CRACK ANALYSIS OF PRESTRESSED CONCRETE U-BEAMS FOR CROSS FLORIDA GREENWAY LAND BRIDGE

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ABSTRACT

Prestressed U-beam bridges compare favorably in cost and appearance to traditional concrete I-beam bridges. Consequently, U-beam bridges are gaining in popularity and usage especially when aesthetic issues are deemed important. U-beam bridges first appeared in Florida during 2000; however, during construction, cracks developed in the webs of the U-beams. The objective of this paper is to publish the results of an analysis of representative cracking of U-beams. For the purpose of the analysis, the U-beam is divided into a series of finite shell-plate elements and the prestressing tendons are simulated as a number of concentrated forces. Two different mechanical models of the U-beams are developed based on the stages of construction. Analytical results show that high tensile stresses occur in the end zone of the U-beam due to the prestressing tendons and that these tensile stress need to be properly considered in bridge design.

Keywords: Prestressed U-beam Bridges; 3-D Finite Element Modeling; Crack Analysis; Tensile Stresses Induced by Prestressing forces; Transfer Zone Stirrup Design.

INTRODUCTION

To offer aesthetically pleasing alternatives for bridge design, yet maintain the economy of prestressed beams manufactured under controlled plant conditions, a new type of concrete beam, called U-beam after the U-shape of its cross section, was developed in 1993 by the Texas Department of Transportation. This type of bridge beam has favorable aesthetic attributes and is cost competitive with other concrete beams [1]. As a result, U-beams are appearing more frequently in bridges throughout the United States especially when aesthetic issues are deemed important. During 2000, Florida built its first U-beam bridge, the Cross Florida Greenway Land Bridge, to carry pedestrians, equestrians and wildlife over heavily traveled I-75. Cracks developed in the webs of the U-beams during the construction of the bridge.

The objective of this investigation is to identify the causes of the cracks and propose a modified anchorage zone stirrup arrangement design method. First, brief descriptions of the bridge and U-beams are given. Then, mechanical models are depicted. Finally, analytical results and design recommendations are discussed.

DESCRIPTION OF BRIDGE

The Cross Florida Greenway Land Bridge [2] is located in Marion County, Florida. The bridge consists of two simple spans measuring 102.3 ft (31.2 m) in length, for an overall bridge length of 204.6 ft (62.4 m). Figure 1 shows the elevation view of the bridge structure. The cross section of the bridge superstructure consists of four precast-prestressed Florida U-beams with a cast-in-place concrete deck of 7.87 in (200 mm) thick, as shown in Figure 2. The design live-load considerations for the bridge structure included AASHTO [3] HS 15 to account for pedestrian loads. The three center-to-center spacings of Florida U-beam lines are 12.14 ft (3.7 m), 15.42 ft (4.7 m), and 12.14 ft (3.7 m). The overhangs from the centerline of the outside girder to the edge of the deck are 6.20 ft (1.95 m), for an overall bridge width of 52.5 ft (16 m). The width of the bridge accommodates a trail section that measures 16.4 ft (5 m) in width, with planting areas that measure 18.05 ft (5.5 m) in width on either side (see Figure 2). The planting areas are soil-filled sections bounded by retaining walls and landscaped with materials consistent with natural approach vegetation. The trail surface is compacted shell material.

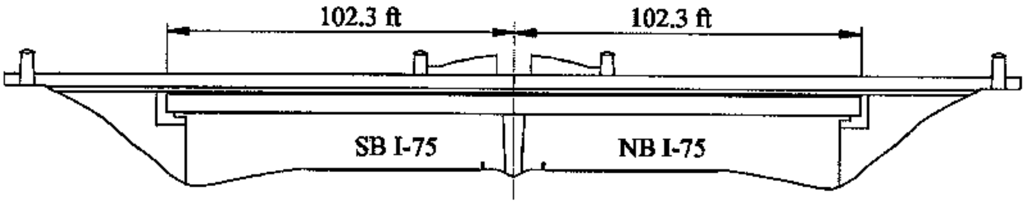


Fig. 1. Bridge Elevation

The precast Florida U-beam is a 5.9 ft (1.8-m) deep precast prestressed concrete tub-shaped girder with inclined webs. The bottom soffit of the U-shaped girder is 4.59 ft (1.4 m) wide, and

