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Design of Pedestrian Steel Foot over Bridge

A Project Report

Submitted by

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BONAFIDE CERTIFICATE

It is certified that this project report **"DESIGN OF PEDESTRIAN STEEL FOOT OVER BRIDGE"** is the bonafide work of **"PRITHIVI KUMAR. V (211415103076), RAHUL.A(211415103079), SAMPATH KUMAR. V (211415103087), SELVIN CASTRO. I (211415103094)** and the project was carried out under the supervision of the following professors.

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ABSTRACT

Foot bridge are needed where a separate pathway has to be provided for the people to cross traffic flows or some physical obstructions such as river. The loads they carry are in relation to high ways or railway bridge quite modest are in most circumstances a fairly light structure is required. They are however frequently used to give a long clear span, and stiffness and become an important consideration. The bridges are often very clearly on view to the public and therefore the appearance merits careful attention.

The project Design of pedestrian steel foot over bridge. The design includes the estimation of loads and factor of safety taking care of all kinds of practical situations. The methodology used was limit state method based on IS 456:2000, IS800:2007 and SP-16.

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CHAPTER 1
INTRODUCTION

A. GENERAL

Klinth the promise of extensive beam generation isolated segments of the landscape are being connected by architecturally novel and challenging foot bridge. The challenges in foot bridge design lie him the fulfillment of the architectural demands for long light and slender structure this project details design and analysis of pedestrian steel foot over bridge. The design includes the estimation of loads and factor of safety taking care of all kinds of practical situations the methodology used was limits state method based on IS 456-2000 IS 800-2007 and sp-16.

B. SCOPE OF THE PROJECT

- To form the foot over bridge in the cities to relieve traffic congestion
- To reduce the accidents take place I proposed area
- For the convenience of the people to cross the roads easily and safely in the heavily congested area

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CHAPTER 2

LITERATURE REVIEW

- ***The Bilbao Steel Bridge spanning the nervion river: Portugalete***

The 165 meter bilbao transport bridge was constructed in 1893 which makes it the first bridge of this kind have been built designed by Alberto Palacio one of the Gustaveeiffel's disciples it spans the mouth of the firth of nervion linking the towns of portugalate and las arenas in the Biscay province of Spain . The bridge is called locally "Puente colgante" which roughly translated means suspension bridge after the previous bridge which was of the suspension bridge the existing bridge was reconstructed of the original. The reconstruction was undertaken by J.JanuAracil between 1939 and 1941 there are two new visitors lifts installed in 50 mtr high pillars of the bridge that allow brave visitors that do not suffer from vertigo to walk over the bridge's platform from where they can see the port and the Abra Bay. On 13thjuly 2006 it was declared a World heritage site by UNESCO.

- ***The Rochefort-Martrou Transporter Bridge spanning the river Charente***

The Rochefort-Martrou transporter bridge was constructed by Ferdinand Arnodin to cross the river Charente it came into service on 8thjuly 1900 at a cost of 586599 French francs it has span of 139.916 mtrs and is 59 meters in height. The deck beams and suspension were replaced and modified between 1933 and 1934. The bridge came out of service in 1967 when it was suspended by a nearby vertical lift bridge. On the 30thapirl 1976 it was classified as a historic monument and refurbished in 1996. It is still in use today and is a tourist attraction.

CHAPTER 3
BRIDGES

A. INTRODUCTION TO BRIDGES

A bridge is a structure built for carrying the road/railway traffic or other moving loads over a depression or gap or obstacle such as river, channel, canyon, valley, road or railway. Depending on the purpose and the obstacle the type of bridge is selected to meet the requirement if a bridge is constructed to carry a highway traffic it is called highway bridge if it used to carry a railway traffic it is called as railway bridge the bridges that are constructed exclusively to carry pedestrians, cycle and animals are known as foot bridges and bridges used to carry canals and pipe lines are known as aqueduct bridge.

B. CLASSIFICATION OF BRIDGES

- **BASED ON MOVEMENT OF STRUCTURAL PARTS**

A bridge may be movable or fixed. A fixed bridge is the one which always remains in one position, a movable bridge is the one which can be opened either horizontally or vertically so as to allow river or channel traffic to pass. Such bridge are constructed over a navigable stream where the normal headway is not sufficient for the vehicles to pass through.

- **BASED ON RESTING**

A bridge may be either of deck type or through type. A deck type is the one in which the roadway/railway floor rests on the top of the supporting structure while a through bridge is the one where the roadway/ railway floor rests on the bottom of the main load supporting structure;

- **BASED ON MATERIAL OF CONSTRUCTIONS**

Bridges are made of different materials such as timber masonry, brick masonry, concrete and steel. Timber bridges are constructed only over very small spans and for temporary purpose, to carry light loads. Masonry bridges are also constructed for shorter span. Concrete bridge, both of reinforced cement concrete as well as of pre-stressed cement concrete are constructed over moderate to high spans to carry all types of loads. Concrete arch bridges have been constructed of span up to 200m similarly steel bridges are constructed both over moderate to high spans as well as for heavy trafficular loads.

C. STEEL BRIDGE

The main advantage of structural steel over other construction material is its strength and durability. It has higher strength to cost ratio in compression when compared concrete. The stiffness to weight ratio of steel is much higher than of concrete. Thus, structural steel is efficient and economical in construction of bridge.

STEELS USED IN BRIDES

Steel used in bridges may be grouped under the following categories;

Carbon steel: This is the cheapest steel available for structural for structural for structural users where stiffness is more important than the strength

High strength steels: It is derived for their highest strength and other required properties for the addition of alloying elements. These are also called weathering steels in Europe.

Heat-treated carbon steel: These are steels with the highest strength. They derive higher strength from some from some form of heat treatment after rolling namely normalization or quenching and tampering.

The physical properties of structural steel such as strength, ductility, brittle fracture, weld ability, weather resistance etc., are important factors for its use in bridge construction these properties depend upon alloying elements the amount of cooling carbon rate of the steel and the mechanical deformation of the steel.

