EXPERIMENTAL INVESTIGATIONS ON STRENGTHENING OF RC BEAMS BY EXTERNAL PRESTRESSING

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ABSTRACT

Experimental investigations have been carried out on five specimens, out of which three were of single-draped tendon profile and two were of straight tendon profile. Rectangular RC beams of section size 150 mm X 275 mm with 4 m length were used for investigation. Crack was induced in RC beams to a limit in which strain in reinforcing steel was around 85 % of the yield strain by monotonically increased static two-point load at flexural zone. Strengthening by external prestressing was done while the member was subjected to superimposed dead load of a bridge girder, equivalent to 25 % of the calculated ultimate load of the specimen. Strengthened members were tested by monotonically increased two-point load. Role of the reinforcing steel were observed from electrical strain gauges, which were fixed throughout the length of the beams. It was observed that the ultimate load carrying capacity of strengthened members have increased 48 % and 17 % for single-draped tendon profile and straight tendon profile respectively.

Keywords: Experiment; RC beams; flexure; strengthening; external prestressing.

1. INTRODUCTION

External prestressing is basically a post-tensioning method, in which tendons are placed entirely outside the concrete members and the prestressing forces are transferred to the member through anchorages and deviators. External prestressing is being used for strengthening of distressed concrete bridges and also for new bridges like segmental constructions. Because its applications are wider in structural engineering field, the technique is becoming popular. In India, some of existing concrete bridges were strengthened using this technique and also it is being used for new construction. In

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continuous application of the technique for strengthening distressed concrete structures, credibility of the technique in improving the performance in service load behaviour has been proved. However, some of the bridges strengthened using external prestressing has developed sign of distress even after strengthening. Therefore, it is very much necessary to investigate the behaviour of concrete members strengthened by external prestressing technique at their post-strengthening stage.

2. REVIEW OF PREVIOUS WORKS

External prestressing is analytically being treated as unbonded tendons and therefore analytical solutions of members prestressed with internal unbonded post-tensioning tendons are applied for externally prestressed members. Many investigations have been done on subjects: stress at ultimate in unbonded tendons and external tendons [1], external prestressing regarding flexural strength [2], tendon stress [3], compatibility of cable [4], friction at deviators [5], second order effects [6], behaviour of multiple deviators [7], behaviour of deviators [8], highly eccentric external tendons [9], stress in unbonded and external tendons [10], assessment of tendon stress [11], flexural behaviour of externally prestressed beams [12], partial prestressing with external steel tendons [13], plastic rotation capacity of members prestressed with external tendons [14], proposal of design equations for stress in External and Internal Unbonded Tendons [15], inelastic behavior [16], partial prestressing with external CFRP tendons [17] . All the above studies and their observations are useful to the members strengthened using external prestressing, since external tendons are analytically treated as unbonded tendons. However, basically the studies are intended to solve issues for new concrete members and not for distressed concrete members. Few works have concentrated on strengthening of distressed concrete members by external prestressing. Harajli [18] tested sixteen beam specimens which were earlier cracked and then strengthened by external prestressing, and observed that flexural strength of the strengthened members were increased by 146 % and induced deflections were reduced by 75 %. Ghallab and Beeby [19] tested twelve prestressed concrete beams strengthened by external tendons using G Parafil rope, out of which three were cracked prior to strengthening. They suggested that if the internal reinforcing steel does not yield at precracked stage, then the strengthened member at ultimate can be analysed as same as uncracked strengthened beams. Another investigation of the same kind is done by Elrefai et al.[20], in which RC beams were precracked at stages: overloaded (internal steel is yielded) and non overloaded (internal steel is not yielded), then they were strengthened by external prestressing using Carbon-Fibre Reinforced Polymer Tendon. They observed that beams overloaded prior to strengthening did not have discernable effect on the beam fatigue life in comparison with that of non overloaded beam. Antony Jeyasehar and Mohankumar [21] investigated the reliability of external prestressing on strengthened beams using only straight tendon profile, which were earlier damaged by corrosion, and observed that stiffness increased in strengthened beams compared to undamaged beam. Sirimontree and Teerawong [22] tested a full-scale prestressed concrete highway bridge girder with RC deck slab, which was earlier cracked up to the level of inelastic cracked stage for

two times and then strengthened by external prestressing, and found that the required external prestressing force to recover structural performance of a damaged girder depends directly on the damage index, which is the ratio of permanent deformation to the crack deformation of the reference undamaged girder.

