Abstract

In India generally the bridges are constructed with cast in situ construction technique. In the bridges prestressing technique is used frequently due to cost effectiveness. The prestressed construction is of two type post-tension and pre-tension. The concept of pretension prestressed construction system is used in construction of precast element. Different types of the bridge superstructure are constructed in India such as Slab, T-shape, I-shape, Box shape, Arch type etc. Depending upon the requirement of the bridge selection of superstructure has been done. In present scenario, aesthetic of the bridge structure is the important aspect in design of bridge. Generally, in India railway bridges are constructed with steel section. During the Delhi Metro project and Mumbai Metro project the concept of precast prestressed bridge superstructure with U-shaped has been implemented. The precast construction reduces time duration of project which result in saving in the cost. The precast girder has good aesthetic as compare to cast-insitu girder due to quality management. Due to the U shape of girder it will obstruct view of machines part of rail which result in good view of rail. The prestressed precast U-shaped girder may be give new dimension to bridge superstructure. The U-shaped girder has better scope as it can replace current trend of design of bridge with I,T or Box shaped girder.

In present study, the analysis and design of U-shaped girder for railway loading is done with specification of Concrete bridge rule under Indian Railway Standards. The parametric study of the superstructure for span with various depth for broad gauge and meter gauge loading is done and economical span to depth ratio is found out. The design sub structure is carried out which include design of bearing, piercap, pier, pilecap, pile. The drawing of super structure and sub structure is prepared.

Abbreviation Notation and Nomenclature

A .... Cross-sectional area of member

A_c .... Area of concrete section

A_{ct} .... Area of concrete in tension zone

A_{ps} .... Area of prestressing strands

A_s .... Area of non-prestressing tension reinforcement

E_c .... Modulus of elasticity of concrete

E_s .... Modulus of elasticity of steel

I .... Second moment of area of section

L .... Effective span

M_d .... Bending moment due to dead load

M_l .... Bending moment due to live load

M_{ult} .... Ultimate Bending moment

P .... Prestressing force

P_i .... Initial prestressing force

P_{eff} .... Effective prestressing force after losses

V_{ult} .... Ultimate shear force

V_{co} .... Shear strength of Uncracked section

V_{cr} .... Shear strength of Cracked section

Z_t .... Section modulus of top fiber of beam

Z_b .... Section modulus of bottom fiber of beam

e ...... Eccentricity of prestressing force with respect to centroid of concrete section

f_c .... Compressive stress

f_{ci} .... Compressive stress of concrete at initial transfer of prestress

f_{cp} .. Compressive stress of concrete at centroid axis due to prestress after all losses

f_{ct} .... Allow compressive stress in concrete at transfer of prestress

f_{cw} .... Allow compressive stress in concrete at service load

f_{ck} .... Characteristic cube strength of concrete

f_{pe} .... Effective prestress in strands

f_{pi} .... Initial stress in strands

f_{pu} .... Characteristic strength of prestressing strands

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Chapter 1

Introduction

1.1 General

Transportation is essential requirement for development of particular area. Transportation is mainly of two types highway and railway transportation. Bridge is key element in transportation. Bridge is consist of many components such as girder, bearing, piercap, pier, abutment, etc. The superstructure of bridge is supporting system of vehicular traffic as shown in Figure1.1. The superstructure of bridge decide cost of bridge and its construction methodology. There are many superstructure sections currently in use in the bridges such as solid slab, voided slab, T-girder, I-girder, Box girder, U-girder, etc as shown in Figure1.2.

The construction method has significant effect on the cost of bridge. Generally, in India the bridge is constructed with cast in-situ method. In present scenario the bridge is constructed with prestressed construction technique. In prestressed technique element is combination of high tensile steel to provide tensile strength and high performance concrete to provide compressive strength, which gave good combination for the super strucuture of the bridge. In the bridge quality is essential requirement, so the concept of precast construction came forward. In precast construction quality control can be maintain.

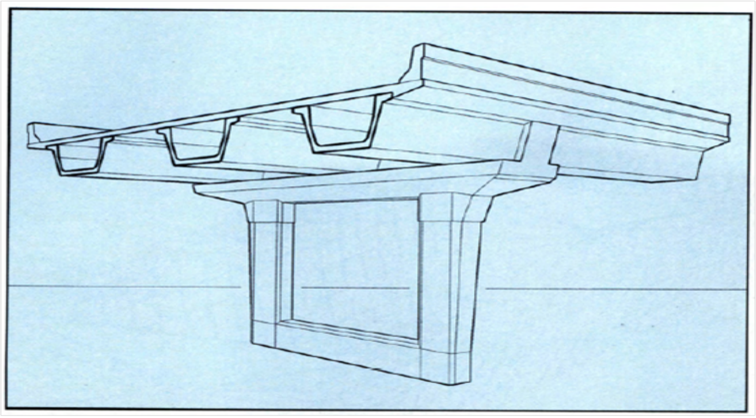

Figure 1.1: Superstructure of Bridge

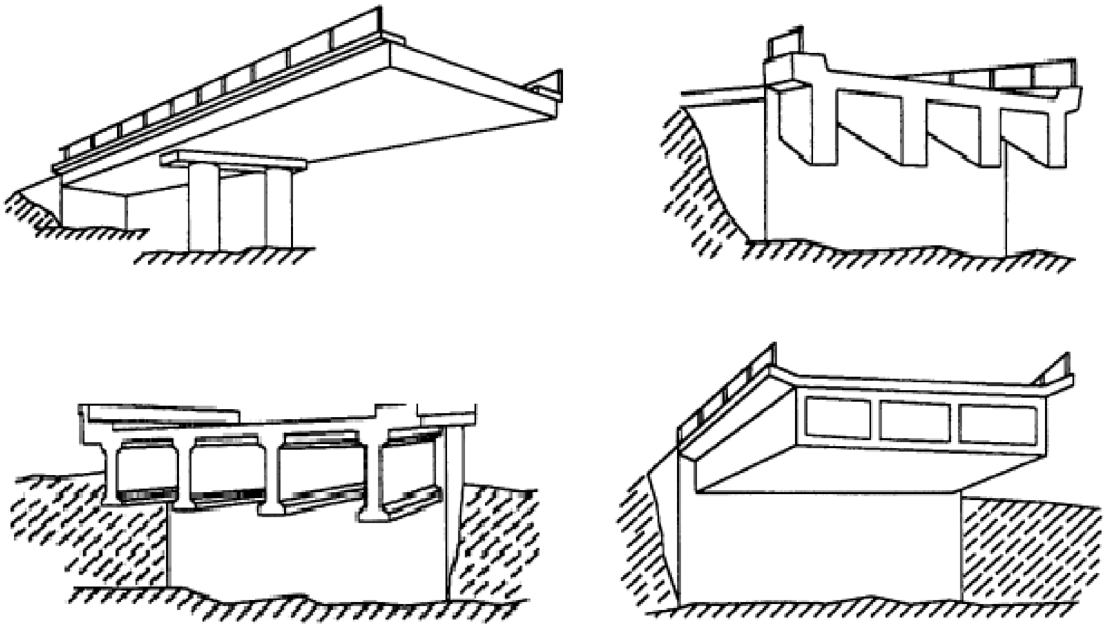

Figure 1.2: Solid Slab, T-girder, I-girder, Box girder

1.2 Principal of Prestressing System

The basic principle of prestressing system can be explained by example of blocks. When group of blocks joined together and allowed to deflect due to their own weight they deflect in downward direction, but when the external force is applied to blocks they came in their original condition as shown in Figure1.3. The prestressing of member can be achieved by transfer of force between prestressed strands and concrete. The strands are stretch and anchor against the concrete, that tension in strands result in the compression in the concrete. This externally applied compressive force is use to replace stresses develop under loading condition. The bottom fiber stress in non-prestressing member is tensile under application of load as shown in Figure1.4. When prestressing force is applied this tensile stress in bottom fiber is balance by compressive stress generate due to prestressing force as shown in Figure1.5.The strands can be placed as internal tendons with in section or as external tendons over the section. The strands may be unbonded or bonded to the concrete. The prestressing system can be pre-tensioned or post-tensioned.

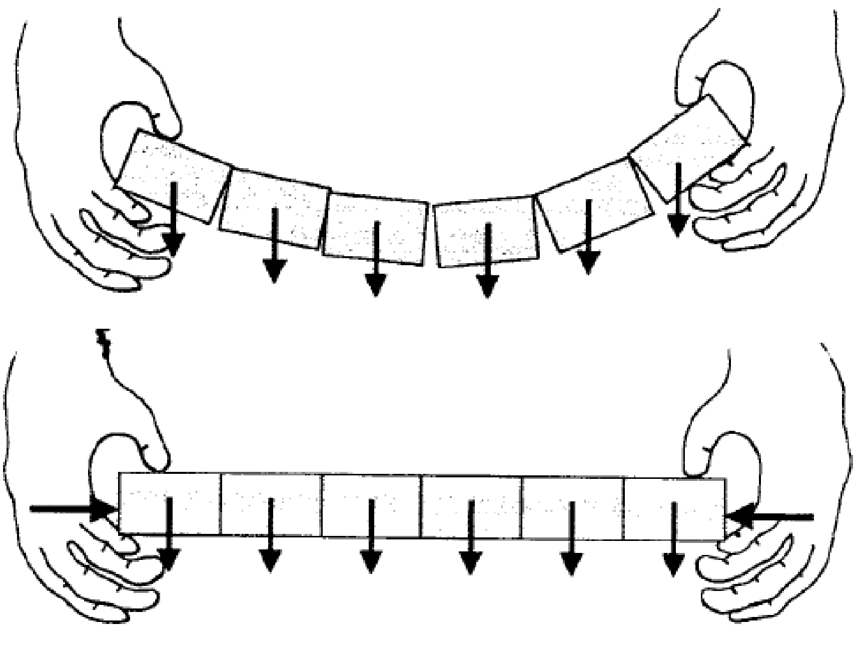

Figure 1.3: Prestressing Blocks

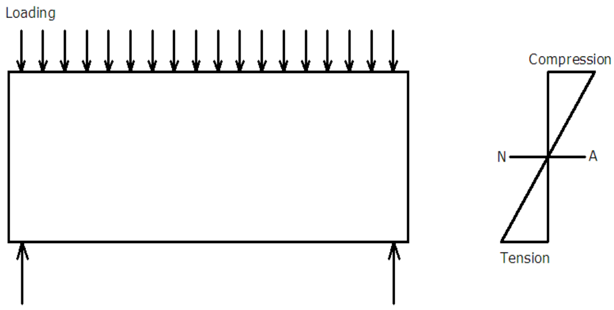

Figure 1.4: Non-Prestressed Section

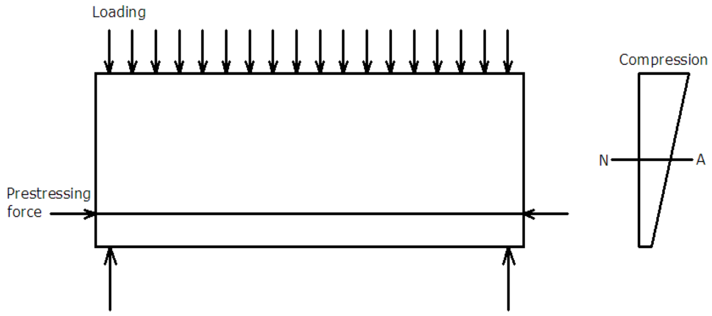

Figure 1.5: Prestressed Section

1.2.1 Post-Tension Prestressed

In post-tension the prestressing is applies to concrete where steel strands are tensioned against the concrete after the hardening of concrete.The typical arrangement for post-tension prestressed system is shown in Figure1.6

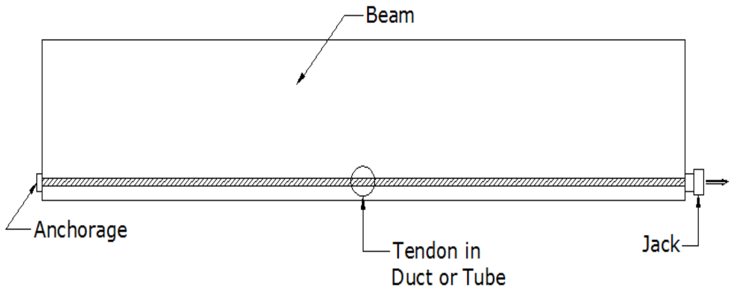

Figure 1.6: Typical arrangement for Post-tension Prestressing

1.2.2 Pre-Tension Prestresses

In pre-tension the prestressing is applies to concrete where steel strands or bars are tensioned against the concrete between abutments before placing of concrete. After hardening of concrete, force in steel is transferred to concrete by releasing the anchors at abutments. The prestressing force is transfer due to bond between concrete and steel. The typical arrangement for pre-tension prestressed system is shown in Figure1.7. Pretension prestressed concrete bridge decks generally made up of precast pretension units with cast in place deck slab for small and medium bridge. The precast prestressed I and T beam have been standardized for the use in construction of bridge decks.

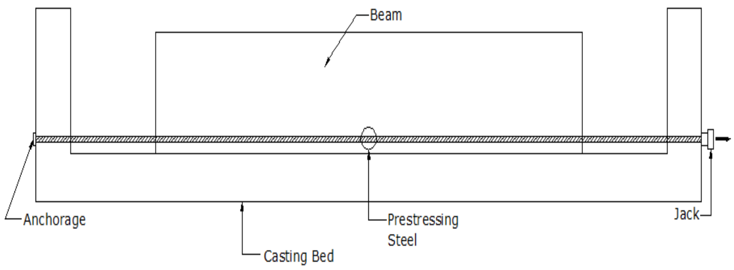

Figure 1.7: Typical arrangement for Pre-tension Prestressing

1.3 Historical Background

The aesthetics is a major design consideration in the construction of bridge. In Texas to achieve aesthetics as well as to maintain economy of precast prestressed girder the new shape was developed. The number of girder and the number of visual break lines have been reduced by replacing I-girder with open top section with sloping webs as shown in Figure1.8. This precast girder was used with cast in-situ slab on top for bridge superstructure.

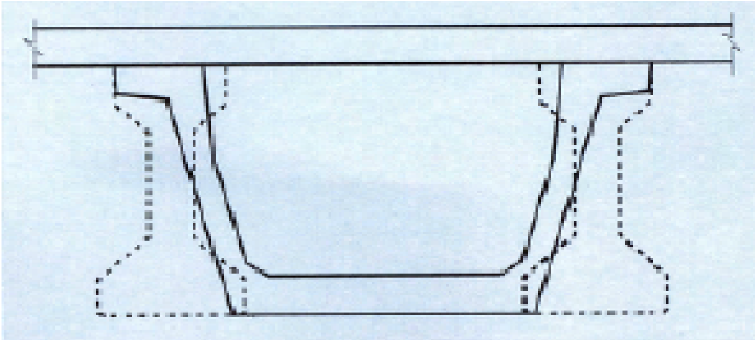

Figure 1.8: Comparison of I-shaped girder with U-shaped girder

In India the precast prestressed U-shaped girder was first time used during Delhi Metro Rail project. In DMR project girder was used with open at top and rail track support by bottom flange. The U-shaped has benefit that bottom part of rail machine is covered in shape and it shows good aesthetic view. In DMR project, girder for length of 25m without increase in thickness of section was cast as shown in Figure1.9.The precast prestressed U-shaped girder is patent shape by SYSTRA, France based company. The company in co-ordination with PB company do design for U-shape girder. The U-shaped girder has better future scope as it can replace current trend of design of bridge with I, T or box shaped girder.

