Fundamentals and Application of PCI Ultra-High-Performance Concrete

Presented at the GCPCI-PCEF Meeting
February 4, 2021

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Outline

- What is UHPC?
- PCI-UHPC
- Research objectives
- Materials and plant production
- Structural design recommendations with PCI-UHPC
- Future opportunities for PCI-UHPC

What is Ultra-High-Performance Concrete?

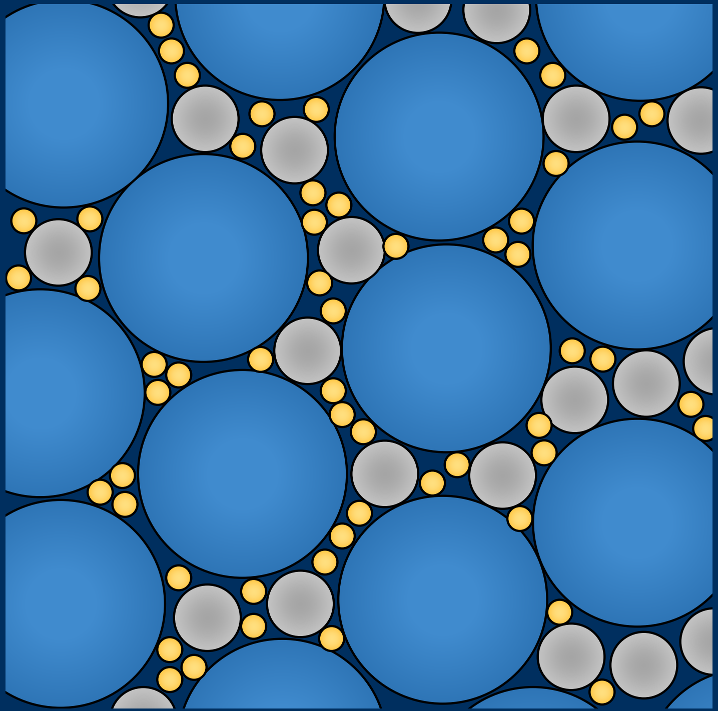
- Fiber-reinforced, cementitious composite
  - Low w/cm (typically < 0.20)

What is PCI-Ultra-High-Performance Concrete?

- Characterized by:
  - Higher compressive strength than currently in AASHTO LRFD-BDS
  - High pre- and post-cracking tensile strength
  - Ensured strain hardening to allow for exceptional flexural and shear behavior
  - Enhanced durability due to high density and discontinuous pore structure

PCI-UHPC Mix Design Based on Local Materials

- Type I/II Cement
- Silica Fume
- Supplementary powder (slag, ground limestone, etc.)
- Masonry Sand
- Steel Fibers
- High-range water reducer
- Admixture to extend flowability



Objectives of the PCI-UHPC Research Project

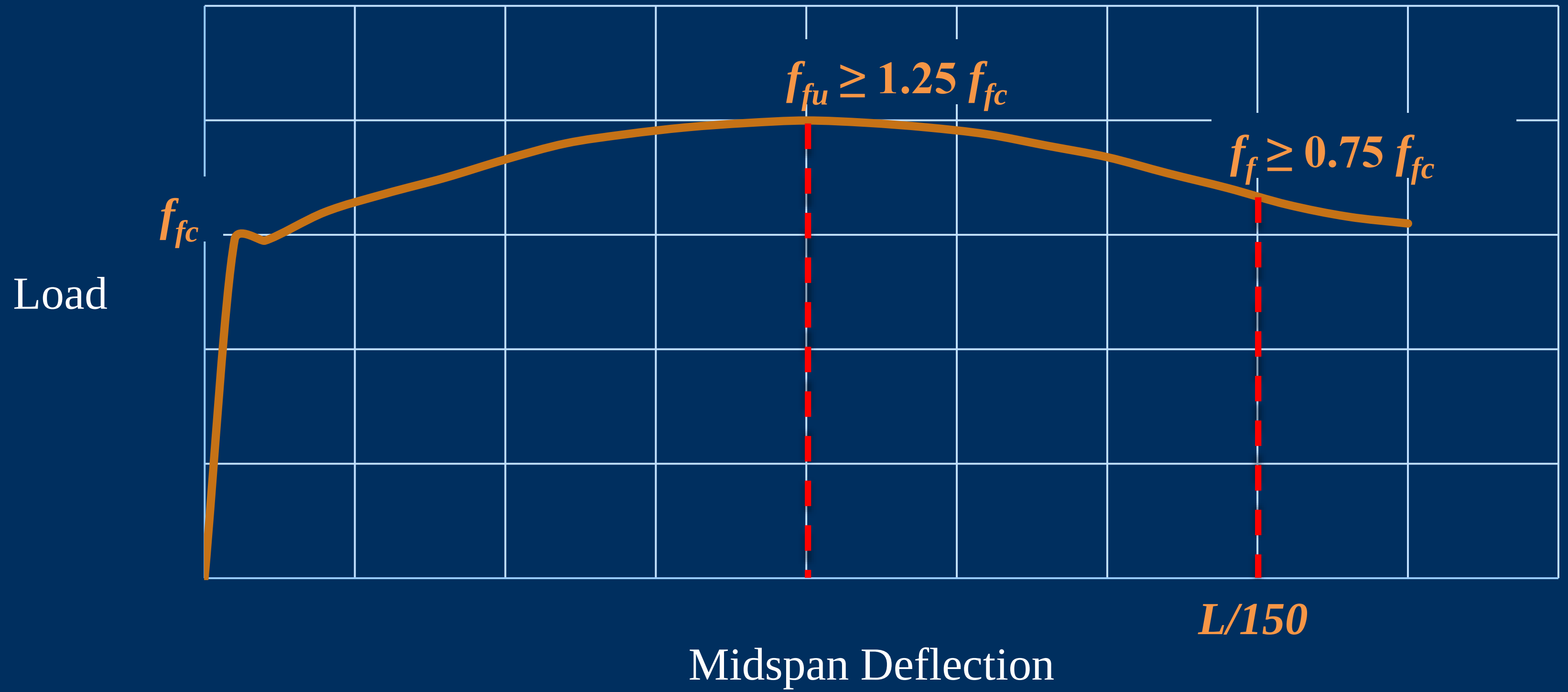
- Rapid implementation of cost-competitive UHPC bridge components and systems
- Train precasters to produce the material for a reasonable cost and with minimal disruption to their current production practices
- Develop materials and structural design guidelines
- Fully worked out design examples to help train designers
- Introduce the least amount of change to the current AASHTO LRFD Bridge Design Specifications, and to ACI 318

Definition of PCI-UHPC for Precast Pretensioned Members

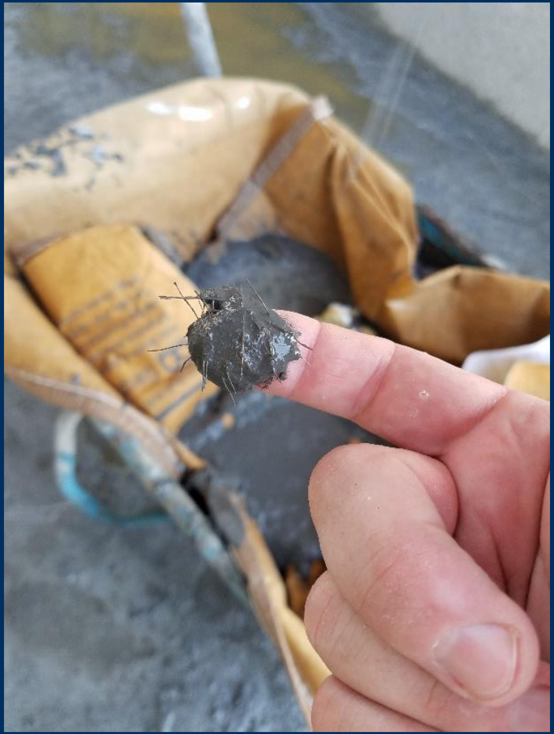
- Compressive strength, ASTM C1856, C109, 3”x6” cylinders
  - At service = 17.4 ksi (120 MPa) Required!
  - At prestress release = 10 ksi (70 MPa) Recommended

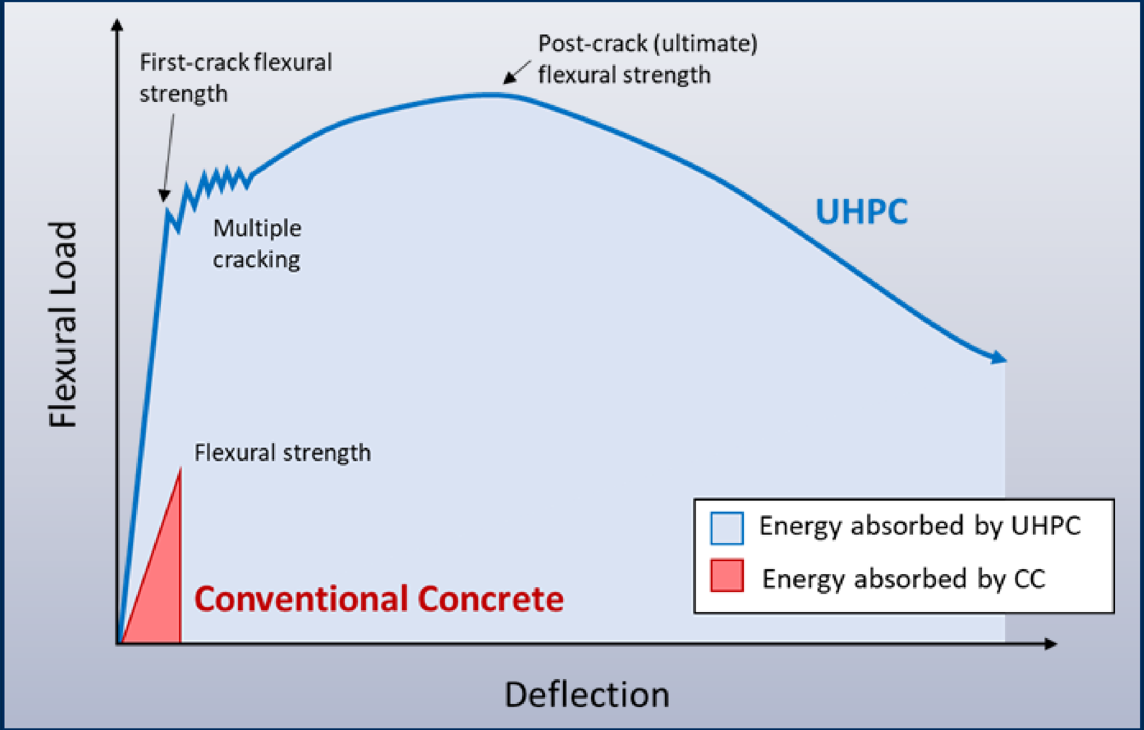
Note: lower strength at release may be permitted for lightly prestressed members.

Flexural Tension Requirements, using ASTM C1609 Standard Testing; 4”x4”x14” prism. IMPORTANT!



Tensile Strength and Ductility





Durability of PCI-UHPC vs. Conventional Concrete

Property	Conventional Concrete	UHPC
Electrical Indicator of Chloride Penetration Resistance, Coulombs	~4,000	32
Chloride Diffusion Coefficient, m^2/s	$\sim 5 \times 10^{-12}$	0.13×10^{-12}

PCI Project Strategy

- AASHTO already recognizes up to 15 ksi concrete. Extending to 17.4 ksi should not be a big challenge.
- 17.4 ksi is adequate compressive strength for most practical applications. Increasing the compressive strength requirement adds more cost with no apparent benefit.
- The distinguishing property of UHPC is its **tensile** capacity. The PCI-UHPC material has high limits and requires at least 2% of high strength, high aspect ratio fibers.
- It is our goal to take the current knowledge, confirm it, simplify it, and put it in practical guidelines.
- To compete with conventional concrete on a first cost basis, we target (1) material cost to 30% of prebagged commercial cost and (2) concrete volume to 50% of conventional products.

