The double tee is a major production item of many integrated precast, prestressed concrete manufacturers in the United States. Thus, the shear reinforcement requirement by ACI-3181 and its omission by producers has attracted the interest of design engineers and prestressers alike.

Since 1971, several companies have conducted tests on full size untopped or topped double tees. One of the authors conducted a test in 1986 related to shear mesh elimination. The test objective was to show that web reinforcement in the stems could be eliminated without impairing the flexural or shear capacities of pretensioned tees. The test objective was met, and several producers invoked the provisions of Chapter 11, ACI-318, to waive the minimum area of shear reinforcement.

The 1983 issue of the ACI-318 Code, however, added a new requirement to tests used as a basis for omitting minimum shear reinforcement. Section 11.5.5.2 states that such tests shall simulate effects of differential settlements, creep, shrinkage, and temperature change. It is common practice among prudent designers to simulate such volume change effects on the bearings by applying a horizontal tensile force \( N = 0.20 V \), where \( V \) is the vertical reaction. This force magnitude is also mentioned in Ref. 1, Section 11.9.3.4.

In flexural members subject to normal loading, the flexure-shear design of reinforcement should be checked in addition to the minimum reinforcement. The concrete contribution is given by:

\[
V_{ci} = 0.6 \sqrt{f'c b_e d} + V_d + \frac{V_t M_{cr}}{M_{max}}
\]

To expand on the available data base, the 1987 and 1988 tests were conducted at the High Concrete Structures, Inc. plant in Denver, Pennsylvania. Details
are presented here, along with results and conclusions. The tests are, in certain respects, an extension of earlier tests conducted by other companies. However, they do have some unique features:

(a) A fully precast 4 in. (102 mm) thick flange.
(b) A longer span, 62 ft (18.9 m), and deeper sections, 26 in. (660 mm) and 34 in. (864 mm), respectively.
(c) A simultaneous horizontal force equal to 20 percent of the applied vertical force.
(d) A higher than usual nominal live load, 54 to 70 psf (264 to 342 kg/m²).
(e) End sleeving of strands.
(f) A relatively slender leg in at least one specimen.

Consistent with PCI’s SFRAD Project No. 2, the test tees contained no web reinforcement except in the outer 5 ft (1.52 m) of the member where it provides protection against accidental damage.

**TEST PROGRAM**

The static load tests were conducted on full scale, totally precast double tees. The specimens were typical production double tees spanning 62 ft (18.9 m), representing similar units used in normal parking structures. The specific properties and dimensions of the two specimens tested are presented in the following paragraphs.

**Description of Specimen 1**

The test specimen consisted of a 10 ft wide x 34 in. deep (3.05 m x 864 mm) double tee cast on October 30, 1987, with normal weight concrete and was designed for a fully precast parking structure application. The member length was 62 ft 6 in. (19.05 m) and the flange was 4 in. (102 mm) thick and cast monolithically with the double tee. The nominal concrete strength at 28 days was $f'_c = 5000$ psi (34.5 MPa), which is commonly used in the plant’s marketing area. The design service load was 70 psf (342 kg/m²). Section properties are listed in Fig. 1.

Fabrication of the tee was in accordance with the drawings reproduced as Figs. 2 and 3. The specimen contained twelve $\frac{1}{2}$ in. (13 mm) diameter, low relaxation, Grade 270 strands tensioned to 75 percent of the Ultimate Tensile Strength (UTS). The second strand level in each stem was sleeved for a length of 8 ft 6 in. (2.60 m) from the ends while the third level was sleeved for 4 ft 6 in. (1.37 m).
The leg mesh vertical wire was 6 gage at 7.5 in. (190 mm) on center, and the mesh extended for a length of 5 ft (1.52 m) at each end. The deck reinforcement consisted of #3 (9.5 mm) reinforcing bars spaced at 12 in. (305 mm) on center and was placed in the middle of the flange thickness. Each bearing area was reinforced with two #4 (12.7 mm) mild steel reinforcing bars. Details of the concrete mix properties, along with the prestress strand data, are displayed in Appendix A. At test time the concrete strength was $f'_c = 6467$ psi (44.6 MPa).

