Strengthening of Reinforced Concrete T- Section Beams Using External Post-Tensioning Technique

Dr. AbdulMuttalib Issa Said
Professor
University of Baghdad-College of Engineering
abdmusawi@yahoo.com

Dr. Ali Hussein Ali Al-Ahmed
Lecturer
University of Baghdad-College of Engineering
ali_hussein_alahmed@yahoo.co.uk

Dhafer Mohsin Al-Fendawy
M.Sc Student
University of Baghdad-College of Engineering
dhaferhafdh@yahoo.com

ABSTRACT

This research is carried out to investigate the externally post-tensioning technique for strengthening RC beams. In this research, four T-section RC beams having the same dimensions and material properties were casted and tested up to failure by applying two mid-third concentrated loads. Three of these beams are strengthened by using external tendons, while the remaining beam is kept without strengthening as a control beam. Two external strands of 12 mm diameter were fixed at each side of the web of the strengthened beams and located at depth of 200 mm from top fiber of the section (d_{ps}). So that the depth of strands to overall depth of the section ratio ($d_{ps}/h=0.8$). For each strengthened beams, the strands have been tensioned by using a hydraulic jack with constant stress of 600 MPa. The main parameter conducting in this research is the strengthening length ratio (L_s/L) which is equal to the length of strengthening region (L_s) divided to the length of beam (L), these ratios are 0.83, 0.67 and 0.50. The experimental results showed that this technique for strengthening is efficient for reducing cracks width and increasing first cracking, service cracking and ultimate load capacities. The percentage increasing in first crack loads were 100%, 133% and 167%, for service crack loads (0.3 mm) were 63%, 75% and 88% and for ultimate loads were 78%, 89% and 67% for strengthening length ratios 0.83, 0.67 and 0.50 respectiviely as compared with the control beam.

Key words: post-tensioning, externally, strengthening, T- section, strands.

تقوية العتبات الخرسانية المسلحة ذات مقطع على شكل حرف (T) باستخدام تقنية الشد اللاحق الخارجي

د. عبد المطلب عيسى سعيد
استاذ
جامعة بغداد-كلية الهنسة

د. علي حسين علي أل احمد
مدرس
جامعة بغداد-كلية الهنسة

ظافر محسن حسن الفنداوي
طالب ماجستير
جامعة بغداد-كلية الهنسة

الخلاصـــة

في هذا البحث تم التحري عمليا عن التقوية لعتبات خرسانية مسلحة من خلال استخدام تقنية الشد الاحق المثبت خارج المقطع الخرساني. حيث تم صب وفحص اربعة عتبات خرسانية ذات مقطع على شكل حرف T بالانكليزية لحد الفشل وذلك بتسليط قوتين مركزتين عند نقطتي الثلث الوسطي للعتب حيث صممت هذه العتبات بنفس الابعاد و المواصفات. ثلاثة عتبات تم تقويتها باستعمال جدائل فولاذية خارجية بينما تركت العتبة الرابعة بدون تقوية كعتب مرجعي للمقارنة. تم استعمال جدائل فولاذية بواقع جديلة واحدة قطر 12 ملم ثبتت في كل جانب من جدع العتب المقوى حيث تم تثبيت هذه الجدائل بعمق 200 ملم من اعلى مقطع العتب حيث ان نسبة عمق الجدائل الى العمق الكلي للمقطع تساوي الى 0.8. تم شد هذه الجدائل باستخدام مضخه هيدروليكية للسيطرة على اجهاد الشد والذي هو ثابت لجميع العتبات بمقدار 600 ميكاباسكال. ان هذا البحث يهدف الى تقوية جزء من طول العتب حيث تم اعتماد نسبة الطول المقواة وهي 0.83, 0.67 و 0.5 حيث تمثل هذه الارقام نسبة الجزء الذي تم تقويته بالنسبة للطول الكلي للعتب. اظهرت النتائج كفاءة هذه التقنية في تقليل عرض التشققات وزيادة في مقاومة العتب للاحمال المسلطة. حيث ان بنقصان نسبة الطول المقواة ازدادت نسب حمل التشقق الاول بمقدار 100% و 133% و 167% وحمل الشق الخدمي (0.3 ملم) بمقدار 63% و 75% و 88% والحمل الاقصى بمقدار 78% و 89% و 67% عن العتب المرجعي لنسب الطول المقواة 0.83, 0.67 و 0.5 على التوالي.

الكلمات الرئيسية: الاجهاد المسبق ;خارجي; التقوية; مقطع على شكل حرف T ; جدائل الحديد

1. INTRODUCTION

The repairing and strengthening of deteriorated and substandard structures has become one of the important challenges confronting civil engineering worldwide. There is a significant and growing need for the strengthening of RC structures. In other hand, the decision to rehabilitate or replace a structure depends upon its importance, the severity of the damage**,** the availability of resources, and the economic factors. While replacement of a structure is generally costly and time consuming, strengthening techniques that make efficient use of labor and economic resources provide attractive alternatives to new construction. Rehabilitation can extend the life of a structure, or if a building or a bridge is to be replaced, temporary repairs can return a critical structure to service until a replacement is operational**, Balbool, 2009**. External post-tensioning technique is considered one of the most powerful techniques that have been used for strengthening concrete structures because of ease of installation of the strands and reduced or no interruptions to the regular function of the structures. The advantage recorded previously, that the tendons were placed on the web of concrete beams; on the contrary the analysis and design of the strengthened members were more complicated**, Tan, and Ng, 1997**.

This study will be spot-light on the behavior of externally prestressed RC T-beams. External tendons are not bonded to the concrete, are free to move between the deviator, and have a nearly constant stress along their lengths. In the present study, the effect of the strengthening length ratio (L_s/L) which is equal to the length of strengthened region (L_s) divided to the length of beam (L) will be conducted. Stresses increment for rebars and strain in concrete at critical section also, the first crack, service and ultimate load capacities are also adopted in this study.

2. OBJECTIVES

The main objective of this study is:

1. Predicting the first cracking, service and ultimate load capacities of RC T-beams strengthened by using external prestressed tendons that subjected to short-term loading. In addition, determine the percentage increasing in these load capacities as compared with beam without strengthening.
2. Ability of installation of externally strands at a distance from supports with governed the purpose of strengthening.
3. Estimating the optimum position to fix the external strands without effected on the behavior of strengthened member.

