International Journal of Science and Research (IJSR)
ISSN (Online): 2319-7064
Index Copernicus Value (2013): 6.14 | Impact Factor (2015): 6.391

Assessment of Stress Loss Evaluation Capability of Design Software and IRC Codes in Post-Tensioning Members

Shan B L[1], Shobha Elizebeth Thomas[2]

[1]P G Scholar, Department of Civil Engineering (Structural Engineering), Sree Buddha College of Engineering, Pattoor, Alappuzha, Kerala

[2]Assistant Professor, Department of Civil Engineering (Structural Engineering), Sree Buddha College of Engineering, Pattoor, Alappuzha, Kerala

Abstract: ***Post tension members with curved cable profile are generally used for longer spans. The curved cable profile improves the shear resistance of the girders. Due to the material property, there will be losses in presress applied initially. Commonly the prestress losses are categorized under two groups. Losses due to friction between cables and materials, due to elastic shortening of concrete, due to wobbling effect and due to anchorage slip are the immediate losses. Creep and shrinkage of concrete and relaxation of stress in steels are the time dependent losses. In Indian Practice the stress losses are calculated base on different codal provisions such as IS 1343-1980, IRC 18-2000 and IRC 112-2010. In this study the effectiveness of the stress loss calculated manually by IRC 112-2010 and design software is compared. In this thesis a prestress concrete bridge girder of 10m span with single curved cables profile is considered for the study. The stress losses are calculated with one and both end stressing. The total losses were calculated using the Bridge design software and IRC112-2011 with same coefficients and dimensions. When the results were compared the relaxation losses was is obtained same from both manual calculation and software. The relative humidity of materials is automatically assed by the software, but it failed to provide a critical stress loss variation. When the losses due to shrinkage and creep with normal calculation and that obtained from software about 10% variation is identified. This causes the major variations in stress loss calculation. For the immediate stress losses such as friction both methods showed the same values. Usually in manual calculation the elastic shortening loss is consider as zero for single cable. But when analysis in software it is providing a value for the loss in single cable. Due to the limitation of software implemented optimization algorithms, the software generate suboptimal design compared to manual design. When those results were compared, it is necessary to improve the efficiency of the Design software.***

Keywords: Design software, Post-tension, Stress lose, Tendons

1. Introduction

Now a days there are many Bridge design and analysis software are available in the market, In olden times the designers use the manual methods for the design purpose. Now Because of time effectiveness designers prefer to use design software than the manuals. Therefore it is necessary to check the effectiveness of design software's available in the market. Calculation of stress losses by means of design software are tedious process. In these days also stress losses are mainly calculated by manual methods even though there are many software are available in the market for stress loss calculation. In this study effectiveness of the stress loss by means of manual methods and software method on a pre stressed concrete bridge is checked. In prestressed concrete, estimation of losses plays vital role in arriving at the residual prestress.

2. Principles of Prestressing

The functions of prestressing are to place the concrete structure under compression in those regions where the load causes tensile stress. Tension caused by the load will first have to call off the compression induced by the prestressing before it may crack the concrete.

Since the tensile strength of the concrete is low, a homogeneous concrete beam has very little flexural strength. To offset these deficiencies, steel reinforcement are provided near the bottom of simple beams to carry tensile stresses. However, a substantial region of concrete below the neutral axis merely retains the reinforcement in that position, but this tensile strength is neglected in computation for the flexural strength in case of reinforced concrete beams. If the tensile reinforcement of the beam is subjected to tensile stresses before applying the external loads, then compressive stresses were induced in the concrete of the beam (and it is done by prestressing). Usually the tensile stresses in the concrete caused due to the external loads are totally absorbed or counteracted by compressive stresses in concrete, resulting from prestressing reinforcement. The concrete, therefore, is being used well in resisting tensile stresses formed by external loads rather than being ignored as in case of reinforced concrete.

Prestressed concrete can be applied to about all concrete constructions where ordinary reinforced concrete is commonly used. But due to high cost or prestressing and good quality material used, its use is made under unusual condition, particularly for the precast members. In addition to structural precast members, viz., joists, beams, slabs, columns, girders, etc, prestressed concrete are used for the framed multi-storeyed buildings. A large variety of industrial structures such like silos, roof trusses, water tanks, piles, pipes, nuclear power stations, factories, steel plants, electric sub-stations, etc can also be built in the prestressed concrete.