**Description of Specimen 2**

The test specimen consisted of a 10 ft wide x 26 in. deep (3.05 m x 660 mm) double tee cast on February 23, 1988, with normal weight concrete. The member length was 62 ft 6 in. (19.05 m) and the flange was 4 in. (102 mm) thick and cast monolithically with the double tee. The nominal concrete strengths were $f'_c = 3000$ psi (20.7 MPa) and $f'_c = 5000$ psi (34.5 MPa), respectively. The service load was 54 psf (264 kg/m²).

Fabrication of the tee was in accord-
Fig. 3. Double tee reinforcement.

Fig. 3. Double tee reinforcement.

ance with the drawings reproduced as Figs. 4 to 6. The specimen contained fourteen ½ in. (13 mm) diameter, low relaxation, Grade 270 strands tensioned to 75 percent UTS. The second strand level in each stem was sleeved for a length of 8 ft (2.44 m) from the ends while the fourth level was sleeved for 3 ft 6 in. (1.07 m).

The leg mesh vertical wire was 6 gage at 7 in. (178 mm) on center, and the mesh extended for a length of 5 ft (1.52 m) at each end. The deck reinforcement in the northern half of the slab consisted of #3 (9.5 mm) reinforcing bars spaced at 12 in. (305 mm) on center and was placed in the middle of the flange thickness. In the southern half of the slab, the deck reinforcement was a 12 x 4 — W2 x W4 mesh. Each bearing area was reinforced with two #4 (12.7 mm) mild steel reinforcing bars. Details of the concrete properties, along with the prestressing strand data, are displayed in Appendix B. At test time the specimen’s concrete strength was $f'_{c} = 5031$ psi (34.7 MPa).

TEST SETUP

Structural steel supports were fabricated to provide a pinned bearing condition as well as supply the required horizontal tensile force equal to 20 percent of the end reactions. These test supports held the slab approximately 2 ft 6 in. (762 mm) above the ground so the tee could freely deflect. One of the supports had a link tilted at 11.3 degrees from the vertical to generate the 20 percent horizontal force (Fig. 7). The tee was placed directly on these supports and welded at the bearing plates to allow for horizontal forces involved. As a safety measure, large timbers were stacked near the pivot roller support, but a few inches of clearance were available under the tee.

Changes in the slab midspan camber (or deflection) were measured from the original, unloaded and cambered position labeled the “zero position” until total deflection changes exceeded 15 in. (381 mm).
Fig. 4. 1ODT26 precast section dimensions and calculated section properties.

<table>
<thead>
<tr>
<th>Width (ft)</th>
<th>Depth (in.)</th>
<th>Weight (lbs per ft)</th>
<th>$A_c$ (sq in.)</th>
<th>$I$ (in.$^4$)</th>
<th>$Y_f$ (in.)</th>
<th>$Y_B$ (in.)</th>
<th>$S_f$ (in.$^3$)</th>
<th>$S_B$ (in.$^3$)</th>
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<tbody>
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<td>Precast</td>
<td>10</td>
<td>26</td>
<td>752</td>
<td>722</td>
<td>34766</td>
<td>6.134</td>
<td>19.866</td>
<td>5668</td>
</tr>
</tbody>
</table>

Fig. 5. Double tee dimensions.

Fig. 8 shows an end view of the double tee setup and Fig. 9 shows the tee loaded with blocks near the end of the test.

**TEST LOADS**

In these tests, large concrete blocks measuring 10 in. x 2 ft x 8 ft (254 x 610 x 2438 mm) and weighing 2 kips (8.9 kN) each were used to simulate the quasi-uniform loading condition. Twenty-one blocks spaced 2 ft 6 in. (762 mm) apart for the central three blocks and 3 ft (914 mm) elsewhere were used to create a uniform loading condition in the first two layers. One or more blocks were added in the third layer until the maximum test load was reached.

The placement of these blocks was structured so that loading would progress in stages. These stages were established to compare the actual flexural behavior with that predicted by rational analysis using a sophisticated method and a computer program.8

For Specimen No. 1, the loading
stages were (see Figs. 10 to 13):

Stage I — Service loading, equivalent to 68 psf (332 kg/m²); maximum bottom tension = 231 psi (1.6 MPa).

Stage II — 0.85 (1.4 DL + 1.7 LL) as per Chapter 20, ACI Code; maximum nominal bottom tension = 1225 psi (8.4 MPa).

Stage III — 1.0 (1.4 DL + 1.7 LL) as required by the ACI Code and most building codes.

Stage IV — \( M_n \), nominal flexural strength assuming \( \phi = 1.0 \).