Figure 1.9: Erection of U-shaped girder

1.3.1 Advantages over Conventional Section

The precast prestressed U shaped girder is open section and this has advantage over conventional girder as described below.

- It is cost effective as the weight of U shaped girder is 10t/m as compared to segmental box girder of 16t/m.
- The erection of girder is fast as it is precast girder.
- The aesthetic view of girder is good.
- Due to U shaped no additional side parapets are required.
- The bottom deck slab supports the rail so overall height of section is reduces.
- The section remains typical throughout span as it will not require increase in thickness at end of span.
- The erection of reinforcement cage is possible in faster way.
- The adjustment of reinforcement and formwork of section become easy.
- The lying of prestressing steel is easy as it is in straight profile.
- The erection and handling of full length inner formwork is easy.
- The availability of girder at time is more due to casting at casting yard.

1.4 Objective of Study

The main objective of this study is to understand the design of U shaped precast prestressed girder. The objective of project is:

- To understand analysis and design of U shaped precast prestressed girder for railway loading as per Indian Railway Standard.
- To study effect of the various depth of section with span in cost of superstructure. To evaluate economical span to depth ratio for the superstructure.

- To study design of substructure for girder.

1.5 Scope of Work

The scope of work is as follows:

- To study detail design procedure for precast pretension prestressed girder.
- Manual calculation of U-shaped precast prestressed girder for broad gauge rail traffic.
- Parametric study for various depth of section with span for the superstructure.
- Design of Substructure components.

1.6 Organization of Major Project

The content of major project is divided into different chapters as follows:
Chapter 1, represents an introduction and overview of the major project work. The Principal of prestressing, Historical back ground and advantages of U shaped section over conventional section. It also includes objectives of study and scope of work.
Chapter 2, In this chapter brief literature review pertaining to Use of U shaped section is presented.
Chapter 3, In this chapter explanation for analysis and design of U shaped girder for broad gauge rail loading according to Indian Railway Standard is described.
Chapter 4, represent parametric study with cost analysis for depth of section with span for superstructure.
Chapter 5, describes design of substructure for broad gauge rail loading.
Chapter 6, consists of summary, conclusions and possibility for future scope of work on basis of the work conducted in the Major project.

Chapter 2

Literature Review

2.1 General

Literature survey has been carried out for use of precast prestressed girder for various bridges and its design specification in various standards. The case study for use of U shaped girder with different purpose. The paper for Finite Element Analysis to use in analysis of bridge deck.

2.2 Literature Review

Various literatures have been studied for design of precast girder and brief review of which has been discussed below.

2.2.1 Case Studies

O.P.Singh, S.C.Gupta and A.Khare[1] presented a case study on Delhi Metro project construction using 25m long precast U girder. The paper discussed the benefits of use of precast member during construction process which helps in reduction of time duration of the project. The paper included the casting and erection process of precast girder and pile cap.

M.Singh, R.Kataria, A.Mhedden, S.Mohammad and P.Bajpai[2] discussed future of elevated metro viaducts with full span precast deck. The case study included the benefits of U shaped girder, precast member in construction. The case study discussed the construction of precast girder and erection of full span girder over site.

Mary Lou Ralls, Luis Ybanez and John J.Panak[3] discussed development of precast prestressed U-beam by Texas Department of Transportation. The paper included development of U-beam with design, production and construction aspects. The two type of U-beam U54 and U40 with different section and design properties. The U54 beam was suitable for length of 36.6m and the U40 beam was suitable for length of 27.4m with spacing of beam ranging from 4 to 4.9m. The U-beam was prestressed with 12.7mm diameter strands in bottom slab with straight profile. The U-beam was design for AASHTO loading specification.

Paulo J.S.Cruz and Dawid F.Wisniewski[4] discussed the construction of the bridge over the Ave river in Portugal with precast construction technology. The bridge superstructure was made up of U-beam with cast in-situ slab. The length of U-beam ranging from 20m to 30m. The deck of bridge was made with two precast prestressed U-beam with spacing of 7.5m from center to center. The U-beam had depth of section as 1.7m and thickness of bottom slab 240mm with increase in thickness where anchorage blocks were located. The width of bottom slab 2.2m. The thickness of web was 180mm.

2.2.2 Books

Krishna N. Raju[5] describes design of various types of bridge in his book D̈esign of Bridge". This book is useful in understanding design of prestresses girder and design of substructure. It is helpful in understanding component of bridge and their detailed drawing.

Krishna N. Raju[6] describes theory and design of prestressed concrete in his book P̈restress Concrete". This book is useful in understanding design procedure for pre-tension prestresses girder. The book explained calculation of losses, prestressing force, checks for design of pre-tension girder.

Nigel R. Hewson[7] describes the prestressed concrete bridge with design and construction aspects. The book give brief understanding to prestress technology and design concepts for prestressed bridges. The construction of prestressed bridge is also described with practical problem and their solution.

V. K. Raina[8] C̈oncrete bridge handbook" useful in understanding the design philosophy concepts of prestressed girder design. The book is also helpful in understanding component of substructure of bridge.

PCI Bridge Manual[9] P̈CI bridge manual" useful in understanding design of precast pre-tension prestressed bridge superstructure. The manual has good example for precast prestressed girder.

2.2.3 Standards

IRS Bridge Rules - Rules specifying the loads for design of superstructure and substructure of bridges[10] is useful in application of railway load on bridge.

IRS Concrete Bridge Rules - Code of practice for plain, reinforced and prestressed concrete for general bridge construction[11] is useful in design of prestressed girder and design of component of substructure.

IRC 83 (part-II) - Standard specification and code of practice for road bridge (Elastomeric bering) [12] is useful in design of the elastomeric bearing.

2.3 Summary

In this chapter, review of relevant literature is carried out. The review of literature includes various parameters related to design of pre-tension precast prestressed girder. The review helps in design of pre-tension prestressed girder and understanding of behavior of various parameter on design.

Chapter 3

Analysis and Design of U-shaped girder

3.1 General

Prestressed construction is ideally suited for long span bridges. The provisions for design of pre-tension prestressed girder is given in IRS Concrete Bridge Rule. A U-shaped section is analyzed and designed for broad gauge railway loading.

3.2 Preliminary Data

In present study the cross section taken for analysis is shown in Figure3.1

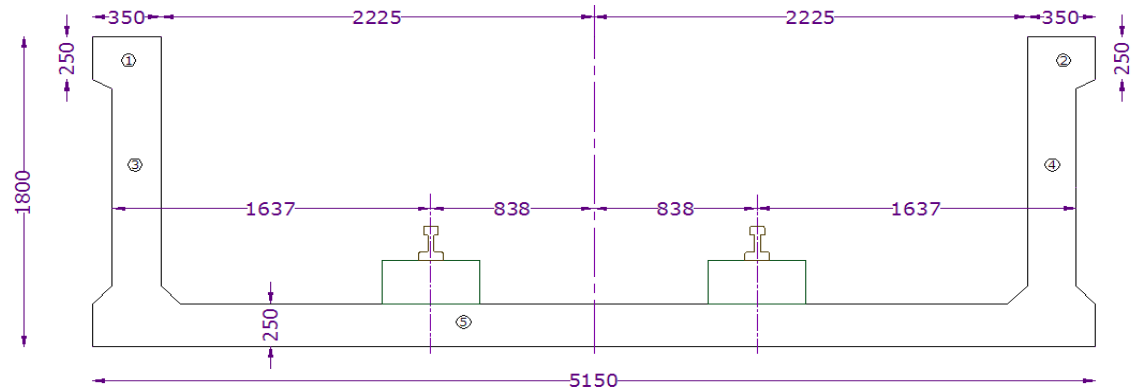

Figure 3.1: Cross Section of U-shaped Girder

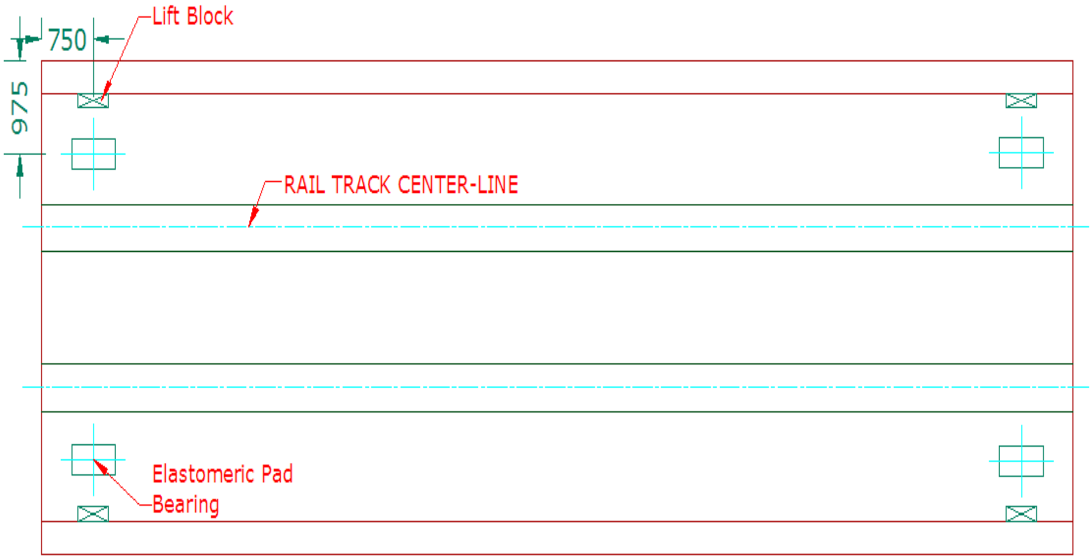

Figure 3.2: Longitudinal Section of Girder

Span of girder	=	15 m
C/c distant between bearing	=	15 - 2*0.75
	=	13.5 m
Loading	=	Railway Loading
Lane	=	Single Lane
Type of gauge	=	Broad Gauge
Location	=	Mumbai
Height from ground to bottom of girder	=	15 m
C/c distance between gauge	=	1676 mm
Compressive Strength of Concrete at Service (f_{ck})	=	50 N/mm^2
Compressive Strength of Concrete at Transfer (f_{ci})	=	35 N/mm^2
Tensile Strength of Prestressing Steel (f_{pu})	=	1860 N/mm^2
Tensile Strength of Non Prestressing Steel (f_y)	=	415 N/mm^2

3.3 Section Properties

Top width of section (B_t)	=	5150 mm
Bottom width of section (B_b)	=	5150 mm
Total height of section (H)	=	1800 mm
Top flange width (b_{f1})	=	350 mm
Top flange thickness (t_{f1})	=	250 mm
Top flange width (b_{f2})	=	350 mm
Top flange thickness(t_{f2})	=	250 mm
Vertical web thickness (t_w)	=	250 mm
Bottom flange width (b_{f3})	=	5150 mm
Bottom flange thickness (t_{f3})	=	250 mm
Neutral Axis distance from bottom (Y_b)	=	491.86 mm
Neutral Axis distance from top (Y_t)	=	1308.14 mm
Neutral Axis distance from left (X_l)	=	2575 mm
Neutral Axis distance from right (X_r)	=	2575 mm
Area of section (A)	=	2112500 mm^2
Moment of Inertia @ X-X axis (I_{xx})	=	626000000000 mm^4
Moment of Inertia @ Y-Y axis (I_{yy})	=	7450000000000 mm^4
Section Modulus of Top section (Z_t)	=	478000000 mm^3
Section Modulus of Bottom section (Z_b)	=	1270000000 mm^3

3.4 Analysis of Girder

3.4.1 Dead Load Calculation

The dead load carried by a bridge member consists of self weight of the girder and any extra fixed load supported by the girder. The following are the dead loads considered during calculation of the bending moment and shear force.The dead load due to deck shuttering, wearing coat, railings, crash barrier, and water main are directly assumed.

- Self weight of girder

- Weight of Ballast cushion
- Railings, kerb or crash barrier
- Water main, if any

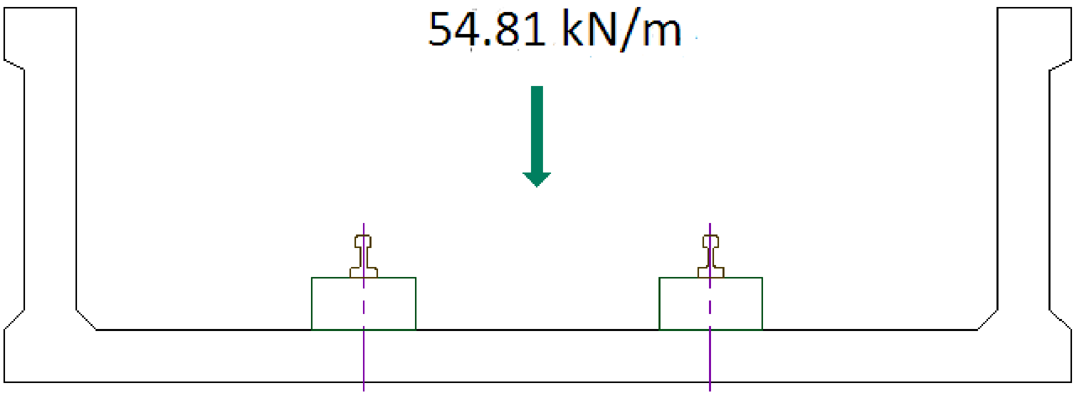

Figure 3.3: Dead Load on girder

Self weight of girder	=	52.81 kN/m
Superimposed dead load	=	2.00 kN/m
Total dead load	=	54.81 kN/m
Longitudinal Bending Moment	=	$\frac{54.81*13.5^2}{8}$
	=	1248.64 kN-m
Transverse Bending Moment	=	$\frac{54.81*(5.15-0.35)}{8}$
	=	32.89 kN-m
Shear Force	=	$\frac{54.81*13.5}{2}$
	=	369.96 kN

3.4.2 Live Load Calculation

Railway bridge loadings should conform to the specification of the Indian Railway Standards (IRS) prescribed by the Ministry of Railways, Government of India. The railway tracks are classified according to the importance of of traffic as main and branch lines. The three types of gauges used in the Indian railways as follows,

- Broad Gauge (BG) :- 1676 mm (5'6")
- Meter Gauge (MG) :- 1000 mm (3'3.375")
- Narrow Gauge (NG) :- 762 mm (2'6")

IRS Bridge rules recommends the use of equivalent uniformly distributed loads (EDUL) on each track and also coefficient of dynamic augment (CDA) for spans varying from 1 to 130 m for both BG and MG loading.

