FIGG Structural Analysis Presentation

University City Prosperity Project (BT-904)

Pedestrian Bridge over Tamiami Trail US 41

Contract No.: ARI73 FM No.: 434688-1

Date: 03/15/18 Time: 9:00 AM Location: MCM Field Office

- **<u>Overview</u>**
  - Discuss temporary construction loading condition and temporary mechanism to capture nodal zone
  - Summarized review of before and after movement bridge conditions
  - Overview of FIGG's analysis presentation
  - Questions & Answers discussion

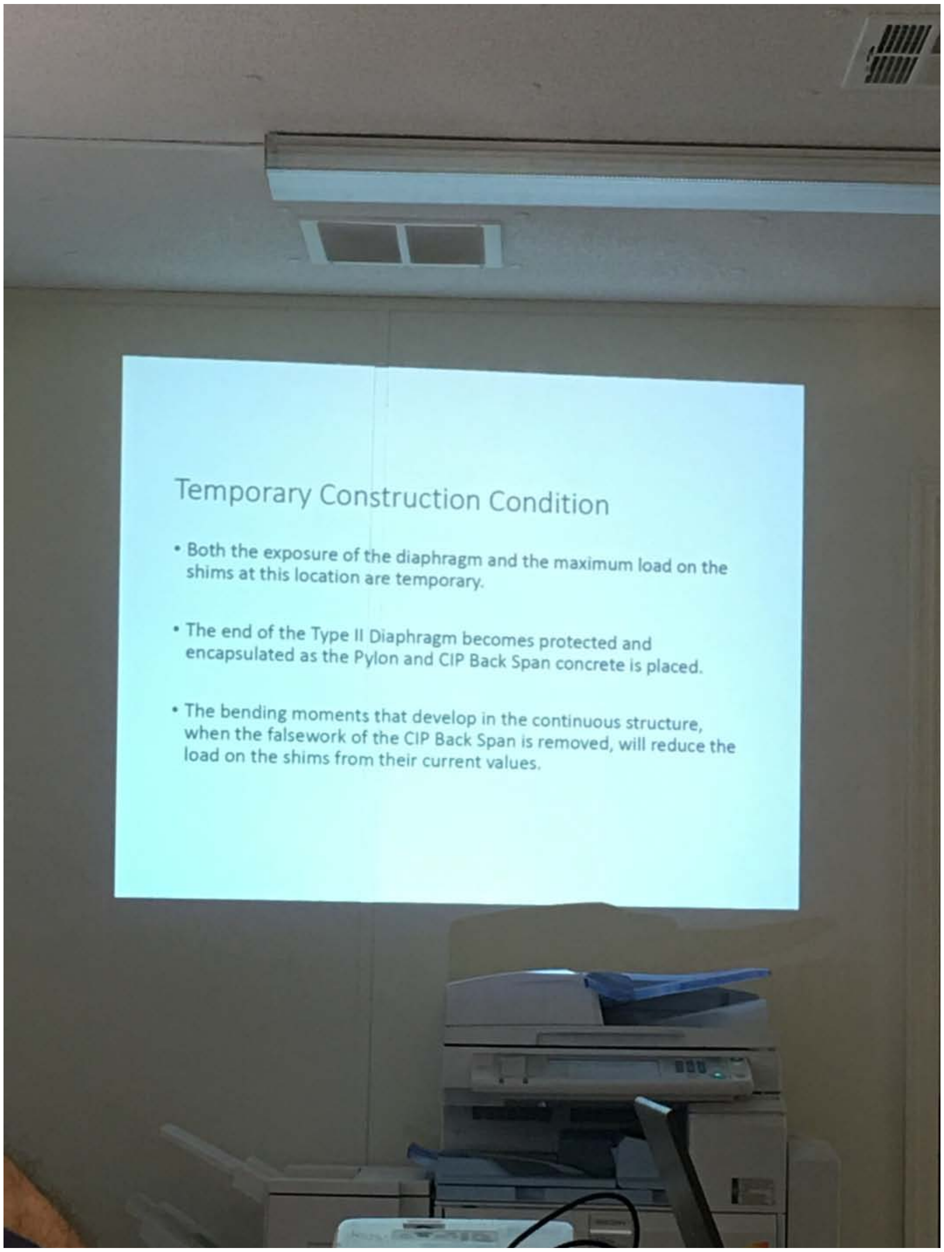
- **<u>Attendees</u>**
  - FIGG: Denney Pate, Eddy Leon, Dwight Dempsey (on the phone)
  - MCM: Rodrigo Isaza, Ernie Hernández, Pedro Cortes
  - FDOT: Alfredo Reyna
  - FIU: John Cal, Patrick Meagher
  - BPA/CEI: Jose Morales, Rafael Urdaneta, Carlos Chapman, Maria Christina Acosta

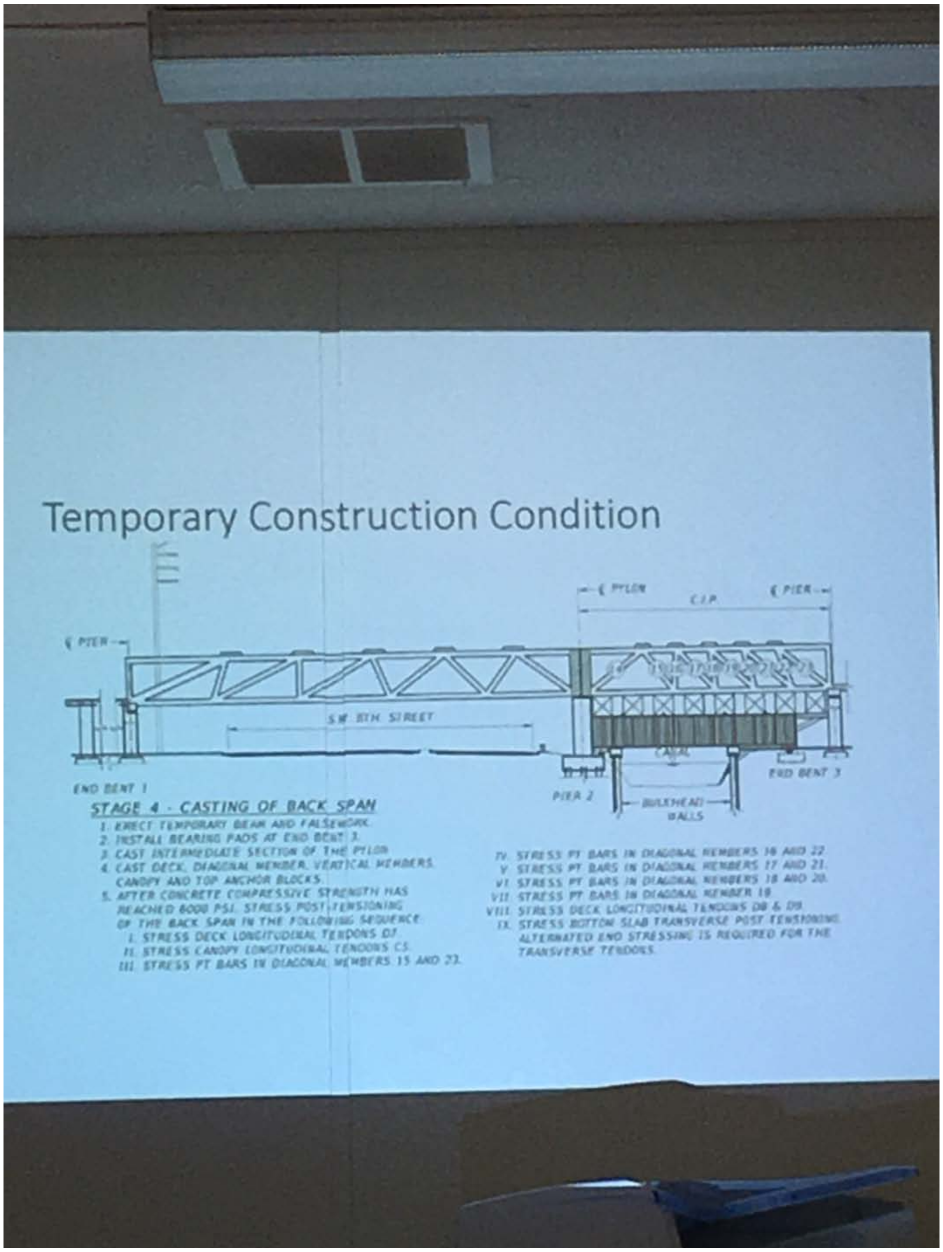
- **<u>FIGG's Presentation Summary</u>**
  - FIGG pointed out that the cracks look more significant in person than on photographs after site inspection performed prior to the presentation
  - Temporary construction loading condition
  - Bridge was loaded onto the permanent supports on 03/10/18
  - Immediately after the move, CEI & FIGG inspection showed nothing after distressing members 2 & 11
  - On 03/13/18, MCM e-mailed FIGG documentation regarding the cracks and FIGG instructed MCM to install the recommended temporary shims in the pylon base directly below member 12 (nodal area of members 11/12) between the permanent support shims
  - FIGG assured that there was no concern with safety of the span suspended over the road
  - The importance of the pylon diaphragm pour and back span construction was discussed
  - A temporary mechanism to capture the nodal zone and the time frame to deliver the plan was discussed
  - Refer to attached analysis presentation photographs (first few slides were not photographed)

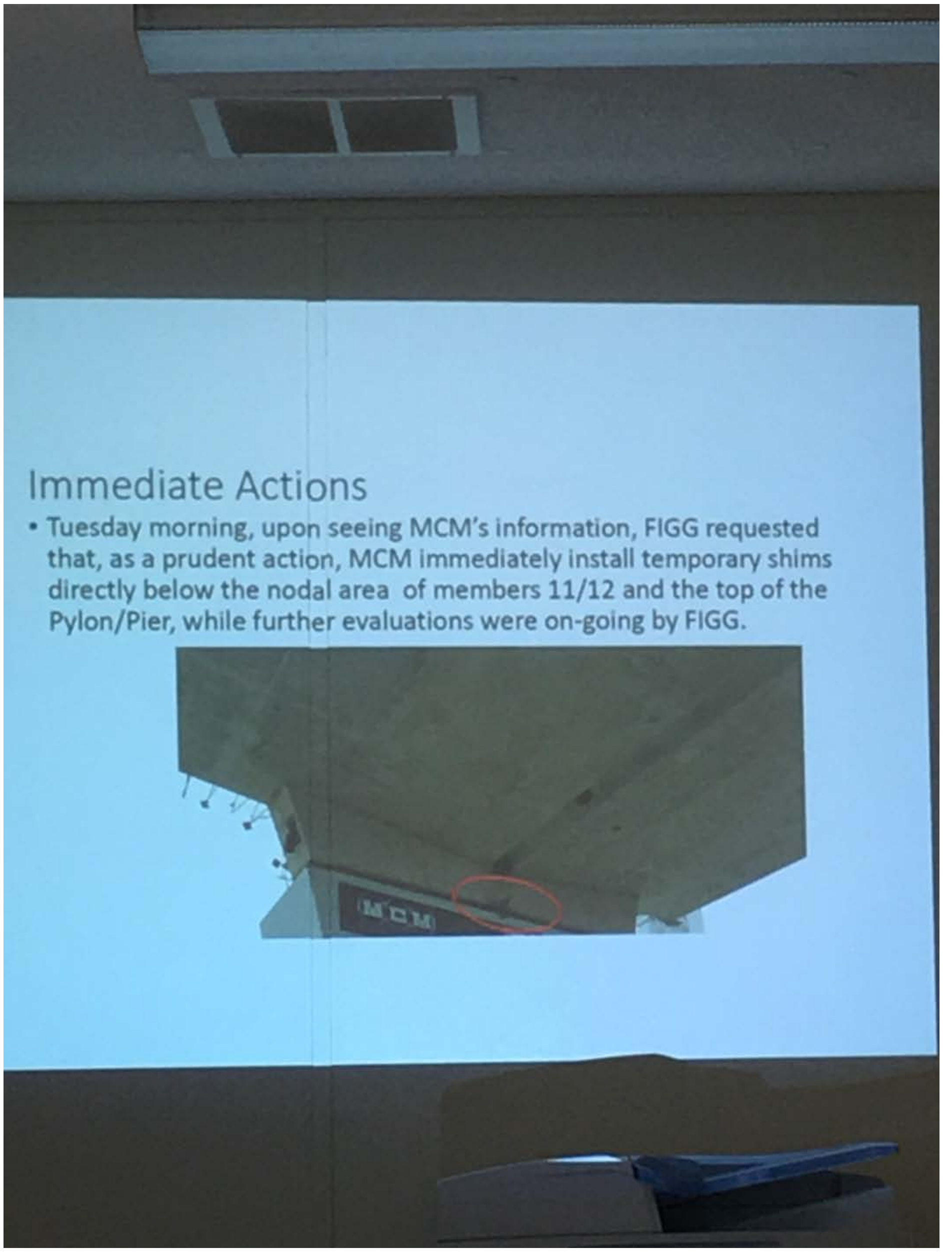
- **Questions & Answers**
  - CEI to FIGG: Do we need temporary shoring?
    - FIGG responded that it was not necessary. Rather than carry weight, carry load off that number/node. Steel channels to 10/9 node & PT Bars to capture some of that force which is better than vertical support. The diagonal member is what needs to be captured
  - FIGG mentioned that no repairs should be done now. Once back span is there, member 11 force will decrease, then repair can begin. FIGG also stated that the prudent action is to share the load carried to 9/10 and construct pylon diaphragm
  - CEI to FIGG: Will the mechanism to capture the load from the node have to be integrated with the pylon diaphragm and will it remain in the structure?
    - FIGG answered that the temporary mechanism to capture the node, preferably will not remain in the structure
  - MCM showed concerned of the timeline between using grout versus gray concrete as per workability.
    - FIGG commented that they will coordinate with MCM to address the concern. FIGG prefers the grout material but if it helps expedite the process to use otherwise, FIGG will be okay
  - MCM to FIGG: Will the temporary shim remain in the structure?
    - FIGG replied that the temporary shim shall be removed, however, if it cannot be removed, we will work something out
  - FIU to CEI: What is the CEI opinion on presentation analysis from FIGG?
    - FDOT to FIGG: FDOT requests a copy of FIGG's analysis presentation to give to their structural group
    - CEI: At this point we cannot comment, will follow up on this request and expedite in 2-3 days with Jake Perez and Luis M. Vargas
  - FIGG comments that the analysis predicts diagonal cracking
  - CEI to FIGG: Requested clarification on amount of transferred PT assumed for the nodal shear stability analysis
    - FIGG: Clamping action only on transverse strands
  - FIU commented to FIGG that nothing predicted this cracking
  - FIGG mentioned that the P.T. bars in their permanent condition have less stress than under construction condition
  - CEI to FIGG: Are there any restrictions of any load on that side?
    - FIGG answered that until further restraining of the node, no load other than necessary is needed. Also, member 11 is going to be tensioned today 03/15/18
  - CEI to FIGG/MCM: Will there be a crack monitoring plan? CEI had been monitoring the cracks and insisted that FIGG/MCM perform the crack monitoring as well
    - FIGG/MCM had no response
  - MCM to CEI: Have the cracks increased in length or depth?
    - CEI confirmed cracks have increased in length daily

- FDOT to FIGG: Are you going to continue to figure out why it happened?
    - FIGG responded that all we "know is that it just happened"
- MCM to FIGG: Will there be a further inspection inside the cracks?
    - FIGG answered that they don't want to core concrete out. They want to move forward and seal cracks before being covered
- FIGG insisted that right now to not do any repairing of cracks until stabilizing the node and pylon diaphragm. The rest of any corrective actions will be after construction of back span
- FIU to FIGG: this concrete is sticky (flowable) because of the titanium dioxide. FIU is concerned to be used under the pylon diaphragm
    - FIGG reassured that means and methods will be considered when used in pour
- FIU to FIGG: Why is the bridge less than 950 tons versus Barnhart's weight?
    - FIGG and CEI confirmed that it was built as per plans and the approximate weight of 950 tons included an increase factor
- MCM to FIGG: What is the time frame for temporary mechanism to capture nodal zone?
    - FIGG: Saturday
- CEI to FIGG: Are you staying for the P.T. procedure?
    - FIGG replied that they will not be staying for the procedure. FIGG was going back right after this presentation because they had work to do on this
- CEI to FIGG: Requesting a copy of the power point presentation
    - FIGG/MCM will provide
- CEI to FIGG: Has it been peer reviewed? CEI requested that it wanted more eyes on this and that the more eyes on this, the better
    - FIGG concurred
- CEI to FIGG/MCM: Provide to CEI the stressing procedure that will be performed on 03/15/18
    - MCM responded that we will provide to CEI. MCM clarified that VSL was currently on site to perform the stressing operation with the corresponding stressing procedure
- FIGG requested to MCM the compressive strength test results. MCM stated that laboratory results on concrete had exceeded the design compressive strength
- CEI to MCM: When do you have in your schedule the completion of the construction of the pylon diaphragm and back span and are you planning on rushing the completion of construction of them?
    - MCM responded that they are following the schedule but that they will expedite the construction of them
- FIU requested the progress meeting to be moved to 03/19/18

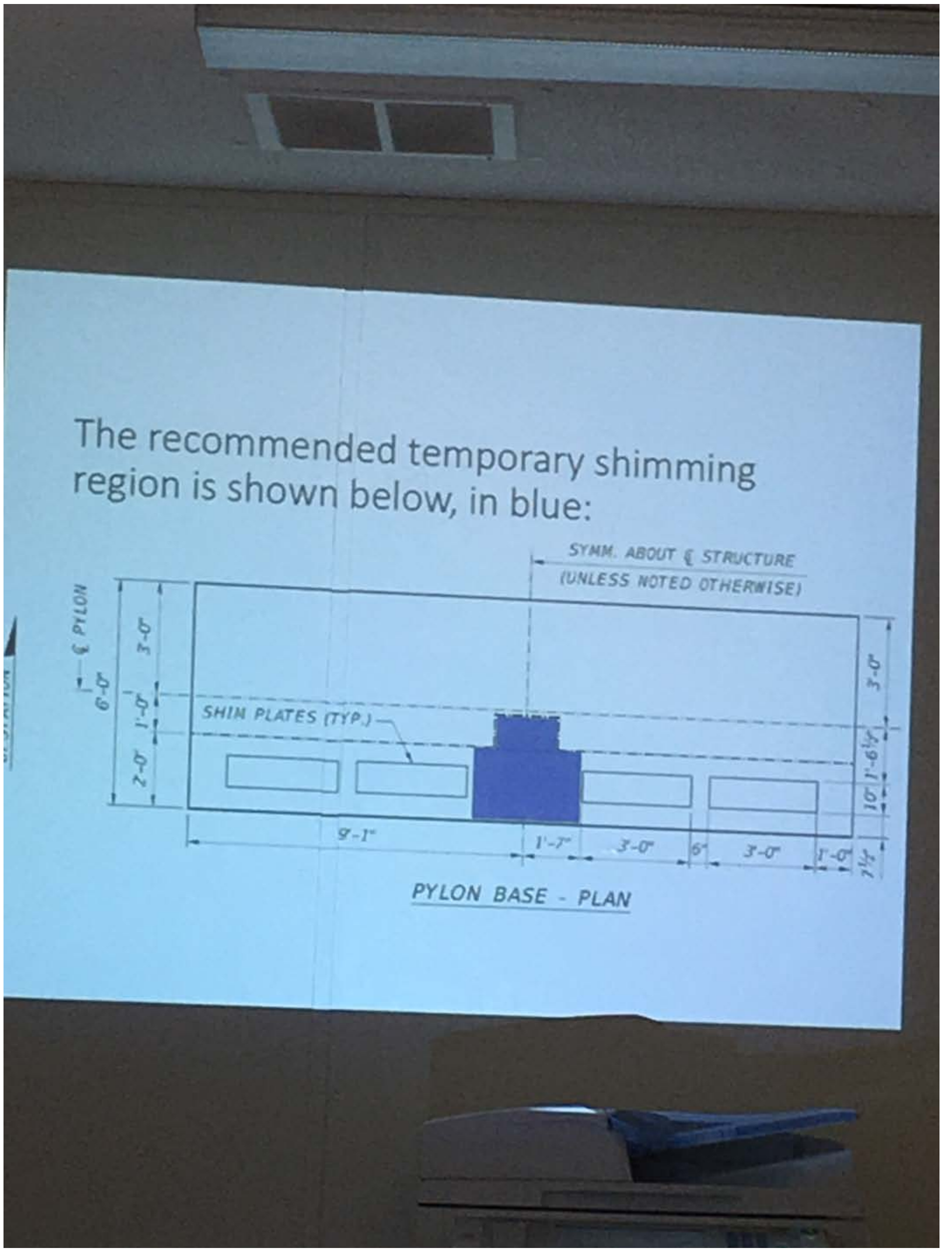
- **<u>Analysis Presentation Photographs</u>**

