Time-dependent characteristics of a Self-Consolidating Concrete mix for PC concrete bridge girders

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ABSTRACT: Creep and shrinkage test results of the proposed SCC mix for bridge girders (in the state of Kansas) are presented. Four bridge girders with an inverted tee profile were used to measure the creep and shrinkage in typical bridge girders. In two of the girder specimens the strands were tensioned to seventy-five percent of ultimate tensile strength. The strands of the other two girder specimens used were left un-tensioned to evaluate the shrinkage effect of the concrete. In addition, the fully tensioned girder specimens were used to determine the transfer length of the pre-stressing strand.

1 INTRODUCTION

Self-Consolidating Concrete (SCC) has rapidly become a widely used material in the construction industry. The primary reasons for the increased use of SCC are the economical advantages that SCC has over normal conventional concrete (NC). The Interim Guidelines for the use of Self-Consolidating Concrete in PCI Member Plants defines SCC as “a highly workable concrete that can flow through densely reinforced or complex structural elements under its own weight and adequately fill voids without segregation or excessive bleeding without need for vibration.”

The Interim Guidelines also state in the commentary that the hardened properties of SCC may be different than those of NC. Where modulus, creep, and shrinkage are important design guidelines, it states that the mix should be properly investigated before using in design. When designing pre-stressed concrete members, these properties are very important for an accurate estimation of time-dependent losses.

2 PROBLEM STATEMENT

Departments of Transportation including the Kansas Department of Transportation (KDOT) would like to use SCC in pre-tensioned bridge members to enhance the aesthetics and improve consolidation in congested areas. However, before allowing the use of SCC in state bridges, KDOT needed to investigate the time-dependent losses of an SCC mix proposed by the local pre-caster. A previous study (“Bond Characteristics of an SCC Mix for Pre-stressed Concrete Bridge Girders,” companion paper at the 2005 National Bridge Conference) using SCC was conducted to investigate the bond between the strand and the concrete. At the time of this study the American Concrete Institute (ACI) and American Association of State Highway Transportation Officials (AASHTO) did not address the issue of members cast with SCC. KDOT wanted a thorough investigation of long term creep and shrinkage properties of a proposed SCC mix before allowing it to be used in state projects.

3 BACKGROUND

American Concrete Institute Building Code Requirements, AASHTO LRFD Bridge Design Specifications, KDOT, and The PCI Design Handbook, Sixth Edition all have slight differences in determining the losses of pre-stressed members. They are listed in the appendix section.

4 EXPERIMENTAL PROGRAM

4.1 *Prestress loss determination*

To determine the time-dependent losses, KDOT funded an experimental program to evaluate the long-term performance of inverted T-shape (IT) members cast with SCC. Four IT’s, twelve foot in length, were cast in order to determine the time-dependent losses of actual bridge girders. A twelve foot length was considered to be an adequate length for this SCC mix because previous tests concluded that a six-foot development length was all that was needed to achieve full bond. The girder type used was the IT600. The cross-section of this shape can be seen in Figure 1. Table 1 presents the geometric properties of the IT600. Dimensions of the cast specimens were measured and then used for all calculations. Of the four girders, two of them had the pre-stressing strand jacked to seventy-five percent of the ultimate guaranteed tensile stress (f_{pu}) and were designated as FT #1 and FT #2, where FT stands for fully-tensioned. These two girders were used to determine the combined effects of creep, shrinkage, and relaxation. In addition, the two fully tensioned specimens were used to evaluate the transfer lengths, as discussed in the following section. The remaining two specimens, used to determine the effect of shrinkage alone, had the pre-stressing strand pulled to a "hand-tight" condition. The designations of these specimens were UT #1 and UT #2, where UT stands for un-tensioned. The strand was placed in these shrinkage specimens to match the transformed section properties of the two specimens that had the strand jacked to seventy-five percent of f_{pu}.

