PRESTRESSED CONCRETE BOX GIRDER BRIDGE WITH CORRUGATED STEEL WEBS

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NIRMA UNIVERSITY

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Major Project

Submitted in Partial Fulfillment of the Requirements for the degree of

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IN

CIVIL ENGINEERING

(Computer Aided Structural Analysis And Design)

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Declaration

This is to certify that

i) The thesis comprises my original work towards the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis And Design) at Nirma University and has not been submitted elsewhere for a degree.

ii) Due acknowledgement has been made in the text to all other material used.

Pitolwala Zuzar N

Certificate

This is to certify that the Major Project Report entitled **"Prestressed concrete box girder bridge with corrugated steel webs"** submitted by **Mr. Pitolwala Zuzar Nurooddinbhai (Roll No: 11MCLC13)** towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad is the record of work carried out by him under our supervision and guidance.The work submitted has in our opinion reached a level required for being accepted for examination. The results embodied in this major project work to the best of our knowledge have not been submitted to any other University or Institution for award of any degree or diploma.

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Abstract

The history of the development of the bridges is closely associated with the history of human civilization. The bridge builders has always a desire to build new type of bridges, either new in concept or new in technique of construction. The hybrid prestressed concrete bridges with corrugated steel webs was originally developed in France in the 1980's and later introduced to Japan in the 1990's. In this structure, the concrete webs are replaced with trapezoidally corrugated steel plates to reduce the dead load of the structure, improving the prestress efficiency, reducing the construction work, and cost are principally main advantages of this structure. Moreover, using corrugated webs allows one to avoid using stiffeners usually needed in flat plate webs.

This work has been carried out for study, the behavior 2 span continuous prestressed concrete box-girder bridges with corrugated steel webs. The analysis of the Conventional Bridge and by replacing the prestressed concrete webs with corrugated webs bridges are done by using professionally available 'Staad Pro. Software' for dead load, superimposed load and moving load as a class AA tracked, class AA wheeled and class A train of vehicle of IRC loading. The design of 2 span continuous 1 rectangular box girder with PC webs as per IRC 18:2000 and replacing the prestressed concrete webs with trapezoidally corrugated steel webs has been carried out as per IRC 22:2008.

It is necessary to design the webs without buckling at the ultimate limit state. Stresses has been calculated for these three buckling modes local, global, interactive shear buckling. Lateral torsional buckling capacity has been calculated.

Study was done to compare the cost difference between the conventional PC box girder bridge with corrugated steel webs box Girder Bridge. Also study was done to compare the unit weight between the conventional PC box girder bridge with corrugated steel webs box Girder Bridge.

Economy mainly depends on various factors like span and superstructure cross sectional dimensions. The present study includes parametric study on PC box girder with corrugated steel webs for two lane Road Bridge by changing span to depth ratio and changing depth of web to depth of corrugation in such a way that it becomes most economical box girder.

Parametric study was done for calculation of most economical L/D ratio and hw/d ratio for 40m, 45m and 50m. For this all costing was done with quantity analysis and rate analysis as per current market rates.

Acknowlegement

I would like to thank my guide **Prof.Jahanvi Suthar**,whose keen interest and knowledge base helped me to carry out the major project work. Her constant support and guidance during my project work equipped me with a great understanding of different aspects of the project work. She has shown keen interest in this work right from beginning and has been a great motivating factor in outlining the flow of my work. I am also thankful to **Prof. N. C. Vyas** and **Prof. Himat Solanki**, Department of Civil Engineering whose keen interest and excellent knowledge base helped me to carry out the dissertation work. Their constant support and interest in the subject equipped me with a great understanding of different aspects of the required architecture for the project work. They have shown keen interest in this dissertation work right from beginning and has been a great motivating factor in outlining the flow of my work.

My gratitude is due to **Dr V V Nori**, Director at Shirish Patel and Associates, for providing his excellent concept related to corrugated web bridges during my summer training.

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Abbreviation Notation and Nomenclature

a .... Flat panel width

A_{st} .... Area of reinforcement

A_q .... Area of stiffener

$A_{xx,3D}$ .... Area of 1 wavelength of corrugation

b .... Horizontal projection of the inclined panel width

b_f .... Width of flange

b_{eff} .... Effective width

BM .... Bending moment

BOA .... Based on area

BOS .... Based on stiffeness

c .... Inclined panel width

C .... Spacing of stiffeners in the web

c/c .... Center to center

CG .... Center of gravity

$C_{w,co}$ .... Warping constant of I girder with corrugated web

$C_{w,Flat}$ .... Warping constant of I girder with flat plate

d .... Corrugation depth

D .... Girder height

DL .... Dead load

E_c .... Modulus of elasticity of concrete

E_c .... Modulus of elasticity of steel

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

f_{st} .... Characteristic tensile strength of steel

f_t .... Maximum principle tensile stress

F_s .... Design tensile force

F_{bs} .... bursting tensile force

F_{wd} .... Design stress of weld

G .... Shear modulus of flat steel web.

G_{eq} .... Equivalent shear modulus

$G_{w,co}$ .... Shear modulus of corrugated steel web.

h_w .... Web height

h_w/d .... Web height to corrugation depth ratio

IRC .... Indian Road Congress

IS .... Indian Standard

$I_{x,co}$ .... Second moment of inertia about major axis

I_{xx} .... Torsion moment of inertia

$I_{y,co}$ .... Second moment of inertia about week axis

$I_{yy,3D}$ .... MOI of 1 wavelength

I_{yy} .... MOI about minor axix

I_{zz} .... MOI about main axix

J_{co} .... Pure torsional constant

K .... Ratio of short to long span (B/L)

k_G .... Global shear buckling coefficient

k_I .... Interactive shear buckling coefficient

k_L .... Local shear buckling coefficient

$2L$ .... 2 Legged stirrups

L .... Length of bridge

LL .... Live load

L_h .... Horizontal wavelength

L/D .... Span to girder depth ratio

m .... Modular ratio

$m1\&m2$ .... Coefficient for moments along the short and long spans

M_B .... Moment in the short span direction

M_L .... Moment in the long span direction

M_{cr} .... Lateral torsional buckling moment

n .... length reduction factor

NA .... Neutral axix
P .... Prestressing force
PC .... Prestressed concrete
r_{eq} .... According effect factor
SF .... Shear force
T .... Torsion moment
t_f .... Thickness of flange
t_w .... Web thickness
t_{eq} .... Equivalent thickness of 2D web
V_u .... Vertical shear force
V_{cr} .... Shear force corresponding to web buckling
V_t .... Torsional shear stress
V_{tc} .... Permissible torsion shear stress
w .... Maximum fold width
W .... Total load
Z .... Section modulus
τ_y .... Yield stress
τ_{cr}^{ie} .... Inelastic buckling stress
τ_{cr}^{e}, G .... Elastic global buckling stress
τ_{cr}^{e}, I .... Elastic interactive buckling stress
τ_{cr}^{e}, L .... Elastic local buckling stress
σ_{cbc} .... Permissible stress in concrete
σ_{st} .... Permissible stress in steel
β .... Global buckling factor
θ .... Corrugation angle
γ_m .... Partial safety factor
ν .... Poisson's ratio

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Chapter 1

Introduction

1.1 General

The history of the development of the bridges is closely associated with the history of human civilization. The bridge builders have always a desire to build new type of bridges, either new in concept or new in technique of construction.Prestressed concrete bridges are used for long spans but the weight and cost of the bridges are more, so the new concept are required for the reduction in the weight and the cost of the bridges.In Prestressed concrete bridges with corrugated steel webs, concrete web is replaced by the corrugated steel plate, thus forming a composite structure with a box section. A typical layout of such bridge is shown in Fig 1.1(a).By replacing the concrete webs with corrugated steel plates, the following benefits can be obtained.
(1) Reduced the self-weight of main girder (25%). (2) Improving the efficiency of main girder. (3) Improved shear resistance. (4) Reduced the manpower in construction work. (5) Reduced the cost.

This type of prestressed concrete box-girder bridges with corrugated steel webs is a major improvement on traditional prestressed concrete box-girder bridges.

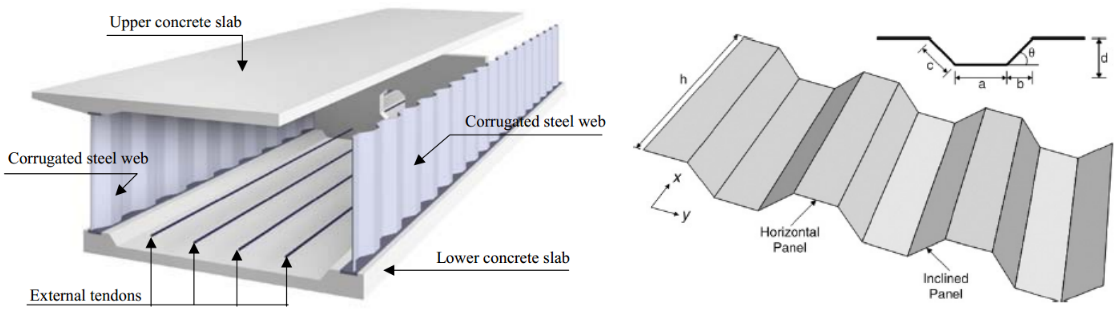


Figure 1.1: (a)Layout of bridge and (b)Corrugated plate profile

This concept is first develop in France in 1980's and built in 1986, the performance of this type of bridge has been receiving deserved attention and has proved its superiority over the others. And later this concept introduced to Japan in the 1990's. When continuous tendons are used, since the web is made of steel plate, it is provided by means of external tendons. Trapezoidal corrugated steel plate profile is as shown in Fig1.1(b).

1.2 Characteristics of Box girder Bridge with Corrugated Steel Webs

The introduction of prestressing plays an important role in PC bridges. In bridges with corrugated steel web, it is possible to reduce the cross-sectional area of concrete that resist the flexural and axial forces. Considering the mechanical characteristics of corrugated web, one of the features is that the concrete slab and steel plate can be treated separately. The corrugated plate is much more flexible than concrete, thus resulting in an "according effect", the forces are transmitted to the concrete slab. The steel plates have considerable ability to resist shear forces. Also corrugated plates have sufficient rigidity to resist buckling forces in out of plane direction [1] [2].

Structural Behavior

- **Flexural Behavior**

As shown in Fig.1.2, the axial stiffness of corrugated steel webs can be neglected in engineering point of view. Furthermore, only upper and lower concrete slabs are considered on resisting the axial forces and bending moments as shown in Fig. 1.3. Based on the many experiments and analysis, the assumption that plane sections remain plane was verified and the similar ultimate flexural moment between corrugated web bridges and conventional prestressed concrete bridge was also verified. Therefore, apart from ignoring the stiffness of corrugated steel webs, the design for bending moments and axial forces is the same as the conventional prestressed concrete bridges.

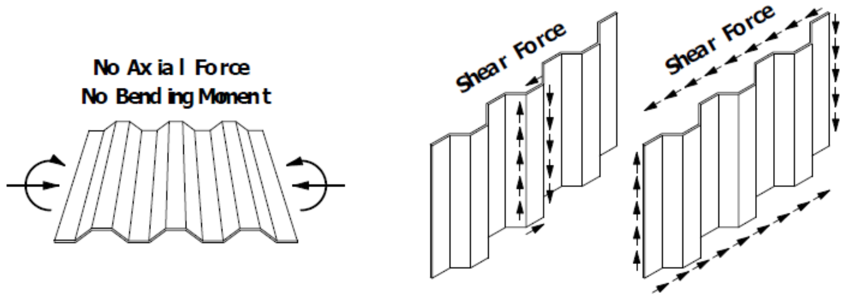


Figure 1.2: Properties of Corrugated Webs

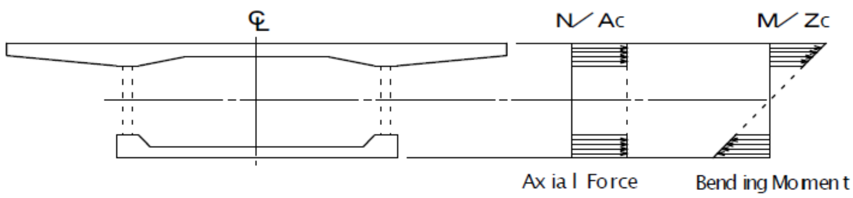


Figure 1.3: Cross Section for Axial Forces and Bending Moments

- **Shear Behavior**

As shown in Fig.1.4, the shear forces are resisted by the corrugated steel webs. Based on the experiments and analysis carried out to date, it has been confirmed that the applied shear forces are mostly resisted by the corrugated steel webs. Therefore, the shear forces are designed by assuming that all applied shear forces are resisted by the corrugated steel webs as shown in Fig.1.4, which is certainly on the safe side.

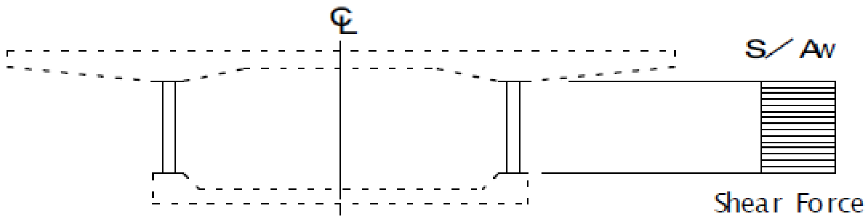


Figure 1.4: Effective Cross Section for Shear Forces

- **Shear Buckling Behavior**

There are three modes of shear buckling of corrugated steel webs as shown in Fig.1.5.Since no post buckling strength can be expected from corrugated steel webs, it is necessary to design the webs without buckling at the ultimate limit state. Formulae and analysis methods for calculating the strength have been proposed for these three buckling modes, and their validity has been verified in many experiments.

a. Local buckling: Mode in which buckling occurs between fold lines of the corrugated steel web.

b. Global buckling: Mode in which the entire corrugated steel web buckles.

c. Interactive buckling: This mode which is a composite of the above two shapes

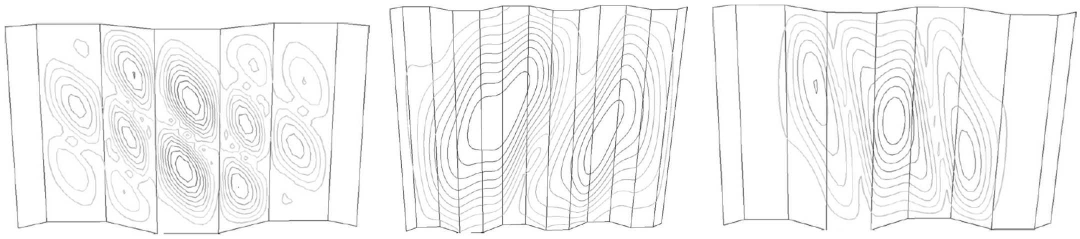


Figure 1.5: Local, Global and Interactive Buckling

- **Torsional Behavior**

Compared with the conventional prestressed concrete box girders, the stiffness in out-of-plane direction of corrugated steel webs is relatively small. Thus, the cross-section tends to deform easily as shown in Fig.1.6. When the cross-section deforms, it causes a reduction in the cross-sectional stiffness or increases warping torsional stresses. Therefore, on curved or skewed bridges it is necessary to place the diaphragms at suitable intervals in order to restrict the cross-sectional deformation. Past researches showed that the effect of cross-sectional deformation is virtually eliminated when the diaphragms are reinforced at suitable intervals.

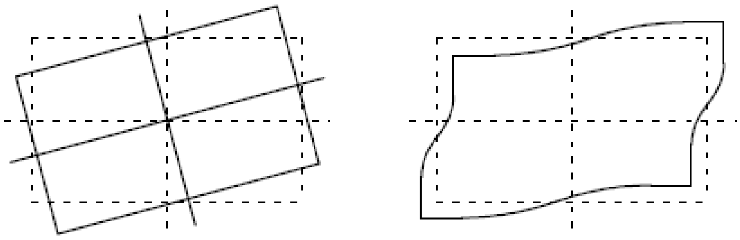


Figure 1.6: St.venant Deformation and Warping Deformation

Method of Connection Between Concrete Slab and Web

The most important part of a PC bridge with corrugated steel webs is the connection between concrete slab and steel plate. For this type of structure to be feasible, it is necessary to transfer the shear force acting on the corrugated steel plates to

concrete slab completely. Hence, if any damage or failure occurs in this part, the structure will loose its performance, and this connection very important. In hybrid structures, the connections between the concrete and steel greatly affect on the structural performance and cost. Initially studs or angle shear connectors were used to connect the concrete slabs and corrugated steel webs.
Following methods of connections are used.

- **Stud Dowel on Flange Plate:**

This method is the most commonly used for composite connection as shown in Fig.1.7. There is not much concern when this method is used for the upper slab, but in case of lower slab, it is necessary to properly evaluate the strength of this connection.

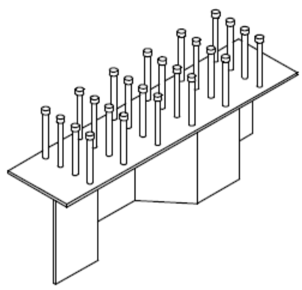


Figure 1.7: Stud Dowels on Flange Plate

- **Direct Embedding of the Plate in Concrete:**

This method was developed considering the construction cost efficiency. It is necessary to evaluate the shear transfer when the steel plate is directly embedded in concrete by appropriate methods as shown in Fig.1.8(a). However, since the corrugated steel plate is directly embedded in concrete, sufficient care should be taken on the maintenance of the interface.

- **Angles Dowels on Flange Plate:**

This method has been adopted in bridges in France. However, it is necessary to weld the angles on the steel flange as shown in Fig.1.8(b), thus increasing the fabrication cost of the connectors.

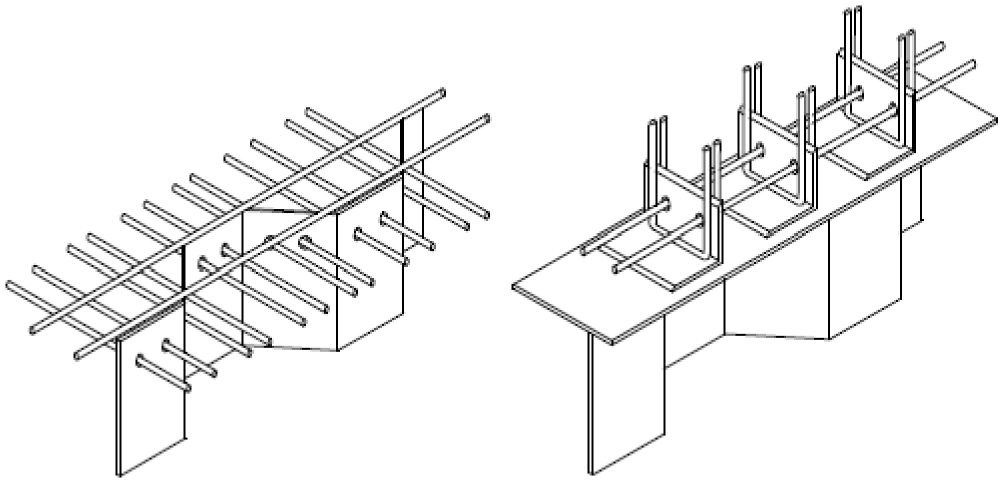


Figure 1.8: (a)Direct Embedding in Concrete (b)Angles Welded to Flange Plate

- **Perfobond Strip Connection:**

A perfobond strip connection is a connection using a plate with holes as shear connector as shown in Fig.1.9. Compared with stud connectors, the stiffness of shear connection is higher. This connection is a comparatively economical because welding of the shear connector is simpler. Nevertheless, the combination between plate and studs is frequently applied since the plate cannot solely resist transverse bending moments.

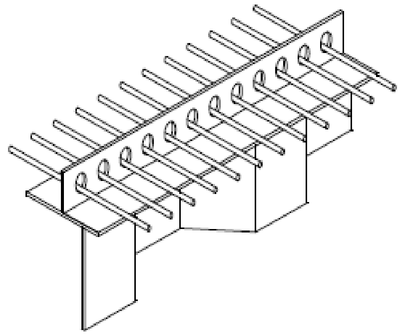


Figure 1.9: Perforated Strip Connection

1.3 Case Studies

- **Shinkai Bridge:**

The Shinkai Bridge, as shown in Fig.1.10(a), was the first corrugated web prestressed concrete bridge built in Japan. It is single span box girder bridges with length of 31m, span of 30m, and width of 14.8m. As shown in Fig.1.10(b), the erection method was the launching girder method, in which the girders were constructed at an on-site fabrication yard. The connections between concrete slabs and webs were stud shear connectors, and the joints between corrugated steel plates were butt welding. Erection of the girders was carried out using equipments to ensure that no torsional moment would be subjected to the girders.

- **Ginzan-Miyuki Bridge:**

The Ginzan-Miyuki bridge in Akita Prefecture, as shown in Fig.1.11 (a), was the second corrugated web bridge constructed in Japan. The construction of this bridge was the incremental launching method using the main girder cross-section as a launching nose with cable supported from pylon towers as shown in Fig.1.11 (b). This

Figure 1.10: Shinkai Bridge (a) completed and (b) during erection

bridge was a five spans continuous girder bridge with length of 210.0m and maximum span of 45.5m. In addition, this bridge was the first corrugated web continuous girder bridge in Japan. The connections between the concrete slabs and corrugated steel plates were stud shear connectors, and the joints between corrugated steel plates were single shear friction with additional plates.

Figure 1.11: Ginzan-Miyuki Bridge (a) completed and (b) during erection

- **Hondani Bridge:**

The Hondani bridge, as shown in Fig.1.12(a), was the third corrugated web bridge constructed in Japan. It is three spans continuous prestressed concrete rigid frame box Girders Bridge, with a length of 198.2m, maximum span of 97.2m, and width of 11.04m. Erection was carried out by the cantilever method. In addition, the connections between the concrete slabs and webs were embedded connection as shown in Fig.1.12 (b), and the joints between web plates were single shear friction joints.


Figure 1.12: Hondani Bridge (a) Completed and (b) Embedded Connection

- **Cognac Bridge:**

The Cognac Bridge, as shown in Fig.1.13 was the world's first corrugated steel web bridge built in France and completed in 1986. It is three spans continuous box girder bridge with the total length of 105m and maximum span of 43.0m. The cross-section is a box with the height of 2.285m with both upper and lower slabs made of concrete and webs slanted at about 35 degree of 8mm thick corrugated steel plate. Construction was carried out by the fixed scaffolding method.

Figure 1.13: Cognac Bridge

1.4 Objective of Study

- To study the continuous prestressed concrete box girder bridge with corrugated steel webs.
- To study the difference in behavior of conventional PC box girder and replacing concrete webs with corrugated webs.
- To study the road bridge superstructure using different type of vehicular loading as per IRC 6:2010.
- To study the difference concept of design in corrugated steel web box girder with conventional PC box girder.
- To study the shear behavior of corrugated web bridge girder.
- To study the shear connectors design.
- To study the impact of span to depth ratio and h_w/d ratio of corrugated web box girder on the estimation of economical girder.
- To evaluate the unit weight.
- To evaluate the cost of the bridge.

1.5 Scope of Work

The scope of work for major project is decided as follows:

- Mainly two types of work have been carried out.

  a. Analysis

  b. Design

  For the PC box Girder Bridge and replacing the concrete webs with the corrugated webs.

- Analysis of super structure is carried out based on Staad pro and design is done using the excel work sheet.

- For analysis and design 2 span continuous rectangular box girder is to be selected.

- For analysis live load is considered as per IRC 6:2010.

- Design of PC box Girder Bridge is done as per IRC 18:2000.

- Design of PC box Girder Bridge with corrugated webs is done as per IRC 22:2008.

- Evaluate change in design by changing span to depth ratio and h_w/d ratio in such a way that it become most economical box girder.

- Compare the unit weight between the conventional PC box girder and corrugated web box Girder Bridge.

- Compare the cost effectiveness between the conventional PC box girder and corrugated web box Girder Bridge.

1.6 Problem Formulation

- The two span continuous superstructure is to be design for Conventional PC box girder bridge and replacing the concrete webs with corrugated steel webs.
- Span: 45 m
- Carriageway: 7.5 m
- Wearing coat: 80 mm
- Diaphragms: 7.5 m c/c in longitudinal direction

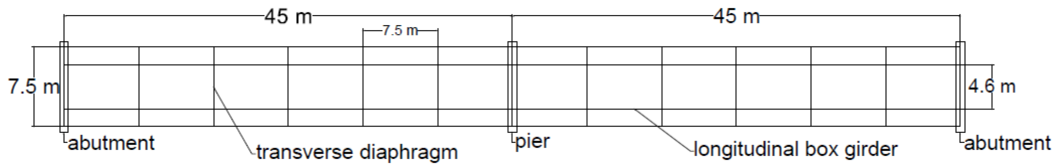


Figure 1.14: Longitudinal Section of Bridge

- PC box girder bridge

As shown in Fig.1.15

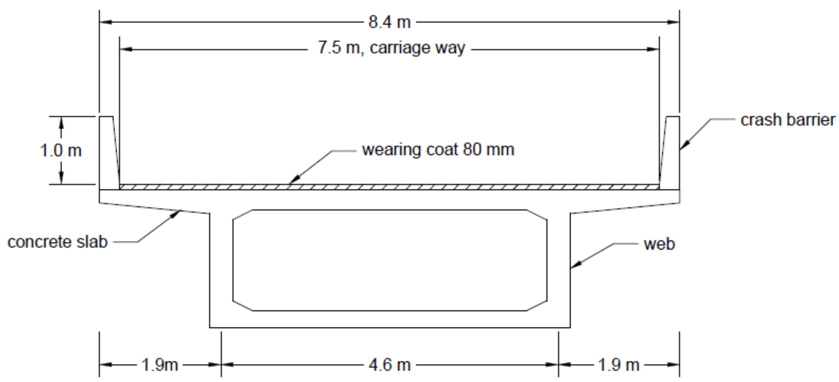


Figure 1.15: Bridge Cross Section

- PC box girder with corrugated steel webs

As shown in Fig.1.16

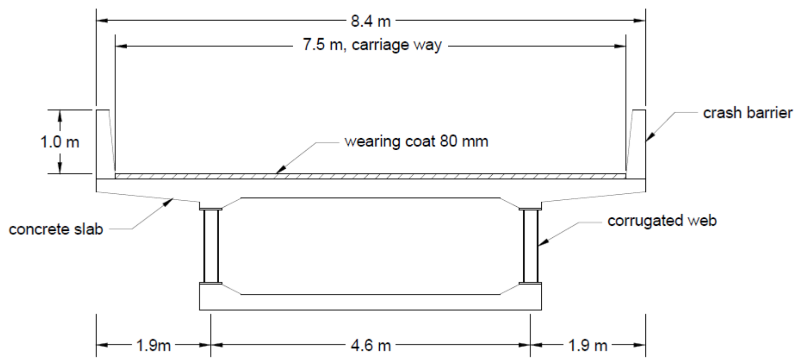


Figure 1.16: Bridge Cross Section

1.7 Organization of Major Project

Chapter 1:*Introduction*, Includes the introductory part of thesis, objective and the scope of work.