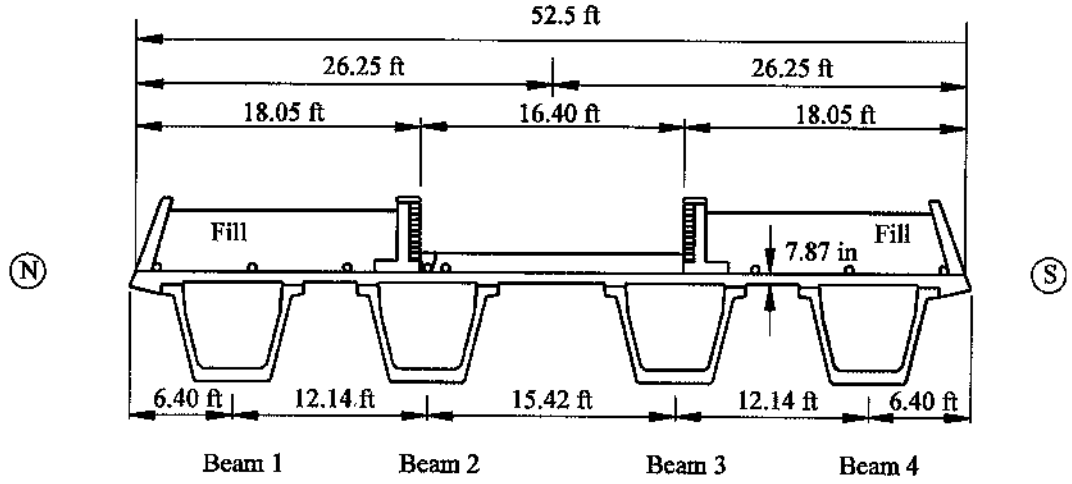


Fig. 2. Typical Cross Section

the inclined webs are each 5.12 in (13 cm) thick with #5 stirrups spaced at 6 inches center to center. Each web is topped by a 1.33 ft (0.405 m) wide flange (see Figure 3). There are two end diaphragms and two intermediate diaphragms in each U-beam (see Figure 3a). The design strength of the concrete for the precast Florida U-beam is 8.5 ksi (59 MPa).

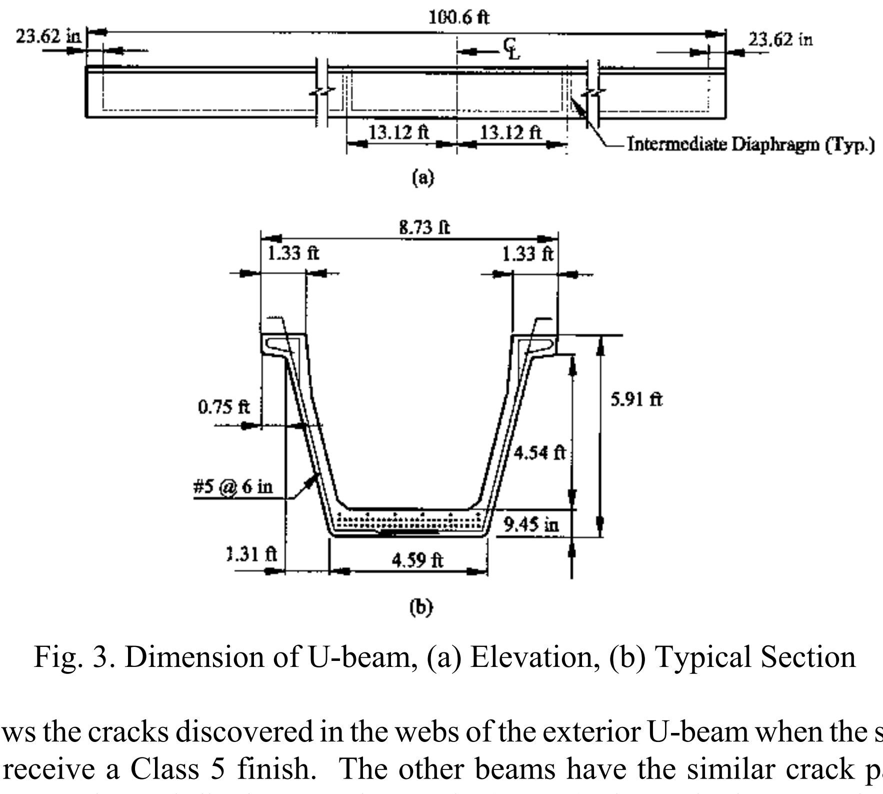


Fig. 3. Dimension of U-beam, (a) Elevation, (b) Typical Section

Figure 4 shows the cracks discovered in the webs of the exterior U-beam when the surfaces were prepared to receive a Class 5 finish. The other beams have the similar crack patterns. The cracks begin near the end diaphragm, about 8 in (20 cm) above the bottom of the beam and extend upward at an angle ranging from 30 to 45 degrees. At some locations there is only one crack; at others there are multiple cracks. Some cracks extend up to 9 in (23 cm) while others extend close to the top flange. Crack width varies from hairline to about 0.012 in (0.3 mm).

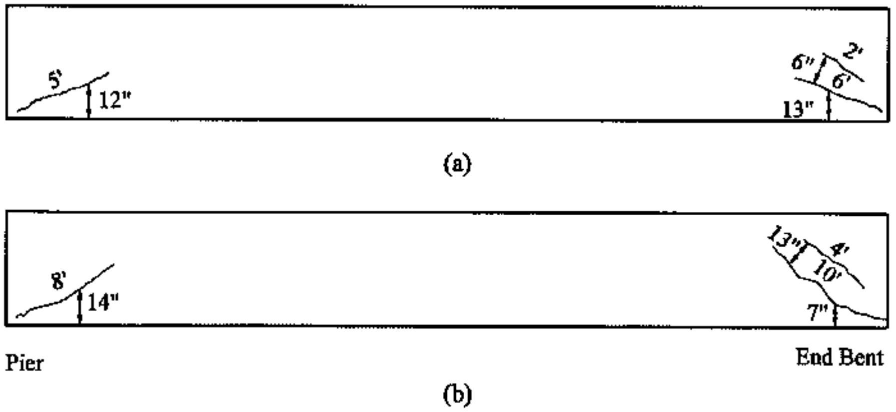


Fig. 4. Typical Cracks of Exterior U-beam (Beam 4) Over I-75 South Bound Span, (a) South Web, (b) North Web

