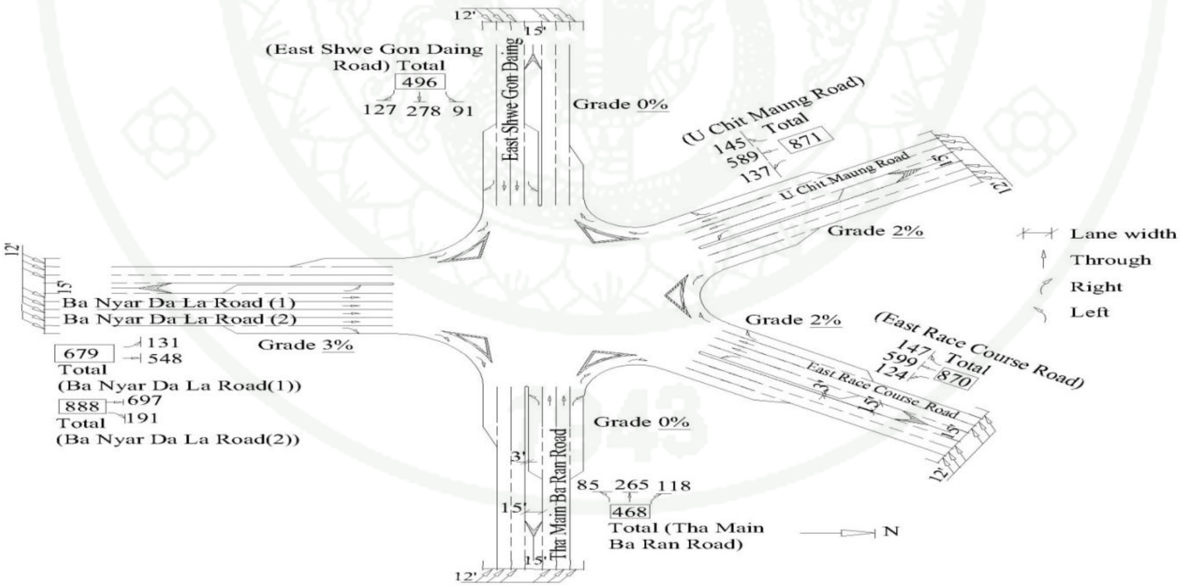
Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
9. Bus blockage adjustment factor (Use Equation-18)	N_B = 0 (No buses Stopping), $f_{bb} = \dfrac{N - \dfrac{14.4N_B}{3600}}{N} = 1.00$
10. Area type adjustment factor (Use Equation-20).	For all other areas, f_a = 1.000
11. Lane utilization adjustment factor (From Table-10).	Use f_{LU} = 0.952 for all through movements.
12. Left-turn adjustment factor	The Left-turn is permitted. A special Procedure is used. All approaches are opposed by multilane approaches. The supplemental worksheet for multilane approaches is used to determine the factor. The left turn is protected Use Equation (25). For exclusive left-turn Lane, f_{LT} = 0.950
13. Right-turn adjustment factor (Use Equation-21).	For all exclusive – lane approaches, f_{RT} = 0.85

Appendix Table H1 (Continued)

Minimum pedestrian green time and adjustment factors for base saturation flow rate	Computation
14. Left-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.
15. Right-turn pedestrian/bicycle adjustment factor.	Supplemental worksheet for pedestrian-bicycle effect is used to determine the factor.

Appendix Table H2 Input Worksheet

General Information		Site Information	
Analyst		Intersection	Tamwe Intersection
Agency or Company		Area Type	□ CBD ■ Other
Date performed	17.1.2011	Jurisdiction	
Analysis Time Period	5:00 – 6:00 P.M	Analysis Year	2011
Intersection Geometry			



Appendix Table H2 (Continued)

