Bridge Factors Factual Report Attachment 32 – FIGG 3/15/18 Power Point Presentation

Miami, FL

HWY18MH009

(49 pages)

FIU - Sweetwater
University City Bridge

Temporary Construction Loading Condition

Additional Review of Pylon Diaphragm Region

(Type II Deck End Diaphragm)

3/13/2018

The Span was moved into place on Saturday March 11, 2018, completed about Noon.

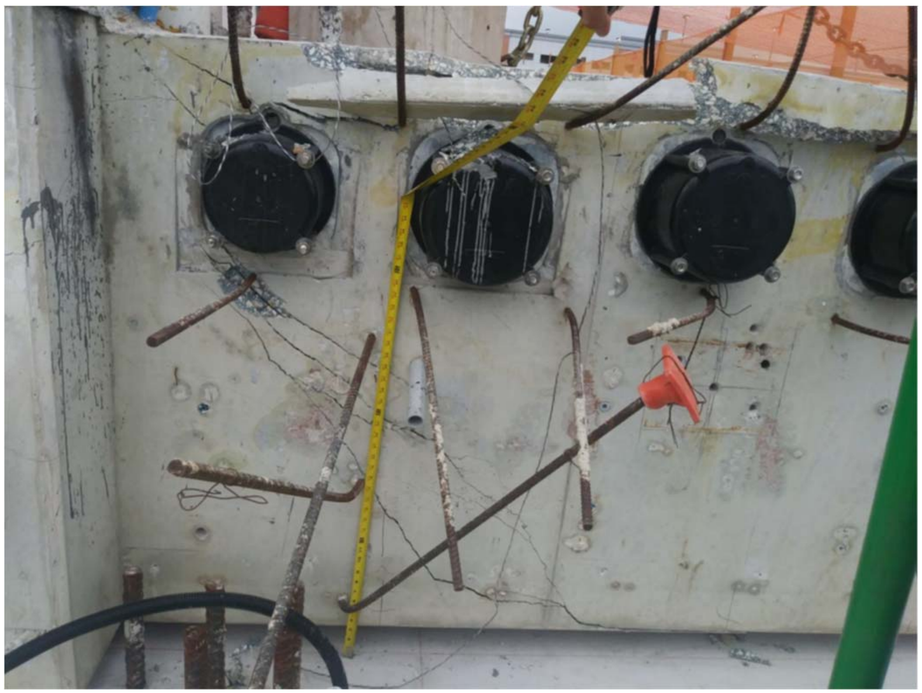
Post-Move Inspection

- Immediately after the span move was completed, Franklin Hines (FIGG) and the Project CEI staff inspected the bridge paying particular attention to regions where previous minor cracking had been noted before the move.

- The pylon diaphragm end of the span was also visually inspected and nothing of particular interest was noted at that time.

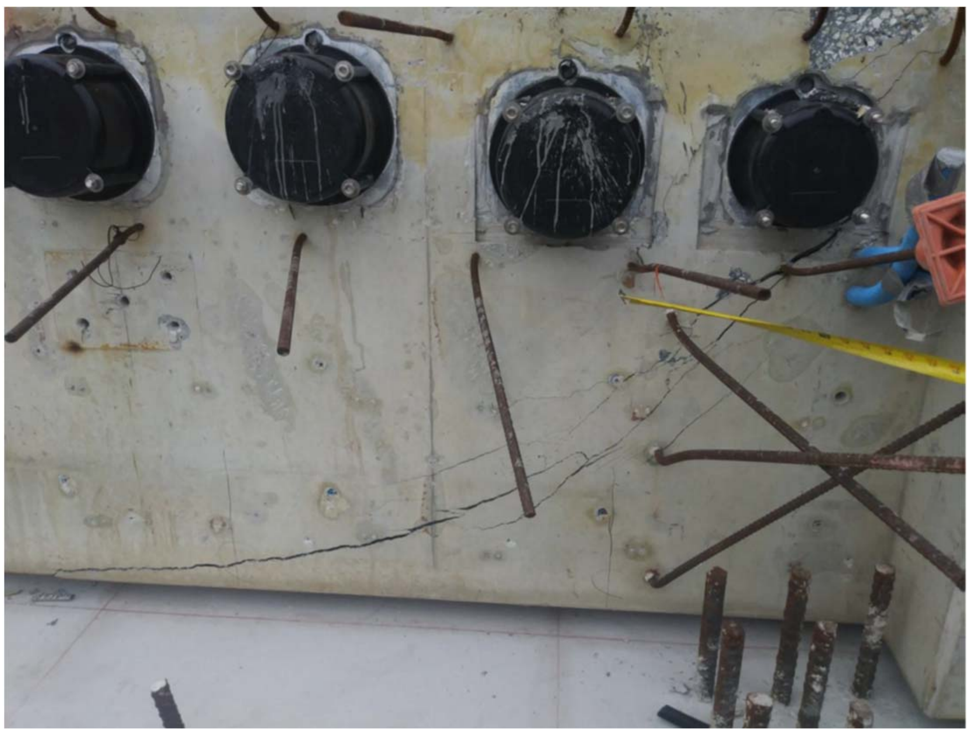
Crack Notification – March 13, 2018

- FIGG was notified by MCM's email of 4:52 pm 3/12/2018 that some cracks and spalls had developed at the pylon diaphragm end of the span. Below are photographs of the West side.



Crack Notification – March 13, 2018

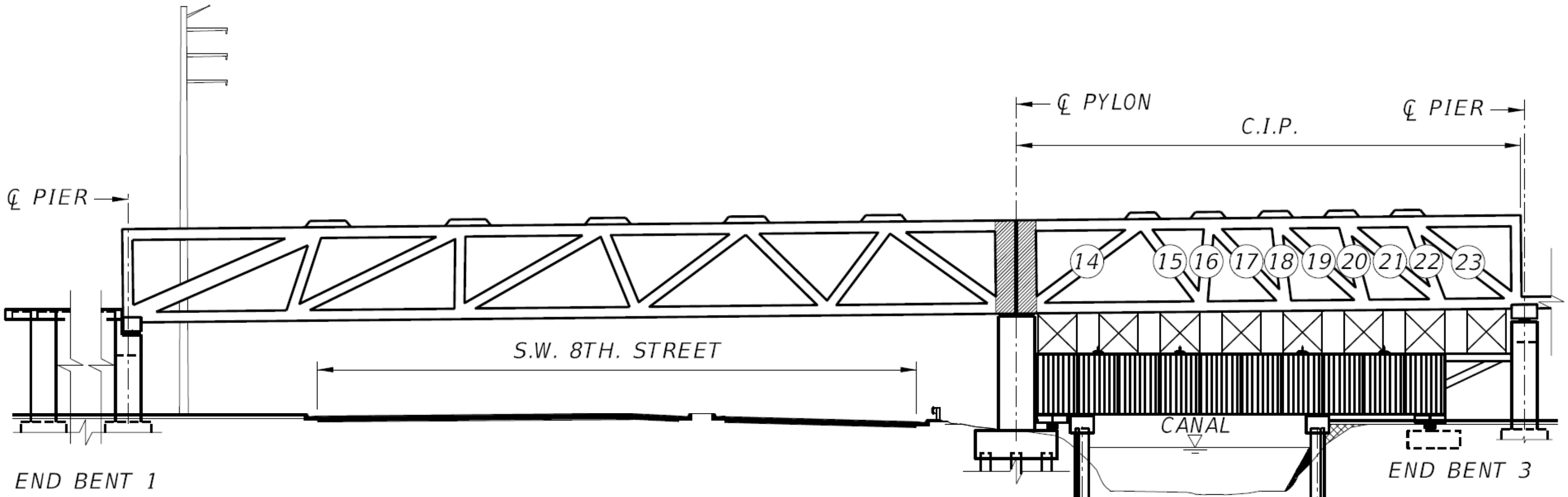
- FIGG was notified by MCM's email of 4:52 pm 3/12/2018 that some cracks and spalls had developed at the pylon diaphragm end of the span. Below are Photographs of the East side



Temporary Construction Condition

- Both the exposure of the diaphragm and the maximum load on the shims at this location are temporary.
- The end of the Type II Diaphragm becomes protected and encapsulated as the Pylon and CIP Back Span concrete is placed.
- The bending moments that develop in the continuous structure, when the falsework of the CIP Back Span is removed, will reduce the load on the shims from their current values.

