LIMIT STATE DESIGN OF R.C. BRIDGE SUPERSTRUCTURE

By

Sweta Ojha

08MCL010

DEPARTMENT OF CIVIL ENGINEERING

AHMEDABAD-382481

May 2010

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Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology Civil Engineering

(Computer Aided Structural Analysis and Design)

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Declaration

This is to certify that

i) The thesis comprises my original work towards the degree of Master of Technology in 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.

Sweta Ojha

Certificate

This is to certify that the Major Project entitled “Limit State Design of R.C. Bridge Superstructure ”submitted by Sweta Ojha (08MCL010), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering 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.

Prof.N.C.Vyas
Guide,
Professor,
Department Civil Engineering,
Institute of Technology,
Nirma University, Ahmedabad

Prof. P. H. Shah
Professor and Head,
Department of Civil Engineering,
Institute of Technology,
Nirma University,Ahmedabad

Dr K. Kotecha
Director,
Institute of Technology,
Nirma University, Ahmedabad

Examiner

Date of Examination

Abstract

Revolutionary changes have taken place in the practices and philosophies in bridge design during the last century. Yet many important factors were lost sight of the design because of inadequate knowledge and experience. This has adversely affected the durability and serviceability of the bridge and has resulted in their premature deterioration and other problems. The bridge designers are now becoming increasingly aware of many new factors and dimensions in the designing of modern bridges. This is going to be great help in designing more beautiful and durable bridges which will be constructed with ease, efficiency and economy and which will remain in service for a very much longer period. There are many factors which affects bridge durability, which is both environmental conditions and design philosophy.

In present study, comparison of limit state design and load and resistance factored design is done. By taking an bridge superstructure data, which is analysed and designed by both methods. And are compared to obtain economical design. The main emphasis is given on limit state method design as per IS 456-2000 and and partial safety factors from IRC-6 amendments no.8, which covers limit state of collapse , limit state of cracking ,limit state of serviceability and partial safety factors. And load and resistance design is done using ASSTHO specifications.

By taking different depth for different span the economical L/D ratio is obtained. Quantity and rate analysis is done to obtain the total economic cost of bridge superstructure. The bridge superstructure is designed by both the methods and compared with working stress design method to find most efficient design philosophy.

Acknowledgements

It gives me great pleasure in expressing thanks and profound gratitude to Prof.N.C.Vyas, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad for his valuable guidance and continual encouragement throughout the thesis work. I heartily thankful to him for his time to time suggestion and the clarity of the concepts of the topic that helped me a lot for my thesis.

My special thanks to Mr.Devang Patel, Principal associate of STUP Consultant Pvt.Ltd who helped me collecting data for design and clearing doubts whenever I got stuck.And Mr.H.Solanki sir who helped me during literature survey.And also i would like to thanks Prof.G.N.Patel,Dr.P.V.Patel and Dr.U.V.Dave for there time to time suggestions during reviews.

At last but not the least my family and friends , who always bear and motivate me throughout this thesis preparation, I am thankful to them.

Sweta Ojha

08MCL010

Abbreviation Notation and Nomenclature

A_{st} .... Area of Steel in tension
A_{sc} .... Area of Steel in compression
A_{sf} .... Area of steel in flange
a_{cs} .... Deflection due to shrinkage
a_{cr} .... distance from the point under consideration to the nearest surface
b_f .... Effective width of flange
b_w .... Width of web
d', d_c .... Effective cover
D_f, h_f .... Depth of flange
D .... Total depth of section
x_u, a, x .... Depth of neutral axis
E_c .... Modulus of elacticity of concrete
E_S .... Modulus of elacticity of steel
E_{ce} .... Modulus of elacticity of concrete
f_{ck} .... Characteristic compressive strength of concrete
f_y .... Yield stress of the material
I_{eff} .... Effective moment of flanged section
I_{gr} .... Moment of inertia of gross-section(uncracked section)
I_r .... Moment of inertia of gross-section ignoring reinforcement (cracked section)
M_r .... Moment of resistance
v_u .... Factored shear force
M_u .... Factored moment
d .... Effective depth of section
τ_{uc} .... Permissible shear stress in concrete
V_{uc} .... Shear strength of concrete
V_{us} .... Shear strength of steel
m .... modular ratio

z, j, d_v .... Lever arm distance

W_{cr} .... Crack width of surface

ϵ_m .... Average steel strain at level cracking considerd

ϵ_1 .... Strain at considered level

Z .... Crack parameter

v, τ .... Nominal shear

ϕ_v .... Resistance factor

M_{cr} .... Moment of resistance

$D.L$ .... Dead load of permanent component

$W.C.L$ .... Wearing coat load

$L.L$ .... Live load of vehicle

γ .... Partial safety factor

θ .... Creep coefficent

$\Delta_{D.L}$ .... Deflection due to dead load

$\Delta_{L.L}$ .... Deflection due to live load

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Chapter 1

Introduction

1.1 General

Bridges are the links in roads, which close the discontinuity across any natural or manmade feature on the ground. Now a day due to requirement of speed of traffic, the requirement of Flyover Bridge, is also increased. Bridges are built spending large sums of money and are expected to remain in service for long period-50 to 100 year of even more. So, it should have a predetermined useful life. Its failure load should be sufficiently greater than the working load in order that the probability of its failure during its life-time is less than a specified limit and the required safety remains inbuilt. The cracking, vibrations and deflection of the bridge under working loads should not be so large as to impair its safety or serviceability during its life-time. The economic considerations with regard to its design, construction and maintenance. For doing so, methodologies like the elastic analyses, are in vogue currently, the limit state approach being the latest trend.

1.2 Design Philosophy

In bridge engineering, there are two principal methods of design in use today. The names used to define these design methods vary depending on the structural material being used and the design codes being referenced, they are classified as:

•Working stress design

•Limit states design

•Load and resistance factored design

For most of the century, the working stress design approach was the standard by which bridges and other structural engineering projects were designed. It was Russians who set the ball rolling, as far as back as 1938 they chose to break away from working stress design though it continued to enjoy wide patronage in the rest of the world. The limit state design method was developed in the erstwhile USSR in the period 1947-49 and approved in 1955. The Russian Standard of reinforced concrete NITU 123-55 was re-elaborating in 1962 through SNIP II-B.1.62 and the design method was adopted by virtue of the recommendation COMECONN in the East Europe also, in 1963. In 1964, CEB brought out its first edition of a model code adopting semi-probabilistic limit state design method followed by the joint FIP/CEB second edition in 1970. The British Unified Code appeared in late 1972 has new limit state .So, the replacement of the traditional code format by the new one is almost the inevitable natural process like shedding of old clothes and taking on the new. Therefore, by 1970's limit states design began to gain acceptance by the general engineering community.In 1986, in order to overcome the disadvantages of limit state method LRFD method was adopted by Canada, America and other European countries.

1.2.1 Working Stress Design Method

Working stress design is an approach in which structural members are designed so that unit stresses do not exceed a predefined allowable stress. Factor of safety applied to the yield or ultimate stress to get permissible stress. Structure designed to support working or service loads without exceeding the permissible stresses in concrete and steel. The allowable stress is defined by a limiting stress divided by a factor of safety. The salient features are described below:

a. Actual stresses are representative of stresses due to the service or working loads that a structure is suppose to carry. Figure1.1(a) and (b) show stress-strain diagrams for concrete and steel respectively. The point where a material behaves elastically is defined as the proportional limit. Once stress and strain are no longer proportional, the material enters in plastic region. The working stress method is logically not applicable to concrete structures, because the range of proportionality is very small.

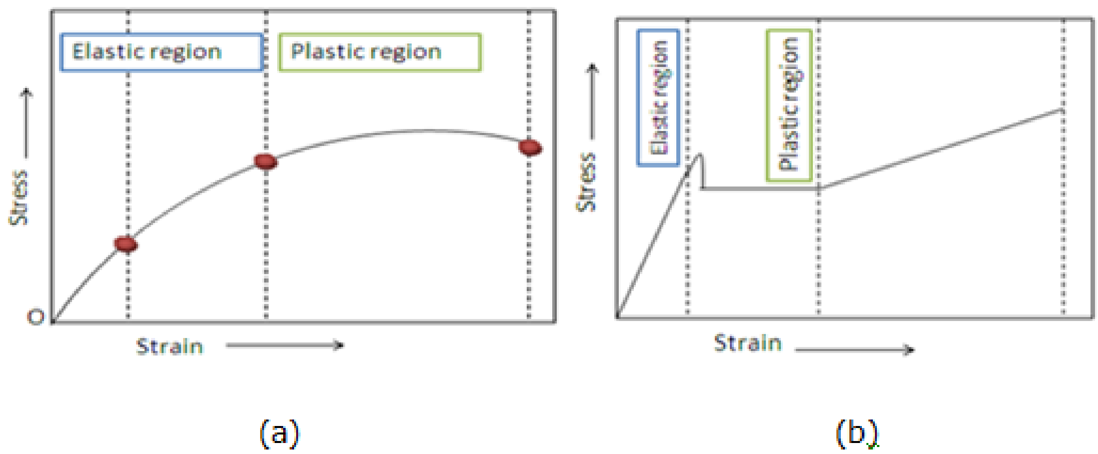

Figure 1.1: Stress-Strain Diagram For (a) Concrete Strength In Compression And (a) Steel Strength In Tension

b. Due to non-linear stress-strain relationship modulus of elasticity also varies; therefore, constant value of modular ratio cannot be used.

c. Factor of safety does not predict true margin of safety.

d. Additional load carrying capacity in the plastic region is not taken in account.

e. It considers ultimate stress as the limit of safety is a function of ultimate strain and ultimate stress.

f. Since the structure is subjected to loads, the loads should form the failure criteria and not the stress.

g. The effect of creep and shrinkage of concrete is totally ignored.

h. Failure load computed by this method in majority of the cases is less than that obtained by experimental results at collapse.

i. It is simple and reasonable reliable method.

1.2.2 Limit State Design Method

A limit state is a condition beyond which a bridge system or bridge component ceases to fulfill the function for which it is designed. The limit state design method was, in part, developed to address the drawbacks to the working stress approach mentioned above. This approach makes use of the plastic region for the design of structural members and incorporates load factors to take into account the inherit variability of loading. The quote from the AISC Manual of Steel Construction defines limit state as a condition representing ”structural usefulness.” The strength required is computed using conventional analysis methods and multiplying computed values by appropriate load factors. If we simply considering dead load times some factor plus live load times the another factor. Specific values for factors are provided by the applicable design code.

Classification of Limit State

There are two broad categories of limit states, namely:
•Limit states of collapse (Ultimate Limit State)
•Limit states of serviceability

Limit State of Collapse

It is limit state on attainment of which the structure is likely to collapse. It relates to stability and ultimate strength of the structure. Design to limit state ensures safety of structure from collapse. The structural failure can be collapse of one or more member occurring (i.e. material failure, breakage due development of plastic hinge at critical section, bulking) as a result of force coming on the member exceeding its strength. And other condition (i.e. sliding, overturning, and sinking) is displacement of structure bodily due to lack of equilibrium between the external forces and resisting reaction. This limit state is attended to by providing resistance greater than the force coming on it and keeping a margin of safety factors.

Limit State of Serviceability

Limit states of serviceability related to performance or behavior of structure at working loads and are based on causes affecting serviceability of the structure. They mainly subdivided into following categories:
(a) Limit states of deflection: Design to limit state safeguards the serviceability of the structure from adverse effect of excessive deflection which affects the shape of structure, creates feeling of lack of safety, poor drainage or ponding effect. The limit state is attended to by prescribing maximum allowable deflections or by prescribing maximum allowable span to depth ratios.

(b) Limit state of cracking: Design to this limit state safeguards the serviceability of the structure against damage due to excessive cracking. Due to cracking it creates exposed surface appearance which leads to corrosion of steel bar; reduces the imperviousness, strength and durability of the structure; creates lots of maintenance problems. Cracking is not dangerous directly but lead to ill effects. This limit state is attended to by imposing restrictions on maximum crack width for important structures and by adhering to appropriate detailing rules and restrictions on bar diameter, spacing, cover etc. for structures.
(c) Other limit states: Structures designed for special or unusual functions need considerations of appropriate limit state. They are vibration, fire resistance and durability.
The salient features of limit state are described below:

a. It considers the actual behavior of the structure during the entire loading history up to collapse.

b. It is adopts the concept of fitness of structure to serve the desired function during the service life span and defines the limiting state of fitness as the 'limit state'.

c. It attempts to define quantitatively the margin of safety or fitness on some scientific mathematical foundations rather than on adhoc basis of experience and judgment.

d. The method adopts the idea of probability of structure becoming unfit, and attempts to achieve the minimum acceptable probability of failure.

e. The method is based on statistical probabilistic principle.

1.2.3 Load and resistance factored design

To overcome the deficiencies of limit state design, load and resistance factored design new approach. To account for the variability on both sides of the inequality in the equation, the resistance side is multiplied by a statistically based resistance factored , ,whose value is usually less than one, and the load side is multipled by statistically based load factored ,whose value is usually greater greater than one. Because the load effect at a particular limit state involves a combination of different types that have different degrees of predictability, the load effect is represented by the summation of values. If the nominal resistance is given by Rn, the safety criterion is

Equation involves both load factors and resistance factors, the design method is called load and resistance factor design. The resistance factored for a particular limit state must account for the uncertainties in material properties, equation that predicts strength, workmanship, quality control and consequence of a failure. The load factored chosen for a particular load type must consider the uncertainties magnitude of loads, arrangement of load and possible combinations of loads. In selecting resistance factors and load factors for bridges, probabililty theory has been applied to data on strength of materials, and statistics on weight of materials and vehicular loads.
The salient features are described below:

a. Account for variability in both resistance and load.

b. Achieves fairly uniform levels of safety for different limit states bridge types without involving probability or statistical analysis.

c. Provides a rational and consistent method of design.

d. Provides consistency with other design specification.

1.2.4 Characteristic Strength of Materials

The characteristic strength for all materials is defined as the value of the strength of concrete, and of the yield or proof-stress of reinforcement, below which not more than 5% of the test results may be expected to fall. The value therefore is

$$f_k = (f_m - 1.64s) \tag{1.1}$$

1.2.5 Characteristic Load

Characteristic load means that value of load which has a per 95% probability of not being of not exceeded during the life of the structure. Ideally this should be determined from mean load and its standard deviations from mean, using the same probability as for the materials that

$$F_k = (F_m + 1.64s) \tag{1.2}$$

1.2.6 Partial Safety Factors and Design Load

The safety of the structure depends on each of the two principal design factors, one for load and other, two different safety factors, one for load and the other for material strength are used instead of a single safety factor. Because each of the two safety factors contributed partially to safety, they termed as partial safety factors. Partial safety factor for load enhancing factor (greater than unity) which when multiplied to characteristic load gives a load known as design load for which the structure is to be designed. It takes into account unforeseen possible increase in load, accurate assessment of load effect, unexpected stress redistribution and variation in dimensional accuracy. Thus, it makes provision for margin of safety.

1.3 Objective of Study

a. To study the limit state method of design for R.C.C highway (flyover) bridge superstructure.

b. To study the load and resistance factored design of R.C.C highway (flyover) bridge superstructure.

c. To analyze and design bridge superstructure under IRC−6 loading.

d. To carry out the cost analysis for highway superstructure design as per limit state method and load and resistance factored design.

e. To carry out parametric study of bridge superstructure at different depth for different span for the economical section.

1.4 Scope of Work

a. Study of design philosophy i.e. working stress, limit state method and load and resistance factored design.

b. Analysis of superstructure is carried out on SAP2000−12 civil software and design is done using excel worksheet.

c. Two Lane Bridge with three I-shaped girders considered for analysis and design.

d. For analysis live load is considered are Class−70R tracked/wheeled and Class−A wheeled as per IRC: 6−1966.

e. For design of superstructure is done by using following codes: i) Limit state method: Using IRC−6 draft code and IS−456:2000

ii) Load and resistance factored design: Using AASTHO specification

f. Comparison of cost for design method for limit state , load and resistance factored design and working stress method.

g. Paramateric study for the economical depth of bridge superstructure of limit state design approach.

1.5 Thesis Organization

The rest of the thesis is organized as follows.

Chapter1,*Introduction*,deal with the basic introduction of working stress and limit state design. And also includes the objective and scope of work.

Chapter2,*Literature Review*,shows review of various books and papers describing design philosophy of R.C.C bridge structure, classification of design methods and construction aspects.

Chapter 3, *Comparison of Design Methods*, describes the comparison of both methods working stress and limit state design and behavior of beam with various useful formula.

chapter 4, *Bridge Superstructure Analysis*, shows the analysis of bridge superstructure by calculating dead load and live load by doing manual calculation and using software SAP2000.

Chapter 5, *Bridge Superstructure Design*, describes the design steps required for working stress and limit state method.

Chapter 6, *Estimation of Quantity and cost*, includes estimation of concrete and steel for deck slab,girders and wearing coat with cost.

Chapter 7, *Parametric Study*, Chapter 8 Includes the parametric study for 15m, 20m and 25m span at depth 1.1m,1.5m and 1.9m gives various L/D ratio to find out the economical L/D ratio. Cost comparison between LSD design and LRFD design. In this parametric study trials are taken at different depths.

Finally, in **chapter 8** concluding remarks and scope for future work is presented.

Chapter 2

Literature Review

2.1 General

Bridges are engineering works that affords passage to pedestrians, animals, vehicles, waterways and services above obstacles or between two points at a height above the ground. And by improving design philosophy of bridge structure we can achieves the safety, serviceability and economy. Literature survey is carried out to review various criteria which are to be considered for the analysis and design for bridge structure. Here various books and papers describing design philosophy of R.C.C bridge structure, classification of design methods and construction aspects are studied.

2.2 Literature Review

Mr.Joglekar [**1**]paper highlights the necessity and urgency of preparing new generation, rationalization codes for bridge in India, in line with international standards. This paper focus on concept of safety and reliability, provision of sound design philosophy i.e. limit state method which is needed to be improved in existing bridge code in Indian.

Dr.V.K.Raina [**2**] paper gives the bridge design education and rational approach to structural design of bridge. This paper clears the reason to adopt new design philosophy in the new code practice and split load factors.

Demetrios E.Tonias [3] intended to serve an overview of the bridge engineering process from the origin of a bridge project through his book “Design, Rehabilitation and Maintenance of Modern Highway Bridge”. This book provides theoretical aspects of design philosophy and bridge design.

Krishna N. Raju [4] describes theory and design of various types of bridge in his book “Design of Bridge”. This book is useful in solving continuous R.C.C slab bridge design by using working stress method as per the codes of the Indian Roads Congress. And shows detailed working drawing of reinforcement, plan, elevation and cross section.

D.J. Victor [5] describes theory and design of various bridge components in his book “Essentials of bridge Engineering”. This book deal with the design of R.C.C bridge having T-shaped girder under IRC loading with reinforcement detailing.

V. K. Raina [6]“Concrete bridge handbook” useful in understanding the design philosophy concepts of elastic design and load factor (limit state) design method. And his book “Analysis, Design and Economics” is useful in solving example with easy analysis techniques.

V.L. Shah and S.R Karve [7]had described limit state theory and design of R.C.structure in his book “Limit State Theory and Design of Reinforced Concrete”. This book is very useful in understanding theoretical aspect of design philosophy including limit state design method. It very useful in solving example of R.C structures such as slab and beams with reinforcement detailing as per I.S:456-2000.

Richards M.Barker [8]had described load and resistance factored design theory and design of R.C.bridge in his book “Design of highway bridges an LRFD Approach”. This book is very useful in understanding theoretical aspect of design philosophy including load and resistance factored design. It very useful in solving sample calculation of bridge superstructure.

Edward G. Nawy [9]had described fundamental appraoch of R.C structure using ACI318-05 in his book “Reinforced concrete”. This book is very useful in understanding fundamental aspect of design philosophy used by ACI318-05.

I.R.C:6-2000 "Standard Specifications and Code of Practice for Road Bridge" Section-II Load and stress [**10**] (fourth revision) is useful in application of vehicular load on the bridge structure. **I.R.C:21-2000 "Standard Specifications and Code of Practice for Road Bridge" Section-III Cement Concrete** [**11**] (third revision) is useful in application of vehicular load on the bridge structure.

I.R.C:6 draft code [**12**]according to the amendment No.8 is here notified from 1 June 2009 used for load combination for working out the stress in members using limit state design approach. **IS-456:2000 "Indian code practice for plain and reinforced concrete"** [**13**] (fourth revision) is useful for limit state design of R.C.C bridge.

Desgin Specifications-2007 by the American Association of State Highway and Transportation Officials." [**14**] (fourth edition) is useful for load and resistance factored design of R.C.C bridge.

Chapter 3

Comparison of Design Methods

3.1 General

The traditional method of design was based on the concept of allowable working load stresses, and was associated conventionally with theory of elasticity. The factor of safety, determining the allowable stresses, catered for a margin of safety. Subjective ignorance regarding the resistance of material at the limiting state failure was thus added to the objective uncertainty associated with imperfection of human observation.

3.2 Limit State Design as per IS456:2000

3.2.1 Introduction

The bridge superstructure is designed by both the methods and compared with working stress design method to find most efficient design philosophy. In the limit state design method, the structure shall be designed to withstand safety all loads likely to act throughout the life. It shall not suffer total collapse under the accidental loads, the objective of design is to achieve a structure that will remain fit for use during its life with acceptable target reliability. In other words, the probability of a limit state being reach during its life time should be very low. The acceptable limit for safety and serviceability requirement before the failure occurs is called a limit state. In general structure must be design for the critical and check for other limit state.

3.2.2 Partial safety factor

Following are the partial safety factors according to the amendment no.8 of IRC:6-2000 given in Table 3.1 ,

Table 3.1: Partial safety factors(IS-456:2000)

Load Type	**partial safety factor**	
	Ultimate strength	**Servicibilty strength**
Dead load	1.35	1
Wearing surface	1.75	1
Live load	1.5	1

3.2.3 Stress block

The stress block diagram of T-beam for finding out the neutral axis.