CLASSIFICATION OF STEEL BRIDGE

Steel bridge can be classified on the basis of the following criteria:

- Classification according to types of structural arrangement.
- Classification according to structural action.
- Classification according to types of connection.

- **CLASSIFICATION ACCORDING TO TYPES OF STRUCTURAL ARRANGEMENT**

Under this criterion the steel bridges may be of the following types;

I-GIRDER BRIDGE: when the span is small, simple I-girders may be as the main load carrying members. For this purpose, wide flange I-section are used. Such type of structural arrangements is used I-section however there are limitations in the maximum size of available I-sections.

PLATE GIRDER BRIDGE: for wide spans built up plate girder sections are used to meet the requirement of the section modulus corresponding to the imposed loads. In railways plate girder bridges are quite popular. They are also used for highway bridges.

TRUSS GIRDER BRIDGE: when the structural requirements of the depth of the girder is more as the main load carrying members. Truss girder bridge are commonly used for spans 20m to 200m.

SUSPENSION BRIDGE: for still longer spans, suspension bridge using high strength steel cables may be provided.

- **CLASSIFICATION ACCORDING TO STRUCTURAL ACTION**

  According to the criteria of structural action steel bridges may be classified as the following types.

- ✓ **SIMPLY SUPPORTED SPAN BRIDGES:** Such types are commonly used when the width of gap to be bridged is large the whole width can be subdivided into a number of individual span each span being simply supported.

- ✓ **CONTINOUS SPAN BRIDGE:** When the width of the gap is quite large and where there are no chance of uneven settlements bridge may be continuity, moments are developed at pier supports resulting in the reduction of stresses at the inner spans.

- ✓ **CANTILEVER BRIDGE:** In case of three span continuous bridge loaded with uniformly distributed load over all the three spans it is observed that there are two points of contra flexure in the central span. Hence if continuous beam of the middle span is cut at these two points of contra flexure and shear resisting joints are made at these two points the resulting configuration will be a cantilever bridge with a central suspended span between these two formed joints.

- ✓ **ARCH BRIDGE:** For deep gorges arch bridge are generally used since they offer economical and aesthetic solution however they require strong abutments to resist the thrust from the arches the arches may be consist of girder section or trusses and may be (a) fixed arches (b) two hinged arches (c) three hinged arches the two hinged arches being more common.

- ✓ **RIGID FRAME BRIDGE:** Rigid frame bridges compromising of single span or two to three continuous span are used for dry over or under crossing for gaps between 10 to 20m these consist of steel girders with continuity at the knee such bridge are suitable for rigid foundation.

- **CLASSIFICATION ACCORDING TO TYPE OF CONNECTION**

  Depending on the types of connections of the joints bridges can be of the following types:

- ✓ **RIVETED BRIDGE:** Great majority trends welding connection are used in the past are riveted bridges

- ✓ **WELDED BRIDGES:** In the modern trends welded connections are used in steel bridges the behavior of welded connections under impact and vibrations is not yet fully known hence except for the main joints welded connection are now increasingly used for built up sections and bracing elements particularly in highway bridges.

- ✓ **BOLTED BRIDGES:** Some of the older were pin-connected but their constructed was abandoned because they are less rigid and at the same time they require constant maintenance.

➢ ***ADVANTAGES OF STEEL BRIDGE***

- They could carry loads longer span with minimum dead weight leading to smaller foundation.
- Steel has the advantage where speed of construction is vital as many elements can be prefabricated and erected at the site.

D. FOOT OVER BRIDGE

Foot over bridge is used to cross the road which consists of very heavy traffic where the pedestrians find difficult while crossing and mainly to avoid the death rate takes place through accidents in peak areas most of the cities in India have adopted the foot over bridge for safety movements of pedestrians.

E. DECK SLAB

The design of steel pedestrian foot bridges differ depending on where they are to be situated show large they have to be a simple beam bridge is usually required for longer span in this project the span of the beam is more.

CHAPTER 4

DESIGN

A. DESIGN OF DECK SLAB

Lv = 2.5m

Ly = 3m

Ly/Lv = 3/2.5=1.2<2

F_{ck} = 25n/mm2

F_y= 415N/mm2

Since the ratio of long to short span is less than 2 the slab should be designed as two-way slab with provision for torsion at corners.

- ***DEPTH OF SLAB***

As the loading class exceeds the value of 3 N/mm2 adopt span/depth ratio as 25.

Depth=span/depth ratio=3000/25=120mm

D=120mm

D=150mm

Effective span =clear span+ effective depth

=3+0.12=3.12m

- ***LOADS***

Self-weight of the slab=0.15 x 25=3.75KN/m^2

Live load =5KN/m^2

Floor finish =1.25KN/m^2

Total load =10KN/m^2

Factored load =1.5 x 10=15Kn/m^2

Ultimate design moments and shear force

Referring table 26 of IS 456

Ay = 0.056 αx = 0.072

Mux = αyWuLx2 = 0.072 x 15 x 3.12sq = 10.51KNm

Vux = 0.5WuLx2 = 0.5 x 15 x 3.12sq = 23.4kN

- ***Check for depth***

Mmax = 0.144Fckbd^2

D = root off 15 x 10^6 / 0.144 x 25 x 1000

D = 65mm<120

Hence the effective depth selected is sufficient to resist the design ultimate moment.

- ***Reinforcement (short and long span)***

Mu=0.87fyAst D (1-AstFy/bdFck)

10.51 x 10^6 = 0.87 x 415 x Ast x 125(1-Ast x 415/100 x 125 x 25)

232.85=Ast-1.328 x 10^{-4} Ast^2

Ast = 240mm^2

Adopt 8mm dia at 200mm C/C in shorter span direction .

For y direction

Effective depth = 125-8 = 117mm

8.172 x 10^^{6} = 0.87 x 415 x Ast x 117(1-Astx415/100x117x25)

Ast = 200mm^2

Provide 8mm dia at 250mm C/C in longer span direction.