MECHANICAL MODELS OF BRIDGE AND U-BEAMS

Consistent with bridge construction procedures, two mechanical models with two different loading stages were developed and are shown in Figure 5. The first model is an open thin-wall structure (see Figure 5a) that simulates the first loading stage. The first stage loadings consist of the prestressing tendons, the U-beam self-weight, and the self-weights of the deck and stay-in-place metal forms. The second model is a closed box girder (see Figure 5b) that supports the superimposed dead loads of the fill and barriers. To simplify the analysis of the second loading stage, the bridge is first treated as a plan grid model (see Figure 6) to determine the distributed load of each girder due to the fill and the barriers. Then, the distributed load is applied to the closed box girder model (see Figure 5b). The U-beam and box girder are divided into a number of quadrilateral shell plate elements (see Figures 7 and 8) and analyzed by the finite element method. There are a total of 150 elements in the beam longitudinal direction and 32 elements in the transverse direction for the U-beam model and 50 elements for the box girder model. The width and length of each shell element is about 8 in (20 cm) and the aspect ratio is approximately unity, except for the diaphragm elements, the mechanical behaviors of which are not a concern of this investigation. The total number of finite elements is 4840 for the U-beam model and 7580 for the box girder model. Both the U-beam and box girder are assumed homogeneous and elastic in the analysis.

PRESTRESSING FORCE MODEL

The strand pattern and de-bonding schedule of the prestressing tendons are illustrated in

