ANALYSIS AND DESIGN OF COMPOSITE BOX GIRDER BRIDGE

By

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DEPARTMENT OF CIVIL ENGINEERING

AHMEDABAD-382481

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ANALYSIS AND DESIGN OF COMPOSITE BOX GIRDER BRIDGE

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering

(Computer Aided Structural Analysis And Design)

By

Priyanka Pal

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May 2010

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.

Priyanka Pal

Certificate

This is to certify that the Major Project entitled "Analysis and Design of Composite Box Girder Bridge" submitted by Miss Priyanka Pal (08MCL011), 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 her under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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Abstract

Composite steel-concrete box girders are commonly used in curved bridges,interchanges,and ramps. Composite box girders are particularly strong in torsion and efficiently resist the large torsional demands created by horizontal bridge curvature and vehicle centrifugal forces.

This work was carried out for study, the study of behavior of composite box girder bridges as per new revised codes. The analysis of the composite bridge is done by using professionally available 'SAP software' for dead load, superimposed load and moving load as a class A and class 70R IRC loading. The finite element model is used to generate composite bridge superstructure model in SAP2000.Analysis and design of rectangular simply supported box girder& simply supported trapezoidal, continuous box girder & continuous trapezoidal girder bridge has been carried out .

The motive behind present study is to prepare some useful interface for preliminary design of composite road bridge system as per IRC:22 and IS 800:2007 provisions and then to find an economical section for the system.There are many changes in design provision of 2007 code.

The other aim of study is to determine the most suitable and economical section so as to achieve satisfactory performance of the structure satisfying new code provisions.

Economy mainly depends on various factors like span and superstructure cross sectional dimensions . The present study includes parametric study on steel-concrete composite two lane road bridge with various alternatives consisting of variation in span and span to depth ratio and designs as per IS 800:2007.

Parametric study was done for calculation of most economical L/D ratio for 15m, 20m and 25m spans as per IS 800-2007. For this all 9 alternatives costing was done with quantity analysis and rate analysis as per current market rates. From these most economical L/D ratios with minimum cost was found 13.64, 16.67 and 14.71 respectively for span 15m, 20m and 25m.

Study was also done to find out the cost difference between simply supported box and trapezoidal section and also continuous box and trapezoidal section design .It is found that in simply supported rectangular box girder bridge is 4.74% economical than simply supported trapezoidal bridge while in continuous span trapezoidal open section is 0.03 % economical than continuous rectangular box span.

Acknowledgements

I am deeply indebted to **Prof. N.C. Vyas** for his constant guidance and motivation. He has devoted significant amount of his valuable time to plan and discuss the thesis work. Without his experience and insights, it would have been very difficult to do quality work.
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Abbreviation Notation and Nomenclature

A_{st} .... Area of Steel
BM .... Bending moment
b_1 .... Width of stiff bearing on the flange
b_{bf} .... Width of bottom flange
b_s .... Outstand width of the stiffener
b_{tf} .... Width of top flange
c .... Spacing of stiffeners in the web
c/c .... Centre to centre
$C.G$ .... Center of gravity
d .... Effective depth of girder
D .... Total depth of girder
DL .... Dead load
d_s .... Thickness of deck slab
d_w .... Depth of web
d_{ws} .... Depth of web splice
E_c .... Modulus of elasticity of concrete
E_s .... Modulus of elasticity of steel
f_{cd} .... Design compressive stress
f_{ck} .... Characteristic compressive strength of concrete
F_q .... Stiffener force
F_{qd} .... Design resistance of the intermediate stiffeners
F_x .... External load or reaction at the support
F_{xd} .... Design resistance of load carrying stiffener corresponding to buckling
F_v .... Yield strength of the tension field
F_y .... Yield stress of the material
F_{yf} .... Yield stress of the flange
F_{yw} .... Yield stress of the web

F_{wd} .... Design stress of weld
h .... Outstand of stiffener
$I.R.C.$ .... Indian Road Congress
I_{xx} .... Moment of inertia about the major axis
I_{yy} .... Moment of inertia about the minor axis
K .... Constant
K_v .... Shear buckling coefficient
L .... Length (Span of Bridge)
L/D .... Span to Depth Ratio
l_o .... The distance between points of contra-flexure/zero moments
L_f .... Length of fillet weld
LL .... Live load
L_w .... Length of butt weld
m .... Modular ratio
M_B .... Moment in the short span direction
M_L .... Moment in the long span direction
M_{DL} .... Dead load moment
M_{LL} .... Live load moment
M_{SIDL} .... Super imposed dead load moment
n .... Number of girders
n_1 .... Dispersion of the load through the web at 45 degree
n_2 .... Dispersion length
N_c .... Number of shear connectors
P_c .... Design strength of one shear connector
p_t .... Percentage of Steel
q_w .... Force on weld
q_1 .... Minimum force for stiffener to web weld connection
q_2 .... Force for stiffener to web weld connection due to external loading
R .... The reaction of the girder at the support

R_{nw} .... Strength of the weld
R_{tf} .... Resultant longitudinal shear
r_{yy} .... Radius of gyration in minor axis
s .... Weld leg length
SF .... Shear force
$SIDL$ .... Superimposed dead load
t_{bf} .... Thickness of bottom flange
t_e .... Effective thickness of plate
t_{tf} .... Thickness of top flange
t_w .... Thickness of web
V .... Vertical shear force
V_{cr} .... Shear force corresponding to web buckling
V_L .... Longitudinal shear force
V_n .... Nominal shear resistance
W .... Total load
X_u .... Neutral axis from top of slab
Z_{pe} .... Plastic section modulus
Zxx_{bot} .... Section modulus from bottom
Zxx_{top} .... Section modulus from top
β_b Ratio of the elastic modulus to plastic modulus with respect to compression fiber
ϵ .... Yield stress ratio
τ_b .... Shear stress corresponding to web buckling
τ_c .... Permissible shear stress
τ_{cr} .... Elastic critical stress
σ_v .... Actual shear stress
γ_{mo} .... Partial safety factor against yield stress and buckling
μ .... Poisson's ratio

Contents

List of Tables

List of Figures

Chapter 1

Introduction

1.1 General

Steel concrete composite construction has been increasingly popular in advance countries like USA and Uk and is fast catching-up in developing countries. It is more ideally suitable for flyovers and bridges in metros with minimum disruption to the community. This type of few constructions is now coming up in India during last decade because of the potential benefits. The recent examples of successful implementation of such are grade separator at Andresganj and Mayapuri flyover constructed along ring road in Delhi, the cable stayed Vidhyasagar setu (second Hoogly Bridge) and flyover in Garihat at Ghatkopar in Mumbai. In composite construction, there is most effective utilization of materials like concrete in compression and steel in tension. Shear connectors are the main part for resisting horizontal shear in steel-concrete composite road bridge. Composite section has higher stiffness and higher ductility of steel that gives better seismic resistance. Some main advantages of composite construction is like saving in steel weight about 30to 50 % over the non-composite beam and gives greater stiffness means they can be shallower for the same span. Medium span composite bridges/flyovers are normally constructed from welded built-up steel plate girders with variety of reinforced concrete decks. Box girders, though technically advantageous for longer spans and look very attractive in some cases, are comparatively expensive due to higher fabrication costs, if continuous plated flange is used. Due to un symmetrical nature of the cross-section, shrinkage always causes compression in steel top flange / sagging bending in steel section, lending to greater deflection.

1.2 General Box Girder Bridges

The use of box girder in elevated highway construction several advantages. The highway may be curved in plan, resulting in torsion even when the loading is symmetrical, and the supports may not be disposed in best way to resist torsion. The torsional strength inherent in the closed box section, with its ability to distribute resisting moments and shears across the width of bridge, is therefore advantageous. The interior of box can be used for services, and in larger span could be used for traffic. Along with these advantages, the box -shape girder is an aesthetically pleasing structure. The design complication of warping, distortion and shear lag still occur. Intermediate diaphragms are used to limit distortion

1.3 Advantages of Steel-Concrete Composite Construction

- Most economic utilization of materials viz. concrete is in compression and steel is in tension and shear.
- High ductility of steel material leads to better seismic resistance and fatigue of the composite section.
- Composite sections have higher stiffness and hence experience lesser deflection than non-composite steel section.
- Keeping span / loading unaltered, lower structural steel section will be required

for composite construction compared to non-composite steel construction.

- More use of steel ensures better quality control for the major part of the structure.
- Compared to concrete bridges of longer spans, faster construction can be achieved by utilizing rolled and/or prefabricated components. Also, speedy construction facilitates quicker return of the invested capital.
- Quality assurance of the steel material along with availability of proper paint system
- Life cycle cost analysis is competitive compared to all concrete or non composite structures.
- Saving in overall depth of the girder in turn reduces the cost of embankment in a flyover / bridge, when compared with RCC spans.
- Reduction in overall weight of structure compared to RCC construction, which reduces foundation costs.
- Cost of form work is lesser compared to RCC construction.
- Cost of handling and transportation is minimized for using major part of the structure fabricated in the workshop

1.4 Behaviour of Composite Box Girder Bridge

When subjected to bending, box girder behaves similarly to plate girders; they are subjected to buckling, shear lag and local slenderness effects. When subjected to a torsional moment, either from eccentric loading of from curvature the structure, the box forms a much stiffer structure than a plate girder. Due to application of eccentric load on box girder, it results in Combination of three components -bending, torsion and distortion. For example, considering a general loading on a box section, as shown in Figure 1.1, in which a single vertical eccentric load is replaced by sets of forces representing vertical, torsional and distortional loading. The general loading in Figure 1.4 can be represented as two different components of loading, one causing bending and the other causing torsion as shown in Figures 1.1(b) and 1.1(c), respectively. The torsional loading component can be subdivided further into a pure torsional component and a distortional component as shown in Figures 1.1(d) and 1.1(e), respectively. Although the pure torsional component will normally result in negligible longitudinal stresses, the distortional component will always tend to deform the cross-section, thus creating distortional stresses in the transverse direction and warping stresses in the longitudinal direction. The distortion of the cross-section will be resisted by cross frames and diaphragms and hence an accurate analysis involves evaluating the distortional warping and shear stresses and the associated distortional bending stresses in the transverse frames. The resistance to torsion in a box girder in the form of shear flow around the box as shown in fig 1.1(d).

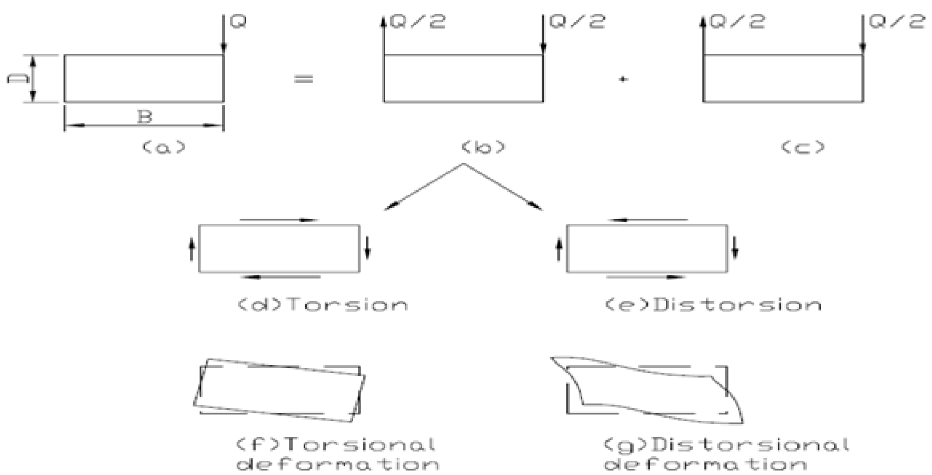


Figure 1.1: Idealization of eccentric loading in box girder

1.5 Shear Connectors for Composite Box

Shear connectors are the most important structural element in a composite bridge deck, provided at the junction of the concrete slab and longitudinal steel girders. The main function of the shear connector is to prevent the separation between the steel girder and the in situ concrete slab by transferring the horizontal shear force along the contact surface without slip. In the case of composite girder deck, the deflection is comparatively less than that of non composite girder decks due to the increased moment of inertia of the composite section. Commonly used type of connectors is as per IRC 22-1986. There are three main types of shear connectors, viz. Rigid shear connectors, Flexible shear connectors and Anchorage type shear connectors. The majority of the effective connectors should be within the effective width of girder, connector outside the effective width will be required to carry local or transverse effects or the small longitudinal shear spread beyond the effective width.

1.6 Objective of Study

- To study the composite behavior in the composite road bridge superstructures consisting of concrete slab and steel box girder joined together with shear connectors and study the difference in behavior of box girder section and trapezoidal section. For these two section spans are taken,
  1)Simply supported span
  2) continuous span
- To study shear connectors design.
- To study the behavior of composite road bridge superstructure using different type of vehicular loading as per IRC:6-1966.
- To study codal stipulation related to composite box girder bridge.
- In box Girder Bridge, study will be carried out for section as shown in fig.1.2

- To carry out parametric study to find out most economical l/d ratio.
- To study cost analysis of box girder and trapezoidal girder for simply supported and continuous span.

1.7 Scope of Work

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

- Mainly two types of work will be carried out, one is analysis and other is design of steel concrete composite box Girder Bridge.
- Analysis of superstructure is carried out based on SAP2000 software and design is done using excel work sheet.
- For design and analysis Road bridges considered are with:
  1)Simply supported
  2)continuous
  a. Closed rectangular box section.
  b. trapezoidal section.
- For analysis live load is considered as different vehicular load like 70R Tracked/wheeled And Class A as per IRC:6-1966..
- Design is based on Limit state method.
- For design of steel concrete composite box girder road bridge concrete deck slab, box girder of steel, diaphragms, shear connector and different types of stiffeners will be designed.
- Shear connector design.
- Parametric study for economical span to depth ratio

Problem formulation:

a. Closed rectangular box section:

Span: 20m

Carriageway: 7.5 m

Wearing coat: 85 mm

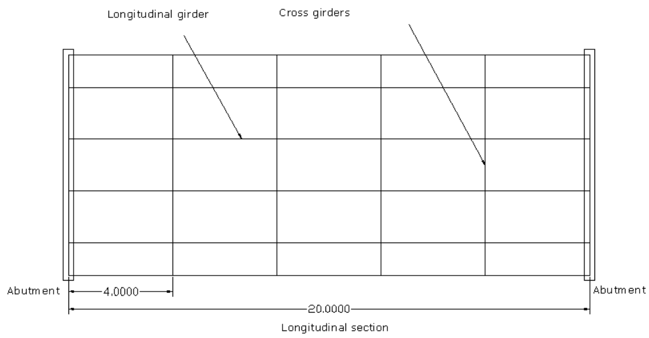


Figure 1.2: Longitudinal Section of Simply Supported Bridge

a. Open trapezoidal section:

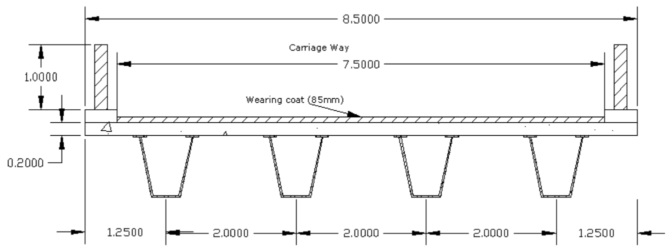


Figure 1.3: Bridge section

1.8 Organization of Major Project

The rest of the thesis is organized as follows. Notice how chapters are referred by means of *slash*refhandoff command. Also see in handoff chapter how handoff is labeled.

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 composite action between steel-concrete composite road bridge with shear connectors.

Chapter 3, *Behaviour of Box Girder Bridge*, In this chapter,behaviour of box girder , stresses and Comparison of box girder with I-girder are included.

Chapter 4, *Design Philosophies*, Includes the different type of loading on bridge, superstructure and code stipulation as per the IRC:6 and IRC:22-2008.

Chapter 5, *Analysis Of Composite Box-Girder Bridge*, Includes the analysis of bridge structure using professional software and design procedure of deck slab and longitudinal girder.

Chapter 6, *Design Of Composite Box-Girder Bridge*, Design of deck slab , longitudinal girder and shear connector for simply supported box girder are included in this chapter.

Chapter 7, *Estimation of cost*, Includes estimation of deck slab concrete and reinforcement quantity, box girder component quantity, connection quantity and rate analysis.

Chapter 8, *Parametric study for economical span to depth ratio*,Includes the parametric study for 15m, 20m and 25m span with various L/D ratio to find out the economical L/D ratio. Cost comparison between closed section and open section.In this parametric study trials are taken by varying L/D ratio.

Chapter 9 Includes summary, conclusion and future line of action for major project.

Chapter 2

Literature Review

2.1 General

The analysis, design and experimental procedure and their conclusions for steel concrete composite box girder bridge are presented by many authors. The different analytical and experimental models are prepared to simulate the actual behavior of composite box girder bridge. There are many assumptions involved with the procedures. The researchers came up with new realistic concepts from time to time to achieve the more realistic behavior. In this chapter, the study carried out by different authors based on composite action in bridge due to shear connectors is presented.

2.2 Literature Review

Various literatures have been referred for behavior of shear connectors and brief review of literature is discussed below.

Handbook On Composite Construction [1] :In this book *"Composite construction"* the book 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 table for composite sections and pigeaud's curves are also given in this handbook.

Dr. T K Bandyopadhyay et al.[2]in their paper on *"Design Aspects of Steel-concrete composite Bridges".* reviewed the behavior and design of composite bridge structure. This paper includes advantages of steel-concrete composite concrete, assessment of different code stipulations, design procedure for super structure and loading parameters. All code stipulations given in paper consider the section as compact section and non-compact section. While IS considers position of Neutral axis. Only two types of vehicular loading are considered for analysis purpose. Design of composite I-girder on the bases of the yield stress of the steel girder, the yield stress of the reinforcing steel in slab and the ultimate strength of the equivalent concrete stress block are carried out in this paper.

Design guide for composite box Girder Bridge [3]:In this book *"A Critical Review On curved composite bridge is done"*This book provide guidance on the design of composite box girder, generally in accordance with BS5400.The guide describes features of initial and detailed design and explains how the standard is applied to the design of these structure .How diagrams are provided as further guidance to the use of the standard. Two worked example are included based on designs for actual structures.

Dr. K. Natarajan [4] in their paper on *"Analysis and Design of Steel-concrete composite Bridges".* reviewed the behavior and design of composite bridge structure. This paper includes advantages of steel-concrete composite concrete, Fundamental theory highlighting the behavior of composite section in terms of stress and strain variation is mentioned, parameters of analysis and design of composite bridges, design procedure for superstructure of composite bridge, limit state design and some of the code provisions of AASHTO,BS,CAN/CSA were included for comparison. Theories in case of full slip, no slip and partial slip are explained.

Chang-Su Shim et al.[5] in their paper on *"Desgn Of Shear connection in com-*

*posite steel and concrete with precast decks ".*carried out study on design of shear connection in composite steel and concrete bridge with precast deck. The author discussed design considerations and experimental work of the push test. For this experiment, shear pockets filled with a non-shrinkable mortar with same elastic modulus as precast concrete are used. The ultimate strength and fatigue endurance are investigated through push tests. Based on experiment, it was concluded that ultimate strength decreases as the thickness of the bedding layer increases.

Steel-Concrete Compoite Bridges by David collings [6]: In this book *"This book mainly emphesis on steel-concrete bridge behavior,analysis and design "*this book provides behavior of boxes and shear connector for composite boxes. Computed a problem for box bridge carrying railway over bridge.

B.I.Maisel et al. [7] in their paper on *"Concrete box-girder bridges ".*Study of types of structural action of box-section beams.

Punashri P.Phadnis et al.[8] in their paper on *"Analysis And Design Of 3-Span Continous steel concrete bridges ".*In this paper attempt is made to highlight the advantage of composite construction. Analysis and design of 3 spans continuous composite bridge has been carried with reference to IS code provisions. Comparative study is carried out for cost effectiveness of composite superstructure construction with the reinforced concrete construction.

Design Of Modern Steel Highway Bridges[9]: In this book *"This book mainly emphesis on design of composite bridge "* This book gives general behavior of box girder bridge and carried our design of straight steel box girder using AASHTHO specification.

Design Of Steel structure[10]: In this book *"This book mainly emphasis on design of steel structure "*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.

Design and construction of highway bridges[11] :In this book *"This book* *mainly emphasis on design of bridge structure* "has described the procedure involved in analysis and design of composite bridges

Design of bridges[**12**]:In this book *"This book mainly emphasis on analysis and design of bridge structure* "has described the procedure involved in analysis and design of composite bridges .

2.3 Relevant Codes

The composite road bridge super structure shall be designed as per the following IRC codes.

IRC: 6-2000[**13**] in this code *"This code includes loading on bridges"*

IRC: 21-1987[**14**] in this code *"This code includes design of concrete bridges"*

IRC: 22-2000(Draft)[**15**] in this code *"This code includes design on bridges according to limit state"*

Chapter 3

Behaviour of Box Girder Bridge

3.1 General

A box girder is formed when two web plates are joined by a common flange at both the top and the bottom. The closed cell which is formed has a much greater torsional stiffness and strength than an open section and it is this feature which is the usual reason for choosing a box girder configuration. Although steel or steel-concrete composite box girders are usually more expensive per tonne than plate girders, because they require more fabrication time, they can lead to a more economic solution overall.