3. Scope of Study

- To reduce the time involved in designing the structures.
- To find the dependability of Bridge Design software over the tedious manual designing.
- To evaluate the performance of Bridge design software.
- To develop a better design methodology combines the manual and the Bridge design software.

4. Objective of Study

- To study the IRC different code manual.
- Stress loss calculation of girder by using IRC 112-2010.
- Simulation of different tendon profile on girder by Bridge design software..
- To compare the effectiveness of Bridge design software with manual calculation.

5. Material specifications

For the comparison the grade of concrete is taken as M40 and the Partial factor on strength of concrete γ_c is taken as 1.5. The prestressing time after concreting is 28days as per IRC112:2010.The pre-stressing steel Type and dia. of strands 7ply-12.7mm-II of low relaxation type having the dia of 12.7mm, the other details are taken from IRC: 112:2011and the breaking load of each strand F_{pu} is 183.71kN.

In Reinforcing steel the Characteristic strength of longitudinal steel f_{yk} is 415 N/mm^2. Partial factor on strength of steel γ_s is 1.15 is taken from Cl.6.2.2 of IRC: 112:2011.

Prestressing force at center after friction and slip loss in cable 1and 2 is 157t and the Prestressing force at center after friction and slip in cable 3 is taken as 131t.

The cross section of the deck and girder specifications are shown in the figure 1 and 2.

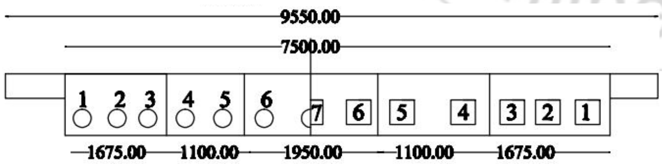

Figure 1: Cross section of slab section.

6. Loss Calculation

The Vertical and Horizontal profile of cables are shown in figure 2 and 3. It is having the profile eccentricity of 0.52m from the neutral axis.

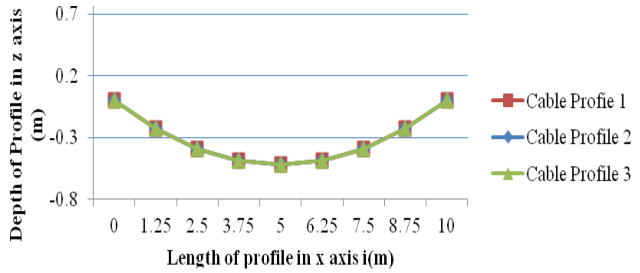

Figure 2: Vertical single tendon profile of 10 m span

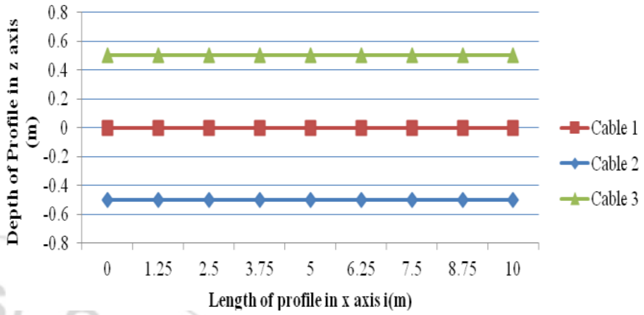

Figure 3: Horizontal single tendon profile of 10 m span

Table 1: Details of Prestressing Cables

Cable No	*No of Strands*	*Dia of Strands (mm)*	*Area of Strands (mm^2)*	*Jack end stress (N/mm^2)*	*Jack end Force (kN)*	*Active*
C1	10	12.7	98.8	1406	1389	1
C2	10	12.7	98.8	1406	1389	1
C3	10	12.7	98.8	1406	1389	1

The calculation of post tensioning stress losses are manually calculated by IRC 112:2010.

a) Loss due to Friction and Wobbling Effect.
[From Table 7.1 of IRC: 112-2011]
Coefficient of Friction k =0.002
Wobble coefficient μ =0.17
Deviation angle of cable in Y-dir. b/w support & mid-span θ_h =0.045 rad
Deviation angle of cable in Z-dir. b/w support & mid-span θ_v =0.000 rad
Gross angle of deviation $\sqrt{(\theta_h^2+\theta_v^2)}$ =0.045 rad
Intial prestress at end= 1406 N/mm^2
[From Cl.7.9.3.2(2) of IRC:112-2011}
Stress at mid-span $P_x=P_i\,e^{-(\mu\theta+kx)}$ =1381 N/mm^2.
Loss of stress at mid-span =25 N/mm^2.