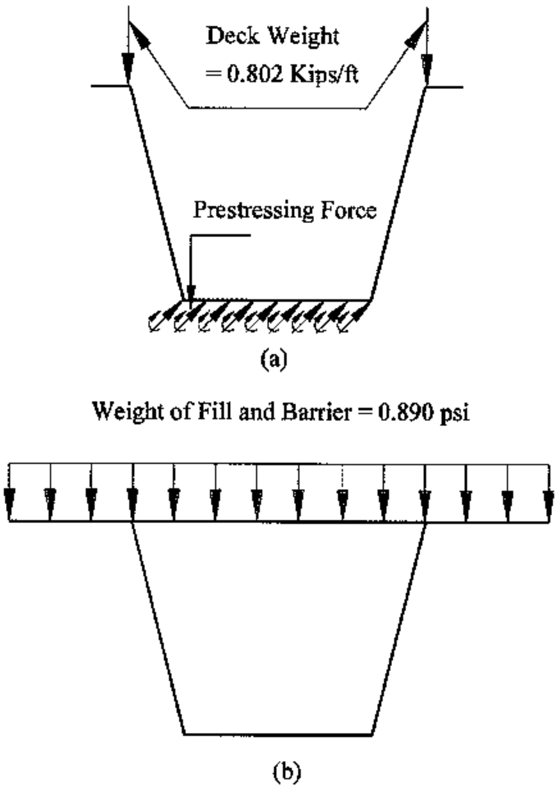


Fig. 5. Loading Stages, (a) First Loading Stage, (b) Second Loading Stage

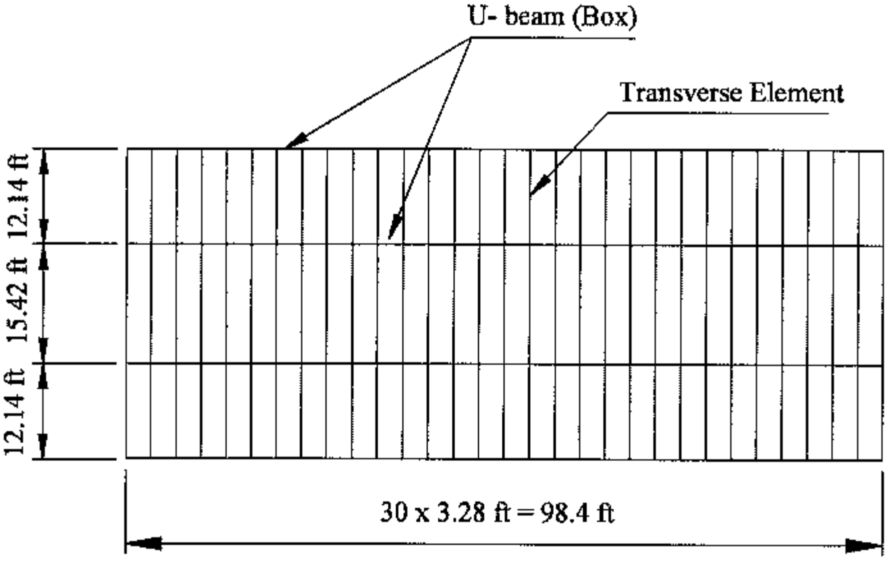


Fig. 6. Bridge Model

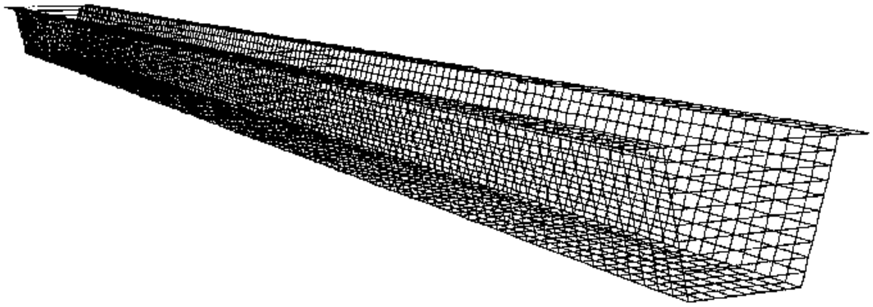


Fig. 7. U-beam Model for First Loading Stage

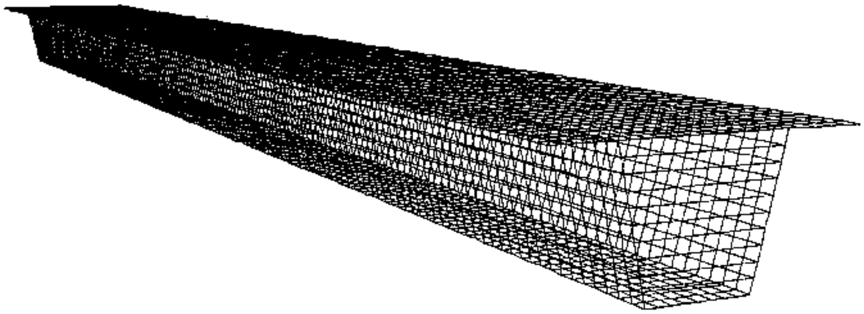


Fig. 8. U-beam Model for Second Loading Stage

Figure 9. The prestressing strands are Grade 270, ϕ 0.6, low-relaxation, and stressed to 44 kips each. The prestressing force is simulated as a number of concentrated forces and moments applied at the half-depth of the bottom slab. According to the de-bonding schedule shown in Figure 9, the forces and moments are separated into five groups which represent the prestressing forces of full- bonded strands and the strands to be de-bonded at 9.84 ft. (3 m), 14.76 ft (4.5 m), 19.96 ft. (6.0 m), and 24.61 ft (7.5 m), individually. The transfer length of the strands is assumed to be 60 strand diameters (LRFD [4] Article 5.11.4.1). The prestressing force of the strands is assumed to vary linearly from zero at the point where bounding commences to a maximum at the transfer length and applied at the related nodes (see Figure 10). The prestressing force model is illustrated in Figures 5 and 10. From Figure 10, it can be seen that the prestressing forces of the full-bonded strands are divided into five concentrated forces and five concentrated moments along the U-beam longitudinal direction. Each of the concentrated forces and moments is further divided into 9 concentrated forces or moments along the transverse direction (see Figure 5a). The magnitude of prestressing force of strands is determined by assuming an effective stress of 160.0 ksi (1.10 Mpa).

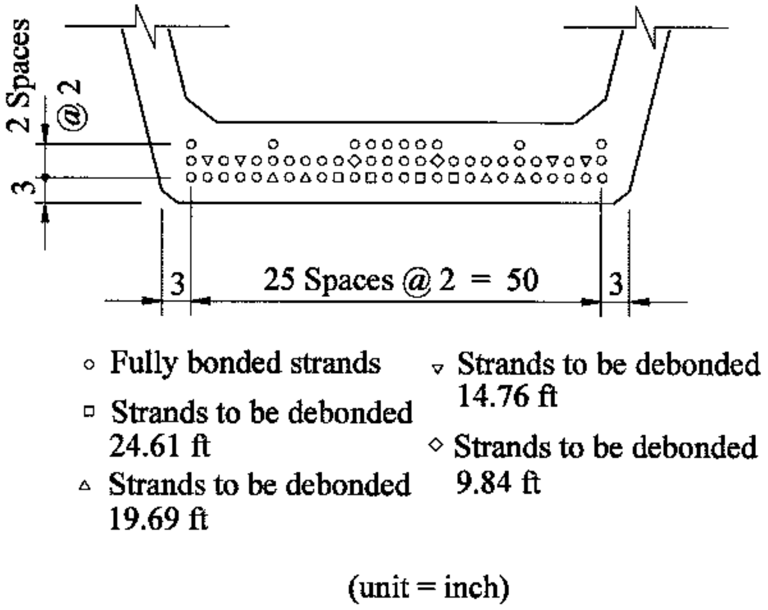


Fig. 9. Strand Pattern and Schedule

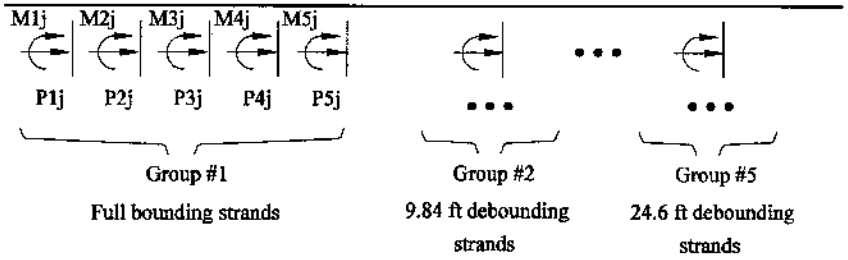
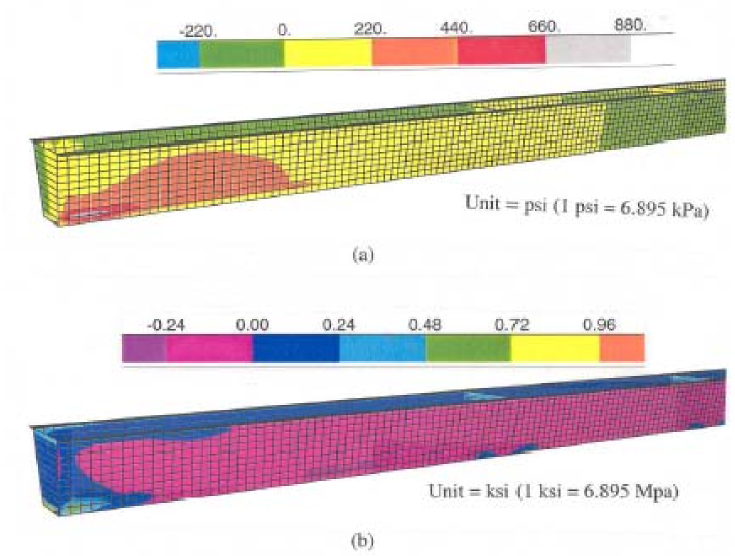


