ROAD SAFETY EVALUATION OF URBAN MAJOR ARTERIALS CASE STUDY OF 1071 MALAZGİRT BOULEVARD IN ANKARA

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ABSTRACT

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Traffic accidents cause damages in social, economic and health of communities, which have severe and serious costs in the long run. Infrastructural elements are also crucial, which can be avoided beginning from planning stages. Road Safety Audit (RSA) is a concept that lists and checks various design and operational characteristics of road projects and segments, to either prevent accidents and/or reduce their severity, implemented in many countries. However, lack of RSA legislation in Turkey is a major problem, especially in urban regions, where major road network changes often happen due to rapid growth of motorization rates and traffic volumes.

In this thesis, a thorough review of RSA literature and guidelines resulted in creation of a checklist for safety evaluation of urban major arterials, divided in four major themes (General Issues, Alignment and Cross-Section Issues, Intersection Issues and Interchange Issues). A RSA report is prepared voluntarily for a recent urban arterial extension in Ankara, 1071 Malazgirt Blvd. Evaluations showed that original design project with 70 km/hr speed limit had minor safety concerns due to landscaping and sight distance requirement at certain locations. However, current increase in the speed limits to 82 km/hr (corresponding to 90 km/hr before penalties), showed severe deficiencies in horizontal and vertical alignments (inappropriate superelevation rates, lack of lateral sigh distances, etc.). While such infrastructural design limitations may be responsible for some of the accidents happened along this corridor, they are most likely undetected due to lack of formal RSA requirements, and listed under "driver/person" errors in statistics.

Keywords: Road safety, Traffic safety, Road safety audit, Road safety evaluation, Road safety review, Road safety inspection

ÖZ

KENTSEL BÜYÜK ARTERLERİN YOL GÜVENLİĞİNİN DEĞERLENDİRİLMESİ ANKARA 1071 MALAZGİRT BULVARI DURUM ÇALIŞMASI

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Trafik kazaları, toplumlar üzerinde sosyal, ekonomik ve sağlık anlamında uzun vadede ciddi maliyetleri olan hasarlara sebebiyet vermektedir. Altyapı ile ilgili sorunlar planlama aşamasından başlayarak önlenebilecek olması yönünden çok önemlidir. Yol Güvenliği Denetimi (RSA), birçok ülkede uygulanan, kazaları önlemek ve / veya şiddetini azaltmak için, yol projelerinin ve mevcut yol segmentlerinin çeşitli tasarım ve işletim karakteristiklerini listeleyen ve kontrol eden bir kavramdır. Bununla birlikte, Türkiye'deki RSA mevzuatının eksikliği, özellikle motorizasyon oranlarının ve trafik hacimlerinin hızlı büyümesi nedeniyle, büyük karayolu ağı değişikliğinin meydana geldiği kentsel bölgelerde büyük bir sorun haline gelmiştir.

Bu tez çalışmasında, kentsel ana arterlerin güvenlik değerlendirmesi üzerine control listelerinin oluşturulması adına RSA literatürünün ve kılavuzlarının, dört ana temaya bölünmüş olarak (Genel Konular, Güzergah ve Kesit Konuları, Hemzemin Kavşak Konuları ve Çok Katlı Kavşak Konuları olmak üzere) kapsamlı bir incelemesi yapılmıştır. Ankara'da yeni yapılan bir kentsel ana arter olan 1071 Malazgirt Bulvarı için gönüllü olarak bir RSA raporu hazırlanmıştır. Değerlendirmeler, 70 km / s hız limitine sahip özgün tasarım projesinin, bazı yerlerde peyzaj ve görüş mesafesi gereksinimi nedeniyle küçük güvenlik sorunları olduğunu göstermiştir. Bununla birlikte, hız sınırının 82 km / saat'e çıkarılması (ceza öncesi müsamaha ile 90 km / saate karşılık gelir), yatay ve düşey yerleşimlerde (uygun olmayan dever oranları, yanal görüş mesafesi eksikliği vb.) ciddi eksiklikler olduğunu göstermektedir. Bu gibi altyapı tasarım limitlerinin, söz konusu koridorda meydana gelen muhtelif kazalardan sorumlu olması beklenirken, resmi RSA uygulamalarının olmamasından dolayı bu kazaların sebeplerinin istatistiklerde "sürücü / insan" hataları altında listelenmesi muhtemel olmaktadır.

Anahtar Kelimeler: Yol güvenliği kontrolü, Yol güvenliği denetimi, Yol güvenliği değerlendirmesi, Yol emniyeti değerlendirmesi, Trafik güvenliği

To My Parents: Leyla & Bülent Demirel

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ABBREVIATIONS

WHO	World Health Organization
GDH	General Directorate of Highways
EU	European Union
AMM	Ankara Metropolitan Municipality
USA	United States of America
RSA	Road Safety Audit
RSI	Road Safety Inspection
DMRB	Design Manual for Roads and Bridges
IHT	Institution of Highways and Transportation
RSR	Road Safety Review

CHAPTER 1

INTRODUCTION

1.1. Introduction

Road safety is a major problem especially in developing countries such as Turkey. Many authorities and governments are trying to reduce the severity of effects that occur because of road safety deficiencies. This situation led to development of new methods and processes. Although, high-income countries have achieved success on improving road safety procedures, there is still more to be improved apart from the fact that developing countries have just started to pay attention to this subject.

Road safety audit (RSA) and road safety inspection (RSI) methods are most commonly known and applied techniques throughout the world in last decades. These methods have been created in order to utilize the procedures in struggling with road safety problems within a systematic and formal way by focusing on prevention and precautions considering all the phases of both road design and construction beginning from preliminary projects to revising existing roads.

In Turkey, RSA and RSI are not being followed officially. Although, there are some studies and programs that are related with road and traffic safety which are being run by ministries, an exact scheme of state-of-the-art has not been implemented yet. By the instructions of European Union Harmonization process which is being held between Turkey and EU, Turkey has been taking actions in order to comply with EU standards regarding to road safety. Because of this reason, in Turkey, fast-growing road network is not subject to RSA. Hence, as an instance, the design and construction of 1071 Malazgirt Boulevard that has been opened to traffic in 2013 in Ankara which is capital city of Turkey, is evaluated from the point of view of RSA perspective.

1.2. Scope of the Study

In this study, principles of RSA from different sources are compiled to create a subset of guides for urban major arterial RSA evaluation, which can be grouped under major topics of:

- General Issues
- Alignment & Cross-Section Issues
- Intersection Issues
- Interchange Issues

However, some of the items under some groups could not be addressed in the case study project due to lack of information such as traffic count, infrastructure etc.

Also, as this thesis goes back to design documents for some evaluations (such as superelevation, horizontal and vertical curves) as if in a RSA case, it also checks the current conditions for other items such as (pavement markings, landscaping, signals) as if in a RSI manner. Thus, RSA terms in this thesis is used as a general concept which does not differentiate between the design or after construction phases.

1.3. Layout of the Thesis

First, a general background is provided for the discussion of the road safety problem in the world and in Turkey, with a focus on traffic safety legislation considering also road safety evaluations that are used currently in Turkey in Chapter 2. Chapter 3 presents the general concepts and items of RSA process, whereas the details of RSA for urban arterials are discussed in Chapter 4. After the introduction of the project design and characteristics of the case study corridor, 1071 Malazgirt Blvd., in Chapter 5, RSA evaluation details are presented in Chapter 6, followed by the overall conlcusion and further recommendations presented in Chapter 7.

CHAPTER 2

BACKGROUND

2.1. Road Safety Problem in the World

According to World Health Organization (WHO), every year, the number of people who die in traffic accidents is over 1.2 million while between 20 and 50 million suffer from non-fatal injuries (WHO, 2015). Moreover, road traffic accidents are the greatest cause of death of young people who are aged between 15 and 29 (WHO, 2015). It is also emphasized that the fatality rates are double in low and middle-income countries compared with the ones in high-income countries. Because of these facts, especially in the last decades, countries work through the implementations that can be done in order to improve road safety. However, Global Status Report on Road Safety 2015 (WHO) underlines the realities that reflect the situation of the efforts which are not sufficient yet. Especially, the subjects that are evaluated as not to have attracted enough attention yet are defined below;

- In most countries, either the laws related with road traffic safety are not enough or the enforcement of these laws are not totally realized by the public (WHO, 2015).
- Speed management is not applied adequately in many of the countries while it has the greatest importance on reducing the fatality level of the accidents that occur on the roadways (WHO, 2015)
- Still, in a huge part of the world, vehicles that are sold do not meet the minimum safety requirements (WHO, 2015)
- Roads that are constructed are not designed in a way that consider all the safety perspectives they must have especially about the vulnerable road users (WHO, 2015)

Also, a detailed statistical survey has been done by WHO in 2009 in order to reveal the seriousness of road traffic safety lacks throughout the world in different 178 countries. This survey includes many aspects of road safety. This report can be considered as a good milestone that forces countries to comprehend the real situations related with road safety in their own traffics and also it gives the authorities of these countries the chance of both being able to compare their status with other countries' and realizing what can be done in order to improve road safety and reduce the fatalities of road accidents. Some general notes have been summarized as given below according to this survey in "Global Status Report" that was prepared by WHO again in 2009;

- Although low-income and middle-income countries have only 48% of the registered cars in the whole world, the road traffic fatality rates are 21.5 and 19.5 per 100 000 population respectively while high-income countries have a rate of 10.3. In most of the high-income countries, the death rate has been declining in the last decades, however, road accident injuries are still a major percent in the causes of death and injuries. (WHO, 2009)
- The pedestrians, cyclists and users of motorized two wheelers who are called as "vulnerable road users" form almost half part of those who die in road traffic crashes. Also, it is indicated that, the percent of the vulnerable road users is higher in poorer countries. (WHO, 2009)
- The adoption and enforcement of traffic safety related laws are not sufficient in many of the countries. Especially drink-driving, excessive speed and use of seat belts, helmets and child restraints are at critical level in the manner of law enforcing. According to this global status report, only half of the countries have legislations about all of these 5 factors and only 15% of them has laws that can be counted as adequate. (WHO, 2009)
- The rate of the countries who necessitate the usage of helmets for both riders and passengers is only 40%. Likewise, the rate of countries in which seatbelt usage is a mandatory is only 57%. (WHO, 2009)

The most significant results that was derived from the studies of this global report is the fact that the improvement of road safety is a multi-disciplinary task which include both the health, transport and police sectors (WHO, 2009). All of these sectors must be involved in the implementation of the necessary countermeasure with a huge collaboration effort. As can be expected, this situation also requires a big amount of funding which causes the governments to ignore the applications that must be done for a successful coordinated endeavor. The worst point that is seen on the report is the reality about the expected increase in road traffic injuries. According to predictions of WHO, by 2030, as shown in *Table 2.1Table 2.1*, the road traffic crashes will become the fifth leading factor in the causes of death list (WHO, 2009). However, in 2015 version of the same report, it has been underlined that while in 68 countries road traffic deaths increased, in 79 it has decreased compared with the results expressed in WHO Global Status Report on Road Safety 2010 (WHO, 2015).

Table 2.1. *Leading Causes of Death, 2015 and 2030 Compared (WHO, 2013)*

2015				2030			
Rank	**Cause**	**Deaths (000s)**	**% deaths**	**Rank**	**Cause**	**Deaths (000s)**	**% deaths**
1	Ischaemic heart disease	7594	13,2	**1**	Ischaemic heart disease	9245	13,2
2	Stroke	6700	11,7	**2**	Stroke	8578	12,2
3	Lower respiratory infections	3223	5,6	**3**	Chronic obstructive pulmonary disease	4568	6,5
4	Chronic obstructive pulmonary disease	3217	5,6	**4**	Lower respiratory infections	3535	5,0
5	Diarrhoeal diseases	1808	3,2	**5**	Diabetes mellitus	2464	3,5
6	HIV/AIDS	1667	2,9	**6**	Trachea, bronchus, lung cancers	2413	3,4
7	Trachea, bronchus,lung cancers	1636	2,9	**7**	**Road injury**	**1854**	**2,6**
8	Diabetes mellitus	1556	2,7	**8**	HIV/AIDS	1793	2,6
9	**Road injury**	**1423**	**2,5**	**9**	Diarrhoeal diseases	1617	2,3
10	Hypertensive heart disease	1137	2,0	**10**	Hypertensive heart disease	1457	2,1
11	Preterm birth complications	1133	2,0	**11**	Cirrhosis of the liver	1201	1,7
12	Cirrhosis of the liver	1028	1,8	**12**	Liver cancer	1186	1,7
13	Tuberculosis	887	1,5	**13**	Kidney diseases	1152	1,6
14	Kidney diseases	871	1,5	**14**	Stomach cancer	1143	1,6
15	Self-harm	836	1,5	**15**	Colon and rectum cancers	1075	1,5

2.2. Integrated Approach for Road Safety

The history of road accidents reaches back to 1899 when the first accident occurred as a man called H. H. Bliss was hit by a horseless carriage as he got out of a streetcar. Hence, since beginning of 19th century, towards safer roads, new frameworks have been tried to be developed. As general point of view of safety, 3 E’s, 4 E’s and 5 E’s approaches have been developed up to now. The approach that is called “3E Principles” had been invented first by Julien H. Harvey, at a meeting in Topeka in 1923, who afterwards became the director of Kansas City Safety Council. While answering some questions related with highway safety, he drew a triangle on the blackboard with “E” letters on each corner which represent Engineering, Education and Enforcement (Damon, 1958). Although this approach is used in road safety mostly, it is also used in a wide variety of safety concepts such as fire prevention.

These three E’s compose a structure that works all together in a co-operational way. However, naturally, each one has different sub concepts. From the perspective of road safety, according to Nineteenth Annual Report and Resolution of the Council of Ministers which was prepared by European Conference of Ministers of Transport in 1972, engineering among these three can be divided into two; “Road Construction” and “Vehicle Manufacture”. Roads must be studied well by doing adequate engineering in order to increase the safety of them (ECMT, 1972). All of these structures and signs are results of road engineers’ works from zebra crossings to guardrails, from design of the road to road safety audits. On the other hand, the design of the vehicles is another essential point in point of view of safety. Vehicles also must be manufactured with the understanding of this fact. Creating suitable designs that comply with the needs of safety is crucial (ECMT, 1972). Secondly, education is important in order to deliver all the people who compose traffic what to do and how to do while in traffic.

These educations must be given to all kind of road users. These include lectures in schools, seminars, trainings and campaigns. While it is important to cover all road users during these education processes, they must be well-prepared considering different age-groups and different kind of road users so that people should be leaded to behave in a way much safer in traffic (ECMT, 1972). Third “E”, “Enforcement” is another key element because apart from trainings and educational activities, in some cases, people must be directed also by using some rules. Law enforcement which can also be called as legislation and regulations namely, is another way of managing and controlling people's behavior in traffic. Such as speeding, drink-driving, following other cars too closely, parking to non-parking areas, paying attention to signalizations and signings etc. must be regulated by law that are applied by authorities. (ECMT, 1972)

However, afterwards, some authorities developed a more expanded approach which has been seen more beneficial. In this manner, the fourth E has been added to the concept of this approach which is "Emergency Services". This principle requires the usage of emergency services in case of accidents in a well-organized way. The authorities that are included in case of emergency situations on roads must be defined as well as their responsibilities. Moreover, a new concept which is called as 5E has been developed eventually. This approach includes;

- Engineering
- Education
- Enforcement
- Emergency
- Evaluation

Evaluation is also very crucial about road safety. Since all the studies and precautions must be based on evidences and scientific reasons, a well evaluation must be done by the relevant authorities. Additionally, evaluation also includes creating enough funds in order to increase the level of countermeasures against road safety hazards by taking the specific needs into account. In conclusion, it must be noted that while some governments and states use only 3 E's, some of them use 4 E's and 5 E's as well depending on the conditions.

2.3. Road Safety in Turkey

As mentioned before, road traffic accidents are a major part of deaths and injuries in the world. Apart from costs to human health, these accidents also have an important cost on economy of the countries. The loss related with traffic accidents by means of economic cost is estimated to be around 518 US$ which is around 1-3% of the countries' gross national products (WHO, 2009). Mostly affected countries are the developing ones in which while the number of accidents is higher related with the ones that occur in developed countries, also the severity of the accidents is heavier. According to WHO, there are 15.1 m registered vehicles in Turkey. The road traffic deaths per 100 000 people is around 13.4. About 10 000 people lose their lives in Turkey every year. These statistics show that Turkey is among the countries in which road traffic safety is subject to be taken care of immediately (WHO, 2009).

As can be seen on *Table 2.2*, Turkey's place is not pleasing in the world. As a middle-income country, Turkey, especially in the last years started to give huge importance to road safety. Some external factors such as European Union (EU) Compatibility Standards forces Turkey in the way of having improvements about road safety. Also, some other programs are conducted with the cooperation of internal and external authorities. "Decade of action for road safety 2011-2020" can be given as one of these examples. In this action plan, for instance, WHO, Ministry of Health of Turkey, General Directorate of Highways (GDH) and Traffic Department of Police are the participants. The main scope of these actions is to decrease the number of accidents, at the same time decreasing the fatality relating to them. In order to be able to have a clearer understanding about the significance of road accidents in Turkey, *Table 2.3* given below should be given a look.

Another important point related with the road accident statistics is the type of faults causing them. Apart from new legislations and regulations about traffic rules such as drunk-driving, wearing seatbelts and so on, the infrastructure of the roads also is a substance matter of road safety. Since, in this thesis, mostly infrastructure of roads will be studied, it is important to underline percentage of it among type of faults in road accidents. The share of road infrastructure deficiency to the others in the distribution of road accident reasons is given in *Table 2.4*;

Table 2.2 *Road Accident Deaths per 1 000 000 inhabitants 2014-2016 (OECD, 2018)*

Location	**2014**	**2015**	**2016**
Australia	49,0	50,60	53,70
Austria	50,3	55,50	49,40
Azerbaijan	117,9	92,70	77,80
Belgium	64,9	64,90	56,10
Bulgaria	91,4	98,60	99,30
Croatia	72,7	82,80	73,60
Czech Republic	65,4	70,00	57,90
Denmark	32,3	31,30	36,80
Estonia	54,8	46,40	47,90
Finland	41,9	49,30	46,60
France	51,0	52,00	52,00
Georgia	137,1	161,90	156,20
Germany	41,7	42,30	38,80
Greece	73,0	73,30	75,90
Hungary	63,50	65,40	61,80
India	108,00	111,60	113,90
Ireland	41,80	34,60	39,00
Israel	34,00	38,40	39,20
Italy	55,60	56,50	54,20
Japan	38,00	38,40	37,00
Korea	93,80	90,60	83,80
Latvia	106,30	95,10	80,60
Lithuania	91,00	83,30	66,80
Moldova	91,10	84,40	87,60
Montenegro	104,50	82,00	104,40
New Zealand	65,00	69,40	69,90
Poland	84,20	77,30	79,70
Romania	91,30	95,50	97,10
Russia	187,40	160,40	140,70
Serbia	75,20	84,70	85,90
Slovak Republic	54,40	57,20	50,70
Slovenia	52,40	58,10	63,00
Spain	36,30	36,40	39,00
Turkey	**45,80**	**96,20**	**91,80**

Table 2.3. *Road Traffic Accidents Statistics (TurkStat, 2017)*

Year	Total number of accidents	Number of accidents involving		Number of persons killed			Number of Persons Injured
		Death or injury	Property damage only	Total	At accident scene	During follow-up(1)	
2007	825 561	106 994	718 567	5 007	5 007	NA	189 057
2008	950 120	104 212	845 908	4 236	4 236	NA	184 468
2009	1 053 346	111 121	942 225	4 324	4 324	NA	201 380
2010	1 106 201	116 804	989 397	4 045	4 045	NA	211 496
2011	1 228 928	131 845	1 097 083	3 835	3 835	NA	238 074
2012	1 296 634	153 552	1 143 082	3 750	3 750	NA	268 079
2013	1 207 354	161 306	1 046 048	3 685	3 685	NA	274 829
2014	1 199 010	168 512	1 030 498	3 524	3 524	NA	285 059
2015	1 313 359	183 011	1 130 348	7 530	3 831	3 699	304 421
2016	1 182 491	185 128	997 363	7 300	3 493	3 807	303 812
2017	1202716	182669	1 020 047	7427	3534	3 893	300 383

(1) Includes the deaths within 30 days after the traffic accidents due to related accident and its impacts for people who were injured and sent to health facilities.

GDH has been working on improvements on the infrastructure of the existing roads such as horizontal and vertical geometry and signings, signalizations, guardrails etc. As a formal advancement of road safety audit and inspection in Turkey, by the directives of EU, in the ends of 2013, Road Safety Management Directive Harmonization Committee has been founded by GDH. In the light of this progresses, short, middle and long termed targets have been defined and some education programs have been organized in order to have foreign experts train the relevant staff in Turkey.

Table 2.4. *Number of faults causing traffic accidents involving death or injury, 2009 – 2016 (TURKSTAT, 2016)*

Year	Total Faults	Driver faults		Passenger Faults		Pedestrian Faults		Road faults		Vehicle Faults	
		Total	(%)	Total	(%)	Total	(%)	Total	(%)	Total	(%)
2009	**155 982**	139 758	89,6	640	0,4	14 181	9,1	958	0,6	445	0,3
2010	**157 970**	141 728	89,7	564	0,4	14 171	9,0	992	0,6	515	0,3
2011	**174 605**	157 494	90,2	677	0,4	14 860	8,5	1 044	0,6	530	0,3
2012	**181 266**	161 076	88,9	797	0,4	17 672	9,7	1 124	0,6	597	0,3
2013	**183 030**	162 327	88,7	774	0,4	16 458	9,0	1 913	1,0	1 558	0,9
2014	**193 215**	171 236	88,6	901	0,5	18 115	9,4	1 841	1,0	1 122	0,6
2015	**210 498**	187 980	89,3	915	0,4	18 522	8,8	1 916	0,9	1 165	0,6
2016	**213 149**	190 954	89,6	869	0,4	18 612	8,7	1 717	0,8	997	0,5

2.4. Traffic Safety and Legislation in Turkey

In Turkey, Highway Traffic Law that is in charge was claimed in 1983. The main objective of the law was described as to define the precautions about the subjects regarding traffic safety and ensuring traffic order in the manner of security of life and property on highways. This law covers the rules, conditions, right and obligations about traffic, application and supervision of these, related authorities and its duties, responsibilities and work methods and other provisions.

The related committees that have been recognized are given in the law. These committees have been founded in order to determine the aims, enforce these aims and provide coordination among themselves. These committees are given below;

-*Highway Safety Supreme Board*

This board consists of minister of justice, interior, treasury, education, public works and settlement, health, transportation, forestry, the ministry to which general directorate of rural services, commander of the Turkish gendarmerie forces , secretariat of the state planning organization, director general of public security and director general of highways under the presidency of prime minister.

-Highway Traffic Safety Board

Chief of the traffic services of public security heads the board and the participants can be listed as; at least the head of any related department of public institution which is n inclusive of highway safety supreme board, general commandership of gendarmerie, Turkish standards institute, drivers and motoring federation, engineering chambers etc.

The responsibilities and duties of the authorities that constitute these boards have been given in the law.

Police directorate is mainly responsible for drivers and vehicles. Its duty is to inspect and impose necessary punishments in case of conditions against legislations. Also, various type of facilities concerned with highways are the field of interest of police headquarters. There are also other duties that belong to security directorate which will not be mentioned here.

GDH also have many responsibilities according to traffic law. Mainly, the duties of GDH can be explained as to implement legislations on the highways for which it is responsible on the construction and maintenance. These legislations include placing the required signs, constructing the necessary intersections and interchanges, parking areas, needful facilities on these roads. On the other hands, determining the road speed limits, working on the projects that are related with the road safety, inspecting the accidents, preparing the statistical data are other duties of this establishment. In the point of view of road design, GDH has published a design manual which is called as Design Handbook and can be considered as a summary of AASHTO Green Book. By this manner, GDH is one of the major interest areas of this thesis.

According to traffic law, ministry of national education, youth and sports is generally responsible for the necessary educations and exams of the ones involved in traffic. Also, issuing the proficiency documents is a duty of this ministry.

Ministry of health, as described in traffic law, is in charge of planning and organizing the first and emergency aid services on the occasions of road accidents, providing a rapid health service to the casualties by facilitating first aid stations and required and educated personnel in these stations, keeping ambulances and medical personnel present and ready at nearby spots of highways.

Transportation ministry is responsible for providing required coordination for highway transportation, doing inspections of the registered vehicles, checking the vehicle inspections stations, controlling the weight and size of the vehicles that are on the traffic according to traffic law.

As defined in the traffic law, the ministry of agriculture, forestry and rural services are responsible for the forestry and rural roads. Main duties are to implement the necessary arrangements, take the required precautions, inspect the stops, intersections and parking areas.

Municipality traffic units have the responsibility of keeping the roads which are under its authorization in a position that will have provided traffic order and safety. On the intersections, where it is considered necessary, putting traffic lights, signings and markers is also another duty of municipalities. Taking required precautions while there is a working on the road, removing or making the dangerous objects visible, organizing traffic educations for the children also can be listed as the duties of municipalities.

Province and county committees are in charge of taking required precautions in order to provide the traffic order and safety, working on the infrastructure of the roads, arranging the rules and routes of the commercial transportations, assigning empty areas as parking areas to the natural and legal people according to traffic law.

In traffic law, after defining these duties, the required regulations are described about all the components of traffic such as road markings, facilities, terms and conditions on vehicles, traffic documents, motorized vehicles, inspection of vehicles, drivers' licenses and drivers, traffic rules, traffic accidents, legal responsibilities and insurance, imposing penalties, education, schools and child education parks, permeable and temporary provisions. However, in traffic law, in the point of view of design of the roads, the only criteria that is discussed is the road hierarchy and the speed limits. It is stated that, the speed limit on the intercity roads is 90 km/h, on the divided highways it is 110 km/h while on the freeways it is defined as 120 km/h.

In accordance with traffic law, highway traffic regulation has been issued in Turkey. Its scope is to define the procedures and principles about the measurements that will be taken about providing road traffic order and safety by the means of security of life and property. This regulation includes the clear statements and definitions of the rules in detail that are under charge of the authorities which were described in the traffic law.

Apart from traffic law and regulations, as a technical specification, there is highway technical specification. In this specification, besides law legislations, the rules and principles of construction of highways have been given. It is a guide that must be followed during the construction of a roadway.

There is also Urban Law which was issued in 1985 in Turkey. However, in this law, there is not a definition of how to design or construct roads. It just declares some statements about road and housing interactions such as the distance of an estate to a road.

2.5. Road Safety Evaluation Efforts in TR

In Turkey, firstly, National Traffic Safety Program was published in 2001 related with road safety evaluation. In this publishing, the aim was to prepare a vision, strategy and an action plan for the time period of 2002-2011. The program was financed mostly by World Bank and managed with a collaboration of a university and a few ministries (Sweroad, 2001). Under the umbrella of this project, a safety audit manual (handbook) was released in 2001. This has been the first handbook of GDH about conducting road safety audit on existing and new roads. The audits are planned to be done in a systematically way by a group of a client, a designer and a safety audit group. For new roads, there are some checklists that are used for auditing procedure. For existing roads, the audit is done by defining black spots considering the places where there are highly frequent accidents. GDH has created a database of these black spots on its highway network and continuously publishes them on its website (Sweroad, 2001).

Afterwards, Ministry of Interior initiated a traffic safety project at the beginning of 2008. This project, later on, has been reorganized and updated as Action Plan of Turkey between 2010-2020. This action included five ministries including GDH. On its way to get on board of EU, Turkey implemented the directives of EC about road safety in 2008 by proper and consistent accident data acquisition, RSA and RSI Training for staff and related software (Çelik, 2011).

CHAPTER 3

ROAD SAFETY EVALUATION

3.1. History of Road Safety Audit

Road safety audit (RSA) was first initiated in UK in 1980s (IHT, 2008). It was formally introduced by some local authorities that work on safety measurements on railways (Jones, 2013). However, later, it spread quickly to a national wide level beginning especially from Australia and New Zealand (IHT, 2008). By 1996, Design Manual for Roads and Bridges (DMRB) and Institution of Highways and Transportation (IHT) claimed two sets of Guidelines and Standards.

The reason why this system spread quickly because the engineers managed to consider the fact that there is no need to wait to see the collisions on the roads in order to see the problematic points and take the required measurements and it could be possible to foresee these locations once a systematic scheme for road design and construction is established. Because of this reason, as mentioned before, RSA spread firstly to Australia, New Zealand, Denmark and Ireland. By 2007, RSA has been a common procedure for most Western Europe Countries (IHT, 2008). Although the first leader on implementing RSA in the North America was Canada, in 1996, the Federal Highway Administration of USA arranged technical visits to Australia and New Zealand in order to learn how RSA is processed and applied so that their engineers had the chance to work with Australian engineers which has been beneficial for practice. Afterwards, a pilot program has been organized among thirteen states to develop RSA practices in USA (Wilson and Lipinski, 2004). These developments led RSA to attract attention and after that the popularity and acceptance of RSA started to increase rapidly. This raise of popularity has not been limited to USA, Australia and New Zealand and it was recognized by almost all of the countries in the world since it has been considered as a very practical and economical way of preventing loss of lives and wealth (Jones, 2013).

3.2. EU Perspective – Road Safety Initiative

EU has been delivering great attention to road safety especially in the last a few decades beginning from 1984, Commission of the European Communities (CEC) states in 1997. In this sense, in 1993, CEC has prepared the first action programme on road safety to emphasize the importance of this problem (CEC, 1997). In this action programme, it was agreed that the results of this paper were going to be discussed three years later. Looking at the results, it was clear that a great effort had been given in order to increase the road safety and all of the objectives had been accomplished in accordance with the strategies that had been defined. The success of this paper led to reality of great reduction in fatality and severity of road accidents (CEC, 1997). Therefore, the commission decided to go on the same policy more eagerly, presenting the new action programme; Road Safety 1997 – 2001 (CEC, 1997). As in the previous one, the main objective of this paper is also to attract attention of the member states on the fact how huge the consequences of road fatalities are. By this manner, the measures that can be taken such as pedestrian friendly cars, seatbelt wearing, road infrastructure development, arrangement of vehicle speeds have been mentioned (CEC, 1997). Later on, despite the developments in the desired ways, Commission published its White Paper on European Transport Policy named as "European transport policy for 2010: time to decide" in 2001 (CEC, 2003). In this paper, the aim was a bit more deterministic by targeting to halve the number of road accidents by 2010 in the EU. Hence, paying regard to same objective, as the 3rd of its kind, Road Safety Action Programme 2003 – 2010 has been presented. In summary, the main efforts that were going to be taken care of regarding this programme can be given as improving road user behavior, improving safety of vehicles and improving road infrastructure (CEC, 2003). Then, at the end of 3rd road safety action programme, road safety issue still kept an important role in the Commission of EU. While it was indicated that in 2009, more than 35000 people died on the roads of the EU, at the same time the evaluation of the 3rd action programme was remarking this number was not totally complying with the targets of halving the fatalities, a major decrease has taken place by 2010 though (CEC, 2010).

Although this objective has not been able to be accomplished by the deadline, the programme still managed to achieve success in reducing the number of road fatalities which can be seen on Figure 3.1. This underachievement showed that, in spite of great progress, the action must be kept on with a higher interest. The commission proposes to continue on the previous target which was to halve the number of road accidents in 3rd road safety action programme till the end of 2020. Thus, the 4th road safety action programme has been claimed with the same main target starting from 2010 named as "Towards a European road safety area: policy orientations on road safety 2011-2020" (CEC, 2010).

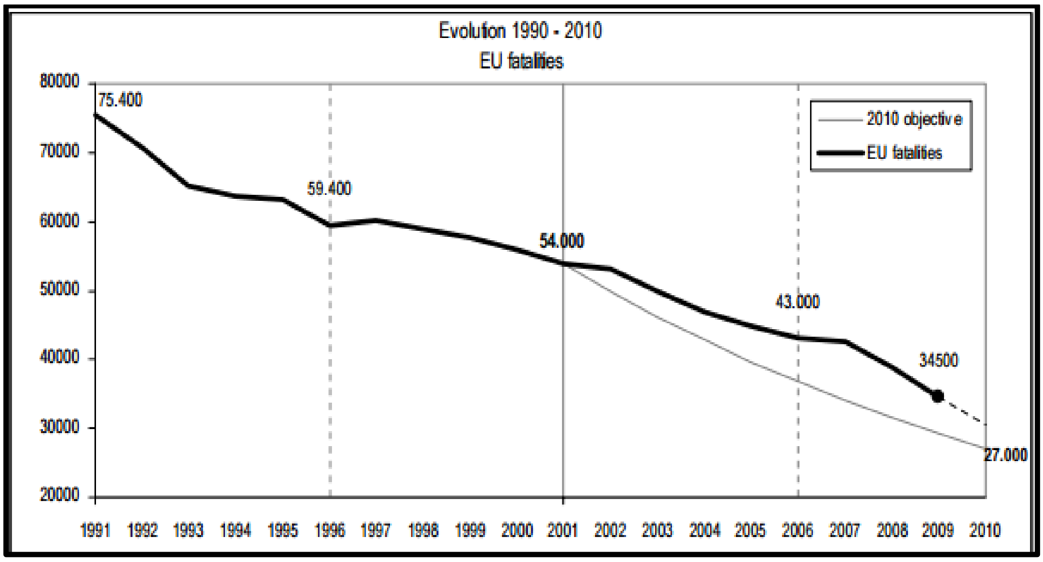

Figure 3.1. Evolution 1990 – 2010 EU Fatalities (CEC, 2010)

Apart from this main target which is halving the number of road accidents, there are also 7 strategic objectives that have been underlined in the action plan. These can be given as below;

- Improve education and training of road users
- Increase enforcement of road rules
- Safer road infrastructure
- Safer vehicles
- Promote the use of modern technology to increase road safety
- Improve emergency and post-injury services
- Focusing on vulnerable road users (CEC, 2010)

After the claim of 4th road safety action plan, CEC published its White Paper on Roadmap to a Single European Transport Area – Towards a competitive and resource efficient transport system. The main objective of this paper was to prepare the European transport area for the future. Hence, it does not only include the policies about road safety but also many other aspects of transportation. On wide range of topics such as eco-driving, urban mobility plans, cargo transportation, rail safety, civil aviation, shipping, passenger rights etc. are all contents of this paper in order to carry the transportation in Europe into a decent level (CEC, 2011). Although, as in 4th road safety action plan, the target has been defined as to halve the number of road fatalities by 2020, in this paper, an additional target has been defined which is to reach to 'zero fatality' by 2050 (Ratcliff, 2017). It was also stated that the applications are planned to be executed with the cooperation between member states and the commission. Thus, the countries are encouraged to establish their national road safety policy authorities which will be helped and guided by commission. At the same time, The European Road Safety Charter which was founded by the commission in 2004 focuses on the civil authorities on the related policies and targets (Ratcliff, 2017).

For a more detailed and specific approach on road infrastructure in the meaning of road safety, EU has published a directive for the member states in 2008 which is called as "Directive 2008/96/EC Of The European Parliament And Of The Council on Road Infrastructure Safety Management" (Ratcliff, 2017). This directive includes the following main articles;

- Road safety impact assessment for infrastructure projects
- RSA for infrastructure projects
- Ranking of high accident concentration sections and network safety ranking
- Accident information contained in accident reports (EP, 2008).

Along with these directives, although national and regional authorities are responsible for these infrastructures that belong to them, EU provides funding for better roads which are in its development region. This region is mainly the Trans-European Road Network (TERN). The directives are valid for the roads which are part of TERN whether they are at design phase, construction phase or existing (Cullen, 2012).

In short, the formal documents that have been published by EU on road safety policy can be summarized as in below;

- "Road Safety Strategy in the EU 1st Road Safety Action Plan (1993-1996): Integrated approach of road safety with qualitative targets and specific priorities.
- 2nd Road Safety Action Plan (1997-2001): Target of reducing the annual number of road deaths by at least 18.000 by 2010.
- 3rd Road Safety Action Plan (2003 -2010): Reduce the number of road deaths by 50% by 2010 comparing to those in 2000.
- European Road Safety Action Plan (2011-2020): Reduce the number of road deaths by 50% by 2020 comparing to those in 2010.

Directive on Road Infrastructure Safety Management (2008/96/EC): Road Safety Audits for Infrastructure Projects, Road Safety Inspections, Safety Assessment and Ranking, Safety Management" (Yannis et al., 2012)

3.3. Road Safety Evaluations: Audit, Inspection and Review

3.3.1. Road Safety Audit and Inspection

Usage of roads are inevitable in our daily lives. Every day, many people are driving or using the mass transportation systems on the roads. The high usage of the roads brings the fact of high number of crashes with it. As mentioned before, the hazards of road accidents are vital in both economical and health manners. This situation leads us to road safety concept. In order to minimize the number of road crashes and also the severity of the accidents which cannot be eliminated, especially in the last decades the studies about road safety concept has been carried on with much interest and importance.

In the last decades, the roadway and roadway infrastructure component of road safety has gained a new methodology which is called as Road Safety Audit and Inspection. That part of road safety can be considered as an answer to this question; is there a reason about why the user or vehicle on this roadway is not able to keep its regular travel?

RSA can be defined as a formal assessment which is done by an independent team composed of multidisciplinary people on existing roads or design process of future roads in order to examine the safety concept. Although, in some countries, especially English-Speaking countries such as USA and Australia, RSA is defined in this way, especially in European Countries, RSA is seen to cover only the design processes of future roads. In these countries, the examination of the existing roads is defined as Road Safety Inspection (RSI) (Nadler, nadler and strnad, 2014). Hence, stages of and steps during performing a road safety can be summarized as in *Table 3.1* and *Table 3.2* respectively. As mentioned, some countries divide RSI out of RSA procedures. This individual RSI includes its own stages. Since it is conducted only on existing roads, the procedure differs as concentrated on existing roads. Although it is similar to existing RSA, it is beneficial to mention its stages. Below in *Table 3.3* is given explaining how to conduct RSI.

3.3.2. Road Safety Review

Apart from RSA and RSI, there is another method for the same examination which can be described as the most traditional one. Road Safety Review (RSR) is the procedure that has been started to be used very long ago. This procedure differs from country to country according to the needs. It did not start as a standardized study. The aim of this was mainly to spot the black points and deficiencies on the road networks and try to increase the safety on these locations. Hence, since traditional RSR is mainly applied after high number of crashes occur on a spot such as intersection or an alignment segment, a more detailed review has gained the need of development of RSA and RSI which have been defined above. (Austroads, 2002)

RSR can be described as the procedure that identifies hazards and/or safety deficiencies in road design, layout and road instruments. RSR is composed of three stages. The first one is the office review. This stage includes the jobs that must be done in office before going on site which is summarized in. Site description and crash data analysis are the main jobs in this stage. The second stage is called as the field review. During field review, road survey, checklist studies and speed data examinations are conducted which are given in *Table 3.5*. And the last stage is to write the final report (Austroads, 2002).

Table 3.1. *Stages of RSA Process (Wilson&Lipinski, 1999)*

Stage 1: Feasibility (Planning Stage): This is done during the development of the project. The options such as possible route corridors, layouts, treatments, facilities, intersection and interchange locations are evaluated while also the impact on the existing road networks are considered.
Stage 2: Draft (Preliminary) Design Stage: It includes consideration of general design standards. These standards can be listed as horizontal and vertical alignment, intersection and interchange types and schemes, sight distances, lane widths, superelevation, conditions for pedestrians and bicyclists.
Stage 3: Detailed Design Stage: This requires the review of all the final situations of elements such as geometric design features, signaling, markings, guardrails, drainage, lighting, intersection and interchange design details etc. Also, special circumstances like elderly or handicapped road users are considered.
Stage 4: Pre-Opening Stage: Last audit just before the opening of the road to traffic in order to make sure about all the safety related concerns are handled well and all of the hazardous conditions are eliminated. This check must be done both during day and night, in both wet and dry conditions and for all the type of use; driving, walking and riding.
Stage 5: Existing Road Audits (Road Safety Inspection): This audit evaluates the safety situation of the existing roads. The review area is about whether the existing roads satisfy the required safety level for all of the road users. This audit is also important since the roads undergo changes by time after being opened to traffic. However, this audit also can be conducted for the roads that are just opened to traffic.

Table 3.2 *Steps during Performing RSA (Wilson&Lipinski, 1999)*

1. Select the Road Safety Audit Team: The auditors selected must be independent from the design team. At the same time, safety engineering skills and experience is also a must. The reason behind the significance of being independent is to have the design evaluated by distinct reviewers. This situation leads the road to be examined by different kind of backgrounds and viewpoints so that the problems can be noticed much more easily. The auditors are selected by the client or the designer.
2. Provide the Background Information In this step, after selecting the proper audit team, the auditors are given the necessary information about the project. This information includes the design standards that were used, traffic volumes, crash reports, plans, reports and other relevant outputs that will be considered during auditing.
3. Holding an Initiation Meeting The auditors are called for a commencement meeting and the data is given to them by the client. In this meeting, also the scope of the audit must be defined and the duty of the auditors and the project manager must be clarified.
4. Evaluate all documents All the documents are evaluated by the auditors. The points which may be problematic are specified. In case of any questionable situation, the designer or the client must be informed. The already defined checklists must be used in order to find out potential problems.
5. Inspect on Field The auditors must go to and see the site. These visits should be both on day and night. This review must be done in accordance with the documents in hand. Also the adjacent roads must be taken into consideration. The views of all the potential road users must be included in the survey.
6. Writing the road safety audit report This report indicates the safety inadequacies and then proposes the precautions for these inadequate points. However, these proposed precautions must be directly for the solution of the problems not describing the problem in details. Especially the problems which own a high level of urgency must be emphasized.
7. Holding a Completion Meeting In this meeting, the auditor team submits all the data they have generated. Also, an independent auditor team must also see the problems and express their opinions about them. The questions of this independent audit team also must be answered.
8. Writing the Response Report The designer or client must prepare a report which has responses to the report that was prepared by the auditor team. This report must cover all the points that were identified in the report of the auditors and also it must indicate the parts which are accepted including the corrective actions that will be taken and which are rejected.
9. Taking Agreed Corrective Action The designer must implement the required solutions which have been agreed on by audit team and the designer.
10. Feedback All the experience that was gained during this audit process must be fed back to the design process while it also must be directed to other projects, other designers, other auditors and standards. Besides, the project that was audited must be observed for up to three years in order to see the results whether the audit was successful or not.

Table 3.3. *Stages of Conducting RSI (Nadler et al., 2014)*

1. Preparation for a Road Safety Inspection a. Accident Evaluation b. Technical Traffic Aspects c. Other Evaluations (such as speed measurements, following distance measurements etc.) d. Review of The Available Documents
2. Section Inspection on Site
3. Creation of Report b. General Information c. Checklist and Accident Evaluation d. List of Measures e. Evaluating the Time Frame
4. Documentation and Monitoring

Table 3.4. *Tasks in Office Review of RSR (Macaya, 2003)*

13. **Site Description:**This description involves the geographic definition, demographic situation and also a short explanation of the history of the area. Then, the characterization of the roadways must be processed. The land classification, functionality of the roadways, speed limit on these roadways etc. should be reviewed. While doing these studies, one does not go to the site, instead, uses the geodesic data that can be gathered such as GIS etc.
14. **Crash Data Analysis:**The reasons of evaluating crash reports in the related area can be given as below; b. To discover the reasons of the crashes c. To define high-risk spots in the point of view of crashes d. To use in the selection of the countermeasures to be taken e. To evaluate the result of the countermeasures that have been taken However, main scope of this analysis can be summarized as to reduce the number and severity of the crashes occur in the related roadways. • Crash Rates In order to be able to generate a complete crash analysis and evaluation, the crash data must be normalized by the crash rates. By using crash rates, it is possible to compare the crash data of roadway with different segments and cross sections. Crash rates can be calculated as follows; • Crash Data Classification In this classification, the crash reports are categorized in three different aspects; location, pattern and cause. o Crash Data by Location In this analysis, the crash reports are examined according to the location of the crash. The roadway is separated into segments so that the crashes are assigned to relevant segment in order to identify the most problematic points on the roadway such as intersections, curves, bridges (Hummer, 1994). o Crash Data by Pattern The crashes are categorized according to the way the crash occurred. There are extremely number of ways a crash can occur but some major classifications can be created. One example is given below in the figure; o Crash Data by Cause There are many different factors that cause the crashes on the roads such as driver, vehicle, roadway and environment effects. In this data classification, these reasons are categorized in order to determine the major reasons of the crashes occur in that area.

Table 3.5. *Tasks in Field Review of RSR (Macaya, 2003)*

13. Road Survey:

The first element of field review is road survey. However, in this element, not only the physical situation of the road is evaluated but also the traffic operations, roadside area, users of the roadway and environmental conditions etc.

a. Geometric Design Elements:
- Speed
- Road Alignment:
- Cross section:
- Intersections:
- Auxiliary lanes:
- Clear Zones and crash barriers
- Pavement:
- Bridges and Culverts
- Lightning

b. Roadway Activity
- Pedestrians and cyclists
- Parking and Public Transportation
- Heavy Vehicles, emergency vehicles and slow-moving vehicles

c. Environmental Considerations
- Weather
- Animals

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14. Traffic Control Devices

Traffic control devices are the elements of roadways that provide the communication with the drivers and users of the roads. These elements must be very clear and understandable. Pavement markings, signs, traffic signals and object markers can be given as an example of these elements. In the guideline, The Manual on Uniform Traffic Control Devices (MUTCD), the control devices are defined as "all signs, signals, markings, and other devices used to regulate, warn, or guide traffic, placed on, over, or adjacent to a street, highway, pedestrian facility, or bikeway by authority of a public agency having jurisdiction".

a. Signs
b. Traffic Signals
c. **Markings and Delineation**

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15. Checklists

Checklists are used in order to identify any potential roadway safety issue. Checklists should be considered in a way they may not always fit to the requirements of the roadway section that is being reviewed. Adaptive decisions should be made before defining and applying the checklists.