3.2 Behaviour and Stresses

3.2.1 Bending, Torsion and Distortion

The general case of an eccentric load applied to a box girder is in effect a combination of three components - bending, torsion and distortion. As a first step, the force can be separated into two components, a pair of symmetric vertical loads and a force couple, as shown in Figure 3.1. However, torsion is in fact resisted in a box section by a shear flow around the whole perimeter and the couple should in turn be separated into two parts, representing pure torsion and distortion, as shown in Figure 3.1. The first two components, vertical bending loads and a torsional shear flow are externally applied forces, and they must be resisted in turn at the supports or bearings. The third component, distortional forces, comprises an internal set of forces, statically in equilibrium, which do not give rise to any external reaction. Distortional effects depend on the behavior of the structure between the point of application and the nearest positions where the box section is restrained against distortion.

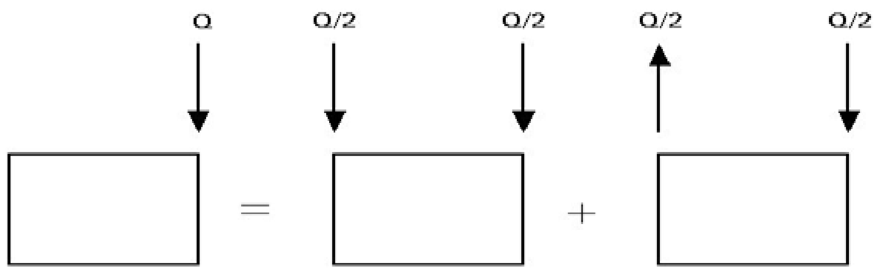


Figure 3.1: Separation of an Eccentric Applied Load into Two Components

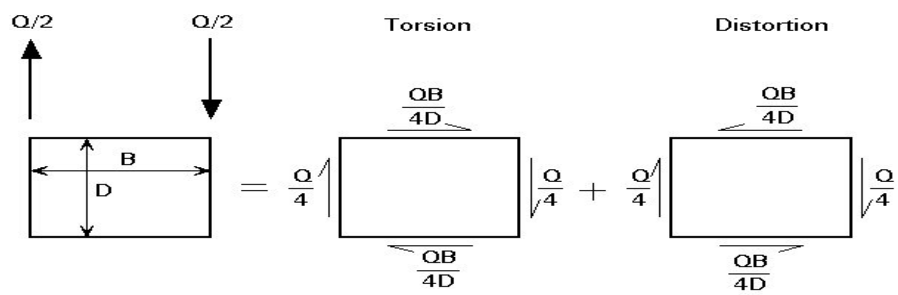


Figure 3.2: Separation of Force Couple into Torsion and Distortion Component

The first two components, vertical bending loads and a torsional shear flow are externally applied forces, and they must be resisted in turn at the supports or bearings. The third component, distortional forces, comprises an internal set of forces, statically in equilibrium, which do not give rise to any external reaction. Distortional effects depend on the behavior of the structure between the point of application and the nearest positions where the box section is restrained against distortion.

3.2.2 Torsion and Torsional Warping

The theoretical behavior of a thin-walled box section subject to pure torsion is well known and is treated in many standard texts. For a single cell box, the torque is resisted by a shear flow which acts around the walls of the box. This shear flow (force/unit length) is constant around the box and is given by q = T/2A, where T is the torque and A is the area enclosed by the box. (In Figure 2 the torque is QB/2 and the shear flow is Q/4D.) The shear flow produces shear stresses and strains in the walls and gives rise to a twist per unit length, which is given by the general expression:

However, it is less well appreciated that this pure torsion of a thin walled section will also produce a warping of the cross-section, unless there is sufficient symmetry in the section. This is illustrated in Figure 3.3 for a rectangular section that is free to warp at its ends. However, in practice boxes are not subject to pure torsion; wherever there is a change of torque (at a point of application of load or at a torsional restraint) there is restraint to warping; because the free warping displacements due to the different torques would be different (restraint is high, for example. over intermediate supports where torsion is restrained). Such restraint gives rise to longitudinal warping stresses and associated shear stresses in each wall of the box.

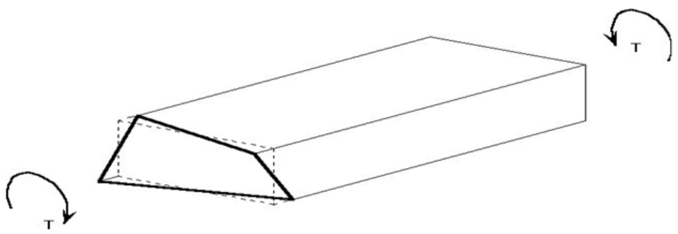


Figure 3.3: Warping of Rectangular Box Subjected to Pure Torsion

of course, for a simple uniform box section subject to pure torsion this warping is unrestrained and does not give rise to any secondary stresses. But if, for example, a box is supported and torsionally restrained at both ends and then subjected to applied torque in the middle, warping is fully restrained in the middle by virtue of symmetry and torsional warping stresses are generated. Similar restraint occurs in continuous box sections which are torsionally restrained at intermediate supports. This restraint of warping gives rise to longitudinal warping stresses and associated shear stresses in the same manner as bending effects in each wall of the box. The shear stresses effectively modify slightly the uniformity of the shear stress calculated by pure torsion theory, usually reducing the stress near corners and increasing it in mid-panel. Because maximum combined effects usually occur at the corners, it is conservative to ignore the warping shear stresses and use the simple uniform distribution. The longitudinal effects are, on the other hand greatest at the corners. They need to be taken into account when considering the occurrence of yield stresses in service and the stress range under fatigue loading. But since the longitudinal stresses do not actually participate in the carrying of the torsion, the occurrence of yield at the corners and the consequent relief of some or all of these warping stresses would not reduce the torsional resistance. In simple terms, a little plastic redistribution can be accepted at the ultimate limit state (ULS) and therefore there is no need to include torsional warping stresses in the ULS checks. If deformation of the cross section is prevented, and if twisting is prevented at the supports, then for straight boxes torsional deformations are often small enough to be neglected.

3.2.3 Distortion

When torsion is applied directly around the perimeter of a box section, by forces exactly equal to the shear flow in each of the sides of the box, there is no tendency for the cross section to change its shape. If torsion is not applied in this manner, there is effectively a set of forces which is trying to extend the length of one diagonal across the section and reduce the other (see Figure3. 4).

Diaphragms or frames can be provided to restrain distortion where large distortional forces occur, such as at support positions, and at intervals along a box, but in

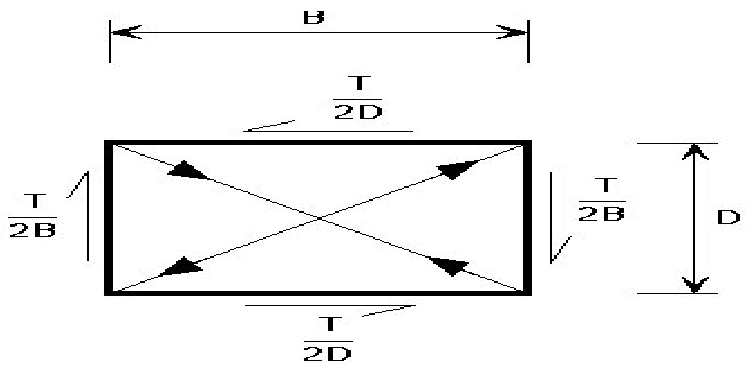


Figure 3.4: Force in Diagonal Members Due to Distortional Component of Applied Torque

general the distortional effects must be carried by other means. Torsion can be applied in this manner if, at the position where the force couple is applied, a diaphragm or stiff frame is provided to ensure that the section remains square and that torque is in fact fed into the box walls as a shear flow around the perimeter. Provision of such diaphragms or frames is practical, and indeed necessary, at supports and at positions where heavy point loads are introduced. But such restraint can only be provided at discrete positions. When the load is distributed along the beam, or when point loads can occur anywhere along the beam such as concentrated axle loads from vehicles, the distortional effects must be carried by other means. If the only resistance to transverse distortional bending is provided by out-of-plane bending of the flange plates there were no intermediate restraints to distortion, the distortional deflections in most situations would be significant and would affect the global behavior. For this reason it is usual to provide intermediate cross-frames or diaphragms; consideration of distortional displacements and stresses can then be limited to the lengths between cross-frames. To illustrate how distortion occurs and is carried between effective restraints, consider a simply supported box with diaphragms only at the supports and which is subject to a point load over one web at mid span. Under the distortional forces, each side of the box bends in its own plane and, provided there is moment continuity around the corners, out of its plane as well. The deflected shape is shown in Figure 3.5.

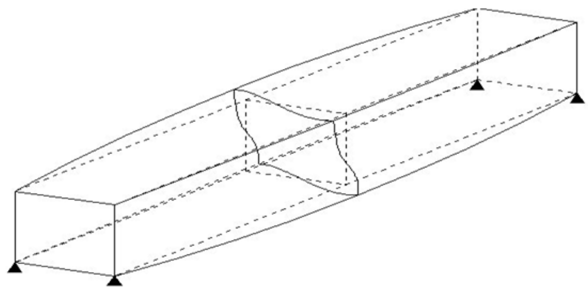


Figure 3.5: Distortional Displacements in Box Girder

The in-plane bending of each side gives rise to longitudinal stresses and strains which, because they are in the opposite sense in the opposing faces of the box, produce a warping of the cross section (in the example shown the end diaphragms warp out of their planes, whilst the central plane can be seen to be restrained against warping by symmetry). The longitudinal stresses are therefore known as distortional warping stresses. The associated shear stresses are known simply as distortional shear stresses. The bending of the walls of a box, as a result of the distortional forces, produces transverse distortional bending stresses in the box section. The introduction of stiff intermediate cross-frames will restrict distortional effects to the lengths between frames (rather than between supports). But they must be stiff enough for this purpose. In general the distortional behavior depends on interaction between the two sorts of behavior, the warping and the transverse distortional bending. The behavior has been demonstrated to be analogous to that of a beam on an elastic foundation (BEF), with the beam stiffness representing the warping resistance and the elastic foundation representing the transverse distortional bending resistance. A diagrammatic representation of the response is shown in figure 3.6. Warping stresses are represented by bending of the beam and distortional bending stresses by the displacement of the foundation.

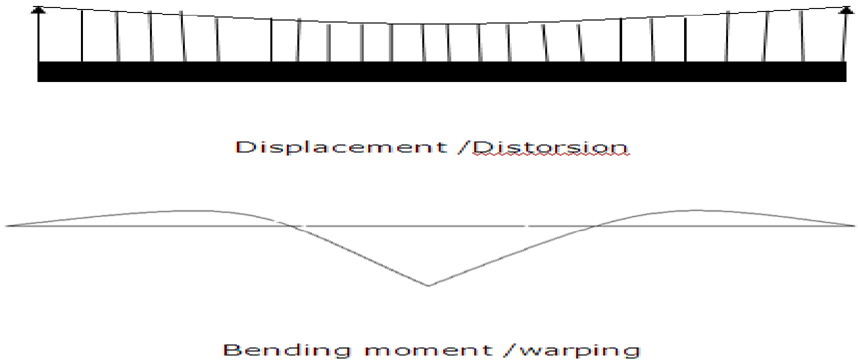


Figure 3.6: Beam on Elastic Foundation Analogy

The introduction of intermediate diaphragms in the box girder can be represented in BEF analogy by the addition of discrete vertical restraints, or springs.

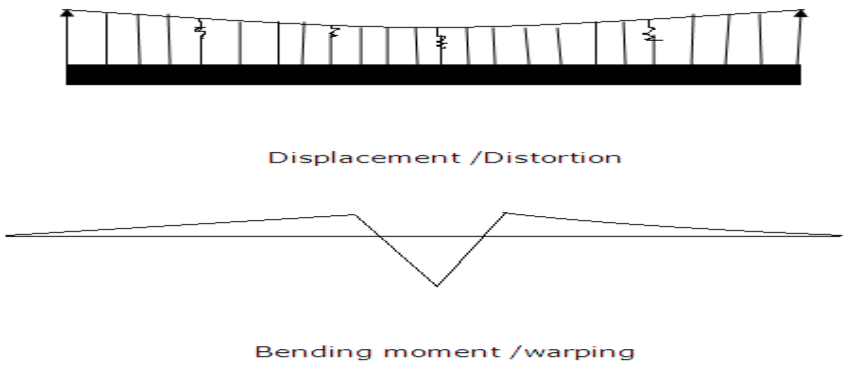


Figure 3.7: BEF Model With Intermediate Springs

3.3 Shear Lag

When the axial load is fed into a wide flange by shear from the webs the flange distorts in its plane; plane sections do not remain plane. The resulting stress distribution in the flange is not uniform in very wide flanges, shear lag effects have to be taken into account for the verification of stresses, especially for short spans, since it causes the longitudinal stress at a flange/web intersection to exceed the mean stress in the flange. Shear lag can be allowed for in the elementary theory of bending, by using an effective flange breadth (less than the real breadth) such that the stress in the effective breadth equals the peak stress in the actual flange (see Figure 3.8). This effective flange breadth depends on the ratio of width to span.

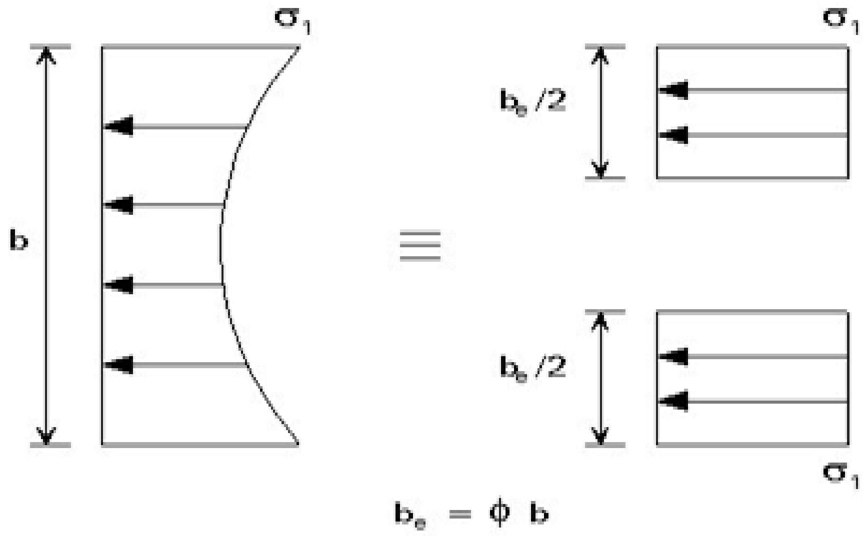


Figure 3.8: Effective widths for shear lag effect

Chapter 4

Design Philosophies

4.1 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 from IRC:6-2000

Dead Load

Live Load

Class 70R Loading:

This loading is to be adopted for bridge located within certain specified municipal localities, National highway, State highway and in certain existing or Contemplated industrial areas. The Fig.4.1shows the details of 70R loading.

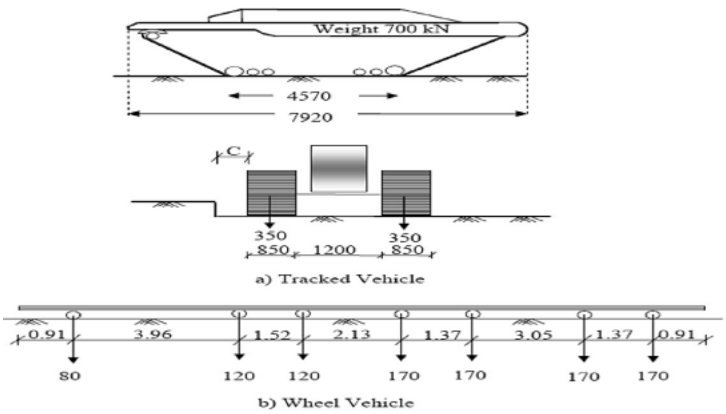


Figure 4.1: IRC Class 70R Loading

Carriageway Width m	*C Minimum m*
3.5 to 5.5	0.3
Over 5.5	1.2

Table 4.1: The Minimum Clearance Between Road Face Kerb and The Outer Edge of The Wheel

4.2 Limit State Method of Design

4.2.1 General:

Normal elastic method is valid for analysis of structure after considering load history, sequence of concrete casting and development of composite strength. In the case of propped construction, most of the initial dead load is resisted through girder-prop system and main girder remains basically unstressed at that stage. In case of unpropped construction the steel girder alone has to carry the initial dead load and consequently stresses. The necessary distinction has to be made in the analysis .In ultimate limit state; however, this distinction is not necessary while checking for flexural strength.

4.2.2 Limit States

Structural safety has to be assessed for each limit state as mentioned below:

Service Limit State

Is the state in which following conditions occur

a. Stress in structural steel has reached the prescribed limit.

b. Deflection reaches the prescribed limit.

c. Concrete crack width reaches the prescribed limit.

d. Slip at interface between steel and concrete becomes excessive.

e. Vibration becomes excessive specially at overhanging foot or cycle path.

Fatigue Limit State

It is the state at which stress range due to application of live load, reach prescribed limit, prescribed limit corresponding to the number of load cycles and detail configuration.

Ultimate Limit State

It is the state when under the worst combination of factored loads the structure or its components reach design strength and collapse.

Design Loads

Table 4.2: Materail Safety Factor

Material	Partial safety factor(γm)	
	Limit state	Fatigue state
Structural steel against yield stress	1.10	1.0
Structural steel against ultimate stress	1.25	
Steel reinforcement	1.15	
Shear connector	1.25	
Bolts & rivets for shop and site fabrication	1.25	
Weld for shop fabrication	1.25	
Weld for site fabrication	1.50	
Concrete(γc)	1.50	1.0

Sectional Classification of Girder

The Sectional Strength at limit state should be considered on their ability to resist local buckling before full plastic strength is developed .in this respect the section may be classified as:

Class-1 or Plastic: Cross-section which can develop plastic hinge and have rotation capacity required for failure of structure by formation of a plastic mechanism.

Class-2 or Compact: Cross-section which can develop plastic moment of resistance but have inadequate plastic hinge rotation capacity for formation of a plastic mechanism due to local buckling.

Class-3 or Semi- Compact: Cross-section in which the extreme fibers in compression can reach yield stress, but cannot develop the plastic moment of resistance due to local buckling.

Class-4 or Slender: Cross-section in which the elements buckle locally, even before reaching yield stress. This code does not deal with this type of section.

