Initial Report on Testing of Beams Removed from I-75 Bridge over Main 'A' Canal

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Background

Four AASHTO Type III beams from the I-75 over Main “A” Canal Bridge in Sarasota County were saw cut from the bridge and delivered to the Structures Research Center in Tallahassee. A portion of the composite slab, 28 inches wide, was included with the beams. Contrary to the original plans, the slabs were not cast-in-place but composed of precast deck panels with a cast-in-place topping. Figure 1 shows the cross section of the beam with the interface between the deck panels and the cast-in-place topping.

Figure 1

The deck panels are transversely keyed with a stepped thickness that alternates between 2.5 and 4.5 inches. Figure 2 shows removal of one of the beams from the bridge.

Figure 2

The beams were delivered with a portion of the diaphragms at each end and with the neoprene bearing pads. The beams were supported on these pads during the tests. Testing was requested since the calculated capacity of the bridges was less than required. In particular, the calculated shear capacities were deficient.

Testing

Five tests were conducted. Figure 3, in conjunction with the values in Table 1, describes the setup for each test.

During testing, the load, deflections at the load and both supports, strains at various locations and the movement of 10 strands were monitored and recorded at 0.1 second intervals. Loading was quasi-static with a rate of increase of approximately 0.25 kips per second.

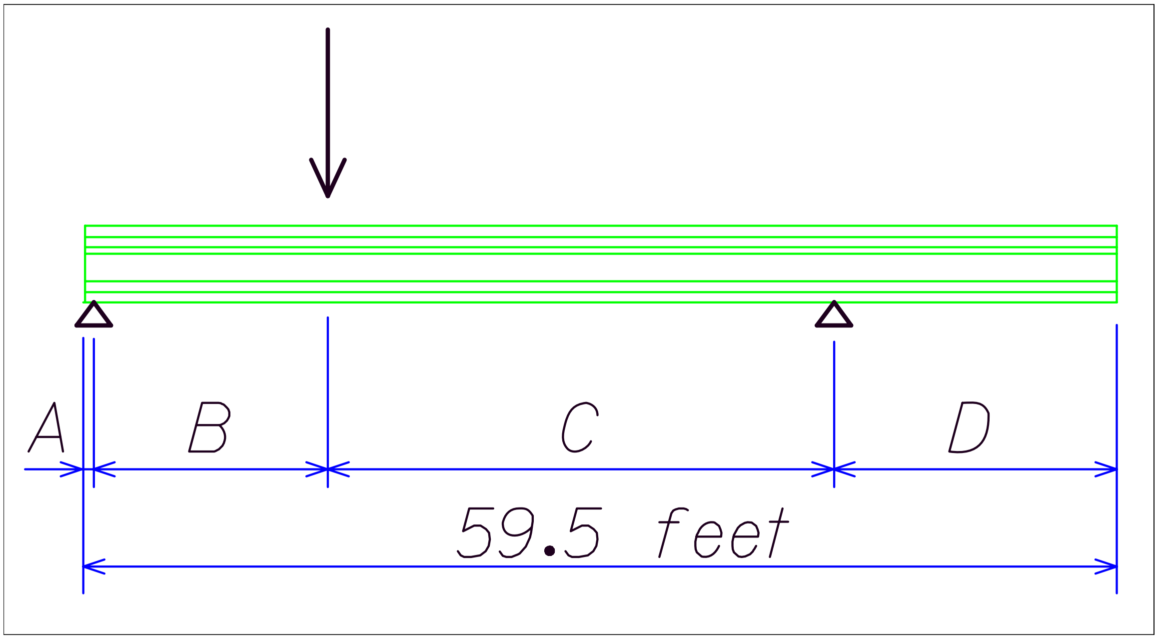

Figure 3

Table 1

Name	A (ft)	B (ft)	C (ft)	D (ft)
4 foot offset	0.46	4.29	40.00	14.75
8 foot offset	0.50	7.79	38.75	12.46
12 foot offset	0.50	11.96	38.58	8.46
16 foot offset	0.46	16.29	39.96	2.79
21 foot offset	0.50	20.96	37.54	0.50

Results

Figures 4 through 8 are the load deflection plots for each of the tests. The deflections in the graphs are the values at the load location adjusted to remove the rigid body motions due to the displacements at the supports.

