Evaluating Three Alternative Designs for Reducing Delay at an Intersection in Dohuk, Iraq

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ABSTRACT

Traffic delay is one of the most important factors used in intersection analysis. This study focused on the average delay by looking at different alternative junction models. These different models analyzed with and without signalization according to cycle length in order to help create alternate designs. Green intervals were optimized to reduce average delay in order to assist the level of service for the given junction.

The traffic and geometric data were taken from the Imam Hamza Intersection as a case study. A delay problem in this intersection of Dohuk city (northern Iraq) was analyzed and redesigned under the present conditions of right-of-way and volumes of traffic. Firstly redesign for signalized intersection was done by expanding the roads of each approach. Then designs of three new interchange models were proposed. A forecasting of traffic volumes for the next fifteen years was taken using each design model.

The simulations for these new models were taken by applying different scenarios for each model in order to find the best results of delay and level of service. Likewise a comparison was made based on the best amount of delay obtained from these new models.

Finally, the level of delay for the peak hours of each model was compared with the current level of delay in the intersection in order to find the total delay hours for next fifteen years and determine the most effective design in terms of reducing delay in the given intersection. The conclusion of this study displayed the second model diamond interchange had the minimum results of delay comparing with the other models.

Keywords: Delay, Signalized intersection, Diamond interchange, Imam Hamza Intersection, Level of service, Traffic jam, Delay time cost

ÖZ

Kavşak analizinde en önemli etkenlerden biri de trafikte yaşanan gecikmelerdir. Bu araştırma alternatif kavşak modellerini ele alarak ortalama bekleme sürelerinin sonuçlarını mercek altına almayı hedeflemektedir. Bu farklı modeller alternatif tasarımlar yaratmak için sinyal süresine göre hem sinyalizasyon ile hem de sinyalizasyonsuz halde analiz edilecektir. Söz konusu olan kavşağın hizmet seviyesini artırmak için yeşil ışığın süreleri en iyi şekilde kullanılarak ortalama bekleme gecikme süresinin azaltılması amaçlanmaktadır.

Trafik ve geometrik veriler örnek olay incelemesi olarak Imam Hamza kavşağından alınacaktır. Irak'ın kuzeyindeki Dohuk şehrinde bulunan bu kavşaktaki bir gecikme problemi mevcut trafiğin sağ şeridi ve trafik yoğunluğunun durumu göz önünde bulundurularak analiz edilip yeniden tasarlanacaktır.

Elmas şeklinde köprülü kavşağın üç yeni modelini tasarlamak için önümüzdeki 15 yılın trafik yoğunluk tahminleri dikkate alınacaktır. Aynı şekilde söz konusu olan yeni modeller ile benzer yollardaki gecikme süresi baz alınarak bir karşılaştırma da yapılacaktır. Bu süre miktarı tespit edilince gecikmeden dolayı oluşan maliyet belirlenebilecektir.

Son olarak, gelecek 15 yılın toplam gecikme süresini bulmak, en etkili modeli belirlemek ve verilen kavşaktaki gecikme süresini düşürmek için her modelin yoğun saatlerdeki gecikme süresi ve kavşaktaki şimdiki gecikme süresi karşılaştırıldı.

Anahtar Sözcükler: Gecikme, siyalizasyonlu kavşak, Elmas şeklinde kavşak, Imam Hamza kavşağı, Hizmet seviyesi, Trafik sıkışıklığı, Gecikme süresi maliyeti

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I would like to give further thanks to my family and to everyone who supported me in Iraq and Cyprus.

"I can do all things through GOD which strengthened me"

Philippians 4:13

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LIST OF ABBREVIATIONS

LOS	Level of service
AASHTO	American Association of State Highway and Transportation Officials
HCM	Highway Capacity Manual
NCHRP	National Cooperative Highway Research Program
IHI	Imam Hamza Intersection
HCS	Highway Capacity Software
VISSIM	Visual Simulation

Chapter 1

INTRODUCTION

1.1 General

The lack of urban public transportation systems in Iraqi cities makes the society highly dependent on passenger vehicles. Since 2003, the number of cars had increased abnormally and that had given way to an increase in traffic accidents especially at major intersections [1].

Consequently, the government had started to build new roads and expand the existing roads in order to accommodate the growing number of cars. But the expansion had done without changing the connecting areas at signalized intersections. This had created frequent traffic congestion as observed in these areas.

To measure the level of service (LOS) for road or intersection, the calculations of delay must be made and these calculations depend on the approaches of each intersection and how many lanes exist in each approach.

Likewise, traffic congestion has many negative consequences with respect to the economy, driver behavior and the capacity of the roads. Excessive delay on intersections has created a growing need for research resulting in technical reports, papers or theses in order to solve this growing global dilemma.

The main purpose of this study is to show the problems of an existing signalized intersection and if possible to provide some guidelines on how to choose between geometrical designs for grade separated interchanges in order to minimize delays.

This research analyzed one of the signalized intersections (Imam Hamza Intersection) in Dohuk, a city in northern Iraq. The highway capacity manual software [2] was utilized to analyze traffic data and AutoCAD Civil 3D was used to suggest three design interchange models as a replacement for the existing intersection depending on AASHTO 2004[3]. Visual Simulation (VISSIM) was used to simulate the new models [4].

1.2 Background

Much research and many articles discussed the traffic delay problem. In order to accurately analyze this problem, the subsequent steps could be following. Firstly, the traffic data must be collected at peak hours. Secondly, data on the physical characteristics of the location (site) must also be available. Thirdly, for redesign, the best-suited type of intersection, a roundabout or signaled interchange, must be determined. Finally the design of the junctions should be compared and checked with the traffic data based on some parameters such as right of way, cost and environmental concerns [6].

Mazloumi tried to show and solve the delay problems at intersections [7] by finding the delay at signalized intersections depends on various factors such as intersection geometry, signal timing, traffic volumes and drivers’ behavior culture in each country. On the other hand, Simões considered in his analyses of intersections using some computer software to simulate the performance of vehicle and allow traffic engineers to experiment several configurations[8].

Design and comparison between multi models of interchanges give a good idea on how to select the best model. Some theses had been written comparing some models and establishing some elements useful for analysis. One suggestion given in analyzing multi geometrical design models is fixing one factor such as traffic volume to achieve the best model [9].

1.3 Objective

The primary aim of this study is to solve the delay problem at Imam Hamza intersection (IHI) in Dohuk city by explaining how to analyze signalized intersections then attempting to redesign this intersection by adding some lanes. Finally three new models of interchanges were suggested to reduce the delay in order to determine the best model by applying many scenarios for each model.
The simulation of all scenarios was by VISSIM software taking into account the predication factors for next fifteen years.

1.4 Research Organization

Chapter 1: explains the general location, background and objective of the study.

Chapter 2: contains literature review about intersections and interchange types, considerations for analyzing signalized intersections and background on Diamond interchange. Finally, some details on traffic simulation are described by VISSIM.

Chapter 3: explains the methodology of the study including collecting traffic data from existing signaled intersection, analyzing data using highway capacity software (HCS) and finally presenting the results of analysis.

Chapter 4: presents alternative solutions for the existing signaled intersection to design some interchange models. A new design corresponding to the predicted traffic volume of the next fifteen years is put forward.

Chapter 5: focuses on simulating new interchange models using VISSIM and suggesting multiple scenarios by comparing and using the best results of delay and average speed among the three models.

Chapter 6: conclusion and recommendations were being explained.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

In this chapter some research, reports and studies explaining the problems of intersections and the ways to fix them were presented. It included a discussion on how to make the decision to change the type of intersection and how to improve each interchange.

2.2 Intersection

An intersection is the area where two or more roads join or cross each other. This area is very critical for safety and for delay prevention. In general, intersections can be classified into three categories: at-grade intersection, grade separation without ramps and interchanges [3].

At grade signalized intersection connects three, four or five legs and all connections are made in one area. Thus, appropriate signals for each leg (approach) should be installed for safety reasons. Some considerations that should be taken into account at signaled intersections include capacity, demand, delay, and level of service. In chapter three, the acceptable limitations were analyzed [2].

2.2.1 The Requirements for Analyzing Signalized Intersection

In the process of analyzing traffic capacity and level of service (LOS) of signalized intersections some conditions such as the value and distribution of traffic movements, geometric characteristics and all of the details of signalization of intersection must be taken into consideration. By finding the delay for each approach, the level of service (LOS) for each approach can be determined. This is done by comparing with the standard limitation as shown in Table 2-1 [2].

Table 2-1: HCM standard limitation for signalized intersection [2]

Level of service (LOS)	Total Average Delay (Sec.)
A	10
B	>10 – 20
C	>20 – 35
D	>35 – 55
E	>55 – 80
F	>80

From this table, it can be noted that delay is an important parameter to measure the efficiency of each intersection.

2.2.2 Analyzing Traffic Data in an Intersection

In civil engineering when analyzing any construction (buildings, dams, traffic intersections, highways, etc.) reliance on computer software was preferred. Modern engineering software took into consideration the standard specifications and limitations imposed by government organizations.

Highway capacity standard specifications were used in HCS software to analyze all traffic conditions and all delay calculations. HCS was considered the bible of roadway capacity analyses [10].This software which was used by the U.S. Department of Transportation's Federal Highway Administration (FHWA) was developed by the Center for Microcomputers in Transportation (Mc*Trans*) [11].

HCS determined many types of delays, especially for signalized intersection, and then compared them with the limitation of level of service (LOS) which was saved inside it and provided the final report for all results.

The requirement of planning and design any intersection, the designer needs to know the number of lanes that can be applied by forecasting the traffic volume for the future depending on some real data taken at present and after that calculate the level of service (LOS) corresponding to the minimum delay cycle length [12].

2.3 Interchange

According to AASHTO policy on geometric design of highway an interchange is a system of interconnecting roadways with one or more grade separations which have movement of traffic between two or more roadways on different levels [3].

2.3.1 Types of Grade Separated Interchange

There are many shapes of grade separated interchanges and each figure depends on some factors at the site. Diamond interchange and full cloverleaf interchange are two of many shapes of interchanges as shown in Figure 2-1 and Figure 2-2.

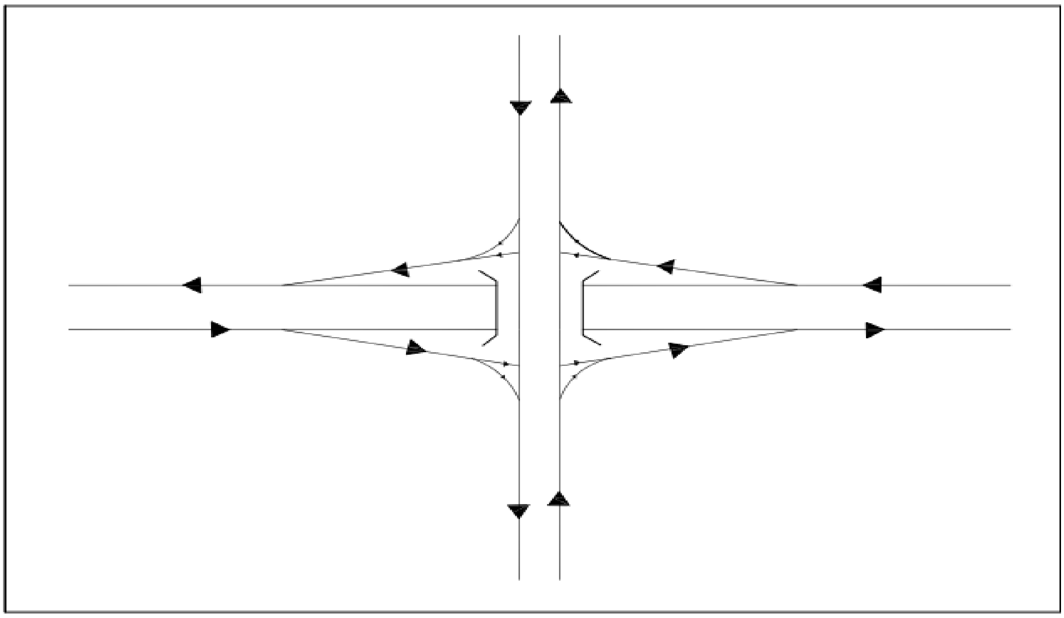


Figure 2-1: Diamond interchange

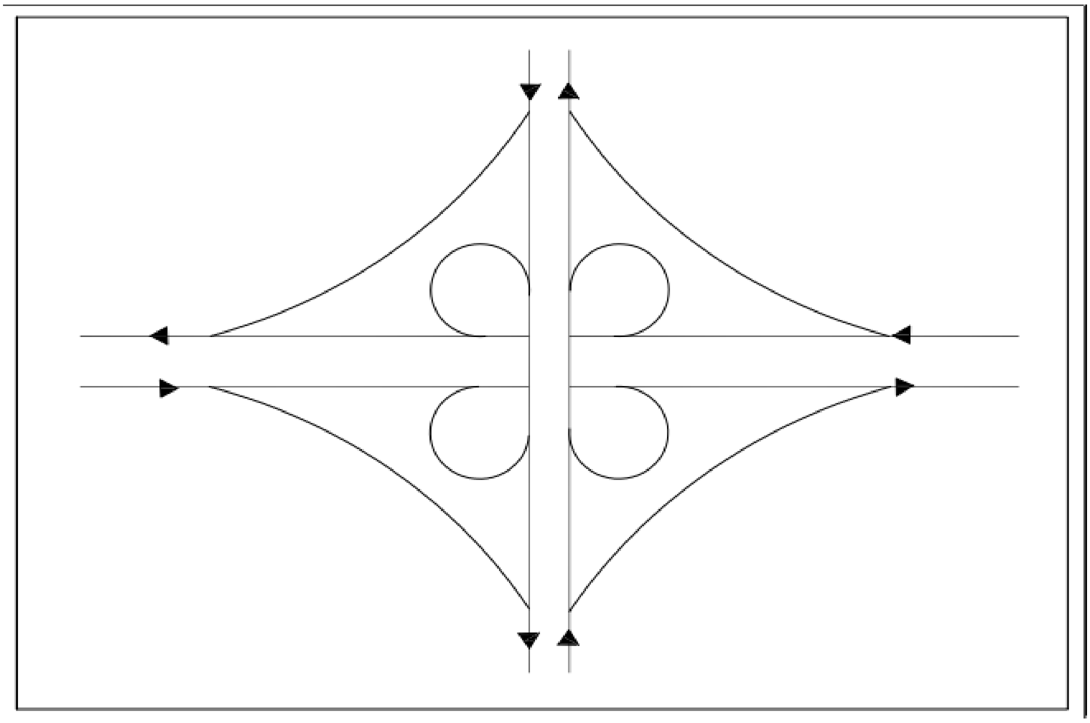


Figure 2-2: Cloverleaf interchange

Different variations of these interchanges may have different shapes according to the area used, types of roads, cost of project, and volumes of traffic.

Some useful recommendation for selecting the type of interchange can be listed below [13]:

- Right-of-way availability: When the available right of way is limited, diamond interchange is most suitable; a cloverleaf interchange requires a larger right-of-way, due to the space requirements of the loop ramps.
- Construction cost: The diamond interchange has the lowest cost of all the interchange types, due to its compact design which results in smaller of right-of-way requirement. The cost of cloverleaf interchange may be higher than diamond because of the need to build at least four huge loops.
- Traffic issues: Un-signalized diamond interchange should be used when traffic volumes are very low (under 1500 vph). But if the volumes are between (1500 and 5500 vph), signalized system should be used at the intersection area.
- Pedestrian areas are more suitable for diamond interchanges but not so convenient for cloverleaf.

2.3.2 Background of Diamond Interchange

Because of the limited area of the studied site used in this research, diamond interchange type was selected and in this section can be see some details for this type used in different countries.

Diamond interchange is the simplest and most common type of interchange. A diamond interchange is formed when connecting a major with a minor roadway and is fitted with ramps in each quadrant consequently all traffic on the major road way can enter and exit at high speeds. The type of structure connection between major and minor will occur either by using an overpass or an underpass depending on some factors such as traffic flow, topography and cost of the project. This is shown in Figure 2-3. In the picture on the left, the major road uses the underpass. In the picture on the right the major road uses the overpass [14].

-A- -B-

Figure 2-3: Roundabout diamond interchange Nicosia- Cyprus A: Archaggelos junction, B: Roundabout lakatamias [14]

A sampling of other diamond interchanges can be shown in Figure 2-4 and Figure 2-5:

Figure 2-4: Roundabout diamond interchange Esteghlal Sq. Mashhad –Iran [14]

-A- -B-

Figure 2-5: Diamond interchange A: Al-Farouq interchange, B: Singar interchange Mosul-Iraq [14]

The using of U-turn with underpass for major road in diamond interchange can be seen in Figure2-6:

Figure 2-6: Diamond interchange U turns exclusive N-Lee Trevino Dr. interchange in El Paso-Texas-USA [14]

In Table 2-2 the advantages of overpass and underpass construction can be observed [15].

Table 2-2: Overpass, underpass geometry and function comparison [15]

Crossroad location relative to existing ground	Major Road Location Relative to Crossroad	
	Overpass	Underpass
Below	Offers best sight distance along major road	Not applicable
At	Offers best possibility for stag construction Elimination drainage problems	Reduce traffic noise to adjacent property Provides best view of ramp geometry
Above	Not applicable	Ramp grades decelerate exit-ramp vehicles and accelerate entrance-ramp vehicles. Eliminates drainage problems. Typically requires least earthwork.

The alternatives for design diamond with overpass and underpass road types are being explained in chapter four.

2.4 Traffic Simulations

In recent years, the rapid growth of technological applications and need for tools to make quick and accurate decisions for the future had led to the development of the simulation concept. Because the traffic simulation models are becoming an increasingly important tool for traffic control, simulators need to generate scenarios, optimize control and predict network behavior at the operational level. Sometimes computer models can be used to simulate the influence of governmental measures like road pricing or building of new streets [16].

The use of computer simulation started when Gerlough (1955) published his dissertation, “Simulation of Freeway Traffic on a General Purpose Discrete Variable Computer”. Since then, computer simulation has become a widely used tool in transportation engineering with a variety of applications from scientific research to planning, training and demonstration.

Many microscopic simulation packages are used to analyze traffic models such as VISSIM (Visual Simulation), PARAMICS, CORSIM (Corridor Simulation) and many other types of software. The reasons of selecting specific software for the research of this thesis are being discussed in chapter five.

2.5 Simulation by VISSIM

VISSIM simulation system allows district and microscopic simulation, random traffic flow, junction and network analysis. VISSIM software system is composed of two large program states, traffic simulator and signal generator.

An urban interchange is a road intersection whose research scope is relatively small, so VISSIM simulation software can be very effective in describing the interaction behavior between vehicles, and can validate improvement measures of an urban interchange simply and quickly. It is also useful in determining the key factors that affect the traffic operation of the interchange [18].

According to one research in which VISSIM software was used to simulate traffic data in San Diego, California, its advantages include [19]:

- Integrates freeways and surface streets seamlessly;
- Allows for pre-timed and actuated signals and ramp meters;
- Driver behavior parameters are adjustable to provide flexibility in calibration and validation;
- No limits on the number of nodes, links and vehicles on any simulation;

- Can use GIS layers and/or photos to help define inputs and reference animation output;
- VISSIM can be used to model complicated facilities, such as major freeway interchanges with ramp metering.

Chapter 3

ANALYSIS OF SIGNALIZED INTERSECTION

3.1 Introduction

The main idea behind this chapter is to collect traffic data and use computer software to analyze this data in order to determine the problem at the Imam Hamza Intersection (IHI) in Dohuk city. The attempting to see if it would be possible to solve this problem by changing the green signal time to obtain the best state of level of service (LOS) without changing the geometric dimensions.

3.2 Location of Dohuk

Because of the border gate between Iraq and Turkey, Dohuk is considered an important commercial province since all goods imported from Turkey to Iraq pass through the city. Dohuk is located 36.85°North 43.02°East and is surrounded by Mountain ranges. The population of Dohuk is approximately 900,000 [5].

Figure 3-1: Location of Dohuk from Iraqi road map

3.3 Traffic Data Collection

The most important thing which can be used to illustrate the problem at an intersection is the data taken from the site and analyzing the data to obtain the results. Many methods can be used to collect traffic data for an intersection. Among these methods video recording is most advantageous due to limited human resource requirements during the data collection process.

3.3.1 Video Data Collection

There are many advantages relating to the use of video camera for collecting traffic data [20]:

- Efficiency, the video has no human errors.
- The video can be replayed many times.
- The camera can be used to observe the behavior of some drivers (drivers' behaviors are different from city to city in the same country).

- The camera can record the proportion of drivers that respond to the instructions of a traffic policeman.
- It is simple and fast method for a team that has no members to collect data manually as shown in Figure 3-2.
- Appropriate for locations with large land area.

-A-	-B-

Figure 3-2: Imam Hamza Intersection (IHI) A: Normal traffic volume, B: High traffic volume

The data was collected on the first day of the work week. The reason for choosing the first day is that peak hours in traffic in Dohuk city usually occur on Sunday because it is the first day in the week in Iraq (this day always has many traffic jams, especially in the morning and afternoon). Two recordings were made on Sunday the 3rd and 10th of February 2013 from 2:45 pm to 3:45 pm. In addition, the normal work day in Iraq goes from 8 am to 3 pm.

3.3.2 Background for the Location

Imam Hamza Intersection (IHI) is T shaped as shown in Figure 3-2 connecting two main roads and one minor road. The first two have four lanes each, and the smaller road has three lanes. The right turns for all are exclusive. In this study, each approach at intersection has been defined to ensure the simple inputting and outputting for analyzing process as shown in Figure3-3.

- Eastbound (EB): the vehicles coming from the west.
- Northbound (NB): the vehicles coming from the south.
- Southbound (SB): the vehicles coming from the north.

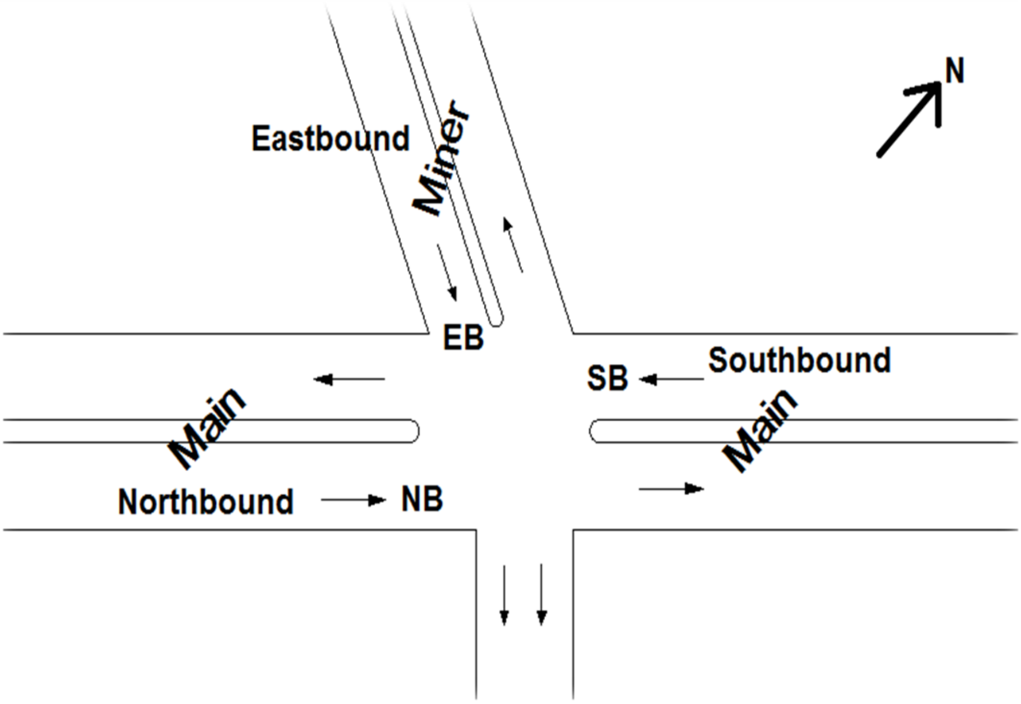


Figure 3-3: Define roads for IHI

3.3.3 Data Preparation

Firstly, the camera was set up in a good location to get an excellent view of the intersection (all approaches should be visible).

Secondly, after the process of recording had finished, the video was saved on the hard drive (HDD), and then the data was transferred carefully from the video and inserted in the tables in excel software to review it at any time as shown in Table 3-1 and Table 3-2.

Finally, all traffic data was entered into highway capacity software HCS2000 (which developed by McTrans center [11], University of Florida).

All these points were repeated for second video recording. Figure 3-4 shows a screenshot of this software.

Figure 3-4: HCS 2000 software [4]

In addition to video traffic data, some physical measurements (lane width, number of lane in each approach) were recorded and some information likes percentage of heavy vehicles, speed of pedestrians, etc. about the site was obtained. The data was then fed into the HCS2000 software.

