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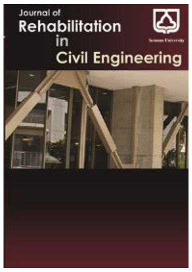

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Ultimate Tendon Stress in CFRP Strengthened Unbounded HSC Post-Tensioned Continuous I-Beams

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ABSTRACT

The use of unbounded tendons is common in prestressed concrete structures and evaluation of the stress increase in unbonded tendons at ultimate flexural strength of such structure has posed a great challenge over the years. Based on the bending experiment for two-span continuous post-tension beams with unbounded tendons and externally applied CFRP sheets, the monitoring of the stress increment of unbounded tendons is made in the loading process. For these aims, in this paper there are presented results of two continuous un-bonded post-tensioned I-beams were cast with high strength concrete (HSC) and monitored by electrical strain gauges. The beams are made of which are compared with the theory proposed by different codes. The results indicate that the ACI 318-2011 provides better estimates than AASHTO-2010 model whereas this model provides better estimates than BS 8110-97. Comparison of experimental ultimate tendon stress increase of strengthened and non-strengthened beams casted with HSC indicates that increase in tendon stress at an ultimate state in strengthened unbounded post-tensioned beam is lower than non-strengthened unbounded post-tension beam casted with HSC.

1. Introduction

Post-tensioned beams with unbounded tendons and externally applied CFRP sheets is a kind of new technique for strengthening and has the merits of both post-tensioned unbounded prestressed concrete and strengthening with externally CFRP sheets. To analysis and perform flexural design of such concrete members with unbounded tendons, the ultimate stress in the prestressed steel must be known. Due to lack of continuity between unbounded tendons and surrounding concrete, the theory of strain compatibility cannot be applied to the strain of un-bonded tendons in such concrete beams subjected to loading, so estimation of stress increase in the tendons due to external loading is difficult. In the case of post-tensioned members hundreds of elements, both slabs and beams cast with normal strength concrete were tested by Warwaruk et al. [1]; Cooke et al [2]; Elzatany and Nilson [3]; Du and Tao [4]; Chouinard [5]; Harajli and Kanj [6]; Ozkul et al. [7]. Through these investigations, there were separated parameters, which have influence on the stress increment in unbounded tendons. These are; span-to-depth ratio, concrete compressive strength (normal strength and high strength concrete), yield strength, tendon profile, tensile strength and amount of non-prestressed and prestressed reinforcement, type of loading (single point load, third-point loading and uniformly distributed load), loading pattern in continuous members (uniform loading, alternate spans, adjacent spans, external span or internal span) and stress in the tendon after time dependent losses. These parameters have caused formulating of tendon stress was more complicated. Since the early 1950s, researchers have suggested many experimental and analytical equations for prediction of stress in tendon at an ultimate state, which have been evaluated and reviewed by Harajli and Kanj; Naaman and Alkhairi [8]; Ament et al. [9]; Harajli [10]; Manisekar and Senthil [11]; Dall'Asta et al. [12]; He and Liu [13], that these equations don't account all of important parameters and worked with nearly low strength concrete.

With recent advancements in concrete technology, and the availability of various types of mineral and chemical admixtures and very powerful superplasticizers, concrete with a compressive strength of up to 100 MPa can now be produced commercially with an acceptable level of variability using ordinary aggregates. Since HSC is well accepted for prestressed concrete construction, it is necessary that more data and information on ultimate stress increase in unbounded tendons of HSC post-tensioned members be available.

Externally bonding fiber reinforced polymer (FRP) sheets with an epoxy resin is an effective technique for strengthening and repairing the reinforced concrete (RC) beams under flexural loads but the effect of externally bonding FRP on the ultimate tendon stress have been not considered. The purpose of this research is to deliver information about behavior of members cast with HSC, prestressed with unbounded tendons and strengthened by externally bonding CFRP sheets. The conclusion of this paper can provide the reference date for the design of such prestressed concrete continuous members.

2. Experimental program

Two continues unbounded post-tensioned high strength concrete I-beam tests were conducted; one strengthened with CFRP sheets named SUPN1-12 and one non-strengthened named UPN1-12. The post-tensioned beams were designed according to ACI 318-11 [14].

