Accident Investigation Report - Pedestrian Bridge Collapse at Florida International University

UniversityCity Prosperity Project
Miami, Florida

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Abbreviations and Terminology

AASHTO	American Association of State Highway Transportation Officials
ABC	Accelerated Bridge Construction
ASTM	American Society for Testing and Materials
BPA	Bolton Perez & Associates
CEI	Construction Engineering and Inspection
CJ	Construction joint
DCR	Demand-to-capacity ratio
DL	Dead load
FEA	Finite element analysis
FDOT	Florida Department of Transportation
FIU	Florida International University
HDPE	High-density polyethylene
IPR	Independent peer review
kip	Kilopound (1,000 pounds)
ksf	Kips per square foot
LL	Live load
LRFD	Load and Resistance Factor Design
MCM	Munilla Construction Management
MEP	Mechanical, electrical, and plumbing
MOT	Maintenance of traffic
NERO	Northeast Regional Office (FIGG)
psf	Pounds per square foot
psi	Pounds per square inch
PT	Post-Tensioning
PVC	Polyvinyl chloride
RFC	Released for Construction
RFI	Request for Information
RFP	Request for Proposal
SERO	Southeast Regional Office (FIGG)
SPMT	Self-propelled Modular Transporter
TSG	The Structural Group

Executive Summary

The Main Span of the FIU/Sweetwater Pedestrian Bridge ("Bridge") collapsed at 1:47 p.m. ET on March 15, 2018, tragically killing six people, including five vehicle passengers and one construction worker, and injuring several others. The collapse occurred just as the Post-Tensioning ("PT") contractor, Structural Technologies, had finished restressing the PT bars in Diagonal Truss Member 11 close to the north end of the Main Span. Restressing of these bars was not part of the original stressing sequence set forth in the Designer's (FIGG Bridge Engineers ("FIGG")) plans, but was prescribed by FIGG in response to cracking at the base of Member 11 and the North Diaphragm, as discovered and reported to FIGG by General Contractor Munilla Construction Management ("MCM") and the Construction Engineering and Inspection ("CEI") firm Bolton Perez & Associates ("BPA") in February and March 2018. In the days and weeks prior to the collapse, FIGG reviewed the information sent by MCM and BPA, conducted structural analyses, and repeatedly assured all involved parties that there were no safety concerns associated with the cracking. Unfortunately, this was not the case.

Figure 1. Frame from dashboard video of Main Span during collapse, showing that the failure originated near the north end of the Main Span.[1]

(a) Bottom of Member 11 *(b) Corner of North Diaphragm*

Figure 2. Cracks at north end of Main Span (Node 11/12) on March 14, 2018, showing examples of cracking discovered in the Main Span prior to the collapse.[2]

[1] National Transportation Safety Board, Bridge Factors Group Chairman's Factual Report ("NTSB Bridge Factors Report"), at Video Attachment labeled "Bridge Factors - Original Video from Driver Travelling Eastbound on SW 8th Street.MOV."

[2] NTSB Bridge Factors Report, Photos 90 and 97.

Since the time of the collapse, the National Transportation Safety Board ("NTSB") has been actively investigating the factual circumstances surrounding the collapse in an effort to determine the probable cause for the Bridge's failure ("NTSB Investigation"). MCM has been an active participant in the NTSB Investigation and has provided, and continues to provide, information to the NTSB to assist in this effort. This Report is MCM's final submission in this regard, as requested by the NTSB and as provided for in the Code of Federal Regulations at 49 C.F.R. § 831.14.

In preparing this Report, MCM[3] has reviewed extensive factual information (including, but not limited to, witness interviews, photographs, and the original Bridge design plans and calculations) and has performed numerous calculations and analyses in order to evaluate all pertinent design and construction aspects for potential contributions to the cause of collapse. Based on this detailed analysis, MCM concludes that the collapse occurred due to the following:

- There were serious errors and omissions in FIGG's original design calculations and drawings for the Bridge, including a significant underestimation of critical connection forces;
- Louis Berger did not conduct an adequate Independent Peer Review ("IPR") of FIGG's design and calculations, and did not independently detect FIGG's design calculation and drawing errors;
- FDOT, who employed licensed professional engineers on the Project, did not recognize multiple design errors and omissions contained within FIGG's calculations and drawings;
- FIGG lacked proper understanding of the causes for the observed cracking during construction in February and March 2018, selected an incorrect repair of the observed cracking, and failed to appreciate and warn of the dangers posed by both the cracking and the selected repair; and
- FIGG's repeated assurances (both written and verbal) that there were no safety concerns with the Bridge suspended over the road, dictated that FDOT and FIU not seek closure of the roadway to live traffic during re-stressing operations pursuant to governing traffic control standards and FDOT protocols.

The factual bases for these conclusions are outlined in detail throughout this Report and are also summarized in more detail in the final "Conclusions" set forth in Section 8 below.

[3] MCM, as a General Contractor, is not expected nor allowed to perform engineering design; in order to analyze any aspect of the Bridge design, it is required to retain appropriately licensed expert professionals.

1 Introduction

MCM has closely reviewed the information submitted by the various parties during the NTSB Investigation and has correspondingly performed detailed analyses of the design, construction, transport, repair, and tragic collapse of the Bridge. Accordingly, this Report contains MCM's factual findings, probable causes, and safety recommendations pursuant to 49 C.F.R. § 831.14. MCM hopes this report may be helpful to the NTSB investigators.

1.1 Project Description

In 2015, Florida International University ("FIU") awarded a $9.3 million dollar contract to MCM and FIGG for the construction of the FIU – UniversityCity Prosperity Project (the "Project").[4] The Project was primarily intended to provide a safer pedestrian crossing between FIU's campus and the adjacent City of Sweetwater, where there is substantial off-campus student housing in an area with an unfortunate history of car/pedestrian accidents. The Project was also intended, however, to offer an inviting outdoor public space for FIU students and the general public, incorporating a signature structure that would serve as an attraction in and of itself. The 174-foot walkway of the Bridge was planned to have spanned over the busy Tamiami Trail (SW 8th Street) at SW 109th Avenue, as shown in Figure 3 below.

