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PRESTRESSING OF RC ELEMENTS WITH INTERNAL FRP REINFORCEMENT – SIMILARITIES AND DIFFERENCES IN COMPARISON WITH STEEL REINFORCED ONES

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ABSTRACT

Fibre reinforced polymer (FRP) reinforcement for reinforced concrete (RC) elements is increasing its popularity. The main benefits from its application are the lack of corrosion, high strength, reduced CO_2 emissions for the production and installation and lack of magnetic interference. At the same time it has relatively high price and the higher deformability in comparison with steel reinforcement. Classical approach for solving the problem with relatively low Young modulus for GFRP and BFRP reinforcement is via design, based predominantly on SLS considerations. More economical approach is via prestressing of this type of reinforcement. The latest developments and tendencies in this direction are connected with solving a range of existing problems, such as developing of appropriate anchoring devices, estimation of losses of prestress in terms of FRP reinforcement and consideration of different modes of failure including creep rupture effects. The proposed paper investigates the latest developments of the research in the area and critically analyses the existing problems and possible solutions in comparison with adopted approaches and techniques for steel reinforced prestressed elements.

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1. Introduction

BFRP and GFRP reinforcements are attractive option for future applications due to their resistance to corrosion, high strength, lack of magnetic properties and relatively low price in comparison with other types of FRP reinforcement. At the same time they possess low Young modulus, which is the reason for higher deformability of reinforced concrete (RC) elements reinforced with BFRP and GFRP bars.

Among the most popular solutions of such structural problem are the reinforcing in accordance with Serviceability Limit State (SLS) or application of pre-stressing on the reinforcement. The first approach in case of B/GFRP is quite expensive resulting in amount of reinforcement significantly higher than the requirements in accordance with Ultimate Limit State (ULS). The second one is obviously more attractive and initial research connected with it indicates good opportunities for future applications.

2. Background

Many research investigations, such as (ISIS Canada, 2007) indicate that the application of internal FRP reinforcement can increase the service life and the load carrying capacities of reinforced and prestressed concrete structures, as such type of reinforcement exhibit low relaxation losses and good fatigue properties

Among the first publications in the area of prestressing with FRP materials are a wide range of publications by a research group in Cambridge University UK. In Burgoyne's study on prestressing via polyaramid tendons he obtained very positive results for such type of reinforcement (1992). Balafas and Burgoyne (2012) indicated that developed range of approaches and considerations about application of governing principles for optimal design of external prestressing tendons including depth against reinforcing area diagrams.

One of the first practical applications in developing FRP prestressed concrete was achieved in 1998 during the construction of Taylor Bridge in Headingley, Manitoba. Two types of carbon FRP (CFRP) reinforcements were used in the project. The girders were pretensioned by using Leadline bars. The bridge is remotely monitored to estimate the effects of the long-term behaviour and durability of the FRP materials comparative to conventional steel reinforcements and the obtained results are very positive (Shehata at al., 1999).

Braimah (2000) investigated beams pre-tensioned with CFRP rods and with steel prestressing strand. He pointed out that at the sustained load level the steel pre-tensioned beam shows higher mid-span deflection in comparison with the CFRP pre-tensioned beams.

Between the first attempts for using BFRP bars as prestressing reinforcement Jonsson (2011) states that the ultimate bearing resistance of a beam with prestressed BFRP tendons is not significantly higher than without prestressed, on the other hand SLS resistance is significantly higher and the deflection is smaller in comparison with un-prestressed beams. Guðmundsson (2012) investigated concrete beams, prestressed with BFRP tendons and commented that BFRP prestressed beams without shear reinforcement are weak to transvers loading even if the a/d ratio is high.

Pearson and Donchev (2013) pointed out that for elements reinforced with post-tensioned internal BFRP bars higher level of prestressing significantly reduces the deformation at the same level of loading. Grouting has a beneficial effect on increasing the ultimate capacity and decrease the deformability of BFRP reinforced beams.

The indicated above initial research clearly defines the opportunity to use relatively deformable BFRP and GFRP reinforcement for prestressing purposes, this way achieving reduction of the deformability of the beams. At the same time certain limitations about prestressing with BFRP and GFRP reinforcing bars, such as creep rupture (ACI, 2006) and difficulties connected with construction of appropriate anchoring devices have to be kept in mind. According to ACI (2006) review of available information about GFRP creep rupture the results for 50 years lifespan are varying between 29% and 55% of the ultimate load. Similar results are indicated in (Banibayat, 2011) for BFRP reinforcement under relatively high temperature and alcali environment.