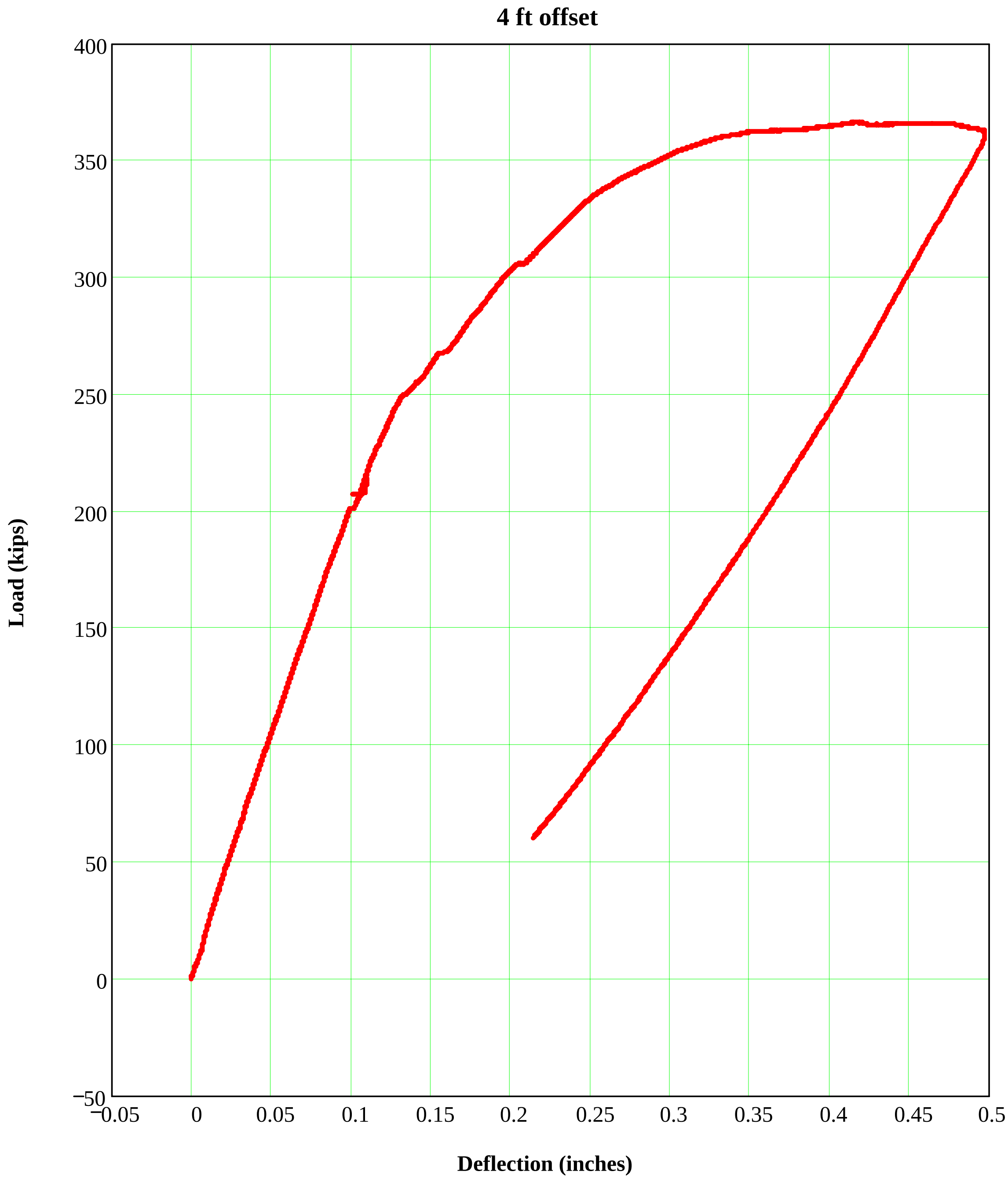
4 ft offset
Load (kips)
400
350
300
250
200
150
100
50
0
−50
−0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
Deflection (inches)