1.1.1 The Bridge

The primary portion of the Bridge that crossed the Tamiami Trail, referred to herein as the "Main Span," was built using an engineering method known as Accelerated Bridge Construction ("ABC"). In this particular context, ABC involved staging the construction of the Main Span in a designated yard on the FIU campus just adjacent to the final Bridge location, and then transporting and placing the Main Span over the roadway onto permanent support structures, as shown in Figure 4. The purpose of ABC is to limit the disruption of vehicular traffic to the greatest practical extent during construction, which was desirable to FIU and the Florida Department of Transportation ("FDOT") given the high traffic area in which the Bridge was to be constructed.

The other portion of the Bridge, the "Back Span," which would have traversed the canal just north of the Main Span over Tamiami Trail, was intended to be built using conventional bridge-building methods after the Main Span was completed. A diagram of the planned Bridge appears below in Figure 5, and a rendering of the initial structure prepared by FIGG for the original FIU proposal in 2015 appears in Figure 6.

[4] The Project was funded largely by two federal programs: The Transportation Investment Generating Economic Recovery (TIGER) fund and the Transportation Alternative Program (TAP). NTSB Bridge Factors Report, at pp. 5-8. Other financial contributors included FIU, the City of Sweetwater, and FDOT. *Id*. at pp. 9-21.

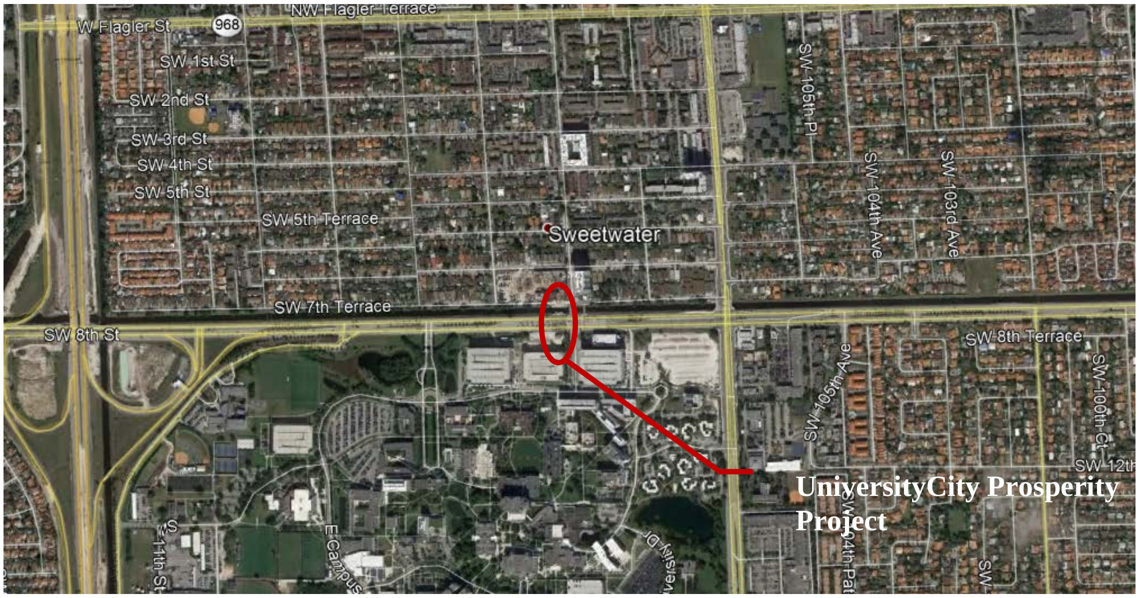

Figure 3. Location of the UniversityCity Prosperity Project.[5]

