EXPERIMENTAL STUDY ON THE BEHAVIOUR OF HIGH-STRENGTH PARTIALLY PRESTRESSED T-SECTIONBEAMS

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Abstract

In recent years, high strength concrete (HSC) has been widely used. With advent of super plasticizers and micro-silica; HSC with strength higher than 100 MPa can be reached. This helped in many fields of construction; specifically pretensioned bridge girders. This high strength permitted longer spans, larger spacing between girders thus reducing total bridge cost. Lately, designers have been using partially prestressed members technique, which tends to decrease the prestressing steel and eventually leads to more economical sectional design. The Use of HSC partially prestressed girders is very promising, however HSC was found to be more brittle. Accordingly, the girder may exhibit brittle/less ductile behavior and less deformability. There is also lack of knowledge about the effect of HSC on cracking pattern of such members. This study presents experimental observations for the deflection and cracking behaviour along with ductility study of six tested partially prestressed beams.

Keywords: Partially Prestressed, High-strength Concrete, Deflection Behaviour, Cracking Behaviour, Serviceability.

1- Introduction

In recent years, high strength concrete (HSC) has been widely used. With advent of super plasticizers and micro-silica; HSC with strength higher than 100 MPa can be reached [1]. HSC shows a number of differences compared to normal strength concrete. The brittle behavior comes on top of the list, together with the relatively high Young's Modulus. Also the modulus of rupture, which represents the cracking boundary for the concrete section, its value for HSC is still argumentative till this point. Thus HSC exhibits less deformation, with higher cracking load. HSC helped in many fields of construction; one of these fields is pretensioned bridge girders, which has become an accepted practice in many countries. The high strength permitted longer spans, larger spacing between girders thus reducing total bridge cost.

Contrary to the fully prestressed members, designers adopted partially prestressed members, as having sections free of cracks is not always a serviceability requirement. This technique leads to a cracked section accompanied with reduction in sectional stiffness and increase in deflection. At the same time partial prestressing decreases the prestressing steel, thus produces more economical sections. Using HSC with partially prestressed girders is very promising; however HSC ductility is questioned by several researchers. There is a lack of knowledge about the effect of HSC on the serviceability requirements of such partially prestressed members. Thus a need exists for reassessment of the design provisions for the analysis of these girders [2]. Accordingly, the primary objective of this research was to investigate the deformations for HSC partially prestressed members, and determine the influence of different parameters as the reinforcement ratio and concrete strength on the deflection and cracking behavior.

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In this paper an experimental program for testing six beams is presented. Observations and results are discussed in terms of deflection and cracking behaviour, along with the ductility. The beams varied in compressive strength, prestressing and non-prestressing reinforcement ratios. Results were compared with the current provisions of Egyptian Code of Practice ECP 203-2007 to expand its applicability.

2- Specimens' Details

Six HSC post-tensioned partially prestressed T-beams, with clear simple span of 4.5 m were tested. All beams were designed to have tensile failure, with safe shear capacity. It was considered necessary to test the beams with a realistic span-to-depth ratio that is appropriate and typically used in the design of bridge girders, accordingly all beams had depth of 250 mm and breadth of 150 mm, maintaining a span-to-depth ratio of 18, which is considered an acceptable value in practice. The flange width was 350 mm for all beams. Five of the tested beams were prestressed with one 0.6 inch steel strand, for the remaining two beams one and two 0.5 inch steel strands were used respectively to vary the prestressing steel area. All strands were prestressed by 75% of its ultimate tensile strength, which is 1860 MPa. Three mixes with target compressive strength of 45, 85 and 100 MPa were designed for the experimental program. Several trails were conducted to assure that the required strength is reached after 28 days. The average 28-days cube compressive strength for the three batches were 46.5, 84.5 and 101.3 MPa. The prestressed strands were positioned through the beam span with a polygonal profile as shown in Fig. (1) to decrease the eccentricity at supports and avoid tension stresses. The strand deviation was realized using an appropriate radius of curvature to reduce the friction losses. Beside prestressing reinforcement, ordinary non-prestressing steel bars were used as additional main reinforcement with different steel areas to study the effect of non-prestressing steel ratio. Table 1, presents the details of the experimental specimens in terms of the test parameters.

