DESIGN OF BRIDGES

For BE/B.TECH/BCA/MCA/ME/M.TECH/Diploma/B.Sc/M.Sc/BBA/MBA/Competitive Exams & Knowledge Seekers

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PREFACE

This book has been written for the Medical/Pharmacy/Nursing/ME/M.TECH/BE/B.Tech students of All University with latest syllabus for ECE, EEE, CSE, IT, Mechanical, Bio Medical, Bio Tech, BCA, MCA and All B.Sc Department Students.

The basic aim of this book is to provide a basic knowledge in Design of Bridges.

Design of Bridges Syllabus students of degree, diploma & AMIE courses and a useful reference for these preparing for competitive examinations.

All the concepts are explained in a simple, clear and complete manner to achieve progressive learning.

This book is divided into five chapters. Each chapter is well supported with the necessary illustration practical examples.

I am greatly indepted to M/S. Arputhasamy Publications for publishing this book in such a short span of time with great interest.

Finally I thank my teachers, students & parents whose invaluable support made the whole project possible.

I would be glad to receive any comments and suggestions for the improvement of this book.

Dedicated to my beloved parents.

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INTRODUCTION

UNIT 1

Classification, Investigation and Planning, Choice of type, I.R.C. Specifications for Road Bridges, Standard live, loads, other forces acting on bridges, General Design considerations - Bearings, substructures and footing for bridges.

A bridge is a structure providing passage over an obstacle such as valley, road, railway line, canal, river etc, without closing the way beneath. The required passage may be for road, railway, canal, pipe line, cycle track (or) Pedestrian.

Bridges may be of RC or steel construction. The beams are supported on intermediates piers and end abutments. In India bridges for roads are designed as per the recommendation of the India Road Congress. Rail bridges are designed on the recommendation of specifications of the Indian Railway standard code of Practice.

OBJECTIVES

The objectives for these guidelines are to:

Describe processes or stages of work to be followed for a bridge site investigation. Describe information required to design bridge foundations. Indicate standards of skill, workmanship and reporting, which are to be applied.

STAGES OF INVESTIGATION

Field Reconnaissance Survey For most bridge investigations access and environmental constraints have major influences on cost. It is therefore necessary for a field reconnaissance survey to be conducted as the first stage of a geotechnical investigation. This may be undertaken by MRWA or by a consultant specifically engaged for this survey.

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Information on the following should result: Legal and physical aspects of access to site and bridge alignment – both riverbed and adjoining properties.

- Availability of any services or supplies of water, electricity, earthworks plant.
- Buried or overhead services.
- Photographs of surface conditions.
- Traffic control requirements. The possible effects of alternative investigation techniques on the environment (for example, ground disturbance, vegetation removal, water discharge, noise etc).
- On-ground survey details.
- Tide, river level or other natural constraints.

The physical relationship of the proposed construction to the immediate natural surroundings and any existing developments. The field reconnaissance survey must be diligently prepared and conducted to allow for reliable cost estimates to be prepared. Experienced and suitably qualified personnel should perform the survey. Further stages of the investigation should be held until the field reconnaissance survey has been completed and reported to MRWA. Cost estimates for the major part of the investigation will be based partly on this reconnaissance survey.

Desk Top Study:

Every site investigation should commence with a desk study directed towards collecting, collating and reviewing the following:

- Design drawings from any previous structure at the site.
- Previous site investigation reports, borehole logs, penetrometer results and construction experience e.g. piling records.
- Geological and Topographical maps, survey data and records.
- Hydrological data.
- Aerial photographs.
- Regional seismicity data.
- Survey records, local knowledge and resources.

Selection Criteria For Bridge Site

1. The choice of the right site is a crucial decision in the planning and designing of a bridge.
2. It may not be possible always to have a wide choice of sites for a bridge.
3. This is particularly so in case of bridges in urban areas and flyovers.
4. For river bridges in rural areas, usually a wider choice may be available.

- **The characteristics of an ideal site for a bridge across a river are:**

  a. A straight reach of the river.
  b. Steady river flow without cross currents:
  c. A narrow channel with firm banks
  d. Suitable high banks above high flood level on each side.
  e. Rock or other hard in erodible strata close to the river bed level.
  f. Economical approaches danger of floods, the approaches should be free from obstacles such as hills, frequent drainage crossings, scared places, graveyards or built up areas or

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troublesome land acquisition

g. Absence of sharp curves in the approaches;

h. Absence of expensive river training works;

i. Avoidance of excessive underwater construction.

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Selection Criteria For Bridge Site

- •For selecting a suitable site for a major bridge, the investigating engineer should make a reconnaissance survey to get impression of the landscape and to decide on the type of the structure to the site.
- •Care should be taken to investigate a number of probable alternative sites and then decide on the site which is likely to serve the needs of the bridge at the least cost.
- •A brief description of the reasons for the selection of a particular site should be furnished in the investigation report along with salient details of alternative sites investigated and rejected.

Selection Criteria For Bridge Site Preliminary Study (techno-economic Feasibility Survey)

Different studies performs during **Preliminary Study** are:

- Topography
- Catchment area
- Hydrology
- Geo-technical data
- Seismology
- Navigation
- Construction resources
- Nearby bridges
- Traffic data

CLASSIFICATION:

The bridges are mainly classified according to

(1) Life - a) Permanent Bridges
b) Temporary Bridges

(2) Road Level – a) Deck Bridges

b) Through Bridges

c) Semi through Bridges

(3) Materials – a) Timber bridges

b) Masonry bridges

c) Steel bridges

d) RCC bridges

e) Prestressed concrete bridges

(4) Position of high Flood Level -

a) Submersible (or) Low Level bridge

b) Non Submersible (or) High Level bridge

(5) Introduction of bridge - a) Straight bridge

b) Skew bridge

(6) Types of superstructure – a) Arch bridge

b) Girder bridge

c) Truss bridge

d) Suspension bridge

(7) Clearance in Navigation channels – a) Movable bridges

b) Transporter bridges

(8) Loading – a) Class A bridges

b) Class B bridges

c) Class AA bridges.

COMPONENTS OF BRIDGE:

Sub structure

(i) Foundation
(ii) Abutments
(iii) Pier
(iv) Wing Walls & Return walls
(v) Rivetment
(vi) Apron
(i) Approaches to bridge.

Super structure

(i) Bearings
(ii) Deck
(iii) Hand rails
(iv) Girder (or) Truss
(v) (v) Roadway,

etc Loads on bridges

The following are the various loads tobe considered for the purpose of computing stresses, wherever they are applicable.

- Dead load
- Live load
- Impact load
- Longitudinal force
- Thermal force
- Wind load
- Seismic load
- Racking force
- Forces due to curvature.
- Forces on parapets
- Frictional resistance of expansion bearings
- Erection forces

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Dead load – The dead load is the weight of the structure and any permanent load fixed thereon. The dead load is initially assumed and checked after design is completed.

Live load – Bridge design standards specify the design loads, which are meant to reflect the worst loading that can be caused on the bridge by traffic, permitted and expected to pass over it. In India, the Railway Board specifies the standard design loadings for railway bridges in bridge rules. For the highway bridges, the Indian Road Congress has specified standard design loadings in IRC section II. The following few pages brief about the loadings to be considered.
For more details, the reader is referred to the particular standard.

Railway bridges: Railway bridges including combined rail and road bridges are to be designed for railway standard loading given in bridge rules. The standards of loading are given for:

· Broad gauge- Main line and branch line
· Metre gauge- Main line, branch line and Standard C
· Narrow gauge- H class, A class main line and B class branch line The actual loads consist of axle load from engine and bogies. The actual standard loads have been expressed in bridge rules as equivalent uniformly distributed loads (EUDL) in tables to simplify the analysis. These equivalent UDLvalues depend upon the span length. However, in case of rigid frame, cantilever and suspension bridges, it is necessary for the designer to proceed from the basic wheel loads. In order to have a uniform gauge throughout the country, it is Advantageous to design railway bridges to Broad gauge main line standard loading. The EUDLs for bending moment and shear force for broad gauge main line loading can be obtained by the following formulae, which have been obtained from regression analysis:

There are three types of standard loadings for which the bridges are designed namely,
IRC class AA loading, IRC classes A loading and IRC class B loading.

Foot Bridges and Footpath on Bridges – The live load due to pedestrian traffic should be treated as uniformly distributed over the pathway. For the design of footbridges or footpaths on railway

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bridges, the live load including dynamic effects should be taken as 5.0 kN/m^2 of the footpath area. For the design of foot-path on a road bridges or road-rail bridges, the live load including dynamic effects may be taken as 4.25 kN/m^2except that, where crowd loading is likely, this may be increased to 5.0 kN/m^2

Impact load The dynamic effect caused due to vertical oscillation and periodical shifting of the live load from one wheel to another when the locomotive is moving is known as impact load. The impact loads determined as a product of impact factor, and the live load. The impact factors are specified by different authorities for different types of bridges. The impact factors for different bridges for different types of moving loads.

Longitudinal forces – Longitudinal forces are set up between vehicles and bridge deck when the former accelerate or brake.

This loading is taken to act at a level 1.20 m above the road surface. No increase in vertical force for dynamic effect should be made along with longitudinal forces. The possibility of more than one vehicle braking at the same time on a multi-lane bridge should also be considered.

Thermal forces– The free expansion or contraction of a structure due to changes in temperature may be restrained bits form of construction. Where any portion of the structure is not free to expand or contract under the variation of temperature, allowance should be made for the stresses resulting from this condition. The coefficient of thermal expansion or contraction for steel is 11.7 x 10-6 /^{o}C

Wind load – Wind load on a bridge may act.
Horizontally, transverse to the direction of span.
Horizontally, along the direction of span.
Vertically upwards, causing uplift .
Wind load on vehicles

Racking force– This is a lateral force produced due to the lateral movement of rolling stocks in railway bridges. Lateral bracing of the loaded deck of railway spans shall be designed to resist, in addition to the wind and centrifugal loads, a lateral load due to racking force of 6.0 kN/m treated as moving load. This lateral load need not be taken

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into account when calculating stresses in chords or flanges of main girders.

Forces on parapets- Railings or parapets shall have a minimum height above the adjacent roadway or footway surface of 1.0 m less one half the horizontal widths of the top rail or top of the parapet. They shall be designed to resist a lateral horizontal force and a vertical force each of 1.50 kN/m applied simultaneously at the top of the railing or parapet.

Seismic load– If a bridge is situated in an earthquake prone region, the earthquake or seismic forces are given due consideration in structural design. Earthquakes cause vertical and horizontal forces in the structure that will be proportional to the weight of the structure. Both horizontal and vertical components have to be taken into account for design of bridge structures.IS:1893 – 1984 may be referred to for the actual design loads.

Forces due to curvature - When a track or traffic lane on a bridge is curved allowance for centrifugal action of the moving load should be made in designing the members of the bridge. All the tracks and lanes on the structure being considered are assumed as occupied by the moving load. This force is given by the following formula:

Erection forces– There are different techniques that are used for Construction of railway bridges, such as launching, pushing, cantilever method, lift and place. In composite construction the composite action is mobilized only after concrete hardens and prior to that steel section has to carry dead and construction live loads. Depending upon the technique adopted the stresses in the members of the bridge structure would vary. Such erection stresses should be accounted for in design. This may be critical, especially in the case of erection technologies used in large span bridges.

Load combinations

Stresses for design should be calculated for the most sever combinations of loads and forces. Four load combinations are generally considered important for checking for adequacy of the bridge. These are given in Table 7.2 and are also specified in IS 1915
- 1961.

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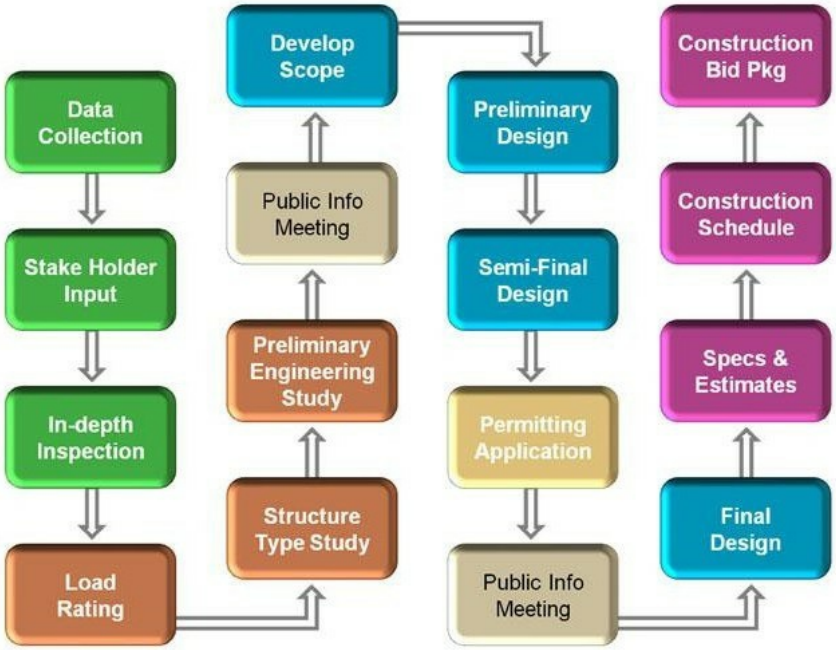
The Design Process



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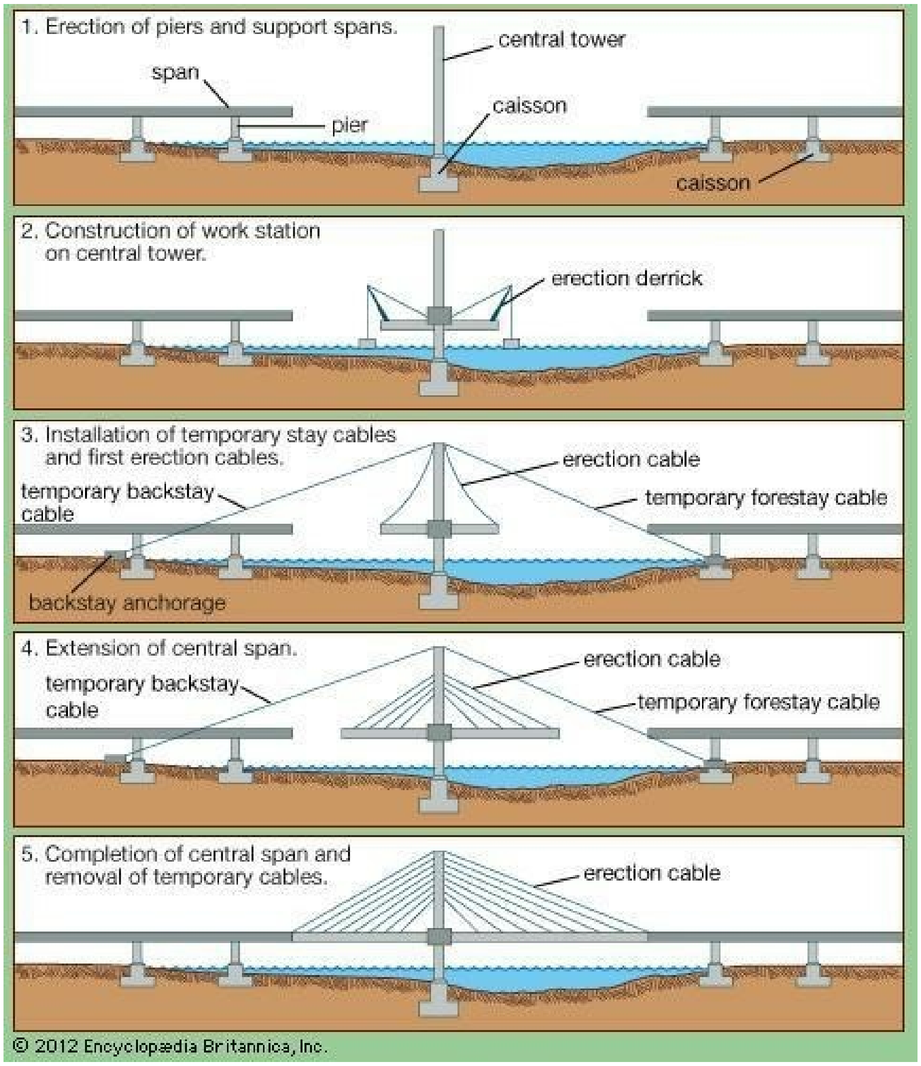

Foundation:

Foundation is the lowest part of a structure which provides base for superstructure which provides base for superstructure. It is an artificial arrangement provided to transmit the loads on the structure including the live load and dead load of the structure to the soil.

