Orlando, FL

November 7-8, 2024

2024 TRANSPORTATION SYMPOSIUM

Structures Research Center Update

Christina Freeman & Olga Iatsko

FDOT Structures Research Center

FDOT Structures Lab

- Outdoor facility: a 2.5-acre yard for outdoor testing, storing specimens, Impact pendulum, and two semi-trucks with steel blocks for bridge testing.
- Indoor facility: 50’ x 100’ strong floor, 2 movable load frames, static (1000 kips)/dynamic (460 kips), 7 DAQ systems (2 with remote capabilities), 4 high-speed cameras for NDI.
- Staff: 10 full-time and 5 part-time contract employees.
- Projects: 3 in-house, 10 contracted, 2 bridge tests.

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https://www.fdot.gov/structures/StructuresResearchCenter/default.shtm

Ready to Use Topics

Project Title	**End Date**
Repair of Impact Damaged Utility Poles with FRP	6/30/2015
Large Reinforcing Bars Spliced In Ultra-High-Performance Concrete	03/14/2023
Shear Friction Capacity of Corrugated Pipe Connection in Precast Footings	06/28/2022
Aluminum Lightweight Orthotropic Deck Evaluation Project	2/17/2017
Inspection of Flexible Filler Tendons	12/14/2023

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

New and Emerging Topics

Project Title	**End Date**
Evaluation of GFRP Spirals in Corrosion Resistant Concrete Piles	07/21/2023
Evaluation of Concrete Pile to Footing or Cap Connections	5/31/2024
Bond Performance of Post-Tensioning Tendons with Corrosion Inhibitor	11/2024
Half-Round Bearing Stiffeners for Skewed Steel I-Girders	06/20/2024

Repair of Impact Damaged Utility Poles with FRP

This utility pole (left) was damaged in a collision, but it was successfully repaired (right).

Objectives: Damaged utility poles can be repaired with fiber-reinforced polymers (FRP) without removing or replacing the them.

The objective was to develop repair guidelines for economically and effectively restoring an impact-damaged utility pole.

Variables: FRP repair system, Impact energy, pole and dent geometry, Material of Spiral Reinforcement.

Methods: FEM, Pendulum Impact, Test, Four-point Flexure Test, Full-scale Cantilever Flexure Test, Cyclic Load Fatigue Test

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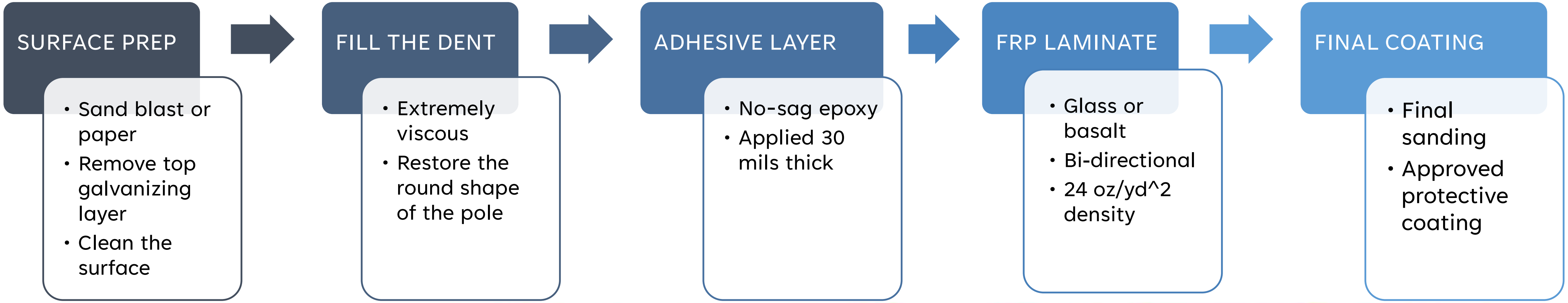
Repair of Impact Damaged Utility Poles with FRP

FIBER REINFORCED POLYMER (FRP) REPAIR SYSTEM



SURFACE PREP

- Sand blast or paper
- Remove top galvanizing layer
- Clean the surface

FILL THE DENT

- Extremely viscous
- Restore the round shape of the pole

ADHESIVE LAYER

- No-sag epoxy
- Applied 30 mils thick

FRP LAMINATE

- Glass or basalt
- Bi-directional
- 24 oz/yd^2 density

FINAL COATING

- Final sanding
- Approved protective coating

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Repair of Impact Damaged Utility Poles with FRP

PENDULUM IMPACT TEST

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Repair of Impact Damaged Utility Poles with FRP

CONCLUSIONS:

1. The results of this study indicate that the FRP composite repair systems considered in this study were effective in restoring both field-damaged and laboratory-damaged tapered utility poles to acceptable capacities.
2. Dented poles (with dent of less than 30% diameter) tested in a four-point bending setup could be returned to the original plastic capacity many times with a single layer of FRP.
3. All the damaged and repaired large-scale specimens achieved more than 90% of the estimated undamaged yield capacity.
4. The design of the repair was complicated by the location and extents of the dent, potentially with the cross-sectional geometry of the pole being different than the original tapered geometry (at the center of the dent location usually).
5. Majority of poles considered contained an integral access port (hand hole) or series of pedestrian buttons that required placement of a vertical layup of the laminates.

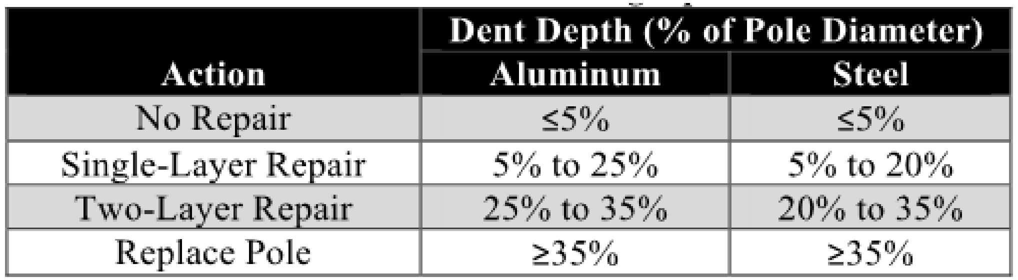
Applicability of Repair to Impact Damaged Utility Poles with FRP

INITIAL INSPECTION:

1. Perform a close investigation for cracks due to the impact.
2. Brittle cracks can lead to premature failure even with FRP repair.

MEASURE GEOMETRY:

1. Geometry of FRP wrap depends on pole features.
2. Access ports and crosswalk signal buttons in should remain accessible.
3. Repair should extend at least 6 in beyond the edges of the dent and should encompass the circumference of the pole.



	Dent Depth (% of Pole Diameter)	
Action	**Aluminum**	**Steel**
No Repair	≤5%	≤5%
Single-Layer Repair	5% to 25%	5% to 20%
Two-Layer Repair	25% to 35%	20% to 35%
Replace Pole	≥35%	≥35%

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Large Reinforcing Bars Spliced In Ultra-High-Performance Concrete

Objectives: Although UHPC has been researched extensively, previous research for reinforcing bar splice and development lengths have focused on #9 and smaller diameter bars. Typically, larger diameter bars are used for substructures.

The objective was to determine the reinforcing bar splice and development length for rebar diameters larger than #8.

Potential Applications: Prefabricated bridge substructure elements can be used to accelerate construction. UHPC is an ideal material for joining precast components, such as:

- Drilled Shaft to Precast Bent Cap Connection
- Footing to Precast Column Connection
- Precast Beam Connections

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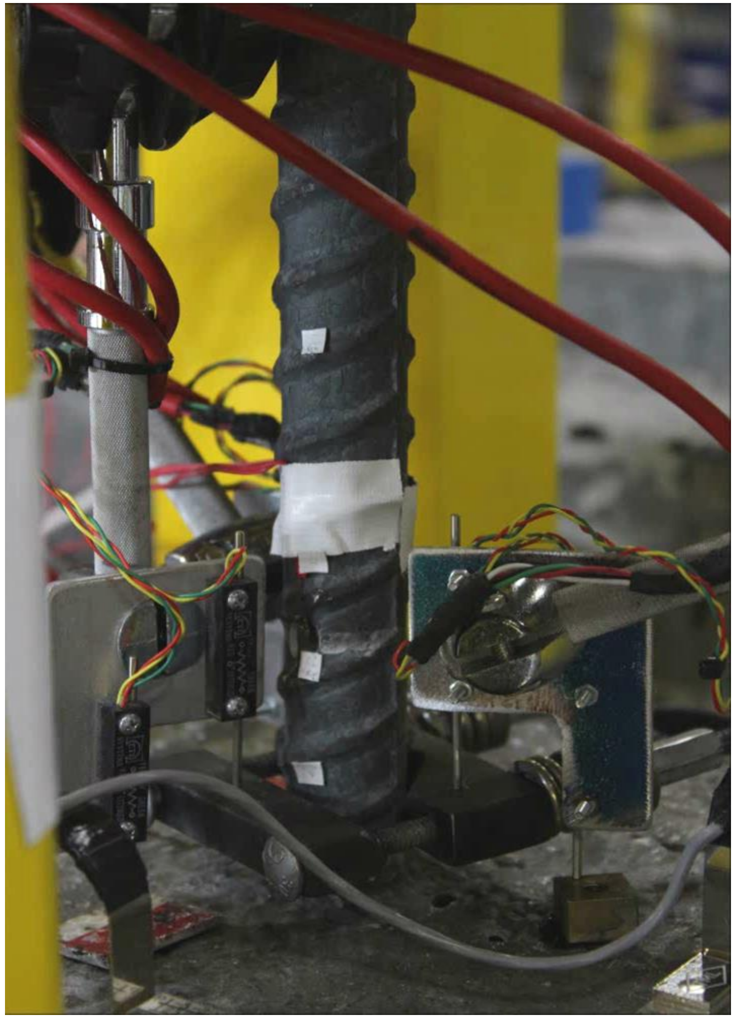
Large Reinforcing Bars Spliced In Ultra-High-Performance Concrete



MATRIX OF PARAMETERS:

- The required splice length was determined for steel deformed reinforcing bars embedded in UHPC, considering three primary variables:
  - Bar size (#8, #9, #10 and #11 bars)
  - Bar spacing (contact, 6 in. (152.4 mm), and 8.5 in. (215.9 mm)), and
  - Concrete cover (1.75 in. (44.5 mm), 2.75 in. (69.9 mm), and 3.75 in. (95.3 mm))
- 128 Individual reinforcing bar tests were completed.
- A single readily available propriety UHPC mix with 2% steel fiber by volume was used.

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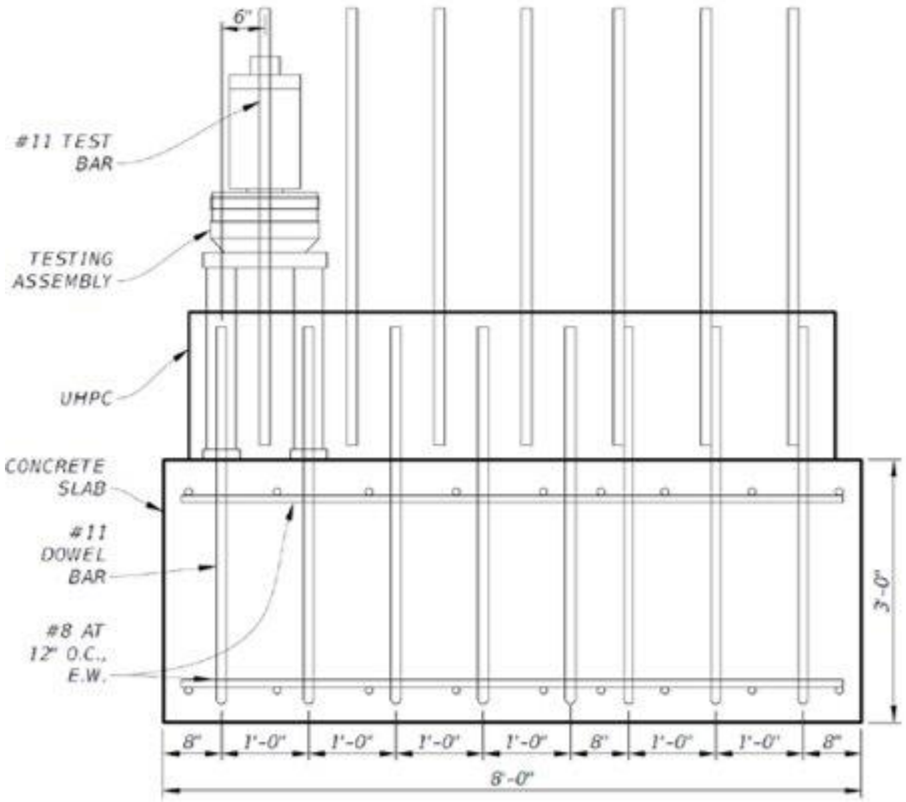
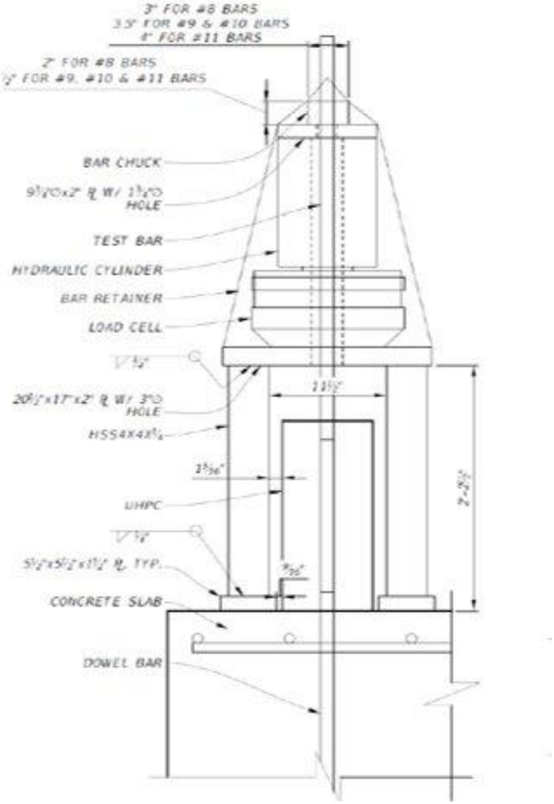
Large Reinforcing Bars Spliced In Ultra-High-Performance Concrete

TEST SETUP





- The testing equipment, shown in consisted of a hollow core load cell and hollow core cylinder in sequence.
- Splice was placed in direct tension, without any confining compressive stresses.

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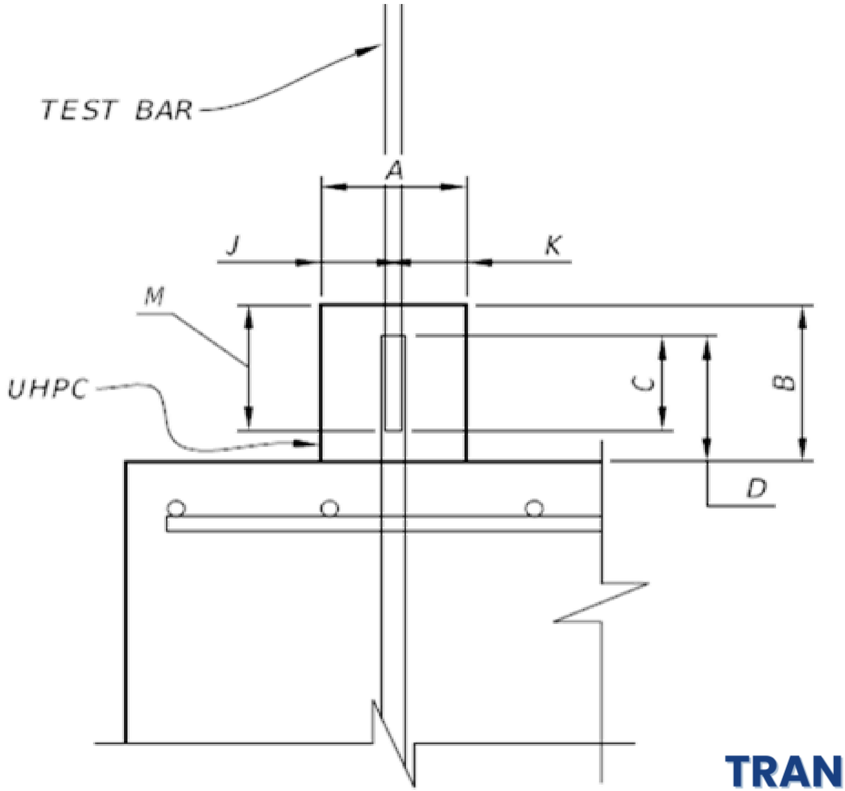
Large Reinforcing Bars Spliced In Ultra-High-Performance Concrete

CONCLUSIONS:

Additional splice or embedment length needs to be specified by designers to account for reinforcing bar length and placement construction tolerances.