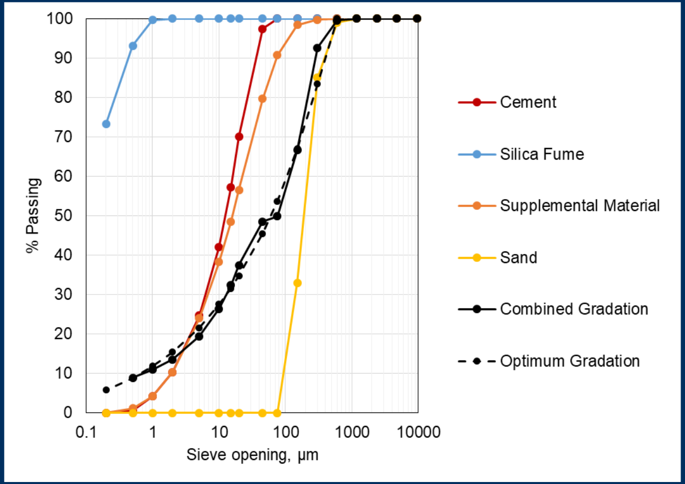
Development of Mix Designs using Locally available Materials

Mix Design and Testing

Predict mix proportions based on particle packing.

Trial batch in lab to achieve 9-inch flow.

Trial batch in plant and verify performance.



Mix Design and Testing

Predict mix proportions based on particle packing. → Trial batch in lab to achieve 9-inch flow. → Trial batch in plant and verify performance.

Evaluate:
- Flow spread
- Compressive strength
- Flexural performance
- ...

Temperature and Flowability

- Goal is to have as much flow spread as possible without segregation: 8 to 11 inches at point of placement
- Temperature before placement should be as low as possible: 65 to 85° F, preferably close to 65!
- Temperature after placement and finishing should be as high as possible: 160° for PCI standard curing and 194° for UHPC thermal post curing.

Performance Achieved

Property	Target (PCI-UHPC)	Phase I (Box Beam)	Phase II (Decked I-Beam)
Compressive Strength			
28-days (lab-cured), psi	--	18,970	21,410
At service (match-cured), psi	≥ 17,400	19,780	22,290
Flexural Strength			
First-Peak, psi	≥ 1,500	1,960	1,770
Peak, psi	≥ 2,000	3,170	3,450
Peak, % of first peak	≥ 125%	162%	200%
Residual at L/150, % of first-peak	≥ 75%	137%	146%

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Structural Design

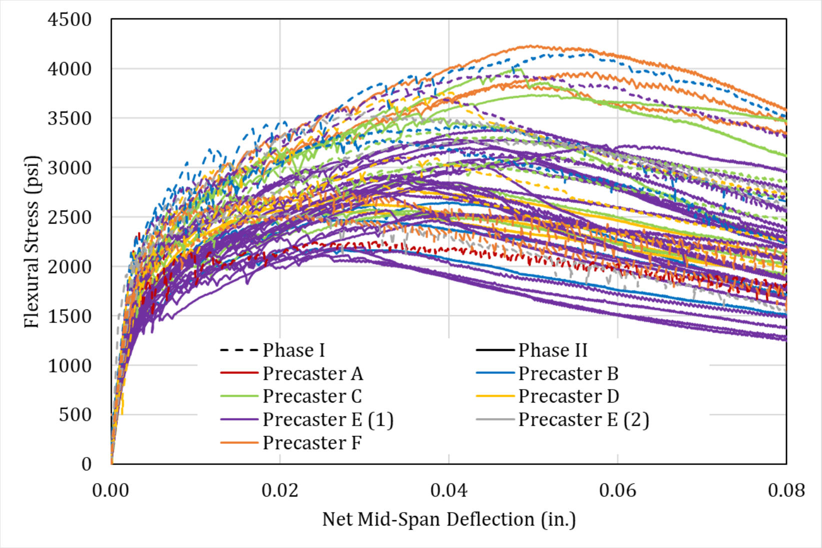
Structural Design Guidelines

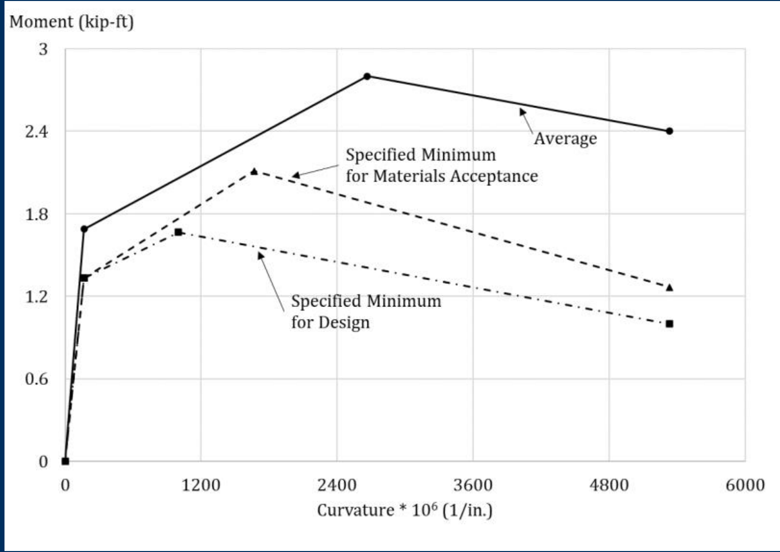
- Flexure, Creep, Shrinkage, Prestress Losses
- Vertical Shear
- Interface Shear
- Strand Bond
- End Zone Reinforcement

Flexure, Service Limit State

- Linear elastic uncracked section analysis, as currently in AASHTO LRFD Bridge Design Specifications (AASHTO)
- Concrete modulus, assumed = 6,500 ksi
- Initial Prestress Loss: same as in AASHTO, conservatively ignoring autogenous shrinkage
- Long Term Effective Prestress= 202.5-40.5 = 162 ksi
- Allowable compressive stress limits as currently in AASHTO
- Tensile stress at release to 0.75 ksi
- Tensile stress at service to 1.00 ksi

Inverse Analysis





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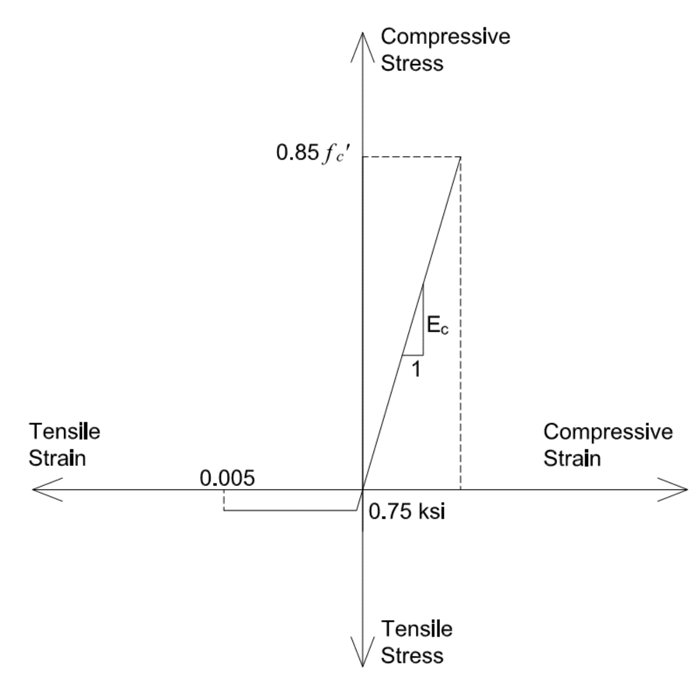
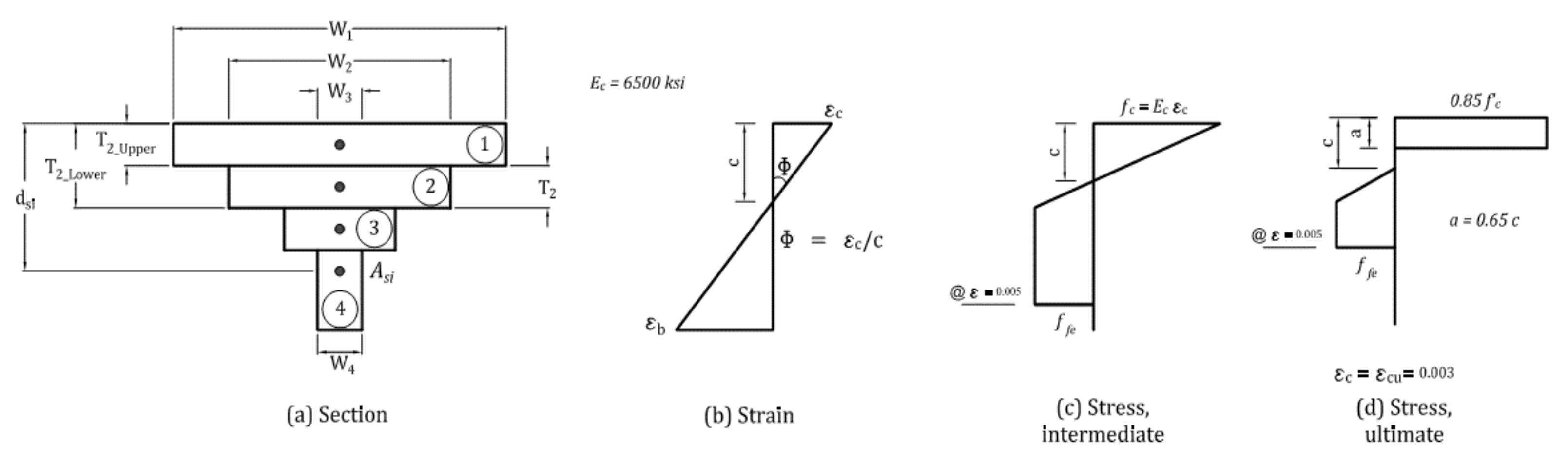
Inverse Analysis Results



Flexural Strength Design Process



(a) Develop moment-curvature curve; Determine peak moment, M_{n1}

(b) Use ultimate strain of 0.003, and rectangular stress block to get M_{n2}

(c) The peak capacity is the larger of M_{n1} and M_{n2}

Recommended Short Cut for Prestressed Members

- For prestressed concrete, strand is the dominant tension element

- No change to strain compatibility analysis in AASHTO

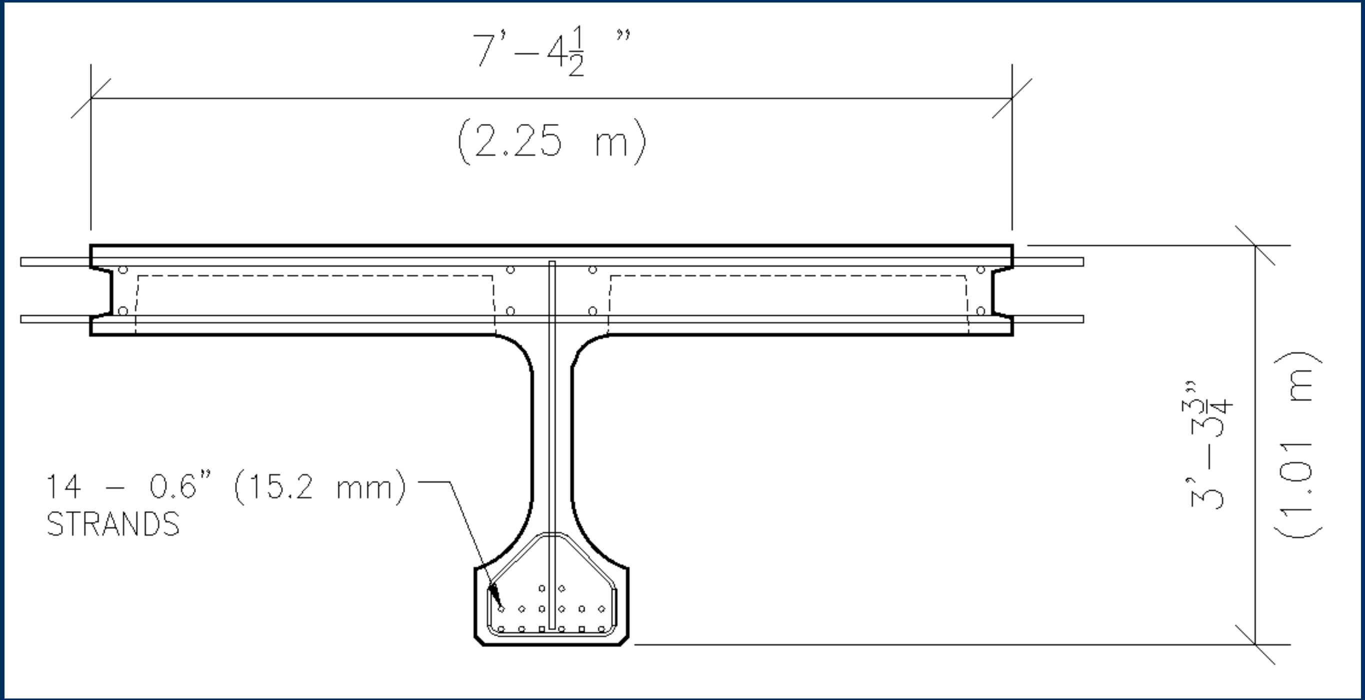
- Use available commercial software

Recommended Design in Transverse Direction

- Examples: top flange of decked I-beam and box beam are
- Not prestressed
- Ribbed slab are structurally optimum
- For T-sections:
  - No rebars for negative moment
  - Likely, will need rebars in the stems for positive moment
- Resistance factor: (a) fibers only, use 0.75; (b) fibers with bars, use

