Design of Post Tensioned and Prestressed Concrete Box Girder-Slab for a Bridge Deck

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Abstract— Bridge construction today has achieved a worldwide level of importance. Bridges are the key elements in any road network and use of pre-stress girder type bridges gaining popularity in bridge engineering fraternity because of its better stability, serviceability, economy, aesthetic appearance and structural efficiency. I & T-beam bridges are one of the most commonly used types of bridge and it is necessary to constantly study, update analysis techniques and design methodology. Structurally they are simple to construct. Hence they are preferred over other types of bridges when it comes to connecting between short distances. The report examines in detail the application of segmental precast Design of Post Tensioned and Prestressed Concrete Box Girder-Slab for a Bridge Deck in achieving long spans in bridge structures. Numerous examples from throughout the world indicate that such construction can provide an effective means of achieving long span in the range of 100 to 500 ft. Used segmental pre-cast bridge structure member is manufactured in a number of short units which during erection are joined together, end to end, and post-tensioned to form the completed superstructure. Adherence to these simple rules will not prevent all damage in moderate or large earthquakes, but life threatening collapses should be prevented, and damage limited to repairable proportions. Considering the time variant, I have been focused on a box girder of span 40 meters and analyzing it with respect to pre-tensioning and post-tensioning and incorporating the graphs for the various loads and deflections. In this case designing is done using STAAD- Pro software. Software will be used for 3D design model of 40m span Box Girder-Slab for a Bridge Deck & will be tested in order to study its behaviour during the construction process and under permanent loads. Shear force, Bending moment, axial force, base shear, are studied with accordance to earthquake response effects. The analysis results were compared with analytical predictions obtained by means of a prototype model developed for the nonlinear and time-dependent analysis of segmental erected, reinforced and prestressed concrete structures. Generally good agreement was obtained, showing the adequacy of the model to reproduce the structural design of the different elements in the post-tensioned pre-stressed bridge structure.

Keywords: PSC Bridge; Pre-Stressed Concrete Segment; Box Deck Design, I-Beam Deck; Post Tensioned Box Girder; Post-Tensioned Deck Slab

I. INTRODUCTION

The main difference between reinforced concrete and prestressed concrete is the fact that reinforced concrete combines concrete and steel bars by simply putting them together and letting them to act together as they may wish. Prestressed concrete, on the other hand, combines high strength concrete with high strength steel in an "active" manner. This is achieved by tensioning the steel and holding it against the concrete, thus putting concrete into compression. This active combination results in a much better behaviour of two materials. Steel is ductile and now is made to act in high tension by prestressing. And concrete is a brittle material with its tensile capacity now improved by being compressed, while its compressive capacity is not really harmed. Thus prestressed concrete is an ideal combination of two modern high strength materials.

A. *Bridge:*

Bridge is a structure built across an obstruction to easily pass over it. Obstruction may be a river, stream, canal, valley, ditch, highway, railway, etc. from ancient times bridge has played an important role in development of a place by improving its connectivity with other places thus improving trade and economy. There are various types of bridges. But these days for construction long span major bridges PSC bridges are ideally suited. Prestressed concrete bridges mostly came into used because of their rapidity, ease of construction, and competing in costs with other alternatives such as steel and reinforced concrete. Ganga bridge in Patna is an example for major PSC bridge in India.

II. LITERATURE REVIEW

The development of prestressed concrete can be studied in the perspective of traditional building materials. In the ancient period, stones and bricks were extensively used. These materials are strong in compression, but weak in tension. For tension, bamboos and coir ropes were used in bridges. Subsequently iron and steel bars were used to resist tension. These members tend to buckle under compression. Wood and structural steel members were effective both in tension and compression. In reinforced concrete, concrete and steel are combined such that concrete resists compression and steel resists tension. This is a passive combination of the two materials. In prestressed concrete high strength concrete and high strength steel are combined such that the full section is effective in resisting tension and compression. This is an active combination of the two materials. The following sketch shows the use of the different materials with the progress of time.

III. RESEARCH METHODOLOGY

A. *Modelling Procedure:*

The Deck slab is designed for IRC loading with live load at different positions on the deck. The Bending moment and shear force are calculated by using Pigeaud's curve. The load from deck slab is transferred to the main girders by Courbon's method. The live load bending moment and shear force are calculated and the girder is designed for pre-stressed concrete girder.

The finite element 3D modeling is done in STAAD Pro with dead load and live load applied and the final stresses, principles, deflection, etc are tabulated.

Segmental construction employs precast segments cast of highest quality concrete, in sizes which can be transported and erected. The segments are usually reinforced with mild steel and are designed to be connected by post-tensioning after erection.

The created model is intended to represent the reference bridge that is to be built and therefore it is of high importance that the geometry of the bridge model corresponds with the reality. A geometric axis to determine the orientation of the bridge and the location of every component such as pier, abutment and bearings is first created. The placement of the piers has been arranged in terms that all spans have equal length.