Required Embedment Length (M, D) in Terms of Bar Diameters					
		Bar Size			
		#8	#9	#10	#11
Cover	1.75 in. (44.5 mm)	8	9.8	11.7	12.9
	2.75 in. (69.9 mm)	-	-	-	11.3
	3.75 in. (95.3 mm)	8	6.9	8.4	9.3

Required Splice Length (C) in Terms of Bar Diameters					
		Bar Size			
		#8	#9	#10	#11
Cover	1.75 in. (44.5 mm)	6	7.3	9.7	11.1
	2.75 in. (69.9 mm)	-	-	-	9.7
	3.75 in. (95.3 mm)	6	5	6.6	7.3



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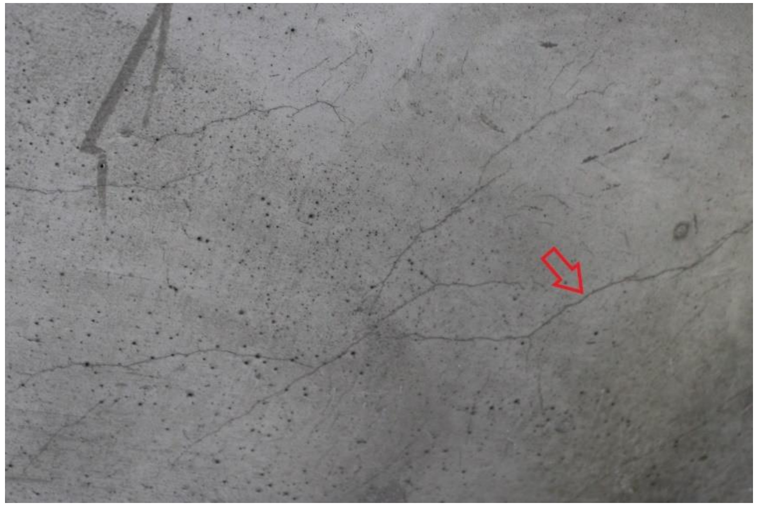
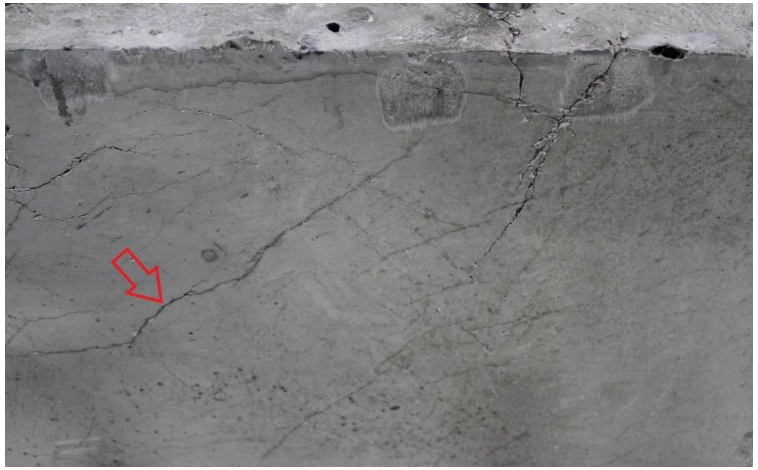
Large Reinforcing Bars Spliced In Ultra-High-Performance Concrete





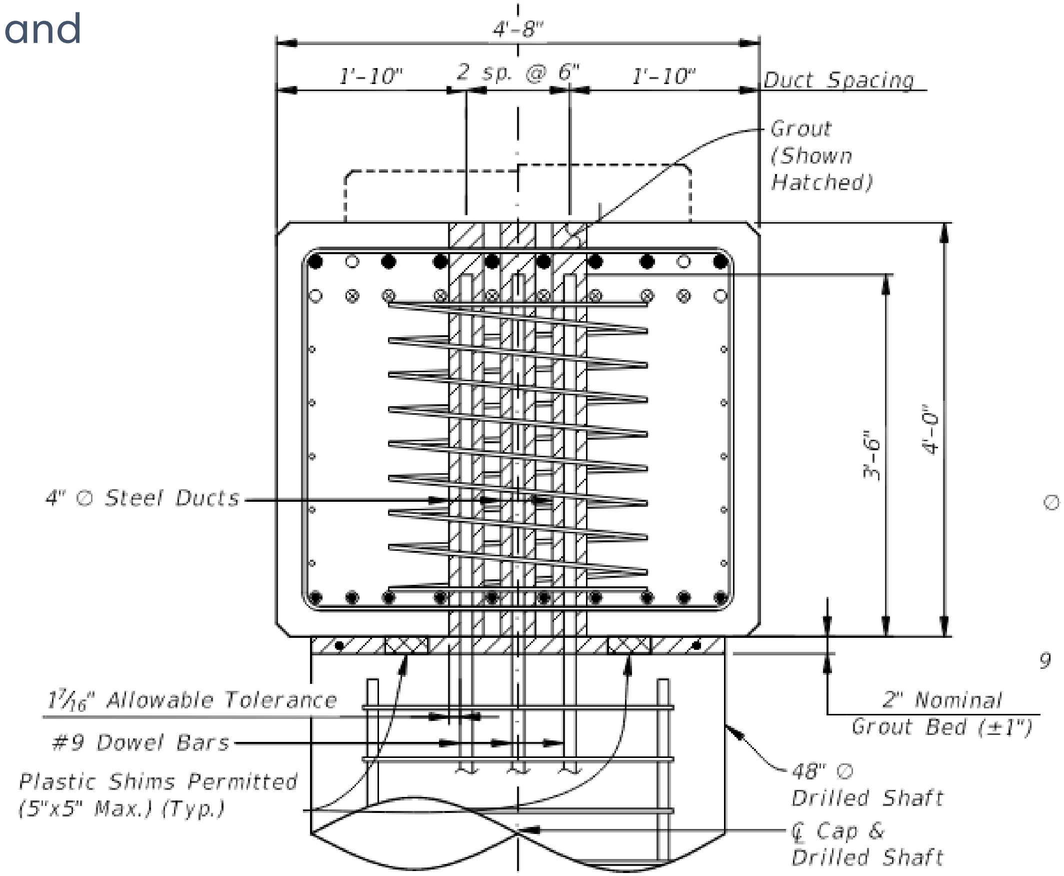
OBSERVATIONS:

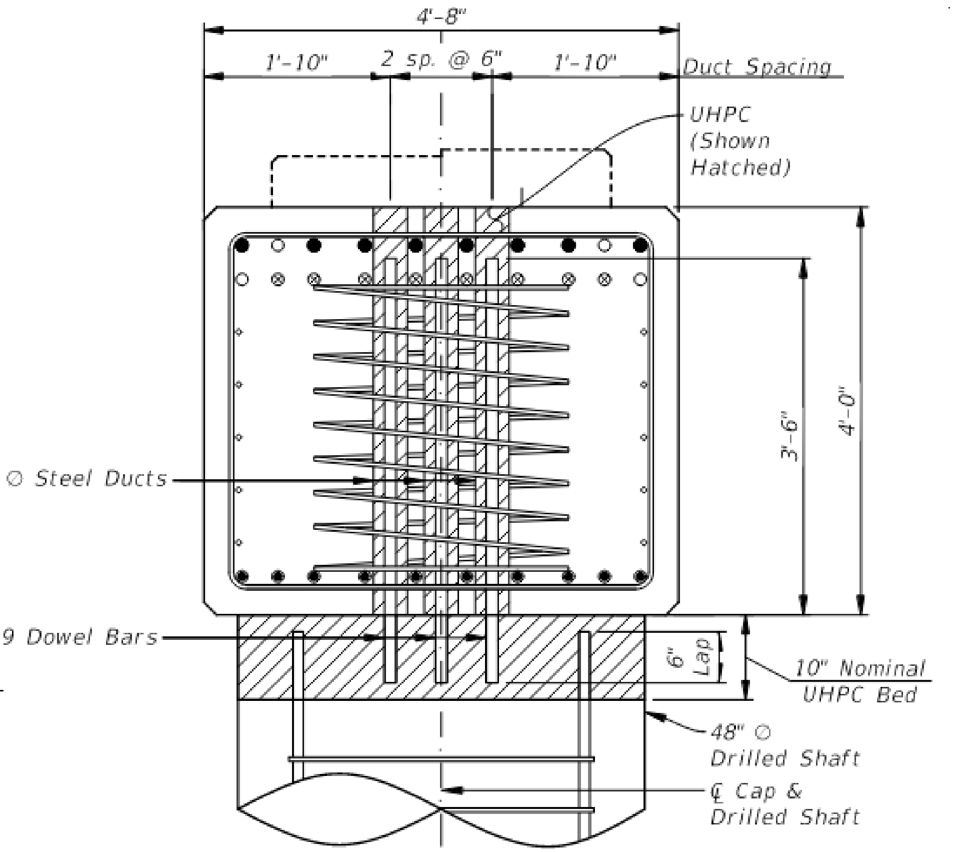
For one set of the tests, shrinkage cracking was apparent in the UHPC prior to testing and the results of those tests showed lower than expected bond strength. However, the results of tensile material testing did not indicate reduced capacity. Tensile sample testing is not sufficient to evaluate UHPC with shrinkage cracks.

Large Reinforcing Bars Spliced In Ultra-High-Performance Concrete

Drilled Shaft to Precast Bent Cap Connection

- Increase horizontal and vertical tolerances.





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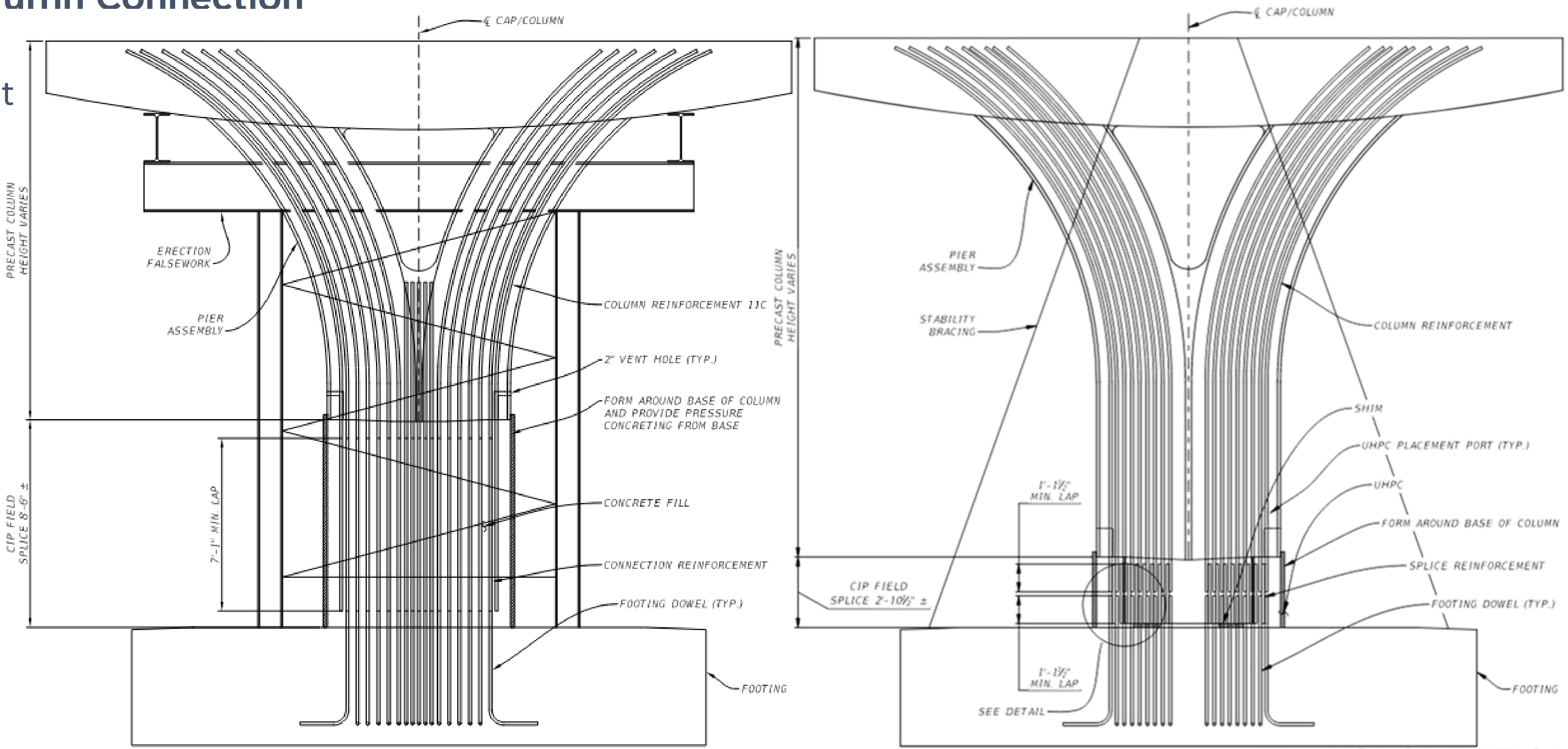
Large Reinforcing Bars Spliced In Ultra-High-Performance Concrete

Footing to Precast Column Connection

- Decrease splice height and volume of field-placed material.
- Possibly eliminate erection falsework.



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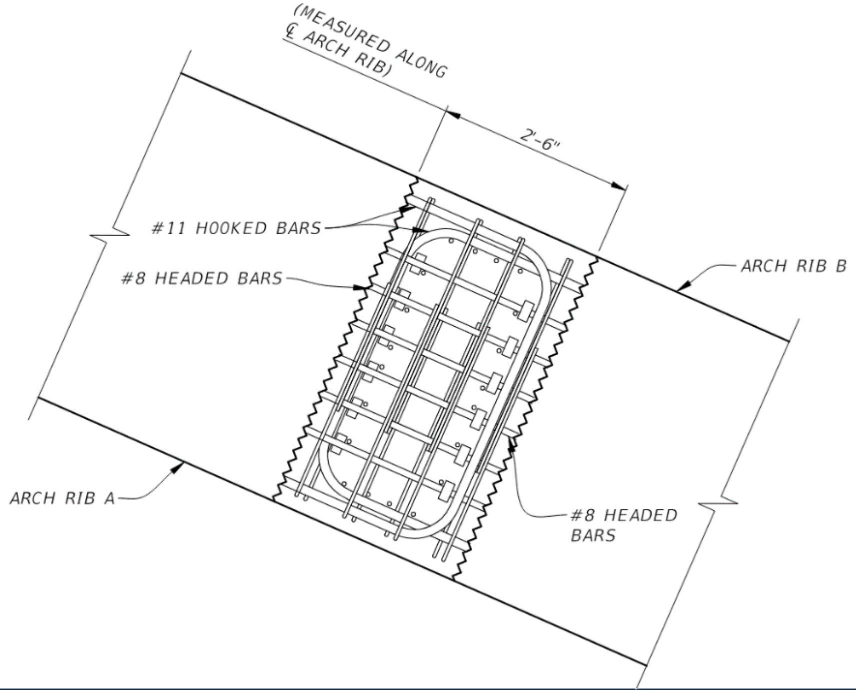
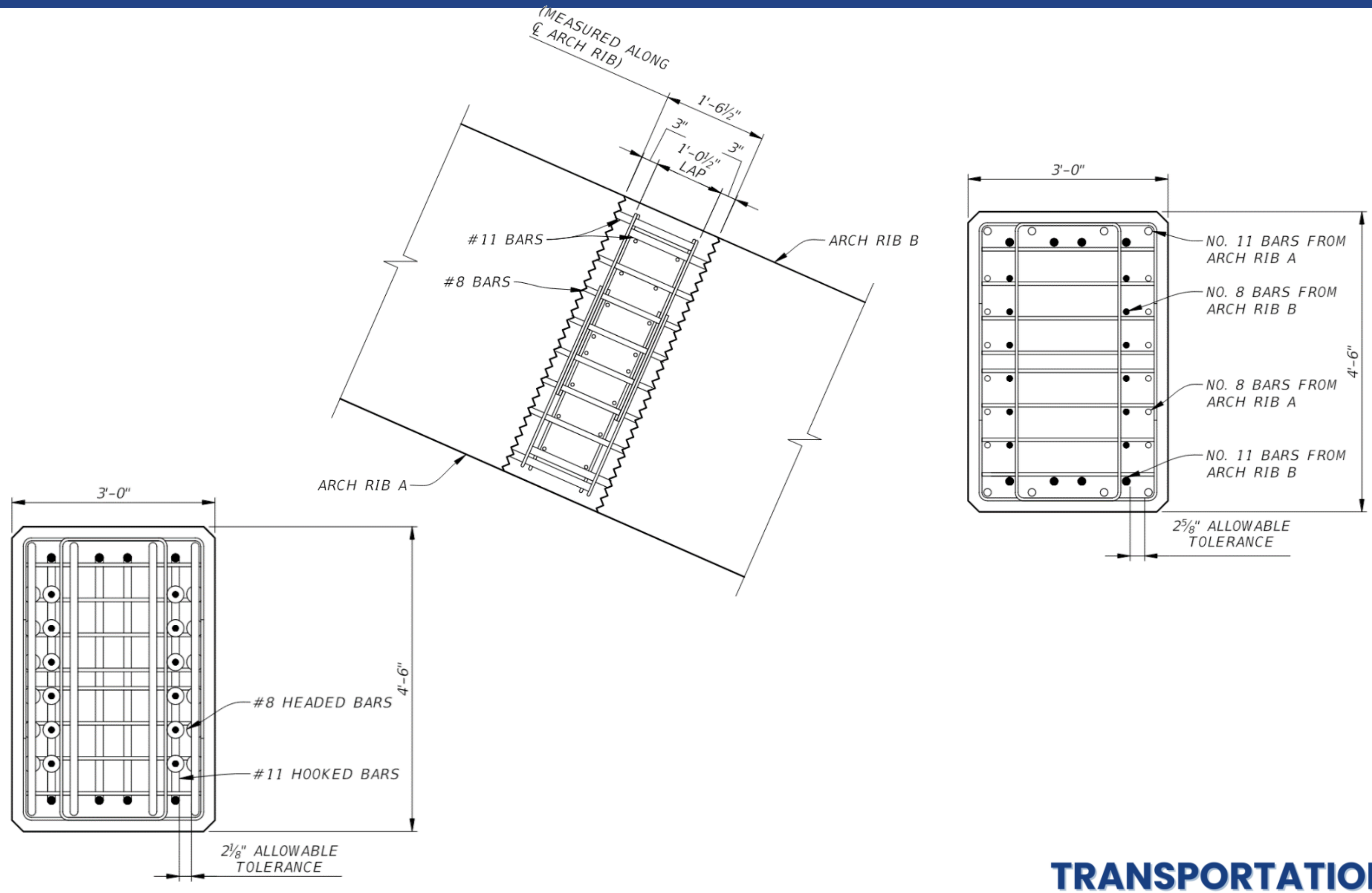
Large Reinforcing Bars Spliced In Ultra-High-Performance Concrete

Precast Beam Connections

- Decrease closure pour size.
- Eliminate hooked and headed bars.
- Increase tolerances.