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16. Speed Data

The subjects related with speed are very important since the speed is the article that defines the safety, time, comfort and economics conditions of the roadways.

a. Speed Data Collection
b. Speed Data Analysis

3.3.3. Road Safety Audit versus Road Safety Review

A road safety audit is defined as a formal and systematic examination of a future road or intersection by an independent team of professionals while RSI is of existing road or intersection. However, as mentioned before, also RSA term is used for the examination of existing roads in some countries. However, it cannot be defined as a procedure that makes rating or ranking a project, check against compliance standards, an accident investigation, a redesign or an informal process (Wilson&Lipinski, 1999). On the other hand, RSR is a traditional safety assessment procedure aims to detect hazards and deficiencies in road design, layout and items. Although in general manner that look similar, it is important to understand the difference between the application of RSR and RSA since, in spite of the fact that many authorities already have adapted a road safety procedure, RSA should be clarified as a different method than RSR. The difference can be summarized in *Table 3.6* given below;

Table 3.6. *Differences between RSA and RSR (FHWA, 2014)*

Road Safety Audit	**Traditional Road Safety Review**
Performed by a team independent of the project	The safety review team is usually not completely independent of the design team.
Performed by a multi-disciplinary team	Typically performed by a team with only design and/or safety expertise.
Considers all potential road users	Often concentrates on motorized traffic.
Accounting for road user capabilities and limitations is an essential element of an RSA	Safety Reviews do not normally consider human factor issues.
Always generates a formal RSA report	Often does not generate a formal report.
A formal response report is an essential element of an RSA	Often does not generate a formal response report.

3.4. Checklists: The "Core" of the Road Safety Audit and Review

Checklists, firstly, have been used in UK in order to aid the traditional safety audits as mentioned before. However, by time, checklists have become the heart of safety audits and reviews. Hence, it is very vital to prepare a consistent and correct checklist in order to be able to have a decently prepared safety review by carefully taking the relevant situation into consideration. The usage of checklists is essential so that the auditors will not skip or miss any safety issue during audit process. (Wilson&Lipinski, 1999)

One of the most important instruments of RSA, inspections and reviews are the checklists that are created specifically for the case in hand or taken as a common one that has been recommended by some authorities. These checklists provide the relevant points that must be paid attention as an overview. Many authorities have their own checklists, however, for unique projects, unique checklists can be generated. Also, it should be noted that these checklists are mostly given separately for different stages of the audits. These help the auditor, inspector or reviewer to consider the points that require investigation. Hence, they can be defined as the main guidelines for the auditors. The checklist papers also include a column of “comments” so that a detailed description can be made about the related issue topic. In some cases, a column of “level of risk” can be added in order to categorize the issues according to their level of emergency so that the ones with high priority may be given a quick review. (Wilson&Lipinski, 1999)

Since there are different checklists for different authorities, stages and projects, first of all, the suitable checklist must be selected by the auditor team. While selecting this, the master checklists that have been created by some well accepted authorities can be used. However, the purpose of these checklists is not just to tick off the issues on them. Each topic must be evaluated by engineering judgement for a detailed identification and investigation. (Wilson&Lipinski, 1999)

3.5. Road Safety Evaluation Checklist for Urban Major Arterials

Although RSA, RSI and RSR can be conducted on any type of road network, each road type should be treated accordingly while preparing the checklists for safety review and audit. Since the items that each road type includes can show an alteration, it is important to identify the road type in the beginning of the checklist preparation process.

Auditor can use an already developed checklist while they can create a new one that suits to the roadway that is being studied. Mostly, before getting into the detailed checklists, a preliminary master checklist is prepared. The checklists can be divided as municipal (urban) and rural checklists. Hence, a specification must be made about the type of the roads. Urban roads can be divided into three categories; arterial roads, collector roads, local roads. Each of the road types would have a common checklist (MUDGI, 2013). Master checklists, then, are categorized according to design stages for the roads that are new and/or on upgrade phases which are also divided into planning, preliminary, detailed design, pre-opening and post opening phases. Additionally, it includes a category for the existing roads. After this classification, the subjects of the safety audit are categorized such as general, alignment and cross sections, intersections, interchanges, road surface, visual aids, physical objects, environmental conditions road users and access and adjacent development (UNB, 1999).

After examining the correct master topic of the checklist category, the detailed checklists arise. These checklists are composed of safety questions that require attention for the road category that is relevant with the road which is on subject. The questions are related on critical points that should be evaluated during audit. By a decent completion of a suitable checklist the audit team will lead to a well-studied safety examination. Although there are already prepared and suggested checklists, specific checklists can be created depending on the existing case. An example of a checklist for an existing road is given on appendix. In the next chapter, the checklist that will be used in this thesis will be explained in detail.

3.6. Road Safety Audit and Inspection for Urban Arterial Projects

Since an urban major arterial can be considered as a combination of both urban and rural roads, auditors can mix already developed checklists according to the needs.

Both checklists that belong to urban and rural road networks have items that fit to urban major arterials hence there is not a certain limitation to the use of checklists. The use of these checklists depends on the situation and need of the location. Especially, data available affects the preparation of checklists. Mostly, for an urban road, the main items, availability which should be taken into consideration while deciding on checklists are given below;

- Traffic Volumes
- Pedestrian Volumes
- Location Map of Key Pedestrian Nodes
- How is the Traffic Control at Specified Spots
- Pedestrian Collision - Collision History, and Collision Reports
- Aerial Photographs of specified Locations
- Speed Limits and Speed Surveys
- Future Planned Improvements
- Inventory of Missing Sidewalks, Informal Pathways, and Pedestrian Opportunity Areas
- Key land use features that influence crossings
- Location of bike lanes (MUDGI, 2013)

Each of these items are crucial for RSA procedure. In case of availability of all or most of these data, all aspects regarding with alignment and cross section, intersections, road surface, visual aids, physical object, road users, access and adjacent developments and parking should be evaluated (UNB, 1999). Both examples of regular and municipal checklists that have been gathered from a guideline of University of New Brunswick Transportation Group is presented in the appendix section. Considering both checklists, a recommended checklist table (*Table 3.7*) is prepared for urban major arterials by referencing to the tables given in the appendix. Although these are recommended for an urban arterial, in this thesis, the concept will cover only the issues related with General, Alignment and Cross Section, Intersections and Interchanges.

Table 3.7. *Recommended RSA Checklist for Urban Arterials*

Part-I General & Geometric Design Based Audit Items	
General (G)	
(G.A) Traffic Barrier Warrants **(G.B)** Landscaping **(G.C)** Temporary Work	**(G.D)** Headlight Glare **(G.E)** Accident Reports
Alignment and Cross Section (A&C)	
(A&C.A) Classification **(A&C.B**) Design Speed/Posted Speed **(A&C.C**) Route Selection / Alignment **(A&C.D**) Cross Sectional Elements **(A&C.E**) Drainage **(A&C.F**) Lane Width **(A&C.G)** Cross Slopes/Superelevation	**(A&C.H)** Pavement Widening **(A&C.I)** Alignment **(A&C.J)** Horizontal Alignment **(A&C.K)** Vertical Alignment **(A&C.L)** Combined Vertical & Horizontal **(A&C.M)** Sight Distances **(A&C.N)** Readability by Drivers
Intersections (X)	
(X.A) Quantity **(X.B)** Type **(X.C)** Location/Spacing **(X.D)** Visibility **(X.E)** Layout **(X.F)** Maneuvers **(X.G)** Auxiliary/Turning Lane	**(X.H)** Sight Distances **(X.I)** Markings **(X.J)** Signs **(X.K)** Signals **(X.L)** Signal Phasing **(X.M)** Warnings
Interchanges (I)	
(I. A) Location/Spacing **(I. B)** Weaving Lanes **(I. C)** Ramps **(I. D)** Exit Terminals	**(I. E)** Entrance Terminals **(I. F)** Service Road Systems **(I. G)** Lane Balance/Continuity **(I. H)** Auxiliary/Turning Lanes
Part-II Other Audit Items	
Road Surface	
Skid Resistance Pavement Defects Surface Texture	Ponding Manholes
Visual Aids & Physical Objects	
Pavement Markings Delineation Poles and Other Obstructions Hazardous Object Protection	Lighting Signs Culverts
Access and Adjacent Development	
Right of Way Proposed Development Driveways	Roadside Development Building Setbacks Bridge Structures

CHAPTER 4

A GUIDE FOR THE PROPOSED URBAN RSA CHECKLIST

4.1. General (G) Issues

During evaluation of checklists, the most important criteria is the design speed of the roadway. Here, in the roadway that is subject to this thesis has a design speed of 70 km/h according to the project. However, municipality uses 82 km/h as posted speed which will permit up to 90 km/h regarding with the legislation. Hence, both speeds will be checked during the studies where possible considering the checklists given in *Table 3.7*. In this chapter, general issues related with RSA are defined. Sub-items of general issues and the required controls regarding with those are given in *Table 4.1*. Details of how to make these controls will be described. However, some of the items, which are not be able to be examined in case study chapter, will not explained.

Table 4.1. *General issues in RSA*

Sub Item	**Issues**
(G.A) Traffic Barrier Warrants	**1.** Presence of non-traversable or fixed object hazards within clear zone
	2. Does a potential risk exist for vehicles crossing over the median into the path of an opposing vehicle?
	3. Accident history of area
(G.B) Landscaping	**1.** Landscaping along road in accordance with guidelines?
	2.Required clearances and sight distances restricted due to future plant growth?
(G.C) Temporary Work	**1.** Interaction between temporary work and traffic flow
	2. Is temporary work adequately signed?
	3. Does temporary work signage remain even though construction is complete?
	4. Visibility of temporary work area from approaching traffic.
(G.D) Headlight Glare	**1.** Severity of head light glare during night time operations.
(G.E) Accident Reports	**1.** Accident reports available for specific facility?
	2. Frequency of accidents at facility
	3. Common accident characteristics discussed in reports

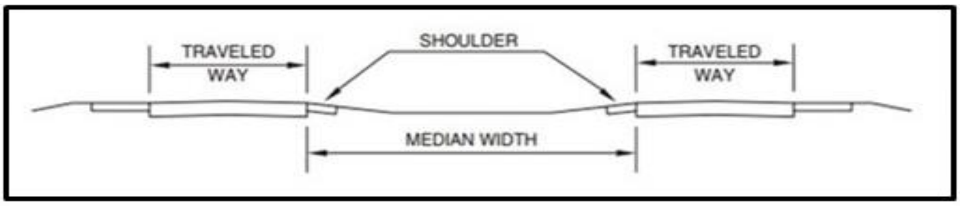
4.1.1. (G.A) Traffic Barrier Warrants

“Clear Zone” term is used to define the traversable section on a roadway beyond the edge of the through lane which is clear of unyielding objects. Shoulders, bicycle lanes, and auxiliary lanes are included in this zone (AASHTO, 2011). Apart from head-on crashes, objects such as sign poles, trees, bridge piers, culverts etc. that stand beside through travel lane increase the severity of fatalities on errant vehicles (AASHTO, 2011). Although clear zone design is explained in detail in AASHTO Roadside Design Guide manual, the parameters that should be considered are composed of many different features such as side slopes and curve factors and does not cover the roadways with curbs. Therefore, for guidance of clear zone width, *Table 4.2* is designated in roadway design manual of Texas Department of Transportation which was revised lastly in 2018 will be used.

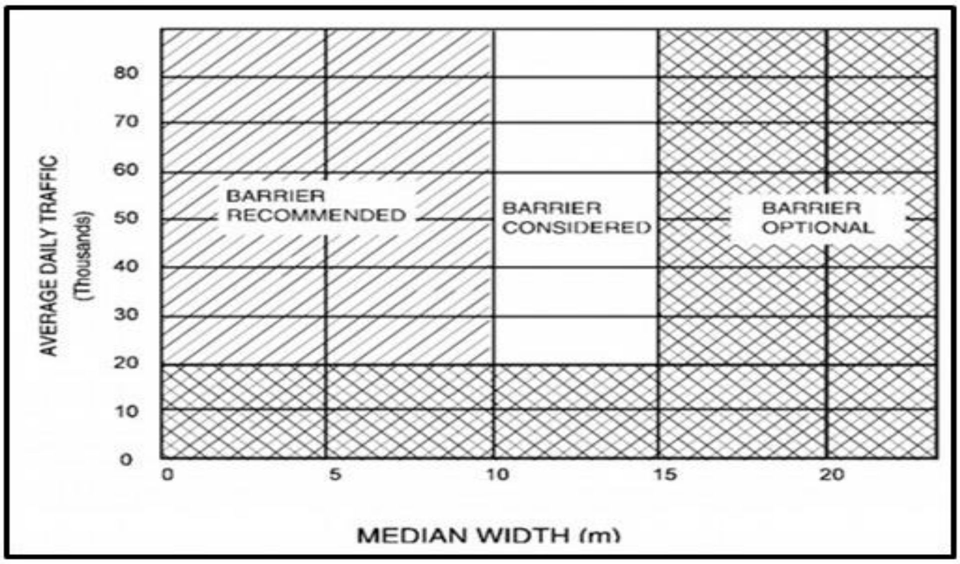
Table 4.2. *Required Clear Zones (TDT, 2018)*

Location	Functional Classification	Design Speed (mph)	Avg. Daily Traffic	Clear Zone Width (ft)[3,4,5]	
-	-	-	-	Minimum	Desirable
Rural	Freeways	All	All	30 (16 for ramps)	
Rural	Arterial	All	0 - 750 750 - 1500 >1500	10 16 30	16 30 --
Rural	Collector	≥ 50	All	Use above rural arterial criteria.	
Rural	Collector	≤ 45	All	10	--
Rural	Local	All	All	10	--
Suburban	All	All	<8,000	10[6]	10[6]
Suburban	All	All	8,000 - 12,000	10[6]	20[6]
Suburban	All	All	12,000 - 16,000	10[6]	25[6]
Suburban	All	All	>16,000	20[6]	30[6]
Urban	Freeways	All	All	30 (16 for ramps)	
Urban	All (Curbed)	≥ 50	All	Use above suburban criteria insofar as available border width permits.	
Urban	All (Curbed)	≤ 45	All	4 from curb face	6
Urban	All (Uncurbed)	≥ 50	All	Use above suburban criteria.	
Urban	All (Uncurbed)	≤ 45	All	10	--

Median is defined as the fragment that divides the roadway between opposing traffic lanes. Medians are required especially for the roads with four or more lanes. Median width is described as the distance between the edges of opposing traveled ways including the shoulders on left-hand side. An illustration of a typical median cross section is given on Figure 4.1 (a). The width of the median depends on the need of a median barrier. Medians should be as wide as possible since it adds operational advantages on the roadway. However, economic reasons limit the desirably wide median usages. But, in 1990s, some states noticed an increase in the number and severity of cross-median crashes even on the sections where the medians are quite wide. For this reason, FHWA conducted a study and found out that median barriers can significantly decreases the occurrence and severity of cross-median crashes. With this fact, a recommendation chart has been prepared for the usage of median barriers which is given in Figure 4.1 (b).



(a)



(b)

Figure 4.1. Typical median cross section (a) and barrier requirement chart based on ADT and median width (b) (AASHTO, 2011)

4.1.2. (G.B) Landscaping

Landscaping process should be done based on keeping the existing character of the existing environment by adaptation to the new road. It includes following operations; conserving the existing plantation, transplantation where possible, planting of new vegetation and rehabilitation of natural species. Its functions can be defined as; contribution to aesthetics, lowering the construction cost, creating usefulness, interest and beauty for pleasure (AASHTO, 2011). On the other hand, although it used to be thought in former that landscaping was just a concept of developing aesthetics and natural beauty adjacent to roadways, some studies showed that landscaping also helps in increasing road safety by providing traffic calming as well (Mok et. Al., 2005). Hence, it also gains importance to pay attention to landscaping design in order to contribute to improvement of safety of roads. However, according to Landscape and Aesthetics Design Manual (TxDOT, 2017), landscaping process should be in a way that vegetation should not affect sight-distance clearance, provide aesthetics, not obstruct any signs, have low height around intersection area, not be located near merging roads and not lead to occurrence of any unsafe situation for motorists.

4.2. Alignment & Cross-Section (A&C) Issues

Road safety issues, which should be considered during a RSA on major urban arterials, that are related with alignment and cross-section are given in *Table 4.3* and *Table 4.4*. Most of the issues have multiple controls. The background that will be used while conducting the safety audit for the case study will be explained in detail. Although, all the required items are given, some of them are not available for examination in this case study. Therefore, the background data will not be presented for these items.

Table 4.3. *Alignment & Cross – Section Issues in RSA (Sub Items A-E)*

(A&C.A) Classification	**1.** Check the appropriateness of the classification and design for the proposed project's design volume and traffic composition
	2. Is the design of the proposed project flexible enough to accommodate unforeseen increases in volume or changes in traffic characteristics
(A&C.B) Design Speed /Posted Speed	**1.** Check the appropriateness of the design speed for horizontal and vertical alignment, visibility, etc.
	2. Check the continuity of the design speed and the posted speed.
	3. Is the posted speed on each curve adequate?
	4. Is the traffic following the posted speed?
(A&C.C) Route Selection / Alignment	**1.** Are horizontal and vertical curves minimized?
	2. Do excessive grades affect heavy vehicle operations and service levels?
	3. Check for poor combinations of features
(A&C.D) Cross Sectional Elements	**1.** Determine if the proposed project has a suitable cross section for the ultimate requirements of the road including: - classification - design speed - level of service/peak service volumes
	2. Determine if adjustments in dimensions can be made for future expansion possibilities
(A&C.E) Drainage	**1.** Is the drainage channel appropriate for topography, maintenance and snow drifting?
	2. Is there possibility of surface flooding or overflow from surrounding or intersecting drains and water courses?
	3. Does the proposed roadway have sufficient drainage?

Table 4.4. *Alignment & Cross – Section Issues in RSA (Sub Items F-N)*

(A&C.F) Lane Width	**1.** Is the lane width sufficient for road design / classification?
(A&C.G) Cross Slopes /Superelevation	**1.** Do crown and cross slope designs provide sufficient storm water drainage and facilitate de-icing treatments?
	2. Do different rates of cross slope exist along adjacent traffic lanes?
(A&C.H) Pavement Widening	**1.** Is sufficient pavement width provided along curves where off-tracking characteristics of vehicles are expected?
(A&C.I) Alignment	**1.** Are there excessive curves that cause sliding in adverse weather conditions?
(A&C.J) Horizontal	**1.** Check that a transition curve is required between a tangent and a circular curve
	2. Is the superelevation with transition curves suitable in relation to effects of drainage
(A&C.K) Vertical	**1.** Are there excessive grades which could be unsafe in adverse weather conditions?
	2. Is a climbing lane provided where overtaking and passing maneuvers are limited due to terrain?
	3. Is a climbing lane provided in areas where the design gradient exceeds the critical length of the grade?
	4. Verify that escape lanes are provided where necessary on steep down grades. If not, are escape lanes feasible?
	5. Is there adequate provision of passing opportunities?
	6. Is there sufficient spacing between passing zones?
(A&C.L) Combined Vertical and Horizontal	**1.** Check the interaction of horizontal and vertical alignments in the road (ie., roller coaster alignments, sequencing of horizontal/vertical curves, etc.)
(A&C.M) Sight Distances	**1.** Check that there is decision sight distance provided for interchange and intersection signing throughout the project
(A&C.N) Readability by Drivers	**1.** Check for sections of roadway having potential for confusion -alignment problems -old pavement markings not properly removed -streetlight/tree lines don't follow road alignment

4.2.1. (A&C.A) Classification

Classification of roads is crucial for engineers, administrations and other shareholders so that they can clearly identify them among themselves. There are different types of classification of roads which can be given as design types, numbering and administrative purposes. However, for transportation purposes, functional classification has been developed which groups the roads according to their character of service they provide and given in *Table 4.5*. Functional systems are divided into two; rural areas and urban areas. In urban areas, the trips that occur between central business districts, major suburban areas and major inner-city communities are defined to be involved in urban principal arterial systems (AASHTO, 2011).

Table 4.5. *Functional classification*

Rural Areas	Urban Areas
Rural Principal Arterial System	Urban Principal Arterial System
Rural Minor Arterial System	Urban Minor Arterial System
Rural Collector System	Urban Collector Street System
Rural Local Road System	Urban Local Street System

4.2.2. (A&C.B) Design / Posted Speed

Appropriateness of the design speed for horizontal and vertical alignment (A&C.B.1)

In order to evaluate the horizontal curves' design appropriateness, maximum superelevation rate must be known. According to AASHTO (2011), the highest superelevation rate that is used generally on highways is 10%. However, this value must be defined considering four different conditions; climate conditions, terrain conditions, type of area and frequency of very-slow-moving vehicle. So, high rates are used where snow and ice are not present in winter times. Additionally, where congestion is possible, such as intracity roads, it is practical to apply a lower rate of superelevation, mostly 4 to 6 percent (AASHTO, 2011).

Superelevation Rate:

The recommended values for superelevation in order to apply regarding with different design speeds and radii for maximum superelevation rate of 4% are given in *Table 4.6* below that is gathered from AASHTO (2011).

Transition Curve Length:

For the application of superelevation on the road, an adequate transition design must be done. In order to have a safe transition from normal crown slope to defined superelevation rate, the lengths of superelevation runoff and tangent runout distances must be calculated. Superelevation transition is composed of these two distances. Superelevation runoff distance is the length required to change outside lane cross slope from flat position to applied superelevation or exact opposite while tangent runout is the length required to change the outside lane cross slope from normal crown slope to flat position or the contrary (AASHTO, 2011). Transition design is divided into two categories; tangent to curve transition and spiral curve transition. Where a spiral curve is used before and after main circular horizontal curve, it is called as spiral curve transition. In these different cases, also the design of transition differs. However, in this project, there is no spiral curve but only circular curves. Minimum length of superelevation runoff used to be calculated on the basis of equalizing the runoff distance with the distance traveled in 2.0 seconds. However, later, it came out that this method does not give adequate results. Then, runoff distance is suggested to be calculated using maximum relative gradient criterion. According to this criterion, between the longitudinal grades of the axis of rotation and the edge of the pavement, the length of superelevation runoff should be calculated taking the maximum allowable difference into consideration. "Current practice is to limit the grade difference, referred to as the relative gradient, to a maximum value of 0.50 percent or a longitudinal slope of 1:200 at 80 km/h [50 mph]" (AASHTO, 2011). However, in the application of relative gradient criterion, runoff lengths would be doubled in case of four lanes highway and tripled in case of six lanes highway. Yet, these values of runoff lengths mostly are not practicable. Therefore, an adjustment factor is used in order to downgrade the runoff lengths so that it would be a number that is logical to apply on site (AASHTO, 2011). The equation that is used for calculation of runoff length and the variables that are included in this equation are;

$$L_r = \frac{(wn_1)e_d}{\triangle}(b_w) \qquad \textit{Eq. 4.1 (AASHTO, 2011)}$$

Where;

L_r = minimum length of superelevation runoff, m

w = width of one traffic lane, m (typically 3.6 m)

n_1 = number of lanes rotated

e_d = design superelevation rate, percent

b_w = adjustment factor for number of lanes rotated

$\triangle$= maximum relative gradient, percent

Adjustment factor for number of lanes rotated can be calculated with the formula;

$$b_w = [\![1 + 0.5(n_1 - 1)]\!] \,/\, n_1 \qquad \textit{Eq. 4.2} \quad \textit{(AASHTO, 2011)}$$

Maximum relative gradient is to be defined using *Table 4.7*. After the evaluation of superelevation runoff lengths, tangent runout lengths will be examined. The length of tangent runout is calculated to be at the same rate regarding with superelevation runoff length so that the relative gradients will be same for both. The aim for this proportion is to have a smooth transition using the same rate of change for both surface runoff and tangent runout lengths (AASHTO, 2011). Since they are directly proportional, the equation that is used in order to calculate tangent runout length is;

$$L_t = \frac{e_{NC}}{e_d} L_r \qquad \text{Eq. 4.3 (AASHTO, 2011)}$$

where:

L_t = minimum length of tangent runout, m

e_{NC} = normal cross slope rate, percent

e_d = design superelevation rate, percent

L_r = minimum length of superelevation runoff, m

<u>Horizontal Sight Distance:</u>

The drivers must be able to observe ahead easily while driving for safe and efficient operation. While on railroads, a trained operator and a signaling system is enough, on roads the operators' training and experience are varied. Hence, a decent sight distance must be applied to the roads by the designers so that the drivers on the road would be able to avoid hitting any object which is called stopping sight distance. On two-lane highways, there is also a sight distance that must be provided for a safe passing of the vehicle without hitting any vehicle coming from opposite direction since it has to use opposite traffic lane during this passing operation which is called passing sight distance. However, for multi-lane roads, this condition is eliminated as the passing vehicles do not interfere with opposing traffic (AASHTO, 2011). This criterion is valid for both horizontal and vertical alignment.

Table 4.6. *Superelevation rates for different design speeds and radii for e_{max} = 4% (AASHTO, 2011)*

R_{min} (m) e (%)	**Design Speed (km/h)**								
	20	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**
NC	163	371	679	951	1310	1740	2170	2640	3250
RC	102	237	441	632	877	1180	1490	1830	2260
2.2	75	187	363	534	749	1020	1290	1590	1980
2.4	51	132	273	435	626	865	1110	1390	1730
2.6	38	99	209	345	508	720	944	1200	1510
2.8	30	79	167	283	422	605	802	1030	1320
3.0	24	64	137	236	356	516	690	893	1150
3.2	20	54	114	199	303	443	597	779	1010
3.4	17	45	96	170	260	382	518	680	879
3.6	14	38	81	144	222	329	448	591	767
3.8	12	31	67	121	187	278	381	505	658
4.0	8	22	47	86	135	203	280	375	492

Table 4.7. *Design speed based (a) Maximum Relative Gradients, b) Stopping Sight Distance and Design Controls for (c) Crest Vertical Curves and (d) Sag Vertical Curves (St (AASHTO, 2011)*

Design Speed (km/h)	**(a) Maximum Relative Gradient (%)**	**(b) Stopping Sight Distance (m)**	**Rate of Vertical Curvature K**	
			(c) Crest type	**(d) Sag type**
20	0.80	20	1	3
30	0.75	35	2	6
40	0.70	50	4	9
50	0.65	65	7	13
60	0.60	85	11	18
70	0.55	105	17	23
80	0.50	130	26	30
90	0.47	160	39	38
100	0.44	185	52	45
110	0.41	220	74	55
120	0.38	250	95	63
130	0.35	285	124	73

About the evaluation of horizontal alignment, another subject is the sight distance across the inside of the curves. In case there are some obstructions (I.e. buildings, structures, median barrier, cut slope) inside the curve and they are not in a condition to be removed in order to increase the sight distance, the required changes must be done by modifying the alignment or cross section. Hence, sight distance phenomena requires the investigation of each individual horizontal curve for this manner (AASHTO, 2011). In the examination of sight distance for horizontal curves, the sightline is a chord of the curve and the sight distance is the distance taken from centerline to centerline of the inner lane. Although AASHTO (2011) has prepared a diagram that can be used for quick check of horizontal sight line distance control, this diagram is based on stopping sight distance where the vertical alignment is flat. But the roadway has some steep vertical grades on horizontal curves and it must be checked one by one since the grade affects stopping sight distance. Stopping sight distance is composed two items; brake reaction time and braking distance. Brake reaction time is the duration that is elapsed between the instant when the driver notices the obstacle standing ahead of him and the instant the driver actually applies braking. Although many studies and laboratory tests have been made, the results differed a lot. In these tests, it was found out that the average brake reaction time was around 0.65 seconds. However, some drivers' reaction time increased up to 3.5 seconds in spite of being under simple conditions in some tests. Depending on these tests, a duration of 2.5 seconds has been recognized as a suitable brake reaction time which exceeds 90th percent of drivers' brake reaction time by AASHTO in Green Book 2011. Other component of stopping sight distance is braking distance which is the length covered by the driver after applying braking until fully stopping. Studies show that most of the drivers can decelerate at a rate greater than 4.5 m/s^2 while 90 percent of test drivers decelerate at a rate greater than 3.4 m/s^2. Therefore, a deceleration rate of 3.4 m/s^2 has been chosen as an appropriate value for calculation of breaking distance. As a result, stopping sight distance is the summation of these two items in length. The equation that is used to calculate stopping sight distance on grades is given;

$$SSD = 0.278Vt + \frac{V^2}{254\left[\left(\frac{a}{9.81}\right) \pm G\right]}$$ Eq. 4.4 (AASHTO, 2011)

Where;

SSD = Stopping sight distance in meters

V = Design Speed in km/h

t = Brake reaction time in seconds (used as 2.5 seconds)

a = deceleration rate in m/s^2

G = grade in rise/run (m/m)

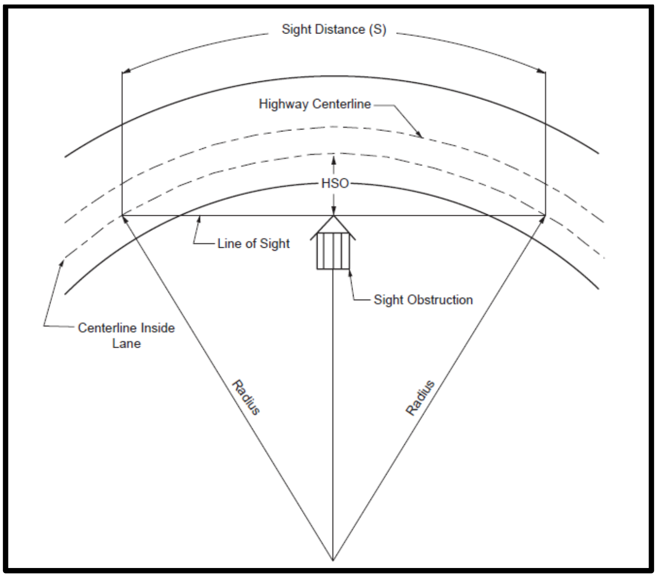


Figure 4.2. Illustration of Horizontal sight Distance (AASHTO, 2011)

For the eye height of 1.08 m and object height of 0.60 m, stopping sight distance is evaluated for horizontal sight line distance on horizontal curves. Using the actual SSD on grades, the horizontal sight line distance is determined and then sight distance can be examined accordingly. An illustration of sight distance on a horizontal curve is presented in Figure 4.2; the equation that is used for calculation of HSO is;

$$HSO = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right]$$ Eq. 4.5 (AASHTO, 2011)

Vertical Curves:

Curves are placed for smooth transition of gradient changes in the vertical alignment. The vertical curves should satisfy the requirements for clear visibility, enhanced vehicle control, satisfactory sight and decent drainage conditions. These curves are divided into two according to their layouts; sag vertical curves and crest vertical curves which can be observed in Figure 4.3. Design check for this different kind of vertical curves also differ. For crest curves, only stopping sight distance and passing sight distance are subject to design control while for sag curves; headlight sight distances, passenger comfort, drainage control and general appearance criteria are. However, all the vertical curves should provide sufficient sight distances. It is recommended in AASHTO (2011) that at least stopping sight distance criteria should be met in design of vertical curves.

$$K = \frac{L}{A}$$ Eq. 4.6 (AASHTO, 2011)

Where;

K = the length of curve per percent algebraic difference in intersecting grades

L = length of the vertical curve, m

A = Algebraic difference in grades, percent

Usage of parameter "K" is a convenient way of defining vertical curves in terms of curvature without using incoming and outgoing parameters and lengths. This parameter covers all the parameters and states them in one value only, therefore, AASHTO has prepared design check tables for vertical curves based on this parameter. Yet, as mentioned before, design controls are different for sag and crest curves. But the main ideas for sight distances are similar with general manners. Sight distances are separated into two on vertical curve design control; stopping sight distance and passing sight distance.

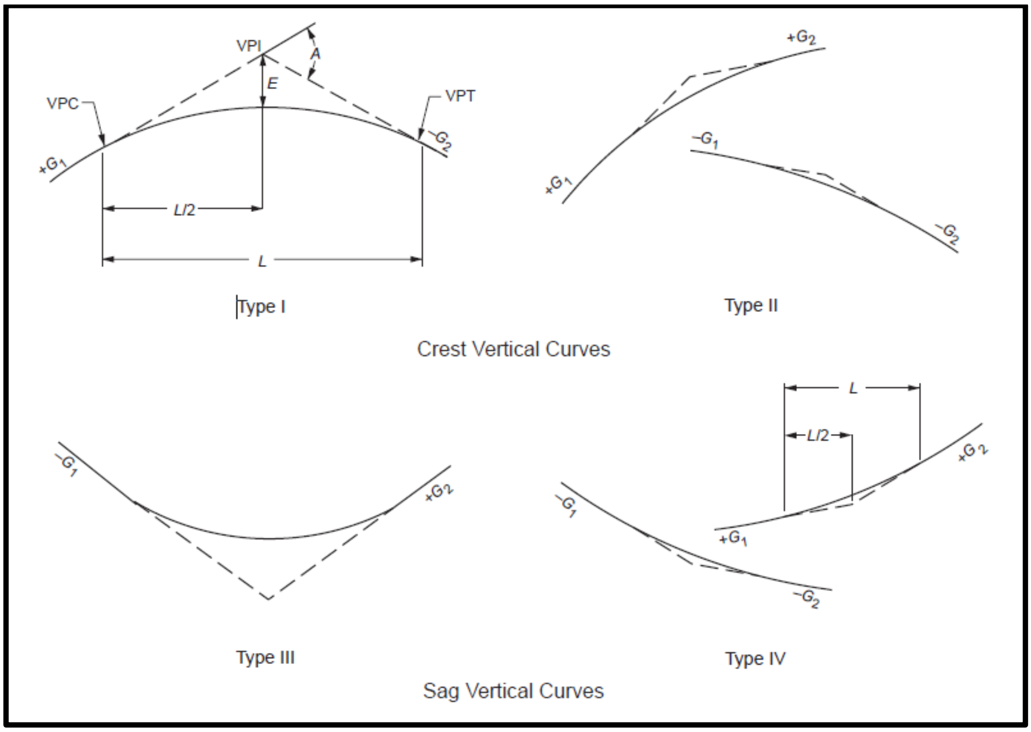
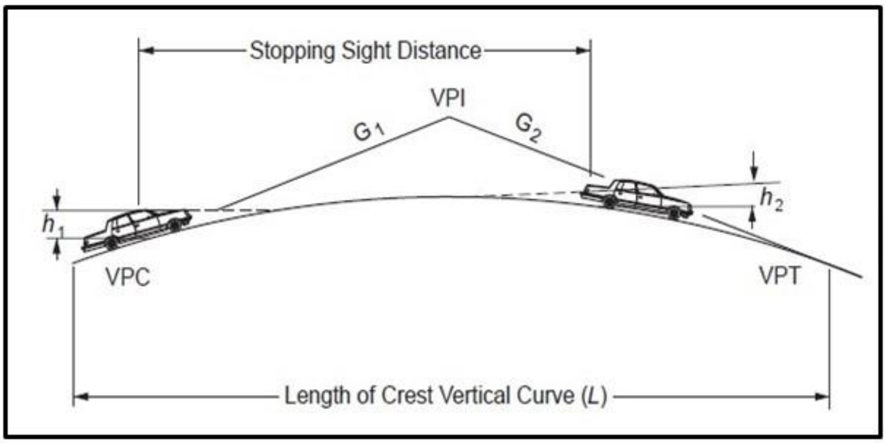


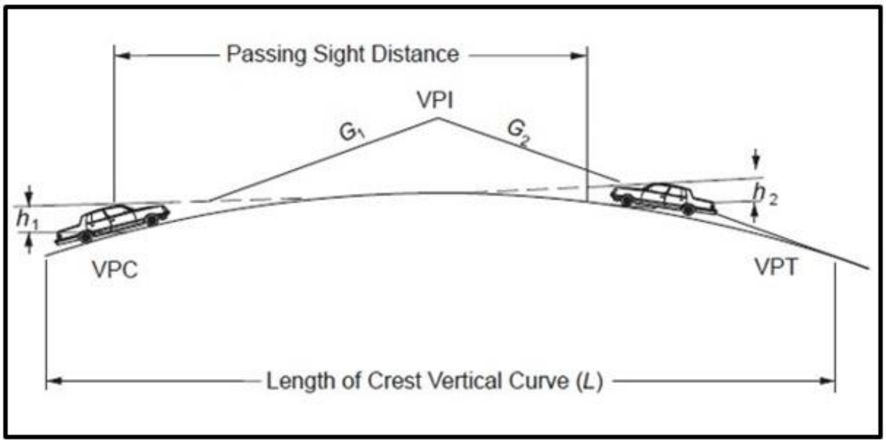
Figure 4.3. Types of vertical curves (AASHTO, 2011)

For stopping sight distance, height of the driver's eye (h1) is considered to be 1.08 m while the height of the object (h2) is considered to be 0.60 m which are described visually in Figure 4.4. Accordingly, the equation that is used for calculation of length of the crest curve with respect to stopping sight distance is given in detail in ASSHTO (2011). But, for easiness of design control, K parameter is used by AASHTO (2011) with the aid of *Table 4.7.* Design speed based (a) Maximum Relative Gradients, b) Stopping Sight Distance and Design Controls for (c) Crest Vertical Curves and (d) Sag Vertical Curves (St (AASHTO, 2011).

In the evaluation of sag vertical curves, the most critical criteria that is used for design control is the headlight sight distance. Although the roadway in this project is lightened with lamps, in case of inadequacy or breakdown of lamps, headlight sight distance still has to be considered. Many agencies use directly headlight distance in determining the length of sag vertical curves. Though, there are also other criteria that have to be checked additionally. Headlight sight distance is calculated by the fact that the headlight of the vehicles has a height of 0.60 m and spreads upwards with a 1-degree divergence. Yet, most agencies ignore this condition and take it as if it spreads



(a)



(b)

Figure 4.4. SSD (a) and PSD (b) Parameters Used in Design Control of Crest Curves (AASHTO, 2011)

horizontally. It can be summarized as the requirement of headlight beams to reach to the point that is same with the stopping sight distance so that the driver can see the object while in operation. Although the equation that is used in calculation of headlight sight distance is given in AASHTO (2011), as in crest curve for stopping sight distance, a simple table is presented for convenient use of design control which is presented in *Table 4.7.* Design speed based (a) Maximum Relative Gradients, b)

Stopping Sight Distance and Design Controls for (c) Crest Vertical Curves and (d) Sag Vertical Curves (St (AASHTO, 2011). However, this table also includes design controls for other criteria. Apart from headlight sight distance, passenger comfort should be taken into consideration since, in contrast with crest curves, the centrifugal force and the gravitational force act towards the same direction in sag curves which increases the total force acting on the vehicle by combining them. The vehicle can have a comfortable travel when the centripetal acceleration is limited to 0.3 m/s2. Yet, this criterion is not taken into consideration because of the fact that mostly it does not govern the calculation of the length of the vertical curves (AASHTO, 2011). Another design criterion is about drainage control. This criterion is valid for the Type III sag vertical curves given in the Figure 4.3. For a decent drainage, minimum grade of 0.30 percent should be provided within 15 m of the level point inside the sag vertical curve which is equal to 51 m in terms of parameter “K”. When the K value is less than this value, drainage control does not govern. On the other hand, the check for passing sight distance is not a subject to this project because of the fact that it is not needed to calculate the passing sight distance if the roadway has more than two lanes in each direction (AASHTO, 2011).

Vertical Alignment:

From the point of view of vertical grades, the terrain can be defined to have a rolling topography. This condition also leads to high values of grades in the vertical alignment. Although not much known about the maximum allowable grades, some studies have been done up to now by reasonable guideline authorities. According to these studies, it can be summarized that for a design speed of 110 km/h, a maximum grade of 5 percent is allowed while for a design speed of 50 km/h the maximum grade allowed is between 7 and 12 percent. For the design speed from 60 to 100 km/h, allowable grade values fall between these mentioned values. Though, AASHTO (2011) has prepared *Table 4.8* for urban arterials that is showing the recommended maximum grades.

Check the continuity of the design speed and the posted speed (A&C.B.2)

Although the design speed is 70 km/h, local authority set the posted speed as 90 km/h. Referencing to highway traffic law, which states that the vehicles travelling at a speed 10% higher than the posted speed, posted speed has been arranged to be 82 km/h. Being 90 km/h, the posted speed is almost 29 % higher than design speed.

Table 4.8. *Maximum Grade for Urban Arterials (AASHTO, 2011)*

Type of Terrain	**Maximum Grade (%) for Specified Design Speed (km/h)**					
	50	**60**	**70**	**80**	**90**	**100**
Level	8	7	6	6	5	5
Rolling	9	8	7	7	6	6
Mountainous	11	10	9	9	8	8

Is the posted speed on each curve adequate? (A&C.B.3)

As described in the previous item, posted speed refers to 90 km/h. Both horizontal and vertical curves will be examined according to this posted speed by using the same definitions and criteria that have been presented and used while checking for design speed which is 70 km/h.

Is the traffic following the posted speed? (A&C.B.4)

Posted speed limits usually refers to the 85th percentile speed of traffic which is retrieved by evaluating the speeds of a sample of vehicles. 85th percentile speed is mostly the speed that falls within a range of 15 km/h of the speed that is travelled by most of the drivers.

4.2.3. (A&C.D) Cross Sectional Elements

Determine if the proposed project has a suitable cross section for the ultimate requirements of the road including: Classification, design speed and level of service/peak service volumes (A&C.D.1)

According to AASHTO (2011), design speed for urban arterials range from 50 to 100 km/h and during design of these roads, traffic volume of next 20 years should be taken into consideration. Although level of service should be C or D for urban arterials, in order to have a better roadway composition, level of service C should be sought. Width of the roadway must be sufficient for accommodation of medians, curbs and flowing traffic but there is not a specific value defined in AASHTO (2011). Number of lanes depend on the capacity required. For urban arterials, number of lanes including opposite direction vary from 4 to 8 lanes. About the sidewalks, the distance from the edge of travel to adjacent structure is called border and including sidewalk width, this border must have at least 2.4 m of width (AASHTO, 2011).

Determine if adjustments in dimensions can be made for future expansion possibilities (A&C.D.2)

Expansion possibilities differ from segment to segment on the roadway. While there are available rooms where the roadway can be expanded, in some zones this possibility exists only by some demolition applications. In order to be able to define these segments, the roadway has been divided into zones which are presented in Figure 4.5.

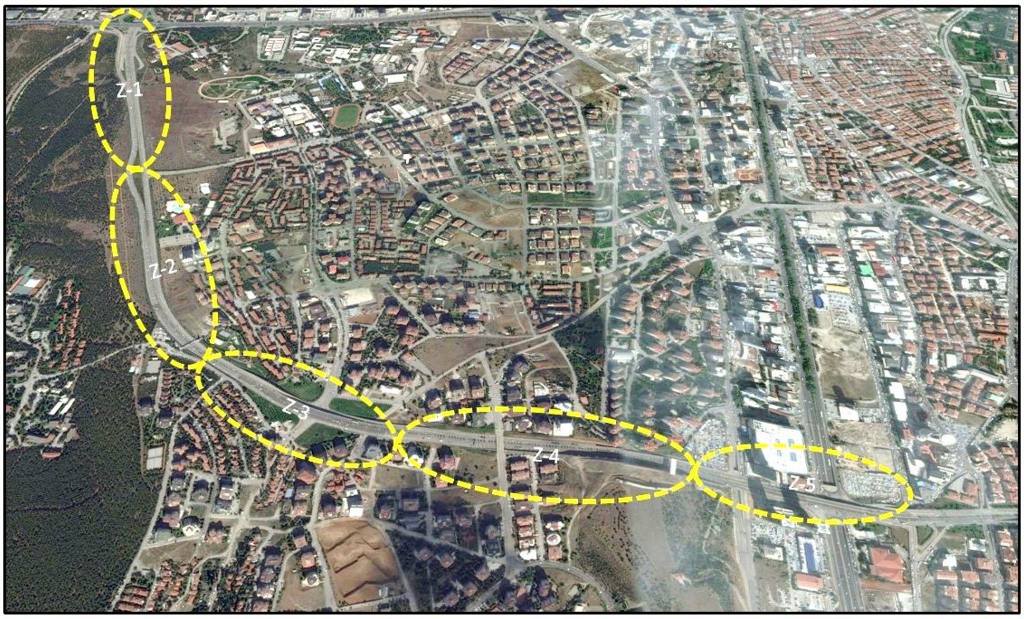


Figure 4.5. Defined Zones for Investigation of Expansion Possibility

4.2.4. (A&C.F) Lane Width

Is the lane width sufficient for road design / classification? (A&C.F.1)

Lane widths should be between 3.0 and 3.6 meters while on high-speed, free-flowing main arterials 3.6 meters of lane width is desirable (AASHTO, 2011). In Turkey, except for expressways with tolls, most of the main roads are designed with a lane width of 3.5 meters. While two-lane rural roads can be designed with a lane width of 3.0 meters, expressways with tolls have a lane width of 3.6 meters.

