DESIGN CONSIDERATION FOR PRETENSIONED
PRESTRESSED BEAMS

K. Maruyama and S. Rizkalla

ABSTRACT

The use of welded wire fabric as shear reinforcement has become popular among precast concrete manufacturers because of its relative ease of placement and saving of time and money. This paper summarizes the results of the first phase of a comprehensive research program designed to investigate the influence and effectiveness of welded wire fabric as shear reinforcement as compared to commonly used conventional stirrups.

Eleven pretensioned prestressed single tee beams with identical flexural reinforcement and shear span-to-depth ratio were tested. Nine of them were loaded statically up to failure, and two were under cyclic loading. In addition, two non-pretensioned beams were tested under static loading.

Test results revealed a common premature failure for nine beams tested statically due to slippage of the prestressing strands. The load level at slippage was considerably less than that predicted using the current codes' approach. The slippage significantly influenced the shear failure mode for these categories of beams. The effects of the different types of shear reinforcement on the crack behavior, overall deformation, shear behavior and ultimate strength are discussed.

Based on the test results, a mechanism to describe the overall behavior of the beam is proposed, and practical considerations and design recommendations are presented.

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INTRODUCTION

In the last few decades precast concrete structural members have become very popular in the construction field due to the saving of construction time and better quality control. The commonly used cross-sections of the beams include mainly the thin-webbed tee and channel section. Because of the narrow webs, the use of conventional closed hoop stirrups is inconvenient and time-consuming. Welded wire fabric (WWF) as shear reinforcement is an economical alternative and has become very popular with the precast concrete manufacturers due to its relative ease of placement and saving of casting time.

Welded wire fabric has been used in concrete industries for many years, especially in concrete slabs, pipes and walls. It has demonstrated very good performance in controlling cracking and preventing deterioration after cracking. There are, however, very few published studies available with regard to the use of welded wire fabric as shear reinforcement in beams.


Taylor and El-Hammadi (1980) carried out the test program on the effectiveness of welded wire fabric for controlling the shear crack behavior of reinforced concrete T-beams.

Because of the complexity of general shear mechanism, it must require a lot of extensive works to verify the effectiveness of welded wire fabric as shear reinforcement. Mirza and MacGregor (1981) pointed out the potential problem of welded wire fabric in its less ductility. Less ductile property of welded wire fabric may influence the shear behavior under fatigue loading.

As to design of pretensioned prestressed concrete beams, the development length of strands should be reconsidered when the shear behavior is dominant. Truss analogy (Thurley et al. 1975; and Collins et al. 1980) reveals that shear force produces additional tensile force in longitudinal reinforcement. Furthermore, large shear cracks should occur near ultimate stage, and those cracks must change the stress distribution in prestressing strands. This implies that the current code specification for the development length of strand (CAN3-A23.3-M84; 1984) may be insufficient.
Objectives

The objectives of this study are summarized as follows:

1) Using the pretensioned prestressed single tee beams of which shape and dimension are similar to those of commercial products, the effectiveness of welded wire fabric as shear reinforcement was examined in comparison with conventional stirrups.

2) As the shear span-to-depth ratio was small, the shear crack should influence the overall behavior of beams. The second objective of this study is to propose a mechanism to describe the overall behavior of the beams and the influence of shear cracking on the deterioration of bond stresses causing premature slippage of the prestressing strands.

3) Based on the test results and the proposed mechanism, practical considerations and design recommendations are presented.

EXPERIMENTAL PROGRAM

The experimental program was designed mainly to study the behavior of the thin-webbed pretensioned prestressed concrete beams with different types of shear reinforcement in a low range of shear span-to-depth ratio. The load was applied statically up to failure. All other parameters believed to affect the shear behaviors were kept identical except the type of shear reinforcement configuration.

In order to examine the influence of bond characteristics between prestressing strand and concrete on the failure mode, the non-prestressed beams were also tested. In addition, a couple of pretensioned beams were cyclically loaded at a load level of shear crack initiation.

Test Specimen

Fig. 1 shows the shape and dimension of the test specimen. All thirteen beams were 3600 mm long with 650 mm wide flange, 490 mm overall depth, a flange thickness of 60 mm, and a variable web width of 80 mm to 140 mm. The beams were tested at a span of 3000 mm with two point loading and a constant a/d value of 2.69.