3. LITERATURE REVIEW

Cooke, et al., 1981. conducted an experimental investigation to study the effect of (L/d_{ps}) ratio and the amount of prestressing steel on the stress at ultimate stage in unbonded tendons. They tested nine simply supported fully prestressed one-way slabs with unbonded tendons. The slabs were divided into three groups with varying (L/d_{ps}) ratio. Each group had a varying amount of prestressing steel.

Yaginuma, and Kitada, 1988. tested three series of unbounded partially prestressed concrete beams. All beams were strengthened using straight tendons covering whole length of the beams. Two (L/d_{ps}) ratios of 18 and 32 were taken into account. They concluded that the stresses in prestressed strands increased as decreasing (L/d_{ps}) ratio.

Harajli, and Kanj, 1992. tested 16 RC beams. These beams were externally strengthened using steel tendons along the beams and subjected to cyclic fatigue loading. They concluded that using a straight horizontal profile is less effective in increasing the flexural capacity than a deviated profile due to reduction in depth of the straight strands throughout loading.

Tan, and Ng, 1997. tested six T-section RC beams, each of 3.3 m in length. The beams were externally strengthened with external strands along the span. They used straight and draped tendons for comparison. They concluded that, the beams with draped tendons showed wider spread of cracks, greater tendon stress increase and greater ductility compared with beams of straight tendons.

Tan, et al., 2001. conducted two series of simply supported T section-RC beams, one strengthened with external steel tendons and the other with carbon FRP tendons. They concluded that the exterior tendons anchored at inter-span locations could efficiently enhanced beams capacity.

Ng, 2003. tested seven T-section RC beams strengthened with external tendons with constant strand depth of 200 mm. It concluded that the span to depth ratio has insignificant effect on the tendon stress at ultimate stage in externally prestressed beams. Also he proposed a modified bond reduction coefficient for evaluation the flexural strength of externally prestressed beams based on strain compatibility and force equilibrium.

Sivaleepunth, et al., 2005. conducted an experimental investigation on RC beams with external tendons by varying the geometry of loading whether it is one-point loading or two-point loading. A comparison has been done of the experimental results with the results of prediction equations recommended by ACI 318M-99 and AASHTO LRFD design codes. It is found that all beams have similar behavior and first cracks observed in the midle of beams.. Also, it is found that the flexural cracks occurred at approximately 50% of the ultimate load.

4. MATERAILS USED FOR CASTING BEAMS

4.1 Concrete

The ingredients of concrete are: cement, fine aggregate, coarse aggregate and mixing water. An ordinary strength concrete mix was prepared using Portland cement (Type IV). For all beams, the cylindrical compressive strength of concrete (f_c') was 30 MPa at 28 days.

4.1.1 Cement

Sulphate resistance Portland cement from Karbala city company plant popular name (Lafarge Bridge) is used throughout this investigation. All quantity of cement was tested chemically and physically. The properties conform to the **Iraqi Specifications No. 5, 1984.** for Portland cement and British standard, this test was conduct with the help of laboratory of southern cement company.

4.1.2 Fine aggregate

Natural sand from Al-Akhaidher quarries in Karbala city has been used for concrete mixes. The fine aggregate has (4.75mm) maximum size with rounded-shape particles and smooth texture with fineness modulus of (2.84). The sand has been washed and cleaned with water several times and conform to the **Iraqi specification No.45, 1984**.

4.1.3 Coarse aggregate

Crushed gravel from Al-Sudor region whith maximum size of 15 mm has been used throughout this research. The crushed river coarse aggregate was washed. The specific gravity and absorption were (2.66) and (0.66%) respectively and conform to the **Iraqi specification No.45, 1984**.

Slump test is the direct method which it gives measure to workability for concrete mix. So that, in the present study the slump is taken 75 to 100 mm that make the mix medium workability. The curing of concrete is achieved normally by spray water for 28 days with covered the beams with roughness clothes.

4.2 Steel Rebars

Three sizes of deformed bars have been utilities in the present study which their diameters are: ϕ8 mm, ϕ10 mm and ϕ10 mm with yield stresses (f_y) of 530, 559, and 615 MPa respectively conform to **ASTM standard A615**.

4.3 Stress-Relived Strands

conform to **ASTM standard A416**. These strands of ϕ12 mm in diameter were made from seven wires by twisting six of them around one wire which is slightly larger than them. The ultimate strength and the modulus of elasticity were 1860 MPa and 190000 MPa respectively.

5. EXPERIMENTAL WORK

5.1 General

The experimental work is based on casting and testing up to failure four T section-RC beams having the same dimensions and internal reinforcement. Three of these beams have been strengthened by using external tendons (strands), while the remaining beam is kept without strengthening as a control beam. The primary effect is the length of the strengthening region (L_s). Where part of the span is strengthened to enhance the flexural behavior of simply supported T section-RC beams.

5.2 Specimens

All specimens were reinforced concrete with T-section having dimensions as: *hf =75 mm, bf=350 mm,* b_w*= 150 mm* and *h=250 mm*. The total length of the specimens is 3200 mm and the effected length (***L***) is 3000 mm. These beams were internally reinforced by 2ϕ12 mm as bottom reinforcement (tensile reinforcement), 4ϕ10 as top reinforcement (compressive reinforcement) and ϕ8 mm@100 mm as stirrups (shear reinforcement). All four beams were subjected to two mid-third concentrated loads up to failure. Fig. 1 shows the full details and setup of the tested beams.

Two external strands of diameter 12 mm (cross-sectional area of 98 mm^2) have been fixed at each side of the web of the strengthening beams and located at eccentricity (***e***) of 100 mm from center of gravity of the section, ***C.G.C,*** (or depth of strands=200 mm from top fiber of the section, d_{ps}) with one deviator at mid-span so that the depth of strands to overall depth of the section ratio ($d_{ps}/h=0.8$). A special anchorage system has been achieved to fix these external tendons with the web of the strengthened beams.

The main parameter conducting in this research is the strengthening length ratio (L_s/L) which is equal to the length of strengthening region (L_s) divided to the length of beam (***L***). In this research the strengthening ratios were 0.83, 0.67 and 0.50.

