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SEGMENTAL BRIDGE DESIGN OF PRESTRESSED BOX SUPERSTRUCTURE FOR CANTILEVER CONSTRUCTION

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ABSTRACT: Bridge "aesthetics" with "economy" is a challenging task for engineers. Superstructure plays a leading role in bridge aesthetics and economy. By using "segmental box type superstructure" aesthetic with economy can be achieved as curtailing of pre-stressing cables became possible. Many times, it is the only option left for "Construction of Bridge in Difficult Climate Region" and "Construction of flyover without disturbing traffic". Analysis and design of cantilever segmental bridges are more complicated and is always a challenging task. Hence here attempt is made to accommodate procedure for three dimensional analysis and design of the same followed by illustrated example. The objective is to spread knowledge of the design of Pre-stressed Segmental Box type superstructure and to show how to automize the same. Efforts made in this study may enhance understanding of how to make the design process somewhat easy by providing automization with the use of the programming languages and by interlinking the same manually; without having particular bridge software package.

KEYWORDS: Box type bridge superstructure, Segments, Segmental Cantilever Bridge, Three Dimensional Analysis of Bridge.

1 INTRODUCTION

In following pages analysis and design of "Precast Post-tensioned Prestressed Segmental Box type Bridge Super-Structure" to be constructed by "cantilevering" are discussed.

1.1 Calculations of Sectional Properties:-

1.1.1 Preliminary Dimensioning and Geometry:

Very first step in designing is assumption of first trial geometry to calculate self weight of segments and sectional properties at different sections, as generally the depth of the box girder as well as the thickness of lower flange reduces parabolically moving towards the mid span and thus the lower profile of the bridge is curved. For this assumption following equations are available but the choice is largely empirical [14]:

$$h_1 = \frac{L\ \{1 + 4(L/100)\}}{11\ \{3 + 4(L/100)\}} \tag{1}$$

$$h_0 = L/40 \text{ to } L/60 \tag{2}$$

$$t_f = (d/36) + 10 \text{ cm} \tag{3}$$

$$b_w = (h/36) + \text{diameters of ducts} + 5 \text{ cm} \tag{4}$$

where: h_1 = Depth of the beam over the support,
h_0 = Depth of the beam at mid span,
t_f = Top Flange Thickness in cm,
b_w = Web Thickness in cm,
L = Span between the piers,
l= Length of the half cantilever (l =L/2),
d = clear span between the webs in cm,
h= Height of box girder in cm.

Now as per the clause no. 9.3.2.1 of IRC: 18-2000, in case of cantilever construction, if pre-stressing cables are to be anchored in web, the web thickness shall not be less than 350 mm or less than that recommended by the pre-stressing system manufacturer, subjected to design requirement. It should be noted that these values are indicative only, and proper calculation must always be made to obtain more accurate values.

1.1.2 Box segment arrangement:-

Distance between the webs may be as much as 5 to 6 m [14]. Wide spacing of webs suggested in the foregoing is justified for spans in excess of 100 m and that for spans of about 60 m web center of about 4m are better [14]. A single box girder is sufficient for bridges up to 11 m in width [14]. For widths between 11 and 16 m a single box girder with three webs may be adopted [14]. For wider bridges, separate box girders should be used as shown in figure 1.

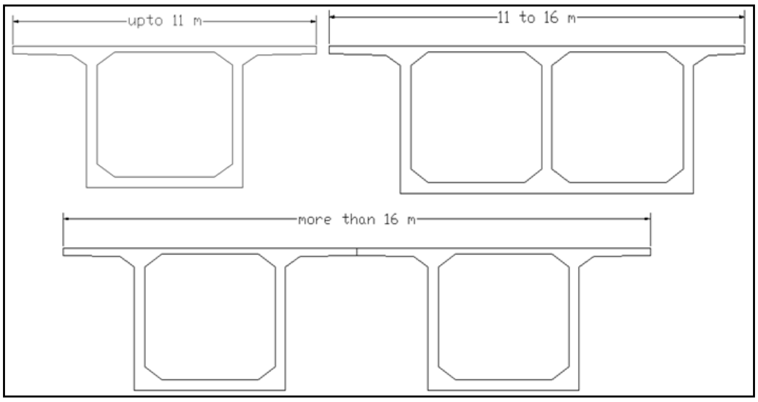


Figure 1: Selection of Number of Boxes as per width

Segments are usually 3 to 5m long [8]. Segments can weigh up to 250 tons in precast cantilevering construction [2]. But for precast constructions the lengths of segments are generally kept between 3 to 3.5 m, by giving due consideration to transportation facilities and keeping in mind that each section must be prestressed, in order to support the next one [14]. Hence, for each section cast, at least one cable in each web for each section must be stressed.

Intermediate Diaphragms within the box are usually not necessary [9]. Also it is concluded in the Journal of Bridge Engineering (ASCE) that behavior of the box girder does not appear to be significantly influenced by the absence of support diaphragms [3]. Other guidelines of IRC: 18-2000 for Box girder's dimensioning shall also be followed.

1.1.3 Properties of Section:-

This task of finding out properties of the section such as cross-sectional areas (A_i), position of centroid from top and bottom of girder, moment of inertia at top and bottom (I_t and I_b), modulus of sections from bottom and top (Z_b and Z_t) etc. can be automized and carried out easily by using an Auto-LISP program "hgeo.lsp" [1]. "hgeo.lsp" program not only draw different sections span-wise but also gives the section properties of the different section as output files (.mpr files); in a number equal to the number of segments plus one.

1.1.4 Geometry Model Generation for SAP Non-linear software:-

To carry out 3-D analysis, like in SAP-2000 Non-linear software with shell elements, the geometry of model in the form of connected shell elements would require. This task can be carried out easily by using another Auto-LISP program "hsapshell.lsp" [1].

1.2 Load Calculation and Analysis:-

Load calculations and analysis can be carried out rapidly and accurately by using finite element based software such as SAP by importing model generated.

1.2.1 Dead Load: - [Dead Loads: Clause No.205 of I.R.C: 6-2000]

The dead load carried by a girder consist of the portion of weight of the superstructure (and fixed loads carried thereon), which is supported wholly or in part by the girder including its own weight. The model geometry generated (.dxf) by Auto LISP program “hsapshell.lsp” can directly be imported in SAP-2000 environment as shell elements [1]. Then after assigning shell thicknesses and support conditions (fixed at support of cantilever), the Dead Load Analysis can be carried out. SAP considers the self weight of the 3-D shell elements and hence no need to define self weight.

1.2.2 Super Imposed Dead Load:- [SIDL: Clause 209 of IRC: 6-2000]

Superimposed dead loads (SIDL) including wearing coat, crash barrier, footpaths, wearing surface, waterproofing, architectural ornamentation, pipes, conduits, cables and other immovable appurtenances installed on the structure. SIDL should be applied after completion of two half cantilevers and after they made continuous, by providing continuity cables at mid span and hence behaving as fixed girder. Clause No. 209 of I.R.C: 6-2000 shall be followed.

1.2.3 Live Load: - [Live Loads: Clause No. 207 of I.R.C: 6-2000]

Clause number 207 of IRC: 6-2000 specifies three classes of vehicular loadings (hypothetical loading) designated as class 70-R, class AA and class A for the design of permanent bridges for which analysis is to be carried out as per number of lanes. Combinations, Impact Factors and reduction in the longitudinal effect shall be considered as per Clause No. 207.4, 211 and 208 of I.R.C: 6-2000. Live Load Analysis can be performed in non-linear FEM software like SAP by using the Moving load options after defining lanes and the maximum values of Bending moments (BMs) and shear forces (SFs) at various sections can be picked up from outputs.

1.2.4 Distribution of loads on longitudinal girders:

Load distribution in bridge is very complicated. It is deeply rooted in domains of higher mathematics and ideally involves orthotropic plate analysis. For box girders, distribution of loads can be best accessed by preparing a three dimensional (3D) model and then by placing loads in analytical software [13]. For this task a 3D model (.dxf) can be imported in FEM software.

1.2.5 Analysis Results

Results, i.e. Shear Forces (SFs) and Bending Moments (BMs) at various sections of segments for following load cases combinations are to be recapitulated.

Load cases:

1. Dead Load of box girder (DL)
2. Super Imposed Dead load (SIDL)
3. Vehicular Live Load cases (LL)

Load Combination [14]:

1. DL
2. DL+ SIDL+ Secondary Pre-stressing parameter (i.e. total permanent moment and shear)
3. DL+ SIDL+ Secondary Pre-stressing parameter+ Live Load case giving worst effect [Maximum Absolute Values of design parameters (corresponding to min. algebraic value)]

2 ESTIMATION OF LIMITING ECCENTRICITIES, PRE-STRESSING FORCE REQUIRED, NUMBER OF CABLES REQUIRED AND CURTAILMENT PLANNING OF CABLES.

2.1 Estimation of Limiting Eccentricities and Pre-stressing Forces Required:

In figure 2, G is the centroid and A' is lower boundary of the central core, the resultant pre-stressing force must be such that, under the effect of the total loading (dead and live), the center of thrust lies at A'.

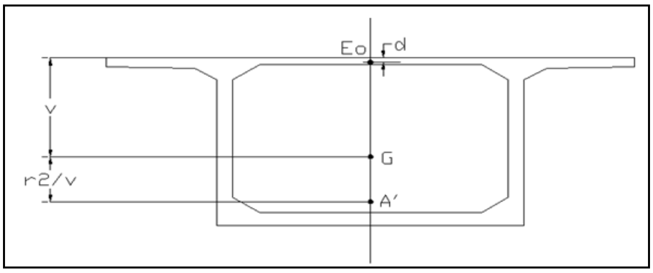


Figure 2: Lever Arm for Cantilever Box Section.

Hence, if d is the distance of the tendon from the upper face, it follows that at every section:

$$F = P = \frac{Mp + Ms}{v+(r2/v)-d} = \frac{Mp + Ms}{e+(r2/v)}; \text{ (as } e=v-d) \quad (5)$$

where: F = P = Pre-stressing Force
Mp= Moment due to Dead Loads
Ms= Moment due to Live Loads
v= Distance of centroid from upper face
d= Distance of tendon from upper face
e= eccentricity of tendons = v – d.

One method of determining the cable profile would be to calculate for each section the limiting positions “E1” and “E2”. However use of such a procedure tends to obscure the primary object of pre-stressing, which is the creation of pressure lines of suitable shapes and in suitable positions [4]. It’s therefore better and also simpler to base the calculation on the limiting eccentricities between which the resultant line of thrust must lie. If TPM = Total Permanent Moment, R1, R'1 are the limiting stresses corresponding to the M_1= TPM + Negative LLBM and R2, R'2 are the limiting stresses corresponding to the M_2= TPM + Positive LLBM. These limiting eccentricities and stresses for cantilevered girders are represented in figure 3a and 3b respectively.