2.1. Concrete Mix Design

In this study, the mix design with a target compressive strengths of 95 MPa have been designed by the second author. Table 1 shows the mix proportions of the high strength concrete (HSC) with an average slump test value of 6.5cm. The chemical analysis of cement and micro silica are given in Table 2. The specific surface (Blaine) of Portland cement was 3100 cm2/gr.

Table 1. The normal concrete mix proportions ($1m^3$)

Concrete Type	Water (L)	Cement (Kg)	Gravel (Kg)	Sand (Kg)	Micro silica (Kg)	Filler (Kg)	S.P (L)
HSC	170	575	850	800	70	9.75	6

S.P: SuperPlasticizer

Table 2. Chemical analysis of cement and micro silica

Constituent	Cement	Micro silica
SiO_2	21.74	93.86
Al_2O_3	5.02	1.32
Fe_2O_3	4.05	0.87
CaO	61.72	0.49
MgO	1.23	0.97
SO_3	2.88	0.10
Cl	0.035	0.04
Na_2O+0.658K_2O	0.6	0.974
C_3S	48.53	--
C_2S	25.12	--
C_3A	5.52	--
C_4AF	12.23	--
Ignition loss	1.80	--

Tests were carried out at two ages (i.e., transfer of prestressed force as well as 28 days) and an average value of three specimens was considered as 95 MPa (see later in Table 3).

2.2. Unbounded post-tensioned beams fabrication

2.2.1. Beam specimens

A summary of the test beams and design variables is provided in Table 3. The typical dimensions and relevant reinforcement details of beams are shown in Fig. 1. The areas of the ordinary bonded reinforcement and the prestressing steel were selected to produce reinforcing indexes within the practical range of design.

Table 3. Summary of test parameters

Beams	$f'_{c,i}\,(MPa)$ transfer age	$f'_{c,28}\,(MPa)$	Steel bars: A_s Midspan and internal support (mm^2)	Steel bars: A'_s Midspan and internal support (mm^2)	Prestressing steel: A_p (mm^2)	Prestressing steel: d_p Midspan (mm)	Prestressing steel: d_p Internal support (mm)
UPN1-12	80	95	2Φ12 (226)	2Φ12 (226)	1 (0.6 in) 140	370	370
SUPN1-12	77	93	2Φ12 (226)	2Φ12 (226)	1 (0.6 in) 140	370	370

2.2.2. Preparation of tendons

The 13.9 mm diameter tendons (consisting of 7 wires) were cut to the required length of 10 meter. Three locations (designated as A, B, C) along the prestressing tendons were smoothed and cleaned and the electrical strain gauges were installed at this locations. Fig. 1 shows the electrical strain gauges mounted on tendons. One electrical strain gauge type FLK-2-11-5LT is attached on each location of A, B and C. The tendon profile (Fig. 1) (laying pattern) of the tendon consisting of variable tendon eccentricity of beams is presented by Eq. (1):

$$e(x) = \begin{cases} -0.027x^3 + 0.0822x^2 + 0.0265x + 0.002 & 0 \le x \le 2.175 \\ 0.0661x^3 - 0.6468x^2 + 1.8759x - 1.53 & 2.175 \le x \le 4.35 \end{cases} \quad (1)$$

2.3. Prestressing method, monitoring and measurements of prestressed beams

The prestressing method involved in the study was post-tensioning unbounded system. Post-tensioning was carried out 7 days after concrete casting. The terminals of the electrical strain gauges mounted on the tendons were connected to the data acquisition system. The data acquisition system monitored and recorded the tendon strains during jacking and loading operation to the computer system for further analysis. The tendons in the beams were jacked up to required jacking force using a 22-ton capacity hydraulic jack. The prestressing operation is shown in Fig. 1.