The tested beams is remarked as AT-0, AT-1, AT-2 and AT-3. AT-0 is the control beam and the three beams (AT-1, AT-2, and AT-3) are strengthened beams. The remark at the end of symbol of the beams refers to the strengthening region (L_s), where number (1,2and 3) refers to (2500,2000, and 1500) mm (L_s) respectively as shown in Fig. 2. Also, Table 1 illustrates the properties of the strengthened beams.

5.3 Anchorage System

Two plates with dimensions of 200 by 200mm and thickness of 10 mm have been bent to form an angle-shape section with unequal legs of 120 mm and 80 mm. These angle-shapes were then welded back to back together and with additional stiffeners to form a shape that able to resist any expected deformation when subjected to load as shown in Fig. 3. The mechanical transfer system has been attached by drilling four holes in the web of beams at distances according to strengthened length (L_s) required for each strengthened beam and then fixed with bolts.

5.4 Deviator

To reduce second order effect, one deviator was used at mid span for all strengthened beams. This deviator was fabricated from a plate of 6 mm thickness surrounding beam web in the bottom and welded to perpendicular stiffener plates on both sides as shown in Fig. 4.

5.5 Jacking Process

The maximum initial post-tensioning prestressed stress was 600 MPa. To control the stress of strands, a balance device has been used as shown in Fig. 5 to insure distributed load neutrally and avoided laterally deformation.

5.6 Mesuring Devices

In the present study, device equipments are provided such as hydraulic jack for post-tensioning requirements, universal testing machine for applying load, strain tools measurement, crack width tool, and dial gauge for recording central deflection.

6. EXPERIMENTAL RESULTS

All beams were tested up to failure by applying two mid-third point load with load division of 7 kN. This test is carried out using a universal testing machine with full capacity of 200 ton.

6.1 Crack Pattern

The first cracks for beam **AT-0** (control beam) propagated at load 21 kN, spread and distributed along the span. Seven to eight cracks at each side beyond the center line of the beam were appeared, their length were ranged between (67 to 171) mm. The cracks within the zone of pure bending moment were longer than the cracks outside this zone. However, as the load increased, the cracks moderately propagated upward to the flange through few steps after first cracking load. Increasing load caused cracks progressed upward rapidly and significantly until they reached their final length which ranged between (190 to 220) mm at ultimate load of 63 kN. It is worth to mention that at load 56 kN, the crack width was 0.3 mm. So that according to **ACI committee R224-07**, this load is considered as a service load. Fig. 6 shows the crack pattern at three stages of loading, first crack, service and ultimate stages of loading.

For strengthened beam **AT-1** ($L_s/L=0.83$) at load step 42 kN, only five short cracks were appeared at each side beyond the center line of the beam within the pure bending moment zone with length ranged between (47 to 99) mm. These cracks were very small in length. Accordingly, as load increased, the cracks were progressed and increased in length upward to compression zone with generated new cracks outside the pure bending moment zone. The service load was 91 kN which corresponding to crack width of 0.3 mm. These cracks were progressed continously till they reached their final length which ranged between (97 to 148) mm. The ultimate load at this stage was 112 kN. Fig. 7 shows the crack pattern generated in this beam.

For strengthened beam **AT-2** ($L_s/L=0.67$), Eight cracks were appeared at load step 49 kN. These cracks were spread and distributed within the pure bending moment zone, their length was ranged between (68-138) mm. Increasing load caused a slightly increasing in crack width and length. The service load was 98 kN with increasing the crack width to 0.3 mm at this load. The ultimate load was 119 kN and the cracks reached their final length as shown in Fig 8. From this figure, it could be noted that inclined cracks were created outside the strengthening region and beyond the anchorage system.

For strengthened beam **AT-3** ***($L_s/L=0.50$)***, only two cracks at each side of the beam were noticed at load 42 kN outside the strengthening region with very small width. Thereby, when the load increased up to 56 kN, the cracks length had increased outside the strengthened region with generating new cracks within the strengthening zone their length ranged between (108 to 115) mm. As the load increased, the cracks were increased and directed toward the compression flange till failure at load 105 kN. It is worth to mention that at this load the crack width was 0.3 mm, so that this load is also considered as service load. This cracks pattern is similar the result of further study, ***Tan, and Ng, 1997***. Fig. 9 shows the stages of loading for this beam.

Fig. 10 shows all tested beams after failure. Also, the results obtained from these four tested beams could be summerised as illustrated in Table 2. From this table, It can be noticed that the percentage increasing in first crack loads were 100%, 133% and 167%, for service loads were 63%, 75% and 88% and for ultimate loads were 78%, 89% and 67% for strengthening ratios 0.83, 0.67 and 0.50 respectively as compared with the control beam.

The load-cracks width curve for all tested beams is shown in Fig. 11. It could be noticed from this figure that, when reducing the strengthening ratio (***L_s/L***), the width of cracks at same level of load is reduced, due to increase the stiffness of the member within the strengthening zone.

6.2 Failure Mode

When a concrete member has a low steel ratio, that mean steel yielding before concrete crushing. In this research, all tested beams were designed with low steel ratio. The failure mode was predicted by measuring concrete strains at mid-span section of beams according to readings obtained from demec points throgout loadings. While, stresses in the ordinary reinforcement were calculated as strains at steel level (***d_s***) multiplied by steel modulus of elasticity but no greater than yield stress. Forgoing, the test focused on the member when failed only by yielding steel or crushing concrete. The control beam was failed by yielding of bottom steel but the stresses of concrete at top fiber was less than the ultimate strain. Generally, strengthened beams by external tendons have been enhanced slightly the strains in concrete and stresses in ordinary steel as compared with control beam at the same level of load. As decreasing ***(L_s/L)*** ratios from 0.83 to 0.5, the strains in concrete and the stresses in steel rebars were decreased at the same level of load as shown in Figs. (12 and 13) respectively. From Fig. (13) it could be noticed that the stresses in steel rebars for strengthened beams at transfer stage of loading having a negative signs. This is because the cambering effect due to strands tensioning.