b) Loss due to Slip at Anchorages.
Anchorage slip S = 6 mm
[From Cl.7.9.3.2 (3) of IRC: 112-2011]
Slope of stress profile $\beta=P_{L/2}\,(e^{-(\mu\theta+kx)}-1)/(L/2)$ = 0.0028.
Slip length $L_s=\sqrt{(S.Es/\beta)}$ =20.46m.
Stress at Jack end due to slip loss =1406-2x20460x0.0028= 1292 N/mm^2.
Stress at the end of slip length from jack = 1406-20460x0.0028 =1349 N/mm^2.
Loss of stress at mid-span =84 N/mm^2.

c) Losses due to Elastic Deformation of Concrete

Increase in concrete stress at mid-span point of Cable-4:

Due to stressing Cable-1= $P_1/A_n+P_1.e_{yp4.}\ e_{yp1}/I_n$= 4.2 N/mm^2.

Due to stressing Cable-2= $P_2/A_n+P_2.e_{yp4.}\ e_{yp2}/I_n$= 4.2 N/mm^2.

Due to stressing Cable-3= $P_3/A_n+P_3.e_{yp4.}\ e_{yp3}/I_n$= 4.2 N/mm^2.

Gross stress increase in concrete $f_{c4,es}$ =13Mpa.

Bending moment due to D.L.=232kMm.

Fibre stress at the level of Cable-4 due to D.L.=3.4 N/mm^2.

Net fibre stress at the level of Cable-4 due to D.L. $f_{c4,es}$=13-3.4= 9.6 N/mm^2.

Stress loss due to the elastic shortening $f_{c1,es}.(E_{sp}/E_c)$ =57 N/mm^2.

Total Immediate loss of prestress = 25+84+57 =166N/mm^2.

d) Losses due to Relaxation of Steel

[From Cl.7.9.3.3 of IRC:112-2011]

Initial pre-tress at mid-span = 1406 N/mm^2.

Immediate losses of prestress at mid-span =166 N/mm^2.

Net initial prestress at mid-span f_p =1240 N/mm^2.

=0.667f_{yp}.

Nominal relaxation ρ_{1000} = 2.09%.

[From Table 6.2, Cl.A2.11] k=0.143.

at stress transfer (1 day) $\rho_1=\rho_{1000}$ (1 x24/1000)^k = 1.22%.

at stress transfer (3 day) $\rho_3=\rho_{1000}$ (3 x24/1000)^k = 1.43%.

at opening to traffic (180 days) $\rho_{180}=\rho_{1000}$ (180 x24/1000)^k = 2.57%.

Relaxation at Long term (ꝏ) 3xρ_{1000} for 0.7f_{yp} = 7.5%.

Loss at stress transfer = 1240x1.22/100 = 15 N/mm^2.

Relaxation loss at construction =1240x1.43/100 =18 N/mm^2.

Loss at opening to traffic =1240x2.57/100 =32 N/mm^2.

Relaxation loss at Long term (ꝏ)=1240x7.5/100 =93 N/mm^2.

e) Losses due to Shrinkage of Concrete

[From Cl.6.4.2.6 of IRC:112-2011]

Cross Section area of concrete, A_c = 811398 mm^2.

Perimeter of girder exposed to atmosphere, u =1650mm.

Notional size of member, h_0= $2A_c/u$ = (2x811398 / 1650) =984mm.

[From Table 6.7] Value of k_h = 0.70.

Coefficients for cement, α_{ds1}= 4

α_{ds2}= 0.12.

[From Table 6.5 of IRC:112-2011]

Mean compressive strength, f_{cm} =40+10 =50 N/mm^2.

f_{cm0} = 12.5 N/mm^2.

[From Table 14.1 of IRC:112-2011]

Relative humidity of Ambient environment, RH =80%.

RH_0 =100%.

Age of concrete at the end of curing (shrinkage starts) t_s= 7days.

Unrestrained shrinkage value,

ε_{cd0} =0.85x((220+110.α_{ds1}).$e^{(-\alpha}$$_{ds2}$.$f_{cm}/f_{cmo}$))x1.55)1-$(RH/RH_0)^3$ x10^{-6}.= 0.000263

β_{ds}= $(t-t_s)/((t-t_s)+ 0.04h_0^{3/2})$.