4.2.5. (A&C.G) Cross Slopes / Superelevation

Do crown and cross slope designs provide sufficient storm water drainage and facilitate de-icing treatments? (A&C.(G.1)

“Each roadway of a divided arterial may be sloped to drain to both edges, or each roadway may be sloped to drain to its outer edge, depending on climatic conditions and the width of median. Roadways on divided arterials should have a normal cross slope of 1.5 to 2 percent” (AASHTO, 2011).

4.2.6. (A&C.H) Pavement Widening

Is sufficient pavement width provided along curves where off-tracking characteristics of vehicles are expected? (A&C.H.1)

Offtracking is the event that rear wheels of a vehicle do not follow the front wheels when the vehicle, more intense on large vehicles, goes through a horizontal curve or makes a turn. This depends on the speed and the friction between wheels and pavement which changes according to whether there is superelevation or not. The more the speed is, the more possibility for offtracking occurs. With or without superelevation, high or low speed, the amount of offtracking and widening required as the consequence of this offtracking is related with the design vehicle and the radius of curvature. Design vehicle is chosen considering the types and frequency of vehicles that will use the roadway in question. As the size of the design vehicle increases and the rate of curvature decreases, the amount of widening needed increases. The items of that are used in calculation of roadway widening are; the track width of the design vehicle, U; the lateral clearance per vehicle, C; the width of front overhang occupying inner lanes, F_A; the width of rear overhang, F_B; the width allowance for driving on curves, Z (AASHTO, 2011). The track width for a vehicle, also known as swept path, is calculated using the equation;

$$U = u + R - \sqrt{R^2 - \sum L_i^2}$$ Eq. 4.7 (AASHTO, 2011)

Where;

U = track width on curve in meters

u = track width on tangent in meters

R = radius of curve or turn in meters

Li = wheelbase of design vehicle between consecutive axles in meters

The width of front overhang is the radial distance between the outer edge of the path of front tire and the outer edge of the front of the vehicle body (AASHTO, 2011). It is calculated by the equation;

$$F_A = \sqrt{R^2 + A(2L + A)} - R$$ Eq. 4.8 (AASHTO, 2011)

Where;

FA = width of front overhang in meters

R = radius of curve in meters

A = front overhang of inner-lane vehicle in meters

L = wheelbase of single unit tractor in meters

FB is the radial distance between the outer edge of the tire path of the inner rear wheel and the inside edge of the vehicle body. For passenger cars, the width of the vehicle is 0.3 m greater than the width of out-to-out width of the rear wheels which makes FB equal to 0.15 m while for trucks, since the width of the vehicle body is same with the width of out-to-out rear wheels, FB is considered as zero. Z is the distance taken into consideration in order not to ignore the maneuver variation while driving on the curve. This distance is an empirical value which is calculated using the equation (AASHTO, 2011);

$$Z = 0.1(\frac{V}{\sqrt{R}})$$ Eq. 4.9 (AASHTO, 2011)

Where;

Z = extra width allowance in meters

V = design speed of the roadway

R = radius of curve

Hence, amount of widening becomes the distance between the width of traveled way on curve and tangent section (AASHTO, 2011). The width of the traveled way on curve section is calculated by;

$$W_c = N(U + C) + (N - 1)F_A + Z$$ Eq. 4.10 (AASHTO, 2011)

Where;

Wc = width of traveled way on curve in meters

N = number of lanes

U = track width of design vehicle in meters

C = lateral clearance in meters

FA = width of front overhang of inner-lane vehicle in meters

Z = extra width allowance in meters in meters

The items related with widening that have been described above are illustrated in Figure 4.6.

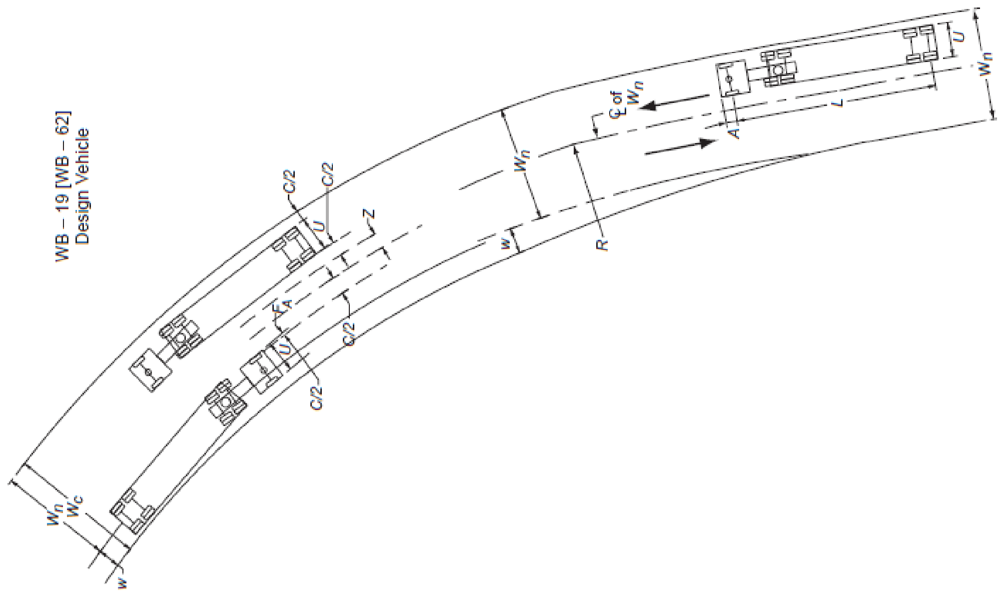


Figure 4.6. Widening components (AASHTO, 2011)

Design vehicle that was chosen by the designer for this roadway is not known. However, observing the traffic, it can be said that trucks with trailers can be utilized. According to AASHTO (2011), “interstate semitrailer” design vehicle with the code of WB-19 is appropriate for assumption of design vehicle for the project. The dimensions for this vehicle are examined on *Table 4.9*.

Table 4.9. *Design Vehicle Dimensions (AASHTO, 2011)*

Design Vehicle Type	Symbol	Overall Height	Overall Width	Overall Length	Overhang Front	Overhang Rear
Passenger Car	P	1.30	2.13	5.79	0.91	1.52
Single-Unit Truck	SU-9	3.35–4.11	2.44	9.14	1.22	1.83
Single-Unit Truck (three-axle)	SU-12	3.35–4.11	2.44	12.04	1.22	3.20
Buses						
Intercity Bus (Motor Coaches)	BUS-12	3.66	2.59	12.36	1.93	2.73[a]
	BUS-14	3.66	2.59	13.86	1.89	2.73[b]
City Transit Bus	CITY-BUS	3.20	2.59	12.19	2.13	2.44
Conventional School Bus (65 pass.)	S-BUS 11	3.20	2.44	10.91	0.79	3.66
Large School Bus (84 pass.)	S-BUS 12	3.20	2.44	12.19	2.13	3.96
Articulated Bus	A-BUS	3.35	2.59	18.29	2.62	3.05
Combination Trucks						
Intermediate Semitrailer	WB-12	4.11	2.44	13.87	0.91	1.37[a]
Interstate Semitrailer	WB-19*	4.11	2.59	21.03	1.22	1.37[a]
Interstate Semitrailer	WB-20**	4.11	2.59	22.40	1.22	1.37[a]
"Double-Bottom" Semitrailer/Trailer	WB-20D	4.11	2.59	22.04	0.71	0.91
Rocky Mountain Double-Semitrailer/Trailer	WB-28D	4.11	2.59	29.67	0.71	0.91
Triple-Semitrailer/Trailers	WB-30T	4.11	2.59	31.94	0.71	0.91
Turnpike Double-Semitrailer/Trailer	WB-33D*	4.11	2.59	34.75	0.71	1.37[a]
Recreational Vehicles						
Motor Home	MH	3.66	2.44	9.14	1.22	1.83
Car and Camper Trailer	P/T	3.05	2.44	14.84	0.91	3.66
Car and Boat Trailer	P/B	–	2.44	12.80	0.91	2.44
Motor Home and Boat Trailer	MH/B	3.66	2.44	16.15	1.22	2.44

4.2.7. (A&C.J) Horizontal

Check that a transition curve is required between a tangent and a circular curve. (A&C.J.2)

Among agencies, there is general disagreement about the use of transition curves according to some reviews. Yet, AASHTO (2011), recommends that there should be a maximum radius of curvature above which the curve will not benefit from the advantages of usage of transition curves from the point of view of safety and operation. Some agencies including AASHTO, studied to define limiting radii for different design speeds so that transition curves will not be used needlessly. These limitations have been based on lateral acceleration rates which have been found to vary between 0.4 and 1.3 m/s^2 among the results. Considering also crash potentials, the upper limit of these results, which is 1.3 m/s^2, has been adopted by AASHTO (2011) and the maximum radius for use of transition curves have been presented depending on the design speed in *Table 4.10*.

Table 4.10. *Maximum radius for use of transition curves (AASHTO, 2011)*

Metric	
Design speed (km/h)	**Maximum radius (m)**
20	24
30	54
40	95
50	148
60	213
70	290
80	379
90	480
100	592
110	716
120	852
130	1000

4.2.8. (A&C.K) Vertical

Are there excessive grades which could be unsafe in adverse weather conditions? (A&C.K.1)

For urban arterials, grades can have important role on its operational performance. While steep grades can affect the safety conditions especially for heavy vehicles, flat grades may cause drainage problems. Minimum grade for flat sections is recommended not to be less than 0.5 % since especially curbed sections will not be able to provide sufficient longitudinal flow for drainage (AASHTO, 2011).

4.2.9. (A&C.M) Sight Distances

Check that there is decision sight distance provided for interchange and intersection signing throughout the project (A&C.M.1)

Although stopping sight distance is sufficient for the emergency situations where the driver has to come to complete stop, there is also another sight distance that must be considered. Decision sight distance is the length that will be passed through in the duration in which the driver has to make complex decisions where there exists an intersection, a lane change or a section change since these points might make the drivers obliged to make urgent maneuvers. These maneuvers are the ones which include only having changes in the path or speed but not stopping because in some situations, stopping is more risky (AASHTO, 2011). Hence, using empirical data, AASHTO prepared a decision sight distance table (*Table 4.11*) relevant to different design speeds. According to that table, on urban roads, required decision sight distance is 275 meters for a design speed of 70 km/h.

Table 4.11. *Decision Sight Distance Control Values AASHTO, 2011)*

Metric						U.S. Customary					
Design Speed (km/h)	Decision Sight Distance (m)					Design Speed (mph)	Decision Sight Distance (ft)				
	Avoidance Maneuver						Avoidance Maneuver				
	A	B	C	D	E		A	B	C	D	E
50	70	155	145	170	195	30	220	490	450	535	620
60	95	195	170	205	235	35	275	590	525	625	720
70	115	325	200	235	275	40	330	690	600	715	825
80	140	280	230	270	315	45	395	800	675	800	930
90	170	325	270	315	360	50	465	910	750	890	1030
100	200	370	315	355	400	55	535	1030	865	980	1135
110	235	420	330	380	430	60	610	1150	990	1125	1280
120	265	470	360	415	470	65	695	1275	1050	1220	1365
130	305	525	390	450	510	70	780	1410	1105	1275	1445
						75	875	1545	1180	1365	1545
						80	970	1685	1260	1455	1650

Avoidance Maneuver A: Stop on rural road—*t* = 3.0 s

Avoidance Maneuver B: Stop on urban road—*t* = 9.1 s

Avoidance Maneuver C: Speed/path/direction change on rural road—*t* varies between 10.2 and 11.2 s

Avoidance Maneuver D: Speed/path/direction change on suburban road—*t* varies between 12.1 and 12.9 s

Avoidance Maneuver E: Speed/path/direction change on urban road—*t* varies between 14.0 and 14.5 s

4.3. Intersection (X) Issues

Issues related with intersections are crucial for RSA and the ones that are seen to be taken into consideration for urban major arterials are given in *Table 4.12*. Although the items given in this table should be used during study of RSAs, some of these items were not able to be controlled in case study. Hence, details and background data will be given only for the items that will be used.

Table 4.12. *Intersection Issues in RSA*

Intersections	
(X.A) Quantity	**1.**Is the number of intersections appropriate given the surrounding network?
(X.B) Type	**1.**Are types of intersections selected appropriate for traffic and safety aspects of the project?
	2.Can intersection designs accommodate all design vehicle classifications?
(X.C) Location / Spacing	**1.**Is there sufficient spacing between intersections?
	2.Does horizontal/vertical alignment affect the location/spacing of the intersections?
	3.Junctions and access adequate for all permitted vehicle movements?
(X.D) Visibility / Conspicuity	**1.**Does the horizontal and vertical alignment provide adequate visibility of the intersection?
	2.Are sight lines to the intersection obstructed?
(X.E) Layout	**1.**Are the lane widths adequate for all vehicle classes?
	2.Are there any upstream and downstream features which may affect safety? (I.e., "visual clutter", angle parking, high volume driveways)
	3.Are separate through lanes needed but not provided?
(X.F) Maneuvers	**1.**Are vehicle maneuvers obvious to all users?
	2.Identify any potential conflicts in movements.
(X.G) Auxiliary / Turning Lanes	**1.**Are they of appropriate length?
	2.Is there advance warning of approaching auxiliary lanes?
	3.Is sight distance for entering/leaving vehicles adequate?
	4.Are tapers installed where needed? Are they correctly aligned?
(X.H) Sight Distances	**1.**Are all sight distances adequate for all movements and road users?
	2.Are sight lines obstructed by signs, bridge abutments, buildings, landscaping, etc.?
	3.Could sight lines be temporarily obstructed by parked vehicles, snow storage, etc.?
	4.Do grades at intersecting roadways allow desirable sight distance?
(X.I) Markings	**1.**Are pavement markings clearly visible in day and night time conditions?
	2.Check retroreflectivity of markings.
(X.J) Signs	**1.**Check visibility and readability of signs to approaching users.
	2.Check for any missing/redundant/broken signs.
	3.Are stop/yield signs used where appropriate?
(X.K) Signals	**1.**Have high intensity signals/target boards/shields been provided where sunset and sunrise may be a problem?
	2.Check location and number of signals. Are signals visible?
	3.Are primary and secondary signal heads properly positioned?
(X.L) Signal Phasing	**1.**Are minimal green and clearance phases provided?
	2.Is the signal phasing plan consistent with adjacent intersections?
(X.M) Warnings	**1.**Is adequate warning provided for signals not visible from an appropriate sight distance? (I.e., signs, flashing light, etc.)
	2.Are lateral rumble strips required and properly positioned?
	3.Are pavement markings appropriate for the intersection?

4.3.1. (X.C) Location / Spacing

Junctions and access adequate for all permitted vehicle movements? (X.C.3)

It was already defined that although it is not known, design vehicle is assumed to be WB-19 interstate semi-trailer. AASHTO (2011) claims that the width of turning roadways are defined based on the volume of turning traffic and types of turning vehicles. The corner radii should be given according to the minimum radii the vehicles can make their turns (AASHTO, 2011). Considering that the design vehicle is assumed as WB-19, minimum turning path related to this design vehicle is given in Figure 4.7 according to AASHTO (2011).

As can be seen in Figure 4.7, minimum design turning radius is 13.66 meters. All the corner radii in the project will be examined accordingly in order to see whether they are appropriate or not. However, turning radius is not the only criteria on designing the turning roadways at intersections. Also, the swept width is another subject since vehicles, especially long ones, are not able to make their front and rear wheels follow the same path. Hence, AASHTO (2011) presents *Table 4.13* that summarizes the required pavement width at intersections depending on the radius of the inner edge and design vehicle.

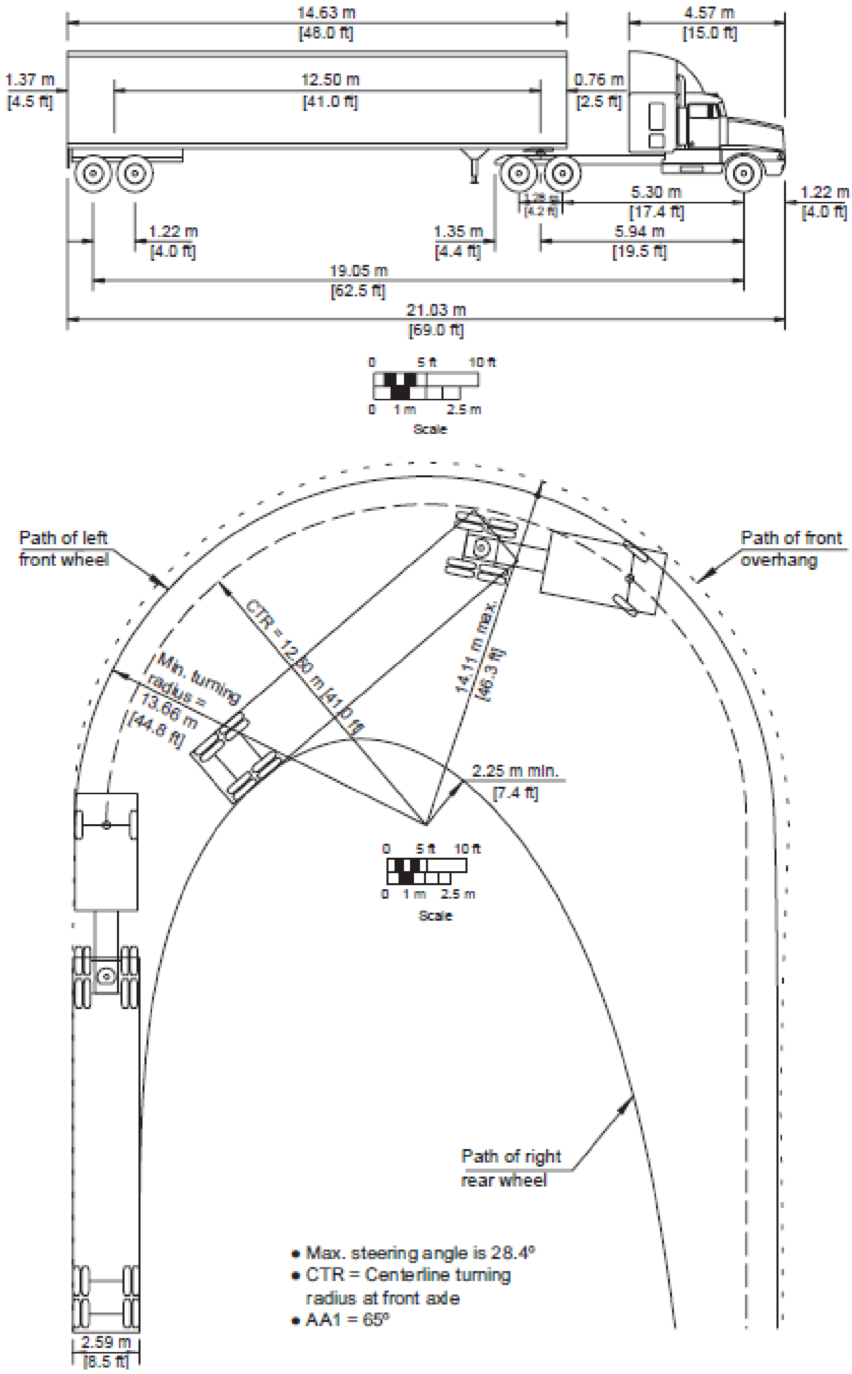


Figure 4.7. Minimum Turning radius for WB-19 Design Vehicle (AASHTO, 2011)

Table 4.13. *Required Pavement Width at Intersections Regarding with Design Vehicles (AASHTO, 2011)*

Metric																				
Radius on Inner Edge of Pavement, *R* (m)	Case I, One-Lane Operation, No Provision for Passing a Stalled Vehicle																			
	P	SU-9	SU-12	BUS-12	BUS-14	CITY-BUS	S-BUS-11	S-BUS-12	A-BUS-11	WB-12	WB-19	WB-20	WB-20D	WB-28D	WB-30T	WB-33D	MH	P/T	P/B	MH/B
15	4.0	5.5	6.3	6.6	7.2	6.5	5.7	5.5	6.7	7.0	13.5	—	8.8	—	11.6	—	5.5	5.7	5.4	6.5
25	3.9	5.0	5.4	5.7	5.9	5.6	5.1	5.0	5.7	5.8	8.5	9.5	6.8	9.6	7.9	12.0	5.0	5.1	4.9	5.5
30	3.8	4.9	5.2	5.4	5.7	5.4	5.0	4.9	5.5	5.5	7.8	8.5	6.3	8.6	7.3	10.3	4.9	5.0	4.8	5.3
50	3.7	4.6	4.8	5.0	5.2	5.0	4.7	4.6	5.0	5.0	6.3	6.7	5.5	6.8	6.1	7.7	4.6	4.7	4.6	4.9
75	3.7	4.5	4.6	4.8	4.9	4.8	4.5	4.5	4.8	4.7	5.7	5.9	5.1	6.0	5.5	6.6	4.5	4.5	4.5	4.7
100	3.7	4.4	4.5	4.7	4.8	4.7	4.5	4.4	4.7	4.6	5.3	5.5	5.0	5.6	5.2	6.0	4.4	4.5	4.4	4.5
125	3.7	4.4	4.5	4.6	4.7	4.6	4.4	4.4	4.6	4.5	5.2	5.3	4.8	5.3	5.0	5.7	4.4	4.4	4.4	4.5
150	3.7	4.4	4.4	4.6	4.6	4.6	4.4	4.4	4.6	4.5	5.0	5.2	4.8	5.2	4.9	5.5	4.4	4.4	4.4	4.4
Tangent	3.6	4.2	4.2	4.4	4.4	4.4	4.2	4.2	4.4	4.2	4.4	4.4	4.4	4.4	4.4	4.4	4.2	4.2	4.2	4.2

4.3.2. (X.D) Visibility / Conspicuity

Do the horizontal and vertical alignment provide adequate visibility of the intersection? (X.D.1)

Intersections are the points where both vehicles, pedestrians and bicycles tend to continue their movements in accordance with each other. Hence, this situation makes intersections gain importance since they require additional attention. Movement of all these traffic users at these points naturally include conflicts. These conflicts lead to need of enhanced solutions in order to reduce the negative results of these conflicts. Because of this reason, horizontal and vertical alignments at intersections should be designed as straight and flat as possible (AASHTO, 2011).

Horizontal alignments approaching to the intersection must be designed in a way so that all the users shall be able to notice all the components of the intersection. These components include vehicles and pedestrians maneuvering, traffic signs and lights as well. AASHTO (2011) recommends to place the intersections at right angles as much as possible in order to make all the components visible. Likewise, vertical geometry also is preferred to be as flat as practical. However, grades that are less than 3 percent are permissible while grades that are higher than 6 percent are strongly undesirable. Additionally, composition of horizontal and vertical geometry must be taken into consideration. For instance, sharp horizontal curves following a crest vertical curve is not recommended anywhere especially at intersection locations (AASHTO, 2011).

Are sight lines to the intersection obstructed? (X.D.2)

All the drivers using the roadway must have a clear view of all the intersection points apart from required stopping sight distances and decision sight distances. The users approaching an intersection must be able to view the whole intersection area and the signings without any obstructions that could restrain sight lines of them (AASHTO, 2011).

4.3.3. (X.H) Sight Distances

Are sight lines obstructed by signs, bridge abutments, buildings, landscaping, etc.? (X.H.2)

There are some areas, along legs of intersections, which have to be clear of any obstructions. In case of existence of any obstruction within these areas, the sight view of the driver will have been blocked and could lead to danger by avoiding the chance of seeing any conflicting vehicle. This mentioned area is defined as sight triangle. The size of these triangles depends on the design speed of the intersecting roads and the type of the traffic control at the intersection. These areas are defined by observing some driver behavior and plotting them on space-time profile. There are 2 kinds of sight triangles; approach sight triangle and departure sight triangle (AASHTO, 2011).

Each side of an intersection should contain a triangular area in which there will not be any obstructions and the length of the legs of these triangular areas must be sufficient to stop or slow down after the driver notices a conflicting vehicle without colliding it. Approach sight triangle is valid for the vehicles approaching intersection while departure sight triangle is the one related with the vehicles stopping already at an intersection and about to cross or get into the intersection. The vertex point of these triangles are located on the drivers moving on the minor road of the intersection. This point is also considered as decision point which means at this point the driver must start slowing down or taking off. The perpendicular distance from this point to the axis of major road is one side of the triangle when the other side of the triangle represents the length on which the vehicle without the right-of-way can see the vehicle with the right-of way, the driver of the potentially conflicting vehicle can also see the first vehicle (AASHTO, 2011). Typical illustrations are given on Figure 4.8.

While determining the obstructions inside the sight triangle, the profiles of the intersecting roads must be taken into consideration since the height of the driver's eye must be over any object within sight triangle. In case of existence of such objects within sight triangle, like buildings, fences, parked vehicles, walls, roadway structures, trees and bushes etc., the height of these objects must be lowered or totally removed where possible. Hence, identification of an object obstructing sight view at intersections requires the evaluation of both plan and profile. As before, during calculation of other sight distances in previous chapters, the height of the driver is assumed to be 1.08 m above the ground of the roadway. The height of the object is assumed to be 1.33 meters which is the height of the 15th percentile of the car population less an allowance of 250 mm. Hence, using the same height of the object and the driver's eye leads to reciprocal evaluation for intersecting vehicles (AASHTO, 2011).

As mentioned before, the dimensions of the sight triangles depend both on design speed of approaching roads and type of traffic control used at that intersection. Different types of traffic controls are presented below;

Case A—Intersections with no control

Case B—Intersections with stop control on the minor road

- Case B1—Left turn from the minor road
- Case B2—Right turn from the minor road
- Case B3—Crossing maneuver from the minor road

Case C—Intersections with yield control on the minor road

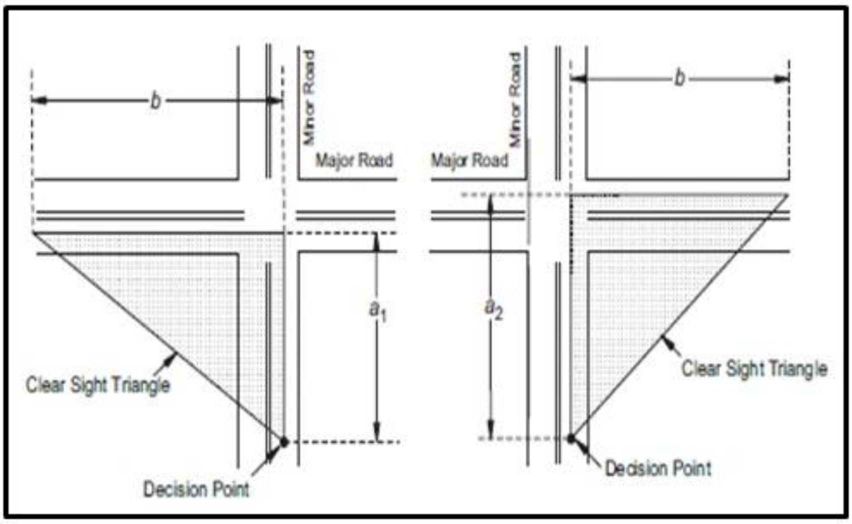
- Case C1—Crossing maneuver from the minor road

- Case C2—Left or right turn from the minor road

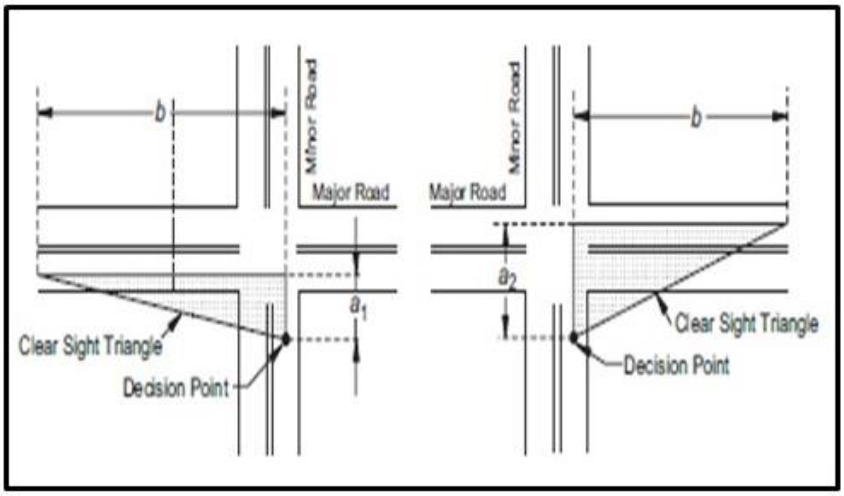
Case D—Intersections with traffic signal control

Case E—Intersections with all-way stop control

Case F—Left turns from the major road (AASHTO, 2011)



(a)



(b)

Figure 4.8. Approach sight triangle (Left) (a) and Departure Sight Triangle (Right) (b) Illustrations (AASHTO, 2011)

For intersections which belong to Case A, where there is not a yield sign, stop sign or a traffic signal etc., the method of calculating sight triangle becomes familiar to calculation of stopping sight distance since the vehicle is expected to be able to stop without collision after the driver notices the potential conflict. As in calculation of stopping sight distance which has been presented in previous chapters, 2.5 second of reaction time is considered. According to field observations, it has been derived that the vehicles reduce their speed to approximately 50 percent of their approach speed when they are about to get into an uncontrolled intersection and this reduction has been observed to take place with a deceleration rate of 1.5 m/s^2. Summing up all the data retrieved, *Table 4.14* is presented in order to define the length of the sight triangle legs depending on the relevant design speed values (AASHTO, 2011).

Table 4.14. *Length of Sight Triangle Leg for Case A (AASHTO, 2011)*

Metric		U.S. Customary	
Design Speed (km/h)	Length of Leg (m)	Design Speed (mph)	Length of Leg (ft)
20	20	15	70
30	25	20	90
40	35	25	115
50	45	30	140
60	55	35	165
70	65	40	195
80	75	45	220
90	90	50	245
100	105	55	285
110	120	60	325
120	135	65	365
130	150	70	405
—	—	75	445
—	—	80	485

Do grades at intersecting roadways allow desirable sight distance? (X.H.4)

The roadways with grades steeper than 3% are considered to have effect on the desirable sight distances at intersections (AASHTO, 2011).

4.3.4. (X.I) Markings

Are pavement markings clearly visible in day and night time conditions? (X.(I.1)
Pavement markings are essential elements of intersections and they lead the drivers to move through correct paths by assisting to channelization (AASHTO, 2011). Hence, it is strongly crucial to have clearly visible markings on the pavements.

4.4. Interchange (I) Issues

Issues, which are subject to urban major arterials while conducting RSAs, are presented in *Table 4.15*. The items that will be studied during case study will be explained in detail including background data while others will not be given since they are not available to examine.

Table 4.15. *Interchanges Issues in RSA*

(I.A) Location / Spacing	**1.** Does the location of the interchange service the needs of the surrounding community?
	2. Determine if spacing between interchanges in the network is sufficient.
(I.B) Weaving Lanes	**1.** Ensure appropriate length and number of weaving lanes.
(I.C) Ramps	**1.** Is the design speed appropriate for site limitations, ramp configurations, and vehicle mix?
	2. Adequate distance between successive entrance and exit noses?
	3. Is design of main lane adequate at exit/entrance terminals?
(I.D) Exit Terminals	**1.** Is the length adequate for deceleration?
	2. Is adequate sight and decision sight distance provided?
	3. Are spiral curves warranted? If so, do spirals begin and end at appropriate locations?
(I.E) Entrance Terminals	**1.** Is the length appropriate for acceleration and safe and convenient merging with through traffic?
	2. Are spiral curves warranted? If so, do spirals begin and end at appropriate locations?
	3. Is visibility obscured by traffic barriers and other obstructions?
(I.F) Service Road Systems	**1.** Is there adequate distance between the highway and the service road to allow for future development?
	2. Does service road traffic adversely affect traffic flow along the highway?
	3. Is there sufficient access to/from the service road?
(I.G) Lane Balance / Basic Lanes / Lane Continuity	**1.** Is the number of lanes appropriate for safe operations and to accommodate variations in traffic patterns?
	2. Is there coordination of lane balance and basic lanes?
	3. Is lane continuity maintained?
(I.H) Auxiliary / Turning Lanes	**1.** Are they of appropriate length?
	2. Is there advance warning of approaching auxiliary lanes?
	3. Is sight distance for entering/leaving vehicles appropriate?
	4. Are tapers installed where needed? Are they correctly aligned?
	5. Is the service road being used for its original intent?

4.4.1. (I.C) Ramps

Is the design speed appropriate for site limitations, ramp configurations, and vehicle mix? (I.C.1)

The superelevation that must be applied on a road with respect to its design speed and radius is presented in *Table 4.16*.

Table 4.16. *Design Speed Regarding with Radii and Superelevation Rate (max e = 4%) (AASHTO, 2011)*

Metric									
	V_d = 20 km/h	V_d = 30 km/h	V_d = 40 km/h	V_d = 50 km/h	V_d = 60 km/h	V_d = 70 km/h	V_d = 80 km/h	V_d = 90 km/h	V_d = 100 km/h
e (%)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)	*R* (m)
NC	163	371	679	951	1310	1740	2170	2640	3250
RC	102	237	441	632	877	1180	1490	1830	2260
2.2	75	187	363	534	749	1020	1290	1590	1980
2.4	51	132	273	435	626	865	1110	1390	1730
2.6	38	99	209	345	508	720	944	1200	1510
2.8	30	79	167	283	422	605	802	1030	1320
3.0	24	64	137	236	356	516	690	893	1150
3.2	20	54	114	199	303	443	597	779	1010
3.4	17	45	96	170	260	382	518	680	879
3.6	14	38	81	144	222	329	448	591	767
3.8	12	31	67	121	187	278	381	505	658
4.0	8	22	47	86	135	203	280	375	492

AASHTO (2011) also presents a guideline *Table 4.17* about the design speed of ramps as related to design speed of the main arterial. Considering that the design speed of main highway is 70 km/h, recommended design speed for the ramps is to be 40 km/h as the lower range (AASHTO, 2011).

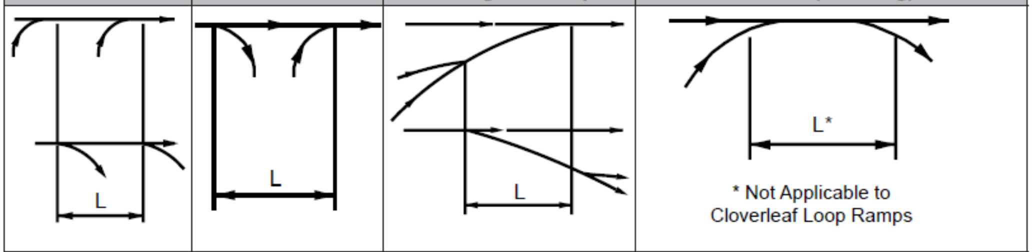
Table 4.17. *Ramp Design Speeds Recommended a Related to Highway Design Speed (AASHTO, 2011)*

Metric								
Highway design speed (km/h)	50	60	70	80	90	100	110	120
Ramp design speed (km/h)								
Upper range (85%)	40	50	60	70	80	90	100	110
Middle range (70%)	30	40	50	60	60	70	80	90
Lower range (50%)	20	30	40	40	50	50	60	70

Adequate distance between successive entrance and exit noses? (I.C.2)

When two or more ramp terminals are located following each other, a desired space must be given between these terminals in order to have a sufficient weaving length so that the movements of the vehicles will be put into operation with least discomfort. However, these distances are dependent on the type of interchanges. Hence, *Table 4.18* has been prepared in order to define minimum required space distances between successive ramp terminals regarding with the types of interchanges by AASHTO (2011).

Table 4.18. *Minimum Recommended Spacing between Successive Ramp Terminals (AASHTO, 2011)*



EN-EN or EX-EX		EX-EN		Turning Roadways		EN-EX (Weaving)			
Full Freeway	CDR or FDR	Full Freeway	CDR or FDR	System Interchange	Service Interchange	System to Service Interchange		Service to Service Interchange	
						Full Freeway	CDR or FDR	Full Freeway	CDR or FDR
Minimum Lengths Measured between Successive Ramp Terminals									
300 m (1000 ft)	240 m (800 ft)	150 m (500 ft)	120 m (400 ft)	240 m (800 ft)	180 m (600 ft)	600 m (2000 ft)	480 m (1600 ft)	480 m (1600 ft)	300 m (1000 ft)

Notes: FDR—Freeway distributor road EN—Entrance
CDR—Collector distributor road EX—Exit

4.4.2. (I.D) Exit Terminals

Is the length adequate for deceleration? (I.D.1)

When the ramp is composed of more than one lane, independent from the design speed of it, there is a distance of 450 meters which will create the deceleration lane with an additional auxiliary lane in order not to reduce the number of lanes on the through direction for parallel types. Also, a distance of taper must be provided in order to place the auxiliary lane. As another criteria, AASHTO (2011) recommends that the curve on the ramp must have a radius of 300 meters at least which can be seen on Figure 4.9.

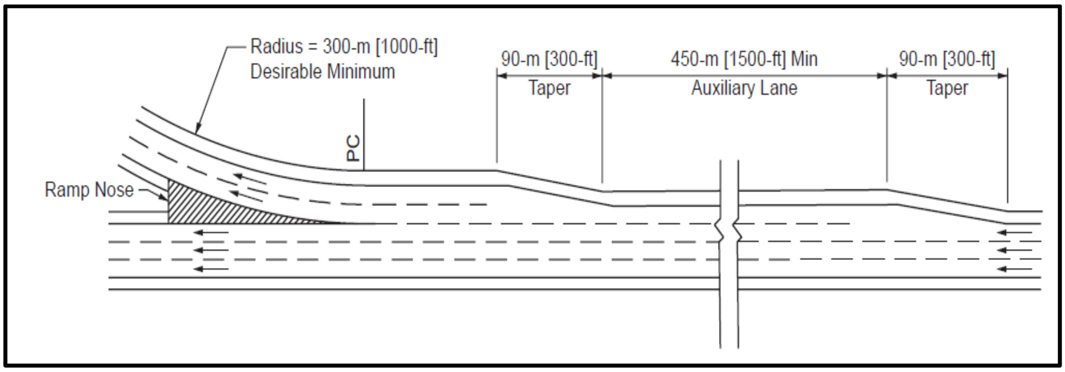


Figure 4.9. Typical Designs for two-lane exit terminals (AASHTO, 2011)

Is adequate sight and decision sight distance provided? (I.D.2)

Sight distance at exit terminals on the main line should be provided at least equal to the stopping sight distance of the through traffic, where practical, 25 percent more than it. The view of entire exit terminal and the exit nose should be sufficiently clear for the drivers. Decision sight distance is also desirable at the approach nose of the exit terminals so that motorists will have the required time in order to understand the situation and take the action according to the direction they would like to take without any risky movements. SSD and DSD values are given on *Table 4.19* and *Table 4.20* below, respectively;

Table 4.19. *Stopping Sight Distance (AASHTO, 2011)*

Metric					U.S. Customary				
Design Speed (km/h)	Brake Reaction Distance (m)	Braking Distance on Level (m)	Stopping Sight Distance		Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Stopping Sight Distance	
			Calculated (m)	Design (m)				Calculated (ft)	Design (ft)
20	13.9	4.6	18.5	20	15	55.1	21.6	76.7	80
30	20.9	10.3	31.2	35	20	73.5	38.4	111.9	115
40	27.8	18.4	46.2	50	25	91.9	60.0	151.9	155
50	34.8	28.7	63.5	65	30	110.3	86.4	196.7	200
60	41.7	41.3	83.0	85	35	128.6	117.6	246.2	250
70	48.7	56.2	104.9	105	40	147.0	153.6	300.6	305
80	55.6	73.4	129.0	130	45	165.4	194.4	359.8	360
90	62.6	92.9	155.5	160	50	183.8	240.0	423.8	425
100	69.5	114.7	184.2	185	55	202.1	290.3	492.4	495
110	76.5	138.8	215.3	220	60	220.5	345.5	566.0	570
120	83.4	165.2	248.6	250	65	238.9	405.5	644.4	645
130	90.4	193.8	284.2	285	70	257.3	470.3	727.6	730
					75	275.6	539.9	815.5	820
					80	294.0	614.3	908.3	910

Note: Brake reaction distance predicated on a time of 2.5 s; deceleration rate of 3.4 m/s^2 [11.2 ft/s^2] used to determine calculated sight distance.

Table 4.20. *Decision Sight Distance Values (AASHTO, 2011)*

Metric						U.S. Customary					
Design Speed (km/h)	Decision Sight Distance (m)					Design Speed (mph)	Decision Sight Distance (ft)				
	Avoidance Maneuver						Avoidance Maneuver				
	A	B	C	D	E		A	B	C	D	E
50	70	155	145	170	195	30	220	490	450	535	620
60	95	195	170	205	235	35	275	590	525	625	720
70	115	325	200	235	275	40	330	690	600	715	825
80	140	280	230	270	315	45	395	800	675	800	930
90	170	325	270	315	360	50	465	910	750	890	1030
100	200	370	315	355	400	55	535	1030	865	980	1135
110	235	420	330	380	430	60	610	1150	990	1125	1280
120	265	470	360	415	470	65	695	1275	1050	1220	1365
130	305	525	390	450	510	70	780	1410	1105	1275	1445
						75	875	1545	1180	1365	1545
						80	970	1685	1260	1455	1650

Avoidance Maneuver A: Stop on rural road—t = 3.0 s

Avoidance Maneuver B: Stop on urban road—t = 9.1 s

Avoidance Maneuver C: Speed/path/direction change on rural road—t varies between 10.2 and 11.2 s

Avoidance Maneuver D: Speed/path/direction change on suburban road—t varies between 12.1 and 12.9 s

Avoidance Maneuver E: Speed/path/direction change on urban road—t varies between 14.0 and 14.5 s

4.4.3. (I.E) Entrance Terminals

Is the length appropriate for acceleration and safe and convenient merging with through traffic? (I.F.1)

For single lane and tapered merges, AASHTO (2011) recommends a length for acceleration lane in order to have a safe and convenient travel during entry. These recommended lengths are given according to design speed, reached speed and initial speed on *Table 4.21*. When the entrance terminal merges as multi-lane and in a parallel type, AASHTO (2011) still recommends to use *Table 4.21*, but it recommends to locate a taper of 90 m following the end of acceleration lane which is given on Figure 4.10.

Table 4.21. *Minimum Acceleration Lengths for Entrance Terminals (AASHTO, 2011)*

Metric									
Acceleration Length, *L* (m) for Entrance Curve Design Speed (km/h)									
Highway		Stop Condition	20	30	40	50	60	70	80
Design Speed, *V* (km/h)	Speed Reached, V_a (km/h)	and Initial Speed, V'_a (km/h)							
		0	20	28	35	42	51	63	70
50	37	60	50	30	—	—	—	—	—
60	45	95	80	65	45	—	—	—	—
70	53	150	130	110	90	65	—	—	—
80	60	200	180	165	145	115	65	—	—
90	67	260	245	225	205	175	125	35	—
100	74	345	325	305	285	255	205	110	40
110	81	430	410	390	370	340	290	200	125
120	88	545	530	515	490	460	410	325	245

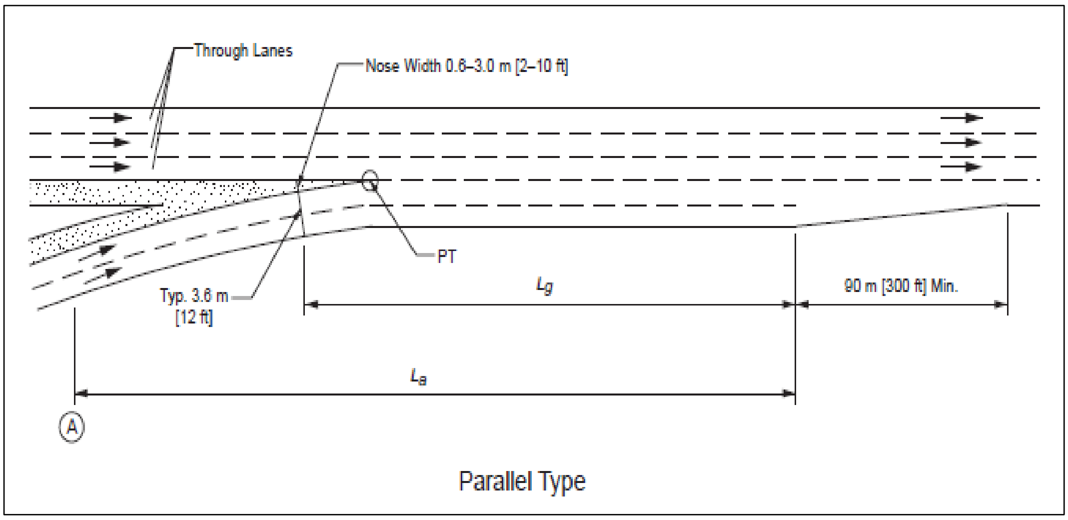


Figure 4.10. Minimum Acceleration Length Terminals for Multi-Lane and Parallel Type Entrance Terminals (AASHTO, 2011)

4.4.4. (I.(G.) Lane Balance / Basic Lanes / Lane Continuity

Is the number of lanes appropriate for safe operations and to accommodate variations in traffic patterns? (I.H.1)

One of the crucial steps of designing roadways is the designation and arrangement of basic number of lanes on them. Notwithstanding the irregularities on a route, the number of lanes is defined over a significant length. Hence, along these sections, consistency of number of lanes should be provided. Number of lanes is designed according to design hour volume which is representing the morning and evening rush hour volumes during weekdays. Local variations of traffic volumes on these substantially long segments are ignored (AASHTO, 2011).

