Fatigue Strength of Joints in a Precast Prestressed Concrete Double Tee Bridge

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The feasibility of using precast, prestressed double tees to form a monolithic bridge system is examined by conducting static and fatigue load tests on a 1:3.5 scale model of a two-span, transversely and longitudinally post-tensioned, continuous double tee beam system. Constant amplitude fatigue loading was applied on the model at typical locations simulating HS20-44 AASHTO truck loading.

The behavior of the bridge system was evaluated with regard to structural integrity. Crack widths in the longitudinal and transverse joints were monitored with increasing cycles of fatigue loading. A finite element analysis of the bridge system was carried out using orthotropic modeling, and the deflections were compared with experimental values.

The ultimate load, computed from plastic analysis, was found to be in good agreement with the measured value. The study established the feasibility and structural adequateness of the precast, prestressed concrete double tee concept for short and medium span highway bridges in Florida and elsewhere.

The need to replace the multitude of deteriorated highway bridges in the United States has stimulated efforts to intensify the search for more economical bridge systems.

Recently, an extensive program was initiated by the Florida Department of Transportation to study the feasibility of using a precast prestressed double tee system for bridges with spans up to 80 ft (24.4 m). The experimental investigation comprised two parts: (1) study the behavior of a simply supported half scale bridge model with three double tees joined together by simple longitudinal V joints and transversely post-tensioned; and (2) examine the effect of prestressed steel profiles and shielded strands on the ductility of double tees. A summary of this research work was reported by El Shahawy in the September-October 1990 PCI JOURNAL.[1]

The purpose of this paper is to report the results of a series of fatigue tests on a 1:3.5 scale model of a two-span continuous double tee bridge structure post-tensioned both transversely and longitudinally.

BACKGROUND

Double tees have been frequently used in the past for rural and secondary roads. However, because of their structural strength, cost effectiveness and ease of construction, there is growing interest in using double tees in longer spans [up to 80 ft (24.4 m)]. Nevertheless, with large load requirements, coupled with more slender structural members and higher working stresses, there is a need to check the fatigue behavior and ultimate strength of longitudinal and transverse joints in double tee structural systems.

Csagoly et al[2] designed and tested a system of three precast half-scale model double tees, representing a prototype bridge of 60 ft (18 m) span. The double tees were joined together by simple longitudinal V joints and transverse post-tensioning. The degree of prestress was established to ensure sufficient local punching shear strength and overall monolithic behavior.

Turner et al[3] tested the effects of repetitive loading on the serviceability and strength of composite panel form bridges. A study of the causes of cracking in composite bridge decks, structural adequacy in the cracked condition, structural performance in compression as for conventional concrete construction, and remedial measures for cracked decks was made by Fagundo et al.[4]

Based on fatigue tests on a 127 ft (39 m) span prestressed concrete bridge, Rosli[5] found that the deflections remained small in comparison with the span of the bridge and showed only minor increases after application of several million load cycles. The prestress in the bridge slab was fully effective, even at high load levels.

In their shear tests on AASHTO beams, bulb tees and double tees, Csagoly et al[2] observed that shielding of strands decreased the shear capacity both at serviceability and ultimate limit states. Reynolds and Gamble[6] described the instrumentation developed to study the behavior of prestressed concrete bridges in the field.

After only a few years in service, the joints in bridges may become potential problem areas. Therefore, in the design of such structures it is important to allow for the proper transmission of normal and shear stresses in the bridge deck and to protect the prestressing tendons from corrosion due to water entering the ducts. The reduction or elimination of maintenance costs become significant design considerations in the construction of bridges.

This paper discusses the design concepts, describes the static and fatigue tests on a 1:3.5 scale model of a two-span transversely and longitudinally post-tensioned continuous double tee bridge system (Fig. 1), and examines the feasibility of using precast double tees to form a monolithic bridge system. Constant amplitude fatigue loading was applied on the model at typical locations simulating HS20-44 AASHTO truck loading (Fig. 2).

The bridge system was evaluated with regard to structural integrity (monitored by crack widths), behavior of longitudinal and transverse joints with increasing cycles of fatigue loading, and local punching shear resistance of the slab. A finite element analysis of the bridge system was carried out using orthotropic modeling, and the load-deflection behavior was compared with experimental results. The ultimate load, computed from plastic analysis, agreed closely with the measured value.

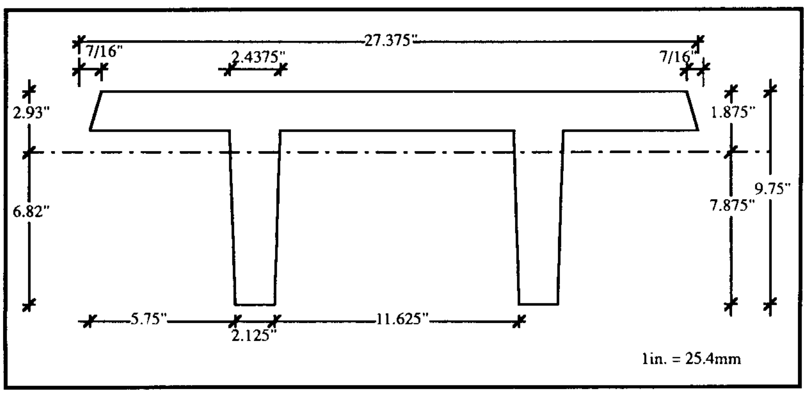

Fig. 1. Details of the model beam.

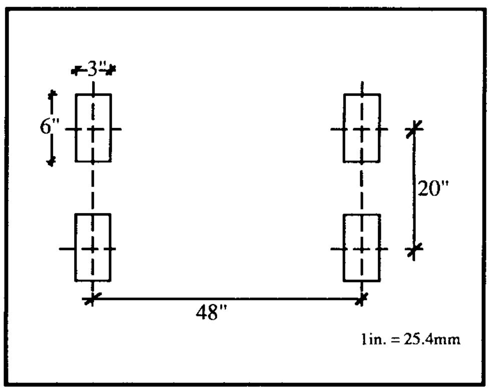

Fig. 2. Simulated truck contact loading areas.

DESIGN OF BRIDGE MODEL

Design Live Load

The 1:3.5 scale two-span continuous bridge deck was designed for a live load of HS 20-44 in which the variable spacing between axles was assumed to be 14 ft (4.27 m). The impact allowance is given by:

$$I = \frac{50}{L + 125} < 0.30 \qquad (1)$$

where

I = impact fraction

L = length in feet of portion of span loaded to produce maximum stress in member

Thus, for the prototype with a span of 70 ft (21.3 m), the computed impact factor was 0.26. For the model, the truck loading was simulated by scaling down the equivalent of the HS20-44 loading applied at four points as shown in Fig. 2. The simulated load of the HS20-44 truck was computed to be 7.41 kips (33 kN) including the impact factor.

Therefore, the model bridge was subjected to a maximum load of 8000 lb (35.6 kN) with the load ratio of 0.0625 in the frequency range of 3 to 4 Hz. Considering the lateral distribution of load on the transversely post-tensioned model bridge, the model beam was designed for a moving concentrated load of 3.5 kips (15.6 kN).

Table 1a. Section properties.

Parameter	Quantity
A	86.44 in.^{2}
I_g	681.0 in.^{4}
y_b	6.82 in.
y_t	2.93 in.
z_b	99.85 in.^{3}
z_t	232.42 in.^{3}
k_b	2.70 in.
k_t	1.16 in.

Metric (SI) conversion factors: 1 in. = 25.4 mm; 1 in.^{2} = 645 mm^2; 1 in.^{3} = 16387 mm^3; 1 in.^{4} = 416231 mm^4.

Table 1b. Concrete properties and allowable stresses.

Parameter	Stress, psi
f_c'	5000
$f_{ci}' = 0.8 f_c'$	4000
$\sigma_{ci} = 0.6 f_c'$	2400
$\sigma_{ti} = -3\sqrt{f_{ci}'}$	− 190
$\sigma_{cs} = 0.40 f_c'$	2000
$\sigma_{ts} = -6\sqrt{f_{ci}'}$	− 424

Metric (SI) conversion factor: 1 psi = 0.006895 MPa.

Double Tee Model

The double tee model has the section properties shown in Table 1.

Because of symmetry, only two critical sections were analyzed, i.e., the intermediate support Section B and the near midspan Section D taken at $3L/8$ from the left support. The maximum possible eccentricities at Sections D and B (Fig. 3a) are 4.80 and 1.875 in. (122 and 48 mm), respectively. The computed moments at these sections are tabulated in Table 2. The maximum and minimum moment envelopes were determined for the two-span continuous beam by combining the moments due to self weight and moving wheel loads.

Table 2. Moments at sections.

Parameter	Near midspan Section D in.-kips	Intermediate support Section B in.-kips
Distance from left support	$3L/8$ = 7.5 ft	L = 20 ft
Dead load moment	30.24	− 54.0
Live load moment	201.89	−121.68
Minimum moment	− 2.09	− 54.0
Maximum moment	203.56	−134.83

Metric (SI) conversion factor: 1 in.-kip = 113 N-m.

