Comparative Study I And Box Girder Bridge

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Abstract- *A Box girder bridge is a bridge in which the main beams comprise girders in the shape of a hollow box. The box girder normally comprises either pre-stressed concrete, structural steel, or a composite of steel and reinforced concrete. The box is typically rectangular or trapezoidal in cross-section. Box girder bridges are commonly used for highway flyovers and for modern elevated structures of light rail transport. Although normally the box girder bridge is a form of beam bridge, box girders may also be used on cable-stayed bridges and other forms. Box girder bridges are popular around the world because of aesthetics and because of the structural advantages resulting from the high torsional rigidity of box sections.*

Earlier studies of the Box girders were analyzed considering box girder as beam. The studies on the box girder by discretization are scanty. In the present study the performance of box girder and I-girder were considered. The Pre-stressing steel and stresses and stresses developed at various sections and the deformations and quantities required for concrete and steel for both the bridges are computed.IRC class AA and class A loadings were considered for IRC:6-2000 for loading and analysis were carried out for moving loads, dead loads and earth quake loads. The load combinations were taken as per IRC: 6-2000. Analysis was carried out by STAAD PRO Using this software time is saved and it minimized manual errors. Comparing the percentage of steel and Percentage of concrete required is more in the case of I-girder when compared to box Girder Bridge. Pre-stressing steel is more in the case of I-girder.

Comparing various aspects it is concluded that the performance of the box girder bridge is better than that of I-girder Bridge. It is also provides accommodating space for cables and pipelines through the annular space of box girder.

I. INTRODUCTION

1.1 Back ground

Pre-stressed concrete is a method for overcoming concrete's natural weakness in tension. It can be used to produce beams, floors or bridges with a longer span than is practical with ordinary reinforced concrete. Pre-stressing tendons (generally of high tensile steel cable or rods) are used to provide a clamping load which produces a compressive stress that balances the tensile stress that the concrete compression member would otherwise experience due to a bending load. Traditional reinforced concrete is based on the use of steel reinforcement bars, rebar's, inside poured concrete.

1.2 Applications

Prestressed concrete is the main material for floors in high-rise buildings and the entire containment vessels of nuclear reactors. Unbounded post-tensioning tendons are commonly used in parking garages as barrier cable. Also, due to its ability to be stressed and then de-stressed, it can be used to temporarily repair a damaged building by holding up a damaged wall or floor until permanent repairs can be made.

The advantages of prestressed concrete include crack control and lower construction costs, thinner slabs - especially important in high rise buildings in which floor thickness savings can translate into additional floors for the same (or lower) cost and fewer joints, since the distance that can be spanned by post-tensioned slabs exceeds that of reinforced constructions with the same thickness. Increasing span lengths increases the usable unencumbered floor space in buildings; diminishing the number of joints leads to lower maintenance costs over the design life of a building, since joints are the major focus of weakness in concrete buildings.

The first prestressed concrete bridge in North America was the Walnut Lane Memorial Bridge in Philadelphia, Pennsylvania. It was completed and opened to traffic in 1951. Prestressing can also be accomplished on circular concrete pipes used for water transmission. High tensile strength steel wire is helically-wrapped around the outside of the pipe under controlled tension and spacing which induces a circumferential compressive stress in the core concrete. This enables the pipe to handle high internal pressures and the effects of external earth and traffic loads.

1.5 Advantages and Disadvantages

Compared to I-beam girders, box girders have a number of key advantages and disadvantages. Box girders offer better resistance to torsion, which is particularly of benefit if the bridge deck is curved in plan. Additionally, larger girders can be constructed, because the presence of two webs allows wider and hence stronger flanges to be used. This in turn allows longer spans. On the other hand, box girders are more expensive to fabricate, and they are more difficult to maintain, because of the need for access to a confined space inside the box.

1.6 OBJEVTIVE

Analyzing the box girder and I-girder by discretization.

1. To study the design of box girder and I-girder bridge using STAAD PRO -2006.
2. To study the performance of box girder and I-girder bridge under class–AA Tracked and wheeled loading.
3. To identify the critical stress zone.
4. Analyzing the deformations, bending moments and shear forces by keeping same span and width in both the girders for various loads according to IRC: 6-2000, IRC: 18-2000.
5. To estimate the quantities for steel and concrete and to discuss the economics between the two types.

1.7 SCOPE

1. Single span multi cell box girder will be considered.
2. Equivalent cross section of the box girder is considered to find out the maximum bending moment and shear force.
3. Hallow circular cross section of the cable ducts are not considered in the analysis.

1.8 ORGANIZATION OF THE THESIS

In the first chapter, a brief description on introduction to the project, literature review, objectives, scope is presented. Second chapter deals with the literature review. Third chapter contains methodology. Fourth chapter deals with design procedure for pre-stressed box girder bridge as per IRC–18-2000. Fifth chapter contains design procedure for pre-stressed I-Girder bridge. Sixth chapter contains results and discussions. And the last chapter contains conclusions. At the end of the report, references and annexure are presented.

II. LITERATURE REVIEW

LÜ, ZHITAO, PAN, ZUANFENG[1] (2009) studied about the Issues in design of long-span prestressed concrete boxgirder bridges. Prestressed concrete box girder bridges are widely used because of advantageous structural behavior and perfect shapes. However, in the past few years, excessive deflection at the midspan and cracks in the box girders have commonly appeared in some long-span prestressed concrete box girder bridges after a period of operation, reducing the serviceability and durability of the bridges. The causes of cracks and excessive deflection are still not clear. Long-span prestressed concrete box girder bridges are studied from several aspects, such as longitudinal prestress design, vertical prestress design, vertical prestress loss, composition design, configuration of stirrups, characteristics of concrete. Two methods of longitudinal prestress design are compared, and the study indicates that the layout of bent-down tendons is an effective means for avoiding diagonal cracks in the webs. Moreover, the influences of shear deformation and development of diagonal cracks in thin-walled box girders on long-term deflection could not be ignored.

After going through the literature review most of the studies are on either box girder bridge or an I-girder bridge. Studies on the quantity requirements for both the girders keeping the span and width same are scanty. Hence in the present study both box girder and I-girder are considered for studying their behavior, stress computations, prestressing steel requirements and concrete requirements and are compared.