Is there coordination of lane balance and basic lanes? (I.H.2)

In order to have decent traffic flow on highways through and beyond an interchange, a balance should be provided in the number of lanes on both major road and ramps. As mentioned before, basic number of lanes are designated according to the traffic volume values. However, while typical examples of lane balance is illustrated on Figure 4.11, lane balance coordination is another issue that should be taken care of at interchanges which require the following rules;

- The number of lanes on the major road beyond entrance terminal should not be less than the number of lanes merging minus one while it can be equal to the sum of all lanes merging.

- The number of lanes approaching the exits on the major road should be equal to the number of lanes on the major road beyond exit plus number of lanes on the exit ramp minus one. However, some exemptions are defined for cases such as; closely spaced interchange where the end of the taper at entrance terminal and the beginning of the taper at exit terminal is less than 450 m and a continuous auxiliary lane is placed between these terminals. In these cases, the number of lanes approaching can be equal to number of lanes beyond exit point plus the number of lanes on exit by dropping the auxiliary lane.
- Reduction of lanes on a major road should be more than one at one point (AASHTO, 2011).

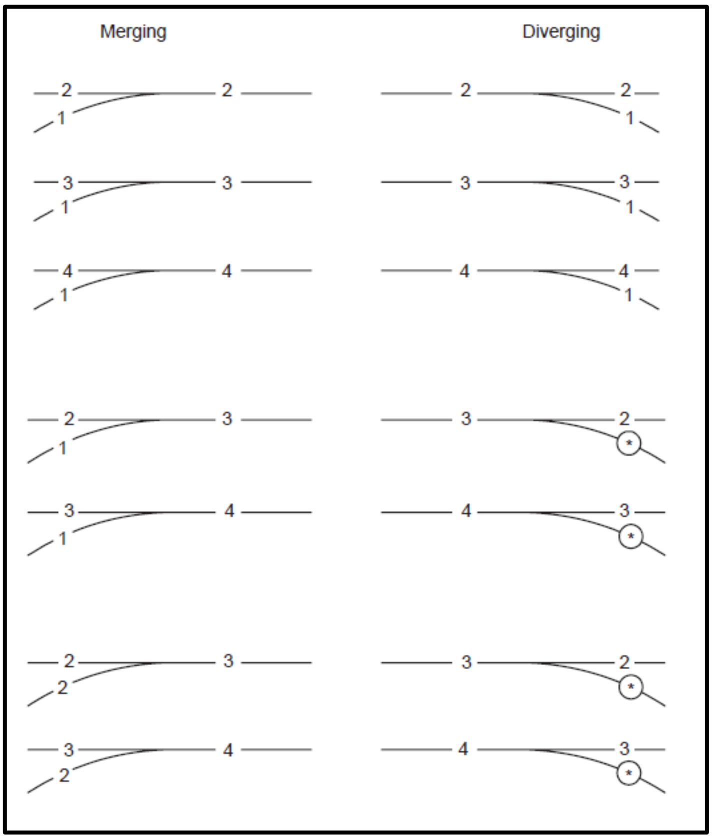


Figure 4.11. Typical Examples of Lane Balance (AASHTO, 2011)

Is lane continuity maintained? (I.H.2)

Definitions of lane balance and lane continuity may seem to be similar which could lead to confusion. However, lane continuity can be described as keeping the major roadway with the same number of lanes before, throughout and beyond the interchange. Hence, an interchange with appropriate configuration of lane balance may fail to compliance with the lane continuity requirements while the contrary is also possible (AASHTO, 2011). Coordination of Lane Balance and Lane Continuity illustration can be found on Figure 4.12.

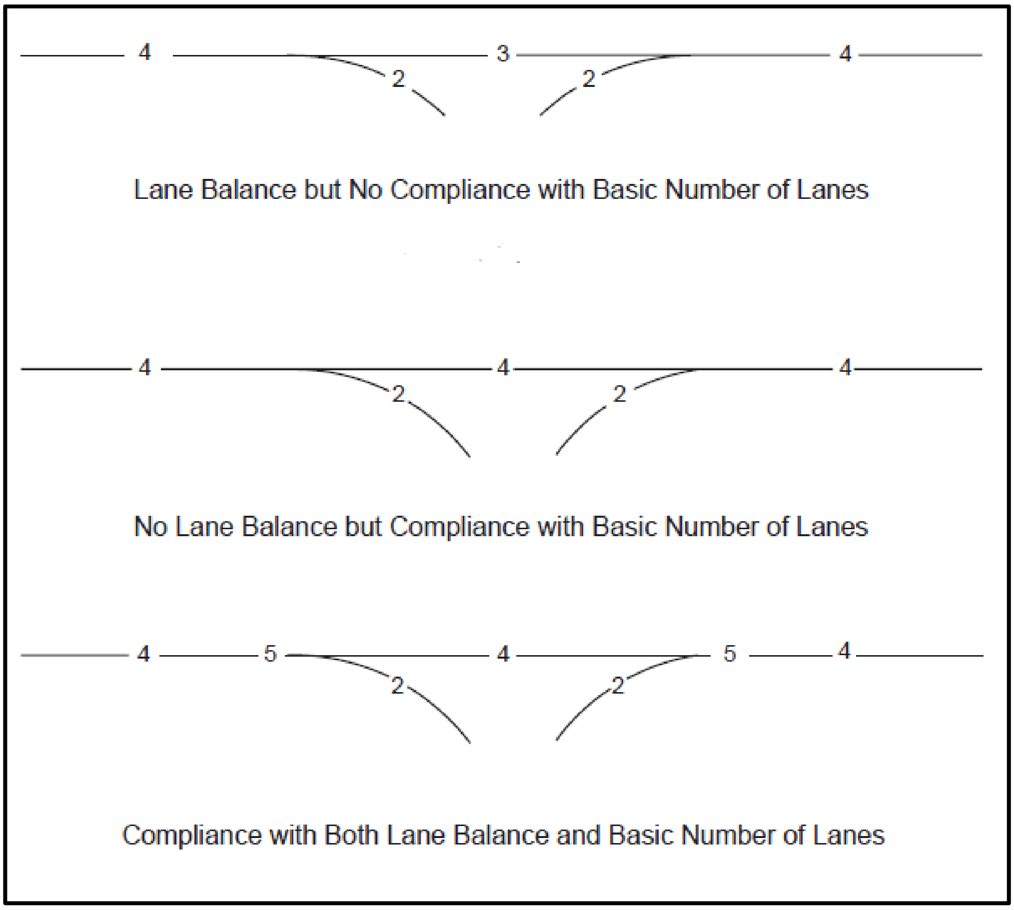


Figure 4.12. Coordination of Lane Balance and Lane Continuity (AASHTO, 2011)

CHAPTER 5

1071 MALAZGİRT BOULEVARD PROJECT

5.1. Study Area

The study area of this thesis is the existing road that has been constructed recently and opened for traffic on February 2014 in Ankara which is the capital city of Turkey. Turkey is a country that is located between Europe and Asia. Although, it is considered to be in Europe Continent by some authorities, since the country is not a member of EU, in some sources the country is said to be in Asia. However, Turkey has lands on both continents physically and it is bordered by Mediterranean Sea, Aegean Sea and Black Sea while it also has land neighbors which are Georgia, Azerbaijan (Nakhcivan), Armenia, Iran, Iraq, Syria on the east side (Asia) and Greece and Bulgaria on the west side (Europe). Turkey has a population of 79,814,871 and almost 25 million out of this is composed by only 3 cities out of 81. These 3 cities can be listed as İstanbul with 14,804,116, Ankara with 5,346,518 and İzmir with 4,223,545 population (TurkStat, 2016). Ankara, being the second biggest city by means of population both in Turkey and Europe, hosts the road which is in question in this thesis. The location of Ankara over the map of Turkey is as seen on Figure 5.1.

Figure 5.1. Location of the Study Corridor in Ankara, Turkey (Google Maps, 2018)

The road that is subject to this thesis has a story that goes back to almost 25 years ago. By being an extension of the existing Anatolian Blvd., the plan of the road had been prepared by GDH in 1980's and it had been added into "Ankara Land Use Plan - 1990" in as an internal ring road that would connect Ankara-İstanbul (Fatih Sultan Mehmet Blvd.), Ankara-Konya (Mevlana Blvd.) and Ankara-Eskişehir (İnönü Blvd.) roads to each other. This plan had been approved by Ankara Metropolitan Municipality (AMM) and the construction of Anadolu Blvd. had been completed in 1988 pursuant to this plan. However, Anadolu Blvd. had been constructed between only Fatih Sultan Mehmet Blvd and İnönü Blvd. and it had stopped at the border of Middle East Technical University (METU). METU has been established in 1956 under the name of "Middle East High Technology Institute" and during its presence it has succeeded to keep its position as one of the best among all the other universities in Turkey. Nowadays, it continues its education with 43 undergraduate, 107 graduate and 69 doctorate programs with a total number of alumni around 120.000. The campus area is around 4500 hectares while the forest area is around 3000 hectares. The layout of the related area before the construction of 1071 Malazgirt Boulevard is given on Figure 5.2.

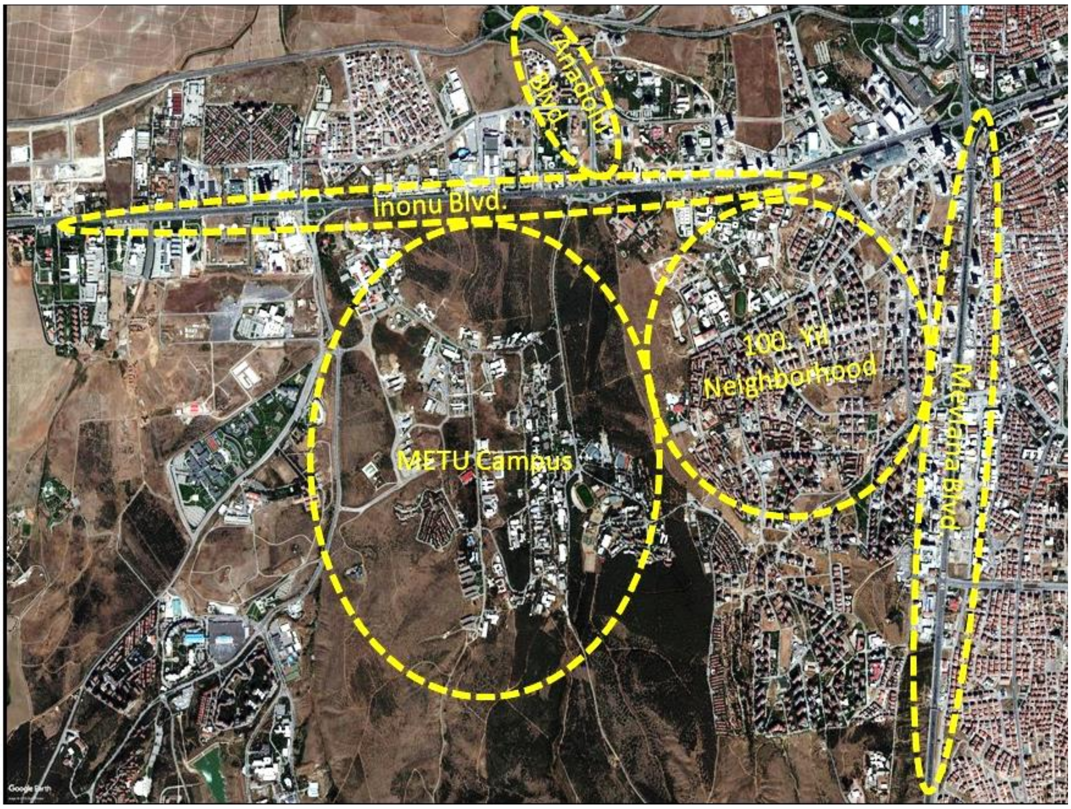


Figure 5.2. Location of METU and Layout of Study Area Before Construction of 1071 Malazgirt Boulevard

According to "Ankara Land Use Plan – 1990", Anadolu Blvd. has to pass through METU Campus in order to be able to complete the connection providing also the section between İnönü Blvd. and Mevlana Blvd. Because of this reason, AMM asked for the required authorization from METU to get the permission for the construction of extension of Anadolu Blvd towards inside of METU Campus. Then, in 1993 and 1994, METU approved the 4 km long İnönü Blvd – Mevlana Blvd. connection alignment, 1.8 km of which passes through METU Campus and they processed the layout of the road onto METU Land Use Plan. Since then, the edge of the eastern border of campus for that section had been reserved for use of road construction area (Rectorship of METU, 2013). As told before, 1071 Malazgirt Blvd., also known as

METU Road, partly lies in the former lands of METU Campus and the remaining part of the alignment rests on the neighborhood that is called as 100. Yil Isci Bloklari. The campus has 3 gates which are named as A1, A4 and A7. A7 is the gate on the west side of the campus which is irrelevant with the road. However, A1 and A4 gates are directly affected by the construction of this new road. Anadolu Blvd., as seen on the figure above, used to end at the A1 gate of campus which will be shown in detail later in this thesis. One of the points that have been affected by this METU Road is this initiating point of the road since there is both an interchange of two main axes of Ankara and the gate of one of the biggest universities of Turkey. After this point, the road which has 8 lanes and almost 40 m width goes along the edge of the campus and leaves the campus on the east side next to A4 gate. Leaving the campus, the road passes through 100. Yil İsci Bloklari and ends up after crossing Mevlana Blvd. with a bridge. There are 6 crossing with other roads in the project. While 5 of them are multi-level crossings (interchanges), only one of them is level crossing (intersection). The layout of 1071 Malazgirt Boulevard including the locations of the crossings and related points is presented on Figure 5.3 and the notations on this figure are explained in *Table 5.1*. These crossings will be explained in the next chapters.

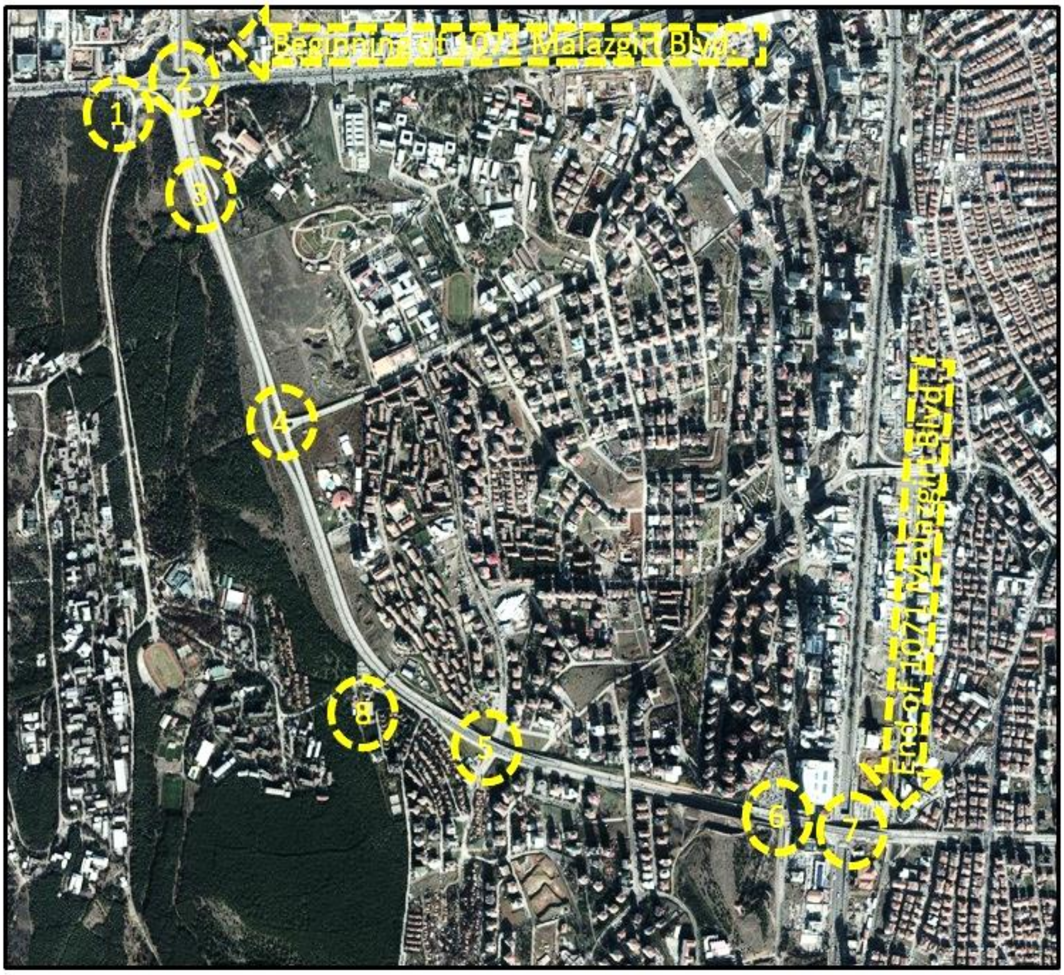


Figure 5.3. Layout of 1071 Malazgirt Boulevard

Table 5.1. *Explanation of Labels Given on Layout of 1071 Malazgirt Boulevard*

1	A1 Gate of METU Campus
2	METU Interchange (Inonu Boulevard)
3	U-Turn Interchange
4	Ogretmenler Avenue Intersection
5	100. Yil Interchange
6	Muhsin Yazicioglu Avenue Interchange
7	Mevlana Boulevard Interchange
8	A4 Gate of METU Campus

The scope of the study is to present all the workings that have been done during the preparation of METU Road Project and show the alternative that was chosen by applying the RSA processes also by comparing the project on paper and the existing situation that is constructed on site. A checklist will be prepared that will be used in evaluation of safety for the chosen alternative. Finally, after pointing out the important points, recommendations and proposals will be expressed as conclusion. Hence, it is aimed that the points that must be taken into consideration while designing and constructing a major urban arterial road will be clarified and they will lead the upcoming similar projects in order to serve much safer and comfortable transportation system for all the users.

5.2. Features of the Current Design

At the end of negotiations between METU committee and AMM, the final alternative has been chosen for İnönü Blvd and A1 gate intersections. The selected alternative was the one with a bridge that overpasses the traffic that comes from west side on İnönü Blvd. and tends to join 1071 Malazgirt Blvd. towards south side over the road of A1 gate connection roads of METU campus and the traffic that comes from north side on Anadolu Blvd. and/or gets out of A1 gate and tends to move towards east side which goes to city center is u-turned by the previously mentioned bulb-shaped underpass below 1071 Malazgirt Blvd. The existing layout of the A1 gate intersection tried to be kept similar only by omitting out the passage that used to let vehicles which get out from A1 gate and join to İnönü Blvd. directly towards east side which heads to city center since this connection passage was coinciding with the inlet of the new bridge that overpasses the A1 gate connection roads.

The corridor that had been reserved for this alignment was clear and this corridor was used from beginning to end. Hence, the corridor did not cause any discussions between authorities and it was approved by all of them. Apart from İnönü Blvd and A1 gate intersections, the other intersection that falls in METU campus area did not lead to any divergence since both sides agreed on to have a level roundabout intersection. This T-intersection was designed in order to create the connection of 100. Yil Neighborhood to 1071 Malazgirt Blvd. as a passage other than the one which is outside campus area. However, by 2017, although the construction of the road was done according to intersection design with roundabout, the roundabout has not been constructed yet so that only one direction of the road has connection to Öğretmenler Avenue. The other intersections which are outside campus area were decided by AMM and no design alternative has been studied for any of them since they were all clear also in Ankara Land Use Plan.

The next interchange location is the one that used to exist which was serving for Çiğdem Neighborhood, 100. Yil Neighborhood and A4 gate of the campus. The intersection did not have any roundabout or traffic lights. It was a level, simple, uncontrolled 3-legs intersection. However, Cigdem Neighborhood had a considerable amount of population and the access to A4 gate of the campus had a considerable amount of traffic volume. Hence, an adequate solution had to be thought in this location and considering also the topography, the former intersection being on the low point, a bridge crossing with a roundabout below it has been designed. The parallel legs of the interchange allowed the merge and diverge of the traffic to 1071 Malazgirt Blvd. There is also a road which is called as 1505. Avenue and this avenue is one of the main roads in the vicinity crossing the new boulevard so that an underpass was designed so as not to interrupt the traffic of this road. Similar to 1505. Avenue, another main avenue that intersects with the 1071 Malazgirt Blvd., 1506. Avenue also had to cross the new boulevard without any interruption. Hence, a bridge that lets this avenue overpass the new boulevard was designed. In the last part, there is one avenue and another main boulevard which is called Mevlana Boulevard intersecting 1071 Malazgirt Blvd. While the avenue named as Muhsin Yazicioglu Avenue is underpassing the new road with a concrete box structure, 1071 Malazgirt Blvd. is overpassing Mevlana Blvd. with an intersection below the bridge. The parallel legs of the new road have access to this intersection downgrading to its elevation. Below this intersection, there is another concrete box structure, which used to exist before, that provides the traffic of Mevlana Blvd. move in north and south direction without interruption. Also, there is a bridge that will be used for access to foreign ministry zone that will be constructed later to which no connection road exists yet. The connection roads will be provided after the construction of the area. Hence, this bridge that overpasses the new road will not be subject to this thesis. After crossing Mevlana Blvd., the project ends.

5.2.1. Alternative Routes for A1 Gate during Design Stages

Although the corridor of the alignment had been defined almost 25 years ago, some points had still remained unsolved. These points were the intersection points with other roads. The location of these intersections was given in the previous section briefly. The first and the most important intersection to be solved was the one that is on the İnönü Blvd. including the gate of the METU Campus. The name of this gate is A1 and it serves as the main gate of the campus. This fact made it obliged to be considered as another intersection inside the main intersection. Hence, after construction of the new METU road, the serviceability of A1 gate had to be kept as much as possible while providing a fully working interchange that will function in each direction between Anadolu Blvd., the boulevard to which the new 1071 Malazgirt Blvd. (METU Road) will be an extension, İnönü Blvd. and new METU Road. Normally, if the gate of campus did not exist, the interchange was going to be constructed as a regular cloverleaf junction that is used very common on İnönü Blvd for intersecting other main roads. However, in this case, the legs on this side (south-west) have become very problematic and tough to solve whilst the other three were still able to be regular cloverleaf legs. For this manner, the design company, AMM and a committee that was established by rectorship of METU consisting of academic staff from different relevant departments had discussions and meetings in order to find out a decent solution that would affect the gate of the campus least. During these discussions, many alternatives were studied upon the requests of committee. These studies can be summarized in 3 types in general; level intersection, underpass and overpass solutions on A1 gate.

These three alternatives did not have only one type of solution. Each of them has been studied in detail and 27 sub-alternatives came out at the end in total. First alternative was to create a level crossing intersection that was going to serve as both the two legs of the main interchange on this side and the enter and exit route for campus. However, in spite of studying some sub-alternatives both with or without roundabouts and traffic lights, it turned out that these solutions were not going to work properly since the interchange legs were to belong to a main arterials interchange which were supposed to work like express way interchange legs although the location is inside the city because these boulevards are wide main roads with high traffic capacity and design speed. One example of the studied level intersection is given on Figure 5.4;

Figure 5.4. Satellite photo of one of the level intersection alternative studies on A1 Gate

After the studies of level intersections on A1 gate, overpass solutions were proposed for the connection of traffic that comes from west side on İnönü Blvd. and tends to head to 1071 Malazgirt Blvd. towards south side. However, in that case, the traffic that tends to go to city center direction after reaching to interchange from Anadolu Blvd. direction or getting out of the campus from A1 gate was unable to have the passage on this side since there was no space for this leg in this case and also there was an external obstacle because there exists a metro station cut and cover structure where this leg was supposed to lay on. Hence, a u-turn underpass was considered on the south side of the interchange as a separate structure that would provide this passage for the relevant traffic. This example is given on Figure 5.5;

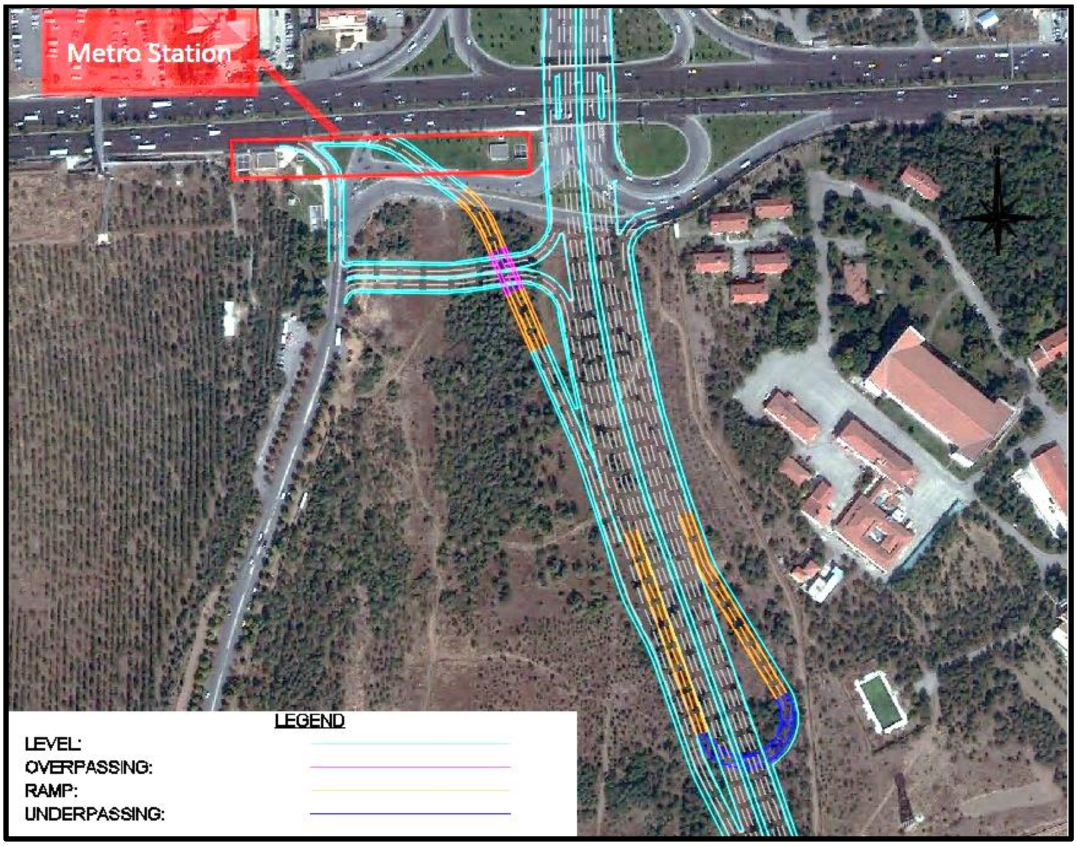


Figure 5.5. Satellite photo of one of the overpass alternative study on A1 Gate

Yet, in case of these alternatives, it was evident that the existing layout and landscape was being affected by the new intersection. Hence, an alternative that would affect the surrounding area and trees least were thought which was to place this leg that will serve for the traffic which comes from west side on İnönü Blvd. and tends to go to 1071 Malazgirt Blvd. towards south side underground. However, this solution had some difficulties considering the topography, soil type and cost. The tunnel was supposed to pass through a soft soil with shallow cover depth. Because of this reason, this alternative has never been leant towards by AMM. The leg needed for traffic that comes from north side on Anadolu Blvd. and/or gets out of campus from A1 gate and tends to go to city center direction towards east side was considered to be the u-turn underpass as in the previous overpass solution again. Although, some studies were done upon minor changes in the plan and profile of this solution, one of the examples can be seen on Figure 5.6 below.

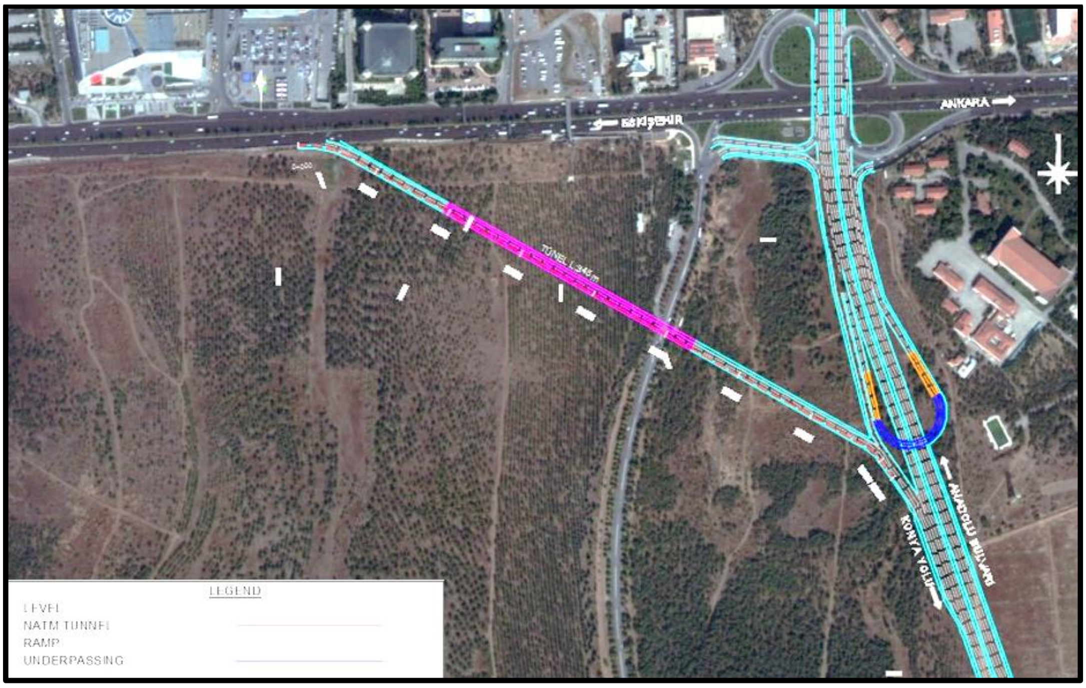


Figure 5.6. Satellite photo of one of the underpass alternative study on A1 Gate

5.2.2. Application Project

In this thesis, RSA will be conducted on both the application project and the existing situation of 1071 Malazgirt Boulevard. The total length of the project is 4180 kms. The general layout of application project can be seen on Figure 5.7.

Figure 5.7. General Layout of Application Project of 1071 Malazgirt Boulevard

Some items are audited by using data that is designed during project while other items will be observed on site. From the point of view of geometrical design, there are four horizontal curves while there are twelve vertical curves. The labels of point of intersections (PI) of horizontal curves, which are named as S-1 (HC1), S-2 (HC2), S3 (HC3) and S-4 (HC4), are given on Figure 5.8 while the notations used in these labels are explained in *Table 5.2*.

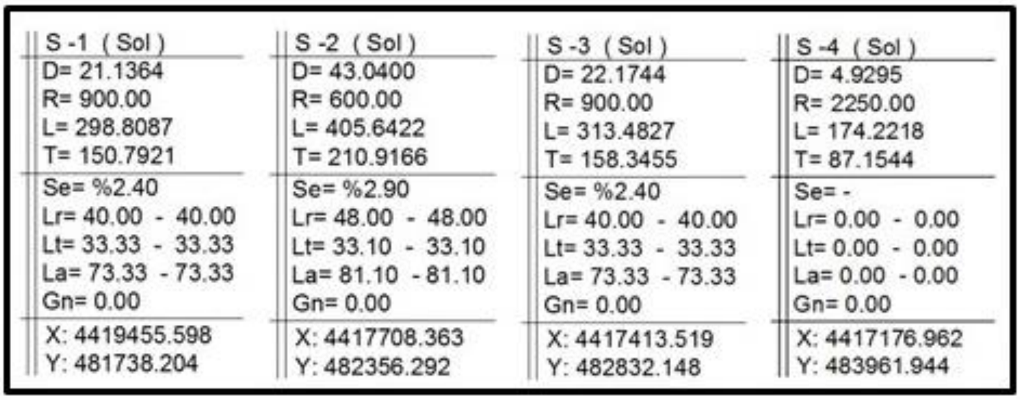
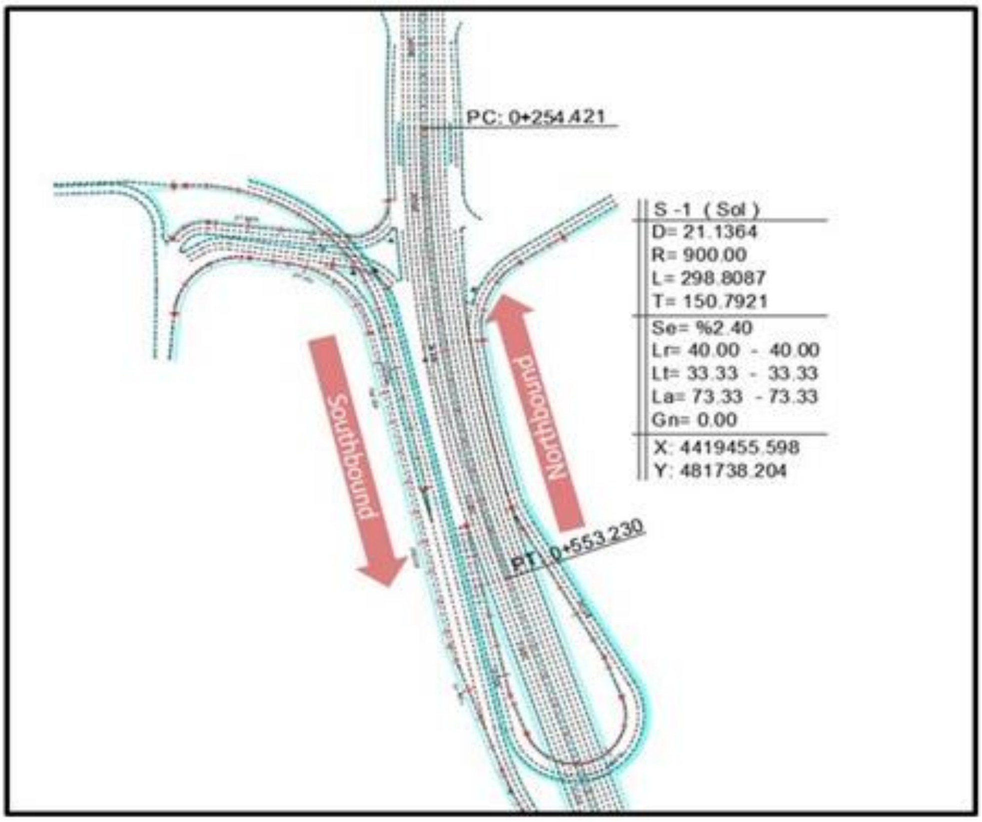


Figure 5.8. Horizontal Curve Labels of Application Project

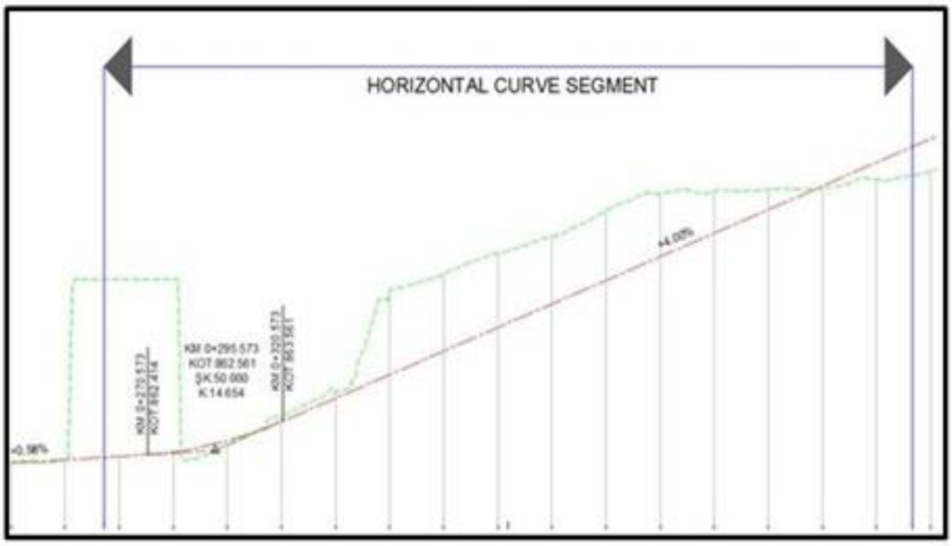
Table 5.2 *Notation used in PI labeling*

S_i	Names for Point of Intersections (PIs) (S-1 through S-4 in the project)
Sağ /Sol	Turning direction of PI (Sol: Left; Sağ: Right)
D	Angle of Deflection in grads
R	Radius of the Curve in meters
L	Length of the Curve in meters
T	Length of the tangent of curve in meters
Se	Rate of Superelevation
Lr	Superelevation runoff distance in meter
Lt	Superelevation tangent runout distance in meters
La	Sum of Lr and Lt
Gn	Widening of horizontal curve
X/ Y	Easting and northing coordinates of PI

The locations of these horizontal curves and their layout on longitudinal profile of the alignment are presented from Figure 5.9 to Figure 5.12.

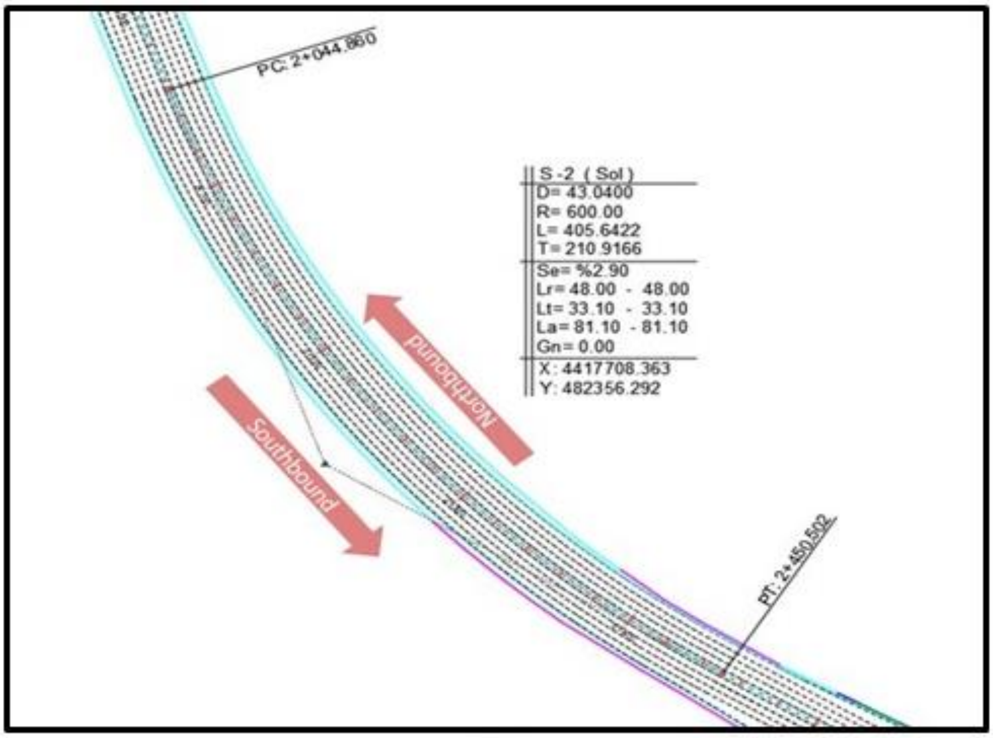


(a)

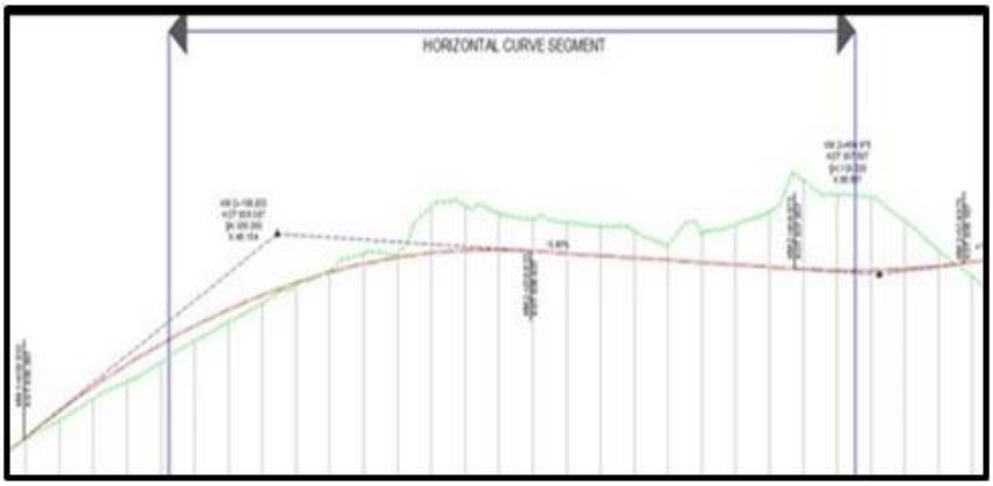


(b)

Figure 5.9. Plan View of HC1(a) and its Layout on Profile View of Alignment (b)

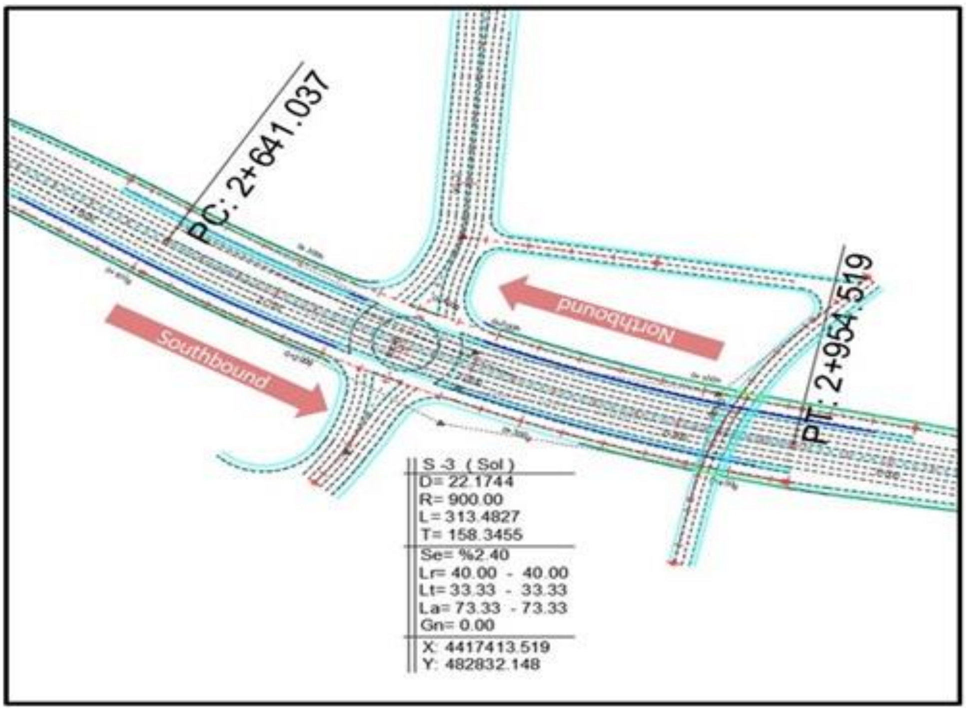


(a)

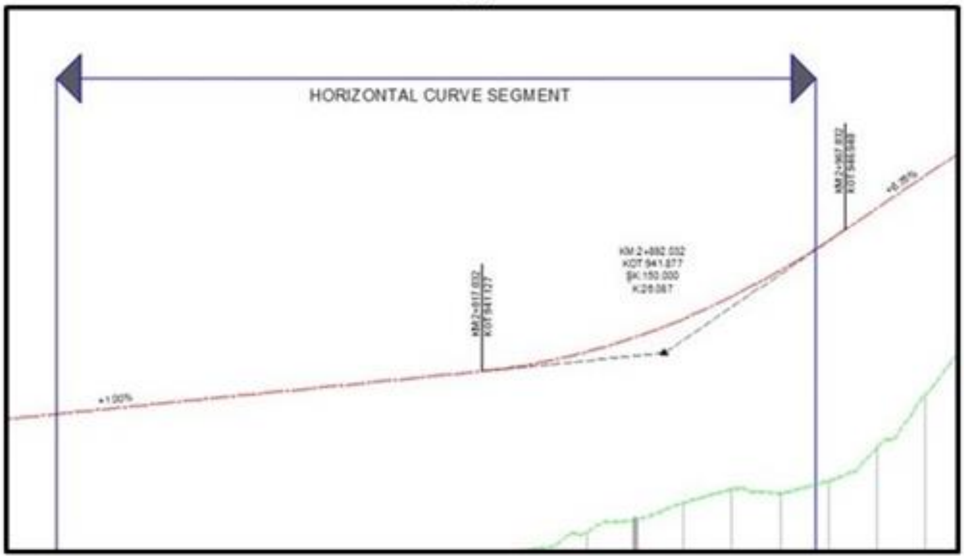


(b)

Figure 5.10. Plan View of HC2(a) and its Layout on Profile View of Alignment (b)

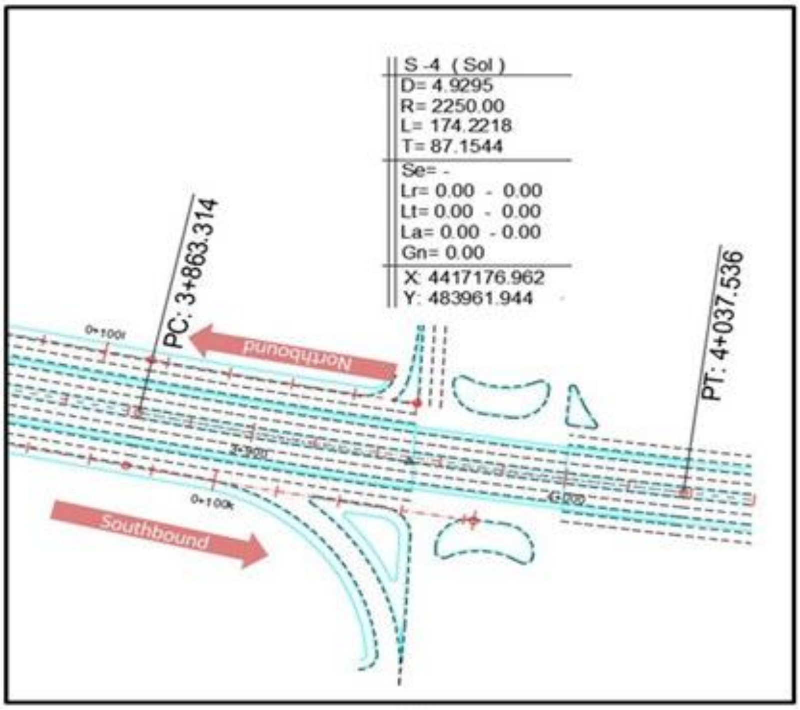


(a)

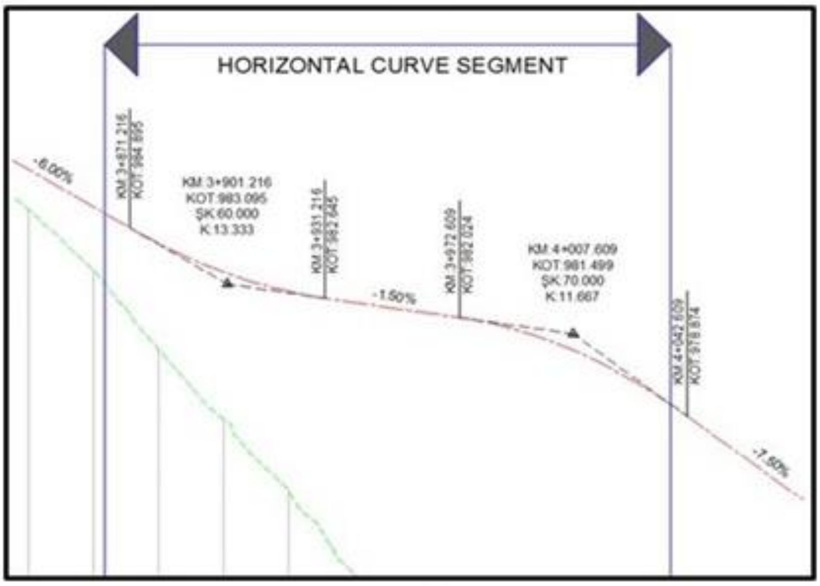


(b)

Figure 5.11. Plan View of HC3(a) and its Layout on Profil View of Alignment (b)



(a)



(b)

Figure 5.12. Plan View of HC4(a) and its Layout on Profil View of Alignment (b)

The labels of point of intersection of vertical curves are given on Figure 5.13 while the notations used in these labels are explained in *Table 5.3*.