Prestressing Force and Ultimate Flexural Strength

The maximum stresses occurring at the top and bottom extreme fibers are required to be less than the allowable stresses at all times. The initial prestressing force, F_i, required for each stem of the double tee was computed to be 17.5 kips (78 kN) using the stress inequality conditions and the Magnel diagram. A seven-wire strand [7/16 in. (11 mm) diameter] with f_{pu} = 250 ksi (1724 MPa) was chosen to satisfy the above requirements.

Taking into account both the prestressed and nonprestressed steel, the nominal ultimate flexural strength was determined to be 385 in.-kips (43500 N-m). The level of post-tensioning in the transverse strands (Fig. 3a) was evaluated from the static tests performed on half-scale model beam tests.[2]

The test results indicated the transverse post-tensioning force in the exterior and interior strands to be 10,078 and 7,875 lb (45 and 35 kN). This force produces compressive stresses of 250 and 150 psi (1.7 and 1.0 MPa) in the concrete at the ends and in the interior portions of the model bridge deck, respectively.

Shear Resistance and End Zone Reinforcement

The maximum shear force envelope for the factored dead and wheel loads at different sections and the corresponding shear stresses were determined, and shear reinforcement provided in the regions based on the strength requirement. The transfer of the prestressing force in the double tees was by direct bearing through the anchorages.

The end zones were reinforced by a closely spaced grid of both horizontal and vertical bars. Seven D4 stirrups at 2 in. (51 mm) spacing were provided over a length of 13 in. (330 mm) in addition to those required for shear to account for the splitting tensile force.

Handling and Temperature Stresses

In each stem, two 3/8 in. (9.5 mm) prestressing strands stressed to 7440 lb (3375 kgf) were provided at depths of 1.5 and 8.25 in. (38 and 210 mm) from the top of the member. Their purpose was to dissipate handling stresses resulting from the removal of the member from the formwork. The depths were chosen so that the net effect of the two strand forces produced axial compression and zero moments about the centroid of the section.

In addition, a 4 x 4 – W 2.9 x 2.9 WWF mesh was provided in the flange to serve as temperature steel. Fig. 3b shows the reinforcement, tendon profile and details of the end bearing plates.[7,15]

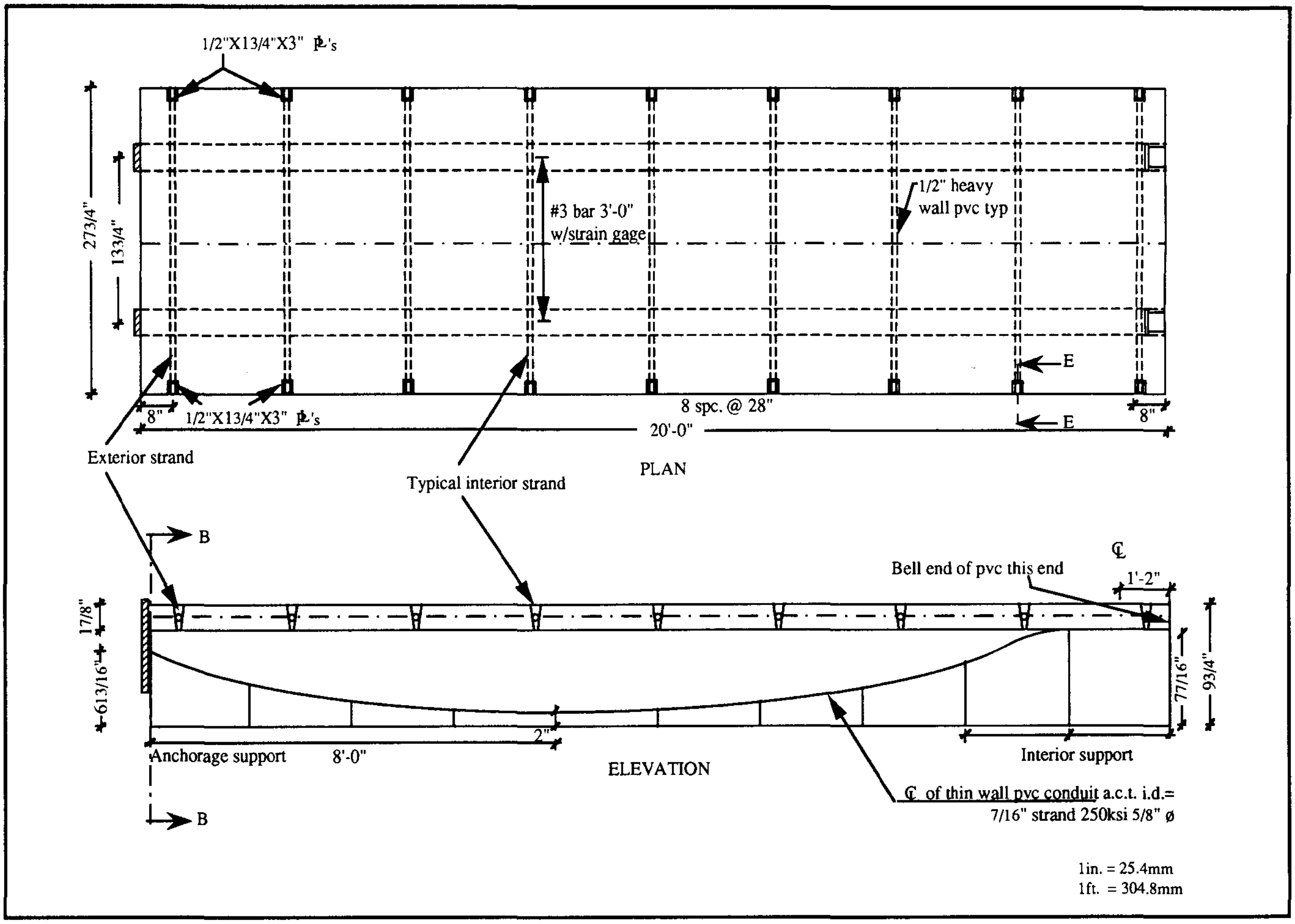

Fig. 3a. Details of the reinforcement and cable profile.

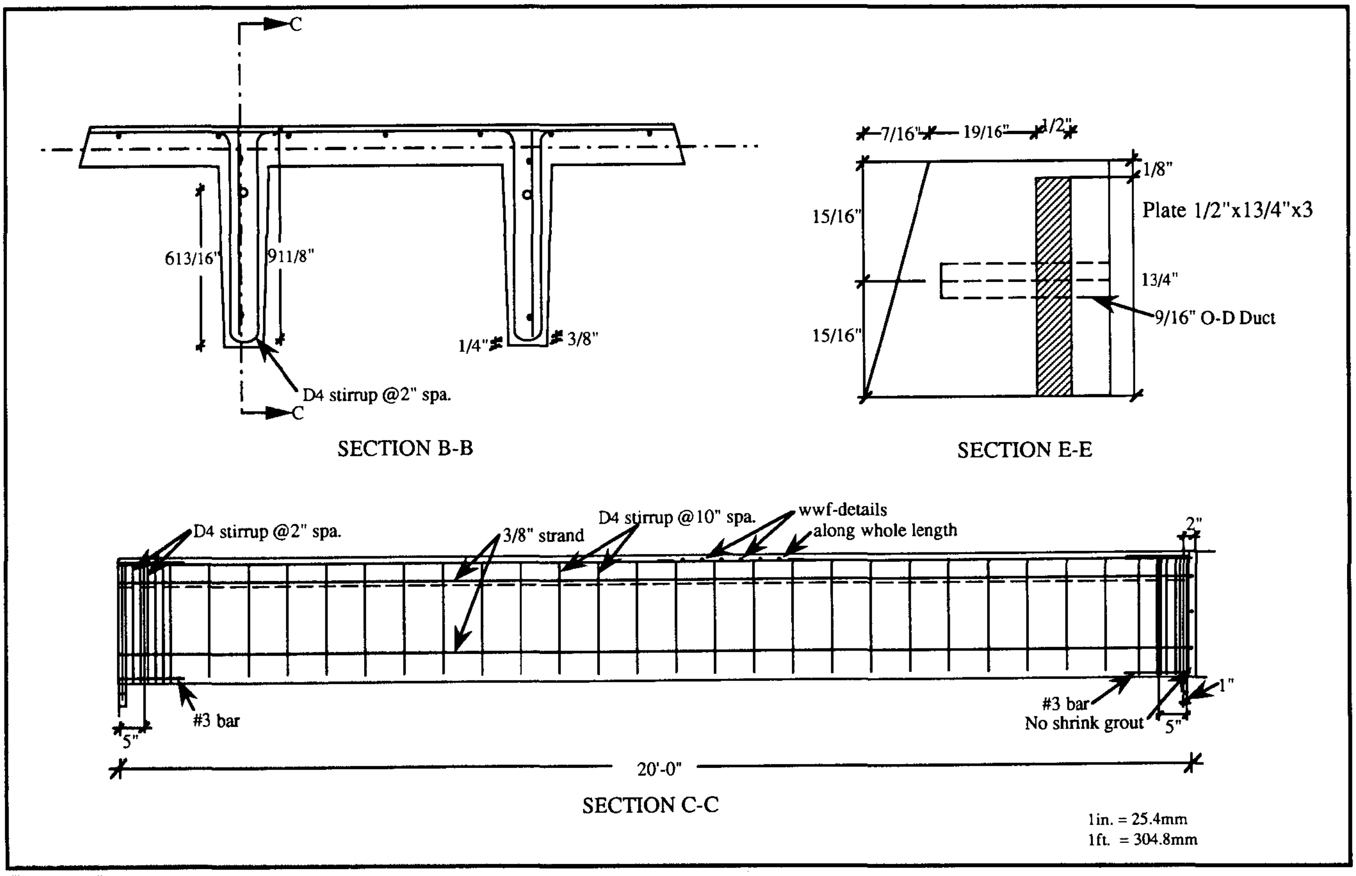

Fig. 3b. Fabrication details of the double tee beam and end bearing plate.