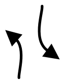
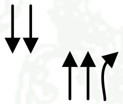
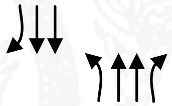
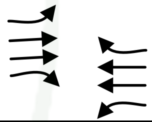
Volume and Timing Input	Site Information																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	124	599	147	131	548	-	137	589	145	-	697	191	91	278	127	85	265	118
% heavy vehicles, %HV		0			0			0			0			0			0	
Peak-hour factor, PHF		0.97			0.96			0.96			0.98			0.97			0.98	
Pretimed (P) or actuated (A)		P			P			P			P			P			P	
Start-up lost time, I_1 (s)		-			-			-			-			-			-	
Extension of effective green time, e (s)		-			-			-			-			-			-	

Appendix Table H2 (Continued)

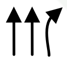
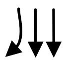
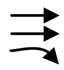
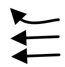
Volume and Timing Input	Site Information																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Arrival type, AT		3			3			3			3			3			3	
Approach pedestrian volume, v_{ped} (p/h)		80			80			60			80			40			40	
Approach bicycle volume, v_{bic} (bicycle/h)		0			0			0			0			0			0	
Parking (Y or N)		N			N			N			N			N			N	
Parking maneuvers, N_m (maneuvers/h)		0			0			0			0			0			0	
Bus stopping, N_b (buses/h)		0			0			0			0			0			0	
Min. timing for pedestrians, G_p (s)		16.4			16.4			16.1			16.4			21.8			21.8	

Appendix Table H2 (Continued)

Volume and Timing Input	Site Information					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gone Daing Road	Tha Main Ba Ran Road

Signal Phasing Plan				
Diagram	01 	02 	03 	04 
Timing	G = 15 Y = 4	G = 60 Y = 4	G = 60 Y = 4	G = 45 Y = 4
Protected turns	Protected turns Pedestrian			Cycle Length, C = 196 s

Appendix Table H3 Volume Adjustment and Saturation Flow Rate Worksheet

Volume Adjustment	Computation																
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	124	599	147	131	548	-	137	589	145	-	697	191	91	278	127	85	265	118
Peak-hour factor, PHF		0.96			0.95			0.94			0.98			0.95			0.94	
Adjusted flow rate, v_P = V/PHF (veh/h)	129	624	153	138	577	-	146	627	154		711	195	96	293	134	90	282	126
Lane group																		
Adjusted flow rate in lane group (veh/h)	129		777	138		577	146	781			906		96	427		90	408	

Appendix Table H3 (Continued)

Volume Adjustment	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Proportion of LT or RT (P_{LT} or P_{RT})	1.00		1.00	1.00		1.00	1.00	1.00			1.00		1.00	1.00		1.00	1.00	
Saturation Flow Rate	Computation																	
Base saturation flow, s_o (pc/h/lane)	1900	1900	1900	1900	1900	1900	1900	1900			1900		1900	1900		1900	1900	
Number of Lanes, N	1	1	3	1	1	2	1	3			3		1	3		1	3	

Appendix Table H3 (Continued)

Saturation Flow Rate	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Lane width adjustment factor, f_w	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00			1.00		1.00	1.00		1.00	1.00	
Heavy-vehicle adjustment factor, f_{HV}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00			1.00		1.00	1.00		1.00	1.00	
Grade Adjustment factor, f_g	.990	.990	.990	.985	.985	.985	.990	.990			.985		1.00	1.00		1.00	1.00	
Parking adjustment factor, f_p	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00			1.00		1.00	1.00		1.00	1.00	

Appendix Table H3 (Continued)

Saturation Flow Rate	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Bus Blockage adjustment factor, f_{bb}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00			1.00		1.00	1.00		1.00	1.00	
Area type adjustment factor, f_a	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00			1.00		1.00	1.00		1.00	1.00	
Lane utilization adjustment factor, f_{LU}	1.00	1.00	.952	1.00	1.00	.952	1.00	.952			.952		1.00	.952		1.00	.952	
Left-turn adjustment factor, f_{LT}	.950	.348	1.00	.950	.290	1.00	.276	1.00			1.00		.462	1.00		.452	1.00	

Appendix Table H3 (Continued)