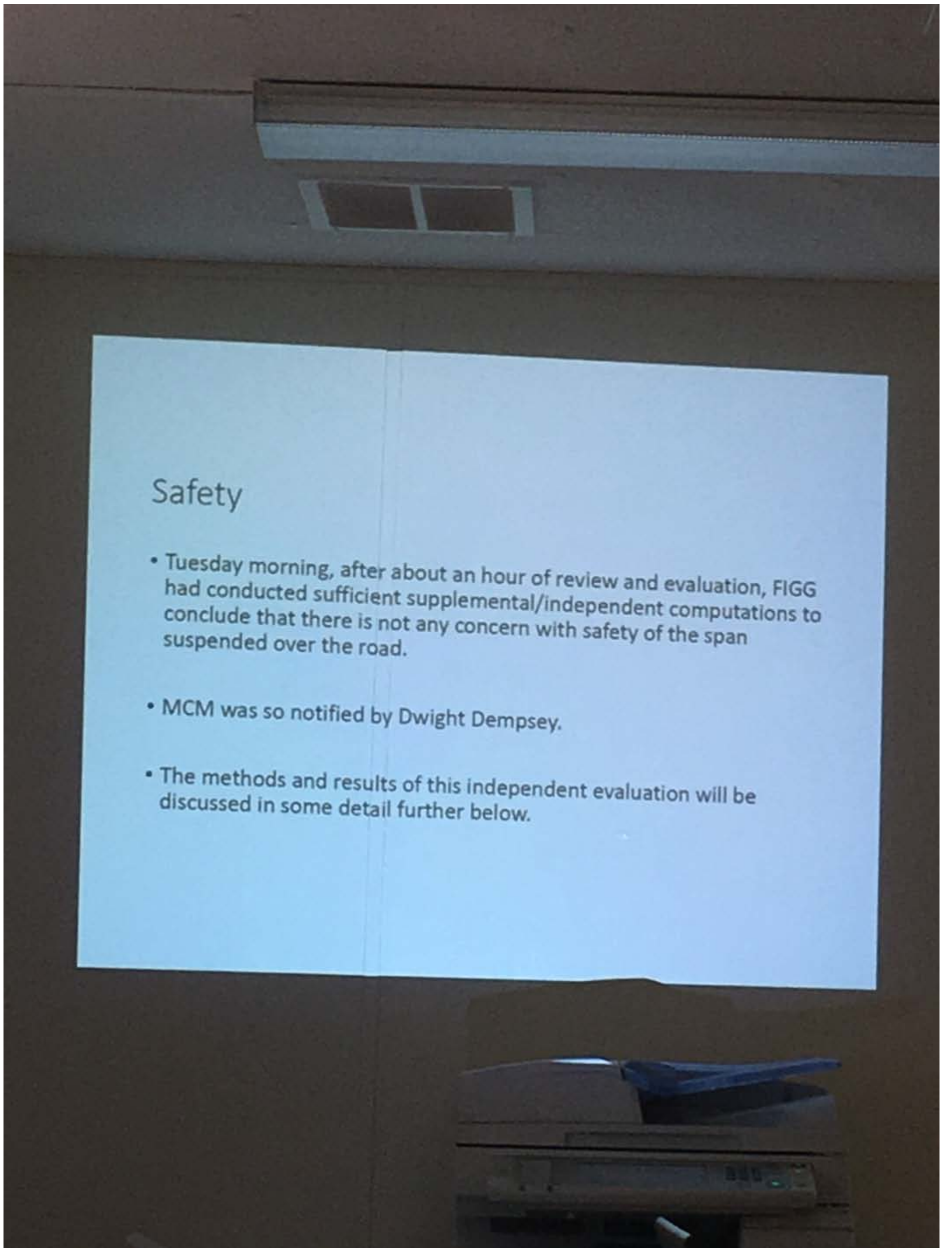


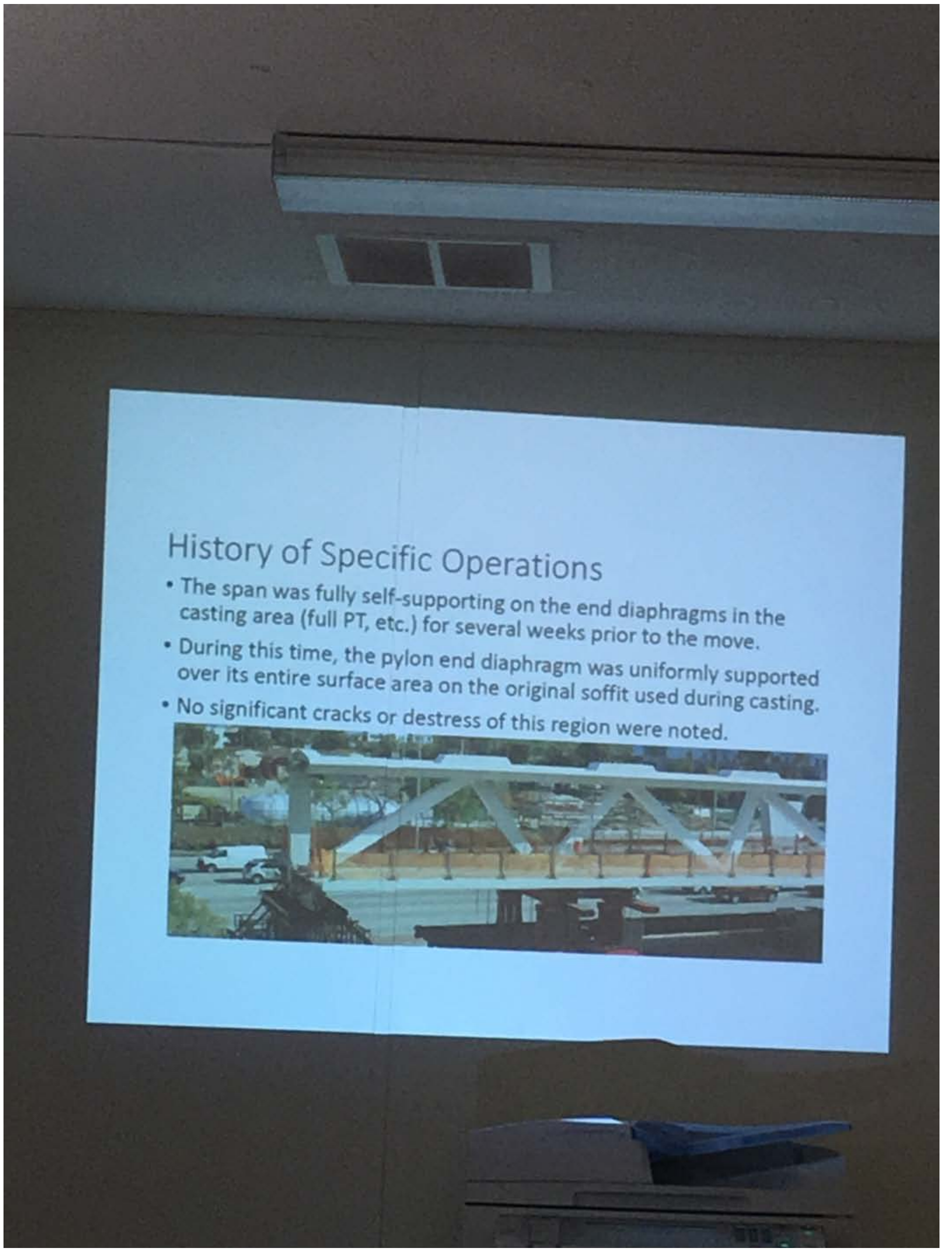

Temporary Construction Condition
℄ PIER
℄ PYLON
C.I.P.
℄ PIER
S.W. 8TH STREET
END BENT 1
PIER 2
BULKHEAD WALLS
END BENT 3
STAGE 4 - CASTING OF BACK SPAN
1. ERECT TEMPORARY BEAM AND FALSEWORK.
2. INSTALL BEARING PADS AT END BENT 3.
3. CAST INTERMEDIATE SECTION OF THE PYLON.
4. CAST DECK, DIAGONAL MEMBER, 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.

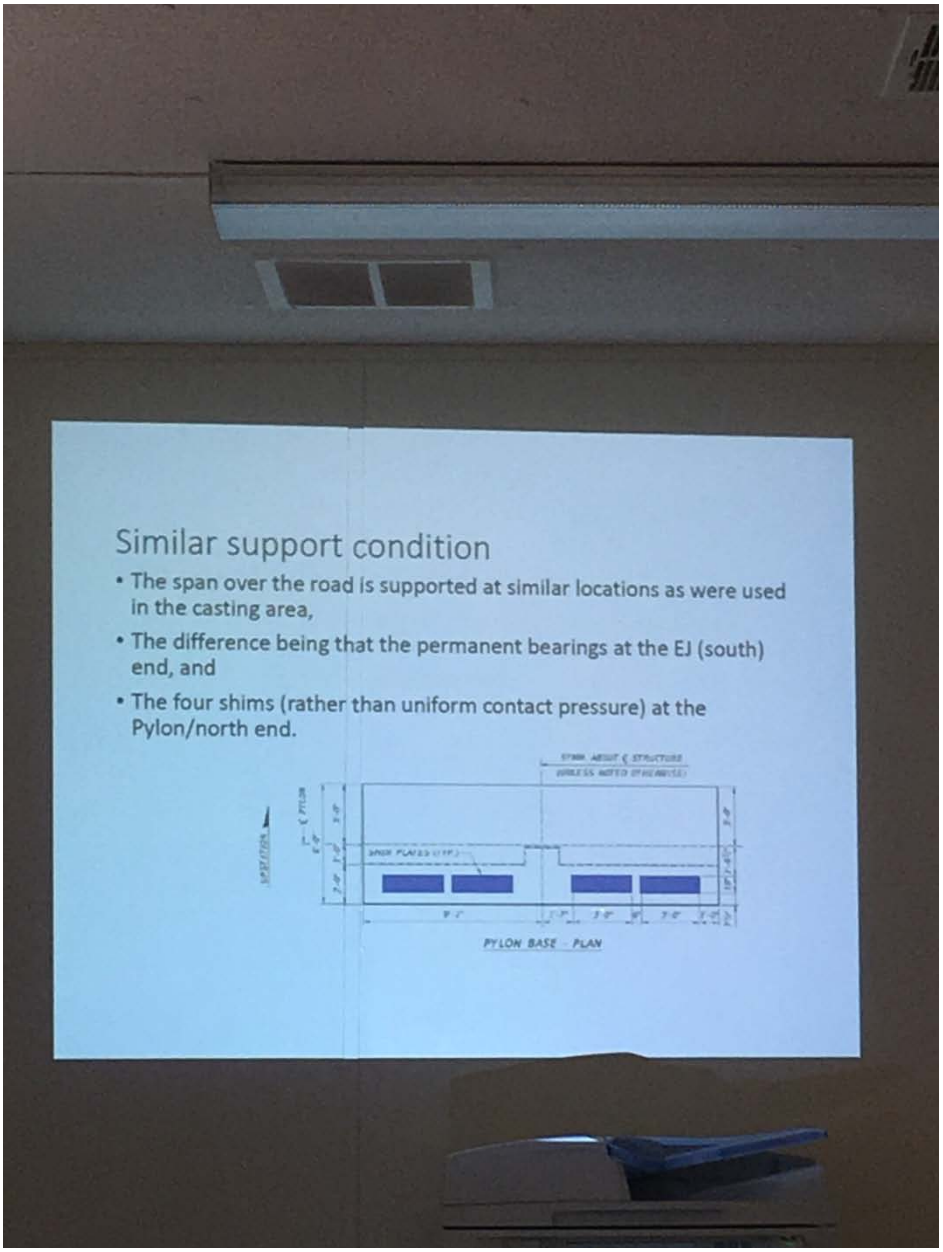

Immediate Actions
• Tuesday morning, upon seeing MCM's information, FIGG requested that, as a prudent action, MCM immediately install temporary shims directly below the nodal area of members 11/12 and the top of the Pylon/Pier, while further evaluations were on-going by FIGG.
MCM

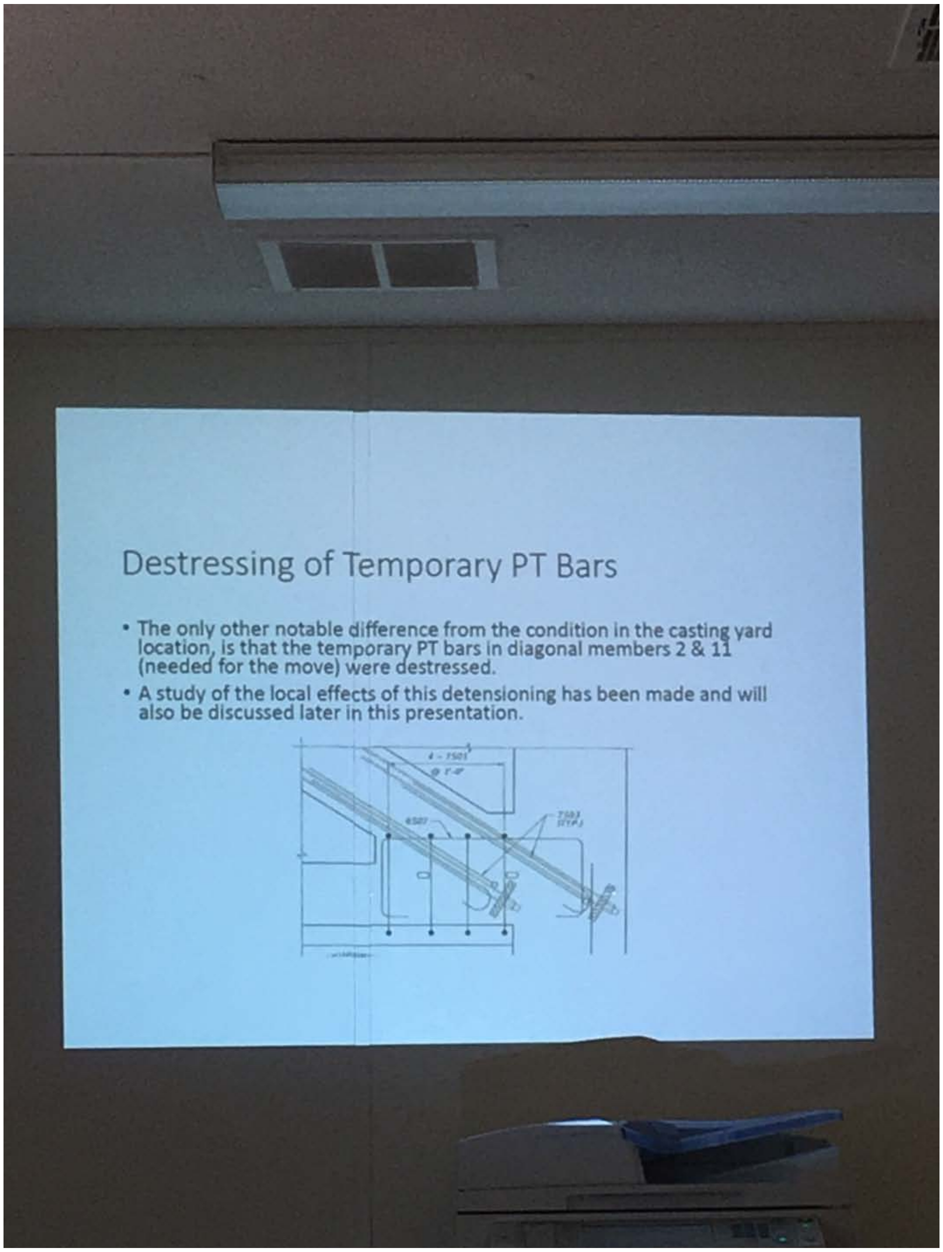
The shims placed during the span move were:
SYMM. ABOUT ℄ STRUCTURE
(UNLESS NOTED OTHERWISE)
℄ PYLON
6'-0"
3'-0"
1'-0"
2'-0"
SHIM PLATES (TYP.)
3'-0"
1'-6½"
10"
7½"
9'-1"
1'-7"
3'-0"
6"
3'-0"
1'-0"
PYLON BASE - PLAN

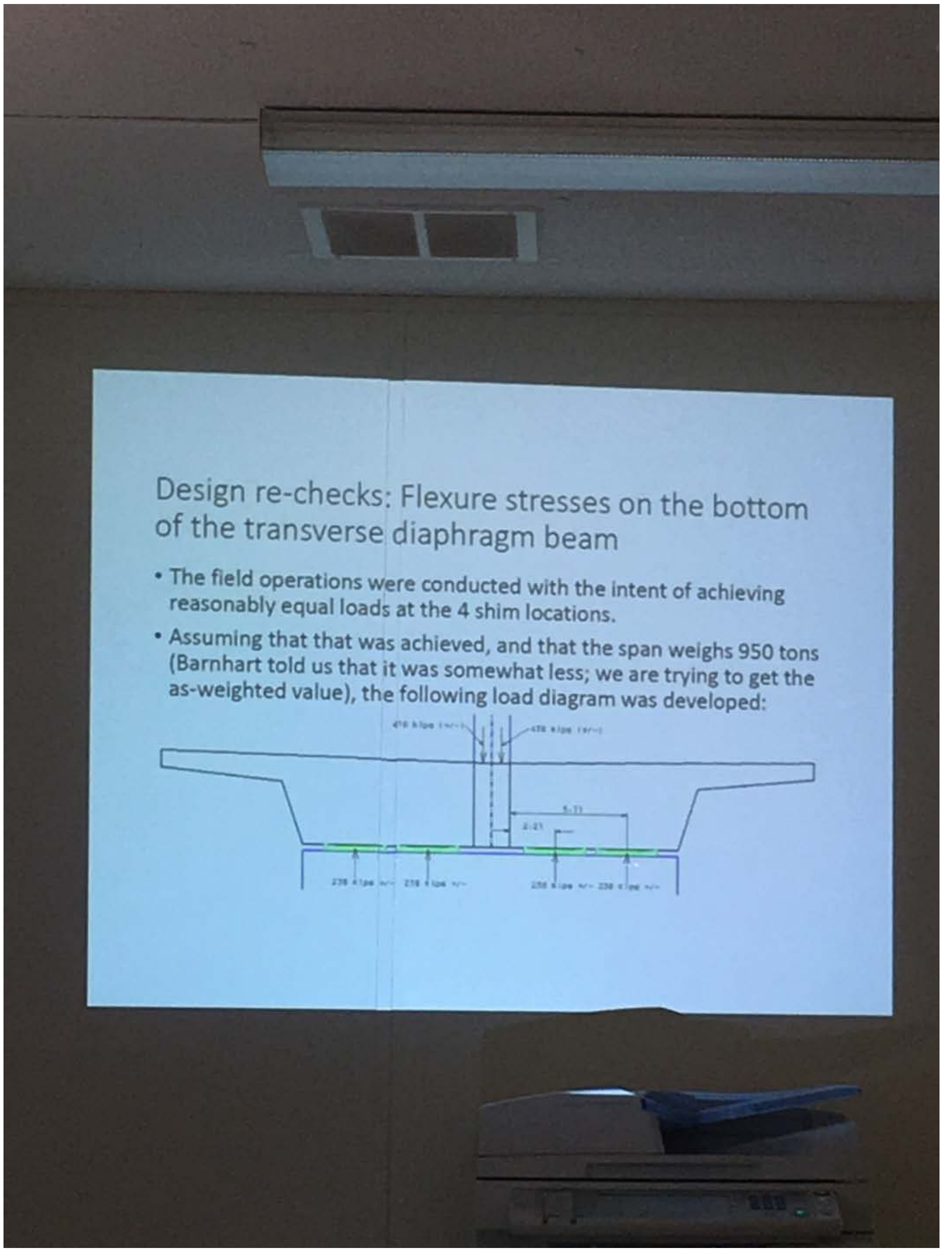

The recommended temporary shimming region is shown below, in blue:
SYMM. ABOUT ℄ STRUCTURE
(UNLESS NOTED OTHERWISE)
℄ PYLON
6'-0"
3'-0"
1'-0"
2'-0"
SHIM PLATES (TYP.)
3'-0"
1'-6½"
10"
9'-1"
1'-7"
3'-0"
6"
3'-0"
1'-0"
7½"
PYLON BASE - PLAN