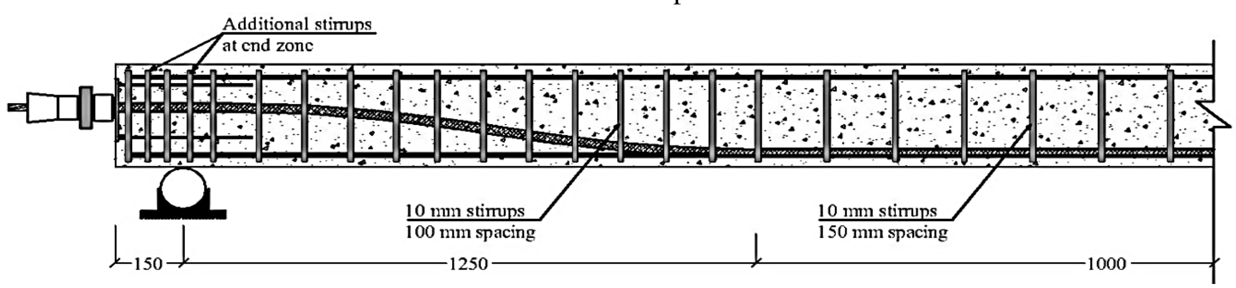


Figure1- Typical sectional elevation for beams

Table 1: Details of the experimental specimens

Beam Designation	f_{cu} (MPa)	Prestressing Strands	μ_p Asp/bd$_P$ (%)	Non-Prestressed steel	μs A_s/bd (%)	Variable
T-85-2-2	84.5	1-0.6"	0.43	2Y10	0.47	Control
T-45-2-2	46.5	1-0.6"	0.43	2Y10	0.47	f_{cu}
T-100-2-2	101.3	1-0.6"	0.43	2Y10	0.47	f_{cu}
T-85-1-2	84.5	1-0.5"	0.31	2Y10	0.47	μ_p
T-85-2-1	84.5	1-0.6"	0.43	2Y6	0.17	μ_s
T-85-2-3	84.5	1-0.6"	0.43	4Y10	0.95	μ_s

The beams were reinforced for shear using normal rectangular stirrups, with diameter of 10 mm, uniformly distributed every 100 mm through the first and last 1.4 m, and between them uniformly distributed every 150 mm. In the first 200 mm of the beam two more stirrups were added to withstand excessive stresses at the anchorage zone that may occur from the prestressing force. The stirrups were tied to two top longitudinal steel bars, 10 mm in diameter. The nominal yield stress of the stirrups steel and the top steel bars was 400 MPa. The flanges were reinforced with extra longitudinal bars at the edge of the flange each side. And a clip was tied to each stirrup. The extra bars and the clips were 6 mm diameter with nominal yield stress of 240 MPa.

The designation for each beam has the first letter as T, indicating that the beam is T-section. The first number represents the concrete target compressive strength, which ranges from 45 MPa to 100 MPa. The second number has the value of 1 or 2. This represents different prestressing steel ratios 0.31 & 0.43, respectively. The third number also has the value of 1, 2 or 3 but indicates the different non-prestressing steel ratios that range from 0.17 to 0.95.

3- Experimental Instrumentation & Loading Procedure

Two electrical strain gauges were installed on the bottom reinforcement bars for each beam before casting. They were positioned at mid-span. For measuring the strain in the concrete, four electrical strain gauges were installed on the concrete surface using epoxy based material. Two of them were used to measure the compression strain in the top flange. They were located at mid-span and next to the spreader beam support. The other two were used to measure the tension strain and were located on the bottom of the web, under the compression strain gauges. A donut shaped load cell was positioned between the anchor plate and the beam's end plate to accurately measure the loss of prestressing force.

The beam's deflection was monitored using four linear variable differential transducers (LVDTs), which were positioned at mid-span, under the spreader beam supports and at 1.48 m from mid-span. The strain and deflection measurements were recorded approximately every 5 kN increment up to the failure load. The cracking behavior has been observed in terms of crack height, width and spacing within the constant moment zone in the beam. Crack widths were measured at constant level, 15 mm from the bottom surface of the concrete, which is the same level of non-prestressing reinforcement. Fig. (2) shows schematic drawing for the test setup and instrumentation positions.