BRIDGE FOUNDATIONS

The foundations of a bridge are particularly critical because they must support the entire weight of the bridge and the traffic loads that it will carry.

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Common Foundation Types

- Spread Footing
- Piling
- Drilled Shafts

Types of Substructures

- Abutment-Type Substructures

- Abutment and Retaining Walls
- Anchored Walls
- Mechanically Stabilized

Earth Walls

- Prefabricated Modular Walls

- Pier-Type Substructures
- Concrete Pier
- Steel Pier
- Composite Steel & Concrete Pier

Loads on Substructures

- Loads from Superstructure
- Loads on Substructure
- Load

Combinations Type Of

Excavation

"Dry" Excavation:

- Generally stable with no waterway or de-stabilizing groundwater

"Wet" Excavation:

- In or near water or significant groundwater

Bridge Bearing

- Bridge Substructures

Loads on Substructure

- Abutment
- Piers

Bearing

Bearing is a structural device positioned between bridge superstructure and substructure

Roles of Bearing:

- Transmit load from superstructure to substructure.
- Accommodate relative movements between superstructure and substructure.

Types:

Fixed Bearing - Rotational movement only

Expansion Bearing -

Rotational movement

Translational movement

Types of Bearing

- Rocker Bearin
- Pin Bearing
- Roller Bearing
- Slider Bearing
- Elastomeric Bearing
- Curved Bearing
- Pot Bearing
- Disk Bearing

Forces and Movements on Bearing

Forces on Bearing

- Vertical forces - from dead loads and live load
- Horizontal forces - from wind, earthquake

Movements on Bearing

- Horizontal translation caused by creep, shrinkage, and thermal

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Expansions.

- Rotations caused by traffic, construction tolerances, and uneven Settlement of foundations

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If no movement is allowed, then the bearing must be able to resist The force due to the deformation.

Elastomeric Bearing

Made up of natural or synthetic rubber Very flexible in shear but very stiff against volumetric change Can accommodate both rotational and translational movements through the deformation of pad Steel or fiberglass is typically used to reinforced the pad in alternate layers to prevent it from “bulging” under high load, allowing it to resist higher loads.

Elastomeric Bearing with Slider

Steel slider with Teflon (PTFE _ polytetrafluoroethylene) coated surfaces may be used in combination with elastomeric bearing to allow for more translations.

Pot Bearing

Pot bearing consists of steel container (“Pot”) with elastomeric pad Inside Can resist much larger loads than conventional elastomeric bearing notation is accommodated by deformation of the elastomeric Sliding surface is used to allow for translation

Consider the following factors when selecting a bearing to use:

- Vertical and Horizontal Loads
- Translational and Rotational Movements
- Available Clearance (footprint/ height)
- Environment (corrosion/ temperature range)
- Initial Cost
- Maintenance Cost
- Availability
- Owner’s Preference

UNIT II

SHORT SPAN BRIDGES

Load distribution theories - Analysis and Design of Slab Culverts, Tee beam and Slab bridges, Design of panel and cantilever for IRC loading.

INTRODUCTION:

Bridges may be of RC or steel construction. The beams are supported on intermediates piers and end abutments. In India bridges for roads are designed as per the recommendation of the India Road Congress. Rail bridges are designed on the recommendation of specifications of the Indian Railway standard code of Practice.

DESIGN AND ANALYSIS OF SLAB CULVERT:

(a) General Design features:

The RC Slab –type deck is generally used for small spans. This type of superstructure is economical for spans up to about 8m. Slab decks are simpler to construct due to easier fabrication of formwork and reinforcements and placements of concrete, generally, slab decks are supported at the ends on piers or abutments. The deck slab is designed as a one-way slab to support the dead load and live load with impact.

The deck slab is generally designed to resist the worst effect of either one lane of IRC class AA tracked vehicle or tow lanes of Class A load train, based on. Analytical investigations, victor has reported the use of class AA wheeled vehicles for spans up to 4m & class AA tracked vehicles for spans greater than 4m for computation of live load BM for computations of Maxi shear due to live loads in two lane bridges Class AA tracked vehicle should be used for all spas form 1m to 8m.

The design of distribution reinforcement is based on 0.3 times the live load moment M_L and 0.2 times the dead load moment M_d and is given by formula = $0.3M_L$ +

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$0.2\ M_d$.

(b) Analysis of deck slabs:

(i) Slabs spanning in one direction:

For slab spanning in one direction, the dead load moments can be computed directly assuming the slab to be simply supported between the supports. Bridge deck slabs have to be designed for IRC loads specified as class AA or A depending on the importance of the bridge. For slabs supported on two opposite sides, the maximum BM caused by a wheel load may be assumed to be resisted by an effective width of slab measured parallel to the supporting edges.

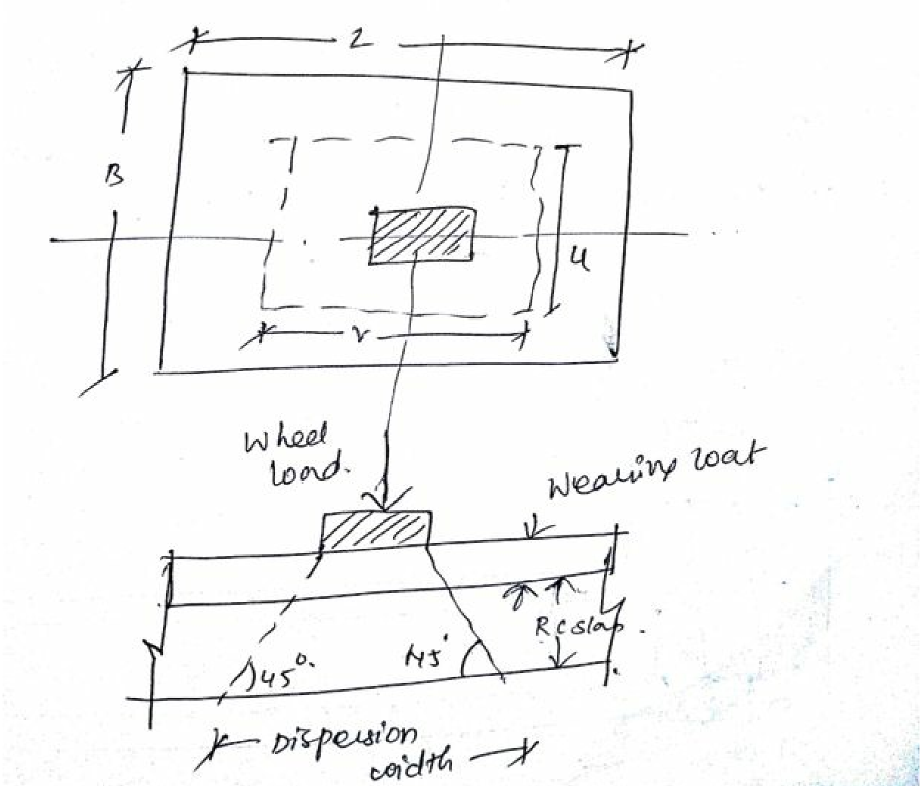
For a single concentrated load the effective width of dispersion,

$$be = k.x\,(1- x/l) + bw$$

(ii) Slabs Spanning in two directions:

In case of bridge decks with T- beams and cross girder, the deck slab is supported on all the four sides and is spanning in two directions. The moments in the two directions can be computed by using the design curves developed by Mr. M.Pigeaud.

The method developed by pigeaud is applicable to rectangular slabs. Supported freely on all the four sides and subjected to a symmetrically placed concentrated load as shown.



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The dispersion of load may be assumed to be at 45 degree through the wearing coat and deck slab.

The BM are

computed. M1 =

$(m_1+\mu m_2)$ W

M2 = $(m_1+\mu m_2)$ W

The values of moment co-efficient m1 & m2, depend upon the parameter (u/B), (v/L) & K.

DESIGN PROBLEM:

Design an RC slab culvert for a national highway to suit the

following data. A two-lane carriage way (7.5m wide)

Foot path on either side (1 m

wide) Clear span = 6m

Wearing coat = 80mm

Width of bearing = 0.4

m

Materials – M25 Grade concrete & Fe 415 grade HYSD bars

Loading: IRC Class AA tracked vehicle. Design the RC deck slab and sketch the details of reinforcements in the longitudinal and cross sections of the slab.

Step (1) Given data:

A two-lane carriage way (7.5m

wide) Foot path on either side (1

m wide) Clear span = 6m

Wearing coat = 80mm

Width of bearing = 0.4

m f_{ck} =25 N/mm^2

f_y = 415 N/mm^2

Loading: IRC Class AA tracked vehicle.

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Step (2): Permissible stresses:

Permissible stresses in concrete in bending compression = 8.3

N/mm^2 Permissible tensile stress in steel = 200

N/mm^2 m = 10, j = 0.9, Q = 1.1

Step (3): Depth of slab and effective span

Assume thickness of slab at 80mm per meter of span for highway bridge decks.

Therefore overall slab thickness = 80 x 6 = 480

mm Adopt overall slab thickness as 500mm

Using 25mm dia bars with clear cover of

25mm Effective depth d = 500 – (25+12.5)

= 462.5mm

Bearing width = 0.4m =

400mm Effective span is the

least of

(1) Clear span + Effective depth = 6+0.4625 = 6.4625 m

(2) Centre to centre of bearings = 6+0.40 = 6.40

m Effective span L = 6.40m

Step (4): Dead load bending moment

Dead wt. of slab = 0.5 x 24 = 12 KN/m^2

Dead wt. of Wearing coat = 0.08 x 22 = 1.76 KN/m^2

Total load = 13.76 KN/m^2

Dead load BM = $Wl^2/8$

$$= \frac{13.76 \times 6.4}{8}$$

= 70.45 KN-m

Step (5): Live load bending moment

Impact factor = 25-15/4(6.4-5)

= 19.75%

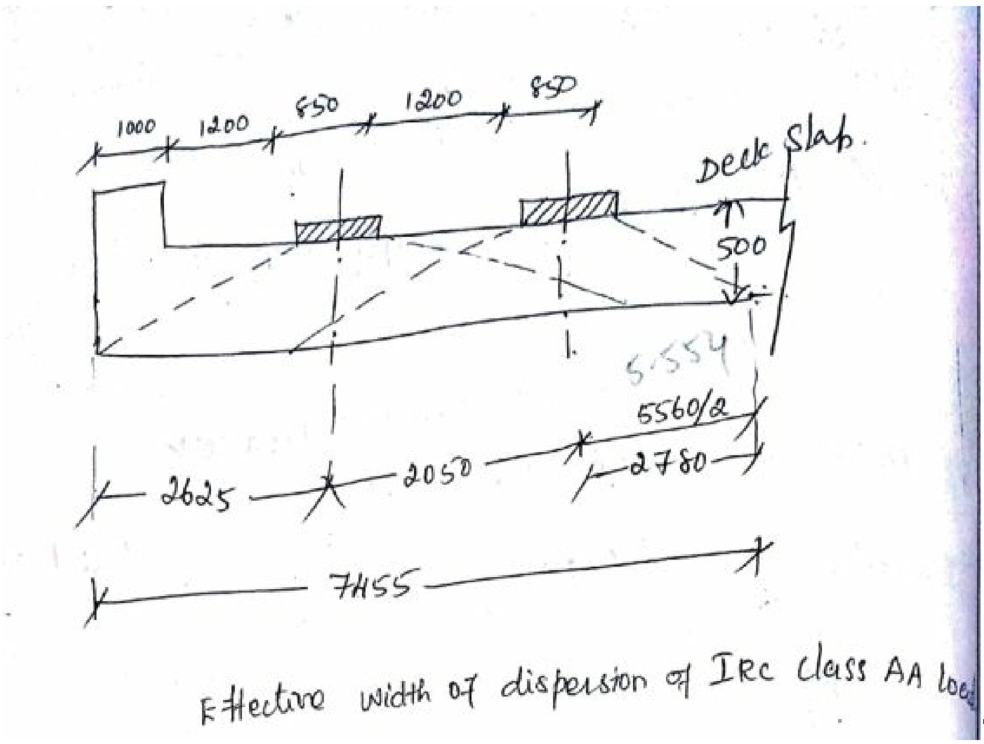
Effective width of slab , perpendicular to span

be = k.x (1- x/l) + bw = 1.01 m

Position of load for Maximum B.M

beff =2.84 x 3.2 [1-3.2/6.4] + 1.01

=5.554 m



The tracked vehicle is placed close to the kerb with the required minimum clearance as shown.

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Effective width of dispersion of IRC Class AA loads.

Net effective width of dispersion = 7.455 m

Total load of two tracks with impact = 700 x 1.197

Average Intensity of load = Load / Area

= 23.61 KN/m^2

Maxi BM due to live load is given by,

$$M_{max} = 179.81 - 66.86$$

$$= 112.94 \text{ KN.m}$$

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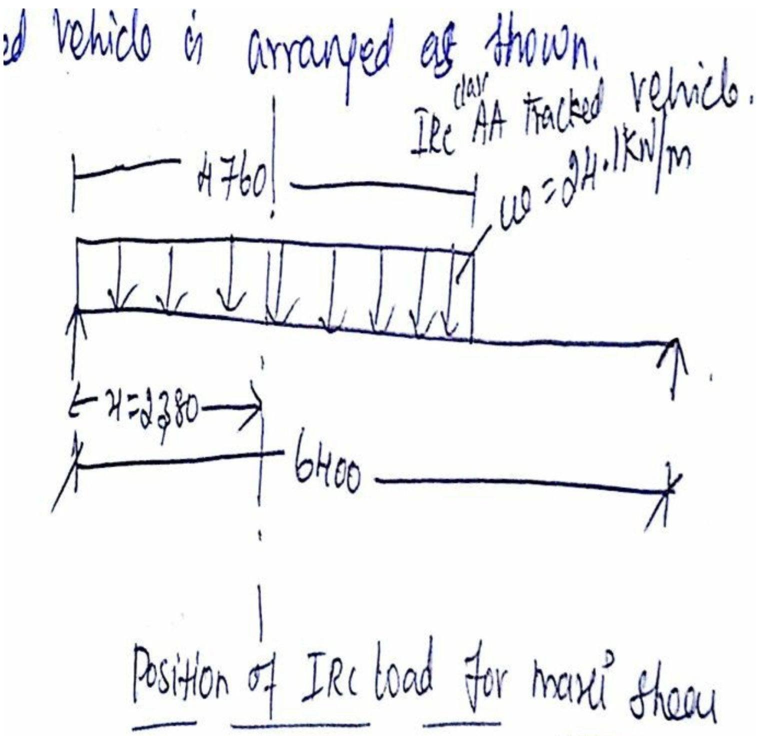
Total design BM = D.L + L.L

=112.94 + 70.45

BM =183.34 KN.M

Step (6): Shear due to class AA tracked Vehicle:

For maxi shear at support, the IRC class AA tracked vehicle is arranged as shown.