Material properties for the bridge have been defined from the design code in Staad and material are chosen appropriately for the different parts of the bridge. The cross sections for the superstructure, piers and abutments are modeled separately in AutoCAD and subsequently assigned with the corresponding material. The cross sections for the pier and foundations are modeled as the drawings for the reference bridge. The boundary conditions for the bearings are defined for each bearing along the bridge axis.

The self-weight was introduced to the structure and loads were applied. The loads applied to the structure are defined and applied respectively. The loads applied are defined further in section.

B. *Steps to Be Followed:*

Present thesis work is being done in following steps:

1) Modelling of bridge deck with the three types of girder for central zone of India.
2) Modelling has to be done in STAAD-PRO.
3) Comparison of result.
4) Study for optimum design.
5) Study and comparison of cross section in terms of base shear, moment at base, size of cross-section on the basis of design given by software.

For thesis work some of the constraints are being kept constant so that comparison of the three types of bridge girder can be carried out with an unbiased method. For the same, some of the design constant are kept constant and are as follows:

1) List of properties of materials used such as grade of concrete, high tensile steel, untensioned steel. Usually, concrete of grade M40 is used for post tensioned girders.
2) Assume preliminary dimension, based on experience. The overall depth is usually about 75 to 85 mm for every meter of span. The thickness of deck slab is about 150 to 200 mm with transverse prestressing and about 200 to 250 mm in composite construction. The minimum thickness of web of precast girder is 150mm plus diameter of the cable duct. The bottom width of precast beam may vary from 500 to 800 mm.
3) Compute section properties. It is permissible to compute this based on the full section without deducting cable ducts.
4) Compute dead load moments and stresses for girders.
5) Calculate Live load moments and stresses for girders for the severest applicable condition of loading according to the code specified by IRC 6-2000. Any rational method may be used for load distribution among the girders.
6) Determine the Magnitude and Location of the prestressing force at the point of maximum moment. The prestressing force must meet two conditions.

a) It must provide sufficient compressive stress to offset the tensile stresses which will be caused by the bending moments. b. It must not induce either tensile or compressive stresses which are in excess of those permitted by the specification (IRC 18-2000).

7) Select the prestressing tendons to be used and work out the details of their locations in the member. Avoid grouping of tendons, as grouting of some after first stage of prestressing may lead to leakage of grout into remaining as yet ungrouted ducts if sheaths are not leak proof. Establish the concrete strength at the time of prestressing and check stresses under the initial and final prestress conditions.
8) Determine the profile of the tendons, and check the stresses at critical point along the member under initial and final condition. The profile of the tendons may be parabolic for the full length from one anchorage to other. Alternatively, a tendon may have a central straight portion with parabolic profile at either end. That draping of the tendons helps to bring the stresses in concrete at every section within the permissible range. This also serves to reduce the net shear at any section.
9) Check the ultimate strength to ensure that the requirements of IRC 18-2000 are met. The ultimate load is to be taken as 1.25G + 2SG + 2.5Q where G, SG and Q denote permanent load, super imposed load and live load including impact respectively.
10) Work out the shear stresses at different sections and design the shear reinforcement.
11) Check the stresses at jacking end due to cable forces and these should be within permissible limits.
12) 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.

C. *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.

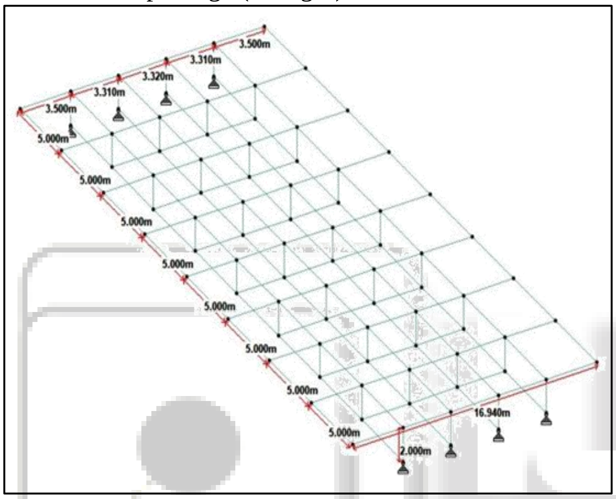
D. *Deck Slab:*

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

Box Girder Design specification:

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



A. Materials:

Concrete: Grade of concrete = M45
Density of concrete = 24.525 KN/m^3
Un tensioned 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
19 represents the maximum number of strands in a prestressing cable and 15 represents the diameter of each strand or tendons with 19 strands

B. Material Properties:

Modulus of elasticity of concrete Ec = 5000√fck
= 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 Es = 1.95x105 N/mm^2
Modular ratio m = Es/Ec = 5.81

C. Permissible Stresses: (Cl: 7 of IRC 18-2000)

1) Concrete:

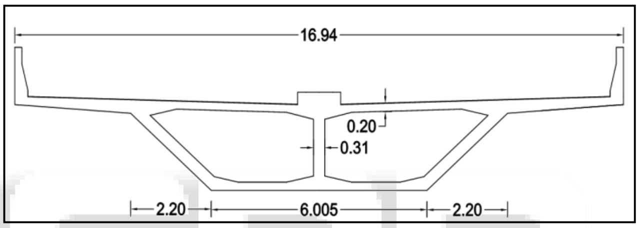
1) Temporary stress in extreme fibre (Cl:7.1 of IRC 18-2000)

