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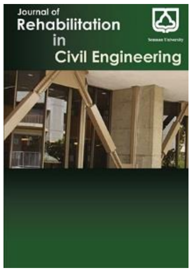

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Serviceability Response of Rehabilitated Unbonded Post-tensioned Indeterminate I-Beams Consisting UHSSCC

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ABSTRACT

The ultra-high strength self-compacting concrete, UHSSCC is the new generation type of concrete with a compressive strength higher than 80MPa. The application of this type of concrete on the serviceability state in CFRP strengthened unbonded post-tensioned indeterminate I-beam is monitored and the results are compared theoretically using different standards. For this aim, full scale I-beam of 9m length was cast, by UHSSCC. During the beam service load test, the stress and strain of materials, and also deflection and crack widths were monitored at different locations using different types of sensors. Based on the experimental measurements and observations, the beam serviceability response was compared theoretically by different methods. As the considerations prepared in the standards do not cover strengthening of such members with unbonded tendons, and also no trace of deflection prediction for such continuous member, one can find in the open literature. It is therefore, this investigation was planned. A comparison between theoretical and monitored results was performed for serviceability response and it was found that although, the stress of materials are well within the standards limitations for crack widths of 0.1 and 0.2 mm of bonded tendon, but the full service load is reached at a higher load, while the flexural crack, experience a width of 0.3 mm. It is also apparent that the loads corresponding to the conventional suggested deflection limits will cause to exceed serviceability state of strengthened unbonded beam, and new limitations are introduced for crack widths of 0.1, 0.2 and 0.3 mm to predict service deflection of beams.

1. Introduction

Self-compacting concrete, SCC is relatively new category of high performance concrete. The mix is smooth enough that it freely passes around and through the reinforcements; it fills the formwork completely and consolidates under its own weight without segregation.

Ultra-high strength self-compacting concrete, UHSSCC offers many advantages over concrete constructions. The ultra-high compressive strength can be used advantageously in structures to reduce dead load and subsequently to reduce total load on the foundation system. The relatively higher compressive strength per unit volume and per unit weight will also significantly reduce the dead load of flexural members.

When the strength of concrete gets higher, some of its characteristics and engineering properties become different from those of normal-strength concrete, NSC [1, 2]. These differences in material properties may have important consequences in terms of the structural behaviour and design of UHSSCC members. The design provisions contained in the major building codes are, in reality, based on tests conducted on NSC. While designing a structure using UHSSCC, the designer particularly in the Southeast Asian region—usually ignores the enhanced properties of concrete and possible changes in the overall response of the structure because of lack of adequate code guidance [3] and so, this is the worst when designing a structure using ultra high strength concrete. Earlier research concluded that although the tensile strength increase with an increase in compressive concrete strength, the serviceability stress in tendons of unbonded post-tensioned members are critical as the strain compatibility is not apply due to to prestressing tendons slip relative to the surrounding concrete.

The calculation of unbonded tendon strain generally requires a complicated procedure. The procedure becomes more complicated when the member even strengthened by CRFP, tendon eccentricity changes with applied loads etc., are included in the analysis. Time dependent effects due to creep and shrinkage of concrete and relaxation of prestressed strands, and the effect due to temperature gradient across the section depth are important in the analysis to predict accurate deflections, strains and stresses in concrete structures at the serviceability limit state.

It is important in the analysis that the unbonded tendon stress be known due to service loads to check the deflection and crack width limitations. To determine the strain in unbonded tendons due to the different loading range, the structural analysis has to be performed to find displacements for the given loads. This becomes difficult as the unbonded tendon stresses are not known a priori, thus iterative methods are generally required. Simplified methods are also available to determine the behavior of concrete structures with internal unbonded tendons due to service loads considering cracking. Most of these methods are however limited to simply support non-strengthened beams with symmetrical loads and tendon profiles consisting of normal (vibrated) concrete.

Advances in concrete technology in many countries have now made practical use of concrete (vibrated or non-vibrated, SCC) with strengths up to 90 MPa. These concretes, with very high compressive strength, can result on the serviceability responses of structural members especially while considering the unbonded post-tensioned tendons where, for such unbonded tendons, the strain compatibility is not apply. It has been found that a very limited research works available while considering the serviceability response in terms of maximum deflection and flexural crack width attainable, may be smaller in continuous unbonded posttensioned rectangular beams consisting of high strength vibrated concrete strengthened by FRPs [4]. In daily service life of structures, serviceability considerations is an important factor in design of UHSSCC members under flexure; consequently the use of UHSSCC in unbonded post-tensioned beams strengthened with CFRP and serviceability that has not been focus neither in Building Codes nor in much of the limited previous experimental research, will be focused in this study.

