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Urban Road Design for Bole Bulbulla Condominium

A Thesis in partial fulfillment of Bsc Degree in Civil Engineering

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We certify that research work titled “Urban Road Design for Bole Bulbulla Condominium” is our own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / refertred.

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Abstract

The project deals with the design of highway road based on AACRA manual and it has nine chapter and the contents and the dutier accomplished in each chapter is explained below

- Chapter one is introduction about highway design
- The second chapter is about traffic survey and analysis

The AADT has been determined

The third chapter as about type are sectional design standard here road is classified

- The fourth chapter ids about geometric design
- The horizontal and vertical dimension has been designed
- The fifth chapters is about earthwork computation
 - Mass haval diagram has been constructed
- The sixth chapter is about pavement design
 - Based on CBR result and axle load computed the thickness of the pavement has been determine
- The seventh chapter is about drainage design
 - By calculating the catchment the drainage system was designed using rotational method and manning roughness coefficient
- The eighth chapter is about sign post and read manning
- The ninth chapter is about bill of quantity
 - The total cast of the project was computed

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CHAPTER ONE

1.1 INTRODUCTION

Transportation plays an enormous role in our everyday lives. Each of us travels somewhere almost every day, whether it is to get to work or school, to go shopping, or for entertainment purposes. In addition, almost everything we consume or use has been transported at some point. There are so many ways of transportation, but in this document, we concern on road transportation.

Highways are the important part of our life, the economy of the society as well as it is Part of the infrastructure. Also, it is important for the change location of humans, goods and information. The better of this structure is the faster, more effective and cheaper, can be the capacities of the society used. Traffic security efficiency, economy, environmental protections are some points which considered during design of highway (AAIT, Lecture Note).

The development of road transportation plays a great role in a country's economic development. Roads are the basic infrastructure in which its service extends facilitating the growth and keeping the different sectors of the economy functional. (AAIT, Lecture Note)

A country should have adequately designed enough number of roads connecting its main parts. This helps the sufficient mobility of products, raw materials and labor as well as it has a far-reaching effect embracing administration convenience and creating national integration and cohesion, which enhance for a better economic achievement of the country, it is this and unmentioned needs which motivate us to do our final year project on High way (AAIT, Lecture Note).

. Addis Ababa the capital state of Ethiopia is facing high economic growth and this growth is creating problems upon old infrastructures, one of the old infrastructures creating problem is transport system. This problem can be decreased through creating new transport system and constricting infrastructures ((AACRA), 2004)

Addis Ababa's municipality under its subordinate, Addis Ababa City Road Authority, have been constructing roads throughout the city. It is believed that these roads have enriched the socio-economic development and placed the city in similar position to other metropolitans ((AACRA), 2004)

1.1 Project Background

This project lies in the south-east zone of Addis Ababa region. it is a round in sub-city of Bole. Specific location is Bulebula condominium area road which covers about 2.711 km. which is located in the South-east of the country bordering Oromia regional state to East, Akaki-kalit sub-city to the South West, Ethiopia airline to North and Kirikus sub-city to the North West. The road starts and finish in side of sub- city of Bole.

1.1.1 Project location and topography

This project lies in the south-east zone of Addis Ababa region. it is a round in sub-city of Bole. Specific location is Bulebula condominium area road which covers about 2.711 km. which is located in the South-east of the country bordering Oromia regional state to East, Akaki-kalit sub-city to the South West, Ethiopia airline to North and Kirikos sub-city to the North West. The road starts and finish in side of sub- city of Bole.

1.2. Objective

The objective of this project is to know the design of geometric design, pavement design, drainage design, CBR test, earth work computation, bill of quantity calculation, design procedures and engineering judgments. The other objective is to make us remember and revise all the courses we have taken so far. It also enables us to share knowledge, skill and experience with each other; which is useful for us when we are going to be exposed to real working environment.

1.3. Limitations and Assumptions

There are limitations on this thesis, mainly on time restriction to collect actual data like traffic density, traffic flow (volume) and format / availability of data required for the research work. Therefore, only few days of count was collected which may result in accuracy error. And also, manual counts for the vehicle density could result in human error.

The CBR testing machine of our campus was reported malfunctioning by the lab technicians. Therefore, we were forced to take the samples to profitable private company



Figure 1-1 Topography of the project

CHAPTER TWO

Traffic Survey and Analysis

2.1 General

The institute of transport engineers defines traffic engineering as a subset of transportation engineering as follows.

Transportation engineering is the application of technology and scientific principles to the planning functional design, operation, and management of facilities for any mode of transportation in order to provide for the safe, rapid, comfortable, convenient, economical, and environmentally compatible movement of people and goods. And Traffic engineering is that phase of transportation engineering which deals with the planning, geometric design and traffic operations of roads, streets, and highways, their networks, terminals, abutting lands, and relationships with other modes of transportation. (Roess, Prassas, & McShane, 2004)

2.2 Traffic Survey

Traffic survey is the study of the components of traffic and their behavior. The components of traffic generally can be

- Vehicles(motorists)
- Pedestrians
- Bicycles

Table 2-1: Vehicle Categories (AACRA)

Category	Cars	Light	Medium	Heavy	Articulated
Axles	2	2	3	4	>4
Tiers	4	6	10	14	>14
Length	<3m	3m – 7.5m	3m – 7.5m	>7.5m	>7.5m
GVW	<3.5T	3.5T – 12T	>12T	>12T	>12T
Includes	Cars, Utilities, Minibus, 4WD	Bus, 1 Axle Truck	2 Rear Axle Truck	4 Axle Truck	Large Truck

Some traffic characteristics can be measured directly, while others, because of their inherent nature, can only be estimate. The large number of locations for which traffic information is often required normally prevents direct measurement because of the high cost involved. In this case, reliance on estimates made from short-term counts is the most cost-effective option. (AACRA, AACRA Traffic and Axle Load Study Manual, 2004)

2.2.1 Volume study

Traffic counts are the most basic of traffic studies and are the primary measure of demand; virtually all aspects of traffic engineering require volume as an input, including highway planning and design, decisions on traffic control and operations, detailed signal timing, and others. (Roess, Prassas, & McShane, 2004)

Demand would be the volume plus the vehicles of motorists wishing to pass the site during the study hour who were prevented from doing so by congestion. The latter would include motorists in queue waiting to reach the study location, motorists using alternative routes to avoid the congestion around the study location, and motorists deciding not to travel at all due to the existing congestion (Roess, Prassas, & McShane, 2004)

Capacity would be the maximum volume that could be accommodated by the highway at the study location ability of existing speed regulations and/or to set new ones on a rational basis. (Roess, Prassas, & McShane, 2004)

Traffic volume studies are conducted to collect data on the number of vehicles and/or pedestrians that pass a point on a highway facility during a specified time period. This time period varies from as little as 15 minutes to as much as a year depending on the anticipated use of the data. The data collected also may be put into subclasses which may include directional movement, occupancy rates, vehicle classification, and pedestrian age. Traffic volume studies are usually conducted when certain volume characteristics are needed, some of which follow: (Garber & Lester, Transport and Highway Engineering, 2009)

1. **Average Annual Daily Traffic (AADT)** is the average of 24-hour counts collected every day of the year. AADTs are used in several traffic and transportation analyses for:
 - a. Estimation of highway user revenues
 - b. Computation of crash rates in terms of number of crashes per 100 million vehicle miles
 - c. Establishment of traffic volume trends
 - d. Evaluation of the economic feasibility of highway projects
 - e. Development of freeway and major arterial street systems
 - f. Development of improvement and maintenance programs
2. **Average Daily Traffic (ADT)** is the average of 24-hour counts collected over a number of days greater than one but less than a year. ADTs may be used for:
 - a. Planning of highway activities
 - b. Measurement of current demand
 - c. Evaluation of existing traffic flow
3. **Peak Hour Volume (PHV)** is the maximum number of vehicles that pass a point on a highway during a period of 60 consecutive minutes. PHVs are used for:
 - a. Functional classification of highways

- b. Design of the geometric characteristics of a highway, for example, number of lanes, intersection signalization, or channelization
- c. Capacity analysis
- d. Development of programs related to traffic operations, for example, one-way street systems or traffic routing
- e. Development of parking regulations
- 4. **Vehicle Classification (VC)** records volume with respect to the type of vehicles, for example, passenger cars, two-axle trucks, or three-axle trucks. VC is used in:
 - A. Design of geometric characteristics, with particular reference to turning-radii
 - B. requirements, maximum grades, lane widths, and so forth
 - 1. Capacity analyses, with respect to passenger-car equivalents of trucks
 - 2. Adjustment of traffic counts obtained by machines
 - 3. Structural design of highway pavements, bridges, and so forth
- 5. **Vehicle Miles of Travel (VMT)** is a measure of travel along a section of road. It is the product of the traffic volume (that is, average weekday volume or ADT) and the length of roadway in miles to which the volume is applicable. VMTs are used mainly as a base for allocating resources for maintenance and improvement of highways.

There are two ways of conducting volume survey on a particular highway. The choice of the method will depend on many factors such as the purpose and scope of the study being conducted.

1. Manual method

Manual counts are relatively inexpensive and can generally be set up in a short period of time. They also involve some unique challenges, including the coordination of personnel, accurate observation and recording of data, and organization of data in a manner that can be easily understood and interpreted. The major disadvantage of hand counters is that the data must be manually recorded in the field at periodic intervals. This disrupts the count periodically and calls for close coordination between observers to ensure that the data collected is consistent. For the most part, therefore, it is advantageous to use equipment capable of recording and storing the data electronically in a machine-readable form that can be directly downloaded to a computer using a data interface. (Roess, Prassas, & McShane, 2004)

2. Portable count techniques

All count equipment using pneumatic road tubes or other temporary detection equipment is referred to as “portable.” This is because the equipment used can be transported from location to location to conduct various counts as needed. (Roess, Prassas, & McShane, 2004)

.Here for our project we use Manual counting method.

2.2.2spotSpeed studies

Speed characteristics are strongly related to safety concerns and are needed to assess the viability of existing speed regulations and/or to set new ones on a rational basis.

When conducting manual counts, practical human constraints must be taken into account. Some necessary considerations include:

All personnel must be properly trained concerning the details of the study and the expected results. This includes making them familiar with field sheets and equipment to be used as well as their specific counting assignments. No study should place so many observers at a location that they distract drivers and cause traffic disruptions. (Roess, Prassas, & McShane, 2004)

Locations for Spot Speed Studies

The following locations generally are used for the different applications listed: (Garber & Lester, Transport and Highway Engineering, 2009)

1. Locations that represent different traffic conditions on a highway or highways are used for *basic data collection*.
2. Mid-blocks of urban highways and straight, level sections of rural highways are sites for *speed trend analyses*.
3. Any location may be used for the solution of a *specific traffic engineering problem*. When spot speed studies are being conducted, it is important that unbiased data be obtained. This requires that drivers be unaware that such a study is being conducted. Equipment used therefore, should be concealed from the driver, and observers conducting the study should be inconspicuous. Since the speeds recorded eventually will be subjected to statistical analysis, it is important that a statistically adequate number of vehicle speeds be recorded.

Time of Day and Duration of Spot Speed Studies

The time of day for conducting a speed study depends on the purpose of the study. In general, when the purpose of the study is to establish posted speed limits, to observe speed trends, or to collect basic data, it is recommended that the study be conducted when traffic is free-flowing, usually during off-peak hours. However, when a speed study is conducted in response to citizen complaints, it is useful if the time period selected for the study reflects the nature of the complaints. The duration of the study should be such that the minimum number of vehicle speeds required for statistical analysis is recorded. Typically, the duration is at least 1 hour and the sample size is at least 30 vehicles. (Garber & Lester, Transport and Highway Engineering, 2009)

For this project, the speed study is conducted at off peak hour around 1:PM (7:AM Ethiopian local time)

2.2.3 Design period

Design period is the length or duration of time during which the pavement structure is expected to function satisfactorily without the need for major intervention (rehabilitation such as overlays or reconstruction) or the duration on time until the pavement structure reaches its terminal condition (failure condition).

Selecting appropriate design period depends on: -

- Functional importance of the road.

For important roads and for roads with high traffic volume, rehabilitation would cause major inconvenience to road users and thus, it generally appears economical to construct roads with longer design periods.

AACRA pavement design manual recommends design life to be

- New granular and semi-rigid pavement =20 years
- New rigid pavement= 40 years
- New gravel pavement =10 years
- Asphalt overlays= 15 years
- Granular overlay=10 years

For our project road, we select the design period of 15 yrs. because of asphalt over lays

- Traffic volume.
- Location & terrain of the project.

For roads in difficult locations and terrain where regular maintenance proves to be costly and time consuming because of poor access and non-availability of nearby construction material sources, a longer design period is also appropriate

- Financial constraints.
- Difficulty in forecasting traffic
- When accurate traffic estimates cannot be made or from a purely economic point of view, it may be advisable to reduce the design period or pavement structures may be planned to be constructed in stages or upgraded. (AAiT, Highway engineering II)

Table 2-2 Design Speed for different road types

Road Type	Proposed Speed Limit or Speed Environment	Minimum Design Speed for Cars
Suburban Freeway Through Carriageway	100	110
Suburban Freeway to Freeway Ramp	80	80
Suburban Freeway to Arterial Road Ramp	60/70*	60/70
Suburban Freeway Collector-Distributor Road	80	90
Inner Urban Freeway Through Carriageway	80	90
Inner Urban Freeway to Freeway Ramp	60	60
Inner Urban Freeway to Arterial Road Ramp	60/70*	60/70
Inner Urban Freeway Collector-Distributor Road	70	80
Freeway Loop Ramp	Truck Advisory Sign 35	40
Continuous Frontage Road	70	80
Arterial Road Controlled Access	70	80
Discontinuous Frontage Road	60**	60**
Arterial and sub-arterial road with frontage access	60	70
Collector roads and industrial/commercial access roads	60	60
Low Volume Residential Collector roads (<3000 vpd)	50	50
Local Residential Street	30	30
Access place	15	15
Temporary roads (e.g. construction sidetracks)	40-60	40-60

Since our road type is a collector road, we have used a minimum design speed of 60km/hr, which can be seen on the highlighted section of the above table.

2.2.5 Traffic Forecast

Travel demand is expressed as the number of persons or vehicles per unit time that can be expected to travel on a given segment of a transportation system under a set of given land-use, socioeconomic, and environmental conditions. Forecasts of travel demand are used to establish the vehicular volume on future or modified transportation system alternatives. The methods for forecasting travel demand can range from a simple extrapolation of observed trends to a sophisticated computerized process involving extensive data gathering and mathematical modeling. The travel demand forecasting process is as much an art as it is a science. Judgments are required concerning the various parameters—that is, population, car ownership, and so forth that provide the basis for a travel forecast. The methods used in forecasting demand will depend on the availability of data and on specific constraints on the project, such as availability of funds and project schedules. (Garber & Lester, Transport and Highway Engineering, 2009)

There are two basic demand forecasting situations in transportation planning. The first involves travel demand studies for urban areas, and the second deals with intercity travel demand. Urban travel demand forecasts, incorporated in most travel forecasting procedures. Land-use characteristics are a primary determinant of travel demand. The amount of traffic generated by a parcel of land depends on how the land is used. Socioeconomic characteristics of the people living within the city also influence the demand for transportation. Lifestyles and values affect how people decide to use their resources for transportation. (Garber & Lester, Transport and Highway Engineering, 2009)

2.2.6 Axle Load Survey

Axle load surveys involve the classification of vehicles and the physical weighting of each axle on the vehicle which is selected for testing during the survey period. A representative range of vehicles should be selected for testing covering the spectrum of vehicles in the classification system and passing through the survey site during the survey period. (AACRA, AACRA Traffic and Axle Load Study Manual, 2004)

The types of survey sites used for conducting axle load studies include:

- Permanent stations where the data collection is continuous process for throughout the year
- Temporarily stations for particular section of the road (AACRA, AACRA Traffic and Axle Load Study Manual, 2004)

In pavement design method, the traffic load is determined in terms of the number of repetitions of an 18,000-lb (80 kilo Newton's (k N)) single-axle load applied to the pavement on two sets of dual tires. This is usually referred to as the equivalent single-axle load (ESAL). (Garber & Lester, Transport and Highway Engineering, 2009). Other loads which differ from the 80 ken single axle load are turned to the standard by multiplying them with the equivalent factor.

AACRA gives the equivalence factor as

$$EquivalenceFactor = \frac{AxleLoad(kg)^{4.5}}{8160}$$

AACRA has provided typical ESA values for Addis Ababa based on vehicle class.

2.2.7 Design Traffic loading

Traffic is the most important factor in pavement design. The consideration of traffic should include both the loading magnitude and configuration and the number of load repetitions. There are three different procedures for considering vehicular and traffic effects in pavement design: fixed traffic, fixed vehicle, and variable traffic and vehicle. (Huang, 2004)

Fixed Traffic

In fixed traffic, thickness of pavement is governed by a single wheel load, and the Number of load repetitions is not considered as a variable. If the pavement is subjected to multiple wheels, they must be converted to an equivalent single-wheel load (ESWL) so that the design method based on a single wheel can be applied. This method has been used most frequently for airport pavements or for highway pavements with heavy wheel loads but light traffic volume. Usually, the heaviest wheel load anticipated is used for design purposes. (Huang, 2004)

Fixed Vehicle

In the fixed vehicle procedure, the thickness of pavement is governed by the number of repetition of a standard vehicle or axle load, usually the 18-kip (80-kN) single axle load. If the axle load is not 18 kip(80-kN) or consists of tandem or tridem axles, it must be converted to an 18-kip(80-kN) single axle load by an equivalent axle load factor(EALF). The number of repetitions under each single or multiple axle load must be multiplied by its EALF to obtain the equivalent effect based on an 18-kip(80-kN) single axle load. A summation of the equivalent effect of all axle loads during the design period results in an equivalent single axle load (ESAL), which is the single traffic parameter for design purpose. (Huang, 2004)

AACRA design manual is based on this traffic loading system. This project will also be based on this design loading method.

Variable traffic and vehicle

For variable traffic and vehicles, both traffic and vehicle are considered individually, so there is no need to assign an equivalent factor for each axle load. The loads can be divided into a number of groups, and the stress, strains, and deflections under each load group can be determined separately and used for design purpose. This procedure is best suited for the mechanistic method of design. Where in the response of pavement under different loads can be evaluated by using a computer. (Huang, 2004)

2.3 Calculations of Traffic surveys

Spot Speed calculations

We have conducted speed survey for the proposed road. But since the road is not finished its not open for traffic. We decided to study a road segment nearby that we assume will reflect the actual road users after its completion.

We have measured the speed of 50 vehicles passing through a 100m road segment. The time that takes a vehicle to pass the 100m is recorded and the speed is computed using the formula

$$speed = \frac{distance}{time} = 3.6 \frac{100}{t(sec)} kmph$$

The result is presented here tabulated:

Table 2-3 Spot speed Survey Result

vehicles No	distance(m)	time (sec)	speed(kmph)
1	100	8.29	43.43
2	100	17.0	21.18
3	100	7.07	50.92
4	100	9.41	38.26
5	100	8.87	40.59
6	100	8.55	42.11
7	100	7.11	50.63
8	100	7.68	46.88
9	100	7.84	45.92
10	100	6.24	57.69
11	100	7.58	47.49
12	100	6.18	58.25
13	100	5.01	71.86
14	100	5.26	68.44
15	100	7.55	47.68
16	100	8.95	40.22
17	100	7.36	48.91
18	100	9.27	38.83
19	100	7.5	48.00
20	100	8.15	44.17
21	100	7.24	49.72
22	100	6.98	51.58
23	100	7.96	45.23
24	100	9.94	36.22
25	100	7.3	49.32
26	100	8.75	41.14
27	100	5	72.00
28	100	8.35	43.11
29	100	7.1	50.70
30	100	5.92	60.81
31	100	7.64	47.12
32	100	7.77	46.33
33	100	7.78	46.27
34	100	5.5	65.45
35	100	6.46	55.73
36	100	8.04	44.78
37	100	6.31	57.05
38	100	7.44	48.39
39	100	8.61	41.81
40	100	7.2	50.00

41	100	7.9	45.57
42	100	8.29	43.43
43	100	9.2	39.13
44	100	7.7	46.75
45	100	8.62	41.76
46	100	5.52	65.22
47	100	8.01	44.94
48	100	7.64	47.12
49	100	7.76	46.39
50	100	5.39	66.79

From the table, the average speed is calculated to be 48.83 kmph

To determine whether the number of sample is enough for these speed study we check using the formula $N = \frac{Z\alpha^2}{d}$

Where Z = number of standard deviations corresponding to the required confidence

$Z = 1.96$ for 95% confidence level

d = limit of acceptance error kmph = 2.65

α = standard deviation kmph

$$\alpha = \sqrt{\frac{\sum_{i=1}^n (U_i - U_{\text{mean}})^2}{N - 1}}$$

$$N = \frac{1.96 * 9.54^2}{2.65} = 49.79$$

The number of sample which is 50 vehicles is enough for this confidence level and error margin.

Axle load calculation

For collector road category, the maximum AADT is given in AACRA to be $AADT_o = 4000 \text{ vec/d}$

Assuming the time between AADT survey and opening of the road to be two years

Traffic Analysis

Traffic count data



Figure 2.1 during Traffic Volume Counting

Table 2.4 Traffic Count Result

Vehicle category	The fist 15min	The second 15min	The third 15 min	The forth 15min
car	276	260	296	337
LIGHT	21	19	23	26
MEDIUM	3	3	1	2
articulated	0	0	0	0
total	320	282	320	365

Total number of count in both direction=365veh/15min. The peak hourly volume is used for our project since it is educational purpose and thus will be safer evens the worst case.

Traffic forecasting

The growth rate (r) = 6% since our project road (bole bulbulla road) is collector and found in economic zone 3 (as we can see from map of economic zone)

AADT (two way) Calculation

Construction period (x) is the time required to construct the road.

$X=2$ yrs mostly used that is anticipated period between survey and opening year

$$PHV = \max(365, 315, 322 \text{ \& } 303) = 395$$

$$AADT_0 = 365 * 4 * 24/24 = veh/d = ADT$$

$$AADT_1 = AADT_0 * (1 + R)^x = 1580(1 + 0.06)^2 = 1775 \text{ When the road opened}$$

$$AADT_N = AADT_1 * (1 + r)^{15} = 1775 * (1 + 0.06)^{15} = 3925 v/d \text{ at the last year of design period.}$$

From the table and the figure given bellow the growth factor for the project has been determined. The road which is around BoleBulbulla is located at economic zone three3). For economic zone three and our road is classified as collector and the growth factor is found from the table to be 6%.

Table 2-4: Growth Rate (AACRA)

Road Classification	Economic Growth Zone					
	1	2	3	4	5	6
Arterial	7.0	11.2	7.0	11.1	7.2	9.3
Sub-Arterial	6.5	9.3	6.5	9.2	6.6	8.2
Collector	6.0	7.4	6.0	7.4	6.1	6.8
Local	5.5	5.5	5.5	5.5	5.5	5.5

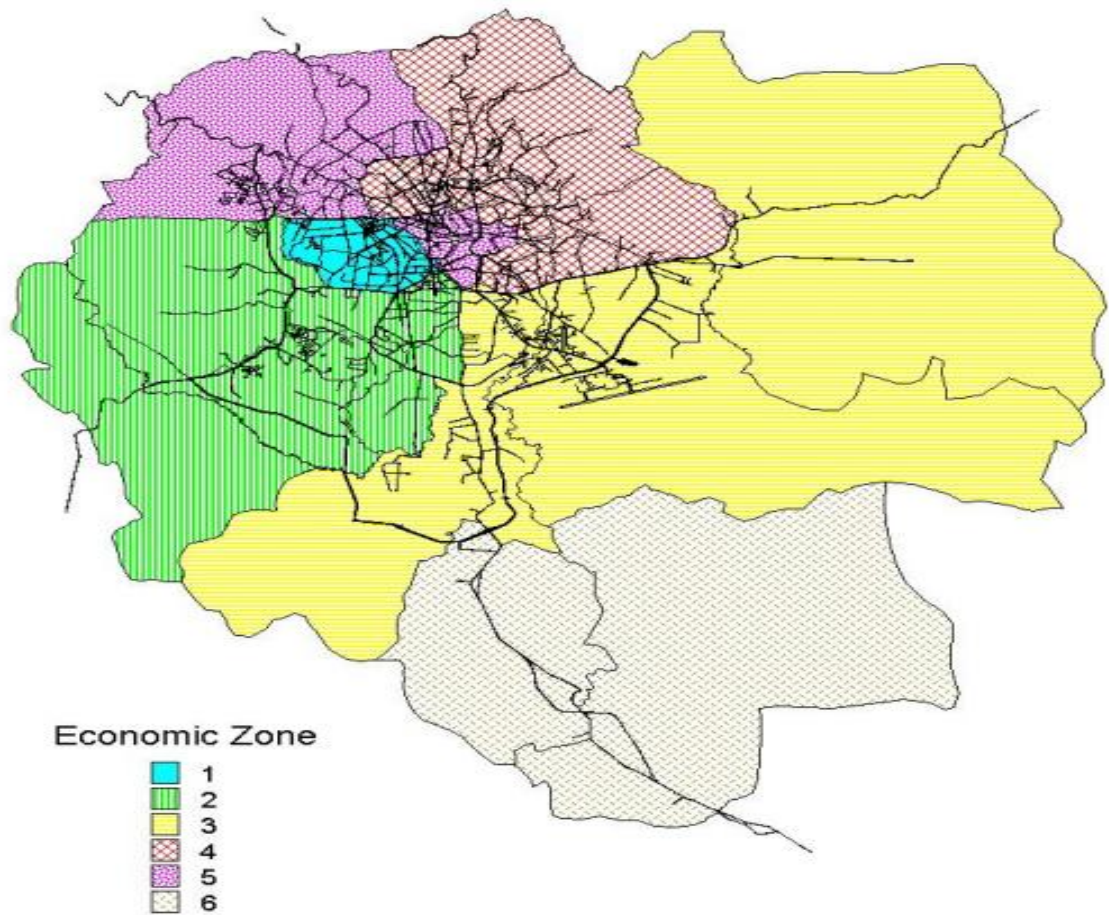


Figure 2-1: Addis Ababa Economic Zones (AACRA)

Cumulative growth factor GF

$GF = 23.3$ for $r = 6\%$ & for $designperiod = 15years$ from the table below. Or it can be calculated manually by the formula $GF = (1 + 0.01r)((1 + .01r)^{15} - 1)/.01r$

Table 2-5: Growth Factor for Different Design periods (AACRA)

Design Period (years)	Growth Rate (% pa)					
	0	2	4	6	8	10
5	5	5.2	5.4	5.6	5.9	6.1
10	10	10.9	12.0	13.2	14.5	15.9
15	15	17.3	20.0	23.3	27.2	31.8
20	20	24.3	29.8	36.8	45.8	57.3
25	25	32.0	41.6	54.9	73.1	98.3
30	30	40.6	56.1	79.1	113.3	164.5
35	35	50.0	73.7	111.4	172.3	271.0
40	40	60.4	95.0	154.8	259.1	442.8

$$subgarde = N_{ss} * 365 * GF$$

$$cementedmaterial = N_{sc} * 365 * GF$$

$$N_{as} = 1.1N_{where}$$

$$asphalt = N_{as} * 365 * GF$$

$$N = N_{IEFl} + N_{mEFm} + N_{hEFh} + N_{aEFa}$$

N=IS the initial daily traffic loading is calculated from in accordance with section 6 of AACRA manual. EF Equivalent factor can be calculated by

$$EF = \left(\frac{axleload\ KG}{8160} \right)^{4.5} \quad \text{or from the table we can obtain respective equivalent factor}$$

NL is the daily number of light vehicles

NM is the daily number of medium vehicles

NH is the daily number of HEAVY vehicle

Na is the daily number of articulated vehicles

Here we take typical value for calculation

$$N = 384 * 0.03 + 0 * 0.73 + 11 * 1.31 = 395 \text{ EAL/d}$$

$$N_{AS} = 1.1N = 1.1 * 395 = 435.5 \text{ veh/day}$$

$asphalt(CESAL) = N_{as} * 365 * GF = 435.5 * 365 * 23.3 = 3598000 = 1.2 * 10^6 \text{ ESA}$ will be used later in chapter 6 when designing pavement.

CHAPTER THREE

Road Types and Selection of Design Standards

3.1 TYPES OF ROAD

3.1.1 Over view

Road transport is one of the most common modes of transport. Roads in the form of track ways, human pathways etc. were used even from the pre-historic times. Since then many experiments were going on to make the riding safe and comfort. Thus, road construction became an inseparable part of many civilizations and empires. In this chapter, we will see the different generations of road and their characteristic features. (Mathew & Rao, 2007)

3.1.2 History of highway engineering

The history of highway engineering gives us an idea about the roads of ancient times. Roads in Rome were constructed in a large scale and it radiated in many directions helping them in military operations. Thus, they are considered to be pioneers in road construction and these three countries are main roll on Ancient roads that are, Roman, British, French etc. (Mathew & Rao, 2007)

3.1.3 Modern roads

The modern roads by and large follow Macadam's construction method. Use of bituminous concrete and cement concrete are the most important developments. Various advanced and cost-effective construction technologies are used. Development of new equipment's helps in the faster construction of roads. Many easily and locally available materials are tested in the laboratories and then implemented on roads for making economical and durable pavements. Scope of transportation system has developed very largely. Population of the country is increasing day by day. The life style of people began to change. The need for travel to various places at faster speeds also increased. This increasing demand led to the emergence of other modes of transportation like railways and travel by air. While the above development in public transport sector was taking place, the development in private transport was at a much faster rate mainly because of its advantages like accessibility, privacy, flexibility, convenience and comfort. This led to the increase in vehicular traffic especially in private transport network. Thus road space available was becoming insufficient to meet the growing demand of traffic and congestion started. In addition, chances for accidents also increased. This has led to the increased attention towards control of vehicles so that the transport infrastructure was optimally used. Various control measures like traffic signals, providing Roundabouts and medians, limiting the speed of vehicle at specific zones etc. were implemented. With the advancement of better roads and efficient control, more and more investments were made in the Road sector especially after the World wars. These were large projects requiring large investment. For optimal utilization of funds, one should know the travel pattern and travel behavior. This has led to the emergence of transportation planning and demand management. (Mathew & Rao, 2007)

AACRA classifies roads in to five groups.

- Freeway
- Arterial
- Sub-Arterial
- Collector
- Locals

Table 3-1: Typical Traffic Percentage Composition for the Different Road Categories of Addis Ababa (AACRA)

Road Classification	AADT (two way)	Typical Percentage				
		Car	Light	Medium	Heavy	Articulated
Freeway	-	-	-	-	-	-
Arterial	10,000	80%	17%	2%	0%	1%
Sub-Arterial	9,000	89%	9%	1%	0%	1%
Collector	4,000	91%	8%	1%	0%	0%
Local	1,500	97%	3%	0%	0%	0%

3.1.4 FREEWAYS

Freeway status is applied only to high volume arterial roads with full control of access.

Freeways are grade separated multi-lane roads with no property access allowed. Certain

Classes of vehicles may be precluded from using a freeway.

These characteristics lead to the need for high standard roads may be constructed in stages, the first stage being a two-lane two-way road with the interchanges constructed in the first stage. The design of the interchange ramps for these roads requires special consideration to avoid inappropriate movements at the ramp terminals on the freeway. If no median is introduced, wrong way movements may be attempted and entering vehicles may move into the oncoming lane with hazardous consequences. A median must therefore be introduced through the interchange.

3.1.5 Urban Arterial & sub-Arterial Roads

Urban Arterial & sub-Arterial Roads are the major traffic in Addis Ababa. Freeway are a special type of arterial road and are deal with separately in the above. Urban arterials are usually dual function facilities providing service to:

- Through traffic (primary function)
- Local traffic and property access (secondary function)

In some cases, the two functions are separated with a roadway for each function. In other cases, the secondary function is completely suppressed and access point is limited to fairly widely spaced intersection (e.g. a bypass road).

Roads are an integral part of the urban fabric providing services to a wide range of users. Careful attention to the needs of all users (vehicles, cycles, public transport, pedestrians, and public utility authorities) is required.

The impact on urban form and the requirements of the Addis Ababa Master plan also is considered. Sub-arterial roads are the less important roads with a through traffic function. They would be expected to carry lower volumes of traffic, and may be single carriageway roads.