Saturation Flow Rate	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Right-turn adjustment factor, f_{RT}	1.00	1.00	.850	1.00	1.00	1.00	1.00	.850			.850		1.00	.850		1.00	.850	
Left-turn ped/bike adjustment factor, f_{Lpb}	1.00	.990	1.00	1.00	.993	1.00	.995	1.00			1.00		.986	1.00		.987	1.00	
Right-turn ped/bike adjustment factor, f_{Rpb}	1.00	1.00	.921	1.00	1.00	1.00	1.00	.941			.921		1.00	.948		1.00	.948	

Appendix Table H3 (Continued)

Saturation Flow Rate	Computation																	
	East Race Course Road			Ba Nyar Da La Road (1)			U Chit Maung Road			Ba Nyar Da La Road (2)			East Shwe Gone Daing Road			Tha main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjustment saturation flow, s (veh/h), (Use Equation-13) $s = s_o N f_w f_{HV} f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$	1787	648	4206	1778	539	3563	517	4297			4184		866	4373		848	4373	

Appendix Table H4 Supplemental Worksheet for Permitted Left Turns Opposed by Multilane Approach

Permitted Left Turns Adjustment Factor	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gone Daing Road	Tha Main Ba Ran Road
Cycle Length, C (s)	196					
Total actual green time for LT Lane group, G (s)	79	79	60		45	45
Effective Permitted green time for LT lane group, G (s)	64	64	60		45	45
Opposing effective green time, g_o (s)	60	60	60		45	45
Number of Lanes in LT Lane group, N	1	1	1		1	1
Number of Lanes in opposing approach, N_o	4	3	4		3	3
Adjusted LT flow rate, v_{LT} (veh/h)	129	138	146		96	90
Proportion of LT volume in LT lane group, P_{LT}	1.000	1.000	1.000		1.000	1.000
Adjusted LT flow rate for opposing approach, v_o (veh/h)	577	624	711		282	293
Lost time for LT lane group, t_L	0	0	0		0	0
LT volume per cycle LTC = v_{LT} C/3600	7.023	7.513	7.949		5.227	4.900
Opposing lane utilization factor, f_{LUo}	0.952	0.952	0.952		0.952	0.952

Appendix Table H4 (Continued)

Permitted Left Turns Adjustment Factor	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gone Daing Road	Tha Main Ba Ran Road
Opposing flow per lane per cycle, $v_{olc} = \dfrac{v_o C}{3600 N_o f_{LUo}}$ (veh/C/Ln)	8.250	11.895	10.165		5..376	5.586
$g_f = G\,[e^{-0.882(LTC^{0.717})}] - t_L$, $g_f < g$ (except for exclusive Left-turn lanes) (Use Equation-27)	0	0	0		0	0
Opposing Platoon ratio, R_{po} (From Table-11)	1.000	1.000	1.000		1.000	1.000
Opposing queue ratio, $qr_o = \max\,[1 - R_{po}\,(g_o/C),0]$	0.694	0.694	0.694		0.770	0.770
$g_q = \dfrac{v_{olc} qr_o}{0.5 - [v_{olc}(1-qr_o)/g_o} = t_L$, $v_{olc}\,(1-qr_o)/g_o \leq 0.49$ (note case-specific parameters) (Use Equation-28)	12.503	18.790	15.741		8.760	9.123
$g_u = g - g_q$ if $g_q \geq g_f$, (Use Equation-29) or $g_u = g - g_f$ if $g_q < g_f$ (Use Equation-30)	51.497	45.210	44.259		36.240	35.877
E_{L1} (Refer to Table) and (Use Equation-31,32,33)	2.315	2.438	2.668		1.744	1.762

Appendix Table H4 (Continued)

Permitted Left Turns Adjustment Factor	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gone Daing Road	Tha Main Ba Ran Road
$P_L = P_{LT}\left[1+\frac{(N-1)g}{(g_f+\frac{g_u}{E_{L1}}+4.24)}\right]$ (except with multilane subject approach) (Use Equation-34)	1.000	1.000	1.000		1.000	1.000
$f_{min} = 2\,(1 + P_L)/g$ (Use Equation-36)	0.063	0.063	0.067		0.089	0.089
$f_m = [g_f/g] + [g_u/g]\left[1+\frac{1}{1+P_L(E_{L1}-1)}\right]$ ($f_{min} \leq f_m \leq 1.000$) (Use Equation-35)	0.348	0.290	0.276		0.462	0.452
$f_{LT} = [f_m + 0.91\,(N-1)]/N$ (except for permitted left turn) (Use Equation-37)	0.348	0.290	0.276		0.462	0.452