Chapter 2:*Literature Review*, in this chapter, review of relevant literature is carried out. The review of literature includes, concepts of PC box Girder Bridge with corrugated steel webs.

Chapter 3:*Structural Behavior of Corrugated Webs*, Includes shear buckling and lateral torsional buckling behavior of corrugated steel webs, and notation of different geometric properties of the webs, different cross sectional properties of corrugated steel webs.

Chapter 4:*Box girder bridge*, Include the analysis and design of the different parts of the conventional bridge. In this chapter cost estimation and unit weight also calculated.

Chapter 5:*Box girder bridge with corrugated steel webs*, Include the analysis and design of the longitudinal girder with corrugated web. In this chapter cost estimation and unit weight also calculated.

Chapter 6:*Parametric study*, Include the parametric study for 40m, 45m, and 50m span with various L/D ratio to find out the economical L/D ratio. Here also h_w/d ratios taken into account for the find out the buckling behavior of the corrugated web.

Chapter 7: Includes summary, conclusion and future line of action for major project.

Chapter 2

Literature Review

2.1 General

Literature survey is carried out to know the actual behavior of trapezoidal corrugated steel plate when it used in prestressed concrete box Girder Bridge. The different analytical and experimental modal are prepared by various authors to simulate the actual behavior of composite box Girder Bridge. In literature survey, main emphasis is given on various books, IRC codes, published papers.

2.2 Literature Review

Various literatures have been referred for behavior of corrugated steel webs and brief review of literature is discussed below.

Hisao Tategami et al.1: This paper "Recent trend of prestressed concrete box girder bridges with corrugated steel webs in japan" includes characteristics of PC bridges with corrugated steel web, the evaluation of sectional stiffness of such bridges, method of connection between concrete slab and steel web, method of connection between the corrugated steel plates.

Shoji Ikeda et al.2: This paper "Development of hybrid prestressed concrete bridges with corrugated steel web construction" include features of corrugated web bridges, structural behavior of corrugated web box girder, connections, joints of corrugated web, example of corrugated web bridges.

Jiho Moona et al.3: This paper presents the "Shear strength and design of trapezoidally corrugated steel webs". Derive the global shear buckling coefficient, The interactive shear buckling coefficient and the shear buckling parameter for corrugated steel webs are then proposed based on the 1st order interactive buckling equation.

Jongwon Yi et al.4: This paper include "Interactive shear buckling behavior of corrugated webs". The interactive buckling is rather complex and is an intermediate type of shear buckling between local buckling and global buckling, which involves several panels. In this study, a series of finite element analyses was carried out to study the geometric parameters affecting interactive shear buckling modes and strength. Based on the analysis results, the interactive shear buckling strength formula is proposed.

Jiho Moon et al.5: This paper "Lateral- torsional buckling of I-girder with corrugated webs under uniform bending" described the bending and the pure torsional rigidity. Then the location of the shear center and calculating the warping constant are proposed. Using the proposed method, the lateral torsional buckling strength of I girder with corrugated webs under uniform bending can be calculated easily.

Y. L. Mo et al.6: In this paper "Torsional design of hybrid concrete box girders" a series of systematic tests on hybrid concrete box girders subjected to torsion has been performed. According to the test results, an analytical model was developed.

Using the developed analytical model, a step-by-step procedure for torsional design of such bridges is presented in this article. Based on the design procedure proposed, a girder is designed by the analytical model and checked to satisfy structural codes.

Bertagnoli Gabriele et al.7: In this paper "orthotropic modal for the analysis of beam with corrugated steel webs" include finite element modal for steel and composite beams with corrugated webs. A parametric study of the 2D flat equivalent model is presented in order to investigate the influence of geometric parameters on the structural behavior and to check the robustness of the simplified method in predicting different failure modes.

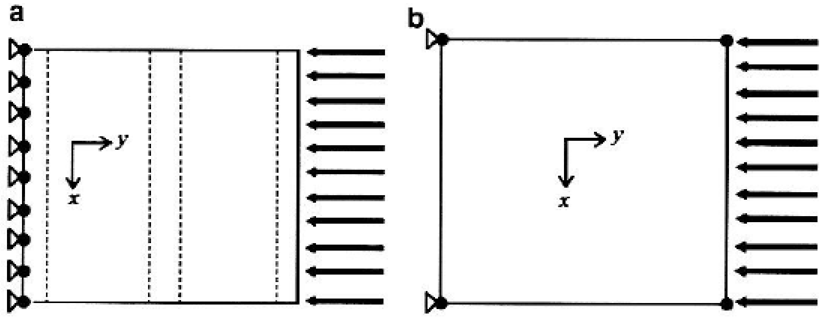


Figure 2.1: Loading and Boundary Condition of 3D and 2D Plate

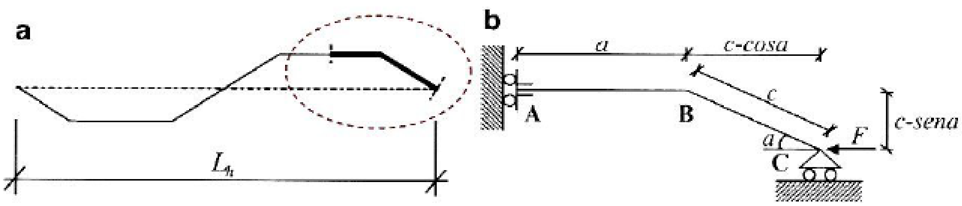


Figure 2.2: Wavelength of Corrugated Web and Equivalent Static Scheme

$$r_{eq} = \frac{\Delta_{y,3D}}{\Delta_{y,flat}} = 48\frac{c^2 sin^2 \alpha t_{eq}}{L_h t^3}\left(a + \frac{c}{3}\right) \tag{2.1}$$

$$G_{eq} = G\frac{t_{eq}}{t} = \frac{E}{2(1+\nu)}\frac{t_{eq}}{t} \tag{2.2}$$

$$E_x I_{yy,3D} = E_x \frac{t_{eq}^3}{12} \tag{2.3}$$

$$E_x A_{xx,3D} = E_x t_{eq} \tag{2.4}$$

$$t_{eq} = \sqrt{\frac{12 I_{yy,3D}}{A_{xx,3D}}} \tag{2.5}$$

$$E_{x,eq} = \frac{A_{xx,3D} E}{t_{eq} L_{x,eq}} \tag{2.6}$$

$$E_{y,eq} = \frac{E}{r_{eq}} \tag{2.7}$$

Where,

r_{eq} = According effect factor.

G_{eq} = Equivalent shear modulus.

$I_{yy,3D}$ = MOI of 1 wavelength.

$A_{xx,3D}$ = Area of 1 wavelength.

L_h = is the wavelength shown in Fig2.2.

E = Modulus of Elasticity.

t_{eq} = Equivalent thickness of 2D web.

KwangHoe Jung et al.8: In this paper "Verification of incremental launching construction safety for the ilsun bridge, the world's longest and widest prestressed concrete box girder with corrugated steel web section" to verify the construction safety of the Ilsun Bridge, this investigation focuses on the span-to-depth ratio, buckling shear stress of the corrugated steel webs, optimization of the length of the steel launching nose, detailed construction stage analysis, and the stress level endured by the corrugated steel webs during the launching process.

Handbook on Composite Construction9: This book has described introduction, advantage of steel-concrete composite construction, composite action in beam, effective width, modular ratio, resistance to vertical shear, resistance to combined bending and shear, different type of shear connectors with deformation design and detailing. Also it contains codal stipulation design procedures like, design of deck slab, longitudinal girder, cross bracing and shear connectors and four different design examples of I-girder. The property tables for composite sections and pigeaud's curves are also given in this handbook.

Design of bridges10: This book includes the analysis and design procedures of prestressed concrete bridges and the composite bridges.

Design and Construction of highway bridges11: This book includes the analysis and design procedures of prestressed concrete bridges and the composite bridges.

Essentials of bridge engineering12: This book includes the analysis and design procedures of prestressed concrete bridges and the composite bridges.

Grillage analogy in bridge deck analysis13: This book includes the analysis of different types of bridges with different vehicular loading.

Design of steel structure14: This book has described the procedure involved in designing structural components like tension member, compression member, member subjected to flexure like gantry girder and plate girder. Typical problems have been solved using limit state design method as per IS: 800-2007.

2.3 Relevant Codes

Following IRC and IS codes shall be used for design of road bridge superstructure.

IRC 6:2010: in this code "This code includes loading on bridges".

IRC 18:2000: in this code "This code includes design of PC Box girder bridges".

IRC 21:2000: in this code "This code includes design of concrete bridges according to limit state".

IRC 22:2008: in this code "This code includes design of composite bridges according to limit state".

IS 1343:1980: in this code " This code includes design of prestressed concrete members".

IS 800:2007: in this code "This code includes design on steel members according to limit state method".

IS 456:2000: in this code "This code includes design r.c.c members according to working stress method and limit state method".

Chapter 3

Structural behavior of corrugated webs

3.1 General

The use of corrugated webs is a potential method to achieve adequate out-of-plane stiffness and shear buckling resistance without using stiffeners; therefore, it considerably reduces the cost of beam fabrication and the weights of superstructures. The efficiency of prestressing is enhanced because the corrugated web carries only shear forces and the flanges carry the moment due to the accordion effect. In order to benefit from these characteristics, prestressed concrete box girder bridges with corrugated webs are used extensively.

Name of bridge	a(mm)	b(mm)	d(mm)	c(mm)	n	w/h_w	d/t_w
Sinkai bridge	250	200	150	250	0.90	0.21	16.67
Matunoki bridge	300	260	150	300	0.93	0.14	15
Hondani bridge	330	270	200	330	0.91	0.10	22.22
Cognac bridge	353	319	150	353	0.95	0.20	18.75
Maupre bridge	284	241	150	284	0.92	0.11	18.75
Dole bridge	430	370	220	430	0.93	0.17	22

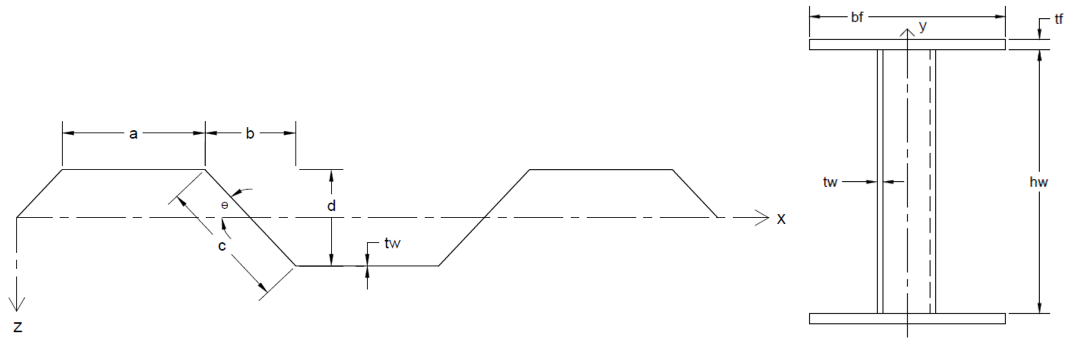


Figure 3.1: Geometric notation of corrugated web

3.2 Elastic Shear Buckling of Trapezoidally Corrugated Steel Webs

The shear strength of corrugated web I-girder is primarily a function of the web height and thickness, the corrugation geometry, and materials properties, although initial web geometric imperfections may also play a significant role. The corrugations provide stability to the web, eliminating the need for the transverse stiffeners. For a corrugated steel web, it is assumed that the web carries only shear forces due to the accordion effect. Because of this characteristics, the corrugated steel webs fail due to shear buckling or yielding. The bending moment can reasonably be assumed to be carried entirely by the flanges. Thus shear strength can be determine without consideration of moment-shear interaction. There are three different shear buckling modes.

(1)Local buckling

(2)Global buckling

(3)Interactive buckling

- **Local Buckling:**

The presence of local shear buckling is characterized by the buckling of individual sub-panels as shown in Fig.3.2. It is assumed that corrugated webs are treated as a series of flat rectangular sub-panels supporting each other along their vertical edges and by the flange along their horizontal edges. The elastic local shear buckling stress of the corrugated webs,τ_{cr}^{e}, L can be determined by the classical plate buckling theory and expressed as.[3]

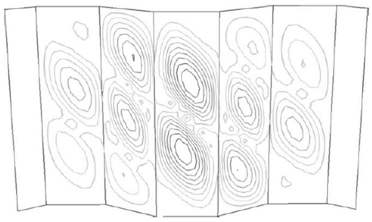


Figure 3.2: Local Buckling

$$\tau_{cr}^{e}, L = k_L \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t_w}{w}\right)^2 \tag{3.1}$$

Where,

E = Young's modulus of elasticity.

ν = Poisson's ratio.

w = The maximum fold width (maximum of flat panel width a and inclined panel width c.

t_w = The web thickness.

k_L = The local shear buckling coefficient, Assuming that the panel has simply supported edges, k_L is given by.

$$k_L = 5.34 + 4\left(\frac{w}{h_w}\right)^2 \tag{3.2}$$

k_L is a function of the aspect ratio of the sub-panel, w/h_w. It is found that the w/h_w on actual bridges that have been constructed to date, are generally smaller than 0.2, as shown in Table 1. The difference between Eq. (3.2) and kL = 5.34 is smaller than 2.9% when $w/h_w \leq 0:2$. Therefore, $k_L = 5.34$ is recommended for practical design purposes for fold simply supported edges and 8.98 for fold fixed edges

- **Global Shear Buckling behavior:**

Global shear buckling is characterized by the formation of diagonal buckles through the entire web similarly to a flat plate web. The global shear buckling stress of the corrugated webs by treating the corrugated web as an orthotropic flat web. Elastic global shear buckling stress of the corrugated steel webs τ_{cr}^{e}, G is given by [3].

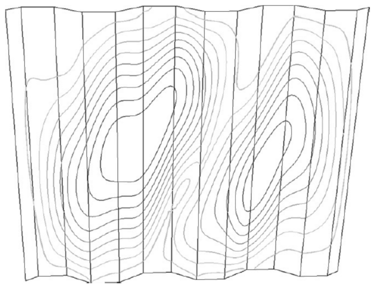


Figure 3.3: Global Buckling

$$\tau_{cr}^{e}, G = k_G \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t_w}{w}\right)^2 \tag{3.3}$$

$$k_G = \frac{36\beta}{\pi^2\sqrt{n}}[2((d/t_w)^2+1)(1-\nu^2)]^{3/4} \tag{3.4}$$

Where,

β=global buckling factor that depends on the boundary condition, 1 for simply supported and 2 for fixed edges.

n = length reduction factor = $\frac{a+b}{a+c}$

For $n = 1, \beta = 1, \nu = 0.3$ putting this values in above equation (3.4) and simplifying the k_G.

$$k_G = 5.72 \left(\frac{d}{t_w}\right)^{1.5} \tag{3.5}$$

It is found that the k_G can be simplified as a function of d/t_w only, for corrugated profiles that are used for the webs of bridges $\frac{w}{h_w} \leq 0.2$, $\frac{d}{t_w} \geq 10$.

When the local and global elastic shear buckling stressτ_{cr}^{e}, L and τ_{cr}^{e}, G exceed 80% of the shear yield stress τ_y inelastic buckling will occurs τ_{cr}^{ie}.

$$\tau_{cr}^{ie} = \sqrt{0.8\tau_y\tau_{cr}} \tag{3.6}$$

Where, $$\tau_{cr}^{ie} \leq \tau_y$$

- ### Interactive Shear Buckling Behavior:

Interactive shear buckling mode is attributed to the interaction between global and local shear buckling modes and governs the shear buckling strength. The buckled shapes of the interactive buckling mode are not as definitive as those of the local or global buckling mode but vary depending on the geometry of corrugated webs. The elastic interactive shear buckling stress of corrugated steel web, τ_{cr}^{e}, I is given by [3].

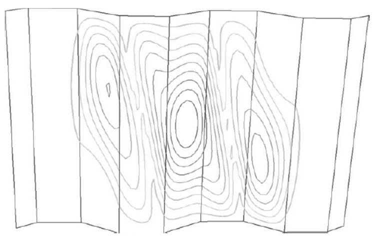


Figure 3.4: Interactive Buckling

$$\frac{1}{\tau_{cr}^{e}, L} + \frac{1}{\tau_{cr}^{e}, G} = \frac{1}{\tau_{cr}^{e}, I} \tag{3.7}$$

Substituting Eq.(3.1) and Eq.(3.3) into Eq.(3.7), τ_{cr}^{e}, I takes the form of the classical plate buckling equation and can be expressed as.

$$\tau_{cr}^{e}, I = k_I \frac{\pi^2 E}{12(1-\nu^2)} (\frac{t_w}{h_w})^2 \tag{3.8}$$

$$k_I = \frac{k_L k_G}{[k_L + k_G(\frac{w}{h_w})^2]} \tag{3.9}$$

For a practical design, k_I is simply calculated with $k_L = 5.34$ and $k_G = 5.72(d/t_w)^{1.5}$

$$k_I = \frac{30.54}{5.34(d/t_w)^{-1.5} + 5.72(w/h_w)^2} \tag{3.10}$$

3.3 Shear Buckling Design of Trapezoidally Corrugated Steel Webs

The elastic shear buckling strength of corrugated steel webs is controlled by the elastic interactive shear buckling strength.If the $\tau_{cr}^{e}, Iis > 0.8\tau_y a$ the inelastic buckling will occur and τ_{cr}^{ie}inilastic is given by below equation. Thus, the shear buckling parameter of corrugated webs, λ_s is defined as [3].

$$\tau_{cr}^{ie} = \sqrt{0.8\tau_{cr}^{e}, I\tau_y} \tag{3.11}$$

$$\lambda_s = \sqrt{\frac{\tau_y}{\tau_{cr}^{ie}}} \tag{3.12}$$

Where, τ_y is the shear yielding stress of the webs, Substituting Eq. (3.8) into Eq. (3.11), λ_s can be expressed as

$$\lambda_s = 1.05\sqrt{\frac{\tau_y}{k_I E}}(\frac{h_w}{t_w}) \tag{3.13}$$

Once a determination is made of λ_s, the shear buckling strength, considering material inelasticity, residual stress, and initial imperfections, can be determined from the buckling curve. The buckling curve is adopted from the design manual for PC bridges with corrugated steel webs. The buckling curve equations that were used are given by.

$$\frac{\tau_{cr}}{\tau_y} = 1 \qquad \lambda_s < 0.6 \tag{3.14}$$

$$\frac{\tau_{cr}}{\tau_y} = 1 - 0.614(\lambda_s - 0.6) \qquad 0.6 \leq \lambda_s < \sqrt{2} \tag{3.15}$$

$$\frac{\tau_{cr}}{\tau_y} = \frac{1}{\lambda_s^2} \qquad \lambda_s \geq \sqrt{2} \tag{3.16}$$

3.4 Lateral Torsional Buckling of Girder with Trapezoidally Corrugated Steel Webs

Generally, lateral-torsional buckling is a major design aspect of flexural members composed of thin-walled I-girders. When a slender I-girder is subjected to flexure about its strong axis with insufficient lateral bracing, out-of-plane bending and twisting may occur as the applied load approaches its critical value. At this critical value, lateral-torsional buckling occurs. This phenomenon occurs because the corrugated web is eccentrically attached to the flange.

Using the formula of the lateral-torsional buckling strength of the I-girder with flat webs with the corrugated web section properties, the elastic lateral-torsional buckling strength M_{cr} of the I- girder with corrugated webs can be expressed as [5].

$$M_{cr} = \sqrt{\left(\frac{\pi^2 E I_y}{L_{LT}^2}\right)\left(G_{co} J_{co} + \frac{\pi^2 E C_{wco}}{L_{LT}^2}\right)} \tag{3.17}$$

Where,

M_{cr} = Lateral torsional buckling moment.

E = Young modulus of elasticity.

$I_{y,co}$ = Second moment of inertia about week axis.

$I_{x,co}$ = Second moment of inertia about major axis.

$G_{w,co}$ = Shear modulus of corrugated steel web.

G = Shear modulus of flat steel web.

$C_{w,co}$ = Warping constant of I girder with corrugated web.

$C_{w,Flat}$ = Warping constant of I girder with flat plate.

J_{co} = Pure torsional constant.

$$G_{w,co} = \frac{a+b}{a+c}G = nG \tag{3.18}$$

$$J_{co} = \frac{1}{3}(2b_f t_f^3 + h_w t_w^3) \tag{3.19}$$

The variation in $C_{w,co}$ and $G_{w,co}$ with θ is as shown in Fig.(3.5). The y axis represents $G_{w,co}/G$ and $C_{w,co}/C_w$. It can be found that $G_{w,co}/G$ decreases with incensing θ and values of $G_{w,co}/G$ are less than 1. While $C_{w,co}/C_w$ increases with incensing θ and values of $C_{w,co}/C_w$ are greater than 1. This behavior implies that the warping constant of the I girder with corrugated webs is larger than that of the I girder with flat webs, while the shear modulus of the corrugated plates is smaller than that of the flat plates [5].

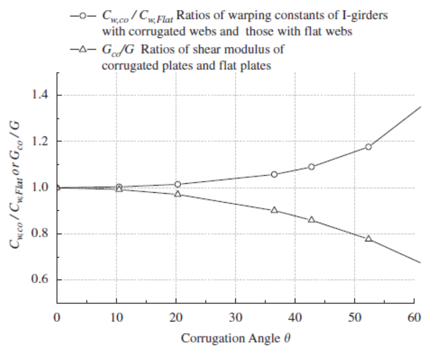


Figure 3.5: Variation in $G_{w,co}/G$ and $C_{w,co}/C_w$ with θ

The variation in M_{cr} with θ is as shown in Fig.(3.6). It can be found that $M_{cr}/M_{cr,flate}$ increases slightly with increasing θ. The lateral torsional buckling strength of the I girder with corrugated webs are larger than that of the I girder with flat webs when $\theta = 60 degree$ [5].

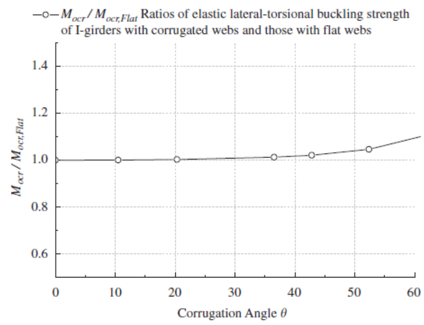


Figure 3.6: Variation in $M_{cr}/M_{cr,flate}$ with θ

Lindner has suggested an empirical formula of the warping constant of the I-girder with corrugated webs $C_{w,co}$, which is defined as [5].

$$C_{w,co} = C_{w,Flat} + \frac{C_w L^2}{E\pi^2} \tag{3.20}$$

$$C_w = \frac{(2d_{max})^2 h_w^2}{8u_x(a+b)} \tag{3.21}$$

$$u_x = \frac{h_w}{2Gat_w} + \frac{h_w^2(a+b)^3(I_{x,co}+I_{y,co})}{600a^2 E I_{x,co} I_{y,co}} \tag{3.22}$$

Where,

$d_{max} = d/2$.

Chapter 4

Box Girder Bridge

4.1 Structural data

Effective span of bridge = 45m

No of span = 2

Total width of bridge = 8.4m

Carriageway width = 7.5m

Total depth of bridge = 2030mm

L/D ratio = 22.16

Wearing coat = 80mm

Thickness of deck slab = 300mm

Thickness of Soffit slab = 230mm

Thickness of Intermediate Diaphragm = 350mm

Clear depth of diaphragm = 1500mm

Spacing of diaphragm = 7.5m

Web thickness = 350mm

Web thickness at the support = 500mm

C/C distance between web = 4.6m

Size of haunch = $300 * 150$mm

Concrete grade for slab = $25 N/mm^2$

Concrete grade for web = $60 N/mm^2$

Steel grade = 415 N/mm^2

Density of concrete = 25 kN/m^3

Density of wearing coat = 22 kN/m^3

In this study, cross section taken for analysis is as shown in Fig4.1

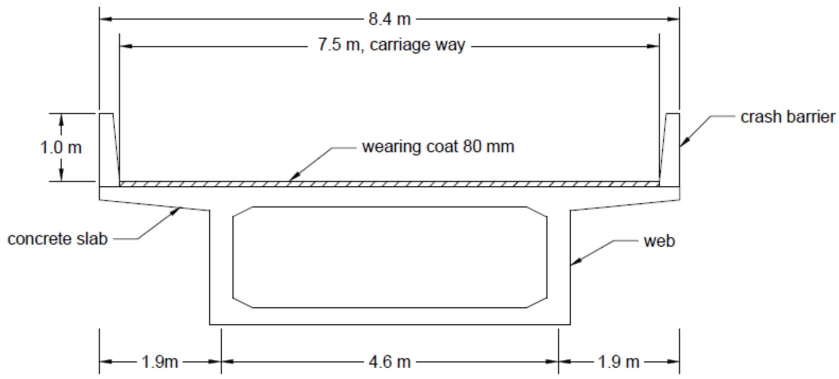


Figure 4.1: Cross Section of Bridge

In this study, longitudinal section taken for analysis is as shown in Fig4.2

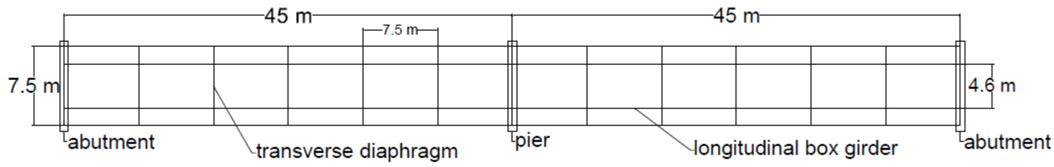


Figure 4.2: Longitudinal Section of Bridge

4.2 Loading on Bridge

The section - II of I.R.C. gives the specifications about the load and stresses applicable while designing the road bridges. The following loads, forces and stresses should be considered in design, where applicable:

a. Dead Load

b. Live Load

c. Impact or dynamic effect of live load

d. Wind load

e. Longitudinal forces caused by the tractive effort of vehicles or by breaking of vehicles.

f. Longitudinal forces due to frictional resistance of expansion bearings.

g. Centrifugal forces due to curvature

h. Horizontal forces due to water currents

i. Buoyancy

j. Earth pressure

k. Temperature stresses

l. Secondary stresses

m. Erection stresses

n. Forces and effects due to earthquake

Following loads are taken

Dead Load

Live Load

According to IRC classification, the main live loads for road bridges can be put into the following types.