6.3 Curvature

The results viewed with decreasing the strengthening length ratio from 0.83 to 0.5, the curvature is reduced as compared with the control beam due to increase the stiffness of beams due to strands effect. Fig. 14 shows the moment-curvature responses at mid-span section for tested beams. For strengthened beams, the curvature values at early stages of loading having approximately constant values this behavior might be due to constant moment within strengthened region. Also, it is clear form this figure that at transfer stage when strands have been tensioned, the curvature values have negative sign due to camber effect. The curvature is depends on concrete top and bottom extreme strains fiber recorded throughout the test. Eq. (1) is used for uncracked section. While, Eq. (2) is used for cracked section.

$$\emptyset_i = \frac{\varepsilon_{cbi} - \varepsilon_{cti}}{h} \tag{1}$$

$$\emptyset_i = \frac{\varepsilon_{cdi}}{d} \tag{2}$$

Where ϕ_i is the curvature at load step *i*.

ε_{cbi} is the bottom extreme strain fiber of concerete at load step *i*.

ε_{cti} is the top extreme strain fiber of concerete at load step *i*.

ε_{cdi} is the concrete strain at depth *d* above the neutral axis at load step *i*.

6.4 Load-Deflection Response

Central deflection has been recorded for each beam during the test by using dial gage located at mid-span of beam. The load-deflection responses for the strengthened beams were significantly stiffer than the control beam because of greater flexural rigidity (***EI***) for strengthened beam than control beam. Decreasing (***Ls/L***) ratio from 0.83 to 0.5 caused increasing in deflection within the inter range of loading, because the value of (***EI***) decreased as decreasing strengthening ratio. Fig. 15 shows the load-mid span deflection cuvers for tested beams.

The **ACI Code 318M-11**, limits the maximum permissible deflections to ***L/180*** (***Ds2***)***, or L/360*** (***Ds1***) for immediate deflection due to live load. The first limitation is for members which are not likely to be damaged by large deflection. While, the second limitation is for members which are likely to be damaged by large deflection.

6.5 Stresses of Strands

Strain in strands is determined by measuring the displacement between two demec points dividing by the distance between them. On the other hand, stresses in the strands within the elastic range are calculated by Eq. (3). Increments of strands stresses above initial stress (600MPa) are shown in Fig. 16. Increment stresses in strengthening beams are approximately similar this is because of constant (***dps /h***) ratio which equal to 0.8. Increment strand stresses measured experimentally by applying Eq. (3) and predicated analytically by applying Equation (18-4) of the **ACI Code 318M-11** are presented in Table (3).

$$\Delta f_{ps} = \Delta \varepsilon_{ps} \times E_{ps} \tag{3}$$

Where Δf_{ps} is the increment of strand stress

$\Delta \varepsilon_{ps}$ is the increment of strand strain

E_{ps} is the modulus of elasticity of the strand

7. CONCLUSIONS

1. Generally, using external strands technique for strengthening RC beams is effecient to increase beams capacities.
2. Decreasing strengthening length ratio affected on delaying the appearance of first cracks.
3. Decreasing strengthening length ratio lead to increasing first cracks load, service cracks load and ultimate loads. The percentage increasing in first crack loads were 100%, 133% and 167%, for service loads were 63%, 75% and 88% and for ultimate loads were 78%, 89% and 67% for strengthening ratios 0.83, 0.67 and 0.50 respectively as compared with the control beam.

4. Strengthened beams by external tendons would enhanced slightly the strains in concrete and stresses in ordinary steel as compared with control beam. As decreasing ***(L_s/L)*** ratios from 0.83 to 0.5, the strains in concrete and the stresses in steel rebars were decreased at the same level of load.
5. The results viewed with decreasing the strengthening length ratio from 0.83 to 0.5, the curvature is reduced as compared with the control beam due to increase the stiffness of beams due to strands effect.
6. Increments of strands stresses above the initial stress is approximately similar for all strengthened beams due to constant (***d_{ps} /h***) ratio.

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NOMENCLATURE

d_s depth of ordinary reinforcement from top fiber of the section
d_{ps} depth of strand from top fiber of the section
EI flexural rigidity
E_{ps} modulus of elasticity of the strand
f_c' cylindrical compressive strength of concrete
f_y yield stress of steel reinforcement
h overall depth of the section
L length of beam
L_s length of strengthened region
RC reinforced concrete
Δf_{ps} increment of strand stress
$\Delta \varepsilon_{ps}$ increment of strand strain
ε_{cbi} bottom extreme strain fiber of concerete at load step *i*.
ε_{cdi} concrete strain at depth *d* above the neutral axis at load step *i*.
ε_{cti} top extreme strain fiber of concerete at load step *i*.
ϕ_i curvature at load step *i*.