Effective width of dispersion is given by

be = k.x (1- x/l) + bw = 5.256 m

Width of dispersion =

2625+2050+2628

=7303 mm

=7.303 m

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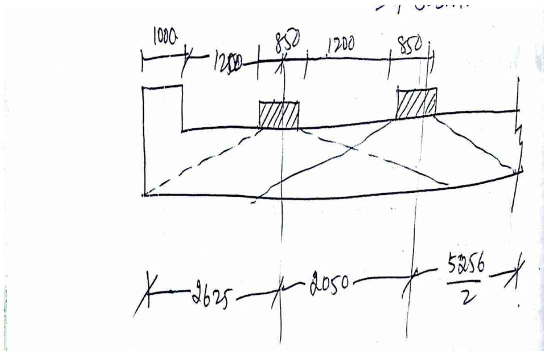

Average of Intensity of load W = Load/ Area

$= 24.1\ KN/m^2$

Shear force V_A = 72 KN

Dead load shear = 116.85 KN

Total design shear = 44.8 + 72.05 = 116.85 KN

Step (7):Design of Deck slab:

$Mu = 0.138\ fck\ bd^2$

D = 230.5mm < 462.5 mm

Main Reinforcement:

Mu = 0.87fy Ast d [1- (Ast.fy/b d fck)] From this Ast = 1142.98 mm^2

Spacing of 25mm dia bars = (ast /Ast) x 1000

= 450 mm Adopt 25mm dia bars at 450mm c/c Main Reinforcement. BM for distribution Moment:

$= 0.3M_L + 0.2Md$

= 47.92 KN-M

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Using 12 mm dia bars

Effective depth = 462.5 (12.5+6)

= 444 mm

Ast = 30 mm^2

Provide 12 mm dia bars at 100mm C/c distribution reinforcement. Minimum Reinforcement:

Step (8): Check for Shear stress:

= 0.12% b D = 600 mm^2

Shear stress in the slab are checked as follows

Design shear stress τ_v = V/bd = 0.249

N/mm^2

p = 100 Ast / b d

= 0.24

τ_c = 0.36

N/mm^2 For solid slabs = K τ_c, K =

1

Since $\tau_v < \tau_c$

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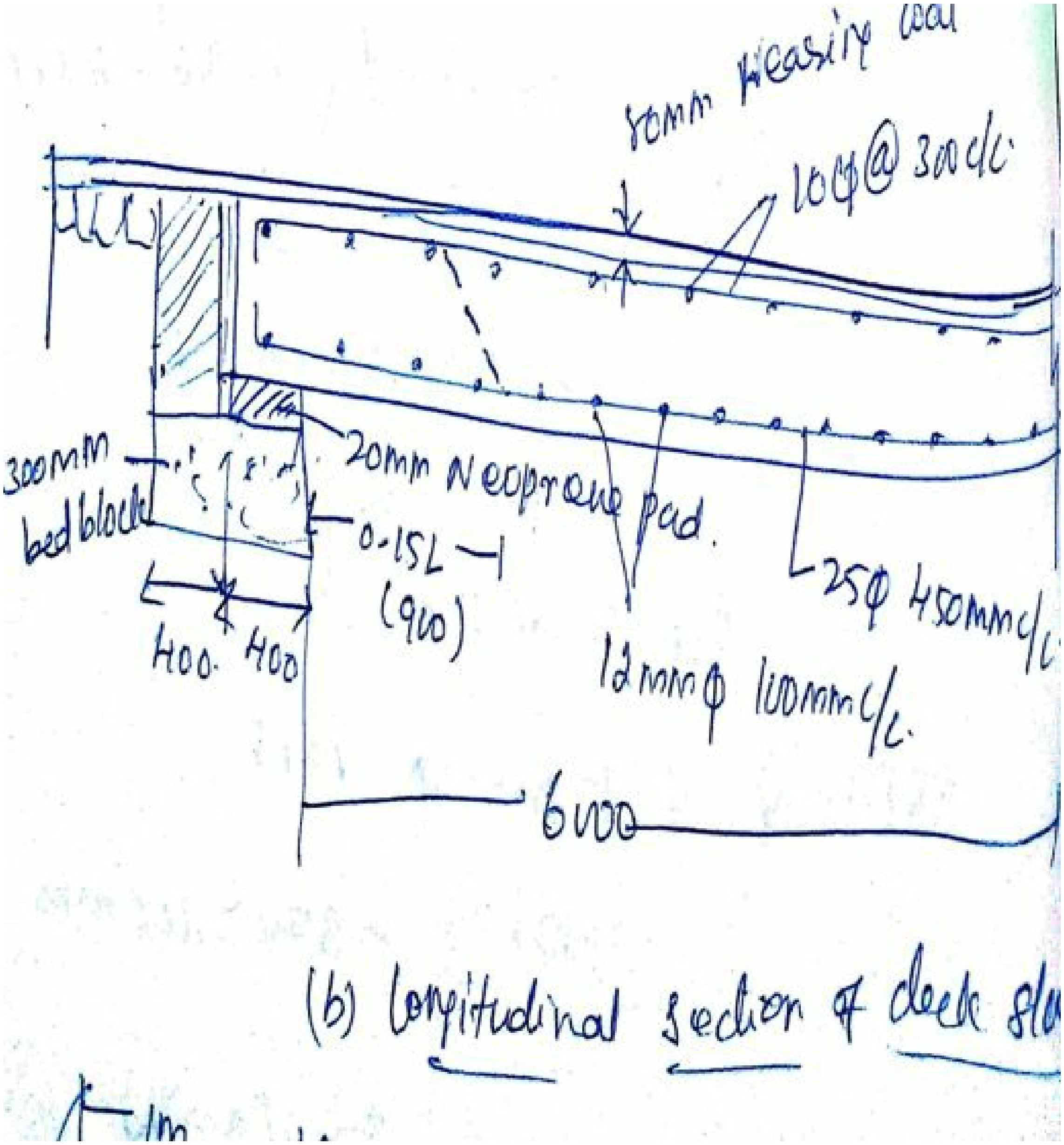

The shear stresses are within safe permissible limits.

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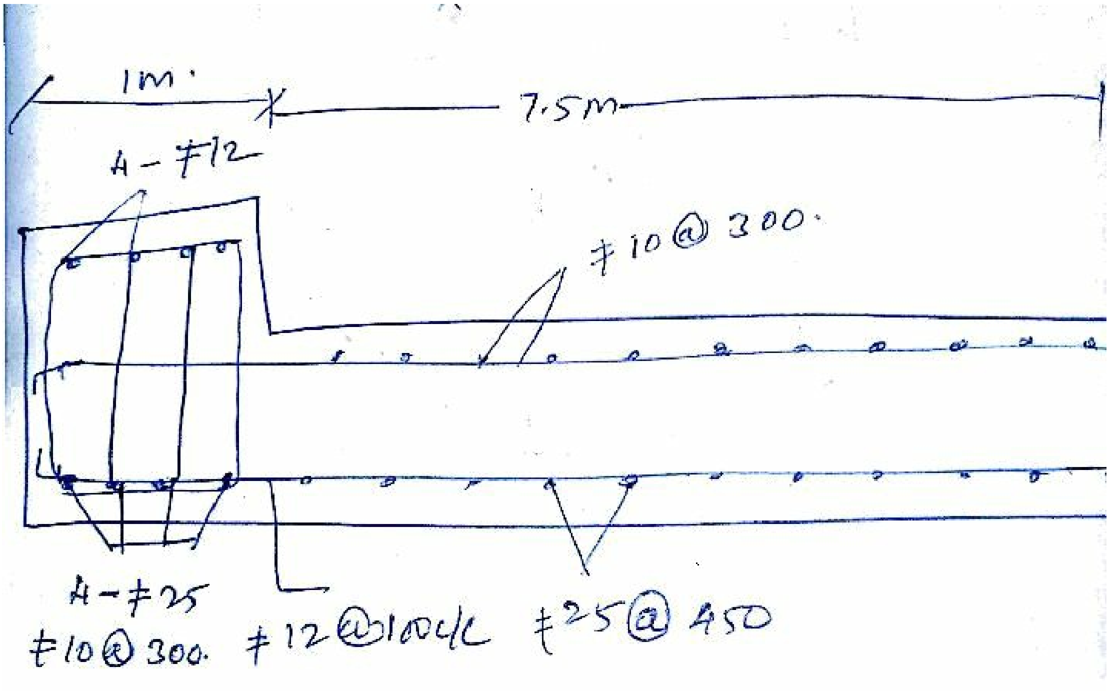

(a) Cross section of deck slab.

Analysis and Design of T-Beam & slab Decks:

A typical tee beam deck slab generally comprises of the longitudinal girders, continuous deck slab b/w the tee beams and cross girders to provide lateral rigidity to the bridge deck. The distribution of live loads among the longitudinal girders can be estimated by any of the following rational methods.

1. Courbon's method
2. Hendry-Jaegan method
3. Guyon-Massonnet-method

The longitudinal girders are spaced at intervals of 2 to 2.5m and cross girders are provided at 4 to 5 m intervals.

Among those methods, courbon's method is the simplest and is applicable when the following conditions are satisfied.

(1) The ratio of span to width of deck is greater than 2 but less than 4

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(2) The longitudinal girders are interconnected by at least five symmetrically spaced cross girders.
(3) The cross girders extended to a depth of at least 0.75 of the depth of the longitudinal girder.

Courbon's method is popular due to the simplicity of computations.

DESIGN PROBLEM:

Design of R.C.C Tee beam and slab deck to suit the following data.

Clear width of road way	= 7.5 m
Effective span of girders	= 16m
Width of kerbs	= 600 mm
Thickness of wearing coat	= 80 mm
Number of main girder	= 4
Spacing of main girder	= 2.5 m
Spacing of cross girder	= 4 m

Type of loading: IRC class AA tracked vehicle Materials M 20 grade concrete & Fe 415 grade steel. Design the interior panel and main girders and cross girders and sketch the typical details of reinforcement.

Solution:

Step (1) Given Data :

Clear width of road way	= 7.5 m
Effective span of girders	= 16m
Width of kerbs	= 600 mm
Thickness of wearing coat	= 80 mm
Number of main girder	= 4
Spacing of main girder	= 2.5 m
Spacing of cross girder	= 4 m

f_{ck} =25 N/mm^2

f_{y} = 415 N/mm^2

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Loading: IRC Class AA tracked vehicle.

Step 2 Permissible stresses

σ_{cb} = 6.7 N/mm ^{2}

σ_{ct} = 200 N/mm ^{2}

Step 3: Cross section of deck slab:

Four Main girders are provided @ 2.5

Centers. Adopt thickness of deck slab 80 to

250mm Width of Main girder

=300mm

Kerbs 600 mm wide & 300mm deep

Cross girder are provided @ every 4m intervals

Depth of main girder = 1600 mm at the rate of 100mm per meter

span. The depth of cross girder is taken as equal to that depth of main

girder.

The cross section of the deck & the plan showing the spacing of main & cross girders as shown.

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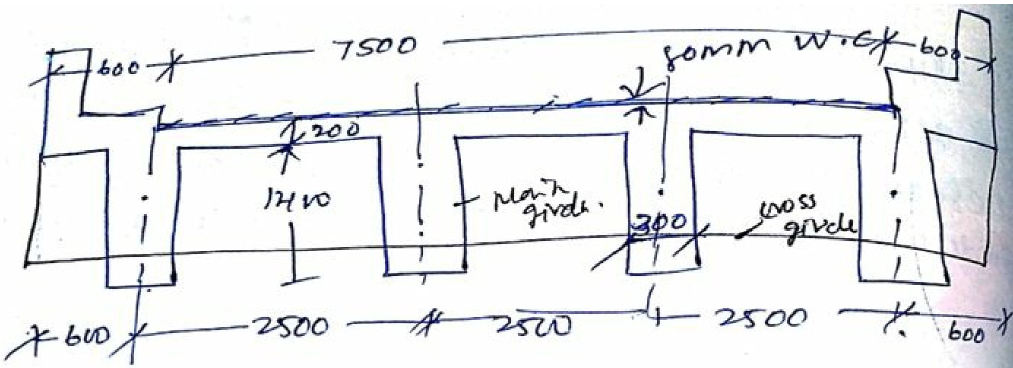
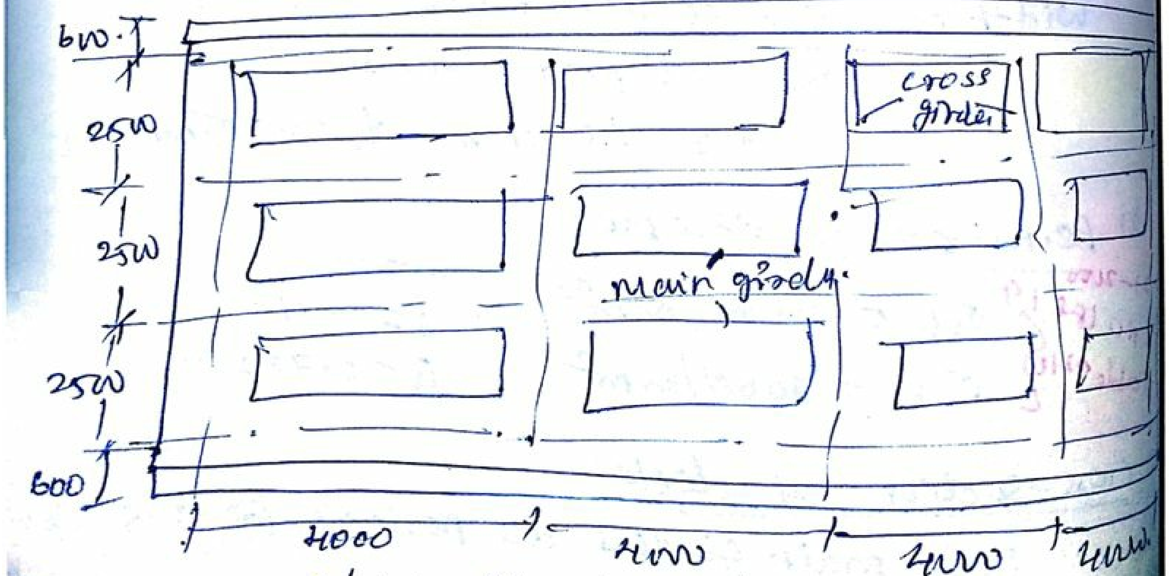


(a) Cross section of Bridge deck.



T-ee beams slab bridge deck (b) Plan of Bridge deck

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Step 4: Design of Interior slab Panels:

(a) Bending moments:

Dead wt of slab = 1 x 1x 0.20 x 24 = 4.80 KN/m^2

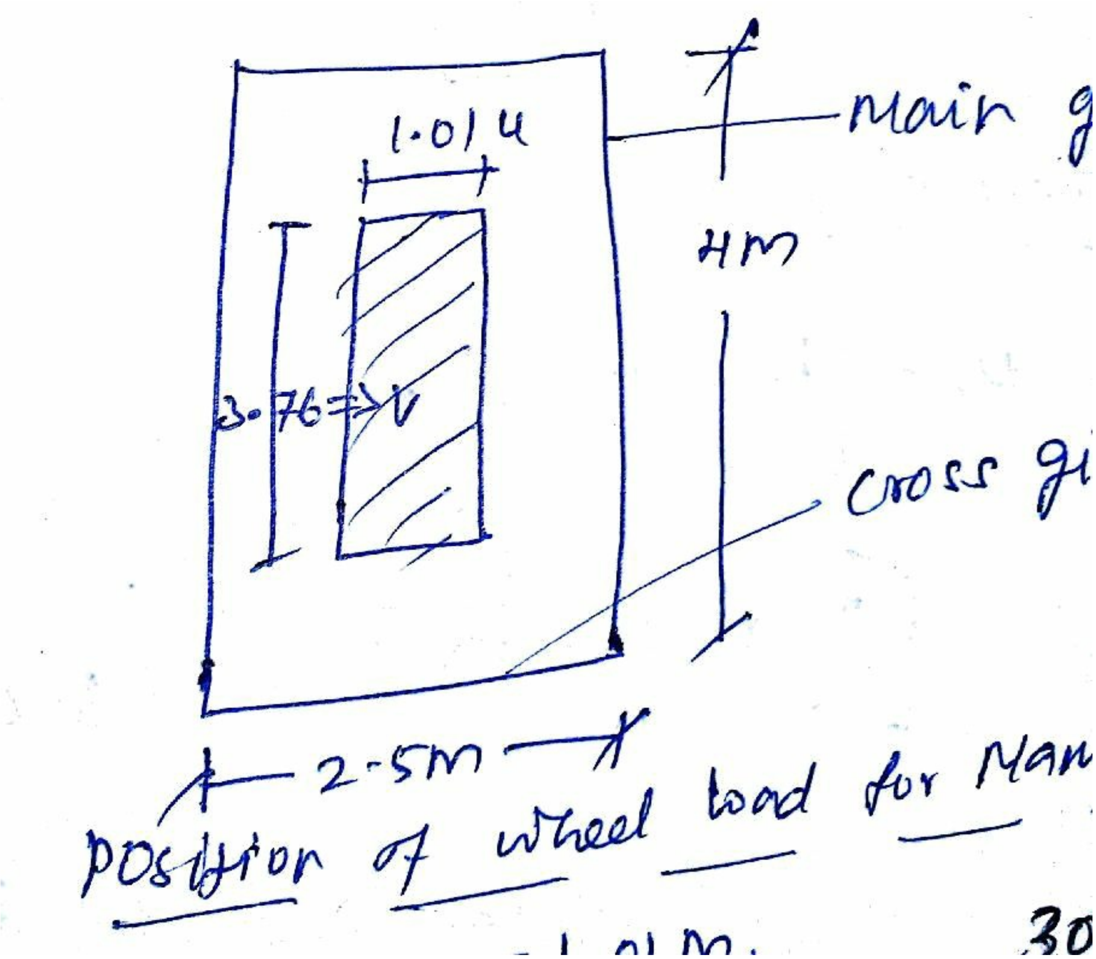
Dead wt of W.C = 0.08 x 22

= 1.76 KN/m^2

Total dead load = 6.56 KN/m^2

Live load in IRC for tracked vehicle

One wheel is placed at the centre of panel.