3. RESEARCH SIGNIFICANCE

In concrete members prestressed with unbonded post-tensioning tendons, the stress-increase in unbonded tendons happened almost after yielding of untensioned steel and went up to the concrete crushing in the extreme compressive fiber of the member. This phenomenon leads to the presence of plastic hinge in internal post-tensioned members, which were observed and reported by researchers [10] & [11]. In case of precracked RC or prestressed concrete beams strengthened by external prestressing, it could be observed that failure of the member occurs immediately after the yielding of untensioned steel and therefore, there is only a little presence of plastic hinge [18],[19]&[20]. This also exhibited the reduction in ductility. In view of this, it is better to investigate the behaviour of strengthened members by taking into account of the role of untensioned steel at ultimate state. An experimental investigation was carried out to study the behaviour by closely monitoring the reinforcing steel. This paper presents the results on testing of five RC beams, which were pre cracked to a known limit (reinforcing steel reaches 85 % of yield strain) and then strengthened by external prestressing using single-draped tendons and straight tendons.

4. EXPERIMENTAL INVESTIGATIONS

4.1 Material testing of cement, concrete, reinforcing steel and high tensile steel wire
OPC 53 grade cement of specific brand and particular manufacturing unit was used continuously for the investigation of cement strength, concrete strength and casting of beams, to avoid variations in data. Test on standard consistency of cement was carried out as per Indian Standard, IS: 4031 (Part 4)-1988 [23], and the P was observed as 29 %. P is defined as the percentage of weight of water added to total weight of cement. Cement strength test was performed by casting cement mortar cube of size 70 mm x 70 mm x 70 mm in the ratio CM 1:3 as per IS: 4031 (Part 6)-1988 [24], and the average 28 days strength was observed as 55 MPa. Based on the cement strength the water-Cement ratio was obtained as 0.47 as per IS: 10262-2009 [25]. Accordingly the mix design was carried out using w/c ratio 0.47 as per ACI method. Three w/c ratios such as 0.43, 0.47 and 0.51 were selected for trail mixes and 10 nos. of cubes were cast for each ratio. Compression tests on cubes were carried out and the average compressive strength of cubes cast with water-cement ratios 0.43, 0.47 and 0.51 were observed as 55 MPa, 47.30 MPa and 42.5 MPa respectively. The cubes cast with 0.51 w/c ratio had given better results than that of others. Therefore water-cement ratio of 0.51 was fixed and accordingly the mix ratio of 1: 1.59: 2.79 was decided to achieve concrete strength of 40 MPa. Tension test on 12 mm reinforcement bar was carried out as per ASTM standards: E8/E8M-09 [26]. The yield strain and the yield stress were observed

as 0.0023 and 433.70 MPa respectively. Failure strain and the failure load were observed as 0.034 and 54 kN respectively. Young's modulus of the specimens was observed as 2.1×10^5 MPa. Tension test on High tensile steel wire (7 mm dia) was conducted as per ASTM standard: A 370 [27] and ultimate strength was observed as 1478 MPa.

4.2 Casting of beams
RC beam of rectangular section of size 150 mm X 275 mm with 4 m length was designed. Span-depth ratio of 14.97 was chosen for all the specimens. The reinforcement details are given in Table 1.

