Design of Bridge Superstructure With Limit State Method as per IRC:112-2011

By

Nirav C. Vaghasia
11MCLC17

DEPARTMENT OF CIVIL ENGINEERING
AHMEDABAD-382481

May 2013

Design of Bridge Superstructure With Limit State Method as per IRC:112-2011

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

Nirav C. Vaghasia
11MCLC17

DEPARTMENT OF CIVIL ENGINEERING
AHMEDABAD-382481
May 2013

Declaration

This is to certify that

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

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

Nirav C. Vaghasia

Certificate

This is to certify that the Major Project entitled **Design of Bridge Superstructure With Limit State Method as per IRC:112-2011** submitted by **Mr.Nirav C. Vaghasia (11MCLC17)**, towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad is the record of work carried out by him under 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.

Mr.Harsh Morbia
Guide and Asst.Professor,
Department of Civil Engineering,
Institute of Technology,
Nirma University,
Ahmedabad.

Dr. 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 Examintion

Abstract

The past two decades have seen unprecedented growth of knowledge in the field of concrete bridges, development of new structural forms, new methods of computer-based analysis and design. 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 of great help in designing more aesthetic and durable bridges which will be constructed with ease, efficiency and economy and which will remain in service for much longer period.In present study, comparison of limit state design and working stress design is done.

In India bridge superstructure have traditionally been designed using working stress methods, but the new IRC:112-2011 Code now specifies a limit state design procedure for these structures. The main objective of this study was to compare working stress design (WSD) with limit states design (LSD) methods particular to bridge superstructure.

The main emphasis is given on limit state method design as per IRC:112-2011. By taking different span and bridge superstructure and analysis and design is carried out. The bridge superstructure is designed by limit state methods and compared with working stress design method to find most efficient design philosophy.

Acknowledgements

I would like to first of all thank **Shri. Himat Solanki** and **Prof. Harsh Morbia** Guide M.Tech. Department of Civil Engineering (CASAD) Institute of Technology, Nirma University, Ahmedabad whose keen interest and excellent knowledge base helped me to carry out the dissertation work. His constant support and interest in the subject equipped me with a great understanding of different aspects of the required architecture for the project work. He has shown keen interest in this dissertation work right from beginning and has been a great motivating factor in outlining the flow of my work.

My sincere thanks and gratitude to **Dr. P. V. Patel**, Professor, Department of Civil Engineering Institute of Technology, Nirma University, Ahmedabad for his continual kind words of encouragement and motivation throughout the Dissertation work.

I further extend my thanks to **Dr. P. H. Shah**, Head, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad and **Dr K. Kotecha**, Director, Institute of Technology, Nirma University, Ahmedabad for providing all kind of required resources during my study.

I would like to thank The Almighty, my family and all my friends, for supporting and encouraging me in all possible ways throughout the dissertation work.

-Nirav C. Vaghasia
11MCLC17

Abbreviation Notation and Nomenclature

A_c .... Cross sectional area of concrete

A_s .... Cross sectional area of reinforcement

$A_{s,min}$ .... Minimum cross sectional area of reinforcement

A_{sw} .... Cross sectional area of shear reinforcement

E_c . Tangent modulus of elasticity of normal weight concrete at a stress of σ_c= 0 and at 28 days

$E_{c,eff}$ .... Effective modulus of elasticity of concrete

E_{cm} .... Secant modulus of elasticity of concrete

E_s .... Design value of modulus of elasticity of reinforcing steel

M .... Bending moment

M_{Ed} .... Design value of the applied internal bending moment

SLS .... Serviceability limit state

T .... Torsional moment

T_{Ed} .... Design value of the applied torsional moment

ULS .... Ultimate limit state

V .... Shear force

V_{Ed} .... Design value of the applied shear force

b .... Overall width of a cross-section, or actual flange width in a T or L beam

b_w .... Width of the web on T, I or L beams

d .... Diameter;Depth

d .... Effective depth of a cross-section

f_{cd} .... Design value of concrete compressive strength

f_{ck} .... Characteristic compressive cylinder strength of concrete at 28 days

f_{cm} .... Mean value of concrete cylinder compressive strength

f_{ctk} .... Characteristic axial tensile strength of concrete

f_{ctm} .... Mean value of axial tensile strength of concrete

f_{tk} .... Characteristic tensile strength of reinforcement

f_y .... Yield strength of reinforcement

f_{yd} .... Design yield strength of reinforcement

f_{yk} .... Characteristic yield strength of reinforcement

f_{ywd} .... Design yield of shear reinforcement

h .... Overall depth of a cross-section

L .... Length;Span

t .... Thickness

t_0 .... The age of concrete at the time of loading

u .... Perimeter of concrete cross-section, having area Ac

x .... Neutral axis depth

z .... Lever arm of internal forces

α .... Angle;factor;ratio

β .... Angle;ratio;coefficient

γ_M .... Partial factor

ε_c .... Compressive strain in the concrete

ε_{c1} .... Compressive strain in the concrete at the peak stress f_c

ε_{cu} .... Ultimate compressive strain in the concrete

ε_u .... Strain of reinforcement or prestressing steel at maximum load

ε_{uk} .... Characteristic strain of reinforcement or prestressing steel at maximum load

θ .... Angle

υ .... Poisson's ratio

ν .... Strength reduction factor for concrete cracked in shear

ρ_w .... Reinforcement ratio for shear reinforcement

σ_c .... Compressive stress in the concrete

ϕ_n .... Equivalent diameter of a bundle of reinforcing bars

φ(t,t0) .... Creep coefficient, defining creep between times t and t0,related to elastic deformation at 28 days

φ(∞,t0) .... Final value of creep coefficient

Contents

List of Tables

List of Figures

Chapter 1

Introduction

1.1 General

Bridge is a structure which provides a passage to people, vehicles, railways or pipelines to cross various obstacles to travel. Engineers build bridges over obstacles such as lakes, rivers, canyons, and dangerous roads and railway tracks. Without bridges, people would need boats to cross waterways and would have to travel around canyons and ravines.

Bridges range in length from a few meters to several kilometers. They are among the largest structures built by man. The demands on design and on materials are very high. A bridge must be strong enough to support its own weight as well as the weight of the people and vehicles that use it. The structure also must resist various natural occurrences, including earthquakes, strong winds, and changes in temperature. Most bridges have a concrete, steel, or wood framework and an asphalt or concrete roadway on which people and vehicles travel.

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 analysis, are in vogue currently, the limit state approach being the latest trend.

1.2 History and Development of Bridges

The history and development of bridge is closely associated with the history of human civilization. The art of bridges therefore attracted the attention of engineer and builder from beginning of the civilization. It may be well presumed that the idea of building a bridge across an obstacle occurred in human mind by observing natural phenomenon such as tree trunk fallen accidentally by storm across a small water-course or a piece of stone in a form of an arch over a small opening caused by erosion of soil below and the creepers hanging from tree to tree allowing monkeys to cross from one bank to the other. These may be initial forebears of the arch and the suspension bridges. The primitive man imitated nature and learned to build beam and suspension bridges. The person, who deliberately cut a tree so that it fell across a stream and offered him a crossing, was the first bridge builder. Since the primitive man was a wanderer in search of food and shelter from the elements, the first type of structures he built was bridge. Men were setting down to community life and were giving more thought to permanence of bridges. Fig. 1.1 shows the development of bridges.

1.3 Objective of Study

a. To study the limit state method of design for R.C.C bridge superstructure by IRC:112-2011

b. To analyze and design bridge superstructure under IRC:6-2010 loading.

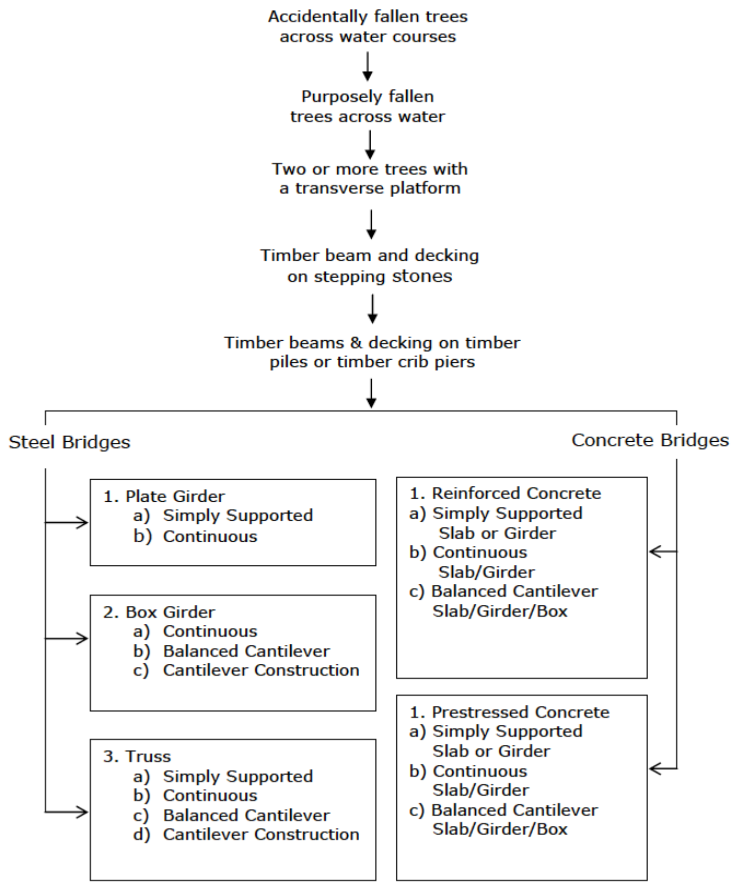

Figure 1.1: Development of various bridges from ancient times to the modern age

1.4 Scope of Work

a. Study of design philosophy i.e. working stress, limit state method.

b. Analysis of superstructure is carried out on STAAD-Pro. Software and design is done using excel worksheet.

c. Two Lane Bridge with three I-shaped and T-shape girders and solid deck slab with different span considered for analysis and design.

d. For analysis live load is considered are Class-70R tracked/wheeled and Class-A wheeled as per IRC: 6-2010.

e. For design of superstructure is done by using following codes:

IRC-6:2010, IRC-112:2011 and IRC-21:2000

f. Comparison of limit state design method and working stress design method.

1.5 Organization of Report

The Major Project is divided into six chapters. They are as follows:
Chapter 1 Includes the introductory part of thesis, objective and the methodology of the study.
Chapter 2 The literature review regarding information of various type of bridge, loading on bridge, superstructure introduction,and brief from research paper is covered.
Chapter 3 Design Philosophy Includes types of limit states
Chapter 4 Covers Analysis and design of deck slab,It also covers analysis and design procedure along with sample calculation and detailed drawings.
Chapter 5 includes Analysis of I- Girder for bridge superstructure. It covers design methodology along with one sample calculation, analysis, design and detailed drawings.
Chapter 6 includes parametric study for comparison of reinforcement and depth
Chapter 7 includes conclusion for all cases obtained from analysis and design. It also includes summary and future scope of work.

Chapter 2

Literature Review

2.1 General

Several books and research papers have been studied for analysis and design of bridge superstructure, brief review of these has been discussed below.

In addition to determining the geometry of a bridge, designers must consider various design elements. Basically, bridges are viewed from two perspectives. Traveling over the bridge deck, the driver of a vehicle sees the travel way, bridge railings, and the view to either side. If the bridge crosses over another roadway, water or land both on its side and underneath can also be viewed from this perspective. It is important for bridge designers to keep in mind that these two perspectives may require consideration of additional aesthetic treatments for the bridge.

For the design of the bridge deck, the major components include the width of the travel way and pedestrian and other non vehicular accommodations. Other components include railings, lighting fixtures, crash barrier and other design details.

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 India

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.

N.Krishna Raju[3] 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.

V. K. Raina[4] "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[5] 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.

D.J. Victor[6] describes theory and design of various bridge components in his book "Essentials of bridge Engineering". This book deals with the design of R.C.C bridge having T-shaped girder under IRC loading with reinforcement detailing.

C.S.Surana and R.Agrawal[7] This book deals with the well established computer-aided method of grillage analogy as applied to analysis of bridge decks. The method ,applicable to various types of bridge deck (such as slab bridges, T-beam bridges and box-girder bridges),can handle rigid or flexible support conditions,and right,skew or curved plan layouts.

IRC:6-2010[8] "Standard Specifications and Code of Practice for Road Bridge" Section-II Load and stress (fifth revision) is useful in application of vehicular load on the bridge structure.

IRC:112-2011[9] "Code of Practice for Concrete Road Bridges" This code strives to establish common procedures for the design and construction of concrete road bridges including footbridges In India. It covers design principles, detailed design criteria and practical rules, material specifications, workmanship , quality control and all such aspects which affect the characteristics/ability of the bridge to meet the aims. This code deals with the structural use of plain concrete, reinforced concrete, prestressed concrete and composite construction using concrete elements in bridges and is applicable to all structural elements using normal weight concrete. Requirement of blast resistance and fire resistance are not covered in the code.

IRC:21-2000[10]"Standard Specifications and Code of Practice for Road Bridge" Section-III Cement Concrete (Pain and reinforced) third revision is

K.S.Rakshit[11]"Design and construction of highway bridges" This book is dealing with the design,construction and maintenance of highway bridges.

BS EN 1992-2:2005[12] "Eurocode 2-Design of concrete structures Part-2:Concrete bridges-design and detailing rules" This code describes the principles and requirements for safety, serviceability and durability of concrete structures, together with specific provisions for bridges. It is based on the limit state concept used in conjunction with a partial factor method.

Chapter 3

Design Philosophy

3.1 Introduction

Limit state design of an engineering structure must ensure that (1) under the worst loadings the structure is safe, and (2) during normal working conditions the deformation of the members does not detract from the appearance, durability or performance of the structure. Despite the difficulty in assessing the precise loading and variations in the strength of the concrete and steel, these requirements have to be met. Three basic methods using factors of safety to achieve safe, workable structures have been developed over many years; they are

1. The permissible stress method in which ultimate strengths of the materials are divided by a factor of safety to provide design stresses which are usually within the elastic range.

2. The load factor method in which the working loads are multiplied by a factor of safety.

3. The limit state method which multiplies the working loads by partial factors of safety and also divides the materials' ultimate strengths by further partial factors of safety.

The permissible stress method has proved to be a simple and useful method but it does have some serious inconsistencies and is generally no longer in use. Because it is based on an elastic stress distribution, it is not really applicable to a semi-plastic

material such as concrete, nor it is suitable when the deformations are not proportional to the load, as in slender columns. It has also been found to be unsafe when dealing with the stability of structures subject to overturning forces.
In the load factor method the ultimate strength of the materials should be used in the calculations. As this method does not apply factors of safety to the material stresses, it cannot directly take account of the variability of the materials, and also it cannot be used to calculate the deflections or cracking at working loads. Again, this is a design method that has now been effectively superseded by modern limit state design methods.

The limit state method of design, now widely adopted across Europe and many other parts of the world, overcomes many of the disadvantages of the previous two methods. It does so by applying partial factors of safety, both to the loads and to the material strengths, and the magnitude of the factors may be varied so that they may be used either with the plastic conditions in the ultimate state or with the more elastic stress range at working loads. This flexibility is particularly important if full benefits are to be obtained from development of improved concrete and steel properties.
The purpose of design is to achieve acceptable probabilities that a structure will not become unfit for its intended use-that is, that it will not reach a limit stale, thus any way in which a structure may cease to be fit for use will constitute a limit slate and the design aim is to avoid any such condition being reached during the expected life of the structure. The two principal types of limit state are the ultimate limit slate and the serviceability limit slate.

3.1.1 Ultimate limit state

This requires that the Structure must be able to withstand, with an adequate factor of safety against collapse, the loads for which it is designed to ensure the safety of the building occupants and/or the safety of the structure itself. The possibility of buckling or overturning must also be taken into account, as must the possibility of accidental damage as caused, for example, by an internal explosion.

3.1.2 Serviceability limit states

Generally the most important serviceability limit states are:

1. Deflection - the appearance or efficiency of any part of the structure must not be adversely affected by deflections nor should the comfort of the building users he adversely affected.

2. Cracking - local damage due to cracking and spalling must not affect the appearance, efficiency or durability of the structure.

3. Durability - this must be considered in terms of the proposed life of the structure and its conditions of exposure.

Other limit states that may be reached include:

4. Excessive vibration - which may cause discomfort or alarm as well as damage.

5. Fatigue - must he considered ii cyclic loading is likely.

6. Fire resistance - this must be considered in terms of resistance to collapse, flame penetration and heat transfer.

7. Special circumstances - any special requirements of the structure which are not covered by any of the more common limit states, such as earthquake resistance, must be taken into account.

The relative importance of each limit stale will vary according to the nature of the structure. The usual procedure is to decide which is the crucial limit state for a particular structure and base the design on this, although durability and fire resistance requirements may well influence initial member sizing and concrete class selection. Checks must also be made to ensure that all other relevant limit states are satisfied by the results produced. Except in special cases, such as water-retaining structures, the ultimate limit state is generally critical for reinforced concrete although subsequent serviceability checks may affect some of the details of the design. Prestressed concrete design, however, is generally based on serviceability conditions with checks on the ultimate limit state.

In assessing a particular limit state for a structure it is necessary to consider all the possible variable parameters such as the loads, material strengths and all constructional tolerances.