2.4. CFRP properties and bonding procedure

The Young's modulus (Efu) and ultimate tensile stress (ffu) of the CFRP sheet and the properties of epoxies used for bonding the CFRP sheets were obtained from the producer and given in Tables 4 and 5. The process of applying CFRP sheet to concrete; involved surface preparation, priming, resin under coating, CFRP sheet application, and resin over coating. After the beams surface preparation, a two-part primer was applied to the prepared concrete surface. Next, a two-part epoxy resin was applied to the primed concrete surface, followed by application of the CFRP sheet. The CFRP sheet was installed over the concrete surface by starting at one end and moving along the length of the CFRP sheet until completed. Finally, a resin over coating was applied over the CFRP sheet. Concrete beams strengthened with CFRP sheets were cured for at least seven days at room temperature before testing. Details of CFRP strengthening of the test specimens are shown in Table 6.

Table 4. Mechanical properties of the CFRP sheet

Material	Density *(gr/cm³)*	Thickness *(mm)*	Ultimate tensile stress, f_{fu} *(MPa)*	Young's modulus, E_f *(MPa)*	Ultimate strain, e_{fu} *(%)*
CFRP	1.76	0.131	4300	238000	1.8

Table 5. Mechanical properties of the bonding adhesive

Material	Density (gr/cm^3)	Compression strength (MPa)	Tensile strength (MPa)	Young's modulus (MPa)	Shear strength (MPa)
Epoxy resin adhesive	1.11	97.4	76.1	3600	54.8
Epoxy resin primer	1.77	>90	>25	12800	>15

Table 6. Details of CFRP strengthening of the test specimens

Beam no.	CFRP width (mm)	Positive moment strengthening		Negative moment		End anchorage
		No. of layers	Strengthen length (mm)	No. of layers	Strengthen length (mm)	
UPN1-12	-	-	-	-	-	-
SUPN1-12	200	1	6528	1	2673	Yes

The specimens were loaded with two concentrated point loads applied simultaneously at the middle of each two spans. The loading arrangements are shown in Fig. 1. The test measurements included deflections at the midpoints of the right and left spans with two displacement transducers LVDTs. Depending on the diameter of the steel bars three different types of electrical strain gauges were mounted on the main ordinary reinforced bars, stirrups , CFRP sheets and prestressing tendon at specific locations to monitor the development of conventional and prestressing steel strains throughout the loading history. The strain on concrete surface at different locations were also monitored by electrical strain gauges and mechanical demec points (steel less steel discs) 200 mm apart (Fig. 1).

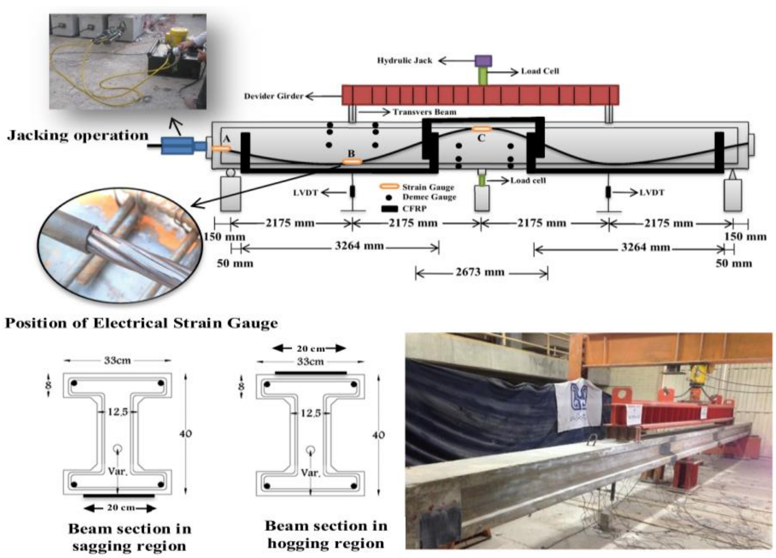


Fig. 1. Typical dimensions, reinforcement details and geometry of test beams

3. General behavior of tested beams

Two different failure modes were observed for tested beams (Fig. 2) and are described as follows. The control post-tensioned beam casted with HSC, UPN1-12, failed in the flexural manner. The tensile steel yielded (Py= 172.6 kN)

prior to concrete crushing at both the central support (Pu = 347 kN) and mid-span section. The wide flexural cracks were occurred at mid-span (wcrmax=21 mm) and central support (wcrmax= 21 mm). These cracks are well extended to the compressive regions. The rupture of tendon was sudden and accompanied by a loud noise and shove end anchorage indicating a rapid release of energy and a total loss of load capacity.