Table 3-1: Traffic volume data for 1st peak hour in IHI

1st video results						
Number of cycle length	**EB (veh.)**		**SB (veh.)**		**NB (veh.)**	
	Thru	left	Thru	left	Thru	left
1	13	81	123	15	176	51
2	11	92	6	1	126	32
3	0	4	96	6	46	12
4	6	40	114	20	153	30
5	7	80	132	16	85	35
6	8	83	122	19	148	35
7	5	36	119	16	170	60
8	19	85	72	10	96	30
9	7	63	151	17	149	44
10	10	69	124	18	128	50
11	11	64	199	27	160	58
12	10	72	151	17	127	54
13	11	64	142	18	149	59

Table 3-2: Traffic volume data for 2nd peak hour in IHI

2nd video results						
Number of cycle length	**EB (veh.)**		**SB (veh.)**		**NB (veh.)**	
	Thru	left	Thru	left	Thru	left
1	4	25	59	6	45	15
2	4	25	56	5	65	12
3	3	33	36	5	63	9
4	5	28	69	4	45	11
5	6	26	43	3	57	11
6	7	23	67	8	56	15
7	3	29	56	7	55	16
8	2	30	50	9	64	12
9	5	29	61	7	52	15
10	5	23	61	9	52	17
11	3	24	61	5	56	16
12	5	27	59	10	52	15
13	6	20	52	5	61	14
14	5	24	65	8	58	17
15	3	30	55	3	80	29
16	8	26	86	8	122	28
17	7	37	75	13	129	43
18	11	47	80	3	108	39
19	9	65	101	12	139	36
20	9	51	181	17	170	39
21	10	61	85	6	136	22
22	15	67	60	7	134	59

3.4 Data Analysis

The delay and level of service (LOS) are the measurements of efficiency for each signalized intersection. There are many methods to analyze traffic data and in this study, one of these methods was observed using field measurement as input in software.

3.4.1 Definition of key Parameters

In addition to delay and LOS some criteria were determined from the results of simulations in the software such as:

- Lane group capacity: the maximum hourly rate at which vehicles can reasonably be expected to pass through the intersection under prevailing traffic, roadway, and signalization conditions.
- V/C Ratio: The ratio of volume to capacity (v/c), represented by (X_{cm}), is typically referred to as the measure of the degree of saturation at an intersection. Table 3-3 explains the relation between capacity conditions with the amount of V/C [2].

Table 3-3: V/C ratio for signalized intersection [3]

Critical V/C Ratio X_{cm}	**Capacity condition**
$X_{cm} \leq 0.85$	Under capacity
$0.85 < X_{cm} < 0.95$	Near capacity
$0.95 < X_{cm} \leq 1.0$	At capacity
$X_{cm} < 1.0$	Over capacity

- Lane group delay: The control delay for a given lane group.
- Delay: The additional travel time experienced by a driver, passenger, or pedestrian.

- Level of service (LOS): Is the average delay per vehicle estimated for each lane category and aggregated for each approach for the intersection. It is the qualitative measurement describing operational conditions within a traffic stream such as speed, travel time, traffic interruptions, and convenience.
- Peak-hour factor: The hourly volume during the maximum-volume hour of the day divided by the peak 15-min flow rate within the peak hour.
- Average queue spacing: is the average length between the back bumper and front bumper of two successive vehicles in queue (0.5 m is used for this spacing).
- All red: the time when all vehicles stopping in cycle length in signalized intersection.

There are many parameters to be considered in order to input data in HCS 2000 software. The next section deals with most of the data that should be entered to find the result of level of service and delay.

3.4.2 Intersection Geometry and Traffic Volume Inputs

Volume data and peak hour factor can be entered manually, in addition to other variables that can be entered such as the number of lanes, average queue spacing, duration and available queue storage length as shown in Figure 3-5.

From Equation (3.1) can calculate the Peak hourly factor for each approach [21]:

$$\text{Peak hourly factor(PHF)} = \frac{\text{Total volume } (V_{60})}{4 * \text{Max}(V_{15})} \qquad \text{Eq.}(3.1)$$

where:
V_{60}: Total volume for 60 minutes in one each direction approach.
V_{15}: Total volume for 15 minutes in one each direction approach.

3.4.3 Volumes Data Selection

The selection between two hours video recording were depend on total traffic volumes per each hour as shown in Tables 3-4 and Table 3-5.

Table 3-4: 1st video recording of total traffic volumes data for IHI

EB		NB		SB	
THRU	LEFT	THRU	LEFT	THRU	LEFT
118	833	1551	200	1713	550

Table 3-5: 2nd video recording of total traffic volumes data for IHI

EB		NB		SB	
THRU	LEFT	THRU	LEFT	THRU	LEFT
135	750	1518	160	1799	490

Comparing the traffic data between the two peak hours include:

The volume of total traffic in EB & NB in the first video has increased by 6.9 and 4.16 % respectively for the second hour but in the SB, it has shown little difference, not more than 1.13 %.

Therefore, the results from the first video can be validated because it represents the real problem at the intersection.

In the analysis, the volumes used in this software should be equal to the duration of time which is used in calculating the peak hour factor [21], therefore, from Eq.(3.2) can be applied the values of traffic volumes for each 15 minutes three segments. Finally, the time of 0.75 (45 minutes) hour was used to find the peak hour factor, delay and LOS as shown in Table 3-6 and Table 3-7 and Figure 3-5.

$$\text{Peak hourly factor(PHF)} = \frac{\text{Total volume } (V_{45})}{3 * Max(V_{15})} \qquad \text{Eq. } (3.2)$$

Table 3-6: Traffic volumes for 0.75 hour

EB		NB		SB	
THRU	LEFT	THRU	LEFT	THRU	LEFT
97	697	1437	437	1258	165

Table 3-7: Peak hour factor calculation for IHI

Northbound , PHF calculation					
	Total V 1st (15 min.)	Total V 2nd (15 min.)	Total V 3rd (15 min.)	PHF for Left	PHF for Thru
Left	108	137	123	**0.90**	**0.96**
Thru	501	499	437		
Eastbound , PHF calculation					
Left	217	199	217	**0.97**	**0.80**
Thru	30	20	36		
Southbound , PHF calculation					
Left	38	48	41	**0.88**	**0.95**
Thru	339	373	347		

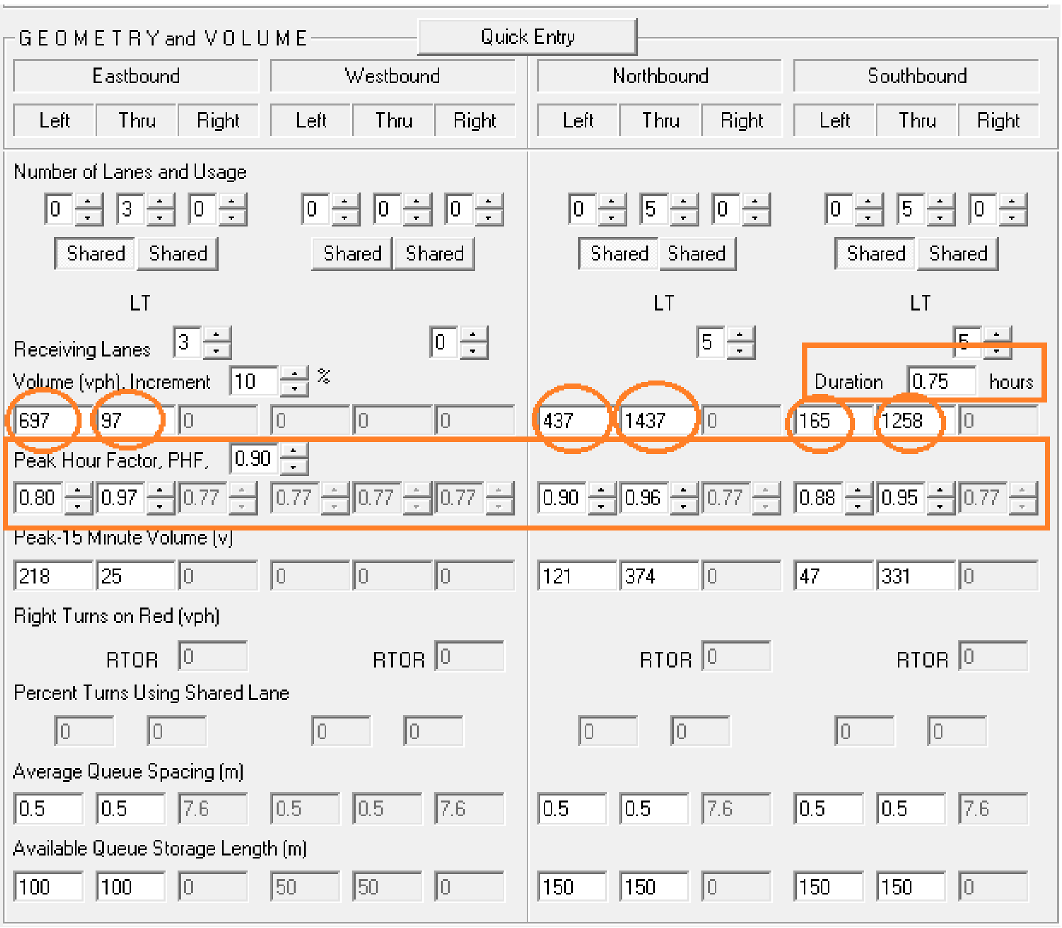


Figure 3-5: Geometry and volume part for HCS2000

3.4.4 Operating Parameters

In this section operating parameters can be defined; other parameters assist to understand the software as shown in Figure 3-6.

- Unmet demand: the number of vehicles on a signalized lane group that have not been served at any point in time. (assume 50 for EB and 100 for NB & SB).
- Pedestrian speed: is the average walking speed of pedestrians, in meter per second, the speed is approximately 1.5 m/sec.

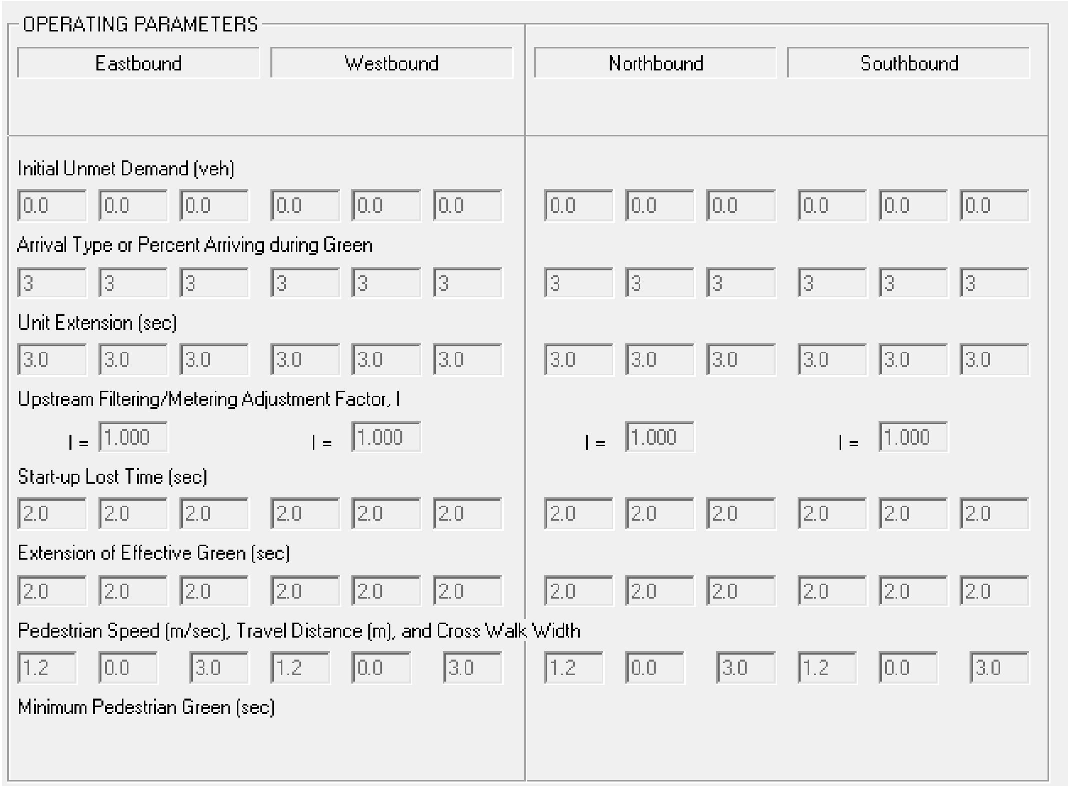


Figure 3-6: Operating parameters part in HCS2000

3.4.5 Signal Timing

- Phase: is the part of the signal cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals.
- Cycle length: The total time for a signal to complete one cycle at intersection [22] (Max. Cycle length according to HCM2000 is 150 sec. and Min. is 60 sec [2]).

In this study, Imam Hamza intersection had three phases as shown in Figure 3-7 because all vehicles in each approach moving together. And for each phase had cycle length can be entered the values of green, yellow and red times as shown in Figure 3-8.

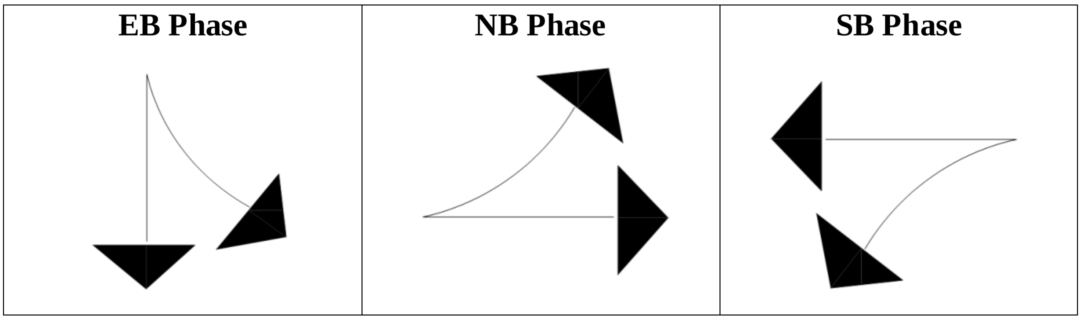


Figure 3-7: Phase diagrams

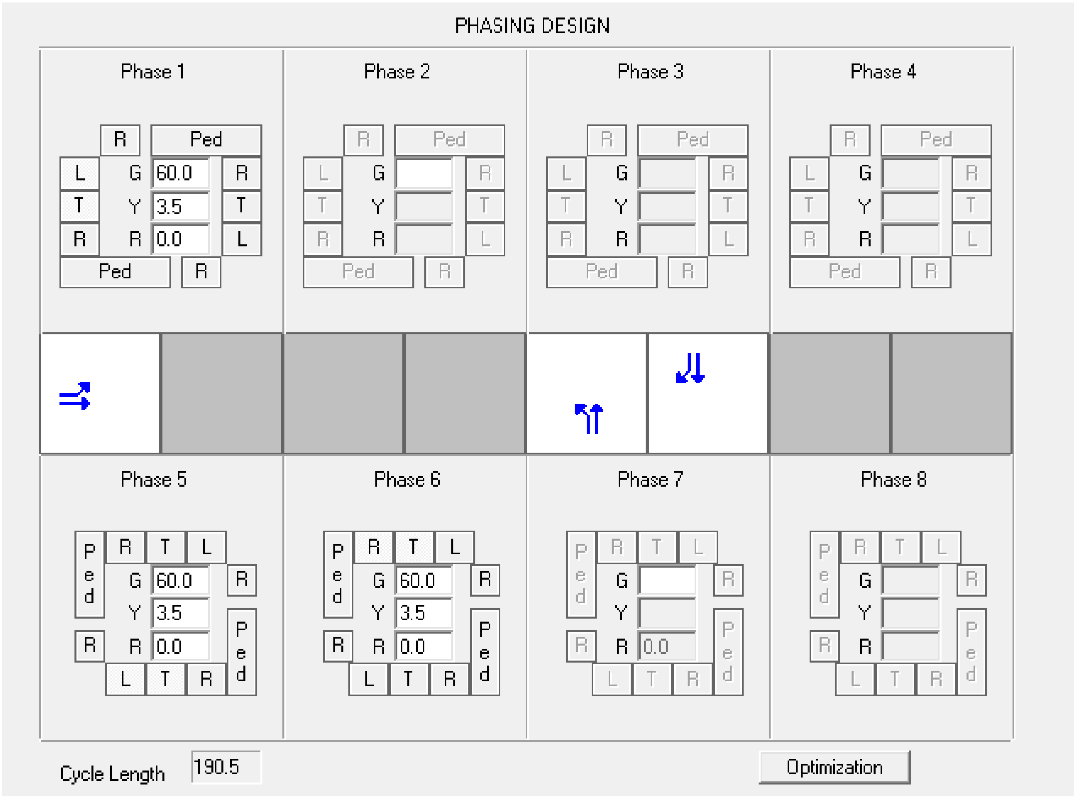


Figure 3-8: Phasing design part in HCS2000

3.4.6 Saturation Flow Rate

This is the equivalent hourly rate at which previously queued vehicles can traverse an intersection approach under prevailing conditions [23], from Equation 3.3 can find the saturation flow rate:

$$S = S_0 N f_W \, f_{HV} \, f_g \, f_p \, f_{bb} \, f_a \, f_{LU} \, f_{LT} \, f_{RT} \, f_{Lpb} \, f_{Rpb} \qquad \text{.Eq. (3.3)}$$

where:

S : saturation flow rate (veh/h),

S_0: Base saturation flow rate per lane (pc/hour/lane),

N : Number of lanes in approach,

f_W : factor for lane width,

f_{HV} : factor for heavy vehicles,

f_g : factor for approach grade ;

f_p : factor for existence of a parking lane,

f_{bb}: factor for blocking effect of local buses that stop within intersection,

f_a : factor for area type,

f_{LU} : factor for lane utilization,

f_{LT} : factor for left turns,

f_{RT} : factor for right turns,

f_{Lpb} : pedestrian adjustment factor for left-turn movements,

f_{Rpb} : Pedestrian-bicycle adjustment factor for right-turn movements.

The values of these factors used in this study can be seen in Table3-8

Table 3-8: The values of factors used in equation (3.3) for IHI

	EB	SB	NB
S_0	1800	1800	1800
N	3	5	5
f_W	0.9	0.9	0.9
f_{HV}	1.0	1.0	1.0
f_g	1.0	1.0	1.0
f_p	1.0	1.0	1.0
f_{bb}	1.0	1.0	1.0
f_a	1.0	1.0	1.0
f_{LU}	0.95	0.95	0.95
f_{LT}	0.957	0.993	0.987
f_{RT}	0.85	0.85	0.85
f_{Lpb}	0.9	0.9	0.9
f_{Rpb}	1.0	1.0	1.0

3.5 Results of the Analysis

The choice of the suitable hour for planning and designing is important for providing an adequate level of service (LOS) for every hour of the year. Because of this, the study chose two hours for collecting traffic volumes, after that, these volumes were inputted in HCS to obtain the result for delay and level of service LOS) in this intersection (IHI)as shown in Figure 3-9.

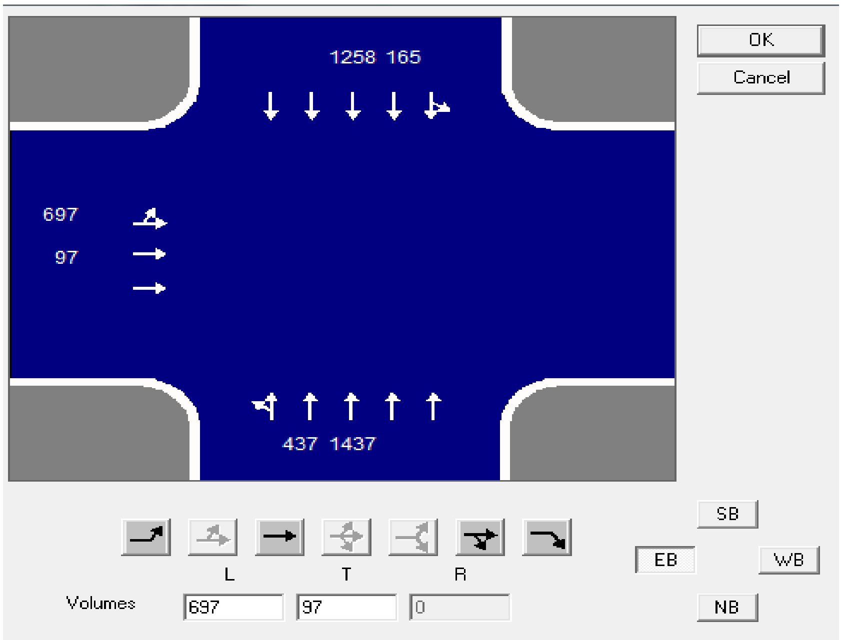


Figure 3-9: HCS inputting traffic volumes and number of lanes

After all the parameters required by the software to run the analysis was put in, the following results were obtain as shown in Figure 3-10.

RESULTS

	Eastbound	Westbound	Northbound	Southbound
	LT		LT	LT
Lane Group Adjusted Volume, (vph)	0 971 0	0 0 0	0 1983 0	0 1512 0
Lane Group Capacity, (vph)	838		1980	1992
Lane Group v/c Ratio	1.16		1.00	0.76
Critical Lane Group	#		#	#
Lane Group Delay, (sec/veh)	534.3		279.1	112.6
Lane Group Level of Service	F		F	F
Final Unmet Demand, (v)	149.7		102.3	0.0
Approach Delay, (sec/veh)	534.3		279.1	112.6
Approach Level of Service	F		F	F

Cycle Length 290.5 sec Intersection Delay 278.2 sec/veh Intersection LOS F

Figure 3-10: Output results for present traffic data

When the actual values of traffic volumes were put in the level of service (LOS) obtained was F because the intersection delay was more than 80 sec. The actual cycle length was 290.5 sec. This amount includes a total of 80 sec. green time for EB and 100sec. green time for NB and SB plus 10 sec. for all red. Table 3-10 shows the cases of LOS according to the delay values [2].

If different values of cycle length were entered, the results obtained were as shown in Table 3-9.

Table 3-9: Results by changing cycle length

Case	Cycle length (sec.)	Green time (sec)			Intersection delay	Level of service(LOS)
		EB	NB	SB		
1	120.5	30	35	45	388.1	F
2	130.5	30	40	50	383.7	F
3	140.5	30	45	55	388.2	F
4	150.5	30	50	60	400.4	F
5	160.5	30	55	65	416.2	F
6	170.5	30	60	70	437.1	F
7	180.5	30	65	75	459.3	F
8	190.5	30	70	80	484.4	F
9	200.5	30	75	85	513.7	F
10	210.5	30	80	90	545.3	F

Note1: in these cases assume yellow ≡ 3.5 and all red ≡ 0
Note 2: no. of lanes for EB ≡ 3 shared, for NB ≡ 5 shared , for SB ≡ 5 shared

Table 3-10: The values of delay with LOS in signalized intersection [2]

LOS Delay(sec)	Description
A ≤ 10.00	• Free flow & non- delays. • No waiting longer than one red signal. • Traffic flow is extremely good, and most vehicles arrive during the green time.
B $10.1 - 20.00$	• Stable operation & short delay times. • This level generally includes good traffic flow, short cycle lengths, or both.
C $20.1 - 35.00$	• Stable operation & Acceptable delays. • Higher delays may result from normal traffic flow, longer cycle lengths, or both. • Individual cycle failures may begin to appear at this level.
D $35.1 - 55.00$	• Approaching unstable & possible delays. • Waiting more than one red signal indication. • Longer delays may be causes by some combination of unfavorable traffic flow, long cycle lengths, or high v/c ratios.
E $55.1 - 80.00$	• Unstable operation & considerable delays. • Waiting though several signal cycles. Long queues form upstream of intersection. • High delay values because of long cycle lengths, and high v/c ratios.
F ≥ 80.00	• Slow traffic flow & overload delays. • This level occurs when arrival flow rates exceed intersection capacity, and is considered to be unacceptable to most drivers. • Poor traffic flow, long cycle lengths, and v/c ratios approaching 1.0 may contribute to these high delay levels.

3.6 Benefit of the Level of Service

The conclusion of the analysis can be enumerated as follows:

- The large volume of vehicles passing through this intersection is the real problem. This is especially the case for the main road because the value of intersection delay reaches ≈ 5 minutes for each vehicle. This number is not acceptable for traffic engineering nor for drivers who use this intersection.
- When changing the green interval in each approach, the resulting of delay is still not acceptable as shown in table 3-10.

From the two points, realize that the problem of delay and level of service cannot be solved by changing cycle lengths. Some other method was being tried to solve this problem. In the chapter four, the proposed method to solve delay and level of service for Dohuk city intersection (IHI) is presented.