3.1.6 Collector roads & Local streets

The road layout should conform to the requirements of the external road network and satisfy the transport provisions of the city Master Plan.

The external road network should be designed and located to provide routes which are more convenient for potential through traffic within the network. Major roads should be complete and of adequate capacity to accommodate through network movements. The internal road system should not provide through routes that are more convenient than the external road network. Where a road is needed that provides a convenient through route, then a sub-arterial road (local crossing route) could be considered with appropriate capacity and abutting land use.

Designers are encouraged to develop road layouts that will promote the objective of the road hierarchy. Innovation can lead to improvements in amenity, safety and other elements such as streetscapes, legibility and ongoing maintenance costs.

The design features of each type of road convey to the driver its primary function and encourage appropriate driver behavior. This can be achieved by complying with the following requirements wherever possible:

- Traffic volumes and speeds on any collector and local road should be compatible with the residential, school, industrial, or shopping functions of that road
- Limiting the length of a local should ensure its status as an access street is retained, where the traffic, in terms of speed and volume will enable the integration of pedestrian, bicycle and vehicular movements
- Limiting the length will also ensure that access convenience is not unduly impaired as a result of speed restraints.

The length of collector road within a development should be minimized. ie a bus route through a precinct is required, and then this must be along a collector road located to service those parts of the area not served by bus routes on the peripheral arterial or sub-arterial roads. If no internal bus route is required, short “stub” collector roads branching out into local roads via a roundabout could be considered.

The time required for drivers to travel on all access streets within the development should be minimized.

Where access streets form part of a pedestrian or bicycle network, access links should provide suitable connectivity with adjoining access streets or open space systems so to ensure such pedestrian and bicycle networks are functionally efficient.

The road network should ensure that no road links with another road which is more than two levels higher or lower in the hierarchy. Connections between internal roads should be T-junction or controlled by roundabouts.

It is preferred that the access road network layout should be such that speed control is inherent in the layout design, i.e. bends and some intersection types provide speed calming rather than devices such as speed bumps, raised platforms or chicanes.

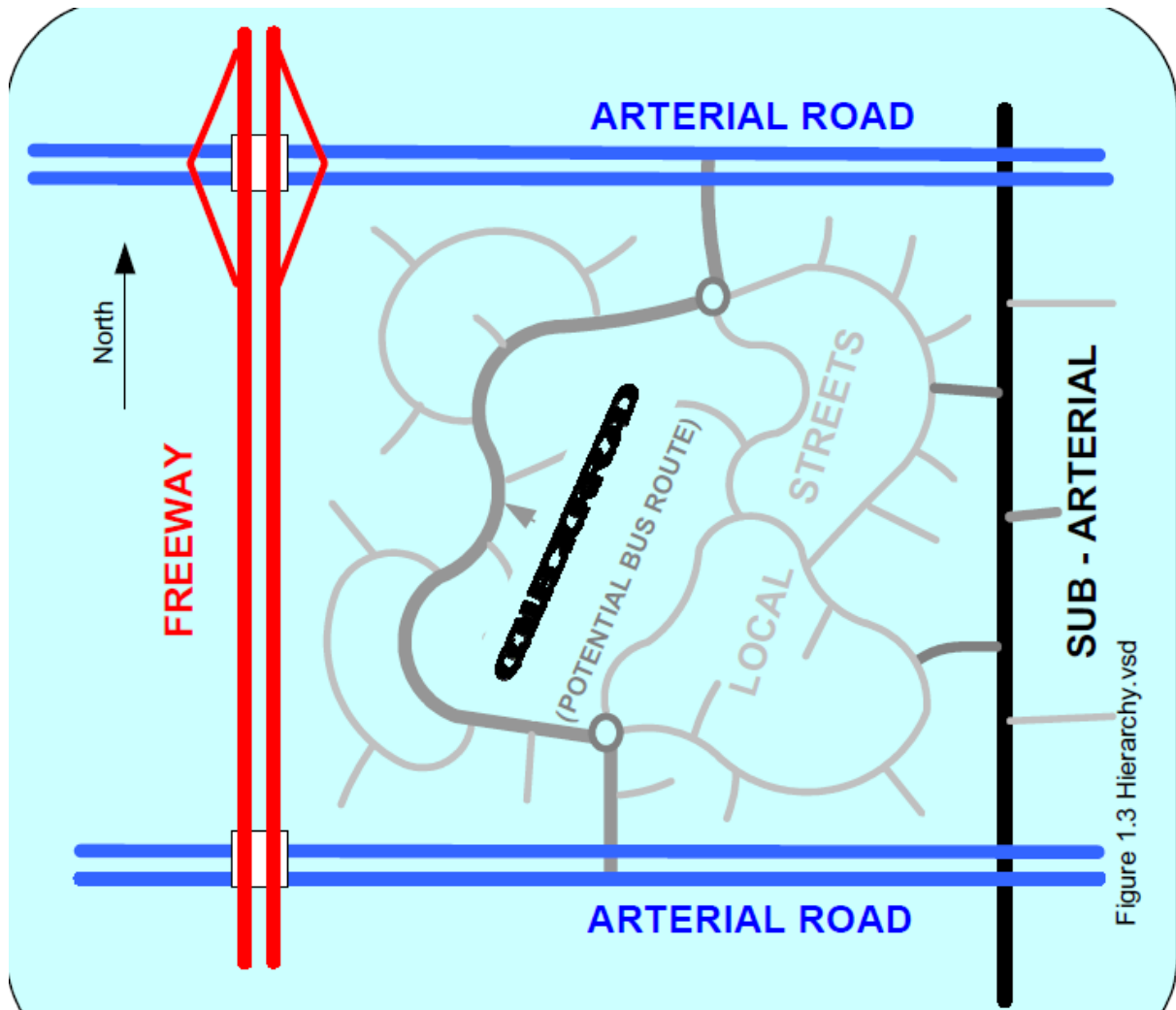


Figure 3-1: Road Classification (AACRA)

3.2 Terrain Classification

The geometric design elements of a road depend on the transverse terrain through which the road passes.

3.2.1 Flat

- Transverse terrain slope less than 5 percent.



Figure 3-2 Flat Terrain (AAiT)

3.2.2 Rolling

- Transverse terrain slope from 5 percent to 25 percent.



Figure 3-3 Rolling Terrain (AAiT)

3.2.3 Mountainous

- ✓ This class of terrain imposes definite restrictions on the standard of alignment obtainable and often involves long steep grades and limited sight distance
- ✓ Transverse terrain slope from 25 percent to 50 percent.



Figure 4-4 Mountainous (AAiT)

3.2.4 Escarpment

- If the standards associated with each of the above terrain types cannot be met. We refer to escarpment.
- The earthwork quantities are considerable, with transverse terrain slope in excess of 50 percent).



Figure 4-5 Escarpment Terrain (AAiT)

3.3 Fitting the road to the terrain

GENERAL

A visually attractive and physically unobtrusive road will be one that integrates well the environment through which it passes.

A general rule for designs is to achieve a flowing line, with a smooth and natural appearance in the land and a sensuous rhythmic continuity for the driver this effect results from following the natural contours of the land using graceful and gradual horizontal and vertical transitions and relating the alignment to permanent features such as rivers and mountains (aesthetics in transportation US DOT, 1980)

The opposite of occurs with designs using a combination of long tangents with short curves whether in the horizontal or vertical plane.

In urban areas, the type of surroundings will depend on the land use of the areas through which the alignment passes. The hard lines of an industrial area will be complimented more by the design which incorporates features such as retaining walls and other structures more in keeping with that urban form. The urban form of the area being traversed will dictate the detail of the design whatever that form. A length of urban road may call for several changes in the design concept as the road passes from one form to another.

Table 4-2: Terrain Classification of The Road

Station	left elv	right elv	distance	slope	Terrain
0+0	2186.2	2185.6	50	1.2	FLAT
0+20	2187.2	2187	50	0.4	FLAT
0+40	2188.8	2187.5	50	2.6	FLAT
0+60	2189.9	2188	50	3.8	FLAT
0+80	2190	2189.2	50	1.6	FLAT
0+100	2191	2189.8	50	2.4	FLAT
0+120	2192	2191	50	2	FLAT
0+140	2193	2191.7	50	2.6	FLAT
0+160	2194.2	2193.1	50	2.2	FLAT
0+180	2194.7	2194	50	1.4	FLAT
0+200	2196	2194.6	50	2.8	FLAT
0+220	2197	2195.6	50	2.8	FLAT
0+240	2205.5	2202.7	50	5.6	ROLLING
0+260	2205.5	2207.7	50	4.4	FLAT
0+280	2205.5	2202.7	50	5.6	ROLLING
0+300	2206.2	2202.4	50	7.6	ROLLING
0+320	2205.5	2202.7	50	5.6	ROLLING
0+340	2205.5	2202.5	50	6	ROLLING
0+360	2205.5	2202.5	50	6	ROLLING
0+380	2205.5	2202.8	50	5.4	ROLLING
0+400	2205.5	2202.5	50	6	ROLLING
0+420	2205.5	2202.3	50	6.4	ROLLING
0+440	2205.5	2202.5	50	6	ROLLING
0+460	2205.5	2202.2	50	6.6	ROLLING
0+480	2205.5	2203	50	5	ROLLING
0+500	2205.5	2203	50	5	ROLLING
0+520	2205.5	2203.2	50	4.6	FLAT
0+540	2206.2	2203.7	50	5	ROLLING
0+560	2206.2	2203.7	50	5	ROLLING
0+580	2204.9	2203.8	50	2.2	FLAT
0+600	2204.9	2203.8	50	2.2	FLAT

0+620	2207.2	2203.8	50	6.8	ROLLING
0+640	2206.7	2203.9	50	5.6	ROLLING
0+660	2206.5	2204.1	50	4.8	FLAT
0+680	2206.8	2204.4	50	4.8	FLAT
0+700	2207.1	2203.9	50	6.4	ROLLING
0+720	2208	2203.8	50	8.4	ROLLING
0+740	2206.9	2203	50	7.8	ROLLING
0+760	2206.7	2202.6	50	8.2	ROLLING
0+780	2206.9	2202	50	9.8	ROLLING
0+800	2206.7	2202.5	50	8.4	ROLLING
0+820	2206.5	2202.5	50	8	ROLLING
0+840	2206.5	2202.5	50	8	ROLLING
0+860	2206.5	2202.9	50	7.2	ROLLING
0+880	2206.7	2202.1	50	9.2	ROLLING
0+900	2206.8	2201.8	50	10	ROLLING
0+920	2206.5	2201.6	50	9.8	ROLLING
0+940	2206.6	2202.1	50	9	ROLLING
0+960	2206.1	2202.5	50	7.2	ROLLING
0+980	2205.2	2203	50	4.4	FLAT
1+000	2204.3	2202.3	50	4	FLAT
1+020	2203.9	2202	50	3.8	FLAT
1+040	2206	2200.8	50	10.4	ROLLING
1+060	2205.7	2200	50	11.4	ROLLING
1+080	2206	2200	50	12	ROLLING
1+100	2206	2202	50	8	ROLLING
1+120	2203.9	2202	50	3.8	FLAT
1+140	2207.5	2202.5	50	10	ROLLING
1+160	2208	2201.8	50	12.4	ROLLING
1+180	2207.9	2201	50	13.8	ROLLING
1+200	2208.9	2199	50	19.8	ROLLING
1+220	2206.7	2202.1	50	9.2	ROLLING
1+240	2206.5	2202.7	50	7.6	ROLLING
1+260	2206.7	2202.8	50	7.8	ROLLING
1+280	2206.8	2203	50	7.6	ROLLING
1+300	2206.7	2203	50	7.4	ROLLING
1+320	2206.6	2203	50	7.2	ROLLING
1+340	2206.4	2203	50	6.8	ROLLING
1+360	2205.9	2202	50	7.8	ROLLING
1+380	2205.5	2202	50	7	ROLLING
1+400	2204.3	2201	50	6.6	ROLLING
1+420	2203	2200.5	50	5	ROLLING
1+440	2201	2200	50	2	FLAT
1+460	2201.1	2203	50	3.8	FLAT
1+480	2201	2205	50	8	ROLLING

1+500	2203.4	2206	50	5.2	ROLLING
1+520	2203	2207	50	8	ROLLING
1+540	2204.2	2208.2	50	8	ROLLING
1+560	2205	2209.3	50	8.6	ROLLING
1+580	2206	2210.5	50	9	ROLLING
1+600	2207	2211.6	50	9.2	ROLLING
1+620	2209.2	2212.5	50	6.6	ROLLING
1+640	2210.1	2213.5	50	6.8	ROLLING
1+660	2211	2214.3	50	6.6	ROLLING
1+680	2212.8	2215	50	4.4	FLAT
1+700	2214	2217	50	6	ROLLING
1+720	2215.9	2218.2	50	4.6	FLAT
1+740	2218.2	2219.5	50	2.6	FLAT
1+760	2219.7	2220.8	50	2.2	FLAT
1+780	2221	2222.4	50	2.8	FLAT
1+800	2222.3	2222.5	50	0.4	FLAT
1+820	2223.5	2223.5	50	0	FLAT
1+840	2224.3	2224.3	50	0	FLAT
1+860	2224.9	2225	50	0.2	FLAT
1+880	2225.7	2225.8	50	0.2	FLAT
1+900	2226.3	2226.5	50	0.4	FLAT
1+920	2227	2227.2	50	0.4	FLAT
1+940	2227.5	2227.2	50	0.6	FLAT
1+960	2228.5	2228.3	50	0.4	FLAT
1+980	2228.8	2229	50	0.4	FLAT
2+000	2229.5	2229.1	50	0.8	FLAT
2+020	2230.1	2229.5	50	1.2	FLAT
2+040	2230.2	2229.8	50	0.8	FLAT
2+060	2231.4	2229.8	50	3.2	FLAT
2+080	2231.5	2231.5	50	0	FLAT
2+100	2232.5	2231.5	50	2	FLAT
2+120	2232.5	2231.5	50	2	FLAT
2+140	2233.2	2230.5	50	5.4	ROLLING
2+160	2233	2231.5	50	3	FLAT
2+180	2233	2231.8	50	2.4	FLAT
2+200	2233.5	2232	50	3	FLAT
2+220	2233.9	2232.2	50	3.4	FLAT
2+240	2233.9	2233.4	50	1	FLAT
2+260	2233.5	2233.5	50	0	FLAT
2+280	2234.2	2233.7	50	1	FLAT
2+300	2234.2	2234.2	50	0	FLAT
2+320	2234	2234	50	0	FLAT
2+340	2234	2234	50	0	FLAT
2+360	2234	2234	50	0	FLAT

2+380	2234	2234	50	0	FLAT
2+400	2234	2234	50	0	FLAT
2+420	2235.2	2234.2	50	2	FLAT
2+440	2235.4	2234	50	2.8	FLAT
2+460	2235.3	2233.7	50	3.2	FLAT
2+480	2235.3	2233.9	50	2.8	FLAT
2+500	2235.5	2234	50	3	FLAT
2+520	2235.8	2234.2	50	3.2	FLAT
2+540	2235.8	2234.2	50	3.2	FLAT
2+560	2235.8	2230.7	50	10.2	ROLLING
2+580	2235.3	2231.4	50	7.8	ROLLING
2+600	2234.5	2232.2	50	4.6	FLAT
2+620	2233.9	2232.6	50	2.6	FLAT
2+640	2233.5	2232	50	3	FLAT
2+660	2233	2231.5	50	3	FLAT
2+680	2233	2231.6	50	2.8	FLAT
2+700	2232.5	2231	50	3	FLAT
2+720	2232.1	2230.5	50	3.2	FLAT
2+740	2231.5	2230	50	3	FLAT
2+760	2230.8	2229.8	50	2	FLAT
2+780	2230.1	2228.8	50	2.6	FLAT
2+800	2230.4	2228	50	4.8	FLAT
2+820	2230.5	2226.8	50	7.4	ROLLING
2+840	2229	2225.9	50	6.2	ROLLING
2+855	2228.5	2225.5	50	6	ROLLING

From the data given above the terrain of the road is classified as FLAT

3.3 Selection of design standards

Here we do not have design standard class in AACRA manual

CHAPTER FOUR

Geometric Design

Geometric design in a highway design, deals with the visible dimensions of a road way. Properly designed highway geometries provide optimum efficiency in the operation of traffic with safety.

Categories of geometric design

- 1) vertical and horizontal alignment
 - 2) sight distance
 - 3) cross sectional elements
- **Design Control Factors;** for the above categories, there should be a design control to satisfy them. These controls are the following. (AAIT lecture note)
- Topography
 - Design speed
 - Traffic factor
 - Volume and capacity traffic
 - Environmental factor

Topography/Terrain classification

The geometric design elements of a road depend on the transverse terrain through which the road passes. Transverse terrain properties are categorized into four classes as shown in the above section of this thesis, they are Flat, Rolling, Mountainous, and Escarpment.

Design speed

It is the most important factor controlling the geometric design element of a highway. Design speed is a speed determined for design and correlation of the physical features of a highway that influences vehicular operation. It is the maximum safe speed that can be obtained over a specified section of a highway when conditions are so favourable that the design feature of the highway governs. Different speed standards are assigned for different classes of the road. The choice of a design speed depends on primarily on the surrounding terrain and the functional class of the highway. In our case the design speed is 60km/hr since the road type is collector road.

Other factors determining the selection of design speed include traffic volume and composition, costs of right of way and construction, and aesthetic considerations.

The speed that a driver adopts on a highway depends on factors such as:

- Physical characteristics of a high way and its surrounding
- Weather condition in the area.
- Presence of other vehicles and their nature.
- Speed limitations place up on the vehicle either by law or mechanical devices.

Traffic factor Vehicular characteristics related to cars, buses trucks, motor cycles all have different speed and acceleration characteristics. The same traffic behaviour is also affected by physical, mental and psychological characteristics of drivers and pedestrians.

Volume and capacity factor A highway is designed for not the traffic flow but slighter lower values, so a reasonable value of traffic volume is decided by multiplying the peak flow with a factor.

Environmental factor Landscaping, air pollution and aesthetic conditions also affect road geometries.

4.1 Route Selection and Highway Alignment

4.1.1 Route selection

Always route selection is needs perfect engineering decision and experience in highway design unless **operational** and **construction** cost of the project will be increased, generally we consider the following factor which affect route selection.

Special route

There are special routes designated to serve specific purposes as shown below.

Strategic highway corridor Network (STRAHNET)

The United States Department of Defense has a program called Highways for National Defense (HND) to ensure the mobility of united state force during national defense operations. To support this program a strategic highway corridor network (STRAHNET) WAS established. the (STRAHNET) includes highway which are important to the United States strategic defense policy and which provide defense access continuity and emergency capabilities for the movement of personal material and equipment in both peacetime and wartime (NYSDOT, 2017)

Route selection depends on:

1. Construction materials
2. Sub-grade conditions
3. Cuts and embankments
4. Areas of instability
5. Erosion and soft ground
6. Requirements for frequency and size of culverts
7. Bridge sites (AAIT, Lecture Note)

- But in our case route selection is not an issue since our project is already existing road.

LEVEL OF SERVICE AND HIGHWAY CAPACITY

Level of service' describes in a qualitative way the operational conditions for traffic from the viewpoint of the road user. It gauges the level of congestion on a highway in terms of variables such as travel time and traffic speed.

- **Driver Population.** Under ideal conditions, a driver population consisting primarily of weekday commuters is assumed. However, it is known that other driver populations do not exhibit the same behavior.
- **Grade.** The effect of a grade depends on both the length and the slope of the grade. Traffic operations are significantly affected when grades of 3 percent or greater are longer than 1/4 mi (400m) and when grades are less than 3 percent and longer than 1/2 mi (800m). The effect of heavy vehicles on such grades is much greater than that for passenger vehicles.
- **Lane Width.** Traffic flow tends to be restricted when lane widths are narrower than 12 ft (3.65m). This is because vehicles have to travel closer together in the lateral direction, and motorists tend to compensate for this by driving more cautiously and by increasing the spacing between vehicles, thus reducing the maximum flow on the highway.
- **Lateral Obstruction.** In general, when roadside or median objects are located too close to the edge of the pavement, motorists in lanes adjacent to the object tend to shy away from the object, resulting in reduced lateral distances between vehicles. This lateral reduction in space also results in longer spacing's between vehicles and a reduction in the maximum flow on the highway. This effect is eliminated if the object is located at least 6ft (1.8m) from the edge of the roadway. Note, however, that lateral clearances are based mainly on safety considerations and not on flow consideration.
- **Speeds, Space mean speed,** are also used in level-of-service analysis because flow has a significant effect on speed.
 1. **Road way characteristics:** This are associated with the geometric characteristics and design elements of the facility, which include type of facility, number of lanes, lane width, shoulder width, horizontal and vertical alignments, lateral clearance, design speed, and availability of queuing space at intersections. For example, a curved road has lesser capacity compared to a straight road.

Capacity is independent of the demand. It speaks about the physical amount of vehicles and passengers that a road can afford. It does not depend on the total number of vehicles demanding service. Generally the highway capacity depends on certain conditions as listed below; convenience have decreased perceptibly at this level

Factors affecting level of service

1. Speed and travel time

2. Traffic conditions: Capacity is expressed in terms of units of some specific thing (car, people, etc.), so it also does depend on the traffic conditions. The traffic conditions are associated with the characteristics of the traffic stream on the segment of the highway. These include the distribution of the different types of vehicles in the traffic stream or traffic composition such as the mix of cars, trucks, buses etc. and the directional and lane distribution of the traffic volume on the highway segment. Furthermore, it includes peaking characteristics, proportions of turning movements at intersections etc.

2. Traffic interruptions/restrictions

3. **Control conditions:** This primarily applies to surface facilities and includes the types of traffic control devices in operation, signal phasing, allocation of green time, cycle length, and the relationship with adjacent control measures.

3. Freedom to travel with desired speed

4. Driver comfort and convenience

5. Operating cost.

A term level-of-service closely related to capacity and often confused with it is service volume. When capacity gives a quantitative measure of traffic, level of service or LOS tries to give a qualitative measure. Service volume is the maximum number of vehicles, passengers, or the like, which can be accommodated by a given facility or system under given conditions at a given level of service.

Approximately 50% of its maximum value. Level of service F denotes the 'breakdown' condition at which both speeds and flow levels tend towards zero.

Determining the capacity and LOS of a highway

Factors such as lane width, lateral obstruction, traffic composition, grade and driver population also affect the maximum flow on a given highway segment. The effect of each of these factors on flow is discussed.

For a given road or facility, capacity could be constant. But actual flow will be different for different days and different times in a day itself. The intention of LOS is to relate the traffic service quality to a given flow rate of traffic. It is a term that designates a range of operating conditions on a particular type of facility. Highway capacity manual (HCM) provides some procedure to determine level of service. It divides the quality of traffic into six levels ranging from level A to level F. Level A represents the best quality of traffic where the driver has the freedom to drive with free flow speed and level F represents the worst quality of traffic.

Highway Capacity

HIGHWAYS CAPACITY AND LEVEL OF SERVICE

Hourly volume (V) The highest hourly volume within a 24-hour period

In order to determine a road's level of service, a comprehension of the relationship between hourly volume, peak hour factor and service flow is vital:

Level of service (LOS) qualitatively measures both the operating conditions within a traffic system and how these conditions are perceived by drivers and passengers. It is related with the physical characteristics of the highway and the different operating characteristics that can occur when the highway carries different traffic volumes. Speed-flow-density relationships are the principal factor affecting the level of service of a highway segment under ideal conditions

Level of service A

Level of service B

Level of service C

Level of service D

Level of service E

Level of service F

Level of service A: This represents free-flow conditions where traffic flow is virtually zero. Only the geometric design features of the highway may limit the speed of the car. Comfort and convenience levels for road users are very high as vehicles have almost complete freedom to maneuver.

Service B: Represents reasonable free-flow conditions. Comfort and convenience levels for road users are still relatively high as vehicles have only slightly reduced freedom to maneuver. Minor accidents are accommodated with ease although local deterioration in traffic flow conditions would be more discernible than in service A.

Service C: Delivers stable flow conditions. Flows are at a level where small increases will cause a considerable reduction in the performance or 'service' of the highway. There are marked restrictions in the ability to maneuver and care is required when changing lane. While minor incidents can still be absorbed, major incidents will result in the formation of queues. The speed chosen by the driver is substantially affected by that of the other vehicles.

Level of service D: The highway is operating at high-density levels but stable flow still prevails. Small increases in flow levels will result in significant operational difficulties on the highway. There are severe restrictions on a driver's ability to maneuver, with poor levels of comfort and convenience

Level of service E: Represents the level at which the capacity of the highway has been reached. Traffic flow conditions are best described as unstable with any traffic incident causing extensive queuing and even breakdown. Levels of Basic Elements of comfort and convenience are very poor and all speeds are low if relatively uniform.

Level of service F: Describes a state of breakdown or forced flow with flows exceeding capacity. The operating conditions are highly unstable with constant queuing and traffic moving on a 'stop-go' basis.

The capacity of a facility defined as the maximum hourly flow rate at which the maximum number of vehicles, passengers, or the like, per unit time, which can be accommodated under prevailing roadway, traffic and control conditions with a reasonable expectation of occurrence. For most cases, to analyze the capacity we used the peak 15 minutes of the peak hour.

- **Traffic Composition.** The presence of vehicles other than passenger cars-such as trucks, buses, and recreational vehicles-in a traffic stream reduces the maximum flow on the highway because of their size, operating characteristics, and interaction with other vehicles.

LOS determination

The input data that we have:

Table 4-1 Counted one Directional Peak hour volume

Vehicle class	V ₁₅ (1 ST MIN)	V ₁₅ (2 ND MIN)	V ₁₅ (3 RD MIN)	V ₁₅ (4 TH MIN)
Car	103	90	114	76
Truck	2	3	4	3
Total	105	93	118	79

Total volume (V_t) =100+168+222+79=395veh

For Multi lane highway

$$\text{Percentage of truck (PT)} = \frac{\text{NO OF TRUCK}}{\text{TOTALNOVEHICLE}} = \frac{12}{372} = 2.96\%$$

Recreational vehicle (TR) = 0

$$\text{Adjustment factor for heavy vehicle FHV} = \frac{1}{1 + \text{PT} \cdot \text{ET} - 1 + \text{TR}(\text{ER} - 1)}$$

Adjustment factor for truck and buses (ET)= will be obtained by interpolation principle:

1 mile =1.61km↔2.711km9project road length) =1.684mil

PT===ET

2 5

2.77 x

4 4

$$\leftrightarrow \frac{(4-2)}{4-2.76} = \frac{(4-5)}{4-x} \dots X=4.62$$

ET= and substituting

$$\text{FHV} = \frac{1}{1 + 0.0296 \cdot 4.62 - 1 + 0(\text{ER} - 1)} = 0.903$$

$$\text{Peak hourly factor (PHF)} = \frac{\text{VT}}{\text{V}_{15\text{max}} \times 4} = \frac{372}{110 \times 4} = 0.845$$

$$\text{Daily design hourly volume (DDHV}_{15}) = \text{DDHV}_{15} \cdot (1+r)^{15}$$

$$= 372(1+0.06)^{15} = 891.5 \quad \text{Use } 892 \text{ veh/lane}$$

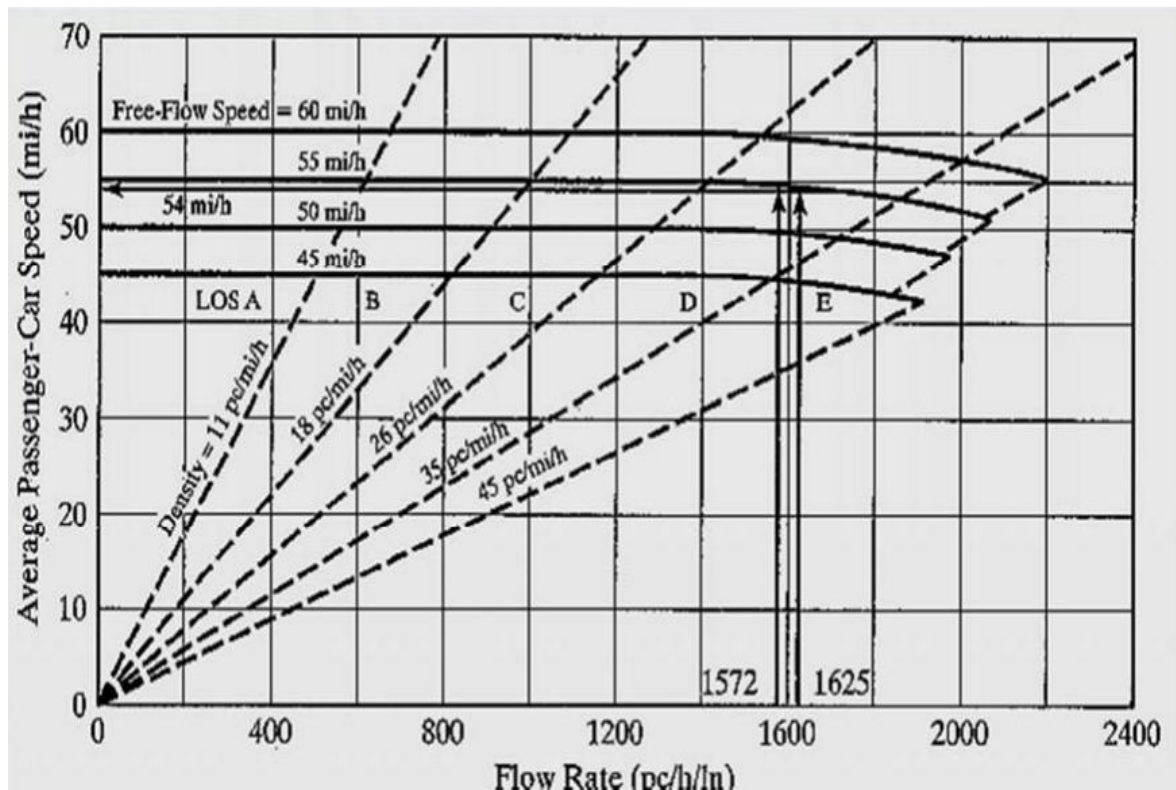


Figure 4-1 Level of Service Chart

fp=1 assuming that our project road is familiar for driver & assuming that the level of service at the end of 15 year be level of service "E"

Maximum service

$$speed(FFS) = 48.83 \text{ km/hr} = 30.33 \text{ mi/hr}$$

For level of service "E" and average Passenger Car speed of 48.83km/hr the value of flow Rate (MSFi) is 1380ve/h/l from the above chart

Now let us compute for number of lane

$$N = \frac{DDHV}{PHF * MSFI * FHV * fP} = \frac{892}{0.845 * 1380 * 0.903 * 1}$$

=0.85 lane Use N=1 lane (in single direction)

Recommendation: we recommend one additional lane in order to provide parking lane.

4.1.2 ALIGNMENT

The overall quality and appearance of a road will be determined by the quality of the alignment design (horizontal and vertical)and its relationship to the surrounding environment.