Figure 4

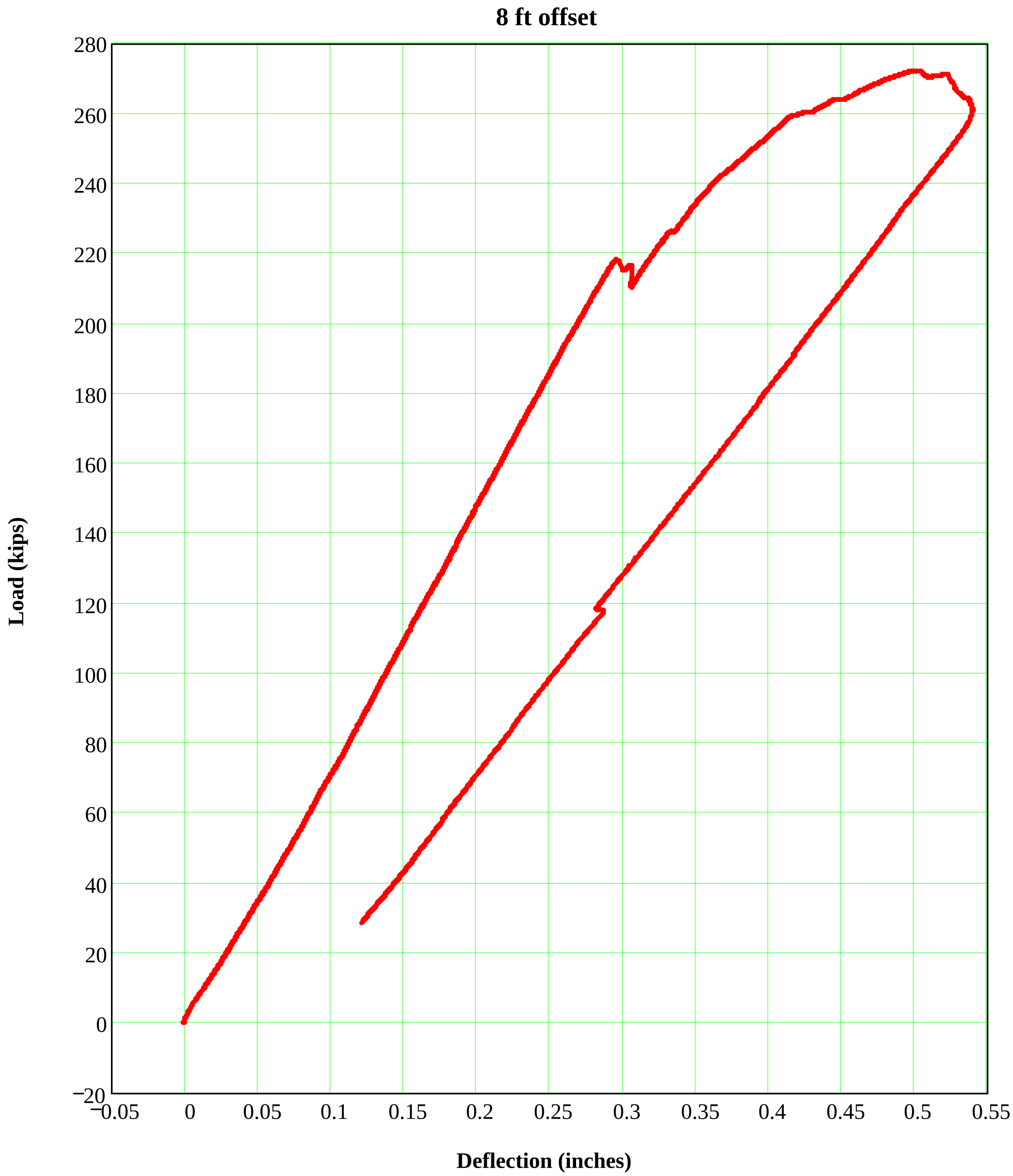
8 ft offset
Load (kips)
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
−20
−0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
0.55
Deflection (inches)