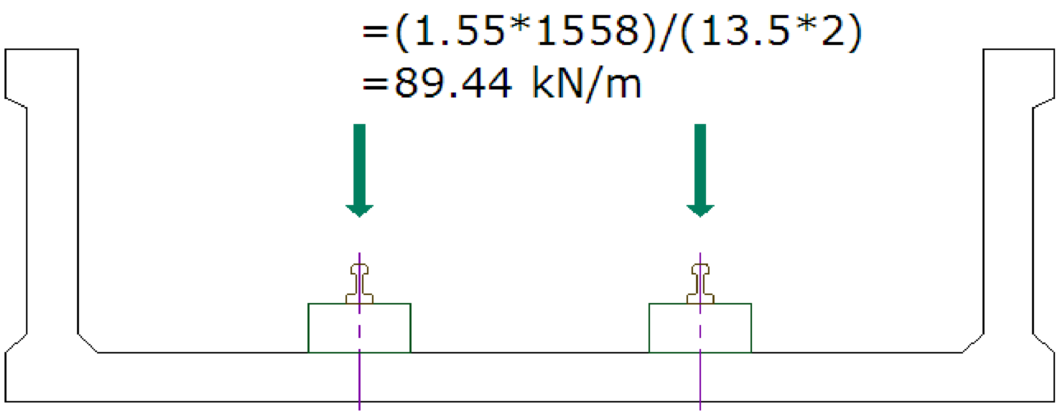

Figure 3.4: Live Load on girder

EUDL for Bending Moment	=	1558 kN
EUDL for Shear Force	=	1740 kN
Co-efficient of Dynamic Auggement	=	0.55
Longitudinal Bending Moment	=	$\frac{1.55*1558*13.5}{4}$
	=	8150.29 kN-m
Transverse Bending Moment	=	45.46 kN-m
Shear Force	=	$\frac{1.55*1740}{2}$
	=	1348.5 kN

3.4.3 Wind Load Calculation

Wind load calculation is done as per Indian Standard IS 875 (part-3). For basic wind pressure calculation with choice of wind velocity due consideration shall also be given to degree of exposure appropriate to locality and to local meteorological data. The wind pressure provided to bridge shall not be considered to be carrying any live load when the wind pressure at deck level exceeds the following limits,

Bridges	Wind Pressure (kN/m^2)
Broad gauge Bridges	1.47
Meter and Narrow gauge Bridges	0.98
Foot Bridges	0.74

Table 3.1: Wind Pressure Limit

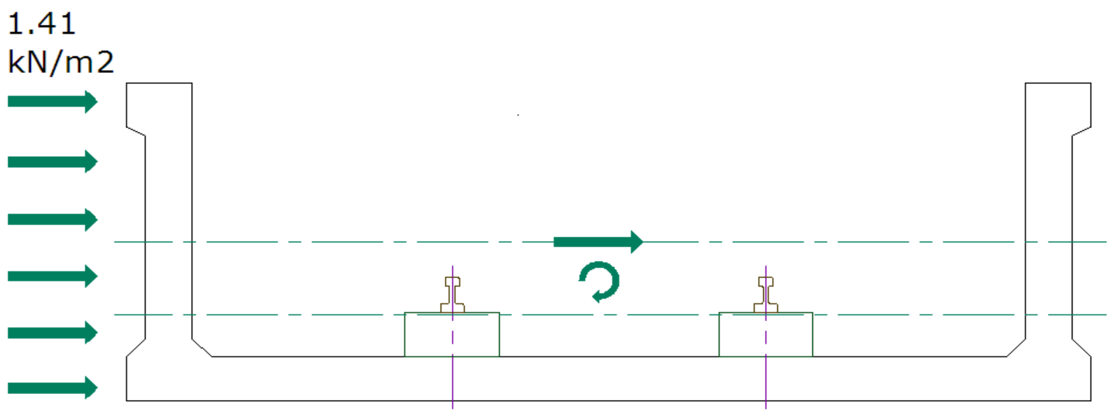

Figure 3.5: Wind Load on girder

$$
\begin{aligned}
\text{Location} &= \text{Mumbai} \\
\text{Basic wind speed } (V_b) &= 44 \text{ m/s} \\
\text{Probability factor } (k_1) &= 1.07 \\
\text{Terrain factor } (k_2) &= 1.03 \\
\text{Topographic factor } (k_3) &= 1.00 \\
\text{Design wind speed } (V_z) &= V_b k_1 k_2 k_3 \\
&= 44 * 1.07 * 1.03 * 1.00 \\
&= 48.49 \text{ m/s}
\end{aligned}
$$

$$
\begin{aligned}
\text{Design wind pressure } (P_z) &= 0.6{V_z}^2 \\
&= 0.6 * 48.49^2 \\
&= 1.41 \text{ kN}/m^2 \\
\text{Design wind force } (F_z) &= P_z A \\
&= 1.41 * 15 * 1.8 \\
&= 38.07 \text{ kN} \\
\text{Torsional Moment} &= F_z e \\
&= 38.07 * (0.5 * 1.8 - 0.491) \\
&= 15.53 \text{ kN-m}
\end{aligned}
$$

3.4.4 Derailment Load Calculation

Derailment load is consider as per clause give in Indian Railway Standard Bridge Rule manual. The derailment load affect the stability of girder due to overturn of girder. The calculation for torsional moment develop due to derailment is shown below,

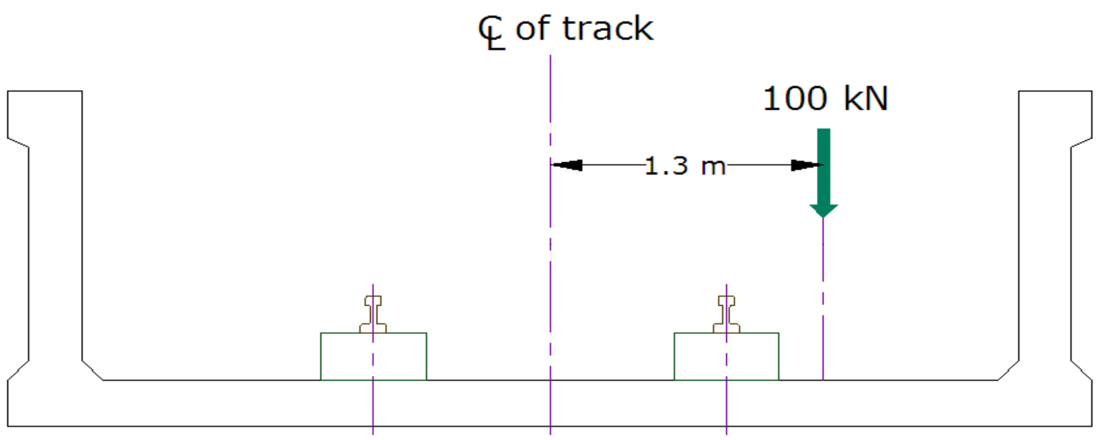

Figure 3.6: Derailment Load on girder

$$
\begin{aligned}
\text{Derailment load} &= 100 \text{ kN} \\
\text{Eccentricity from track centerline} &= 1.3 \text{ m} \\
\text{Torsional Moment} &= 130 \text{ kN-m}
\end{aligned}
$$

3.5 Prestressing Force

The prestressing force and eccentricity is decided to balance tensile stress at time of service. The prestressing force is depended on eccentricity, diameter of strand and number of strands.

$$
\begin{aligned}
\text{Strand diameter} &= 15.4 \text{ mm} \\
\text{Area of strand} &= 139.35\ mm^2 \\
\text{No. of strands} &= 64 \\
\text{No. of layers} &= 2 \\
\text{Centroid of strands from bottom} &= \frac{(32*100)+(32*175)}{64} \\
&= 137.5 \text{ mm} \\
\text{Eccentricity} &= 491.86 - 137.5 \\
&= 354.36 \text{ mm} \\
\text{Prestressing force for single strand} &= 0.75 * 1860 * 139.35 \\
&= 194.39 \text{ kN} \\
\text{Total prestressing force } (P_i) &= 194.39 * 64 \\
&= 12441.17 \text{ kN}
\end{aligned}
$$

3.6 Losses

The reduction in prestressing stress due to elastic deformation shall be deemed to be instantaneous, while reduction in prestressing stress due to creep of concrete, shrinkage of concrete and relaxation of steel is time dependent. These losses shall be described as below,

3.6.1 Elastic Deformation

The loss due to elastic deformation of concrete shall be computed based on the modular ratio and the average stress in concrete at level of steel. If initial stress is known than percentage loss of stress in steel due to elastic deformation of concrete can be computed.

$$Loss = \alpha f_{ci} \tag{3.1}$$

Where,

α = Modular ratio

f_{ci} = Concrete stress due to prestress

$$\begin{aligned}
\text{Modulus of elasticity of concrete } (E_c) &= 34000 \text{ N}/mm^2 \\
\text{Modulus of elasticity of steel } (E_s) &= 195000 \text{ N}/mm^2 \\
\text{Modular ratio } (\alpha) &= \frac{E_s}{E_c} \\
&= \frac{195000}{34000} \\
&= 5.7 \\
\text{Concrete stress due to prestress } (f_{ci}) &= \frac{P_i}{A} + \frac{P_i e^2}{I} - \frac{M_d e}{I} \\
&= \frac{12441.17*10^3}{2112500} + \frac{12441.17*10^3*354.36^2}{626*10^9} \\
&= -\frac{1248.64*10^6*354.36}{626*10^9} \\
&= 7.70 \text{ N}/mm^2 \\
\text{Loss due to elastic deformation} &= 5.7*7.7 \\
&= 44.19 \text{ N}/mm^2
\end{aligned}$$

3.6.2 Creep of Concrete

The loss in strand due to creep of concrete shall be calculated on the assumption that creep is proportional to stress in the concrete for stress of up to one-third of the cube strength at transfer. The loss of prestress is obtained from the product of the creep strain in concrete adjacent to strands and modulus of elasticity of prestressing steel.

$$Loss = \epsilon_{cc} f_{ci} E_s \tag{3.2}$$

Where,

ϵ_{cc} = Creep strain

f_{ci} = Concrete stress due to prestress

E_s = Modulus of elasticity of steel

Creep strain (ϵ_{cc})	=	0.000043
Concrete stress due to prestress (f_{ci})	=	7.70 N/mm^2
Modulus of elasticity of steel (E_s)	=	195000 N/mm^2
Loss due to creep of concrete	=	$0.000043 * 7.7 * 195000$
	=	64.60 N/mm^2

3.6.3 Shrinkage of Concrete

The loss of prestress in the strands due to shrinkage of the concrete may be calculated from modulus of elasticity of strands and shrinkage per unit length.

$$Loss = \epsilon_{sc} E_s \tag{3.3}$$

Where,
ϵ_{cc} = Strain due to shrinkage
E_s = Modulus of elasticity of steel

Strain due to shrinkage (ϵ_{sc})	=	0.0003
Modulus of elasticity of steel (E_s)	=	195000 N/mm^2
Loss due to shrinkage of concrete	=	$0.003 * 195000$
	=	58.50 N/mm^2

3.6.4 Relaxation of Strand

The thousand hour relaxation loss value shall be obtained from the manufacturer of prestressing steel. Where there is no experimental data available and the relaxation loss may assumed 2.5 percent for low relaxation of an initial prestress.

Type of relaxation	=	Low relaxation
Percentage loss	=	2.5
Loss due to relaxation of strand	=	$0.025 * 0.75 * 1860$
	=	34.88 N/mm^2

3.6.5 Anchorage Slip

The magnitude of the loss of stress due to the slip in anchorage is calculated as described below,

$$Loss = \frac{E_s \delta}{L} \tag{3.4}$$

Where,

E_s = Modulus of elasticity of steel

δ = Anchorage slip

L = Length of element

Modulus of elasticity of steel (E_s)	=	195000 N/mm^2
Anchorage slip (δ)	=	5 mm
Length of element (L)	=	15000 mm
Loss due to anchorage slip	=	$\frac{195000*5}{15000}$
	=	65 N/mm^2

3.6.6 Total Loss

Total loss	=	Elastic deformation + Creep of concrete
		+ Shrinkage of Concrete + Relaxation
		+ Anchorage slip
Total loss	=	44.19 + 64.60 + 58.50 + 34.88 + 65
Total loss	=	267.2 N/mm^2
Effective prestressing stress	=	$0.75 * 1860 - 267.2$
	=	1127.8 N/mm^2
Effective prestressing force (P_{eff})	=	$1127.8 * 139.35 * 64$
	=	10058.4 kN

3.7 Stresses

The stress at top and bottom fiber should be check at time of transfer and at time of service stage.

3.7.1 Stress at Transfer

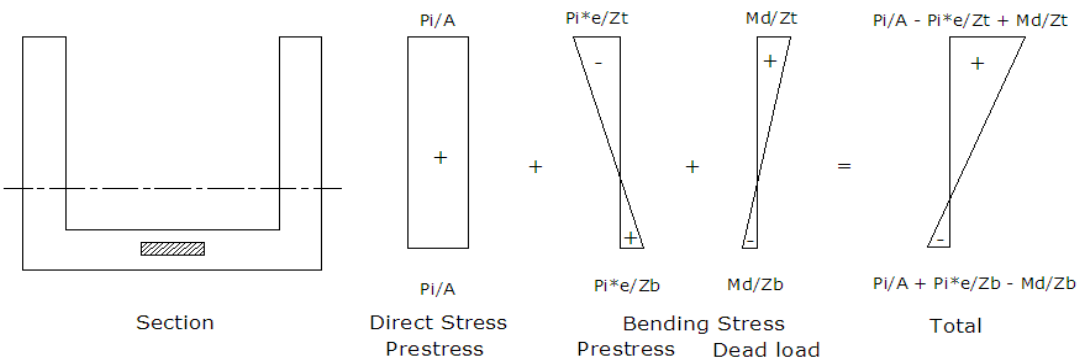

Figure 3.7: Stress at Transfer

Top fiber stress at transfer (Tensile stress),

$$f_{tt} = \frac{P_i}{A} + \frac{P_i e}{Z_t} + \frac{M_d}{Z_t} \tag{3.5}$$

Bottom fiber stress at transfer (Compressive stress),

$$f_{ct} = \frac{P_i}{A} + \frac{P_i e}{Z_b} - \frac{M_d}{Z_b} \tag{3.6}$$

Where,

P_i = Initial prestressing force

A = Area of cross section

e = Eccentricity

Z_t = Top section modulus of section

Z_b = Bottom section modulus of section

M_d = Bending moment due to dead load

Top fiber stress at transfer	=	-0.81 N/mm^2
	$\geq$	-1.00 N/mm^2
Bottom fiber stress at transfer	=	8.41 N/mm^2
	$\leq$	0.5*f_{ci}
	$\leq$	17.50 N/mm^2