For Specimen 2, the loading stages were (see Figs. 14 to 16):

Stage I — Service loading, equivalent to 56 psf (273 kg/m²); maximum bottom tension = 490 psi (3.4 MPa).

Stage II — 0.85 (1.4 DL + 1.7 LL); maximum bottom tension = 1582 psi (10.9 MPa).

Stage III — 1.0 (1.4 DL + 1.7 LL); \( M_n \), nominal flexural strength, is calculated based on \( \phi = 1.0 \).

Schematic diagrams of each stage are presented showing placement of weights. Also included in Figs. 10 through 16 are tabulations at each stage of applied loads, actual midspan moments, moments at 0.4 \( L \) and predicted deflections.

The placement of loads was carried out in a continuous (monotonic) process from start to finish with occasional delays to permit inspection and deflection measurements. The total time for each test was about 2½ to 3 hours.

**Test Results for Specimen 1**

The test slab carried a maximum load of 106,000 lbs (471 kN) in addition to its self weight. At this load, the test was stopped for safety reasons since deflection change approached 27 in. (686 mm)
TYPICAL BLOCKS WEIGH 20 K EACH

PIVOT ROLLER SUPPORT (SEE DETAIL BELOW)

Fig. 7. Test setup.

Fig. 8. A view of the pivot roller support.
and the tee was only 6 in. (152 mm) above ground. The rate of creep deflection was high, and further loading was neither prudent nor beneficial.

If the secondary moments due to the horizontal forces are neglected as usual, then the total applied moment at 0.50 L reached 14614 in.-kips (1651 kN-m) under the maximum test loading. The calculated nominal strength using the PCI Design Handbook strain compatibility approach (Ref. 4, p. 4-63) is 14742 in.-kips (1666 kN-m) at the same section. In other words, by using the PCI Design Handbook method, the applied test load reached 99 percent of the nominal strength near the critical section.

Deflection changes from the zero position were:
- 6 in. (16 mm) at the end of Stage I, under one layer of loads equivalent to 68 psf (332 kg/m²).
- 3 in. (76 mm) at the end of Stage II.
- 4.25 in. (108 mm) under two layers of blocks, equivalent to 136 psf (664 kg/m²).
- 14 in. (356 mm) at the end of Stage III, visibly increasing due to high creep rate.
- 27 in. (686 mm) when test loading stopped.

First flexural cracks appeared in the middle of the span at the hold-down device location. The loads consisted of one layer of blocks plus two additional blocks at the east end and corresponded to a superimposed moment of 3812 in.-kips (431 kN-m) at the 0.40 L section. As loads were added, the flexural cracking region became more extensive and progressed toward the supports. At the maximum test load, the nearest visible flexural shear cracks were between 1.3 and 1.8 ft (3.96 and 5.49 m) from each bearing. Initially, the crack widths were \( \frac{3}{16} \) in. (5 to 6 mm) near the central portion of the span and about \( \frac{3}{25} \) in. (1 mm) for the cracks nearest to the supports.

The loss of stiffness due to cracking of the concrete was smooth and gradual, as shown in the moment versus deflection curve (Fig. 17). Throughout the test

Fig. 9. 1ODT34 beyond Stage III (1.4 DL + 1.7 LL) and near the end of test.
**STAGE I: SERVICE LOADS**

Precast double tee self weight:  

Given service live load: 70 psf x 10 ft  

Moment due to given service live load:  
- at midspan: 1.5 (0.700)(62)^2  
- at 0.40 L: 0.96 x 4036  

Applied test load moments:  
- at midspan: 1.5 (0.677)(62)^2  
- at 0.40 L: 0.96 x 3904  

Maximum calculated bottom tension under test load  
- = 213 psi  

Calculated camber change under test load  
- = 0.69 in.  

Measured camber change: 5/8 in.  

---

**Test Results for Specimen 2**

The test slab carried a maximum load of 86,000 lbs (383 kN) in addition to its self weight. At this load, the test was stopped since deflection change exceeded 30 in. (762 mm), the tee was only 2 in. (51 mm) above ground and the rate of creep deflection was high. After a few minutes the tee legs touched the ground.