Fig. 10. Prestressing Force Model in Longitudinal Direction

RESULTS

The exterior U-beam was analyzed based on the mechanical models described above. The exterior U-beam is subjected to two line loads of 0.802 kips/ft (11.7 kN/m) due to the concrete deck and stay-in-place metal forms applied along the both sides on the top of the U-beam (see Figure 5a). The dead load due to the fill and barriers distributed to the exterior beam is assumed to be a uniform and was determined as 0.892 psi (6.137 kN/m^2), based on the grid model shown in Figure 6. The shear stress and tensile stress distributions for the first loading stage are shown in Figures 11a and 11b, respectively. From these figures, a high concentration of stresses can be observed in the end zone of the U-beam near the bottom of the web. The maximum shear stress and tensile stress are about 820 psi (5.66 Mpa) and 720 psi (4.97 Mpa), respectively. Figure 12 shows the distributions of shear stress and tensile stress due to the soil fill and barriers in the second loading stage. It can be seen from this figure that the maximum shear stress and tensile stress at the end of U-beam near the bottom of the web are about 145 psi (1.0 Mpa) and 155 psi

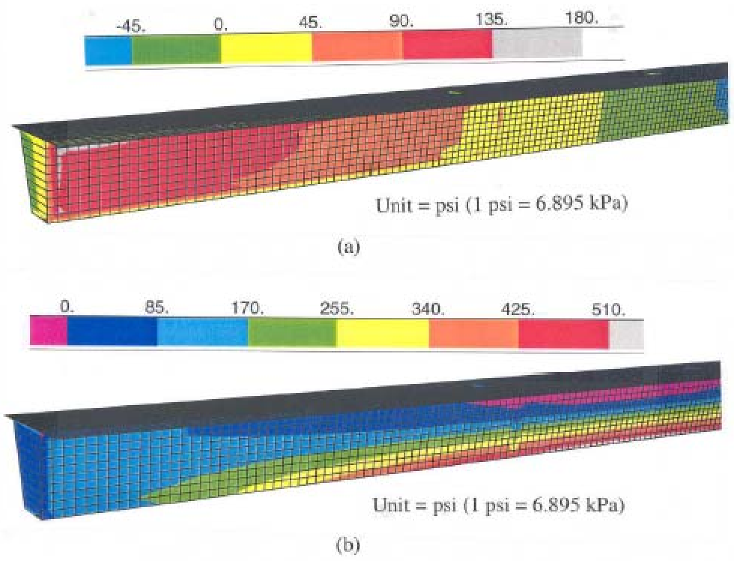
(1.07 Mpa), individually. The total maximum tensile stress near the U-beam end, approximately 870 psi (6.0 Mpa), is large enough to cause cracking. It should be noted that this analysis has not considered the effect of the construction load and that the effective prestressing force is estimated based on an approximate pre-stress loss value suggested by AASHTO Specifications. Tensile stresses higher than the analytical results may be expected during construction.



(a)

(b)

Fig.11. Stress Distribution at First Loading Stage, (a) Shear Stress Distribution, (b) Maximum Tensile Stress Distribution.



(a)

(b)

Fig. 12. Stress Distribution at Second Loading Stage, (a) Shear Stress Distribution, (b) Maximum Tensile Stress Distribution.

Figure 13 shows the distribution of shear stress induced by the prestressing forces. It can be seen from this figure that the prestressing forces cause a very high concentration of shear stress in the end zone of the U-beam and that the maximum shear stress is about 560 psi (3.9 Mpa). Table 1 gives the principal tensile stresses and their directions due to different types of loadings at Node No. 2 located in Figure 14a. From this table, observe that the maximum tensile stress caused by the prestressing force is nearly the half of the total tensile stress. The direction of the resultant maximum tensile stress is about 45 degrees corresponding well with the observed angles of the cracks.