The longitudinal tension reinforcements were two 15 W deformed bars and two 13 mm 7-wire stress-relieved prestressing strands. Two 93 (9.5 mm) deformed bars were used as longitudinal compression reinforcements. The tension reinforcement ratio \( \rho = \frac{A_s + A_p}{b d} \) is 1.24 %. The 15 W bars were anchored using standard 90 degree bends at the ends of the beams with a side cover of 25 mm. The development length of the strand for this beam was 1400 mm. There was no special treatment for the end regions of the beam to improve the bond and anchorage strength. This may be of question. The details are discussed in the later chapter.
Fig. 1 Test Specimen
The type of shear reinforcement is a main variable in this study. Totally seven different types were selected such as four types of welded wire fabric (smooth or deformed wire, with or without an additional horizontal wire at mid-height of the beam), conventional single legged and double legged stirrups, and welded stirrups mesh. In each type, the stirrup spacing was kept identical at 152 mm. The anchorage of welded wire fabric and welded stirrups mesh satisfied the requirements according to both the PCI/WRI Ad Hoc Committee (1980) and the latest ACI Code (1985).

The notation and the details of the specimen are summarized in Table 1. The material properties are shown both in Table 1 and Table 2.

<table>
<thead>
<tr>
<th>Name</th>
<th>Type of stirrup</th>
<th>$A_v$ ($\text{mm}^2$)</th>
<th>$s$ (mm)</th>
<th>$p_v$ (%)</th>
<th>$f_{vy}$ (MPa)</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>PS1-O</td>
<td>Non</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>Static</td>
</tr>
<tr>
<td>PS2-S6W</td>
<td>Single leg</td>
<td>29.0</td>
<td>152</td>
<td>0.176</td>
<td>536</td>
<td></td>
</tr>
<tr>
<td>PS3-D2</td>
<td>Double leg</td>
<td>62.3</td>
<td>152</td>
<td>0.378</td>
<td>335</td>
<td></td>
</tr>
<tr>
<td>PS4-M2</td>
<td>Welded mesh</td>
<td>31.2</td>
<td>152</td>
<td>0.189</td>
<td>393</td>
<td></td>
</tr>
<tr>
<td>PS5-D</td>
<td>Non</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>PS6-VD</td>
<td>WWF, D4.7</td>
<td>25.9</td>
<td>152</td>
<td>0.155</td>
<td>645</td>
<td></td>
</tr>
<tr>
<td>PS7-WSH</td>
<td>WWF, D4.7 with additional wire</td>
<td>30.6</td>
<td>152</td>
<td>0.183</td>
<td>564</td>
<td></td>
</tr>
<tr>
<td>PS8-WS</td>
<td>WWF, D4.7</td>
<td>30.6</td>
<td>152</td>
<td>0.183</td>
<td>558</td>
<td></td>
</tr>
<tr>
<td>PS9-WDH</td>
<td>WWF, D4.7 with additional wire</td>
<td>25.9</td>
<td>152</td>
<td>0.155</td>
<td>676</td>
<td></td>
</tr>
<tr>
<td>RS1-O</td>
<td>Non</td>
<td>---</td>
<td>---</td>
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<td>---</td>
</tr>
<tr>
<td>RS2-VD</td>
<td>WWF, D4.7</td>
<td>25.9</td>
<td>152</td>
<td>0.155</td>
<td>645</td>
<td>Cyclic</td>
</tr>
<tr>
<td>PC1-VD</td>
<td>WWF, D4.7</td>
<td>25.9</td>
<td>152</td>
<td>0.155</td>
<td>645</td>
<td></td>
</tr>
<tr>
<td>PC2-VD</td>
<td>WWF, D4.7</td>
<td>25.9</td>
<td>152</td>
<td>0.155</td>
<td>645</td>
<td></td>
</tr>
</tbody>
</table>

Note: PS-: Pretensioned prestressed beam under static loading
RS-: Prestressed beam under static loading
PC-: Pretensioned prestressed beam under cyclic loading
-0: Without web reinforcement
-VD: Welded wire fabric, deformed wire
-WS: Welded wire fabric, smooth wire
-WSH: WWF, D4.7, one additional horizontal wire at mid depth
Prestressing

The strands were stressed one by one with the help of a hydraulic jack. The applied jacking force was measured by means of load cells and electrical resistance strain gages attached to the strands.

The prestress force was applied at the eighth day by cutting the prestressing strands at the ends with an oxyacetylene torch. The compressive strength of concrete at that age was about 30 MPa while the nominal strength at 28 days of age was 40 MPa. The strain in strands was monitored by a strain indicator up to the testing day.