III. METHODOLOGY

A bridge is a structure providing passage over an obstacle without closing way beneath. The required passage may be for a road, a railway, pedestrians, a canal or a pipeline. The obstacle to be crossed may be a river, a road, railway or a valley.

Bridge may be classified according to the material of construction of super structure as timber, masonry, iron, steel, reinforced concrete, prestressed concrete, composite or aluminium. In the above types of bridges prestressed concrete bridges are playing a vital role in the present day construction. Because of the advantages over reinforced concrete bridges prestressing become popular. Basically there are two types of prestressing Pretensioning and Post tensioning.

1. Design the end block. The end block should be rectangular in section of width equal to the width of the bottom flange of the girder.

Design of End Block:

The portion of the prestressed concrete girder surrounding the anchorages of the tendons at an end of girder is called an end block. It is usually made rectangular in section with width equal to width of the flange of girder. The purpose of the end block is to distribute the concentrated prestressing forces at the anchorages and to facilitate gradual transmission of the forces to the basic cross section. The length of the end block should be about one half the depth of the girder, but not less than 600 mm or its width. The concentrated force at an anchorage causes bursting tensile force and spalling tensile force in the end block. The bursting tensile force is estimated from tabular values given in IRC 18-2000 as a function of the force in the tendon and the ratio of the loaded width to the total width at the anchorage without overlap**.**

Deck slab:

Experiments on prestressed concrete slabs have shown that they assure a large factor of safety against both cracking and failure. It may therefore the permissible to disregard the condition of ensuring permanent compression which is often considered as a base for designing members of prestressed concrete structures.

IV. DESIGN OF POST TENSIONED PRESTRESSED CONCRETE BOX GIRDER SLAB BRIDGE DECK

4.1.1 DESCRIPTION

Dimensions of the box girder:-

Total length of the girder = 40.00m
Depth of box girder = 2.30m
c/c support of box girder = 38.4m
Overall width of box girder = 16.940m
Clear carriage way width = 7.50m
Over hanging length of deck slab = 3.2475m
Bottom Width = 6.00m
Kerb thickness = 370mm
Side slope = 2.5%
Thickness of top flange (at edges) = 200 mm

4.1.2 MATERIALS:

Concrete

Grade of concrete = M_{45}
Density of concrete = 24.525 KN/m^3

Untensioned steel

Type of untensioned steel = Fe 500

Prestressing steel

Type of HTS = untensioned stress relieved low relaxation steel confirming to IS 14268

Type of prestressing system proposed = 19K15 type

4.1.3 MATERIAL PROPERTIES:

Modulus of elasticity of concrete $E_c = 5000\sqrt{f_{ck}}$ = 33541.02 N/mm^2

Prestressing steel:

Cross sectional area of each strand = 140mm^2

Type of sheathing duct proposed = corrugated HDPE sheathing ducts.

Diameter of the duct for 19K15 = 100 mm ID
= 107 mm OD

The wobble coefficient per meter length of steel K = 0.002

Coefficient of friction μ = 0.17
Modulus of elasticity $E_s = 1.95 \times 10^5$ N/mm^2
Modular ratio m = E_s/E_c = 5.81

4.1.4 PERMISSIBLE STRESSES: (CI: 7 of IRC 18-2000)

Concrete:

a. Temporary stress in extreme fibre (CI:7.1 of IRC 18-2000)

Permissible compressive stress = 0.5 x f_{ck} = 0.5 x 45 = 22.5 N/mm^2
Permissible tensile stress = 0.1 x σ_c = 0.1 x 22.5 = 2.25 N/mm^2
Minimum strength of concrete at the time of stressing = 40.5 N/mm^2

b. Stress in extreme fibre at service (CI:7.2 of IRC 18-2000)

Permissible compressive stress in service =0.33f_{ck} = 0.33 x 45 =14.85 N/mm^2

Untensioned steel:

Permissible flexural stress = 240N/mm^2
Permissible stress in shear = 200N/mm^2

Prestressing steel:

Ultimate tensile strength = 1862 N/mm^2
Maximum jacking stress = 0.765xUTS = 0.765x1862 = 1424.43 N/mm^2

4.1.15 CALCULATIONS OF ORDINATES FOR CABLE PROFILE:-

$Y = 4ex(l-x)/l^2$

At center x=39.1/2 =19.550

Total area of cables:

CABLE NO	ANCHORAGE TYPE	OUTER DIAMETER
1	19K15	107mm(O.W)
2	19K15	107mm(O.W)
3	19K15	O.W
4	19K15	O.W
5	19	I.W
6	19	I.W
7	19	I.W
8	19	I.W
9	19	Soffit slab
10	19	Soffit slab

a) LOSS OF PRESTRESS DUE TO SHRINKAGE OF CONCRETE:-
b)

Stressing is considered to be done after concrete attaining 100% strength behind the anchorage. But not less than 28 days from completion of last pour of concrete. Strain due to residual shrinkage at concrete considering the age of concrete at the time of Stressing of 28days.
Refer table no 3 of IRC: 18-2000

$\varepsilon_s = 1.9 \times 10^{-4}$

Loss of prestress due to shrinkage of concrete = $\varepsilon_{cs} \times E_s$
$= 1.9 \times 10^{-4} \times 2 \times 10^5 = 38$ N/mm^2

c) LOSS OF PRE-STRESS DUE TO CREEP OF CONCRETE:-

The loss due to creep of concrete is calculated by creep method. Considering 100% maturity of concrete at the time of stressing the residual creep strain.

$\in c$ per 1 mpa= $4\times 10^{-4.}$

4.1.17 STEEL IN LONGITUDINAL DIRECTION:-

As per clause 15.3 of IRC: 18-2000;
0.18% of the gross sectional area of the web considering 1m height web.
Area of steel required for 310 mm thick web
$A_{st} = 0.18 \times 310 \times 1000/100 = 558$ mm^2
Area of steel required for 450 mm thick web
$A_{st} = 0.18 \times 450 \times 1000/100 = 810$ mm^2
Area of steel required for 500 mm thick web
$A_{st} = 0.18 \times 500 \times 1000/100 = 900$ mm^2
Provide 10 mm ϕ bars @ 200 c/c.