The failure modes of concrete beams retrofitted with FRP materials and the techniques used in analysing the failure modes were reviewed by Toutanji et al. [5] and Xiong et al. [6]. The behaviour of concrete beams strengthened with externally bonded steel plates, FRP plates, carbon fibre fabric and GFRP sheets was studied both experimentally and analytically. To date, extensive research work has been conducted on the flexural strength of concrete beams bonded with various types of FRP composites [7]. However, considering implementation of ultra-high strength concretes (i.e., NC and SCC) on posttensioned continuous members, a few research reports are available to consider the effect of externally bonding FRP on the unbonded tendon stress of such systems [8-10].

The objective of this investigation is to study the effectiveness of CFRP sheets and UHSSC (f'_c=95 MPa) on serviceability of full-scale unbonded post-tensioned continuous I-beam (L=9 m, b_f =330 mm, b_w =125 mm, h_f =80 mm, h=400 mm). This objective is achieved by conducting the following tasks: i) flexural testing of unbonded post-tensioned UHSSC continuous I-beam strengthened with CFRP sheets reinforced with one unbonded tendon and two tensile reinforcement bars of 18 mm diameter; ii) monitoring the effect of unbonded prestressing tendon, tensile bars and CFRP sheets on the serviceability response; iii) evaluating the flexural crack width and height, and deflection. Comparisons with theoretical model to examine the monitored test beam behaviour in serviceability considerations were also performed.

2. Plastic phase properties and results discussion

The continuous I-beam cast with the SCC having an average compressive strength of 95 MPa in hardened state and the fresh phase tests including: J ring, V funnel, L box and slump flow were also carried out to check the UHSSCC properties of mixes. The tests results on plastic phase of SCC are shown in Table 1. Evaluating the test results, the precast/prestressed concrete institute (PCI) guidelines for SCC [11] was considered as a reference (Table 1), and it was concluded that, the designed mix can be considered as SCC in plastic phase and therefore it was used to cast the post-tensioned I-beam.

3. Beam Detail, Instrumentation and Test Procedure

The bending flexural test was conducted on continuous unbonded post-tensioned ultra-high strength self-compacting concrete I-beam, strengthened by CFRP sheets named as, SUPS1-18. The letters S, U, P and S, is stand for strengthened, unbonded, post-tensioned and self-compacting UHSC respectively, and the number 1 and 18 are indicated as one unbonded tendon of variable eccentricity and 2 ordinary reinforced

Table 1. The results of tests on UHSSCC fresh concrete

Mix	Slump flow		J-ring			L-Box			V-Funnel (Sec.)
	dia. (cm)	T_{50} (Sec.)	dia. (cm)	$\triangle_H$ (cm)	T_{50} (Sec.)	h_2/h_1	T_{20} (Sec.)	T_{40} (Sec.)	
SUPS1-18	71	2.8	69	0.45	3.3	0.84	1.4	3.1	6.3
PCI (2003)	85-55	--	Same as slump flow	1.5≤	2-7	0.8-1	--	--	6-12

Fig. 1. Casting and loading procedure of SUPS specimen

bars of 18 mm diameter located at top and bottom, throughout the length of the beam spans, along with 10 mm diameter bars at a spacing of 100 mm centre to centre for shear reinforcement. The post-tensioned beam were designed according to ACI 318-11 [12] and the post-tensioning was carried out 7 days after concrete casting and the tendons were jacked up to required jacking force using a 22-ton capacity hydraulic jack. The casting procedure and loading of the beam are shown in Fig. 1. To monitor the beam during the test, the electrical resistance disposable strain gauges were pasted on the CFRP sheets and on internal tendon and reinforcing bars at different locations. The demec and electrical gauges were also attached along the height of beam to measure the concrete strains; these values can be used to find out the strain distribution and the moving neutral axis depth of the beam tested. The load was applied step by step in a load control manner of test beam. During the test, vertical deflections were measured using LVDTs (Fig. 2). The purpose of this research is to deliver more information about serviceability response of such members cast with UHSSC strengthened by externally bounding CFRP sheets. The conclusion of this paper can provide the reference date for the design of such continuous prestressed concrete members.