KM:0+295.573 KOT:862.561 ŞK:50.000 K:14.654	KM:0+663.726 KOT:877.289 ŞK:100.000 K:587.574	KM:1+318.522 KOT:902.369 ŞK:100.000 K:54.640	KM:1+578.522 KOT:907.569 ŞK:100.000 K:25.000	KM:2+108.833 KOT:939.387 ŞK:300.000 K:46.154	KM:2+464.975 KOT:937.607 ŞK:100.000 K:66.667
KM:2+892.032 KOT:941.877 ŞK:150.000 K:26.087	KM:3+564.432 KOT:987.264 ŞK:220.000 K:46.316	KM:3+764.910 KOT:991.274 ŞK:110.000 K:13.750	KM:3+901.216 KOT:983.095 ŞK:60.000 K:13.333	KM:4+007.609 KOT:981.499 ŞK:70.000 K:11.667	KM:4+120.000 KOT:973.070 ŞK:60.000 K:5.839

Figure 5.13. Labels of PIs of Vertical Curves

Table 5.3. *Notations Used in Labels of PIs of Vertical Curves*

KM:	Chainage
KOT:	Elevation
ŞK:	Length of Vertical Curve
K:	Curve Parameter

The chainage list of these geometrical features is given in *Table 5.4*.

Table 5.4. *Chainage of Geometric Features*

Chainage	**Feature**	**Chainage**	**Feature**	**Chainage**	**Feature**	**Chainage**	**Feature**
0+254	TO1	1+528	BVC4	2+641	TO3	3+863	TO4
0+270	BVC1	1+628	EVC4	2+817	BVC7	3+871	BVC10
0+320	EVC1	1+958	BVC5	2+954	TF3	3+931	EVC10
0+553	TF1	2+044	TO2	2+967	EVC7	3+972	BVC11
0+613	BVC2	2+258	EVC5	3+454	BVC8	4+037	TF4
0+713	EVC2	2+414	BVC6	3+674	EVC8	4+042	EVC11
1+268	BVC3	2+450	TF2	3+709	BVC9	4+090	BVC12
1+368	EVC3	2+514	EVC6	3+819	EVC9	4+150	EVC12
TO: Origin of Tangent Curve TF: Final of Tangent Curve BVC: Beginning of Vertical Curve EVC: End of Vertical Curve							

CHAPTER 6

CASE STUDY:ROAD SAFETY EVALUATION OF 1071 MALAZGİRT BLVD

In this thesis, the safety evaluation of the newly constructed road which is called as 1071 Malazgirt Blvd will be processed. Since the construction of the road is completed, the safety evaluation will belong to existing road throughout the discussions while, in some parts, the features of the design phase will be subject to studies in order to have adequate comparisons of design and construction situations. As mentioned before, RSA is defined as safety evaluation of both design phases, during construction and after construction phases (on existing roads) by some countries, whereas, in some countries (i.e. USA), it is considered to include only design phase and during construction studies and RSI is used for the examination of the existing roads. On the other hand, again as mentioned before, RSR is a traditional method for safety evaluation of existing roads by focusing on the black spots that are already defined. However, in this thesis, both the features of existing situation and design parameters will be evaluated because the data available leads the evaluation to a combination of them. Hence, it might not be correct to name this evaluation either as RSI or as RSA because of the previously mentioned definition difference among the states and authorities.

The first thing in starting for the evaluation of 1071 Malazgirt Blvd. would be to define the checklist that will be used. There are many checklists that are used by states and authorities and private companies in safety evaluations. For this thesis, instead of creating a new checklist, a combination of an existing one that was given in Road Safety Audit Guidelines which was developed by University of New Brunswick Transportation Group in 1999 since it was based on the checklists generated upon the experiences in Australia, New Zealand, United Kingdom, United States, and Canada also by differing the rural and municipal conditions based upon the phases. The checklists published in this guideline were separated to 2 different categories as highway and municipal roads after presenting a master checklist that is used for common purpose to find out which checklist parameters are related to existing project. However, this roadway demonstrates the properties of both highway and municipal roads because of being an urban major arterial. Although it has 8 lanes and multi-level interchanges, the road is inside the boundary of municipality. Hence, a composite evaluation will be conducted. By this manner, in the previous chapter, a recommended checklist, which is a combination of already developed highway and urban checklists, has been presented that is considered as suitable for an urban major arterial. Yet, because of the lack of data, not all the items that are defined in the recommended checklist will be evaluated.

6.1. RSA on General Issues

6.1.1. (G.A) Traffic Barrier Warrants

Presence of non-traversable or fixed object hazards within clear zone (G.A.1)

According to *Table 4.2*, the suitable term that matches the current road is urban location with a classification defined as “All (Curbed)”. Since the design speeds are given in US units, metric units should be converted. As defined before, both design speed of 70 km/h and 90 km/h will be evaluated. 70 km/h equals to 44 mph while 90 km/h equals to 56 mph. For the design speed of 44 mph, which is less than 45 mph, desired clear zone width is 6 feet which equals to almost 2 meters while for 56 mph the table refers to suburban located roads. In this case, average daily traffic (ADT) decides on the width of the required clear zone width. ADT is told to be around 60,000 by Transportation Department of Metropolitan Municipality of Ankara (FN, 2014). Therefore, desirable clear zone width increases up to 30 feet while minimum is 20 feet which equal to 9 meters and 6 meters respectively. When the cross-section of the road is investigated, it is seen that the fence that is surrounding the road is almost 10 m away from the edge of through lane which is almost equal to desirable clear zone width for 90 km/h. However, sidewalk includes sign poles, lightening poles and trees on it on most sections. An illustration of such trees can be seen on Figure 6.1 (a). The distance to these objects from the edge of the through lane is mostly less than 3.5 meters which is the width of shoulder and sidewalk. In some cases, there are also objects like steel stair structure of pedestrian bridges that are close to through lane less than 2 meters which is given in Figure 6.1 (b). Hence, for each design speed, clear zone widths are not within desirable limits.

(a)

(b)

Figure 6.1. Example of unyielding objects within Clear Zone of a) trees (Km 1+100) and b) steel structure (Km 2+350)

Does a potential risk exist for vehicles crossing over the median into the path of an opposing vehicle? (G.A.2)

Although there are many types of medians, in this project, a 4 m of median width has been used, 3 m of which is composed by raised concrete barriers. Considering that ADT is around 60,000 on 1071 Malazgirt Blvd., barrier is recommended up to 10 meters wide medians referencing to Figure 4.1. Having a median with a width of 4 m, the usage of barrier is proper and the potential risk of cross-over crashes seems to be lowered. The existing median barrier can be seen on Figure 6.2.

Figure 6.2. Existing Median Barrier (Km 3+300)

Accident history of area (G.A.3)

There is no data available in this area that can be used to study the accident history.

6.1.2. (G.B) Landscaping

Landscaping along road in accordance with guidelines (G.B.1)

In the previous chapter, it is already mentioned that there are vegetations planted within the clear zone. Additionally, there is a point where vegetation exists adjacent to merging ramp terminal which violates proper landscaping guidelines by limiting the sight views which can distract the drivers of the vehicles on the through lane. This case is visible on Figure 6.3. Plantings placed on median is a subject that will be investigated in detail in the sight distance chapter.

Figure 6.3. Plantings obstructing sight view (Km 0+900)

Required clearances and sight distances restricted due to future plant growth? (G.B.2)

As mentioned before, throughout the whole roadway existing plants already occupy the required clear zone and as these plants will grow, negative effects will be more serious. Moreover, there are additional locations where the plants would have a chance to obstruct sight views of the road users in the future although they are not that obstructive currently one of which can be seen on Figure 6.4;

Figure 6.4. Planting that can obstruct sight view in the future after growth (Km 0+350)

6.1.3. (G.C-E) Temporary Work, Headlight Glare, Accident Reports

Since any temporary work has not been conducted during the study of this thesis, an evaluation cannot be made. Although median barriers reduce the severity of headlight glare, there is not such a device available to measure it. Accident reports that belong to the area are not present to public.

6.2. RSA on Alignment & Cross-Section Issues

6.2.1. (A&C.A) Classification

Since the volume of the roadway system is not known, any comment about the classification appropriateness and the possibility of future congestions cannot be made.

6.2.2. (A&C.B) Design/Posted Speed

Appropriateness of the design speed for horizontal and vertical alignment (A&C.B.1)

There are 4 horizontal curves that take place on the main alignment of the road. The design speed of the road is 70 kph while the posted speed is 82 kph. However, municipality uses 82 km/h as posted speed which will permit up to 90 km/h regarding with the legislation as mentioned before. Yet, Ankara is a city where the temperature of the weather can fall down to -15 C degrees and snow conditions do exist often in winter. Additionally, although this is considered as an 8-lanes highway, it is an urban road where congestion can occur. Considering these facts, referencing to AASHTO, maximum superelevation rate of 4% is assumed to be used in design.

Table 6.1. *Horizontal Curve Design Parameters Summary Table*

	S1			S2		
	Existing Value	Required Values for Design/Posted Speed		Existing Value	Required Values for Design Speed Vd (km/h)	
		70 km/hr	90 km/hr		70 km/hr	90 km/hr
Radius (m)	900	-	-	600	-	-
Se (%)	2.4	2.4 (✔)	3.0(✖)	2.9	2.9 (✔)	3.6 (✖)
Lr (m)	40	38.18 (✔)	55.85 (✖)	48	46.13 (✔)	67 (✖)
Lt (m)	33.33	31.82 (✔)	37.23 (✖)	33.10	31.81 (✔)	37.22 (✖)
	S3			S4		
	Existing Value	Required Values		Existing Value	Required Values	
		70 km/hr	90 km/hr		70 km/hr	90 km/hr
Radius (m)	900	-	-	2250	-	-
Se (%)	2.4	2.4 (✔)	3.0 (✖)	0	0 (✔)	R.C. (✖)
Lr (m)	40	31 (✔)	45 (✖)	0	0 (✔)	25.67 (✖)
Lt (m)	33.33	25.83 (✔)	30 (✔)	0	0 (✔)	25.67 (✖)

Superelevation Rate:

As given above, the radius of the horizontal curves (HC) are 900 meters, 600 meters, 900 meters and 2250 meters in order. Although it can be seen that the radius is totally same for HC1 and HC3 curves, number of lanes involved in each curve differs. The recommended values for superelevation to apply regarding with different design speeds and radii for maximum superelevation rate of 4% are given in *Table 4.6*. For a design speed of 70 km/h, minimum superelevation that must be applied for a curve with 900 meters radius is 2.4% which is same with the given one in the project. For a curve with 600 meters radius, the superelevation rate that must be applied is 2.8%. Again, it fits with the given one in the project which is 2.9% which lies between 3.0 % and 2.8 % for the given curve radius. For a curve with a radius of 2250 meters, the table says that normal crown shall be kept which is valid for the design as well. Hence, it can be said that the design values match with the requirements according to AASHTO Green Book (2011).

Transition Curve Length:

In the project, as given before, there are 4 horizontal curves in total. The last curve, namely HC4, does not include superelevation application since it keeps its normal crown slope because of its high curve radius. HC1 and HC2, again as mentioned before, are same in terms of curve radius, applied superelevation, runoff length and tangent runout. However, number of lanes rotated are different for these two curves which should have resulted different values of superelevation runoff and tangent runout lengths. Hence, these two different curve types will be evaluated below while it is also important to note that width of one traffic lane, design speed and accordingly maximum relative gradient are same for all horizontal curves. Using Eq. 4.1, Eq. 4.2 and *Table 4.7*, runoff lengths are calculated and given in *Table 6.2*.

Table 6.2. *Design Control for Horizontal Curves for Design Speed of 70 km/h*

	Design speed = 70 km/h ; △ = 0.55 % ; w = 3.5 m	
	HC1	**HC2**
n_1	4	4
b_w	$\llbracket 1+0.5(4-1)\rrbracket/4 = 0.625$	$\llbracket 1+0.5(4-1)\rrbracket / 4 = 0.625$
e_d	2.40 %[a] (✔)	2.90 %[a] (✔)
L_r	$=\frac{(3.5*4)*2.4}{0.55}*(0.625)=38.18$ m[b] < 40.00 m[a] (✔)	$=\frac{(3.5*4)*2.9}{0.55}*(0.625)=46.13\ m$[b] < 48.00 m[a] (✔)
L_t	$\frac{2}{2.4}*40=33.33$m[b] < 33.33[a] m (✔)	$\frac{2}{2.9}*48=33.10$m[b] < 33.10[a] m (✔)
SSD_{SB}	$0.278*70*2.5+\frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right)+0.0058\right]}=103.40\ m$	$0.278*70*2.5+\frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.005\right]}=105.12\ m$
SSD_{NB}	$0.278*70*2.5+\frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.04\right]}=$ 111.57 m	$0.278*70*2.5+\frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.0405\right]}=111.68\ m$
HSO_{SB}	$900*\left[1-\cos\left(\frac{28.65*104}{900}\right)\right]=$ 1.50 m[b] < 2.25 m[a] (✔)	$600*\left[1-\cos\left(\frac{28.65*106}{600}\right)\right]=$ 2.34 m[b] > 2.25 m[a] (**X**)
HSO_{NB}	$900*\left[1-\cos\left(\frac{28.65*112}{900}\right)\right]=$ 1.75 m[b] < 3.75 m[a] (✔)	$600*\left[1-\cos\left(\frac{28.65*112}{600}\right)\right]=$ 2.61 m[b] < 3.75 m[a] (✔)
	HC3	**HC4**
n_1	3	4
b_w	$\llbracket 1+0.5(3-1)\rrbracket / 3 = 0.67$	$\llbracket 1+0.5(2-1)\rrbracket / 3 = 0.50$
e_d	2.40 %[a] (✔)	0.00 %[a] (✔)
L_r	$=\frac{(3.5*3)*2.4}{0.55}*(0.67)=30.70$ m[b] < 40.00 m[a] (✔)	No runoff length because of zero superelevation rate
L_t	$\frac{2}{2.4}*40=33.33$m[b] < 33.33 m[a] (✔)	No runout length because of zero superelevation rate
SSD_{SB}	$0.278*70*2.5+\frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right)+0.01\right]}=$ 102.75 m	$0.278*70*2.5+\frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.0707\right]}=118.58$ m
SSD_{NB}	$0.278*70*2.5+\frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.06125\right]}=116.26m$	$0.278*70*2.5+\frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right)+0.015\right]}=102.00$ m
HSO_{SB}	$900*\left[1-\cos\left(\frac{28.65*103}{900}\right)\right]=$ 1.47 m[b] < 2.75 m[a] (✔)	$2250*\left[1-\cos\left(\frac{28.65*119}{2250}\right)\right]=$ 0.79 m[b] < 2.75 m[a] (✔)
HSO_{NB}	$900*\left[1-\cos\left(\frac{28.65*117}{900}\right)\right]=$ 1.90 m[b] < 3.25 m[a] (✔)	$2250*\left[1-\cos\left(\frac{28.65*102}{2250}\right)\right]=$ 0.58 m[b] < 3.25 m[a] (✔)

[a] Application project value
[b] Required value

Horizontal Sight Distance:

For curve HC1:

In the portion where this horizontal curve takes place, the grade is upwards with 4.00 % towards southbound while the first part of it falls onto a vertical curve with a length of 50 meters as can be seen on Figure 5.9, before this vertical curve, horizontal curve lays partly on 0.58 % grade upwards on southbound which makes the critical grade 0.58 % on this direction while it becomes -4.0 % for the opposite direction (northbound). For the traffic that goes to direction of southbound, median barriers are critical from the point of view of stopping sight distance while for the traffic that goes towards northbound, sidewalk side of the curve is subject to horizontal sight distance. Stopping sight distances and horizontal sight distances for both directions will be calculated below with a radius of 900 meters using Eq. 4.4 and Eq. 4.5;

$$SSD = 0.278 * 70 * 2.5 + \frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right) + 0.0058\right]} = 103.40$$

$$\cong 104\ meters\ for\ southbound$$

$$HSO = 900 * \left[1 - \cos\left(\frac{28.65 * 104}{900}\right)\right] = 1.50\ meters\ for\ southbound$$

On southbound, the critical sight distance falls to the median side. As mentioned before, on median, there is a concrete barrier along the roadway. The width of the lane which is closest to median barrier is 3.5 meters while there is a paved offset to the barrier with a width of 0.5 m on the median side. In this case, the distance from median face to the centerline of this lane becomes 2.25 meters (3.5/2+0.5) which is greater than calculated HSO with a value of 1.50 meters. Hence, sight distance is adequate for this direction on this curve.

$$SSD = 0.278 * 70 * 2.5 + \frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right) - 0.04\right]} = 111.57$$

$$\cong 112\ meters\ for\ northbound$$

$$HSO = 900 * \left[1 - \cos\left(\frac{28.65 * 112}{900}\right)\right] = 1.75 \, meters \, for \, northbound$$

On northbound, the sight distance lays on the inner part of the curve where there exists sidewalk. Between the curb of this sidewalk and inner boundary of the most-inner lane, there is a shoulder with a width of 0.5 m as on median side. Also, the width of the sidewalk is 3 meters. However, along the roadway, trees are present in the middle of sidewalk. In this case, the distance from the centerline of inner lane to the inner edge of sidewalk becomes 3.75 meters (3.5/2+0.5+3/2) which is greater than the required HSO with the value of 1.75 meters calculated above. Hence, sight distance is adequate for this direction as well on this curve.

For curve HC2:

Referencing to Figure 5.10 (a) and Figure 5.10 (b), the portion where this horizontal curve takes place, vertical alignment lays partly on a vertical curve which has a length of 300 meters and a downwards grade with 0.50 % on southbound. Hence, critical grade on this direction becomes -0.50 %. However, for northbound, the critical grade falls on the vertical curve. The horizontal curve starts at the 90th meter of 300 m long vertical curve with an incoming grade of 6.00 % and outgoing grade of -0.50 %. In this case, critical grade becomes 4.05 % (90/300*(6+0.5)-6) by calculation of proportional grade change on the vertical curve. On northbound, this grade is downwards. For the traffic that goes southbound, median barriers are critical for stopping sight distance while for the traffic that goes towards northbound, sidewalk side of the curve is important. Stopping sight distances for both directions will be calculated below with a radius of 600 meters;

$$SSD = 0.278 * 70 * 2.5 + \frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right) - 0.005\right]} = 105.12$$

$$\cong 106 \, meters \, for \, southbound$$

$$HSO = 600 * \left[1 - \cos\left(\frac{28.65 * 106}{600}\right)\right] = 2.34 \, meters \, for \, southbound$$

On southbound, the sight distance falls on the median side. In this case, available sight distance is 2.25 meters as in curve HC1 which is less than calculated HSO with a value of 2.34 meters. Hence, sight distance is not adequate for this direction on this curve. However, 9 cm is not an important distance for roadways. Even the irregular shape of the concrete barrier may provide that much sight distance. For this reason, this curve also can be counted as adequate.

$$SSD = 0.278 * 70 * 2.5 + \frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right) - 0.0405\right]} = 111.68$$

$$\cong 112 \; meters\; for\; northbound$$

$$HSO = 600 * \left[1 - \cos\left(\frac{28.65 * 112}{600}\right)\right] = 2.61 \; meters\; for\; northbound$$

Northbound, the sight distance falls on the inner part of the curve where there exists sidewalk as in curve HC1. In this case, the distance from the centerline of inner lane to the inner edge of sidewalk again is 3.75 meters (3.5/2+0.5+3/2) which is greater than the required HSO with the value of 2.61 meters calculated above. Hence, sight distance is adequate for this direction on this curve.

<u>For curve HC3:</u>

Referencing to Figure 5.11 (a) and Figure 5.11 (b), the portion where this horizontal curve takes place, vertical alignment lays partly on a vertical curve which has a length of 150 meters and an upwards grade with 1.00 % on southbound. Hence, critical grade on this direction becomes 1.00 %. However, on northbound, the critical grade falls on the vertical curve. The horizontal curve ends at the last 15th meter of 150 m long vertical curve with an incoming grade of 1.00 % and outgoing grade of 6.75 %. In this case, critical grade becomes 6.175 % (1+135/150*(6.75-1)) by calculation of proportional grade change on the vertical curve. On northbound, this grade is downwards. For the traffic that goes southbound, median barriers are critical for stopping sight distance while for the traffic that goes northbound sidewalk side of the roadway becomes important where there exists the barrier with the parapet of the bridge structure. Stopping sight distances and horizontal sight distances for both directions will be calculated below with a radius of 900 meters;

$$SSD = 0.278 * 70 * 2.5 + \frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right) + 0.01\right]} = 102.75$$

$$\cong 103 \; meters \; for \; southbound$$

$$HSO = 900 * \left[1 - \cos\left(\frac{28.65 * 103}{900}\right)\right] = 1.47 \; meters \; for \; southbound$$

On southbound, the sight distance lays on the median side. On bridge structures, there is a paved offset from the sidewalk curb with a width of 0.50 m and a curbed sidewalk with the same width of offset. After the sidewalk, there is the concrete parapet of the bridge. The width of the lane which is closest to median barrier is 3.5 meters while there is an offset and sidewalk with a width of 1 m in total on the median side. In this case, the distance from parapet structure face to the centerline of this lane becomes 2.75 meters (3.5/2+1) which is greater than calculated HSO with a value of 1.47 meters. Hence, sight distance is adequate for this direction on this curve.

$$SSD = 0.278 * 70 * 2.5 + \frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right) - 0.06125\right]} = 116.26$$

$$\cong 117 \; meters \; for \; northbound$$

$$HSO = 900 * \left[1 - \cos\left(\frac{28.65 * 117}{900}\right)\right] = 1.90 \; meters \; for \; northbound$$

On northbound, the sight distance falls on the outer part of the roadway where there exists a paved offset and a barrier with the parapet of the bridge structure since this portion lays on a bridge. On this bridge, offset with a width of 1.5 m exists. In this case, the distance from the centerline of inner lane to the face of parapet becomes 3.25 meters (3.5/2+1.5) which is greater than the required HSO with the value of 1.91 meters calculated above. Hence, sight distance is adequate for this direction on this curve.

For curve HC4:

Referencing to Figure 5.12 (a) and Figure 5.12 (b) this horizontal curve lays on two different vertical curves which have lengths of 60 and 70 meters respectively. On the segment between these two vertical curves, the grade is 1.50 % downwards on southbound. First vertical curve has an incoming grade of 6.00 % downwards and an outgoing grade of 1.50 % downwards. This grade goes into second vertical curve and leaves with a 7.50 % grade downwards again. Therefore, the critical grade on northbound becomes 1.50 %. However, for southbound, the critical grade becomes the point where the grade changes on second vertical curve. The horizontal curve ends at the last 5th meter of 70 m long vertical curve with an incoming grade of -1.50 % and outgoing grade of -7.50 %. In this case, critical grade becomes -7.07 % (-1.5-65/70*(7.5-1.5)) by calculation of proportional grade change on the vertical curve. For the traffic that goes southbound, median barriers are critical for stopping sight distance while for the traffic that goes northbound, outer side of the roadway becomes important where there lies the shoulder and the barrier with the parapet of the bridge structure. Stopping sight distances for both directions will be calculated below with a radius of 2250 meters;

$$SSD = 0.278 * 70 * 2.5 + \frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right) - 0.0707\right]} = 118.58$$

$$\cong 119\ meters\ for\ southbound$$

$$HSO = 2250 * \left[1 - \cos\left(\frac{28.65 * 119}{2250}\right)\right] = 0.79\ meters\ for\ southbound$$

On southbound, the sight distance lays on the median side. As mentioned before, on median side for bridge structures, there is an offset with a width of 0.5 m and then a curbed sidewalk again with a width of 0.5 m. On the outer edge of this sidewalk, there exists the parapet structure of the bridge. The width of the lane which is closest to median barrier is 3.5 meters as always. In this case, the distance from parapet face to the centerline of this lane becomes 2.75 meters (3.5/2+0.5+0.5) which is greater than calculated HSO with a value of 0.79 meters. Hence, sight distance is adequate for this direction on this curve.

$$SSD = 0.278 * 70 * 2.5 + \frac{70^2}{254\left[\left(\frac{3.4}{9.81}\right) + 0.015\right]} = 102.00$$

$$\cong 102 \; meters\ for\ northbound$$

$$HSO = 2250 * \left[1 - \cos\left(\frac{28.65 * 102}{2250}\right)\right] = 0.58 \; meters\ for\ northbound$$

On northbound, the sight distance lays on the inner part of the curve where there exists shoulder and a barrier on the elevated sidewalk with the parapet of the bridge structure since this portion lays on a bridge. On this bridge, total width of these elements up to parapet is 1.5 m. In this case, the distance from the centerline of inner lane to the face of parapet becomes 3.25 meters (3.5/2+1.5) which is greater than the required HSO with the value of 0.58 meters calculated above. Hence, sight distance is adequate for this direction on this curve.

<u>Vertical Curves:</u>

As mentioned before, there are 12 vertical curves along the roadway in total while 6 of them are considered as sag vertical curve (VSC) and 6 of them as crest vertical curve (VCC). The existing parameters of these vertical curves given in the project are presented in *Table 6.3*.

K parameters calculated using Eq. 4.6 are 588, 54, 46, 46, 13 and 11 respectively for crest vertical curves. Looking at *Table 4.7*, on which minimum K parameters are given for different design speeds, considering the design speed as 70 km/h for this project, the last two crest vertical curves are evaluated as inadequate since minimum value of K parameter is defined as 17 for 70 km/h design speed. Hence, VCC5 and VCC6 do not meet the required parameters for stopping sight distance criteria.

Minimum K value that must be considered is 23 according to *Table 6.3*. Existing sag curves have the length of 14, 25, 66, 26, 13 and 5 respectively. As a result, VSC1, VSC5 and VSC6 have inadequate lengths having K values less than 23 which is the minimum value required. These curves must be lengthened in order to have decent sight distances. Though, these limits are valid when there is no fixed-source lighting but still these limits are considered here in case of any malfunction of lighting. On the other hand, third sag vertical curve with a K value of 66 is subject to drainage control parameter. Additionally, VSC3, having K value larger than 51 means that maximum drainage limit is exceeded as the roadway has curbs on each side. As mentioned before, the low point portion of it is not suitable for water drainage because the grade at that point does not provide a convenient flow for drainage. Hence, the length of this sag vertical curve must be shortened for drainage purposes.

Table 6.3. *Existing Vertical Curves' Parameters*

Curve	**G1 (%)**	**G2 (%)**	**L (m)**	**K[a]**	***K[b]***
Crest Curves					
VCC1	4.00	3.83	100	588.24 (✔)	***17***
VCC2	3.83	2.00	100	54.64 (✔)	
VCC3	6.00	-0.50	300	46.15 (✔)	
VCC4	6.75	2.00	220	46.32 (✔)	
VCC5	2.00	-6.00	110	**13.75 (X)**	
VCC6	-1.50	-7.50	70	**11.67 (X)**	
Sag Curves					
VSC1	0.58	4.00	50	14.62 (X)	***23***
VSC2	2.00	6.00	100	25.00 (✔)	
VSC3	-0.50	1.00	100	**66.67 (X)** *	
VSC4	1.00	6.75	150	26.09 (✔)	
VSC5	-6.00	-1.50	60	**13.33 (X)**	
VSC6	-7.50	2.77	60	**5.84 (X)**	
[a] *Application project value* [b] *Required value *Fails for drainage check*					

Vertical Alignment:

As can be seen on *Table 4.8*, having a rolling topography as mentioned before, recommended maximum grade for an urban arterial with a design speed of 70 km/h is 7 percent. When the profile view of the project is checked, it is seen that all the sections have a grade that is less than 7 percent but only the section between Km:4+042 – 4+090 has a grade value of 7.5 percent which is visible on Figure 6.5. This curve is placed after the interchange bridge crossing Mevlana Blvd. It is obvious that vertical clearance needed for overpassing Mevlana Blvd. had to be diminished by downgrading to the existing road within a very short distance which caused to this high grade. Independent from the reason, this section has a grade that is higher than the maximum recommended limit while all the other sections have adequate value of grade regarding with the given *Table 4.8*.

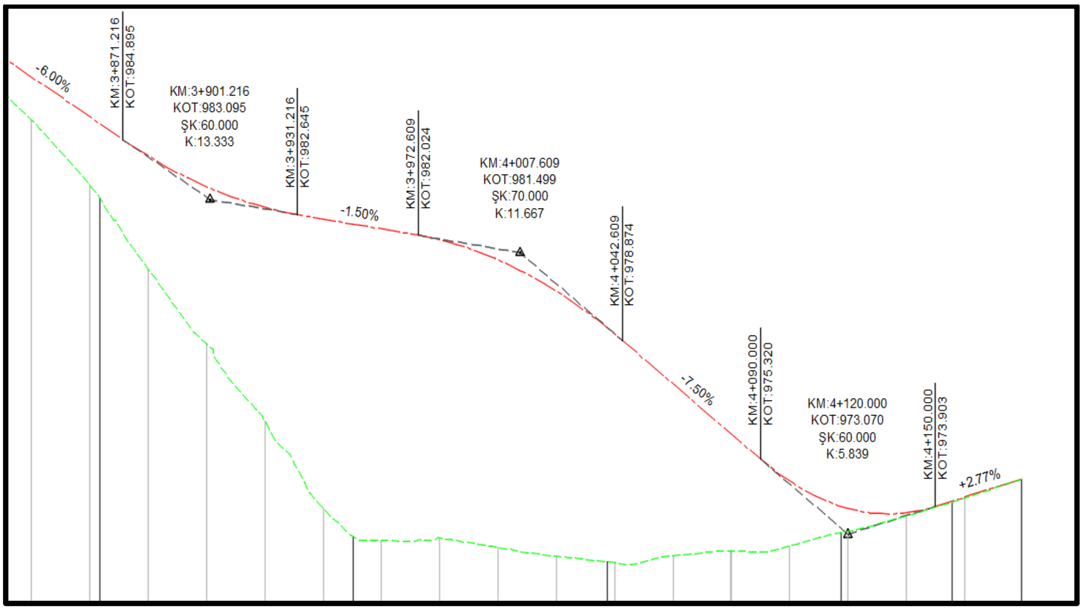


Figure 6.5. Inadequate grade section on vertical alignment between Km:4+042 – 4+090

Check the continuity of the design speed and the posted speed (A&C.B.2)

The continuity of the design speed is seen to be provided when the project is examined. However, some design controls reveal that some of the criteria for the design speed do not meet the requirements. Still, it is visible that from beginning to end of the roadway, the designer kept design speed constant as 70 km/h. Apart from some inadequate geometric features, which must be improved, continuity of the design speed can be said to be preserved. However, the posted speed is much higher than the design speed as mentioned before. Looking at the signings, continuity of posted speed has been conserved which can be seen in Figure 6.6.

(a)

(b)

Figure 6.6. Posted Speed Signings at Km:1+000 (a) and Km:3+400 (b)

Is the posted speed on each curve adequate? (A&C.B.3)

The calculations that have been made for design control of both horizontal and vertical curves for design speed of 70 km/h are made for posted speed of 90 km/h and the summary results of horizontal curves controls are given in *Table 6.4* while vertical curves are given in *Table 6.5*.

Table 6.4. *Design Control for Horizontal Curves for Posted Speed of 90 km/h*

	Design speed = 90 km/h ; △ = 0.47 % ; w = 3.5 m	
	HC1	**HC2**
n_1	4	4
b_w	$[\![1+0.5(4-1)]\!]/4 = 0.625$	$[\![1+0.5(4-1)]\!]\ /\ 4 = 0.625$
e_d	2.40 %[a] < 3.00 %[b] (**X**)	2.90 %[a] < 3.60 %[b] (**X**)
L_r	$= \frac{(3.5*4)*3.0}{0.47} * (0.625) = 55.85$ m[b] > 40.00 m[a] (**X**)	$= \frac{(3.5*4)*3.6}{0.47} * (0.625) = 67.02\ m$[b] > 48.00 m[a] (**X**)
L_t	$\frac{2}{3.0} * 55.85 = 37.23$m[b] > 33.33 m[a] (**X**)	$\frac{2}{3.6} * 67.02 = 37.23$m[b] > 33.10 m[a] (**X**)
SSD_{SB}	$0.278 * 90 * 2.5 + \frac{90^2}{254\left[\left(\frac{3.4}{9.81}\right)+0.0058\right]} = 153.90\ m$	$0.278 * 90 * 2.5 + \frac{90^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.005\right]} = 155.91\ m$
SSD_{NB}	$0.278 * 90 * 2.5 + \frac{90^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.04\right]} =$ 166.57 m	$0.278 * 90 * 2.5 + \frac{90^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.0405\right]} = 166.74\ m$
HSO_{SB}	$900 * \left[1-\cos\left(\frac{28.65*154}{900}\right)\right] = 3.29\ m$[b] > 2.25 m[a] (**X**)	$600 * \left[1-\cos\left(\frac{28.65*156}{600}\right)\right] = 5.06\ m$[b] > 2.25 m[a] (**X**)
HSO_{NB}	$900 * \left[1-\cos\left(\frac{28.65*167}{900}\right)\right] = 3.87\ m$[b] > 3.75 m[a] (**X**)	$600 * \left[1-\cos\left(\frac{28.65*166}{600}\right)\right] = 5.73\ m$[b] > 3.75 m[a] (**X**)
	HC3	**HC4**
n_1	3	4
b_w	$[\![1+0.5(3-1)]\!]\ /\ 3 = 0.67$	$[\![1+0.5(2-1)]\!]\ /\ 3 = 0.55$
e_d	2.40 %[a] < 3.00 %[b] (**X**)	0.00 %[a] < R.C. %[b] (**X**)
L_r	$= \frac{(3.5*3)*3.0}{0.47} * (0.67) = 44.90$ m[b] > 40.00 m[a] (**X**)	$= \frac{(3.5*3)*2.0}{0.47} * (0.55) = 25.67$ m[b] > 0.00 m[a] (**X**)
L_t	$\frac{2}{3.0} * 44.90 = 29.94$ m[b] < 33.33 m[a] (✔)	$\frac{2}{2.0} * 29.93 = 29.93$ m[b] > 0.00 m[a] (**X**)
SSD_{SB}	$0.278 * 90 * 2.5 + \frac{90^2}{254\left[\left(\frac{3.4}{9.81}\right)+0.01\right]} = 151.98\ m$	$0.278 * 90 * 2.5 + \frac{90^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.0707\right]} =$ 178.14 m
SSD_{NB}	$0.278 * 90 * 2.5 + \frac{90^2}{254\left[\left(\frac{3.4}{9.81}\right)-0.06125\right]} = 174.31\ m$	$0.278 * 90 * 2.5 + \frac{90^2}{254\left[\left(\frac{3.4}{9.81}\right)+0.015\right]} =$ 150.74 m
HSO_{SB}	$900 * \left[1-\cos\left(\frac{28.65*152}{900}\right)\right] = 3.21\ m$[b] > 2.75[a] m (**X**)	$2250 * \left[1-\cos\left(\frac{28.65*178}{2250}\right)\right] = 1.76\ m$[b] < 2.75 m[a] (✔)
HSO_{NB}	$900 * \left[1-\cos\left(\frac{28.65*175}{900}\right)\right] = 4.25\ m$[b] > 3.25 m[a] (**X**)	$2250 * \left[1-\cos\left(\frac{28.65*151}{2250}\right)\right] = 1.27\ m$[b] < 3.25 m[a] (✔)
[a] Application project value [b] Required value R.C. (Reverse Crown)		

Table 6.5. *Design Control of Vertical Curves for Posted Speed of 90 km/h*

Curve Name	**G1 (%)**	**G2 (%)**	**L (m)**	**K[a]**	***K*[b]**
Crest Curves					
VCC1	4.00	3.83	100	588.24 (✔)	***39***
VCC2	3.83	2.00	100	54.64 (✔)	
VCC3	6.00	-0.50	300	46.15 (✔)	
VCC4	6.75	2.00	220	46.32 (✔)	
VCC5	2.00	-6.00	110	**13.75 (X)**	
VCC6	-1.50	-7.50	70	**11.67 (X)**	
Sag Curves					
VSC1	0.58	4.00	50	**14.62 (X)**	***38***
VSC2	2.00	6.00	100	**25.00 (X)**	
VSC3*	-0.50	1.00	100	66.67 (✔)	
VSC4	1.00	6.75	150	**26.09 (X)**	
VSC5	-6.00	-1.50	60	**13.33 (X)**	
VSC6	-7.50	2.77	60	**5.84 (X)**	

[a] *Application project value*
[b] *Required value*
**Fails for drainage check*

Is the traffic following the posted speed? (A&C.B.4)

It is not possible to measure the speed of the drivers in order to calculate the 85th percentile speed of the roadway. Hence, there is no data about the free-flow speed.

6.2.3. (A&C.C) Route Selection / Alignment

Since traffic data is not available, it is not possible to evaluate the items related with route selection and alignment.

6.2.4. (A&C.D) Cross Sectional Elements

Determine if the proposed project has a suitable cross section for the ultimate requirements of the road including: Classification, design speed and level of service/peak service volumes (A&C.D.1)

This roadway is classified as an urban arterial that connects major points of urban area with high speed serving for both high-speed vehicles, low-speed vehicles, pedestrians and bicycles. It is connecting to major urban arterials which are Dumlupinar Blvd. and Mevlana Blvd. and ABB has designed these boulevards with a typical cross-section and design speed considering the level of service consistency among them. The same criteria were used for the design of this roadway. For this roadway, design speed is defined as 70 km/h which is within the given range. Yet, ABB has not published any traffic volume data. Because of this reason, there is no donnee to be used for evaluation of traffic volume. The usage of median is not defined also. Yet, in order to be considered as a divided urban arterial, there must be minimum of 1.2 meters wide median which is 4 meters in 1071 Malazgirt Blvd. For urban roads, the median width should provide enough space for left turns but there is no left turn on this roadway. Hence existing median width seems adequate. There are 8 lanes in total which is suitable while the width of the sidewalks is 3 meters that is convenient with the standards. Typical cross-section of the roadway can be seen in Figure 6.7.

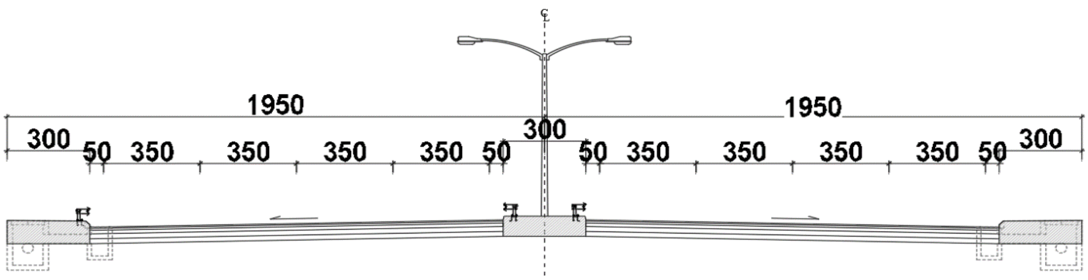


Figure 6.7. Typical Cross Section of the roadway (dimensions in mm)

Determine if adjustments in dimensions can be made for future expansion possibilities. (A*&C.D.2)*

Looking at Figure 4.5, In zone 1, since there are interchanges and intersections, expansion will cost to rebuilding of some structures and interchange legs. After this zone, the roadway lays on an open surface with only an adjacent land and an intersection. Hence, in zone 2, with the condition of reconstruction of this level intersection and occupying some parts of adjacent privately-owned land, expansion is possible. A frontage road and residential buildings are present in zone 3. There is also a bridge structure with certain width of roadway. Although there is a park area which seems free of structures as a green area, expansion in this zone will be problematic because of these surroundings. In zone 4, despite that some of them does not exist currently, near the roadway there are residential zones. Additionally, there is a high earth retaining structure on one side with a bridge crossing the roadway on this structure. In case of expansion, these structures also must be demolished. In the last zone, both sides are wrapped by trade centers and there exist an underpass structure and a bridge. It is for sure that any expansion process would be extra costly in Zone 5.

6.2.5. (A&C.E) Drainage

Is the drainage channel appropriate for topography, maintenance and snow drifting? (A&C.E.1)

There is no channel on the roadway since drainage design is made according to urban roads. The drainage of the surface flow is made by using manholes and infrastructural pipelines.

Is there possibility of surface flooding or overflow from surrounding or intersecting drains and water courses? (A&C.E.2)

When the basins on the related area are observed, apart from one basin, they have extremely small areas. Because of this reason, an external flood is not expected except for this large basin. This basin has an area of 2.6 km^2 which can be considered as a risky flood basin also considering the steep grade. Although there is another street right on the riverbed, since the drainage capacity of this street is not known, depending on the intensity of a rainfall with high frequency, the low point of the roadway may be flooded. At the discharge point of roadway for that basin, there is the bridge structure. However, below this bridge, there is a level intersection with roundabout. When this intersection is observed, it can be said that there is no additional precaution taken against flood risk. The basin area (a) and the intersection which lays on the discharge point of this basin (b) can be seen on Figure 6.8.

