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TRANSPORTATION RESEARCH BOARD

A Simplified Full Depth Precast Concrete Deck Panel System

Tuesday, April 30, 2019
1:00-2:30 PM ET

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Purpose

To discuss research from the National Cooperative Highway Research Program (NCHRP)'s Research Report 895: Simplified Full Depth Precast Concrete Deck Panel Systems.

Learning Objectives

At the end of this webinar, you will be able to:

- Describe new advances of full depth precast concrete deck panels for highway bridges
- Describe use of ultra-high performance concrete on highway bridges
- Determine current needs for bridge construction projects

TRB Webinar: A Simplified Full Depth Precast Concrete Deck Panel System
Tuesday, April 30, 2019 1:00 PM - 2:30 PM EDT

"Simplified Full-Depth Precast Concrete Deck Panel Systems"

NCHRP 12-96 (NCHRP Report 895)

Sameh S. Badie, Ph.D., PE, George Washington University (PI)
George Morcous, Ph.D., PE, University of Nebraska-Lincoln (Co-PI)
Maher K. Tadros, Ph.D., PE, econstruct.USA (Co-PI)

NCHRP 12-96

NCHRP 12-96 Panel

Chair:	Ahmad Abu-Hawash, PE (Iowa DOT)
AASHTO Monitor:	Hussam Z. "Sam" Fallaha, PE (Florida DOT)
Members:	Ihab Said Darwish, Ph.D., PE, SE (Alfred Benesch)
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	Carin Roberts-Wollmann, Ph.D., PE (VA Tech.)
	William P. Saffian, PE (New Hampshire DOT)
	Richard B. "Dick" Stoddard, PE (WA State DOT)

NCHRP 12-96 Staff

Senior Program Officer: Waseem Dekelbab, Ph.D., PE, PMP

FHWA Liaison: Benjamin Graybeal, Ph.D, PE

TRB Liaison: Stephen F. Maher

- Goals of the Project
- Features of proposed deck panel system
- Analytical investigation
- Experimental investigation
- Design Guidelines
- Proposed changes to AASHTO LRFD Bridge Design Specifications

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Goals of the Project

1) Develop a simplified full depth precast concrete deck panel system:
   (a) Satisfactory composite action
   (b) Meet ABC goals
   (c) Simplified grouting
   (d) Minimize shear pockets and joints
   (e) Relax tight tolerances
2) Investigate analytically and/or experimentally various design issues
3) Develop guidelines for design and construction
4) Propose revisions to the AASHTO LRFD Bridge Design Specifications

- Goals of the Project
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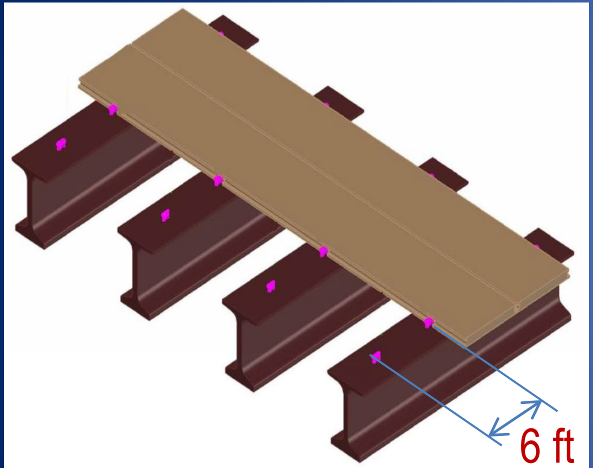
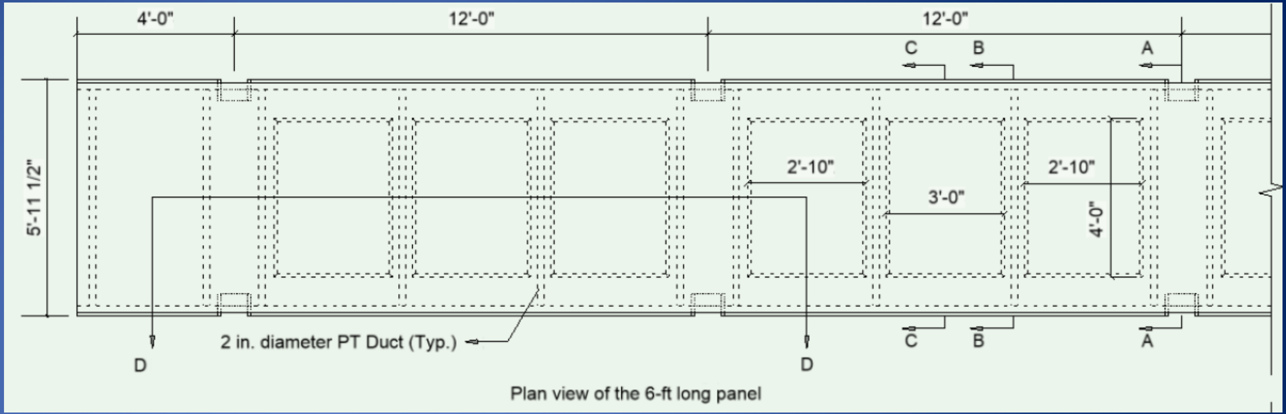
Features of proposed deck panel system

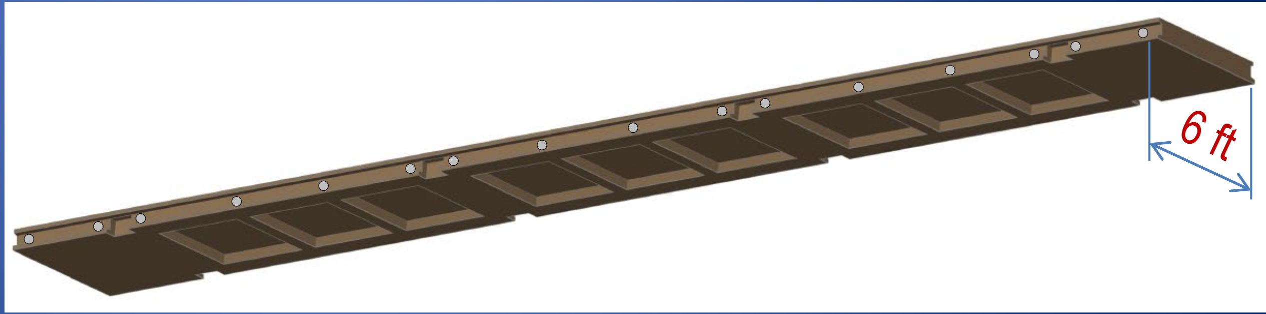
1. Discrete (isolated) horizontal shear connections at 6 ft ** filled with UHPC
2. Haunch between girder top and deck bottom is filled with flowable grout (or may be left unfilled to save time and material)
3. Panels are 6-ft long in direction of traffic with no interior shear pockets (length may be doubled to 12-ft with one interior pocket)
4. Ribbed (waffle) slab to reduce the weight (may be solid)
5. Transverse joints are filled with UHPC with spliced rebars, or conventional concrete with post-tensioning (PT)
6. Unique connection hardware for concrete girders

** NCHRP 12-65 (2008) proved the feasibility of extending the spacing to 4 ft
LRFD Specs. (2014 to present): Spacing = 4 ft

Simplified full depth precast concrete deck panel system





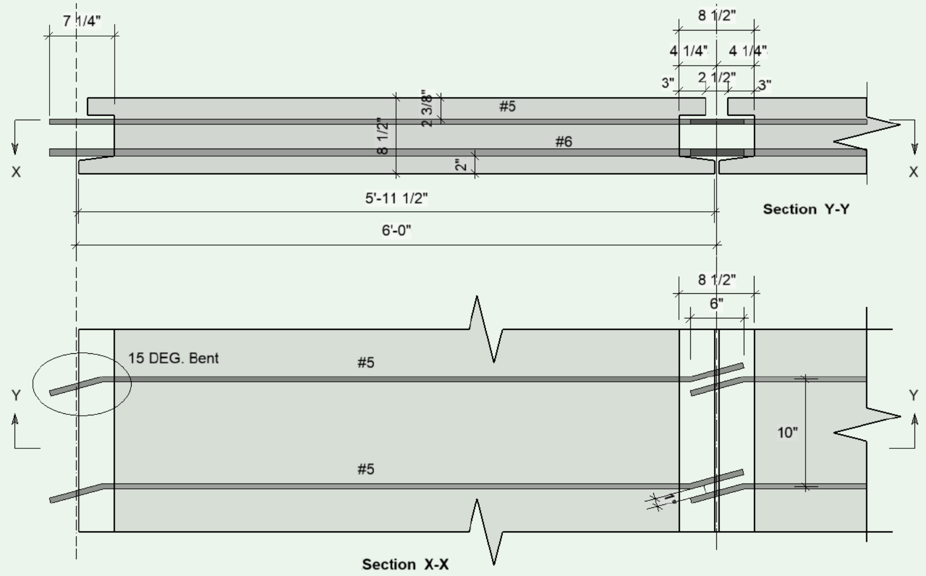


- The panel can be made solid
- The panel can be made 12-ft long with one interior pocket

Panels with Longitudinal PT

Panels with no PT

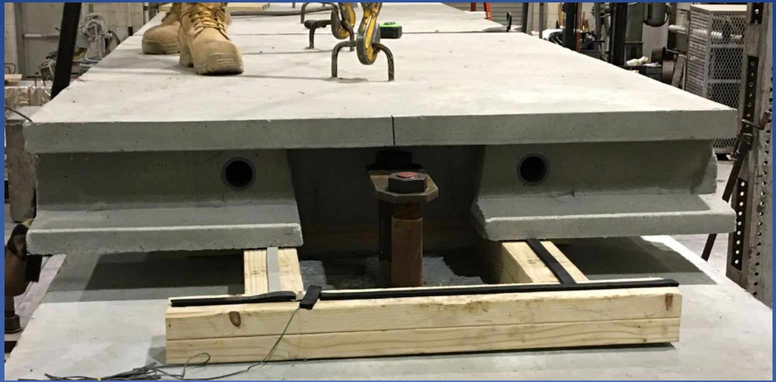
The panel-to-panel transverse joints are filled with UHPC