Figure 5

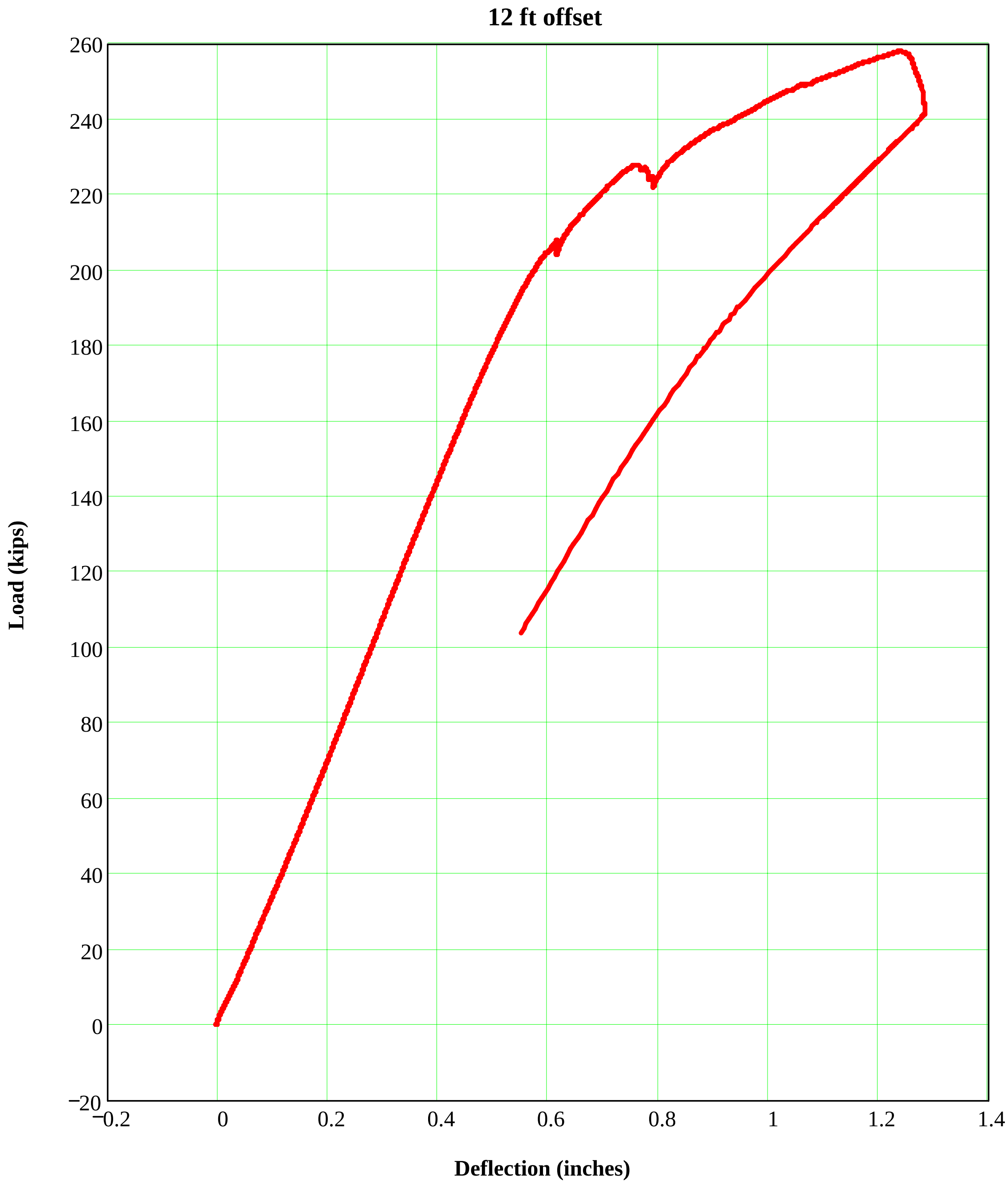
12 ft offset
Load (kips)
260
240
220
200
180
160
140
120
100
80
60
40
20
0
−20
−0.2
0
0.2
0.4
0.6
0.8
1
1.2
1.4
Deflection (inches)