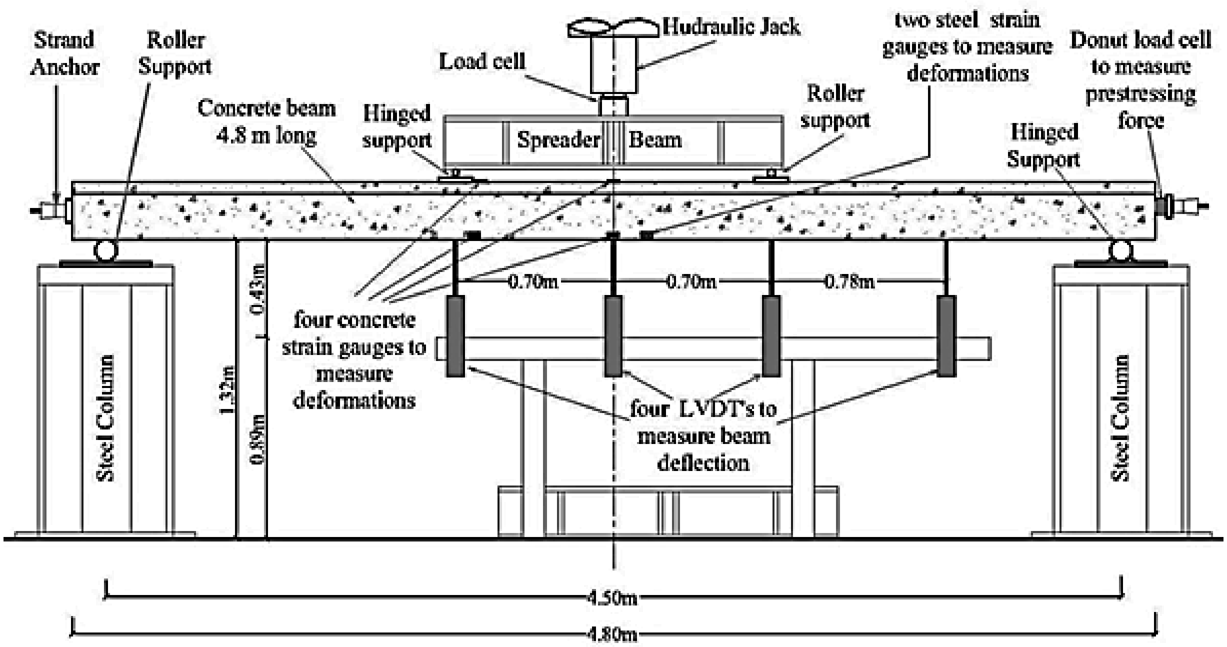


Figure 2- Schematic of the test setup

The beams were tested using quasi-static monotonic two point concentrated loads. The load was cycled several times before the beam was loaded to failure. The aim of these loading cycles was to study the deflection behavior pre-cracking and post-cracking, and to examine the beam's loss of stiffness due to micro-cracking.

4-Experimental Results

4-1-Load-Deflection Behaviour

It was observed that Load-deflection relation maintained its linear behaviour for all the tested beams before cracking occurred (cycle 1-2-3). Fig. (3) shows the typical load versus mid-span deflection during the first four cycles for the control beam. It clearly shows loss of stiffness and increase of deflection rate after passing the cracking load through the third cycle. After the third cycle a residual deflection was detected when the load was released, proving that cracks took place and the stiffness was reduced. During the final loading (indicated as 5th Cycle), it has been noticed that the rate of deflection at higher load level increased excessively and almost linearly to failure. This was attributed to yielding of reinforcement as shown in Fig. (4).

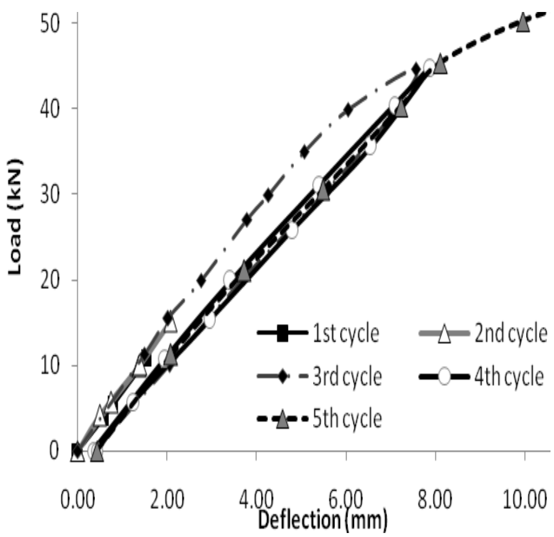


Figure 3 - Typical load-deflection behaviour at mid span deflections during load cycles