An important concept in highway design is that every project is unique. The setting and characteristics of the area, the values of the community, the needs of the highway users, and the challenges and opportunities are unique factors that designers must consider with each highway project. for each potential project, designers are faced with the task of balancing the need of the

highway improvement with the need to safely integrate the design into the surrounding natural and human environments (Flexibility In High Way Design, 1997)

A holistic approach to the design is required where the various elements of the road design are considered as a whole in the context of the environment through which the road passes and the expectations of the community affected by the road. To achieve the best result, designers must consider the engineering requirements of the road in question together with the characteristics of the area in which the road is located. The designer must consider the character of the area in the context of its physical location and give proper weight to: -

- City master plan
- Physical characteristics of the area including property access requirements
- Use of the corridor (destination spots, pedestrians, cycles, industry, urban residential etc.)
- The type and scale of the road to be design
- Vegetation in the corridor (density, type)
- Views available, scenic values
- Historic features (including cultural heritage)
- Features to preserve (including natural environment)
- Existing roads in the corridor and the provision of intersections (if an at grade road) or road closures, grade separations or interchanges (if a freeway)

Designers need to employ imagination, ingenuity and flexibility to get the best results. It is important that all of the necessary information on both physical constrains and a community expectation is obtained early in the process. It's also important that the special expertise of landscape architect environmental specialists and others is employed at the beginning before controlling decisions are made and changes can still be adopted. ((AACRA), 2004)

This section is devoted to the design of the horizontal and vertical alignments as a coordinated whole in harmony with the landscape, the environment and the community in general ((AACRA), 2004)

Design principles

In engineering terms road alignment have to service traffic in terms of providing a route that meets constraints imposed by vehicle dynamics occupant comfort and topography. This means that most road alignments are in fact complex three dimensional splines that do not have a simple mathematical definition. ((AACRA), 2004)

This problem has historically been addressed by reducing the three-dimensional alignment to two dimensional alignments. In each case the alignments are made up of geometric elements that are convenient to calculate and construct yet still ensure that vehicle dynamics constrains are met. ((AACRA), 2004)

Even with the advent of computers this approach has still proven to be the most convenient for design and construction. To use complex three-dimensional models would introduce unnecessary complexity into the process. Because horizontal and vertical geometry are calculated independently,

it is necessary to ensure that the two alignments are properly coordinated and complement each other. ((AACRA), 2004)

4.2 Horizontal Alignment

Horizontal alignment which comprises a series of intersecting tangents and circular curves (with or without transition curves) is a most important feature affecting the safety, efficiency and cost of a road. Several studies have indicated that highway curves exhibit higher accident rates than tangent sections and that the accident rate increase as the radius of curvature is decreased. Generally, vehicle operating speeds decreases as the overall horizontal curvature increases thus road user costs are affected by the bendiness of a road. Also speeds on individual curves are lowered as their radii are reduced while construction costs usually increase as radii increases (especially in hilly terrain). (O'Flaherty, 2003)

In addition, in horizontal alignment, design should be consistence to the topography, provide good drainage system and minimize earth work. And the alignment selected for particular project should run without sudden changes from easy to sharp curves.

The design elements of the horizontal alignment are the following

1. Simple horizontal (Un- transition) curve and transition curve (**spiral**)
2. Tangent or straight section.
3. Super elevation section.

4.2.1 Tangent Section

From an aesthetical point of view, tangent sections may often be beneficial in flat terrain but are less so in rolling or mountainous terrain. From a safety standpoint, they provide better visibility and more passing opportunities. And it is a part of road alignment where we don't have both horizontal and vertical curves and it is always not exceeding **4km** to avoid accident from fatigue of the driver and headlight glare.

4.2.2 Horizontal Circular Curves

Simple horizontal curves

The horizontal alignment consists of straight section of the road(known as tangent)connected by curves. The curves are usually segments of circles,whiche have radii that will provide for a smooth flow of traffic. The design of the horizontal alignment entails the determination of the minmum radius, determination of length of the curve ,and the computation of of the horizontal offsetes from the tangent to the curve facilitat locating the curve in the field,in some cases,to avoid a sudden change from a tangent with infinite radius to a curve of finiat radius, a curve with radii varying from infinite to the radius of the circular curve is placed between the circular curve and the tangent.Such a curve is known as a spiral or transition curve. There are four types oe horizontal curve, simple, compound, reversed, and spiral. (Garber, Traffic & Hiwghway Engineering, 2009)

SIGHT DISTANCES

Sight distance is the length of roadway ahead visible to the driver. The minimum sight distances Available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. (AAIT, Lecture Note)

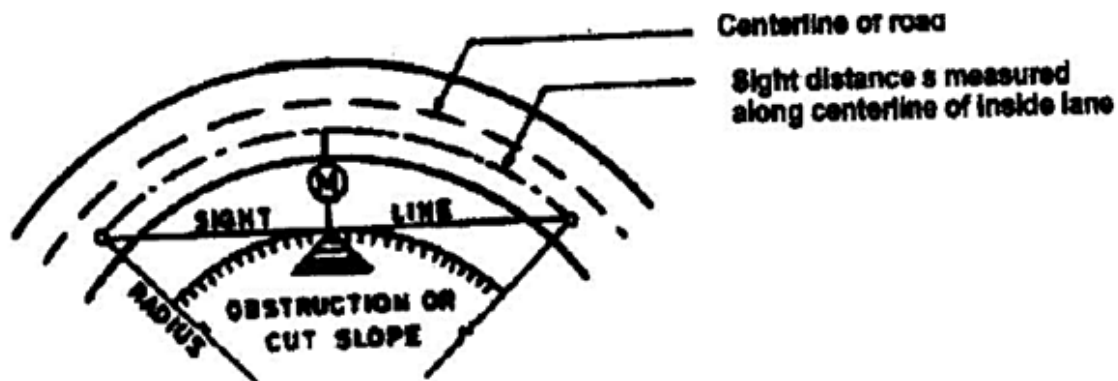


Fig-2 sight distance (AAIT, Lecture Note)

$$\text{SightLine } S = 2R * \sin\left(\frac{\Delta}{2}\right)$$

$$\text{Middleordinate } (M) = R \left(1 - \cos\left(\frac{\Delta}{2}\right)\right)$$

There are two sight distances that should be maintain in roads

➤ Stopping Sight Distance

There are three types of stopping sight distance to consider. These are stopping sight distance for crest vertical curves, stopping sight distance for sag vertical curves under bridges or other vertical sight obstruction (also called "headlight sight distance"), and stopping sight distance for horizontal curves. Sag vertical curve sight distance is not a critical design element where sight lines are unrestricted. (NYSDOT, 2017)

Stopping Sight Distance (SSD)

$$SSD = D1 + D2$$

Where

$$D1 = \text{Reaction distance (m)} = RT * V/3.6$$

$$D2 = \text{Breaking distance (m)} = \frac{V_2}{254 F1 + 0.01G}$$

G = Longitudinal grade % (Positive for up grading, Negative for down grading)

RT = Reaction time (sec) = 2.5sec

V = Design speed

$F1$ =longitudinal friction factor

➤ Overtaking/passing/ Sight Distance

Overtaking Sight Distance is of relevance on two way roads in rural areas. In urban areas, overtaking sight distance is not usually a design consideration on Local Street. Arterial roads and freeways are normally dual carriageway or multi-lane, and overtaking sight distance is again not an issue. However, the initial development of a freeway or arterial road may involve the construction of a single carriageway operated two-way until traffic volumes increase and duplication is necessary. Table 9.3.5.A provides guidelines for use in this case. Allowing overtaking sight distance will increase the capacity of the initial two-way road, but any additional cost of achieving better sight distance would need to be considered. ((AACRA), 2004)

Sight Distance on Horizontal Curves: - Sight Distance on Horizontal Curves is frequently an issue in urban road design. This is due to the need to use curves of relatively low radius to meet site constraints on the road alignment. These curves may involve the sight line crossing the verge or residual median.

If this occurs, any road furniture, landscape planting, structure or batter will need to be located clear of the sight line. ((AACRA), 2004)

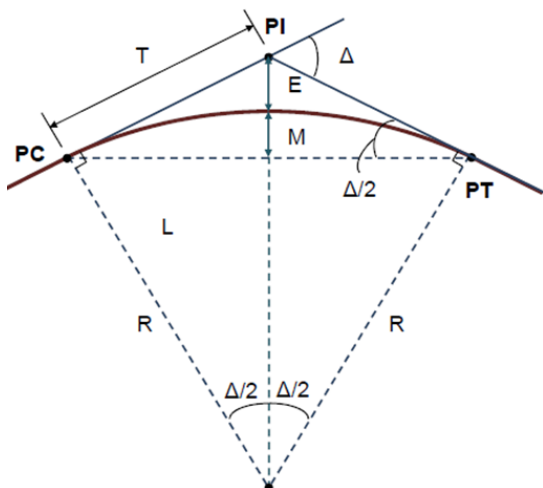
Table 4-2: overtaking sight distance for two way roads (AACRA)

Design Speed	Overtaking Sight Distance
60	300
70	350
80	450
90	600
100	750
110	900

Horizontal Curves Design

Minimum Curve Radius Determination

The minimum curve radius that is required to keep a vehicle moving in a certain speed on a circular path is equal to



$$R_{min} = \frac{V^2}{127 e_{max} + f_{max}}$$

Where, R = radius of curve

V = design speed

e_{max} = max cross fall

f_{max} = max side friction

Horizontal Curve Formulas

Figure 4-2: Simple Horizontal Curve Elements

$$T = R \tan \Delta / 2 \quad M = R (1 - \cos \Delta / 2)$$

$$C = 2R \sin \frac{\Delta}{2} \quad L_c = R * \Delta_{rad}$$

$$E = R \frac{1}{\cos \frac{\Delta}{2}} - 1 \quad PC = PI - T$$

$$PT = PC + L_c$$

The deflection angles obtained from **inroad** software reading.

Table 4-3 Designed Horizontal curves of the road

Curve No	Angel	radius	PI (Point of INTERSECTION)	Remark
1	17.2°	333.110	0+652.892	Simple curve
2	24.54°	230.082	1+038.847	Simple curve
3	115.52°	128.8	1+452.33	Spiral curve
4	20.30°	279.435	1+808.869	Simple curve
5	11.23	503.11	2+342.33	Simple curve

The detail analysis and design of the horizontal curves is given below for each curve.

CURVE 1

$$\Delta^\circ = 17.2^\circ \quad PI = 0 + 652.892e = 5\%, f = 0.17$$

$$R = \frac{V^2}{127(e+f)} \quad \text{Where } V = V_{sd} = 60 \text{ Km/h}$$

$f = 0.17$ Coefficient of side friction for truck according to **AACRA** manual which will be safe for car

$e = 5\%$ For safety, appearance and comfort

Therefore

$$R_{min} = \frac{60^2}{127(0.05 + 0.17)} = 128.8$$

$$L_c = R * \Delta_{rad}$$

$$L_c = R * \Delta^\circ * \frac{\pi}{180}$$

$$L_c = 128.7m * 17.2^\circ * \frac{\pi}{180}$$

$$L_c = 38.646m$$

But AACRA requires a minimum curve length of 100m

Therefore, provide minimum curve length of

$$L_c = 100m$$

Revise all horizontal curve elements with $L_c = 100m$

$$L_c = R * \Delta * \pi/180$$

$$100 = R * \Delta * \pi/180$$

$$R = \frac{100}{17.2 * \frac{\pi}{180}}$$

$$R = 100 * \frac{180}{17} * \pi$$

$$R = 333.28$$

$$R = \frac{V^2}{127 * e + f}$$

$$v = \sqrt{(* 127 * (e + f))}$$

$$V = \sqrt{333.28 * 127 * (0.05 + 0.17)} = 96.49km/h$$

The road has the above speed capacity but it's not used because since the area is residential, and the road class is collector, the speed will not be a safe speed.

$$T = R * \tan \Delta^\circ/2$$

$$T = 333.28m * \tan \frac{17.2^\circ}{2} = 50.4m$$

$$Pc = PI - T = 0 + 652.892 - 50.4 = 0 + 602.49$$

$$PT = PI + L = 0 + 602.49 + 100 = 0 + 702.49$$

$$m = 333.28m * 1 - \cos \frac{17.2^\circ}{2}$$

$$m = 3.747$$

$$E = 333.28m * \frac{1}{\cos \frac{17.2^\circ}{2}} - 1 = 3.79m$$

Calculate **stopping sight distance** and compare m provided with m_{min}

$$SSD = D1 + D2$$

$$D1 = V * Tr / 3.6$$

$$D1 = 60 * 2.5 / 3.6$$

$$D1 = 41.667m$$

$$D2 = \frac{V^2}{254 f + 0.01G}$$

Where: -

F is longitudinal friction coefficient for truck that will be safe for cars = 0.29

G is max. Grade (−4.072) from geometric alignment

$$D2 = \frac{60^2}{254 \cdot 0.29 - 0.01 * 4.072}$$

$$D2 = 56.63$$

$$SSD = 41.667 + 56.63$$

$$SSD = 98.3m$$

To find the minimum ordinate (m) that is adequate for the provision of the minimum sight distance (SSD) is

$$m_{min} = R(1 - \cos(\frac{SSD * 28.65}{R}))$$

$$m_{min} = 333.28 * (1 - \cos(\frac{98.3m * 28.65}{333.28})) = 3.7m$$

$$m_{min} = 3.7m < 3.74 \dots OK! M = 3.7m$$

Curve setting out

- for the first station on the curve, $l = 0 + 620 - 0 + 602.49 = 17.510m$
- $\delta = \frac{1}{2} * \frac{l}{R} * \frac{180}{\pi} = \frac{1}{2} * \frac{17.510}{333.28} * \frac{180}{\pi} = 1.506^\circ$

Table3-6: Curve setting out for curve 1

station	Curve length	Total length	radius	Angel	Total angle
0+620	17.15	17.51	333.28	1.506	1.506
0+640	20	37.51	333.28	1.720	3.226
0+660	20	57.51	333.28	1.720	4.946
0+680	20	77.51	333.28	1.720	6.666
0+700	20	97.51	333.28	1.72	8.386
0+702.49	2.49	100	333.28	0.214	8.6

Checking for Δ

$$\Delta = 2 * \text{total angel}$$

$$\Delta = 2 * 8.6 = 17.2 \text{ oK!!!!}$$

CURVE2

$$\Delta^\circ = 24.54, \text{PI} = 1+038.847, \quad e = 5\%, f = 0.17$$

For the same assumption as curve 1

$$R_{min} = \frac{60^2}{127(0.05 + 0.17)} = 128.7m$$

$$L_c = R * \Delta r d = R * \Delta^\circ * \frac{\pi}{180}$$

$$L_c = 128.7 * 24.54 * \frac{\pi}{180} = 55.095m$$

$$\text{But from AACRA manual, } L_{min} = 100m$$

Therefore provide $L_c = 100m$

Revise R with this value of L_c

$$L_c = R * \Delta^\circ * \frac{\pi}{180}$$

$$R = \frac{L_c}{\Delta^\circ * \frac{\pi}{180}}$$

$$R = \frac{100}{24.54 * \frac{\pi}{180}} = 233.598m$$

$$T = R * \tan(\Delta/2)$$

$$T = 233.598 * \tan (24.52/2)$$

$$T = 50.762m$$

$$Pc = PI - T$$

$$= 1 + 038.847 - 50.762$$

$$PC = 0 + 988.085$$

$$PT = PC + L$$

$$= 988.085 + 100$$

$$PT = 1 + 088.085$$

$$M = R * \left(1 - \cos \frac{\Delta}{2} \right)$$

$$M = 233.598 * \left(1 - \cos \frac{24.52}{2} \right) = 5.327m$$

$$E = 233.598m * \left(\frac{1}{\cos \frac{24.52}{2}} - 1 \right)$$

$$E = 5.452m$$

Setting out

- for the first station on the curve, $l=1+000 - 1+988.085=11.915m$
- $\delta = \frac{1}{2} * \frac{l}{R} * \frac{180}{\pi} = \frac{1}{2} * \frac{11.915}{233.598} * \frac{180}{\pi} = 1.462^\circ$

Table 4.4 Curve setting out for Curve 2

STATION	LENGHT	Total length	Radius	angle	Total angel
1+000	11.915	11.915	233.598	1.462	1.462
1+020	20	31.915	233.598	2.454	3.916
1+040	20	51.915	233.598	2.454	6.370
1+060	20	71.915	233.598	2.454	8.824
1+080	20	91.915	233.598	2.454	11.278
1+088.085	8.085	100	233.598	0.992	12.27

Checking for Δ

$$\Delta = 2 * \text{total angel}$$

$$12.27 * 2 = 24.52 \text{ OK !}$$

CURVE 4

$$\Delta^\circ = 20.30 \quad PI = 1 + 808.869 \quad e = 5\%, \quad f = 0.17$$

For the same assumptions, as above

$$R_{min} = \frac{60^2}{127(0.05 + 0.17)} = 128.7m$$

$$Lc = R * \Delta r d = R * \Delta^\circ \frac{\pi}{180}$$

$$Lc = 128.7 * 20.30 * \frac{\pi}{180} = 45.576m$$

$$\text{But from AACRA manual, } L_{min} = 100m$$

$$\text{Therefore provide } L_c = 100m$$

$$R = \frac{Lc}{\Delta^\circ * \frac{\pi}{180}}$$

$$R = 100 * \frac{180}{20.3} * \pi = 282.388m$$

$$T = R * \tan \frac{\Delta}{2} \dots T = 282.388 * \tan \frac{20.3}{2} = 50.555m$$

$$Pc = PI - T$$

$$PC = 1 + 808.869 - 50.55$$

$$PC = 1 + 758.314$$

$$PT = PI + Lc$$

$$PT = 1 + 758.314$$

$$PT = 1 + 858.314$$

$$M = R * \left(1 - \cos \frac{\Delta}{2} \right) = 282.388 * \left(1 - \cos \frac{20.3}{2} \right)$$

$$M = 4.419m$$

$$E = 282.388m * (1/\cos(20.3/2) - 1)$$

$$E = 4.490M$$

Minimum value of M for provision of minimum SSD = 98.3 which is calculated early

$$M_{min} = 282.388 * \left(1 - \cos \frac{28.65}{84.658} \right) = 3.167 < 4.419M \text{ OK!}$$

Setting out

- for the first station on the curve, $\frac{l}{-}=1+760 - 1+758.314=1.686m$
- $\delta = \frac{1}{2} * \frac{l}{R} * \frac{180}{\pi} = \frac{1}{2} * \frac{1.686}{282.388} * \frac{180}{\pi} = 0.171^\circ$

Table4-5: Curve setting out for Curve 4

SATION	LENGHT	TOTAL LENGHT	ANGLE	RADIUS	TOTAL ANGLE
1+760	1.686	1.686	0.171	282.388	0.171
1+780	20	21.686	2.03	282.388	2.201
1+800	20	41.686	2.03	282.388	4.231
1+820	20	61.686	2.03	282.388	6.261
1+840	20	81.686	2.03	282.388	8.291
1+858.314	18.314	100	1.859	282.388	10.15

Checking for Δ

$$\Delta = 2 * \text{total angel}$$

$$2 * 10.15 = 20.3 \text{ OK!}$$

FORCURE 5

$$\Delta^\circ = 11.23, \quad PI = 2 + 342.33e = 5\%, f = 0.17$$

For the same assumptions, as above

$$R_{min} = \frac{60^2}{127(0.05 + 0.17)} = 128.7m$$

$$L_c = R * \Delta r d = R * \Delta^\circ \pi / 180$$

$$L_c = 128.7 * 11.23 * \pi / 180 = 25.212m$$

$$\text{But from AACRA manual, } L_{min} = 100m$$

$$\text{Therefore provide } L_c = 100m$$

Revise R and Compute the horizontal curve elements

$$R = \frac{L_c}{\Delta^\circ * \frac{\pi}{180}}$$

$$R = 100 * \frac{180}{11.23} * \pi = 510.462m$$

$$T = R * \tan \frac{\Delta}{2}$$

$$T = 510.462 * \tan \frac{11.23}{2} = 50.186m$$

$$Pc = PI - T$$

$$PC = 2 + 342.33 - 50.186$$

$$PC = 2 + 292.144m$$

$$PT = PI + Lc$$

$$PT = 2 + 292.144 + 100$$

$$PT = 2 + 392.144m$$

$$M = R * \left(1 - \cos \frac{\Delta}{2} \right) = 510.462 * \left(1 - \cos \frac{11.23}{2} \right) = 2.449m$$

$$E = 510.462m * \left(\frac{1}{\cos \frac{11.23}{2}} - 1 \right) = 2.461m$$

$$M_{min} = 510.462 * \left(1 - \cos 84.658 \right) * \frac{28.65}{510.462}$$

$$M_{min} = 1.754 < 2.449 \dots \dots \dots OK!$$

Setting out

Sample calculation of circular curve: curve 5 setting station at 20m interval

- for the first station on the curve, $L = 2+300 - 2+292.144 = 7.856m$
- $\delta = \frac{1}{2} * \frac{1}{R} * \frac{180}{\pi} = \frac{1}{2} * \frac{7.856}{510.462} * \frac{180}{\pi} = 0.441^\circ$

Table4-6: Curve setting out for Curve 5

Station	length	Total length	angle	radius	Total angle
2+300	7.856	7.856	0.441	510.462	0.441
2+320	20	27.856	1.123	510.462	1.564
2+340	20	47.856	1.123	510.462	2.687
2+360	20	67.856	1.123	510.462	3.810
2+380	20	87.856	1.123	510.462	4.933
2+392.144	12.144	100	0.682	510.462	5.615

Checking for Δ

$$\Delta = 2 * \text{total angle}$$

$$2 * 5.615 = 11.23 \text{ OK!}$$

Here we recommend adverse super division because the radius is larger.

Transition curves

In practice transition curves are included in curve design, between curved and straight sections of the road if the curve radius to be used at a given design speed is less than that for a wholly circular curve without super elevation. Their main purpose is to provide a driver travelling at high speed with an easy to follow path so that the radial acceleration increases and decreases gradually as the

vehicles enters and leaves the circular curve. Super elevation or the elimination of adverse camber, is also progressively applied on or within the length of the transition curve from the arc end; if the transition length defined by the design speed is less than that required by super elevation turnover, a longer transition is usually applied to meet super elevation needs. Transition curves also facilitate widening on curves and improve the appearance of a road. (O'Flaherty, 2003)

The essential feature of a well-designed transition curve is that its radius of curvature decreases gradually from infinity at the tangent transition curve intersection to the radius of the circular curve at the transition curve circular curve intersection. (O'Flaherty, 2003)

For curves with transitions, it is normal practice to match super elevation runoff with the transition. That is, the point of 0% cross fall corresponds to the start of the transition (for a vehicle entering the curve) and the full super elevation for the curve ($e\%$) is attained at the end of the transition. The super elevation development is extended back from the start at the rotation rate to the point of normal cross fall on the approach tangent. ((AACRA), 2004)

The theoretical basis for the uniform development of the super elevation along the length of the transition is that when combined with the uniform increase in curvature provided by the clothoid spiral transition. There will be a uniform attainment of the side friction demand that will be used on the horizontal curve. ((AACRA), 2004)

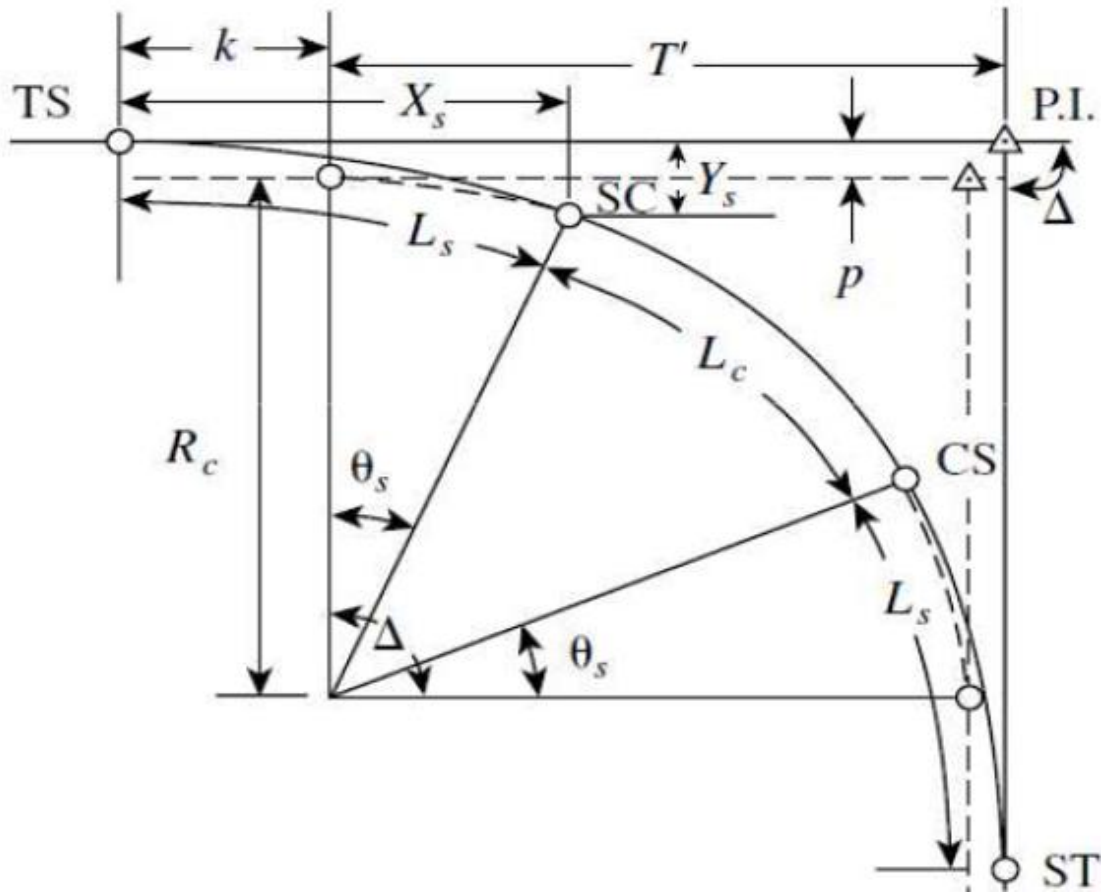


Figure 4-3: Transitional Curve Parameters

Nomenclature of spiral parts:

$$TS = PI - (T' + K)$$

$$SC = TS + LS$$

$$CS = SC + LC$$

$$ST = CS + LC$$

$$A = \overline{L_s * R}$$

$$YS = \frac{Ls^3}{6A^2} - \frac{Ls^7}{336A^6} + \frac{Ls^{11}}{42240A^{10}}$$

$$Xs = Ls - \frac{Ls^5}{40 * A^4} + \frac{Ls^9}{3456 * A^8}$$

$$K = Xs - Rc(\sin\theta_s)$$

$$T' = (R + P)\tan \Delta/2$$

$$P = YS - RC (1 - \cos \theta_s)$$

Transition (spiral) curve calculation

To get length of the transitional curve, we have the following three lengths of super elevation. The maximum of these values shall be used for design.

$$\text{based on rate of rotation cross fall}(Le1) = 0.278 * V * (e2 - e1)/r$$

Where

$$v = \text{designed speed} = 60\text{km/h},$$

$$e1 = -3\%, \text{ cross fall}$$

$e2 = 5\%$, super elevation at the ends of development length(m/m); +ve when slopping upward from the control line -ve when slopping down ward from the control line

r = rate of rotation

$$Le1 = 0.278 * 60 * \frac{0.05 - -0.03}{0.035} = 38.125m$$

$$\text{Based on relative grade } (Le2) = 100 * w * \frac{e2-e1}{Gr}$$

Where Gr =relative grade

$Gr = 1$. Because the road is two lanes from the code AACRA

$$Le2 = 100 * 3.5(0.08)/1 = 28m$$

$$Le3 = 40m \text{ from AACRA manual sec11 Table 11.4.6 B}$$

$$Le3 = 40m \text{ governs}$$

Therefore provide $Ls = 40m$

Curve 3

$$R_{min} = \frac{60^2}{127(0.05 + 0.17)} = 128.8$$

And checking the minimum curve length

$$L = \Delta * R * \frac{\pi}{180} = 115.52 * 128.8 * \frac{3.1415}{180} = 259.7 \text{ USE } 260m > 100m \text{ ...ok!}$$

For curve 3 we consider transition curve since this curve is sharp with large deflection angle to make it smooth.

$$PI = 1 + 452.330 \text{ m}$$

$$Ls = 2R\theta_s$$

$$\theta_s = \frac{Ls}{2 * R} = \frac{40}{2 * 128.8} = 0.155\text{rad} = 8.901^\circ$$

$$\Delta' = \Delta - 2 * \theta_s$$

$$\Delta' = 115.52 - 2 * (8.901) = 97.718$$

$$A = \overline{(Ls * R)} = \overline{(40 * 128.8)} = 71.777m$$

$$X = Ls - \frac{Ls^5}{40A^4} + \frac{Ls^9}{3456A^8}$$

$$X = 40 - \frac{40^5}{40 * 71.777^4} + \frac{40^9}{3456 * 71.777^8}$$

$$X = 39.904m$$

$$Y = \frac{Ls^3}{6A^2} - \frac{Ls^7}{336A^6} + \frac{Ls^{11}}{42240A^{10}}$$

$$Y = \frac{40^3}{6 * 71.777^2} - \frac{40^7}{336 * 71.777^6} + \frac{40^{11}}{42240 * 71.777^{10}}$$

$$Y = 2.066m$$

$$P = Y - R(1 - \cos \theta)$$

$$= 2.066 - 128.8(1 - \cos 8.901)$$

$$P = 0.515m$$

$$K = X - R(\sin \theta_s)$$

$$= 39.904 - 128.8 * \sin 8.901$$

$$K = 19.975m$$

$$T' = R + P \tan \frac{\Delta}{2} = 128.8 + 0.515 * \tan \frac{115.52}{2}$$

$$T' = 205.031m.$$

$$Lc = R * \Delta'_{rad} = 128.8 * 97.718 * \frac{\pi}{180} = 219.687m$$

Stationing

$$TS = PI - T' + K = 1 + 452.330 m - 205.031 + 19.975 m = 1 + 227.324m$$

$$SC = TS + LS = 1 + 227.324m + 40m = 1 + 267.324m$$

$$CS = SC + LC = 1 + 267.324m + 219.687 = 1 + 487.001m$$

$$ST = CS + LS = 1 + 487.001m + 40m = 1 + 527.011m$$

when the transition curve inter the Circular curve the length run from(ts-sc).

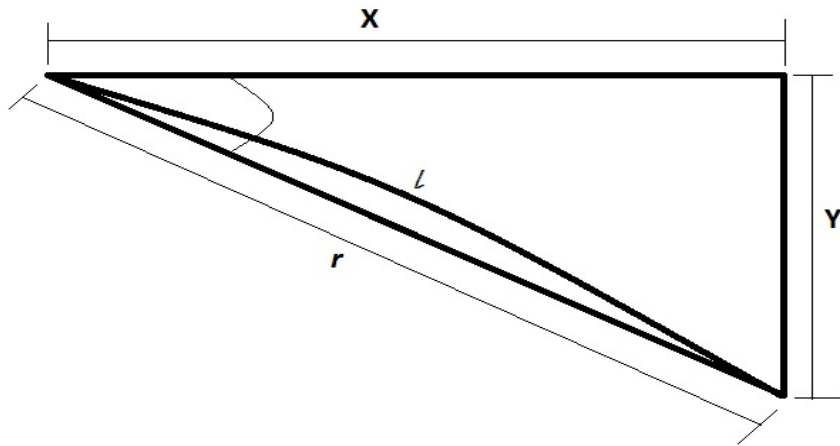


figure 4-4 sample setout

Setting out of curves

NB. We used a 20m interval to do the setting out of both the spiral and circular curves.

