Initial Report on Testing of Simulated Skyway Trestle Span Beams

December 12, 2006

Background

During the Spring of 2006 two 60 foot long Type IV AASHTO girders were constructed in the Dura-Stress yard in Leesburg, Florida to simulate the existing beams on the Skyway approach spans, in particular, the exterior girders that have cracking near the supports as shown in Figure 1. In late summer, eight-inch deep slabs were cast on these beams. A few weeks later, one end of each beam had the CFRP (carbon fiber reinforced polymer) wrap applied, as per the contract documents in the proposed repair. The materials and procedures complied with the winning bid. The beams were then tested to failure after allowing the carbon repairs one to two weeks to cure. Each end of each beam was tested thereby providing a test of the carbon wrapped section and an unwrapped section which functioned as a control.

Figure 1

Test Procedure

Figures 2 and 3 show the four tests in elevation. Figure 2 shows the elevation of the test with the load offset 7 feet two inches from the end. Figure 3 is similar but with the load offset 15 feet. The expectation was that the two different offsets would demonstrate different failure mechanisms. In both cases, loads were applied under manual control at a rate of around 250 pounds per second. At several load levels, the hydraulic system was locked and the beams were examined for cracks. This was done at predetermined levels and after audible indications of cracking. For all tests the load, deflections, strand displacements and strains were recorded continuously at a rate of 10 Hertz.