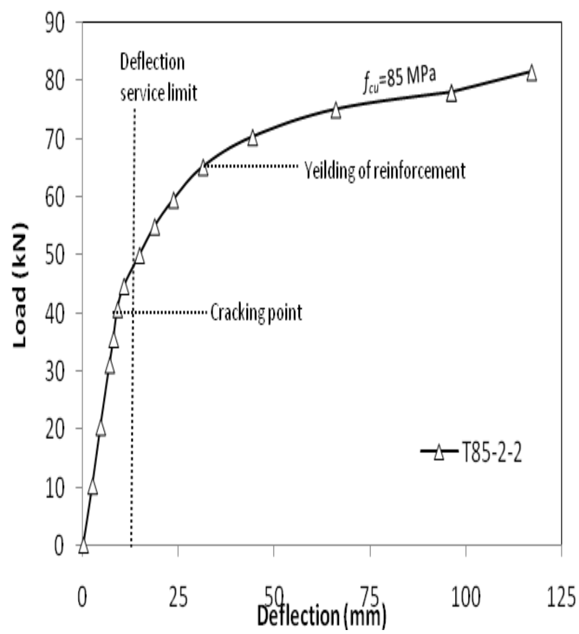


Figure 4 - Load-deflection Behaviour at mid span for control beam

For beams with various compressive strengths, load deflection behaviour is plotted in Fig. (5). It is clear that as concrete strength increases beams behaved in stiffer manner. The deflection increased as the compressive strength decreased. Prior cracking the stiffer manner is attributed to the higher elastic modulus of HSC. The cracking load itself for HSC increases as it is highly dependent on the concrete modulus of rupture, which is relative to the concrete compressive strength. Thus, the uncracked phase was prolonged and reduced the overall deflection at the same load level.

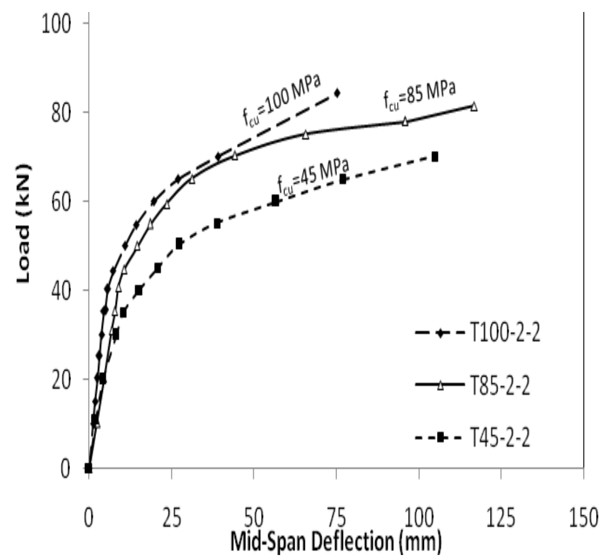


Figure 5- Loa-deflection behaviour at mid span for beams with various compressive strengths

While studying the effect of increasing prestressing and non-prestressing reinforcement, both Fig.(6.a) and (6.b) indicate that deflection decreases as reinforcement ratio increases. However the section capacity relatively increases, thus at failure load, both upper and lower limit beams showed almost the same ultimate deflection.

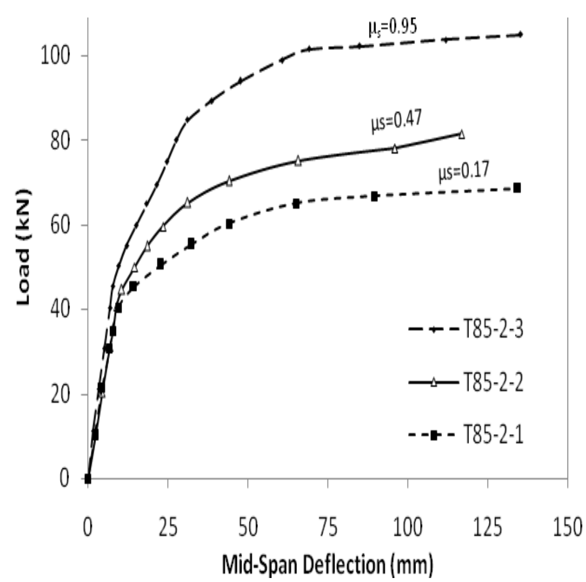


Figure 6-a-Various non-prestressing steel ratios

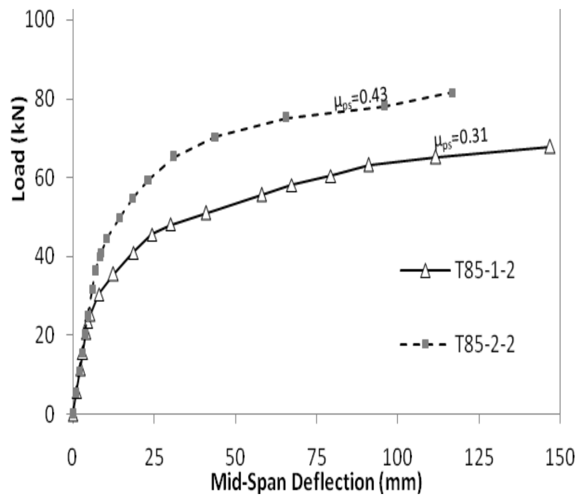


Figure 6-b-Various prestressing steel ratios

Figure 6- Load-deflection behaviour at mid span

4-2-Cracking Behavior

Cracking moment in non-prestressed members depends mainly on the modulus of rupture of the material. However, for prestressed members, the prestressing force contributes in raising the cracking load level. In the experimental program, despite all beams were jacked with the same force, variation in anchorage losses affected the prestressing force. Thus both T85-2-2 & T100-2-2 cracked at the same load as the losses of the later were higher.The beam with lowest compressive strength cracked at 25% lower load due to the lower modulus of rupture as seen in table 2.While non-prestressing reinforcement hardly affected the cracking moment value, prestressing reinforcement role was obvious through beam T-85-1-2, which cracked at 40% lower level than the control beam.