3.2 Bending and the equivalent rectangular stress block

For most reinforced concrete structures it is usual to commence the design for the conditions at the ultimate limit state, followed by checks to ensure that the structure is adequate for the serviceability limit state without excessive deflection or cracking of the concrete.For this reason the analysis consider the simplified rectangular stress block which can be used for the design at the ultimate limit state. The rectangular

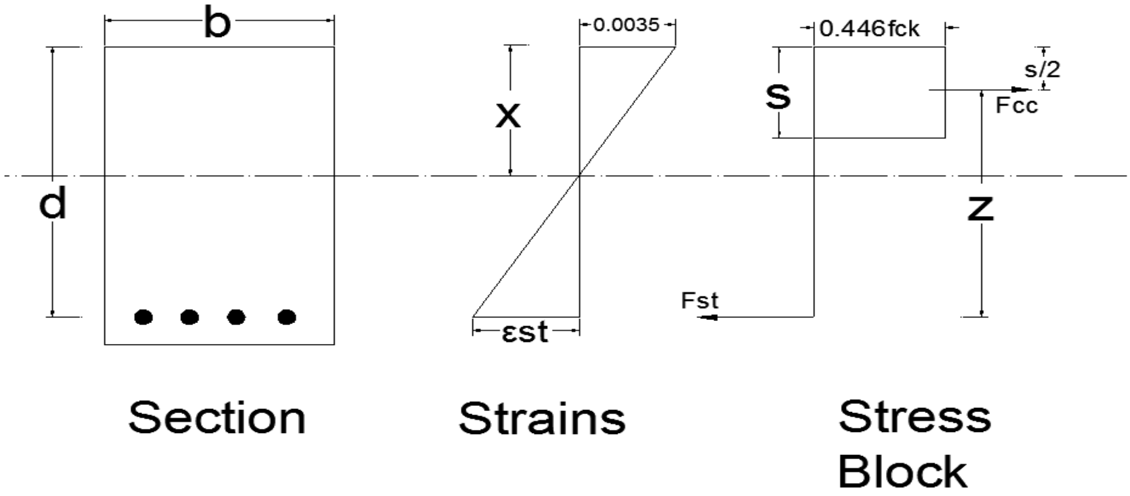

Figure 3.1: Rectangular Stress Block Diagram

stress block as shown in figure 3.1 may be used in preference to the more rigorous rectangular-parabolic stress block. This simplified stress distribution will facilitate the analysis and provide more manageable design equations, in particular when dealing with non-rectangular cross-sections or when undertaking hand calculations.
It can be seen from figure 3.1 that the stress block does not extend to the neutral axis of the section but has a depth $s = 0.8x$ This will result in the centroid of the stress block being $s/2 = 0.4x$ from the top edge of the section, which is very nearly the same location as for the more precise rectangular-parabolic stress block. The design equations derived in sections as following.

3.2.1 Design equation for bending

Bending of the section will induce a resultant tensile force in the rein forcing steel, and a resultant compressive force in the concrete F_{cc} which acts through the centroid of the effective area of concrete in compression,as shown in figure 3.1.
For equilibrium,the ultimate design moment,M, must he balanced by the moment of resistance of the section so that

$$M = F_{cc}z = F_{st}z \tag{3.1}$$

where z the lever arm between the resultant forces F_{cc} and F_{st}
F_{cc}= stress $\times$ area of action
$= 0.446 f_{ck} \times bs$
and

$$z = d - s/2 \tag{3.2}$$

so that substituting in equation 3.1
$M = 0.446 f_{ck} bs \times z$
and replacing s from equation 3.2 gives

$$M = 0.892 f_{ck} b(d - z)z \tag{3.3}$$

Rearranging and substituting $K = M/bd^2 f_{ck}$
$(z/d)^2 - (z/d) + K/0.892 = 0$
Solving this quadratic equation:

$$z = d[0.5 + \sqrt{(0.25 - K/0.892)}] \tag{3.4}$$

in equation 3.1
$F_{st} = (f_y/\gamma_s)A_s$ with $\gamma_s = 1.15$
Hence

$$A_s = \frac{M}{0.87 f_{yk} z} \tag{3.5}$$

Equations 3.5 can he used to design the area of tension reinforcement in a singly reinforced concrete section to resist an ultimate moment,M.
equation 3.4 for the lever arm z can also be used to draw a lever arm curve as shown in figure 3.2. This curve may he used to determine the lever arm, instead of solving equation 3.4

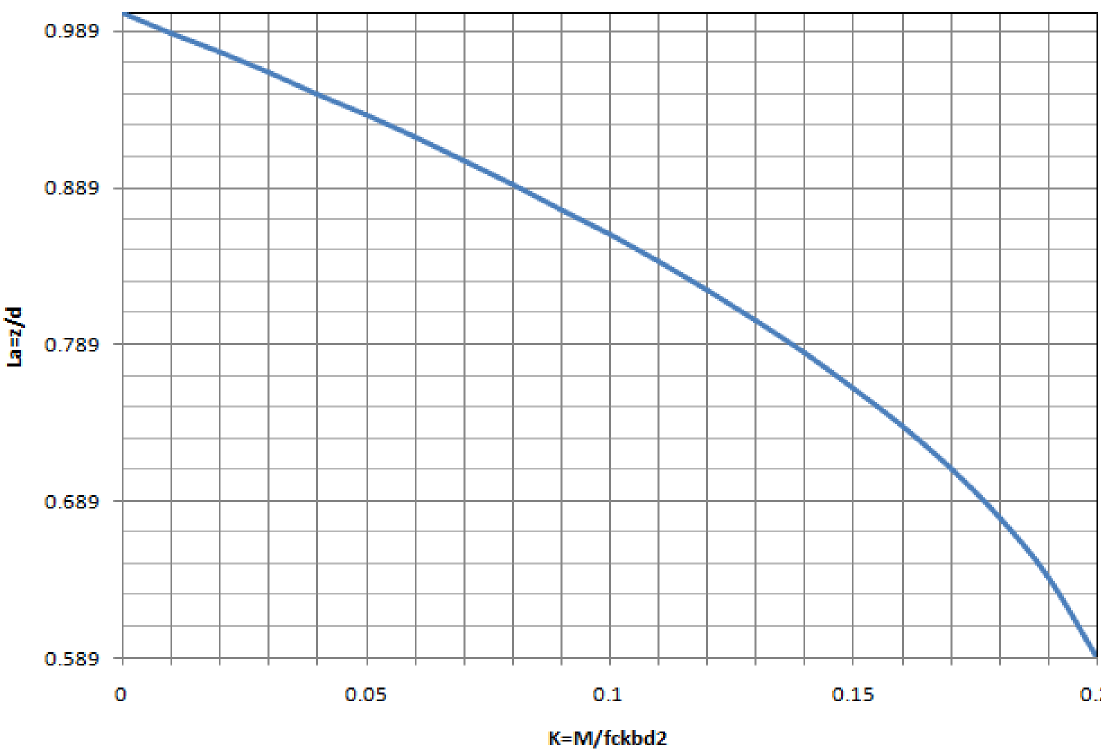

Figure 3.2: Lever-arm curve

3.3 Ultimate state of shear

Figure 3.3 represents the distribution of principal stresses across the span of a homogenous concrete beam. The direction of the principal compressive stresses takes the form of an arch, while the tensile stresses have the curve of a catenary or suspended chain. Towards mid-span. where the shear is low and he bending stresses are dominant. the direction of the stresses tends to be parallel to the beam axis. Near the supports. where the shearing forces are greater. the principal stresses become inclined and the greater the shear force the greater the angle of inclination. The tensile stresses due to shear are liable to cause diagonal cracking of the concrete near to the support so that shear reinforcement must be provided. This reinforcement is either in the form of stirrups or inclined bars (used in conjunction with stirrups). The concrete

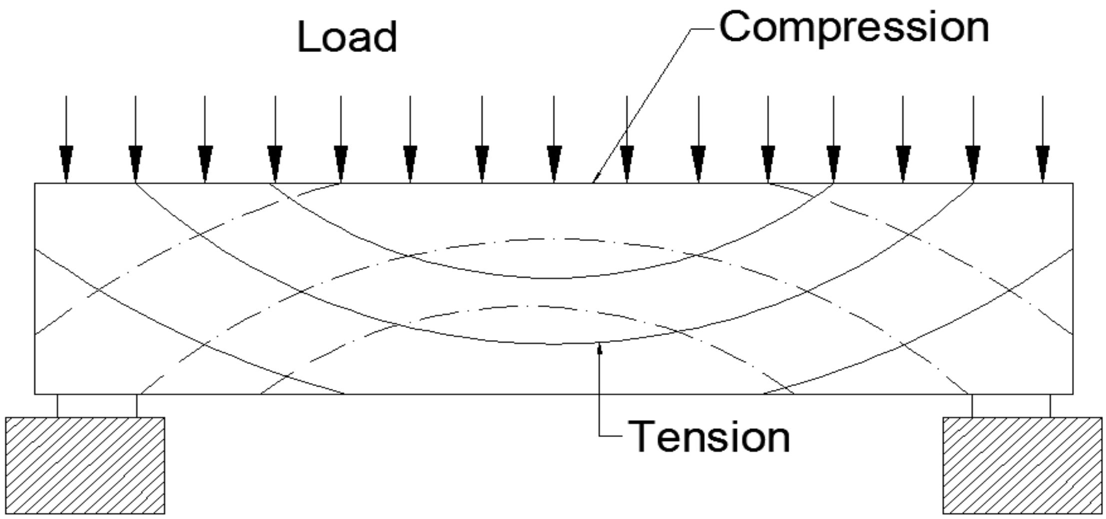

Figure 3.3: Principle stresses in a beam

itself can resist shear by a combination of the un-cracked concrete in the compression zone, the dowelling action of the bending reinforcement and aggregate interlock across tension cracks but, because concrete is weak in tension, the shear reinforcement is designed to resist all the tensile stresses caused by the shear forces.Even where the shear forces are small near the centre of span of a beam a minimum amount of shear reinforcement in the form of links must he provided in order to form a cage supporting the longitudinal reinforcement and to resist any tensile stresses due to factors such as thermal movement and shrinkage of the concrete.

The actual behaviour of reinforced concrete in shear is complex. and difficult to analyse theoretically, hut by applying the results from many experimental investigation, reasonable simplified procedures for analyses and design have been developed. In

IRC 112-2011 a method of shear design is presented which will be unfamiliar to those designers who have been used to design methods based on previous Indian Standard design codes. This method is known as *The Variable Strut Inclination Method*. The use of this method allows the designer to seek out economies in the amount of shear reinforcement provided.

3.3.1 Concrete section that do not require design shear reinforcement

The concrete sections that do not require shear reinforcement are mainly lightly loaded floor slabs and pad foundations or solid deck slab. Beams are generally more heavily loaded and have a smaller cross-section so that they nearly always require shear reinforcement. Even lightly loaded beams are required to have a minimum amount of shear links. The only exceptions to this are very minor beams such as short span. lightly loaded lintels over windows and doors.
Where shear forces are small the concrete section on its own may have sufficient shear capacity ($V_{Rd,c}$) to resist the ultimate shear force (V_{Ed}) resulting from the worst combination of actions on the structure. although in most cases a nominal or minimum amount of shear reinforcement will usually be provided.
In those section where $V_{Ed} \leq V_{Rd,c}$ then no calculated shear reinforcement is required.
The shear capacity of the concrete,$V_{Rd,c}$,in such situation is given by an empirical equation:

$$V_{Rd.c} = [0.12K(80\rho_1 f_{ck})^{0.33} + 0.15\sigma_{cp}]b_w d \tag{3.6}$$

with a minimum value of:

$$V_{Rd.c} = (v_{min} + 0.15\sigma_{cp})b_w d \tag{3.7}$$

where:

$V_{Rd,c}$=the design resistance of the section without shear reinforcement

$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$
where d is depth in milimeters
$v_{min} = 0.031K^{3/2}f_{ck}^{1/2}$
σ_{cp} is limited to $0.2f_{cd}$Mpa $\rho_1 = \frac{A_{sl}}{b_w d} \leq 0.02$
A_{sl} is the tensile reinforcement

3.3.2 The variable strut inclination method for sections that do require shear reinforcement

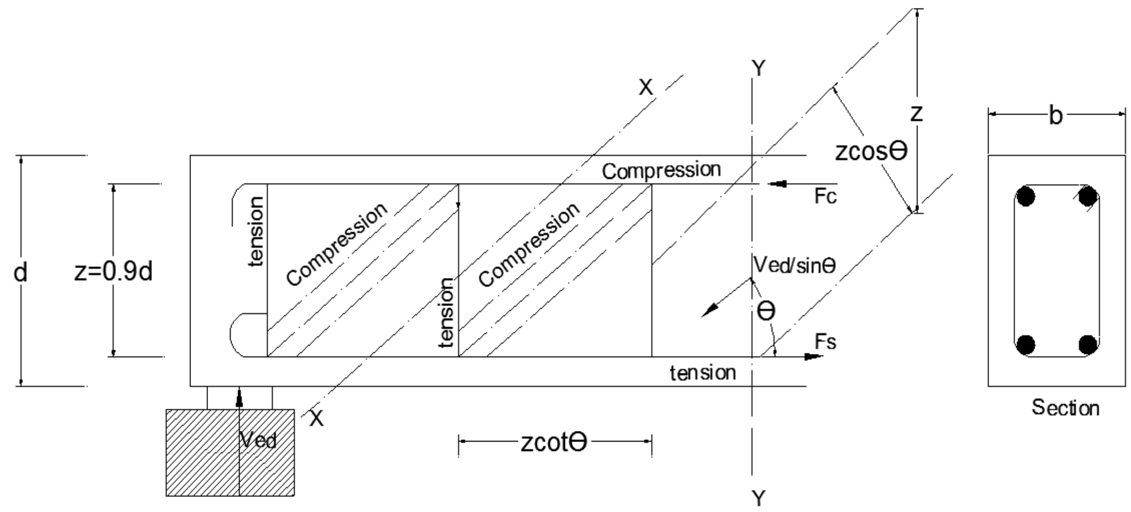

Figure 3.4: Assumed truss model for the variable strut inclined method

In order to derive the design equations the action of a reinforced concrete beam in shear is represented by an analogous truss as shown in figure 3.4 The concrete acts as the top compression member and as the diagonal compression members inclined at an angle θ to the horizontal The bottom chord is the horizontal tension steel and the vertical links are the transverse tension members.It should be noted that in this model of shear behaviour all shear will be resisted by the provision of links *with no direct contribution from the shear capacity of the concrete itself.*
The angle θ increases with the magnitude of the maximum shear force on the beam

and hence the compressive forces in the diagonal concrete members. It is set by IRC 112-2011 to have a value between **22** and **45** degrees. For most cases of predominately uniformly distributed loading the angle θ will be 22 degrees but for heavy and concentrated loads it can be higher in order to resist crushing of the concrete diagonal members.

The analysis of the truss to derive the design equations will be carried out in the following order:

a. Consideration of the compressive strength of the diagonal concrete strut and its angle θ;

b. Calculation of the required shear reinforcement A_{sw}/s for the vertical ties;

c. Calculation of the additional tension steel A_{sl} required in the bottom chord member;

The diagonal compressive strut and the angle θ

The shear force applied to the section must be limited so that excessive compressive stresses do not occur in the diagonal compressive struts, leading to compressive failure of the concrete. Thus the maximum design shear force $V_{Rd,max}$ is limited by the ultimate crushing strength of the diagonal concrete member in the analogous truss and its vertical component.

with reference to figure 3.4, the effective cross sectional area of concrete acting as the diagonal strut is taken as $b_w \times zcos\theta$ and the design concrete stress $f_{cd} = f_{ck}/1.5$.

The ultimate strength of the strut =ultimate design stress $\times$ cross-sectional area

$= (f_{ck}/1.5) \times (b_w \times zcos\theta)$

and its verticle component=$(f_{ck}/1.5) \times (b_w \times zcos\theta) \times sin\theta$

so that

$V_{Rd.c} = f_{ck}b_w cos\theta sin\theta/1.5$

which by conversion of the trigonometrical function can also be expressed as

$$V_{Rd.Max} = \frac{f_{ck}b_w z}{1.5(cot\theta + sin\theta)} \tag{3.8}$$

In IRC:112-2011 this equation is modified by the inclusion of a strength reduction factor v_1 for concrete cracked in shear
Thus

$$V_{Rd.Max} = \frac{f_{ck} b_w z v_1 \alpha_{cp}}{1.5(cot\theta + sin\theta)} \tag{3.9}$$

Where the strength reduction factor

$$v_1 = 0.6[1 - \frac{f_{ck}}{310}] \tag{3.10}$$

f_{ck} in MPa
and to ensure that there is no crushing of the diagonal compressive strut
$V_{Rd,max} \geq V_{Ed}$
This must he checked for the maximum value of shear on the beam, which is usually taken as the shear force, V_{Ed} at the face of the beams supports so that
$V_{Rd,max} \geq V_{Ef}$

The Vertical shear reinforcement

Using the method of sections it can be seen that,at section X-X in figure 3.4. the force in the vertical link member V_{wd} must equal the shear force V_{Ed},that is
$V_{wd} = V_{Ed} = f_{ywd} A_{sw}$
$= \frac{f_{ywd} A_{sw}}{1.15}$
$= 0.87 f_{yk} A_{sw}$
If the links are spaced at a distance s apart,then the force in each link is reduced proportionately and is given by
$V_{wd} \frac{s}{zcot\theta} = 0.87 f_{yk} A_{sw}$
or
$V_{wd} = V_{Ed}$
$= 0.87 \frac{A_{sw}}{s} z f_{yk} cot\theta$
$= 0.87 \frac{A_{sw}}{s} 0.9 d f_{yk} cot\theta$
Thus rearranging

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{0.78df_{yk}cot\theta} \tag{3.11}$$

3.3.3 Shear between the web and flange of a flanged section

The provision of shear links to resist vertical shear in a flanged beam is on the assumption that the web carries all of the vertical shear and that the web width,b_w,is used as the minimum width of the section in the relevant calculations.

Longitudinal complementary shear stresses also occur in a flanged section along the interface between the web and flange as shown in figure**??** This is allowed for by providing transverse reinforcement over the width of the flange on the assumption that this reinforcement acts as tics combined with compressive struts in the concrete. It is necessary to check the possibility of failure by excessive compressive stresses in the struts and to provide sufficient steel area to prevent tensile failure in the ties. The variable strut inclination method is used in a similar manner to that for the design to resist vertical shear in a beam.

The design is divided into the following stages:

1.Calculate the longitudinal design shear stresses, v_{Ed} at the web-flange interface.