Table 1: Details of Reinforcements and HTS wire

Beam designation	Details of tensile reinforcement	Details of compressive reinforcement	Details of HTS wire	Effective prestress f_{pe} MPa	Type of tendon profile
EPS-B1	2-12 mm dia	2-8 mm dia	2-7 mm dia	287.7	Single-draped
EPS-B2	2-12 mm dia	3-8 mm dia	2-7 mm dia	430.5	Single-draped
EPS-B3	2-12 mm dia	2-8 mm dia + 1-10 mm dia	2-7 mm dia	328.65	Single-draped
EPS-B4	2-12 mm dia	2-8 mm dia	2-7 mm dia	315	Straight
EPS-B5	2-12 mm dia	3-8 mm dia	2-7 mm dia	516.6	Straight

Electrical strain gauges (5 mm) were pasted to the tensile reinforcement at 150 mm interval throughout the effective span of the beams. They were housed in the sleeves for protection during casting. wooden mould was fabricated to cast the beam. The beam casting was carried out by following all the procedures such as preparation of materials, weighing materials, proper mixing of concrete using mixer machine, vibrating concrete, casting of representative cubes and finishing of beams. The casting of beam is shown in Fig. 1.

Figure 1. Casting of beam

4.3 Preparation of beam and Instrumentation
End plates were fixed at both the ends of the beam to facilitate external prestressing. The beam was supported in a simply supported condition, in which one end was hinge support and other one was a roller. Electrical Strain gauges (5 mm) which were pasted to the tensile reinforcement at the time of casting were connected to the MGC plus data logger so that it will store the data in the computer as MS Excel file. Similarly, three electrical strain gauges (60 mm) were connected to the data logger to observe the concrete strain in extreme compressive fiber, one at either side of two-point loads and one at mid span.

4.4 Testing of beams
The beam was subjected to static monotonic two-point load to induce crack to a limit around 85 % of the yield strain of the tensile reinforcement. The load was given at a load increment of 1.47 kN till the strain in the reinforcing steel reaches around 85 % of yield strain. The corresponding loads were observed as 32.4 kN and no. of cracks developed in the flexural region were marked. Then the load was released and the deviator and spacer arrangements were fixed with the beam. HTS wires on either side were kept in position and electrical strain gauges were pasted to the tendons to record the strain values. In the next phase of the experiment, sand bags were placed on the beam to give UDL of 3.16 kN/m, so that it will simulate the super imposed dead weight of a bridge girder. It was equal to 25 % of the calculated ultimate load of the beam. Sand bag loading is shown in Fig. 2. Strengthening by external prestressing was done while keeping the sand bag load on the beam. External prestressing was carried out using wire jacks by locking in one side and pulling in another side. Electrical strain gauges were used for recording strain in HTS wire. In case of single-draped tendon profile system, two strain gauges were pasted for each tendon to observe the variations in prestressing forces between two segments of the profile in the tendon. This instrumentation was done for facilitating to observe any loss in prestress due to friction at deviators. Where as in straight tendons, only one strain gauge was used as there is no deviation. Prestressing was done simultaneously on both sides. Sand bags were removed after the completion of strengthening. In phase-III of the experiment, the strengthened member was subjected to two-point load till its failure. The test arrangement is shown in Fig. 3. The load increment of 2.94 kN was given constantly. The member was failed in concrete crushing under one of the two-point loadings.

Figure 2. Sand bags loading on the beam

Figure 3. Test arrangement of the strengthened beam (deviated tendon profile)

5. RESULTS AND DISCUSSION

The complete strain history of the reinforcing steel for all the five specimens right from EPS-B1 and EPS-B5 are shown in Figs. 5 - 9. Various stages such as pre-strengthening, sand bag loading, after prestressing, yielding of steel at post-strengthening and ultimate were taken for discussion. In all the stages, reinforcing steel has behaved differently. It is clear that external prestressing of deviated tendon profile have induced negative strain in reinforcing steel consistently along with deflection recovery in specimens, which are shown in Fig. 5, Fig. 6 and Fig. 7. However, external prestressing of straight tendon profile (Fig.4) induced negative strain only in few locations at flexural zone for EPS B4 and almost nil in case of EPS B5 though the effective prestress was higher than that of EPS B4, which is shown in Fig. 8 and Fig. 9. This is because of the free movement of straight tendons at flexural zone and no deviators in flexural zone. This difference is also exhibited in ultimate flexural capacity between the two types of tendon profiles, which is shown in Fig. 10.