ANALYTICAL MODELING OF BRIDGE SYSTEM

The double tee bridge system was analyzed using a linear orthotropic model. The difference in the flexural rigidities in the two mutually perpendicular directions of the bridge system was taken into account. The virtual work method was used to predict the collapse load satisfying the plastic moment condition of the two-span bridge system.[8]

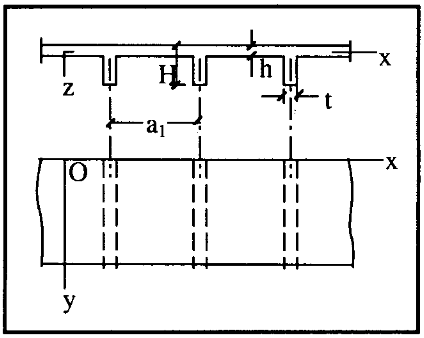

Fig. 4. Double tee bridge system idealization for computation of rigidities.

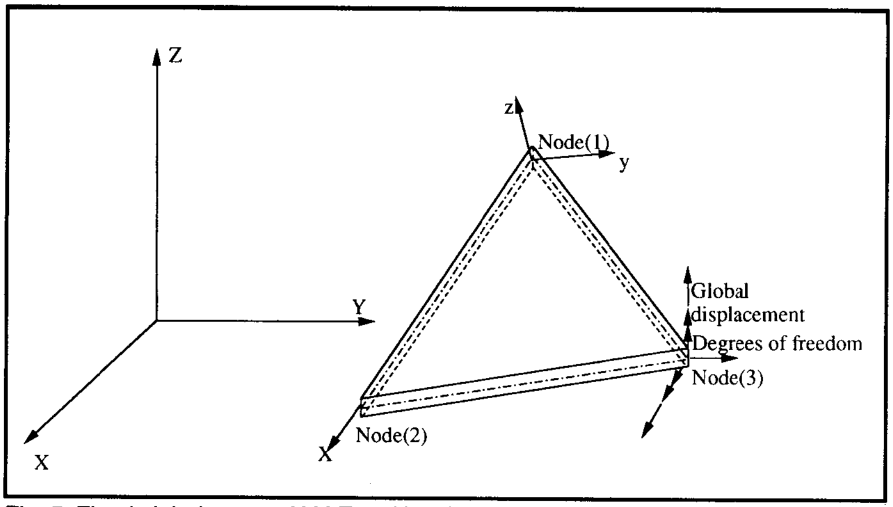

Fig. 5. Fixed global system X-Y-Z and local system x-y-z for plate element.

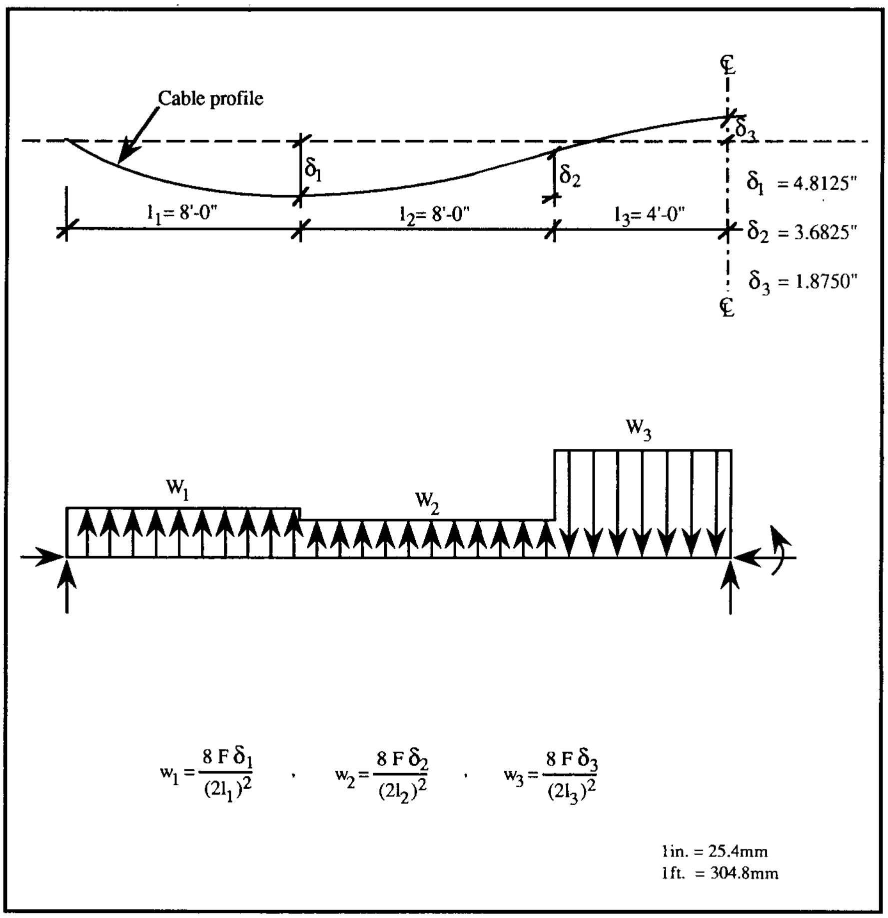

Fig. 6. Equivalent loading of the continuous beam.

Orthotropic Linear Elastic Model

The double tee bridge system was idealized as a slab reinforced by a set of equidistant ribs (Fig. 4). A finite element analysis, using the ADINA program,[10] was adopted to study the model.

In the program a three-node flat plate element (Fig. 5) was used. This element has six degrees of freedom per node corresponding to the global cartesian axes. Nodal lines were assumed along the beam longitudinal axes, the central transverse and longitudinal joints, load application points, and selected transverse post-tensioning strands. The idealized structure has a total of 280 plate elements and 165 nodal points with 737 degrees of freedom.

Materials Properties

The material constants were as follows:

E = 4.287 x 10^6 psi
(30 x 10^6 kPa)
ν = 0.2
D_x = 0.2919 x 10^7 lb-in.
(0.33 x 10^6 N-m)
D_1 = 0.0
D_y = 0.10665 x 10^9 lb-in.
(0.12 x 10^8 N-m)
D_{xy} = 0.2639 x 10^7 lb-in.
(0.30 x 10^6 N-m)

The rotation about the Z-axis was restrained at all nodal points. A hinge support was assumed at one end and a roller support at the interior and other end supports. The rotation about the Y-axis was also restrained at the nodal points on the rigid supports.

Applied Loads

The HS 20-44 highway loading on the bridge model was applied in the form of scaled down concentrated loads of 2000 lb (8900 N) at four nodal points. The prestressing forces

Table 3. Equivalent loads (Fig. 6).

Stem No.	Prestressing force, lb	w_1 lb/in.	w_2 lb/in.	w_3 lb/in.
1	13,000	13.576	10.403	21.158
2	12,650	13.211	10.403	20.589
3	14,700	15.352	11.764	23.926
4	10,250	10.705	8.202	16.683
5	11,200	11.697	8.963	18.229
6	10,600	11.07	8.483	17.253
7	10,550	11.018	8.443	17.171
8	12,350	12.898	9.883	20.101

Metric (SI) conversion factors: 1 lb = 4.448 N; 1 lb/in. = 0.175 N/mm.

in the transverse and longitudinal directions were specified as concentrated nodal forces. The prestressing forces, measured after 4 million cycles of fatigue loading, were used as input data in the analysis.

The effect of the change in tendon profile in the stems is to produce a transverse vertical force on the concrete member. Fig. 6 shows the equivalent loads for the tendon profile and Table 3 summarizes the calculated equivalent loads in each of the stems. The equivalent loads were applied as concentrated loads at the corresponding nodal points.

The deflections in the model were measured with respect to the initial profile of the system after post-tensioning. The measured values were compared with the computed results for two load cases:

(a) Prestress force together with equivalent loads; and

(b) Simulated HS 20-44 loading together with prestress forces and equivalent loads.

The actual deflections were obtained by appropriate addition of the computed values for the two load cases.