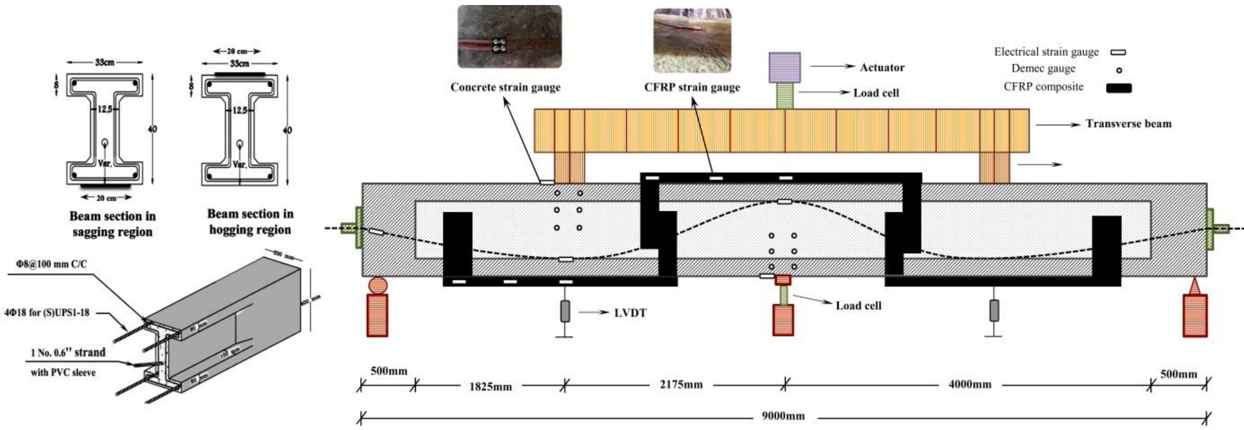

Fig. 2. Beam details and measurement schemes for SUPS test specimen

Table 2. Summary of test parameters

Specimen label	Concrete		Steel bars	Prestressed steel strand			FRP sheets	
	f'_{ci} (MPa)	f'_c (MPa)	As , A′s (mm^2)	Diameter and area of strand (mm, mm^2)	d_p (mm)	f_{pe} (MPa)	n_f, A_f (mm^2)	SL_{hog} and SL_{sag} (mm)
SUPS1-18	81	95	2Φ18 (509)	15.24 (140)	370	826.56	1 (26.21)	2670 and 6530

Designations, areas and diameter of none prestressing and prestressing steel, effective stress in prestressing steel, area of the FRP reinforcement, strengthened length of hogging (SL_{hog}) and sagging (SL_{sag}) regions are summarized in Table 2.

4. Material Properties

4.1 Concrete

The concrete strength of beam was measured by three 100 mm concrete cube specimens made at the time of casting and were kept with the beam during curing. The average 28-day concrete cube strength (f_{cu}) was 120 MPa. The relationship of cylinder strength (f'_c) and cube strength assumed as $f'_c=0.8f_{cu}$ and thus the average compressive strength (f'_c) was 95 MPa. The specific surface (Blaine) of Portland cement was 3100 cm^2/gr. Compressive tests were carried out at two ages (i.e., transfer of tendon prestressed force to the beam, as well as 28 day) and an average value of three specimens was considered for each age.

4.2Non-prestressed and prestressing steel preparations

The 13.9 mm diameter tendons (consisting of 7 wires) were cut to the required length of 10 meter. Three locations along the prestressing tendons were smoothed and cleaned and electrical strain gauges types FLK-2-11-5LT were installed at these locations. Fig. 1 shows the electrical strain gauges mounted on tendon. The terminals of the electrical strain gauges mounted on the tendon and other types of bars 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 (Fig. 2). The tendon profile (laying pattern) of the tendon consisting of variable tendon eccentricity is presented in Eq. (1):

$$e(x)=\begin{cases}-0.027x^3+0.0822x^2+0.0265x+0.002 & 0\le x\le 2.175\\ 0.0661x^3-0.6468x^2+1.8759x-1.53 & 2.175\le x\le 4.3\end{cases} \quad (1)$$

The yield strength, ultimate strength, and ultimate strain of the steel bars, and prestressing strand are also listed in Table (3).

4.3 CFRP properties and bonding procedure

The Young's modulus (E_{fu}), ultimate tensile stress (f_{fu}) 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 beam 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 beam strengthened with CFRP sheets were cured for at least 20 days at environment temperature before testing. Details of CFRP strengthening of the test specimen are shown in Fig. 3.