Figure 4. Test arrangement of the strengthened beam (straight tendon profile)

As it was reported earlier in literature [18], [19] & [20] that there is quick failure of the strengthened members after yielding of reinforcing steel, it was observed from this experiment too that the failure by means of concrete crushing had occurred after the yielding of reinforcing steel (Shown in Fig. 5 to Fig. 9). However there was little delay in failure of EPS B2 and EPS B3 since the area of compressive reinforcement for these beams were provided higher than that of EPS B1, which is shown in Fig.10. It is also to note that the increase in strain in reinforcing steel concentrated in the same location where maximum strain occurred in the pre-strengthening stage. The yielding of steel and the failure were occurred in the same location, which is evident from all the illustrations from Fig. 5 to Fig. 9.
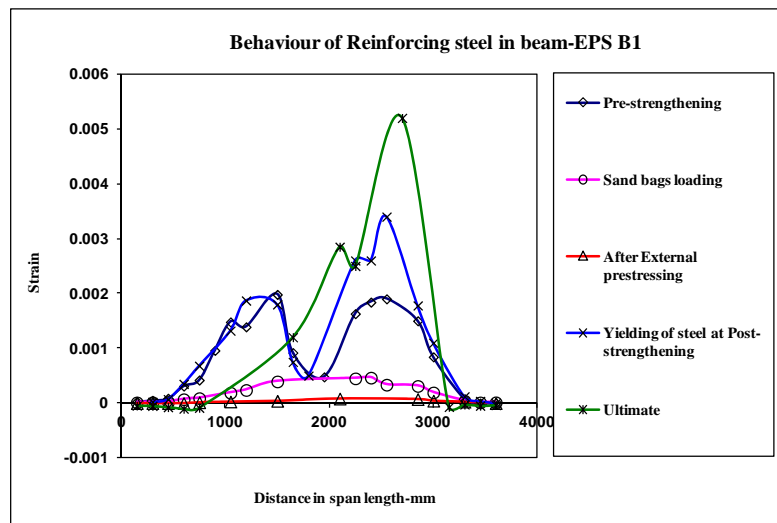
Figure 5. Strain history in reinforcing steel for the beam EPS B1
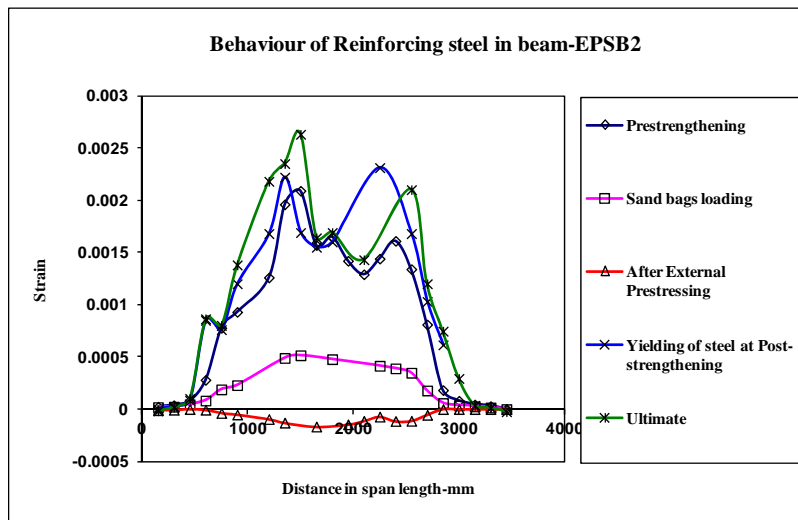
Figure 6. Strain history in reinforcing steel for the beam EPS B2
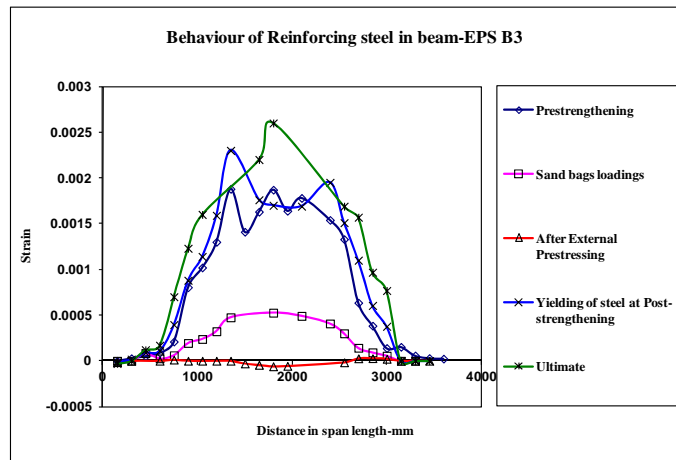
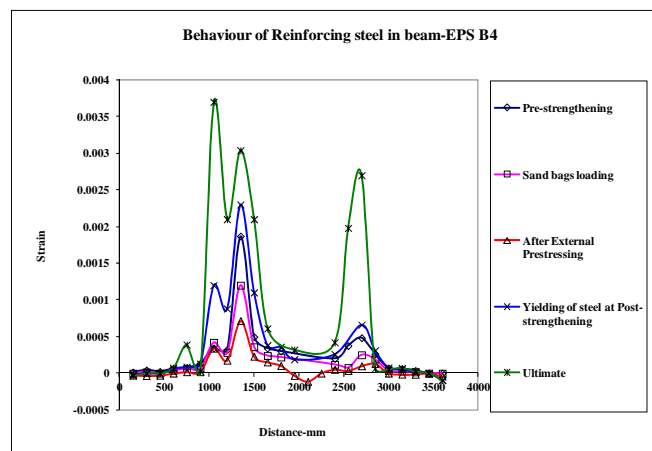Figure 7. Strain history in reinforcing steel for the beam EPS B3