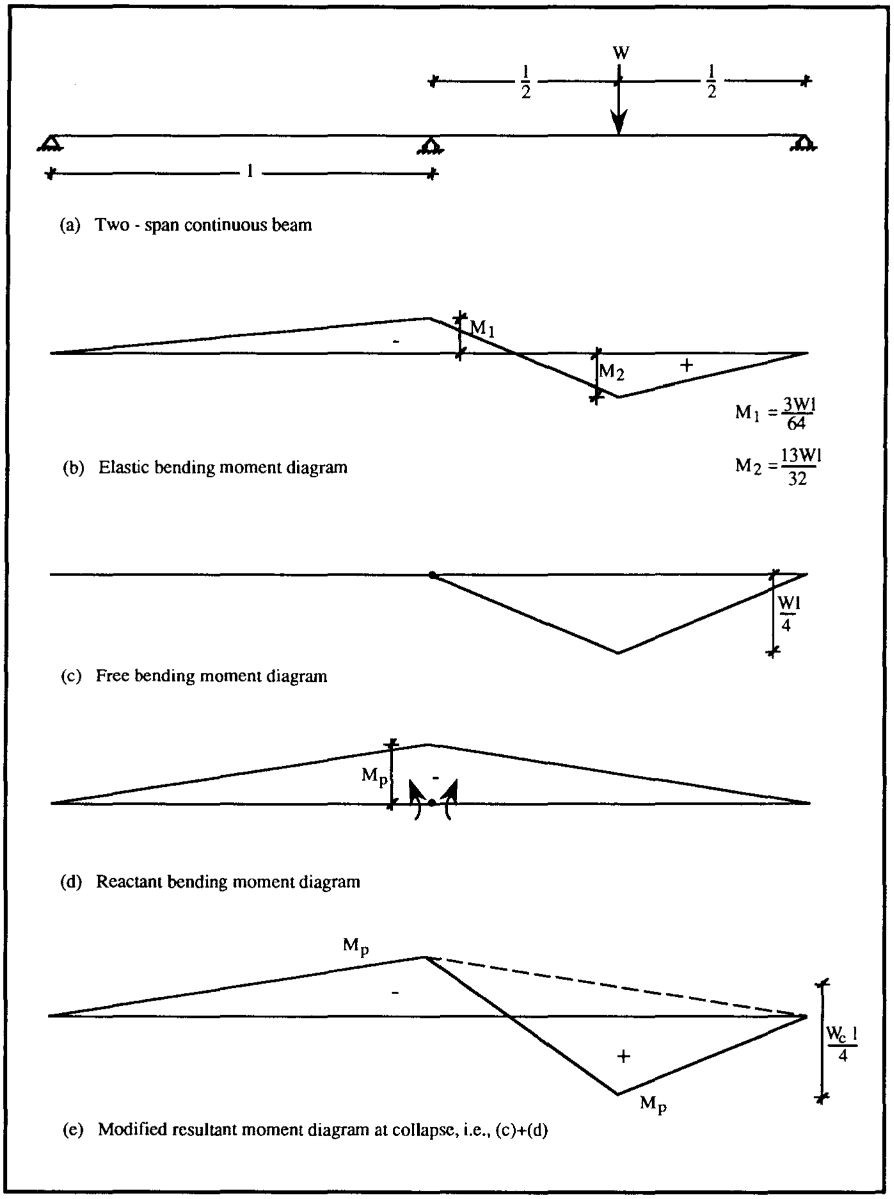

Fig. 7. Equilibrium diagram.

Collapse Load

Fig. 7 shows the equilibrium diagram for the bridge system idealized as a two-span structure. The collapse load, W_c, is obtained by considering the total ordinate of the free moment diagram at the plastic hinge position as follows:

$$W_c = \frac{6M_P}{l} \quad (2)$$

EXPERIMENTAL PROGRAM

Concrete Mix

The concrete mix design specified by the Florida Department of Transportation was used for casting the model beams. Table 4 shows the materials per cubic yard for a design compressive strength of 5000 psi (35 MPa) concrete. The fineness moduli of fine and coarse aggregates were determined to be 2.34 and 5.59, respectively.

The compression tests on 6 x 12 in. (152 x 305 mm) cylinders, conducted at ages of 26 to 90 days, showed compressive strength values ranging from 5060 to 7780 psi (35 to 54 MPa). The 3 x 6 in. (76 x 152 mm) grout cylinders had compressive strengths in the range of 5235 to 6900 psi (36 to 48 MPa) whereas the 2 x 2 x 2 in. (51 x 51 x 51 mm) grout cubes showed compressive strengths of 1250 psi (after 20 hours) and 5250 psi (after 40 hours) [8.6 and 36 MPa]. The post-tensioning was carried out 44 hours after grouting the joints.

Table 4. Summary of mix proportions.

Materials	Materials per cubic yard (saturated surface dry aggregates)	Absolute volume
Cement (lb)	658	3.36
Fly ash (lb)	—	—
Coarse aggregate (lb)	1815	1.5
Fine aggregate (lb)	1132	6.909
Admixture (oz)	33	0
Air entraining agent (oz)	1	0.81
Water (gal)	32.39	4.32
Water (lb)	270	0

Metric (SI) conversion factors: 1 cu yd = 0.7646 m^3; 1 lb = 4.448 N; 1 oz = 0.278 N; 1 gal = 3.785 l.

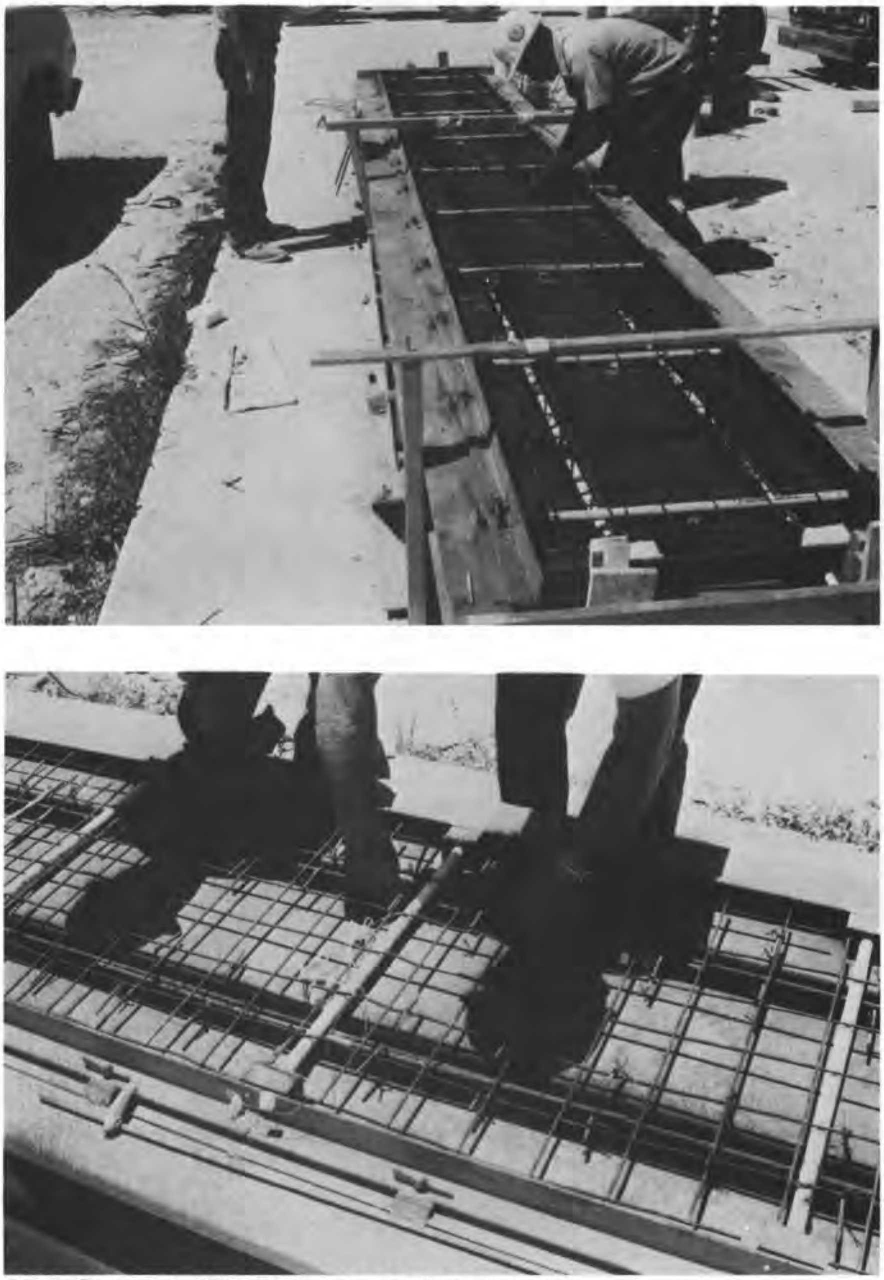

Fig. 8. Formwork of the test beam and reinforcement.

Double Tee Beam Specimens and Loading Procedure

The test specimens were fabricated at Southern Prestress, Inc. From there, they were transported to Florida Atlantic University. Fig. 8 shows the steel formwork used in casting the double tees. The beams were removed from the steel molds 18 hours after they were cast.

PVC pipes were used to provide the longitudinal tendon profiles in all the stems. Transverse duct holes were provided in the beams at intervals of 28 in. (711 mm) along the entire length.

Fig. 9 gives a schematic representation of the test setup showing the major dimensions.

The 50 ft (15 m) long HP beams were positioned on two large steel sawhorses to provide the reaction force to the actuator loading. These beams are held in place by transverse 16 ft (4.9 m) long I-beams at the ends and tied to the sawhorses by tie rods. The model bridge system was supported on a specially designed reinforced masonry wall and footing. The cyclic loading was applied at selected locations by positioning the electromechanically controlled 55 kip MTS actuator that was mounted on the I-beams.