$$\varphi = 0.75 + 0.30\left(\frac{M_{nb}}{M_n}\right) \leq 1.0$$

Product Testing in Flexure, PCI-UHPC Decked I-Beam



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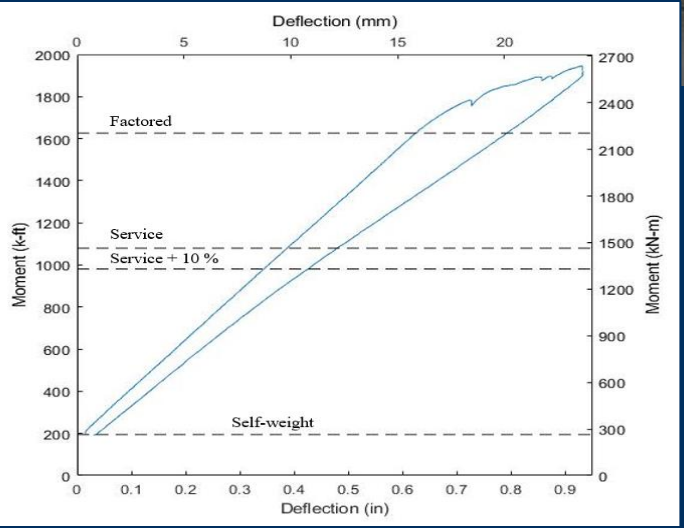
Decked I-Beam for FACCA, Inc, Ontario, Canada

- 50’ long decked bridge girder
- Tests in flexure (3-pt), shear (both ends), and local deck and diaphragm tests

Flexure Testing

- Loaded to about 10% over factored moment
- No visible cracking
- However, strain data suggests cracking at about 1800 kip-ft



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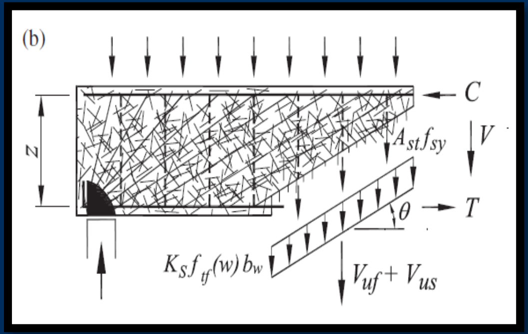
Vertical Shear

Shear Strength Design Recommendation

* Use AASHTO's general MCFT, with modifications
* $V_n = V_c + V_s + \boldsymbol{V_f}\textbf{(new)} + V_p$
  * $V_c = 0.0316\beta\sqrt{f_c'}b_v d_v$
  * $\varepsilon_s = \frac{(M_u/d_v)+(V_u-V_p)-P_e}{(E_s A_s+E_p A_{ps})}$
  * **Use negative strain** $\boldsymbol{\varepsilon_s = \frac{(M_u/d_v)+(V_u-V_p)-P_e}{(E_s A_s+E_p A_{ps}+E_c A_{ct})}}$
  * $\beta = 4.8/(1+750\varepsilon_s)$
  * $\theta = 29 + 3{,}500\varepsilon_s$
  * $\boldsymbol{V_f = f_{rr}\cot\theta\, b_v d_v}$
  * f_{rr} = residual rupture stress, recommended = **0.75 ksi**



f_{rr} is the key parameter!

Experimental Shear Program

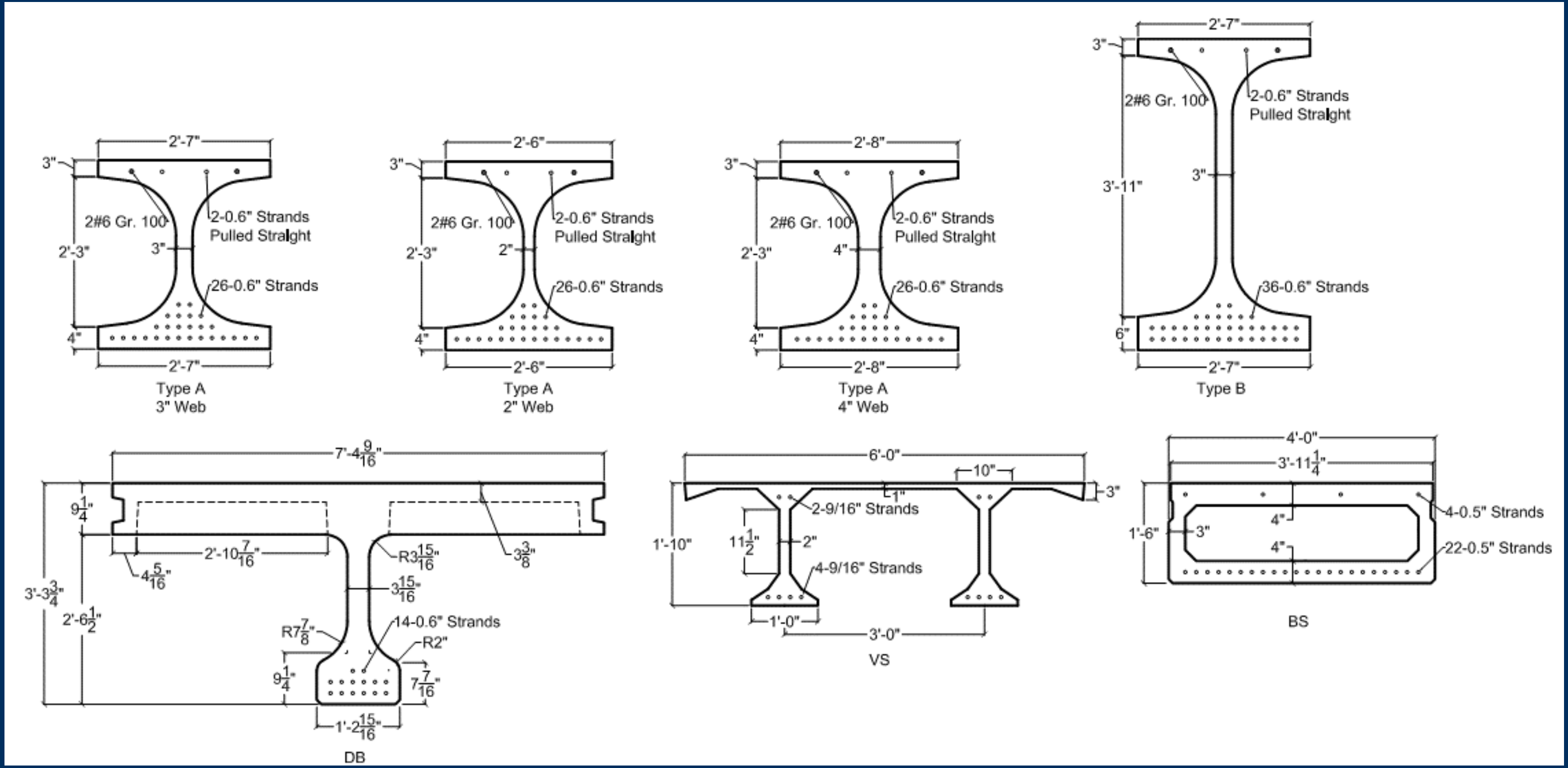
Shear Component Testing Considered:

Prestress level; Stirrups; Web Thickness; Fiber Length; Shear Span/Depth Ratio; Member Size and Shape; Tension Tie Demand; Effect of Thermal Curing

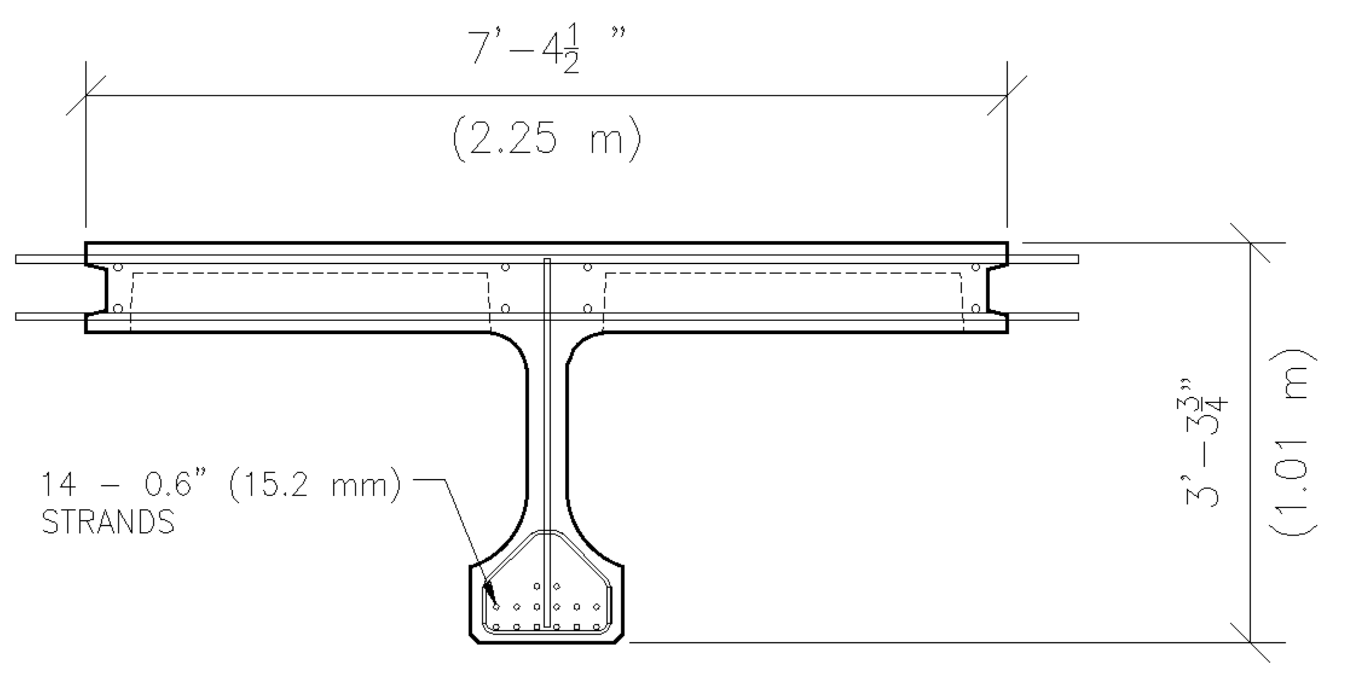
Full Product Shear Testing:

- Ribbed building floor slabs
- Bridge box slabs
- Building and Bridge Decked I-Beams