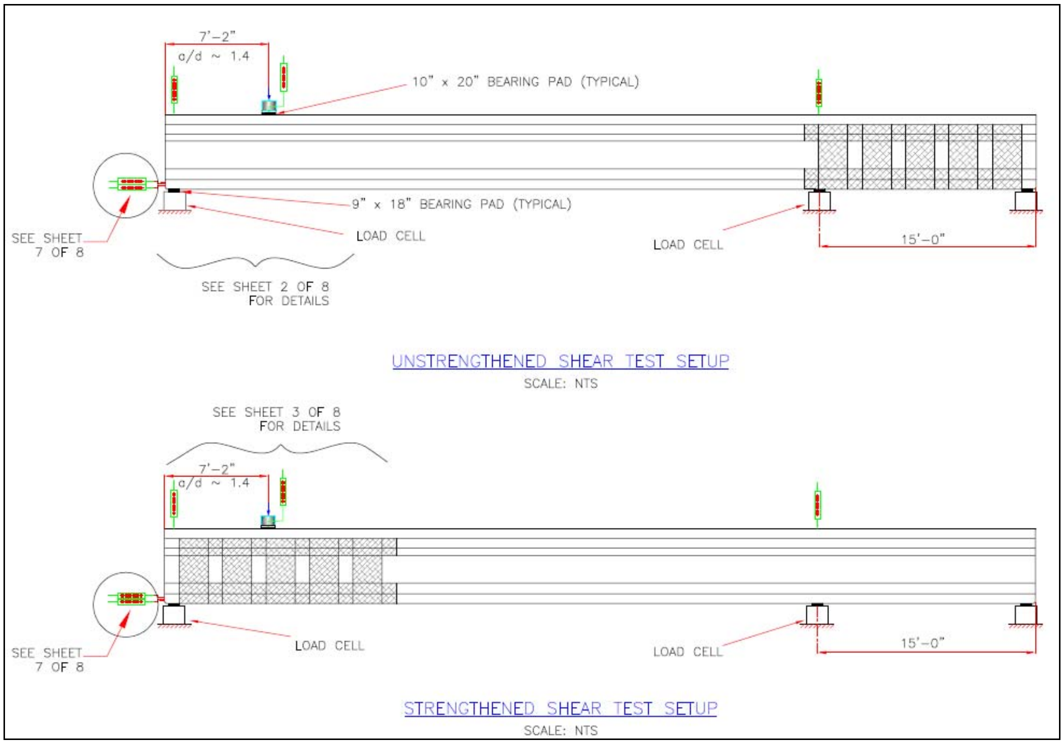

Figure 2

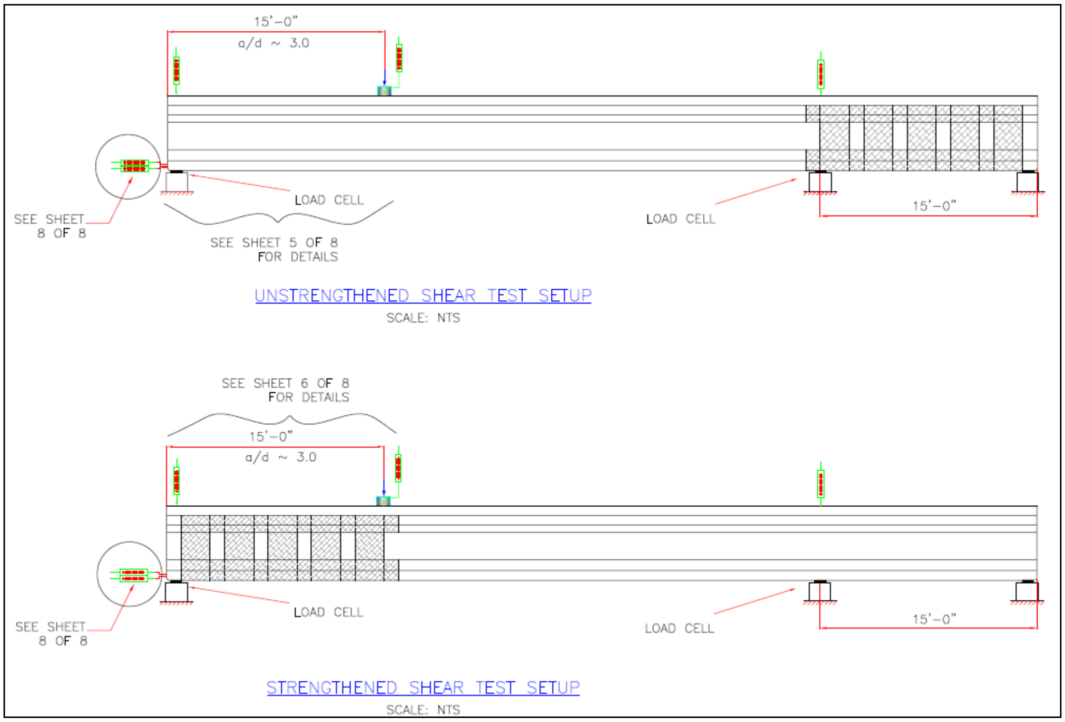

Figure 3

Test Results

Figure 4 shows the load deflection curves for the two tests with the load offset 7 feet 2 inches.