The longitudinal shear stresses are at a maximum in the regions of the maximum changes in bending stresses that, in turn, occur at the steepest parts of the bending moment diagram. These occur at the lengths up to the maximum hogging moment over the supports and at the lengths away from the zero sagging moments in the span of the beam.

the change in the longitudinal force ΔF_d in the flange outstand at a section is obtained from

$$\Delta F_d = \frac{\Delta M}{(d - h_f/2)} \times \frac{b_{f0}}{b_f} \tag{3.12}$$

where

b_f= the effective breadth of the flange b_{f0} = the breadth of the outstand of the flange

$= (b_f - b_w)/2$ b_w=the breadth of the web h_f = the thickness of the flange ΔM= the change in moment over the distance Δx There fore

$$\Delta F_d = \frac{\Delta M}{(d - h_f/2)} \times \frac{b_f - b_w/2}{b_f} \tag{3.13}$$

The longitudinal shear stress,v_{Ed},at the vertical section between the outstand of the

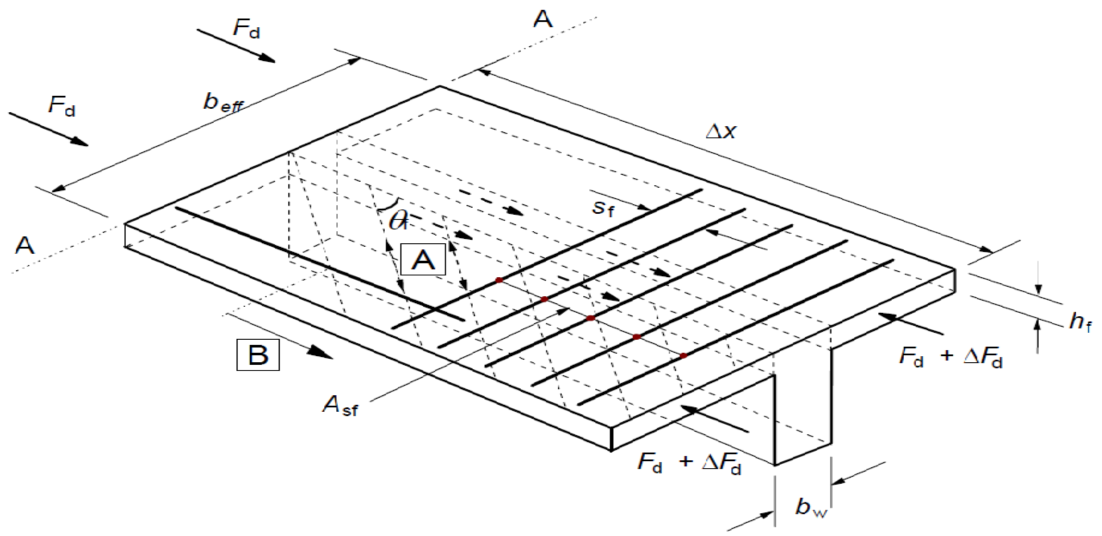

Figure 3.5: Shear between flange and web

flange and the web is caused by the change in the longitudinal force,ΔFd, which occurs over the distance Δx, so that

$$v_{Ed} = \frac{\Delta F_d}{h_f \times \Delta x} \tag{3.14}$$

The maximum value allowed for Δx is half the distance between the section with zero moment and that where maximum moment occurs. where point loads occur Δx should not exceed the distance between the loads.

If v_{Ed} is less than or equal to 0.4 f_{ctd} no extra reinforcement above that for flexure is required and proceed directly to step 4.

2. Check the shear stresses in the inclined strut

As before, the angle θ for the inclination of the concrete strut is restricted to a lower and upper value and IRC:112-2011 recommends that, in this case:
$1.0 \leq cot\theta_f \leq 2.0$ for compression flanges($45^0 \geq \theta_f \geq 26.5^0$)
$1.0 \leq cot\theta_f \leq 1.25$ for tension flanges($45^0 \geq \theta_f \geq 38.6^0$)
To prevent crushing of the concrete in the compressive struts the longitudinal shear stress is limited to: $v_{Ed} \leq v f_{cd} sin\theta_f cos\theta_f$ where the strength reduction factor $v = 0.6(1 - f_{ck}/310)$

3. Calculate the transverse shear reinforcement required

The required transverse reinforcement per unit length,A_{sf}/s_f,may be calculated from the equation:

$$\frac{A_{sf}}{s_f} \geq \frac{v_{Ed} h_f}{f_{yd} cot\theta_f} \tag{3.15}$$

4. The requirements of transverse steel

IRC:112-2011 requires that the area of transverse steel should be the greater of (a) that given by equation 5.l8 or (b) half that given by equation 5.l8 plus the area of steel required by transverse bending of the flange. The minimum amount of transverse steel required in the flange is $A_{s.min} = 0.26 b d_f f_{ctm}/f_{yk} > 0.0013 b d_f \ mm^2/m$ where b=1000 mm

3.4 Flexural cracking

Members subject to bending generally exhibit a series of distributed flexural cracks, even at working loads. These cracks are unobtrusive and harmless unless the width becomes excessive, in which ease appearance and durability suffer as the reinforcement is exposed to corrosion.
The actual widths of cracks in a reinforced concrete structure will vary between wide limits and cannot he precisely estimated, thus the limiting requirement to be satisfied is that the probability of the maximum width exceeding a satisfactory value

is small. The maximum acceptable value suggested by IRC:112-2011 is 0.3 mm for all exposures. Other codes of practice may recommend lower values of crack widths for important members and requirements for special cases, such as water retaining structures, may be even more Stringent.
Flexural cracking is generally controlled by providing a minimum area of tension reinforcement and limiting bar spacing or limiting bar sizes. If calculations to estimate maximum crack widths are performed, they are based on the quasi-permanent combination of loads and an effective modulus of elasticity of the concrete should he used to allow for creep effects.

3.4.1 Mechanism of flexural cracking

This can he illustrated by considering the behavior of a member subject to a uniform moment.
A length of bean as shown in figure 3.6 will initially behave elastically throughout, as the applied moment M is increased. when the limiting tensile strain for the concrete is reached, a crack will form and the adjacent tensile zone will no longer be acted on by direct tension forces. The curvature of the beam, however, causes further direct tension stresses to develop at some distance from the original crack to maintain equilibrium. This in turn causes further cracks to form, and the process continues until the distance between cracks does not permit sufficient tensile stresses to develop and cause further cracking. These initial cracks are called 'primary cracks'and the average spacing in a region or constant moment is largely independent of reinforcement detailing. As the applied moment is increased beyond this point. the development of cracks is governed to a large extent by the reinforcement. Tensile stresses in the concrete surrounding reinforcing bars are caused by bond as the strain in the reinforcement increases. These stresses increase with distance from the primary cracks and may eventually cause further cracks to form approximately mid-way between the prin1ar cracks. This action may continue with increasing moment until the bond between concrete and steel is incapable of developing sufficient tension in the concrete to cause further cracking in the length between existing cracks. Since the develop-

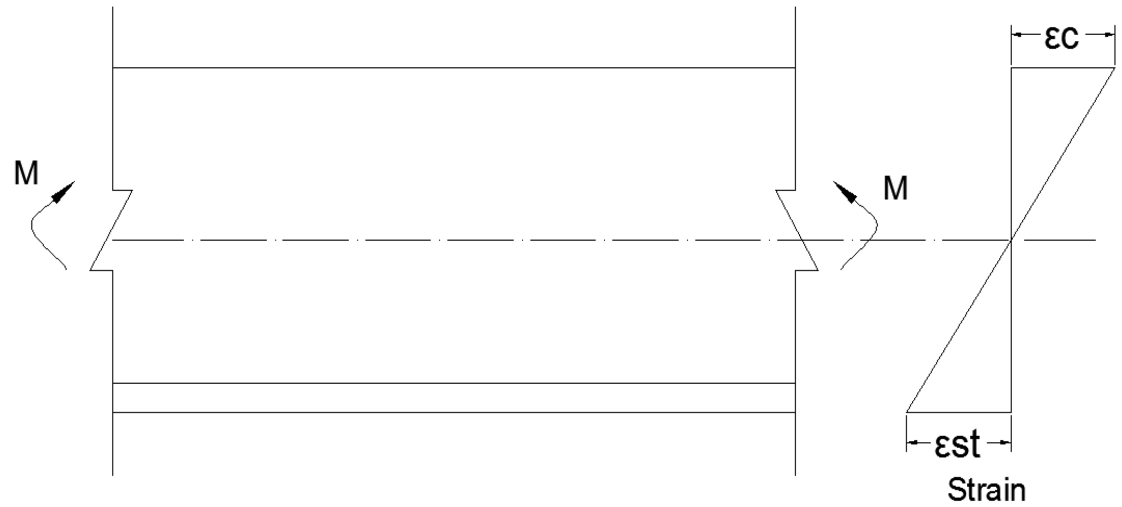

Figure 3.6: Bending of a length of beam

ment of the tensile stresses is caused directly by the presence of reinforcing bars, the spacing of cracks will be influenced by the spacing of the reinforcement.
If bars are sufficiently close for their 'zones of influence' to overlap then secondary cracks will join up across the member, while otherwise they will form only adjacent to individual bars. According to IRC:112-2011 the average crack spacing in flexural member depends in part On the efficiency of bond, the diameter of reinforcement bar used and the quantity and location of the reinforcement in relation to the tension face of the section.

3.4.2 Estimation of Crack Width

If the behavior of the member in figure 3.7 is examined, it can be seen that the overall extension per unit length at a depth y below the neutral axis is given by

$$\varepsilon_1 = \frac{1}{(d-x)}\varepsilon_s \tag{3.16}$$

where ε_s is the average strain in the main reinforcement over the length considered, and may be assumed to be equal to σ_s/E_s where σ_s is the steel stress at the cracked

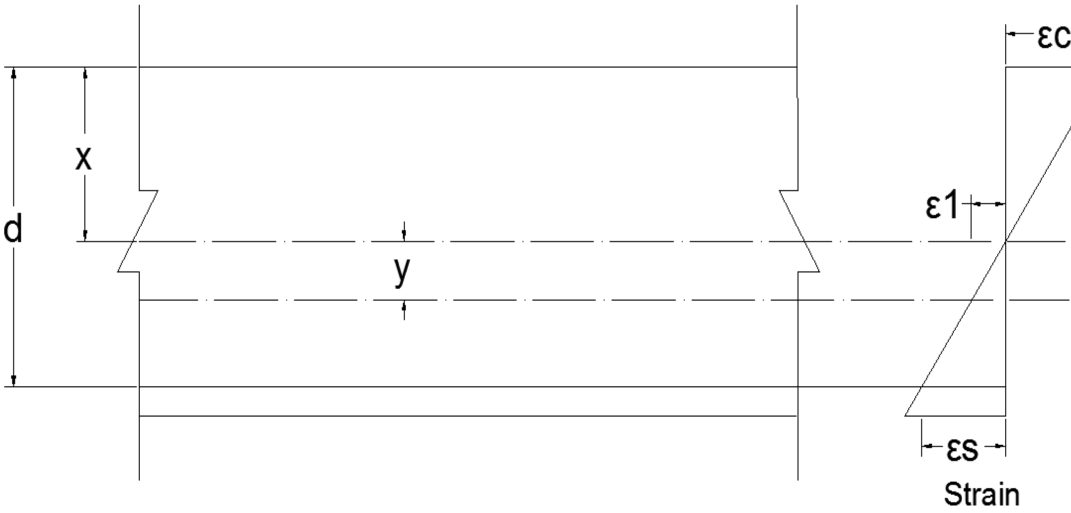

Figure 3.7: Bending strains

section. ε_1 is the strain at level y which by definition is the extension over the unit length of the member, Hence, assuming any tensile strain of concrete between cracks is small. since full bond is never developed, the total width of all cracks over this unit length will equate to the extension per unit length, that is

$$\varepsilon_1 = \frac{y}{d-x}\frac{\varepsilon_s}{E_s} = \Sigma w \tag{3.17}$$

where Σw = the sum of all crack widths at level y.
The actual width of individual cracks will depend on the number of cracks in this unit length, the average being given by unit length/average spacing s_{rm}.Thus average crack width

$$w_{av} = \frac{\Sigma w}{av.numberofcracks} \tag{3.18}$$

$$= \frac{\varepsilon_1}{\frac{1}{s_{rm}}} \tag{3.19}$$

$$= s_{rm}\varepsilon_1 \tag{3.20}$$

The designer is concerned however with the maximum crack width which has an acceptably low probability of being exceeded. For design purposes the design maximum crack width, w_k, can be based on the maximum spacing, $s_{r.max}$. Hence the design crack width at any level defined by y in a member will thus he given by

$$w_k = s_{r.max}\varepsilon_1 \tag{3.21}$$

The expression for the design crack width given in IRC: 112-2011 is of the above form and is given as

$$w_k = s_{r.max}(\varepsilon_{sm} - \varepsilon_{cm}) \tag{3.22}$$

where w_k=the design crack width
$s_{r.max}$=the maximum crack spacing
ε_{sm}=the mean strain in the reinforcement allowing for the effects of tension stiffening of the concrete, shrinkage etc.
ε_{cm}=the mean strain in the concrete between cracks
The mean strain, ε_{sm}, will be less than the apparent value ε_1 and $(\varepsilon_{sm} - \varepsilon_{cm})$ is given by the expression

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{sc} - k_t \frac{f_{ct,eff}}{\rho_{p.eff}}(1 + \alpha_e \rho_{p.eff})}{E_s} \geq 0.6\frac{\sigma_{sc}}{E_s} \tag{3.23}$$

where σ_s is the stress in the tension steel calculated using the cracked concrete Section.
k_t,is a factor dependent on the duration of the load which may be taken as 0.5
The maximum crack spacing, $S_{r.max}$, is given by the empirical formula

$$S_{r.max} = 3.4c + \frac{0.425k_1k_2\phi}{\rho_{\rho.eff}} \tag{3.24}$$

where ϕ is the bar size in mm or an average bar size where a mixture or different sizes have been used and c is the cover to the longitudinal reinforcement. k_1 is a factor that accounts of the bond properties of the bonded reinforcement (0.8 for deformed bars,1.6 for bars with an effectively plain surface) and k_2 is a coefficient accounting for the nature of the strain distribution which for cracking due to flexure can be taken as 0.5. $\rho_{\rho.eff}$, is the effective reinforcement ratio, $A_s/A_{c,eff}$ where A_s is the area of reinforcement within an effective tension area of concrete $A_{c,eff}$ as shown in figure 6. 12.

The effective tension area is that area of the concrete cross-section which will crack

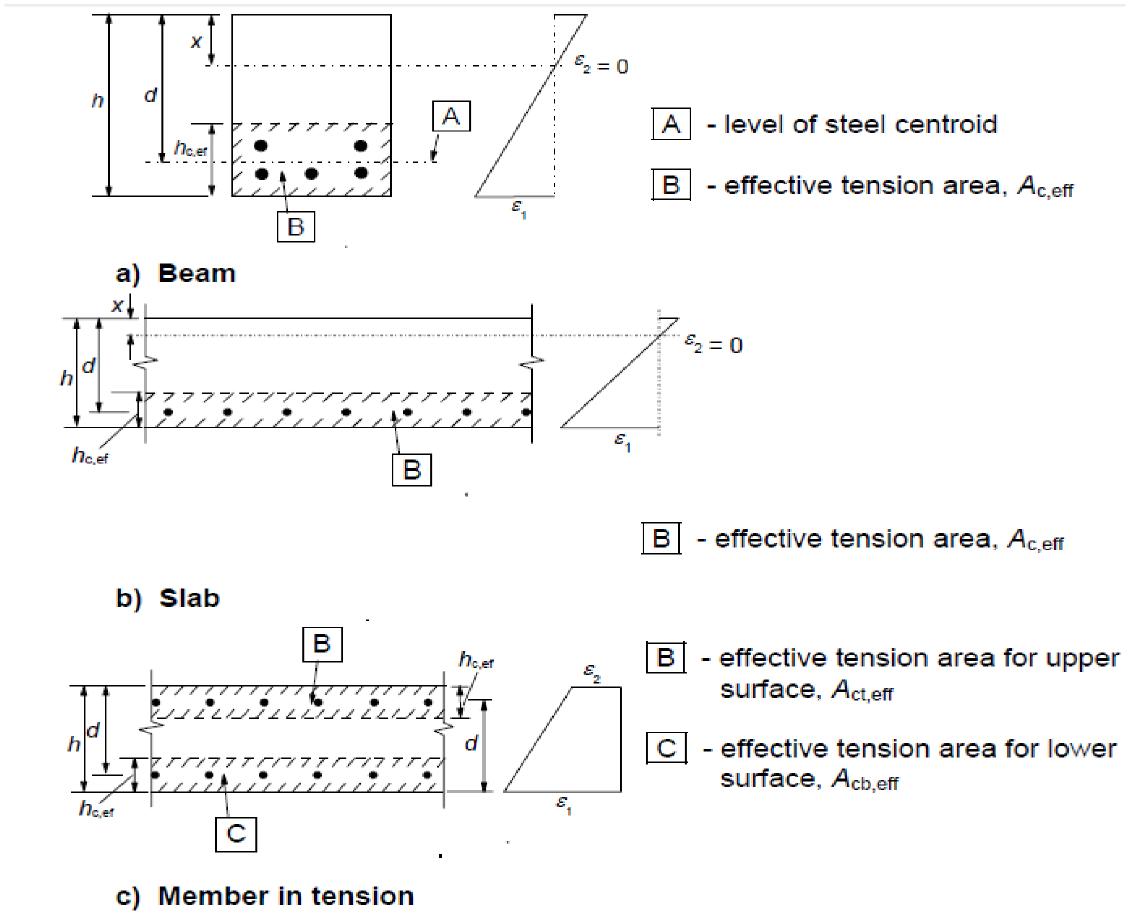

Figure 3.8: Effective tension area

due to the tension developed in bending. This is the cracking which will he controlled by the presence of an appropriate type, amount and distribution of reinforcement. Generally the effective tension urea should he taken as having a depth equal to 2.5 times the distance from the tension face of the concrete to the centroid of the reinforcement. although for slabs the depth of this effective area should he limited to$(h-x)/3$. An overall upper depth limit of $h/2$ also applies.

Although not directly incorporated into the above formulae. it should he noted that crack widths may vary across the width of the soffit of a beam and are generally likely to be greater at positions midway between longitudinal reinforcing bars and at the corners of the beam. Where the maximum crack spacing exceeds $5(c + \phi/2)$ then an upper bound to crack width can he estimated by using $S_{r.max} = 1.3(h - x)$

3.4.3 Control of crack widths

It is apparent from the expressions derived above that there arc tour fundamental ways in which surface crack widths may he reduced:

(1) reduce the stress in the reinforcement σ_s which will hence reduce ε_{sm}

(2) reduce the bar diameters ϕ which will reduce bar spacing and have the effect reducing the crack spacing $S_{r.max}$

(3) increase the effective reinforcement ratio $\rho_{\rho.eff}$

(4) use high bond rather than plain bars.