Temporary Construction Condition



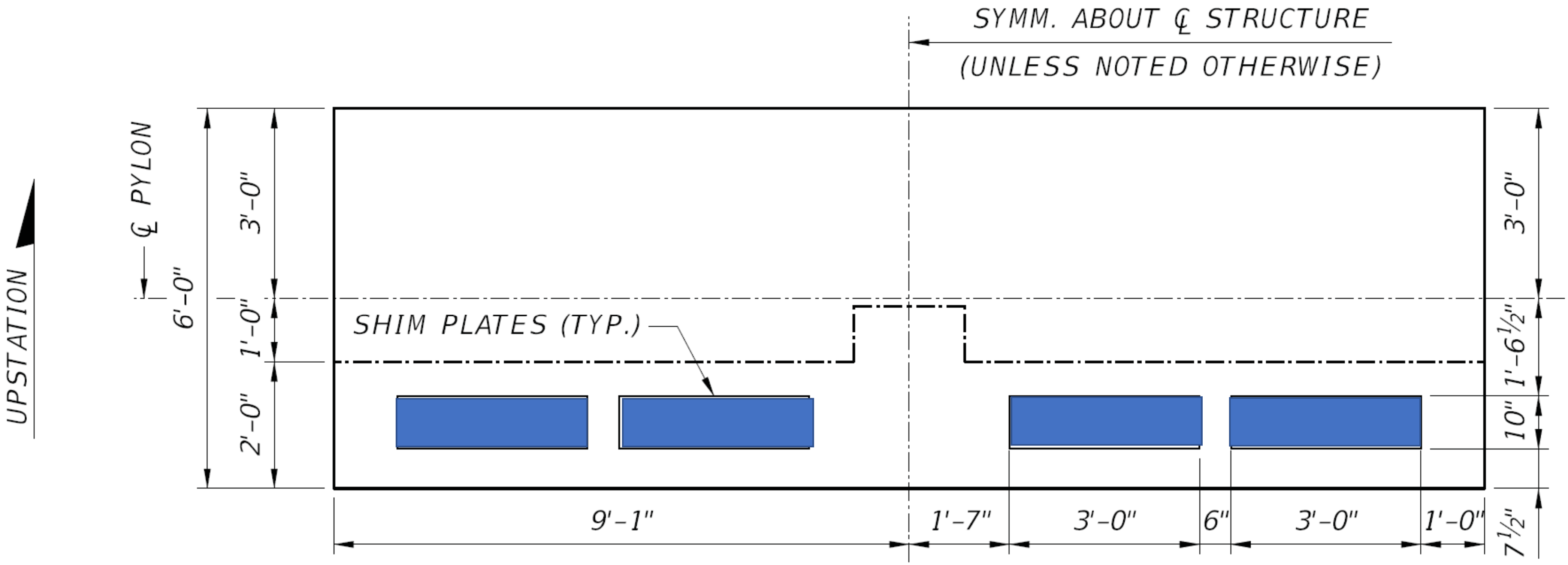
STAGE 4 - CASTING OF BACK SPAN

1. ERECT TEMPORARY BEAM AND FALSEWORK.
2. INSTALL BEARING PADS AT END BENT 3.
3. CAST INTERMEDIATE SECTION OF THE PYLON
4. CAST DECK, DIAGONAL MEMBER, VERTICAL MEMBERS, CANOPY AND TOP ANCHOR BLOCKS.
5. AFTER CONCRETE COMPRESSIVE STRENGTH HAS REACHED 6000 PSI, STRESS POST-TENSIONING OF THE BACK SPAN IN THE FOLLOWING SEQUENCE:
   - I. STRESS DECK LONGITUDINAL TENDONS D7.
   - II. STRESS CANOPY LONGITUDINAL TENDONS C5.
   - III. STRESS PT BARS IN DIAGONAL MEMBERS 15 AND 23.
   - IV. STRESS PT BARS IN DIAGONAL MEMBERS 16 AND 22.
   - V. STRESS PT BARS IN DIAGONAL MEMBERS 17 AND 21.
   - VI. STRESS PT BARS IN DIAGONAL MEMBERS 18 AND 20.
   - VII. STRESS PT BARS IN DIAGONAL MEMBER 19.
   - VIII. STRESS DECK LONGITUDINAL TENDONS D8 & D9.
   - IX. STRESS BOTTOM SLAB TRANSVERSE POST-TENSIONING. ALTERNATED END STRESSING IS REQUIRED FOR THE TRANSVERSE TENDONS.

Immediate Actions

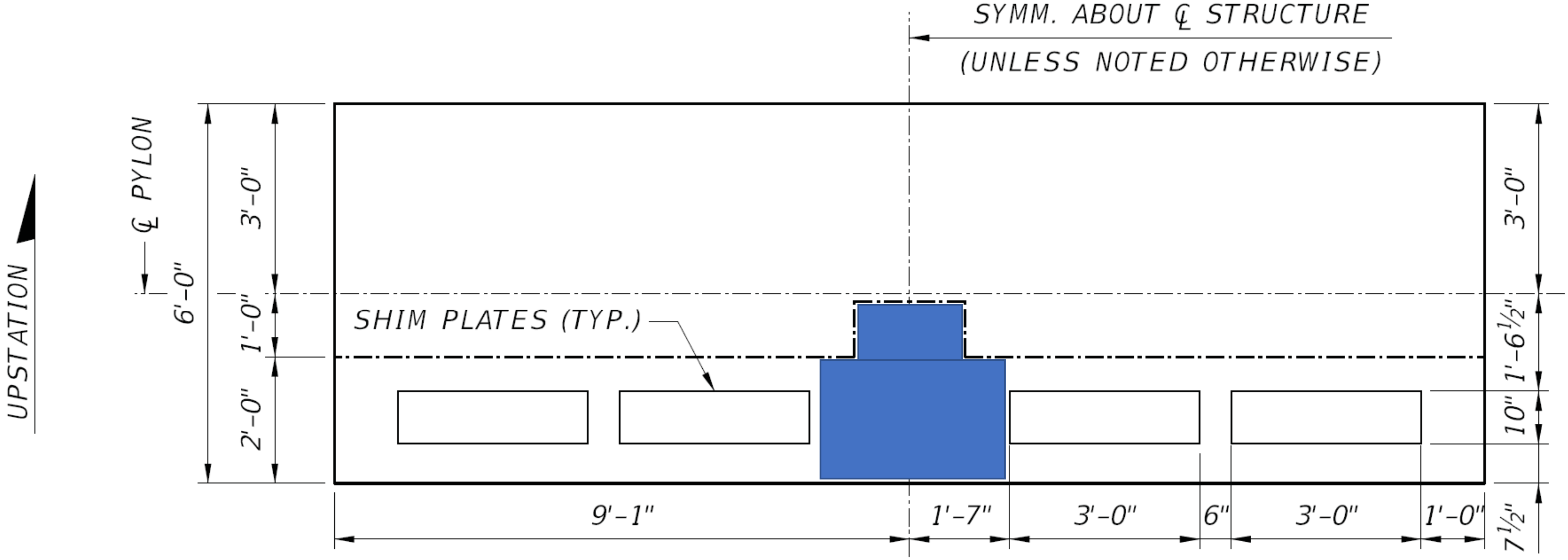
- Tuesday morning, upon seeing MCM's information, FIGG requested that, as a prudent action, MCM immediately install temporary shims directly below the nodal area of members 11/12 and the top of the Pylon/Pier, while further evaluations were on-going by FIGG.

The shims placed during the span move were:



PYLON BASE - PLAN

The recommended temporary shimming region is shown below, in blue:



PYLON BASE - PLAN

Safety

- Tuesday morning, after about an hour of review and evaluation, FIGG had conducted sufficient supplemental/independent computations to conclude that there is not any concern with safety of the span suspended over the road.

- MCM was so notified by Dwight Dempsey.

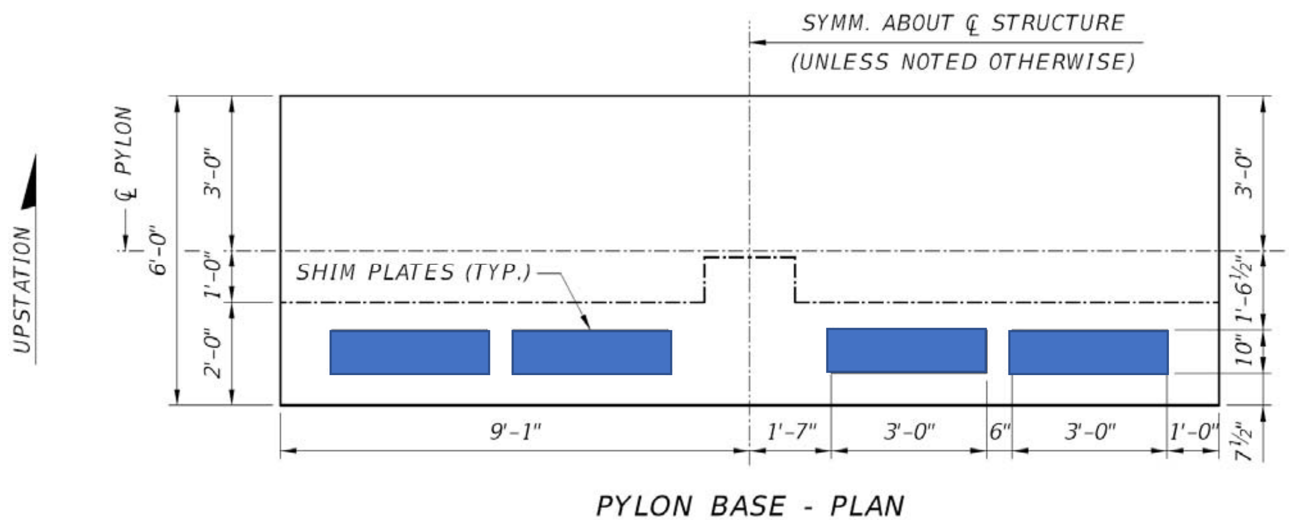
- The methods and results of this independent evaluation will be discussed in some detail further below.

