Experimental Study on the Behaviours of Post-tensioned Concrete Members with Unbonded Tendons

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Abstract. This paper presents experimental results regarding the behaviours of eight simply supported partially prestressed concrete beams with internally unbonded tendons, focusing particularly on the effect of three different variables: concrete compressive strength, $f_{ck}$; jacking stress, $f_j$; and the prestressing index, $\omega$. Increasing the concrete compressive strength from 35 MPa to 60 MPa was seen to lead to an increase in load-carrying capacity by about 10%. The load capacity was also affected, though to a lesser extent, by the jacking stress. The failure load of partially prestressed concrete beams with internally unbonded strands was increased by about 2.5%, to 10%, when the jacking stress increased from 0.5 to 0.7 of the strand’s ultimate strength, $f_{pu}$. The test results showed that the prestressing index was the most influential parameter among the examined variables. The experimental results of the stress increase in unbonded strands at failure and the ultimate load resistance of members were also compared to the analytical results of various standards and research methodologies for verification.

1. Introduction
Unbonded prestressed concrete members are commonly implemented in most important construction work. Due to the importance of this type of construction element, many experimental studies have been carried out to examine the design of these structures. As there is no compatibility between the unbonded prestressing steel and surrounding concrete, the total stress in the prestressing steel will depend on the overall change in length of the prestressing steel between the end anchorages, however. This causes any analysis of this type of structural member to be member-dependent rather than section-dependent. Many equations have thus been developed to determine the performance and the ultimate stress of unbonded tendons. In this paper, the performance of eight test beams is examined based on experimental results of tests of the ultimate stress in unbonded tendons and the ultimate load resistance of test members; these are then compared to the analytical results determined by various research methodologies and design codes.

2. Review of stress in unbonded post-tensioned tendons at ultimate stress
Many researchers have attempted to develop empirical or semi-empirical equations to simulate the stress in unbonded tendons at the failure stage of post-tensioned concrete members. Based on the concept of bond reduction, the coefficient $\mu$, which is defined as the ratio of the strain increase in the unbonded tendon to the strain increase in the equivalent bonded tendon, Naaman and Alkhairi [1] proposed the following expression to predict the ultimate stress ($f_{utb}$) in unbonded steel:
where $f_{ub}$ is the effective stress in unbonded prestressed reinforcement after all losses; $E_{ps}$ is the modulus of elasticity of the unbonded tendon; $\varepsilon_{cu}$ is the ultimate concrete compression strain, generally considered equal to 0.003; $d_p$ is the distance from the extreme compression fibre to the centroid of prestressing reinforcement; $c$ is the neutral axis depth; $L_1$ is the length of loaded span or sum of lengths of loaded spans affected by the same tendon; $L_2$ is the length of tendon between end anchorages, divided by the number of hinges in the assumed failure mechanism; $f_{py}$ is the specified yield strength of prestressed reinforcement; $k$ is the load type factor ($k = 1.5$ for one-point loading at midspan and $3$ for two-point loading or uniformly distributed loading); and $L$ is the span length from anchorage to anchorage. Thus, $L_1/L_2$ equals one in the case of simply supported beams [1].

Based on experimental results for 26 tested beams, along with the test data available from other researchers, Harajli and Kanj [2] developed Equation (2) for ultimate stress in unbonded post-tensioned steel $f_{ub}$:

$$f_{ub} = f_{pe} + \gamma_o f_{pu} \left(1 - 3 \frac{A_{ps} f_{pe} + A_s f_y}{b d_p f_c'} \right) \tag{2}$$

where

$$\gamma_o = L_1 \left(0.12 + \frac{2.5}{S/d_p} \right) \tag{3}$$

in which $f_{pu}$ is the ultimate strength of the prestressing reinforcement; $A_{ps}$ is the area of the prestressed reinforcement in tension zone; $A_s$ is the area of ordinary tension reinforcement in tension zone; $f_y$ is the yield stress of ordinary tension reinforcement; $f_c'$ is the specified compressive strength of concrete; $b$ is the width of the compression flange of the member; and $S$ is the span length from anchorage to the anchorage of simply supported member or length of the span under consideration for the continuous member.