If the secondary moments due to the horizontal forces are neglected, then the total applied moment at 0.45 L reached 12363 in.-kips (1397 kN-m) under the maximum test loading. The calculated nominal strength using the PCI Design Handbook strain compatibility ap-
Stage II: 0.85 (1.4 DL + 1.7 LL)

Total \( w = 0.85 (1.4 \times 0.809 + 1.7 \times 0.700) \) = 1.974 klf

Less existing dead load = 0.809 klf

Required additional load = 1.165 klf

Required additional moment:

at midspan: \( 1.5(1.165)(62)^2 \) = 6,719 in.-kips

at 0.40 \( L \): 0.96 \times 6,719 = 6,450 in.-kips

Applied test load moments:

at midspan: = 6,904 in.-kips

at 0.40 \( L \): = 6,584 in.-kips

Maximum calculated bottom tension under test load = 1,225 psi

Calculated camber change under test load, including assumption of cracked sections: 2.17 + 0.37 = 2.54 in.

Measured camber change = 3.00 in.

---

Deflection changes from the zero position were:

- 1 in. (25 mm) at the end of Stage I, equivalent to 56 psf; no visible cracking.
- 4 in. (102 mm) at the end of Stage II.
- 7.25 in. (184 mm) under two layers minus blocks marked a, b, and c on Fig. 16.
- 30 in. (762 mm) when last load was placed at centerline, visibly increasing due to high creep rate.

First flexural cracks appeared in the middle of the span at the hold-down device location. The loads consisted of one full layer of blocks and corresponded to a superimposed moment of 3750 in.-kips (424 N-m) at the 0.40 \( L \) section. As loads
Stage III: 1.0 (1.4 DL + 1.7 LL)

Total \( w = 1.4 \times 0.809 + 1.7 \times 0.700 \) = 2,323 kIf

Less existing dead load

Required additional load

Required additional moment:

at midspan: \( 1.5 \times (1.514)(62)^2 \) = 8,727 in.-kips

at \( 0.40 L \); \( 0.96 \times 8,727 \) = 8,378 in.-kips

Applied test load moment:

at midspan:

at \( 0.40 L \);

Fig. 12. Stage III: 1.0 (1.4 DL + 1.7 LL).

were added, the flexural cracking region became more extensive and progressed toward the supports. At the maximum test load, the nearest visible flexural shear cracks were between 12 and 14 ft (3.66 and 4.27 m) from each bearing. Initially, the crack widths were \( \frac{3}{16} \) to \( \frac{5}{16} \) in. (5 to 8 mm) near the middle of the span and about \( \frac{1}{64} \) in. (1 mm) for the cracks nearest to the supports.

Here again, the loss of stiffness due to cracking of the concrete was smooth and gradual, as shown in Fig. 18. Throughout the test loading, flexural cracks did not propagate to cause flange separation, nor was there any sign of impending premature shear failure at the ends or near the one-quarter points. Diagonal hairline cracks appeared near two stem supports at the test end, but were so small [\( \frac{3}{160} \) in. (0.17 mm)] as to be considered inconsequential.
STAGE IV: \( M_n \), NOMINAL FLEXURAL STRENGTH ASSUMING \( \phi = 1.0 \)

The theoretical strengths, "Provided \( M_n \)," are equivalent to \( M_n \) since \( \phi = 1.0 \) and based on the BEAM Program by Jacques & Aswad, Inc.:

- at midspan: \( M_n = 15,043 \) in.-kips
- at 0.40 \( L \): \( M_n = 14,539 \) in.-kips

When the PCI Design Handbook approach for strain compatibility is used (Ref. 4, p. 4-63), the following strengths are obtained:

- at midspan: \( M_n = 14,742 \) in.-kips
- at 0.40 \( L \): \( M_n = 14,248 \) in.-kips

Using the loading scheme below, the total actual applied load moments at the end of the test were:

- at midspan: \( M = 14,614 \) in.-kips
- at 0.40 \( L \): \( M = 14,040 \) in.-kips

Therefore, neglecting as usual the contribution of the horizontal reactions to the bending moment, the applied test load moment at 0.50 \( L \) is within 0.9 percent of the expected nominal strength.

**Analysis for Deflection and Shear Reinforcement**

Using the BEAM Program,\(^5\) one computer run was made for each loading stage assuming actual concrete strengths. These runs were made to predict the midspan deflection change under the various superimposed load configurations. The results are shown in Figs. 17 and 18. The correlation between the predicted and measured values is remarkably good for loads significantly higher than the service loads and slightly beyond Stage II, 0.85 (1.4 DL + 1.7 LL).