For the first station on the curve,

$$L = 1 + 240 - 1 + 227.324 = 12.676m:$$

$$A = \overline{l * R} = \overline{12.676 * 128.8} = 40.406m$$

$$x = l - \frac{l^5}{40A^4} + \frac{l^9}{3456A^8}$$

$$X = 12.676 - \frac{12.676^5}{40 * 40.406^4} + \frac{12.676^9}{3456 * 40.406^8}$$

$$\underline{X = 12.673}$$

$$Y = \frac{l^3}{6A^2} - \frac{l^7}{336A^6} + \frac{l^{11}}{42240A^{10}}$$

$$Y = \frac{12.676^3}{6 * 40.406m^2} - \frac{12.676m^7}{336 * 40.406m^6} + \frac{12.676m^{11}}{42240 * 40.406m^{10}}$$

$$\underline{Y = 0.208m}$$

$$r = \overline{X^2 + Y^2} = \overline{12.673^2 + 0.208^2} = 12.675m$$

$$\delta = \tan^{-1} \frac{Y}{X} = \tan^{-1} \frac{0.208}{12.673} = 0.940^\circ$$

- **For the second station on the curve,**

$$L = 1 + 460 - 1 + 440 = 20$$

$$\text{The new } L = (12.676 + 20) = 32.676m$$

$$A = \overline{l * R} = \overline{32.676 * 128.8} = 64.874m$$

$$x = l - \frac{l^5}{40A^4} + \frac{l^9}{3456A^8}$$

$$X = 32.676 - \frac{32.676^5}{40 * 64.874^4} + \frac{32.676^9}{3456 * 64.874^8}$$

$$\underline{X2 = 32.623}$$

$$Y = \frac{l^3}{6A^2} - \frac{l^7}{336A^6} + \frac{l^{11}}{42240A^{10}}$$

$$Y = \frac{32.676m^3}{6 * 64.874^2} - \frac{32.676m^7}{336 * 64.874^6} + \frac{32.676m^{11}}{42240 * 64.874^{10}}$$

$$\underline{Y2 = 1.380}$$

$$r = \sqrt{X2^2 + Y2^2} = \sqrt{32.623^2 + 1.38^2} = 32.652m$$

$$\delta = \tan^{-1} \frac{Y2}{X2} = \tan^{-1} \frac{1.38}{32.623} = 2.422^\circ$$

- For the third station on the curve,

$$L = 1 + 467.324 - 1 + 460 = 7.324m$$

$$\text{The new } L = 32.676m + 7.324m = 40m$$

$$A = \overline{l * R} = \overline{40 * 128.8} = 71.777m$$

$$X = Ls - \frac{Ls^5}{40A^4} + \frac{Ls^9}{3456A^8}$$

$$X = 40 - \frac{40^5}{40 * 71.777^4} + \frac{40^9}{3456 * 71.777^8}$$

$$X = 39.904m$$

$$Y = \frac{Ls^3}{6A^2} - \frac{Ls^7}{336A^6} + \frac{Ls^{11}}{42240A^{10}}$$

$$Y = \frac{40^3}{6 * 71.777^2} - \frac{40^7}{336 * 71.777^6} + \frac{40^{11}}{42240 * 71.777^{10}}$$

$$Y = 2.066m$$

$$r = \sqrt{X^2 + Y^2} = \sqrt{(39.904)^2 + (2.066)^2} = 39.957m$$

$$\delta = \tan^{-1} \frac{Y}{X} = \tan^{-1} \frac{2.066}{39.904} = 2.964^\circ$$

Computation for circular curve (SC-CS)

Setting out

$$R = 128.8$$

$$\Delta = 97.718^\circ$$

$$SC = TS + LS = 1 + 227.324m + 40m = 1 + 267.324m$$

$$CS = SC + LC = 1 + 267.324m + 219.687 = 1 + 487.001m$$

Let the first station on the curve be 1+300

$$l = 1 + 300 - 1 + 267.324 = 32.676m$$

$$\delta = \frac{1}{2} * \frac{l}{R} * \frac{180}{\pi} = \frac{1}{2} * \frac{32.676}{128.8} * \frac{180}{\pi} = 7.272^\circ$$

$$\text{Chord length: } C = 2 * R * \sin \delta = 2 * 128.8 * \sin 7.272 = 32.605m$$

Table 4-4: Setting out for Curve with transition circular part

Station	Length of curve	radius	Total length of the curve	Deflection angle	comulative(δ)
1+267.324	0	128.8	0	0	0
1+300	32.676	128.8	32.676	7.272	7.272
1+350	50	128.8	82.676	11.127	18.399
1+400	50	128.8	132.676	11.127	29.526
1+450	50	128.8	182.676	11.127	40.653
1+487.001	37.001	128.8	219.677	8.234	48.887

Checking there is any problem during the setting out

$$\Delta = 97.718 = 2 * 48.887 \text{ no problem!}$$

When the transition curve leaves the curve (cs-st)

$$CS = SC + LC = 1 + 267.324m + 219.687 = 1 + 487.001m$$

$$ST = CS + LS = 1 + 487.001m + 40m = 1 + 527.011m$$

- For the first station on the curve,

$$L = 1 + 500 - 1 + 487.001 = 12.999m$$

$$A = \overline{l * R} = \overline{12.999 * 128.8} = 40.918m$$

$$x = l - \frac{l^5}{40A^4} + \frac{l^9}{3456A^8}$$

$$X = 12.999 - \frac{12.999^5}{40 * 40.918^4} + \frac{12.999^9}{3456 * 40.918^8}$$

$$X = 12.996$$

$$Y = \frac{l^3}{6A^2} - \frac{l^7}{336A^6} + \frac{l^{11}}{42240A^{10}}$$

$$Y = \frac{12.999^3}{6 * 40.918^2} - \frac{12.999^7}{336 * 40.918^6} + \frac{12.999^{11}}{42240 * 40.918^{10}}$$

$$Y = 0.219$$

$$r = \sqrt{X^2 + Y^2} = \sqrt{12.996^2 + 0.219^2} = 12.998m$$

$$\delta = \tan^{-1} \frac{Y}{X} = \tan^{-1} \frac{0.219}{12.996} = 0.965^\circ$$

- For the second station on the curve,

$$L = 1+520-1+487.01 = 32.999m$$

$$A = \sqrt{l * R} = \sqrt{32.999 * 128.8} = 65.194m$$

$$x = l - \frac{l^5}{40A^4} + \frac{l^9}{3456A^8}$$

$$X = 32.999 - \frac{32.999^5}{40 * 65.194^4} + \frac{32.999^9}{3456 * 65.194^8}$$

$$X=32.945$$

$$Y = \frac{l^3}{6A^2} - \frac{l^7}{336A^6} + \frac{l^{11}}{42240A^{10}}$$

$$Y = \frac{32.999^3}{6 * 65.194^2} - \frac{32.999^7}{336 * 65.194^6} + \frac{32.999^{11}}{42240 * 65.194^{10}}$$

$$Y = 1.407$$

$$r = \sqrt{X^2 + Y^2} = \sqrt{32.945^2 + 1.407^2} = 32.975m$$

$$\delta = \tan^{-1} \frac{Y}{X} = \tan^{-1} \frac{1.407}{32.945} = 2.445^\circ$$

- For the third station on the curve,

$$L=1+527.011-1+487.011=40$$

$$A = \sqrt{L_s * R} = \sqrt{40 * 128.8} = 71.777.$$

$$x = L_s - \frac{L_s^5}{40A^4} + \frac{L_s^9}{3456A^8}$$

$$X = 40 - \frac{40^5}{40 * 71.777^4} + \frac{40^9}{3456 * 71.777^8}$$

$$X=39.904$$

$$Y = \frac{L_s^3}{6A^2} - \frac{L_s^7}{336A^6} + \frac{L_s^{11}}{42240A^{10}}$$

$$Y = \frac{40^3}{6 * 71.777^2} - \frac{40^7}{336 * 71.777^6} + \frac{40^{11}}{42240 * 71.777^{10}}$$

$$Y = 2.066m$$

$$r = \sqrt{X^2 + Y^2} = \sqrt{39.904^2 + 2.066^2} = 39.957m$$

$$\delta = \tan^{-1} \frac{Y}{X} = \tan^{-1} \frac{2.066}{39.904} = 2.964^\circ$$

4.2.3 Super elevation

It is normal practice for horizontal curves to be super elevated. This allows a component of the vehicle weight to provide some of the centripetal forces that is needed for the vehicle to move in a circular path.

If a curve is not super elevated the curve is said to have adverse or negative super elevation. Therefore 'e' is then expressed as a negative value in %.

The amount of super elevation is chosen primarily on the basis of safety, but other factors are comfort and appearance. The super elevation that is applied to horizontal curve should take into account the following. ((AACRA), 2004)

- Tendency to increase the tracking of the rear wheels of slow moving vehicles toward the center
- Stability of high laden commercial vehicles
- Stability of vehicles loads
- Difference between inner and outer formation level especially in flat country
- Length available to introduce the necessary super elevation
- The need to avoid major changes in side friction demand between successive horizontal curves
- The amount of centripetal force provided by super elevation versus that provided by side

Maximum values of super elevation and increases in side friction

According to AACRA, a maximum value of super elevation to be used in urban areas is 5%. The minimum radius of horizontal curve (R_{min}) that is suitable for a given speed requires the use of maximum allowable super elevation and maximum allowable coefficient of side friction. With horizontal curves that are larger than R_{min} , it is normal practice to provide the same proportions of super elevation and side friction that apply to R_{min} . This practice helps (partly but not completely)

ensure that any increases in side friction demand between successive horizontal curves is kept within acceptable limits. ((AACRA), 2004)

Large increases in side friction demand over what drivers have become accustomed to can change vehicle response that is not anticipated by some drivers. Furthermore, these situations may create hazards for motorcyclists because they have difficulty in maintaining control when there is a sudden change in side friction demand. ((AACRA), 2004)

Therefore, the need to limit changes in side friction demand becomes a desirable secondary control for ensuring geometric consistency. However, the limits on decrease in speed between horizontal alignments elements are (10km/h desirable max. 15km/h absolute max.) are the primary requirement for geometric consistency. The desirable maximum changes in speed will often ensure that changes in side friction demand are acceptable. ((AACRA), 2004)

But in our cause to be conservative we use side friction coefficient of truck and we keep 60km/h constant and by adjusting the radius of the curve.

Table 4-5: Maximum Design Values for Side Friction (Cars on Sealed pavements) (AACRA)

Design Speed km/h	Coefficient of Side Friction	
	Absolute Maximum	Desirable Maximum
40	0.35	0.30
50	0.35	0.30
60	0.33	0.24
70	0.31	0.19
80	0.26	0.16
90	0.20	0.13
100	0.16	0.12
110	0.12	0.12

Table 4-6: Maximum Design Values for Side Friction (Trucks on Sealed pavements) (AACRA)

Operating Speed km/h	Coefficient of Side Friction	
	Absolute Maximum	Desirable Maximum
40	0.25	0.21
50	0.25	0.21
60	0.24	0.17
70	0.23	0.14
80	0.20	0.13
90	0.15	0.13
100	0.12	0.12

Minimum super elevation and adverse super elevation

It is a normal practice to super elevate all curves to a value that is at least equal to the normal cross fall on straights. This is normally 3% in order to ensure adequate surface drainage. However, there are situations where the application of super elevation can cause pavement drainage problems. In some situations when the grade is nearly flat water will not run off the road properly. ((AACRA), 2004)

Problems with drainage of the pavement surface may be overcome by modifying the combination of the grading the super elevation and the application of the super elevation. If the horizontal curve radius is large enough there may be scope to leave the curve un-super elevated (that is have adverse super elevation) may warrant the use adverse super elevation on large radius curves however the use of adverse super elevation must be considered to be exceptional practice.

Further problems with using adverse super elevation are

- Greater tendency of vehicle to move towards the outside (in terms of radius) of the traffic lane on multilane roads
- Greater instability of vehicle loads (exacerbated by greater suspension movement due to the weight component acting in the outwards direction)

On two-lane two-way roads, the normal crowned pavement structure is carried through the curve. Vehicles on the outer lane will have adverse super elevation and vehicles on the inner lane will have positive super elevation. Hence a vehicle that crosses from the inner lane to the outer lane will experience a rapid increase in side friction demand with increases probability of the driver losing control.

These problems with adverse super elevation limit its use on freeways to

- Large radius curves that can be regarded as straights and where pavement drainage can be improved
- Temporary roadways temporary connections

On urban roads, adverse super elevation can be tolerated more than on rural roads. This mainly due to lower vehicle speed and driver familiarity with the road.(AACRA manual Geo des sec11.4.2)

Super elevation on horizontal curves with radius $> R_{min}$

It is normal practice to have super elevation provide a significant amount of the centripetal force that is needed for a vehicle to move in a circular path. This is because super elevation is a positive and permanent feature whereas side friction is subject to road way surface and vehicle variations even so it is common to have two to three times as much centripetal force provided by side friction as that provided by super elevation

There are a number of methods to determine the super elevation (and hence resultant side friction) for curves with a radius larger than the minimum radius for a given design speed.

The method that will normally be used for new works, is for the super elevation to be varied linearly from 0 to $r=\infty$ to e_{max} for R_{min}

This then means that all curves that are designed for a given speed will have approximately the same proportions of super elevation and side friction demand although for construction expediency super elevation values are normally rounded (upwards) to a multiple of 1% so that there is a corresponding adjustment of side friction.

Other methods have been used in the past so that there are likely to be many cases where the reuse of existing pavement will dictate a different super elevation. This is acceptable if the resultant side friction is suitable for the curve design speed and consistent with that for any adjacent curves.

With the linear distribution method, the super elevation e for a curve of radius R that is greater than R_{min} is given by

- $$e = \frac{v^2 e_{max}}{127R} (e_{max} + f_{max})$$

Note that f_{max} may be either the absolute maximum value or the desirable maximum value for the design speed V .

The value of e is usually rounded upwards (e.g. 4.0% but 4.0% becomes 5%) and the corresponding coefficient of side friction is calculated from

- $$e = \frac{v^2}{127R} - e \text{ rounded}$$

With different possibilities for e_{max} and f_{max} (absolute maximum vs desirable maximum) different values of super elevation may be attributed to a given combination of radius and design speed. However, the subjective basis of the linear distribution method (and indeed most other methods) and the practice of rounding the super elevation value allows a practical rationalization to be made

For urban areas rationalization has been achieved by distributing from the combination of 5% maximum super elevation and the desirable maximum value of coefficient of side friction. For design speeds, less than or equal to 100km/h curves with a radius less than that corresponding to 5% super elevation and the desirable maximum coefficient of side friction. Maintain the 5% super elevation with increasing side friction until f_{max} . This practice helps ensure that the relatively low maximum super elevation is applied before vehicles have to make use of increasing side friction. (AACRA manual Geo des sec11.4.4)

Minimum length of super elevation on horizontal curves

In constrained situations such as mountainous terrain or urban roads curves should be fully super elevated even if only instantaneously but it is desirable that there be at least 30m of fully super elevated curve.

Super elevation development length

Super elevation is developed by rotating the roadway cross section about some axis most commonly the horizontal control line.

Super elevation development length is defined as the length required to rotate the pavement from the point of normal cross fall on the approach tangent (straight) to the point where the full super elevation for the curve is attained in turn this super elevation development length has two components

- **Super elevation runoff length** –this is the length from the point where the pavement has been rotated to zero cross fall to the point where the full curve super elevation has been attained

- **Tangent run out**- this is the residual length from the point of normal cross fall to the point zero cross fall (this component lies on the approach tangent)

Therefore, the normal minimum lengths for super elevation runoff for a curve are

- 40m for transitioned curves which ties a 40m minimum transition length
- 50m for an un transitioned curve when the curve design speed is greater than 80km/h with this length normally being equidistant about the curve tangent point
- 30m for an un transitioned curve when the curve design speed is less than or equal to 80km/h with this length normally being equidistant about the curve tangent point

There are two criteria that are used to determine the length of super elevation development

- Maximum rate of rotation of cross fall
- Relative grade between the edge of the carriage way and the control line

Maximum rate of rotation of cross fall

The following maximum rates of rotation are applicable

- 0.025 radians per second for most road types including roads with vehicles that carry livestock
- 0.035 radians per second in low speed areas 70km/h this includes roads in steep terrain and geometry consisting of reverse curves with little or no straight between the curves
- 0.04 radians per second in low speed areas (<60km/h) with extremely constricted horizontal alignment i.e. sections of road in steep terrain and geometry consisting of small radius reverse curves with little or no straight between the curves.
- The minimum super elevation development length based on the above criteria is given by

$$Le = 0.278V(e2 - e1)/r$$

Where

Le=super elevation development (m)

V=design speed (km/h)

e1, e2= cross fall or super elevation at ends of development length (m/m) **+ve** when sloping upwards from the control line **-ve** when sloping down wards from the control line

r=rate of rotation of the road cross fall (radians/second)

Relative grade

Relative grade is the percentage difference between the grade of the edge of the carriage way and grade of the axis of rotation. This difference should be kept below the values shown in the table below to achieve a reasonably smooth appearance.

The minimum super elevation development length based on the criteria in table 11.4.6.A (AACRA manual sec 11)

$$Le = w(e2 - e1)/Gr$$

Where

Le=super elevation development (m)

W=maximum width from axis of rotation to edge of running lane (m)

e1, e2= cross fall or super elevation at ends of development length (m/m) +ve when sloping upwards from the control line –ve when sloping down wards from the control line

Gr =allowable relative grade

Table 4-7: Maximum Relative Grades Between Edge of Carriageway and Control Line (AACRA)

Design Speed (km/h)	Relative grade – per cent		
	One lane ¹	Two lanes ²	More than two lanes ³
40 or under	0.9	1.3	1.7
60	0.6	1.0	1.3
80	0.5	0.8	1.0
100	0.4	0.7	0.9
120 or over	0.4	0.6	0.8

In addition to the above two formulas we incorporate the following standard transition curve length from **AACRA manual (sec) 11 table 11.4.6 B (i.e. Ls=40)** for spiral curve (curve 3).

Super elevation sample calculation

The following table gives the list of the horizontal curves on the road and their location.

Table4-8: List of Simple Horizontal Curves

Simple Horizontal Curves				
Curve number	Point of curvature(PC)	Point of intersection(PI)	Point of tangent(PC)	
1	0+602.49	0+652.89	0+702.49	
2	0+988.085	1+038.847	1+088.085	
3	Binging transition curve	Binging circular curve	End of transition curve	End of circular curve
	1+227.3	1+267.32	1+487.008	1+527.01
4	0+758.314	1+808.869	1+858.314	
5	2+292.144	2+342.33	2+392.14	

Sample calculation for simple curve super elevation

Curve 1

Le=38..... we take 50% this value to know the beginning of SE runoff length as per AACRA manual.

Starting of super elevation runoff station

- $PC - Le/2 = 602.49 - 19 = 583.49m$ (0+583.49)

Starting of fully super elevated station

- $PC + Le/2 = 602.49 + 19 = 621.49m$ (0+621.49)

Starting of existing super elevation

- $PT - Le/2 = 702.49 - 19 = 683.49m$ (0+683.49)

Ending of super elevation run off

- $PT + Le/2 = 702.49 + 19 = 721.49m$ (0+721.49)

Fully super elevated length

- $PT - \frac{Le}{2} - PC + \frac{Le}{2} = 683.49 - 621.49 = 62m$

Curve 3 ('Spiral Curve)

$Le = L_s$ where L_s is transition curve length, because it is preferable as we indicate in the theory part.

$Le = 40$

Starting of super elevation runoff station

- $TS = 1+227.32$

Starting of fully super elevated station

- $SC = 1+267.32$

Starting of existing super elevation

- $CS = 1+487.008$

Ending of super elevation run off

- $ST = 1 + 527.02$

Fully super elevated length

- $CS - SC = 487.008 - 267.32 = 219.69$
- Table for both simple and spiral curve super elevation (e) at specific point of 'X' is given below.

Curve number	Le	Le/2	Location	Station	e%	Remark
3 (spiral)	40	20	TS	1+227.32	-3%	Normal cross fall
			TS + X	1+242.32	0%	Outer lane at flat level(X=15)
			SC	1+267.32	5%	At fully super elevated
			CS	1+487.001	5%	Ending of super elevated
			CS + X	1+512.001	0%	Outer lane at flat(X=25)
			ST	1+527.001	-3%	Normal cross fall
			PC - Le/2	1+758.314	-3%	Normal cross fall

4	38	19	$(PC - L_e/2) + X$	1+772.564	0%	Outer lane at flat level($X=14.25$)
			$PC + L_e/2$	1+777.314	5%	At fully super elevated
			$PT - L_e/2$	1+839.314	5%	Ending of super elevated
			$(PT - L_e/2) + X$	1+863.064	0%	Outer lane at flat($X=23.75$)
			$PT + L_e/2$	1+877.314	-3%	Norma cross fall

Curve number	L_e	$L_e/2$	Location	Station	e%	Remark
5	38	19	$PC - L_e/2$	2+273.144	-3%	Normal cross fall
			$(PC - L_e/2) + X$	2+287.394	0%	Outer lane at flat level($X=14.25$)
			$PC + L_e/2$	2+311.144	5%	At fully super elevated
			$PT - L_e/2$	2+373.144	5%	Ending of super elevated
			$(PT - L_e/2) + X$	2+396.894	0%	Outer lane at flat($X=23.75$)
			$PT + L_e/2$	2+411.144	-3%	Norma cross fall

Table4-9: Position of Super Elevation at Selected Stations

curve no/	PC	PT	Le/2	B Le	ST F SE	ST EX F SE	END SE	FULL SE LE
1	0+602.49	0+702.49	19	0+583.49	0+621.49	0+683.49	0+721.49	62
2	0+988.085	1+088.085	19	0+969.085	1+007.085	1+069.085	1+107.085	62
4	1+758.314	1+858.314	19	1+739.314	1+777.314	1+839.314	1+877.314	62
5	2+292.144	2+392.144	19	2+273.144	2+311.144	2+373.144	2+411.144	62

Where B Le= beginning of SE runoff length

ST F SE=starting of fully super elevated length

ST EX F SE= starting of existing fully super elevation

END SE= ending of super elevation

LE= length of fully super elevation

Super elevation calculation for spiral curve

Note: - we use the next formula to calculate the above tabular components.

$e = X \cdot 8 / Le$ where X = Horizontal distance at any point of horizontal curve.

Le=Super elevation run off length

8=Total super elevation in %(from -3 up to +5)

e =Super elevation in % at specific point.

- Sample charts for both simple and spiral curve super elevation

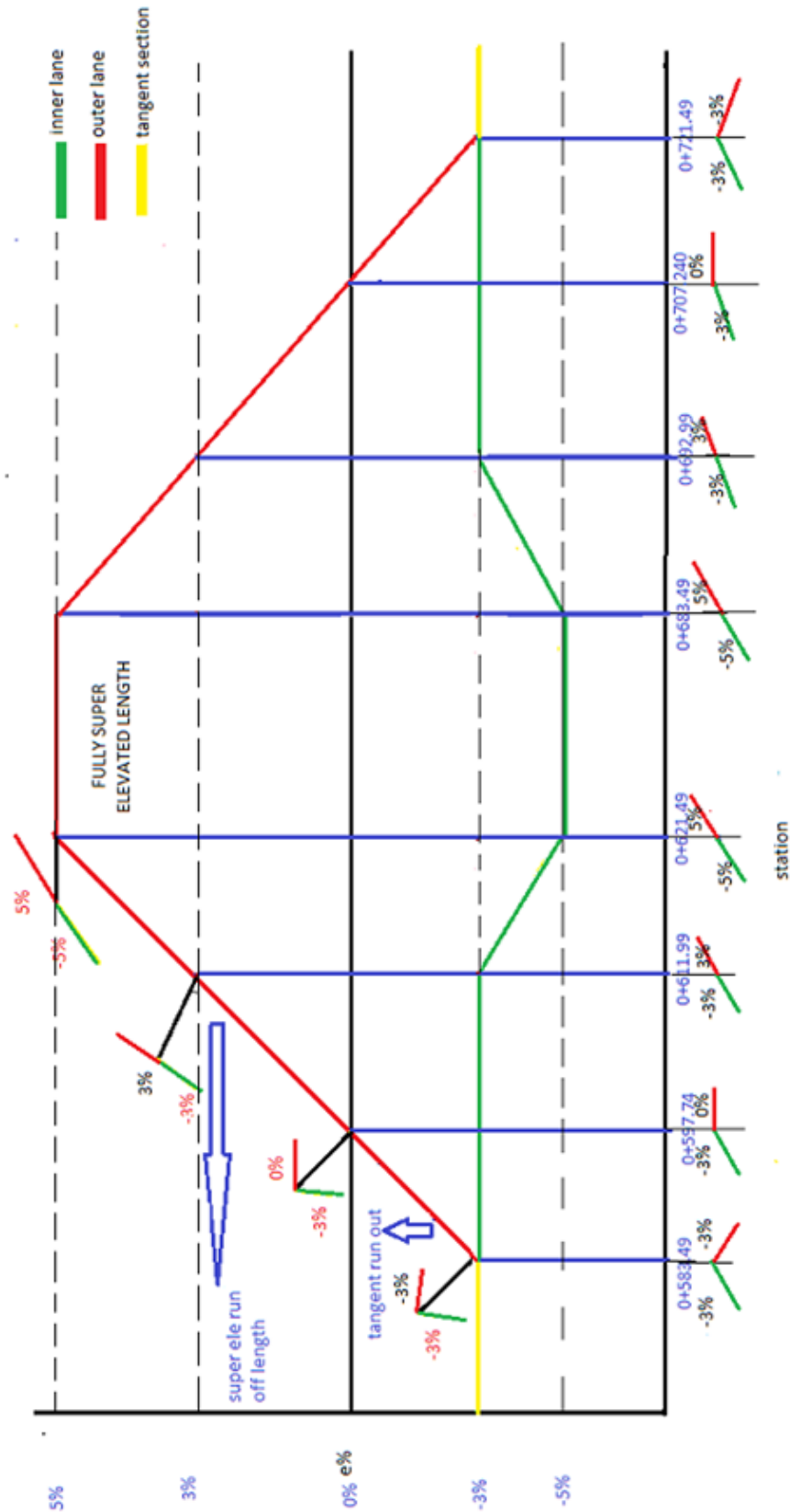


Figure 4-4: Super Elevation Diagram for Curve 1 (simple curve)

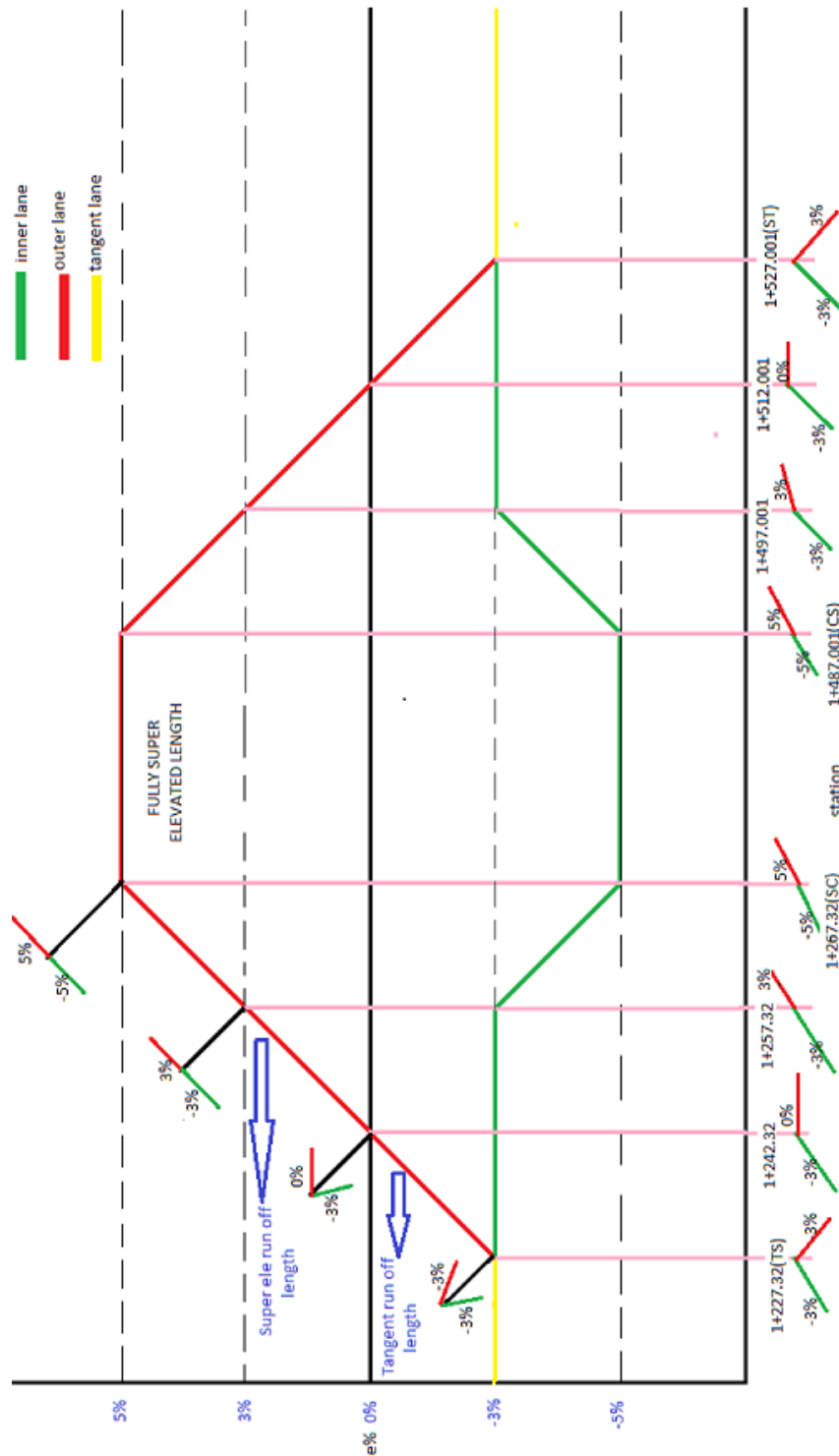


Figure 4-5:
Elevation Diagram for Curve 3 (curve with transitional curve)

Super

4.2.4 CURVE WIDENING

On smaller radius, horizontal traffic lanes may need to be widened in order to maintain the lateral clearances that apply to vehicles on straight sections of road.

The rear wheels of a vehicle off track with respect to the front wheels on a curve and this also causes the front overhang of a vehicle to have a lateral component. The off tracking can be either what is commonly called low speed off tracking (or more correctly when vehicles have a low side friction

demand). Or high speed off tracking (or more correctly off tracking when vehicles' have a high side friction demand.) with the former form of off tracking the rear wheels off track towards the center of the curve. With the latter form the rear wheels off-track outwards from the center of the curve. The swept path widths are based on low speed/low side friction demand off tracking even though normal operation on some curves may involve high speed/high side friction demand off tracking that will be less than the low speed off tracking. (AACRA manual Geo des sec 11,7,1)

The amount of widening per lane depends on

- The radius of the curve
- Type (size) of vehicle operating on the road
- An allowance for steering variation by different drivers

However, there is a lower practical limit to widening due to construction feasibility. For a two-lane road curve widening is omitted when the total widening is less than m 0.5. (AACRA manual Geo des sec 11,7,1)

- In our project, we omitted curve widening because all horizontal radiuses are $> R_{min}$ or sufficient radius and in our road, all most we neglect heavy trucks.

4.2.5 CROSS SECTIONAL ELEMENTS

By cross-section elements are meant that feature of a roadway which forms its effective width. There are two types of elements, basic and ancillary. The basic features of major interest are the width of carriageway (including the number of lanes), central reservation (or median strip), shoulders (including verges), lay bus, camber of the carriageway, and the side slopes of cuttings and embankments. Ancillary elements include safety fences and crash attenuation devices, anti-dazzle screens, and noise barriers. (O'Flaherty, 2003)

Carriageway width

Factor which influence the width of a carriageway are

- The design volume, i.e. the greater the traffic volume the wider the carriageway and, normally, the greater the number of lanes
- Vehicle dimensions, i.e. heavy commercial vehicles require wider carriageways to ensure adequate clearance when passing each other.
- The design speedier vehicles travelling at high speed especially commercial vehicles, require wider carriageways to ensure safe clearances between passing vehicles
- The road classification, i.e. the higher the road classification the greater the level of service (and width of carriageway) expected.

Internationally, it is generally accepted that lane width should normally be at least 3.5m, although narrower lanes are often used for economic or environmental reason on both rural and urban roads. However, research has established that the accident rates for vehicles running off the road, and for head-on and sideswipe accident between vehicles travelling in opposite direction, decrease with increasing lane width up to 3.65m on two-lane two-way rural roads. Lanes on British roads are most commonly 3.65m. (O'Flaherty, 2003)

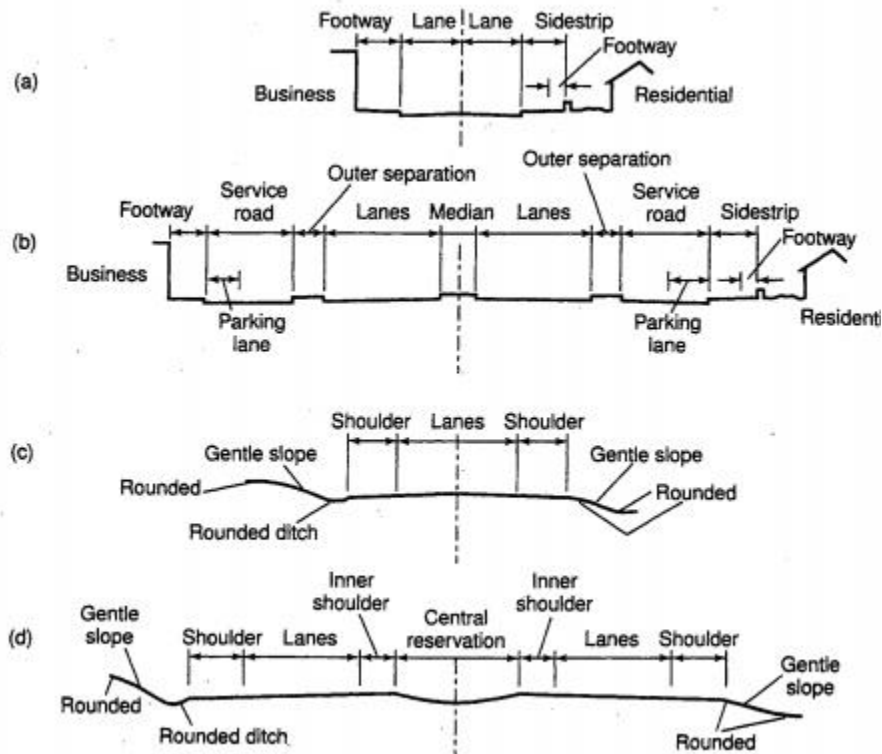


Figure 4-6: Cross section (AACRA)

For our project road in BoleBulbulla condominium (residential) as we indicate in the previous section our road is collector road and as per AACRA manual in geometric design **sec 7 Table7.6.1-A** we can get total cross-sectional area of the road. But we decided to use dual carriageway width because of environmental needs, population density and by considering the future economic growth of the city.

