UNDERPASS PEDESTRIAN CROSSING CONNECTING THE TWO CAMPUSES OF ADDIS ABABA INSTITUTE OF TECHNOLOGY

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This is to certify that the thesis prepared by Tiruye Beshah, entitled: *Underpass Pedestrian Crossing Connecting the Two Campuses of Addis Ababa Institute of Technology* and submitted in partial fulfillment of the requirements for the Degree of Master of Sciences (Structure) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

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This Thesis is my original work and has not been presented for a degree in any other university, and that all sources of material used for the thesis have been duly acknowledged.

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ABSTRACT

Road traffic accident is becoming the major problem globally. Our country is also facing the problem currently with increasing trend of the magnitude of the problem from time to time. Addis Ababa, the capital of the nation is the major area where traffic accident is occurring in the country.

One of the main reasons for road traffic accident is thought to be due to imbalance between the number of pedestrians, vehicle number and size of the road. These problems are also the main contributors for the road traffic accident in Addis Ababa.

To solve the above problems providing an appropriate path for pedestrians is vital. Constructing pedestrian underpass around Addis Ababa Institute of Technology is one solution for preventing the road traffic accident that happens in this area.

Civil Designer, AchiCAD, Excel and AutoCAD software were used for the design analysis of the pedestrian underpass.

The minimum loading requirements for determining loads and forces was taken from ERA's Bridge Design Manual-2002 and AASHTO Geometric Design Manual 2001.

The final design of foundations was conducted based on the safe bearing capacity of the foundation computed from the geotechnical investigations.

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I also would like to extend my gratitude to my family especially to my baby Naod Atnafu who missed my motherly care while I was busy in doing my master's Degree and this Thesis project.

Finally I would like to thank the Ethiopian Roads Authority (ERA) who has given me the scholarship to study my master's degree. I also would like to appreciate the support I got from the Highway Engineers Consultants (HEC) and from Addis Ababa traffic police office for providing me the necessary information to write up the document.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF PICTURES

LIST OF TABLES

ILLUSTRATIONS

$A_s =$ area of nonprestressed tension reinforcement (mm^2)

A'_s =area of compression reinforcement (mm^2)

$A_{sk} =$ area of skin reinforcement per unit height in one side face (mm^2)

a = depth of equivalent rectangular stress block (mm); the anchor plate width (mm); the lateral dimension of the anchorage device measured parallel to the larger dimension of the cross-section (mm)

b = width of the compression face of the member (mm); the lateral dimension of the anchorage device measured parallel to the smaller dimension of the cross-section (mm)

b_{eff} = lateral dimension of the effective bearing area measured parallel to the smaller dimension of the cross-section (mm)

$b_w =$ web width or diameter of a circular section (mm)

c = distance from the extreme compression fiber to the neutral axis (mm); cohesion factor (MPa); required concrete cover over the reinforcing steel (mm); spacing from centerline of bearing to end of beam (mm)

d = distance from compression face to centroid of tension reinforcement (mm)

D = external diameter of the circular member (mm)

f'_c =Specified compressive strength of concrete for use in design (MPa)

g = Gravitational acceleration (m/s^2)

E = Young's Modulus in (MPa)

E_c = modulus of elasticity of concrete (MPa)

E_s = modulus of elasticity of reinforcing bars (MPa)

EI = flexural stiffness ($N - mm^2$)

$e =$ base of Napierian logarithms; eccentricity of the anchorage device or group of devices with respect to the centroid of the cross-section; always taken as positive (mm); minimum edge distance for anchorage devices as specified by the supplier (mm)

e_m = average eccentricity at midspan (mm)

f_f = permissible fatigue stress range (MPa)

f_{min} = algebraic minimum stress level (MPa)

f_r = modulus of rupture of concrete (MPa)

f_y = specified minimum yield strength of reinforcing bars (MPa)

f'_y =specified minimum yield strength of compression reinforcement (MPa)

h_f = compression flange depth (mm)

I_{cr} = moment of inertia of the cracked section, transformed to concrete **(**m^4)

I_e = effective moment of inertia (mm4)

$I_g =$ moment of inertia of the gross concrete section about the centroidal axis, neglecting the reinforcement (mm^4)

$M_{cr} =$ cracking moment (N-mm)

$M_U =$ factored moment at the section (N-mm)

$n =$ modular ratio $= E_s/E_c$**;** number of anchorages in a row; projection of base plate beyond the wedge hole or wedge plate, as appropriate (mm)

$P_U =$ factored axial force effect or factored tendon force (N); factored tendon load on an individual anchor (N)

$r =$ radius of gyration of gross cross-section (mm)

$r/h =$ ratio of base radius to height of rolled-on transverse deformations

$s =$ spacing of reinforcing bars (mm)

$s_{max} =$ maximum permitted spacing of transverse reinforcement (mm)

$V_c =$ nominal shear resistance provided by tensile stresses in the concrete (N)

$V_n =$ nominal shear resistance of the section considered (N)

$V_s =$ shear resistance provided by shear reinforcement (N)

$V_u =$ factored shear force at section (N)

$\nu_u =$ average factored shear stress on the concrete (MPa)

$y_t =$ distance from the neutral axis to the extreme tension fiber (mm)

$Z =$ crack control parameter

I. INTRODUCTION

Since pedestrians are a part of every roadway environment, their interaction with traffic should be a major consideration in highway planning and design. Pedestrians have also some features which can make them vulnerable to traffic accidents. To mention some of the characteristics of pedestrians which can make them vulnerable to traffic accident include their less predictable action than a motorists, many consider themselves outside the law in traffic matters, have a basic resistance to changes in grade or elevation when crossing roadways (avoid underpass or over pass pedestrian facilities), and they walk in a path representing the shortest distance between two points. Because of the demands of vehicular traffic in congested urban areas, it is often very difficult to make adequate provisions for pedestrians. Yet provisions should be made, because pedestrians are the lifeblood of our urban areas, especially in the downtown and other retail areas. In general, the most successful shopping sections are those that provide the most comfort and pleasure for pedestrians. Underground pedestrian systems are systematic underground pedestrian spaces that have multiple functions for transport, public and commercial usage - such as underground shopping streets, and subway stations with underground concourses. Such underground pedestrian systems are normally located in sub centers and regional centers in metropolises and strongly influence city functions (AASHTO Geometric design manual, 2001).

The public prefers sidewalks and underpasses while overpasses are largely being provided for children en route to school. If underpasses are used, it has to be well lighted, cleaned, and the area has to be safe from thieves or robbers. Bicycle riding in the under pass has to be forbidden since it is hazardous for pedestrians. Overpasses may create nuisances and hazards because of pedestrians who throw objects at cars passing under (NZ, 2007).

Pedestrian traffic accidents frequently occur in urban areas where there are little or no safety features for the pedestrians. The traffic accidents could be two-motor vehicle accidents or single-motor vehicle accidents while accidents that result from vehicles running off the roadway are significant in terms of relative frequency and severity (John E. Baerwald, 1965).

II. STATEMENT OF THE PROBLEM

Road traffic accident is becoming the main problem of Addis Ababa city. Every day a number of pedestrians are dying because of road traffic accident. Also each year the city is losing millions of dollars as a result of the traffic accident.

One of the reasons that contribute a lot for the problem is lack of appropriate pathways for pedestrians even if other factors can also be mentioned like vehicle design, speed of operation, road design, road environment, and driver skill and behavior. Because of the absence of safe path ways for pedestrians in the city, they are forced to cross streets through inappropriate way risking themselves to traffic accident. It is everyone's responsibility to look for a solution to alleviate such a big problem of the city.

According to the information obtained from Addis Ababa Traffic Police Office, in the year 2011/2012 there was 11,529 registered traffic accidents that happened in Addis Ababa city. As a result of these accidents 369 people died, 1190 major injuries and 820 minor injuries occurred and 9150 traffic accidents damaged property. All the above accidents were estimated to cost 52,013,101.00 birr loss only in 2011/2012 year. The street from 'Arat-kilo' roundabout to 'Sidist kilo' roundabout has a length of 1.2km and it is a part of the street from Grand-palace to Shiromeda. On this street where the two campuses of Addis Ababa institute of Technology are separated by it, there were 126 car to car collusion accidents, 11 car-to-materials collusion accidents and 16 pedestrian traffic accidents in the aforementioned year only. This was estimated to worth 493,000.00 birr loss (Addis Ababa Traffic Police report, 2011/2012).

The main obstacle for pedestrians around Addis Ababa Institute of Technology is the main arterial road of Grand Palace – Shiromeda traffic congestion. Addis Ababa Institute of Technology is a governmental institution with over 5,000 students (Addis Ababa Institute of technology Registrar students' profile, 2012). It has dormitories and dining facilities located on the West side of the Grand palace – Shiromeda Street while Classrooms, library and administrative offices are situated on the East side of the Street. Hence students are expected to cross the main street several times a day for different reasons. This will make them to be at risk for the traffic accident in the area than any other body. The frequent interruption of the traffic flow by the institute's students at the crossing road on the Grand-palace Shiromeda Street created capacity problems along the roadway. This street is also a location for numerous rear-end vehicular accidents and pedestrian accidents to the nearby residents and visitors. According to traffic counts done on the street in 2010, there were up to 1,000 pedestrians recorded crossing the Grand palace – Shiromeda Street conflicting with over 1,875 vehicles traveling on this street during a one-hour period. The annual average daily traffic (AADT) on this street in 2010 was 45,000 vehicles a day (HEC traffic study, 2010). This value indicates the traffic problem of the mentioned area. The large volume of pedestrians and the high number of vehicular/ pedestrian conflicting along the mentioned street is of key concern. It is logical to think that the Addis Ababa Institute of Technology community and the local residents would like to see this problem solved by accommodating pedestrian traffic with either an overpass or underpass across the grand-Palace Shiromeda Street.The specific tasks done include, conducting a literature search on potential feasible crossing scenarios, project future volumes that would potentially be using the proposed crossing and to provide a design for it.

III. OBJECTIVE

General Objective:

- To provide feasible solution for pedestrian crossings that connects the two campuses of the Addis Ababa Institute of Technology.

Specific Objectives:

- To make a design to separate vehicle and pedestrian traffic to avoid road traffic accident without compromising student safety.
- To create extra space for constructing rooms in the pedestrian underpass for different business activities.

IV. MATERIAL AND METHODS

The design of the pedestrian underpass was done for the Grand palace – Shiromeda Street in Addis Ababa. The underpass will be located on this street around Addis Ababa Institute of Technology just 2.3km north of the Grand palace.

Picture-4.1: pedestrian under pass location on the Street of Grand-palace Shiromeda to connect the two campuses of Addis Ababa Institute of Technology.

❖ Underpass Design Considerations

Consideration of a below grade crossing came with its own set of considerations, concerns and issues. The fact that this type of structure would convey pedestrian under the traffic on Grand Palace-Shiromeda Street immediately brings to mind issues like 24-hour a day lighting. Proper lighting is imperative in any tunnel or underpass built to improve pedestrian security. Whenever possible, any measure to improve illumination in underpass will be implemented for pedestrians to feel safer using them. In the design of the pedestrian under pass crossing that connects the two campuses of Addis Ababa Institute of Technology, besides overhead lighting natural light coming in through skylight in median opening in the ceiling from the outside was also provided.

Unlike overpasses or at-grade crosswalks, tunnels and underpasses require special considerations for head room clearance. A minimum height of two point five (2.5) meter is typically required for tunnels or underpasses. A height of three (3.0) meter is preferred, particularly for underpasses extending over 18 meter in length. Whenever possible, the minimum clearance requirements ought to be exceeded for a more effective design.

The minimum mandatory width for underpass is 3.5 meter, although a greater width would be desirable. An increased width would provide pedestrians a less threatening, more open environment and serve to reduce conflicts between opposing traffic, especially bicycles. Both entry and exit points should be completely visible from either end thereby eliminating any chance of danger around a bend or just out of sight. Drainage is another consideration for underpass especially when they are below grade. If an underpass is not properly drained then there is the potential for internal flooding in the summer months if water is permitted to pond. If it is not possible for gravity flow to be used then a pump systems may be required. Though most new pump systems are pretty reliable, it is still a good idea to build in redundancy with a stand-by pump (ATCS, 2008).

While doing the design of the pedestrian underpass crossing that connects the two campuses of Addis Ababa Institute of Technology, the following parameters were used based on the above standards. Safety issues in the pedestrian under pass will be addressed with a security system linked to campus police; drainage problems will be solved with a pump and underground storage system, and constructability issues will be solved by a structural design approach aimed at minimizing impacts. The underpass approach ramps were designed to meet AASHTO standard with specific grade, landing area, and cross slope requirements. The approach ramps and necessary retaining walls were designed to minimize visual impacts to the setting and are aesthetically compatible with the historic context of the campus. The underpass width was proposed to be 20m while its height was 3m based upon the future pedestrian usage and length of 55 meter. Construction material such as concrete, partition walls, paver walkways, cast stone elements, decorative balustrade walls, decorative fencing, ceramic tile, and bronze hand railings will be used.

The following data were collected and utilized in doing the design proposal of the pedestrian underpass; traffic data, surveying data, traffic accident that happened in the area in the year 2011/2012 and soil extension of the area. The traffic data utilized was the one collected by the Highway Engineers and Consultants (HEC) P.L.C in 2010 for the purpose of Grand-Palace to Shiromeda Street design upgrading. The surveying data was obtained partially from the HEC P.L.C and the rest from Addis Ababa road net work contour. The traffic accident(on the street from Arat-Killo round about to Sidist-Killo round about) data was extracted from the Addis Ababa Traffic police office daily traffic accident record of the city after a formal letter was written for the necessary cooperation to the traffic office from the Civil Engineering department. The data about the soil extension of the area was obtained from HEC P.L.C and was used for the purpose of estimating the baring capacity of the soil for the sub structure design.

Civil Designer software, AutoCAD soft ware and Excel program were used in doing the design where as ArchiCAD software was used in doing the three dimensional view.

V. RESULTS

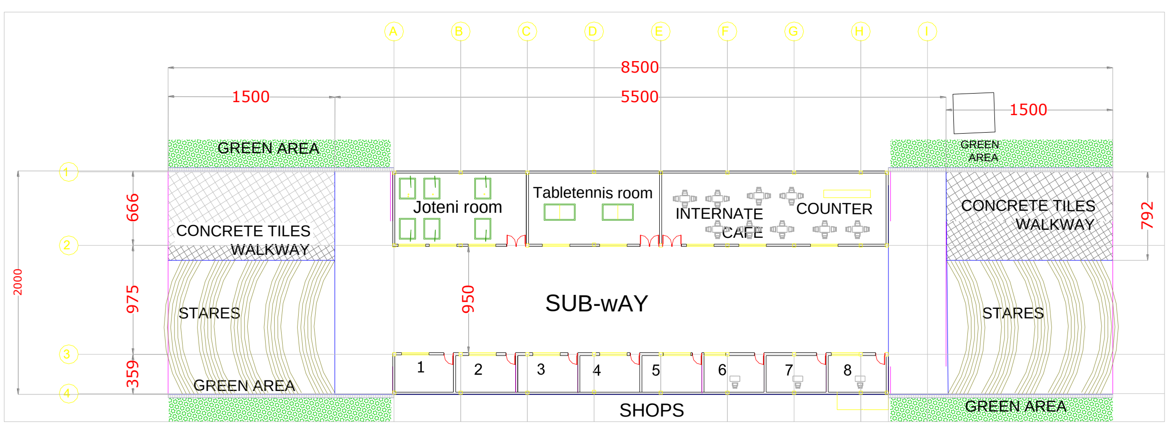
To materialize the pedestrian underpass crossing that connects the two campuses of Addis Ababa Institute of Technology, the Architectural, the Structural and the financial Cost break down was done and included in the following paragraphs.

a. The Architectural design

The Architectural design of the pedestrian under pass crossing illustrated in the following figure was done after getting an opinion from an architect to keep its esthetic value and its standard.

The sub-way was designed in such a way as to have entertainment rooms such as pool house, table tennis room and internet cafe, shops and other rooms for different purposes. The following drawing shows, the detail partitions and their dimension in the sub-way.

Fig.5.1 Architectural design of the pedestrian under pass crossing that connects the two campuses of Addis Ababa Institute of Technology



.NB: All dimrnsions are in cm.

b. Structure design proposal

For any bridge, selection of type of structure to be adopted requires careful examinations of all the factors governing- economy, safety, durability, time of erection and maintenance cost, etc. Therefore, taking all the above conditions into consideration box girder types of superstructures were used for this project.

Design criteria and loading

The following criteria and standards have been used in the design of the bridge over the pedestrian crossing (Addis Ababa city Road net work plan, 20O1):

Bridge carriageway width (four lane) = 14m

Side walk and curb width = 5.0m

Median Width = 2.0m

Green Area (Only on the right side) = 13.0m

Clear Span =20.0m

Clear height of first abutment = 3.0m

Clear height of second abutment =3. 0m

Total width (carriage and walk way) of bridge = 55.0m

Loading

HL-93 live load was used combined with the dead load of bridge components as per the recommended load combination of Ethiopian Road Authority (ERA) bridge design manual. HL-93 loading is a combination of truck and lane load or tandem axle and lane load and the design was performed taking the maximum effect of these loads combined with that of the dead load multiplied by the corresponding load factors.

HL-93 live load is a combination of Design Truck load or Design Tandem load and Design Lane load. Design truck load was applied as per article 3.8.3 of ERA Bridge

Design Manual. Accordingly, three axles of the design truck the first of which is 35KN and the 2^{nd} and the rear axles of 145KN were applied at the specified axle spacing. A total of 325KN load was used as design truckload on superstructures of bridges. In addition to the truck load a design lane load of 9.3KN/m uniformly distributed in the longitudinal direction was applied on each traffic lane of bridges. It was the combined effect of Truck and Lane load or Tandem axle and lane load whichever was greater that was taken for design of bridge components. Tandem axle loads consist of a pair of 110 KN axles spaced at 1.2m intervals. The total load of tandem axle loads is far more less than that of design truckload (ERA, 2003).

Substructure design

Types of Substructures

The height of substructures of the bridge is within applicable limit for masonry structures. Masonry structures are considered to be economical and feasible for heights up to 9 to 10m. Therefore the substructures of this bridge are designed to be composed of masonry abutments and wing walls. It is optional to use plain concrete leveling course under all substructures, which shall be decided after excavation of foundation materials. When required, reinforced concrete (R.C) footings will be designed for substructures of bridge. The abutments and wing walls were designed to be of masonry, with R.C seat and back wall at its top. The purpose of these concrete parts is to prevent any crack due to earth pressure and temperature changes and to provide good working level. The wing wall also has R.C parapet coping at its top. The masonry walls were designed as gravity retaining walls with factor of safety 2.0 for overturning and 1.5 for sliding according to ERA Bridge Design Manual. The back fill material behind the walls shall be free draining, non-expansive and non-corrosive. The allowable stresses were checked for the following load combinations (ERA BDM, 2003):

Live load, ERA HL-93 loading

Dead loads, including 10cm wearing surface

Wind loads, according to ERA BDM

Impact, according to ERA BDM

Longitudinal forces, according to ERA BDM

Loading Substructure components are designed for maximum reactions of dead and live loads of superstructures, their own dead loads and earth pressure loads. The masonry abutments and wing walls are designed as gravity retaining structures under the action of the above indicated load cases

Material Properties

A) Reinforcement steel

The steel industries available in the country produce grade 60-reinforcement steel for diameter of bar equal to and greater than 20mm, and grade 40 steel for those less than 20mm diameter. The minimum yield strength of grade 60 reinforcement steel is 400Mpa, while that of grade 40 is 276Mpa. These and other strength parameters are used in the design of the superstructures of the bridges. Minimum clear cover of reinforcing bars are recommended and shown on drawings as ERA Bridge design Manual. (Table 9-5)

Water/Cement Ratio	≤0,40	≤0,45	≤0,50
SITUATION	COVER (mm)	COVER (mm)	COVER (mm)
Direct exposure to salt water	80	100	120

Cast against earth (i.e. Bottom of footings)	60	75	90
Exterior other than above	40	50	60
Interior other than above (i.e. hollow structures) • Up to ∅35 Bar • ∅45 and ∅55Bars	 32 40	 40 50	 48 60
Bottom of cast-in-place slabs • Up to ∅35 Bar • ∅45 and ∅55Bars	 25 40	 25 50	 30 60
Precast soffit form panels	20	20	24
Precast Reinforced Piles • Non-corrosive environments • Corrosive environments	 32 60	 40 75	 48 90
Precast Prestressed Piles	40	50	60
Cast-in-place Piles • Non-corrosive environments • Corrosive environments - General - Protected • Shells • Auger cast, tremie concrete or slurry construction	 40 60 60 40 60	 50 75 75 50 75	 60 90 90 60 90

Table 4-1 Cover for Unprotected Main Reinforcing Steel (mm)

B) Concrete

Design parameters of C-30 concrete were used in the structural computations of superstructure of the bridge. These strength parameters were specified on design drawings and technical specifications to be attained during construction stage. Resistance factors are recommended on the design manuals to account the imperfection in production of these construction materials. Accordingly the appropriate resistance factors for shear and bending moment of structural components were taken during the design of these components.