Indicating the same limits for GFRP reinforcement in ISIS (2006) is noted that laboratory testing in not necessarily representative of field performance. Some information from conducted experiments with BFRP reinforcing bars (Pearson, Donchev and Salazar, 2013) are in support of this view and give initial indication that the laboratory conditions for such estimation could be quite conservative.

For the purposes of this publication, the attention is focused on relatively short term behaviour of prestressed FRP elements, when creep rupture effects are not governing. Possible solutions for mitigating the effects of creep rupture are outside the scope of this work as well.

3. Main sources of pre-stress losses

Pre-stress loss for design of PC element in concrete structures is an important design parameter which must be taken into consideration with FRP reinforcement. ACI Committee 318 (2011) states that pre-stress losses have little effect on ultimate design strength of steel reinforced members, but can affect service conditions such as deflections, cracking load and camber. Overestimation on pre-stress losses at service loads can be nearly as detrimental as underestimation.

Pre-stressing losses can be determined analytically or experimentally. Analytical methods to estimate pre-stressing losses are divided into the following levels (NCHRP REPORT 496, 2003):

a) Lump-sum method to estimate total pre-stressing losses (oversimplified method for preliminary design);

b) Detailed methods to estimate pre-stress losses separately due to each particular source;

c) Accurate determination of cumulative losses by time-step methods.

For the purpose of this publication the application of the second type of estimation is adopted. To determinate pre-stress losses from experimental techniques requires a back-calculation from the test data by using theory of mechanics concepts.

A summary of the sources of the separate pre-stressing losses and the stage of their occurrence is given in table 1. From this table, the total loss in pre-stress can be calculated for post-tensioned and pre-tensioned steel reinforced members as follows (Nawy, 2006):

Pre-tensioned members:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pR} + \Delta f_{pCR} + \Delta f_{pSH}. \qquad (1)$$

Post-tensioned members:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pR} + \Delta f_{pCR} + \Delta f_{pSH} + \Delta f_{pF} + \Delta f_{pA}, \qquad (2)$$

where Δf_{pES} is applicable only when tendons are jacked sequentially, and not simultaneously.

Table 1. Types of prestress loss (after Nawy, 2006)

	Stage of occurrence		Tendons stress loss	
Type of prestress loss	**Pretensioned members**	**Post-tensioned members**	**During time nterval (t_i, t_j)**	**Total or during life**
Elastic shortening of concrete (ES)	At transfer	At sequential jacking	______	Δf_{pES}
Relaxation of tendons (REL)	Before and after transfer	After transfer	$\Delta f_{pR}(t_i, t_j)$	Δf_{pR}
Shrinking of concrete (SH)	After transfer	After transfer	$\Delta f_{pS}(t_i, t_j)$	Δf_{pSH}
Creep of concrete (CR)	After transfer	After transfer	$\Delta f_{pC}(t_i, t_j)$	Δf_{pCR}
Anchorage seating loss (A)	______	At transfer	______	Δf_{pA}
Total	Life	Life	$\Delta f_{pT}(t_i, t_j)$	Δf_{pT}

3.1. Elastic shortening of concrete (ES)

When pre-tensioned tendons are released from the anchorage or post-tensioning is applied, this can cause an elastic deformation of the concrete. The loss of force in each group of steel tendons of area A_P varies along its length and based on BS EN 1992-1-1 (2004) can be approximately estimated from formula below:

For pretensioned members: $$\Delta P_{el}(x) = A_p \frac{E_P}{E_{cm}(t)} \sigma_c(x), \quad (3)$$

where A_p is the cross-sectional area of tendon,

E_p is the modulus of elasticity of prestressing steel,

E_{cm} is the modulus of elasticity of concrete at time t,

$\sigma_c(x)$ is the stress in the concrete adjacent to the bar at transfer,

$E_P/E_{cm}(t)$ is the modular ratio, with the modulus for concrete based on its age at transfer.

For post-tensioned members:

$$\Delta P_{el} = A_p E_p \sum\left[\frac{j\Delta\sigma_c(t)}{E_{cm}(t)}\right], \quad (4)$$

where $\Delta\sigma_c(t)$ is the variation of stress at the centre of gravity of the tendons applied at time t,

j is a coefficient equal to (n-1)/2n where n is the number of identical tendons successively prestressed. As an approximation j may be taken as ½ for the variations due to permanent actions applied after prestressing.