Chapter 4

ALTERNATIVE SOLUTION FOR TRAFFIC CONGESTION

4.1 Introduction

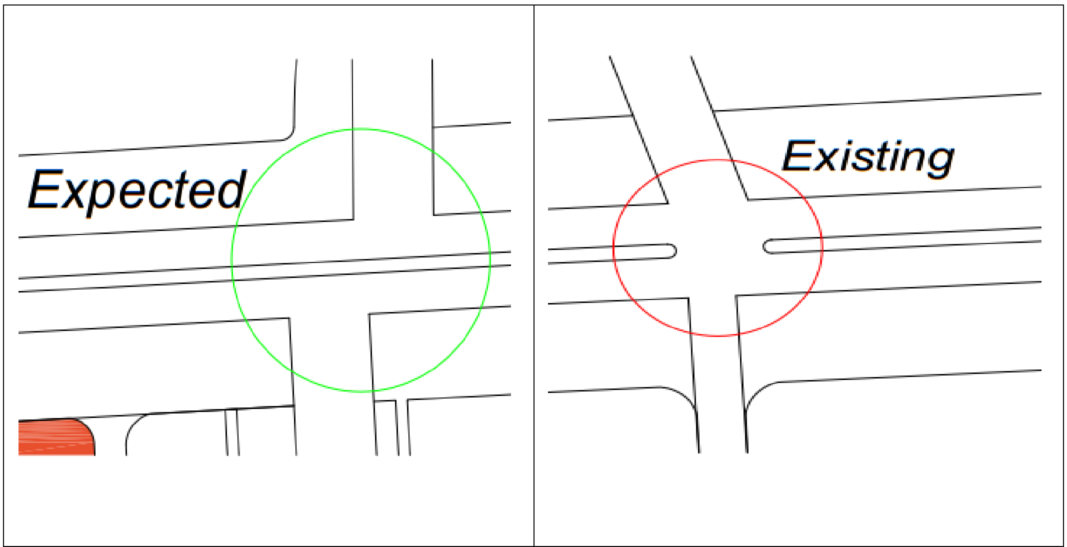
The objective of this chapter is to explain some redesign methods for signaled intersections. Consequently by using the same traffic volumes and projecting for the next fifteen years a redesigned interchange model is discussed.

There are many types of intersections used to connect major and minor roads. An intersection having more vehicles in one approach than the other is referred to as Grade-separated junction. It has at the entrance and exit slip roads which produce a diamond interchange junction or roundabout junction or half-cloverleaf interchange, etc.

The decision to redesign an intersection is based on economic factors and traffic continuous flow advantage. Sensitivity to delay, future traffic forecast and right-of-way (land area) have considerable influence on the choice of junction type.

4.2 Traffic Volume Projection at the Study Location

According to the city's urban plan for the future, the location of the intersection will change as shown in Figure 4-1. The new plan will connect two major roads with two minor roads; therefore, in our design, all details for future plans such as volumes of vehicles for next 15 years and area of land must be taken into account. Figure 4-2 shows a forecast of the volumes of vehicles for year 2028 depending on the values of the peak hours taken in February 2013. In this study, because of the lack of history traffic volumes in Dohuk city, an increase factor for traffic volumes was assumed as 5% depending on “Project Traffic Forecasting Handbook” [24].



-A-	-B-

Figure 4-1: Part of master plan map for Dohuk city, A: Expected location for Imam Hamza Intersection, B: The existing of intersection studied.

The predication of traffic volume calculated by compound growth equation as shown in Eq. (4.1) [24]:

$$V_{2028} \equiv V_{2013}(1+i)^{15} \quad \text{Eq.(4.1)}$$

Where:

V_{2028}: Traffic volumes for year 2013,

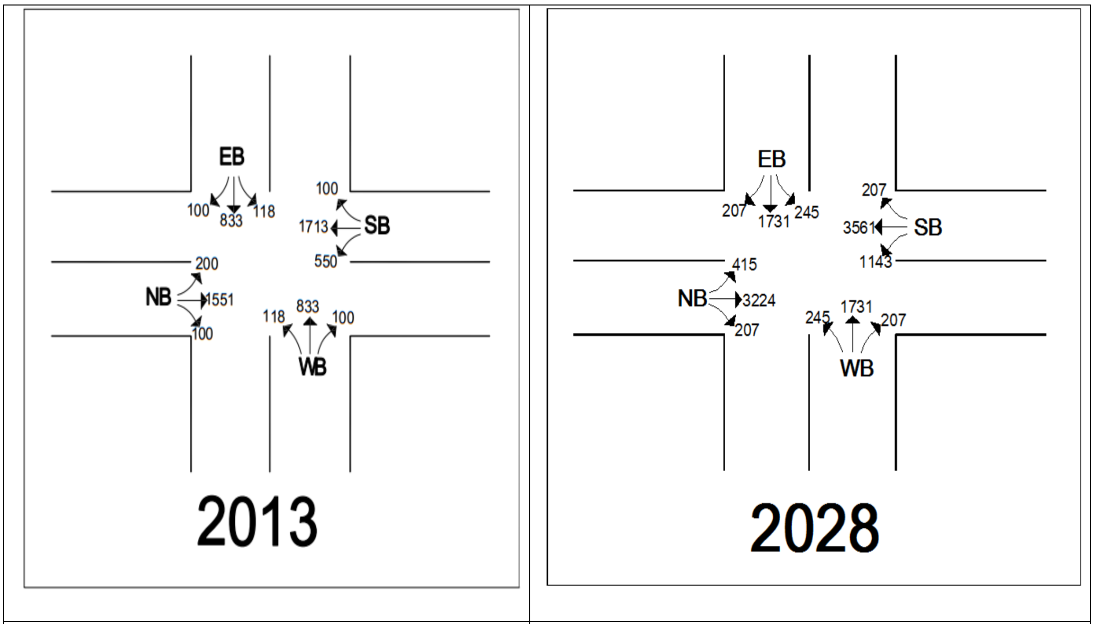
V_{2013}: Traffic volumes for year 2028,

i: annual growth rate, %

When apply Eq. (4.1) for each approach can find the predication volumes for year 2028 as shown in Table 4-1.The comparing between the traffic volumes in 2013 with traffic volumes in 2028 can be seen in Figure 4-2.

Table 4-1: The traffic hourly volume will be used in design

Intersection approaches		Traffic volumes (2013) vph	Total traffic volumes (2013) vph	Traffic volumes (2028) vph	Total traffic volumes (2028) vph
EB	Through	833	1051	1731	2184
	Left	118		245	
	Right	100		207	
SB	Through	1713	2363	3561	4912
	Left	550		1143	
	Right	100		207	
WB	Through	833	1051	1731	2184
	Left	118		245	
	Right	100		207	
NB	Through	1551	1851	3224	3848
	Left	200		415	
	Right	100		207	



-A-	-B-

Figure 4-2: Forecasting of traffic volumes A: Traffic volumes for 2013 B: Traffic volumes for 2028

4.3 Redesign of the Intersection

The redesign can be done in two ways:

- Firstly, the existing volumes and actual geometries for the signaled intersection can be increased by changing the cycle time and increasing the number of lanes to enhance the delay and LOS at the intersection.
- Secondly, three new models are suggested for the interchange which take into account forecasted future traffic volume.

4.3.1 Revising Signal Timing for Signalized Intersection

By changing the cycle length at the intersection as shown in chapter three, the results obtained prove the inefficiency of the intersection to absorb even current volume of vehicles. These attempts were without adding any lanes or changing any dimensions of the road in each approach; therefore, in this section adding and changing the shape of the intersection can be discussed in order to find delay and LOS for each case [25].

4.3.1.1 Add one lane in EB approach

In this case, one lane is added in EB approach to make four lanes in EB and keep five lanes in both the NB and SB. In addition one lane is added at the exit to receive vehicles approaching in order to keep a continuous flow of vehicles as shown in Table 4-2.

Table 4-2: 1st Redesign include (EB=4, NB=5, SB =5)

	Cycle length	Green time (sec)			Delay approach			Int. Delay	LOS approach			LOS
					EB	NB	SB		EB	NB	SB	
1	80.5	17	21	32	421.4	703.1	33.8	415.2	F	F	C	F
2	90.5	20	25	35	349.1	594	38.7	352.8	F	F	D	F
3	100.5	20	30	40	537.6	454.1	38.8	331.6	F	F	D	F
4	110.5	25	33	42	316.8	456.5	45.7	287.0	F	F	D	F
5	120.5	30	35	45	171.3	510.3	50.3	280.8	F	F	D	F
6	130.5	30	40	50	299.9	417.9	50.2	267.8	F	F	D	F
7	140.5	30	45	55	426.8	246.5	50.6	263.8	F	F	D	F
8	150.5	30	50	60	553.1	289.5	51.3	266.2	F	F	D	F
9	160.5	30	55	65	682.1	243.6	52.3	274.2	F	F	D	F
10	170.5	30	60	70	808.3	205.8	53.4	285.2	F	F	D	F
11	180.5	30	65	75	937.2	174.2	54.6	299.6	F	F	D	F
12	190.5	30	70	80	>999	146.9	55.9	315.3	F	F	E	F
13	200.5	30	75	85	>999	128.1	57.2	335.3	F	F	E	F
14	210.5	30	80	90	>999	118.1	58.7	359.2	F	F	E	F

Note 1: in all cases assume yellow ≡ 3.5 and all red ≡ 0
Note 2:the volumes using in these cases the same volume using in analyses :
EBT=97 ,EBL=697 // NBT=1437 ,NBL=437 // SBT=1258 ,SBL=165

Conclusion for these cases:

- Although there are many improvements in the cycle length, the level of service is still F because the amount of delay at the intersection is still more than 80 sec.
- The adding of lanes has no effect on the overall LOS.
- The volumes used in this table reflect the present conditions and if values for the future were to be used, the results would be even worse.

4.3.1.2 Add two lanes in EB approach and one lane in NB, SB approaches

In this case two lanes are added in EB approach and one lane in SB, NB to become five lanes in EB and six lanes in both the NB and SB as shown in Table 4-3.

Table 4-3: 2nd Redesign include (EB=5, NB=6, SB =6)

	Cycle length	Green time (sec)			Delay approach			Intersection Delay	LOS approach			LOS
					EB	NB	SB		EB	NB	SB	
1	80.5	17	21	32	87.5	366.1	26.2	190.5	F	F	C	F
2	90.5	20	25	35	70.5	275.3	30	147.8	E	F	C	F
3	100.5	20	30	40	167.8	159.5	31	117.8	F	F	C	F
4	110.5	25	33	42	71.3	162.3	36.1	99.8	E	F	D	F
5	120.5	30	35	45	59.1	207.5	39.7	118.4	E	F	D	F
6	130.5	30	40	50	75.1	130.5	40.6	88	E	F	D	F
7	140.5	30	45	55	104.1	94.4	41.6	78.7	F	F	D	E
8	150.5	30	50	60	184.9	82.2	42.8	91.2	F	F	D	F
9	160.5	30	55	65	287.9	76.6	44.1	115.5	F	E	D	F
10	170.5	30	60	70	391.6	73.9	45.4	133.3	F	E	D	F
11	180.5	30	65	75	493.8	72.7	46.8	155.4	F	E	D	F
12	190.5	30	70	80	598.5	72.2	48.2	178.5	F	E	D	F
13	200.5	30	75	85	699.5	72.3	49.6	201.0	F	E	D	F
14	210.5	30	80	90	802.3	72.8	51.1	224.1	F	E	D	F
Note: in all cases assume yellow ≡ 3.5 and all red ≡ 0 Note 2:the volumes using in these cases the same volume using in analyses : EBT=97 ,EBL=697 // NBT=1437 ,NBL=437 // SBT=1258 ,SBL=165												

The conclusion for this analysis can be summarized in these following points:

- Level of service is F for all.
- The change in SB an improved LOS of D compared with other approaches.
- In this case, the volume used reflects the current conditions; therefore even if a good LOS is obtained it will most likely not be acceptable in the future.

4.3.1.3 Add three lanes in EB approach and two lanes in other approaches

Adding three lanes is not acceptable because the land area is not sufficient to accommodate this expansion. Also complications may arise during maintenance. The results in Table 4-4 show the proposed delay and LOS in IHI.

Table 4-4: 3rd Redesign include (EB=6, NB=7, SB =7)

	Cycle length	Green time (sec.)			Delay approach			Intersection Delay	LOS approach			LOS
					EB	NB	SB		EB	NB	SB	
1	80.5	17	21	32	45.1	125.3	22.8	73.2	D	F	C	E
2	90.5	20	25	35	45.1	76.3	26.2	52.5	D	E	C	D
3	100.5	20	30	40	59.5	57.2	27.3	47.6	E	E	C	D
4	110.5	25	33	42	50.7	60.5	31.8	48.6	D	E	C	D
5	120.5	30	35	45	48.3	69.7	35.1	53.3	D	E	D	D
6	130.5	30	40	50	56.7	62.7	36.2	52.4	E	E	D	D
7	140.5	30	45	55	66.5	59.7	37.4	53.6	E	E	D	D
8	150.5	30	50	60	78.6	58.6	38.7	56.2	E	E	D	E
9	160.5	30	55	65	95.8	58.3	40	60.3	F	E	D	E
10	170.5	30	60	70	126.4	58.7	41.4	67.6	F	E	D	E
11	180.5	30	65	75	199.9	59.3	42.8	84.3	F	E	D	F
12	190.5	30	70	80	286.5	60.2	44.3	104.0	F	E	D	F
13	200.5	30	75	85	373.5	61.2	45.7	123.9	F	E	D	F
14	210.5	30	80	90	459.8	62.4	47.2	143.6	F	E	D	F

Note: in all cases assume yellow ≡ 3.5 and all red ≡ 0
Note 2:the volumes using in these cases the same volume using in analyses :
EBT=97 ,EBL=697 // NBT=1437 ,NBL=437 // SBT=1258 ,SBL=165

The conclusions of this design are as follows:

- Case 3 & 4 have minimum value of intersection delay at 110,120 sec. cycle length. This means the best cycle length can apply for this intersection.
- Although there are more lanes, the LOS is still not greatly improved because there is such a great volume in the through traffic direction in major road.

The conclusions of all these analysis results in Tables 4-2, 4-3 and 4-4 are the traffic volumes so high and need seriously to change the type of intersection.

4.3.2 Design of Interchange

It is not easy to make a decision to change an intersection from an at-grade intersection to a grade-separated intersection. The reasons for the change must be clear in order to persuade government officials. However by showing some predictions and possible solutions for the problem at the intersection, they can be convinced.

4.3.2.1 Factors Considered

Some factors must be taken into account in the analysis before change can be proposed; these factors include [3]:

- Design Designation: It should be determined whether each intersecting highway will be terminated, rerouted or provided with a grade separation or interchange. The main concern being unhindered traffic flow for all junction approaches or for most of them.
- Site Topography: At some sites, a grade separated interchange may be more feasible than an at-grade intersection due to local topographical conditions.
- Traffic Volume: In general the traffic volume of interchanges at cross streets is heavier thus warranting a new design.
- Safety: High traffic volume greatly increases the risk of traffic accidents. A reduction in collisions ensures support for a new type of interchange.

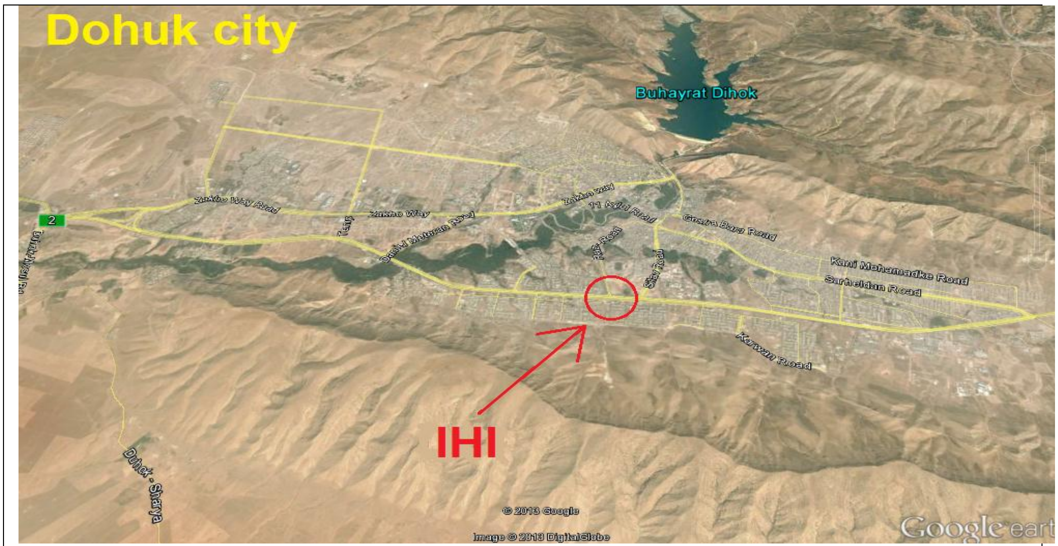
- Congestion: An interchange may be redesigned where the intersection cannot be modified to provide an acceptable level of service due to constant traffic congestion.
- Road-User Benefits: When interchanges are designed and operated efficiently, they significantly reduce the travel time and costs when compared to at-grade intersections. Thus analysis should prove that road-user benefits will exceed the costs over the service life of the interchange.

4.3.2.2 The Effect of Factors on Studied Intersection

The conditions of the Dohuk city site can be compared with the criteria listed in previous section for a new interchange model. This is shown in Table 4-5.

Table 4-5: Comparing between factors with conditions of site studied location

Factors	Description
Site topography	Because of the land is not flat in Dohuk city, It should be take the coordinates of all points by a survey instrument to draw contouring map for the location as shown in Appendix A, B and C including all the pillars and barriers needed for design.
Traffic volumes	The present total traffic volume for (IHI) intersection is 5265 vph and is expected to rise to 13130 vph in 15 years. That requires an efficient interchange type to ensure free flow of traffic as shown in Table 4-1.
Congestion	Through reviewing previous results of the level of service (LOS), the degree of congestion in this intersection is quite evident.
Road-User Benefits	The importance of this junction is that it connects the north and south parts of the city so that the traffic flow is constant (Figure 4-3).



-A-

-B-

Figure 4-3: Google earth image for the location of junction, A: Dohuk city map, B: Location of intersection [14]

4.3.2.3 Interchange Type for Existing Location

Sometime in the future it is necessary to convert the three-way junction into a four-way junction, as shown in the urban city design map for the future in Figure 4-1. The objective of a four-way junction is to keep the free flow for higher volumes at the approach. The predication of traffic volumes in NB and SB have a lot of vehicles per hour as shown in Table 4-1; therefore, they should be the most important consideration in finding the best type of intersection in order to ensure continuous free flow especially for through lanes in these approaches [13].

The simplest type of interchange is diamond interchange as shown in Figure 4-4 and is a suitable type for intersections having few turning movements from the major to minor and connecting with slip roads.

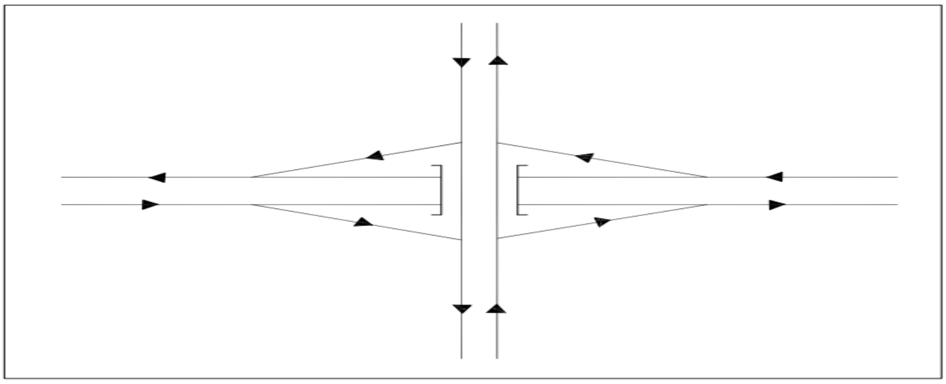


Figure 4-4: Typical diamond interchange

Sometimes the topography and limited use of land (right-of-way) make the designer select some interchange types such as grade-separated roundabout or grade-separated overpass or underpass. All these types have good features to keep the free flow for major lanes. In this study three models of diamond interchange are proposed.

- **Grade separated (Major road underpass) (Model 1)**

This model consists of underpass for the major road and signalization for the minor road with exclusive U turn for major as shown in Figure 4-5.

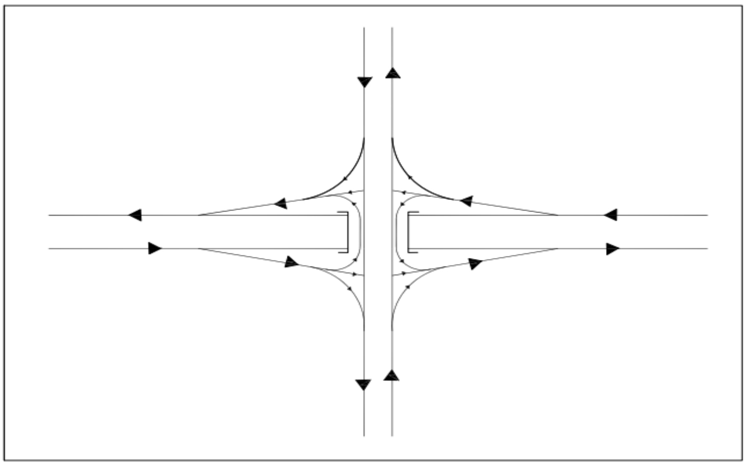


Figure 4-5: Simple drawing for Model 1

Characteristics of this model:

The through direction for major road (NB, SB) has free traffic flow, exclusive U turn lane for (NB, SB), exclusive right turns for all approaches and all ramps and minor road are at the level of the surface ground.

- **Grade separated (Minor road overpass with roundabout) (Model 2)**

In this model, major road separated from the minor road by a roundabout overpass as shown in Figure 4-6.

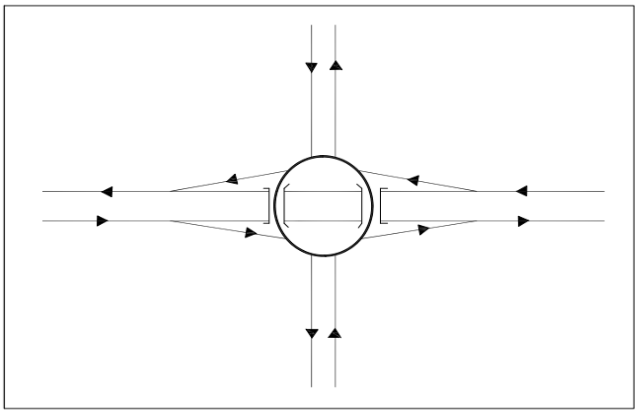


Figure 4-6: Simple drawing for Model 2

Characteristics of this model:

1. Through direction for major road (NB, SB) has free flow traffic.
2. The major road is at the level of the surface ground.
3. Ramps have slopes for acceleration and deceleration.

- **Grade separated (Minor road overpass) (Model 3)**

In this model the minor road is an overpass with signalization as shown in Figure 4-7.

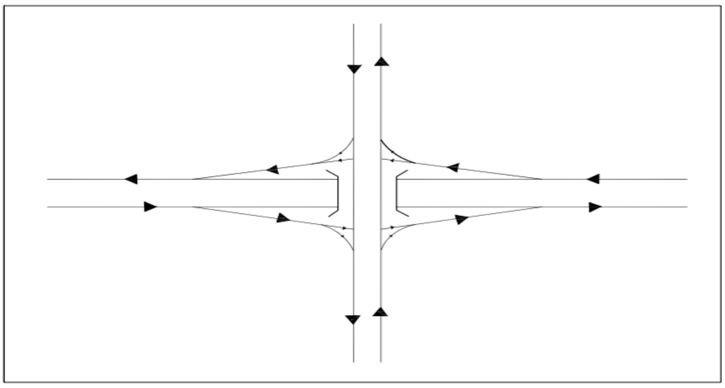


Figure 4-7: Simple drawing for Model 3

Characteristics of this model:

1. Through traffic for the major road (NB, SB) is on the level of the ground and free flowing (major road finishing level close to natural ground level).
2. The minor road is a signalized overpass with ramps.
3. No U turn in this model.

4.4 Geometric Design for Alternative Models

For designing each one of these models, the alignment and profiles for each approach will be drawn and checked for limitations using AASHTO [3]. The easiest way to design all geometric elements is to use Autodesk Land Desktop or AutoCAD Civil 3D. These software programs work with points in three dimensional coordinates and can draw and check with the standards of AASHTO [3]. In this study AutoCAD civil 3D [27] is used because it has many options compared with other software programs and because many unique operations can be performed on this software as shown in Figure4-8.

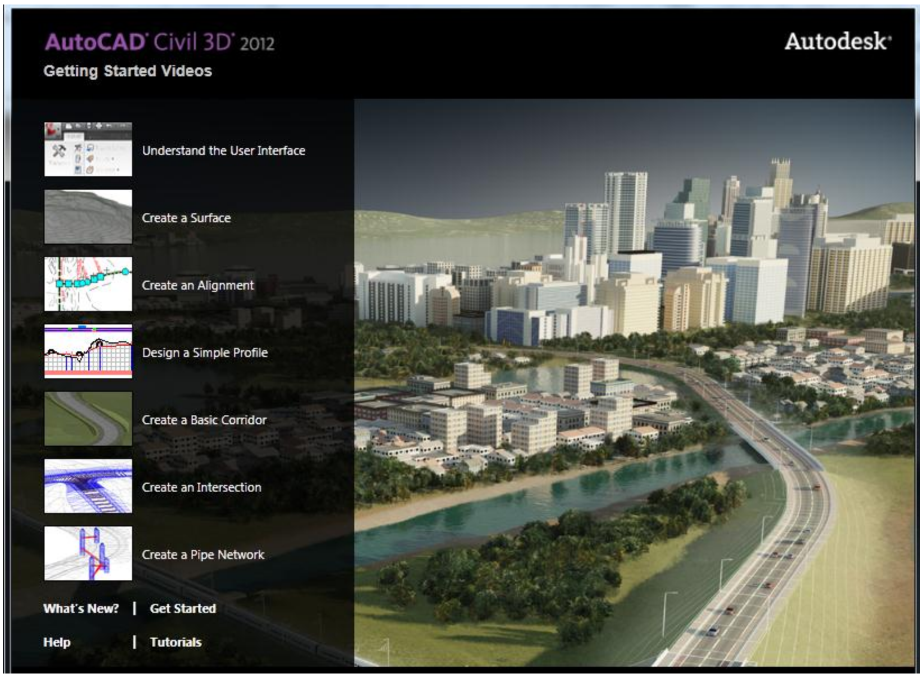


Figure 4-8: AutoCAD Civil 3D [27]

4.4.1 Design for Model 1(Major road as underpass)

Firstly, before starting to design, the center line for each approach should be noted. The design speed for roadway and ramps will be defined later along with the new alignments for each road with its corresponding ramp.