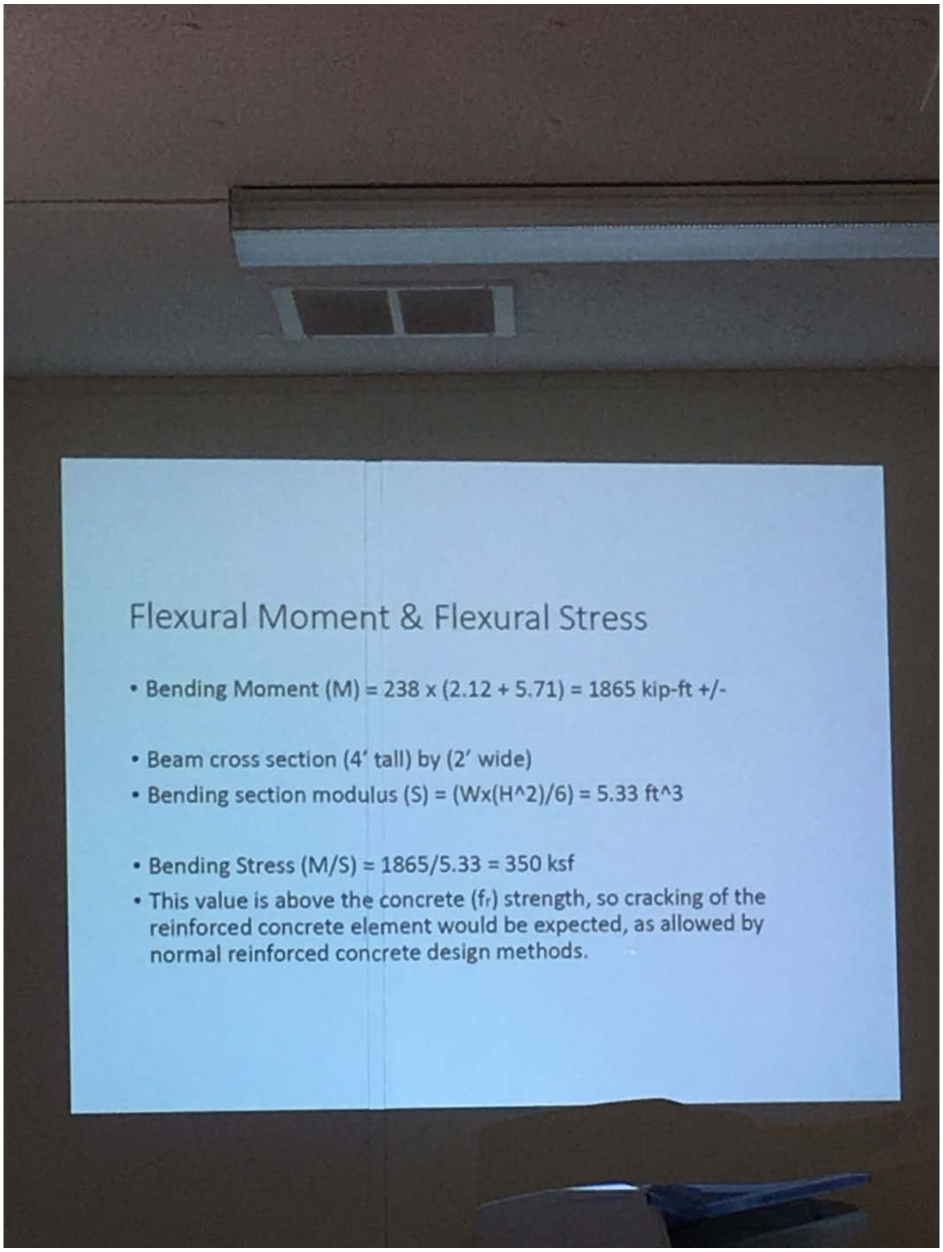

Safety
• Tuesday morning, after about an hour of review and evaluation, FIGG had conducted sufficient supplemental/independent computations to conclude that there is not any concern with safety of the span suspended over the road.
• MCM was so notified by Dwight Dempsey.
• The methods and results of this independent evaluation will be discussed in some detail further below.

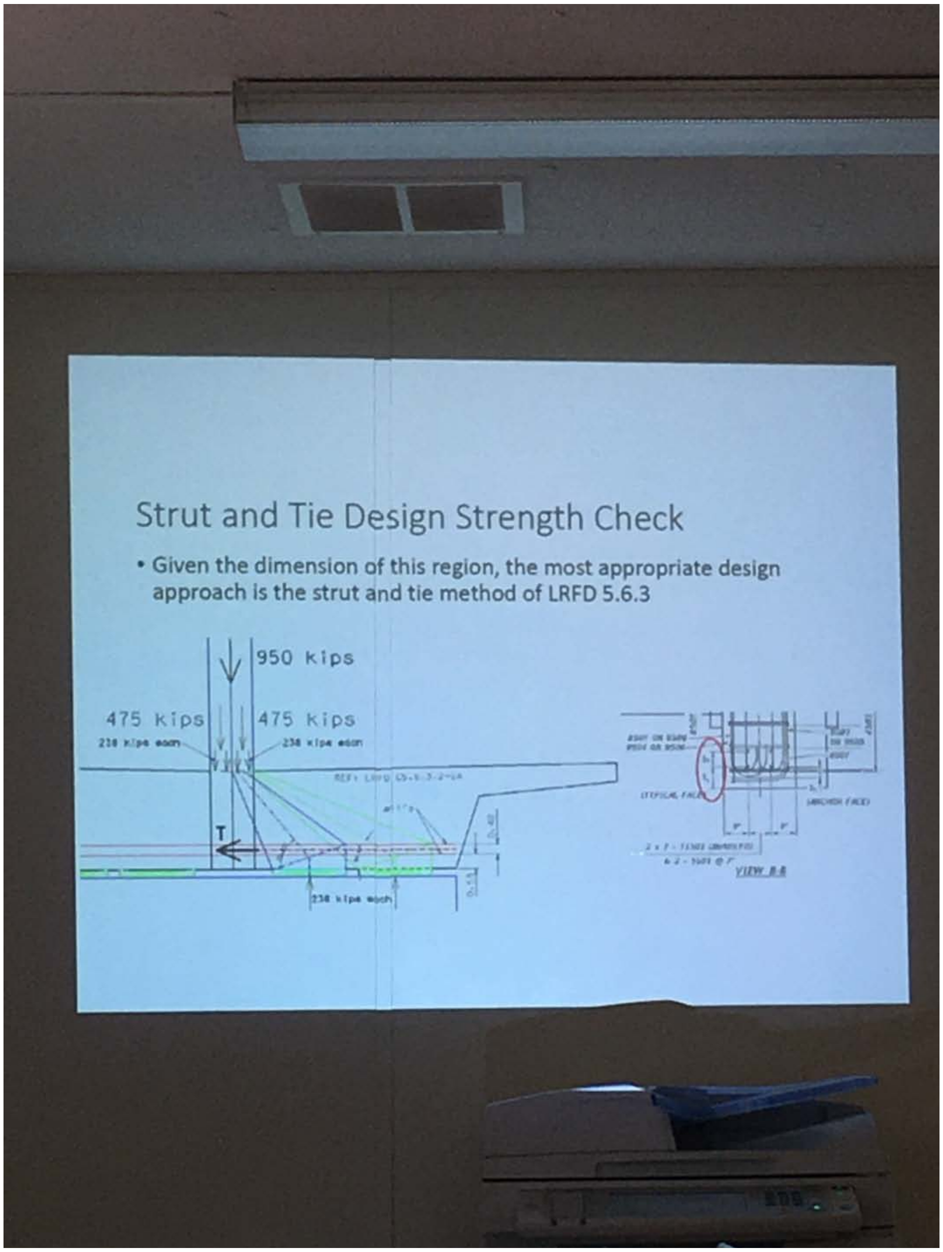

History of Specific Operations
• The span was fully self-supporting on the end diaphragms in the casting area (full PT, etc.) for several weeks prior to the move.
• During this time, the pylon end diaphragm was uniformly supported over its entire surface area on the original soffit used during casting.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

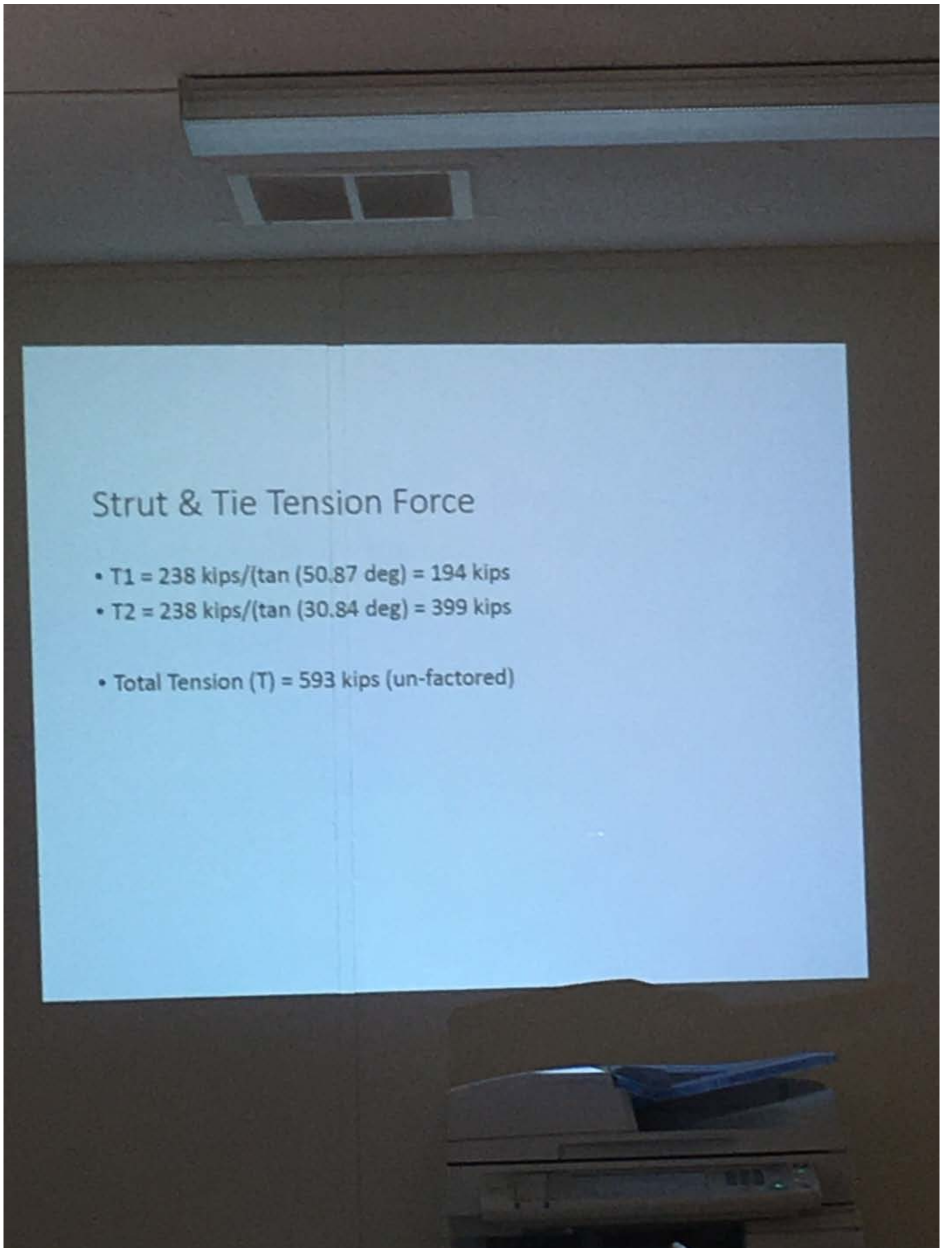

Similar support condition
• The span over the road is supported at similar locations as were used in the casting area,
• The difference being that the permanent bearings at the EJ (south) end, and
• The four shims (rather than uniform contact pressure) at the Pylon/north end.
PYLON BASE - PLAN

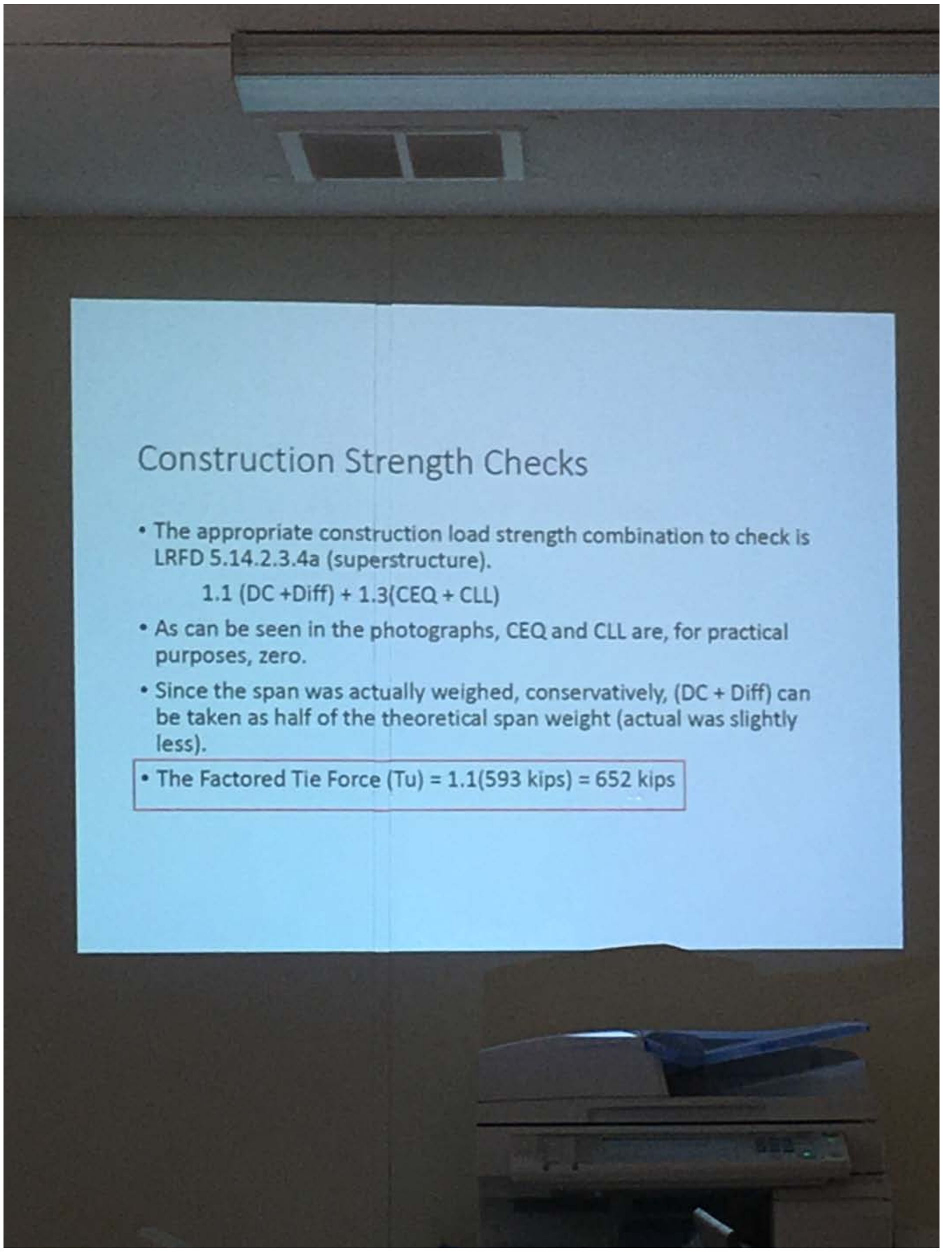

Destressing of Temporary PT Bars
• The only other notable difference from the condition in the casting yard location, is that the temporary PT bars in diagonal members 2 & 11 (needed for the move) were destressed.
• A study of the local effects of this detensioning has been made and will also be discussed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

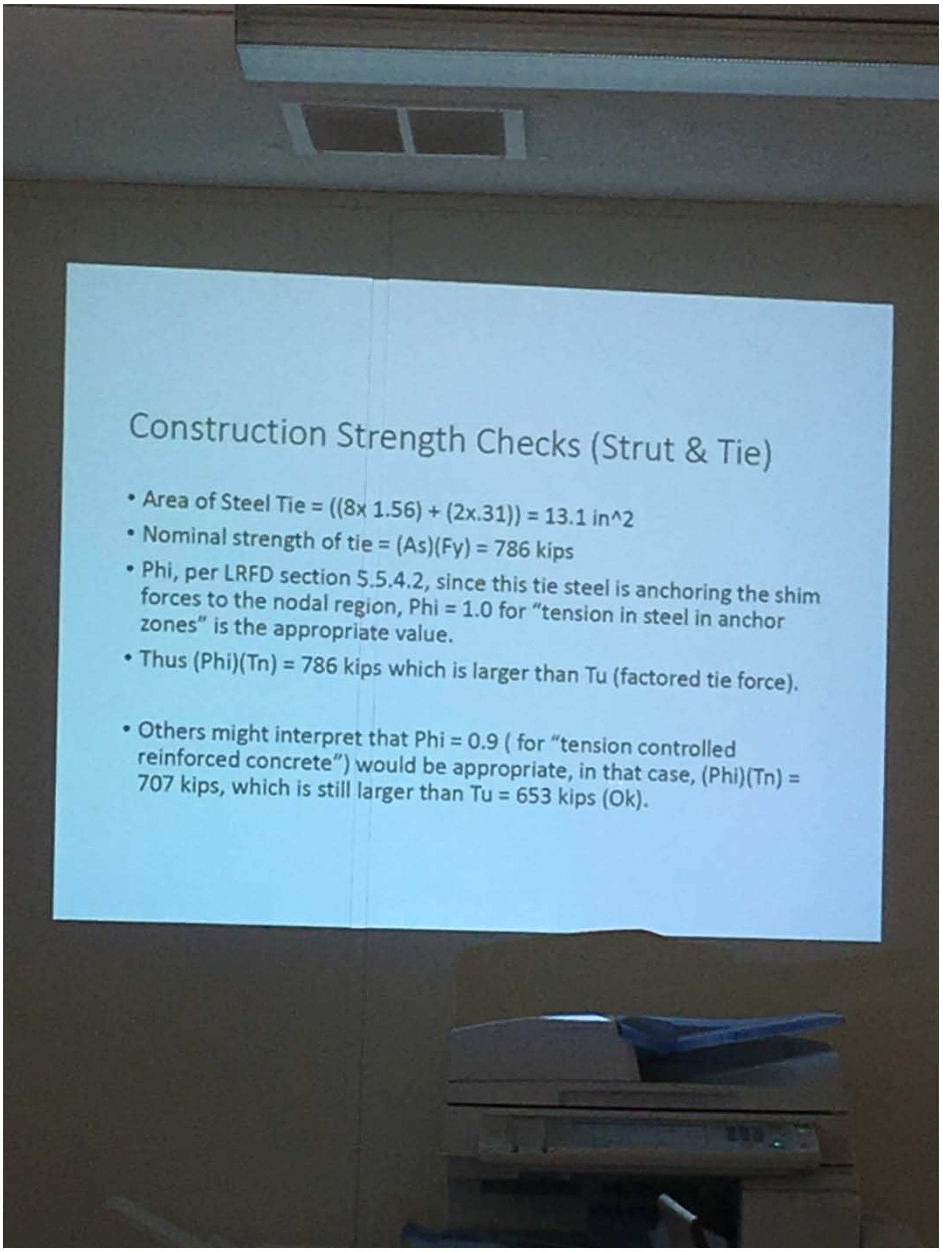

Design re-checks: Flexure stresses on the bottom of the transverse diaphragm beam
• The field operations were conducted with the intent of achieving reasonably equal loads at the 4 shim locations.
• Assuming that that was achieved, and that the span weighs 950 tons (Barnhart told us that it was somewhat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