Collector Roads

Table 4-14 shows the cross section for collector roads in residential areas. These roads may be bus routes. A road reserve of 15.0m is required. ((AACRA), 2004)

Table 4-10: Cross Section for Collector Roads in Residential Areas. (AACRA)

Border Width	Footpath Width	Carriageway Width*
2 x 4.0	2 x 1.2	7.0

Border

The area between the carriageway and the property line. On freeway, it includes the verge. On other urban roads, it includes the footpath and space for the placement of utility services, landscape planting, and indented bus bays and indented parking. (AACRA manual GEO Des sec 7)

4.3 Vertical Alignment

The vertical alignment of a highway consists of straight section of highway, grades or tangents, connected by vertical curves. Thus, the design of vertical alignment involves the selection of suitable

grades for the tangent sections and the design of vertical curves. The vertical curvature is governed by sight distance criteria whereas the gradient is related to vehicle performance and level of service. In addition to the topography of the area, the route through which the road traverses have a significant impact on the design of the vertical alignment. **(AACRA)**

During vertical alignment, the following should be considered

- Gradient of the route should be between the allowable max and min
- Critical length should be within limits
- Vertical and horizontal curves should not over lap
- Cut and fill should be balanced
- Provision of adequate sight distance over all crests
- Avoidance short grade between two vertical curves
- Avoidance of the combination of two vertical curves in the same direction

The two major aspects of vertical alignment are vertical curvature, which is governed by sight distance criteria, and gradient. Gradient is related to vehicle performance and level of service. There are two types of vertical curves-summit (or crest) and sag (or valley) curves, which are introduced at vertical grade changes. **(AACRA)**

Stopping sight distance

Stopping sight distance is the sum of two distances: the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied and the distance required to stop the vehicle from the instant brake application begins. These are referred to as brake reaction distance and braking distance, respectively.

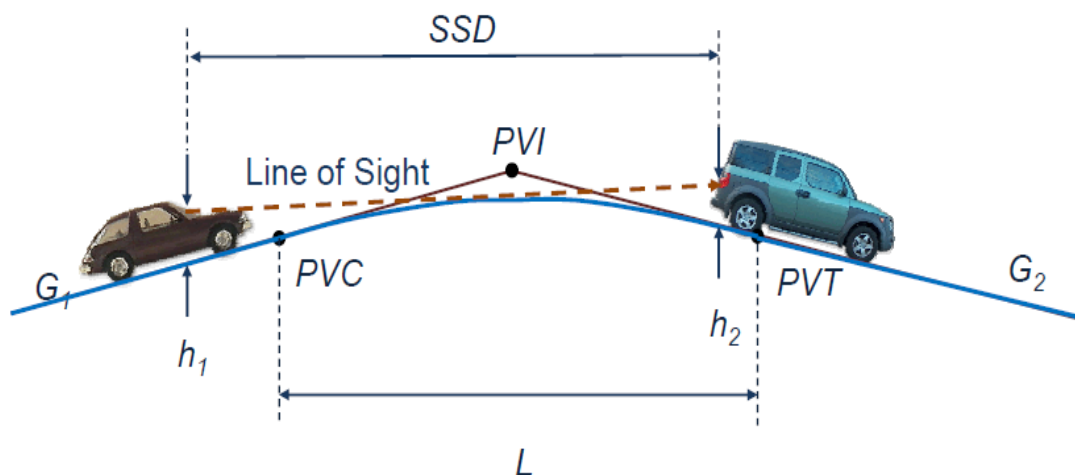


Figure 4-7: Sight Distance on Crest Vertical Curves

4.3.1 Gradients

Although the use of steep tangent gradients in hilly terrain generally results in lower road construction and environmental costs, it also adds to road user costs through delays and extra fuel and accidents. The extra user costs mostly noticeable when traffic is heavy and commercial vehicles form a large proportion of the traffic stream. On uphill gradients in hilly terrain extra accidents often

results from frustrated drivers of faster vehicles over taking where normally they might not do so. speeds normally increase on downhill gradients and if drivers have a break to, say, traverse a curve at the bottom of the hill and the road is icy or wet serious accidents can result from vehicles skidding out of control.

Gradients of up to about 7percent have relatively little effect on the speeds of passenger cars. However, the speeds of commercial vehicles are considerably reduced on long hills with gradients in excess of about 2 per cent. When the uphill distances are short gradients of 5 or 6 per cent may have little detrimental effect on commercial vehicles speeds, drivers usually accelerate upon entering an uphill section and the extra momentum can overcome the effect of the gradients over these shorter distances. (ACRA MANUAL sec-11)

The grades are selected based on

- ✓ The amount of earthwork (cut/fill) should be as minimum as possible to reduce cost.
- ✓ Design speed and topographic factors.
- ✓ Vehicle operating cost.
- ✓ Minimum grade of 0.5% should be provided for drainage purpose.
- ✓ Grades are selected as much as possible not to cause high fill.

Gradients and grade controls

Consideration;

1. for the maximum limit.

- ❖ Visibility related to sight distance.
- ❖ Cost of vehicle operation
- ❖ Cut and fill (earth work)
- ❖ Passengers comfort.
- ❖ General appearance and
- ❖ Existing road intersections & adjacent land use.

2. for the minimum limit

- ❖ Drainage purpose

Maximum Gradients

It is desirable to have a very long length of steep grade. On both the upgrade and the down grade, the lower operating speed of trucks may cause inconvenience to other traffic.

For very low levels of traffic flow represented by only a few four-wheel drive vehicles other references advocate a maximum traversable gradient of up to 18 percent. Small commercial vehicles can usually negotiate an 18 per cent gradient; whilst two-wheel drive trucks can successfully manage gradients of 15-16 per cent except when heavily laden.

Minimum Gradients

The minimum gradient for the usual case is 0.5 percent. However, flat and level gradients on uncurbed paved highways are acceptable when the cross slope and carriageway elevation above the surrounding ground is adequate to drain the surface laterally. With curbed highways or streets, longitudinal gradients should be provided to facilitate surface drainage.

Critical length of gradient

Critical length may be defined at the point at which a truck reaches a certain speed or the point at which it has lost a certain amount of speed.

4.3.2 Vertical Curve

K VALUE

Provide K values for sags and crest curves based upon comfort criteria. Discomfort is felt by a person subjected to rapid changes in vertical acceleration. To minimize such discomfort when passing from one grade to another. It is usual to limit the vertical acceleration generated on the vertical curve to a value less than 0.05g where g is the acceleration due to gravity. On low standard roads. At intersections or where economically justified, a limit of 0.10g may be used.(AACRA)

The value c is a constant that depends on the eye and the situation

For crest vertical curves:

$$C = 200 [\sqrt{h_1} + \sqrt{h_2}]^2$$

For sag vertical curve headlight sight distance,

$$C = 200 [h + D \tan q].$$

Crest Curves (summit)

Crest vertical curves are curves when the algebraic difference of the two gradients is positive and when viewed from the side, are convex upwards.

This includes vertical curves at hill crests, but it also includes locations where an uphill grade becomes less steep, or a downhill grade becomes steeper. (O'Flaherty, 2003)

The most important design criterion for these curves is stopping sight distance. This is the distance a driver can see over the crest of the curve. If the driver cannot see any obstructions in the roadway, such as a stalled vehicle or animal, the driver may not be able to stop the vehicle in time to avoid a crash.

The minimum lengths of crest curves have been designed to provide sufficient stopping sight distance. The design is based on minimum allowable "K" values, as defined by the formula:

$$K = L/A$$

Where

K = limiting value, horizontal distance required to achieve a 1% change in grade

L = length of vertical curve (m)

A = Algebraic difference in approach and exit grades (%)

Where K is the length of the vertical curve per percent change in A . Since K is a function of design speed, it can be used as a convenient “shortcut” to compute the minimum length for a crest vertical curve

ACCRA recommends the following K values for crest curves

Table 4-11: Crest K Values Based on SSD for Trucks (AACRA)

Speed	Truck Stopping Distance		Crest K Value $h_1 = 2.40\text{m}$ $h_2 = 0.20\text{m}$	
	Desirable Min. $R_T 2.5\text{sec}$	Absolute min $R_T 2.0\text{ sec}$	Desirable Min.	Absolute Min
40	50	44	6	5
50	69	62	6	5
60	91	82	10	8
70	116	106	17	14
80	143	131	25	22
90	173	160	37	32
100	210	197	55	48
110	259	244	85	75

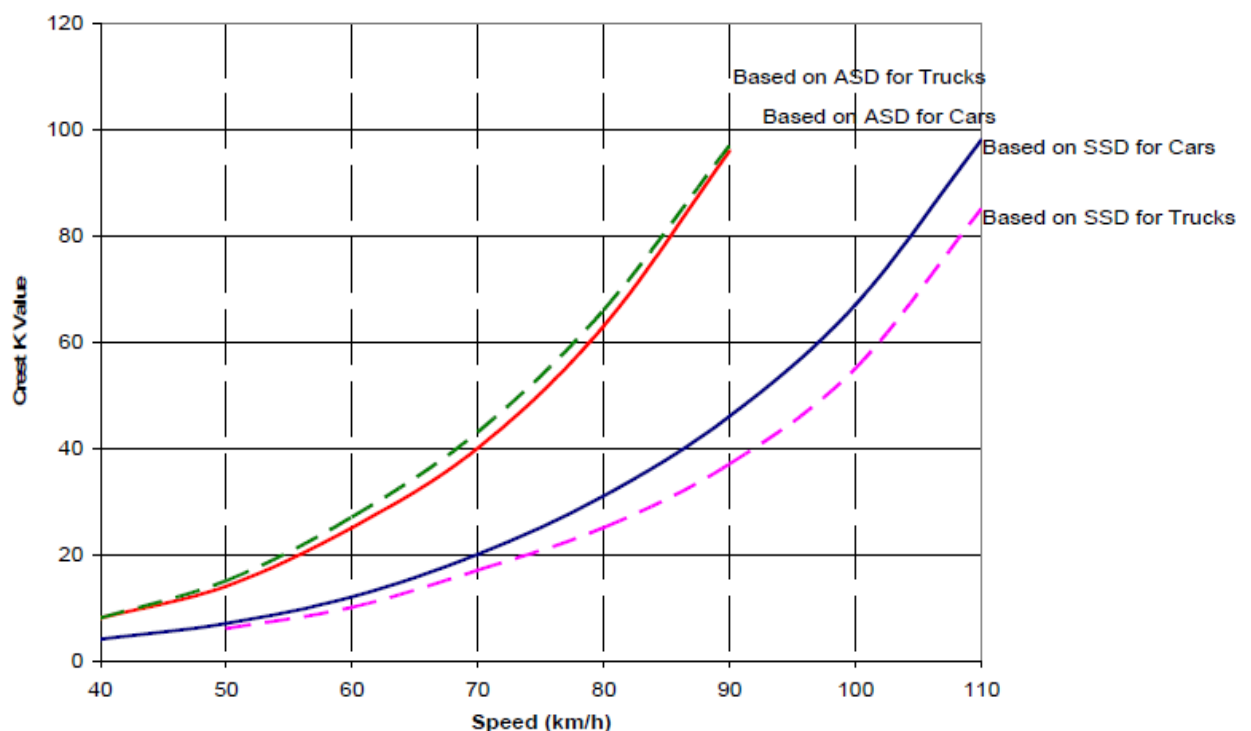


Figure 4-8: Crest K Values (AACRA)

Minimum lengths of crest vertical curves are determined based on sight distance criteria and it's generally satisfactory from the standpoint of safety, comfort, and appearance.

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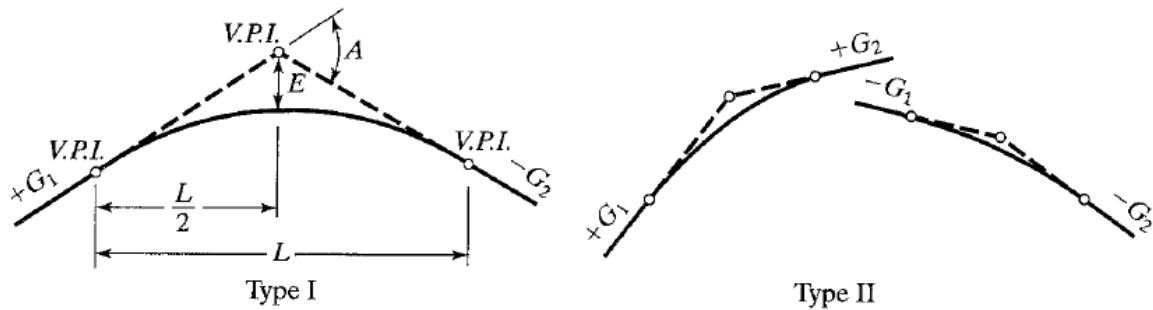


Figure 4-9: Types of Vertical Crest Curves

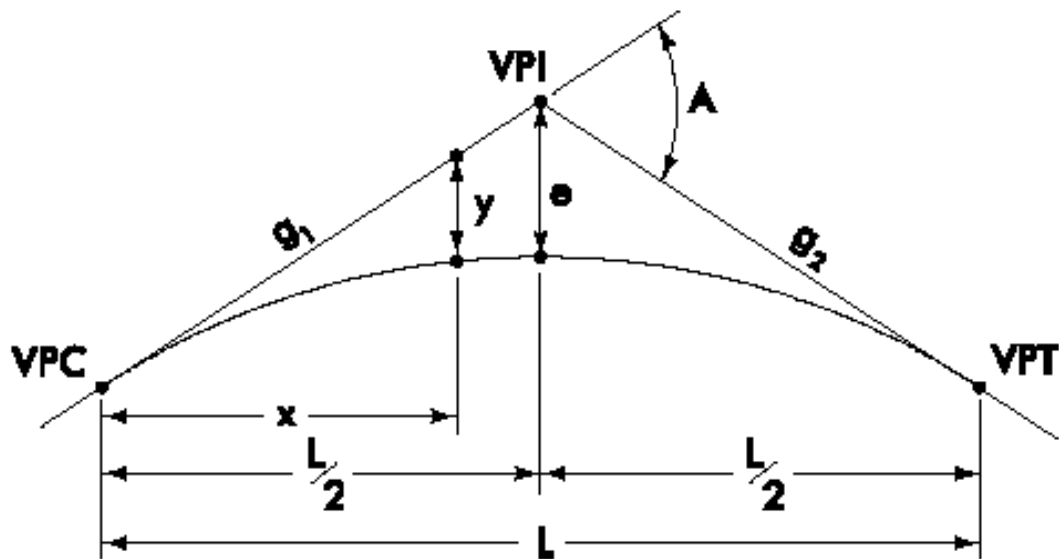


Figure 4-10: Crest Vertical Curve Elements

$$L_{min} = \frac{AS^2}{(\overline{h_1} + \overline{h_2})^2} \text{ when } S \leq l$$

$$L_{min} = 2S - \frac{200(\overline{h_1} + \overline{h_2})^2}{A} \quad \text{when } S > L$$

DESIGN CONTROLS

The minimum lengths of crest vertical curves for different values of A to satisfy the minimum stopping sight distances for different design speed

According to AACRA manual, SSD is based upon the design speed for cars (with 2.5 sec car driver reaction time) and for trucks (with 2.0 sec truck driver reaction time). And recommend minimum K value is tabulated as

SAG Vertical Curve

Sag vertical curves are curves when the algebraic difference of the two gradients is negative and when viewed from the side, are concave upwards.

This includes vertical curves at valley bottoms, but it also includes locations where an uphill grade becomes steeper, or a downhill grade becomes less steep.

At least four different criteria are used for establishing lengths of sag vertical curves.

These are

- Headlight sight distance,
- Passenger comfort,
- Drainage control, and
- General appearance

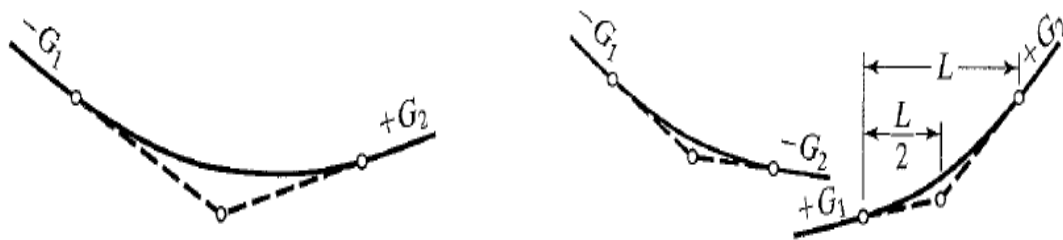


Figure 4-11: Types of Sag Vertical Curves

A. Head light sight distance

Headlight sight distance has been used directly by some agencies and for the most part is the basis for determining the length of sag vertical curves. When a vehicle traverses a sag vertical curve at night, the portion of highway lighted ahead is dependent on the position of the headlights and the direction of the light beam. A headlight height of 600 mm and a 1-degree upward divergence of the light beam from the longitudinal axis of the vehicle is commonly assumed. The upward spread of the light beam above the 1-degree divergence angle provides some additional visible length of roadway, but is not generally considered in design. The following equations show the relationships between S, L, and A, using S as the distance between the vehicle and point where the 1-degree upward angle of the light beam intersects the surface of the roadway:

- Distance illuminated by headlight at night
- Height of headlight= 0.600 m
- Upward divergence of headlight beam= 1°

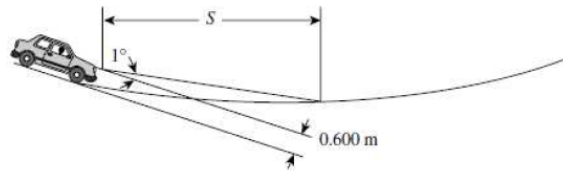


Figure 4-12: Stopping Sight Distance for Sag Vertical Curves

$$L_{min} = \frac{AS^2}{200(0.6 + Stan1^\circ)} = \frac{AS^2}{120 + 3.5S} \quad \text{When } S \leq L$$

$$2S - \frac{200(0.6 + Stan1^\circ)}{A} = 2S - \frac{120 + 3.5S}{A} \quad \text{When } S > l$$

Where:

L= length of sag vertical curve in meter

S= light beam distance in meter

A=algebraic difference in grades in percent

For overall safety on highways, a sag vertical curve should be long enough that the light beam distance is nearly the same as the stopping sight distance. Accordingly, it is appropriate to use stopping sight distances for different design speeds as the value of S in the above equations

The minimum lengths of sag curves have been designed to provide sufficient stopping sight distance. The design is based on minimum allowable "K" values, as defined by the formula:

$$K = L/A$$

Where

K = limiting value, horizontal distance required to achieve a 1% change in grade

L = length of vertical curve (m)

A = Algebraic difference in approach and exit grades (%)

Minimum Values for Sag Vertical Curves

Table 4-12: Minimum K Values for Sag Vertical Curves Based on Headlight and Trucks SSD (AACRA)

Speed	Sight Distance	K h = 0.75m q = 1 ⁰
40	50	9
50	69	12
60	91	18
70	116	29
80	143	32
90	150*	33
100	150*	33
110	150*	33

* Maximum range of headlights assumed 150m

Table 4-13: Minimum K Values for Sag Vertical Curves Based on Vertical Acceleration (AACRA)

Speed	K Value	
	a = 0.05g	Aa = 0.10 g
40	3	1
50	4	2
60	6	3
70	8	4
80	11	6
90	14	7
100	17	9
110	20	10

A. Passenger comfort

The effect on passenger comfort of the change in vertical direction is greater on sag vertical curve than on crest vertical curves because gravitational and centripetal forces are in opposite directions, rather than in the same direction. Comfort due to change in vertical direction is not readily measured because it is affected appreciably by vehicle body suspension, vehicle body weight, tire flexibility, and other factors. Limited attempts at such measurements have led to the broad conclusion that riding is comfortable on sag vertical curves when the centripetal acceleration does not exceed 0.3m/s². The general expression for such a criterion is

$$L = A * \frac{V^2}{395}$$

Where,

L=length of sag vertical curve

A=algebraic difference in grade %

V=design speed km/h

B. Drainage

A minimum grade of 0.30 percent should be provided within 15 m of the level point. This criterion corresponds to K of 51 m per percent change in grade.(that is L_{max}=51A) The drainage criterion

differs from other criteria in that the length of sag vertical curve determined for it is a maximum, whereas, the length for any other criterion is a minimum.

C. Appearance

For general appearance of sag vertical curves, some use was formerly made of a rule-of thumb for minimum curve length of $30A$, $K = 30$. This approximation is a generalized control for small or intermediate values of A . Compared with headlight sight distance, it corresponds to a design speed of approximately 80 km/h. On high-type highways, longer curves are appropriate to improve appearance.

Comparison of ASSHTO and AACRA manual

DESIGN STANDARD		AACRA	AASHTO
Design Speed (Km/h)		60	60
Grade	Minimum	0.5%	0.5%
	Maximum	4.0%	7%
Minimum K Value	Sag	18	18
	CREST	12	11
Passing Sight Distance (m)		-----	195
Minimum Stopping Sight Distance (m)		91	85

Table 4-19 comparison of ASSHTO and AACRA manual

Vertical Curve Design

Vertical curve 1 (Crest curve)

sample calculation

We have the following curve data from vertical alignment we have prepared using INROADS software.

Station of PVI = 0 + 238.896

Elevation of PVI = 2204.462

The minimum value of K is 12 for 60km/h from AACRA manual. But the curve length when $K=12$ is used is too small for curve setting out. This is due to the minimum change between the two grades (g_1 and g_2). Therefore, we have decided to use a higher value of ' K ' in order to make larger curve length.

Using $K=80$

$$g_1 = 1.773\% \text{ \& } g_2 = -0.507\%$$

$$A = |g_2 - g_1| = |1.177 + 0.507| = 2.28\%$$

$$L = K * A = 80 * 2.28 = 182.4m$$

First calculate the required minimum stopping sight distance. Here we have calculated the required SSD for all tangent section slopes in the road and the maximum of these values is used as the governing SSD for all tangent slopes.

$$SSD = 0.278V * t + \frac{V^2}{254(f+0.01G)} = 0.278 * 60 * 2.5 + \frac{60^2}{254 * 0.29 - 0.04072} = 98.6$$

Table 4-20: SSD for all Grades in the Vertical Alignment

$V(\text{km/h})$	T	f (longitudinal friction)	Grade (g)	SSD	SSD from code
60	2.5	0.29	1.773	87.7573501	71
60	2.5	0.29	-0.507	91.44284332	71
60	2.5	0.29	-1.773	93.75578414	71
60	2.5	0.29	0.507	89.73344409	71
60	2.5	0.29	2.551	86.62164542	71
60	2.5	0.29	-2.551	95.28701027	71
60	2.5	0.29	4.072	84.55567352	71
60	2.5	0.29	-4.072	98.55666057	71
60	2.5	0.29	-1.436	93.11934533	71

Case I Assume $S > L, A = g_1 - g_2 = -0.507 - 1.773 = -2.28\%$

$h_1 = 1.05\text{m}$ $h_2 = 0.2\text{m}$

$$C = 200(h_1^2 + h_2^2)^{1/2}$$

$$C = 200(1.05^2 + 0.2^2)^{1/2} = 433.3\text{m}$$

SSD = 98.3m Calculated earlier

Stationing of vertical curve

$$\text{Chainage of BVC} = \text{chainage of PVI} - L/2$$

$$\text{Chainage of BVC} = 0 + 238.896 - 0.5 * 182.388 = \mathbf{147.702\text{m}}$$

$$\text{Elevation of BVC} = \text{Elevation of PVI} - g_1 * L/2$$

$$\text{Elevation of BVC} = 2204.462 - ((1.773 * 0.5 * 182.388)/100) = \mathbf{2202.85\text{m}}$$

$$\text{Chainage of EVC} = \text{Chainage of PVI} + g_2 * L/2 \text{ or Chainage of BVC} + L$$

$$\text{Chainage of EVC} = 147.702\text{m} + 182.388 = \mathbf{329.49\text{m}}$$

$$\text{Elevation of EVC} = \text{ElevPVI} - g_2 * L/2$$

$$\text{Elevation of EVC} = 2204.462 - 0.5 * 182.388 * 0.507/100 = \mathbf{2204\text{m}}$$

Setting out vertical curve

We have used 20m interval for all curves

$$\text{Chainage of BVC} = 0 + 147.702\text{m}$$

$$\text{The first station (v1)} = 0 + 160$$

$$\text{Length, } X = 160 - 147.702 = 12.298\text{m}$$

$$\text{Target ELV} = g_1 * \frac{x}{100} + \text{ElevBVC} = 1.773 * \frac{12.298}{100} + 2202.85\text{m} = 2203.068\text{m}$$

$$\text{Offset } y' = \frac{A \cdot X^2}{200L} = 2.28 * \frac{12.298}{200} * 182.388 = 0.009$$

$$\text{Curve Elv (Y)} = \text{Tanget ELV} - y' = 2203.068 - 0.009 = \mathbf{2203.058m}$$

$$\text{station}(v2) = 0 + 180$$

$$X = 0 + 180 - 0 + 147.02 = 32.298m$$

$$\text{Tanget ELV} = g1 * x/100 + \text{ElvBVC}$$

$$= 1.773 * 32.298/100 + 2202.85 = 2203.422m$$

$$\text{Offset } y' = A * \frac{X^2}{200L} = 2.28 * \frac{32.298^2}{200} * 182.388 = 0.06520184m$$

$$\text{Curve Elv } Y = \text{Tanget ELV} - y' = 2203.422 - 0.06520184 = \mathbf{2203.357m}$$

Vertical Curve3 (Sag)

$$g1 = 2.511, \quad g2 = 4.072, \quad A = 1.561$$

For the same reason as curve 1 we have decided to use a higher value of K,

Using $K = 90$

$$\text{head lights } SD = 298.743$$

$$\text{Take } SSD = 298.743$$

$$\text{Take } L \text{ for Sag} = A * K = 90 * 1.561 = 140.504m$$

Stationing of the curve

$$\text{Station } PVI = 1 + 516.881$$

$$\text{Elevation of } PVI = 2212.186m$$

$$\text{Elevation of } BVC = 2212.186m - 2.511\% * 0.5 * 140.504m = \mathbf{2210.422m}$$

$$\text{Chainage of } BVC = \text{Station } PVI - \frac{L}{2} = 1 + 516.881 - \frac{140.504}{2} = 1 + 446.629m$$

$$\text{Chainage of } EVC = \text{Station } PVI + \frac{L}{2} = 1516.881 + \frac{140.504}{2} = 1 + 587.133m$$

$$\text{Elevation of } EVC = 2 + 212.186m + 4.072 * 140.504 * 0.5 = \mathbf{2215.046m}$$

Setting out vertical curve

Using a 20m interval for stationing

$$\text{Chainage Of } BVC = 1 + 446.629$$

$$\text{Let the first station be } (v1) = 1 + 460$$

$$\text{Distance, } X = 1 + 460 - 1 + 446.629 = 13.371m$$

$$\text{Tanget ELV} = \text{ElvBV} + g_1 * \frac{x}{100} = 2.511 * \frac{13.371}{100} + 2210.422m = 2210.757746m$$

$$\text{Offset } y' = A * \frac{x^2}{200L} = 1.561 * \frac{13.371^2}{200} * 140.504 = 0.010$$

$$\text{Curve Elv } Y = \text{Tanget ELV} + y' = 2210.757745m - 0.00993 = \mathbf{2210.77m}$$

station (v2)=1+460

$$X = 1 + 460 - 1 + 446.629 = 33.371$$

$$\text{Tanget ELV} = g_1 * x/100 + \text{ElvBVC} =$$

$$= 2.511 * \frac{33.371}{100} + 1446.629 = 2211.259946m$$

$$\begin{aligned} \text{Offset } y' &= \frac{A * X^2}{200L} = 1.561 * \frac{33.371^2}{200} * 140.504 = 0.06186m \\ \text{Curve Elv } Y, \\ &= \text{Tanget ELV} - \text{Offset } y' = 2211.259946 - 0.06186 = 2211.32m \end{aligned}$$

NB The rest of the calculation is completed on excel sheet below

We have designed four vertical curves for the project in accordance to the vertical alignment. The sammary of all vertical curves is provided in the table below.