Elastic shortening losses occur at the time the pre-stress force is transferred to the concrete and thus can be eliminated from long term calculations. The increase in relaxation losses from transfer to final loss calculations are very small compared to the creep and shrinkage losses and are not included in calculations. Elastic shortening and relaxation were determined from the change in strain just after detensioning and subtracted from the measurements that were recorded from the two fully-tensioned specimens. Then the values of shrinkage, obtained from the two un-tensioned specimens, was subtracted from the above mentioned fully-tensioned specimens along with the elastic shortening and relaxation to determine the effects of creep alone. To record the long term strains that all the specimens had undergone, Vibrating Wire Strain Gages (VWSG) were used. Gages with a 152.4mm gage length were chosen for this project. All the gages were connected to a data-logger that could take data readings at any desired time interval. Readings were taken every five minutes for the first day and then the collection interval was changed to two hours. Gages were placed at the height of the top strand (101.6mm from the top), the neutral axis of the section (208.5mm from the bottom and in one case this value changed slightly and thus the recorded value was used), and the last gage placed at the bottom strand height (50.8mm from the bottom). The setup of the gages is shown in Figure 2. The gage at the bottom was primarily used to gather the long term strains. To determine the stress at the strand height, Hooke's Law was used.

$$\sigma = E_{ps}\varepsilon \qquad (1)$$

where E_{ps}=modulus of elasticity (MPa); ε = recorded strain value (mm/mm).

Strains were zeroed just prior to detensioning. Therefore, the subsequent strain changes were due to pre-stress losses. A load cell was placed at the "dead" end of the pre-stressing bed in order to get an exact pre-stress force at detension. The nominal value of the jacking stress, f_{pj}, was calculated to be 1396 MPa. However experimentally it was found to be 1365 MPa and this value will be used for all calculations using code and experimental data. For the methods that were compared to the experimental data, no intermediate days are calculated, just final values. In order to estimate creep and shrinkage values for periods less than two years, the expression by Corley and Sozen (1966) was used. The following equation made it possible to compare creep and shrinkage values for periods less than two years.

$$R = 0.13\ln\left(t+1\right) \qquad (2)$$

where R = the total time-dependent proportion; and t = time (days).

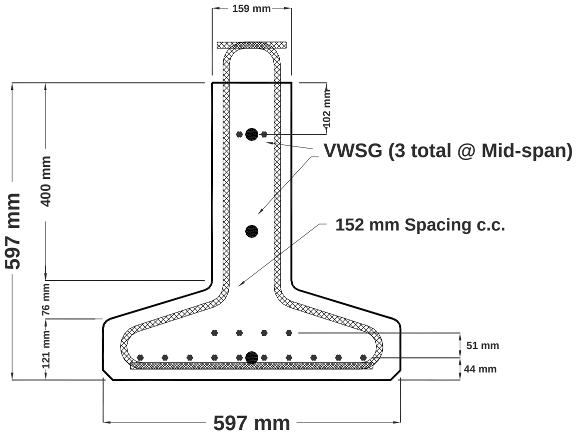

Figure 1. Cross section of IT600.

Table 1. List of measured geometric properties of IT600.

A=165,096 mm^2	I = 8,272,242 mm^2	E_{ps} =196,501 MPa
E_{ci} = 24,821Mpa	Y_{bot} = 215 mm	e = 98 mm
M_{sw} = 498 KN.mm	RH = 65%	A_{ps} = 1579 mm^2
f_{pj} = 1363 Mpa		V/S = 73 mm

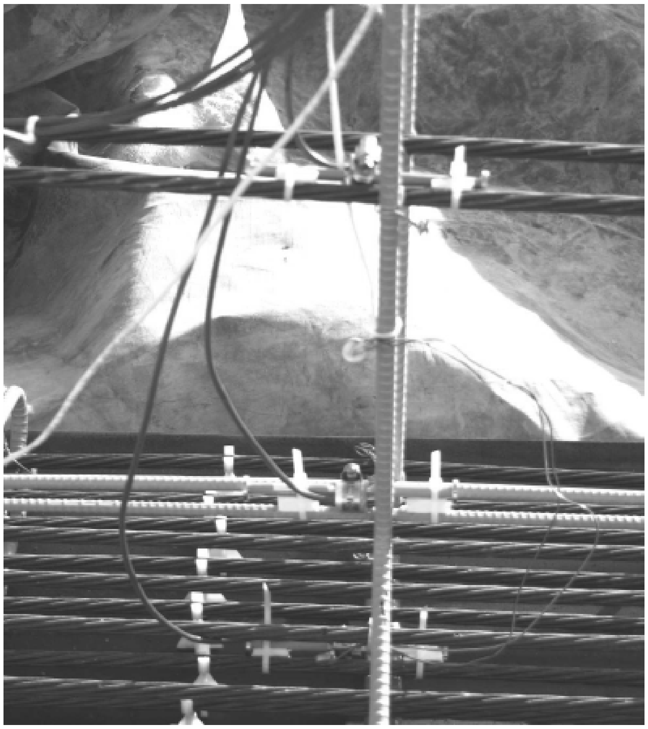

Figure 2 Location of VWSG’s.