3.7.2 Stress at Service

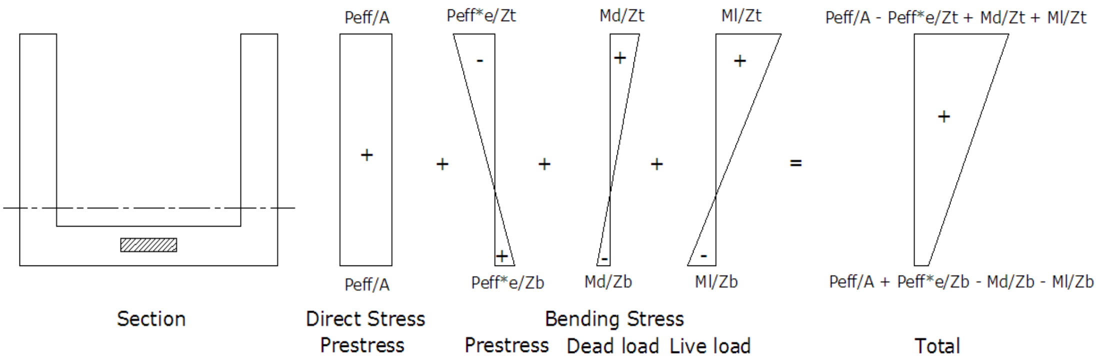

Figure 3.8: Stress at Service

Top fiber stress at transfer (Compressive stress),

$$f_{cw} = \frac{P_{eff}}{A} + \frac{P_{eff}e}{Z_t} + \frac{M_d}{Z_t} + \frac{M_l}{Z_t} \tag{3.7}$$

Bottom fiber stress at transfer (Tensile stress),

$$f_{tw} = \frac{P_{eff}}{A} + \frac{P_{eff}e}{Z_b} - \frac{M_d}{Z_b} - \frac{M_l}{Z_b} \tag{3.8}$$

Where,

P_{eff} = Effective prestressing force

A = Area of cross section

e = Eccentricity

Z_t = Top section modulus of section

Z_b = Bottom section modulus of section

M_d = Bending moment due to dead load

M_l = Bending moment due to live load

Top fiber stress at transfer	$=$	18.57 N/mm^2
	$\leq$	0.4*f_{ck}
	$\leq$	20.00 N/mm^2
Bottom fiber stress at transfer	$=$	-0.03 N/mm^2
	$\geq$	0.00 N/mm^2

3.8 Limit state of Collapse : Flexural

The assessment of the structure under design load shall ensure that the structure does not collapse under critical condition of flexural. The effect of creep and shrinkage of concrete, temperature difference and differential settlement need not be considered at the ultimate limit state.
Ultimate Bending Moment due to load,

$$M_{ult} = 1.4M_d + 2.0M_{sd} + 2.0M_l \tag{3.9}$$

Where,
M_{ult} = Ultimate bending moment
M_d = Bending moment due to dead load
M_{sd} = Bending moment due to super-imposed load
M_l = Bending moment due to live load

Neutral Axis distance from top fiber,

$$x_u = \frac{0.87 f_{pu} A_{sp}}{0.4 f_{ck} b_w} \tag{3.10}$$

Where,
x_u = Neutral axis distance from top fiber
f_{pu} = Tensile strength of prestressing steel
A_{sp} = Area of prestressing steel

f_{ck} = Compressive strength of concrete

b_w = Thickness of web

Ultimate Flexural Strength,

$$M_r = f_{pb} A_{sp} (d - 0.5 x_u) \tag{3.11}$$

Where,

x_u = Neutral axis distance from top fiber

f_{pb} = Permissible Tensile strength of prestressing steel = $0.87 f_{pu}$

A_{sp} = Area of prestressing steel

d = Depth of web

Ultimate bending moment due to load (M_{ult})	=	17984.96 kN-m
Neutral axis distance from top fiber (x_u)	=	1443.18 mm
Permissible stress in prestressing stee (f_{pb})	=	1618.20 N/mm^2
Effective depth of prestressing strands (d)	=	1662.50 mm
Area of prestressing steel (A_{ps})	=	8918.40 mm^2
Ultimate flexural strength (M_r)	=	18579 kN-m

3.9 Limit State of Collapse : Shear

The assessment of the structure under design load shall ensure that the structure does not collapse under critical condition of shear force. The effect of creep and shrinkage of concrete, temperature difference and differential settlement need not be considered at the ultimate limit state.
Ultimate Shear Force due to load,

$$V_{ult} = 1.4 V_d + 2.0 V_{sd} + 2.0 V_l \tag{3.12}$$

Where,

V_{ult} = Ultimate shear force

V_d = Shear force due to dead load

V_{sd} = Shear force due to super-imposed load

V_l = Shear force due to live load

Shear Strength of Uncracked section,

$$V_{co} = 0.67bh\sqrt{({f_t}^2 + f_{cp}f_t)} \tag{3.13}$$

Where,

V_{co} = Ultimate shear strength of uncracked section

b = width of web

h = height of web

f_t = Tensile stress in concrete = $0.24\sqrt{f_{ck}}$

f_{cp} = Stress in concrete at centroid

Shear Strength of Cracked section,

$$V_{cr} = 0.037bd\sqrt{f_{ck}} + \frac{M_{cr}}{M}V \tag{3.14}$$

Where,

V_{cr} = Shear strength of cracked section $\leq 0.1bd\sqrt{f_{ck}}$

d = distance from extreme compressive fiber to centroid of strands

M_{cr} = Cracking moment = $(0.37\sqrt{f_{ck}} + f_{pt})\frac{I}{y}$

M = Ultimate bending strength

V = Ultimate shear strength

Ultimate shear force (V_{ult})	$=$	3193 kN
Width (b_w)	$=$	250 mm
Height (h)	$=$	1800 mm
Depth at centroid of strand (d)	$=$	1662.5 mm
Tensile stress in concrete (f_t)	$=$	$0.24\sqrt{50}$
	$=$	1.70 N/mm^2
Stress in concrete at centroid (f_{cp})	$=$	$\frac{10058.4*10^3}{2112500}$
	$=$	4.76 N/mm^2
Shear strength of uncracked section (V_{co})	$=$	998 kN
Cracking moment (M_{cr})	$=$	2538.35 kN-m
Shear strength of cracked section (V_{cr})	$=$	669 kN
Ultimate shear strength of section (V_r)	$=$	669 kN
	$\leq$	3193 kN

Shear reinforcement is required.

3.10 Limit State of Collapse : Torsion

The assessment of the structure under design load shall ensure that the structure does not collapse under critical condition of torsion moment. The effect of creep and shrinkage of concrete, temperature difference and differential settlement need not be considered at the ultimate limit state.

Ultimate Torsional Moment,

Torsional Moment = Torsion due to Wind + Torsion due to Derailment

Torsional Moment = 15.54 + 130

Torsional Moment = 145.54 kN-m

Torsional Shear Stress,

Part	h_{max}	h_{min}	T	v_t	Remark
3	1300	250	24.41	2.23	Stirrup Req.
4	1300	250	24.41	2.23	Stirrup Req.
5	5150	250	96.71	1.89	Stirrup Req.

Table 3.2: Torsional Shear Stress

3.11 Limit State of Serviceability : Deflection

The deflection of the structure or any part of structure shall not affect the appearance or efficiency of the structure.

$$\begin{aligned}
\text{Deflection due to Dead load } (\delta_d) &= \frac{5wl^4}{384EI} \\
&= 1.07 \text{ mm} \\
\text{Deflection due to Live load } (\delta_l) &= \frac{5wl^3}{384EI} \\
&= 2.88 \text{ mm} \\
\text{Deflection due to Prestress } (\delta_p) &= \frac{P_{eff}el^2}{8EI} \\
&= 4.04 \text{ mm} \\
\text{Allow Deflection } (\delta_{allow}) &= \frac{Span}{300} \\
&= 45.00 \text{ mm}
\end{aligned}$$

Deflection check is okay.

3.12 Limit State of Serviceability : Cracking

The structure shall not produce any tensile stresses which result in no cracks at bottom fiber.

$$\begin{aligned}
\text{Tensile stress at service} &= \text{-0.03 N}/mm^2 \\
&\approx \text{0.00 N}/mm^2
\end{aligned}$$

No crack calculation is required.

3.13 Debonding of Prestressing Steel

The debonding of steel is done to reduce stress in concrete due to prestressing force.

Location	0.0L	0.03L	0.07L	0.10L	0.20L
Length	0.0	0.5	1.0	1.5	3.0
Layer-1	6	12	14	20	30
Layer-2	20	20	26	26	32
Prestressing Force	4086.2	5029.2	6286.5	7229.5	9744.1
Eccentricity	334.17	344.99	343.11	349.47	353.15
Bottom Fiber Stress	3.01	3.61	4.41	5.03	6.66
Remark	Ok	Ok	Ok	Ok	Ok
Top Fiber Stress	-0.92	-0.89	-0.84	-0.87	-0.84
Remark	Ok	Ok	Ok	Ok	Ok

Table 3.3: Bebonding Schedule of Prestressing steel

3.14 Supplementary Reinforcement

3.14.1 Bottom Deck Slab

Longitudinal Reinforcement,

Diameter of bar = 8 mm

Spacing of bar = 175 mm

Area of reinforcement = 287 mm^2

Provide 8mm # @ 175mm c/c as longitudinal reinforcement in bottom deck slab.

Bottom Transverse Reinforcement,

Diameter of bar = 20 mm

Spacing of bar = 150 mm

Area of reinforcement = 2095 mm^2

Provide 20mm # @ 150mm c/c as bottom transverse reinforcement.

Top Transverse Reinforcement,

Diameter of bar	=	10 mm
Spacing of bar	=	150 mm
Area of reinforcement	=	524 mm^2

Provide 10mm # @ 150mm c/c as top transverse reinforcement.

3.14.2 Vertical Web

Longitudinal Reinforcement,

Diameter of bar	=	8 mm
Spacing of bar	=	175 mm
Area of reinforcement	=	287 mm^2

Provide 8mm # @ 175mm c/c as longitudinal reinforcement in vertical web.

Stirrups Reinforcement,

Diameter of bar	=	25 mm
Spacing of bar	=	150 mm
Shear force	=	3921 kN
Shear strength	=	4598 kN

Provide 25mm # @ 150mm c/c as stirrups reinforcement in vertical web.

3.14.3 Top Flange

Longitudinal Reinforcement,

Diameter of bar	=	8 mm
Spacing of bar	=	150 mm
Area of reinforcement	=	335 mm^2

Provide 8mm # @ 150mm c/c as longitudinal reinforcement in top flange.

Stirrup Reinforcement,

Diameter of bar	=	8 mm
Spacing of bar	=	150 mm
Area of reinforcement	=	335 mm^2

Provide 8mm # @ 150mm c/c as stirrup reinforcement in top flange.

3.15 Summary

	Allow	Actual	
No. of strands		64	
Initial Prestressing Force		12441.17	kN
Loss of Prestress		19.2	%
Effective Prestressing Force		10058.4	kN
Stresses			
At Transfer			
Compressive stress	17.50	8.41	N/mm^2
Tensile stress	-1.00	-0.81	N/mm^2
At Servide			
Compressive stress	20.00	18.57	N/mm^2
Tensile stress	0.00	-0.03	N/mm^2
Flexural Strength	18579	17985	kN
Shear Strength	4598	3921	kN
Deflection due to load	54.00	3.96	mm
Deflection due to prestress	45.00	4.04	mm
Stress for Cracking	0.00	-0.03	N/mm^2
Concrete		31.69	m^3
		221830	Rs.
Prestressing Steel		1.05	T
		63000	Rs.
Non-prestressing Steel		6.96	T
		313200	Rs.
Finishing		258	m^2
		129000	Rs.
Handling		8000	Rs.
Total Cost		**735030**	Rs.

Chapter 4

Parametric Study

4.1 General

The various span to depth ratio are required to evaluated for quantity and cost analysis of the superstructure to get economical span to depth ratio of section. To evaluate economical span to depth ratio parametric study is done for 15m, 20m and 25m span with various depth. The parametric study is done for broad gauge and meter gauge loading.

4.2 Various Depth

The cross section shown in Figure4.1 is use for parametric study with span of 15m, 20m and 25m. The summary of cost analysis is done with 3 different span for broad and meter gauge loading.

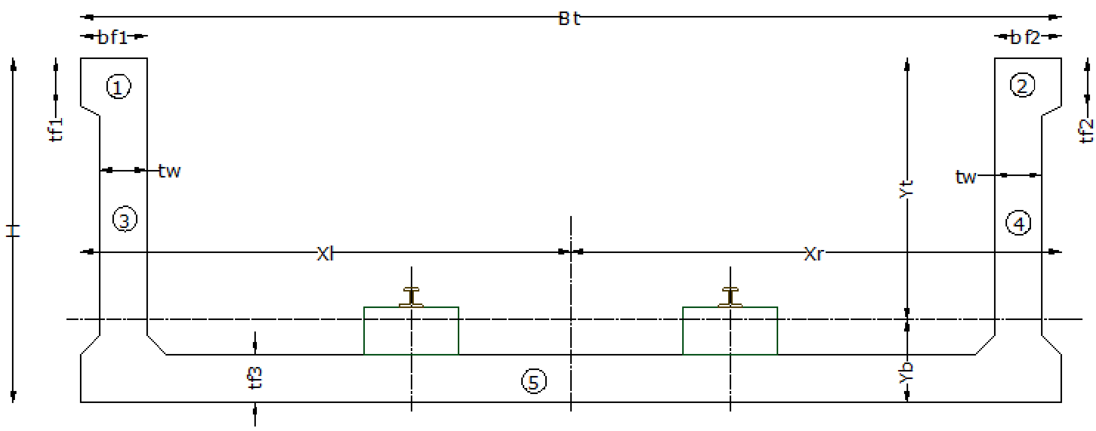

Figure 4.1: Cross Section of U-shaped Girder

Top width of section (B_t)	=	5150 mm
Bottom width of section (B_b)	=	5150 mm
Top flange width (b_{f1})	=	350 mm
Top flange thickness (t_{f1})	=	250 mm
Top flange width (b_{f2})	=	350 mm
Top flange thickness(t_{f2})	=	250 mm
Vertical web thickness (t_w)	=	250 mm
Bottom flange width (b_{f3})	=	5150 mm
Bottom flange thickness (t_{f3})	=	250 mm

For Board gauge loading,

Total height of section (H)	=	2100 mm
	=	2400 mm
	=	2700 mm

For Meter gauge loading,

Total height of section (H)	=	1500 mm
	=	1800 mm
	=	2100 mm

4.3 Analysis

The analysis of superstructure is done with manual calculation. The bending moment, shear force and torsional moment for broad gauge and meter gauge are tabulated in Table 4.1 and 4.2. The corresponding graphical representation is done in Figures. For Broad Gauge Loading,