Test Specimens



Product Testing in Shear, PCI-UHPC Decked I-Beam

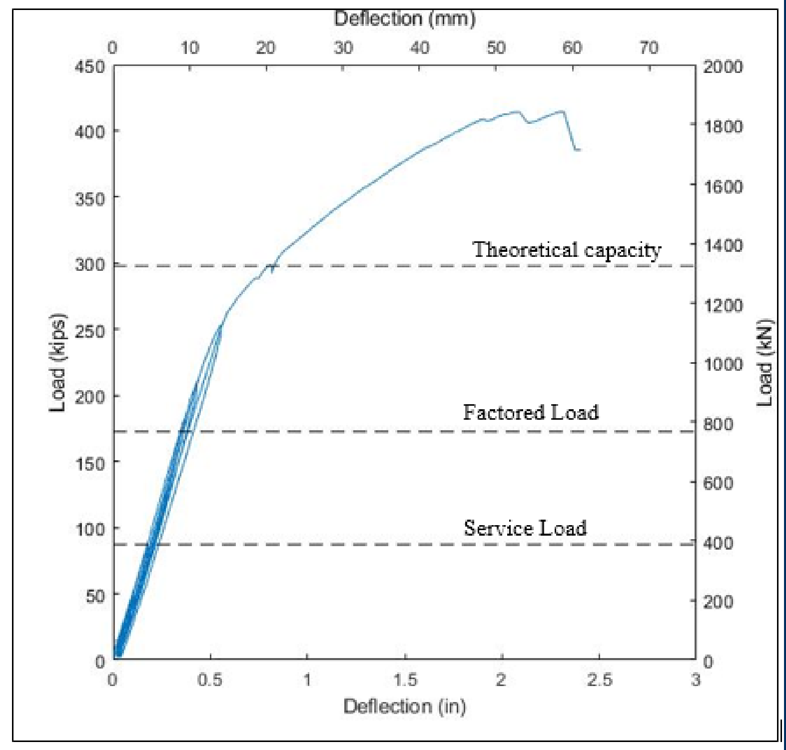


Bridge Decked I-Beam

- This beam was meant to show that a beam with an integrated deck panel provides a fast and efficient design

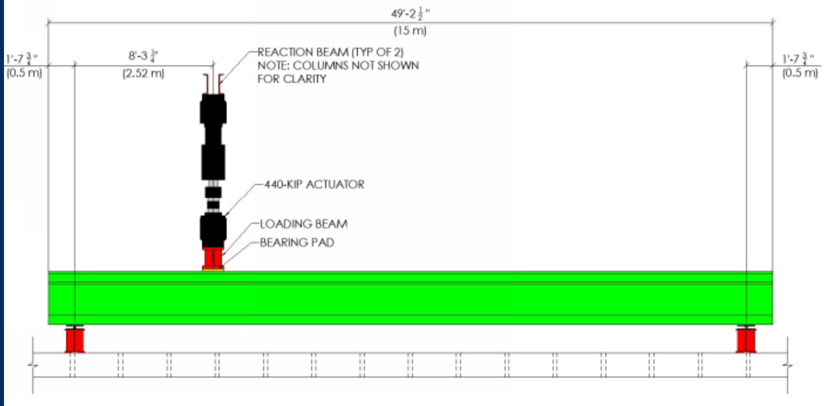
DIB Shear Tests-

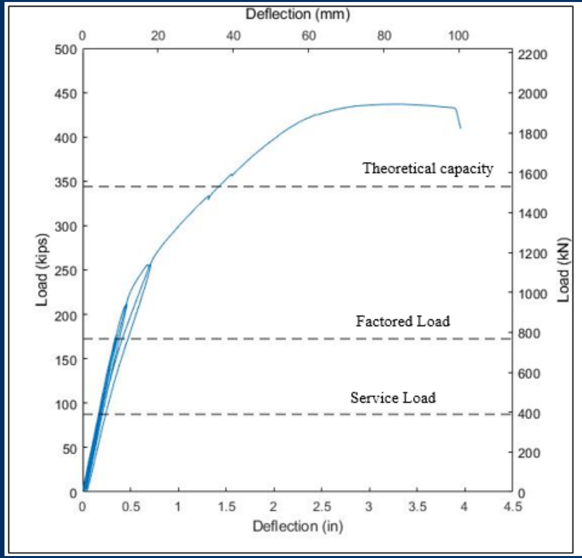
End With No Stirrups

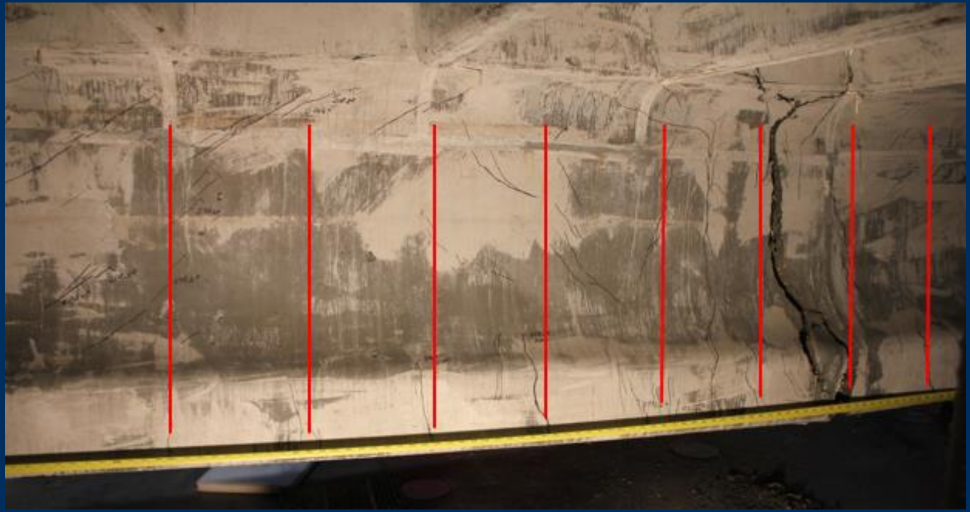


DIB Shear Test, **End With #5@10”**

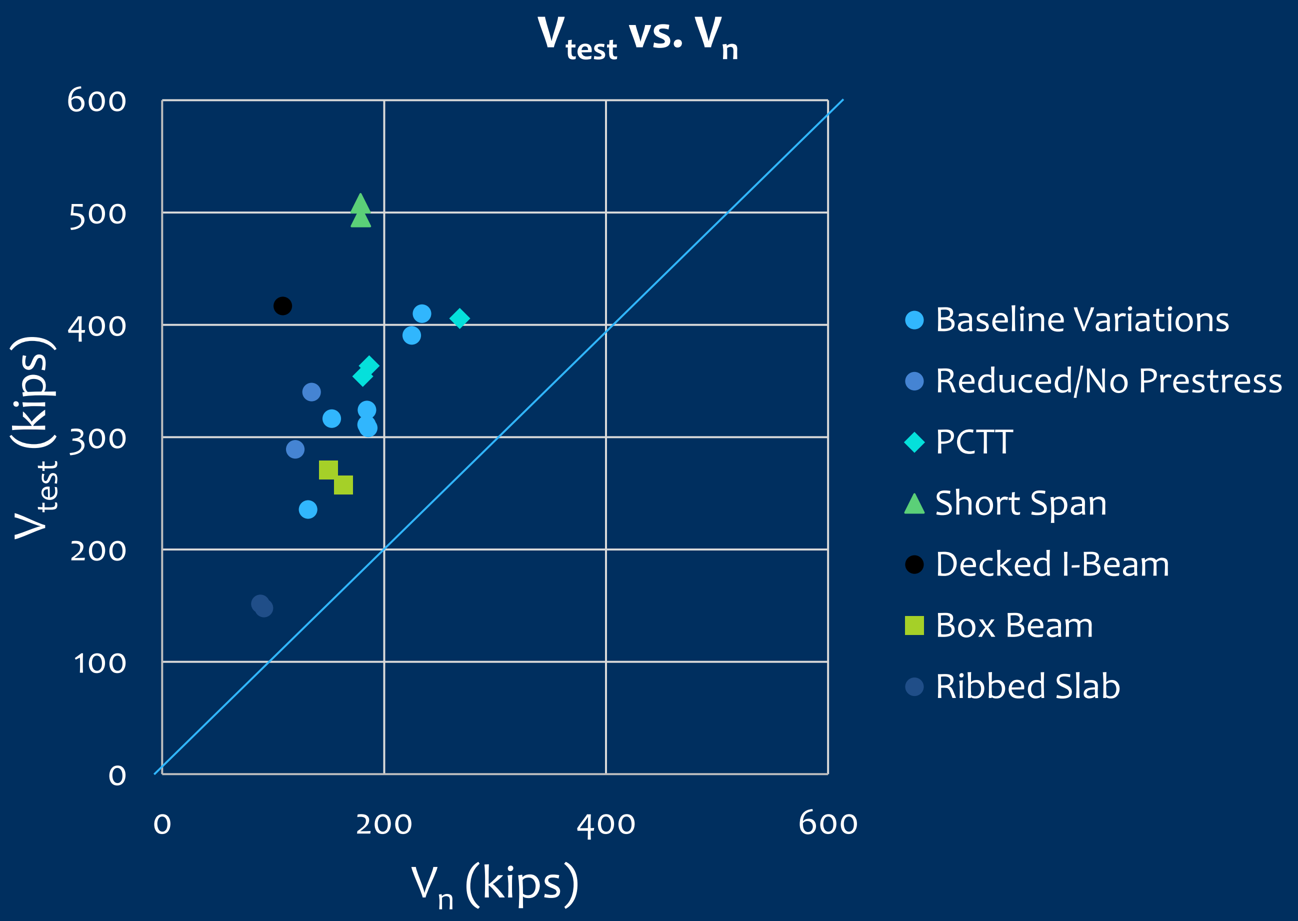
- Failed in flexure (strand rupture) @ 437 kips
  - About the same shear as the other end
  - **Flexure cracks initiated at each stirrup location**