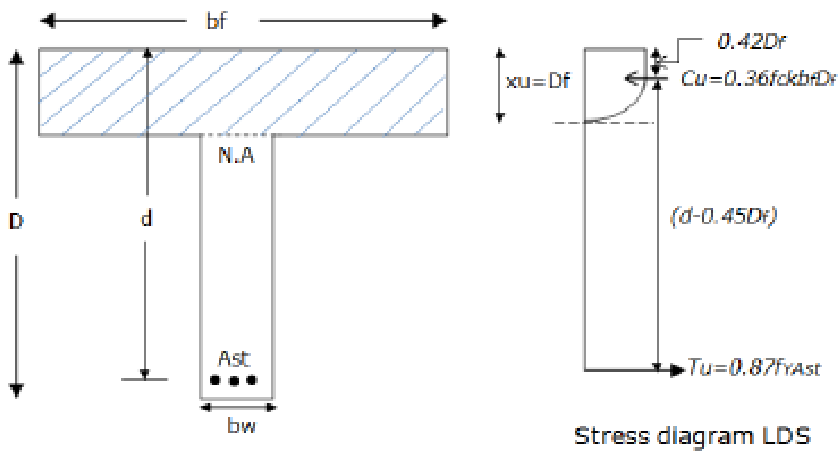

Figure 3.1: Neutral Axis Lies In Flange Stress Diagram LSD

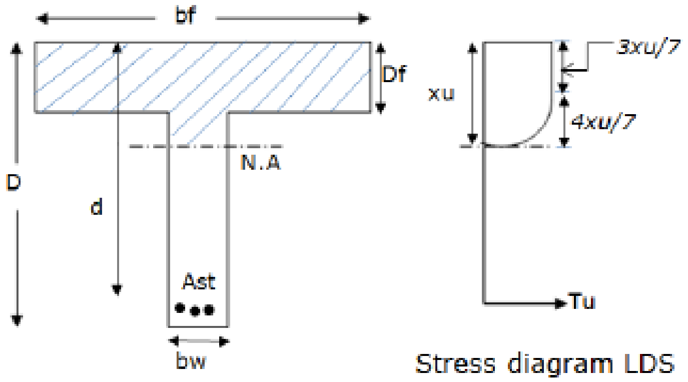

Figure 3.2: Neutral Axis Lies In Web Stress Diagram LSD

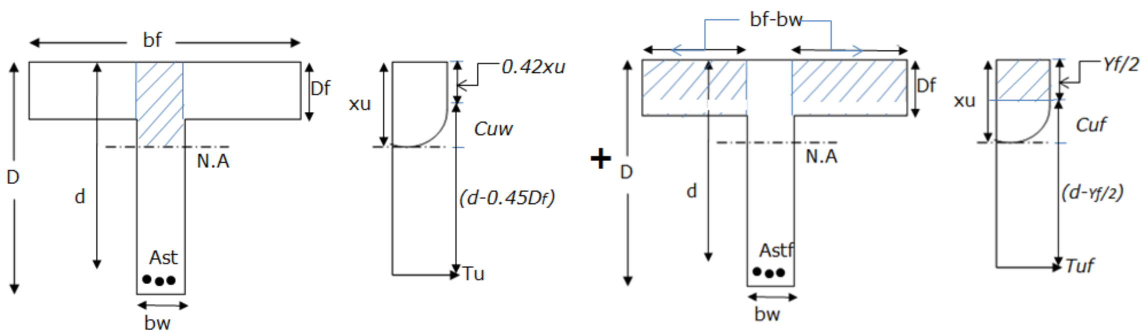

Figure 3.3: Parabolic part of stress-block inside flange (LSD)

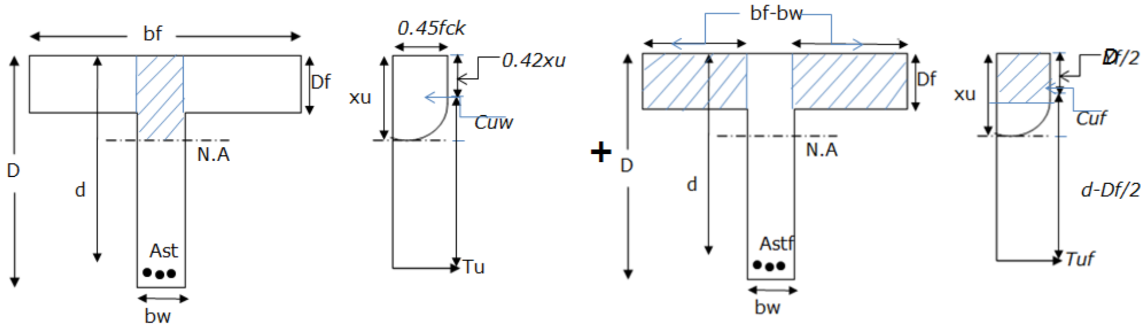

Figure 3.4: Parabolic part of stress-block lying outside flange (LSD)

3.2.4 Limit state of collapse-flexure

Case-I: When Neutral axis lie in flange i.e.(xu<Df) as shown in 3.1.

•xu can be found out from equilibrium condition C_uf=T,

$$x_u = \frac{0.87f_yA_{st}}{0.36f_{ck}b_f} \tag{3.1}$$

•Moment of resistance with respect to compression force resisted by flange = C_uf is given

$$MR = 0.36f_{ck}b_fx_u(d - 0.42x_u) \tag{3.2}$$

•Moment of resistance with respect to tensile force is given by

$$MR = 0.87f_yA_{st}(d - 0.42x_u) \tag{3.3}$$

$$A_{st} = \frac{0.5f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6M_u}{f_{ck}bd^2}}\right] \times b \times d \tag{3.4}$$

•When xu=Df, then

$$MR = 0.36f_{ck}b_fD_f(d - 0.42D_f) \tag{3.5}$$

$$A_{st} = \frac{0.36f_{ck}b_fD_f}{0.87f_y} \tag{3.6}$$

When N.A lie in web i.e. (x>Df) as shown in fig.3.2.It is divided in two part, one consist of concrete in the web portion of width bw and depth xu and other consist of projecting flanges of width (b_f-b_w) and depth Df .

Case: II(a) The depth of rectangular part of stress block is less than the depth of flange i.e. $3x_u/7 < D_f$ or x_u <$7D_f/3$ Df, as shown in 3.2., x_u can be found out from

$$x_u = \frac{0.87f_yA_{st} - 0.45 \times 0.65f_{ck}D_f(b_f - b_w)}{0.36f_{ck}b_w + 0.45 \times 0.15f_{ck}(b_f - b_w)} \tag{3.7}$$

$$M_{ur} = 0.36 f_c k b_w x_u (d - 0.42 x_u) + 0.45 f_{ck} (b_f - b_w) b_f (d - \frac{y_f}{2}) \tag{3.8}$$

$$A_{st} = \frac{0.36 f_{ck} b_w x_u + 0.45 f_c k (b_f - b_w) y_f}{0.87 f_y} \tag{3.9}$$

Where,

$$y_t = (0.15 x_u + 0.65 D_f) \tag{3.10}$$

Case-II(b): The depth of rectangular part of stress block greater than the depth of flange i.e. $3x_u/7 >$ Df or $x_u > 7D_f/3$, as shown in 3.3., xu can be found out from

$$x_u = \frac{0.87 f_y A_{st} - 0.45 \times 0.65 f_{ck} D_f (b_f - b_w)}{0.36 f_{ck} b_w} \tag{3.11}$$

$$M_{ur} = 0.36 f_c k b_w x_u (d - 0.42 x_u) + 0.45 f_{ck} (b_f - b_w) b_f (d - D_f) \tag{3.12}$$

$$A_{st} = \frac{0.36 f_{ck} b_w x_u + 0.45 f_c k (b_f - b_w) D_f}{0.87 f_y} \tag{3.13}$$

3.2.5 Limit state of collapse- shear

Generally, shear failure in reality occurs under the combined action of shearing forces and bending moments, is characterized by very small deflection and lack of ductility .This failure many times sudden and without any warning. For this reason the shear failure is considered very undesirable and is usually avoided. Nominal shear stress is,

$$\tau_v = \frac{V_u}{bd} \tag{3.14}$$

For solid slab shear strength of concrete shall be τ_{uc}k, k taken from (cl:40.2.1.1), nominal shear stress not exceed the half the values given in table-20 of IS-456:2000. In case of beam, the external shear ultimate state is jointly carried by the concrete and web steel, and equilibrium equation written as:

$$V_u = V_{uc} + V_{us} \tag{3.15}$$

where,

$$V_{uc} = \tau_{uc} bD \tag{3.16}$$

$$V_{us} = 0.4bD \tag{3.17}$$

If $V_u c \geq V_u$, then no need to provide shear reinforcement.

3.2.6 Limit state of serviceability

Limit state state philosophy of design considers the performance of a structure or rather the fitness of structures to serve the desired function satisfactorily, i.e the serviceability of the structure, as one of the important criteria of structural design besides safety, economy and durability.

3.2.7 Limit State of Deflection

There are two types of deflection are:

a. Short-term deflection: this is due to initial elastic deformation of the member due to load and permanent imposed load under service condition.
   Moment of inertia of gross cross-section (Igr) ignoring reinforcement, for flanged beam section (gross uncracked section):

$$x = \frac{b_w D/2 + (b_f - b_w)D_f^2/2}{b_w D + (b_f - b_w)D_f} \tag{3.18}$$

$$I_{gr} = \frac{b_f x^3}{3} - \frac{(b_f - b_w)(x - D_f^2)^3}{3} - \frac{b_w (D - x)^3}{3} \tag{3.19}$$

Moment of inertia of gross cross-section (Ir) ignoring reinforcement, for flanged beam section (cracked section):

When neutral axis lies in web,

$$b_f D_f (x - \frac{D_f}{2}) = m A_{st} \, (d - x) \tag{3.20}$$

$$x = \frac{b_f D_f^2/2 + m A_{st} d}{b_f D_f + m A_{st}} \tag{3.21}$$

$$I_r = \frac{b_f x^3}{3} - \frac{(b_f - b_w)(x - D_f)^3}{3} + m A_{st}(d - x)^2 \tag{3.22}$$

When neutral axis lies in flange,

$$I_r = \frac{b_f x^3}{3} + m A_{st}(d - x)^2 \tag{3.23}$$

Effective moment of inertia of flanged beam section (Ieff):

For simply supported beams and cantilevers,

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M} \frac{z}{d} \left(1 - \frac{x}{d}\right) \frac{b_w}{b}} \tag{3.24}$$

b. Long term deflection: long-term deflection is caused due to creep and shrinkage under sustained load and additional short-term deflection due to temporary live loads.

Deflection due to shrinkage:

$$a_{c3} = k_3 \psi_{cs} l^2 \tag{3.25}$$

Where,

$$\psi_{cs} = k_4 \frac{\varepsilon_{cs}}{D} \tag{3.26}$$

$$k_4 = 0.72 \frac{p_t - p_c}{\sqrt{p_t}} \leq 1 for 0.25 \leq P_t - P_c < 1 \tag{3.27}$$

or

$$k_4 = 0.65 \frac{p_t - p_c}{\sqrt{p_t}} \leq 1 \tag{3.28}$$

Deflection due to creep:

$$a_{cc(perm)} = a_{icc(perm)} - a_{i(perm)} \tag{3.29}$$

Permissible deflection:

For dead load,$\Delta_{allowable}$= L/350

For live load,$\Delta_{allowable}$= L/800

For total load,$\Delta_{allowable}$= L/600

3.2.8 Limit State Cracking

To check the width of cracking:

$$W_{cr} = \frac{3 a_{cr} \varepsilon_m}{1 + 2\left(\frac{a_{cr} - C_{\min}}{D - x}\right)} \tag{3.30}$$

$$\varepsilon_m = \varepsilon_1 - \frac{b(D - x)(a - x)}{3 E_S A_{st}(d - x)} \tag{3.31}$$

Permissible deflection:

The total load taken to check crack width D.L+50% L.L.

For modrate enviormental condition = 0.3 mm

For severe enviormental condition = 0.2 mm

3.3 Load and resistance factor design as per AASTHO specification

LRFD is a method of proportioning structures such that no applicable limit state is exceeded when the structure is subjected to all appropriate design load combinations. The proposed LRFD method provides reasonably accurate solution approach. In addition, LRFD basic principle is same as limit state design method ,the difference will be better calibration of the load and resistance factors, a more reasonable combination of various load effects and a better use of the strength of the total structure. This considers the design life of various materials or various types of structures so that paying a little higher price during construction to get more service life can be justified. LSD basically the same as the LRFD being proposed, but the term is more descriptive since the method includes criteria for performance under both serviceably limit state and ultimate limit state. The basic design expression in the AASHTO (2007) LRFD Bridge Specification that must be satisfied for all limit states, both global and local, is given as:

$$\phi R_n \geq \sum \eta_i \gamma_i Q_i \tag{3.32}$$

where Q_i, is the force effect. R_n is the nominal resistance, γ is the statistically based load factor applied to the force effects, ϕ is the statistically based Resistance factor applied to nominal resistance, and Q_i, is a load modification factor. For all nonstrcngth limit states, = 1 .0.

The load modifierη is a factor that takes into account the ductility, redundancy and operational importance of the bridge. It is given for loads for which a maximum value of γ is appropriate by:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \tag{3.33}$$

where, $\eta_D$is the ductility factor, $\eta_R$is the redundancy factor,and η_I is the operation importance factor. The first two factors referees to the consequence of a bridge being out of service. For all nonsrength limit states, η_D=η_R=1.

Ductility is important to the safety of a bridge. If ductility is present, over loaded portions of the structure can redistribute the load to other portions that have reserve strength. This redistribution is dependent on the ability of the overloaded component and its connections to develop iiie1tstic deformations without failure.

The value to be used for the strength limit state ductility factor are:
$\eta_D \geq 1.05$ for nonductile components
$\eta_D = 1$ for conventional designs and all other limit state
$\eta_D = 1$for components for addition ductilty

Redundancy significantly affects the safety margin of a bridge structure. A statically indeterminate structure is redundant, that is, has more restraints than are necessary to satisfy equilibrium.
$\eta_R \geq 1.05$ for nonredundant member
$\eta_R = 1$ for conventional levels of redundancy
$\eta_R \geq 0.95$ for expectional redundancy

Bridges can be considered of operational importance if they are on the shortest path between residential areas and a hospital or school or pro vide access for police, fire, and rescue vehicles to homes, businesses, and

industrial plants. In the event of an earthquake, it is important that all lifelines, such as bridges, remain open. Therefore, the following requirements apply to the extreme event limit state as well as to the strength limit state: $\eta_I \geq 1.05$ for a bridge of opertional importance

$\eta_I = 1$ for typical bridge

$\eta_I \geq 0.95$forrelatively less important bridges

3.3.1 Partial safety factors

Following are the partial safety factors according to the ASSTHO specification given in Table 3.2 ,

Table 3.2: Partial safety factors(AASTHO

Load Type	partial safety factor	
	Ultimate strength	**Servicibilty strength**
Dead load	1.25	1
Wearing surface	1.5	1
Live load	1.75	0.75

3.3.2 Stress block

The section has to be considered as a T-section for findng out neutral axis,

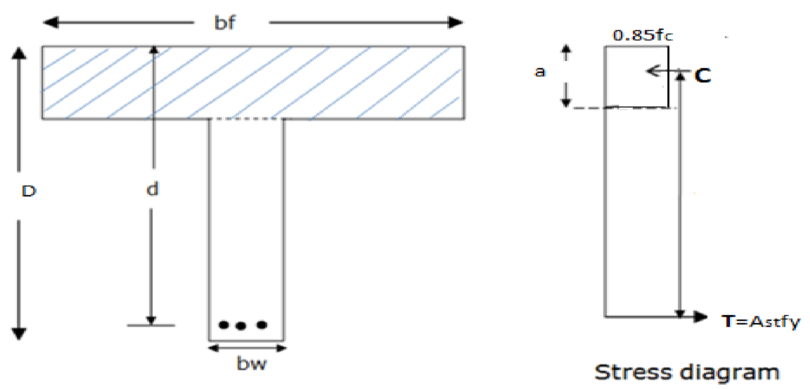

Figure 3.5: Neutral axis inside flange LRFD

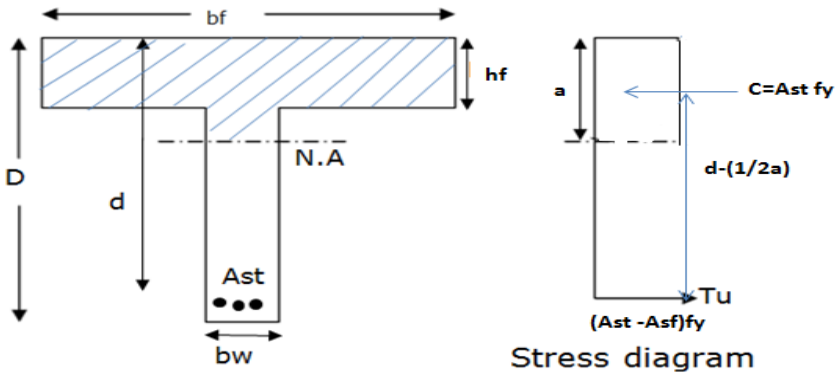

Figure 3.6: Neutral axis outside flange LRFD

3.3.3 LRFD - Flexure Limit State

Case-I: Depth of neutral axis less than flange hf,

This case can be treated similarly to the standard rectangular section provided that the depth a of the equivalent rectangular block is less than the flange thickness. The flange width bf of the compression side should be used as the beam width analysis. Force equilibrium where C=T,

$$0.85 f_c b a = A_s f_y \tag{3.34}$$

$$a = \frac{A_s f_y}{0.85 f_c b} \tag{3.35}$$

The nominal moment strength would thus be,

$$M_n = A_{sf} f_y (d - a/2) \tag{3.36}$$

Since the force contribution in tension zone is neglected. It does not matter whether part of the flange is in the tension.

Case - II: Depth of neutral axis larger than flange hf, In this case, a>hf the depth of the equivalent rectangular stress block a could be smaller or larger than the flange thickness hf. This type of T-beam (a>hf) can be treated in a manner similar to that for doubly reinforced rectangular cross section.Force equilibrium where C=T,

$$0.85f_c(b - b_w)h_f = (A_{sf}f_y) \tag{3.37}$$

where, Asf is an imaginary compressive steel area whose force capacity is equivalent to the force capacity of the compressive flange overhange. Consequently, an equilvelent area Asf of compression reinforcement to develop the overhange flange would have a value of

$$A_sf = \frac{0.85f_c(b - b_w)h_f}{f_y} \tag{3.38}$$

For a beam to be considered as a real T-beam , the tension forc Asfy generated by the steel should be greater than the compression force capacity of the total flange area

$$a = \frac{A_{st}A_{sf}f_y}{0.85f_cb_w} \tag{3.39}$$

The nominal moment strength would thus be,

$$M_n = ((A_{st} - A_{sf})f_y(d - a/2)) + (A_{sf}f_y(d - 0.5h_f)) \tag{3.40}$$

3.3.4 LRFD-Shear Limit State

Nominal shear stress v is divded by f_c to obtain the ratio of v/f_c,this ratio is higher than 0.25,larger cross section required. The nominal shear stress is

$$v = \frac{V_u}{\phi_n b_v d_v} \tag{3.41}$$

Total shear strength of concrete and steel in web portion,

$$V_n = V_c + V_s \tag{3.42}$$

$$V_c = 0.083\beta\sqrt{f_c}b_v d_v \tag{3.43}$$

$$V_s = \frac{V_u}{\phi_v} - 0.083\beta\sqrt{f_c}b_v d_v \tag{3.44}$$

or,

$$V_s = \frac{A_v f_y d_v}{S} \tag{3.45}$$

If$V_u \geq 0.5\phi V_c$ no need to provide shear reinforcement.

3.3.5 LRFD-Service Limit State

The service limit state shall be taken as restrictions on deformation(deflection) and crack width under regular service conditions.

3.3.6 Deformation

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g \tag{3.46}$$

$$I_{cr} = \frac{1}{3}bx^3 + nA_s(d-x)^2 \tag{3.47}$$

$$M_{cr} = f_r\frac{I_g}{y_t} \tag{3.48}$$

Permissible deflection:

For dead load,$\Delta_{allowable}$= L/350

For live load,$\Delta_{allowable}$= L/800

For total load,$\Delta_{allowable}$= L/600

Cracking control

$$f \leq f_{sa} = \frac{Z}{(d_c A)^{1/3}} \tag{3.49}$$

where,

f_c = concrete cover,

Z = crack width parameter value given in table 3.3

A = area of concrete $f \leq 0.6 f_y$, then the crack width is under the permissible limit.

Table 3.3: Crack width parameter

Exposer condition	Z(N/mm)	Crack width
Moderate	30000	0.41
Severe	23000	0.3
Buried structure	17000	0.23

Chapter 4

Bridge Superstructure Analysis

4.1 General

A R.C.C flyover superstructure section is taken here to analyze using manual calculations and SAP software to find the maximum bending moment and shear force.

4.2 Structural Data

In this study the cross section taken for analysis is shown in fig.4.1 and fig.4.2

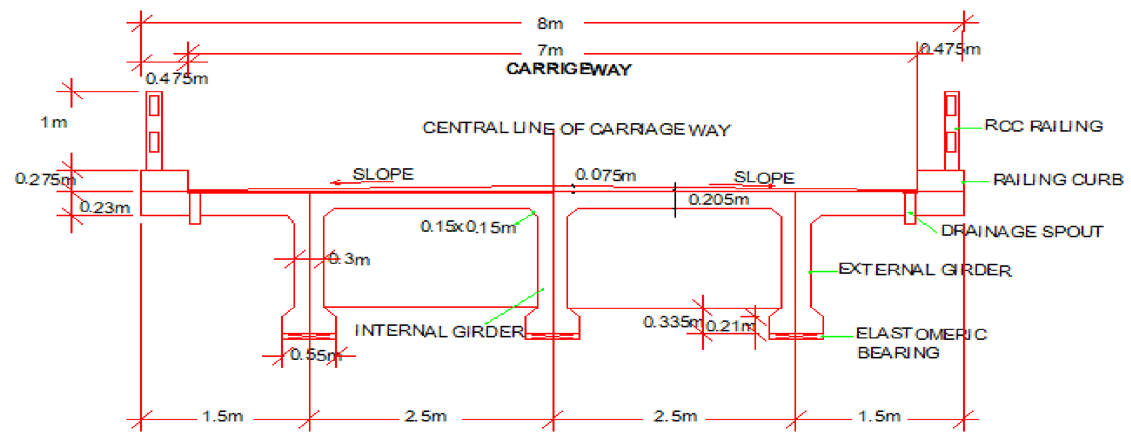

Figure 4.1: Cross Section of Bridge Deck Slab

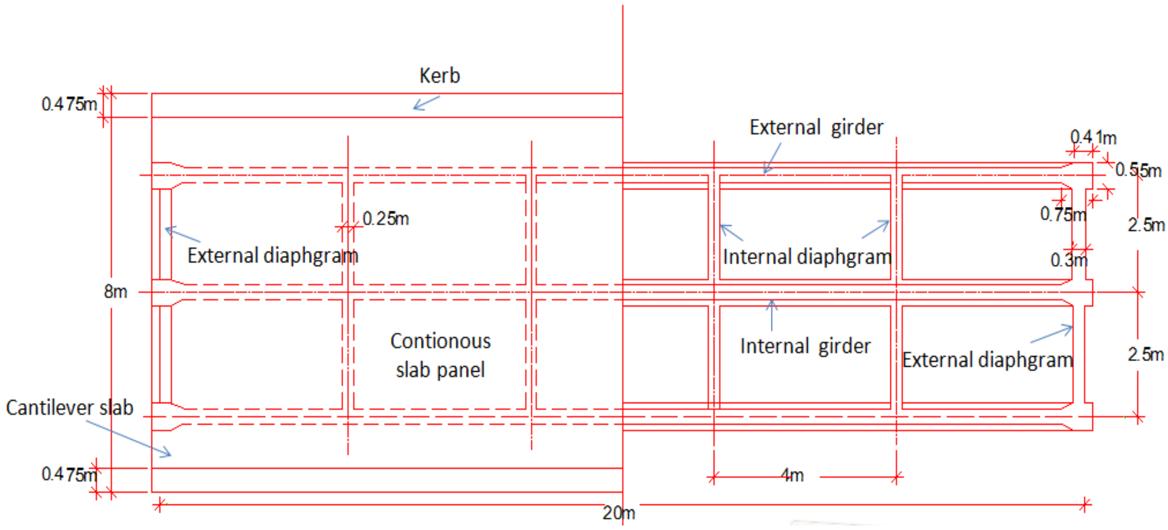

Figure 4.2: Longitudinal Bridge Cross Section

Effective span of Bridge	=	20 m
c/c distance of cross-girder	=	4 m
No. of diaphgram	=	6
Width of carriageway	=	7.5 m
Length of cantilever portion	=	1.35 m
RCC Grade	=	M30
Grade of reinforcement	=	Fe415
Unit weight of RCC	=	24 kN/m^3
Unit weight of wearing coat	=	22 kN/m^3
Unit weight of structural steel	=	78.5 kN/m^3
Width of bridge	=	8 m
No. of longitudinal girder	=	3
c/c distance of longitudinal-girder	=	2.5 m
Width of Curb	=	475 mm
Width of parapet	=	250 mm
Height of curb	=	200 mm
Height of parapet	=	1000 mm
Thickness of intermediate Deck Slab	=	205 mm
Thickness of cantilever Deck Slab	=	230 mm

4.3 IRC Loading

In order to analyze the bridge to these moving loads, IRC-6 (loads and stresses) recommends certain standard hypothetical loading systems. The bridge is then designed for the maximum response values under these standard loads. The live load which gives maximum bending moment and shear force at different position and which are used for analysis is shown in fig.4.3, fig.4.4 and fig.4.5.