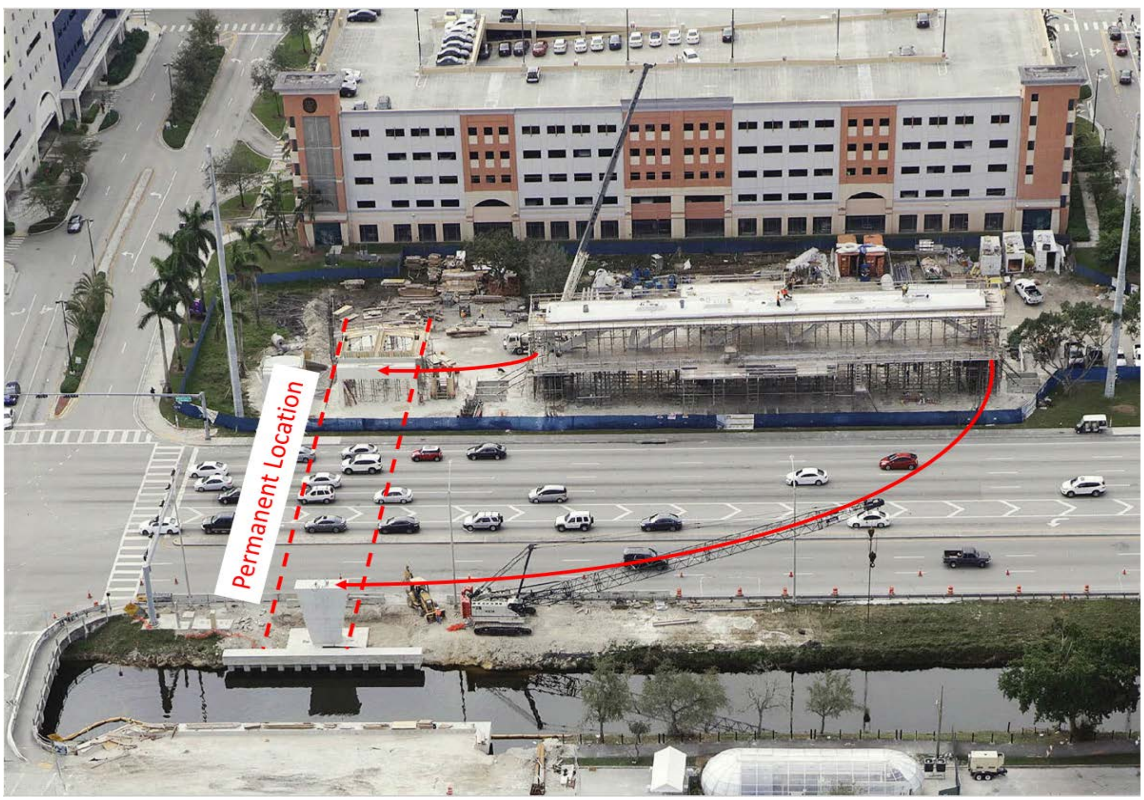

Figure 4. Illustration of Accelerated Bridge Construction (ABC) method (looking south).[6]

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5 Image from Google Earth Pro.

6 MCM Photo Submission to NTSB, MCM_NTSB_OSHA-005115.

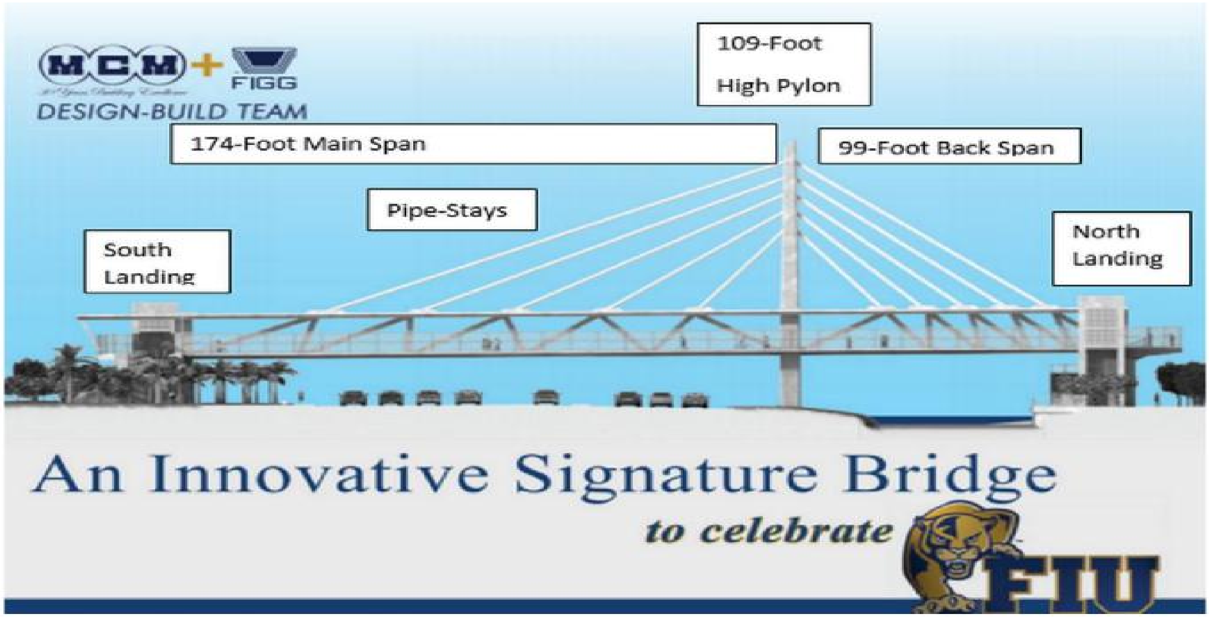

Figure 5. Diagram of planned Bridge.[7]

Figure 6. Rendering of the proposed Bridge, facing southeast.[8]

[7] Proposal for the BT-904 | FIU UniversityCity Prosperity Project, MCM and FIGG Design-Build Team, September 30, 2015, ("Design-Build Proposal").

[8] *Id*.

The Bridge went under contract in early 2016[9] and was scheduled to be completed by early 2019. The contract for the construction of the Bridge called for a "Design-Build" project, pursuant to which MCM, as the General Contractor, contracted with FIGG, a nationally-acclaimed, award-winning engineering firm based out of Tallahassee, Florida, to design and oversee construction planning for the Bridge. FIGG is well known for designing other iconic bridges, such as the Leonard P. Zakim Bridge in Boston and the Sunshine Skyway Bridge in Tampa Bay, and thus MCM and FIU had full confidence in FIGG's abilities on this Project.

1.1.2 Parties Involved

The formal legal relationship between MCM and FIGG for the Bridge is reflected in the Design-Build subcontract.[10] Consistent with a typical Design-Build project,[11] FIU was the Project "Owner," FIGG was the "Designer," and MCM was the "General Contractor." As such, these parties had the following primary responsibilities:

- ***FIU*** **–** As the Project Owner, FIU issued a Request for Proposals ("RFP") based on an initial design and technical criteria developed by its consultant, TY Lin International, took proposals, and ultimately hired a team (MCM and FIGG) to complete the Project. While FIU provided conceptual design drawings for the Bridge and coordinated general civil design items, FIGG, as the Designer, was to provide the final design plans, construction drawings, and specifications for the Bridge. In addition to providing initial conceptual designs, FIU was also consistently informed of the Project status, provided Project oversight and review, and authorized Project change orders and expenditures.[12]

- ***MCM*** **–** Contracted directly with FIU to act as the General Contractor, or "Design-Build Firm," for the construction of the Bridge. As such, MCM was tasked with procuring subcontracting parties to carry out the construction, installation, inspection, and completion of the Bridge.[13]

- ***FIGG*** **–** As the "Designer" and Engineer of Record ("EOR"), FIGG was the Design-Build contractor of MCM, and was thus responsible for the overall engineering of the Bridge, including design, testing, drawings, and calculations, and for providing monitoring and oversight for certain construction activities related to implementation of its engineering designs and plans. FIGG was ultimately responsible for providing MCM with all information necessary to construct the Project as a complete and fully operational system in accordance with the requirements and contract documents provided by FIU.[14]

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[9] NTSB Bridge Factors Report, Attachment 15.