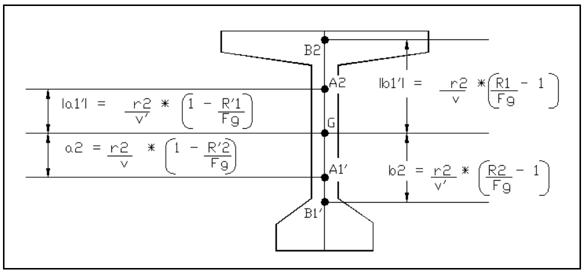


Figure 3a: Limiting Eccentricities (Cantilever)

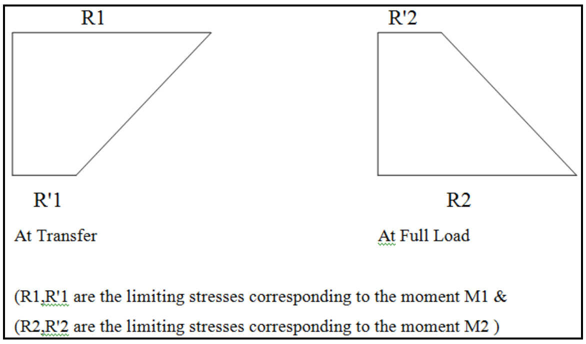


Figure 3b: Limiting Stresses (Cantilever)

Note: v’= Distance of centroid from lower face; a1', a2 and b1', b2 are limiting eccentricities for tension and compression respectively.

a1' and a2 are the limiting eccentricities for tension; that is, the limits which must not be exceeded (in absolute value) in order to avoid excessive tension (or in order that the stress should not fall below the minimum permissible value) at the opposite face. b1' and b2 are the limiting eccentricities for compression; that is, the limits which must not be exceeded (in absolute value) if compressive stresses in excess of the permissible limits are to be avoided at the face nearest to the particular limit concerned. The absolute values of these eccentricities are given by the expressions:

For Cantilevered Girders:

$|a1'| = (-r^2 / v') (1 - R'1/ Fg)$ $\qquad |b1'| = (-r^2 / v) (R1/ Fg - 1)$

$a2 = (r^2 / v) (1 - R'2/ Fg)$ $\qquad b2 = (r^2 /v') (R2 / Fg - 1)$

In the case of beam with a constant eccentricity at every section, the pre-stressing moment 'F.e'or "P.e" would have too large value at all other sections other than that at which the moment is a maximum, if F remains constant. So a varying pre-stressing force at constant eccentricity can be obtained and the best use of the tendons can be achieved by curtailing them. But while doing so at every section, the magnitude of F must lie between two limiting values F_{min} and F_{max}. If F reaches one or other of these values, a limiting permissible stress is reached at one or other face.

In other words, for Cantilevered Girders, Sufficient tendons must be provided at any section to counteract the tensions due to the applied load at the upper face, but their numbers must be so limited that at transfer both the compression at upper face and tension at lower face must be within the permissible values. Denoting the absolute (constant) values of the eccentricity by e, the statement can be represented by the following relations:

For Cantilevered Girders:

(1) F minimum: $$\frac{Fmin}{A}\left(1 + \frac{e}{r^2/v}\right) + f'2 = R'2$$

And since, $F'2 = -M2 / (I/v)$

Therefore $$\frac{Fmin}{A}\left(\frac{e + r^2/v}{r^2/v}\right) = R'2 + \frac{M2}{(I/v)}$$

Therefore $$Fmin = \frac{M2 + R'2(I/v)}{e + r^2/v} \qquad (6)$$

(2) F maximum:

(a) $$\frac{Fmax}{A}\left(1+\frac{e}{r^2/v}\right)+f'1 = R1$$

and since, $f'1 = -M1 / (I/v)$

Therefore $$\frac{Fmax}{A}\left(\frac{e+r^2/v}{r^2/v}\right) = R1 + \frac{M1}{(I/v)}$$

therefore $$Fmax = \frac{M1 + R1\,(I/v)}{e + r^2/v} \tag{7}$$

(b) $$\frac{Fmax}{A}\left(1-\frac{e}{r^2/v'}\right)+f1 = R'1$$

and since, $f1 = M1 / (I/v')$

therefore $$\frac{Fmax}{A}\left(\frac{e-r^2/v'}{r^2/v'}\right) = -R'1 - \frac{M1}{(I/v')}$$

therefore $$Fmax = \frac{M - R'1(I/v')}{e - r^2/v'} \tag{8}$$

2.1.2 Calculations of Secondary Moment due to Pre-stressing Using Theorem of Three Moments

Calculations of Secondary Moment due to Pre-stressing can be carried out using Equation of Theorem of Three Moments written below [13]:

$$M_{ab} + 2\cdot M_{ba} + 2\cdot k\cdot M_{bc} + k\cdot M_{cb} = K_{ab} + k\cdot K_{bc} \tag{9}$$

where $K = \frac{6\times P\times\int(ex\,dx)}{L^2}$ and

$k = \frac{I_{ab} / L_{ab}}{I_{bc} / L_{bc}} = 1$

2.2 Estimation of Cables Required and Curtailment Planning of Cables (Theoretically):

Number of continuity cables required (Per deck of half bridge)

$$= \frac{\text{Total Pre-stressing force required } (Fc_{req}) \text{ per deck}}{\text{Force by cable after loss}}$$

where;

$$Fc_{req} = \frac{Mc}{e_c + (r^2 / v')} \quad \text{(in ton)} \qquad (10)$$

where: e_c = eccentricity of continuity cable = – (v'–d) in m
v'= Distance of centroid from lower face
d= Distance of tendon from upper face (m),
Mc= Total Max. Positive Moment at cantilever tip
= Total Secondary Moments + Max Positive Moment at cantilever tip
= (m+M) + Max Positive Moment at cantilever tip

These Secondary Pre-stressing Moments (m and M) can be calculated using following equations [13].

$$m = \frac{e \times E \times \sum (dx/h)}{\sum (dx/I)} \qquad (11)$$

and

$$M = \frac{□sei \times \sum (M' \times dx/I)}{1 + □sei \times \sum (dx/I)} \qquad (12)$$

where: E = Modulus of Elasticity
e = Exp. per unit length= $\Delta T \times$Thermal coefficient
□sei= Co-efficient [13] [14]
M'= DLBM + Secondary Pre-stressing Moment
dx= lengths of segments
h= height of box at different sections
I= Moment of Inertia at different sections

2.3 Estimation of Cables Required and Curtailment Planning of Cables (Graphically)

Graphical representation of “Limiting zone for cable profile without and with curtailment of cables” and “Estimation of number of cables required at different sections and curtailment planning of cables” can also be carried out graphically by passing a curve between F_{min} and F_{total} as shown in figure 4 and 5 respectively.

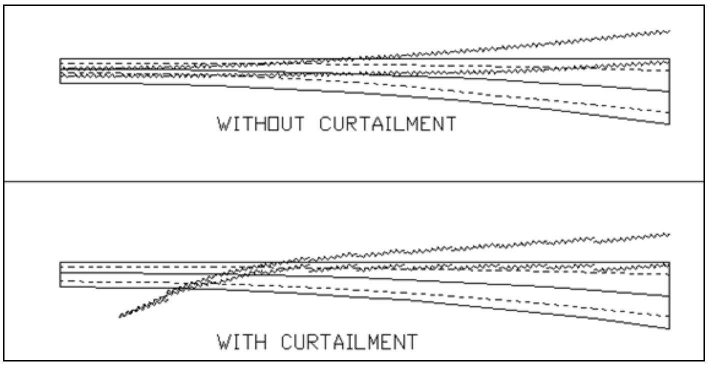


Figure 4: Limiting Zone (zigzag) for Cable Profile

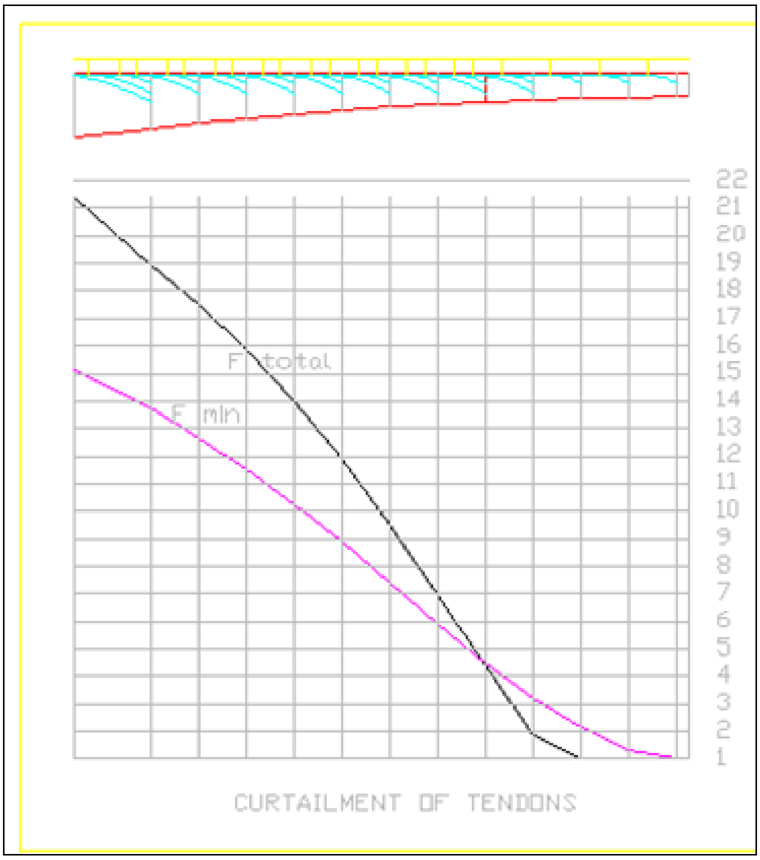


Figure 5: Estimation of Cables Required.

2.4 Pre-stressing of Cables:

The maximum permissible jacking force considered is 0.765 times the maximum ultimate tensile strength of the strand. The positions of cable are to be provided in such a manner that fulfills the permissible stress criteria as per I.R.C: 18-2000 Clause 7 at different sections.

3 ESTIMATION OF LOSSES: (IRC:SP:65-2005, IRC:18-2000)

The amount of pre-stress decrease due to
(a) Instantaneous losses (e.g. friction, seating of anchorages and elastic shorting) and
(b) Time dependent Losses (e.g. creep, shrinkage and relaxation of steel) are to be calculated as per clauses of codes as tabulated in table 1 below:

Table 1- Clauses of codes for loss calculations

Losses Due To	Clause of IRC: 18-2000
Friction :	Clause No 11.6
Seating of Anchorages :	Clause No 11.5
Elastic Shortening :	Clause No 11.1
Creep of Concrete :	Clause No 11.2
Shrinkage of Concrete :	Clause No 11.3
Relaxation of Steel :	Clause No 11.4

Above loss calculations at different sections can be carried out in tabular format in following sequence.

No	Description
1	Ordinates of Cable in Elevation at Different Section for Box Girder
2	Angle made by the tangents of cables in elevation (degree)
3	Ordinates of cable in plan at diff sections
4	Angle made by tangents of cables in plan
5	Distance x from point of application of pre-stress force for each cable
6	Calculation of force after frictional losses
7	Summary of force after friction loss
8	Summary of stresses after friction loss
9	Calculation of slip loss and stresses after slip loss
10	Summary of stress after frictional and slip losses
11	Summary of force after friction and slip loss
12	Summary of data require for calculation of gradual losses at different sections
13	Calculation of gradual loss at diff. sections
14	Summary of calculation of gradual losses at different sections.

Some related clauses of different losses are discussed for ready reference.

3.1 Loss due to Slip and Friction (IRC-18:2000 Cl. 11.6,Table -5)

Stresses after friction loss can be calculated using following equation as per IRC-18.

$\sigma_{po} = \sigma_{po}(x)\ e^{z}$ (13)

Where; $z = kx+\mu\sqrt{(\theta_v^2 + \theta_h^2)} = kx+ \mu\theta$
σ_{po} = Steel stress at jacking end
e= base of Naperian Logarithms
k = wobble co-efficient per meter length of steel
μ = coefficient of friction
θ= cumulative angle in radians through which the tangent to the cable profile has turned between points of operation of σ_{po} and $\sigma_{po}(x)$. [θ_v is vertical cumulative angle and θ_h is horizontal cumulative angle]
x= distance between points of σ_{po} and $\sigma_{po}(x)$.

Loss due to slip (As per IRC-18, 2000)

As per standard Manufacturer's recommendations 6 mm reverse slip of tendons is considered. Slip Point is to be found out such that area of stress diagram remains less than 117 kg/mm.