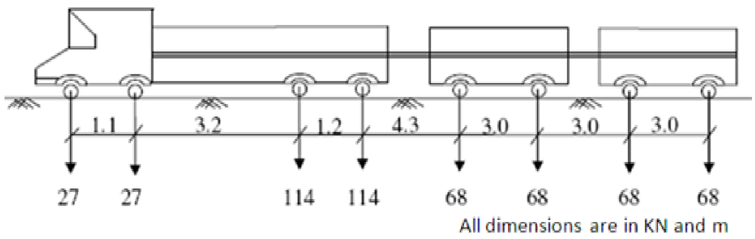

Figure 4.3: Class A Wheeled Vehicle

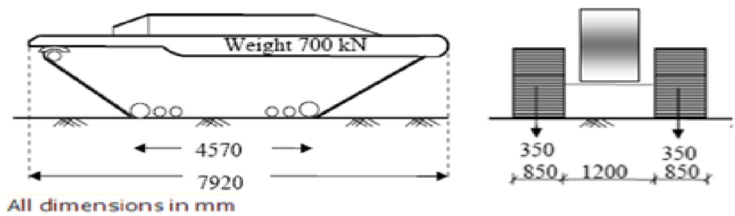

Figure 4.4: Class 70R Tracked Vehicle

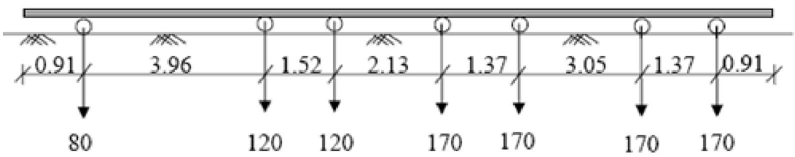

Figure 4.5: Class 70R Wheeled Vehicle

4.4 Slab Analysis

The deck slab is analyzed for dead loads and live loads. Mostly class A wheeled for cantilever portion and class 70R tracked vehicle on two way spanning slab gives critical forces as it is heaviest vehicle.The moments in the two directions can be computed by using the design curves developed by M. Pigeaud.

4.4.1 Cantilever slab:

Dead Load calculation

Table 4.1: Cantilever Slab Dead Load Moment

Components	**D.L m2**	**C.G m**	**Moment KN.m**	**S.F KN**
Slab	7.452	0.675	5.03	
Wearing coat(W.C)	2.228	1.35	3.01	
Parapet	5.28	1.35	1.35	
Kerb	3.135	1.11	4.7	
Total			14.09	20.42

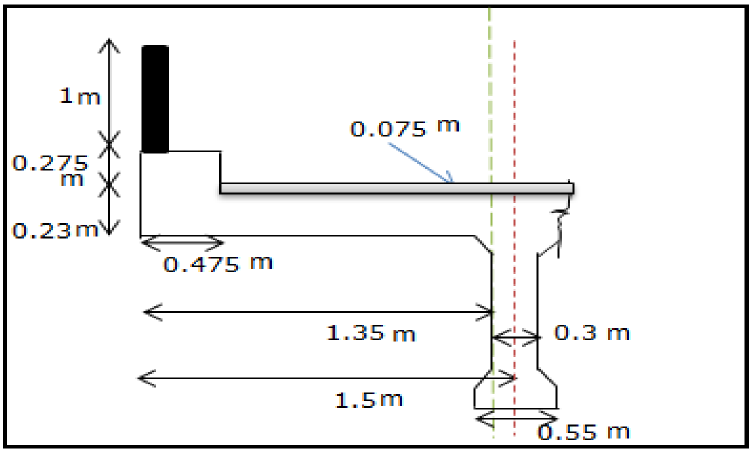

Figure 4.6: Cantilever Slab

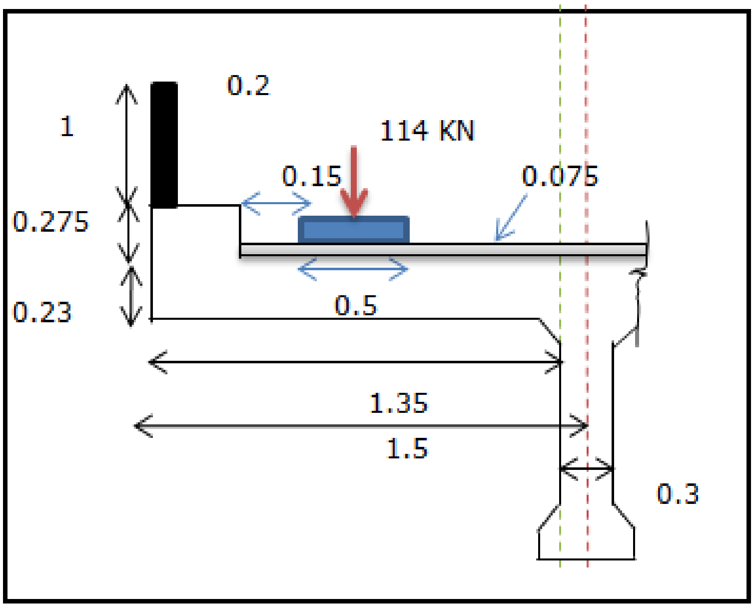

Figure 4.7: Class A Heaviest Wheel Near To Minimum Distance from Kerb

Live Load calculation

On the basis of the criteria of minimum clearance from the kerb.Class-A two wheel live load will be critical on the cantilever portion of the deck slab.

Placing wheel of 114KN axle of Class-A @ 0.15m from kerb.

Effective width, beff	=	1.2*a + b1
where, a	=	1.35-[0.2+0.15+0.25]
a	=	0.475 m
Width of wheel along span	=	0.25 m
b1	=	0.25+(2*0.075)
	=	0.4 m
beff across the span	=	1.2*0.48+0.4
	=	0.97 m <1.2m
beff along span	=	0.5+2*(0.23+0.075)
	=	1.11 m <1.8m

Load intensity	=	57*1.53/0.97
	=	90 kN/m
B.M at the face of support	=	47.45 kN.m
S.F at the face of support	=	74 kN
Total Moment	=	62 kN.m
Total shear force	=	94 kN

4.4.2 Continuous slab

Placing one track at the center of the panel to get maximum bending moment. Here, the slab panel of size (4-0.25) x (2.5-0.3) i.e. 3.75mx2.2m.Here L/B ratio is 1.7 which is less than 2, therefore slab is spanning in two direction.

Dead load calculation:

Dead Load Calculation:

Self-weight = 0.205 * 24	=	4.9 kN/m^2
Weight of wearing coat = 0.075 * 22	=	2 kN/m^2
Total	=	6.92 kN/m^2
Total W = 6.92 * 4 * 2.5	=	69.2 kN
from Pigeaud's curve		
K =1.6 , 1/K=0.63		
m1	=	0.028
m2	=	0.042
M_B =W x (m_1 + 0.15 m_2)	=	2.37 kNm
M_L =W x (m_2 + 0.15 m_1)	=	3.19 kNm
Design BM including continuity factor		
MB = 2.46x0.8	=	1.86 kNm
ML = 3.32x0.8	=	2.55 kNm

Live load calculation:

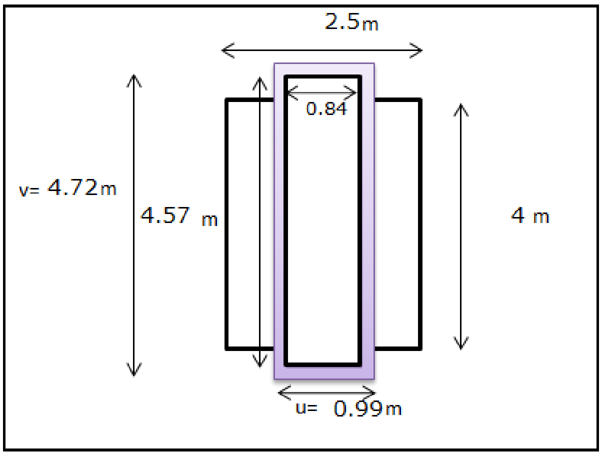

Figure 4.8: Concentrated Load on the Slab Panel

Class-70R Tracked Vehicle :

Live load is class 70R tracked vehicle. One wheel is placed at the centre of panel as shown in fig.

W	=	350 kN		
u	=	0.85 + 2 * 0.075	=	1 m
v	=	4.57 + 2 * 0.075	=	4.72 m
(u/B)	=	1/2.5	=	0.4
(v/L)	=	4.57/4	=	1.25
(B/L)	=	2.5/4	=	0.67

Rferring to Pigeaud's curves (refer fig.)

$m_1 =$ 0.085

$m_2 =$ 0.048

Short span moment M_B = W x (m_1 + 0.15 m_2)

= 32.27 kNm

Long span moment M_L = W x (m_2 + 0.15m_1)

= 21.27 kNm

Design BM including continuity and impact factor :

M_B	=	32.27 x 0.8 x (1 + 0.25)	=	32.27 kNm
M_L	=	21.27 x 0.8 x (1 + 0.25)	=	21.27 kNm

Total design bending moment

M_B = 1.96 + 32.27 = 34.3 kNm

M_L = 2.65 + 21.17 = 23.9 kNm

Dead Load shear = 3.5 kN

For L.L load shear ,

Dispersion in direction of span = 0.84+2x(0.075+0.205)

= 1.4 m

The load is kept at = 0.7 m from edge of beam

Effective width of panel,b_eff = Kx(1-x/L)+b_w

B/L =1.6

K =2.48

Effective width = 6.2 m

Load per m width = 85.34 kN

Shear force = 109.7 kN

Shear force with impact = 137.15 kN

Total S.F = 141 kN

Table 4.2: Slab bending moment and shear force

Cantilever slab						
Thickness	Loading	Unfactored		Factored		Remark
(m)		B.M(kNm)	S.F(kN)	B.M(kNm)	S.F(kN)	
0.23	at face	62	94	89	138	LSD
0.26	at face	59	90	95	146	LRFD
Continous slab panel						
Thickness	Loading	Unfactored		Factored		Remark
(m)		B.M(kNm)	S.F(kN)	B.M(kNm)	S.F(kN)	
0.205	$L_{shorter}$	34.23	94	51	141	LSD
	L_{longer}	23.93		36		
0.235	$L_{shorter}$	35	95	60	165	LRFD
	L_{longer}	24		40		

4.5 Diaphragm Analysis

Diaphragm is analyzed as intermediate and external diaphragm. Intermediate diaphragm is analyzed for dead load and Class 70R tracked vehicular load while as external diaphragm is analyzed for jack force for lifting the super structure for replacement of bearing.

4.5.1 Internal diaphragm

Dead load calculation: As shown in fig 4.8 slab load transfer to cross beam and total dead load on girder is shown in fig 4.9.

Internal diaphragm

Dimensions:

Length = 2.5m

Width = 0.25m

Depth =1.165m

self-wt. of cross girder	=	0.25x0.96x2.4
	=	5.76 kN/m
Dead load from slab wt.	=	2x1/2x2.5x1.25x0.205x24
	=	15.38 kN
Uniformly distributed	=	15.4/2.5
	=	6.15 kN/m
total dead load on cross girder	=	5.76+6.1
	=	11.91 kN/m
Reaction on each cross girder	=	13.14x5/3
	=	21.9 kN

Live load calculation:

For maximum B.M class -70R tracked vehicle on cross girder as shown in fig4.9

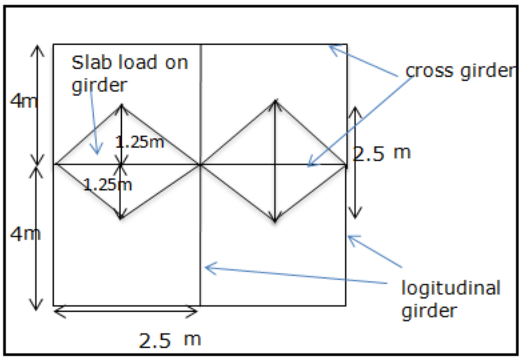

Figure 4.9: Dead Load On Cross Girder

load coming on diaphragm is	=	(350x(4-1.143)/4)
	=	250kN
Reaction on longitudinal girder	=	166.7kN
Max. B.M. in cross girder under load	=	190.5 kNm
load coming on diaphragm including impact factor	=	238.11 kN
Dead load B.M	=	16.45 kNm
Live load B.M	=	238 kNm
Total B.M	=	254 kNm
Dead load S.F	=	21.9 kN
Live load S.F	=	198.4kN
Total S.F	=	220.3 kNm

4.5.2 External diaphragm

Dead load calculation:

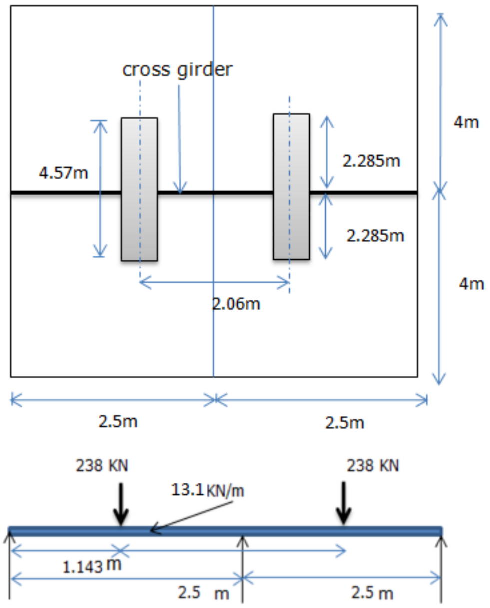

Figure 4.10: Position of Vehicle for Maximum B.M in Cross Girder

Dimensions:

Length	=	2.5 m
Width	=	0.25 m
Depth	=	1.165 m
self-wt.	=	209.7 kN
slab wt.	=	533.9 kN
w.c.wt.	=	141 kN
Railing kerb	=	314.4 kN
Girder weight	=	671.4 kN
Total Dead load	=	1661 kN

Total load of superstructure on one side = 1043 kN

Live Load calculation:

Total 8 jacks per end will be used to lift the superstructure i.e.2 outside and 2 inside between the girders for design consider worst case of girder at centre of 2 jecks.

force per jeck = 130 kN

load per girder = 260 kN

Max. B.M=wl/4 = 325 kNm

Max. S.F=wl/2 = 650 kN

Table 4.3: Diaphgram bending moment and shear force

External diaphgram						
Thickness	Depth	Unfactored		Factored		Remark
(m)		B.M(kNm)	S.F(kN)	B.M(kNm)	S.F(kN)	
0.25	1.165	325	650	447	895	LSD
0.3	1.165	338	676	408	826	LRFD
Internal diaphgram						
Thickness	Loading	Unfactored		Factored		Remark
(m)		B.M(kNm)	S.F(kN)	B.M(kNm)	S.F(kN)	
0.25	1.165	254	220	377	324	LSD
0.3	1.165	272	237	468	402	LRFD

4.6 Girder Analysis

In order to simplify the computation of load distribution some rational methods like, Courbon's method, Guyon Massonet method, Hendry Jaegar method are used. IRC prefers Courbon's method, which is simple easily calculated manually. And using SAP2000 software both can be verified.

4.6.1 Courbon's method:

Courbon's method is the simplest method to calculate live load distribution factor. when the live load are positioned nearer to the kerb as shown in fig. 4.10, the centre of gravity of live load acts eccentrically with the centre of gravity of the girder system. Due to this eccentricity, the load shared by each girder is increased or decreased depending upon the position of the girders.Forces are distributed by reaction factor.

Reaction factor is calculated for position of vehicle for maximum force in all girders. The reaction factor is given by equation 4.2.

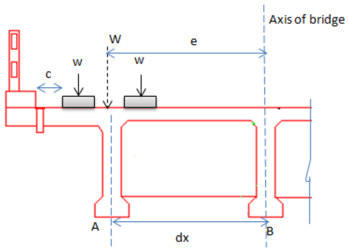

Figure 4.11: Position of Live Load for Maximum Moment in Girder 'A'

$$R_x = \left(\sum \frac{W}{n}\right)\left[1 + \left(\frac{\sum I}{\sum dx^2 \times I}\right) dx \times e\right] \tag{4.1}$$

Courbon's method can be used when the following conditions are satisfied,

a. Courbon's method can be used when the following conditions are satisfied,

b. The ratio of span to width of deck is greater than 2 but less than 4.

c. The longitudinal girders are interconnected by at least five symmetrically spaced cross girders.

d. The cross girder extends to a depth of at least 0.75 times the depth of the longitudinal girders.

4.6.2 SAP 2000:

SAP 2000 is very efficient finite element based powerful tool to analyses any kind of structure for different loading. It is user-friendly software with graphical interface. The analysis is done in SAP2000 using bridge modular and inputting the bridge dimensions in it and bridge model is shown in fig.4.12.

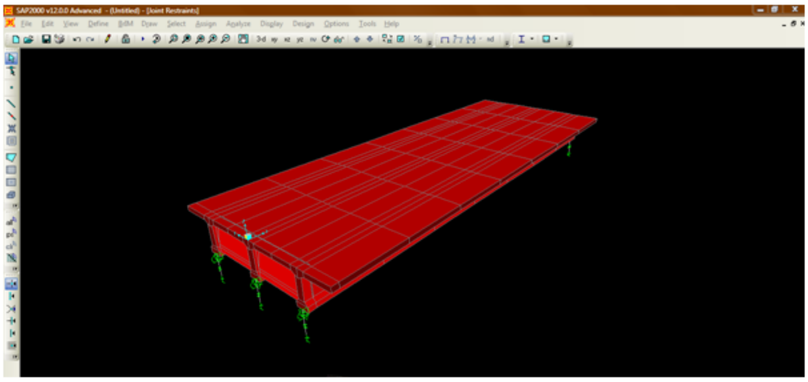

Figure 4.12: SAP2000 Bridge Model

4.6.3 Live load position

The analysis is done for Deal Load, Super Imposed Dead Load and in vehicle load all IRC vehicle . To get maximum forces in external girder, vehicle is placed at minimum distance (‘c’) provided in IRC6-2000 clause 207.1.3, from the kerb. The position of vehicle is shown in fig 4.13.

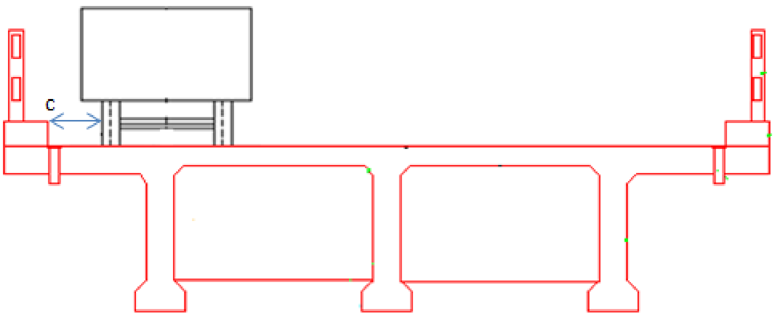

Figure 4.13: Class-70R At Minimum Distance From Kerb For External Girder

The maximum bending moment obtain center of girder, which is due to IRC 70 R vehicle. The position of vehicle is as shown in fig.4.14.

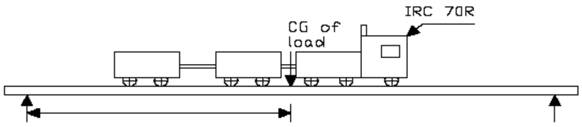

Figure 4.14: Position of Class-70R Vehicle For Max. Bending Moment

The maximum shear forces obtain at support of girder, which is due to IRC 70 R vehicle. The position of vehicle is as shown in fig.4.15.