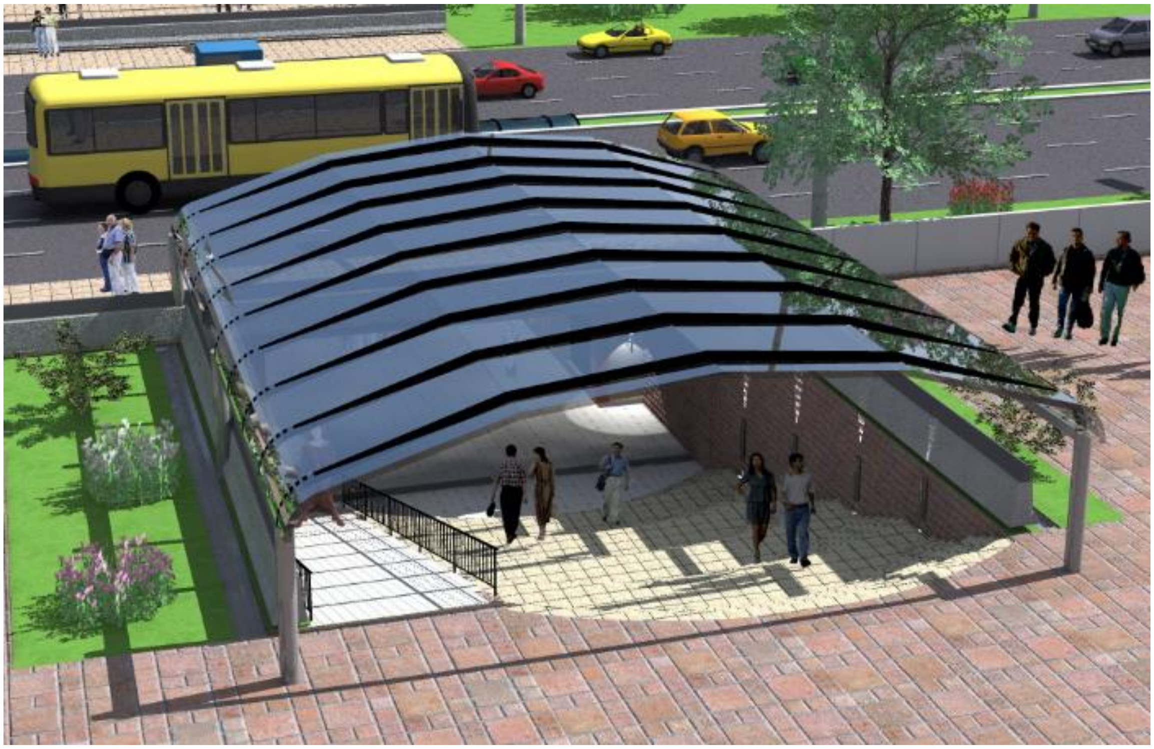
C) Stone Masonry

The unit weight of stone masonry used in the design of substructure components was taken from ERA bridge design manual. (Table 3-4)

MATERIAL		DENSITY (kg/m^3)	Force effect (kN/m^3)
Bituminous Wearing Surfaces		2250	22.5
Cast Iron		7200	72
Cinder (volcanic stone) Filling		960	9.6
Compacted Sand, silt, or Clay		1925	19.3
Concrete	Normal	2400	24
Loose Sand, Silt, or Gravel		1800	18
Soft Clay		1700	17
Rolled Gravel or Ballast		2250	22.5
Steel		7850	79
Stone Masonry		2725	27.3
Wood	Hard	960	9.6
	Soft	800	8
Water	Fresh	1000	10

Table 4-2 Densities and Force Effects of Different Materials

Picture 5-1: top view of the pedestrian underpass connecting the two campuses of Addis Ababa institute of Technology.



Picture 5-2: entrance/exit side view of the pedestrian underpass connecting the two campuses of Addis Ababa Institute of Technology.

Picture 5-3: Interior view of the pedestrian underpass connecting the two campuses of Addis Ababa Institute of Technology.

c. Pedestrian Underpass Crossing Cost Breakdown

The cost break down of the pedestrian underpass crossing is done and the detail cost specification is illustrated in the following table. The total cost of the pedestrian underpass crossing is estimated to be 16,429,459.08 birr. Road way Excavation to Embankment is estimated to cost 2,209,250.72 birr while the cost of Sub-base and base Coarse is 506,275.00 birr. The cost of Priming, Primer Sealing and Sealing and Structure will be 1,247,400.00 and 12,466,533.35 birr respectively.

Specification Ref. No.	Description	Unit	Quantity	Rate	Amount
204	Road way Excavation to Embankment				
204P2(1)	Soft Excavation to Embankment	m^3	3974.513	125	496,814.18
204P3	Cut to spoil				
204P3(1)	Soft Excavation	m^3	5542.55	70	387,978.52
204P3(2)	Intermediate Excavation	m^3	3965.443	175	693,952.56
204P3(3)	Hard Excavation	m^3	2379.266	265	630,505.47
301	Sub-base Coarse				
301P1(ii)	Unstabilized gravel sub-base compacted to 96% of AASHTO Test S-11 density, 200mm layer thickness crushed stone	m^3	962.5	230	221,375.00
302	Base Coarse				

302P1(2)(ii)	Crushed stone base compacted to 100% of AASHTO Test S-11 density, 200mm layer thickness	m^3	770	370	284,900.00
401	Priming, Primer Sealing and Sealing				
401P1	Supply and Spray Primer, Primer-binder coat				
401(1)	MC-30 cut-back bitumen applied at 1.0 liters per sq.m.	lit.	3850	37	142,450.00
403	Hot Laid Asphalt Concrete Surfacing				
403P1	Supply and Application of Tack Coat at 0.3 lit/ square meter of residual bitumen	lit.	1155	40	46,200.00
403P3	Dense graded Asphalt AC in Intermediate (Binder) Course, 60mm Nominated Compacted Layer Thickness, 20mm Nominal aggregate Size with Class 85/100 pen. Bitumen	m^2	3850	150	577,500.00
403P4	Dense graded Asphalt AC in Wearing Course, 40mm Nominated Compacted Layer Thickness, 14mm Nominal aggregate Size with Class 85/100pen. Bitumen	m^2	3850	125	481,250.00
601	Structure				

601 P1	Structural Excavation	m^3	841.051	150	126,157.68
601 P2.2	Backfilling to excavation (Selected material)	m^3	148	235	34,780.00
603 P1	Cast in situ concrete	m^3	970.32	4265	4,138,414.80
604 P1.5	Steel reinforcement for structure(40)	ton	79.40239	31895	2,532,539.19
	Steel reinforcement for structure(60)	ton	81.4643	34725	2,828,847.72
606 P16	Concrete bridge railing	Lm	65	2425	157,625.00
606 P9	Elastomeric Bearings	No.	50	10000	500,000.00
701 P2.2	Class 'B' Stone masonry	m^3	1370.88	1567	2,148,168.96
Total	**Sum**	**Birr**			**16,429,459.08**

Table 5-1: Pedestrian Underpass Crossing Cost Breakdown

VI. DISCUSSION

In doing the design proposal of pedestrian underpass crossing that connects the two campuses of Addis Ababa Institute of Technology, what ignited my mind to forward a solution is the existing traffic problem in the area and the problem that students are facing due to the presence of a highway street between the compounds of the institute which I am witnessing in the area on daily basis. Different alternatives have been looked at to forward the best solution for the problem. Some of the reasons considered while forwarding the alternative solutions for the problem include the location of the area i.e. being a down town, the fact that students of the institute could be the major victims, and being able to provide a lasting solution for the problem.

Each alternative design for a pedestrian crossing, regardless of whether it is constructed at-grade, above grade or below grade, will have its own distinct advantages and disadvantages.

At-Grade Crossing

At-grade street crossings are the most common type of pedestrian crossings due to their relatively low-cost, design simplicity and typical location at the intersection of two streets. These crossings can be divided into two basic groups; those located at intersections or the ones at mid-block locations. Mid-block crossings tend to be less safe than crosswalks located at intersections. This is particularly true because of the simple facts that, at intersections, vehicles are usually going slower when a certain direction is given the "WALK" sign and we have become used to seeing pedestrian crossing at intersections. Conversely, mid-block crosswalk can be unexpected "inconvenience" and thereby driver reactions tend to vary more than at intersections.

Advantages of at grade crossing:

- Typically will have a substantially lower construction cost than either above or below grade facilities
- Construction time is much less than other crossing types. This will mean less impact on existing traffic, both pedestrian and vehicular
- Minimal or no impact on existing utility lines and drainage systems
- Easier to make connections to other transportation such as taxi and bus stations

Disadvantages of at grade crossing:

- Disrupts the flow of both pedestrian and vehicular traffic by having to wait for traffic signal changes
- Potential conflict will exist between pedestrians and vehicular traffic making it less safe than a grade separated crossing.

Grade Separated Crossings

Grade separation is achieved by redirecting one mode of travel either above or below the other thereby providing each with continuous channels for traffic flow. (ATCS, 2008)

Pedestrian bridges and underpasses are primarily provided to assist pedestrians crossing busy roads in relative safety; they can also reduce vehicle delay, increase highway capacity, and reduce vehicle crashes when appropriately located and designed. However, they are expensive and have practical limitations. For instance, pedestrians have to travel considerably further to use these facilities than they would otherwise do by crossing roads at grade. For many pedestrians, the risk of being hit by a vehicle while crossing roads is not perceived to be high whereas travelling the extra distance to use a facility is seen as an unnecessary waste of time and energy. Apart from extra travel distance and time taken to use bridges and underpasses some pedestrians, particularly the disabled, find that ramp slopes are too steep to negotiate.

When possible, working with existing topography to design an overpass or underpass will produce the most efficient structure. In addition to utilizing naturally-occurring grade changes, this method will also minimize overall costs and land-use impacts to the surrounding environment. Convenience is paramount to an effective grade separated crossing. Pedestrians will not use a poorly located crossing which takes them out of their way, or does not deliver them to a point they wish to travel. The same holds true if the crossing is perceived to be very lengthy and adds much distance to the route of travel. An overpass that is inconvenient will also be underutilized. Pedestrians may prefer a more time saving and direct option, potentially leading to hazardous conditions or creating impacts on the land.

Above Grade Crossings

Pedestrian Overpass

Pedestrian overpasses are structures that provide pedestrians, including bicyclists, a route over an obstruction such as a road. They allow for the continuous and uninterrupted flow of traffic on both the road and the path. By their nature they are typically supported on columns or other structures that elevate them over the obstacle.

Overpasses, particularly those crossing over high-volume roads require large vertical clearance to accommodate the various types of vehicles that may pass underneath. These structures have to be of sufficient width to accommodate two-way traffic which can include walkers, bicyclists, wheel chairs and other wheeled but typically not motorized vehicles. The width of the overpass and the associated ramps need to be essentially the same.

Advantages of pedestrian overpass:

- Pedestrian and vehicular traffic are channelized separately allowing traffic to move without interruption
- Because conflicts between pedestrians and vehicles are eliminated, the potential for accidents is reduced

Disadvantages of pedestrian overpass:

- An elevated, above grade structure is very expensive relative to an at-grade crossing
- Height can be obtrusive into the existing urban landscape
- Ramps connecting to the bridge take up a lot of space
- Providing direct access to Taxi and Bus stations could be difficult or impossible and could require obtaining right-of-way
- Access to some businesses could be blocked or difficult to reach
- Pedestrians may not be willing to walk the extra distance (200+ meter) to cross the bridge

Below Grade Crossings

Underpass/Tunnel

Underground pedestrian systems are systematic underground pedestrian spaces that have multiple functions for transport, public and commercial usage - such as underground shopping streets, and subway stations with underground concourses. Such underground pedestrian systems are normally located in sub centers and regional centers in metropolises and strongly influence city functions. Adopting a systems approach to analyzing urban pedestrian space helps to conceptualize the relationship between underground pedestrian systems and other pedestrian spaces within the broader urban environment. Firstly, viewing the city as a complex system, an underground pedestrian system is a vital subsystem of public space systems, expanding space and movement below the surface pedestrian systems. Secondly, an underground pedestrian system uses walking as a mode of transport to link and aggregate activities such as retailing and mass transit within an underground setting. Thirdly, an underground pedestrian system has an important functional feature through the provision of underground public passageways functioning as a medium that integrates ground level spaces with underground spaces.

Pedestrians prefer to travel on level grade with minor undulations and very few or no major changes in elevation. A below grade crossing is an ideal way to accommodate pedestrian travel especially when the feature to be crossed is an elevated road. This can result in a below grade crossing that is actually at the same grade as the path but under the grade of the intersecting road.

Unfortunately, the natural ground around Addis Ababa Institute of Technology is not elevated high enough for underpasses at the same graded as the existing paths along

Grand-Palace to Shoromeda Street. This means that in order to have below grade crossing (underpass) either the underpass has to be lower than the path grade or the elevation of the road has to be raised. But the existing buildings located from either side of the street near Addis Ababa Institute of Technology prohibit raising the height of street. So the only alternative left is to dig the ground below the path grade.

Advantages and Disadvantages

Advantages of underpass:

- Pedestrian and vehicular traffic are channelized separately allowing traffic to move freely, with less interruption.
- Because conflicts between pedestrians and vehicles are eliminated, the potential for accidents is reduced.

Disadvantages of underpass:

- Very costly – involves bridge construction
- Potential safety concerns to users
- Drainage handling issues and possible need for pump system
- Potential "hang-out" area

Recommended pedestrian crossing in the specific area

Pedestrian under pass crossing is recommended to connect the two campuses of Addis Ababa Institute of Technology based on the following facts;

- For the specific location at the Addis Ababa Institute of technology, the students and the Institute's community will be safe from road traffic accident and the pedestrian under pass will provide them the shortest distance to cross from one compound to the other as compared to the above two alternatives pedestrian crossings mentioned.

- If pedestrian under pass is constructed, students will be safe to move from one compound of the Institute to the other any time of the day by providing adequate illumination and by assigning guard on both the exit and entry point of the under pass.
- The pedestrian under pass will make the two campusses of the Institute to be considered as one compound since it will be possible to go from one compound to the other without external barrier. This will allow students to use the necessary services in the two compounds any time of the day especially at night time.
- Space areas that will be created in the pedestrian under pass will be used for different services including internet café, secretarial services and recreational services.
- Constructing pedestrian under pass crossing to connect the two campuses of Addis Ababa Institute of Technology is not without problems. These problems can be overcome by making the environment esthetic, luxurious, providing adequate illumination along the underpass and by assigning a guard on both the exit and entry points of the underpass. Constructing pedestrian underpass is expensive as compared to doing at grade pedestrian pass. But this is not comparable with the loss of human life and property as a result of the traffic accident while it is easily being prevented by constructing pedestrian under pass.

VII. CONCLUSION

In conclusion, constructing pedestrian under pass crossing that connects the two campuses of Addis Ababa Institute of Technology is recommended to bring an alternative solution for the traffic problem on the mentioned street and to provide a safe way for the students and staffs of the institute.

Hence the design of the pedestrian under pass crossing with the bridge structure over it on the Grand-palace to Shiromeda Street is done based on this logical ground.

VIII. SUGGESTION FOR FUTURE WORK

The original intent of this project is to eliminate the at-grade pedestrian crossing; thus improving safety. We have great expectation that the design of this pedestrian underpass will provide benefit if it is going to be implemented. The beneficiaries are the general public and the students of Addis Ababa university especially students of the Addis Ababa Institute of Technology. The pedestrian underpass bridge will provide students life safety from the traffic accident. This will avoid the financial cost being spent for life and property insurance for the traffic accident. Over all, if this pedestrian underpass bridge is going to be constructed in the area, we believe that the city will benefit from it since it can reduce one of the major problems of the city that is traffic accident.

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X. ANNEXES

Structural Design of Reinforced Concrete Bridge

1. Design Data

1.1 General

Case -Statical calculation

Span (m):- $= 20$

Number of Girder $= 9$

C/C Support $= 20.6$

Clear Width $= 14$

Number of Lanes $= 4$ Lane plus Walkway 5m

Face of railing to End of span $= 0.575$

Multiple presences Factor

For one lane loaded $= 1.2$

For two lane loaded $= 1$

For three lane loaded $= 0.85$

For >three lane loaded $= 0.65$

Specification Used -ERA Bridge Design Manual 2011/AASHTO LRFD Bridge Design Spec.2007 Rear Axle Load $= 145\text{KN}$

$\text{P} = 72.5\text{KN}$

Lane Load in the Longitudinal Direction $= 9.3\text{KN/m}^2$

Design Tandem Axle Load $= 110\text{KN}$

Wearing Surface $= 2.25\text{KN/m}^2$

Highway Railing Load, $\text{Ph} = 44.48\text{KN}$

1.2 Material Property Used:

1.2.1 Concrete:

Type of Concrete: $\mathrm{C}-30$

$$f'_c = 24\text{Mpa}$$

$$f_c = 9.6\text{Mpa}$$

Density of Concrete $\gamma_c = 24\text{KN/m}^3$

$E_c = 0.043 \times \gamma_c \; \overline{f'_c} = 24768\text{Mpa}$

1.2.2 Reinforcement:

For diameter $\geq 20\text{mm}$, $f_{yk} = 400\text{Mpa}$

$$f_s = 160\text{Mpa}$$

For diameter $< 20mm$, $f_{yk} = 300\text{Mpa}$

$$f_s = 138\text{Mpa}$$

$E_s = 200000\text{Mpa}$

Modular Ratio, $n = \frac{E_s}{E_c} = 8.075$

Take $n = 8$ Use the Nearest Integer

1.3 Load and Resistance Factors (LRFD) Design Methodology:

$f_{\text{moment}} = \varphi = 0.9$

$f_{\text{shear}} = \varphi = 0.9$

$b = 0.85$

Dead Load limit state factor =1.25

Live Load Limit state factor = 1.75

Dead Load Service Limit State factor = 1

Live load service limit state factor = 1.3

Fatigue Live Load Factor = 0.75

Dynamic impact allowance

Fatigue abd Fracture = 15%

All other Limit state = 33%

Modulus of rupture $= f_r = 0.63\ \overline{f'_c} = 3.086\text{Mpa}$ Eq.9.50

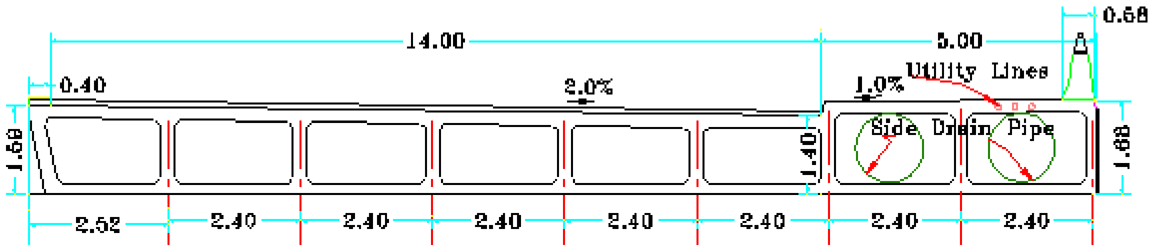
Z in Eq. 9.5 = 30000KPa

For diameter $\geq 20\text{mm}$, $r_b = 0.85 b_1 \frac{f'_c}{f_{y^*}}\ 599.843\ 599.843 + f_y\ = 0.026$

For diameter $< 20mm$, $r_b = 0.85 b_1 \frac{f'_c}{f_{y^*}}\ 599.843\ 599.843 + f_y\ = 0.039$

$$0.75 \times r_b = 0.029$$

2. Preliminary Dimension



TYPICAL CROSS-SECTION

Clear Road Way Width = 14m

Face of Curb to end slab = 5m

Face of Railing to end slab = 0.575m

Total Top Width = 20.15m

Span (C/C of Bearings) = S = 20.6m

Girder Depth=0.065S = 1.236, Take D = 1.4m (Recommended minimum Art.5.11)

C/C of Girder Spacing (a) = 2.4m Perpendicular

Overhang Distance (c) = 0 Perpendicular

Cantilever length = 0 Perpendicular

Girder web thickness = 0.25

Clear Span between Girders = 2.4 Perpendicular

Top Slab Thickness $t = a\ 12$ to $a\ 15 = 0.2\text{m}$ to 0.16m

Take $t = 0.22\text{m}$

Fillets $= 0.1 \times 0.1 = 0.01$ Between Girders and Deck

No. of box cells = 8

Bottom slab thickness = (clear span between girders)/16 = 0.134

Take 0.2m

Overhang end slab thickness = 0.22m

Overhang slab thickness near Ext. girder = 0m

Fillet = 0.1m

Exterior diaphragm depth = 1.4m

Width of diaphragm =0.25m

Total number of diaphragm=2

3. Design of Slab

3.1 Design of Deck Slab

3.1.1 Loadings

a) Dead Loads

Dead loads computation

Slab 22cm thick $= t \times 24 = 5.28\,\text{KN} \quad m^2$

Asphalt 10cm thickness $= 0.10 \times 22.5 = 2.25\,\text{KN} \quad m^2$

$$W_{DL} = 7.53\text{K}\,\text{KN} \quad m^2$$

$$M_{DL} = \frac{W_{DL} \times S_2 \; \times 0.80}{8}$$

Where 0.80 is a continuity factor

Span length S = Clear span = 2.4m (Art. 3.24.1.2)

$$M_{DL} = \frac{1}{8 \;\; W_{DL} \times S_2 \times 0.80} = 4.337\,\text{KNm} \quad m$$

b) Live Loads

Live Load moment for continuous slab (Art. 3.24.3.1)

$$M_{LL} = \frac{1}{32 \;\; S + 2 \;\; P20} \times 0.80\,, \text{where } S = \text{ span length in feet} \;\; \text{Art. } 3.24.1.2 \;\; ,$$

$$S = 2.4\text{m}$$

$\text{MLL} = 4023.53\,\text{lbft} \quad \text{ft}$ $\qquad\qquad P = 72.5\text{KN}$

$\text{MLL} = 17.897\text{KNm/m}$

Impact Factor (Art. 3.8.2.1)

$$\text{IM} = 0.33$$

Live load plus impact

$M_{LL} + IM = 23.803KNm/m$

c) Factored Design moment

Total Design moment

$$M_{Total} = 1.25 \times M_{DL} + 1.75 \times M_{LL} + IM = 47.076KNm/m$$

3.1.2 Reinforcement

$Mu = 47.076KNm/m$ $\quad$ $\text{Ø} = 0.9$

Assume a = 19.29mm b = 1000mm

$f_y = 300N/mm^2$ $\quad$ f'_c =24N/mm^2

$D = 220mm$ $\quad$ $diameter = 16mm$

$cover = 50mm$ $\quad$ $d = 162mm$

$$A_s = \frac{Mu}{\text{Ø}\, f_y \left(d - \frac{a}{2}\right)} = 1144.405mm^2 / m$$

$$a = \frac{As \times fy}{0.85 \times fc'b} = 16.829mm$$

$$\text{Required } A_s = 1144.405mm^2/m$$

$$\text{Spacing, s} = b \times \pi \times diameter^2 \times 0.25/A_s = 175.691mm$$

Use diameter 16 mm bars @170mm (Top and bottom reinforcement.-transverse)

A_s provided =1182.717mm^2 $\quad$ BAR S_1 & S_2

Distribution Reinforcements (Art. 3.24.10.2)

For main reinforcement perpendicular to traffic, the distribution reinforcement is given as percentage of the main slab reinforcement as given below:

A_s(distribution) % =220/ $\overline{S}$ $\leq$ 67%

Therefore, A_s(distribution) % =78.402% (S is in feet) where S =2.4m

A_s(distribution) % =0.67

A_s distribution =0.67× A_s provided =792.421mm^2/m

Spacing =253.731mm F of bar=16mm

Use diameter 16mm bars c/c 250mm (bottom reinforcement - longitudinal)

A_s provided = 804.248mm^2 BAR S_4

Temperature and shrinkage reinforcements (ERA Art. 9.4)

A_s (temp and shrink.) is not less than, $A_s \geq 0.75\ A_g/f_y$ whereA_g= gross area of section (mm^2)

As (temp and shrink .) = 550mm2/m

Spacing =365.567mmBar Diameter =16mm

Max spacing =3.0 ×thickness =660mm

450mm =450mm

Use diam. 16 c/c 300mm (top. Longitudinal Reinforcement) BAR S_4

Bottom Slab Reinforcement (ERA Bridge Design Manual 2002,Chapter 7)

Bottom Slab Reinforcement in Cast-in-place Box girders shall be a uniformly distributed reinforcement of 0.4%of flange area be placed in a bottom slab parallel to the girder span

= $800\text{mm}^2/\text{m}$

0.5%of cross-sectional area of slab is placed in a bottom slab transverse to the girder span

= $1000\text{mm}^2/\text{m}$

Maximum Spacing allowed = 450mm

Spacing of diameter 16 mm bar for reinforcement parallel to span = 251.327mm Ok!