Appendix Table H5 Supplemental Worksheet for Pedestrian – Bicycle Effects on Permitted Left Turns and Right Turns

Pedestrian – Bicycle Adjustment Factor						
Permitted Left Turns	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
						
Effective pedestrian green time, g_p (s)	60	60	60		45	45
Conflicting Pedestrian Volume, Vped (p/h)	80	80	60		40	40
$v_{pedg} = v_{ped}$ (C/g_p) ($v_{pedg} \leq 5000$) (Use Equation-38)	261	261	196		174	174
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \leq 1000$) (Use Equation-40) $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if ($1000 < v_{pedg} \leq 5000$) (Use Equation-41)	0.131	0.131	0.098		0.087	0.087
Opposing queue clearing green, g_q (s)	12.503	18.790	15.741		8.760	9.123
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p if $g_q \geq g_p$ then $f_{Lpb} = 1.0$	0.208	0.313	0.262		0.195	0.203
$OCC_{pedu} = OCC_{pedg}$ [1-0.5 (g_q/g_p)] (Use Equation-42)	0.117	0.110	0.085		0.079	0.078

Appendix Table H5 (Continued)

Pedestrian – Bicycle Adjustment Factor						
Permitted Left Turns	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Opposing flow rate, v_o (veh/h)	577	624	711		282	293
$OCC_r = OCC_{pedu}\ [e^{-(5/3600)Vo}]$ (Use Equation-44)	0.052	0.046	0.032		0.053	0.052
Number of cross-street receiving lanes, N_{rec}	3	3	3		3	3
Number of turning lanes, N_{turn}	1	1	1		1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, (Use Equation-46) or $A_{pbT} = 1 - 0.6\ (OCC_r)$ if $N_{rec} > N_{turn}$ (Use Equation-47)	0.969	0.972	0.981		0.968	0.969
Proportion of left turn, P_{LT}	1.000	1.000	1.000		1.000	1.000
Proportion of left turns using protected phase, $P_{LTA} = (1 - \text{permitted phase } f_{LT})/0.95$ (Use Equation-48)	0.686	0.747	0.762		0.566	0.577
$f_{Lpb} = 1.0 - P_{LT}\ (1 - A_{pbT})\ (1 - P_{LTA})$ (Use Equation-49)	0.990	0.993	0.995		0.986	0.987

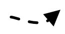
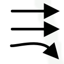
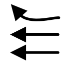
Appendix Table H5 (Continued)

Pedestrian – Bicycle Adjustment Factor						
Permitted Right Turns	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
						
Effective pedestrian green time, g_p (s)	60		60	60	45	45
Conflicting Pedestrian Volume, v_{ped} (p/h)	80		60	80	40	40
Conflicting Pedestrian Volume, v_{bic} (bicycle/h)	0		0	0	0	0
$v_{pedg} = v_{pedg} (C/g_p)$	261		196	261	174	174
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \leq 1000$) or $OCC_{pedg} = 0.4 + v_{pedg}/1000$ if ($1000 < v_{pedg} \leq 5000$)	0.131		0.098	0.131	0.087	0.087
Effective green, g (s)	60		60	60	45	45
$v_{bicg} = v_{bic} (C/g)$ ($v_{bicg} \leq 1900$) (Use Equation-39)	0		0	0	0	0
$OCC_{bicg} = 0.02 + v_{bicg} / 2700$	0		0	0	0	0
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg}) (OCC_{bicg})$ (Use Equation-45)	0.131		0.098	0.131	0.087	0.087

Appendix Table H5 (Continued)

Pedestrian – Bicycle Adjustment Factor						
Permitted Right Turns	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
						