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Shear Friction Capacity of Corrugated Pipe Connection in Precast Footings

Florida International University researchers used these specimens to test shear friction capacity for precast pile caps and precast piles that do not have steel crossing the interface.

Objectives: FDOT developed a concept for a “pocket connection” between precast pile caps and precast piles. The “pocket” is a void made in the precast pile cap that the precast pile element fits into. The void is filled with concrete to create the connection.

The goal of this project was to evaluate the strength of the precast pocket connection without steel crossing the interface.

Variables: Interface surface roughness and preparation, reinforcement crossing the interface, Applied normal force, Concrete strength, and Concrete curing conditions.

Methods: Push-through test

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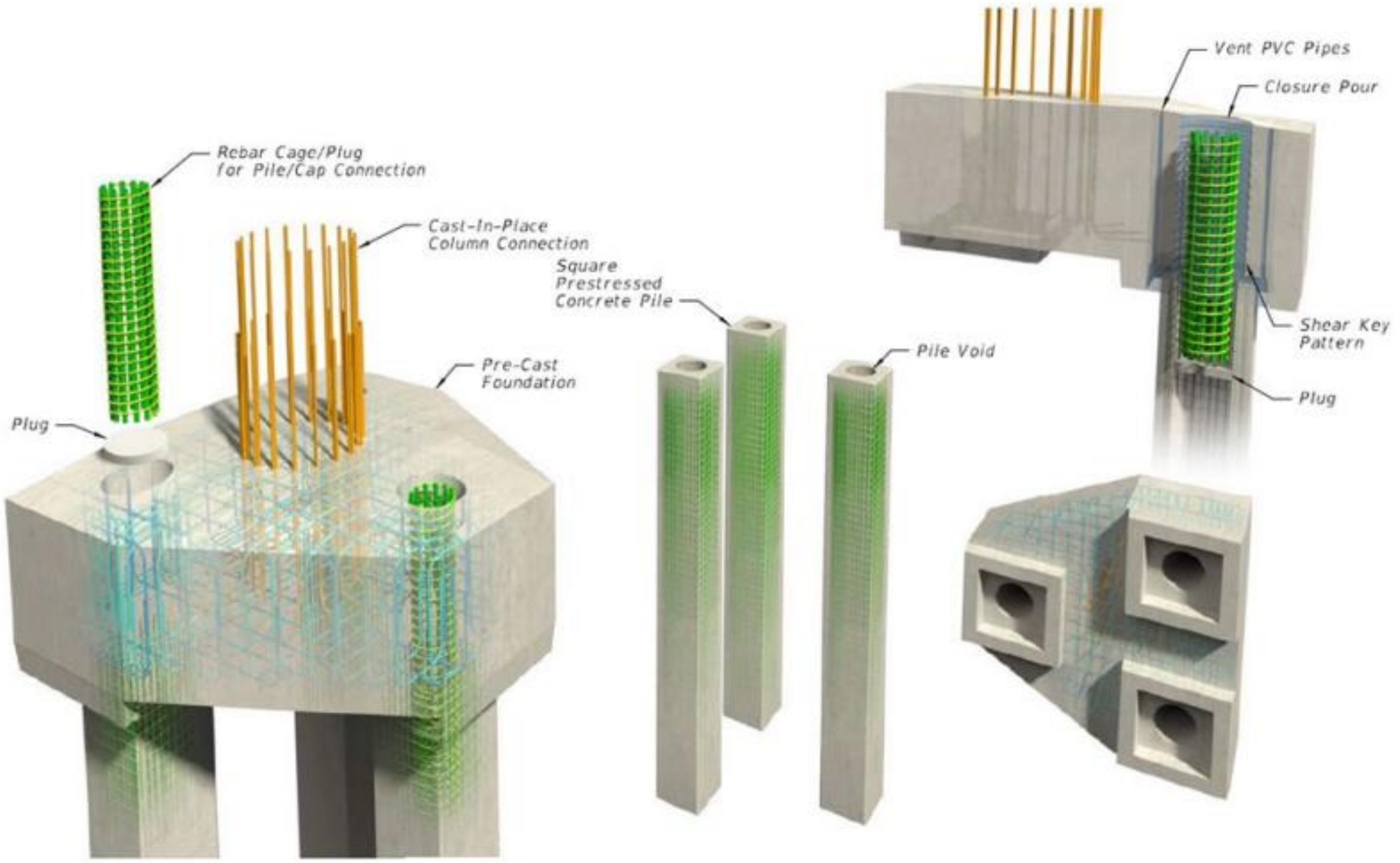
Shear Friction Capacity of Corrugated Pipe Connection in Precast Footings

FDOT CURRENTLY RECOMMENDED CONNECTION DETAILS BETWEEN PRECAST PILES AND PILE CAPS



Use a removable corrugated pipe in the plug-cap interface. (SDM 25.4.3.7)

The surface shall be presoaked and prepared to obtain a “saturated surface dry” (SSD) condition. (SDM 25.4.3.7)

Provide an exposed aggregate finish surface at all interfacing surfaces. This finish is specified as a 1/4-inch amplitude finish. (SDG 1.15)

Include shrinkage-reducing admixture into filling concrete and provide a seven-day moist cure. (SDM 25.4.3.7)

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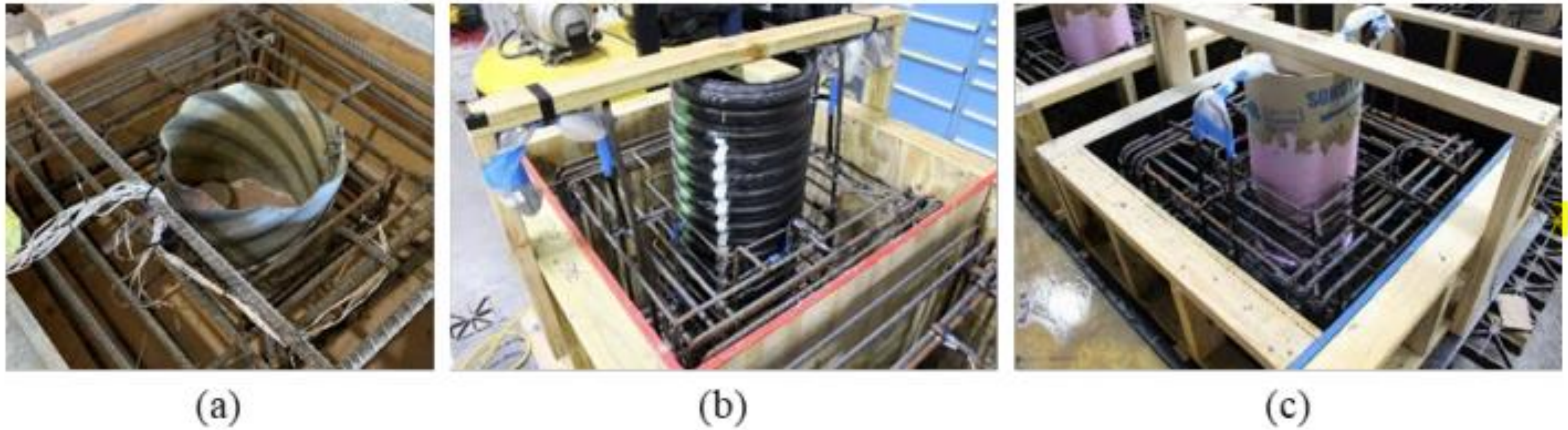
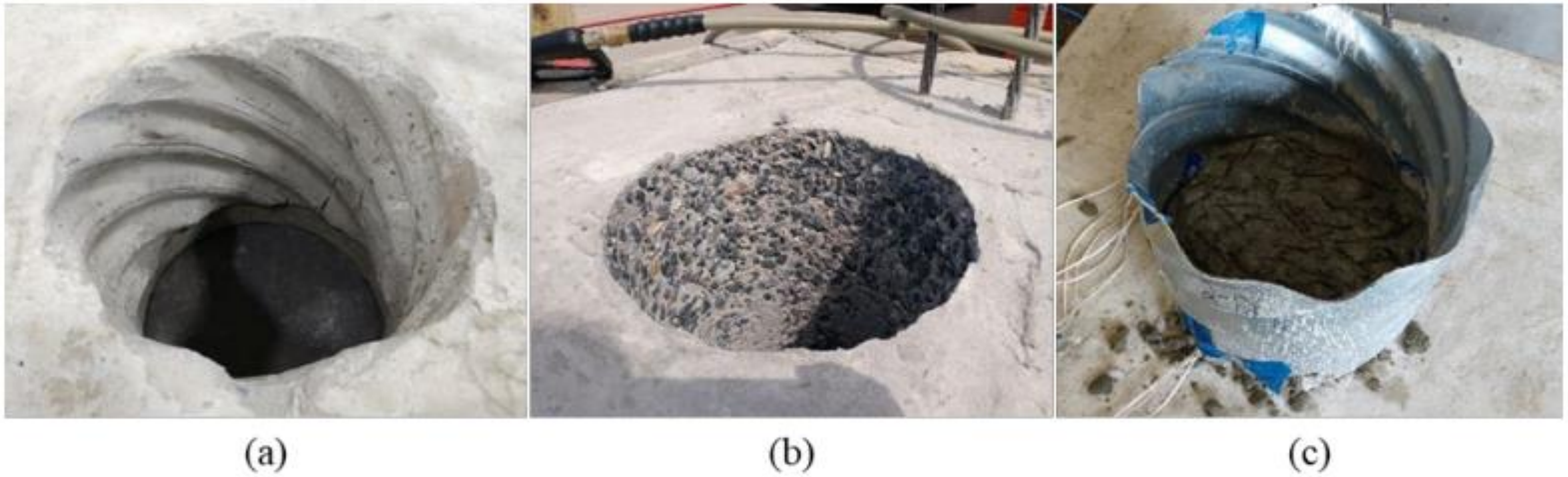
Shear Friction Capacity of Corrugated Pipe Connection in Precast Footings

SPECIMEN FABRICATION


(a) (b) (c)

Construction of cap and installation of blockout for void with:
- (a) corrugated metal pipe,
- (b) corrugated plastic pipe,
- (c) Sonotube with paste retarder


(a) (b) (c)

Surface preparation for second series of specimens:
(a) sandblasted,
(b) paste retarder,
(c) corrugated metal duct

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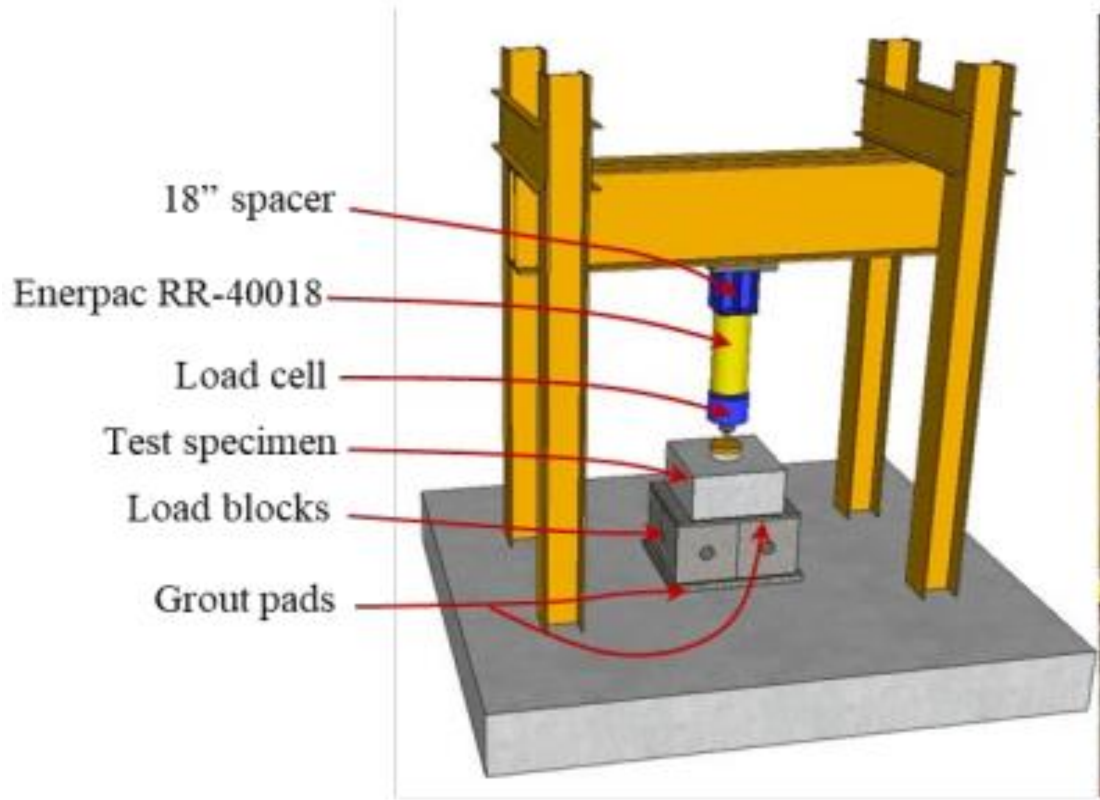
Shear Friction Capacity of Corrugated Pipe Connection in Precast Footings

TEST SETUP



- The load was applied to the specimens using a 750-kip hydraulic jack and a 600-kip load cell attached to a load frame
- The specimens were placed on top of four load blocks
- The load was applied at a rate of 0.2 kips per second until a load of 200 kips for all specimens
- The 200-kip load was held on the specimens at this point while the specimens were inspected for cracks
- Load was then applied at the same load rate (0.2 kips per second) until failure of the interface or test capacity was reached.

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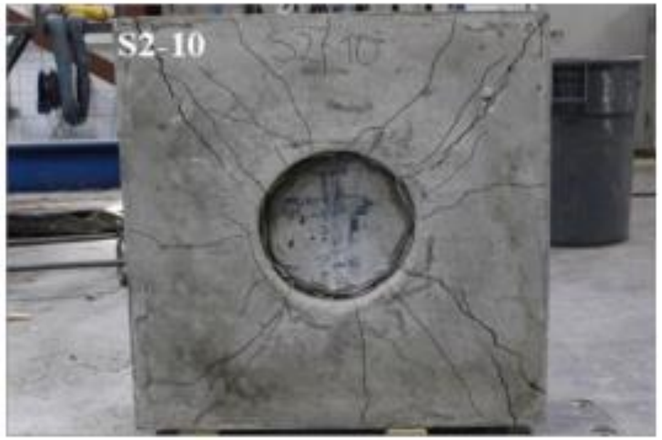
Shear Friction Capacity of Corrugated Pipe Connection in Precast Footings

CONCLUSIONS:

- All specimens with the 12-inch diameter plug failed due to a shear friction failure at the interface between the plug and cap
- Specimens with an exposed aggregate finish with 1/4-inch surface roughness had the highest normalized strength among all specimens tested
- Specimens with a smooth interface and 1/16-inch surface roughness are sensitive to the casting procedure
- The corrugated metal pipe provided only minor cohesion between the plug concrete and metal pipe and failed at lower loads than specimens with the corrugated interface with a 1/16-inch surface roughness concrete finish.
- The corrugation size and spacing affect the strength of the interface. Providing single or double ribs at the base of the pocket increased the normalized strength of specimens compared to those with a smooth interface
- Edge distance had a noticeable effect on the normalized strength only when the edge distance was decreased in two directions.



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Aluminum Lightweight Orthotropic Deck Evaluation Project

Objectives: Identify/Develop a Viable Lightweight Deck System with Solid Surface to Replace Steel Open Grid Deck on Typical Florida Bascule Bridges.