Table 4-14:Summary of Vertical Curves

Curve no	g1	g2	PVI sat	Elva PVI	type of Curve	L	K	Elva BVC	sta BVC	Elva EVC	StaEVC	Ymax/min
1	1.773	-0.507	238.896	2204.462	CREST	182.4	80	2202.845	147.696	2203.999616	330.096	2203.942
2	-0.507	2.511	1046.117	2200.367	SAG	211.26	70	2200.902	940.487	2203.019369	1151.747	2199.57
3	2.511	4.072	1516.881	2212.186	SAG	140.49	90	2210.422	1446.636	2215.046376	1587.126	2211.912
4	4.072	-1.436	2102.853	2236.045	CREST	385.56	70	2228.195	1910.073	2233.276679	2295.633	2233.390

Table 4-15:Vertical Curves Setting Out

Vertical Curve 1 (Crest)					
Elevation of Tangent (m)	Station	Cumulative length (m)	Tangent Elevation	Offset y'(m)	Curve Elevation (m)
2202.85	147.702	12.298	2203.068044	0.009453172	2203.05859
2202.85	160	32.298	2203.422644	0.06520184	2203.357442
2202.85	180	52.298	2203.777244	0.170953797	2203.60629
2202.85	200	72.298	2204.131844	0.326709044	2203.805134
2202.85	220	92.298	2204.486444	0.532467581	2203.953976
2202.85	240	112.298	2204.841044	0.788229407	2204.052814
2202.85	260	132.298	2205.195644	1.093994524	2204.101649
2202.85	280	152.298	2205.550244	1.449762929	2204.100481
2202.85	300	172.298	2205.904844	1.855534625	2204.049309
2202.85	320	192.298	2206.259444	2.31130961	2203.948134

2202.85	330.09	201.788	2206.427701	2.545067248	2203.882634
Vertical Curve 2 (Sag)					
Elevation of Tangent (m)	Station	Cumulative length (m)	Tangent Elevation	Offset y' (m)	Curve Elevation (m)
2200.903	940.487	13.883	2200.832613	0.013767695	2200.846381
2200.903	960	33.883	2200.731213	0.082008391	2200.813222
2200.903	980	53.883	2200.629813	0.207394919	2200.837208
2200.903	1000	73.883	2200.528413	0.389927281	2200.91834
2200.903	1020	93.883	2200.427013	0.629605475	2201.056619
2200.903	1040	113.883	2200.325613	0.926429501	2201.252043
2200.903	1060	133.883	2200.224213	1.280399361	2201.504613
2200.903	1080	153.883	2200.122813	1.691515053	2201.814328
2200.903	1100	173.883	2200.021413	2.159776577	2202.18119
2200.903	1120	193.883	2199.920013	2.685183934	2202.605197
2200.903	1140	213.883	2199.818613	3.267737124	2203.08635
2200.903	1151.747	225.63	2199.759056	3.636539128	2203.395595

Vertical Curve 3 (Sag)

Elevation of Tangent (m)	Station	Cumulative length (m)	Tangent Elevation	Offset y' (m)	Curve Elevation (m)
2210.422	1446.629	13.371	2210.757746	0.009931435	2210.767677
2210.422	1460	33.371	2211.259946	0.061861815	2211.321808
2210.422	1480	53.371	2211.762146	0.158232212	2211.920378
2210.422	1500	73.371	2212.264346	0.299042625	2212.563388
2210.422	1520	93.371	2212.766546	0.484293053	2213.250839
2210.422	1540	113.371	2213.268746	0.713983497	2213.982729
2210.422	1560	133.371	2213.770946	0.988113958	2214.75906
2210.422	1580	153.371	2214.273146	1.306684434	2215.57983
2210.422	1587.126	160.497	2214.45208	1.430928907	2215.883009

Vertical Curve 4 (Crest)

Elevation of Tangent (m)	Station	Cumulative length (m)	Tangent Elevation	Offset y' (m)	Curve Elevation (m)
2228.195	1910.073	19.927	2229.006427	0.028363238	2228.978064
2228.195	1930	39.927	2229.820827	0.113868952	2229.706958
2228.195	1950	59.927	2230.635227	0.256517523	2230.37871
2228.195	1970	79.927	2231.449627	0.456308952	2230.993318
2228.195	1990	99.927	2232.264027	0.713243238	2231.550784
2228.195	2010	119.927	2233.078427	1.027320381	2232.051107
2228.195	2030	139.927	2233.892827	1.398540381	2232.494287
2228.195	2050	159.927	2234.707227	1.826903238	2232.880324
2228.195	2070	179.927	2235.521627	2.312408952	2233.209218

2228.195	2090	199.927	2236.336027	2.855057524	2233.48097
2228.195	2110	219.927	2237.150427	3.454848952	2233.695578
2228.195	2130	239.927	2237.964827	4.111783238	2233.853044
2228.195	2150	259.927	2238.779227	4.825860381	2233.953367
2228.195	2170	279.927	2239.593627	5.597080381	2233.996547
2228.195	2190	299.927	2240.408027	6.425443238	2233.982584
2228.195	2210	319.927	2241.222427	7.310948952	2233.911478
2228.195	2230	339.927	2242.036827	8.253597524	2233.78323
2228.195	2250	359.927	2242.851227	9.253388952	2233.597838
2228.195	2270	379.927	2243.665627	10.31032324	2233.355304
2228.195	2290	399.927	2244.480027	11.42440038	2233.055627
2228.195	2295.633	405.56	2244.709403	11.74849383	2232.960909

CHAPTER FIVE

EARTH WORK

5.1 Earth Work Quantities and Mass Haul Diagram

Earthwork is simply the quantity of material to be moved or put in place to convert the natural condition and configuration within the Row of the road to the section and grades described in the plans. The final element in the location process is to establish the horizontal and vertical alignments of the highway project and to prepare highway plans and specifications for estimating project costs and preparation of bids by contractors. (Garber, Traffic & Highway Engineering, 2009)

Earthwork is necessary to

- ❖ Set grade line as close as possible to natural ground level
- ❖ Set grade line so there is a balance between excavated volume and volume of embankment
- ❖ clearing, grubbing, roadway and drainage excavation,
- ❖ excavation for structures, embankments, borrow, overhaul, machine grading, sub grade scarifying, rock fill,
- ❖ All the operations of preparing the sub grade foundation for highway or runway pavement.

Classification of earthwork operations

- ❖ Clearing and grubbing
- ❖ Excavation
 - a) Roadway and drainage excavation
 - b) Excavation for structures
 - c) Borrow excavation
- ❖ **Clearing and grubbing:** the removal of trees, stumps, roots, down timber, rotten wood, rubbish and other objectionable materials from an area marked on the plans. Clearing refers to the removal of materials above existing ground surface, and Grubbing means the removal of roots, stumps and similar objects to a nominal depth below the surface. It is the first operation to be undertaken in road projects.
- ❖ **Excavation:** the process of loosening and removing earth or rock from its original position in a cut and transporting it to a fill section or to a waste deposit.
 - a) **Roadway and drainage excavation:** the excavation and grading of the roadway and ditches, including the removal and disposal of all excavated material and all work needed for the construction and completion of the cuts, embankments, slopes, ditches, approaches, intersections and similar portions of the work.
 - b) **Excavation for structures:** the excavation of material in order to permit the construction of pipe culverts, concrete box culverts, foundations for bridges,

retaining walls, and practically all other structures that may be required in a particular job.

- c) **Borrow excavation:** when sufficient material for the formation of embankments and other elements of the roadway structure is not available from excavation performed within the limits of the right-of-way, additional suitable material is generally taken from borrow pits. (AAiTlecture)

Classification of Excavation

- ❖ **Rock excavation:** material that cannot be excavated without blasting or the use of rippers and all boulders or other detached stones
- ❖ **Common excavation:** excavation and disposal of all materials of whatever character encountered in the work, which are not classified as rock, borrow
- ❖ **Borrow excavation:** excavation of approved material required for construction of embankments.
- ❖ **Unsuitable excavation:** the removal and disposal of deposits of saturated or unsaturated mixtures of soil and organic matter not suitable for embankment material. (Chandra, 2005)

5.2 Shrinkage and Swell

Excavated in-situ material used in embankments or fills may swell or shrink. It is important to determine the properties of a material in order to evaluate how these properties will affect the earthwork volume estimates. (AAiTlecture)

Excavation of material causes it to loosen, and thus its excavated volume will be greater than its *in situ* volume. However, when filled and compacted, it may occupy a less volume than when originally in situ. For example, light sandy soil is less by about 11% after filling, whilst large rocks may bulk by up to 40%. To allow for this, a correction factor is generally applied to the cut or fills volumes. (Schofield & Breach, 2007)

5.3 Area and Volume Computation

5.3.1 Estimation of Earth Work Quantities

The estimation of the quantity of material which must be excavated (cut), and the quantity of material required to raise the elevation of the existing ground (fill), is very useful in the development of a cost estimate for road projects. (AAiTlecture)

One of the major objectives in selecting a particular location for a highway is to minimize the amount of earthwork required for the project. Therefore, the estimation of the amount of earthwork involved for each alternative location is required at both the preliminary and final stages.

To determine the amount of earthwork involved for a given grade line, cross sections are taken at regular intervals along the grade line. The cross sections are usually spaced 50 ft. apart, although this distance is sometimes increased for preliminary engineering. (Garber, Traffic & Highway Engineering, 2009)

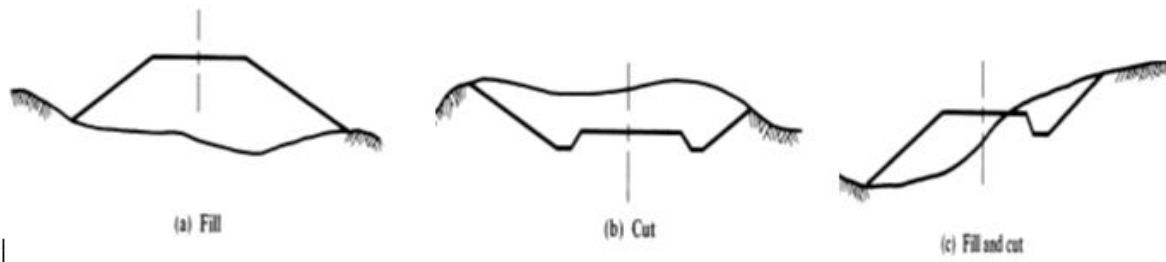


Figure 5-1: Typical Types of Earth Work Cross Sections (Garber)

These cross sections are obtained by plotting the natural ground levels and proposed grade profile of the highway along a line perpendicular to the grade line to indicate areas of excavation and areas of fill. Figure 5-1 shows three types of cross sections. When the computation is done manually, the cross sections are plotted on standard cross-section paper, usually to a scale of 1 in. to 10 ft. for both the horizontal and vertical directions. The areas of cuts and fills at each cross section are then determined by the use of a planimeter or by any other suitable method. Surveying books document the different methods for area computation. The volume of earthwork is then computed from the cross-sectional areas and the distances between the cross sections. (Garber, Traffic & Highway Engineering, 2009)

A common method of determining the end areas is the Coordinate method. With the coordinates of all the corners of a cross-section known, the area may be computed by means of the coordinate method. The point of intersection of the center of formation with the centerline of the road (c) is used as the origin. The cut above the formation are written as plus (+) and those below as minus (-). The distances to the right are written as plus (+) and those to the left as minus (-). (AAiT Lecture)

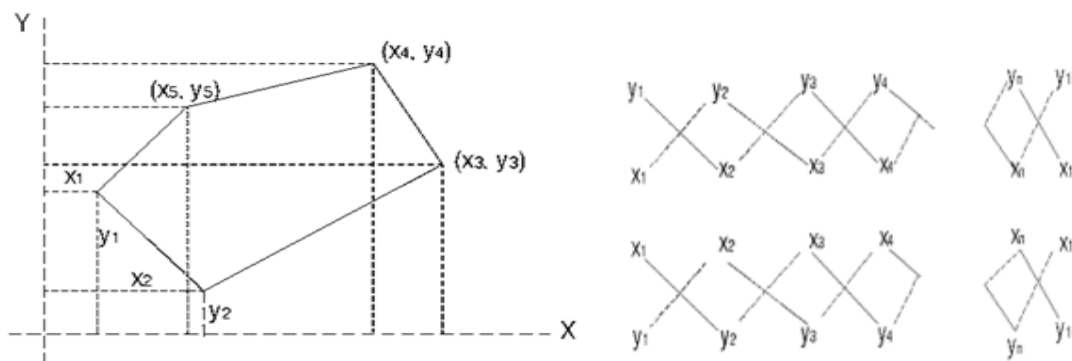


Figure 5-2: Coordinate method of Area Computation

The products of the coordinates along full lines is taken positive and along dashed lines negative. Thus, the area

$$Area = \frac{1}{2} [x_1(y_2 - y_n) + y_2(x_3 - x_1) + y_3(x_4 - x_2) + \dots + y_n(x_1 - x_{n-1})]$$

Volume sample calculation for one station

For station 5+520

PE=2203.063 CE=2204.749 LE=2205.65 RE=2204.4

Where PE=proposed elevation CE= center of elevation Le= left elevation

Re= right elevation

W=width of the road and L1 and L2= length of back slope

LE= left elevation – proposal elevation

LE=2205.65-2203.063=2.587

CE=center elevation –proposal elevation

CE= 2204.749-2203.063=1.686

RE=right elevation- proposal elevation

RE=2204.4-2203.063=1.337

The back slope=1;2 for L1=5.174 for L2=2.674

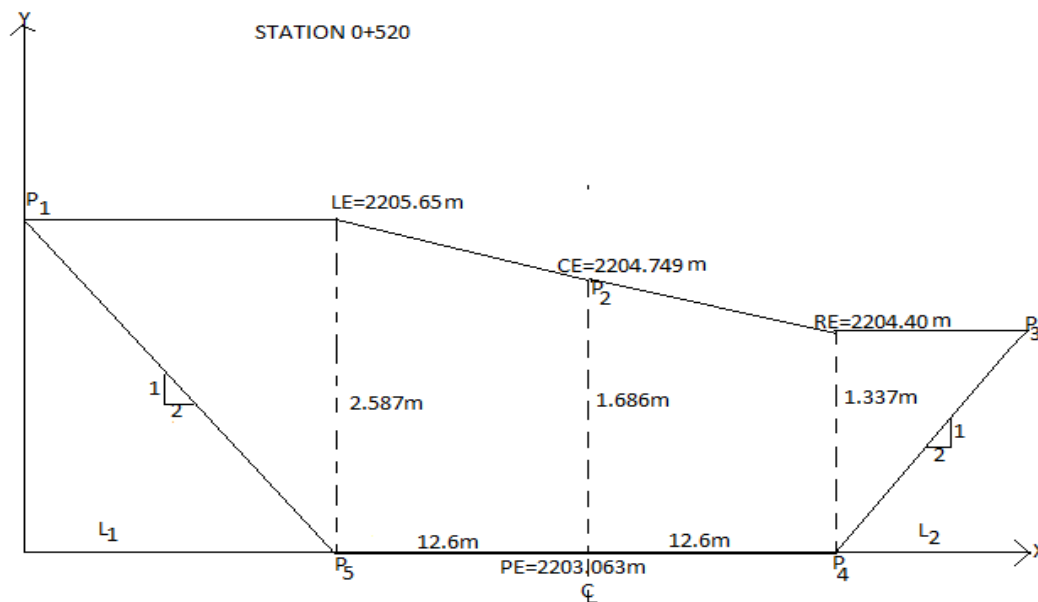


Figure 5-3: Cross Sectional Profile for Station 0+520

By using coordinate method, we are going to calculate the above cross section

Where P=coordinates (X, Y)

Finding coordinates starting P1

$$P1 = (0.2, 5.87)$$

$$P2 = (L1 + W/2, 1.686) = (17.774, 1.686)$$

$$P3 = (L1 + W + L2, 1.337) = (33.048, 1.337)$$

$$P4 = (30.374, 0) \text{ AND } P5 = (5.174, 0)$$

point	X(m)	Y(m)
P1	0	2.587
P2	17.774	1.686
P3	33.048	1.337
P4	30.374	0
P5	5.174	0

COORDINATES OF THE PROFILE

$$AREA = \left| \sum_{i=1}^n x_i y_{i+1} - \sum_{i=1}^n y_i x_{i+1} \right| / 2$$

$$A = |0(1.686 - 0) + 17.774(1.337 - 2.587) + 33.048(0 - 1.686) + 30.374(0 - 1.337) + 5.174(2.587 - 0)| / 2$$

$$A = |-69.168| / 2$$

$$A = 34.584 \text{ m}^2 \text{ cut}$$

For station 0+540

$$PE = 2202.935 \text{ CE} = 2205.342 \text{ LE} = 2206.1 \text{ RE} = 2204.65$$

Where PE=proposed elevation CE= center of elevation Le= left elevation Re= right elevation

W=width of the road and L1 and L2= length of back slope

LE= left elevation – proposal elevation

$$LE = 2206.1 - 2202.935 = 3.165$$

CE=center elevation –proposal elevation

$$CE = 2205.342 - 2202.935 = 2.407$$

RE=right elevation- proposal elevation

$$RE = 2204.65 - 2202.935 = 1.715$$

The back slope=1;2 for L1=6.328 for L2=3.43

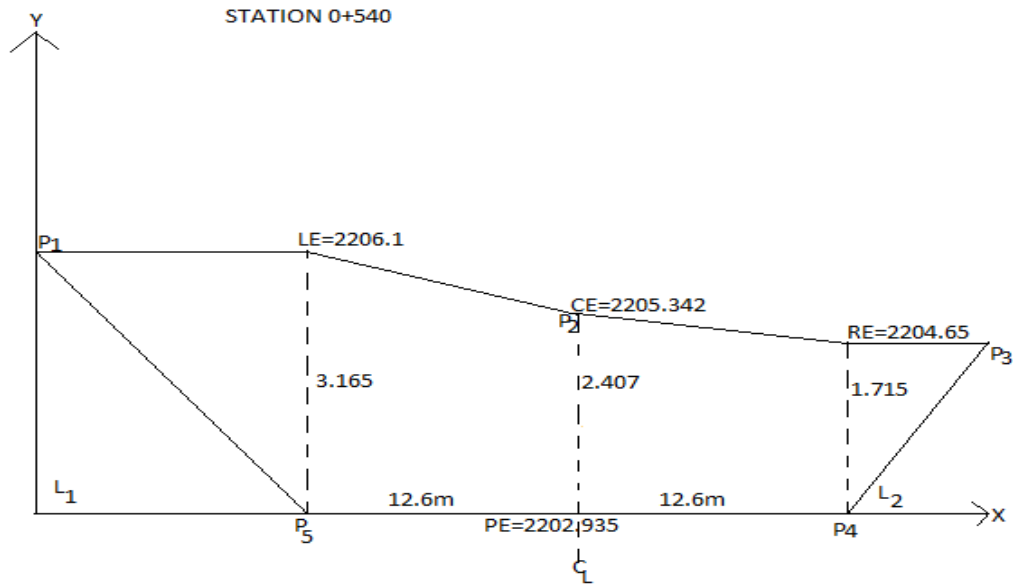


Figure 5-4: Cross Sectional Profile for Station 0+540

By using coordinate method we are going to calculate the above cross section

Where P=coordinates (X,Y)

Finding coordinates starting P1

$$P1 = (0, 3.165)$$

$$P2 = (L1 + W/2, 2.407) = (18.928, 2.407)$$

$$P3 = (L1 + W + L2, 1.715) = (34.958, 1.715)$$

$$P4 = (31.528, 0) \text{ AND } P5 = (6.328, 0)$$

point	X(m)	Y(m)
P1	0	3.165
P2	18.928	2.407
P3	34.958	1.715
P4	31.528	0
P5	6.328	0

COORDINATES OF THE PROFILE

$$\text{AREA} = \left| \sum_{i=1}^n x_i y_{i+1} - \sum_{i=1}^n y_i x_{i+1} \right| / 2$$

$$A = |0(2.407 - 0) + 18.928(1.715 - 3.165) + 34.958(0 - 2.407) + 31.528(0 - 1.715) + 6.328(3.165 - 0)| / 2$$

$$A = |-72.816|$$

$$A = 72.816 \text{ m}^2 \text{ cut}$$

Computing the volume with taking the average area of 0+520 and 0+540 then multiple with the length. Area of 0+520=34.584 and 0+540=72.816

$$A_{avg} = 34.584 + 72.816$$

$$A_{avg} = 53.70m^2$$

$$V = A_{avg} * L$$

$$V = 53.70 * 20$$

$$V = 1074m^3_{cut}$$

5.3.2 Mass haul diagram (MHD)

It is established design practice to achieve the most economical solution when designing a road. A key aspect in achieving this requirement is to seek a balance in earth works and to reduce the haulage distance from cuts and fills. Earth works computer systems identify the net quantities of cut and fill material together with the different types of cut materials. E.g. rock (non- rip able material). They will also produce a mass haul diagram which converts the net cut volumes to embankment volumes. Where a grade line is not constrained by other influences/restrictions, obtaining the most economical grading solution requires the designer to use a mass haul diagram, to enable him/her to balance the earth works, optimize the length of leads from cuts to fill, locate job borrow areas; and match the areas of spoil to the most appropriate spoil site(s). The information generated can then be used to adjust the profile until a satisfactory earth works volume and balance is achieved. ((AACRA), 2004)

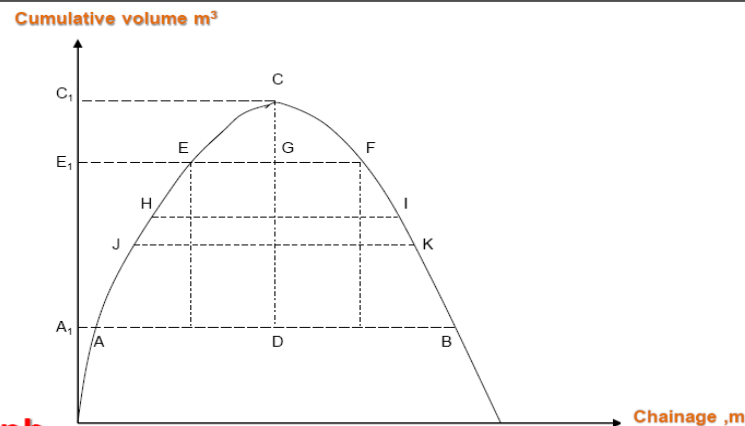
Construction of the mass haul diagram

The mass haul diagram is a curve plotted on a distance base. The ordinate at any point of which represents the algebraic sum up to that point of the volumes of cuttings and embankments from the start of the project or from any arbitrary point. In obtaining the algebraic sum, cuttings are considered positive and embankments negative. ((AACRA), 2004)

The designer in calculating earthwork quantities should allow for bulking and compaction factors (shrinkage and swell). In situ material when excavated increases in volume i.e. it bulks up (swell). When this material is placed and compacted it decreases in volume (shrinkage). Factors are specific to the material being handled and designers should seek advice from geotechnical engineers. ((AACRA), 2004)

Analyze the MHD

From the graph,



Line	Description
EF	Freehaul distance
CG	Freehaul volume
CD	Haul volume
HI	Average haul distance – centre line of CD
GD	Overhaul volume
JK	Average overhaul distance - centre line of GD

Figure 5-5: Mass Haul Diagram (AAiT Lecture note)

Definitions

- **Haul:** It is defined as the total volume of excavation multiplied by average haul distance. It is the area between the curve and balancing line,
- **Free-haul:** It is the distance up to which the hauling is done by the contractor free of charge. For this distance, the cost of transportation of the excavated material is included in the excavation cost.
- **Overhaul:** It is the excavated material from a cutting moved to a greater distance than the free-haul, the extra distance is overhaul.
- **Borrow:** The volume of material brought into a section due to a deficient
- **Waste:** The volume of material taken from a section due to excess
- **Limit of economical haul** is the maximum haul distance. When this limit is reached it is more economical to waste and borrow material. (Schofield & Breach, 2007)

Use of Mass-Haul Diagram

1. The distances over which cut and fill will balance.
2. Quantities of materials to be moved and the direction of movement.
3. Areas where earth may have to be borrowed or wasted and the amounts involved.
4. The best policy to adopt to obtain the most economic use of plant.

5. The best use of plant for the distances over which the volumes of cut and fill are to be. (Schofield & Breach, 2007)

Properties of the MHD

Consider Figure: 5.4 (a) in which the ground XYZ is to be leveled off to the grade line A-B. Assuming that the fill volumes, after correction, equal the cut volumes, the MHD would plot as shown in Figure: 5.4(b).

Thus:

- (1) Since the curve of the MHD represents the algebraic sums of the volumes, then any horizontal line drawn parallel to the base AB will indicate the volumes that balance. Such a line is called a *balancing line* and may even be represented by AB itself, indicating that the total cut equals the total fill.
- (2) The rising curve, shown broken, indicates cut (positive), the falling curve indicates fill (negative).
- (3) The maximum and minimum points of a MHD occur directly beneath the intersection of the natural ground and the formation grade; such intersections are called *grade points*.
- (4) As the curve of the MHD rises above the balance line AB, the haul is from left to right. When the curve lies below the balance line, the haul is from right to left.
- (5) The total cut volume is represented by the maximum ordinate CD.
- (6) In moving earth from cut to fill, assume that the first load would be from the cut at X to the fill at Y, and the last load from the cut at Y to the fill at Z. Thus, the haul distance would appear to be from a point mid-way between X and Y, to a point mid-way between Y and Z. However, as the section is representative of volume, not area, the haul distance is from the center of mass of the cut volume to the center of mass of the fill volume. The horizontal positions of these centers of mass may be found by bisecting the total volume ordinate CD with the horizontal line EF. (Schofield & Breach, 2007)

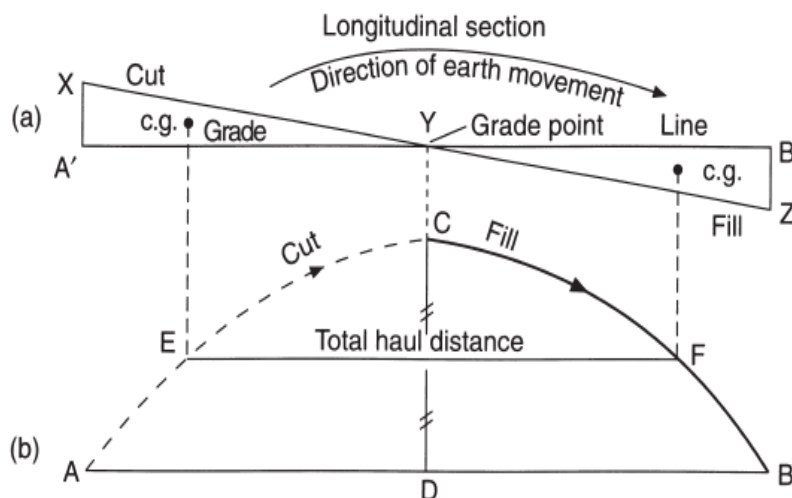


Figure 5-6: Mass Haul Diagram Properties

Table 5-1 Earth Work Volume computation

station	area	cut adjusted	fill adjusted	Cumulative volume
0+000.000	10.953	0	0	0
0+020.000	14.226	251.79	2.127	249.664
0+040.000	20.912	351.38	0	601.043
0+060.000	28.002	489.143	0	1090.187
0+080.000	29.015	570.175	0	1660.362
0+100.000	40.775	697.898	0	2358.26
0+120.000	32.966	737.404	0	3095.664
0+140.000	29.31	622.753	0	3718.417
0+160.000	31.874	611.834	0	4330.251
0+180.000	24.087	559.61	0	4889.861
0+200.000	21.956	460.43	0	5350.29
0+220.000	22.093	440.488	0	5790.778
0+240.000	20.226	423.193	0	6213.972
0+260.000	18.044	382.703	1.926	6594.749
0+280.000	16.688	347.316	2.782	6939.283
0+300.000	22.706	393.932	0.856	7332.36
0+320.000	22.747	454.524	1.491	7785.392
0+340.000	32.634	553.805	1.829	8337.368
0+360.000	24.113	567.472	0.338	8904.502
0+380.000	22.768	468.817	0.361	9372.958
0+400.000	23.072	458.402	0.361	9830.999
0+420.000	24.845	479.17	0	10310.17
0+440.000	29.385	542.299	0	10852.47
0+460.000	38.196	675.811	0	11528.28
0+480.000	46.881	850.776	0	12379.05
0+500.000	52.829	997.104	0	13376.16
0+520.000	50.406	1032.348	0	14408.51
0+540.000	64.342	1147.475	0	15555.98
0+560.000	56.97	1213.12	0	16769.1
0+580.000	44.851	1018.21	0	17787.31
0+600.000	47.223	920.738	0	18708.05
0+620.000	45.249	924.715	0	19632.77
0+640.000	42.723	879.714	0	20512.48
0+660.000	80.42	1231.422	0	21743.9
0+680.000	99.921	1803.409	0	23547.31
0+700.000	126.367	2262.883	0	25810.19
0+720.000	82.739	2091.061	0	27901.25
0+740.000	92.885	1756.243	0	29657.5
0+760.000	73.568	1664.53	0	31322.03
0+780.000	92.446	1660.14	0	32982.17

station	area	cut adjusted	fill adjusted	Cumulative volume
0+800.000	100.889	1933.35	0	34915.52
0+820.000	100.843	2017.321	0	36932.84
0+840.000	83.803	1846.46	0	38779.3
0+860.000	75.511	1593.141	0	40372.44
0+880.000	72.284	1477.951	0	41850.39
0+900.000	77.922	1502.061	0	43352.45
0+920.000	64.106	1420.286	0	44772.74
0+940.000	85.034	1491.407	0	46264.15
0+960.000	74.36	1593.94	0	47858.09
0+980.000	75.303	1496.627	0	49354.71
1+000.000	61.288	1365.912	0	50720.62
1+020.000	38.498	997.861	0	51718.49
1+040.000	21.545	600.431	0	52318.92
1+060.000	23.704	452.488	6.294	52765.11
1+080.000	47.162	708.66	6.294	53467.48
1+100.000	46.066	932.282	0	54399.76
1+120.000	42.939	890.054	0	55289.81
1+140.000	62.322	1052.618	0	56342.43
1+160.000	51.427	1137.497	0	57479.93
1+180.000	30.609	820.363	0	58300.29
1+200.000	15.322	459.306	163.453	58596.14
1+220.000	12.564	278.852	170.893	58704.1
1+240.000	6.958	195.219	36.846	58862.47
1+260.000	3.417	103.759	113.308	58852.93
1+280.000	1.618	50.35	132.352	58770.92
1+300.000	2.597	42.141	100.559	58712.5
1+320.000	1.223	38.2	103.801	58646.9
1+340.000	0.111	13.339	146.399	58513.84
1+360.000	0.003	1.133	197.896	58317.08
1+380.000	0.033	0.357	187.492	58129.94
1+400.000	0.35	3.832	133.626	58000.15
1+420.000	2.19	25.402	68.793	57956.76
1+440.000	7.993	101.834	24.865	58033.73
1+460.000	14.759	227.527	16.486	58244.77
1+480.000	19.61	343.696	11.09	58577.38
1+500.000	32.751	523.608	0	59100.98
1+520.000	44.629	773.797	0	59874.78
1+540.000	57.601	1022.302	0	60897.08
1+560.000	72.062	1296.627	0	62193.71
1+580.000	84.371	1564.33	0	63758.04
1+600.000	100.375	1847.461	0	65605.5
1+620.000	113.243	2136.176	0	67741.68
1+640.000	120.919	2341.614	0	70083.29

station	area	cut adjusted	fill adjusted	Cumulative volume
1+660.000	134.337	2552.552	0	72635.84
1+680.000	135.235	2695.72	0	75331.56
1+700.000	136.055	2712.901	0	78044.46
1+720.000	136.214	2722.685	0	80767.15
1+740.000	132.74	2689.539	0	83456.69
1+760.000	120.072	2528.122	0	85984.81
1+780.000	113.804	2338.756	0	88323.56
1+800.000	108.069	2218.728	0	90542.29
1+820.000	104.886	2129.548	0	92671.84
1+840.000	93.98	1988.652	0	94660.49
1+860.000	90.069	1840.484	0	96500.98
1+880.000	88.79	1788.59	0	98289.57
1+900.000	66.018	1548.083	0	99837.65
1+920.000	60.153	1261.716	0	101099.4
1+940.000	54.782	1149.35	0	102248.7
1+960.000	46.461	1012.428	0	103261.1
1+980.000	50.341	968.017	0	104229.2
2+000.000	45.323	956.631	0	105185.8
2+020.000	35.985	813.076	0	105998.9
2+040.000	30.588	665.733	0	106664.6
2+060.000	27.339	579.269	0	107243.9
2+080.000	22.25	495.886	0	107739.8
2+100.000	21.348	435.981	0	108175.7
2+120.000	19.392	407.403	0	108583.1
2+140.000	19.18	385.726	0	108968.9
2+160.000	25	441.803	0	109410.7
2+180.000	23.463	484.635	0	109895.3
2+200.000	30.977	544.408	0	110439.7
2+220.000	31.966	629.434	0	111069.1
2+240.000	36.622	685.884	0	111755
2+260.000	42.992	796.141	0	112551.2
2+280.000	45.862	888.538	0	113439.7
2+300.000	43.851	897.126	0	114336.8
2+320.000	49.182	930.328	0	115267.2
2+340.000	53.579	1027.617	0	116294.8
2+360.000	55.437	1090.165	0	117384.9
2+380.000	67.428	1228.649	0	118613.6
2+400.000	66.386	1338.138	0	119951.7
2+420.000	66.152	1325.378	0	121277.1
2+440.000	67.175	1333.269	0	122610.4
2+460.000	70.272	1374.466	0	123984.8
2+480.000	70.671	1409.422	0	125394.3
2+500.000	66.206	1368.765	0	126763

station	area	cut adjusted	fill adjusted	Cumulative volume
2+520.000	66.231	1324.367	0	128087.4
2+540.000	64.487	1307.177	0	129394.6
2+560.000	64.261	1287.481	0	130682.1
2+580.000	62.341	1266.027	0	131948.1
2+600.000	64.111	1264.519	0	133212.6
2+620.000	52.799	1169.097	0	134381.7
2+640.000	49.783	1025.823	0	135407.5
2+660.000	43.831	936.14	0	136343.7
2+680.000	24.545	683.762	0	137027.4
2+700.000	14.696	392.413	0.007	137419.8
2+711.477	5.023	113.155	0.231	137532.8
		139165.5	1632.757	

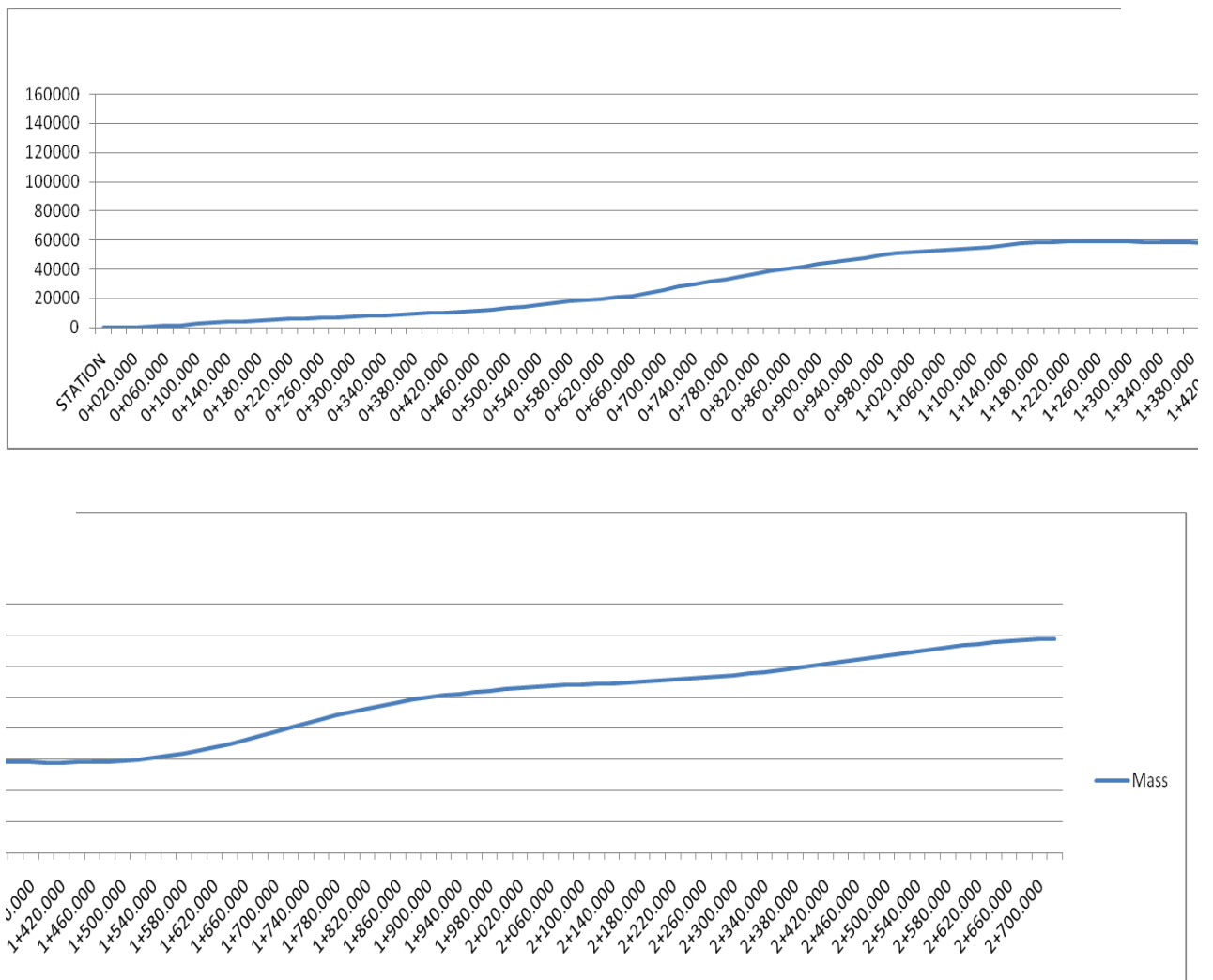


Figure 5-7 Mass Haul Diagram

Our project mass haul diagram is cut section because of the material is very weak ,to avoid misphasing, and also to attained the maximum gradient

CHAPTER SIX

PAVEMENT

6.1 Introduction

Highway pavements are divided into two main categories: rigid and flexible. The wearing surface of a rigid pavement usually is constructed of Portland cement concrete such that it acts like a beam over any irregularities in the underlying supporting material. The wearing surface of flexible pavements, on the other hand, usually is constructed of bituminous materials such that they remain in contact with the underlying material even when minor irregularities occur. Flexible pavements usually consist of a bituminous surface under laid with a layer of granular material and a layer of a suitable mixture of coarse and fine materials. Traffic loads are transferred by the wearing surface to the underlying supporting materials through the interlocking of aggregates, the frictional effect of granular materials, and cohesion of fine materials.