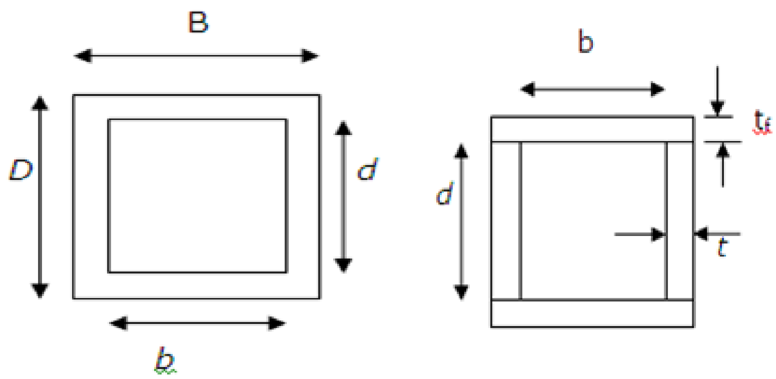


Figure 4.2: Rectangular Hollow Section and Built-Up Section

Table 4.3: Limiting Width to Thickness Ratio

Compression element			Ratio	*Class of section*		
				Class1 Plastic	Class2 compact	Class 3 Semi-compact
Outstanding element	Rolled section		b/ tf	9.4ε	10.5ε	15.7ε
Of compression flange	Welded section		b/ tf	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/ tf	29.3ε	33.5ε	42ε
	Axial compression		b/ tf	*Not applicable*		
Web of an I-H- or box section	Neutral axis at mid- depth		d/ tw	84ε	105ε	126ε
	Generally	If r1 is negative	d/ tw		$105.0\varepsilon/1+r1$	
				$84.0\varepsilon/1+r1$	$105.0\varepsilon/1+r1$	$126.0\varepsilon/1+r1$
		If r1 is positive	d/ tw			
				But=42 ε	*But=42 ε*	*But=42 ε*
	Axial compression		d/ tw	*Not applicable*		*42 ε*

Effective Width of Concrete Slab

For strength calculation of composite girder, Effective width b_{eff} of deck slab on either side of the girder to satisfy

$$b_{eff} = (\frac{L_0}{8}) \leq (\frac{B_1}{2}) or (\frac{B_2}{2}) \tag{4.1}$$

Therefore, total effective width b_{eff} of deck slab is restricted to the limit as indicated below:

a. For inner beams

$$b_{eff} = (\frac{L_0}{4}) \leq (\frac{B_1 + B_2}{2}) \tag{4.2}$$

For equal spacing of girder ie. B_1= B_2=B

$$b_{eff} = (\frac{L_0}{4}) \leq B \tag{4.3}$$

L=Actual span of girder

L_o=the effective span taken as the distance between point of zero moments (Lo=L for simply supported girders)

B= Equal center to center distance of transverse span of inner slab

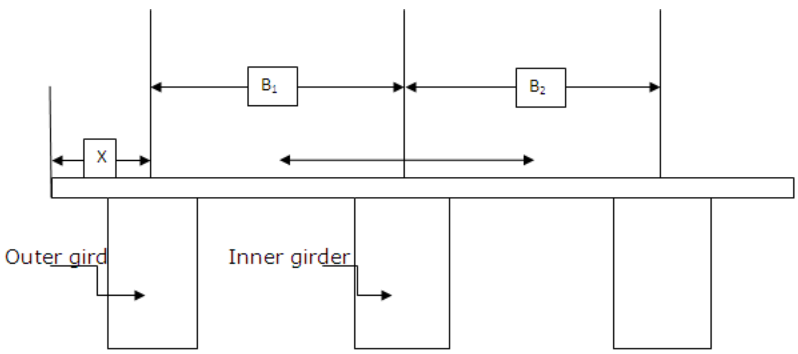


Figure 4.3: Limiting Width to Thickness Ratio

Table 4.4: Moment of Resistance of Composite Section With Plastic or Compact Structural Steel Section(Positive moment)

Case	Position of plastic neutral axis	Value of Xu	Moment capacity Mp
1	Within slab	Xu=aAs/beff	$\mathrm{Mp} = \mathrm{Asfy}\,\frac{(\mathrm{dc}+0.5\mathrm{ds}-0.42\mathrm{Xu})}{\gamma\mathrm{m}}$
2	Plastic neutral axis in steel flange $\mathrm{beffds} < \mathrm{aAs} < (\mathrm{beffds}+2\mathrm{aAf})$	$\mathrm{Xu} = \mathrm{ds} + \frac{(\mathrm{aAs}-\mathrm{beffds})}{2\mathrm{bfa}}$	$\mathrm{Mp} = \mathrm{fy}\,\frac{[As(\mathrm{dc}+0.08\mathrm{ds})-bf(xu-ds).(Xu+0.16ds+tf)]}{\gamma\mathrm{m}}$
3	Plastic neutral axis in web $\mathrm{beffds}+\mathrm{aAf} < \mathrm{aAs}$	$\mathrm{Xu} = \mathrm{ds} + \mathrm{tf} + \frac{\mathrm{a(As}-2\mathrm{A\ f)\ beffds}}{2\mathrm{bwa}}$	$\mathrm{Mp} = \mathrm{fy}\,\frac{\left[\begin{array}{c}\mathrm{As}\,(\mathrm{dc}+0.08\mathrm{ds}) - 2\mathrm{Af}\,(0.5\mathrm{t}+.58\mathrm{ds}) - \mathrm{tw}\,(\mathrm{Xu}-\mathrm{ds}-\mathrm{tf})\,.\\ (\mathrm{Xu}+0.16\mathrm{ds}+\mathrm{tf})\end{array}\right]}{\gamma\mathrm{m}}$

Effective Cross-section for Strength Calculation

In calculating the strength of the cross-section of the composite girders the following should be considered:

For positive moment: Concrete in effective width to be included but not the reinforcements.

For negative moments: Concrete to be neglected but longitudinal reinforcement within effective are to be included.

Design against vertical shear and its effect on plastic moment capacity

The factored shear force, V, in a beam due to external action shall satisfy: Where, Vd= design shear strength calculated as given below

$$v_d = \left(\frac{V_n}{\gamma_{m0}}\right) \tag{4.4}$$

γ_{m0}=Partial safety factor against shear failure

V_n =may be governed by plastic shear resistance or strength of web as governed by shear buckling as given below 1) Plastic shear resistance The nominal plastic shear resistance under pure shear is given by: $V_n = V_p$ Where,

$$v_p = \left(\frac{A_v f_{yw}}{\sqrt{3}}\right) \tag{4.5}$$

Where, A_v is the shear area, f_{yw} is the yield strength of the web and partial safety factor γmo =1.10

Shear area may be calculated as below:

For rectangular hollow section of uniform thickness

Loaded parallel to depth (d)

$$A_v = A_d = (\frac{Vd}{b+d}) \tag{4.6}$$

Loaded parallel to width (b)

$$A_v = A_b = (\frac{Vb}{b+d}) \tag{4.7}$$

Where, A actual area of cross-section

b Overall breath of tubular section, breath of I-flanges

d Clear depth of web between flanges

2) Shear buckling resistance

The nominal shear strength, V_n of the web with or without intermediate stiffeners as governed by buckling may be evaluated by:

a) Simple post-critical method: This method is based on shear buckling strength can be used for webs of I-girder, with or without intermediate transverse stiffeners, provided that the web has transverse stiffeners at the support. The nominal shear strength is give by: $V_n = V_{cr}$

Where,

V_{cr} =shear force corresponding to web buckling

$$v_{cr} = (A_v \tau_{bnby}) \tag{4.8}$$

Where = Shear stress corresponding to web buckling, determined as follows:

When

$$\tau_w \leq 0.8 \tag{4.9}$$

Then,

$$\tau_b = (\frac{f_{yw}}{\sqrt{3}}) \tag{4.10}$$

When

$$0.8 \leq \tau_w \leq 1.2 \tag{4.11}$$

Then,

$$\tau_b = (1 - 0.8(\tau_w - 0.8))(\frac{f_{yw}}{\sqrt{3}}) \tag{4.12}$$

When

$$\tau_w \geq 1.2 \tag{4.13}$$

Then

$$\tau_b = (\frac{f_{yw}}{\sqrt{3}\tau_w})^2 \tag{4.14}$$

Where,

τ_w=non -dimensional web slenderness ratio for shear buckling stress

$$\tau_w = \sqrt{(\frac{f_{yw}}{\sqrt{3}\tau_{cr,e}})} \tag{4.15}$$

The elastic critical shear stress of the web,$\tau_{cr,e}$ is give by:

$$\tau_w = \frac{k_v \pi^2 E}{(12(1-\mu^2))(\frac{d}{t_w})^2} \tag{4.16}$$

Where, μ=Poission's ratio K_V= 5.35 When transverse stiffeners are provided at supports

$$K_v = 4 + \frac{5.35}{(\frac{c}{d})^2} \tag{4.17}$$

$$K_v = 5.35 + \frac{4}{(\frac{c}{d})^2} \tag{4.18}$$

Where, c,d are the spacing of transverse stiffeners and depth of web ,respectively.

Design for serviceability limit

Stresses and deflection

For calculating stresses and deflection, the value of modular ratio, m shall be taken as, for short term effect or loading

$$m = \frac{E_s}{E_c} \geq 7.5 \tag{4.19}$$

for permanent or long term effect or loading (K_c =creep factor=0.5)

$$m = \frac{E_s}{E_c k_c} \geq 15 \tag{4.20}$$

Where, E_s =Modulus of elasticity of steel=2x105 in N/mm^2 E_c = Modulus of elasticity of cast-in-situ concrete at 28 days

$$E_c = (5000\sqrt{f_{ck}}) \tag{4.21}$$

F_{ck}= Characteristic cube compressive strength of concrete in N/mm2

Limiting Stresses of Serviceability

The total elastic stress considering the different stage of construction in the steel beam should not exceed 0.87fy and the bending in concrete should not exceed one-third of its characteristic strength.

Limit for Deflection

Calculated deflection of composite girder under live load and impact shall not exceed 1/800 of span of the girder.

In any case under worst combination of D.L, super -imposed dead load, live load and impact effects, the total deflection of the girder shall not exceed 1/600 of span.

Control of Cracking of Concrete

Adequate reinforcement in terms of diameter and spacing as per IRC: 21 are to be provided in composite girders, at the zone of negative moment, to prevent cracking adversely affecting appearance and durability of structure. Crack width calculation as well as limiting crack width is given in IRC: 21 may be followed to discretion of engineers.

Fatigue

Fatigue is to be checked under live load with impact the appropriate load factor .Stress are to be assessed by elastic theory and elastic properties of the section with no adjustment for support moment.

4.3 Shear Connector

4.3.1 Shear connector

Spacing and design of shear connectors Ultimate limit strength (Strength criteria) Calculate shear$V_L$at interface corresponding to vertical shear is as given below,

$$V_L = \Sigma[\frac{V \times A_{ce} \times Y}{I}]_{dl,ll} \tag{4.22}$$

Where, V_L=Longitudinal shear per unit length
V =The vertical shear force due to dead load and live load (including impact) separately at each state of load history
A_{ec} =The transformed compressive area of concrete above the neutral axis of the composite section with appropriate modular ratio depending nature of load(whether short term i.e. live load, or long term i.e. dead load
Y=C.G Distance of transformed concrete area from neutral axis
I=Moment of inertia of the whole composite section using appropriate modular ratio

Dl,ll=Different load history, i.e. sustained load or composite action dead load, transient load or composite action live load. These load are to be considers with appropriate load factor.
Spacing of shear connector is given as :

$$S_{L1} = \frac{\Sigma Q_u}{V_L} \tag{4.23}$$

Q_u is the ultimate static strength of one shear connector which is to be taken from table and the summation is over the number of shear studs at one section.

Type of shear connector		Connector material	ultimate static strength in KN per connector for concrete strength			
Stud connector			25	30	40	50
Nominal Diameter(mm)	Overall height(mm)					
25	100	Material with a characteristic yield strength of 385MPa minimum elongation of 18% and a characteristic strength of 495 Mpa	103	118	146	154
22	100		79	91	113	119
20	100		66	75	93	99
20	75		62	71	89	99
16	75		42	48	60	63
12	**65**		**24**	**27**	**34**	**35**

Channels:150 mm long (min)	As per IS 2062				
ISMC 125		195	219	243	268
ISMC 100		184	204	228	250
ISMC 75		170	193	218	238

Table 4.5: Ultimate Static Strength of Shear ConnectorsQ_u for Different Concrete Strength)

Table 4.6: Nominal Fatigue StrengthQ_r(In Kn)

Type of Connector	Connector Material	N=No. of columns		
		$2x10^6$	5x10^5	1x10^5
Headed Studs Φ25	F_y=385	27	37	45
Headed Studs ϕ22	F_u=495	21	29	34.5
Headed Studs ϕ20	Elongation=18%	17	23	28
Headed Studs ϕ25		11	15	18
Channel 150mm long	IS:2062	55	70	93

4.3.2 Serviceability limit state (Limit state of fatigue)

Calculate longitudinal shear per unit length, Vr at interface due to live load and impact load is as given below.

$$V_r = \Sigma[\frac{V_r A_{ce} Y}{I}]_{ll} \tag{4.24}$$

Where, A_{ec},V_r,Y,I are as explained above

V_r = vertical shear difference due to maximum shear envelop due to live load and impact e

L_l=is live load with impact

Spacing of shear connector from fatigue is given as

$$S_R = \frac{\Sigma Q_r}{V_r} \tag{4.25}$$

Q_r is the nominal fatigue strength of one shear connector which is to be taken from Table 4.9e

Detailing of Shear Connector

Details as shown in following sketch are to be followed:

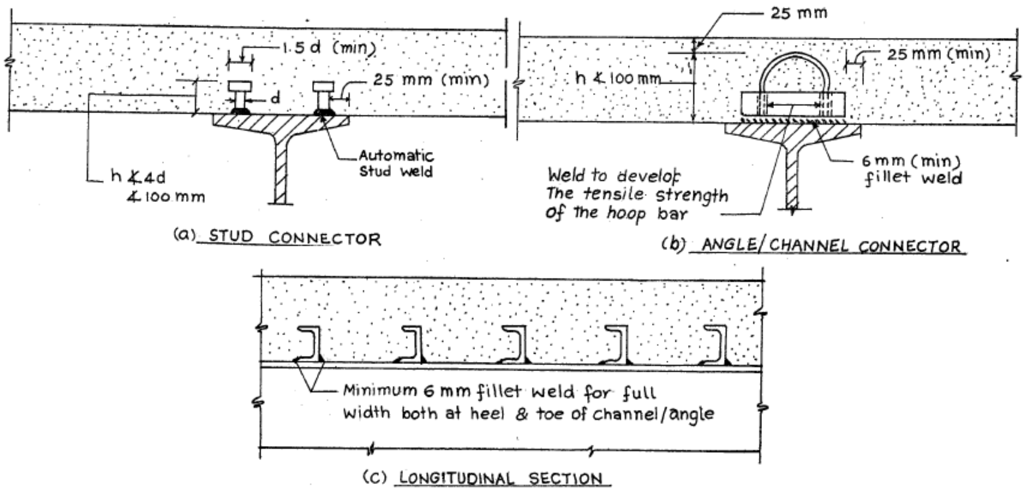


Figure 4.4: Details of Connector on Steel Girder

Cover to Shear Connector

The clear depth of concrete cover over the top of the shear connector shall not be less than 25mm.the horizontal clear concrete cover to any shear connector shall not be less than 50mm as shown in figure 4.7.

Limiting criteria for spacing of shear connector

1) Where a steel compression flange that would otherwise be in a lower class is assumed to be in class1 or class2 because of restraint provided by shear connectors, the centre -to-centre spacing of the shear connectors in the direction of the compression should satisfy the following
a)Where the slab is in contact over full length(e.g. Solid slab)

$$S_L \leq 21 t_f \sqrt{\frac{250}{f_y}} \tag{4.26}$$

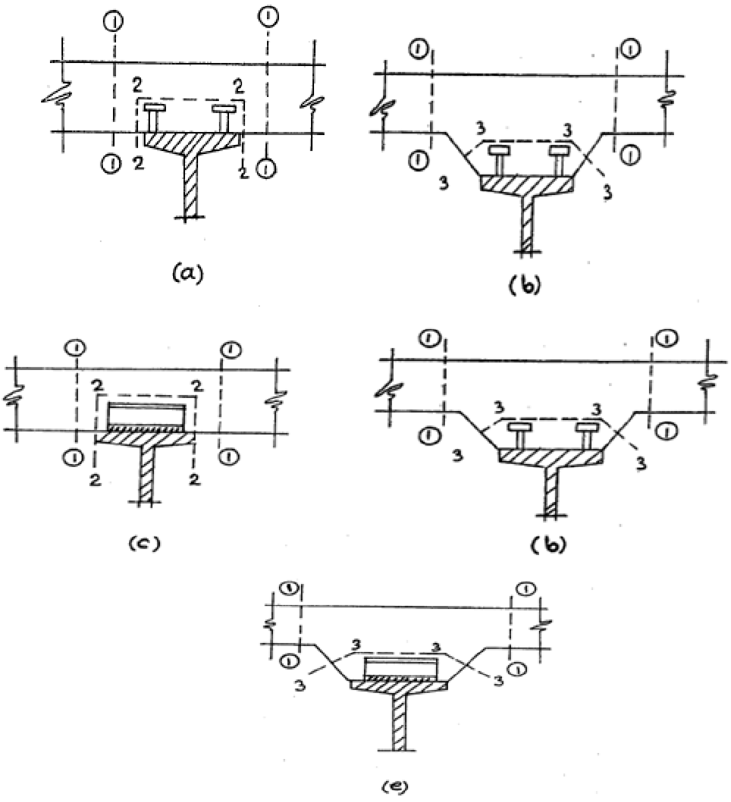


Figure 4.5: Typical Shear Planes

b) Where the slab is not in contact over full length (e.g. Slab with ribs transverse to the beam)

$$S_L \leq 14t_f\sqrt{\frac{250}{f_y}} \tag{4.27}$$

Where , τ_f is the thickness of the flange
F_y is the yield strength of the flange in N/mm2
S_L is the maximum spacing of shear connector
In addition, the clear distance from the edge of the compression flange to the nearest line of shear -connectors should not be greater than

$$9t_f\sqrt{\frac{250}{f_y}} \tag{4.28}$$

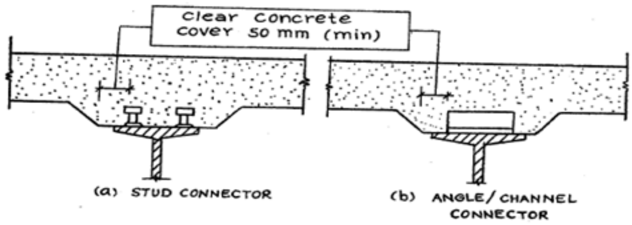


Figure 4.6: Cover to Shear Connector

or 50mm whichever is less.

2) In all cases, shear connector shall be provided throughout the length of beam may be uniformly spaced between shall be provided between critical cross-sections. The maximum spacing of shear connectors in the longitudinal direction shall be limited to 600 mm or three times the thickness of the concrete slab or four times the height of the connector (including any hoop which is an integral part of the connector) whichever is least.

3) Minimum spacing should be such, as to allow proper concrete flow and compaction around the connectors and for stud connectors it should not be less than 75mm.

Transverse shear check

Shear connector transfer longitudinal shear from steel girder to slab concrete abutting them, where from the same is transferred to the rest of slab through transverse shear strength of slab as well as transverse reinforcements provided .the strength and amount of reinforcement is to be checked for following relations. The shear force transferred per meter length VL shall satisfy both the following conditions:

$$V_L \leq 0.632L\sqrt{f_{ck}} \tag{4.29}$$

or

$$V_L \leq 0.232L\sqrt{f_{ck}} + 0.1A_{st}f_{st}.n \tag{4.30}$$

Where, V_L=Longitudinal shear force per unit length calculated for ultimate limit state
f_{ck}=Characteristic strength of concrete in MPa
f_{st}=Yield stress of transverse reinforcement in MPa
L =Length (mm) of possible shear planes envelop as indicated in fig.
N =Number of times each lower transverse reinforcing bar is intersected by a Shear surface (i.e. the number of rows of shear connector at the section of the beam).generally for T-beam n=2 and for L-beam n=1 A_{st}=sectional area (in cm^2) of transverse reinforcement per meter run of beam
The amount of transverse steel in the bottom of the slab shall not be less than cm^2/m

$$\frac{2.5V_L}{f_{st}} \tag{4.31}$$

Where, V_L is in KN/m.

4.4 Transverse Reinforcement

Planes which are critical for longitudinal shear failure, in the process of transfer of longitudinal shear from the girder to slab, are of four main types, as shown in fig 6.21.If the concrete by itself is insufficient to take the longitudinal shear, sufficient transverse reinforcement shall be provided to transfer longitudinal shear Force from the girder to the effective width of slab. The area of transverse reinforcement per unit length of beam will be the sum total of all the reinforcement (A_t, A_h or A_b as shown in figs.4.8) ,which are intersected by the shear plane and are fully anchored on both sides of the shear plane considered.

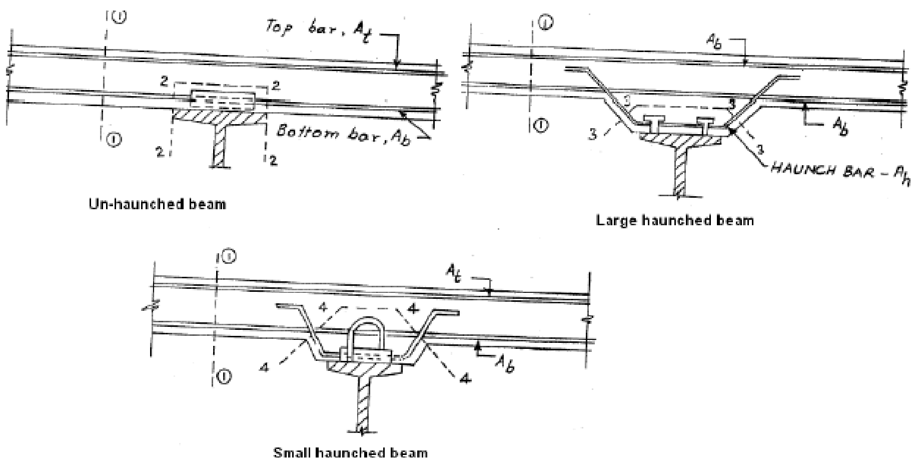


Figure 4.7: Transverse Reinforcement Across Shear Connector

4.4.1 Total Shear Reinforcement

The total transverse reinforcements, As, per unit length of beam in case of shear plane 1-1 which crosses the whole thickness of slab will be sum of $(A_t + Ab)$.Area of reinforcement At and Ab include those provided for flexure. The total reinforcements across plane 2-2 is A_s=2 Ab and that across plane 3-3 is As=2 Ah as these planes do not cross the full thickness of the slab .In case of plane 4-4, the total transverse reinforcement is $A = (A_b + A_h)$.

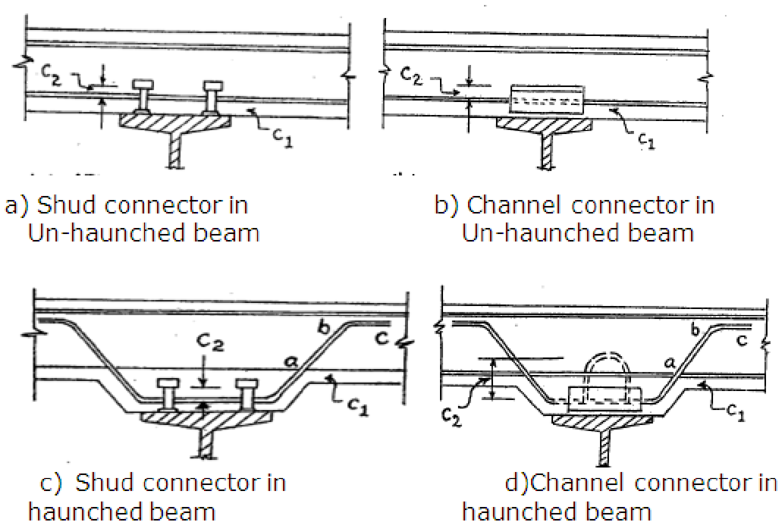


Figure 4.8: Arrangement of Transverse Reinforcement

Chapter 5

Analysis of Composite Box-Girder Bridge

5.1 General

In present study composite box-girder bridge is taken for cost effective economy comparisons with different L/d ratio. This chapter covers the analysis of composite highway bridge with SAP software .