Number of cross - street receiving lanes, N_{rec}	3		3	3	3	3
Number of turning lanes, N_{turn}	1		1	1	1	1
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$, $A_{pbT} = 1 - 0.6\ (OCC_r)$ if $N_{rec} > N_{turn}$	0.921		0.941	0.921	0.948	0.948
Proportion of left turn, P_{RT}	1.000		1.000	1.000	1.000	1.000
Proportion of right turns using protected phase, P_{RTA}	0		0	0	0	0
$f_{Rpb} = 1.0 - P_{RT}\ (1 - A_{pbT})\ (1 - P_{RTA})$ (Use Equation-50)	0.921		0.941	0.921	0.948	0.948

Appendix Table H6 Capacity and LOS Worksheet

Capacity Analysis	Computation												
Phase Number	1	1	2	2	2	2	3	3	3	4	4	4	4
Phase Type													
Lane group													
Adjusted flow rate, v (veh/h)	129	138	0	777	0	577	146	781	906	96	427	90	408
Saturation flow rate, s (veh/h)	1787	1778	648	4206	539	3563	517	4297	4184	866	4373	848	4373
Lost time, t_L (s), $t_L = I_1 - Y - e$ (Use Equation-8)	4	4	4	4	4	4	4	4	4	4	4	4	4
Effective green time, g (s), $g = G + Y - t_L$	15	15	64	60	64	60	60	60	60	45	45	45	45
Green ratio, g/C	0.077	0.077	0.327	0.306	0.327	0.306	0.306	0.306	0.306	0.230	0.230	0.230	0.230
Lane group capacity, c = s (g/C) (veh/h) (Use Equation-51)	138	137	212	1287	176	1090	158	1315	1280	199	1006	195	1006

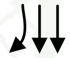
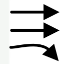
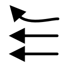
Appendix Table H6 (Continued)

Capacity Analysis	Computation												
v/c ratio, (Use Equation-52)	0.935	1.007	0	0.604	0	0.529	0.924	0.594	0.708	0.482	0.424	0.462	0.406
Flow ratio, v/s		0.078		0.185			0.282			0.111			
Critical lane group/phase (✓)		✓		✓			✓			✓			
Sum of flow ratio for critical lane groups, $Y_c = \sum$ (Critical lane groups, v/s)													
Total Lost time per cycle, L (s) (Use Equation-9)	16												
Critical flow rate to capacity ratio, $X_c = (Y_c)(C)/(C-L)$ (Use Equation-53)	0.714												

Appendix Table H6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation										
	East Race Course Road		Ba Nyar Da La Road (1)		U Chit Maung Road		Ba Nyar Da La Road (2)	East Shwe Gon Daing Road		Tha Main Ba Ran Road	
Lane group											
Adjusted flow rate, v (veh/h)	129	777	138	577	146	781	906	96	427	90	408
Lane group capacity, c (veh/h)	350	1287	313	1090	158	1315	1280	199	1006	195	1006
v/c ratio, X = v/c	0.369	0.604	0.441	0.529	0.924	0.594	0.708	0.482	0.424	0.462	0.406
Total green ratio, g/C	0.404	0.306	0.404	0.306	0.306	0.306	0.306	0.230	0.230	0.230	0.230
Uniform delay, d_1 (Use Equation-56) $d_1 = \frac{0.5C[1-(g/C)]^2}{1-[\min(1,X)g/C]}$ (s/veh)	40.91	57.90	42.36	56.32	65.81	57.69	60.25	65.35	64.38	65.01	64.09
Incremental delay calibration, k	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500
Upstream filtering / metering adjustment factor, I	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000

Appendix Table H6 (Continued)