Table 2: Measured response at service load

Beam Designation	1st crack load (kN)	Load Ps (kN)	No. of Cracks	Av. crack width (mm)	Av. Crack spacing (mm)	Max. Crack spacing (mm)
T-85-2-2	40.00	59.51	11	0.16	138	175
T-45-2-2	29.87	40.14	10	0.13	144	285
T-100-2-2	39.36	50.14	8	0.11	181	270
T-85-1-2	23.9	40.94	10	0.225	157	160
T-85-2-1	37.89	45.37	7	0.2	225	335
T-85-2-3	44.85	69.50	14	0.12	95	155
* Ps is the measured load at service maximum crack width of 0.25 mm.						

The cracks in the constant moment zone were examined. Generally, the cracks started perpendicular to the centerline of the beam, and then they propagated almost vertically till it reached the flange. With load progression the cracks extended to top flange. As previously mentioned by several researchers experimental findings, the steel stirrups required for shear reinforcement actedas crack initiator [3]. All the beams had the same behaviour as shown in Fig. (7). The measurements show that the crack width increases almost linearly with the applied load. When reinforcement yields, the crack width increases with much higher rate.

While monitoring crack width propagation, results showed that at same load level relative to the cracking load, crack width increases with almost same rate and value for the beams with different compressive strength, as seen in Fig.(8), this behaviour is maintained during the service load. It points out that compressive strength may affect the cracking moment through the modulus of rupture, yet it does not influence the crack width propagation rate during service load.

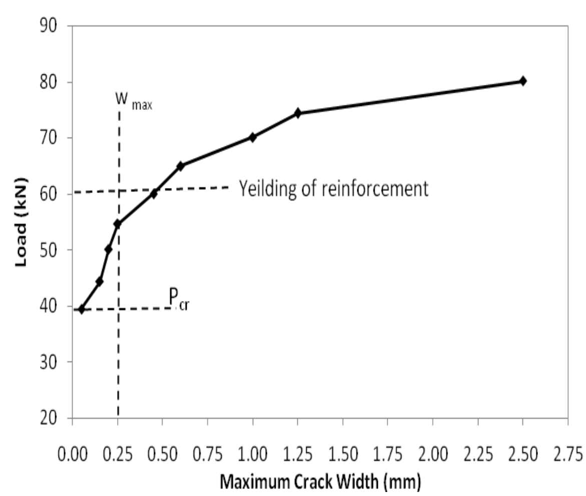


Figure 7- Typical load-crack width relationship for beam T-100-2-2

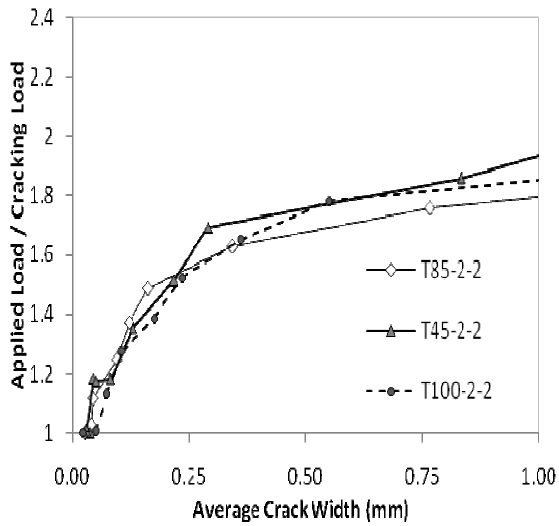


Figure 8- Applied to cracking load ratio vs. crack width for beams with various compressive strengths

On the other hand, changing the reinforcement ratio highly affected the crack width propagation as seen in Fig. (9). As non-prestressing steel ratio increased, crack width decreases, while crack spacing decreases as shown in Fig. (10). This concurs with most of literature conclusion, that ordinary reinforcement is considered the prime parameter used for crack control. It was noticed that beam T85-2-1 had relatively quick propagation of crack width, this is attributed to the ordinary mild steel reinforcement used with 240 MPa yielding stress. Although crack spacing decreased to 80 mm from 200 mm while increasing ordinary steel ratio from 0.17 to 0.95, the permissible crack width was reached at much higher load level, increasing from 44.5 KN to 80.5 KN load level.