Figure 6

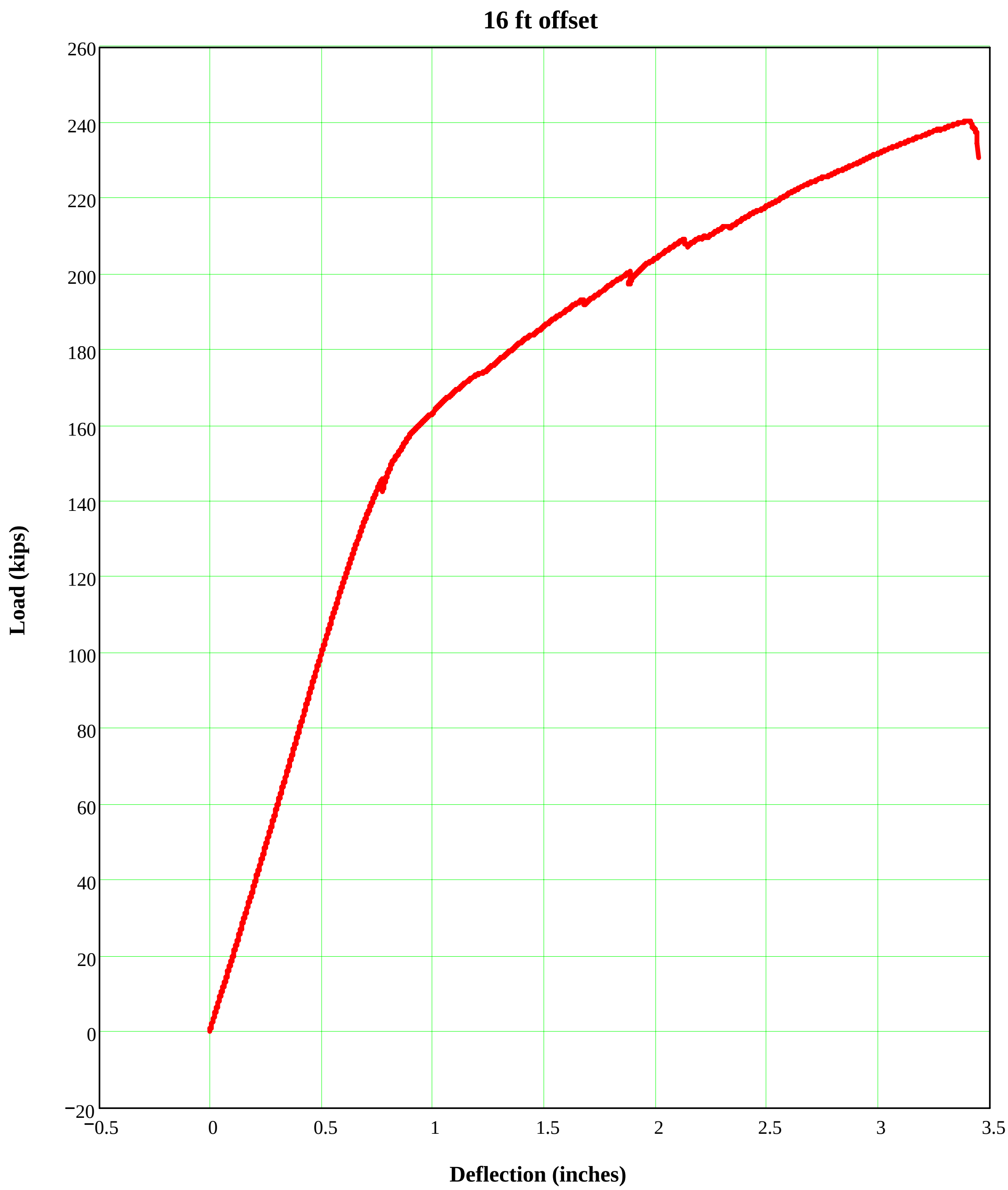
16 ft offset
Load (kips)
260
240
220
200
180
160
140
120
100
80
60
40
20
0
−20
−0.5
0
0.5
1
1.5
2
2.5
3
3.5
Deflection (inches)