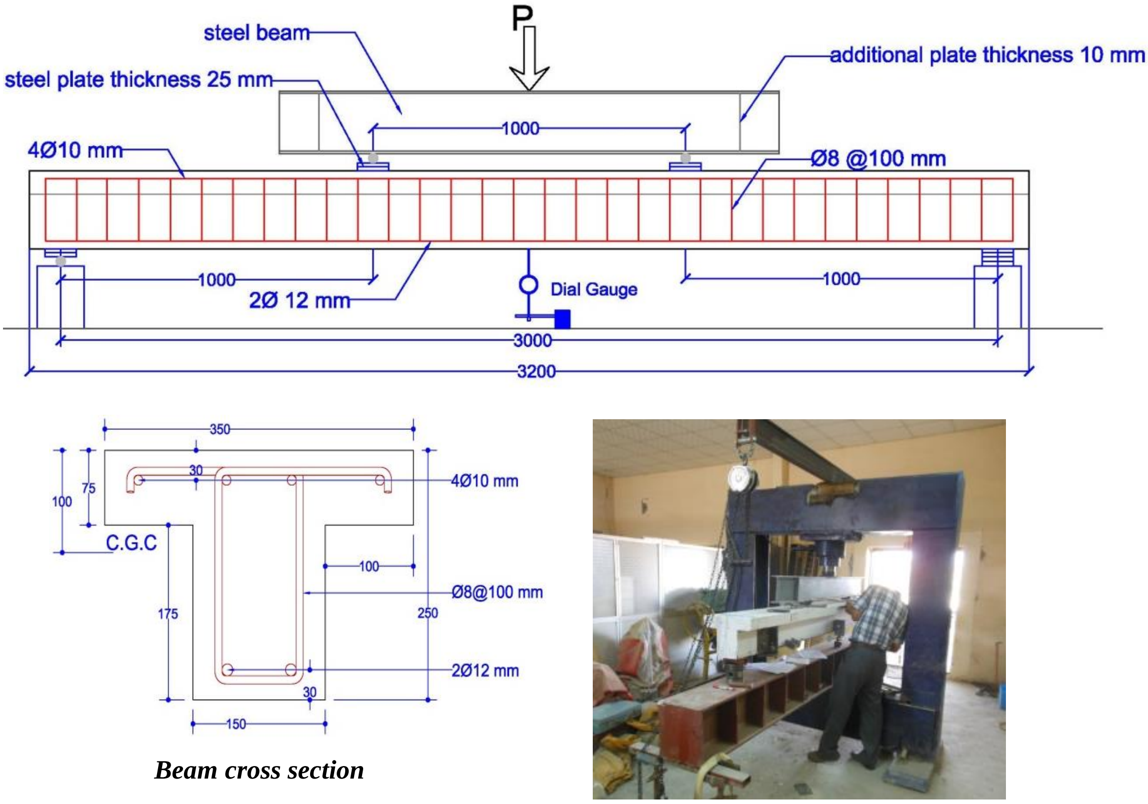

Figure 1. Layout and setup of typical tested beam (all dimensions are in mm).

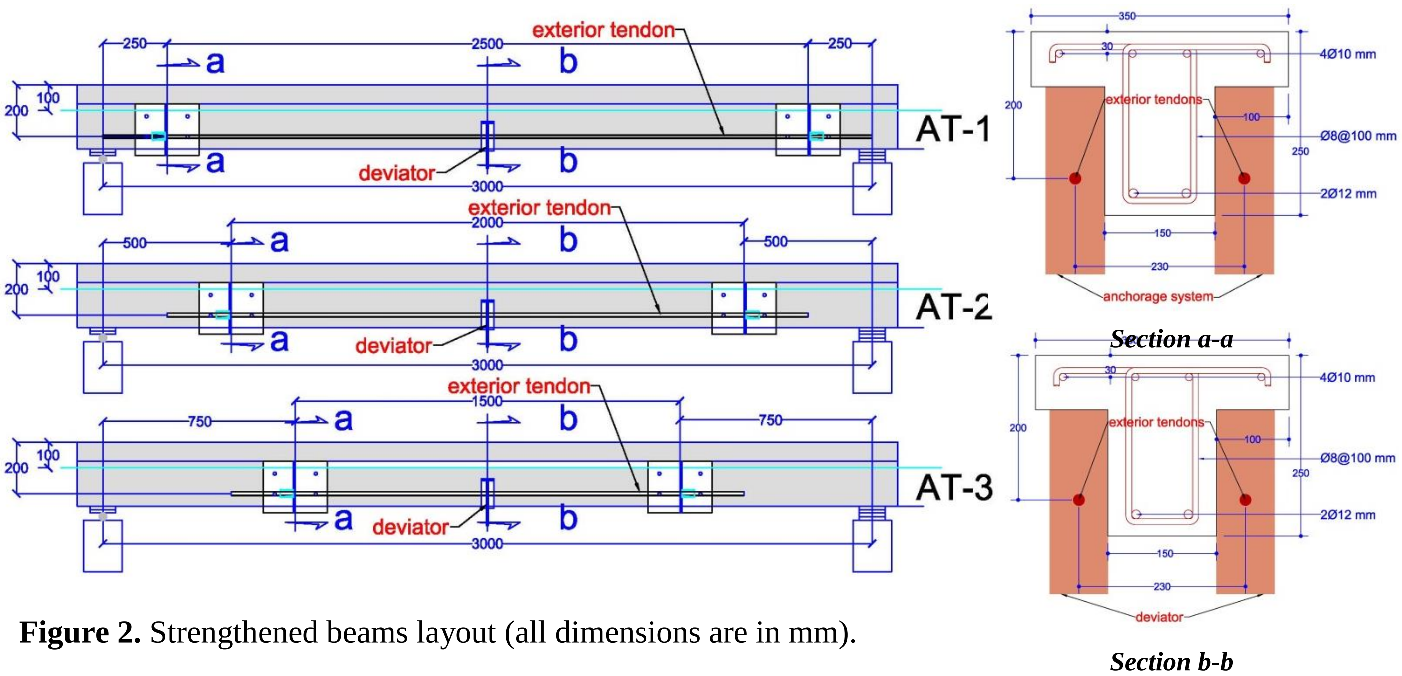

Figure 2. Strengthened beams layout (all dimensions are in mm).

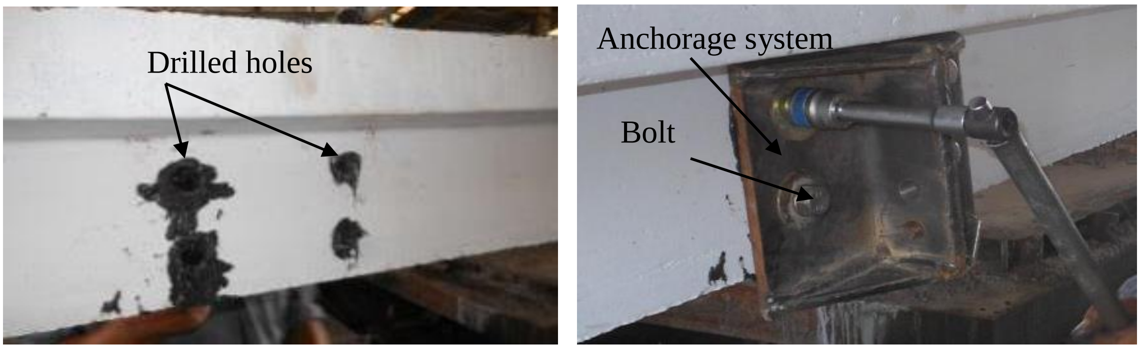

Figure 3. Operation used to fix the anchorage system.

Figure 4. Deviator.