U = 0.85 + 2 x 0.08 = 1.01 m

V = 3.60 + 2 x 0.08 = 3.76 m

U or V [Contact area length + 2 x W.C]

B = 2.5

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m L =

4.0 m

U/B = 1.0 / 2.5 =

0.404 V/L = 3.76/4.0

= 0.94

K = B/L = 2.5/4 = 0.625

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Referring to Pigeaud's Curve

Moment co-efficient m_1 & m_2 = 0.024

The bending moments in the short & Long span

$M1 = (m_1 + \mu\ m_2)$

$W\ M2 = (m_2 + \mu\ m_1$

$)\ W$

Short span B.M

$M \text{ or } M1 = (m_1 + \mu\ m_2)\ W$

$\mu = 0.15$

$= [0.085+0.15x0.024]350$

Total load on 700 KN per each span 350 KN

$M1 = 31.01$ KN – m

As the slab is continuous,

Design BM = 0.8 M_B (80% Actual)

Design BM including impact & continuity factor

M_B (short) = 1.25 x 0.8 x 31.01

= 31.01 KN-m

M_L (long) = $(m2 + \mu\ m1$

$)\ W$ Design (or) M_2 = 12.86

KN-m

(b) Shear forces:

Dispersion in the direction of span = 0.85 +2(0.08+0.2)

=1.41

For maxi shear, load in kept such that the while dispersion is in span, the load is kept at 1.41/2 = 0.705 from the edge of the beam.

Effective width of slab = $\alpha\ a\ [1- a/lo]$

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+ b, Clear length of panel (or) lo =

3.76 m (u) Clear span L =

2.2 m

B/L = 3.76/2.2 = 1.70

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α Values from table for continuous slab

$\alpha = 2.56$

a = 0.705 from face of support

$b_{eff} = 1.228 + 3.76$

= 4.998

Load per Meter width = 350/4.99 = 70.16 KN

Shear force = 47.67 KN

Shear force with impact = 59.58 KN.

(c) Dead load BM & SF:

Dead load = 6.56 KN/M2

Total load on panel = 4 x 2.5 x 6.56

= 65.60 KN

U/B =1 and V/ L =1 as panel is loaded with Uniformly distributed load.

K = B/L = 2.5/4 = 0.625

and 1/K = 1.6

From Pigeaud's curve

$M_1 = 0.049$ & M2 = 0.015 $M_1 = (m_1 + m_2)$ W

= 3.362 KN-m

Taking Continuity into effect = 0.8 x 3.362

= 2.689 KN-

m M_2 = (m_2 +μ

m_1) W

= 1.466 KN-m

Taking continuity into effect = 0.8 x 1.466

= 1.172 KN-m

Dead load shear force = 7.216 KN.

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(d) Total design moment and shear:

Total M_d=31.01+3.688=33.698

kN-m Total

M_l=12.845+1.174=14.019kN-m

Total shear force=59.5+7.216=66.716kN.

(e) Design of section:

Effective depth d= $((33.698*10^6)/(0.762*1000))^{1/2}$

=210.29mm

Adopt eff. Depth=210mm

Adopt overall

depth=250mm

A_{St} = $((33.968*10^6)/(200*0.91*210))$

=881mm^2

Short span

Use 16mm dia HYSD bars at 150mm c/c

(Ast=1341mm^2) Spacing of reinforcement in slabs

not to exceed 150mm.

Effective dept for long span using 10mm dia bars = 210-8-

5=197mm Ast= $((14.019*10^6)/(200*0.91*197))$

=391 mm^2

Long span

Use 100 mm dia bars at 150 mm c/c (Ast=520)

(F)Check for shear stress:

Nominal shear stress τ_v = V/bd

= $((66.716*10^3)/(210*10^3))$ = 0.317 N/mm^2

100Ast/bd = 0.41 N/mm^2

K τ_c=1.1X0.41 = 0.415N/mm^2

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Since, $\tau_v < \tau_c$. Therefore, Shear stresses are within safe permissible limits.

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Step (5) Girder design

Live load bending moment in girder-

Eff span of girder =16m

Impact factor for IRC class AA loading =10%

Bending moment including impact and reaction factors in outer girder A is

computed as, M = 2401*1.1*0.382
= 1008.9 kN-m

Dead load B.M in girder A

Overall depth of girder = 1600 depth of mm

Depth of rib = 1600-200 = 1400mm
Self wt of rib = 300mm

Self wt of rib/m = 0.3X1.4X24X1 = 10.08 kN-m

The cross girder is assumed to have the same cross sectional dimension of main

girder. Wt of cross girder= 10.08 kN-m
Reaction of main girder= 10.08X2.5

= 25.2 KN

Reaction from deck slab on each girder = 16.25k

kN-m Total dead load on girder = 16.25+10.08
= 26.33 kN-m

The max B.M in the exterior girder A is computed as

$B.M_{max}$= ((926.33*16^2)/8) + ((25.2*16)/4)+((25.2*16)/4

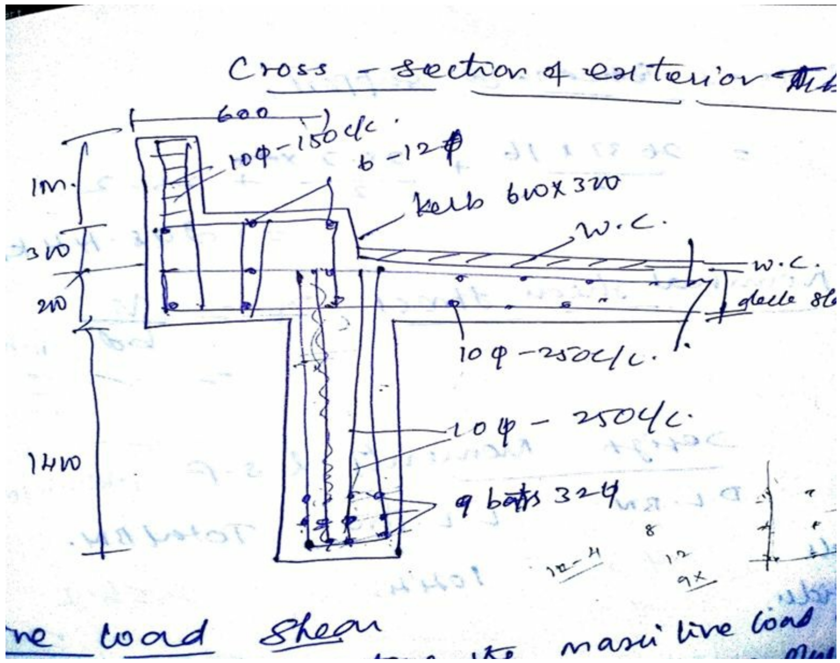
= 1044.16 kN-m

B.M in centre = 26.33*8*(8/2)+25.4*4

=1047.36 kN-m

Total design moment

Design moment in girder, DL moment = 1047.36 kN-m



LL moment = 1008.9 kN-m Total =2056.26 kN-m

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Step (6): Design of cross girder:

Self wt of girder (or) self wt of rib/m = 10.08

kN-m Dead load from slab= 2X

(1/2)X2.5X1.25X6.56

= 10.08 kN-m

Uniformly distributed load = 20.5/2.5 = 8.2 kN-m

Total load on the cross girder = 10.08+8.2 =18.28 kN-m

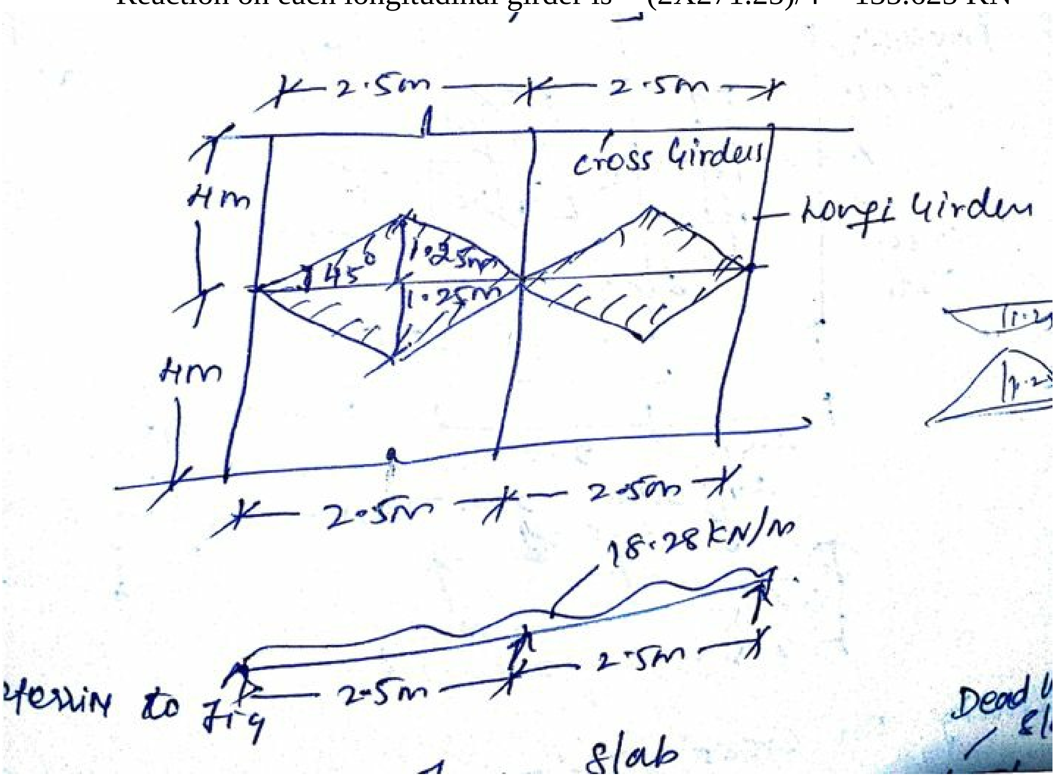
Assuming the cross girder to be rigid, reaction on each cross girder = ((18.25X50)/3)

= 30.46 kN-m

Load coming on cross girder = (350(4-0.9))/4 =

271.25 KN Assuming the cross girder as rigid,

Reaction on each longitudinal girder is = (2X271.25)/4 = 135.625 KN



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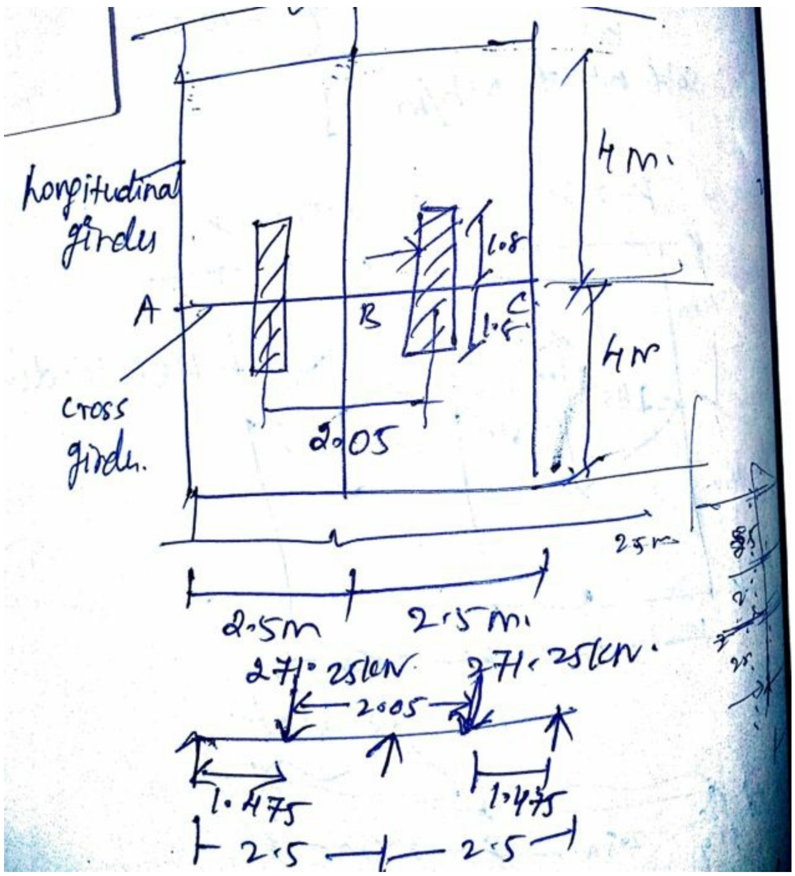

Max B.M in cross girder under the load = 135.625*1.475 = 200

kN-m LL B.M including impact = 1.1*200 = 220.50 kN-m

DL B.M at 1.475m from support = $30.47*1.475-(18.28*1.475^2)/2$ =

25.05 kN-m Total design B.M = 220.05+25.05 = 245.10 kN-m

Live load shear including impact = ((2*27125)/*2)*1.1 =

149.18 kN-m Total design shear = 30.47+147.18 =179.65 kN

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Assuming an eff depth for cross girder is

1540mm, Ast = M/ (σ_{ast}jd)

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$= ((245.10X10^6)/ (200X0.9X1540))$

$= 884.19\ mm^2$

Provide 3 bars of 20mm dia (Ast=941 mm^2)

Shear stress, $\tau_v = ((179.65*10^3)/(300*1540)) = =0.38$

N/mm^2 Using 10 mm 2 legged stirrups,

$S_v = ((\sigma_{as}A_{sv}d)/V_u) = ((200*2*79*1540)/ (179.65*10^3))$

$= 270\ mm$

Adopt 10 mm dia 2legged stirrups at 150mm c/c throughout the length of the cross girder.

UNIT-III

LONG SPAN GIRDER BRIDGES

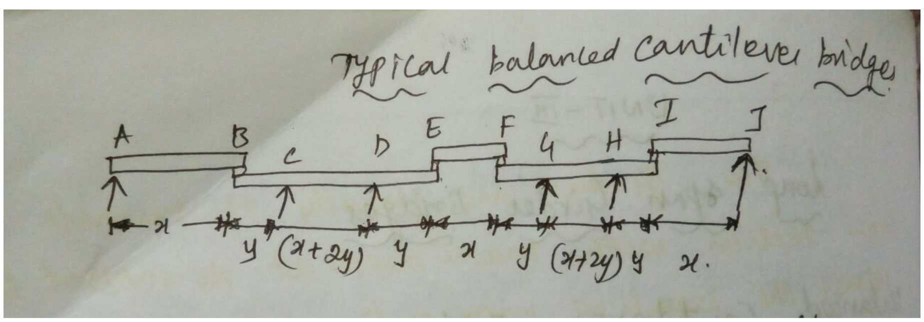
Design Principles of Continuous Bridges, Design of balanced cantilever bridges - Deck slab - Main girder.

BALANCED CANTILEVER BRIDGES:

General Features:

For spans more than 15 to 20m.The tee beam bridges are not economical .Since, the depth of girder are larger with large quantities of reinforcement. Continuous span structures or R.C arches are not safe to construct on yielding soils. Since, a slight settlement of the structure affects the nature of stresses considerably. On such occasions where soils are of slightly yielding nature and for spans in the range of 20 to 30m, balanced cantilever bridges offer an ideal solution for the following reasons:

- Advantage of continuity.
- Reduction in moments.
- Not affected by settlement of supports.
- Basically a balanced cantilever bridge consists of freely supported spans with cantilever extending on either side serving as supports for adjacent spans. Arrangement Of Supports typical balancedcantileverbridge with simply supported spans and cantilever spans as shown



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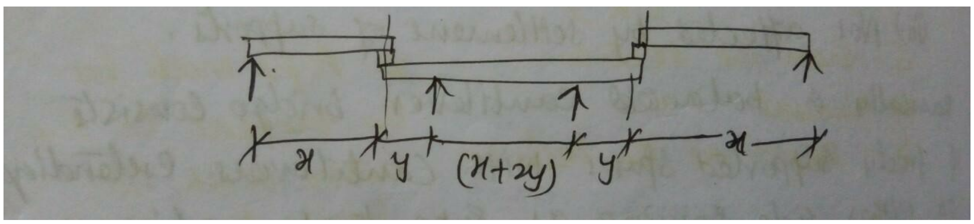
AB, EF, IJ = Simply Supported Spans

CB, DE, HI = Cantilever Spans

For economy, the main criterion is that the maxi moments in the beams are the least. For this the dimensions X and Y should be adjacent accordingly. Under normal condition the ratio

(X/y) = 4 to 5

This reduced the BM to about 50% of the moment for a simply supported beam of span(x+2y).The simplest arrangement of balanced cantilever bridge is of minimum three spans as shown in fig



The central span can be up to 30m.If the length of the bridge exceeds, and then a 5 span arrangement is selected with a simply supported span at the centre.