Chakrabarti [3] used the test results and observations for 33 unbonded post-tensioned concrete members along with test data collected by other researchers to develop Equation (4) to evaluate the stress at ultimate in unbonded strand $f_{ps}$:

$$f_{ps} = \frac{f_{pe} + 10000 + A}{(1 - B)} \tag{4}$$

where

$$A = \frac{f_{c}'}{100 \rho_s} * \frac{d_p}{d} * \frac{60000}{f_y} * (1 + \rho_s/0.025) \leq 20000 \tag{5}$$

$$B = \frac{r f_{c}'}{100 \rho_s f_{pe}} \leq 0.25 \tag{6}$$

in which $\rho_s$ is the ordinary reinforcement ratio ($\rho_s = A_s/\rho d_s$); $d_s$ is the distance from the extreme compression fibre to the centroid of the tension ordinary reinforcement; $r = 1$ for $l/d_p \leq 33$, $r = 0.8$ for $l/d_p > 33$; and $l$ is the span length.

Lee et al. [4] instead proposed Equation (7), which takes into account the effect of each of the bonded reinforcements and the plastic hinge length:

$$f_{ub} = f_{pe} + \Omega_u E_{ps} \varepsilon_{cu} \left(\frac{d_p}{c} - 1\right) \frac{L_1}{L_2} \leq 0.94 f_{py} \tag{1}$$

$$\Omega_u = \frac{k}{(L/d_p)} \tag{1}$$
where \( A_s' \) is the area of ordinary compression reinforcement and \( f \) is a coefficient of the loading type, which equals 3.0 for two-point loading or uniform loading and 10.0 for a singular concentrated load. 

Au and Du [5] proposed Equation (8) to predict the value of \( f_{ub} \) assuming that the ultimate concrete compression strain equals 0.003 and the ratio of the equivalent length of the plastic region to neutral axis depth \( (c) \) at ultimate equals 9.3:

\[
f_{ub} = f_{pe} + 0.0279 \frac{E_p (d_p - c)}{l_e} \leq f_{py} \quad \text{(in MPa)}
\]

where \( l_e \) is the length of the tendon between the end anchorages divided by the number of plastic hinges \( (n) \) required to develop a failure mechanism in the span under consideration. In the case of simply supported beams, the number of plastic hinges \( (n) \) is equal to 1 and \( l_e \) is thus equal to \( l \).

In an attempt to enrich the data on stress in unbonded prestressing steel, Diep and Niwa [6] used the concept of plastic hinge length in a parametric study to predict the stress at ultimate in an unbonded tendon \( f_{ub} \) as follows:

\[
f_{ub} = f_{pe} + \frac{0.95A_{ps}f_{py} + A_s f_y - A_s' f'y' - 0.85f_c'(b - b_w)h_f}{0.85\beta_1f'_c h_w}
\]

where \( L_o = L_d - 0.535 d_{ps} \); \( L_d \) is the distance between two point loads; \( f_y' \) is the yield stress of ordinary compression reinforcement; \( b_w \) is the width of the web; and \( \beta_1 \) is the reduction factor of the concrete compression block.

The AASHTO-LRFD 2014 [7] adopted Equation (11) to estimate \( f_{ub} \):

\[
f_{ub} = f_{pe} + \frac{900 (d_p - c)}{L} (1 + N_s/2) \leq f_{py} \quad \text{(in ksi)}
\]

where \( N_s \) is the number of support hinges crossed by the tendon between anchorages or discretely bonded points.

The ACI 318M-19 code [8] adopted Equations (12) and (13) to estimate \( f_{ub} \), instead, however:

when \( l/h \leq 35 \)

\[
f_{ub} = f_{pe} + 70 + \frac{f'_c}{100 \rho_{ps}} \quad \text{(in MPa)}
\]

\( f_{ub} \leq \text{the least of (} f_{pe} + 420 \text{) and (} f_{py} \text{)} \)

when \( l/h > 35 \)

\[
f_{ub} = f_{pe} + 70 + \frac{f'_c}{300 \rho_{ps}} \quad \text{(in MPa)}
\]

\( f_{ub} \leq \text{the least of (} f_{pe} + 210 \text{) and (} f_{py} \text{)} \)

where \( l \) is the length of the clear span measured face to face of support in mm; \( h \) is the overall thickness, height or depth of member in mm; and \( \rho_{ps} \) is the prestressing reinforcement ratio \( (\rho_{ps} = A_{ps} / bd_p) \).