[10] NTSB Bridge Factors Report, Attachment 18.

[11] According to the Design-Build Institute of America (DBIA), the design-build form of project delivery is a system of contracting whereby one entity performs both architectural/engineering and construction under one single contract. This is different from the Design-Bid-Build method, in which the project owner first hires an engineer/architect to design the facility and then hires a contractor separately to bid on and construct the work.

[12] *E.g.*, NTSB Bridge Factors Report, Attachments 11, 12, 15, 16, and 17.

[13] *E.g.*, NTSB Bridge Factors Report, Attachment 15.

[14] *E.g.*, NTSB Bridge Factors Report, Attachments 18.

Other parties were engaged in a variety of capacities on the Project. Aside from FIU, MCM, and FIGG, the primary participants on the Project were as follows:

- ***Barnhart Crane and Rigging, Co. ("Barnhart")*** – Responsible for transporting the Main Span from the temporary staging area to its final location on the permanent piers/foundation. Barnhart worked with FIGG to develop the engineering plans for the transport and hired other subcontractors related to the same.[15]

- ***BPA*** – As a CEI firm, BPA was hired by FIU as the CEI to monitor the progress of the Project and ensure compliance with the master Design-Build contract and related standards. BPA supervised, inspected, and otherwise ensured that MCM, FIGG, and other subcontractors complied with controlling safety standards, governing regulations, contractual duties/obligations, and industry safety standards.[16]

- ***Bridge Diagnostics, Inc. ("BDI")*** – Hired by Barnhart to perform "structural monitoring" during, transport and installation of the Main Span, largely by providing instrumentation for monitoring the movement according to Barnhart's plans.[17]

- ***The Corradino Group, Inc. ("Corradino")*** – Hired by BPA to perform engineering, inspection and management services, and consulting on the Project. Essentially, Corradino served as a sub-consultant and "over-the-shoulder reviewer" for BPA.[18]

- ***FDOT*** – Reviewed and approved the Bridge design and provided oversight for the Project in relation to maintenance of traffic, right-of-way permitting and monitoring, and overall impact of the Project on the public. FDOT performed inspections, authorized certain plans and permits, coordinated construction activities with FIU and the contractors, and was informed of and provided input into major construction efforts.[19]

- ***Louis Berger, U.S., Inc./The Louis Berger Group, Inc. ("Louis Berger")*** – As a construction engineering firm that provides, among other things, civil engineering, construction management, and structural engineering services, FIGG hired Louis Berger to provide Independent Peer Review ("IPR") of FIGG's design work.[20]

- ***RC Group, LLC ("RC Group")*** – Responsible for monitoring and inspecting shoring operations on the Main Span.[21]

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15 *E.g.*, NTSB Bridge Factors Report, Attachment 63, at FCA-A5, Barnhart Crane & Rigging Scope of Work, and FCA-A6(30), Bridge Movement Plan.

16 *E.g.*, NTSB Bridge Factors Report, Attachments 14 and 17.

17 *E.g.*, NTSB Bridge Factors Report, at Figure 3.

18 *Id*.

19 *E.g.*, NTSB Bridge Factors Report, Attachments 8, 9, and 10.

20 *E.g.*, NTSB Bridge Factors Report, Attachments 19 and 63, at FCA 6.5-7 (pp. 384-385 of pdf), and FCA 6.5-30 (p. 451 of pdf).

21 *E.g.*, NTSB Bridge Factors Report, Attachment 66, MCM-5 and MCM-6.

- ***Structural Technologies*** **–** Performed PT activities on the Bridge throughout the Project, including on Truss Member 11 on the day of the collapse.[22]

- ***The Structural Group of South Florida, Inc. ("TSG")*** **–** As the primary construction subcontractor on the Project, TSG provided labor and supervision for construction components of the Project, including construction of Foundations, the Deck, the Truss Web, the Canopy, concrete slabs, formwork, and falsework/shoring.[23]

1.1.3 Construction Timeline

After FIU secured the contract with MCM, and MCM subcontracted with FIGG, engineering and construction planning, geotechnical investigation, site preparation, permitting, and other preparatory work took place throughout 2016 and 2017, with MCM, FIGG, and FIU hiring subcontractors to perform the necessary work. Construction of the Main Span commenced in the staging area on or about March 20, 2017, when contractors started building the formwork and falsework/shoring.[24] Falsework was installed between May 16 and June 12, 2017; formwork for the Deck and Web Truss Members was installed between June 12 and June 23, 2017; steel, PT ducts, conduits, and pipe were installed between June 26 and August 17, 2017; and concrete was then poured for the Main Span Deck on October 17 and 18, 2017, for the Truss on November 6, 2017, and for the Canopy on December 14, 2017.[25]

After concrete was placed and finished, the Main Span was prepared for transport to and installation at its final location. Among other things, Main Span Deck PT tendons were stressed between January 15 and February 1, 2018; Deck PT ducts were grouted between January 30 and February 5, 2018; Web Truss PT bars were stressed between January 23 and February 7, 2018; Web Truss PT ducts were grouted between February 7 and February 16, 2018; Canopy PT tendons were stressed between January 15 and February 21, 2018; and Canopy PT ducts were grouted between February 21 and March 2, 2018. Formwork and falsework/shoring removal was completed on the Main Span on February 24, 2018, and bearing pads were then installed at the South Bent and Pylon between March 2 and March 5, 2018. The Main Span was moved into place and installed on March 10, 2018. Additional key project milestones are shown in Table 1.

