Fundamentals and Application of PCI Ultra-High-Performance Concrete

Presented at the GCPCI-PCEF Meeting
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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!

Load

Midspan Deflection

\[ f_f \geq 0.75f_{fc} \]

\[ f_{fu} \geq 1.25f_{fc} \]

\[ L/150 \]
Tensile Strength and Ductility

- First-crack flexural strength
- Multiple cracking
- Flexural strength
- Post-crack (ultimate) flexural strength

Comparison between UHPC and Conventional Concrete in terms of energy absorption.
Durability of PCI-UHPC vs. Conventional Concrete

<table>
<thead>
<tr>
<th>Property</th>
<th>Conventional Concrete</th>
<th>UHPC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Electrical Indicator of Chloride Penetration</td>
<td>~4,000</td>
<td>32</td>
</tr>
<tr>
<td>Resistance, Coulombs</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chloride Diffusion Coefficient, m²/s</td>
<td>~5 × 10⁻¹²</td>
<td>0.13 × 10⁻¹²</td>
</tr>
</tbody>
</table>
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.

![Graph showing particle size distribution](image)
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
- ...

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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

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

7'-4\frac{1}{2}''

(2.25 m)

14 - 0.6'' (15.2 mm) STRANDS

3'-3\frac{3}{4}''

(1.01 m)
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
Vertical Shear
Shear Strength Design Recommendation

* Use AASHTO’s general MCFT, with modifications

\[ V_n = V_c + V_s + V_f (\text{new}) + V_p \]

* \( V_c = 0.0316 \beta \sqrt{f_r \gamma_b} d_v \)

* \( \varepsilon_s = \frac{(M_u/d_v)+(V_u-V_p)-P_e}{(E_s A_s+E_p A_{ps})} \)

* Use negative strain \( \varepsilon_s = \frac{(M_u/d_v)+(V_u-V_p)-P_e}{(E_s A_s+E_p A_{ps}+E_c A_{ct})} \)

* \( \beta = 4.8/(1 + 750 \varepsilon_s) \)

* \( \theta = 29 + 3,500 \varepsilon_s \)

* \( V_f = f_{rr} \cot \theta b_v d_v \)

* \( f_{rr} \) is the key parameter!

\( f_{rr} = \text{residual rupture stress, recommended} = 0.75 \text{ ksi} \)
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

UHPC Beam Shear Testing -- $V_{\text{test}}$ vs. $V_n$

- PCI-UHPC (2020)
- Graybeal (2005)
- Baby et al. (2010)
- Voo et al. (2006)
- Voo et al. (2010)
Tension Tie is Important. Two specimens with low anchorage gave relatively low shear capacity

\[ A_s f_y + A_{ps} f_{ps} \geq \left( \frac{V_u}{\phi_v} - 0.5 V_s - V_p \right) \cot \theta \quad \text{(AASHTO)} \]
Most Importantly! Demand is much lower than capacity

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

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

Proposed Model

\[ V_{ni} = c A_{cv} + \mu A_{vf} f_y \]

Fluted Joint Details as Specified by AFGC (2013)
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
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

NOTE: EMBEDDED COIL RODS TO HAVE A WATER PROOF GREASE APPLIED TO THE PORTION OF THE ROD THAT WILL BE EMBEDDED IN CONCRETE.

14" x 1" Ø GRADE 100 COIL ROD WITH A WELDED TOP NUT @ 24" SPA. (10 REQUIRED)

4" Ø CORRUGATED STEEL DUCT @ 24" SPA. TO BE GROUTED WITH UHPC AT THE SAME TIME AS THE POCKET AND THE HAUNCH BETWEEN THE GIRDER AND THE DECK. (10 DUCTS REQUIRED)
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 \( \ell/d_b \)
(20 of 35 test results)
Optimized Products developed in the PCI-UHPC Program
Decked I-Beams
**Comparison with Conventional Concrete,**

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

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

**Nebraska NU 1100 (43.31") with 8" conventional deck**
## Two Stage UHPC Cross Section

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

![Two-Stage UHPC Decked I-Beam based on NU1100](image)

Two-Stage UHPC Decked I-Beam based on NU1100

- Total depth: 51.31 in.
- Compressive strength at service: 18 ksi
- Compressive strength at release: 10 ksi
- Volume of beam: 12.00 CY
- Volume of deck: 13.7 CY
- Beam plus deck: 25.70 CY
- # of 0.7” Strands: 32
- Shear Reinforcement: No
- Deck Reinforcement: Transverse Only
## One-Stage Decked I Beam - Best Solution

<table>
<thead>
<tr>
<th>UHPC Decked-I-Beam</th>
<th>Percent reduction due to use of UHPC</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Total depth (in.)</strong></td>
<td><strong>51.31</strong></td>
</tr>
<tr>
<td>Compressive Strength at service, ksi</td>
<td>18</td>
</tr>
<tr>
<td>Compressive strength at release, ksi</td>
<td>10</td>
</tr>
<tr>
<td>Volume of beam, CY</td>
<td>23.85</td>
</tr>
<tr>
<td>Volume of deck, CY</td>
<td>1.35</td>
</tr>
<tr>
<td>Beam plus deck, CY</td>
<td>25.20</td>
</tr>
<tr>
<td># of 0.7” Strands</td>
<td><strong>24</strong></td>
</tr>
<tr>
<td>Shear Reinforcement</td>
<td>NO</td>
</tr>
<tr>
<td>Deck Reinforcement</td>
<td>Transverse Only</td>
</tr>
</tbody>
</table>

UHPC Decked I-Beam based on NU1100
U-Beams

46'-0"
11'-6"
11'-6"
6'-0"
6'-0"
11'-6"
11'-6"
7'-0"
5'-0"
1'-0"
1'-0"
1'-0"
3'-6"
1'-8"
1'-8"
1'-4"
4"
1'-2"
4'-0"
8'-6"
6'-0"
88-0.7" TEMPLATE 2" SPA.
Box Slabs

10 Spaces 4' = 36'
Optimization of Northeast Extreme Tee (NEXT)

Volume reduced from 43 to 23 cubic yards for a 90 ft long piece
Optimization of Square Piles
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