The tensile steel at central support of beam SUPN1-12 yielded at a load of 290 kN. The load was applied further and the beam was failed at a load of 398 kN by intermediate crack (IC) debonding of CFRP sheet at hogging and sagging region and rupture of end strap at hogging region. The wide flexural cracks were occurred at mid-span (wcrmax=8 mm) and central support (wcrmax= 7 mm). By strengthening the UPN1-12 beam using CFRP sheet, the bearing capacity at yielding load increased from 172.6 to 290 kN (an increase of 68%) and at ultimate state increased from 346 to 398 kN (an increase of 15%). Also more number of flexural cracks are occurred and developed towards the neutral axis before beam failure.

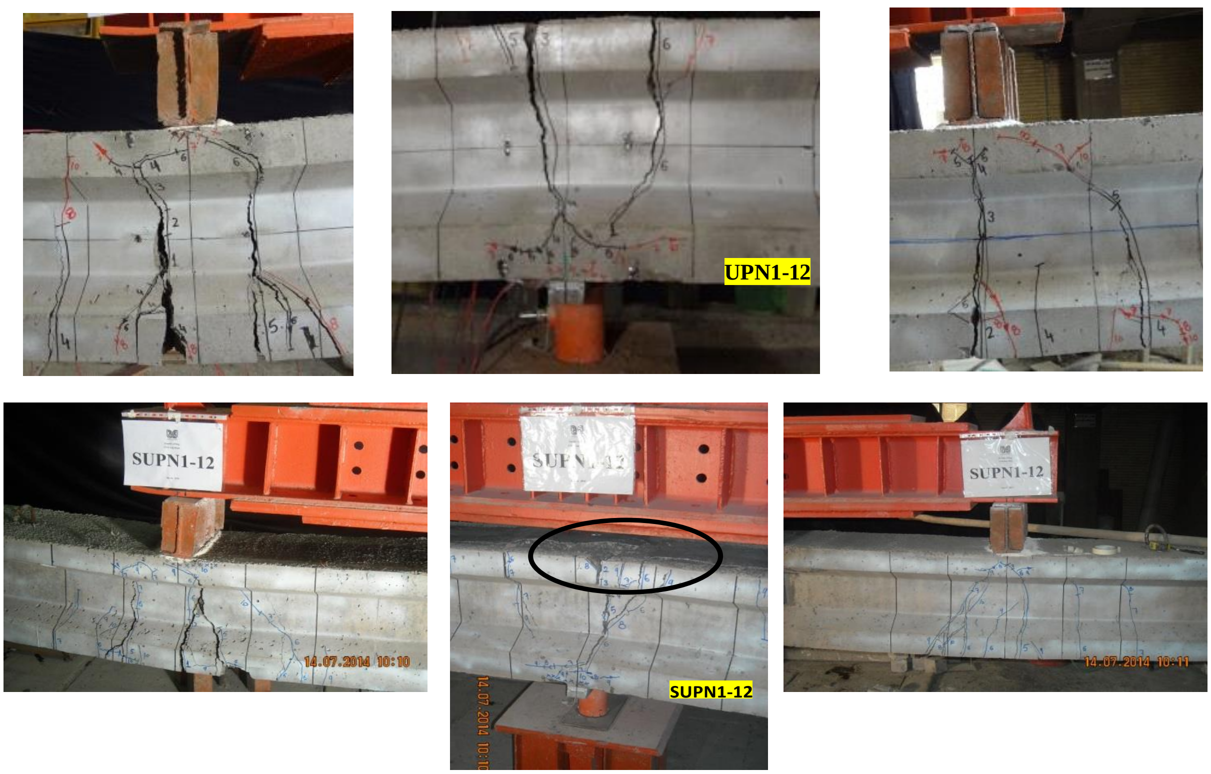


Fig. 2. Typical crack pattern of tested beams

Fig. 3 shows the applied load versus average deflection of the right and left spans for two tested beams. Two tested beams exhibited three stage responses up to failure; representing the concrete precracking stage, concrete postcracking to tension steel preyield stage, tension steel postyield stage to failure. In the uncracked elastic stage, the same behavior was observed for two tested beams, indicating very similar beams stiffness prior to concrete cracking.