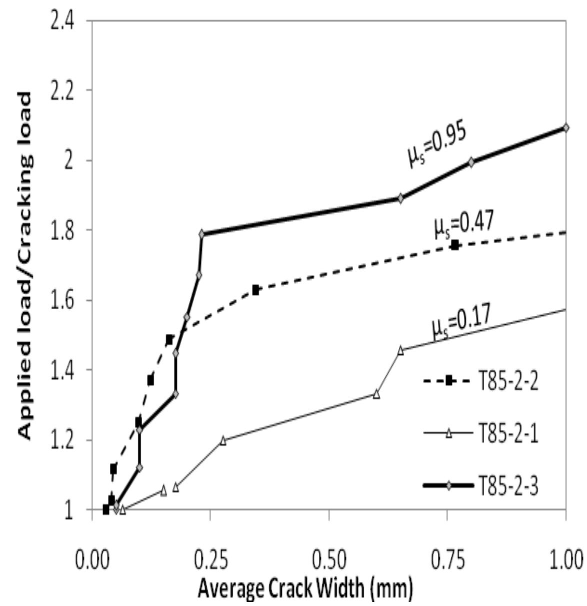
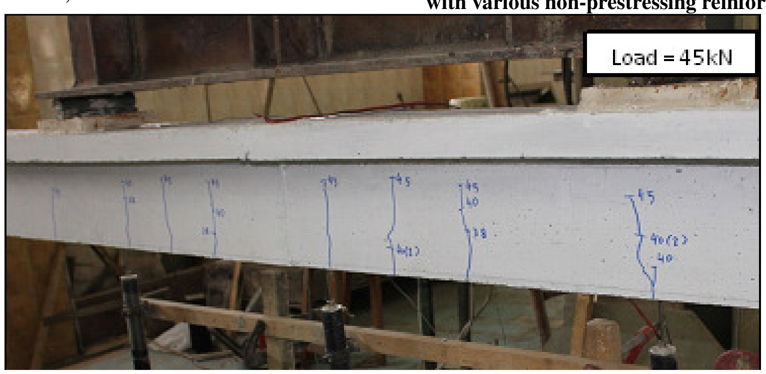
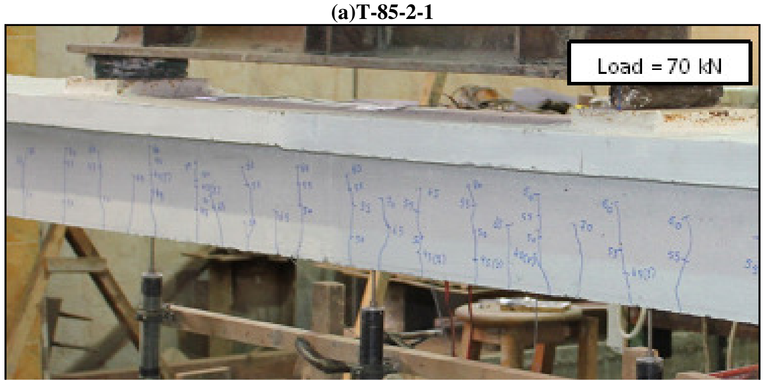


Figure 9- Applied to cracking load ratio vs. crack width for beams with various non-prestressing reinforcement



(a)T-85-2-1



(b)T-85-2-3

Figure 10- Crack Pattern for tested beam at service crack width of 0.25 mm

4-3-Ductility

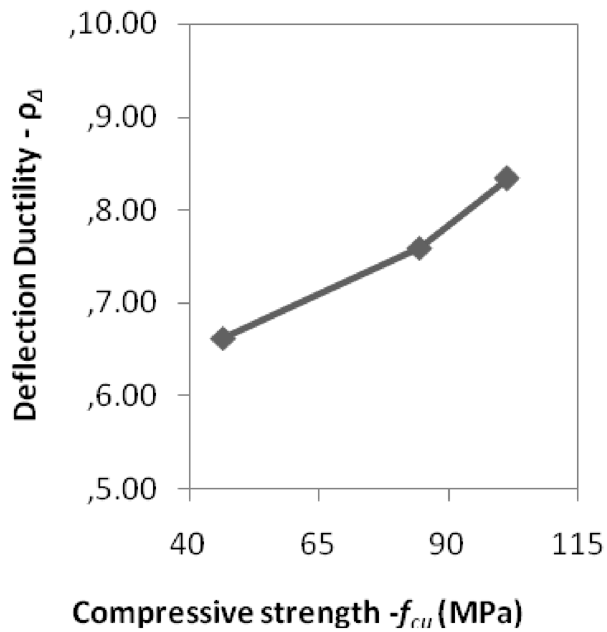
To compare between beams in terms of the ability to sustain inelastic deformation before collapse, both curvature and deflection ductility indices were calculated from the experimental results. It was important to measure both indices as curvature ductility represents mainly the sectional behavior ductility; while deflection ductility which evaluates the whole member's ductility behavior. Generally, the ductility index is the ratio of ultimate to yield deformations. Since partially prestressed members contain both prestressing steel and non-prestressing steel, many methods were provided to determine the yielding point of the member. A graphical method recommended by Namaan et al [4] was used to point out the yielding point by transforming the load-deflection curve into simplified bilinear relation defining the yielding point.