Methods: Visual and NDE inspection, Full-scale static and cyclic testing, Heavy vehicle simulation, Wearing surface testing

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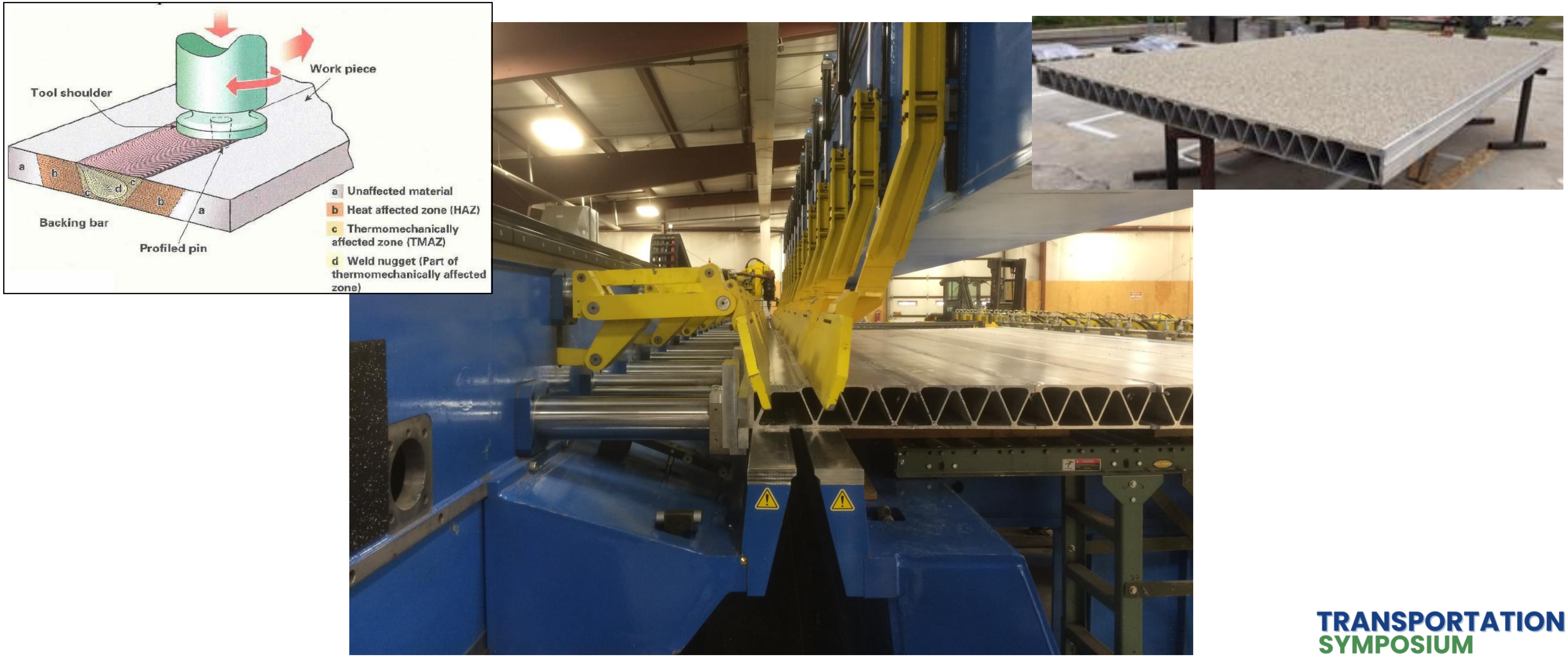
Aluminum Lightweight Orthotropic Deck Evaluation Project



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Aluminum Lightweight Orthotropic Deck Evaluation Project

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Aluminum Lightweight Orthotropic Deck Evaluation Project

TEST SETUP:

- Nine static tests, one cyclic test, and heavy vehicle simulation
- Varied loading points and support conditions. The support conditions for the steel stringers varied.
- Loads were applied corresponding to the factored Service II, Strength I and Strength II limit states.

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Aluminum Lightweight Orthotropic Deck Evaluation Project

CONCLUSIONS:

- Structurally, the test specimen performed well. The maximum demand-to-capacity ratio for strain measurements was 0.7.
- Deflection measurements were very close to the design limit, exceeding the Span/800 limit by less than 1/32”.
- Fatigue is a potential design concern for the panel, as the stress range is predicted to be approximately equal to the constant amplitude fatigue threshold.
- The wearing surface proved to be an effective friction surface after rigorous testing. It is expected to perform well in-service, although frequent inspections during the trial period are appropriate since this deck system is a new technology.
- One key difference between the aluminum lightweight deck and open grid steel deck systems is the aluminum lightweight deck provides a solid driving surface, which is preferred.

Aluminum Lightweight Orthotropic Deck Evaluation Project

FIELD EVALUATION

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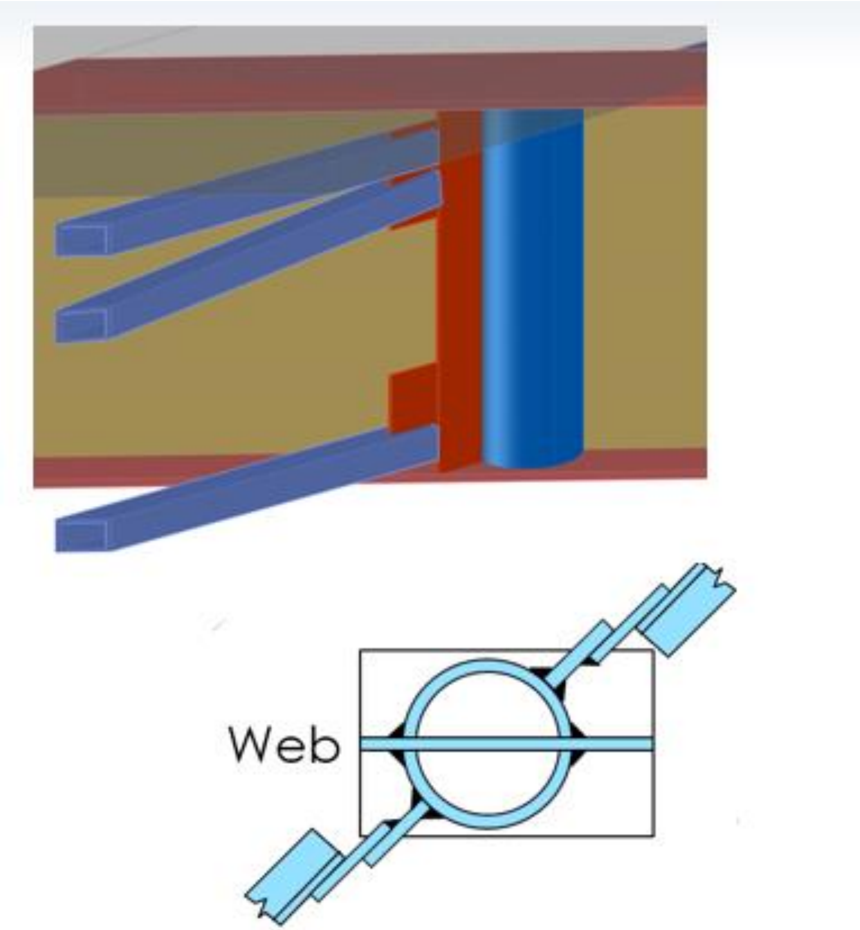
Half-Round Bearing Stiffeners for Skewed Steel I-Girders



Objectives: Determine the fatigue sensitivity of the half-round bearing stiffener connection over the intermediate support in continuous skewed steel girder bridges.

Skewed bridge design requires extra care because the structural behavior of skewed bridges can differ significantly from non-skewed bridges

Support skew has the potential to cause additional effects, including:

- Additional structural effects/responses from lack-of-fit. Fit-up forces need to be accounted for design and detailing of connections.
- Amplifying live load effects and fatigue impact because of support skew and differential deflection.

Two bridges have already been built in Florida with this detail and are in service.

Methods: 2D Grid Analysis, 3D FEM Analysis

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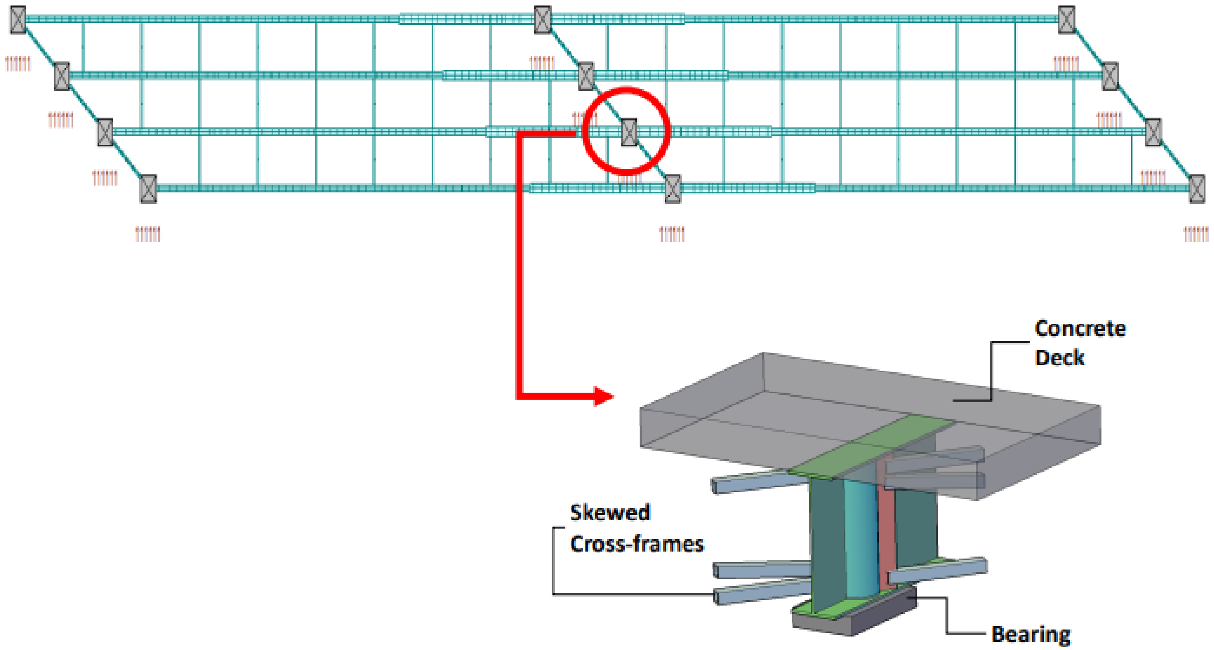
Half-Round Bearing Stiffeners for Skewed Steel I-Girders

ANALYTICAL STUDY:



26 bridges representative of FDOT skewed continuous steel girder bridges were analyzed.

Midas Civil and Midas FEA NX programs were utilized to create 2-D and 3-D finite element (FE) models.

Tensile stress variation in the girder top flange over the intermediate pier for fatigue test planning, and end cross-frame member forces for:

- Sizing and detailing of the HRBS connection
- Stress concentration considerations for the connection
- Fatigue categorization

Half-Round Bearing Stiffeners for Skewed Steel I-Girders

CONCLUSIONS:

Refined analysis models considered the following geometry:

- The diameter of the HRBS was selected so that the flange extends at least 2-in. beyond the HRBS. A variation of thickness for the HRBS ranging from 3/8 to 3/4 in. was used to determine the appropriate thickness.
- Clipped and non-clipped conditions were considered for the HRBS. In addition, two stiffener connection plate details were modelled consisting of both welded and non-welded conditions with the girder flanges where applicable.

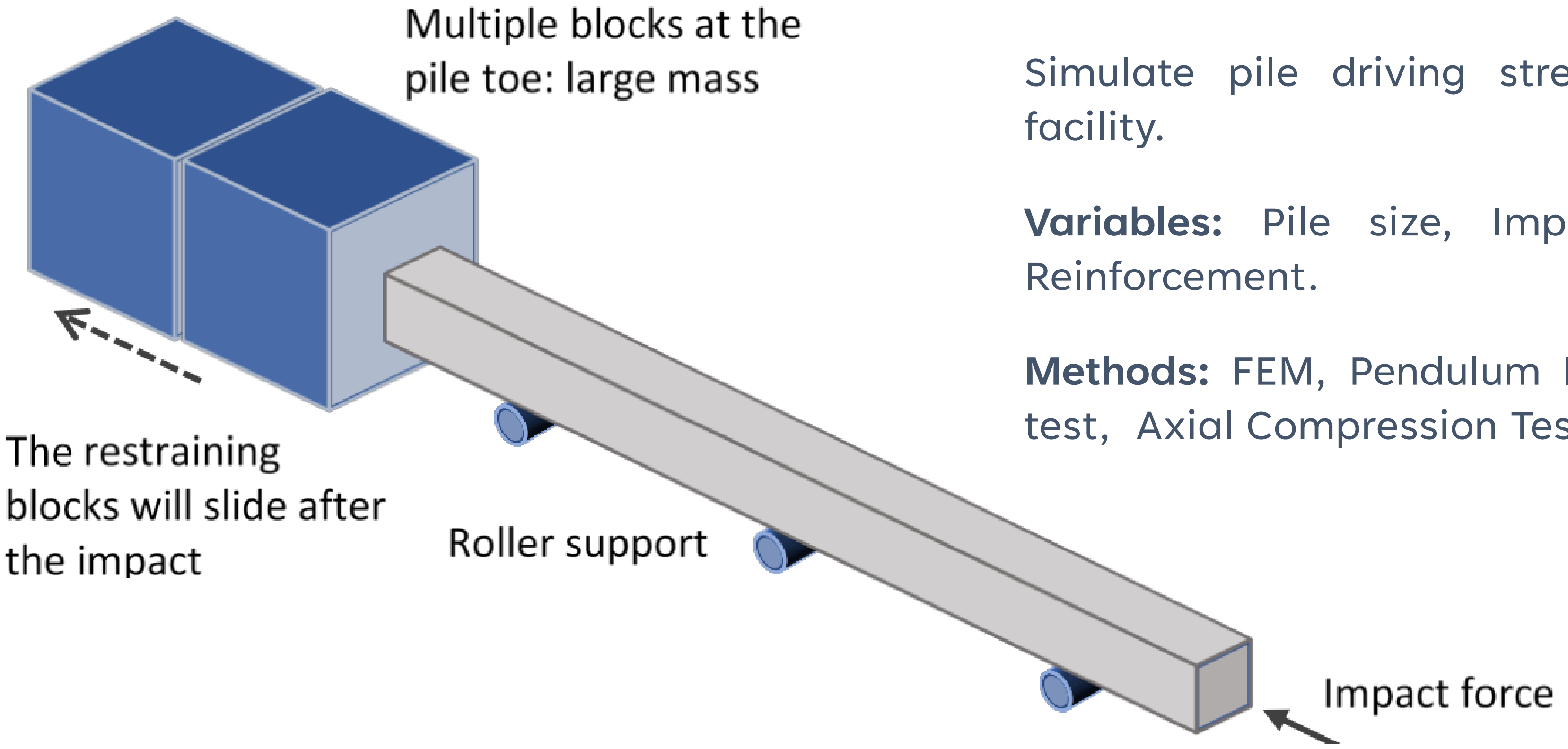
For the bridges studied, the top flange fatigue stress range from Fatigue I factored loading showed a range between 1.24 to 3.57 ksi with an average of 2.34 ksi.

A category C’ fatigue stress range is appropriate, based on previously completed physical testing.

It is anticipated that HRBS would be allowed on any steel I-girder bridge that has a skew complying with the limits in the FDOT Structures Design Guidelines.