Table 3. Mechanical properties of prestressing strand and deformed bars

Type of steel	Modulus of elasticity (N/mm^2)	Yield strength (N/mm^2)	Yield strain (%)	Ultimate strength (N/mm^2)	Ultimate strain (%)
Strand	1.8×10^5	1687	1.00	1812	3.53
rebar	2.1×10^5	542	0.22	880	15

Table 4. Strengthening material properties

Material	Density (gr/cm^3)	t_f (mm)	f_{fu} (MPa)	E_{fu} (GPa)	ε_{fu} (%)
CFRP Sheet	1.76	0.131	4300	238	1.8

Table 5. Epoxy bonding material properties

Compression strength at 7 day (MPa)	Bending strength at 7 day (MPa)	Bond resistance (MPa)
85	28	2.07

Fig. 3. Strengthening of UHSSCC beam

5. Behavior of prestressed beams under monotonic loads

The global load-deflection curve of a prestressed concrete beam under monotonic loading until failure is qualitatively described by Fig. 4; it is assumed that the beam has never been loaded before. Point A is the loading level leading to tension concrete cracking. P_c is the cracking load of the beam, calculated by using a conventional model for prestressed concrete beams. For the calculation of the deflection along branch 0-A of the curve, the inertia is fixed at the value of I_o (inertia before cracking). The global stiffness of the beam is noted as k_0. For the cracking phase A-B, the behavior of the beam is modelled by a simple linear interpolation between point A and point B. Point B corresponds to the level of loading which leads to the yielding of ordinary steel bars (P_{y1}). At this load level, much of the span is cracked and considerable damage of the concrete at the interface with the tensile ordinary reinforcing bars is expected. As a result, the residual steel-concrete bond is assumed to be very low and the tension stiffening effect is neglected. Consequently, the inertia of the beam is fixed equal to I_c (the inertia of the cracked cross-section where the concrete tension is not taken into account) for the calculation of the deflection at the value of P_{y1}. The resulting apparent stiffness of the beam (branch 0-B) is k_c. Point C corresponds to the load level leading to the yielding of the prestressing tendons. The yielding load of the unbonded tendons, denoted P_{y2}, is calculated from the estimation of the additional global lengthening before and after cracking. The increase in the deflection between point B and point C, for an incremental bending load equal to P_{y2}-P_{y1}, is calculated by an inertia still fixed at I_c. As the ordinary reinforcing bars have yielded, the elastic modulus of the ordinary steel bars is fixed at the post-yielding hardening value, k_y. Finally, point F corresponds to the failure of the beam classically calculated assuming the crushing of the concrete compression zone or failure of the tension reinforcement. The increase in deflection for an incremental load equal to P_u-P_{y2} is calculated by fixing the elastic modulus of both the ordinary and prestressing steel bars at their post-yielding hardening values k_f. As a result, the global stiffness k_f is significantly less than k_y [13].

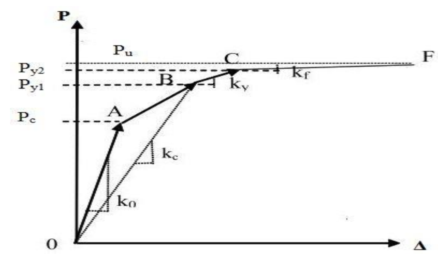

Fig. 4. The global behavior for prestressed beams under monotonic loads [13]

6. Test Results and Discussions

To recognize the importance of serviceability response of FRP strengthening on pre or post-tensioned concrete elements, it seems it is enough to mention the considerations prepared in clause 4.8 of Fib [14] which do not cover strengthening of elements with unbonded tendons. It is therefore, this investigation was planned to prepare information regarding to serviceability response of FRP strengthened unbonded posttensioned beam.

6.1 Serviceability states

6.1.1 Stress limitation

Under service load conditions, the stress limitations which have to be considered for materials stress are as follows:

Extreme fiber stress in tension (f_t): In ACI 318-14 [15] code the primary analysis of prestressed concrete beams is based on service conditions, and on the assumption that stresses in the extreme concrete fiber are limited to values that correspond to elastic behavior. ACI defines three classes of behaviors for prestressed flexural members. Class U members are assumed to behave as uncracked members. Class C members are assumed to behave as cracked members. The behavior of class T members is assumed to be in transition between uncracked and cracked. In other words, prestressed flexural members shall be classified as class U, class T, or class C based on ft, the computed extreme fiber stress in tension in the precompressed tensile zone calculated at service loads, as follows (without any suggestion for their corresponding flexural crack width (w_{cr})):

$$(a)\text{ class U: } f_t \le 0.62\sqrt{f_c'} \qquad (2)$$

$$(b)\text{ class T: } 0.62\sqrt{f_c'} < f_t \le 1.0\sqrt{f_c'} \qquad (3)$$

$$(c)\text{ class C: } f_t > 1.0\sqrt{f_c'} \qquad (4)$$

Extreme fiber stress in compression (f_c): If external tensile reinforcement is added and as the compression force equals the total tensile force, a significant change in the state of concrete stress may be expected. To prevent excessive compression, producing longitudinal cracks and irreversible strains, the following limitations (Eq. 5) are applied for the concrete compressive stress based on ACI-318 [15] without any suggestion for its corresponding strain:

$$f_c < 0.45f'_c \qquad (5)$$

Noneprestressed and prestressing tendon stress: Tensile stress in the steel under serviceability conditions which could lead to inelastic deformation of the steel shall be avoided as this will lead to large and permanently open cracks. This requirement will be met provided that under service load, the tensile stress in ordinary reinforcement does not exceed $0.8f_y$ [14]. Also the stress in prestressing tendons should not exceed $0.75f_{py}$ ($f_{py}=0.9f_{pu}$) after allowance for losses [14].