		L/D	Bending	Shear	Torsional
Span	Depth	Ratio	Moment	Force	Moment
15	2.1	7.1	9716.2	1760.5	150.2
	2.4	6.3	9801.6	1785.8	155.2
	2.7	5.6	9887.0	1811.1	160.4
20	2.1	9.5	15663.5	2103.7	156.9
	2.4	8.3	15824.0	2138.4	163.6
	2.7	7.4	15984.4	2173.1	170.5
25	2.1	11.9	23393.4	2485.2	163.7
	2.4	10.4	23652.2	2529.2	171.9
	2.7	9.3	23911.1	2573.3	180.6

Table 4.1: Analysis result for Broad Gauge Loading

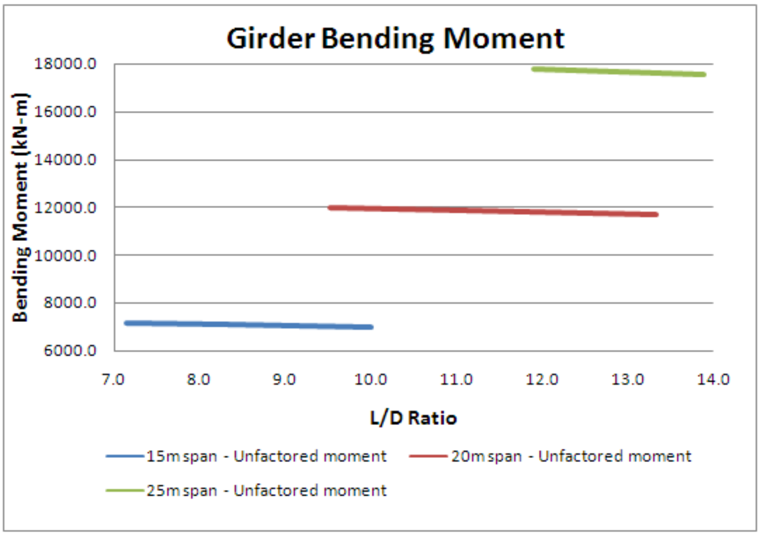

Figure 4.2: Bending moment of girder along L/D ratio for Broad Gauge Loading

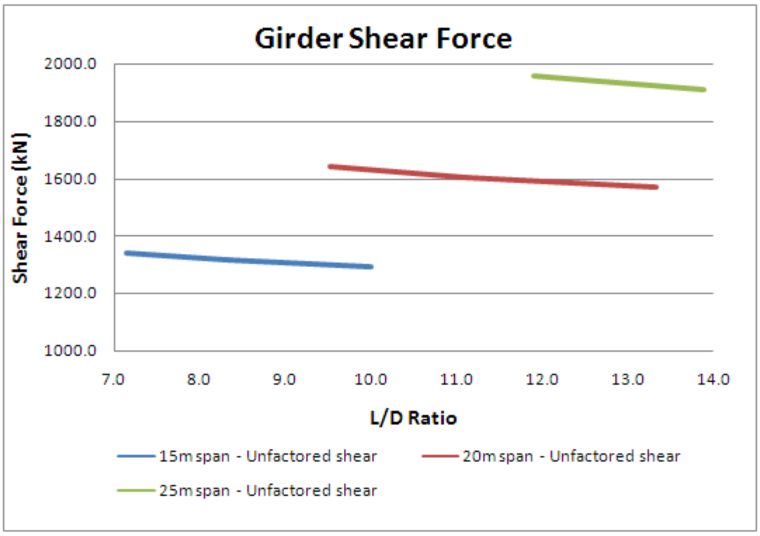

Figure 4.3: Shear force of girder along L/D ratio for Broad Gauge Loading

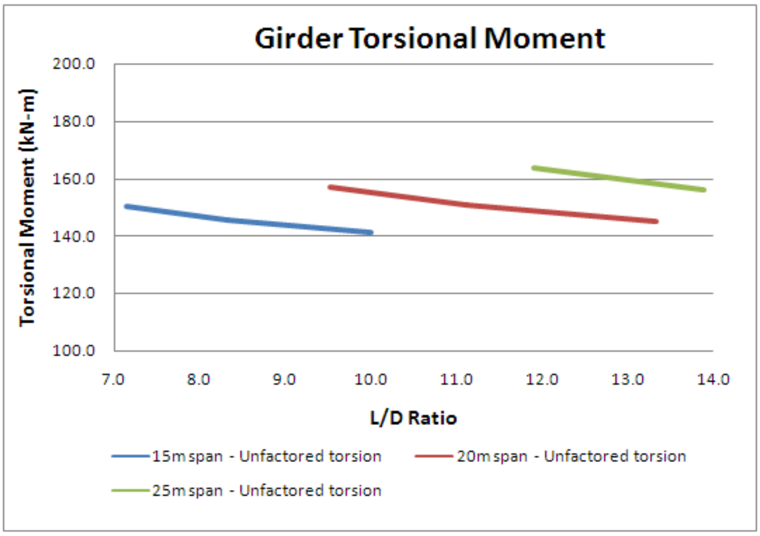

Figure 4.4: Torsional Moment of girder along L/D ratio for Broad Gauge Loading

For Meter Gauge Loading,

Span	Depth	L/D Ratio	Bending Moment	Shear Force	Torsinal Moment
15	1.5	10.0	6995.8	1289.9	141.2
	1.8	8.3	7081.2	1315.2	145.5
	2.1	7.1	7166.7	1340.5	150.2
20	1.5	13.3	11681.1	1570.5	145.0
	1.8	11.1	11841.6	1605.2	150.7
	2.1	9.5	12002.0	1639.9	156.9
25	1.8	13.9	17517.0	1910.9	155.9
	2.1	11.9	17775.9	1955.0	163.7

Table 4.2: Analysis result for Meter Gauge Loading

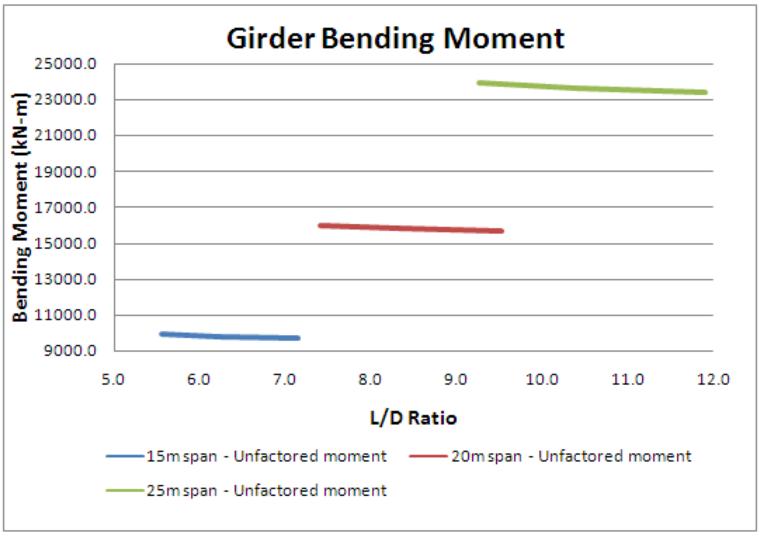

Figure 4.5: Bending moment of girder along L/D ratio for Meter Gauge Loading

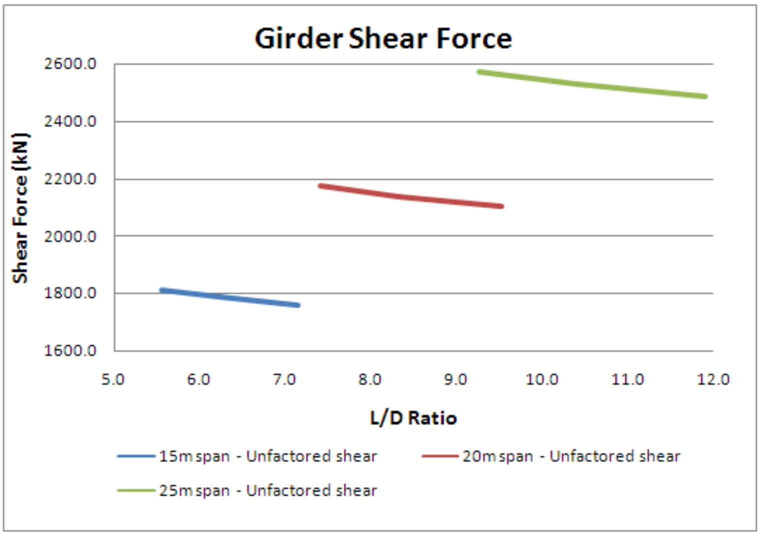

Figure 4.6: Shear force of girder along L/D ratio for Meter Gauge Loading

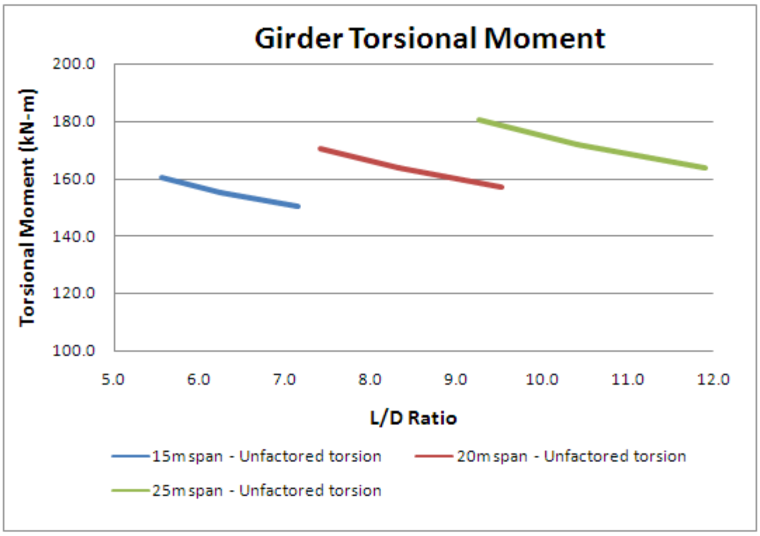

Figure 4.7: Torsional Moment of girder along L/D ratio for Meter Gauge Loading

4.4 Cost Analysis

The design is done by prepared spreadsheet. The overall analysis and design methodology is described in chapter 3. For all the various spans and span to depth ratio girder is designed. In the cost analysis the cost of concrete, prestressing steel, Non-prestressing steel, finishing and handling are included.

For different span to depth ratio Figures show that the cost of girder did not affect due to Non-prestressing steel and Handling cost, but the cost is affected by concrete, prestressing steel and finishing cost. It is observed that as the depth increased the cost of concrete and finishing increased and the cost of prestressing steel decreased. The cost analysis for different span with different span to depth ratio for Broad gauge and Meter gauge loading is described as below,

4.4.1 Cost Analysis for Broad Gauge Loading

Span 15 m					
Depth			**2.1**	**2.4**	**2.7**
L/D ratio			7.1	6.3	5.6
Concrete	Quant.	m^3	33.94	36.19	38.44
	Cost	Rs	237580	253330	269080
Prestressing Steel	Quant.	T	0.92	0.82	0.72
	Cost	Rs	55200	49200	43200
Non-prestressing Steel	Quant.	T	7.50	7.50	7.50
	Cost	Rs	337500	337500	337500
Finishing	Quant.	m^2	276	294	312
	Cost	Rs	138000	147000	156000
Handling	Cost	Rs	8000	8000	8000
Total Cost		Rs	**776280**	795030	817780

Table 4.3: Cost analysis for 15m span with Broad gauge loading

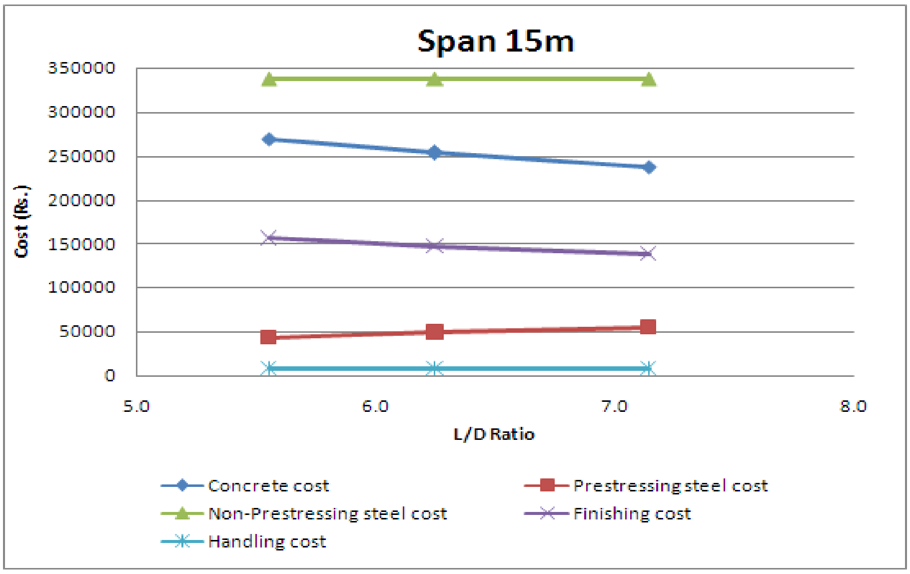

Figure 4.8: Cost analysis for 15m span with L/D ratio for Broad Gauge Loading

Span 20 m					
Depth			**2.1**	**2.4**	**2.7**
L/D ratio			9.5	8.3	7.4
Concrete	Quant.	m^3	45.25	48.25	51.25
	Cost	Rs	316750	337750	358750
Prestressing Steel	Quant.	T	2.19	1.66	1.49
	Cost	Rs	131400	99600	89400
Non-prestressing Steel	Quant.	T	10.00	10.00	10.00
	Cost	Rs	450000	450000	450000
Finishing	Quant.	m^2	368	392	416
	Cost	Rs	184000	196000	208000
Handling	Cost	Rs	8000	8000	8000
Total Cost		Rs	**1090150**	1091350	1114150

Table 4.4: Cost analysis for 20m span with Broad gauge loading

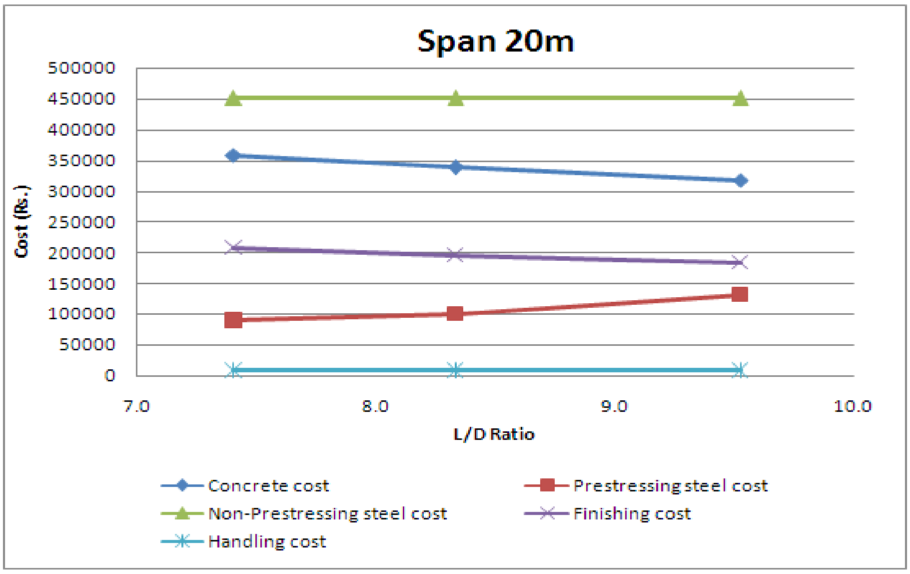

Figure 4.9: Cost analysis for 20m span with L/D ratio for Broad Gauge Loading

Span 25 m					
Depth			**2.1**	**2.4**	**2.7**
L/D ratio			11.9	10.4	9.3
Concrete	Quant.	m^3	56.56	60.31	64.06
	Cost	Rs	395920	422170	448420
Prestressing Steel	Quant.	T	3.66	3.28	2.79
	Cost	Rs	219600	196800	167400
Non-prestressing Steel	Quant.	T	13.60	13.60	13.60
	Cost	Rs	612000	612000	612000
Finishing	Quant.	m^2	460	490	520
	Cost	Rs	230000	245000	260000
Handling	Cost	Rs	8000	8000	8000
Total Cost		Rs	**1465520**	1483970	1495820