Evaluation of GFRP Spirals in Corrosion Resistant Concrete Piles

The concept



Objectives: This project investigated replacing the CFRP spirals with GFRP (Glass Reinforced Polymers (GFRP)) spirals

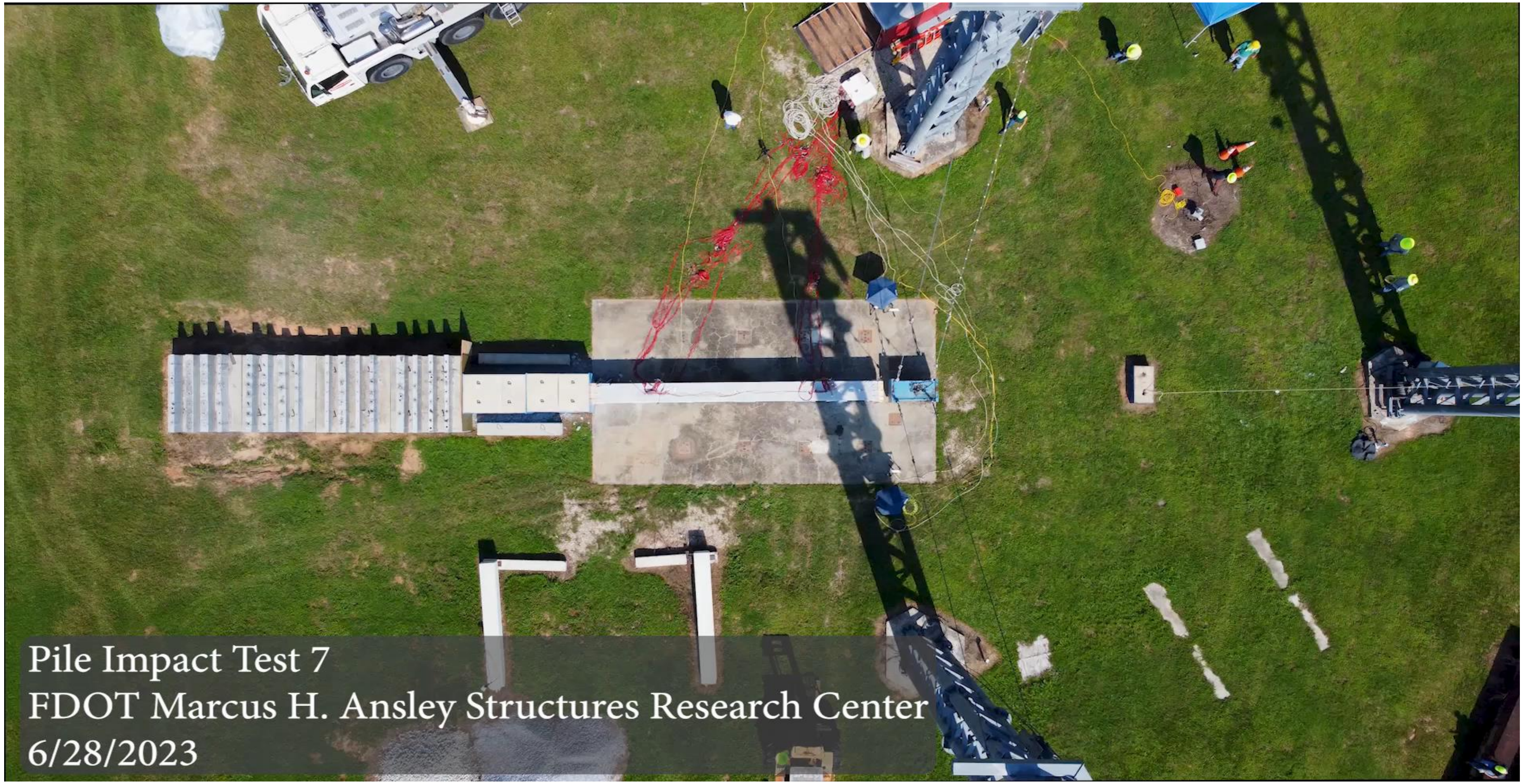
Simulate pile driving stresses with the FDOT pendulum facility.

Variables: Pile size, Impact energy, Material of Spiral Reinforcement.

Methods: FEM, Pendulum Impact, Test, Four-point Flexure test, Axial Compression Test.

Evaluation of GFRP Spirals in Corrosion Resistant Concrete Piles

The SRC Pendulum facility was utilized to apply impact loading comparable to pile driving


Pile Impact Test 7
FDOT Marcus H. Ansley Structures Research Center
6/28/2023

- Horizontal setup
- 24” x 24” pile
- 5 ksi stress
- Flectionless supports
- Large blocks at the pile toe

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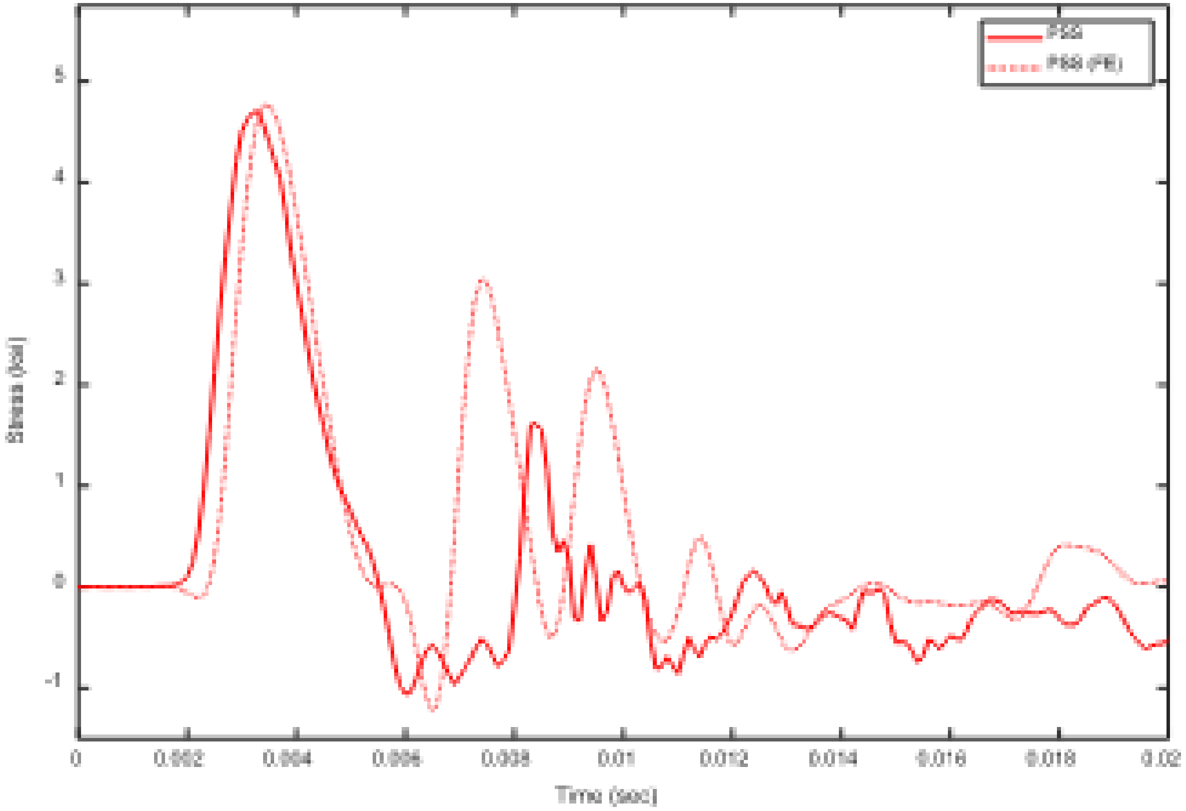
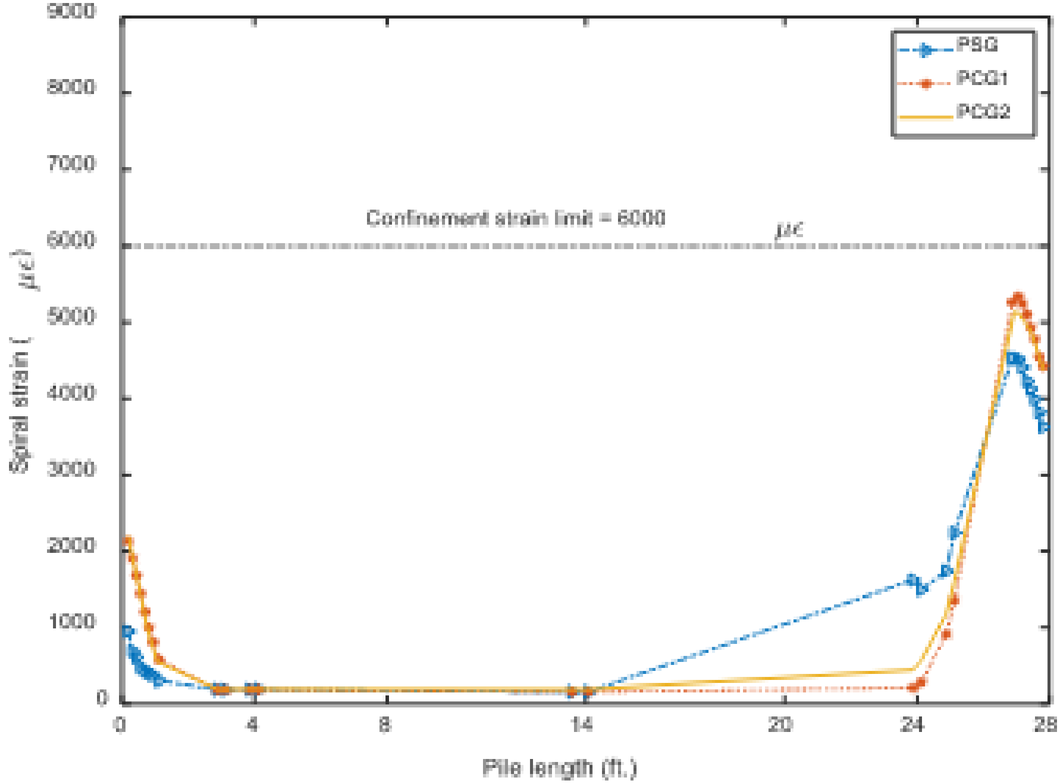
Evaluation of GFRP Spirals in Corrosion Resistant Concrete Piles

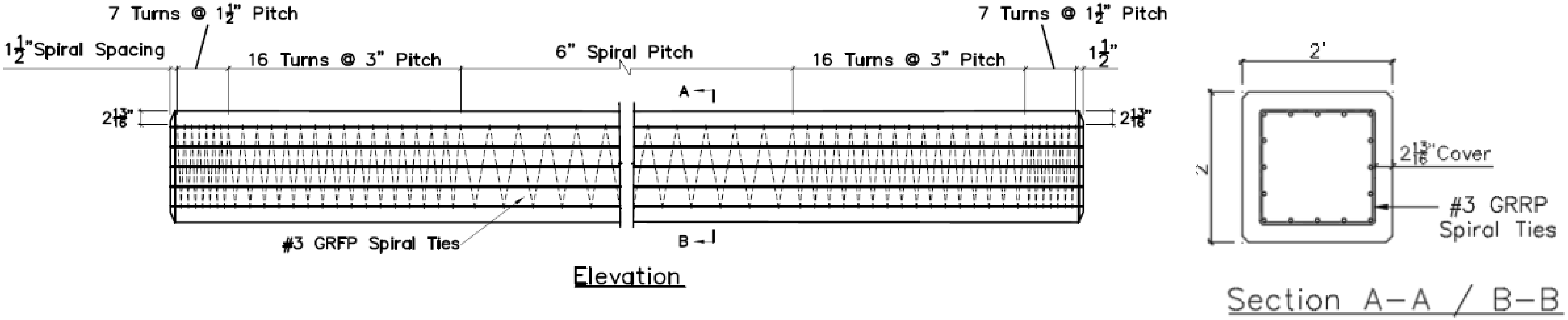


FE VS Test stress results PSS (15 ft.)



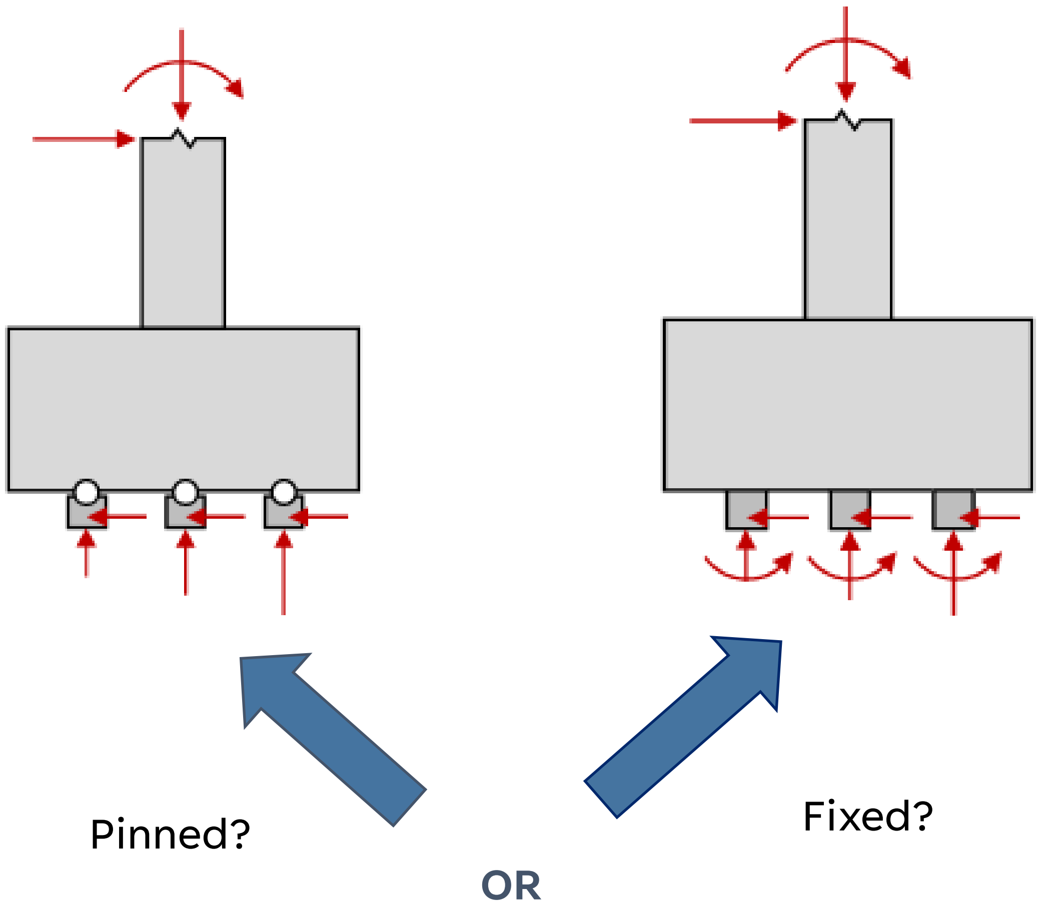
FRP spiral strain comparison for PSG, PCG1 and PCG2 (20 ft. drop heig

Evaluation of GFRP Spirals in Corrosion Resistant Concrete Piles



Summary: PCG1 (7 turns) and PCG2 (11 turns) showed similar compression and tension stress measurements. Therefore, the extra end-spiral turns for PSG2 provided no significant advantage under the reported test conditions. Consequently, the spiral pattern for PCG1 proved to be sufficient and recommended for standard pile design.

Evaluation of Concrete Pile to Footing or Cap Connections



Objective: To better understand the pile to cap or footing connection allowing FDOT to provide better design guidance along with more informed design reviews.

Background: FDOT Standard Design Guide (SDG 3.5.1.C) requires a pile embedment of 48” into a reinforced concrete footing for adequate development of the full bending capacity of the pile. 12” embedment is considered for pinned head condition (SDG 3.5.1.B).

Variables: Pile size, embedment depth, axial load, and interface reinforcement.

Methods: FEM, Full-scale Lab Test

Instrumentation: Load cells at hydraulic jack, LVDTs, Surface Strain Transducers, Vibrating wire gauge, and Digital Image Correlation (DIC).