The Canadian Standard (CAN3 – A23.3 M84) [9] in turn recommended Equation (14) to predict \( f_{ub} \)

\[
f_{ub} = f_{pe} + 8000 \left( \frac{d_p - c_y}{l_e} \right) \leq f_{py} \quad \text{(in MPa)}
\]

where
\[ c_y = \frac{\varphi_p A_{ps} f_{py} + \varphi_s A_s f_y - \varphi_c A_c f_{cy} - 0.85 \varphi_c f_{cy} h_f (b - b_w)}{0.85 \varphi_c \beta f_{cy} b_w} \]  

in which \( \varphi_p, \varphi_s, \) and \( \varphi_c \) are resistance factors for the prestressed tendon, non-prestressed reinforcement, and concrete, which can be considered equal to 0.9, 0.85, and 0.6, respectively.

Dutch Practice [10] recommended Equation (16) to evaluate \( f_{ub} \),

\[ f_{ub} = 1.05 f_{pe} \]  

while the German code (DIN 1045: 2001) [11] recommended Equation (17) for the same purpose.

\[ f_{ub} = f_{pe} + E_p \left( \frac{d_{ps}}{17L_1} \right) \leq f_{py} \]  

According to AASHTO 2014 [7] and ACI 318-19 [8], the nominal bending moment can be computed from the moment equilibrium, using equation (18) for rectangular sections:

\[ M = A_{ps} f_{pz} \left( d_{ps} - \frac{a}{2} \right) + A_s f_y (d - \frac{a}{2}) + A_s f_{cy} \left( \frac{a}{2} - d' \right) \]

where \( a \) is the depth of the equivalent rectangular stress block.

3. Test programme
This paper presents test results for eight partially prestressed concrete beams with a 2.7 m clear span and 250 x 350 mm cross-sectional dimensions under two-point loading. These beams were tested in the structural laboratory at the University of Baghdad. Seven-wire low-relaxation strands of 99.6 mm\(^2\) cross-sectional area were used as unbonded prestressing steel with an ultimate strength of 1,860 MPa. The yield strength, modulus of elasticity, and the maximum elongation of the prestressing strands were 1725 MPa, 197500 MPa, and 5%, respectively.

Three diameters of deformed ordinary steel bars were used: 10, 16, and 25 mm. The compression zone was reinforced with two bars of 10 mm diameter, while for tensile reinforcement, test specimens were reinforced with two bars of 25 mm diameter and two bars of 16 mm diameter for beams of \( \omega = 0.4 \) and \( \omega = 0.6 \), respectively. For shear reinforcement, an appropriate amount of steel was used to ensure that no shear failure occurred prior to flexural failure. Accordingly, bars of 10 mm diameter were utilised as closed stirrups at 100 mm c/c spacing in the shear span, and two further steel stirrups were used in the flexural span. Table 1 illustrates the characteristics of the steel reinforcement used in this study. The beam specifications and concrete mechanical properties at testing age and the reinforcement areas are given in Table 2, while the cross-sectional dimensions of the beams and their profiles, including stirrup distribution, are shown in Figures 1 and 2, respectively.

Figure 1. Cross-sectional dimensions for (a) beams with \( \omega = 0.4 \) (b) beams with \( \omega = 0.6 \)
Two values of compressive strength were used in the study, with concrete mixtures designed to achieve target cylinder compressive strengths of 35 and 60 MPa at 28 days for 150 × 300 mm specimens. The cement: sand: aggregate: super-plasticizer proportions by weight for the 35 MPa mix were 1: 2.0: 2.65: 0.01, with a water/cement ratio of 0.375 and a final cement content of 400 kg/m$^3$; while the proportions for 60 MPa mix were 1: 1.4:1.9: 0.01, with a water/cement ratio of 0.270 and a final cement content of 520 kg/m$^3$. The characteristics of the materials used in concrete mixtures are shown in Tables 3 to 8.
Table 3. Typical properties of superplasticizer

| Form                  | Specific gravity | pH     | Chloride content |
|-----------------------|------------------|--------|------------------|
| Yellowish liquid       | 1.07±0.02        | 6±1.0  | nil              |