Permissible compressive stress = 0.5 x fck = 0.5 x 45 = 22.5 N/mm^2
Permissible tensile stress = 0.1 x σc = 0.1 x 22.5 = 2.25 N/mm^2
Minimum strength of concrete at the time of stressing = 40.5 N/mm^2

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

Permissible compressive stress in service =0.33fck = 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

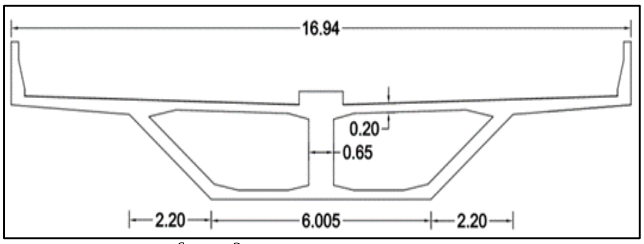
D. Sectional Properties: (At Midspan)



A = [(0.2 x 3.25) x 2] + [2 x ½ x (0.1 x 3.25)] + [2.9 x 0.310 x 2]+[4 x 0.15 x 0.75 x 0.5 x 2]+
[0.506 x 0.3 x 2]+[4.5615 x 0.2 x 2]+[2.875 x 2 x 0.2]+[2.3 x 0.310]+[2 x ½ x 0.9525x 0.1]
A = 7.944 = 7.944 X 10^6 mm^2

$$Y_b = \frac{[(0.2 \times 3.25 \times 2 \times 2.2)] + [2.1/2 \times 0.1 \times 3.25 \times 2.066] + [0.5.6 \times 0.3 \times 2 \times 2.15] + 4.5615 \times 0.2 \times 2.2] + [4 \times ½ \times 0.75 \times 0.15 \times 2.05] + [4 \times ½ \times 0.75 \times 0.15 \times 0.25] + [0.310 \times 2.3 \times 2.3/2] - [1/2 \times 2.22 \times (2/3 \times 2+08) \times 2] - [1/2 \times 2.22 \times 2 \times (1/3 \times 2 \times 0.300)] +[0.2 \times 6.005 \times 0.2/2]}{7.944 \times 10^6}$$

Y_b= 1431 mm
Y_t = 2.3-1.431 = 0.87mm
Moment of inertia I = [(3.4275 x 0.2^3/12) + (3.2475 x 0.2 x 1.71^2)] + [(3.2475 x 0.1^3/36) + (3.2475 x 0.1 x 1.576^2) x 2] + [(0.2^3 x 10.445)] + 2x[(2.22 x 1.650^3/12) + (2.22 x 1.650 x 0.335^2)] 2x[(2.220 x 1.650^3/36)+(1.11 x 1.650 x 0.06^2)+(2.22 x 1.650^3/36)+(1.11 x 1.650 x 0.61^2)]+ [(6 x 0.2^3/12)+(6 x 0.2 x 0.390^2)]+[(0.310 x 1.685^3/12)+(0.310 x 1.685 x 0.352^2)]
I = 5.074x10^{12} mm^4
Section modules for bottom most fibers Z_b= 5.074x10^{12}/1431 = 3.54x10^9 mm^3
Section modules for top most fibers Z_t = 5.074x10^{12}/870 = 5.832x10^9 mm^3
Distance of bottom kern point from N.A K_b = Z_t/A = (5.832 x 10^9/ 7.944 x 10^6) = 728 mm
Distance of top kern point from N.A K_t = Z_b/A = (3.54 x 10^9/ 7.944 x 10^6) = 442.5 mm
Radius of gyration r =√(I/A) = 796.40 mm
(AT SUPPORT)



Area = 9.35×10^6 mm^2
Moment of inertia I = 5.56×10^{12} mm^4
Y_b= 1380 mm, Y_b= 920 mm
Section modulus for bottom most fibre Z_b= $5.56 \times 10^{12}/1380$ = 4.028×10^9 mm^3
Section modulus for top most fibre Z_t = $(5.56 \times 10^{12})/920$ = 6.043×10^9 mm^3
Distance of bottom kern point from N.A $K_b = Z_t/A$ = $(6.043 \times 10^9/ 9.35 \times 10^6)$ = 645 mm
Distance of top kern point from N.A $K_t = Z_b/A$ = $(4.028 \times 10^9/ 9.35 \times 10^6)$ = 430 mm
Radius of gyration r =√(I/A) = 594 mm
Permissible stress as per IRC 18-2000
Characteristic compressive strength of concrete at 28 days = F_{ck} = 45 N/mm^2
F_{cj} =0.8 F_{ck} = 0.8 x 45 = 36 N/mm^2
Allowable comp stress at transfer F_{ci}= 0.4 F_{cj} = 16.2N/mm^2
Allowable tensile stress at transfer F_{ci}= -0.1 F_{ci} = -1.62 N/mm^2
Allowable stress at service = 0.33 F_{ck} = 0.33 x 45 = 14.88 N/mm^2
No tensile stress shall be permitted at service