Spacing of diameter 16 mm bar for reinforcement transverse to girder span = 201.062mm

Ok!

Therefore Use dia. 16 mm bar with spacing = 250mm

For bottom Slab reinforcement parallel to girder BAR S_3

And Use dia. 16 mm bar Top with spacing = 200mm

For bottom Slab reinforcement transverse to girder BAR S_5

For bottom slab Reinforcement Transverse to girder span (Distribution Reinforcement)

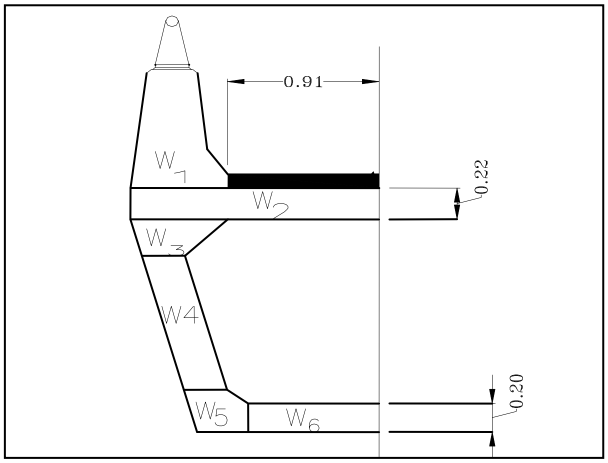
All transverse reinforcement in the bottom slab shall be extended to the exterior face of the outside web in each group and shall be anchored by a standard 90° hook.

4. Design of longitudinal girders

4.1. Loads

4.1.1 Dead Loads

a) Exterior Girder



a =2.4 c =0 d_w =0.25 b_w=0.25 overhang=0m

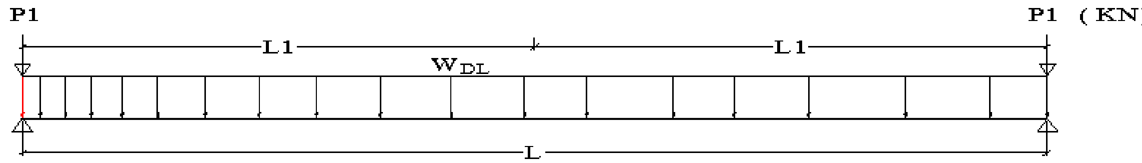
D =1.4 bottom Slab t = 0.2m Top slab t=0.22m fillet =0.1

Uniform Dead Load per Linear Load

	Dead loads (KN/m)
W_1	8.4
W_2	7.92
W_3	2.64
W_4	6
W_5	2.4
W_6	3.84
W_8(wearing surface)	2.048
SUM W_{DL}	33.248

Weight of Diaphragms :(at the two extreme ends)

Concentrated Load $P_1 = d_w \times 1.18 \times 2.25 \times 24/2 = 7.965$



$W_{DL} = 33.2475$KN/m

$P_1 = 7.965$KN $\qquad$ L_1=10.3m

$P_2 = 7.965$KN $\qquad$ L =20.6m

Shear Forces and Bending Moments due to Deal Loads on Exterior Girder

$V_{DL}\ x = P_1 + 0.5P_2 + \frac{WL}{2} - \ P_1 + W \times X \qquad$ if $0 \leq x < \frac{L}{2}$

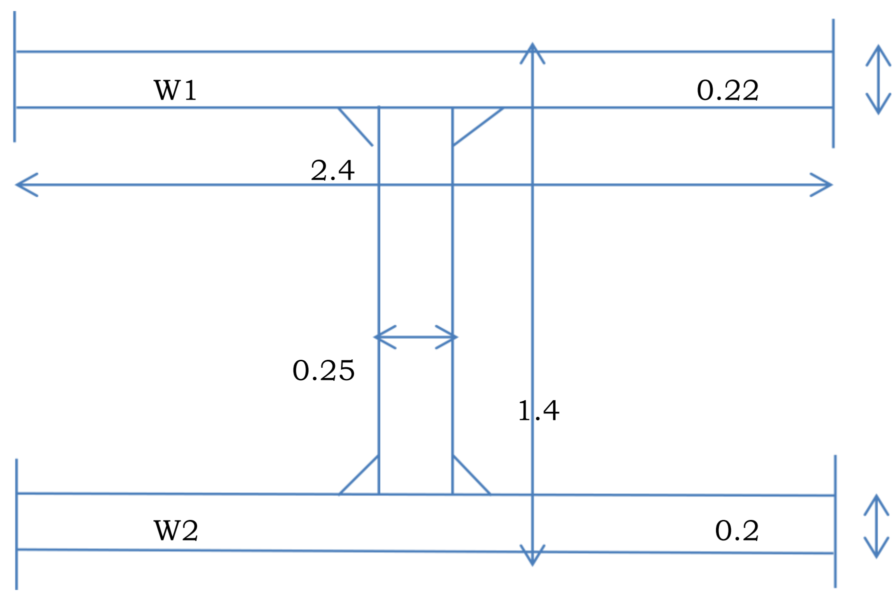
$V_{DL}\ x = P_1 + 0.5P_2 + \frac{WL}{2} - \ P_1 + P_2 + W \times X \qquad$ if $x > \frac{L}{2}$

$M_{DL}\ x = \ \ P_1 + 0.5P_2 + \frac{WL}{2}\ \ X - P_1 \times X - \ W \times x^2\ \ 2 \qquad$ if $0 \leq x < \frac{L}{2}$

$M_{DL}\ x = \ \ P_1 + 0.5P_2 + \frac{WL}{2}\ \ X - P_1 \times X - \ W \times x^2\ \ 2 - P_2\ \ X - .5L \qquad$ if $x > \frac{L}{2}$

x(m)	V_{DL} (KN)	M_{DL} (KNm)
0.00	346.43	0.00
1.030	312.19	339.19
2.060	277.94	643.10
3.090	243.70	911.75
4.120	209.45	1145.12
5.150	175.21	1343.22
6.180	140.96	1506.05
7.210	106.72	1633.60
8.240	72.47	1725.88
9.270	38.23	1782.90
10.300	3.98	1804.63

b) Interior Girder

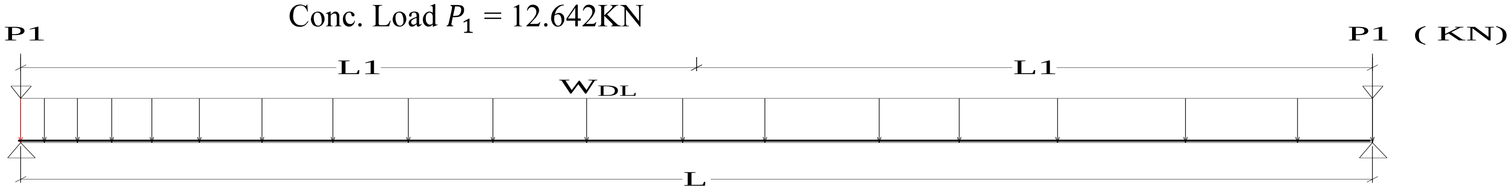


Uniform Dead Load per Linear Load

	Dead loads (KN/m)
W_1 (top slab)	11.35
W_5 (bottom slab)	10.32
W_8 (girder)	8.40
W_9 (wearing surface)	5.40
W_{10} (fillet)	0.48
Sum W_{DL}	35.95

Weight of Diaphragms :(at the two extreme ends)

Conc. Load P_1 = 12.642KN



W_{DL}=35.952KN/m

P_1=12.642KN L_1=10.3m L=20.6m

Shear Forces and Bending Moments due to Deal Loads on Interior Girder

V_{DL}(x) = P_1+ wL/2 - (P_1+ W× x) if 0≤ x <L/2

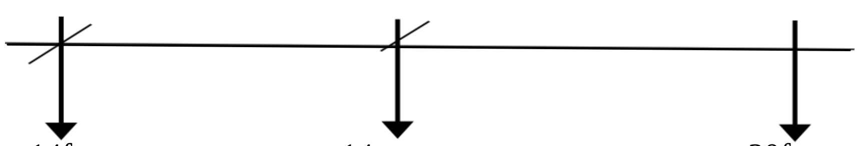
M_{DL}(x) = (P_1+wL/2) × x – P_1×x- W×x^2 /2 if 0≤ x <L/2

x (m)	V_{DL} (KN)	M_{DL} (KNm)
0.00	370.31	0.00
1.030	333.28	362.34
2.060	296.24	686.55
3.090	259.21	972.61
4.120	222.18	1220.53
5.150	185.15	1430.31
6.180	148.12	1601.94
7.210	111.09	1735.44
8.240	74.06	1830.79
9.270	37.03	1888.00
10.300	0.00	1907.07

4.1.2 Live Loads

a**) Design Truck Load: HL-93**

P/4=35KN P=145KN P=145KN



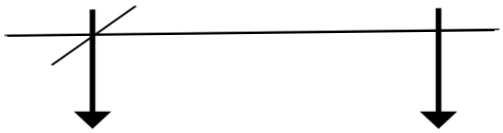
14ft 14 - 30ft

(4.267m) (4.267- 9.144m)

P = wheel load = 72.5KN

LONGITUDINAL ARRANGEMENT

P=145KN P=145KN



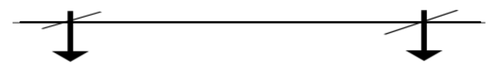
6ft (1.80m)

TRANSVERSE ARRANGEMENT

b) Design Tandem

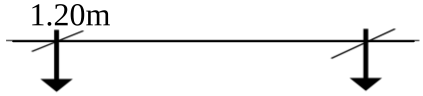
P=110KN P=110KN

1.80m



P=110KN P=110KN

1.20m



$$P = \text{wheel load} = \frac{1}{2} \times 110\text{KN} = 55.0\text{KN}$$

LONGITUDINAL ARRANGEMENT TRANSVERSE ARRANGEMENT

4.1.2.1 Dynamic Load Allowance

Section 3.13, the vehicular dynamic load allowance IM

IM = 0.33 Therefore IM = 0.33

The live loads shall be factored by 1+IM/100 = 1.33

4.1.2.2 Transverse Load Distribution

The applications of live load for the design of deck overhang according to ERA Bridge Design Manual 2002, Art. 3.9 is

• For the design of the deck overhang - 300 mm from the face of the curb or railing, and

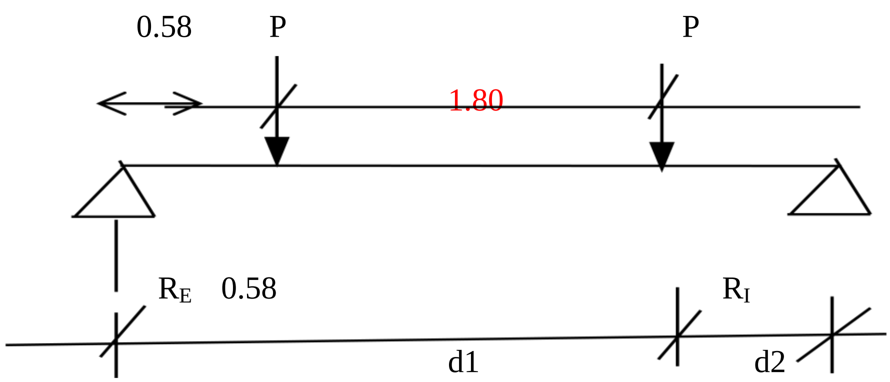
• For the design of all other components - 600 mm from the edge of the design lane.

A) Distribution Factor for Shear (Sec. 13.4: Table 13-7 & 13-8)

Exterior Girder:

Case-1: One Design lane loaded

The lever rule is applied assuming that the slab is simply supported over the longitudinal beams (Table 13-8)



$a' = 0.58m$ $a = 2.4m$

$d_1 = 2.38m$ $d_2 = 0.02m$ $d_1 + d_2 = 2.4m$

The distribution coefficient to the exterior girder for shear

R_{EX1} (shear) = $1/a \times m \times P(d_1 + d_2 + d_2)$ = 0.92P Where, m=1.2

Case-2: Two or more design lanes loaded

The distribution of live load per lane for shear in exterior girder is determined according to the formulas given in Table 13-8.

R_{EX2} (shear) = $(0.64+de/3800) \times R_{in}$ shear= 0.529 per lane, de=c-railing-$b_w/2$= 0

This factor is for one lane load which is equivalent to two lines of wheels, and thus multiplied by 2 = 1.058P

Therefore, R_{EX} (shear) in exterior girder is maximum of the above two values, R_{EX1} or R_{EX2} R_{EX} (shear) = 1.058P

Interior Girder:

Case-1: One Design lane loaded

The distribution of live load per lane for shear in interior girder is determined according to the formulas given in Table 13-7.

$$R_{INT\ 1}\ (\text{shear}) = (S/2900)\times 0.6\times(d\ \ L)^{0.1} = 0.682$$

$$\text{Where } 1800 \leq S \leq 4900$$

$$6000 \leq L \leq 73000$$

$$890 \leq d \leq 2800$$

$$Nc \leq 3$$

$$S=2.4m$$

Case-2: Two or more design lanes are loaded

The distribution of live load per lane for shear in interior girder

$$R_{INT\ 2}\ (\text{shear}) = (S/2200)\times 0.9\times(d\ \ L)^{0.1} = 0.826$$

$$\text{Where } 1800 \leq S \leq 4900$$

For two lines of wheels $6000 \leq L \leq 73000$

$$R_{INT\ 2}\ (shear) = 1.653P$$

$$890 \leq d \leq 2800$$

$$Nc \leq 3$$

$$S=2.4m$$

Therefore, R_{INT} (shear), in interior girder is maximum of the above two values, R_{INT1} or R_{INT2} R_{INT} (shear) = 1.65P

B) Distribution Factor for moment (Sec.13.4: Table 13-3 and 13-4)

Exterior Girder

Case-1: One Design lane loaded

The lever rule is applied assuming that the slab is simply supported between the longitudinal beams (Table 13-4)

R_{EXT1} (moment) = 0.92P

Case-2: Two or more design lanes loaded

Where $W_e \leq S$

R_{EXT1} (moment) = $W_e/4300$ = 0.0291P

W_e = half the web spacing, plus the total overhang (mm) = 125mm

For two lines of wheels = 0.058

Therefore, R_{EX} (moment) in exterior girder is maximum of the above two values, R_{EX1} or R_{EX2} R_{EX} (moment) = 0.92

Interior Girder

Case-1: One Design lane loaded

Where $2100 \leq S \leq 4000$

R_{INT1} (moment) =(1.75+S/1100)×(300/L)×0.35×(1/Nc)×0.45

$18000 \leq L \leq 73000$

The distribution factor of live load per lane for moment in interior girder:

$Nc \geq 3$

RINT1 (moment) = (1.75+S/1100)×(300/L)×0.35×(1/Nc)×0.45 =0.351

L=20.6m S=2.4m

Case-2: Two or more design lanes loaded

R_{INT2} (moment) = (13/Nc)×0.3 ×(S/430)(1/L)×0.25 = 0.539

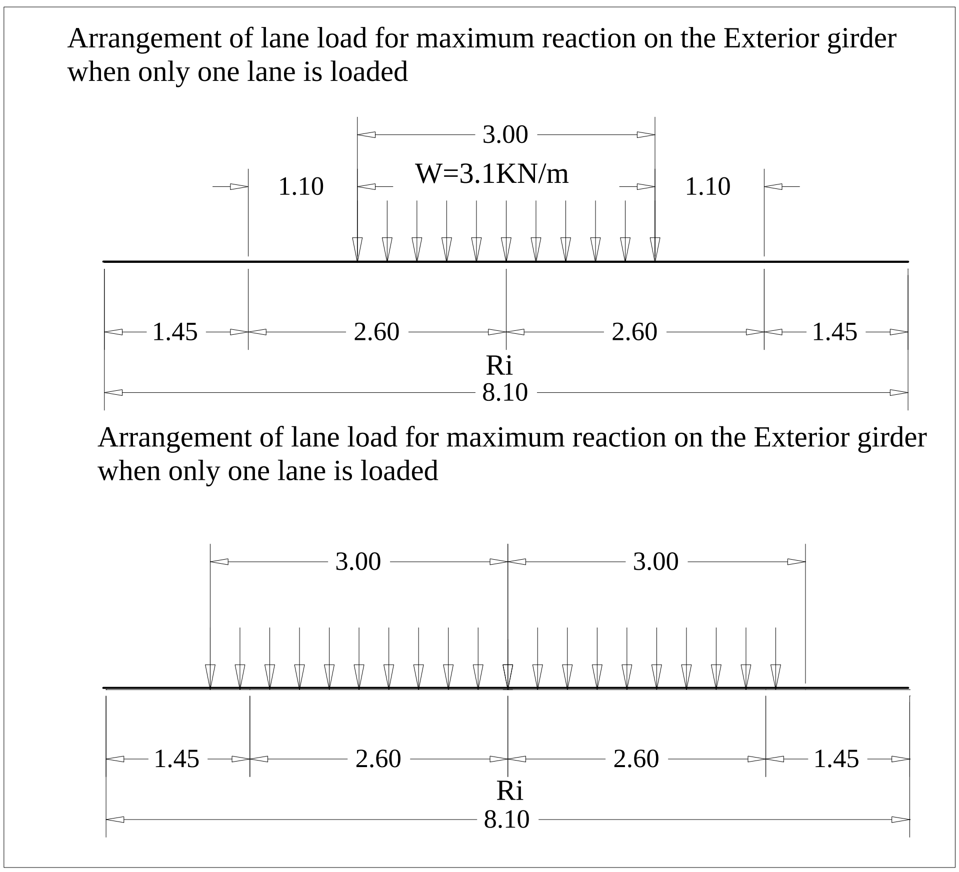
Where $2100 \leq S \leq 4000$

The distribution factor for wheel load (i.e. two lines of wheels) is 2 times the maximum of the above two values $18000 \leq L \leq 73000$

R_{INT} (moment) = 1.078P $Nc \geq 3$

C) Lane Loading

Exterior Girder



c = 0   a = 2.4

Multiple Presence factor for one lane loaded = 1.2

Multiple Presence factor for two lane loaded = 1

Distributed lane load including multiple presence factors shall be computed as follows.