In the cracked preyield stage, the stiffness and yield load of the CFRP strengthened beam was slightly larger than that of the control beam and significant decreases in beams stiffness was not observed after yielding the tensile steel at sections of negative and positive moments.

The moment-flexural crack width diagrams of post-tensioned beams are compared and shown in Fig. 4. This Figure shows that crack width is significantly reduced with strengthening of post-tensioned beam.

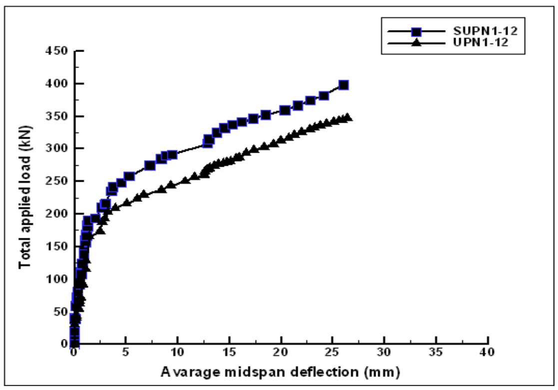


Fig. 3. Applied load versus average midspan deflection

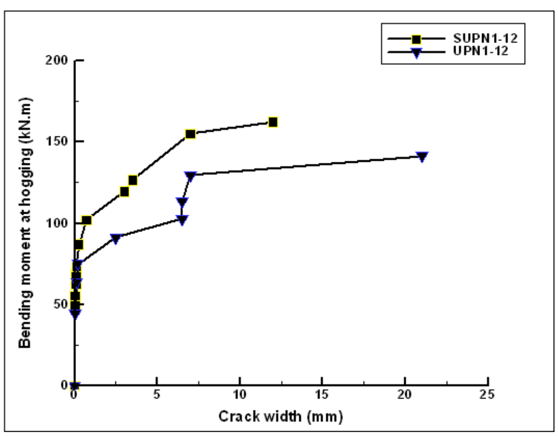
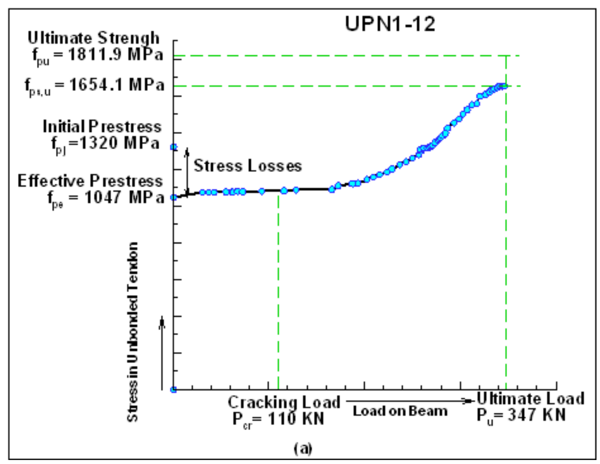


Fig. 4. Flexural crack widths of strengthened and unstrengthened beams

4. Relationship between total P and fps in tested beams

In order to get a clear understanding of the behavior of strengthened HSC beams (i.e., from jacking force up to failure load), the increase in tendon stress monitoring was performed. The variation of prestressing steel stress as the load increases shown in Fig. 5. For two tested HSC beams the initial measured stress in tendon (at jacking operation) was 0.73fpu (fup is ultimate strength of prestressing steel) after total stress losses (immediate and time depended losses after two month) the amount of measured stress in UPN1-12 reduced to 0.58fpu. During load test, stress in tendon increased by loading and reached to 0.91fpu at ultimate load (an increase of 58 %). The change in tendon stress during load test for the SUPN1-12 beam was 528 MPa, increasing by 50%. It was noted that increase in tendon stress decreased by strengthening with CFRP sheets.