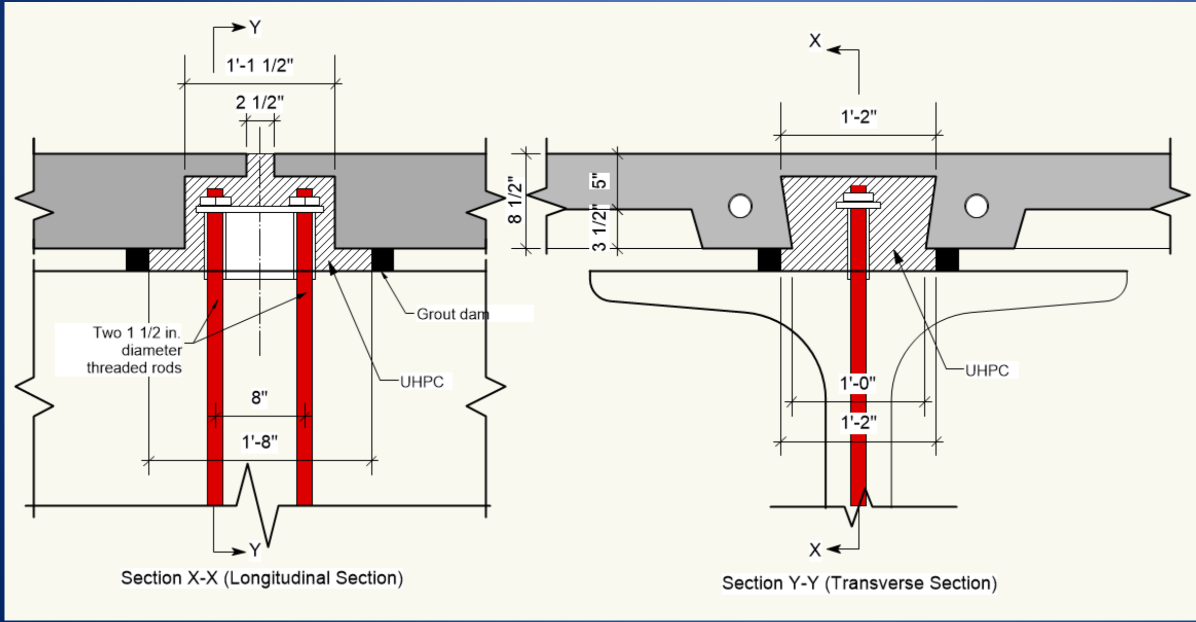


On concrete girders

Innovative connection hardware

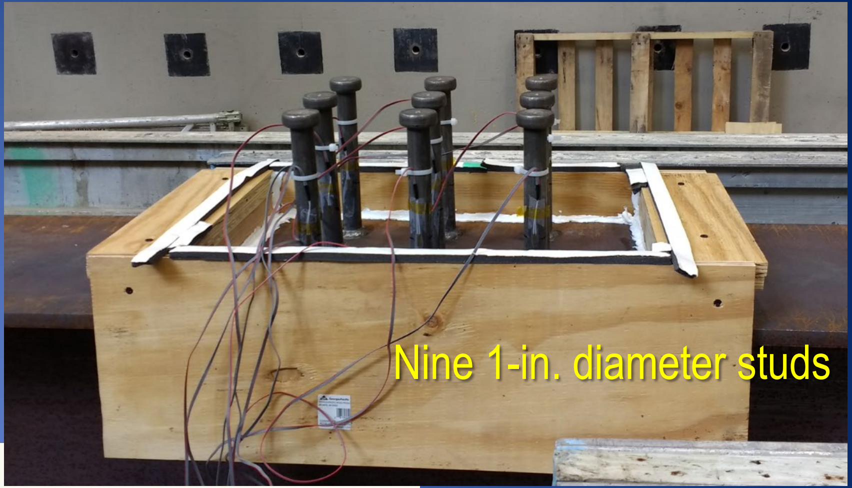
joints are filled with UHPC

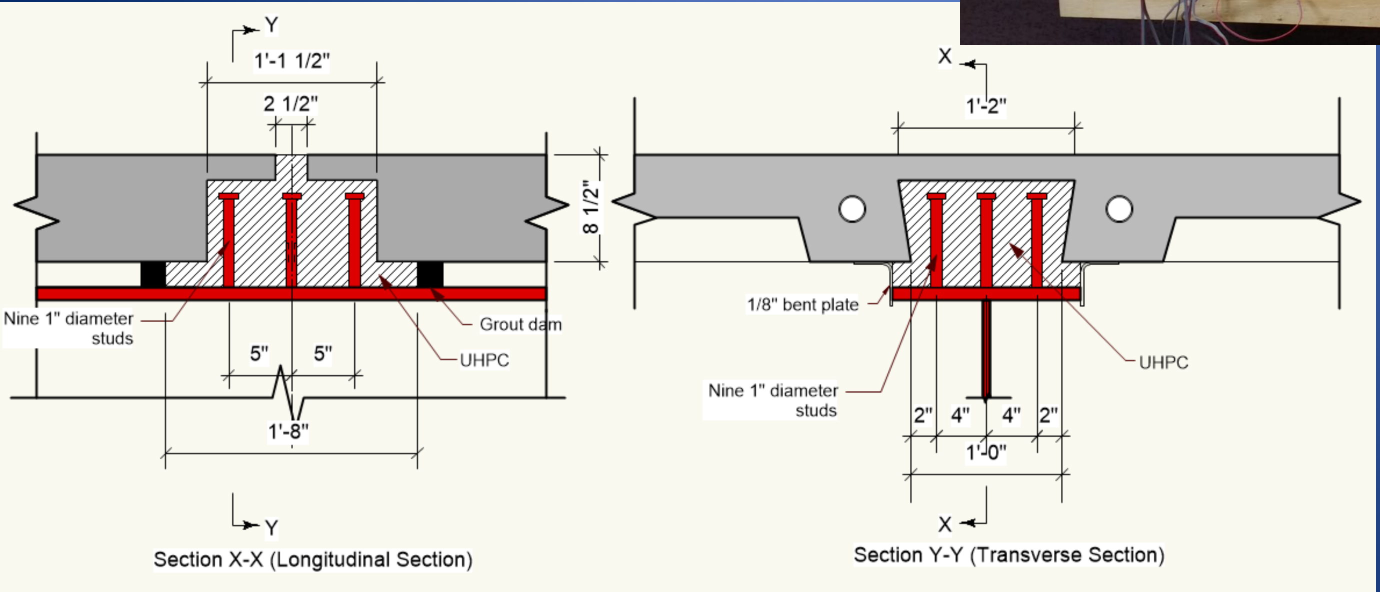




On steel girders

Use large size studs
(1.0 or 1.25 in. diameter studs)
to reduce the size of the joint

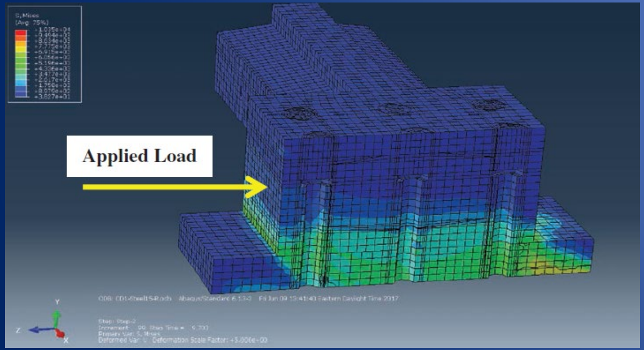


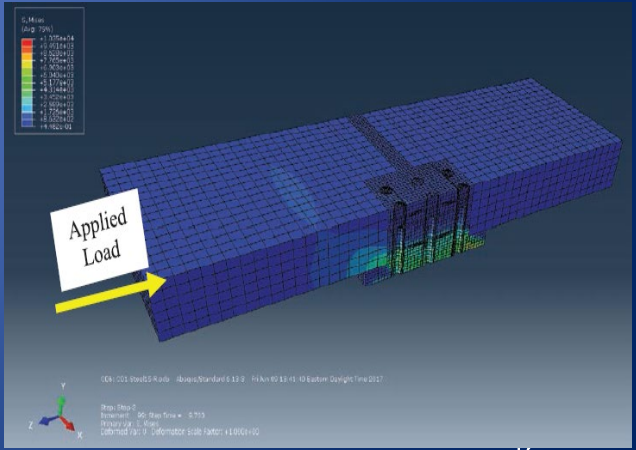


Discrete (isolated) joint connection system

The shear connector joints are filled with UHPC.

The haunch may be filled with flowable concrete.



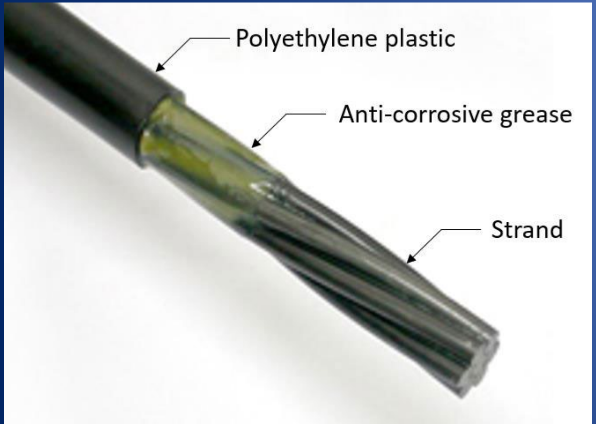


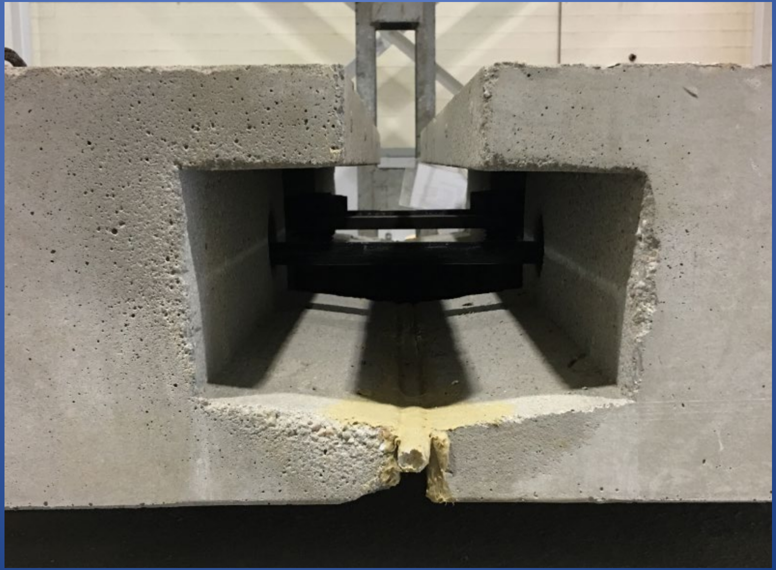
Innovative longitudinal PT system (duct-in-duct, unbonded PT system):

Sheathed strands placed in larger PVC tubes

The PVC tubes are not grouted (unbonded system) and not spliced

Simply supported bridges: PT is applied after the shear connector joints are grouted and cured





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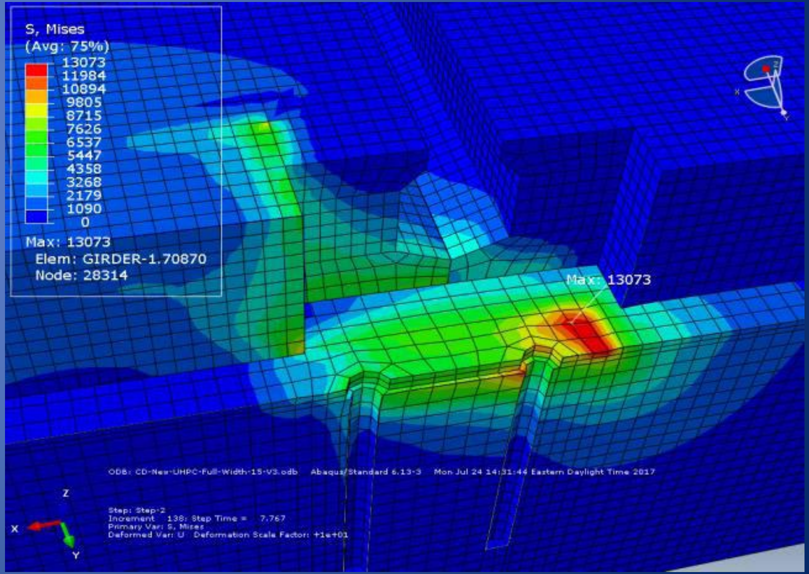
Analytical investigation

1. Deck design
2. Beam design
3. Deck-beam composite system:
   - Distribution factors
   - Flexure design
   - Deflection
   - Vertical shear
   - Interface shear
   - Top flange buckling
   - Effect of simplification of deck post-tensioning

Tools:

1- Vierendeel model

2- Non-linear finite element analysis



Deck slabs with discrete joints

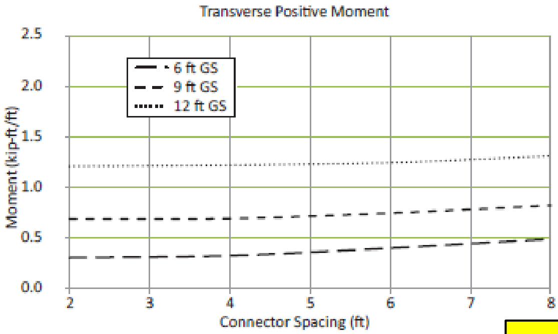
Deck design

Design aids were developed for HL93 and 100 psf.

A commercial finite element package was used in the analysis.

The deck was modeled using plate elements (bending-only surface elements).

HL93



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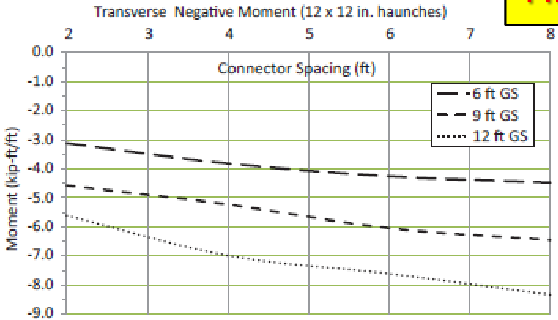

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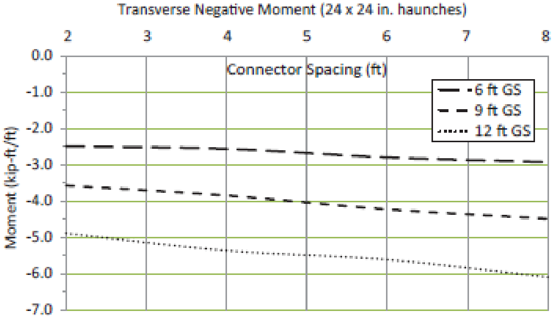

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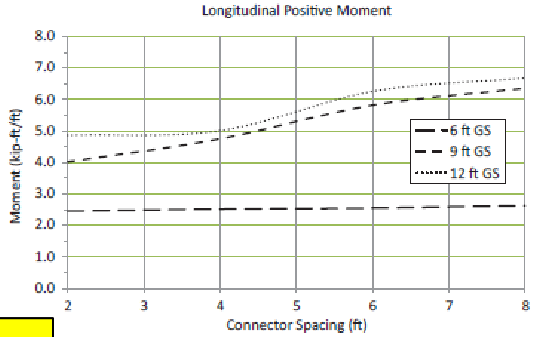

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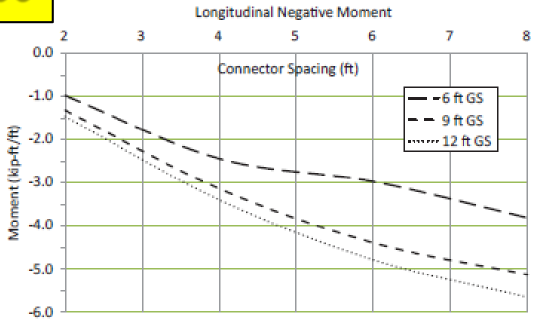

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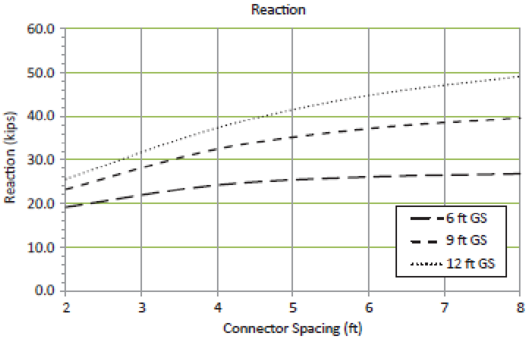