Table 4. Physical properties of cement

| Physical properties                      | Test results | Iraqi Specification Limits |
|------------------------------------------|--------------|----------------------------|
| Specific surface area (Blaine method) (m²/kg) | 390          | ≥ 230                      |
| Initial setting time (hours : minutes)    | 1:45         | ≥ 45 min                   |
| Final setting time (hours : minutes)      | 3:30         | ≤ 600 min                  |
| Compressive strength for 3-day (MPa)      | 19.23        | ≥ 15 MPa                   |
| Compressive strength for 7-day (MPa)      | 33.03        | ≥ 23 MPa                   |

Table 5. Chemical analysis of cement

| Chemical compound | Abbreviation | Content (%) | Iraqi Specification Limits |
|-------------------|--------------|-------------|----------------------------|
| Lime              | CaO          | 61.52       | -                          |
| Silica            | SiO₂         | 20.8        | -                          |
| Alumina           | Al₂O₃        | 4.76        | -                          |
| Iron oxide        | Fe₂O₃        | 3.9         | -                          |
| Magnesia          | MgO          | 2.91        | ≤ 5 %                      |
| Sulphate          | SO₃          | 2.3         | ≤ 2.8 %                    |
| Loss on Ignition  | L.O.I        | 2.24        | ≤ 4 %                      |
| Insoluble material| I.R.         | 1.02        | ≤ 1.5 %                    |
| Lim Saturation Factor | L.S.F   | 0.88        | 0.66-1.02                  |
| Free Lime         | F.L.         | 1.4         | -                          |
| Main compounds    |              |             |                            |
| Tricalcium Silicate| C₃S         | 44.56       | -                          |
| Dicalcium Silicate| C₂S         | 25.43       | -                          |
| Tricalcium Aluminate| C₃A       | 5.59        | ≥ 5 %                      |

Table 6. Sieve analysis of sand

| Sieve size (mm) | Cumulative passing (%) | Limit of Iraqi Specification #45/1984 Zone No. III [13] |
|-----------------|-------------------------|--------------------------------------------------------|
| 10              | 100                     | 100                                                    |
| 4.75            | 94.0                    | 90 – 100                                               |
| 2.36            | 87.2                    | 85 – 100                                               |
| 1.18            | 80.0                    | 75 – 100                                               |
| 0.60            | 70.8                    | 60 – 79                                                |
| 0.30            | 24.8                    | 12 – 40                                                |
| 0.15            | 6.4                     | 0 – 10                                                 |

Table 7. Chemical and physical properties of sand.

| Properties                        | Test results | Iraqi Specification #45/1984 [13] |
|-----------------------------------|--------------|-----------------------------------|
| Fineness modulus                  | 2.368        | 2.3-3.1                            |
| Absorption (%)                    | 2.20         | ----                              |
| Dry loose unit weight (kg/m³)     | 1581         | ----                              |
| Sulphate content SO₃ (%)          | 0.33         | 0.5 (max. value)                  |
| Material finer than 0.075 mm sieve (%) | 2.4         | 5 (max. value)                    |

Table 8. Sieve analysis and sulphate content of coarse aggregate.
| Sieve size (mm) | Cumulative passing (%) | Iraqi Specification # 45/1984 [13] |
|----------------|-------------------------|----------------------------------|
| 37.5           | 100                     | 100                              |
| 19             | 96.6                    | 95-100                           |
| 14             | 54.5                    | -                                |
| 10             | 22.3                    | 30-60                            |
| 5              | 0.2                     | 0-10                             |
| 2.36           | 0.1                     | -                                |
| 0.075          | 0.1                     | 3%                               |

Sulphate content SO₃ (%) 0.09 0.1%

All specimens were examined using a uniaxial electrical resistance strain gauge to measure the strain in both steel and concrete. The pre-wired strain gauges were of 118±0.5 Ω resistance for steel and 119±0.5 Ω resistance for concrete.