5.1.1 Structural Data

Data for simply supported box Girder Bridge

C/C Bearing	20 m
Over All Length of Girder	20 m
No. Of longitudinal Girder	4
No. Of Cross Girder	6
C/C Of Cross Girder	4m
Over All Width of Deck	8.5m
Clear Carriage Way Width	7.5 m
Curb Width	0.5 m

C/C Spacing Of Girder	2m
Slab Thickness	0.20 m
Wearing Coat	0.085m
Loading	Class 70R

In this study, cross section taken for analysis is as shown in Fig.5.2

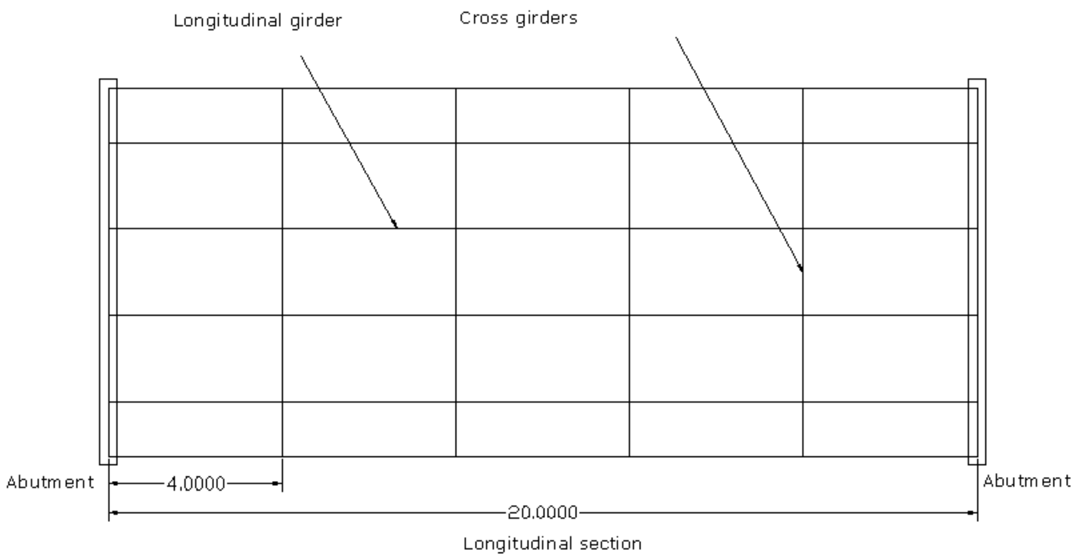


Figure 5.1: Longitudinal Section of Simply Supported Bridge

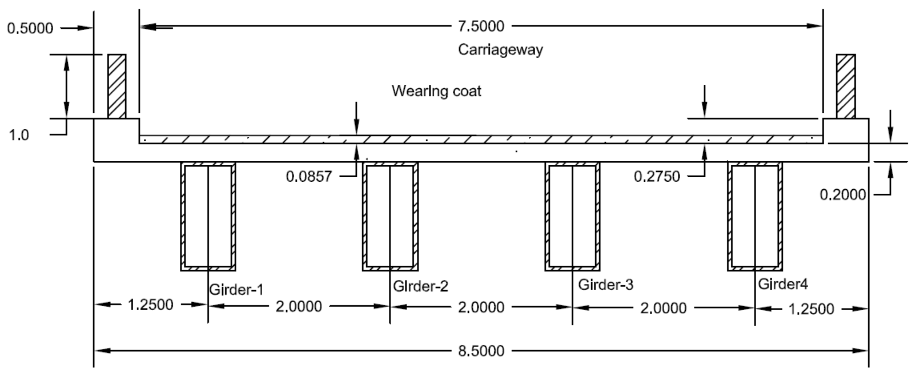


Figure 5.2: Cross Section of composite Box-girder Without Shear Connectors

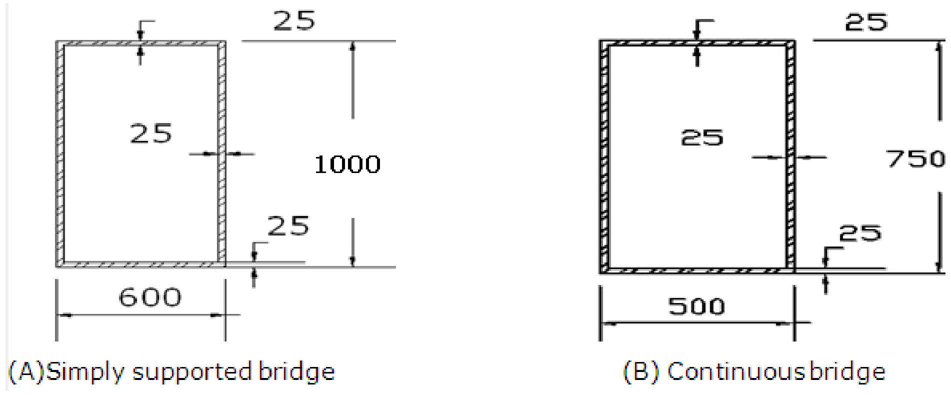


Figure 5.3: Assumed Box Cross Section for Design

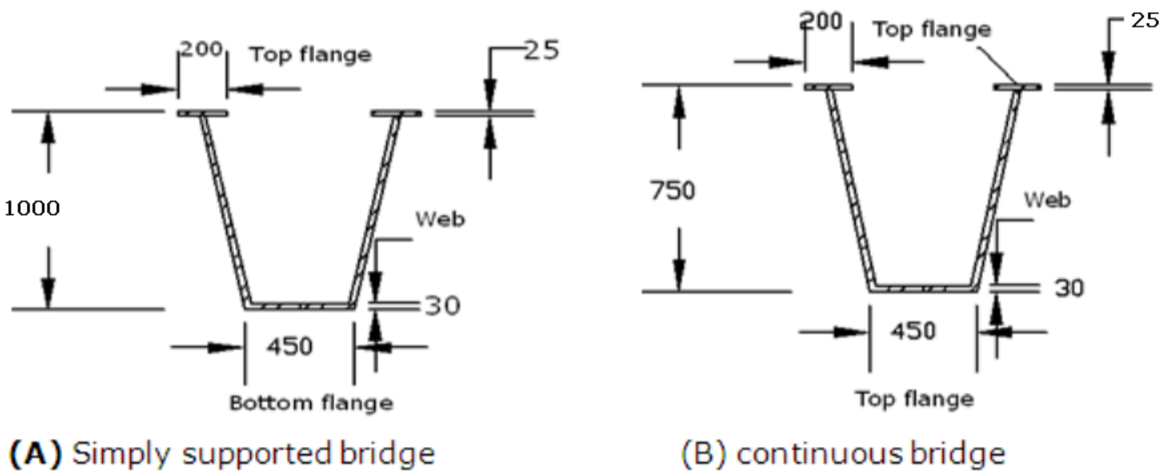


Figure 5.4: Assumed Trapezoidal Section for Design

RCC Grade = M30

Grade of reinforcement = Fe 415

Grade of structural steel = Fe 250

Unit weight of RCC = 25 kN/m^3

Unit weight of wearing coat = 22 kN/m^3

Unit weight of structural steel = 77 kN/m^3

The analysis are carried out as per following data,

Table 5.1: Analysis Results of Different Spans

SPANS (m)	*c/c dist. between cross girder(m)*	*No. of cross girder*	*c/c distance between longitudinal girder*	*No. of longitudinal girder*	*Depth taken (m)*	*Span to depth ratio*
15	3.75	5	2	4	1200,1100 1060	13,13.64, 14.16
20	4	6	2	4	1700,1200 1150	11.77,16.67, 17.4
25	4.16	7	2	4	2000,1700, 1400	12.5,14.71, 17.86

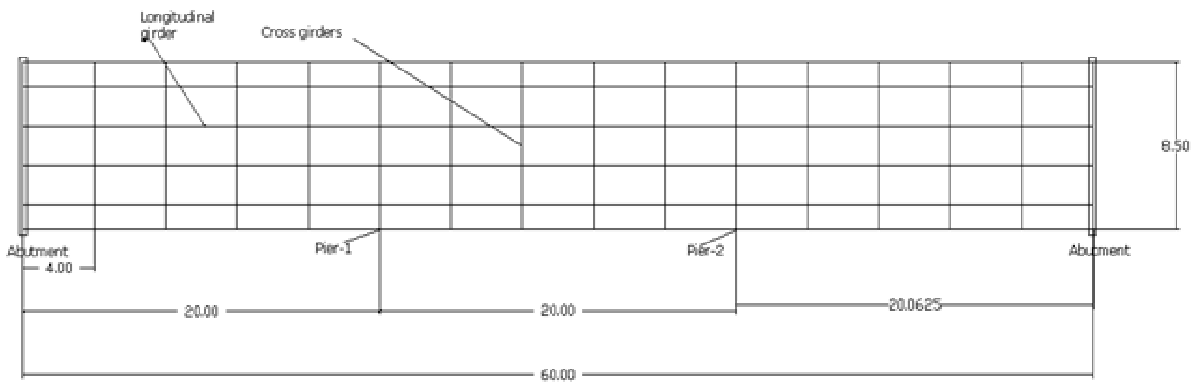


Figure 5.5: Longitudinal Section for Continuous Bridge

Table 5.2: Data for Analysis for Different Sections Bridge

	Spans (m)	*c/c Distance between Cross Girder(m)*	*No. of Cross Girder*	*c/c Distance between Longitudinal girder*	*No. of Longitudinal girder*	*Depth Taken (m)*
Simply supported box girder	20	4.0	6	2	4	1000
Continuous box girder ridge	60	4	16	2	4	750
Simply supported Trapezoidal Girder bridge	20	4.0	6	2	4	1000
Continuous Trapezoidal Girder bridge	60	4	16	2	4	750

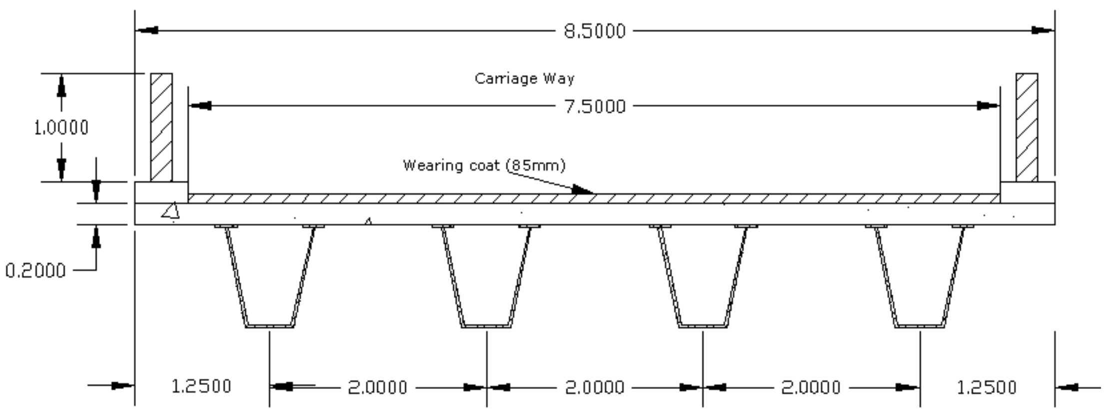


Figure 5.6: Cross Section for Trapezoidal Bridge

5.2 Modeling of Box-Girder Bridge in SAP2000

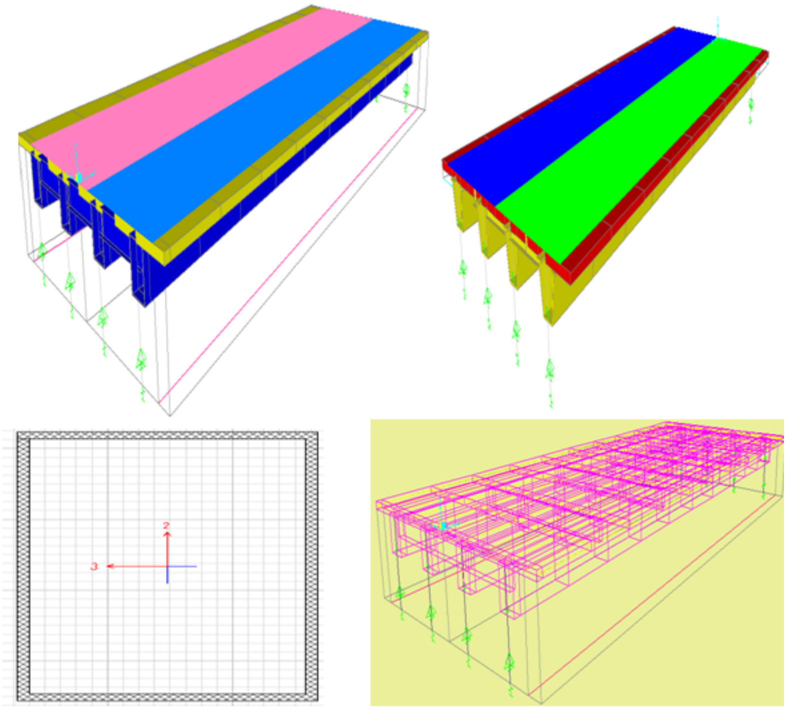


Figure 5.7: Model of Composite Box-Girder in Sap2000

5.3 Analysis of Box Girder Bridge

The analysis is done for Deal Load, Super imposed dead Load, vehicle load Class A and Class 70R IRC vehicle cases in Sap2000. From the software we have taken maximum bending moment and shear force and torsion. For the design force consideration we have taken carriageway combinations as 1.35DL + Impact +1.5 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 70 R or 2 lane of Class A vehicle on carriageway

5.3.1 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 = 20 m
- No of longitudinal girder = 4
- C/c spacing of longitudinal girder = 2.0 m
- Cross girder = 6
- Cross girder spacing of girder = 4 m.

As the ratio of longer dimension to shorter dimension is 4/2 = 2.0, therefore the slab is considered as two way slab.

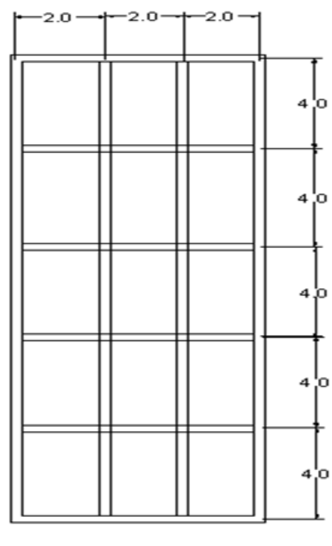


Figure 5.8: Slab Panel

The deck slab is analysed for D.L and L.L. The dead load consists of self weight, super imposed dead Load, and vehicle load as Class A and Class 70R 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 A and Class 70R IRC vehicle cases are taken. Deck slab is further divided in slab panels and cantilever slab.

Analysis Deck Slab

The deck slab panel is designed as two way slab using Pigeaud's curves. The bending moment are computed as equation 5.1. M1 = (m1 +m2) W

$$M_1 = (m_1 + \mu m_2)w \tag{5.1}$$

$$M_2 = (m_2 + \mu m_1)w \tag{5.2}$$

Where, K = Ratio of short to long span (B/L)
M_1 = Moment in the short span direction
M_2 = 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.

Analysis and Design of Cantilever Slab

The cantilever deck slab is analysed 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.

5.3.2 Analysis of Longitudinal Girders

The girder is designed for flexure and shear. Steel girder is designed for different stages of loading starting from construction of girder to open for use. In design procedure the girder stresses are to be checked for different stages, because concrete is gaining up to 70 percentage strength of its ultimate strength it will act as non-composite member, like stresses due to only self weight of girder, forces due to self weight of concrete and wet weight of deck slab including shuttering (ie.stage1) and forces due to whole section with SIDL and vehicle loading (ie. Stage 2). The stage wise design is required to check the section for construction sequence, after that the composite section is checked for ultimate strength. For analysis and design of girder, codal provision of IRC 6, IRC 21, IS800-2007 and IRC 22-2008 are used. Analysis is done with Sap2000 software and design is done Using spreadsheet. The typical sample calculations of longitudinal girder for 20m box section and for tabulation are done for other section taken. Analysis is carried out for following sections:

a. Simply supported box girder

b. Simply supported trapezoidal girder

c. Continuous box girder

d. Continuous trapezoidal girder

Courbon's Method

According to Courbons's method, the reaction Ri of the cross beam on any girder i of a typical bridge consisting of multiple parallel beams is computed assuming a linear variation of deflection in the transverse direction. The deflection will be maximum on the exterior girder on the side of eccentric load (or c.g. of loads if there is a system of concentrated loads) and minimum of the other exterior girder. The reaction Ri is then given by,

$$R_i = \frac{PI_i}{\sum I_i} + \left\{ \frac{PI_i}{\sum I_i} \cdot \frac{e\, d_i \,.\sum I_i}{\sum I_i \,.\, d_i^2} \right\}$$

$$R_i = \frac{PI_i}{\sum I_i}\left\{1 + \frac{\sum I_i}{\sum I_i \, . \, d_i^2} \, . \, ed_i\right\} \tag{5.3}$$

where,

P = total live load

Ii = moment of inertia of longitudinal girder i

e = eccentricity of the live load (or c.g. of loads in case of multiple loads)

di = distance of girder i from the axis of the bridge. When the intermediate and the end longitudinal girders have the same moment of inertia, the quantity Ii in the second term within brackets of equation (3.1) gets cancelled and the term outside the bracket now reduces to P/n, where n is the number of longitudinal girders. This reduces the amount of computation considerably. In view of the simplicity in calculation, this method is very popular.