4.1.18 FOR SOFFIT SLAB OF BOX GIRDER:

(1) Longitudinal direction : as per clause 15.4 of IRC 18-2000

Thickness of soffit slab = 200 mm
Considering 1m length area of steel required $A_{st} = 0.18 \times 200 \times 1000/100$
= 360 mm^2
Spacing of 10 mm dia bars on one face = (70 x 2/360) x1000
= 388 mm
Spacing should not exceed 200 mm
Hence provide 10 mm ϕ @ 200 mm c/c

(2) Transverse direction:

Considering 1m width area of steel required = 388 x 2 = 776 mm^2
Spacing of 10 mm ϕ bars on one face = (2 x 70/776) x 1000
= 180 mm
Hence provide 10 mm ϕ @180 mm c/c

4.1.19 TORSIONAL MOMENT DUE TO LIVE LOAD:

Distance between centres of box girder to the wheel of tracked vehicle
= 8.47-(0.5+1.625+2.05) = 4.295m
Twisting moment with impact factor = 700 x 4.3 x 1.1 = 3311 KN-m
As per clause 14.2.3.3 (IRC 18-2000)
$Vt = T/(2xh_{wo}xA_o)$
A_o= 2.085 x 0.65 = 1.35 x 10^6 mm^2
$V_t = 3311 \times 10^6/(2 \times 650 \times 1.35 \times 10^6) = 1.88$ N/mm^2
V_{tc} = 0.42 (from table 7 of IRC 18-2000)
$V_t > V_{tc}$ additional torsional reinforcement are required.
V_{tu} = 1.88-0.42 = 1.46 N/mm^2
Balance twisting moment = 1.46 x2 x h_{wo} x A_o
= 1.46 x 2 x 650 x (2085 x 650)
= 2572 x 10^6 N-mm
Torsional reinforcement is to be provided such that
$A_{sv}/S_v = T/(A_o \times 0.87 \times f_y)$
$A_{sv}/S_v = 2572 \times 10^6/(2085 \times 650 \times 0.87 \times 500)$
= 4.36 mm
Area of longitudinal reinforcement
$A_{sl} \geq A_{sv}/S_v \times (\text{perimeter}/2) \times f_{yv}/f_{yl}$
Where A_o = 2 x (650+20685) = 5470 mm
$A_{sl} = 4.36 \times (5470/2) \times (500/500) = 5962.3$ mm^2
Provide 12 mm ϕ bars, area of each bar = 113 mm^2
No of bars = 5962.3/113 = 52.76 say 52 no's
Use 12 mm ϕ bars 26 no's at each face are provided.

4.1.20 DESIGN OF DECK SLAB OF BOX GIRDER:

B.M due to dead load:
a. Weight of deck slab = (0.2x25) = 5 KN/m.
b. Weight of wearing coat = (0.06x22) = 1.32 KN/m.
c. Weight of crash barrier = 2x2.5 (0.5+0.13)/2 =15.75 KN/m.
Total = 22.07KN/m.
B.M due to dead load = Wl^2/8 = (22x5^2)/8 = 68.75 KN-m.
Taking continuity effect maximum B.M = 68.75x0.8 = 55 KN-m.

B.M due to Live Load:

K for continuous slab = 2.6; dispersion width = 0.85+2(0.2+0.06) =1.37 m.
B_w =3.6+2(0.06) =3.72 m.
X = 1050 mm.
B_e = kx[1-x/l]+B_w =2.6x1.05[1-1.05/5.0]+3.72 = 5.87
Dispersion width = (5.87+2.05) = 7.926
Load per meter width = (2x350)/(7.926x1.37) = 64.45 KN/m.

Maximum B.M due to Live Load = (64.45x1.37)2.075 – (64.45x1.37)x(1.025)
= 190.66 – 94.18 = 92.71 KN-m
Design B.M due to Live Load including impact and taking continuity effect
= 92.71x1.0x0.8 = 74.17 KN-m

Design of deck slab section:
B.M due to dead load = 68.75 KN-m
B.M due to live load = 74.17 KN-m
Total Load = 74.17+68.75 = 143 KNm
The design bending moment = 143
Grade of concrete = M_{45}, m=10, k = 0.365 , j = 0.878 , q = 1.843
Effect depth required d = $\sqrt{(M/Rb)} = \sqrt{(143x10^6)/(1.843x1000)}$ = 178.55.
Depth provided is 200 mm.
Main reinforcement
A_{st} Required = M/(σ_{st} x j x d) = (143 x 10^6)/200 x 0.878 x 200
= 4071 mm^2
Use 200 ø bar @ . [(π/4)x$(16)^2$ /4071] x 1000 = 77.13 mm
Provide 80 mm c/c reinforcement in longer direction.
Distribution Steel: = 0.15 DL + 0.3 LLBM

= (0.15 x 68.75) + (0.3 x 74.17) = 32.56 KN-m
A_{st} = (32.56 x 10^6) / (200 x 0.878 x[200 – 25]) = 1000 mm^2
Provide 12 mm HYSD bars 1100 mm c/c
. Moment due to Live Load on cantilever:
Class A loading can operate on cantilever
b_w = 0.25 + 2x0.006 = 0.37
Effective width(b_e) = 1.2X + b_w
= 1.2(1.735) + (0.37) = 2.452 m
Load per meter width = (114 / 2.452) = 46.50 KN/m

B.M due to Live Load including impact = 1.1 x 46.50 x(20/100) = 10.22 KN/m

Design of section:
Moment due to dead load :
Crash barrier: (25 x 0.351 x1.0) = 8.775
=8.775 x (3.47 – 0.5/2) = 28.25 KN-m
Cantilever slab: (25 x [(0.3+0.2)/2] x 3.47 = 43.375 KN-m
B.M due to dead load = 71.625 KN-m
B.M due to live load =10.22 x1 = 10.22 KN-m
Total = 81.845 KN-m
Effective depth required d = $\sqrt{M_u}$/ Qb =√(81.845 x 10^6)/(1.843 x 1000) = 210 mm.
Over all depth = 210 + 40 = 250
A_{st} Required = M/ σ_{st} x j x d = (81.845 x 10^6) / (200 x 0.878 x 210) = 2219 mm^2
Provide 16 mm HYSD bars = (201 / 2219) x 1000 = 100 mm
So hence provide 16 mm @ 100 mm c/c
B.M for distribution steel = 0.3 x B.M due to LL + 0.2 x B.M due to DL
= (0.3 x 10.22) + (0.2 x 71.625) = 17.40 KN-m
A_{st} Required = (17.40 x 10^6) / (200 x 0.878 x 200) = 495.44 mm^2
Min reinforcement provide A_{st} = (0.18 / 100) x 1000 x 240 = 432 mm^2
Provide 10 mm HYSD bars = (78.5/495) x 1000 = 150 mm c/c.