Smooth PVC ducts with a 20 mm internal diameter were utilised for the unbonded tendons. The strands were cut to specified lengths (the length of the girder plus one extra metre), and the strain gauges were fixed on the strands: three strain gauges were installed on the first strand, with one strain gauge attached on the second strand at the midspan section. The strands were placed in the ducts, which were embedded in the moulds prior to concrete casting. The strain gauge wires were connected to the strain indicator to record the strain of the strands during the prestressing process. Two bearing steel plates were used at the ends of each specimen. The plates had 10 mm thickness with two holes of 20 mm diameter. Split-wedge and barrel-type anchor grips were used for anchorage purpose on the unbonded tendons, with the split-wedge being in two parts. The grips were placed at the ends of the strands, which were marked at the end of the grips in order to measure any elongation of the strands after prestressing to determine the level of prestrain induced in each strand. The first strand was stressed to the required prestressing value, then the jack was released, and the process repeated the second strand. The corresponding prestrain was monitored accurately by means of the strain gauges attached to the surface of each prestressing strand, with readings from the pressure gauge of the hydraulic jack used in the post-tensioning operation used as backup. The post-tensioning procedure, shown in Figure 3, was conducted about 30 minutes before testing as the specimens were installed on the testing machine.

Figure 3. Posttensioning process for tested beams

Two dial gauges were used, one at the end of each strand, to check whether any strand slip occurred during the loading process. The deflection of the specimens was evaluated by means of four LVDTs.
One was installed under the mid-span section, while two of were fixed at the two point-load sections; the last was put under the frame below the first to take into consideration any potential frame movement. The resulting data were automatically recorded using a computerised data acquisition system.

After the specimen was placed on the testing machine, as shown in Figure 4, all readings of the load cell, strain gauges, dial gauges, and LVDTs were zeroed. The load was then increased incrementally by means of a hydraulic jack of 100-ton capacity. A load cell of 100-ton capacity was used to monitor the load, which acted on the transverse steel beam. The load cell was connected to a computerised data acquisition system, which thus recorded the monotonically applied load automatically. Cracks were marked on the specimens for each load step after the appearance of the first crack. The maximum width of the crack in the flexural region was measured, along with the spacing between cracks. The crack pattern was also photographed after the test on three sides of the cross-section. Ultimate loads and modes of failure were recorded in each case. Load-deflection relationships were also monitored, along with the strains of each of the concrete samples, both with conventional reinforcement, and unbonded tendons, throughout the tests.

![Figure 4. Test setup of experimental beams](image)

### 4. Results and observations

The performance of the test beams was monitored in terms of ultimate load-bearing capacity, crack pattern, load-deflection characteristics, and strain increase in the unbonded tendon. All beams failed in flexure.

#### 4.1 Ultimate flexural strength and failure

Yielding of the bonded steel occurred in all test beams before the crushing of concrete. The major factor influencing the ultimate load-bearing capacity of the test beams was thus the amount of ordinary tensile reinforcement, which provides an additional tensile force that leads to an increase in the ultimate strength and the stiffness of the beam after cracking.

| Beam ID | Ultimate moment (kN.m) |
|---------|------------------------|
|         |                        |
|                | Exp. (1) | ACI 318-19 [8] (2) | AASHTO [7] (3) | (2) (1) | (3) (1) |
|----------------|----------|--------------------|----------------|--------|--------|
| IB35-10-0.5-1  | 182.9    | 186.0              | 177.7          | 1.03   | 1.047  |
| IB35-10-0.5-2  | 119.7    | 117.8              | 121.9          | 0.982  | 0.966  |
| IB35-10-0.7-1  | 187.7    | 199.4              | 192.9          | 0.973  | 1.034  |
| IB35-10-0.7-2  | 130.5    | 133.9              | 138.8          | 0.940  | 0.965  |
| IB60-10-0.5-1  | 199.4    | 200.1              | 191.7          | 1.040  | 1.044  |
| IB60-10-0.5-2  | 130.1    | 125.1              | 128.0          | 1.016  | 0.977  |
| IB60-10-0.7-1  | 204.3    | 216.0              | 208.1          | 0.982  | 1.038  |
| IB60-10-0.7-2  | 141.8    | 142.5              | 144.7          | 0.98   | 0.985  |
| Average        |          |                    |                | 0.993  | 1.007  |
| Standard deviation σ |        |                    |                | 0.033  | 0.037  |
| Coefficient of variation COV |     |                    |                | 0.033  | 0.037  |