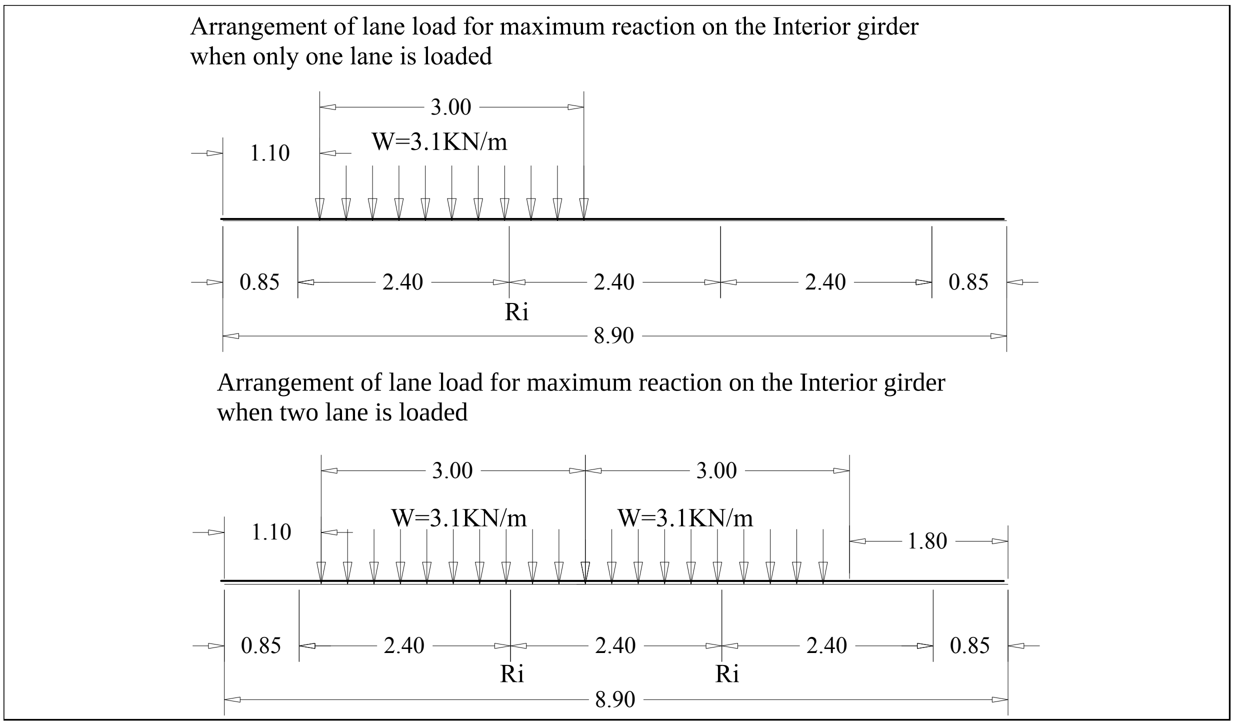
For one lane loaded case, Re (1) = -0.465KN/m

For two lane loaded case, Re (2) = 2.532KN/m

Two lane loading case governs the design!

Re = 2.532KN/m

Interior Girder



a = 2.4 c = 0

Distribution factor for moment

For one lane loaded

$R_i = (1.75+S/1100)(300/L)^{0.35}(1/Nc)^{0.45} = 0.546$

Where: S = 2400mm L = 20600mm

For two or more lanes loaded

$R_i = (13/Nc)^{0.3}(S/430)(1/L)^{0.25} = 0.723$

Therefore, R_{in} moment (1) or for half lane loading (i.e. for one line of load)

R_{in} moment (2) multiplied by two and the maximum is taken.

R_{int} moment =1.447

The two lane load case governs the design, R_i=1.447KN/m

4.1.2.3 Shear and moment due to lane loading

W_e= 2.532KN/m L = 20.6m

W_i= 1.447KN/m

Exterior $R_a = R_b$ = 26.076KN

Interior $R_a = R_b$ = 14.900KN

X	M_{ex}	M_{in}	V_{ex}	V_{in}
0	0.00	0.00	26.08	14.90
1.03	25.52	14.58	23.47	13.41
2.06	48.35	27.62	20.86	11.92
3.09	68.49	39.14	18.25	10.43
4.12	85.95	49.11	15.65	8.94
5.15	100.72	57.55	13.04	7.45
6.18	112.81	64.46	10.43	5.96
7.21	122.21	69.83	7.82	4.47
8.24	128.92	73.67	5.22	2.98
9.27	132.95	75.97	2.61	1.49
10.3	134.29	76.74	0.00	0.00

Summary of Transversal Distributed Loads

Dead Loads

Exterior Girder- W_e = 33.248 P_1 = 7.965

Interior Girder- W_i = 35.952 P_1 = 15.93

Track and Tandem Loads

Exterior Girder- Shear at support = 1.058P

Moment and shear at span = 0.92P

Interior Girder- Shear at support = 1.653P

Moment and shear in span = 1.078P

Lane Loading

Exterior Girder- W_e = 2.532KN/m

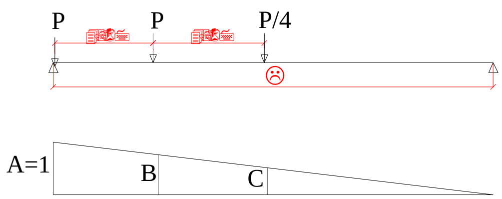
Interior Girder- W_i = 1.447KN/m

4.1.2.4 Shear Forces and Bending Moments due to Live Loads (due to truck or tandem)

a) Influence Lines for Shear Forces and Bending Moments

a-1) Design Truck: HL-93

Influence Lines for Shear Force at "x" distance from end support



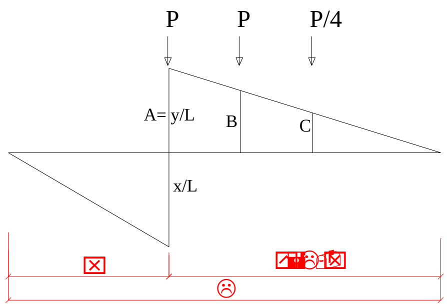
Influence Line for reaction at support

V(x=0) = P (A + B +C/4)

A=1.0

B=(L-4.267)/L

C=(L-8.534)/L



Influence line for shear at distance 'x'

Coefficients

A=y/L=(L-x)/L

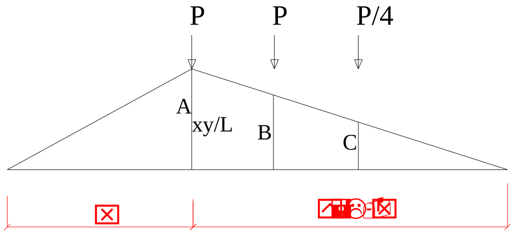
B=(y-4.267)/L=(L-x-4.267)/L

C=(y-8.534)/L=(L-x-8.534)/L

V(x) = P (A + B +C/4)

$$V(x) = P\left(\frac{L-x}{L} + \frac{L-x-4.267}{L} + \frac{L-x-8.534}{4L}\right)$$

Influence Lines for Bending Moment at "x" distance from end support



Influence line for
moment at distance 'x'

Loading 1

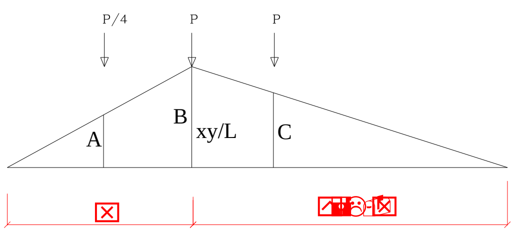
Coefficients

A=xy/L=x(L-x)/L

B=x(y-4.267)/L= x(L-x-4.267)/L

C=x(y-8.534)/L=x(L-x-8.534)/L

Mx = P (A + B + C/4)



Loading 2

A=(x-4.267)y/L=(x-4.267)(L-x)/L

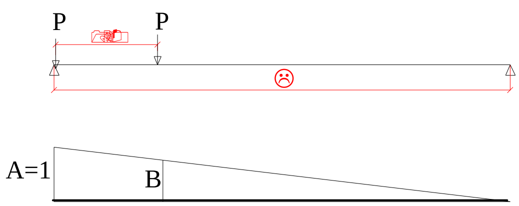
B=xy/L= x(L-x)/L

C=(y-4.267)x/L=(L-x-4.267)x/L

Mx = P (A/4 + B + C)

a-2) Design Tandem

Influence Lines for Shear Force at 'x' distance from end support

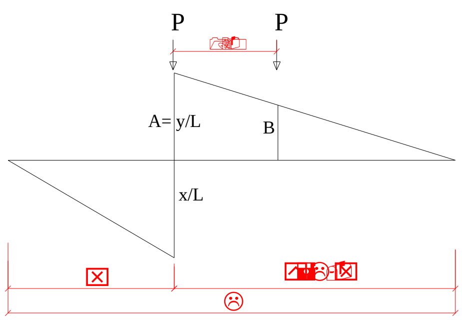


Influence Line for
reaction at support

A=1.0

B=(L- 1.20)/L

V(x=0) = P (A + B) = 1.0*P + P(L-1.20)/L



Influence line for
shear at distance 'x'

Coefficients

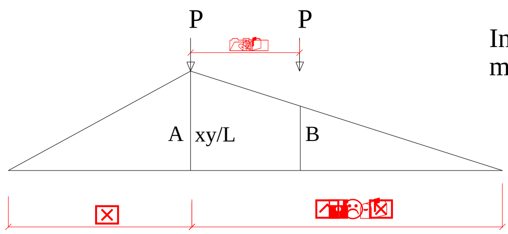
A=y/L=(L-x)/L

B=(y-1.20)/L=(L-x-1.20)/L

Vx = P (A + B)

$$Vx = P \left(\frac{(L-x)}{L} + \frac{(L-x-1.20)}{L} \right)$$

Influence Lines for Bending Moments at 'X' distance from end support



Influence line for
moment at distance 'x'

Coefficients

A=xy/L= x(L-x)/L

B=(y- 1.20)x/L=(L-x-1.20)x/L

$$Mx = P \; (A + B)$$

$$Mx = P \left(\frac{(L-x)x}{L} + \frac{(L-x-1.20)x}{L} \right)$$

b) Shear forces due to Live load plus impact

b-1) Exterior Girder

Design Truck: $V_{LL} + IM = (1.00 + IM) \times (\text{Dist. Factor}) \times (P_{truck}) \times (A + B + C/4)$

Design Tandem: $V_{LL} + IM = (1.00 + IM) \times (\text{Dist. Factor}) \times (P_{tandem}) \times (A + B)$

IM = 0.33 L = 20.6m

Dist. Factor = 1.0579

P truck = 72.5KN

P tandem = 55KN

x (m)	LiveLoad 1: Design Truck				Live Load 2: Design Tandem			V_{LL+I} (MAX)
	Coefficients			V_{LL+IM}	Coefficients		V_{LL+IM}	
	A	B	C	(KN)	A	B	(KN)	(KN)
0	1.000	0.793	0.586	197.82	1.000	0.942	150.26	197.82
1.03	0.950	0.743	0.536	186.35	0.950	0.892	142.52	186.35
2.06	0.900	0.693	0.486	174.87	0.900	0.842	134.79	174.87
3.09	0.850	0.643	0.436	163.40	0.850	0.792	127.05	163.40
4.12	0.800	0.593	0.386	151.92	0.800	0.742	119.31	151.92
5.15	0.750	0.543	0.336	140.44	0.750	0.692	111.57	140.44
6.18	0.700	0.493	0.286	128.97	0.700	0.642	103.83	128.97
7.21	0.650	0.443	0.236	117.49	0.650	0.592	96.09	117.49
8.24	0.600	0.393	0.186	106.02	0.600	0.542	88.35	106.02
9.27	0.550	0.343	0.136	94.54	0.550	0.492	80.62	94.54
10.3	0.500	0.293	0.086	83.06	0.500	0.442	72.88	83.06

b-2) Interior Girder

Design Truck: $V_{LL} + IM = (1.00 + IM) \times (\text{Dist. Factor}) \times (P_{truck}) \times (A + B + C/4)$

Design Tandem: $V_{LL} + IM = (1.00 + IM) \times (\text{Dist. Factor}) \times (P_{tandem}) \times (A + B)$

IM = 0.33 L = 20.6m

Dist. Factor = 1.653

P truck = 72.5KN

P tandem = 55KN

x (m)	Live Load 1: Design Truck				Live Load 2: Design Tandem			V_{LL+I} (MAX)
	Coefficients			V_{LL+IM}	Coefficients		V_{LL+IM}	
	A	B	C	(KN)	A	B	(KN)	(KN)
0	1.000	0.793	0.586	309.10	1.000	0.942	234.79	309.10
1.03	0.950	0.743	0.536	291.17	0.950	0.892	222.69	291.17
2.06	0.900	0.693	0.486	273.24	0.900	0.842	210.60	273.24
3.09	0.850	0.643	0.436	255.31	0.850	0.792	198.51	255.31
4.12	0.800	0.593	0.386	237.38	0.800	0.742	186.42	237.38
5.15	0.750	0.543	0.336	219.44	0.750	0.692	174.33	219.44
6.18	0.700	0.493	0.286	201.51	0.700	0.642	162.24	201.51
7.21	0.650	0.443	0.236	183.58	0.650	0.592	150.15	183.58
8.24	0.600	0.393	0.186	165.65	0.600	0.542	138.05	165.65
9.27	0.550	0.343	0.136	147.72	0.550	0.492	125.96	147.72
10.3	0.500	0.293	0.086	129.79	0.500	0.442	113.87	129.79

c) Bending Moments due to Live load plus impact

c-1) Exterior Girder

Design Truck: $V_{LL} + IM = (1.00 + IM) \times (\text{Dist. Factor}) \times (P_{truck}) \times$

Max(Sum Coeff's)

Design Tandem: $V_{LL} + IM = (1.00 + IM) \times (\text{Dist. Factor}) \times (P_{tandem}) \times (A + B)$

IM = 0.33 L = 20.6m

Dist. Factor = 0.92 P truck = 72.5KN P tandem = 55KN

When the live load is Design Truck

	Coef. (Loading 1)			Loading 1	Coeff. (Loading 2)			Loading 2	Maximum
x				sum Coef.				sum (coef.)	M_{LL+IM}
(m)	A	B	C/4	A+B+C4/4	A/4	B	C	A/4+B+C	(KNm/m)
0	-	-	-	0.000	-	-	-	-	0.00
1.03	0.98	0.77	0.14	1.882	-	0.98	0.765	1.744	166.92
2.06	1.85	1.43	0.25	3.531	-	1.85	1.427	3.281	313.28
3.09	2.63	1.99	0.34	4.950	-	2.63	1.986	4.613	439.08
4.12	3.30	2.44	0.40	6.136	-	3.30	2.443	5.739	544.32
5.15	3.86	2.80	0.43	7.091	0.17	3.86	2.796	6.824	629.01
6.18	4.33	3.05	0.44	7.813	0.33	4.33	3.046	7.707	693.13
7.21	4.69	3.19	0.42	8.304	0.48	4.69	3.193	8.358	741.43
8.24	4.94	3.24	0.38	8.564	0.60	4.94	3.237	8.777	778.63
9.27	5.10	3.18	0.31	8.591	0.69	5.10	3.178	8.965	795.27
10.3	5.15	3.02	0.22	8.387	0.75	5.15	3.017	8.921	791.36

When the live load is Design Tandem

x (m)	*Coefficients*		*Sum coef.*	M_{LL+IM}
	A	*B*	*(A + B)*	*(KNm/m)*
0	0.000	0.000	0.000	0.00
1.03	0.979	0.919	1.897	127.66
2.06	1.854	1.734	3.588	241.47
3.09	2.627	2.447	5.073	341.40
4.12	3.296	3.056	6.352	427.48
5.15	3.863	3.563	7.425	499.69
6.18	4.326	3.966	8.292	558.04
7.21	4.687	4.267	8.953	602.52
8.24	4.944	4.464	9.408	633.14
9.27	5.099	4.559	9.657	649.90
10.3	5.150	4.550	9.700	652.79

c-2) Interior Girder

Design

Truck: V_{LL} + IM = (1.00+IM) × (Dist. Factor) × (P_{truck}) × Max. (Sum Coeff's)

Design Tandem: V_{LL} + IM= (1.00+IM) × (Dist. Factor) × (P_{tandem}) × (A+B)

IM = 0.33 L = 20.6m

Dist. Factor = 1.078

P truck= 72.5 KN

P tandem= 55 KN

When the live load is Design Truck

				Loading 1				Loading 2	Maximum
x	Coef. (Loading 1)			sum Coef.	Coeff. (Loading 2)			sum Coef.	M_{LL+IM}
(m)	A	B	C	A+B+C4/4	A/4	B	C	A/4+B+C	(KNm/m)
0	-	-	-	0.000	-	-	-	-	0.00
1.03	0.98	0.77	0.14	1.882	-	0.98	0.765	1.744	195.56
2.06	1.85	1.43	0.25	3.531	-	1.85	1.427	3.281	367.03
3.09	2.63	1.99	0.34	4.950	-	2.63	1.986	4.613	514.42
4.12	3.30	2.44	0.40	6.136	-	3.30	2.443	5.739	637.72
5.15	3.86	2.80	0.43	7.091	0.17	3.86	2.796	6.824	736.93
6.18	4.33	3.05	0.44	7.813	0.33	4.33	3.046	7.707	812.06
7.21	4.69	3.19	0.42	8.304	0.48	4.69	3.193	8.358	868.65
8.24	4.94	3.24	0.38	8.564	0.60	4.94	3.237	8.777	912.23
9.27	5.10	3.18	0.31	8.591	0.69	5.10	3.178	8.965	931.73
10.3	5.15	3.02	0.22	8.387	0.75	5.15	3.017	8.921	927.14

When the live load is Design Tandem

x (m)	Coefficients		Sum (coef.)	M_{LL+IM}
	A	B	(A + B)	(KNm/m)
0	0.000	0.000	0.000	0.00
1.03	0.979	0.919	1.897	149.57
2.06	1.854	1.734	3.588	282.90
3.09	2.627	2.447	5.073	399.98
4.12	3.296	3.056	6.352	500.83
5.15	3.863	3.563	7.425	585.43
6.18	4.326	3.966	8.292	653.79
7.21	4.687	4.267	8.953	705.90
8.24	4.944	4.464	9.408	741.78
9.27	5.099	4.559	9.657	761.41
10.3	5.150	4.550	9.700	764.80

4.1.3 Seismic Force Effects

Earthquake zones: EBCS Zone -2

Since the project site is located in zero Seismic Zone no earthquake force is considered during the design of this RC Box Girder Bridge.

4.1.4 Factored Loads

Load Factors and Load Combinations

The load factors and load combinations are according to ERA's Bridge Design Manual 2002section 3.3.

The load combination to be used for design is Strength - I Limit state, Table 3-1.

$$\text{Factored Load} = g_b \times DL + 1.75 \times (LL + IM) + 1.75 \times Lane$$

$$= 1.25 \times DL + 1.75 \times (LL+IM) + 1.75 \times Lane$$

g_b =1.25

Factored Shear Forces

	Exterior Girder				Interior Girder				Design S.F.
x	V_{DL}	V_{LL+I}	V_{lane}	$1.25V_{DL}+1.75V_{LL}+IM + Lane$	V_{DL}	V_{LL+I}	V_{lane}	$1.25V_{DL}+1.75V_{LL}+IM + Lane$	V_{max}
(m)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)
0.00	346.43	197.82	26.08	824.87	370.31	309.1	14.9	1029.88	1029.88
1.03	312.19	186.35	23.47	757.41	333.28	291.2	13.4	949.61	949.61
2.06	277.94	174.87	20.86	689.96	296.24	273.2	11.9	869.33	869.33
3.09	243.70	163.40	18.25	622.51	259.21	255.3	10.4	789.06	789.06
4.12	209.45	151.92	15.65	555.06	222.18	237.4	8.9	708.78	708.78
5.15	175.21	140.44	13.04	487.60	185.15	219.4	7.5	628.51	628.51
6.18	140.96	128.97	10.43	420.15	148.12	201.5	6.0	548.23	548.23
7.21	106.72	117.49	7.82	352.70	111.09	183.6	4.5	467.96	467.96
8.24	72.47	106.02	5.22	285.25	74.06	165.7	3.0	387.68	387.68
9.27	38.23	94.54	2.61	217.79	37.03	147.7	1.5	307.41	307.41
10.30	3.98	83.06	0.00	150.34	0.00	129.8	0.0	227.13	227.13

Factored Bending Moments

	Exterior Girder				Interior Girder				Design B.M.
x	M_{DL}	M_{LL+IM}	M_{lane}	$1.25M_D + 1.75M_{LL}$+IM	M_{DL}	M_{LL+IM}	M_{lane}	$1.25M_D + 1.75M_{LL}$ +IM	M_{max}
(m)	(KNm)	(KNm)	(KNm)	(KNm)	(KNm)	(KNm)	(KNm)	(KNm)	(KNm)
0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.0	0.00	0.00
1.03	339.19	166.92	25.52	760.75	362.34	195.6	14.6	820.67	820.67
2.06	643.10	313.28	48.35	1436.72	686.55	367.0	27.6	1548.83	1548.83
3.09	911.75	439.08	68.49	2027.93	972.61	514.4	39.1	2184.48	2184.48
4.12	1145.12	544.32	85.95	2534.37	1220.53	637.7	49.1	2727.61	2727.61
5.15	1343.22	629.01	100.72	2956.04	1430.31	736.9	57.6	3178.23	3178.23
6.18	1506.05	693.13	112.81	3292.95	1601.94	812.1	64.5	3536.34	3536.34
7.21	1633.60	741.43	122.21	3553.36	1735.44	868.6	69.8	3811.63	3811.63
8.24	1725.88	778.63	128.92	3745.57	1830.79	912.2	73.7	4013.81	4013.81
9.27	1782.90	795.27	132.95	3853.01	1888.00	931.7	76.0	4123.48	4123.48
10.30	1804.63	791.36	134.29	3875.68	1907.07	927.1	76.7	4140.63	4140.63

4.2 Exterior Girder Design

4.2.1 Design for Flexure

a) Design Loads

The factored loads in the above tables are the design loads and the structure shall be designed to carry the expected design loads as calculated above.

b) Effective Compression Flange Width, b_{eff}, (AASHTO Art. 8.10.1.1)

Exterior Girder

The total width of slab effective as T-girder flange shall not exceed one-fourth of the span length of the girder. The effective flange width overhanging on either side shall not exceed six times the thickness of the slab or one half the clear distances to the next web. Therefore, the effective compression flange width, $b_{eff,}$ is the minimum of the following:

a. 1/4× span = 5.15m

b. $2\times6h_f + b_w = 2.89$m

c. b_w+2×1/2(clear spacing b/n webs) =1.45m

b_w+(Ext. cantilever + 1/2×clear spacing b/n girders) =1.45m

Effective compression flange width, b_{eff} = 1.45m

L =20.6m b_w =0.25m

h_f =0.22m (average) Overhang L =0m

Clear Span b/n girder=2.4m

Use compression flange width, b_{eff} = 1.45m

c) Spacing of Reinforcement Art. 5.10.3 Spacing of Reinforcement AASHTO 2005

The clear distance between parallel bars in a layer shall not be less than 1.5 bar diameter, maximum size of aggregate or 1.1/2 inches.