E. Design Loads:

Unit wt of concrete = 25 KN/m^3
Self wt girder alone is considered as dead load
Self wt of crash barrier, kerb walls, median and wearing coat are considered as superimposed dead load.
IRC- 70R, loading is considered for live load.
1) Self wt of girder = $(8 \times 10^6 \times 40 \times 25)/10^6$ = 8000 KN
2) Total superimposed dead load

Wearing coat = 2 KN/m^2
Crash barrier:
Area = 0.351 m^2
Intensity of load = 0.351 x 23.544 = 8.264 KN/m
Total superimposed dead load = 1160 KN

F. Seismic Force:

As per cause 22.2 in IRC-6-1966 the vertical seismic coefficient shall be considered in the case of structure built in Zones IV.
The vertical seismic coefficient applicable may be taken as half of the horizontal seismic coefficient.
Horizontal seismic force: $f_{eq} = \alpha \times \beta \times \lambda \times G$
α = horizontal seismic coefficient depending on location = 0.05 (table 1)
β = A coefficient depending upon the soil foundation system = 1.2 (table 2)
λ = A coefficient depending upon the importance of bridge =1.5
G = 0.5 (As per combination IX)
F_{eq} = 0.05 X 1.2 X1.5 X 0.5 = 0.045 KN
Vertical seismic force = ½ f_{eq} = 1/2 x 0.045= 0.0225KN

Consider halt of live load due to perpendicular to the traffic are = 16.14/2 = 8.07KN
Total load at the centre = 0.0225 + 8.07 = 8.0925 KN
Total max B.M at centre = (8.0925 x 40) / 4 = 81 KN-m

G. Design Moments:

Bending moment due to self wt= M_d = (8000 x 40)/8 = 40000 KN-m
B.M due to super imposed dead load = (1160 x 40)/8 = 5800 KN-m
BM due to LL with impact factor = 1.25 x (96 x 40) = 4800 KN-m
B.M due to Seismic force= 81 KN-M
Total BM at transfer = 50600 KN-m

H. Calculation of Pre-Stressing Force and Final Stress:

The basic value of permeable stress top and bottom as per considered for arriving to the pre stressing force as follows
At transfer
Stress at top : $(P_i/A) - (P_i \times e/Z_t) + (M_d/Z_t) \leq -1.62$ N/mm^2
Stress at bottom $(P_i/A) + (P_i \times e/Z_b) - (M_d/Z_b) \leq 16.2$ N/mm^2
At service (considering 20% loss of pre- stress)
Stress at top $0.8 \times (P_i/A) - 0.8(P_i \times e/Z_t) + (M_d/Z_t) \leq 14.85$ N/mm^2
Stress at bottom $0.8 \times (P_i/A) + 0.8 \times (P_i \times e/Z_b) - (M_d/Z_b) \leq 0$ N/mm^2
Where P_i = Initial Pre stressing force, e = eccentricity
$(P_i/8 \times 10^6) - (P_i \times e/5.832 \times 10^9) + ((40 \times 10^9)/(5.832 \times 10^9)) \leq -1.62$ N/mm^2
$(P_i/8 \times 10^6) + (P_i \times e/3.54 \times 10^9) - ((40 \times 10^9)/(3.54 \times 10^9)) \leq 16.2$ N/mm^2
$0.8(P_i/8 \times 10^6) - 0.8(P_i \times e/5.832 \times 10^9) + ((50.6 \times 10^9)/(5.832 \times 10^9)) \leq 14.85$ N/mm^2
$0.8(P_i/8 \times 10^6) + 0.8(P_i \times e/3.54 \times 10^9) - ((50.6 \times 10^9)/(3.54 \times 10^9)) \leq 0$ N/mm^2
Initial pre stressing force= 51250 KN
Prestressing force (19K15) in each cable = 3660 KN
Sum of depth of all cables from bottom / no of cables = 0.1288
Eccentricity e = 1.4317-0.128 = 1.3037 m
At mid span
Stress at top = $(51.2 \times 10^6/8 \times 10^6) - (51.2 \times 10^6/5.832 \times 10^9) + (40 \times 10^9/5.832 \times 10^9)$
= 1.808 > -1.62 (tension)
Stress at bottom = $(51.2 \times 10^6/8 \times 10^6) + (51.2 \times 10^6/3.54 \times 10^9) - (40 \times 10^9/3.54 \times 10^9)$
= 13.96 < 16.2 (compression)
Stress at top = $(0.8 \times 51.2 \times 10^6/8 \times 10^6) - (0.8 \times 51.2 \times 10^6/5.832 \times 10^9) + (50.6 \times 10^9/5.832 \times 10^9)$
= 4.636 < 14.85(compression)
Stress at bottom = $(0.8 \times 51.2 \times 10^6/8 \times 10^6) + (0.8 \times 51.2 \times 10^6/3.54 \times 10^9) - (50.6 \times 10^9/3.54 \times 10^9)$
= 11.532 > 0 (compression)
No tension is allowed; Hence ok
Total area of cables:

CABLE NO.	ANCHORAGE TYPE	OUTER DIAMETER	AREA OF EACH STRAND	NO.OF CABLES	TOTAL AREA
1	19K15	107mm (O.W)	140	02	4480
2	19K15	107mm (O.W)	140	02	5320
3	19K15	O.W	140	02	5320
4	19K15	O.W	140	02	5320
5	19	I.W	140	01	2240
6	19	I.W	140	01	2660
7	19	I.W	140	01	2660
8	19	I.W	140	01	2660
9	19	Soffit slab	140	01	2660
10	19	Soffit slab	140	01	2660

Total no of all cables=35980mm^2
Total ultimate strength of cables =1862 × 35980/1000
=> 66994.76KN
Sum of depth of all cables/no of cables = 33.8/280 = 0.13
e =1.431 - 0.13 = 1.301
At support x = 0.
Sum of depth of all cables /no of cables = 0.9473
Eccentricity = 1.431-0.9473 = 0.4837

I. *Losses of Prestress:*

Losses of pre stress are calculated as per clause 11-IRC-18-2000

1) *Loss of Prestress Due to Friction*

Between cable and sheathing stress stress in cable "x" meters from jack end
$\sigma_{po}= \sigma_{po}(x)\times e^{(\mu\theta \times kx)}$
σ_{po}= the steel of Naperian logerithms(in net) =>2.7183
μ = the coefficient of friction
θ = the cumulative angle in radians through Which the tangent to the cable profile has turned b/w the points of operation of σ_{po} and σ_{po} (x)
$\sigma_{po}(x)$ = the steel stress at a point distance x from the Jacking end
K = the wobble coefficient per meter length of steel
X = the distance between points of operation of σ_{po} and $\sigma_{po}(x)$In m
For corrugate HDPE K= 0.0020; μ = 0.17
$\sigma_{po}(19.55) = 2.713$ ^ $(0.17\Theta + 0.002x19.55)$
σ_{po}= 1862 x 0.765x2.7138 ^ ((0.17 x 0.664 + (0.002 x 19.55))
Maximum jacking pressure=1481.42 N/mm^2

a) Loss of Pre-Stress Due to Slip of Anchorage:

If $\frac{\delta x}{dx} = \frac{Fpo}{AE}$.

$$\delta x = \frac{Fpo(x)dx}{AE}$$

$$\Sigma\delta x = \frac{\Sigma Fpo(x)dx}{Ae}$$

$\Sigma\delta x = should\ not\ exceeds\ 7mm$
A=cross section area of cables.
E=modulus elasticity of cable = 2×10^5 N/mm^2
Internal dia of the dust=107mm
External dia the dust=112mm
Minimum clear cover to dust from outer face =75mm
C.G. of cable from surface =75 + 112/2 =.131mm
c/c spacing between cables = 350 × 350mm
c/c distance between anchor blocks =395 mm

Stress in prestressing force = (55385 x 1000)/35980
=1539.32 N/mm^2
Percentage loss = (64.293 x 100)/1539.32 = 4.17%
Elongation of Jack at each end:-
Length of Jack L=760mm
Stress at jack end=1424.43mm
Elongation of cable within the Jack portion = stress x L/E
= 1424.43 x 760/2 x 10^5
= 5.41mm
Extra length at stressing end required for griping purpose=1000mm
Extra length required at non stressing end =1350mm

b) Loss of Prestress Due to Shrinkage of Concrete:

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
$\mathcal{E}_s = 1.9 \times 10^{-4}$
Loss of prestress due to shrinkage of concrete = $\mathcal{E}_{cs} \times E_s$ = 1.9 x 10^{-4} x 2 x 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= 4×10^{-4}
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 = √(M/Rb) = √(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 = $\sqrt{M_u / Qb} = \sqrt{(81.845 \times 10^6)/(1.843 \times 1000)}$ = 210 mm.
Over all depth = 210 + 40 = 250
A_{st} Required = M/ σ_{st} x j x d = $(81.845 \times 10^6) / (200 \times 0.878 \times 210) = 2219\ mm^2$
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 = $(17.40 \times 10^6) / (200 \times 0.878 \times 200) = 495.44\ mm^2$
Min reinforcement provide A_{st} = $(0.18 / 100) \times 1000 \times 240 = 432\ mm^2$
Provide 10 mm HYSD bars = (78.5/495) x 1000 = 150 mm c/c.

V. RESULTS & DISCUSSIONS

The analysis results from the STAAD.PRO presented in the following tables illustrate the maximum bending moment and shear forces occurred at the sections considered in the geometric modeling of box girder. 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.