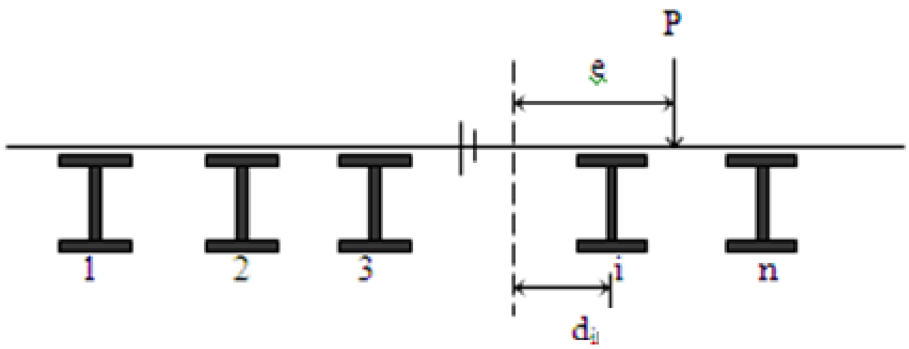


Figure 5.9: Slab Panel

Data for composite longitudinal girder:

- Span = 20 m
- Slab thickness = 200 mm
- Web dimension = 1150x 25 mm
- Top flange dimension = 600 x 25 mm
- Bottom flange dimension = 600 x 25 mm
- Modular ratio for SIDL = 14.61, for LL =7.3

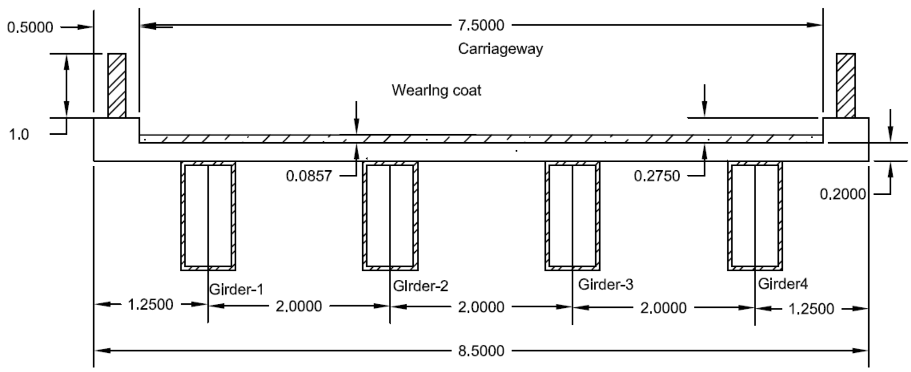


Figure 5.10: Cross section of Composite Box-Girder Without Shear Connectors

5.3.3 Analysis Results

SAP output results are shown as below. The results of shear force in entire section for 1.35DL+IL+1.5LL case are tabulated in Table below. Here the results are shown for maximum force at that location for particular position of vehicle. For 20m simply supported box girder bridge(l/d=16.67)

Table 5.3: Moment for Simply Supported Box Girder Bridge

	G1		*G2*		*G3*		*G4*	
	L/4	L/2	L/4	L/2	L/4	L/2	L/4	L/2
Dead load(kN.m)	642.57	866.95	608.80	823.49	608.8	823.49	642.57	866.95
SIDl(kN.m)	246.44	281.26	182.78	264.94	182.78	264.94	246.44	281.26
Live load(kN.m)	2480.95	3185.45	2007.10	2578.30	2052.18	2643.21	2776.17	3521.73

Table 5.4: Shear Force for Simply Supported Box Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN)	183.75	187.80	187.80	183.75
SIDl(kN)	42.97	37.73	37.73	42.97
Live load(kN)	1026.84	929.76	936.20	1254.22

Table 5.5: Torsion for Simply Supported Box Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN.m)	6.4	5.19	5.19	6.4
SIDl(kN.m)	4.43	2.05	2.05	4.43
Live load(kN.m)	234	252	274.9	372.1

Table 5.6: Moment for Continuous Box Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN.m)	582.8	649.4	649.4	582.8
SIDl(kN.m)	253	266.1	266.1	253
Live load(kN.m)	2510	1959	2023	2783

Table 5.7: Shear Force for Continuous Box Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN)	169.6	200.5	200.5	169.6
SIDl(kN)	75.38	80.61	80.61	75.38
Live load(kN)	827.2	874.6	888.9	961.2

Table 5.8: Torsion for Continuous Box Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN.m)	26.8	15.36	15.36	26.8
SIDl(kN.m)	11.55	6.35	6.35	11.55
Live load(kN.m)	465.4	464.7	451.5	614.7

Table 5.9: Moment for Simply Supported Trapezoidal Girder Bridge

	G1		*G2*		*G3*		*G4*	
	L/4	L/2	L/4	L/2	L/4	L/2	L/4	L/2
Dead load(kN.m)	935.4	1260	732.5	992	732.5	992	935.4	1260
SIDl(kN.m)	313.5	420.6	217.5	296.8	217.5	296.8	313.5	420.6
Live load(kN.m)	3661	4798	2684	3445	2684	3445	3661	4798

Table 5.10: Shear Force for Simply Supported Trapezoidal Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN.m)	255.2	195	195	255.2
SIDl(kN.m)	87.56	56.44	56.44	87.56
Live load(kN.m)	1117	871.5	871.5	1117

Table 5.11: Torsion for Simply Supported Trapezoidal Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN.m)	3.62	3.35	3.35	3.62
SIDl(kN.m)	2.12	2	2	2.12
Live load(kN.m)	266.1	177.5	177.5	266.1

Table 5.12: Moment for Continuous Trapezoidal Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN.m)	731.8	727.6	754.2	771.2
SIDl(kN.m)	244.3	226.9	233	254.1
Live load(kN.m)	3478	2059	2010	3226

Table 5.13: Shear Force for Continuous Trapezoidal Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN)	218.4	222.9	242.6	217.1
SIDl(kN)	73.69	68.49	73.32	73.4
Live load(kN)	1052	928	868.9	1009

Table 5.14: Torsion for Continuous Trapezoidal Girder Bridge

	G1	*G2*	*G3*	*G4*
Dead load(kN.m)	27.69	17.12	17.12	27.69
SIDl(kN.m)	7.57	4.53	4.53	7.57
Live load(kN.m)	400.8	277.3	258.4	307.5

SAP results are tabulated shear force and bending moment among all girders for all L/D ratios are recapitulated in Table5.15

Table 5.15: Moment for Simply Supported Box Girder Bridge

span (m)	*Dead Load Moment* (kNm)	*SIDL Load Moment* (kNm)	*Live Load Moment* (kN.m)	*Total B.M* (kN.m)	*Dead Load Reaction* (kN)	*SIDL Reaction* (kN)	*Live Load Reaction* (kN)	*Total S.F* (kN)
15	471.54	141	2514	3126.54	135.23	27	1164	1326.23
	445.05	141	2522	3108.05	127.61	27	1166	1320.61
	455.41	142	2605	3202.41	130.07	27	1212	1369.07
20	856.67	280.96	3469.15	4606.78	181.64	43	1254	1478.64
	866.95	281.26	3521.74	4669.95	183.75	43	1254	1480.75
	877.27	284.84	4012.45	5174.56	184.57	43	1251	1478.57
25	1412.98	462.89	5437.03	7312.9	238.24	59	1326	1623.24
	1360.61	464.91	4992.6	6818.12	228.15	59	1334	1621.15
	1492.18	469.92	5300.25	7262.35	249.45	60	1336	1645.45

Chapter 6

Design of Composite Box-Girder Bridge

6.1 Design of Deck Slab

Design Data:

Type of superstructure	=	20 m
c/c of longitudinal main girder	=	2 m
c/c of longitudinal cross girder	=	4 m
Cantilever width,L left	=	1.25 m
L right	=	1.25 m
Grade of steel	=	Fe 415
Grade of concrete	=	M 25
Clear cover	=	30 mm
Clear carriageway width	=	7.5m
Overall width of deck slab	=	8.5m
Width of slab	=	200 mm
Width of wearing coat	=	85 mm

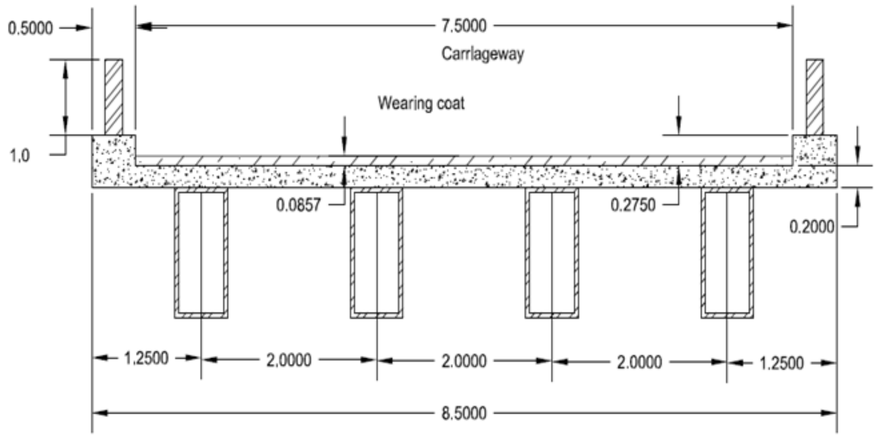


Figure 6.1: Bridge Cross-Section

R.C.C Grade	=	M30
Grade of reinforcement	=	Fe 415
Grade of structural steel	=	Fe 250
Unit weight of RCC	=	25 kN/m^3
Unit weight of wearing coat	=	22 kN/m^3
Unit weight of structural steel	=	77 kN/m^3

6.1.1 Analysis Results

Table 6.1: Analysis Result for Cantilever Panels

Span	Dead load Bending Moment (kNm)	Dead load Shear Force (kN)	Live load Bending Moment (kNm)	Live load Shear Force (kN)
15	8.06	10.67	15.49	152.4
20	8.06	10.67	15.49	152.4
25	8.06	10.67	15.49	152.4

Table 6.2: Analysis Result for Interior Panels

	Dead Load Moment		Live Load Moment		Short span Design	Long span Design
Span	Short span (kNm)	Long span (kNm)	Short span (kNm)	Long span (kNm)	Moment (kNm)	Moment (kNm)
15	3.29	1.159	49.24	15.21	52.54	16.37
20	3.29	1.159	49.24	15.21	52.54	16.37
25	3.29	1.159	49.24	15.21	52.54	16.37

Impact Factor

Impact factor for class A= 1.62

Impact factor for Class 70R-wheeled= 1.25

Continuous span Impact factor for class 70R,

span in x-direction = 1.25

span in y-direction= 1.25

Impact factor for class A

span in x-direction=1.5

span in y-direction= 1.6

6.1.2 Design of cantilever slab

Table 6.3: S.F and B.M in Cantilever Slab

Components	D.L m2	C.G m	Moment KN.m
Slab	5.7	0.475	2.708
W.c	0.842	0.45	0.379
Parapet	4.125	0.7	2.888
Total	10.67		8.064

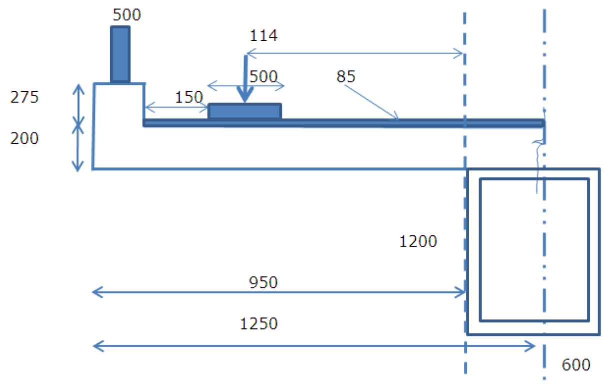


Figure 6.2: Cantilever Section of Deck

Effective width,b_{eff} = S1.2 X a + b_1

b_{eff} across the span	=	0.48m
b_{eff} along the span	=	1.07m
Load intensity	=	57×1.62/0.48
Load intensity	=	193 kN/m
B.M at the face of support	=	15.48kN.m
Distribution moment	=	6.26 kN.m
Design B.M	=	23.55

Slab Design

For main steel:

d_{reqd}	=	67.0 mm
$d_{available}$	=	165 mm OK
$Ast_{required}$	=	679.97mm^2
10mm @ 250 c/c (top)	=	314.15mm^2
10mm @ 200 c/c	=	392.69mm^2
Minimum steel required	=	300 mm^2

Prov. Half steel at top and half at bottom

Prov.10mm dia @ 300 mm = 261.67 mm^2at top and bottom

Check for Shear

Dead Load shear	=	14.40 kN
Shear force with impact	=	152.37kN
Total S.FV_u	=	181.17 kN
τv	=	1.098 N/ mm^2
100As/bd	=	0.16
τc	=	0.290 N/mm^2
K	=	1.2
V_{uc}	=	57.42kN < V_u=181.17 KN
τ_v	=	1.10 N/mm^2
τ_{cmax}	=	3.5 N/mm^2
	=	1.75 N/mm^2> τ_v

Hence no need to provide shear reinforcement

6.1.3 Design of Interior Slab Panal

Dead Load Analysis

Selfweight	=	6 KN/ mm^2
Weight of wearing coat	=	1.87KN/ mm^2
Total	=	7.87 KN/ mm^2
Total W	=	7.87 $\times$2 $\times$ 4kN
Total W	=	62.96kN
B	=	2
L	=	4
K=B/L	=	0.5
1/K	=	2

As slab panal is loaded equally by UDL so,

U/ B	=	1
V/L	=	1
m_1	=	0.047
m_2	=	0.01

$M_B = (m_1 + \mu m_2)w$

$M_L = (m_2 + \mu m_1)w$

Dead load bending moment

Along Shorter span ,M_B =3.055 kNm

Along longer span,M_L = 1.07kN.m

Live Load Analysis

W=350kN

U=1.01

V=0.57

B = 2

L=4

B/L=0.5

U/B=0.505

V/L =1.0

Now,from pigeaud's curve m_1 =0.075 and and m_2=0.013

Bending moment including impact

M_B=49.238 KN.m

M_L =15.21 KN.m

Design Moment=D.LBM +L.LBM

M_B = 52.54KN.m

M_L =16.37 KN.m

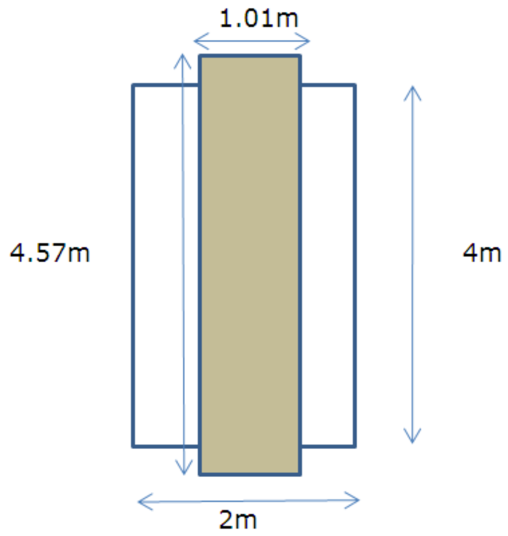


Figure 6.3: Slab Panal

Slab Design

For main steel :

d_{reqd}	=	123.40 mm
$d_{available}$	=	162mm OK
Ast (short)	=	$980.10mm^2$
Provide 16 mm @ 200 c/c	=	$1005mm^2$

For distribution steel:

d_{reqd}	=	111mm
Ast(long)	=	$430.68mm^2$
Prov. 10 mm dia @ 150mm	=	$523.60mm^2$

Check for Shear

Dead Load shear	=	7.4 KN
live load shear force with impact	=	39.40 KN
Total S.F	=	46.84 KN
τv	=	0.42 N/mm
τc	=	0.41 (From table 19 IS 456)
Permissible Shear Strength	=	0.492 N/mm^2 Ok

Check for Deflection

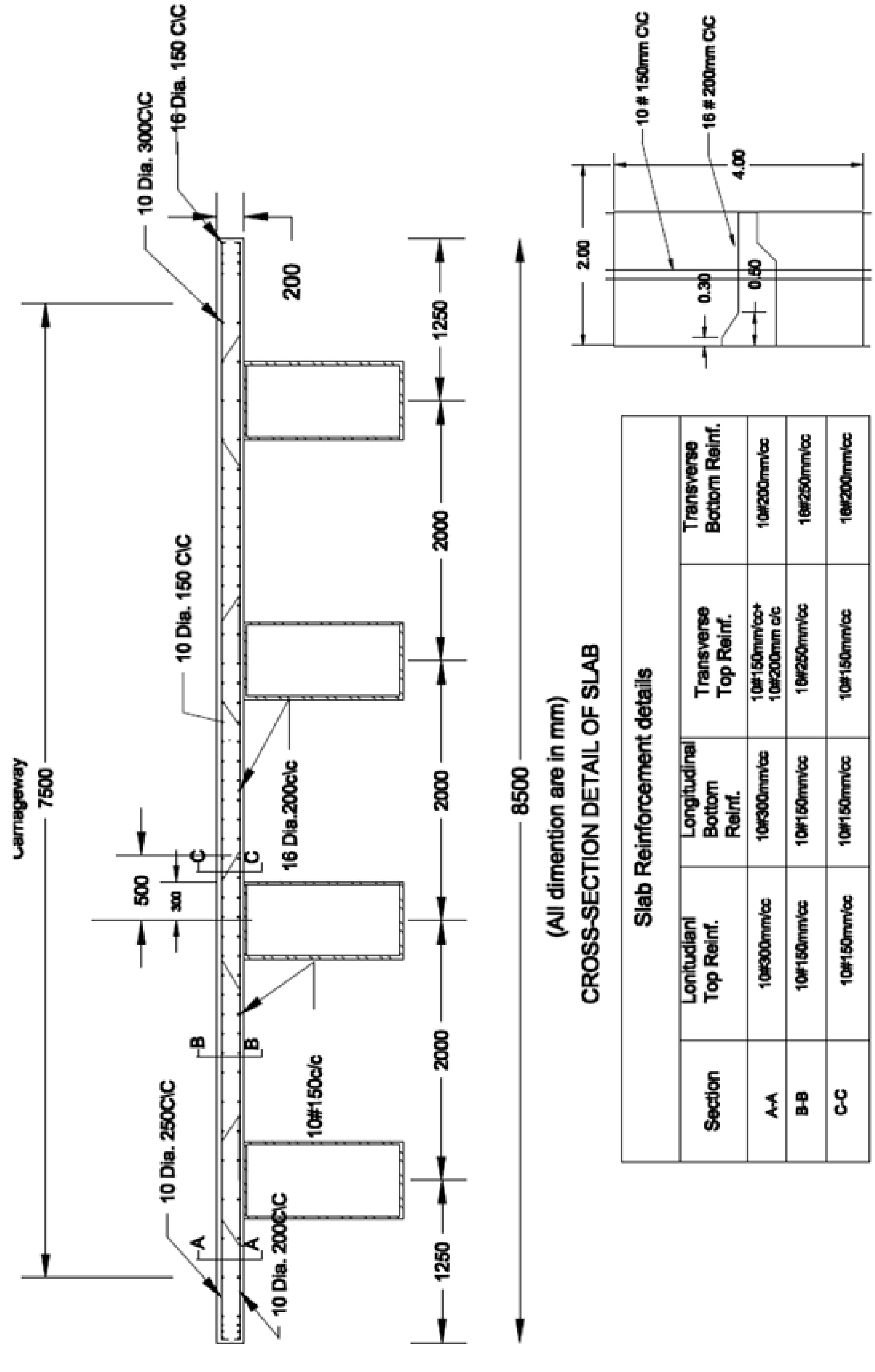
Deflection shall be checked for shorter span

Permissible Span/d = 20 ratio

Actual Span/d= 12.34 <20

Hence safe

Detailed drawing is shown in sheet no.1



Section	Lonitudianl Top Reinf.	Longitudinal Bottom Reinf.	Transverse Top Reinf.	Transverse Bottom Reinf.
A-A	10#300mm/cc	10#300mm/cc	10#150mm/cc+ 10#200mm c/c	10#200mm/cc
B-B	10#150mm/cc	10#150mm/cc	16#250mm/cc	16#250mm/cc
C-C	10#150mm/cc	10#150mm/cc	10#150mm/cc	16#200mm/cc

Figure 6.4: Sheet No.1-Detailing of Deck Slab

6.2 Design of Longitudinal Girder

Structural Data:

Effective span of Bridge		=	20 m
c/c distance of cross-girder		=	4 m
No. of cross-girder		=	6 No.
Width of bridge		=	8.5 m
No. of longitudinal girder		=	4 No.
c/c distance of longitudinal-girder		=	2 m
fy of steel		=	250 N/mm^2
Modulus of elasticity E		=	2.E+05 N/mm^2
RCC grade M	fck	=	30 N/mm^2
Thickness of slab		=	200mm
Load factor		=	1.5
Loadinf		=	70R
Condition		=	Simply supported

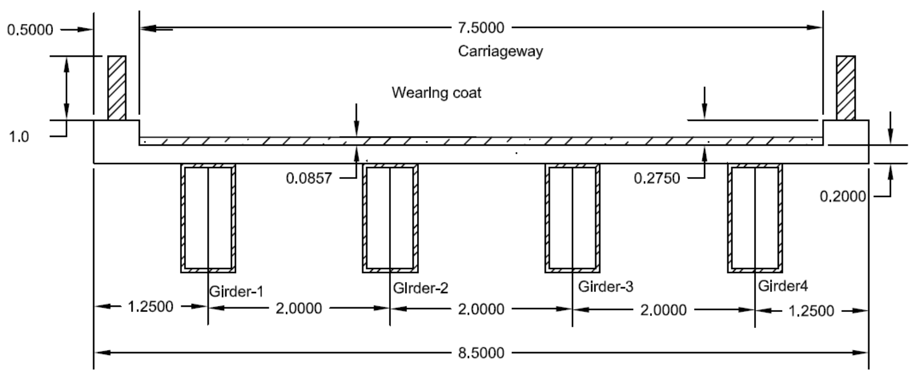


Figure 6.5: Cross section of Composite Box-girder Without Shear Connectors

Dead load of deck per metre

a)Weight of deck slab = 40.80 kN/m

b)Secound stage (Balanced DL)

1 Safety kerb = 6.60 kN/m

2 Parapet kerb = 2.40 kN/m

3 Railing = 3.00 kN/m

4 Wearing coat = 14.03 kN/m

Total = 26.03 kN/m

DL moment

Total DL = 66.83 KN/m

Assume weight of steel girder including shear connector

@ 15% of total DL (Approx)

= 10.02 KN/m

Total 1^st stage DL= 50.82 KN/m

Total 2^nd stage DL= 26.03 KN/m

Assuming uniform sharing ,Load per girder is:

1^st Stage DL= 12.71 KN/m

2^nd Stage DL = 6.51 KN/m

D.L.M per girder 1st Stage DL= 857.65 KN.m

D.L.M per girder 2^{nd}Stage DL = 439.17 KN.m

6.2.1 Analysis Results(from SAP)

	Bending moment kN.m	*Shear force kN*	*Deflection mm*
Dead load	866.95	183.75	24
SIDL	281.26	42.97	
LL(with impact)	3521.74	1254.23	
Total	4669.95	1480.95	

Design of section

Design moment for non-composite section are

1)Due to self weight of steel girder and deck slab= 857.65 kN.m

2)Add 10% for weight of formwork etc.= 85.765 kN.m

Total = 943.42kN.m

Design moment for composite section

Design moment = 2^{nd} Stage DL moment +LL moment

Assuming a steel stress for M.S girder as = 150.00 N/mm^2

Analysis Data:

Optimum depth of box girder=$(M_K/f_{yf})^{0.33}$

K=d/tw = 1000/25= 40

d = $(4669.95E6/250)^{0.33}$= 862.15 mm

Provided depth = 950 mm

Optimum thickness of web=$(M/f_y f_{k2})^{0.33}$

= 23.52mm

Provided thickness of web = 25 mm

Depth of section d		=	950	mm
	dw	=	900	mm
	b (top)	=	600	mm
	tf (top)	=	25	mm
	b (bottom)	=	600	mm
	tf (bottom)	=	25	mm
	tw	=	25	mm