From Table 4-6 the speed for ramp can be chosen depending on the speed available for major road (the design speed for major road is 80 km per hr.). On the other hand the existing points help to know the natural surface ground of location (contour line map). In addition to this, the amount of excavation should be considered to minimize cost during the construction of the interchange. Appendix A shows all the details of model 1 with all the elements used in designing this model.

Table 4-6: Ramp and roadway design speed relationship [3]

Roadway design speed km/h	Ramp design speed km/h
60	30-50
70	40-60
80	40-70
90	50-80
100	50-90

4.4.1.1 Alignment

- The length of major alignment is more than 500 m, two ways; each way has 5 lanes before ramps and changes to 3 lanes (width of lane 3.6m) as shown in Figure 4-8.

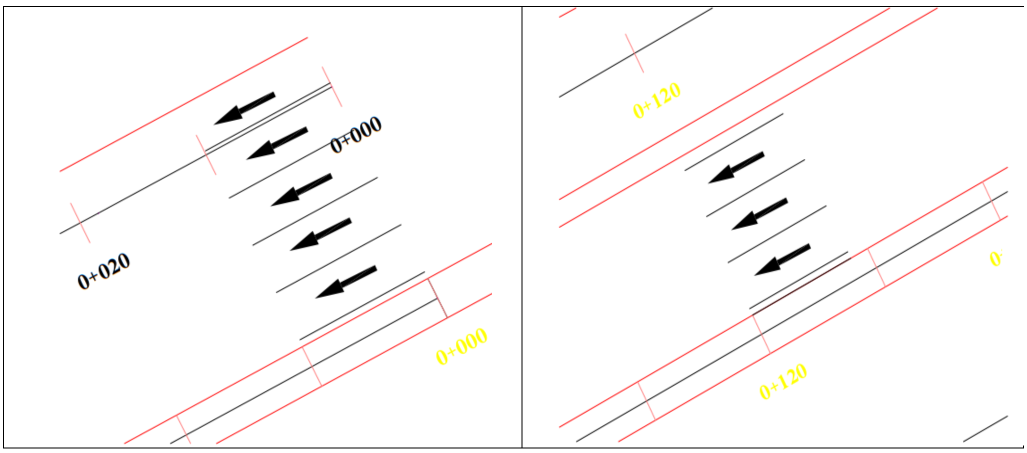


Figure 4-9: Number of lanes of Major road

- The lengths of each of the two minor alignments are more than 200m. The two ways are divided each into 3 lanes.
- Four ramps connect the major and minor roads. Each one has a different length depending on the centerline; each ramp has at least 2 lanes.
- Horizontal curves used in all alignments depend on the design speed as shown in Table 4-7 (design speed for major and minor roads is 80 km/h and for ramps is 40 km/h).

Table 4-7: Minimum Radius when e=6 % [3]

Speed km/h	20	30	40	50	60	70	80
Min. Radius	15	30	55	90	135	195	250

4.4.1.2 Profile

The profile represents the longitudinal section of road with all elevations of natural ground level and finishing level (new design elevation) in each station.

- For the major road the new design profile line has slope coming down, sag vertical curve and slope coming up as shown in Figure 4-10.

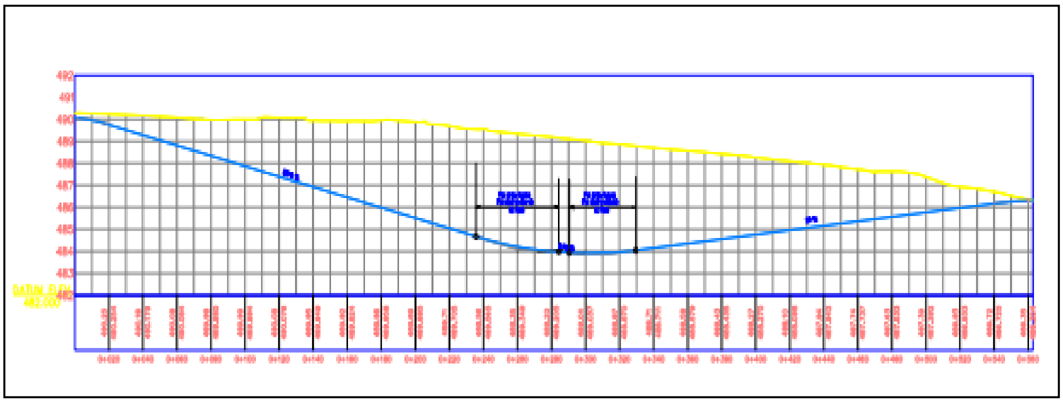


Figure 4-10: Major alignment profile [27]

- Each ramp and minor roads have profiles which have a finishing level close to natural elevation because no more change in elevation for these roads is necessary as in Model 1 (just cut or fill no more than 1 m).

4.4.1.3 Median

- The width of median and shoulder is equal to 3 m and 1.2m in the major road according to AASHTO limitation [3] as shown in Table 4-8.

Table 4-8: Width of median and shoulders for underpass [3]

	Minimum median	Minimum shoulder
road has 4 lanes	3 m	1.2 m
road has 6 lanes	6.6m	3m

- For minor roads the width of median is equal to 2 m.

4.4.1.4 Vertical clearance

Vertical clearance required for the major road is 5m depending on AASHTO limitation:

- Vertical clearance for all structures above road and shoulders must be at least 0.3m greater than the highest legal vehicle [3].
- The minimum clearance according to AASHTO is 4.4m but 5m is recommended in case of snow or ice accumulation.

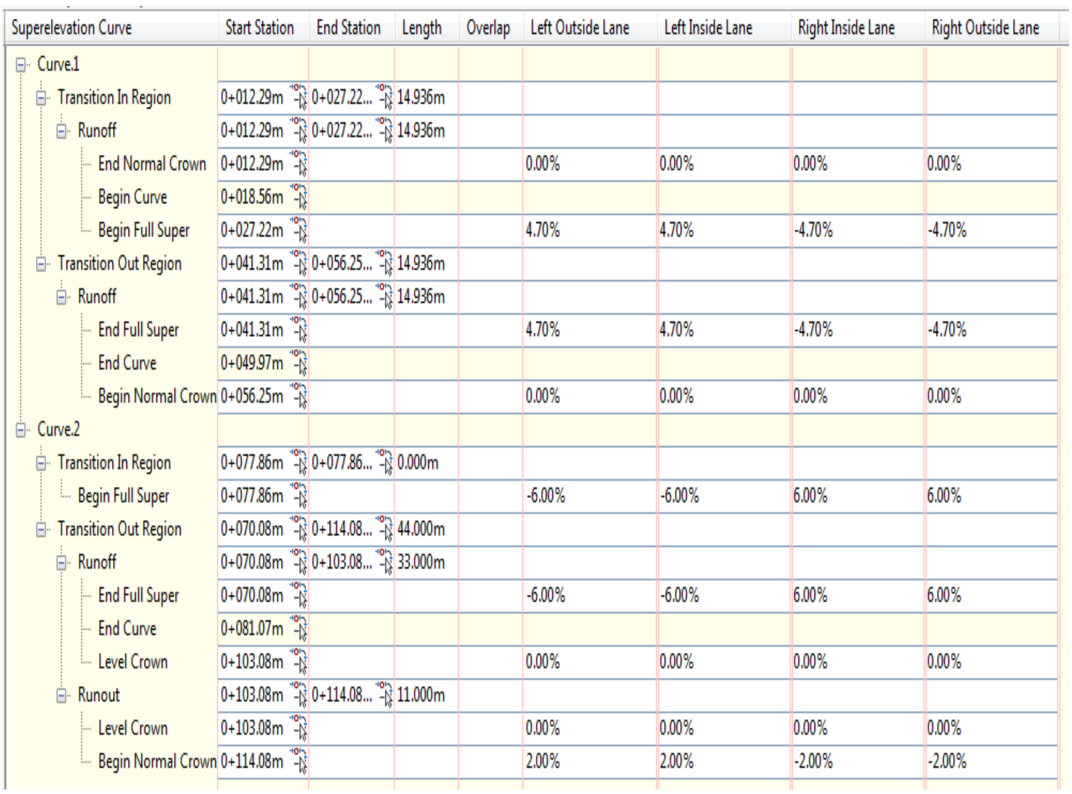
4.4.1.5 Superelevation

Superelevation is very important to provide balance for vehicles in curves especially in the connecting areas between ramps and main roads. This area has some rotation in the longitudinal slope to ensure the water drainage in these areas. The limitation according to AASHTO depends on the radius of curves and speed design as shown in Table 4-9.The superelevation has two parts for design runoff and runout. Figure 4-11 illustrates the design of superelevation for one ramp.

Table 4-9: Values of runoff and runout for two lanes rotated [3]

	Runoff	Runout	Runoff	Runout	Runoff	Runout
Speed	$e_{max} = 4\%$		$e_{max} = 6\%$		$e_{max} = 8\%$	
30	29	14	43	14	57	14
40	31	15	46	15	62	15
50	32	16	49	16	65	16
60	36	18	54	18	72	18
70	39	20	59	20	79	20
80	43	22	65	22	86	22

*e_{max} repersents superelevation rate



Superelevation Curve	Start Station	End Station	Length	Overlap	Left Outside Lane	Left Inside Lane	Right Inside Lane	Right Outside Lane
Curve.1								
Transition In Region	0+012.29m	0+027.22...	14.936m					
Runoff	0+012.29m	0+027.22...	14.936m					
End Normal Crown	0+012.29m				0.00%	0.00%	0.00%	0.00%
Begin Curve	0+018.56m							
Begin Full Super	0+027.22m				4.70%	4.70%	-4.70%	-4.70%
Transition Out Region	0+041.31m	0+056.25...	14.936m					
Runoff	0+041.31m	0+056.25...	14.936m					
End Full Super	0+041.31m				4.70%	4.70%	-4.70%	-4.70%
End Curve	0+049.97m							
Begin Normal Crown	0+056.25m				0.00%	0.00%	0.00%	0.00%
Curve.2								
Transition In Region	0+077.86m	0+077.86...	0.000m					
Begin Full Super	0+077.86m				-6.00%	-6.00%	6.00%	6.00%
Transition Out Region	0+070.08m	0+114.08...	44.000m					
Runoff	0+070.08m	0+103.08...	33.000m					
End Full Super	0+070.08m				-6.00%	-6.00%	6.00%	6.00%
End Curve	0+081.07m							
Level Crown	0+103.08m				0.00%	0.00%	0.00%	0.00%
Runout	0+103.08m	0+114.08...	11.000m					
Level Crown	0+103.08m				0.00%	0.00%	0.00%	0.00%
Begin Normal Crown	0+114.08m				2.00%	2.00%	-2.00%	-2.00%

Figure 4-11: Some of superelevation results by AutoCAD Civil 3D for one ramp [27]

4.4.1.6 Longitudinal Distance to Attain Grade Separation

The distance from ground level to underpass level is critical especially for roads inside urban areas. The distance used in this model equal to 250 meter calculated from Appendix E (if vertical clearance (H) =5m, grade=3% and design speed =80 km/hr).

4.4.2 Design for Model 2 (Minor road with roundabout)

To design this model, the National Cooperative Highway Research Program (NCHRP report 672 for Roundabout) [28] was followed. By using the different elevations in the natural ground surface between major and minor roads, a minor overpass with ramps can be built, having slopes as shown in Appendix B which includes a detailed illustration of the second model.

4.4.2.1 Number of Lanes Required for Roundabout

When calculating the volumes entering the roundabout from the minor road and ramps of the major road 5633 vehicles per hour are observed. Because of this excessive amount of traffic, 3 lanes are assumed as shown in Tables 4-10 which illustrates how to determine the number of lanes based on volume of vehicles [28].

Table 4-10: Type of roundabout with the volumes of vehicles [28]

Volume Range	No. of Lanes
0 to 1000 veh/h	Single-lane
1000 to 1300 veh/h	Single-lane or two-lane
1300 to 1800 veh/h	Two-lane
Above 1800 veh/h	More than two

4.4.2.2 Diameter of Inscribed Circle for Roundabout

The diameter of the roundabout is very important for design and safety. In this study, 3 lanes with two circles 80m diameter are used and are connected with tangent lines as shown in Figure 4-12. Table 4-11 illustrates the relation between the number of lanes and the diameter of a roundabout according to NCHRP [28].

Table 4-11: Radius of roundabout types [28]

Roundabout type	Diameter range (m)
Mini-Roundabout	14 to 27 m
Single-Lane Roundabout	40 to 55 m
Multilane Roundabout (2 lanes)	50 to 67 m
Multilane Roundabout (3 lanes)	67 to 91 m

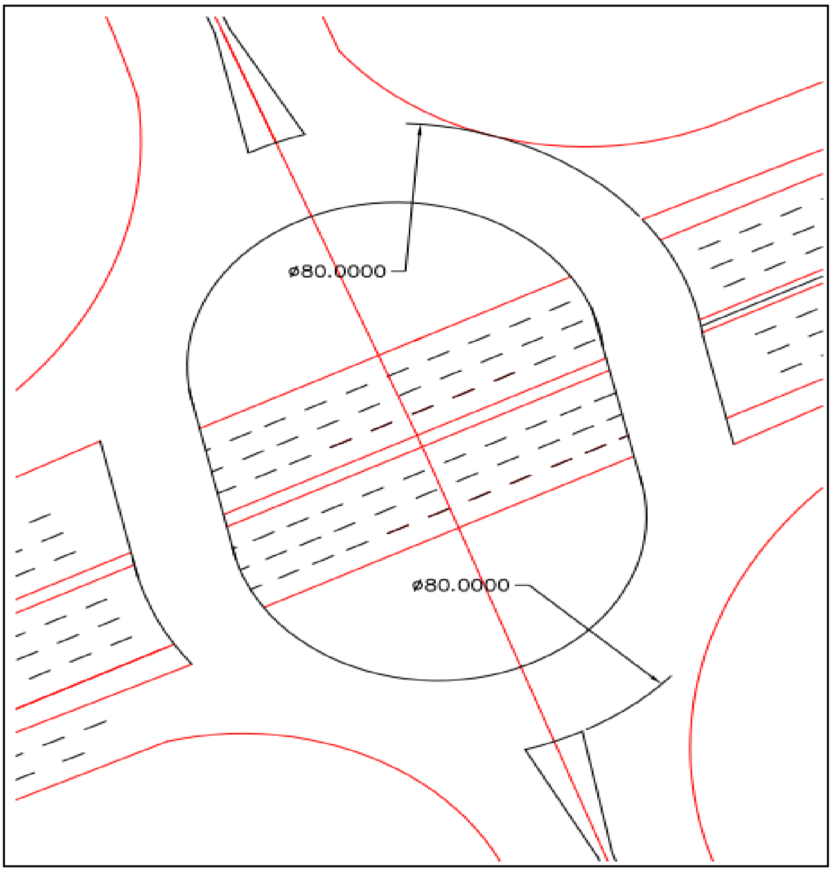


Figure 4-12: Dimension of Circles used in Roundabout [27]

4.4.2.3 Design of the Splitter Islands

Splitter islands assist to control speed and guide traffic into the roundabout; its dimensions are designed according to NCHRP report [28] and AASHTO [3]. Figure 4-13 shows one island designed according to standard specification.

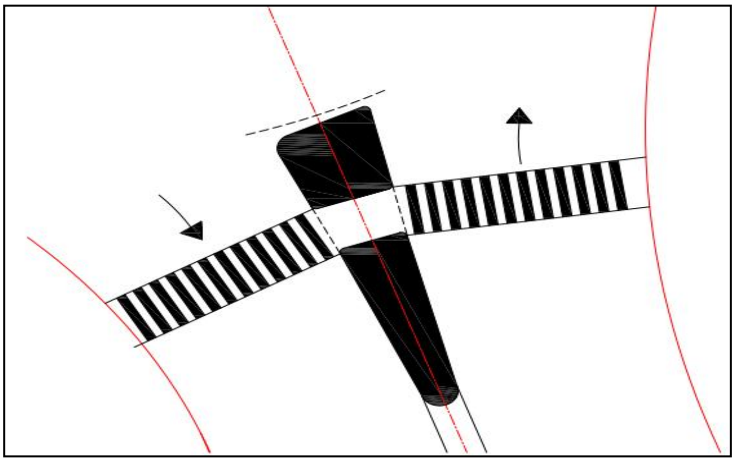


Figure 4-13: Minor roads' island connecting with roundabout [28]

4.4.2.4 Design of Some Roundabout Elements

The Comparing between some elements used in this model with minimum standards can be seen in Table 4-12.

Table 4-12: Comparison between specification and the measurement used in the model 2 [28]

Element	Min. dimension (m)	Dimension use in model (m)
Width Entry	11 To 13.7 for 3 lanes	13.5
Circulatory roadway width	12.8 To14.6 for 3 lanes	12.5
Entry radii	$\geq$20m	Using more than 20

- Width entry depends on the number of lanes (3 lanes).
- In multilane roundabouts, the circulatory roadway width depends upon the number of lanes and the types of vehicles.
- Entry radii for multilane roundabouts should typically exceed 20 m to encourage adequate natural paths and avoid sideswipe collisions on entry.

According to one research which dealt with roundabout Design Standards from City of Colorado [29] some standards for designing roundabout are spelled out as shown in Table 4-13 and Figure 4-14.

Table 4-13: Maximum Radii [29]

Radius	**Multilane Roundabout (Radius Max.) m**
R1 Entry	244.5 - 286.5
R2 Circulating	286.5 – 338
R3 Exit	244.5 - 286.5
R4 Left turn	286.5 – 338
R5 Right turn	244.5 - 286.5

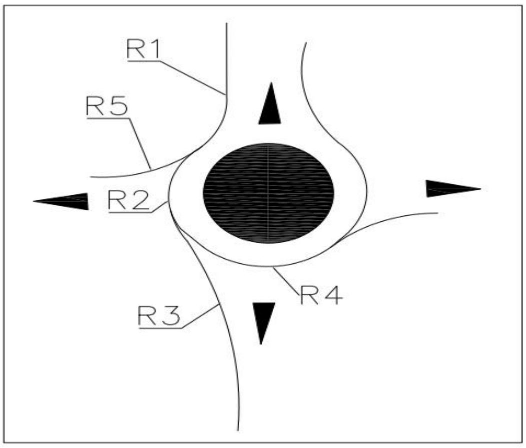


Figure 4-14: Radii of elements in roundabout [29]

4.4.2.5 Number of Lanes for Major and Minor Roads in Model 2

The number of lanes in the major road is the same as in Model 1:

- For the major road 5 lanes before ramps changes to 3 lanes after ramps.
- For the minor road there are 3 lanes in each approach.
- Ramps have slopes because they connect the major at grade with the minor overpass. The width of each ramp is at least two lanes.

4.4.2.6 Design of Ramps

To design the elements of the ramps NCHRP report 730 [30] was used. Table 4-14 show the maximum grad can be used according to design speed. The design speed for ramp used equal to 40 km/hr.

Table 4-14: Relation between design speed and maximum grade allowable [30]

Design speed (km/h)	Maximum grade
24-40	6-8 %
40-48	5-7 %
64	4-6 %
72.5-80	3-5 %

From Appendix B can be see the profile of finishing level with grade.Likewise in Figure 4-15 and Table 4-5 can be see the ramps with the gradient used in this model.

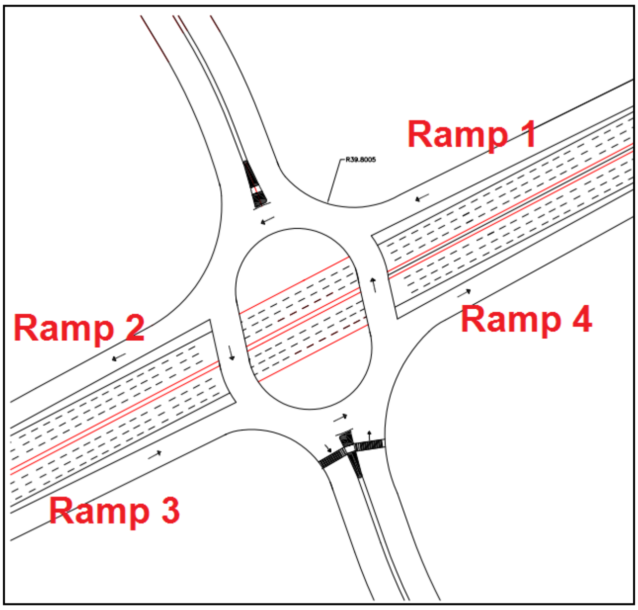


Figure 4-15: Ramps in Model 2

Table 4-15: The gradient used in model 2

Ramps	Actual grade use in model
Ramp 1	2.09 %
Ramp 2	-3.47 %
Ramp 3	3.76 %
Ramp 4	-2.10 %

The length of acceleration and deceleration in ramps considered so important, Table 4-16 and Table 4-17 used to compare the measurement used in design with the minimum values according to AASHTO [2].

Table 4-16: Comparing lengths of acceleration used in model 2 with AASHTO [3]

Ramps	Actual acceleration length	Green Book minimum acceleration Length
Ramp 1	240 m	145 m
Ramp 3	225 m	145 m

Table 4-17: Comparing lengths of deceleration used in model 2 with AASHTO [3]

Ramps	Actual deceleration length	Green Book minimum deceleration Length
Ramp 2	230 m	100 m
Ramp 4	240 m	100 m

4.4.3 Design for Model 3 (Minor road overpass with two signalized intersections)

In this model:

- The major road is at grade level and has continuous free flow for each through direction.
- Four ramps connecting the major and minor roads.
- The minor road has on overpass.
- Two signalized intersections where the two ramps connect with minor.

Appendix C shows the model with all its detailed alignments, profiles, and top view with all dimensions.

4.4.3.1 Design at Intersection

This version of AutoCAD Civil 3D 2012 has option to design intersections according to AASHTO 2004. It defines the intersection point as shown in Figure 4-16 and Figure 4-17.

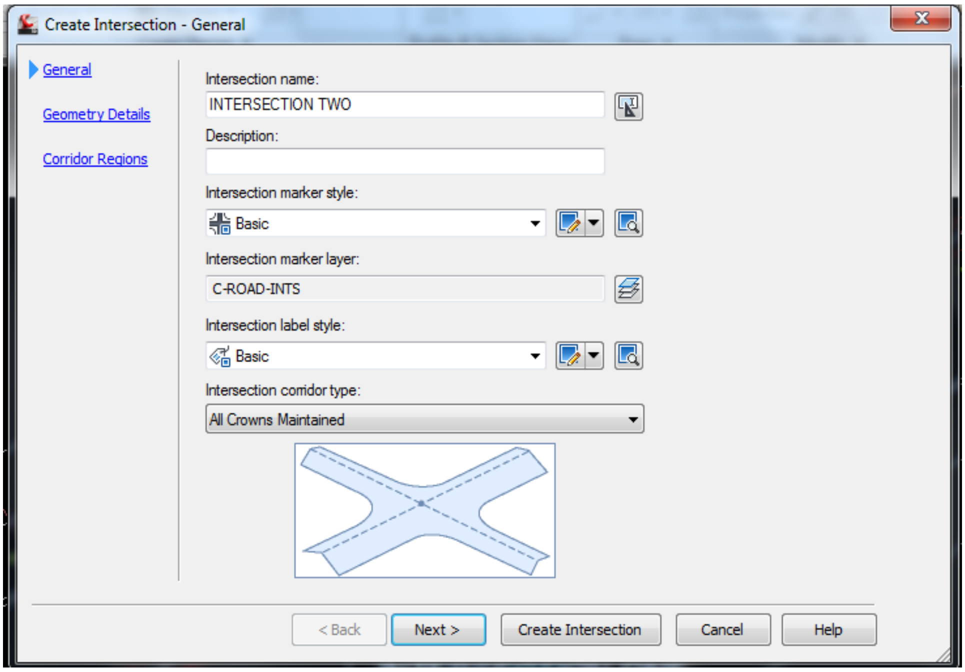


Figure 4-16: Create intersection by AutoCAD Civil 3D [27]

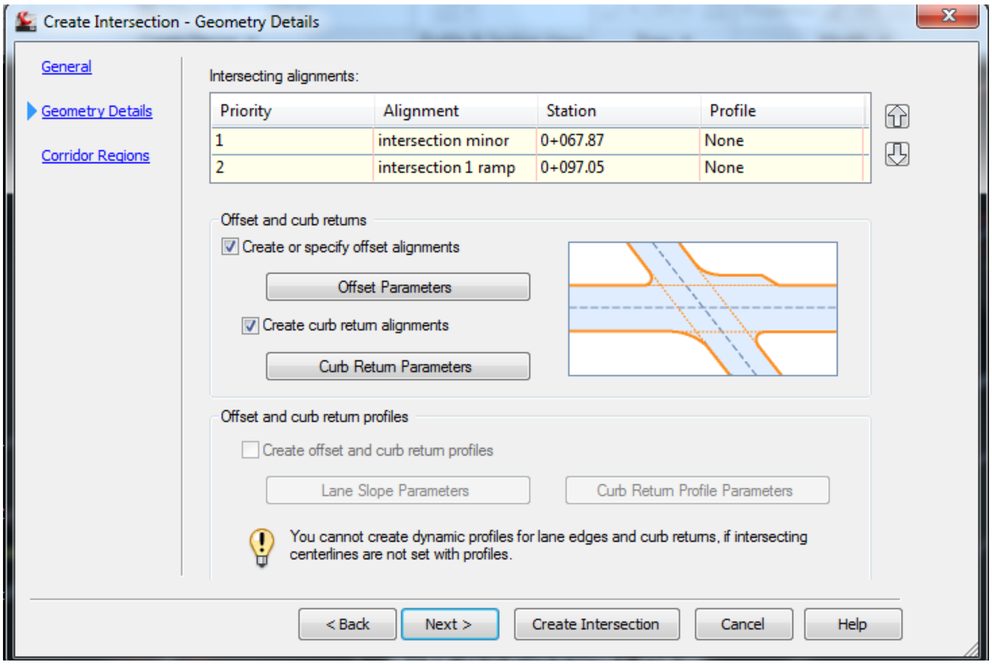


Figure 4-17: Input the details to create intersection [27]

- In this model, there are no triangle islands for right-turns on the minor road to minimize the cost compared to four exclusive right-turns.
- According to specification of AASHTO [3], vehicle type of WB-30T was used to design radii of curb in model 3 as shown in table 4-18.