Assume d=32mm

1.5×diam. = 48mm

1.5×max. size of aggregate =1 inch =38.1mm

1.1/2 inch =38.1mm

Hence the minimum clear distance between parallel bars in a layer with no lapping is =48mm.

Assuming vertical lapping of bars, one over the other,

The minimum clear distance between parallel bars in a layer is =48mm

Therefore, the minimum center to center spacing of bars in a layer is = 80mm

Use center to center spacing of bars in a layer = 80mm

Assuming horizontal lapping of bars, one aside the other,

The minimum clear distance between parallel bars in a layer is =48mm

Therefore, the minimum center to center spacing of bars in a layer is = 112mm

Minimum clear distance between two layers of bars is 1 inch =25mm

Thus, minimum center to center spacing of bars when there is lap = 89mm

Use vertical center to center spacing of bars b/n layers = 90mm

d) Section Capacity and Reinforcement

The maximum design moment is:

M_{max} = 3875.67KNm

Try					
Total =12	No. of Bars	Dia = 32	bars		d (m)
1st row	6	bars	6	d1	0.077
2nd row	4	bars	12	d2	0.167
3rd row	2	bars	14	d3	0.257
4th row	0	bars	14	d4	0.347
5th row	0	bars	14	d5	0.437
6th row	0	bars	14	d6	0.527
7th row	0	bars	14	d7	0.617
8th row	0	bars	14	d8	0.707
9th row	0	bars	14	d9	0.797
10th row	0	bars	14	d10	0.887

Cover =50 D = 1.4m

Stirrup Dia. =12 b =1.45m

$$d' = \frac{a_i \times d_i}{a_i} = 0.1413\text{m}$$

d = D - d' =1.2587m

For rectangular beam analysis ($a < h_f$), use the following formulas:

$r = A_s/bd$

$$a = A_s \frac{f_y}{0.85 \times f_c' \times b}$$

$$ØM_n = Ø \times A_s \times f_y \times \left(d - \frac{a}{2}\right)$$

Assume rectangular beam analysis, $A_s = 9896.017\text{mm}^2$

d =1.2587m b =1.45 f_y = 400MPa

f_c' = 24Mpa b_w = 0.25m

h_f = 0.22m Ø =0.9 $r = A_s/bd = 0.006$

$a = \dfrac{A_s \times f_y}{(0.85 \times f_c' \times b)} = 133.820mm$

Since a < h_f, it is rectangular beam as assumed.

$ØM_n = Ø \times A_s \times f_y \times (d-a/2) = 4245.88KNm$

Therefore, the section capacity is

$ØM_n = 4245.881KNm \quad \gg \quad M_{max} = 3875.679KNm$ OK!!

e) Checking maximum steel area

$$r_f = \frac{A_{sf}}{b'_w d} = 0$$

$b'_{\text{bottom flange reinf. distribution width}} = 1.31m$

$b_1 = 0.85$ $f_c' = 24N/mm^2$

$f_y = 400N/mm^2$ $b_{w(weighted)} = 0.7875m$

$$r_{max} = 0.75 \times r_b = 0.75 \times \frac{b_w}{b} \times 0.85 \times b_1 \times \frac{f'_c}{f_y} \times \frac{600}{600 + f_y} + r_f = 0.0195$$

$$r_{provided} = \frac{A_s - A_{sf}}{b_w d} = 0.01 \quad \ll \quad r_{max} \qquad \text{OK !!}$$

f) Check for minimum reinforcement (Section 9.4.5)

$ØM_n \geq 1.2M_{cr}$. $b_{top} = 1.45m$

$M_{cr} = f_r \times I_g / y_t$ $h_f = 0.22m$

$fr = 0.63 \times \sqrt{f'_c} = 3.086N/mm^2$ $b_w = 0.25m$

Centroid of cross section D=1.4m

$y_t = \frac{Ai*yi}{Ai} = 0.714m$ $b_{bot} = 1.45m$ $h_{bot} = 0.2m$

$Ai \ = 0.854m^2$

$Ai * yi \ = 0.610 m^3$

Gross moment of inertia, I_g, about centroid

$I_1 = 1/12 \times b_{top} \times h_f^3 + A_1 \times d_1^2 \ = 0.107 m^4$

$I_2 = 1/12 \times b_w \times (D - h_f - h_{bot})^3 + A_2 \times d_2^2 = 0.020 m^4$

I3=1/12×bbot×(hbot)3 + A3×d32 = 0.110

$I_g \ = 0.237 m^4$

The cracking moment, M_{cr},

$M_{cr} = f_r \times I_g / y_t \ = 1025.519$KNm

$1.2M_{cr} = 1230.622$KNm

$M_{max} \ = 3875.679$KNm >> $1.2M_{cr}$. OK!

g) Serviceability Requirements

Fatigue stress limits (Section 9.6.2)

Fatigue stress limits will be checked for the service load conditions. The permissible stress range is given by Eq. 9.19.

r / h =0.30	MDL=1804.63KNm	MLL+IM=791.36KNm
$p = A_s/bd = 0.006$	b=1.450m	d=1.2587m
hf=0.22m	n=8	As= 9,896 mm^2

$f_f \ = \ 45 - 0.33 \ f_{min} + 55 \ (r/h)$

$K = [pn + 0.5 \ \frac{t_s}{d}^{\ 2}] \ / \ (pn + (t_s/d) \ = 0.284$

$k_d = \ 0.358$m T-beam

If it is rectangular beam

J =1 - k/3 = 0

If it is T-beam

$$j = \frac{6 - 6\left(\frac{h_f}{d}\right) + 2\left(\frac{h_f}{d}\right)^2 - \frac{\left(\frac{h_f}{d}\right)^3}{2pn}}{6 - 3\left(\frac{h_f}{d}\right)} = 0.926$$

The minimum stress, f_{min}, is:

$$f_{min} = \frac{M_{DL}}{As \times j \times d} = 156.531 N/mm^2$$

The maximum stress, f_{min}, is caused by the total load (MDL+LL+IM)

$$f_{max} = \frac{M_{DL} + LL + IM}{A_s \times j \times d} = 225.172 N \; mm^2$$

The actual stress range, deltaf_f, is:

Delta $f_f = f_{max} - f_{min} = 68.641 N \; mm^2$

The fatigue stress limit,f_{min}, is

$$f_f = 145 - 0.33 \times f_{min} + 55\left(\frac{r}{h}\right) = 109.845 N \; mm^2 \gg 68.641 \, N \; mm^2 \qquad OK!!$$

Control of Cracking by Distribution of Reinforcement (Sec. 9.4.5)

To control flexural cracking of the concrete, tension reinforcement shall be well distributed within maximum flexural zones.

Components shall be so proportioned that the tensile stress in the steel reinforcement at service limit state,f_s does not exceed

$$f_s = \frac{Z}{\left(d_c \times A\right) \times \frac{1}{3}} \leq 0.6 f_y$$

$$A = \text{Effective tension area per No. of bars} = b_w \times \frac{2y'}{N} = 0.005m^2$$

$$y' = d' = 0.141m$$

d_c = distance measured from extreme tension fiber to center of the closest bar

$$= 0.077m$$

$$b_w = 0.25m$$

Z = crack width parameter, assumed = 30KN/mm No. of bars= 14

Therefore, for crack control the maximum allowable stress is

$$f_{sa} = \frac{Z}{d_c \times A \times \frac{1}{3}} \leq 0.6f_y = 411.129N/mm^2$$

The maximum stress,f_{max} at service load is

$$f_{max} = \frac{M_{DL} + LL + IM}{A_s \times j \times d} = 225.172\,N \quad mm^2 \ll f_{sa} = 240N \quad mm^2 \qquad \text{OK!!}$$

h) Bar Cutting

Development of Reinforcement (Sec. 9.4.3)

Positive moment reinforcement: At least one-third of the positive moment reinforcement in simple span members shall extend along the same face of the member beyond the centerline of the support.

The basic development length, l_{db}, in mm is

For bars diam. 35 and smaller,

$$l_{db} = 0.02Ab \times \frac{f_y}{\overline{\overline{f'_c}}} \geq 0.06db \times f_y$$

$$db = 32mm$$

$$l_{db} = 0.02Ab \times \frac{f_y}{\overline{\overline{f'_c}}} = 1313.331mm$$

$$f_y = 400\text{Mpa} \qquad f'_c = 24\text{Mpa}$$

The tension development length, l_d is $0.06\text{db} \times f_y = 768\text{mm}$

$l_d = l_{db} \times \text{modification factor} = 1313.331\text{mm}$ $\text{mod. Factor} = 1$

Lap splices of Reinforcement in Tension

The length of lap for tension lap splices shall not be less than either 300mm or the 1.3 times the development length, Lap splices for diam. 32 bar = 1707.330mm

Flexural Reinforcement Extension Length (Sec. 9.4.5)

Except at supports of simple spans and at the free ends of cantilevers, reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than:

a) The effective depth of the member = 1.259m

b) 15 times the nominal diameter of bar = 0.48m

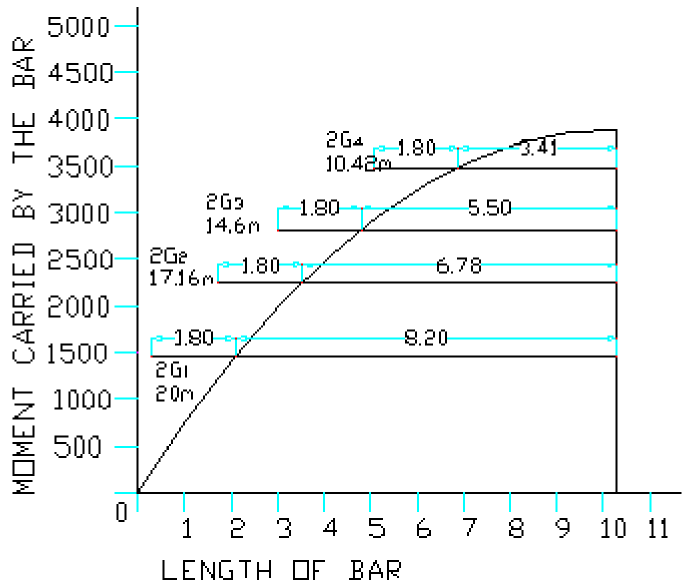
c) 1/20 of the clear span = 1m

Therefore, extension length is the maximum of the above values and development length,

l_d= 1.313m

Span Length = 20.6m

X (m)	Mx (KNm)
0.00	0.00
1.03	760.75
2.06	1436.72
3.09	2027.93
4.12	2534.37
5.15	2956.04
6.18	3292.95
7.21	3553.36
8.24	3745.57
9.27	3853.01
10.30	3875.68



Bar Cutting & Resisting Moment for Exterior Girder

Rectangular Analysis $f_M = fA_s f_y \left(d - \frac{a}{2}\right)$

$D = 1.4m$ $b_{eff} = 1.45m$ $b_w = 0.25m$

Bottom reinforcement dist. Width $b'_w = 1.31m$

$h_f = 0.22m$ $f = 0.9$

$f_y = 400Mpa$ $f'_c = 24Mpa$

n- (Number of bottom bars in bottom slab) = 6

m- (Number of upper bars in bottom slab) = 6

$d_c = 77mm$

Name	No of Bars	Dia. (mm)	A_s (mm^2)	d' (m)	d =D-d' (m)	$p = \frac{A_s}{bd}$	$a = \frac{A_s f_y}{0.85f'_c\, b}$ (mm)	Rect. or T-beam	Rect. Beam $\emptyset Mn = \emptyset \times A_s f_y \; d - \frac{a}{2}$ (KNm)
G4	12	32	9651	0.165	1.235	0.00539	131	Rect.	**4064.7**
G3	10	32	8042	0.146	1.254	0.00442	109	Rect.	**3472.1**
G2	8	32	6434	0.141	1.259	0.00353	87	Rect.	**2814.8**
G1	6	32	4825	0.077	1.323	0.00252	65	Rect.	**2241.6**
G0	4	32	3217	0.116	1.285	0.00173	44	Rect.	**1462.4**

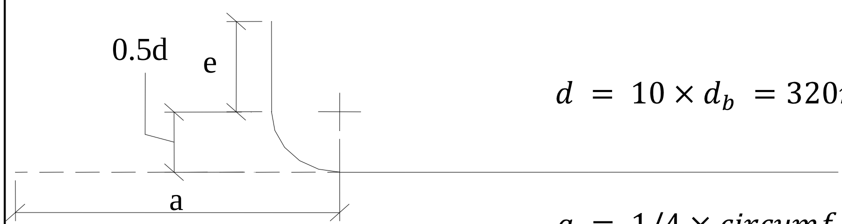
Type	No. of bars	Total No. of bars	R or T	Resisting Moment	Cutting Moment
G0	4	4	R	1462.411	no cutting
G1	2	6	R	2241.604	1462.411
G2	2	8	R	2814.797	2241.604
G3	2	10	R	3472.098	2814.797
G4	2	12	R	0	3472.098

Length of Flexural Reinforcement Bar

Length of Flexural Reinforcement Bar	
Bar - G0 = 4× Diam. 32 bars	21.97m
Bar - G1 = 2× Diam. 32 bars	20.00m
Bar - G2 = 2× Diam. 32 bars	17.16m

Bar - G3 = 2×Diam.32 bars	14.60m
Bar - G4 = 2×Diam.32 bars	10.42m

Standard Hook Detail

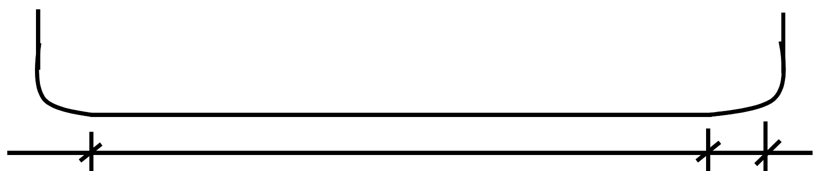


$$e = 12 \times d_b = 12 \times 32mm = 384mm$$

$$d = 10 \times d_b = 320mm$$

$$a = 1/4 \times circumf. + 12 \times d_b = 636mm$$

TOTAL LENGTH = 21.89 $12d_b = 360.00$



i) Skin Reinforcement

If the depth of side face of a member exceeds 3ft (0.91m), longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance d/2 nearest the flexural tension reinforcement.

Area of skin reinforcement, (on each side face)

$A_{sk} \geq 0.012 \times (d - 30)\ in^2 / ft$

Depth of side face $= D - h_f - h_b = 980mm \gg 3ft = 914.4mm$

Skin Reinforcement shall be provided!

$A_{sk} \geq 0.012 \times (d - 30) in^2 / ft = 0.2664 in^2 / ft = 564 mm^2 / m$

$d = 1.326m$ $\quad$ $diam. = 16mm$

Spacing $= a_s / A_s \times 1000 = 356.493mm$

Maximum spacing, S_{max}, lesser of d/6 or 12" = 221mm

Use diam. 16mm bars c/c 220mm

4.2.2 DESIGN FOR SHEAR

Shear strength

Design of cross sections subject to shear shall be based on

$V_u \leq ØV_n = Ø\ V_c + V_s \qquad Ø = 0.9$

Where V_u = factored shear forces at the section

V_n = the nominal shear strength, determined as the lesser of

$V_n = V_c + V_s$ or $\qquad V_n = 0.25 \times f'_c \times b_v \times d_v$

V_c = the nominal shear strength provided by the concrete, determined by:

$V_c = 0.083 \times b \times \sqrt{f'_c} \times b_v \times d_v$, where b = 2

V_s = the nominal shear strength provided by the shear reinforcement

$V_s = A_v \times f_v \times d / s$

Shear strength provided by concrete

Shear stress provided by concrete

$v_c = 0.083 \times b \times \sqrt{f'_c} = 0.813Mpa$

The shear strength carried by concrete,

$V_c = v_c \times b_w \times d = 0.813Mpa$

Shear strength provided by shear reinforcement

Where the factored shear force, V_u, exceeds shear strength $ØV_c$, shear reinforcement shall be provided to satisfy the equation, $V_u \leq ØV_n = Ø\ V_c + V_s$

When shear reinforcement perpendicular to the axis of the member is used

$$V_s = \frac{V_u}{\text{Ø}} - V_c = A_v \times f_y \times \frac{d}{s}$$

Therefore, spacing of shear reinforcement, s, is:

$$S = A_v \times f_y \times \frac{d}{Vs} = A_v \times f_y \times \frac{d}{\frac{V_u}{\text{Ø}} - V_c} = A_v \times f_y \times \frac{d}{\frac{V_u}{\text{Ø}} - 00.742 \times b_w \times d}$$

Minimum shear reinforcement (Eq.12.34)

$$A_v = 0.083 \times \sqrt{f'_c} \times b_w \times S \ \ f_y$$

$$S_{max} = \frac{A_v \times f_y}{0.083 \times \sqrt{f'_c} \times b_w}$$

$$f'_c = 24MPa \qquad f_y = 300MPa$$

$$A_v = diam.12 = 226.195mm^2$$

Maximum spacing of transverse shear reinforcement

$$\text{If } V_u < 0.10 \times f'_c \times b_w \times d,\ S \leq 0.8d \leq 600mm$$

$$\text{If } V_u \geq 0.10 \times f'_c \times b_w \times d,\ S \leq 0.4d \leq 300mm$$

Sections located less than a distance d from support may be designed for the same shear as that computed at a distance d

X (m)	V_x (KN)
0.00	1029.88
1.03	949.61
2.55	(Vu at distance d)
2.06	869.33
3.09	789.06
4.12	708.78
5.15	628.51

X (m)	V_x (KN)
6.18	548.23
7.21	467.96
8.24	387.68
9.27	307.41
10.30	227.13

$D = 1.4m$ $f_y = 300Mpa$

$b_{eff} = 1.325m$ $f'_c = 24Mpa$ $f = 0.9m$

$b_w = 0.25m$ $A_v = \text{diam}.12 = 226.195mm2$

x (m)	V_u (KN)	d (m)	$V_c = 0.813 b_w d \times 10^{-3}$ (KN)	$V_s = \frac{V_u}{\emptyset} - V_c$ (KN)	$0.10 \times f'_c \times b_w \times d$ (KN	S_{max1} (mm)	$S_{max2} = \frac{A_v \times f_y}{0.083 \times \sqrt{f'_c} \times b_w}$ (mm)	Spacing by analysis $S = A_v \times f_y \times \frac{d}{\frac{V_u}{\emptyset} - V_c}$ (mm)	Spacing provided (mm)
0.00	1029.88	1.285	261.07	883.24	770.7	300	668	99	80
1.03	949.61	1.285	261.07	794.05	770.7	300	668	110	80
2.55	869.33	1.323	268.90	697.03	793.8	300	668	129	80
2.06	789.06	1.323	268.90	607.83	793.8	600	668	148	100
3.09	708.78	1.259	255.84	531.69	755.3	600	668	161	100
4.12	628.51	1.259	255.84	442.50	755.3	600	668	193	100
5.15	548.23	1.259	255.84	353.31	755.3	600	668	242	150
6.18	467.96	1.254	254.79	265.16	752.2	600	668	321	150
7.21	387.68	1.254	254.79	175.96	752.2	600	668	483	300
8.24	307.41	1.235	251.05	90.51	741.1	600	668	-	300
9.27	227.13	1.235	251.05	1.32	741.1	600	668	-	300
10.30	0.00	1.235	251.05	-	741.1	600	668	-	300

4.2.3 Deflection

a) Computation of Gross Moment of Inertia

Centroid of cross section for exterior girder

For simplicity of calculations, the slab surface is assumed level, i.e., without cross fall.