All the test specimens were placed on neoprene bearing pads resting on the support walls. The double tees were tied together by transverse post-tensioning. The V joints between the double tees were filled with mortar (Fig. 10a). A transverse joint of 2 in. (51 mm) was inserted between the beams at the intermediate support. Here, stirrups were placed and later filled with mortar (Figs. 10b and 10c). This prevented point contact and subsequent local crushing of the concrete.

A hand operated hydraulic jack was used to apply the prestressing force which was monitored by a load cell and load meter. Instrumentation was set up for measuring strains, deflections, and crack widths. EA strain gages with a 120 ohm resistance and a gage factor of 2.05 were fixed on 3 ft (0.91 m) long members with #3 bars and placed in the concrete specimens

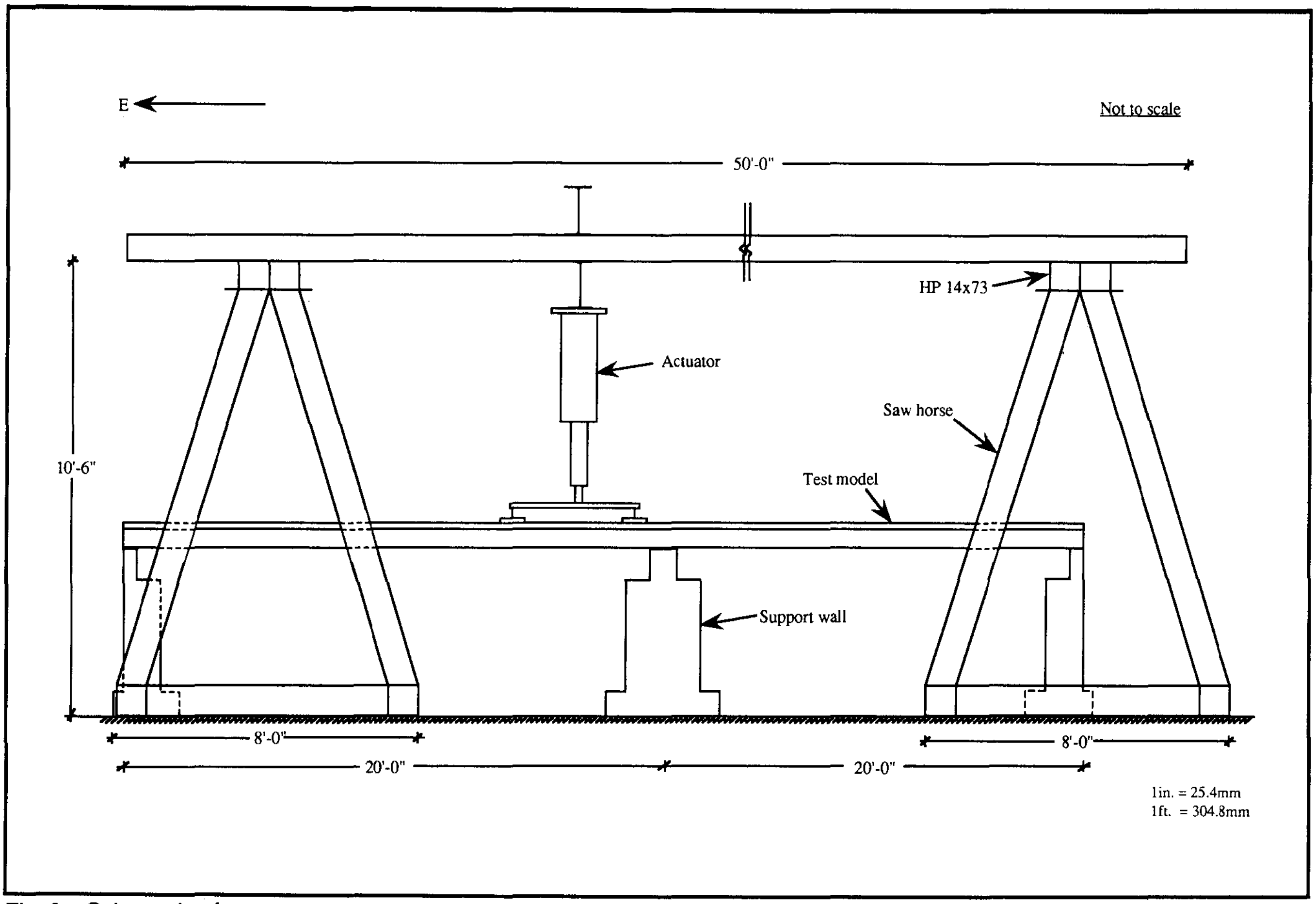

Fig. 9a. Schematic of test setup.

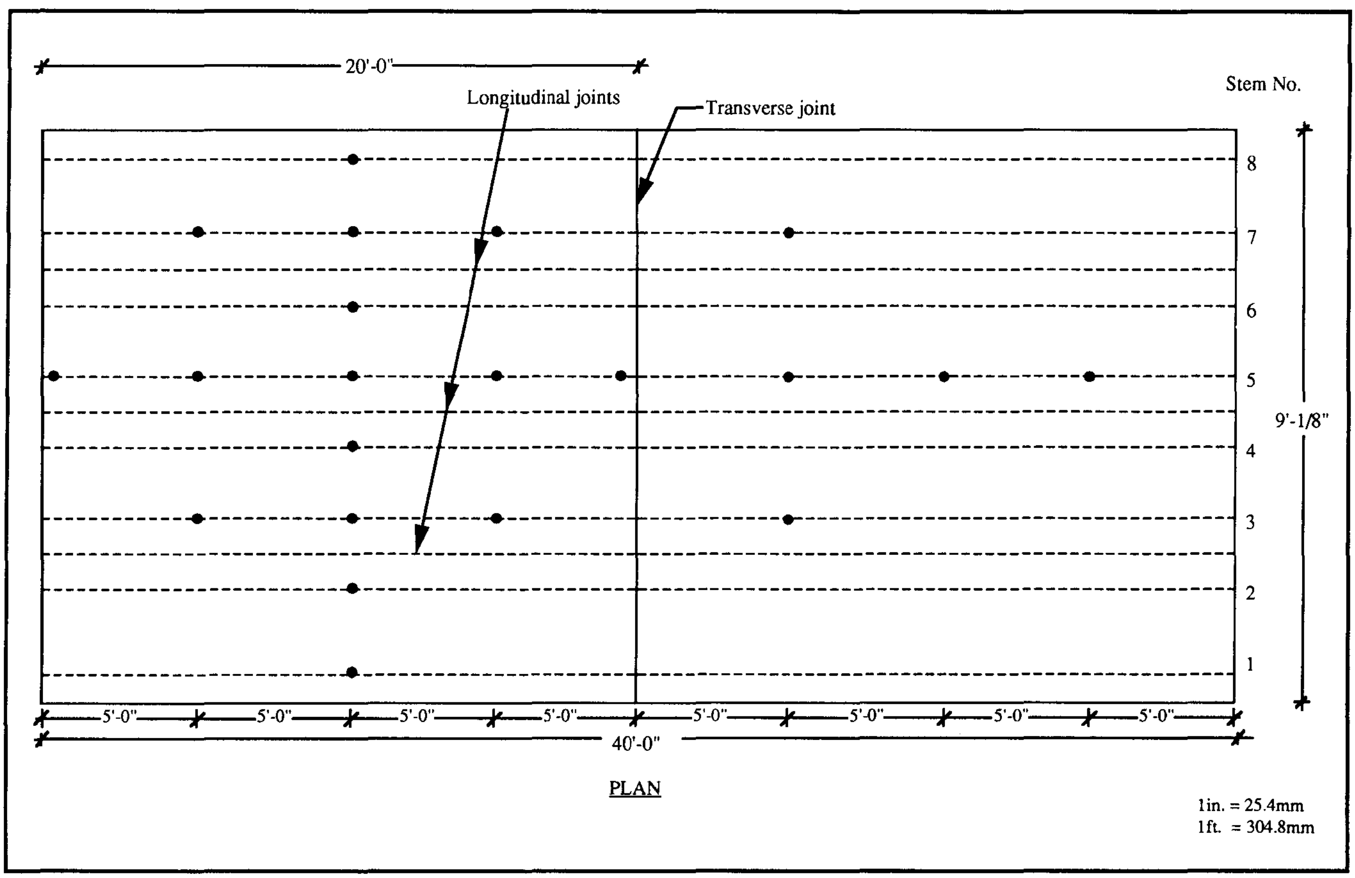

Fig. 9b. Dial gage locations for a typical load position (Load Position 4).

Fig. 10a. Embedded end plates and the V joint between beams.