Deck slabs with discrete joints

Vierendeel Model was used to investigate deflection, interface shear, and vertical shear

The investigation has shown that the simple Euler–Bernoulli Beam Theory yields conservative results

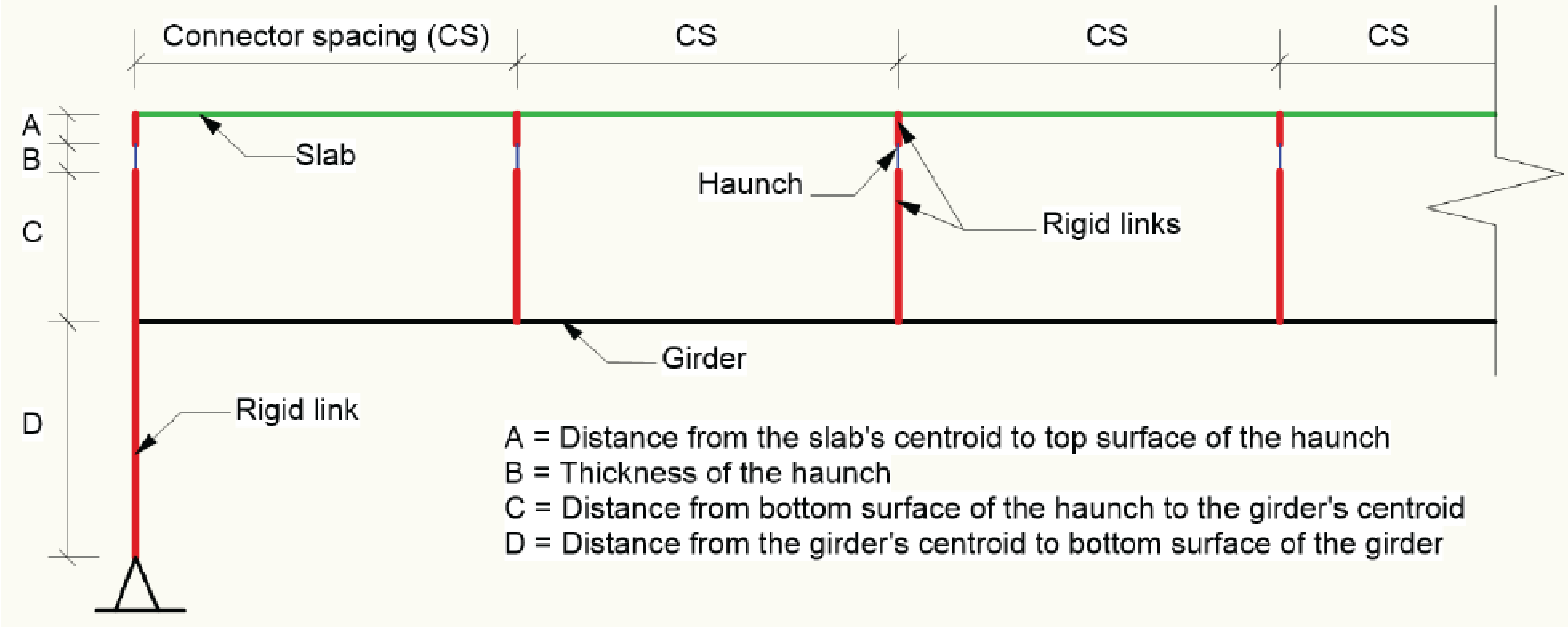

Figure 3.8. Vierendeel Model of a slab–girder composite system with discrete joints.

Deck slabs with discrete joints

Distribution factors

A three-dimensional finite element model was used in the investigation. The slab and the haunch were modeled using the 8-node linear reduced integration brick elements.

The AASHTO LRFD provides conservative results.

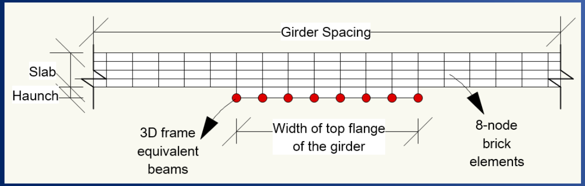

Table 20. Effect of changing the design parameters on the distribution factor

	DFM (Two lanes)			DFV (Two lanes)		
Parameter under investigation	B6	B8	LRFD	B6	B8	LRFD
1- Change girder concrete strength of Example 1 From 8 ksi to 12 ksi						
Base line (Concrete St. = 8 ksi)	0.391	0.403	0.533	0.690	0.690	0.671
(Concrete St. = 12 ksi)	0.397	0.414		0.694	0.694	
2- Change slab concrete strength of Example 1 From 4 ksi to 8 ksi						
Base line (Concrete St. = 4 ksi)	0.391	0.403	0.533	0.690	0.690	0.671
(Concrete St. = 8 ksi)	0.365	0.376		0.679	0.679	
3- Change slab thickness of Example 1 From 7.5 in. to 10 in.						
Base line (Thickness = 7.5 in.)	0.391	0.403	0.533	0.690	0.690	0.671
(Thickness = 10 in.)	0.366	0.360		0.540	0.540	
4- Change haunch length of Example 3 From 12 in. to 24 in.						
Base line (Length = 12 in.)	0.652	0.650	0.830	1.061	1.061	1.130
(Length = 24 in.)	0.649	0.640		1.061	1.062	
5- Change haunch thickness of Example 3 From 2 in. to 6 in.						
Base line (tThickness = 2 in.)	0.652	0.650	0.830	1.061	1.061	1.130
(Thickness = 6 in.)	0.667	0.657		1.071	1.071	

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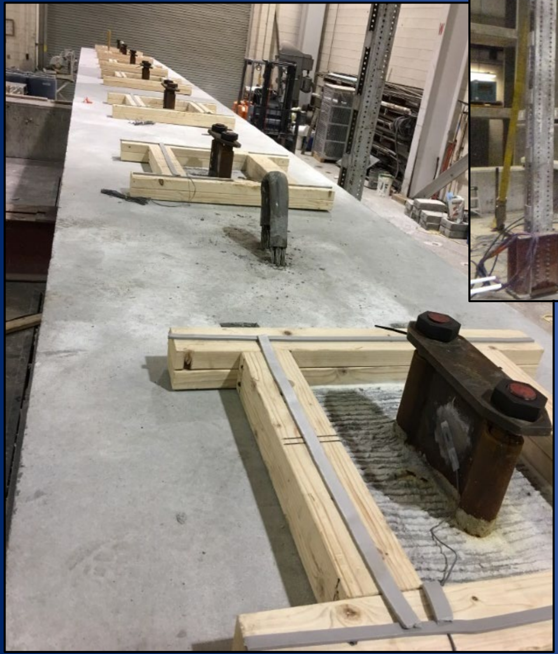
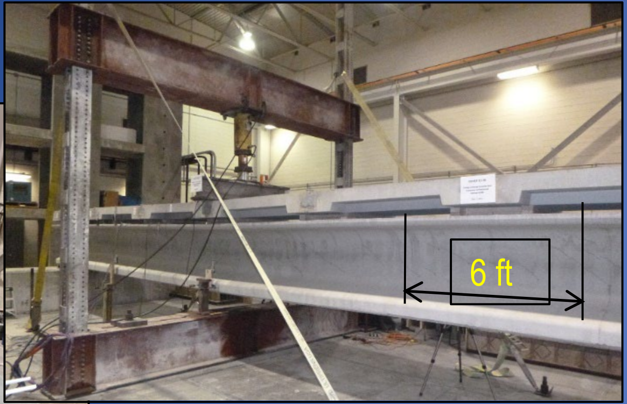
Experimental investigation

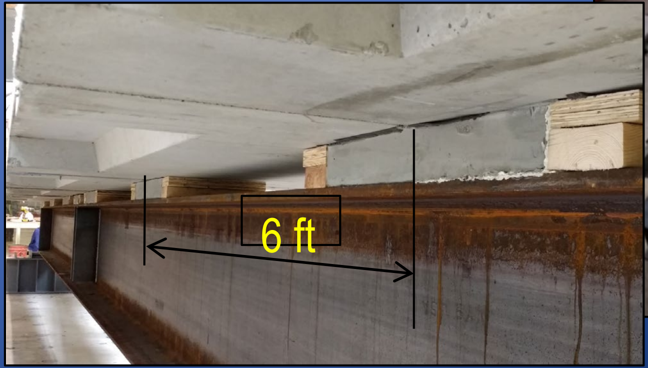
With concrete girder (UNL)	With steel girder (GWU)
Push-off specimens	Push-off specimens
Large scale composite beam with unbonded longitudinal PT Tested for: - Strength for interface shear, flexure & vertical shear	Large scale composite beam with no longitudinal PT Tested for: - Fatigue: 6.8 million cycles - Strength for interface shear & flexure
Ribbed Panels & the haunch was unfilled	

Discrete (isolated) joint connection system

The shear connector joints are filled with UHPC







Experimental investigation

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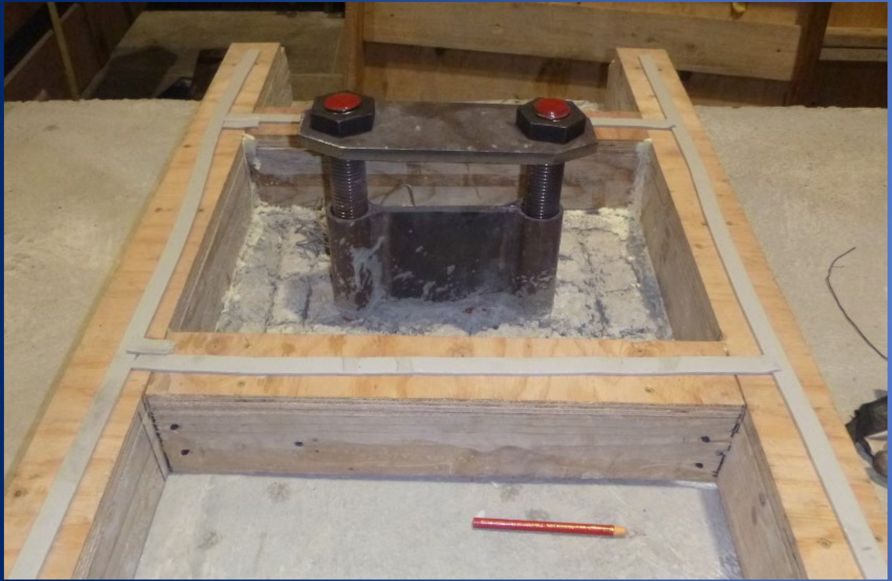
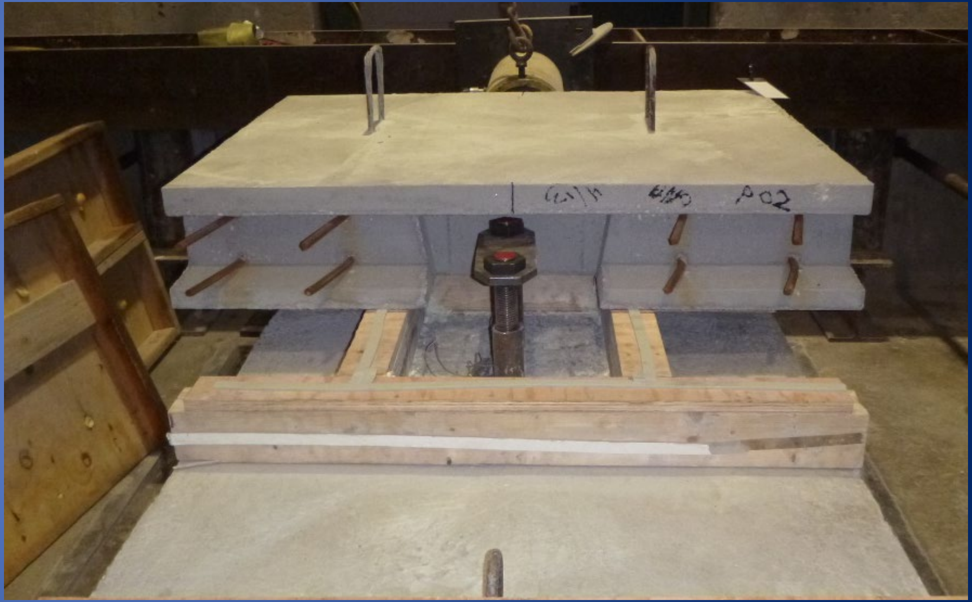
Ribbed Panels & the haunch was unfilled	