3.2 Time dependent Losses

3.2.1 Loss due to Elastic Shortening, Relaxation of steel, Creep and Shrinkage (IRC-18)

(a) Losses due to Elastic Shortening =0.5 x Modular Ratio x Stress in Concrete (Adjacent to tendon averaged along length)
(b) Losses due to Relaxation of Steel = Found out from Ration of Average Stresses after Elastic Shortening & UTS.
(c) Losses due to Shrinkage = Strain between 7 days & 90 days and strains beyond 90 days are considered.
(d) Losses due to Creep = Generally to be considered for 80 to 90 % maturity.

3.3 Permissible Stresses in Concrete: (IRC: SP: 65-2005 Cl.No.5.3, IRC: 18-2000 Cl.No 7)

3.3.1 Permissible Temporary Stresses at Transfer of Concrete:-

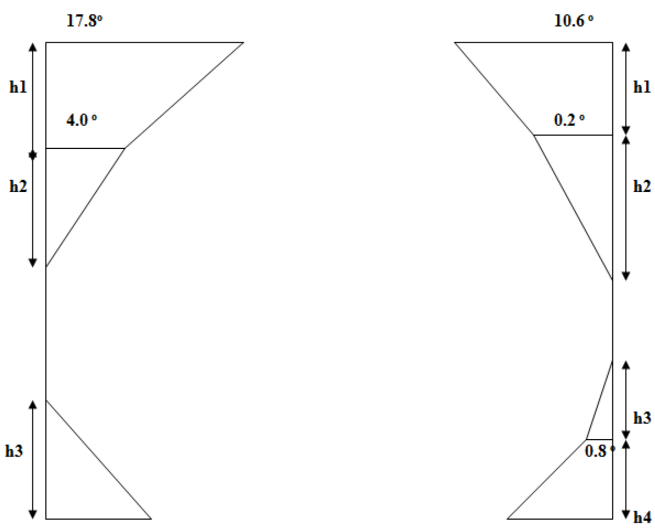
(a) Temporary Compressive stress shall not exceed 0.50 f_{cj} subject to maximum of 20Mpa; where f_{cj} is concrete strength at j days subject to maximum of f_{ck}.
[f_{ck}= Characteristic strength of concrete at 28 days]
(b) Temporary Tensile stress shall not exceed 1/10th of the Compressive stress.

3.3.1.2 Permissible Stresses during Service.

(a) The Compressive stress under service load shall not exceed 0.33 f_{ck}.
(b) No Tensile stress shall be permitted in the concrete during service.

3.3.2 Temperature Difference (IRC: 6 –Cl. 218)

Effect of temperature difference within the super structure shall be derived from positive temperature difference which occur when condition are such that solar ration and other effect cause a gain in heat through the top surface of the superstructure. Conversely, reverse temperature differences are such that heat lost from the top surface of the bridge deck as a result of re-radiation and other effect. Positive and reverse temperature difference for the purpose of design shall be as per IRC 6:2000 clause no 218 and as shown in figure 6. The stresses resulting from temperature effect not exceeding in the value of two third of modular of rupture may be permitted in prestressed concrete bridges [14].



h1 = 0.3 h < 0.15m

h2 = 0.3 h > 0.10m

< 0.25m

h3 = 0.3 h > 0.15m

h1 = h4 = 0.2 h < 0.25m

h1 = h3 = 0.25h < 0.20m

Figure 6: Temperature Variation Diagram

3.3.3 Checking of Stress: (I.R.C:18-2000 Cl. No 5)

The Stresses are to be checked at each construction stage. Stress due to continuity tendon pre-stressing shall also be included at some middle sections where they are acting. Thus stresses along with secondary stresses for cantilever box girder are to be checked at each construction stage and for following different discrete stages:

For Cantilever Girders:-

(a) Service Dead Load + Pre-stress with full losses (At transfer at each stage)

$f(top) = P/A + P.e / Z_t – M_d / Z_t$ (14)
$f(bottom) = P/A– P.e / Z_b + M_d / Z_b$ (15)

(b) Service Dead Load + Live Load + Pre-stress with full losses (At service)

$f(top) = P/A + P.e / Z_t – M_d / Z_t – M_l / Z_t$ (16)
$f(bottom) = P/A– P.e / Z_b + M_d / Z_b + M_l / Z_b$ (17)

(c) Service D.L. + 50 % L.L. + Prestressed with full losses + Reverse temperature stress (At service)

$f(top) = P/A + P.e/Z_t – M_d/Z_t –0.5×M_l/Z_t–M_t/ Z_t$ (18)
$f(bot)=P/A– P.e/Z_b + M_d/Z_b +0.5×M_l/Z_b–M_t/Z_b$ (19)

Where, P = Pre-stressing force in KN,
M_1= Live load bending moments in KN-m,
M_d=Dead load bending moments in KN-m,
e= eccentricity of tendons,
Z_t= modulus of sections from top,
Z_b= modulus of sections from bottom.

4 CHECK FOR STRESSES

4.1 Completion Stresses

Check for Stress at different sections during construction i.e. “completion stresses” are to be carried out. It is very important to find out stresses at each and every construction stage and at each and every sections instead of finding only final stresses. Stresses due to secondary moments are also to be added to access the completion stresses. As in segmental construction at each stage of construction moment and pre-stressing forces are added. So corresponding stresses are developed and are to be calculated at all sections at every increment of segment.

4.2 Check for Stresses

Stress calculations at different sections can be done in tabular format in following sequence:
(a) Check for stress conditions during construction
(b) Check for stresses under service condition

(c) Check for shear at different sections
(d) Check for ultimate strength at various sections

4.3 Check for Shear Stress (At Diff Sections):

Check is to be carried out as per following.

(A) I.R.C: 18-2000, Clause No 14

Calculations for shear are only required for ultimate load. Ultimate Shear Resistance (Vc) shall be considered for section both un-cracked (Vco) and cracked (Vcr) in flexure and lesser value shall be taken. If necessary, Shear reinforcement (clause no 14.1.4) shall be provided.

Ultimate loads = 1.25 G + 2.0 SG + 2.5 Q

where, G = permanent dead load
SG = superimposed dead load
Q = live load including impact

Requirement of Shear Reinforcement:

If V < Vc/2 then No Shear Reinforcement is required.

If V > Vc/2 then Minimum Shear Reinforcement shall be provided as per:

$$\frac{A_{sv} \times 0.87 \times F_{yv}}{S_v \times b} = 0.4$$

If V < Vc/2 then Shear Reinforcement shall be provided as per:

$$\frac{A_{sv}}{S_v} = \frac{V - V_c}{0.87 \times F_{yv} \times d_t}.$$

(B) IRC: SP: 65-2005, Clause No. 5.4.2.1

Ultimate Shear Capacity (Vc) calculated as per clause 14 of IRC: 18: 2000 shall be multiplied by a factor of 0.90 for internal bonded tendons.

4.4 Check for the Ultimate Strength

(A) I.R.C: 18-2000, Clause No 13

Prestressed member shall be checked for failure conditions at an ultimate load of 1.25G+2.0SG+2.5Q under moderate condition and 1.50G+2.0SG+2.5 Q under severe conditions. The two alternative condition of failure shall be calculated by the following formulae and the smaller of the two values shall be taken which should be less than ultimate resistance of section:

(1) Failure of yield of steel (under reinforced section)

$$M_{ult} = 0.9\, d_b\, A_s\, f_p \quad (19)$$

where, A_s = the area of the high tensile steel
f_p = the ultimate tensile strength for steel

d_b = depth of girder from the max. Compression edge to the Center of gravity of the steel tendons

(2) Failure by crushing of concrete:

$$M_{ult} = 0.176\, b\, d_b^2\, f_{ck} + (2/3)\, 0.8(B_f - b)\, (d_b - t/2)\, t\, f_{ck} \tag{20}$$

where, b = width of web
B_f = width of top flange
t = thickness of top flange

(B) IRC: SP: 65-2005, Clause No. 5.5.2

AS per clause no. 5.5.2 of IRC: SP: 65-2005, the flexural capacity (M_{ult}) calculated as per clause 13 of IRC: 18-2000 shall be multiplied by a factor of 0.95 for internal bonded tendons.

5 ILLUSTRATED EXAMPLE

5.1 Introduction

The complete analysis and design of segmental cantilever bridge super-structure is covered with atomization at possible locations. Problem is selected of the bridge having span of 80.50 m. Elevation with segment and section numbers, Preliminary cross-section dimensions at support, near mid span and longitudinal geometry are determined and are as shown in Figure 7a, 7b and 7c respectively. The design is carried out for one of the half-section (i.e. for 40.25 m span) and then two half cantilevers are made continuous by providing continuity cable at mid portion. The following are the parameter used in the design calculation.

(a) Center to center Span: -------------------------------- 80.50 m
(b) Depth at support and mid: -------------------------- 4.30 m and 1.60m
(c) Nos. of Lane: --- 3-lanes
(d) Total width of girder (transverse):------------------ 14.40 m
(e) Nos. of webs for carriage way: --------------------- 3 Nos
(f) Cantilever length on outer side: --------------------- 1.975m
(g) Cantilever length on inner side: --------------------- 1.275m
(h)Type of High tensile steel used: --------------------- 12T13
(i) U.T.S. of strands: ------------------------------------- 0.765xUTS
(j) Modulus of Elasticity of steel: ---------------------- 1.95x 10^6t/m2
(k) Materials for sheathing duct------------------------ Galvanized steel
(l) Diameter for sheathing duct-------------------------- 75mm
(m)Grade of concrete (Density):------------------------ 40 Mpa (2.5t/m2)
(n)Thickness of wearing-coat considered: ------------ 0.062m
(o)Size of Crash Barriers considered: ------------------ 0.5m×0.9 m
(p) Width of footpath on outer-sides: ------------------ 1.5 m
(q) Haunch size: -- 0.60 m×0.50 m

(r) Length of first, middles and end segment ----------- 4.96 m, 3.14 m & 0.75 m

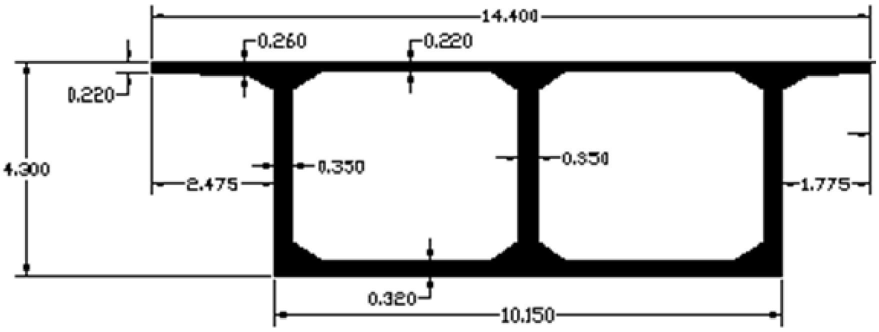


Figure 7a: Cross-Sectional Dimensions at support

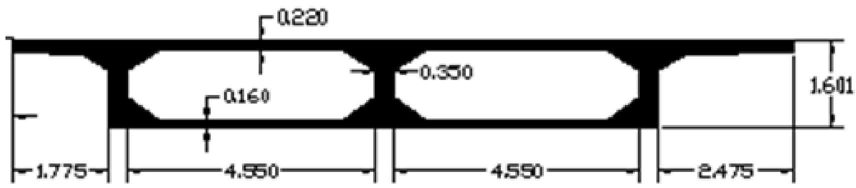


Figure 7b: Cross-Section at mid span

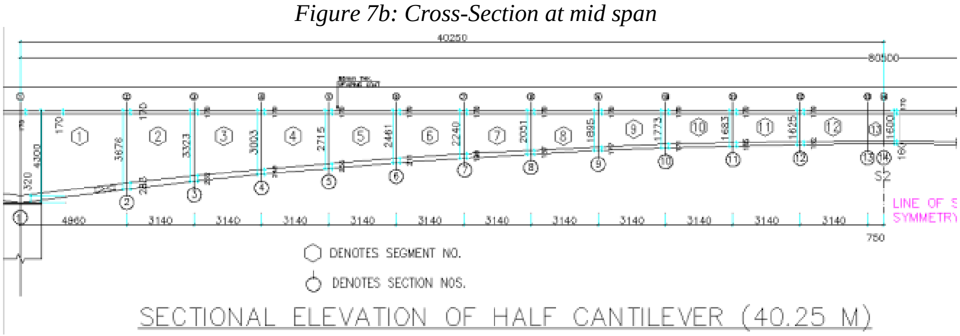
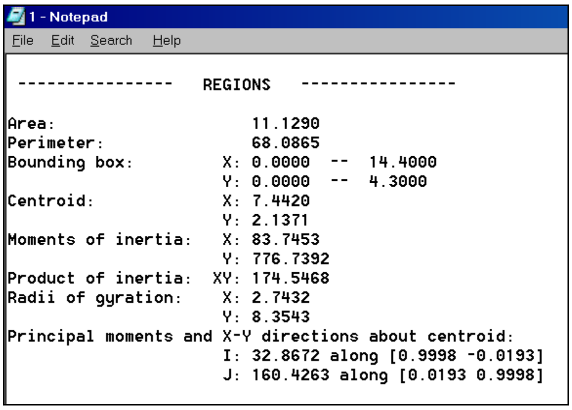


Figure 7c: Elevation of Half Cantilever.