The use of steel at reduced stresses is generally uneconomical and, although the approach is used in the design of water-retaining structures where cracking must often he avoided altogether. it is generally easier to limit the bar diameters. increase $\rho_{\rho.eff}$ use high bond bars in preference to plain bars.

To increase $\rho_{\rho.eff}$ the effective tension area should be made as small as possible. This is best achieved by placing the reinforcement close to the tension face such that the depth of tension area $2.5(h-d)$ is made as small as possible recognizing, nevertheless, that durability requirements limit the minimum value of cover.

The calculation of the design crack widths indicated above only applies to regions within the effective tension zone. Since cracking can also occur in the side face of beam it is also good practice to consider the provision of longitudinal steel in the side faces of beams. Reinforcement detailing, however, has been shown to have a large effect on flexural cracking, and must in practice be a compromise between the requirements of cracking, durability and constructional ease and costs.

Chapter 4

Analysis and Design of Solid Deck Slab

4.1 General

In this simplest type of bridge superstructure, the deck slab also serves as the principal load carrying element. Slab bridges are easiest to construct and are frequently used for comparatively smaller spans. The form is very efficient at distributing point loads because of its two way spanning ability and high torsional strength. It is relatively easy to construct and this is reflected in its construction cost. The principal disadvantage is a its high self-weight which can be counteracted to some extent, by providing suitable variation in thickness or by providing voids. It may be reinforced concrete or of prestressed concrete. The concrete slab, which may be solid, voided, or ribbed, is supported directly on the substructures. Slab bridges require more concrete and steel than girder bridges of the same spans, but construction cost is usually lower and their formwork is simpler and less expensive.The limit of span of slab bridges depends on the magnitude of load and the relative costs of formwork, materials and labour. The small headroom under slab bridges can also have some bearing on economy through cheaper formwork.

Solid reinforced concrete slab of constant depth is normally used for spans upto 10 m (Fig. 4.1 a). For larger spans, say upto 15 m, haunching or variable depth is adopted

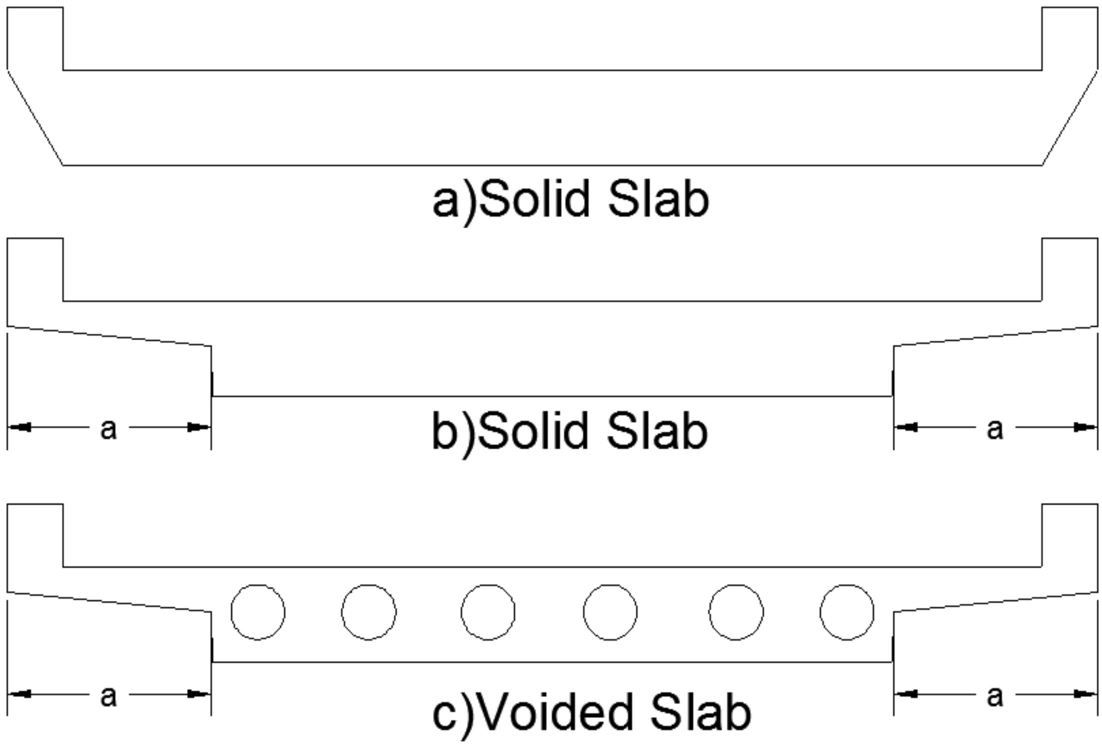

Figure 4.1: Slab Bridges

to reduce dead load (Fig. 4.1 b). A solid slab of uniform depth is preferred in highly skewed crossings, particularly if significant curvature and variation in width of the deck is involved.

Voided slab bridges (Fig. 4.1 c) are adopted to reduce the self weight of the bridge. The voids are usually circular or rectangular. The depth of voids is generally restricted to sixty per cent of the depth of the slab so that the slab continues to behave like a single plate. If this limit of void-depth is exceeded, the slab may behave more like a cellular deck. The voids may either run for the full span length or, alternatively, these may be provided in the central span length only so that solid section is available near the supports where shear is large.

Voided R.C. slabs with depth upto 100 cm may be adopted for span range of 8 to 15 m. However, for spans between 15 and 30 m, voided prestressed concrete slabs of depth upto 1.2 m are cheaper. For moderate skew crossings having spans of 15 to 25 m, this type of deck with longitudinal prestressing is useful but for highly skewed crossings, reinforced concrete decks are preferred for ease of construction. If the voided section is found inadequate in shear, it should be kept solid near supports.

In R.C. slab bridges, span-depth ratio of 15 for simple spans and 20 to 25 for Continuous spans are usually adopted for both solid and voided slabs. For cast-in-situ, prestressed concrete voided slab bridges, this ratio is nearly 30. In precast prestressed voided slabs, the ratio ranges between 25 and 30. The deck slab overhang, designated as 'a' in Figs. 4.1b and 1.1c may be provided to produce the desirable aesthetic effect and also to reduce the dead load and the width of sub-structure.

4.2 Loadings on Bridges

The design of the bridge superstructure is based on a set of loading condition which the component or element must withstand. The loading has profound effect upon the design, construction and eventually upon the cost of any bridge of a given span. Besides carrying their own weight, the bridge decks are designed for certain loadings imposed partly by the vehicles and the users and partly by nature. In order to maintain uniformity in design, loading standards have been laid down. In India, these standards for highway bridges (The Codes of Practices) are prepared by Indian Roads Congress (IRC), a statutory body formed by the government of India under the Ministry of Surface Transport. These codes are followed faithfully in the design of bridges.

The deck of the highway bridge has to support moving loads in the form of vehicles, men and materials and transmit their effects to the foundation. It has also to support and carry the self-weight of its various components. The structure is also subjected to vibrations under moving loads giving rise to the additional loads, which is known as impact loading. The details of some other loads and forces such as earthquake, wind etc. which also become important in some cases could be referred from the codes of practice.

The bridge engineer must take in to account a wide variety of loads which vary based on,

a. Duration (permanent or temporary)

b. Direction (vertical, longitudinal, etc.)

c. Deformation (concrete creep, thermal expansion, etc.)

d. Effect (shear, bending, torsion, etc.)

The following is a list of the main forces whose effects should be analyzed to estimate the load effects at all critical sections in the structure.

- Dead load of the structure Self weight may come in stages
- Live load On roadway, cycle tracks and footpaths
- Breaking force Generated by the application of brakes on the live load
- Wind load
- Earthquake force
- Lateral horizontal loads on parapets and kerbs
- Centrifugal force in horizontally curved decks
- Flood water current force in the bridge longitudinal and transverse directions
- Effect of afflux head
- Effect of cross-current force in bridge longitudinal direction
- Buoyancy effect
- Earth pressure
- Self-induced horizontal force caused at bearings by movement/rotation of deck due to temperature variation, creep and shrinkage of deck concrete, elastic-shortening of deck due to pre-stress, etc

- Thermal effect
- Secondary effects like effect of eccentric connections and shrinkage and creep of deck concrete
- Effect of possible differential settlement of supports
- Loads resulting from temporary erection conditions and partial span dislodgement condition

All members should be designed to sustain safely any combination of the above forces that can coexist. Typical combinations of loads and forces to be considered in design and allowable increases in permissible stresses for certain combinations are given in the code
Only the important loads to be used in the analysis of decks are described here.

4.2.1 Loading Requirements

The deck of the highway bridge has to support moving loads in the form of vehicles, men and materials and transmit their effects to the foundation. It has also to support and carry the self weight of its various components. The structure is also subjected to vibrations under moving loads giving rise to what is known as impact loading.
The details of some other loads and forces such as earthquake, wind etc. which also become important in some cases could be referred from the Codes of Practice [8,9]. Only the important loads to be used in the analysis of decks are briefly described here.

4.2.2 Dead Loads

The bridge superstructure is to be analysed for its self weight and dead loads imposed on it as well. The dead loads imposed on the bridge consist of permanent stationary load such as that of wearing coat, kerb, parapets etc. In estimating the dead loads, the unit weights of materials specified in reference [8] may be adopted.

Dead loads invariably form a relatively large loading component and result in significant design forces and deformations. It is, however, never a problem to either estimate these loadings accurately or compute their effects on the structure.

4.2.3 Live loads

The main loading on highway bridges is due to the vehicles moving on it, which are transient and hence difficult to estimate accurately. In order to analyse the bridge for these moving loads, IRC Code [7] recommends certain standard hypothetical loading systems. The bridge is then designed for the maximum response values under these standard loads.

The live loads usually consist of a set of wheel loads which are patch loads due to tyre contact area. These patch loads may be treated as point loads acting at the centre of the contact area. This simplification is found to be acceptable in the analysis.

According to Indian Roads Congress classification, the main live loads for road bridges can be put into the following four types [8]:

4.3 Data Specification

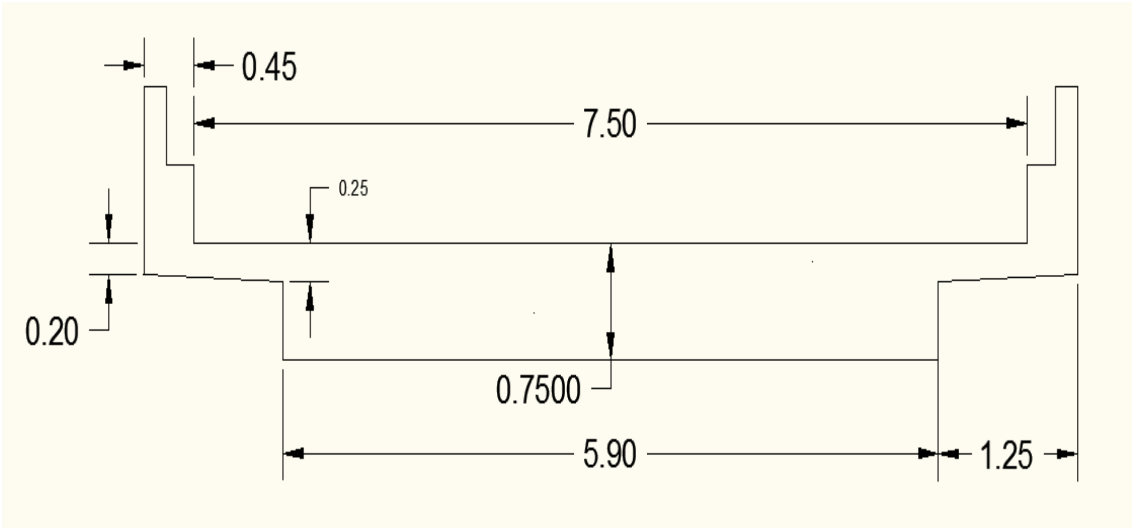

Figure 4.2: Cross section of solid deck slab

Effective Span= 15 m

Depth of slab= 750 mm

Width of slab= 8.4 m

Thickness of wearing coat at centre= 65 mm

Thickness of wearing coat at end= 52 mm Width of crash barrier= 450 mm

End clearance= 1200 mm

Length of cantilever= 1250 mm

Impact factor= 4.5/(6+L)= 0.21= 1.21

Consider dispersion width in transverse direction is equal to

(width of span−length of cantilever)

$\therefore$ B= 5.9 m

Characteristic strength of concrete=fck= 40 MPa

Characteristic strength of concrete=fyk= 415 MPa

4.3.1 Dead Load of Superstructure

(a) Self weight of deck

(1)0.5(0.2+.25)×1.25 × 2 × 2.5 = 1.41 t/m

(2)0.75 × 5.9 × 2.5 = 11.06 t/m

(b) Weight of wearing coat

0.5(0.065+0.052)×2.2 =0.1287 t/m^2 (min.0.2 t/m^2) = 1.5 t/m

(c) Crash barrier

2 × 0.29 × 2.5= 1.45 t/m

Total Dead Load = 15.42 t/m

4.3.2 Analysis of Solid Deck Slab for Live Load

Analysis of solid deck slab is done using staad pro and spreadsheet.Carriageway width of deck slab is 7.5m. Therefore Load combination as per IRC6:2010,one lane of 70R or two lanes of Class A vehicle is taken for analysis. From that 70R wheeled vehicle is governing for maximum bending moment and shear force.Therefore calculation for 70R Wheeled vehicle is as shown following section.

4.3.3 Longitudinal Placement for Maximum Bending Moment

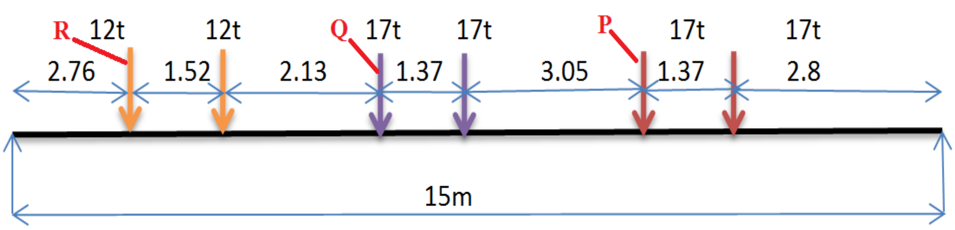

Figure 4.3: Loading for max. bending moment

From figure 4.6

1.For 12t axle load(R)

Load on 1 tyre(p)= 6 t

Width= 150+(p-1)57= 435 mm (IRC 6:2010 Annex-A(2) Pg.61)

contact width= 50 mm (IRC 6:2010 Annex-A(2) Pg.61)

Width= 435 - 50=385 mm

Maximum tyre pressure=5.273 kg/cm^2 (IRC 6:2010 Annex-A Pg.58)

Breadth of tyre= 295.55 mm

Length of dispersion along the span= 1.92 m > 1.52 m (c/c distance between 12t Axles)

Total length of dispersion along span= 3.44 m

2.For 17t axle load(P,Q)

Load on 1 tyre(p)= 8.5 t

Width= 150+(p-1)57= 577.5 mm (IRC 6:2010 Annex-A(2) Pg.61)

Contact width= 50 mm (IRC 6:2010 Annex-A(2) Pg.61)

Width= 577.5-50=527.5 mm

Maximum tyre pressure=5.273 kg/cm^2 (IRC 6:2010 Annex-A Pg.58)

Breadth of tyre= 305.589 mm

Length of dispersion along the span= 1.93 m > 1.37 m (c/c distance between 17t Axles)