Table 4-18: Recommended radii for curbs [3]

Angle of turn (degree)	Design vehicle	Radius(m)	Offset (m)	Tapper(L:T)
60	WB-30T	29	0.8	15:1
75	WB-30T	26	1.0	15:1
90	WB-30T	25	0.8	15:1
105	WB-30T	22	1.0	15:1
120	WB-30T	20	1.1	15:1

As shown in Figure 4-18 & Figure 4-19, intersection two and intersection one respectively had angles between ramps and the minor road of 71° and 108°; therefore, the radius and tappers were drawn according to Table 4-18. The minimum radius is no less than 26m and 22m respectively and tapper had 15m length.

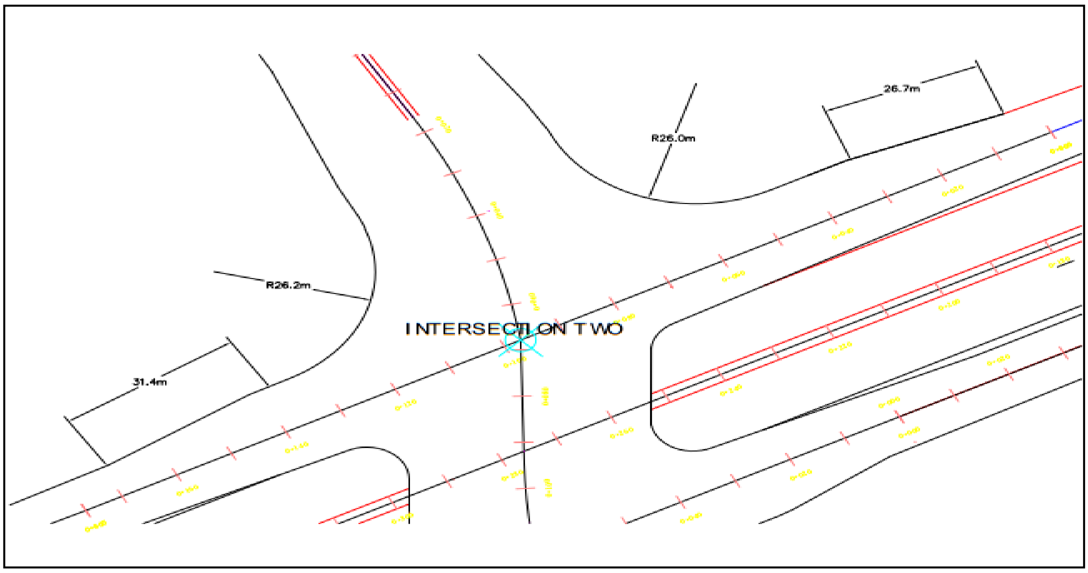


Figure 4-18: Intersection two connecting ramp1 with ramp2 with minor road [27]

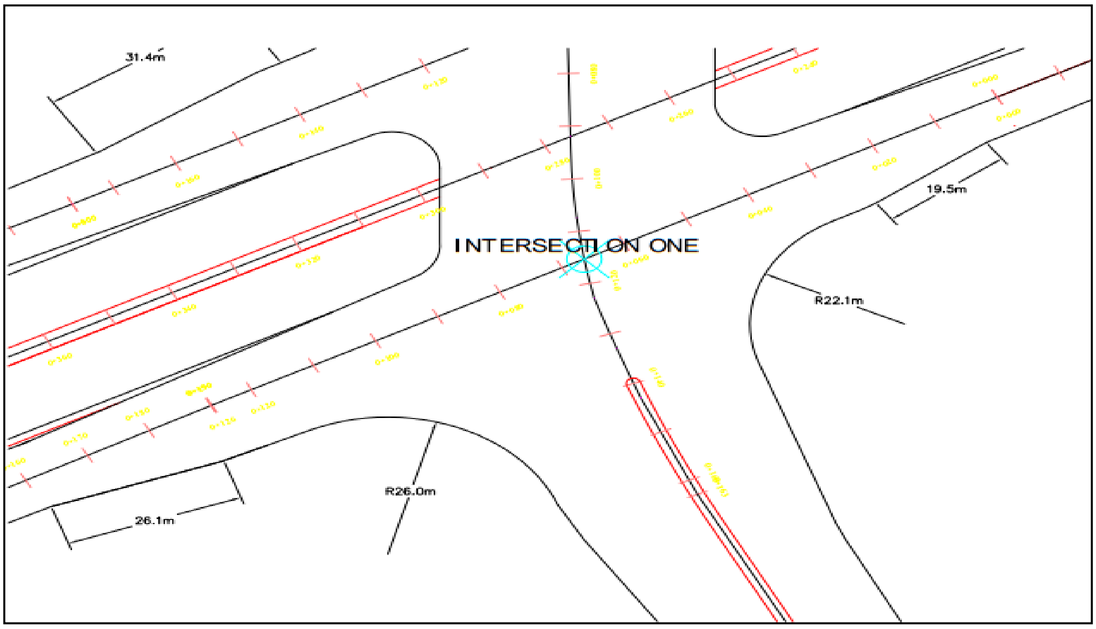


Figure 4-19: Intersection one connecting ramp3 with ramp4 with minor road [27]

4.4.3.2 Other Design Elements

- Major and minor roads have the same number of lanes as in model two.
- Ramps have slopes.

- Traffic signals are installed for each direction and are shown in the chapter five to ensure a better cycle length.

4.5 Selection of the Best Model

The successful of selection the best model should simulate these models with one computer software. In the next chapter the software selected was explained and the scenarios for each model.

Chapter 5

DISCUSSION OF SIMULATION RESULTS

5.1 Introduction

In order to compare the efficiency of each design, they should be simulated and the results checked, then the best one can be selected for the design of the road. There are several softwares that simulate reality and future traffic data, each software had some advantages compared with others.

5.2 Comparison of CORSIM, VISSIM and PARAMICS

There are many simulation packages which can be used to simulate traffic data such as CORSIM, VISSIM and PARAMICS, as shown in Table 5-1 describe the advantages of each software [25].

Table 5-1: Comparison between three simulation software according to their functions [25]

Function	CORSIM	VISSIM	PARAMICS
Network limitation	900 nodes, unlimited links and vehicles	None, except for memory limit on computer	None, except for memory limit on computer
Traffic control	Yield sign, stop sign, pre-timed actuated signal, ramp metering control roundabout	Priority rules, stop sign, pre-timed signal, actuated signal, roundabout	Priority junction, stop sign, pre-timed signal, actuated signal, roundabout
Multi-model transportation	Car, trucks, pedestrian	Car, trucks, bus, rail, tram, bike and pedestrian	Car, trucks, bus and Pedestrian
Measure of performance	Traffic volume Delay time Travel time Control delay Stopped delay Queue time Queue length Vehicle speed	Traffic volume Vehicle speed Mean speed Travel time Total delay Stopped delay Average queue length Maximum queue length Vehicle stops within the queue Bus/ tram wait time	Point/ link flow Point/ link speed Headway Occupancy Acceleration Density Link/ bus/ total delay Turn/queue/ link counts

Based on the comparisons it appears that the VISSIM model is the most efficient simulation model because VISSIM is used for the evaluation of various alternatives and offers excellent modeling of complicated networks using priority rules for roundabout.

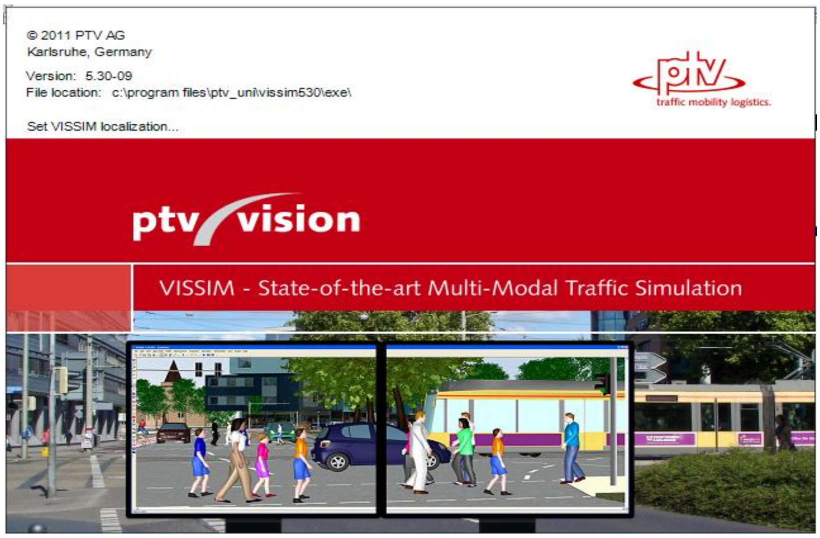


Figure 5-1: VISSIM software [4]

5.3 Applying Scenarios in VISSIM

The application of some scenarios can be discussed in this section based on the models proposed in chapter four. Different alternatives can be proposed in order to get the best result of delay and travel time. These also take into account projections for traffic volume in 2028 as seen in Figure 4-2.

5.3.1 Applying Scenarios for Model 1

First, the roads were defined and a name and volume was given for each one to simply organize input and output data as shown in Table 5-2 and Figure 5-2

Table 5-2: Define roads for study

Road	Description	Input projected traffic Volume
A	Exit ramp connecting major road with Minor (SB)	1380
B	Minor road (EB)	2250
C	Exit ramp connecting Major with Minor (NB)	650
D	Minor road (WB)	2250

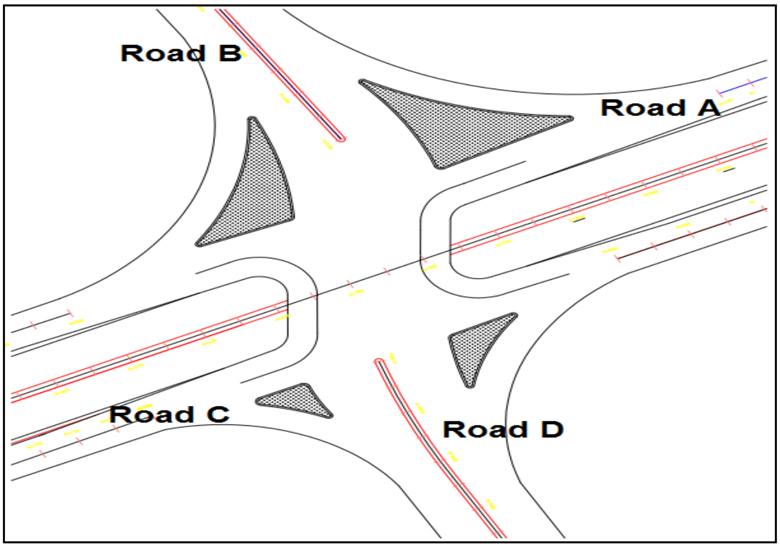


Figure 5-2: Define roads in model 1

In VISSIM the traffic volume is distributed for each road. Figure 5-3 illustrates the distribution according to traffic data.

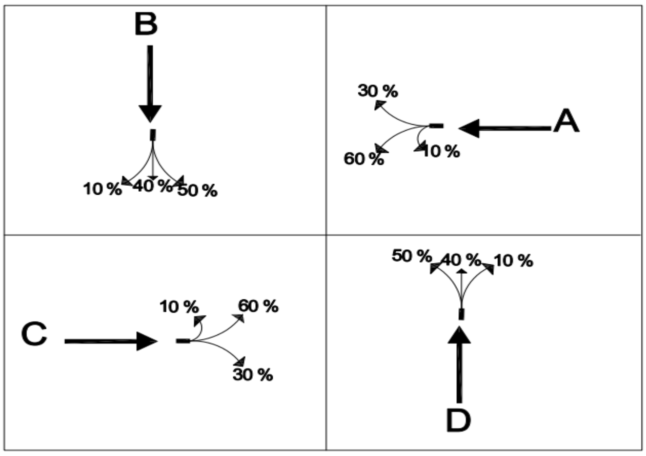


Figure 5-3: Distribution of traffic volume for Model1

There are some variables that can be changed in the model design to obtain better results for delay, which are considered as the measurement of level of service (LOS), such as cycle length, numbers of lanes, waiting in intersection and number of phases used. On the other hand the results determined from VISSIM were for travel time and delay as shown in every scenario table then using Eq. (5.1) to find the junction weighted average delay (J.W.A.D) for each scenario.

Junction weighted average delay: is the ratio between total delays of all approaches to the total number of vehicles in these approaches.

$$\text{junction weighted average delay} \equiv \frac{\sum(\text{delay per approach})_{A,B,C,D}}{\sum(\text{number of veh.})_{A,B,C,D}} \qquad .Eq.(5.1)$$

1st scenario: in this case the cycle length was assuming 178 sec. with 3, 4, 1 and 4 number of lanes in A, B, C and D roads respectively as shown in the Table 5-3.

Table 5-3: First scenario signalization data for Model 1

Road	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.
A	178	119.4	107.1	109	11673.9	30	3
B		114.6	91.4	256	23398.4	55	4
C		108	95.6	42	4015.2	30	1
D		55.7	41.4	310	12834	53	4
Intersection total values				717	51921.5		
junction weighted average delay ≡ 72.41							

2nd scenario: in this case assuming the cycle length 150 sec. with the same number of lanes as in the first scenario as shown in Table 5-4.

Table 5-4: Second scenario signalization data for Model 1

Road	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.
A	150	92.1	79.9	97	7750.3	25	3
B		75.1	51.9	282	14635.8	50	4
C		77.6	65.1	51	3320.1	25	1
D		48.7	34.4	338	11627.2	50	4
Intersection total values				768	37333.4		
junction weighted average delay ≡ 48.61							

3rd scenario: in this case, assuming the cycle length was 140 sec. as shown in with the same numbers of lanes as shown in Table 5-5.

Table 5-5: Third scenario signalization data for Model 1

Road	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.
A	140	92	97.8	125	12225	25	3
B		80.3	57	277	15789	45	4
C		63.2	50.7	48	2433.6	25	1
D		53.6	39.4	323	12726.2	45	4
Intersection total values				773	43173.8		
junction weighted average delay ≡ 55.85							

4th scenario: using 130 sec. cycle length was decrease the time of the junction weighted average delay as well as shown in Table 5-6.

Table 5-6: Fourth scenario signalization data for Model 1

Road	cycle length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.
A	130	81.6	69.3	120	8316	23	3
B		79.1	55.7	310	17267	43	4
C		66	53.3	51	2718.3	21	1
D		44.4	30.1	302	9090.2	43	4
Intersection total values				783	37391.5		
junction weighted average delay ≡ 47.75							

5th scenario: cycle length assumed was 120 sec. while was keeping the same number of lanes as the first scenario as shown in Table 5-7.

Table 5-7: Fifth scenario signalization data for Model 1

Road	cycle length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.
A	120	86.6	74.6	105	7833	20	3
B		70.4	47.1	295	13894.5	40	4
C		62.1	49.5	56	2772	20	1
D		40.6	26.4	339	8949.6	40	4
Intersection total values				795	33449.1		
junction weighted average delay ≡ 42.07							

6th scenario: in this case, assume 115 sec. as a minimum value of cycle length with the same number of lanes as the first scenario as shown in Table 5-8.

Table 5-8: Sixth scenario signalization data for Model 1

Road	cycle length	Travel time (sec.)	delay	# Veh.	total delay per approach	Green (sec.)	# of lanes at inter.
A	115	92.7	80.4	122	9808.8	20	3
B		66.0	42.7	294	12553.8	40	4
C		71.6	59.1	50	2955	17	1
D		41.3	27.0	332	8964	38	4
Intersection total values				798	34281.6		
junction weighted average delay ≡ 42.95							

7th scenario: in this case, assume 117 sec. as a value of cycle length with the same number of lanes as the first scenario as shown in Table 5-9.

Table 5-9: Seventh scenario signalization data for Model 1

Road	cycle length	Travel time (sec.)	delay	# Veh.	total delay per approach	Green (sec.)	# of lanes at inter.
A	117	88.2	76	119	9044	20	3
B		65.6	42.3	285	12055.5	40	4
C		68.2	55.6	50	2780	19	1
D		40.4	26.2	341	8934.2	38	4
Intersection total values				795	32813.7		
Junction weighted average delay ≡ 41.27							

8th scenario: because of the little difference for junction weighted average delay the change of cycle length was by a second to obtain better accuracy as shown in Table 5-10.

Table 5-10: Eighth scenario signalization data for Model 1

Road	cycle length	Travel time (sec.)	delay	# Veh.	total delay per approach	Green (sec.)	# of lanes at inter.
A	116	90.5	78.2	120	9384	20	3
B		66.7	43.4	287	12455.8	40	4
C		72.4	59.8	50	2990	18	1
D		41.0	26.7	339	9051.3	38	4
Intersection total values				796	33881.1		
Junction weighted average delay ≡ 42.56							

9th scenario: assume 118 sec. for cycle length while keeping the same numbers of lanes as shown in Table 5-11.

Table 5-11: Ninth scenario signalization data for Model 1

Road	cycle length	Travel time (sec.)	delay	# Veh.	total delay per approach	Green (sec.)	# of lanes at inter.
A	118	90.5	77.5	118	9145	20	3
B		66.7	40.2	297	11939.4	40	4
C		72.4	47.8	51	2437.8	20	1
D		41.0	28.9	338	9768.2	38	4
Intersection total values				804	33290.4		
Junction weighted average delay ≡ 41.40							

10th scenario: in this case, cycle length was 118 sec. but changed the numbers of lanes waiting in the intersection on road C from one to two lanes as shown in Table 5-12.

Table 5-12: Tenth scenario signalization data for Model 1

Road	cycle length	Travel time (sec)	delay	# Veh.	total delay per approach	Green (sec.)	# of lanes at inter.
A	118	89.6	77.5	126	9765	20	3
B		63.3	40	295	11800	40	4
C		38.2	25.7	53	1362.1	20	2
D		45.6	31.3	338	10579.4	38	4
Intersection total values				812	33506.5		
Junction weighted average delay ≡ 41.26							

11th scenario: in this case, assume 119 sec for cycle length. And keep the same numbers of lanes as the tenth scenario as shown in Table 5-13.

Table 5-13: Eleventh scenario signalization data for Model 1

Road	cycle length	Travel time (sec)	delay	# Veh.	total delay per approach	Green (sec.)	No of lanes at inter.
A	119	84.9	72.5	126	9135	20	3
B		75.7	37.8	299	11302.2	40	4
C		35.8	25.8	54	1367.4	20	2
D		47.1	31.8	339	10780.2	39	4
Intersection total values				818	32584.8		
Junction weighted average delay ≡ 39.86							

5.3.2 Conclusion for Results of Model 1

Two variables affect the selection of the best scenarios. The first one is delay and the second one is travel time. Figure 5-4 shows the delays for each road. When comparing the best results in Figure 5-4 with Table 3-9 (The values of delay with LOS in signalized intersection) the 11th scenario has the least amount of delay so that the level of service can appear as:

- Road A...LOS (E), Road B .LOS (D), Road C & D...LOS(C).

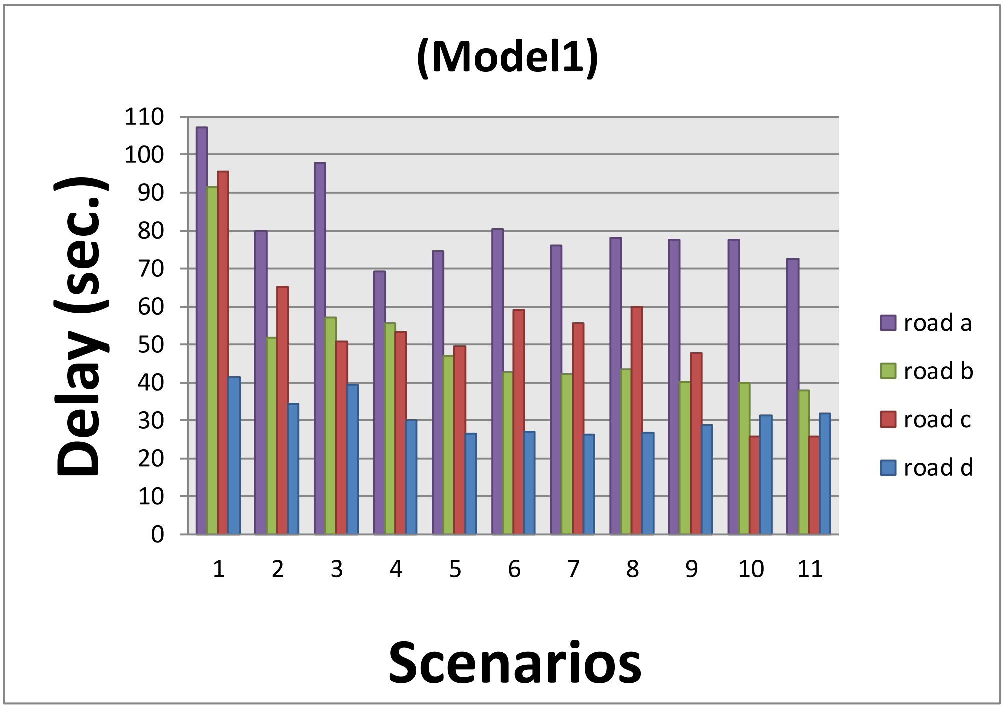


Figure 5-4: Delay diagram for each road in Model 1

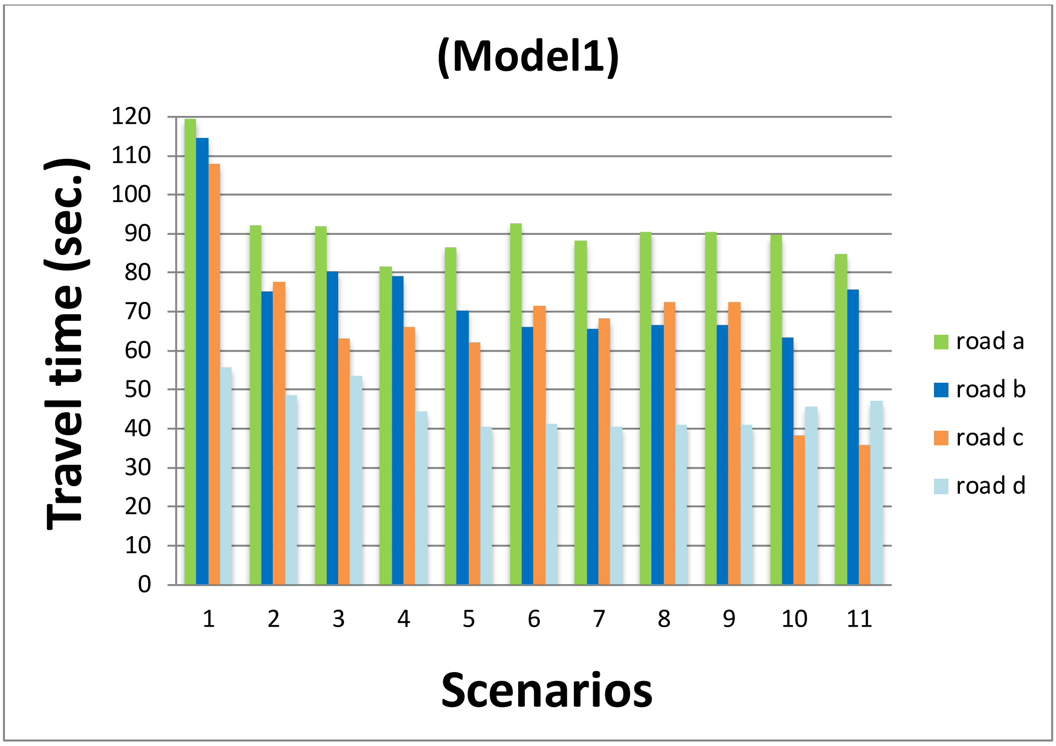


Figure 5-5: Travel time for each road in Model 1

The summary of all scenarios for Model 1 can be seen in appendix D

5.3.3 Applying Scenarios for Model 2

VISSIM software had good features for establishing priority between two or three ways. Because of this feature could help in the scenarios to obtain the minimum delay results. The change of some geometric elements, such as add lanes or add traffic signals, was being done. Define of roads studied in this model can be seen in Figure 5-6 and the distribution of traffic volumes in this roads can be shown in Figure 5-7.