5.1.1 Calculations of Sectional Properties:-

Figure-8a and 8b below shows the "sample output file of program (hgeo.lsp)" containing sectional properties of sections and "summary of outputs of (hgeo.lsp)" respectively [1].



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 ----------------   REGIONS   ----------------

Area:                         11.1290
Perimeter:                    68.0865
Bounding box:              X: 0.0000  --  14.4000
                           Y: 0.0000  --  4.3000
Centroid:                  X: 7.4420
                           Y: 2.1371
Moments of inertia:        X: 83.7453
                           Y: 776.7392
Product of inertia:       XY: 174.5468
Radii of gyration:         X: 2.7432
                           Y: 8.3543
Principal moments and X-Y directions about centroid:
                           I: 32.8672 along [0.9998 -0.0193]
                           J: 160.4263 along [0.0193 0.9998]
```

Figure 8a: Sample output of "hgeo.lsp".

Section No.	Depth (m)	Area (sqm)	v (m)	v ' (m)	I (m4)	I / v (m3)	I / v ' (m3)	R 2 / v (m3)	r 2 / v ' (m3)	Remark
1	4.300	11.129	2.163	2.137	32.867	15.196	15.379	1.365	1.382	Outputs of AutoLISP program "hgeo.lsp", which serve as inputs for this calculations
2	3.676	10.137	1.797	1.878	21.990	12.235	11.708	1.207	1.155	
3	3.323	9.576	1.595	1.728	16.981	10.645	9.829	1.112	1.026	
4	3.003	9.067	1.415	1.588	13.098	9.257	8.250	1.021	0.910	
5	2.715	8.611	1.256	1.459	10.119	8.055	6.935	0.935	0.805	
6	2.461	8.206	1.119	1.342	7.857	7.024	5.853	0.856	0.713	
7	2.240	7.854	1.001	1.238	6.161	6.153	4.976	0.783	0.634	
8	2.051	7.555	0.904	1.147	4.907	5.431	4.277	0.719	0.566	
9	1.895	7.307	0.825	1.071	3.998	4.848	3.734	0.664	0.511	
10	1.772	7.112	0.763	1.009	3.358	4.399	3.328	0.619	0.468	
11	1.682	6.969	0.719	0.963	2.931	4.075	3.043	0.585	0.437	
12	1.625	6.878	0.692	0.934	2.677	3.872	2.867	0.563	0.417	
13	1.601	6.839	0.680	0.921	2.574	3.786	2.794	0.554	0.409	
14	1.600	6.838	0.679	0.921	2.570	3.783	2.791	0.553	0.408	

Figure 8b: Summary of outputs of "hgeo.lsp".

5.1.1.1 Geometry Model for SAP software:-

Figure 9 below shows the model (.dxf) generated by AUTOLISP program ("hsapshell.lsp") to import in SAP non-linear software for 3D analysis [1].

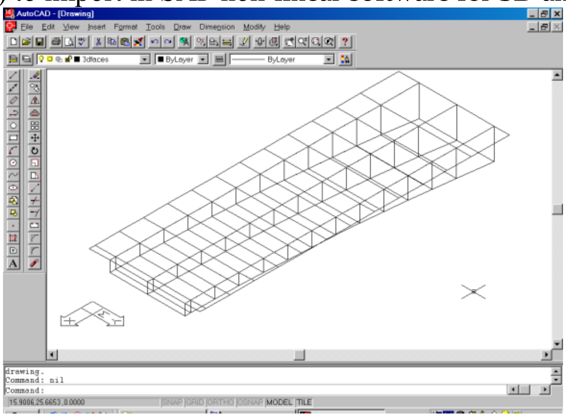


Figure 9: A 3-D Model to import in SAP software.

5.1.2 Load Calculation and Analysis:-

5.1.2.1 Dead Load Analysis in SAP:-

Figure 10 below shows view of DL analysis in SAP Non-linear platform.

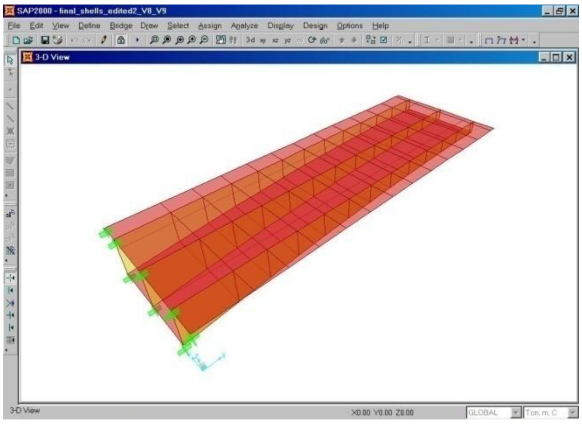


Figure10: View of Dead load analysis on 3D model imported in SAP Non-linear platform

5.1.2.2 Super Imposed Dead Load (SIDL) Calculations & Analysis:-

Calculations of SIDL:

Density of wearing coat material = 2.20 t/m^3.
Density of material for crash barrier = 2.50 t/m^3.
Density of Footpath material = 2.30 t/m^3.
Thickness of wearing coat = 0.62 m.
Width of wearing coat = 12.40 m.
Cross Sectional Area of crash barrier = 0.40 m^2.
Dimensions of Footpath (width x depth) = 1.50 m×0.30 m.

(1)UDL (Longitudinal dir) due to wt. of wearing coat
=0.062x2.2=0.1364t/m^2 or =12.4x0.062x2.2=1.70t/m
(2)UDL (Longitudinal dir) of wt. of crash barrier
=0.4x2.5 = 2.00 t/m.
(3)UDL (Longitudinal dir) due to wt. of footpath
= 0.3x2.3 = 0.69 t/m^2 or = 1.5×0.3×2.3 = 1.035 t/m.

Figure 11 below shows the model of whole fixed span for SIDL analysis. SIDL will be applied after completion of two half cantilevers and after they made continuous.

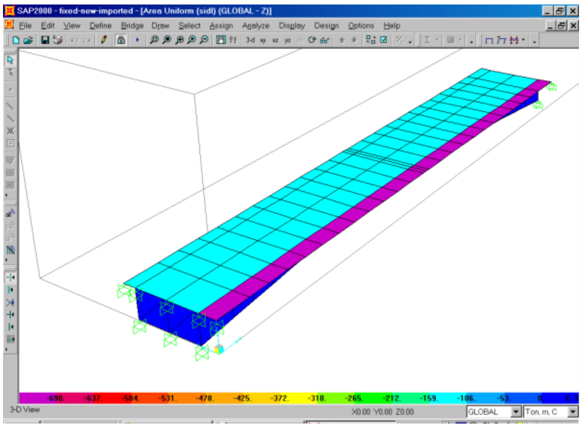


Figure 11: View of SIDL analysis on 3D model imported in SAP Non-linear platform

5.1.2.3 Live Load Calculations:-

Live load analysis is carried out after defining lanes [figure 12a] in SAP non-linear platform, as per IRC –6 considering vehicular loads as shown (including Impact Factors in axel loads) in Figure-12b and 12c below.

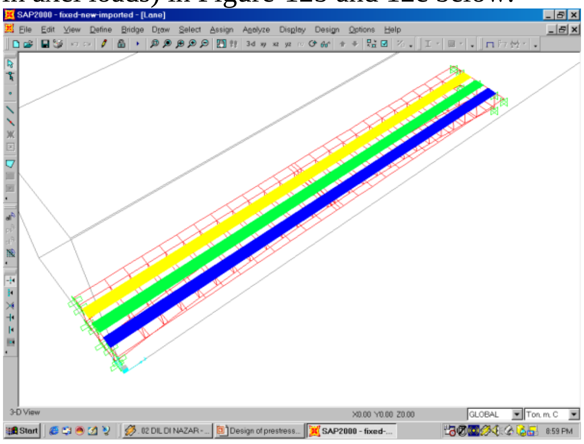


Figure 12a: Live Load analysis in SAP non-linear platform after defining lanes.

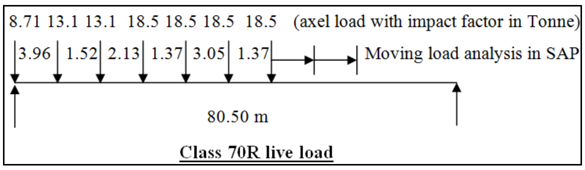


Figure 12b: Axel loads of Class 70R Load .

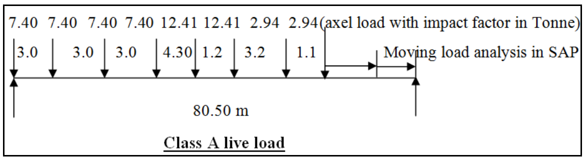


Figure 12c: Axel loads of Class A Load.

Following sub conditions in this live loading option are analyzed in SAP Non-linear and the result of worst condition is to be considered in the design.

(A) One lane of 70R live load and one lane of Class A live load:

- Class-70R LL on left eccentric position + Class-A LL besides on right
- Class-70R LL on left eccentric position + Class-A LL on right eccentric position
- Class-70R LL on right eccentric position +Class-A LL besides on left

(B) Three lanes of Class-A live load:

- Three Class-A Vehicular LL (left eccentric)
- Three Class-A Vehicular LL (right eccentric)
- Three Class-A Vehicular LL (middle position)

5.1.2.4 Analysis

5.1.2.5 Analysis Results

Output results from SAP-2000 nonlinear software for different load cases are to be recapitulated. Table-2 below shows sample output of D.L. analysis.

Table 2 - Sample Output result of Analysis for D.L.

TABLE: Section Cut Forces - Analysis								
	Output Case	Case Type	F1	F2	F3	M1	M2	M3
Text	Text	Text	Ton	Ton	Ton	Ton-m	Ton-m	Ton-m
1	DEAD LOAD	LinStatic	2.01E-11	2.32E-12	674.526	-23.836	-12169.317	-8.38E-11
2	DEAD LOAD	LinStatic	1.99E-11	2.35E-12	562.991	-23.385	-9103.408	-9.46E-11
3	DEAD LOAD	LinStatic	2.01E-11	2.58E-12	498.254	-22.585	-7437.962	-9.07E-11
4	DEAD LOAD	LinStatic	1.88E-11	2.22E-12	437.611	-21.428	-5969.297	-1.02E-10
5	DEAD LOAD	LinStatic	2.01E-11	2.65E-12	380.656	-19.948	-4685.196	-9.29E-11
6	DEAD LOAD	LinStatic	2.01E-11	1.41E-12	326.983	-18.180	-3574.716	-7.92E-11
7	DEAD LOAD	LinStatic	2.24E-11	2.75E-12	276.262	-16.154	-2628.067	-1.02E-10
8	DEAD LOAD	LinStatic	3.38E-11	1.60E-12	228.016	-13.912	-1836.731	-7.79E-11
9	DEAD LOAD	LinStatic	3.22E-11	2.18E-12	181.841	-11.488	-1193.569	-6.21E-11
10	DEAD LOAD	LinStatic	2.67E-11	1.01E-12	137.334	-8.919	-692.711	-5.50E-11
11	DEAD LOAD	LinStatic	8.17E-14	-2.10E-12	94.094	-6.238	-329.550	-1.57E-10
12	DEAD LOAD	LinStatic	-1.60E-11	3.55E-12	51.792	-3.476	-100.623	-5.46E-11
13	DEAD LOAD	LinStatic	-3.40E+00	-5.68E-02	3.811	-1.041	-1.639	-7.38E-01

** SF and BM due to haunches to be added manually*

[Note- Additional moments due to self weight of splays or haunches (as not considered in the modeling) are added in output results. So at each section moment of $wx^2/2$ shall be added; where w is load in running meter (3.125 t/m) and x is distance of section considered (meter) from mid span.]

5.1.2.6 Recapitulation of Bending Moment and Shear Forces at various sections:

The BMs and SFs at various sections are recapitulated as shown in Table - 3 and 4 below:

Table 3- Summary of B.M. at various sections *Table 4- Summary of S.F. at various sections*

Sr. No.	Section At Dist. X (m)	Dead Load Moment (tm)	SIDL Moment (tm)	Live Load Moment (governing)			
				ClassA 3lane		70R+ClassA	
				Min (tm)	Max (tm)	Min (tm)	Max (tm)
1	0.000	-14700	-2206	-2802	0	-2671	0
2	4.960	-11049	-1614	-2183	11	-2086	11
3	8.100	-9053	-1279	-1814	32	-1738	32
4	11.240	-7284	-975	-1466	67	-1408	67
5	14.380	-5731	-702	-1141	118	-1100	117
6	17.520	-4382	-461	-847	188	-819	187
7	20.660	-3228	-251	-586	273	-571	272
8	23.800	-2260	-72	-368	365	-361	364
9	26.940	-1471	76	-195	467	-193	466
10	30.080	-855	193	-78	560	-78	559
11	33.220	-407	278	-16	636	-17	634
12	36.360	-125	332	-1	686	-1	685
13	39.500	-3	318	0	625	0	623
14	40.250	0	0	0	625	0	623

Sr. No.	Section At Dist. X (m)	Dead Load Shear Force (t)	SIDL Shear Force (t)	Live load Shear Force	