Table 4.5: Cost analysis for 25m span with Broad gauge loading

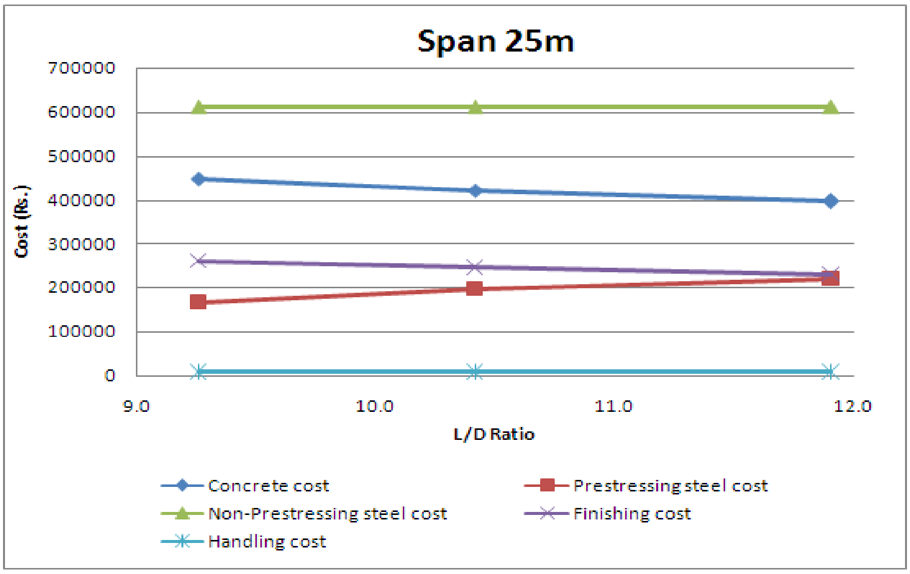

Figure 4.10: Cost analysis for 25m span with L/D ratio for Broad Gauge Loading

Span	Depth	L/D ratio	Total cost
15	2.1	7.1	776280
	2.4	6.3	795030
	2.7	5.6	817780
20	2.1	9.5	1090150
	2.4	8.3	1091350
	2.7	7.4	1114150
25	2.1	11.9	1465520
	2.4	10.4	1483970
	2.7	9.3	1495820

Table 4.6: Total Cost of girder with various L/D ratio for Broad gauge loading

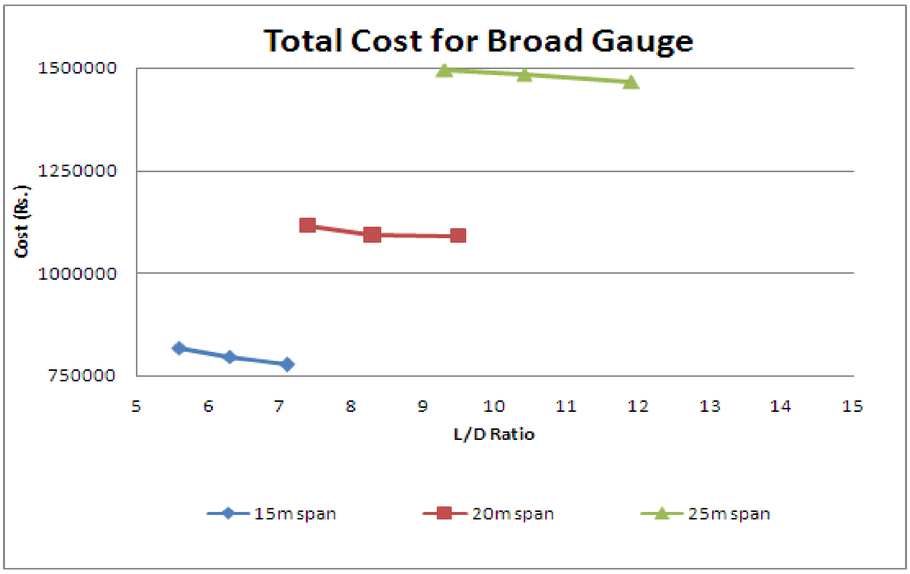

Figure 4.11: Total cost of girder with various L/D ratio for Broad Gauge Loading

4.4.2 Cost Analysis for Meter Gauge Loading

Span 15 m					
Depth			**1.5**	**1.8**	**2.1**
L/D ratio			10	8.3	7.1
Concrete	Quant.	m^3	29.44	31.69	33.94
	Cost	Rs	206080	221830	237580
Prestressing Steel	Quant.	T	1.08	0.82	0.66
	Cost	Rs	64800	49200	39600
Non-prestressing Steel	Quant.	T	6.96	6.96	6.96
	Cost	Rs	313200	313200	313200
Finishing	Quant.	m^2	240	258	276
	Cost	Rs	120000	129000	138000
Handling	Cost	Rs	8000	8000	8000
Total Cost		Rs	**712080**	721230	736380

Table 4.7: Cost analysis for 15m span with Meter gauge loading

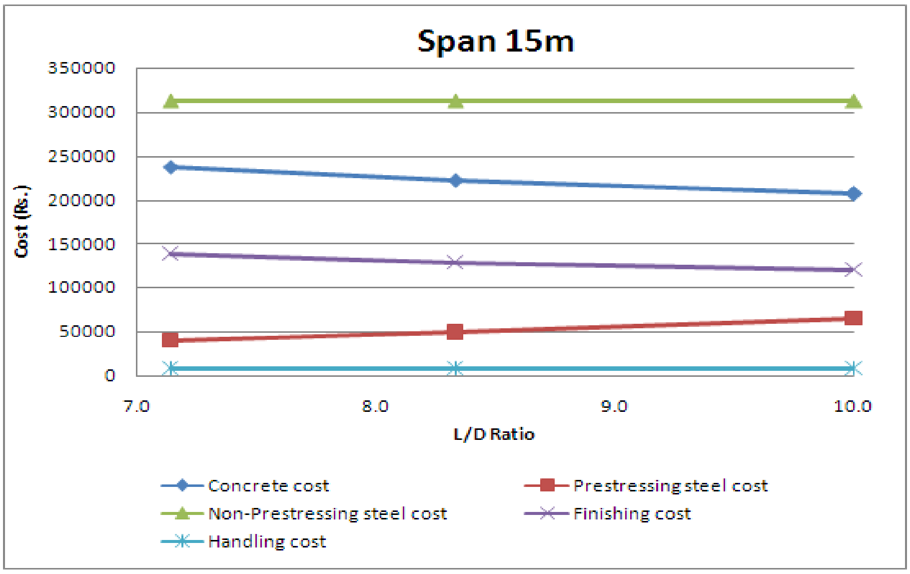

Figure 4.12: Cost analysis for 15m span with L/D ratio for Meter Gauge Loading

Span 20 m					
Depth			**1.5**	**1.8**	**2.1**
L/D ratio			13.3	11.1	9.5
Concrete	Quant.	m^3	39.25	42.25	45.25
	Cost	Rs	274750	295750	316750
Prestressing Steel	Quant.	T	2.19	1.84	1.49
	Cost	Rs	131400	110400	89400
Non-prestressing Steel	Quant.	T	9.27	9.27	9.27
	Cost	Rs	417150	417150	417150
Finishing	Quant.	m^2	320	344	368
	Cost	Rs	160000	172000	184000
Handling	Cost	Rs	8000	8000	8000
Total Cost		Rs	**991300**	1003300	1015300

Table 4.8: Cost analysis for 20m span with Meter gauge loading

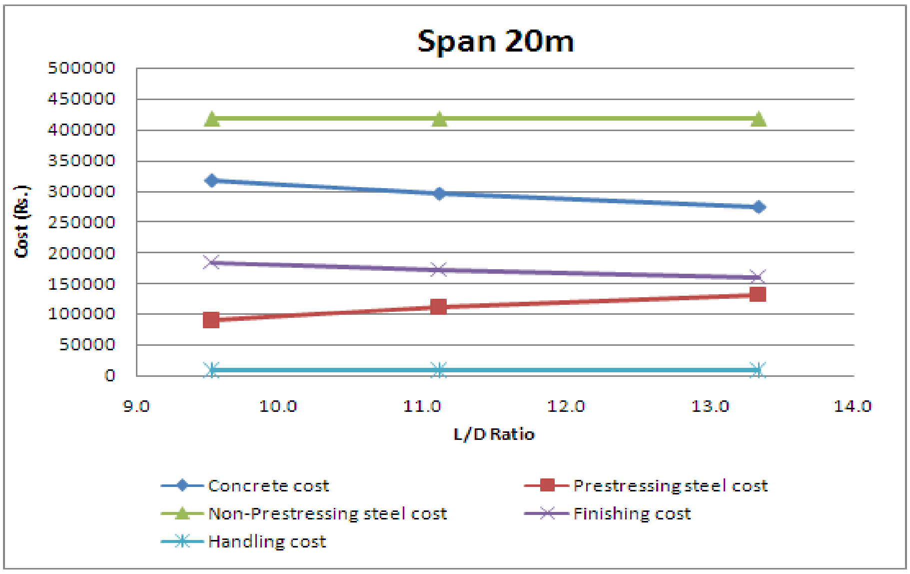

Figure 4.13: Cost analysis for 20m span with L/D ratio for Meter Gauge Loading

Span 25 m				
Depth			**1.8**	**2.1**
L/D ratio			13.9	11.9
Concrete	Quant.	m^3	52.81	56.66
	Cost	Rs	369670	395920
Prestressing Steel	Quant.	T	3.83	2.95
	Cost	Rs	229800	177000
Non-prestressing Steel	Quant.	T	11.60	11.60
	Cost	Rs	522000	522000
Finishing	Quant.	m^2	430	460
	Cost	Rs	215000	230000
Handling	Cost	Rs	8000	8000
Total Cost		Rs	**1344470**	1332920

Table 4.9: Cost analysis for 25m span with Meter gauge loading

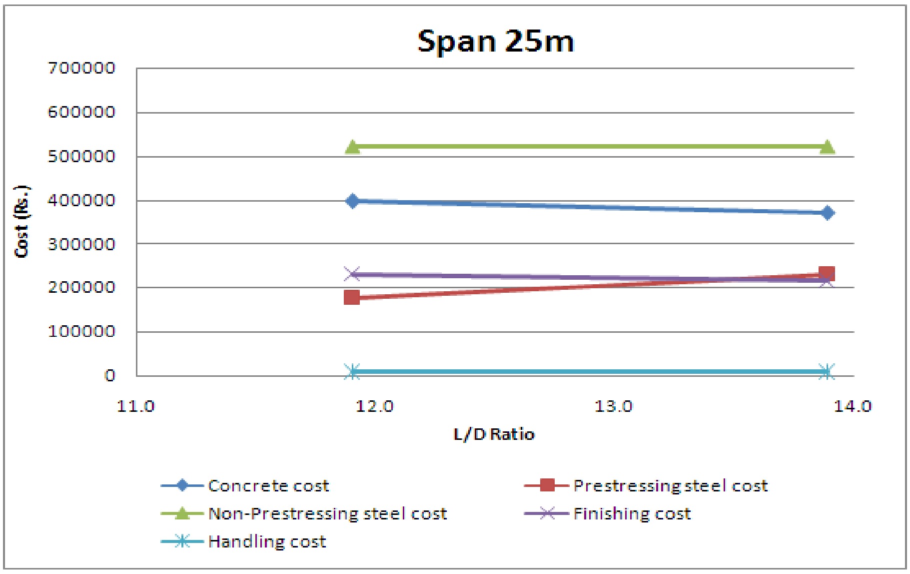

Figure 4.14: Cost analysis for 25m span with L/D ratio for Meter Gauge Loading

Span	Depth	L/D ratio	Total cost
15	1.5	10.0	712080
	1.8	8.3	721230
	2.1	7.1	736380
20	1.5	13.3	991300
	1.8	11.1	1003300
	2.1	9.5	1015300
25	1.8	13.9	1344470
	2.1	11.9	1332920

Table 4.10: Total Cost of girder with various L/D ratio for Broad gauge loading

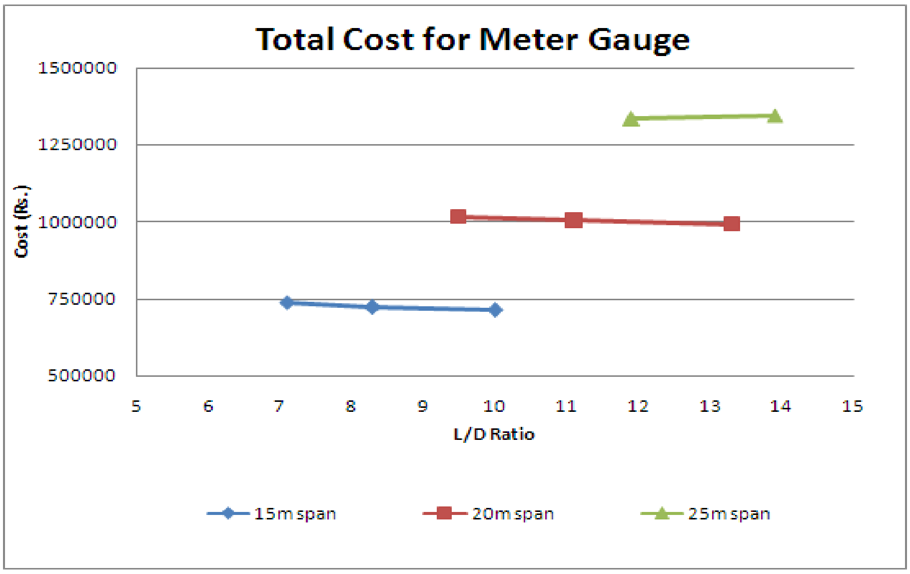

Figure 4.15: Total cost of girder with various L/D ratio for Broad Gauge Loading

4.5 Summary

Parametric study is done to found out the economical span to depth ration for 15m, 20m and 25m with Broad gauge and Meter gauge loading. It is observed ratio 7.1, 9.5 and 11.9 are economical L/D ratio for 15m, 20m and 25m span respectively under Broad gauge loading and ratio 10.0, 13.3 and 11.9 are economical L/D ratio for 15m, 20m and 25m span respectively under Meter gauge loading. The cost of girder is changes by 27 percent when span change by 5m.