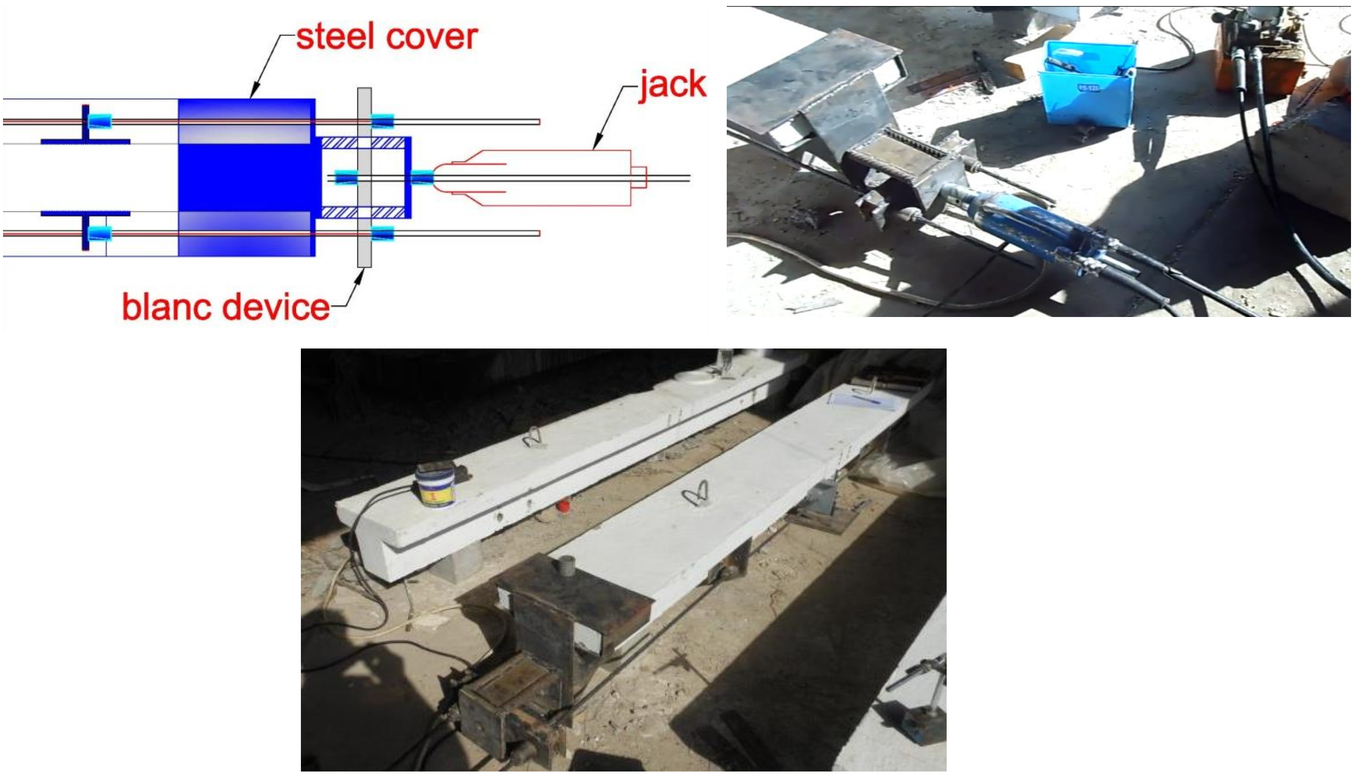

Figure 5. Jacking prosses to balance the force.

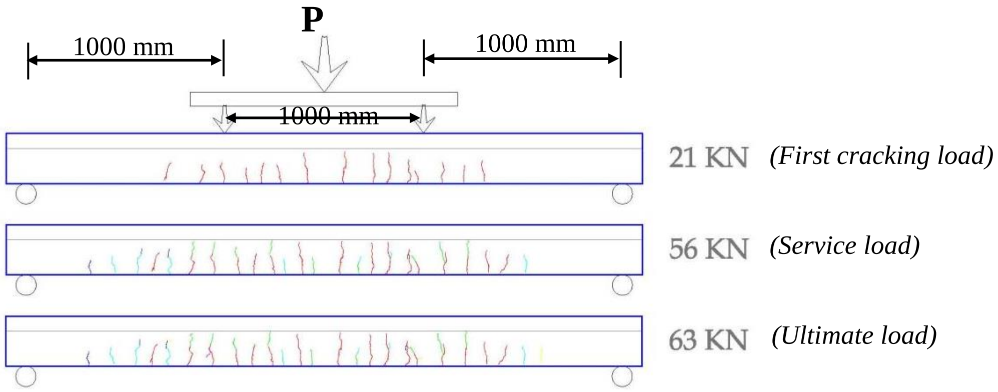

Figure 6. Cracks pattern for beam AT-0 (control beam).

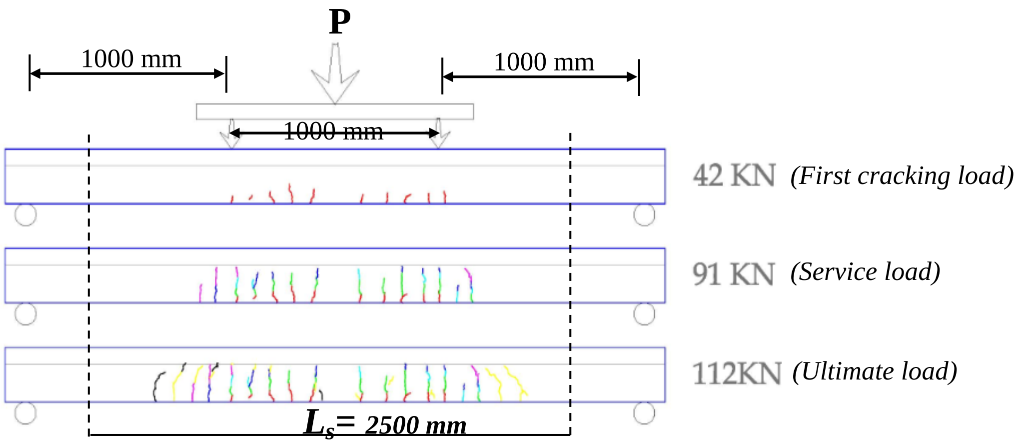

Figure 7. Cracks pattern for strengthened beam AT-1 (***L****s*/***L***=0.83).