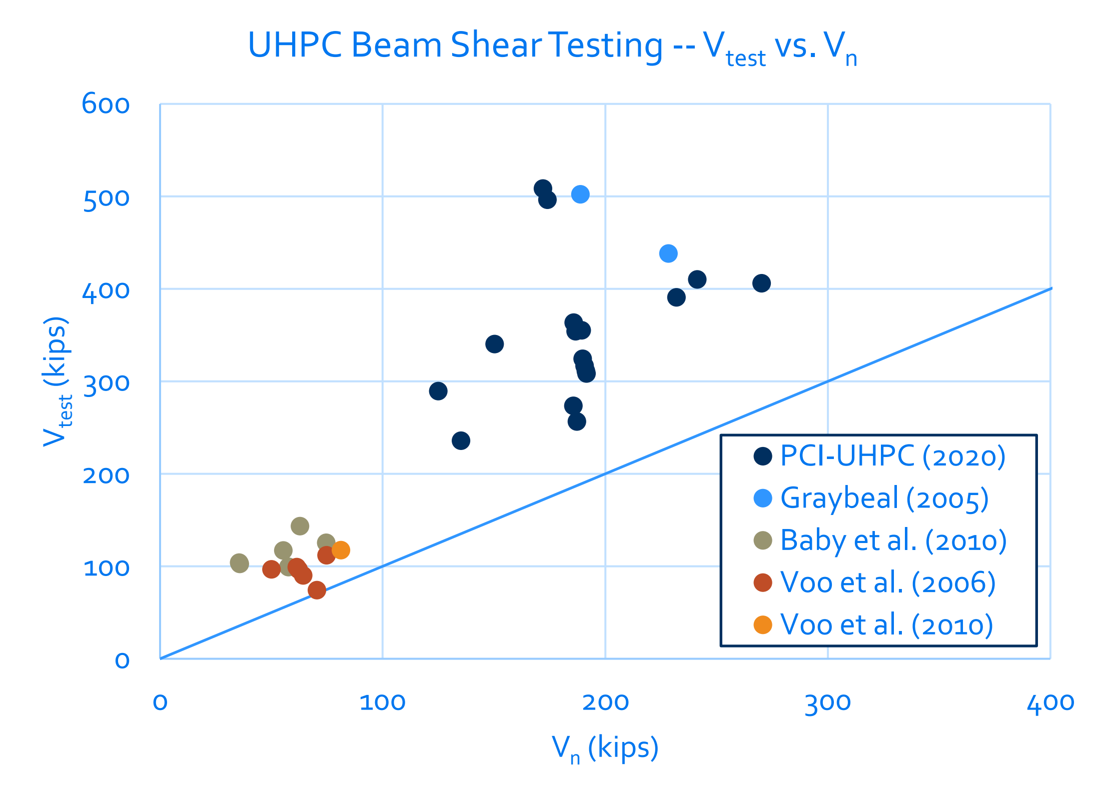




Experimental vs. Theoretical Shear Strength, $f_{rr} = 0.75$ ksi



…Including Tests by Others



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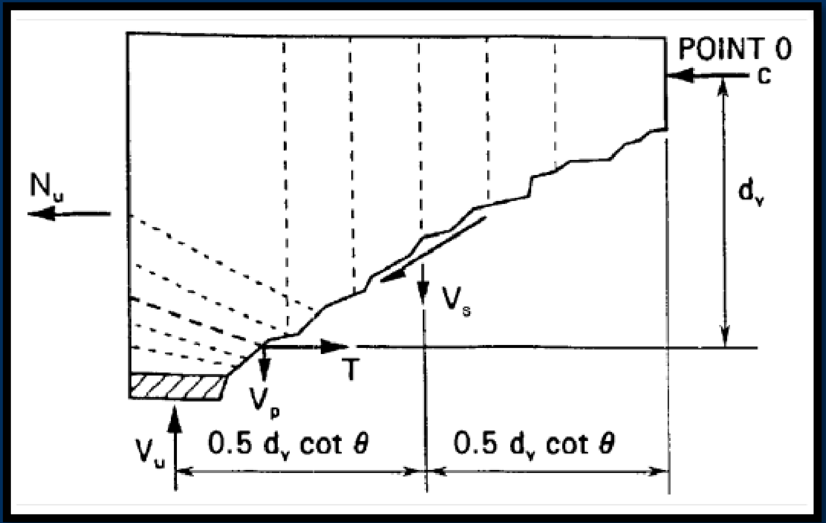
Tension Tie is Important. Two specimens with low anchorage gave relatively low shear capacity



$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5V_s - V_p\right)\cot\theta \quad \text{(AASHTO)}$$

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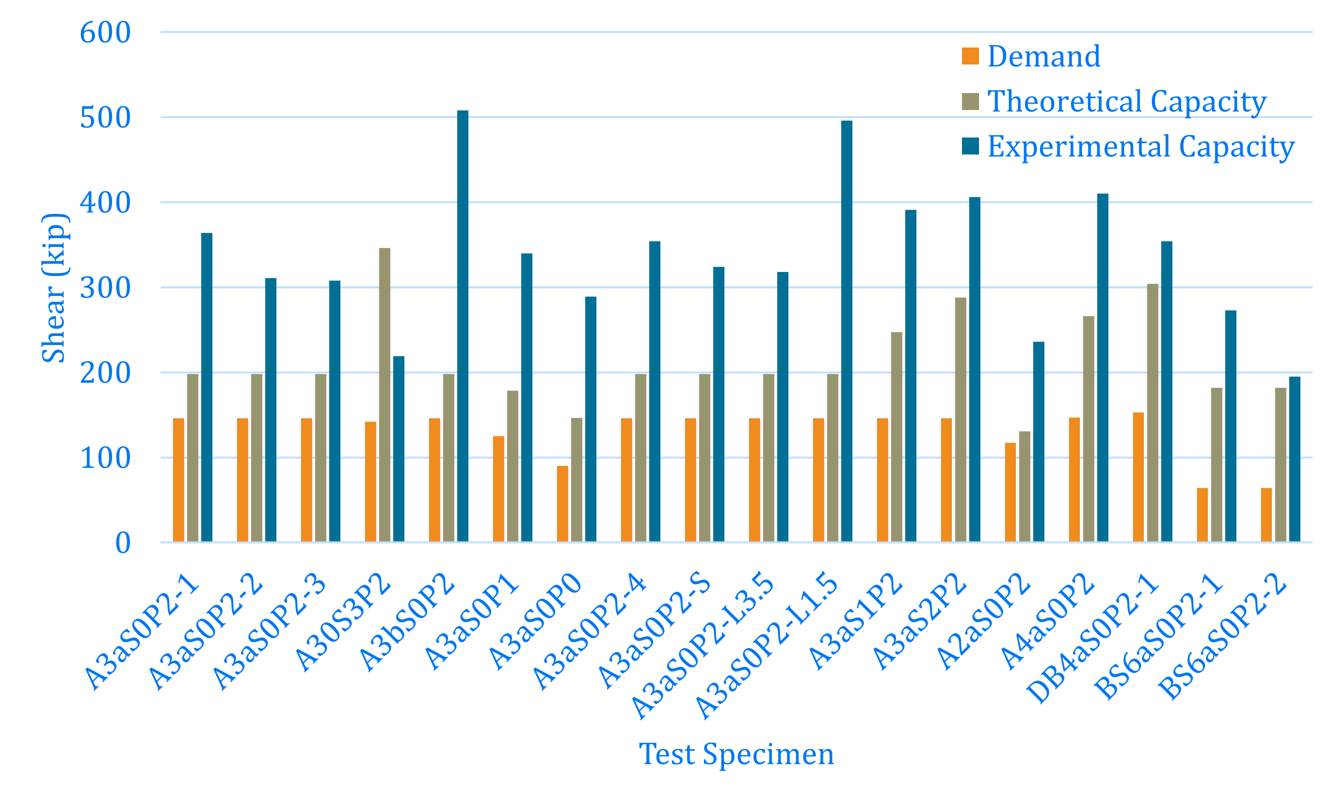
Most Importantly! Demand is much lower than capacity

Specimen	Experimental Capacity /Theoretical Capacity	Experimental Capacity/Demand
A3aS0P2-1	1.84	2.49
A3aS0P2-2	1.57	2.13
A3aS0P2-3	1.56	2.11
A3bS0P2	2.57	3.48
A3aS0P1	1.91	2.72
A3aS0P0	1.97	3.21
A3aS0P2-4	1.79	2.42
A3aS0P2-S	1.64	2.22
A3aS0P2-L3.5	1.61	2.18
A3aS0P2-L1.5	2.51	3.40
A3aS1P2	1.58	2.68
A3aS2P2	1.41	2.78
A2aS0P2	1.80	2.02
A4aS0P2	1.54	2.79
DB4aS0P2-1	1.16	2.31
BS6aS0P2-1	1.50	4.27
BS6aS0P2-2	1.07	3.05



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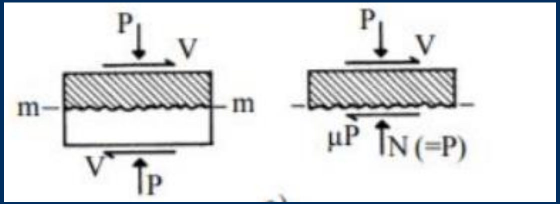
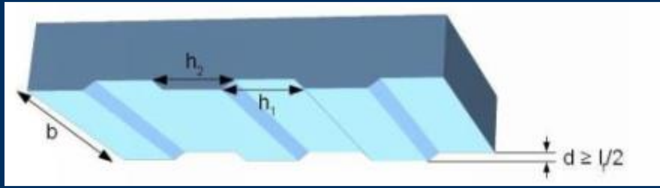
Interface Shear

Interface Shear Behavior



Shear Friction Hypothesis (Birkeland H. and Birkeland P., 1966)



Fluted Joint Details as Specified by AFGC (2013)

Proposed Model

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

Methods of Connection

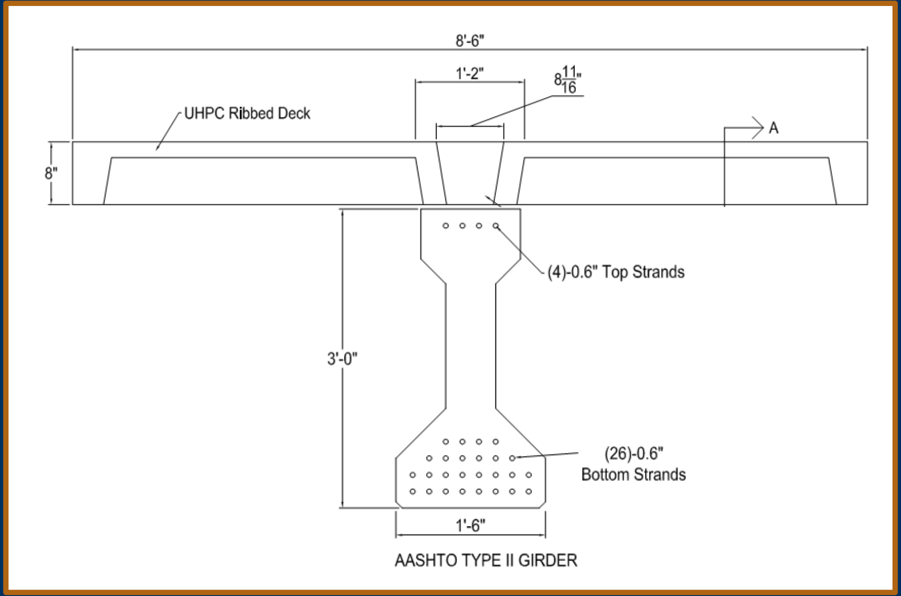
- Best solution to “roughen” the interface is to use a form liner
- Mechanical interlock is more significant than cohesion
- Need to use connecting bars for uplift reaction

Composite Bridge Beam

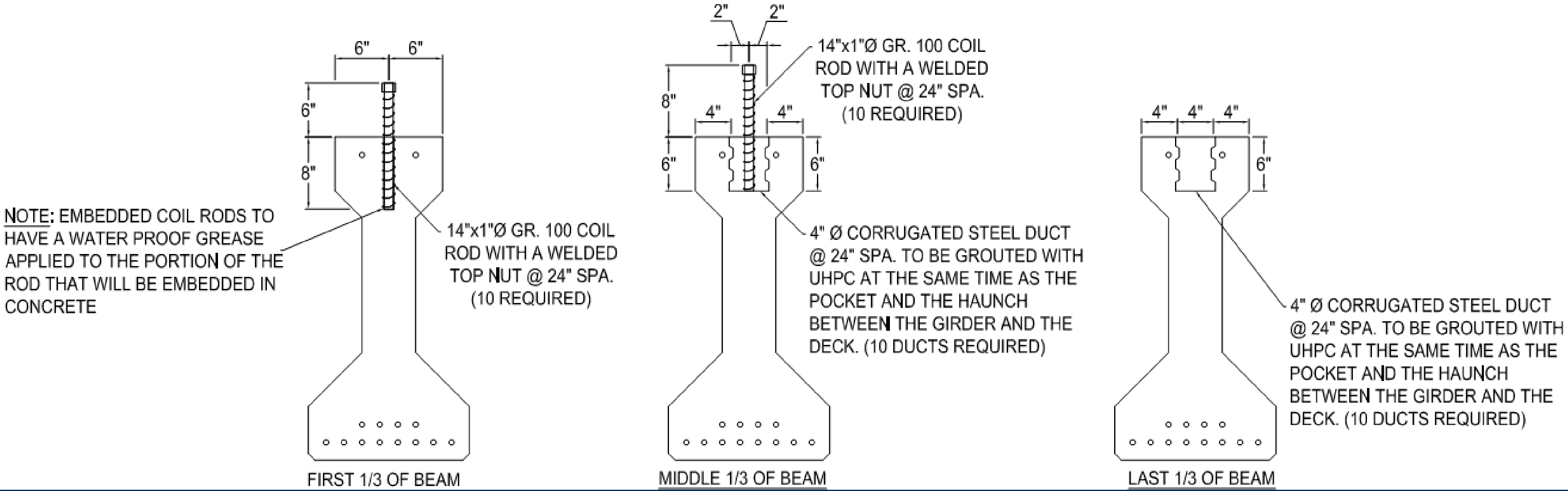
The objectives of this test are:

(1) To assess the adequacy of three different interface shear connections

(2) To demonstrate possible adjustments for camber and cross slope controls



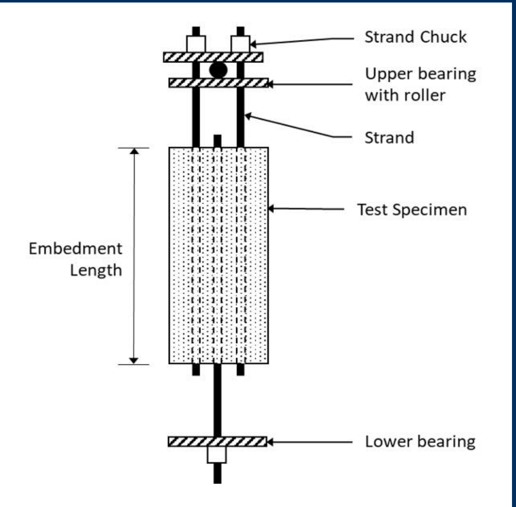
Connection Details



Decked I-Beam Assembled at SCP Tampa Plant, Ready for Shipment to FDOT Lab

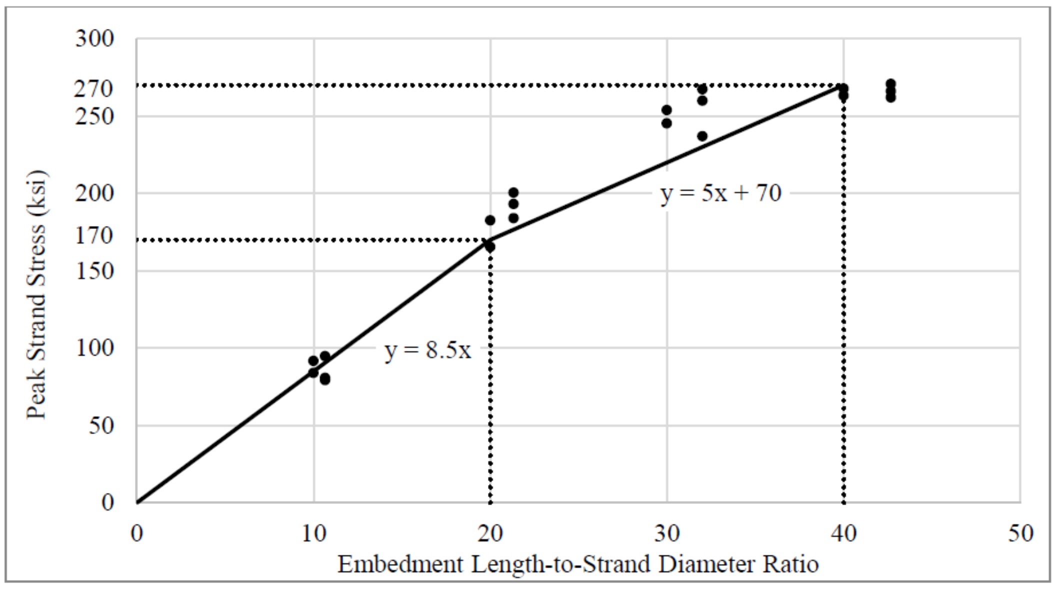
- Decked I-Beam

Strand and Bar Development



$$L_d = L_t + 0.2d(f_{ps} - f_{pe})$$

where $L_t = 20d_b$



Confirming work by FHWA

Peak Strand Stress vs ℓ/d_b (20 of 35 test results)

Optimized Products developed in the PCI-UHPC Program

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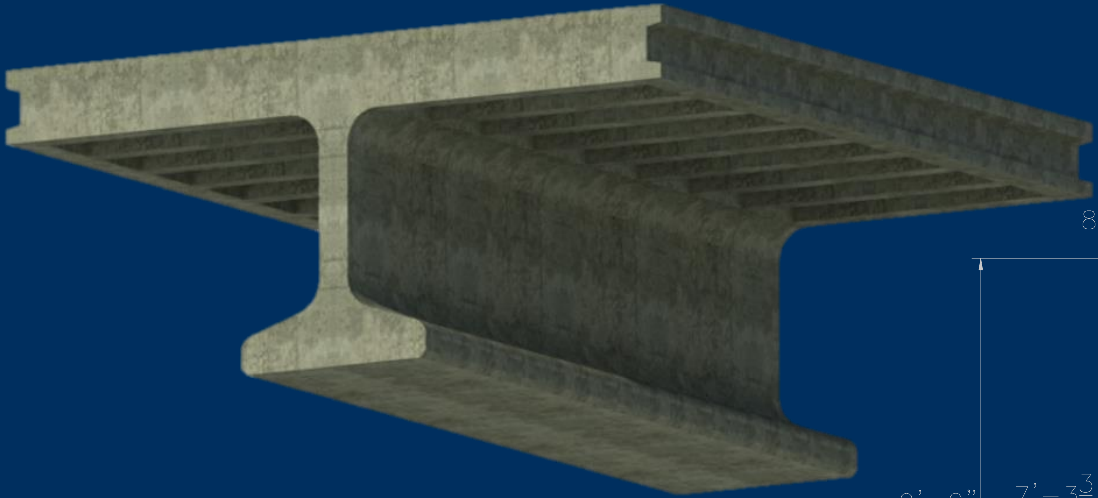
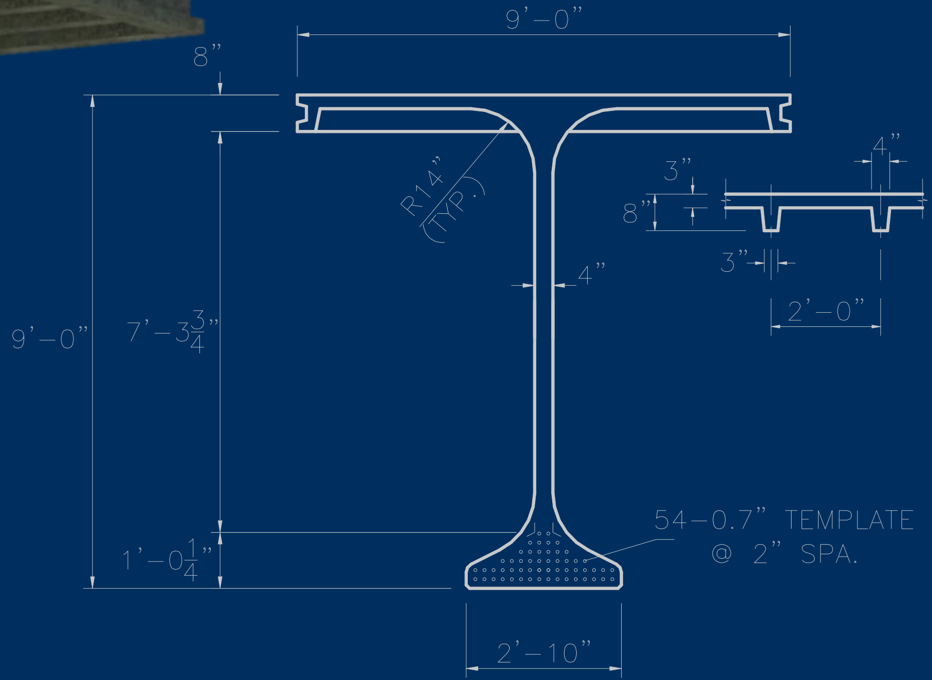
Decked I-Beams

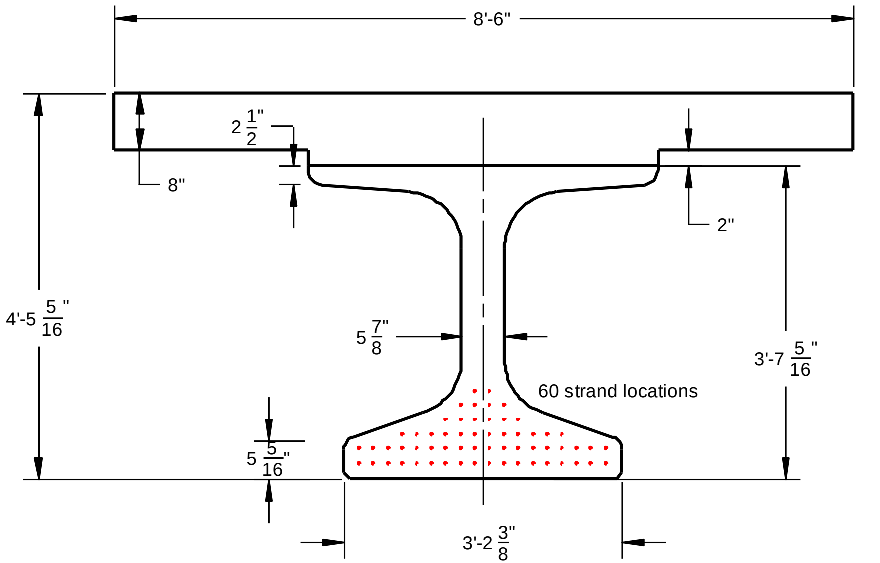




Comparison with Conventional Concrete,

Span = 110 ft, Width = 50 ft, spacing = 8.5 ft.

	Conventional NU 1100
Total depth (in.)	**53.31**
Compressive Strength at service, ksi	8
Compressive strength at release, ksi	6
Volume of beam, CY	20.00
Volume of deck, CY	25.80
Beam plus deck, CY	45.80
# of 0.7” Strands	**32**
Shear Reinforcement	YES
Deck Reinforcement	Both Directions



Nebraska NU 1100 (43.31") with 8" conventional deck

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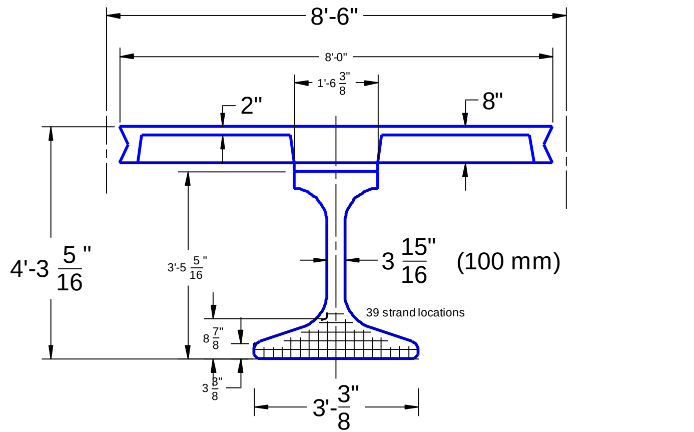
Two Stage UHPC Cross Section

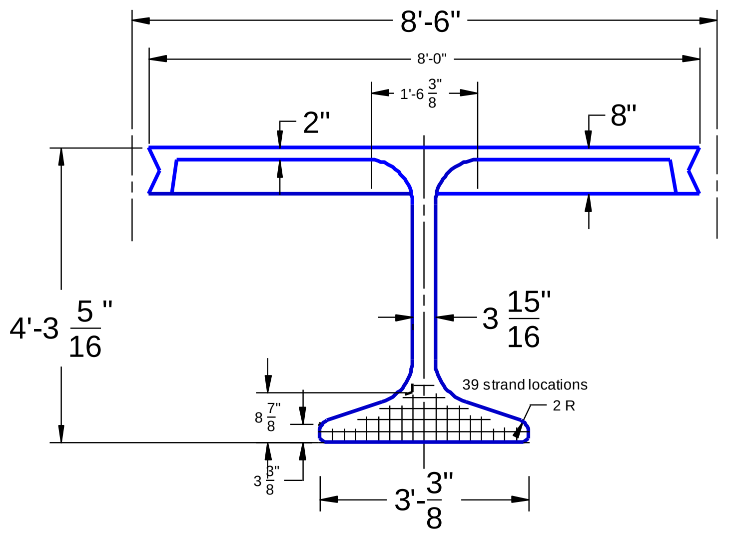
	Two-Stage UHPC, Modified NU100+ribbed slab	Percent reduction due to use of UHPC
Total depth (in.)	**51.31**	
Compressive Strength at service, ksi	18	
Compressive strength at release, ksi	10	
Volume of beam, CY	12.00	**40%**
Volume of deck, CY	13.7	**47%**
Beam plus deck, CY	25.70	**44%**
# of 0.7" Strands	**32**	
Shear Reinforcement	NO	
Deck Reinforcement	Transverse Only	Significant