History of Specific Operations

- The span was fully self-supporting on the end diaphragms in the casting area (full PT, etc.) for several weeks prior to the move.
- During this time, the pylon end diaphragm was uniformly supported over its entire surface area on the original soffit used during casting.
- No significant cracks or destress of this region were noted.

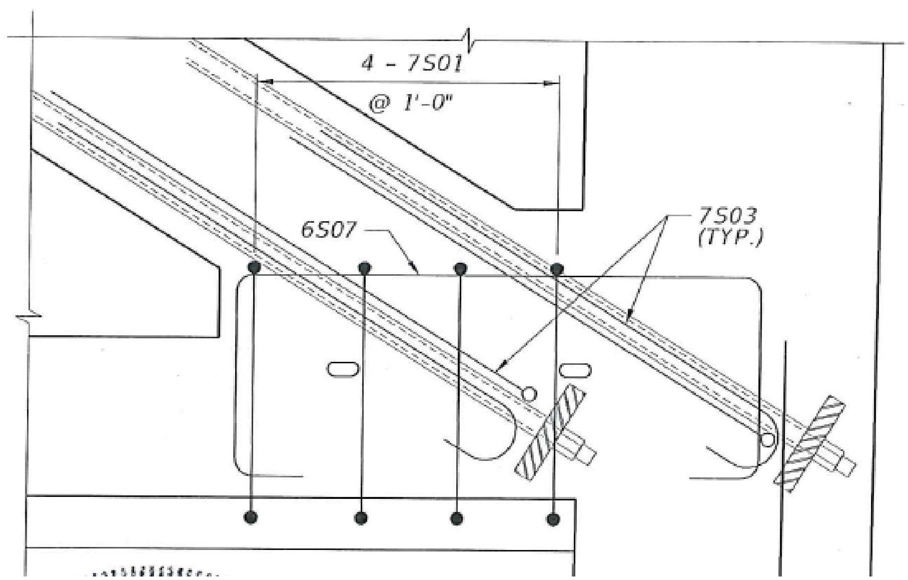
Similar support condition

- The span over the road is supported at similar locations as were used in the casting area,
- The difference being that the permanent bearings at the EJ (south) end, and
- The four shims (rather than uniform contact pressure) at the Pylon/north end.



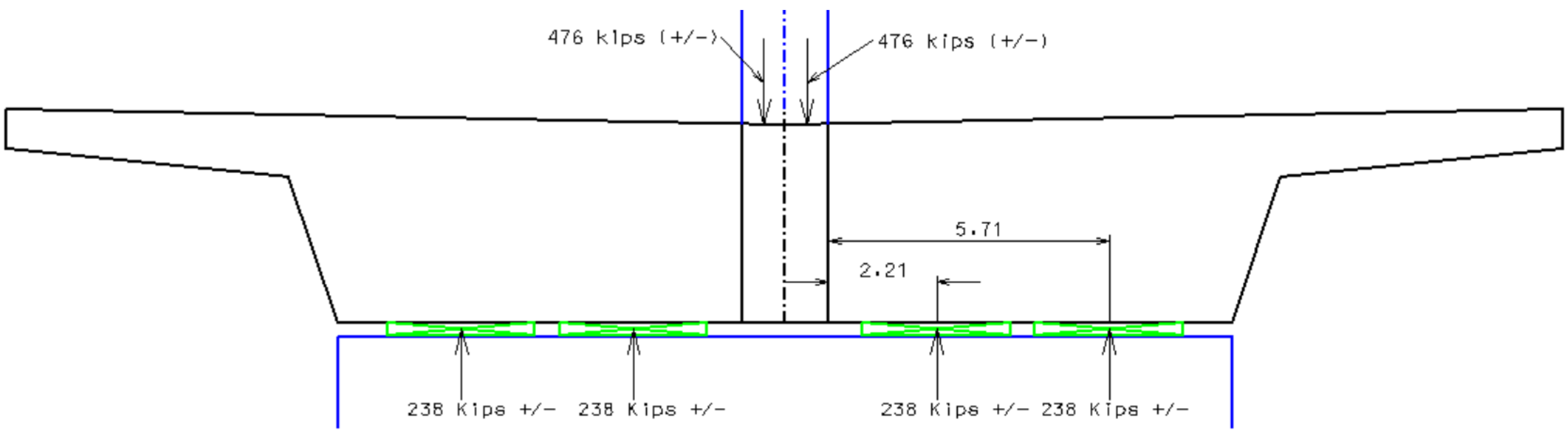
Destressing of Temporary PT Bars

- The only other notable difference from the condition in the casting yard location, is that the temporary PT bars in diagonal members 2 & 11 (needed for the move) were destressed.
- A study of the local effects of this detensioning has been made and will also be discussed later in this presentation.



Design re-checks: Flexure stresses on the bottom of the transverse diaphragm beam

- The field operations were conducted with the intent of achieving reasonably equal loads at the 4 shim locations.
- Assuming that that was achieved, and that the span weighs 950 tons (Barnhart told us that it was somewhat less; we are trying to get the as-weighted value), the following load diagram was developed:



Flexural Moment & Flexural Stress

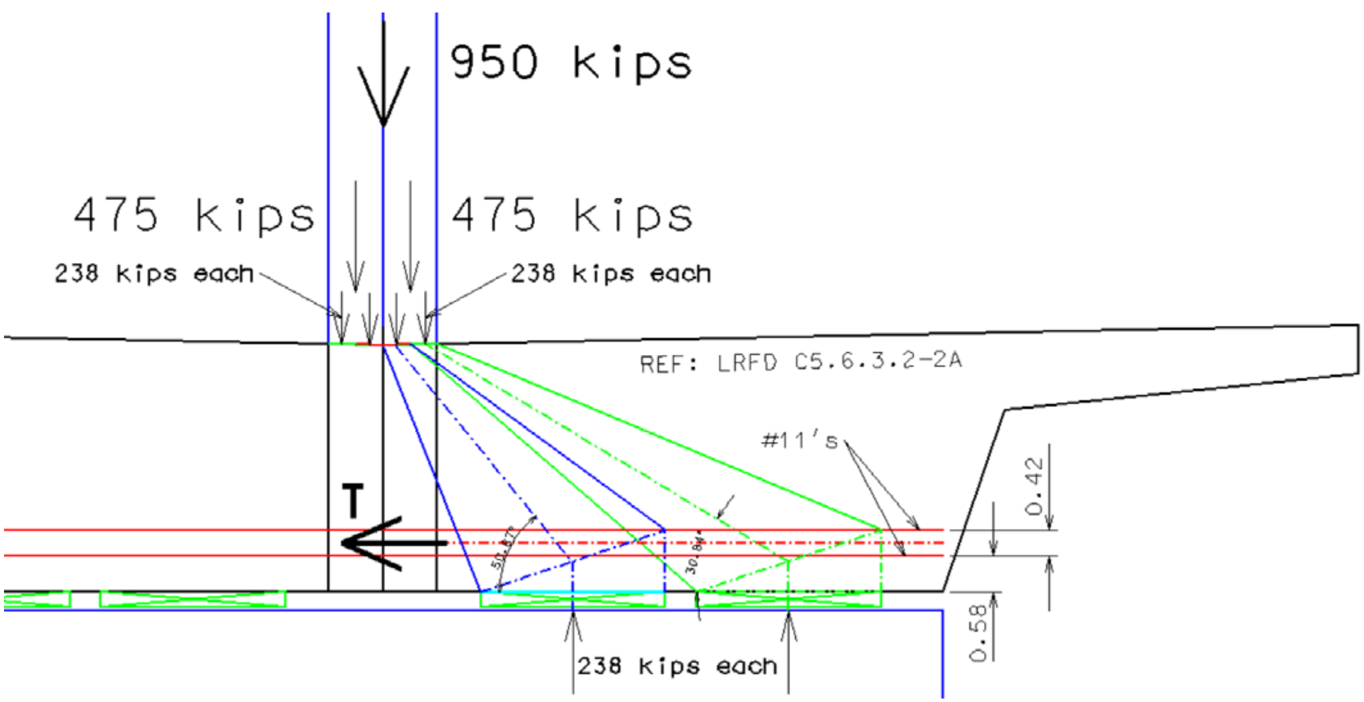
- Bending Moment (M) = 238 x (2.12 + 5.71) = 1865 kip-ft +/-