Chapter 5

Substructure Design

5.1 General

The substructure of bridge is important part of the bridge. The substructure of bridge includes bearing, pier-cap, pier, pile-cap, pile. The various parameter and design of the component of substructure is done as per concrete bridge rule under IRS standards.

5.2 Load

5.2.1 Dead Load Calculation

Self weight of girder	=	52.81 kN/m
Superimposed dead load	=	2.00 kN/m
Total dead load	=	54.81 kN/m
Longitudinal Bending Moment	=	$\frac{54.81*13.5^2}{8}$
	=	1248.64 kN-m
Transverse Bending Moment	=	$\frac{54.81*(5.15-0.35)}{8}$
	=	32.89 kN-m
Shear Force	=	$\frac{54.81*13.5}{2}$
	=	369.96 kN

5.2.2 Live Load Calculation

EUDL for Bending Moment	=	1558 kN
EUDL for Shear Force	=	1740 kN
Co-efficient of Dynamic Auggement	=	0.55
Tractive Force	=	490 kN
Breaking Force	=	368 kN
Total Longitudinal Force	=	858 kN
Longitudinal Bending Moment	=	8150.29 kN-m
Transverse Bending Moment	=	45.46 kN-m
Shear Force	=	1348.5 kN

5.2.3 Wind Load Calculation

Location	=	Mumbai
Basic wind speed (V_b)	=	44 m/s
Probability factor (k_1)	=	1.07
Terrain factor (k_2)	=	1.03
Topographic factor (k_3)	=	1.00
Design wind speed (V_z)	=	$V_b k_1 k_2 k_3$
	=	$44 * 1.07 * 1.03 * 1.00$
	=	48.49 m/s
Design wind pressure (P_z)	=	$0.6{V_z}^2$
	=	$0.6 * 48.49^2$
	=	1.41 kN/m^2
Design wind force (F_z)	=	$P_z A$
	=	$1.41 * 15 * 1.8$
	=	38.07 kN
Torsional Moment	=	$F_z e$
	=	$38.07 * (0.5 * 1.8 - 0.491)$
	=	15.53 kN-m

5.2.4 Earthquake Load Calculation

Location	=	Mumbai
Zone	=	3
Basic seismic co-efficient (α_o)	=	0.04
Soil Type	=	Medium Soil
Soil Foundation system factor (β)	=	1.0
Importance factor (I)	=	1.5
Design seismic co-efficient (α_h)	=	$\alpha_o \beta I$
	=	$0.04 * 1.0 * 1.5$
	=	0.06
Force	=	$Weight * \alpha_h$
Force along Lateral direction	=	$(52.81 * 15 + 1740 * 0.5) * 0.06$
	=	99.73 kN
Force along Longitudinal direction	=	$(52.81 * 15) * 0.06$
	=	47.53 kN

5.2.5 Load Combination

The calculation of load with combination for single bearing is shown below,

Load Combination	Vertical Load	Lateral Load	Longitudinal Load
	kN	kN	kN
1.4DL + 2SDL + 2LL	1625.70	44.10	150.00
1.4DL + 2SDL + 1.6WL	287.75	15.23	0.00
1.4DL + 2SDL + 1.6EL	287.75	39.89	19.0
1.4DL + 2SDL + 1.75LL + 1.25WL	1467.69	50.48	131.25
1.4DL + 2SDL + 1.75LL + 1.25EL	1467.69	70.68	146.10

Table 5.1: Load on Single Bearing

5.3 Bearing

The neoprene pad is very common as bearing for bridges due to its economy and easy maintenance. The neoprene bearing or elastomeric bearing has good physical properties such as compactness, weather resistance and flame resistance.

5.3.1 Parameters

Maximum vertical load on bearing	=	1625.7 kN
Maximum lateral load on bearing	=	70.68 kN
Maximum longitudinal load on bearing	=	150 kN
Grade of Concrete (f_{ck})	=	40 N/mm^2
Total strain due to creep,shrinkage	=	0.000343

From IRC: 83 (part-II),

Width of bearing	=	320 mm
Length of bearing	=	630 mm
Area of bearing (A_2)	=	$630 * 320$
	=	201600 mm^2
Width of concrete bed block	=	470 mm
Length of concrete bed block	=	780 mm
Area of concrete bed block (A_1)	=	$780 * 470$
	=	366600 mm^2
Area ratio	=	$\frac{A_1}{A_2}$
	=	1.818
	$\leq$	2
Allowable contact pressure	=	$0.25 f_{ck}\sqrt{\frac{A_1}{A_2}}$
	=	$0.25 * 40 * \sqrt{1.818}$
	=	13.48 N/mm^2

$$
\begin{aligned}
\text{Effective bearing area required} &= \frac{Load}{AllowPressure} \\
&= \frac{1625.7*10^3}{13.48} \\
&= 120571\ mm^2 \\
&\leq 201600\ mm^2 \\
\text{Actual bearing stress } (\sigma_m) &= \frac{1625.7*10^3}{201600} \\
&= 8.06\ \text{N}/mm^2
\end{aligned}
$$

5.3.2 Design

Layer in bearing,

$$
\begin{aligned}
\text{Thickness of internal elastomeric layer } (h_i) &= 10 \text{ mm} \\
\text{Thickness of external elastomeric layer } (h_e) &= 5 \text{ mm} \\
\text{Thickness of steel plate } (h_s) &= 3 \text{ mm} \\
\text{Number of steel plate} &= 3 \\
\text{Total thickness of bearing} &= 2*5+3*3+2*10 \\
&= 39 \text{ mm}
\end{aligned}
$$

5.3.3 Checks

Shape factor,

$$
\begin{aligned}
\text{S} &= \frac{LoadedArea}{Areaallowtobulge} \\
&= \frac{(630-12)*(320-12)}{10*2*(630+320)} \\
&= 10 \\
&\geq 6 \\
&\leq 12
\end{aligned}
$$

Shear strain along longitudinal,

$$
\begin{aligned}
\gamma_d &= \text{strain due to creep,shrinkage + strain due to longitudinal force} \\
&= \left(\frac{0.5*3.43*10^{-4}*15000}{39}\right) + \left(\frac{150*10^3}{201600}\right) \\
&= 0.79 \\
&\approx 0.7
\end{aligned}
$$

Shear strain along lateral,

$$\begin{aligned}\gamma_d &= \text{strain due to creep,shrinkage + strain due to longitudinal force}\\ &= \left(\frac{0.5*3.43*10^{-4}*15000}{39}\right) + \left(\frac{70.68*10^3}{201600}\right)\\ &= 0.42\\ &\leq 0.7\end{aligned}$$

Allowable Angle of rotation,

$\alpha_d = \beta n \alpha_{bimax}$

where,

$\beta = 0.1\sigma'_m$

n = Number of internal elastomeric layer

$\alpha_{bimax} = \dfrac{0.5\sigma_m h_i}{bs^2}$

σ_m = Allowable bearing pressure

h_i = Thickness of internal elastomeric layer

b = Dimension of bearing along direction of rotation

s = Shape factor of bearing

Allowable angle of rotation along longitudinal	=	0.00129
Allowable angle of rotation along lateral	=	0.00261

Friction,

Actual Shear strain along longitudinal	=	0.79
Actual Shear strain along lateral	=	0.42
Allow shear strain	$\leq$	$0.2 + 0.1\sigma_m{}'$
	$\leq$	$0.2 + 0.1 * 8.06$
	$\leq$	1.006

Shear stress due to compression,

$= 1.5\frac{\sigma_m{}'}{s}$

$= 1.5\frac{8.06}{10}$

$= 1.209$ N/mm^2

Shear stress due to horizontal deformation,

$= 0.79$ N/mm^2

Shear stress due to rotation,

$= 0.5(\frac{b}{h_i})^2\alpha_{bi}$

$= 1.53$ N/mm^2

Total shear stress,

$= 1.209 + 0.79 + 1.53$

$= 3.53$ N/mm^2

≤ 5 N/mm^2

5.4 Design of pier cap

5.4.1 Parameters

Maximum vertical load on bearing	=	1625.7 kN
Maximum lateral load on bearing	=	70.68 kN
Maximum longitudinal load on bearing	=	150 kN
Width of bearing	=	320 mm
Length of bearing	=	630 mm
Width of concrete bed block	=	470 mm
Length of concrete bed block	=	780 mm
Width of pier-cap	=	$2 * 150 + 2 * 390 + 2 * 750 + 50$
	=	2630 mm
Length of pier-cap	=	$2 * 5150 + 50$
	=	10350 mm
Grade of concrete (f_{ck})	=	40 N/mm^2
Grade of steel (f_y)	=	415 N/mm^2

5.4.2 Analysis and Design

Along Longitudinal direction,

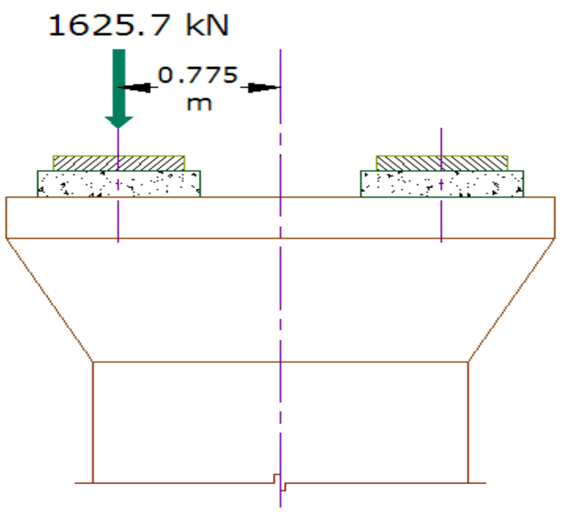

Figure 5.1: Pier-cap section along Longitudinal direction

Bending Moment,

$= 2*1625.7*0.775 + 0.5*49.31*1.315^2$

= 2562.47 kN-m

Shear Force,

= 2*1625.7 + 49.31*1.315

= 3316.24 kN

Along Lateral direction,

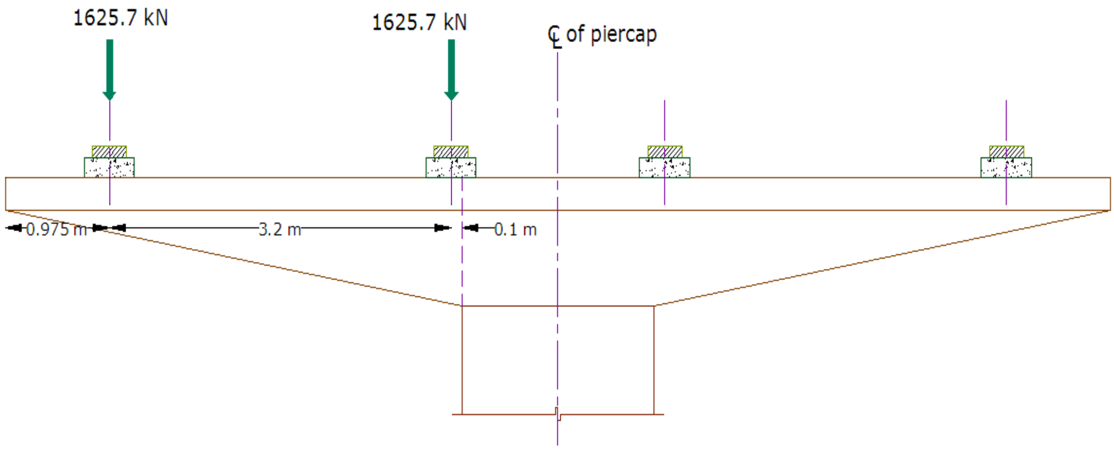

Figure 5.2: Pier-cap section along Lateral direction

Bending Moment,

$= 1625.7*3.3 + 1625.7*0.1 + 0.5*49.31*4.275^2$

= 5978 kN-m

Shear Force,

= 1625.7 + 1625.7 + 49.31*4.275

= 3462.2 kN

$$
\begin{aligned}
\text{Required Area of steel } (A_{st}) &= \tfrac{0.5 f_{ck} bd}{f_y}[1 - \sqrt{1 - \tfrac{4.6M}{f_{ck} bd^2}}] \\
&= \tfrac{0.5*40*1000*1125}{415}[1 - \sqrt{\tfrac{4.6*5978*10^6}{40*1000*1125^2}}] \\
&= 17572\ mm^2
\end{aligned}
$$

Provide,

2 layers of 32mm # @ 75mm c/c as Top reinforcement.

1 layer of 32mm # @ 75mm c/c as Bottom reinforcement.

$$
\begin{aligned}
\text{Percentage of steel (pt)} &= \tfrac{100 A_{st}}{bd} \\
&= \tfrac{100*21500}{1000*1125} \\
&= 1.91 \\
\text{Shear stress in concrete } (\tau_c) &= 0.9\ \text{N}/mm^2 \\
\text{Shear strength of concrete } (V_c) &= \tau_c bd \\
&= 0.9*1000*1125 \\
&= 1012.5\ \text{kN} \\
\text{Required Shear strength } (V_s) &= V - Vc \\
&= 3462.2 - 1012.5 \\
&= 2449.7\ \text{kN} \\
\text{Shear strength } (V_s) &= \tfrac{0.87 f_y A_{sv} d}{s} \\
&= \tfrac{0.87*415*4*\frac{\pi}{4}*16^2*1125}{125} \\
&= 2612.5\ \text{kN}
\end{aligned}
$$

It is okay.

Provide, 4 legged 16mm # @ 125mm c/c as stirrup reinforcement.