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[22] *E.g.*, NTSB Bridge Factors Report, at Figure 3.

[23] *Id*.

[24] Falsework is a temporary framework used to support a structure during its construction, also known as shoring in concrete work. Formwork is temporary molds for casting concrete. When concrete hardens, the formwork and falsework/shoring are removed. Reinforcing steel/PT ducts are placed in formwork prior to pouring concrete.

[25] A concrete pour was previously attempted on August 31, 2017 but was redone due to problems with Cemex's batch plant that prevented a continuous pour. The concrete was wholly removed, and the pour was redone in its entirety to preclude any residual problems. Demolition and removal took nearly a month, from August 31, 2017 to September 22, 2017. Hurricane Irma also occurred during this time period, causing delays.

Table 1. Key Project Milestone Dates

Milestone	Proposed Completion[26]	Actual Completion[27]
FIU-MCM Contract Execution	Jan 14, 2016	Jan 14, 2016
Notice to Proceed (NTP)	Jan 14, 2016	Jan 21, 2016
Issue RFC Foundation Plans	Jul 12, 2016	Mar 28, 2017
Issue RFC Substructure Plans	Aug 23, 2016	Mar 28, 2017
Issue RFC Superstructure Plans	Nov 10, 2016	Jun 16, 2017
S. Landing Foundations	Feb 6, 2017	May 8, 2017
S. Landing Pier and Elevator Shaft	Mar 20, 2017	Nov 6, 2017 (Pier)
Pylon Foundations	Mar 29, 2017	Dec 26, 2017
Pylon V-Pier	Apr 26, 2017	Jan 15, 2018
N. Landing Foundations	May 3, 2017	
N. Landing Pier and Elevator Shaft	Jun 15, 2017	
Main Span Superstructure Fabrication	Jul 19, 2017	Feb 24, 2018
Move Main Span to Final Position	Jul 26, 2017	Mar 10, 2018
Back Span	Oct 13, 2017	
Pylon and Stays	Dec 7, 2017	
Punch List and Project Completion	Jul 17, 2018	

1.1.4 Cracking Timeline

As discussed further in Section 2 below, between February 6 and March 15, 2018, BPA, Corradino, and MCM noticed and reported three separate instances of cracking around several Diagonal members of the Main Spain (collectively referred to as "Truss") and the North Diaphragm. BPA drafted several reports related to the same, which MCM promptly delivered to FIGG seeking instructions on how to proceed. In response to BPA's and MCM's reports, photographs, and inquiries, FIGG informed BPA, MCM, and others that the observed cracking did not present a design or safety concern. FIGG also conducted internal analyses, made a site visit to the Bridge the morning of March 15, 2018, and led a meeting to discuss its analyses regarding the cracking and to further assure FIU, MCM, BPA, and FDOT that the cracks posed no concern.

[26] *See* Design-Build Proposal.

[27] *See* 2018-01-FIU-UCPP-Full Schedule DD31JAN18.pdf, unless otherwise noted; NTSB Bridge Factors Report, Attachments 15 and 18.

1.2 Incident Description

The Main Span failed on March 15, 2018, at approximately 1:47 p.m. ET, despite the parties having followed FIGG's instructions to conduct remedial re-tensioning of Truss Member 11. Within minutes after the Structural Technologies crew completed re-tensioning of the PT bars in Truss Member 11—in accordance with FIGG's instructions—the Main Span collapsed.

Tragically, six people succumbed to fatal injuries, and a number of others were injured. Immediately after the collapse, federal, state, and local emergency response, law enforcement, and investigatory teams were on site. NTSB took control of the scene, with federal and local law enforcement providing rescue efforts and logistical support to render emergency aid, preserve evidence, relocate the fallen Main Span, and open the roadway. MCM has been cooperating with and actively assisting the NTSB Investigation since this time.

Dashboard video camera footage of the collapse is publicly available. One such video is taken from a moving vehicle traveling east toward the Bridge. The north end of the Main Span is situated on the left side of the frame. It is apparent from the video that the failure precipitating the collapse initiated on the north end of the Main Span (Figure 7). Figure 8 shows the aerial views of the collapsed Main Span.

Figure 7. Frame from dashboard video of Bridge collapse.[28]

[28] *See* FN 1.

Figure 8. Aerial photo of collapsed Main Span and rescue efforts.[29]

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[29] NTSB Bridge Factors Report, Photo 107.

2 Design

In 2014 and 2015, through its consultant TY Lin International, FIU developed the "Design Criteria"[30] for its original Request for Proposals ("RFP") on the Project.[31] This Design Criteria included the architectural vision for the Bridge; the engineering and structural requirements; FDOT, American Association of State Highway and Transportation Officials ("AASHTO"), Federal Highway Administration ("FHWA"), and other standards to which the design must conform; geometric layout and clearance requirements of the Bridge; and design loading (including construction loads), deflection limits, and material requirements to be followed during Bridge construction. All proposals by prospective Design-Build teams were required to adhere to all requirements set forth in the Design Criteria.