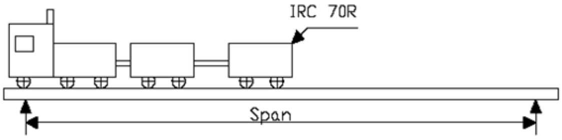

Figure 4.15: Position of Class-70r Vehicle for Max. Shear Force

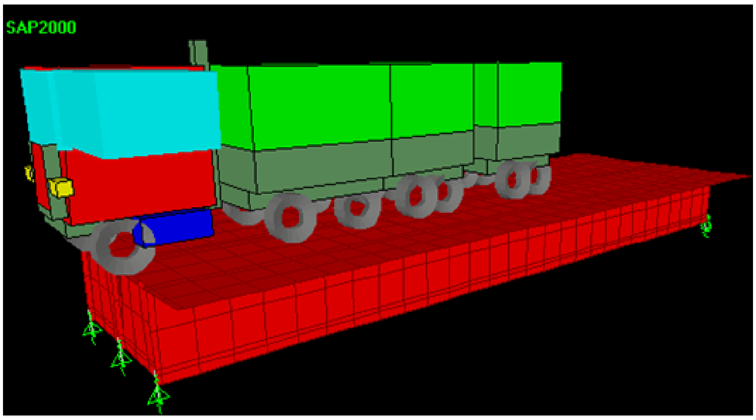

Figure 4.16: IRC Class-70R Vehicle on Moving Bridge Deck

4.6.4 Girder Force

The design force for girder is obtained from results of forces for individual girder. The maximum bending moment and shear force from all vehicle load and dead load among all girder is used for designing all girders. The results obtained from SAP 2000 and courbons method, for maximum forces at different sections out of all girders are recapitulated in Table. And maximum design forces are computed in table. NOTE:Units in table for bending moment in KN.m and shear force in KN

Table 4.4: Comparsion of bending moment

Total bending moment(KN.m)					
Section	**Length**	**External girder**		**Internal girder**	
	(m)	**Courbon's method**	**SAP2000**	**Courbon's method**	**SAP2000**
0L	0	0	0	0	0
0.1L	2	1537	1436	1085	1014
0.25L	5	3497	3268	2400	2243
0.3L	6	3974	3714	2715	2537
0.4L	8	4650	4346	3162	2955
0.5L	10	4918	4596	3322	3105

Table 4.5: Comparsion of shear force

Total shear force(KN)					
Section	**Length**	**External girder**		**Internal girder**	
	(m)	**Courbon's method**	**SAP2000**	**Courbon's method**	**SAP2000**
0L	0	0	0	595.99	557
0.1L	2	1084.98	1014	485.78	454
0.25L	5	2400.01	2243	374.5	350
0.3L	6	2714.59	2537	291.04	272
0.4L	8	3161.85	2955	172.27	161
0.5L	10	3322.35	3105	14.98	14

Table 4.6: Unfactored moment and forces on external girder(SAP2000)

External girder							
		D.L		**W.C.L**		**L.L(Class70-R)**	
Section	**L(m)**	**B.M**	**S.F**	**B.M**	**S.F**	**B.M**	**S.F**
0L	0	0	244	0	34	0	445
0.1L	2	538	191	62	27	836	377
0.25L	5	1170	122	129	17	1969	377
0.3L	6	1309	99	144	14	2261	276
0.4L	8	1502	54	165	7	2679	174
0.5L	10	1562	0	172	0	2862	31

Table 4.7: Unfactored moment and forces on internal girder (SAP2000)

Internal girder							
		D.L		**W.C.L**		**L.L(Class70-R)**	
Section	**L(m)**	**B.M**	**S.F**	**B.M**	**S.F**	**B.M**	**S.F**
0L	0	0	319	0	34	0	204
0.1L	2	569	254	62	27	383	173
0.25L	5	1212	160	129	17	902	173
0.3L	6	1357	132	144	14	1036	126
0.4L	8	1562	74	165	7	1228	80
0.5L	10	1621	0	172	0	1312	14

Table 4.8: Factored moment and forces on external girder for LSD

External girder							
		1.35*D.L		**1.75*W.C.L**		**1.5*L.L(Class70-R)**	
Section	**L(m)**	**B.M**	**S.F**	**B.M**	**S.F**	**B.M**	**S.F**
0L	0	0	329	0	60	0	668
0.1L	2	726	257	108	48	1254	566
0.25L	5	1580	165	225	30	2953	566
0.3L	6	1767	134	252	24	3391	414
0.4L	8	2028	73	288	12	4018	262
0.5L	10	2108	0	300	0	4293	46

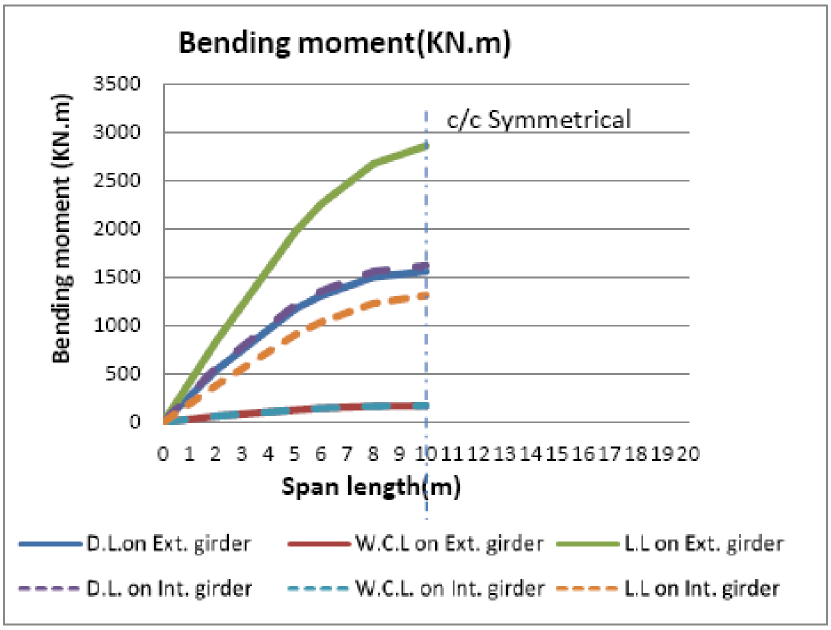

Figure 4.17: Variation in B.M of various load along longitudinal span

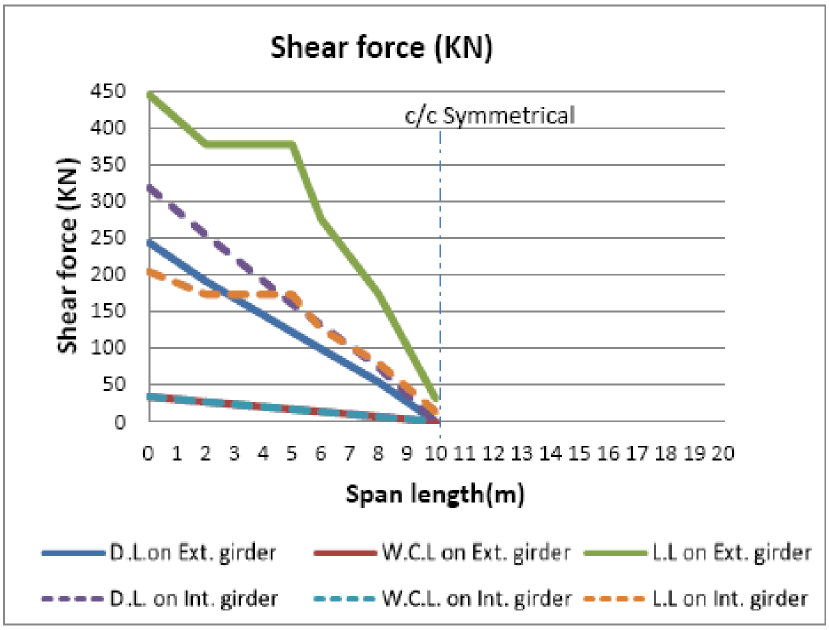

Figure 4.18: Variation in S.F of various load along longitudinal span

Table 4.9: Factored moment and forces on internal girder for LSD

Internal girder							
		1.35*D.L		**1.75*W.C.L**		**1.5*L.L(Class70-R)**	
Section	**L(m)**	**B.M**	**S.F**	**B.M**	**S.F**	**B.M**	**S.F**
0L	0	0	430	0	60	0	306
0.1L	2	768	343	108	48	575	260
0.25L	5	1636	216	225	30	1353	260
0.3L	6	1831	178	252	24	1554	190
0.4L	8	2109	100	288	12	1842	120
0.5L	10	2188	0	300	0	1968	21

Table 4.10: Factored moment and forces on external girder for LRFD

External girder							
		1.25*D.L		**1.5*W.C.L**		**1.75*L.L(Class70-R)**	
Section	**L(m)**	**B.M**	**S.F**	**B.M**	**S.F**	**B.M**	**S.F**
0L	0	0	305	0	51	0	756.5
0.1L	2	672.5	238.8	93	40.5	1421.2	640.9
0.25L	5	1463	152.5	193.5	25.5	3347.3	640.9
0.3L	6	1636	123.8	216	21	3843.7	469.2
0.4L	8	1878	67.5	247.5	10.5	4554.3	295.8
0.5L	10	1953	0	258	0	4865.4	52.7

Table 4.11: Factored moment and forces on internal girder for LRFD

Internal girder							
		1.25*D.L		**1.5*W.C.L**		**1.75*L.L(Class70-R)**	
Section	**L(m)**	**B.M**	**S.F**	**B.M**	**S.F**	**B.M**	**S.F**
0L	0	0	398.8	0	51	0	346.8
0.1L	2	711.3	317.5	93	40.5	651.1	294.1
0.25L	5	1515	200	193.5	25.5	1533.4	294.1
0.3L	6	1696	165	216	21	1761.2	214.2
0.4L	8	1953	92.5	247.5	10.5	2087.6	136
0.5L	10	2026	0	258	0	2230.4	23.8

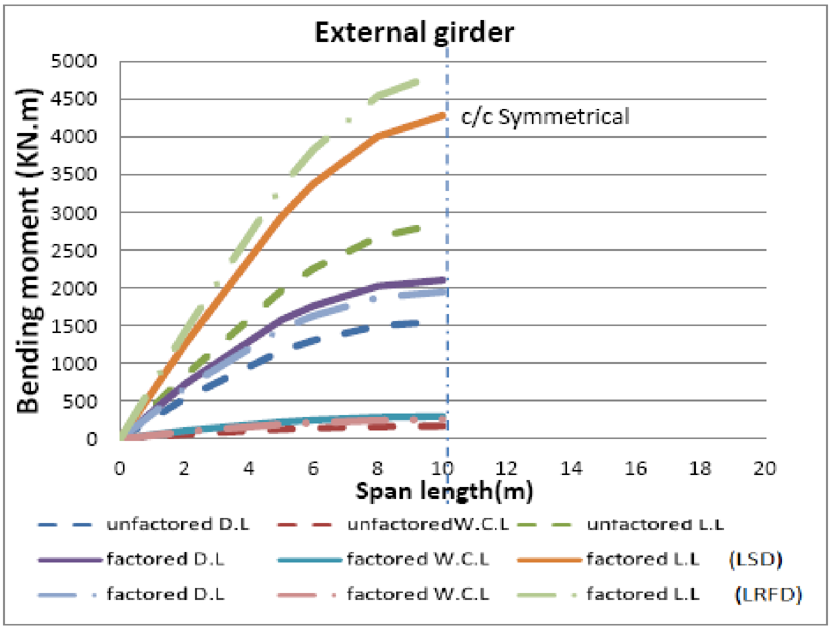

Figure 4.19: Factored bending moment on external girder

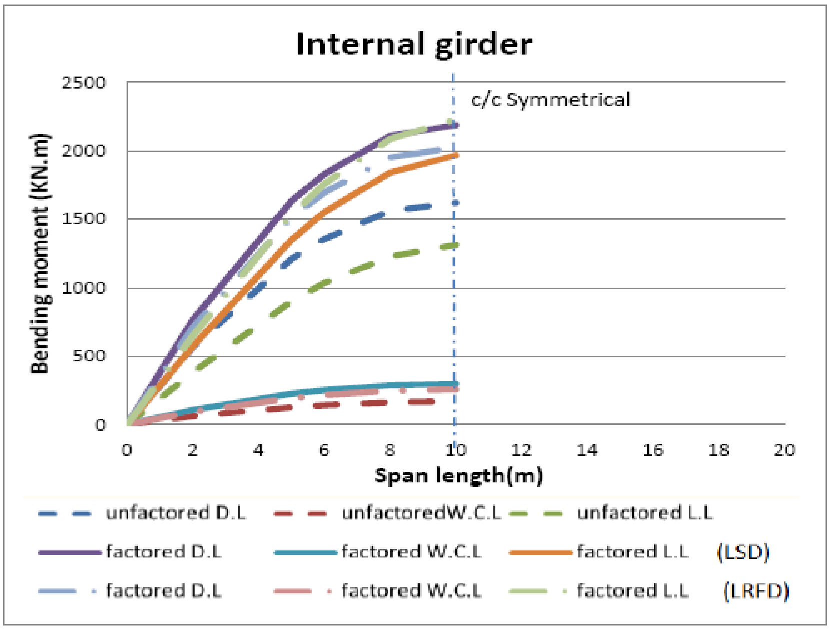

Figure 4.20: Factored bending moment on internal girder

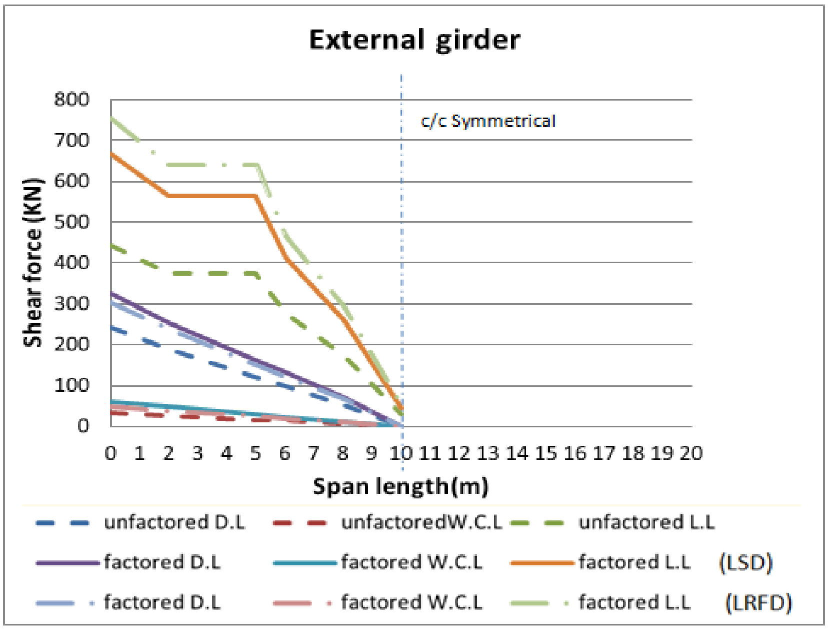

Figure 4.21: Factored shear force on external girder

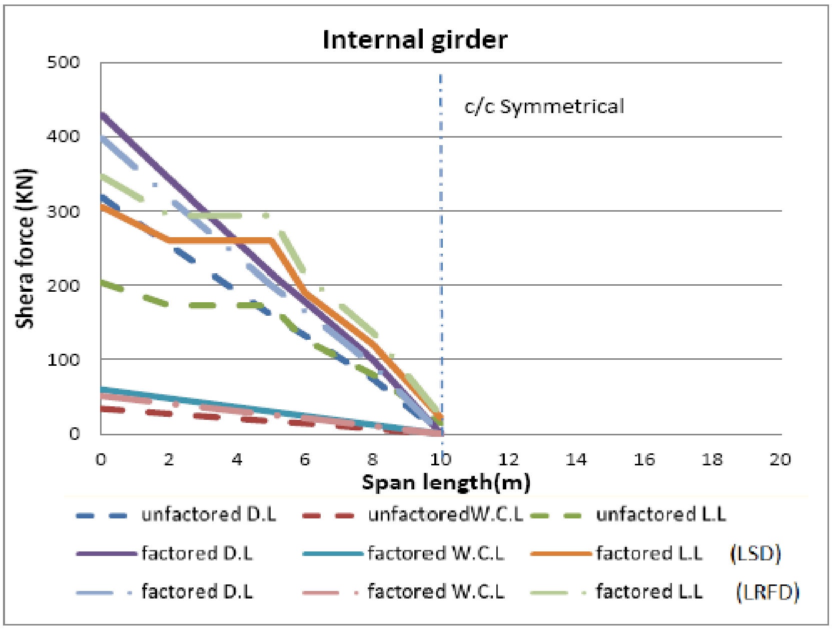

Figure 4.22: Factored shear force on internal girder

4.7 Summary

The difference shows that Courbon's method gives higher moment on girder which is conservative side, at the same time it increases cost of girder. Thus ,it can be concluded that by exact analysis design optimization can be done. The maximum bending moment and shear force is on external girder as compared to internal girder. And also show the factored moment which will be required for limit state design and for load and resistance design, which is high then the unfactored monent which will be useful for working stress design.And it is also observed that partial load factores from AASTHO specfication gives more moment & forces for live load and less for dead load as compared to partial load factors from IRC-6 amendment no.8.

Chapter 5

Bridge Superstructure Design

5.1 General

R.C.C flyover superstructure sections is taken here to design for the maximum bending moment and shear force using limit state design philosophy and load and resistance factored design.

5.1.1 Problem Formulation

In this study the cross section and data taken same as in chapter 4 which is already analysed for design, section shown in fig:

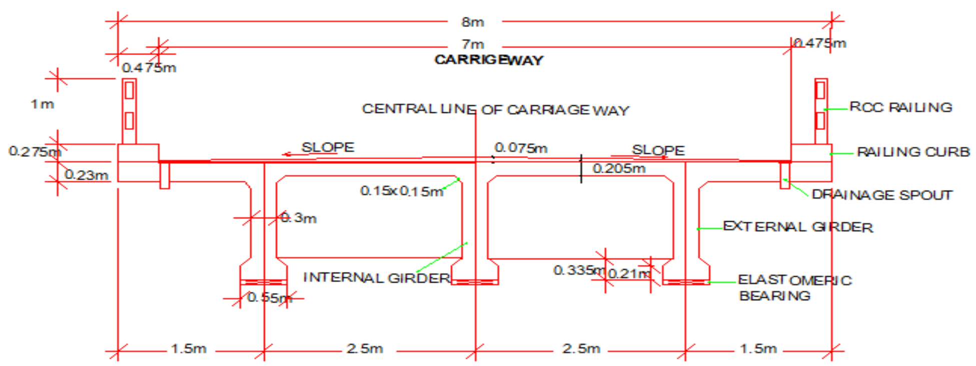

Figure 5.1: Cross Section of Bridge Deck Slab

5.2 Limit State Design Method

5.2.1 Cantilever Slab Design

1) **Design force and moment:**

Unfactored design moment, M	=	61.6 kNm
Unfactored design force, V	=	94 kN

Using,basic load combination,1.35D.L+1.75W.C+1.5L.L

Factored Design moment, Mu	=	89 kNm
Factored Design force, Vu	=	138 kN

2) **Dimension:**

Thickness, D	=	230 mm
Effective cover, d'	=	30 mm

3) **Depth check:**

$k_{u\,\max} = \frac{700}{1100+0.87f_y}$	=	0.48
$R_{U\,\max} = 0.36 f_{ck} k_{u\,\max}(1 - 0.42k_{u\,\max})$	=	4.13
$d_{required} = \sqrt{\frac{Mu}{R_{U\,\max} b}}$	=	147 mm
dprovided=230-30-12/2	=	194 mm

dprovided > drequired , hence safe.

4) **Moment check:**

$M_{urmax} = R_{umax} b d_{req}^2$	=	155 kNm >> M_u

Therefore,section in under-reinforced.

5) **Main steel:**

$A_{st} = \frac{0.5f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6M_u}{f_{ck}bd^2}}\right] \times b \times d$	=	1531 mm^2
Provide 12mm@140mmc/c (top r/f)	=	808 mm^2
Provide 12mm@150mmc/c(bottom r/f)	=	754 mm^2

Total Ast provided = 1562 mm^2

Ast provides > Ast required, safe.

6) **Distribution Steel:-**

Ast required=0.12% of Db for Fe-415

Ast = 0.12x230x1500/100 = 414 mm^2

Prov. half steel at top and half at bottom

Provide 12mm@250mmc/c = 452 mm^2

Total Ast provided = 904 mm^2

Hence, safe.

7) **Check for shear:-**

Vu = 138 kN

pt(%) = Astx100/bd = 0.65

τ_c (From table 19, IS456:2000) = 0.96

for, 230mm slab,

k = 1.125

$V_u c$= (τ_c x k) x bd = 282.85 kN > Vu

τ_v = 0.53 N/mm^2

$\tau_c max$ (From table 20, IS456:2000) = 3.1 N/mm^2

= 1.55 N/mm^2 > τ_v

Hence no, need to provide shear reinforcement.

8) **Check for deflection:-**

Using servicibilty,basic load combination,D.L+W.C+L.L

Assume section 1000x230mm

$M_{L.L}$ = 47.45 kNm

$M_{D.L}$ = 14.09 kNm

Asc (pc%=1%) = 2262 mm^2

Ast (pt%=4%) = 8042.48 mm^2

Es	=	200000 N/mm^2
Ec	=	27386 N/mm^2
m	=	7
$I_{cr} = \frac{Bd^3}{12}$	=	1.01E+09 mm^4
$f_{cr} = 0.75\sqrt{f_{ck}}$	=	3.8 N/mm^2
y_t = D/2	=	115 mm
$M_r = \frac{f_{cr} I_{cr}}{y_t}$	=	34 kNm

Short term deflection:

$bx^2 + (m-1)A_{sc}(x-d') = mA_{st}(d-x)$

x	=	78 mm
z=d-x/3	=	168 mm
$I_r = bx^3/3 + (m-1)A_{sc}(x-d')^2 + mA_{st}(d-x)^2$	=	9.68E+08 mm^4

Perm. load , $I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M}\frac{z}{d}\left(1-\frac{x}{d}\right)\frac{b_w}{b}}$

$I_{eff} < I_r$, so, $I_{eff} = I_r$	=	9.68E+08mm^4
$ai_(perm) = \delta_{DL} = \frac{M_{D.L} l^2}{4E_c I_{eff}}$	=	0.120 mm/m

Long term deflection due to shrikage:

k3 =0.5, k4=1 from eq(3.28)

$\psi_{cs} = k_4 \frac{\varepsilon_{cs}}{D}$	=	1.24E-06
$a_{c3} = k_3 \psi_{cs} l^2$	=	1.4 mm

Long term deflection due to creep

Eec=Ec/(1+Θ) =13041 N/mm^2

Θ =1.1

m=15.33

m-1=14.33

$bx^2 + (m-1)A_{sc}(x-d') = mA_{st}(d-x)$

x	=	99 mm
z=d-x/3	=	161 mm
$I_r = bx^3/3 + (m-1)A_{sc}(x-d')^2 + mA_{st}(d-x)^2$	=	1.55E+09 mm^4

Perm. load , $I_{eff} < I_r$,so,$I_{eff} = I_r$	=	1.55E+09 mm^4
ai.cc(perm)	=	1.3 mm
acc(perm)=ai.cc(perm)-ai(perm)	=	0.3 mm
allowable dead load deflection(L/350)	=	4.3 mm
Total perm. load deflection	=	3.69 mm< 4.3 mm
Deflection due to L.L		
Live load , Ieff	=	1.28E+09 mm^4
allowable live load deflection(L/800)	=	1.9 mm
$ai_{L.L} = \frac{M_{L.L}l^2}{3E_cI_{eff}}$	=	1.32 mm<1.9mm

9) **Check for crack**

$M_{D.L}$ + 50% $M_{L.L}$	=	36 kN
A_{st}	=	8042 mm^2/mm
A_{sc}	=	2262mm^2/mm
E_c	=	27386.12 mm^2/mm
E_s	=	200000 N/mm^2
E_{ce}	=	13693.06 N/mm^2
m	=	14.60593907
d	=	194 mm
D	=	230 mm
b	=	1000 mm

Determine neutral axis at working load,

x = 56.74 mm

d-x = 137.26 mm

I_c = 2.27E+09 mm^4

$\varepsilon_1 M(D-x)/E_{ce}I_c$

$\varepsilon_1 = 0.00104$

$\varepsilon_m = \varepsilon_1 - \frac{b(D-x)(a-x)}{3E_S A_{st}(d-x)}$

$\varepsilon_m = 0.0010$

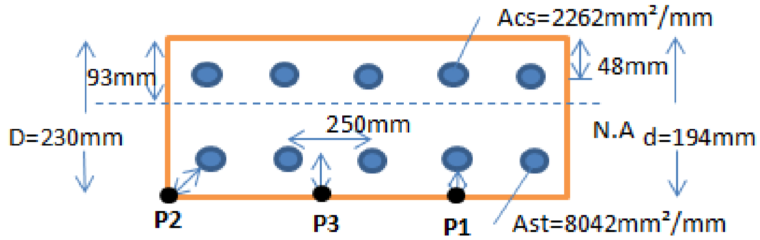

Figure 5.2: Cracked section of cantilever slab

(a) Crack width directly under a bar on tension face at point P1

a_{cr}=nominal cover+distribution bar dia/2 = 48 mm

Crack width=$3a_{cr}\varepsilon_m$ = 0.14 mm

<0.3mm ,Safe

(b) Crack width at bottom corner of beam at P2

$acr = \sqrt{((48^2) + (48^2))} - (12/2)$ = 62 mm

Crack width,

$W_{cr} = \frac{3a_{cr}\varepsilon_m}{1+2(\frac{a_{cr}-C_{\min}}{D-x})}$ = 0.16mm

<0.3mm ,Safe

(c) Crack width between bars mid-way at P3

$acr = \sqrt{((75^2) + (48^2))} - (12/2)$ = 83 mm

Crack width,

$W_{cr} = \frac{3a_{cr}\varepsilon_m}{1+2(\frac{a_{cr}-C_{\min}}{D-x})}$ = 0.19 mm

<0.3mm ,Safe

5.2.2 Continous Slab panel design

1) **Design force and moment:**

Unfactored design moment, Mx	=	34.23 kNm
Unfactored design moment, My	=	23 kNm
Unfactored design force, V	=	94 kN

Using,basic load combination,1.35D.L+1.75W.C+1.5L.L

Factored Design moment, Mux	=	51.25 kNm
Factored Design moment, Muy	=	35.72
Factored Design force, Vu	=	142 kN

2) **Dimension:**

Thickness, D	=	205 mm
Slab panel size,	=	2.5 m x 4 m
Effective cover, d'	=	30 mm

3) **Depth check:**

$k_{u\,\max} = \frac{700}{1100+0.87f_y}$	=	0.48
$R_{U\,\max} = 0.36f_{ck}k_{u\,\max}(1 - 0.42k_{u\,\max})$	=	4.13
$d_{required} = \sqrt{\frac{Mu}{R_{U\,\max}b}}$		
dxreq	=	111 mm
dyreq	=	93 mm
dprovided =205-30-12/2	=	159 mm

dprovided > drequired , hence safe.