As shown in Table 9, the moments at failure increased from 119.7, 130.5, 130.1, 141.8 kN.m to 182.9, 187.7, 199.4, 240.3 kN.m, respectively, for beams IB35-10-0.5-2, IB35-10-0.7-2, IB60-10-0.5-2, and IB60-10-0.7-2 on an increase in the amount of bonded steel; their moment capacities were thus increased by 52, 44, 53, and 69%, respectively. The second influencing factor was the concrete compressive strength. Comparing the failure moments of beams with 35 MPa concrete compressive strength with the corresponding values for beams with 60 MPa showed that the failure moments increased by 9.0, 8.7, 8.8, and 8.7 %, respectively. Increasing the jacking stress from 0.5 $f_{pu}$ to 0.7 $f_{pu}$ also led to increases in the moment at failure by 2.6 and 2.5 for the cases where $\omega = 0.4$ and by 9.0% for the case where $\omega = 0.6$.

The nominal failure moment computed from the moment equilibrium equation, as adopted by AASHTO 2014 [7] and ACI 318M-19 [8], showed good correlation with the experimental data, with a standard deviation and coefficient of variation (COV) of 3.3 and 3.7%, respectively.

4.2 Crack pattern

Several cracks developed in the pure bending moment region between the two loads. As the load increased, new cracks appeared outside of this region, however, as shown in Figure 5.
The cracks in beams with a lower amount of tensile reinforcement increased in width more rapidly than those in the beams that had greater levels of tensile reinforcement. The average crack spacing of beams over the constant moment region varied between 110 mm and 147 mm.

4.3 Load-deflection response

The load-deflection curves exhibited three distinct stages: uncracked elastic, which ended at the initiation of the first crack; cracked elastic, which ended with the yielding of the bonded steel; and plastic. The rate of elongation in the third stage was greater than that in the second stage, which was in turn greater than the in first stage. Figure 6 clarifies that ductility was increased with the decrease in the amount of the tensile reinforcement. In the case of beams with larger quantities of tensile reinforcement, the ductility values ranged between 1.32 and 1.66, while for those beams with less reinforcement, ductility values ranged between 2.31 and 2.57. The deflection values at failure were about 20 mm to 30 mm ($l/135$ to $l/90$).
Figure 6. Experimental load-midspan deflection at different loading stages

4.4 Load-stress increase in unbonded strands

The stress increase in unbonded tendons was evaluated based on the average of the readings from the strain gauges installed on the strands and the resulting stress-strain curve of the strand. The curves of load-stain increase in the unbonded tendons, as shown in Figure 7, had the same shape as those in the load-deflection curves with regard to the three stages. There appears to be a direct relationship between the strain increment due to the applied load in the unbonded strands and the increase in concrete compressive strength, with an inverse relationship between the former and any increase in either the amount of ordinary tensile steel reinforcement or jacking stress. In this study, the value of the ultimate stress in the unbonded tendon increased with the increase of the jacking stress, as this was a summation of the effective prestress and the stress increment in the unbonded tendon. A comparison was thus made between experimental values of ultimate stress in unbonded strands and several equations mentioned previously, as shown in Table 10 and Figure 8. The best prediction was found to be generated by the equation proposed by Naaman and Al-Khairi, which had a coefficient of variation of 0.026, followed by that created by Diep and Niwa, with a COV of 0.042; the worst prediction was achieved by the German Code, with a COV of 0.127, followed by the Dutch Practice [10] version, with a COV of 0.113.
Figure 7. Load-strain increment in unbonded strands of test beams