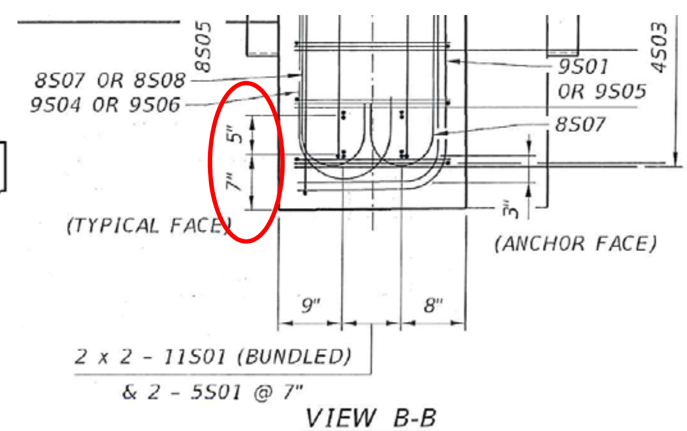
- Beam cross section (4' tall) by (2' wide)
- Bending section modulus (S) = (Wx(H^2)/6) = 5.33 ft^3

- Bending Stress (M/S) = 1865/5.33 = 350 ksf
- This value is above the concrete (f_r) strength, so cracking of the reinforced concrete element would be expected, as allowed by normal reinforced concrete design methods.

Strut and Tie Design Strength Check

- Given the dimension of this region, the most appropriate design approach is the strut and tie method of LRFD 5.6.3





Strut & Tie Tension Force

- T1 = 238 kips/(tan (50.87 deg) = 194 kips
- T2 = 238 kips/(tan (30.84 deg) = 399 kips

- Total Tension (T) = 593 kips (un-factored)

Construction Strength Checks

- The appropriate construction load strength combination to check is LRFD 5.14.2.3.4a (superstructure).

  1.1 (DC +Diff) + 1.3(CEQ + CLL)

- As can be seen in the photographs, CEQ and CLL are, for practical purposes, zero.
- Since the span was actually weighed, conservatively, (DC + Diff) can be taken as half of the theoretical span weight (actual was slightly less).
- The Factored Tie Force (Tu) = 1.1(593 kips) = 652 kips

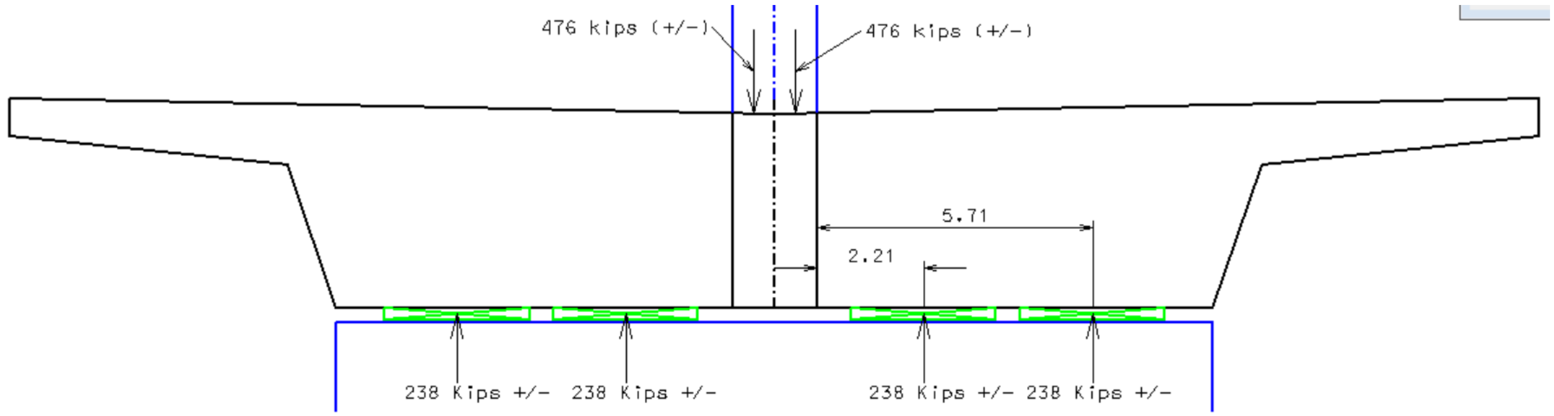
Construction Strength Checks (Strut & Tie)

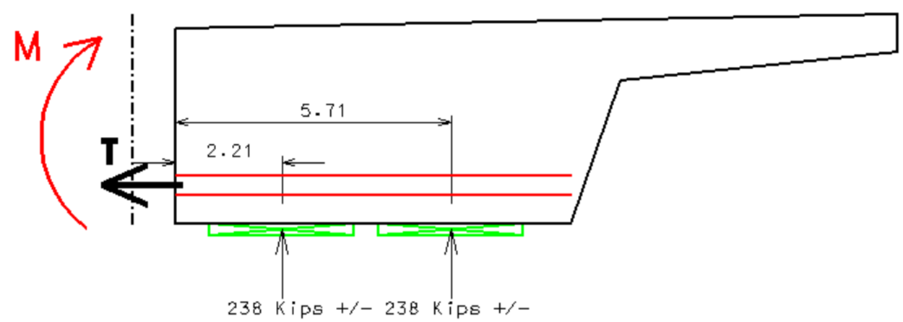
- Area of Steel Tie = ((8x 1.56) + (2x.31)) = 13.1 in^2
- Nominal strength of tie = (As)(Fy) = 786 kips
- Phi, per LRFD section 5.5.4.2, since this tie steel is anchoring the shim forces to the nodal region, Phi = 1.0 for “tension in steel in anchor zones” is the appropriate value.
- Thus (Phi)(Tn) = 786 kips which is larger than Tu (factored tie force).

- Others might interpret that Phi = 0.9 (for “tension controlled reinforced concrete”) would be appropriate, in that case, (Phi)(Tn) = 707 kips, which is still larger than Tu = 653 kips (Ok).

Bending Check – Beam Theory

- As previously noted, the strut and tie method is more applicable to this region. However, the conventional beam theory method can serve as a confirmation check to the strut and tie results.





Bending Check – Beam Theory

- As previously noted:

  Bending Moment (M) = 238 x (2.12 + 5.71) = 1865 kip-ft +/-

- The normal reinforced concrete behavior is assumed, where the compression in the concrete has to equal the tension in the rebar
- And … the moment created by the distance between the T and C forces must meet the demand moment

Conventional Method

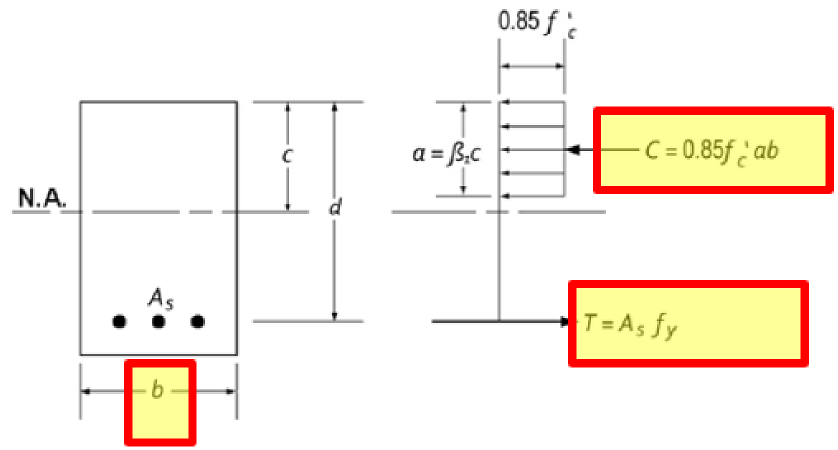
Rectangular Beam Analysis

Data:

- Section dimensions – b, h, d, (span)
- Steel area - As
- Material properties – f'c, fy

Required:

- Nominal Strength (of beam) Moment - Mn
- Required (by load) Design Moment – Mu
- Load capacity



$$c = \frac{a}{\beta_1}$$

$$\varepsilon_t = \frac{d-c}{c} 0.003 \geq 0.005$$

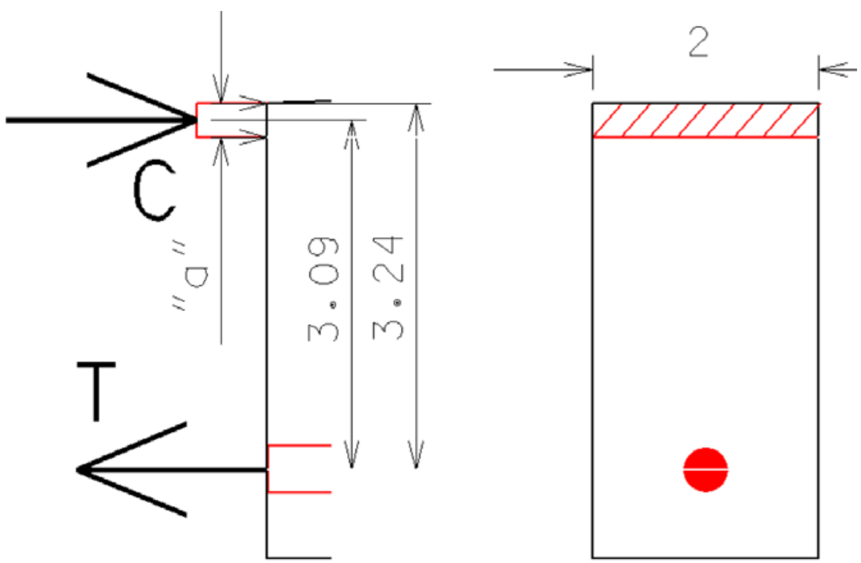
$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

$$\phi M_n \geq M_u$$

1. Calculate d
2. Check As min
3. Calculate a
4. Determine c
5. Check that $\varepsilon_t \geq 0.005$ (tension controlled)
6. Find nominal moment, Mn
7. Calculate required moment, ϕ Mn ≥ Mu

Bending Check – Beam Theory



- Steel Area = 13.1 in^2
- T = (13.1)(60 ksi) = 786 kips
- f'c = 8.5 ksi = 1224 ksf
- (a)(.85 f'c)(b) = C = 786 kips
- Solving, "a" = 0.38 ft
- Mn = (T)(d –(a/2)) = 786x3.09'
  = 2398 kip-ft (nominal capacity)
- Phi = 0.9, so (Phi)(Mn) = 2158 kip-ft
- Which is larger than Mu = 2015 kip-ft
- Check OK

Nodal Shear Transfer of Vertical Loads

- The diaphragm cross-section area is approximately (2’)(4’) = 8 SF
- The shear from the two shim on one side is approximately 476 kips.

- Thus the average shear stress from the vertical loads is approximately 60 ksf, which is within normal ranges for 8.5 ksi concrete

Nodal Shear Transfer of Vertical Loads

- For the shear friction transfer between the diaphragm and the nodal region here, conservatively, the transverse PT is not considered and only the mild steel is evaluated.

- Also, the "Cohesion" term of LRFD's shear friction equation 5.8.4-3 is conservatively ignored.

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = \underbrace{\cancel{cA_{cv}}}_{\text{Taken as 0.0}} + \mu \left(A_{vf} f_y + \underbrace{\cancel{P_c}}_{\text{Taken as 0.0}}\right) \qquad (5.8.4.1\text{-}3)$$

Nodal Shear Transfer of Vertical Loads

- For monolithic concrete, Mu = 1.4
- Thus, V_{ni} = (1.4)(17.91)(60 ksi)

$$\boxed{V_{ni} = 1504 \text{ kips}}$$

- The limits on V_{ni} of equations 5.8.4.1-4 and 5.8.4.1-5 are also met.

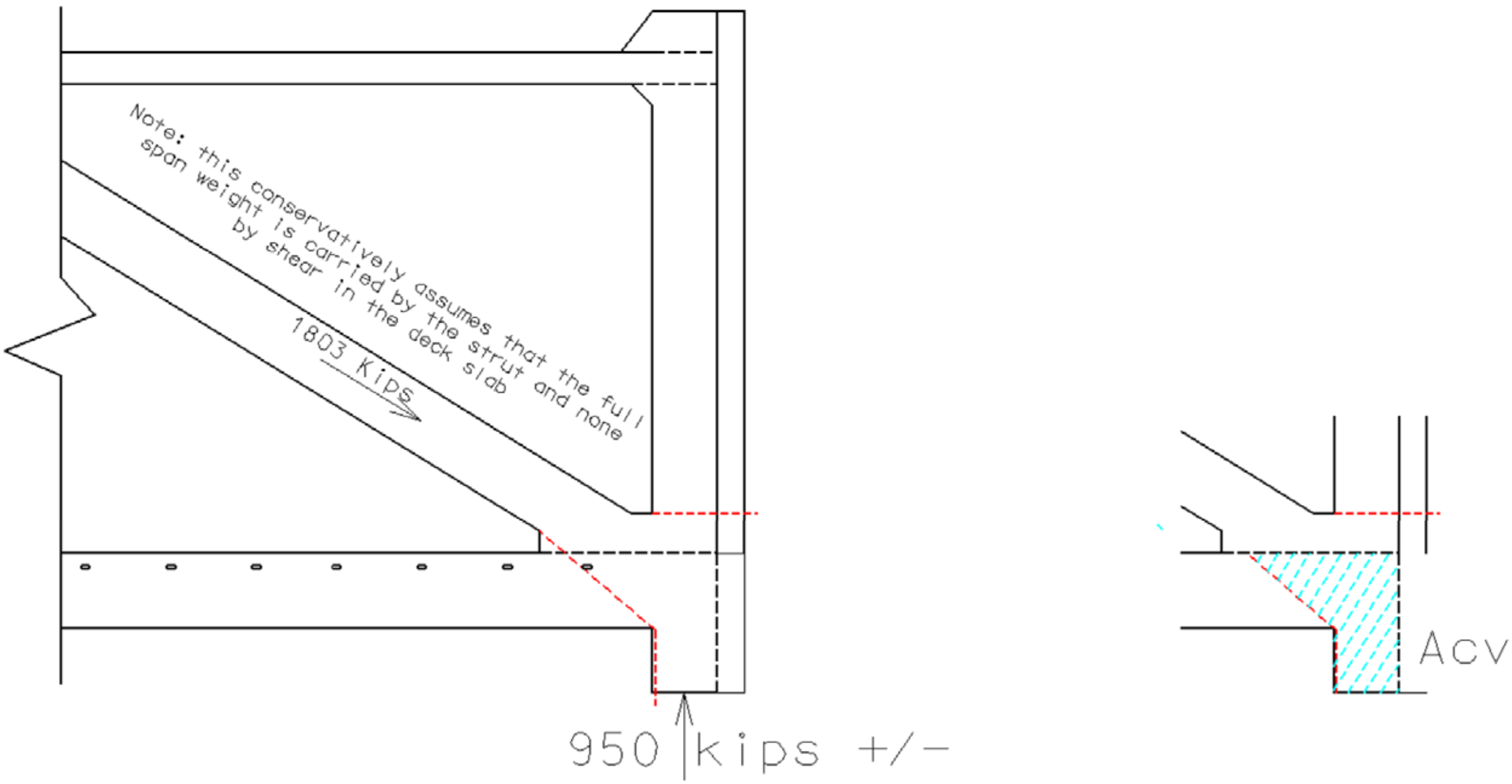
Number	Name	Area	Total Area	
8	11S01	1.56	12.48	
3	5S04	0.31	0.93	
3	4S02	0.2	0.6	
3	5S04	0.31	0.93	
3	4S02	0.2	0.6	
1	8S01	0.79	0.79	
1	8S02	0.79	0.79	
1	8S05	0.79	0.79	
			17.91	**in^2**

Nodal Shear Transfer of Vertical Loads

- Factored Shear Demand = (1.1)(476 kips) = 524 kips
- Reduced capacity (Phi)(Vni) = (0.9)(1504) = 1,354 kips
- Easily OK.

Total Nodal Shear Stability

- The total "node" must remain attached to the diaphragm/deck in order for the longitudinal tendons to capture the longitudinal force component of the strut.