				ClassA 2lane Max (t)	70R lane Max (t)
1	0.000	675	138	171	163
2	4.960	563	121	166	160
3	8.100	498	110	160	155
4	11.240	438	100	152	149
5	14.380	381	89	144	142
6	17.520	327	78	134	133
7	20.660	276	67	122	123
8	23.800	228	56	110	112
9	26.940	182	46	99	101
10	30.080	137	35	87	89
11	33.220	94	24	75	76
12	36.360	52	13	63	64
13	39.500	4	1	6	8
14	40.250	0	0	0	0

5.2 Estimation of Limiting Eccentricities, Pre-stressing Force and Number of Cables Required and Curtailment Planning of Cables:

5.2.1 Estimation of Limiting Eccentricities and Pre-stressing Force Required:

Table 5a and 5b below represent "Inputs" and "Calculations" for pre-stressing forces required (in ton) at different sections respectively.

Table 5a - "Inputs" to calculated forces (in ton)

Section No.	Depth (m)	Area (sqm)	v (m)	v ' (m)	I (m4)	I / v (m3)	I / v ' (m3)	R 2 / v (m3)	r 2 / v ' (m3)	Remark
1	4.300	11.129	2.163	2.137	32.867	15.196	15.379	1.365	1.382	Outputs of AutoLISP program "hgeo.lsp", which serve as inputs for this calculations
2	3.676	10.137	1.797	1.878	21.990	12.235	11.708	1.207	1.155	
3	3.323	9.576	1.595	1.728	16.981	10.645	9.829	1.112	1.026	
4	3.003	9.067	1.415	1.588	13.098	9.257	8.250	1.021	0.910	
5	2.715	8.611	1.256	1.459	10.119	8.055	6.935	0.935	0.805	
6	2.461	8.206	1.119	1.342	7.857	7.024	5.853	0.856	0.713	
7	2.240	7.854	1.001	1.238	6.161	6.153	4.976	0.783	0.634	
8	2.051	7.555	0.904	1.147	4.907	5.431	4.277	0.719	0.566	
9	1.895	7.307	0.825	1.071	3.998	4.848	3.734	0.664	0.511	
10	1.772	7.112	0.763	1.009	3.358	4.399	3.328	0.619	0.468	
11	1.682	6.969	0.719	0.963	2.931	4.075	3.043	0.585	0.437	
12	1.625	6.878	0.692	0.934	2.677	3.872	2.867	0.563	0.417	
13	1.601	6.839	0.680	0.921	2.574	3.786	2.794	0.554	0.409	
14	1.600	6.838	0.679	0.921	2.570	3.783	2.791	0.553	0.408	

Table 5b - "Calculations" of pre-stressing forces

Section No.	Mp (DLBM)	Mq (SIDLBM)	MH (tm)	TPM (tm)	Ms1	Ms2	M1	M2	Eccentricity (e)	r^2 / v	$e + (r^2/v)$	Fp (min) required	Fp (Total) required
(i)	(ii)	(iii)	(iv)	(v)=ii+iii+iv	(vi)	(vii)	(viii)=v+vi	(ix)=v+vii	(x)	(xi)	(xii)=x+xi	(xiii)=ii/xii	(ivx)=viii/xii
1	-14700	-2206	695	-16211	-2802	0	-19013	-16211	2.003	1.365	3.368	4364	5645
2	-11049	-1614	695	-11968	-2183	11	-14151	-11957	1.637	1.207	2.844	3885	4975
3	-9053	-1279	695	-9637	-1814	32	-11451	-9605	1.435	1.112	2.547	3555	4496
4	-7284	-975	695	-7564	-1466	67	-9030	-7497	1.255	1.021	2.276	3201	3968
5	-5731	-702	695	-5738	-1141	118	-6879	-5620	1.096	0.935	2.032	2821	3386
6	-4382	-461	695	-4148	-847	188	-4995	-3960	0.959	0.856	1.815	2415	2753
7	-3228	-251	695	-2784	-586	273	-3370	-2511	0.841	0.783	1.625	1987	2074
8	-2260	-72	695	-1637	-368	365	-2005	-1272	0.744	0.719	1.462	1545	1371
9	-1471	76	695	-700	-195	467	-895	-233	0.665	0.664	1.328	1108	674
10	-855	193	695	33	-78	560	-45	593	0.603	0.619	1.222	700	37
11	-407	278	695	566	-16	636	550	1202	0.559	0.585	1.144	356	0
12	-125	332	695	902	-1	686	901	1588	0.532	0.563	1.094	114	0
13	-3	318	695	1010	0	625	1010	1635	0.520	0.554	1.073	3	0
14	0	318	695	1013	0	625	1013	1638	0.519	0.553	1.073	0	0

Note: MH = Secondary Pre-stressing Moment, TPM = Total Permanent Moment, Ms1 = Negative LLBM, Ms2 = Positive LLBM (all moments in t-m)

In above table secondary moment due to pre-stressing (MH) can be calculated using Theorem of Three Moments with first trial of pre-stressing forces required as explained in 2.1.2 and as calculated in next topic E.2.1.2.

5.2.1.2 Calculations of Secondary Moment due to Pre-stressing Using Theorem of Three Moments

Table 6 below shows calculations of Secondary Moment due to Pre-stressing Using Theorem of Three Moments:

Table 6 - Secondary Moment due to Pre-stressing

Section No	delta (P)	E	x	dx	delta(p)*e*x*dx
1	669.530	2.003	0.000	0.000	0
2	478.889	1.637	4.960	4.960	14565
3	528.499	1.435	8.100	3.140	14564
4	581.907	1.255	11.240	3.140	19458
5	633.098	1.096	14.380	3.140	23661
6	678.526	0.959	17.520	3.140	27018
7	528.847	0.841	20.660	3.140	21794
8	437.747	0.744	23.800	3.140	18366
9	407.828	0.665	26.940	3.140	17311
10	343.965	0.603	30.080	3.140	14798
11	241.577	0.559	33.220	3.140	10639
12	111.424	0.532	36.360	3.140	5105
13	2.795	0.520	39.500	3.140	136
			Total	694.104	187415

where, delta (P) = difference of pre-stressing force required at two consecutive sections.

Mab + 2 Mba + 2 k Mbc + k Mcb = Kab + k Kbc

Where: $K = \dfrac{6 \times P \times \int (ex\ dx)}{L^2} = Kab = Kbc$

And $k = \dfrac{Iab / Lab}{Ibc / Lbc} = 1$

Now,

Mab +2 Mba +2 k Mbc +k Mcb = Kab +k Kbc

i.e. Mab + 4 Mb + Mcb = 694.104 + 1×694.1044

i.e. Mab + 0 + Mab = 1388.209 tm

i.e. Mab=1388.21/2 = 694.104 tm say ≈695.000 tm.

5.2.2 Estimation of Cables Required and Curtailment Planning of Cables (Theoretically)

Cables used: 9T13 - 12.7 mm 7-ply (Class 2)

PRE-STRESSING FORCE PER CABLE =9×183700×0.00010197=168.5897 t.

FORCE PER CABLE AFTER LOSSES =0.765×168.5897×0.8=103.1769 ton.

Total No of Cables Required = Max F req / 103.1769

=5644.633 / 103.1769

=54.708 = Say 55 Numbers.

Table 7 below shows calculations for curtailment planning of cable (theoretically)

Table 7- Curtailment Planning of Cable Theoretically

Section	Fp (min)	No of Cables	Fp (total)	No of Cables	Maximum	No of Cables	No of Cables
No.		Required		Required	Force (F)	Required	Provided
1	4364.178	15	5644.633	19	5644.633	19	20
2	3884.525	13	4975.103	17	4975.103	17	18
3	3554.644	12	4496.214	15	4496.214	15	16
4	3200.535	11	3967.715	13	3967.715	13	14
5	2820.769	10	3385.808	11	3385.808	11	12
6	2414.890	8	2752.710	9	2752.710	9	10
7	1986.785	7	2074.184	7	2074.184	7	8
8	1545.336	5	1370.973	5	1545.336	5	6
9	1107.590	4	673.890	3	1107.590	4	5
10	699.762	3			699.762	3	4
11	355.796	2			355.796	2	3
12	114.219	1			114.219	1	2
13	2.795	1			2.795	1	1

Date Considered for design:-

Delta T	=	10^0 (degree)
Thermal Co-efficient	=	1.17E–05
E (Modulus of Elasticity)	=	3.16E+06
□sei	=	2.00E+00
Reduction Factor	=	0.4
Exp. / unit length = e	=	0.000117 (per unit length)

Table 8 below shows calculations of secondary moments due to Temperature Difference:

Table 8- Secondary Moments (Temperature Diff.)

Section	dx	h	I	(dx/h)	(dx/I)	M '	(M' *dx/I)
1	4.960	4.300	32.867	1.153	0.151	7943	1199
2	3.140	3.676	21.990	0.854	0.143	5367	766
3	3.140	3.323	16.981	0.945	0.185	4292	794
4	3.140	3.003	13.098	1.046	0.240	3371	808
5	3.140	2.715	10.119	1.156	0.310	2523	783
6	3.140	2.461	7.857	1.276	0.400	1818	727
7	3.140	2.240	6.161	1.402	0.510	1174	598
8	3.140	2.051	4.907	1.531	0.640	664	425
9	3.140	1.895	3.998	1.657	0.785	206	162
10	3.140	1.772	3.358	1.772	0.935	-125	-117
11	3.140	1.682	2.931	1.866	1.071	-410	-439
12	3.140	1.625	2.677	1.932	1.173	-573	-672
13	3.140	1.601	2.574	1.961	1.220	-692	-844
14	0.750	1.600	2.570	0.469	0.292	-695	-203
			Total	**19.021**	**8.055**		**3986**

Note- M'=DLBM+Secondary Pre-stressing Moment (ton-m)

$$m = \frac{e \times E \times \sum (dx/h)}{\sum (dx/I)} = 873.73 \text{ t-m}$$

$$M = \frac{\square sei \times \sum (M' \times dx/I)}{1 + \square sei \times \sum (dx/I)} = 131.9639 \text{ t-m}$$

Total Secondary Moments = 1005.69 t-m
Max Positive Moment at cantilever tip = 1638.00 t-m
Total Max. Positive Moment at cantilever tip (M) =1005.69+1638 =2643.69 t-m

Assuming tendons at a distance (d) = 0.08 m
Their eccentricity (e) = – (v'–d) = –0.84 m
Lever Arm =e + (r 2 / v') = 1.25 m

$$F\ req = \frac{M}{e + (r2/v')} = 2116.85\ ton$$

Pre-stressing Force to be provided per deck = 1058.424 ton

Cables used : 12T8 - 7.9 mm 7-ply (Class 1)