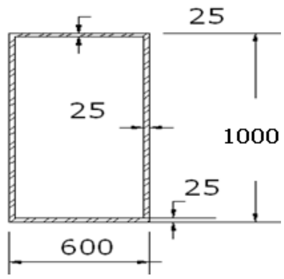


Figure 6.6: Final Section for Design

Table 6.4: Section Property

Sr. No	Area	h	Ah	Ah2	MOI about self	MOI About C.G
1	15000	987.5	14812500	1.46E+10	781250	1.46E+10
2	47500	500	23750000	1.19E+10	1.79E+09	1.37E+10
3	15000	12.5	187500	2343750	781250	3125000
total	77500		38750000	2.65E+10	1.79E+09	2.83E+10

I_{xx} = 3.E+10 mm^4

I_{yy}= 902473958.3 mm^4

y_{bottom}= 500.00 mm

y_{top} = 500.00mm

Elastic modulus Z_x = 6E+07 mm^3

Elastic modulusZ_y = 6E+07mm^3

r_x= 604.21mm

r_y= 107.91 mm

Z_{tg} = 6E+07 mm^3

= 0.057 m^3

Z_{bg} = 6E+07 mm^3

= 0.057 m^3

Moment Capacity Check :

Moment Capacity Check : Moment Resistance=
$v_d = (\frac{f_y \beta_b Z_p}{\gamma_{mo}})$
Considering that the flange only resist the bending moment
The plastic section modulus below the equal area axis
= 10132813 mm^3
The plastic section modulus above the equal area axis
=10132813 mm^3
Total section modulus = 20265625 ^{3}
Moment of resistance = 4.606E+09 Nmm
= 4705.82kNm
$>$ 4669.95 kNm Safe

Shear Capacity Check

Shear Capacity Check $v_d = (\frac{f_y \times d \times t_w}{\gamma_{mo} * \sqrt{3}})$
3280.399 kN
$>$ 1480.95kN Safe

Stress in the box girder due to self weight of girder plus weight of slab,formwork etc

M_{DL}= 943.42 kN.m
σ_{tg} = 16672.6 KN/mm^2
= 16.67 N/mm^2
σ_{bg} = 16672.6 KN/mm^2
= 16.67 N/mmm^2
permissible Steel stress = 150.00 N/mm^2 OK

Modular Ratio :

Modular ratio : For superimposed load Es/K_cXE_c =14.61

For L.L Es/Ec = 7.303

Effective width of Longitudinal girder :

Equivalent width =Effective Flange width/m

=625mm

Area of composite section = Area of compound section + Equivalent steel area of deck slab

=1070377083mm ^{2}

Centroidal axis of equivalent composite section

Taking moment about bottom of girder

X1$\times areaofcompositesection$=(Area of box section $\times It'sC.Gdistancefrombottom$) + (Area of transformed steel Area $\times C.Gdistancefrombottom$)

X1 × 10703770833 = 5.35182E+12 + 150000000

= 5.35197E+12

X1 = 500.01mm

Moment of Inertia of Equivalent Compound Section

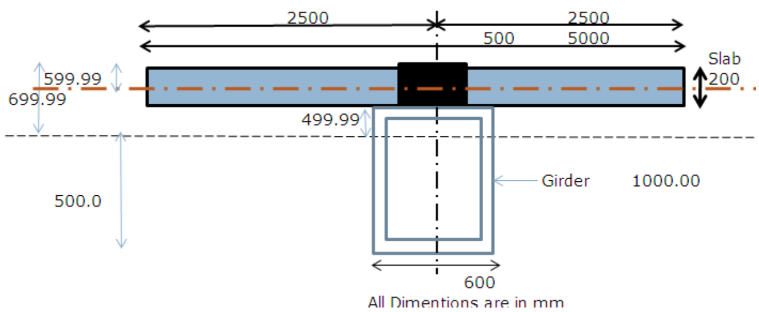


Figure 6.7: Equivalent Compound Section

Steel box (own axis)	=	7.74E+09	mm^4
Concrete secton	=	4.17E+08	mm^4
(Transformed area own axis)			
Steel box (centroidal axis)	=	715277	mm^4
Concrete secton	=	4.5E+10	mm^4
(Transformed area centroidal axis)			
Total	=	5.32E+10	mm^4

Stress due to 2nd stage DL+LL moment on composite section Stress at top of slab

=MDL+MLL/Zts$\times m$

= 60.96 N/mm^2

Stress at top of steel girder =MDL+MLL/Ztg

= 43.54 N/mm^2

Stress at bottom of steel girder =MDL+MLL/Zbg

= 43.54 N/mm^2

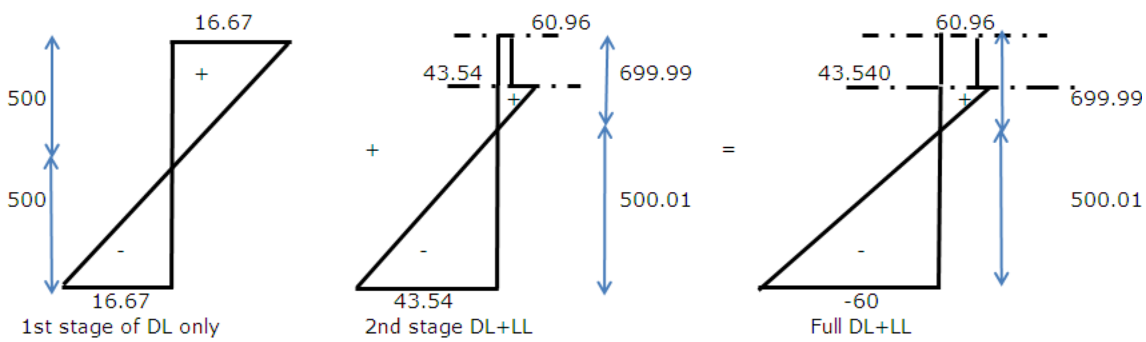


Figure 6.8: Stress Diagram

Final Stresses in Composite Box Girder

Stress at	Stresses (N/mm^2)due to		
	First Stage D.L	Secon stage D.L+L.L	Total load
Top of slab	-	60.95652	60.95652
Top of girder	16.67257	43.54017	60.21274
Bottom of girder	-16.6726	-43.5416	-60.2142

Position of Plastic Neutral Axis and Ultimate Moment of Resistance

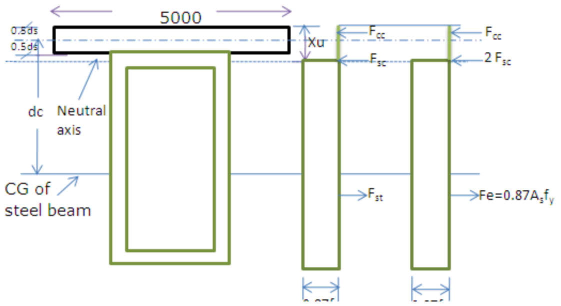


Figure 6.9: Position of Plastic Neutral Axis

Case 2 Plastic Neutral Axis in Steel Flange

$b_{eff} \times d_s$= 1.0E+06

$a \times A_s$=1.6E+06

$b_{eff} \times d_s + 2aA_f$= 1.6E+06

Satisfy the condition

xu = 202.998 mm

Mp = 10049.29 kN.m>safe 4669.95

Shear Resistance of the Web

Check for serviceability

d/tw=(1000-2×25)/25=38<200 O.K.

Check for flange buckling

d/tw=950/25=24 < 345 O.K.

Hence the minimum web thickness requirement are met.

Check Shear Force corresponding to buckling

Let us consider the simple post-critical method.

Vd=Vn/ γ_{mo}

$V_n = v_p = (\frac{A_v f_{yw}}{\sqrt{3}})$

Av = $A \times d/$(b+d) = 5.E+04mm^2

parell to depth

Av=A$\times b/$(b+d) = 3.E+04 mm^2

parell to width

Vp = 4330kN

Vd = 3936.48 KN > 1481.0 OK

V_n=V_{cr}

Vcr= shear force corresponding to web buckling

$V_{cr} = Av\tau_{cre}$

assuming C/d= 1.35

=1350

Provide C =1400 mm

as C/d >1

Kv=5.35 +4/$(c/d)^2$

Kv = 7.5

τ_{cre} = 895.276 MPa

τ_w = 0.4 ok

as

τ_b = 144.34 N/mm^2

Vcr=Av$\times\tau_b$

4330.13 kN> 1480.95 kN

So intermediate stiffeners not required

Reduction in Bending Resistance Under High Shear Force

= 0.6 $\times V_d$ = 2361.89 No reduction < v= 1480.95kN

If V<0.6 Vd

Then there is no reduction in plastic bending resistanec of the section

Check for Shear Capacity of the End Panel :

(without using tension field action)
$v_{dp} = (\frac{A_v f_{yw}}{\sqrt{3}})$
V_{dp}=3428.02kN

$v_{cr} = dt_w \tau_b$
V_{cr}=4330.12 kN
$H_q = (1.25V_{dp}(1 - \frac{v_{cr}}{v_{dp}}))$
So,H_q= 4285.02 kN
$R_{tf} = (\frac{H_q}{2})$
R_{tf}= 2142.51 kN
$A_v = t_w d$
A_v= 23750mm^2
$v_p = (\frac{A_v f_{yw}}{\sqrt{3} \times \gamma_{mo}})$
V_n=6232.76 kN>2142.51kN
The end panal is safe to carry the shear due to anchoring forces.

Check for Moment Capacity of the End Panel :

$M_{ft} = (\frac{H_q}{10})$ =407.08kNm
$Y = (\frac{C}{2})$
=700mm
$I = (\frac{1 \times t_w \times c_3}{12})$
=5.7E+09mm^4
$M_q = (\frac{I \times f_y}{y \times \gamma_{m0}})$
=1865.06kN.m > 407.08 Safe
As Mq>Mft ,hence the end panel can carry bending moment due to anchor forces

Figure 6.10: Bearing stiffeners

Design of Stiffeners

Check for Bearing Stiffeners :

At the support

Check for web crippling

Assume width of support = 400 mm

minimum stiff bearing length provided by support

b_1 = 200 mm

Thickness of flange=25mm

Dispersion length(1:2.5),n_2= 62.5 mm

$F_w = (\frac{(b_1+n_2)\times f_{yw}}{\gamma_{m0}})$

F_w =1491.5 > 1480.95 kN Safe

Check for web buckling

Slenderness ratio of the web = 2.5d/t=95 f_{cd} = 114.0 N/mm^2

b_1 = 200 mm

n_1 = 500 mm

f_{qd} = 1995 > 1480.9kN Safe

Hence not necessary to design stiffeners

Design of End Bearing Stiffeners

Choose the dimension of stiffener force due toM_{tf}=M_{ft} /c =290.77 kN

Total compression force F_c=1771.179kN

Area of stiffeners required

$A_q > \frac{(0.8xF_{cx} \times \gamma_{mo}}{f_{yq}})$

=6236.50mm^2

Provide stiffeners of two flat of size 160 X20mm

Area=6400.0mm^2 >A_q

(a) Check for out stand :

$14 \times t_q \times \epsilon$ =350mm

b_s=160mm <350mm

Hence, the criterion for the out stand has been satisfied

(b) Buckling check:

I_x= 5.46E+07 mm^4

Effective area =10340 mm^2

Radius of gyration =81.7

Flange is restrained against rotation and lateral deflection

L_e=665mm

Λ=10

F_{cd}=227.0N/ mm^2

Buckling resistance of the stiffener

$P_d = f_{cd} \times Ae$ =2347.18 kN > 1771.7kN

Hence, the stiffener is safe against buckling

(c)Check the stiffener as load bearing stiffener $b_1 = 0$ mm

$n_2 = 62.5$ mm

Local capacity of web

$F_w = (\frac{(b_1+n_2)\times f_{yw}}{\gamma_{m0}})$ =355.11 kN

Bearing stiffener is designed for $(F_c - F_w)$

$= 1165.06$kN

Bearing capacity of stiffener alone

$F_w = (\frac{A_e \times f_{yq}}{\gamma_{m0}})$

=5681.8 kN >1165.1kN Safe

=2727.27 kN >525.00kN Safe

Hence the stiffener is safe ,no need to design bearing stiffeners

Design of Intermediate Stiffener

Stiffener B is the most critical intermediate stiffener

(a) Minimum stiffeners :

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

$I_s \geq 0.75 d t_w{}^3$

I_s=937500mm^4

Try intermediate stiffener of two flats of 90 x 20 mm

Provided=1.2536E+07 mm_4

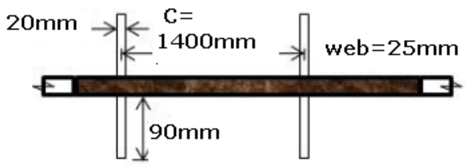


Figure 6.11: Intermediate Stiffeners

Hence, the stiffener have more than the required stiffness

(b)Check for out stand

out stand of the stiffeners=b_s=90 $14 t_q \epsilon = 280$

90mm < 280safe

hence the criteria of out stand is satisfied

(c) Buckling check : $F_w = (\frac{V - F_q}{\gamma_{m0}})$

Where,

V =factored shear force

V_{cr} =shear buckling resistance

=4330.13kN

Shear force @ B VB= 1273.617kN

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

$20t_w$=500 mm

I_x=1.38E+07 mm^4

Area=25180mm^2

r_x=23.44mm

Λ=28.4mm

F_{cd}= 225N/$mm2$

Buckling resistance of the stiffener=F_{cd}*Area

=5365.35kN

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

F_q=3172.29 kN

F_{qd}=5365.35kN

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

F_{xd}=F_{qd}=5365.35

M_q=0

$F_q - F_x$=2647.293kN

=0.59 <1 Safe

Hence the stiffener is safe at point load

Design of horizontal stiffener at 1/5 from compression flange

C/d=1.474 Try intermediate stiffener of two flats 180x20mm

Hence, the stiffener have more than the required stiffness

(b) Check for out stand : $14t_q\epsilon = 350$

b_s=180 mm < 350

Hence, the criterion for the out stand has been satisfied

Design of Horizontal Stiffener at Neutral Axis (N.A.):

(a) Minimum stiffeners : $I_s \geq d_2{t_w}^3$ d_2 =twice the clear distance from the compression

flange to the neutral axis

= 405.99mm

Required =6E+06mm^4

Try intermediate stiffener of two flats of 80 x 15 mm

Provided= 7.81E+06mm

Hence, the stiffener have more than the required stiffness (b) Check for out stand :

$14 \times t_q\epsilon = 350$

= 80 mm <350

Hence, the criterion for the out stand has been satisfied.

Connection Details :

(a)Design of weld at web flange junction

$q_w = vAy/2I_x$

=0.32kN/mm

Assume weld leg length s= 5 mm

$f_{dw} = f_u/(\gamma_{mw} \times \sqrt{3})$

=92.38

$R_{nw} = 0.7xsxf_{wd}/1000$

=0.323 kN/mm

Hence, provide 5 mm continuous weld on both side

(b)Weld for End Stiffener:

Assuming a weld on each side of the stiffener is

$q_1 = {t_w}^3/5b_s$

= 0.42 kN/mm

Length of weld = 920 mm

q_2 = 0.01 kN/mm

q_w = 0.4kN/mm

Force on each weld= 0.2kN/mm

Weld leg length s= 4mm

R_{nw} = 0.50kN/m

Hence, provide 4 mm continuous weld on both side

(c)Weld for Horizontal Stiffener :

Assuming a weld on each side of the stiffener is $q_w = 125{t_w}^2/h$

q_w=0.104 kN/mm

Weld leg length s= 3mm

= 0.33 kN/mm

Hence, provide 3 mm continuous weld on both side

Design of Web Splice:

Factored bending moment=2709.825mm

Factored shear force= 1054.3725 mm

Span of the girder = 1405830 mm

Thickness of web= 25 mm

Depth of web = 950 mm

Gross M.I. of girder= 2.E+09mm ^{4}

Gross M.I. of web only = 1.8E+09 mm^4

Bending Moment Resisted By Web plate (M_w)

M_w = 2707.5kNm

Depth of web splice d_s= 900 mm

Thickness of web splice t_s= 13.93 mm

= 18 mm

Width of web splice =600mm

Weld thickness= 16.00 mm shop weld

Distance between slot edge to outer edge

= 80mm

Width of slotb_{sl} =180mm

Depth of slot d_{sl}=740mm

$\overline{x}$=346.8 mm

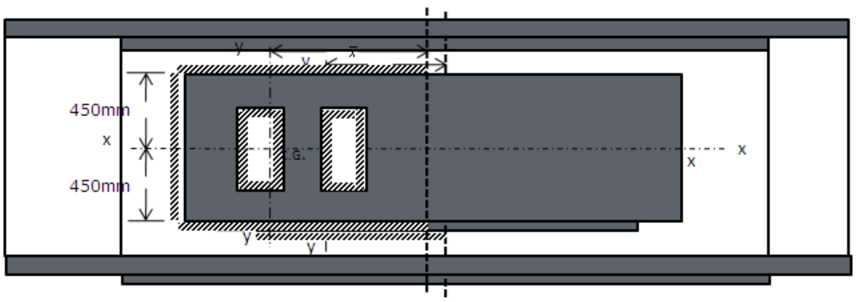


Figure 6.12: web splice

Weld length l_{weld}=5780mm

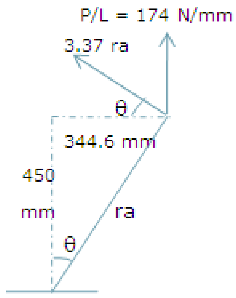
Resistance offered by the weld per mm length against translation

P/L=174N/mm

Resistance against rotation per mm length of weld at a point distance from the C.G.

S = Kr

$K = M/(I_{xx} + I_{yy})$



Moment of inertia of weld lengths

I_{xx}= 5.37E+08mm^3

I_{yy}= 1.92E+08mm^3

K = 3.71

Resistance against rotation at A per mm length of weld

$S_a = Kr_a$

= 3.709 ra N/mm^2

Total vertical component at A per mm length of weld

$V = (P/L) + s_a sin\Theta$

= 1333.96N/mm

Total horizontal component at A per mm length of weld H =1514.755N/mm

Resultant resistance per mm length at A

$\sqrt{(H^2 + V^2)}$=2223.51 N/mm

Let the maximum shear stress intensity in the weld be q N/mm2

0.7 x 16 x 1 x q = 2223.71

q = 180.2145 N/mm^2 <189N/mm^2safe

Design of Flange Splice :

Finding out tensile and compression force carried by flanges

$A_{f(top)}$ = 15000 mm2

$A_{f(bottom)}$ = 15000 mm2

Compression force = 2249 kN

Tensile force = 2405 kN

Design of Butt weld :

At top flange

Length of butt weld = 600 mm

Thickness of plate = 25 mm

Strength of weld =$L_w t_e f_y/\gamma_{mw}$

= 3000 kN Safe

At bottom flange

Length of butt weld = 600mm

Thickness of plate = 25 mm

Strength of weld =$L_w t_e f_y/\gamma_{mw}$

= 3000 kN Safe

Design of welding

At top flange

Size of welding plate

W_{fs}= 550 mm

t_p = 25 mm

L_{fs} = 250 mm

Maximum size of weld= 23.5 mm

Assume size of weld = 8 mm (shop welding)

Required length of weld = 2130 mm

Available weld length = 1050 mm

Assume width of slot W_s = 75 mm

No. of slot = 4

Length of slotL_s = 180 mm safe

Length of weld provided= 2130mmsafe

At bottom flange

Size of welding plate

W_{fs} = 550mm

t_p= 25 mm

L_{fs} = 300 mm

Maximum size of weld= 23.5 mm

Assume size of weld = 8 mm

Required length of weld= 2272.250567 mm

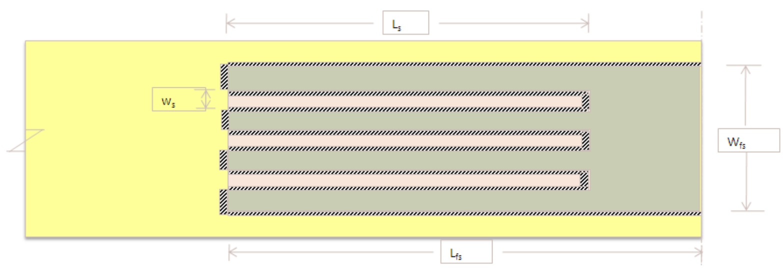
Available weld length= 1150 mm

Assume width of slotW_s=40 mm

No. of slot= 3

Length of slotL_s = 190mm safe

Length of weld provided= 2290 mm safe



Design of Cross Girder

Length of cross girder =2m

Factored B.M =566.36kN.m

Factored S.F=114.5 kN

Try ISMB 400 for cross girder

depth of section = 400

Total area = 7845.99 mm^2

Z_{xx} = 1022920 mm^3

Z_p = 22900 mm^3

d_f = 16 mm

t_w= 8.9mm

d_f=368mm

Moment of resistance=$\beta_b Z_p f_y/\gamma_{mo}$

= 4163.63kN.m > 566.36kN.m Safe

Shear resistance =$f_y d t_w/\gamma_{mo}\sqrt{3}$

= 3120.073 > 114.5 kN Safe

Connection of Cross Girder to Web

Let thickness of weld throat = 6 mm

Try ISA 100x100x8

d= 250 mm

Total length of weld=1400 mm

Vertical shear stress at weld=13.63 MPa

Horizontal shear stress due to bending at extreme fibers

=288.96 MPa

Resultant stress =302.59MPa

Design stress =302.590>227 MPa Safe

Fatigue Strength :