Two other computer runs were made with the purpose of calculating the required shear reinforcement based on ACI 318\(^1\) Eqs. (11-11) and (11-13). Results of these runs are summarized on the next page:

Fig. 13. Stage IV: nominal flexural strength (\( \phi = 1.0 \)) and maximum load configuration.
STAGE I: SERVICE LOADS

Precast double tee self weight:  
Given service live load: 54 psf x 10 ft  
Moment due to given service live load:  
at midspan: 1.5 (0.540)(62)^2  
at 0.40 L: 0.96 x 3,114

Applied test load moments (Run No. 5A):  
at midspan:

Maximum calculated bottom tension under test load  
(No visible signs of flexural cracking)
Calculated camber change under this test load  
Measured camber change

DLb = 0.752 klf  
LL = 0.540 klf

= 3,114 in.-kips  
= 2,989 in.-kips

= 3,220 in.-kips  
= 3,152 in.-kips

= 490 psi  
= 1.25 in.

= 1.00 in.

Fig. 14. Stage I: service loads.

- Specimen 1: using $f'_c = 6467$ psi (44.6 MPa)
  - Required web reinforcement based on $V_{ci}: A_n = 0$ in.$^2$ ft
  - Required web reinforcement based on $V_{cm}: A_n = 0$ in.$^2$ ft

- Specimen 2: using $f'_c = 5031$ psi (34.7 MPa)
  - Required web reinforcement based on $V_{ci}: A_n = 0.03$ in.$^2$ ft
  - Required web reinforcement based on $V_{cm}: A_n = 0$ in.$^2$ ft

Note: Web reinforcement is the total area for two stems.

CONCLUSIONS

1. The omission of web reinforcement in tee stems, except in the outer 5 ft (1.52 m), did not impair the shear or flexural capacities of two full size double tee specimens when subjected to static, quasi-uniform loading.
STAGE II: 0.85 (1.4 DL + 1.7 LL)

Total \( w = 0.85 \times (1.4 \times 0.752 + 1.7 \times 0.540) \) = 1.675 klf

Less existing dead load = -0.752 klf

Required additional load = 0.923 klf

Required additional moment:
- at midspan: \( 1.5 \times (0.923)(62)^2 \) = 5,323 in.-kips
- at 0.40L: \( 0.96 \times 5,323 \) = 5,110 in.-kips

Applied test load moments (Run No. 6):
- at midspan: = 5,380 in.-kips
- at 0.40L: = 5,135 in.-kips

Maximum calculated bottom tension under test load = 1,582 psi

Minor flexural hairline cracks showed up in the central 0.20L of the slab after four blocks close to midspan were added. Midspan moment was \( M = 4,590 \) in.-kips and deflection change reached 2.06 in.

Calculated camber change under test load, including assumption of cracked sections: 2.57 + 1.97 = 4.54 in.

Measured camber change = 4.00 in.

Fig. 15. Stage II: 0.85 (1.4 DL + 1.7 LL).

2. The presence of horizontal forces at the bearings and partial end sleeving of strands did not cause any special distress or precipitate a premature shear failure.

3. Deflections at load levels substantially beyond cracking loads can be accurately predicted using rational methods of analysis. The transition from uncracked to cracked sections is gradual and smooth, reflecting the steady propagation of cracks from the center toward the ends. There is no sudden loss of stiffness.
STAGE III: $M_n$, NOMINAL FLEXURAL STRENGTH ASSUMING $\phi = 1.0$

The theoretical strengths, "Provided $M_n$," are equivalent to $M_n$ since $\phi = 1.0$ and based on the BEAM Program by Jacques & Aswad, Inc.:

- at midspan: $M_n = 12,687$ in.-kips
- at $0.45L$: $M_n = 12,427$ in.-kips
- at $0.40L$: $M_n = 12,167$ in.-kips

When the PCI Design Handbook approach for strain compatibility is used (Ref. 4, p. 4-63), the following strengths are obtained:

- at midspan: $M_n = 12,433$ in.-kips
- at $0.45L$: $M_n = 12,178$ in.-kips
- at $0.40L$: $M_n = 11,923$ in.-kips

Using the loading scheme below, the total actual applied load moments at the end of the test were:

- at midspan: $M = 12,521$ in.-kips
- at $0.45L$: $M = 12,363$ in.-kips
- at $0.40L$: $M = 11,961$ in.-kips

Under the maximum load, the total immediate deflection change is reached 30 in. $(\pm)$, increasing very rapidly due to the high creep. After a few minutes the slab touched the ground. Therefore, neglecting as usual the contribution of the horizontal reactions to the bending moment, the applied test load moment at $0.45L$ is 101 percent of the expected nominal strength by the PCI Design Handbook approach (and more than 99 percent of the nominal strength based on Jacques & Aswad's BEAM Program).