The center of gravity is calculated from bottom of girder

Part	Area, A_i (m^2)	Centroid, y_i (m)	$A_i y_i$ (m^3)
Top Slab	0.26	1.29	0.34056
Bottom slab	0.29	0.10	0.029
Girder	0.25	0.69	0.169
Sum	**0.80**		**0.54**

Centroid of area

$$y_1 = sum\ A_i \times \frac{y_i}{A_i} = 0.674m$$

Span $= 20.6$ $d_w = 0.25$

a $= 2.4$ D $= 1.4l$

c $= 0$ $T_{top} = 0.22$

w $= 0.25$ $T_{bottom} = 0.2$

Gross moment of Inertia for exterior girder

Part	Area, A_i (m^2)	y_i (m)	$ycg = y_i - y_1$ (m)	I_g (m^4)	$A_i \times (ycg)^2$ (m^3)	$I_g + A_i \times ycg^2$ (m^4)
Top Slab	0.26	1.29	0.62	0.001287	0.100	0.1014
Bottom slab	0.29	0.10	0.57	0.000967	0.096	0.0965
Girder	0.25	0.69	0.02	0.019608	0.000	0.0197
					Sum I_g =	**$0.2176m^4$**

b) Computation of Effective Moment of Inertia

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \times \mathbf{I_g} + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq \mathbf{I_g}$$

$$f_r = 0.63 \times \sqrt{f'_c} = 3.086 \mathrm{N/mm^2}$$

$$M_{cr} = f_r \times \frac{I_g}{y_t} = 996.493 \mathrm{KNm}$$

Weight of superstructure

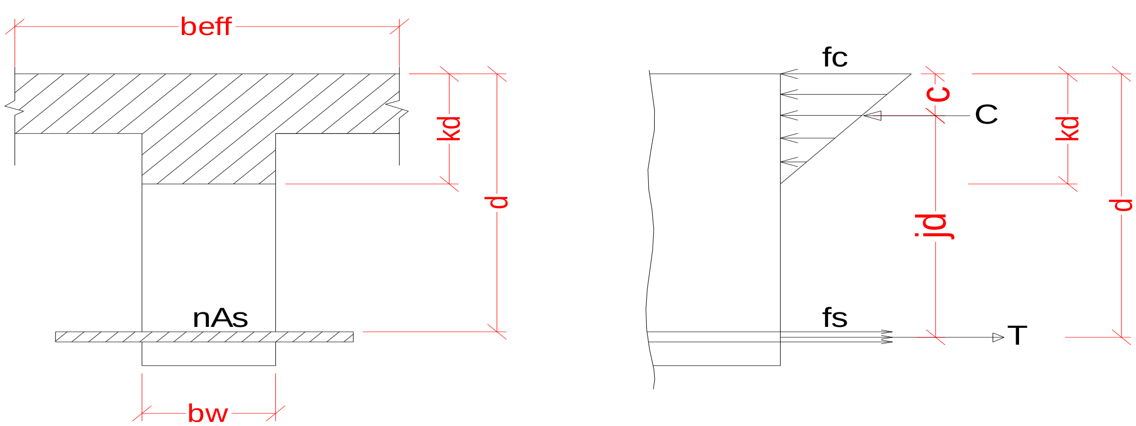
Part	W (KN)	W (KN/m)
Top Slab	197.68	9.32
Bottom Slab	217.15	10.24
Girder	183.46	8.65
Sum W (KN/m)		**28.22**

$$M_a \text{ Total Slab} = W \times \frac{L^2}{8} = 1585.472$$

$$MDl._{ext} = W \times \frac{L^2}{8} = 1804.6$$

$$M_a = 1804.63 \mathrm{KNM}$$

Transformed section



$k_d = 0.358m$ previous calculation $d' = 0.141m$

$b_{eff} = 1.45m$ $D = 1.4m$

$b_w = 0.25m$ $h_f = 0.22m$

$n = 8$ $j = 0.926m$

$A_s = 9896.017mm^2$ $d = 1.259m$

Centroid of cross section (y_1 measured from bottom)

Part	Area, A_i (m^2)	y_i (m)	$A_i y_i$ (m^3)
Slab	0.26	1.29	0.34056
Girder	0.25	0.69	0.169
Transformed steel, nA_s	0.08	0.14	0.011
Sum	0.59		0.52

$$y_1 = \frac{A_i \times \quad y_i}{A_i} = 0.885m$$

Computation of Moment of Inertia of Cracked section, Icr

Part	Area, Ai m^2	$ycg = y_i - y_1$ (m)	Icg (m^4)	$A_i \times (ycg)^2$ (m^3)	$I_g + A_i \times ycg^2$ (m^4)
Slab	0.26	0.4045	0.001287	0.0432	0.0445
Girder	0.25	0.1955	0.019608	0.0094	0.0290
Transformed steel, nA_s	0.08	0.7442	-	0.0438	0.0438
				Sum =	0.1173m4

Effective moment of Inertia, I_e

$$I_e = \frac{M_{cr}}{M_a}^3 \times I_g + \quad 1 - \frac{M_{cr}}{M_a}^3 \quad I_{cr} = 0.134m^4 \leq I_g = 0.218m^4$$

Computation of Dead Load Deflection using effective moment of Inertia, I_e

The maximum deflection is calculated by using the formula;

$$\text{Deflection, max.} = 5 \times W \times \frac{L^4}{384 \times E \times I_e} = 24.0m$$

$E = E_c = 24768Mpa \qquad W = 34.021KN/m$

$\text{Long term deflection} = \text{Instantaneous deflection} \times \text{factor}$

$L = 20.6m$

$$\text{Factor} = 3 - 1.2 \times \frac{A_s{}'}{A_s} = 3 \qquad A_s' = 0$$

$\text{Long term deflection} = 72.001mm$

4.3 Interior Girder Design

4.3.1. Design for Flexure

a) Design Loads

The factored loads in the above tables are the design loads and the structure shall be designed to carry the expected design loads as calculated above.

b) Effective Compression Flange Width, b_{eff}, (AASHTO Art. 8.10.1.1)

Interior Girder

The total width of slab effective as T-girder flange shall not exceed one-fourth of the span length of the girder. The effective flange width overhanging on either side shall not exceed six times the thickness of the slab or one half the clear distances to the next web. Therefore, the effective compression flange width b_{eff} is the minimum of the following:

Interior Girder

The effective compression flange width b_{eff} is the minimum of the following:

a. $1/4$ span $= 5.15\text{m}L = 20.6\text{m}$

b. $2\times6h_f + b_w = 2.89\text{m}hf = 0.22\text{m}$

c. $b_w+2\times1/2$(clear spacing b/n webs) $= 2.5\text{m}bw = 0.25\text{m}$

Effective compression flange width, $b_{eff} = 2.5\text{m} \times$ Clear S b/n gir. $= 2.25\text{m}$

Use compression flange width, $b_{eff} = 2.5\text{m}$

c) Spacing of Reinforcement (Art. 5.10.3) New AASHTO

The clear distance between parallel bars in a layer shall not be less than 1.5 bar diameter, maximum size of aggregate or 1.1/2 inches.

$1.5 \times$ diam. $= 48\text{mm}$ Assume $d = 32\text{mm}$

$1.5 \times$ max. size of aggrergate $= 1$ inch $= 38.1\text{mm}$

$1.1/2$ inch $= 38.1\text{mm}$

Hence the minimum clear distance between parallel bars in a layer with no lapping is $=$ 48mm

Assuming vertical lapping of bars, one over the other,

The minimum clear distance between parallel bars in a layer is $= 48\text{mm}$

Therefore, the minimum center to center spacing of bars in a layer is $= 80\text{mm}$

Use center to center spacing of bars in a layer $= 80\text{mm}$

Assuming horizontal lapping of bars, one aside the other, T

he minimum clear distance between parallel bars in a layer is $= 48\text{mm}$

Therefore, the minimum center to center spacing of bars in a layer is $= 112\text{mm}$

Minimum clear distance between two layers of bars is 1 inch $= 25\text{mm}$

Thus, minimum center to center spacing of bars when there is lap $= 89\text{mm}$

Use vertical center to center spacing of bars b/n layers $= 90\text{mm}$

d) Section Capacity and Reinforcement

The maximum design moment is:

$\text{Mmax} = 4140.631\text{KNm}$

Try					
Total = 14	No. of bars	Diam. = 32	bars		d (m)
1st row	6	bars	6	d1	0.078
2nd row	6	bars	12	d2	0.168
3rd row	2	bars	14	d3	0.258
4th row	0	bars	14	d4	0.348
5th row	0	bars	14	d5	0.438
6th row	0	bars	14	d6	0.528
7th row	0	bars	14	d7	0.618
8th row	0	bars	14	d8	0.708
9th row	0	bars	14	d9	0.798
10th row	0	bars	14	d10	0.888

Cover =50 D = 1.4m

Stirrup Dia. =12 b =2.5m

$$d' = \frac{a_i \times d_i}{a_i} = 0.1423\text{m}$$

d = D - d' =1.2587m

For rectangular beam analysis ($a < h_f$), use the following formulas:

$r = A_s/bd$

$$a = A_s \frac{f_y}{0.85 \times f_c' \times b}$$

$ØM_n = Ø×A_s×f_y× \ \ d-\frac{a}{2}$

Assume rectangular beam analysis, A_s=11259.468mm^2

d =1.258m b =2.5 f_y = 400MPa

f_c' = 24Mpa b_w = 0.25m

h_f = 0.22m=220mm Ø =0.9

r= A_s/bd = 0.009

a = $A_s×f_y/(0.85×f_c'×b)$ = 88.310mm

Since a < h_f, it is rectangular beam as assumed.

$ØM_n$ =Ø × A_s × f_y ×(d-a/2) = 4919.052KNm

Therefore, the section capacity is

$ØM_n$ =4919.052KNm ≫ M_{max} = 4140.631KNm OK!!

e) Checking maximum steel area

$$r_f = \frac{A_{sf}}{b'_w d} = 0$$

$b'(\text{bottom flange reinf. distribution width}) = 1\text{m}$

b_1=0.85 f'_c =24N/mm^2

f_y =400N/mm^2 $b_{w(weighted)}$ = 1.325m

$$r_{max} = 0.75 \times r_b = 0.75 \times \frac{b_w}{b} \times 0.85 \times b_1 \times \frac{f'_c}{f_y} \times \frac{600}{600 + f_y} + r_f = 0.02$$

$$r_{provided} = \frac{A_s - A_{sf}}{b_w d} = 0.007 \ll r_{max} \quad \text{OK !!}$$

f) Check for minimum reinforcement (Section 9.4.5)

$ØM_n \geq 1.2M_{cr}$ $M_{cr} = f_r \times \frac{I_g}{y_t}$

$f_r = 0.63 \times \sqrt{f'_c} = 3.086 \text{N/mm}^2$

Centroid of cross section

$$Y_t = \frac{A_i \times y_i}{A_i} = 0.836\text{m}$$

$$A_i \times y_i = 0.908\text{m3}$$

$$A_i = 1.085\text{m2}$$

$b_w = 0.25\text{m}$ $\quad$ $h_f = 0.22\text{m}$ $\quad$ $h_{bop} = 0.2\text{m}$

$D = 1.4\text{m}$ $\quad$ $b_{top} = 2.5\text{m}$ $\quad$ $b_{bop} = 1.45\text{m}$

Gross moment of inertia,I_g about centroid

$$I_1 = \frac{1}{12} \times b_{top} \times {h_f}^3 + A_1 \times {d_1}^2 = 0.115\text{m}^4$$

$$I_2 = \frac{1}{12} \times b_w \times \; D - h_f - h_{bop} \;^3 + A_2 \times {d_2}^2 = 0.025\text{m}^4$$

$$I_3 = \frac{1}{12} \times b_{bop} \times \; h_{bop} \;^3 + A_3 \times {d_3}^2 = 0.158$$

$$I_g = 0.298\text{m}^4$$

The cracking moment,M_{cr}

$$M_{cr} = f_r \times \frac{I_g}{y_t} = 1101.305\text{KNm}$$

$$1.2M_{cr} = 1321.566\text{KNm}$$

$M_{max} = 4140.631\text{KNm} \gg 1.2M_{cr}$ $\quad$ OK!

g) Serviceability Requirements

Fatigue stress limits (Section 9.6.2)

Fatigue stress limits will be checked for the service load conditions. The permissible stress range is given by Eq. 9.19.

$F_f = 145 - 0.33\, f_{min} + 55 \quad \frac{r}{h}$

$K = [pn + 0.5 \quad \frac{t_s}{d} \quad ^2 \;] \,/\, (pn + (t_s/d) = 0.353$

$kd \; = 0.443m \qquad\qquad T - beam$

$\frac{r}{h} = 0.3 \qquad\qquad M_{\;DL} = 1907.074KNm \qquad M_{LL} + IM = 927.143KNm$

$b = 2.500m \qquad d = 1.258m \qquad p = \frac{A_s}{bd} = 0.009$

$A_s = 11259.468mm^2 \qquad n = 8 \qquad\qquad h_f = 0.22m$

If it is rectangular beam

$j = 1 - k/3 = 0$

If it is T-beam

$$j \; = \frac{6 \; - \; 6 \;\; \frac{h_f}{d} \;\; + 2 \;\; \frac{h_f}{d} \; ^2 \; - \frac{\frac{h_f}{d} \; ^3}{2pn}}{6 \; - \; 3 \;\; \frac{h_f}{d}} \;\; = 0.922$$

The minimum stress f_{min} is:

$f_{min} \;\; = \; \frac{MDL}{A_s \times j \times d} \; = 146.037N/mm^2$

The maximum stress, f_{max} is caused by the total load $M_{DL} + LL + IM$

$f_{max} = \; \frac{M_{DL} + LL + IM}{A_s \times j \times d} \; = 217.034N/mm^2$

The actual stress range delta f_f is:

$\text{delta}\; f_f \; = f_{max} \; - \; f_{min} = 70.997N/mm^2$

The fatigue stress limit, f_f is

$f_f \;\; = \; 145 \; - \; 0.33 \times \; f_{min} + \; 55 \quad \frac{r}{h} \quad = 113.308N/mm^2 \quad >> \quad 70.997N/mm^2 \qquad OK\,!!$

Control of Cracking by Distribution of Reinforcement (Sec. 9.4.5)

To control flexural cracking of the concrete, tension reinforcement shall be well distributed within maximum flexural zones. Components shall be so proportioned that the tensile stress in the steel reinforcement at service limit state,f_s does not exceed

$$f_s = \frac{Z}{\sqrt[3]{d_c \times A}} \leq 0.6 \times f_y$$

$$A = \frac{\text{Effective tensionarea}}{\text{No. of bars}} = b_w \times \frac{2y'}{N} = 0.005m^2$$

$y' = d' = 0.142m \qquad b_w = 0.25m \qquad \text{No. of bars} = 14$

d_c = distance measured from extreme tension fiber to center of the closest bar = 0.078m

Z = crack width parameter, assumed = 30KN/mm

Therefore, for crack control the maximum allowable stress is

$$f_{sa} = \frac{Z}{\sqrt[3]{d_c \times A}} \leq 0.6 \times f_y = 408.403\ N\ \ mm^2$$

The maximum stress, f_{max} at service load is

$$f_{max} = \frac{M_{DL} + LL + IM}{A_s \times j \times d} = 217.034N\ \ mm^2 \ll f_{sa} = 240N\ \ mm^2 \qquad \text{OK!!}$$

h) Bar Cutting

Development of Reinforcement (Sec. 9.4.3)

Positive moment reinforcement: At least one-third of the positive moment reinforcement in simple span members shall extend along the same face of the member beyond the centerline of the support.

The basic development length, l_{db}, in mm is

For bars diam. 35 and smaller, $l_{db} = 0.02Ab \times \frac{f_y}{\overline{f'_c}} \geq 0.06db \times f_y$

$l_{db} = 0.02Ab \times \frac{f_y}{\overline{f'_c}} = 1313.331mm$

$db = 32mm$ $f_y = 400Mpa$ $fc' = 24Mpa$

$0.06db \times f_y = 768mm$ $\text{mod. Factor} = 1$

The tension development length,l_d is

$l_d = l_{db} \times \text{modification factor} = 1313.331mm$

Lap splices of Reinforcement in Tension

The length of lap for tension lap splices shall not be less than either 300mm or the1.3 times the development length

Lap splices for diam. 32 bar = 1707.33mm

Flexural Reinforcement Extension Length (Sec. 9.4.5)

Except at supports of simple spans and at the free ends of cantilevers, reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than:

a) $\text{The effective depth of the member} = 1.258m$

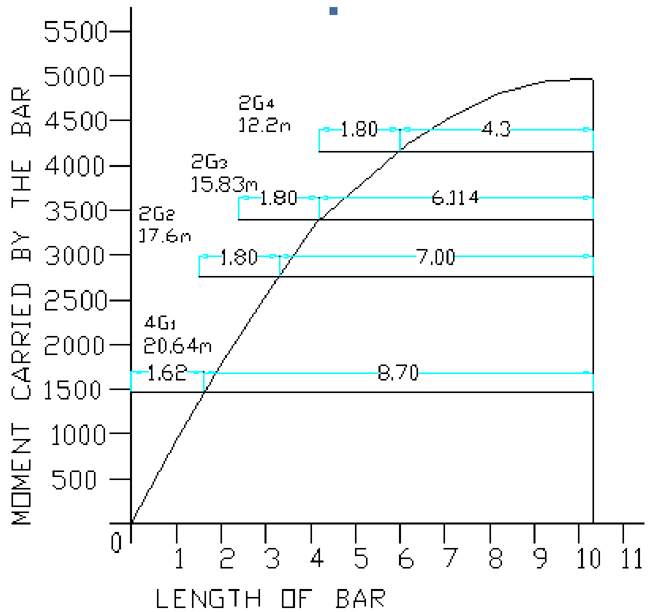
b) $15 \text{ times the nominal diameter of bar} = 0.48m$

c) $1/20 \text{ of the clear span} = 1m$

Therefore, extension length is the maximum of the above values and development length, l_d =1.313m

Span Length =20.6m

X (m)	Mx (KNm)
0.00	0.00
1.03	820.67
2.06	1548.83
3.09	2184.48
4.12	2727.61
5.15	3178.23
6.18	3536.34
7.21	3811.63
8.24	4013.81
9.27	4123.48
10.30	4140.63



Bar Cutting & Resisting Moment for Interior Girder

Rectangular Analysis $\quad fM = fA_s \times f_y \left(d - \frac{a}{2}\right)$

$D = 1.4\text{m} \qquad b_{eff} = 2.5\text{m} \qquad b_w = 0.25\text{m}$

Bottom reinforcement dist. Width, $b_{w'} = 1\text{m} \qquad h_f = 0.22\text{m}$

$f = 0.9 \qquad f_y = 400\text{Mp} \qquad f'_c = 24\text{Mpa}$

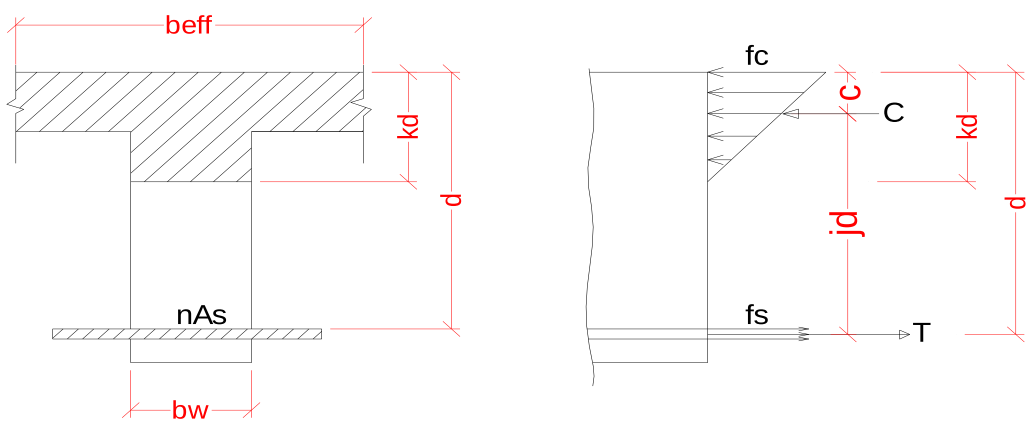
n (no. of bottom bars in bottom slab) $= 6$

m (no. of upper bars in bottom slab) $= 6$

$d_c = 78\text{mm}$

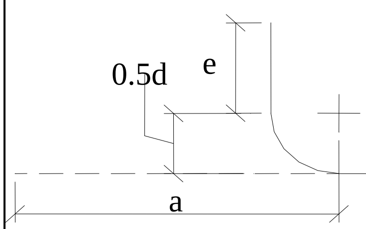
	No of Bars	Diam. (mm)	A_s mm^2	d' (m)	d =D-d' (m)	$p = \frac{A_s}{bd}$	$a = \frac{A_s f_y}{0.85 f'_c\, b}$ (mm)	Rect. or T-beam	Rect. Beam $ØMn = Ø \times A_s f_y \left(d - \frac{a}{2}\right)$ (KNm)
G4	14	32	11259	0.142	1.258	0.00358	88	Rect.	4919.1
G3	12	32	9651	0.166	1.234	0.00313	76	Rect.	4155.9
G2	10	32	8042	0.199	1.201	0.00268	63	Rect.	3385.4
G1	8	32	6434	0.185	1.216	0.00212	50	Rect.	2756.9
G0	4	32	3217	0.117	1.283	0.00100	25	Rect.	1471.3

Type	No. of bars	Total No. of bars	R or T	Resisting Moment	Cutting Moment
G0	4	4	R	1471.253	no. cutting
G1	4	8	R	2756.940	1471.253
G2	2	10	R	3385.351	2756.940
G3	2	12	R	4155.855	3385.351
G4	2	14	R	4919.052	4155.855



Length of Flexural Reinforcement Bar	
$\mathrm{Bar} - \mathrm{G0} = 4 \times \mathrm{Diam.\,32\,bars}$	21.89m
$\mathrm{Bar} - \mathrm{G1} = 4 \times \mathrm{Diam.\,32\,bars}$	20.64m
$\mathrm{Bar} - \mathrm{G2} = 2 \times \mathrm{Diam.\,32\,bars}$	17.60m
$\mathrm{Bar} - \mathrm{G3} = 2 \times \mathrm{Diam.\,32\,bars}$	15.83m
$\mathrm{Bar} - \mathrm{G4} = 2 \times \mathrm{Diam.\,32\,bars}$	12.20m

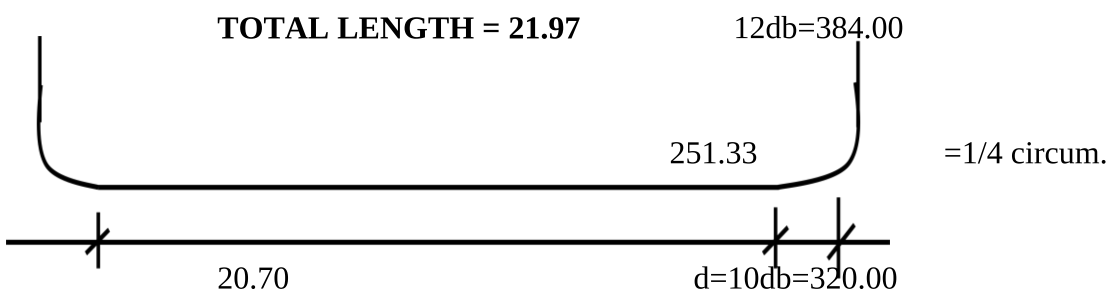
Standard Hook Detail



$$e = 12 \times d_b = 12 \times 32mm = 384mm$$

$$d = 10 \times d_b = 320mm$$

$$a = 1/4 \times circumf. + 12 \times d_b = 636mm$$



i) Skin Reinforcement

If the depth of side face of a member exceeds 3ft, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance d/2 nearest the flexural tension reinforcement.

Area of skin reinforcement, (on each side face)

$A_{sk} \geq 0.012 \times (\mathrm{d} - 30)\ \mathrm{in}^2 / \mathrm{ft}$

Depth of side face $= \mathrm{D} - \mathrm{h_f} - \mathrm{h_b} = 980\mathrm{mm} \gg 3\mathrm{ft} = 914.4\mathrm{mm}$

Skin Reinforcement shall be provided!