➢ ***CHECK FOR SHEAR STRESS***

Considering the shorter span and unit width of slob

Ωv = Vu/bd = 23.4 x 10^3/1000 x 125 = 0.87N/mm^2

Pt = 100Ast/bd = 100 x 240/1000 x 125 = 0.192%

Ωc = 0.319N/mm^2

K = 1.3

kΩc1.3 x 0.319 = 0.41N/mm^3>Ωv

Considering unit width of slab in the short span.

(l / d) = 20

Pt = 0.2

Kt = 1.5

(l/d) max=20 x 1.5 = 30

(l/d) actual =312/12 = 26

Deflection is satisfied

- ***Check for crack control***

Reinforcement provided

=0.012 x 150 x 1000 = 180mm^2

Spacing

=3 x 125 = 375mm

Dia of reinforcement<d/8<150.8<18.75

8 < 18.75

The cracks are within safe permissible limit.

- ***TORSION REINFORCEMENTAT CORNER***

=0.75 x 240.5 = 180.38mm^2

Length over which torsion steel is provided=1/5 x 25 = 0.5

Reinforcement in edge strip

Ast = 0.12%

=180mm^2

B. DESIGN OF CROSS GIRDERS

LL=5KN/m^2=5x3=15KN/m

Floor finish = 1KN/m^2=1x3=3KN/m

DL=11.25KN/m

Total load = 29.25KN/m

Total factored load=45KN?mreaction R1=65KN

- ***SELECTION OF SUITABLE SECTION***

Depth of section(h)=250mm

Widthofflange(b)=125mm

Thicknessof flange=8.2mm

Elastic section modulus=297.4x10^3 mm^3

Section classification

b/Tf=7.622

factoredslfwgt =0.279x1.5=0.42KN/m

total load =45.5KN/n

maximum bending movement 51KNm

plastic section=51x10^6x1.1/250

$=224400mm^2$

Design shear force =wl/2=68.25KN

Design of shear strength=200KN>68.25KN

Check for design capacity of the section

M_d=81.1KNm

Hence the design capacity of the member is more than max bending moment.

δ=4.25

Allowable maximum deflectiom=10m

The deflection is less than the allowable deflection.

C. DESIGN OF ARCH

➢ **MEMBER FORCES**

Method of joints

Ra+Rg=325KN

EM@A=0

Rg=162.5

Ra=162.5KN

EMJ=0

H=243.75KN

Joint A

EFv=0

162.5+Fahsin33.74=0

Fah=-295.45KN

Fab=245.68KN

Joint h

EFh=0

Fhi=-264.17KN

Fbh=66KN

Joint B

EFh=0

Fbc=245.68KN

EFv=0

Fbi=-1.37KN

Joint iEfh=0Fu=-246KN

Fcd=245.68kN

EFv=0

Fbh-65+Fbisin46.88=0

Fbi=-1.37KN

JOINT I

EFh=0

Fijcos7.5-Fincos21.8-Fincos46.88=0

Fij=-246KN

EFv=0

-Fic+Fusin7.5-Finsin46.88-Finsin21.8=0

Fic=67KN

JOINT C

EFh=0

-Fcb+Fcd+Fcjcos50.2=0

Fcd=Fcb=245.68KN

EFv=0

-65+67=-Fcjsin50.2

Fcj=-2.6KN

ARCH MEMBER (COMPRESSION)

Fah=Fgl=-295.45KN

Fhi=Flk=-264.17KN

Fij=Fkj=-245KN

HORIZONTAL MEMBER (TENSION)

Fab=fGF=245.68KN

Fbc=Ffe=245.68KN

Fcd=Fed=245.68KN

STRAIGHT MEMBER (TENSION)

Fbh=66KN

Fcj=67KN

Fdj=65KN

DIAGONAL MEMBER (COMPRESSION)

Fbi=-1.37KN

Fcj=-2.6KN

DESIGN OF TENSION MEMBERS

TENSION MEMBERS

A tension member is the one which is intended to resist axial tension. Tension members are also called ties or hangers. In contrast to compression member the disposition of material in atie has no effect on its structural efficiency so that compact section such as roads may be used without reduction in allowable stress for tensile force to be axial it is necessary that the load is applied through the centroid of the section of the member.

TYPES OF TENSION MEMBER

In general tension member can be divided into four groups

- Wires and cables
- Rods and bars

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- Single structural shapes and plates
- Built-up section

DESIGN OF ATENCION MEMBER

In the design of a tension member based on the tensile force acting on the member the designer has to arrive at the type and size of the member the type of member is chosen based on the type of the structure and location of the member.

The design is iterative involving a choice of atrial section and an analysis of its capacity the various steps are as follows.

The net area required An to carry the design load T is obtained by the equation

$An=Tu/(fu/^{\delta}m1)$

From the required net area the gross area may be computed by increasing the net area about 25%to 40%. The required gross area may be also be checked against that required from the yield strength of the gross section as follows.

$Ag=Tu/(fu/^{\delta}m0)$

The number of bolts or welding required for the connection is calculated. They are arranged in a suitable pattern and the net area of the chosen section is calculated the design strength of the trial section is evaluated using eqns in the case of plates and threaded bars and additionally using eqns in the case of angles.

The design strength is either small or too large compared to the design force a new section is chosen and step 3 is repeated until a satisfactory design is obtained as slenderness ratio of the member is checked.