The bridge in use = 365 days/year = 24 hrs/day

Maximum trips of vehicle in 1 hours at

maximum load level=25 per hrs

Design life of the bridge= 100 years

Category classification f_{fn} =92(Table 6.2 of IRC:22)

γ_{mf}= 1 (Table 25)

Number of stress cycles N_{sc} = 2.0E+07cycles Safe

$f_f = \tau_{fn}\sqrt[3]{5+06/N_{sc}}$= 68.47N/$mm^2$

Design fatigue strength = 68.47 N/mm^2

Calculation of actual stress range:

$f_{min} = 0$

f_{max} = 156.38N/mm^2

f=156.38 N/mm^2 Not safe

Shear stress at support= 0.00 N/mm2

Fatigue assessment is not required

$\tau_f = \tau_{fw}\sqrt[5]{5+06/N_{sc}}$ =68.47 N/mm^2

Design fatigue strength in shear= 68.47 N/mm2 safe

From Table 17.1, log C for category 118 and N ¿ 5 ×105=12.301

$logN = logCmlogf_f$

we can also compare the number of cycles permitted at the actual stress range of f_f= 68.47 N/mm^2

Thus N =1328.91

Note:Deatailng of girder is compiled in sheet No.-2

6.2.2 Design of Shear Connector

Shear due to 2nd stage of D.L	=	462 kN
Assuming equal sharing of S.F	=	115.6 kN

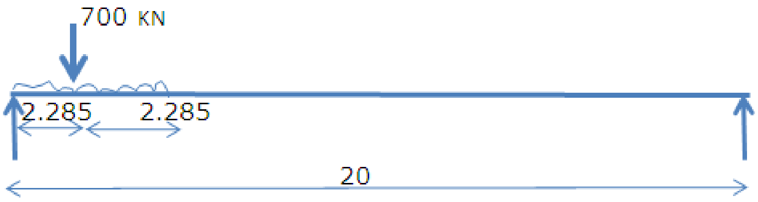


Figure 6.13: 70R Track Vehicle Placed near The support

Ra	=	620.02 kN
Shear with 10 % impact	=	682.02 kN
L.L shear on central girder	=	211.4 kN
For 20m span Impact factor		
Steel bridges	=	25
Concrete bridges	=	10
Average impact factor	=	1.176%
LL shear with impact	=	248.43 KN
Shear for intermediate girder	=	238.7KN
LL shear with impact	=	280.48.46KN

a) Shear force diagram for second stage D.L

All Dimensions are in m

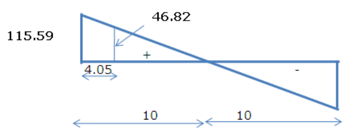


Figure 6.14: Shear force diagram for second stage D.L

b) S.F.Diagram for single lane of IRC 70R loading All Dimensions are in m

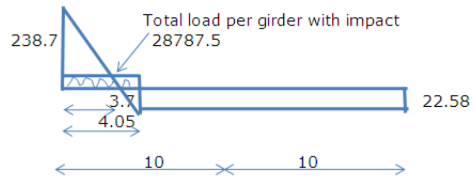


Figure 6.15: S.F.Diagram for single lane of IRC 70R loading

c)Net S.F diagram (DL+LL) All Dimensions are in m

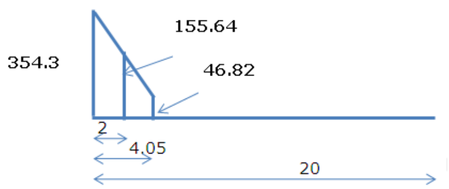


Figure 6.16: Net S.F diagram (DL+LL)

As per clause 606.4.1,The longitudinal shear per unit length $V_L = \Sigma[\frac{VA_{ce}Y}{I}]_{dlll}$
VL = 337.2047KN/cm

Spacing of shear connector $S_{L1} = \frac{\Sigma Q_u}{V_L}$ using 12 mm dia.65mm high stud,

Q = 27KN

If 3 Shear connectors are placed in 1 transverse line then, Spacing =160 mmc/c

Limiting Criteria for Spacing of Shear Connectors When the slab is in full contact over the full length

$S_L \leq 21t_f\sqrt{\frac{250}{f_y}}$
SL 300 733.46 Ok

Design of shear reinforcement as per IRC 22-2008

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

Dia. and Spacing

Top and bottom steel provided in is slab 16mm @200mmc/c

10mm @ 150mm c/c

For shear plane 1-1 $A_s = A_t + A_b$ but At and Ab should be 50% of As

For shear plane 2-2 $A_s = 2A_b$

As available for shear plane 1-1 and 2-2 is:

= 2.011 mm^2/mm A_{st}

(Min. transverse rein.) = 0.0513mm2/mm

The shear force transferred per meter length V_L shall satisfy both the following conditions:

$V_L \leq 0.632L\sqrt{f_{ck}}$

L= 100cm =1000mm 8.526≤ 316 OK or

$V_L \leq 0.232L\sqrt{f_{ck}} + 0.1A_{st}f_{st}.n$ 8.526≤ 199.44 OK

As longitudinal shear per unit length is very less than the shearing resistance of shear planes ,hence safe

A_{st}(minimum transverse reinforcement)=0.052mm^2/mm

Minimum permissible spacing of connector along longitudinal direction

should be minimum of the three:

600mm

4 times the height of connector=260

3 times the thickness of concrete slab=600

Provide 260mm is minimum spacing of connector

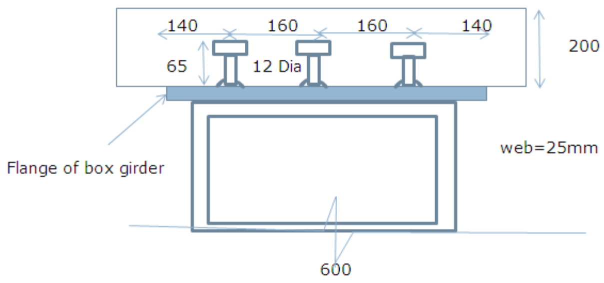


Figure 6.17: Cross-section of Composite Girder Showing Position of Shear Connector(All Dimensions are in mm)

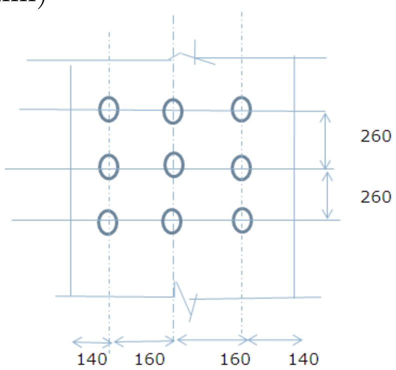


Figure 6.18: Longitudinal Section of Composite Girder Showing Position Of Shear Connector(All Dimensions are in mm)

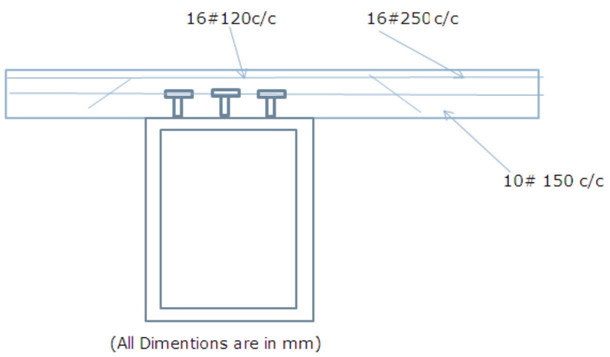


Figure 6.19: Details of Transverse Shear Reinforcement

6.3 Design Comparison

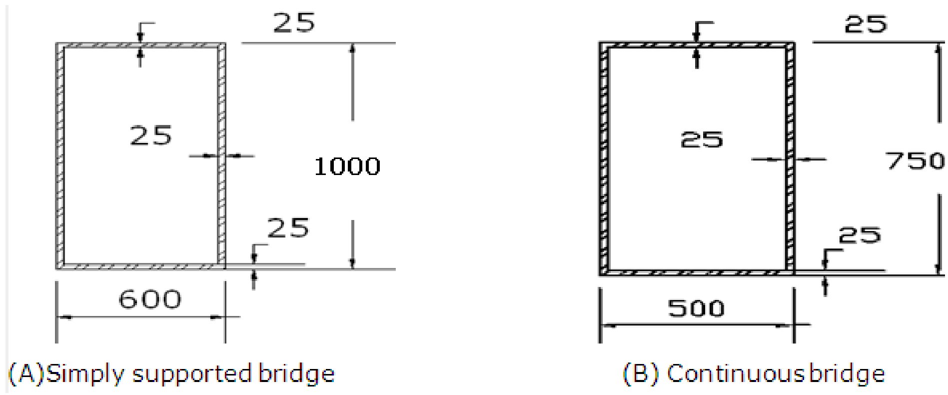


Figure 6.20: Assumed Cross section for design

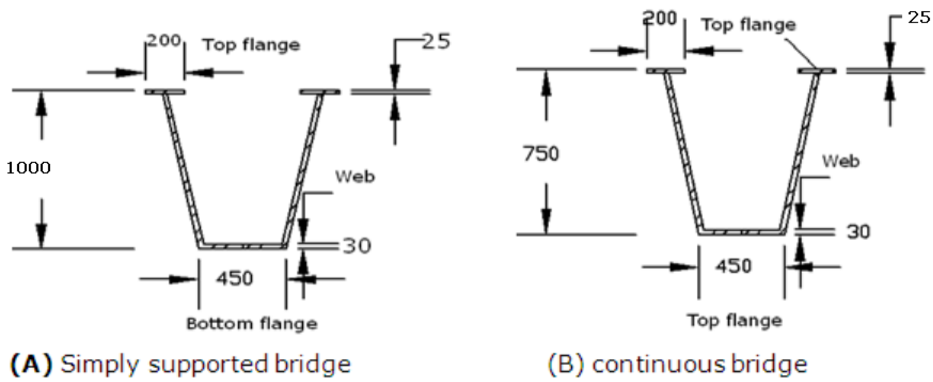


Figure 6.21: Assumed Trapezoidal Section for Design

Table 6.5: Slab analysis results for different type of bridge

Design Types	Design Span	Dead Load Moment Short span (kNm)	Dead Load Moment Long span (kNm)	Live Load Moment Short span (kNm)	Live Load Moment Long span (kNm)	Short span Moment (kNm)	Long span Moment (kNm)
Simply supported box girder	20.10	3.30	1.16	49.24	15.21	52.29	16.29
Simply supported trapezoidal girder	20.00	2.92	1.31	60.05	24.66	62.97	25.97
Continuous box girder	60.10	2.78	1.11	60.24	25.94	63.02	27.05
Continuous trapezoidal girder	60.00	2.87	72.52	55.78	15.21	52.29	16.29

Table 6.6: Slab design for different type of bridge

Interior	*Panel*	*Simply Supported Box Girder*	*Simply Supported trapezoidal Girder*	*Continuous Box Girder*	*Continuous trapezoidal Girder*
Long Span	Diameter	10	12	10	20
	Spacing	150	140	150	220
	Ast Provided	523.60	807.84	523.60	1428.00
Short span	Diameter	16	20	16	20
	Spacing	200	250	250	200
	Ast Provided	1005.3	1256.6	804.2	1570.8
Cantilever	Span				
	Diameter	10	20	10	16
	Spacing	300	170	250	220
	Ast Provided	261.80	1848.00	314.16	913.92

Table 6.7: SAP Results

Types	*Dead Load Moment (kNm)*	*SIDL Load Moment (kNm)*	*Live Load Moment (kNm)*	*Total Moment (kN.m) ent*	*Dead Load Reaction (kN)*	*Sidl Load Reaction (kN)*	*Live Load Reactction (kN)*	*Total Reaction (kN)*
Simply supported box girder	866.95	281.26	3521.74	4669.95	183.75	42.97	1254.23	1480.95
Simply supported trapzoidal girder	856.67	280.96	3469.15	4606.78	181.64	42.88	1253.81	1478.33
Continious box girder	462.89	5437.03	238.24	6138.16	58.98	1326.39	133.6	1518.97
Continious trapzoidal girder	771.2	254.09	3225.7	4250.99	242.64	74.24	1008.83	1325.71

Table 6.8: Design Compilation

	Simply Supported Box Girder	*Simply Supported Trapezoidal Girder*	*Continuous Box Girder*	*Continuous Trapezoidal Girder*
optimum depth	863	730	675	727
optimum web thickness	24	20	25	25
Moment Capacity Check	4705.8 safe	3420 Safe	3452 Safe	3945 Safe
Shear Capacity Check	3280.4 Safe	3104 Safe	2296.26 Safe	2296.61 Safe
Stress @ Top of slab	60.95	26.7	19	18.76
Stress @ Top of Girder	60.21	83.9	75	22.32
Stress @ Bottom of Girder	-60.21	-128.2	87.84	101.88
Position of NA Xu= Mu=	In flange 202.99 10049	In Web 1453.226 3332.823	In web 1447.5 4850	In web 2149.38 5006.62
S.F Corresp onding To Buckling	4330.1 Not Required	1036.9 Not Required	820.77 Stiffeners Required	1334 Stiffeners Required
Check For end panel Shear Capacity Moment Capacity	6232.8 Safe 1865.1 safe	2935.96 Safe 2130.68 Safe	2296.28 Safe 1145.83 Safe	2296.61 Safe 1145.81 Safe

Table 6.9: Girder components

Types	*Vertical stiff- ners Width (mm)*	*thk. (mm)*	*No.*	*Bea flange Width (mm)*	*ring stiff- ners thk. (mm)*	*No.*	*Horizo flange Width (mm)*	*ntal stiff- ners thk. (mm)*	*No.*	*Shear conn- ector No.*
Simply supp orted box girder	90	20	3	270	20	2	270	20	2	1231
Simply suppo rted trapz oidal girder	100	15	2	100	15	14	100	15	14	615
Conti nious box girder	90	15	2	150	15	21	150	15	21	1846
Conti nious trapz oidal girder	90	15	3	150	20	56	150	20	56	2182

Table 6.10: Connection of girder

	Simply Supporeted Box Girder	Simply Supporeted trapazoidal Girder	Continuous Box Girder	Continuous trapazoidal Girder
web to flange connection (mm)	5	3	4	3
continuous Weld length (mm)	15000	20000	60000	60000
vertical stiffener to web (mm)	3	3	3	3
Intermittent weld length (mm)	150	150	150	150
Bearing stiffener to web (mm)	12	3	3	3
Intermittent weld length (mm)	150	150	150	150
Web splice to web (mm)	16	7	8	8
continuous Weld length (mm)	6060	8410	3610	3610

Chapter 7

Estimation of Cost

7.1 General

This chapter includes the methodology of estimation of cost for bridge superstructure. The estimation of cost for any structure includes quantity analysis and rate analysis. The estimation of cost is necessary for selection of final design alternative amongst all the available various designs alternatives.

7.2 Quantity Analysis

The quantity analysis 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 analysis 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. It should be noted that the quantity analysis mentions all the items in the estimate. The quantity analysis does not give the list of materials required.Quantity analysis compilation is done for l/d ratio=16.67.

7.2.1 Estimation of Concrete and Wearing Coat Quantity

Table 7.1: Estimation of Concrete and Wearing Coat Quantity

Description	**Volume /weight**
Slab (One Panel)= 4x2x0.2	1.6 m3
Number of Panel	15No.
Total Volume in m3 = 1.6 x 30	24 m3
Wearing coat = (20x7.5x0.085)	12.75 T

7.2.2 Estimation of Reinforcement in Slab Quantity

Table 7.2: Reinforcement in Cantilever Slab

Item No.	**Particulars of item and details of works**	**Length (m)**	**Breadth (m)**	**Height or Depth m**	**Quantity m3**
Cantilever slab Rein.					
Across bridge					
Top reinforcement					
10mm @ 250mm c/c	81	2		0.89 kg/m	144.18
Bottom reinforcement					
10mm @ 250mmc/c	341	2		0.89 kg/m	606.98
Along bridge					
Top reinforcement					
Bottom reinforcement					
10mm @ 250mm c/c	60	20		0.89 kg/m	1068

Table 7.3: Reinforcement in Interior Slab

Particulars of item and details of works Slab reinforcement	***No.***	***Length* (m)**	***Breadth* (m)**	***Height or Depth* (m)**	***Quantity* (m^3)**
Across bridge					
Bottom reinforcement 12 mm @ 200 mm c/c No. of bar (20000/200) + 1 101	101	7.5	x	0.89 kg/m	674.175 kg
Top reinforcement 10 mm @ 150 mm c/c No. of bar = (20000/150) + 1 = 134.34	134.33	7.5	x	0.89 kg/m	896.675 kg
Along bridge					
Top and Bottom reinforcement 10 mm @ 150 mm c/c No. of bar ((5500/150) + 1)*2 50	50	20	x	0.89 kg/m	3951.6 kg
				Total	5522.45

7.2.3 Estimation of Structural Steel and Shear Connectors

Table 7.4: Estimation of Structural Steel

Item no.	Particulars of Item and Detail of works	No.	Length	Breath	H/Deth /weight (Kg m)	Quantity (Kg) /cum
1	web plate	8	20	0.025	0.95	29830
	(2plates)					
	950x25					
2	Bottom flange	4				
	600x25		20	0.6	0.025	9420
3	top flange	4	20	0.6	0.025	9420
	600x25					
4	Cross girders	29	2	X	61.6	3572.8
	3					Kg
5	Shear connector	1231				
6	Bearing stiffener	2				
	270x20		0.27	0.02	0.95	0.01026
7	Vertical stiffener	8				
	90x20		0.09	0.02	0.94	0.013536
8	horizontal stiffener					
	180	16	20	0.02	0.18	1.152
	20					cum
9	Web splice	16				
	900		0.018	0.6	0.9	0.15552
	18					cum
	600					
					Total	52244.11

7.2.4 Estimation of Weld Quantity for Connection in Box Girder

Table 7.5: Estimation of Weld for Connection in Girder

Item No.	Particulars of item and details of works	No.	Length m	Breadth m	Height or m	Quantity m3
1	**Web to flange connection** weld quantity mm thick. continuous weld	16	20	—	10	3200 m
2	**Vertical stiffener to web connection** weld quantity mm fillet wed 150 mm long alternative either side	4	1.46	—	3	17.5 m
3	**Bearing stiffener to web connection** weld quantity mm fillet weld 150 mm long intermittently on both side	3	1.45	—	4	17.4 m
4	**horizontal stiffener to web connection** mm fillet weld 150 mm long intermittently on both side	4	20	—	9	720
					Total	3954.9

7.3 Rate Analysis

In order to determine the rate of a particular item, the factors affecting the rate of that item are studied carefully and then finally a rate is decided for that item.With the use of that rate and estimated quantity the total tentative cost of the whole structure can be obtained. For cost estimate rate analysis of concrete is worked out wherein the rates of cement and other ingredients are considered based on current market rates. The rates of structural steel are based on current market rates.

The rates taken are as below,

Concrete : 3600 Rs./m3

Wearing coat : 4000 Rs./t

Connection : 250 Rs./length

Shear connector : 50 Rs./No.

Structural steel : 38 Rs./kg

7.3.1 Total cost

Table 7.6: Total Material Cost for 20m Span Composite Road Bridge

Span=20	L/D=16.67				
		Quantity	Rate		cost(Rs.)
Total slab concrete		34 m^3	3500	Rs/m^3	119000
Total slab reinforcement		4262.21kg	40	Rs/kg	170488.4
Wearing coat		28.05 m^3	4000	Rs/m^3	112200
Total structural steel in box girder		52244.1075kg	38	Rs/kg	1985276.09
Total connection length		438.04m	250	Rs/m	109510
Shear connector		1231No.	50	Rs/No.	61550
Total cost		=		Rs.	2448228
Cost per meter		=		Rs	122411.4

For the study purpose cost of composite bridge superstructure is taken per m of bridge length. This is done because, for different span of deck the rate changes so there is no direct comparison is possible with respect to total cost. Thus cost of bridge can be compared per meter length of bridge.

7.4 Summary

Estimation of different items like concrete, reinforcement, girder steel, shear connectors and weld length is carried out. For rate analysis, rate considered as a current market rate. After estimation and costing it is found that total cost of bridge superstructure for 20m span with L/D ratio 16.67 is 2448228 Rs.

Chapter 8

Parametric Study for Economical Span to Depth Ratio

8.1 General

The various span to depth ratio, design alternatives are required to be evaluated for quantity and costing 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 15m, 20m and 25m span by taking various span to depth (L/D) ratios like as given in table 8.1. Total 9 cases for different spans with various L/D ratio are discussed in this chapter.

Table 8.1: Data for Analysis for different sections Bridge

	SPANS (m)	**c/c dist. between cross girder(m)**	**No. of cross girder**	**c/c distance between longitudinal girder**	**No. of longitudinal girder**	**Depth taken (m)**
Simply supported box girder	20	4.0	6	2	4	1000
Continuous box girder bridge	60	4.0	16	2	4	750
simply supported trapezoidal Girder Bridge	20	4.0	6	2	4	1000
Continuous trapezoidal Girder Bridge	60	4.0	16	2	4	750

8.2 Parametric Study for Simply Supported Girder

The overall analysis methodology and step by step design procedure for 20m span is described in chapter 6. A typical cross section of 15m, 20 m and 25m simply supported span is as shown in fig 8.1 .It was analyzed with various L/D ratio alternatives using SAP-2000 software and design and analysis are compiled in table 8.2 Analysis results of parametric study are obtained by analysis in SAP software for 15m, 20m and 25m with different L/D ratio are tabulated in table 8.2. Corresponding graphical variations are also shown in Fig.8.2 and 8.3.