Lane Group Capacity, Control Delay, and LOS	Computation										
	East Race Course Road		Ba Nyar Da La Road (1)		U Chit Maung Road		Ba Nyar Da La Road (2)	East Shwe Gon Daing Road		Tha Main Ba Ran Road	
Lane group											
Duration of analysis period (h),T	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250
Incremental delay, d_2 (Use Equation-57) $d_2 = 900T\,(X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}$ (s/veh)	2.98	2.11	4.46	1.84	53.82	1.98	3.33	8.13	1.31	7.68	1.22
Initial queue delay, d_3 (s/veh)	0	0	0	0	0	0	0	0	0	0	0
Progression adjustment factor, PF (From Table-13)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Delay, $d = d_1\,(PF) + d_2 + d_3$ (s/veh) (Use Equation-54)	43.9	60.0	46.8	58.2	119.6	59.7	63.6	73.5	65.7	72.7	63.3
LOS by lane group	D	E	D	E	F	E	E	F	E	F	E

Appendix Table H6 (Continued)

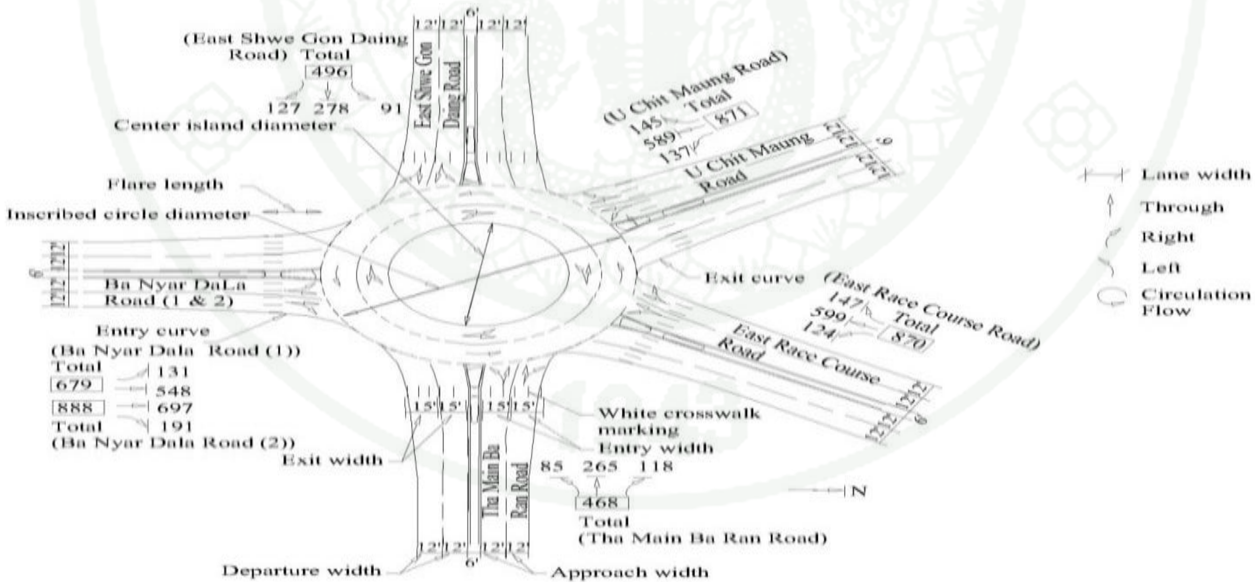
Lane Group Capacity, Control Delay, and LOS	Computation					
	East Race Course Road	Ba Nyar Da La Road (1)	U Chit Maung Road	Ba Nyar Da La Road (2)	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Approach delay, d_A (Use Equation-58), $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	57.7	56.0	69.1	63.6	67.1	65.0
LOS by approach	E	E	E	E	E	E
Approach flow rate, v_A (veh/h)	906	715	927	906	523	498
Intersection delay, d_I (Use Equation-59) $d_I = \frac{\sum(d_A)(v_A)}{\sum(v_A)}$ (s/veh)	62.9	Intersection LOS Table-14			E	

Appendix I

Tamwe Roundabout Intersection

Appendix Table I1 General data and geometric design elements of Tamwe roundabout intersection

General Information		Site Information	
Analyst		Intersection	Tamwe Intersection
Agency or Company		Area Type	□ CBD ■ Other
Date performed	17.1.2011	Jurisdiction	
Analysis Time Period	5:00 - 6:00 P.M	Analysis Year	2011
Intersection Geometry			