Total length of dispersion along span= 3.30 m

Transverse placement for Max.BM

For 17t axle load(Q)

c/c dist between tyres = 2.790-0.860= 1.93 m

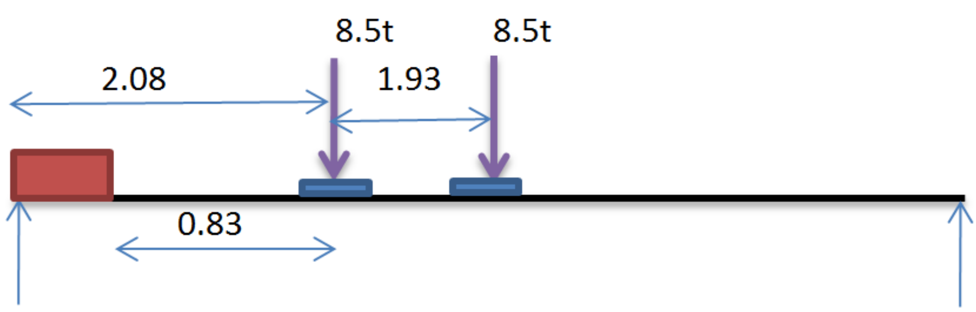

Figure 4.4: Transverse placement of 17t(Q) Axles

Calculation of effective width

$$b_{eff} = \alpha a(1 - \frac{a}{l_0}) + b_1 \tag{4.1}$$

(IRC 112-2011 Annex B-3 Pg.278)

b_{eff}= the effective width of the slab on which the load acts,

l_0= effective span= 15 m

a=the distance of the c.g. of concentrated load from the nearer support,

a= 7.095 m

b_1= the breadth of concentration area of the load

b_1= 0.43 m

b/l_0= 0.39

α= 1.352

b_{eff}= 5.49 m

Total effective width across the span= 5.50 m

Intensity of the load= 2.26 t/m^2

For 17t axle load(P)

c/c dist between tyres = 2.790-0.860= 1.93 m

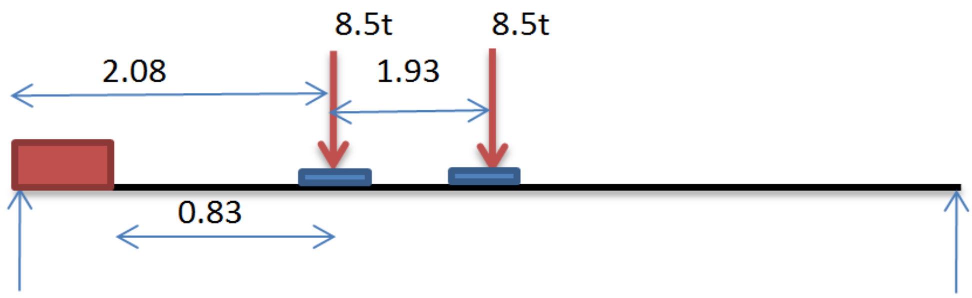

Figure 4.5: Transverse placement of 17t(P) Axles

Calculation of effective width

From equation(4.1)

l_0= effective span= 15 m

a= 3.485 m

b_1= 0.43 m

b/l_0= 0.39

α= 1.352

b_{eff}= 4.05 m

Total effective width across the span= 4.78 m

Intensity of the load= 2.61 t/m^2

For 12t axle load(R)

c/c dist between tyres = 2.790-0.860= 1.93 m

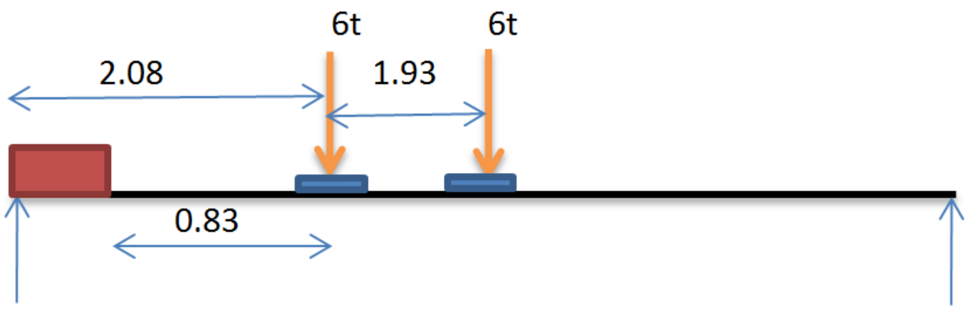

Figure 4.6: Transverse placement of 12t(R) Axles

Calculation of effective width

From equation(4.1)

l_0= effective span= 15 m

a= 3.52 m

b_1= 0.43 m

b/l_0= 0.39

α= 1.352

b_{eff}= 4.07 m

Total effective width across the span= 4.79 m

Intensity of the load= 1.76 t/m^2

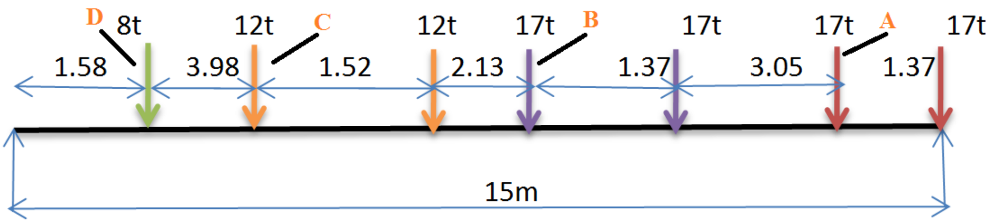

Figure 4.7: Loading for max. shear force

4.3.4 Longitudinal placement for max SF

1.For 12t axle load(C)

Total length of dispersion along span= 3.44 m

2.For 17t axle load(B)

Total length of dispersion along span= 3.30 m

3.For 17t axle load(A)

Total length of dispersion along span= 2.34 m

4.For 8t axle load(D)

Load on 1 tyre(p)= 4 t

Width= 150+(p-1)57= 321 mm (IRC 6:2010 Annex-A(2) Pg.61)

Contact width= 50 mm (IRC 6:2010 Annex-A(2) Pg.61)

Width= 271 mm

Max.tyre pressure=5.273 kg/cm^2 (IRC 6:2010 Annex-A(2) Pg.61)

Breadth of tyre= 279.92 mm

Length of dispersion along the span= 1.90 m > 3.98 m Hence, No Overlap

Total length of dispersion along span= 1.90 m

Transverse placement for Max.SF

For 17t axle load(B)

c/c dist between tyres = 2.790-0.860= 1.93 m

Calculation of effective width

From Equation (4.1)

l_0= effective span= 15 m

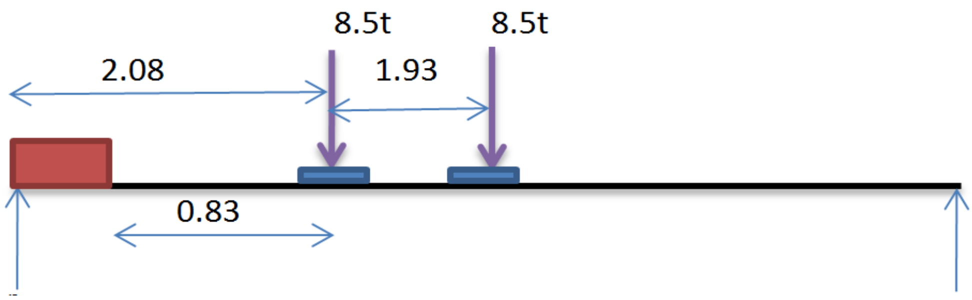

Figure 4.8: Transverse placement of 17t(B) Axles

a= 5.105 m

b_1= 0.43 m

b/l_0= 0.39

α= 1.42

b_{eff}= 5.22 m

Total effective width across the span= 5.37 m

Intensity of the load= 2.33 t/m^2

For 17t axle load(A)

c/c dist between tyres = 2.790-0.860= 1.93 m

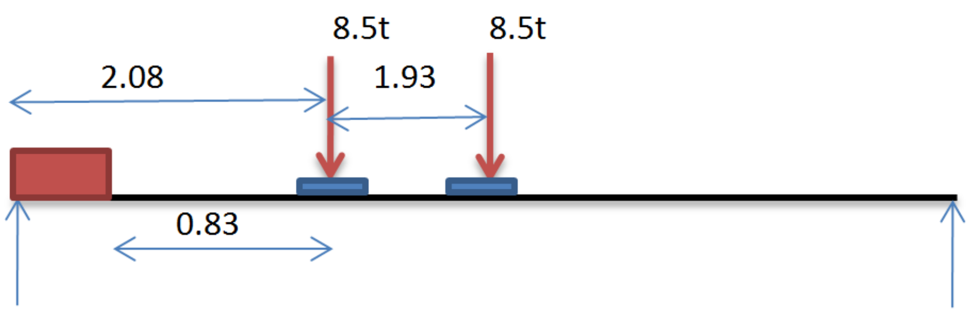

Figure 4.9: Transverse placement of 17t(A) Axles

Calculation of effective width

From equation (4.1)

l_0= effective span= 15 m

a= 0.685 m

b_1= 0.43 m

b/l_0= 0.39

α= 1.42

b_{eff}= 1.36 m

Total effective width across the span= 3.44 m

Intensity of the load= 3.62 t/m^2

For 12t axle load(C)

c/c dist between tyres = 2.790-0.860= 1.93 m

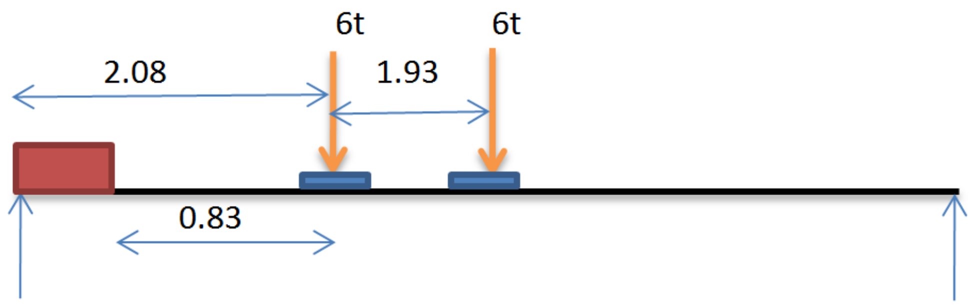

Figure 4.10: Transverse placement of 12t(C) Axles

Calculation of effective width

From equation (4.1)

l_0= effective span= 15 m

a= 6.32 m

b_1= 0.43 m

b/l_0= 0.39

α= 1.42

b_{eff}= 5.62 m

Total effective width across the span= 5.57 m

Intensity of the load= 1.52 t/m^2

For 8t axle load(D)

c/c dist between tyres = 2.790-0.860= 1.93 m

Calculation of effective width

From equation (4.1)

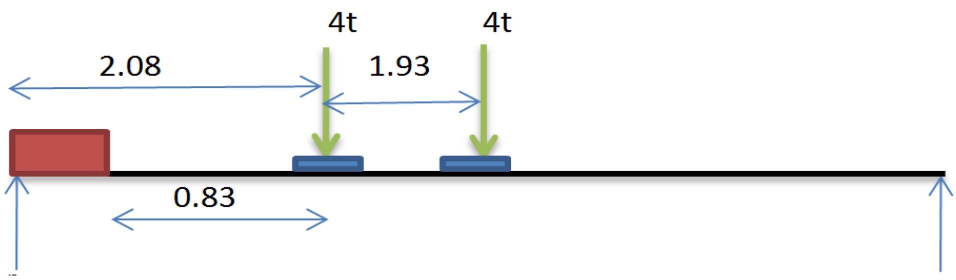

Figure 4.11: Transverse placement of 8t(D) Axles

l_0= effective span= 15 m

a= 1.58 m

b_1= 0.40 m

b/l_0= 0.39

α= 1.42

b_{eff}= 2.42 m

Total effective width across the span= 3.97 m

Intensity of the load= 1.28 t/m^2

4.3.5 Effective UDL for Maximum Forces

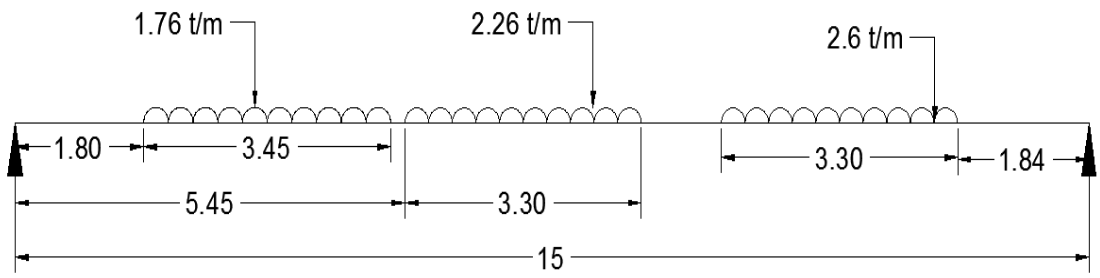

Figure 4.12: Effective UDL for maximum forces

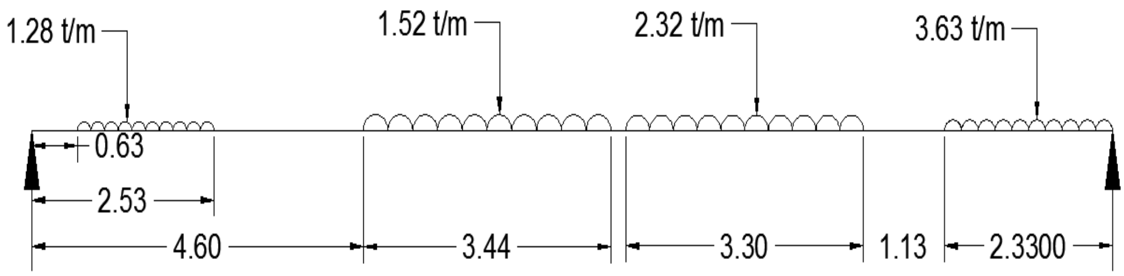

Figure 4.13: Loading for Maximum live shear force

Table 4.1: Analysis Result Unfactored

	Bending Moment $kN.m$	**Shear force** kN
Dead Load	720.90	235.75
Live Load	493.74	150.10

4.4 Limit State Design of Solid Deck slab

4.4.1 Data

BM due to Live Load= 493.71 $kN.m$

BM due to Dead Load= 720 $kN.m$

Partial safety factor for Live Load=γ_m=1.5

Partial safety factor for Dead Load=γ_m=1.35

Design BM due to Live Load= 740.61 $kN.m$

Design BM due to Dead Load= 972 $kN.m$

f_{ck}= 40 N/mm^2

f_{yk}= 415 N/mm^2

D= 950 mm

clear cover=c= 50 mm

d= 884 mm

width= 8400 mm

cantilever= 1250 mm

solid slab= 5900 mm

depth of cantilever= 250 mm

b= 1000 mm (Unit strip)

Total Design BM= 1712.61 $kN.m$

4.4.2 Ultimate capacity of Deck Slab

1.Design Compressive strength

$f_{cd} = \frac{\alpha f_{ck}}{\gamma_m}$ (IRC-112:2011,Cl.6.4.2.8,Pg.49)

$\alpha = 0.67$

$\gamma_m = 1.5$ for basic and seismic combination

$= 1.2$ for accidental combination

$f_{cd} = 17.87\ N/mm^2$

2. Assume Tension Reinforcement

Diameter of Bar $\phi = 32\ mm$

Min.$A_{st} = 0.26\ \times f_{ctm} \times b \times d/fy$ (IRC-112:2011,Cl.16.6.1.1,Pg.181)

Min.A_{st}= 1755.46988 mm^2

Max spacing= 2h or 250 mm

Spacing= 90 mm

Ast= 8,931.56 mm^2

3.Find Depth of neutral Axis

Depth of neutral axis=X

$X = \frac{1.305 f_y A_{st}}{\lambda \eta f_{cb} b}$ (IRC-112:2011,Annex-A2.9(2),Pg.242)

λ= 0.8

η= 1

X= 338.42 mm

4.Moment Capacity Check

BM= 1977.62 $kN.m$ > 1712.61 $kN.m$ $\therefore$ **OK**

4.4.3 Check for Serviceability

(1)Before creep has occurred the cracked section properties will be based on the short term modulus for all action

E_{cm}= 33 GPa (IRC-112:2011,Table 6.5,Pg.38)

E_s= 200 GPa

Modular Ratio m= 6.06

d_c= depth to neutral axis then equating strains for cracked section

$d_c = -A_sE_s \pm \frac{\sqrt{(A_sE_s)^2+2bA_sE_sE_{c,eff}}}{bE_{c,eff}}$

d_c= 259.93 mm

cracked Moment of area=I_{NA}

$I_{NA} = A_s(d - d_c)^2 + \frac{E_{c,eff}bd_c^3}{3E_s}$

I_{NA}= 4444414841 mm^4

Concrete Stress=σ_c

σ_c= 18.87 N/mm^2

Limiting concrete Stress=0.48 $\times$ f_{ck}= 19.2 N/mm^2 $>$ 18.87 N/mm^2 $\therefore$ OK

(2) After all creep has taken place the cracked section properties will be based on the long-term and the short-term modulus for the various action

$E_{c,eff} = \frac{E_{cm}}{1+\phi(t,t_0}$ (IRC-112:2011,Cl.12.4.2(2),Pg.132)

creep co-efficient = $\phi(t, t_0)$= 1.50 (IRC-112:2011Table 6.9,Pg.47)

$E_{c,eff}$=13.21 GPa

m= 15.14

d_c= 385.27

I_{NA}= 3948111044 mm^4

limiting Compressive stress of concrete =0.36f_{ck} (IRC-112:2011,Cl.12.2.1,Pg.120)

Stress in concrete=σ_c= 13.78 N/mm^2 $<$ 14.4 N/mm^2 $\therefore$ **OK**

Limiting stress in steel= 0.8f_{yk} (IRC-112:2011,Cl.12.2.1,Pg.120)

steel Stress= 280.34 N/mm^2 $<$ 332 N/mm^2 $\therefore$ **OK**

Crack Control

(IRC-112:2011,Cl.12.3.4,Pg.125) Crack Width=W_k

$$w_k = s_{r.max}(\varepsilon_{sm} - \varepsilon_{cm}) \tag{4.2}$$

$$s_{r.max} = 3.4c + \frac{0.425k_1k_2\phi}{\rho_{\rho.eff}} \tag{4.3}$$

Diameter of bar $\phi = 32\ mm$

clear cover $= c = 50\ mm$

for deformed bars , k_1= 0.8

for bending, $k_2 = 0.5$

Equation

$h_{c,eff}$ is minimum of

1. 2.5(h-d)= 165 mm

2. h-(X/3)= 887.194 mm

3. h/2= 500 mm

$A_{c,eff}$= 165000 N/mm^2

$\rho_{\rho,eff}$= 0.054

$S_{r.max}$= 270.49 mm

Spacing Limit=5$(c + \phi/2)$= 330 mm > 90 mm $\therefore$ OK

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{sc} - k_t\frac{f_{ct,eff}}{\rho_{p.eff}}(1 + \alpha_e\rho_{p.eff})}{E_s} \geq 0.6\frac{\sigma_{sc}}{E_s} \tag{4.4}$$

k_t= 0.5

α_e= 6.06

$f_{ctm} = f_{c,eff}$= 3

σ_{sc}= 238.7010741 N/mm^2

$\varepsilon_{sm} - \varepsilon_{cm}$= 0.00100 > 0.00071

W_k= 0.27 mm

Limiting W_k= 0.30 mm $\therefore$ OK

4.4.4 Distribution Steel

Provide 20 % of main steel (IRC-112:2011,Cl.16.6.1,Pg.181)

Ast= 1786.312 mm^2

4.5 Working Stress Method

BM due to Live Load= 740.61 $kN.m$

BM due to dead Load= 972 $kN.m$

DESIGN BM= 1712.61 $kN.m$

σ_{cbc}= 13.33 MPa

σ_{st}= 200 MPa

m= 10

$k = \frac{1}{1+\frac{\sigma_{st}}{\sigma_{cbc}\times m}}$

k=0.40

$j = 1 - \frac{k}{3}$

j=.87

$Q = 0.5 \times \sigma_{cbc} \times j \times k$

Q=2.31

$d_{req} = \sqrt{\frac{M}{Q\times b}}$

d_{req}=861 mm

A_{st}=$\frac{M}{\sigma_{st}\times j\times d}$ A_{st}=10400 mm^2

Distribution Steel: BM=0.3BM_{LL}+0.2BM_{DL}

BM=416.58 $kN.m$

A_{st}=2529.79 mm^2

Table 4.2: Comparison of WSD and LSD

	WSD	**LSD**
Depth mm	861	950
A_{st} mm^2	10978	8931

4.6 Detailed Drawings

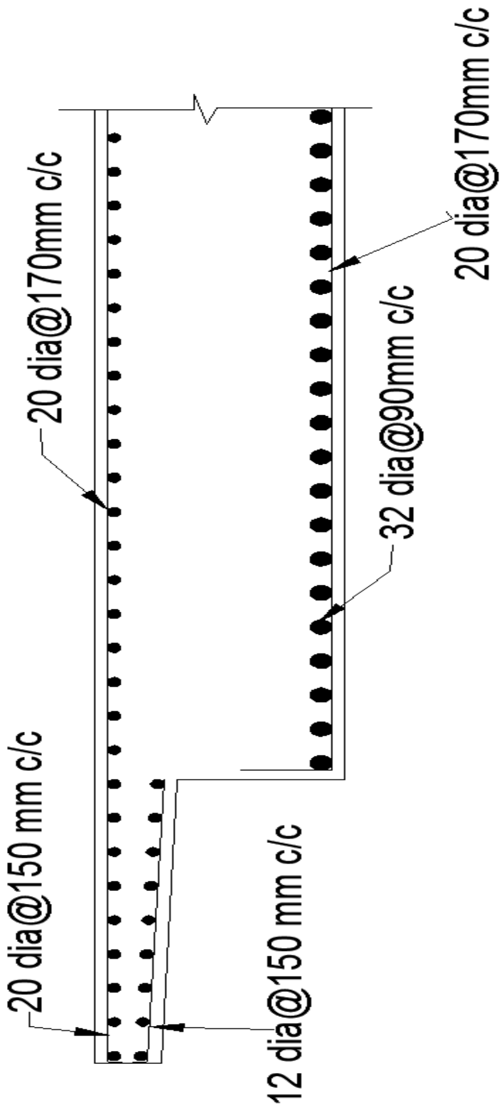

Figure 4.14: Reinforcement detailing for solid deck slab

Chapter 5

Analysis and Design of I Girder

5.1 General

The T-beam bridge is by far the most commonly adopted type in the span range of 10 to 25 m. The structure is so named because the main longitudinal girders are designed as T-beam integral with part of the deck slab,which is cast monolithically with the girders. Simply supported T-beam spans of over 25 m are rare as the dead load then becomes too heavy. A recent construction with a single span of 35 m for Advai bridge in Goa, which is probably the longest span of simply supported reinforced concrete T-beam bridge in India. The super Structure may be arranged to confirm to on of the following three types,as shown in fig 5.1

(a)Girder and slab type. in which the deck slab is supported on and cast monolithically with the longitudinal girders. No cross beams are provided. In this case, the deck Slab is designed as a one-way slab spanning between the longitudinal girders. The system does not possess much torsional rigidity and the longitudinal girders can spread laterally at the bottom level. This type is not adopted in recent designs.