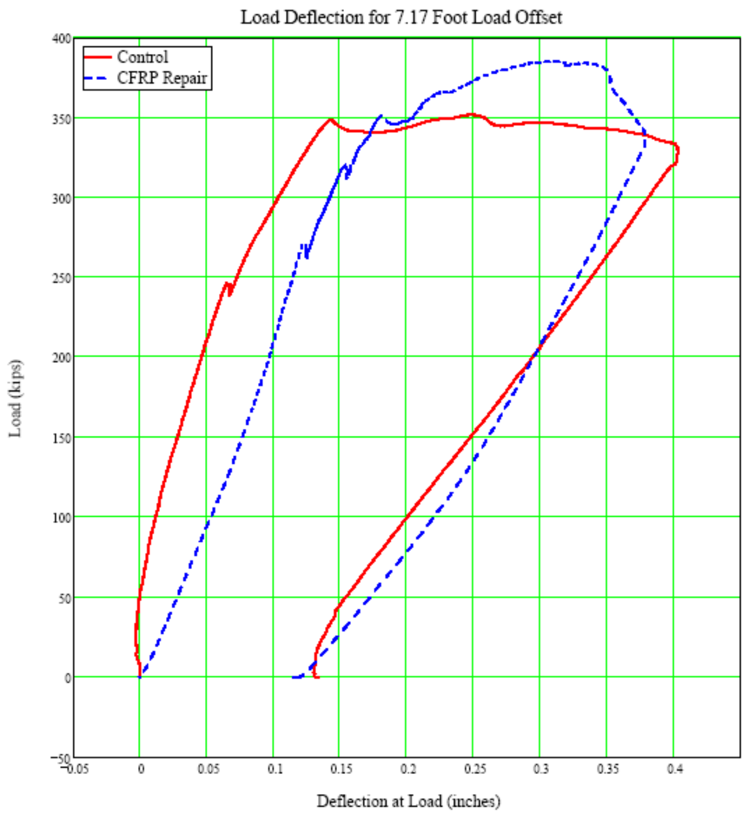

Figure 4

The breaks on the ascending portion of the curves are locations where the loading was stopped and the beams were examined for cracks. Figure 5 is the load deflection graph for the two tests with the load offset 15 feet.

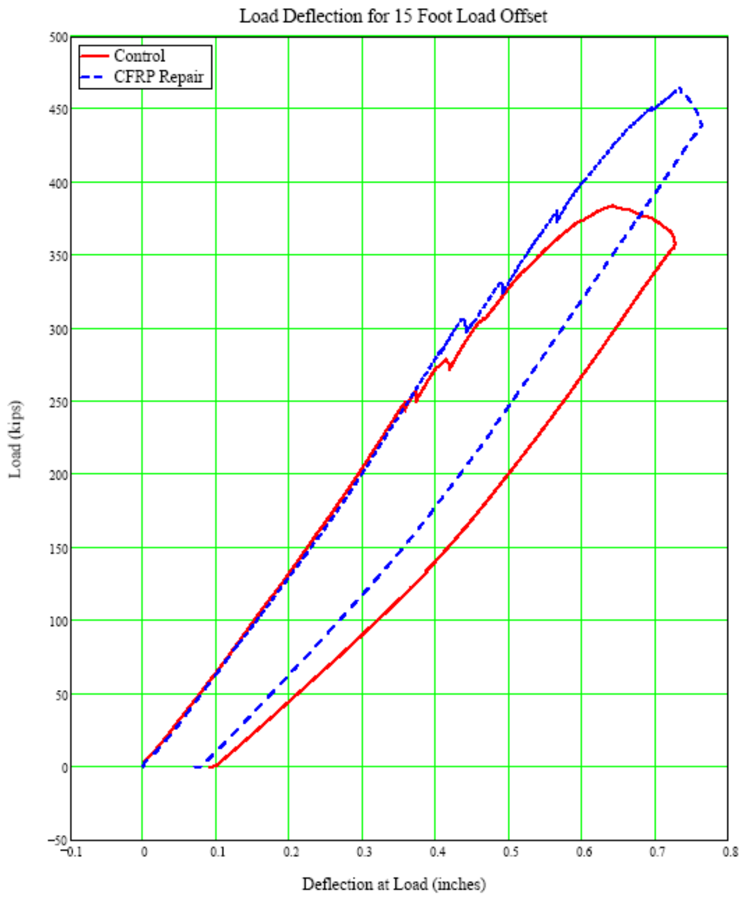

Figure 5

Strand slip was monitored during testing. A bracket was attached to the beam flange and displacement transducers were attached such that strand movement relative to the end of the beam could be recorded. Figures 6 through 9 provide the results from this monitoring concurrent with the deflection at the load location.