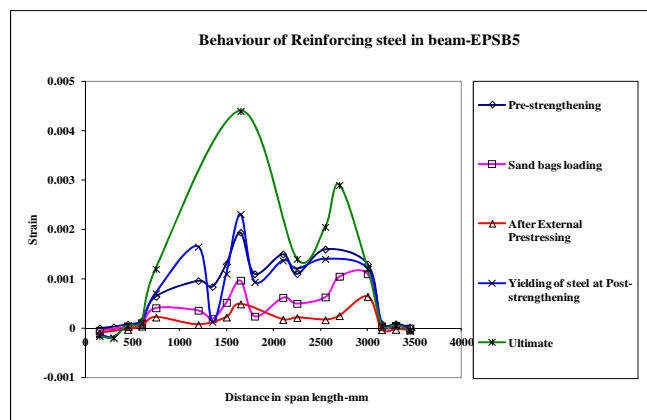Fig.8 Strain history in reinforcing steel for the beam EPS B4

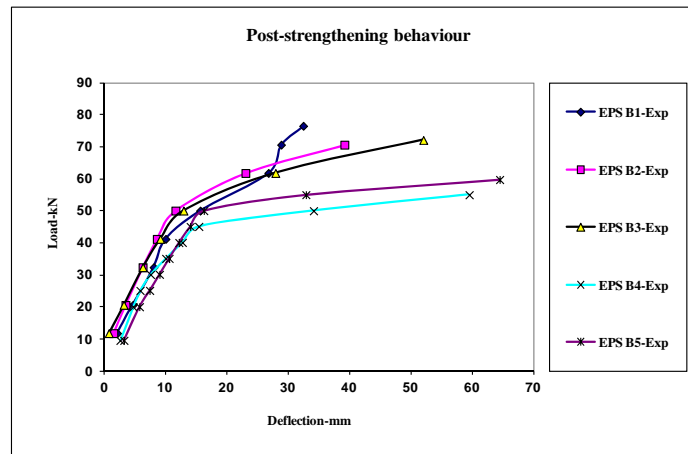Figure 9. Strain history in reinforcing steel for the beam EPS B5

Figure 10. Load-Deflection relation at post-strengthening stage

5.1 Deflection recovery due to external prestressing

Deflection recovery due to strengthening by external prestressing was monitored at every stage and recorded in data logger. Two types of comparison have been made regarding deflection due to external prestressing such as deflection recovery and reduction in deflection at post-strengthening stage. Strengthening by external prestressing was carried out while keeping sand bag load on the beam. The deflection recovery was measured from starting of prestressing operation till locking the prestressing force at the locking end. Secondly, measured deflections were compared between stages before strengthening and after strengthening. The load taken for comparison is the maximum load applied for cracking the beams at pre-strengthening stage. The details are shown in Table. 2. It is observed that external prestressing using single-draped tendon profile system has recovered 100 % deflection, where as strengthening using straight tendons of external prestressing has recovered only 49 %. When comparing deflection between stages of pre-strengthening and post-strengthening, external prestressing has reduced to maximum of 58 % and 25 % deflection at post-strengthening stage for deviated tendon and straight tendon profiles respectively. Therefore, it is clearly observed that external prestressing of deviated tendon profile has performed better than that of straight profile in increasing flexural capacity, deflection recovery and reducing the deflection at post-strengthening stage.

Table 2: Comparison of deflection between stages before and after strengthening

Specimen	Deflection Recovery			Measured Deflection			
	Deflection at sand bag load (mm)	Deflection after Ext. prestressing (mm)	Percentage recovery	Load* (kN)	at Pre-strengthening (mm)	at Post-strengthening (mm)	Percentage reduction
EPS B1	2.73	0.4	85	32.4	13.04	8.14	38
EPS B2	2.31	-0.21	100	26.5	11.85	4.98	58
EPS B3	2.8	0.42	86	26.5	9.72	4.9	50
EPS B4	4	2.64	34	20.14	5.94	4.82	19
EPS B5	6.4	3.28	49	30.46	12.23	9.22	25

*maximum load applied for cracking the beams at pre-strengthening stage.