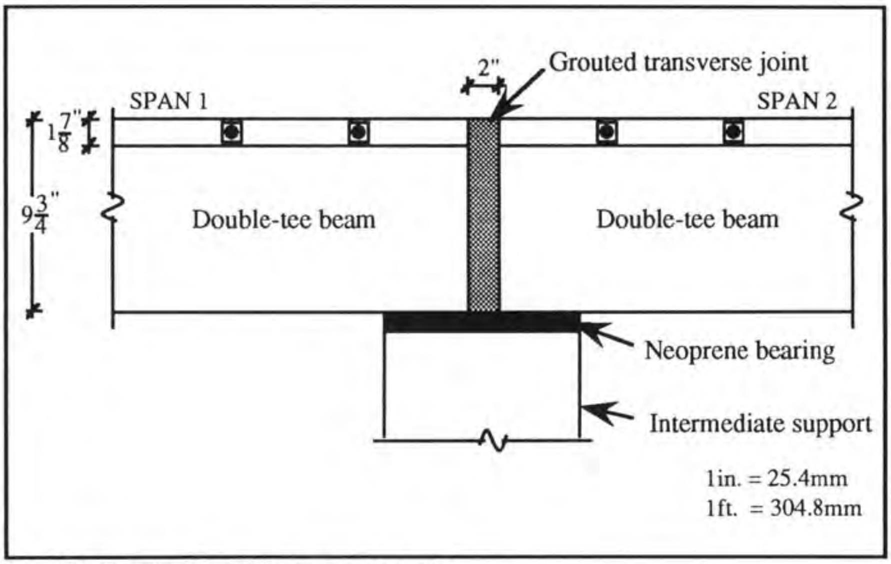

Fig. 10b. Details of transverse grouted joint.

Fig. 10c. Transverse joint.

at midspan and near the ends of the beam.

To prevent the strain gages from being damaged, they were covered with bitumen and enclosed with aluminum foil. Forty reinforcing bars were instrumented and each bar was calibrated for load vs. strain. Deflections of the stems were measured with mechanical dial gages. Fig. 11 shows the four fatigue test load positions and the ultimate collapse load test location.

For each load position, the model bridge was subjected to a cyclic loading at a frequency varying between 3.2 Hz and 4.0 Hz for 2 million cycles. The maximum and minimum loads were 8000 lb and 500 lb (3630 and 227 kgf), respectively. The loads were monitored with an MTS 406 controller. Deflections and strain measurements were taken for a maximum static load of 8000 lb (3630 kgf) after 100,000, 250,000, 500,000, 750,000, 1.0 x 10^6, 1.25 x 10^6, 1.5 x 10^6, and 2 x 10^6 cycles. The joint crack width was measured with an LVDT. The static load was applied at increments of 1000 lb up to 8000 lb (454 to 3630 kgf). Deflections were measured at each load increment.

The transverse grouted joint between the double tees over the interior support developed cracks at the bottom over the neoprene bearing pads during fatigue loading in the test position 2 at the end of 744,000 cycles. These cracks were pressure grouted a few times during testing. The amplitudes of oscillations at midspan of Stem 4 (typical) decreased after grouting the transverse joint. The measurement of post-tensioning forces indicated considerable prestress loss possibly due to strand slip in both transverse and longitudinal directions. Hence, the test for load position 2 was considered as invalid.

The model bridge system was subjected to ultimate loading after completion of fatigue loading at the end of 8 million cycles. The load-deflection plot (Fig. 12) indicates the ductile behavior of the double tee bridge model. The ultimate collapse load, deflections, crack width, and crack patterns were observed, and the experimental collapse load compared with the predicted value.

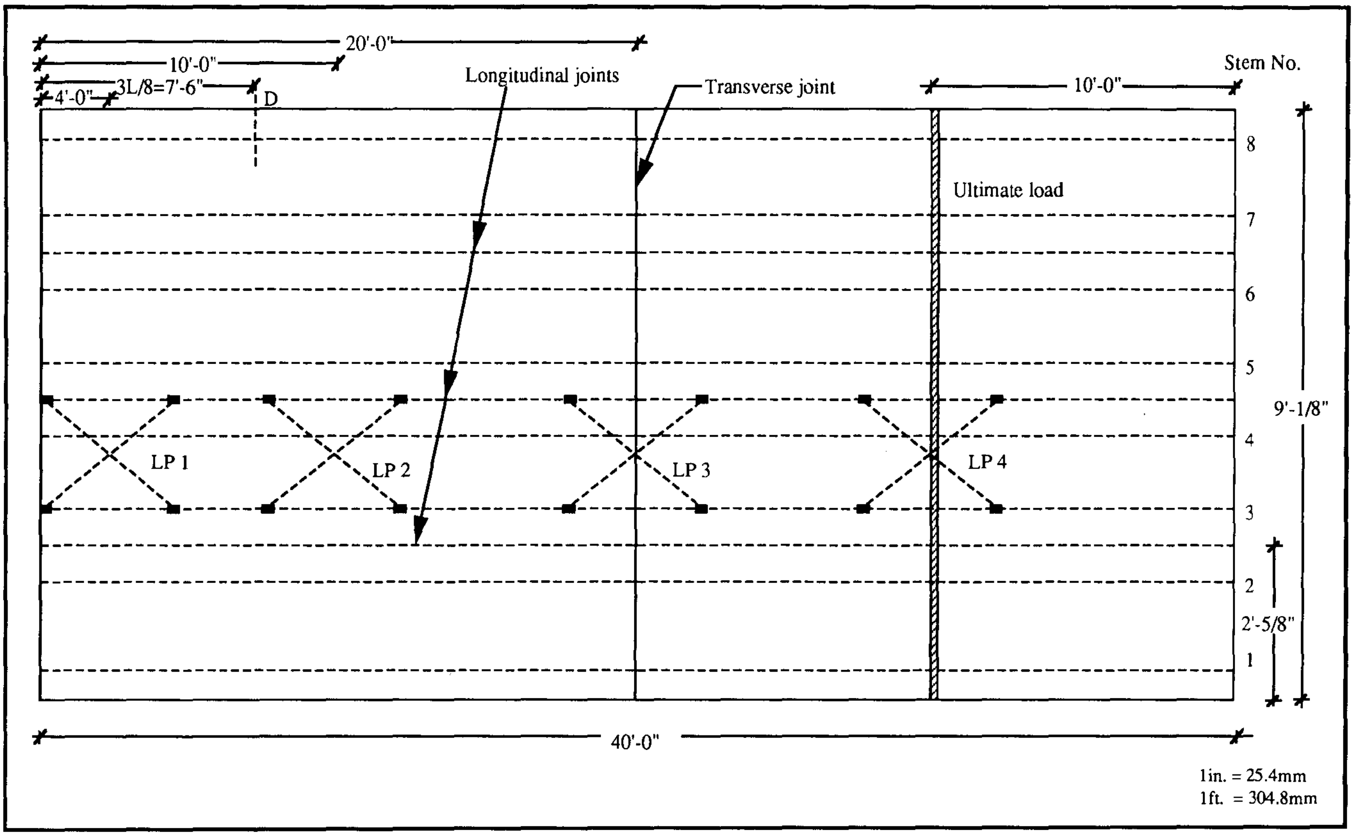

Fig. 11. Location of loading device, Load Positions 1 to 4 and ultimate load.

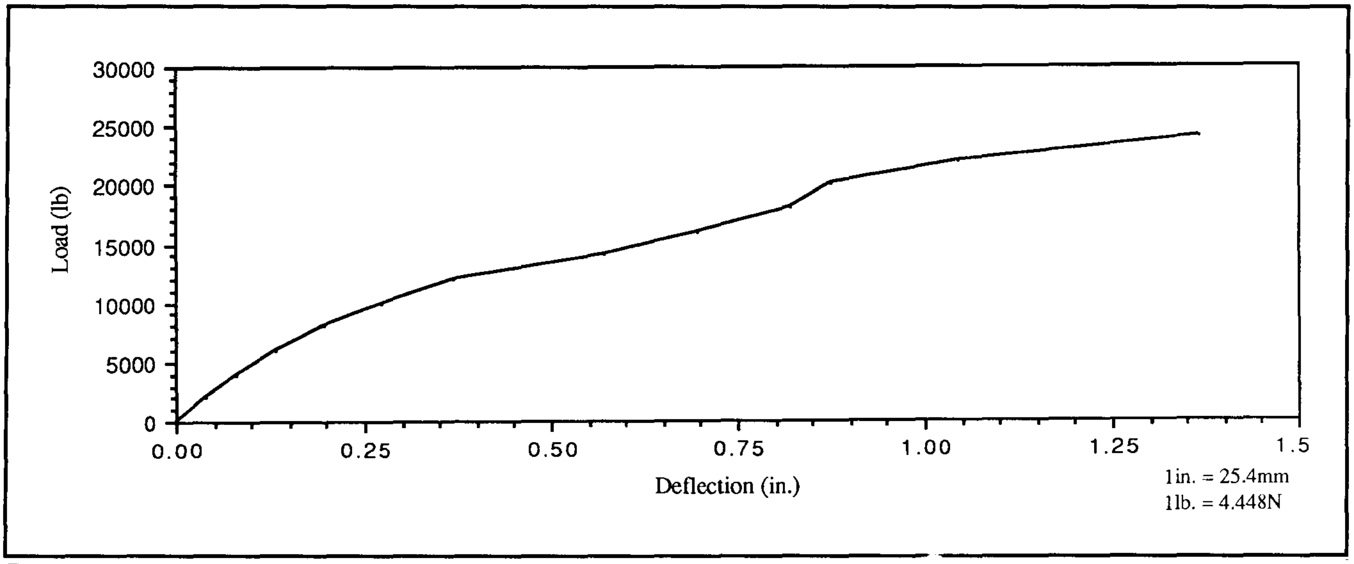

Fig. 12. Ultimate load test showing load vs. deflection.