ES loss of prestress for FRP reinforcement is expected to be the same as for steel reinforcement with replacement of the E-modulus for steel with the one for FRP.

3.2. Relaxation losses (REL)

After the concrete member is cast around the pretensioned bars, additional factors such as time and temperature applied in accordance with corresponding curing method are influencing relaxation prestress losses. EN 1992-1-1 (BSI, 2008) states that the starting point is the relaxation loss at 1000 hours from an initial stress of 70% of the actual tensile strength of the steel tendons.

According to ISIS Canada, (2007) the losses due to relaxation for CFRP are negligible when the initial stress is equal to 50% of the ultimate tensile stress. However, relaxation losses for different type of FRPs vary. The relaxation losses in FRP tendons are a combination of three sources and the total relaxation loss can be calculated by assessing these three effects separately. ACI (2006) defines these three effects as follows:

$$REL = REL1 + REL2 + REL3. \quad (5)$$

3.2.1. REL1 – Relaxation of polymer

When stress is applied to the tendons, a portion of the load is carried in the resin matrix. The matrix, which is a viscous-elastic material, relaxes and loses its contribution to the load carrying capacity. This relaxation can be obtained by:

$$n_r = \frac{E_r}{E_f}. \quad (6)$$

Where, n_r is the modular ratio of the resin. E_r is the elastic modulus of the resin, E_f is the modulus of the fiber.

The volume of fibers in the tendon can be determined from $\nu_f + \nu_r = 1.0$, where ν_f and ν_r are the volume fractions of fiber and resin, respectively. REL1 can be expressed as follow:

$$REL1 = n_r \times \nu_r. \quad (7)$$

3.2.2. REL2 - Straightening of Fibers

The fibers in a pultruded section are nearly, but not completely parallel. Therefore, stressed fibers flow through the matrix and straighten, and this straightening appears as a relaxation loss in typical applications. An assumed 1% to 2% relaxation of the transfer stress is adequate to predict this portion of the relaxation loss calculation.

3.2.3. REL3- Relaxation of Fibers

CAN/CSA-S806-02 report indicates that the fiber relaxation is dependent upon the fiber type. According to this report, in the absence of specific information, the following values of relaxation may be used (with t = time in days), expressed as a percentage of the transfer stress:

$$\text{for CFRP:} \quad \text{Relaxation (\%)} = 0.231 + 0.345 \log(t) \quad (8)$$

$$\text{for AFRP:} \quad \text{Relaxation (\%)} = 3.38 + 2.88 \log(t). \quad (9)$$

The behaviour of BFRP and GFRP reinforced elements in this aspect is not sufficiently investigated.

3.3. Shrinkage loss (SH)

Shrinkage of concrete is affected by several factors such as mixture properties, type of cement, type of aggregate, curing time, size of the member, time between the end of external curing, application of prestressing and the environmental conditions. Narayanan and Beeby, (2005) mentioned that as the concrete shrinks, the prestressing steel must compress by the same strain which causes a loss of prestress and a change in the concrete stress. According to BS EN 1992-1-1 (2008) the prestress loss due to concrete shrinkage can be determined as:

$$\varepsilon_{cs} - \frac{\Delta\sigma c}{E_{ce}} = \frac{\Delta\sigma p}{E_p} \tag{10}$$

where ε_{cs} is the shrinkage strain;

E_{cs} is the effective concrete modulus;

E_p is the modulus of elasticity of the prestressing steel;

z_{cp} is the eccentricity of the tendons;

I_c is the second moment of area of the concrete section;

A_c is the area of the concrete section.

According to ACI Committee 209 (1997) the average value of ultimate shrinkage strain in both steam-cured and moist-cured concrete is given as $\varepsilon_{SH} = 780 \times 10^{-6}$ m/m. PCI (1999) stipulates for standard conditions an average value for nominal ultimate shrinkage strain $(\varepsilon_{SH})_u = 820 \times 10^{-6}$ m/m. This average value is affected by temperature, length of initial moist curing, concrete composition and ambient relative humidity. To take such effects into account, ε_{SH} should be multiplied by a correction γ_{SH} :

$$\varepsilon_{SH} = 780 \times 10^{-6}\, \gamma_{SH} \tag{11}$$

the loss in prestressing in pretensioned members is

$$\Delta f_{pSH} = \varepsilon_{SH} \times E_{ps}, \tag{12}$$

where Δf_{pSH} is shrinkage of concrete and E_{ps} is the modulus of elasticity of prestressing reinforcement.