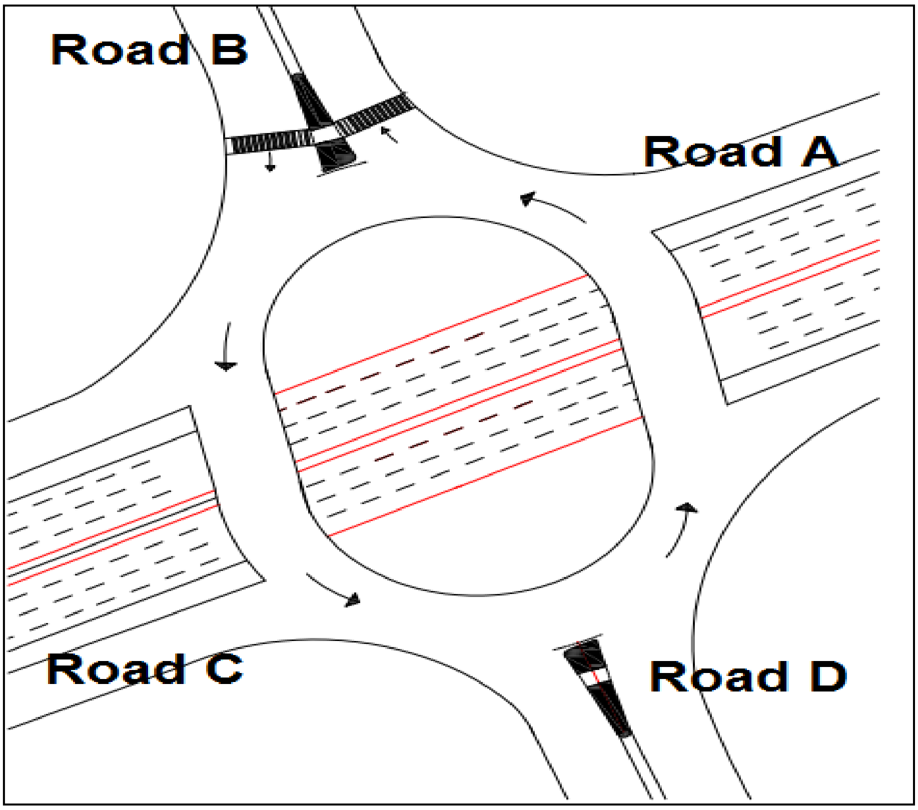


Figure 5-6: Define roads in Model 2

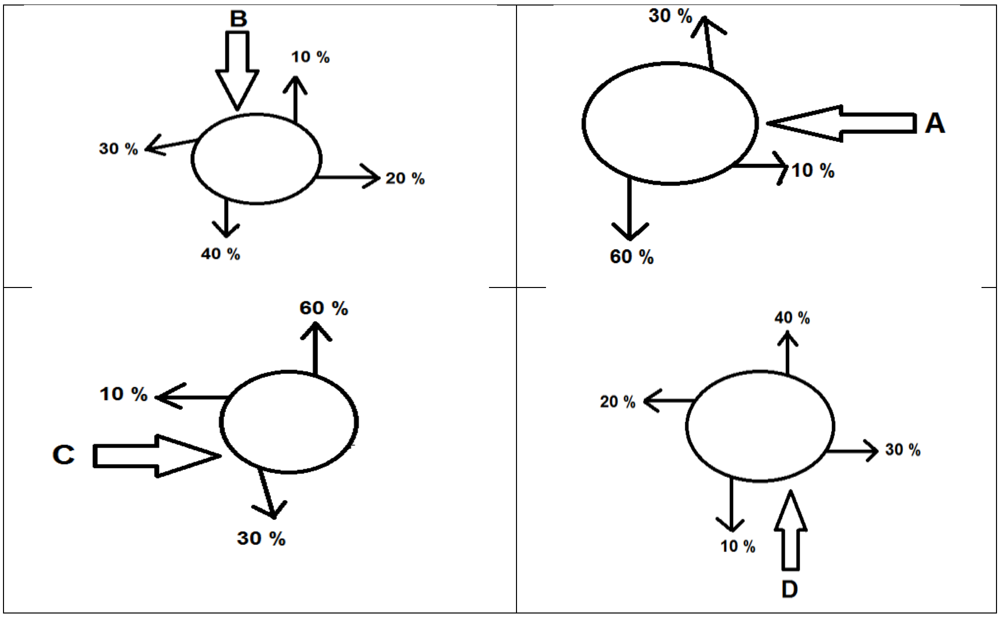


Figure 5-7: Distribution of traffic volume for Model 2

1st Scenario: in this case the priority was given to all roads that enter the roundabout the results can be seen in Table 5-14.

Table 5-14: First scenario delay data for Model 2

Road	Travel time (sec.)	Delay	# Veh.	Total delay per approach	LOS
A	120.4	104.8	96	10060.8	F
B	27.9	8.2	323	2648.6	A
C	16.2	2.6	77	200.2	A
D	91.5	76.9	133	10227.7	F
Intersection total values			629	23137.3	
Junction weighted average delay ≡ 36.78					

2nd Scenario: in this scenario the priority was given to road A and road C as shown in Table 5-15.

Table 5-15: Second scenario delay data for Model 2

Road	Travel time (sec)	delay	# Veh.	total delay per approach	LOS
A	19.1	5.8	195	1131	A
B	93.0	73.8	205	15129	F
C	14.6	1.1	73	80.3	B
D	49.2	35.2	195	6864	E
Intersection total values			668	23204.3	
Junction weighted average delay ≡ 34.7369					

3rd Scenario: in this scenario the priority was given to road B and road D as shown in Table 5-16.

Table 5-16: Third scenario delay data for Model 2

Road	Travel time (sec.)	Delay	# Veh.	Total delay per approach	LOS
A	127.3	111.9	82	9175.8	F
B	23.8	3.9	333	1298.7	A
C	26.2	12.6	76	957.6	B
D	101.7	87.4	150	13110	F
Intersection total values			641	24542.1	
Junction weighted average delay ≡ 38.28					

4th Scenario: the priority was given to vehicles inside roundabout as shown in Tables 5-17.

Table 5-17: Fourth scenario delay data for Model 2

Road	Travel time (sec.)	Delay	# Veh.	Total delay per approach	LOS
A	62.8	46.2	155	7161	E
B	80.4	48.5	243	11785.5	E
C	22.4	9.0	73	657	A
D	19.7	7.4	228	1687.2	A
Intersection total values			699	21290.7	
Junction weighted average delay ≡30.45					

5th Scenario: in this case added an exclusive right turns to each approach with priority for vehicles inside roundabout as shown in Table 5-18.

Table 5-18: Fifth scenario delay data for Model 2

Road	Travel time (sec.)	Delay	# Veh.	Total delay per approach	LOS
A	26.4	12.9	115	1483.5	B
B	58.9	39.3	202	7938.6	E
C	28.6	14.9	72	1072.8	B
D	21.7	7.3	227	1657.1	A
Intersection total values			616	12152	
Junction weighted average delay ≡19.72					

5.3.4 Conclusion for Model 2

When compare the results with the specification of highway capacity manual [2] as shown in Table 5-19, can find that the 5th scenario is the best one in all approaches except for Road B whose level of service (LOS) is E.

Table 5-19: Level-of-service criteria for roundabout HCM [2]

Delay (sec.)	Level of service (LOS)
0-10	A
>10-15	B
>15-25	C
>25-35	D
>35-50	E
>50	F

Figure 5-8 and Figure 5-9 illustrate the results of delay and travel time of Model 2, the summery for all scenarios of Model 2 can be seen in Appendix D.

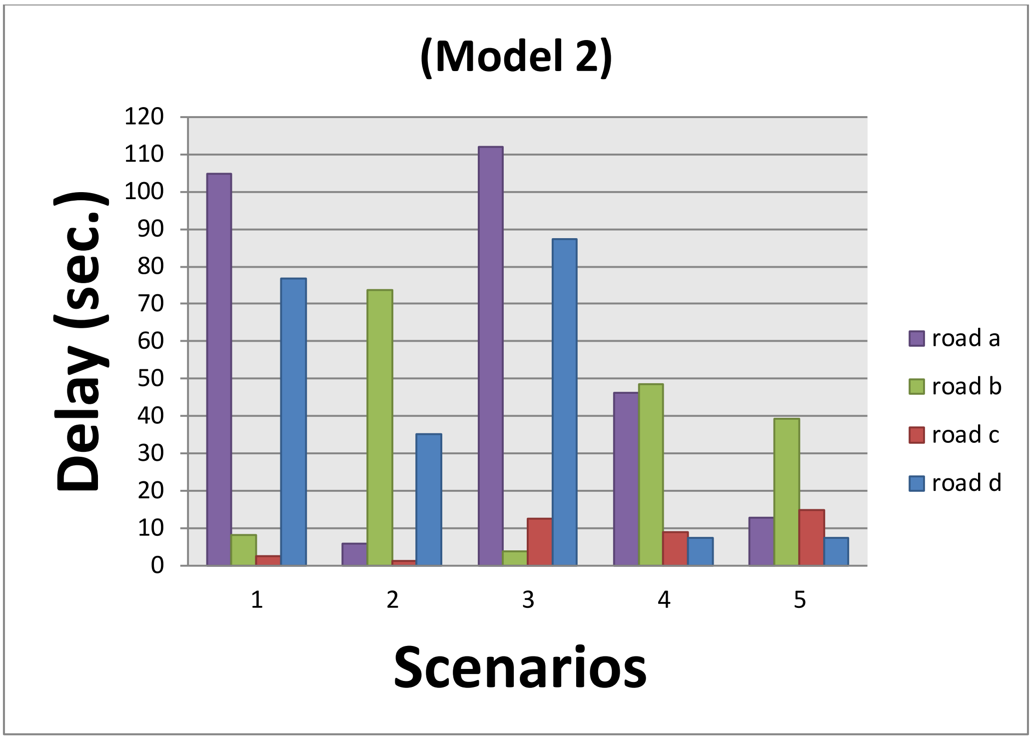


Figure 5-8: Delay diagram for each road in Model 2

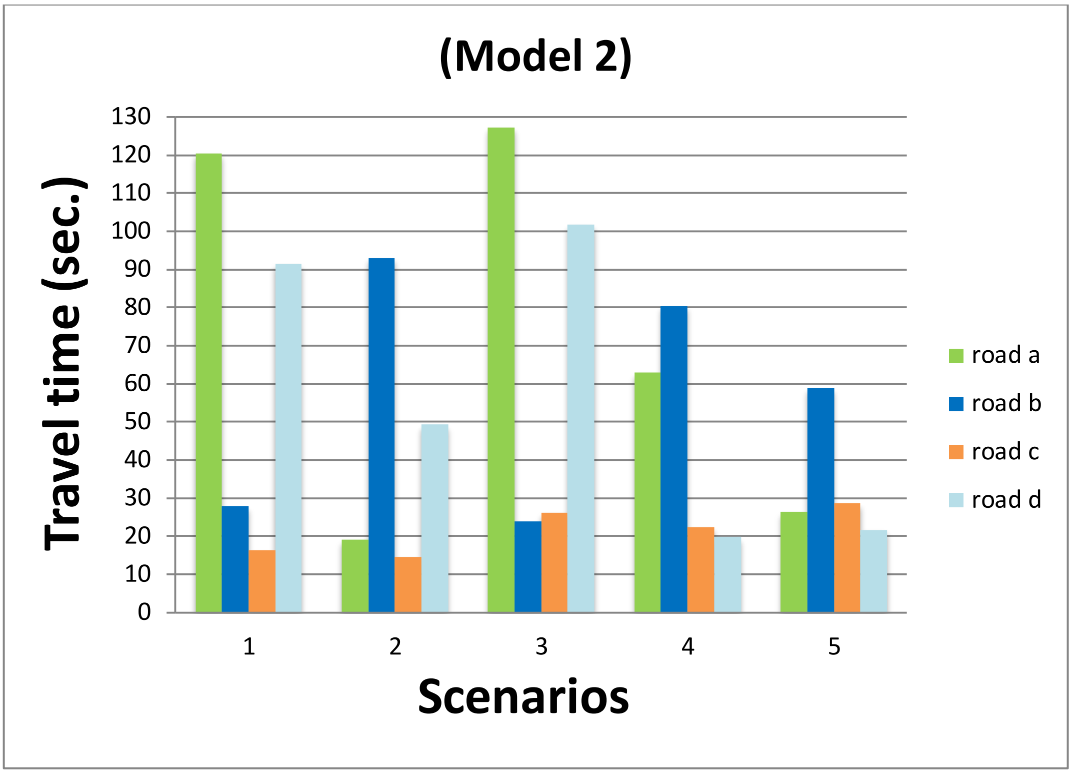


Figure 5-9: Travel time for each road in Model 2

5.3.5 Applying Scenarios for Model 3

The difference between Model one and Model three is that the last one does not have an exclusive U turn. This is done to minimize the area used for bridge and ramps on the minor road.

- Input data is the same as the other models 1380, 2250, 650, 2250 for Roads A, B, C and D respectively.
- Traffic signals are installed on Roads A,B,C and D as shown in Figure 5-10
- Assume travel cycle time and simulate it to obtain the best result of Level of service (LOS) and minimize Delay.
- The distribution of traffic volumes is shown in Figure 5-11.

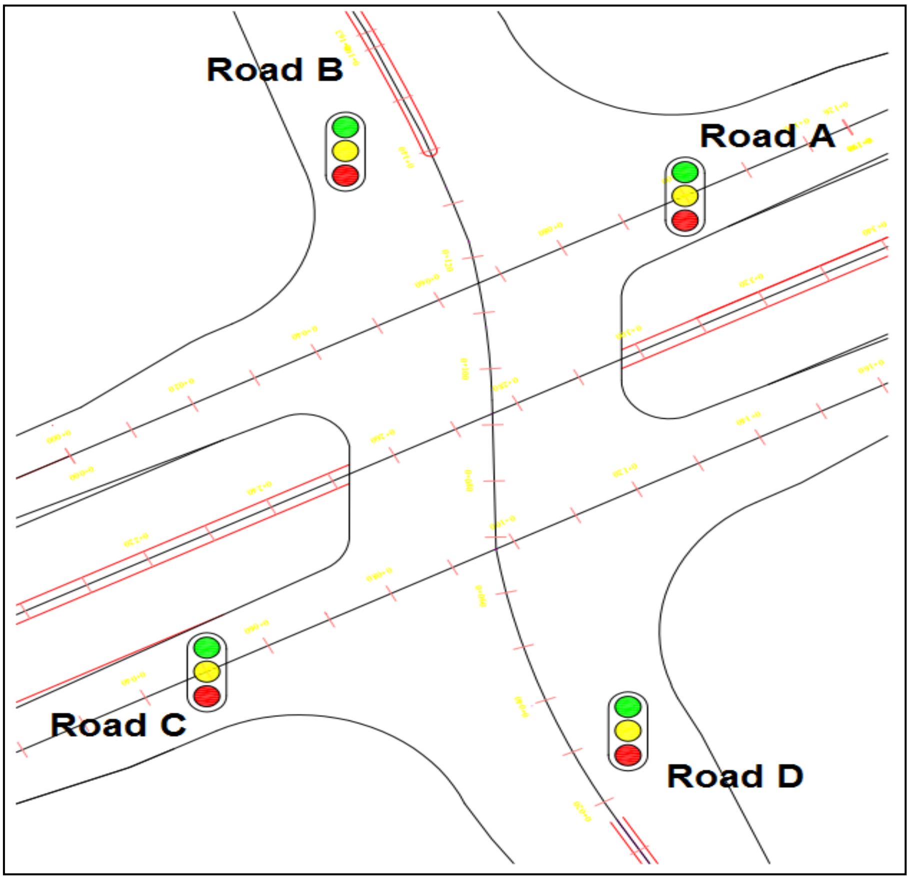


Figure 5-10: Define roads in Model 3

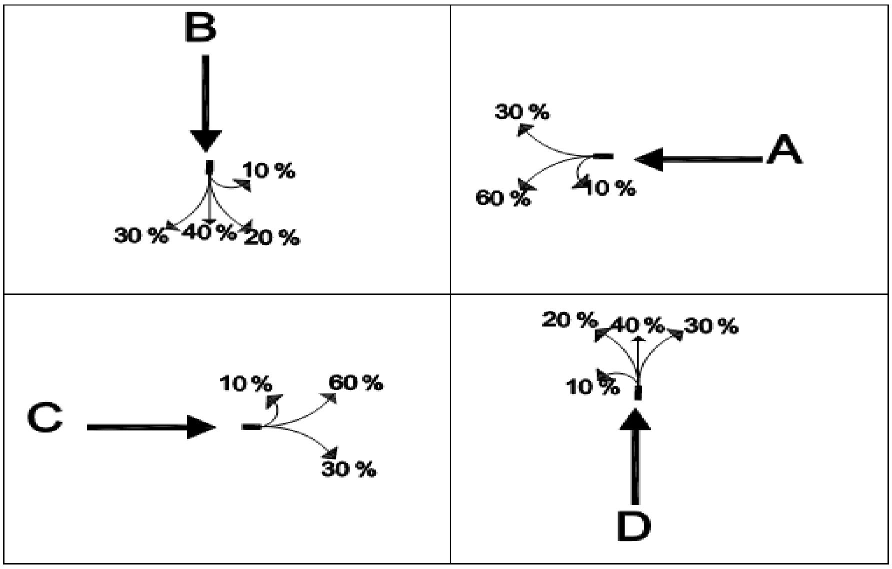


Figure 5-11: Distribution of traffic volume for Model 3

1st scenario: at the first scenario 131 sec. was assumed for cycle length with specific numbers of lanes: 2 for road A, 3 for road B, 2 for road C and 3 for road D as shown in Table 5-20.

Table 5-20: First scenario signalization data for Model 3

Road	Cycle Length	Travel time (sec.)	Delay	# Veh.	total delay per approach	green	# of lanes at inter.	LOS
A	131	94.0	80.4	112	9004.8	30	2	F
B		81.2	61.7	213	13142.1	40	3	E
C		59.9	48.0	71	3408	23	2	D
D		90.5	76.4	189	14439.6	37	3	E
Intersection total values				585	39994.5			
junction weighted average delay ≡ 68.36								

2nd scenario: assume 123 sec cycle length with the same number of lanes as the first scenario as shown in Table 5-21.

Table 5-21: Second scenario signalization data for Model 3

	Cycle Length	Travel time (sec.)	Delay	# Veh.	total delay per approach	green	# of lanes at inter.	LOS
A	123	88.3	74.8	107	8003.6	28	2	E
B		70.8	51.3	210	10773	40	3	E
C		52.0	39.8	79	3144.2	23	2	D
D		103.6	89.4	163	14572.2	32	3	F
Intersection total values				559	36493			
junction weighted average delay ≡ 65.28								

3rd scenario: in this case, the cycle length was 120 sec. with keeping the same numbers of lanes as shown in Table 5-22.

Table 5-22: Third scenario signalization data for Model 3

	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.	L O S
A	120	89.0	73.3	103	7549.9	28	2	E
B		61.8	42.4	211	8946.4	38	3	D
C		57.1	45.2	77	3480.4	20	2	D
D		98.9	85.3	201	17145.3	34	3	F
Intersection total values				592	37122			
junction weighted average delay ≡ 62.70								

4th scenario: added one lane to road A and road C as well as reduce the value of cycle length to 115 sec. as shown in Table 5-23.

Table 5-23: Fourth scenario signalization data for Model 3

	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.	L O S
A	115	51.3	37.9	149	5647.1	28	3	D
B		66.6	47.3	197	9318.1	35	3	D
C		51.9	39.6	76	3009.6	17	3	D
D		95.8	82	194	1590.8	34	3	F
Intersection total values				616	33882.8			
junction weighted average delay ≡ 55.0								

5th scenario: keep the same number of lanes as in the fourth scenario and assume 120 sec. for cycle length as shown in Table 5-24.

Table 5-24: Fifth scenario signalization data for Model 3

	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.	L O S
A	120	56.1	43.0	125	5375	28	3	D
B		59.7	40.2	214	8602.8	38	3	D
C		55.4	43.1	77	3318.7	17	3	D
D		86.8	72.6	212	15391.2	37	3	E
Intersection total values				682	32687.7			
junction weighted average delay ≡ 52.05								

6^{th} scenario: assume 125 sec. for cycle length and 3 lanes for all roads (A, B, C and D) as shown in Table 5-25.

Table 5-25: Sixth scenario signalization data for Model 3

	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.	LOS
A	125	57.8	44.5	129	5740.5	28	3	D
B		66.8	47.2	215	10148	39	3	D
C		55.8	43.4	83	3602.2	18	3	D
D		91.4	77.2	196	15131.2	39	3	E
Intersection total values				623	34621.9			
junction weighted average delay ≡ 55.57								

7^{th} scenario: keeping the same cycle length from the previous scenario and add another lane to road B & D as shown in Table 5-26.

Table 5-26: Seventh scenario signalization data for Model 3

	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green	# of lanes at inter.	LOS
A	125	57.8	44.4	129	5727.6	28	3	D
B		59.0	39.6	230	9108	39	4	D
C		55.8	43.3	83	3593.9	18	3	D
D		56.1	42.2	228	9621.6	39	4	D
Intersection total values				670	28051.1			
junction weighted average delay ≡ 41.86								

8^{th} scenario: in this case assume 115 sec. for cycle length and the same number of lanes as the seventh scenario as shown in Table 5-27.

Table 5-27: Eighth scenario signalization data for Model 3

	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green (sec.)	# of lanes at inter.	LOS
A	115	51.0	37.7	148	5579.6	28	3	D
B		58.7	39.2	210	8232	35	4	D
C		51.9	39.6	76	3009.6	17	3	D
D		52.2	38.1	242	9220.2	34	4	D
Intersection total values				676	26041.4			
junction weighted average delay ≡ 38.52								

9th scenario: assume 110 sec. for cycle length and keep the same number of lanes as shown in Table 5-28.

Table 5-28: Ninth scenario signalization data for Model 3

Road	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green (sec.)	# of lanes at inter.	LOS
A	110	54.2	39.9	143	5705.7	26	3	D
B		56.6	37.1	230	8533	34	4	D
C		56.0	43.7	74	3233.8	16	3	D
D		59.5	43.2	236	10195.2	33	4	D
Intersection total values				683	27667.7			
junction weighted average delay ≡ 40.51								

10th scenario: assume 120 sec. cycle length with the same number of lanes in previous scenarios as shown in Table 5-29.