RESULTS AND DISCUSSIONS

The primary objectives of the investigation were to evaluate:

(a) The longitudinal V joint behavior under fatigue loading;

(b) Local punching shear strength of the deck slab;

(c) Load carrying capacity, ductility, distribution, and size of flexural cracks; and

(d) Ultimate load behavior and capacity of the double tee bridge system for a given level of post-tensioning in the transverse direction.

Longitudinal V Joint Fatigue Behavior

The double tee bridge system was designed with minimum levels of post-tensioning in the longitudinal and transverse directions to obtain monolithic behavior under service loads. The fully precast beams were tied together by a simple V joint and transverse post-tensioning. The longitudinal joint was grouted with non-shrink grout.

The longitudinal joint between Stems 4 and 5 below the wheel load position exhibited cracking for Load

Case 1 at 80,000 cycles. The extent of cracking was localized at the exterior concentrated load region and the observed crack width was 0.005 in. (0.13 mm). No evidence of cracking was observed in any of the longitudinal joints under wheel loads for the Load Case 3 even after 2 million cycles of loading. However, Load Case 4 was more severe in causing localized cracking in the longitudinal joint.

This cracking was not visible on the top face of the deck. Nevertheless, it was observed at the bottom surface (with a magnifying glass) between Stems 4 and 5. The observed crack width was 0.005 in. (0.13 mm). The crack widths measured at the end of different stages of fatigue loading were found to be in the range of 0.003 to 0.004 in. (0.076 to 0.10 mm) which were well below the permissible crack widths shown in Table 5. The initiation of the localized longitudinal crack in Load Case 1 could be attributed to shear caused by the severity of the concentrated wheel loads acting at the edge of the double tee.

Shear Strength[11-13]

Shear is generally not critical when the deck slabs carry distributed loads or line loads because in such cases the maximum shear force per unit length of deck is relatively small. However, it can be critical in the vicinity of concentrated loads.

In the present investigation, the truck loading on the bridge system was simulated by scaling down the equivalent HS 20-44 loading. For the 1:3.5 scale model, the simulated load was computed to be 7.41 kips (3360 kgf) including the impact factor. Therefore, both a maximum and minimum load of 8000 and 500 lb (3630 and 230 kgf) were applied cyclically at a frequency range of 3 to 4 Hz.

The shear reinforcement provided in the double tees followed ACI 318-83 and AASHTO specifications. The 1⅞ in. (48 mm) thick deck slab did not exhibit any punching shear cracks/failure at any of the simulated wheel load positions for all the test load cases. A single layer of flange mesh reinforcement (4 x 4 W2 – 0 x W2 – 0 WWF), furnished as temperature steel in the deck slab, improved the flexural behavior and resistance to vertical loads.

Load Carrying Capacity, Ductility, and Spacing of Flexural Cracks

Typical displacements measured on stems for different fatigue load positions were plotted and the trend examined with respect to increasing number of cycles. The trends were estimated using the least squares approach. The lines satisfying the least squares criterion are shown in Fig. 13.

Typical values of measured deflections for Load Position 4 are compared with those obtained from orthotropic linear elastic models in Fig. 14. The predicted values from orthotropic modeling agree reasonably well with the measured values along the loaded Stem 4.

Fig. 15 shows the crack growth, spacing and pattern over the depths of stems. It can be observed from the experimental data that there is a gradual increase in crack width only in the initial stages, after which stable behavior is observed with only marginal increases in crack width. It is also clear that a further increase in the number of loading cycles has no significant effect on crack width. It was noticed that the cracks remained visible even after unloading in the static test.

Ultimate Load Behavior

The ultimate collapse load of 34.65 kips (15720 kgf) was computed idealizing the bridge system as a two-span continuous beam using the equilibrium method. The computed value agrees very closely with the observed collapse load of 35.5 kips (16100 kgf). The actual collapse load might have been higher if the post-tensioning strands were grouted.

The unbonded condition prevented the strands from developing higher forces when the section rotated, due to the lack of compatibility of strains between the strand and the concrete. Visible cracks initiated above a load of 15,000 lb (6800 kgf) and crack widths increased almost linearly with increase in the line load. The maximum observed deflection was limited to 3.25 in. (83 mm) since the loading had to be discontinued due to the limitations of the loading frame. The variation in crack width over the interior support was found to be linear with increasing load.

Fig. 16 shows the spacing of cracks near the region of the applied line load. The overall behavior of the model bridge satisfied the requirements of load carrying capacity, ductility, spacing and size of flexural cracks, deflection, and general lack of cracking due to shear.

Table 5. Typical values of maximum allowable and measued crack widths.[14]

Source	Exposure condition	Allowable crack width (static loading) in.
ACI Committee 224	Interior exposure (dry air, protective membrane)	0.016
BS—5400 Bridge Code (1978)	Exposure: Severe	0.0078
	Very severe	0.0039
Abeles	Air or protective membrane	
	(a) Cracking not permitted under dead load	0.012
	(b) Cracking permitted under dead load	0.010
Present study on double tee bridge system	Fatigue loading (Load Position 4)	Measured crack widths (in.)
	No. of cycles:	
	509,200	0.003
	750,000	0.003
	1,250,000	0.003
	2,000,000	0.004

Metric (SI) conversion factor: 1 in. = 25.4 mm.

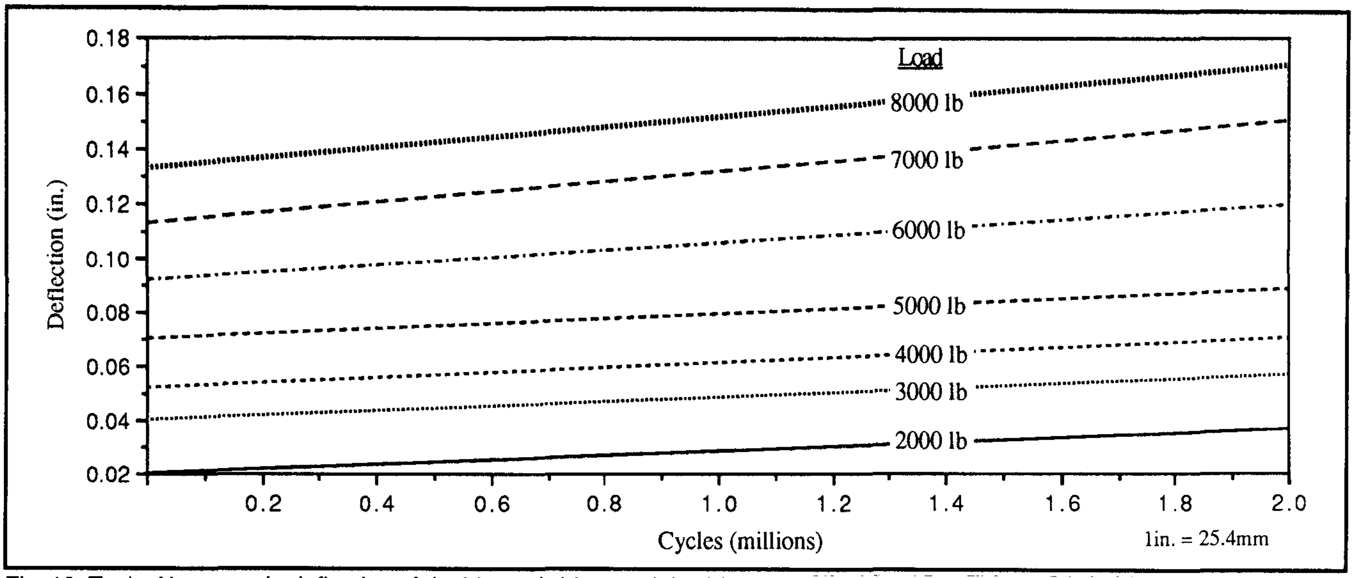

Fig. 13. Typical increase in deflection of double tee bridge model subjected to constant amplitude cyclic loading.

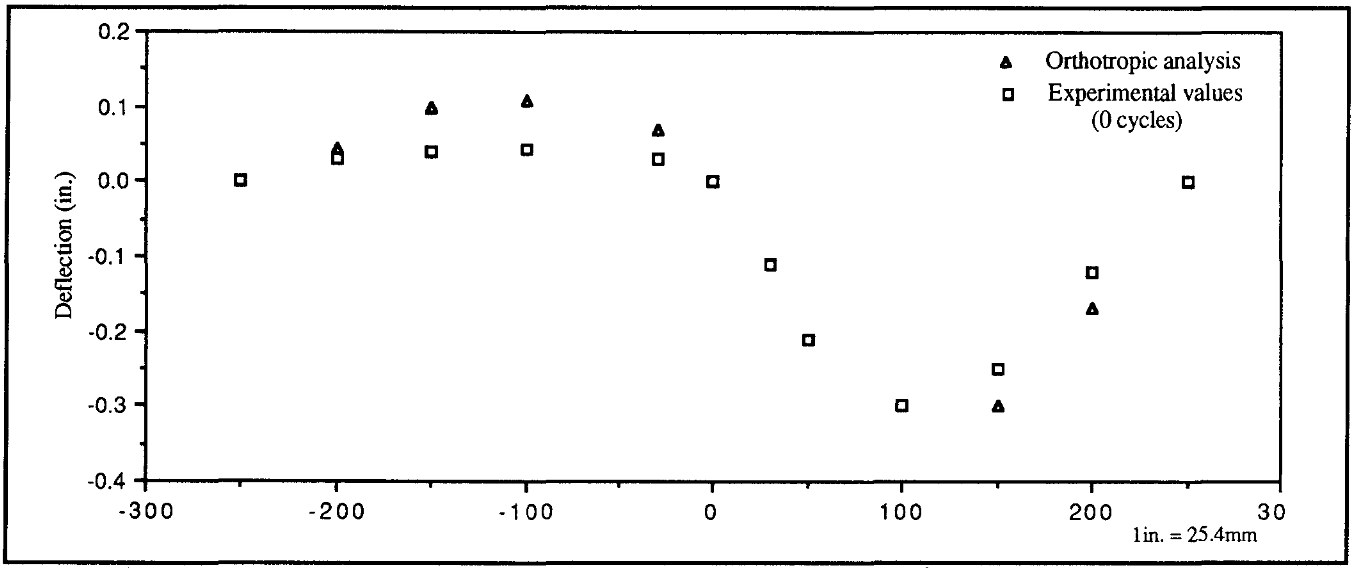

Fig. 14a. Comparison of computed and measured deflections (longitudinal section) along Stem 4, Load Position 4.