Figure 8. Experimental vs. predicted values of ultimate stress in unbonded strands
Table 10. Ultimate stress in the unbonded strands of tested beams

| Beam ID      | Experimental results | Naaman and Al-Khairi [1] | Harajli and Kanj [2] | Chkrabarti [3] | Lee et al. [4] | Au and Du [5] | Diep and Niewa [6] | AASHTO LRFD 2014 [7] | ACI 318M-19 [8] | Canadian Standards (CAN3 – A23.3 M84) [9] | Dutch Practice [10] | German Code (DIN 1045-201) [11] |
|--------------|----------------------|--------------------------|----------------------|----------------|----------------|----------------|---------------------|------------------------|-----------------|----------------------------------------|------------------|--------------------------------------|
| IB35-10-0.5-1| 1233                 | 1182                     | 1123                 | 1189           | 1191           | 1223           | 1139                | 1234                   | 1125            | 1338                                   | 977              | 1725                                 |
| IB35-10-0.5-2| 1518                 | 1422                     | 1245                 | 1218           | 1293           | 1310           | 1376                | 1330                   | 1125            | 1484                                   | 977              | 1725                                 |
| IB35-10-0.7-1| 1576                 | 1516                     | 1495                 | 1550           | 1489           | 1573           | 1511                | 1606                   | 1497            | 1710                                   | 1367             | 1725                                 |
| IB35-10-0.7-2| 1740                 | 1714                     | 1562                 | 1578           | 1591           | 1659           | 1725                | 1702                   | 1497            | 1725                                   | 1367             | 1725                                 |
| IB60-10-0.5-1| 1386                 | 1315                     | 1239                 | 1353           | 1344           | 1280           | 1280                | 1303                   | 1209            | 1444                                   | 977              | 1725                                 |
| IB60-10-0.5-2| 1566                 | 1597                     | 1368                 | 1408           | 1447           | 1343           | 1605                | 1373                   | 1209            | 1551                                   | 977              | 1725                                 |
| IB60-10-0.7-1| 1687                 | 1639                     | 1578                 | 1693           | 1641           | 1636           | 1652                | 1675                   | 1581            | 1725                                   | 1367             | 1725                                 |
| IB60-10-0.7-2| 1762                 | 1725                     | 1707                 | 1743           | 1744           | 1699           | 1725                | 1725                   | 1581            | 1725                                   | 1367             | 1725                                 |
| **Average of** |                      | **Expected results**     |                      |                |                |                |                     |                        |                |                                        |                  |                                      |
| $f_{ub}/f_{ub}$ (exp.) | 0.970               | 0.912                    | 0.941               | 0.942          | 0.94           | 0.961          | 0.958               | 0.868                  | 1.022          | 0.751                                   | 1.121            |                                      |
| **Standard deviation** | 0.250               | 0.046                    | 0.068               | 0.044          | 0.054          | 0.04           | 0.055               | 0.075                  | 0.045          | 0.085                                   | 0.142            |                                      |
| **Coefficient of variation** | COV          | 0.026                    | 0.05                | 0.072          | 0.047          | 0.057          | 0.042               | 0.057                  | 0.086          | 0.044                                   | 0.113            | 0.127                                 |
5. Conclusions

1. Increases in the amount of tensile steel led to increases in the ultimate bearing capacity by 52, 44, 53, and 69%, and decreases in ductility, stress increase and total stress in the unbonded tendon.

2. The increase in concrete compressive strength from 35 MPa to 60 MPa led to increases in the ultimate bearing capacity of 9.0, 8.7, 8.8, and 8.7 %, and increases in stress increase and the total stress in the unbonded tendon.

3. The increase in the jacking stress from 0.5 to 0.7 of the strand ultimate strength $f_{pu}$ led to an increase in the ultimate bearing capacity by about 2.5% for the case where $\omega = 0.4$ and by 9.0% for the case where $\omega = 0.6$. This increase in jacking stress also led to a decrease in stress increase in the unbonded tendon.

4. The deflection at failure ranged from approximately $l/135$ to $l/90$.

5. The equation proposed by Naaman and Al-Khairi [1] for predicting stress in the unbonded tendon had the best correlation with the experimental results among the equations presented in this study.

6. References

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