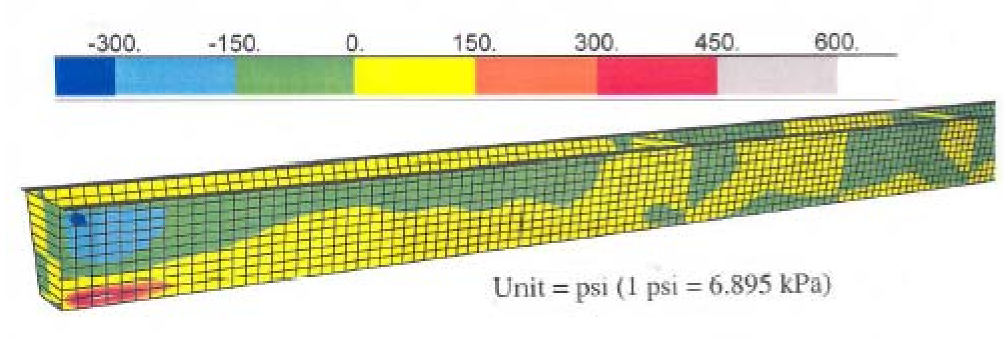


Fig. 13. Shear Stress Distribution Induced by Prestressing Force

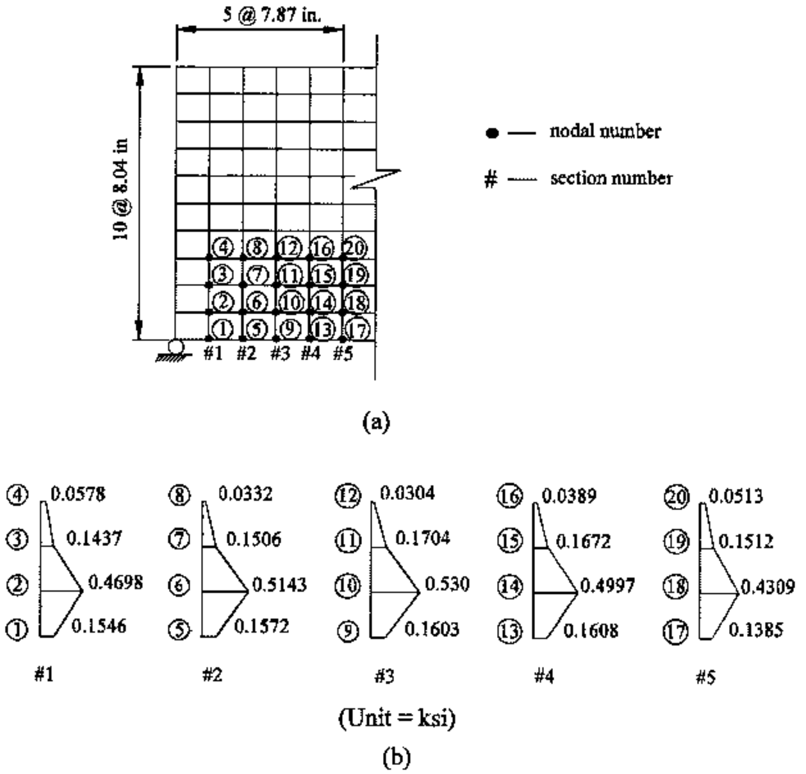


Fig. 14. Distribution of Shear Stress induced by Prestressing Forces, (a) Nodal and Section Numbering, (b) Distribution of Shear Stress

Table 1. Typical Principal Stress in End Zone due to Different Loads

Node Number	U-beam Self-weight		Prestressing Force		Deck Self-weight		Fill and Barriers	
	Magnitude (psi)	Direction (Degree)	Magnitude (psi)	Direction (Degree)	Magnitude (psi)	Direction (Degree)	Magnitude (psi)	Direction (Degree)
2	130.8	40.54^0	404.2	51.19^0	143.8	40.37^0	155.3	39.91^0

To further confirm the magnitudes of the shear stresses due to the prestressing forces in the end zone of the U-beam, five cross-sections in the end zone were investigated. The locations of these cross-sections are illustrated in Figure 14a. Figure 14b demonstrates the shear stress distributions for the five cross-sections along the vertical direction of the web. The variation of the shear stress between two nodes is assumed to be linear. The total positive shear force (downward) in each cross-section is equal to the shear stress area (see Figure 14b) multiplied by the web thickness and is provided in Table 2. To illustrate how the longitudinal prestressing force relates to the vertical shear force, Table 2 gives the ratios of the vertical shear force to the total prestressing force. From this table, it can be seen that the total downward shear force is about 4% of the total longitudinal prestressing force. It is interesting to note that this percentage is similar to that of the AASHTO Standard Specification [3] Article 9.22.1, which says that vertical stirrups acting at a unit stress of 20.0 ksi (137.9 Mpa) to resist at least 4 percent of the total prestressing force shall be placed within the distance of one fourth of the height of the pretensioned beam from its end. AASHTO LRFD [4] Article 5.10.10.1 is roughly equivalent to this provision. However, both AASHTO Standard and LRFD Specifications have not clearly addressed whether more stirrups are required to resist the shear due to the design live and dead loads and how this provision was determined.

Table 2. Vertical Shear Induced by Prestressing Force

Item	Section Number				
	1	2	3	4	5
Total Prestressing Force, N_{ps} (kips)	1650.7	1650.7	1650.7	1650.7	1650.7
Vertical Shear Q_{ps} (kips)	59.2	62.6	65.5	63.1	55.7
Q_{ps}/N_{ps} * 100%	3.59%	3.79%	3.97%	3.82%	3.38%

In 1962, Marshall and Mattock [5] performed a significant experimental investigation of vertical tensile stress induced by longitudinal prestressing tendons for precast prestressed concrete I-girders. Twenty-five I-girder specimens with girder heights ranging from 22.5 in to 25.0 in (57 cm to 64 cm) were tested. They found: (1) maximum vertical tensile stress occurs at the end face of the girder and closes to zero at a distance of about one quarter the girder depth from the end face; (2) total vertical tension force increases with the ratio of girder height (h) to transfer length (lt) and is about 2.53% and 3.44% of the total prestressing force for h/lt =1.63 and 1.76 respectively; and (3) their proposed vertical stirrup design equation does not consider the effect of the shear force induced by prestressing forces. The authors believe that the current AASHTO Specification is based on Marshall and Mattock's research results and is only meant to control horizontal cracks of prestressed beams. To evaluate their test results, the U-beam was reanalyzed with diaphragms removed. The vertical tensile stress distribution is illustrated in Figure 15a. It can be observed from this figure that very high tensile stresses occur at the end zone. Figure 16 demonstrates the web average vertical tensile stress distributions for five horizontal sections whose locations and numbering are shown in Figure 16a. Assuming that the vertical stresses between two nodes vary linearly, the total vertical tensile forces were calculated and given in Table 3. From this table, it can be seen that the maximum tensile force occurs at about two-fifths of the girder depth from the bottom and is 4.85% of the total prestressing force.