IRC Class-AA loading

IRC Class-AA loading comprises either a tracked vehicle of $700kN$ or wheeled vehicle of $400kN$ loads. Fig.4.3(a) shows the class-AA tracked vehicle and Fig 4.3(b), shows the class-AA wheeled vehicle. All bridges located on National highways and State highways have to be designed for this heavy loading.

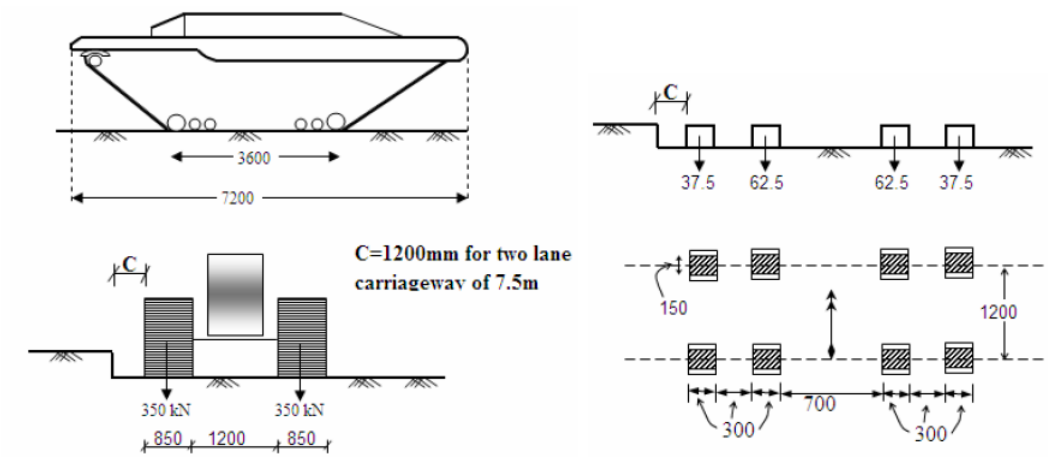


Figure 4.3: (a)IRC class AA Tracked vehicle (b)IRC class AA Wheeled vehicle

IRC Class-A loading

IRC Class-A loading consist of a wheel load train comprising a truck with trailers of specified axle spacing and loads as detailed in Fig.4.4 This type of loading is adopted on all roads on which permanent bridges and culverts are constructed

- **Impact**

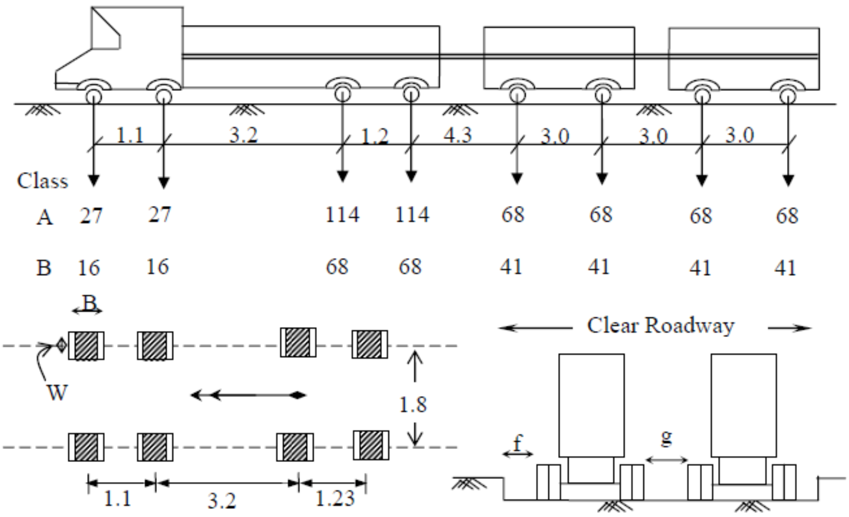


Figure 4.4: IRC class A Train of vehicle

In order to account for the dynamic effects of the sudden loading of a vehicle onto a structure, an impact factor is used as multiplier for certain structural elements. Live load stresses are then multiplied by this factor. Impact factor increases live load values.

For Class A and Class B Loading:

Impact factor for reinforced concrete bearing = $4.5/(6 + L)$

Where, L is length in meters of the span.

x-direction = 1.088

y-direction = 1.439

For Class AA and Class 70R Loading:

a. **For Span less than 9 m**

- Tracked vehicles: 25% for span up to 5 m linearly reducing to 10% for spans of 9 m
- Wheeled vehicles: 25%

b. **For Span of 9 m or more for reinforced concrete bridges**

- Tracked vehicles: 10% for span up to 40 m and in accordance with the curve Fig. for spans in excess of 40 m.
  x-direction = 1.1
  y-direction = 1.25

- Wheeled vehicles: 25% for spans upto 12 m and in accordance with the curve in Fig. for spans in excess of 12 m.
  x-direction = 1.08
  y-direction = 1.25

4.3 Analysis of Box Girder Bridge

The analysis is done for Deal Load, Super imposed dead Load, vehicle load Class AA and Class A IRC vehicle cases in Staad pro. From the software we have taken maxi- mum bending moment and shear force and torsion for the design of PC girder. For the design force consideration we have taken carriageway combinations as 1.5DL + 2.5*impact* Live Load. As our carriageway width is 5.3 and above but less than 9.6 for that purpose we have considered live load combination of either one lane of class AA tracked or 2 lane of Class A vehicle on carriageway.

4.3.1 Analysis of Deck Slab

Bridge Deck provides the surface on which traffic passes. For sample calculation of deck slab, two way spanning of slab is taken.
Data:
Span = 90m

No of longitudinal girder = 2
C/c spacing of longitudinal girder = 4.6 m
Spacing of cross girder = 7.5m
As the ratio of longer dimension to shorter dimension is 7.5/4.6 = 1.63, therefore the slab is considered as two way slab.

The deck slab is analyzed for D.L and L.L. The dead load consists of self weight, super imposed dead Load, and vehicle load as Class AA and Class A IRC vehicle cases are taken. In calculation of bending moment and shear force vehicles are adjusted in such a way that it gives maximum force in element. The analysis is done for Deal Load, Super imposed dead Load, vehicle load Class AA and Class A IRC vehicle cases are taken. Deck slab is further divided in slab panels and cantilever slab.

The deck slab panel is designed as two way slab using Pigeaud's curves. The bending moment are computed as equation 4.1 and 4.2

$$M_B = W(m_1 + \mu m_2) \tag{4.1}$$

$$M_L = W(m_2 + \mu m_1) \tag{4.2}$$

Where,
K = Ratio of short to long span (B/L)
M_B = Moment in the short span direction
M_L = Moment in the long span direction
$m1\&m2$ = Coefficient for moments along the short and long spans
μ= Poisson's ratio for concrete generally assumed as 0.15
W = Load from the wheel under consideration

Dead Load Analysis

Wearing coat $= 1.76kN/m^2$

Self weight of slab $= 7.5kN/m^2$

Total weight $= 9.26kN/m^2$

Total weight = W $= 281.39kN$

$B = 4.25m$

$L = 7.15m$

$u = 4.25m$

$v = 7.15m$

$u/B = 1$

$v/L = 1$

$k = (B/L) = 0.59$

$1/k = 1.68$

Using pigeaud's curve for fully loaded panel with udl.

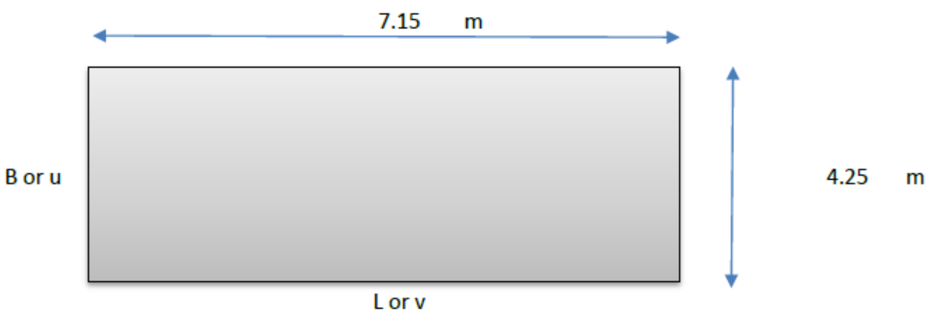


Figure 4.5: Slab Panel

using $k, m_1 = 0.05$

using $(1/k), m_2 = 0.015$

$M_B = W(m_1 + \mu m_2) = 14.70kNm$

$M_L = W(m_2 + \mu m_1) = 6.33kNm$

Live Load Analysis

- **Class AA tracked vehicle**

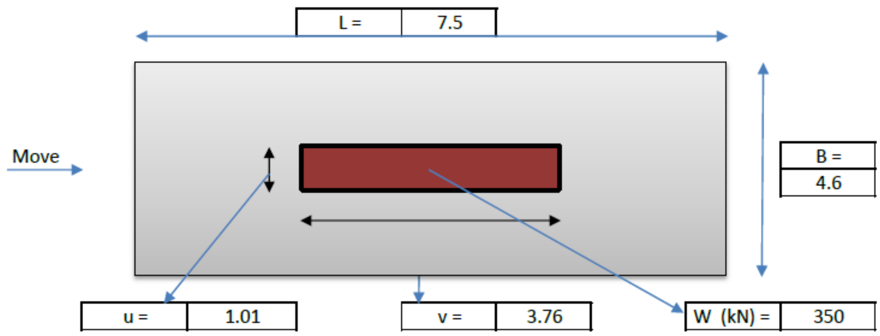


Figure 4.6: Position of Vehicle

Total weight = W = $350kN$

$B = 4.6m$

$L = 7.5m$

$u = 1.01m$

$v = 3.76m$

$u/B = 0.22$

$v/L = 0.50$

$k = (B/L) = 0.61$

Using pigeaud's curve.

$m_1 = 0.14$

$m_2 = 0.048$

$M_B = W(m_1 + \mu m_2) = 51.52kNm$

$M_L = W(m_2 + \mu m_1) = 24.15kNm$

- **Class AA wheeled vehicle**

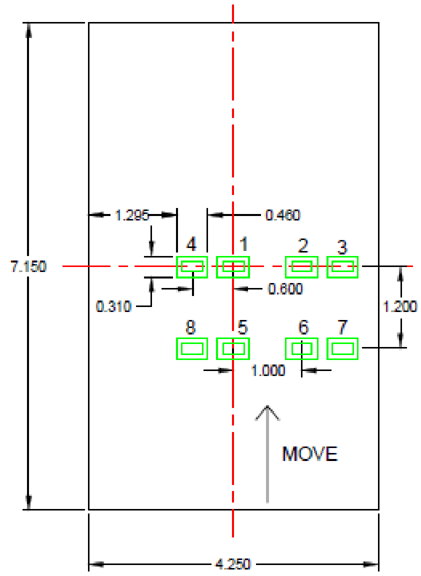


Figure 4.7: Position of Vehicle

Table 4.1: Class A vehicle moments

position ofwheel	$M_B(kNm)$	$M_L(kNm)$
1	17.87	17.34
2	4.78	6.20
3	1.05	2
4	5.47	5.58
5	7.06	1.21
6	3.58	4.65
7	0.79	1.50
8	4.1	4.39
Total	44.71	43.17

Table 4.2: Design Moments

	$M_B(kNm)$	$M_L(kNm)$
final design moments(DL+LL)	66.22	49.51

4.3.2 Analysis of Cantilever Slab

The cantilever deck slab is analyzed for D.L and L.L. The dead load consists of self weight, super imposed dead Load, vehicle load Class A is taken on the basis of the criteria of minimum clearance from crush barrier, as class A two-wheel live load will be critical on cantilever portion of deck slab. And maximum bending moment and shear force is calculated.

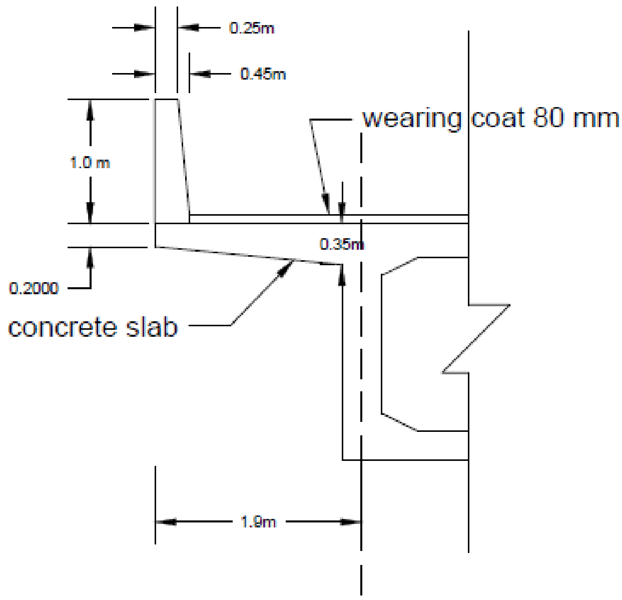


Figure 4.8: C/S of Cantilever Slab

Dead Load Analysis

self weight of slab = $6.875kN/m$

wearing coat = $1.76kN/m$

crash barrier = $8.75kN$, distance from support = $1.737m$

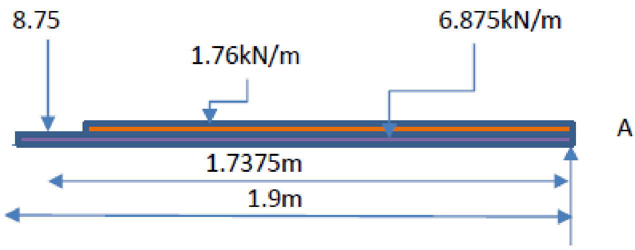


Figure 4.9: DL Loading Detail

Taking moment @ A BM = $29.46kNm$

Live Load Analysis

- **Class A wheeled vehicle**

In this case the axel load $114kN$ will give maximum moment.

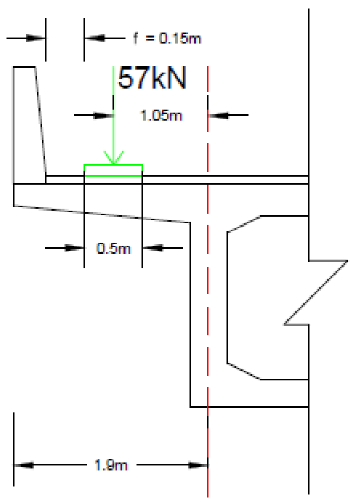


Figure 4.10: Vehicle Position

Effective width,$b_{eff} = 1.2x + b_w$

$x = 1.05m$

b_w = the breath of the concentration area of the load in the direction parallel to movement of vehicle.

$b_w = 0.41m$

$b_{eff} = 1.67m$

LL/m width including impact = $49.12kNm$

4.3.3 Analysis of Diaphragm

Diaphragm provides lateral stability to bridge deck. Diaphragm monolithic with deck slab shall be provided at the bearings and intermediate location on design requirements. The thickness of diaphragm shall not be less than the minimum web thickness of the main longitudinal girder. The depth of the diaphragm at bearings shall be suitably adjusted to allow access for proper inspection of bearings and to facilitate positioning of jacks for future lifting up of the super-structure.

Diaphragm is designed as intermediate and external diaphragm. Intermediate diaphragm is designed for dead load and vehicular load while as external diaphragm is designed for Jack force for lifting the super structure for replacement of bearing.

The diaphragm is analyzed for D.L and L.L.The dead load consists of self weight, super imposed dead Load, vehicle load Class AA Tracked is placed center of the transverse direction for getting maximum BM.

Dead Load Analysis

Weight of deck slab and wearing coat = $9.26kN/m^2$

DL of slab and diaphragm = $13.12kN/m$

self weight of diaphragm = $13.125kN/m$

Total DL = $26.24kN/m$

Live Load Analysis

- **Class AA tracked vehicle**

Load/track (W) = $350kN$

Load on diaphragm/track = $308.40kN$

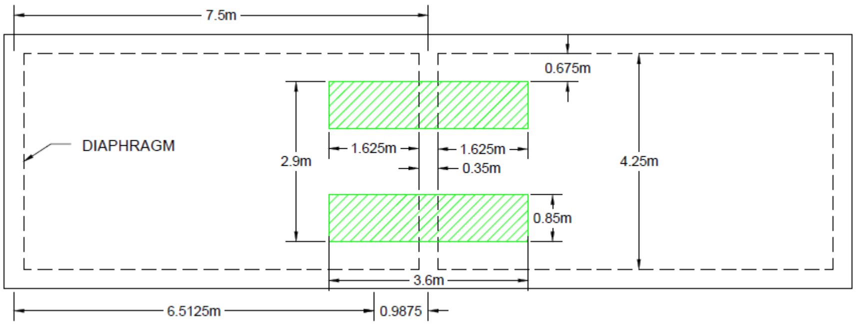


Figure 4.11: Vehicle Position

Load on diaphragm/track with impact = $385.50kN$

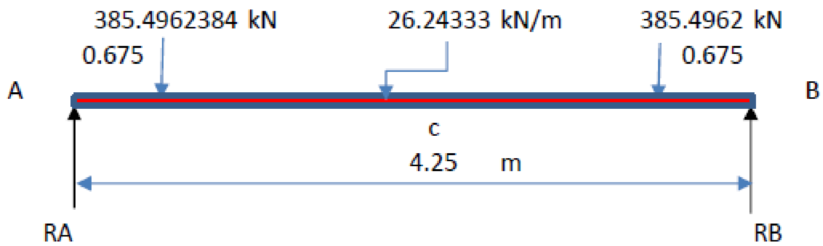


Figure 4.12: Loading Detail

$RA + RB = 882.53kN$

$RA = RB = 441.26kN$

Tacking moment @ C = BM = $319.46kNm$

4.3.4 Analysis of Longitudinal Girders

The analysis is done in Staad pro by grillage analogy method.By using the grillage analogy method the different section properties are to be calculated for different grillage members. The analysis is done for Deal Load, Super Imposed Dead Load, live load.In vehicles IRC Class A and IRC Class AA tracked vehicles are taken. In this thesis I have consider class AA track vehical but for 2 lane large span condition class 70R vehicle should be taken.

Grillage Analogy

Grillage analogy is probably one of the most popular computer-aided method for analysing bridges.The method consists of representing the actual system of bridge by an equivalent grillage of beams.The dispersed bending and torsional stiffnesses of the system are assumed for the purpose of analysis, to be concentrated in these beams.The stiffnesses of the beams are chosen so that the prototype bridge deck and the equivalent grillage of beams are subjected to identical deformations under loading.The actual loading is replaced by an equivalent nodal loading.The grillage analogy in this case has advantage of being relatively inexpensive in computer time and simple to comprehend.

Longitudinal grid lines are usually placed coincident with webs of the actual structure. The transverse medium consisting of top and bottom slabs only (with no diaphragm), is represented by equally spaced transverse grid lines along the span. The transverse grid lines are placed along each diaphragm including at supports. Additional grid lines representing the top and bottom slabs are placed in between the diaphragms.A closer spacing of transverse grid lines will result in more continuous structural behavior and will provide greater details of forces.

As per the problem formulation 2 span continuous box girder bridge with 45m span is consider.As shown in Fig4.13 4 longitudinal grid lines have been assumed. The end ones at A and A' are necessiated because the cantilever projections are large and wheels of live load could go on the cantilever slabs beyond B and B'.The moment of inertia of the cross section of the bridge about a common axis is computed and this is divided equally among the two longitudinals at B and B'.Similarly torsional inertia of the closed trapezoidal section is computed and one half of this is assigned to each of the longitudinals at B and B'.Thus the entire inertia is concentrated along grid lines at B and B'.The remaining Longitudinals A and A' are assigned zero inertia values. The 7 transverse grid lines between 2 diaphragm are assumed as shown in Fig4.14.The flexural and torsional inertia values of these transverse member are computed.The transverse member also have diaphragms and hence moment of inertia of diaphragm are added in this transverse grillage member.

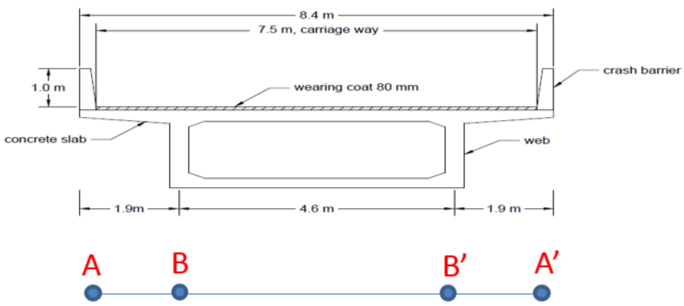


Figure 4.13: Location of Longitudinal Grid Line

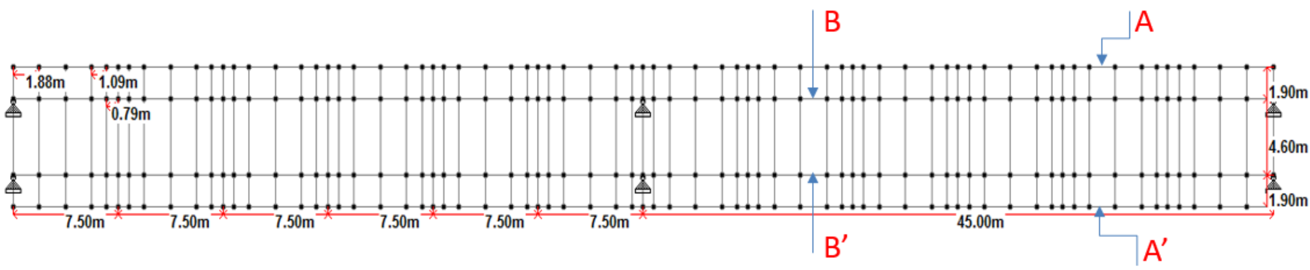


Figure 4.14: Top View of Gride Lines

Section Properties

- **Longitudinal Direction**

Where,

I_{zz} = moment of inertia @ main axix

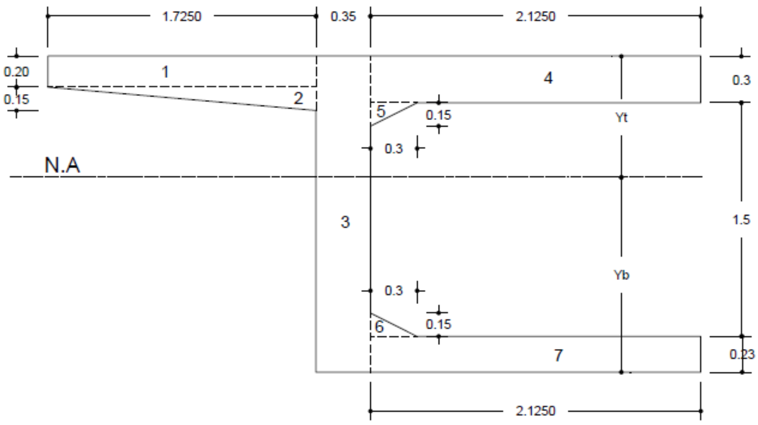


Figure 4.15: Half Cross Section Details

Table 4.3: Section property in longitudinal direction

Sr No	Type	No	Width(m)	Height(m)	Area(m^2)	Area x CG	$I_{zz}(m^4)$	$I_{yy}(m^4)$
1	Rec	2	1.725	0.2	0.69	0.069	0.32	7.82
2	Tri	2	1.725	0.15	0.258	0.0646	0.0728	2.452
3	Rec	2	0.35	2.03	1.42	1.44	1.074	7.53
4	Rec	2	2.125	0.3	1.275	0.19	0.514	1.912
5	Tri	2	0.3	0.15	0.045	0.0157	0.00834	0.185
6	Tri	2	0.3	0.15	0.045	0.0787	0.0425	0.185
7	Rec	2	2.125	0.23	0.977	1.872	1.265	1.465

I_{yy} = moment of inertia @ minor axix

I_{xx} = torsional moment of inertia

$$I_{xx} = \frac{4A^2}{\frac{s1}{t1} + \frac{s2}{t2} + \frac{2s3}{t3}} \tag{4.3}$$

Where,

A=area bounded by the center line of the closed C/S

$s1$ and $s2$ = width

$s3$ = height

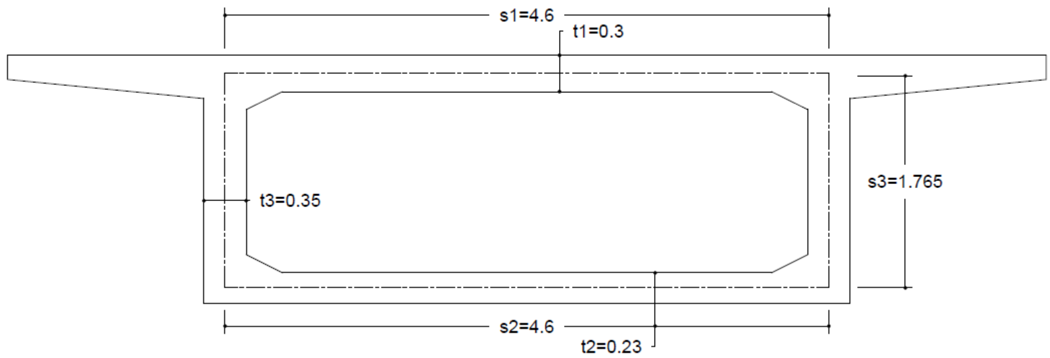


Figure 4.16: C/S Details for Torsional M.O.I

$t1, t2$ and $t3$ = thickness

Therefor,Properties for 1 longitudinal girder

$Y_t = 0.778m$

$Y_b = 1.251m$

Area $= A = 4.71/2 = 2.356m^2$

$I_{zz} = 3.297/2 = 1.648m^4$

$I_{yy} = 21.55/2 = 10.77m^4$

$I_{xx} = 5.8/2 = 2.9m^4$

- **Transverse Direction**

1 (without diaphragm)