Two-Stage UHPC Decked I-Beam based on NU1100

One-Stage Decked I Beam- Best Solution

	UHPC Decked-I-Beam	Percent reduction due to use of UHPC
Total depth (in.)	**51.31**	
Compressive Strength at service, ksi	18	
Compressive strength at release, ksi	10	
Volume of beam, CY	23.85	-
Volume of deck, CY	1.35	-
Beam plus deck, CY	25.20	**45%**
# of 0.7" Strands	**24**	
Shear Reinforcement	NO	
Deck Reinforcement	Transverse Only	Significant



UHPC Decked I-Beam based on NU1100

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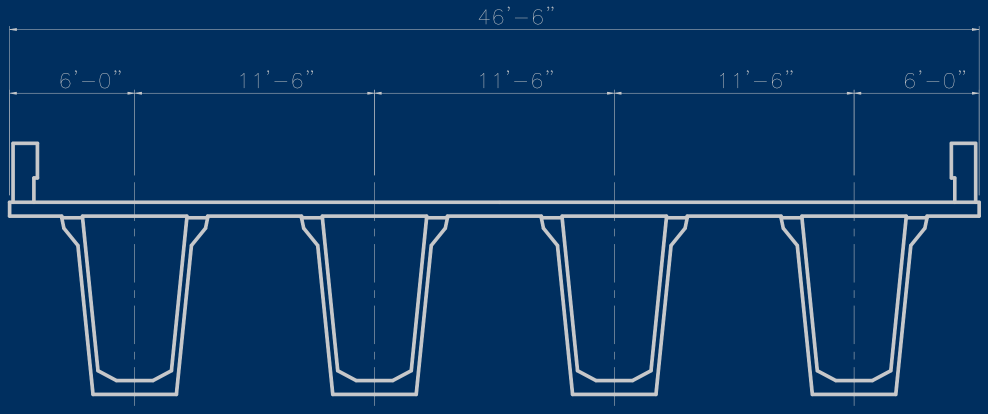
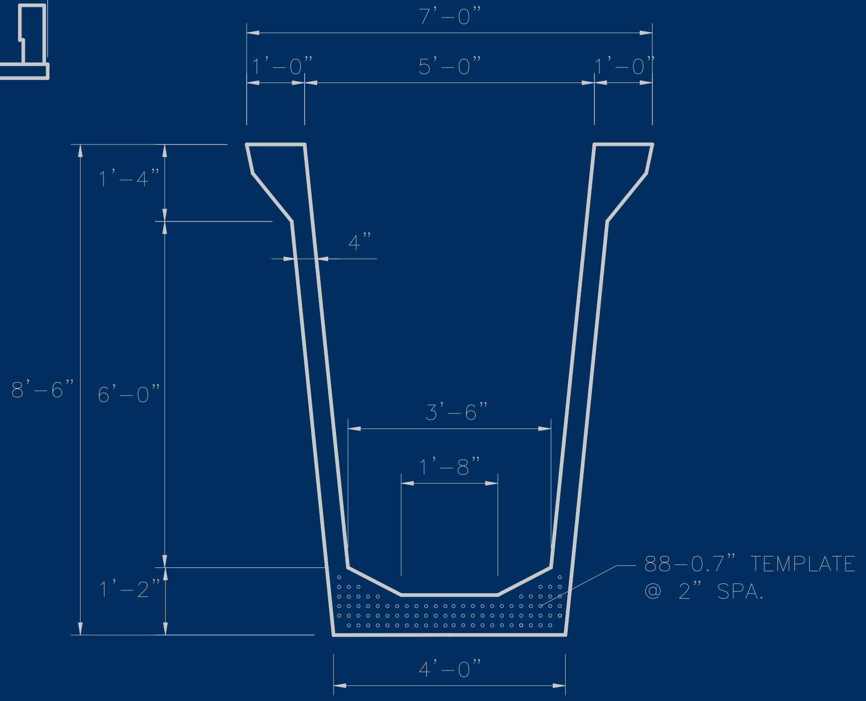
U-Beams

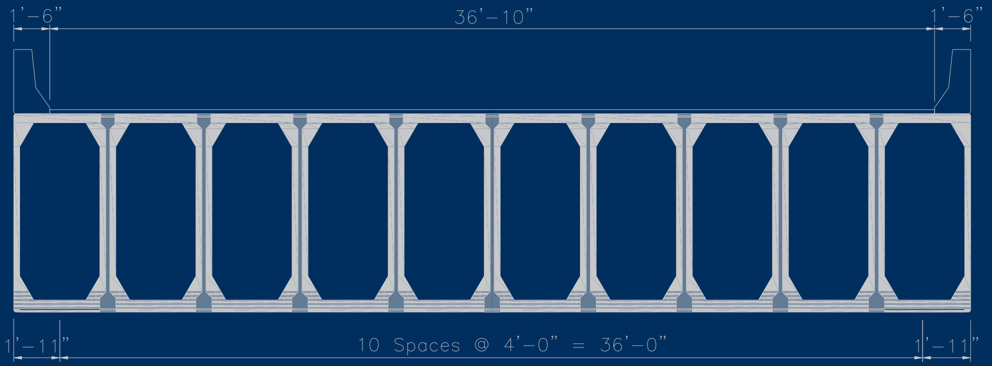


46'–6"
6'–0"
11'–6"
11'–6"
11'–6"
6'–0"

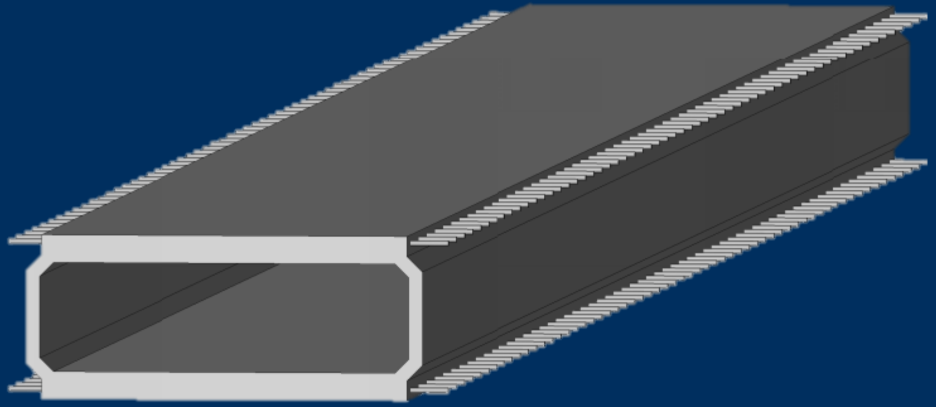


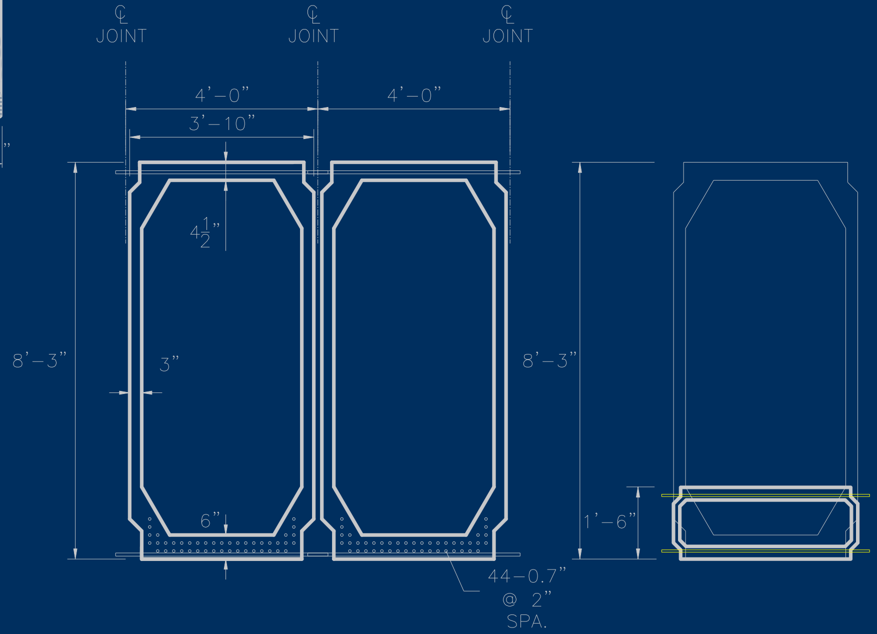
7'–0"
1'–0"
5'–0"
1'–0"
1'–4"
4"
8'–6"
6'–0"
3'–6"
1'–8"
1'–2"
88–0.7" TEMPLATE
@ 2" SPA.
4'–0"

Box Slabs



1'–6"
36'–10"
1'–6"
1'–11"
10 Spaces @ 4'–0" = 36'–0"
1'–11"

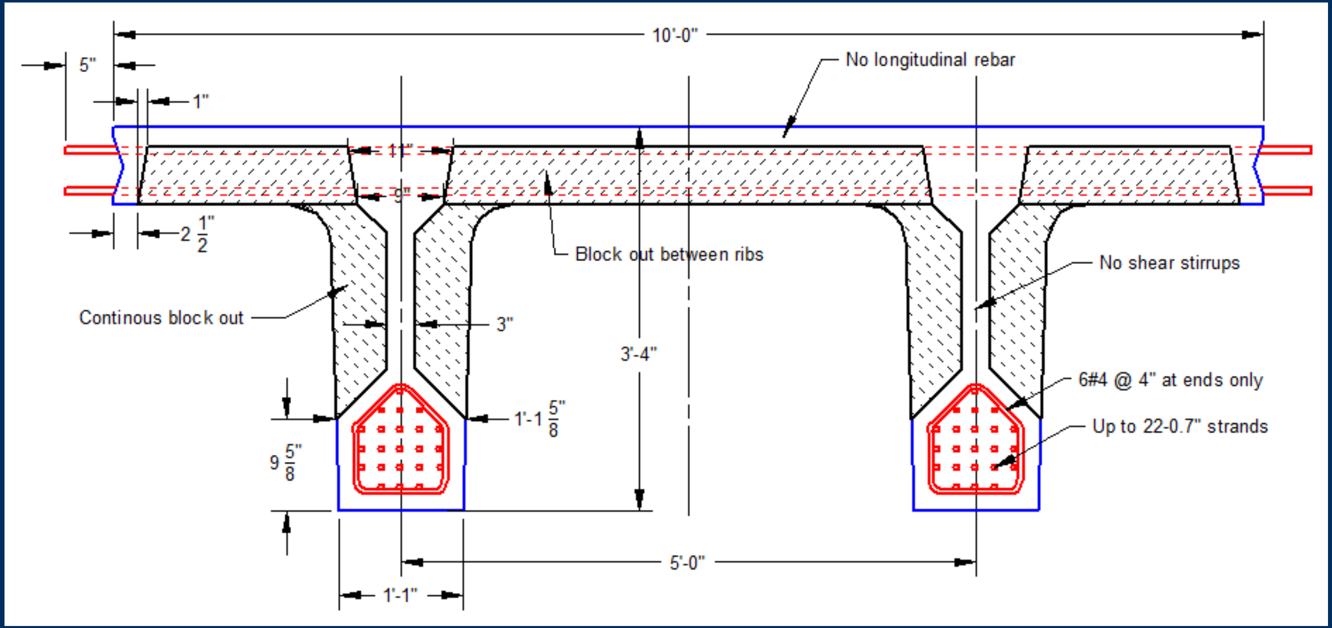


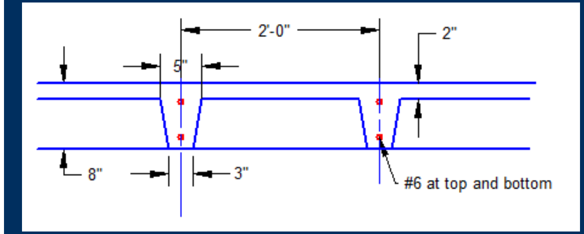


℄ JOINT
℄ JOINT
℄ JOINT
4'–0"
4'–0"
3'–10"
4½"
8'–3"
3"
6"
8'–3"
1'–6"
44–0.7" @ 2" SPA.