FRP stress (f_f): In a similar way, based on ACI-440 [16], the FRP stress under service load should be limited as Eq. (6):

$$f_f < \eta f_{fu} \qquad (6)$$

where $\eta < 1$ is the FRP stress limitation coefficient. This coefficient depends on the type of FRP and should be obtained through experiments. Based on creep rupture tests, indicative values of η = 0.8, 0.5 and 0.3 may be suggested for CFRP, AFRP and GFRP, respectively.

6.1.2 Cracking propagation

Fig. 5 shows the service cracks propagation and their development patterns of tested SUPS1-18 beam, under the full service load. Based on ACI-318 [15], the service load is considered to be applied, when a compressive stress of concrete at bottom fiber of inner support section or top fiber of mid-span section one, equals to $0.45f'_c$ (with no recommendation for its corresponding strain value). There are no any established equations for obtaining the complete stress-strain curves for UHSSCC and the descending branch neither in ACI-363 standard [17] for high strength concrete nor in ACI [15] standard, to model the compressive stress-strain behavior of this type of concrete. Therefore, based on Fib [18] the stress-strain equation for high strength concrete up to grade C120 as shown in Eq. (7) was used for accessing the concrete service stress and strain limitations while the full service load is applied. So for tested beam by substituting in Eq. (7), the compressive strain of 0.001 is achieved for the concrete service compression stress limitation of $0.4f'_c$.

$$\sigma_c = \frac{k.\eta-(\eta)^2}{1+(k-2).\eta} \times f_{cm} \qquad (7)$$

where:

σ_c = concrete compression stress

ε_c = concrete compression strain

f_{cm} = is compressive strength of standard concrete cylindrical specimen at 28 days age

E_{ci} = concrete modulus of elasticity equal to $E_{co}\alpha_E[f_{cm}+8/10]^{1/3}$

$E_{c1} = f_{cm} / E_{ci}$

$E_{co} = 20.5\times10^3 \text{MPa}$

ε_{c1} = strain at maximum compressive stress equal to $1.6\times(f_{cm}/10)^{0.25}/1000$

$k = E_{ci} / E_{c1}$

$\eta = \varepsilon_c / \varepsilon_{c1}$ is the plasticity number

α_E = the factor reflecting the effect of type of aggregate on modulus of elasticity

The concrete was not initially pre-cracked and the development of the cracks during the test is highly influenced by the CFRP sheet. Besides, for prestressed concrete, the propagation of the cracks is not quasi-instantaneous because of the prestressing load, which acts against bending crack propagation and widening. The first visible cracking loads at inner support and two mid-span regions are monitored and presented in Table 6.

In the strengthened beam, SUPS1-18, the loads are applied incrementally at different steps. The first visible flexural micro crack of width of 0.02 mm occurred at bottom flange in the positive moment region (i.e., at left mid span) with a cracking load of 76 kN (Table 6 and at step 1 loading of Fig. 5). Two more steps of small load increment was applied (steps 2 and 3) and at step 3, the only first available crack was simultaneously propagated further in the height of the beam without grew wider (causing further decrease in neutral axis depth), and the second visible flexural micro crack was also occurred at left mid span. By slightly increasing the load (i.e., step 4 for a load of 94 kN (Fig. 5 and Table 6), one more visible flexural micro crack of width 0.04 mm opened at top flange of inner support and moved towards the bottom flange while, the