(b)Girder, slab and diaphragm type, wherein the slab is supported on and cast monolithically with the longitudinal girders. Diaphragms connecting the longitudinal girders are provided at the support locations and at one or more intermediate locations within the span. But the diaphragms do not extend up to the deck slab and hence the deck slab behaves as an one-way slab spanning between the longitudinal girders.

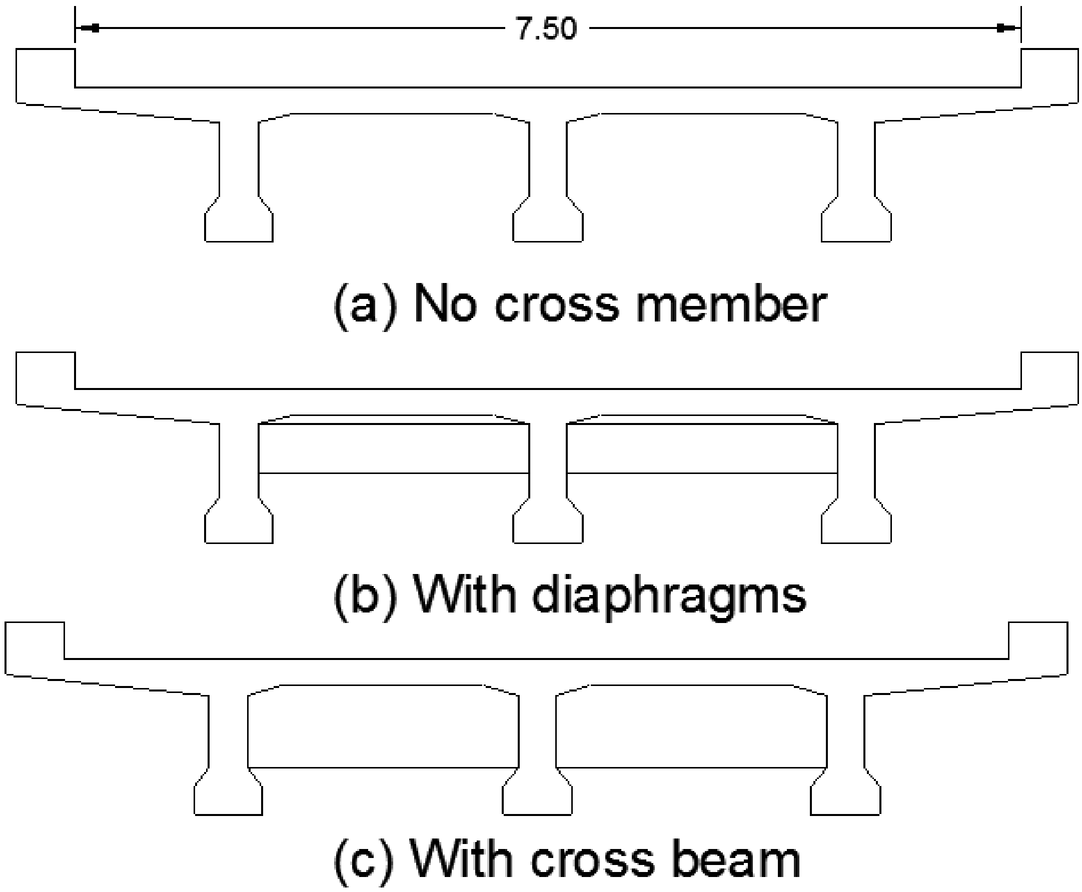

Figure 5.1: Typical cross section of T-beam bridges

This type of superstructure possesses a greater torsional rigidity than the girder and slab type.

(c)Girder, slab and cross beam type, in which the system has at least three cross beams extending up to and cast monolithically with the deck slab. The panels of the floor slab are supported along the four edges by the longitudinal and cross beams. Hence tee floor slab is designed as a two-way slab. This leads to more efficient use of the reinforcing steel and to reduced slab thickness and consequently to reduced dead load on the longitudinal girders. The provision of cross beams stiffens the structure to n considerable extent, resulting in better distribution of concentrated loads among the longitudinal girders. With two-way slab and cross beams, the spacing of longitudinal girders can be increased. resulting in less number of girders and reduced cost of formwork

Components of a Slab-Girder Bridge

The T-beam superstructure consists of the following components as also indicated in Fig 5.2.

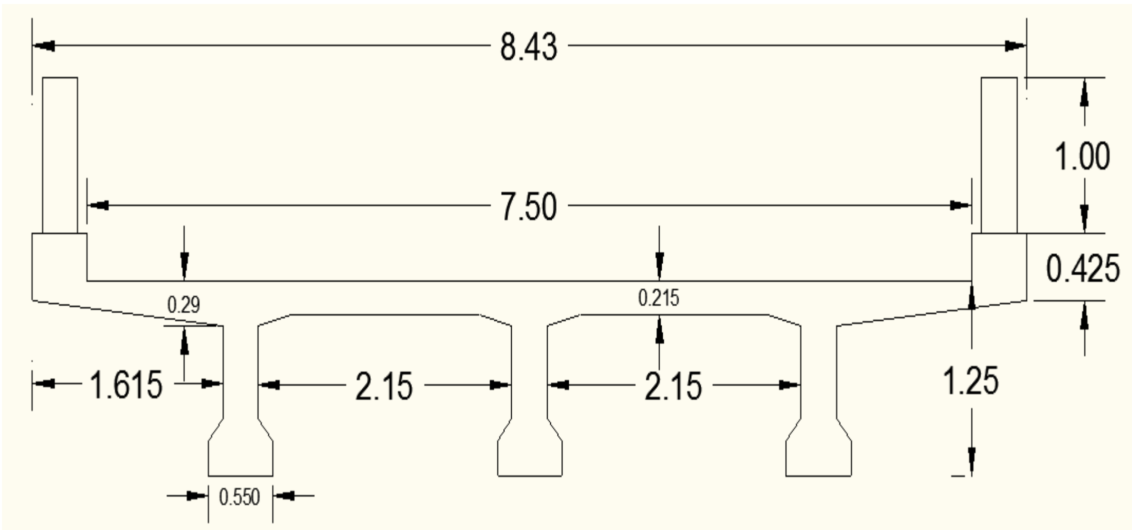

Figure 5.2: Cross section of T-beam bridge

a. Deck slab

b. Cantilever portion

c. Footpaths, if provided, kerbs and handrails

d. Longitudinal girders, considered in design to be of T-section

e. Cross beam or diaphragms

f. Wearing Course.

Standard details are used for kerbs and hand rails. The width of the kerb may vary from 475 *mm* to 600 *mm*. Wearing course can be of asphaltic concrete 0f average thickness 56m or of cement Concrete of M 30 grade for an average thickness of 75 *mm*. Footpaths of 1.5 *m* width are to be provided on either side for bridges located in municipal areas;these may be omitted for bridges on rural stretches of roads. It is, however, desirable to provide footpaths even for a bridge on a rural section, if the overall length of the bridge is large.

Number and Spacing of Main girders

The illustration shown in Fig. 5.2 features three main girders, applicable for a two-lane carriageway of 7.5 m width. If the width of the bridge is adopted as 12.0 m, at least four main girders will be necessary. The lateral spacing of the longitudinal girders will affect the cost of the bridge. Hence in any particular design, the comparative estimates of several alternative arrangements of girders should be studied before adopting the final design. with closer spacing, the number of girders will be increased, but the thickness of deck slab will be decreased. Usually this may result in smaller cost of materials. But the cost of formwork will increase due to larger number of girder forms. as also the cost of vertical supports and bearings. Relative economy of two arrangements with different girder spacings depends upon the relation between the unit cost of materials and the unit cost of formwork. The aim od the design should be to adopt a system which will call for the minimum total cost. For the conditions obtaining in India, a three-girder system is usually more economical than a four-girder system for a bridge width of 8.7 m catering to two-lane carriageway.

Cross Beams

Cross beams are provided mainly to stiffen the girders and to reduce torsion in the exterior girders. These are essential over the supports to prevent lateral spread of the girders at the bearings. Another function of the cross beams is to equalize the deflections of the girders carrying heavy loading with those of the girders with less loadeng. This is particularly important when the design loading consists of concentrated wheel loads, such, as Class 70 R or Class AA wheeled vehicles, to be placed in the most unfavorable position. When the spacing of cross beams is less than about

1.8 times that of longitudinal girders. the depth of the end cross girders should be such as to permit access for inspection of bearings and to facilitate positioning of jacks for lifting of superstructure for replacement of bearings.

Prior to 1956, T-beam bridges had been built without any cross beams or diaphragms, necessitating heavy ribs for the longitudinal beams as in Fig. 5.1 (a). In some cases, only two cross beams at the end have been used. The provision of cross beams facilitates adoption of thinner ribs with bulb shape at bottom for the main beams as in Fig. 5.2. The current Indian practice is to use one cross beam at each support and to provide one to three intermediate cross beams. Diaphragms have been used instead of cross beams in some cases in the past. provision of one cross beam at each end and one at the centre is definitely advantageous in reducing deflection and increasing ultimate load capacity, though the additional benefit in providing more than three cross beams is not significant.

Cantilever Portion

The cantilever portion usually carries the kerb, handrails, footpath if provided and a part of the carriageway. The critical section for bending moment is the vertical section at the junction of the cantilever portion and the end longitudinal girder. For the computation of bending moment due to live load, the effective width for cantilever is assessed from the formula given in IRC-112:2011 Annex B of the Bridge Code as also included as equation (A.2) in Appendix A.

5.2 Grillage Analogy

In recent years, the grillage analogy Method, which is a computer oriented technique, is increasingly being used in the analysis and design of bridges. The method is also suitable in cases where bridge exhibits complicating features such as heavy skew, edge stiffening and. isolated supports. The use of computer facilitates the investigation of several load cases in shortest possible lime.

The method consists of 'converting' the bridge deck structure into a network of rigidly connected beams at discrete nodes i.e. idealizing the bridge by an equivalent grillage. The deformations at the two ends or a beam element are related to the bending and torsional moments through their bending and torsional stiffness.

5.2.1 General Guidelines for Grillage Layout

Idealization of deck into equivalent grillage Because of the enormous variety of deck shapes and support conditions, it is difficult to adopt hard and fast rules for choosing a grillage layout of the actual structure However, some basic guidelines regarding the location, direction, number, spacing etc. of the longitudinal and transverse grid lines forming the idealized grillage mesh are followed in the deck analysis. But each type of deck has its own special features and need some particular arrangements for setting idealized grid lines.

(a)Location and Direction of Grid Lines : Grid lines are to be adopted along lines of strength.In the longitudinal direction, these are usually along the centre line of girders, longitudinal webs, or edge beams, wherever these are present. Where isolated bearings are adopted, the grid lines are also to be chosen along the lines joining the centres of bearings. In the transverse direction, the grid lines are to be adopted, one at each end connecting the centres of bearings and along the centre lines of transverse beams, wherever these exits.

In general, the grid lines should coincide with the centre of gravity of the sections but some shift or deviation is permissible, if this simplifies the grid layout or if it assigns more clearly and easily the sectional properties of the grid members in the other direction.

(b)Number and Spacing of Grid Lines : Wherever possible, an odd number of longitudinal and transverse grid lines are to be adopted. The minimum number of longitudinal grid lines may be three and the minimum number of transverse grid lines per span may be five.

The ratio of spacing of transverse grid lines of those of longitudinal grid lines may be chosen between 1.0 and 2.0. This ratio usually reflects the span to width ratio of the bridge. Thus, for a short span and wide bridge, it should be close to 1.0 and for long span and narrow bridge, this ratio may be kept closer to 2.0.

Grid lines are usually uniformly placed, but their spacing can be varied, if required, depending upon the situation. For example, closer transverse grid lines should be adopted near a continuous support as the longitudinal moment gradient is steep at such locations.

It may be noted that in the grillage analysis, an increase in number of grid lines consequently increases the accuracy of computation, but the effort involved is also more and soon it becomes a case of diminishing return. In a continuous girder bridge, more than one longitudinal physical beam can be represented by one grid line. For slab bridges, the grid lines need not be closer than two to three times the depth of slab.

5.2.2 Grillage Idealization of Slab-on-Girders Bridge

The idealization of beam and slab bridge by an assembly of interconnected beams seems to confirm more readily to engineering judgement than for slab bridges. The T- and I-beams are by far the most commonly adopted type of bridge decks consisting of longitudinal girders at definite spacing; connected by top slab, with or without transverse cross-beams. Usually, the diaphragms connecting the longitudinal girders, are provided at the supports.

The logical choice of longitudinal grid lines for T-beam or I-beam decks is to make them coincident with the centre lines of physical girders and these longitudinal members are given the properties of the girders plus associated portions of the slab, which they represent. Additional grid lines between physical girders may also be set in order to improve the accuracy of the result. Edge grid lines may be provided at the edges of the deck or at suitable distance from the edge. For bridge with footpaths, one extra longitudinal grid line along the centre-line of each footpath slab is also provided. The above procedure for choosing longitudinal grid lines is applicable to both right and skew decks.

When intermediate cross-girders exist in the actual deck, the transverse grid lines represent the properties of cross girders and associated deck slabs. The grid lines are set-in along the centre-lines of cross-girders. Grid lines are also placed in between these transverse physical cross-girders, if after considering the effective flange widths of these girders, portions of the slab are left out. If after inserting grid lines due to these left-over slabs, the spacing of transverse grid lines is still greater than two times the spacing of longitudinal grid lines, the left-over slabs are to be replaced by not one but. two or more grid lines so that the above recommendation for spacing is satisfied. When there is a diaphragm over the support in the actual deck, the grid lines coinciding with these diaphragms should also be placed. A typical T-beam bridge with grillage lay-out. is shown in Fig. 5.3.

When no intermediate diaphragms are provided, the transverse medium i.e. deck

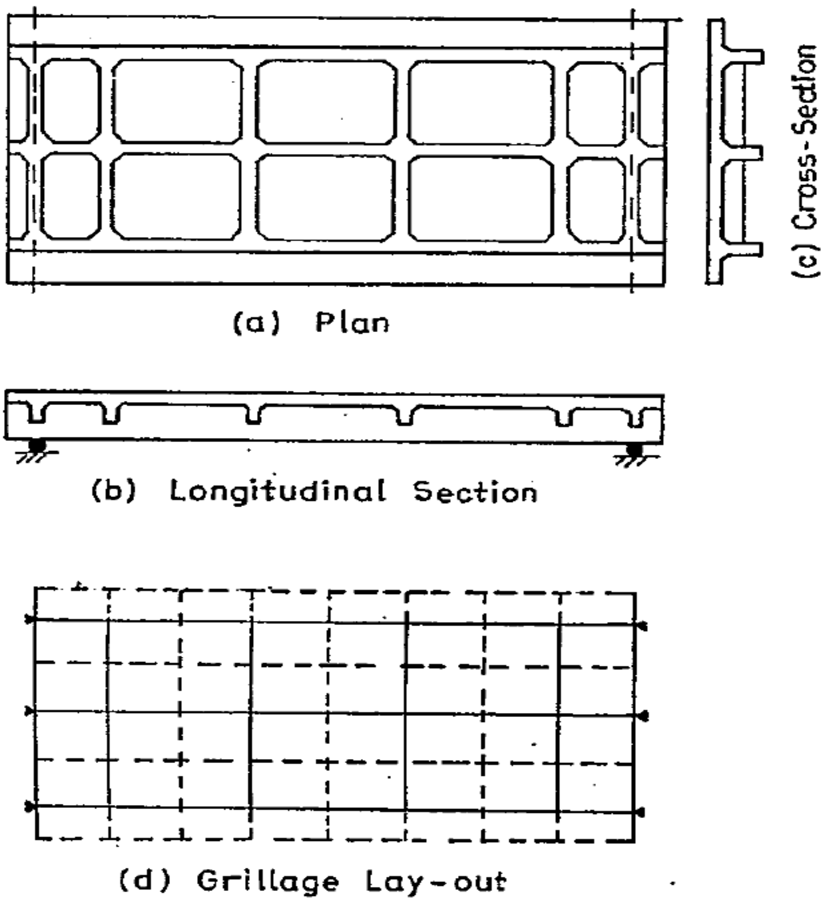

Figure 5.3: T-beam bridge and grillage lay-out

slab is conceptually broken into a number of transverse strips and each strip is replaced by a grid line. The spacing of transverse grid lines is somewhat arbitrary but about 1/8 of effective span is generally convenient. As a guideline, it is recommended that the ratio of spacing of transverse and longitudinal grid tines be kept between 1 and 2 and the total number of lines be odd. This spacing ratio may also reflect the span-width ratio of the deck. Therefore, for square and wider decks, the ratio can be kept as 1 and for long and narrow decks it can approach to 2.