4.2 *Transfer length setup*

Measurements of the transfer length were accomplished by measuring concrete surface strains with a mechanical strain gage. Whittemore points, stainless steel discs with a machined hole in the center, were adhered along the bottom flange of the specimen of both fully-tensioned specimens prior to detensioning, Figure 3. The location chosen for the line of points was the center of gravity between the bottom two layers of pre-stressing strand, 68 mm from the bottom. Readings were then taken just prior to detensioning and after detensioning, shown in Figure 4. The procedure outlined by Russell & Burns (1993) was used to evaluate the data. Measured changes in strain were then plotted against the length of the specimen. The concrete strains were the numerical difference between the initial reading and the final reading (just after detensioning). To eliminate any anomalies, measured strains were smoothed by averaging the data over three gage lengths. The equation used to smooth the data is shown as follows:

$$(strain)_i = \frac{(strain)_{i-1} + (strain)_i + (strain)_{i+1}}{3} \qquad (3)$$

where i = the current strain reading.

So at any given strain point, that strain and the values just ahead and behind were averaged to obtain the “smoothed” average.

Transfer lengths were then determined by plotting the strains versus the specimen length. The method known as the “95% Average Maximum Strain,” also detailed in Russell and Burns (1993), was used to estimate the transfer lengths. The first step is to plot the “smoothed” strain profile. Then the maximum strain that the specimen underwent is determined. Next, 95% of this maximum strain value is computed and a line corresponding to this value is drawn. Lastly the transfer length is determined by taking the intersection of the 95% maximum strain line and the “smoothed” strain profile line.

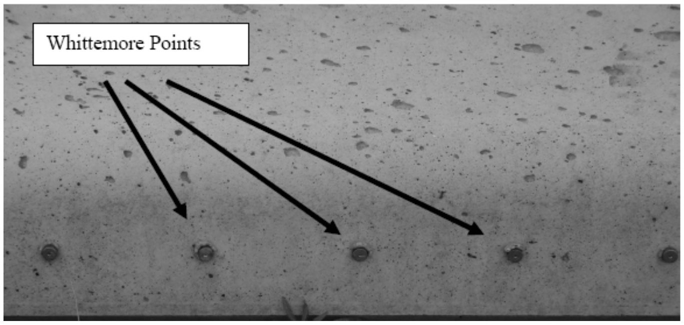

Figure 3 Whittemore points adhered to the bottom the specimen.

Figure 4. Measuring concrete surface displacements with a Whittemore gage.

Transfer length is implied by both ACI and AASHTO to be expressed as:

$$L_{tr} = \frac{f_{se}}{3} d_b \qquad (4)$$

where d_b = diameter of strand (in) f_{se} = effective stress is pre-stressing strand after allowance of pre-stress losses (ksi).

For the IT specimen geometry and strand configuration used in this study, the transfer length was calculated (using equation 4) to be 24 inches.

5 RESULTS

5.1 *Prestress losses*

By using the methods for determining pre-stress losses as described earlier and presented in the appendix, all the values for elastic shortening, creep, shrinkage, and relaxation were calculated, see Table 1. It was found that the ACI and PCI methods gave the same results; therefore they are presented in the same row. The experimental values for elastic shortening, creep, shrinkage, and relaxation are also given. However, it must be noted that data collection for this paper was stopped at 144 days and collection of strains will continue for a much longer time in order to obtain more complete data.

Table 2. Summary of time-dependent losses.

Method	Elastic Shortening	Creep	Shrinkage	Relaxation	Effective prestress Loss
AASHTO	121.3	183.4	50.3	13.1	992.8
ACI / PCI	110.3	239.2	46.9	18.6	951.5
KDOT	119.3	183.4	50.3	11.0	999.7
Experimental	122.0	129.6	3.4	7.6	1096.3

* All values in MPa, data recorded at 144 days

Strains have been recorded throughout the life of the specimens. By using equation 1, time-dependent losses were estimated at several days and compared to the experimentally-determined values. It must be noted that the strains from FT #1 and FT #2 were averaged and then converted to stresses. The results are shown in Table 2. Figure 5 presents the long term losses in graphical form.