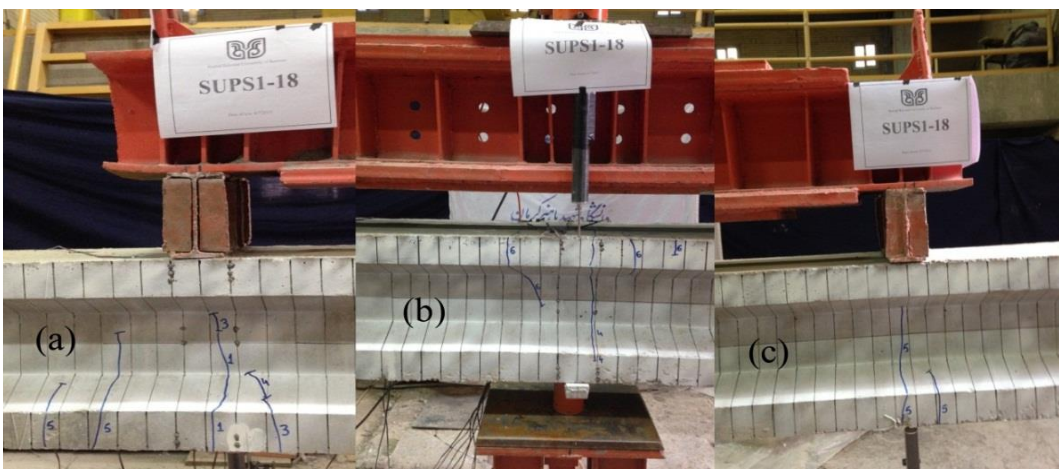

Fig. 5. Service crack pattern of SUPS1-18 specimen at the (a) left mid span, (b) inner support, and (c) right mid-span

Table 6. Cracking load at inner support and two mid-span regions

Beam	Cracking load (inner support)		Cracking load (at left mid-span)		Cracking load (at right mid-span)	
	P_{cr} (kN)	w_{cr} (mm)	P_{cr} (kN)	w_{cr} (mm)	P_{cr} (kN)	w_{cr} (mm)
SUPS1-18	94	0.04	76	0.02	104	0.04

previous second crack which was occurred at load step of 3 at left mid span, propagated towards the web of I-beam without grew wider in the first and second existing crack in this region. At load increment of step 5 for a load of 104 kN, without any propagation in height of the beam for available cracks, two more visible micro cracks were simultaneously appeared at each bottom flange of left and right mid spans consisting a width of 0.04 and 0.03 mm at right and left mid span respectively. However, at this step of loading, no any new crack was occurred at the inner support region (Fig. 5). On further load increment of step 6, only 3 cracks were occurred at top fiber of negative region and were propagated further in the height of the beam towards the bottom flange without causing further decrease in neutral axis depth for the only one existing crack of step 4 loading, but with grew wider in existing crack (Fig. 5 and 6). At this step, the inner support cracks reached a width of 0.3 mm with a corresponding concrete stress and strain of $0.45f'_c$ and 0.000945 (≈0.001) respectively (i.e., the limitation of ACI-318 [15] for full service load) with a Load of 300 kN (Table 8). Therefore, for strengthened beam, SUPS1-18 the full service load is also considered to be reached at step 6 of loading, while compressive stress of concrete at bottom fiber of inner support section equals to $0.45f'_c$ (as recommended in Eq. (7) by Fib [18] or with a corresponding strain of 0.001). Fig. 5 also illustrates that for full service load, the number of cracks that occurred at the left mid span were equal to that at the inner support. The observed crack propagation in SUPS1-18 is similar to past research findings on RC [19] or bonded prestressed concrete [20], due to unbonded strands in rectangular unbonded posttensioned strengthened beams.

The load verses maximum crack width and height at inner support and mid-span section of tested beam is illustrated in Fig. 6.

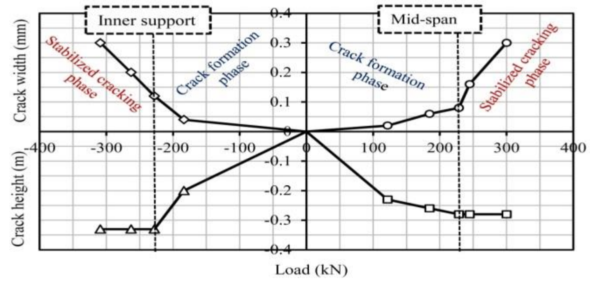

Fig 6. Load Vs. maximum crack width and maximum crack height at inner and mid-span sections of tested beam

As it is shown, two phases of crack formation and stabilized cracking are separated. The crack formation phase includes; increasing the number, width and height of cracks which is until the loads of 208 kN, for the crack height of 280 and 330 mm and crack width of 0.16 and 0.2 mm at midspan and inner support respectively. The stabilized cracking includes increasing of crack width with stabilized cracks number and height, which was occurred at the loads of 300 and 310 kN, crack height of 280 and 330 mm and crack widths of 0.3 mm at midspan and inner support respectively.