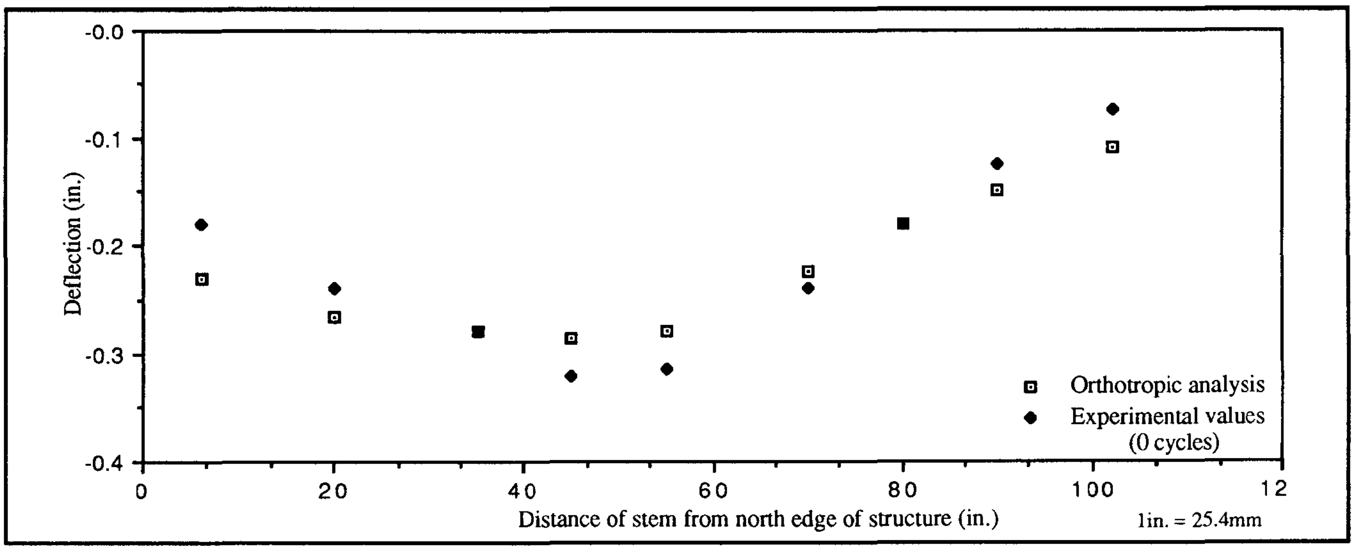

Fig. 14b. Comparison of computed and measured deflections (transverse section)
at 10 ft (3.05 m) from west support, Load Position 4.

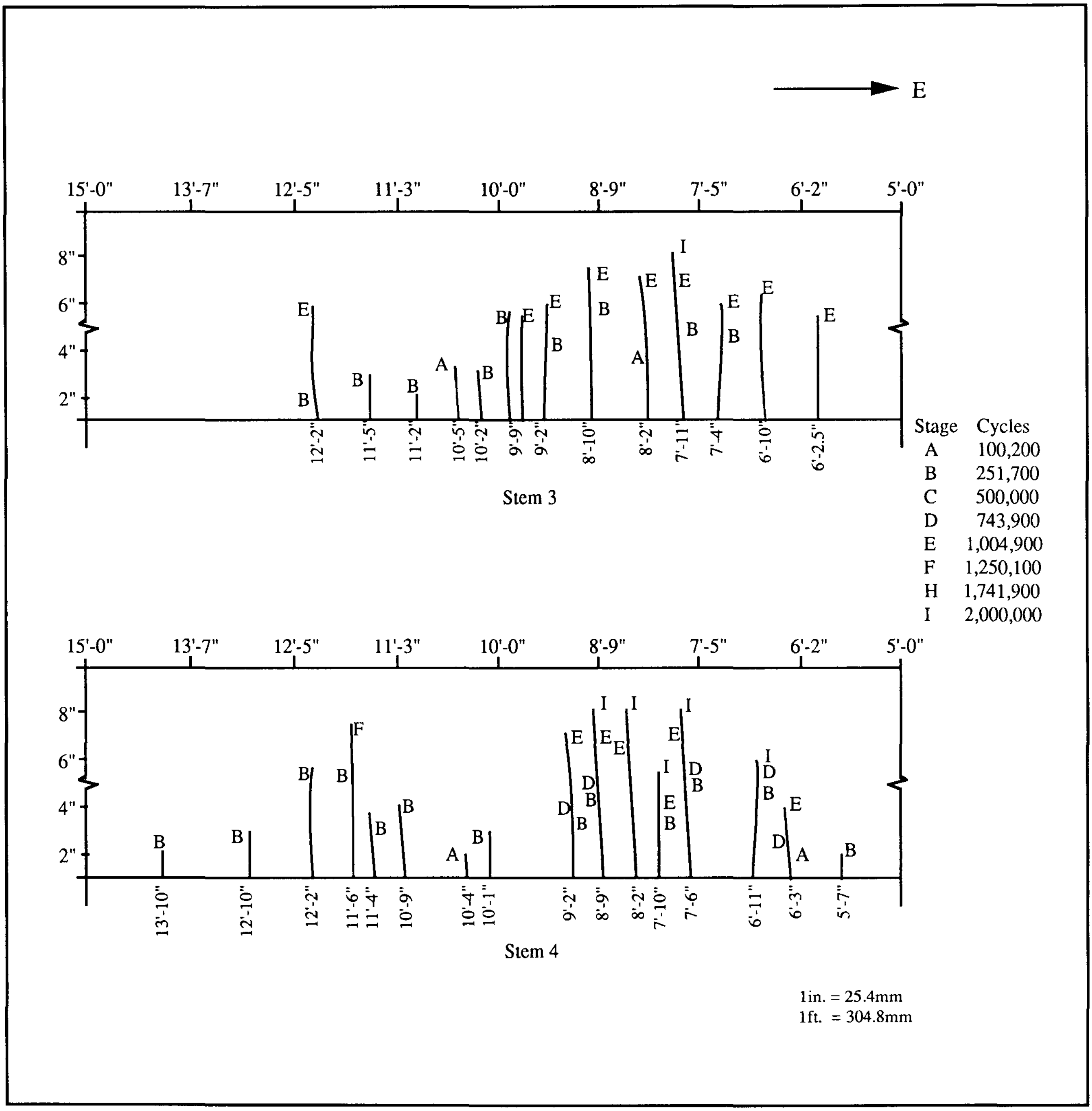

Fig. 15. Schematic representation of cracks, Stems 3 and 4, Load Position 4.

CONCLUSIONS

The following conclusions are drawn based on the experimental and analytical investigation of the precast, prestressed, double tee beam bridge system.

1. The double tee bridge system, assembled with post-tensioning in both the longitudinal and transverse directions, showed monolithic behavior under both static and fatigue loading conditions.

2. Even after 8 million cycles of constant amplitude fatigue loading, the deflection increase was about 30 percent. However, no appreciable crack propagation or increase in strand stresses was observed indicating there was no significant loss in flexural stiffness. This shows that the overall behavior of the system was satisfactory from a serviceability and strength viewpoint.

3. The behavior of longitudinal joints under static and fatigue loading was satisfactory as the bridge maintained its structural integrity even after 8 million cycles. Localized cracking was observed at a particular location between Stems 4 and 5 below the wheel load positions. However, the measured crack width was less than the allowable value required by various codes of practice.

4. No visible cracking was observed at the transverse joint over the tensile zone above the interior support for typical wheel load positions, even after 8 million cycles of fatigue loading, indicating adequacy of the prestressing force.

5. The deck slab with a thickness of 1⅞ in. (47.5 mm) behaved very well under a simulated wheel load of

8000 lb (35.6 kN) without showing any punching shear failure around any of the wheel load positions for all the test load cases.

6. The increase in the number of cycles of fatigue loading had no significant effect on the crack width. However, the cracks remained visible even after unloading during the static test.

Fig. 16. Crack pattern at ultimate load.

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