Table 3. Strand stress at various ages (MPa).

Time (days)	AASHTO	ACI/PCI	KDOT	Experimental
Transfer	1234.2	1213.5	1213.5	1241.1
25	1137.6	1117.0	1137.6	1179.0
50	1117.0	1089.4	1117.0	1158.3
75	1103.2	1075.6	1103.2	1130.7
96	1096.3	1068.7	1096.3	1123.8
120	1089.4	1054.9	1089.4	1117.0
144	1082.5	1048.0	1082.5	1096.3
Long term	992.8	951.5	999.7	1034.2*

* Estimated by extrapolating curve to 2 years.

5.2 *Transfer length results*

The concrete strain profile verses length for both FT #1 and FT #2 are shown below. FT #1, seen in Figure 6, had transfer lengths of 813 mm on one end and 584 mm on the other. FT #2, seen in Figure 7, had transfer lengths of 610 mm on one end and 711 mm on the other. The ends of the specimens that had the greatest transfer lengths both were flame-cut at detensioning while the ends with the smaller values underwent a gradual release during detensioning.

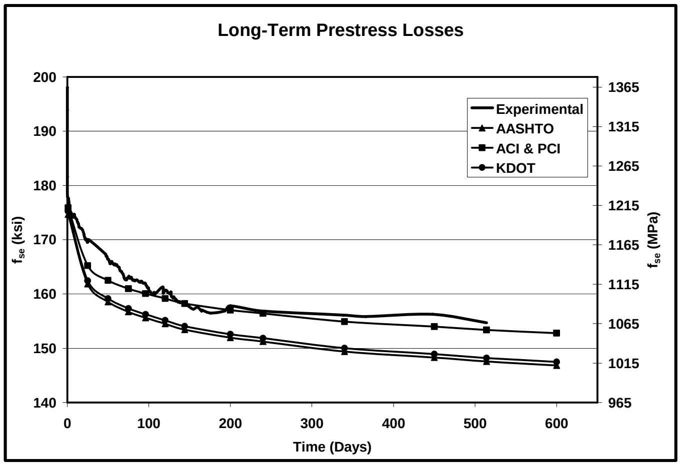

Figure 5. Long term effective pre-stress losses.

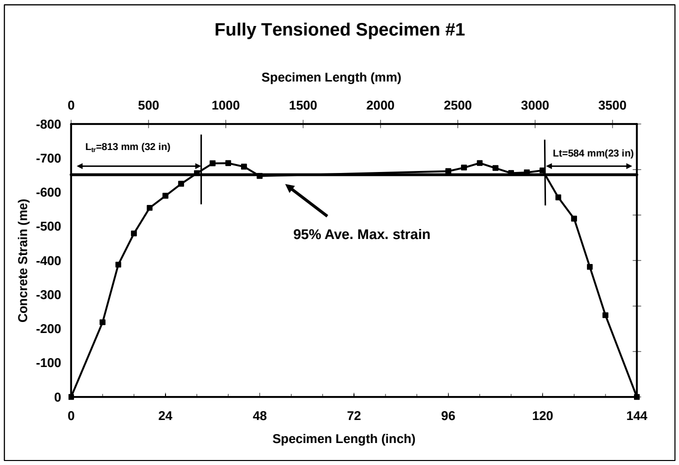

Figure 6. Transfer length profile for FT #1.

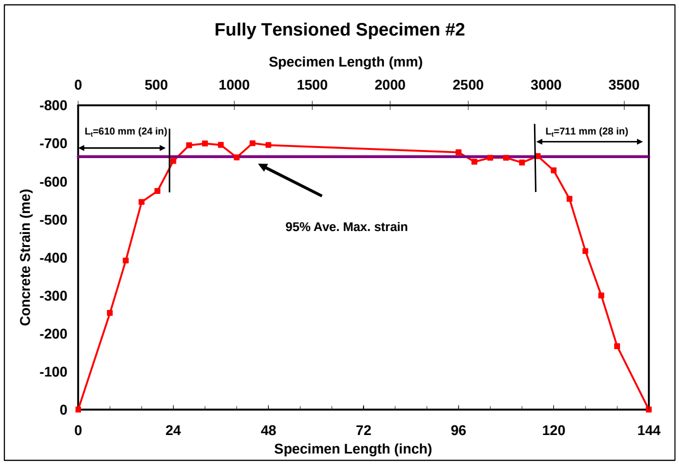

Figure 7. Transfer length profile for FT #2.