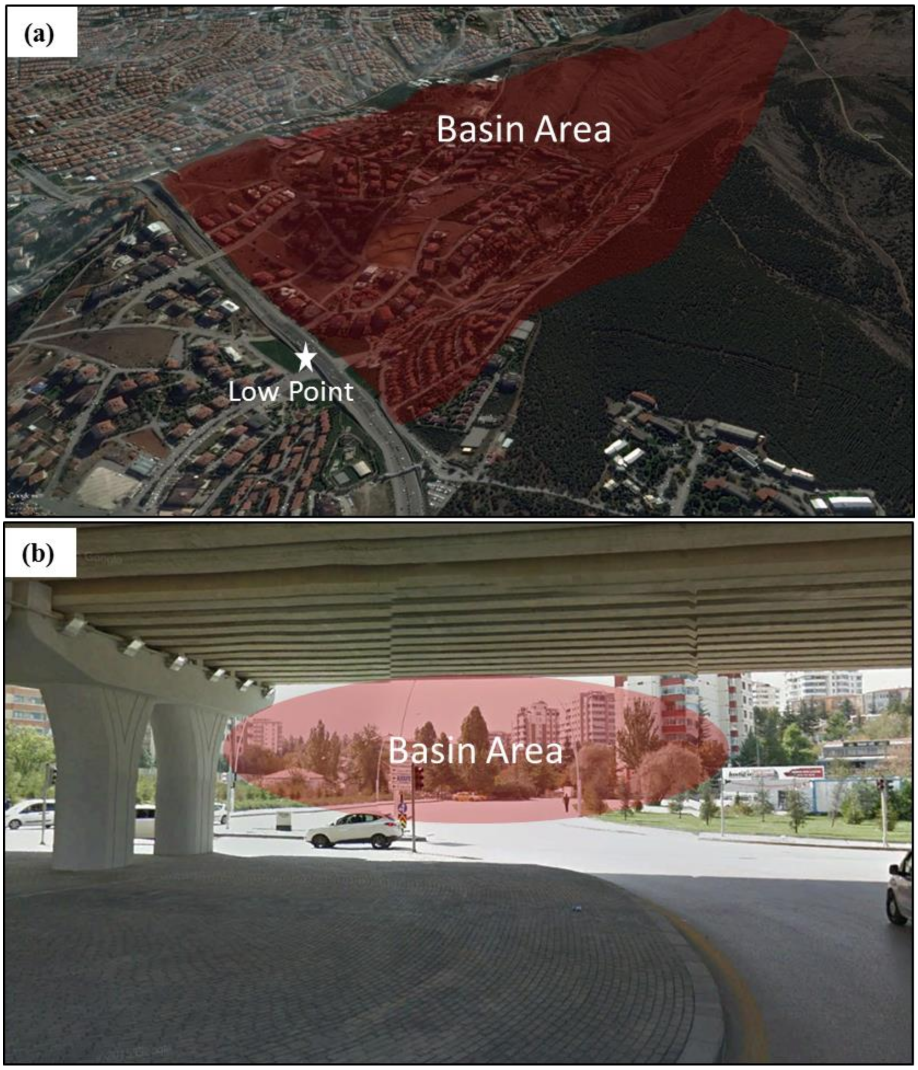


Figure 6.8. Flood-risky basin area (a) and the intersection at discharge point of this basin (b)

Does the proposed roadway have sufficient drainage? (A*&C.E.3)*

Since the stormwater drainage system design is not available to us, it is not possible to control the design of pipe system underground. In order to be able to calculate and compare the required drainage capacity with the existing one, the grade, layout, number and size of the pipes must be available.

6.2.6. (A&C.F) Lane Width

Is the lane width sufficient for road design / classification? (A&C.F.1)

On this roadway, width of the lanes are 3.5 meters. Considering that most of the main arterials in Ankara have 3.5 m of lane width and design manual allows the range between 3.0 and 3.6, it can be said that the lane width of this project is appropriate.

6.2.7. (A&C.G) Cross Slopes /Superelevation

Do crown and cross slope designs provide sufficient storm water drainage and facilitate de-icing treatments? (A&C.(G.1)

The normal crown slope for drainage in this project is 2 % which is adequate according to standards. About de-icing, no extra treatment is seen on the pavement. However, it should be noted hereby that considering the snowy and icy conditions of area, maximums superelevation rate has been designed as 4 %.

Do different rates of cross slope exist along adjacent traffic lanes? (A&C.(G.2)

Cross slopes manage to stay same for all lanes throughout the roadway when it is checked visually. However, by precise measurements, without a running traffic, small differences may be seen which are avoidable.

6.2.8. (A&C.H) Pavement Widening

Is sufficient pavement width provided along curves where off-tracking characteristics of vehicles are expected? (A&C.H.1)

Dimensions of design vehicle WB-19 is retrieved from *Table 4.9*, which are explained below;

For WB-19;

A = 1.22 meters

u = 2.59 meters

Lane width = 3.5 meters

C = 3.5 – 2.59 = 0.91 meters

L = WB_1 = 5.94 meters

WB_2 = 12.50 meters

Summary of the pavement widening calculations and results are given in *Table 6.6*.

Table 6.6. *Pavement Widening Parameters*

	HC1		**HC2**		**HC3**		**HC4**	
R (m)	900		600		900		2250	
N	4		4		3		2	
U	2.70		2.75		2.70		2.63	
FA	0.009		0.013		0.009		0.004	
Design Wc* (m)	14		14		10.5		7	
Design Speed (km/h)	**70**	**90**	**70**	**90**	**70**	**90**	**70**	**90**
Z	0.23	0.30	0.29	0.37	0.23	0.30	0.15	0.19
Required Wc (m)	**14.70 (X)**	**14.77 (X)**	**14.97 (X)**	**15.05 (X)**	11.08 (✔)	**11.15** (X)	7.23 (✔)	7.27 (✔)
* existing value (width of the traveled way)								

Details of calculations for design speed of 70 km/h are given below. For a curve with a radius of 900 m, which are curves HC1 and HC3;

Using Equation 4.7;

$$U = u + R - \sqrt{R^2 - \sum L_i^2} = 2.59 + 900 - \sqrt{900^2 - \sum(5.94^2 + 12.50^2)} =$$

$2.70\ meters$

Using Equation 4.8;

$$F_A = \sqrt{R^2 + A(2L + A)} - R = \sqrt{900^2 + 1.22 * (2 * 5.94 + 1.22)} - 900 =$$

$0.009\ meters$

Using Equation 4.9;

$$Z = 0.1\left(\frac{V}{\sqrt{R}}\right) = 0.1 * \left(\frac{70}{\sqrt{900}}\right) = 0.23\ meters$$

For a curve with a radius of 600 m, which is curve HC2;

Using Equation 4.7;

$$U = u + R - \sqrt{R^2 - \sum L_i^2} = 2.59 + 600 - \sqrt{600^2 - \sum(5.94^2 + 12.50^2)} = 2.75\ meters$$

Using Equation 4.8;

$$F_A = \sqrt{R^2 + A(2L + A)} - R = \sqrt{600^2 + 1.22 * (2 * 5.94 + 1.22)} - 600 = 0.013\ meters$$

Using Equation 4.9;

$$Z = 0.1\left(\frac{V}{\sqrt{R}}\right) = 0.1 * \left(\frac{70}{\sqrt{600}}\right) = 0.29\ meters$$

For a curve with a radius of 2250 m, which is curve HC4;

Using Equation 4.7;

$$U = u + R - \sqrt{R^2 - \sum L_i^2} = 2.59 + 2250 - \sqrt{2250^2 - \sum(5.94^2 + 12.50^2)} = 2.63\ meters$$

Using Equation 4.8;

$$F_A = \sqrt{R^2 + A(2L + A)} - R = \sqrt{2250^2 + 1.22 * (2 * 5.94 + 1.22)} - 2250 = 0.0036\ meters$$

Using Equation 4.9;

$$Z = 0.1\left(\frac{V}{\sqrt{R}}\right) = 0.1 * \left(\frac{70}{\sqrt{2250}}\right) = 0.15\ meters$$

Hence;

For curve HC1;

Using Equation 4.10;

$$W_c = N(U + C) + (N - 1)F_A + Z = 4 * (2.70 + 0.91) + (4 - 1) * 0.009 + 0.23 = 14.70\ meters$$

However, there is no widening on curve HC1 as in all other curves. The width of the traveled way is 3.5*4=14 meters (four lanes with a width of 3.5 meters each). As a conclusion, the widening needed for this curve is around 0.70 meters which has not been applied on the curve.

For curve HC2;

Using Equation 4.10;

$$W_c = N(U + C) + (N - 1)F_A + Z = 4 * (2.75 + 0.91) + (4 - 1) * 0.013 + 0.29 = 14.97\ meters$$

The width of widening should have been around 1.0 meters (14.97-14=0.97 meters). In the roadway, there is no widening also for this curve as in all others. This means the traveled way width is not adequate for curve HC2.

For curve HC3;

Using Equation 4.10;

$$W_c = N(U + C) + (N - 1)F_A + Z = 3 * (2.70 + 0.91) + (3 - 1) * 0.009 + 0.23 = 11.08\ meters$$

In this curve, there are three lanes with a width of 3.5 meters each which makes the total width of the traveled way as 10.5 meters (3*3.5). The width of widening should have been around 2 meters (11.08-10.5=0.58 meters). In AASHTO (2011), it is noted that widening amount which is less than 0.6 meters can be disregarded. Hence, it can be said that curve HC3 provides the requirements with no widening.

For curve HC4;

Using Equation 4.10;

$W_c = N(U + C) + (N - 1)F_A + Z = 2 * (2.63 + 0.91) + (2 - 1) * 0.0036 + 0.15 = 7.23\ meters$

In this curve, there are two lanes with a width of 3.5 meters each which makes the total width of the traveled way as 7 meters (2*3.5). The width of widening should have been around 0.23 meters (7.23-7=0.23 meters). However, as mentioned before AASHTO (2011) says that the widening with an amount less than 0.6 m may be eliminated. Thus, widening of curves against off-tracking is adequate for curve HC4.

6.2.9. (A&C.I) Alignment

Are there excessive curves that cause sliding in adverse weather conditions? (A&C.(I.1)

As stated before, maximum superelevation rate applied for the roadway is 4 %. However, because of high radius of curvature of existing curves, maximum superlevation that is applied is 2.8 %. Considering that normal crown is 2.0 %, the curves with these superelevation rates are not expected to create adverse effects under bad weather conditions.

6.2.10. (A&C.J) Horizontal

Check that a transition curve is required between a tangent and a circular curve. (A&C.J.1)

Referring to *Table 4.10*, the maximum radius of curvature for usage of transition curve is 290 meters for a design speed of 70 km/h while it is 480 meters for design speed of 90 km/h. In that case, stating again that the radii of curves are 900, 600, 900 and 2250 meters for the existing four horizontal curves without transition curves, the usage of transition curves was not necessary for any of them for both design speed and posted speed.

Is the superelevation with transition curves suitable in relation to effects of drainage? (A&C.J.2)

There is not any curve with transition in the project. The drainage condition of the curves without transition has been explained before in the thesis.

6.2.11. (A&C.K) Vertical

Are there excessive grades which could be unsafe in adverse weather conditions? (A&C.K.1)

The limitations for vertical grades have been given in Chapter 4.2. Although the limit is 7 %, there is a section with a grade of 7.5 % which could create crash potential being between intersections. Also, there are some other sections with grades of 6.0 and 6.75 % which could again lead to inadequate circumstances under bad weather conditions being close to intersection zones. Additionally, having a grade which is less than 0.5 % is not desirable because of poor drainage circumstances which is caused by the difficulty of water-flow. However, on this roadway, there is not a section with a grade less than 0.5 %.

Is a climbing lane provided where overtaking and passing maneuvers are limited due to terrain? (A&C.K.2)

There are 4 lanes on each direction of traffic. Being an urban arterial, percentage of heavy vehicles also are not deemed to be high. Although it is not a law that climbing lanes cannot be placed on multilane roadways, it is preferred to be constructed when there is an expectation of such a need in the following years within service life of the road which is mostly about 20 years. On this roadway, there is no need with this traffic composition because of having 4 lanes in each direction though there are steep grades but with small lengths.

Is a climbing lane provided in areas where the design gradient exceeds the critical length of the grade? (A&C.K.3)

As given before, the only segment with a grade more critical than allowed limitations is the last section with a grade of 7.5 %. This grade is climbing towards a two-lane bridge structure. Despite it is sufficient for now, in the future there might be a need for a climbing lane. In this case, the bridge structure must be demolished and a wider will have to be reconstructed again.

Verify that escape lanes are provided where necessary on steep down grades. If not, are escape lanes feasible? (A&C.K.4)

For the escape lanes, there is not a universal guideline specifying the parameters. Engineering judgement must be done by observing the critical downward grade segments. Damaged guardrails, spilled oil, gouged pavement surfaces may reveal that the heavy vehicles are having difficulty while on these steep grades. At the same time, crash data is another fact that will indicate the need of an escape ramp. For this roadway, such a need is not observed.

Is there adequate provision of passing opportunities? (A&C.K.5)

Provision of passing opportunities are mostly question for two-lane and two-way roadways. Having four lanes on each direction, adequate provision for passing opportunities is already maintained for this project.

Is there sufficient spacing between passing zones? (A&C.K.6)

Not a subject of multi-lane roadways as in the preceding item.

6.2.12. (A&C.L) Combined Vertical and Horizontal

Check the interaction of horizontal and vertical alignments in the road (ie., *roller coaster* alignments, sequencing of horizontal/vertical curves, etc.) (A&C.L.1)

As stated before, there are 4 horizontal curves in the project. When these curves are evaluated together with vertical curves, some deficiencies are found out. The first horizontal curve includes a sag vertical curve which is practically desired according to AASHTO (2011). Before the second horizontal curve starts, a sag vertical is placed some part of which coincides with the horizontal curve. The same type of layout also occurs at the exit of horizontal curve. This time, a crest vertical curve partly falls at the end of the horizontal curve and continues after it. In the middle of vertical curves, it is not pleasant to start and end a horizontal curve since it could misdirect the drivers. The third curve also includes a sag vertical curve at the end of it. It is again not desirable since some part of vertical curve overlays the horizontal curve. It might create misleading for the drivers since the horizontal curve looks like a sharp angle. The same condition is valid for the last curve as well. These combinations also may affect required sight distances.

6.2.13. (A&C.M) Sight Distances

Check that there is decision sight distance provided for interchange and intersection signing throughout the project. (A&C.M.1)

Evaluation of this sight distance for intersections and interchanges is done considering the sight distance values depending on design speed given in *Table 4.11*. For a design speed of 70 km/h, required sight distance is 275 m while it is 360 m for a design speed of 90 km/h. The first area that will be examined from the point of view of sight distance is the multilevel interchange that starts by diverging exit ramps. This interchange serves as an exit and entrance to 100. Yil Neighborhood from and to the roadway. The layout is given in Figure 6.9.

The horizontal and vertical geometry in this curve will be evaluated by means of decision sight distance. For both direction of traffic, the signings have been prepared before the start of the ramps. On southbound, the headlight beams on the sag vertical do not reach the object. The object is the signing in this case and its height is assumed to be 4 meters high while the height of the driver's eye is 1.08 meters which can be seen on Figure 6.10.

Figure 6.9. Layout of 100. Yil Neighborhood Interchange

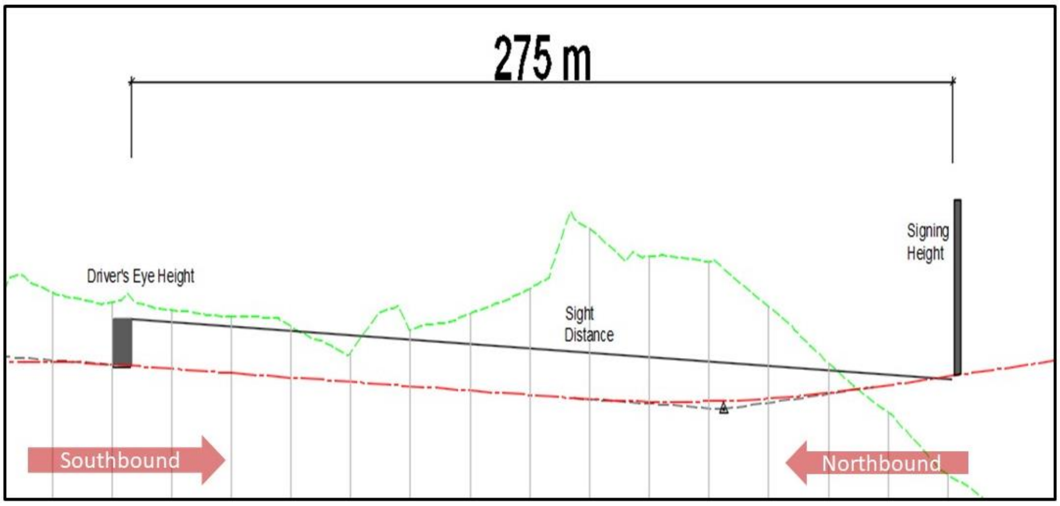
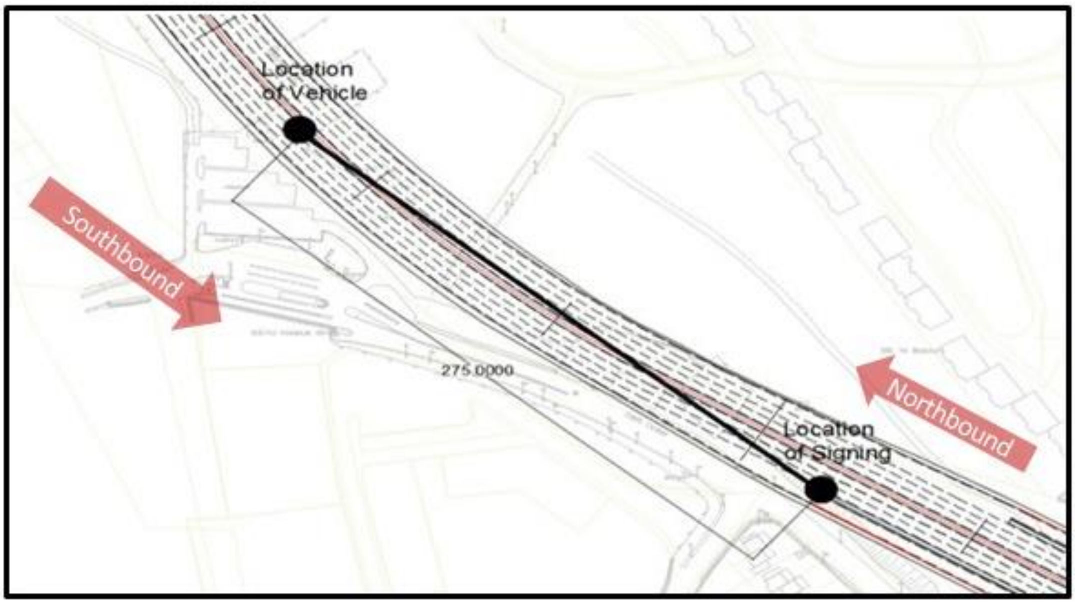
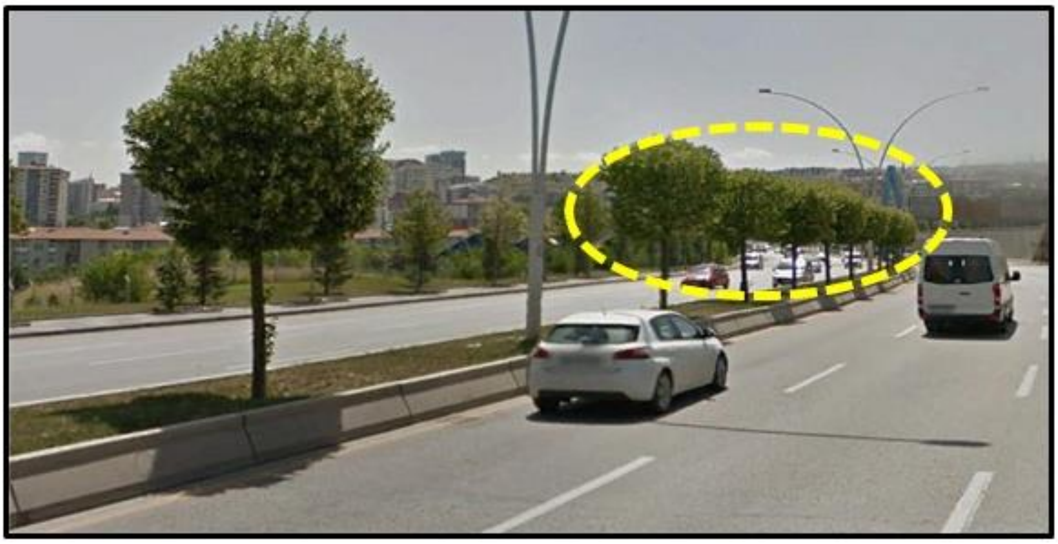


Figure 6.10. Decision Sight Distance for 100. Yil Neighborhood Interchange on Vertical Alignment



(a)



(b)

Figure 6.11. Decision Sight Distance on Horizontal Geometry (a) and Objects Obstruction Sight on Median at Km:2+200 (b)

There is also a horizontal curve within the section of this sight distance. The location of the signing with a required sight distance of 275 meters is sketched on plan view in order to see whether it is provided or not. The sight view in horizontal is obstructed because the objects on median which means horizontally decision sight distance also is not provided for this interchange (Figure 6.11). Hence, with a requirement of 360 meters of sight distance, posted speed also is not considered to be appropriate. However, on Northbound direction, there is not a horizontal or vertical curve that might affect the sight distance at this interchange.

After this interchange, the ramps of the interchange of Mevlana Blvd. takes place. On southbound, the location for the required decision sight distance falls on a crest vertical curve at this interchange. The layout of this interchange is given in Figure 6.12.

Figure 6.12. Layout of Mevlana Blvd. Interchange

Placing the driver's eye height to 1.08 meters again, the sight distance is insufficient because of vertical curve in order to have visibility on the signing object the height of which is assumed as 4 meters as before. Hence, the sight distance is not insufficient also for posted speed which requires 360 meters of DSD. This condition has been evaluated as in Figure 6.13.

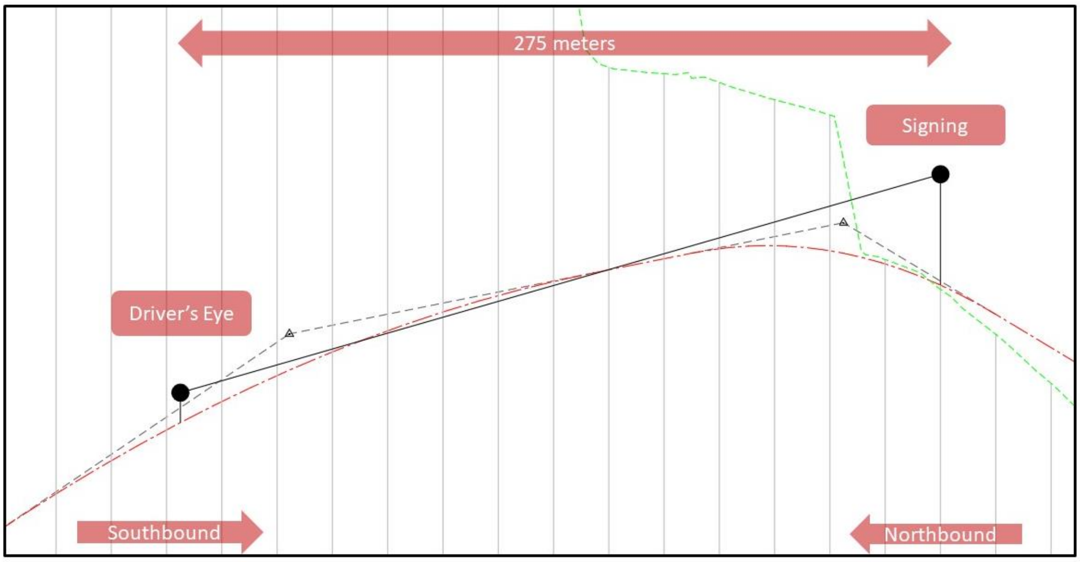


Figure 6.13. Decision Sight Distance for Mevlana Blvd. Interchange on Vertical Alignment

On the opposite direction, so-called northbound, the decision sight distance overlays the end of the project. Hence, no evaluation can be made here. Also, it must be said that, sight distance portions do not involve any horizontal curve, thus, horizontal sight distance will not be controlled for this interchange. On the roadway, there is not any interchange or intersection for which decision sight distance would be checked.

6.2.14. (A&C.N) Readability by Drivers

Check for sections of roadway having potential for confusion (A&C.N.1)

Being a newly constructed road, the markings are mostly visible throughout the roadway. The vegetation and street lamps follow the path of the road. But, on the bridge structure at A1 Gate of METU, which is one way, the side delineations are not properly installed (Figure 6.14) which can, especially at nights, misdirect the drivers. Other than that, it can be said that there is no facility that may confuse the drivers.

Figure 6.14. Absence of Delineation on Bridge Structure and Its Approach Roadway

6.3. RSA on Intersections Issues

6.3.1. (X.A) Quantity

Is the number of intersections appropriate given the surrounding network? (X.A.1)

Since traffic data is not available, it is not able to evaluate whether the number of intersections is appropriate or not.

6.3.2. (X.B) Type

It is not possible to evaluate the appropriateness of the type of the intersection since traffic data is not available.

6.3.3. (X.C) Location / Spacing

Is there sufficient spacing between intersections? (X.C.1)

There is only one intersection that was planned in the project phase on the main roadway which is around Km:1+500. However, this intersection has not been constructed yet completely and does not allow left-turns but only a ramp that connects a minor road (Ogretmenler Avenue) to the boulevard which creates a T intersection. Other intersections are located before the ramps which connect minor roads to main roadway. These intersections are considered as separate intersections since they are independent from 1071 Malazgirt Blvd. Hence, adding the lack of traffic data, it is not possible to examine this issue.

Does horizontal/vertical alignment affect the location/spacing of the intersections? (X.C.2)

As stated in the preceding item, since there is only one intersection, it is not meaningful to evaluate the spacing of intersections.

Junctions and access adequate for all permitted vehicle movements? (X.C.3)

METU A1 Gate Intersection;

The possible critical turning movements are drawn in Figure 6.15 and detailed explanations are given below;

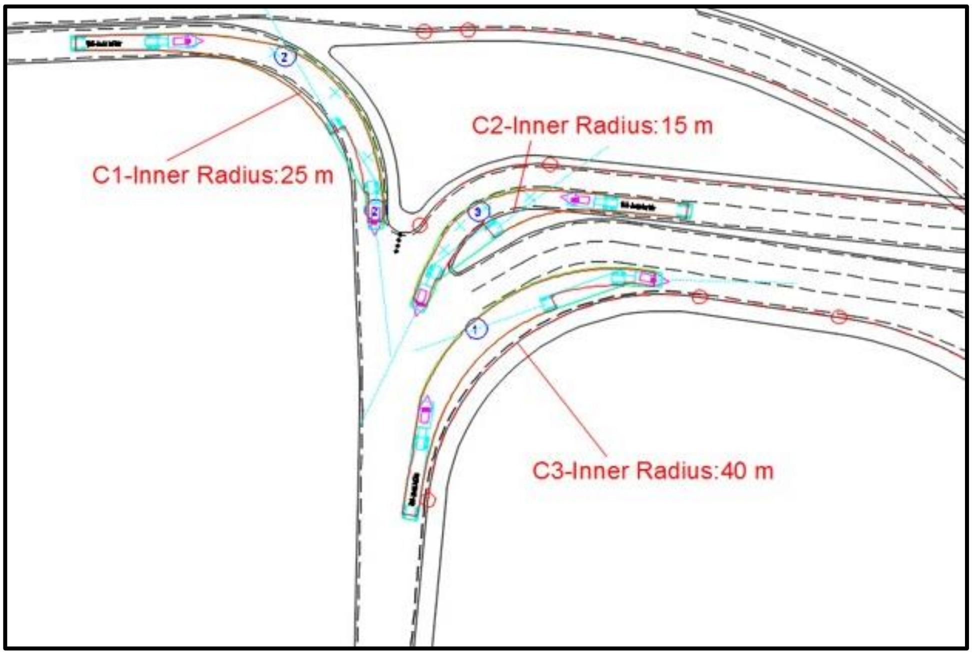


Figure 6.15. Design Vehicle (WB-19) Turning Paths at METU A1 Gate Intersection

Curve 1 (C1): Lane width is designed as 5 meters and adding 0.5 meters of safety lanes, the pavement width becomes 6 meters in total while the radius of the inner curve is 25 meters. Although the radius is greater than the minimum required according to AASHTO (2011), minimum required pavement width which is 8.5 meters is not satisfied. On Figure 6.15, it is visible that turning path of the design vehicle is encroaching upon outside of the road.

Curve 2 (C2) : This leg has 2 lanes arriving at the intersection with 3.5 meters width each. Inner radius of the curve is 15 meters. In this case, minimum pavement width must be 13.5 meters which is much lower with a width of 4 meters (lane width is 3.5 meters and safety offset to sidewalk is 0.5 meters). It can be seen **Error! Reference source not found.** that the path of the vehicle is occupying the inner sidewalk.

Curve 3 (C3) : Inner radius of the curve is 40 meters and the width of the lane is around 8 meters. As seen on the figure, vehicle path fits on the pavement. However, this is valid only for most inner lane. Other lanes have a radius of 22.5 meters with 3.5 meters lane width which does not comply with standards. The path of a vehicle turning on this lane can be seen occupying the adjacent inner lane on Figure 6.15.

Öğretmenler Ave Intersection:

Curve 1 and Curve 2 (C1 and C2) : Both curves have the same layout with 4 meters of pavement width and 20 meters of inner radius. However, although the lane markings are not drawn precisely, pavement width can be considered as around 8 meters since there is a gap between the turning lane and adjacent lane. But the layout of the turning roadway still does not provide the suitability as can be seen on Figure 6.16.

Curve 3 (C3) : The radius of the roundabout is 15 meters. However, the width of the pavement about the circle is around 10 meters. Hence, as it is visible on Figure 6.16, turning path fits on the existing roadway.

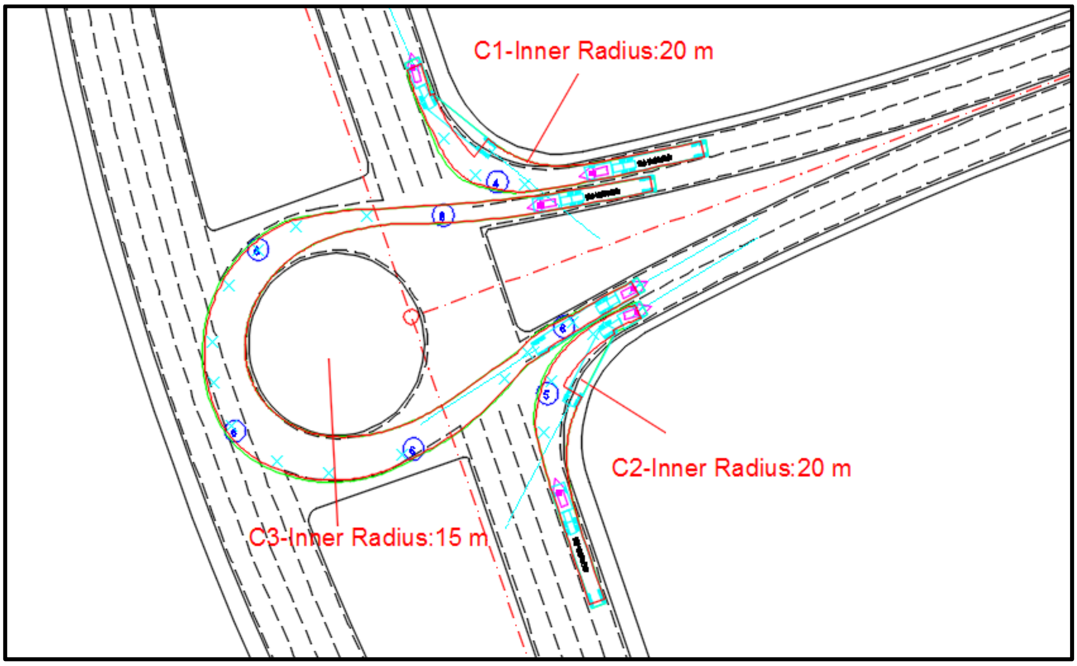


Figure 6.16. Design Vehicle (WB-19) Turning Paths at Öğretmenler Ave Intersection

100. Yil Neighborhood Intersection:

Curve 1, 2, 3, 4 (C1, C2, C3 and C4): This intersection lays under the bridge structure of the Yüzüncüyıl Neighborhood interchange. These curves have the same radius and almost same skewness, hence, possess the same characteristics. Radius of these curves are 20 m and width of each lanes are 3.5 meters excluding 0.5 meters of safety lane. However, legs of the interchange have a width of 6 meters including safety lanes but serving as single lane. Turning paths of vehicle, which have been drawn in a way the turning lanes are most outer ones, can be seen on Figure 6.17. Paths show that the lane width and radii of these curves are not proper in order to let the design vehicle move without off-tracking.

Curve 5 (C5): This curve belongs to the roundabout in the center of the intersection. It has a radius of 15 meters and cannot provide a smooth movement of design vehicle without occupying other lanes or the roundabout itself although it has a pavement width of 12 meters.

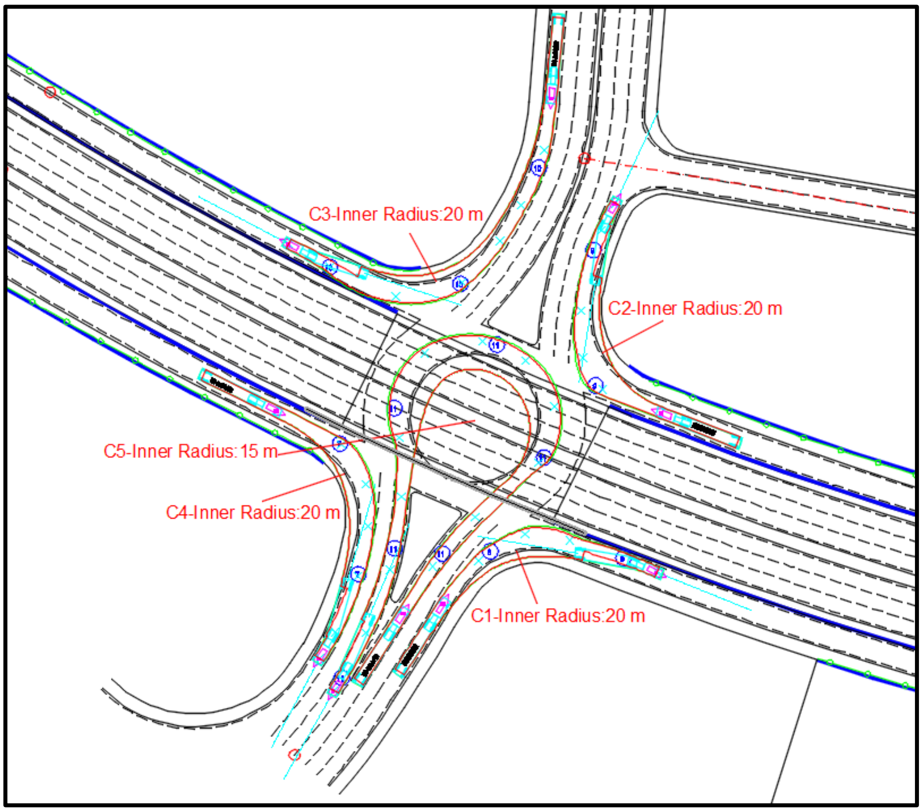


Figure 6.17. Design Vehicle (WB-19) Turning Paths at Yüzüncüyıl Neighborhood Roundabout Intersection

6.3.4. (X.D) Visibility / Conspicuity

Does the horizontal and vertical alignment provide adequate visibility of the intersection? (X.D.1)

On this roadway, although most of the intersections do not contain any visibility problem, a few points can be counted as inappropriate from the point of view of vertical and horizontal geometry effect on conspicuity. At A1 Gate intersection, the leg approaching from 1071 Malazgirt Boulevard to the intersection has a crest vertical curve with a sharp horizontal curve on it. This situation causes worsening of visibility which can be seen on Figure 6.18.

Figure 6.18. Visibility Reduction at A1 Gate Intersection because of Horizontal and Vertical Alignment Layout

Are sight lines to the intersection obstructed? (X.D.2)

On this roadway, sight lines are clear in all intersections for the drivers approaching on the main boulevard. Sight triangles for other connecting roads will be examined in the following chapters.

6.3.5. (X.E) Layout

Are the lane widths adequate for all vehicle classes? (X.E.1)

All the lane widths on the main roadway and the intersection legs are 3.5 meters which is appropriate according to AASHTO (2011) as mentioned before. However, where the minimum required curve radii are not satisfied, lane widths must be modified accordingly. In the previous chapters, this phenomena has been presented in detail. In most cases, lane widths are not sufficient for the assumed design vehicle (WB-19) and existing radii with respect to AASHTO (2011).

Are there any upstream and downstream features which may affect safety? (I.e., "visual clutter", angle parking, high volume driveways) (X.E.2)

Such problems are not observed on the roadway at intersection areas.

6.3.6. (X.F) Maneuvers

Are vehicle maneuvers obvious to all users? (X.F.1)

All the intersections have been investigated on site from the point of view of obviousness of maneuvers and it can be said that maneuvers are all visible to users which is possible to determine when examined physically on site. The vehicles that start to take its movement at the intersection are subject to being seen easily and clearly. Any problem about this issue has not been recognized. These situations are presented on Figure 6.19.

(a)

(b)

Figure 6.19. Maneuvers at A1 Gate Intersection (a) and 100. Yil Neighborhood Intersection (b)

Identify any potential conflicts in movements. (X.F.2)

Possible conflict points exist at some intersections on the roadway. The number of these points must be reduced as much as practical. In this project, when examined, location of these conflict points can be given as follows;

A1 Gate Intersection includes 2 conflict points. One of them occurs at the point where vehicles approaching from Dumlupinar Boulevard and 1071 Malazgirt Boulevard meet each other. Second point is where the drivers will try to change their lanes in order to make their turns whether towards Kizilay (City Center) direction by using U-turn underpass or southbound on the main arterial. These conflict points can be seen on Figure 6.20.

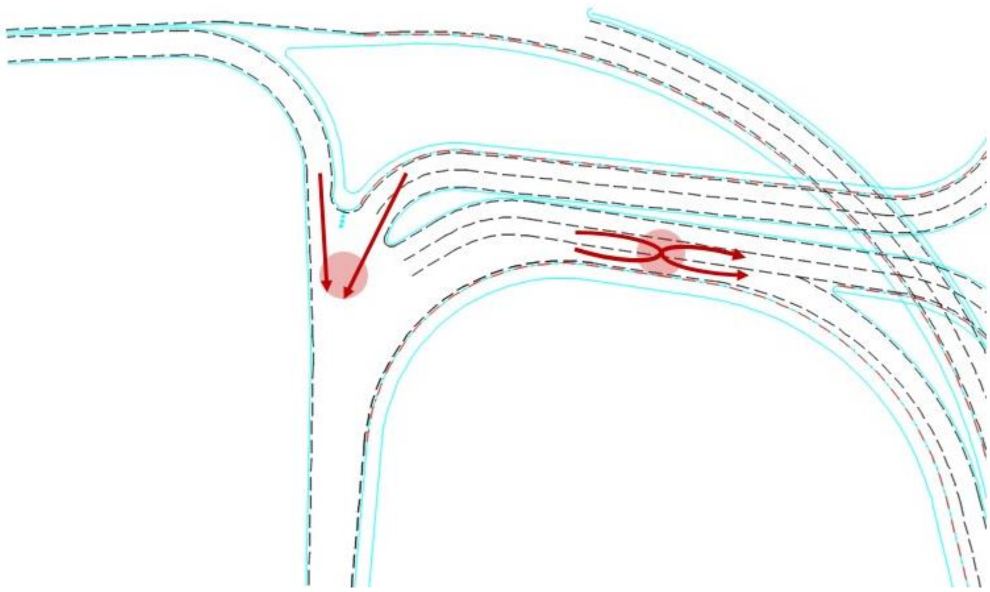


Figure 6.20. Point of Conflict at METU A1 Gate Intersection

6.3.7. (X.G) Auxiliary / Turning Lanes

There are not any auxiliary or turning lanes at intersections on the roadway around intersection areas. Hence, evaluations related with these lanes cannot be made.

6.3.8. (X.H) Sight Distances

Are all sight distances adequate for all movements and road users? (X.H.1)

Stopping sight distance and decision sight distance calculations have been made in the previous chapters. In this chapter, sight triangles will be investigated.

Are sight lines obstructed by signs, bridge abutments, buildings, landscaping, etc.? (X.H.2)

A1 Gate Intersection is the type of traffic control of Case A since there is not any type of control at this intersection. It is a three-leg intersection and the design speed is assumed to be around 30 km/h since it is not possible to know the exact value which was defined by the designer. According to *Table 4.14*, for this kind of intersections, the legs of sight triangle can be considered as 25 meters by assuming the design speed of the approaching roads as 30 km/h. When 25 meters of leg length is drawn on plan layout, the resulting sight triangle becomes as given on Figure 6.21. Looking at physical situation on site within this triangle, it can be said that there is an advertising billboard which could stay inside sight view of drivers. Other than this, there is not any obstruction for this sight triangle.

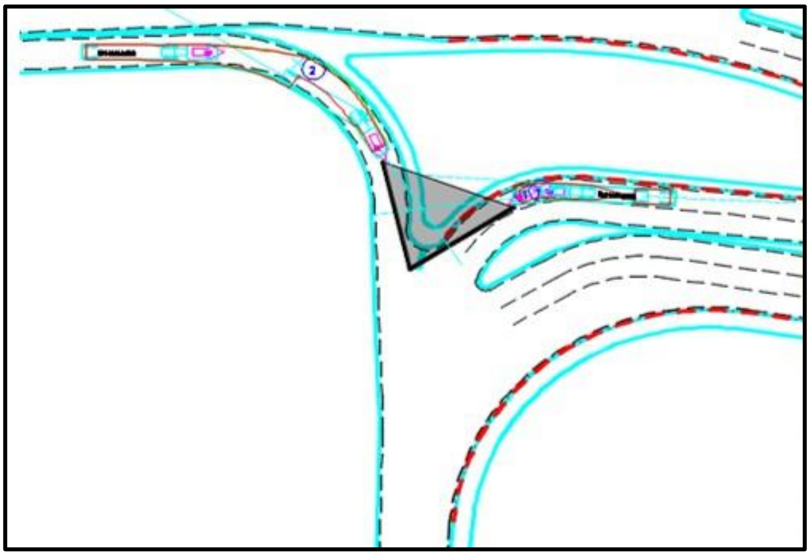
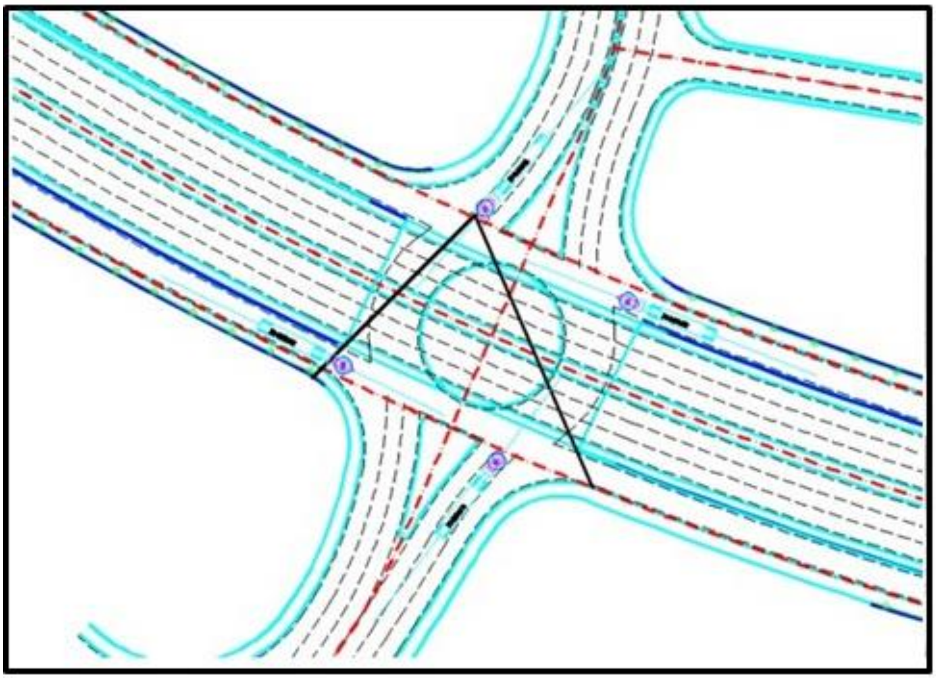


Figure 6.21. Sight Triangle Plan Layout at METU A1 Gate Intersection

The intersection under the bridge of 100. Yil Neighborhood interchange is a type of intersection with traffic signals. When this kind of intersections are subject to consideration of sight distances, the evaluation is as simple as checking whether the first vehicle stopping at an approach is able to see the first vehicles stopping on each other approaching roads or not. Apart from these conditions, there is not an approach or departure sight distance evaluation. Hence, it can be said that signalization may be a suitable solution against crashes that occur because of sight view inadequacy at high-volume intersections. Sight triangles for vehicles waiting at signals are drawn in order to define whether drivers are able to see the first cars waiting on the other approaching roads which can be seen on Figure 6.22 (a) and (b).