The main girders consist of tee beam and slab, the spacing between being arranged depending upon the number of lanes of traffic.

Design Features:

The girders are usually of variable cross section. With maximum depth at the support to clear the negative moments, and with minimum depth at the centre of span and end supports.

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Influence lines are drawn for all critical section such as “the supports centre of middle span and simply supported spans for BM and SF.

Each section is designed for the maxi BM obtained from the arrangement of I.R.C loading. The sections of the beams can be suitably varied from supports towards the centre.

Shear Variation:

Due to the change in the section of the beam, the variation of shear can be V obtained by the relation

$S = (M/h) \tan \alpha$

S - Shear at section

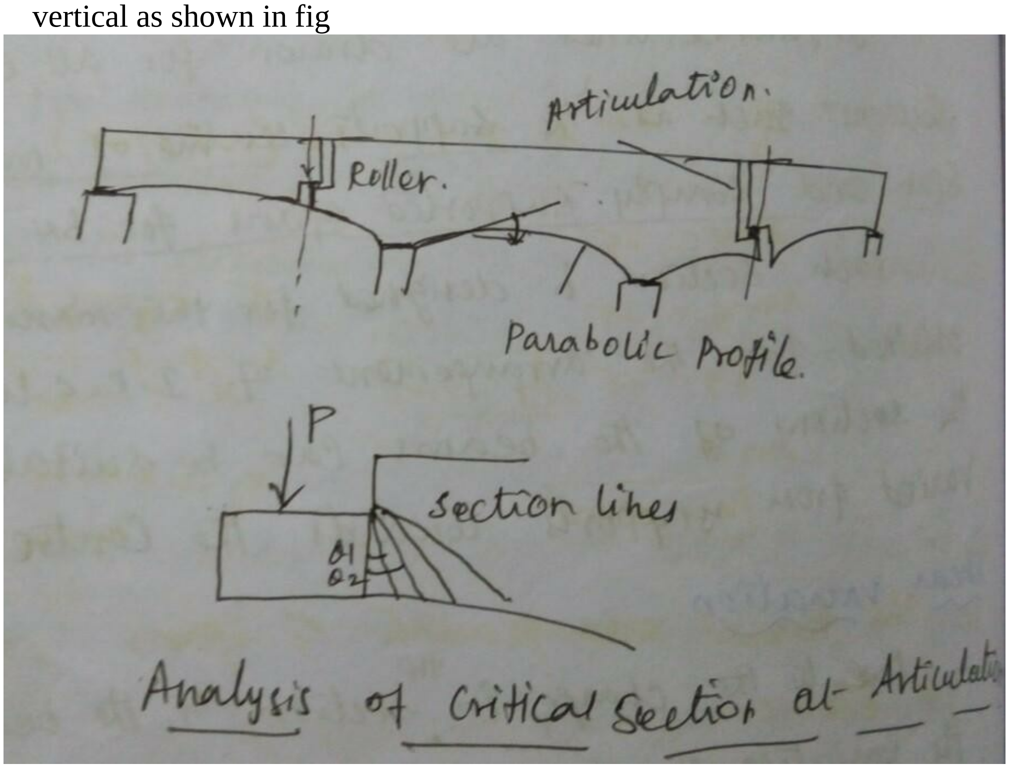
M - Moment at Section

H - Depth at the section

α – Angle made by the tangent to the curve with the horizontal.

Articulation:

The junction of cantilever and simply supported span is referred to as ARTICULATION. Maximum tensile stresses are checked in section 1, 2,3,4,etc inclined at $\theta_1, \theta_2, \theta_3, \theta_4$,.. To the vertical as shown in fig



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The critical sections are designed for BM, normal thrust and shear. The beam is supported on steel or concrete rocker and roller bearings or neoprene pad bearings at the supports.

DESIGN PROBLEM:

Design double cantilever bridges to suit the following data:

Total length of bridge=77m

Road Width=1.5m between kerbs Foot

Paths=1.8m on either side Spacing of

Tee Beams=1.8m Heading IRC class

AA tracked vehicle

Materials: M25 grade concrete and Fe415 grade HYSD bars.

Design the salient structural elements of the bridges and sketch the details of reinforcement.

Solutions:

Step (1): Arrangements of Spans

Total length of the bridge deck= 77m j=1.10

Central span=27m Q=0.90

End span=25m m=10

σ_{st} = 210 N/m^2

x/y =4m and x+y=25(end span)

x+y=25

4y+y=25

5y=25, y=5m

And x=20m x/y=4 and x+2y=27

4y+2y=27

6y=27, y=27/6 = 4.5m

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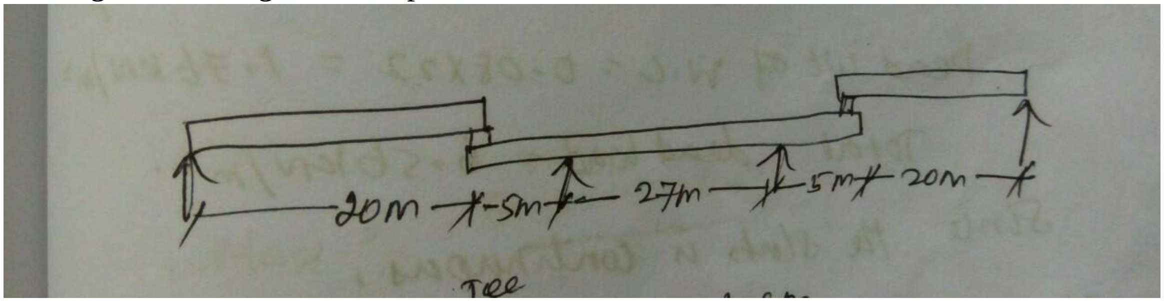
The general arrangement of spans is shown:

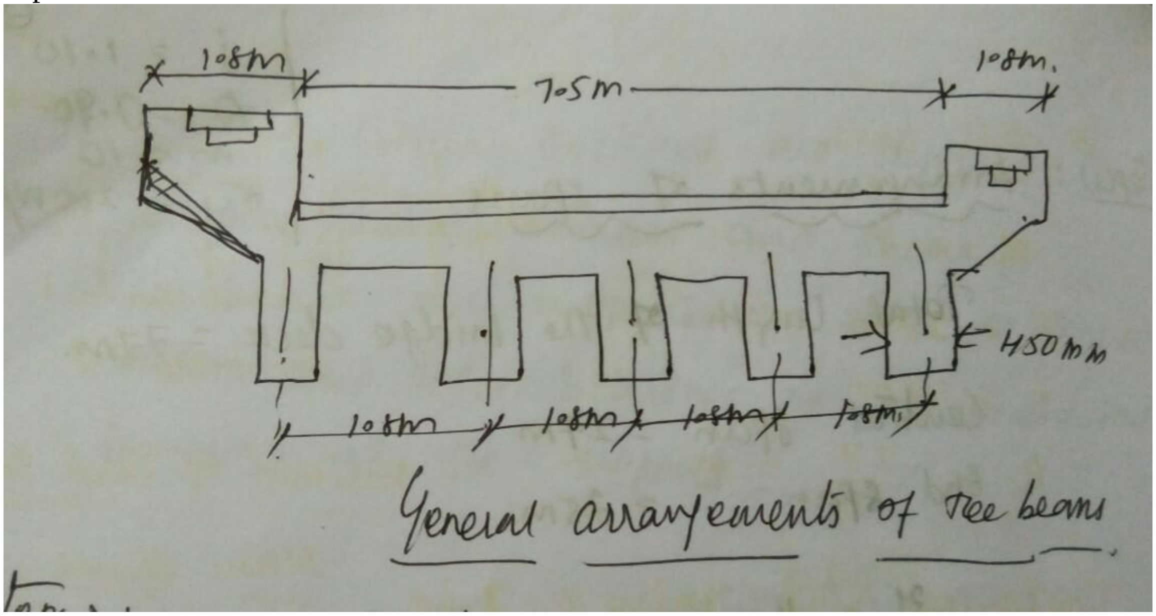


Spacing’s of Tee beans=1.8m The

width of beam=450mm Foot

paths=1.8m on either sides

The cross section of bridges deck showing the general arrangement of Tee beans and foot path is shown:



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Step (2): Design Of Deck Slab

Thickness of deck slab assumed as 200mm

Thickness of wearing coat=80mm

Width of main girder=450mm

Spacing between main girder=1.8m

The slab is supported and continuous over Tee beams.

Dead wt of slab=0.2×24=4.80 KN/m^2

Dead wt of W.C =0.08×22=1.76 KN/m^2

Total dead load=6.56 KN/m^2

Since the slab is continuous

Maxi D.L.BM=0.8X6.56X1.8^2/8 =2.125 KN-m

The live load BM will be maxi when the I.R.C class AA tracked vehicle wheel is placed at the centre of span.

b_w= 0.85+2(0.08) = 1.01m.

Effective width of slab perpendicular to span in expressed as, b_e

= αa (1-a/10) + b_w.

In this case,

D= 0.9

B=27m.

L=1.8m.

B/L=15

Continues slab, (value ∠ =

2.60 $B_{w\,=}$ 3.6+ (2x0.08) =3.76m

B_e= ∠ a (1-

a/10) B_e= 4.93m

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B_e=2.6x0.505(1-(0.505/1.8))+3.76=4.7m

Average intensity of load = 350/ (1.01x4.93) =>70.3KN/m^2

Max= {(70.3x1.01x0.9)/2}-{(70.3x1.01^2)/5)} =>23KN-m
Live load moment=1.25x0.8x23=23kN-m

Total design moment= (Dead load Bending Moment + Live Load Bending Moment)

=2.125+23

=25.125+23

=25.125KN-m

Effective depth d = $\sqrt{(\frac{M}{\sigma st.j.b})}$=152mm

Adopt effective depth d = 175mm

Overall depth D=200mm Ast=
$(\frac{M}{\sigma st.Q.b})$=806$mm^2$~840$mm^2$
Provide 16mm ϕ@150mmc/c (Ast provided = 1341mm)

Mini steel =0 .12%bd=240mm^2

Provide 10mmϕbars @ 250mm/cc

The reinforcement in the long span divided is designed for moment given by Design

moment = 0.3M_L+0.2M_D=7.32 KN-m

Using 10mmϕ bars

Effective depth=175-8-5=162mm

Ast= (7.32E6/210x0.9x162)=253mm^2 ~240mm^2

Maximum shear in the slab occurs when the Load position is as shown For

this position of load,

X=0.505m. B=27m L=1.8m B/L=15 $B_{w=}$3.76m k=2.60.

Average intensity of Load=(350x1.25)/(1.01x4.7) = 92.16KN-m^2

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Maximum live load shear=92.16x1.01x (1.295/1.8)=6.7KN

Dead Load shear= (6256x1.8)/2=5.9KN

Total Design Shear =67+5.9=72.9KN

τ_v = V/bd=0.41N/mm^2

100Ast/bd=0.76

For M25 grade concrete & reinforcement ratio 0.76

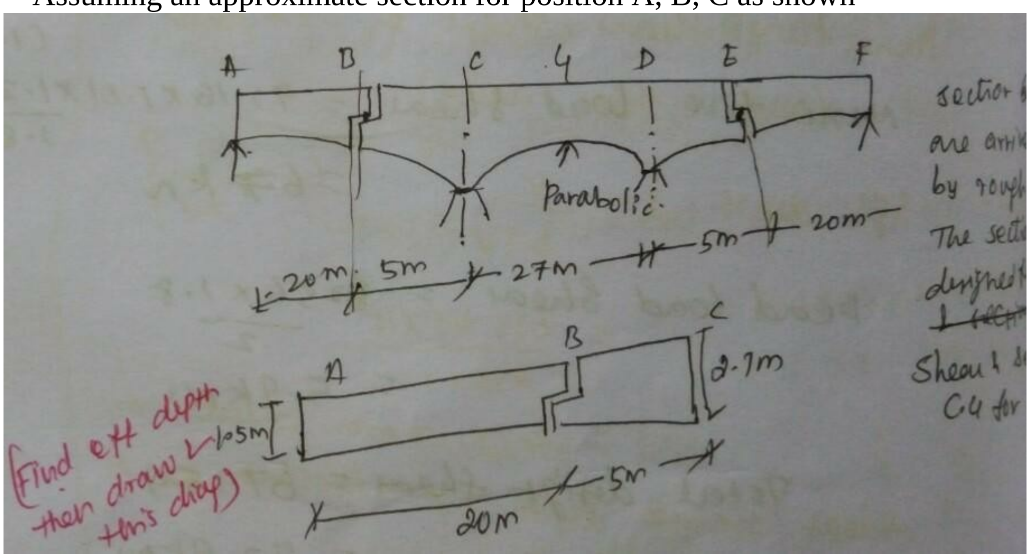
(Table 12B of Irc21-2000) &12c
K=1.20

Permissible shear stress is calculated in concrete slab= k τ_c =1.20x0.35=0.43N/mm^2 $\tau_v < \tau_c$. Hence the shear stresses are within safe permissible limits.

step3: Design of Longitudinal Girders

a) Preliminary Design

Assuming an approximate section for position A, B, C as shown



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1) Loads on AB

Weight of slab= 0.2x1.8x2.4 = 8.6KN/m

Weight of w.c =0.08x 1.8x22 = 3.168 KN/m

Self weight of Girder = 1.5x 0.45x24 = 16.20KN/m

Weight of fillet (150x150)

2x0.15x0.15x24x1/2=0.54KN/m

Dead Load Shear @ A (approximate value) = 350kN

Total Shear = 635.48KN

Permitting Shear stresses τ_v =1N/mm^2

(With shear reinforcement)

Effective depth d= V/b. τ_v =1412.17mm approx (1500)

Adopt overall depth of 1500mm at A

2) Cantilever BC

Referring the fig (i)

Reaction at B due to dead wt of girder AB =285.48KN.m

BM@c due to self wt of cantilever at c,

= {(1.5x0.45x5x5/2) + (1/2x1.2x5x0.45x5/2)=256.5KN.m

BM due to wt of slab, n.c and fillet over cantilever bc

= ½(8.64+3.168+0.54)5^2=154.35KN.m

Total dead load B.M @c

= (285.48+1427.4+256.5+154.35)

=2182.73KN.m

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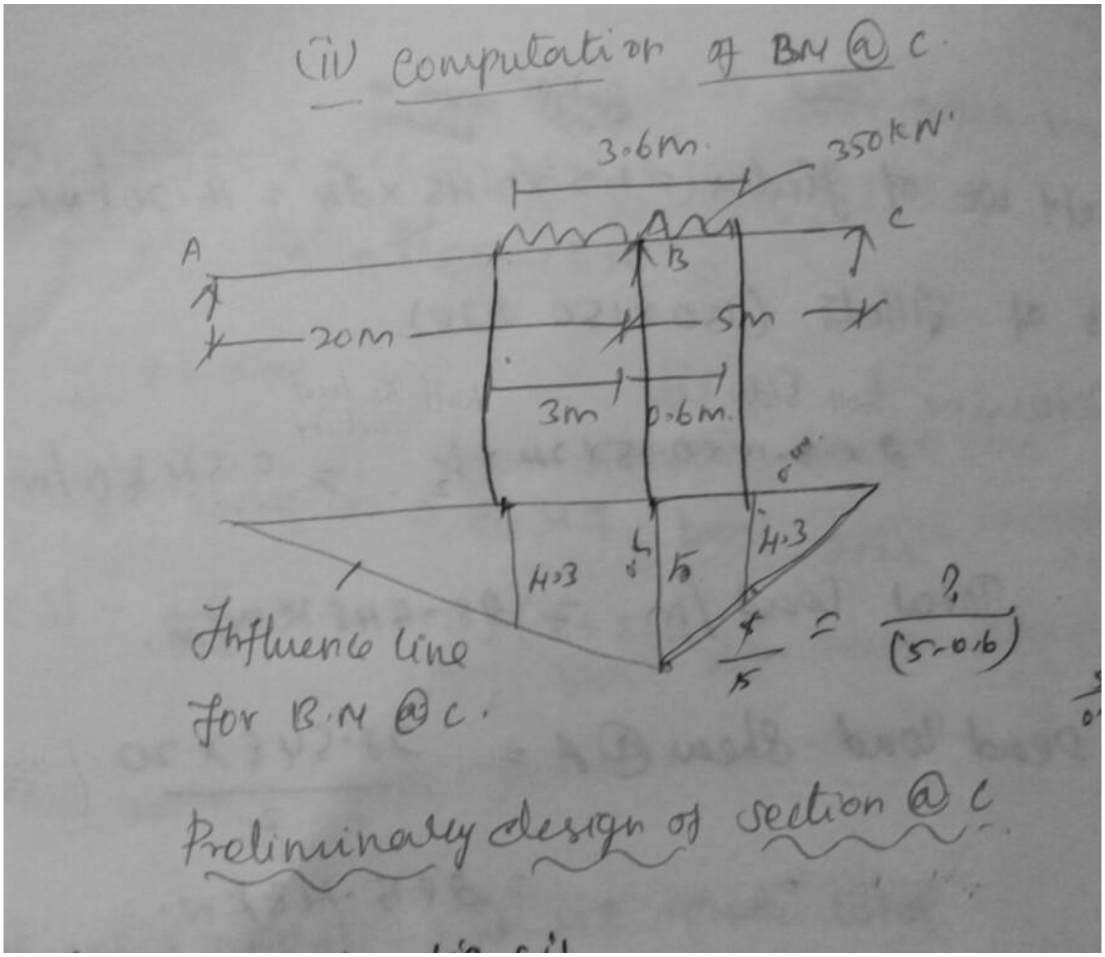

Referring to fig (2)

Live load BM @ c is given by ½(4.3+5)(3+0.6)x(350/3.6)=1627.5KN.m Total

design BM @ C = 1627.5+2125.73=3751.23kN.m

Using M25 grade & Fe415,

$M=Qbd^2$

By solving d = 2752.86mm=2700mm

Since the section c will be designed as a doubly reinforced section an overall depth of 2.7m can be adopted.