Therefor,Properties for 1 (without diaphragm) transverse grillage member

$d_1 = 0.628m$

$d_2 = 1.136m$

Area $= A = 0.994m^2$

$I_{xx} = 0.785m^4$

$I_{yy} = 0.291m^4$

$I_{zz} = (t_1 d_1^2 + t_2 d_2^2)1.875 = 0.778m^4$

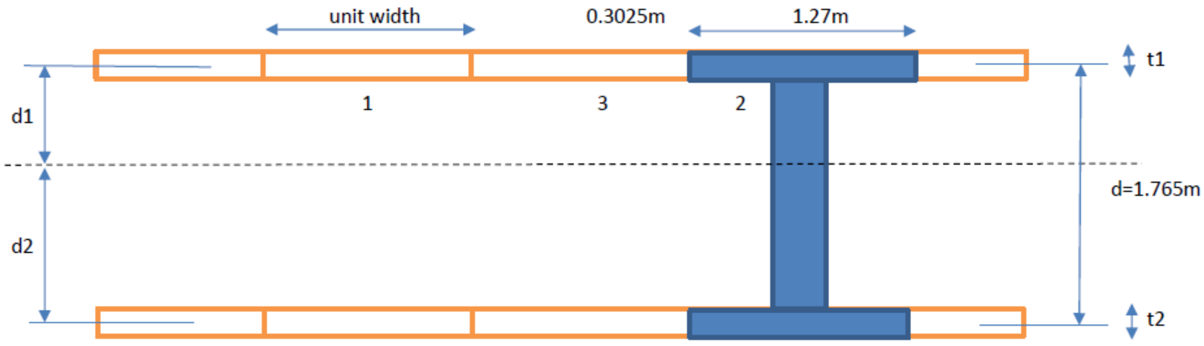


Figure 4.17: Transverse Direction C/S Properties

2 (with diaphragm)

Effective width $b_{eff} = b_w + 0.2l_o = 1.27m$ where $l_o = 4.6m$

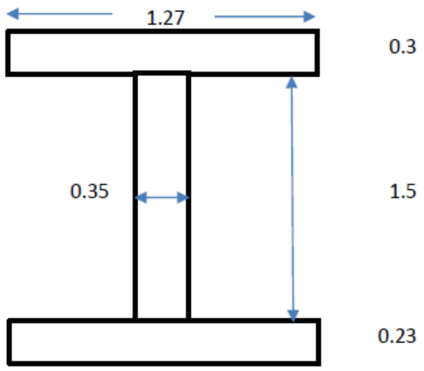


Figure 4.18: Diaphragm with Effective Width

Therefor,Properties for 2 (with diaphragm) transverse grillage member

Area $= A = 1.1981m^2$

$I_{xx} = 0.619m^4$

$I_{yy} = 0.958m^4$

$$I_{zz} = \frac{3}{10}(\frac{b^3d^3}{(b^2+d^2)} - \frac{b_1^3d_1^3}{(b_1^2+d_1^2)}) \tag{4.4}$$

$I_{zz} = 0.642m^4$

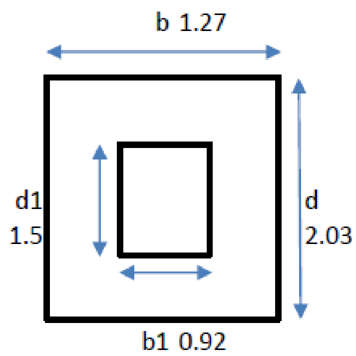


Figure 4.19: Box Section of Diaphragm

3 (Near diaphragm)

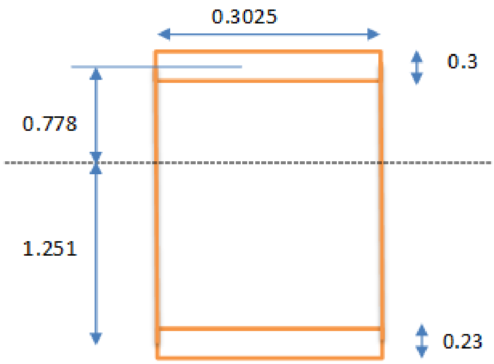


Figure 4.20: near diaphragm

Therefor,Properties for 3 (Near diaphragm) transverse grillage member

Area $= A = 0.1603m^2$

$I_{xx} = 0.125m^4$

$I_{yy} = 1.2E + 3m^4$

$I_{zz} = 0.123m^4$

Dead Load Analysis

Load calculation per girder

Self weight $= 4.71 * 25 = 117.80/2 = 58.90kN/m$

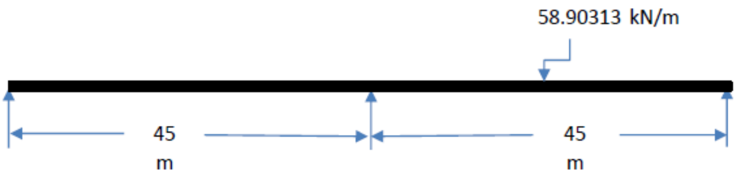


Figure 4.21: Self Weight per Girder

Self weight of cross diaphragm $=55.78/2 = 27.89kN$

Super imposed DL

Wearing coat $= 6.6kN/m$

Crash barrier $= 8.75kN/m$

Final DL per girder as shown in Fig4.22

Figure 4.22: Final DL Loading on Girder

Table 4.4: SF and BM due to DL

		SF (kN)	BM(kNm)
DL	Mid support	2087.27	18749.06
	Mid span		10588.5

Live Load Analysis

LL analysis has been done using grillage analogy method using staad pro software.In this analysis IRC class AA tracked vehicle and IRC class A vehicle is used.Total 6 cases based on vehicle positions as shown in below Table4.5 and Fig4.23 4.24 has been carried out to finding out maximum shear force, maximum bending moment and maximum torsion moment in the girder.

Table 4.5: Vehicles position

Sr No	IRC vehicle	No of vehicle	Position
1	Class AA tracked	1	left crash barrier
2		1	center
3	Class A	1	left crash barrier
4		1	center
5		2	left crash barrier
6		2	cente

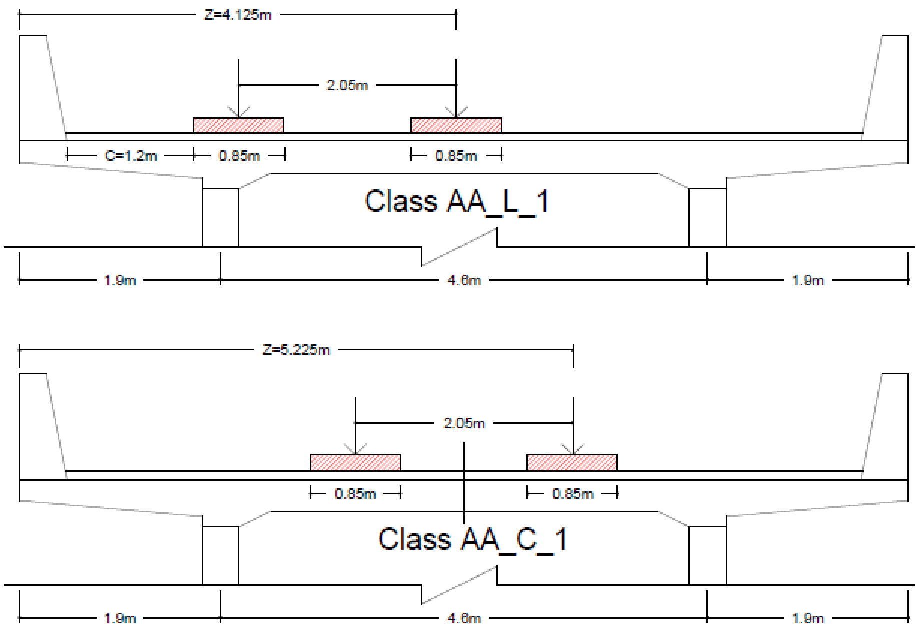


Figure 4.23: IRC class AA tracked vehicle position

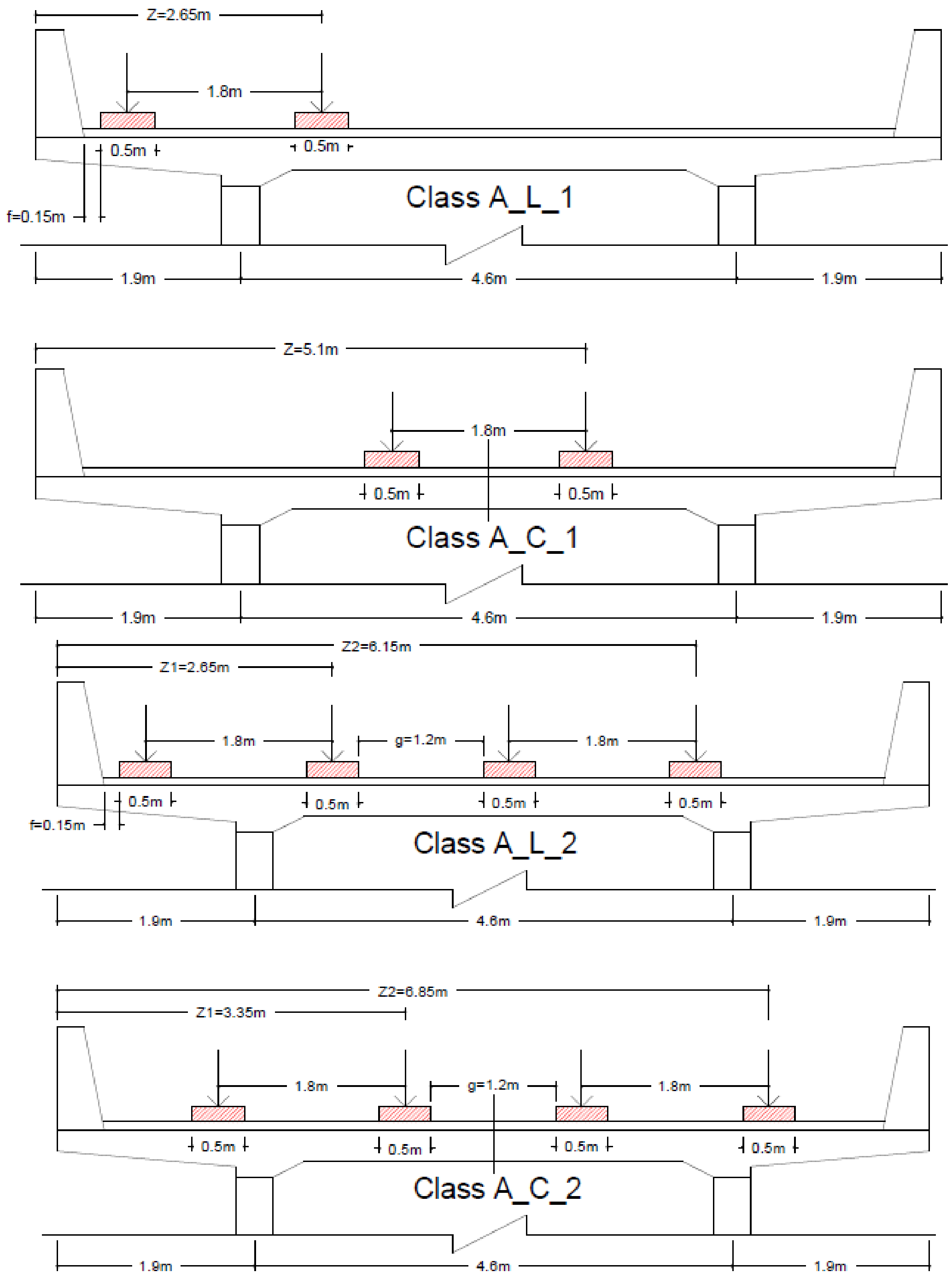


Figure 4.24: IRC class A vehicle position

Table 4.6: SF,TM and BM due to LL

	SF(kN) Mid support	TM(kNm)	BM(kNm) Mid support	BM(kNm) Mid span
class A C1	458.32	21.48	3921.28	2219.37
class A C2	916.66	14.18	7842.56	4438.74
class A L1	883.86	649.99	4627.89	3122.36
class A L2	1159.36	371.15	8230.13	4937.02
class AA C1	669.41	42.92	6221.58	3012.50
class AA L1	969.92	412.32	6856.28	3549.95

4.4 Design of Box Girder Bridge

Design data:

$\sigma_{cbc} = 8.33N/mm^2$

$\sigma_{st} = 200N/mm^2$

$k = 0.294$

$j = 0.902$

$Q_{bl} = 1.105$

4.4.1 Design of Deck Slab and Soffit Slab

Reinforcement Design

Effective depth required = $d_{eff,reqd} = 244.8mm$

Depth available = $d_{available} = 245mm$ ok

Deck Slab Shorter Span

$A_{st,req} = 1498.33mm^2$

$A_{st,min} = 540mm^2$

Provide 16mm dia @ 130mm c/c in shorter span equally distributed top and bottom

$A_{st,pro} = 1547.81mm^2$

Deck Slab Longer Span

$A_{st,req} = 1120.12mm^2$

$A_{st,min} = 540mm^2$

Provide 16mm dia @ 170mm c/c in longer span equally distributed top and bottom

$A_{st,pro} = 1183.62mm^2$

Soffit Slab Shorter Span

$A_{st,min} = 414mm^2$

Provide 12mm dia @ 270mm c/c in shorter span equally distributed top and bottom

Soffit Slab Longer Span

$A_{st,min} = 690mm^2$

Provide 12mm dia @ 160mm c/c in Longer span equally distributed top and bottom

Check for Shear

Dead load Shear force = $21.30kN$

Live Load Shear force with impact = $61.20kN$

Total Shear force = $82.50kN$

$\tau_v = 0.33N/mm^2$

For $300mm$ depth of slab,k=1

$\tau_c = 0.34N/mm^2$ ok

4.4.2 Design of Cantilever Slab

Reinforcement Design

Design BM=$78.58kNm$

Effective depth required = $d_{eff,req} = 266.71mm$

Depth available = $d_{available} = 290mm$ ok

Main Reinforcement

$A_{st,req} = 1502.01mm^2$

$A_{st,min} = 804.86mm^2$

$A_{st,pro}$by deck slab panel due to bent up (shorter span)

$A_{st,pro}/2 = 773.90mm^2$

Therefor,$A_{st,req} = 728.1mm^2$

Provide 16mm dia @ 260mm c/c and 12mm dia @ 160mm c/c

Distribution Reinforcement

Based on $A_{st,min} = 804.86mm^2$

Provide 8mm dia @ 120mm c/c

Check for Shear

Total Shear force = $82.15kN$

$\tau_v = 0.28N/mm^2$

For 350mm depth of slab,k=1

$\tau_c = 0.32N/mm^2$ ok

4.4.3 Design of Diaphragm

Reinforcement Design

Design BM=319.46kNm

Effective depth required = $d_{eff,req} = 537.77mm$

Depth available = $d_{available} = 1437.5mm$ ok

$A_{st,req} = 1231.92mm^2$

$A_{st,min} = 1006.25mm^2$

Provide 4# bars of 20mm dia

Check for Shear

Total Shear force = $441.26kN$

$\tau_v = 0.88 N/mm^2$

$\tau_c = 0.32 N/mm^2$

Therefor shear reinforcement is required

Use 10mm dia bar with 2 leg

Shear force resisted by the stirrups = $300.39 kN$

Provide 10mm dia 2L @ 150mm c/c

Skin Reinforcement

$A_{st} = 0.1\%$ of the c/s area

$A_{st} = 525 mm^2$

Use 12mm dia bar

No of bar required = 4.64 say=6 No

Provide 6# bars of 12mm dia, 3# on each face

4.4.4 Design of Web Girder

Table 4.7: Design Bending Moment

	DL(M_g) kNm	LL(M_q) kNm(including impact)	$M_d = M_g + M_q$ kNm	$M_u = 1.5M_g + 2.5M_q$ kNm
Mid span	10588.523	9053.14	19641.663	38515.63
Mid support	18749.06	5430.72	24179.78	41700.39

Table 4.8: Design Shear force

	DL(V_g) kN	LL(V_q) kN(including impact)	$V_d = V_g + V_q$ kN	$V_u = 1.5V_g + 2.5V_q$ kN
Mid support	2087.27	1275.29	3362.56	6319.13

Table 4.9: Design Torsion Moment

LL(M_t) (kNm)	$M_d = 2.5M_t(kNm)$
649.99	1624.97

Permissible Stresses

For M-60 grade concrete

$f_{ck} = 60N/mm^2$

$f_{ci} = 45N/mm^2$

$f_{ct} = 0.45f_{ci} = 20.5N/mm^2$

$f_{cw} = 0.33f_{ck} = 20N/mm^2$

$f_{tt} = f_{tw} = 0$

Loss ratio = 0.8

C/S properties of main girder

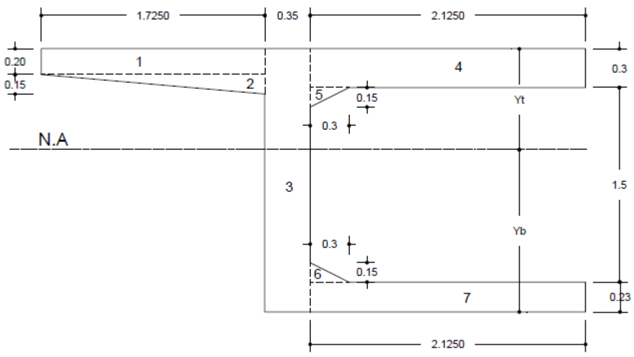


Figure 4.25: Cross Section of Main Girder

$Y_t = 0.778m$

$Y_b = 1.251m$

Area $= A = 2.356m^2$

$I_{zz} = 3.297/2 = 1.648m^4$

$Z_t = 2.115m^3$

$Z_b = 1.317m^3$

Check for Minimum Section Modulus

$M_d = 24179.78kNm$

$f_{br} = nf_{ct} - f_{tw} = 16.2N/mm^2$

$$f_{inf} = \frac{f_{tw}}{n} + \frac{M_d}{nZ_b} \tag{4.5}$$

$f_{inf} = 22.94N/mm^2$

$$Z_{b,req} = \frac{M_q + (1-n)M_g}{f_{br}} \tag{4.6}$$

$Z_{b,req} = 0.57E + 9mm^3 < Z_{b,pro} = 1.32E + 9mm^3$

Hence section provided is adequate

Prestressing Force

For the 2 span continuous, a "concordent cable profile" is selected such that the secondary moment are zero.the cable profile selected is shown in Fig4.26. The maximum possible eccentricity at the mid support section C is determined by providing suitable cover to house the cable.

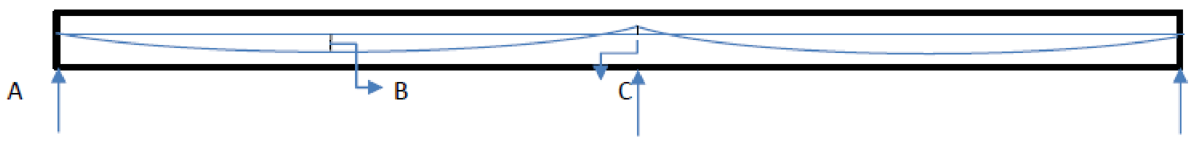


Figure 4.26: Concordent Cable Profile

Resultant eccentricity at C = 414.60mm

Resultant eccentricity at B = 371mm

Prestressing force(P) obtained from the relation shown in below

$$P = \frac{Af_{inf}Z_b}{Z_b + Ae} \tag{4.7}$$

$P = 31045008.9N = 31045kN$

Using freyssinest system, anchorage type 37k-15 (37 strands of 15.2 mm diameter) in 130mm cable ducts,

Characteristic strength of each strand = 265kN

Force in each cable = 7844kN

No of cable required = 4/ girder

Total prestressing force (P)=31376kN

Area of 15.2mm dia tendon = 140mm^2

Area of 37 strand = 5180mm^2

Total area of 4 cable = A_p = 20720mm62

Check for Stresses

- **Center at span section**

At the stage of transfer

$$\sigma_t = \frac{P}{A} - \frac{Pe}{Z_t} + \frac{M_g}{Z_t} \tag{4.8}$$

$\sigma_t = 12.81N/mm^2$

$$\sigma_b = \frac{P}{A} + \frac{Pe}{Z_b} - \frac{M_g}{Z_b} \tag{4.9}$$

$\sigma_b = 14.12N/mm^2$

At the service load state

$$\sigma_t = n\frac{P}{A} - n\frac{Pe}{Z_t} + \frac{M_g}{Z_t} + \frac{M_q}{Z_t} \tag{4.10}$$

$\sigma_t = 15.54N/mm^2$

$$\sigma_b = n\frac{P}{A} + n\frac{Pe}{Z_b} - \frac{M_g}{Z_b} - \frac{M_q}{Z_b} \tag{4.11}$$

$\sigma_b = 2.81N/mm^2$

- **Mid support section**

At the stage of transfer

$$\sigma_t = \frac{P}{A} + \frac{Pe}{Z_t} - \frac{M_g}{Z_t} \tag{4.12}$$

$\sigma_t = 10.60N/mm^2$

$$\sigma_b = \frac{P}{A} - \frac{Pe}{Z_b} + \frac{M_g}{Z_b} \tag{4.13}$$

$\sigma_b = 17.67N/mm^2$

At the service load state

$$\sigma_t = n\frac{P}{A} + n\frac{Pe}{Z_t} - \frac{M_g}{Z_t} - \frac{M_q}{Z_t} \tag{4.14}$$

$\sigma_t = 4.14N/mm^2$

$$\sigma_b = n\frac{P}{A} - n\frac{Pe}{Z_b} + \frac{M_g}{Z_b} + \frac{M_q}{Z_b} \tag{4.15}$$

$\sigma_b = 18.65N/mm^2$

The stresses in general are within the maximum permissible limit of $20N/mm^2$

Check for Ultimate Flexural Strength

- **Center of span section**

Data:

$M_{u,req} = 38515.63kNm$

$A_p = 20720mm^2$

$f_p = 1862N/mm^2$

$b_w = 350mm$

$D_{f,avg} = 287.5mm$

$d = 1150mm$=depth of web from the maximum compression edge to the CG of steel tendon

Failure by yielding of steel

$$M_u = 0.9dA_sf_p \tag{4.16}$$

$M_u = 39917.4kNm$....................1

Failure by crushing of concrete

$$M_u = 0.176bd_b^2f_{ck} + \frac{2}{3}(0.8)(b - b_w)(d - 0.5D_f)D_ff_{ck} \tag{4.17}$$

$M_u = 94421.3kNm$....................2

$M_{u,pro}$ = minimum of 1 and 2 = $39917.4kNm$

- **Mid support section**

Data:

$d = 1666mm$

Failure by yielding of steel

$$M_u = 0.9dA_sf_p \tag{4.18}$$

$M_u = 57847.8kNm$....................1

Failure by crushing of concrete

$$M_u = 0.176bd_b^2 f_{ck} + \frac{2}{3}(0.8)(b - b_w)(d - 0.5D_f)D_f f_{ck} \tag{4.19}$$

$M_u = 177289kNm$....................2

$M_{u,pro}$ = minimum of 1 and 2 = $57847.81kNm$

Hence, Design is satisfies the limit state of collapse

Check for Ultimate Shear Strength

- **Section uncracked in flexure**

Data:

$V_{u,req} = 6319.13kN$

f_t = maximum principle tensile stress = $0.24\sqrt{fck} = 1.85N/mm^2$

f_{cp}=compressive stress at centroidal axis due to prestress = $10.65N/mm^2$

θ = vertical component of prestressed force = $0.00369rad$

$$V_{co} = 0.67b_w h\sqrt{f_t^2 + 0.8f_{cp}f_t} + nPsin\theta \tag{4.20}$$

$V_{co} = 2091.4kN$

Balanced shear = $6319.14 - 2091.4 = 4227.74kN$

Use 16 mm diameter 4L stirrups

Spacing, $S_v = 130mm$

Provide 16mm dia 4L @ 130mm c/c near the supports

Check for Torsion

Data:

$T = 1624.97kNm$

h_{wo}=wall thickness of members where the stress is determined

V_t=Torsional shear stress

V_{tc}=Permissible torsion shear stress

If $V_t > V_{tc}$, Then reinforcement shall be provided

Therefor,

$$V_t = \frac{T}{2h_{wo}A_o} \tag{4.21}$$

$V_t = 0.286N/mm^2$

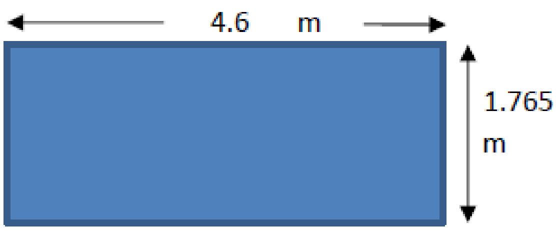


Figure 4.27: Box Section

$V_{tc} = 0.42N/mm^2$

Therefor, Torsional reinforcement are not required

Supplementary reinforcement for web

The longitudinal reinforcement should not be less than 0.18%of gross sectional area