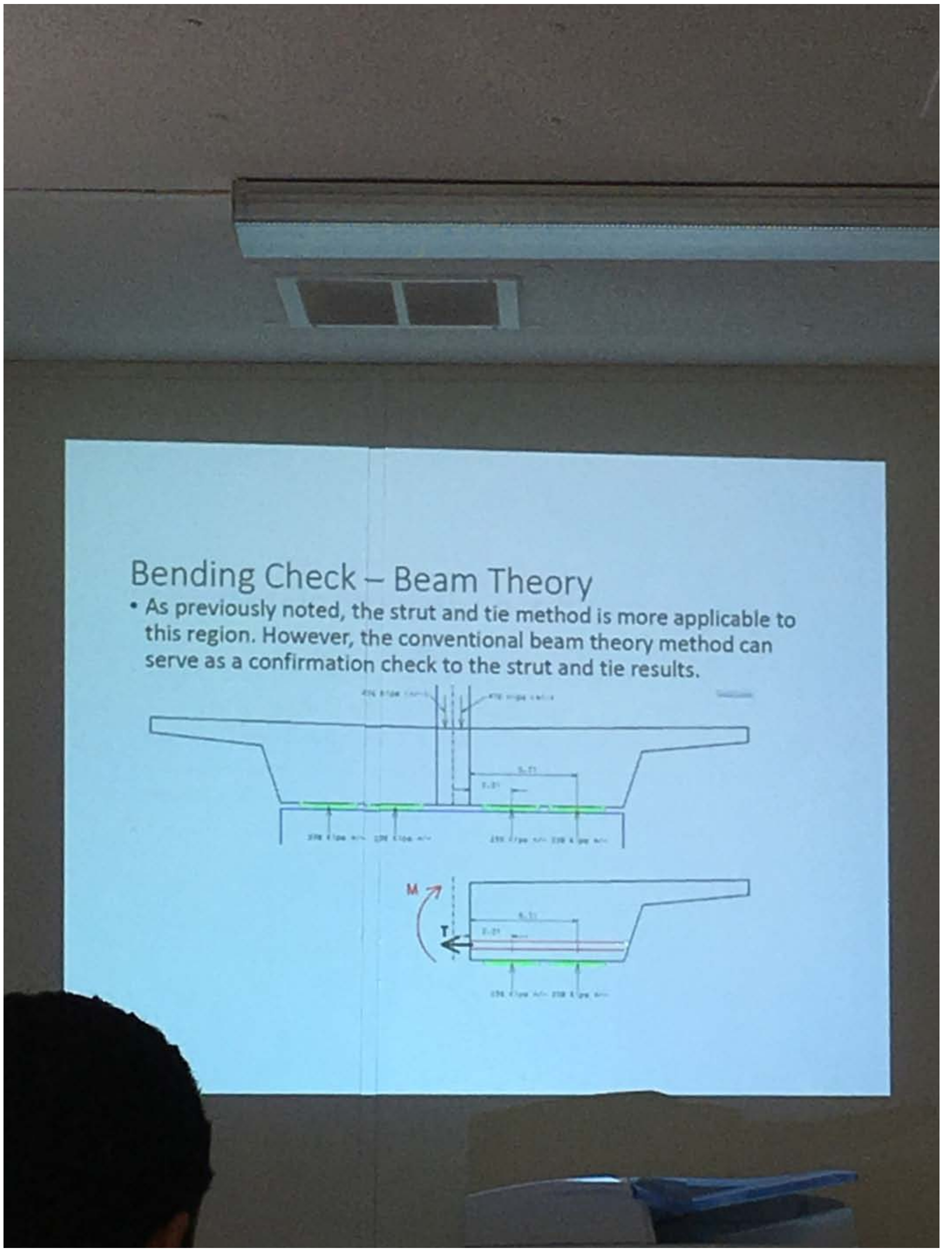

Flexural Moment & Flexural Stress
• Bending Moment (M) = 238 x (2.12 + 5.71) = 1865 kip-ft +/-
• Beam cross section (4' tall) by (2' wide)
• Bending section modulus (S) = (Wx(H^2)/6) = 5.33 ft^3
• Bending Stress (M/S) = 1865/5.33 = 350 ksf
• This value is above the concrete (fr) strength, so cracking 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
950 kips
475 kips
475 kips
T

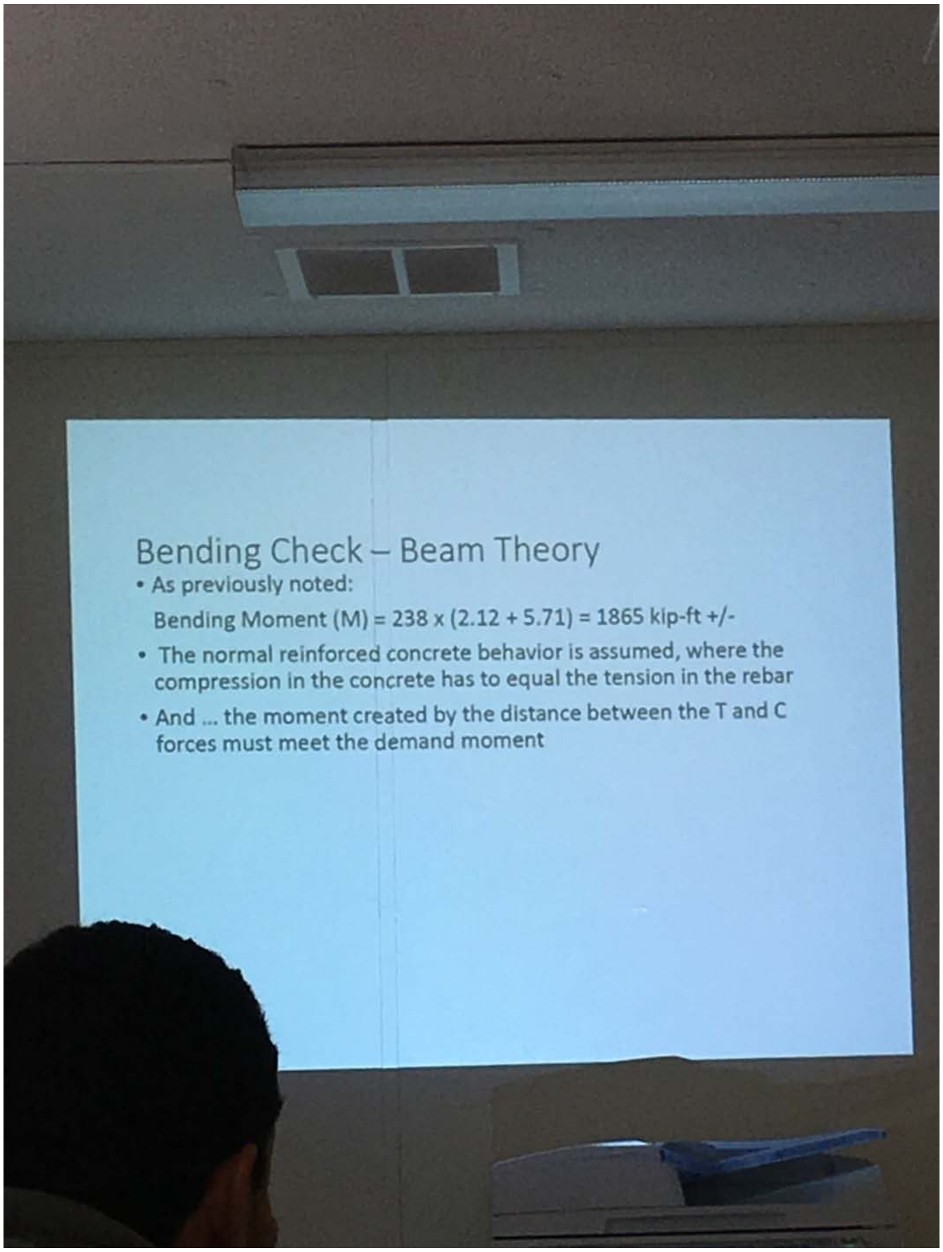

Strut & Tie Tension Force
• T1 = 238 kips/(tan (50.87 deg) = 194 kips
• T2 = 238 kips/(tan (30.84 deg) = 399 kips
• Total Tension (T) = 593 kips (un-factored)

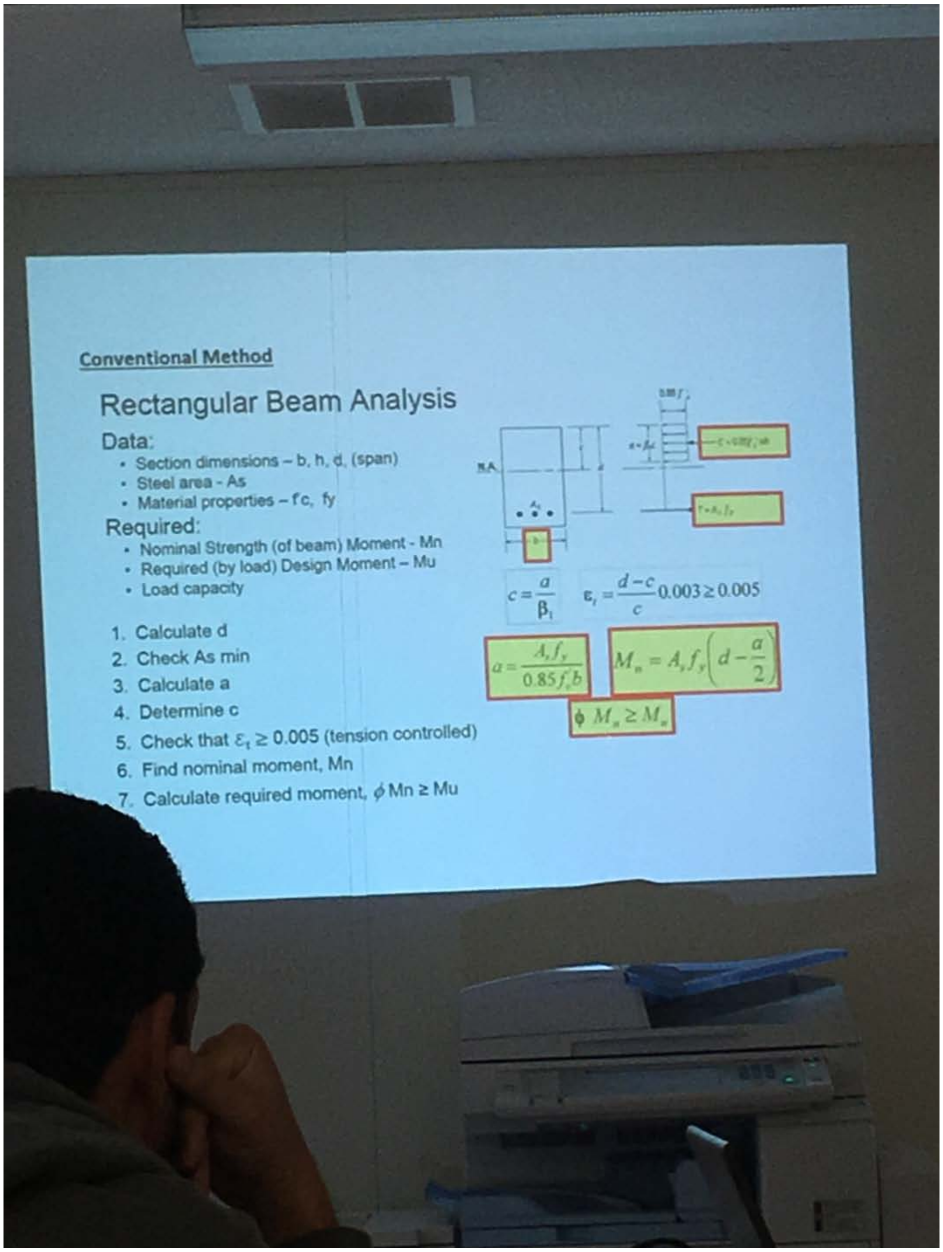

Construction Strength Checks
• The appropriate construction load strength combination to check is LRFD 5.14.2.3.4a (superstructure).
1.1 (DC +Diff) + 1.3(CEQ + CLL)
• As can be seen in the photographs, CEQ and CLL are, for practical purposes, zero.
• Since the span was actually weighed, conservatively, (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

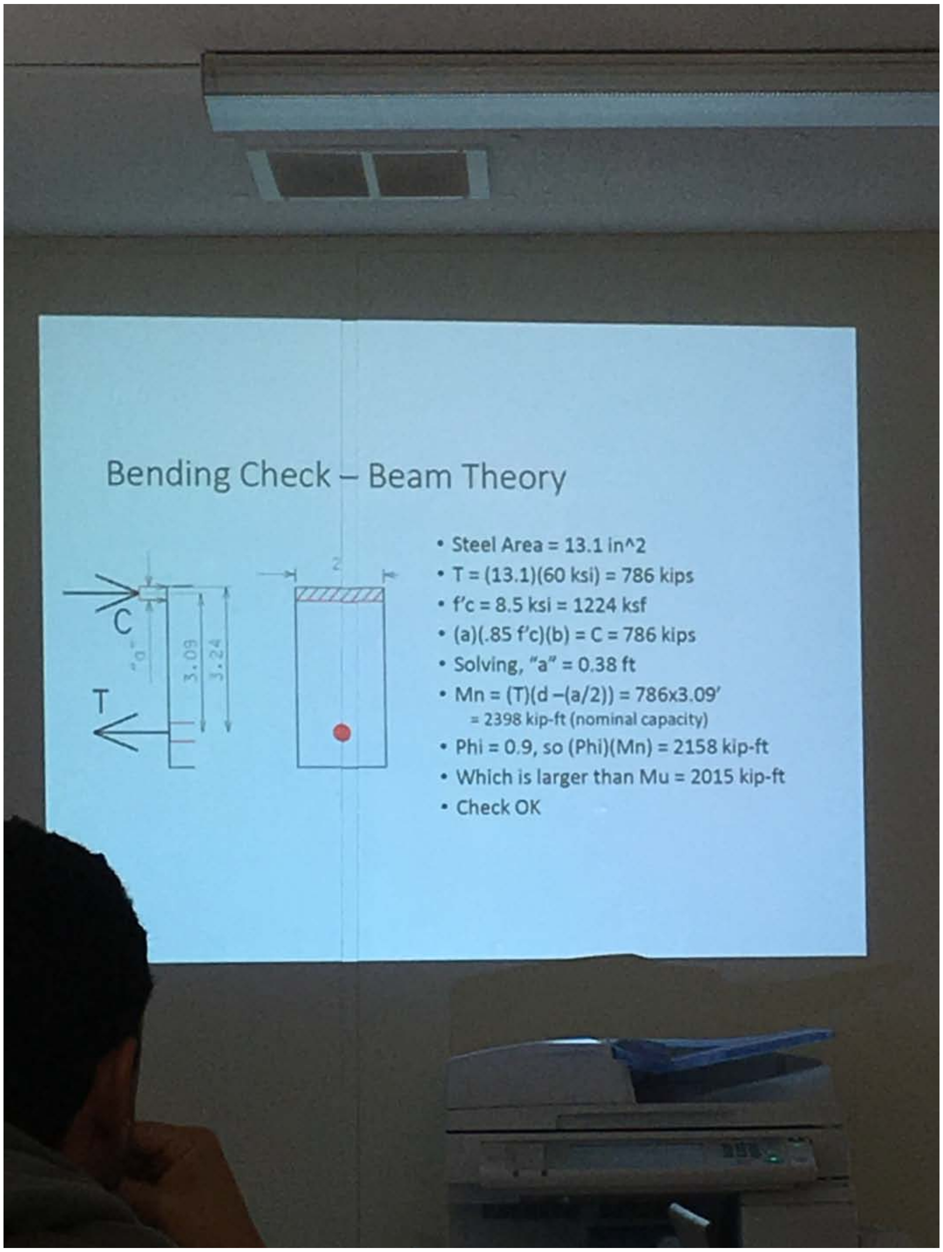

Construction Strength Checks (Strut & Tie)
• Area of Steel Tie = ((8x 1.56) + (2x.31)) = 13.1 in^2
• Nominal strength of tie = (As)(Fy) = 786 kips
• Phi, per LRFD section 5.5.4.2, since this tie steel is anchoring the shim forces to the nodal region, Phi = 1.0 for "tension in steel in anchor zones" is 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).

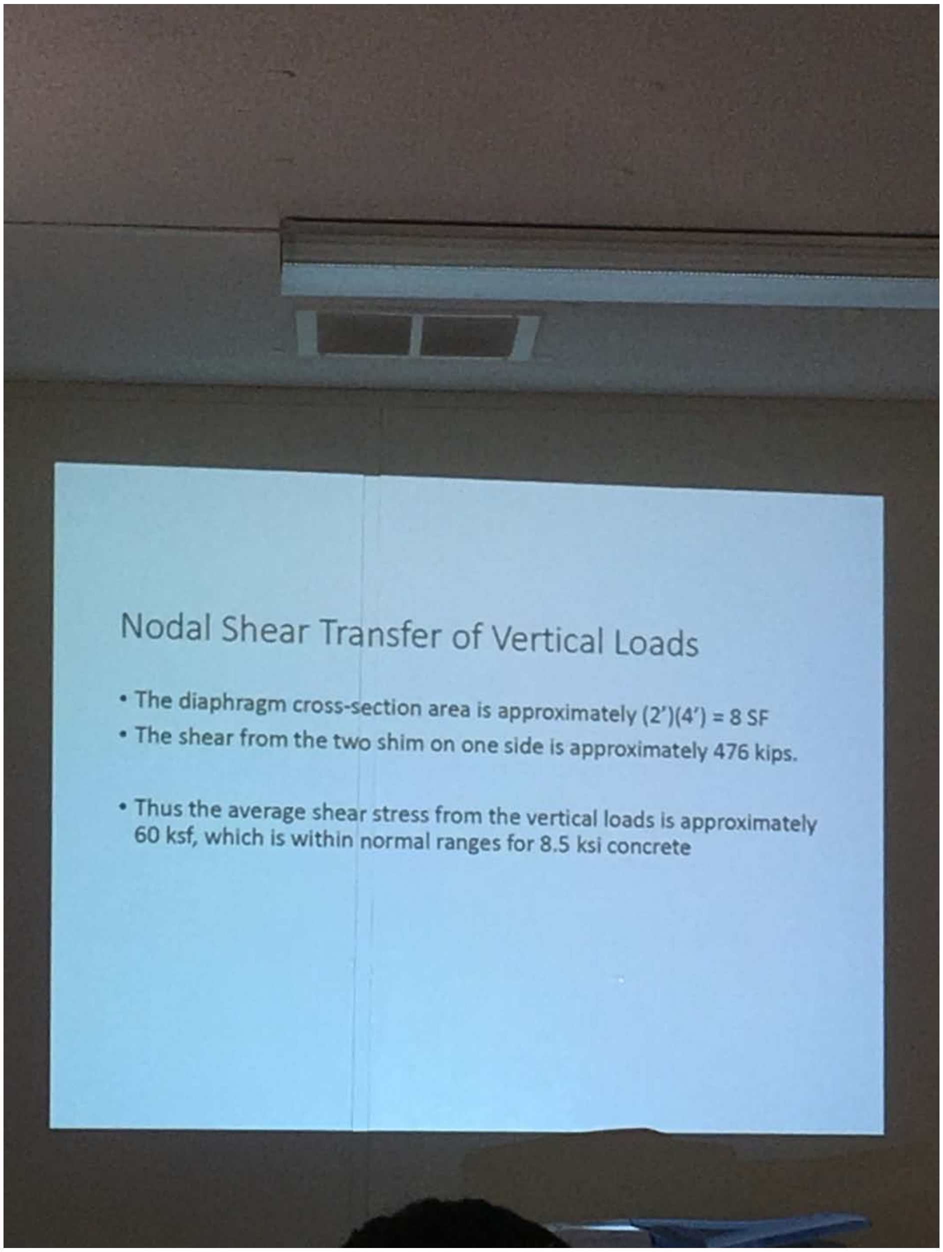


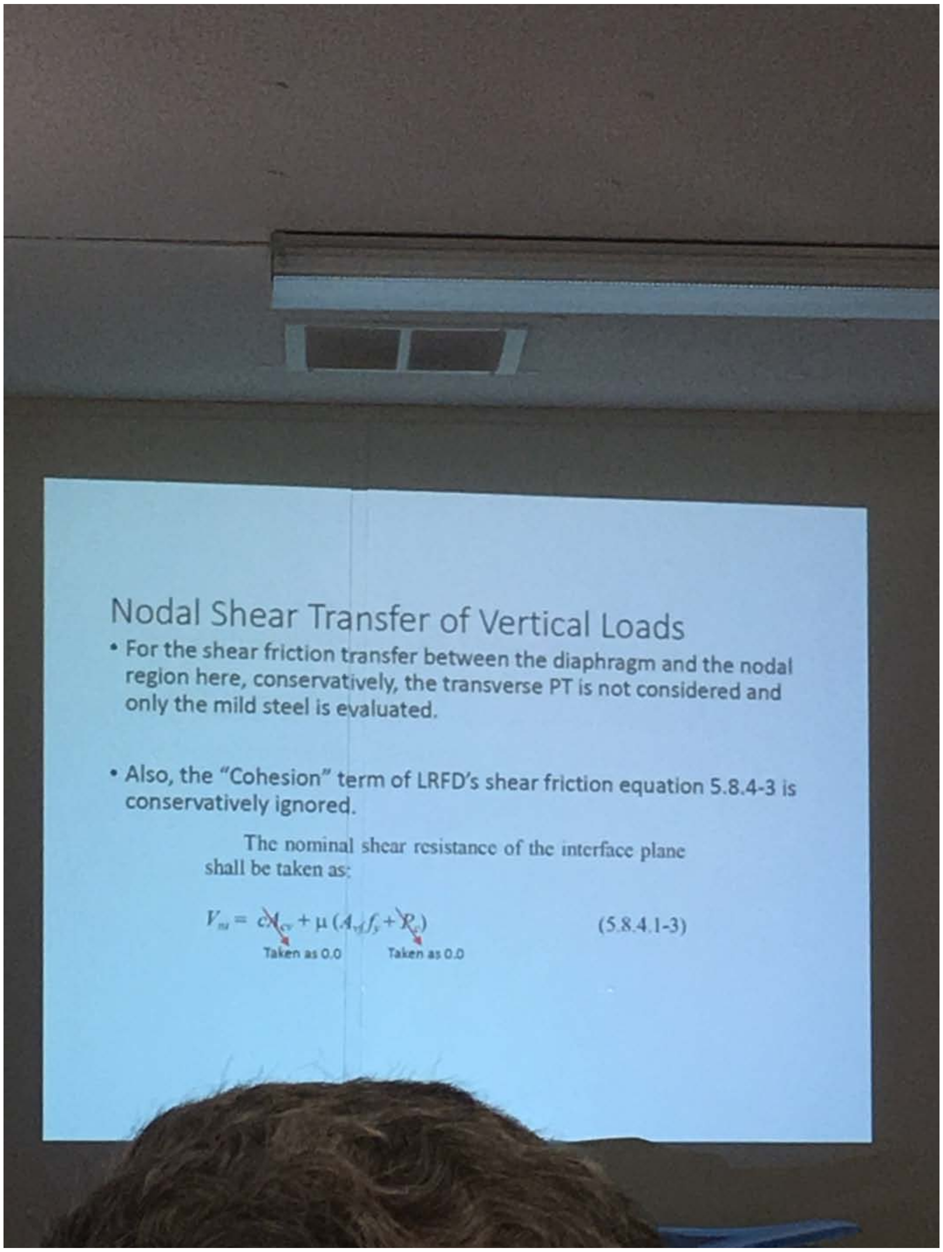
Bending Check – Beam Theory
• As previously noted, the strut and tie method is more applicable to this region. However, the conventional beam theory method can serve as a confirmation check to the strut and tie results.
M
T