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3) Section for mid span at 4

The preliminary section assumed at center of span is shown

Load on dead load due to deck, w.c and fillets= (8.64+3.168+0.54)=12.34KN/m

Self wt of girder=0.5(2.7+2.1) (0.45×24) =25.92 KN/m

Total load= (12.32+25.92) =38.26 KN/m

$(+)_{ve}$ moment due to D.L =1/2×27×6.75×38.24=3486 KN/m

$-_{ve}$ moment due to D.L =1/2×5×2.5×2×38.26

= -478.25 KN-m

$-_{ve}$ moment due to cantilever loads,

285.5×2.5×2= -1427.5 KN/m

Net $+_{ive}$ BM due to dead load at 4,

(3486.4 – 478:25 – 1427.5) = 1580.4 KN/m

Maxi $+_{ive}$ BM due to line load at 4

= 2×1/2(5.8+6.75)1.8(350/3.6)

= 2196.25 KN/m

Total design BM at 4= (1580.4+2196.25)

= 3776.65 KN/m

For lar beam action at centre

Approximate depth d = $\sqrt{3776.65 \times 10^{6} / 1.1 \times 450}$

= 2762.17 mm

Since Tee beam action will be prevalent an overall depth of 2100 mm will be sufficient.

(iv)The section chosen at different parts such as cantilevers, mid span and supports is detailed below:

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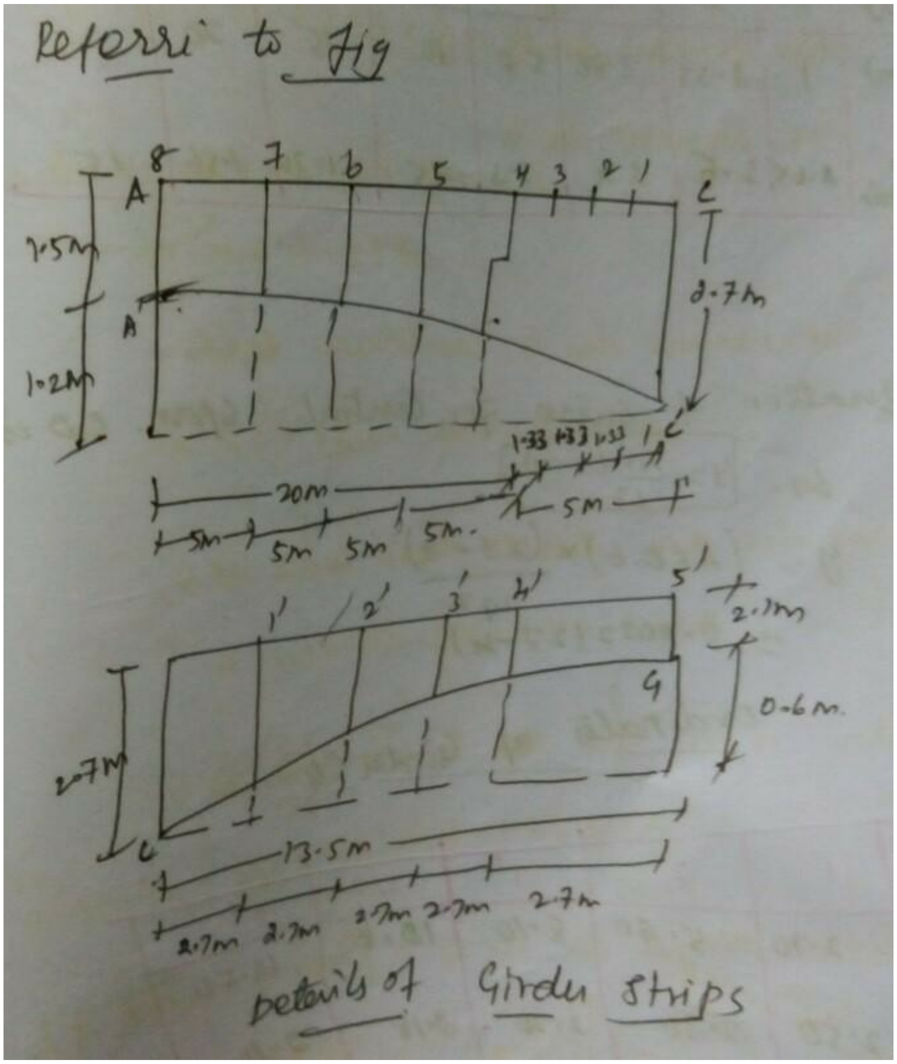

Equation of curve AC´(origin C´)is

$Y = (48/22)(L - X)$

$\underline{\hspace{2cm}} = (4)(1.2)\times(50 - x)/ 50^2$

$= 0.00192\times(50 - x)$

The ordinates of the girder at section 1,2,3,4,5,6,7,8, are tabulated in table below:

point	1	2	3	4	5	6	7	8
x(m)	1	2.33	3.66	5.0	10	15	20	25
depth of girder	2.65	2.5	2.4	2.3	1.75	1.70	1.56	15

Equation of curve for central span CD is given by,

$Y=4r/L^2 (L - X)$

$\underline{\hspace{2cm}} Y= (4\times0.6) \times (27 - x)/ 27^2$

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$= 0.0032(27 - x)$

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Ordinates of girder

Point	1´	2´	3´	4´	5´
X(m)	2.70	5.40	8.10	10.8	13.50
depth of girder(m)	2.50	2.30	2.20	2.18	2.10

Analysis of continuous bridges

The bending moments and shear forces at various sections of a continuous girder can be evaluated by using influence lines. Influence lines are constructed for salient sections at regular intervals of span for bending moments and shear forces. The IRC live loads are suitably positioned on the influence lines so that maximum bending moment and shear force are developed at any given section. The dead load and live load moments are combined to yield the design moments. The reinforcement in the section is designed to resist the maximum moments and shear forces developed in the section.

Design Example

A three span continuous Reinforced Concrete Bridge with the girder of variable cross section is required for the cross section is required for the crossing of a national highway Design the deck slab and main girders to suit the following data

Total length of bridge = 70m

Span length, end spans = 20 m

Central span = 30m

Width of carriage way = 7.5 m

Kerbs = 600mm on either side

Spacing's of main girders = 2.9m

Spacing's of cross girders = 4.0m

Concrete: M -20 grade, Steel: Fe-415 tor steel

Loading: I.R.C.class AA tracked vehicle

Design the bridge deck and draw typical sections showing reinforcement details in the deck slab and girders. The design should conform to the provisions of I.R.C code 21

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1. Selection of Dimensions Of Girders[6]

The longitudinal elevation showing the main girders of the three span AB, BC and CD is shown in fig..Assuming a parabolic profile for girders, depth of girder at A=1/20 span

h= (1/20×20) = 1m

Depth of girder at B = (h+2h) = 3h= 3m

r_B = 2

The cross section of the deck showing the dimensions of the various structural components is shown in fig:

Thickness of deck slab = 250mm

Width of girders = 500mm Spacing's

of girders = 2.9 m Thickness of

wearing coat = 80mm

Kerbs 600mm wide by 300 mm depth are provided on either side

Cross girders are provided at 4m intervals in end spans and 6 m intervals in central span

Width of cross girder = 300 m

2. Design of Deck Slab

(a) *Bending Moments*

Dead weight of slab = (1×1×0.25×24) = 6

Dead weight of W.C = (0.08×22) = 1.76

Total dead load = 7.76 kN/m^2

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

u = (0.85+2×0.08) = 1.01 m

v = (3.60+2×0.08) = 3.76 m

(U/B) = (1.01/2.7) = 0.348

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$(V/L) = (3.76/4.0) = 0.94$

$K = (B/L) = (2.9/4.0) = 0.725$

Referring to Pigeaud's curves(Refer fig:) m_1=0.09

$m_{2=}$0.035

$M_{B\ =}W\ (m_1+0.15\ m_{2)}$

=350(0.09+0.15×0.035)

=33.32 kN-m

Design B.M including impact and continuity factor is given by M (Short Span)

= (1.25×0.8×33.32) = 33.32 kN-m

Similarity (Long Span) =350(0.035+0.15×0.09) = 16.975 kN-m

(b) Dead Load Bending Moments

Dead load=7.76 kN/m^2

Total load one panel = (40×2.9×7.76) =90 KN

(U/B)= (v/L) = 1 as panel is loaded with uniformly distributed load

K= (B/L) = (2.9/4.0)=0.725,(UK)=1.379

From Pigeaud's curves (Refer fig:)

$m_{1=}$0.048 and m_2=0.025

M_B = 90(0.048+0.15×0.025) = 4.65kN-m

Taking continuity into effect,

M_B= (0.8×4.65)=3.726 kN-m

M_L = 90(0.025+0.15×0.048) = 2.88kN-m

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Taking continuity into account

M_L = (0.8×2.88) = 2.304 kN-m

(c) Design Moments

Total M_B = (33.32+3.726) =37.046 KN=m

$M_{L\,=\,(16.975+2.304)\,=}$ 19.279 kN-m

Adopted overall depth =250 mm

Effective depth =230 mm

A_{st} =(37.046×10^6)/(200×0.9×230) = 895 mm^2(Short Span)

Use 12mm ǿ at 100 mm centers (A_{st} = 1131 mm^2) Effective

depth for long span using 10 mm diameter bars

= (230-6-5) = 219 mm

$A_{st\,=}$ (19.279×10^6)/(200×0.9×219) = 489 mm^2

(Long Span)

Use 10 mm ǿ at 150 mm centers (A_{st} =524 mm^2)

The reinforcement in deck slab is shown in fig:

3. Stiffness and Distribution Factors

Referring to fig:the value of r_B and r_C =2.0,r_A = r_D =0.For these values from fig: The

stiffness coefficients are obtained as,

$K_{BA=}$ 13.5 k_{BC} =22.8

From fig: the carry over factors are, C_{AB}=

-1.01, C_{BC} = -0.78, C_{CD} = -0.40

C_{BA} = -0.40, C_{CB} = -0.78, C_{DC} = -1.01

The end A is simply supported .Hence the stiffness factors K_{BA} is modified by using the equation

Modified value of K'_{BA}= (1-$C_{BA}C_{AB}$) $KBA_{=8.046}$

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The distribution factors are collected using the following data

D_{BA}= (K/∑K) =0.3468=D_{cd}

D_{BC}= (1-0.3468) =0.6535=D_{CB}

4. Moments at supports B and C

The final moments at support B and C in terms of fixed end moments are calculated as follows:

(a) Load in Span AB

M_B= [(1-D_{BA}-U)/(1-U)]M_1

Where U =$C_{BC}.C_{CB}.D_{BC}.D_{CB}$

= (-0.78)(-0.78)(0.6532)(0.6532)

= 0.2595

M_B = [(1-0.3468-0.2595)/ (1-0.2595)] M1

= 0.5316 M_1

M_C = (V/1-U) M1

V =$C_{BC}.D_{BC}.D_{CD}$

= (-0.78×0.6532×0.3468) = -0.1766

M_C = [(-0.1766)/ (1-0.2595)] M=-0.2384M_1

(b) Load in span BC

M_B = ($D_{BA}.M_{BC}$.-W.M_{CB})/(1-U)

W = $C_{CB}.D_{CB}.D_{BA}$

= (-0.78×0.6532×0.3468) = -0.1766

M_B = (0.3468M_{BC}+0.1766M_{CB})/ (1-0.2595)

= (0.4683M_{BC}+0.2384M_{CB})

M_C = ($D_{CD}.M_{CB}$-V.M_{BC})/(1-U)

= (0.3468M_{CB}+0.1766M_{BC})/ (1-0.2595) =(0.4683M_{CB}+0.2384M_{BC})

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(C)Load in span CD

M_B = [(W/1-U)] M3= [(-0.1766)/(1-0.2595)]M_3

=-0.2384M_3

M_C = [(1-D_{CD}-U)/(1-U)]M_3

= [(1-0.3468-0.2595)/ (1-0.2595)] M3

=0.5316M_3

Also $M_1=M_{BA}-C_{AB}.M_{AB}$

$M_3=M_{CD}-C_{DC}.M_{DC}$

5 Influence Line Coefficients for Moments at support B is calculated for incremental position of load in spans, AB, BC an CD respectively as complied

In this table the coefficients are in terms of the length “L”. The span lengths of the design problem L1=20m, L2=30m, L3=20m. The coefficients for M are multiplied by the respective span length depending upon the position of the loads on span and influence line coefficients are derived.

The influence line ordinates for bending moment at support B is derived by multiplying the respective lengths of the spans L1,L2 and L3 depending upon the load position from 0.1 to 3.0. The influence line ordinates are complied in table 14.2 and the influence line plotted on the span is shown in fig

6. Influence Line ordinates at various Sections

Similarly the influence line ordinates are derived for bending moments at section 0.2L, 0.4L, 0.5L, 0.6L, 0.8L, 1.2L, 1.4L and 1.5L and they are complied in table. The influence line ordinates for these various sections are shown

Similarly influence line ordinates for shear force at support section A and B complied The

influence for shear at support are shown in fig

7. Dead Load Bending Moments

(a) Self-weight of Deck slab, Wearing Coat and Kerbs

Total dead of deck of deck slab and wearing coat= 7.76KN/m^2

Load due to kerb RC post etc (lump sum) = 0.24

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Total load = 8kN/m^2

Load transmitted to girder at 0.1 L section are as follows In

span AB= (8x2.9x2) =47KN

In span BC= (8x2.9x3)=70KN

Load transmitted at A= (8x2.9x1)=23.5KN

Load transmitted at B= (8x2.9x2.5)=58.0kN

B) Self weight of main girders

The self-weight of girders acting at various sections from 0.1L to 1.5L is complied in table. The main girders are varying depth and of constant width of 500mm. the depth of main girder

Self weight of cross section Girder

Cross girder have the same depth as that of main girders and they are spaced at 4m intervals in end span and 6m interval in central span. Width of cross girder =300mm. The self weight cross section girder acting at various sections.

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UNIT -IV
PRESTRESSED CONCRETE BRIDGES

PRESTRESSED CONCRETE

Using this high tensile strength steel alloys producing permanent pre compression in area subjected to tensioning.