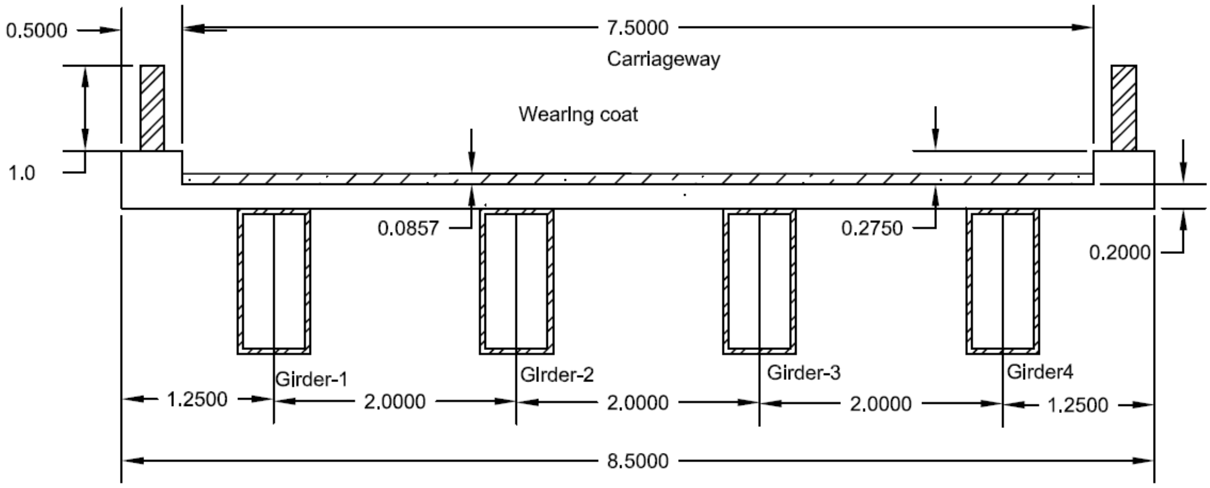


Figure 8.1: Cross Section Details for 15m, 20m and 25m span

Table 8.2: Maximum BMD and SFD in Girder

span (m)	Dead Load Moment (kNm)	SIDL Load Moment (kNm)	Live Load Moment (kN.m)	Total B.M (kN.m)	Dead Load Reaction (kN)	SIDL Reaction (kN)	Live Load Reaction (kN)	Total S.F (kN)
15	471.54	141	2514	3126.54	135.23	27	1164	1326.23
	445.05	141	2522	3108.05	127.61	27	1166	1320.61
	455.41	142	2605	3202.41	130.07	27	1212	1369.07
20	856.67	280.96	3469.15	4606.78	181.64	43	1254	1478.64
	866.95	281.26	3521.74	4669.95	183.75	43	1254	1480.75
	877.27	284.84	4012.45	5174.56	184.57	43	1251	1478.57
25	1412.98	462.89	5437.03	7312.9	238.24	59	1326	1623.24
	1360.61	464.91	4992.6	6818.12	228.15	59	1334	1621.15
	1492.18	469.92	5300.25	7262.35	249.45	60	1336	1645.45

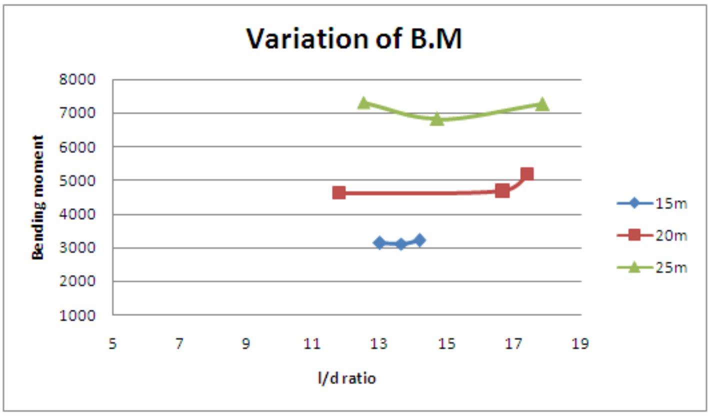


Figure 8.2: Variation in Total Bending moment

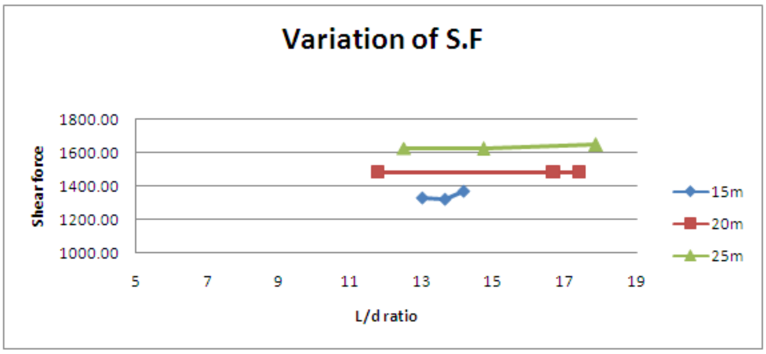


Figure 8.3: Variation in Total Shear Force

8.2.1 Costing

Initially the flange dimension and web dimension are selected in such a way that it satisfies the bending and shear stress check. For span more than 20m, with decreasing the depth of section, L/D ratio increases and weight and cost reduces up to the point where dimensions of the section satisfies the bending and shear stress check. After this point if there is decrease in the depth of section, L/D ratio increases but it's web and flange dimensions are such that they do not satisfy the bending and shear stress check and as such one has to increase the web and flange thickness (section become stiff) to satisfy the bending and shear stress check. Thus, weight and cost of the sections increases. Fig 8.4, 8.5 and 8.6 shows the concrete cost, reinforcement cost, wearing coat cost, girder steel cost, shear connector cost and connection cost for 15m, 20m and 25m respectively with different L/D ratio. Table 8.4, 8.6 and 8.8 shows that the deck slab concrete, slab reinforcement and wearing coat cost does not affects the L/D ratio. From the Fig. 8.4, Fig. 8.5, 8.6 it is clear that total cost of super structure is mainly affected by girder steel cost. Table 8.4 shows that L/D ratio 13.64 is most economical L/D ratio for 15m span among all L/D ratio alternatives. Table 8.6 shows that L/D ratio 16.67is most economical L/D ratio for 20m span among all L/D ratio alternatives. Table8.8 shows that L/D ratio 14.71 is most economical L/D ratio for 25m span among all L/D ratio alternatives.

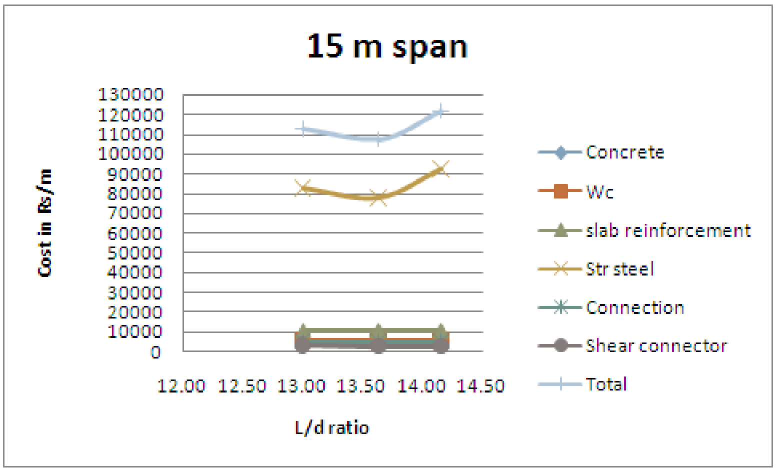


Figure 8.4: Variation in Cost of Material, Other Items and Total Cost per meter for 15m span With Various L/D Ratio

Table 8.3: Quantity of Different Items and Total Cost Per meter for 15m Span With Various L/D Ratio

15	13 Quantity	13.64 Quantity	14.16 Quantity	Rate		13 cost	13.64 cost	14.16 cost
Total slab concrete	25.5	25.5	25.5	3500	Rs/m3	89250	89250	89250
Total slab reinfor cement	6002.2	6002.16	6002.2	40	Rs/kg	156960.4	156960.4	156960.4
Wearing coat	21.038	21.0375	21.038	4000	Rs/m3	84150	84150	84150
Total srtuctural steel in box girder	32512	30628.298	36328	38	Rs/kg	1235467	1163875	1380447
Total connection length	301.74	299.32	301.62	250	Rs/m	75435	74830	75405
Shear connector	810	760	910	50	Rs/No.	45500	40500	38000
Total cost					Rs.	1686763	1609566	1824212
Cost per meter					Rs	112450.8	107304.4	121614.1

Table 8.4: Cost of Different Items and Total Cost Per meter for 15m Span With Various L/D Ratio

L/D	Concrete	Wc	Slab reinforcement	Structural steel	Connection	Shear connector	Total
13	5950	5610	10464.03	82364	5029	3033.3	112450.85
13.64	5950	5610	10464.03	77591.69	4988.67	2700	107304.38
14.16	5950	5610	10464.03	92029.77	5027	2533.3	121614.13

Table 8.5: Quantity of Different Items and Total Cost Per meter for 20m Span With Various L/D Ratio

20	11.77 Quantity	16.67 Quantity	17.4 Quantity	Rate		11.77 cost	16.67 cost	17.4 cost
Total slab concrete	34	34	34	3500	Rs/m3	119000	119000	119000
Total slab reinfor	6816.51	6816.5	6816.5	40	Rs/kg	170488.4	170488	170488
cement Wearing coat	28.05	28.05	28.05	4000	Rs/m3	112200	112200	112200
Total structural steel in box girder	50058.1649	51628	50058	38	Rs/kg	2078799	1961870	1902210
Total connection length	398.92	397.32	398.92	250	Rs/m	105395	99330	99730
Shear connector	850	900	850	50	Rs/No.	70500	45000	42500
Total cost Cost per meter					Rs. Rs	2656382 132819.1	2507889 125394	2446129 122306

Table 8.6: Cost of Different Items and Total Cost per meter for 20m Span With Various L/D Ratio

L/D	Concrete	Wc	Slab reinforcement	Structural steel	Connection	Shear connector	Total
11.77	5950	5610	8525.42	98093.51	5269.75	3525	126973
16.67	5950	5610	8525.42	95110.51	4966.5	2250	122411
17.4	5950	5610	8525.42	103940	4986.5	2125	131136

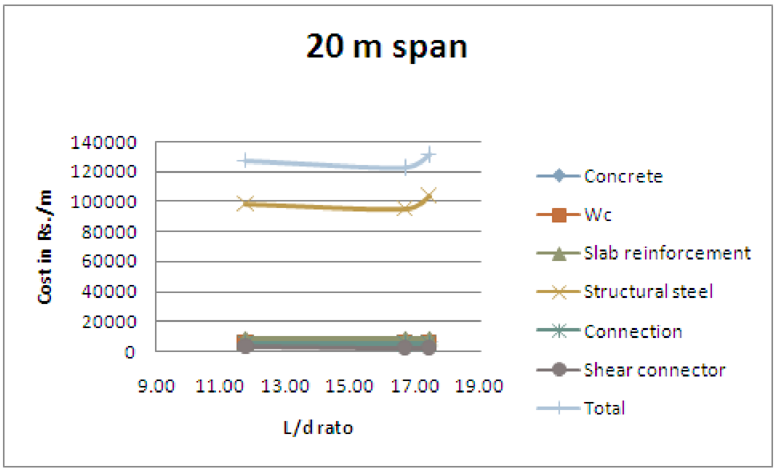


Figure 8.5: Variation in cost of Material, Other Items and Total Cost per meter for 20m Span With Various L/D Ratio

Table 8.7: Quantity of Different Items and Total Cost Per meter for 25m Span With Various L/D Ratio

25	12.5 Quantity	14.71 Quantity	17.86 Quantity	Rate		12.5 cost	14.71 cost	17.86 cost
Total slab concrete	42.5	42.5	42.5	3500	Rs/m3	148750	148750	148750
Total slab reinfor cement	7067.19333	7067.2	7067.2	40	Rs/kg	156960.4	156960	156960
Wearing coat	35.0625	35.063	35.063	4000	Rs/m3	140250	140250	140250
Total srtuctural steel in box girder	76967.6569	67548	71551	38	Rs/kg	2924771	2566808	2718950
Total connection length	537.3	527.3	530.02	250	Rs/m	134325	131825	132505
Shear connector	1710	1410	1100	50	Rs/No.	85500	70500	55000
Total cost					Rs.	3590556	3215093	3352415
Cost per meter					Rs	143622.3	128604	134097

Table 8.8: Cost of Different Items and Total Cost Per meter for 25m Span With Various L/D Ratio

L/D	Concrete	Wc	Slab reinforcement	Structural steel	Connection	Shear connector	Total
12.5	5950	5610	6278.42	116990.8	5373	3420	143622.25
14.71	5950	5610	6278.42	102672.3	5273	2820	128603.73
17.86	5950	5610	6278.42	108758	5300	2200	134096.62

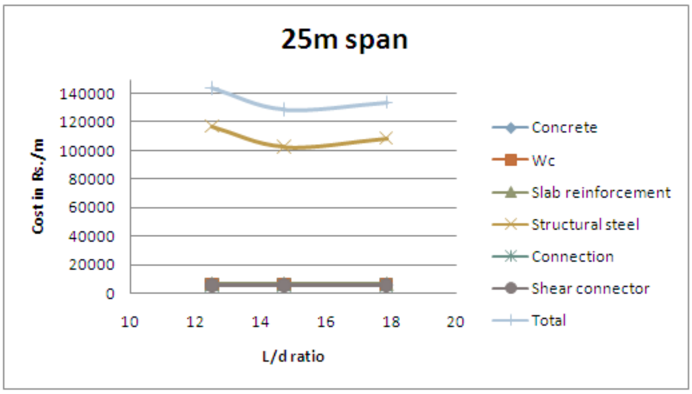


Figure 8.6: Variation in Cost of Material, Other Items and Total Cost Per meter for 25m Span With Various L/D Ratio

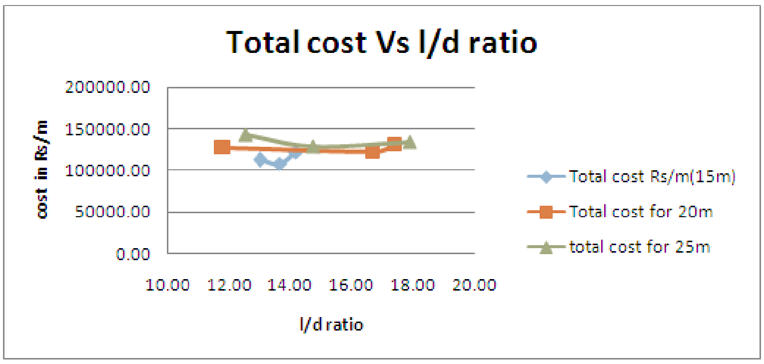


Figure 8.7: Variation in Total Cost Per meter for 15m,20 and 25 m span With Various L/D Ratio

8.3 Comparison Between Closed Box Girder and Open Trapezoidal Web Girder

Table 8.9: Cost of Different Items and Total Cost Per meter for Different Span

Types	Concrete	Wearing Coat	Slab reinfor cement	Structural steel	Conne-ction	Shear conne ctor	Total cost	% Variation
Simply supported box girder	5950	5610	8524.42	96923.004	5475.5	3078	129135	4.7434
Simply supported trapezoidal girder	5950	5610	16067.507	98390.077	4430.813	1538	135261	
Continuous box girder	5950	5610	7493.2067	82561.957	4184.375	1538	110113	0.0328
Continuous trapezoidal girder	5950	5619.35	7482.8665	82689.154	4139.884	1818	110077	

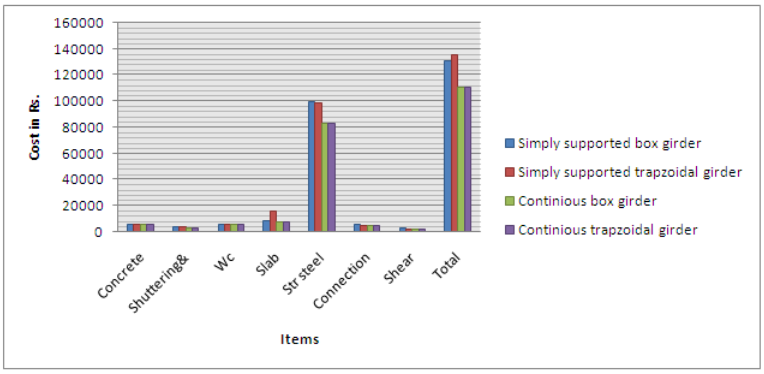


Figure 8.8: Comparison of Different Bridges

8.4 Summary

Parametric study is carried out to find out the effective economical L/D ratio for 15m, 20m and 25m. It is found that L/D ratio 13.64, 16.67and 14.71are most economical L/D ratio for 15m, 20m and 25m spans respectively. Cost comparison between closed rectangular box girder and open trapezoidal girder is carried out. It is found that in simply supported rectangular box girder bridge is 4.74 % economical than simply supported trapezoidal bridge while in continuous span trapezoidal open section is 0.03 % economical than continuous rectangular box span.

Chapter 9

Summary and Conclusion

9.1 Summary

The main objective of the work was to study the composite behavior in the composite road bridge superstructures consisting of concrete slab and steel girder joined together with shear connectors and to find out minimum cost or economical span to depth ratio for 15m, 20m and 25m spans & various depths.

Composite road bridge is analyzed using SAP software. Excel spreadsheet are prepared for deck slab design, longitudinal composite girder design as per IRC-22:2008 and IS 800:2007, shear connectors, cross girder, 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 A and class 70R IRC loading are considered.

Total 9 alternatives of span to depth ratio as compiled in table 5.1are analyzed using SAP software and design are done using prepared spreadsheet. Composite girder is designed to satisfy bending check, shear check and deflection check. 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(taken including shuttering cost and scaffolding cost), structural steel cost,connection cost and shear connectors cost are considered.

Estimation and costing are compared between different bridge type like simply supported closed and open section and continuous closed and open section.

The section designs are carried out for 15m, 20m and 25m span as per the IS 800:2007 and IRC22:2008 provisions. All the sections are safe in bending and shear stress and deflection for respective spans.

Four types of problems as under are analysed & designed

1. Simply supported closed rectangular box section.

2. Continuous closed rectangular box section.

3. Simply supported open trapezoidal section.

4. Continuous open trapezoidal section.

9.2 Conclusions

Based on above study the following conclusions are drawn:

- Maximum live load moment is carried out when the two class 70R IRC loading moving at a time on two lanes.

- As per IRC:22-2008 for composite girder,various aspects like section classification,plastic M.R of the section & shear connector design is studied in detail.

- Initially section dimension are so selected that as increasing in L/D ratio up to certain limit where it satisfies bending and shear check. After this point, if there is increase in L/D ratio, section are not satisfying bending and shear check. So one has to increase the web and flange thickness to satisfy bending and shear stress check and hence, weight and cost of the girder sections increases.
- It is observed that as L/D ratio increases total B.M and S.F increases.
- As the span increases the total cost of super structure per meter increases then decrease & again increase.
- For different spans like 15m, 20m and 25m, the economical L/D ratio is found to be 13.64, 16.67 and 14.71 respectively.
- With various l/d ratio concrete & wearing coat remains almost same but steel cost increases with increase in l/d ratio.
- Area of steel required for deck slab decreases with increase in span .
- Number of shear connectors are more in closed box section as compared to open section.
- It is found that in simply supported rectangular box girder bridge is 1.17 % economical than simply supported trapezoidal bridge while in continuous span trapezoidal open section is 0.03% economical than continuous rectangular box span.

9.3 Future Scope of Work

- In this study straight composite road bridge are taken, the work can be extended for skew type of composite road bridge.

- 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.
- Study can be extended by applying different types of bracing system inside the box section.

Appendix A

List Of Useful Websites

- www.asce.org
- www.steel-insdag.org
- www.compositeworld.com
- www.elsevier.com
- www.kscl.com
- www.sciencedirect.com
- www.engconfintl.org
- www.steelbridge08.com

Appendix B

List of Papers Presented/Communicated

List of Paper Presented

Priyanka Pal,"Analysis and Design of Continuous 3-Span Composite Box Girder Bridge", Dr. S.N. Patel Seminar, Birla Vishvakarma Mahavidyalaya, Vallabh Vidyanagar, January 2010.

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[13] *IRC: 6-2000Standard Specification and Code of Practice for Road Bridges Section II - Load & Stresses,* $4^{t}hRevision$.

[14] *IRC: 21-1987 Standard Specification and Code of Practice for Road Bridges, Section III Cement Concrete (Plain and Reinforced),*$2^{n}d$ *Revision.*

[15] *IRC: 22-2000 Standard Specification and Code of Practice for Road Bridges,Section VI Composite Construction(Limit state design),* $2^{n}d$ *Revision.*