Testing Procedure

Fig. 2 shows the test set-up. For statical loading, the load was applied in 30 KN (15 KN in one span) intervals up to shear crack initiation, and in 15 KN intervals up to failure. At each interval, the strains of longitudinal bars, prestressing strands and stirrups were measured. The deflection of the beams was also measured by LVDT's. These readings were recorded by the Data Acquisition System.

Concrete strain and crack width were measured by the demec gage at each load interval. The reading was conducted manually.

Table 2 Material properties

<table>
<thead>
<tr>
<th>Type</th>
<th>Size (mm)</th>
<th>Area (mm²)</th>
<th>Strength (MPa)</th>
<th>Elongation (%)</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Yield</td>
<td>Ultimate</td>
</tr>
<tr>
<td>15M deformed</td>
<td>14.9</td>
<td>173.9</td>
<td>399</td>
<td>606</td>
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<tr>
<td>6M deformed</td>
<td>6.08</td>
<td>29.0</td>
<td>335</td>
<td>506</td>
</tr>
<tr>
<td>#2 smooth</td>
<td>6.30</td>
<td>31.2</td>
<td>393</td>
<td>517</td>
</tr>
<tr>
<td>#2 Mesh</td>
<td>6.30</td>
<td>31.2</td>
<td>645</td>
<td>645</td>
</tr>
<tr>
<td>WWF, D4.7 (WD)</td>
<td>5.74</td>
<td>25.9</td>
<td>676</td>
<td>676</td>
</tr>
<tr>
<td>WWF, D4.7 (W)</td>
<td>5.74</td>
<td>25.9</td>
<td>558</td>
<td>570</td>
</tr>
<tr>
<td>WWF, W4.7 (WS)</td>
<td>6.24</td>
<td>30.6</td>
<td>564</td>
<td>618</td>
</tr>
<tr>
<td>WWF, W4.7 (WSH)</td>
<td>6.24</td>
<td>30.6</td>
<td>410</td>
<td>575</td>
</tr>
<tr>
<td>P/S strand</td>
<td>12.7</td>
<td>99</td>
<td>1823</td>
<td></td>
</tr>
</tbody>
</table>
The similar procedure was done for the case of cyclic loading. The maximum load level was 180 KN at which the first shear crack initiated, and the minimum load level was 50 KN.

TEST RESULTS AND DISCUSSION

Prestress Losses

The prestress losses were evaluated by readings of strain gages attached to the prestressing strands. The readings of strains were taken by strain indicators periodically from the time of prestressing to the testing day. Fig. 3 shows the distribution of prestress losses after 30 days of casting.

As to the transfer length, both the ACI code and the CSA code specify it as 50 dh.

The test results showed very good agreement with those codes. The percentage loss at the mid span was about 10%. The value is a little higher than the calculated one which is about 15% based on the PCI Recommendations for Estimating Prestress Losses (1985). The age of testing varied from 31 days to 56 days after casting. However, the change of prestress losses after 30 days was observed very little.

Failure Mode

All nine pretensioned prestressed concrete beams under statical loading showed premature failure due to slippage of the prestressing strands. The typical load-deflection curve at mid span was shown in Fig. 4. The first flexural crack was observed at the load of 70 or 75 KN. About the load of 90 KN, the first shear crack initiated with steep inclination beneath the loading point. With increase of load a couple of shear cracks appeared as shown in Fig. 5. Until the slippage of strands occurred, the load-deflection curve was quite identical for each beam.

Fig. 3 Distribution of Prestress Losses
Fig. 4 Load-Deflection Curve

- V (KN)
- PS3-D2
- PS6-WD
- PS5-0
- slippage of strand

Fig. 5 Shear Crack Pattern (PS6-WD)
The slippage of strand was detected by the strain readings. As stated by Ghosh and Fintel (1983), the first slip does not mean the ultimate or the failure of the beam. A little more load was picked up until the maximum strength was observed. It should be noticed that after reaching the maximum load the beam could deflect in a large amount without reduction of strength. The test was terminated by the stroke limit of an actuator (100 mm). At this stage, the shear crack width was more than 1.0 mm in any case. Notwithstanding the large shear crack width, the concrete of top flange was neither punched out nor crushed.