PRETENSIONING

Placing of concrete around reinforcement tendons that have been stressed through the desire degree (steel tension before the concrete is placed).

POSTENSIONING

Reinforcing tendons are strengthen by jacks while keeping them inverted in voids left pre-hand during curing of the concrete. This space are then pumped full grout to bond steel tightly to the concrete (steel tensioned as concrete harden).

ECCENTRICITY PRESTRESSING

Desirable in prestressed not productive at the end of the span. Draped or harped profile temporality held in place before the concrete hardened bonding not all stands or wires are active at the end of the span. For post tensioning eccentricity install ducks in desire profile.

TYPES OF LOSSES IN PRESTRESSED

Pretensioning

- Elastic deformation of concrete
- Relaxation of stress
- Shrinkage of concrete
- Creep of

concrete Post

tensioning

- No losses due to elastic deformation if all the wires are simultaneously tensioned

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- Relaxation of steel
- Shrinkage of concrete
- Creep of concrete

COMPARISION OF PRETENSIONED AND POSTTENSIONED CONCRETE

PRETENSIONED CONCRETE	POSTENSIONED CONCRETE
• Steel is tensioned prior to that of concrete. It is released once the concrete is placed and harden the stresses are transfer along the wires by means of bond.	• Concrete is placed first then the wires are tensioned and anchored at the ends.
• Suitable for short span and precast product like sleepers and electric poles.	• Suitable for long span brides.
• In prestressing the cables are basically straight and horizontal placing them in inclined or curved position is difficult.	• The wires can be kept eccentrically in parabolic profile.
• A postensioned cables can be align in any manner to suit the bending moment diagram due to the external load system. Therefore, it is more economical particularly long span bridges.	• Structural advantages are more. When compared to the post tension.

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• Losses are more.	• Losses are less when compared to the prestressed concrete.

DESIGN OF POST TENSIONED PRESTRESSED SLAB BRIDGE DECK OR DECK SLAB

Design a post tensioned prestressed slab bridge deck for a national highway to suitable the following data

- Clear span 10 meters
- Width of bearing 400mm on either side
- Thickness of wearing coat 80 mm
- Live load IRC tracked AA vehicle
- Type of structure I type
- M40 and 7mm high tensile wire with an ultimate strength of 1500N/mm^2
- The cables consist of 12 wires and anchorage of 150mm diameter for sublimentary reinforcement FE415 HYSD bars
- Compression strength at transfer $f_{ci} = 35\ N/mm^2$
- Loss

ratio=0.8 Solution

Step1: Given data

- Clear span = 10m
- Width of bearing = 400mm
- Clear width of road ways = 7.5m
- Footpath (on either side) = 1m

- Thickness of wearing coat = 80mm
- Live load

  IRC class AA tracked vehicle
- Type of structure
- Materials

f_{ck} = 40N/mm^2

Step2: Permissible stresses

Permissible compressive stress in concrete at transfer and working load refer IRC18

f_{ct}<0.5F $_{ci}$

f $_{ct}$<0.5X35

f $_{ct}$

<17.5N/mm^2 f

$_{ct}$ ≥20N/mm^2

Permissible compressive stress in concrete under service load, f

$_{cw}$ =0.33f $_{ck}$

=0.33X40

=13.2N/mm^2

f $_{tt}$=0

Allowable tensile stress in concrete under the service load f $_{tw}$ =0

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Step3: Depth of the slab

Assuming, the thickness of the slab at 50mm per meter of span for highway bridge deck.

Overall thickness of slab = 10X50mm

= 500mm

Thickness of bearing = 400mm

Effective span of bearing = clear span + effective depth

= 10 + 0.5

= 10.5m (or)

Effective span of bearing = clear span + width of bearing

= 10 + 0.4

=10.4m

(Take the value whichever is lesser) Therefore effective span =10.4m

Step4: Dead load bending moment and shear force

Bending moment = $WL^2/8$

Shear force = WL/2

Dead load

(a) Slab = b x d x unit weight of concrete

= 1 x 0.5 Xx25

=12.50N/mm^2

(b) Wearing coat =0.08 X 22

=1.76N/mm^2

Total dead load

load =14.26N/mm^2

B.M or M_g =192.7kN-m

S.F or V_g =74.152kN

Step5: live load bending moment

Impact factors = 100 + 10/100

= 1.1

Effective length of road= 3.6 + 2(0.5 + 0.08)

= 4.76m

$a = L_{eff}/2$

k = 9.5/10.4

= 0.9134

b_w =0.85 + (0.08)

=1.01

Effective width b_{eff} $=\alpha \times a \times (1-a/L_{eff}) + b_w$

=7.17 m

Width of dispersion =8.67m

Impact load =700 x 1.1

=770 kN

Average intensity of load =770/4.67x8.45

$=18.65 kN/m^2$

Maximum BM due to live load =177.98KN-m

Step6: Shear force

Shear force due to class AA cracked vehicle for maximum shear at the support section the IRC class AA tracked vehicle is given by,

L/b =9.5/10.4 =

0.9134 α =2.37

b_{eff} =2.37X2.38(1-2.38/9.5) + 1.01

b_{eff} =7.35m

Width of the dispersion for two tracks,

=7.345m

Average intensity of load =770/4.76x7.35

= 22kN/m^2

Total shear force, V_A = dead load shear force + live load shear force

= 72.67 + 80.761

= 153.15kN

STEP-7: CHECK FOR MINI SECTION MODULUS:

Dead load moment Mg = 189.28 KN-m Live load moment Mq = 186.86 KN-m

Consider 1m width, over all thickness = 500mm Section modulus z =41.47x10^6 cu.mm

The permissible stress in concrete at transfer (Fct) is obtained from IRC 18-2000. Fct = 17.5 N/sq.mm Ftw = 0 n=0.8

Fcw = 13.2 n/sq.mm Ftt = 0 Fbr=

(n Fct-Ftw)=(17.5x0.8 – 0) = 14 N/sq.mm

The mini section modulus is given by

$$Z = \frac{Mq + (1-n)Mg}{Fbr}$$

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(186.86+(1-0.8)x 189.28)x10^6)14

41.67x10^6 ≥ 16.05x10^6

cu.mm Z(min) 16.05x10^6 < 41.67x10^6

cu.mm

The section provided is sufficient to resist the service load .

STEP-8: MINI PRESTRESSING FORCE:

$$P = \frac{A(F(inf).Zb + F(sup).\ Zt)}{Zb+Zt}$$

Where A = 500x100 = 5x10^5

sq.mm F(inf)

$$= \frac{F(tw)}{n} + \frac{Mq+Mg}{nZb}$$

= 11.28 N/sq.mm

F(sup) = F(tt) - Mg Zt

= - 4.54 N/sq.mm

$$P = \frac{5x10^5 \text{ x } 41.67 \text{ x } 10^6(11.28-4.54)}{(41.67+41.67) \text{ x } 10^6}$$

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P=1685 KN

Using cables containing 12 wires of 7mm dia stressed to 1200N/Sq.mm

Force in each cable = (12 x 3.14 x 7x7 x 1200)4

= 554.17 KN

Spacing of cables = (554.17 x 1000)1685

= 330 mm c/c

STEP – 9 : ECCENTRICITY OF CABLES:

The eccentricity of cables at centre of span is obtained from the relation

$$e = \frac{Zt\,Zb\,(Finf - Fsup)\,A}{(Fsub\,Zt + Finf\,Zb)}$$

= 195mm

The cables are arranged in a parabolic profile with a maximum eccentricity of 195mm at center of span , reducing to zero eccentricity at supports.

STEP -10: CHECK FOR STRESS AT SERVICE LOADS:

P = 1685 KN

e = 195 mm

A = 5x10^5 sq.mm

Zf = Zb = 41.67x 106

cu.mm Mg = 189.28 KN-m

Mq = 186.86 KN-m

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$P = (1685 \times 10^3) 5 \times 10^5$
$= 3.37$ N/ sq.mm

$Pe/z = (1685 \times 10^3 \times 195) / 41.67 \times 10^6$
$= 7.88$ N/sq.mm

$Mg/z = (189.28 \times 10^6) / 41.67 \times 10^6$
$= 4.54$ N/sq.mm

$Mq/z = (186.86 \times 10^6) / 41.67 \times 10^6$
$= 4.48$ N/sq.mm

Stress at transfer

At top of slab = 3.37-7.88+4.54 = 0.03 N/sq.mm

At bottom of slab = 3.37+7.88-4.54 =

6.71N/sq.mm STRESS AT WORKING LOAD

At top of slab = 0.8(3.37-7.88)+4.54+4.48 = 5.492N/sq.mm
At bottom of slab = 0.8(3.37+7.88)-4.54-4.48 =
-0.02N/sq.mm

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STEP -11 CHECK FOR ULTIMATE STRENGTH

Considering 1m width of slab = b = 1000mm

$$Ap = \frac{12x38.5x1000}{328}$$

=1408sq.mm

(a) FAILURE BY YIELDING OF STEEL

Mu = 0.9 d Ap Fp

d = the depth of slab from main comp edge to CG of steel tendons

= 445mm

Fp = 1500 Mpa

Mu = 0.9 x 445 x 1408 x 1500

= 845.85 KN-m

(b) FAILURE BY CRUSHING OF CONCRETE

Mu = 0.176 b(d^2) fck

= 0.176 x 1000 x 445 x 445 x 40

= 1394.09 KN-

m Mu = 845.85

KN-m

According to IRC 18-2000

Required ultimate moment = 1.5Mg + 2.5 Mq

= 751.09 KN-m

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Hence the ultimate moment capacity of the section (Mu= 845.85 KN-m) is greater than the required ultimate moment (751.09KN-m)

STEP – 12 : CHECK FOR ULTIMATE SHEAR STRENGTH:

Ultimate shear force Vu = 1.5Vg + 2.5Vq

= 1.5x72.8 + 2.5x80.75

= 311.075 KN

Accordingly to IRC 18-2000 the ultimate shear resistance of support section uncracked by flexure is given by

$Vco = 0.67\ b\ h\ (\ (Ft)^{2}\ +\ 0.8\ F_{cp}\ Ft\ )$

^0.5 b = 1000mm, h = 500mm

$Ft = 0.24\ (Fck)^{0.5}$

$= 0.24x\ (40)^{0.5}$

= 1.51 N/sq.mm

F_{cp} = comp prestress at centroidal axis

$= \frac{0.8 \times 1685 \times 10^3}{500 \times 1000}$

= 2.69 N/sq.mm

Eccentricity of cables at centre of span e = 195mm The cables are concentric at support section

$Vco = 0.67\ b\ h\ (\ (Ft)^{2} +\ 0.87\ \ F_{cp}\ \ Ft\ )^{0.5}$

$= 0.67 \times 1000 \times 500x\ ((1.51)^{2} + 0.87\ x2.69\ x1.51)$^0.5

= 807.75 KN

Since the ultimate shear force Vu is < than 50% the ultimate shear resistance Vco, no shear reinforcement is required

STEP -13 SUPPLEMENTARY_REINFORCEMENT:

A_{st} = 0.15% Gross C.S.A

= (0.15/100) x 1000x500

= 750 Sq.mm

Provide 10 mm dia bars Fe415 steel bars @ 200 mm c/c , top bottom faces in the longitudinal and transverse direction

STEP-14 DESIGN OF END BLOCK RENIFORCEMENT

At support section cables carrying a force of 554.17KN are spaced at intervals of 330mm c/c

The bursting tension is computed using table no 10.2 (IRC 18-2000) F bst = bursting tension force , P k = tendon force

Ypo = side of loaded area , 2Yo = side of end block

P_k = 554.17 KN

2Ypo = 150mm 2Yo = 330mm

Ypo/Yo = 150/330 = 0.454

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Interpolating the value of Fbst/Pk for

Ypo/Yo Fbst/Pk = 0.185

Fbst = 0.185x554.17

= 103 KN

Using 10mm dia, Fe 250 grade mild steel bars as end block

reinforcement Area of steel = $103 \times 10^3 / 0.87 F_y$ = 474sq.mm

Provide 10mm dia bars at 100mm c/c in vertical and horizontal direction in front of anchorage at 100&200mm respectively

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UNIT – V
Design of plate girder bridges

General features

Plate girder bridges are the most common type of steel bridges generally used for railway crossing of streams and rivers, The earliest forms of steel bridges constructed happen to be the plate girder bridges due to the simplify of the structural form and their elegant aesthetics. Plate girder bridges are adopted for simply supported span in the range of 20m to 50m and for continuous spans up to 250m.

In the case of railway bridges, the plate girders support the sleeper over which the steel rails are fastened. Each rail is supported on a plate girder. So that the wheel loads are transmitted directly to the plate girder without effect of torsion. The twin plate girders are braced laterally stability. Cross bracing consisting of angles are provided at the ends and intervals of 4m to 5m. The lateral bracings and the end cross frames resist the lateral loads on the plate girder.

Element of a plate girder and their design

As plate girder are preferred for construction of railway bridges.

The load recommended by Indian railway standards for each kind of track are called Equivalent uniformly distributed loads [EUDL]

EUDL for broad gauge railway tracks.

(1) Web

The web of a girder can be constant height or varying height. The girder with varying depth of web is called haunched girder. The girder with constant depth of web. Its depth is dependent on maximum bending moment. The depth of the web can be decided based on 'Economical depth' of the plate girder.

It is given by

$$D=5x\sqrt[3]{\left(\frac{m}{fb}\right)}$$

Where,

M – Design BM after incorporating impact effect.

f_b- permissible bending stress in steel which is taken as 0.66fy (fy-yield stress of steel)

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The thickness of the web can be calculated based on the shear stress. The thickness should also provide the necessary bearing area.

A mini thickness of 8mm (IRC24) is adapted to provide for wear caused by corrosion. Inadequate dimensioning of a web may lead to buckling. The minis recommend thickness of web plate for different values of yield stress is given in IS800-1984

(2) Flanges

A flange should be preferably being a single plate unless a plate of suitable thickness is not available. The width of plate depends on the span to width ratio which ranges from 40 to 45. The flanges should be connected to the web by welds to transmit the horizontal shear force combined with any vertical loads which are directly applied. The thickness of flange plate may be calculated base on the approximate requirements of the flange area. The area of flange is given by,

$A_f = (m/f_{bd}) - (A_w/6)$

Where

d- Depth of web,

a_w- area of web.

However, the outstand of the flange should not be greater than 20 times of thickness of plate.

(3) Intermediate stiffeners:

In order to avoid web failures(diagonal buckling) as well as to comply with lower web thickness, the web must be adequately supported laterally by stiffness.

Two types of stiffeners

1. Vertical stiffener located over the length of the span

2. Bearing stiffeners located at the supports of the span (4).

Vertical stiffeners

The vertical stiffeners are provided at spacing not greater than 1.5d and not less than 0.33d, d-depth of the web.

The web panel dimensions between two stiffeners should not be greater than 270 times the thickness of the web. The length of outstanding leg of the vertical stiffeners may be

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taken as 12 times the tk of web. The vertical stiffeners should provide moment of inertia, which should not be less than

$$I=1.5d^3t^3/c^2$$

Where

I - M.O.I of the pair of stiffeners about the centre of the web (or) single stiff about the face of the web,

t- Thickness of web, d- Depth

of web,

c- Clear distance between vertical stiffeners,

These stiffeners and connected to the web plate. So as to withstand the S.F at the interface between the web and the stiffener which is given by,

$F=125t^2/h$

F-SF in kN/m

t- Thickness of web in mm

utstand of stiffeners in mm

(5). End bearing stiffeners

EBS are provided at the pts of supports, the end bearing stif, strengthen the web and transmit heavy reactive forces to the flanges as columns. The sectional area of an end.b.st consist of the stiff together with some length of web (20tk of web) on either side of the determine the radius of gyration to check capacity of the stiff as a column should be greater than the applied load or reaction

(6). Lateral bracing for plate girder

LB is a system of cross framers located in the horizontal plane and installed for connecting flange in order to resist lateral deformation. Lateral deformation is induced by wind loads, which act normal to centre line of the web. Lateral bracing is required if the span exceeds 20m. Since the plate girders of railway bridges are considerably deep, the need for lateral bracing is justified . The increase depth creates a large surface area of the web over which the wind forces can act.