$A_{st} = 630mm^2$

Use 16mm dia bar

No of bar required =3.13 say 4 No

Design of End Block

$2Y_o$ = side of end block = $500mm$

$2Y_{po}$= side of loaded area (bearing plate) = $420mm$

P_k= load in tendon = $7844kN$

F_{bs}= bursting tensile force

$Y_{po}/Y_o = 0.84$

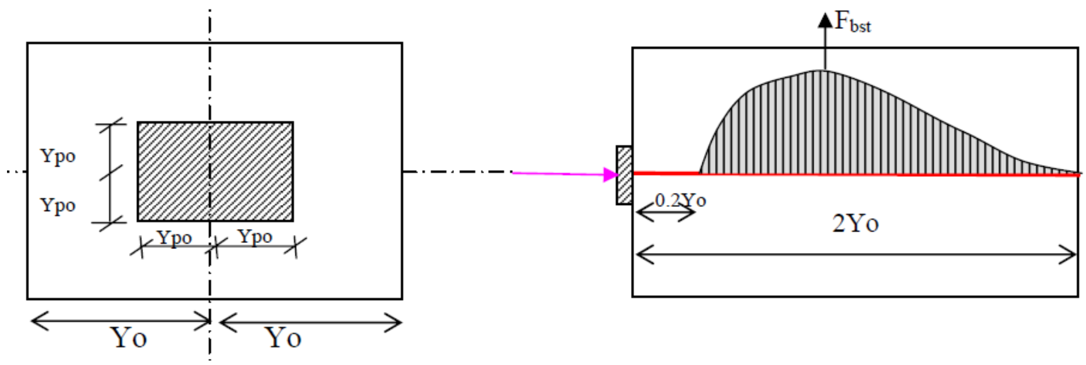


Figure 4.28: Tensile Stress Distribution of End Block

$F_{bs}/P_k = 0.08$

$F_{bs} = 627.52kN$

$A_{st,req} = 1738.04mm^2$

Use 10mm dia bar 4L stirrups

No of row provided = 6

$A_{st,pro} = 1886.4mm^2$

The reinforcement is provided in $0.2Y_o$ to $2Y_o$ region = $50mm$ to $500mm$

Spacing = $90mmc/c$

Provide 10mm dia bar @ 90mm c/c and then spacing 180mm c/c up to 2m length of web

4.5 Drawings

Deck slab, Soffit slab and Cantilever slab

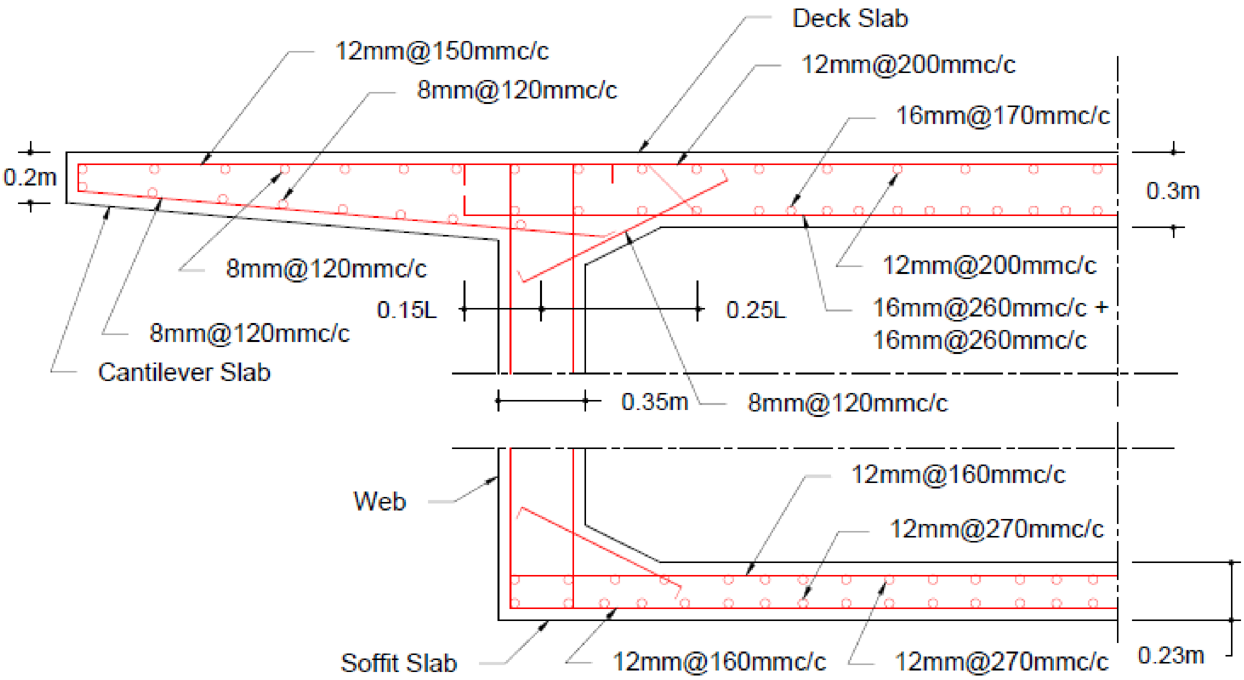


Figure 4.29: Reinforcement Detailing for Slabs

End Block Detailing

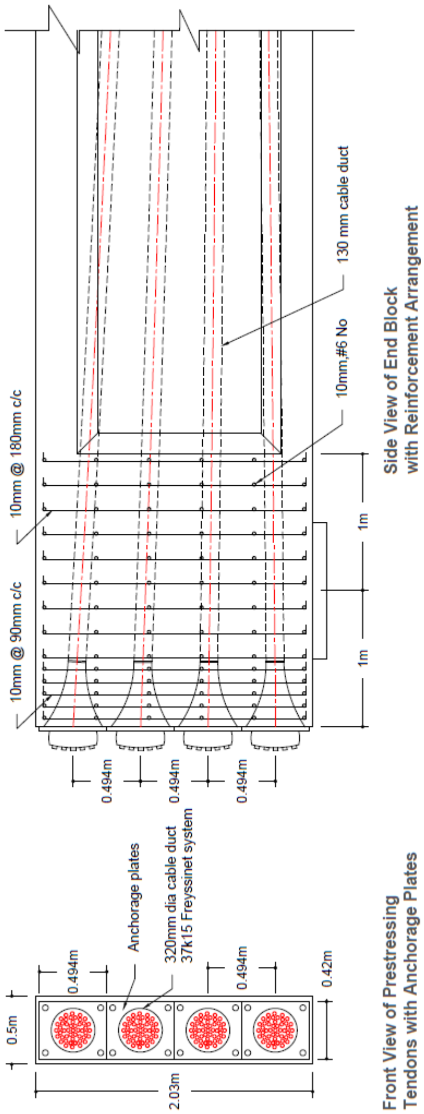


Figure 4.30: End Block Detailing

Web Reinforcement and Location of Prestressed tendons

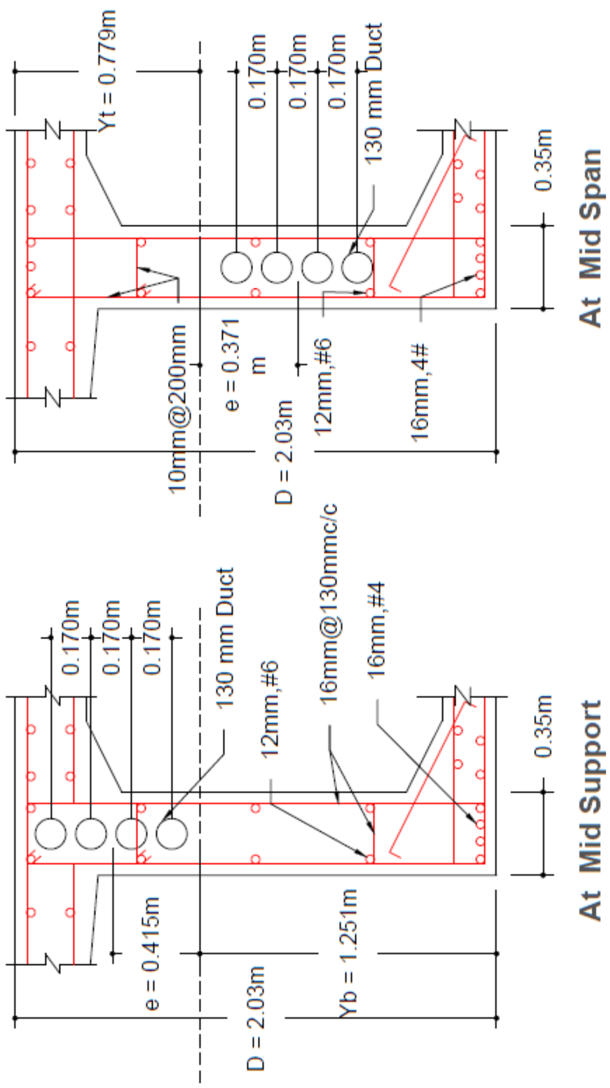


Figure 4.31: Web Reinforcement

4.6 Estimation of cost

The estimation of cost for any structure includes quantity estimation and rate analysis. The estimation of cost is necessary for selection of final design alternative amongst all the available various designs alternatives. The quantity estimation is a schedule or list of quantities of all the possible items required for construction of any structure. These quantities are worked out by reading the drawing of the structure. Thus the quantity estimation indicates the amount of work to be done under each item, which when priced per unit of work gives the amount of cost of that particular item.

Quantity of Concrete

Table 4.10: Quantity of concrete

Sr No	Description	Area(m^2)	Length(m)	Volume(m^3)
1	Slabs and Webs	4.712	90	424.10
2	Haunches (lateral)	0.09	43.8	3.942
3	Diaphragm	6.285	4.55	28.60
Total volume of concrete (m^3)				456.64

Quantity of Shuttering

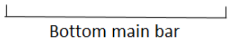
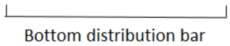
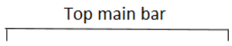
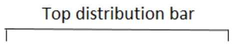
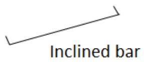
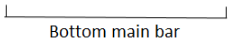
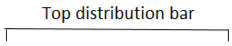
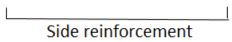
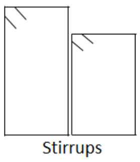
Table 4.11: Quantity of Shuttering

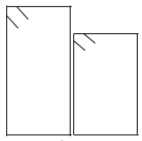
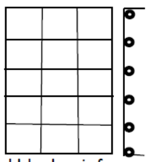
Sr No	Description	Area(m^2)
1	Deck slab	349.14
2	Cantilever slab	347.67
3	Soffit slab	445.5
4	Webs	499.56
5	Haunches	110.08
6	Intermediate diaphragm	138.27
7	End diaphragm	32.67
Total area of shuttering (m^2)		1922.89

Quantity of HYSD Steel

Table 4.12: Quantity of Reinforcement

Deck slab							
Sr No	Description	Dia / spacing	No	Length (m)	Total length (m)	Unit weight (kg/m)	Total weight (kg)
1	Bottom main bar	16mm @260mmc/c	347	8.65	3001.55	1.58	4742.4
2	Bottom main bar	16mm @260mmc/c	346	5.36	1854.56	1.58	2930.2
3	Bottom distribution bar	16mm @170mmc/c	33	90.2	2976.6	1.58	4703
4	Top main bar	12mm @200mmc/c	451	5.36	2417.36	0.889	2149
5	Top distribution bar	12mm @200mmc/c	28	90.14	2523.9	0.889	2243.8
6	Inclined bar	8mm @120mmc/c	1372	0.76	1092.11	0.395	431.38
Cantilever slab							
7	Top main bar	12mm @150mmc/c	1202	2.61	3137.22	0.889	2789
8	Top distribution bar	8mm @120mmc/c	32	90.1	2883.07	0.395	1138.8
9	Inclined main bar	8mm @120mmc/c	1500	2	3000	0.395	1185
10	Bottom distribution bar	8mm @120mmc/c	30	90.1	2702.9	0.395	1067.6

Soffit slab							
Sr No	Description	Dia / spacing	No	Length (m)	Total length (m)	Unit weight (kg/m)	Total weight (kg)
11		12mm @160mmc/c	563	5.094	2867.92	0.889	2549.6
12		12mm @270mmc/c	20	90.14	1802.88	0.889	1602.8
13		12mm @160mmc/c	563	5.09	2867.92	0.889	2549.6
14		12mm @270mmc/c	20	90.14	1802.88	0.889	1602.8
15		8mm @120mmc/c	1372	0.796	1092.11	0.395	431.38
Diaphragm							
16		20mm	52	5.09	264.68	2.469	653.49
17		20mm	39	5.09	198.51	2.469	490.12
18		12mm	78	5.09	397.02	0.889	352.95
19		10mm 150mmc/c	390	7.08	2761.2	0.617	1703.7

Web							
Sr No	Description	Dia / spacing	No	Length (m)	Total length (m)	Unit weight (kg/m)	Total weight (kg)
20	Bottom main bar	16mm	8	90.04	720.34	1.58	1138.1
21	Top distribution bar	16mm	6	90.04	540.25	1.58	853.6
22	Side reinforcement	12mm	12	90.04	1080.5	0.889	960.57
23	 Stirrups	16mm 130mmc/c	1324	7.08	9374	1.58	14811
24	 End block reinforcement	10mm hori. 10mm vert.	336 224	0.39 2.03	131.04 454.72	0.617 0.617	80.85 280.56
Total weight (kg)						53441.12	

Quantity of Prestressing Steel

Table 4.13: Quantity of Tendons

Sr No	Description	No	Weight(kg/m)	Length(m)	Total weight(kg)
1	Cable 37k-15	8	40.47	720.34	29152

Bill of Quantity

Table 4.14: Bill of Quantity for Box Girder

Sr No	Description	Quantity		Rate		Cost(Rs)
1	Concrete	456.64	m^3	5500	Rs/m^3	2511526.88
2	Shuttering	1922.89	m^2	110	Rs/m^2	211517.40
3	H.T steel	29152	kg	170	Rs/kg	4955839.65
4	HYSD steel	53441.1	kg	50	Rs/kg	2672055.28
5	Scaffolding	456.64	m^3	160	Rs/m^3	73062.6
Total Cost(Rs)						10424002

4.6.1 Estimation of Unit Weight

Table 4.15: Unit weight

Sr No	Description	Weight(kN)
1	Concrete	11416.03
2	H.T steel	285.98
Total weight(kN)		11702

4.6.2 Summary of Result

Estimation of cost and unit weight of different items like concrete, reinforcement, H.T steel tendons is carried out for conventional prestressed concrete box girder bridge.

Cost of bridge per meter $= 1,15,823 Rs/m$

Weight of bridge per meter $= 130 kN/m$

Chapter 5

Box Girder Bridge with Corrugated Steel Webs

5.1 General

In present study PC box girder bridge with corrugated steel webs is taken for under stand the analysis and design phenomena. This chapter cover the analysis,design and estimation of cost and unit weight for this type of bridge.

The web corrugation profile can be viewed as uniformly distributed stiffening in the transverse direction of the beam. When girders with corrugated webs are compared with those with stiffened flat webs, it can be found that trapezoidal corrugation in the web enables the use of thinner webs and corrugated web I-Beams eliminate costly web stiffeners. Due to less cost and higher load carrying capacity, corrugated web I-beams provide a high strength-to-weight ratio compare to I-beams with flat plate. The general corrugated web girder profile and geometric notation corrugated web are as shown in Fig5.1 below.

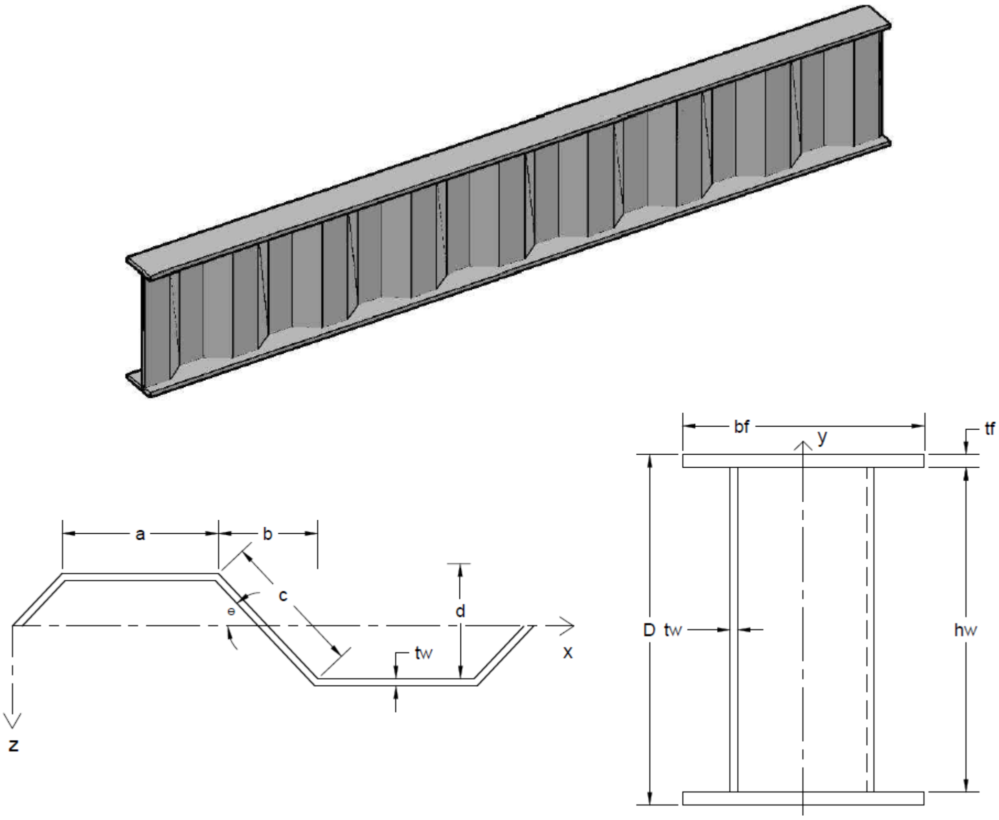


Figure 5.1: Corrugated Web Girder and Geometric Notation

Where,

a=flat panel width

b=horizontal projection of the inclined panel width

c=inclined panel width

θ=corrugation angle

d=corrugation depth

t_w=web thickness

b_f=width of flange

t_f=thickness of flange

h_w=web height

D=girder height

5.2 Structural Data

Density of steel=7.85kN/m^3

Density of concrete=25kN/m^3

Density of wearing coat=22kN/m^3

f_y=250N/mm^2

E=200000N/mm^2

γ_m=1.1

γ_{mst}=1.15

f_{st}=415N/mm^2

f_{ck}=25N/mm^2

In this study, cross section taken for analysis is as shown in Fig5.2

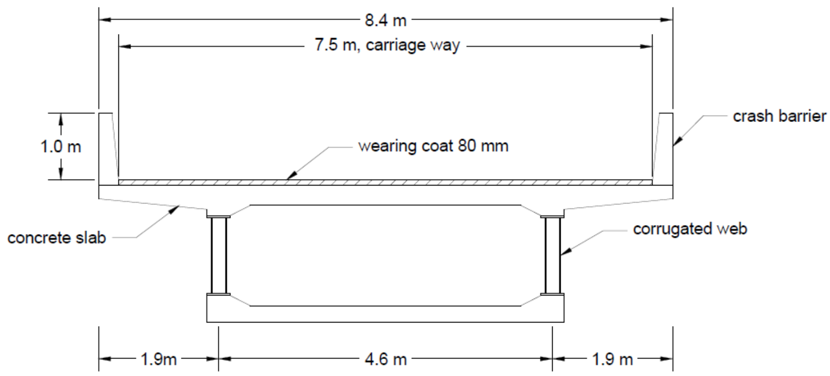


Figure 5.2: Cross Section of Bridge

In this study, longitudinal section taken for analysis is as shown in Fig5.3

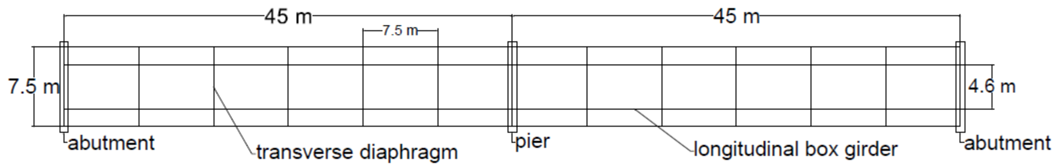


Figure 5.3: Longitudinal Section of Bridge

5.2.1 Data Specification

The economy does not depend directly on single variable, it depends on various combinations of variables. In present study alternatives taken for economical design are span to girder depth ratio (L/D) and web height to corrugation depth ratio (hw/d) for spans 40m, 45m, 50m as shown in table below.

Span to girder depth ratio(L/D)

Span to girder depth ratio(L/D) also affects the economy. If span to girder depth ration is higher, at that time depth of girder decreases. The small section lead to decrease the material cost.it requires more stiffening device for shear and more no of external prestressing tendon for flexural therefore it increase the cost and weight. At the same time lower L/D ratio give higher girder section and increases the material cost but it requires less stiffening device and less external prestressing tendons therefore it decreases the cost and weight. The span to depth ratio plays very important role in economy. The analysis for 40m, 45m, and 50m of 2 span continuous bridge is carried out using staad pro software using grillage analogy method for various L/D ratio. Out of all L/D ratio most economical L/D ratio is obtained.

Web height to corrugation depth ratio(hw/d)

Web height to corrugation depth ratio affect the shear buckling behavior and lateral torsional buckling behavior. It also affect the cost and unit weight of the bridge.

To understand the shear buckling behavior and lateral torsional buckling behavior 3 corrugation depth (d) are taken 150mm, 180mm, 210mm for particular depth of bridge with particular span.

Table 5.1: Data of various span

Sr no	Span(m)	$D(mm)$	$d(mm)$	$hw(mm)$	L/D	hw/d
1	40,45,50	750	150 180 210	690	53.33,60,66.7	5 4.17 3.57
2	40,45,50	1000	150 180 210	940	40,45,50	6.67 5.56 4.76
3	40,45,50	1200	150 180 210	1140	33.33,37.5,41.7	8 6.67 5.71

To compare the cost and weight of conventional box girder bridge to box girder bridge with corrugated steel webs for the span of 45m the L/D ratio (37.5) and hw/d ratio(8) are taken.

5.3 Analysis of Box Girder Bridge with Corrugated Steel Webs

Analysis and Analysis result of deck slab, cantilever slab, diaphragm are same as based on conventional box girder bridge.

5.3.1 Analysis of Longitudinal Girder

The girder is design for flexure and shear. The analysis is done for dead load, live load and for live load IRC Class A and IRC Class AA tracked vehicles are taken.The analysis is done using grillage analogy method using staad pro. For this analysis different section properties are to be calculated for the different grillage member.

Section properties

To find out the sectional property of composite longitudinal girder first the thickness of the trapezoidally corrugates steel web converted into equivalent thickness based on area(BOA) and based on stiffness(BOS) in x-direction. Other parameter of the corrugated web are assumed based on the case study of the different bridges.

Data:

a=250mm

b=200mm

c=250

θ=36.40

d=150mm

t_w=10mm

b_f=350mm

t_f=30mm

h_w=1140mm

D=1200mm

l_1=1 wave length of corrugation=$(2a + 2c)$=1000mm

L_1=length of flat plate=$(2a + 2b)$=900mm

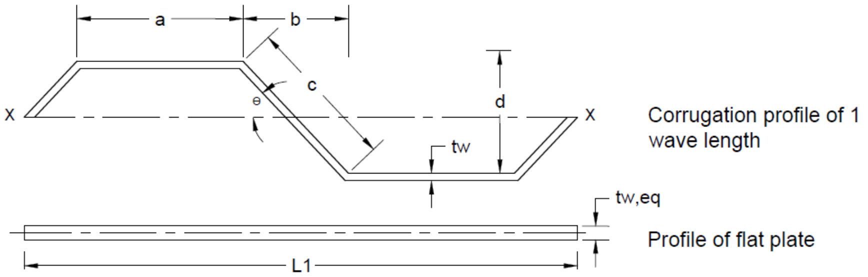


Figure 5.4: Top view of 3D and 2D web

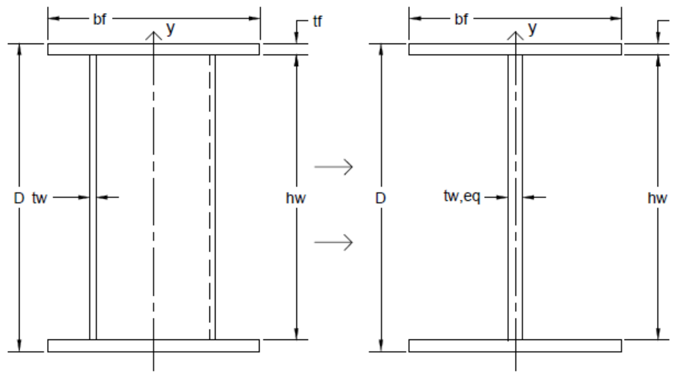


Figure 5.5: C/S of 3D and 2D web

Based on Area(BOA):

Area,3D = Area,2D

$$l_1 t_w = L_1 t_{w,eq} \tag{5.1}$$

$t_{w,eq} = 11.11mm$

Based on Stiffness(BOS):

$I_{xx}, 3D = I_{xx}, 2D$

$$2at_w(\frac{d}{2})^2 + \frac{t_w d^3}{6sin\theta} = \frac{L_1 t_{w,eq}^3}{12} \tag{5.2}$$

$t_{w,eq} = 79.37mm$

- **Longitudinal Direction**

To find out the properties of composite longitudinal girder the steel I section is converted into concrete by multiplying modular ratio.