A. *Moments and shear forces in the box girder:*

The following tables 5.1 and 5.2 shows the maximum and minimum bending moment and shear forces in the box girder.

section	Moments in X-direction		Moments in Y-direction		Shear force in X-direction		Shear force in Y-direction	
	Max	Min	Max	Min	Max	Min	Max	Min
1	63.79	-67.391	55.1906	-67.384	1.45784	-3.0068	6.9017	-6.4148
2	64.8921	-70.56	64.8921	-70.56	1.12342	-3.2239	6.8445	-7.3567
3	88.195	-93.0269	70.2136	-72.307	1.15818	-4.0091	4.1458	-8.3744
4	98.202	-90.2261	71.2459	-72.774	1.20065	-4.4646	3.2694	-8.5495
5	95.6639	-86.4736	67.9847	-72.490	1.20049	-4.6593	5.9463	-8.3964
6	91.7719	-82.948	63.0382	-74.906	1.14658	-4.4508	5.0296	-7.6494
7	145.257	-144.569	55.2934	-55.974	1.43383	-3.9492	3.8522	-6.6747
8	142.17	-82.6654	49.9873	-58.197	1.21512	-3.3802	3.7120	-6.2463

From the above tables 5.1 it was observed that there was a maximum bending moment occurring in the direction of X-axis at a distance 18 meters from the support. And maximum bending moment in the direction of Y-axis occurring at centre of the box girder. The Maximum shear force in the box girder occurred at the support.

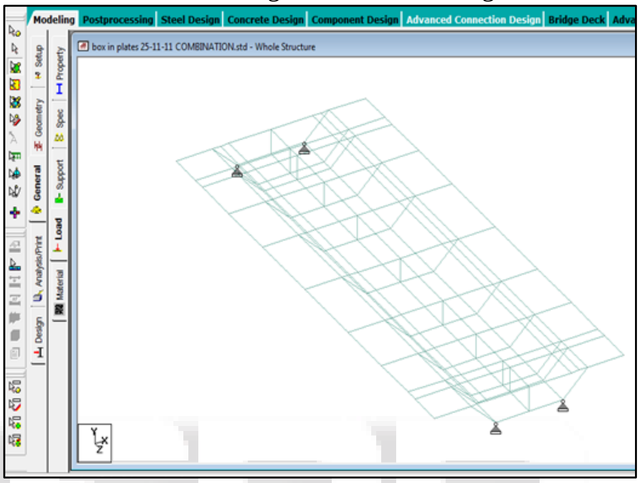
B. *Deformations:*

Section	Deformations (mm)	
	X-direction	Y-direction
1	0.327	0.225
2	0.225	3.143
3	0.184	3.825
4	0.270	4.633
5	0.334	4.545
6	0.391	3.137
7	0.218	4.854
8	0.347	2.891

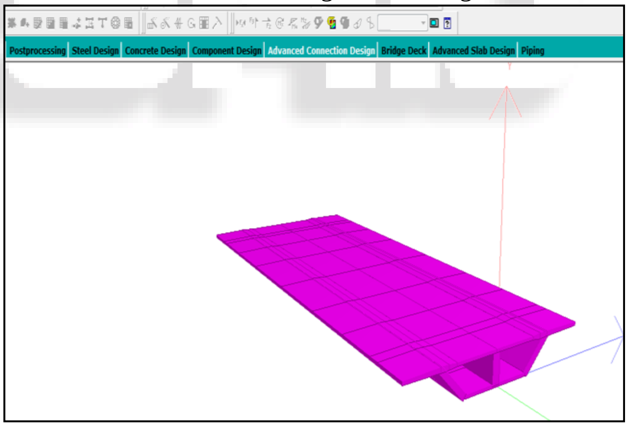
Table 5.2: Deformations in the box girder.

C. *Modelling from STAAD Pro*

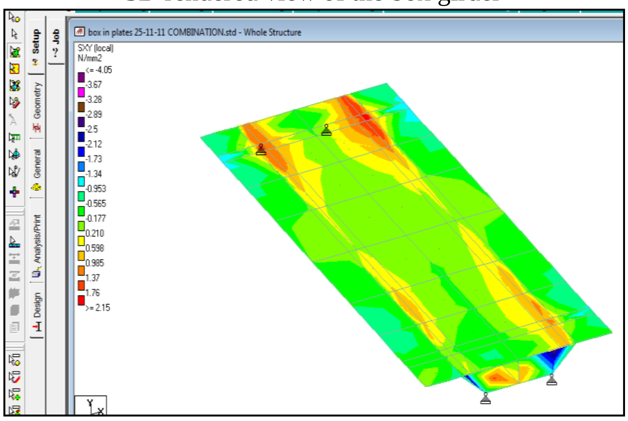
The geometric modeling of the box girder is shown in fig 5.1.bending stress distribution is shown in fig 5.2 and maximum stress concentration identified at the mid span of the box girder. Shear stress distribution due to load combinations in box girder is shown in the fig 5.3 and maximum stress concentration identified at the supports.3D rendered view of the box girder is shown in fig 5.4.