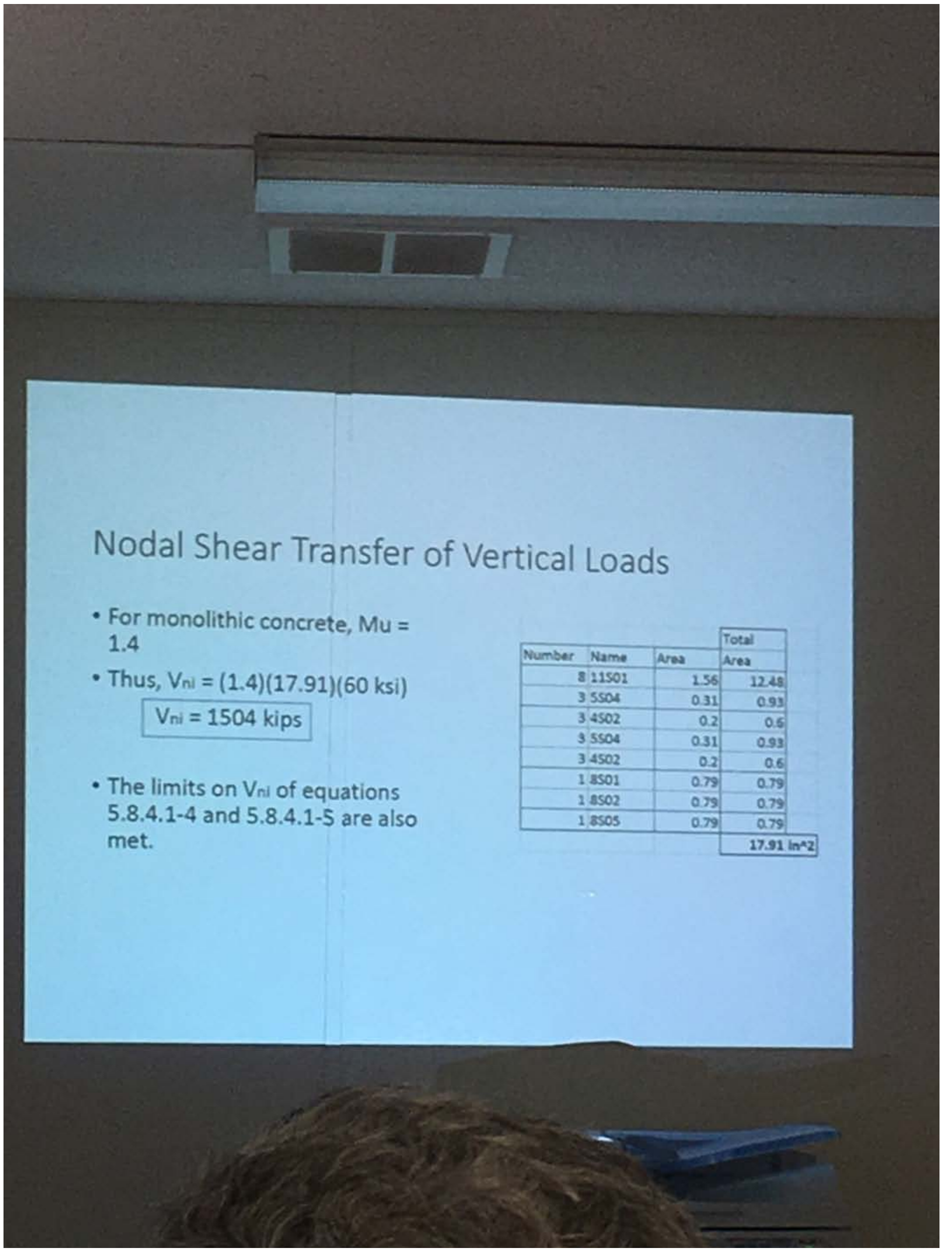

Bending Check – Beam Theory
• As previously noted:
Bending Moment (M) = 238 x (2.12 + 5.71) = 1865 kip-ft +/-
• The normal reinforced concrete behavior is assumed, where the compression in the concrete has to equal the tension in the rebar
• And ... the moment created by the distance between the T and C forces must meet the demand moment

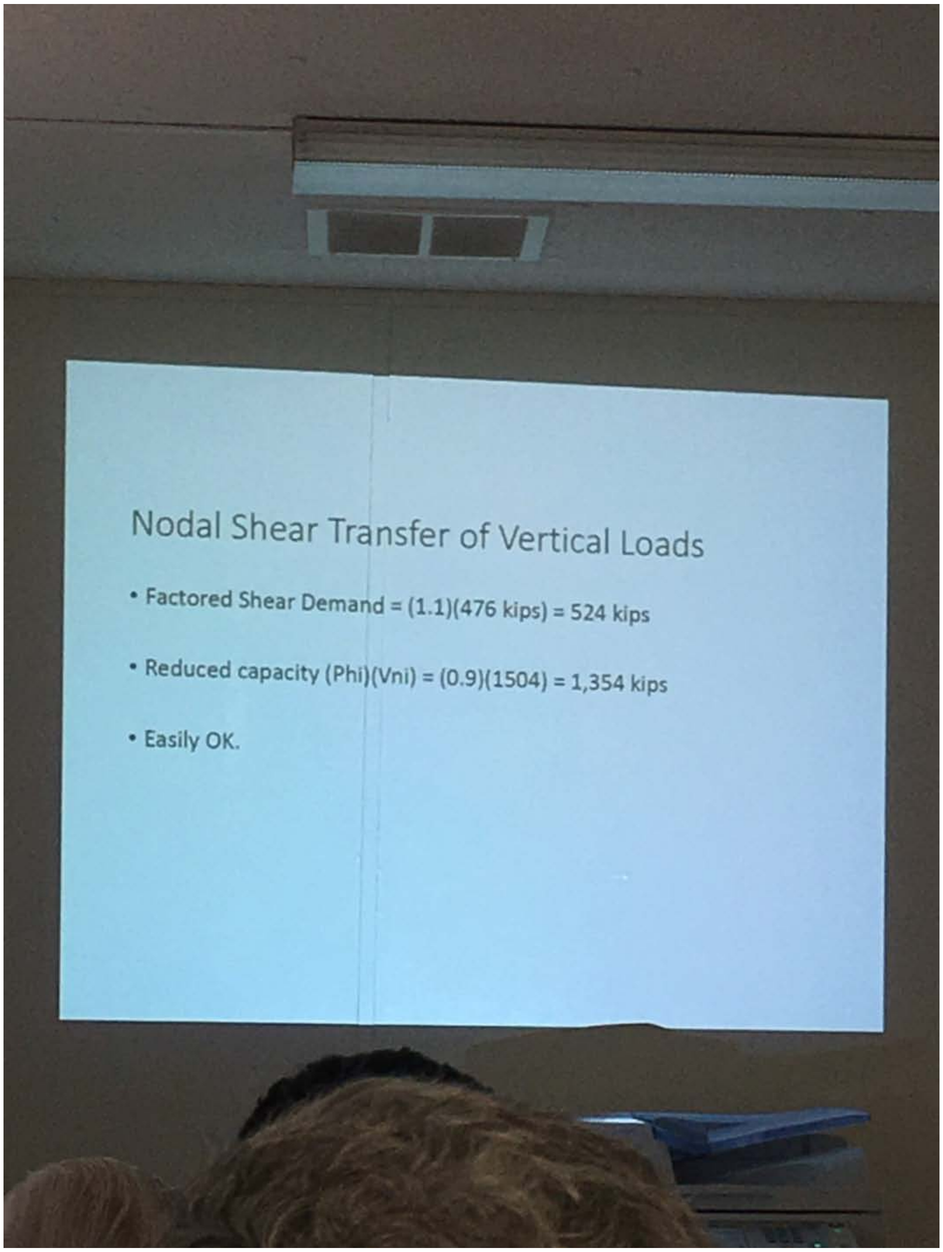

Conventional Method
Rectangular Beam Analysis
Data:
• Section dimensions – b, h, d, (span)
• Steel area - As
• Material properties – f'c, fy
Required:
• Nominal Strength (of beam) Moment - Mn
• Required (by load) Design Moment – Mu
• Load capacity
1. Calculate d
2. Check As min
3. Calculate a
4. Determine c
5. Check that $\varepsilon_t \geq 0.005$ (tension controlled)
6. Find nominal moment, Mn
7. Calculate required moment, ϕ Mn ≥ Mu
$c = \frac{a}{\beta_1}$
$\varepsilon_t = \frac{d-c}{c} 0.003 \geq 0.005$
$a = \frac{A_s f_y}{0.85 f'_c b}$
$M_n = A_s f_y \left(d - \frac{a}{2} \right)$
$\phi M_n \geq M_u$

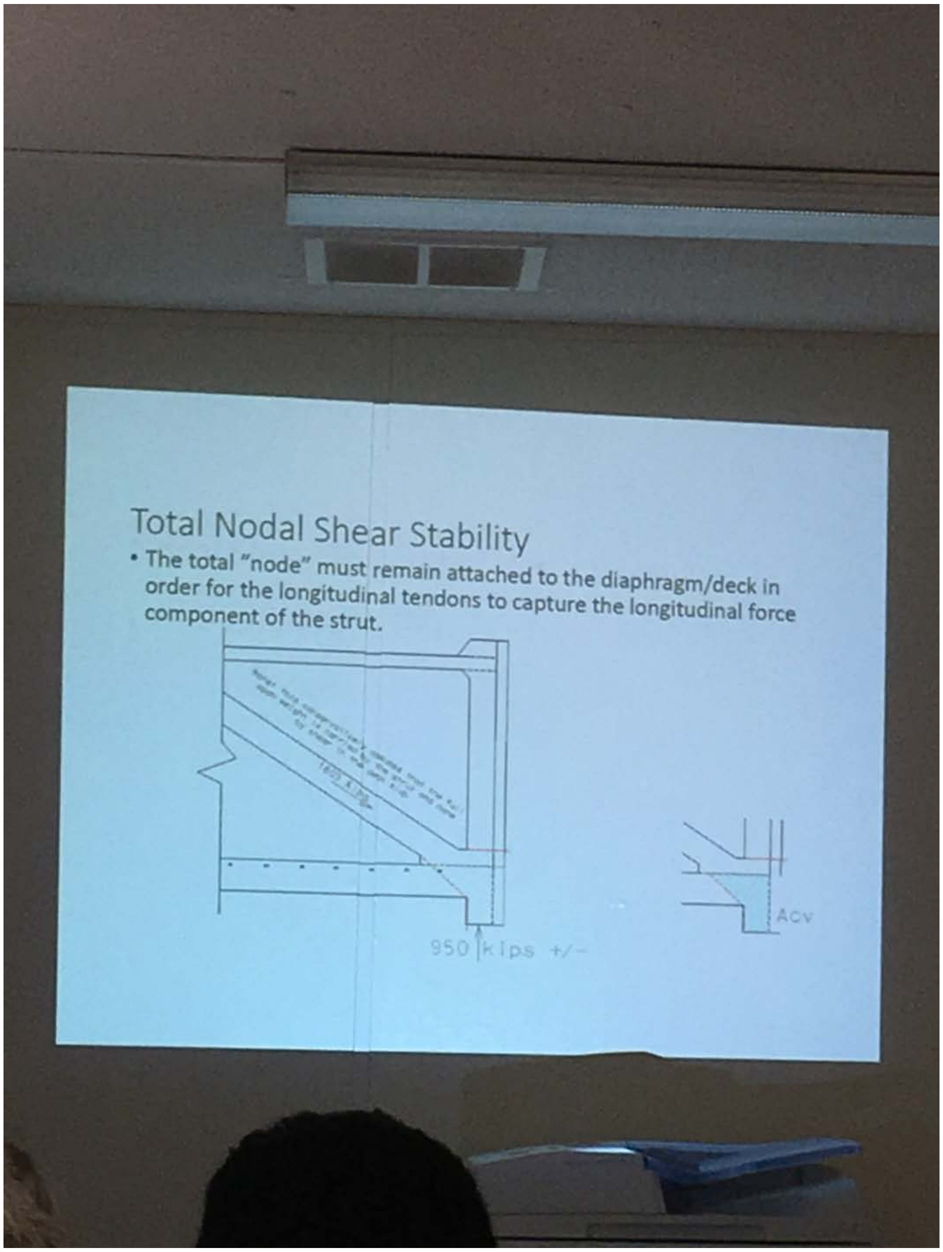

Bending Check – Beam Theory
C
T
"a"
3.09
3.24
2
• Steel Area = 13.1 in^2
• T = (13.1)(60 ksi) = 786 kips
• f'c = 8.5 ksi = 1224 ksf
• (a)(.85 f'c)(b) = C = 786 kips
• Solving, "a" = 0.38 ft
• Mn = (T)(d –(a/2)) = 786x3.09'
= 2398 kip-ft (nominal capacity)
• Phi = 0.9, so (Phi)(Mn) = 2158 kip-ft
• Which is larger than Mu = 2015 kip-ft
• Check OK

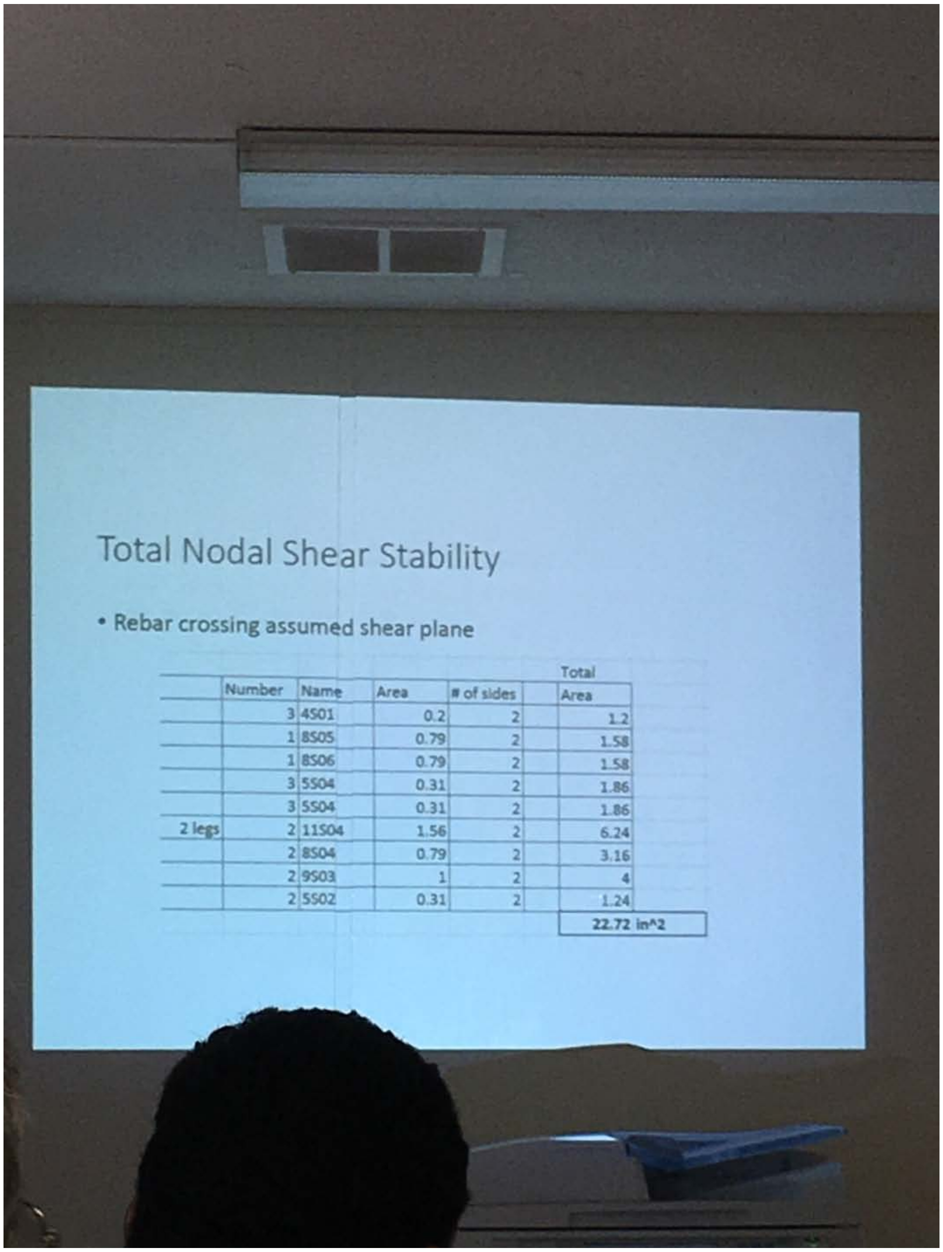

Nodal Shear Transfer of Vertical Loads
• The diaphragm cross-section area is approximately (2')(4') = 8 SF
• The shear from the two shim on one side is approximately 476 kips.
• Thus the average shear stress from the vertical loads is approximately 60 ksf, which is within normal ranges for 8.5 ksi concrete


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:
$V_{ni} = \cancel{cA_{cv}} + \mu (A_{vf} f_y + \cancel{P_c})$ (5.8.4.1-3)
Taken as 0.0
Taken as 0.0


Nodal Shear Transfer of Vertical Loads
• For monolithic concrete, Mu = 1.4
• Thus, Vni = (1.4)(17.91)(60 ksi)
Vni = 1504 kips
• The limits on Vni 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

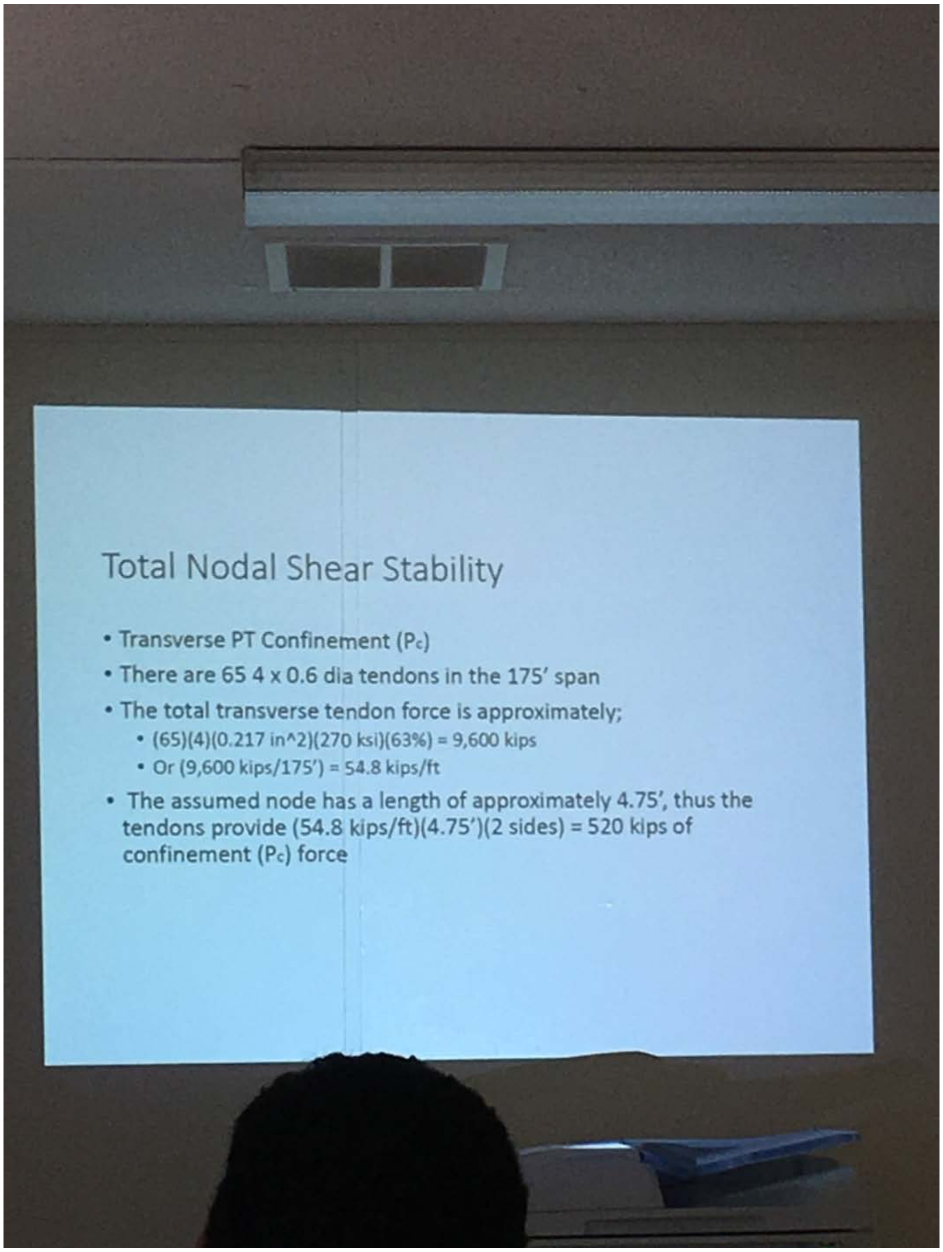

Nodal Shear Transfer of Vertical Loads
• Factored Shear Demand = (1.1)(476 kips) = 524 kips
• Reduced capacity (Phi)(Vni) = (0.9)(1504) = 1,354 kips
• Easily OK.