Drying shrinkage strain,ε_{cd} = $\beta_{ds}.k_h.\varepsilon_{ds0.}$

Autogenous shrinkage strain at 't' $\varepsilon_{as(t)}$= (2 f_{ck} – 25) x 10^{-6}.

β_{as}= 1-$e^{(-0.2.t^{\wedge}0.5)}$.

Autogenous shrinkage strain, ε_{ca} = $\beta_{as}.\varepsilon_{as(\mu)}$.

Total shrinkage strain, ε_{cs}= ε_{cd} + ε_{ca}.

Stress loss due to shrinkage strain of concrete,$\Delta\sigma_s$ = $\varepsilon_{cs}.E_{ps}$.

Table 2: Details of Shrinkage loss calculation

		At 1st stage prestressing	*At full stress transfer*	*At construction*	*Long term*
t $t-t_s$	day day	8 1	29 22	31 24	ꝏ ꝏ
β_{as} β_{ds}		0.43 0.00	0.66 0.02	0.67 0.02	1.00 0.98
ε_{as} ε_{cd} ε_{cs}	10^{-6}. 10^{-6}. 10^{-6}.	24 00 24	36 4 40	37 4 41	55 180 235
$\varepsilon_{cs}.E_{ps}$	N/mm^2	5	8	8	46

f) Losses due to Creep of Concrete

Age of concrete in years at the moment considered t=100years.

Age of concrete at loading in days t_0 =15days.

[From Table 14.1 of IRC: 112-2011]

Relative humidity RH=80%.

Notional size of member, h_0= (2x811398 / 1650) =984mm.

[From Table 6.5 of IRC: 112-2011]

Mean compressive strength, f_{cm}= 50 N/mm^2.

$\beta(f_{cm})$ =18.78/$\sqrt{fcm}$=2.7.

$\beta(t_0)$=1/(0.1+$t_0^{0.2}$) = 0.55.

Coefficients allowing for concrete strength,

α_1= $(43.75/f_{cm})^{0.7}$=0.91.

α_2= $(43.75/f_{cm})^{0.2}$=0.97.

α_3= $(43.75/f_{cm})^{0.5}$=0.94.

Factor allowing for RH, φ_{RH} = (1+(1-(RH/100)α_1 /(0.1$h_0^{0.33}$))α_1= 1.16.

Notional creep coefficient φ_0 = φ_{RH} $\beta(f_{cm}).\beta(t_0)$= 1.69.

Coeff. dependent on RH β_H = 1.5[1+(0.012RH)^{18}]h_0 +250$\alpha_3$$\leq$ 1500α_3= 1403.

Facto allowing for concrete age at loading, $\beta_c(t,t_0)$= $((t-t_0)/(\beta_H+t-t_0))^{0.3}$=0.99.

Creep coefficient $\varphi(t,t_0)$=φ_0. $\beta_c(t,t_0)$= 1.67.

[From Cl. 6.4.2.7(2) IRC:112-2011]

As the maximum temperature is greater than 40 ° c the creep coefficient may be increased by 10% in absence of accurate data.

Creep coefficient = 1.84.

Table 3: Details of Creep losses

		At 1st stage prestressing	*At full stress transfer*	*At construction*	*Long term*
t $t-t_0$	day day	15 0	29 14	31 16	ꝏ ꝏ
β_c		0.00	0.25	0.26	0.99
$\varphi(t,t_0)$		0.00	0.46	0.48	1.85

Due to stressing Cable-1: $P_1/A_n+ P_1.e_{yp4}.e_{yp1}/I_n$ = 1.6 + 2.6 =4.2 N/mm^2.

Due to stressing Cable-2: $P_2/A_n+ P_2.e_{yp4}.e_{yp2}/I_n$ = 1.6 + 2.6 =4.2 N/mm^2.

Due to stressing Cable-3: $P_3/A_n+ P_3.e_{yp4}.e_{yp3}/I_n$ = 1.6 + 2.6 =4.2 N/mm^2.

Gross stress at level of Cable-4 = 12.4 N/mm^2.

Bending moment due to D.L. of girder =232KNm.

Bending moment due to SIDL =5.4 N/mm^2.

Net stress at the level of Cable-4 at construction, σ_{c1}=12.4 - 5.4 =7.0 N/mm^2.