The load at slippage was summarized in Table 3. Based on the strain compatibility analysis and the equation of the development length, such as $l_d = 0.145 f_p (f_p - 0.67 f_{ed})$ (CSA Code 1984), the load at slippage was evaluated as 150 KN. The test results showed lower values than the calculated one although the influence of the type of web reinforcement was recognized. Referring to the shear crack pattern as shown in Fig. 5, the approximate calculation was done for the development length and the load at slippage. The result indicated that the actual development length should be taken from the point where a large shear crack crossed the prestressing strand, not from the point of the maximum moment appeared.

On the contrary, two non-prestressed beams failed in shear such as a large diagonal crack finally running through the top flange outside the loading point. (See Fig. 6) As mentioned before, the cross-sectional properties of the beam were almost the same as those of the pretensioned beam. The only difference was that the prestressing strands were not pretensioned.
As to the bond behavior of untensioned prestressing strand, Salmons and McCrate (1977) proposed the equation for the embedment length of straight strand, such as \( L = 0.337f_s + 6.00 \) (in.). The notation of \( f_s \) stands for the maximum stress in the strand (ksi). As the available embedment length of the beam is 1400 mm, the equation leads to that the maximum strand stress is 964 MPa. This value can be related to the applied load of 115 KN in one span according to the strain compatibility analysis. The ultimate shear strength was less or similar to this load as listed in Table 3.

Two pretensioned beams under cyclic loading failed also in shear as shown in Fig. 7. Since the applied upper load was 90 KN which initiated a first shear crack, the slippage of strand was not expected at the early stage. With increase in number of cycles the additional shear cracks were formed. The shear crack at the middle of a shear span became large and led to failure. At failure all four legs of welded wire fabric which a large shear crack crossed were broken off.

![Shear Crack Pattern (PCI-WD)](image)

**Ultimate Strength**

The strength results were summarized in Table 3, and the calculated results were in Table 4. To estimate the shear strength of the beam without web reinforcement, a couple of code equations such as the ACI code (1985), CSA code (1984) and JSCE recommendation (1984) were used. The contribution of web reinforcement was taken into account by the truss analogy with compression strut angle of 45 degree.

Initially, all beams were expected to fail in shear. However, the large shear crack reduced the development length of prestressing strands, and led to premature failure in all cases of pretensioned beam. Although it is not clear what the maximum load stands for, the value can be an index to examine the contribution of web reinforcement and the load transfer mechanism.
### Table 3 Test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f'_c$ (MPa)</th>
<th>Re-bar yield</th>
<th>First yield</th>
<th>Slip of stirrup strand</th>
<th>$V_{cr}$</th>
<th>$V_{max}$</th>
</tr>
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<tbody>
<tr>
<td>PS1-0</td>
<td>44.4</td>
<td>---</td>
<td>98</td>
<td>---</td>
<td>113</td>
<td>90</td>
</tr>
<tr>
<td>PS2-S6W</td>
<td>43.5</td>
<td>120</td>
<td>135</td>
<td>135</td>
<td>136</td>
<td>98</td>
</tr>
<tr>
<td>PS3-D2</td>
<td>44.7</td>
<td>120</td>
<td>145</td>
<td>135</td>
<td>145</td>
<td>95</td>
</tr>
<tr>
<td>PS4-M2</td>
<td>43.2</td>
<td>120</td>
<td>130</td>
<td>120</td>
<td>133</td>
<td>96</td>
</tr>
<tr>
<td>PS5-O</td>
<td>40.5</td>
<td>---</td>
<td>105</td>
<td>---</td>
<td>113</td>
<td>87</td>
</tr>
<tr>
<td>PS6-WD</td>
<td>38.1</td>
<td>120</td>
<td>135</td>
<td>120</td>
<td>135</td>
<td>101</td>
</tr>
<tr>
<td>PS7-WSH</td>
<td>39.2</td>
<td>120</td>
<td>120</td>
<td>113</td>
<td>120</td>
<td>90</td>
</tr>
<tr>
<td>PS8-WS</td>
<td>40.2</td>
<td>120</td>
<td>113</td>
<td>115</td>
<td>120</td>
<td>99</td>
</tr>
<tr>
<td>PS9-WDH</td>
<td>41.5</td>
<td>120</td>
<td>120</td>
<td>120</td>
<td>120</td>
<td>94</td>
</tr>
<tr>
<td>RS1-O</td>
<td>41.3</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>45</td>
</tr>
<tr>
<td>RS2-WD</td>
<td>40.1</td>
<td>83</td>
<td>113</td>
<td>90</td>
<td>---</td>
<td>45</td>
</tr>
<tr>
<td>PC1-WD</td>
<td>43.7</td>
<td>Failure after 49,745 cycles</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>PC2-WD</td>
<td>44.4</td>
<td>Failure after 25,497 cycles</td>
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</table>