The next intersection is the one that exists under the bridge structure crossing Mevlana Boulevard. This is an intersection with traffic signalization control and it does not contain a roundabout. It was described in detail before how to evaluate approach and departure sight triangles at intersections with traffic signal control. The first vehicle that stops on one approach must have a sight view on the first vehicles stopping on each other approaching road. When this criterion is studied at this intersection, it is possible to say that the drivers are able to have sufficient sight. Especially the fact that abutments of the bridge are placed away from the edge of the roadways by adding other two spans in order to have a gap for installation of u-turns leads to increased sight. Photos taken on site are presented on Figure 6.23 in order to visualize the sight views of the vehicles;



(a)

(b)

Figure 6.22. Sight Triangle Plan Layout (a) and View of Driver (b) at Intersection below the Bridge of 100. Yil Neighborhood Interchange

Figure 6.23. Sight Views of Drivers at Mevlana Intersection

Could sight lines be temporarily obstructed by parked vehicles, snow storage, seasonal foliage, etc.? (X.H.3)

In winter times, required clean-up must be done in order to have decent sight views on each road since Ankara experiences tough weather conditions. However, parked vehicles are one of the biggest reasons of blocking sight lines especially at intersections. This situation is valid in each type of intersection and the same tradition takes place also at A1 Gate Intersection. As can be seen on Figure 6.24, cars are parked wherever is possible for parking without considering sight line obstruction by the drivers.

Figure 6.24. Parked Vehicles Obstructing Sight Lines at A1 Gate Intersection

Do grades at intersecting roadways allow desirable sight distance? (X.H.4)

None of the intersections discussed from the point of view of sight view does not have any grades steeper than 3 % on the approaching roadways. Hence, grades of the roadways are considered as not to have any effect on sight distances at intersections.

6.3.9. (X.I) Markings

Are pavement markings clearly visible in day and night time conditions? (X.(I.1)

All the pavement markings are observed to be clearly visible on site at intersection areas.

Check retroreflectivity of markings. (X.(I.2)

Measurement of retroreflectivity requires special equipments. because of not having these equipments, it is not able to evaluate this issue.

6.3.10. (X.J) Signs

Check visibility and readability of signs to approaching users (X.J.1)

It is also important that the signs are visible to the drivers. They must be clearly readable by the approaching users. The situation of the signs has been examined on site and they are considered to be appropriate.

Check for any missing/redundant/broken signs (X.J.2)

There is not a missing, redundant or a broken sign at any intersection.

Are stop/yield signs used where appropriate? (X.J.3)

The stop and yield signs have been used where appropriate.

6.3.11. (X.K) Signals

Have high intensity signals/target boards/shields been provided where sunset and sunrise may be a problem? (X.K.1)

The signals have sufficient intensity at daytimes also by the aid of shields over each of them. An example of the such signals are given on Figure 6.25.

Figure 6.25. Example of Signals with High Visibility

Check location and number of signals. Are signals visible? (X.K.2)

At each intersection, the signals have been located on both sides standing at the intersection beginning. These locations and numbers are adequate for the drivers to be able to see them. Visibility of signals have been shown on Figure 6.25 before.

Are primary and secondary signal heads properly positioned? (X.K.3)

Primary signals have been positioned at their required locations. In addition to primary signals, pole-mounted signals also exist as secondary signal heads which increase the visibility of them by the drivers which can be seen also on Figure 6.25.

6.3.12. (X.L) Signal Phasing

Since the traffic counts are not available, controls regarding with signal phasing cannot be done. However, for major urban arterials, signal phasing is one of the most crucial points from the point of view of RSA.

6.3.13. (X.M) Warnings

Is adequate warning provided for signals not visible from an appropriate sight distance? (I.e., signs, flashing light, etc.) (X.M.1)

All signals have clear visibility.

Are lateral rumble strips required and properly positioned? (X.M.2)

There are not any rumble strips placed on the roadway at intersections. Drivers should not come up with unusable areas such as islands or median or sidewalk widening suddenly without any warning. Rumble strips should be placed at this kind of intersections (AASHTO, 2011). However, such cases do not exist. Hence, there is no need for rumble strips.

Are pavement markings appropriate for the intersection? (X.M.3)

Unused paved areas must be delineated by pavement markings so that the drivers shall be directed through dedicated roadways (AASHTO, 2011). At the intersections on this road, pavement markings follow the border of roadways adjacent to sidewalks. Hence, it can be said that pavement markings are appropriate.

6.4. RSA on Interchanges Issues

6.4.1. (I.A) Location / Spacing

Because of unavailable traffic data, this item cannot be examined.

6.4.2. (I.B) Weaving Lanes

Ensure appropriate length and number of weaving lanes. (I.B.1)

Since the traffic volume is not known, the length and number of weaving lengths cannot be examined.

6.4.3. (I.C) Ramps

Is the design speed appropriate for site limitations, ramp configurations, and vehicle mix? (I.C.1)

There are three interchanges on this boulevard. While Inonu Boulevard can be considered as a cloverleaf type of interchange with three quadrants, 100. Yil Neighborhood Interchange and Mevlana Boulevard Interchange shall be considered as single point urban interchange with overpassing structures and intersections beneath these overpassing bridges.

The leg that connects eastbound traffic on Dumlupinar Blvd. to southbound traffic on 1071 Malazgirt Boulevard which is seen on Figure 6.26 has a radius of curvature of 120 m in horizontal curve and applied superelevation on this curve is 2.7 %. Under these circumstances, design speed is to be calculated as 30 km/h according to *Table 4.16*. Yet, considering *Table 4.17*, lower range design speeds for such a ramp are 40 km/h and 50 km/h when the design speed of the main highway are 70 km/h and 90 km/h respectively. This results in inappropriate design speed although radius of the curve is within the limits when the design speed of the main road is considered to be 70 km/h. However, for this leg, on site, there is not a posted speed sign. This could cause the users to keep their speed as same as they have on Inonu Boulevard that they are coming from and which has a posted speed of 90 km/h and lose control of the vehicle eventually.

(a) (b)

Figure 6.26. Overpassing Bridge Leg on South-west Quadrant of Inonu Boulevard Interchange

The other leg of this interchange, which carries the traffic from northbound to eastbound, was not able to be placed on the south-west quadrant of the interchange because of the presence of A1 Gate of METU Campus. This leg has been placed on 1071 Malazgirt Boulevard as a U-turn underpass structure. This underpass both serves as a U-Turn for the vehicles moving on Malazgirt Boulevard and also one of the legs of cloverleaf interchange which can be seen on Figure 6.27. Inner radius of this horizontal curve is 31.3 meters. However, posted speed for this leg is 30 km/h.

According to *Table 4.16*, with a superelevation rate of 4.0 % which is the maximum allowable on this roadway, design speed also must be 30 km/h. But, considering *Table 4.17*, design speed of this ramp was supposed to be 40 km/h and 50 km/h. As a result, it can be said that the design speed of this ramp shall not be evaluated as appropriate for both design and posted speed of the main roadway.

(a) (b)

Figure 6.27. U-Turn Leg Underpassing Structure Plan View (a) and Entrance Photo *(b)*

Third leg of the interchange that has been constructed newly within the concept of 1071 Malazgirt Boulevard is the leg that carries the traffic coming from southbound on 1071 Malazgirt Boulevard and takes a turn towards city center which is on east side. Radius of the horizontal curve is designed as 50 meters (Figure 6.28-a) with a superelevation rate of 3.7 %. Considering these parameters, design speed is to be calculated as 30 km/h with respect to *Table 4.16*. At the same time, it is observed on site that the posted speed is also 30 km/h. However, as presented before, when the design speeds of the main arterial are 70 km/h and 90 km/h, low range limit for the design speed of the ramps are recommended not to be lower than 40 km/h and 50 km/ respectively. Apart from having a lower design speed than recommended, this leg also has a serious discontinuity problem which can be seen on Figure 6.28 (b). Design of the ramp does not match with the existing ramp constructed. According to the design of this leg, the wall adjacent to the roadway must have been demolished and a proper positioning of the horizontal curve must have been done. However, because of not demolishing the existing wall, the leg has been confined and a discontinuity occurs in the middle of the curve which may mislead drivers.

(a) (b)

Figure 6.28. Plan View of the South-East Leg on Inonu Blvd. Interchange (a) and Discontinuity on the Ramp (b)

Other two interchanges shall be considered as single point urban interchange type. 100. Yil Neighborhood Interchange (Figure 6.29-a) has the ramps which comprise of outer lanes. These lanes are expanded and used as ramps that reach the intersection below the overpassing structure. Mevlana Boulevard (Figure 6.29-b) also uses the same method but there are added one more auxiliary lane on each side. Posted speed for these ramps are observed as to be 50 km/h. Considering that lower range recommendation values are 40 km/h and 50 km/h for the ramps with a main arterial which has design speed of 70 km/h and 90 km/h, posted speed of the ramp is evaluated as inadequate for both design and posted speed of main arterial.

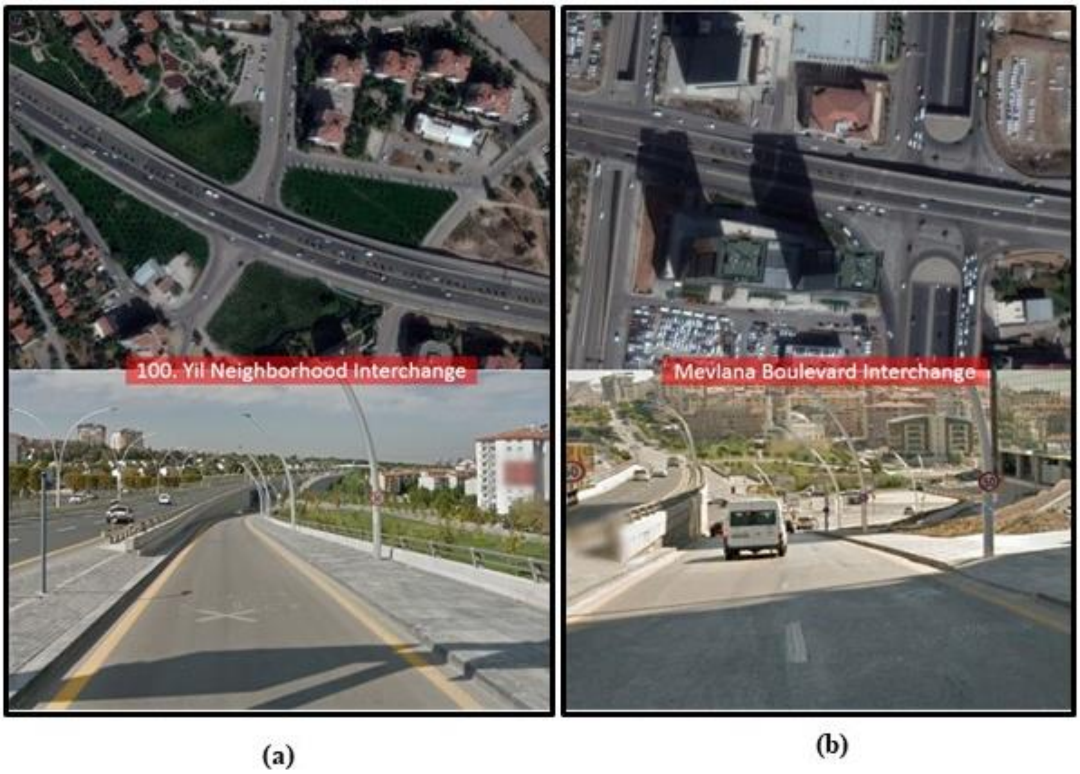


Figure 6.29. Plan View and Photo of Ramps of 100. Yil Neighborhood Interchange (a) and Mevlana Blvd. Interchange (b)

Adequate distance between successive entrance and exit noses? (I.C.2)

It must be noted that, for cloverleaf interchanges, the distances given on *Table 4.18* are not valid. For this kind of interchanges, the spacing between consecutive ramp terminals depend on the curve radii of the loops and roadway and median widths (AASHTO, 2011). Because of this reason, in this chapter, only single point urban type interchanges will be examined. As mentioned before, there are two single point urban interchanges. One of them is 100. Yil Neighborhood Interchange which has a composition of EX-EN which means first ramp terminal is an exit terminal while the next one is an enter terminal. Considering this composition, minimum recommended spacing between these exit and enter ramp terminals is 150 meters which prevails for freeway roads.

Is design of main lane adequate at exit/entrance terminals? (I.C.3)

The horizontal and vertical geometry of the main lane at exit/entrance terminals should be flat with high capacity of visibility. The cross-section of the road should remain same and free from the interchange structures. Because the main route must provide a continuity for the vehicles that will pass through and a comfortable space for lane changes for the vehicles that try to change their directions. Although there is not a certain numerical limitation on the grade of the roads or the curve radii for horizontal curves, a general evaluation can be made on the two interchanges of 1071 Malazgirt Boulevard.

As can be seen on Figure 6.30, horizontal geometry at the ramp terminals of interchanges can be considered as flat as desired. However, when the vertical profiles of the main road at the relative sections are considered, high grades such as 6.75% and 7.50% are seen to exist. Such high grades cannot be counted as appropriate.

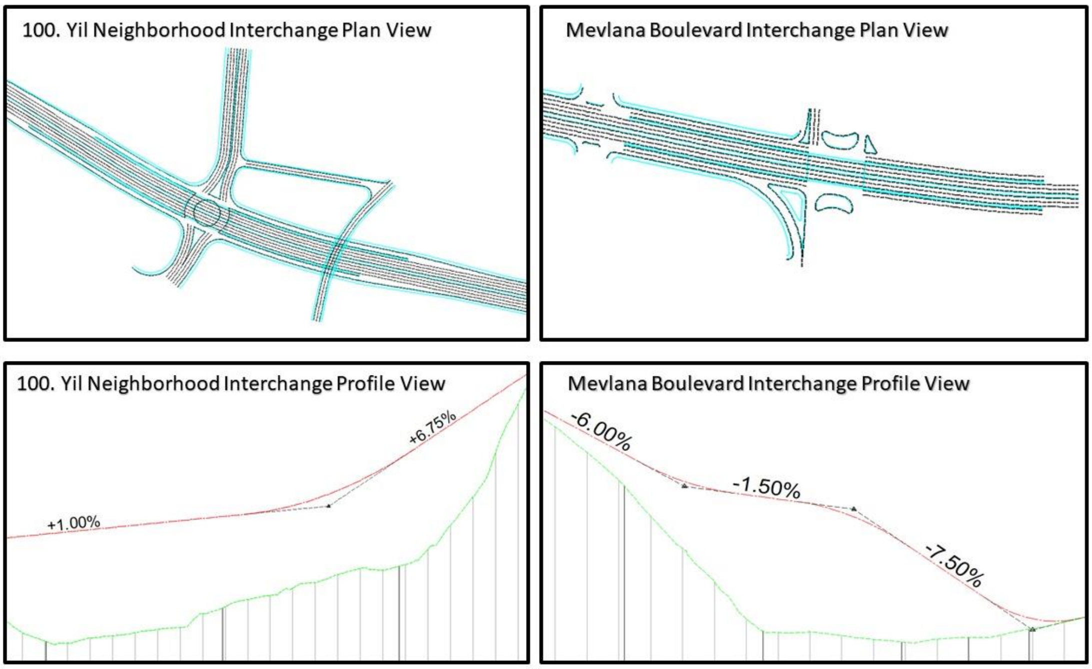


Figure 6.30. 100. Yil Neighborhood and Mevlana Boulevard Interchanges Plan and Profile Views

6.4.4. (I.D) Exit Terminals

Is the length adequate for deceleration? (I.D.1)

In this project, there is only one exit terminal interchange ramp with deceleration lane.

Other terminals either have auxiliary lane or drop one lane to exit ramp. The only deceleration lane is designed as two lanes for the exit terminal that exists on İnönü Boulevard for the vehicles coming from west side and heading to south by taking the ramp which has a radius of 120 meters with a superelevation rate of 2.70%. When the current situation is evaluated, it is seen that the taper length is around 30 meters while the length of the deceleration length of auxiliary lane is around 10 meters that can be observed on Figure 6.31. Compared with values 90 meters and 450 meters respectively, considering Figure 4.9, these values are extremely low compared with the recommended values that are given by AASHTO (2011). Additionally, the radius of the curve on the ramp is 120 meters while the minimum value for this curve is recommended as 300 meters which is given again on Figure 4.9. Hence, it is evident that the design of the deceleration lane at this exit terminal is not adequate.

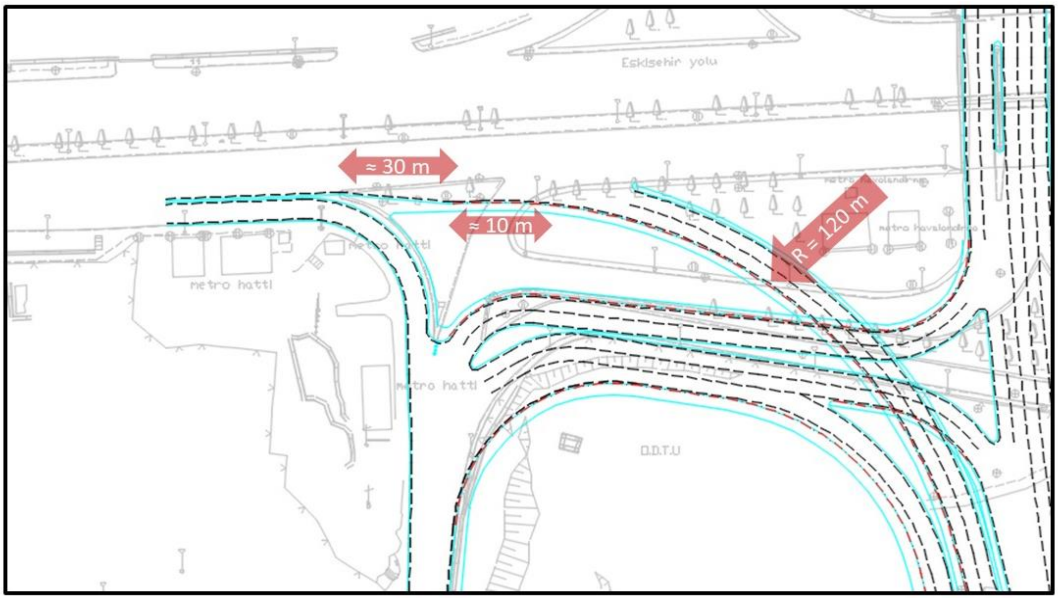


Figure 6.31. Taper and Deceleration Lengths And Radius Of Curvature At The Exit Terminal

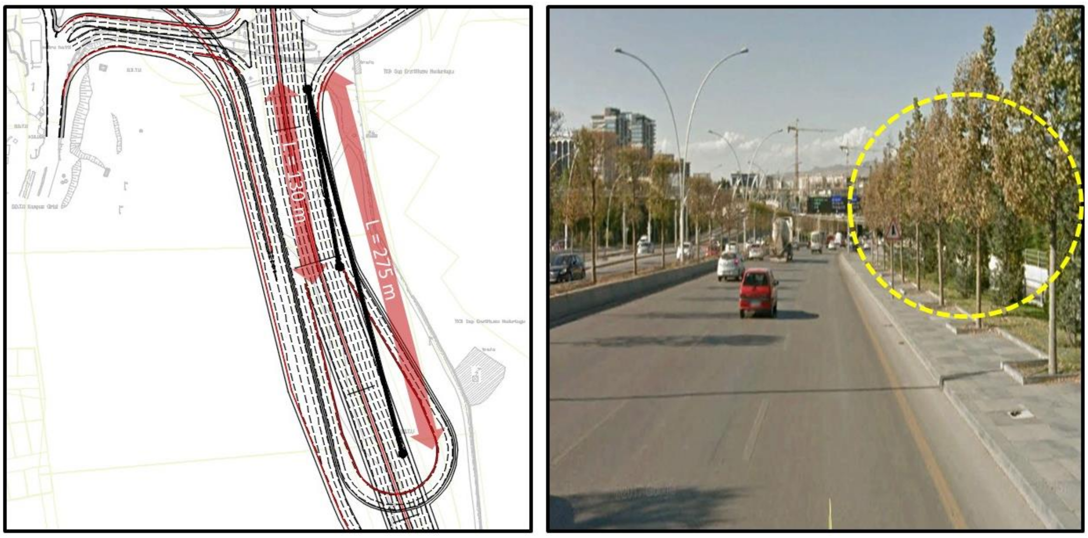
Is adequate sight and decision sight distance provided? (I.D.2)

According to the *Table 4.19*, the vehicles that are travelling at speed of 70 km/h and 90 km/h are supposed to have stopping sight distances of 105 and 160 meters respectively. However, as stated before, for sight distance at approach of exit noses, this value should be increased %25 more which can be rounded to 130 and 200 meters. Additionally, where practical, it is desirable to provide decision sight distance which are 275 and 360 meters for "avoidance maneuver E" type which is valid for urban roads. There are 7 exit terminals that will be examined on 1071 Malazgirt Boulevard. These locations can be named as; METU Interchange (Direction A & B & C), U-Turn Interchange (Direction A), 100. Yil Neighborhood Interchange (Direction A & B), and Mevlana Boulevard Interchange (Direction B). The evaluation from the point of view of sight and decision sight distances of these exit terminals are given below;

METU Interchange:

Figure 6.32. Layout of Exit Terminals at METU Interchange

The vehicles travelling on 1071 Malazgirt Boulevard from southbound and exiting from U-Turn structure (Direction A & B, respectively) approach to the exit nose of the ramp that diverges to city center direction at METU Interchange. For both directions A & B, while stopping sight distances are satisfied on horizontal geometry, decision sight distances cannot be provided hence there are obstructions on the sights which can be seen in Figure 6.33 and Figure 6.34 for a design speed of 70 km/h. However, on Direction A, when SSD and DSD are evaluated considering posted speed which corresponds to 90 km/h, while SSD is provided, DSD is obstructed which can be seen on Figure 6.35. SSD and DSD for posted speed are both inappropriate on Direction B. On the other hand, vertical geometry is not evaluated since there is not an existing vertical curve on the segment subject to sight distance.



(a) **(b)**

Figure 6.33. SSD and DSD on Direction A at METU Interchange (a) and Plantation Obstructing DSD (b) for a design speed of 70 km/h

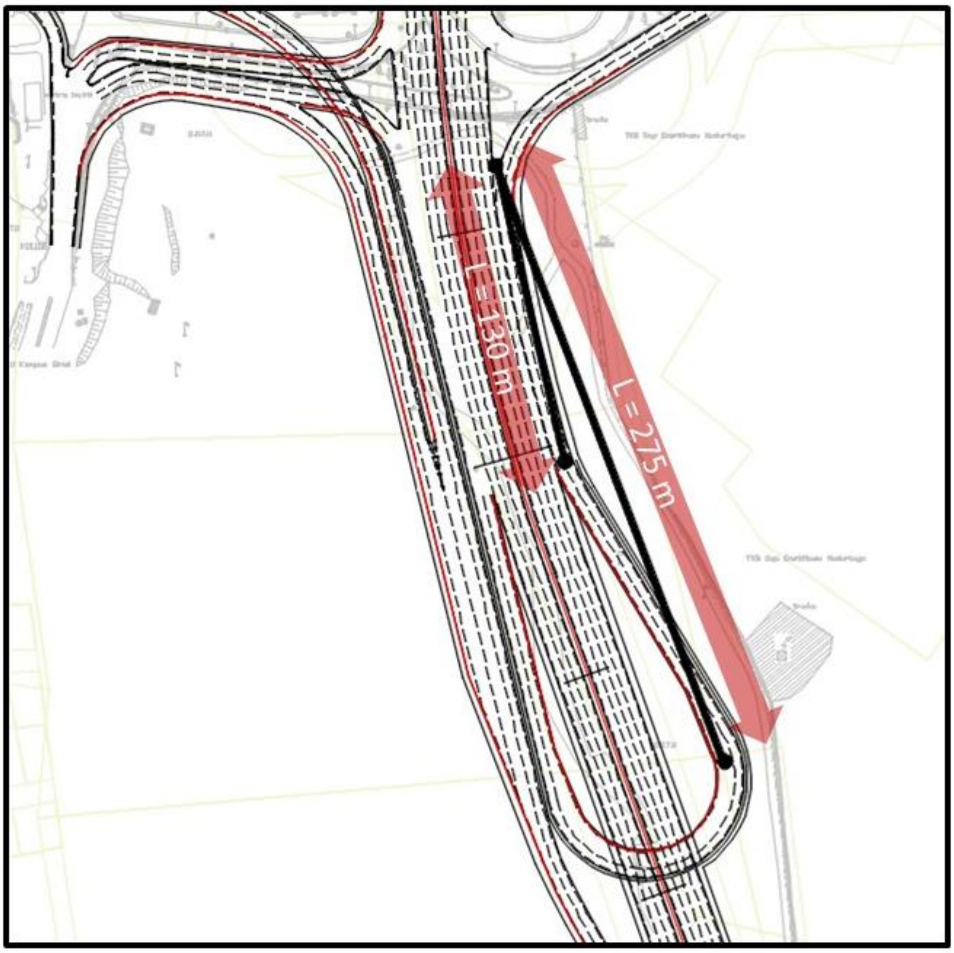


Figure 6.34. SSD and DSD on Direction B at METU Interchange for design speed of 70 km/h

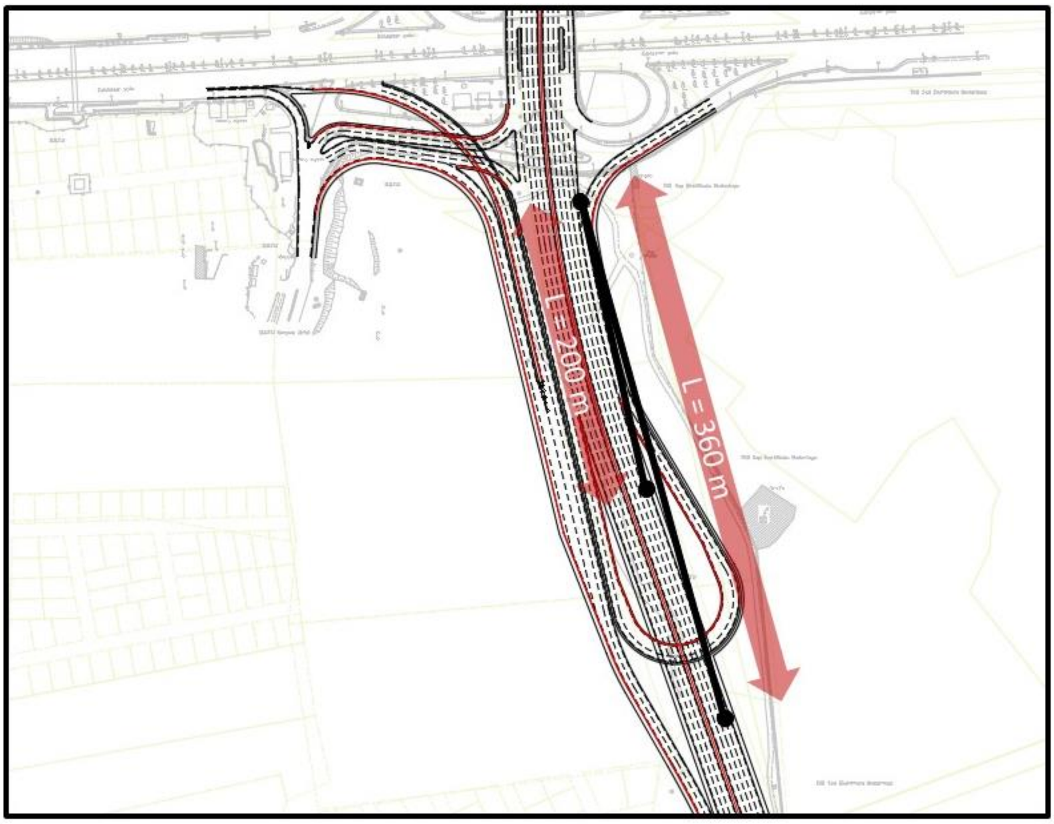


Figure 6.35. SSD and DSD on Direction A at METU Interchange for posted speed 90 km/h

Additionally, the vehicles approaching to METU Interchange through Inonu Boulevard from west (Direction C) direction are subject to make a decision considering the exit noses of both the exit terminal heading to METU Campus A1 Gate and exit terminal ramp of the interchange heading towards 1071 Malazgirt Boulevard. In this section, horizontal geometry and vertical geometry are both flat which leads to the result that the required stopping sight distance and decision sight distance are both satisfied.

U-Turn Interchange:

At U-Turn interchange, the vehicles approaching exit nose with design speed from north (Direction A) have the appropriate stopping sight distance while the decision sight distance, especially of the motorists which are travelling on the most left lane, is obstructed by the median because of the high vegetation placed on it and the left-sided horizontal curve which can be seen on plan view in Figure 6.36. Similarly, for posted speed, SSD is still appropriate but DSD is not which can be seen on Figure 6.37. The vertical geometry is not needed to be checked since the section does not lie on a crest vertical curve.

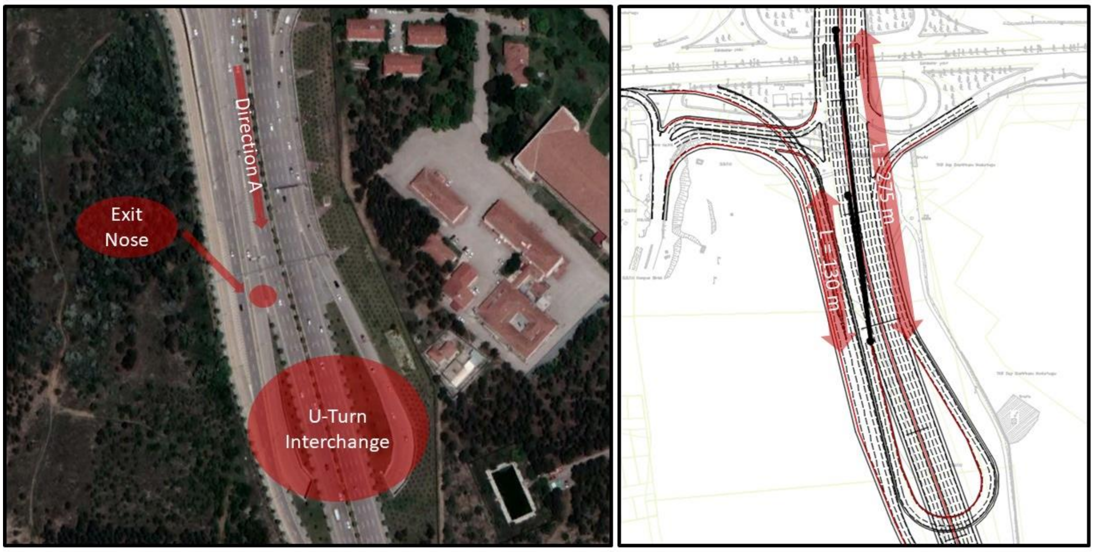


Figure 6.36. Location of Exit Noses and SSD and DSD on Direction A at U-Turn Interchange for design speed

Figure 6.37. Location of Exit Noses and SSD and DSD on Direction A at U-Turn Interchange for posted speed

100. Yil Neighborhood Interchange:

There are 2 exit terminals at this interchange which are on opposing directions that can be seen on Figure 6.38. For direction A, considering design speed, stopping sight distance is appropriate while decision sight distance is not satisfied because of the vegetation on median and the existence of pier of the pedestrian bridge which is located right before the exit nose SSD and DSD and the obstructing items can be seen on Figure 6.39 (a&b). For direction B, both sight distances are adequate since the horizontal geometry is flat at the section before exit nose which can be seen on Figure 6.40. When the posted speed is considered, again, SSD is appropriate while DSD is not adequate for Direction A which can be seen on Vertical geometries on both directions are not evaluated since there does not exist at the segments subject to sight distance any crest vertical curve.

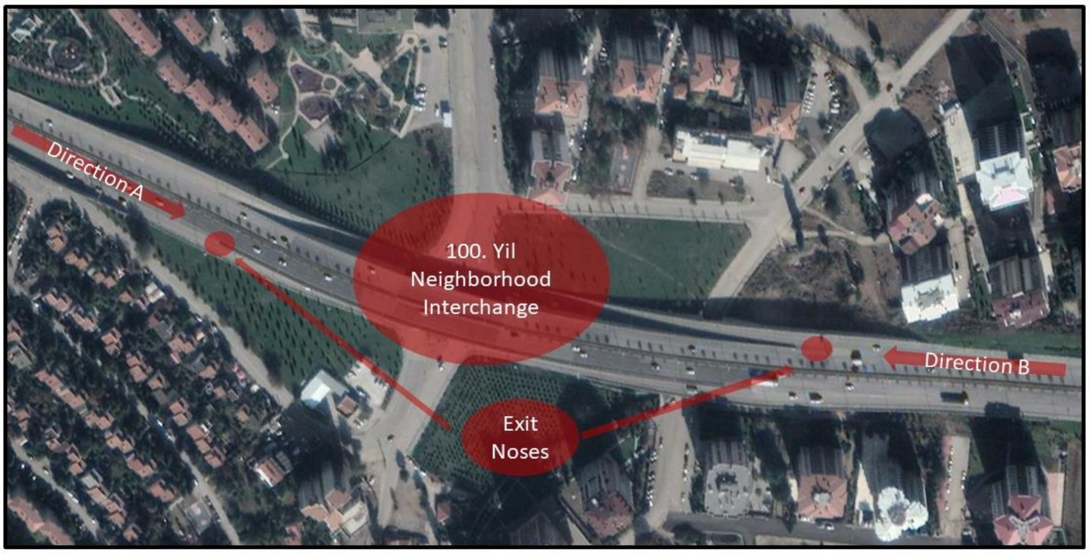
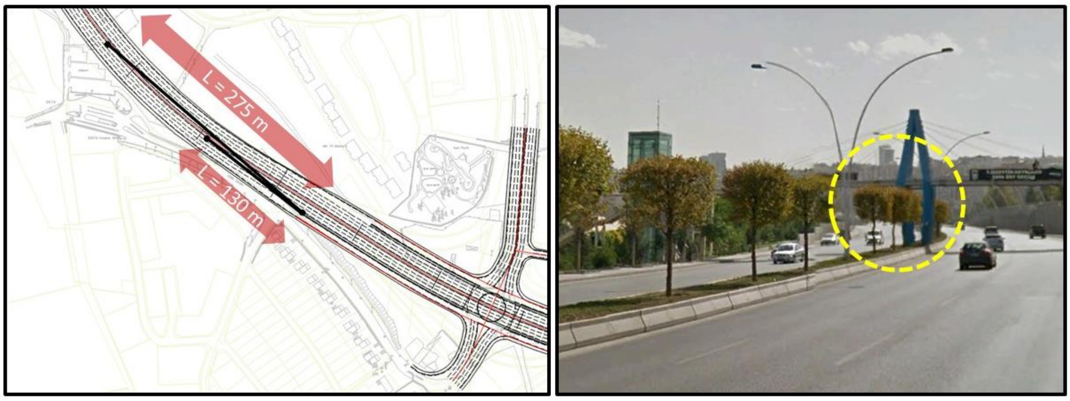


Figure 6.38. Layout of Exit Terminals at 100. Yil Neighborhood Interchange



(a) **(b)**

Figure 6.39. SSD and DSD on Direction A (a) and Obstructing Items on Direction A at 100. Yil Neighborhood Interchange (b) for design speed

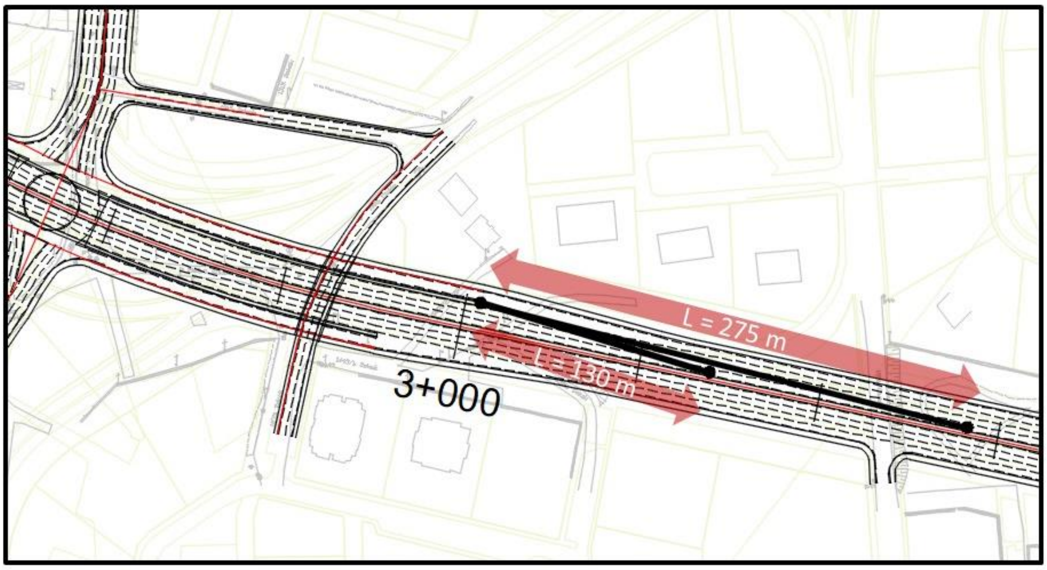


Figure 6.40. SSD and DSD on Direction B at 100. Yil Neighborhood Interchange for design speed

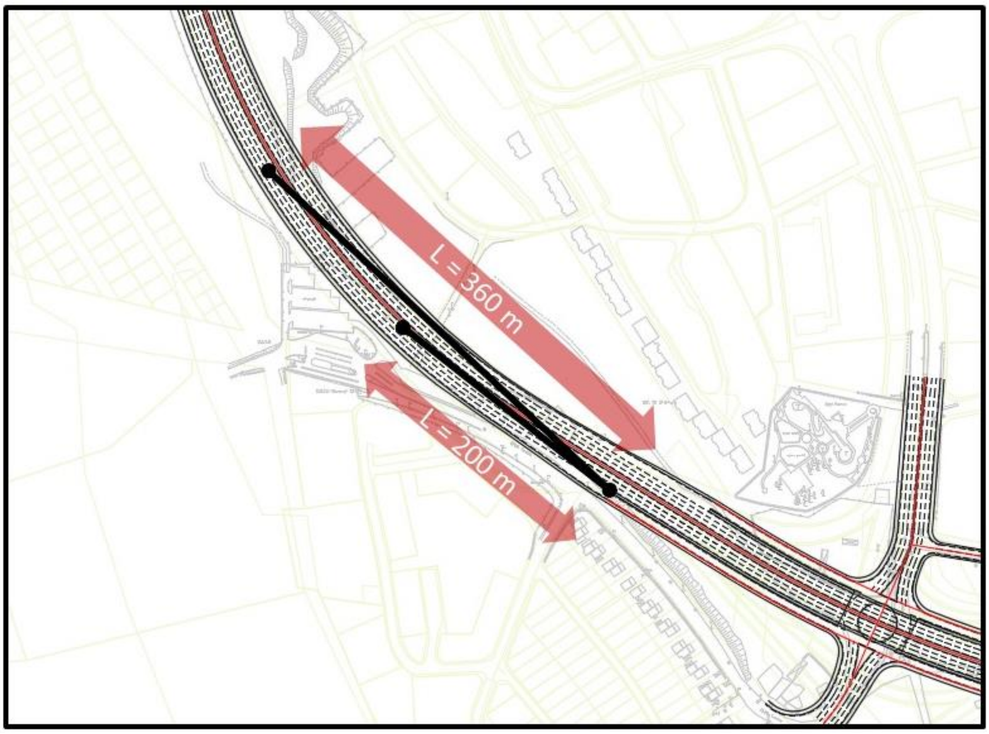


Figure 6.41. SSD and DSD on Direction A for posted speed

Mevlana Boulevard Interchange:

Only west side of this interchange will be evaluated from the point of view of sight distance at exit terminals since 1071 Malazgirt Boulevard ends at this interchange. Layout of the interchange and the exit nose is given on Figure 6.42. The vehicles approaching to the exit nose from direction A have a clear view of the exit nose when the horizontal geometry considered for both design and posted speed since the segment before the location of the exit terminal lies on a straight geometry horizontally.

Figure 6.42. Layout of Exit nose at Mevlana Interchange

However, when the vertical geometry is examined, it is seen that there are successive crest vertical curves at the segment subject to sight distances. Distinctly, the height of the object is not considered to be 0.60 m as in sight distance calculations in the previous chapters, it is recommended that the point to be seen by the driver's eye height, which is 1.08 m as in previous calculations, should be the pavement at the location of the exit nose. Accordingly, from the height of the driver's eye, tangent lines are drawn to the elevation of the top of the pavement and it is clearly seen that both of the sight distances are not satisfied at this exit nose (Figure 6.43 (a)). Drivers do not have a clear view of the exit nose from the points that are given for required stopping sight distance and decision sight distance which can be observed on Figure 6.43 (b). However, in previous chapters, sight distance is examined for the head signings and it was stated that the signings had a clear view in order to see them. As a result, although the drivers can see the signing for the exit terminal, they are not able to see the exit nose on time. Naturally, since required DSD and SSD values are greater for posted speed than the ones which are for design speed, both sight distances are inadequate for posted speed as well.

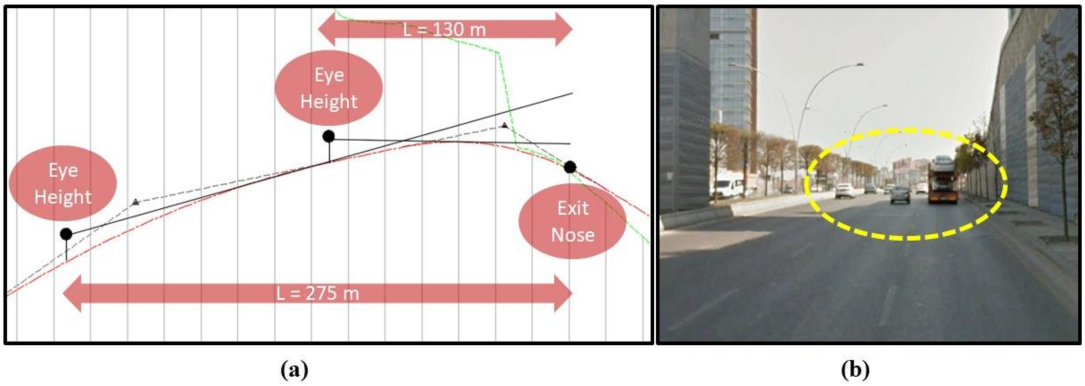


Figure 6.43. DSD on Vertical Profile for the Exit Nose (a) and Unsufficient Sight View of Exit Nose at Direction A at Mavlana Interchange

Are spiral curves warranted? If so, do spirals begin and end at appropriate locations? (I.D.3)

There are two exit terminal interchange ramps on which it can be checked that whether spiral curves are warranted or not. While both of these ramps belong to METU Interchange, one of them is the exit ramp for the vehicles travelling on Inonu Boulevard from west side and heading to southbound towards 1071 Malazgirt Boulevard and the other is the ramp that directs the traffic coming through Malazgirt boulevard towards city center which can be seen on Figure 6.44. Design speed for both exit terminals is 30 km/h as mentioned before and the recommended maximum curve radius value that could include spiral curve for this speed by AASHTO (2011) is 54 meters which have been presented in *Table 4.10*. Having curve radii of 120 m and 50 m respectively, it can be said that the ramp with the curve which has a radius lower than 54 meters should include a spiral curve while it is not warranted for the other ramp.

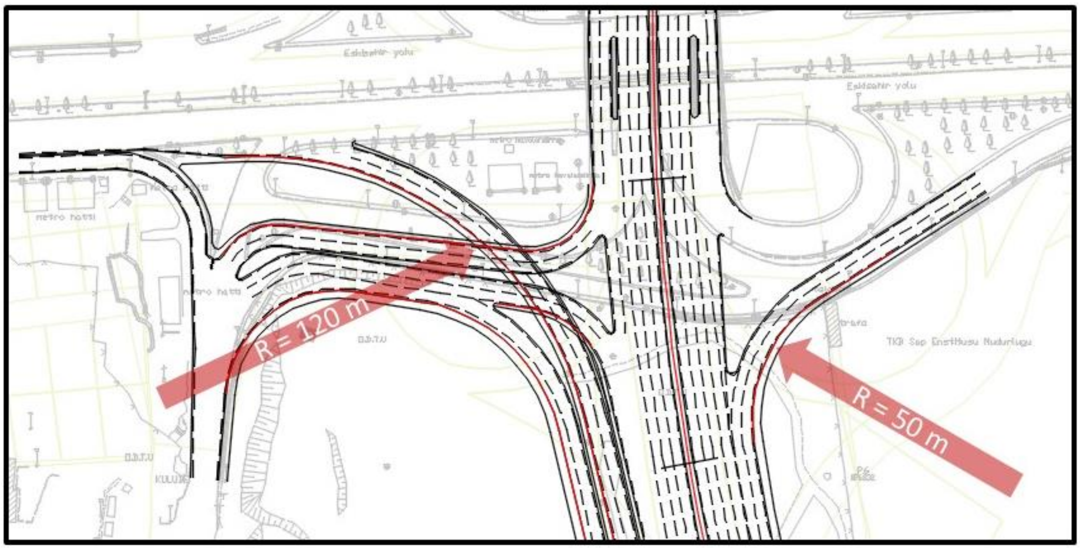


Figure 6.44. Exit Terminal Ramps Subject to Check of Spiral Curve Warrant at METU Interchange

6.4.5. (I.E) Entrance Terminals

Is the length appropriate for acceleration and safe and convenient merging with through traffic? (I.E.1)

There are two entrance terminal interchange ramps on 1071 Malazgirt Boulevard project. The first entrance ramp is located at the merging point of traffic coming from METU Campus A1 Gate towards south and the traffic exiting from Inonu Boulevard at METU Interchange towards 1071 Malazgirt Boulevard that can be seen on Figure 6.45. *Length of Entrance Terminal at Merge Point of METU A1 Ramp and South-West Quadrant Ramp of METU Interchange*. At this point, the entrance ramp is single lane and tapered. The design speed of entering ramp is 30 km/h and the roadway the ramps is merging to has a design speed of 50 km/h. According to *Table 4.21*, required length for acceleration lane is 30 meters which is around 60 meters in the project of 1071 Malazgirt Boulevard. Hence, this entrance terminal is considered to be appropriate.