HORIZONTAL MEMBERS

Number of bolt required=T/strength of one bolt

T=factored tensile load in the member

T=245.68KN

$Pshear=0.462fux0.78x\pi d^{2/4}$

Here the grade of bolt is 4.6

D=diameter of the bolt

Assume 20mm size bolt

Pshear=$0.462x400x0.78x\pi x20^{2/4}$

=45.28KN

In case of double section Pshear=2x45.28

=90.56KN

Pbearing=2 x t x dia of bolt x fu(plate)

=2x12x20x410

=196.8KN

Number of bolt required=245.68/90.56=3

Gross yield section

Tdg=fyAg/$^{\delta}$m0

Fy=yield strength of the material in MPa

Ag=gross area of the cross section

$^{\delta}$m0=partial safety factor

As per IS 800-2007, it is 1.10

Agross=245.68x10^3x1,1/250=3582.92mm$^{\wedge 2}$

For double angle section=1080.4x2=2161.98mm$^{\wedge 2}$

From steel table, pg:36

Provide 2 nos of ISA 100x100x8mm back to back

Gross sectional area =3078mm$^{\wedge 2}$

Rxx=30.7mm

Ryy=47.8mm

Ixx=Iyy+145.1x10$^{\wedge 4}$mm$^{\wedge 4}$

Tdg=fy/$^{\delta}$m0xAg

=250x3078/1.1

=699KN

NET SECTIONAL RUPTURE

Tdn=αxAnetxfu/$^{\delta}$m1

$^{\delta}$m1=1.25

Anet=2((100-8/2-22) x 8+(100-8/2)x8)

$=2720mm^2$

Tdh=803KN

Adopt lesser one

Td=699KN

=700KN(say)

Assume e=30mm

P=50mm

Lvg=30+50+50=130mm

Shearing off

Avg=Lvgxt

$=130x8=1040mm^2$

Tearinf off

$Atg=65x8=520mm^2$

Net shearing area

Avb=[Lvg-dn-dn-dn/2]xt

=[130-22-22-11]x8

$=600mm^2$

Net tearing area

$Atn=(65-22/2)x8=432mm^2$

Fy=250MPa

Fu=410MPa

$^{\delta}m0=1.1$

$^{\delta}m1=1.25$

Tdb1=1040x250/1.1+0.9x432x410/1.25

=264KN

$Tdb2=0.9Avnfu/(^{\delta}m1)+Atgxfu/^{\delta}m0$

=0.9x600x410/1.25+520x250/1.1

=220KN

For double angle Tdh1=264x2=528

Tensile capacity=220x2=440

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➢ *DESIGN OF TENSION MEMBER (STRAIGHT MEMBER)*

Tu=65KN

Number of bolt=Tu/strength of one bolt

Pshaer=0.462fux0.78xπx14$^{2/4}$

=22KN

$P_{bearing}$=2xt x d_{bolt} x fu(plate)

=2x12x14x410

=138KN

Bolt value=22KN

Number of bolt required=65x10^3/22x10^3=3

Agross=286mm^2

From steel table pg8;

ISA 50x50x5mm

Sectional area=479mm^2

Ixx=11x10mm^4

Tdg=fyAg/$^{\delta}$m$_0$

=250x479/1.1=109KN

RUPTURE CRITERIA

Anc=(50-5/2-15)x5=162.5mm^2

Ago=(50-5/2)x5=237.5mm^2

Fu=410MPa

$^{\delta}$m1=1.1

Bs=shear lag width

=50+25-5=70mm

$^{\beta}$=1.4-0.0769(50/5) x (250/410)(70/100) ≤ 1.44

=1.07 ≤ 1.44

T_{dbn}=0.9x410x162.5/1.25+1.70x237.5x250/1.1

=106KN

A_{vg}=L_{vg}x t

=130x5=650mm^2

Atg=30x5=150mm^2

Avn=(130-15-15-15/2)x5

=462.5mm^2

Atn=112.5mm^2

Tdb1=650x250/1.1+0.9x112.5x410/1.25

=118.5KN

Tdb2=0.9x462.5x410/1.25+150x250/1.1

Tensile capacity=113KN

- ***DESIGN OF COMPRESSION MEMBER***

T=295.45

Provide2 nosofISA 150x150x10mm back to back

Agross=5806mm^2

Rxx=46.3mm

Trial section ISA 150x150x10

b/tf=150/10=15<15.7

ɩ=(80-120)

kL/Rxx=1x3x10^3/46.3=65<180

hence ok

kL/Ryy=1x3x10^3/67.8=44<180

hence ok

fcd=196MPa

strength of compression member=196x5806

=1138KN>295.45KN

Hence the chosen section is safe .

- ***DESINGN OF DIAGONAL MEMBERS***

T=2.6KN

Area required (gross)=2.6x10^3/60=43.33mm^2

Trial section ISA 25x25x3mm

Sectional area =141mm^2

h/tf=25/3=8.33<9.4

the section is plastic

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ISA 50x50x3mm

Sectional area=295mm^2

Rxx=15.3mm

kL/Rxx=2.5x10^3/15.3=163.4<180

hence ok

kL/Rxx=163.4andfy=250MPa

fcd=61.2MPa

strength of compression member =62.2x295=18KN>2.6KN

Hence the chosen section is safe.

D. *DESIGN OF BEAM COLUMN*

- ***DESIGN OF BEAM COLUMN (uni-axial bending)***

Factored axial load = 162.5KN

Factored moment = 243.75x4.5 = 1100KNm

Step 1 Trial section

Single joist with additional plates on both flange be used as columns.

Steel joist

ISHB 450

W=925 KN

Sectional area = 43789 mm^2

Radius of gyration = 102.2 mm

tf=13.7 mm

h=450 mm

b=250 mm

effective length of column = 0.8L-0.8X4500=3600mm

KL=36000mm

Rzz=230.6 mm

KL/rzz=15.6

Equivalent axial load,Pe = P+2Mz/d

=162.5+2x1100x10x10x10/450

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=5051KN

From table 10 of IS800-2007,buckling class of cross section

Welded I section,

tf<40mm, Buckling about axis ZZ,

Buckling class is b,

From table 9b,

Design compressive stress fcd = 226MPa

Capacity of column = 226x43789/1000 = 9896>5051KN

Hence choose ISHB 450 as trial section

Step 2 Cross section classification

=250/fy = 1.0

b / tf= (250/2)/13.7=9.12<9.4

Hence the flange is compact.

WEB

d=450-2tf-2R

=450-2x13.7-2x15

d=392.6mm

tw=11.3mm

d/tw = 34.74<42

Hence the crossection is compact.

Step 3:Check for resistance of cross section to the combined effect

The interaction equation is

N/Nd+Mz/Mdz ≤1.0

My,Mz = factored applied moment about minor and major axis of the cross section respectively.