### Table 4 Calculated results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_c$ (KN)</th>
<th>$V_s$ (KN)</th>
</tr>
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<tr>
<td></td>
<td>ACI (V&lt;sub&gt;c&lt;/sub&gt;)</td>
<td>CSA (V&lt;sub&gt;c&lt;/sub&gt;)</td>
</tr>
<tr>
<td>PS1-O</td>
<td>103</td>
<td>95</td>
</tr>
<tr>
<td>PS2-S6W</td>
<td>(same as above)</td>
<td>38.9</td>
</tr>
<tr>
<td>PS3-D2</td>
<td>(same as above)</td>
<td>52.1</td>
</tr>
<tr>
<td>PS4-M2</td>
<td>(same as above)</td>
<td>31.1</td>
</tr>
<tr>
<td>PS5-O</td>
<td>102</td>
<td>94</td>
</tr>
<tr>
<td>PS6-WD</td>
<td>101</td>
<td>93</td>
</tr>
<tr>
<td>PS7-WSH</td>
<td>102</td>
<td>94</td>
</tr>
<tr>
<td>PS8-WS</td>
<td>(same as above)</td>
<td>42.0</td>
</tr>
<tr>
<td>PS9-WDH</td>
<td>(same as above)</td>
<td>43.2</td>
</tr>
<tr>
<td>RS1-O</td>
<td>46</td>
<td>35</td>
</tr>
<tr>
<td>RS2-WD</td>
<td>(same as above)</td>
<td>41.2</td>
</tr>
</tbody>
</table>
Generally, code equations give lower values of strength because they involve the safety factor. This trend may be emphasized for shear strength of prestressed concrete beams. Lyngeberg (1976) compared some test results with the ACI code, Danish code and the theory of plasticity. He concluded that both codes gave the lower limit of the test results, and that the theory of plasticity could show the good prediction of the ultimate shear resistance.

According to the calculation results in Table 4, all three codes give lower shear strength than the actual shear cracking strength for prestressed beams. For non-prestressed beams the trend is similar. The predicted value from any code equation is rather the shear cracking strength than the ultimate shear strength.

Two prestressed beams under cyclic loading were only for examining the failure mode at this stage. However, the number of load cycle at failure was less than 50,000 for both cases. The load level of first shear crack initiation seems very high for fatigue strength.

Contribution of Web Reinforcement

Since the shear failure was not clearly recognized in all nine prestressed beams, the contribution of web reinforcement cannot be read directly from the strength results. As listed in Table 3, the load at slippage of strand or the maximum load varied in beams with different type of web reinforcement. The results may indicate that the conventional stirrups are better than the welded wire fabrics.

Although the premature failure makes it difficult to see the contribution of web reinforcement directly, a couple of stirrups already get yield before slippage of strands. This is a common feature for all nine prestressed beams. The load at the first yield of stirrup is summarized in Table 3. The value looks similar for all prestressed beams because the amount of $A_{sy}$ was controlled to be approximately the same.

The load increment from the first shear crack initiation ($V_{cr}$) to slippage of strands may be an index to evaluate the contribution of web reinforcement.

Fig. 8 Contribution of Stirrups
The lead increment was compared with the stirrup contribution calculated by the truss analogy in which the angle of compression strut is 45 degree. (See Fig. 8.) The beam with conventional stirrup showed a good coincidence with the truss analogy. On the other hand, the beams with welded wire fabric did not have full contribution of stirrups.

Shear Crack Width

In order to evaluate the shear crack width, demec points were attached in three directions, such as horizontal, vertical and diagonal directions. The basic gage length was 200 mm in each direction as shown in Fig. 9. When the shear crack angle is measured, the deformation of a range can be divided into two components such as the vertical and the parallel movement to the shear crack. The vertical movement to the shear crack is defined as the shear crack width (W), and the parallel movement as the slide (S) as shown in Fig. 10. Theoretically, a shear crack angle and deformations in two directions are enough to determine the two components. However, a shear crack does not always run across the center of a range. In that case the deformations in two pertinent directions were used for evaluating the shear crack width.