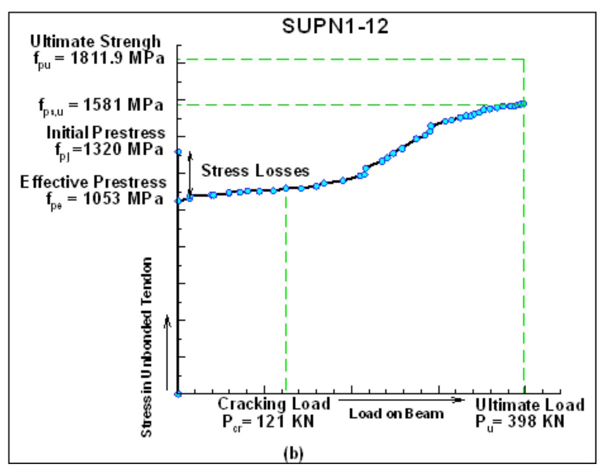


Fig. 5. Variation of prestressing steel stress with applied load for tested beams

5. Comparison of predictions and test results

To have a macroscopically understanding to the precision of equation recommended in relative codes, the results from the analysis of tested beams cast with HSC are compared with ones from the prediction equations given in codes, such as ACI318-2011, BS8110-97 [15] and AASHTO-2010 [16]. Predicted-to-measured increase in tendon stress at an ultimate state ratio was drawn for all the equations given in codes in Fig. 6. Overestimates appear as a predicted-to-measured ratio greater than one and the underestimates as less than one.

5.1. ACI318-2011 equation

ACI-318 recommends the following equation, originally proposed by Mattock et al. [17] and later modified by Mojtahedi and Gamble [18] to account for the influence of the span-to-depth ratio:

for $L/d_p \leq 35$:

$$f_{ps,u} = f_{pe} + 70 + \frac{f_c'}{100\rho_p} \leq f_{py} \text{ or } (f_{pe} + 420) \text{ MPa} \quad (1)$$

for $L/d_p > 35$:

$$f_{ps,u} = f_{pe} + 70 + \frac{f_c'}{300\rho_p} \leq f_{py} \text{ or } (f_{pe} + 200) \text{ MPa} \quad (2)$$

Where fpe is effective pre-stress in the pre-stressing steel, f_c' is compressive strength of concrete, $\rho_p = \frac{A_{ps}}{bd_{ps}}$

Ap is area of prestressing tendons, b is width or effective width of the section or flange in the compression zone, dp is depth from concrete extreme fiber to centroid of pre-stressing steel, fpy is yield strength of pre-stressing steel and L is length of the span.

Substituting the average test values of the material strength of the test beams into Eq. (2-3) and devide to the experimental results is shown in Fig. 6 , The ACI underestimated ultimate tendon stress of two tested beams. This is due to the fact that Eq. (2-3) was derived for fully prestressed simply supported beams and considers only the effect of span-depth ratio, compressive strength of concrete and prestressing reinforcement index, but ignores the effect of strengthened, non-prestressed steel and multi-span or loading pattern in continuous beams.

5.2. AASHTO-2010

Based on research by MacGregor [19] and MacGregor et al. [20] AASHTO LRFD Bridge Design Specification suggests the following equations to determine fps,u :

$$f_{ps,u} = f_{pe} + 6300\left(\frac{d_p - c}{\ell_e}\right) \leq f_{py} \text{ MPa and } \ell_e = \left(\frac{2\ell_i}{2 + N_s}\right) \quad (4)$$

Where c is neutral axis depth at ultimate, ℓe and ℓi are the effective tendon length and length of tendon between anchorages, respectively; and Ns is number of support hinges required to form a mechanism crossed by the tendon. Adopting the average value of the tested strength of the material in the tested beams to Eq. (4) and divide to the experimental results is shown in Fig. 6, where two tested beams are under correlation line; thus, Eq. (4) predicts conservative values for two tested beams.

5.3. BS8110-97

Considering the investigations carried out by Pannell [21], and Pannell and Tam [22], BS8110 adopted the following equation to calculate the stress of prestressing tendon at the ultimate state:

$$f_{ps,u} = f_{pe} + \frac{7000}{L/d_p}(1 - 1.7\frac{f_{pu}A_p}{f_{cu}bd_p}) \leq 0.7 f_{pu} \text{ MPa} \quad (5)$$

Where fcu is the strength of concrete taken from cube tests. Similar to the AASHTO code, the BS code predicts conservative values for two tested beams.