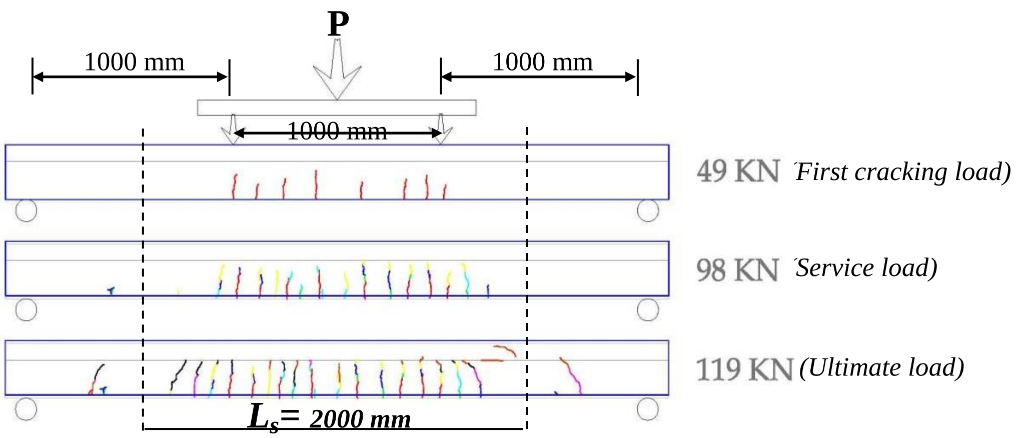

Figure 8. Cracks pattern for strengthened beam AT-2 (***L****s*/***L***=0.67).

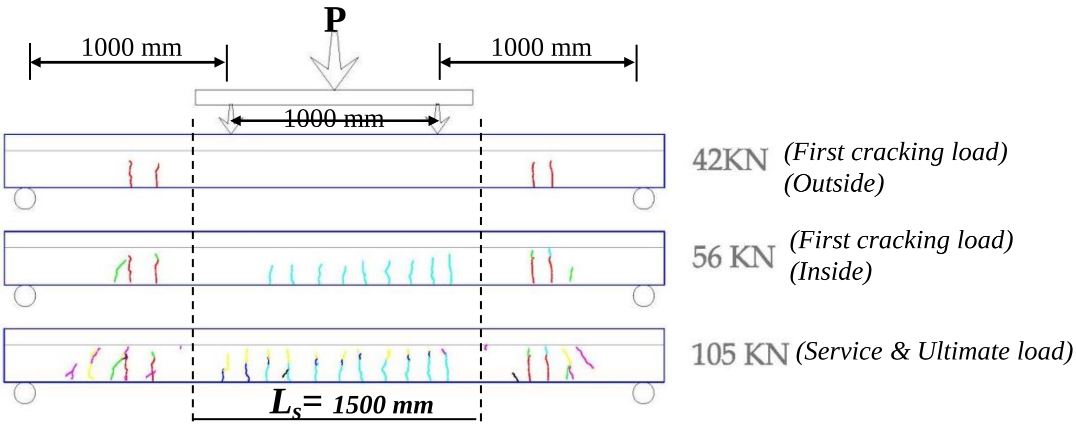

Figure 9. Cracks pattern for strengthened beam AT-3 (***L_s/L***=0.50).

Figure 10. Tested beams after failure.

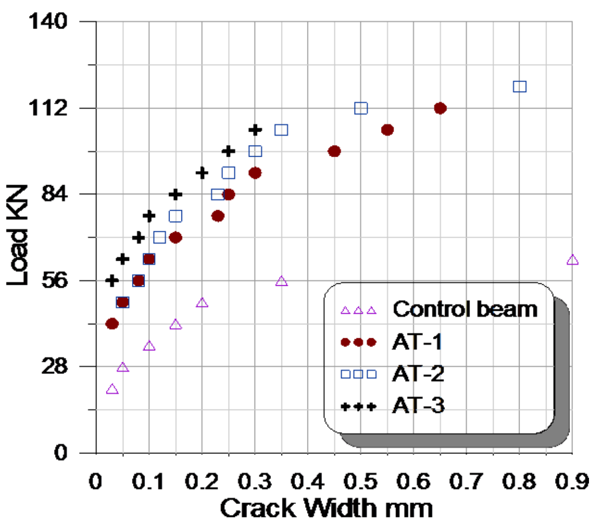

Figure 11. Load-cracks width curves for tested beams.

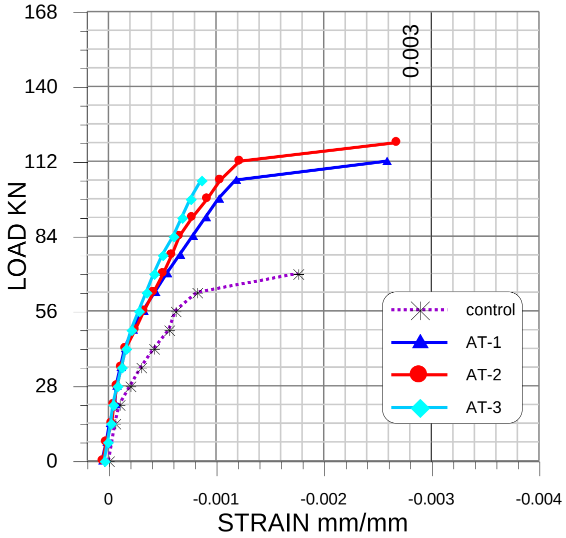

Figure 12. Strain at top fiber of concrete at mid-span section of tested beams.

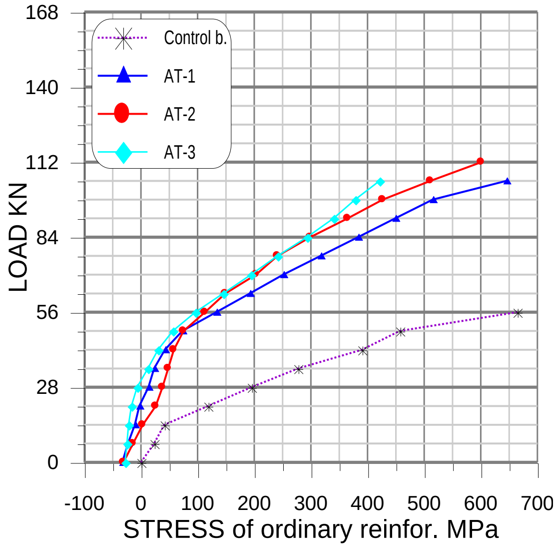

Figure 13. Stresss of ordinary steel rebars of tested beams.