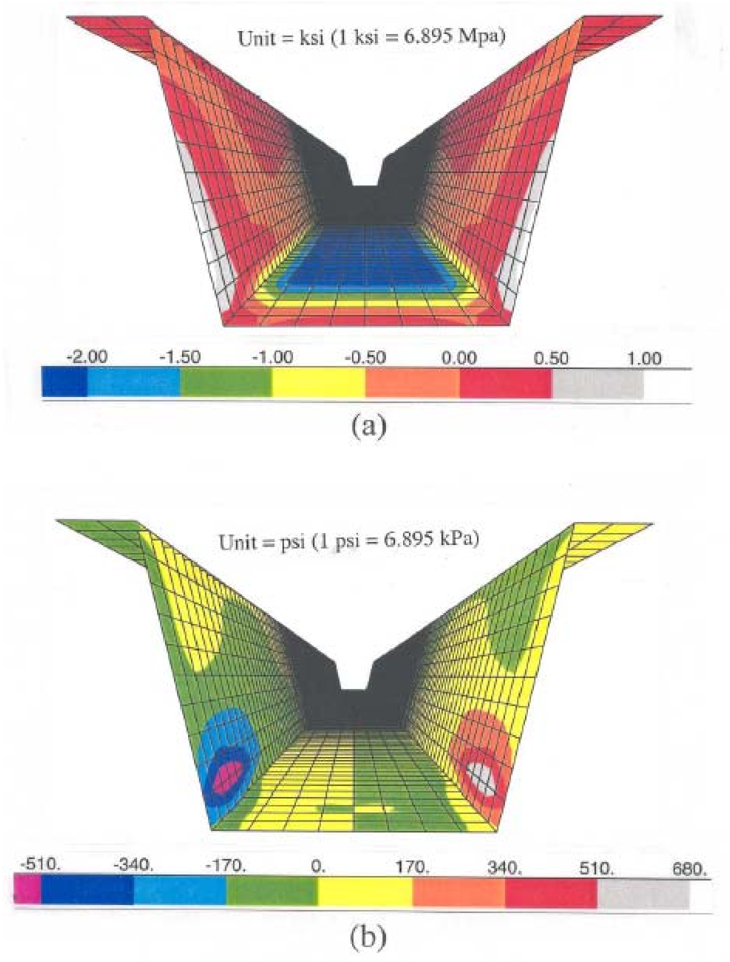


Fig. 15. Distribution of Stresses due to Prestressing Tendons,
(a) Vertical Stress, (b) Shear Stress.

The analytical results well match the observed test results [5]. Figure 15b shows the shear stress distribution caused by the prestressing forces. It can be observed that a high concentration of shear stresses occurs within the transfer length. This type of shear stress is often beyond the distance of one fourth of the beam height from the beam end face and may cause the prestressed beam cracking in an inclined direction when loaded by additional dead and live loads. High principal tensile stresses due to prestressing forces within the transfer length can be observed from Figure 17. For this reason, the authors suggest that the AASHTO Specifications be modified to read: “In addition to stirrups provided within the transfer length distance to resist the shear force due to the design dead loads and live loads, vertical stirrups acting at a unit stress of 20.0 ksi (137.9 Mpa) to resist at least 4 percent of the total prestressing force shall be provided in this same region. The total number of vertical stirrups provided per unit length shall not be less than the number of vertical stirrups required for the tensile force induced by prestressing tendons within the distance of one quarter of the beam height from the nearest beam end.” A more reasonable design method remains to be developed.

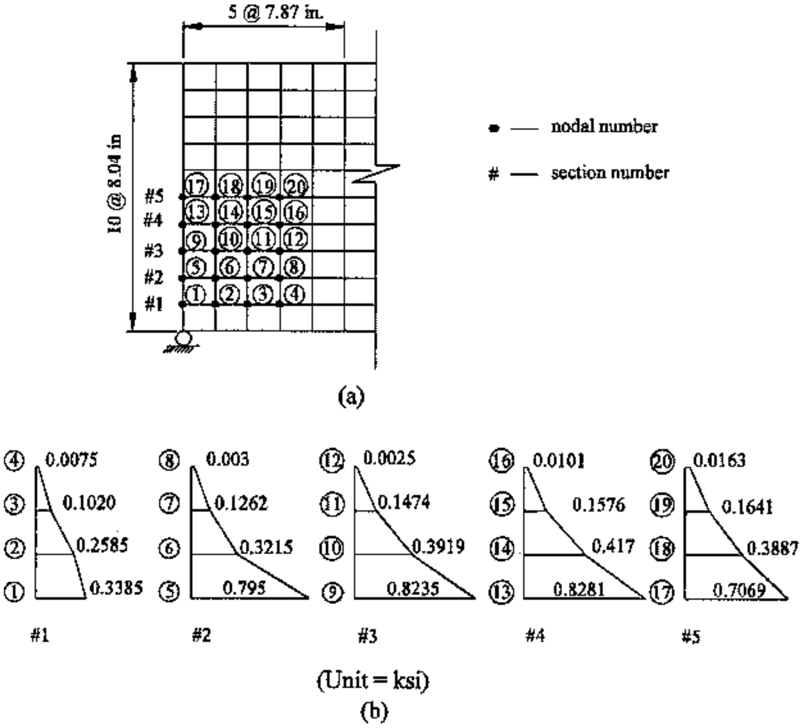


Fig. 16. Distribution of Vertical Tensile Stress of Web Without End Diaphragms, (a) Nodal and Section Numbering, (b) Distribution of Vertical Tensile Stress