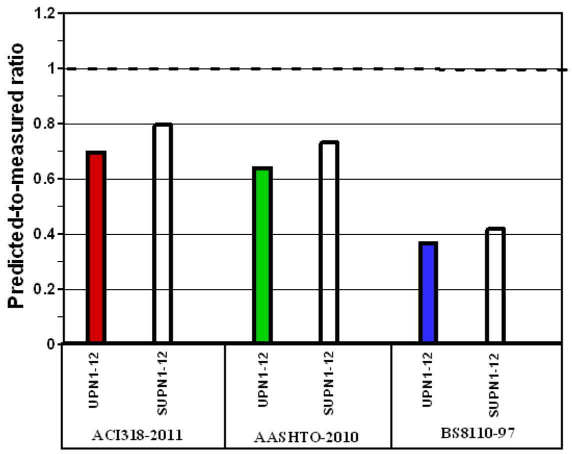


Fig. 6. Comparison between experimental and predicted increase in tendon stress of different standards

The ACI method underestimated the ultimate tendon streess for two tested beams by an average of 10%. The AASHTO method resulted in underestimation for two beams by an average of 12.5 %. The BS method understimated the ultimate tendon streess for two tested beams by an average of 27.5 %.

6. Comparison of experimental ultimate tendon stress increase of strengthened and non-strengthened beams casted with HSC

Fig. 7 shows the comparison of experimental monitored increase in tendon stress at an ultimate state Δfps,u . The experimental (monitored) ultimate tendon stress increase for strengthened beam was lower when compared to non-strengthened beams. As indicated in Fig. 5, the tendon stress in the UPN1-12 beam, increased from 1047 to 1654 MPa (an increase of 58%) while that for SUPN1-12 beam, increased from 1053 to 1581 MPa (an increase of 50%).

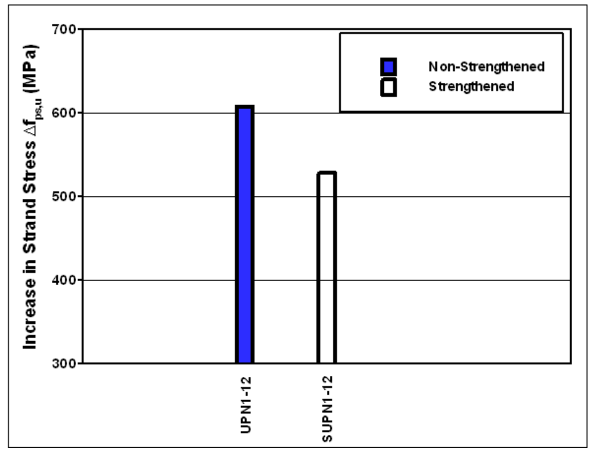


Fig. 7. Comparison between experimental and predicted increase in tendon stress of different standards

7. Conclution

The increase in tendons stress (Δfps,u) of indeterminate (continuous) strengthened post-tensioned unbonded I-beams consisting high strength concrete HSC were investigated experimentally by testing and monitoring of two beams of 9m length. The following results are obtained.

The increase in tendon stress at an ultimate state in strengthened beam consisting of HSC is lower (about of 5%) than non-strengthened beam prepared by HSC of almost the same concrete strength.

Stress in tendon at an ultimate state was estimated using the ACI, AASHTO and BS methods. The ACI (about 10%), AASHTO (about 12.5%) and BS (about 27.5%) methods underestimated the ultimate tendon stress in two tested beams.This results means that, ACI, AASHTO and BS methods are non-conservative to estimate ultimate tendon stress in strengthened and non-strengthened unbounded HSC post-tensioned continuous beams.

It is apparent that ACI produces more accurate results than the AASHTO and AASHTO produce more accurate results than the BS.

In the uncracked elastic stage, the same behavior was observed for two strengthened and non-strengthened beams, indicating very similar beams stiffness prior to concrete cracking. In the cracked preyield stage, the stiffness and yield load of the CFRP strengthened post-tensioned beam was slightly larger than that of the control beam and significant decreases in beams stiffness was not observed after yielding the tensile steel at sections of negative and positive moments.

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