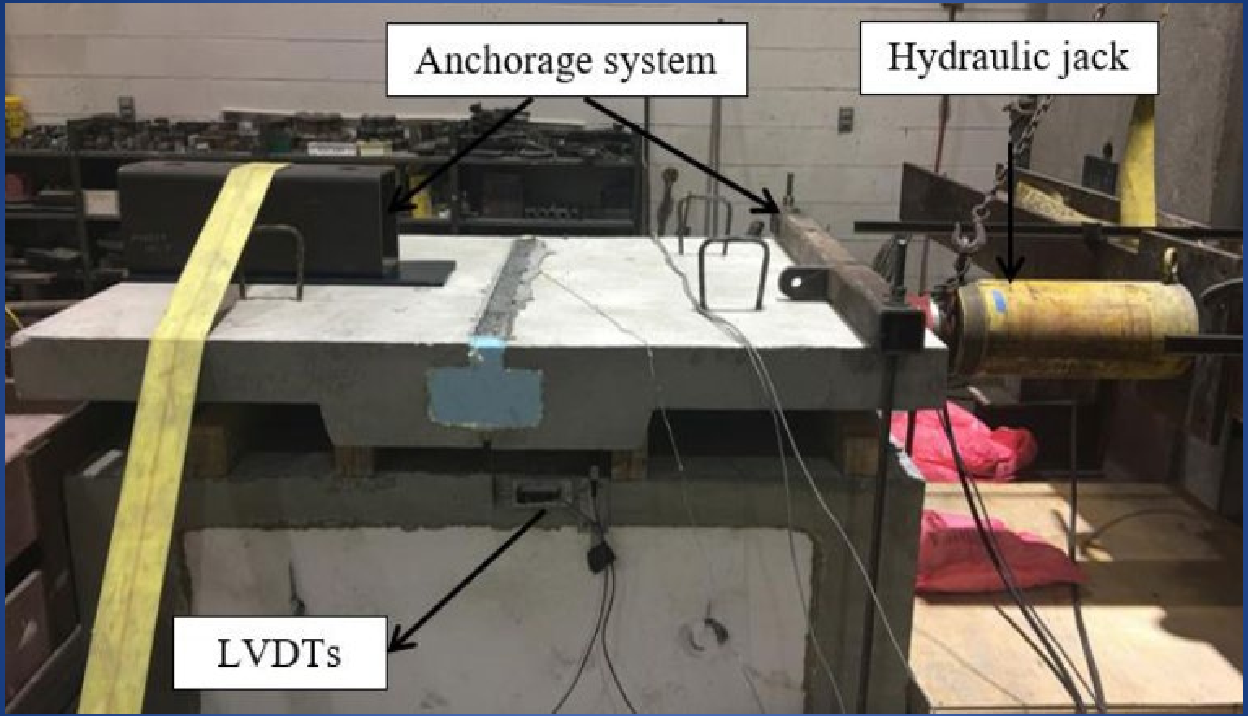
Experimental investigation with concrete girder (Push-off Specimens)





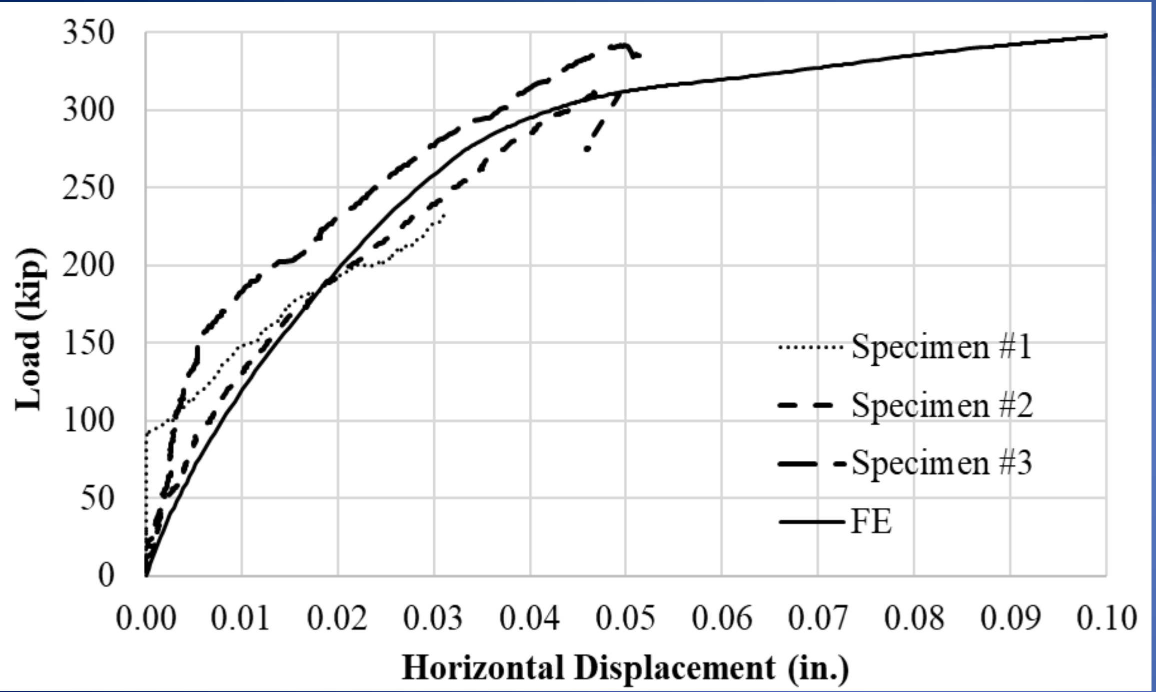
Predicted horizontal shear capacity = 236 kip

Experimental investigation with concrete girder (Push-off Specimens)



Average interface shear capacity = **295 kips** (predicted = 236 kips)
UHPC compressive strength = 14.8 ksi

Experimental investigation with concrete girder (Push-off Specimens)

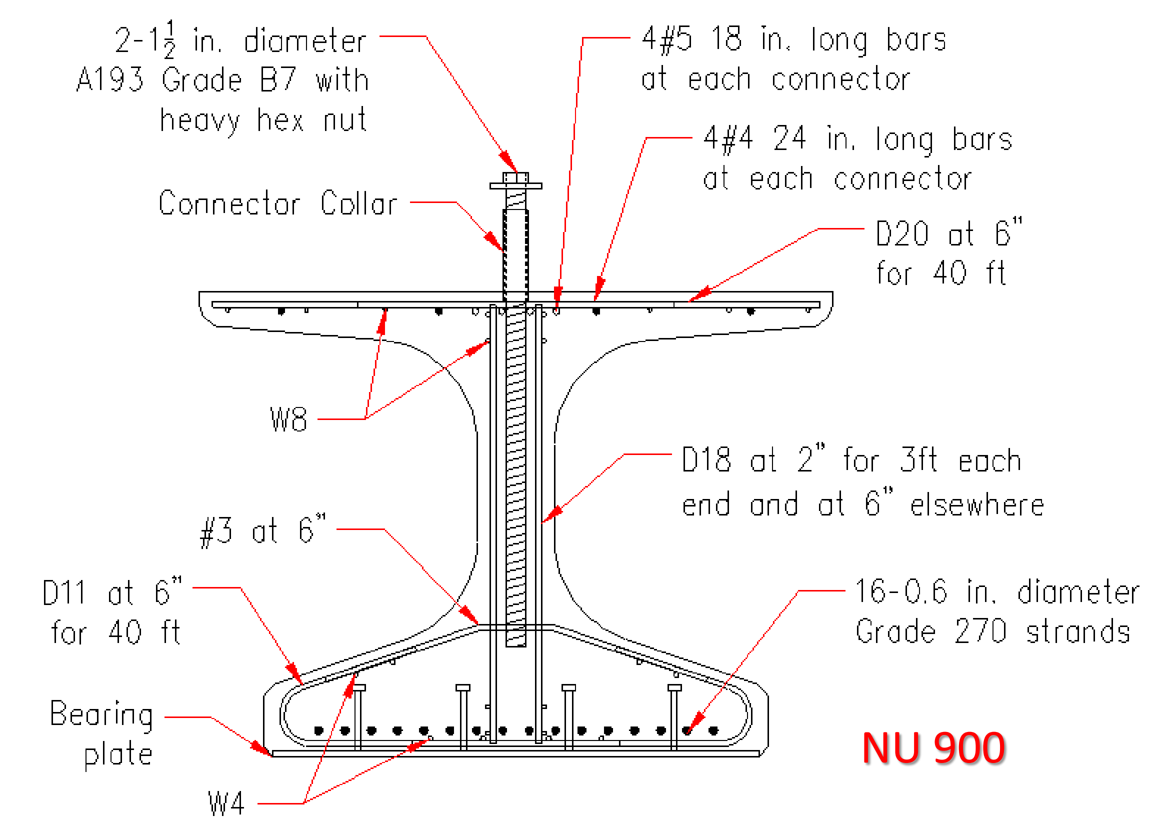


Concrete deck failure

Average interface shear capacity = **295 kips** (predicted = 236 kips)

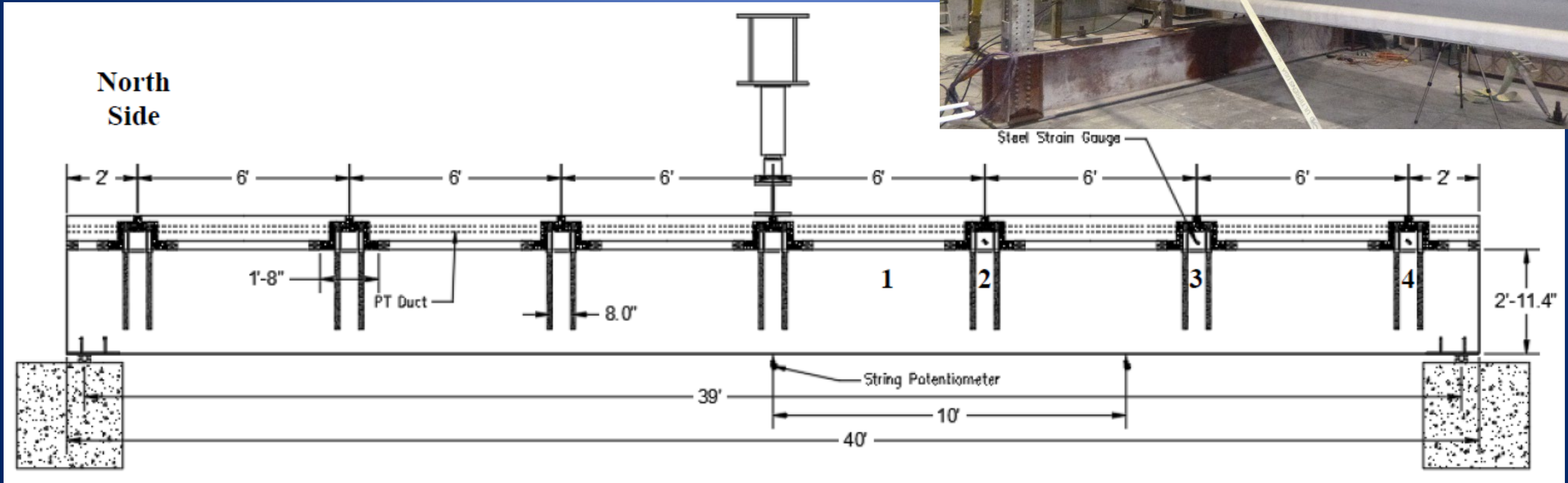
UHPC compressive strength = 14.8 ksi

Experimental investigation with concrete girder (Large-scale composite beam)



Experimental investigation with concrete girder (Large-scale composite beam)

Predicted interface shear strength = 236 kips
and
Predicted composite flexural capacity = 3,401 kip-ft



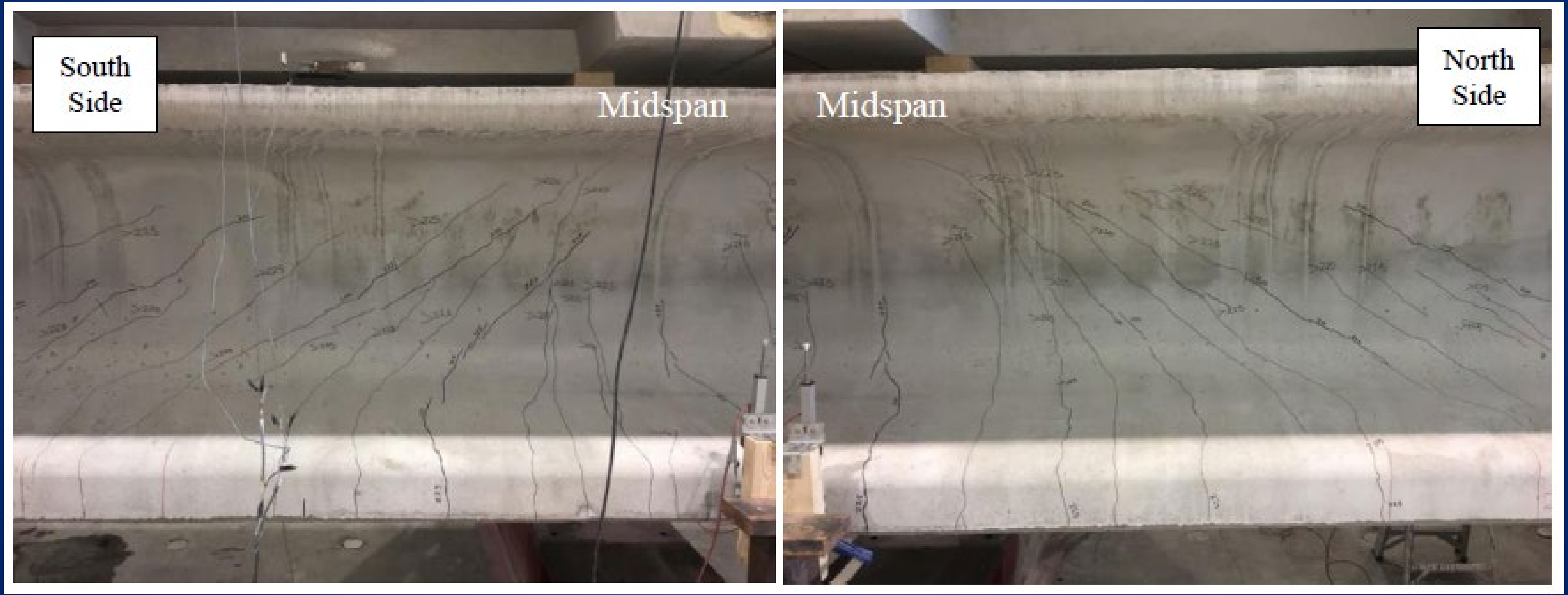
Maximum applied load = 338 kips

Measured interface shear per joint = **325 kips** (predicted 236 kips)