Mid-span curvature was calculated using the top mid-span concrete strain from the experimental results, and the calculated neutral axis depth based on strain compatibility approach. The moment-curvature diagram for mid-span section was plotted except for one beam, were the measurements contained some errors. To determine the yielding curvature point, the load that triggered the yield deflection was used to determine the yielding moment. Using the moment-curvature diagram the yielding curvature was indicated. Table 3 presents both deflection and curvature deformations and their ductility indices.

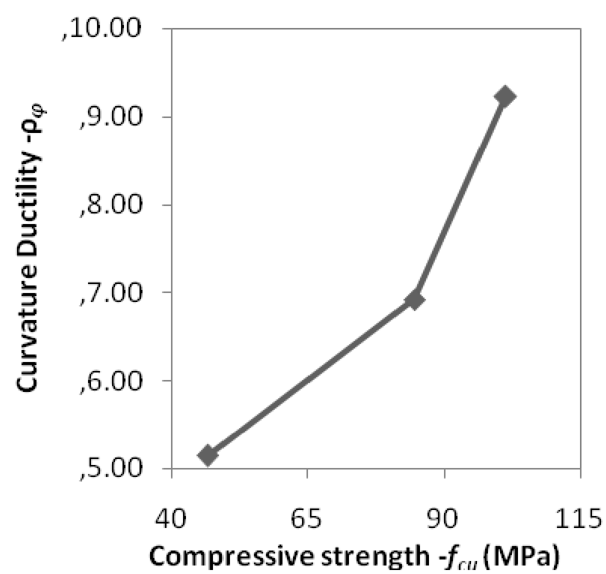
Table 3- Yielding and ultimate and ductility indices for both deflection & curvature

Beam Desig-nation	Δ_y (mm)	Δ_u (mm)	Deflection Ductility (μ_Δ)	Φ_y (mm^{-1})	Φ_u (mm^{-1})	Curvature Ductility (μ_Φ)
T-85-2-2	15.42	117.05	7.59	5.47	37.85	6.92
T-45-2-2	15.97	105.8	6.62	7.48	38.47	5.14
T-100-2-2	9.11	75.94	8.34	6.85	63.22	9.23
T-85-1-2	11.47	146.53	12.78	-	-	-
T-85-2-1	15.45	139.94	9.06	3.75	35.08	9.35
T-85-2-3	17.7	135.52	7.66	8.45	55.96	6.62

The results show that as compressive strength increases from 45 MPa to 100 MPa the ductility index for both curvature and deflection increases, as shown in Figure (11). This confirms with several studies [5], [6]. Increasing the ordinary reinforcement ratio from (0.17) to (0.47) reduced the ductility immensely as shown in Fig. (12), however further increase in the non-prestressing reinforcement reduced the ductility with much lower rate. Some studies attributed this reduction in ductility to the tension steel, which diminishes the rotation capacity of the member [6]. As for the prestressed reinforcement ratio, there were no sufficient data to adequately determine its influence on ductility.



(a) Deflection ductility



b)Curvature ductility

Figure 11- Ductility indices versus concrete compressive strength

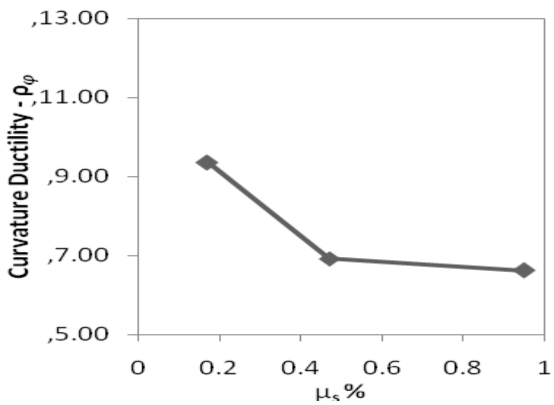


Fig. 12-Curvature ductility index versus non- prestressing reinforcement ratio

5- Determination of service load based on Egyptian Code of Practice

According to the Egyptian code of practice for concrete structures [7], there are several boundaries that limit the service load of a prestressed member. Maximum allowable stress applied on the member, either compression or tension is considered as one of the most influential boundaries that fall under the working stress design method. The Egyptian code classifies the partially prestressed members to category "D", which limit the fictitious tensile stress in concrete to ($0.85\sqrt{f_{cu}}$), neglecting the cracks and reinforcement. Meanwhile, maximum compressive stress allowed is (0.40 fcu).