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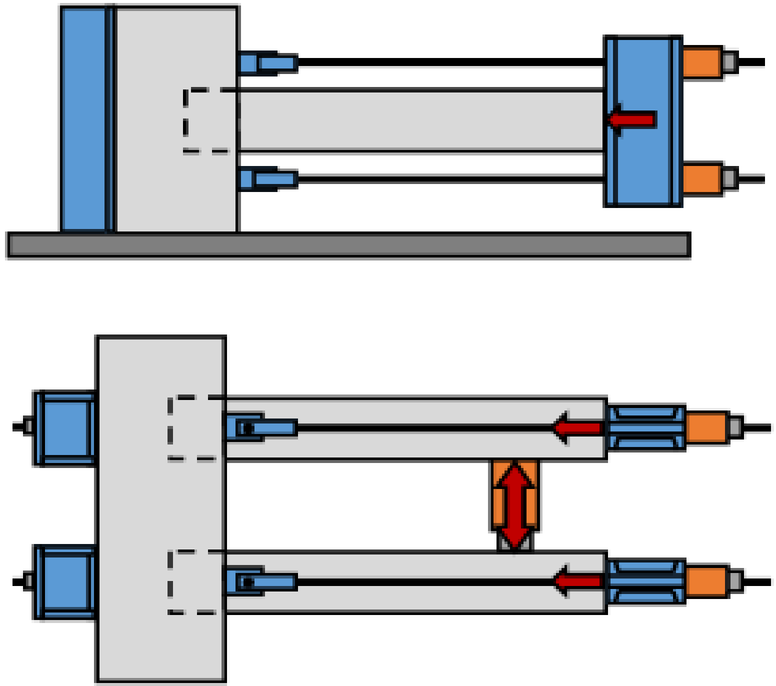
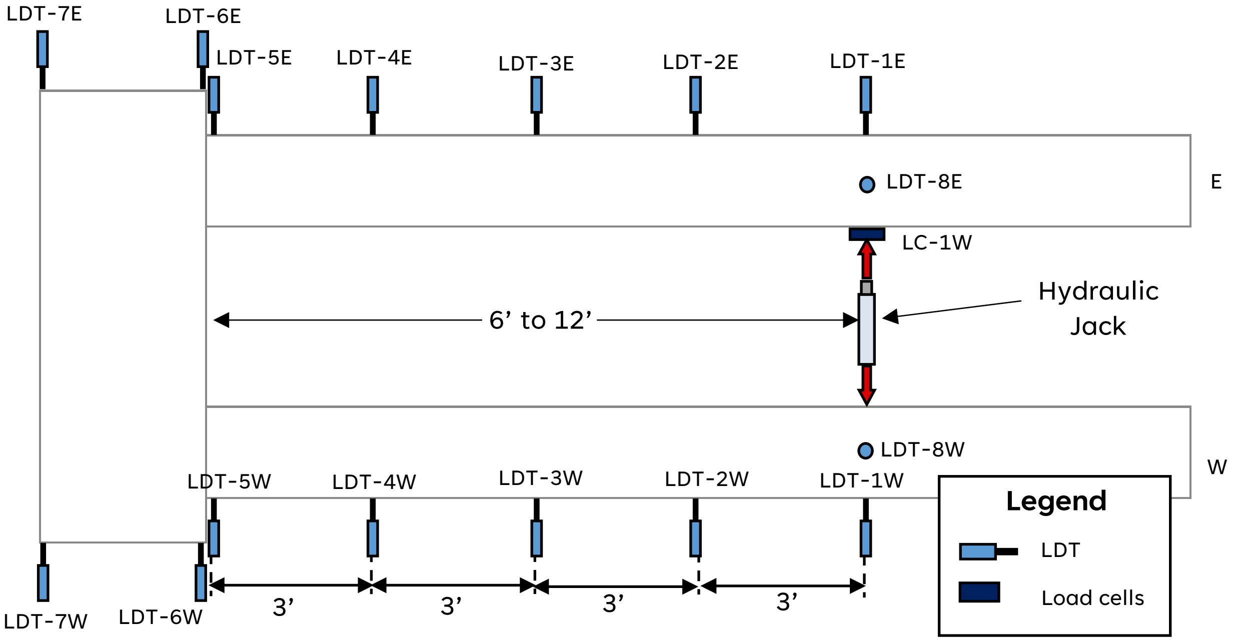
Evaluation of Concrete Pile to Footing or Cap Connections

SELF-REACTING FRAME TEST SETUP



INSTRUMENTATION PLAN



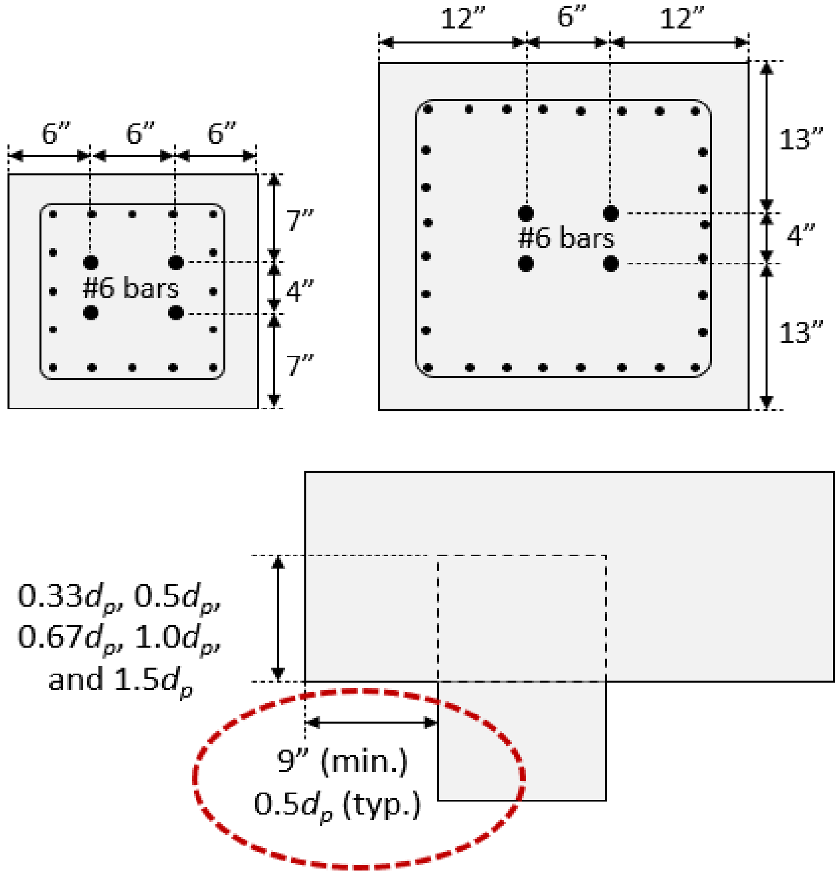
Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Evaluation of Concrete Pile to Footing or Cap Connections

TEST MATRIX

Specimen No.	Pile Size	Embedment Length		Interface Reinforcement	Axial Load	Pile Cap f'_c
1	18”	$0.33d_{pile}$	6”	w/o interface reinforcement	$0A_gf'_c$	Class IV
2	18”	$0.33d_{pile}$	6”	w/o interface reinforcement	$0.1A_gf'_c$	Class IV
3	18”	$0.33d_{pile}$	6”	w/interface reinforcement	$0A_gf'_c$	Class IV
4	18”	$0.5d_{pile}$	9”	w/o interface reinforcement	$0A_gf'_c$	Class IV
5	18”	$0.5d_{pile}$	9”	w/o interface reinforcement	$0.1A_gf'_c$	Class IV
6	18”	$0.67d_{pile}$	12”	w/o interface reinforcement	$0A_gf'_c$	Class IV
7	18”	$1.0d_{pile}$	18”	w/o interface reinforcement	$0A_gf'_c$	Class IV
8	18”	$1.5d_{pile}$	27”	w/o interface reinforcement	$0A_gf'_c$	Class IV
9	30”	$0.4d_{pile}$	12”	w/o interface reinforcement	$0A_gf'_c$	Class IV
10	30”	$1.0d_{pile}$	30”	w/o interface reinforcement	$0A_gf'_c$	Class IV

SPECIMEN DESIGN



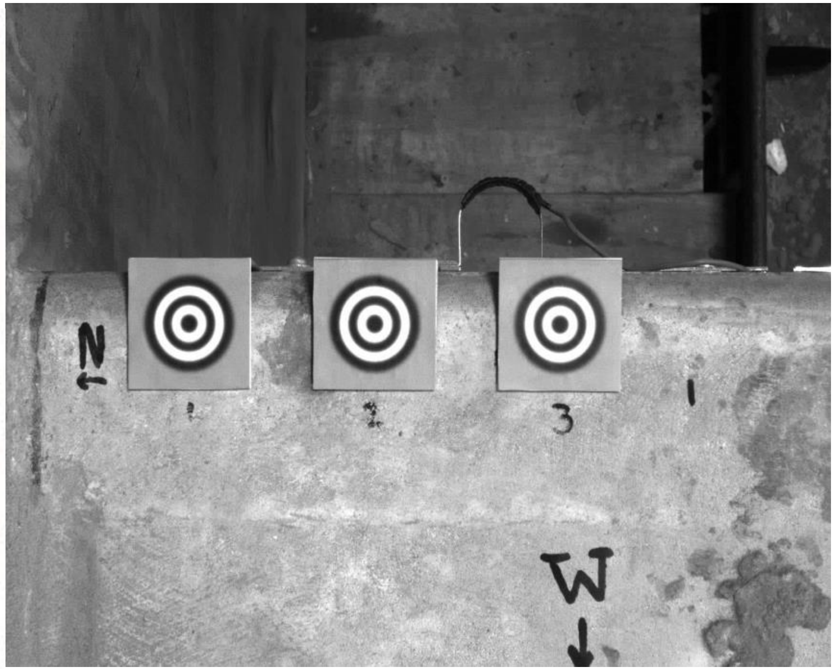
Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Evaluation of Concrete Pile to Footing or Cap Connections

SELF-REACTING FRAME TEST SETUP

CANTILEVER FLEXURE STRENGTH TEST

CRACK OPENING TRACKING BY IMETRUM CAMERA

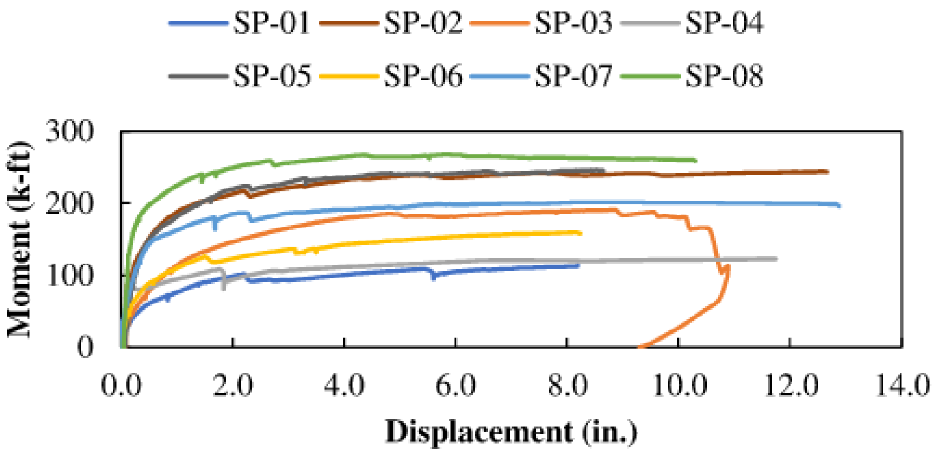


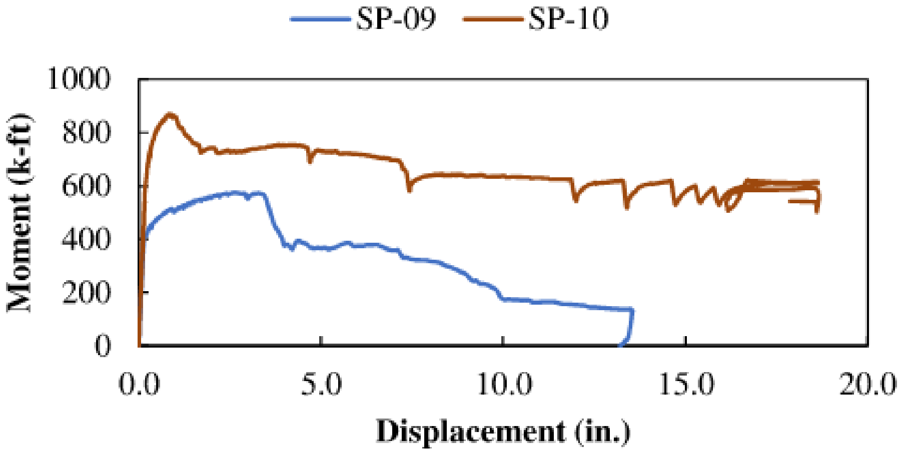
TRANSPORTATION SYMPOSIUM

Source: *https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm*

Evaluation of Concrete Pile to Footing or Cap Connections

FLEXURAL CAPACITY OF PILE-FOOTING CONNECTION





Specimen	Pile Size	Embedment	Failure Moment (k-ft)	Percentage of Pile Capacity	Failure Mechanism
SP-01	18”	6” (0.33d_p)	114.1	34%	Strand Development
SP-04	18”	9” (0.50d_p)	122.8	37%	Strand Development
SP-06	18”	12” (0.67d_p)	159.4	48%	Strand Development
SP-07	18”	18” (1.00d_p)	201.4	61%	Strand Development
SP-08	18”	27” (1.50d_p)	267.6	81%	Strand Development
SP-09	30”	12” (0.40d_p)	574.5	48%	Strand Development
SP-10	30”	30” (1.00d_p)	868.1	73%	Punching Shear

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Evaluation of Concrete Pile to Footing or Cap Connections

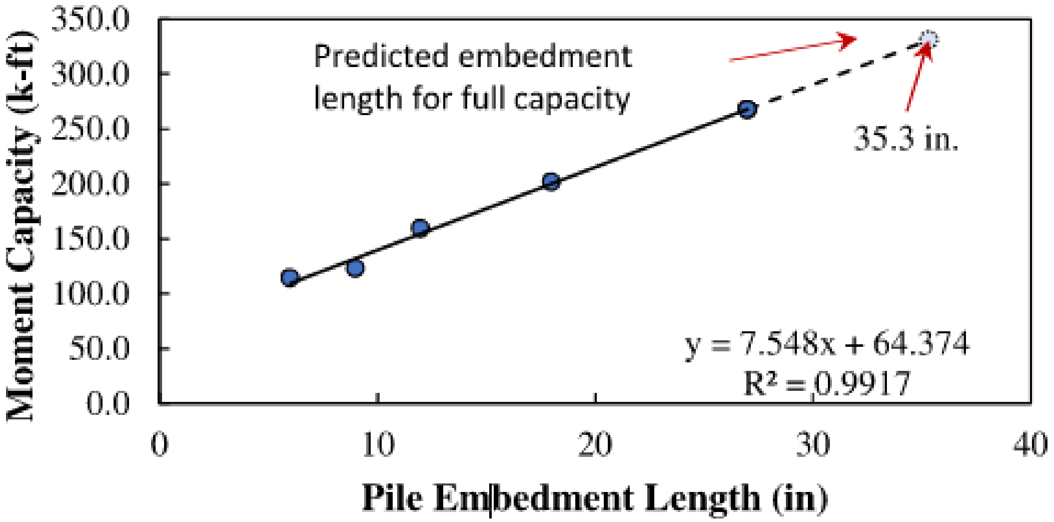
Linear variation between embedment length and moment capacity of the connection:

- Estimated 32.5-inch embedment for full moment capacity of 18-inch pile
- Estimate development length:
- Application of axial load increased the capacity by an average of 107%

AASHTO LRFD BDS → 57.7-inch
ElBatanouny and Ziehl (2012) → 33.4-inch

$$l_d = \left(\frac{f_{se}}{5000}\right) d_b + \left(\frac{f_{ps} - f_{se}}{1800}\right) d_b$$

PREDICTED EMBEDMENT LENGTH FOR FULL PILE CAPACITY



TRANSPORTATION SYMPOSIUM

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Evaluation of Concrete Pile to Footing or Cap Connections

PROPOSED STRAND DEVELOPMENT LENGTH FORMULA

ACI Building Code

Required development length:

$$l_d = \left(\frac{f_{se}}{3000}\right) d_b + \left(\frac{f_{ps} - f_{se}}{1000}\right) d_b$$

Does not consider any clamping or confining stresses

ElBatanouny and Ziehl

Required development length:

$$l_d = \left(\frac{f_{se}}{5000}\right) d_b + \left(\frac{f_{ps} - f_{se}}{1800}\right) d_b$$

AASHTO LRFD BDS

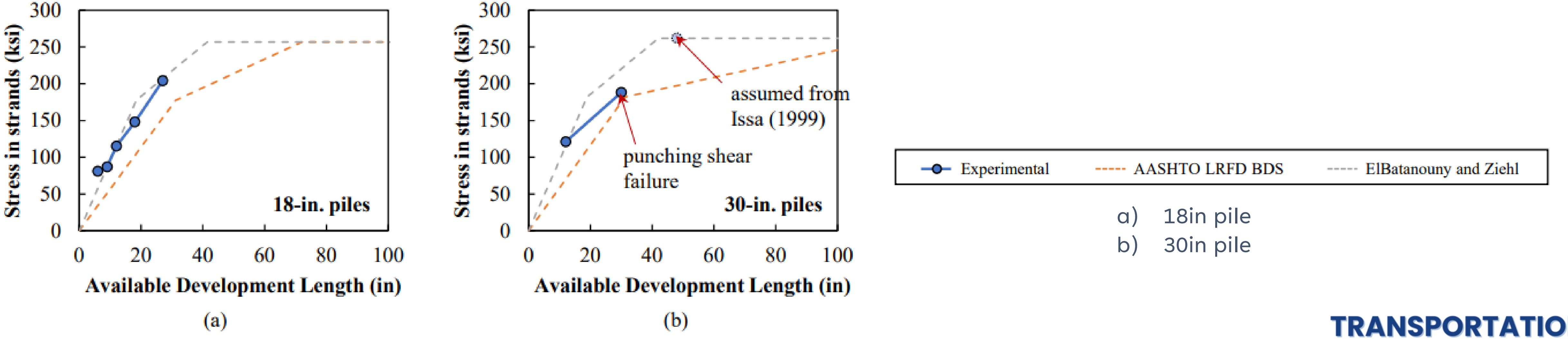
Required transfer length:

$$l_t = 60 d_b$$

Required development length:

$$l_d \geq k\left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b$$

Where: $k = 0.58 \approx \frac{3}{5}$

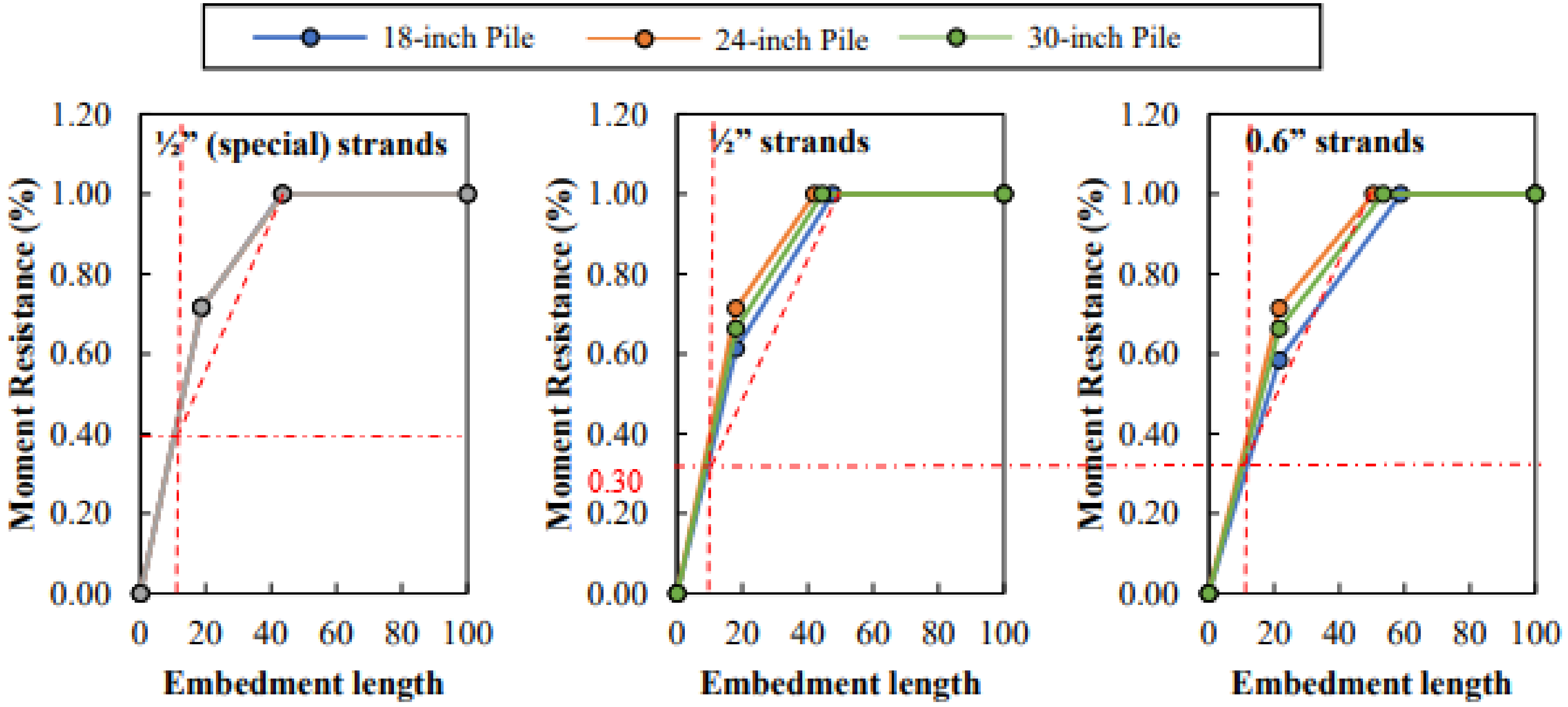


a) 18in pile
b) 30in pile

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Evaluation of Concrete Pile to Footing or Cap Connections

NORMALIZED MOMENT RESISTANCE VERSUS EMBEDMENT LENGTH



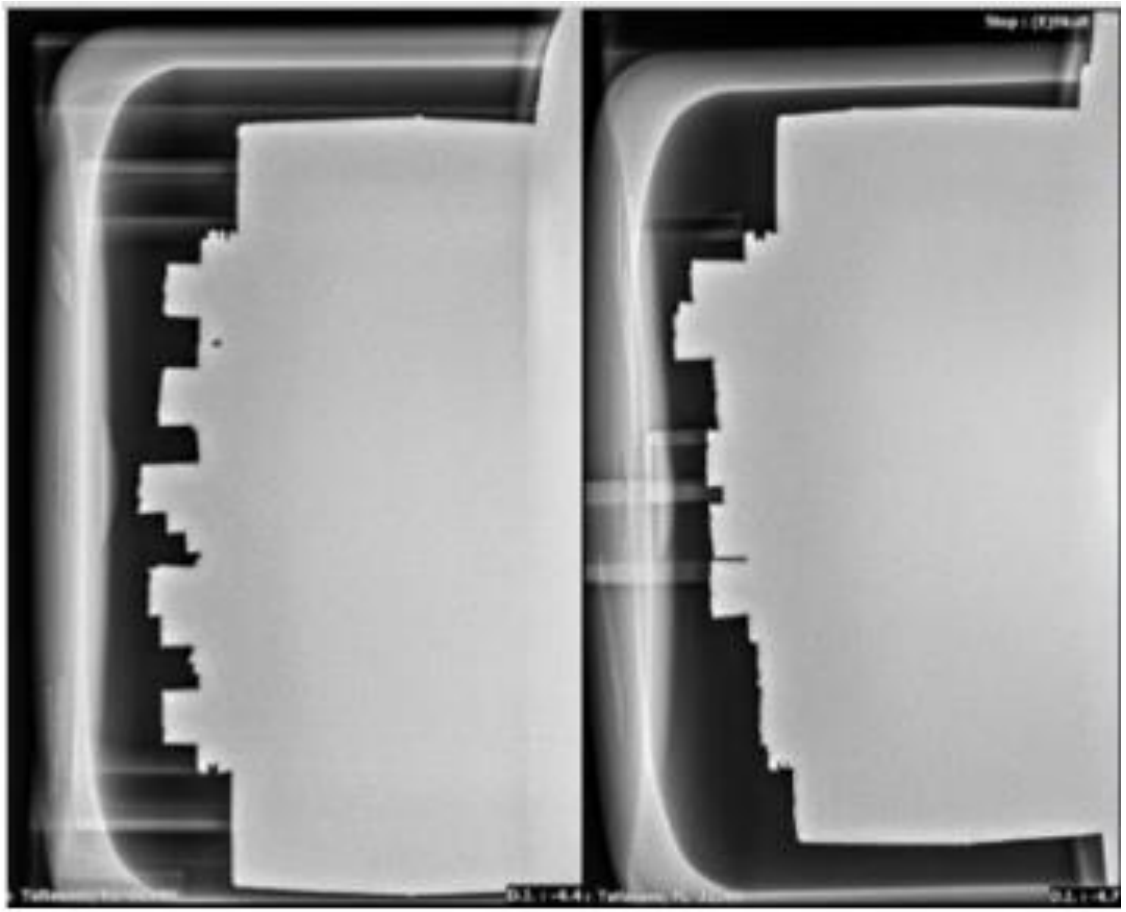
½" special strands configuration

½" strand configuration

0.6" strand configuration

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Inspection of Flexible Filler Tendons



Objectives: This research focused on developing an inspection protocol for post-tensioning (PT) ducts employing flexible filler.

Identify NDE methods for identifying micro-cracking and assessing stress levels in concrete and non-invasive NDE methods for identifying corrosion in tendons and anchorages.

Variables: Duct geometry, number of strands, concrete cover

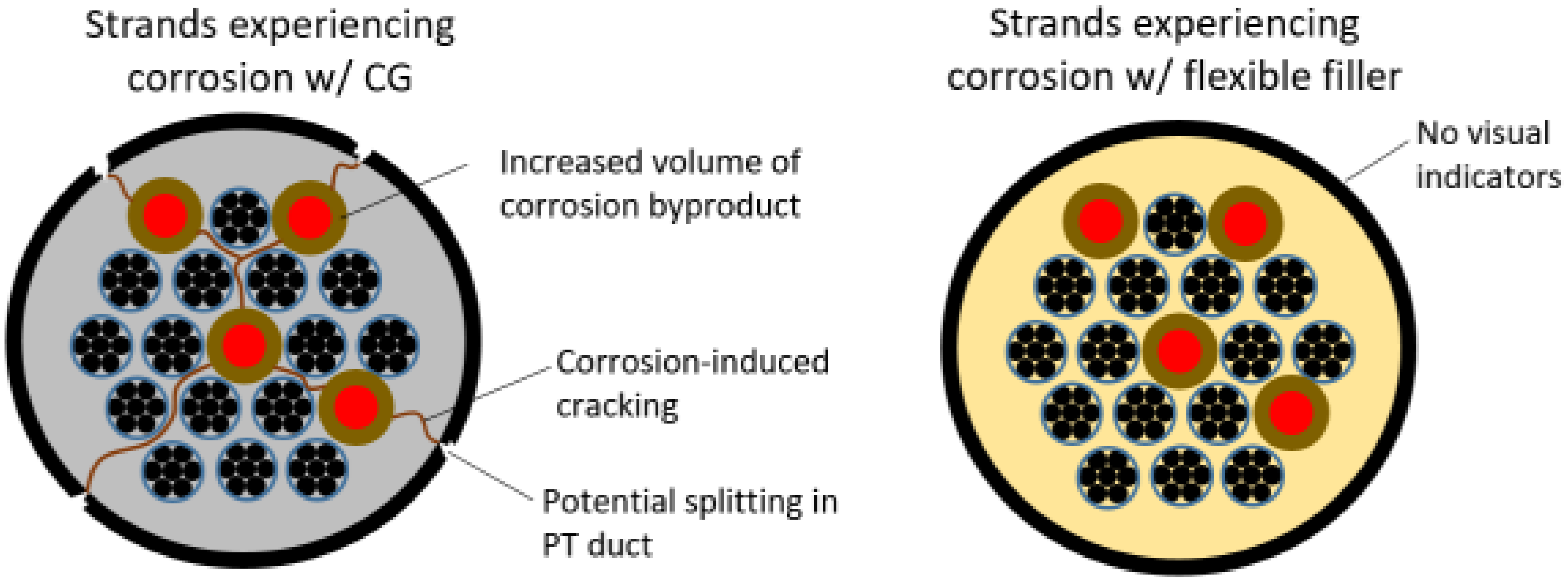
Methods: Ultrasound interferometry and thermoelaticticity on small-scale specimens, and radiography for anchorage cap imaging on full-scale specimens (4-point bending).

TRANSPORTATION
SYMPOSIUM

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Inspection of Flexible Filler Tendons

COMPARISON OF VISUAL INDICATORS FOR CEMENTITIOUS GROUT AND FLEXIBLE FILLERS

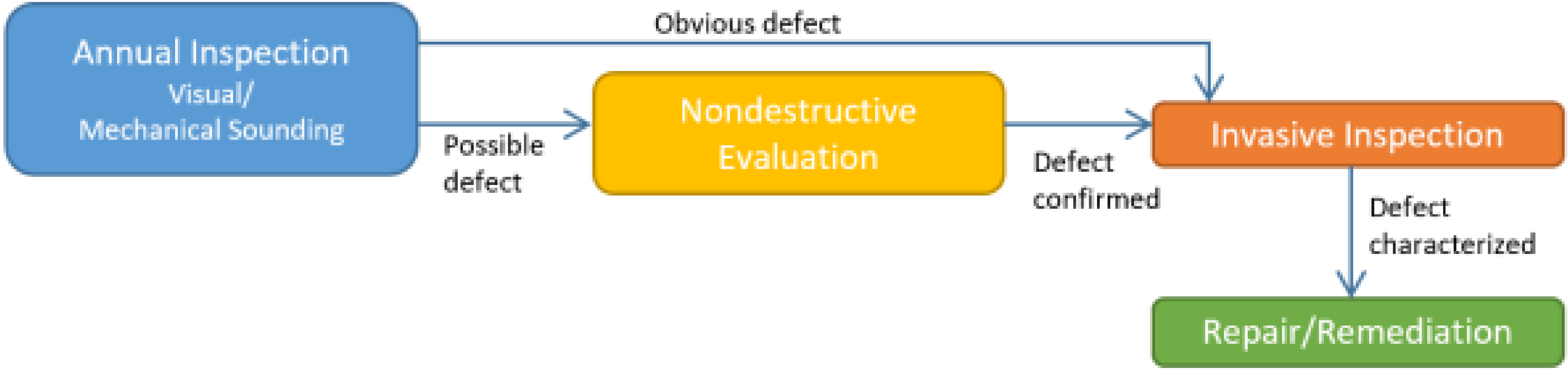


TRANSPORTATION SYMPOSIUM

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Inspection of Flexible Filler Tendons

GENERAL INSPECTION PROTOCOL FOR PT SEGMENTAL BRIDGES



Annual Inspection
Visual/
Mechanical Sounding

Obvious defect

Possible defect

Nondestructive Evaluation

Defect confirmed

Invasive Inspection

Defect characterized

Repair/Remediation

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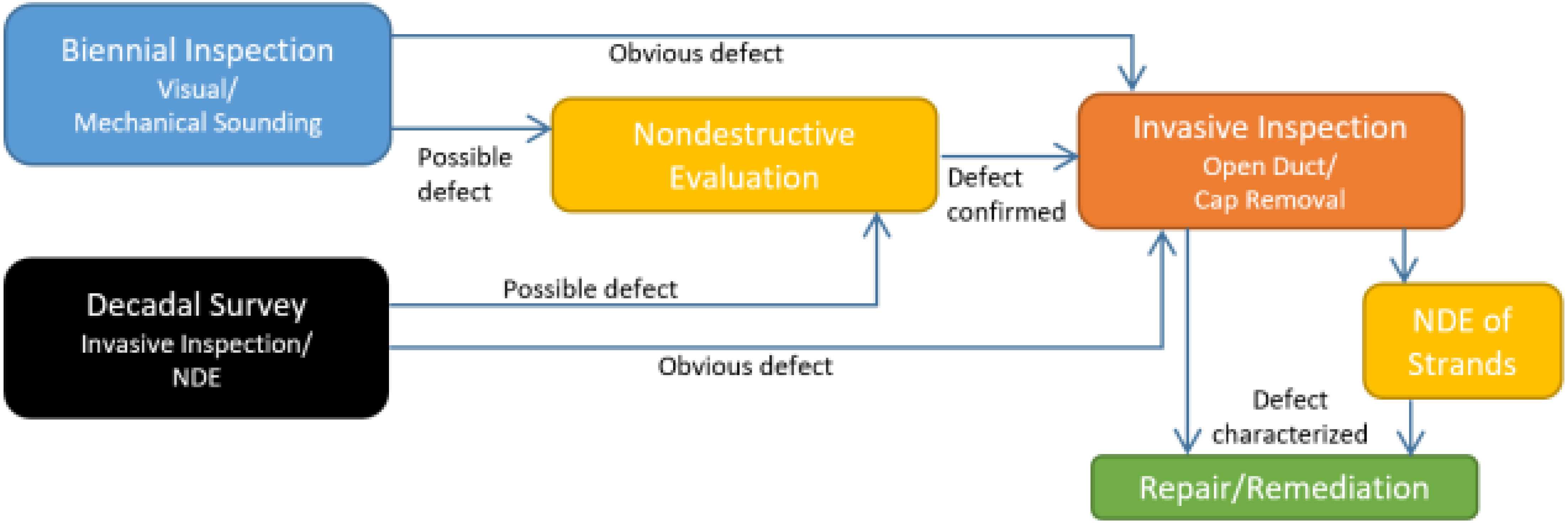
Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Inspection of Flexible Filler Tendons

PROPOSED INSPECTION PROTOCOL FOR PT SEGMENTAL BRIDGES



TRANSPORTATION SYMPOSIUM

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Inspection of Flexible Filler Tendons

RADIOGRAPHY TESTING

Portable X-Ray System POSKOM PXM-20BT

- Low power 60-80 kV, 1.3-1.6mAs
- Minimum Personal Protective Equipment (PPE)
- Suitable for anchorage cap scanning
- Lightweight 20 pounds
- Commonly utilized by veterinarians

Portable YXLON X-Ray machine

- High power higher power capabilities (250kV, 2.4mAs)
- Can scan concrete to a depth of 12in
- Suitable for scanning internal tendons
- Extensive Personal Protective Equipment (PPE) required
- Requires 30ft safety radius during scanning

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

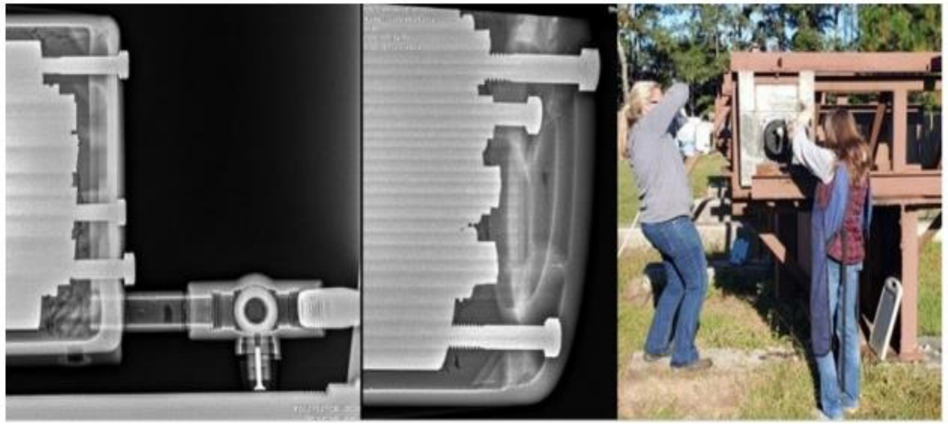
Inspection of Flexible Filler Tendons

RADIOGRAPHY TESTING

Portable X-Ray System POSKOM PXM-20BT

Specimen 1 anchorage cap results

Specimen 1 anchorage trumpet results



External duct mockup: filled anchorage cap results

- Identifies the positions of strands and grips, defects within the cap, voids, or corrosion
- Rapid additional assessment to complement existing inspection procedures.
- Flexible filler material (FF) did not interfere with image quality
- Generates 2D image