5.2 Stress in Tendons

Effective prestress applied to all the beams are given in Table 1. As far as deviated tendon profile was considered, the profiles of the tendon stress at ultimate state f_{ps} were consistent for all the three specimens, is shown in Fig. 11. The stress in tendons at ultimate state f_{ps} , were 1407 MPa, 1121.4 MPa and 1182.9 MPa respectively for EPS B1, EPS B2 and EPS B3 respectively. Also, they increased the flexural capacity by 48 % when compare to the calculated ultimate capacity of unstrengthened beam (calculated by ACI method [28]). Stress in deviated tendons was not constant throughout the length and they exhibited different stress values for two segments. The two segments were such that one from pulling end to the deviator and other from deviator to locking end. It was also observed that the segment near pulling end shown higher stress values in tendons than that of other end. Around 10% loss of prestress due to friction at deviators has occurred. In case of straight tendons, the effective prestress was increased to 516.6 MPa for EPS B5 because the EPS B4 was not performed well in deflection recovery with the effective prestress of 315 MPa. However, the increase in prestress has achieved only 49 % of the deflection recovery and achieved only 17% in increasing the ultimate flexural capacity. The tendon stress at ultimate f_{ps} , were 861 MPa and 1066.8 MPa for EPS B4 and EPS B5 respectively. The profile of stress in tendon is shown in Fig. 12.
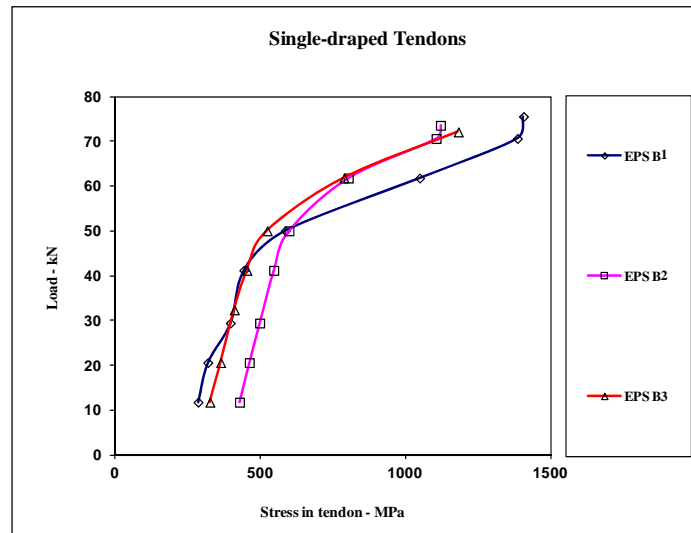

Figure 11. Load-Stress relation in tendons for single draped tendon profile
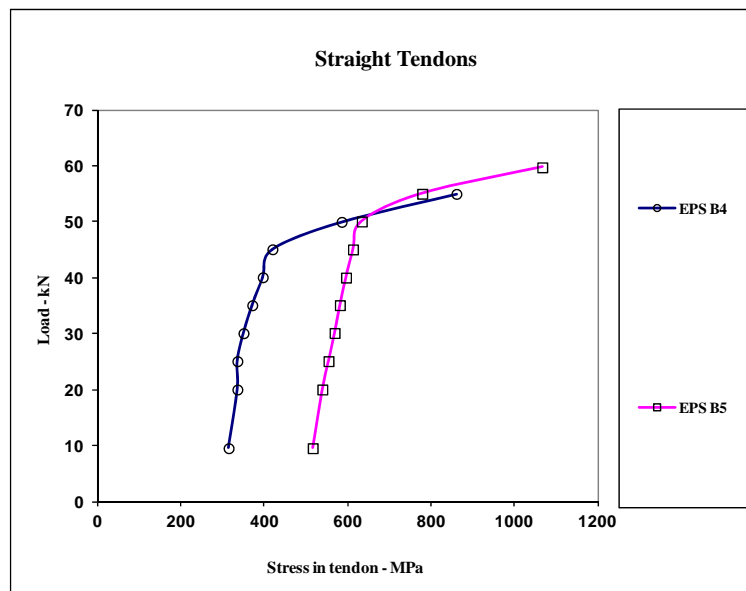
Figure 12. Load-Stress relation in tendons for straight tendon profile

6. CONCLUSIONS

1) Flexural capacity of RC beams has been increased by 48 % and 17 % due to strengthening by external prestressing of single-draped tendon and straight tendon profiles respectively.

2) External prestressing of deviated tendon profile has performed better than that of straight profile in increasing flexural capacity, deflection recovery and reducing the deflection at post-strengthening stage.

3) Beam strengthened using deviated tendons achieved 100% deflection recovery. Beams strengthened using straight tendons achieved only 49%, since there was no deviator in the flexural zone.

4) It was observed from the experiment that around 10% loss of prestress has occurred due to friction at deviators.

5) Yielding of reinforcing steel has a significant role in post-strengthening behavoiour of RC members strengthened by external prestressing.

6) Plastic hinge has not much pronounced in the ultimate state when compare to that of new concrete members prestressed with external tendons.

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NOTATIONS

f_{pe} = Effective prestress in tendons

f_{ps} = Stress at ultimate in external tendons