V. DESIGN OF POST - TENSIONED PRESTRESSED CONCRETE I GIRDER SLAB BRIDGE DECK

5.1.1 DIMENSIONS OF I- GIRDER

Span length = 4 lanes = 7.5 x 2 = 15m
Crash barrier = 500 – 130 = 370mm
Thickness of wearing coat = 60mm
Live load = IRC class AA tracked vehicle
Grade of concrete = M45
Spacing of cross girder = 5m
Loss ratio = 0.8
Fe 415 HYSD bars are available for use.

5.1.2 PERMISSIBLE STRESSES:

For M45 grade concrete and Fe 415 steel (IRC 21:2009)
σ_{cb} = 13.33 N/mm^2
σ_{st} = 200 N/mm^2
Modular ratio m = E_s/E_C = 10
From (IRC-18:2000)
Permissible stresses for M40 grade concrete
F_{ck} = 45 N/mm^2
F_{ci} = 0.45 f_{cj} = 0.45 x 32 = 14.4 N/mm^2

$F_{cj} = 0.8\ f_{ck} = 32\ N/mm^2$
Allowable tensile stress at transfer = $-\ 0.1\ f_{ci} = -0.1 \times 14.4 = -1.44\ N/mm^2$
Allowable compressive stress service = $0.33\ f_{ck} = 0.33 \times 45 = 13.2\ N/mm^2$
Modulus elasticity of concrete $Ec = 5000\sqrt{f_{ck}} = 5000\sqrt{45} = 31622.77\ N/mm^2$
Modulus elasticity of steel $Es = 2 \times 10^5\ N/mm^2$
m = 10
$K = (m x \sigma_{cbc})/(m x \sigma_{cbc} x \sigma_{st})$
$L = ½ \times \sigma_{cbc} \times j \times k = 1.843$
$J = 1 – k/3 = 1 – 1/3 \times 0.365 = 0.878$, $\sigma_{st} = 200\ N/mm^2$

5.1.3 CROSS SECTION OF DECK:

4 main girders are provided at 3.31m interval.
Thickness of deck slab = 250mm
Wearing coat = 60mm
Crash barriers of 500mm wide and 1.1m deep are provided at each end.
Spacing of cross girders = 5m
The overall depth of main girders is assumed
at 500mm per meter of span
Therefore overall depth of girder = 50 x 40 = 2000mm = 2m
Cross sectional area:
$A = (2000 \times 800) – (4 \times ½ \times 0.25 \times 0.15) – (2 \times 1.1 \times 0.25)$
$A = 0.975\ m^2$
Moment of inertia
$I = 6800 \times 2000^3/12 – (2 \times 0.25\ 1.1^3/12) – (4 \times 0.25 \times 0.15^3/36)$
$I = 0.477\ m^4$
$y_b = y_t = 2/2 = 1m$

Section modulus $z_b = z_t = 0.477/1 = 0.477\ m^3$

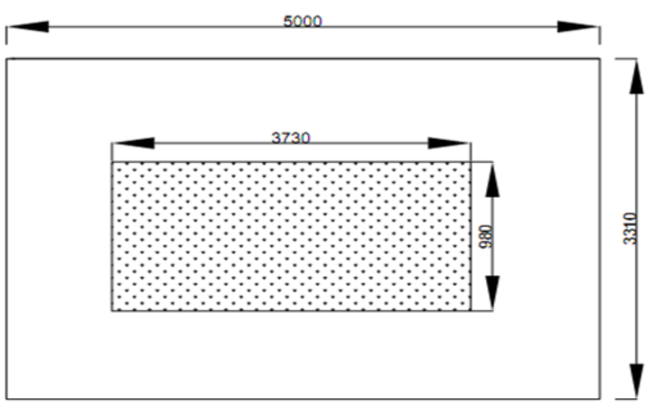


Fig 5.1 Cross section of I-girder

5.1.4 DESIGN OF INTERIOR SLAB PANEL

The slab panels 3.3m by 5m is supported by all four sides

a) BENDING MOMENT :-

Dead weight of slab = (1 x 1 x 0.25 x 24) = 6 KN/m^2
Dead weight of wearing coat = (0.06 x 22) = 1.32 KN/m^2
Total dead load = 7.32 KN/m^2

U = 0.85 + (2 x 0.065) = 0.98m
V = 3.6 + (2 x 0.065) = 3.73m
U/B = 0.98/3.31 = 0.296
V/L = 3.73/5 = 0.746
K = B/L = 3.31/5 = 0.662
Referring to piguard's curves
$M1 = w (m_1 + 0.15\ m_2)$
= 350(0.101 + 0.15 x 0.022

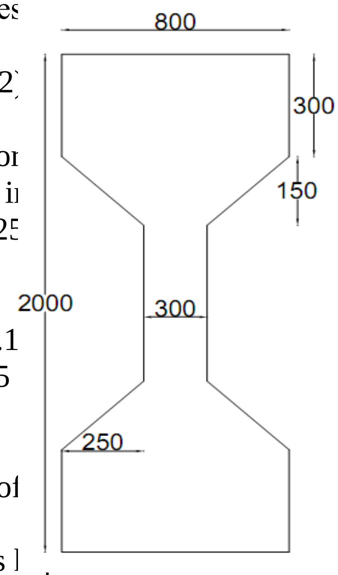


Fig 5.2 Plan showing interio
Bending moment including i
Mb along shorter span = 1.25
m
$Ml = w (m_2 + 0.15m_1)$
Ml = 350 (0.022 + 0.15 x 0.1
Ml along longer span = 1.25
13 KN-m
b) SHEAR FORCES:-
Dispersion in the direction o
+ 0.25)] = 1.47m
For maximum shear, load is l
whole dispersion is in span the load is kept at (1.47/2 = 0.735m) from the edge of the beam as shown in fig.