Table 5-29: Tenth scenario signalization data for Model 3

	Cycle Length	Travel time (sec.)	delay	# Veh.	total delay per approach	green (sec.)	# of lanes at inter.	LOS
A	120	51.4	38.0	126	4788	28	3	D
B		56.6	37.0	227	8399	38	4	D
C		55.6	43.2	79	3412.8	17	3	D
D		53.3	39.8	255	10149	37	4	D
Intersection total values				687	26748.8			
junction weighted average delay ≡ 38.93								

5.3.6 Conclusion for Model 3:

The minimum values of junction weight average delay were in 8th and 10th scenarios when the cycle length is from 115 to 120 sec. But the level of service (LOS) in these scenarios remains not acceptable as following:

- Road A……LOS (D)
- Road B …..LOS (D)
- Road C …..LOS (D)
- Road D.......LOS (D)

Figure 5-12 and Figure 5-13 illustrate the results of delay and travel time of Model 3.The summery of all scenarios can be seen in Appendix D

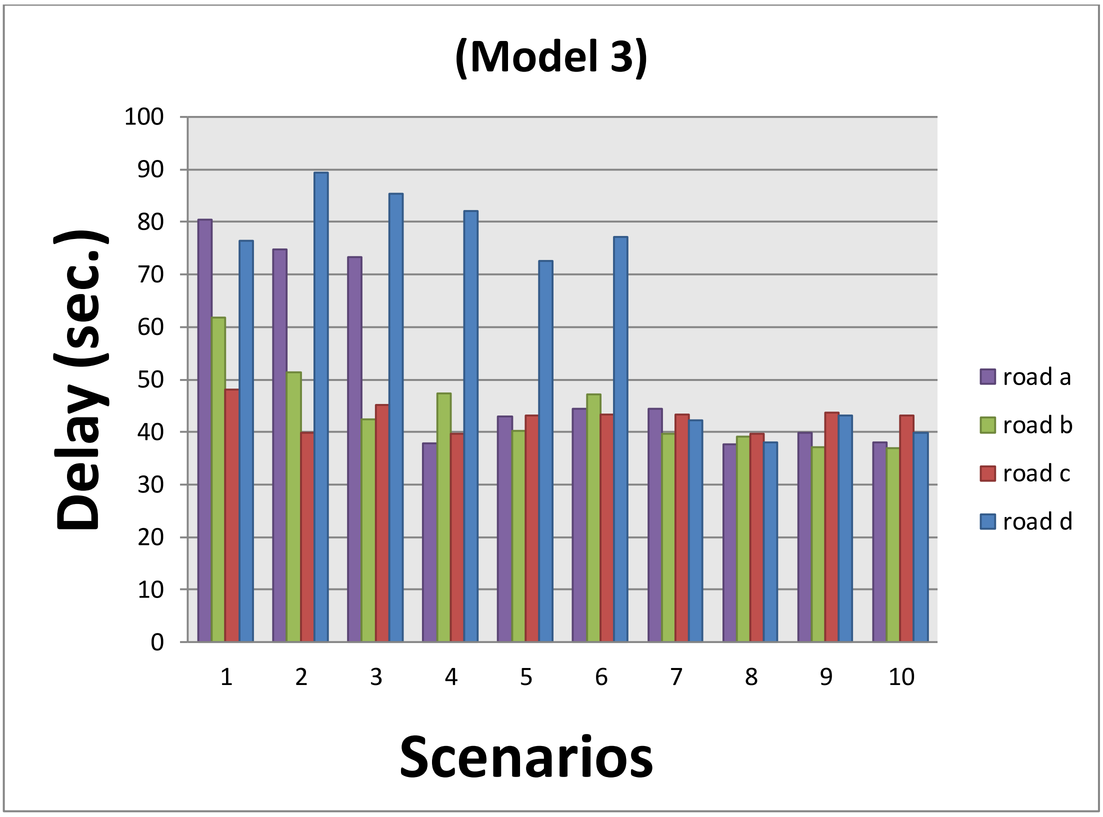


Figure 5-12: Delay diagram for each road in Model 3

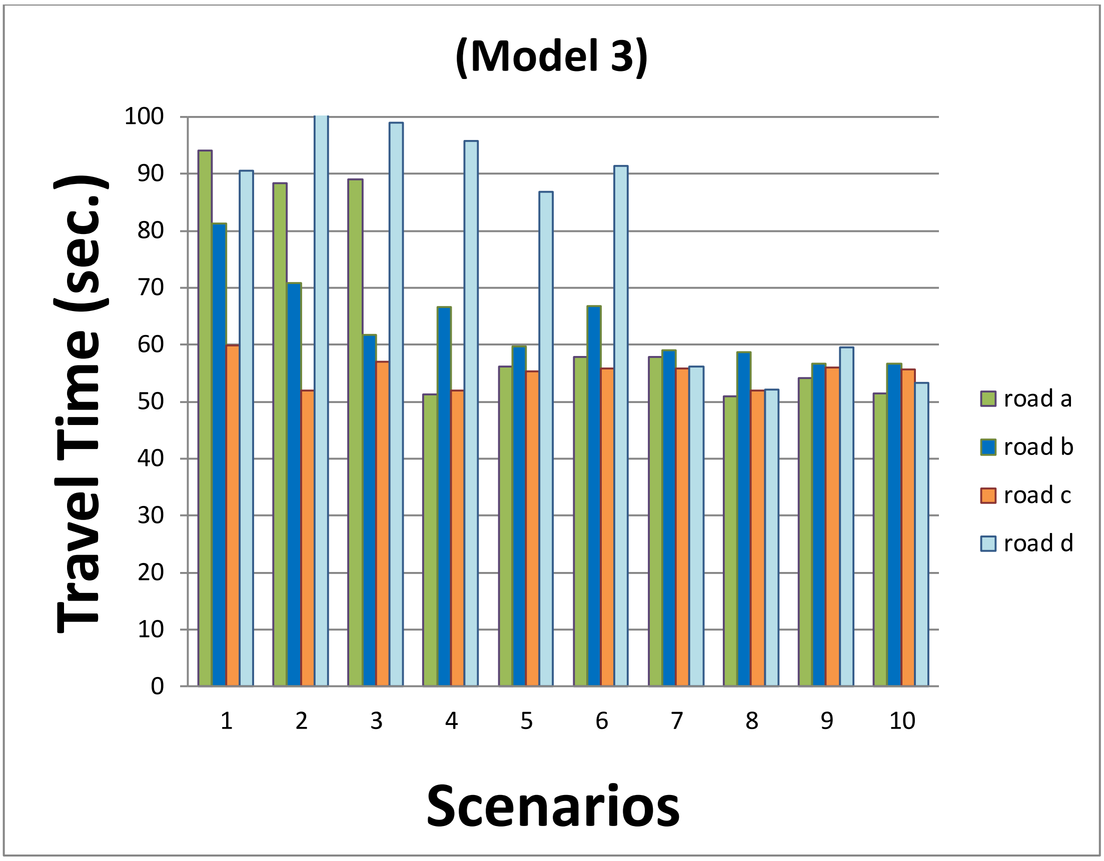


Figure 5-13: Travel time for each road in Model 3

5.4 Comparison of Alternative Models

Because delay is one of the most important factors affecting people all over the world, improving the delay on one person (city) has an impact on the whole community. The cost of delay was projected for fifteen years as shown in these calculations:

5.4.1 Minimum Value of Junction Weighted Average Delay (J.W.A.D)

The value of junction weighted average delay in this study was considered as a measure between the models. Table 5-30 shows the best scenario which has the minimum value of J.W.A.D for each model and also has the minimum values of delay and travel time.

Table 5-30: The best results from all scenarios from three Models

Road	Model 1			Model 2			Model 3		
	Delay	T.T	LOS	Delay	T.T	LOS	Delay	T.T	LOS
A	72.5	84.9	E	12.9	12.9	B	37.7	51.0	D
B	37.8	75.7	D	39.3	39.3	E	39.2	58.7	D
C	25.8	35.8	C	14.9	14.9	B	39.6	51.9	D
D	31.8	47.1	C	7.3	21.7	A	38.1	52.2	D
J.W.A.D	39.86			19.72			38.52		
No. of vehicles	818			616			676		

5.4.2 Delay per hour for Existing and New Models

To find the delay per hour per all vehicles should be convert the results from delay per sec per vehicle to delay per one hour total vehicles as shown in Table 5-31.

Table 5-31: Delay per hour for one day

	Intersection delay (sec/veh)	Number of vehicles	Total delay (sec)	Total Delay (hours)
Existing	278.2	250	69550	19.31
Model 1	39.86	818	32605.48	9.05
Model 2	19.72	616	12147.52	3.37
Model 3	38.52	676	26039.52	7.23

5.4.3 Benefit of Delay

In this study, the comparing between the results of delay for three new models with the existing delay intersection can be seen in Table 5-32. Assuming two peak hours in a day, 26 day work in a month and 12 month in a year. From these values of delay can find the benefit from saving delays hours. Finally model 2 proves more efficient than the other models particularly in light of a fifteen year projection.

Table 5-32: Delay per hour's comparison

Cases	Day		Month	One year	15 year
	One peak hour	Two peak hours	52 peak hours	624 peak hours	9360 peak hours
Existing	19.31	38.62	1004.12	12049.44	180,741.6
Model 1	9.057	18.114	470.964	5651.568	84,773.52
Model 2	3.374	6.748	175.448	2105.376	31,580.64
Model 3	7.233	14.466	376.116	4513.392	67,700.88

5.4.4 Maximum Average Speed

When analyzing the travel time results for all scenarios and comparing between the average speeds for the best scenarios in each model, it is evident that model two has the maximum average speed.

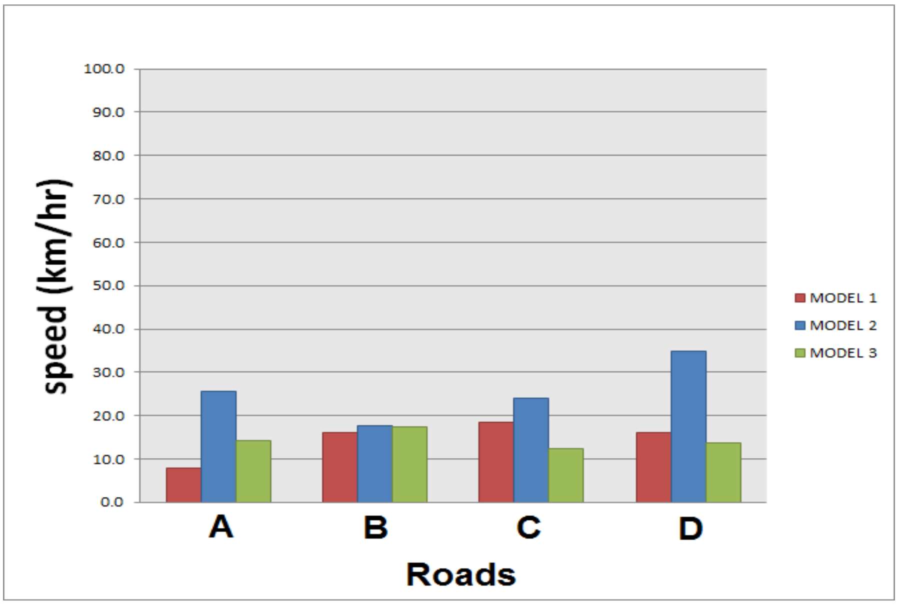


Figure 5-14: Average speed values at each leg of the junction for model 1, model 2 and model 3

Chapter 6

CONCLUSION AND RECOMMENDATION

6.1 Conclusion

The delay and level of service data output presented in this study were based on results obtained using micro simulation VISSIM software. Three different models of interchange were simulated in order to calculate how those interchanges work with regard to the traffic volume prediction. The results obtained from multiple scenarios after altering the geometrical construction and traffic elements of each model are listed in detail in chapter five.

The best results for delay calculations in these scenarios were compared and contrasted with the existing delay levels of the IHI intersection. Likewise, the results of intersection delay represented by the peak hour for each model were then doubled to accommodate two peak hours per day. This standard was then extrapolated to calculate the levels of delay for the next 15 years.

Using the dual peak hour delay for 15 years standard, the results for the second model were 31,580 hour. This was a marked improvement to the 84,773 hour and 67,700 hour peak hour delay for the first and third model respectively. This in turn is a great improvement to the existing delay level of the IHI intersection which stands as 180,741 hours and that not including a prediction of the next 15 years.

The conclusion from these calculations was that the level of delay for model two was better than the first, third and existing models. This can be illustrated by the following equivalence comparison: If the intersection remains in the current state there will be a loss of roughly 7500 days' worth of delay time over the next 15 years. When applied to the first model it would be reduced to roughly 3500 days of delay and roughly 2800 days of delay for the third model when total delay was divided by 24 hours. And yet for the second model the delay time for the next 15 years would be equivalent to roughly 1250 days of delay. The effectiveness of this study can be very simply verified by pointing out that for the next 15 years more than an 80% improvement in delay time between the current intersection and the proposed second intersection model can be projected.

6.2 Recommendation

The following are recommended:

- The limited land of area (right-of-way) leads to the choice of a diamond interchange. Of the three models proposed the second model proved to be more efficient and economic. On the other hand, if free land was available in the area, another type with continuous flow for major and minor roads such as a cloverleaf interchange could be proposed.
- If the budget available for the project exceeds 10 million USD, multi-level interchange could be considered [31].

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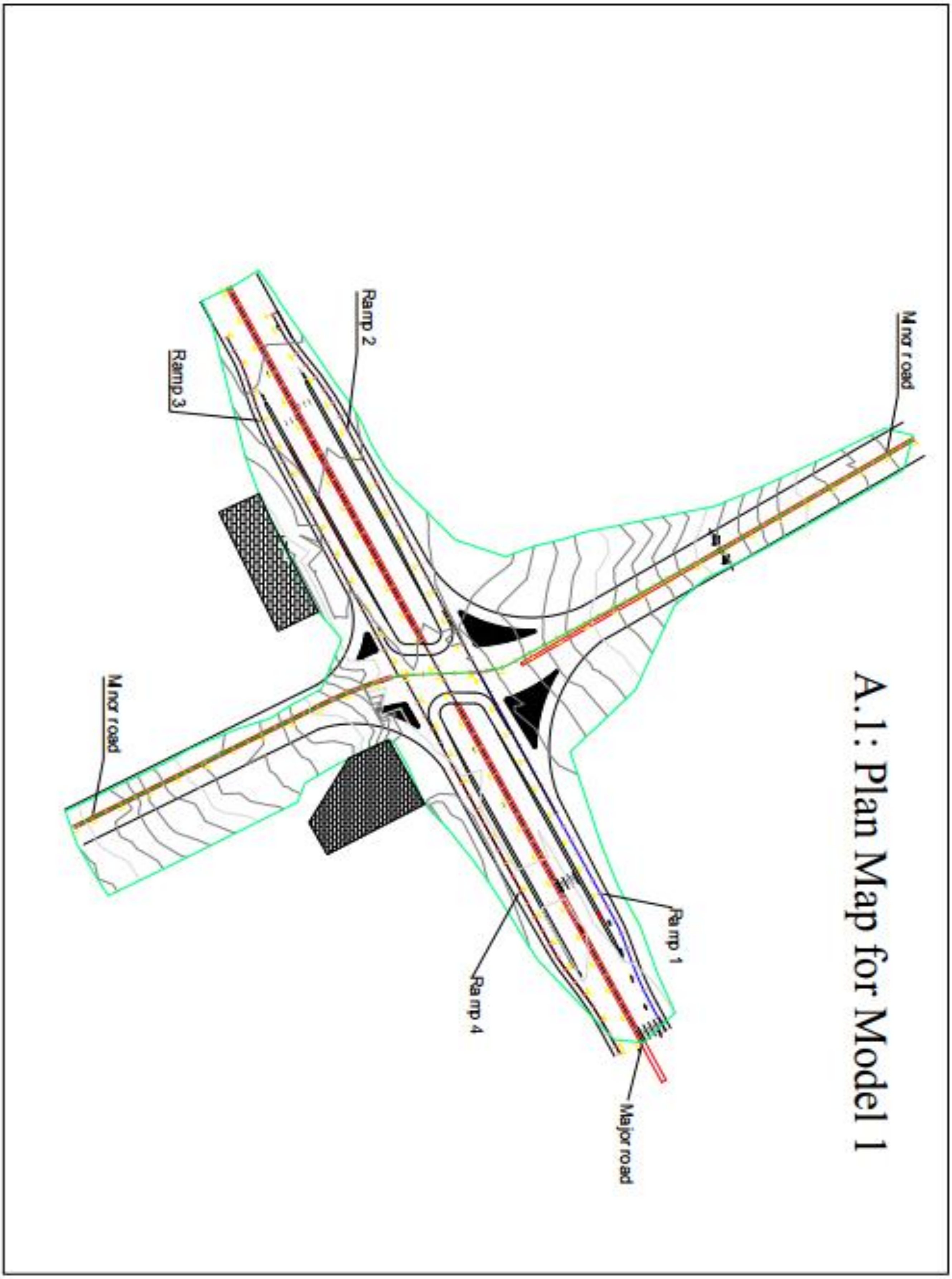
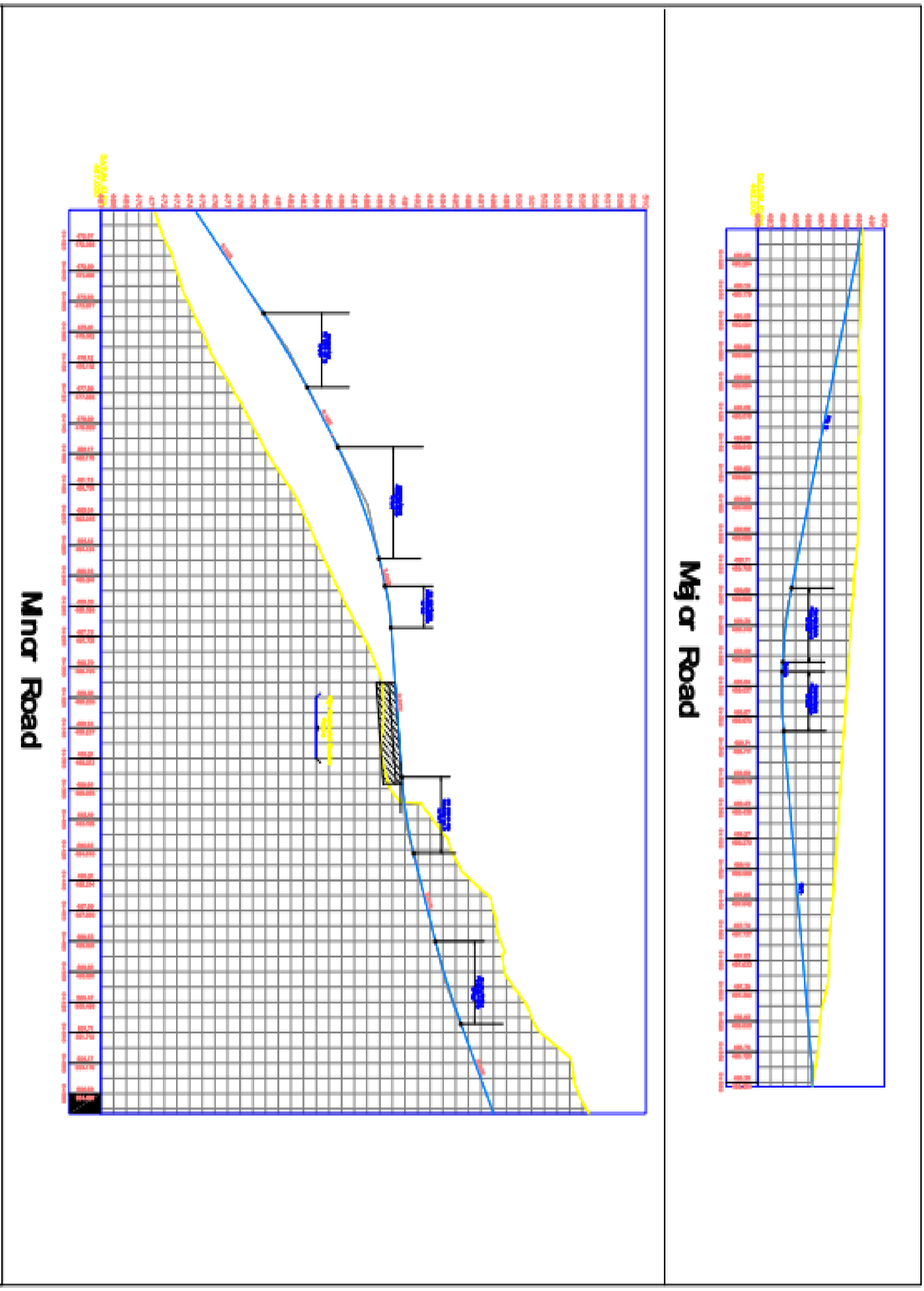
APPENDICES

Appendix A: Model 1

A.1: Plan Map for Model 1

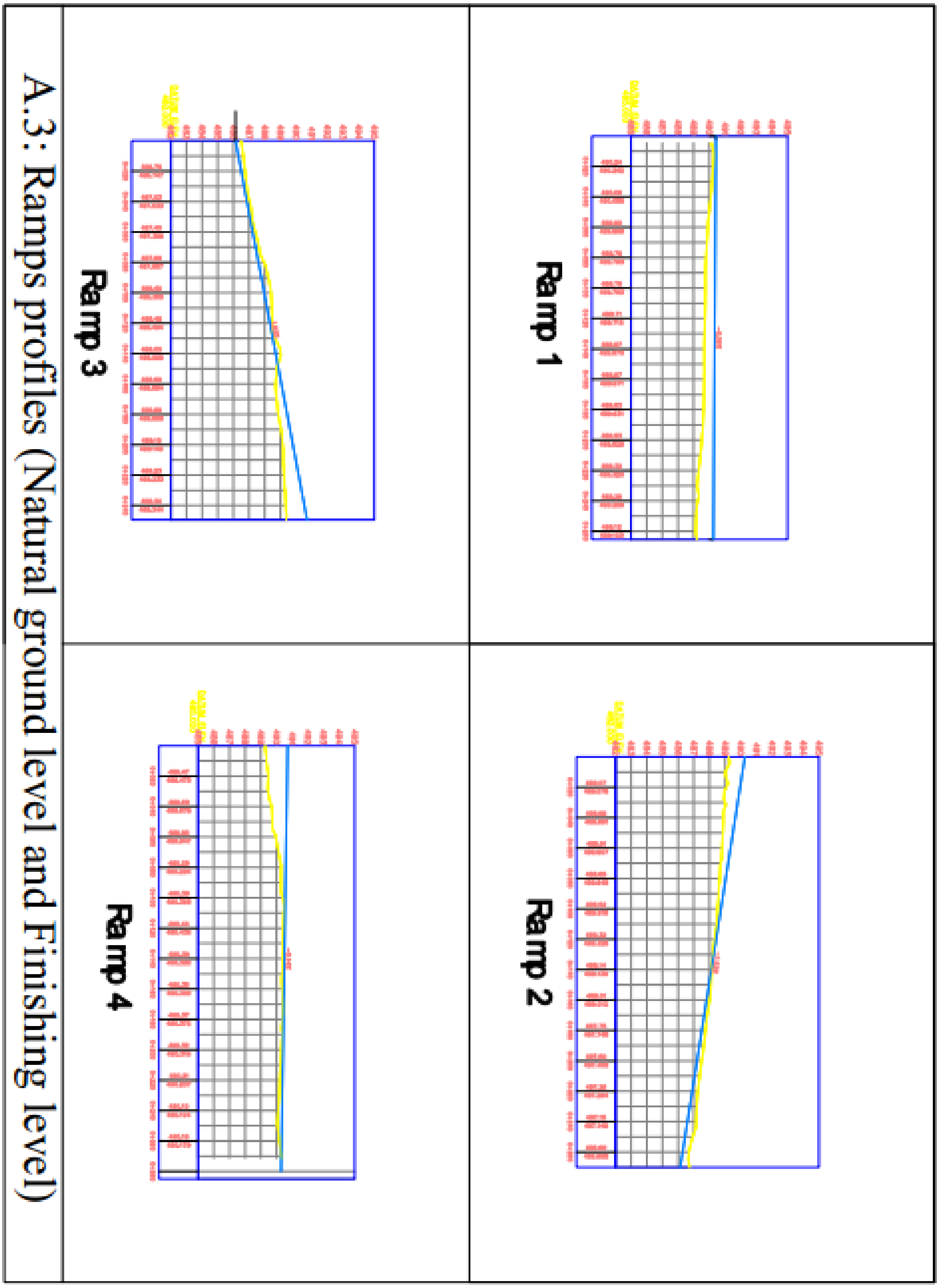


A.2: Major and Minor profiles (Natural ground level and Finishing level)



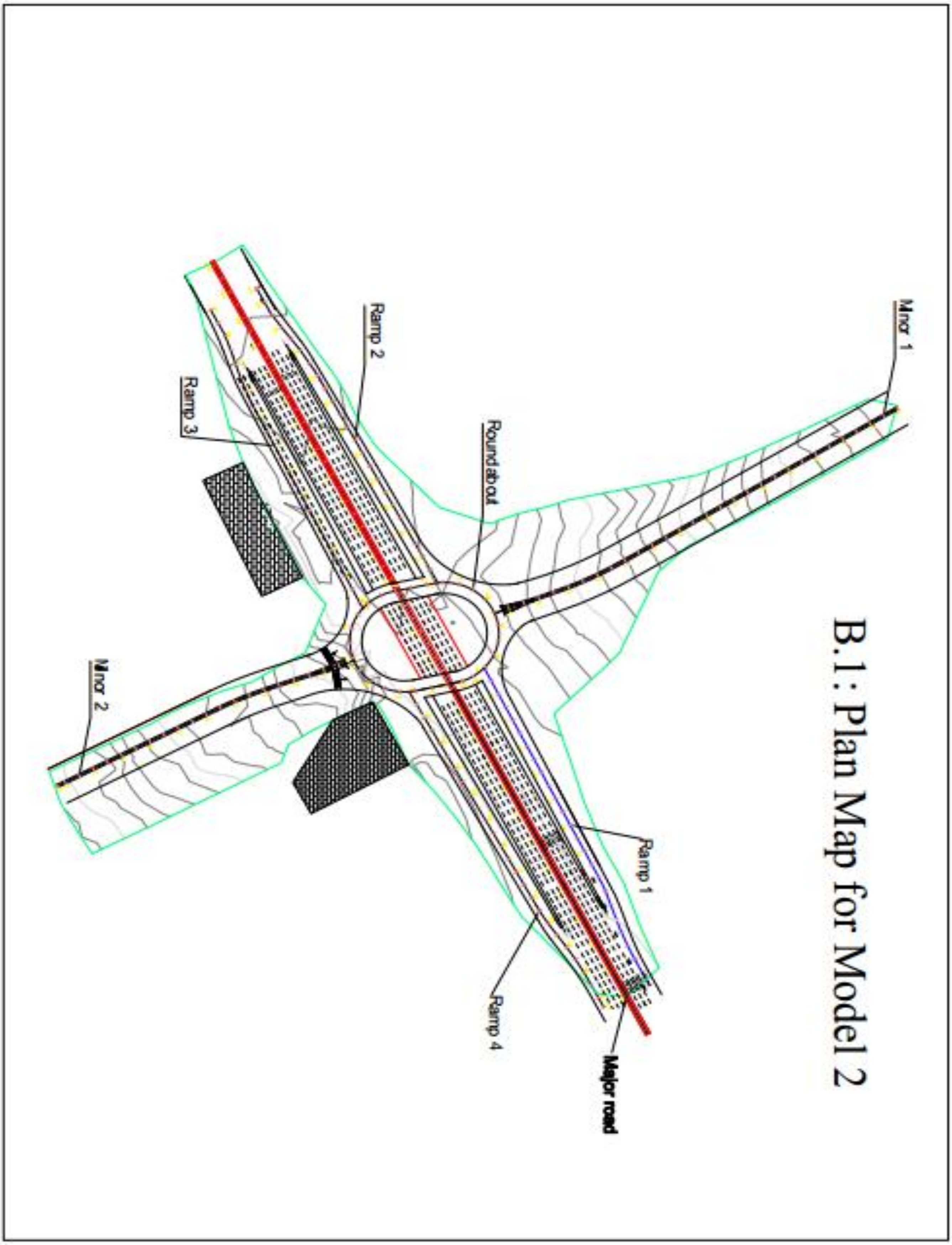
A.2: Major and Minor prfiles (Natural ground and Finishing level)