Optimization of Northeast Extreme Tee (NEXT)

Volume reduced from 43 to 23 cubic yards for a 90 ft long piece





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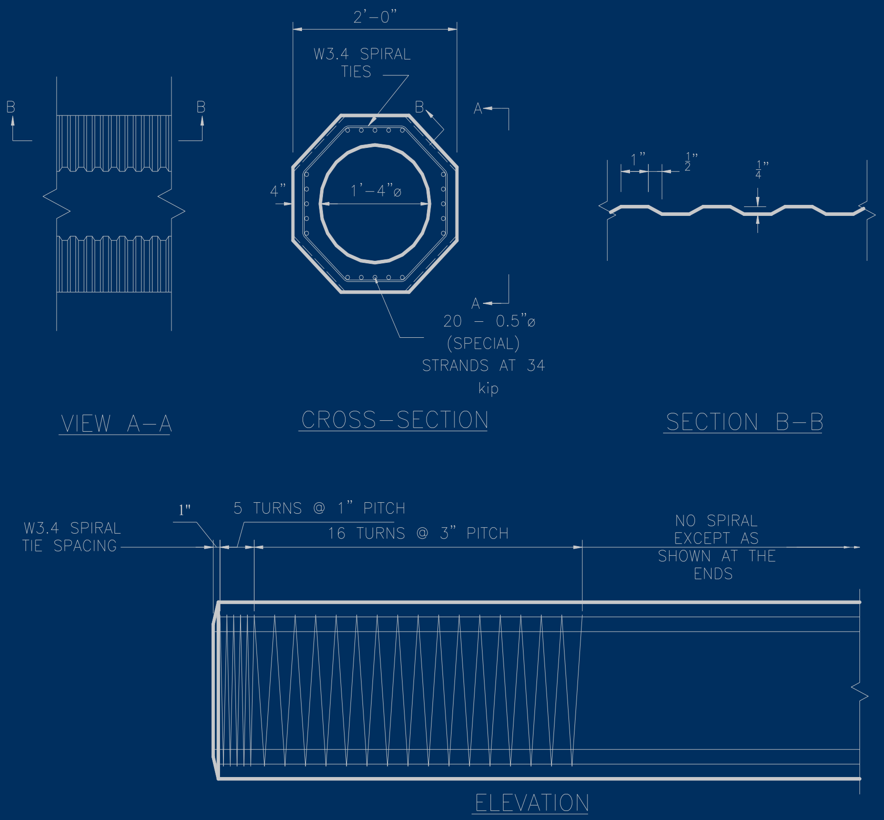
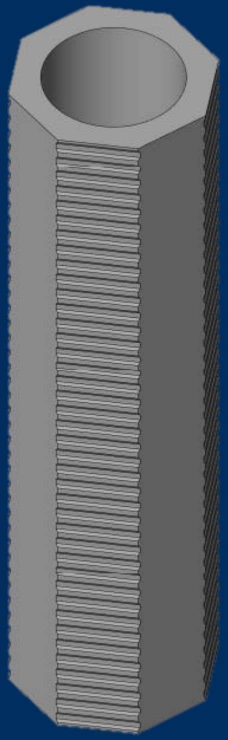
Optimization of Square Piles





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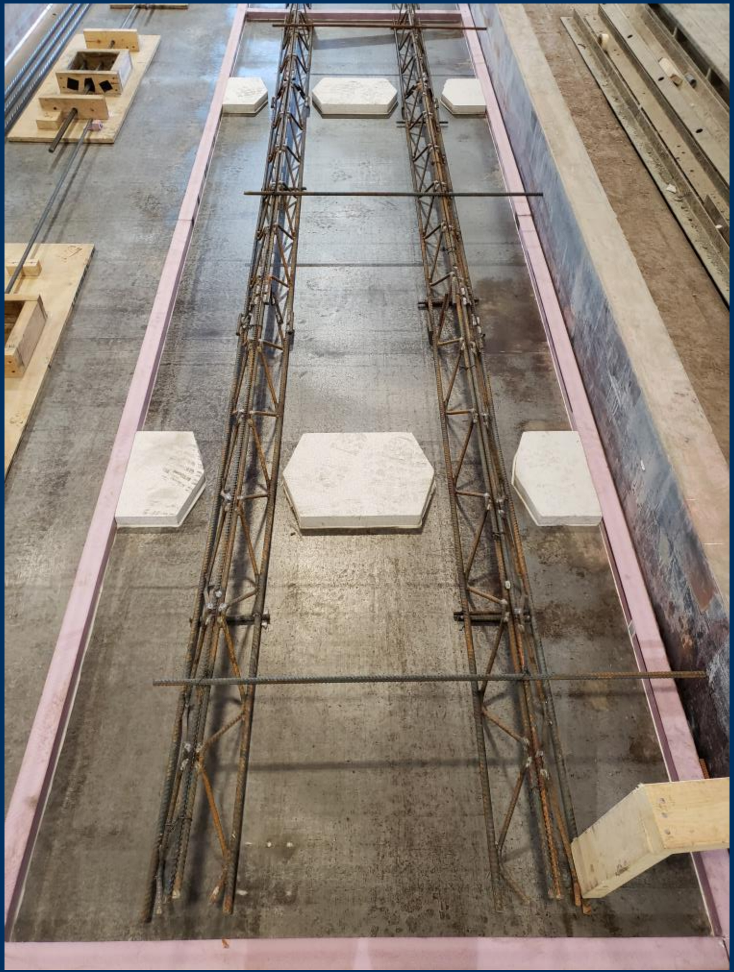
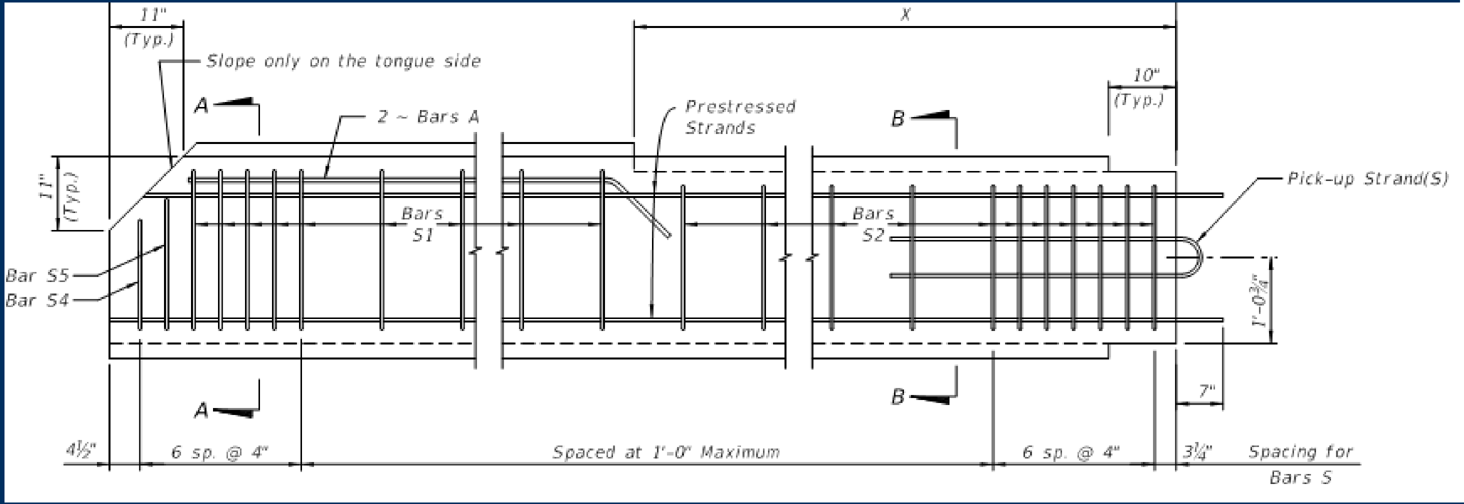
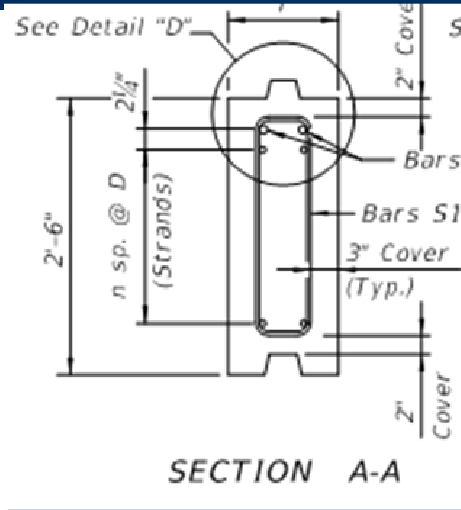
Deck Sub-Panels

1.5” thick UHPC deck sub-panel with a wire truss reinforcement

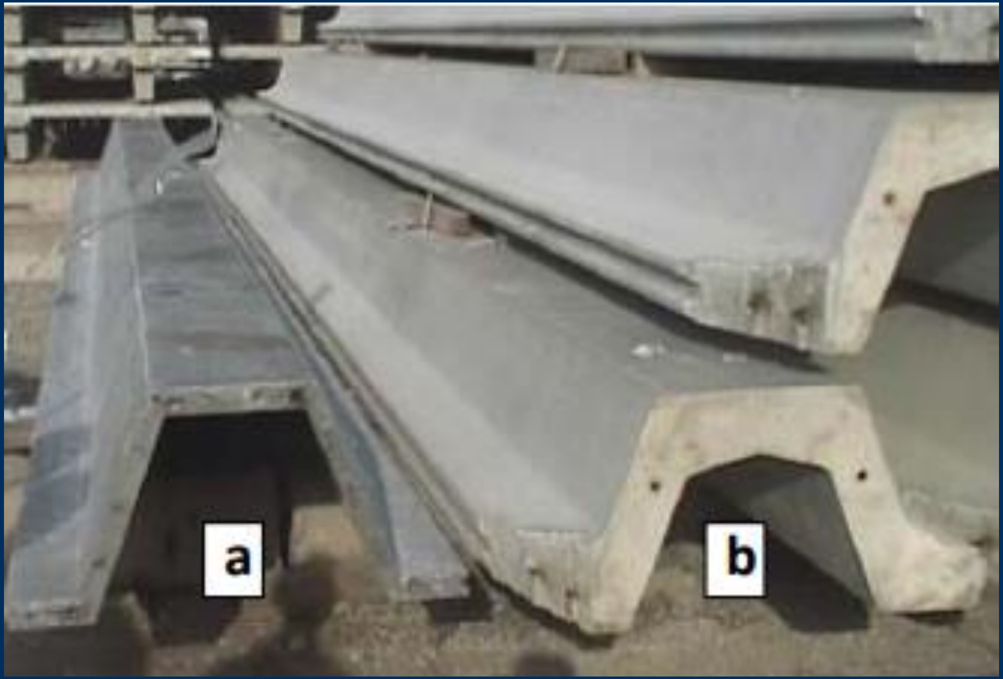


Typical Conventional Concrete Sheet Pile, 10-12" Thick





Sheet Pile in the Netherlands: UHPC (a) versus Conventional concrete (b)



(Grünewald 2004)
(Walraven and Schumacher 2005, Walraven 2007)
(Walraven 2007)

When can we start designing with PCI-UHPC?

The time is NOW!

Recipe for Success

1. Start with something simple
2. Many spans; relatively short 60-80 ft spans
3. Preferably aggressive environment site
4. Simple cross section; the Florida box slab is a top candidate
5. Aim for 50 percent reduction in conventional concrete volume
6. Aim for 80 percent reduction in rebars
7. Be conservative in your design

Summary and Conclusions

- UHPC produced with local precasters at 30% of previous cost
- Products must be structurally optimized to have about 50% volume. Little rebar. Easier to fabricate
- These two conditions result in cost competitive bridges. Durability, shipping, foundations, shoring, etc., are bonus
- PCI-UHPC It is good for all applications and all span ranges
- PCI research aims to give simple guidelines:
  - Based on current AAHTO provisions
  - Reflect the best knowledge we currently have from previous research and international codes