3) **Moment check:**

$M_{urmax} = R_{umax}bd_{req}^2$	=	105 kNm	$>> M_{ux}$

Therefore,section in under-reinforced.

4) **Reinforcement :**

$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] \times b \times d$

Astx	=	978 mm^2
Provide 12mm@200mmc/c (bottom r/f)	=	565 mm^2
Provide 12mm@160mmc/c(top r/f)	=	707 mm^2
Total Ast provided	=	1272 mm^2
Astx provides > Astx required, safe.		
Asty	=	621 mm^2
Provide 10mm@170mmc/c(top & bottom r/f)	=	665 mm^2
Asty provides > Asty required, safe.		
Extra bar @ support, ast	=	636 mm^2
Provide 10mm@120mmc/c	=	654.4985 mm^2

6) **Check for shear:-**

Vu	=	1420 kN
pt(%) = Astx100/bd	=	0.9
tc (From table 19, IS456:2000)	=	0.6
for, 230mm slab,		
k	=	1.125
Vuc= (tc x k) x bd	=	263 kN > Vu
tv	=	0.9 N/mm^2
tcmax (From table 20, IS456:2000)	=	3.1 N/mm^2
	=	1.55 N/mm^2

Hence no, need to provide shear reinforcement.

7) **Check for deflection:-**

Using servicibilty,basic load combination,D.L+W.C+L.L

Assume section 1000x205mm

$M_{L.L}$	=	32.27 kNm
$M_{D.L}$	=	4.16 kNm
Asc (pc%=5%)	=	7972 mm^2
Ast (pt%=6%)	=	9703 mm^2
Es	=	200000 N/mm^2
Ec	=	27386.13 N/mm^2
m	=	7 N/mm^2
$I_{cr} = \frac{Bd^3}{12}$	=	1302 mm^4
$f_{cr} = 0.75\sqrt{f_{ck}}$	=	3.8 N/mm^2
y_t = D/2	=	102.5 mm
$M_r = \frac{f_{cr} I_{cr}}{y_t}$	=	48.705 kNm

Short term deflection:

$bx^2 + (m-1)A_{sc}(x-d') = mA_{st}(d-x)$

x	=	18 mm
z=d-x/3	=	153 mm
$I_r = bx^3/3 + (m-1)A_{sc}(x-d')^2 + mA_{st}(d-x)^2$	=	1.41E+09 mm^4

Perm. load , $I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M}\frac{z}{d}\left(1-\frac{x}{d}\right)\frac{b_w}{b}}$

$I_{eff} < I_r$, so, $I_{eff} = I_r$	=	1.41E+09 mm^4
ai(perm)=Mperm*l^2/4*Ec*Ieff	=	0.047672 mm

Long term deflection due to shrikage:

k3 =0.063, k4=0.133

$\psi_{cs} = k_4 \frac{\varepsilon_{cs}}{D}$	=	1.73E-07
$a_{c3} = k_3 \psi_{cs} l^2$	=	0.024591 mm

Long term deflection due to creep

Eec=Ec/(1+ Θ)	=	13041.01

Θ =1.1

m=15.33

m-1=14.33

bx+(m-1)Asc(x-d)=mAst(d-x)		
x	=	19.5 mm
z=d-x/3	=	152.4 mm
Ir=bx3/3+(m-1)Asc(x-d')^2+mAst(d-x)^2	=	1.48E+08
Perm. load , Ieff = Ir	=	1.79E+09 mm^4
ai.cc(perm)	=	1.21 mm
acc(perm)=ai.cc(perm)-ai(perm)	=	0.18 mm
Total perm. load deflection	=	1.280513 mm < 7.14mm
allowable dead load deflection(L/350)	=	7.14 mm
Deflection due to L.L		
Live load , Ieff	=	1.79E+09 mm^4
a(L.L)=ML.L*L/3*Ec*Ieff	=	0.343 mm<3.125mm
allowable live load deflection(L/800)	=	3.125 mm

7) **Check for crack**

MW (D.L+50% L.L)	=	18.9 kN
Ast	=	9703 mm^2/mm
Asc		7971 mm^2/mm
Ec	=	27386.12 N/mm^2
Es	=	200000 N/mm^2
Ece	=	13693 N/mm^2
m	=	14.6
d	=	157 mm
D	=	205 mm
b	=	1000 mm
Now,determine neutral axis at working load,		
x =		86.86 mm

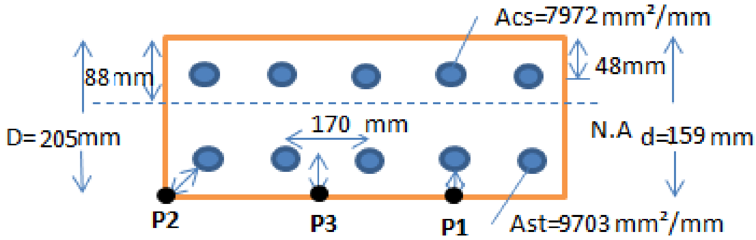

Figure 5.3: Cracked section of continous slab panel

d-x = 70 mm

Ic = 1703294291 mm^4

$\varepsilon_1 =$ 0.000192178

ε_m 0.000148396

(a) Crack width directly under a bar on tension face

acr (cmin)= = 48 mm

Crack width = 0.02 mm

<0.3mm ,Safe

(b) Crack width at bottom corner of beam

$acr = \sqrt{((48^2) + (48^2))} - (12/2)$ = 62 mm

0.02 mm

$W_{cr} = \frac{3a_{cr}\varepsilon_m}{1+2\left(\frac{a_{cr}-C_{\min}}{D-x}\right)}$ <0.3mm ,Safe

(c) Crack width betwwen midway of bars

$acr = \sqrt{((125^2) + (48^2))} - (12/2)$ = 128 mm

$W_{cr} = \frac{3a_{cr}\varepsilon_m}{1+2\left(\frac{a_{cr}-C_{\min}}{D-x}\right)}$ = 0.09 mm

<0.3mm ,Safe

5.2.3 Diaphgram design

Internal diaphgram design

1) **Design force and moment:**

Unfactored design moment, M	=	254 kNm
Unfactored design force, V	=	220 kN

Using,basic load combination,1.35D.L+1.75W.C+1.5L.L

Factored Design moment, Mu	=	377 kNm
Factored Design force, Vu	=	324 kN

2) **Dimension**

Length	=	2500 mm
Thickness	=	250 mm
Depth	=	1165 mm
Cover	=	40 mm

3) **Depth check:**

$R_{U\max} = 0.36 f_{ck} k_{u\max}(1 - 0.42 k_{u\max})$	=	0.47
$k_{u\max} = \frac{700}{1100+0.87 f_y}$	=	4.13

$d_{req} = \sqrt{\frac{Mu}{R_{U\max} b}}$

drequired	=	604 mm
dprovided	=	1115 mm

dprovided > drequired , hence safe.

4) **Steel :**

Longitudinal Reinforcement:

$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] \times b \times d$

Ast req	=	985 mm^2
Provide 4 no.-20mm dia bar	=	1256 mm^2
Ast req >Ast prov		Safe

Skin reinforcement

Ast(0.1% of web area)	=	252.25 mm^2
4nos. Of 10 mm @300mmc/c	=	314 mm^2
(on each face)		

5) **Design of shear reinforcement:-**

Vu	=	327.20kN
pt(%)	=	0.45
$\tau_u c$	=	0.37(From table 19 IS 456)
τ_v	=	0.293N/mm
Shear resistance by concrete ,		
Vuc	=	107.76kN
Shear resistance by min r/f ,		
Vusv.min	=	116.5kN
Vur.min	=	107.76+116.5
Vur.min	=	224.26KN $<$ Vu(327.2kN)
Minimum shear reinforcement sufficent.		
Min. Shear reinforcement required		
Asv/S= 0.4*b/0.87*fy	=	276.96 mm^2/m
Vus=Vu-Vuc	=	219.44kN
Asv (2-legged 10mm dia.)	=	157 mm^2
Spacing,s	=	280mm <300 mm
Hence provide spacing,s	=	280 mm
providing 10mm 2-legged strriups@280mmc/c		
Asv/S (provided)	=	544 mm^2 $>$ (Asv/S req=277mm^2)

6) **Check for deflection**

allowable L/D ratio	=	2.5
permissible L/D ratio	=	2500/1165=2.145<2.5 Hence safe

7) **Check for crack**

MW	=	338 kN
Ast		1256 mm^2
Ec	=	27386.12 N/mm^2
Es	=	200000 N/mm^2
Ece	=	13693.06 N/mm2
m	=	14.6
d	=	1116 mm
D	=	1165 mm

Determine neutral axis at working load,

x =		337.92 mm
d-x =		778.08 mm
Ic =		1.43E+10 mm^4
$\varepsilon_1 = \frac{M_w(d-x)}{E_c e I_c} =$		0.00114
$\varepsilon_m = \varepsilon_1 - \frac{b(D-x)(a-x)}{3E_S A_{st}(d-x)}$	=	0.0007

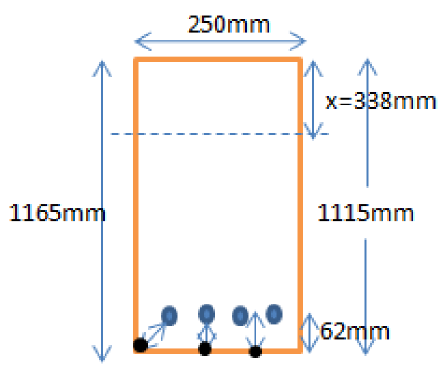

Figure 5.4: Cracked section of internal diaphgram

(a) Crack width directly under a bar on tension face

acr (cmin)=nominalcover+stirrp dia	=	50 mm
Crack width	=	0.11 mm
		<0.3mm ,Safe

(b) Crack width at bottom corner of beam

$acr = \sqrt{(60^2) + (60^2)} - (20/2)$	=	70.9 mm
	=	0.145 <0.3mm ,Safe

(b) Crack width between midway of bar

$acr = \sqrt{(30^2) + (60^2)} - (20/2)$	=	57 mm
	=	0.12 <0.3mm ,Safe

External diaphgram design

1) **Design force and moment:**

Unfactored design moment, M	=	338 kNm
Unfactored design force, V	=	667 kNm

Using,basic load combination,1.35D.L+1.75W.C+1.5L.L

Factored Design moment, Mu	=	447 kNm
Factored Design force, Vu	=	894 kN

2) **Dimension**

Length	=	2500 mm
Thickness	=	250 mm
Depth	=	1165 mm
Cover	=	40 mm

3) **Depth check:**

$R_{U\max} = 0.36 f_{ck} k_{u\max}(1 - 0.42 k_{u\max})$ = 0.479107

$k_{u\max} = \frac{700}{1100+0.87 f_y}$ = 4.133149

$d_{req} = \sqrt{\frac{Mu}{R_{U\max} b}}$

drequired = 669 mm

dprovided = 1115 mm

dprovided > drequired , hence safe.

4) **Steel :**

Longitudinal Reinforcement:

$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] \times b \times d$

Ast req = 1225 mm^2

Provide 5 no.-20mm dia bar = 1570 mm^2

Ast req >Ast prov Safe

Skin reinforcement

Ast(0.1% of web area) = 252.25 mm^2

4nos. Of 10 mm @300mmc/c = 314 mm^2

(on each face)

5) **Design of shear reinforcement:-**

Vu = 894 kN

pt(%) = 0.378

$\tau_u c$ = 0.37(From table 19 IS 456)

τ_v = 0.8N/mm

Shear resistance by concrete ,

Vuc = 223kN

Shear resistance by min r/f ,

Vusv.min = 116.5kN

Vur.min = 223+116.5

Vur.min = 350kN < Vu(894kN)

Minimum shear reinforcement sufficent.

Min. Shear reinforcement required

Asv/S= 0.4*b/0.87*fy = 277mm^2/m

Vus=Vu-Vuc = 660kN

Asv (2-legged 10mm dia.) = 157 mm^2

Spacing,s = 90mm ¡300mm

Hence provide spacing,s = 110mm

providing 10mm 2-legged strriups@110mmc/c

Asv/S (provided) = 1614mm > (Asv/S req=277mm)

6) **Check for deflection**

allowable L/D ratio = 2.5

permissible L/D ratio = 2500/1165=2.145<2.5 Hence safe.

7) **Check for crack**

MW = 338 kN

Ast 1256 mm^2

Ec = 27386 N/mm^2

Es = 200000 N/mm^2

Ece = 13693 N/mm^2

m = 14.6

d = 1116 mm

D = 1165 mm

Determine neutral axis at working load,

x = 589 mm

d-x = 527 mm

2(d-x)/3 = 352 mm

Ic = 22091429722 mm^4

$\varepsilon_1 = \frac{M_w(d-x)}{E_c e I_c} =$ 0.00129

$\varepsilon_m = \varepsilon_1 - \frac{b(D-x)(a-x)}{3E_S A_{st}(d-x)}$ = 0.00108

(a) Crack width directly under a bar on tension face

acr (cmin)=nominalcover+stirrp dia = 50 mm

Crack width = 0.16 mm

<0.3mm ,Safe

(b) Crack width at bottom corner of beam

$acr = \sqrt{(60^2) + (60^2)} - (20/2)$ = 70.9 mm

0.2 mm

<0.3mm ,Safe

(b) Crack width at bar mid way

$acr = \sqrt{(30^2) + (60^2)} - (20/2)$ = 57 mm

0.12 mm

<0.3mm ,Safe

5.2.4 External girder

At mid span section 0.5L=10m

1) **Check for flange depth:**

$b_f = \frac{L_o}{6} + 6D_f + b_w$

bf = (14000/6)+6217.5+300

= 3938.33mm

>2000mm here bf sufficent

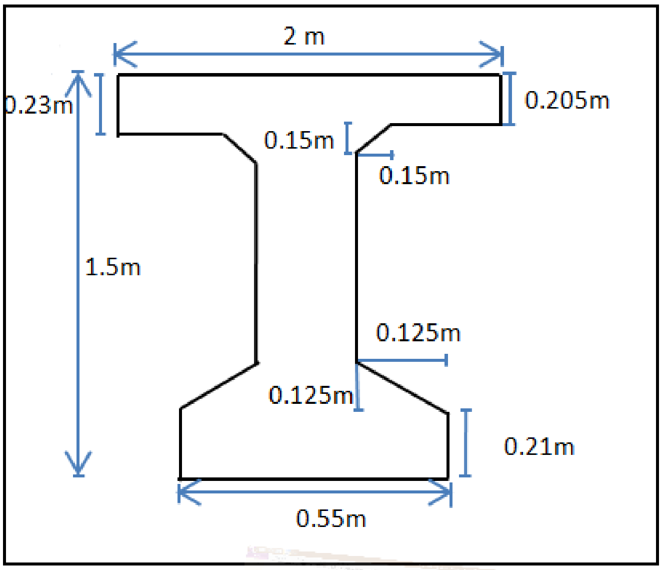

Figure 5.5: Sectional properties of external girder

2) **Check for depth:**

kumax = 700/(1100+0.87415)

= 0.48

Rumax = 0.36*30*0.479(1-0.42*0.479)

= 4.13

$d_{required} = \sqrt{\frac{Mu}{R_{U\ \max} b}}$

dreq = $\sqrt{5980.2692175 * 10^6 / 4.133 * 2000}$

= 850.55mm

3) **Check for width:**

required width =2C1+N+(N-1)C2

C1 = 40mm

C2 = 28mm

N = 8 nos.

width req = 500mm $<$ provided 550mm,i.e.safe.

4) **Exterior girder factored moments:**

Basic load combination for finding ultimate strength

[1.35DL+1.75WCL+1.5(LL)]

Total factored moment, M_u = 5980 kNm refer table4.8

Check for moment:(Limit State Of Collapse-Flexure)

Assume neutral axis,xu=Df

$$\begin{aligned} \text{Mur1} &= 0.36 f_{ck} b_f D_f (d - 0.42 D_f) \\ &= \text{0.36*30*2000*217.5(1441.2-0.42*217.5)} \\ &= \text{6341kNm ¿ Mu} \end{aligned}$$

$$A_{st} = \frac{0.5 f_{ck}}{f_y} \left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] \times b \times d$$

$$\begin{aligned} \text{Ast} &= \text{(0.5*30/415)(1-}\sqrt{1 - 4.6 * 5980.27 * 10^6/30 * 2000 * 1441.2^2}) \\ &\quad \text{*(2000*1441)} \\ &= 12214.67 mm^2 \end{aligned}$$

$$\begin{aligned} \text{xu} &= \text{(0.87*415*12214.67)/(0.36*30*2000)} \\ &= \text{204.17 mm} \end{aligned}$$

Here,Neutral axis lies in flange.

Table 5.1: Longitudinal reinforcement at 0.5L external girder

Longitudinal Reinforcement:						
LAYER	Φ	No	Ast(mm^2)	Y(mm)	c.g(mm)	deff(mm)
4	-	-	-	-		
3	28	4	2461.76	126	58.8	1441.2
2	28	8	4923.52	70		
1	28	8	4923.52	14		
TOTAL		20	12308.8			

At mid span section 0.25L=5m

1) **Check for flange depth:**

$b_f = \frac{L_o}{6} + 6D_f + b_w$

bf = (14000/6)+6217.5+300

= 3938.33mm

>2000mm here bf sufficent

2) **Check for depth:**

kumax = 700/(1100+0.87415)

= 0.48

Rumax = 0.36*30*0.479(1-0.42*0.479)

= 4.13

$d_{required} = \sqrt{\frac{Mu}{R_{U\,\max} b}}$

dreq = $\sqrt{4758.038424 * 10^6 / 4.133 * 2000}$

= 758.68 mm

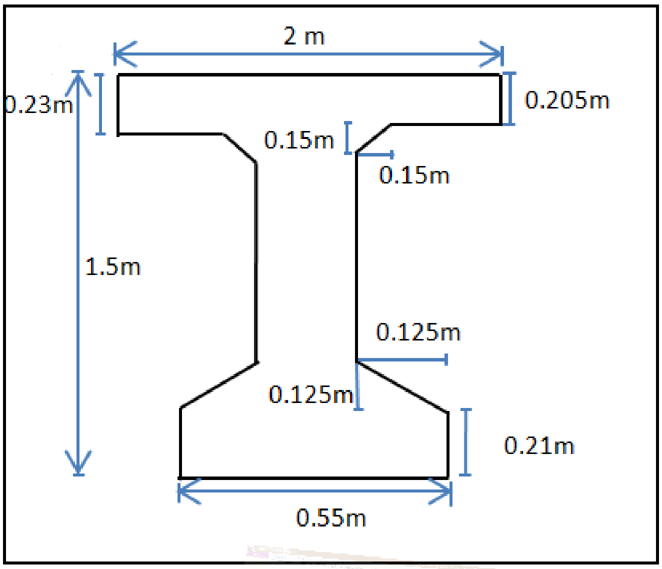

Figure 5.6: Sectional properties of internal girder

3) **Check for width:**

required width =2C1+N+(N-1)C2

C1 = 40mm

C2 = 28mm

N = 8 nos.

width req = 500mm < provided550mm,i.e.safe.

4) **Exterior girder factored moments and shear:**

Basic load combination for finding ultimate strength

[1.35DL+1.75WCL+1.5(LL)]

Total factored moment, M_u = 5980 kNm refer table4.8

Check for moment:(Limit State Of Collapse-Flexure)

Assume neutral axis,xu=Df

$$\begin{aligned} \text{Mur1} &= 0.36 f_{ck} b_f D_f (d - 0.42 D_f) \\ &= 0.36*30*2000*217.5(1441.2\text{-}0.42*217.5) \\ &= 6341\text{KN.m} > \text{Mu} \end{aligned}$$

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] \times b \times d$$

$$\begin{aligned} \text{Ast} &= (0.5 * 30/415)(1 - \sqrt{1 - 4.6 * 5980.27 * 10^6/30 * 2000 * 1441.2^2}) * (2000 * 1441) \\ &= 12214.67\text{mm} \end{aligned}$$

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f}$$

$$\begin{aligned} \text{xu} &= (0.87*415*12214.67)/(0.36*30*2000) \\ &= 204.17\text{mm} \end{aligned}$$

Here,Neutral axis lies in flange.