The transverse grid lines are also placed at abutments joining the centres of bearings. A minimum of seven transverse grid lines are recommended, including end grid lines. It is advisable to align the transverse grid lines normal to the longitudinal lines wherever cross-girders do not exit. It should also be noted that the transverse grid lines are extended upto the extreme longitudinal grid lines.

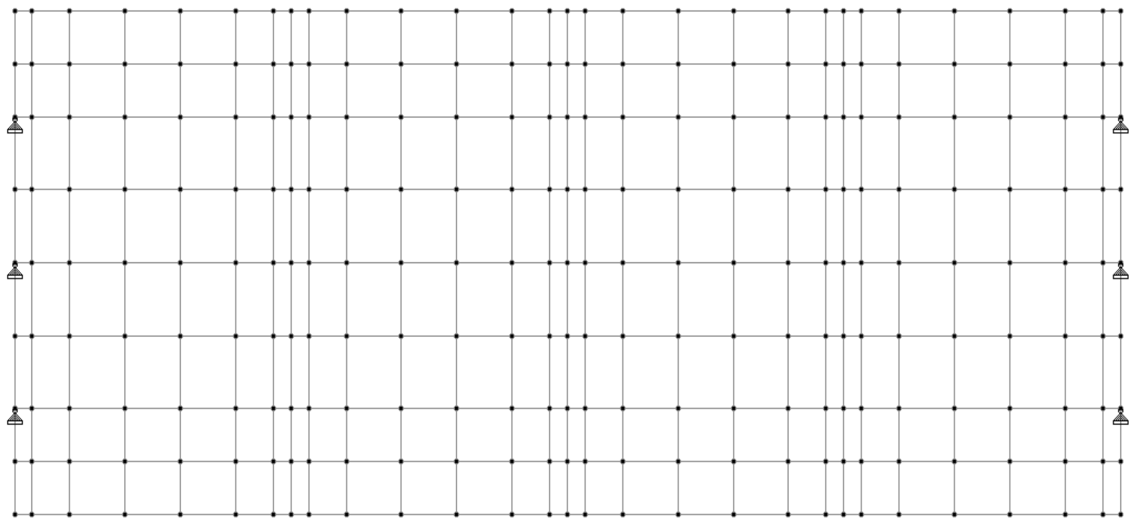

Figure 5.4: Grillage Model for 20m Span in Staad-Pro

5.3 Sample Calculation

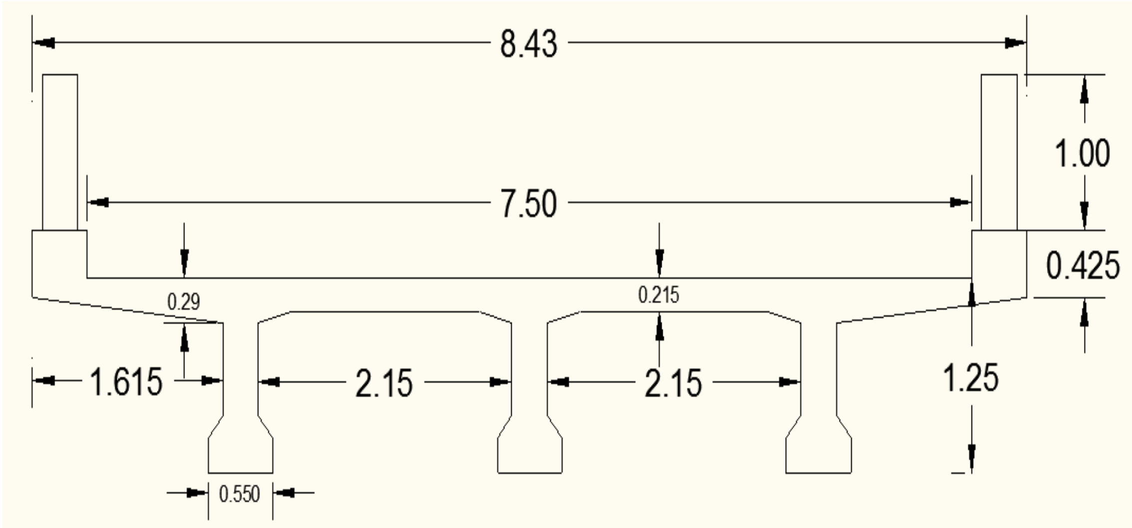

Figure 5.5: Cross Section for 20m Span

For 20m Span

1.Input Data

(Bending moment due to dead load) BM_{DL}= 3441.88 $kN.m$

(Bending moment due to live load) BM_{LL}= 1904.53 $kN.m$

Shear force due to dead load SF_{DL}= 677.43 kN

Shear force due to live load SF_{LL}= 439.76 kN

Impact factor IF= 1.17

Partial safety factor for live load γ_{mLL}= 1.50

Partial safety factor for dead load γ_{mDL}= 1.35

Design dead load bending moment $DBMDL$= 4646.54 $kN.m$

Design live load bending moment $DBMLL$= 3342.45 $kN.m$

Design torsion moment DTM= 143.56 $kN.m$

Design shear force $DSFDL$= 914.53 kN

Design shear force $DSFLL$= 771.78 kN

Total bending moment BM= 7988.99 $kN.m$

Total torsion moment TM= 256.76 $kN.m$

Total shear force SF= 1686.31 kN

f_{ck}= 40.00 Mpa

f_{yk}= 415.00 Mpa

Span L= 10.00 m

E_s= 200.00 GPa

E_{cm}= 33.00 GPa

f_{ct}= 3.00 MPa

Perimeter u= 9190.00 mm

2.Longitudinal Reinforcement

Effective flange width

1. 1/4 of span=5000 mm

2. c/c distance of beam= 2450 mm

3. Breadth of web + 12 times Slab thickness= 3300 mm

Least of three= b_{eff}= 2450 mm

Effective Depth= d= 1394 mm

$k = M/(b_f d^2 f_{ck})$= 0.04

$z = d[0.5 + \sqrt{0.25 - \frac{k}{0.892}}]$

Lever arm = z =1325.03 mm

Depth of stress block

$s = 2(d - z)$= 137.95 mm $s < h_f$ $\therefore$Stress block lies in Flange

Depth of N.A.X= s/0.8= 172.43 mm $X < h_f$ $\therefore$ N.A. lies in Flange

Area of Steel Required= $A_s = M/0.87 f_{yk} z$= 16699.35 mm^2

Reinforcement provided

Dia of bar= 32 mm

No. of bars= 14

A_{st}= 11259.47mm^2

Dia of bar= 25 mm

No. of bars= 12

Ast= 5890.49 mm^2

A_{st} provided= 17419.95 mm^2 $\therefore$OK

Max. $A_{st} = 100A_s/A_c\%$= 3.94 %<4% ∴OK

MR= 8204.56 $kN.m$ ∴OK

Minimum Area of Reinforcement

$A_s = k_c k c_{ct,eff} A_{ct}/f_{yk}$ (IRC-112:2011,Cl.12.3.3,Pg.122)

k_c=1 (IRC-112:2011,Cl.12.3.3,Pg.124)

k= 0.65 (IRC-112:2011,Cl.12.3.3,Pg.124)

$f_{ct,eff}$= 3 (IRC-112:2011,Table 6.5,Pg.38)

A_{ct} web= 418200 mm^2

A_{ct} Flange= 489900 mm^2

A_s Web= 1965.04 mm^2

A_s flange= 2301.94 mm^2

Total Min. A_s= 4266.98 mm^2 Provided A_s>Min.A_s ∴ OK

2. Transverse Steel In Flange

(a) calculated the design longitudinal shear stress v_{Ed}

For a sagging moment the longitudinal shear shear stresses are the greatest over a distance of Δx measured from the point of zero moment and Δx is taken as half the distance to the max. BM at the mid span

Δx= L/4

Δx= 5000 mm

ΔM_{DL}= 3434.96 $kN.m$ (Dead load moment at L/4 distance)

ΔM_{LL}= 2348.73 $kN.m$ (Live load moment at L/4 distance)

ΔM= 5783.69 $kN.m$

The Change in Longitudinal force

$$\Delta F_d = \frac{\Delta M}{(d - h_f/2)} \times \frac{b_f - b_w/2}{b_f} \tag{5.1}$$

ΔF_d= 1961.16 kN

Longitudinal shear stress

$v_{Ed} = \Delta F_d/(h_f \times \Delta x)$

v_{Ed}= 1.96 Mpa

b) Check the strength of the concrete strut.

$v_{Ed} \leq v f_{cd} sin\theta_f cos\theta_f$ (IRC-112:2011,Cl.10.3.5,Pg.98)

v= 0.60 (IRC-112:2011,Cl.10.3.2,Pg.90)

f_{cd}= 0.446 $\times f_{ck}$= 17.84 Mpa

θ_f= 21.8

v_{Ed}= 3.69 Mpa >1.96Mpa $\therefore$ OK

Design of transverse reinforcement

If $v_{Ed} < 0.4 f_{ctd}$ then No transverse reinforcement required.

$f_{ctd} = f_{ctk}/\gamma_m$

f_{ctk}= 3

0.4 $\times$ f_{ctd}= 0.80 Mpa<v_{Ed}

$\therefore$ Transverse reinforcement required

Transverse reinforcement per unit length

$$\frac{A_{sf}}{s_f} \geq \frac{v_{Ed} h_f}{f_{yd} cot\theta_f} \tag{5.2}$$

(IRC-112:2011,Cl.10.3.5,Pg.98)

A_{sf}/s_f= 0.378

Dia of Bar= 16

Spacing of bar = s_f= 200

A_{sf}/s_f= 1.01 $\therefore$ OK

Min.Asf=0.13 $\times$ h_f $\times$ b_f/100= 637 mm^2

Provided Ast= 1005.31 mm^2 $\therefore$ OK

3.Shear Reinforcement

Check Max. shear at face of support

Max. design SF= 1686.31 kN

The maximum effective cross sectional area of the shear reinforcement Asw.max for cotθ=1 is given by

$$\frac{A_{sw.max} f_{ywd}}{b_w s} \leq 0.5 \alpha_{cw} v_1 f_{cd} \tag{5.3}$$

(IRC-112:2011,Cl.10.3.2.2,Pg.91)

$A_{sw.max}/s$=4.84

Min. shear reinforcement

$$\frac{A_{sw.max}}{s} = \frac{0.072\sqrt{f_{ck}}b_w}{f_{yk}} \tag{5.4}$$

$A_{sw.min}/s$=0.33

Provided Stirrups

legged= 4

Dia of Bar= 10

Spacing= 170

Provided Asw/s= 1.85

$$V_{Rd.s} = \frac{A_{sw}}{s} z f_{ywd} cot\theta \tag{5.5}$$

(IRC-112:2011,Cl.10.3.2,Pg.90)

$V_{Rd.s}$=2091.84 $kN > SF \therefore$ OK

Check For Crack Control

Creep Co-efficient=$\Phi(\infty, t_0)$= 1.74 (IRC-112:2011,Table6.9,Pg.47)

$2A_c/u$= 94.67

$E_{c,eff} = E_{cm}/1 + \Phi(\infty, t_0)$= 10.94

α_e=18.27

$$x = -A_s E_s + \sqrt{\frac{(A_s E_s)^2 + 2bA_s E_s E_{c,eff}}{bE_{c,eff}}} \tag{5.6}$$

Depth of N.A. Axis= x= 717.88 mm

calculate the stress in tension steel

Taking moment about the level of copm.force in the concrete

$\sigma_s = \frac{M}{(d-\frac{x}{3})A_s}$

σ_s=403.42 MPa

Calculate $(\varepsilon_{sm} - \varepsilon_{cm})$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{sc} - k_t \frac{f_{ct,eff}}{\rho_{p.eff}}(1 + \alpha_e \rho_{p.eff})}{E_s} \geq 0.6\frac{\sigma_{sc}}{E_s} \tag{5.7}$$

(IRC-112:2011,Cl.12.3.4,Pg.125)

k_t= 0.5

$f_{ct,eff}$= 3

α_e= 6.67

$2.5(h - d)$= 140

$h - (x/3)$= 1210.71

$h/2$= 725

$A_{c,eff}$= 42000

$\rho_{\rho,eff}$=$A_s/A_{c,eff}$= 0.41

$(\varepsilon sm - \varepsilon_{cm})$= 0.00194

$0.6\frac{\sigma_{sc}}{E_s}$=0.0012<0.0194 $\therefore OK$

Calculate The Max. Crack Spacing ($S_{r,Max}$)

$$s_{r.max} = 3.4c + \frac{0.425k_1k_2\phi}{\rho_{\rho.eff}} \tag{5.8}$$

(IRC-112:2011,Cl.12.3.4,Pg.127)

c=clear cover= 40 mm

ϕ_{eq}= 29.19 mm

k_1= 0.8

k_2= 0.5

$S_{r,Max}$= 148.15 mm

Max spacing=5(c+ϕ/2)= 272.98 mm > $S_{r,Max}$ $\therefore$ OK

Calculate Crack Width W_k

$$w_k = s_{r.max}(\varepsilon_{sm} - \varepsilon_{cm}) \tag{5.9}$$

w_k=0.289 mm < 0.3 mm $\therefore$ OK

Design of Torsional Reinforcement

1. design for shear using the variable strut inclination method

V_{Ed}= 1686.31 kN

$V_{Rd.max}$= 2091.84 kN

For Shear links Provided Asw/s= 1.85

2. converts the T-section to an equivalent hollow box section

thickness = t = 208 mm

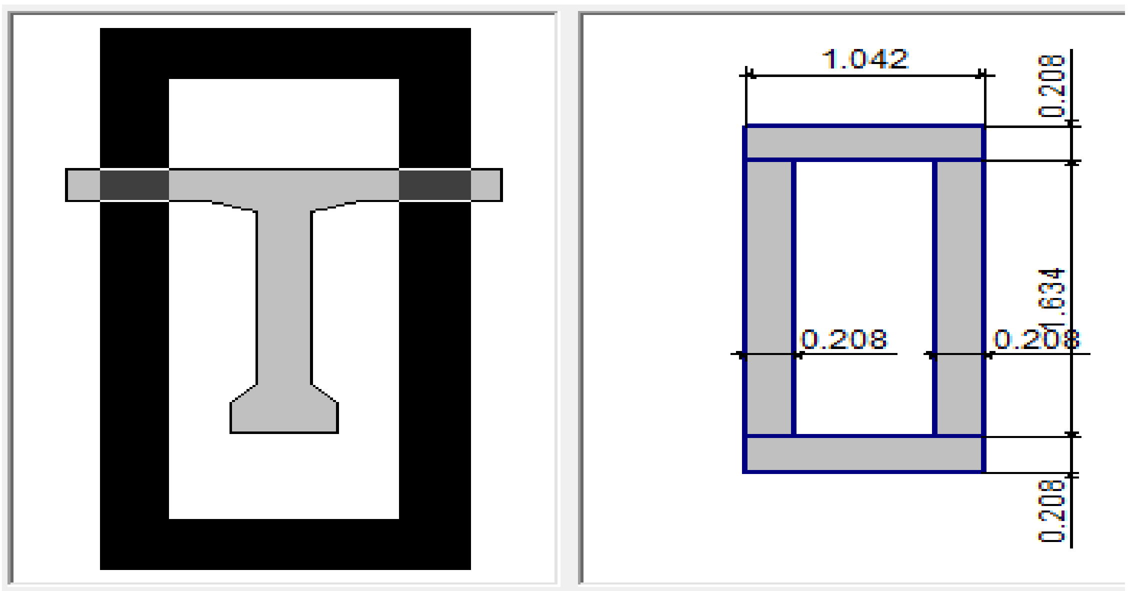

Figure 5.6: Equivalent hollow section

height = h = 2050 mm

width = b = 1042 mm

Area within centreline= A_k= 1536228 mm^2

perimeter of centreline= u_k= 5352 mm

3.Check if concrete section is adequate

$$\frac{T_{Ed}}{T_{Rd.max}} + \frac{V_{Ed}}{V_{Rd.max}} \leq 1.0 \tag{5.10}$$

(IRC-112:2011,Cl.10.5.2.1,Pg.108)

where

$$T_{Rd.max} = 2v\alpha_{cw}f_{cd}A_k t_{ef.i} sin\theta cos\theta \tag{5.11}$$

(IRC-112:2011,Cl.10.5.2.1,Pg.108)

v=0.6(1-f_{ck}/310)

v=0.52

$T_{Rd.max}$=2054.47 $kN.m$ > T_{Ed}=256.76 $kN.m$ $\therefore$ OK

$\frac{T_{Ed}}{T_{Rd.max}} + \frac{V_{Ed}}{V_{Rd.max}}$=0.93 < 1 $\therefore$ OK

Therefore the concrete section is adequate.