5.5 Design of pier

5.5.1 Parameters

Maximum Vertical load on bearing	=	1625.7 kN
Maximum Lateral load on bearing	=	70.68 kN
Maximum Longitudinal load on bearing	=	150 kN
Height of pier	=	1 + 15
	=	16 m
Diameter of pier (h)	=	1.8 m
Grade of concrete (f_{ck})	=	40 N/mm^2
Grade of steel (f_y)	=	415 N/mm^2

5.5.2 Analysis and Design

Along Longitudinal direction,

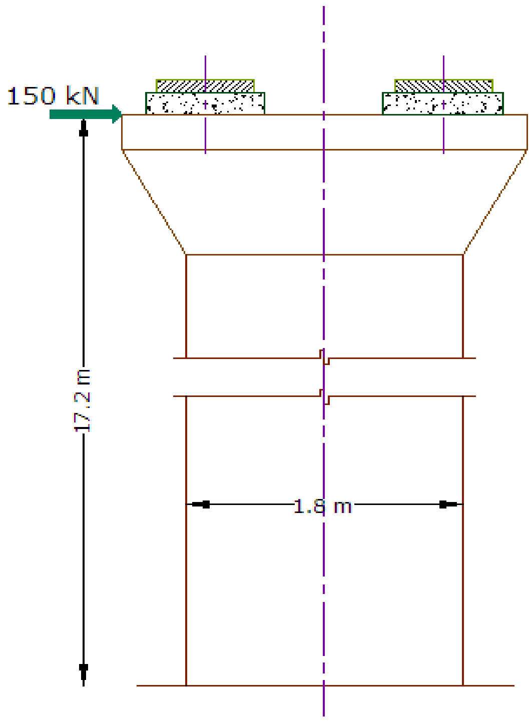

Figure 5.3: Pier section along Longitudinal direction

Bending moment	=	4*150*(16+1.2+0.15+0.039)
	=	10433.4 kN-m

Along Lateral direction,

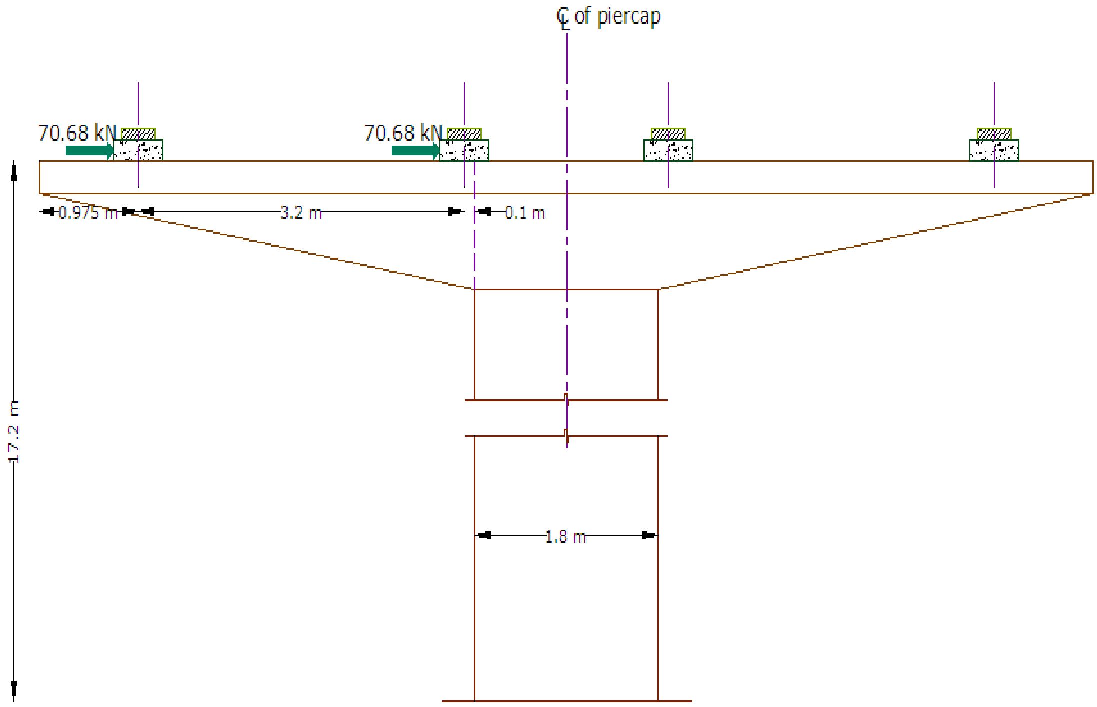

Figure 5.4: Pier section along Lateral direction

$$
\begin{aligned}
\text{Bending moment} &= 4*70.68*(16+1.2+0.15+0.039) \\
&= 4916.21 \text{ kN-m} \\
\text{Vertical load on pier} &= \text{load on bearing + weight of piercap} \\
&= 4*1625.7 + 25*10.35*2.65*(\tfrac{0.3+1.2}{2}) \\
&= 7017.1 \text{ kN} \\
\text{Effective length } (l_e) &= 0.7*16 \\
&= 11.2 \text{ m} \\
\text{Slenderness ratio} &= \frac{l_e}{h} \\
&= \frac{11.2}{1.8} \\
&= 6.22 \\
&\leq 12
\end{aligned}
$$

Pier can be design as short column as per IRS Concrete bridge rules.

Take percentage of steel = 0.8 %

Provide,

28 - 32mm # bars as longitudinal reinforcement in pier.

Ultimate Axial load,

$$P_u = 0.4 f_{ck} b d_c + f_{yc} A'_{s1} + f_{s2} A'_{s2} \tag{5.1}$$

Ultimate Bending moment,

$$M_u = 0.2 f_{ck} b d_c (h - d_c) + f_{yc} A'_{s1} (\frac{h}{2} - d') + f_{s2} A'_{s2} (\frac{h}{2} - d_c) \tag{5.2}$$

Axial capacity without moment,

$$P_{uz} = 0.45 f_{ck} A_c + f_{yc} A_{sc} \tag{5.3}$$

Biaxial bending moment ratio,

$$(\frac{M_x}{M_{ux}})^{\alpha n} + (\frac{M_y}{M_{uy}})^{\alpha n} \leq 1.0 \tag{5.4}$$

Where, f_{ck} = Grade of concrete = 40 N/mm^2

b = Diameter of pier = 1800 mm

d_c = Depth of concrete in compression = 900-75 = 825 mm

f_{yc} = Design compressive strength of reinforcement = 0.67*415 = 278.05 N/mm^2

A_{s1} = Area of steel in compression = 11270 mm^2

f_{s2} = Stress in reinforcement on other face = $\frac{P}{A} - \frac{M}{Z}$ = -6.01 N/mm^2

A_{s2} = Area of steel in tension = 11270 mm^2

h = Diameter of pier = 1800 mm

A_{sc} = Area of steel = 22540 mm^2

M_x = Bending moment along longitudinal direction = 10433.4 kN-m

M_y = Bending moment along lateral direction = 4916.21 kN-m

M_u = Ultimate bending moment

αn = Constant depend on P/P_{uz} ratio

Ultimate axial load,

$$P_u = 0.4*40*1800*825 + 278.05*11270 - 6.01*11270$$
$$= 26825.9 \text{ kN}$$

Ultimate bending moment,

$$M_u = 0.2*40*1800*825*(1800-825) + 278.05*11270*(900-75) - 6.01*11270*(900-825)$$
$$= 14163.2 \text{ kN-m}$$

Axial capacity without bending,

$$P_{uz} = 0.45 * 40 * \frac{\pi}{4} * 1800^2 + 278.05 * 22540$$
$$= 52071.7 \text{ kN}$$

Biaxial bending moment ratio,

$$= (\frac{10443.4}{14163.2})^{1.5} + (\frac{4916.21}{14163.2})^{1.5}$$
$$= 0.83$$
$$\leq 1.0$$

It is okay.

Provide,

28 - 32mm # bars as longitudinal reinforcement in pier.

10mm # @ 150mm c/c as spiral reinforcement in pier.

5.6 Design of pile cap

5.6.1 Parameters

Maximum vertical load	=	load on pier + weight of pier
	=	8098.6 kN
Maximum Longitudinal shear force	=	600 kN
Maximum Longitudinal bending moment	=	10443.4 kN-m
Maximum Lateral shear force	=	282.8 kN
Maximum Lateral bending moment	=	4916.21 kN-m
Diameter of pier	=	1800 mm
Diameter of pile	=	750 mm
Depth of pilecap	=	1000 mm
Minimum c/c distance between pile	=	3*Diameter of pile
	=	2150 mm
Provide c/c distance between pile	=	2400 mm
Width of pile cap	=	2400 + 600 + 2*150
	=	3300 mm
Length of pile cap	=	3300 mm
Grade of concrete (f_{ck})	=	40 N/mm^2
Grade of steel (f_y)	=	415 N/mm^2

5.6.2 Analysis and Design

Along Longitudinal direction,

Maximum load on pile along longitudinal	=	$\frac{8098.6}{2} + \frac{10443.4}{2.4}$
	=	8400.7 kN
Bending Moment in pile cap	=	8400.7*0.3
	=	2520.2 kN-m
Required area of steel	=	8327 mm^2

Along Lateral direction,

Maximum load on pile along lateral	=	$\frac{8098.6}{2} + \frac{4916.21}{2.4}$
	=	6097.7 kN
Bending Moment in pile cap	=	6097.7*0.3
	=	1829.3 kN-m
Required area of steel	=	5940 mm^2

Provide,

32mm # bar @ 90mm c/c along longitudinal direction on both face.

25mm # bar @ 75mm c/c along lateral direction on both face.

Percentage of steel	=	$\frac{100*8944}{1000*925}$
	=	0.97
Shear stress of concrete (τ_c)	=	0.73 N/mm^2
Shear strength of concrete (V_c)	=	0.73*1000*925
	=	675 kN

No shear reinforcement required.

Check for two-way shear,

Total vertical load	=	8098.6 kN
Shear stress	=	$\frac{Load}{Perimeter}$
	=	$\frac{8098.6*10^3}{\pi*1800*925}$
	=	1.55 N/mm^2
	$\leq$	$0.75\sqrt{f_{ck}}$
	$\leq$	4.74 N/mm^2

It is okay.

5.7 Design of pile

Maximum vertical load on pile	=	$\frac{8400.7}{2}$
	=	4200.35 kN
Maximum Longitudinal force on pile	=	$\frac{600}{4}$
	=	150 kN
Maximum Lateral force on pile	=	$\frac{282.8}{4}$
	=	70.68 kN
Diameter of pile	=	750 mm
Length of pile	=	3000 mm
Maximum Longitudinal moment on pile	=	$150 * 3$
	=	450 kN-m
Maximum Lateral moment on pile	=	$70.68 * 3$
	=	212.1 kN-m
Grade of concrete (f_{ck})	=	40 N/mm^2
Grade of concrete (f_y)	=	415 N/mm^2
Provide area of steel	=	24 - 25#
	=	11760 mm^2

Ultimate Axial load,

$$P_u = 0.4 f_{ck} b d_c + f_{yc} A'_{s1} + f_{s2} A'_{s2} \tag{5.5}$$

Ultimate Bending moment,

$$M_u = 0.2 f_{ck} b d_c (h - d_c) + f_{yc} A'_{s1} (\frac{h}{2} - d') + f_{s2} A'_{s2} (\frac{h}{2} - d_c) \tag{5.6}$$

Axial capacity without moment,

$$P_{uz} = 0.45 f_{ck} A_c + f_{yc} A_{sc} \tag{5.7}$$

Biaxial bending moment ratio,

$$(\frac{M_x}{M_{ux}})^{\alpha n} + (\frac{M_y}{M_{uy}})^{\alpha n} \leq 1.0 \tag{5.8}$$

Where,

d_c = Depth of concrete in compression = 375-75 = 300 mm

f_{yc} = Design compressive strength of reinforcement = 0.67*415 = 278.05 N/mm^2

A_{s1} = Area of steel in compression = 5880 mm^2

f_{s2} = Stress in reinforcement on other face = $\frac{P}{A} - \frac{M}{Z}$ = -1.36 N/mm^2

A_{s2} = Area of steel in tension = 5880 mm^2

h = Diameter of pier = 750 mm

A_{sc} = Area of steel = 11760 mm^2

M_x = Bending moment along longitudinal direction = 450 kN-m

M_y = Bending moment along lateral direction = 212.1 kN-m

M_u = Ultimate bending moment

αn = Constant depend on P/P_{uz} ratio

Ultimate axial load,

$$\begin{aligned} P_u &= 0.4*40*750*300 + 278.05*5880 - 1.36*5880 \\ &= 5226.9 \text{ kN} \end{aligned}$$

Ultimate bending moment,

$$\begin{aligned} M_u &= 0.2*40*750*300*(750-300) + 278.05*5880*(375-75) - 1.36*5880*(375-300) \\ &= 1299.9 \text{ kN-m} \end{aligned}$$

Axial capacity without bending,

$$\begin{aligned} P_{uz} &= 0.45 * 40 * \frac{\pi}{4} * 750^2 + 278.05 * 11760 \\ &= 11222 \text{ kN} \end{aligned}$$

Biaxial bending moment ratio,

$= (\frac{450}{1299.9})^{1.5} + (\frac{212.1}{1299.9})^{1.5}$

$= 0.26$

≤ 1.0

It is okay.

Provide,

24 - 25mm # bars as longitudinal reinforcement in pile.

10mm # @ 150mm c/c as spiral reinforcement in pile.

Chapter 6

Summary and Conclusion

6.1 Summary

In present scenario, aesthetic of the bridge structure is the important aspect in design of bridge. Generally, in India railway bridges are constructed with steel section. During the Delhi Metro project and Mumbai Metro project the concept of precast prestressed bridge superstructure with U-shaped has been implemented. The precast construction reduces time duration of project which result in saving in the cost. The precast girder has good aesthetic as compare to cast in-situ girder due to quality management. Due to the U shape of girder it will obstruct view of machines part of rail which result in good view of rail. The prestressed precast U-shaped girder may be give new dimension to bridge superstructure. The U-shaped girder has better scope as it can replace current trend of design of bridge with I,T or Box shaped girder.

In present study, the literature review for U-shaped girder used in various case studies and design procedure for design of pre-tension precast prestressed girder is done. The analysis and design of U-shaped girder with 15m span for broad gauge railway loading is done with specification of concrete bridge rule under Indian Railway Standards. The parametric study of the superstructure for span of 15m, 20m, and 25m with various depth under broad gauge and meter gauge loading is done and economical span to depth ratio is found out. The design sub structure is carried out which include design of bearing, piercap, pier, pilecap, pile. The detailed drawing of superstructure and substructure is prepared.

6.2 Conclusions

Based on the study carried out in this report the following conclusions can be drawn.

- The girder is subjected to flexural moment and shear force due to dead load, superimposed dead load and live load and torsional moment due to wind load and derailment load.
- The loss of prestress is 19.2 percent in 15m girder with broad gauge loading, which is approximately same as it is assume 20 percent.
- The L/D ratio 7.1, 9.5 and 11.9 are economical L/D ratio for 15m, 20m and 25m span respectively under Broad gauge loading.
- The L/D ratio 10.0, 13.3 and 11.9 are economical L/D ratio for 15m, 20m and 25m span respectively under Meter gauge loading.
- The cost of girder is changes by 27 percent when span change by 5m.
- When span increases the cost of girder with various depth become approximately same.

6.3 Future Scope of Work

The study in this report is limited to analysis and design of U-girder with single track under broad gauge and meter gauge loading. The present study can be extended to include following aspects.

- Present study can be studied with two rail track in girder.

- The study is limited to broad gauge and meter gauge loading, which can be studied with standard gauge loading for metro rail.
- The study is limited to railway bridges, which can be studied for highway bridges.

Appendix A

Detail Drawings

- General Arrangement detail drawing.
- Detail Drawing of U-shaped girder.
- Detail Drawing of Sub-structure.
- Detail Drawing of Pier-cap.
- Detail Drawing of Pier, Pile-cap and Pile.

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