Mdy,Mdz = design strength under corresponding moment acting alone,

Mndy,Mndz = design reduced flexural strength under combined axial force and respective uni-axial moment acting alone.

N=Factored applied axial force.

Nd=Design strength in tension or compression due to yielding.

Nd=Ag fy/Ym0

Ag=gross area of cross section

Ym0=Partial safety factor in yielding

Nd=43789x250/1.1

Nd=9952KN

N=162.5KN

(1)When the factor design shear force does not exceed 0.6Vd (Vd=design shear strength of c/s)

Mdz = BpZpfy /Ym0

V = 162.5KN

Vd = Vn/Ym0

Vd=design shear strength

Vn = Av fyw /√**3**

Av = shear area

Welded connection = dtw=39.26x11.3

=4436.4 mm^2

fyw=250MPa

Vn=640KN

Vd=582KN

V<0.6Vd

162.5<349.2KN

Mdz =1xZpx250/1.1 = 227.3Zp

Zp = 2[400x40(20+225)+250x13.7(6.85+211.3)+11.3x211.3x211.3/2]

Zp = 9.84x10 3 mm^3

Elastic section modulus Zxx = 8.787x1000 mm3

Shape factor = Zp/Z = 9.84x1000 / (8.787x1000) = 1.12

Mdz = 227.3x9.84x1000 = 2236.6KNm

N/Nd+Mz/Mdz ≤ 1.0

162.5/9952 + 1100/2236.6 = 0.51<1.0

Member buckling resistance in compression

KL/rz = 15.6, fcd = 226MPa

Ppz = 226x43789/1000 = 9891KN > 162.5KN

Hence the section is safe.

➢ *DESIGN OF BASE PLATE*

f = P/A +(M/I) x y

=162.5 x 1000 /(1400x800)+(1100x10/(1400x1400x1400x800)/12)x1400/2

=4.35<8MPa

I = t3 x1/12

y = t/2

M/I = f/y

M/(t3/12) = f/0.5t

F = fy/FoS = 250/1.1 = 227.27 MPa

Maximum pressure = 4.35 MPa

The moment at X-X = (3x435x435/2)+(4.35-3)x435/2x2/3x435

=369.9x1000 Nmm

M/(t3/12) = f/0.5t

368.9x1000/(t3/12)x0.5t = 227.27

t = 99 (take 100mm)

Dimension of base plate

B = 1400mm

D = 800mm

t = 100mm

➢ *DESIGN OF FOUNDATION*

Interior column

axial load = 162.5KN

moment = 1100KNm

SBC of soil = P/A + M/I x y

=(162.5 x 1)/L x B + 1100/LxB/12 x L/2

SBC of soil is 250KPa

250BL = 162.5 x 0.167Lx + 1100/ 0.167L

41.75BL2 – 27.14L – 1100 = 0

If B = 1m, L = 5.5m

If B = 2m, L = 3.8m

CHECK FOR ONE WAY SHEAR

For M30 for 0.25% of steel,

Design shear strength of concrete = 0.37 N/mm^2

Shear, V= (P/L2) x L((L-a)/2)-d)

= 817500 – 500d

=V

0.37 x 3800 x d = 817500 – 500d

d = 430 mm

check for depth of two way shear

shear strength = 0.25√ fck

=0.25 √ 30 = 1.4N/mm^2

Taking section at d/2 around column, we get

V = 250 (7.6-(0.53+0.43)(0.4+0.43))

V=1700KN

Zp = 1700/2(a+d+b+d)d

= 1700 x 1000/2(530+430+400+430)430

= 1.1 N/mm^2< 1.4N/mm^2 (Hence ok)

DEPTH FROM BENDING

Section YY:

Mlong = P/LB x B x L-a/2 x L-a/4

= 162.5[$(L-a)^2$ / 8L]

=57.5KNm

SECTION XX:

Mshort = P($(B-b)^2$/8B)

=26KNm

Reinforcement required:

Longitudinal direction

M/bd2 = 57.5 x 1000 x 1000 / 2000 x 430 x 430

$=0.15\ N/mm^2$

Percentage of steel = 0.084 %

Ast = 0.084 x 2000 x 430 / 100

$= 723mm^2$

Provide 10 mm dia at 100 mm C/C spacing

Short direction

M/bd2 = 26 x 1000 x 1000 / 3800 x 430 x 430

$= 0.037\ N/mm^2$ p = 0.084

$Ast = 1372mm^2$

Hence provide 16 mm dia at 140 mm C/C

E. BEAM – COLUMN UNDER BIAXIAL BENDING

The geometry and loading of most framed structures are three – dimensional, and the biaxial bending of isolated beam columns with test results has been studied by several researchers and they have shown at the elastic biaxial bending of beam-column is similar to its in-plane behaviour, thus, in case the major and minor axis deflections and twist begin immediately after applying the load and increases rapidly as the elastic buckling load is approached, Although the first yield prediction of strength slender beam-columns, based on these methods, are of sufficient accuracy , they are found to be conservative yielding occurs before failure.

- ***DESIGN OF COLUMN***

LL = 5KN/m2 = 5x5x3 =45 KN

Floor Finish = 1KN/m2 = 1x3x3 = 9KN

DL = unit weight x volume = 25x3x3x0.15 = 33.75 KN

Total load = 87.75 KN

Take 90 KN acts over the slab of 3m x3m

Load acting over the slab = 22.5 KN

P1 = 22.5KN

Mz = 22.5 x 0.75 = 16.875KNm

My = 16.875KNm

Step 1: Trial section

Select ISHB 200 having the following properties

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$A = 4750\ mm^2$

Ry = 45.1 mm

H = 200 mm

b = 200 mm

bf = 9 mm

KL = 3600 mm

KL/rz = 3600 / 87.1 = 41.33

Pef = P + (2Mz/d) + (7.5My/b)

=22.5 + (2x16.875x1000 / 200) +(7.5 x 16.875 x1000 / 200) = 824 KN

From table 10 of IS 800-2007, for h/b >1.2,

Tf $\leq$ mm

Buckling about YY axis, buckling class is b

KL / rz = 41.33

Fy = 250 MPa

fcd = 204.4 MPa

capacity = 204.4 x 4750 = 970 > 824 KN

Hence choose ISHB 200 as a trial section.