Table 3. Vertical Tensile Force Induced by Prestressing Force

Item	Section Number				
	1	2	3	4	5
Total Prestressing Force, N_{ps} (kips)	1650.7	1650.7	1650.7	1650.7	1650.7
Vertical Tensile Force V_{ps} (kips)	43.0	67.9	79.2	80.1	73.7
V_{ps}/N_{ps} * 100%	2.60%	4.12%	4.80%	4.85%	4.47%

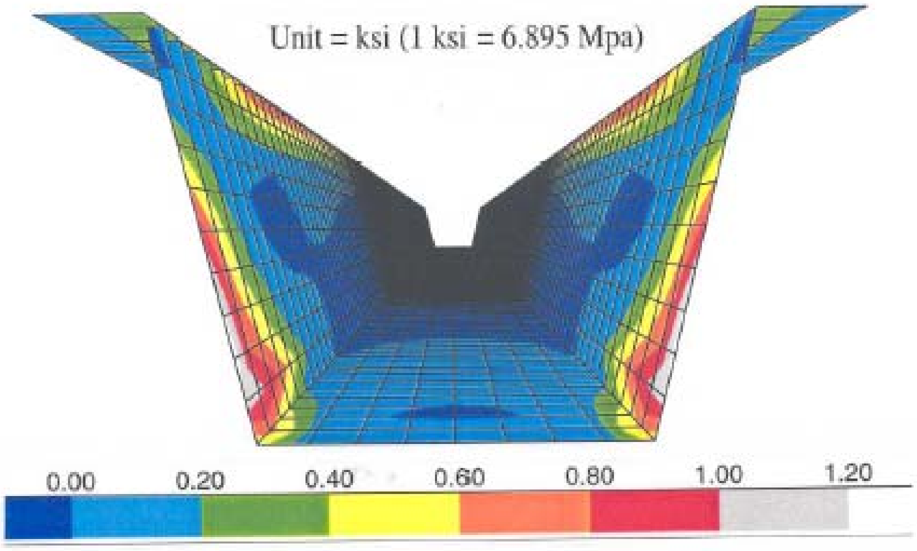


Fig. 17. Distribution of Principal Tensile Stress due to Prestressing Tendons

The U-beam is taken for a further example: The required web reinforcement for the design loading in the end zone is 0.1008 in^2 /in (2.56 mm^2 /mm). According to the proposed method, the required vertical reinforcement within the transfer length for the tensile stresses due to the prestressing forces is 0.0909 in^2 /in (2.31 mm^2 /mm). The total amount of the stirrup reinforcement within the transfer length is 0.1917 in^2 /in (4.87 mm^2 /mm). The required stirrup reinforcement, within the distance of one fourth of the height of the U-beam from the end, determined by AASHTO Specification Article 9.22.1 is 0.1846 in^2 /in (4.69 mm^2 /mm), which is approximately the amount determined by the proposed method. Note that the stirrup reinforcement provided in the end zone by the original design is only 0.1008 in^2 /in (2.56 mm^2 /mm) – not nearly a sufficient amount to control cracking.

CONCLUSIONS

1. The longitudinal prestressing tendons of an U-beam will induce a high concentration of shear stresses in the end zone near the web bottom. The downward shear force within the transfer length is about 4% of the total prestressing force. The effect of this type of shear stress has not been considered in the current AASHTO Specifications.
2. The mechanical models presented in this paper well reflect the actual structural behavior. The analytical results reasonably match the observed crack patterns.
3. The main cause of the U-beam web cracking in the Cross Florida Greenway Land Bridge is that the original design had not considered the effect of the longitudinal prestressing force on the vertical shear within the transfer length and there is insufficient stirrup reinforcement provided in the end zone. The total principal tensile stress where the cracks exist is more than 870 psi (6.0 Mpa), a stress large enough to induce cracks.

RECOMMENDATIONS

In the design of the shear reinforcement in the anchorage zone of a prestressed U-beam or other similar types of prestressed beams, the required transverse reinforcement should consist of two parts: (a) the portion required by the design live and dead loads, which can be determined by AASHTO Standard Specifications Article 9.20 or LRFD Article 5.8 and (b) the portion required by the shear force due to the longitudinal prestressing tendons, which can be determined by assuming the vertical stirrups acting at a unit stress of 20 ksi (138.0 Mpa) to resist 4% of the total prestressing force placed within the distance of the transfer length. The authors suggest that the AASHTO Specifications for prestressed beam anchorage zone design be modified to read: "In addition to stirrups provided within the transfer length distance to resist the shear force due to the design dead loads and live loads, vertical stirrups acting at a unit stress of 20.0 ksi (137.9 Mpa) to resist at least 4 percent of the total prestressing force shall be provided this same region. The total number of vertical stirrups provided per unit length shall not be less than the number of vertical stirrups required for the tensile force induced by prestressing tendons within the distance of one quarter of the beam height from the nearest beam end."

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The opinions and conclusions expressed in this paper are those of the authors and are not necessarily those of the Florida Department of Transportation.

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