6 CONCLUSIONS

Total observed losses for the experimental specimens were slightly less than those predicted by the current AASHTO, ACI/PCI, and KDOT design expressions.

Measured transfer lengths for the proposed SCC mix and specimen geometry were in general accordance with the current AASHTO and ACI/PCI design assumptions, with the maximum measured transfer lengths being 33 percent larger than the calculated value.

Using the proposed SCC mix, it was found that effective prestress losses were in general accordance with current AASHTO, ACI/PCI, and KDOT code equations and no special design considerations need to be taken.

REFERENCES

AASHTO. 2004. *LRFD bridge design specifications, 3rd edition*. Washington, DC. American Association of State Highway and Transportation Officials.

ACI Committee 318. 2002. *Building code requirements for structural concrete (ACI 318-02) and commentary (ACI 318R-02)*. Farmington Hills, MI. American Concrete Institute.

Corley, W.G. & Sozen, M.A. 1966. Time dependent deflections of reinforced concrete beams. *Journal of the American Concrete Institute, Proceedings*63(3): 373-386.

Interim guidelines for the use of self-consolidating concrete in precast/ prestressed concrete institute member plants, first edition. 2003. Chicago, IL.

KDOT. 2003. *Kansas department of transportation design manual, bridge section(3)*. Kansas Department of Transportation.

PCI. 1999. *PCI design handbook, fifth edition*. Chicago, IL Precast/Prestressed Concrete Institute.

Russell, B.W. & Burns, N.H. 1993. Design guidelines for transfer, development and debonding of large diameter seven wire strands in pretensioned concrete girders. *Texas Department of Transportation, Research Project 3-5-89/2-1210*:286

Zia, P., Preston, H.K., Scott, N.L., & Workman, E.B. 1979. Estimating prestress losses. *Concrete International*1(3): 32-38.

APPENDIX

ACI (ACI Committee 318. 2002) and PCI methods (PCI 1999)

Both use the findings of Zia et al (1979) for calculations.

Elastic Shortening of Concrete (ES)

For members with bonded tendons,

$$ES = K_{es} E_s \frac{f_{cir}}{E_{ci}} \tag{1}$$

where K_{es} = 1.0 for pre-tensioned members; K_{es} = 0.5 for post-tensioned members when tendons are tensioned in sequential order to the same tension. With other post-tensioning procedures, the value for K_{es} may vary from 0 to 0.5.

$$f_{cir} = K_{cir} f_{cpi} - f_g \tag{2}$$

where K_{cir} = 1.0 for post-tensioned members; K_{cir} = 0.9 for pretensioned members.

Creep of Concrete (CR)

For members with bonded tendons,

$$CR = K_{cr} \frac{E_s}{E_c} \left(f_{cir} - f_{cds} \right) \tag{3}$$

where K_{cr} = 2.0 for pre-tensioned members; K_{cr} = 1.6 for post-tensioned members

Shrinkage of Concrete (SH)

$$SH = 8.2x10^{-6} K_{sh} E_s \left(1 - 0.06\frac{V}{S}\right)\left(100 - RH\right)$$

in which K_{sh} = 1.0 for pre-tensioned members or K_{sh} is taken from Table 1 (Russell et al. 1993) for post-tensioned members.

Relaxation of Tendons (RE)

$$RE = \left[K_{re} - J\left(SH + CR + ES\right)\right]C$$

where the values of K_{re}, J, and C are taken from Tables 2 and 3 (Russell et al. 1993).

AASHTO method

Taken from the Third Edition (AASHTO 2004) for pre-tensioned members

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

where Δf_{pT} = total loss (ksi); Δf_{pES} = loss due to elastic shortening (ksi); Δf_{pSR} = loss due to shrinkage (ksi); Δf_{pCR} = loss due to creep of concrete (ksi); and Δf_{pR2}= loss due to relaxation of steel after transfer (ksi).