Sectional properties:

H = 200 mm

bf = 200 mm

tf = 9 mm

$A = 4750\ mm^2$

rz = 87.1 mm

ry = 45.1 mm

$Iz = 3608.4 \times 10\ 4\ mm^4$

$Iy = 967.1 \times 10\ 4\ mm^4$

Cross section classification

$= \sqrt{}\ 250/fy = 1$

Outstanding flanges

b/tf = 100/9 = 11.11

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Class 3, semi – compact 15.7 E >11.11

WEB

d = H-2tf – 2R = 200 – 2x9 – 2x9 = 164mm

d/tw = 164/6.1 = 26.88 < 42 E

hence the web is class 3 , semi compact

Step 3: Compression resistance of the crossection

The design compression resistance of the cross section

Nd = Ag fy / Ym0 = 4750 x 250/(1.1 x 1000) = 1079.5 KN

Step 4: The design major axis bending resistance of cross section

Mdz = Bb Zpfy / Ym0 = 0.91 x 3.94 x 10000 x 250 / 1.1

=81.5 KNm> 16.875KNm

Zpz = 2[200x9x(4.5+91)+ 6.1 x 91 x 91 /2]

= 3.94 x 10 5 mm3

ELASTIC SECTION MODULUS

Zxx = 360.8 x 10^3 mm^3

Shape factor = Zp / Z = 3.94 x 10 5 /(360 x 1000) = 1.1

Mdy = Bb Zpyfy/Ym0

Zpy = 2 tf bf2 / 4 + (H-2tf)t^2w /4

Mdy = 1 x 1.82 x 10000 x 250 / 1.1 = 41.36KNm > 16.875KNm

Hence the bending resistance is fine along major Z-Z axis and minor **Y-Y axis**

Step 5: Shear resistance of cross section

The design plastic shear resistance of cross section

Vp = Av(fyw/√ 3)/ Ym0

Load parallel to web.

Max shear force V = [16.875-(-16.875)]/4.5 = 7.5KN

Av = Htw = 200 x 6.1 = 1220 mm_2

$Vp = 1220(250\sqrt{3})/(1.1 \times 1000) = 160\ KN > 7.5KN$

Hence the shear resistance of cross section is alright.

LOAD PARALLEL TO FLANGES:

$V = 16.875/4 = 4.22\ KN$

$Av = 2\ bf\ tf = 2 \times 200 \times 9 = 3600\ mm_2$

$Vp = 3600 \times (250\sqrt{3})/(1.1 \times 1000) = 472KN > 4.22KN$

Hence the shear force is alright

SHEAR BUCKLING:

$d / tw = 200/6.1 = 33 < 67E$

hence no shear buckling check is required.

Step 6: Cross sectional resistance

$(N/Nd) + (My/Mdy) + (Mz/Mdz) \leq 1.0$

N = factored applied axial force = 22.5 KN

$Nd = Ag\ fy/Ym0 = 4750 \times 250/(1.1 \times 1000) = 1079.5\ KN$

My = 16.875KNm

Mdy = 41.36KNm

Mz = 16.875KNm

Mdz = 81.5KNm

$2.5/1079.5 + 16.875/41.36 + 16.875/81.5 = 0.64 < 1.0$

Step 7: Member buckling resistance in compression

$Nd = Aefcd$

For buckling about the major Z-Z axis ,

$KL/rz = 41.33$

$Fcrz = 1213.35\ N/mm^2$

$KL/ry = 79.78$

$Fcr,y = 325.47N/mm^2$

Stress reduction factor = 0.38

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Pdz = Aefcd

Fcd = 86.75N/mm^2

Pdz = 475. X 86.75 = 412KN>22.5KN

Pdy = Aefcd

X = 0.67

Fcd = 154.08N/mm^2

Pdy = 4750 x 154.08 = 731.8KN > 22.5

Member buckling resistance in bending

M = 16.875KNm

Md = Bb Zpfbd

Determination of Mcr

= 188.9 KNm

Non – dimensional lateral torsional slenderness ratio

=0.21 for rolled section

Reduction factor for lateral torsional buckling

= 0.5[1+0.21(0.688-0.2)+0.688+0.688]

=0.787

Xlt = 1/[0.787+(0.787x0.787 – 0.68x0.688)]0.5

=0.855

Lateral torsional buckling resistance

=0.855(250/1.1) x 0.91 x 3.94 x 100000

=69.67KNm

M/Md = 16.875/69.67 = 0.24 $\leq$ 1.0 Hence OK

9) Member buckling resistance in combined bending and axial compression

Determination of moment application factor

Kz = 1 + (Yz – 0.2)P/Pdz

Cmy ,Cmz = equivalent uniform moment factor

Kz = 1 + (1.4 – 0.2)22.5/412 = 1<1+0.8 x 22.5 /412

=1.0$\leq$ 1.0

Cmz = 0.6

Ky = 1.0 $\leq$ 1.0

Ny = P/Pdy = 0.03

Cmlt = 0.4

Klt = 0.98>_ 0.98

Check with interaction formula

[P/Pdy] + [(KyCmyMy)/Mdy] + [KltMz/Mdz]

[22.5/731.8] + [1x0.6x16.875/41.36] + [0.98x16.875/81.5] = 0.48<1.0

Hence the section is suitable to resist the combined effects.

- ***DEIGN OF BASE PLATE***

f = (P/A) + (M/I) x y

0.9 < 8MPa

Select the size of the base plate is 300 x 300 mm

The moment at XX = 4720.8 Nmm

f = 250 /1.1 = 227.27 MPa

M/I = f/y where y = t/2 and I = t3/12

t = 11 mm can be taken as 15mm

Dimension of base plate.