Net stress at the level of Cable-4 at opening to traffic, σ_{c2}= 4.6 N/mm^2.
Net stress at the level of Cable-4 at long term, σ_{c3}=4.6 N/mm^2.
Secant modulus of elasticity of concrete E_{cm}=33 x 10^3 N/mm^2.
Tangant modulus of elasticity of concrete E_c = 1.05 x E_{cm} =34.70 x 10^3 N/mm^2.
Loss of prestress at construction, $\Delta\sigma_{cr}(t_1) = \varphi(t_1)\ \sigma_{c2}E_p/E_c$ = 19.00 N/mm^2.
Loss of at opening of traffic, $\Delta\sigma_{cr}(t_2) = \Delta\sigma_{cr}(t_1)+(\varphi(t_2)- \varphi(t_1))\ \sigma_{c2}E_p/E_c$ = 32.00 N/mm^2.

Loss of prestress at long term, $\Delta\sigma_{cr}(t_3) = \Delta\sigma_{cr}(t_1)+(\varphi(t_3)- \varphi(t_1))\ \sigma_{c3}E_p/E_c$ = 55.00 N/mm^2.

Total losses of prestress at Long term

93+46+55 = 194 N/mm^2

7. Results and Discussions

The stress evaluation capability checked by IRC 112:2010 and Bridge Design software, but the results are not same.

When these both results were compared the total stress losses from IRC 112:2010 is 21.23% and the losses from Bridge design software is 26.34%. The various stress losses in each points are shown in below table.

Table 4: Single profile stress variation due to one end stressing

Length	Intial Stress (N/mm^2)	Stress losses from IRC 112:2010		Stress losses from Design Software	
		Total losses	% of Loss	Total losses	% of Loss
0.0	1405.26	259.68	18.47	246.691	17.5424
1.3	1405.26	262.06	18.64	250.529	17.8153
2.5	1405.26	264.41	18.81	253.835	18.0504
3.8	1405.26	266.74	18.97	257.403	18.3041
5.0	1405.26	269.05	19.14	260.98	18.5585
6.3	1405.26	271.46	19.31	265.218	18.8598
7.5	1405.26	273.85	19.48	269.466	19.1619
8.8	1405.26	276.21	19.65	273.172	19.4254
10.0	1405.26	278.55	19.81	275.764	19.6097

Table 5: Multiple profile stress variation due to one end stressing

Length	Intial Stress (N/mm^2)	Stress losses from IRC 112:2010		Stress losses from Design Software	
		Total losses	% of Loss	Total losses	% of Loss
0.0	1405.26	312.36	22.22	296.73	21.10
1.3	1405.26	314.74	22.39	300.89	21.39
2.5	1405.26	317.09	22.55	304.41	21.64
3.8	1405.26	319.42	22.72	308.24	21.91
5.0	1405.26	321.73	22.88	312.08	22.19
6.3	1405.26	324.14	23.05	316.68	22.51
7.5	1405.26	326.53	23.22	321.30	22.84
8.8	1405.26	328.89	23.39	325.27	23.13
10.0	1405.26	331.23	23.56	327.91	23.31

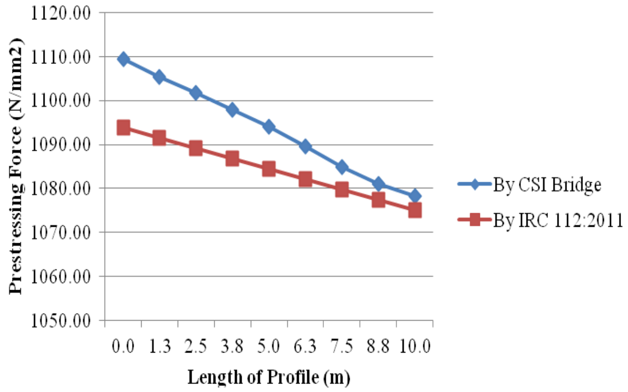

Figure 4: Single profile stress variation due to one end stressing

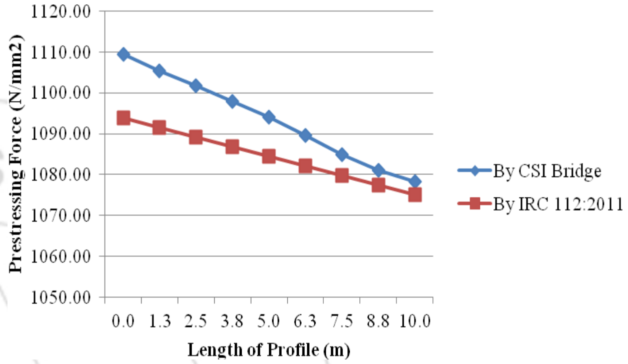

Figure 5: Multiple profile stress variation due to one end stressing

The stress loss from these four cases is ploted in Fig 4 to Fig 5. The percentage difference in stress loss calculated from IRC and Bridge design software is varying from 0.93% at 0m span and 0.2% at 10m span in single cable profile with one end stressing. While in both end stressing the percentage difference is varying from 0.93% at 0m span and 0.58% at 5m span .