Flexible pavements are further divided into three subgroups: high type, intermediate type, and low type. High-type pavements have wearing surfaces that adequately support the expected traffic load without visible distress due to fatigue and are not susceptible to weather conditions. Intermediate-type pavements have wearing surfaces that range from surface treated to those with qualities just below that of high type pavements. Low-type pavements are used mainly for low-cost roads and have wearing surfaces that range from untreated to loose natural materials to surface treated earth. (J.Garber, Traffic & Highway Engineering, 2009)

BASIC CONCEPTS

The basic idea in building a road or parking area for all weather use by vehicles is to prepare a suitable sub grade or foundation provide necessary drainage, and construct a pavement that will;

- Have sufficiently total thickness and interval strength to carry expected traffic loads.
- Be sufficient impermeable to prevent the penetration or internal accumulation of moisture and
- Have atop surface that is smooth, skid resistance, and resistant to wear distortion, and deterioration by weather and deicing chemicals.

The sub grade ultimately carries all traffic loads; therefore, the structural function of a pavement is to support a wheel load on the pavement surface and transfer and spread that load to the sub grade without exceeding either the strength of the sub grade or the internal strength of the pavement itself. Wheel load W , being transmitted to the pavement surface through the tire at an approximately uniform vertical pressure p_0 . The pavement then spreads the wheel load to the sub grade so that the maximum pressure on the sub grade is only p_1 , by proper selection of pavement materials and with adequate pavement thickness p_1 will be small enough to be easily supported by the sub grade. (Asphalt Institute(MS-4), 2007)

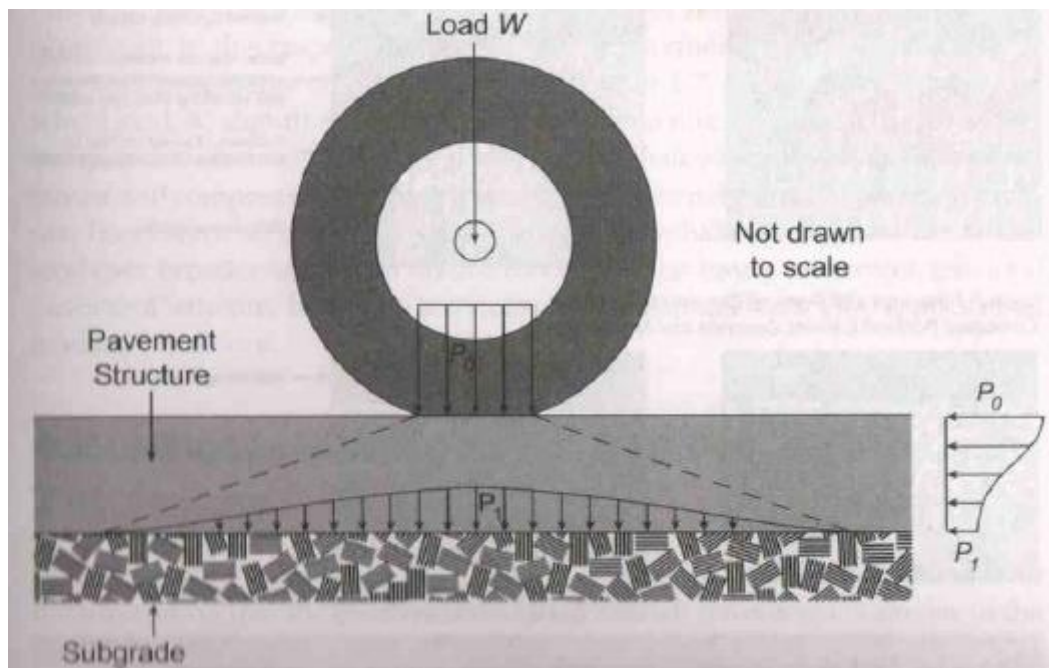


Figure 6-1: Stress Distribution on Pavements

STRUCTURAL COMPONENTS OF A FLEXIBLE PAVEMENT

Figure: 6.2 show the components of a flexible pavement: the sub grade or prepared Roadbed, the sub base, the base, and the wearing surface. The performance of the pavement depends on the satisfactory performance of each component, which requires proper evaluation of the properties of each component separately. (J.Garber, Traffic & Highway Engineering, 2009)

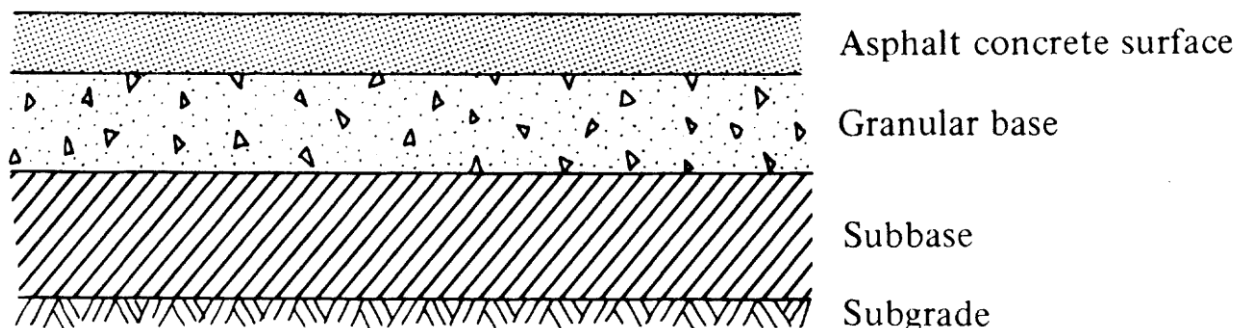


Figure 6-2: Structural Components of Pavements

6.2 Sub grade soil (Prepared Road Bed)

The sub grade is usually the natural material located along the horizontal alignment of the pavement and serves as the foundation of the pavement structure. It also may consist of a layer of selected borrow materials, It may be necessary to treat the sub grade material to achieve certain strength properties required for the type of pavement being constructed. This will be discussed later (J.Garber, Traffic & Highway Engineering, 2009)

SUBGRADE SOILS

Soil is the most important foundation and construction material for pavement structures.

- Foundation material for all pavements as undisturbed in situ sub grade material or Transported and reworked embankment material.

- Construction material for pavement structures either in its natural form (sand and Gravel) or in a processed form as stabilized layer.

Soil investigation is, thus, an integral part of the location, design and construction of Highways. (AAiT, Highway engineering 2)

Soil Surveys and Investigations

Soil survey for highway purposes involve the exploration of the soils along the highway routes and the identification of suitable soils for use as sub base and fill materials. The results of soil investigation provide pertinent information about soil and rock for a decision on one or more of the following subjects:

- i) Selection of roadway alignment;
- ii) Decision on the need for sub grade or embankment foundation treatment;
- iii) Investigation of slope stability in cuts and embankments;
- iv) Location and design of ditches and culverts;
- v) Selection and design of the roadway pavement;
- vi) Location and evaluation of suitable borrow and construction materials; and
- vii) Design of foundations for bridges and other structures.

In selecting the alignment for a new highway, the first step is normally to define a number of Conceivable corridors between the end termini of the road. The next step is to select the best Corridor for the proposed road and define within it one or more different alignments. These Alignments are compared, and a final selection is made for design purposes. The process Involves continuous searching and selecting, using increasingly more detailed knowledge of sub grade soils at each decision-making stage.

Before a field investigation is carried out at the site, preliminary information regarding soil condition can often be obtained from the following sources:

1. **Geological and agricultural soil maps** these often indicate the types of soil or geological formation that cover the area being investigated.
2. **Aerial photographs.** Terrain information visible on-air photos can be used for Identification of most of the common bedrock types associated residual soils, transported soils, and organic soils.
3. **Satellite images.** Satellite images are employed as a supplemental to air photos or as a substitute for air photos for geological studies and soil investigations.
4. **Area reconnaissance.** Reconnaissance survey aids in securing broad understanding of soil conditions and associated engineering problems that may be encountered.

The visual examination of vegetation cover, roadway cuts, and valleys in the area can give clue. The depth to water level in adjacent wells may indicate the elevation of the groundwater table. (AAiT, Highway Engineering 2)

7.3 Sub base Course

Located immediately above the sub grade, the sub base component consists of material of a superior quality to that which is generally used for sub grade construction. The requirements for sub base materials usually are given in terms of the gradation, plastic characteristics, and strength. When the quality of the sub grade material meets the requirements of the sub base material, the

sub base component may be omitted. In cases where suitable sub base material is not readily available, the available material can be treated with other materials to achieve the necessary properties. This process of treating soils to improve their engineering properties is known as *stabilization*. (J.Garber, Traffic & Highway Engineering, 2009)

7.4 Base Course

The base course lies immediately above the sub base. It is placed immediately above the sub grade if a sub base course is not used. This course usually consists of granular materials such as crushed stone, crushed or uncrushed slag, crushed or uncrushed gravel, and sand. The specifications for base course materials usually include more strict requirements than those for sub base materials, particularly with respect to their plasticity, gradation, and strength. Materials that do not have the required properties

can be used as base materials if they are properly stabilized with Portland cement, asphalt, or lime. In some cases, high-quality base course materials also may be treated with asphalt or Portland cement to improve the stiffness characteristics of heavy-duty pavements. (J.Garber, Traffic & Highway Engineering, 2009)

7.5 Surface course

Asphalt pavement is a general term applied to any pavement that has a surface constructed with asphalt normally, it consists of a surface course(layer) of mineral aggregate coated and cemented with asphalt, and one or more supporting courses which maybe the following

- Asphalt base, consisting of asphalt –aggregate mixtures
- Crushed stone (rock), slag or gravel
- Portland cement concrete

An asphalt pavement structure consists of all asphalt-bound layers and unbound (gravel or crushed stone) layers above the prepared sub grade. The upper or top layer consists of an asphalt surface. This surface can be a chip seal (for low-volume roads) or a high quality HMA for higher-volume roads and parking lots. The surface of an asphalt pavement exposed to vehicular traffic must be tough to resist distortion and provide a smooth, skid-resistant riding surface. It must be waterproof and sloped to shed surface water to the roadside, thus protecting the entire asphalt pavement structure and the sub grade from the weakening effects of moisture. It must resist wear caused by traffic and still retain necessary anti- skid properties. It must also be bonded to the layer or course beneath it. (Asphalt Institute(MS-4), 2007)

Pavement design process

The aim of structural design is to limit the level of pavement distress, measured primarily in terms of riding quality, rut depth and cracking to predetermined values. It is assumed that adequate routine and periodic maintenance is carried out during the design period of the road and that at the end of the design period a relatively low level of deterioration has occurred. (AACRA, pavement design manual, 2004)

- There are fundamental steps to be followed in designing a new road pavement standard axles(ESA) that will use road over the selected design life;

- Assessing the strength of the sub grade soil over which the road built and the environmental and cross-sectional effects;
- Selecting the most economical combination of pavement materials and layers thicknesses that will provide satisfactory service over the design life of the pavement (it is usually necessary to assume that an appropriate level of maintenance is also carried out).

Course Thickness

Because of normal construction level tolerances and the consequential variation in pavement course thickness, minimum course thickness must be applied to ensure the assumption of the structural design is achieved.

Granular courses should not normally be less than a minimum of 125mm thick. However, where the granular portion of the pavement is between 200mm and 250mm thick, this may be reduced to a minimum of 100mm for two separated courses if required.

Asphalt courses less than 50mm thick should not be seen as contributing structurally to the pavement but as a surfacing only. Surfacing asphalt courses are generally 35mm thick. Asphalt courses 50mm and greater are treated as structural courses. (AACRA, pavement design manual, 2004)

Sub grade soil CBR result and layer of thickens selection

For this project two samples from two different stations have been taken. The CBR value of the first sample is 3.1%, whereas the CBR value of the second sample is 9.2%. Because the CBR values of the two samples are greater than 3%, we can use the material as a sub grade as per AACRA manual. In order to be safe, we have decided to use the CBR value of 3.1% as our design sub grade CBR value. But since the AACRA chart is produced for CBR values of 3, 5, 10, 15 and 20 we have decided to use a CBR value of 3%. This is less than our average CBR value to be safe.



Figure 6-3 During Sampling



Figure 6-4 during Weighing



Figure 6-5 During Soaking



Figure 6-6 During Dial Reading

$$subgarde = N_{ss} * 365 * GF$$

$$cementedmaterial = N_{sc} * 365 * GF$$

$$N_{as} = 1.1N_{where}$$

$$asphalt = N_{as} * 365 * GF$$

$$N = N_{LEFl} + N_{mEFm} + N_{hEFh} + N_{aEFa}$$

N=IS the initial daily traffic loading is calculated from in accordance with section 6 of AACRA manual. EF Equivalent factor can be calculated by

$$EF = \left(\frac{axleload\ KG}{8160} \right)^{4.5} \quad \text{or from the table we can obtain respective equivalent factor}$$

NL is the daily number of light vehicles

NM is the daily number of medium vehicles

NH is the daily number of HEAVY vehicle

Na is the daily number of articulated vehicles

Here we take typical value for calculation

$$N = 384 * 0.03 + 0 * 0.73 + 11 * 1.31 = 395 \text{ EAL/d}$$

$$N_{as} = 1.1N = 1.1 * 395 = 435.5 \text{ veh/day}$$

$$asphalt(CESAL) = N_{as} * 365 * GF = 435.5 * 365 * 23.3 = 3598000 = 1.2 * 10^6$$

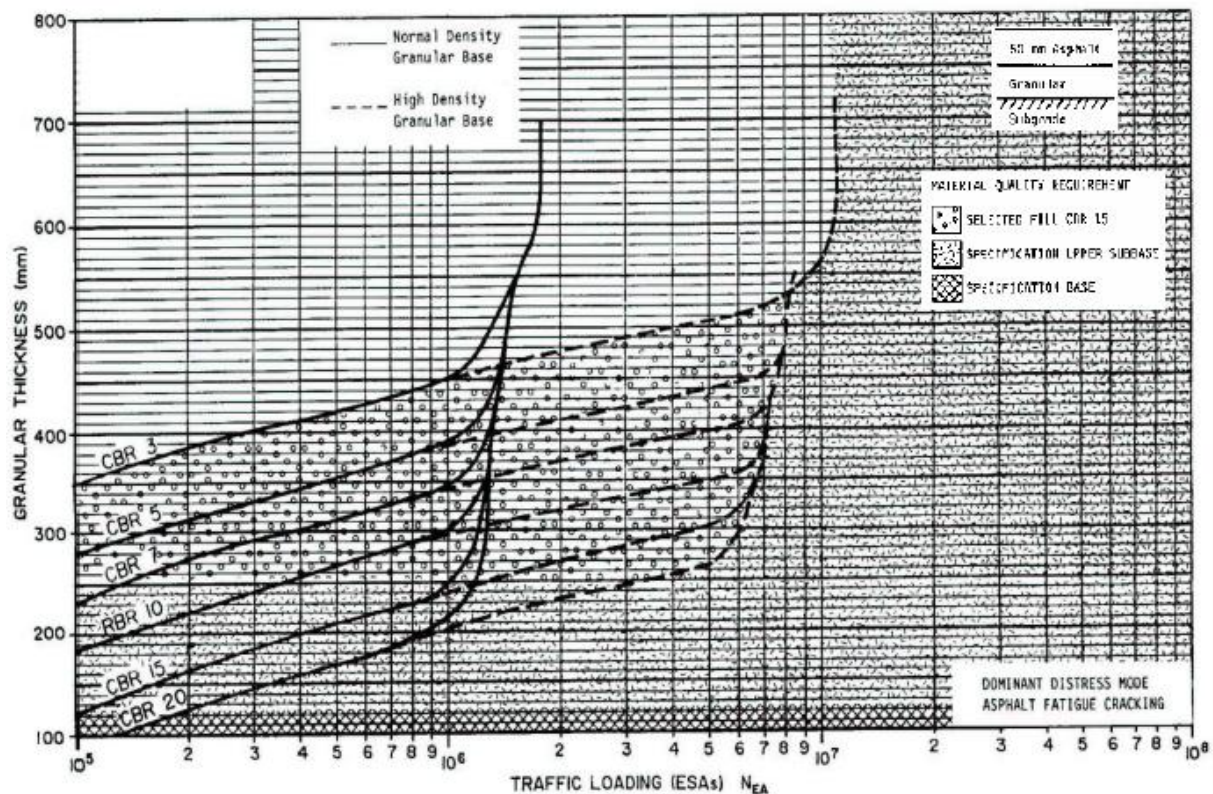


Figure 6-7: AACRA Pavement Design Char

We designed the following pavement according to AACRA manual.

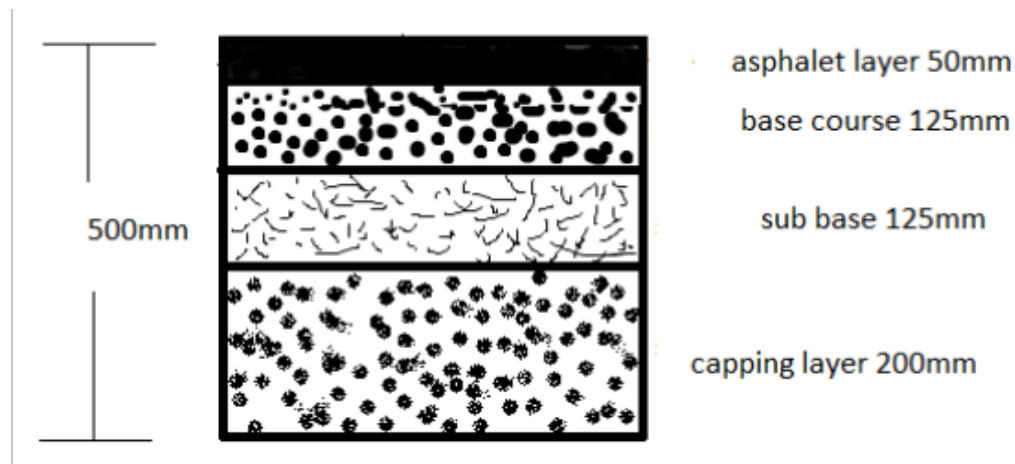


Figure 6-8: Designed Pavement Thickness of the Project

Recommendation: since our project road is subjected to dominantly of cars implies that the pavement will be durable for the designed period.

Stage Construction

Because weight and traffic volume will normally increase each year, pavement originally built thick enough to handle immediate traffic volume may not be thick enough and strong enough to handle future needs, with asphalt pavement, this problem can be economically met by first building the thickness required to cope with initial and short-term loading and then adding, when needed, additional layers of asphalt to increase total pavement thickness. This procedure is called stage construction. it avoids extensive investment in the beginning; when a new asphalt pavement layers is added; the wearing surface is equal to or better than the original.

When road-building funds are limited, a strong all-weather asphalt base can be placed and opened to traffic for a year or more. When additional constructed funds are opened to traffic for a year or more. When additional construction funds are available, the asphalt base can be placed on all new streets to serve until the development is finished and heavy construction traffic ceases. The asphalt surface is then added after first repairing any damage or settlement suffered by the base while supporting the construction traffic. The streets are then at full strength and in an attractive condition appropriate to a new living area. (Asphalt Institute(MS-4), 2007)

CHAPTER SEVEN

DRAINAGE

Introduction

Drainage is one of the most important factors in pavement design. Until recently this factor had not received the attention it merits. One misconception is that good drainage is not required if the thickness design is based on saturated conditions. This concept may have been true during the old days when the traffic loading and volume were small. As the weight and number of axle load increase, water can cause more damage to pavement, such as by pumping and by degradation of paving material

Not merely by loss of shear strength. Theoretically; an internal drainage is not required if the infiltration into the pavement is smaller than the drainage capacity of the base, sub base, and sub grade. Because the infiltration and drainage capacity vary a great deal and are more difficult to estimate. (Houng, 2004)

EFFECT OF WATER ON PAVEMENT

1. It reduces the shear strength of unbounded granular material and sub grade soil.
2. It causes pumping of concrete pavement, with subsequent faulting, cracking, and general shoulder (parking) deterioration.
3. Under high hydrodynamic pressure generated by moving traffic, pumping of fines in the base course of flexible pavement can also occur, with resulting loss of support.
4. Water cause differential heaving over swelling soil

METHOD OF CONTROLLING WATER IN THE PAVEMENT

- **PREVENTION;** in order to minimize infiltration of surface water in to pavement Good surface drainage; transverse slopes are introduced in all tangent section of road way. Except where super elevation of curves directs all water toward the inside. apply Strong pavement section implies full depth asphalt.
- **REMOVAL;** this is after water enters the pavement layer should remove quickly before damage happen using either of the following
 - Drainage layer or blanket; on top of sub grade as extra layer or not considered in thickness design or base course itself if it satisfies strength of base course and permeability for drainage layer. Geotextiles are filter fabrics that can be used to protect the drainage layer from clogging
 - Longitudinal drain
 - Transverse drains like culvert. (Huang, 2004)

7.2 Structures Required for Drainage

There are two general categories of drainage structure;

1. **Major structure**; long span bridge & multiple span culvert (clear span >20ft)
2. **Minor structure**; short span bridge & culvert (clear span <20ft). (Nicholas Garber)

Highway drainage systems are designed to convey predetermined discharges in order to avoid significant flood hazards. Provisions are also made to convey floods in excess of the predetermined discharges in a manner that minimizes flood hazards. Flood discharges are often referred to as peak discharges as they occur at the peak of the stream's flood hydrograph (discharge over time). Peak discharge magnitudes are a function of their expected frequency of occurrence, which in turn relates to the magnitude of the potential damage and hazard.

The highway designer's main interest in hydrology rests in estimating runoff and peak

Road geometry

It includes longitudinal grade and transverse cross fall will influence length of flow paths and speed at which excess rainfall will shed from the travel lane surface. The most effective method of reducing the depth of flows on road pavement is by limiting the length of flow paths. At super elevation transition where the most critical location to consider flow depth designer can either increase Rate of rotation but with satisfactory appearance, or reduce width of contributing width.

7.2.1 Roadside Ditches (drains)

They capture water from pavement or adjacent verge or both.

Consideration in road side drain selection

- Road location: curbed in urban that provide barrier between pavement and verge and also helps to define vehicle paths especially at intersection.
- Verge level: flood protection
- Containment: for high fill embankment
- Water cycle:

Examples of side drain; Krebs, channel & gutters. And other side drains are table drain, catch rain (for rural area) & depressed median.

7.2.2 CROSS DRAIN STRUCTURE (CULVERT)

Culvert comprises of three basic components: the inlet section of conduit and outlets. All these components in addition to the water level at the outlets and elevation of the road surface will control hydraulic performance of the culvert.

DESIGN CONSIDERATION of culvert

- Head water; the head water depth for culvert is the difference between the up stream elevated water depth and the invert of the inlet. As head water rise the velocity in the culvert will also increase.
- Existing water course alignment: generally, culverts should be aligned with the original channel.
- Flat terrain: here a series of culvert in parallel spread across a road embankment is more favorable than one localized high capacity culvert.

Culvert grade

The minimum culvert grade should much that of natural channel slope.

SURFACE INLETS AND OUT LETS

After roadside drain have captured surface runoff from the road or verge. Regular inlets to a piped system or alternative outlets to waterways are required to the capacity level otherwise flooding.

Types of inlets

- Kerb or side entry opening: with deflector slab at steep grade
- Grated opening on range of grades but with problem of blockage.
- Combined of the two

Location of inlet structure

- Low points in road side drain(sag) here small spacing between the pits
- Super elevation transition to ensure flow from one road drain does not cross the carriageway to adjacent drain/gutter.
- In advance of pedestrian crossing
- In advance of intersection
- At location for maintenance purpose such as at change in direction or maximum spacing for cleaning purpose (maximum 100m adopted for these reasons. ((AACRA), 2004)

Issues with respect to vehicle operation and road surface drainage

- ✓ **Aquaplaning** (hydroplaning): film of water on road that can result in loss of friction between the tire and road. But in our cause, this is not an issue because our design speed is 60km/h which is less than 80 km/h.
- ✓ **Spray**: vehicle spray during and immediately after rain event reduces visibility of another vehicle, pedestrian, and pavement marking.
- ✓ **Roadway capacity**: as water encroaching into travel lane will affect roadway capacity.
- ✓ **Vehicle stability**: stability of vehicle in floodwater is a function of water depth and also velocity. (AACRA, DRINAGE SEC-10)

HYDROLOGY

In designing road surface drainage estimated peak flows are required for a number of different recurrence intervals.

- ✓ **Frequent storms:** must be considered for the depth of flow on road pavement to ensure that is no loss in stability of vehicle (aquaplaning). aquaplaning is normally only considered for frequent storm.
- ✓ **Minor storms:** storm water structure need to be located at suitable spacing along roads to prevent nuisance flooding and inconvenience to both motorist and pedestrian.
- ✓ **Major storms:** this must be considered to ensure that flows are conveyed safely during these rare events

Major/Minor Drainage System

- Minor system: is sized to limit inundation during more common storm events. Recommended ARI=5year, ARI= 10 year (for town center).
- ✓ Major system: is designed to convey less frequent major storm event wit out loss of life or significant damage of property. (ARI=100 year recommended value if economical and practical). ((AACRA), 2004)

Rainfall Intensity

- Quantity of runoff depends on intensity and duration.
- Duration = Time of Concentration
- ✓ the time required for water from the remotest place to reach a specific point on the • Rain water expelled from the road surface

i. Infiltration

ii. Runoff

iii. Evaporation- insignificant

Infiltration depends on:

- ✎ Type and gradation of soil
- ✎ Soil covers, moisture content of the soil
- ✎ Presence of impervious layers near the surface.
- ✎ Rate of infiltration on bare soil is less than on a turfed soil.
- ✎ Frozen soil is impervious
- ✎ Rate of infiltration is assumed to be constant during any specific design storm

Run off depends on:

- ✎ Nature of the ground, degree of saturation, and slope of the surface
- ✎ Rate of runoff greater on smooth surfaces

Computation of Runoff

The most widely used flow estimation methods:

- Rational Method;
- NRCS Runoff Curve Number Methods;

- Statistical analysis of stream data; and
- Regional regression equations.

Rational Formula- accurate way of estimating runoff up to areas of 0.5 km²

$$Q = 0.00278 \cdot CIA$$

Where;

Q= runoff (m³/sec)

C=coefficient, representing ratio of runoff to rainfall

I= intensity of rainfall (mm/hr) for a duration equal to the time of concentration

A= catchment area tributary to the design location, ha

Since our catchment area is less than 50 ha OR 0.5km² We use the rational method to estimate the peak runoff rate.

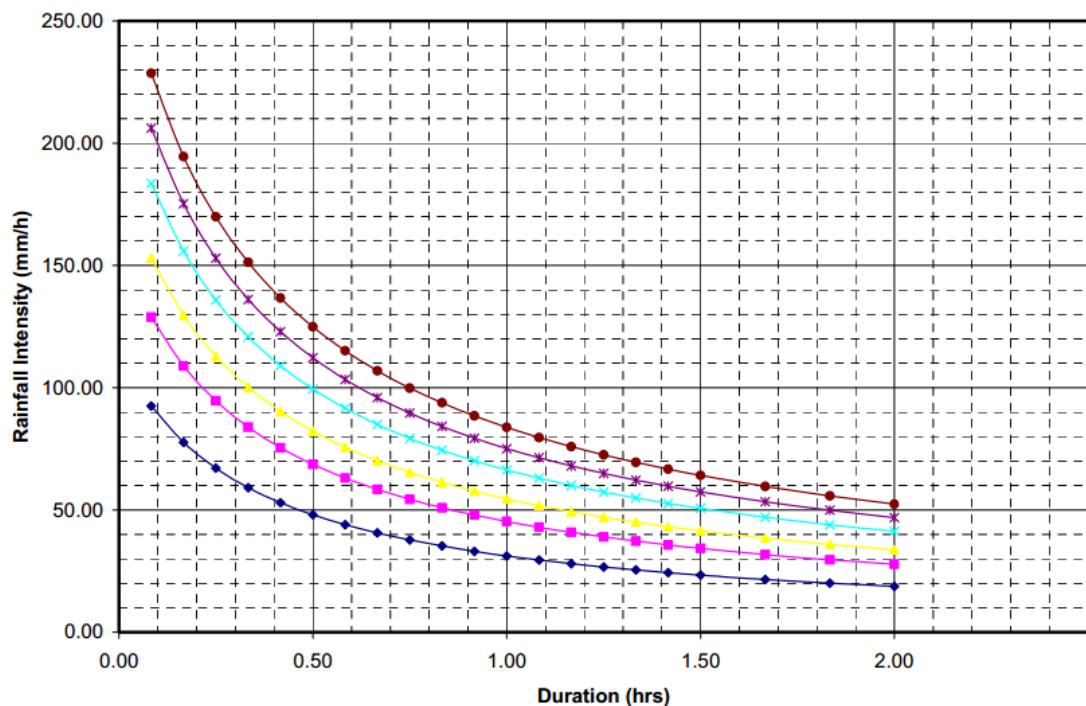


Figure 7-0-1: Intensity Duration Curve for Addis Ababa BoleBulbulla (AACRA)

The following steps are followed in order to compute surface drainage.