L = 300mm

B = 300mm

T = 15mm

P shear = 0.462 x 400 x 0.78 x II/4 x 14 x14

=22.2 KN

Beam column size ISHB 200which has the following properties

A = 4750 mm^2

Zxx = 360.8 x 1000 mm^3

Actual stress = 22.5 x 1000 /4750 = 4.7 N/mm^2

Bending stress = 16.875 x 1000x1000/360.8 x 1000

Total length available for bending along the periphery of ISHB 200

2(200 + 200 – 6.1 +200 – 9) = 1169.8 mm

➢ ***DESIGN OF SQUARE FOOTING***

Plane area = 0.8m x 0.8m

Axial load = 22.5KN

Moment = 16.87KNm

Reaction = 22.5/0.8x0.8 = 35.156KN

Safe bearing capacity of soil = 250 KN/m^2

Depth for one way shear

For M30 , for 0.25% of steel = 0.35N/mm^2

D = 18 mm

Depth for two way shear

IS critical section at d/2 from face as in flat slab.

Perimeter = 4(0.4+0.018) = 1.672m

Shear = 35.156(0.8 x 0.8) – (0.4 x 0.4) = 1.672m

Shear value = 0.25

D = 365mm

Depth for bending M = 0.5625KNm

Reinforcement

Mu = 0.87fyAstd [1-fyAst/fck x bd]

Ast = 1209mm^2

Therefore provide 8 nos of 16mm dia bars of 160mm C/C

F. MIDDLE COLUMN

➢ ***DESIGN OF MIDDLE COLUMN***

Axial load = 325KN

Assume the section as ISHB 250

Height of the column = 4.5m

tf = 12.5mm

tw = 6.9mm

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h = 250mm

b = 125mm

rzz = 103.5mm

ryy = 26.5mm

Area = 4755mm^2

h/b = 2>1.2

tf = 12.5mm<40mm

Hence we should use buckling curve a about ZZ axis and b about YY axis

KL/rz = 34.6

Fcd = 212

Pd = 212 x 4755

KL/ryy = 135.8

Fcd = 83 N/mm^2

Pd = 83 x 4755

Pd = 394mm > 325mm hence the column is safe

- ***DESIGN OF BASE PLATE***

For ISMB 250

h = 250mm

b = 125mm

tf = 12.5mm

tw = 6.5mm

Bearing strength of concrete = 0.45fck = 0.45 x 20 = 9N/mm^2

Required area of base plate = 325 x 1000 /9 = 36111mm^2

Use the base plate of size = 300 x 170 = 51000mm^2

F= [P/A] + [M/Z] x y

=4.64MPa <5MPa

Bearing strength of concrete = 0.45fck = 0.45x 20 = 9N/mm^2

Required area of base plate = 325 x 1000 /9 = 36111mm^2

Use the base plate of size 450 x 250 mm

Projection (a) = 100mm

(b) = 62.5mm

W = 325 x 1000 / 450 x 250 = 2.88MPa

ts = 16.7 mm can be taken as 17mm > 12.5mm

Hence provide 450 x 250 x 17mm plate. Also provide four 20 mm diameter and 300mm long anchor bolts to connect the base plate to the foundation concrete.

WELD CONNECTING BASE PLATE TO COLUMN

Use a 6mm fillet weld all around the column section to hold the base plate in place.

Total length available for welding along the periphery of ISHB 250

= 2(125+125-6.9+250-12.5) = 961.2mm

- ***DESIGN OF FOUNDATION***

Axial load = 325KN

Safe bearing capacity of soil = 250KN/m^2

Assume fy = 415 N/mm^2

fck = 20 N/mm^2

Area = 325/250 = 1.3m^2

Adopt 1.2m x 1.2m square base of constant depth

Reaction = 325/1.2 x 1.2 = 225.69KN/m^2

Depth for one way shear

Assuming the main shear = 0.35 N/mm2 for 0.25% for steel.

d= [P (L-b)]/[2P+700L^2] in meters

d= 0.21m = 210mm

Depth for two way shear

IS critical section at d/2 from face of column.

Perimeter = 4(b + d) = 4(0.125+0.21) = 1.34m

Shear = 226(1.2 x 1.2 – 0.335 x 0.335) = 300KN

Shear value = 0.25fck = 0.25 x 20 = 1.12N/mm^2

d = 300 x 1000 / 1.12 x 1340 = 200mm

Depth from bending

M = P[(L-b)2/8L] = 325[(12-0.125)2/8 x 1.2] = 39.12 KN

Kfck = 0.138 x 20 = 2.76

d = 108.68mm < 120 mm

Reinforcement required

M/bd2 = 0.72

From Sp 16 steel percentage = 0.21 [>min 0.15%]

Ast = 534mm^2

Provide 10mm dia bar 130mm C/C spacing.

Total depth of footing = 300mm.

G. STAIR CASE

- ***DESIGN***

DESIGN OF STAIR CASE

Floor height =4.5m

Width of stairs = 3m

Rise = 0.15m

Tread = 0.3m

Landing = 3m

Length of each flight = 15 x 0.5 = 4.5m

Effective span = 15 x 0.3 +0.3 = 4.8m

Depth = span / 20 = 4800 / 20 = 240mm

Adopt depth, d = 220mm

LOAD

Dead load of slab on slope = 0.24 x 1 x 25 = 6KN/m

Dead load on horizontal span, W = 6.7 KN/m

Dead load on one step = 0.5 x 0.15 x 0.3 x 25 = 0.56 KN/m

Load on steps per meter length = 1.86KN/m

Floor finish = 1.44 KN/m

Total load = 6+6.7+1.86+1.44 = 16KN/m

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Factored load, Wu = 16 x 1.5 = 24KN/m

Bending moment, M = wl2/8 = 69.12KNm

CHECK FOR DEPTH

D = 138.06 < 220 mm

REINFORCEMENT

Mu = 0.87 fyAst d [1-(fy x Ast) / (fck x bd)]