The percentage difference in stress loss calculated from IRC and Bridge design software is varying from 1.11% at 0m span and 0.14% at 10m span in single cable profile with one end stressing. While in both end stressing the percentage difference is varying from 1.11% at 0m span and 0.69% at 5m span .

8. Conclusion

In the case of friction losses there is only 1.5% to 2% of stress losses from the friction and wobble effect. It reduces the losses as per the length increases. The loss calculation as per the IRC 112:2010 and the losses from bridge design software are same.

The amount of anchorage losses depend upon the slippage of the wire. Normally 5.5% to 6% stress will be loss. In the case of anchorage losses there is some difference in both manually and software stress losses. The design software found the loss value in a different way. Here when the losses will decreases with respect the length of profile increases.

There are no elastic deformation losses by the absence of multiple cables tendon profile. But in the software there having shown the losses due to elastic shortening of cables.

The major part of stress will be losses due to the relaxation of steel. In the case of normal relaxation 2.09% of stress wills loss after the immediate losses. When the stress transfer stage it having only 1.22% of stress losses and the opening traffic having 2.75% of stress will be losses. After a long time that is ꙍ days the maximum stress loss will be 7.5%.

Shrinkage of concrete 3% to 4% stress will be loses. The Perimeter of the girder, Relative humidity and grade of cement are the major parts that affect the shrinkage losses.

The creep reduces about 3.5% to 4.5% stress. The Perimeter of the girder, Temperature, Relative humidity and grade of cement are the major parts that affecting the creep. If the small change in RH there will be a huge change in the stress variation. As per IRC112:2010, more than 40 degree the creep coefficient increase by 10%. In software have no ability to find the correct creep coefficient value.

References

[1] Nazim Siddiqui, "Parametric Study And Comparison Of I-Girder Bridge With Different Support Configuration" International Journal of Scientific Engineering and Applied Scie nce (IJSEAS) - Volume- 1, Issue- 5, ISSN: 2395 -3470, August 2015

[2] Vishal U. Misal, "Analysis and Design of Prestressed Concrete Girder", International Journal of Inventive Engineering and Sciences (IJIES)

[3] H.Paul Zia, "Estimating Prestress Losses" Concrete International Journal Volume- 1, Issue- 5,pp 32-38, June 1979

[4] Adekunle Philips Adewuy, "Analytical Investigation Of Prestressed Concrete Structures Incorporating Combined Post-Tensioned And Post-Compressed Reinforcements", ARPN Journal of Engineering and Applied Sciences, VOL. 6, NO. 12, ISSN 1819-6608, pp 55-61, December 2011

[5] Tatiana García-Segura," Hybrid harmony search for sustainable design of post-tensioned concrete box-girder pedestrian bridges", Engineering Structures 92, pp 112–122, 2015

[6] J. E. Breen, "Verification of Load Distribution and Strength of Segmental Post-Tensioned Concrete Bridges", Butterworth-Heinemann Lid,Vol 13, pp. 113-127, 1991

[7] D.W. Cullington, "Continuous Acoustic Monitoring of Grouted Post-Tensioned Concrete Bridges", NDT&E International, Vol 34, pp. 95–105, 2001

[8] Yail J. Kim, "Anchorage Configuration for Post-Tensioned NSM CFRP Upgrading Constructed Bridge Girders", Engineering Structures, Vol 79, pp. 256–266, 2014

[9] R.G.Pillai, "Time-Variant Servicereliability of Post-Tensioned, Segmental, Concrete Bridges Exposed to Corrosive Environments", EngineeringStructures, Vol 32, pp.25962605, 2010

[10] Wen-WeiWan, "Strengthening multiple span simply-supported girder bridges using post-tensioned negative moment connection technique", EngineeringStructures, Vol 33, pp.663–673, 2011.