Fig. 16. Stage III: nominal flexural strength ($\phi = 1.0$) and maximum load configuration.
Fig. 17. Midspan moment versus deflection change.

Fig. 18. Midspan superimposed moment versus deflection change.
RECOMMENDATIONS

Given the results of these tests, and considering the prior tests by the first author and other researchers, and the successful use of double tees without web reinforcement on hundreds of projects since 1971, the authors offer the following recommendations applicable to precast tees.

Shear reinforcement may be omitted except in the outer 5 ft (1.52 m), provided:

(a) The tees are simply supported.
(b) Loads are substantially uniform.
(c) Tee depth is between 24 and 36 in. (610 and 914 mm) and span-to-depth ratio is less than 30.
(d) Required shear reinforcement based on $V_{ew}$, Eq. (11-13) of ACI-318, is essentially zero.
(e) Calculated bottom tension stress is kept below the cracking stress, $7.5 \sqrt{f_e}$.

ACKNOWLEDGMENTS

The work in this paper is part of a project sponsored by High Concrete Structures, Inc. The authors would like to thank Ken Baur, P.E., for his support, encouragement and comments. Any opinions, findings and recommendations are those of the authors and do not necessarily reflect the views of Penn State University or the sponsors.

Thanks are also due to Jacques and Aswad, Inc., for the use of their BEAM Program to make several computer runs for the stress, strength, and rational deflection analyses.

REFERENCES

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, Michigan, 1983, 111 pp.

METRIC (SI) CONVERSION FACTORS

<table>
<thead>
<tr>
<th>Metric</th>
<th>Conversion Factor</th>
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<tbody>
<tr>
<td>1 ft = 0.305 m</td>
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</tr>
<tr>
<td>1 in. = 25.4 mm</td>
<td></td>
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<tr>
<td>1 pcf = 16.02 kg/m³</td>
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<td>1 lb per ft = 1.488 kg/m</td>
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<tr>
<td>1 psi = 0.006895 MPa</td>
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<tr>
<td>1 kip-ft = 1356 N-m</td>
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<tr>
<td>1 kip = 4448 N</td>
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</table>

NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by December 1, 1989.
APPENDIX A

I. Precast Concrete Data:

Unit weight: 149 lbs/ft³; Slump: 5 in. Cement: Type III.

Admixtures:

- Air entraining agent: 8 ounces
- Water reducing agent: 20 ounces
- HRWR: 106 ounces
- Accelerator: 13 ounces

Cylinder strengths:

- At release: Average of two breaks: 3230 psi
- At 7 days: Average of two breaks: 5149 psi
- At 24 days (test time): Average of two breaks: 6467 psi

Dates: Pour date: 10/30/87; stripping date: 10/31/87; test date: 11/23/87.

II. Prestressing Strand Properties:

Specifications: ASTM A-416, Grade 270 K, Low relaxation

Cross section metallic area: 0.154 in.² per strand

Breaking strength (UTS): 43,703 lbs; elongation in 24 in.: 6.9 percent

Yield strength at 1 percent extension: 38,982 lbs

Modulus of elasticity: 29,100 ksi

APPENDIX B

I. Precast Concrete Data:

Unit weight: 151 lbs/ft³; Slump: 4¼ in. Cement: Type III.

Admixtures:

- Air entraining agent: 8 ounces
- Water reducing agent: 20 ounces
- HRWR: 106 ounces
- Accelerator: 13 ounces

Cylinder strengths:

- At release: Average of two breaks: 3678 psi
- At 7 days: Average of two breaks: 4837 psi
- At test time: Average of two breaks: 5031 psi

Dates: Pour date: 2/23/88; stripping date: 2/24/88; test date: 3/10/88

II. Prestressing Strand Properties:

Specifications: ASTM A-416; Grade 270 K, Low relaxation

Cross section metallic area: 0.154 in.²/strand

Breaking strength (UTS): 44,321 lbs; elongation in 24 in.: 6.4 percent

Yield strength at 1 percent extension: 40,793 lbs

Modulus of elasticity: 29,100 ksi