$A_{sk} \geq 0.012 \times (d - 30)in^2 / ft = 0.2664in^2 / ft = 560mm^2 / m$

$d = 1.323m$ $\quad\quad$ $diam. = 16mm$

$Spacing = a_s/A_s \times 1000 = 359.039mm$

Maximum spacing, S_{max}, lesser of $d/6$ or $12" = 220.33mm$

Use diam. 16mm bars c/c 220mm

4.3.2 DESIGN FOR SHEAR

Shear strength

Design of cross sections subject to shear shall be based on

$V_u \leq ØV_n = Ø \; V_c + V_s \quad\quad Ø = 0.9$

Where V_u = factored shear forces at the section

V_n = the nominal shear strength, determined as the lesser of

$V_n = V_c + V_s$ or $\quad\quad V_n = 0.25 \times f'_c \times b_v \times d_v$

V_c = the nominal shear strength provided by the concrete, determined by:

$V_c = 0.083 \times b \times \sqrt{f'_c} \times b_v \times d_v$, where $b = 2$

V_s = the nominal shear strength provided by the shear reinforcement

$V_s = A_v \times f_v \times d / s$

Shear strength provided by concrete

$v_c = 0.083 \times b \times \sqrt{f'_c} = 0.813Mpa$

The shear strength carried by concrete,

$V_c = v_c \times b_w \times d = 0.813Mpa$

Shear strength provided by shear reinforcement

Where the factored shear force, V_u, exceeds shear strength $ØV_c$, shear reinforcement shall be provided to satisfy the equation, $V_u \leq ØV_n = Ø \;\; V_c + V_s$

When shear reinforcement perpendicular to the axis of the member is used

$$V_s = \frac{V_u}{Ø} - V_c = A_v \times f_y \times \frac{d}{s}$$

Therefore, spacing of shear reinforcement, s, is:

$$S = A_v \times f_y \times \frac{d}{Vs} = A_v \times f_y \times \frac{d}{\frac{V_u}{Ø} - V_c} = A_v \times f_y \times \frac{d}{\frac{V_u}{Ø} - 00.742 \times b_w \times d}$$

Minimum shear reinforcement (Eq.12.34)

$$A_v = 0.083 \times \sqrt{f'_c} \times b_w \times S \;\; f_y$$

$$S_{max} = \frac{A_v \times f_y}{0.083 \times \sqrt{f'_c} \times b_w}$$

$f'_c = 24MPa$ $\qquad$ $f_y = 300MPa$

$A_v = diam.12 = 226.195mm^2$

Maximum spacing of transverse shear reinforcement

If $V_u < 0.10 \times f'_c \times b_w \times d, \qquad S \leq 0.8d \leq 600mm$

If $V_u \geq 0.10 \times f'_c \times b_w \times d, \qquad S \leq 0.4d \leq 300mm$

Sections located less than a distance d from support may be designed for the same shear as that computed at a distance d

X (m)	V_x (KN)	
0.00	1029.88	
1.03	949.61	
2.55		(Vu at distance d)
2.06	869.33	
3.09	789.06	
4.12	708.78	
5.15	628.51	
6.18	548.23	
7.21	467.96	
8.24	387.68	
9.27	307.41	
10.30	227.13	

$D = 1.4m$ $\qquad f_y = 300Mpa$

$b_{eff} = 2.5m$ $\qquad f'_c = 24Mpa$

$b_w = 0.25m$ $\qquad A_v = \text{diam}.12 = 226.195mm^2$

$f = 0.9m$

x (m)	V_u (KN)	d (m)	$V_c = 0.813 b_w d \times 10^{-3}$ (KN)	$V_s = \frac{V_u}{Ø} - V_c$ (KN)	$0.10 \times f'_c \times b_w \times d$ (KN	S_{max1} (mm)	$S_{max2} = \frac{A_v \times f_y}{0.083 \times \sqrt{f'_c} \times b_w}$) (mm)	Spacing by analysis $S = A_v \times f_y \times \frac{d}{\frac{V_u}{Ø} - V_c}$ (mm)	Spacing provided (mm)
0.00	1029.88	1.283	260.77	883.54	769.8	300	668	99	80
1.03	949.61	1.283	260.77	794.35	769.8	300	668	110	80
2.55	949.61	1.216	247.05	808.07	729.3	300	668	102	80
2.06	869.33	1.216	247.05	718.87	729.3	300	668	115	100
3.09	789.06	1.201	244.06	632.67	720.5	300	668	129	100
4.12	708.78	1.201	244.06	543.47	720.5	600	668	150	100
5.15	628.51	1.201	244.06	454.28	720.5	600	668	179	150
6.18	548.23	1.234	250.81	358.34	740.4	600	668	234	150
7.21	467.96	1.234	250.81	269.14	740.4	600	668	311	300
8.24	387.68	1.258	255.63	175.13	754.6	600	668	487	300
9.27	307.41	1.258	255.63	85.93	754.6	600	668	993	300
10.30	227.13	1.258	255.63	-	754.6	600	668	-	300

4.3.3 Deflection

a) Computation of Gross Moment of Inertia

Centroid of cross section for exterior girder

For simplicity of calculations, the slab surface is assumed level, i.e., without cross fall.

The center of gravity is calculated from bottom of girder.

Centroid of area

$$y_1 = \text{sum } A_i \times \frac{y_i}{A_i} = 0.674\text{m}$$

$\text{Span} = 20.6$ $\quad d_w = 0.25$

$a = 2.4$ $\quad D = 1.4$

$c = 0$ $\quad T_{top} = 0.22$

$w = 0.25$ $\quad T_{bottom} = 0.2$

Part	Area, A_i (m^2)	Centroid, y_i (m)	$A_i y_i$ (m^3)
Top Slab	0.26	1.29	0.34056
Bottom slab	0.29	0.10	0.029
Girder	0.25	0.69	0.169
Sum	0.80		0.54

Gross moment of Inertia for exterior girder

Part	Area, A_i (m^2)	y_i (m)	$ycg = y_i - y_1$ (m)	I_g (m^4)	$A_i \times (ycg)^2$ (m^3)	$I_g + A_i \times ycg^2$ (m^4)
Top Slab	0.26	1.29	0.62	0.002218	0.100	0.1024
Bottom slab	0.29	0.10	0.57	0.000967	0.096	0.0965
Girder	0.25	0.69	0.02	0.019608	0.000	0.0197
					Sum $\mathbf{I_g}$ =	$\mathbf{0.2186m^4}$

b) Computation of Effective Moment of Inertia

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \times \mathbf{I_g} + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq \mathbf{I_g}$$

$$f_r = 0.63 \times \sqrt{f'_c} = 3.086 N/mm^2$$

$$M_{cr} = f_r \times \frac{I_g}{y_t} = 1000.759 KNm$$

Weight of superstructure

Part	W (KN)	W (KN/m)
Top Slab	197.68	9.32
Bottom Slab	217.15	10.24
Girder	183.46	8.65
Sum W (KN/m)		28.22

$$M_a \text{ Total Slab} = W \times \frac{L^2}{8} = 1585.472$$

$$MDl._{ext} = W \times \frac{L^2}{8} = 1804.6$$

$$M_a = 1804.63KNM$$

$k_d = 0.443m$ previous calculation $d' = 0.142m$

$b_{eff} = 2.5m$ $D = 1.4m$

$b_w = 0.25m$ $h_f = 0.22m$

$n = 8$ $j = 0.922m$

$A_s = 11259.47mm^2$ $d = 1.258m$

Centroid of cross section (y_1 measured from bottom)

Part	Area, A_i (m^2)	y_i (m)	$A_i y_i$ (m3)
Slab	0.26	1.29	0.34056
Girder	0.25	0.69	0.169
Transformed steel, nA_s	0.09	0.14	0.013
Sum	0.60		0.52

$$y_1 = \frac{A_i \times y_i}{A_i} = 0.872m$$

Computation of Moment of Inertia of Cracked section, Icr

Part	Area, A_i m^2	$ycg = y_i - y_1$ (m)	Icg (m^4)	$A_i \times (ycg)^2$ (m^3)	$I_g + A_i \times ycg^2$ (m^4)
Slab	0.26	0.4179	0.002218	0.0461	0.0483
Girder	0.25	0.1821	0.019608	0.0081	0.0277
Transformed steel, nA_s	0.09	0.7298	-	0.0480	0.0480
				Sum =	0.1240

Effective moment of Inertia, Ie

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \times I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} = 0.134m^4 \leq I_g = 0.219m^4$$

Computation of Dead Load Deflection using effective moment of Inertia, I_e

The maximum deflection is calculated by using the formula;

$$\text{Deflection, max.} = 5 \times W \times \frac{L^4}{384 \times E \times I_e} = 22.980mm$$

$E = E_c = 24768Mpa$ $\qquad$ $W = 34.021KN/m$

$\text{Long term deflection} = \text{Instantaneous deflection} \times \text{factor}$

$L = 20.6m$

$\text{Factor} = 3 - 1.2 \times \frac{A_s'}{A_s} = 3$ $\qquad$ $A_s' = 0$

$\text{Long term deflection} = 68.940mm$

5. Steel Reinforced Elastic Bearings

Dimensions:

$L = 250$ Length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis

$W = 350\text{mm}$ Width of the bearing in the transverse direction

$h_r = 13\text{mm}$ Thickness of thickest elastomeric layer in elastomeric bearing

$h_s = 4\text{mm}$ Thickness of the steel reinforcement

$h_{rt} = 31\text{mm}$ Total elastomeric thickness

$n = 2\text{mm}$ Number of interior layers of elastomer

Shape factor for the thickest elastomeric layer: (for rectangular bearings without holes)

$$S = \frac{LW}{2h_r \ L + W} = 5609$$

Material Properties:

$G = 1.2$ Shear modulus of the elastomer

$F_y = 400\text{ MPa}$ Yield strength of steel reinforcement

$f'_c = 24\text{ MPa}$ Concrete compressive strength

Loads and Rotations:

$V_s = 679.406$ Maximum total vertical load at the service limit state

$V_L = 309\text{ KN}$ Maximum live vertical load at the service limit state

$H_s = 0\text{ KN}$ Maximum total horisontal load at the service limit state

$V_{smin} = 370\text{ KN}$ Minimum total vertical load at the service limit state

$D_s = 45\text{ KN}$ Maximum shear deformation of the elastomeric at the service limit state

$q_s = 0\text{ mm}$ Rotation about any axis of the pad

$B = 250.0\text{ rad}$ Length of pad if rotation is about transverse axis or width of pad if rotation is about its longitudinal axis

Design Requirements

Compressive Stress:

$s_s -$ Service average compressive stress due to the total load

$s_L -$ Service average compressive stress due to live load

$s_{Sd} -$ Service average compressive stress due to the total vertical load

$s_{Ld} -$ Service average compressive stress due to vertical live load

Minimum bearing surface area, from maximum limit of the compressive stress i.e. 11.0MPa

$$A_{r1} = \frac{V_s}{s_s \text{ allowable}}$$

$$A_{r2} = \frac{V_L}{s_L}$$

$\max(A_{r1}, A_{r2}) = 69581.04086 \text{mm}^2$

assuming $L = 250\text{mm}$ $W = 350\text{mm}$

$A_a = 87500\text{mm}^2$ OK!

condition $=$ "OK!" if $A_r < A_a$ "Not OK!" otherwise

Average compressive stress due to the total load:

$s_s = 1.66GS = 11.173$

$s_s = s_s$ if $s_s \le 11.0\text{MPa}$ otherwise take 11.0MPa

thus $s_s = 11.0\text{MPa}$

$$s_{Sd} = \frac{V_s}{LW} = 7.765 \quad \text{OK!}$$

condition $=$ "OK!" if $s_{Sd} \le s_s$

"Not OK!" otherwise

Average compressive stress due to live load:

$s_L = 0.66GS = 4.442$

$s_{Sd} = \dfrac{V_L}{LW} = 3.533$ OK!

condition $=$ "OK!" if $s_{Sd} \leq s_L$

"Not OK!" otherwise

Compressive Deflection:

$d = S_{ei} h_{ri}$ Instantaneous deflection

$e =$ from fig 8.4 Instantaneous compressive strain

$$s = \frac{V_s - V_L}{LW} = 4.232$$

$$e = 0.045\%$$

$$d = e\, h_r\, n = 1.17 \quad \text{OK!}$$

$$\text{condition} = \text{"OK!" if } d \leq 3\text{ mm}$$

$$\text{"Not OK!" otherwise}$$

Shear Deformation:

$D_s =$ Maximum shear deformation of the elastomer at the service limit state

$D_{ts} = 4.1796$ Deformation of the elastomer due to temperature change

$a = 10.8 \text{ x } 10^{-6}/o_c$ Coefficient of thermal expansion for normal density concrete

$DT^0 = 15C$ Assumed daily range of temp for the place

$s_L = 25.8m$ Total length of supperstructure

The bearing should satisfy: $2D_s \leq h_{rt}$

The horizontal movement of the superstructure shall be taken as the extreme displacement caused by creep, shrinkage, and post-tensioning, combined with thermal effects.

Calculation of thermal effect:

$$D_{ts} = a \times DT^{0} \times L = 4.1796$$

$$\text{Assuming } D_{ts} = D_s,$$

$$2D_s = 8.3592$$

Thus minimum thickness of elastomer equals to 9

Decide h_{rt} and number of internal layers below using manufacturer's table

Use 31mm elastomer thicknesses, (* it is thicker than the deflection requirement to satisfy uplift requirements) 2 internal 13layers, Plus top and bottom cover layers of 2.5 thick each.   OK!

$$\text{Condition} = \text{"OK!"} \quad \text{if top bottom thickness}$$

$$\leq 0.70 \times \text{internal elastomer thickness} = 9\text{mm}$$

"Not OK!" otherwise

Combined Compression and Rotation:

Rectangular bearings may be taken to satisfy uplift requirements if they satisfy:

$$s_s = 1.0GS \ \frac{q_s}{n} \ \frac{B}{h_r}^{2} = 6.2231$$

$$s_{Sd} = \frac{V_s}{LW} = 7.765 \qquad \text{OK!}$$

$$\text{condition} = \text{"OK!"} \quad \text{if } s_{Sd} > s_s$$

"Not OK!" otherwise

(These provisions ensures that no point in the bearing undergoes net uplift)

Rectangular bearings subjected to shear deformation should also satisfy:

$$s_s = 1.875GS\left(1 - 0.20\left(\frac{q_s}{n}\right)\left(\frac{B}{h_r}\right)^2\right) = 10.287$$

$$s_{Sd} = \frac{V_s}{LW} = 7.765 \qquad \text{OK!}$$

$$\text{condition} = \text{"OK!"} \ \text{if } s_{Sd} > s_s$$

"Not OK!" otherwise

(These provisions prevent excessive compressive stress on an edge)

Stability of Elastomeric Bearings:

Define:

$$A = \frac{1.92h_{rt} / L}{S\sqrt{(1 + 2.0L/W)}} = 0.0272$$

$$B = \frac{2.67}{S\left(S + 2.0\right)\left(1 + \frac{L}{4W}\right)} = 0.0531$$

* For rectangular bearing where L is greater than W, stability shall be investigated by interchanging L and W in Equations.

For bridge deck fixed against horizontal translation:

$$s_s = \frac{G}{2A - B} = 861.587$$

$$s_{Sd} = \frac{V_s}{LW} = 7.765 \qquad \text{OK!}$$

$$\text{condition} = \text{"OK!"} \ \text{if } s_{Sd} < s_s$$

"Not OK!" otherwise

Reinforcement of Bearings:

$\mathrm{DFTH} = 165$ Constant amplitude fatigue threshold

$\mathrm{Fy} = 400\ \mathrm{MPa}$ Yield strength of steel reinforcement

- At the service limit state

$$h_{ss} = \frac{3.0 h_r s_s}{F_Y} = 0.757 \quad \text{(too thin for manufacturing)}$$

$$h_s = 4 \text{OK!}$$

$$\text{condition} = \text{"OK!"} \ \text{if } h_{ss} < h_s$$

"Not OK!" otherwise

- At the fatigue limit state

$$h_{ss} = \frac{2.0 h_r s_s}{DF_{TH}} = 1.835$$

$$h_s = 4 \qquad \text{OK!}$$

$$\text{condition} = \text{"OK!"} \ \text{if } h_{ss} < h_s$$

"Not OK!" otherwise

Summary:

- Plan dimensions: $L = 250\mathrm{mm}$ and $W = 350\mathrm{mm}$

- Thickness:

Reinforcements

3 steel plates having 4mm thickness each.

Elastomer

2 internal layers having 13 mm thickness each.

2 cover layers 3 mm thickness each.

Total Steel Reinforced Elastomeric Bearing Thickness : 43 mm

6. Abutment Design Data and Specification

6.1. Foundation material

The foundation material at the bridge site is slightly weathered rock. (HEC) PLC, 2010, Grand Palace – Shiromeda Road Project Soil extension Study.

The allowable bearing capacity estimated is 600 Kpa.

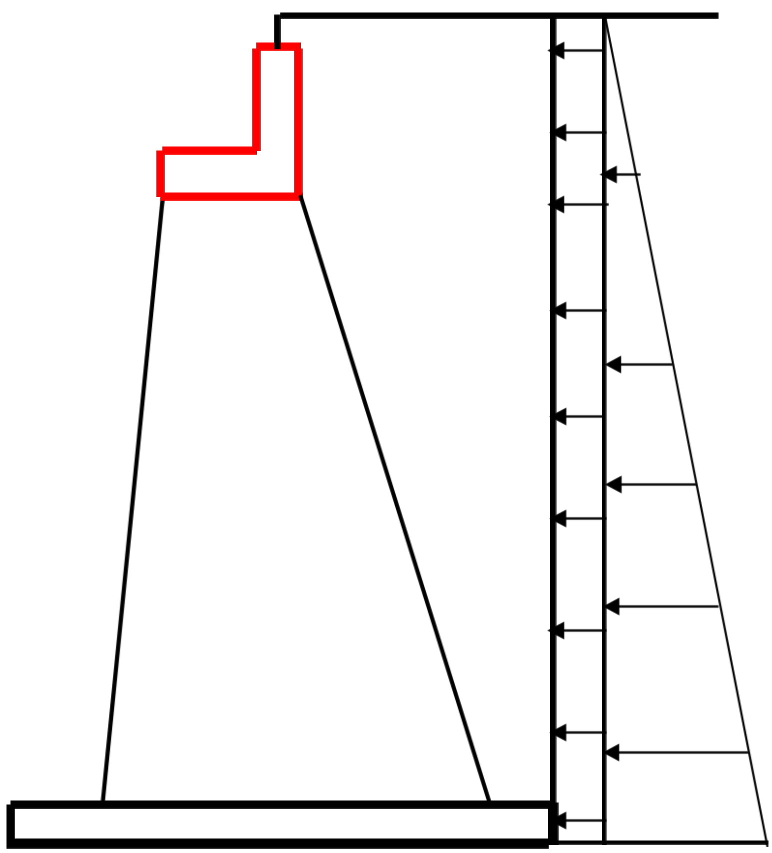
6.2. Backfill Material

Unit weight of material from the laboratory tests the maximum dry density at optimum moisture content (OMC) is calculated to be: $\gamma = 18.3\mathrm{KN/m^3}$

Angle of internal friction $\emptyset = 30\mathrm{deg}$

6.3. Abutment analysis

Computation of active earth pressure



6.4. Rankine's coefficient of active earth pressure

The calculation of active earth pressure based on Rankine's formula is given as thus: Coefficient of active earth pressure using Rankine method is

$$K_a = \frac{1 - \sin\Phi}{1 + \sin\Phi} = 0.333$$

The total active pressure at either Footing or Abutment level is given by

$$P_a = \frac{K_a \times g \times h^2}{2}$$

Where

P_a = Total active pressure at either Footing or Abutment Level

g = Bulk unit weight of the backfill

h = Height to the Abutment or Footing level

6.5. Surcharge pressure

Where traffic load can approach the substructure within a horizontal distance from the top of the wall equal to one-half the wall height, the lateral pressure for design shall be increased by a minimum surcharge acting on the back slope equivalent to 2ft (0.61m) (Hs) of soil having a resultant magnitude of:

$$Ps = K_a \times g \times Hs = 3.721$$

6. 6. Analysis Criteria

In designing the substructure units, the following failure conditions are checked:

(i) Bearing failure

The location of the bearing pressure resultant (R) on the base of the foundation should be within the middle third i. B/6 of the centre of the foundation if soil or B/4 if foundation is rock.

$$\text{Where Qmax} \;=\; V \times \frac{1 + \frac{6e}{B}}{B}$$

$$\text{And} \quad \text{Qmin} \;=\; V \times \frac{1 - \frac{6e}{B}}{B}$$

Also

$$e \;=\; \frac{B}{2} - X_r$$

$$X_r \;=\; \frac{M}{V}$$

$$M \;=\; M_r - M_o$$

The above criteria could include the provision of R.C. base so that the acting pressure is below the allowable at the founding level.

In addition to the bearing failure checking, gravity and semi gravity walls shall be dimensioned to ensure stability against possible failure modes by satisfying the following factors of safety (FS) (un factored DL & LL shall be used to determine the FS against sliding and overturning).

In determining the FS, the effect of passive soil pressure resistance in front of a wall should only be considered when competent soil or rock exists which will not be removed or eroded during structural life.