Elastic Shortening (Δf_{pES})

$$\Delta f_{pES} = \frac{E_p}{E_{ci}} f_{cgp} \tag{4}$$

where: f_{cgp} =sum of concrete stresses at the center of gravity of pre-stressing tendons due to the pre-stressing force at transfer and the self-weight of the member at the sections of maximum moment (ksi); E_p =modulus of elasticity of pre-stressing steel (ksi); and E_{ci} =modulus of elasticity of concrete at transfer (ksi)

Shrinkage (Δf_{pSR})

$$\Delta f_{pSR} = \left(17.0 - 0.150H\right) \tag{5}$$

where: H = the average annual ambient relative humidity (percent)

Creep (Δf_{pCR})

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0\Delta f_{cdp} \geq 0$$

where f_{cgp} =concrete stress at center of gravity of pre-stressing steel at transfer (ksi); Δf_{cdp} =change in concrete stress at center of gravity of pre-stressing steel due to permanent loads with the exception of the load acting at the time the pre-stressing force is applied. Values of Δf_{cdp} should be calculated at the same section or at sections for which f_{cgp} is calculated (ksi).

Relaxation (Δf_{pR2})

At transfer

In pre-tensioned members, the relaxation loss in pre-stressing steel, initially stressed in excess of 0.50 f_{pu}, may be taken as:

For stress-relieved strand:

$$\Delta f_{pR1} = \frac{\log\left(24.0t\right)}{10.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$

For low-relaxation strand:

$$\Delta f_{pR1} = \frac{\log\left(24.0t\right)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$

where t =time estimated in days from stressing to transfer (days); f_{pj} = initial stress in the tendon at the end of the stressing (ksi); and f_{py}=specified yield strength of pre-stressing steel (ksi).

After Transfer

Losses due to relaxation of pre-stressing steel may be taken as:

For pre-tensioning with stress-relieved strands:

$$\Delta f_{pR2} = 20.0 - 0.4\Delta f_{pES} - 0.2\left(\Delta f_{pSR} + \Delta f_{pCR}\right)$$

where Δf_{pES} = loss due to elastic shortening (ksi); Δf_{pSR} = loss due to shrinkage (ksi); and Δf_{pCR} = loss due to creep of concrete (ksi).

For pre-stressing steels with low relaxation properties conforming to AASHTO M 203 (ASTM A 416 or E 328): Use 30 percent of Δf_{pR2} given by equation 12.

KDOT method

As described in the 2003 release (KDOT 2003), the loss of stress in the pre-stressing steel is as follows:

Δf_S = SH + ES + CR_C + CR_S
Δf_S = Loss of stress, psi
SH = Loss due to concrete shrinkage, psi
ES = Loss due to elastic shortening, psi
CR_C = Loss due to creep of concrete, psi
CR_S = Loss due to relaxation of steel, psi

<u>Shrinkage</u>

The shrinkage loss is computed as follows,

$$SH = 17,000 - 150RH$$

where RH is the average relative humidity in percent. For Kansas, the humidity may be assumed at 65 percent.

<u>Elastic Shortening</u>

Elastic shortening is computed as follows:

$$ES = \left(E_s / E_{ci}\right) f_{cir} \qquad (6)$$

where $E_s = 28 \times 10^6$ psi; E_{ci} = Modulus of elasticity of concrete at transfer of stress ($33W^{3/2}\sqrt{f'_{ci}}$ psi.); W = 145 pcf for normal weight concrete; and f_{cir} = Concrete stress at the center of gravity of the pre-stressing steel due top pre-stressing force and dead load of the beam immediately after transfer. (At this stage, the initial stress in the tendon has been reduced by elastic shortening of the concrete and tendon relaxation during placing and curing of the concrete.)

<u>Creep of Concrete:</u>

For pre-tensioned and post-tensioned members

$$CR_C = 12 f_{cir} - 7 f_{cds}$$

Where f_{cds} is the concrete stress at the center of gravity of pre-stressing steel due to all dead loads except the dead loads present at the time pre-stressing force is applied.

<u>Relaxation of Pre-stressing Steel – (Low Relaxation Strand)</u>

$$CR_S = 5,000 - 0.10ES - 0.05\left(SH + CR_C\right)$$

The values of *ES*, *SH*, and CR_C are those computed previously,

Total losses: $\Delta f = SH + ES + CR_C + CR_S$

The minimum loss of pre-stress to be used when computing service load stresses shall be 35,000 psi.