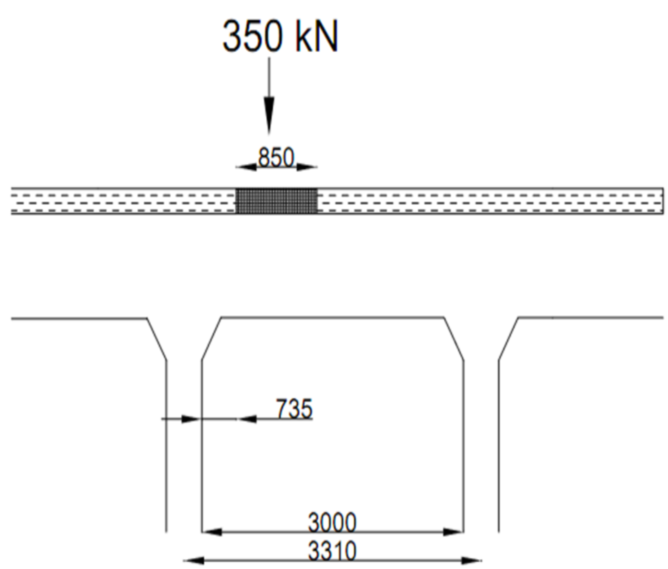


Fig 5.3 Position of Vehicle on I-girder Bridge.

Effective width of slab = kx (1 – x/l) + bw
Breadth of cross girder = 200mm
Clear length of panel = 5 – 0.2 = 4.8m
(B/L) = 4.8/3.0 = 1.6
From table k for continuous slab is obtained as 2.60
Effective width of slab = 2.6 x 0.735 (1 – 0.735/3.0) + [3.6 + (2 x 0.06)]
= 1.442 + 3.72 = 5.162 m
Load per meter width = 350/5.162 = 67.8 KN
Shear force/meter width = 67.80 ((3 – 0.735)/3) = 51.189 KN
Shear force with impact = (1.25 x 51.189) = 63.98 KN

c) DEAD LOAD BENDING MOMENTS AND SHEAR FORCES:-

Dead load = 7.32 KN/m^2
Total load on panel = (5 x 3.31 x 7.32) = 121.146 KN
(U/B) = 1, (V/L) = 1, as panel is loaded with uniformly distributed load
K = B/L = 3.31/5 = 0.662
1/k = 1.51
From pigeaud's curve
m_1 = 0.048, m_2 = 0.021
mb = 121.146 (0.048 + 0.15 x 0.021) = 6.19 KN-m
ml = 121.146 (0.021 + 0.15 x 0.048) = 2.67 KN-m
Design B.M including continuity factor,
Total mb = (0.8 x 6.19) = 4.952 KN-m
Ml = (0.8 x 2.67) = 2.136 KN-m
Dead load shear force = ((7.32 x 3)/2) = 10.98 KN

d) Design moments and shear forces

Total mb = 36.5 + 4.952 = 41.452 KN-m
ml = 13 + 2.136 = 15.136 KN-m

e) DESIGN OF SLAB SECTION AND REINFORCEMENT

Effective depth d = √m/Q b = √ ((41.452 x 10^6)/(0.762 x 1000)) = 233.23 mm
Adopt effective depth d = 235 mm
A_{st} = (M/(σ_{st} x j x d)) = (41.452 x 10^6)/(200 x 0.96 x235) = 918mm^2
(Use 12mm diameter bars at 120 mm centers) 942mm^2
Effective depth for long span using 10mm Ø bars.
= (235 – 12/2 – 10/2) = 224mm
A_{st} = ((15.136 x 10^6)/(200 x 0.96 x 224)) = 352mm^2

But minimum reinforcement using HYSD bars according to IRC – 18:2000,
0.15٪ of cross section
Hence A_{st} = ((0.15/100) x 1000 x 250) = 375mm^2
Use 10mm diameter bars at 150mm centres for crack control (IRC – 21:2000)
Nominal shear stress $\tau_v = V_u/bd$ = (63.98 x 10^3)/(1000 x 235) = 0.252 N/mm^2
The permissible shear stress in slab τ_c is given by
$\tau_c = k_1\, k_2\, \tau_{co}$
Where $k_1 = (1.14 – 0.7d) \geq 0.5$ (where d in m)
= (1.14 – 0.7 x 0.235) = 0.975
$k_2 = (0.5 + 0.25\rho) \geq 1$ where ρ = (100 A_{st} / bD)
ρ = (100 x 942)/(1000 x 235) = 0.4
k_2 = 0.5 + (0.25 x 0.4) = 0.6 ≥ 1
For M_{40} grade concrete τ_w = 0.5 N/mm^2
τ_c = 0.975 x 0.6 x 0.5 = 0.2635 N/mm^2
Since $\tau_v < \tau_c$ the shear stresses are within safe permissible limits.