Total Nodal Shear Stability

- Rebar crossing assumed shear plane

						Total	
	Number	Name	Area	# of sides		Area	
	3	4S01	0.2	2		1.2	
	1	8S05	0.79	2		1.58	
	1	8S06	0.79	2		1.58	
	3	5S04	0.31	2		1.86	
	3	5S04	0.31	2		1.86	
2 legs	2	11S04	1.56	2		6.24	
	2	8S04	0.79	2		3.16	
	2	9S03	1	2		4	
	2	5S02	0.31	2		1.24	
						22.72	**in^2**

Total Nodal Shear Stability

- Transverse PT Confinement (P_c)
- There are 65 4 x 0.6 dia tendons in the 175’ span
- The total transverse tendon force is approximately;
  - (65)(4)(0.217 in^2)(270 ksi)(63%) = 9,600 kips
  - Or (9,600 kips/175’) = 54.8 kips/ft
- The assumed node has a length of approximately 4.75’, thus the tendons provide (54.8 kips/ft)(4.75’)(2 sides) = 520 kips of confinement (P_c) force

Total Nodal Shear Stability

- Acv = (2)(11.81 sf) = 23.62 sf (shear plane total surface)
- Monolithically placed concrete has (per LRFD 5.8.4.3)
  - C = 0.40 ksi = 57.6 ksf
  - Mu = 1.4
  - K1 = 0.25
  - K2 = 1.5 ksi = 216 ksf

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c) \qquad (5.8.4.1\text{-}3)$$

Total Nodal Shear Stability

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c)$$

- c x Acv 57.6 k/sf x 23.62 sf = 1360 kips
- Mu x As Fy = 1.4x22.72x60 = 1908 kips
- Mu x Pc = 1.4 x 520 kips = 730 kips

  = 3947 kips Total = V_{ni}
- FIGG's general preference is to neglect the Cohesion portion when practical. Thus, Vni without "C" = 2638 kips
- Phi = 0.9
- Phi)(Vni) = 3552 kips with "c"
- (Phi)(Vni) = 2374 kips without "c"

Total Nodal Shear Stability

- The factored Demand Nodal Shear = (1.1)(1803 kips) = 1983 kips
- This is less than either of the (phi)(Vni) values, So … Check = OK.
- Note, the upper limits for Vni (LRFD 5.8.4.1-4 and 5.8.4.1-5) were checked and also found to be within limits.

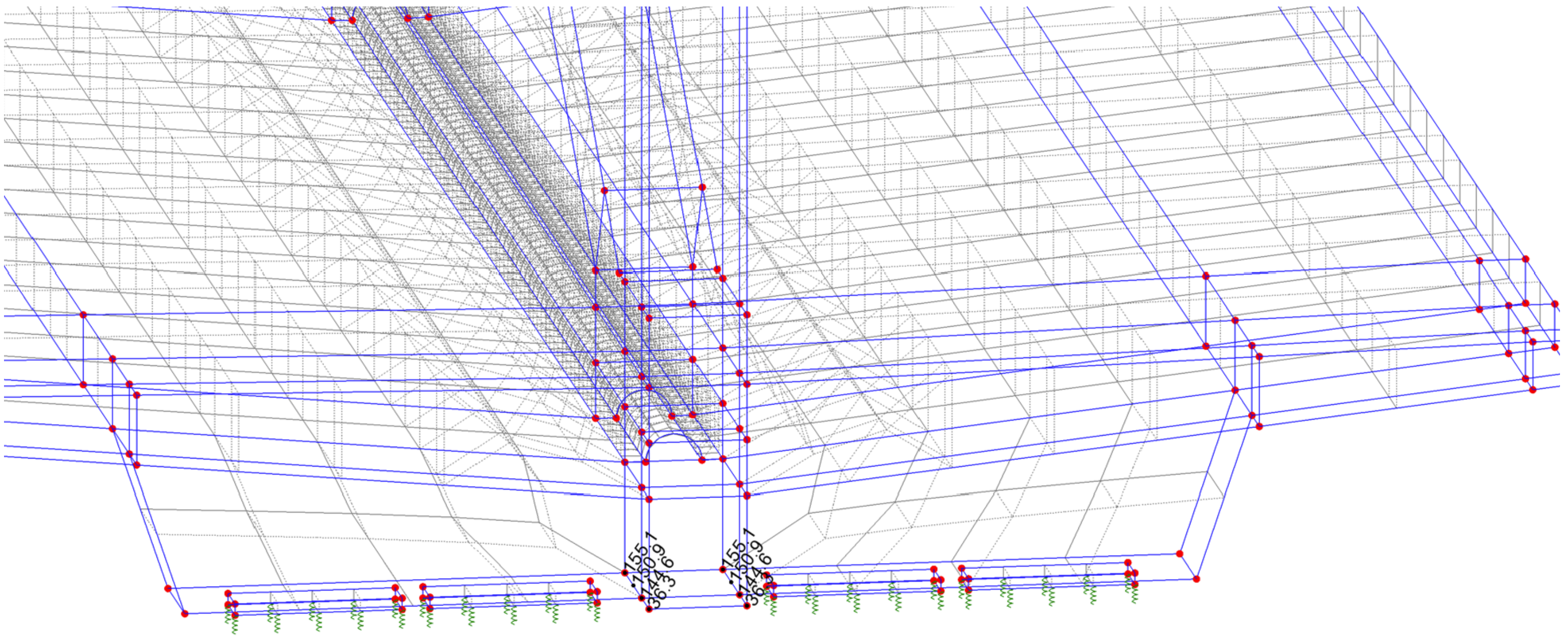
Conclusion

- Based on conservative calculations, it is concluded that the design meets LRFD strength requirements for this temporary condition …

- And therefore the is no safety concern relative to the observed cracks and minor spalls

3 Dimensional Finite Element Evaluations

- 3 dimensional (volume element) finite element evaluations were conducted to understand the local distribution of stress adjacent to the nodal area.
- This evaluation included the 4 shim locations placed at the end of the span movement.

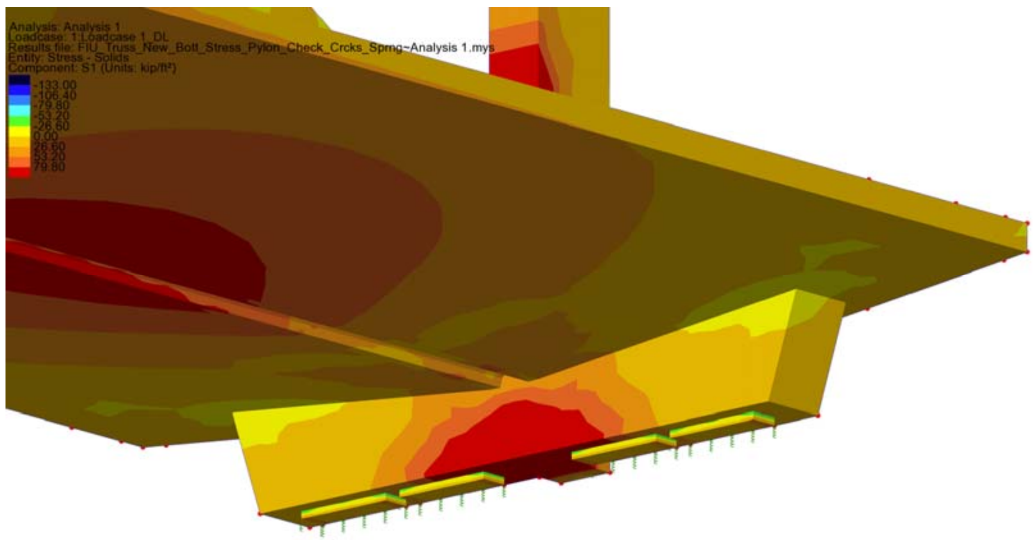


3 Dimensional Finite Element Evaluations

- Several different loads and load combinations were considered to understand both total state of stress and effects of some individual load components:
    - Total Self Weight + all PT (transverse, longitudinal, PT bars)
    - Stress changes from transferring the weight of the span onto the shims
    - Stress changes from stressing (or slackening) the PT bars in Member 11