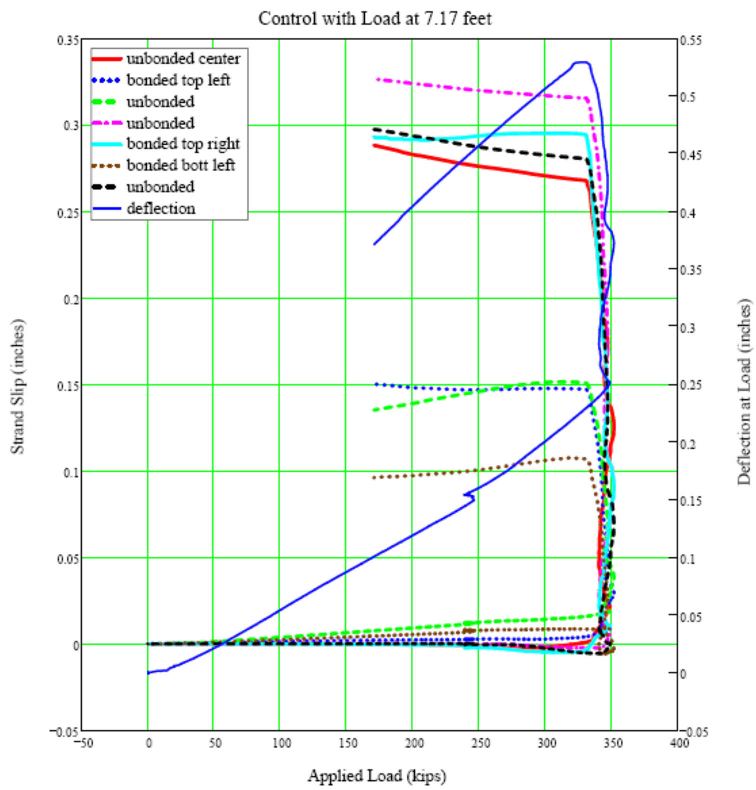

Figure 6

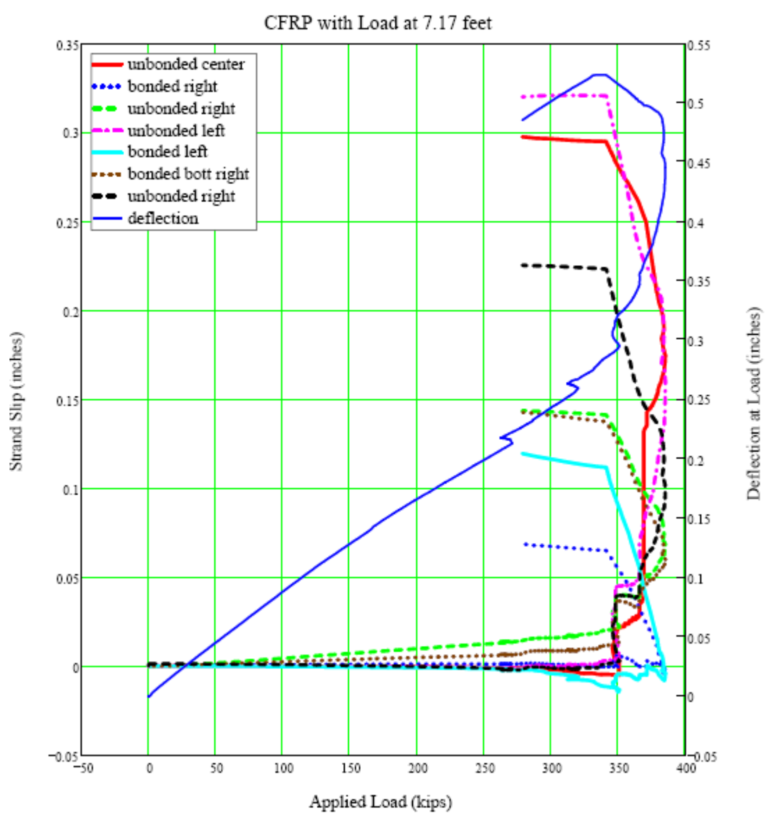

Figure 7

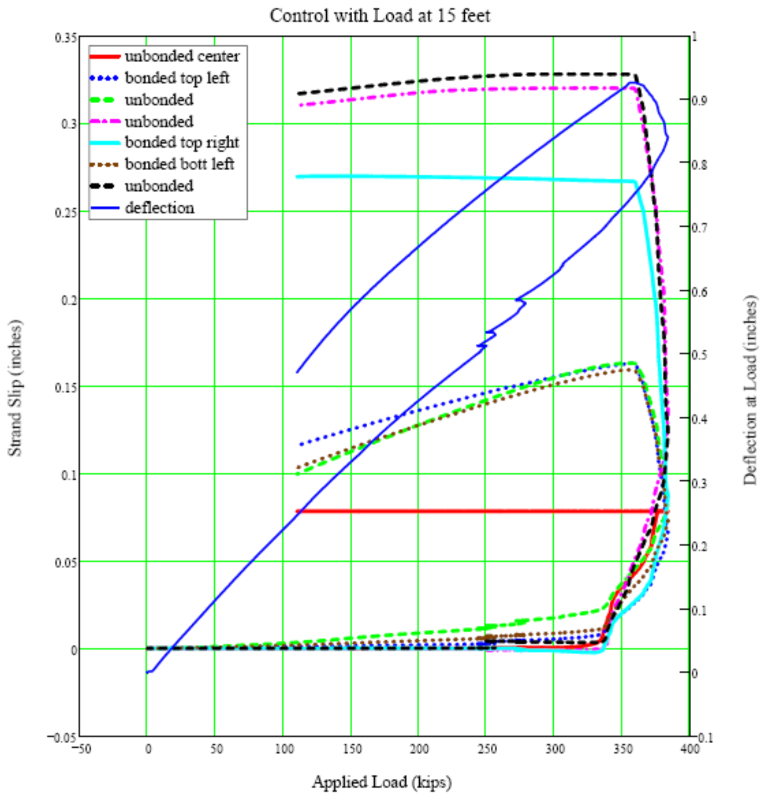

Figure 8

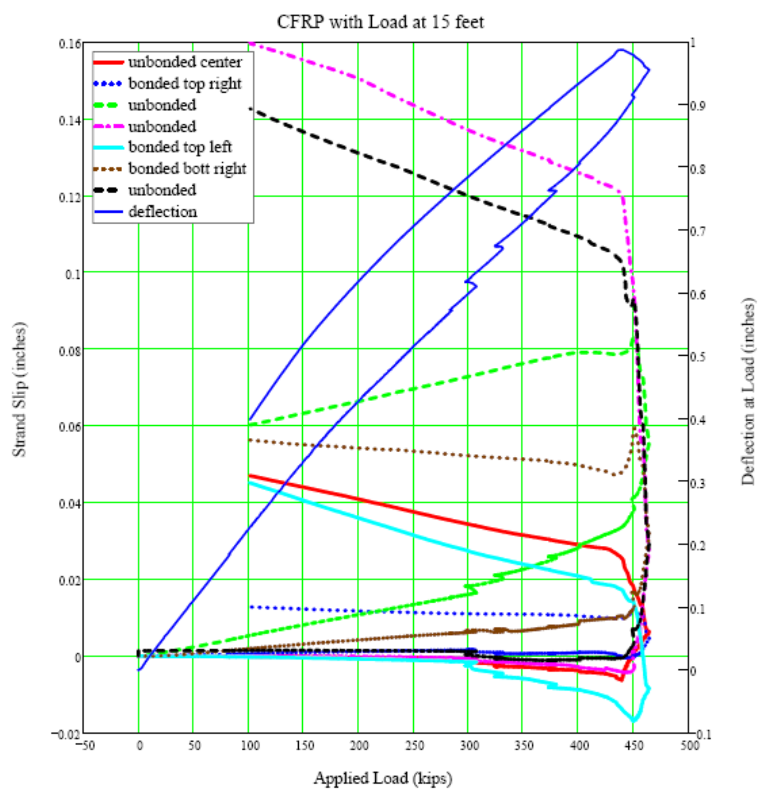

Figure 9

Three 4 by 8 inch cylinders for each beam concrete pour were tested concurrent with the load testing. The beam with the loads at a 7 foot 2 inch offset had a concrete compressive strength of 7222 psi and the beam with the 15 foot offset loads had a concrete compressive strength of 7182 psi. Cylinders for the slab pours were also tested as part of the load test program and had a compressive strength of 6400 psi.

The beam weights were measured after testing. They weighed approximately 63,700 pounds each. Table 1 presents the peak support reaction reached during each test including the additional force due to the beam weight.