At the time the MCM and FIGG Design-Build team submitted its technical proposal to FIU in 2015, FIGG had fully developed the conceptual design of the Bridge and its plans were already at about the 30% level.[32] FIU hired MCM, who had contracted with FIGG, to proceed in accordance with the following design plans set forth in this Section.[33]

2.1 Applicable Design and Construction Codes

FIU's Design Criteria specified the applicable codes and specifications that would govern FIGG's Bridge design. In accordance with the Design Criteria, the General Notes on FIGG's RFC design Sheets B-2 and B-3 specified that, among others codes and specifications, the Seventh Edition of AASHTO LRFD Bridge Design Specifications (with the 2015 interim revisions), the 2009 AASHTO Guide Specification for the Design of Pedestrian Bridges, the 2015 FDOT Structures Design Manual, the 2015 FDOT Standard Specifications for Road and Bridge Construction, and the 2004 AASHTO LRFD Bridge Construction Specifications (with 2006 interim revisions) governed the design and construction of the Bridge.[34]

MCM's current evaluation of both the design and construction elements of the Project, as presented in subsequent sections of this Report, was performed against the requirements of these codes and specifications, which define the accepted industry practice for bridge design, including, but not limited to, nominal load, loads and resistance factors that define an inherent factor of safety against failure, and design equations used to determine section sizes and material strength requirements.

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[30] NTSB Bridge Factors Report, Attachment 12.
[31] NTSB Bridge Factors Report, Attachment 11.
[32] *See* Design-Build Proposal.
[33] NTSB Bridge Factors Report, Attachments 15 and 18.
[34] NTSB Bridge Factors Report, Attachment 38; NTSB Accellion – FIGG Files – UCPP_RFC_Plans_Foundation Bates.pdf, UCPP_RFC_Plans_Substructure Bates.pdf, UCPP_RFC_Plans_Superstructure Bates.pdf.

2.2 Design Concept

2.2.1 Concrete Truss System

FIGG designed the Bridge to incorporate a two-span concrete Truss[35] system, accompanied by a perpendicular and asymmetrically-located tapered "Pylon" with steel tube stays on top of the Bridge, as shown in Figure 9. Although the structure of the Bridge appears similar to a typical cable-stayed bridge, where the stay cables support the weight of the deck, the Bridge was not designed to behave in that manner. Rather, the Bridge's concrete Truss was intended to be fully capable of carrying the Deck, Canopy, and all imposed loads, whereas the simulated stay cables only served to augment the iconic appearance and mitigate structural vibrations.

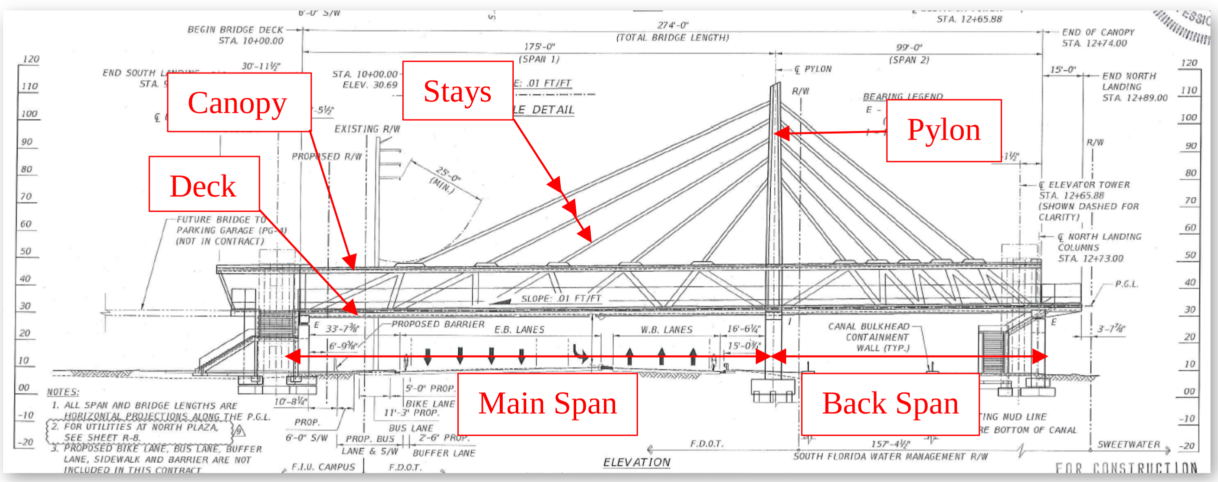

Figure 9. Bridge elevation from RFC General Plan and Elevation – Sheet B-4.[36]

As shown in Figure 10, the bottom flange of the Truss would function as the walking surface ("Deck"), and the top flange was to serve as the "Canopy" providing cover to pedestrians. The Diagonal Truss (Web) Members were in the center of the cross section, forming an approximate I-shape when combined with the top and bottom flanges (Canopy and Deck). Along with the Diaphragms on either end, these Truss Members would ultimately carry the primary weight of the Bridge. Together, the Truss, Canopy, and Deck are referred to as the "Superstructure." The support system below the Superstructure, which consisted of piers, pylons, and ground foundations, is referred to as "Substructures." The loads from the Superstructure were transferred to the Substructures at the north and south "Piers" and the Pylon just south of the canal.

[35] A truss is a combination of diagonal and horizontal structural elements connected together in such a way to create a rigid frame.

[36] NTSB Bridge Factors Report, Attachment 39.