$m = E_s/E_c = 8$

Where,

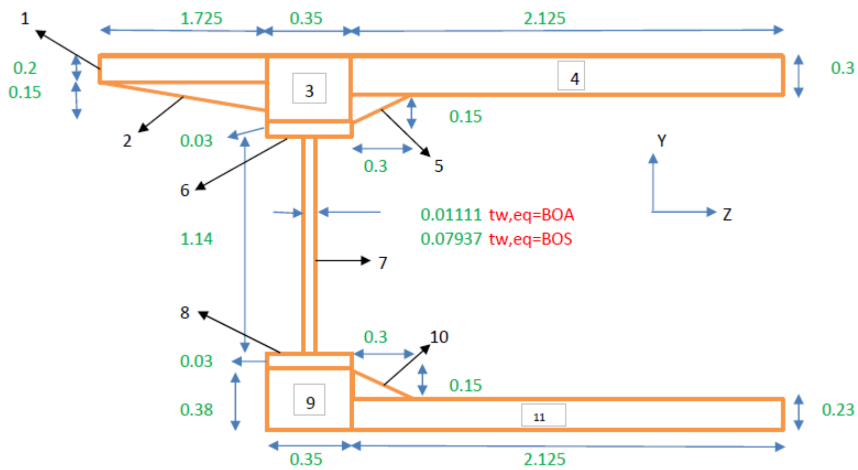


Figure 5.6: Section property in longitudinal direction

I_{zz} = moment of inertia @ main axix

I_{yy} = moment of inertia @ minor axix

I_{xx} = torsional moment of inertia

$$I_{xx} = \frac{4A^2}{\frac{s1}{t1} + \frac{s2}{t2} + \frac{2s3}{t3m}} \tag{5.3}$$

Table 5.2: Section property of 1 longitudinal grillage member

Sr No	$Y_b(m)$	$Y_t(m)$	Area(m^2)	$I_{zz}(m^4)$	$I_{yy}(m^4)$	$I_{xx}(m^4)$
BOA	1.255	0.775	2.21	1.40	9.99	1.76
BOS	1.19	0.84	2.83	1.51	13.29	3.22

- **Transverse Direction**

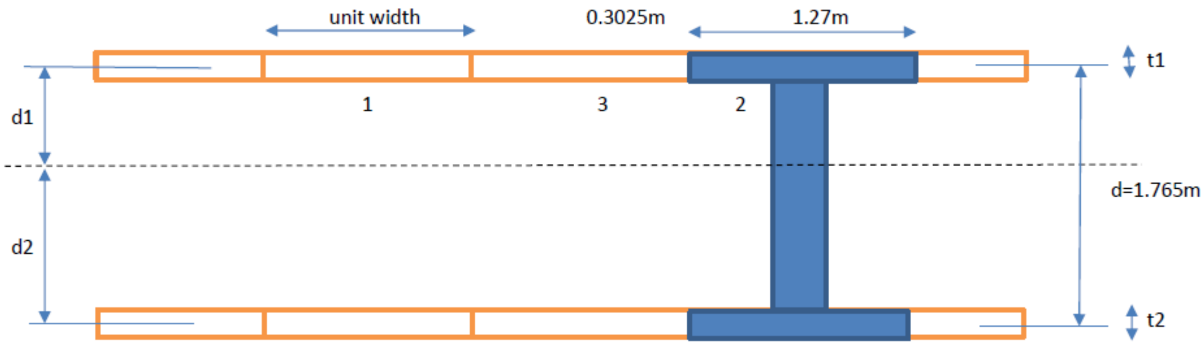


Figure 5.7: Transverse Direction C/S Properties

Table 5.3: Section property of transverse grillage member

No	Type	$d_1(m)$	$d_2(m)$	Area(m^2)	$I_{zz}(m^4)$	$I_{yy}(m^4)$	$I_{xx}(m^4)$
1	BOA	0.62	1.14	0.993	0.780	0.291	0.786
	BOS	0.68	1.07	0.993	0.77	0.29	0.77
3	BOA	0.62	1.14	0.16	0.126	1.2E+3	0.127
	BOS	0.68	1.07	0.16	0.124	1.2E+3	0.124

2 (with diaphragm)

Effective width $b_{eff} = b_w + 0.2l_o = 1.27m$ where $l_o = 4.6m$

Therefor,Properties for 2 (with diaphragm) transverse grillage member

Area $= A = 1.1981m^2$

$I_{xx} = 0.619m^4$

$I_{yy} = 0.0958m^4$

$$I_{zz} = \frac{3}{10}\left(\frac{b^3d^3}{(b^2+d^2)} - \frac{b_1^3d_1^3}{(b_1^2+d_1^2)}\right) \tag{5.4}$$

$I_{zz} = 0.642m^4$

Dead Load Analysis

Below Calculation is for BOA and combined result for BOA and BOS are listed in below Table5.4 Load calculation per girder

Self weight $= 48.53kN/m$

Self weight of cross diaphragm =55.78/2 $= 27.89kN$

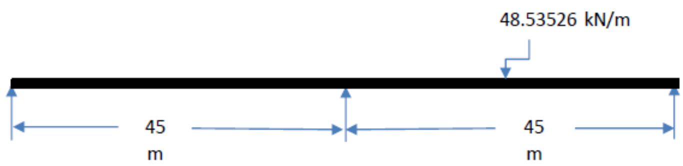


Figure 5.8: Self Weight per Girder

Wearing coat $= 6.6kN/m$

Crash barrier $= 8.75kN/m$

End block for prestressing=18.75kN/m

Final DL per girder as shown in Fig5.9

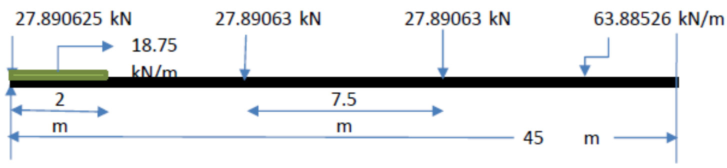


Figure 5.9: Final DL Loading on Girder

Table 5.4: SF and BM due to DL

			SF (kN)	BM(kNm)
BOA	DL	Mid support	1795.98	16135.39
		End support	995.23	9109.11
BOS	DL	Mid support	1796.11	16141.15
		End support	996.43	9106.93

Live Load Analysis

LL analysis has been done using grillage analogy method using staad pro software. IRC Vehicle loadings and Vehicle positions are same as the used in conventional PC box girder bridge.The top view of grillage as shown in Fig5.10. Here result are shown for maximum force.The analysis result for the span of 45m the L/D ratio (37.5) and hw/d ratio(8) are in below Table6.6.

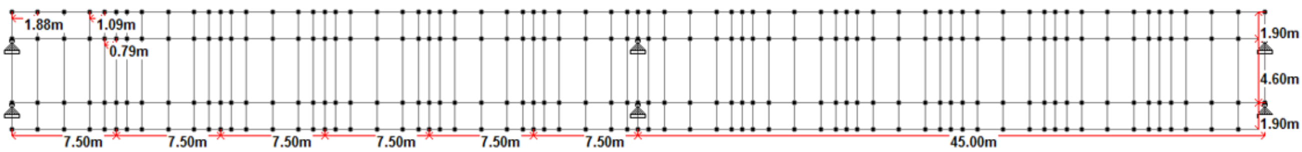


Figure 5.10: Top View of Grid Lines

Table 5.5: SF,TM and BM due to LL

	SF(kN)		TM(kNm)		BM(kNm)			
	Mid support				Mid support		Mid span	
	BOA	BOS	BOA	BOS	BOA	BOS	BOA	BOS
class A C1	469.12	469.14	82.06	95.46	3920.89	3920.57	2221.40	2222.19
class A C2	938.25	938.27	54.17	63.02	7841.79	7841.15	4442.80	4444.38
class A L1	795.12	773	563.99	667.32	4218.84	4155.47	2713.56	2645.87
class A L2	1109.95	1090.56	332.46	395.97	8003.99	7965.71	4717.60	4681.72
class AA C1	678.74	678.74	161.89	194.22	6221.65	6221.25	3013.30	3014.38
class AA L1	939.1	928.44	386.17	467.74	6548.95	6500.70	3295.37	3256.56

5.4 Design of Box Girder Bridge with Corrugated Steel Webs

Design of deck slab, cantilever slab, diaphragm are same as based on conventional box girder bridge. Therefore only the design of longitudinal girder is carried out here.

5.4.1 Design of Longitudinal Girder

The girder is designed for flexure and shear.For design of girder, codal provision of IRC 21, IS800-2007 and IRC 22-2008 are used.For design force consideration we have taken carriageway combination as 1.35DL + Impact +1.5 Live Load..The sample Design calculation for the span of 45m the L/D ratio (37.5) and hw/d ratio(8) are carried out here.

Table 5.6: Design Bending Moment

Type	Location	DL kNm	LL kNm	LL(impact) kNm	DL+LL (impact) kNm	1.35DL+1.5LL (impact) kNm
BOA	Mid span	9109.11	8003.99	8710.23	17819.34	25362.64
	Mid support	16135.39	4717.60	5133.86	21269.26	29483.57
BOS	Mid span	9106.94	7965.71	8668.57	17775.50	25297.22
	Mid support	16141.15	4681.72	5094.81	21235.96	29432.77

Table 5.7: Design Shear Force

Type	Location	DL kNm	LL kNm	LL(impact) kNm	DL+LL (impact) kNm	1.35DL+1.5LL (impact) kNm
BOA	Mid support	1795.98	1109.95	1207.88	3003.86	4236.40
BOS	Mid support	1796.11	190.56	1186.79	2982.90	4204.93

Table 5.8: Design Torsion Moment

Type	LL kNm	LL(impact) kNm	1.5LL(impact) kNm
BOA	563.99	613.75	920.63
BOS	667.32	726.20	1089.31

Section classification

Therefore, section is in compact for BOA and in plastic for BOS as shown in Table5.9

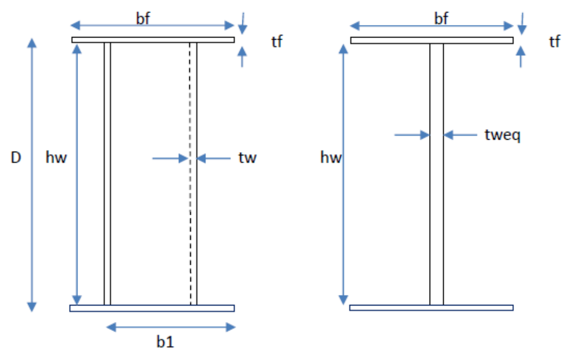


Figure 5.11: geometric notation

Table 5.9: Section classification

Type	Criteria	Value	Limiting Ratio	Class of Section
BOA BOS	Flange criteria b_1/t_f	8.33	8.4 9.4	Plasti Compact
BOA BOS	Web criteria $h_w/t_{w,eq}$	102.6 14.36	84 105	Plasti Compact

For easy calculation the typical deck slab geometry is converted into equivalent rectangular shape and the bottom soffit slab is replaced by equivalent steel plate based on flexural reinforcement. Concrete is crack in flexural so the concrete is neglected in bottom part as shown in Fig5.12

Effective Width

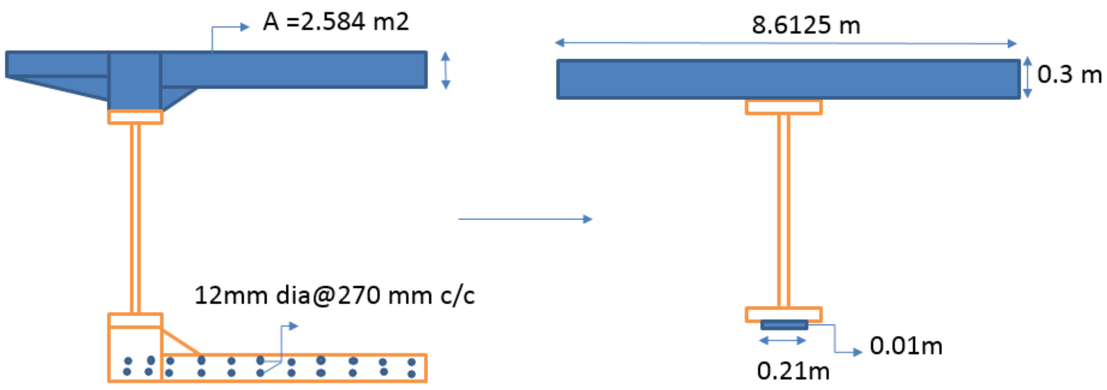


Figure 5.12: Geometric Notation

$$b_{eff} = \psi \left[\frac{B_1}{2} + 0.85x \right] \tag{5.5}$$

Where,

b_{eff}=effective width of slab

ψ=effective breadth ratio based on B/L

B=B_1=c/c distance between web = 4.6m

L = longitudinal span = 45m

x=cantilever projection WRT center of web=1.9m

MID SPAN

ψ=0.86

b_{eff}=3.3669m

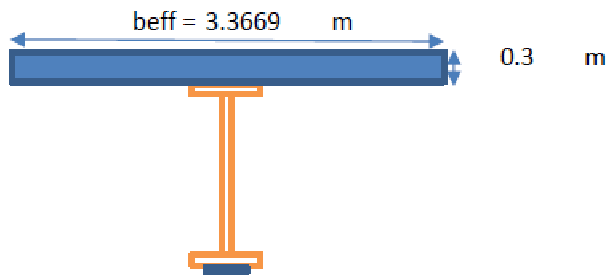


Figure 5.13: Effective Width at Mid Span

MID SUPPORT

ψ=0.41

b_{eff}=1.605m

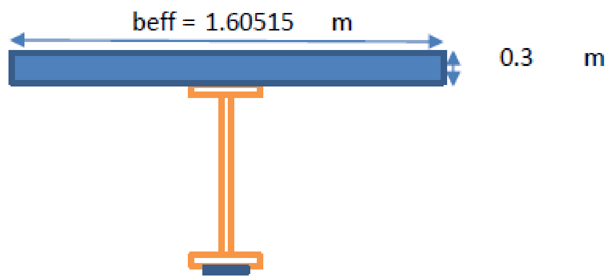


Figure 5.14: Effective Width at Mid Support

Position of Plastic Neutral Axis and Ultimate Moment of Resistance

- **Based on Area**

MID SPAN

Case 1 Plastic NA within the slab

$b_{eff}d_s > \alpha A_s$

$b_{eff}d_s$=1.01

$\alpha = \frac{f_y}{y_m 0.36 f_{ck}}$=25.25

$\alpha A_s = 0.903$

Satisfy the condition

$x_u = (\alpha A_s)/b_eff$

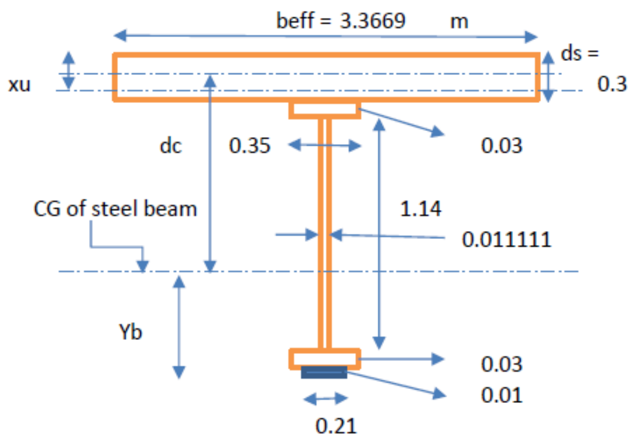


Figure 5.15: Position of Plastic NA

x_u=0.268m

$M_p = A_s F_y(d_c + 0.5d_s - 0.42x_u)/\gamma_m$

$M_p = 6688.8kNm$

MID SUPPORT

Case 1 In absence of tensile reinforcement

$M_p = Z_p f_y / \gamma_m$

Area $= A = 0.0357m^2$

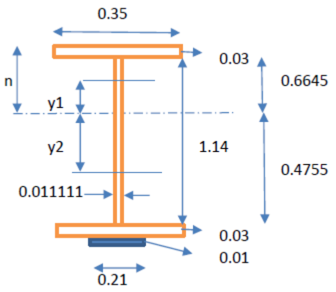


Figure 5.16: Position of NA Without Tensile Reinforcement

$Z_p = 0.017m^3$

$M_p = 3878.7kNm$

Case 2 In present of tensile reinforcement

F_s=Design tensile force in reinforcement $= 290.44kN$

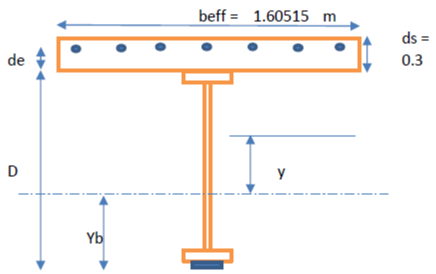


Figure 5.17: Position of Plastic NA with Tensile Reinforcement

Position of plastic NA in web

$y < \frac{D}{2} - t_f$

$M_p = M_p + F_s Z$

$M_p = 4115.78kNm$

- **Based on Stiffness**

MID SPAN

Case 1 Plastic NA in the Web

$x_u = d_s + t_f + (\alpha(A_s - 2A_f) - b_{eff}d_s)/2\alpha t_w$

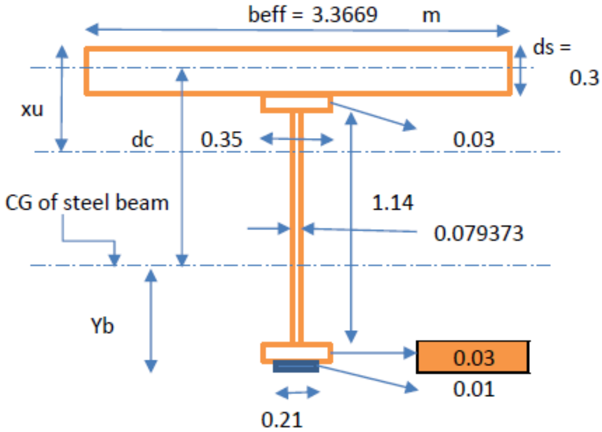


Figure 5.18: Position of Plastic NA

$x_u = 0.661m$

$M_p = f_y(A_s(d_c+0.08d_s)-2A_f(0.5t_f+0.58d_s)-t_w(x_u-d_s-t_f)(x_u+0.16d_s+t_f))/\gamma_m$

$M_p = 17697.7kNm$

MID SUPPORT

Case 2 In absence of tensile reinforcement

$M_p = Z_p f_y/\gamma_m$

Area $= A = 0.114m^2$

$Z_p = 0.039m^3$

$M_p = 8938.61kNm$

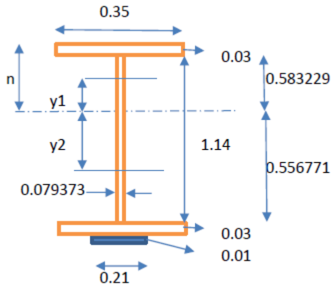


Figure 5.19: Position of NA Without Tensile Reinforcement

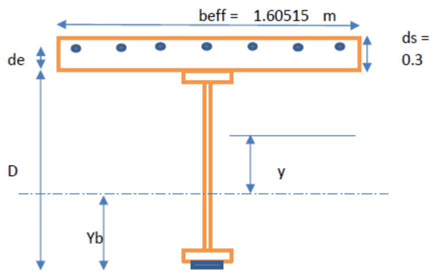


Figure 5.20: Position of Plastic NA with Tensile Reinforcement

Case 2 In present of tensile reinforcement

F_s=Design tensile force in reinforcement = $290.44kN$

Position of plastic NA in web

$y < \frac{D}{2} - t_f$

$M_p = M_p + F_s Z$

$M_p = 9182.87kNm$

The moment provided by of the section is less compared to moment required. Therefore the external prestressing tendons are provided for this remaining moment.The remaining moment are shown in Table5.10

Table 5.10: Required Moment for External Prestressing

Type	Location	$M_{p,req}$ kNm	$M_{p,pro}$ kNm	$M_{p,req}(kNm)$ for external prestressing
BOA	Mid span	25362.64	6688.8	18673.84
	Mid support	29483.57	4115.78	25367.80
BOS	Mid span	25297.22	17697.66	7599.56
	Mid support	29432.77	9182.86	20249.90

Design Against Vertical Shear

- **Case 1 Plastic Shear Resistance**

$V_d = V_n/\gamma_{mo}$=Design shear strength

$V_n = V_p = A_v f_{yw}/\sqrt{3}$ = Nominal plastic shear resistance

$A_v = t_{w,eq} h_w$= Shear area

$A_v = 12666.7mm^2$

$V_p = 1828.33kN$

$V_d = 1662.12kN$

- **Case 2 Shear Buckling Resistance**

$V_n = V_{cr}$

V_{cr}=Shear force corresponding to web buckling

$V_{cr} = A_v \tau_{cr}$

Local buckling involves a single panel, whereas global buckling involves multiple panels, with buckles extending over the entire depth of the web. The interactive buckling is rather complex and Interactive shear buckling mode is attributed to the interaction between global and local shear buckling modes and governs the shear buckling strength.

LOCAL SHEAR BUCKLING

$$\tau_{cr}^{e}, L = k_L \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t_w}{w}\right)^2 \tag{5.6}$$

$$k_L = 5.34 + 4\left(\frac{w}{h_w}\right)^2 \tag{5.7}$$

ν=poisson ratio=0.3

$k_L = 5.53$

$\tau_{cr}^{e}, L = 1598.44 N/mm^2$

GLOBAL SHEAR BUCKLING

$$\tau_{cr}^{e}, G = k_G \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t_w}{w}\right)^2 \tag{5.8}$$

$$k_G = \frac{36\beta}{\pi^2\sqrt{n}}[2((d/t_w)^2+1)(1-\nu^2)]^{3/4} \tag{5.9}$$

β=1.678

n=length reduction factor=L_1/l_1=0.9

$k_G = 589.86$

$\tau_{cr}^{e}, G = 8196.07 N/mm^2$

INTERACTIVE SHEAR BUCKLING

$$\frac{1}{\tau_{cr}^{e}, L} + \frac{1}{\tau_{cr}^{e}, G} = \frac{1}{\tau_{cr}^{e}, I} \tag{5.10}$$

$\tau_{cr}^{e}, I = 1337.58 N/mm^2$

$0.8\tau_y = 115.47 N/mm^2$

Therefore inelastic buckling will occur

$\tau_{cr}^{ie} = \sqrt{0.8\tau_{cr}^{e}, I\tau_y}$

$\tau_{cr}^{ie} = 393 N/mm^2 < \tau_y$

$\tau_{cr}^{ie} = 144.34 N/mm^2$

$\lambda_s = \sqrt{\tau_y/\tau_{cr}^{ie}} = 1$

Using buckling curve equation

$\tau_{cr}/\tau_y = 1 - 0.614(\lambda_s - 0.6) \qquad 0.6 \leq \lambda_s < \sqrt{2}$

$\tau_{cr} = 108.89N/mm^2$

$V_{cr} = 1379.25kN$

Buckling Resistance Moment

$$M_{cr} = \sqrt{\left(\frac{\pi^2 EI_y}{L_{LT}^2}\right)\left(G_{w,co}J_{co} + \frac{\pi^2 EC_{wco}}{L_{LT}^2}\right)} \tag{5.11}$$

$L_{LT} = 7.5m$

$G_{w,co} = nG = 69230.8N/mm^2$

$J_{co} = (1/3)(2b_f t_f^3) + (1/3)(h_w t_w^3) = 6.68E + 6mm^4$

$I_{y,co} = 2.1E + 8mm^4$

$I_{x,co} = 6.8E + 9mm^4$

$u_x = 3.04E - 6mm/N$

$C_w = 2.6E + 12Nmm^2$

$C_{w,flat} = 7.3E + 13mm^6$

$C_{w,co} = 1.5 + E14mm^6$

$M_{cr} = 6548.56kNm$

External Prestressing Tendon

Below calculation for BOA and combined result of BOA and BOS are shown in below Table5.11.

MID SUPPORT

Failure by yielding of steel

$M_{p,req} = M_{u,req} = 18673.84kNm$

$M_{u,req} = 0.9dA_p f_p$

A_p=Area of high tensile steel

$f_p = 1862N/mm^2$

$d = 1642.64mm$

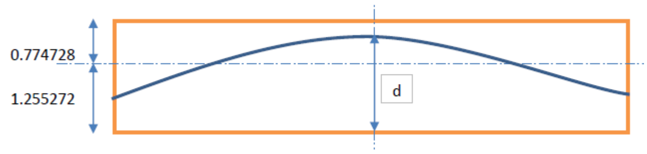


Figure 5.21: Position of NA Without Tensile Reinforcement

$A_p = 9215.51mm^2$

MID SPAN

$d = 1209.18mm$

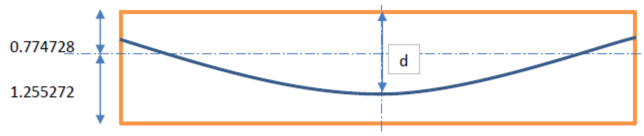


Figure 5.22: Position of NA Without Tensile Reinforcement

Using freyssinet system anchorage type

Using 2 No of 37k-15 tendon

Area of 37k-15=5180mm^2

Assume loss=15%

Total Prestressing force $= P = 16668.5kN$

Shear force are find out due to external prestressing tendons

$V_u = nPsin\theta$

$\theta = 4e/L = 0.0386$

$V_u = 11.23kN$

$V_{u,req} = 2563.05kN$

Table 5.11: Tendon and Shear Force Required for Stiffeners

Type	Location	d(mm)	Tendon	P(kN)	$v_{u,req}(kN)$
BOA	Mid support	1642.64	2,37k-15	16668.5	2563.05
	Mid span	1209.2			-
BOS	Mid support	1612.34	37k-15,19k-15	8334.25	2527.61
	Mid span	1150			-

Design of End Bearing Stiffeners

Below calculation for BOA and combined result of BOA and BOS are shown in below Table5.12.

Area of stiffeners required

$A_q > (0.8F_c\gamma_{mo})/f_y$

$A_q = 9021.94mm^2$

Aesthetic point of view stiffeners are provided only inside

Provide stiffeners of 3 flat of size 190 X 20mm

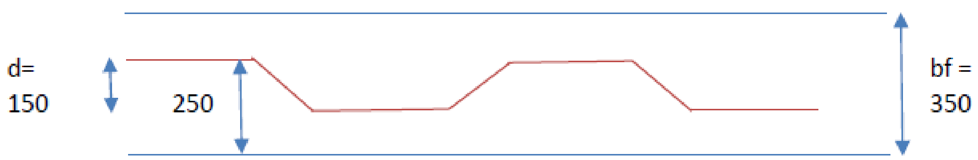


Figure 5.23: Maximum offset for stiffener

Area=11400$mm^2 > A_q$

(a) Check for out stand

$14t_q = 280mm$

bs=190mm $<$ 350mm

Hence, the criterion for the out stand has been satisfied

(b) Buckling check

$I_{xx} = 3.4E + 07mm^4$

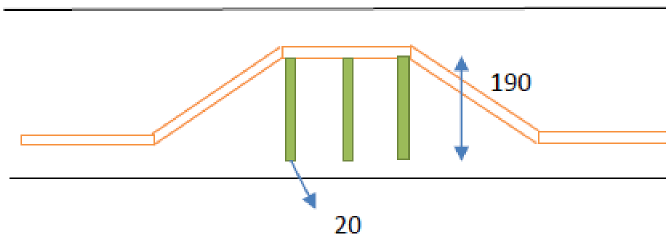


Figure 5.24: Position Stiffener

r_x=Radius of gyration = 54.85

$L_e = 798mm$

$\lambda = 14.55$

$F_{cd} = 225.5N/mm^2$

Buckling resistance of the stiffener

$P_d = fcdA = 2570.7kN > 2563.05kN$

Hence, the stiffener is safe against buckling

(c) Check for load bearing stiffener

$F_w = (b_1 + n_2)t_w(f_y/\gamma_{mo})$=Local capacity of web

$b_1 = 0 =$ bearing length

$n_2 = 75mm$

$F_w = 170.46kN$

Bearing stiffener is design for=$V_{u,req} - F_w = 2392.6kN$

Bearing capacity of stiffener alone=F_w

$F_w = (Af_y)/\gamma_m o = 2590.91kN > 2563.05kN$ ok

Design of Intermediate Stiffeners

(a) Minimum stiffeners

if $C/d \geq \sqrt{2}$

Assume $C = 1.8m$= spacing of stiffener

$I_s \geq 0.75dt_w^2$

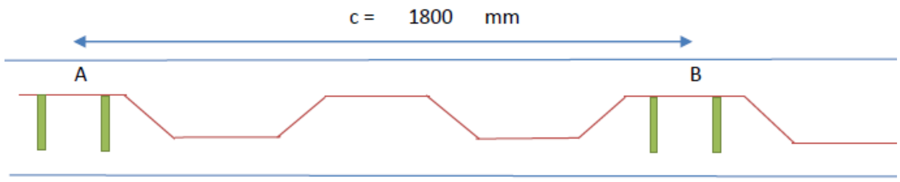


Figure 5.25: Spacing of Intermediate Stiffener

$I_s \geq 855000mm^4$

Try intermediate stiffener of two flats of 150 X 12mm

$I_s = 6.75E + 6mm^4$

(b) Check for outstand

$14t_q = 168mm$

bs=150mm < 168mm

Hence, the criterion for the out stand has been satisfied

(c) Buckling check

$F_q = (V - V_{cr})/\gamma_{mo}$

Where, V =factored shear force at 1.8m away from mid support=3836.4kN

V_{cr}=shear buckling resistance = 1379.25kN

Effective length of web equal to 20tw on each side of the center line of stiffener can be considered along with stiffener.