Table 5.2: Longitudinal reinforcement at 0.25L external girder

Longitudinal Reinforcement:						
LAYER	Φ	No	Ast(mm^2)	Y(mm)	c.g(mm)	deff(mm)
4	-	-	-	-		
3	-	-	-	-	38	1462
2	28	6	3692.64	70		
1	28	8	4923.52	14		
TOTAL		14	8616.16			

Shear Reinforcement Design

Asv/S= 0.4*b/0.87*fy Min. Shear reinforcement required = 609.34mm^2/m

Shear resistance by concrete,Vuc = 332.9172 KN

Shear resistance by min r/f ,Vusv.min = 172.944 KN

Total resistance = 505.8612 KN<Vu

SECTION	S.F.(KN)	width-b(mm)	Eff. Depth(mm)	Long. Rf.	
				Nos.	dia
At support (0.00m)	875.8	550	1441.2	20	28
At 1/4th (5m)	554.8	300	1462	14	28

Table 5.3: Shear reinforcement of external girder

	Pt	Tuc	Tv=T-Tc	Vs	As/S	Provided strriups
At support (0.00m)	1.55	0.73	0.73	217.1	1683.18	2legged 12ϕ stirrups 130mm c/c
At 1/4th (5m)	1.96	0.78	0.78	265.29	907.30	2legged 12ϕ stirrups 240mm c/c

Check for deflection

i) Moment of inertia of Gross uncracked section without reinforcement:-

$$
\begin{aligned}
Ag &= (2000*217.5)+(300*947.5)+(335*550) \\
&= 903500mm \\
Y &= (2000*217.5*(1282.5+(217.5/2)))+ \\
&\quad (300*947.5*1282.5/2)+(550*335*335/2)/903500 \\
&= 905.734mm \\
Ig &= (1/12*2000*217.5^3)+(947.5^3*300/12)+ \\
&\quad (2000*217.5*((1282.5-905.73)+(217.5/2))^2)+ \\
&\quad (947.5*300*((905.73-(1282.5/2)))^2)+ \\
&\quad (335*550*(905.73-(335/2))^2) \\
Ig &= 245819269089.716mm4 \\
yt &= 1500/2 = 750mm \\
\\
fcr &= 3.834N/mm \\
\\
Mr &= 3.83*245819269089.72/750 \\
Mr &= 1256647081.6N.mm \\
Mr &= 1256.65KN.m
\end{aligned}
$$

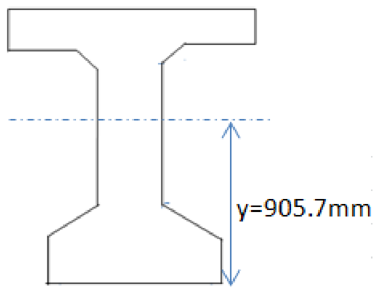

Figure 5.7: Centroidal distance from bottom of I-section

ii) Moment of inertia cracked section with reinforcement:-

$$
\begin{aligned}
\text{Es} &= 200000\text{N/mm} \\
\text{Ec} &= 27386.12\text{N/mm} \\
\text{m} &= \text{Es/Ec}=7.3 \\
\text{x} &= (2000*217.5^2/2)+(7.3*12308.8*1441.2)/(2000*217.5)+(7.3*12308.8) \\
\text{x} &= 336.94\text{mm} \\
\text{z} &= \text{d} -(\text{x}/3) \\
\text{z} &= 1441.2-(336.94/3) \\
\text{z} &= 1328.88\text{mm} \\
\text{Ir} &= (2000*336.94^3/3)-((2000-300)*(336.94-217.5)^3/3) \\
&\quad +7.3*12308.8*(1441.2-336.94)^2 \\
&= 134147902636.459\text{mm4} \\
\text{C} &= 1.2-(1256.65/4114.44)*(1328.89/1441.2)*(1-(336.94/1441.2))*(300/2000) \\
&= 1.17>1, \text{ i.e. Ir=Ieff} \\
\text{Ieff} &= 134147902636.459\ mm^4
\end{aligned}
$$

$$\Delta_{D.L} = \frac{5ML^2}{48E_cI_eff}$$

$$\Delta_{L.L} = \frac{k(3-4k^2}{48E_cI_eff}$$

Table 5.4: Deflection check for external girder

Section	Due to creep(mm)	Due to shrinkage(mm)	$\Delta_{D.L}$ (mm)	Total D.L(mm)	$\Delta_{D.Lallow}$ (mm)	$\Delta_{L.L}$ (mm)	$\Delta_{L.Lallow}$ (mm)
0.5L	5.76	0.3	16.26	22.32	57	1.37	25
0.25L	4.97	0.25	15.35	20.6		0.75	

5.2.5 Internal girder

Sectional properties:

Table 5.5: Crack check for external girder

Section	Crack under tension bar(mm)	Crack at corner (mm)	Crack at midway of bar (mm)	Permissible (mm)
0.5L	0.2	0.25	0.23	0.3
0.25L	0.16	0.23	0.19	0.3

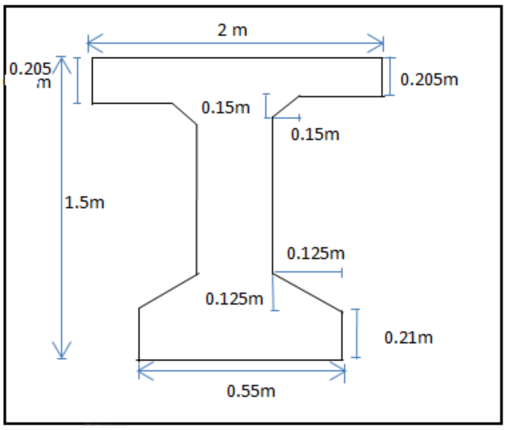

Figure 5.8: Sectional properties of internal girder

At mid span section 0.5L=10m

1) **Check for flange depth:**

bf = (14000/6)+6217.5+300

= 3938.33mm

>2000mm here bf sufficent

2) **Check for depth:**

kumax = 700/(1100+0.87415)

= 0.48

Rumax = 0.36*30*0.479(1-0.42*0.479)

= 4.13

dreq = $\sqrt{5980.2692175 * 10^6/4.133 * 2000}$

= 850.55mm

3) **Check for width:**

required width =2C1+N+(N-1)C2

C1 = 40mm

C2 = 28mm

N = 8 nos.

width req = 500mm < provided550mm,i.e.safe.

4) **Exterior girder factored moments and shear:**

Basic load combination for finding ultimate strength

[1.35DL+1.75WCL+1.5(LL)]

Total factored moment, M_u = 5980 kNm refer table4.8

Check for moment:(Limit State Of Collapse-Flexure)

Assume neutral axis,xu=Df

$$\begin{aligned} \text{Mur1} &= 0.36 f_{ck} b_f D_f (d - 0.42 D_f) \\ &= 0.36*30*2000*205(1462\text{-}0.42*205) \\ &= 6092.49 \text{ KN.m} > \text{Mu} \end{aligned}$$

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] \times b \times d$$

Ast = 8180.3 mm^2

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f}$$

xu = (0.87*415*8180.35)/(0.36*30*2000)

= 136.74 mm

Here,Neutral axis lies in flange.

Table 5.6: Longitudinal reinforcement at 0.5L internal girder

Longitudinal Reinforcement:						
LAYER	Φ	No	Ast(mm^2)	Y(mm)	c.g(mm)	deff(mm)
4	-	-	-	-		
3	-	-	-	-	38	1462
2	28	6	3692.64	70		
1	28	8	4923.52	14		
TOTAL		14	8616.16			

At mid span section 0.25L=5m

1) **Check for flange depth:**

$b_f = \frac{L_o}{6} + 6D_f + b_w$

bf = (14000/6)+6217.5+300

= 3938.33mm

¿2000mm here bf sufficent

2) **Check for depth:**

kumax = 700/(1100+0.87415)

= 0.48

Rumax = 0.36*30*0.479(1-0.42*0.479)

= 4.13

$d_{required} = \sqrt{\frac{Mu}{R_{U\max} b}}$

dreq = $\sqrt{2867.58 * 10^6 / 4.133 * 2000}$

= 588.98 mm

3) **Check for width:**

required width =2C1+N+(N-1)C2

C1 = 40mm

C2 = 28mm

N = 8 nos.

width req = 500mm < provided550mm,i.e.safe.

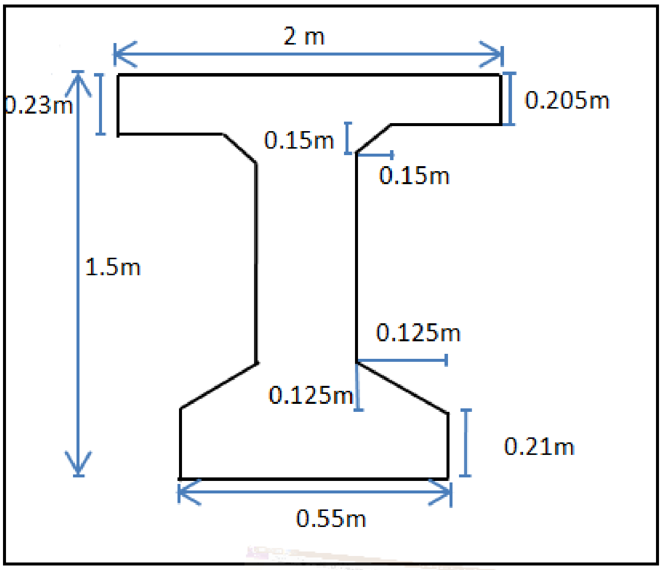

Figure 5.9: Sectional properties of internal girder

4) **Exterior girder factored moments and shear:**

Basic load combination for finding ultimate strength

[1.35DL+1.75WCL+1.5(LL)]

Total factored moment, M_u = 2867.58 kNm refer table4.8

Check for moment:(Limit State Of Collapse-Flexure)

Assume neutral axis,xu=Df

$$\begin{aligned}
\text{Mur1} &= 0.36 f_{ck} b_f D_f (d - 0.42 D_f) \\
&= (0.36*30*2000*205(1474.8\text{-}0.42*205) \\
&= 6149.16 \text{ KN.m} > \text{Mu}
\end{aligned}$$

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right] \times b \times d$$

$$= 5531.6 \text{ mm}$$

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f}$$

$$\begin{aligned}
\text{xu} &= (0.87*415*5531.56)/(0.36*30*2000) \\
&= 92.46 \text{ mm}
\end{aligned}$$

Here,Neutral axis lies in flange.

Table 5.7: Longitudinal reinforcement at 0.25L internal girder

Longitudinal Reinforcement:						
LAYER	Φ	No	Ast(mm^2)	Y(mm)	c.g(mm)	deff(mm)
4	-	-	-	-		
3	-	-	-	-	25	1474.8
2	28	2	1230.88	70		
1	28	8	4923.52	14		
TOTAL		10	6154.40			

Shear Reinforcement Design

Asv/S= 0.4*b/0.87*fy Min. Shear reinforcement required = 609.34mm^2/m

Shear resistance by concrete,Vuc = 332.9172 KN

Shear resistance by min r/f ,Vusv.min = 172.944 KN

Total resistance = 505.8612 KN<Vu

SECTION	S.F.(KN)	width-b(mm)	Eff. Depth(mm)	Long. Rf.	
				Nos.	dia
At support (0.00m)	704.83	550	1462	14	28
At 1/4th (5m)	628.41	300	1474.8	10	28

Table 5.8: Shear reinforcement of internal girder

	Pt	Tuc	Vs=Vu-Vuc	As/S	Provided strriups
At support (0.00m)	1.07	0.66	407.83	772.6226928	2legged 12Φ stirrups 250mm c/c
At 1/4th (5m)	1.39	0.71	314.28	590.2	2legged 12Φ stirrups 300mm c/c

Check for deflection

i) Moment of inertia of Gross uncracked section without reinforcement:-

Ag = 882250

Y = 895.7 mm

Ig = 2.42239E+11 mm4

yt = 1500/2 =750mm

fcr = 3.834N/mm

Mr = 1238.3 KN.m

ii) Moment of inertia cracked section with reinforcement:-

Es = 200000N/mm^2

Ec = 27386.12N/mm^2

m = Es/Ec=7.3

x = 283.38 mm

z = d -(x/3)

z = 1367.54 mm

Ir = (2000*336.94^3/3)-((2000-300)*(336.94-217.5)^3/3)

+7.3*12308.8*(1441.2-336.94)^2

= 1.02308E+11 mm^4

Ieff = 8.76E+10mm^4

= Ir7>Ieff, i.e. Ir=Ieff

Ieff = 134147902636.459mm^4

$\Delta_{D.L} = \frac{5ML^2}{48E_cI_eff}$

$\Delta_{L.L} = \frac{k(3-4k^2}{48E_cI_eff}$

Table 5.9: Deflection check for external girder

Section	Due to creep(mm)	Due to shrinkage(mm)	$\Delta_{D.L}$ (mm)	Total D.L(mm)	$\Delta_{D.Lallow}$ (mm)	$\Delta_{L.L}$ (mm)	$\Delta_{L.Lallow}$ (mm)
0.5L	7.01	0.32	23.27	30	57	2.7	25
0.25L	6	0.25	21		27	2.4	

Table 5.10: Crack check for internal girder

Section	**Crack under tension bar(mm)**	**Crack at corner (mm)**	**Crack at midway of bar(mm)**	**Permissible (mm)**
0.5L	0.16	0.211	0.195	0.3
0.25L	0.13	0.170	0.157	0.3

5.3 Load and resistance factor design

5.3.1 Cantilever Slab Design

1) **Design force and moment:**

Unfactored design moment, M = 59 kNm

Unfactored design force, V = 90 kN

Using,basic load combination,1.25D.L+1.5W.C+1.75L.L

Factored Design moment, Mu = 95 kNm

Factored Design force, Vu = 146 kN

2) **Dimension:**

Thickness, D = 230 mm

Effective cover, d' = 30 mm

3) **Depth check:**

$Dreq = 1.2\frac{S+3000}{30}$ = 147 mm

dreq= 1.2*((2500*1000)+3000)/30 = 220 mm >175mm

Adding sacrifical surfacing 15mm + 25mm to overhanging portion of slab

dprovided=260-30-12/2 = 224 mm

dprovided > drequired , hence safe.

4) **Strength Limit State :**

Using,basic load combination,1.25D.L+1.5W.C+1.75L.L

Assuming the lever arm (d-a/2) is independent of As, we can replace it by jd and solve for approximate As required to resist ϕMn=Mu

j = 0.92

ϕ= 0.9 (AASHTO,A5.5.4.2.1)

$A_s = \frac{M_u/\phi}{f_y j d}$ =1167.460968 mm^2

Maximum reinforcement is limited by the ductility requirement

neutral axis, $a \geq 0.35d = 78.4mm$

$a = \frac{A_s f_y}{0.85 f_c b} = 14mm < 78.4$ mm safe.

Minimum reinforcement (A5.7.3.3.2)

$\rho = \frac{A_s}{bd} \geq 0.03\frac{f_c}{f_y}$ = 0.003860651 >= 0.002168675 ok

min As = 655.8072289 mm^2

Moment capcity check:-

$\phi M_n = \phi A_s f_y (d - \frac{(}{a}2))$ = 94.61 kNm ¿ 89.6 kNm

Main Steel:-

Ast = 1167.460968 mm2

Provide 12 mm @ 160 c/c = 706.8 mm^2 (top r/f)

Provide 12 mm @ 200 c/c = 565.4 mm^2 bottom r/f)

total Astpr = 1272.345025 mm^2

Distribution Steel:-

$percentage = \frac{3540}{\sqrt{S_e}} \geq 67\%$

Se = 1200 mm

percentage = 110.85 % > 67% use 67% dist As = 852.5 mm2

Prov. Half steel at top and half at bottom

Prov. 12 mm dia @ 200 mm = 565.2 mm^2

at top and bottom

Total steel provided = 1130.4 mm > 852.5 mm

Prov. Half steel at top and half at bottom

Vu < $0.5\phi Vc$ Hence no, need to provide shear reinforcement.

5) **Control of cracking:**

$f \leq f_{sa} = \frac{Z}{(d_c A)^{1/3}}$

Z= 30000 N/mm

$bx^2 + (m-1)Asc(x-d) = mAst(d-x)$

x=98.6 mm

$Icr = bx3/3 + (m-1)Asc(x-d')^2 + mAst(d-x)^2$

Icr=566759743.7 mm^4

The tensile stress in bottom steel becomes

$f_s = n(\frac{M_y}{I_{cr}})$

$f_s = 195MPa$

$f_{sa} = 292MPa > 0.6f_y$

$f_{sa} = 0.6f_y = 249MPa > f_s = 195Mpa$

5.3.2 Continous Slab panel design

1) **Design force and moment:**

Unfactored design moment, Mx	=	35 kNm
Unfactored design moment, My	=	24 kNm
Unfactored design force, V	=	95 kN

Using,basic load combination,1.35D.L+1.75W.C+1.5L.L

Factored Design moment, Mux	=	60 kNm

Factored Design moment, Muy	=	40 kNm
Factored Design force, Vu	=	165 kN

2) **Dimension:**

Thickness, D	=	235 mm
Slab panel size,	=	2.5 m x 4 m
Effective cover, d'	=	30 mm

3) **Depth check:**

$Dreq = 1.2\frac{S+3000}{30}$	=	147 mm
dreq= 1.2*((2500*1000)+3000)/30	=	220 mm >175mm

Adding sacrifical surfacing 15mm

dprovided=235-30-12/2 = 199 mm

dprovided > drequired , hence safe.

4) **Strength Limit State :**

Using,basic load combination,1.25D.L+1.5W.C+1.75L.L

Assuming the lever arm (d-a/2) is independent of As, we can replace it by jd and solve for approximate As required to resist ϕMn=Mu

j = 0.92

ϕ= 0.9 (AASHTO,A5.5.4.2.1)

$A_{sx} = \frac{M_u/\phi}{f_y jd}$ =839.7 mm^2

$A_{sy} = \frac{M_u/\phi}{f_y jd}$ =540.5 mm^2

Maximum reinforcement is limited by the ductility requirement

neutral axis,

$a \geq 0.35d = 69.65mm$

$a = \frac{A_s f_y}{0.85 f_c b} = 14mm <$ 78.4 mm safe.

Minimum reinforcement (A5.7.3.3.2)

$\rho = \frac{A_s}{bd} \geq 0.03\frac{f_c}{f_y}$ = 0.003860651 >= 0.002168675 ok

min As = 427 mm^2

Moment capcity check:-

$\phi M_n = \phi A_s f_y (d - \frac{(}{a}2)) = 60.27$ kNm ¿ 57.42 kNm

Main Steel:-

Ast = 839.7 mm2

Provide 12 mm @ 200 mmc/c = 565.4 mm^2 (top r/f)

Provide 12 mm @ 190 mmc/c = 595.2491344 mm^2 bottom r/f)

total Astpr = 1160.7 mm^2

Provide 10 mm @ 120 mmc/c = 655 mm2 eatra top bar

Distribution Steel:-

$percentage = \frac{3540}{\sqrt{S_e}} \geq 67\%$

Se = 1200 mm

percentage = 110.85 % > 67% use 67% dist As = 852.5 mm2

Prov. Half steel at top and half at bottom

Prov. 12 mm dia @ 200 mm = 565.2 mm^2

at top and bottom

Total steel provided = 1130.4 mm > 852.5 mm

Prov. Half steel at top and half at bottom

Vu < $0.5\phi Vc$ Hence no, need to provide shear reinforcement.

5) **Control of cracking:**

$f \leq f_{sa} = \frac{Z}{(d_c A)^{1/3}}$

Z= 30000 N/mm

$bx^2 + (m-1)Asc(x-d) = mAst(d-x)$

x=84.80 mm

$Icr = bx3/3 + (m-1)Asc(x-d')^2 + mAst(d-x)^2$

Icr=1416439093 mm^4

The tensile stress in bottom steel becomes

$f_s = n(\frac{M_y}{I_{cr}})$

$f_s = 175MPa$

$f_{sa} = 250MPa > 0.6f_y$

$f_{sa} = 0.6f_y = 249MPa > f_s = 195Mpa$

Hence, safe.

5.3.3 Diaphgram design

External diapgram

1) **Design force and moment:**

Unfactored design moment, M	=	338 kNm
Unfactored design force, V	=	667 kNm

Using,basic load combination,1.35D.L+1.75W.C+1.5L.L

Factored Design moment, Mu	=	408 kNm
Factored Design force, Vu	=	826 kN

2) **Dimension**

Length	=	2500 mm
Thickness	=	300 mm
Depth	=	1165 mm
Cover	=	40 mm

3) **Depth check:**

$D = 0.7L$	=	0.7*2500
d_{req}	=	560 mm
dprovided	=	1115 mm

dprovided > drequired , hence safe.

4) **Steel :**

Longitudinal Reinforcement:

$A_{st} = \frac{M_u/\phi}{f_y jd}$

Ast req	=	1018 mm^2
Provide 4 no.-20mm dia bar	=	1256 mm^2
Ast req >Ast prov		Safe

5) **Check for shear:** De-

Skin reinforcement

4nos. Of 10 mm @300mmc/c	=	314 mm^2
(on each face)		

sign of shear reinforcement:- Vu = 826 kN

As = 1256 mm^2

a = 408 mm

de = 1115 mm

dv = d-a/2 = 1003.50 mm

dv = 0.9 d_e = 1253.52 mm

dv = 0.72h = 835.2 mm

Select the maximum one.

Calculate the shear stress ratio

$v = \frac{V_u}{\phi_n b_v d_v} = 3.18$ Mpa

v/fc = 0.106119266

Calculate strain ,

estimate θ =40degree and cot θ=1.192 for trial.

$\varepsilon_x = \frac{M_u/d_v + 0.5V_u Cot\theta}{E_S A_s d} = 0.001956524$

$\beta = 2.2$

Calculate the required web reinforcement $V_s = \frac{V_u}{\phi_v} - 0.083\beta\sqrt{f_c} b_v d_v$

Vs = 603.24 kN

Calculate the required spacing of stirrups $s \geq \frac{A_v f_y d_v}{V_s} Cot\theta$= 178 mm

$s \geq \frac{A_v f_y}{0.083\sqrt{f_c b_v}} = 687$ mm

$V_u < 0.1 f_c b_v d_v = 864$ kN Safe.

providing 10 mm 2-legged strriups 140 mm

Internal diaphgram

1) **Design force and moment:**

Unfactored design moment, M	=	272 kNm
Unfactored design force, V	=	676 kNm

Using,basic load combination,1.35D.L+1.75W.C+1.5L.L

Factored Design moment, Mu	=	468 kNm
Factored Design force, Vu	=	402 kN

2) **Dimension**

Length	=	2500 mm
Thickness	=	300 mm
Depth	=	1165 mm
Cover	=	40 mm

3) **Depth check:**

$D = 0.7L$	=	0.7*2500
d_{req}	=	560 mm
dprovided	=	1115 mm

dprovided > drequired , hence safe.

4) **Steel :**

Longitudinal Reinforcement:

$A_{st} = \frac{M_u/\phi}{f_y jd}$

Ast req	=	1018 mm^2
Provide 4 no.-20mm dia bar	=	1256 mm^2
Ast req >Ast prov		Safe

Skin reinforcement

4nos. Of 10 mm @300mmc/c	=	314 mm^2

(on each face)

5) **Check for shear:** Design of shear reinforcement:- Vu = 826 kN

As = 1256 mm^2

a = 408 mm

de = 1115 mm

dv = d-a/2 = 1003.50 mm

dv = 0.9 d_e = 1253.52 mm

dv = 0.72h = 835.2 mm

Select the maximum one.

Calculate the shear stress ratio

$v = \frac{V_u}{\phi_n b_v d_v}$ = 3.18 Mpa

v/fc = 0.001108554

Calculate strain ,

estimate θ =40degree and cot θ=1.192 for trial.