Figure 7

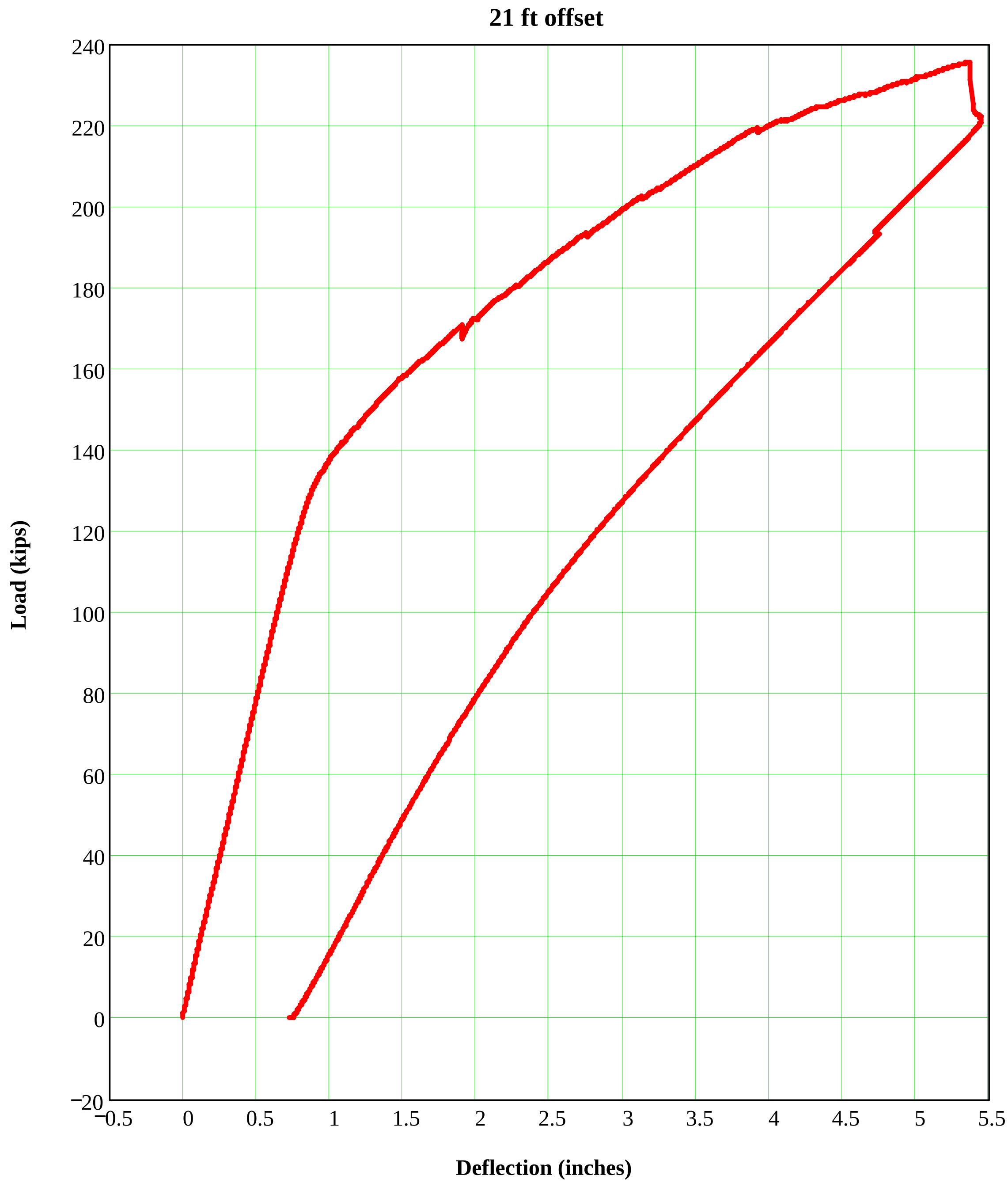
21 ft offset
Load (kips)
240
220
200
180
160
140
120
100
80
60
40
20
0
−20
−0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
Deflection (inches)