A.3: Profiles of Ramps (Natural ground level and Finishing level)

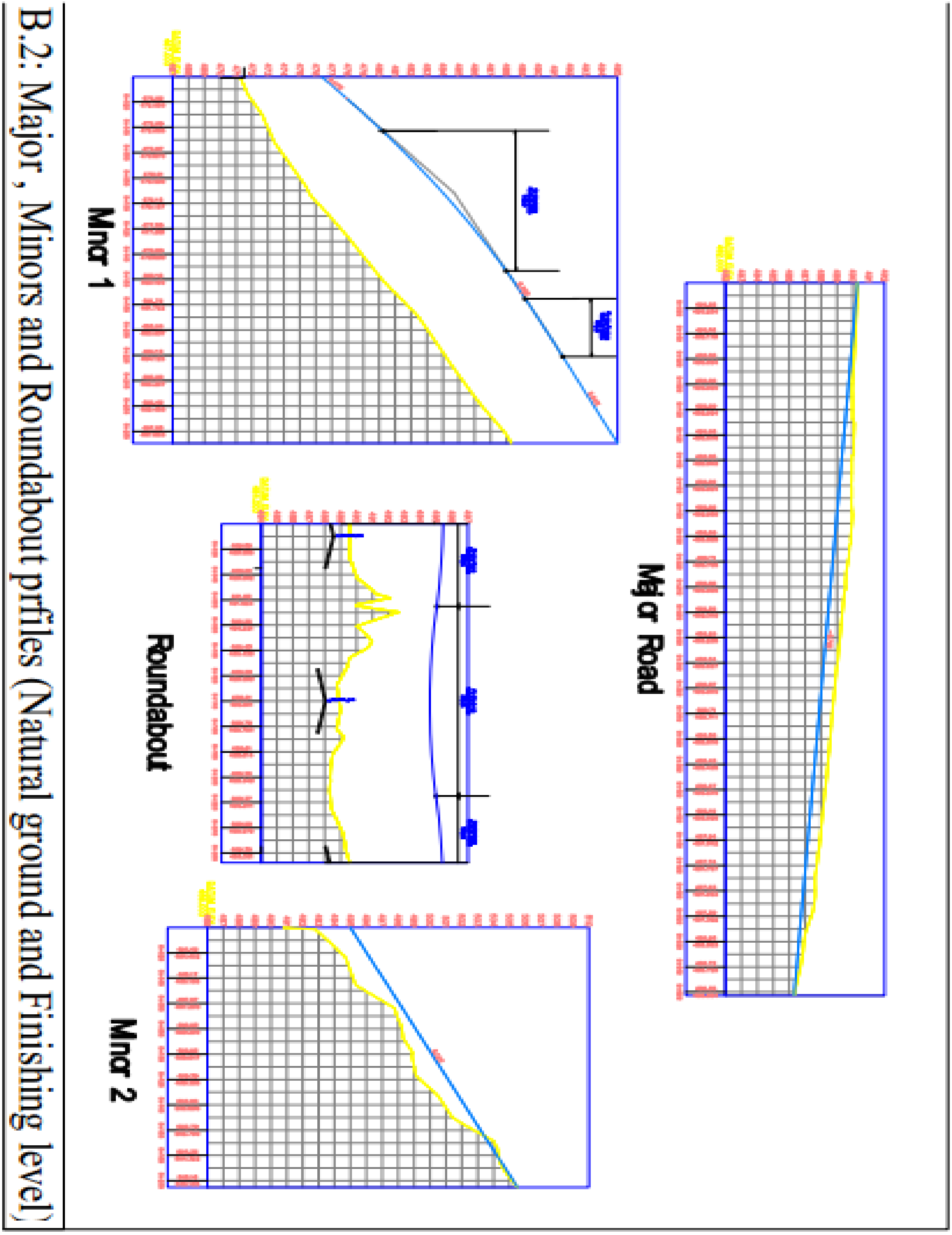


Appendix B: Model 2

B.1: Plan Map for Model 2

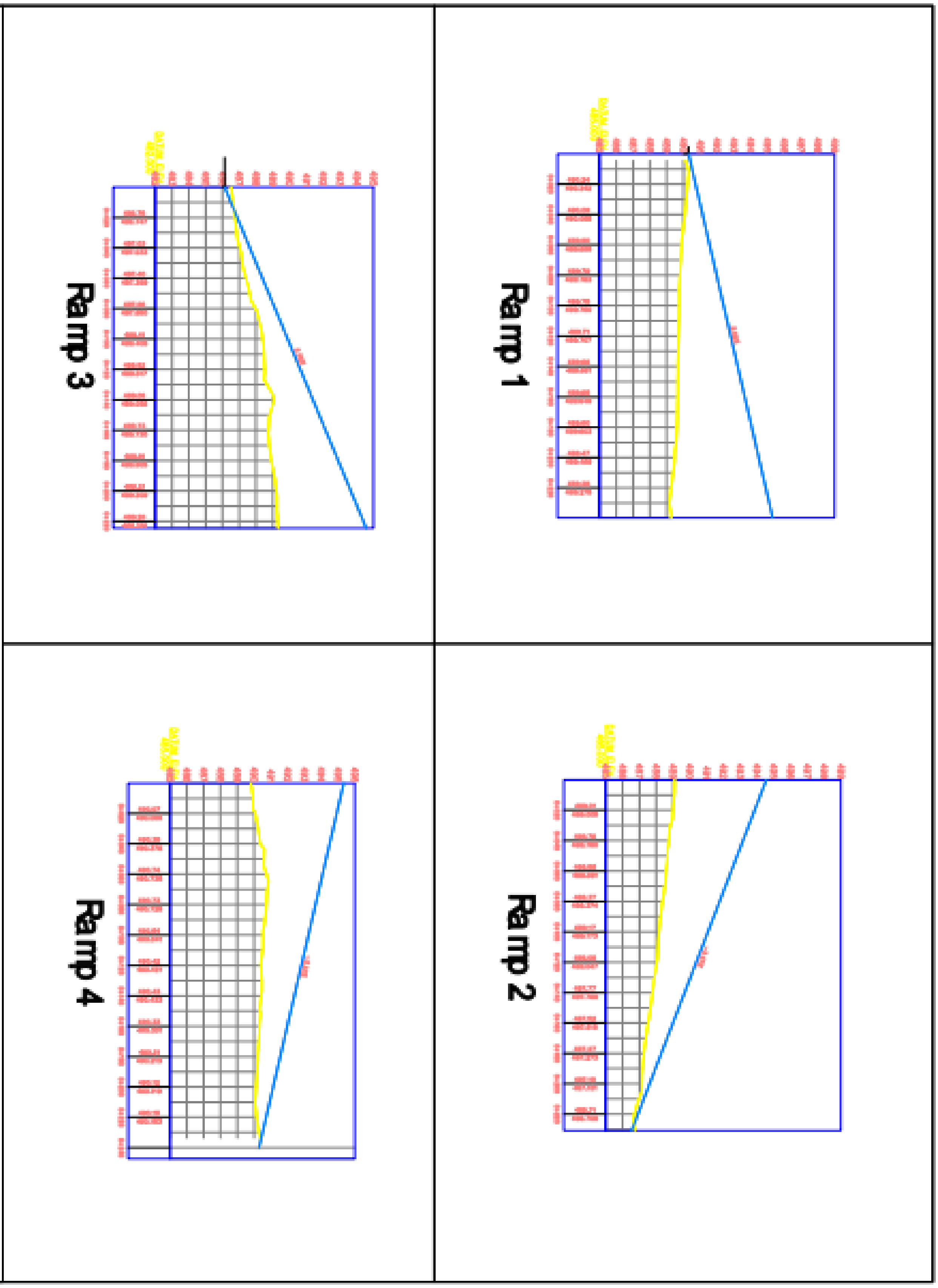


B.2: Major, Minor and Roundabout profiles (Natural ground level and finishing level)



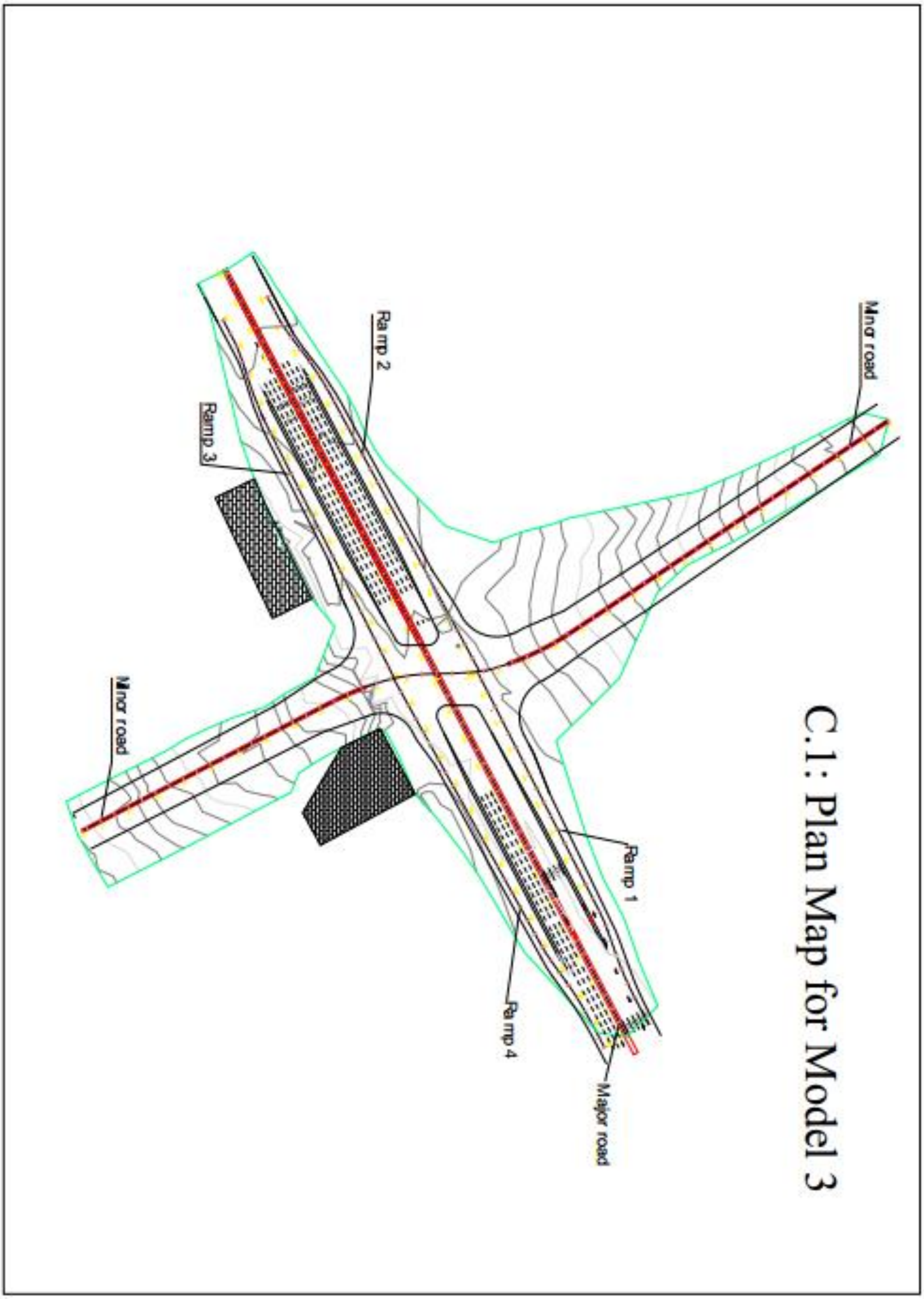
B.3: Profiles of Ramps (Natural ground level and Finishing level)

B.3: Ramps profiles (Natural ground level and Finishing level)



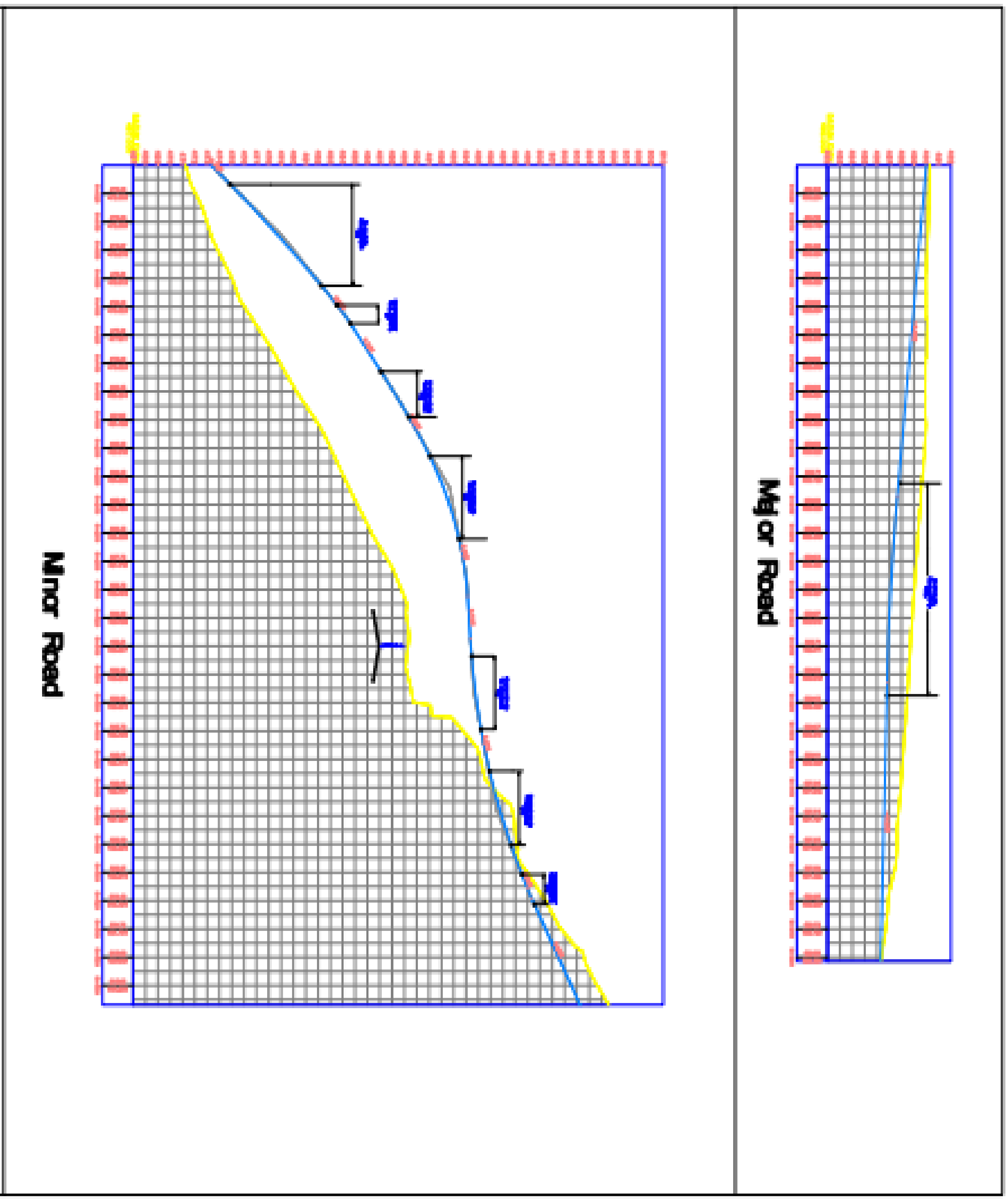
Appendix C: Model 3

C.1: Plan Map for Model 3

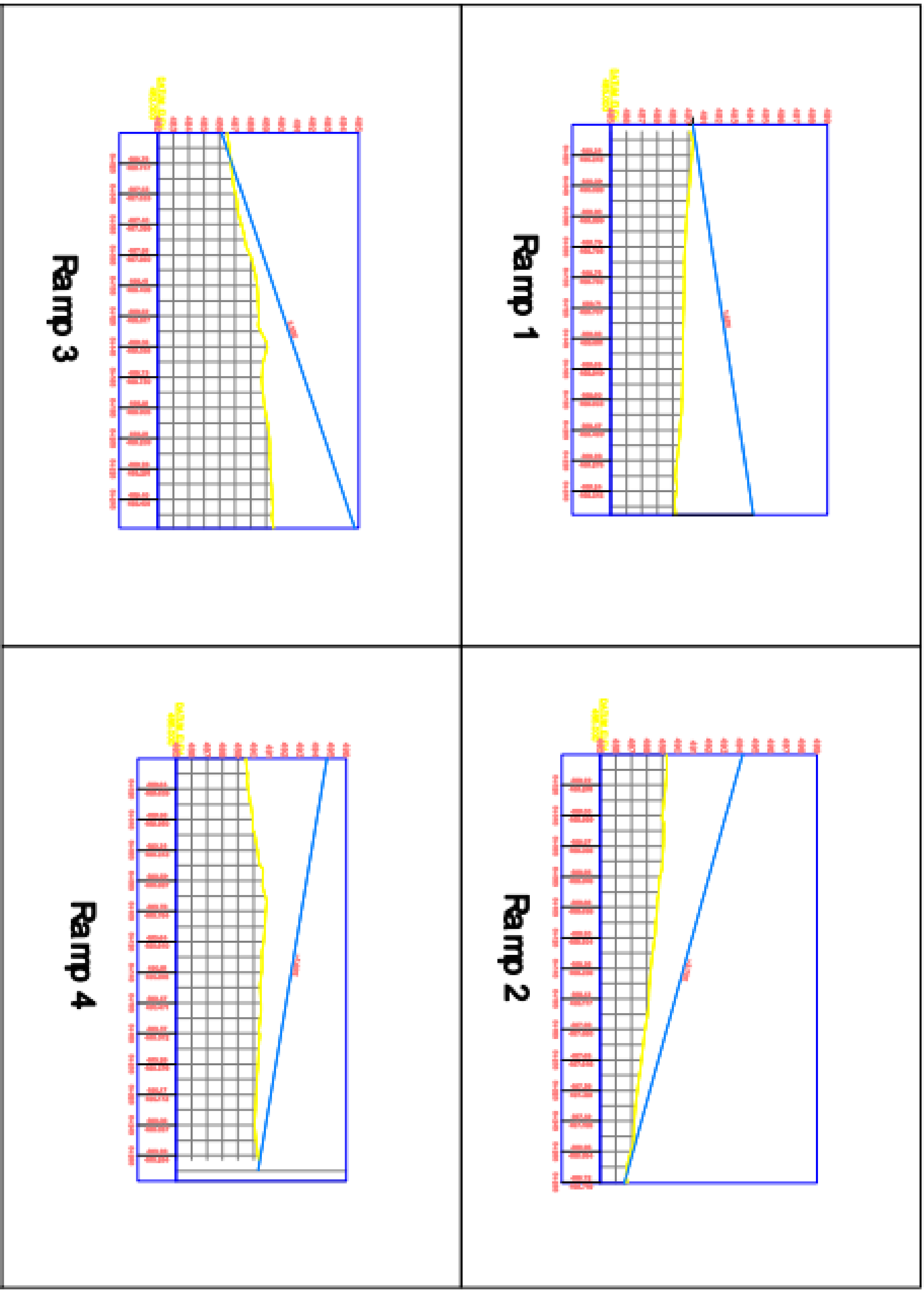


C.2: Major and Minor profiles (Natural ground level and Finishing level)

C.2: Major and Minor prfiles (Natural ground level and Finishing level)



C.3: Profiles of Ramps (Natural ground level and Finishing level)



C.3: Ramps profiles (Natural ground level and Finishing level)

Appendix D: Summery for scenarios of all Models

D.1: Model 1 summery for all scenarios

Model 1 summery for all scenarios

Cycle length	Roads								Green (sec.)				Junction weighted average delay	LOS	No of lanes			
	A		B		C		D											
	T.T	D.T	T.T	D.T	T.T	D.T	T.T	D.T	A	B	C	D			A	B	C	D
115	92.7	80.4	66.0	42.7	71.6	59.1	41.3	27.0	20	40	17	38	42.95	D	3	4	1	4
116	90.5	78.2	66.7	43.4	72.4	59.8	41.0	26.7	20	40	18	38	42.56	D	3	4	1	4
117	88.2	76	65.6	42.3	68.2	55.6	40.4	26.2	20	40	19	38	41.27	D	3	4	1	4
118	90.5	77.5	66.7	40.2	72.4	47.8	41.0	28.9	20	40	20	38	41.40	D	3	4	1	4
118	89.6	77.5	63.3	40	38.2	25.7	45.6	31.3	20	40	20	38	41.26	D	3	4	2	4
119	84.9	72.5	75.7	37.8	35.8	25.8	47.1	31.8	20	40	20	39	39.86	D	3	4	2	4
120	86.6	74.6	70.4	47.1	62.1	49.5	40.6	26.4	20	40	20	40	42.07	D	3	4	1	4
130	81.6	69.3	79.1	55.7	66	53.3	44.4	30.1	23	43	21	43	47.75	D	3	4	1	4
140	92	97.8	80.3	57	63.2	50.7	53.6	39.4	25	45	25	45	55.85	E	3	4	1	4
150	92.1	79.9	75.1	51.9	77.6	65.1	48.7	34.4	25	50	25	50	48.61	D	3	4	1	4
178	119.4	107.1	114.6	91.4	108	95.6	55.7	41.4	30	55	30	53	72.41	E	3	4	1	4

D.2: Model 2 summery for all scenarios

Model 2 summery for all scenarios

Roads								Junction weighted average delay	LOS	Notes
A		B		C		D				
T.T	D.T	T.T	D.T	T.T	D.T	T.T	D.T			
120.4	104.8	27.9	8.2	16.2	2.6	91.5	76.9	36.78	E	priority for all roads
19.1	5.8	93.0	73.8	14.6	1.1	49.2	35.2	34.73	D	priority for A & C
127.3	111.9	23.8	3.9	26.2	12.6	101.7	87.4	38.28	E	priority for B & D
62.8	46.2	80.4	48.5	22.4	9.0	19.7	7.4	30.45	D	priority for vehicles inside roundabout
26.4	12.9	58.9	39.3	28.6	14.9	21.7	7.3	19.72	C	add exclusive right turn for each approach

D.3: Model 3 summery for all scenarios

Model 3 summery of scenarios

Cycle length	Roads								Green (sec.)				Junction weighted average delay	LOS	No of lanes			
	A		B		C		D											
	T.T	D.T	T.T	D.T	T.T	D.T	T.T	D.T	A	B	C	D			A	B	C	D
110	54.2	39.9	56.6	37.1	56.0	43.7	59.5	43.2	26	34	16	33	40.51	D	3	4	3	4
115	51.0	37.7	58.7	39.2	51.9	39.6	52.2	38.1	28	35	17	34	38.52	D	3	4	3	4
115	51.3	37.9	66.6	47.3	51.9	39.6	95.8	82	28	35	17	34	55.0	D	3	3	3	3
120	56.1	43.0	59.7	40.2	55.4	43.1	86.8	72.6	28	38	17	37	52.05	D	3	3	3	3
120	51.4	38.0	56.6	37.0	55.6	43.2	53.3	39.8	28	38	17	37	38.93	D	3	4	3	4
120	89	73.3	61.8	42.4	57.1	45.2	98.9	85.3	28	38	20	34	62.7	E	2	3	2	3
125	57.8	44.5	66.8	47.2	55.8	43.4	91.4	77.2	28	39	18	39	55.57	E	3	3	3	3
125	57.8	44.4	59.0	39.6	55.8	43.3	56.1	42.2	28	39	18	39	41.86	D	3	4	3	4
123	88.3	74.8	70.8	51.3	52.0	39.8	103.6	89.4	28	40	23	32	65.28	E	2	3	2	3
131	94.0	80.4	81.2	61.7	59.9	48.0	90.5	76.4	30	40	23	37	68.36	E	2	3	2	3

Appendix E: Flat Terrain, Distance Required to Effect Grade Separation