Design principles

The design of a plate girder involves the section of the cross section and design of connection between flanges and web, together with the design of intermediate and bearing stiffness and their connections to the web of the plate girder

Various steps involved are as follows

1. Compute the live load and dead load moments and shear forces. The self wt of girder may be assumed as (0.22+1) kN/m.
   where L- span of the girder.
2. The design moments and shear forces are computed by applying impact factors to the live load moments and shears. (The impact factor for steel bridges. Prescribed in IRC codes and IRC bridge rule are outlined in section 1.2 and 1.4 respectively).
   I.F= 0.15+ (8+ (6+2)

3. Approximate depth of girder = 1/8to 1/10 span

   Economical depth = $D=5x \sqrt[3]{\left(\frac{m}{fb}\right)}$

   Where m- design BM

   f_b- permissible bending stress

   For plate girder, the permissible stress is 141 N/mm^2 and 150N/mm^2 from clear depth to tk ratio of web is greater or less than 30 respectively Assume thickness of web as t (not less than 8mm)

   τThe depth of web is obtained as

   $D=V/ \ _V \ t$

   V-Shear force

   t- Thickness of web

   τ_V - The average shear stress specified as 85N/mm2 for mild steel with an yield stress of 236n/mm2 as per IRC 24.

   A suitable web depth is proportioned based on flexure and shear computation

4. Approximate flange are required is

   $A_f= (m/f_{bd})-(Aw/6)$

   A_w area of web

Flange width = L /40 to L/60, outstand of flange beyond the flange angle should not exceed 16t for mild steel and 14t for high tensile steel. Where t-thickness of the thinnest flange plate in the case of riveted connections. For welded connections the flange plate should not project beyond the line of connections to the web by more than 12t

5. The proportioned section is checked for permissible stresses as per the specification of IRC:24

6. The connection between flange and web is designed to resist a maximum horizontal shear force given by,

$$T=Vay/I$$

Where,

V-SF at a section A

- Area of flange

y- Distance of centroid of area v from neutral axis I -

second moment of area

The size of weld is designed to resist this horizontal

7. Spacing of intermediate stiffeners is given by ‘c’ computed as not greater than 1.5d and not less than 0.4d.

Where d is the unsupported depth of the web

The intermediate stiffness are designed. To have a minimum moment of inertial given by

$$I= (1.5d^3t^3)/c^3$$

d- Depth of the web t

- Thickness of web

a - Spacing of stiffness

The outstand of stiffness shall be not more than 16t for rolled section or flat.

The connection of intermediate stiffness to web is designed to resist a horizontal shear force of not less than ($125t^{2/}h$) KN/m

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h- Outstand of stiffener (mm)

8. The end load bearing stiffener is designed as a column assuming the section to consist of the pair of stiffeners together with a length of web on

each side of the centre line of the stiffness and equal to 20tw. The permissible stresses are checked as that for comp member assuming an effective length equal to 0.7 times the length of the stiffener

9. Lateral bracing consisting of angle section are designed to resist the horizontal wind load and racking forces.

Design problems

Design a deck type welded plate Girder Bridge to suit the following data: EFF

span of the girder =30m

Dead load (open floor) =7.5 KN fm

Equivalent total live load for BM calculations/track

=2729kN

Equivalent total live load for shear calculations/track

=2927KN

Top of rail level =105.00

Foundation level =110.50

Width of abutment =4m

Design the main plats girder with intermediate and bearing stiffeners and lateral bracings

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Draw the following views:

1. Half longitudinal elevation and half longitudinal section.
2. Half plan @ top and half plan @foundation.
3. Half cross section centre of span and half through abutment.
4. Longitudinal elevation at main plate girder.
5. Cross section of main plate girder.
6. Plan of girder with lateral bracing

7. Intermediate and bearing stiffeners .Adopt suitable scale for the drawings. Adopt rolled steel section and steel plates with a field stress of 236N/mm^2.

SOLUTION:

tep (1) given data Effective

span =30m.

Average shear stress τ_v=85N/mm$^{2.}$

Broad gauge (1676mm) main line single track.

ETDL for BM per track=2727KN.

ETLL for SF per track =2927KN.

σ_b =141N/mm^2.

Step (2) Dead loads

Dead load of track (open floor) = 7.5KN/ m

Self wt of girder = (0.22+1)

= (0.2×30+1)=7 KN/m.

Total load =14.5KN/m

Step (3) Live loads

ETLL for BM per track= 2727 m

Total the load per girder = 2727 /2 =1463.5kN.

Step (4) Impact factor

The I.F for steel railway bridge is given by co-efficient of dynamic argument (CDA)=I=0.372

(CDA) =I =0.15+8/ (6+L)

Step (5) bending moments

BM due to dead load $=145\times30^2/8$

=1631.25 KN-m

BM due to live load = 1367.5×30 /8

=5113.125 kN-m

BM due to impact on live load

$=0.372\times5113.125$

=190.08 kN-m

Total design BM= M=8646.45kN-m

Step (6) Shear forces

SF due to dead load=$14.5\times30/2$ SF

due to live load =1463.5/2

=731.75 KN

SF due to impact on L.L $=0.372\times731.75$

$=272.21$ KN

Total design SF= V= 1221.46 KN

Step (7) Proportioning of trial section of web plate

Approximate depth of girder =1/8 to 1/10 span

$=30\times10^3/10=300$mm

Economical depth $= 5\sqrt[3]{M/\sigma b}$

$=1971.71$mm

Web depth based on shear consideration assuming 12mm thickness plate (t is not less than 8 mm as per code)

$\tau_v=v/D\times t$

$\tau D=v/ \times t =1221.46\times10^3/85\times12$

$=1197.5$mm

Try a web plate 1600 ×12 mm

Step (8) Flange plates

Approximate flange are required

$A_f = \mu/\sigma_b.d - A_k/6$

$=8646.45\times10^6)(141X1600) - (1600\times12)/6$

$=35126.46\ mm^2$

Flange width L/40 to L/45 (or)L/60

=3000/40 to 3000/45(or)L/60

=750 to 666 (or)500

Adopt B=710 mm

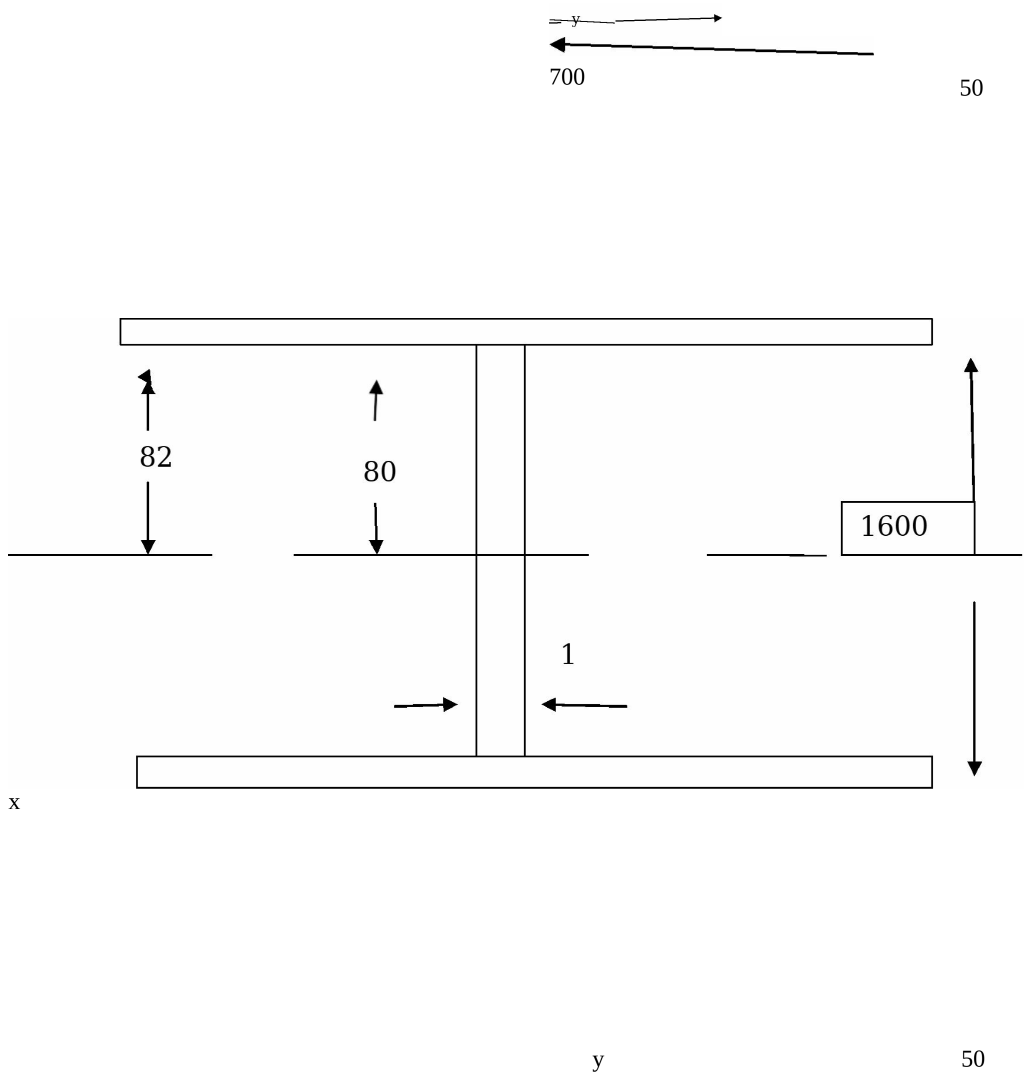
Thickness of plate = 3512.46/710

=50.1mm

Adopt flange plates of size 700× 50mm

Cross section of plate girder:

The section selected as shown



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- Step (9) Check for maximum stresses:

The second moment of area of the section is by I= (

$bd^2/6+Ah^2\times 2)$

$I_{xy}= bd^2+2A_f\, r^2$

$=12\times1600^3/12+2\times700\times50\times825^2$

$=4096\times10^6+4764.3\times10^7$

$=5173.8\times10^7$ mm^4

$I_{yy}=2\times50\times710^3/12 + 1600\times12^3/12$

$= 285.8\times10^7$mm^4 A=

$(2\times500\times700)+(1600\times12)$

$=89200$ mm^2

$r_y=\sqrt{I_{YY}}/A$

$=178.99$ mm

According to IRC-24 the critical compressive stress for I sector having equal moment of inertia about yy axis is given by

$C_S=2677300 \{ 1+0.05(l.tc/ry)\}/ (l/ry)$

'resent example using cross bracings at intervals of 6m where L=6000 mm ,L=eff length of comp flange

D=1700 mm, D= overall depth of girder

ry=178 mm,radius of gyration about the axis of the gross section of the whole glider at the pt of maxi B.M.

$t_c=50$ m

$C_s=2437.42$ N/mm^2

The allowable working for different values of critical stress c_s prescribed in IRC 24.

σPrescribed bending stress corresponding to a value of critical stress c_s=2437 N/mm^2 is obtained as 158 N/mm^2 for $_y$=236N/mm^2.

Actual bending stress

σ_{bc}= σ_{bc}= =M.y/ I_{XX}

=142.04 N/mm^2 <158 N/mm^2

Hence the actual stress are within safe permissible limits.

'ermissible average shear stress depends upon the rate of

d/t=1600/12=133.

Using stiffness spacing 0.9d (or) 0.8d

=1440 =1500

From table(IRC 24) allow average

σShear stress for steel conforming to IS(226)-$_y$ =236 /mm^2 is extrapolated as 87 N/mm^2

τAverage shear stress $_v$ =v/dt

=1221.46× 10^3/1600 ×12

=63.6 N/mm^2 < 87N /mm^2

Hence the average shear stress is within safe permissible limits. Step(10)

Connection between flange and web:

Maxi SF at the junction of the web and flange is given by

τa=v y/I

V=1221.46 kN

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a-area of flange –(fw ×500)=35000mm^2 I

=5173.9×10^{7}mm^2

Y= 825 mm

$$\tau = \frac{1221.46\times10^3\times3500\times825}{5173.9\times10^7}$$

=681.68 N/mm^2

Assuming a continuous fillet weld on either side strength of weld of size ’s’ is

=2 ×0.7 ×S× 102

=142.8S N.mm

142.8S= 681.68

S= 4.77

Use 6mm fillet weld continuous on either side

Step(11) intermediate stiffness

Since d/t =133>85 N/mm^2

Vertical of stiffness as required

Spacing of stiffness = 0.33 d to 1.5 ×16w

=528 to 2400

Adopt 1500 mm spacing

Hence c=1500mm

Greatest unsupported panel dimension of web=1500 mm <270tw

<270 ×12<3240 mm

The intermediate stiffness are designed to have a mini moment of inertia of

$$I=1.5\ d^3f^3/c^2$$

$$=1.5\times1610^3\times12^3/1500^2$$

$$=471\times10^4\ mm^4$$

Using 10 mm thick plate outstand of stiffness should not be greater than 12t=12× 10 =120mm

Adopt a plate 10 mm ×120 mm

$$I=10\times120^2/3$$

$$=576\times10^4 mm^4 > 471\times10^4 mm^4$$

Step (12) connection of vertical stiffness to web

Shear in welds connectivity stiffness to web

$$=125t^2/h(kN/m)$$

T=web tk(mm)

H=outstand of stiffness projector in mm of the stiffness

Shear on welds $=125\times12^2/120$

$$=150$$

kN/m Size on welds =150/0.7

$\times10^2$

$$=2.10\ mm$$

Use 5mm,mini size intermittent welds Eff

length of welds 10 t(10× 12)=120mm

Use 160mm long,5mm fillet welds alternately on either side

Step(13) end bearing stiffness

Maxi shear force =1221.46kN

'he end bearing stiffness is designed as a column (h/t)∄

12

If h=300mm

T=300/12

=25mm

Use 300 ×25mm plates
σ Permissible bearing stress $_p$ =189 N/mm^2
Bearing are required =f/stress

=1221.46 ×10^3/189

=6462 mm^2

If two plates are used ,

Total area provided =300 ×25× 2

=15000>6462 mm^2

The length of web plates which acts along with stiffness plates in bearing the reaction =20t

=20× 12

=240mm

I=25× 612^3/12+2 ×240× 12^3/12

=477.6× 10^{6}mm^4

Area A= (612× 25)+(480 ×12)

$$\frac{\sqrt{477.6} \times 10}{21060}$$

=21060 mm^2 R=

$\sqrt{}$I/A

=

=150.59mm

Eff length of stiffness =1.7 d

=0.7× 1600

=1120mm

F=1120/150.54=7.43

From table

$\sigma\sigma$Permissible stress ac in axial compression is obtained as

ac=138N/mm^2

Area required =f/a

=1221.46× 10^3/138

=8851 mm^2

Step(14) connection between bearing stiffness and web Length available for welding using alternate intermittent welds

=2(1610-40)=3120 mm

Required strength of welds =1771.46 ×10^3/3120

=391.49 N/mm

Size of welds= 391.44/0.7× 10^2

=5.48mm

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Step(15) lateral bracing

For resisting wind racking and centrifugal forces, lateral bracing is provided end cross frames and intermediates cross frames are provided for stress greater than 20mm

A wind force of intensity =1.5 kN/m^2 may be assured prevail

This wind force will act on the other girder may be taken as 25% of the forces in the windward direction.

Co-efficient of wind level on girder=1.25

Depth of girder =1.7mm

Wind load on windward girder=1.6× 1.2× 30

=76.5 kN

Wind load on leeward girder=0.25 ×76.5

=14.125kN

Total wind load=95.625kN

Lateral load due to racking forces =6KN/m

Total racking force=6 ×30

=180 KN

Total lateral load =w.60+r.f

=95.625+180

=275.62kN

=276kN

This total force acts in search a way that half of its is act one end this loading creates diagonal tension in the member

$\theta\sqrt{}$Maxi tension in diagonal =138 cosecθ

Sin =1.676/$\sqrt{2_2+1.676_2}$

Maxi tension in the diagonal =138 × 1.676^2 /1.676

=214KN

Area required =load/stress(0.6fy)

=214× 10^3/0.6 ×250

=1426 mm^2

Use ISA 80× 50 ×10

Area provided =1505mm^2 Maxi

comp force in list=138kN

Length of member =1.676 m Eff

length = 0.65

=0.65× 1.676

=1.084mm

A =1650 mm^2

=1089/21.6 =50.4

From table(IRC 24)

σPermissible stress ac=124 N/mm^2 Safe

load on member 204.6kN>138kN.