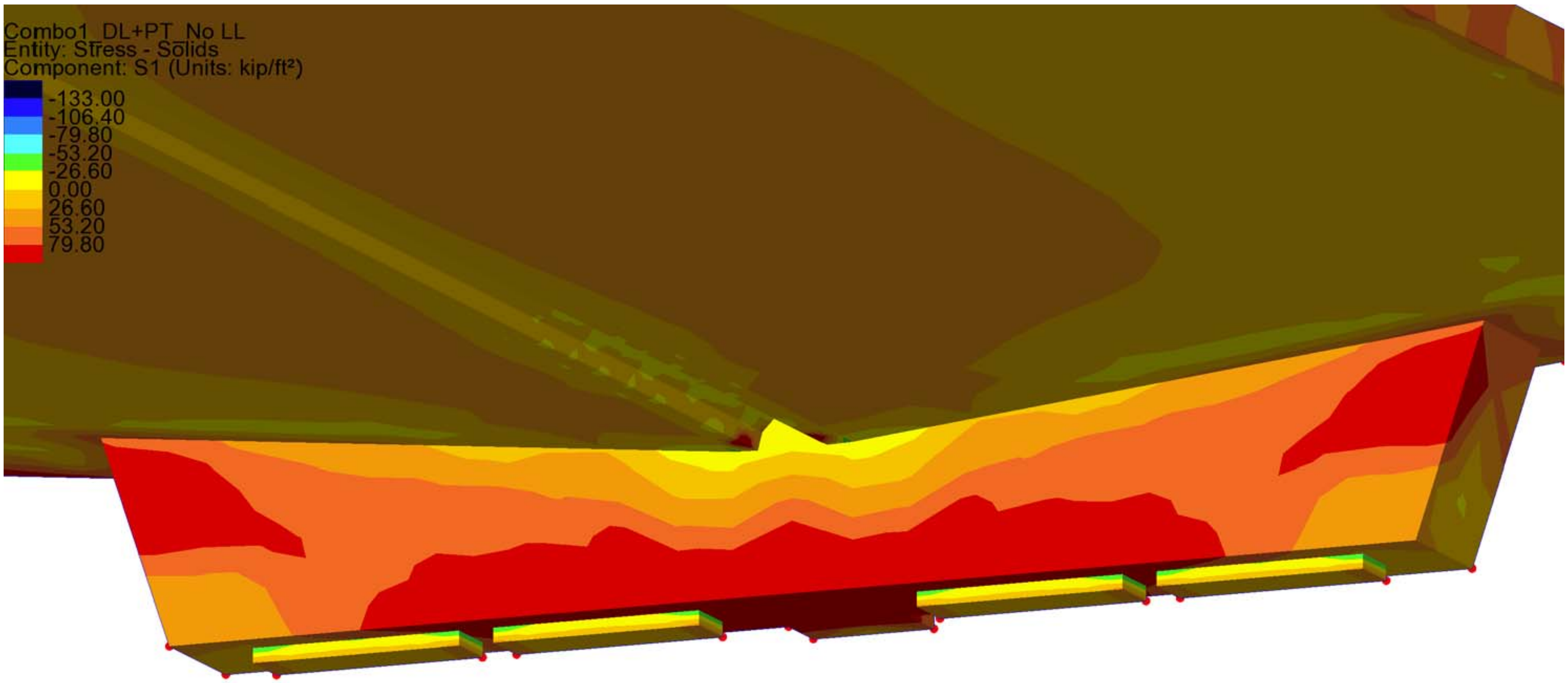
Stress change from placing onto shims

- When the span was placed onto the shims, are the stresses on the diaphragm essentially equal on the North and South Faces, or is one face more highly stressed?

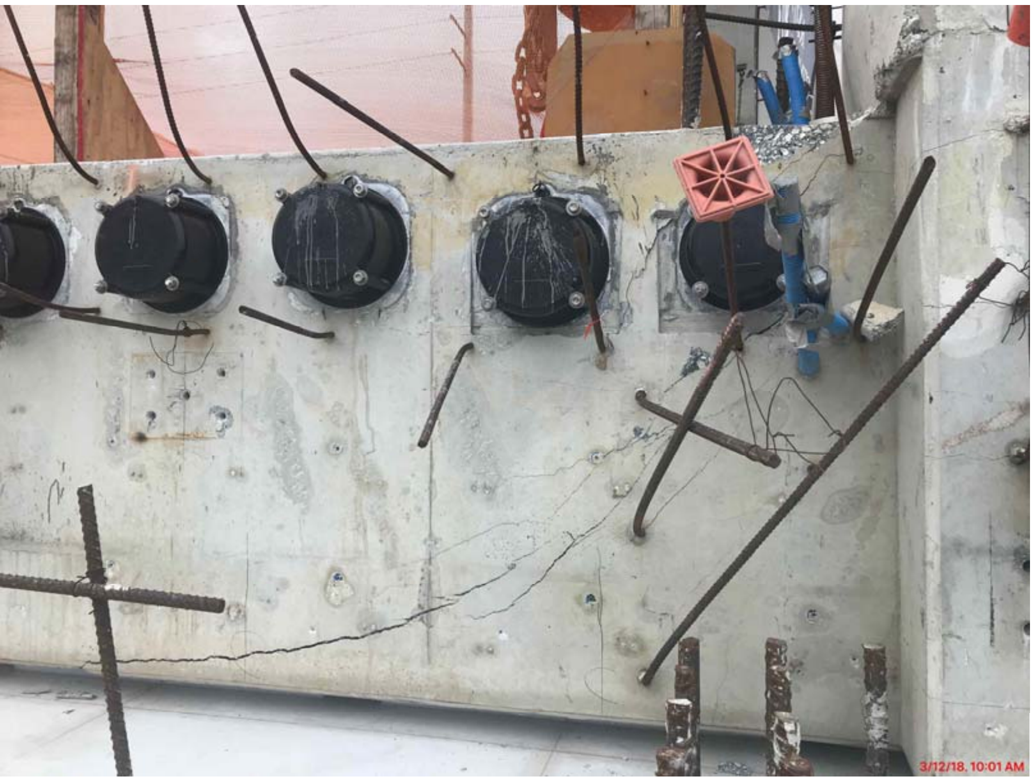


- The results indicate that the stress change on the North and South faces is relatively uniform when the load is transferred to the shims.

Total stress when on shims



Total stress when on shims



Diagonal Crack Pattern

- Both the hand calculations and the 3D volume element analysis concur that some cracks of the nature photographed are possible

- Why the cracks did not develop in the casting area, is likely related to having a substantial portion of the load carried in the central area

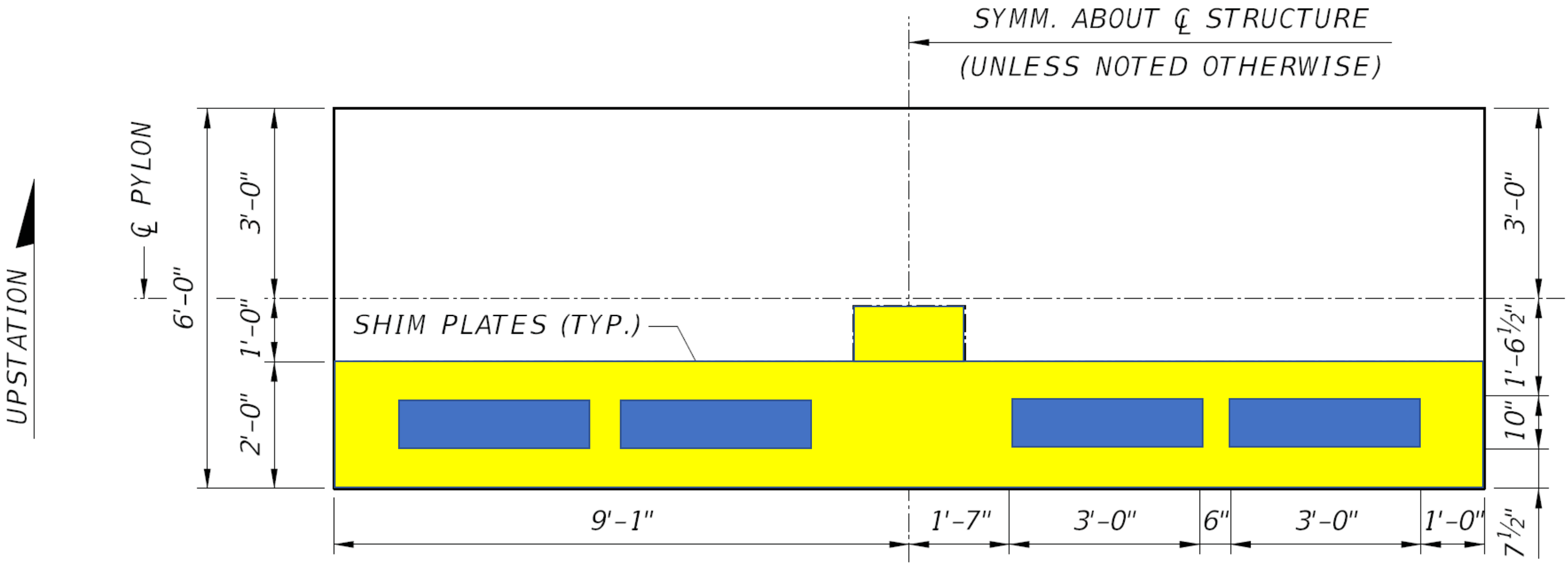
- Having the smaller crack width on the permanently exposed face is preferable, when compared to the other (to be embedded) diaphragm face.