PRE-STRESSING FORCE PER CABLE=12×64500×0.00010197=78.9260 ton
FORCE PER CABLE AFTER LOSSES =0.765×78.92603×0.8 =48.3027 ton
Total No of Continuity Cables Required
= Max F req per deck / 48.30273
=1058.4245 ton / 48.30273
=22.09=Say 23 Nos. per deck for half bridge

5.2.3 Estimation of Cables Required and Curtailment Planning of Cables (Graphically)

Graphical representation of “Limiting zone for cable profile without and with curtailment of cables” and “Estimation of number of cables required at different sections and curtailment planning of cables” are shown in figure 13 and figure 14 respectively by considering R1=2000 t/m^2, R2=1320 t/m^2, R'1 and R'2= 0 t/m^2.

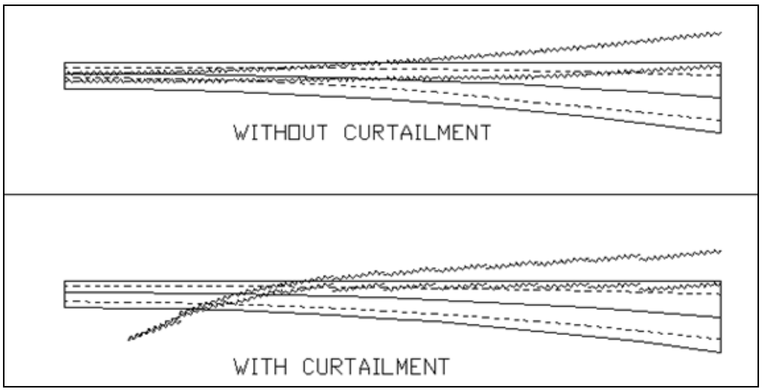


Figure 13: Limiting Zone (zigzag) [Graphically]

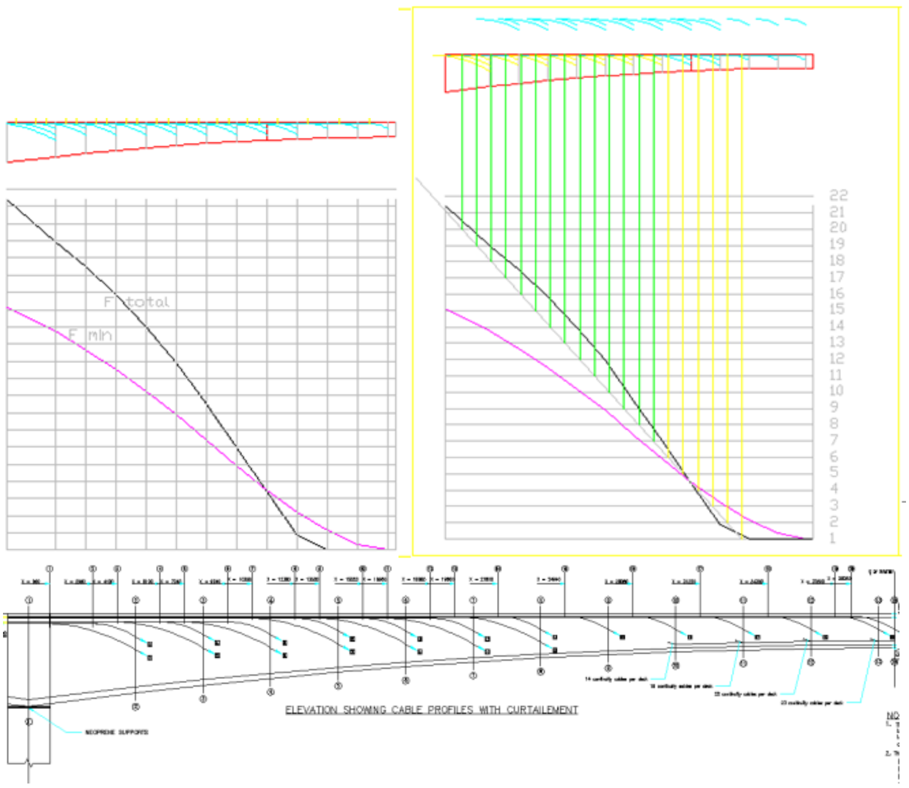


Figure 14: Curtailment Planning (Graphically)

5.3 Loss Calculations

5.3.1 Loss due to Slip & Friction (IRC-18:2000 Cl. 11.6,Table -5)

Values considered for k = 0.0030 and u = 0.200

$$z = kx + \mu\theta$$

$$\sigma_{po} = \sigma_{po} \, . \, e^{z}$$

First Segment Length	=4.96 m
All Middle Segments Lengths	= 3.14 m
End Segment Length	= 0.75 m
Total No of Segments	= 13 Nos.
No of Cables provided per web	= 20 Nos.

Table 9 below shows calculation of force after frictional losses.

Table 9- Calculation of force after frictional losses

Sec No / Cable No	0	1	2	3	4	5	6	7	8	9	10	11	12	13
1	115.8055	128.97111												
2	115.8055	128.97111												
3	114.7082	116.42784	128.97111											
4	114.7082	116.42784	128.97111											
5	113.489	115.19036	116.28058	128.97111										
6	113.489	115.19036	116.28058	128.97111										
7	112.5674	114.25488	115.33624	116.42784	128.971106									
8	112.5674	114.25488	115.33624	116.42784	128.971106									
9	111.3709	113.04049	114.11036	115.19036	116.280576	128.97111								
10	111.3709	113.04049	114.11036	115.19036	116.280576	128.97111								
11	110.0583	111.70825	112.76551	113.83278	114.910151	115.99772	128.97111							
12	110.0583	111.70825	112.76551	113.83278	114.910151	115.99772	128.97111							
13	109.4307	111.07123	112.12246	113.18365	114.254876	115.33624	116.42784	128.9711						
14	109.4307	111.07123	112.12246	113.18365	114.254876	115.33624	116.42784	128.9711						
15	108.2676	109.89068	110.93074	111.98064	113.040485	114.11036	115.19036	116.2806	128.97111					
16	107.2525	108.86037	109.89068	110.93074	111.980644	113.04049	114.11036	115.1904	116.28058	128.9711				
17	105.9885	107.57739	108.59556	109.62336	110.660895	111.70825	112.76551	113.8328	114.91015	115.9977	128.97111			
18	104.9948	106.56877	107.57739	108.59556	109.623364	110.66089	111.70825	112.7655	113.83278	114.9102	115.99772	128.9711		
19	46.06681	46.757411	47.199947	47.646671	48.0976234	48.552844	49.012373	49.47625	49.944519	50.41722	50.894393	51.37608	57.32049	
20	10.78561	10.947296	11.050906	11.155498	11.2610792	11.36766	11.475249	11.58386	11.693492	11.80417	11.915886	12.02866	12.14251	14.33012

Loss due to slip (As per IRC-18, 2000)

As per standard Manufacturer's recommendation 6 mm reverse slip of tendons is considered.

Figure 15 below shows the sample graph of slip loss.

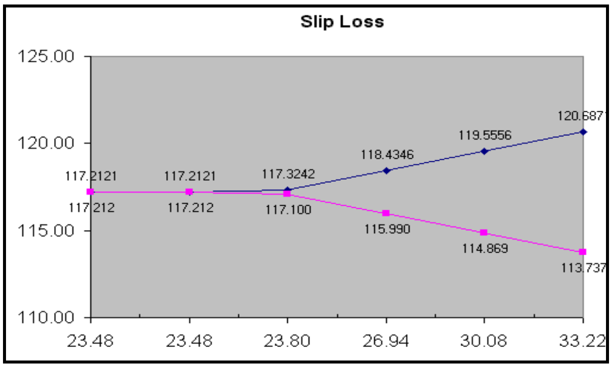


Figure 15: Sample graph showing slip loss.

Area of stress diagram < Es × slip < 1.95×104×6   < 117.00 Kg/mm

Slip Point is to be found out such that area of stress diagram remains less than 117 kg/mm.

5.3.2 Time dependent Losses

5.3.2.1 Loss due to Elastic Shortening, Relaxation of steel, Creep and Shrinkage (IRC-18)

Sample calculation at support section no.1 for gradual loss as explained in 3.2.1 is shown in Table 10 below.

Table 10- Sample calculation for gradual loss sec.1

Grade of Concrete	=	**M 40**	
Modulus of Elasticity	=	**31622.777**	
Stress at C.G. of cables	=	412.219	t/m^2
Loss due to Elastic Shortening	=	**135.480**	**ton**
Net Prestressing force (after elastic shortening)	=	6037.059	ton
Avg. stress in cables immediately after anchoring	=	113.270	kg/mm^2
Ultimate Tensile Strength	=	189.724	kg/mm^2
Ratio (Avg. stress / UTS)	=	0.597	
Relaxation Loss (as per table 4A IRC-18)	=	1.213	%
Relaxation Loss	=	**73.217**	**ton**
Taking 3.0 times higher as per SP - 33	=	**219.652**	**ton**
Net Prestressing force (after relaxation)	=	5963.842	ton
Stress at C.G. of cables	=	367.994	t/m^2
Shrinkage Strain (Cl: 11.3 Table-3 IRC-18)	=	0.000	
Creep Strain (Cl: 11.2 Table-2 IRC-18)	=	0.000	
Loss due to Creep and Shrinkage(bet. 7and 90 days)	=	**254.861**	**ton**
20% increase in loss	=	305.833	ton
Net Prestressing Force	=	5658.009	ton
Stress at C.G. of cables	=	**168.753**	t/m^2
Shrinkage Strain (Cl: 11.3 Table-3 IRC-18)	=	0.000	
Creep Strain (Cl: 11.2 Table-2 IRC-18)	=	0.000	
Loss due to Creep and Shrinkage(beyond 90 days)	=	**230.508**	**ton**
20% increase in loss	=	**276.610**	ton
Net Prestressing Force	=	5381.399	ton
Total Loss in Percentage	=	0.128	

5.3.3 Permissible Stresses in Concrete: (IRC: SP: 65-2005 Cl.5.3, I.R.C:18-2000 Cl. 7)

5.3.3.1 Permissible Temporary Stresses at Transfer of Concrete.

(a) Temporary Compressive stress = $0.50\times f_{cj}$ = 0.5×40 = 20 Mpa (Max. 20Mpa)
(b) Temporary Tensile stress = (Comp. stress)/10 = 20/10 = 2 Mpa.

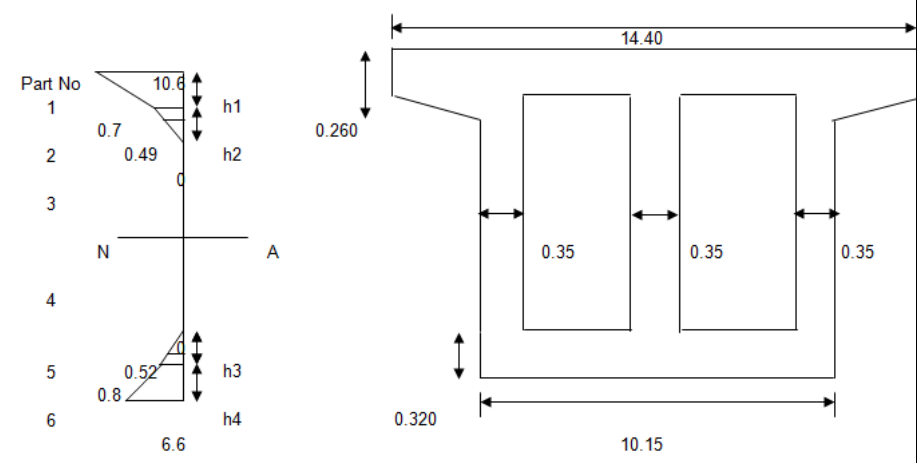
5.3.3.1.2 Permissible Stresses During Service.

(a) Compressive stress under service load = 0.33×40 = 13.20 Mpa.
(b) Tensile stress permitted during service = 0.

5.3.3.2 Temperature Stress (I.R.C:6-2000)

Temperature Stresses can be calculated as per IRC provisions as shown in figure 16 below.

SECTION 1



h1 = h4 = 0.2*h = 0.86 > 0.25 m

h1 = h4 = 0.25 m

h2 = h3 = 0.25*h = 1.075 > 0.20 m

h2 = h3 = 0.20 m

α = Coefficient of Thermal Expansion = 1.17E-05/ °C

E = 5000√F_{ck} = 3162278 t/m²

Area = 11.1290 m²

Zt = 15.196 m³

Zb= 15.379 m³

Part No.	Forces (Ton)	Distance of CG from NA (m)	Moment (t.m)
1	752.553	2.074	1561.098
2	7.925	1.901	15.067
3	1.666	1.830	3.047
4	1.313	-1.774	-2.329
5	49.571	-1.785	-88.463
6	347.371	-2.045	-710.290
Total	**1160.398**		**778.131**

Stress at Top of Girder = 53.061 t/m²

Stress at Bottom of Girder = 154.864 t/m²

Figure 16: Sample Temperature stress calculation

5.4 Check for Stresses

Stresses are to be checked for each stage of construction (i.e. after installation of each segment) as well as for service condition (i.e. after the span is made continuous).

5.4.1 Completion Stresses

Table 11 below shows calculations to check stress at different sections during construction i.e. “completion stresses”.

Table 11- Calculation of Completion Stress

Section 1																	
Section Already Concreted	Section being Concreted	No of Tension Tendon	Prestressing Force	Ecc. (e)	Area	Z top	Z bot	Stress due to Prestress f (top)	F cum(top)	f ' (bot)	f 'cum(bot)	DLBM at each stage	stress f	f '	Resultant stress @ Top	@ Bottom
1 2		6	231.532	1.893	11.129	15.196	15.379	148.936	148.936	-23.078	-23.078	311.914	-20.526	20.281	128.410	-2.797
1 2	2 3	6	231.532	1.893	11.129	15.196	15.379	148.936	297.873	-23.078	-46.156	798.011	-52.515	51.888	245.358	5.732
2 3	3 4	12	229.338	1.903	11.129	15.196	15.379	147.978	445.851	-23.307	-69.463	1478.674	-97.307	96.147	348.544	26.684
3 4	4 5	18	226.901	1.913	11.129	15.196	15.379	146.853	592.704	-23.502	-92.965	2333.395	-153.554	151.723	439.150	58.758
4 5	5 6	24	225.058	1.923	11.129	15.196	15.379	146.105	738.809	-23.750	-116.715	3345.476	-220.157	217.530	518.653	100.816
5 6	6 7	30	222.666	1.933	11.129	15.196	15.379	144.992	883.801	-23.932	-140.647	4500.609	-296.173	292.640	587.628	151.993