(ii) Overturning

Factor of safety: For footing on rock $\quad$ Fos $\geq$ 1.5

For footing on soil $\quad$ Fos $\geq$ 2.0

$$\text{Where } F.so = \varepsilon Mr / \varepsilon Mo$$

$$M_r = \text{Resisting Moments}$$

$$M_o = \text{Overturning Moments}$$

(iii) Sliding

$$\text{Factor of safety : } FSs \geq 1.5$$

$$\text{Where } FSs = \varepsilon V \tan \Phi / \varepsilon H$$

$$V = \text{vertical resisting force}$$

$$H = \text{Disturbing force}$$

ABUTMENT DESIGN DATA

1 Dimensions Data

(a) Superstructure

Item	Dimension
No. of Lanes	8
Top width of roadway	30m
Curb width (top)	25m
Curb width (bottom)	25m
Curb height	0.25m
Clear span	20m (normal direction)
Effective span	20.6m
Grade elevation	2470.074m
Slab thickness	0.22m
Total Slab width	55m

(b) Substructure-Abutment

Height of Abutment (H) = 3.9m

Top width of Abutment = 0.8m

Bottom width of Abutment = 3.29m

Backfill Height = 5.56m

Girder Seat height = 0.3m

Girder Seat width = 0.5m

Back wall height = 1.4m

Back wall width = 0.3m

Back slope of Abutment (H: V) = 12.5

Front Slope of Abutment (H: V) = 110

Surcharge height = 0.9m

Footing level = 2464.174

Depth of footing = 0.3m

2. Material Properties

Dry density = 1.83gm/cc

Optimum moisture content (as %) = 6

Unit Weight (Bulk density) = 19.398KN/m3

Angle of Internal Friction, Φ = 30o

Angle of Inclination β = 0o

Angle of wall friction, δ = 23.33o

Type of Foundation material = Rock

Working Bearing Capacity = 600KN/m2

Unit Weight of Concrete = 25KN/m3

Unit Weight of Masonry = 27KN/m3

3. Earth Pressures

3.1 Rankines Method

(a) Active Earth Pressure (Pa-R)

$$\mathrm{Ka} = 0.333$$

$$\text{Active Pressure at Footing level (Pa} - \text{R)} = 101.390\mathrm{KN/m2}$$

$$\text{Active Pressure at Abutment level (Pa} - \text{R)} = 89.453\mathrm{KN/m2}$$

(b) Surcharge Pressure (Pu)

(i) Footing Level

Surcharge

$\text{Live Load Surcharge (Pull)} = 32.357$ $\qquad$ $\mathrm{He(m)} = 0.61$

$\text{Dead Load Surcharge (Pudl)} = 0$ $\qquad$ $\mathrm{He(m)} = 0$

(i) Abutment Level

$\text{Live Load Surcharge (Pull)} = 0$

$\text{Dead Load Surcharge (Pudl)} = 0$

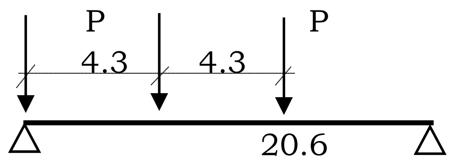
4. Distributed Unfactored Loads from Superstructure

a) Dead Load

Girder	716	KN
Slab	573	KN
Curb	43	KN
Railing	33.6	KN
DL/LM	153.094	KN/m

$\text{Distributed Dead load} = 153.09\mathrm{KN/m}\text{(Total loads)}$

b) Live Load



Max. Live load Reaction will be when one of the wheel loads is placed at one of the supports

$$\mathrm{Ra} = \frac{P}{4}\ 20.6 - 2 \times 4.3\ \ + P\ 20.6 - 4.3\ \ + \frac{p\ 20.6}{20.6} = \ 1.946 \times P$$

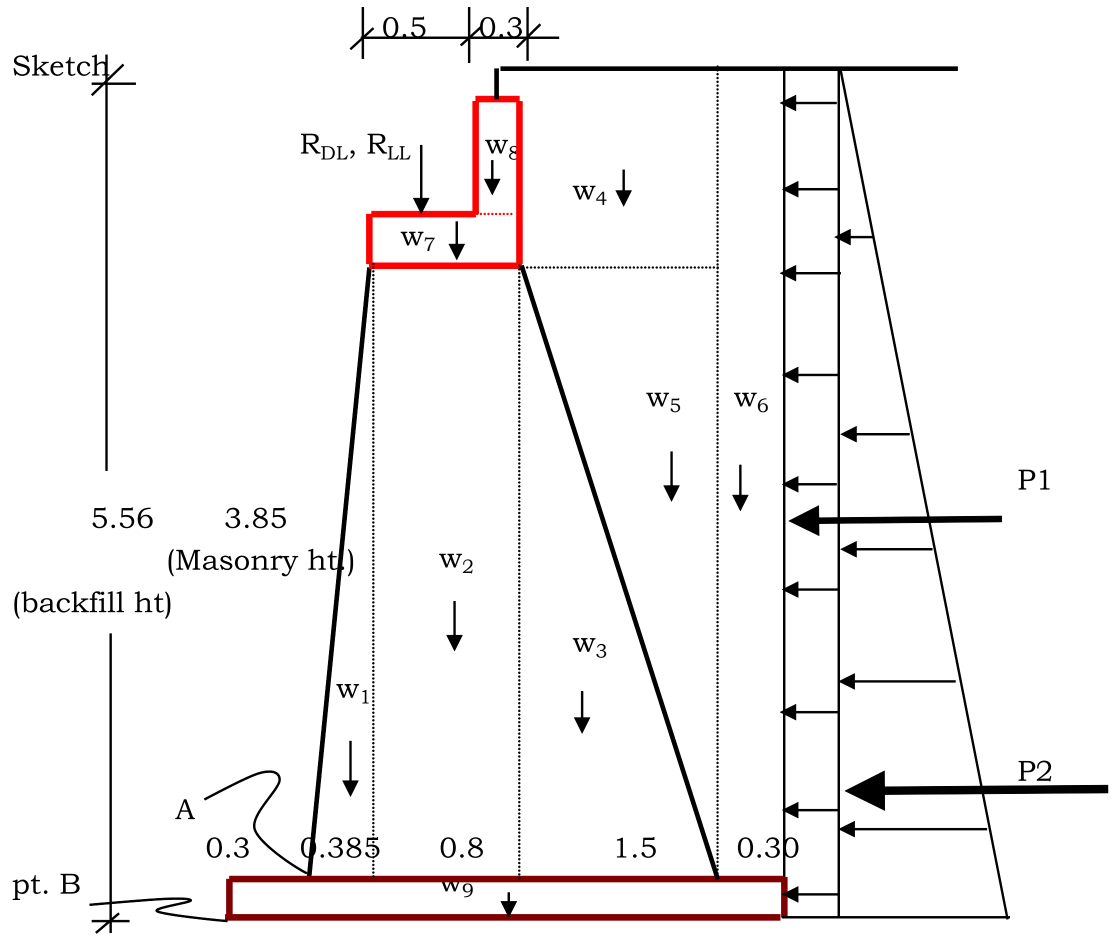
Considering two trucks for the critical case: $P = 2 \times 144KN$

Distributed Live Load Reaction $PLL\ =\ 1.946 \times 144 \times 2/8.92$

$PLL\ =\ 62.960KN/m$

Distributed Live load $= 62.960KN/m$(Total loads)

ABUTMENT ANALYSIS



Loads acting/ Forces and Bending Moment

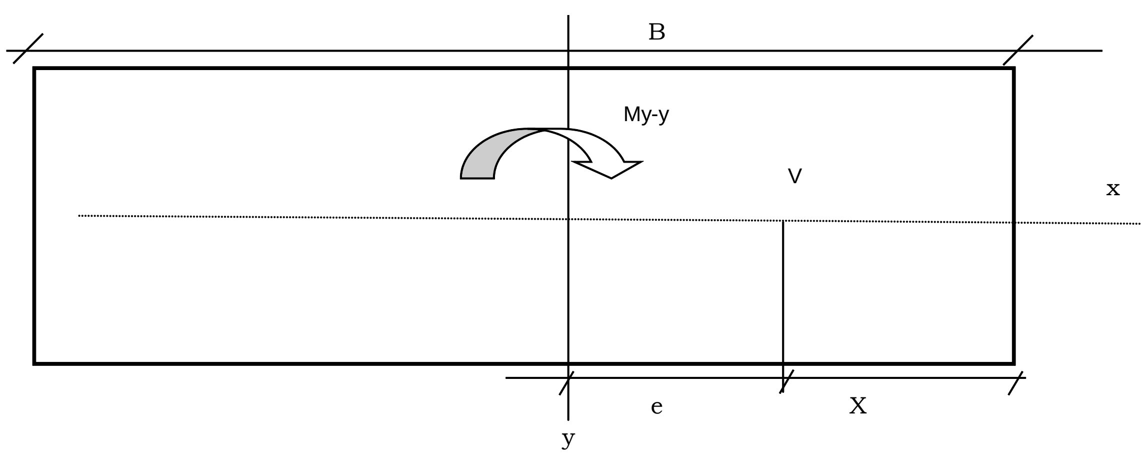
About base of abutment (A)

Load Type	Loading	Item	Force	Arm Length	Moment
		w_1	20.010	0.257	5.136
	DL	w_2	83.160	0.785	65.281
Vertical	Abutment	w_3	77.963	1.685	131.367
	& Backfill	w_4	41.027	1.935	79.387
Loads		w_5	56.012	2.185	122.386
		w_6	0.000	0.000	0.000
		w_7	6.000	0.785	4.710
		w_8	6.600	1.035	6.831
		w_9	0.000	0.000	0.000
		Total	290.771		415.097
	Loads	DL	153.094	0.735	112.524
	from	LL	62.960	0.735	46.275
	superstructure		216.054		158.800
Lateral	LL surcharge	P_1	20.748	2.380	49.379
Pressure	Lateral pres	P_2	89.453	1.587	141.931

About base of footing (B)

Load Type	Loading	Item	Load Value	Lever Arm	Moments
		w_1	20.010	0.557	11.139
	DL	w_2	83.160	1.085	90.229
Vertical	Abutment	w_3	77.963	1.985	154.756
	& Backfill	w_4	41.027	2.235	91.695
Loads		w_5	56.012	2.485	139.189
		w_6	27.700	3.135	86.841
		w_7	6.000	1.085	6.510
		w_8	6.600	1.335	8.811
		w_9	24.638	1.643	40.467
		Total	343.109		629.636
	Loads	DL	153.094	1.035	158.452
	from	LL	62.960	1.035	65.163
	superstructure		216.054		223.616
Lateral Pressure	LL surcharge	P_1	21.931	2.780	60.968
	Lateral pres	P_2	99.947	1.853	185.236

Stability against Bearing



Case 1) Abutment with backfill only

About Pt.	H(m)	B(m)	M	V	Xr	e(m)	B/4	Check	Qmax	Check
A	3.85	2.685	273.17	290.77	0.94	0.40	0.67	OK	205.83	OK
B	4.15	3.29	444.40	343.11	1.30	0.35	0.82	OK	170.70	OK

Footing on soil Footing on rock

e <= B/6 e <= B/4

Case 2) Abutment with backfill and Dead Load only

About Pt.	H(m)	B(m)	M	V	Xr	e(m)	B/4	Check	Qmax	Check
A	3.85	2.685	385.69	353.73	1.09	0.25	0.67	OK	205.98	OK
B	4.15	3.29	509.56	496.20	1.03	0.62	0.82	OK	320.88	OK

Case 3) Abutment with backfill, Dead Load and Live Load

About Pt.	H(m)	B(m)	M	V	Xr	e(m)	B/4	Check	Qmax	Check
A	3.85	2.685	382.59	506.83	0.75	0.59	0.67	OK	436.63	OK
B	4.15	3.29	607.05	559.16	1.09	0.56	0.82	OK	343.34	OK

Stability against Sliding

Case 1) Abutment with backfill only

Angle of Friction a = 35

Point Name	H(m)	B(m)	F_H	F_V	F_V*tana	F.Ss	Check
A	3.85	2.685	89.45	290.77	203.47	2.27	OK
B	4.15	3.29	99.95	343.11	240.09	2.40	OK

Case 2) Abutment with backfill and Dead Load only

Point Name	H(m)	B(m)	F_H	F_V	F_V*tana	F.Ss	Check
A	3.85	2.685	89.45	443.87	310.59	3.47	OK
B	4.15	3.29	99.95	496.20	347.22	3.47	OK

Case 3) Abutment with backfill, Dead Load and Live Load

Point Name	H(m)	B(m)	F_H	F_V	F_V*tana	F.Ss	Check
A	3.85	2.685	110.20	573.90	401.58	3.64	OK
B	4.15	3.29	121.88	559.16	391.27	3.21	OK

Stability against Overturning

Factor of safety: For footing on rock   F.os ≥ 1

For footing on soil   F.os ≥ 2.0

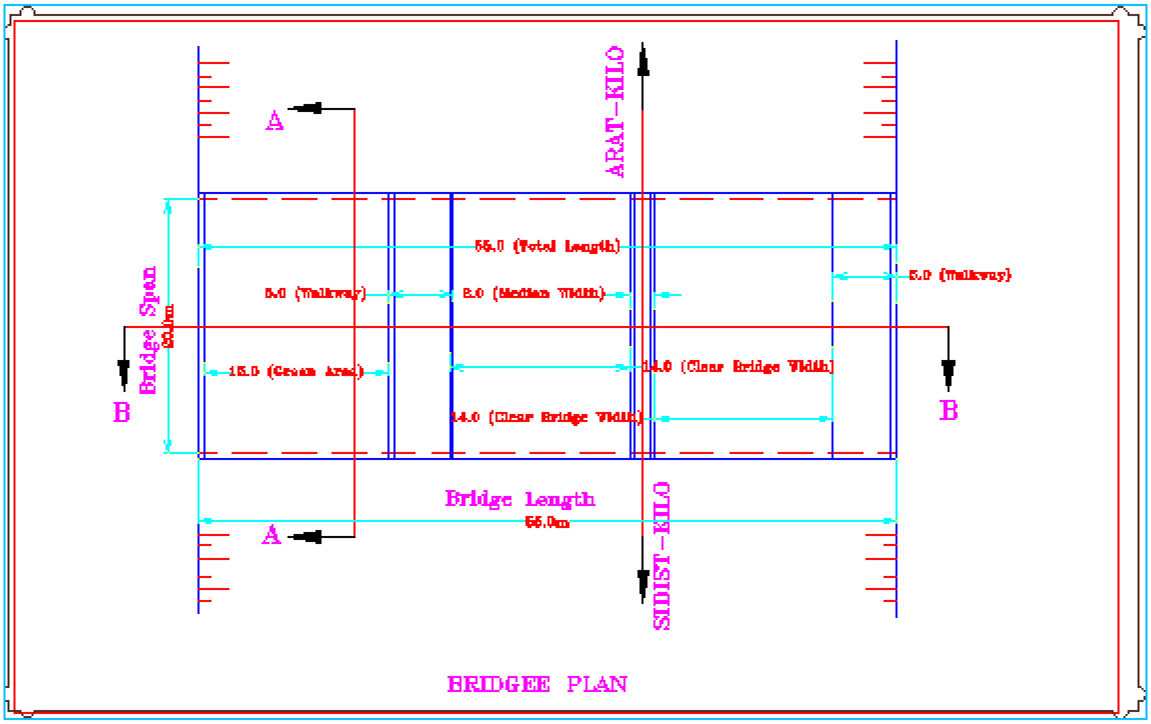
Point Name	H(m)	B(m)	Mr	Mo	F.So	Check
A	3.85	2.69	415.10	141.93	2.92	OK
B	4.15	3.29	629.64	185.24	3.40	OK

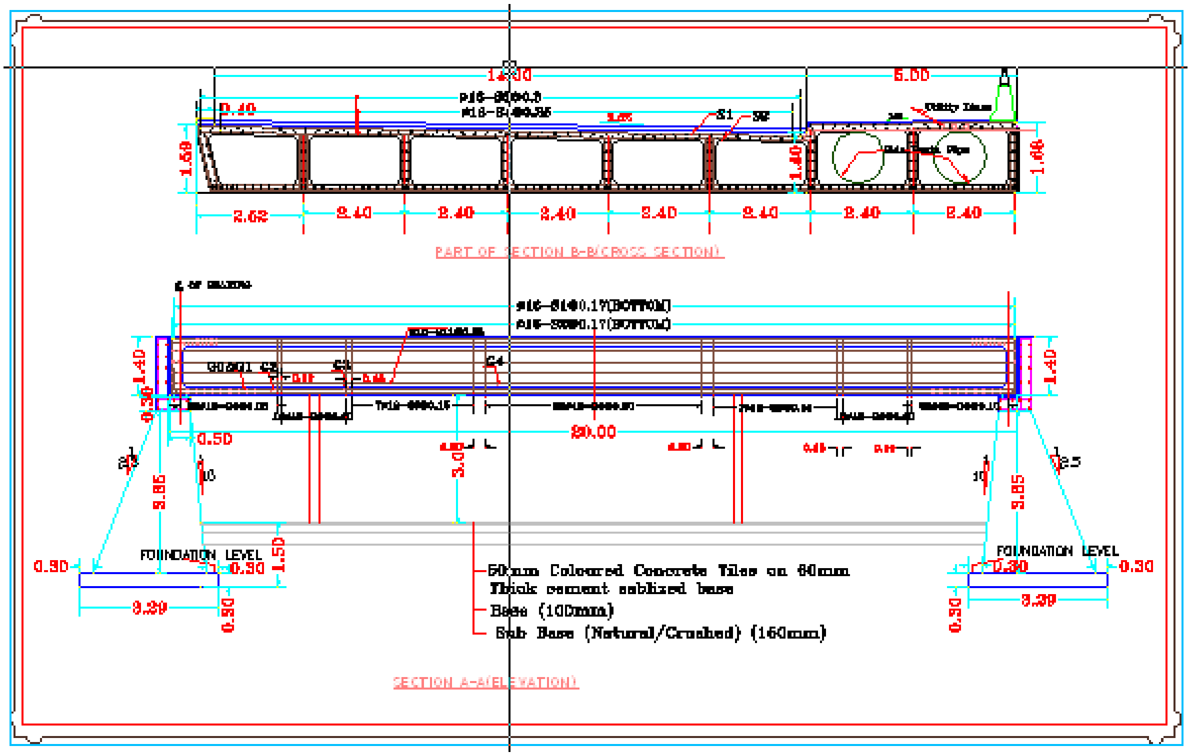
Case 2) Abutment with backfill and Dead Load only

Point Name	H(m)	B(m)	Mr	Mo	F.So	Check
A	3.85	2.69	527.62	141.93	3.72	OK
B	4.15	3.29	788.09	185.24	4.25	OK

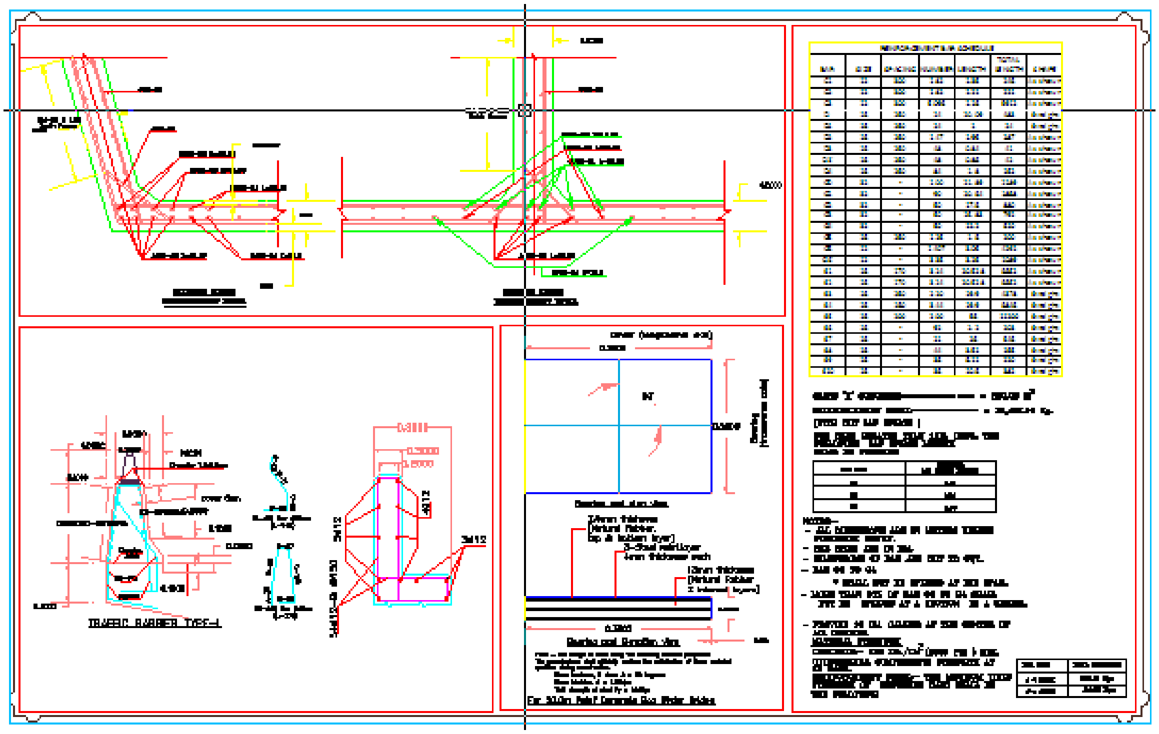
Case 3) Abutment with backfill, Dead Load and Live Load

Point Name	H(m)	B(m)	Mr	Mo	F.So	Check
A	3.85	2.69	573.90	191.31	3.00	OK
B	4.15	3.29	853.25	246.20	3.47	OK


ARAT–KILO
A
55.0 (Total Length)
5.0 (Walkway)
Bridge Span
15.0 (Green Area)
14.0 (Clear Bridge Width)
14.0 (Clear Bridge Width)
B
B
Bridge Length
55.0m
A
SIDIST–KILO
BRIDGEE PLAN



PART OF SECTION B-B(CROSS SECTION)
SECTION A-A(ELEVATION)
FOUNDATION LEVEL
50mm Coloured Concrete Tiles on 60mm
Thick cement sablized base
Base (100mm)
Sub Base (Natural/Crushed) (160mm)
20.00


TRAFFIC BARRIER TYPE-I

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