Figure 8

The five tests envelope the beam capacity and demonstrate three different failure mechanisms. The 4 foot, 8 foot and 12 foot offset tests failed due to strand slip. This is particularly critical in these beams built prior to the use of confinement reinforcing. Type III beams tested in the lab with similar load offsets but with confinement reinforcing exhibited higher loads and a different failure mechanism. The 16 foot offset test did not fail due to strand slip but due to failure of the compression zone from the combined effects of shear and flexure. Analysis indicates it did not reach its theoretical ultimate moment capacity. The 21 foot offset test also failed without strand slip in a flexural mode. This failure was a flexural failure since the moment at failure exceeded the analytical moment capacity.

Conclusions

Figure 9 summarizes the results of the testing along with Table 2. For comparison, the calculated HL-93 factored design loads for the original bridge are shown on the same figure. It should be noted that not only do the tested shear capacities exceed the HL-93 loads at all locations but also that the concurrent failure moments are greater than the design moments associated with the factored shears. The shear capacity of the 8 foot offset test exceeded the factored HL-93 shear by 27%. This was the smallest percentage increase observed.

It is speculated, that the improved performance, in terms of the obvious data trend, for the 4 foot offset test, could be due to the confinement provided by the load restricting strand slip. This is subject to further study but the result is consistent with previous tests, which demonstrate improved performance for loads located within 2 beam depths of the support. The reported shear and moment values in Table 2 are those at the section located at the load and include the additional forces due to the beam weight.

Table 2

Location from End of Beam (feet)	Shear at Failure (kips)	Moment at Failure (ft-kips)
4.75	**342.9**	**1478.5**
8.29	**237.5**	**1873.8**
12.46	**206.7**	**2528.1**
16.75	**180.1**	**3038.2**
21.46	**157.6**	**3474.0**