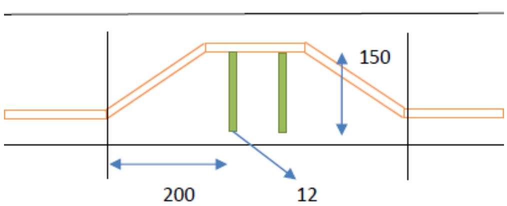


Figure 5.26: Effective Web Length

$20t_w = 200mm$

$I_x = 6.8E + 6mm^4$

Area=$10822.2mm^2$

$r_x = 25.11mm$

$\lambda = 31.78mm$

$f_{cd} = 209N/mm^2$

Buckling resistance of the stiffener=$F_{cd}A$=$2261.84kN > 2233.77kN$ ok

Intermediate stiffener subjected to external load should satisfy the following interaction equation

$F_q = 2233.77kN$

$F_{qd} = 2261.84kN$

$F_x = 525kN$ (class A max. wheel load)

$F_{xd} = F_{qd} = 2261.84kN$

$M_q = 0$

$F_q - F_x = 1708.77kN$

$0.98 < 1$Safe

Hence the stiffener is safe at point load

Connection Details

- **Design of Weld at Web Flange Junction**

Assume fillet weld on each side of the web

$q_w = (VAy)/2I_z$

$q_w = 1.05kN/m$

Provide 8mm fillet weld on both side

- **Weld for End Stiffener**

Assuming a weld on each side of the stiffener is

$q_1 = t_w^2/5b_s = 0.11kN/mm$

Length of weld = $1110mm$

Buckling resistance depends on the slenderness ratio of the web

$\lambda = 2.5(h_w/t_w)$

$n_1 = D/2$=600mm

$f_{cd} = 24.3N/mm^2$

Buckling resistance =291.6kN

$q_2 = 3.55kN/mm$

$q_1 + q_2 = 3.66kN/mm$

Force on each weld = $0.61kN/mm$

Provide 5mm fillet weld.

Table 5.12: Stiffener and Connection Detail

	BOA	BOS
End bearing stiffener	190x20,3No	190x20,3No
Intermediate stiffener	150x12,2No	-
Web to flange connection	8mm fillet weld	6mm fillet weld
Web to stiffener connection	5mm fillet weld	5mm fillet weld

Design of Web Splice

Below calculation for BOA

At a distance of 15m

Factored bending moment=10000kNm

Factored shear force=2000kN

Note:The fillet weld will lie in the plane of load and moment, hence it will be subjected shear due to torsion and vertical shear

$h_w = 1140mm$

Providing clearance = $50mm$

d_s=depth of splice plate = $1040mm$

Provide 2 No of splice plates on each side

t_s=Thickness of splice plate

$t_s = (t_w h_w^2)/(2d_s^2)$

t_s=10mm

BM resisted by the splice plate $M_s = MI_w/I_{gr}$

$I_{zz} = 9.3E - 3mm^4$

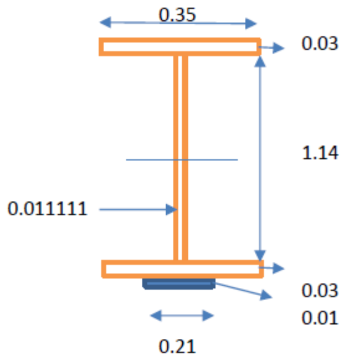


Figure 5.27: Effective Web Length

M_s=13.29$E + 8Nmm$

Thus splice plate subjected to

BM=664.94kNm

SF=1000kNm

Assume length of weld along span = 250mm

b_{sl}=Width of slot =50mm

d_{sl}=Depth of slot =940mm

Total length of weld = 5500mm

Resistance offered by the weld per mm length against translation

$P/L = 363.64N/mm$

Tacking moment of area @ A

$X = 125mm$

$Y = 520mm$

$r = 534.81mm$

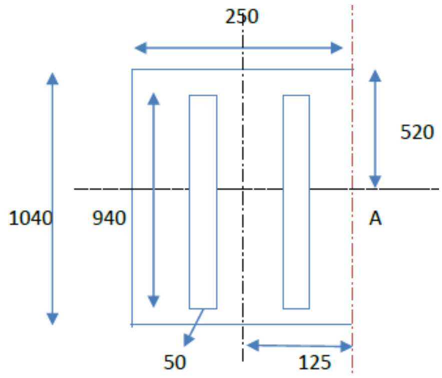


Figure 5.28: Web Splice

$I_{xx} = 5.5E + 8mm^4$

$I_{yy} = 3.1E + 7mm^4$

$J = I_{xx} + I_{yy}$=polar moment of inersia

$J = 5.8E + 8mm^4$

Resistance against rotation per mm length of weld at a distance from the CG

$S = Kr$

$K = M_s/J$=2.29

$S = 1223.90N/mm^2$

Total vertical component per mm length of weld

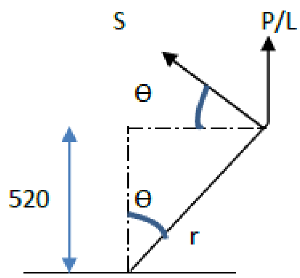


Figure 5.29: Web Splice Forces

$V = (P/L) + Ssin\theta = 363.64N/mm$

Total horizontal component per mm length of weld

$H = Scos\theta = 1223.73N/mm$

Resultant resistance per mm length=$\sqrt{H^2 + V^2}$=1276.61N/mm

Let the maximum shear stress intensity in the weld be q N/mm^2

Assume size of weld = $10mm$

$q = 182.37N/mm2 < 189N/mm^2$safe

Design of Flange Splice

Below calculation for BOA

Finding out tensile and compression force carried by flanges

At a distance of 15m

BM=10000kNm

Compressive force =$(MIb_{tf})/y_{top}$ =7187.89kN

Tensile force =$(MIb_{bf})/y_{bottom}$= 7796.97kN

Design of Butt Flange

At Top & Bottom Flange

Length of butt weld = $350mm$

Thickness of plate = $30mm$

Strength of weld = $2100kN$

Design of Welding

(a) At Top Flange

$W_{fs} = 330mm$

$t_p = 15mm$

$L_{fs} = 770mm$

Maximum size of weld=13.5mm

Assume size of weld = $s = 12mm$

Required length of weld = $4527.52mm$

Available Weld length = $1870mm$

Assume width of slot $= W_s = 50mm$

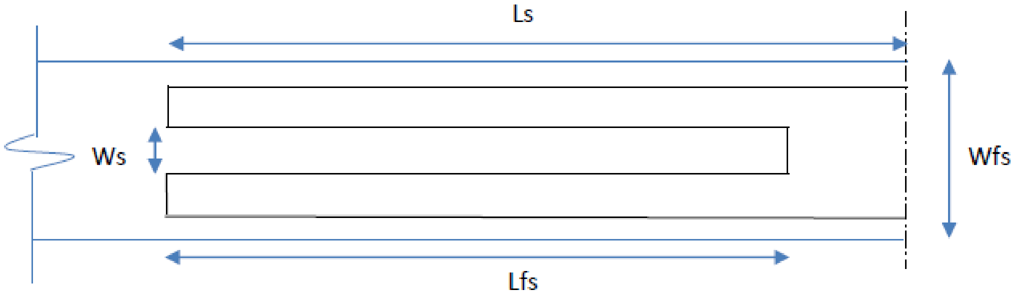


Figure 5.30: Flange Splice

No of slot $= 2$

Length of slot $= L_s = 670mm$

length of weld provided $= 4550mm > 4527.52mm$ ok

(a) At Bottom Flange

$W_{fs} = 330mm$

$t_p = 15mm$

$L_{fs} = 835mm$

Maximum size of weld=$13.5mm$

Assume size of weld $= s = 12mm$

Required length of weld $= 4911.17mm$

Available Weld length $= 2000mm$

Assume width of slot $= W_s = 50mm$

No of slot $= 2$

Length of slot $= L_s = 735mm$

length of weld provided $= 4940mm > 4911.17mm$ ok

Design of Shear Connector

Below calculation for BOA and combined result of BOA and BOS are shown in below Table5.13.

- **Ultimate limit state (strength criteria)**

Longitudinal shear per unit length=V_l

$$V_l = \sum \left(\frac{V A_{ec} Y}{I} \right) dl, ll \tag{5.12}$$

V=the vertical SF due to DL and LL separately at each state of load history

$V_{dl} = 2424.58kN$

$V_{ll} = 1811.82kN$

Dead load and Live Load parameters

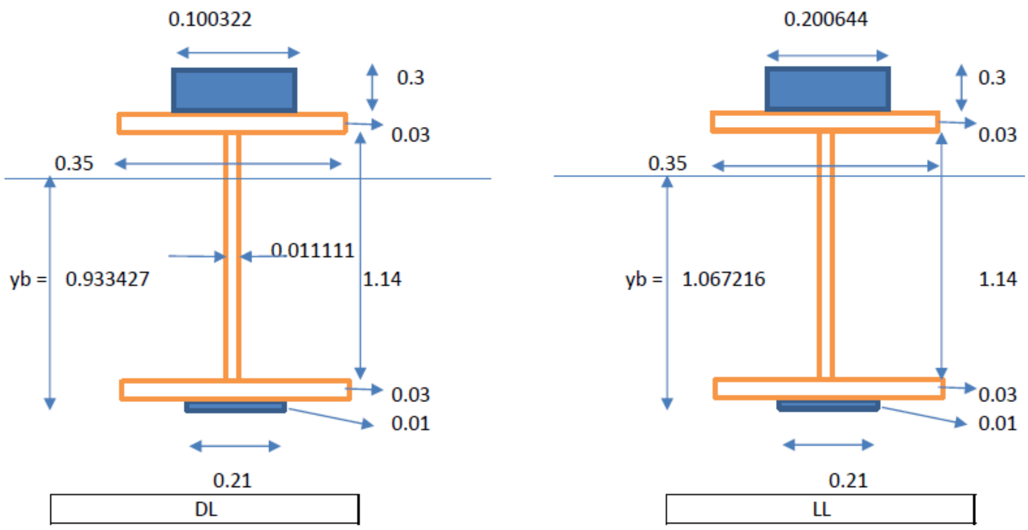


Figure 5.31: Composite Section due to DL and LL(all dimention are in m)

Y=CG distance of transformed concrete area from NA

$Y_{dl} = 0.427m$

$Y_{ll} = 0.293m$

A_{ec}=the transformed compressive area of concrete above NA

$A_{ec}, dl = 0.0301m^2$

$A_{ec}, ll = 0.0602m^2$

I_{zz}=Moment of inersia

$I_{zz}, dl = 0.01959m^4$

$I_{zz}, ll = 0.02358m^4$

Therefor Longitudinal shear $= V_l = 2942.83kN/m$

- **Spacing of Shear Connector**

$$S_l = \frac{\sum Q_u}{V_l} \tag{5.13}$$

Q_u=Ultimate static strength of 1 shear connector

Use $Q_u = 103kN$

If 3 transverse stiffeners are placed in 1 horizontal line

$S_l = 105mmc/c$

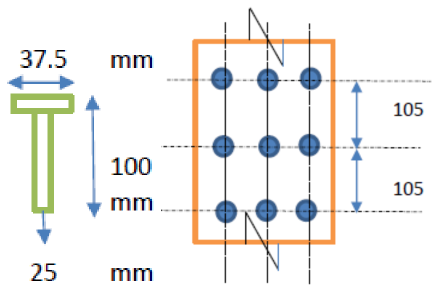


Figure 5.32: Shear connector and Longitudinal section of girder showing spacing of shear connector

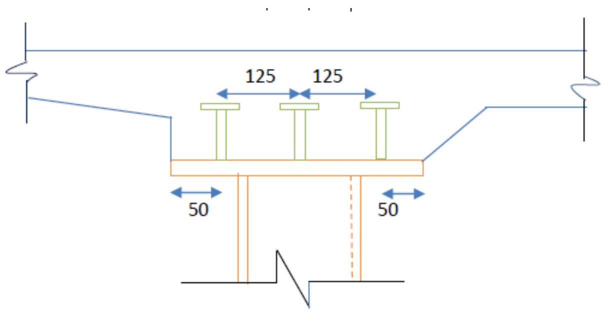


Figure 5.33: Cross section of girder showing spacing of shear connector

- **Limiting criteria for spacing of shear connectors when the slab is contact over the full length**

$S < 21t_f(\sqrt{250/f_y})$

$S = 630mm$ ok

Design of shear reinforcement as per IRC22:2008

The strength and amount of reinforcement to be checked for following 2 conditions

Dia. and Spacing

Top steel provided in slab 12mm dia @ 200mm c/c

Bottom steel provided in slab 8mm dia @ 120mm c/c

$A_b = 419.2mm^2$

The shear force in N/mm of longitudinal girder

$Q = (N_cP)/S = 2942.86kN/m$

Bottom transverse reinforcement should not be less than

$2.5Q/f_y = 1772.81mm^2$

Extra transverse reinforcement is required = $1353.61mm^2$

Hence,provide 16mm dia bar @145mm c/c

Extra transverse bottom reinforcement is provided =$1387.7mm^2$

- **Maximum spacing of shear connectors**

(a) $600mm$

(b)3 x Thickness of concrete slab $=900mm$

(c)4 x Height of shear connector $=400mm$ Therefor Maximum spacing is minimum of above 3

$400mm > 105mm$ ok

- **Minimum spacing of shear connectors**

$75mm < 105mm$ ok

Table 5.13: Details of Shear Connector and Shear Reinforcement

Type	Transverse direction		Longitudinal direction	Shear
	No	Spacing(mm)	Spacing(mm)	Reinfocement
BOA	3	125	105	16mm dia @145mm c/c
BOS	3	125	115	16mm dia @165mm c/c

Check for Shear Capacity of End Panel

Below calculation for BOA

$V_d = 1828.33kN$

$V_n = 1662.12kN$

$V_{cr} = 1379.25kN$

$H_q = 1.25V_d\sqrt{1-(V_{cr}/V_d)}$=Longitudinal shear

$R_{tf} = 566.33kN < V_n = 1662.12kN$ ok

Check for Moment Capacity of End Panel

Below calculation for BOA

$M_{tf} = H_q d/10 = 129.12kNm$

$I = 4.86E + 9mm^4$

$M_q = 1227.27kNm > 129.12kNm$ ok

Check for Applied Shear Stress due to Torsion

$\tau < \tau_{cr}$

$T = 920.63kNm$

q=shear flow=$T/(2A_o)$

$q = 56.70N/mm$

V_v=Vertical shear=qh_w=64.63kN

$\tau = V/A = 5.67N/mm^2 < \tau_{cr} = 108.89N/mm^2$ ok

5.5 Drawings

All drawings are shown below are with respect to BOA calculation

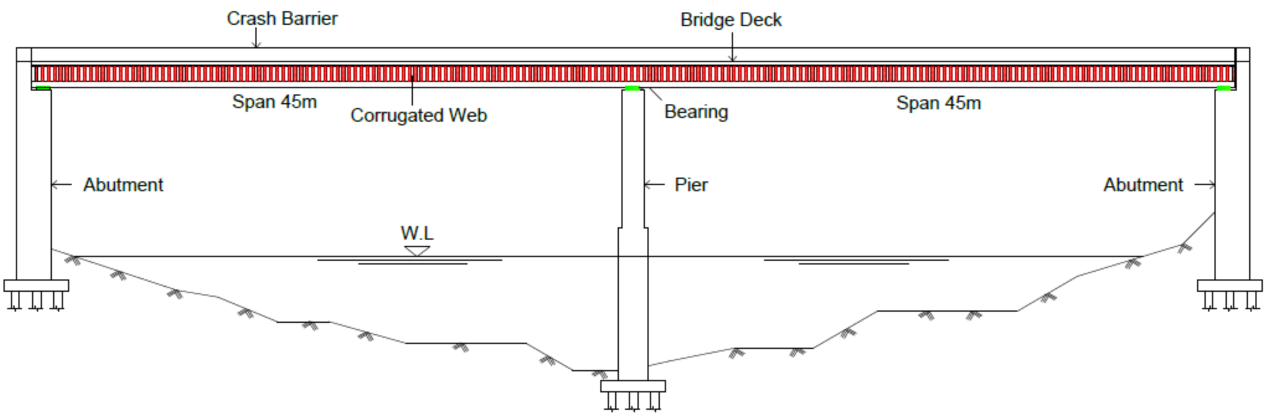


Figure 5.34: Basic Component of Corrugated Web Bridge

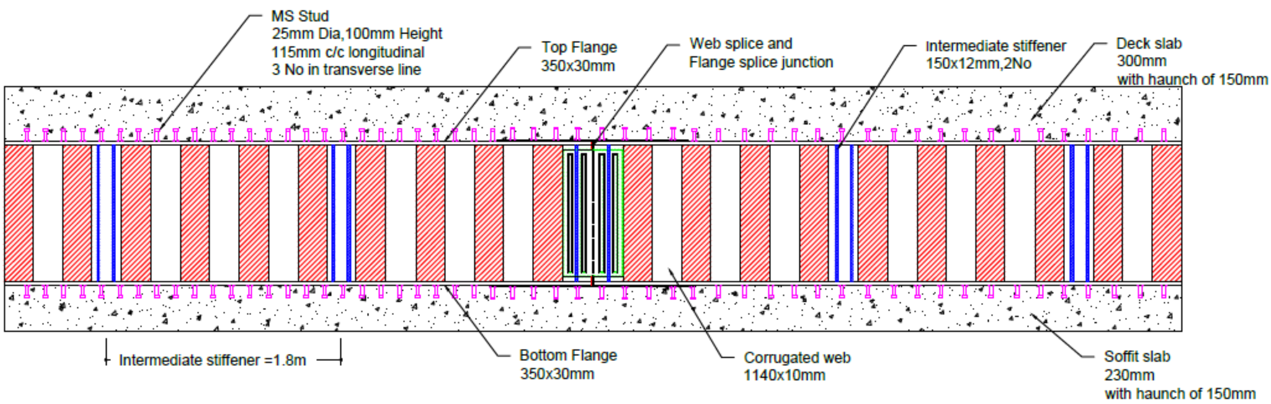


Figure 5.35: Elevation of Longitudinal Girder

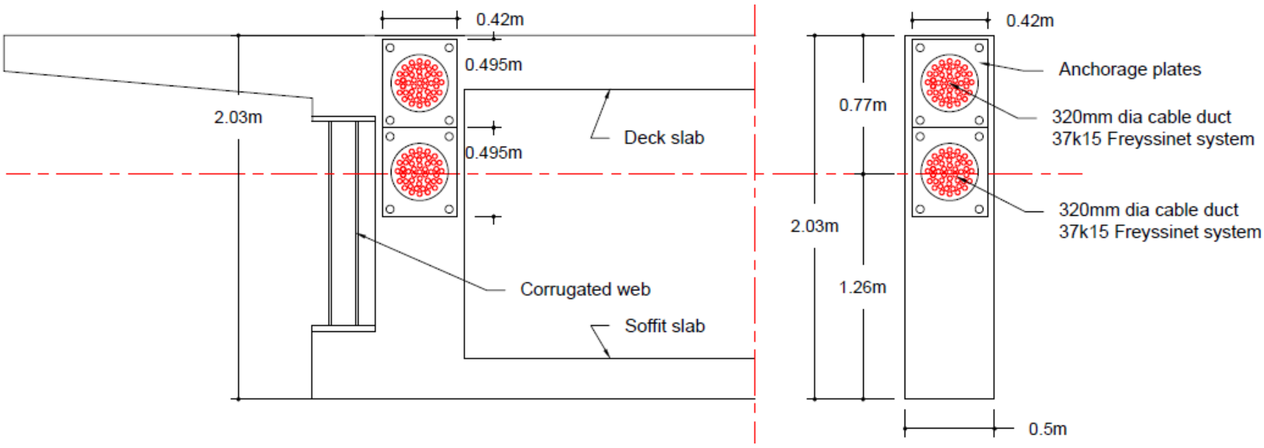


Figure 5.36: Details of Prestressing Tendons at End Block

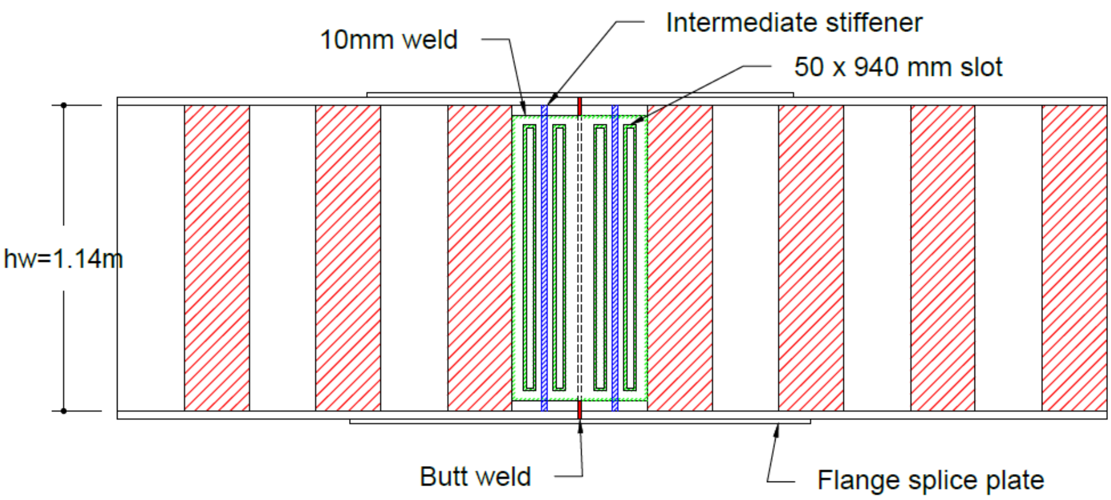


Figure 5.37: Details of Web Splice

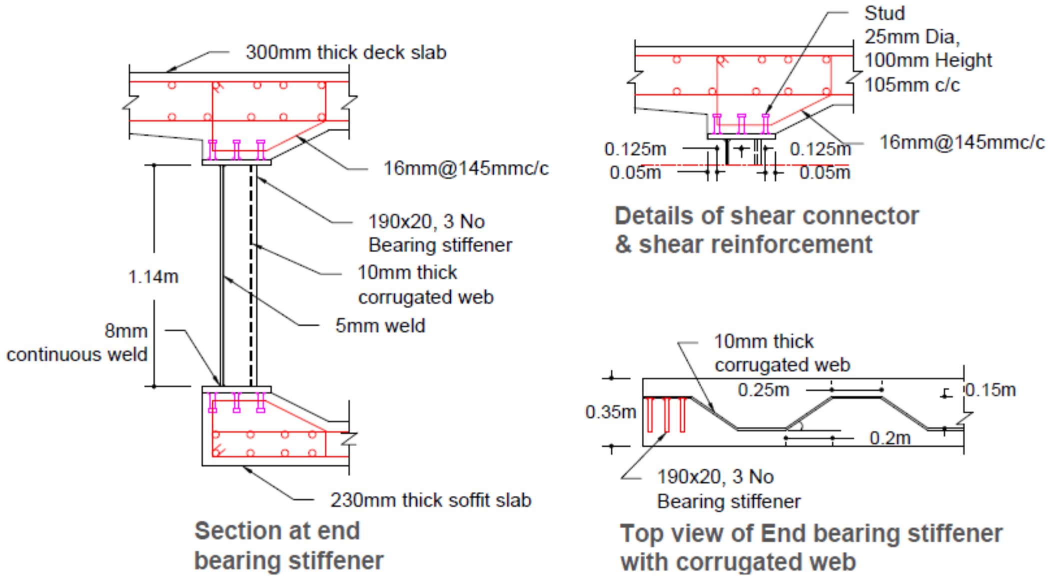


Figure 5.38: Details of End Bearing Stiffener and Shear Connector and Shear Reinforcement

5.6 Estimation of Cost

Quantity of concrete, Quantity of shuttering, Quantity of prestressing steel and Quantity of HYSD steel are found out same as conventional box girder bridge.The summary of this items are listed in below Table5.14. Only the Quantity of structural steel, shear connector and weld are to be finding out.Here Quantity are to be found out for BOA calculation and same quantity are to be found out for BOS calculation.