No observed cracks at the joints

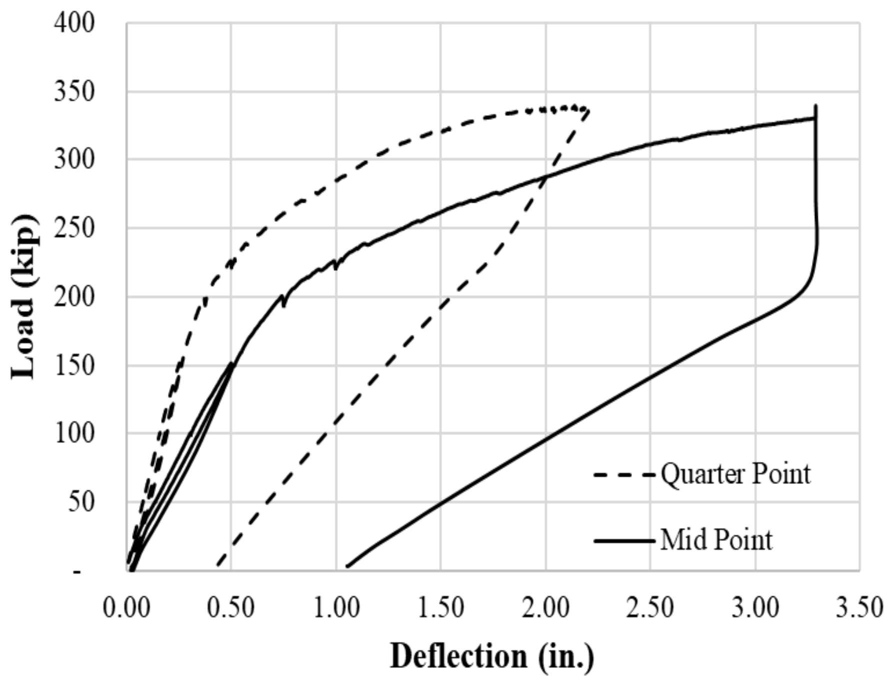
Measured flexural capacity = **3,505 kip-ft** (predicted 3,401 kip-ft)

Flexure & shear-flexure cracks in the girder

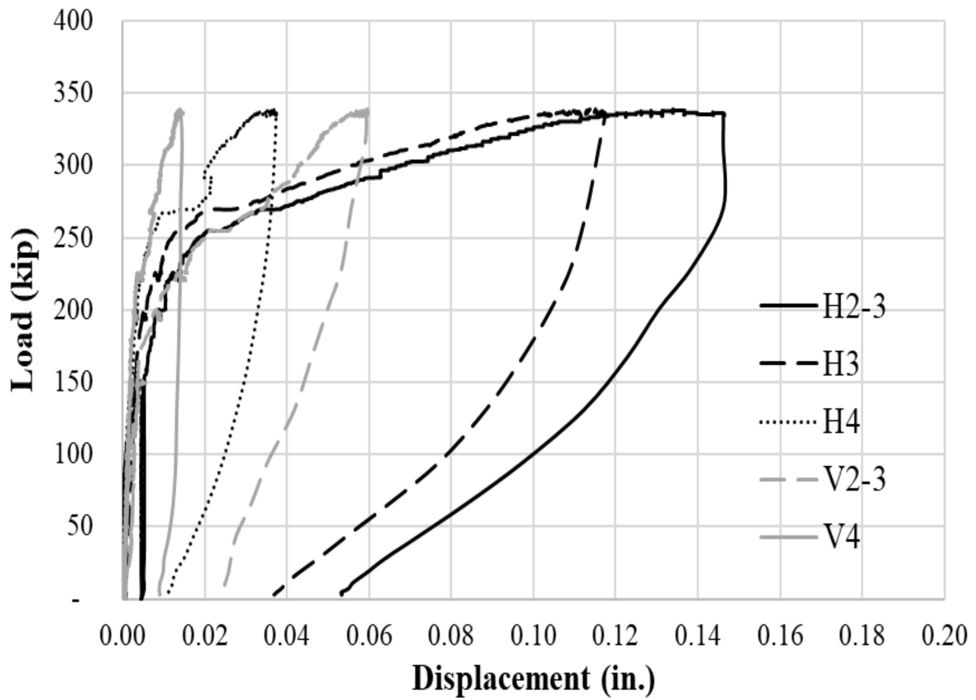


Experimental investigation with concrete girder (Large-scale composite beam)

Interface shear strength



Load-deflection relationship



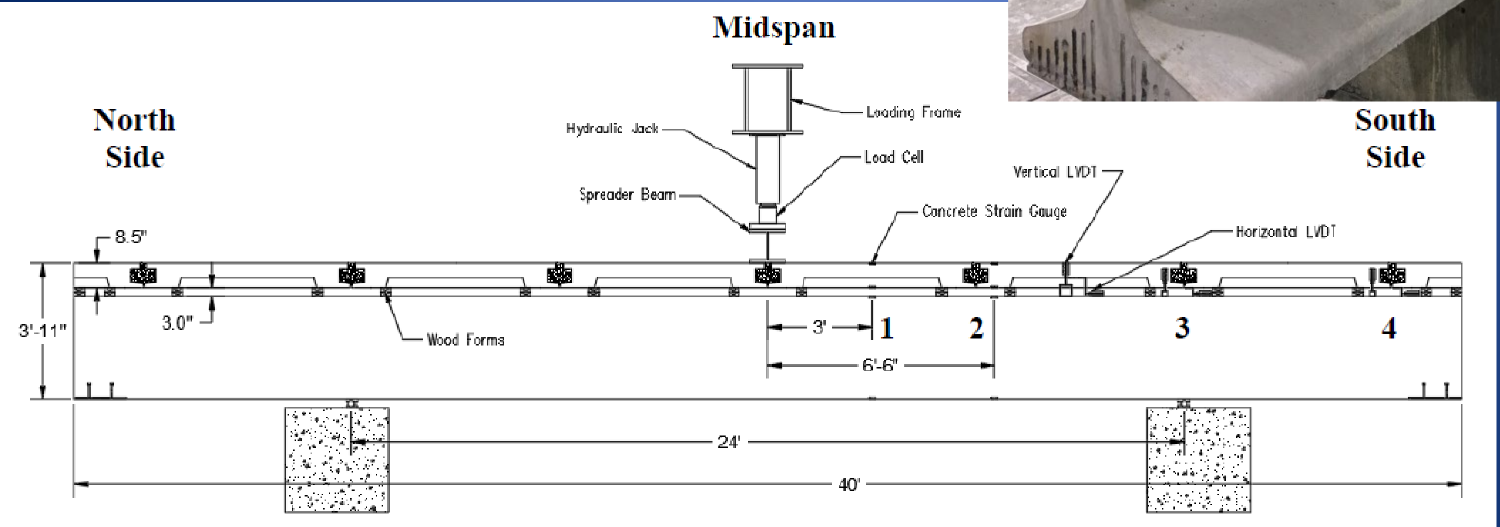
Load-horizontal displacement relationship

Experimental investigation: Interface shear strength

Average interface shear capacity per joint (Push-off & Large Scale Beam) = (295+325) /2 = 310 kips
= 310 kips / 6 ft = 51.7 kip/ft = 4.3 kip/in.

Bridge Name	Span (ft)	Span Type	Girder Size	Girder Depth (ft)	Girder Spacing (ft)	Span / Girder Depth	Interface Shear Demand (kip/in.)	Demand Interface Shear (kip/6 ft)
PCI BDM Ex. 9.1a	120	Simple	BT-72	6	9	20.0	2.86	205.9
Florida Bridge, FL	150	Simple	FL I-72	6	10	25.0	3.38	243.4
Oxford South, NE	110	Continuous	NU1350	4.42	9	24.9	3.58	257.8
Kearney East, NE	166	Continuous	NU1800	5.92	8.5	28.0	3.70	266.4
					Average	**25.9**	**3.6**	**261.9**

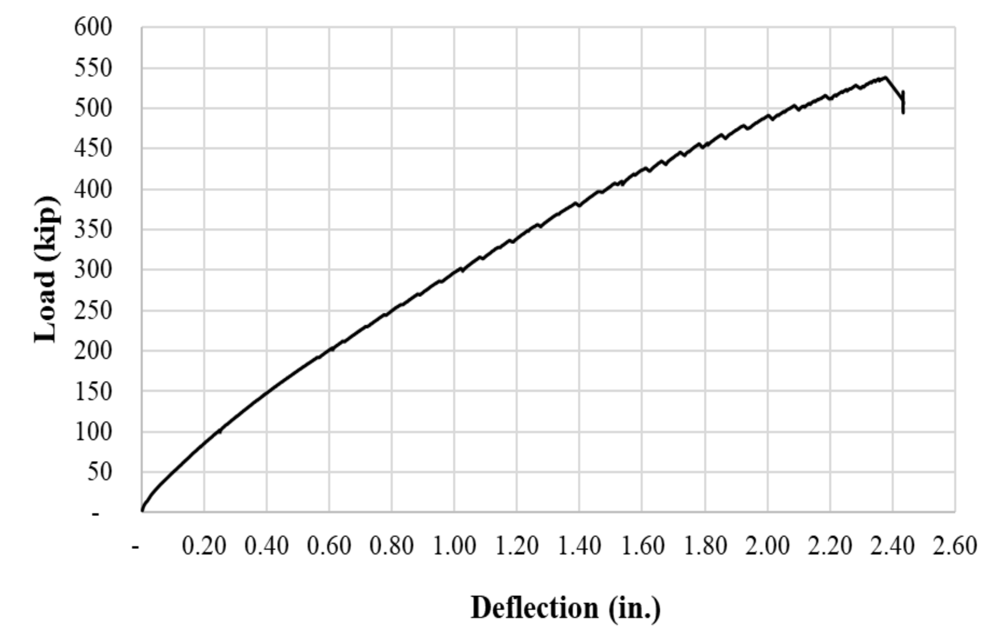
Experimental investigation (Large scale beam):
Vertical shear Strength



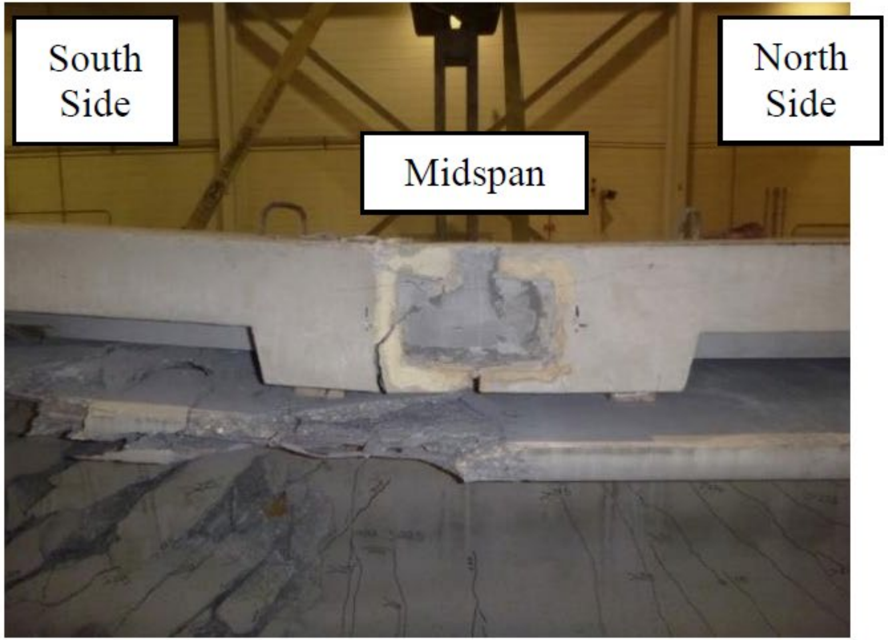
Experimental investigation (Large scale beam): Vertical shear strength

Maximum applied load = 544 kips

Measured vertical shear capacity = **272 kips** (Predicted capacity = 221 kips)



Experimental investigation (Large scale beam): Vertical shear strength



Experimental investigation:

Applying longitudinal PT after the deck is made composite with the beam

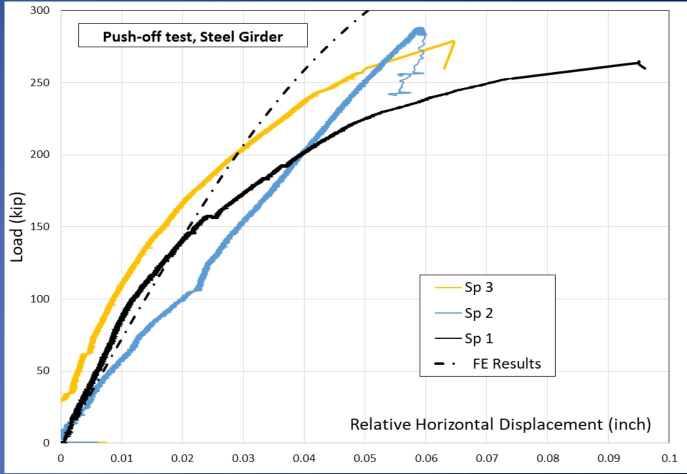
Girder Section	Girder Spacing (ft)		
	8	10	12
NU900	95%	98%	100%
NU1100	91%	94%	97%
NU1350	87%	91%	93%
NU1600	84%	88%	91%
NU1800	82%	86%	89%
NU2000	80%	84%	87%

Ratio of deck PT stress in composite simple span bridge compared to deck PT stress in non-composite deck

Experimental investigation

With concrete girder (UNL)	With steel girder (GWU)
Push-off specimens	Push-off specimens
Large scale composite beam with unbonded longitudinal PT Tested for: - Strength for interface shear, flexure & vertical shear	Large scale composite beam with no longitudinal PT Tested for: - Fatigue: 6.8 million cycles - Strength for interface shear & flexure
Ribbed Panels & the haunch was unfilled	

Experimental investigation (Push-off Specimens)



Measured average horizontal shear capacity = **279.5 kips**

Predicted = 510.0 kips (strength of steel studs, LRFD Chapter 6)

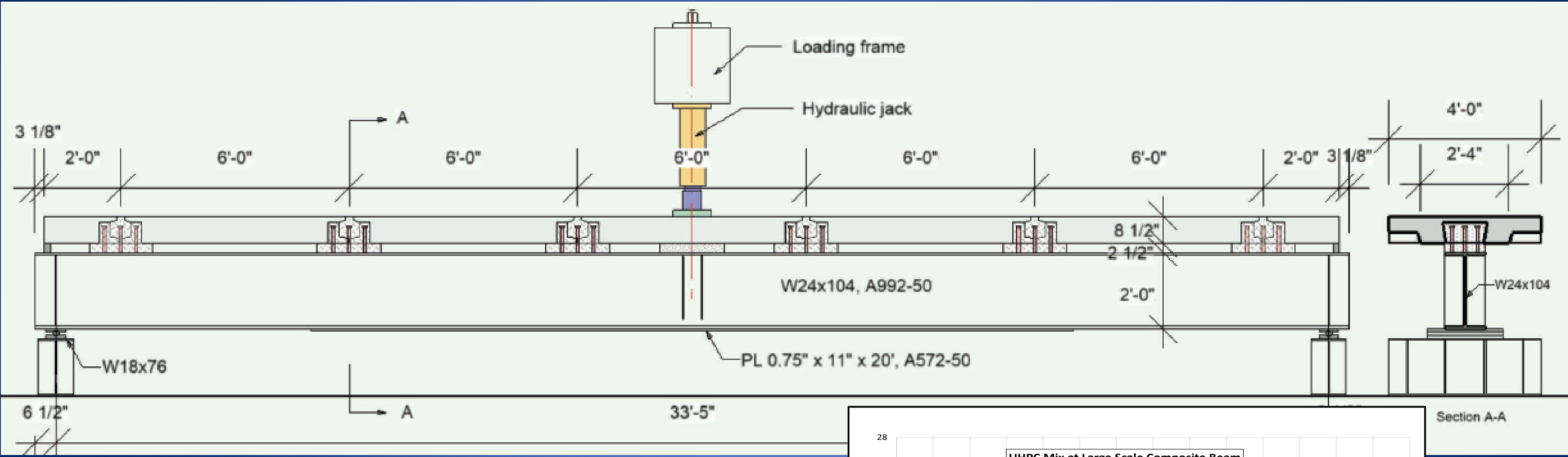
$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \le A_{sc} F_u \quad \text{(Eq. 6.10.10.4.3-1)}$$

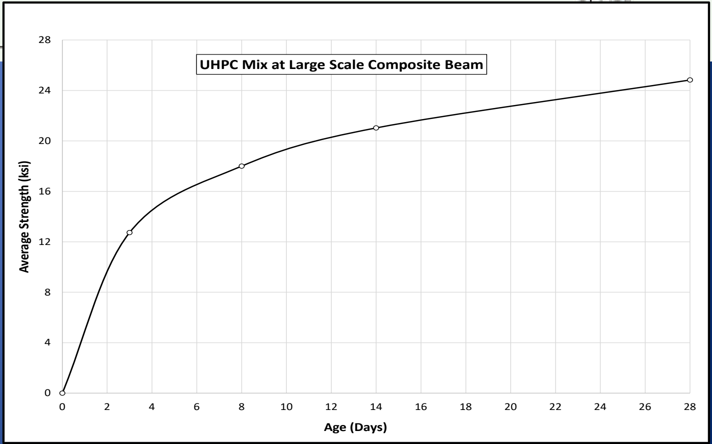
UHPC mix average strength = 14.2 ksi

No cracks were observed in the joints

The precast panel failed by bearing

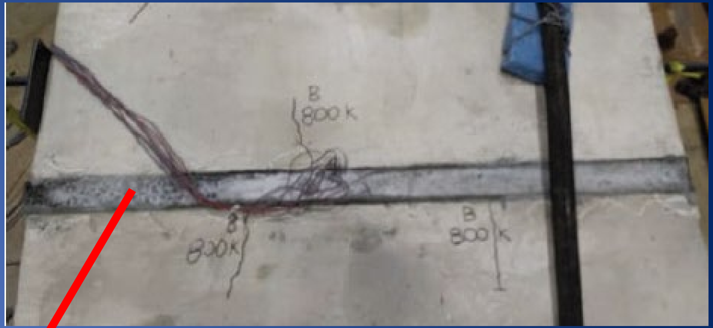
Experimental investigation with composite steel girder

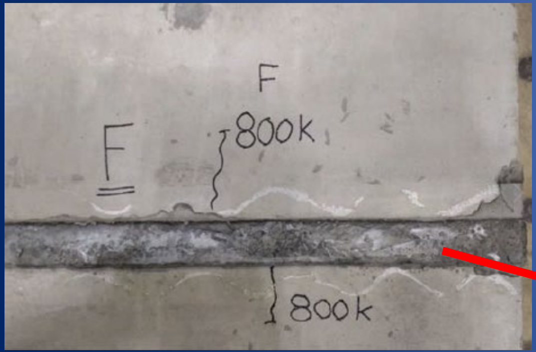




Experimental investigation with composite steel girder

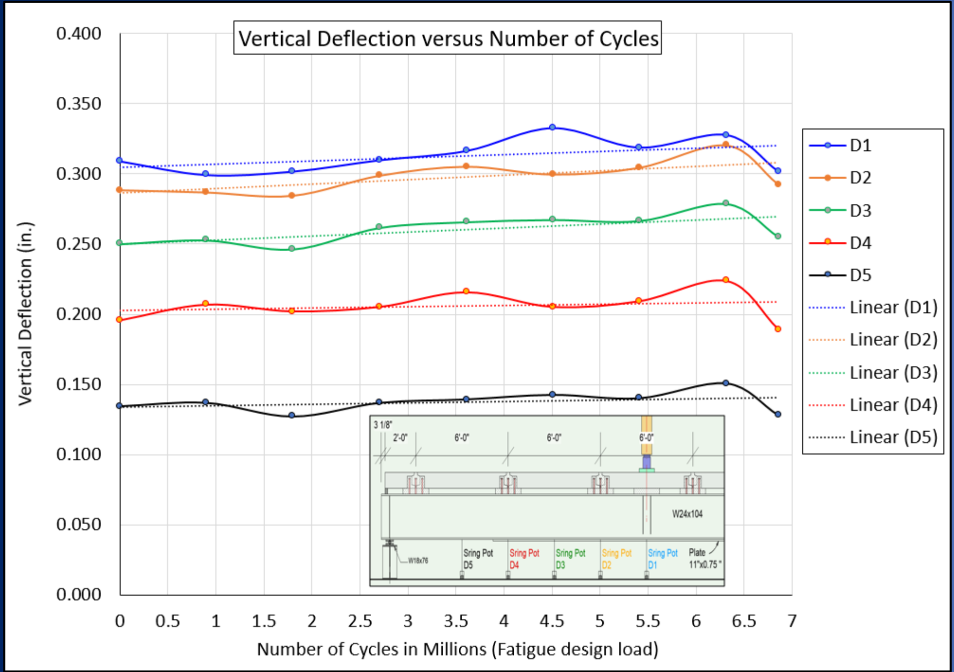
- Fatigue test (effect of 6.8 Million Cycles)
- Very satisfactory performance.
- Minor hair cracks (0.05 in. wide cracks) appeared in the unreinforced shear pocket cover.

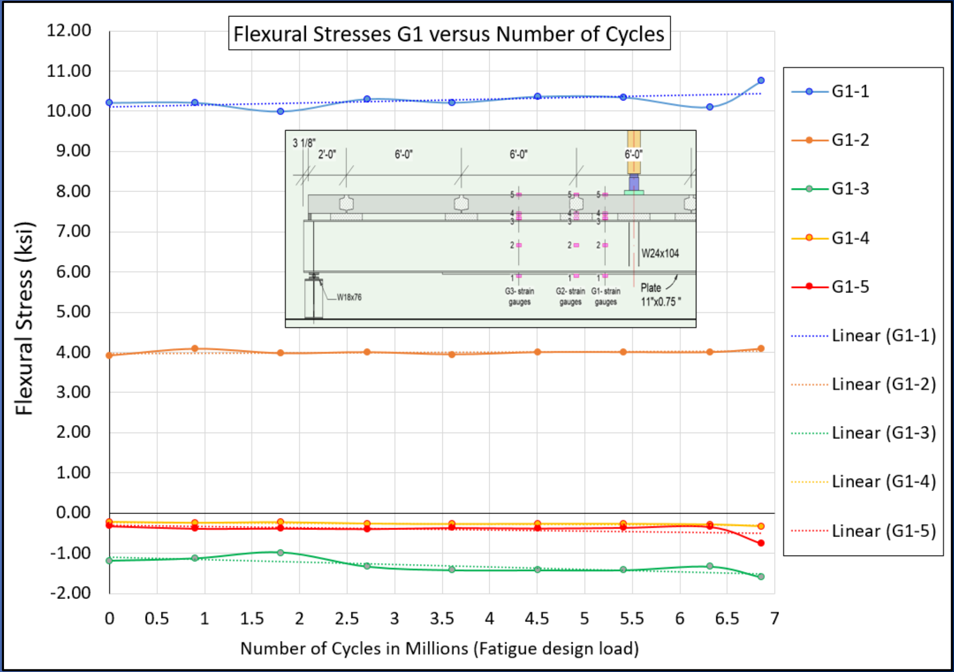




Experimental investigation with composite steel girder

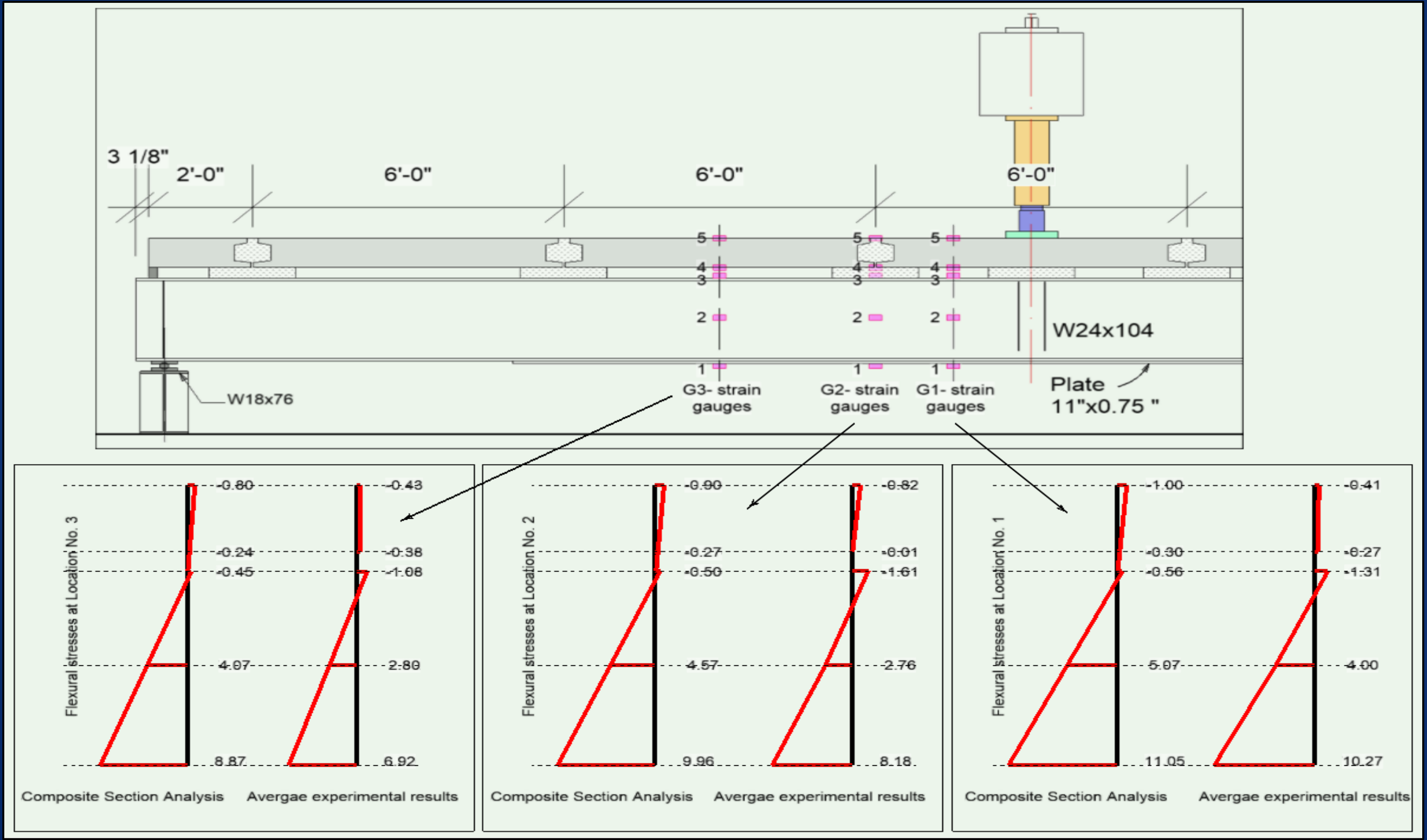
Fatigue test (effect of 6.8 Million Cycles)