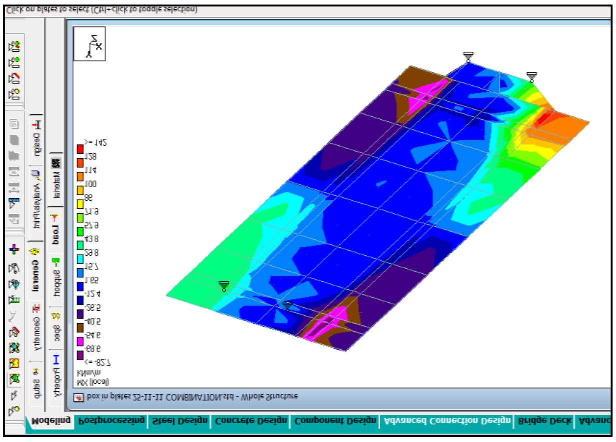
Geometric Modelling of the Box girder



3D rendered view of the box girder



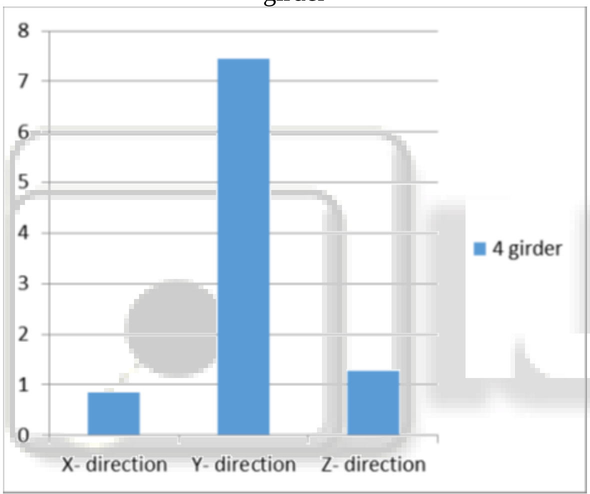
Bending stress distribution due to load combination in the box girder



Shear stress distribution due to load combinations in the box girder bridge

Type of girder	X-direction	Y- direction	Z- direction
Girder No.4	0.985	7.236	1.37

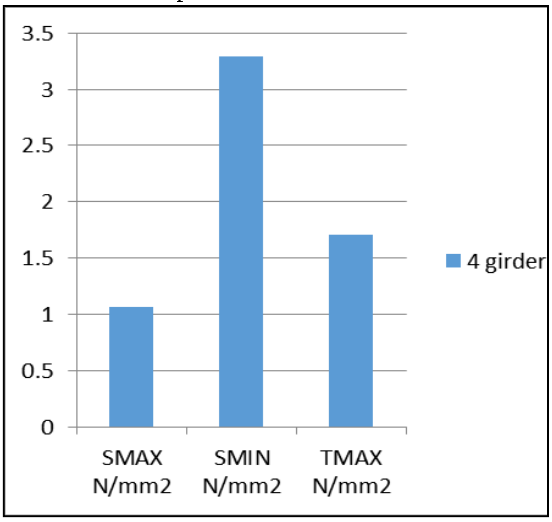
Table 1: Maximum Deflection in bridge in mm Type of girder



Maximum Deflection in bridge in mm

Type of girder	S_{max} N/mm^2	S_{min} N/mm^2	□$_{max}$ N/mm^2
Girder No.4	1.089	3.258	1.64

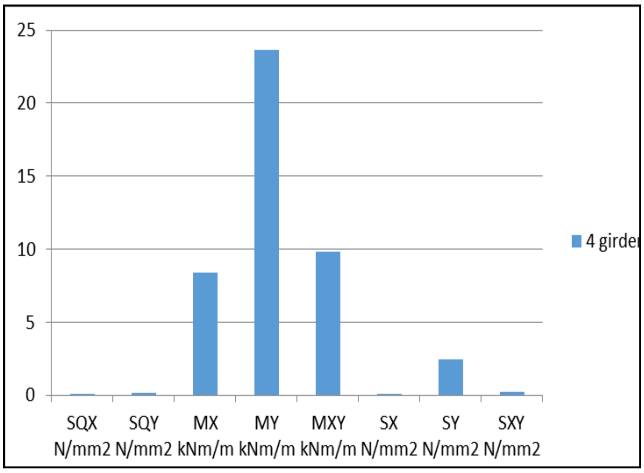
Table 2: Principal stresses in deck slab in N/mm²



Principal stresses in deck slab in N/mm²

Type of girder	M_x KNm/m	M_y KNm/m	M_{xy} KNm/m
Girder No.4	8.462	23.589	9.671

Table 4: Moments in deck slab in KNm/m

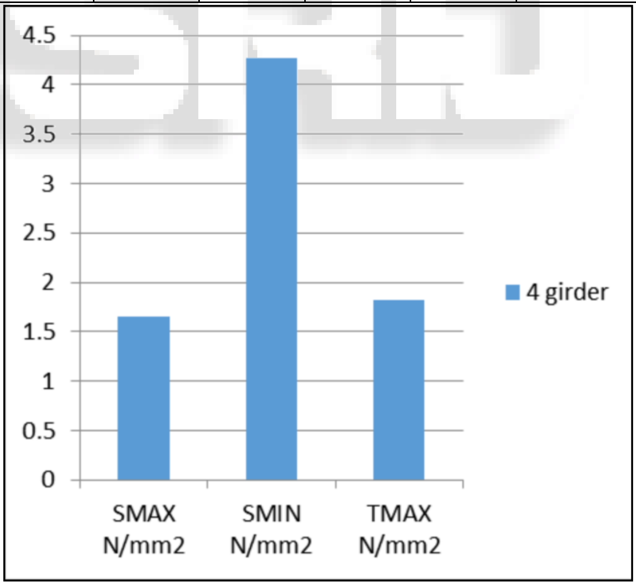


Shear and moments in deck slab in N/mm²

Type of girder	S_{max} N/mm^2	S_{min} N/mm^2	□$_{max}$ N/mm^2
Girder No.4	1.593	4.297	1.69