For having a structural member with acceptable serviceability performance, the Egyptian code of practice also specifies serviceability limit states, which includes both deflection and cracking limits. The allowable immediate deflection for beams due to live load is 1/360 of the span length. From another point of view, the Egyptian code considers four categories for maximum crack width depending on the concrete surface at the tension side; the maximum allowable crack width is set to be 0.3 mm or 0.2 mm for indoor or outdoor members respectively. In this study an approximate value between both has been suggested of 0.25 mm.

In order to determine the maximum service load allowed by the Egyptian code, the loads which caused the previous limits have been either calculated - in case of tension stress limit - or pointed out from load-deflection or load-crack width diagrams. The lowest load represented the maximum allowable service load. Table 4 represents these loads.

Table 4: Applied loads at working and serviceability limits

Beam designation	Allowable load (kN) & based on							
	Working stress limits				Serviceability limits			
	Tension ($0.85\sqrt{f_{cu}}$)		Compression (0.40 fcu)		Deflection (L/360=12.5mm)		Crack-Width (0.25 mm)	
T-84.5-2-2	27.81	(34%)	126.72	(155%)	46.85	(57%)	59.51	(73%)
T-46.5-2-2	23.18	(33%)	71.47	(102%)	36.12	(52%)	40.14	(57%)
T-101.3-2-2	25.94	(31%)	148.54	(176%)	51.25	(61%)	54.63	(65%)
T-84.5-1-2	33.3	(49%)	130.37	(192%)	35.75	(53%)	40.94	(60%)
T-84.5-2-1	26.49	(39%)	130.95	(191%)	42.78	(62%)	44.10	(64%)
T-84.5-2-3	31.81	(30%)	129.38	(123%)	55.04	(52%)	80.32	(77%)

* The underlined value represents the minimum value. i.e. service load.
* The percentage is relative to the ultimate load.

As shown in Table 4, the allowable working tension stress in considered the parameter that controls the service load, approximately 36% of the ultimate capacity. The compression working stress limit, unlike the tension, is highly overestimated for these tested beams. For the deflection and cracking limits, they were reached at about 53% and 65% of the ultimate load, respectively. This indicates that the working load limit for tension stresses is highly conservative. Meanwhile according to the ACI code [8], class C which represents the partial prestressed sections does not indicate an allowable stresses for tension nor compression, as long as the deflection and crack width do not exceed the allowable limits. Both American and Egyptian codes have the same allowable crack width and deflection limits. So for the tested beams it is obvious that neglecting the tension stress limit as specified in ACI would increase the service load by 47%, as it will be controlled by the deflection limit criteria.

Conclusions

From the results of the experimental program reported above, the following conclusions could be pointed out:

* For un-cracked sections, increasing compressive strength from 45 to 100 MPa confirmed obvious

reduction in the deflection by about 30% due to the improved Young's Modulus for HSC. Meanwhile, post cracking behavior also improves due to the higher modulus of rupture, which raises the cracking load thus improves the beams stiffness at the same load level.

* The non-prestressing reinforcement ratio does not influence the deflection behavior prior to cracking, or the cracking moment. However after passing the cracking load, much higher stiffness is acquired for beams with higher reinforcement ratio.

* The prestressing reinforcement ratio also doesnot influence the deflection behavior prior cracking, unlike the cracking load, which is directly related to the prestressing force, thus as the prestressing reinforcement ratio increases, cracking load increases, which reduces the overall deflection at the same load level.

* At same applied to cracking loads ratio, beams with different compressive strength showed the same value of crack width, which clarifies that compressive strength along service load, does not influence the crack width propagation.

* Crack width increases with respect to the applied load in linear fashion. The rate of crack width increases immensely after the non-prestressing reinforcement exceeds the yielding stress.

* The ductility index increases as compressive strength increases from 45MPa to100MPa, while it decreases when the reinforcement ratio increases.

* According to the ECP 203-2007 the tensile working stress limit controls the service load with conservative value of about 36% of ultimate load, while cracking and deflection limits are reached at approximately 60% of ultimate load. However, further experimental and analytical research is needed to study this aspect comprehensively to determine whether it is appropriate to change the tension stress limit specified in the code.

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