4.Calculate the additional link reinforcement required to resist torsion(Note that A_{sw} is for one leg only)

$\frac{A_{sw}}{s} = \frac{T_{Ed}}{2A_k 0.87 f_{yk} cot\theta}$

A_{sw}/s=0.093

5. Therefore for shear plus torsion and based on the area of two legs

Asw/s=1.85+2×0.093=2.03

For torsion reinforcement 2-legged 8 mm stirrups 300 mm c/c with A_{sw}/s=0.34

Total A_{sw}/s=2.18 > 2.03 $\therefore$ OK

6. Calculate the area A_{sl} of the additional longitudinal reinforcement required for torsion

$$A_{sl} = \frac{T_{Ed} u_k cot\theta}{2A_k 0.87 f_{yk}} \tag{5.12}$$

(IRC-112:2011,Cl.10.5.2.1,Pg.109)

A_{sl}=3096.92 mm^2

Dia. of bar= 25 mm

No. of bars= 7

A_{st}=3436.17 mm^2

5.4 Detailed Drawings

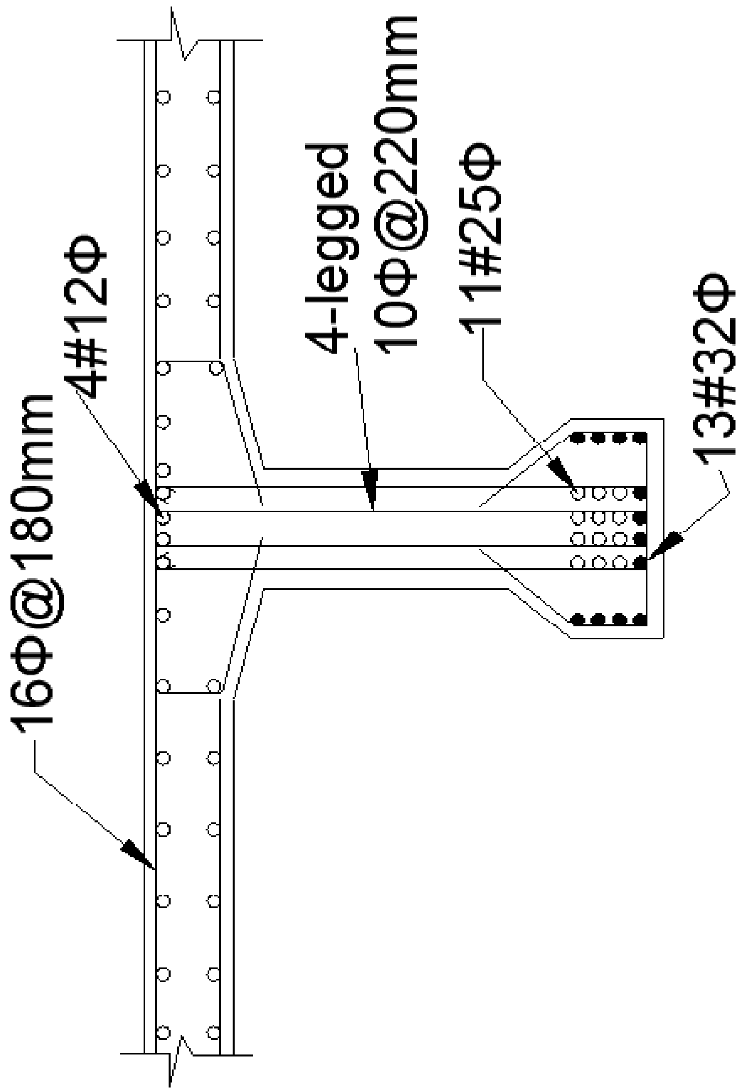

Figure 5.7: Effective UDL for maximum forces

Chapter 6

Parametric Study for Different Span

6.1 General

The alternatives available are required to be evaluated to design the girder by taking different span. The study was done for 10,15,20,25,30m span.

6.2 Design Constraint

The forces calculated and summary of trials shown in this chapter is based on various design constraints and the cross section is shown in fig.6.1 f_{ck}=40 Mpa
f_{yk}=415Mpa
Slab Thicness=250 mm
Width of Diaphragm=300mm

6.3 Summary of Trails

The overall methodology and step by step design is described in chapter 5, a typical case of 20m span is also shown in detail.For want of space and to avoid voluminous

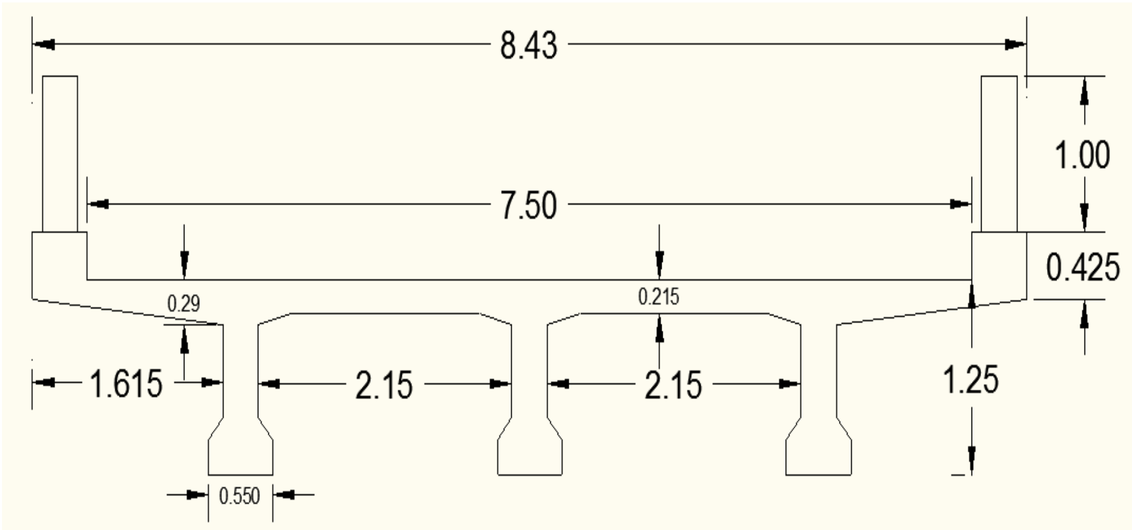

Figure 6.1: Cross section for different span

data coverage, all alternatives analyzed using STAAD-PRO software and designed by spreadsheet are not repetitively explained and enclosed with the work. However summary of the voluminous work done to calculate maximum force and quantity of material for various Span in this para.

Table 6.1: Analysis Result for 70R wheeled vehicle

Span	**Bending Moment** kN.m	**Shear force** kN	**Torsional Moment** kN.m
10	2093.93	932.90	213.06
15	4626.55	1337.45	244.32
20	7989.00	1686.31	256.756
25	12395.18	2067.47	226.252
30	18185.45	2479.46	222.29

Table 6.2: Results for Working Stress Method

Span m	**Depth of Slab** mm	**Depth of Girder** mm	**As for slab** mm^2	**As for Girder** mm^2
10	140	900	935	8315
15	140	1250	935	14177
20	140	1550	935	19552
25	140	1900	935	24588
30	140	2250	935	30356

Table 6.3: Results for Limit state Method

Span m	**Depth of Slab** mm	**Depth of Girder** mm	**As for slab** mm^2	**As for Girder** mm^2
10	150	825	1005	7875
15	200	1150	1005	12282
20	200	1450	1005	16700
25	210	1860	1117	19940
30	260	2210	1257	24538

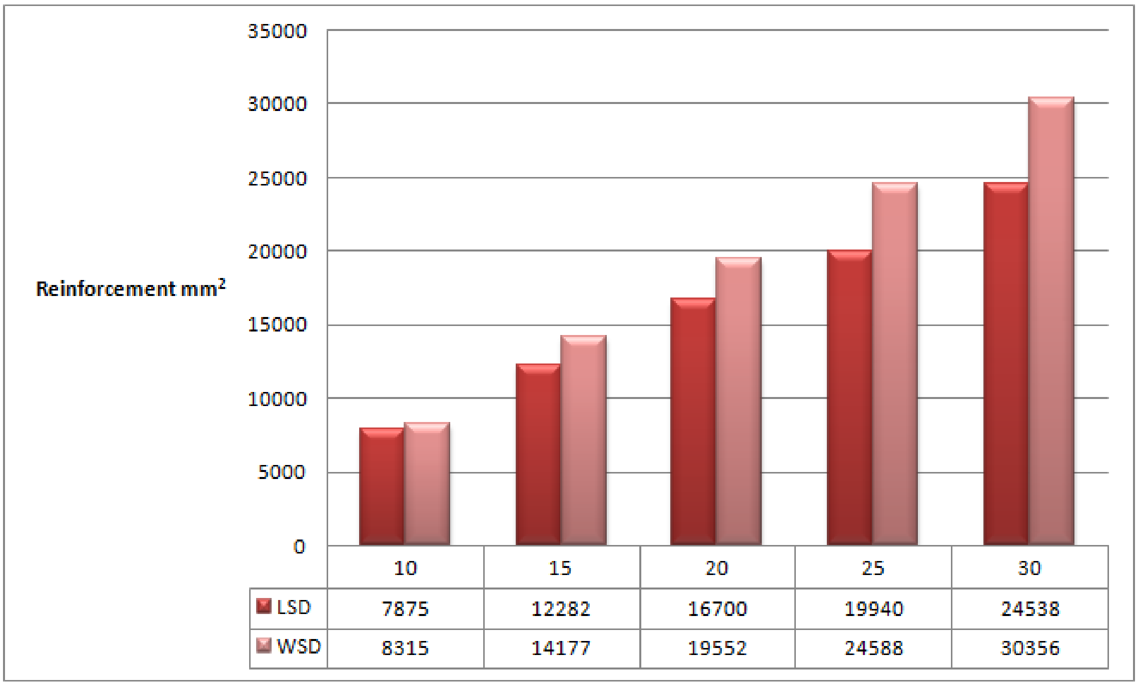

Figure 6.2: Comparison of reinforcement

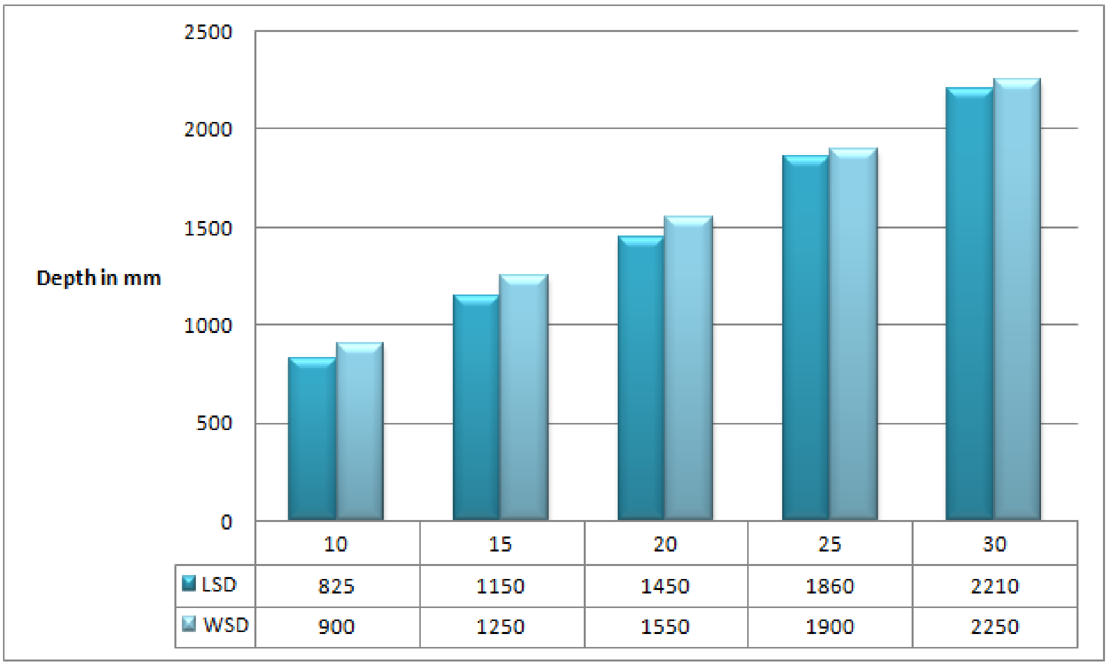

Figure 6.3: Comparison of depth

Chapter 7

Summary and Conclusion

7.1 Summary

The main objective of the work is to study the design philosophy in the R.C.bridge superstructure design of solid deck slab and slab-girder and to find out the requirement of the reinforcement in Limit State Method and Working Stress Method. For slab-girder type superstructure different span such as 10m,15m,20m,25m and 30m. R.C.Bridge is analysed using STAAD-Pro civil software. Excel spread sheets are prepared for the deck slab design, longitudinal girder design as per IRC-112:2011 and as per WSD method. In dead load self-weight of girder, slab. wearing coat and crash barrier load considered. And in live load ClassA and Class 70R as per IRC-6:2010 loading are considered. Total 5 alternatives are done using prepared spread sheet. R.C. girder is designed to satisfy moment capacity check, shear check, crack check and torsion check. And find out required reinforcement is calculated .

7.2 Conclusion

- Maximum live load moment is obtained when the class 70R. IRC loading moving at a time on two lanes for both solid deck slab and slab girder type bridge superstructure.
- The various limit states of flexure, shear,torsion and cracking in verified as per

IRC-112:2011 code and the design is found satisfying all limit states.

- The parametric study is carried out to and the requirement of reinforcement for LSD and WSD for span 10m,15m,20m,25m and 30m as shown in table 6.2 and 6.3.

- It is observed from parametric study that for limit state method, depth and reinforcement for girder is less as compare to working stress method.

7.3 Future Scope of Work

- In this study the sub structure Design is not compared with limit state method. The work can be extended by considering sub structure design with limit state method comparing with working stress method

- In this study the cross section is taken T-beam type cross section, the work can be further extended for box type section,or any other cross sections.

- The work can be further extended by taking Prestressed bridge and design with IRC-112-2011

Appendix A

Effect of Concentrated Loads on Deck Slab

A.1 General

The effect of concentrated loads on slabs spanning in one or two directions or on cantilever slabs may be calculated from the influence fields of such loads or by any other rational method. A value of 0.2 may be assumed for Poisson's ratio. A simplified method for estimating the action of concentrated loads on slab, based on effective width method for cantilever and simply supported slab, is described below, which may be used where more detailed calculations are not performed.

A.1.1 Effective Width

The bending moment per unit width of slab caused by concentrated loads on solid slabs spanning in one direction or on cantilever slabs, may also be calculated by assessing the width of slab that may be taken as effective in resisting the bending moment due to the concentrated loads. For precast slabs, the term 'actual width of slab' used in this Clause shall indicate the actual width of each individual precast unit.

Slabs designed on the above basis need not be checked for shear.

Solid slab spanning in one direction

(i) For a single concentrated load, the effective width may be calculated in accordance with the following equation:

$$b_{ef} = \alpha a(1 - \frac{a}{l_0}) + b_1 \tag{A.1}$$

where

b_{ef} = the effective width of slab on which the load acts,

l_0 = the effective span,

a= the distance of the centre of gravity of the concentrated load from the nearer support,

b_1 = the breadth of concentration area of the load, i.e., the dimension of the tyre or track contact area over the road surface of the slab in a direction at right angles to the span plus twice the thickness of the wearing coat or surface finish above the structural slab,

α = a constant having the following values depending upon the ratio b/l_0 where b is the width of the slab.

(i)Provided that the effective width shall not exceed the actual width of the slab;and provided further that in case of a load near the unsupported edge of a slab, the effective width shall not exceed the above value nor half the above value plus the distance of the load from the unsupported edge.

(ii) For two or more concentrated loads in a line in the direction of the span, the bending moment per unit width of slab shall be calculated separately for each load according to its appropriate effective width of slab calculated as in (i) above.

(iii) For two or more loads not in a line in the direction of the span: If the effective width of slab for one load overlaps the effective width of slab for an adjacent load, the resultant effective width for the two loads equals the sum of the respective effective widths for each load minus the width of overlap, provided that the slab so designed is tested for the two loads acting separately.

Solid slab cantilever

(i) For a single concentrated load, the effective width may be calculated in accordance with the following equation:

$$b_{ef} = 1.2a + b_1 \tag{A.2}$$

where

a=the distance of the centre of gravity of the concentrated load from the face of the cantilever support,

b_1=the breadth of concentration area of the load, i.e., the dimension of the tyre or track contact area over the road surface of the slab in a direction at right angles to the span plus twice the thickness of the wearing coat or surface finish above the structural slab,

Provided that the effective .width-of the cantilever slab shall not exceed one-third the length of the cantilever slab measured parallel to the support. And provided further that when the concentrated load is placed near one of the two extreme ends of the length of cantilever slab in the direction parallel to the support, the effective width shall not exceed the above value, flor shall it exceed half the above value plus the distance of the concentrated load from the nearer extreme end measured in the direction parallel to the fixed edge.

(ii) For two or more concentrated loads

If the effective width of slab for one load overlaps the effective width of slab for an adjacent load, resultant effective width for the two loads shall be taken as equal to,the sum of the respective effective width for each load minus the width of overlap, provided that the slab so designed is tested for the two loads acting separately.

Dispersion of Loads Along the Span

The effect of contact of wheel or track load in the direction of span length shall be taken as equal to the dimension of the tyre contact area over the wearing surface of the slab in the direction of the span plus twice.the overall depth of the slab inclusive of the thickness of the wearing surface.

Dispersion of Loads Through Fills and Wearing Coat

The dispersion of loads through fills and wearing coat shall be assumed at 45^0 both along and perpendicular to the span.

Appendix B

List of Papers Communicated

- Nirav Vaghasia and Prof.Harsh Morbia, *"Design of Bridge Superstructure With Limit State Method as per IRC-112:2011"*, International Conference (NUiCONE),Nirma University,Ahmedabad,India,December 2013.

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] Krishna N.Raju. Design of Bridge. Third edition.

[4] V.K.Raina. Concrete Bridge Engineering. Fifth edition.

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

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

[7] C.S.Surana and R.Agrawal. Grillage Analogy in Bridge Deck Analysis

[8] IRC-6:2010 Standard Specification and Code of Practice for Road Bridge-Loads and Stresses, fifth edition.

[9] IRC-112:2011 Code of Practice for Concrete Road Bridges

[10] IRC-21:2000 Standard Specifications and Code of Practice for Road Bridge" Section-III Cement Concrete (Pain and reinforced) third revision

[11] K.S.Rakshit.Design and construction of highway bridges

[12] BS EN 1992-2:2005. Eurocode 2-Design of concrete structures Part-2:Concrete bridges-design and detailing rules