6 7	7 8	36	220.041	1.943	11.129	15.196	15.379	143.717	1027.518	-24.079	-164.726	5790.875	-381.082	376.536	646.436	211.810
7 8	8 9	42	218.787	1.953	11.129	15.196	15.379	143.330	1170.847	-24.368	-189.094	7210.964	-474.534	468.873	696.314	279.779
8 9	9 10	45	108.231	1.963	11.129	15.196	15.379	71.117	1241.964	-12.266	-201.360	8759.348	-576.429	569.553	665.536	368.193
9 10	10 11	48	107.216	1.973	11.129	15.196	15.379	70.662	1312.626	-12.360	-213.720	10438.283	-686.915	678.721	625.711	465.001
10 11	11 12	51	105.952	1.983	11.129	15.196	15.379	70.038	1382.664	-12.421	-226.141	12251.231	-806.220	796.603	576.444	570.462
11 12	12 13	54	104.959	1.993	11.129	15.196	15.379	69.589	1452.252	-12.509	-238.650	14210.346	-935.144	923.989	517.109	685.339
12 13	13 14	57	46.051	2.003	11.129	15.196	15.379	30.623	1482.876	-5.578	-244.229	14700.665	-967.410	955.871	515.465	711.642
13 14		60	10.782	2.003	11.129	15.196	15.379	7.170	1490.045	-1.306	-245.535	14700.665	-967.410	955.871	522.635	710.336
										Stress due to Secondary Moment					45.736	45.190
										Completion Stress					568.371	665.146

Section 2																
Section Already Concreted	Section being Concreted	No of Tension Tendon		ecce	A	Z top	Z bot	Prestress f	f cum	f '	f ' cum	dlbm	stress f	f '	Resultant stress upper	lower
		0			10.137	12.235	11.708	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2 3		6	225.900	1.537	10.137	12.235	11.708	152.017	152.017	-22.130	-22.130	116.333	-9.509	9.936	142.509	-12.194
2 3	3 4	6	225.900	1.537	10.137	12.235	11.708	152.017	304.034	-22.130	-44.260	447.535	-36.580	38.223	267.455	-6.037
3 4	4 5	12	223.499	1.547	10.137	12.235	11.708	150.949	454.984	-22.468	-66.728	971.087	-79.373	82.939	375.611	16.211
4 5	5 6	18	222.084	1.557	10.137	12.235	11.708	150.538	605.522	-22.894	-89.622	1668.280	-136.359	142.485	469.164	52.863
5 6	6 7	24	223.994	1.567	10.137	12.235	11.708	152.382	757.905	-23.665	-113.287	2523.169	-206.234	215.500	551.671	102.213
6 7	7 8	30	223.340	1.577	10.137	12.235	11.708	152.485	910.390	-24.168	-137.456	3525.464	-288.157	301.105	622.232	163.649
7 8	8 9	36	222.067	1.587	10.137	12.235	11.708	152.160	1062.550	-24.600	-162.055	4667.855	-381.532	398.675	681.018	236.619
8 9	9 10	39	109.853	1.597	10.137	12.235	11.708	75.541	1138.091	-12.451	-174.506	5946.817	-486.069	507.909	652.021	333.403

5.4.2 Check for Stresses (service condition)

Table 12 below shows calculations for check for stress at some sections under service condition:

Table 12- Check for Stress at Some Sections under Service Condition

Sec No	Description	P/M	BOT. OF BOX STRESS T/m2	BOT. OF BOX CUM.STR. T/m2	TOP OF BOX STRESS T/m2	TOP OF BOX CUM.STR. T/m2
1	1 Stress after completion of structure		665.15	665.15	568.37	568.37
	2 Gradual Loss	-254.86	10.29	675.44	-56.49	511.88
	3 SIDL	2206.00	143.44	818.88	-145.17	366.71
	4 Grad. Loss	-230.51	9.31	828.18	-51.09	315.61
	5 Live load	2802.00	182.19	1010.38	-184.39	131.22
	6 Diff. temp.		-154.86	855.51	-53.06	78.16
	7 20% Add. Loss	-97.07	3.92	859.43	-21.52	56.64
2	1 Stress after completion of structure		664.47	664.47	555.62	555.62
	2 Gradual Loss	-238.32	9.82	674.29	-55.41	500.21
	3 SIDL	1614.00	137.85	812.13	-131.92	368.29
	4 Grad. Loss	-246.09	10.14	822.27	-57.21	311.08
	5 Live load	2183.00	186.45	1008.72	-178.43	132.65
	6 Diff. temp.		-165.49	843.23	-66.89	65.76
	7 20% Add. Loss	-96.88	3.99	847.22	-22.52	43.23
3	1 Stress after completion of structure		655.62	655.62	495.35	495.35
	2 Gradual Loss	-206.45	8.58	664.21	-49.39	445.96
	3 SIDL	1279.00	130.12	794.33	-120.15	325.81
	4 Grad. Loss	-205.67	8.55	802.88	-49.21	276.60
	5 Live load	1814.00	184.55	987.43	-170.40	106.20
	6 Diff. temp.		-172.04	815.38	-76.25	29.95
	7 20% Add. Loss	-82.42	3.43	818.81	-19.72	10.23

5.4.3 Check for Shear Stress calculation

(A) I.R.C: 18-2000, Clause No 14

(B) IRC: SP: 65-2005, Clause No. 5.4.2.1

Table 13 below shows calculations for check for shear stress.

Table 13- calculations for check for shear stress.

Description \ Section	1	2	3	4	5	6	7	8	9	10	11	12	13
Width of the web = b (in mm)	1050	1050	1050	1050	1050	1050	1050	1050	1050	1050	1050	1050	1050
Overall Depth "D" (in mm)	3440.00	2940.45	2658.11	2402.06	2172.31	1968.84	1791.67	1640.79	1516.20	1417.90	1345.89	1300.18	1280.75
Clear cover (in mm)	**50**	**50**	**50**	**50**	**50**	**50**	**50**	**50**	**50**	**50**	**50**	**50**	**50**
Grade of Concrete= fck(N/mm²)	40	40	40	40	40	40	40	40	40	40	40	40	40
Sheathing diameter (mm) =	75	75	75	75	75	75	75	75	75	75	75	75	75
Net Prestressing Force= **P** (ton)	5381.40	5203.54	4647.21	4083.10	3520.46	2955.23	2383.57	1810.12	1265.29	959.51	661.12	370.39	117.84
Area of Section = **A** (in m²)	11.129	10.137	9.576	9.067	8.611	8.206	7.854	7.555	7.307	7.112	6.969	6.878	6.839
Moment of Inertia = **I** (in m⁴)	32.867	21.990	16.981	13.098	10.119	7.857	6.161	4.907	3.998	3.358	2.931	2.677	2.574
Section Modulus Top = Z_t (m³)	15.196	12.235	10.645	9.257	8.055	7.024	6.153	5.431	4.848	4.399	3.043	3.872	3.786
C.G. from top = Y_t (in m)	**2.163**	**1.797**	**1.595**	**1.415**	**1.256**	**1.119**	**1.001**	**0.904**	**0.825**	**0.763**	**0.719**	**0.692**	**0.680**
Eccentricity from C/L=**e**(m)	**2.003**	**1.637**	**1.435**	**1.255**	**1.096**	**0.959**	**0.841**	**0.744**	**0.665**	**0.603**	**0.559**	**0.532**	**0.520**
Dead load SF = **DLSF** (ton)	675	563	498	438	381	327	276	228	182	137	94	52	4
Dead load BM = **DLBM** (tm)	14700	11049	9053	7284	5731	4382	3228	2260	1471	855	407	125	3
SIDL Shear = **SIDLSF**(ton)	138	121	110	100	89	78	67	56	46	35	24	13	1
SIDL Moment= **SIDLBM** (tm)	2206	1614	1279	975	702	461	251	72	-76	-193	-278	-332	-318
LL Maximum shear **LLSF** (ton)	171	166	160	152	144	134	122	110	99	87	75	63	6
LL Corresponding BM **LLM**(tm)	**2802**	**2183**	**1814**	**1466**	**1141**	**847**	**586**	**368**	**195**	**78**	**16**	**1**	**0**
LL Corresponding SF **LLSF**(ton)	**171**	**166**	**160**	**152**	**144**	**134**	**122**	**110**	**99**	**87**	**75**	**63**	**6**
LL Maximum Moment **LLM** (tm)	2802	2183	1814	1466	1141	847	586	368	195	78	16	1	0
Foot path LL SF=**FPLLSF** (ton)	16.301	14.292	13.021	11.749	10.477	9.206	7.934	6.662	5.391	4.119	2.847	1.575	0.304
Foot path LL BM=**FPLLM**(tm)	218.708	142.836	99.954	61.066	26.170	-4.732	-31.641	-54.557	-73.480	-88.410	-99.346	-106.290	-109.240
Ultimate Force for Max.shear (ton)	1433.28	1260.41	1150.80	1044.12	939.02	832.79	725.43	621.46	523.78	423.17	324.82	227.59	20.56
Corrosponding Max. BM (tm)	27359.57	20604.17	16818.21	13400.66	10343.65	7653.47	5322.70	3363.71	1773.12	568.95	-254.89	-720.45	-842.13
Corrosponding SF (ton)	1433.28	1260.41	1150.80	1044.12	939.02	832.79	725.43	621.46	523.78	423.17	324.82	227.59	20.56
Maximum moment (tm)	27359.57	20604.17	16818.21	13400.66	10343.65	7653.47	5322.70	3363.71	1773.12	568.95	-254.89	-720.45	-842.13
Section Uncracked:-													
Compressive stress due to prestress f_{cp}=P/A (in t/m2)	483.55	513.34	485.31	450.32	408.85	360.11	303.47	239.60	173.16	134.92	94.87	53.85	17.23
Max. principle tensile stress = f_t=0.24*(fck)$^{0.5}$	1.52	1.52	1.52	1.52	1.52	1.52	1.52	1.52	1.52	1.52	1.52	1.52	1.52
Vco =0.67*b*d*(ft²+0.8*fcp*ft)$^{0.5}$ (in T)	666.61	580.88	513.53	450.89	393.37	340.78	292.83	249.41	211.18	186.24	165.07	147.16	133.36
Vertical Component of Cable force (in T)	0.00	237.01	233.82	229.88	227.54	228.28	228.49	229.81	114.20	113.10	111.32	109.40	45.62