Table 1

Test	Reaction
7.17 foot offset Control	321.4 kips
7.17 foot offset CFRP	349.8 kips
15 foot offset Control	281.1 kips
15 foot offset CFRP	335.7 kips

Discussion

Prior to testing a handling mishap broke off a corner of one of the beams as shown in Figure 10.

Figure 10

It was decided to use this end for the 15 foot offset Control test since the anticipated failure modes meant that this damage would not adversely change the results. A review of the failure modes and Table 1 indicates that this is probably not the case. Even including this test, the coefficient of variation of the maximum reaction for the four tests is less than 8 percent.

Conclusions

All four tests exhibited the same failure mode. Figure 11 shows the layout of the strands and the associated debonding pattern.

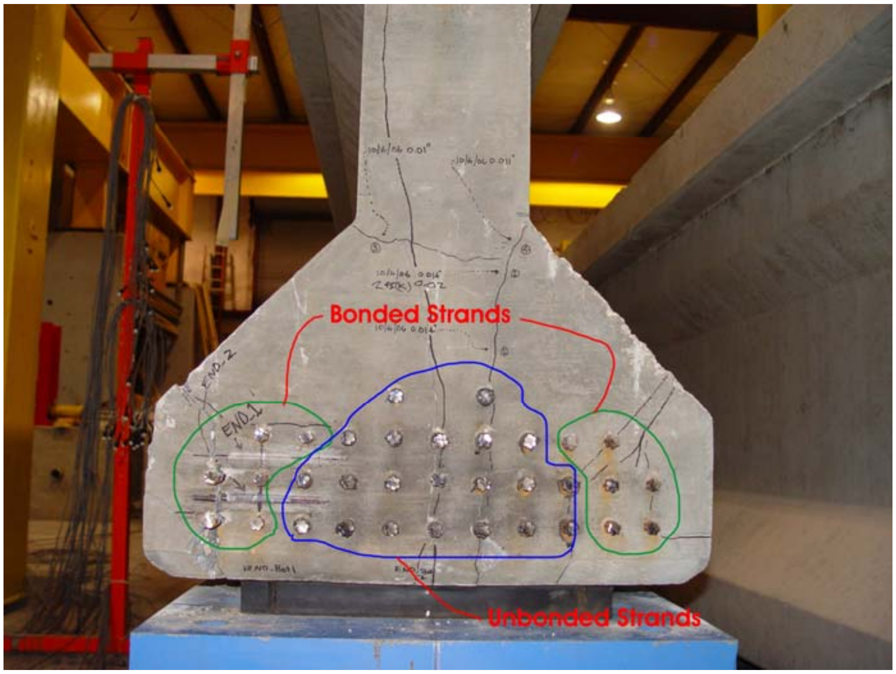

Figure 11

This pattern violates current policy by amount and distribution and is the direct reason for all of the failures. In addition, there is no confining steel around the strands contrary to current FDOT requirements.

The forces at the end of the beam must be resolved at the node as shown in Figure 12.