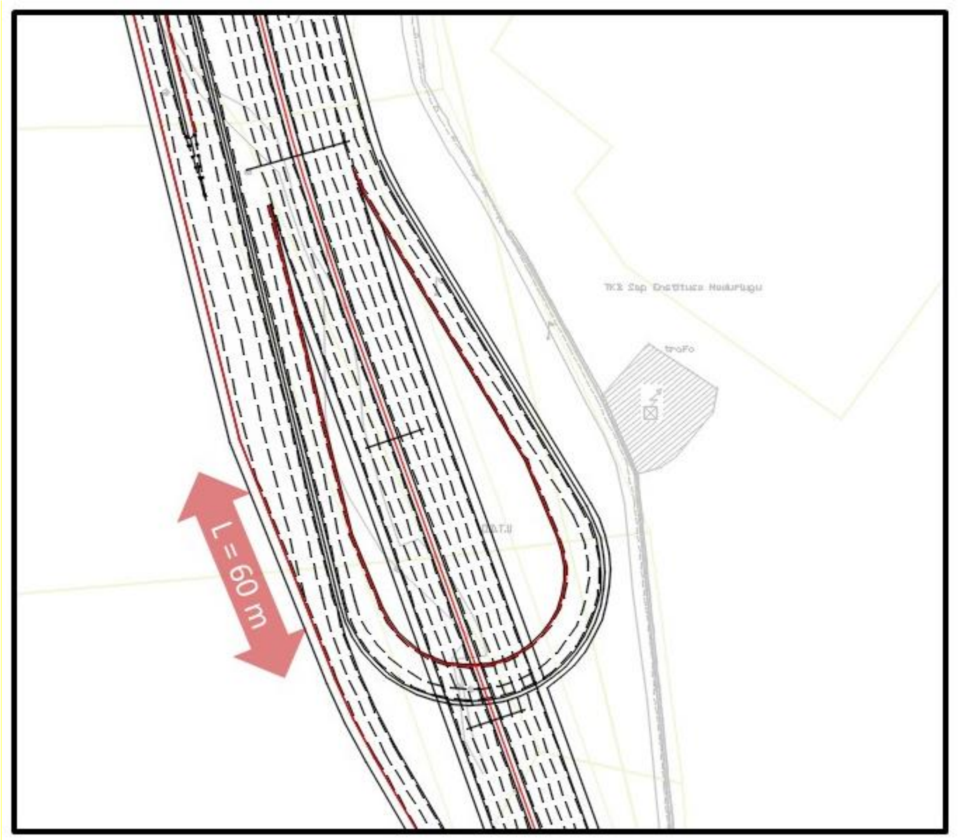


Figure 6.45. Length of Entrance Terminal at Merge Point of METU A1 Ramp and South-West Quadrant Ramp of METU Interchange

The second entrance terminal is on the following segment of the first one. However, in this case, two lanes are merging and the type of it is not tapered again but parallel. Although the same *Table 4.21* is used in determining the length of the acceleration length, design speeds are different compared with the first one. In this case, design speed of the merging road is 50 km/h while the design speed of the major road is 70 km/h which means the required length is 65 m. When the length of the acceleration lane is measured on the project of 1071 Malazgirt Boulevard, it is seen that the value is around 65 meters which is equal to recommended one. However, when the posted speed is taken into consideration for the main roadway, required length of acceleration lane is 175 meters which is highly greater than the existing value. Hence, length of acceleration lane is not adequate for posted speed of main roadway.

Additionally, the length of the taper is measured to be exactly 90 meters which also complies with recommended value that is given on Figure 4.10 and the layout of this entrance terminal can be seen on Figure 6.46.

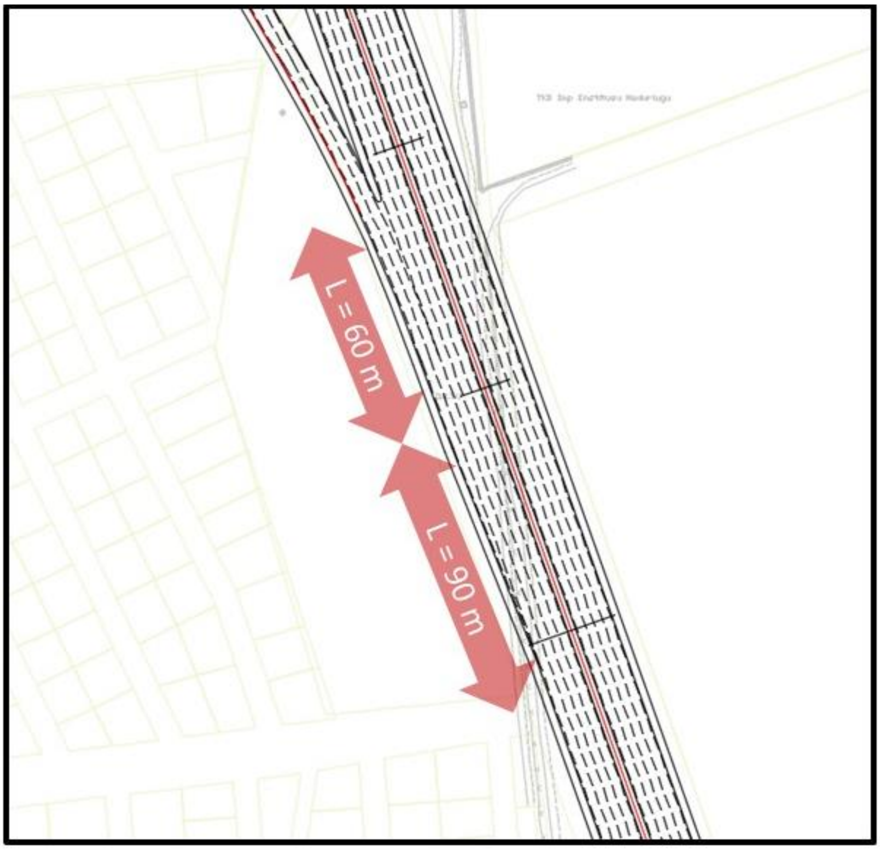


Figure 6.46. Layout of Entrance Terminal at Merge Point of South-West Quadrant Ramp of METU Interchange and 1071 Malazgirt Boulevard

Are spiral curves warranted? If so, do spirals begin and end at appropriate locations? (I.E.2)

As mentioned before, there are two entrance terminals one of which is parallel type and the other one is tapered type. For the entrance ramp which is parallel type, a curve exists while there is not a curve on the tapered type one. This entrance terminal is the one that is at the merge point of south-west quadrant ramp of METU Interchange and 1017 Malazgirt Boulevard. Hence, the control is to be done only for the parallel type and when the radius of the curvature is checked, it is seen to be 700 m looking at

Figure 6.47, which is high enough not to require a spiral curve according to *Table 4.10* in which AASHTO (2011) recommends the use of spirals for the curves with a design speed of 50 km/h when the radius of the curve is less than 148 meters.

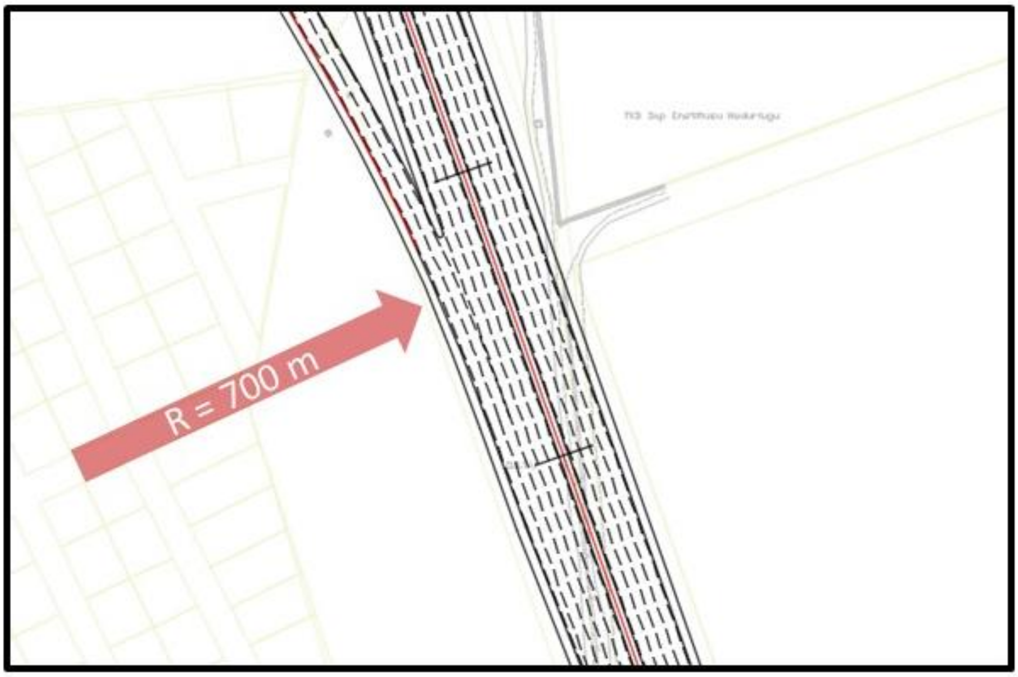


Figure 6.47. Radius of Curve of Entrance Terminal at the Merge Point of South-West Quadrant Ramp of METU Interchange and 1071 Malazgirt Boulevard

Is visibility obscured by traffic barriers and other obstructions? (I.E.3)

Regarding with both of entrance terminals that have mentioned to be only ones on this roadway; it can be said that the views of both entrance terminals are clear and the acceleration lanes have an adequate visibility which can be seen on Figure 6.48.

Figure 6.48. Clear Visibility on Acceleration Lanes at Entrance Terminals

6.4.6. (I.F.) Service Road Systems

Although service road systems are important issue that must be considered during a road safety evaluation of an urban main arterial, in the project of 1071 Malazgirt Boulevard there is not an existing service road. Hence, despite underlining the importance of the issue, an evaluation cannot be made in this thesis.

6.4.7. (I.G.) Lane Balance / Basic Lanes / Lane Continuity

Is the number of lanes appropriate for safe operations and to accommodate variations in traffic patterns? (I.(G.1)

On 1071 Malazgirt Boulevard, on both directions, basic number of lanes is designated as 4 lanes. Laying between two extremely nodes at crossings of important and high-density boulevards (Inonu Boulevard and Mevlana Boulevard), the whole length should be considered as one segment from the point of view of traffic volume although small variations can occur within the boulevard at intersections and interchanges. However, since traffic data does not exist, it is not possible to evaluate whether the number of lanes is adequate or not for this urban arterial.

Is there coordination of lane balance and basic lanes? (I.(G.2)

Vehicles travelling on Inonu Bouelvard towards city center have a basic number of lanes of 4. One of these lanes drops to exit towards METU Campus A1 Gate and only 10 meters ahead taper of exit of interchange ramp starts towards 1071 Malazgit Boulevard with two lanes (Figure 6.49). In this case, if there was an auxiliary lane between these two entrance terminals (since they are too close to each other), exemption could be applied. However, there is not an auxiliary lane and after dropping of one lane towards METU, 3 lanes approach to the exit ramp. Hence, two lanes exiting and three lanes on the major road beyond exit terminal, the balance cannot be provided.

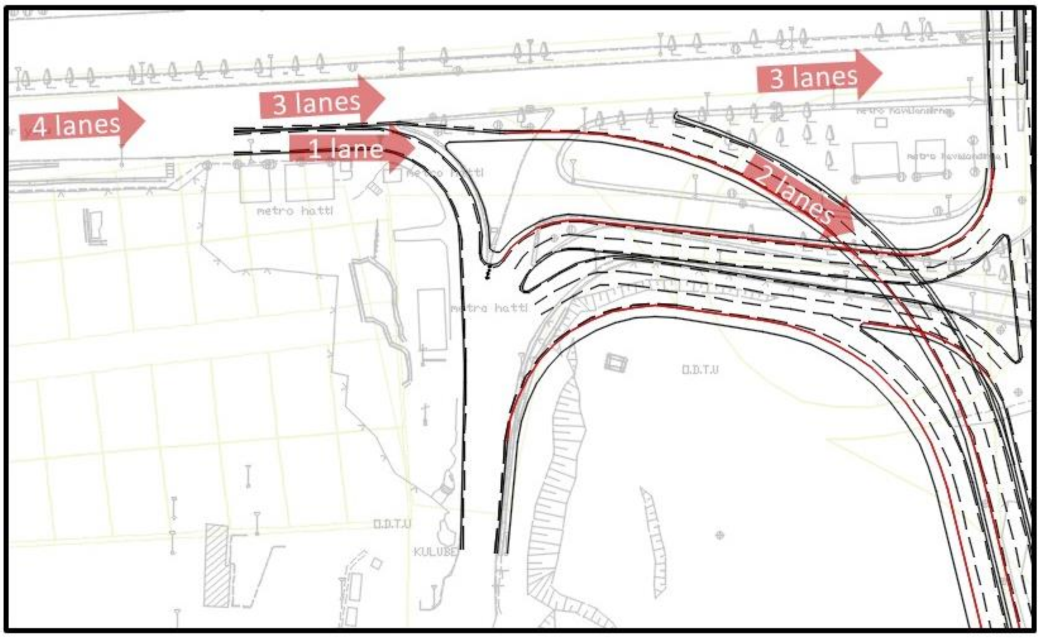


Figure 6.49. Lane Balance Situation at METU Interchange on Inonu Boulevard

At the approach of U-Turn Interchange, there are 5 lanes on the major road. However, as an exemption case, before the exit of this interchange, there is an entrance terminal of two lanes coming from the METU A1 Gate the distance between which is around 150 m that is less than 450 meters. Exit ramp of U-Turn interchange is diverging from the major road with two lanes while the major road continues beyond this exit as three lanes which can be seen on Figure 6.50. In this case, lane balance is allowed to be in such way that the number of lanes approaching (5 lanes) could be equal to the sum of the number of lanes on the major road beyond exit point (3 lanes) and the number of lanes on exit ramp (2 lanes), minus one. However, as can be seen, 3+2-1 = 4 which is not equal to the number of lanes of approach lanes on the major road. Hence, it can be said that, even considering the exemption, lane balance is not appropriate in this section.

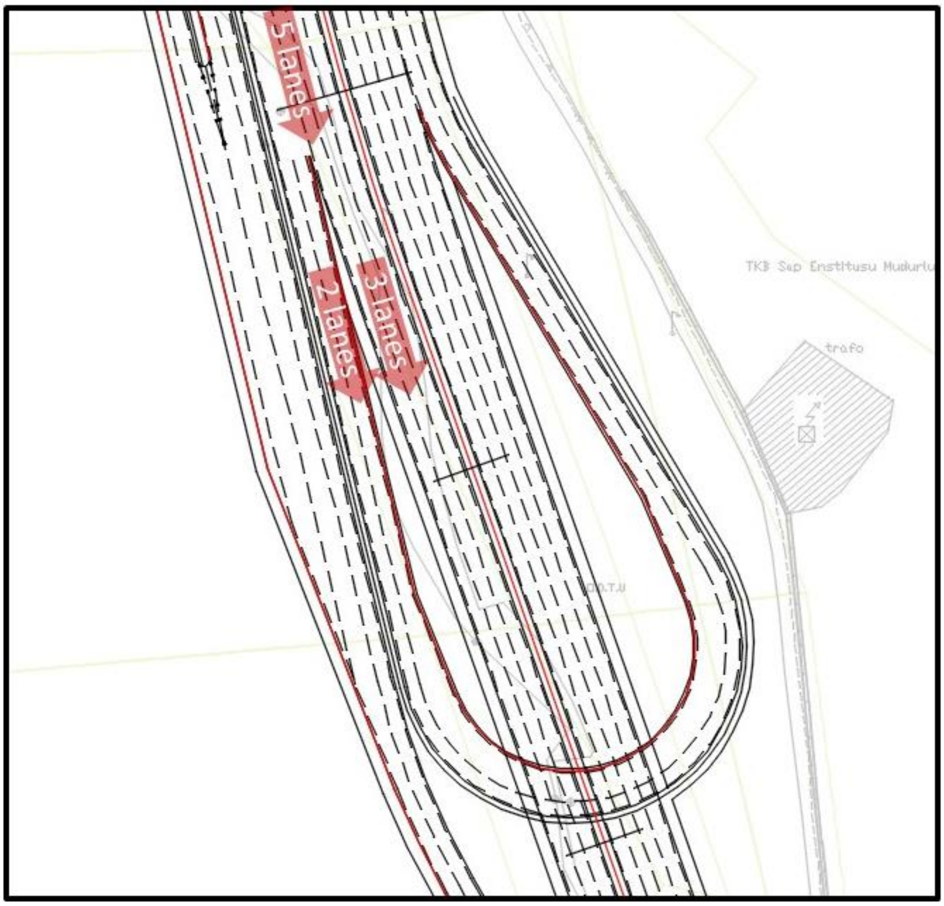


Figure 6.50. Lane Balance Situation at U-Turn Interchange

Between the U-Turn Interchange entrance at METU Interchange exit ramps, there are two auxiliary lanes that merge at the entrance terminal and diverge at the exit terminal while the major road keeps its four lanes throughout the whole related section (Figure 6.51). This is the case where the number of lanes after terminal is equal to the number of merging lanes which makes it appropriate lane balancing at the entrance terminal. However, considering the exemption which is valid for the terminals placed closely to each other less than 450 meters, since the number of lanes approaching the exit (six lanes) is equal to the sum of the number of lanes beyond the exit point (four lanes) and number of lanes on exit ramp (two lanes), lane balance configuration can be accepted adequate for this exit terminal.

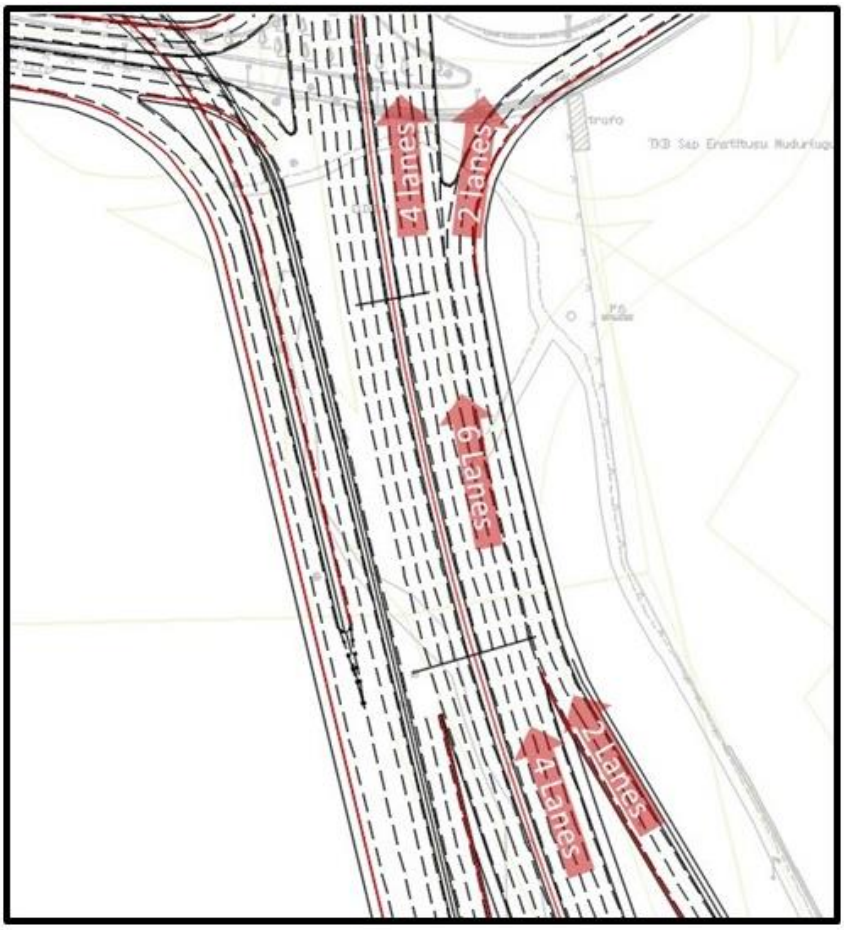


Figure 6.51. Lane Balance Situation between U-Turn Interchange Entrance ramp and METU Interchange Exit Ramp

The entrance terminal which comes from METU A1 Gate and merges 1071 Malazgirt Boulevard has a number of lanes of two while the boulevard is approaching this entrance with three lanes (Figure 6.52). According to ASSHTO (2011), the number of lanes beyond entrance should not be less than the number of lanes merging minus one which makes 4 (2 + 3 – 1). Considering that the number of lanes beyond this entrance on the major road is four, it can be said that lane balance is provided at this merging point.

Figure 6.52. Lane Balance Situation at the Entrance Terminal of South-West Quadrant Ramp of METU Interchange and 1071 Malazgirt Boulevard

At 100. Yil Neighborhood Interchange, there are 2 exits and entrances total in both direction with the same lane balance arrangement (Figure 6.53). At the entrance terminals, one lane ramp merges with the major road approaching with three lanes and the major road continues with four lanes beyond entrance which is appropriate because the sum of the number of lanes merging is equal to the number of lanes beyond the entrance point (3 + 1 = 4). However, at the exit terminals, four lanes of major road approach the exit point while one lane diverges on the ramp and the major road has three lanes beyond the exit which is not appropriate since the number of lanes on the major road beyond the exit is less than the sum of the number of lanes approaching on the major road and the number of lanes on the exit ramp minus one (4 + 1 – 1 = 4).

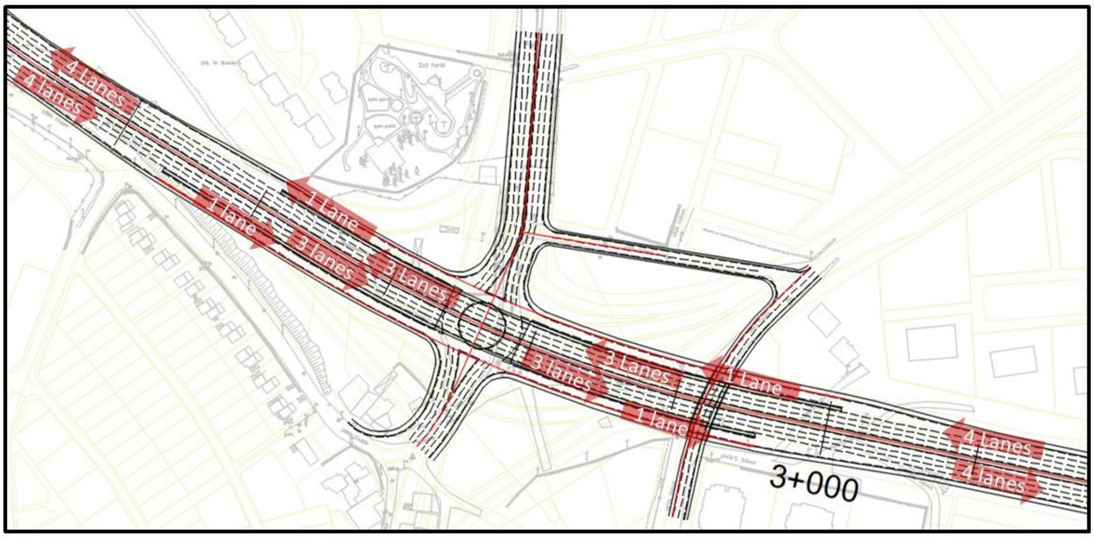


Figure 6.53. Lane Balance Situation at 100. Yil Neighborhood Interchange

Mevlana Boulevard Interchange includes one exit and one entrance terminals. However, both before entrance and exit terminals, there are intersection exit and entrance ramps of Muhsin Yazicioglu Avenue which are closely spaced to each other. For the entrance direction, firstly there is the exit ramp and 15 meters ahead of it there is the entrance ramp. There are four lanes on the major road up to the entrance ramp. After the entrance ramp, an auxiliary lane is placed additionally and there is the exit terminal of the Mevlana Boulevard Interchange exit with a distance of 10 meters away from the preceding exit ramp. Such close ramps are considered as exemption for exit terminals in the subject of lane balance. In this case, the number of lanes beyond the exit on the major road is two lanes while it has four lanes approaching to exit. There are two lanes on the exit ramp one of which is auxiliary lane placed because of the closely placed preceding entrance ramp (Figure 6.54). Hence, lane balance is considered to be appropriate at this direction since the number lanes approaching exit is equal to sum of the number of lanes on exit and major road beyond exit point (2 + 2 = 4). On the other direction, the leg of Mevlana Boulevard is merging the major road with two lanes while the major road approaches this entrance point with two lanes. Beyond this entrance terminal, the major road has four lanes which complies with the recommendations of AASHTO (2011). The total number of the lanes is equal to the number of lanes beyond the entrance terminal (2 + 2 = 4).

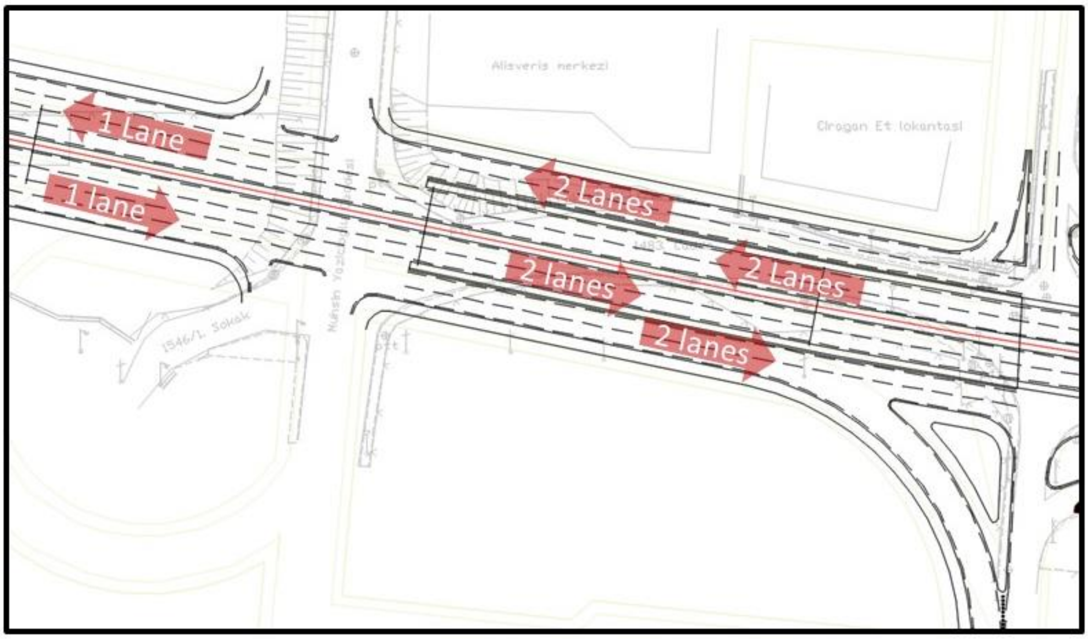


Figure 6.54. Lane Balance Situation at Mevlana Boulevard Interchange

Is lane continuity maintained? (I.(G.2)

In order to evaluate the lane continuity, interchanges should be taken into consideration as a whole. Since U-Turn Interchange and METU Interchange are serving integrated to each other, this section is evaluated as if there is only one interchange and the number of lanes on the major road is investigated through the whole section. On southbound, the number of lanes is beginning with four while it increases to five and then reduces back to three and comes up to four beyond exit at the end which does not comply with the lane continuity. On the other direction (northbound), while the number of lanes start with four, it increases to six, then falls down to five and finally turns back to its original number of lanes which is four. This situation can be seen on Figure 6.55. Hence, lane continuity is not satisfied for this direction as well.

At 100. Yil Neighborhood, on both directions, while there are four lanes both approaching and beyond the interchange (Figure 6.56), throughout the section there are three lanes which means that continuity of basic number of lanes is not provided. Although the part afterwards Mevlana Boulevard Interchange is not included in the project of 1071 Malazgirt Boulevard, it can be stated that because of the reduction of number of lanes throughout the interchange, continuity of basic number of lanes is not procured which is to be seen on Figure 6.57.

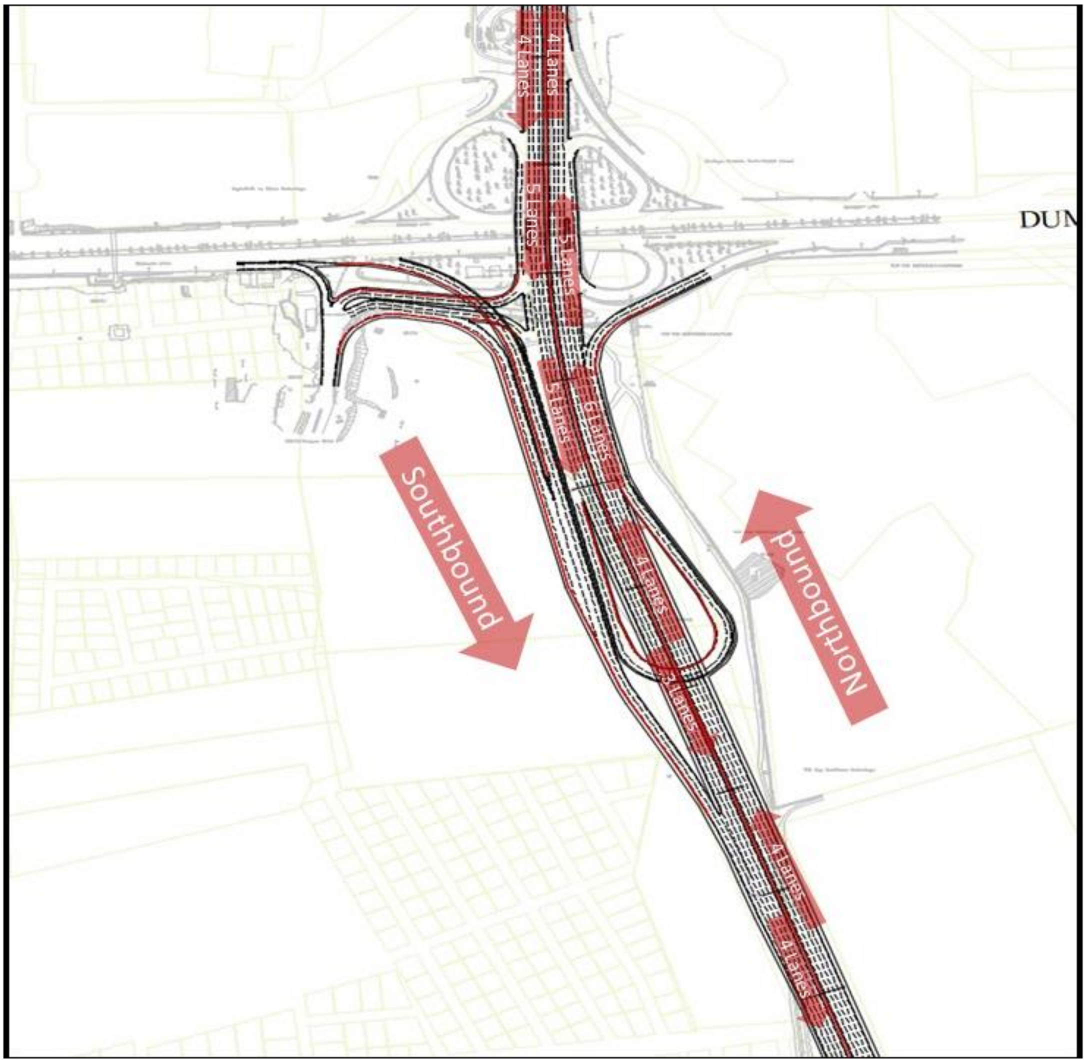


Figure 6.55. Lane Continuity Situation before, throughout and beyond METU and U-Turn Interchanges

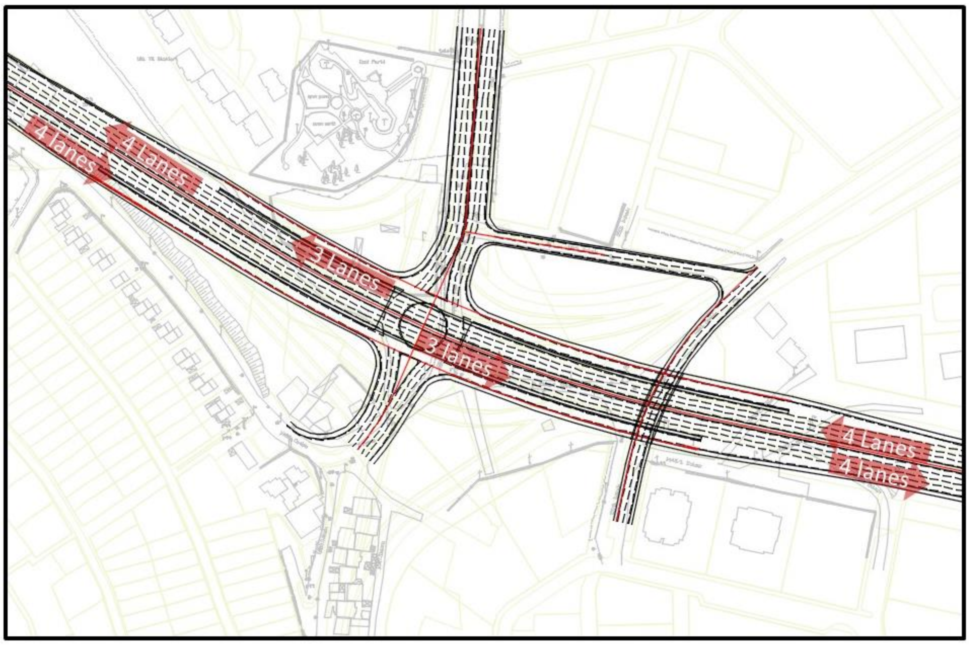


Figure 6.56. Lane Continuity situation before, throughout and beyond 100. Yil Neighborhood Interchange

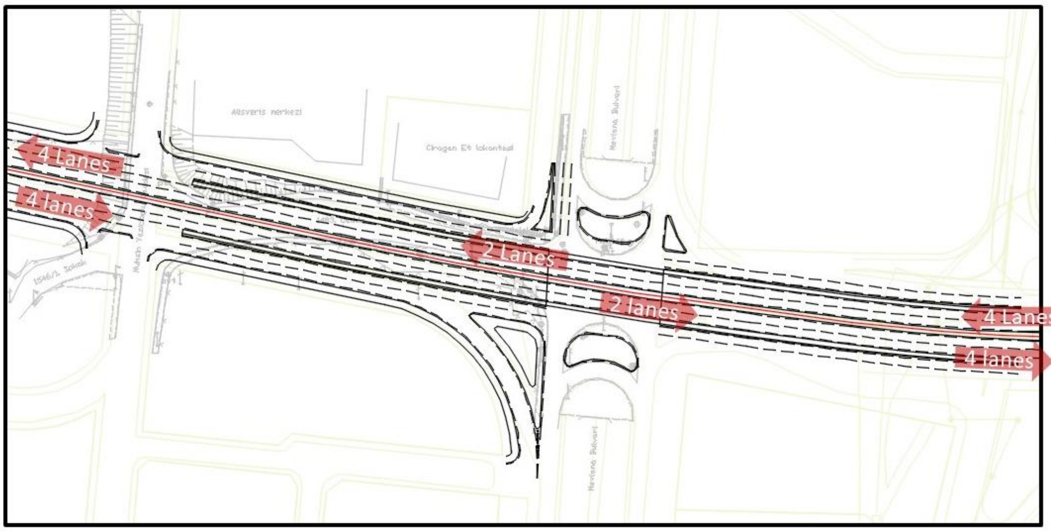


Figure 6.57. Lane Continuity situation before and throughout Mevlana Boulevard Interchange

6.4.8. (I.H) Auxiliary / Turning Lanes

There are not any auxiliary lanes on the interchanges of 1071 Malazgirt Boulevard.

CHAPTER 7

CONCLUSIONS AND FURTHER RECOMMENDATIONS

7.1. Major RSA Principles for Urban Arterials

Although there are many different approaches about the principles of RSA and RSI, it is common to define specific checklists for different kind of road types. In this thesis, the corridor that is subject to evaluation is a n urban major arterial. Hence, apart from general checklist items, there are items that require higher attention regarding to urban major arterials. In this kind of roadways, a checklist has been seen suitable which is a combination of principles that are used for RSA and RSI of both highways and urban streets.

As the arterial shows some characteristics of highways, geometrical items gather importance among the principles that are used such as superelevation, horizontal and vertical alignment. Additionally, the items related with interchanges are crucial. On the other hand, being an urban road, markings, signals, intersections sight distances, drainage, cross sectional elements have importance on RSA evaluation of this kind of roads. So, a range variety of checklist have been compiled in order to perform RSA and RSI studies.

7.2. RSA Results for 1071 Malazgirt Blvd

The main inconvenience about the results of the study is the fact that the design speed is 70 km/h while the posted speed is 82 km/h with an 10 % allowance which lets the drivers drive with speed of 90 km/h. Although most of the criteria are met considering the design speed, posted speed cannot provide the suitableness in terms of design parameters. Hence, some major results can be seenbelow;

- The horizontal and vertical geometry of the roadway is not suitable for an operating speed of 90 km/h.
- There are fixed items within the clear zone.
- Landscaping is not safe.
- Length of vertical curves are not appropriate.
- Width of traveled way on horizontal curves are not sufficient.
- There are corners at intersections with lower radii than the required values.
- There is incompatibility between design and construction of the road. Some parkings that obstructs sight views at intersections exist.
- There are missing signings.
- Lane balance and continuity are not maintained at some interchanges.

Other than these, there are also some problems related with the inspection of existing road instruments such as pavement markings, sight view obstacles etc.

More importantly, if there were a RSA report showing these deficiencies, anybody dying or having a serious injury because of these, the authority would have been kept responsible and paid penalty to the sufferers. Moreover: If RSA had been conducted during design stage, this road would not have been constructed with these design parameters in the first place. Authority takes initiative on its own by sacrificing between Speed & Mobility (Time) without being liable to any standards because of lack of RSA legislation.

7.3. Further recommendations

Better analysis is possible after collecting additional data since some of the data cannot be determined currently. The flow speed can be measured by speed guns, traffic volume can be counted through the segments and also at intersections and interchanges, decent crash data analysis should be compiled and some other crucial measures such as headlight glare, reflectivity of markings etc. should be done. Recommendations that are gathered after defining the results of this study can be summarized as below;

- The posted speed of the road should be lowered to 70 km/h.
- The fixed items within the clear zone should be removed.
- Landscaping should be reviewed
- Width of traveled way on horizontal curves should be increased
- The radii of problematic corners at intersections should be widened
- The incompatibility between design and construction should be resolved
- Parkings should be prohibited where necessary
- Missing signings should be placed
- RSA legislation should be introduced
- Current RSR methodology for rural roads should be converted to RSA
- For Urban Roads, a special RSA methodology should be implemented
- This urban RSA procedure should satisfy the safety requirements of
    - Major Urban Arterials
    - Collector Roads
    - Access Roads

Subsequent to the completion of lacking data, a well-prepared RSA and RSI must be implemented in order to increase the quality of the existing and planned roads and reduce the rate and severity of road accidents. The municipalities must be obliged to use RSA services from independent parties.

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APPENDICES

A. Road Safety and Legislation in the World

Australia: In 1991, two states, New South Wales and Victoria, initiated road safety procedure in Australia. Since, there are organizational differences, the audit procedures differ a bit among the states. Although, almost all of the states have installed road safety audits, local governments are not so contributed to them.

Austria: The RVS guideline 02.02.34 was published in 2007 which is the legislation of RSI of Austria. Then, in order to develop and enhance this guideline and improve the systemization and establish a standardized structure for this purpose, a handbook called "Handbook for carrying out Road Safety Inspection" was claimed. Although the implementation of RSI was not compulsory, after EU Directive was put on enforcement on December 2010, it became compulsory.

In Austria, the start of RSIs was in 2003. Between 2003 and 2007 some sections of roads have been inspected by Austrian Road Safety Board (Kuratorium für Verkehrssicherheit). Some pilot inspections have been conducted in secondary road networks which also leaded to the preparation of RSI handbook.

Belgium: In Belgium, by Belgian Road Safety Institute, an introduction for RSA process was performed and a draft of checklists has been claimed in 2005. The road safety program is managed by the Assembly on Road Safety (Etats généraux de la sécurité routière/Staten Generaal van de Verkeersveiligheid) and it is directed in a way to follow European Commission's targets and timescales.

Bulgaria: Bulgaria, as a state-member of EU, has agreed to implement the directives of EU for road safety. The road safety audits are obligatory for the state roods which are part of Trans-European road network. Also, the roads that are not part of Trans-European road network are undertaking road safety audits however these are not compulsory while they are not funded by EU. Road safety audits include new construction, reconstruction and rehabilitation of the roads. The first group of RS auditors was officially certified in 2013 and up to present only a few RSAs have been done.

Cyprus: Since 2006, for the existing roads, road safety audit has been ongoing in Cyprus. However, according to WHO, up to now, RSA for new projected roads have not been done. The lead agency for road safety is Road Safety Council and the legalization framework belongs to EU Directives.

Czech Republic: Although in Czech Republic, many RSAs have been implemented, there was not a legal basis since they have been introduced EU Directive for road safety. However, this compulsory is valid only for TEN roads.

Denmark: RSA procedure has started in Denmark in 1993. After a few pilot projects, the Danish Handbook for Road Safety Audits has been published in 1997 the procedures of which were based on British procedures.

France: In 1997, a committee for road safety decided on implementation of a road safety audit procedure. Since that date, France has trained many auditors and has made many road safety audits for both new projects and existing roads. In France Road Safety, audit for existing roads are conducted with a three year time interval for the entire country road network. In 2008, a guideline called as “Methodological Guide for Road Safety Inspections” have been published.

Germany: Road Safety Audit procedures started in Germany around 1999. After evaluation of pilot projects and the outcomes that are observed in the other countries, some procedures and checklists have been created.

Greece: “Safe Road Environment” plan was carried into effect in the way of development of road safety audit procedures in Greece in 2002. It includes both the legislation and the guidelines required for the audits.

Ireland: Firstly, road safety audit procedure was introduced in 1999 in Ireland and in 2000 a national Irish standard was produced.

Italy: Italian Ministry of Infrastructure has published the Guidelines for Road Safety (Linee Guida per le analisi di sicurezza della strade) which includes the recommendations for the implementation of the road safety audits.

Netherlands: In Netherlands, first road safety procedure started in 1997 and after evaluation and testing of the procedure for a few years, in 2001, the first guideline for the use of road safety audit was published.

New Zealand: Austroads is the common responsible community for the road safety in both Australia and New Zealand. Hence, same as Australia, the first road safety audit guidelines were introduced in 1990s'. In New Zealand, it is required to have road safety in the entire road network of the country.

Norway: In 1999, guidelines for the road safety audit were published by the Norwegian Public Roads Administration. Afterwards, the same committee has published the "Handbook Road Safety Audits and Inspections" manual in 2006.

Poland: First trial of road safety procedures were carried out in 2001. During these trials, a guideline for Rad Safety audit was produced. In the following years some experts have been trained by the universities in Poland. Road Safety Audit for existing roads are conducted twice a year in autumn and spring.

Portugal: First road safety audits were processed as a pilot study by the National Civil Engineering Laboratory in 1998. In the same year, the audits have been compulsory and the first guidelines were produced in 2001.

Sweden: In Sweden, road safety was considered as a part of internal quality assurance system. However, procedures similar to RSA were started around 1996 and the scope was not to focus on local points but on the entire system of road networks.

United Kingdom: Being the inventor of Road Safety Audit concept, depending on the Road Traffic Act 1974 and its revision which was made in 1988 in order to reduce the road traffic injuries fatalities, the first road safety audit procedures were introduced in

1990. Since that date, it is mandatory to implement road safety audits for national motorways and trunk roads.

USA: First pilot studies started in 1997 in terms of road safety audit in United States. Then, in 2003, an inquiry was conducted by Transportation Research Board.