The maximum shear crack width was compared in Figs. 11 and 12. Once the prestressing strand began to slip, the maximum shear crack width became considerably large. As far as control of shear crack width is concerned, there seems very little influence of the type of web reinforcing until the stirrups got yield. The conventional stirrups, single legged and double legged stirrups were better to confine the crack width after yielding. These stirrups were hooked to both tension and compression reinforcing bars.
Fig. 11 Maximum Shear Crack Width

Fig. 12 Maximum Shear Crack Width
The slippage of prestressing strands prevented the prestressed beams from clear shear failure. However, some stirs already had yield before slippage of strands in any beam. In spite of large shear cracks at the maximum load, the beam could deflect without shear failure. On the other hand, unpretensioned beams failed in shear. These facts should be a key to consider the shear resisting mechanism.

From the observation of failure of a beam, it can be considered that what controls the failure of the beam is the stress or strain condition of concrete in compression zone. But it is not easy to measure the strain condition of concrete because the compression zone is so thin and the strain is not uniform in the top flange.

In order to examine the failure condition of the beam, the authors propose to consider the deformation of the web element in tension because the stress or the strain condition of concrete in compression zone must be related to the deformation of concrete in tension zone. Figs. 13-15 shows one example to express the overall behavior of deformation in tension zone. The total movement in the figure represents the summation of the vertical and the parallel movements to the shear crack in all denec measurement ranges in one shear span. The denec ranges in the web are shown in Fig.16.

The denec measurements could not be conducted very near ultimate. However, the extrapolation of the total movement up to the maximum load indicates that the amount of total movement at the maximum load or at failure looks similar in any beam as shown in Fig.15. This phenomenon makes the following hypothesis possible:

"The stress or strain condition of concrete in compression zone must be unique at ultimate. The way how to reach the unique condition should be specified by how the concrete element of the beam is confined."

As the means of confining concrete, longitudinal reinforcement, web reinforcement and prestressing may be treated in the same manner. When the confinement of concrete is good, the beam can carry more load before reaching ultimate.

In comparison of prestressed beams with non-prestressed beams in Fig.15, the effectiveness of prestress on confinement seems constant for both beams with and without web reinforcement. The contribution of web reinforcement may be similar in prestressed beams and non-prestressed beams as well. The slippage of prestressing strands reduces the confinement effect and decreases the ultimate load. When the slippage occurred, the conventional hooked stirs or stirrups, single legged or double legged, look better for confinement than any kind of welded wire fabric.
Fig. 13 Total Movement

Fig. 14 Total Movement
Fig. 15 Total Movement
DESIGN CONSIDERATIONS

The beam configuration used in this study is a typical commercial pretensioned prestressed beam. Any special devices were not designed. This means that the observations in this study are directly applicable to the actual beams. From the test results, the following should be considered in design.

1. When the shear span to depth ratio is small and the shear behavior is dominant, the current codes (CSA code or ACI code) are not safe for the development length of prestressing strands in pretensioned T-beams. The position where a possible large shear crack will cross a prestressing strand should be taken into account for determining the maximum stress point in a strand.

2. When the shear crack position is considered, the current CSA code equation for development length of prestressing strands such as

\[ l_d = 0.145(f_{ps} - 0.67f_{se})d \]

is effective.

3. When the full development length cannot be provided, the permissible strand stress may be calculated by the above code equation. The maximum moment resistance should be obtained by the tied arch model which is formed by two large shear cracks and longitudinal reinforcements.
REFERENCES


APPENDIX

Notation

$A_v = \text{cross sectional area of web reinforcement}$

$b_w = \text{web width}$

$d_p = \text{nominal diameter of prestressing strand}$

$f_c = \text{specified compressive strength of concrete}$

$f_{ps} = \text{stress in prestressing strand}$

$f_{se} = \text{effective stress in prestressing strand}$

$f_{wy} = \text{specified yield strength of web reinforcement}$

$l_d = \text{development length}$

$p_v = \text{web reinforcement ratio, } = \frac{A_v}{b_w}$

$S = \text{slide, parallel movement to shear crack}$

$s = \text{spacing of web reinforcement}$

$V = \text{shear force}$

$V_c = \text{shear resistance of beam without web reinforcement}$

$V_{cr} = \text{cracking shear resistance}$

$V_{max} = \text{applied maximum shear force}$

$V_s = \text{shear resistance by web reinforcement}$

$V_{slip} = \text{shear force at slippage of prestressing strand}$

$W = \text{shear crack width, normal movement to shear crack}$

$W_{max} = \text{maximum shear crack width}$