$\varepsilon_x = \frac{M_u/d_v + 0.5V_u Cot\theta}{E_S A_s d}$ = 0.001108554

$\beta = 2.2$

Calculate the required web reinforcement $V_s = \frac{V_u}{\phi_v} - 0.083\beta\sqrt{f_c} b_v d_v$

Vs = 205 kN

Calculate the required spacing of stirrups $s \geq \frac{A_v f_y d_v}{V_s} Cot\theta$= 523 mm

$s \geq \frac{A_v f_y}{0.083\sqrt{f_c b_v}}$ = 687 mm

$V_u < 0.1 f_c b_v d_v$ = 864 kN Safe.

providing 10 mm 2-legged strriups 200 mm

5.3.4 External girder

1) Check for flange depth:

bf $\geq$ 1/4 of effective span = 3500 mm

bf $\geq$ = $6t_s + b_w$ = 3320 mm

bf $\geq$ Width of overhang = 1500+3320/2= 3160 mm

bf = 2000 mm o.k

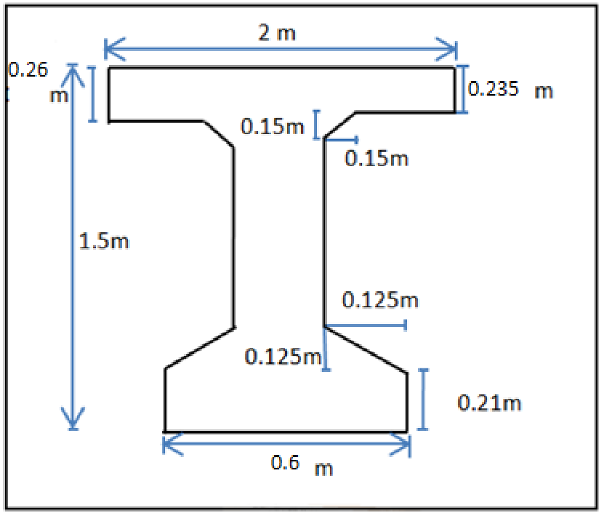

Figure 5.10: Sectional properties of external girder

2) Check for depth:

Dreq=0.7L

Dreq = 1400 mm

Dprov = 1500 mm o.k

3) Check for width:

$b_{min} = 2(C + d_s) + Nd_b + (N - 1)(1.5d_b)$

C = 40 mm

db = 28 mm

ds = 12 mm

N = 8

width req = 598 mm <600mm sufficient width

4) Select Resistance Factors

i) Strength Limit State = ϕ (A5.5.42.1)

Flexure and tension = 0.9

Shear and torsion = 0.9

ii) Nonstrength Limit States = 1 (A1.3.2.1)

5) Select Load Modifiers

	Strength	Service	Fatigue
Ductility,η_D	0.95	1	1
Redundancy,η_R	0.95	1	1
Impoatance,η_I	1.05	N/A	N/A
η=ηD *η_R*ηI	0.95	1	1

6) **Multiple presence factor:**

No.of Loaded Lanes	m
1	1.2
2	1

7) Dynamic Load Allowance: 33%

8) Select Applicabile Load Combination:

Strength -I Limit State

U = 1.25DC+1.5DW+1.75(LL+IM) (ref table 4.11)

Service- I Limit State

U = 1(DC+DW)+1(LL+IM)

9) Exterior girder factored moments and shear:

U= η[1.25DC+1.5DW+1.75(LL+IM)]

Total factored moment = 7789 kNm

10) Check for Neutral axis position

$\omega=\rho\frac{f_y}{f_c}$ =0.03

c=$\frac{1.18\omega d}{\beta_1}$=57 $<< D_f$

Therefore, neutral axis lies in flange.

Table 5.11: Longitudinal reinforcement at 0.5L external girder

Longitudinal Reinforcement:						
LAYER	Φ	No	Ast(mm^2)	Y(mm)	c.g(mm)	deff(mm)
4	28	6	3692.64	222		
3	28	6	3692.64	166	130	1370
2	28	8	4923.52	110		
1	28	8	4923.52	54		
TOTAL		28	17232.32			

Assuming the lever arm (d-a/2) is independent of As, we can replace it by jd and solve for approximate As required to resist ϕ Mn=Mu

j = 0.9 $A_s = \frac{M_u/\phi}{f_y jd}$ =16913 mm^2

Maximum reinforcement is limited by the ductility requirement

$a \geq 0.35d = 478.66mm$ $a = \frac{A_s f_y}{0.85 f_c b} = 137mm <$ 4785 mm safe.

Minimum reinforcement (A5.7.3.3.2)

$\rho = \frac{A_s}{bd} \geq 0.03 \frac{f_c}{f_y}$ = 0.006172884 >= 0.002168675

min As = 5942 mm^2

Moment capcity check:-

$\phi M_n = \phi A_s f_y (d - \frac{a}{2})$ = 8219 kNm ¿7789 kNm

11) **Check for shear:** Design of shear reinforcement:- Vu = 1223 kN (at support)

As = 17232.32 mm^2

a = 137.6 mm

de = 1370 mm

dv = d-a/2 = 1301.18 mm

dv = 0.9 d_e = 1233.00 mm

dv = 0.72h = 792 mm

Select the maximum one.

Calculate the shear stress ratio

$v = \frac{V_u}{\phi_n b_v d_v} = 3.06$ Mpa

v/fc = 0.102066862

Calculate strain ,

estimate θ =40degree and cot θ=1.192 for trial.

$\varepsilon_x = \frac{M_u/d_v + 0.5V_u Cot\theta}{E_S A_s d} = 0.000185719$

$\beta = 2.4$

Calculate the required web reinforcement $V_s = \frac{V_u}{\phi_v} - 0.083\beta\sqrt{f_c} b_v d_v$

Vs = 769.36 kN

Calculate the required spacing of stirrups $s \geq \frac{A_v f_y d_v}{V_s} Cot\theta$= 189 mm

$s \geq \frac{A_v f_y}{0.083\sqrt{f_c b_v}} = 687$ mm

$V_u < 0.1 f_c b_v d_v = 1171$ kN Safe.

providing 12 mm 2-legged strriups 140 mm (at support)

12) **Control of cracking:**

Elastic -Cracked transformed section analysis required to check crack control (A5.7.3.4)

Es = 200000 N/mm

Ec = 26290.68276 N/mm

n=Es/Ec = 7.6

$$f \leq f_{sa} = \frac{Z}{(d_c A)^{1/3}}$$

Z= 30000 N/mm

$bx^2 + (m-1)Asc(x-d) = mAst(d-x)$

x=381 mm

$Icr = bx3/3 + mAst(d-x)^2$

Icr=1.E+11 mm^4

The tensile stress in bottom steel becomes

$f_s = n(\frac{M_y}{I_{cr}})$

$f_s = 226.31 MPa$

$f_{sa} = 470.03 MPa > 0.6 f_y$

$f_{sa} = 0.6 f_y = 249 MPa > f_s = 250.60 Mpa$

Hence,here,fs¡fsa, safe.

13) **Deflection check:**

i) Moment of inertia of Gross uncracked section without reinforcement:-

Ag = (2000*235)+(350*930)+(335*600)

= 996500 mm^2

Y = (2000*235*(1265+(235/2)))+ (350*930*1265/2)+(600*335*335/2))/996500

Y= 892 mm

Ig = $(1/12 * 2000 * 235^3) + (930^3 * 350/12) +$ $(2000 * 235 * ((1265 - 892.44) + (235/2))^2) +$ $(930 * 350 * ((892.44 - (1265/2)))^2) +$ $(335 * 600 * (892.44 - (335/2))^2)$

Ig = 2.66125E+11mm^4

yt = 1500/2 = 750 mm

fcr = 3.45 N/mm^2

Mr = 3.45*266124813631.04/750

Mr = 1224405534 Nmm

Mr = 1224.4 kNm

ii) Moment of inertia cracked section without reinforcement:-

Es = 200000 N/mm^2

Ec = 26290.68276 N/mm^2

m = Es/Ec = 7.61

x = 305 mm

Icr = 1.24692E+11 mm^4

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

Ieff= 1.3013E+11 mm^2 < Igr ok

Ieff $= ((1163.64/3697.6)^3 * 2.529E + 11) + ((1 - (1163.64/3697)^3) * 6.07E + 10)$

EI = 3.4212E+15 Nmm^2

$\Delta_{D.L}$ = 14.15 mm < 57m

$\Delta_{D.L}$ = 3.2 mm < 25m

5.3.5 Internal girder

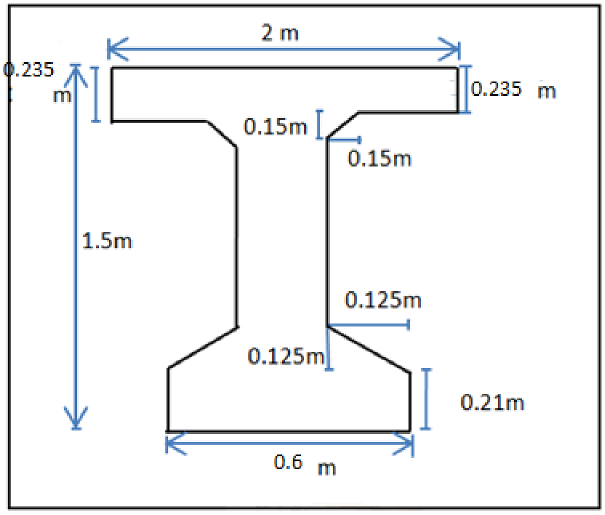

Figure 5.11: Sectional properties of internal girder

1) Check for flange depth:

bf $\geq$ 1/4 of effective span = 3500 mm

bf $\geq$ =$12t_s + b_w$ = 3320 mm

bf $\geq$ Average spacing = 2500 mm

bf = 2000 mm o.k

2) Check for depth:

Dreq=0.7L

Dreq = 1400 mm

Dprov = 1500 mm o.k

3) Check for width:

$b_{min} = 2(C + d_s) + Nd_b + (N - 1)(1.5d_b)$

C = 40 mm

db = 28 mm

ds = 12 mm

N = 8

width req = 598 mm <600mm sufficient width

4) Select Resistance Factors

i) Strength Limit State = ϕ (A5.5.42.1)

Flexure and tension = 0.9

Shear and torsion = 0.9

ii) Nonstrength Limit States = 1 (A1.3.2.1)

5) Select Load Modifiers

	Strength	Service	Fatigue
Ductility,η_D	0.95	1	1
Redundancy,η_R	0.95	1	1
Impoatance,η_I	1.05	N/A	N/A
η=ηD *η_R*ηI	0.95	1	1

6) **Multiple presence factor:**

No.of Loaded Lanes	m
1	1.2
2	1

7) Dynamic Load Allowance: 33%

8) Select Applicabile Load Combination:

Strength -I Limit State U = 1.25DC+1.5DW+1.75(LL+IM)

Service- I Limit State

U = 1(DC+DW)+1(LL+IM)

9) Exterior girder factored moments and shear:

U= η[1.25DC+1.5DW+1.75(LL+IM)]

Total factored moment = 4773.94 kNm

10) Check for Neutral axis position

ω=$\rho\frac{f_y}{f_c}$ =0.03

c=$\frac{1.18\omega d}{\beta_1}$=49.647 << D_f

Therefore, neutral axis lies in flange.

Assuming the lever arm (d-a/2) is independent of As, we can replace it by jd and solve for approximate As required to resist ϕ Mn=Mu

j = 0.9 $A_s = \frac{M_u/\phi}{f_y jd}$ =10126 mm^2

Maximum reinforcement is limited by the ductility requirement

$a \geq 0.35d = 490mm$ $a = \frac{A_s f_y}{0.85 f_c b} = 82mm <$ 490 mm safe.

Minimum reinforcement (A5.7.3.3.2)

$\rho = \frac{A_s}{bd} \geq 0.03\frac{f_c}{f_y}$ = 0.003610292 >= 0.002168675

min As = 6082.8 mm^2

Moment capcity check:-

$\phi M_n = \phi A_s f_y (d - \frac{a}{2})$ = 5148 kNm >4773.94 kNm

11) **Check for shear:**

Design of shear reinforcement:-

Vu = 1312 kN (at support)

As = 11077.92 mm^2

a = 82.4 mm

Table 5.12: Longitudinal reinforcement at 0.5L internal girder

Longitudinal Reinforcement:						
LAYER	Φ	No	Ast(mm^2)	Y(mm)	c.g(mm)	deff(mm)
4	-	-	-	-		
3	28	4	2461.76	166	130	1370
2	28	6	3692.64	110		
1	28	8	4923.52	54		
TOTAL		18	11077.92			

de = 1402 mm

dv = d-a/2 = 1328.80 mm

dv = 0.9 d_e = 1262.20 mm

dv = 0.72h = 792 mm

Select the maximum one.

Calculate the shear stress ratio

$v = \frac{V_u}{\phi_n b_v d_v}$ = 3.66 Mpa

v/fc = 0.121899551

Calculate strain ,

estimate θ =40degree and cot θ=1.192 for trial.

$\varepsilon_x = \frac{M_u/d_v + 0.5V_u Cot\theta}{E_S A_s d}$ = 0.000352353

β = 2.4

Calculate the required web reinforcement

$V_s = \frac{V_u}{\phi_v} - 0.083\beta\sqrt{f_c} b_v d_v$

Vs = 1022 kN

Calculate the required spacing of stirrups

$s \geq \frac{A_v f_y d_v}{V_s} Cot\theta$= 145 mm

$s \geq \frac{A_v f_y}{0.083\sqrt{f_c b_v}}$ = 687 mm

$V_u < 0.1 f_c b_v d_v$ = 1195 kN Safe.

providing 12 mm 2-legged strriups 150 mm (at support)

12) **Control of cracking:**

Elastic -Cracked transformed section analysis required to check crack control (A5.7.3.4)

Es = 200000 N/mm

Ec = 26290.68276 N/mm

n=Es/Ec = 7.6

$f \leq f_{sa} = \frac{Z}{(d_c A)^{1/3}}$

Z= 30000 N/mm

$bx^2 + (m-1)Asc(x-d) = mAst(d-x)$

x=235 mm

$Icr = bx3/3 + mAst(d-x)^2$

Icr=1.E+11 mm^4

The tensile stress in bottom steel becomes

$f_s = n(\frac{M_y}{I_{cr}})$

$f_s = 259.26MPa$

$f_{sa} = 544.93MPa > 0.6f_y$

$f_{sa} = 0.6f_y = 249MPa$¡$f_s = 259.26MPa$

Hence,here,fs¡fsa, safe.

13) **Deflection check:**

i) Moment of inertia of Gross uncracked section without reinforcement:-

Ag = 996500 mm^2

Y = 892 mm

Ig = 2.66125E+11mm^4

yt = 1500/2 = 750 mm

fcr = 3.45 N/mm^2

Mr = 1224405534 Nmm

Mr = 1224.4 kNm

ii) Moment of inertia cracked section without reinforcement:-

Es = 200000 N/mm^2

Ec = 26290.68276 N/mm^2

m = Es/Ec = 7.61

x = 305 mm

Icr = 1.24692E+11 mm^4

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

Ieff = $((1163.64/3697.6)^3 * 2.529E + 11) + ((1 - (1163.64/3697)^3) * 6.07E + 10)$

Ieff= 1.33374E+11 mm^2 < Igr ok

EI = 3.4212E+15 Nmm^2

$\Delta_{D.L}$ = 21.2 mm < 57m

$\Delta_{D.L}$ = 3.13 mm < 25m

5.4 Summary

Table 5.13: Flexure and collapse summary

Collapse					
	Long. Girder	flexure		Shear	
		M.R of section	Actual M.R	S.F of section	Actual S.F
LSD (IS456:2000)	External	5980 kNm	6341kNm	1000kN	522kN
	Internal	4149 kNm	6093 kNm	704kN	477kN
LRFD (ASSTHO)	External	7789 kNm	8219.9 kNm	1075kN	1366kN
	Internal	4774 kNm	5148 kNm	832kN	1195.9kN

Table 5.14: Serviceability summary

Serviceability					
	Long. Girder	Deflection		Cracking	
		Total deflection	Permissible	cracking width	Permissible
LSD (IS456:2000)	External	30mm	33mm	0.21mm	0.3mm
	Internal	31mm		0.25mm	
LRFD (ASSTHO)	External	26mm	33mm	173MPa	470MPa
	Internal	28mm		257MPa	517MPa

Chapter 6

Estimation of Quantity and cost

6.1 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.

6.2 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. The quantity analysis for problem in chapter 4 is given as under.

6.2.1 Estimation of concrete and wearing coat quantity

The total concrete quantity of 20m span, three girder bridge superstructure having 10 diaphgram and also estimated total quantity of wearing surface.

Table 6.1: Concrete and wearing coat quantity

Describtion	**Quantity**	**unit**
Slab	34.3	m3
Girder	32	m3
Diaphragm	8	m3
Total concrete	74.3	m3
Wearing coat	23.265	ton

6.2.2 Estimation of steel quantity

6.2.3 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 as well as high tensile steel are based on current market rates.

The following current market rates:

Concrete = 5116 Rs/m^3 (including scafolding, shuttering,labour)

Steel = 40 Rs/kg

Wearing coat = 4000 Rs/kg

6.2.4 Total cost

Description	Quantity	Rate	Unit	Cost(Rs)
Slab concrete	34	5116	Rs/m^3	175479
Slab R/F	3963	40	Rs/kg	158504
Girder concrete	40	5116	Rs/m^3	203339
Girder R/F	16284	40	Rs/kg	16284
Wearing coat	23	4000	$Rs/tonne$	93060
Total cost				1123218

6.2.5 Summary

Estimation of different items like , concrete including (shuttering and scaffolding), reinforcement of deck slab and girder 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 13 is 1123218 Rs.

Chapter 7

Parametric Study

7.1 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 span to depth ratio parametric study was done for 15m, 20m and 25m span by taking various depths for various L/D ratios. Total 9 cases for different spans of various L/D ratios with three depths are discussed in this chapter.

7.2 Section at various depth

The forces calculated and summary of design quantity trials shown in this chapter is based on various L/D ratios at three various depths and the R.C.C bridge cross section is as shown in fig 7.1 for a span of 15m, 20m and 25m.

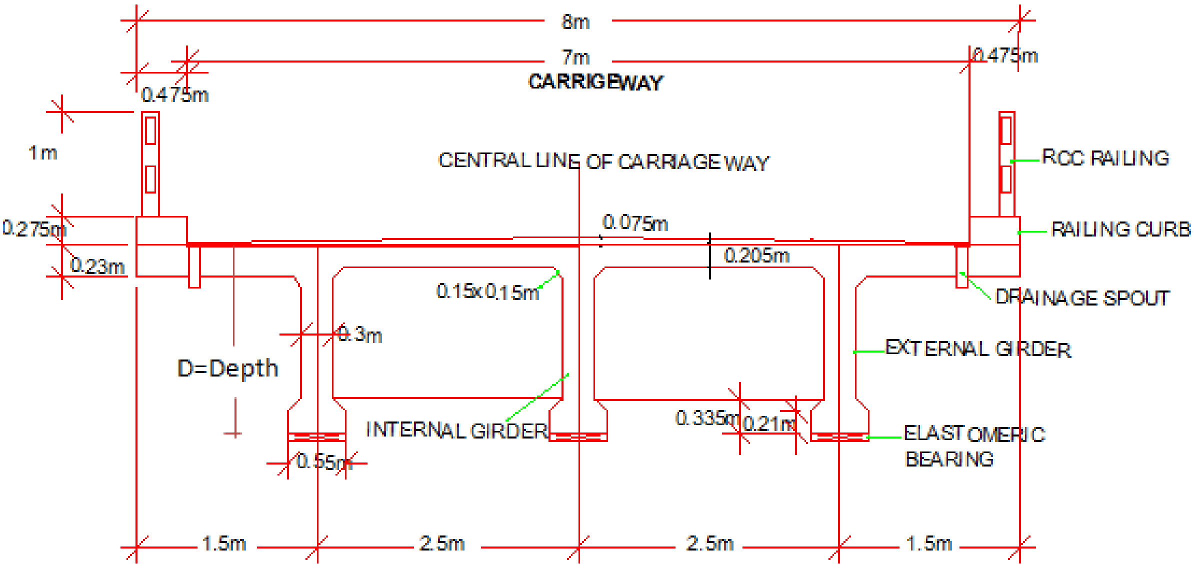

Figure 7.1: Cross Section of Bridge Deck Slab

Material of girder is with f_ck=M30, f_y=Fe415.

D=Trial Depth of girder	1.1m	1.5m	1.9m

7.3 Results

The analysis was done in SAP software. The maximum Bending moment and shear force at different three depths for 15m, 20m and 25m with different L/D ratio alternatives are tabulated in table 7.1 and 7.2. Corresponding graphical variations are also shown in Fig. 7.2,Fig.7.3,Fig.7.4 and Fig.7.5.