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

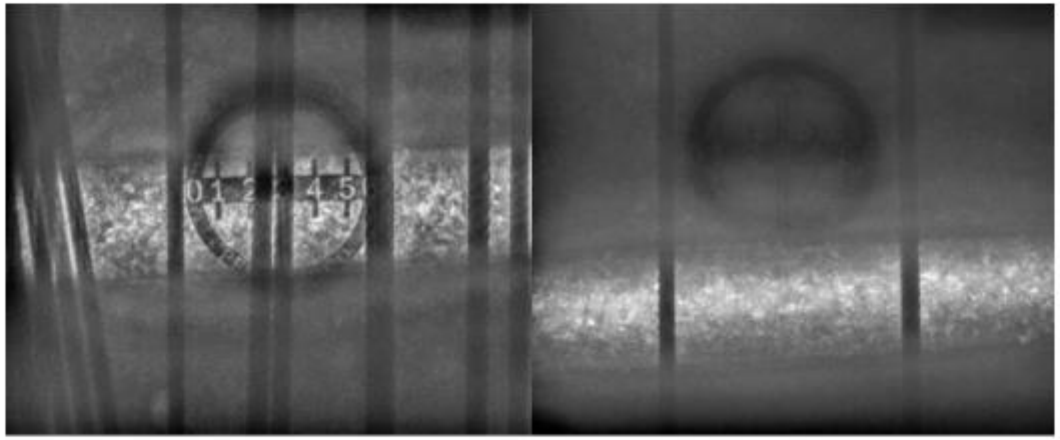
Inspection of Flexible Filler Tendons

RADIOGRAPHY TESTING

Portable YXLON X-Ray machine

Internal Specimen Results

Internal Duct Specimen: duct with strands



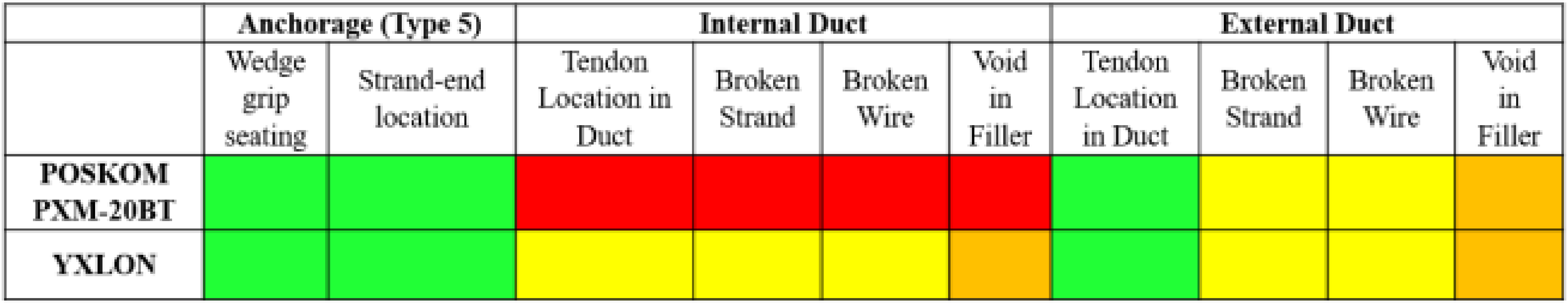
Internal duct specimen: duct without strands

- The YXLON X-Ray is capable of imaging through 12” of concrete.
- Identifies the mild reinforcement in the member
- Provides a highly detailed image of the individual wires comprising the strands.
- Promises for detecting voids and potentially even corrosion

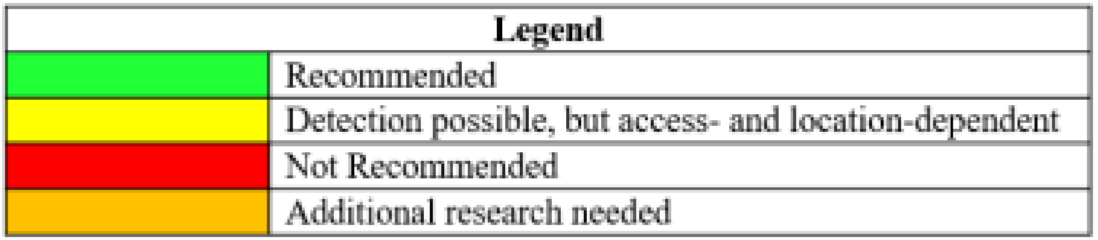
Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Inspection of Flexible Filler Tendons

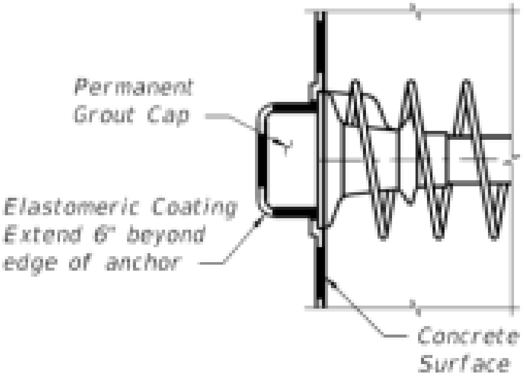
SUMMARY OF RESULTS ON RADIOGRAPHY



	Anchorage (Type 5)		Internal Duct				External Duct			
	Wedge grip seating	Strand-end location	Tendon Location in Duct	Broken Strand	Broken Wire	Void in Filler	Tendon Location in Duct	Broken Strand	Broken Wire	Void in Filler
POSKOM PXM-20BT	🟩	🟩	🟥	🟥	🟥	🟥	🟩	🟨	🟨	🟧
YXLON	🟩	🟩	🟨	🟨	🟨	🟧	🟩	🟨	🟨	🟧



Legend	
🟩	Recommended
🟨	Detection possible, but access- and location-dependent
🟥	Not Recommended
🟧	Additional research needed



TYPE 5

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Inspection of Flexible Filler Tendons

CONCLUSION:

- Inspection methods are generally divided into two categories:
    - direct inspection of the strands/tendon for signs of damage or loss of force;
    - indirect inspection of the post-tensioned concrete for signs that it may be nearing serviceability limits.
- The radiography method does have the potential for direct evaluation of tendons.
- For Type 5 anchorage protection details, strand and wedge-grip dislocations beneath the HDPE cap may be identified without having to remove the cap and expose the flexible filler
- Both types of radiography equipment (low-power medical-grade and high-power construction) should be suitable for these inspections.
- For the case of internal ducts, only the high-powered equipment was shown to be feasible if the overall thickness of the concrete was less than 12 inches.
- For external ducts, both types of equipment can be used to verify the location of the tendon within the duct and, potentially, any strand or wire breakage.

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Bond Performance of Post-Tensioning Tendons with Corrosion Inhibitor

Objectives: Determine whether injection of a corrosion-inhibiting liquid into grouted internal post-tensioned (PT) tendons has a detrimental effect on bond performance & and flexural capacity of girders.

Develop recommendations to include the quantifiable effect of corrosion inhibitor into the AASHTO LRFD code equation for the flexural capacity of PT girders.

Variables: Tendon size, number of strands, tendon profile, type of grout, ductal type.

Methods: Four-point flexure test

Instrumentation: Load Cells at hydraulic jack, LVDTs, Surface Strain Transducers, Vibrating wire gauges, and Digital Image Correlation (DIC).

(Source: FDOT SMO 2021)

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Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Bond Performance of Post-Tensioning Tendons with Corrosion Inhibitor

Nominal flexural strength (M_n)

Fully unbonded tendon

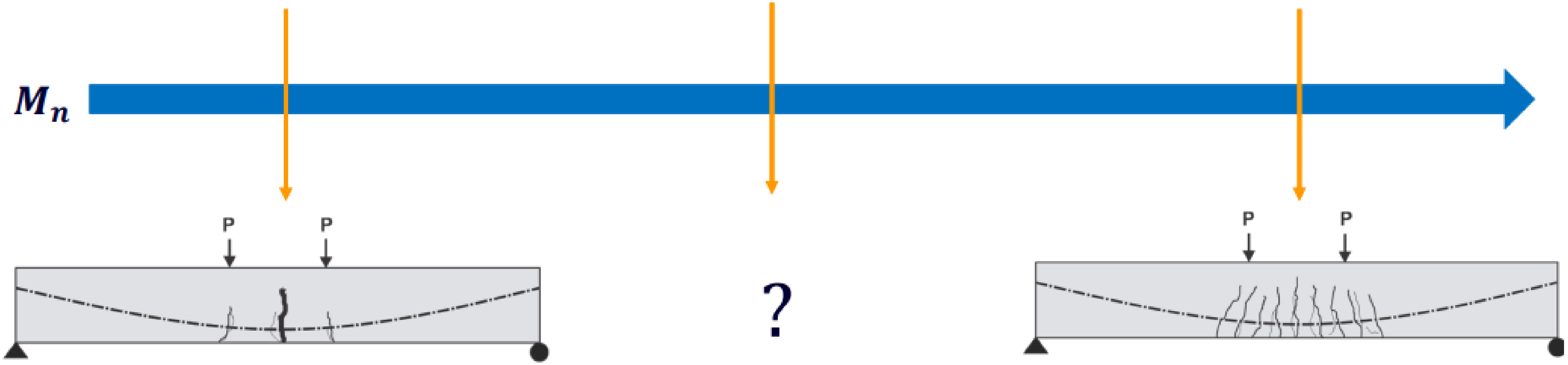
- No grout, strands not bonded
- Strain in strands & adjacent concrete develop independently
- Nominal flexural strength
  - AASHTO models to estimate tendon stress

Partially bonded tendon

- Tendon grouted and Inhibitor injected
- Expected bond degradation
- Nominal flexure strength:
  - TBD

Fully bonded tendon

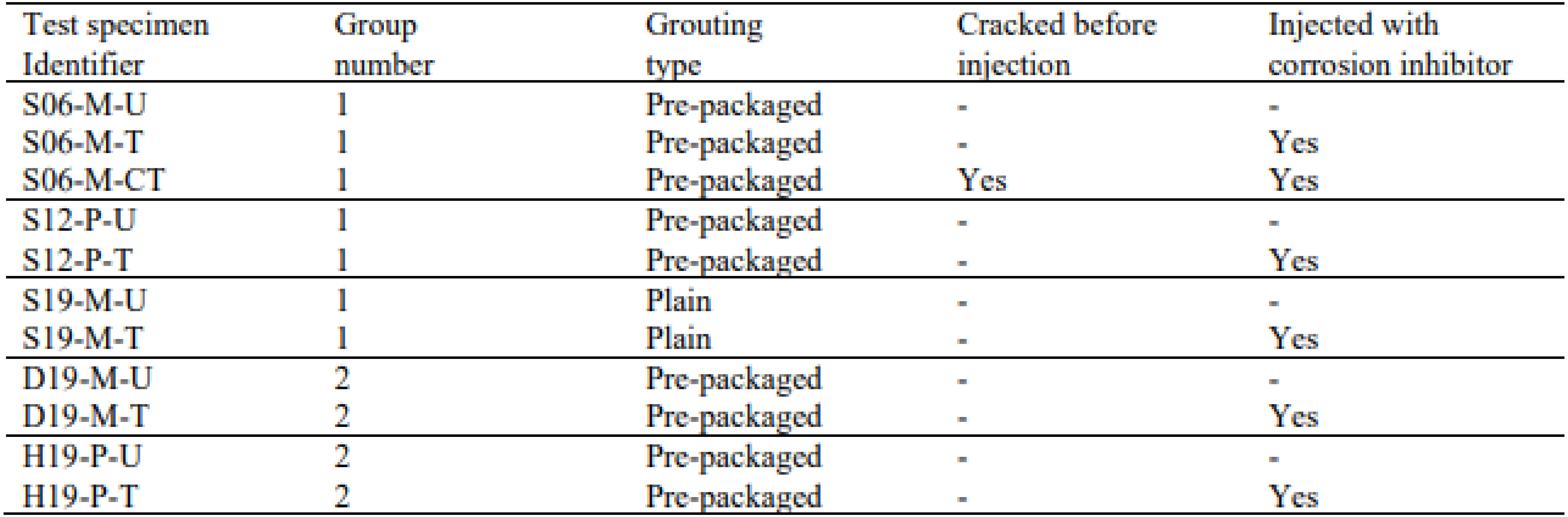
- Tendon grouted
- Bond between strands and grout
- Nominal flexure strength
  - Strain compatibility



Source: UF

Bond Performance of Post-Tensioning Tendons with Corrosion Inhibitor

TEST MATRIX



Test specimen Identifier	Group number	Grouting type	Cracked before injection	Injected with corrosion inhibitor
S06-M-U	1	Pre-packaged	-	-
S06-M-T	1	Pre-packaged	-	Yes
S06-M-CT	1	Pre-packaged	Yes	Yes
S12-P-U	1	Pre-packaged	-	-
S12-P-T	1	Pre-packaged	-	Yes
S19-M-U	1	Plain	-	-
S19-M-T	1	Plain	-	Yes
D19-M-U	2	Pre-packaged	-	-
D19-M-T	2	Pre-packaged	-	Yes
H19-P-U	2	Pre-packaged	-	-
H19-P-T	2	Pre-packaged	-	Yes

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Bond Performance of Post-Tensioning Tendons with Corrosion Inhibitor

TEST SETUP

- 4-point bending test
- I beam pre-cracked
- Treated/untreated specimen couples

INSTRUMENTATION

- Foil strain Gauge (FSG)
- Fiber Optic Sensor (FOS)
- Laser Displacement Transducer (LDT)
- Digital Image Correlation(DIC)

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Projects in Progress

Project Title	**Anticipated End Date**
Assessment and Optimization of the Casting Procedure for Precast UHPC Structural Elements	01/31/2025
Strengthening Piers to Resist Vehicular Collision	04/2024
Bond Performance Between Precast UHPC Substrates and Field Cast UHPC Connections	07/31/2024
Acceptable Crack Width Limit for UHPC Structural Members Under Coastal and Marine Environment	10/31/2025
Evaluation of Ultra-High Performance Concrete (UHPC) Pile Splices	12/31/2025
Evaluation of Skin Friction for UHPC	7/31/2025

Source: https://www.fdot.gov/structures/structuresresearchcenter/activeresearch.shtm

Safety Message

TRANSPORTATION SYMPOSIUM

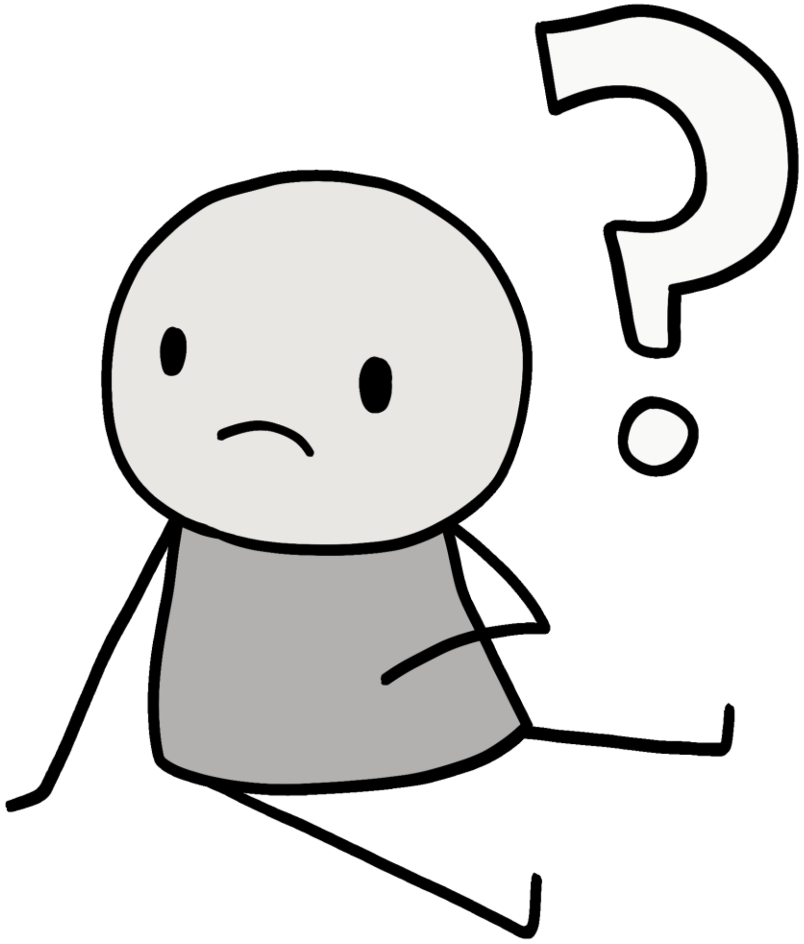
Contact Us ☎

Christina Freeman

- (850) 921-7111
- christina.freeman@dot.state.fl.us

Olga Iatsko

- (850) 921-7105
- olga.Iatsko@dot.state.fl.us



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