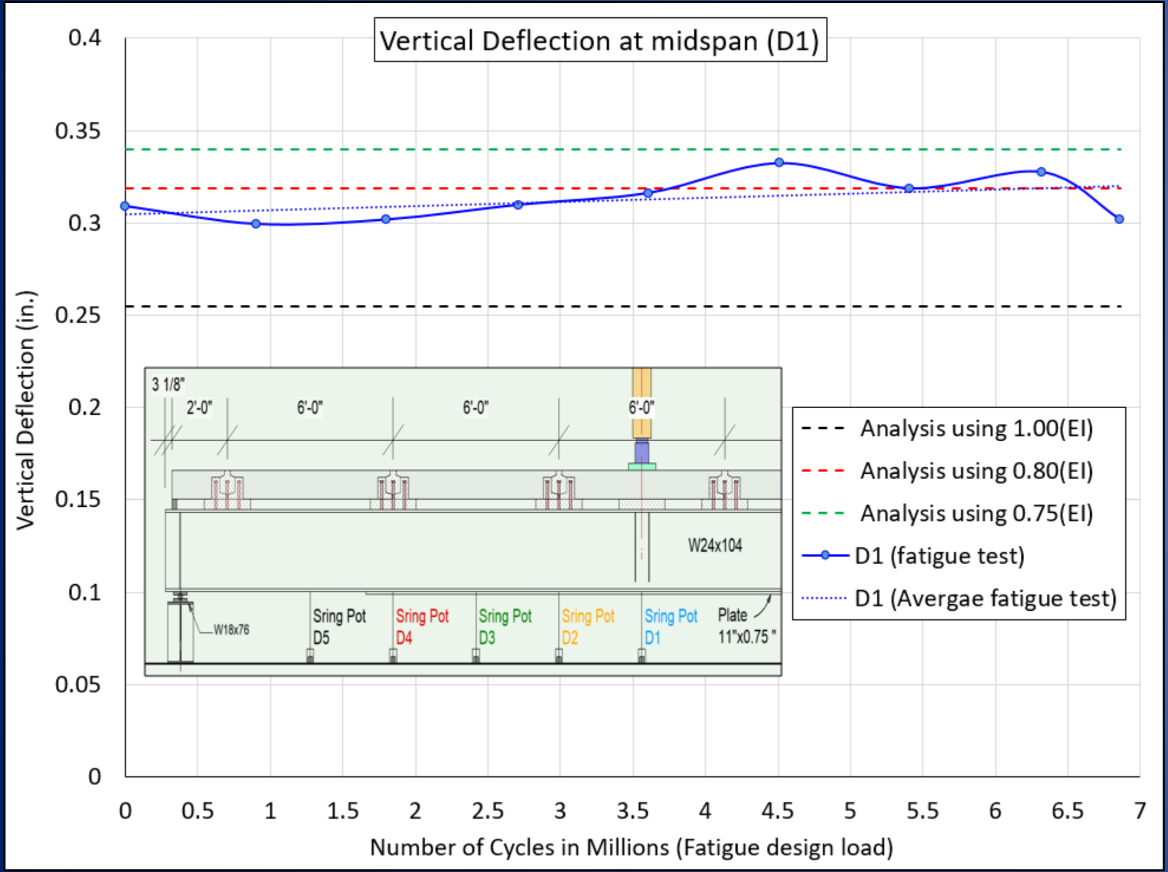
Experimental investigation with composite steel girder

Fatigue test: Full composite action (effect of 6.8 Million Cycles)



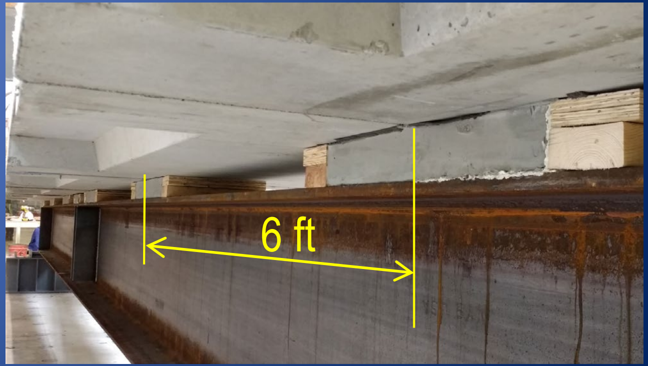
Experimental investigation with composite steel girder, unfilled haunch

Fatigue test (effect of 6.8 Million Cycles)



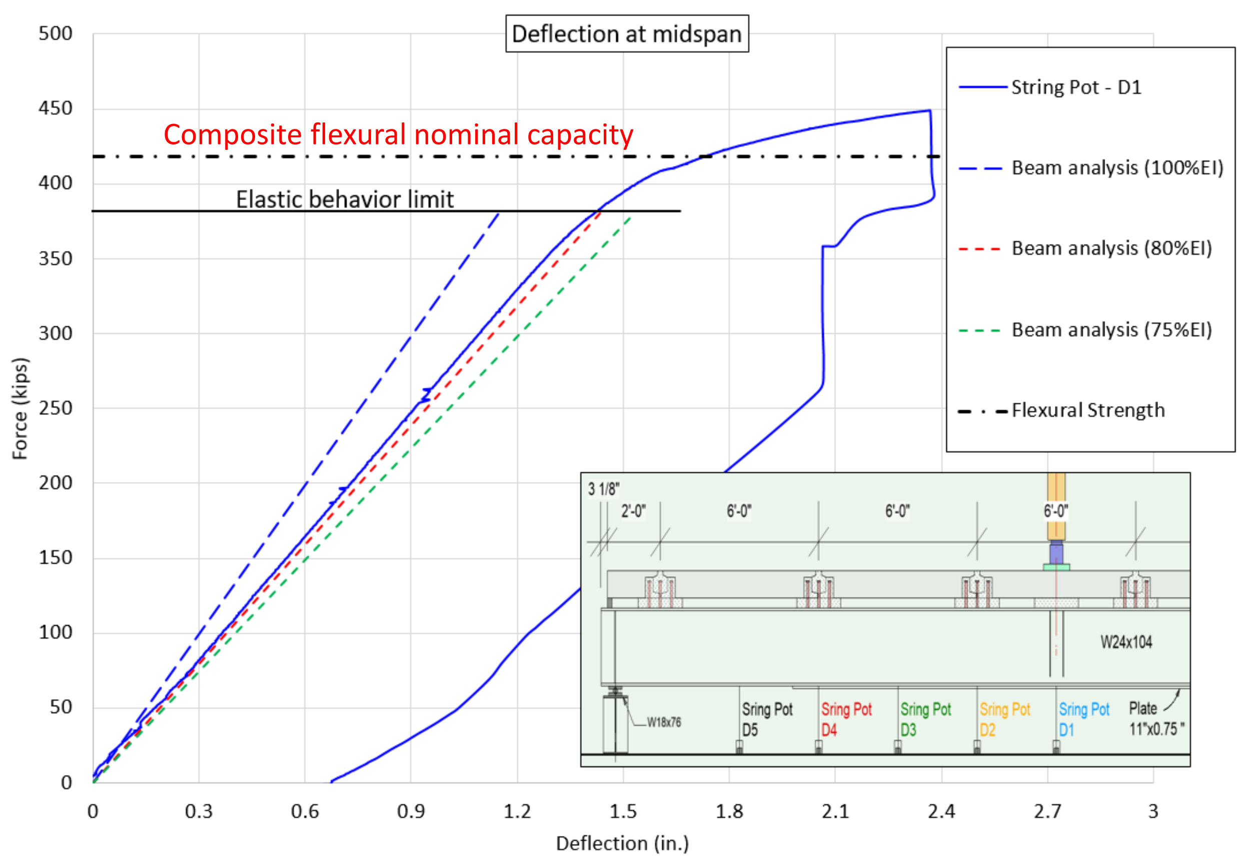
For deflection calculations due to service loads, a 25% reduction in the member stiffness is proposed.

This issue is recognized by the 13th edition of AISC Manual, where 25% reduction in the member stiffness is stated.



Experimental investigation with composite steel girder

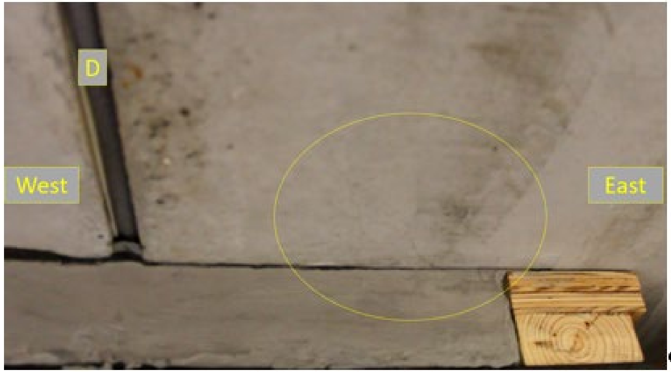
Strength test: Test was stopped at 450 kips (maximum safe capacity of the loading frame)



No interface shear failure was observed

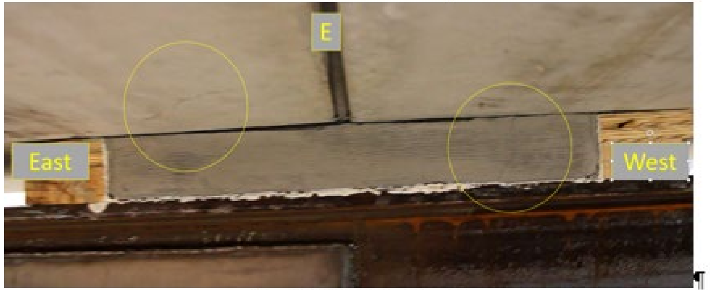
	Measured	Analysis
Flexural capacity (k-ft)	3,758	3,489
Corresponding Interface shear/joint (kips)	**453.8**	424.1
Corresponding load at midspan (kips)	450	418

Joint B (South side)



Joint D (South side)

Joint E (South side)



Joint E (North side)

Joint D (North side)

Joint C (North side)

Experimental investigation with steel girder

interface capacity per joint	Push-off specimens	Large scale composite beam	Average
Measured:	279.5 kips	453.8 kips	**366.6 kips**
Predicted: LRFD Article 6.10.10.4.3	510.0 kips Experimental/Predicted capacity = 72%		

A group reduction factor of 72% is proposed to the LRFD Equation 6.10.10.4.3-1:

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq R_g A_{sc} F_u$$

R_g = Group reduction factor = 0.72 for clusters with 2 rows of studs or more

Similar reduction is proposed by:

International codes, Issa at al. (2003), NCHRP 12-65 (2008) & Provines & Ocel (2014)

EQ. 6.10.10.4.3-1 (AASHTO LRFD Specs.)

$$Q_n = \underbrace{0.5 A_{sc} \sqrt{f'_c E_c}}_{\text{Resistance of the concrete surrounding the shear stud}} \leq \underbrace{A_{sc} F_u}_{\text{Resistance of a shear stud}}$$

Resistance of the concrete surrounding the shear stud

Resistance of a shear stud: Because the shear stud resistance is written in terms of its tensile strength, it implies that a shear stud behaves in pure tension rather than shear, which seems unlikely.

If $F_u = 70$ ksi, $A_{sc}F_u$ will control as long as $f'_c \geq 4.7$ ksi

When other members, like beams, are designed for shear, a 0.6 factor is multiplied by the member's tensile strength to determine its shear strength. No such factor is included.

The AASHTO provisions also do not provide any guidance for grouping clusters of studs close together to facilitate the use of precast deck panels. No guidance is provided to account for shear lag, which would likely be present when using clusters of studs at extended spacings.

International Codes

- The Eurocode 4[1] and Australian Standard[2] provisions both include a 0.8 factor with the stud resistance. This factor implies that shear studs fail somewhere between pure shear and pure tension.
- Although no explicit guidance is provided, the Eurocode 4[1], Australian Standard[2] and the Japanese Standards[3] mention that clustering studs is allowed if consideration is given to account for a greater local demand on the surrounding concrete due to clustering studs.