Table 7.1: Maximum bending moment in girder

			External girder(kNm)		Internal girder(kNm)	
Span (m)	Depth (m)	L/D ratio	Unfactored Moment	Factored Moment	Unfactored Moment	Factored Moment
15	1.1	14	3274	4951	2232	3411
	1.5	10	3373	4951	2352	3411
	1.9	8	3475	5089	2473	3575
20	1.1	18	4397	6434	2877	4289
	1.5	13	4595	6434	2877	4630
	1.9	11	4728	6880	3104	4953
25	1.1	23	5620	8183	4267	6136
	1.5	17	5891	8549	4581	6561
	1.9	13	6158	8910	4898	6989

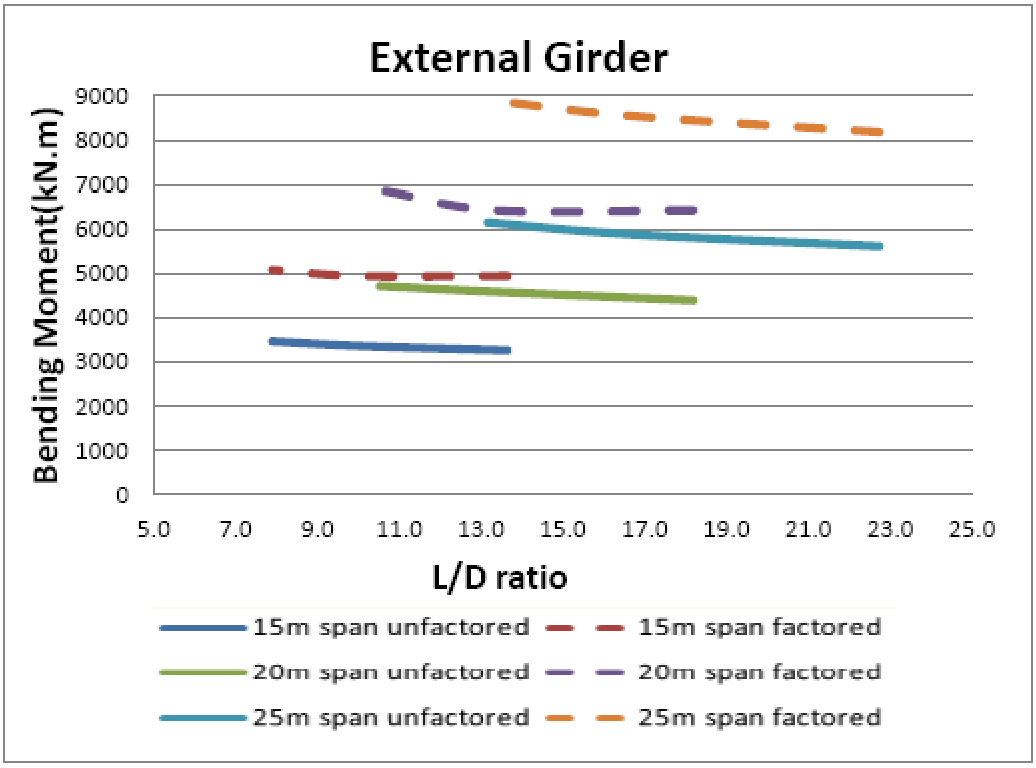

Figure 7.2: Bending moment vartiation in external girder along various L/D ratio

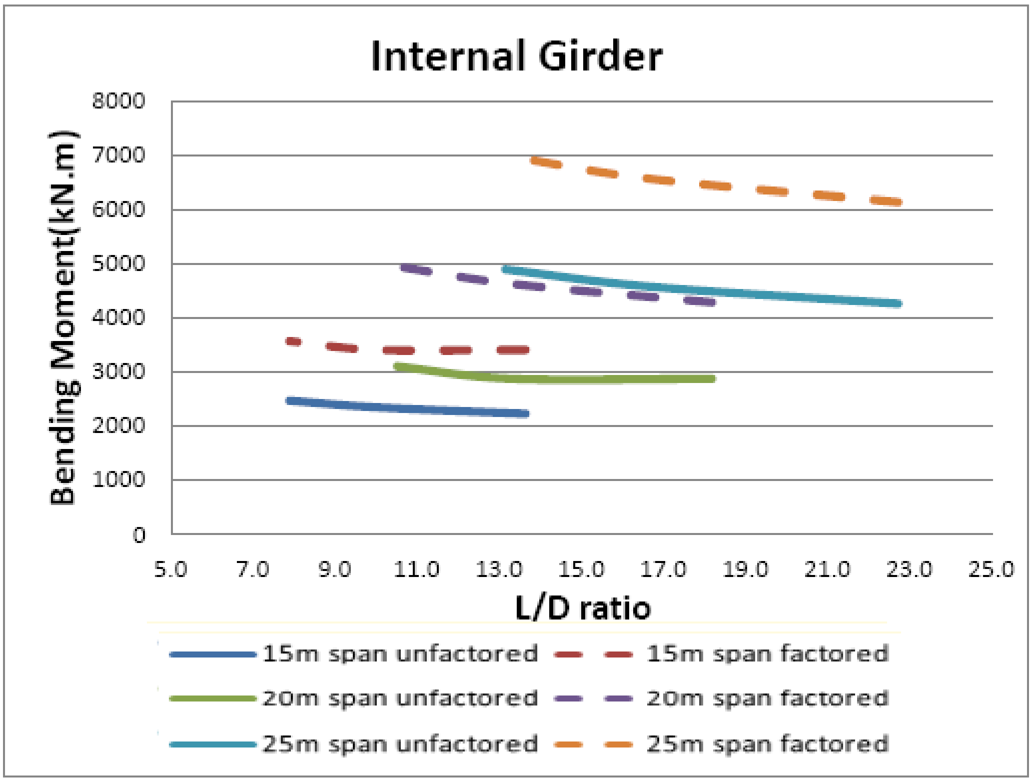

Figure 7.3: Bending moment vartiation in internal girder along various L/D ratio

Table 7.2: Maximum shear force in girder

			External girder(kN)		Internal girder(kN)	
Span	Depth	L/D ratio	Unfactored	Factored	Unfactored	Factored
(m)	(m)		force	force	force	force
15	1.1	14	518	759	325	476
	1.5	10	543	793	449	644
	1.9	8	569	828	480	685
20	1.1	18	681	1000	489	729
	1.5	13	723	1057	557	830
	1.9	11	758	1104	557	830
25	1.1	23	675	984	596	853
	1.5	17	718	1042	646	919
	1.9	13	760	1099	695	986

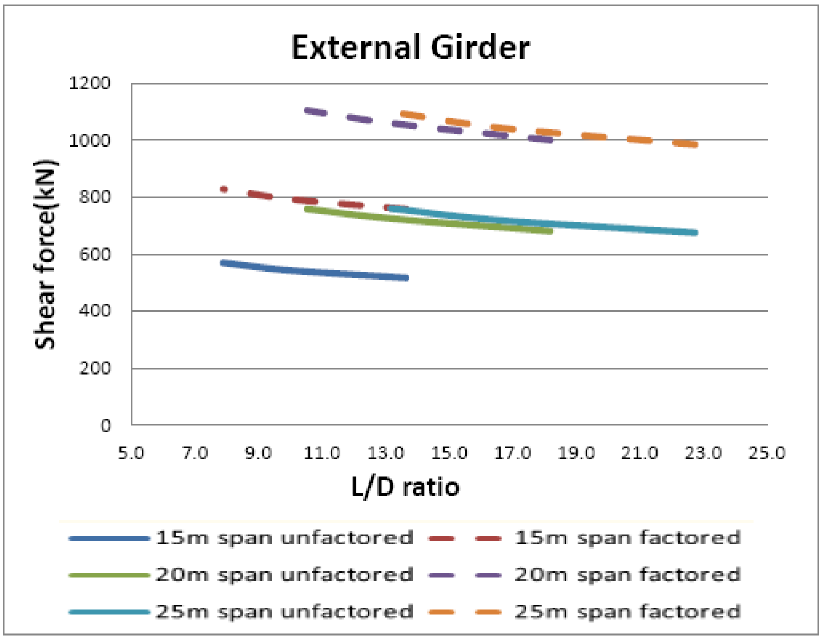

Figure 7.4: Shear force vartiation in external girder along various L/D ratio

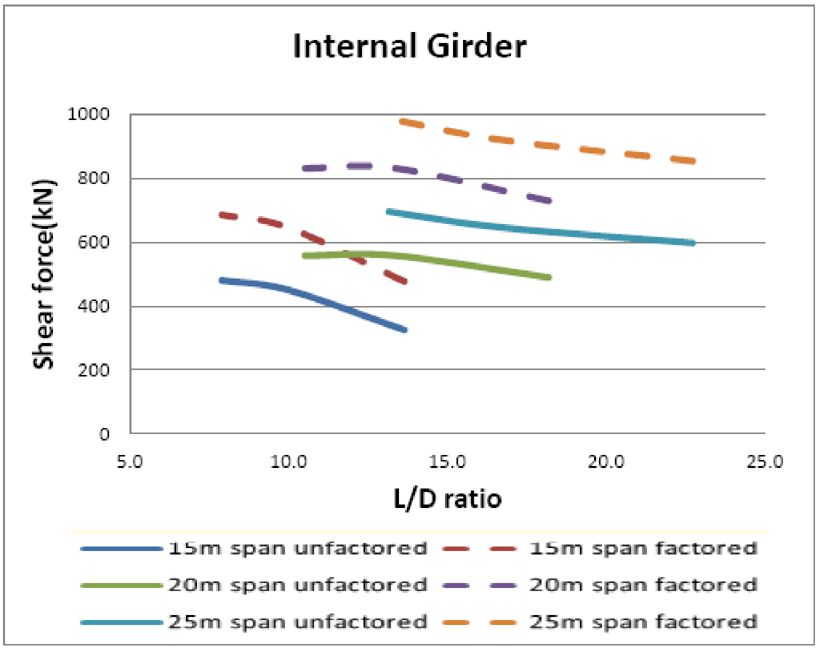

Figure 7.5: Shear force vartiation in internal girder along various L/D ratio

7.4 Results of costing

The design was done by prepared spreadsheet. The overall analysis methodology and step by step design procedure is described in chapter 5. For all the various spans and span to depth ratio deck slab and structural composite longitudinal girder is designed. For all various spans and depth slab and longitudinal girder designed. Intially the depth of longitudinal girder is selected on trial bases to study the behavior of of span 15m, 20m and 25m with three different depth 1.1m ,1.5m and 1.9m. Fig 7.6, 7.7 and 7.8shows the concrete cost, slab reinforcement cost, wearing coat cost and girder reinforcement Cost for 15m, 20m and 25m respectively.

With different L/D ratio. Table 7.3, 7.4 and 7.5 shows that the slab concrete, slab reinforcement and wearing coat cost does not affects the L/D ratio . From the Fig 7.6, 7.7 and 7.8 it is clear that total cost of super structure is mainly affected by girder reinforcement and concrete cost. It is commonly observed that as the depth increases the cost of concrete increases.From table 7.3 observerd that as the girder depth increases the steel cost decreases and from table7.4 as the depth increases steel cost first increases, decrsease then again increaes. L/D ratio 10 is most economical L/D ratio for 15m span 13 L/D ratio for 20m and 25m span among all L/D ratio alternatives. From table 7.4 L/D ratio beyound 17 is uneconomical and structure is even not safe in deflection and cracking.

Table 7.3: Variation of total cost of material for 15m span with varous L/D ratio

Span =15m					
Depth			1.1	1.5	1.9
L/D			**14**	**10**	**8**
Slab Concrete	Quant.	m3	26	26	26
	Cost	Rs	133016	133016	133016
Slab Reinforcement	Quant.	m2	3004	3004	3004
	Cost	Rs	120155	120155	120155
Girder Concrete	Quant.	m3	23	31	39
	Cost	Rs	119808	159283	198758
Girder Reinforcement	Quant.	kg	12198	11222	11291
	Cost	Rs	487921	448896	451647
Wearing coat	Quant.	tonne	17	17	17
	Cost	Rs	69795	69795	69795
Total Cost		**Rs**	103869	**944369**	973372

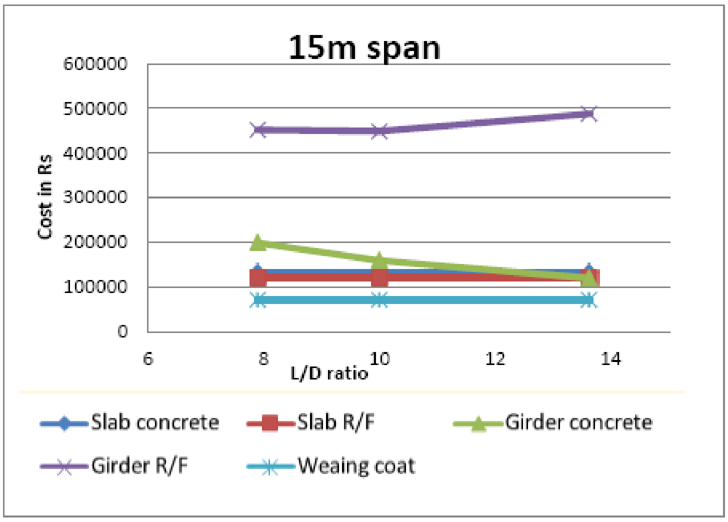

Figure 7.6: Variation of total cost of material for 15m span with varous L/D ratio

Table 7.4: Variation of total cost of material for 20m span with varous L/D ratio

Span =20m					
Depth			1.1	1.5	1.9
L/D			**18**	**13**	**11**
Slab Concrete	Quant.	m3	34	34	34
	Cost	Rs	175479	175479	175479
Slab Reinforcement	Quant.	m2	3963	3963	3963
	Cost	Rs	158504	158504	158504
Girder Concrete	Quant.	m3	30	40	50
	Cost	Rs	152404	203339	255800
Girder Reinforcement	Quant.	kg	16999	16284	17589
	Cost	Rs	679974	651341	703540
Wearing coat	Quant.	tonne	23	23	23
	Cost	Rs	93060	93060	93060
Total Cost		**Rs**	1259421	**1123218**	1386383

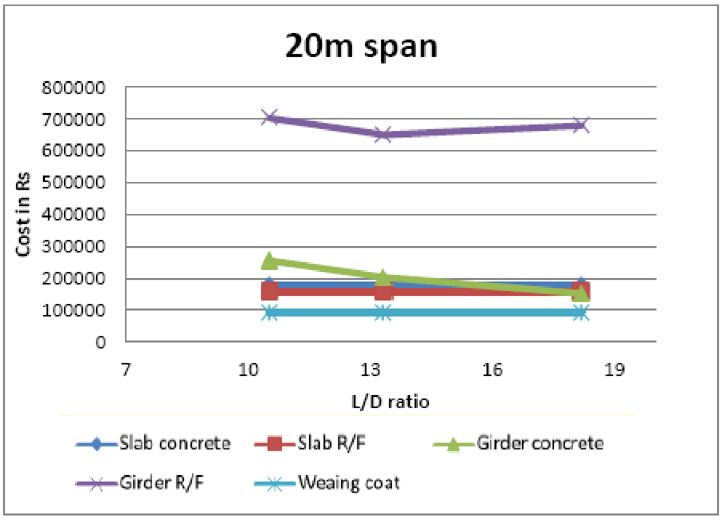

Figure 7.7: Variation of total cost of material for 20m span with varous L/D ratio

Table 7.5: Variation of total cost of material for 25m span with varous L/D ratio

Span =25m					
Depth			1.1	1.5	1.9
L/D			**23**	**17**	**13**
Slab Concrete	Quant.	m3	43	43	43
	Cost	Rs	219349	219349	219349
Slab Reinforcement	Quant.	m2	4921	4921	4921
	Cost	Rs	196852	196852	196852
Girder Concrete	Quant.	m3	35	46	58
	Cost	Rs	176656	235695	294733
Girder Reinforcement	Quant.	kg	19864	18333	16208
	Cost	Rs	794558	733331	648336
Wearing coat	Quant.	tonne	29	29	29
	Cost	Rs	116325	116320	116320
Total Cost		**Rs**	1503741	**1501547**	1475590

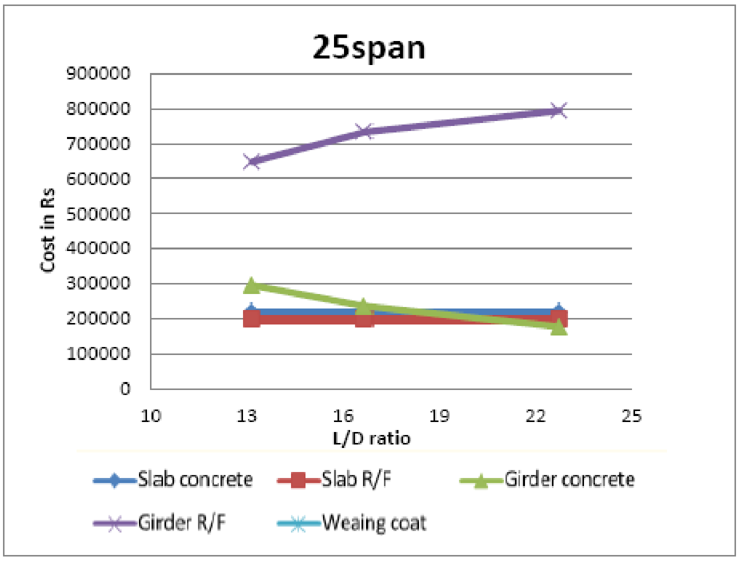

Figure 7.8: Variation of total cost of material for 25m span with varous L/D ratio

Table 7.6: Total cost of 15m, 20m and 25m span

Span	L/D Ratio	Total Cost (Rs)
15	14	1038694
	10	944369
	8	973372
20	18	1259421
	13	1123218
	11	1386383
25	23	1503741
	17	1501547
	13	1475590
	11	1551547

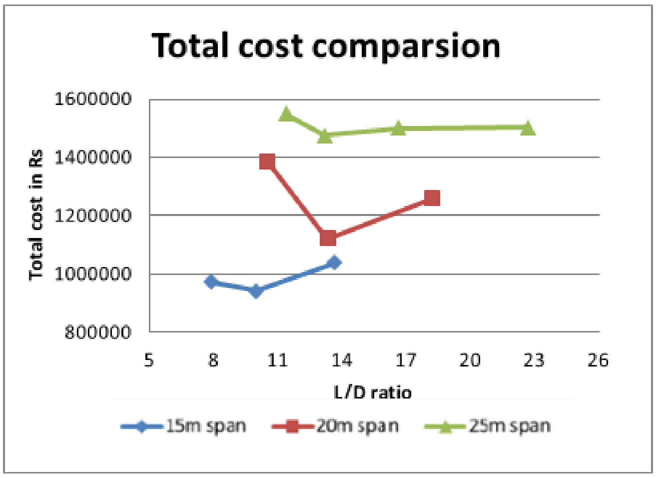

Figure 7.9: Total cost of material for 15m,20m and 25m span with varous L/D ratio

7.5 Total cost comparision of design method

Tables 7.7 shows total cost of concrete, reinforcement and wearing coat for working stress design(WSM), limit state design(LSM) and load and resistance design methods(LRFD) for 20m spans and 13 L/D ratio.It is found that the limit state method design were 7 percent costlier than working stress method design whereas LRFD is 11% high than the LSD.

Table 7.7: Total cost of comparision of WSD,LSD and LRFD

	Concrete	Reinforcement	Wearing coat	Total cost(Rs)
WSD	393932	692600	93060.0	1179592.0
LSD	378584	809845	93060.0	1281529.0
LRFD	419512	933880	93060.0	1446452.0

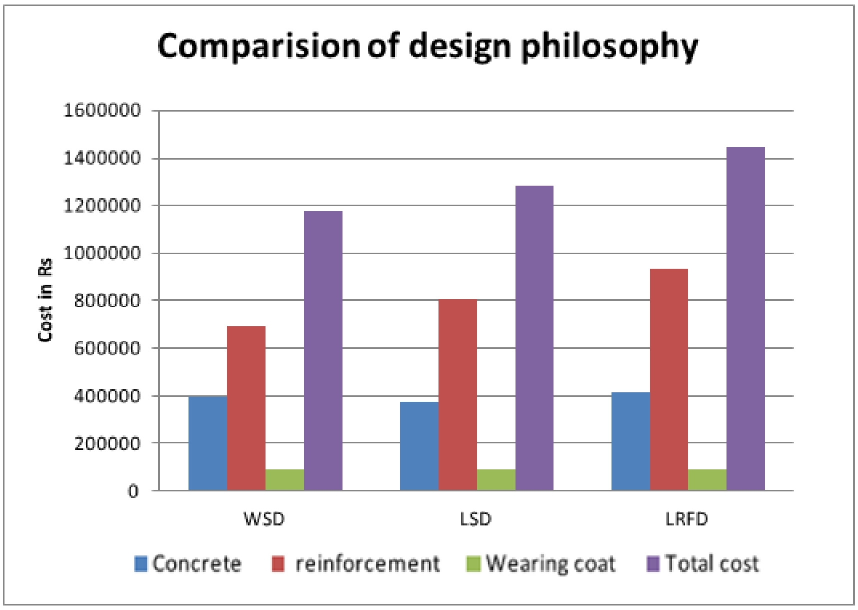

Figure 7.10: Total cost of material for 15m,20m and 25m span with varous L/D ratio

7.6 Summary

Parametric study is carried out to find out the effective economical L/D ratio for 15m, 20m and 25m. It is found that LID ratio 10, 13 and 13 are most economical L/D ratio for 15m, 20m and 25m spans respectively. Cost comparison between WSD design LSM design and LRFD design is carried out for economical L/D ratio 13 for

20m span. It is found that the LSD design were 7 percent costlier than WSD method whereas LRFD is 11% high than the LSD.

Chapter 8

Summary and Conclusion

8.1 Summary

The main objective of the work is to study the design philosophy in the R.C. fly-over bridge superstructures design of slab and girder and to find out the economical span to depth ratio for 15m, 20m and 25m span. R.C. Flyover Bridge is analysed using SAP2000 civil software. Excel spread sheets are prepared for deck slab design, longitudinal girder design as per IS 456:2000 and ASSTHO specification. In dead load self-weight of girder, slab, wearing coat and parapet load considered. And in live load Class-A and Class 70R IRC loading are considered. Total 9 alternatives are done using prepared spread sheet. R.C. girder is designed to satisfy moment capacity check, deflection check and crack check. And costs of all alternatives are carried out for various economical span and l/d ratio safe span to depth ratio.

For 15m span economical l/d ratio is 10, 20m span economical l/d ratio is 13 and for 25m span 13 l/d ratio is economical. Design of 20m span is carried out by limit state design using IS-456:2000 and LRFD using ASSTHO specifiction.

8.2 Conclusion

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

- The various limit state of flexure, shear, deflection and cracking in verified as per IS:456 code and the design is found satisfing all limit state.

- The design is carried out with ASSTHO specifications and it is found to be satifactory in limit state of flexure, shear, deflection and cracking control critria.

- The quantities and the total cost are compared for both the design and found that LRFD total cost 11% more than the LSD design.

- The parametric study is carried out to find the most economical (minimum cost) L/D ratio for span 15m,20m and 25m with L/D ratio as shown in table 7.3 ,7.4 and 7.5.

- It is observed from parametric study that for 15m span economical L/D ratio is 10, while for 20m and 25m span it remain same and equal to 13.

8.3 Future Scope

- To study of superstructure design as per limit state method can be futher carried out with boxgirder section.

- To study of superstructure design as per limit state method can be futher carried out with rectangular and trapzodial shape of composit box girder.

- Increasing the number of girders the study can be futher carried out for optimum number of girders.

Appendix A

Detailing sheet

- Detaling drawing of R.C.C deck slab.
- Detaling drawing of R.C.C longitudinal girder.
- Detaling drawing of R.C.C Diaphgram.

Appendix B

Paper communicated

- Sweta ojha and N.C.Vyas,Limit State Design Approach for Bridge Superstructure,*Dr.S.N.Patel's seminar for structural enginneering students,B.V.M Enginnering College,Vallabh Vidhyanagar, 7 January 2010*,

References

[1] S.G.Joglekar. New generation codes for bridge in india in line with internatonal standard. *Jounral of Indian National Group of International Association for Bridge and Structural Engineering*, April 2009,Vol.39.

[2] Dr.V.K.Raina. Rational approach to structural design of bridge and art of design education. *IE(I) Journal-CV*, Febuary 2004,Vol.84.

[3] Demetrious E.Tonias. *Design, Rehabilitation and Maintance of Modern Highway Bridge*. Fifth edition.

[4] Krishna N.Raju. *Design of Bridge*. Third edition.

[5] D.J.Victor. *Essential of Bridge Engineering*. Fifth edition.

[6] V.K.Raina. *Concrete Bridge Engineering*. Fifth edition.

[7] V.L.Shah and S.R.Karve. *Limit State Theory and Design of Reinforced Concrete*. Fifth edition.

[8] Richards M.Barker. *Design of highway bridges an LRFD Approach*. second edition.

[9] Edward G. Nawy. *Reinforced concrete*. fifth edition.

[10] *I.R.C-6 Standard Specification and Code of Practice for Road Bridge-Loads and Stresses*, fourth edition.

[11] *I.R.C:21-2000 Standard Specifications and Code of Practice for Road Bridge*, third revision edition.

[12] *I.R.C:6 (draft code) Standard Specification and Code of Practice for Road Bridge-Loads and Stresses*, amendment no.8 edition, June 2009.

[13] *Indian code practice for plain and reinforced concrete ,IS-456:2000*, fourth revision edition.

[14] *American Association of State Highway and Transportation Officials-2007*, fourth edition.