5.1.4 DESIGN OF LONGITUDINAL GIRDERS:-

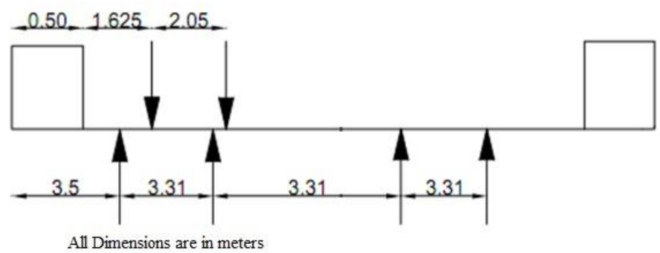


Fig 5.4 Position of Vehicle on I-girder bridge

a) Reaction factors:-
Outer girder A is
Reaction
$R_a = 2w_1/4\, [1 + ((4I \times 4.97 \times 5.095)/(2I \times 4.97^2) + (2I \times 1.66^2))]$
$R_a = 1.422\, w_1$
$R_b = 2w_1/4\, [1 + ((4I \times 1.66 \times 5.095)/(2I \times 4.97^2) + (2I \times 1.66^2))]$
$R_b = 0.808\, w_1$
If w = axial load = 700 KN
W1 = 0.5w = 0.5 x 700 = 350 KN
R_a = 1.422 x 350 = 497.7 KN
R_b = 0.808 x 350 = 282.8 KN

b) Dead load from slab per girder

Weight of crash barrier = 0.375 x 1.1 x 25 = 10.31 KN/m
Weight of deck slab = (0.25 x 3.5 x 24) = 21 KN/m
Total = 31.31 KN/m
Total dead load of deck = (2 x 31.31) + (7.32 x 9.94) = 135 KN/m
It is assumed that the deck load is shared equally by all the four girders.
Dead load/girder = 135/4 = 33.75 KN/m

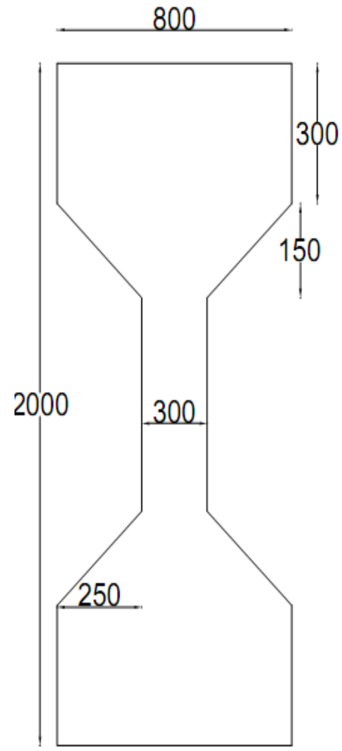
c) Dead load of main girder:-
The overall depth of the girder is assumed as 2000mm at a rate of 50mm for every meter of span
Span of the girder = 40m
Overall depth = 50 x 40 = 2000 = 2mlaced
Dead weight of rib = (1.1 x 0.3 x 24) = 7.92 KN/m
Dead weight of bottom flange =
(0.8 x 24 x 0.3) + [(0.8 + 0.3)/2 x 0.15] x 24
= 5.76 + 1.98 = 7.74 KN/m
Total weight = 7.92 + 7.74 = 15.66 KN/m
Weight of cross girder = (0.2 x 1.4 x 24) = 6.72 KN/m

d) Dead load moments and shear in main girders:
Reaction from deck slab on each girder = 33.75 KN/m
Weight of cross girder = 6 KN/m
Self weight of main girder = 15.66 KN/m
Reaction of main girder = 6 x 3 = 18 KN
Total dead load on girder = (33.75 + 15.66) = 49.41 KN/m

Fig 5.5: Position of dead load on the girder

Mmax = $((49.41 \times 40^2)/8) + (15 \times 40)/4 + (15 \times 15) + (15 \times 10) + (15 \times 5)$
= 10482KN-m
Dead load shear at support
Vmax = (49.41 x 40)/2 + (15 x 7)/2 = 1040.7 KN

e) Live load bending moments in girder
Span of the girder = 40m
Impact factor class AA = 10%
Live load is placed centrally on the span as shown in fig



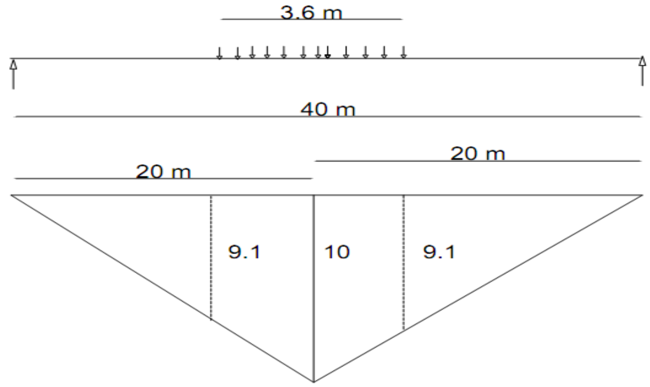


Fig 5.6 Bending moment diagram for moving loads

Bending moment at centre of span
= ((10 + 9.1)/2) x700
= 6685 KN-m
B.M including impact and reaction factors for outer girder i.e
Live load BM = (6685 x 1.1 x 0.711) = 5228.3 KN-m
B.M for inner girder = (6685 x 1.1 x 0.404) = 2970 KN-m

f) Live load shear forces in girders

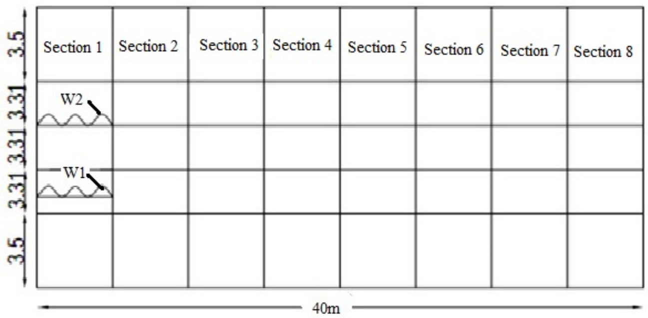


Fig 5.7: Plan showing the load location on the girder

g) Properties of main girder:-

Area = [(0.8 x 0.3) x 2] + [4 x ½ x 0.15 x 0.25] + [1.1 x 0.3] + [2 x 0.15 x 0.3]

A = $0.975m^2 = 97.5 \times 10^4 mm^2$

$y_b = y_t = 2000/2 = 1000mm$

Moment of inertia =

5.1.7 CHECK FOR ULTIMATE FLEXURAL STRENGTH:-

For the centre of the span section Ap= 4900 mm^2

Breath of flange b_f =800 mm

Breath of web b_w= 300 mm

Depth d = 2000-200= 1800 mm

f_{ck}= 40 N/mm^2

f_p= 2000 N/mm^2

According to IRC 18-2000

(i) Failure by yielding of steel

$M_u = 0.9xdxApxf_p$

$= 0.9x1800x4900x2000$

=15876KN-m

(ii) Failure by crushing of concrete:-

$M_u = 0.176xb_wxd^2xf_{ck} + (2/3)x0.8x(b-b_w)(d-(D_f/2))x\ D_{fx}\ f_{ck}$

$= (0.176x300x1800^2x45) + 2/3x0.8x(800-300)x(1800-(300/2))x(300x45)$

$= 6.842x10^9 + (5.28x10^9)$

$M_u = 12122x10^6$ KN-m

According to IS 1343-1980 the ultimate flexural strength of centre span section is computed as follows.