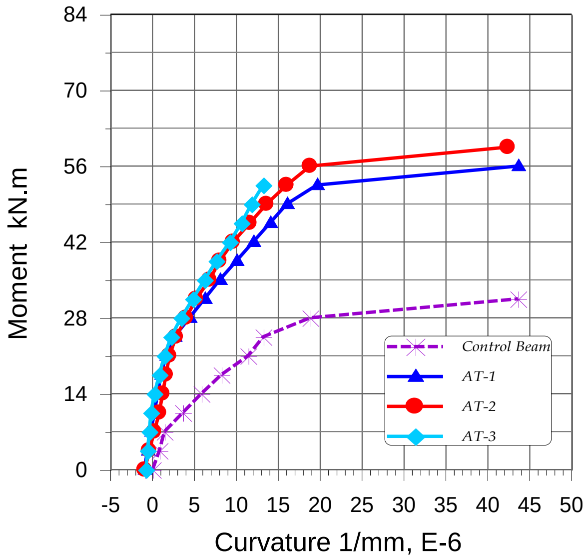

Figure 14. Moment- curvature responses for tested beams.

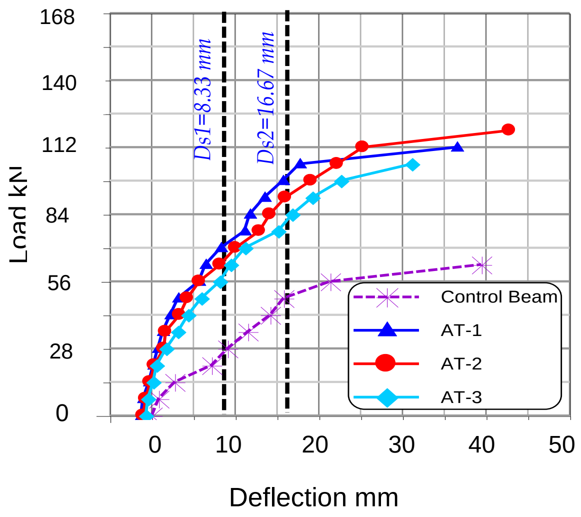

Figure 15. Load- mid span deflection responses for tested beams.

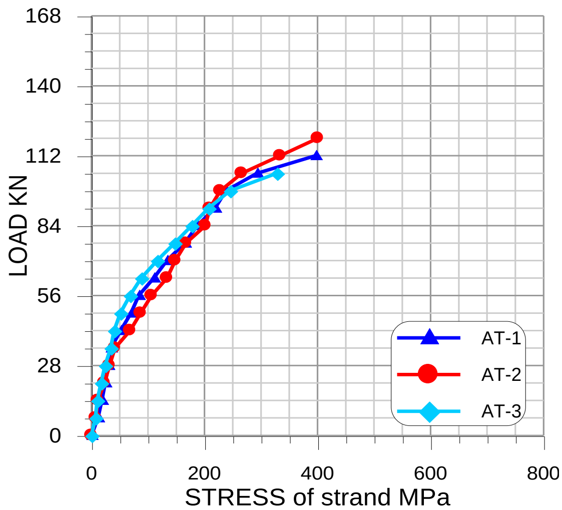

Figure 16. Increment stress in the strands above initial stress.

Table 1. Properties of the strengthened beams.

<table>
<tr><th>Beam designation</th><th>Strengthening length (L_s) mm</th><th>(L_s/L) Ratio</th><th>Configuration of strand</th><th>Depth of strand (d_{ps}) mm</th><th>(d_{ps}/h) Ratio</th></tr>
<tr><td>AT-0</td><td colspan="5">Control Beam</td></tr>
<tr><td>AT-1</td><td>2500</td><td>0.83</td><td>Straight</td><td>200</td><td>0.8</td></tr>
<tr><td>AT-2</td><td>2000</td><td>0.67</td><td>Straight</td><td>200</td><td>0.8</td></tr>
<tr><td>AT-3</td><td>1500</td><td>0.50</td><td>Straight</td><td>200</td><td>0.8</td></tr>
</table>

Table 2. Properties of the strengthened beams.

<table>
<tr><th>Beam designation</th><th colspan="2">First crack load (P_{cr}) kN</th><th>Service crack load (P_s)* kN</th><th>Ultimate load (P_u) kN</th><th colspan="2">% Increasing in first cracking load</th><th>% Increasing in service cracking load</th><th>% Increasing in ultimate load</th></tr>
<tr><td>Control beam</td><td colspan="2">21</td><td>56</td><td>63</td><td colspan="2">-</td><td>-</td><td>-</td></tr>
<tr><td>AT-1</td><td colspan="2">42</td><td>91</td><td>112</td><td colspan="2">100</td><td>63</td><td>78</td></tr>
<tr><td>AT-2</td><td colspan="2">49</td><td>98</td><td>119</td><td colspan="2">133</td><td>75</td><td>89</td></tr>
<tr><td>AT-3</td><td>42
Out</td><td>56
In</td><td>105</td><td>105</td><td>100
Out</td><td>167
In</td><td>88</td><td>67</td></tr>
</table>

% Increasing = $\frac{P(strengthened) - P(control)}{P(control)}$ *x 100*

**Crack width=0.3 mm*

Table 3. Stresses in strands

<table>
<tr><th rowspan="2">Beam designation</th><th rowspan="2">(L_s/L) Ratio</th><th rowspan="2">Δfps Y MPa</th><th rowspan="2">Δfps U MPa</th><th>fps Y MPa</th><th>fps U MPa</th><th rowspan="2">fps MPa ACI Code Eq. (18-4)</th></tr>
<tr><th colspan="2">fps=Δfps+fpe</th></tr>
<tr><td>AT-1</td><td>0.83</td><td>293</td><td>397</td><td>893</td><td>997</td><td rowspan="3">786</td></tr>
<tr><td>AT-2</td><td>0.67</td><td>334</td><td>401</td><td>934</td><td>1001</td></tr>
<tr><td>AT-3</td><td>0.50</td><td colspan="2">328</td><td colspan="2">928</td></tr>
</table>

Y,U the value of stress increment in yield and ultimate stages.
fpe =600 MPa