Appendix Table I1 (Continued)

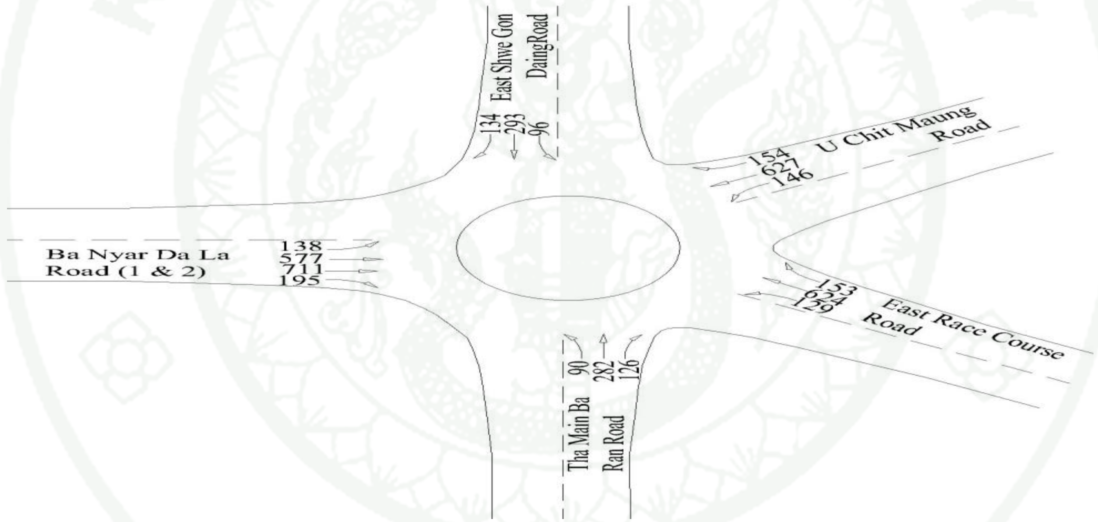
Entry Flow Rate and Geometric Design Elements	Computation														
	East Race Course Road			Ba Nyar Da La Road (1) & (2)			U Chit Maung Road			East Shwe Gon Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Entry flow rate, v (veh/h)	124	599	147	131	1245	191	137	589	145	91	278	127	85	265	118
Adjusted entry flow rate, v (veh/h)	129	624	153	138	1288	195	146	627	154	96	293	134	90	282	126
Entry width (ft)	30			30			30			30			30		
Approach width (ft)	12			12			12			12			12		
Circulatory roadway width (ft) (From Table-8)	32			32			32			32			32		
Exit width (ft)	30			30			30			30			30		
Departure width (ft)	12			12			12			12			12		
Design Speed (mph) (From Table-6)	25			25			25			25			25		
Design Vehicle (FromTable-7)	WB-15 (WB-50)			WB-15 (WB-50)			WB-15 (WB-50)			WB-15 (WB-50)			WB-15 (WB-50)		
Entry radius (ft)	75			75			75			75			75		

Appendix Table I1 (Continued)

Entry Flow Rate and Geometric Design Elements	Computation				
	East Race Course Road	Ba Nyar Da La Road (1) & (2)	U Chit Maung Road	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Exit radius (ft)	150	150	150	150	150
Inscribed circle diameter (ft) (From Table-8)	150	150	150	150	150
Center island diameter (ft) (From Table-8)	86	86	86	86	86
Flare Length (ft)	80	80	80	80	80
Splitter island width (ft)	6	6	6	6	6
Total length of splitter islands (ft)	100	100	100	100	100
Stopping sight distance, d (ft)	152.7	152.7	152.7	152.7	152.7

Appendix Table I2 Tamwe Roundabout Intersection Calculation Worksheet

Double-Lane Roundabout Intersection	
Circulation Flow	Computation



East Race Course Road Circulation Flow	$V_{\text{East Race Course Road}}$ CIRC = 1183 veh/h
U Chit Maung Road Circulation Flow	$V_{\text{U Chit Maung Road}}$ CIRC = 1263 veh/h
East Shwe Daing Road Circulation Flow	$V_{\text{East Shwe Gon Daing Road}}$ CIRC = 1616 veh/h
Ba Nyar Da La Road (1) & Road (2) Circulation Flow	$V_{\text{Ba Nyar Da La Road (1) \& Road (2)}}$ = 664 veh/h
Tha Main Ba Ran Road Circulation Flow	$V_{\text{Tha Main Ba Ran Road}}$ = 1668 veh/h