$A_p = (A_{pw} + A_{pf})$

$A_{pf} = 0.45\ f_{ck}\ x(b- b_w)(D_f/f_p)$

= 0.45x45(800-300)x(300/2000) = 1350 mm^2

$A_{pw} = A_p - A_{pf}$

= 4900 – 1350 = 3550 mm^2

Ratio $((A_{pw}\ x\ f_p)/(\ b_w\ xbx\ f_{ck}))$ = ((3550x2000)/(300x1800x45)) = 0.328

From table-11 of IS: 1343, we have post tensioned beams with effective bond.

$(f_{pu})/(\ 0.87x\ f_p) = 0.822$

$f_{pu} = 0.822x0.87x2000 = 1430.28$

$x_u/d = 0.5846$

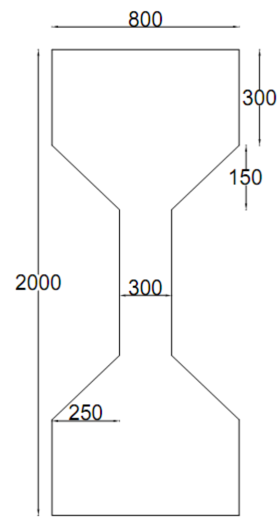
$x_u = 0.58x1800 = 818$ mm

$M_u = [f_{pu}x\ A_{pw}\ x\ (d-(0.42\ x_u)) + 0.45\ x\ f_{ck}x(b-b_w)x(D_f(d-(0.5\ D_f)))$

= 1430.28 x 3550 x [1800-(0.42x818)] + [0.45x45x(800-300)x300(1800-(300/2))]

$= 7.39x10^9 + (4.455x10^9)$

5.1.8 SUPPLEMENTARY REINFORCEMENT:-



Longitudinal reinforcements of not less than 0.15 % of gross cross sectional area provided to safeguard against shrinkage cracking.

$Ast = 0.15/100 \times 0.975 \times 10^6$

$= 1462.5\ mm^2$

20 mm Φ are provided and distributed in the compression flange.

Design of end block :-

$2y_{po}$ = 225 mm

$2y_o$ = 900 mm

The ratio $y_{po} / y_o = 225/900 = 0.25$

Bursting tension is computed using table recommended in IRC: 18-2000

F_{bst}/p_k =0.23

F_{bst} = 0.23 x 1459 = 335.57 KN

= 336 KN

Area of steel to resist this tension is obtained as

$As= (336 \times 10^3)/(0.87 \times 415)$

$= 931\ mm^2$

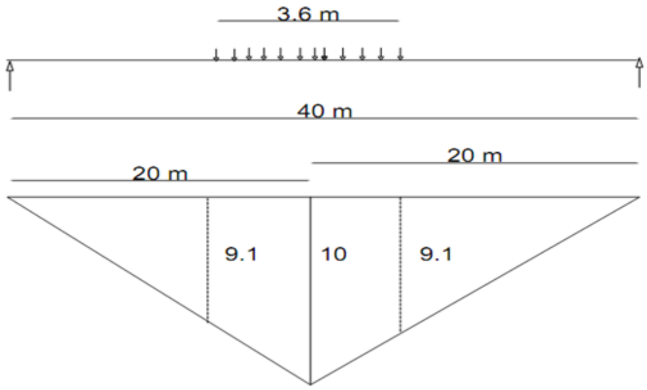


Fig 5.9:Loading on I-girder

VI. RESULTS AND DISCUSSIONS

6.1 INTRODUCTION:

The analysis results from the STAAD.PRO presented in the following tables illustrates the maximum bending moment and shear forces occurred at the sections considered in the geometric modelling of box girder and I-girder bridges. The sections are shown in fig 5.7.And some of the figures are presented to show the geometric view and stress concentration in the analysis part

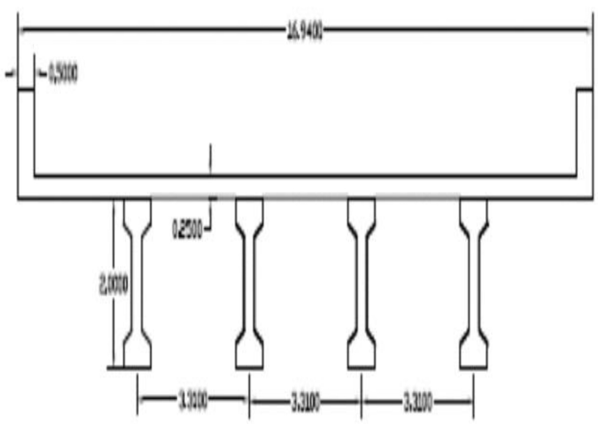


Fig 5.8: Cross section of I-girder

6.1(a): Moments and shear forces in the box girder:

The following tables 6.1 and 6.2 shows the maximum and minimum bending moment and shear forces in the box girder.

Table 6.1 Moments and shear force in the box girder for D.L, L.L and P.F Load 8	0.1213399	-0.0810862	0.172758	-0.108622

From the above tables 6.8 and 6.9 it was observed that maximum shear stress occurred at the supports in box girder and I-girder bridges.

6.5 Cost Estimation:

From the estimation and costing of the box girder and I-girder bridges shown in Annexure-III. It was observed that the cost of I-girder bridge is more when compared to box girder bridge.