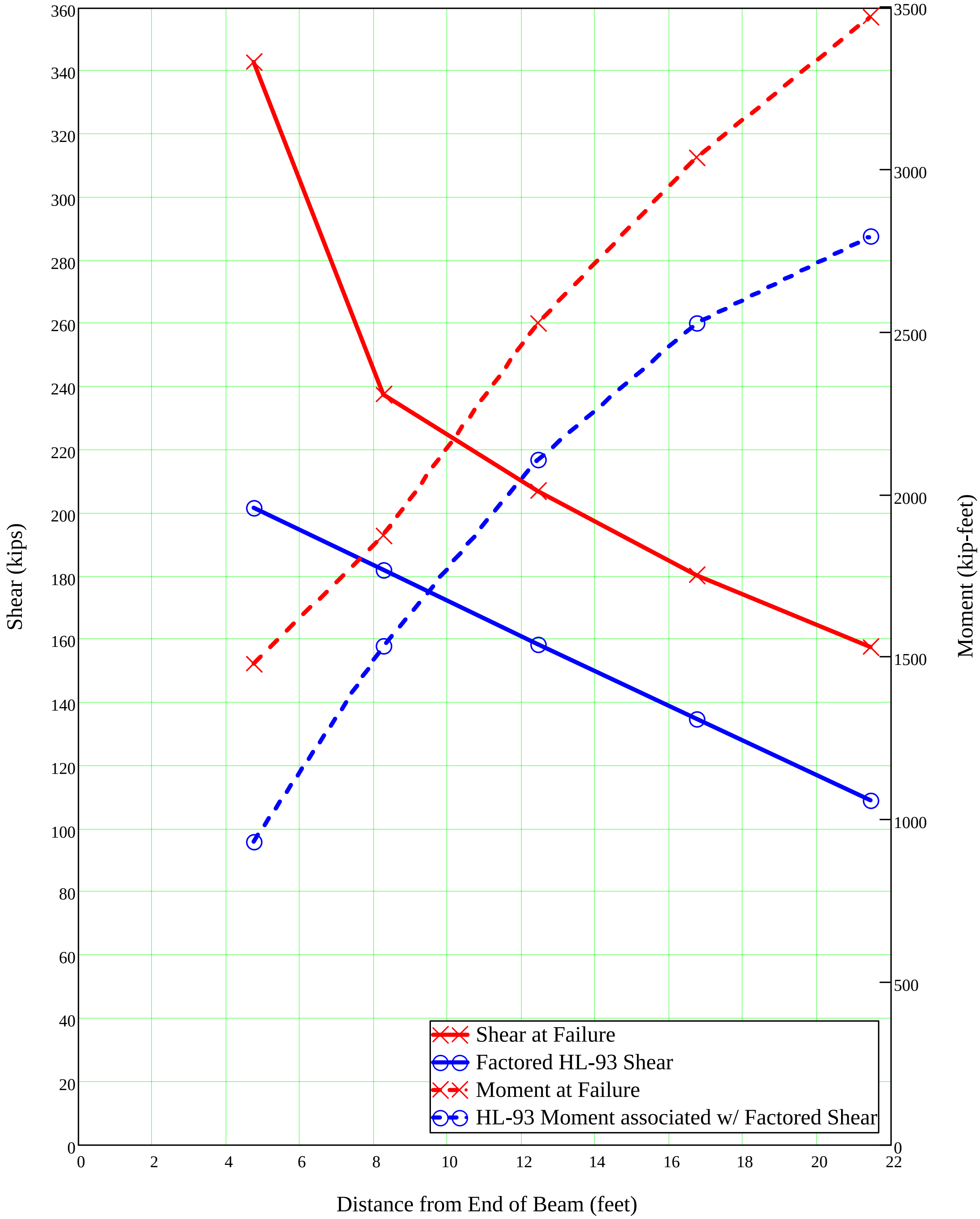
Shear (kips)
Moment (kip-feet)
Distance from End of Beam (feet)
360
340
320
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
3500
3000
2500
2000
1500
1000
500
0
0
2
4
6
8
10
12
14
16
18
20
22
Shear at Failure
Factored HL-93 Shear
Moment at Failure
HL-93 Moment associated w/ Factored Shear

Figure 9

Addendum

After testing, two of the stirrups were removed for material testing. The results in Table A-1 indicate that the bars were Grade 40 as indicated by previous work.

Table A-1

Sample Number	Date Tested	Tensile Strength (psi)	EUL @ .35%, Stress (psi)	Yield Stress (psi)	Ultimate Elongation (%)
1	2/22/07	89980	54,690	54,590	14.5
2	2/22/07	95240	58,700	58,510	11