6.1.3 Serviceability comparison of monitored (experimental) and theoretical strain and stress limitations:

Based on the above mentioned standards for stress limitations, the strain and stress of materials including concrete, CFRP, noneprestressed steel and prestressing tendons are monitored for different flexural crack widths to access the full service load. Tables 7 and 8 contain enhanced load capacities corresponding to different flexural crack widths. The loads corresponding to two permissible flexural crack widths of 0.1 and 0.2 mm based on BS 8110 limitations [21] for class 3 of pre/post tensioned bonded (non-strengthened) members (which is for sever exposure condition, with a maximum flexural crack width of 0.2 mm), and also for

0.1-0.3 mm width, which is usually based on RC structural limitations for different exposure classes, are presented to indicate the trend of increasing crack width and concrete, CFRP and two types of steel strain to find out the full service load. It is clear that materials including ordinary reinforcement, prestressing tendon, concrete and CFRP sheet, experienced stress values of $0.59f_y$ and $0.81f_y$, $0.43f_{py}$ and $0.54f_{py}$, $0.3f'_c$ and $0.4f'_c$, $0.019f_{fu}$ and 0.03ffu at mid-span and inner support sections respectively, which are well within the mentioned standards limitations ($0.8f_y$ for ordinary rebar, $0.75f_{py}$ for prestressing steel, $0.8f_{fu}$ for CFRP and $0.4f'_c$ for concrete) at crack widths of 0.1, 0.2 and 0.3 mm. To encapsulate, the full service load is reached at a higher load, while the flexural crack, experienced a higher width of 0.3 mm.

BS [21] specifies the value of w_{cr} to be 0.2 mm for bonded pre/posttensioned strands in non-strengthened beams at full service load. It can be seen that under full service load, the higher value of w_{cr}=0.3 mm, with an enhancement ratio of γ=1.5 ($w_{cr0.3}/w_{cr0.2}$) have been driven for strengthened posttensioned unbonded beam of this study. This founding is due to i) CFRP strengthening ii) utilizing of UHSSCC and iii) ordinary bonded bars, in unbonded posttensioned member. As in practice, the third factor is almost available in such beams, therefore, the first two factor causes to conclude that; in unbonded posttensioned beam, it is possible to reach a new increase flexural crack width of 0.3 mm while the full service load is applied. In other words, these

Table 7. Monitored serviceability strain and stress of concrete and CFRP materials at mid-span (MS) and inner support (IS) sections of tested beam

Crack width (mm)	$\varepsilon_c \times 10^{-6}$ (mm/mm)		f_c (MPa)		f_c/f'_c		$\varepsilon_f \times 10^{-6}$ (mm/mm)		f_f (MPa)		f_f/f_{fu}	
	MS	IS	MS	IS	MS	IS	MS	IS	MS	IS	MS	IS
0.1	334	522	13	21	0.13	0.2	67	236	16	56	0.003	0.013
0.2	394	719	16	28	0.17	0.3	231	274	55	65	0.012	0.015
0.3	732	949	29	38	0.3	0.4	348	579	83	138	0.019	0.03

Table 8. Monitored serviceability strain and stress of pre and non prestressed steel at mid-span (MS) and inner support (IS) sections of tested beam

Crack width (mm)	$\varepsilon_s \times 10^{-6}$ (mm/mm)		f_s (MPa)		f_s/f_y		$\varepsilon_{ps} \times 10^{-6}$ (mm/mm)		f_{ps} (MPa)		f_{ps}/f_{py}	
	MS	IS	MS	IS	MS	IS	MS	IS	MS	IS	MS	IS
0.1	614	658	129	138	0.24	0.25	4339	4765	781	857	0.47	0.52
0.2	1019	1481	214	311	0.39	0.57	4364	4829	785	869	0.48	0.53
0.3	1519	2101	319	441	0.59	0.81	4471	4895	804	881	0.43	0.54

Table 9. Total load in accordance with allowable deflections (mm) of tested beam (SUPS1-18)

Δ = L/180 = 24.16	Δ = L/240 = 18.12	Δ = L/360 = 12.08	Δ = L/480 = 9.06	$\Delta_{wcr0.1}$= L/ 2351=1.85	$\Delta_{wcr0.2}$ = L/1553 =2.8	$\Delta_{wcr0.3}$ = L/925 =4.7
P (kN)	P (kN)	P (kN)	P (kN)	$P_{wcr0.1}$ (kN)	$P_{wcr0.2}$ (kN)	$P_{wcr0.3}$ (kN)
514.3	489.6	442.3	371.6	212	244	300

two factors, causes to increase the service load even in unbonded posttensioned beam which is beneficial and economy. However, further tests on this subject are argent on both strengthened and non-strengthened unbonded post tensioned members, especially while using self-compacting concretes.
It is also clear that, by strengthening the unbonded posttensioned beam, the monitored materials strain (Table 7 and 8) are well below the service limitation strain as the width of flexural crack moved from 0.1 mm towards 0.2 mm.