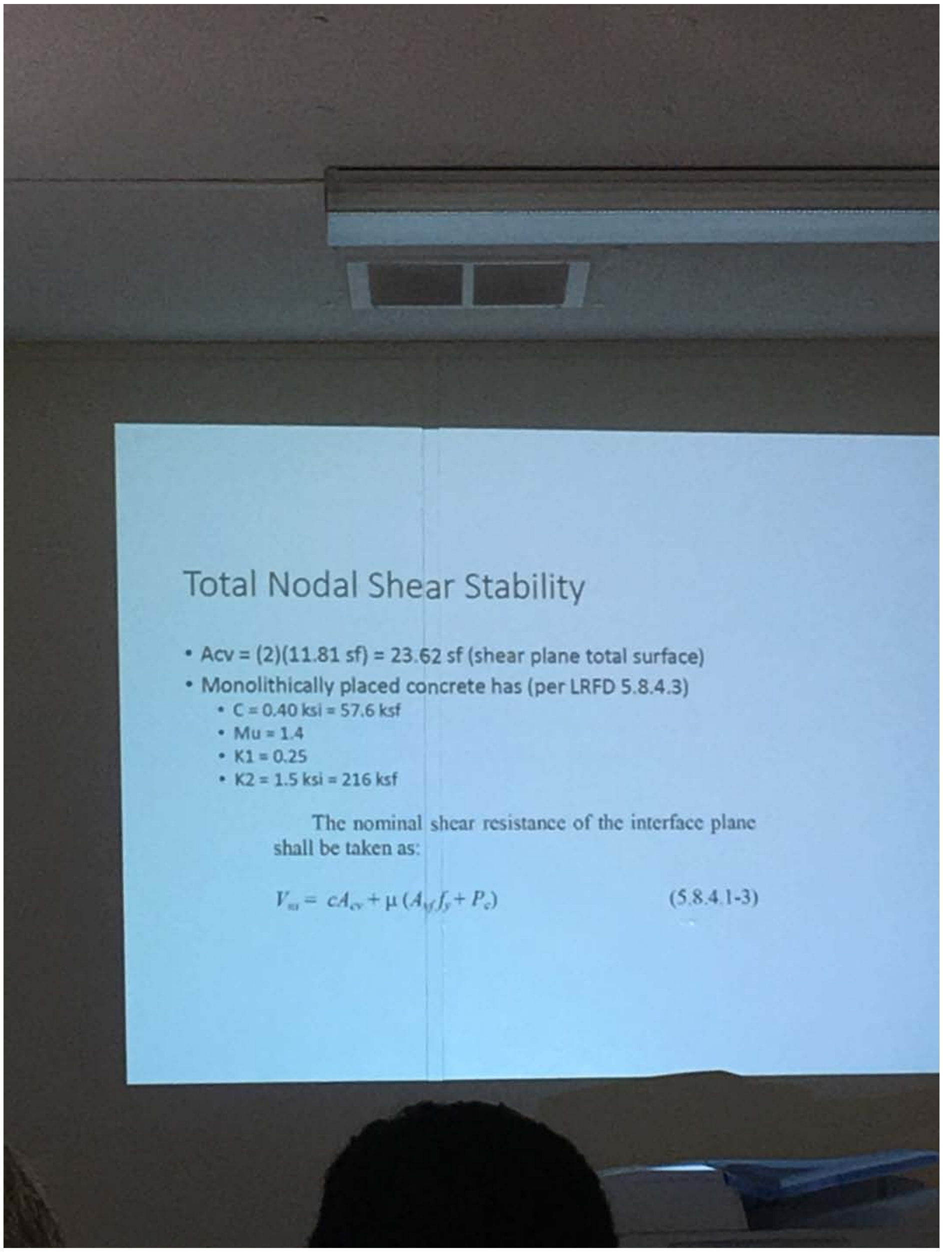

Total Nodal Shear Stability
• The total "node" must remain attached to the diaphragm/deck in order for the longitudinal tendons to capture the longitudinal force component of the strut.
950 kips +/-
Acv

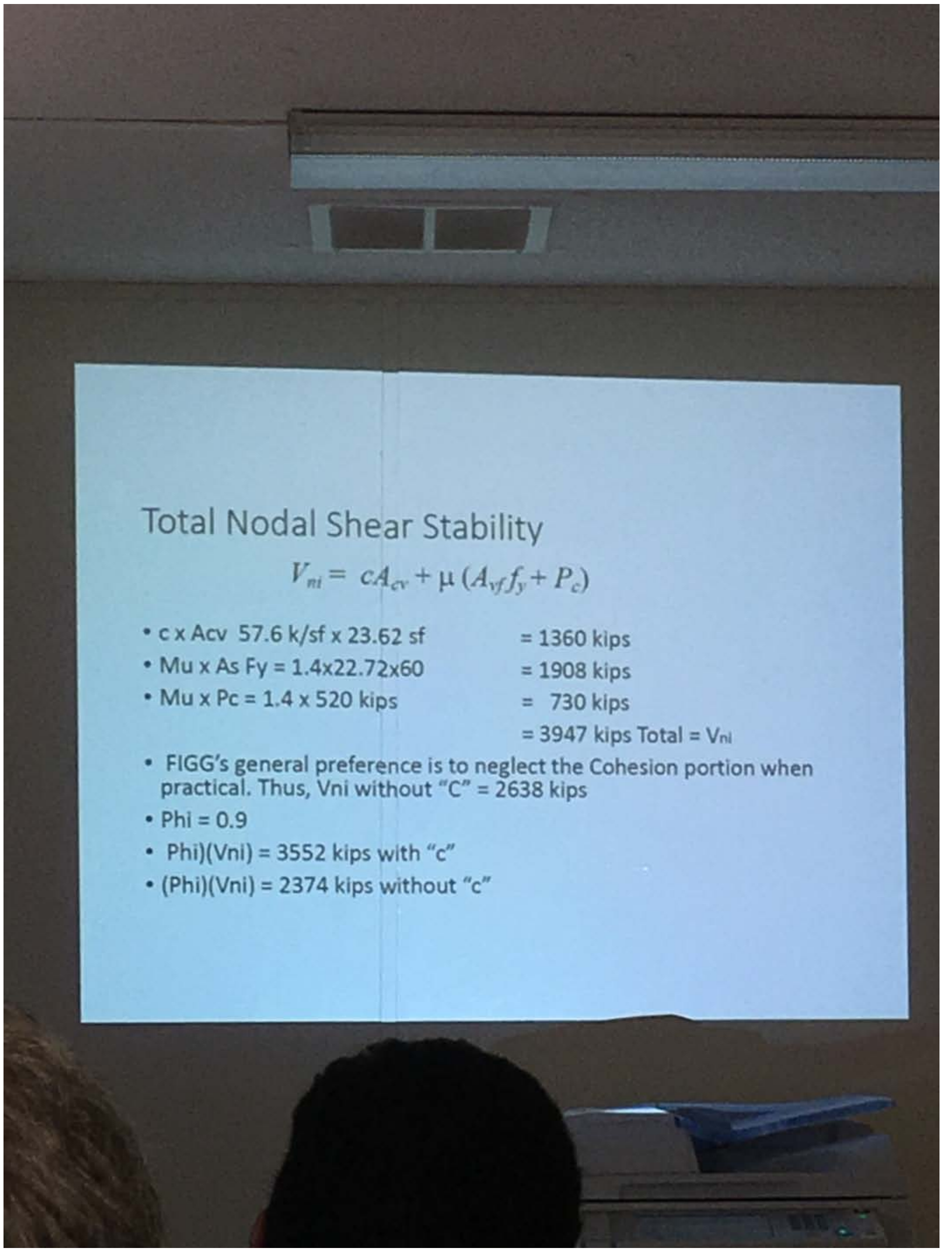


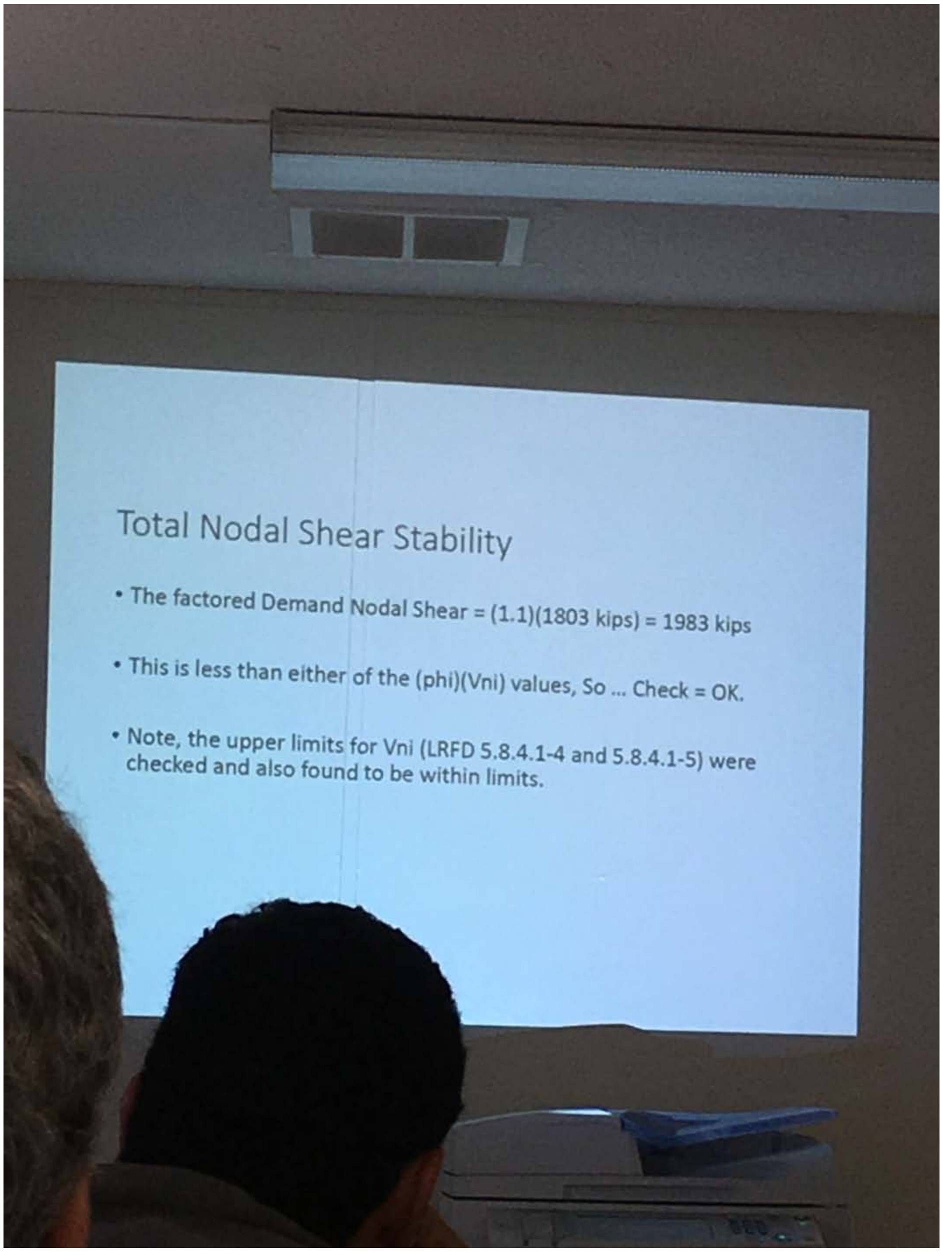
Total Nodal Shear Stability
• Rebar crossing assumed shear plane

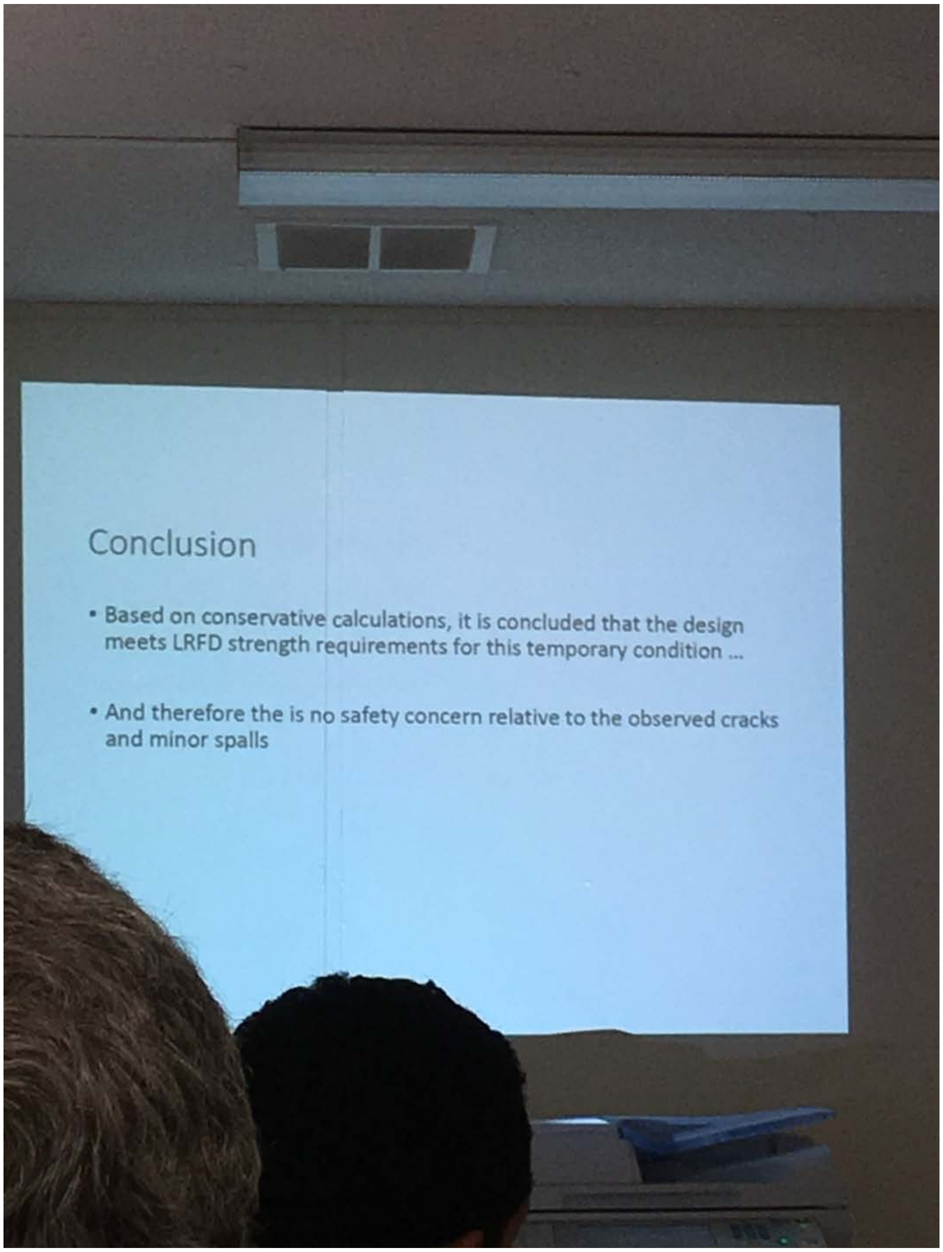
	Number	Name	Area	# of sides		Total Area
	3	4S01	0.2	2		1.2
	1	8S05	0.79	2		1.58
	1	8S06	0.79	2		1.58
	3	5S04	0.31	2		1.86
	3	5S04	0.31	2		1.86
2 legs	2	11S04	1.56	2		6.24
	2	8S04	0.79	2		3.16
	2	9S03	1	2		4
	2	5S02	0.31	2		1.24
						22.72 in^2

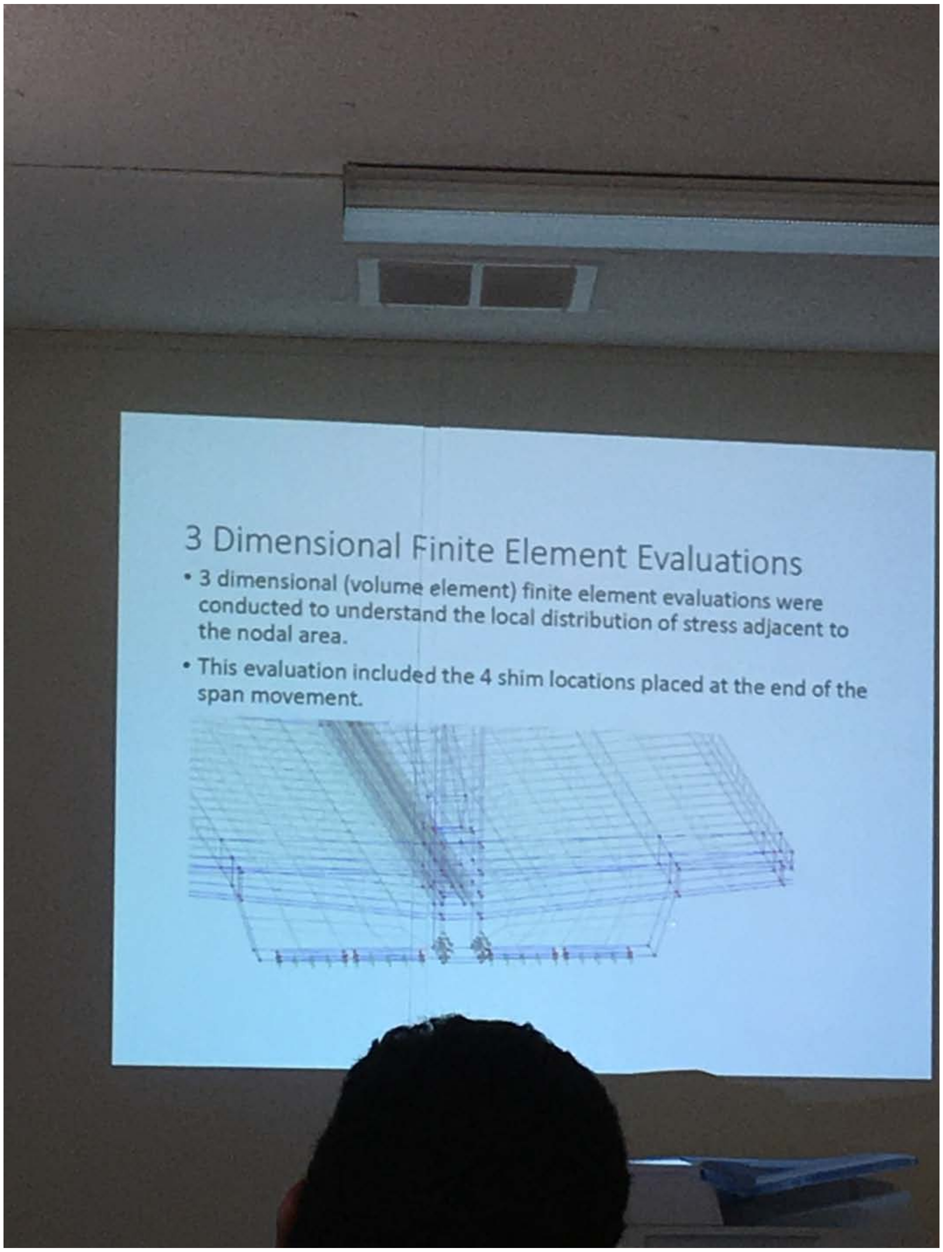

Total Nodal Shear Stability
• Transverse PT Confinement (Pc)
• There are 65 4 x 0.6 dia tendons in the 175' span
• The total transverse tendon force is approximately;
• (65)(4)(0.217 in^2)(270 ksi)(63%) = 9,600 kips
• Or (9,600 kips/175') = 54.8 kips/ft
• The assumed node has a length of approximately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