Local Top Deck Spalls

It is unclear how a change in distribution of contact pressure on the bottom surface …

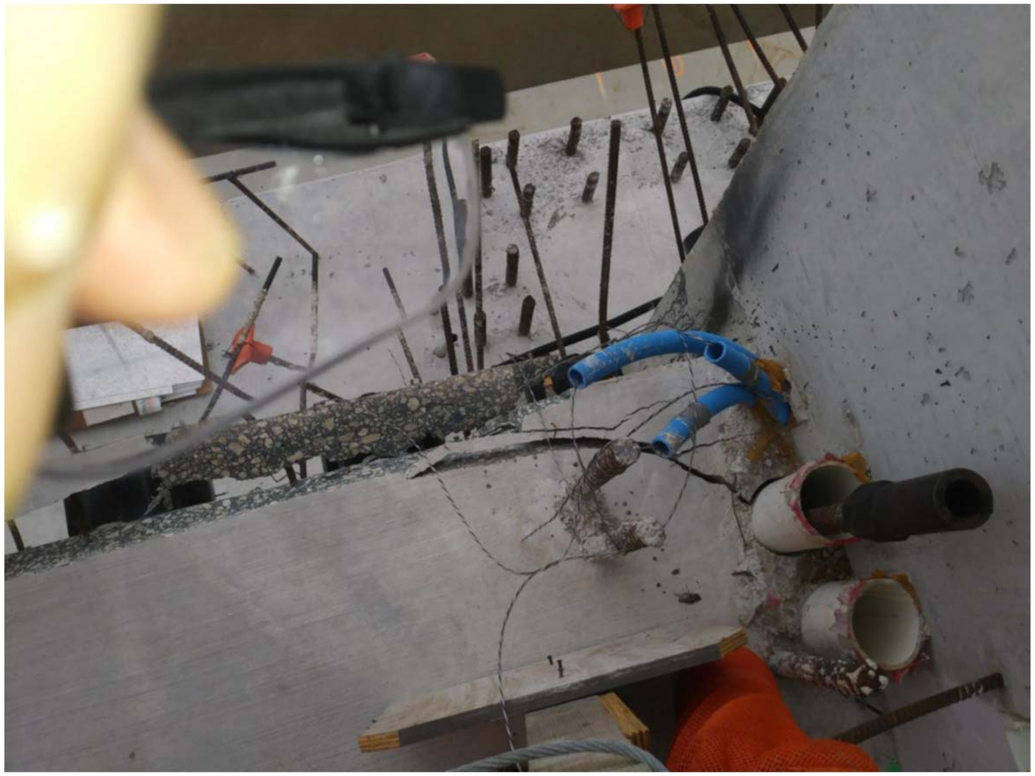
From Being supported like this for several weeks

To being supported on the bottom on the four shims …

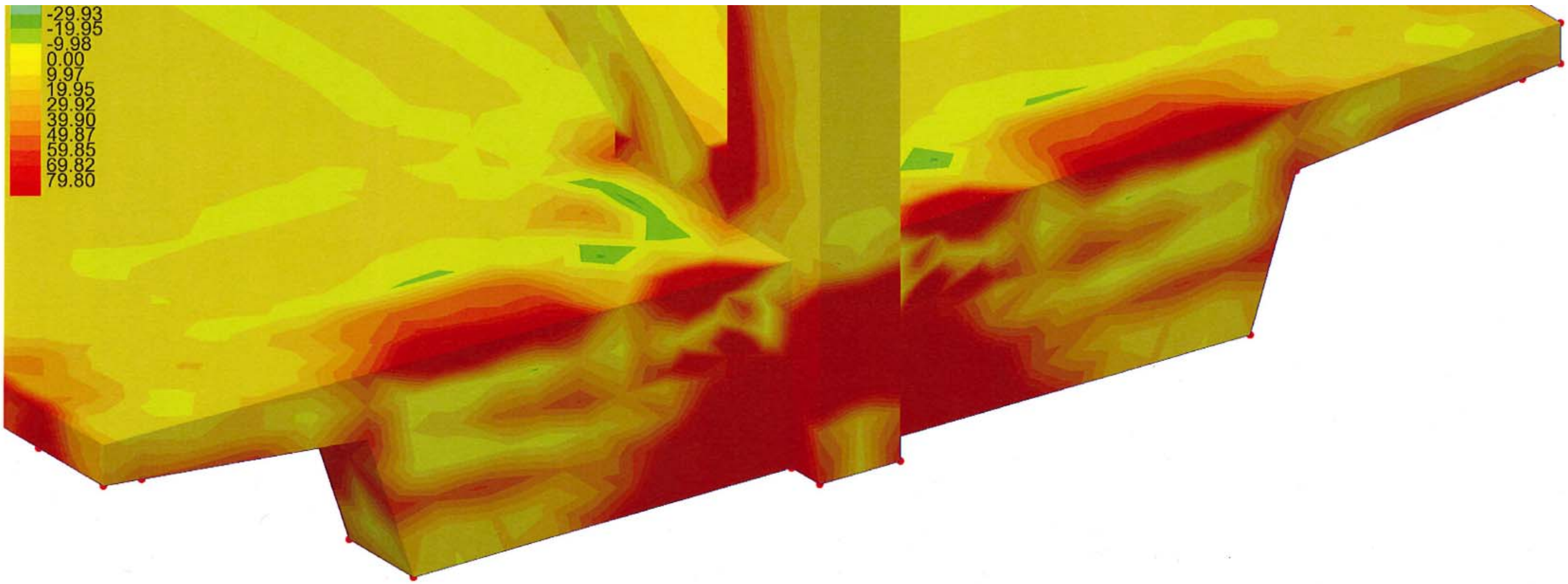


PYLON BASE - PLAN

Could possibly create these top spalls ???

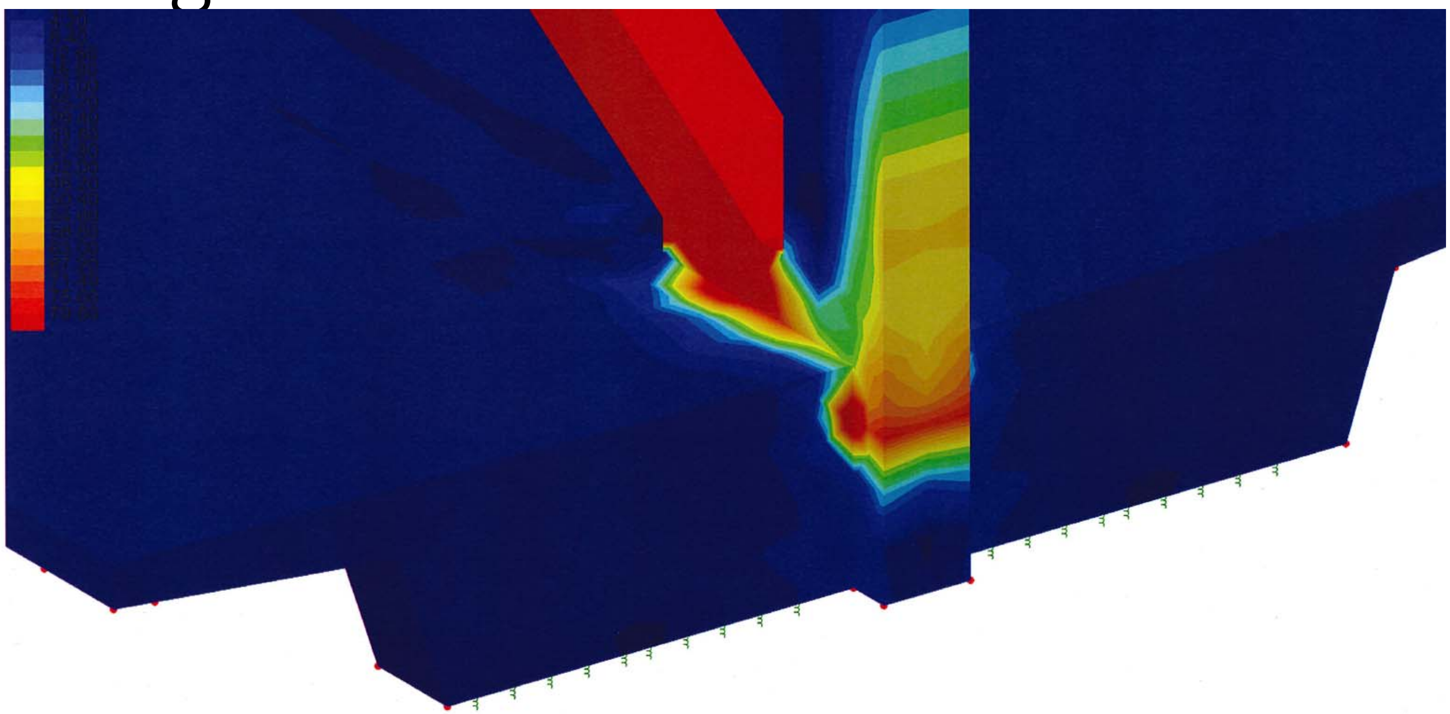


The total (plotted as Principle Tension) stresses from the 3D Model are



- This plot has the diagonal PT bars in Member 11 destressed

The change (principle stresses) solely from destressing the PT bars in Member 11 are



- The analyses (neither total stresses, nor PT bar only stresses) shows any spike in tensile stresses at the corner of the deck/diaphragm/member 12.

Conclusions and Recommendations

- The diagonal type cracks, in excess of FDOT criteria, should be sealed with approved methods and materials (Epoxy injection, etc.)
- The spalled areas have not been replicated by the engineering analyses. However …
- The spalled areas are minor and it is recommended that they be prepared using normal procedures and poured back along with the up coming “pylon diaphragm” pour (different from and prior to the back span on falsework pours)