For post-tensioned members, shrinkage loss is slightly less than pretensioning since some shrinkage has already taken place before post-tensioning. The general formula for loss in prestressing due to shrinkage becomes:

$$\Delta f_{pSH} = 8.2 \times 10\text{-}6\, K_{SH}\, E_{ps}\, (1 - 0.06\quad)(100 - RH), \tag{13}$$

where, RH is relative humidity and V/S is volume to surface ratio. $K_{SH} = 1.0$ for pretensioned member and table 2 gives the values of K_{SH} for post-tensioned members.

According to CAN/CSA S6-06 Prestress losses due to shrinkage of concrete (SH) may be calculated, using empirically-derived equations, as follows.

- for pretensioned members:

$$SH = 117 - 1.5RH \tag{14}$$

- for post-tensioned members:

$$SH = 94 - 0.85RH. \tag{15}$$

Table 2. Values of KSH for Post-Tensioned Members (after Prestressed Concrete Institute, 1999)

Time from end of moist curing to application of prestress, days	1	3	5	7	10	20	30	60
K_{SH}	0.92	0.85	0.80	0.77	0.73	0.64	0.58	0.45

3.4. Creep loss (CR)

Creep is the deformation in the concrete due to the applied stress. It depends on the ambient humidity, the dimensions of the element and the composition of the concrete. BSI (2008) stats that creep is also influenced by the maturity of the concrete when the load is first applied and depends on the duration and magnitude of the loading. The prestress loss due to creep is:

$$\frac{\sigma_c}{E_c}\phi(t, t0) - \frac{\Delta\sigma c}{E_{ce}} = \frac{\Delta\sigma}{E_p} \tag{16}$$

$$\Delta\sigma_{p,c} = \frac{\frac{E_p}{E_c}\phi(t,t0)\sigma_c}{1+\frac{E_p}{E_{ce}}\frac{A_p}{A_c}\left(1+\frac{A_c}{I_c}z^2{}_{cp}\right)} \tag{17}$$

Where, σ_c is the initial concrete stress adjacent to the steel tendons.

The ACI-ASCE Committee expression for estimating the loss in prestressed members due to creep can be defined as follows:

$$\Delta f_{pCR} = K_{CR}\frac{E_{ps}}{E_c}(\bar{f}_{cs} - \bar{f}_{csd}) \tag{18}$$

$$\Delta f_{pCR} = n\, K_{CR}\,(\bar{f}_{cs} - \bar{f}_{csd}) \tag{19}$$

where K_{CR} = 2.0 for pretensioned members;

K_{CR} = 1.60 for post-tensioned members (both for normal concrete);

cs = stress in concrete at level of steel cgs immediately after transfer;

csd = stress in concrete at level of steel cgs due to all superimposed dead loads applied after prestressing is accomplished;

n = modular ratio.

Note that K_{CR} should be reduced by 20% for light weight concrete.

Since modulus of elasticity of steel tendons is generally higher than a corresponding FRP tendon, losses for prestressed FRP tendons due to elastic shortening, shrinkage and creep of concrete will be less than for prestressed steel tendons (ISIS, 2007).

When the elements are pretensioned due to anchorage seating and friction additional losses of prestress are applicable, but they will not be subject of this research.

4. Conclusion

As a result of conducted critical overview for prestressing via pretensioning with FRP reinforcing bars the main losses of prestress to be considered are as follows:

- Elastic shortening of concrete (ES) – similar to steel, but their further development is getting gradually less than for steel reinforcement, due to different Young modulus.

- Shrinkage loss (SH) – modification due to difference in modulus of elasticity for FRP is needed.

- Creep loss (CR) – similar as steel, but for FRP reinforcement is less than for steel reinforcement.

- Relaxation losses (REL) – most dramatic difference in comparison with streel, both theoretical and experimental investigations are needed especially for BFRP and GFRP reinforcement. Taking in consideration the long term deformability of different types of FRP reinforcement with varying load conditions have to be kept in mind.

The overviewed data indicates the need of developing of precise methodology for calculating losses of prestress in FRP reinforced elements.

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