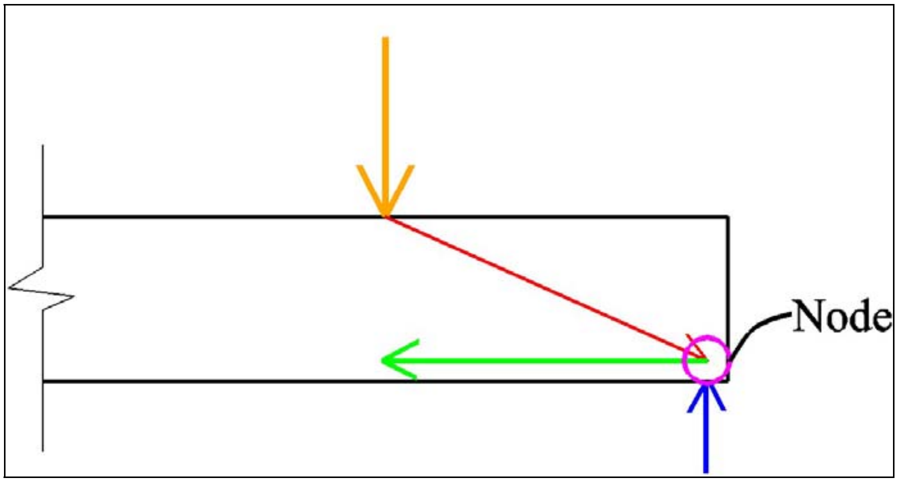

Figure 12

This green arrow represents the tension tie, which in the Skyway beams are the bonded prestressing strands. The unbonded strands do not become integrated into the beam structurally until they are some distance along the beam and outside the node. Therefore, they cannot carry the tension tie forces. The bonded strands are located on the outside of the flange of the beam as shown in Figure 11. The local forces must flow as shown in Figure 13 to allow the compressive forces to be resolved with the tie.

Figure 13

This leads to bursting forces that result in strand slip and the loss of the tension tie and beam integrity. The strand slip failure is obvious for all of the tests from a review of Figures 6 through 9. The cracking for all of the tests was very similar and very similar to the crack in the field shown in Figure 1. In addition, tests were conducted on the original Skyway beams during the time of construction. Figure 14 is the result of one of these tests and the similarity with Figure 13 is apparent.

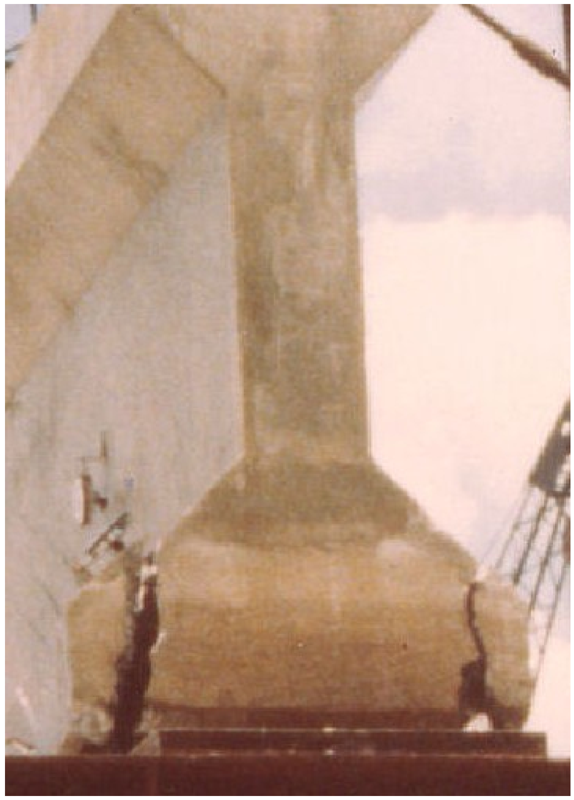

Figure 14

An examination of the exterior girders using the FDOT Prestressed Beam Program, modified to include the stressed strands in the web, indicates that, except for the debonding pattern, the beams are acceptable for HL-93 loads in moment and shear. It is the debonding pattern and the lack of strand confinement in the bearing region that compromises the integrity of these beams.

Our testing indicates the CFRP repairs provide modest increases in the capacity from 9 to 19 percent. Due to the ship impact restraints, the repair is offset from the bearing location, diminishing its effectiveness. To eliminate this tie failure mode would require additional confinement at the bearing location, the node location shown in Figure 12. This could be accomplished in several ways but would probably require removal or modification of the ship impact restraints at the repaired beams.

The beams tested in the lab had stronger concrete than that specified for the Skyway beams. An appreciation for the capacity of the actual beams can be determined from the ratio of the square root of the compressive strengths, which would reduce the field capacities to approximately 85 percent of the lab specimens. Unfortunately, since many of the actual bridge beams are already cracked this capacity may not be meaningful. Since no reinforcing constrains these cracks or inhibits their growth, it is extremely difficult to assign any capacity to the existing beams with cracks and any such assignment would be dependent on the tensile strength of unreinforced concrete. It would therefore be prudent to provide a repair that eliminates this failure mode.