- Step 1: Determine Catchment Area
- Step 2: Determine longest flow path and elevations.
- Step 3: Determine Catchment Property
- Step 4: Calculate Time of concentration we use velocity method .
- Step 5: Determine rainfall intensity we use the provided below

- Step 6: Determine runoff coefficients(roughness coefficient C)

HERE we considered weighted value since the catchment area of our project is including of grass and developed land.

- Step 7: Calculate the Peak flood we use rational method.

Once the quantity of runoff is known, the design of ditches and similar structures is based on the principles of open channel flow.

Design the channel as carried out using the Manning formula.

$$Q = V \cdot A \quad V = \frac{1}{n} R^{2/3} S^{1/2}$$

Where:

V= mean velocity (m/sec)

R= hydraulic radius (m)= Area/wetted perimeter

S=slope of the channel (m/m)

n=Manning's roughness coefficient

sample calculation on drainage structure;

Calculation on drainage structure design

Rational formula

$$Q = \frac{CIA}{360}$$

The following steps are followed.

- Step 1: Determine Catchment Area is an area which contribute flow for the drainage system.

We consider four different catchment area in order to adjust the discharge not cause flooding. we read and approximate the following areas from Google Earth

$$A_1 = 10 \text{ ha}$$

- Step 2: Determine longest flow path and elevations.

$$L = 0.750 \text{ km}, \text{ slope} = (\text{Elv}_2 - \text{Elv}_1) / (1000 * 0.75) = 3\%$$

- Step 3: Determine Catchment Property

Our project road catchment is consists grassland, cobble & asphalt.

- Step 4: **Calculate Time of concentration** : is defined as the time required for a particle of water to flow from the hydraulically most distant point in the watershed to the outlet or design point.

Table 7-1 Intercept Coefficients for Velocity vs. Slope Relationship (AACRA)

Land Cover/ Flow Regime	k_1
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213
Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow); alluvial fans in western mountain regions	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area (shallow concentrated flow); small upland gullies	0.619

$$Tc = L/60V, V = K1 * S^{0.5},$$

Where;

K1 is intercept coefficient

S is slope in percent.

For paved area from the above table $K1 = 0.619$

$$V = 0.619 * 3^{0.5} = 1.072 \text{ m/s}$$

catchment area velocity

$$Tc = 750/60 * 1.072 = 11.66 \text{ min.} = 0.194 \text{ hr}$$

- Step 5: Determine rainfall intensity
- TABLE: design storm selection guidelines

Table 7-2 Road Categories and their Design flood return period

Roadway Classification	Exceedence Probability	Return Period
Urban Principal Arterial System	4% - 2%	25-50-year
Urban Minor Arterial System	4%	25-year
Urban Collector Street System	10%	10-year
Urban Local Street System	20%-10%	5-10-year

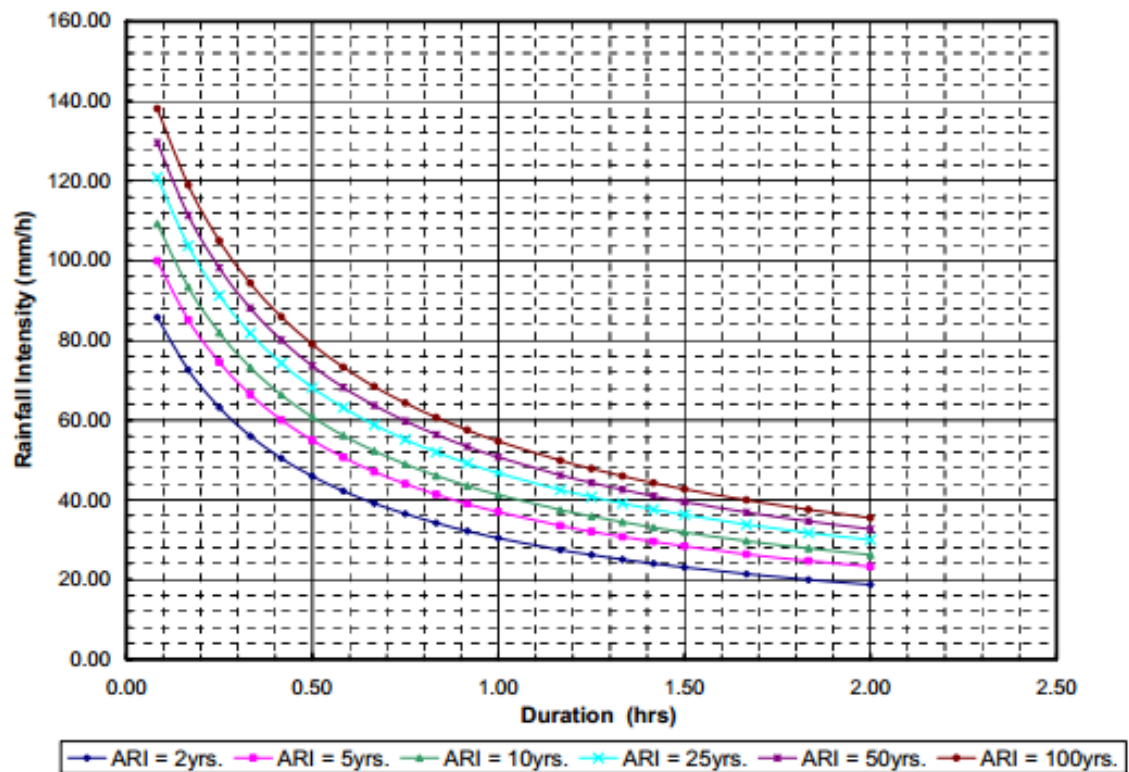


Figure 7-2 Intensity - Duration - Frequency Curve (Addis Ababa) (AACRA)

NB. We considered minor system of drainage hence average return interval (10years) for urban collector road.

For ARI=10 yrs. & $T_c = 11.66/60 = 0.194$ hrs we read $i = 90$ mm/h

- Step 6: Determine runoff coefficients (c)

Table 7-3 Runoff Coefficients for Rational Formula (AACRA)

Type of Drainage Area	C
Business:	
Commercial area	0.70-0.95
Neighbourhood areas	0.50-0.70
Residential:	
Single-family areas	0.30-0.50
Multi-units, detached	0.40-0.60
Multi-units, attached	0.60-0.75
Suburban	0.25-0.40
Apartment dwelling areas	0.50-0.70
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
Lawns:	
Sandy soil, flat, < 2%	0.05-0.10
Sandy soil, average, 2 to 7%	0.10-0.15
Sandy soil, steep, > 7%	0.15-0.20
Heavy soil, flat, < 2%	0.13-0.17
Heavy soil, average 2 to 7%	0.18-0.22
Heavy soil, steep, > 7%	0.25-0.35
Streets:	
Asphaltic	0.70-0.95
Concrete	0.80-0.95
Brick	0.70-0.85
Drives and walks	0.75-0.85
Roofs	0.75-0.95

By assumung area1 consists of 35%(10ha)= ggrass=3.5ha,65%developed=6.5ha

C1=0.2,C2=0.675

$$C_w = (C_1 \cdot a_1 + C_2 \cdot a_2) / A_1$$

Where : C_w =weighted roughness coeffiecnt

C1= roughness coeffiecnt for grass area& a1 area for grass area

C2== roughness coeffiecnt for developed area& a2 area for developed area

$$C_w = (0.2 \cdot 3.5 + 0.675 \cdot 6.5) / 10 = 0.51$$

- Step 7: Calculate the Peak flood

$$\text{➤ } Q = \frac{CIA}{360} = \frac{0.51 \times 90 \times 10}{360} = 1.275 \text{ m}^3/\text{sec}$$

Now let us design the most economical drain structure (ditch) which can convey safely this much discharge.

- Once the quantity of runoff is known, the design of ditches and similar structures is based on the principles of open channel flow.
- Manning's formula assumes steady flow in a uniform channel

The Manning's Formula

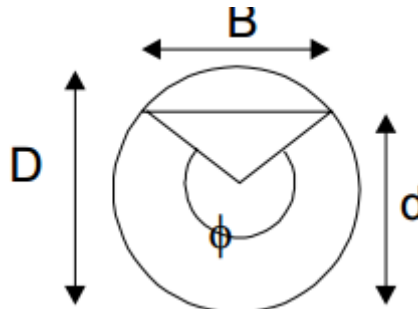
$$Q = V * A$$

$$V = \frac{1}{n} * R^{\frac{2}{3}} * S^{\frac{1}{2}}$$

Where:

V= mean velocity (m/sec)

R= hydraulic radius (m)= Area/wetted perimeter



For circular section

$$A = \frac{1}{8} ((\phi - \sin \phi) D^2$$

$$P = \phi * \frac{D}{2}$$

The hydraulic radius

$$R = \frac{1}{4} \left(1 - \frac{\sin \phi}{\phi} \right) D$$

For most efficient cross section, using the Manning equation, making the perimeter (P) minimum we get the value of $\phi = 151^\circ$

$$A = \frac{1}{8} ((2.634 - \sin 151^\circ) D^2 = 0.269 D^2$$

$$R = \frac{1}{4} \left(1 - \frac{\sin(\phi)}{\phi} \right) D = 0.204 D$$

S=slope of the channel (m/m) here we consider this slope to be same as the natural ground slope.

$$s=1.773\%$$

$$n=\text{Manning's roughness coefficient}=0.013$$

Table 7-4 Recommended Values for Surface Roughness

Type of Pipe	Manning's n
Reinforced Concrete	0.013
Fibre Reinforced Concrete	0.013
UPVC	0.011

$$V = \frac{1}{nR} S^{1/2}$$

$$= \frac{1}{0.013} 0.204D^{2/3} (0.01773)^{0.5}$$

$$Q = V \cdot A$$

$$1.275 = \left(\frac{1}{0.013} 0.204D^{2/3} (0.01773)^{0.5} \right) (0.269D^2)$$

D=1.119 use D=1.2m but by checking V which have max of 6 m/s.

$$V = \frac{1}{nR} S^{1/2}$$

$$= \frac{1}{0.013} 0.204D^{2/3} (0.01773)^{0.5}$$

$$= \frac{1}{0.013} 0.204 * 1.119^{2/3} (0.01773)^{0.5} = 3.83 \text{ m/s ok but we can still increase slope in order to decrease diameter and to be economical.}$$

Let's provide slope 3%

$$1.275 = \left(\frac{1}{0.013} 0.204D^{2/3} * (0.03)^{0.5} \right) (0.269D^2)$$

There for D=1.009 here v=4.65 ok!

Recommendation provide pipe of diameter 1m

And the rest of the calculation is completed on the excel sheet below;

Table 7-5 Drainage Pipe Design Summary

V	Tc	I	C	Q	Bed slope	s	Dc	Dr	VELOCITY
1.072139	0.194315519	90	0.51	1.275	0.03	1.242054	1.00986585	1	4.592715
1.072139	0.168406783	92	0.489	1.749533	0.035	1.341571	1.10469065	1.1	5.287812
1.072139	0.090680575	108	0.5	0.705	0.038	1.397885	0.7736047	0.8	4.451137
1.072139	0.207269887	84	0.499	2.86426	0.0472	1.55794	1.25653081	1.3	5.914049

NB, since all the channel velocities are below the maxi limit 6m/s we are on the safe side.

Where; V=velocity

T_c =time of concentration (hrs)

I =rainfall intensity (mm/h)

C =roughness coefficient

Q =discharge (m^3/sec)

S =bed slope (m/m)

D_c = computed Diameter

D_r = Recommended Diameter

Design of culvert is ignored in our case because we recommend 2 m diameter pipe for cross drainage. Since AACRA manual allow diameter of pipe till 2.4m.

CHAPTER EIGHT

SIGN POSTS AND MARKINGS

Traffic sign controls

Traffic sign control device is the medium used for communicating between traffic engineer and road users. Unlike other modes of transportation, there is no control on the drivers using the road. Here traffic control devices come to the help of the traffic engineer. The major types of traffic control devices used are- traffic signs, road markings, traffic signals and parking control. In general, traffic signs fall into one of Three major categories:

Regulatory signs; - Regulatory signs convey information concerning specific traffic regulations. Regulations may relate to right-of-way, speed limits, lane usage, parking, or a variety of other functions.

Warning signs; -Warning signs are used to inform drivers about upcoming hazards that they might not see or otherwise discern in time to safely react.

Guide signs; -Guide signs provide information on routes, destinations, and services that drivers may be seeking.

Regulatory signs

Regulatory signs shall be used to inform road users of selected traffic laws or regulations and indicate the applicability of the legal requirements. Regulatory signs shall be installed at or near where the regulations apply. The signs shall clearly indicate the requirements imposed by the regulations and shall be designed and installed to provide adequate visibility and legibility in order to obtain compliance. (Rog

The regulatory signs can be further classified into; -

- Right of way series
- Speed series:
- Movement series:
- Parking series:
- Pedestrian series:
- Miscellaneous

Right of way series

These include two unique signs that assign the right of way to the selected approaches of an intersection. They are the STOP sign and GIVE WAY sign for example, when one minor road and major road meets at an intersection, preference should be given to the vehicles passing through the major road. Hence the give way sign board will be placed on the minor road to inform the driver on the minor road that he should give way for the vehicles on the major road. In case two major roads are meeting, then the traffic engineer decides based on the traffic on which approach the sign board has to be placed. Stop sign is another example of regulatory signs that comes in right of way series which requires the driver to stop the vehicle at the stop line. (Ro



Figure 8-1: Right of Way Speed Series

Number of speed signs may be used to limit the speed of the vehicle on the road. They include typical speed limit signs, truck speed, minimum speed signs etc. Speed limit signs are placed to limit the speed of the vehicle to a particular speed for many reasons. Separate truck speed limits are applied on high speed roadways where heavy commercial vehicles must be limited to slower speeds than passenger cars for safety reasons. Minimum speed limits are applied on high speed roads like expressways, freeways etc. where safety is again a predominant reason. Very slow vehicles may present hazard to themselves and other vehicles also.

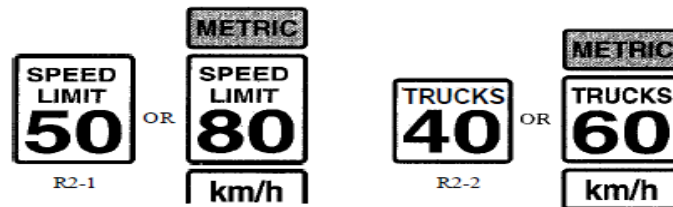


Figure 8-2:Speed Series

Movement series

They contain a number of signs that affect specific vehicle maneuvers. These include turn signs, alignment signs, exclusion signs, one way signs etc. Turn signs include turn prohibitions and lane use control signs. Lane use signs make use of arrows to specify the movements which all vehicles in the lane must take. Turn signs are used to safely accommodate turns in UN signalized intersections

Parking series

They include parking signs which indicate not only parking prohibitions or restrictions, but also indicate places where parking is permitted, the type of vehicle to be parked, duration for parking etc.

Pedestrian series:

They include both legend and symbol signs. These signs are meant for the safety of pedestrians and include signs indicating pedestrian only roads, pedestrian crossing sites etc.

Miscellaneous:

Wide variety of signs that are included in this category are: a "KEEP OF MEDIAN" sign, signs indicating road closures, signs restricting vehicles carrying hazardous cargo or substances, signs indicating vehicle weight limitations etc. source ;-(Tom V. Mathew and K V Krishna Rao) and (traffic engineering roger)



Figure 8-3:Example of Regulatory

Warning signs

Warning signs call attention to unexpected conditions on or adjacent to a highway or street and to situations that might not be readily apparent to road users. Warning signs alert road users to

conditions that might call for a reduction of speed or an action in the interest of safety and efficient traffic operations. The color convention used for this type of signs is that the legend will be black in color with a white background. The shape used is upward triangular or diamond shape with red borders.

(Traffic engineering roger)



Figure 8-4: Example of Warning Signs

Guide signs

Guide signs provide information to road users concerning destinations, available services, and historical recreational facilities. They serve a unique purpose in that drivers who are familiar or regular users of a route will generally not need to use them; they provide critical information, however, to unfamiliar road users. They serve a vital safety function: a confused driver approaching a junction or other decision point is a distinct hazard. They are written black letters on yellow background. Destination signs are used to indicate the direction to the critical destination points, and to mark important intersections. Distance in kilometers is sometimes marked to the right side of the destination. They are, in general, rectangular with the long dimension in the horizontal direction. They are color coded as white letters with green background. (Traffic engineering roger)



Figure 8-5: Example of Guide Signs

Road markings

The essential purpose of road markings is to guide and control traffic on a highway. They supplement the function of traffic signs. The markings serve as a psychological barrier and signify the delineation of traffic path and its lateral clearance from traffic hazards for the safe movement of traffic. Hence, they are very important to ensure the safe, smooth and harmonious flow of traffic. They serve a variety of purposes and functions and fall into three broad categories.

- Longitudinal markings
- Transverse markings
- Object markers and delineators

Longitudinal markings

Longitudinal markings are those markings placed parallel to the direction of travel. It provided for separating traffic flow in the same direction and the predominant color used is white. The vast majority of longitudinal markings involve **centerlines**, **lane lines**, **edge lines**, **No passing zones**, **Warning lines**,

Centerlines

The centerline marking is critically important and is used to separate traffic traveling in opposite directions. Centerline markings shall be placed on all paved urban arterials and collectors that have a traveled way of 20 ft or more and an ADT of 6,000 veh/day or greater. Centerline markings shall also be placed on all paved, two-way streets or highways that have 3 or more traffic lanes(Roger). Since the ADT of our road is less than 6000 veh/day no centerline markings will be required. On two-lane, two-way rural highways, centerline markings supplemented by signs are used to regulate passing maneuvers. A double-solid yellow center marking indicates that passing is not permitted in either direction.



Figure 8-6:Example of Center Line

Lane lines

The typical lane line marking is a single white dashed line separating lanes of traffic in the same direction. A single solid white lane line is used to indicate that lane-changing is discouraged but not illegal. Where lane-changing is to be prohibited, a double-white solid lane line is used.(Roger)



Figure 8-7:Example of Four Lane Line

Edge lines

Edge lines indicate edges of rural roads which have no curbs to delineate the limits up to which the driver can safely venture. They should be at least 150 mm from the actual edge of the pavement. They are painted in yellow or white. All the lines should be preferably light reflective, so that they will be visible during night also. Improved night visibility may also be obtained by the use of minute glass beads embedded in the pavement marking materials to produce a retro reflective surface. (Edge)



Figure 8-8: Example of Edge Lines

No passing zones

No passing zones are established on summit curves, horizontal curves, and on two lane and three lane highways where overtaking maneuvers are prohibited because of low sight distance. It may be marked by a solid yellow line along the center or a double yellow line. In the case of a double yellow line, the left-hand element may be a solid barrier line, the right hand may be either a broken line or a solid line. These solid lines are also called barrier lines. When a solid line is to the right of the broken line, the passing restriction shall apply only to the opposing traffic. (KRI0

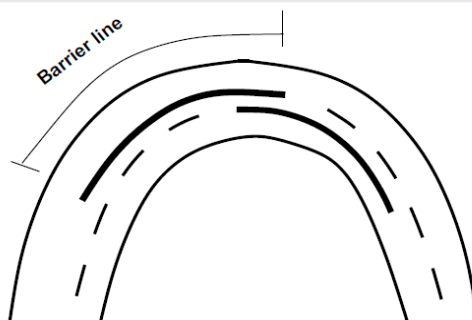


Figure 8-9: Example of No Passing Zone in Horizontal Curves

Warning lines

Warning lines warn the drivers about the obstruction approaches. They are marked on horizontal and vertical curves where the visibility is greater than prohibitory criteria specified for no overtaking zones. They are broken lines with 6 m length and 3 m gap. A minimum of seven line segments should be provided.

All the lines should be preferably light reflective, so that they will be visible during night also. Improved night visibility may also be obtained by the use of minute glass beads embedded in the pavement marking materials to produce a retro reflective surface. (roger)

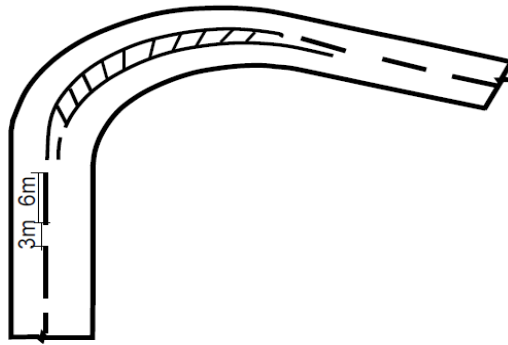


Figure 8-10: Example of Warning Lines

Transverse markings

Transverse markings are marked across the direction of traffic. They are marked at intersections etc. The site conditions play a very important role. The type of road marking for a particular intersection depends on several variables such as speed characteristics of traffic, availability of space etc. **Stop line** markings, markings for **pedestrian** crossing, **direction** arrows. All transverse markings are white

Stop line
Stop line indicates the position beyond which the vehicles should not proceed when required to stop by control devices like signals or by traffic police. They should be placed either parallel to the intersecting roadway or at right angles to the direction of approaching vehicles. Source (Tom V. Mathew and K V Krishna Rao).

Stop lines should be located

- So as to minimize inter green times and clearance times;
- Not less than 3m from conflicting vehicle movements
- Clear of the swept path of vehicles turning from other approaches;
- A minimum of 1.0m from parallel pedestrian crossing at intersections;
- A minimum of 3m from signalized mid- block pedestrian crossings; (AACRA manual section 18 traffic signals)

Pedestrian crossings

Pedestrian crossings are provided at places where the conflict between vehicular and pedestrian traffic is severe. The site should be selected that there is less inconvenience to the pedestrians and also the vehicles are not interrupted too much. At intersections, the pedestrian crossings should be preceded by a stop line at a distance of 2 to 3m for un signalized intersections and at a distance of one meter for signalized intersections. Most commonly used pattern for pedestrian crossing is Zebra crossing consisting of equally spaced white strips of 500mm wide and crosswalk width should be at least 2m. This should be increased when there are heavy pedestrian volumes. Source (Tom V. Mathew and K V Krishna Rao).

Pedestrian crossing should be located;

- As near as possible to the desire lines of pedestrians;
- As near as possible to and no greater than 20 degrees from the shortest path across the carriage way to minimize clearance times;
- As close to, but at least 0.6m clear of parallel vehicle movements.



Figure 8-11: Example of Pedestrian

Directional (pavement) arrows

In addition to the warning lines on approaching lanes, directional arrows should be used to guide the drivers in advance over the correct lane to be taken while approaching busy intersections. Because of the low angle at which the markings are viewed by the drivers, the arrows should be elongated in the direction of traffic for adequate visibility. The dimensions of these arrows are also very important.

Pavement arrows should be used; -

- In auxiliary lanes (left or right) to avoid inadvertent use by through vehicles;
- To allow movements that would not otherwise be allowed under traffic regulations;
- To prohibit movements that would otherwise be allowed under traffic regulations. (

Pavement arrows should not be used to reinforce lane usage that is regulated by traffic regulations. If drivers are breaking the law relating to lane usage it may indicate that the existing lane distribution is inappropriate and may need to be changed. (AACRA)

A minimum of three arrows in any lane should be provided.



Figure 8-12: Example of Pavements arrows (AACRA)

Source; - (AACRA manual section 18 traffic signals) and (Tom V. Mathew and K V Krishna Rao).

Object markers and delineators

Object markers:

Object markers are used to denote obstructions either in or adjacent to the traveled way. Physical obstructions in a carriageway like traffic island or obstructions near carriageway like signal posts, pier etc. cause serious hazard to the flow of traffic and should be adequately marked. They may be marked on the objects adjacent to the carriageway.

Objects within the carriageway

The obstructions within the carriageway such as traffic islands, raised medians, etc. may be marked by not less than five alternate black and yellow stripes. The stripes should slope forward at an angle of 45° with respect to the direction of traffic.

Objects adjacent to carriageway

Sometimes objects adjacent to the carriageway may pose some obstructions to the flow of traffic. Objects such as subway piers and abutments, culvert head walls etc. are some examples for such obstructions. They should be marked with alternate black and white stripes at a forward angle of

45_ with respect to the direction of traffic. Poles close to the carriageway should be painted in alternate black and white up to a height of 1.25 m above the road level. Other objects such as guard stones, drums, guard rails etc. where chances of vehicles hitting them are only when vehicle runs o_ the carriageway should be painted in solid white. Krebs of all islands located in the line of traffic flow shall be painted with either alternating black and white stripes of 500 mm wide or cheered black and white stripes of same width.

Word messages

Information to guide, regulates, or warns the road user may also be conveyed by inscription of word message on road surface. Characters for word messages are usually capital letters. The legends should be as brief as possible and shall not consist of more than three words for any message. Word messages require more and important time to read and comprehend than other road markings. Therefore, only few and important ones are usually adopted. Some of the examples of word messages are STOP, SLOW, SCHOOL, RIGHT TUN ONLY etc. source (Tom V. Mathew and K V Krishna Rao) and (traffic engineering roger)

Table 8-2 Description of traffic sign posts for the project

remark	station	Recommended sign	
		right	left
intersection	0+180		
intersection	0+260		
Tvet collage	0+500		
Horizontal curve with sag curve	0+890		
Horizontal curve	1+140		
Horizontal curve	1+400		
church	1+360		
--grade	2+020 for l and 2+300 for right		
pedestrian	2+120		
intersection	2+400		
intersection	2+600		

Table 8-3 Description of Road markings for the project

Station	Road marks
0+280	Pedestrian cross
0+360	Pedestrian cross
0+720	Pedestrian cross
1+460	Pedestrian cross
2+220	Pedestrian cross
2+500	Pedestrian cross

station	Road marks
0+000 up to 0+600	Broken line (lane line)
0+600 up to 0+700	Solid line (no passing zone)
0+700 up to 0+980	Broken line (lane line)
0+980 up to 1+080	Solid line (no passing zone)
1+080 up to 1+240	Broken line (lane line)
1+240 up to 1+510	Solid line (no passing zone)
1+510 up to 1+750	Broken line (lane line)
1+750up to 1+850	Solid line (no passing zone)
1+850up to2+290	Broken line (lane line)
2+290up to2+390	Solid line (no passing zone)
2+390up to2+711	Broken line (lane line)

Design Speed of Sign Post ‘

we recommended post speed of 40km/hr since the project road is residential area and it is urban area(to be safe) that why we decide this speed we decide the beginning of the road and at the end of the road



Figure 8-13 post speed

CHAPTER NINE

QUANTITY SURVEYING

To estimate how much a civil engineering project may cost or to approve contractor's payment, the actual quantities of materials, labor, equipment etc. that is needed for the construction work must be calculated at the beginning of the work.

Such work of calculating the amount of materials and other incidentals necessary for the realization of the work is called ***quantity surveying***

Quantity surveying is a term or processes used in the construction industry to take measurements of civil works, prepare specifications, and estimate the cost of works either for each trade of work or for the whole project

FINANCIAL RESOURCES

Usually funds are available from among Governmental institution, Private institutions and Donors in the form of loan or assistance.

Table 9-1 Bill of Quantities

Bill No.1: EARTHWORKS	Unit	Original Contract		
		Quantity	Unit rate	mount(Birr)
Clearing and grubbing	ha	6.840	62,398.70	426,807.11
Soft excavation to Spoil	m3	13,608.00	76.75	1,044,414.00
Intermediate Excavation to Embankment	m3	1632.757	281.12	459,000.65
Intermediate Excavation to spoil	m3	137532.753	86.87	11,947,470.25
				13,877,692.01

Bill No.2: Pavements				
a. Carriageway				
Description	Unit	Quantity	Unit rate	Amount(Birr)
25 cm thick capping layer under the subbase layer with maximum compacted layer thickness 250mm.	m3	9,450.00	176.59	1,668,775.50
12.5cm thick Gravel Subbase compaction to 96% of AACRA Test S-11 Test-11 Density(compact layer thickness of maximum 150mm)	m3	5,828.65	300.70	1,752,675.06
12.5cm thick crushed stone base compaction to 98% of AACRA Test S-11 density (maximum compacted layer thickness 150mm)	m3	4,725.00	384.38	1,816,195.50
Supply and spray MC-30 Cut back Asphalt Prime Coat application at a Rate of 1lit/m2 between asphalt concrete and base course	lit	37,954.00	150.00	5,693,100.00
Dense Graded Asphalt Concrete(AC) Wearing course compacted thickness of 50mm	m2	1,897.70	350.00	664,195.00
				9,926,165.56

b. Side walk				
Description		Quantity	Unit rate	Amount(Birr)
20 cm thick Gravel subbase compaction to 96% of AACRA Test S-11 Test S-11 Density	m3	2,688.00	300.70	808,281.60
Bill No.3: Minor Structures				
Description	Unit	Quantity	Unit rate	Amount(Birr)
Back fill Using Imported Selected Material	m3	5,422.00	250.00	1,355,500.00
800mm RC pipe culverts	lm	350.00	1,200.00	420,000.00
1000mm RC Pipe culverts	lm	750.00	1,900.00	1,425,000.00
1100mm RC Pipe Culverts	lm	650.00	2,250.00	1,462,500.00
1300mm RC Pipe culverts	lm	800.00	4,500.00	3,600,000.00
2000mm RC pipe culvert for taransvers dreange2sta 1+046	lm	13.00	10,000.00	130,000.00
RC Concrete cover (C-25) to be constructed on the manhole as detailed on drawings.	No	54	2,707.51	146,205.54
Prefabricated concrete curbing for edge side(400x100*1000mm) using C-25 concrete, price shall also include the necessary false work and form works	lm	5,422.00	150.00	813,300.00
Concrete curbing(430x200*mmx1000mm)using C-25 Concrete price shall also include the necessary false work and form works and U2 surface Finish	lm	5,000.00	200.33	1,001,650.00
Cement mortarted	m3	25.00	2,397.09	59,927.25
				10,414,082.79

Bill No. 4: sign post and marking	Unit	Contract		
		Quantity	Unit rate	Amount(Birr)
cost of excavating for erecting, the necessary concreting , the post and all other expenses)	No	16.00	300.00	4,800.00
Direction Sign Boards	No	10.00	700.00	7,000.00
Lane Line (White broken)	lm	2,041.00	61.69	125,909.29
Lane Line (White continuous)	lm	670.00	120.00	80,400.00
Pedestrian crossing	No	6.00	2,500.00	15,000.00
Zebra Crossing around school area	No	2.00	2,500.00	5,000.00
				238,109.29
Bill No.5. Dayworks	Unit	Contract		
Description		Quantity	Unit rate	Amount(Birr)
Labour				
Unskilled labourer	Hour	4,500.00	15.75	70,875.00
Ganger	Hour	2,500.00	21	52,500.00
Timberman	Hour	1,500.00	26.25	39,375.00
Carpenter	Hour	1,500.00	31.5	47,250.00
Mason	Hour	500.00	33.25	16,625.00
Certified blaster	Hour	150.00	33.25	4,987.50
Pipe layer	Hour	250.00	35.00	8,750.00
Watchman	Hour	1,500.00	21.00	31,500.00
Driver for light vehicles	Hour	1,000.00	42	42,000.00
forman	Hour	3,000.00	35	105,000.00
				418,862.50
				418,862.50

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Conclusion

It is shown above the current performance of the project road in BoleBulbulla Condominium collector road in Addis Ababa.

Importance could be seen on the paper

- It has socio-economic benefit for community
- It gives access for the residence
- It provides the pedestrian with standard walk way
- The sharp curve we phased with geometric design was treated with provision of transitional spiral curve.
- The road project is designed in such way that it is safe, economical and comfortable.
- Mis phasing of curve is adjusted by following principle from Accra manual.
- Any water receiving the pavement was perfected by providing adequate drainage system

Recommendation

- It is believed that the design will provide efficient urban solution.
- The pavement design will e durable since the project road is subjected to cars a about more than 90% of vehicles. So that low load effect will be there.

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