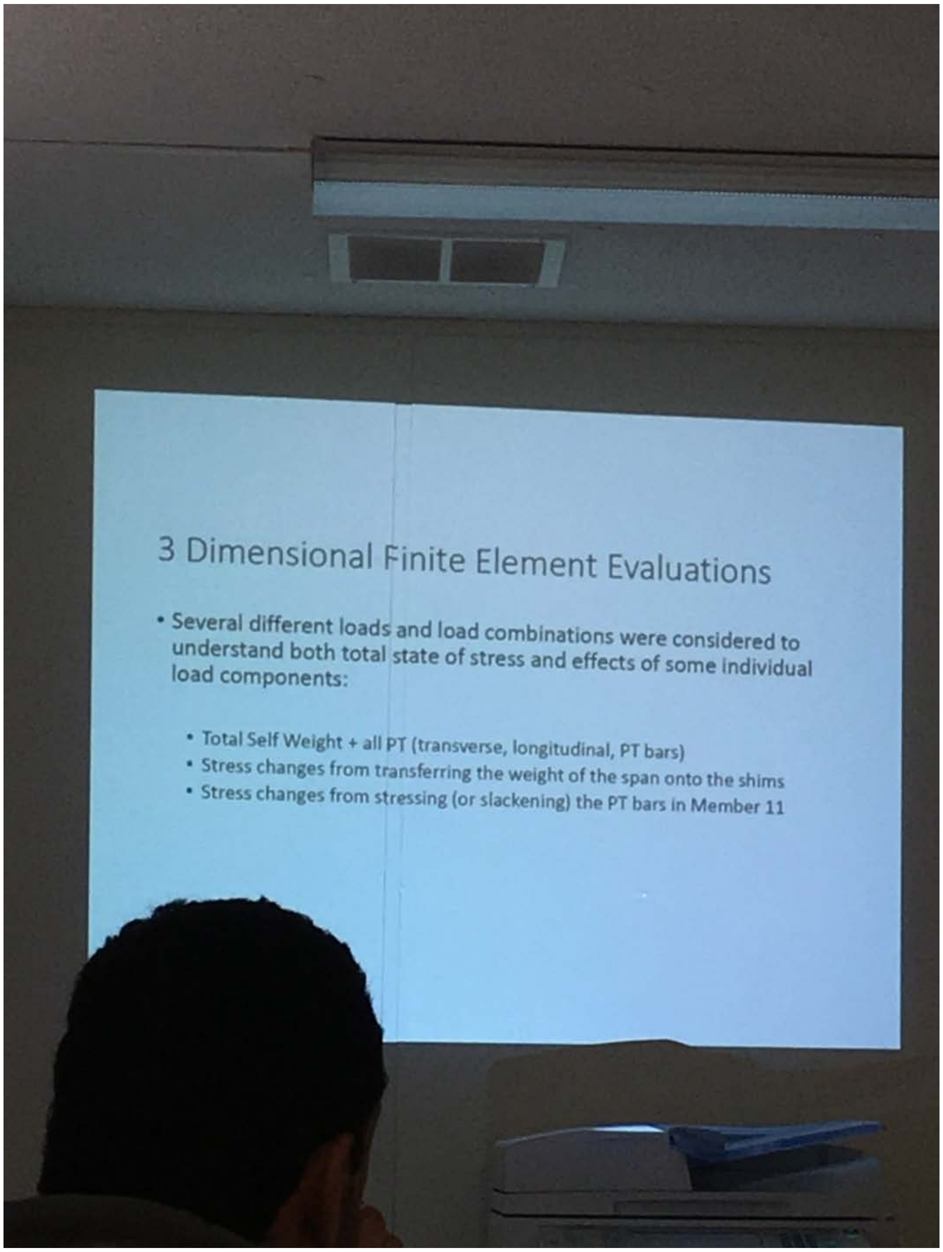

Total Nodal Shear Stability
• Acv = (2)(11.81 sf) = 23.62 sf (shear plane total surface)
• Monolithically placed concrete has (per LRFD 5.8.4.3)
• C = 0.40 ksi = 57.6 ksf
• Mu = 1.4
• K1 = 0.25
• K2 = 1.5 ksi = 216 ksf
The nominal shear resistance of the interface plane shall be taken as:
$V_{ni} = cA_{cv} + \mu (A_{vf} f_y + P_c)$ (5.8.4.1-3)

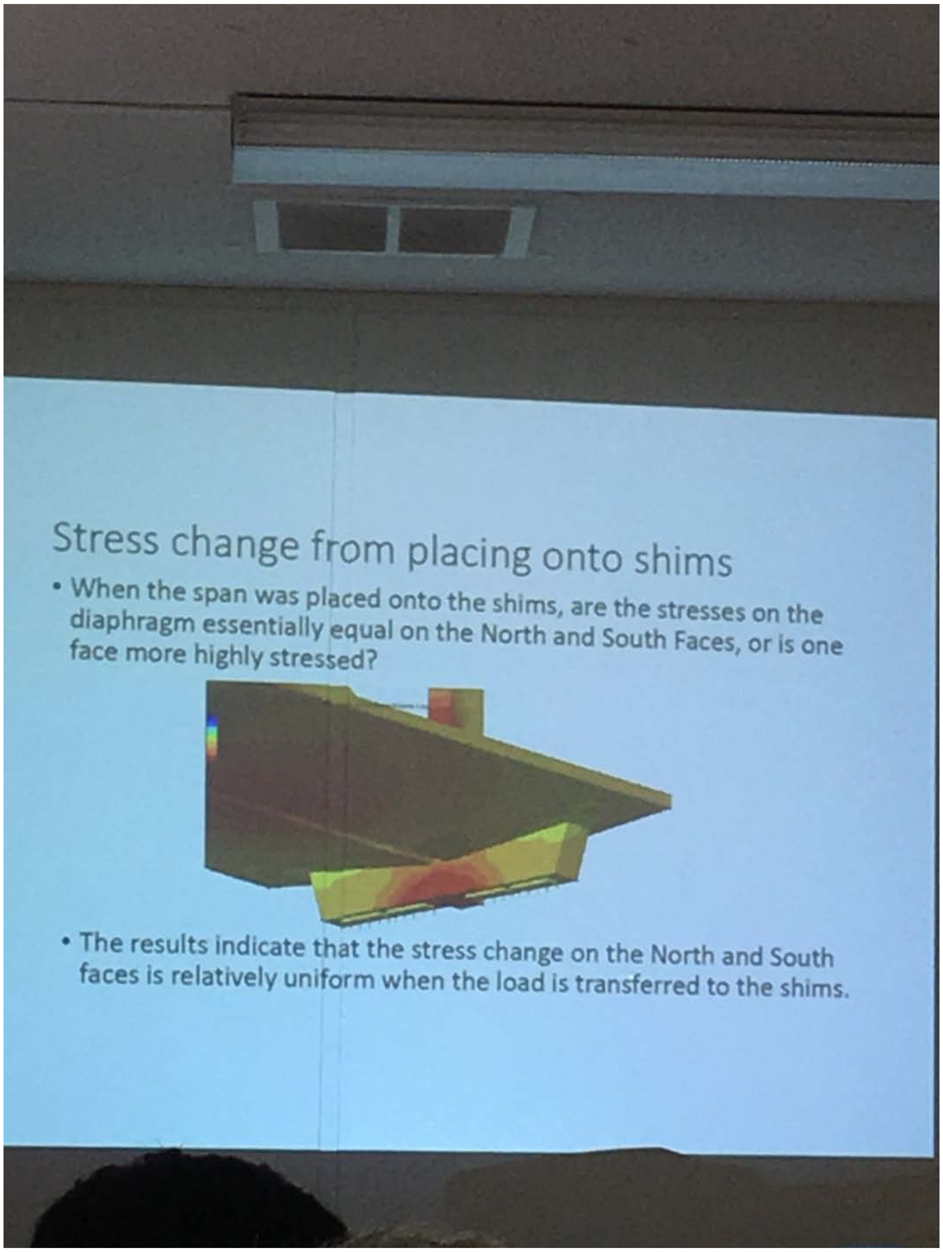

Total Nodal Shear Stability
$V_{ni} = cA_{cv} + \mu (A_{vf} f_y + P_c)$
• c x Acv 57.6 k/sf x 23.62 sf = 1360 kips
• Mu x As Fy = 1.4x22.72x60 = 1908 kips
• Mu x Pc = 1.4 x 520 kips = 730 kips
= 3947 kips Total = V_{ni}
• FIGG's general preference is to neglect the Cohesion portion when practical. Thus, Vni without "C" = 2638 kips
• Phi = 0.9
• Phi)(Vni) = 3552 kips with "c"
• (Phi)(Vni) = 2374 kips without "c"

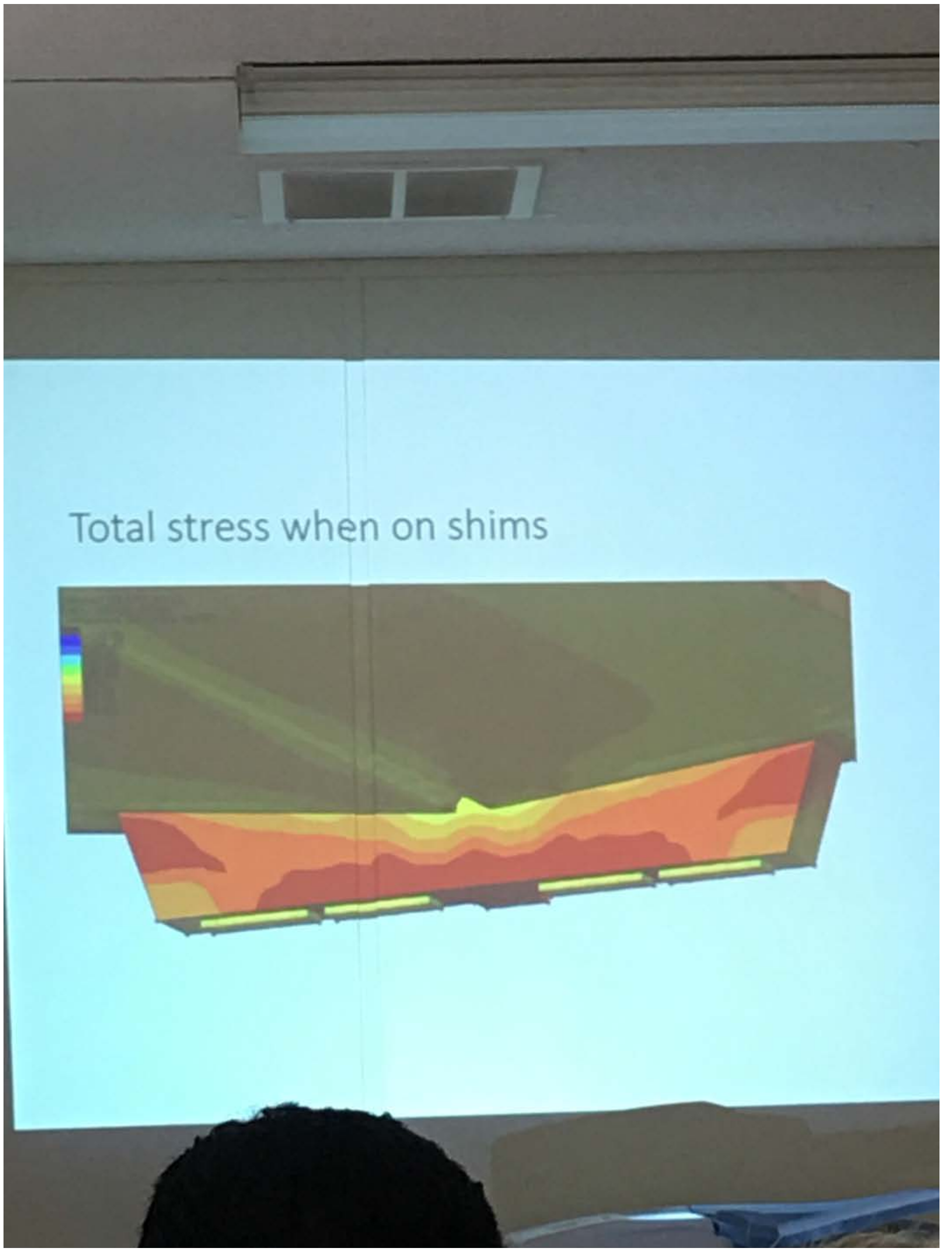

Total Nodal Shear Stability
• The factored Demand Nodal Shear = (1.1)(1803 kips) = 1983 kips
• This is less than either of the (phi)(Vni) values, So ... Check = OK.
• Note, the upper limits for Vni (LRFD 5.8.4.1-4 and 5.8.4.1-5) were checked and also found to be within limits.

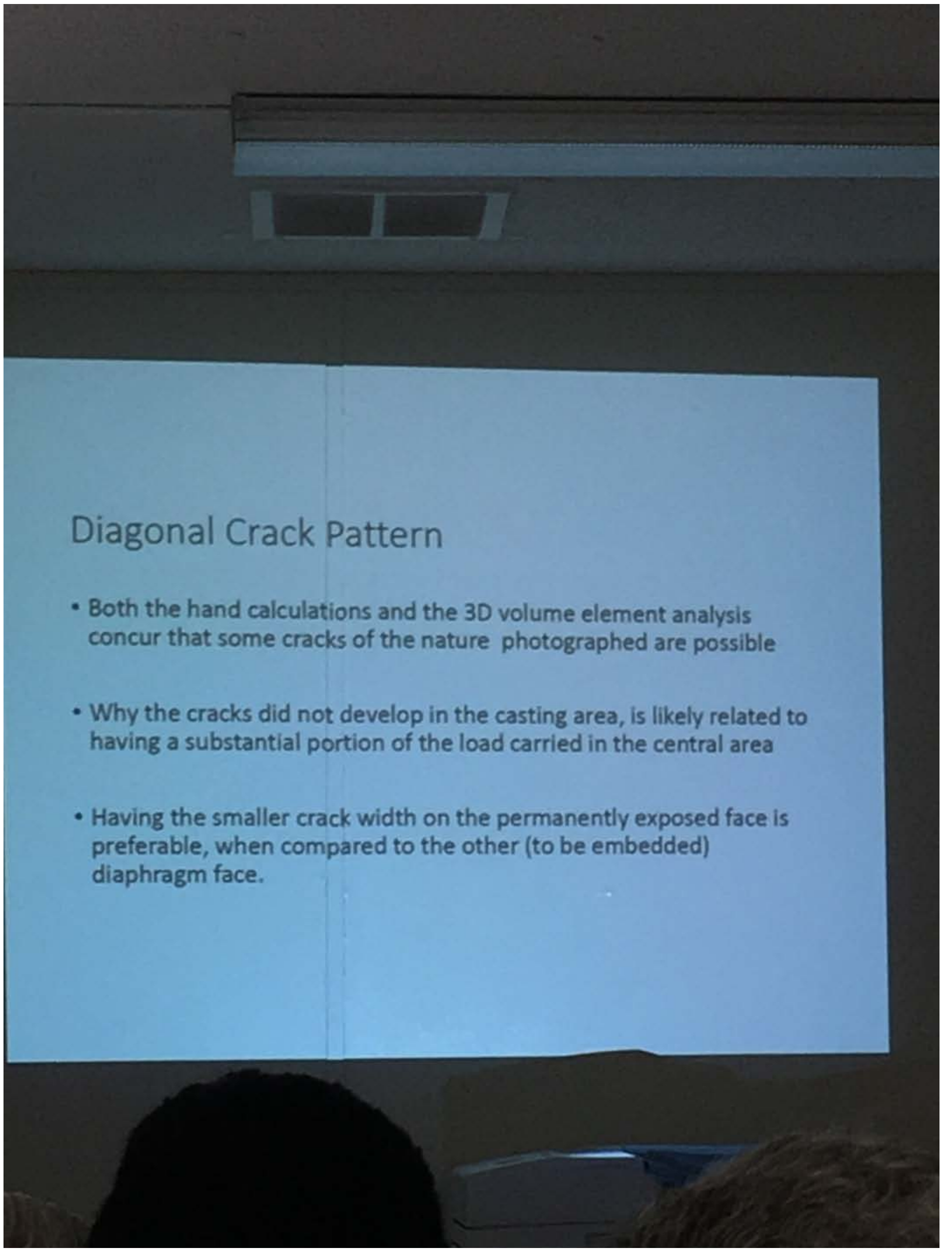

Conclusion
• Based on conservative calculations, it is concluded that the design meets LRFD strength requirements for this temporary condition ...
• And therefore the is no safety concern relative to the observed cracks and minor spalls

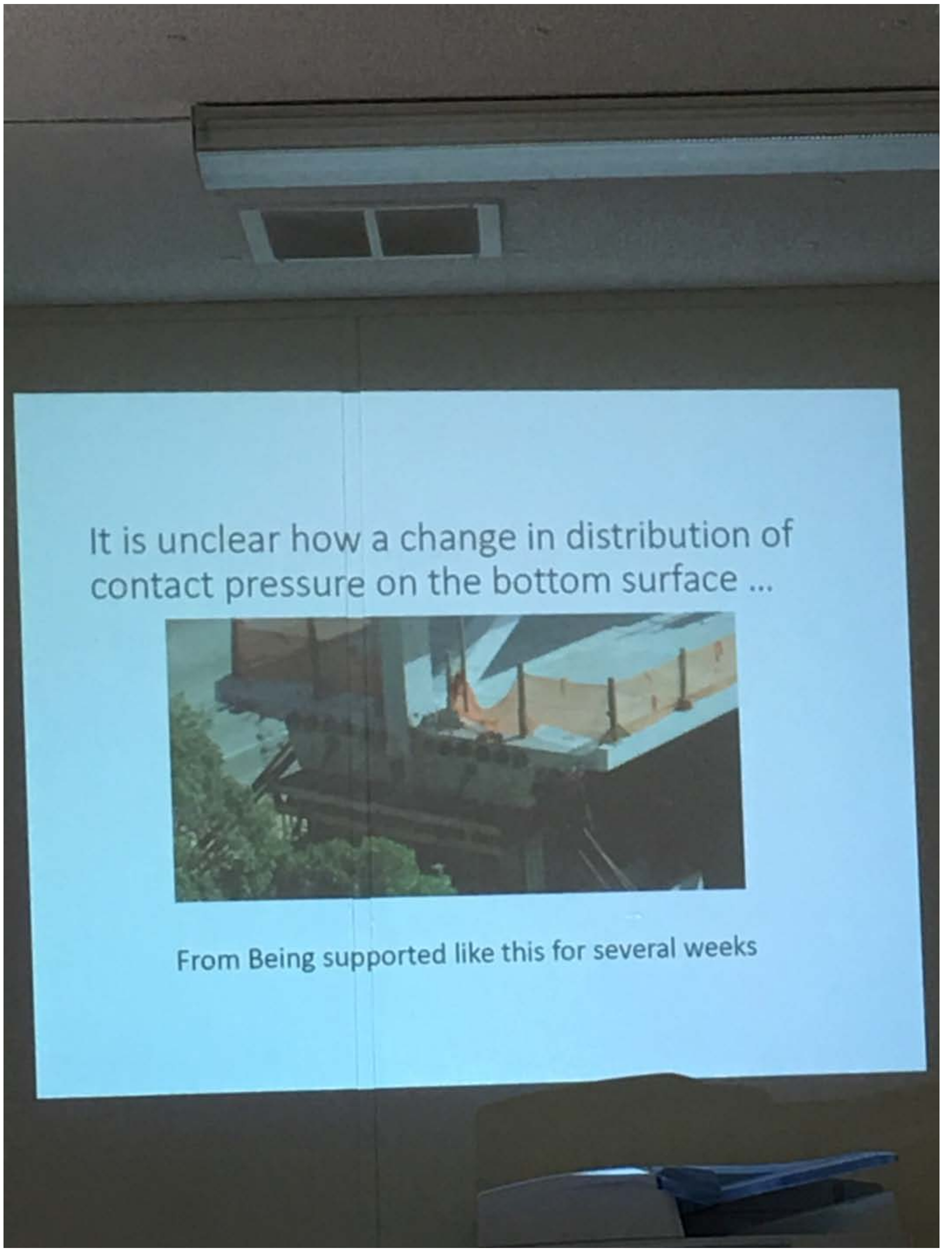

3 Dimensional Finite Element Evaluations
• 3 dimensional (volume element) finite element evaluations were conducted to understand the local distribution of stress adjacent to the nodal area.
• This evaluation included the 4 shim locations placed at the end of the span movement.