Uncracked Capacity Vc (T) (with 0.9 reduction factor)	**599.95**	**736.10**	**672.62**	**612.69**	**558.81**	**512.16**	**469.19**	**431.30**	**292.84**	**269.40**	**248.75**	**230.91**	**161.08**
Section cracked in flexure													
db=Dist bet extreme compression & Tendon CG (m)	3.28	2.78	2.50	2.24	2.01	1.81	1.63	1.48	1.36	1.26	1.19	1.14	1.12
f_{pt} = Pe/Z_b in T/m²	709.30	696.41	626.49	553.50	479.17	403.37	325.96	247.84	173.44	131.59	121.50	50.85	16.18
Mt=(0.37*fck$^{0.5}$+0.8*fpt)*I/y (N-mm)	1.23E+11	9.81E+10	7.93E+10	6.34E+10	5.03E+10	3.95E+10	3.08E+10	2.37E+10	1.82E+10	1.50E+10	1.36E+10	1.07E+10	9.36E+09
Vcr=0.037*b*db*(fck)$^{0.5}$+Mt/M*V (ton) (with 0.9 reduction factor)	**652.49**	**599.85**	**541.98**	**493.05**	**454.35**	**426.06**	**412.25**	**425.62**	**513.06**	**1032.10**	**75.19**	**72.54**	**71.41**
Vcr =0.037*b*db*(fck)$^{0.5}$+Mt/M*V	**652.49**	**599.85**	**541.98**	**493.05**	**454.35**	**426.06**	**412.25**	**425.62**	**513.06**	**1032.10**	**75.19**	**72.54**	**71.41**
Design Shear Force V-Vc (in T)	**833.33**	**660.56**	**608.83**	**551.07**	**484.67**	**406.73**	**313.19**	**195.83**	**116.73**	**40.67**	**249.63**	**155.05**	**-50.85**
Ultimate Shear Force V (in T)	**1433.28**	**1260.41**	**1150.80**	**1044.12**	**939.02**	**832.79**	**725.43**	**621.46**	**523.78**	**423.17**	**324.82**	**227.59**	**20.56**
Max. Shear Stress (in Mpa)	4.70	4.70	4.70	4.70	4.70	4.70	4.70	4.70	4.70	4.70	4.70	4.70	4.70
Max shear=Tv*(b-0.67*dia of duct)*0.8*db (inT)	1571.46	1332.12	1196.85	1074.18	964.10	866.62	781.74	709.45	649.76	602.66	568.16	546.26	536.95
Max shear>= Ultimate shear	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**	**ok**
Reinforcement along the web (in cm²/m)	66.79	62.09	63.43	63.66	62.05	57.59	48.86	33.45	21.63	8.08	33.15	33.70	11.08
Diametre of Bar (in mm)	**12**	**12**	**12**	**12**	**12**	**12**	**12**	**12**	**12**	**12**	**12**	**12**	**12**
No. of Legged for stirrups	**6**	**6**	**6**	**6**	**6**	**6**	**6**	**6**	**6**	**6**	**6**	**6**	**6**
Area of Stirrups (in mm²)	678.24	678.24	678.24	678.24	678.24	678.24	678.24	678.24	678.24	678.24	678.24	678.24	678.24
Spacing of Stirrups req.(mm)	101.55	109.23	106.93	106.54	109.31	117.76	138.82	202.77	313.54	839.54	204.59	201.27	612.20
Stirrups Spacing provided (mm)	100	100	100	100	100	100	100	200	200	200	200	200	200

5.4.4 Check for the Ultimate Strengths

(A) I.R.C: 18-2000, Clause No 13

(B) IRC: SP: 65-2005, Clause No. 5.5.2

Calculations for check for ultimate strength at are tabulated below in Table 14.

Table 14- Check for ultimate strength at various sections

Sample Calculation for Support Section:-

Dead load Moment	dlm	**14700.00** tm
Super Imposed dead load moment	sidlm	**2206** tm
Live load moment with Impact	llm	**2802.00** tm
Live load Torsional Moment	tllm	**356.9992** tm
As = Area of high tensile steel (total area of strands used)	As	**1776.6** cm²
Db=Depth of beam from the max. comp.edge to cg of steel tendon	Db	**328** cm
fp = Ultimate Tensile Strength for steel	fp	**145.1391** kg/cm²
b = Web thickness	b	**105** cm
Bf = Width of flange	Bf	**1440** cm
t = thickness of flange 0.5*100*((0.5*(0.22+0.26))+0.22) =	t	**23** cm
fck = Charectristic Strength of Concrete =	fck	**400** kg/cm²

Condition of exposure Normal 1

Ultimate load moment = 27116.048tm
(1.25*14700+2*2206+2.5*(2802+356.999195348467))*0.9 2.712E+09kgcm

Ultimate moment of resistance :-
Calculation of Ultimate Strength

Failure by Yield of Steel :	**Failure by Crushing of Concrete without Yield of Steel:**
Mult = 0.9 * db * As * fp * 0.95	Mult=0.176*b*Db^2*fck + 2/3*0.8*(Bf-b)*(Db-t/2)*t*fck*0.95
Mult = 7231261178 kgcm 72312.61178 tm 0.9*328*1776.6*145.139101902717*100* 0.95	Mult = **2.725E+09** kgcm 27250.394 tm 0.176*105*328^2*400+2/3*0.8*(1440-105)*(328-23/2)*23*400*0.95
72,312.61>=27116.0481895341 **safe**	27,250.39>=27116.0481895341 **safe**

Description / Section	1	2	3	4	5	6	7	8	9	10	11	12	13
Dead Load mom (tm)	14700.00	11049	9053	7284	5731	4382	3228	2260	1471	855	407	125	3
SIDL moment in tm	2206.00	1614	1279	975	702	461	251	72	-76	-193	-278	-332	-318
Live Load mom (tm)	2802.00	2183	1814	1466	1141	847	586	368	195	78	16	1	0
L.L. Torsional mom tm	357.00	350.6261	341.985	330.6877	318.7417	306.9552	293.7209	279.5466	265.1509	251.5402	239.7867	230.892	66.11861
A_s=Area of Steel in cm²	1776.60	1598.94	1421.28	1243.62	1065.96	888.30	710.64	532.98	444.15	355.32	266.49	177.66	88.83
D_b=depth from comp edge to tendon	328	278.0448	249.811	224.2064	201.2308	180.88	163.17	148.08	135.62	125.79	118.59	114.02	112.07
f_p = UTS of Steel kg/cm²	145.14	145.14	145.14	145.14	145.14	145.14	145.14	145.14	145.14	145.14	145.14	145.14	145.14
b = Web thickness (cm)	105.00	105.00	105.00	105.00	105.00	105.00	105.00	105.00	105.00	105.00	105.00	105.00	105.00
B_f = Width of flange-cm	1440	1440	1440	1440	1440	1440	1440	1440	1440	1440	1440	1440	1440
t_f=thickness of flange(cm)	23	23	23	23	23	23	23	23	23	23	23	23	23
f_{ck} Grade of Conc in kg/cm²	400	400	400	400	400	400	400	400	400	400	400	400	400
M_{ult}	2.71E+09	2.1E+09	1.73E+09	1.4E+09	1.10E+09	8.36E+08	6.06E+08	4.13E+08	2.83E+08	2.05E+08	1.53E+08	1.26E+08	72454187
Mult (steel)= 0.9*db*As*fp*0.95	7.23E+09	5.52E+09	4.41E+09	3.46E+09	2.66E+09	1.99E+09	1.44E+09	9.79E+08	7.47E+08	5.55E+08	3.92E+08	2.51E+08	1.24E+08
(conc)= 0.176*b*Db^2*fck+ 2/3* 0.8*(Bf-b)*(Db-t/2)*t*fck*0.95	2.73E+09	2.2E+09	1.92E+09	1.68E+09	1.47E+09	1.28E+09	1.13E+09	1E+09	9.02E+08	8.22E+08	7.65E+08	7.29E+08	7.14E+08
	safe	**safe**	**safe**	**safe**	**safe**	**safe**	**safe**	**safe**	**safe**	**safe**	**safe**	**safe**	**safe**

6 CONCLUSIONS

The objective of this paper was to spread knowledge of the design of Prestressed Segmental Box type super structure by cantilevering (as per provisions of Indian codes) and to show how to automize the same. Effort made was to enhance understanding of how to make the design process of segmental bridge somewhat easy by providing automization with the use of the programming languages such as Auto LISP and by interlinking the programs. Important features of this paper are as follows:

- During the design main emphasis is given to develop an automized design process by interlinking of programs used (this is achieved by using output of one program as an input for the next program module to be used), and to generate the outputs in tabular formats so that one can easily check the design and minor changes if anywhere required could be carried out easily. Outputs are generated in tabular formats in the form of submission of document.

- Most economical and safe section can be achieved by taking numbers of trials with minimum of time. This type of atomization is most useful in all design offices.
- All input data can be prepared with the use of thumb rules stipulated and by taking number of trials with minimum of time.
- This design could be best employed for bridge construction at difficult climate locations and for the construction of flyovers without disturbing the traffic or of perennial river bridges.
- It could be easily concluded that this option of segmental box girder is more economical compared to conventional precast girders for 80.5 m span by giving due consideration to the followings:
- Lower capacity cranes will be required for the construction as the weight of each segment is very less compared to the conventional girder.
- The benefits of cable curtailments could be achieved, which will reduce the overall cost of the structure significantly.

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