6.6 FIGURES FROM STAAD PRO

The geometric modelling of the box girder is shown in fig 6.1.bending stress distribution is shown in fig 6.2 and maximum stress concentration identified at the mid span of the box girder. Shear stress distribution due to load combinations in box girder is shown in the fig 6.3 and maximum stress concentration identified at the supports.3D rendered view of the box girder is shown in fig 6.4. Bending Stress distribution due to live load is represented in the I-girder is shown fig 6.5 and maxitmum concentration identified at the mid span. Shear Stress distribution due to live load is represented in the I-girder is shown fig 6.6 and maxitmum concentration identified at the supports.

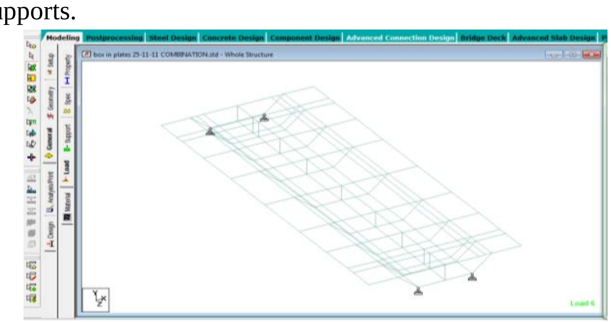


Fig 6.1: Geometric Modelling of the Box girder

VII. CONCLUSIONS

After studying the various parameters of pre-stressed concrete Box Girder Bridge and I-Girder Bridges (keeping span and width as same) and cost analysis of the both bridges the following conclusions were drawn.

1. Pre-stressed Box Girder Bridge have exceptionally good Torsional rigidity in better transverse load distribution.
2. There will be increase in head room for undergoing vehicles.
3. The cost of the box girder bridge and I-girder bridge are estimated and compared it is concluded that the cost of the I-Girder Bridge is higher than cost of the Box Girder Bridge.
4. Quantities of pre-tension cables required for I-Girder Bridge is more than that of Box Girder Bridge.
5. Maximum Stress distribution and prestressing force is maximum in the box girder bridge when compared to I-girder bridge.
6. Deformations are maximum in the I-girder bridge compared to box girder bridge.
7. Concrete area required for Box Girder is less than that of I-Girder.
8. For speedy construction I-Girder Bridge is preferable when compared to Box Girder Bridge.

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4. DI, HU; LAIFA, WANG; JIPING, GUO[4] (2008) calculation of short-term and long-term behavior of pre-stressed concrete box girders considering effect of shear lag. Engineering materials, v 400-402, p 943-948, 2009; issn: 10139826; doi:

ANNEXURE-I

STAAD INPUT FILE FOR BOX GIRDER ANALYSIS

STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 23-Nov-11
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 0 1.885 0; 3 8.47 1.885 0; 4 5.2225 1.885 0; 5 4.4925 1.885 0;
6 3.7425 1.885 0; 7 3.0025 0 0; 8 0 0 -3.03; 9 0 1.885 -3.03;

ANNEXURE-II

STAAD INPUT FILE FOR I-GIRDER ANALYSIS

STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 06-Dec-11
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
2 1.66 0 0; 3 4.97 0 0; 4 8.47 0 0; 6 1.66 0 -5; 7 4.97 0 -5; 8 8.47 0 -5;
10 1.66 0 -10; 11 4.97 0 -10; 12 8.47 0 -10; 13 -1.66 0 0; 14 -4.97 0 0;
15 -8.47 0 0; 16 -1.66 0 -5; 17 -4.97 0 -5; 18 -8.47 0 -5; 19 -1.66 0 -10;
20 -4.97 0 -10; 21 -8.47 0 -10; 23 1.66 0 -15; 24 4.97 0 -15; 25 8.47 0 -15;
26 -1.66 0 -15; 27 -4.97 0 -15; 28 -8.47 0 -15; 30 1.66 0 -20; 31 4.97 0 -20;
32 8.47 0 -20; 33 -1.66 0 -20; 34 -4.97 0 -20; 35 -8.47 0 -20; 37 1.66 0 -25;
38 4.97 0 -25; 39 8.47 0 -25; 40 -1.66 0 -25; 41 -4.97 0 -25; 42 -8.47 0 -25;
44 1.66 0 -30; 45 4.97 0 -30; 46 8.47 0 -30; 47 -1.66 0 -30; 48 -4.97 0 -30

	a) Cross girders 1. 12mm longitudinal bars 2. Stirrups 10mm dia	4 x 8 8 x 99	9.39 3.7			300.48 2930.4
	b) Deck slab portion 1. Main dia bars 12mm 2. Distribution bars 10mm dia	1 x 333	17.224 42.88			5735.6 4802.56

		1 x 112				
	c) Barrier 1. Main bars 12mm ø 2. Distribution reinforcement 8mm ø 150mm c/c	2 x 200 12 x 2	2.43 42.8			972 1029.12

Abstract of steel bars:-

1) 20mm ø steel bars total length 2744.32 x 2.47 kg/m = 6852.57 kg
2) 12 mm ø steel bars total length = 14914.24 x 0.89 kg/m = 13273.676 kg
3) 10 mm ø steel bars total length = 7732.96 x 0.62 kg/m = 4794.435 kg
4) 8 mm ø steel bars total length = 1029.12 x 0.396 kg/m = 412 kg

APPROXIMATE ESTIMATION AND COST OF PRESTRESSED CONCRETE BOX -GIRDER BRIDGE

S.NO	Description	Quantity	Rate	Unit	Rate
1	Pre-stressed concrete grade M_{45}	369.724	5700	1CM	21,07,426
2	Wearing coat	35.85	3560	1CM	1,27,626
3	Cable wires BBRV 19K15 cable	12.13T	101640	1MT	12,29,844
4	HYSD bars	18.37T	46000	1MT	8,45,020

Total = 43,09,916

APPROXIMATE ESTIMATION AND COST OF PRESTRESSED CONCRETE I-GIRDER BRIDGE

S.NO	Description	Quantity m^3	Rate	Unit	Rate
1	Pre-stressed concrete grade M_{45}	408	5700	1CM	23,25,600
2	Wearing coat	35.85	3560	1CM	1,27,647
3	Cable wires BBRV 19K15 cable	17.33T	101640	1MT	17,61,546
4	HYSD bars	25.33T	46000	1MT	11,65,303

Total = 53,80,096