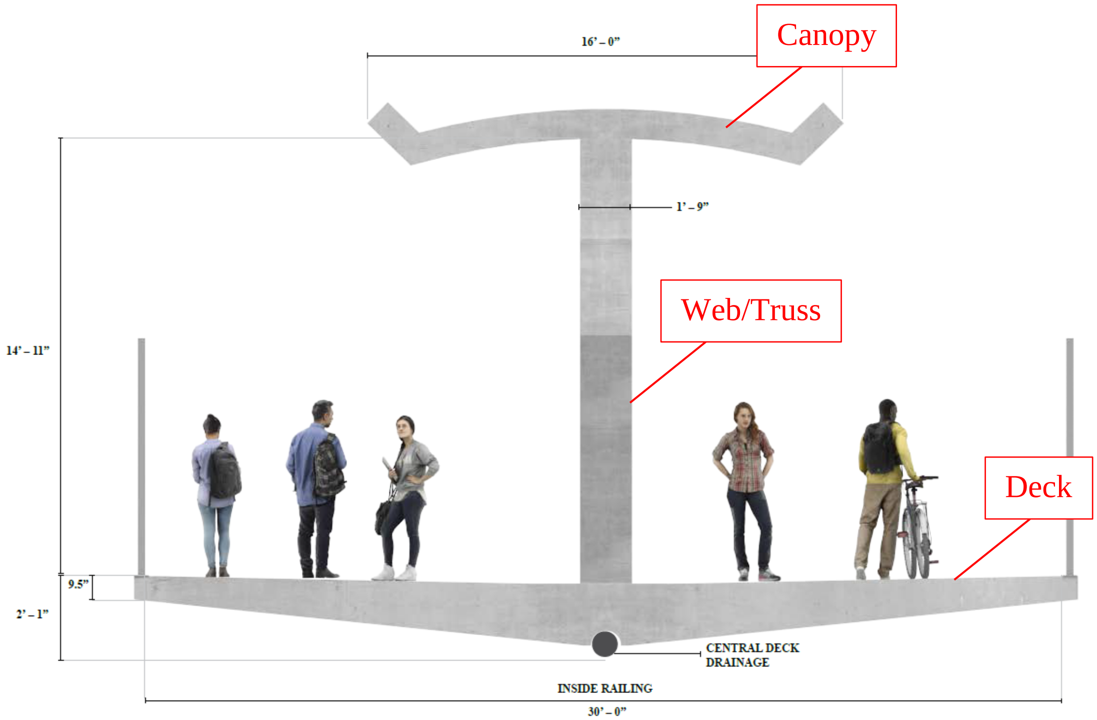

Figure 10. Proposed cross section of Bridge.[37]

In a truss system, diagonal web members can carry either compression (shortening) or tension (stretching) forces, depending on their configuration. Concrete is strong in compression and is a good choice for diagonals that experience only compression. But concrete is not ideal for web members that must also carry tension, as it will crack under relatively small tensile forces.

Therefore, because some of the concrete Diagonal Truss Members on the Bridge carried tension under self-weight, these members were designed to be "prestressed" to keep the concrete in compression even if the Diagonal must carry tension. The concept of prestressing concrete is somewhat akin to using rubber bands to hold items together, where the rubber band is stretched (tensioned) and the items being held together are pressed together (compressed). In prestressed concrete, embedded steel tendons or bars are tightened such that the concrete is pre-compressed. When a tensile force is applied to a prestressed concrete diagonal, sufficient precompression will prevent the concrete from experiencing tension and thus avoid cracking.

FIGG designed the Superstructure prestressing on the Bridge using PT bars/rods—long, high-strength solid round steel bars—passing through the Truss Members/Diagonals. However, several issues surrounding FIGG's design calculations of these post-tensioned elements are central to the collapse of the Main Span and will be discussed in detail in Section 6 below. Figure 11 shows the numbering of the Truss Web Members in the Main Span.

[37] NTSB Bridge Factors Report, Attachment 33, p. 24 of 656.

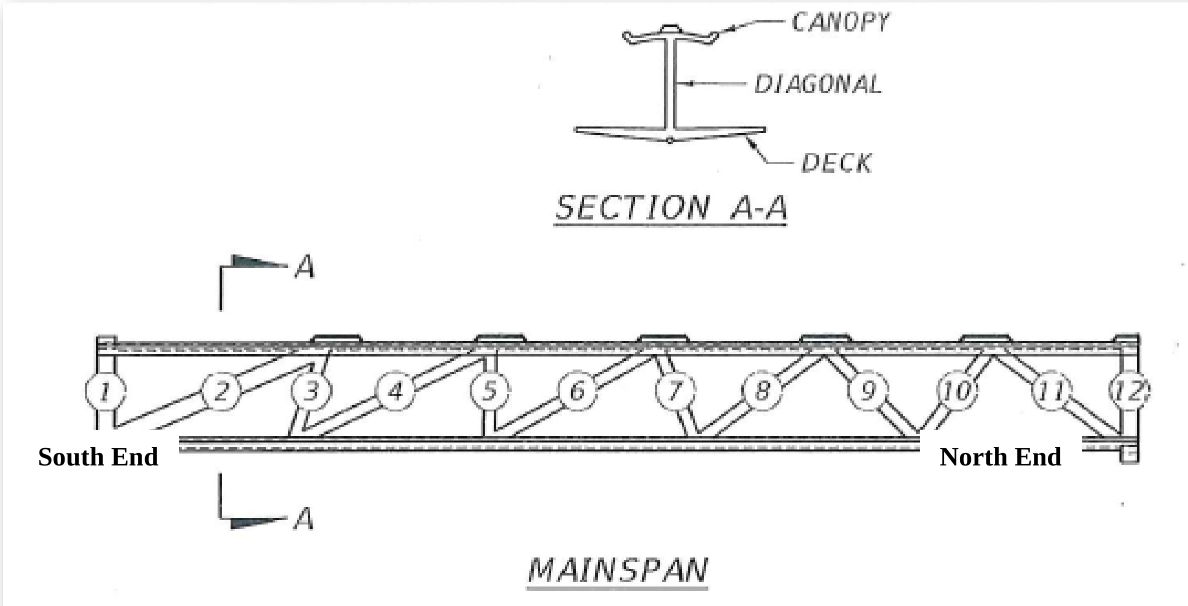

Figure 11. Numbering of Truss Members from 1 (south end) to 12 (north end).