Appendix Table I2 (Continued)

Double-Lane Roundabout Intersection	
Double-Lane Roundabout Capacity	Computation
East Race Course Road approach capacity of a double-lane roundabout	1573 veh/h
U Chit Maung Road approach capacity of a double-lane roundabout	1515 veh/h
East Shwe Gon Daing Road approach capacity of a double-lane roundabout	1261 veh/h
Ba Nyar Da La Road (1) & (2) approach capacity of a double-lane roundabout	1948 veh/h
Tha Main Ba Ran Road approach capacity of a double-lane roundabout	1224 veh/h
Pedestrian crossing volume	
Pedestrian volume (East Race Course Road, Ba Nyar Da La Road (1) & (2))	80 p/h
Pedestrian volume (U Chit Maung Road)	60 p/h
Pedestrian volume (East Shwe Gon Daing Road, Tha Main Ba Ran Road)	40 p/h
Capacity Reduction Factor, M (From Figure-9)	
Capacity Reduction Factor, M (East Race Course Road)	1.0
Capacity Reduction Factor, M (Ba Nyar Da La Road (1) & Road (2)	0.965
Capacity Reduction Factor, M (U Chit Maung Road)	1.0
Capacity Reduction Factor, M (East Shwe Gon Daing Road)	1.0
Capacity Reduction Factor, M (Tha Main Ba Ran Road)	1.0

Appendix Table I2 (Continued)

Double-Lane Roundabout Intersection	
Double-Lane Roundabout Capacity	Computation
Pedestrian effects on entry capacity	
East Race Course Road approach capacity of a double-lane roundabout	1573 x 1.0 = 1573 veh/h
U Chit Maung Road approach capacity of a double-lane roundabout	1515 x 1.0 = 1515 veh/h
East Shwe Gon Daing Road approach capacity of a double-lane roundabout	1261 x 1.0 = 1261 veh/h
Ba Nyar Da La Road (1) & Road (2) approach capacity of a double-lane roundabout	1948 x 0.965 = 1880 veh/h
Tha Main Ba Ran Road approach capacity of a double-lane roundabout	1224 x 1.0 = 1224 veh/h

Appendix Table I3 Capacity and LOS Worksheet

Capacity, Control Delay, and LOS	Computation														
	East Race Course Road			Ba Nyar Da La Road (1) & Road (2)			U Chit Maung Road			East Shwe Gon Daing Road			Tha Main Ba Ran Road		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjusted entry flow rate, v (veh/h)	129	624	153	138	1288	195	146	627	154	96	293	134	90	282	126
Entry flow rate, v_x (veh/h)		870			1567			871			496			468	
Circulatory flow rate, v_{CIRC} (veh/h)		1183			664			1263			1616			1668	
Double-lane roundabout capacity, $C_{m,x}$ (veh/h))		1573			1880			1515			1261			1224	
v/c ratio, X= v/c		0.576			0.862			0.612			0.415			0.407	
Duration of analysis period (h), T		0.250			0.250			0.250			0.250			0.250	
Delay by approach, d_x (Use Equation-7)		5.349			12.179			6.047			4.865			4.945	
LOS by approach		A			B			A			A			A	

Appendix Table I3 (Continued)

Capacity, Control Delay, and LOS	Computation				
	East Race Course Road	Ba Nyar Da La Road (1) & (2)	U Chit Maung Road	East Shwe Gon Daing Road	Tha Main Ba Ran Road
Intersection delay, d_I (Use Equation-59) $d_I = \frac{\sum(d_x)(V_x)}{\sum(V_x)}$	7.866	Intersection LOS (Table-5)			A

CURRICULUM VITAE

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