Table 5.14: Summary of Items

Sr No	Description	Quantity	Unit
1	Concrete	387.04	m^3
2	Shuttering	1346.04	m^2
3	HT steel	14687.4	kg
4	HYSD steel	37455.5	kg
5	Scaffolding	387.04	m^3

Estimation of Structural Steel

Table 5.15: Estimation of structural steel

Sr No	Description	No	Length(m)	Width (m)	Height(m)	Quantity(m^3)
1	Web plate	2	100	0.01	1.14	2.28
2	Flange plate	4	90	0.35	0.03	3.78
3	Vertical stiffener	150	0.15	0.012	1.14	0.306
4	Bearing stiffener	18	0.19	0.02	1.14	0.078
5	Web splice	16	0.4	0.01	1.04	0.067
6	Flange splice	16	1.54	0.23	0.015	0.085
7	Shear connrctor	7724	-	0.025	0.1	7723.29
Total volume of steel in (m^3)						6.60
Total steel in kg						51778.4

Estimation of Weld for Connection

Table 5.16: Estimation of Weld Length for Connection

Sr No	Description	No	Length(m)	Weld size(mm)	Quantity(m)
1	Web to Flange connection	8	100	8	800
2	Vertical Stiffener to Web connection	298	1.12	5	334.51
3	Bearing stiffener to Web connection	36	1.12	5	40.32
4	Web splice to Web connection	32	5.5	10	176
5	Flange splice to Flange connection	32	4.55	12	145.6
6	Flange to Flange Butt Weld connection	16	0.35	30	5.6
Total connection length(m)					1502.03

Bill of Quantity

Table 5.17: Bill of Quantity for Box Girder with Corrugated Steel Web

Sr No	Description	Quantity		Rate		Cost(Rs)
1	Concrete	387.04	m^3	4000	Rs/m^3	1548165
2	Shuttering	1346.04	m^2	110	Rs/m^2	148064.79
3	H.T steel	14687.4	kg	140	Rs/kg	2056232.136
4	HYSD steel	37455.5	kg	50	Rs/kg	1872775.75
5	Scaffolding	387.04	m^3	160	Rs/m^3	61926.6
6	Structural steel	51778.4	kg	45	Rs/kg	2330028.52
7	Weld connection	1502	m	250	Rs/m	375506.67
8	Shear connector	7723.29	No	50	Rs/No	386164.28
Total Cost(Rs)						8778863.75

5.6.1 Estimation of Unit Weight

Table 5.18: Unit Weight

Sr No	Description	Weight(kN)
1	Concrete	9676.03
2	H.T steel	144.08
3	Structural steel	507.95
Total weight(kN)		10328

5.6.2 Summary of Result

Estimation of cost and unit weight of different items like concrete, reinforcement, H.T steel tendons, Structural steel, Shear connector are carried out for prestressed concrete box girder bridge with corrugated steel webs as per BOA and BOS calculation. It is found that the cost and unit weight of the box girder bridge with corrugated steel webs is less compare to conventional box girder bridge.The summary of result are shown in below Table5.19

Table 5.19: Comparison of Cost and Weight

Type		Cost		Weight	
		Total(Rs)	Rs/m	Total(kN)	kN/m
Conventional PC box girder bridge		10424002	115822	11702	130
PC box girder bridge with corrugated steel webs	BOA	8778864	97543	10328	115
	BOS	8266257	91847	10266	114

As per the above result we can say that the PC box girder bridge with corrugated steel webs is 13% lighter compare to conventional PC box girder bridge. So decrease the cost up to 21%.

Chapter 6

Parametric Study

6.1 General

The various span to depth ratio, design alternatives are required to be evaluated for quantity, costing and weight of the superstructure to arrive at effective economical span to depth ratio. To obtain the most effective economical span to depth ratio parametric study was done for 40m, 45m and 50m for two span continuous box girder bridge with corrugated steel webs by taking various span to depth (L/D) ratios. Also with respect to particular L/D ratio changing the Web height to corrugation depth h_w/d ratio and finding out the shear buckling capacity and lateral torsional buckling capacity for different span.

6.2 Results of Analysis

The analysis was done based on grillage analogy method in staad pro software. The design bending moment, design shear force and design torsional moment for 40m,45m, and 50m spans with different L/D and h_w/d ratio are tabulated in Table6.1 6.2 6.3 for all alternatives.

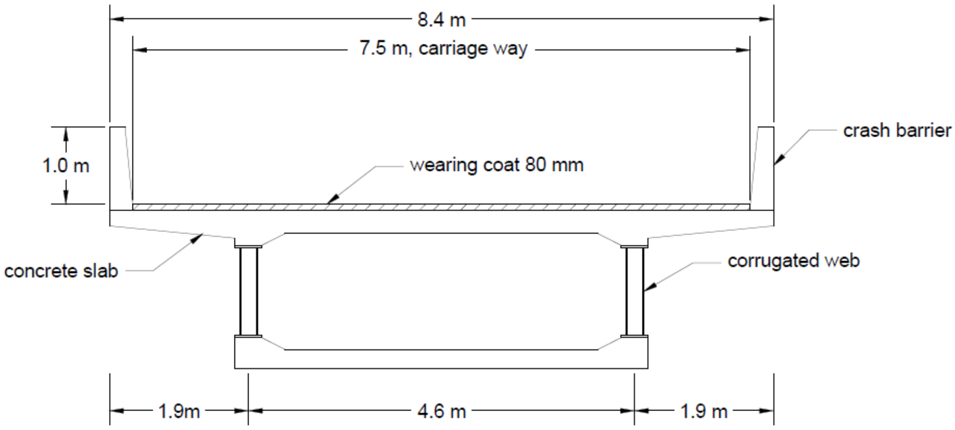


Figure 6.1: C/S Details for 40m,45m,50m of 2 span continuous bridge

Table 6.1: 40m Span Design SF,TM and BM

Span m	D mm	d mm	SF(kN) Mid support		TM(kNm)		BM(kNm) Mid support		BM(kNm) Mid span	
			BOA	BOS	BOA	BOS	BOA	BOS	BOA	BOS
40	750	150	3979.8	3900.3	916.4	1058.5	24469.1	23961.2	21301.8	2987.7
		180	3979.5	3900.1	921.9	1061.1	24472.6	23961.5	21302.9	20986.8
		210	3979.1	3900	927.5	1063	24475.7	23962.1	21303.6	20986.3
40	1000	150	3989.0	3901.1	878.0	1026.8	24512.9	23959.8	21344.2	20997.7
		180	3988.7	3901.3	883.8	1029.1	24517.1	23963.5	21345.5	20998.4
		210	3988.4	3901.3	889.8	1030.6	24522.0	23964.8	21347.1	20997.9
40	1200	150	3995.4	3901.6	865.5	1026.0	24543.7	23959.0	21377.5	21007.3
		180	3995.3	3901.7	871.8	1028.4	24549.3	23961.3	21379.5	21006.8
		210	3995.2	3901.8	878.2	1029.9	24555.7	23963.8	21381.8	21006.7

Table 6.2: 45m Span Design SF,TM and BM

Span	D	d	SF(kN)		TM(kNm)		BM(kNm)			
m	mm	mm	Mid support				Mid support		Mid span	
			BOA	BOS	BOA	BOS	BOA	BOS	BOA	BOS
45	750	150	4230.4	4202.3	966.6	1112.2	29491.2	29435.1	25330.8	25277.6
		180	4229.4	4202.1	971.7	1115.2	29489.3	29436.4	25328.9	29917.5
		210	4228.3	4202.0	977.5	1117.4	29487.3	29437.8	25326.9	25278.3
45	1000	150	4234.3	4204.0	940.2	1100.4	29494.6	29435.1	25346.3	25288.1
		180	4233.2	4203.7	946.3	1103.5	29492.6	29435.8	25344.3	25288.0
		210	4232.0	4203.6	952.6	1105.7	29490.5	29436.9	25342.2	25288.3
45	1200	150	4236.4	4204.9	920.6	1089.3	29483.6	29432.8	25362.6	25297.2
		180	4235.2	4204.7	927.1	1092.1	29481.6	29434.6	25360.5	25296.5
		210	4234.1	4205.2	933.7	1094.0	29479.6	29441.0	25358.2	25298.7

Table 6.3: 50m Span Design SF,TM and BM

Span	D	d	SF(kN)		TM(kNm)		BM(kNm)			
m	mm	mm	Mid support				Mid support		Mid span	
			BOA	BOS	BOA	BOS	BOA	BOS	BOA	BOS
50	750	150	4526.5	4498.0	1030.9	1183.6	35464.9	35412.4	29950.5	29896.2
		180	4525.4	4497.6	1036.8	1186.6	35463.1	35413.2	29948.7	29895.8
		210	4524.3	4497.4	1042.8	1188.8	35461.2	35414.2	29946.8	29895.6
50	1000	150	4530.4	4499.7	998.8	1164.6	35467.2	41655.6	29968.5	29902.2
		180	4529.3	4499.6	1005.2	1167.6	35465.4	35416.7	29966.5	29908.9
		210	4528.2	4499.6	1011.7	1169.8	35463.5	35419.4	29964.4	29910.1
50	1200	150	4532.8	4500.7	974.7	1148.4	35465.8	35412.3	29982.7	29918.2
		180	4531.7	4500.7	981.4	1151.5	35464.0	35415.5	29980.6	29919.7
		210	4530.5	4500.8	988.3	1153.6	35462.1	35418.9	29978.4	29921.4

6.3 Results of Buckling behavior

To know the shear buckling behavior and lateral torsional buckling behavior of corrugated web the parametric study was conducted. The parametric study for buckling behavior consist of changing h_w/d ratio with L/D ratio. The Local buckling, Global buckling, and the Interactive buckling stresses as listed below. The lateral torsional buckling capacity was also calculated to check the torsional behavior.The different stresses and moment value are listed below Table6.4. Corresponding graphical variation are also shown in Fig.6.2 6.3

Table 6.4: Buckling Behavior for 40m,45m,50m Span

L_D_d (m_mm_mm)	L/D	h_w/d	Local N/mm^2	Global N/mm^2	Interactive N/mm^2	M_{cr} kNm
40,45,50_750_150		5.0	1694.6	22372.6	1575.3	4793.9
40,45,50_750_180	53.3	4.2	1483.6	29934.9	1413.6	5315.4
40,45,50_750_210		3.6	1298.3	38451.6	1255.9	5874
40,45,50_1000_150		6.7	1624.6	12054.8	1431.7	5775
40,45,50_1000_180	40.0	5.6	1413.6	16129.5	1299.7	6367.1
40,45,50_1000_210		4.8	1228.3	20718.4	1159.6	7004
40,45,50_1200_150		8.0	1598.4	8196.1	1337.6	6548.6
40,45,50_1200_180	33.3	6.7	1387.5	10966.4	1231.6	7182.7
40,45,50_1200_210		5.7	1202.2	14086.5	1107.6	7867.8

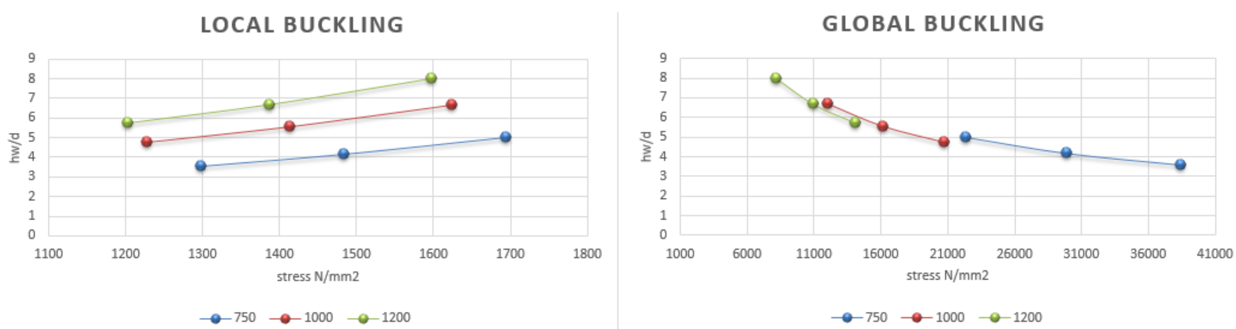


Figure 6.2: 40m,45m,50m span Local and Global Shear

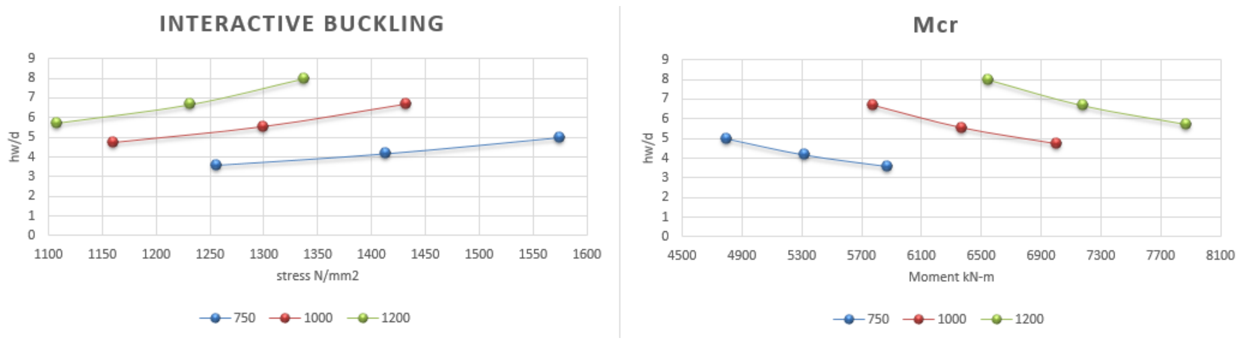


Figure 6.3: 40m,45m,50m span Interactive Shear and Torsional Bucklking

6.4 Results of Estimation

The design was done by prepared spreadsheet. The overall analysis methodology, step by step design procedure and estimation of cost and weight is described in chapter 5 for 45m length of two span continuous bridge for L/D ratio=37.5 and h_w/d ratio=8. For all the various spans and span to depth ratio deck slab and longitudinal girder is designed.

Initially the flange dimension and web dimension (Geometry of corrugated web) are assumed. For every span when L/D ratio decreases the requirement of external prestressing and requirement of stiffener was increasing so increasing the cost of the bridge superstructure. For every span when L/D ratio increases the requirement of external prestressing and requirement of stiffener was decreasing so decreasing the cost of the bridge superstructure.

The concrete cost, reinforcement cost, wearing coat cost, girder steel cost, shear connector cost and connection cost for 40m, 45m and 50m respectively with different L/D ratio and h_w/d ratio are are calculated and summary of result are shown in Table6.5 6.6 6.8. The deck slab concrete, slab reinforcement and wearing coat cost does not affects the L/D ratio. Total cost of super structure is mainly affected by girder steel cost and external prestressing. Table6.5 and Fig.6.4 shows that L/D ratio 33.33 and h_w/d ratio 5.71 is most economical L/D ratio for 40m span among all L/D ratio and

h_w/d ratio alternatives. Table6.6 and Fig.6.5 shows that L/D ratio 37.5 and h_w/d ratio 5.71 is most economical L/D ratio for 45m span among all L/D ratio and h_w/d ratio alternatives. Table6.8 and Fig.6.6 shows that L/D ratio 41.7 and h_w/d ratio 5.71 is most economical L/D ratio for 50m span among all L/D ratio and h_w/d ratio alternatives.

Table 6.5: Estimation for 40m span

L_D_d	L/D	h_w/d	Cost (Rs/m)		Weight(kN/m)	
m_mm_mm			BOA	BOS	BOA	BOS
40_750_150	53.33	5.00	94687.84	92922.47	111.16	110.99
40_750_180		4.17	94889.09	92816.87	111.19	110.99
40_750_210		3.57	94738.54	93109.40	111.22	111.04
40_1000_150	40	6.67	92536.58	86992.57	113.09	112.63
40_1000_180		5.56	92828.35	85849.86	113.14	112.60
40_1000_210		4.76	93285.40	85629.51	113.20	112.62
40_1200_150	33.33	8.00	91044.77	83365.93	114.59	114.00
40_1200_180		6.67	91456.40	83611.69	114.66	114.07
40_1200_210		5.71	91828.22	83149.50	114.72	114.09

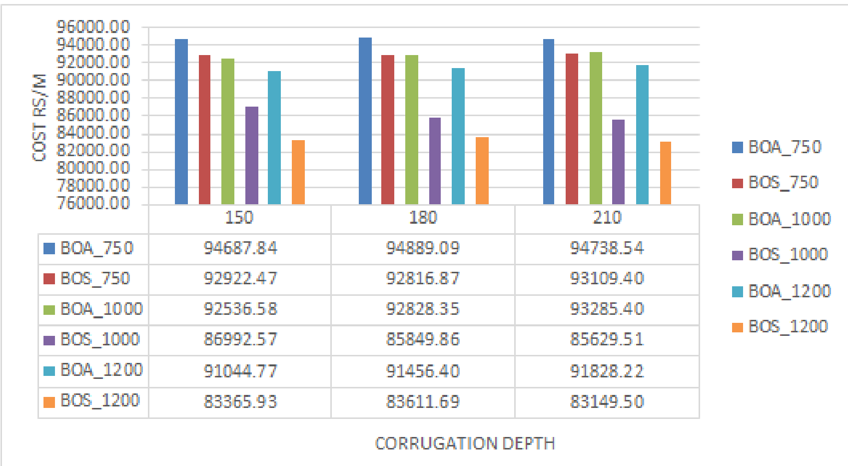


Figure 6.4: 40m Span Cost Comparison

Table 6.6: Estimation for 45m span

L_D_d m_mm_mm	L/D	h_w/d	Cost (Rs/m) BOA	Cost (Rs/m) BOS	Weight(kN/m) BOA	Weight(kN/m) BOS
45_750_150	60	5.00	100578.15	97182.72	112.22	111.02
45_750_180		4.17	100783.81	97127.20	112.24	111.04
45_750_210		3.57	101092.60	97429.00	112.29	111.09
45_1000_150	45	6.67	97093.96	94073.09	114.20	112.86
45_1000_180		5.56	97115.56	91225.05	114.25	112.70
45_1000_210		4.76	97469.46	91159.14	114.30	112.75
45_1200_150	37.5	8.00	97542.93	91847.30	114.76	114.07
45_1200_180		6.67	97835.89	91659.44	114.80	114.10
45_1200_210		5.71	98250.49	91976.34	114.87	114.17

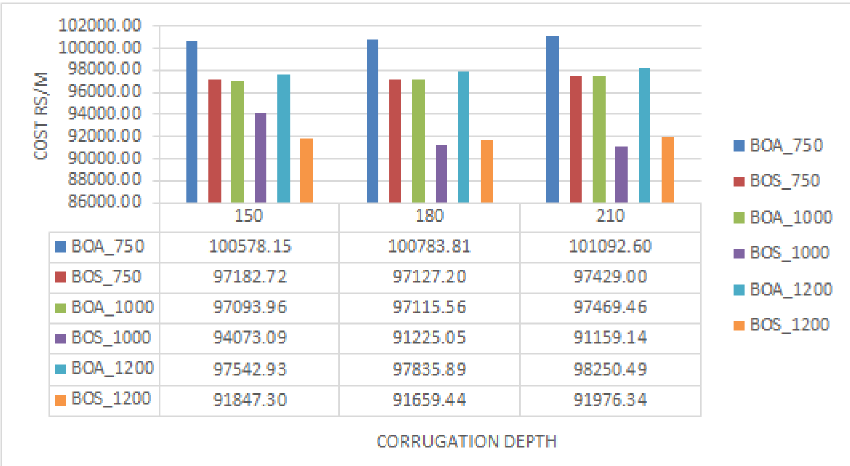


Figure 6.5: 45m Span Cost Comparison

Table 6.7: Estimation for 50m span

L_D_d	L/D	h_w/d	Cost (Rs/m)		Weight(kN/m)	
m_mm_mm			BOA	BOS	BOA	BOS
50_750_150	66.7	5.0	107107.38	104729.55	111.94	111.63
50_750_180		4.2	107304.55	105003.53	111.97	111.68
50_750_210		3.6	107544.25	104851.23	112.00	111.72
50_1000_150	50	6.7	102639.10	97016.96	113.69	113.19
50_1000_180		5.6	102939.01	96753.64	113.74	113.20
50_1000_210		4.8	103220.04	96886.78	113.78	113.27
50_1200_150	41.7	8.0	100592.58	94273.88	115.17	114.61
50_1200_180		6.7	100948.25	92937.23	115.23	114.57
50_1200_210		5.7	101340.66	92701.66	115.29	114.61

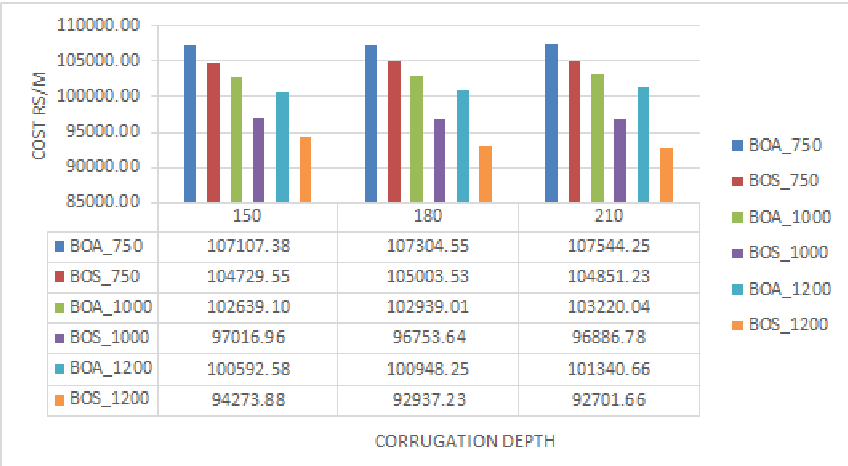


Figure 6.6: 50m Span Cost Comparison

Percentage Reduction in Cost Between BOA and BOS

Table 6.8: Percentage reduction in cost between BOA and BOS

Span	Depth of Girder (mm)		
	750	1000	1200
40	1.92%	7.24%	8.78%
45	3.53%	5.20%	6.15%
50	2.28%	5.85%	7.55%

6.5 Summary

Parametric study is carried out to find out the effective economical L/D ratio for 40m, 45m, 50m.It is found that as per BOS calculation the cost of the bride is less compare to as per BOA calculation. It is found that L/D ratio 33.33 and h_w/d ratio 5.71 are the most economical for 40m, L/D ratio 37.5 and h_w/d ratio 5.71 are the most economical for 45m and L/D ratio 41.7 and h_w/d ratio 5.71 are the most economical for 50m spans respectively.

Chapter 7

Summary and Conclusion

7.1 Summary

The main objective of the work was to study the behavior of convention PC continuous box girder bridge superstructure and then replacing the the concrete web with corrugated steel web. And comparing the cost and weight of the bridge superstructure. Parametric study was done for 40m, 45m, and 50m spans, for various girder depth (D) and for various corrugation depth (d).

The bridge is analyzed using grillage analogy method in staad pro software. Excel spreadsheets are prepare for deck slab, longitudinal girder. For conventional girder design is done as per IRC:18-2000. For composite girder design is done as per IRC:22-2008 and IS:800-2007, web, shear connector, stiffeners and weld connections are design.

Dead load, superimposed dead load and live load are considered for analysis and design. In dead load self weight of steel girder and deck slab are considered. In SIDL wearing coat, kerb, crush barrier, and parapet load are considered. And in live load class AA tracked and class A vehicles of IRC loading are considered.

Total 54 alternatives (27 BOA and 27 BOS) are analyzed using grillage analogy method in staad pro software and design of all 54 alternatives are done using prepared excel spreadsheet. Conventional PC box girder bridge and box girder bridge with corrugated steel webs and its alternatives are designed to satisfy bending and shear. Estimation and costing of all alternatives are carried out to find out the economical (minimum cost) and safe span to depth ratio. In costing concrete cost, shuttering cost, scaffolding cost, structural steel cost, connection cost and shear connectors cost are considered.

7.2 Conclusion

Based on above study the following conclusions are drawn:

- Maximum live load moment is carried out when the two class A loading moving at a time on two lanes on the minimum crash barrier distance.

- As per the literature review the shear buckling behavior and lateral torsional buckling behavior is studied in detail.

- As per IRC:18-2000 for PC box girder bridge and IRC:21-2008 for composite girder varies aspects are studied in detail.

- The PC box girder bridge with corrugated steel webs is 13% lighter compare to conventional PC box girder bridge. So decrease the cost up to 21%.

- Initially the flange dimension and web dimension (Geometry of corrugated web) are assumed. For every span when L/D ratio decreases the requirement of external prestressing and requirement of stiffener was increasing so increasing the cost and weight of the bridge superstructure. For every span when L/D ratio increases the requirement of external prestressing and requirement of stiffener was decreasing so decreasing the cost and weight of the bridge superstructure.

- It is observed that as L/D ratio increases total B.M and S.F increases. Therefor to satisfy the B.M external prestressing tendons increases and for S.F check the stiffeners are increases.

- As the L/D ratio and h_w/d ratio decreases the interactive shear buckling (governing) decreasing and torsional buckling capacity is increasing.

- As the span increases the total cost of super structure per meter decrease because less external prestressing tendons.

- Number of shear connectors are more in BOA calculation compare to BOS calculation.

- As the depth of girder increases the percentage reduction in cost increases in BOS with respect to BOA.

- The corrugated steel plates posses higher out of plane stiffness and shear buckling strength with lesser thickness so the corrugated web can effectively save the steel volume and reduce the cost and the unit weight.

- By using corrugated web the stress, weight, cost are decreasing with increasing lateral torsional buck-ling moment.

7.3 Future Scope of Work

- In this study straight profile of 2 span continuous bridge is taken, the work can be extended for 3 span, 4 span continuous bridges with straight and skew profile.

- The prestress concrete box girder bridge with corrugated steel webs can be applied to long span bridges, for example extradosed bridges and cable stayed bridges with spans in excess of 200m.

- In this study the sub structure cost is not compared with super structure cost. The work can be extended by considering both sub structure and super structure cost and comparing overall economy of bridge as a whole.

Appendix A

List of Papers Published/Communicated

- Pitolwala Zuzar, Prof. N.C.Vyas and Prof. Jahanvi Suthar," *"Prestressed Concrete Box Girder Bridge with Corrugated Steel Webs"*, International Conference on Innovations in Concrete, Hydrabad, India, 23-25 October, 2013 (Abstract Selected)

- Pitolwala Zuzar, Prof. N.C.Vyas and Prof. Jahanvi Suthar," *"Prestressed Concrete Box Girder Bridge with Corrugated Steel Webs"*, International Conference on Trends and Challenges in Concrete Structures, Ghaziabad, India, 19-21 December, 2013 (Abstract Selected)

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