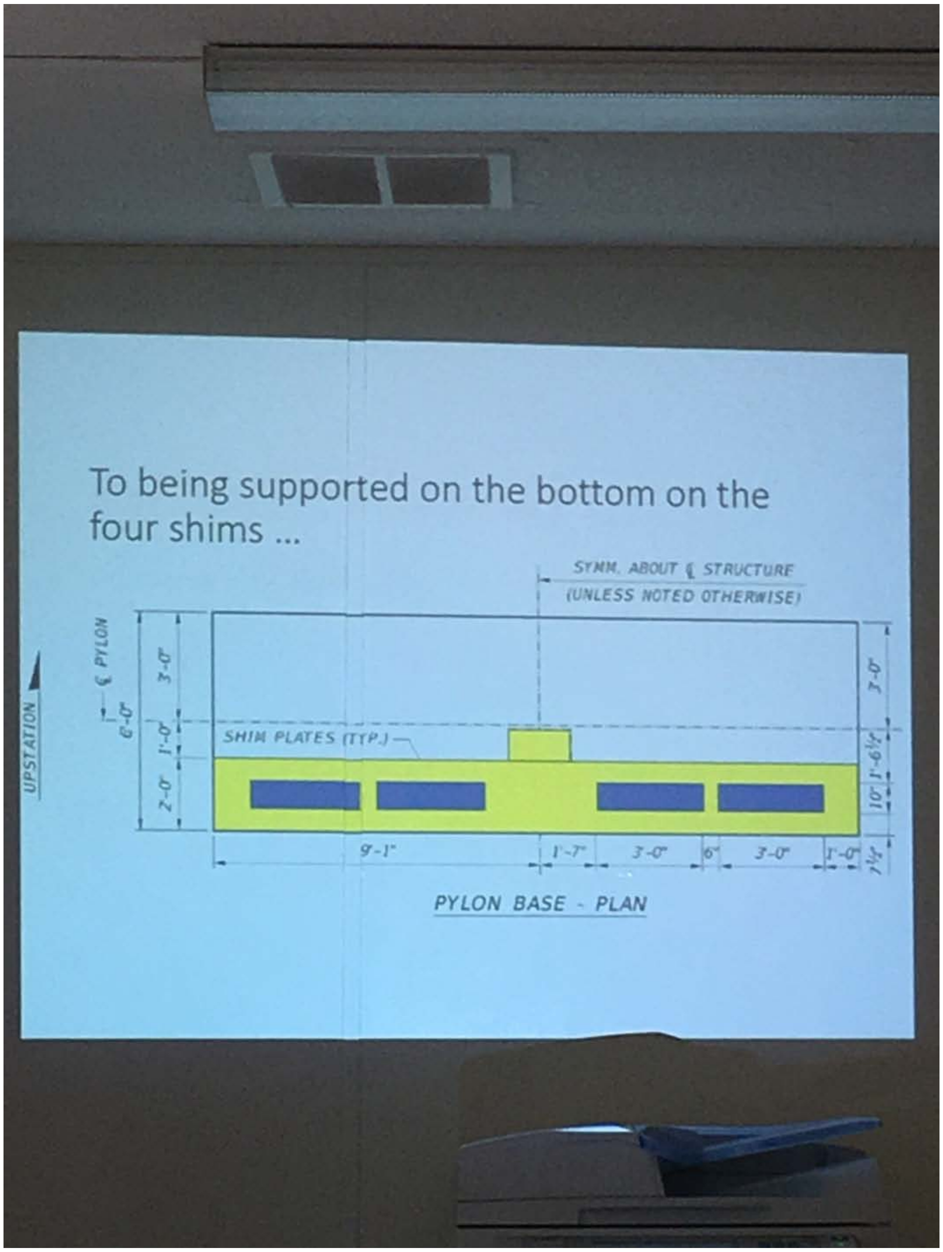

3 Dimensional Finite Element Evaluations
• Several different loads and load combinations were considered to understand both total state of stress and effects of some individual load components:
• Total Self Weight + all PT (transverse, longitudinal, PT bars)
• Stress changes from transferring the weight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

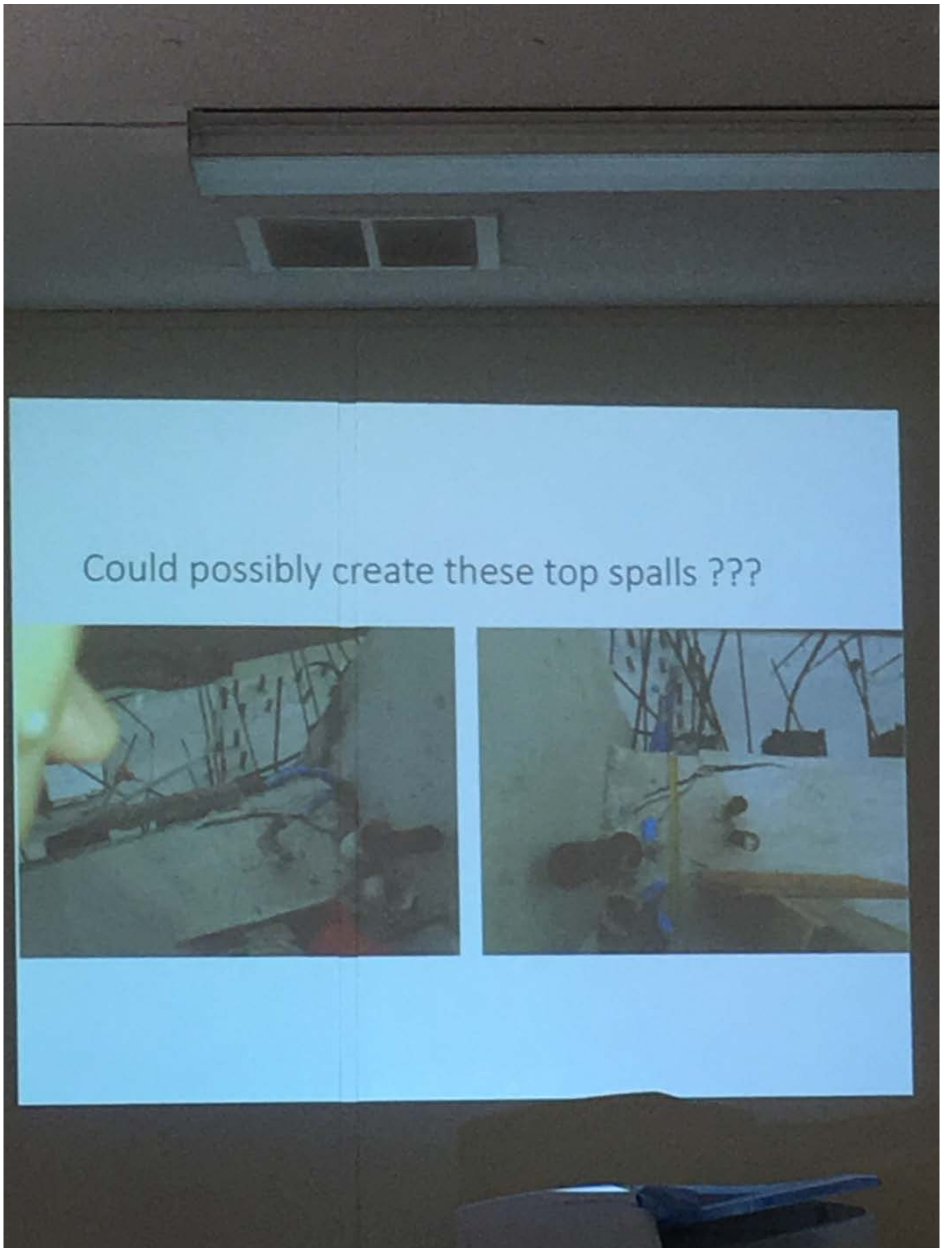

Stress change from placing onto shims
• When the span was placed onto the shims, are the stresses on the diaphragm essentially equal on the North and South Faces, or is one face more highly stressed?
• The results indicate that the stress change on the North and South faces is relatively uniform when the load is transferred to the shims.

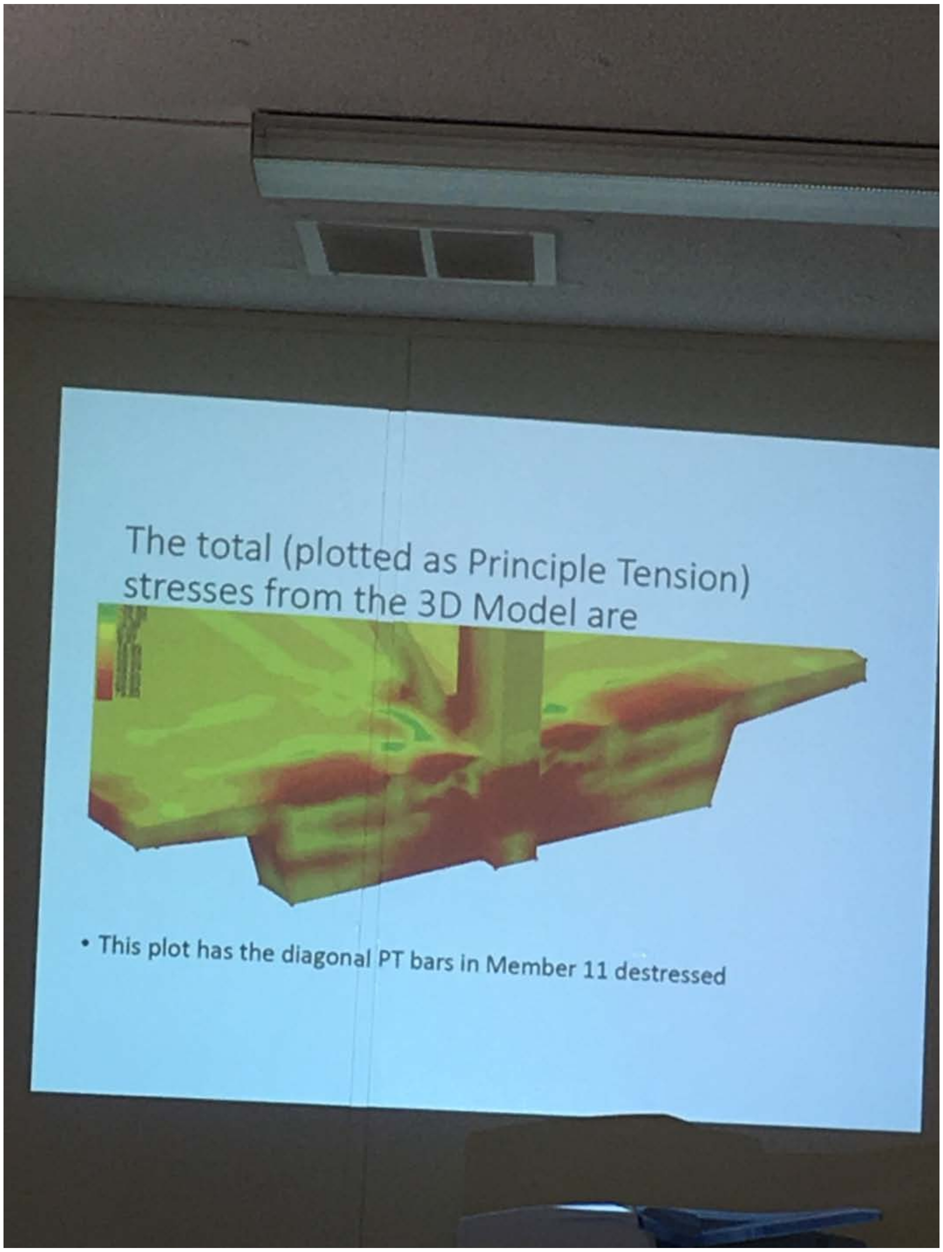

Total stress when on shims

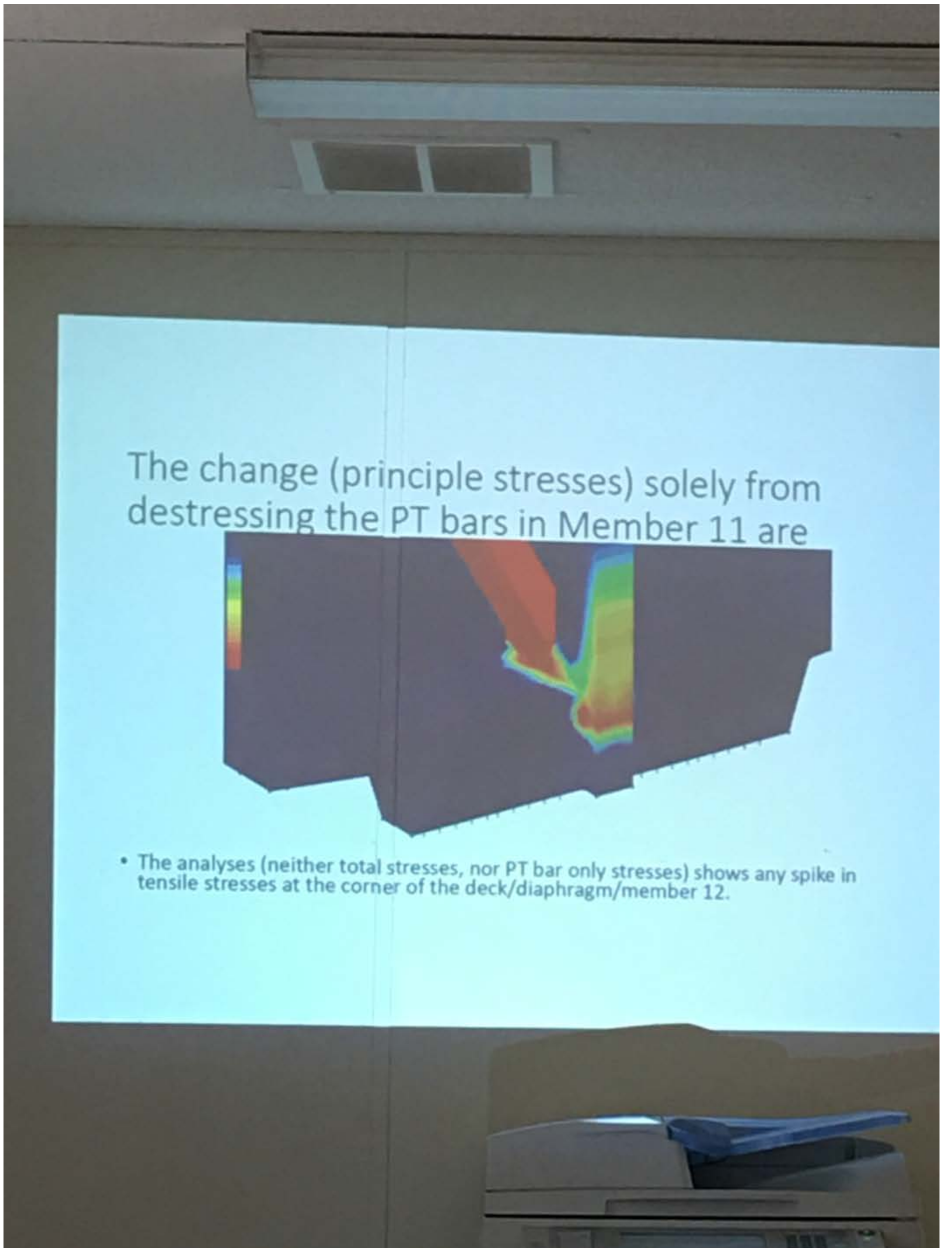

Diagonal Crack Pattern
• Both the hand calculations and the 3D volume element analysis concur that some cracks of the nature photographed are possible
• Why the cracks did not develop in the casting area, is likely related to having a substantial portion of the load carried in the central area
• Having the smaller crack width on the permanently exposed face is preferable, when compared to the other (to be embedded) diaphragm face.

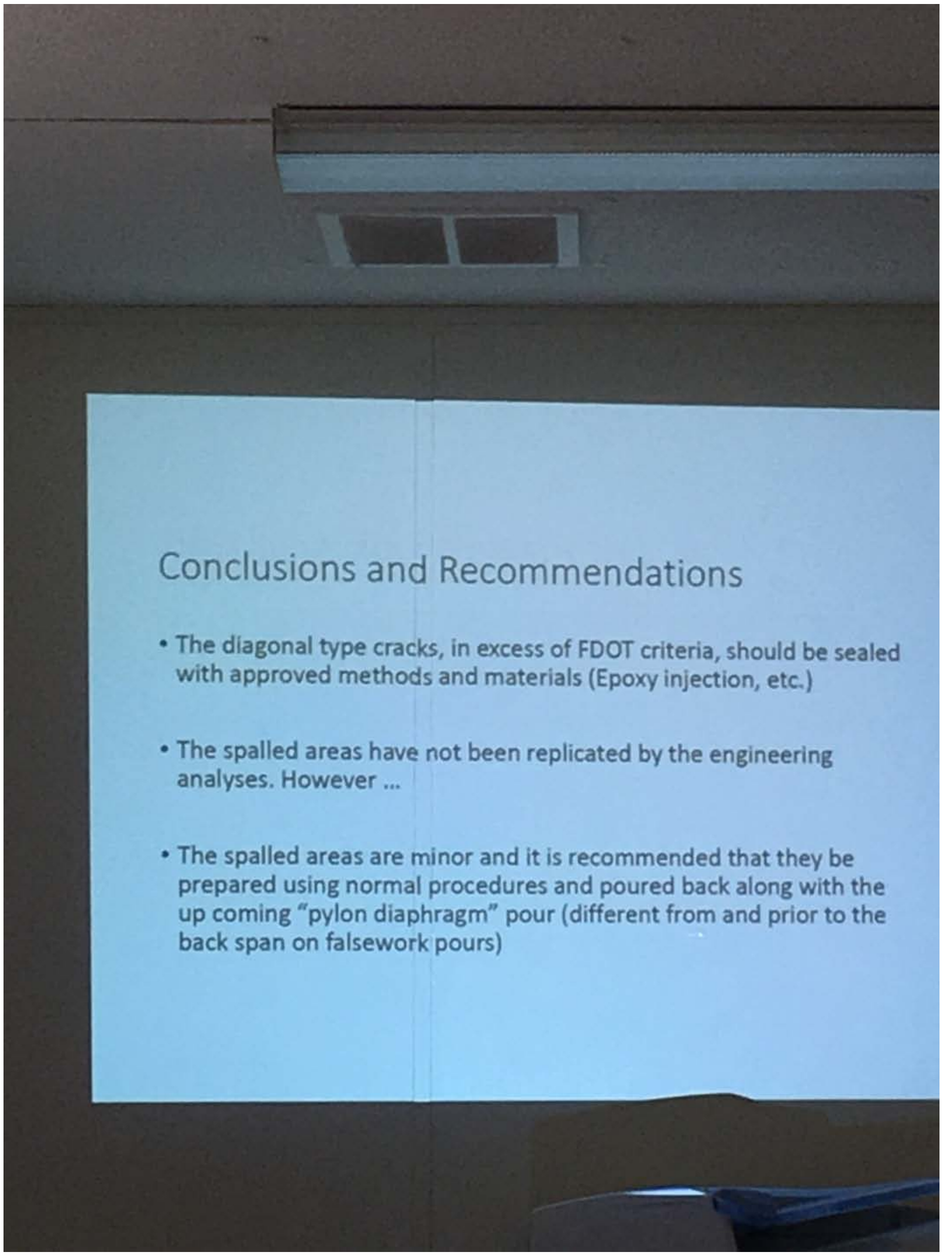

It is unclear how a change in distribution of contact pressure on the bottom surface …
From Being supported like this for several weeks


To being supported on the bottom on the four shims …
SYMM. ABOUT ℄ STRUCTURE
(UNLESS NOTED OTHERWISE)
SHIM PLATES (TYP.)
UPSTATION
℄ PYLON
6'-0"
3'-0"
1'-0"
2'-0"
9'-1"
1'-7"
3'-0"
6"
3'-0"
1'-0"
1'-6½"
10"
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)