6.2 Deflection response

The prestressed concrete member develops deformation under the influence of two usually opposing effects, which are the prestress and transverse loads. The net deflection of such members at any load stage is obtained as Eq. (8):

$$D_{net}=D_L-D_p \quad (8)$$

Where D_{net} is the net deflection, D_L is the deflection due to self-weight and transverse loading, and D_p the deflection due to prestressing stress after allowance of the losses, which is the effective prestressing stress, f_{pe}.
The serviceability state of total applied load versus mid-span deflection of the tested beam, considering the self-weight and prestressing force deflections as mentioned before is shown in Fig. 7.

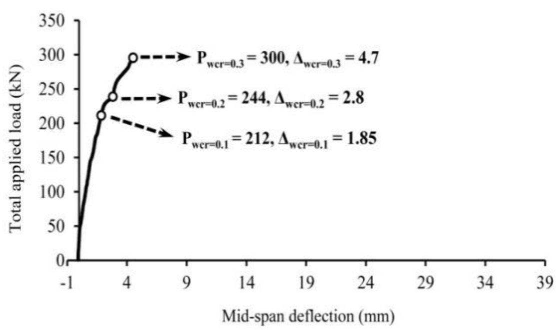

Fig. 7. Serviceability state of load versus mid-span deflection of tested specimen

Unfortunately no trace of deflection prediction for continuous unbonded posttensioned FRP-strengthened beams one can find in the open literature. Accordingly, for the conventionally suggested limits of L/180, L/240, L/360, and L/480 (ACI-318R-14 [15]), the corresponding allowable load limitations are determined. For the 4350 mm long span, these work out to be 24.16, 18.12, 12.08, and 9.06 mm, respectively. The values of experimental loads corresponding to the theoretical allowable deflection limitations, and also deflections corresponding to three crack widths (0.1, 0.2. 0.3 mm) are presented in Table 9. As it is shown, by increasing the crack width from 0.1 mm towards 0.2 mm, an enhancement ratio of 1.15 has occurred for $P_{wcr0.1}$ to $P_{wcr0.2}$ whereas, to access the full service load, this ratio is 1.23 for increasing the crack width from 0.2 to 0.3 mm. It is also apparent that the loads corresponding to the conventional suggested deflection limits (L/180, L/240, L/360, and L/480) will cause to exceed serviceability state, so that such limits are not suitable to predict serviceability deflection of strengthened SUPS beams and new limitations are needed. As it is shown in Table 9 the deflection limitations of L/2351, L/1553 and L/925 for crack widths of 0.1 mm, 0.2 mm and 0.3 mm are proposed for predicting service deflection of SUPS specimens. However, further tests on this subject are argent on both strengthened and non-strengthened unbonded post tensioned members, especially while using self-compacting concretes.

7. Conclusions

Serviceability response of rehabilitated unbonded post-tensioned indeterminate I-beams consisting UHSSCC was monitored and compared theoretically and the following conclusions are drawn:

Based on the above mentioned standards for stress limitations, the strain and stress of materials including concrete, CFRP, non-prestressed steel and prestressing tendons are monitored for three different flexural crack widths of 0.1, 0.2 and 0.3 mm to access the full service load of the beam.

The enhanced load capacities corresponding to three flexural crack widths were found, and for full service load, the higher value of w_{cr}=0.3 mm, with an enhancement ratio of γ=1.5 ($w_{cr0.3}/w_{cr0.2}$) have been driven for strengthened posttensioned unbonded beam. This ratio is mainly due to i) CFRP strengthening, and ii) utilizing of UHSSCC in unbonded posttensioned member, and cause to conclude that; by strengthening unbonded posttensioned UHSSCC beam, it is possible to reach a new increase flexural crack width of 0.3 mm while the full service load is applied, and therefore more beneficial and economy is achievable. However, further tests on this subject are argent.

The loads corresponding to two permissible flexural crack widths of 0.1 and 0.2 mm based on BS 8110 limitations for class 3 of pre/post tensioned bonded (non-strengthened) members (which is for sever exposure condition, with a maximum flexural crack width of 0.2 mm), are presented to indicate the trend of increasing crack width and concrete, CFRP and two types of steel strain to find out the full service load. It was found that, the stress of materials are well within the mentioned standards limitations for crack widths of $w_{cr0.1}$ and $w_{cr0.2}$, however the full service load is reached at a higher load, while the flexural crack, experience a width of 0.3 mm.

In strengthened member, SUPS for all permissible flexural crack widths, materials strains were well below the allowable values.

It was also found that, the loads corresponding to the conventional suggested deflection limits will cause to exceed serviceability state, so that such limits are not suitable to predict serviceability deflection of SUPS beams. Therefore, new deflection limitations of L/2351, L/1553 and L/925 for crack widths of 0.1 mm, 0.2 mm and 0.3 mm are proposed for predicting service deflection of such beams.

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