Table 5: principal stresses in main girder

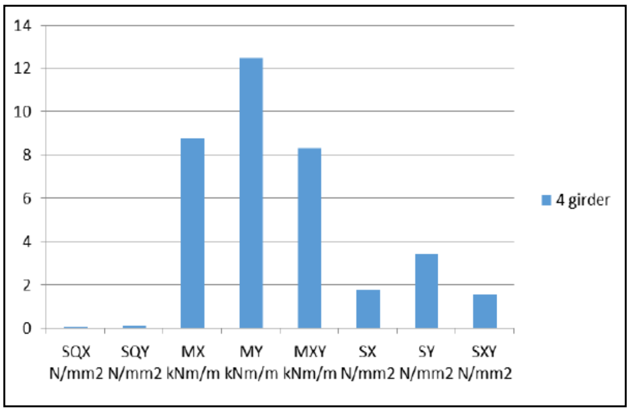
Type of girder	SQ_x N/mm^2	SQ_y N/mm^2	S_x N/mm^2	S_y N/mm^2	S_{xy} N/mm^2
Girder No.4	0.041	0.110	1.778	3.351	1.621



Shear and moments in deck slab in N/mm²

Type of girder	M_x KNm/m	M_y KNm/m	M_{xy} KNm/m
Girder No.4	8.810	12.887	8.441

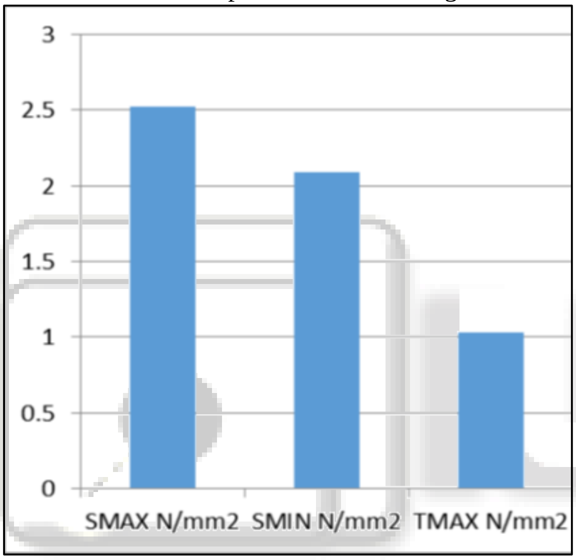
Table 7: Moments in main girder in N/mm²



Shear and moments in main girder in N/mm²

Type of girder	S_{max} N/mm^2	S_{min} N/mm^2	□max N/mm^2
Girder No.4	2.601	2.099	1.057

Table 8: Principal stresses in cross girder



Principal stresses in cross girder

VI. Conclusions

After studying the various parameters of prestressed 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) Prestressed Box Girder Bridge has exceptionally good Torsional rigidity in better transverse load distribution.
2) 40 Length Deck Slab is considered for analysis of precast pre-stressed concrete girder bridges, and for all the cases, deflection and stresses are within the permissible limits.
3) To obtain even better working results the precast pre-stressed concrete girder configuration deck slab can be subjected to pre/post tensioning. The pre-stressing force can be applied more easily and calculation of required jacking force is also simple. This however is not the case in ordinary configuration as it is required to come up with a composite design in case prestressing is considered in the design/construction phase.
4) Ordinary configuration of deck slab creates long term maintenance and serviceability problems as it has more number of exposed components in the structure. This problem can be overcome conveniently in case of precast pre-stressed concrete girder deck slab configuration.
5) We can clearly see the effectiveness of using precast pre-stressed concrete girder configuration as it gives us most of the design parameters within permissible limits of serviceability, deflection and shear compared to ordinary deck slab configuration.
6) There will be increase in head room for undergoing vehicles.
7) 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.
8) Quantities of pre-tension cables required for I-Girder Bridge is more than that of Box Girder Bridge.
9) Maximum Stress distribution and prestressing force is maximum in the box girder bridge when compared to I-girder Bridge.
10) Deformations are maximum in the I-girder Bridge compared to box Girder Bridge.
11) Concrete area required for Box Girder is less than that of I-Girder.

From the above circumstances, I conclude that box Girder Bridge is much effective in all the configurations as its parameters are within the permissible limits. For speedy construction I-Girder Bridge is preferable when compared to Box Girder Bridge.

References

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