---

(1) Eurocode 4. 1994-2:2005. Design of Composite Steel and Concrete Structures, *Part 2-General Rules and Rules for Bridges*.

(2) Australian Standard. (2004). Bridge Design. Part 6: Steel and Composite Construction. Sydney, Australia.

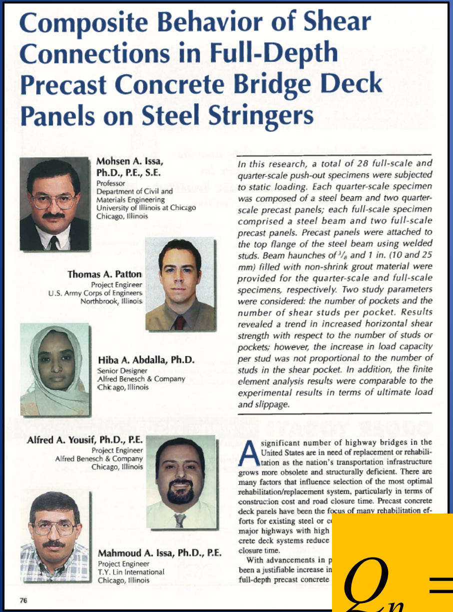
(3) Japan Society of Civil Engineers. (2009). Standard Specifications for Steel and Composite Structures. 1st Edition.

Published Research Results (3 selected samples)

Issa at al., PCI Journal, Sept./Oct. 2003

Badie & Tadros, NCHRP Report 584 (Project 12-65), 2008

Provines & Ocel, National ABC Conference, Miami, FL, 2014



Composite Behavior of Shear Connections in Full-Depth Precast Concrete Bridge Deck Panels on Steel Stringers

Mohsen A. Issa, Ph.D., P.E., S.E.
Professor
Department of Civil and Materials Engineering
University of Illinois at Chicago
Chicago, Illinois

Thomas A. Patton
Project Engineer
U.S. Army Corps of Engineers
Northbrook, Illinois

Hiba A. Abdalla, Ph.D.
Senior Designer
Alfred Benesch & Company
Chicago, Illinois

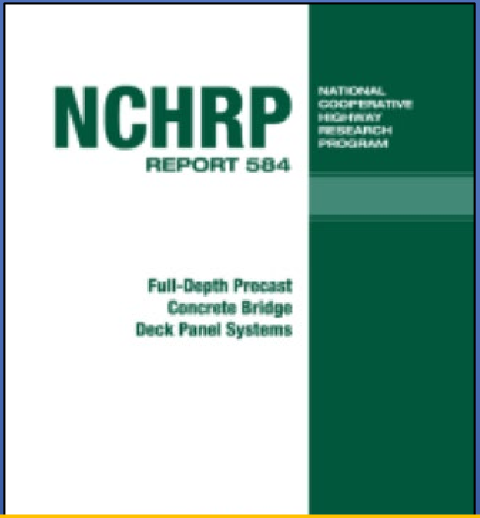
Alfred A. Yousif, Ph.D., P.E.
Project Engineer
Alfred Benesch & Company
Chicago, Illinois

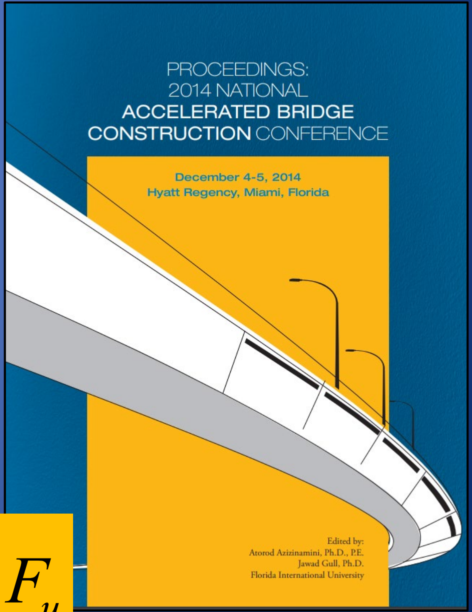
Mahmoud A. Issa, Ph.D., P.E.
Project Engineer
T.Y. Lin International
Chicago, Illinois

In this research, a total of 28 full-scale and quarter-scale push-out specimens were subjected to static loading. Each quarter-scale specimen was composed of a steel beam and two quarter-scale precast panels; each full-scale specimen comprised a steel beam and two full-scale precast panels. Precast panels were attached to the top flange of the steel beam using welded studs. Beam haunches of 3/8 and 1 in. (10 and 25 mm) filled with non-shrink grout material were provided for the quarter-scale and full-scale specimens, respectively. Two study parameters were considered: the number of pockets and the number of shear studs per pocket. Results revealed a trend in increased horizontal shear strength with respect to the number of studs or pockets; however, the increase in load capacity per stud was not proportional to the number of studs in the shear pocket. In addition, the finite element analysis results were comparable to the experimental results in terms of ultimate load and slippage.

A significant number of highway bridges in the United States are in need of replacement or rehabilitation as the nation's transportation infrastructure grows more obsolete and structurally deficient. There are many factors that influence selection of the most optimal rehabilitation/replacement system, particularly in terms of construction cost and road closure time. Precast concrete deck panels have been the focus of many rehabilitation efforts for existing steel or c… major highways with high… crete deck systems reduce… closure time.

With advancements in p… been a justifiable increase in… full-depth precast concrete…





$$Q_n = 0.5 A_{sc} \sqrt{f_c^{'} E_c} \leq A_{sc} F_u$$

- Goals of the Project
- Features of proposed deck panel system
- Analytical investigation
- Experimental investigation
- Design Guidelines
- Proposed changes to AASHTO LRFD Bridge Design Specifications

Design Guidelines

3.4 Design Guidelines

This project resulted in development of design and detailing methods for bridges built with either concrete girders or steel girders supporting full-depth precast deck panel systems with shear connection spacing of up to 6 ft. Details of the precast deck panel and the panel-to-panel and panel-to-girder connections are given in Chapter 2.

- Goals of the Project
- Features of proposed deck panel system
- Analytical investigation
- Experimental investigation
- Design Guidelines
- Proposed changes to AASHTO LRFD Bridge Design Specifications

Proposed changes to AASHTO LRFD Bridge Design Specifications

3.5 Proposed Changes to *AASHTO LRFD Bridge Design Specifications*

This section presents the proposed changes to the *AASHTO LRFD Bridge Design Specifications*, 8th edition (2017). Proposed changes are underlined.

The following list of proposed revisions to the *AASHTO LRFD Bridge Design Specifications* is currently in the process of being developed in a working agenda item format for easier presentation to the relevant Committees of the AASHTO Committee on Bridges and Structures.

WAI presented to T-10 (Concrete structures)
WAI presented to T-14 (Steel structures)

WAI presented to T-10 (Concrete structures)

Item #1: Change the first paragraph of Article 5.7.4.3 (on Interface Shear Resistance) as follows:

Except for specialized connections where shear friction theory may not be directly applicable, the nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c) \qquad (5.7.4.3\text{-}3)$$

Allow using capacity-rated connections

Item #2: Add the following paragraph to Article 5.7.4.3 (on Interface Shear Resistance) after the phrase "…taken as greater than 60.0 ksi." :

For specialized connections, such as that described in the C5.7.4.3, the connection capacity shall be based on more detailed analysis, backed up with full scale experimental verification.

Item #3: Add the following paragraph and figure at the end of Article C5.7.4.3 (on Interface Shear Resistance):

For the specialized connection system developed by Badie et al (2018), and shown in Figure C5.7.4.3-1, the ultimate shear strength of the connection has been determined through experiments and detailed nonlinear finite element analysis to be 310 kips. Such capacity can be directly used in design as V_{ni}, as long as the minimum capacities of the connection materials are as specified in the research, and shown in the figure, are used.

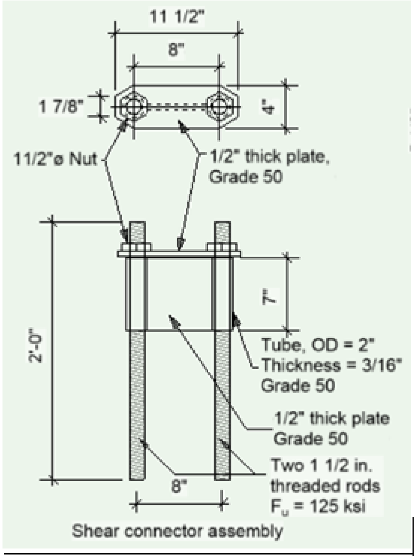


Figure C5.7.4.3-1 Special connection system developed by Badie et al. (2018)

Item #4: Modify the last paragraph of Article 5.7.4.5 as follows:

For beams or girders, the longitudinal center-to-center spacing of non-welded interface shear connectors shall not exceed ~~48.0~~ 72.0 in. ~~or the depth of the member, *h*~~. For cast-in-place box girders, the longitudinal center-to-center spacing of non-welded interface shear connectors shall not exceed 24.0 in.

Extend the connection spacing to 72 in.

Item #5: Modify the last paragraph of Article C5.7.4.5 as follows:

Recent research (Markowski et al., 2005; Tadros and Girgis, 2006; Badie and Tadros, 2008; Sullivan et al., 2011) has demonstrated that increasing interface shear connector spacing from 24.0 to 48.0 in. has resulted in no deficiency in composite action for the same resistance of shear connectors per foot, and girder and deck configurations. These research projects have independently demonstrated no vertical separation between the girder top and the deck under cyclic or ultimate loads. However, the research did not investigate relatively shallow members; hence, the additional limitation related to the member depth is provided. Research by Badie et al. (2018) has demonstrated that spacing between shear connectors can be extended to 72 inches. If the connector spacing is greater than the girder depth, then it is recommended that vertical shear reinforcement be determined assuming the total depth to be that of the girder alone rather than composite member depth.

WAI related to Section 9 of the AASHTO LRFD Specs

Item #1: Modify Article C9.7.5.3 (on *Longitudinally Post-Tensioned Precast Decks*) as follows:

C9.7.5.3

Decks made flexurally continuous by longitudinal post-tensioning are ~~the~~ more preferred solution over non-post-tensioned decks because they behave monolithically and are expected to require less maintenance on the long-term basis.

The post-tensioning ducts should be located at the center of the slab cross-section. Block-outs should be provided in the joints to permit the splicing of post-tensioning ducts.

Panels should be placed on the girders without mortar or adhes relative to the girders during prestressing. Panels can be placed dir with the help of shims of inorganic material or other leveling devic directly on the beams, the space therein should be grouted at the sa connector block-outs.

Promote the idea of applying the PT force on the composite slab-beam system

A variety of shear key formations has been used in the past. Recent prototype tests indicate that a "V" joint may be the easiest to form and to fill.

In the interest of accelerating deck construction, it is possible to consider fully grouting the transverse joints, shear pockets and haunch space between beam tops and deck soffit, all at the same time. Post-tensioning in this case would follow after the grout has gained adequate strength. This option is acceptable as long as global analysis is performed to determine the impact of post-tensioning on the composite member and the loss of prestress from the deck to the underlying structure. For simple span bridges, the deck is generally in longitudinal compression and a minimum average effective prestress of 0.35 ksi due to longitudinal post-tensioning is adequate to account for composite action effects, as shown by Badie et al. (2018).

WAI presented to T-14 (Steel structures)

Item #1: Change the last paragraph of Article 6.10.10.1.2 as follows:

The center-to-center pitch of shear connectors shall not exceed ~~48.0~~ 72 in. for mem
web depth greater than or equal to 24.0 in. For members with a web depth less tha
center-to-center pitch of shear connectors shall not exceed 24.0 in. The center-to-c
shear connectors shall also not be less than six stud diameters.

Extend the connection spacing to 72 in.

Item #2: Add the following paragraph to Article C6.10.10.1.2 (on Interface Shear Resistance) after the last paragraph

Recent research by Badie et al (2018) has demonstrated that clusters consisting of up to 9 studs, may be used to effectively increase the spacing between clusters. Spacing of up to 72 in. has been shown to be effective when UHPC with minimum compressive strength of 14 ksi is used. The corresponding minimum flexural strength has been 1.5 ksi.

Because of the large number of studs in each cluster at a wide cluster spacing, it has been found by Issa et al. (2003), Badie & Tadros (2008), Provines & Ocel (2014) and Badie et al. (2018) that a reduction factor should be used to estimate the strength of the cluster of studs in Eq. (6.10.10.4.3-1).

Item #3: Modify Equation 6.10.10.4.3-1 of Article 6.10.10.4.3 as follows:

$$Q_n = 0.5 A_{sc}\sqrt{f'_c E_c} \le R_g A_{sc} F_u \qquad (6.10.10.4.3\text{-}1)$$

Where:

R_g = Group reduction factor = 0.72 for clusters consisting of 2 or more rows of studs

Apply a reduction factor to the upper limit of Eq. 6.10.10.4.3-1

TRB Webinar: A Simplified Full Depth Precast Concrete Deck Panel System
Tuesday, April 30, 2019 1:00 PM - 2:30 PM EDT

Today's Speakers

- Pedro Silva, *The George Washington University*, silvap@gwu.edu
- Sameh Badie, *The George Washington University*, badies@gwu.edu
- George Morcous, *The University of Nebraska at Lincoln*, gmorcous2@unl.edu

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