Ast = 935mm2

Hence provide 12 mm dia bar at 120mm C/C

DISTRIBUTION REINFORCEMENT

0.12% of cross section area

= 0.0012 x 1000 x 240

=288mm2

Hence provide 8mm dia bar at 170 mm C/C

- **DESIGN OF BEAM SUPPORTING STAIRCASE**

1. Calculation of factor loads:

Dead load:

Self weight of slabs = 3.025KN/m^2

Self weight of steps = 2.625KN/m^2

Live load = 5.00KN/m^2

Total load = 10.65KN/m^2

Factored load = 1.5 x 10.645 = 15.98KN/m^2

Uniformly distributed load = 15.98 x 1.5 = 23.97KN/m

2. Calculation of bending moment and shear force

Maximum bending moment = 26.96KNm

Maximum shear force = 9KN

3. Shear modulus required = 424 x 1000 mm^3

4. Selection of suitable section:

Choose a trial section of ISLB 175 at 0.327KN/m

The properties of section as follows,

Depth section = 175mm

Wide of section = 90mm

Thickness of flange = 6.9mm

Depth of web = h-2(tf + R)

=142.2mm

Thickness of web = 5.1mm

Moment of inertia about major axis, Ixx = 1096.2 x 10 4 mm^4

Elastic section modulus = 125.3 x 1000 mm^3

Adequacy of the section including the self weight of the beam:

Factored self weight of the beam = 1.5 x 0.167 = 0.25KN/m

Total load acting on the beam = 24KN/m

Maximum bending moment = 27KNm

Section modulus required = 118.8 x 1000 mm^3

Since it is less than 143.3 x 1000 mm3 hence the chosen section is adequate.

Shear, V = WL/2 = 36KN

5. Design shear strength of the section :

Design shear strength, Vd = 117KN > 36KN

Also 0.6Vd = 70.2KN

Therefore the design shear force V < 0.6Vd

6. Check for design capacity of the section

d/tw = 27.88< 67E

= 28.33KNm

Where Pb = 0.87

28.33KNm<34.17KNm

Hence the design capacity of the member is more than maximum bending moment

7. Check for deflection

Deflection = 9.54mm

Allowable maximum deflection = L/300 = 3000/300 = 10mm

The deflection is less than the maximum allowable deflection.

- ***STAIR CASE DESIGN***

Axial load = 100 KN

Assume the section ISMB 200

Weight per meter = 254KN

Sectional area = 3233mm^2

H = 200mm

b = 100mm

tf = 10.8mm

tw = 5.7mm

ryy = 21.5mm

rxx = 83.2mm

Ixx = 2235.4 x 10 4 mm^4

Iyy = 150 x 10 4 mm^4

h/b = 2>1.2

Buckling curve a about ZZ axis and b about YY axis

KL/rxx = 0.8 x 2250 /83.2 = 21.6

Fcd = 223.56 MPa

Pd = 223.56 x 3233 = 723KN

KL/ryy = 0.8 x 2.25 / 21.5 = 83.7

Fcd = 143.4 MPa

Pd = 143.4 x 3233 = 464KN

- ***BASE PLATE***

Axial load = 100KN

Bearing strength of concrete = 0.45fck = 0.45 x 25 = 11.25N/mm^2

Size of the base pate = 250 x 200 = 50000mm^2

W = 3.32 MPa

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ts = 6 mm

hence provide 250 x 200 x 6mm plate . Also provide 14mm dia and 150 mm long anchor bolts to connect the base plate to the foundation concrete

WELD CONNECTING BASE PLATE TO COLUMN

Use a 6mm fillet weld all around the column section to hold the base plate in place.

Total length available for welding along the periphery of ISMB 200,

= 2(100+100-5.7+200-10.8)

=767mm

Provide 6mm fillet weld.

- ***DESIGN OF FOUNDATION***

Axial load = 100KN

SBC of soil = 250KPa

fy = 415 MPa

fca = 25N/mm^2

Area = 100/250 = 0.4m^2

= 0.7 x 0.7m

Reaction = 100 / 7x7 = 204KN

Depth for one way shear

Minimum shear = 0.35 N/mm^2

For 0.25% steel

d = 110mm

Depth for two way shear

Perimeter = 4(b + d) = 4(0.1+0.11) = 0.84m

Shear = 204(0.7 x 0.7 – 0.25 x 0.25) = 87.21 KN

Shear value = 1.25N/mm^2

d = 70mm

M = 13.1KN

d = 74mm

Reinforcement required

M/bd2 = 1.54

Steel percentage = 0.25>min of 0.15%

As = 192.5mm^2

Provide 8mm dia bar at 200 C/C

- ***DESIGN OF BEAMS***

Factored load = 20KN/m

Assume width of beam = 250mm

Overall depth of beam = 400mm

Effective cover = 50mm

Effective depth = 350nm

Ultimate moment and shear force = 0.125wu l^2

=22.5KNm

Vu = 0.5wu l

=30KN

Reinforcements

Mu limit = 0.138 Fck bd^2

=0.138 x 25 x 250 x 350 x 350 = 106KNm

Since Mu <Mu limit the section is singly rein forced and under reinforcement section

Ast = 170mm^2

Provide 4 number of 8 mm dia bars as tension reinforcement ans 2 bars of 10mm diameter as hanger bars on compression side,

CHECK FOR SHEAR STRESS

Vu = 30KN

Shear stress = Vu / bd = 0.34N/mm^2

Percentage tensile reinforcement = 100Ast / bd = 0.388

From table 19 of IS 456-2000, Design compressive strength of concrete = 0.431N/mm^2

Assume 2 legged 6mm vertical strips

Spacing of strips = 200mm

Check for deflection control

Pt = 0.388

F = 0.58 x 415 x [170/340]

= 177N/mm^2

Modification factor = 1.65

L/d max = [L/d] basic x modification factor

= [3000/350] = 9< 33 hence safe

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CHAPTER 5
CONCLUSION

Thus the arch type pedestrian foot over bridge is designed as per IS codes of practice. It is the most economical one as we designed as per the code. It can withstand heavy pedestrian loads greater than the normal traffic loads.

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