2.2.2 Accelerated Bridge Construction ("ABC")

The ABC method provided several construction benefits, including less vehicular traffic disruption and congestion. However, constructing the Main Span with the ABC method, but not the Back Span, also added several additional steps and increased the complexity of the design and construction processes. To incorporate ABC, overall construction needed to be split into eight stages:[38]

1. Main Span Superstructure pre-casting in the FIU yard
2. Main Span Substructure casting
3. Self-Propelled Modular Transporter ("SPMT") movement and erection of the Main Span
4. Casting of Back Span
5. Pylon casting
6. Installation of stays
7. Installation of additional Bridge elements (fence, joints, lighting, and drainage)
8. Installation of landings

The extra steps most notably included movement of the Main Span from the FIU yard to its final location over the South Pier and Pylon Pier across SW 8th Street (Step 3) and, thus, holding back casting of the Back Span (Step 4) until after the completion and movement of the Main Span.

Unlike the more traditional construction method where the entire structure acts as one unit at the time of shoring/support removal and transfer of self-weight and other loads to the structure itself, the ABC method required the Main Span to support its own weight and any construction loads by itself for many months before it could be coupled with the Back Span. This required FIGG, as the Designer, as well as any applicable review teams, to separately check not only the design of

[38] NTSB Bridge Factors Report, Attachment 38, Sheet B-109.

the Main Span once connected to the Back Span, but also to verify the design of the Main Span as a standalone structure during all construction stages.

As further discussed below, FIGG made multiple design calculation errors associated with this additional step in the design process; and the independent design peer reviewer, Louis Berger, and the FDOT reviewers failed to catch those errors. The rest of this Report will thus focus primarily on those aspects most relevant to the design, construction, and collapse of the Main Span Truss, and the role of these design errors in the collapse.

2.3 Design Plans

A Project meeting was held on February 25, 2016, during which FIGG presented the 30% plans to the Design-Build team, dated September 2015.[39] Thereafter, the bulk of the design plans were delivered in three distinct packages, Foundations, Substructures, and Superstructure, each of which was submitted at 90%, Final, and Released for Construction ("RFC") milestones.[40]

The Substructures RFC drawings (Foundations, Piers, Pylon, and Landings) were signed and sealed by W. Denney Pate of FIGG on December 9, 2016, January 12, 2017, and February 28, 2017, and were approved by the CEI, FIU, and City of Sweetwater on March 16, 2017. The Superstructure RFC drawings (Main and Back Spans) were signed and sealed by Mr. Pate on April 7, 2017 and June 6, 2017, and were approved by the CEI, FIU, and City of Sweetwater on June 14, 15, and 16 of 2017, respectively.[41] Mr. Pate signed and sealed the Bridge general plan and elevation (Sheet B-4) on May 19, 2017, and this drawing was approved by the CEI, FIU, and the City of Sweetwater on June 27, 29, and 30 of 2017, respectively. The RFC drawings were revised twenty-five times between February 2017 and January 2018.[42] Original release and revision dates for each of these design packages are listed in Table 2.

2.4 Design Calculations

In addition to the design plans themselves, FIGG also delivered design calculation packages for the Foundations, Substructures, and Superstructure categories. The 90%, Final, and RFC Superstructure completion levels calculations were delivered on October 5, 2016,[43] February 10, 2017,[44] and April 11, 2017,[45] respectively. These packages summarized FIGG's analysis and design, including computer model input and output data, and documented the Bridge member serviceability and strength checks for different stages of construction.

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[39] Preliminary Plans Page Turn Meeting; *see also* Design-Build Proposal.

[40] NTSB Bridge Factors Report, Attachment 18, Exhibit C to Agreement, "Design and Deliverable Schedule."

[41] NTSB Bridge Factors Report, Attachment 38.

[42] Email from Erica Hango of FIGG to Alan Ruiz. January 24, 2018 10:06 AM.

[43] Superstructure 90% Design Calculations, UniversityCity Prosperity Project Pedestrian Bridge, FIGG, October 5, 2016.

[44] Superstructure Final Design Calculations, UniversityCity Prosperity Project Pedestrian Bridge, FIGG, February 10, 2017.

[45] NTSB Accellion – FIGG Files – Bridge Calculations - 7. UCPP_RFC_Calculations_Superstructure.pdf, FIGG, April 11, 2017, FBE004193.

FIGG used both LARSA 4D and LUSAS software programs to perform its structural modeling and analysis of the Bridge. LARSA 4D appears to have been FIGG's main analytical tool, whereas the LUSAS finite element model was developed simply to verify the results from LARSA (at least according to the design summary narrative from FIGG[46]). FIGG also developed other worksheets and performed hand calculations to determine externally-applied loads and to check Bridge member and connection capacities. The details of these analyses by FIGG and the reasons for the observed differences in their results are discussed in detail in Section 6 of this Report.

Table 2. Design Submittals

Date	**Submittal**
2/25/2016	30% Plans (Page turn meeting presentation)
5/2016	90% Foundation Plans, 90% North Plaza Foundation Design Calculations, South Plaza Foundation Design Calculations
6/2016	90% Substructure Plans, Final Foundation Plans
6/14/2016	90% Landing Substructure Design Calculations
7/2016	90% Substructure Plans
7/11/2016	90% Pylon Foundation and Footing Design Calculations
7/13/2016	Revised 90% North Plaza and South Plaza Foundation Design Calculations
7/13/2016	Updated 90% Foundation Plans
8/1/2016	Revised 90% Landing Substructure Design Calculations
9/2016	Updated Final Foundation Plans
9/13/2016	Final North Plaza and South Plaza Foundation Design Calculations
9/13/2016	Final Pylon Foundation and Footing Design Calculations
9/26/2016	90% Superstructure Plans
9/28/2016	Final Pylon Substructure Design Calculations
9/29/2016	Final Landing Substructure Design Calculations, Final Substructure Plans
10/5/2016	90% Superstructure Design Calculations
10/31/2016	RFC Foundation Plans
12/9/2016	Updated RFC Foundation Plans
1/11/2017	RFC Pylon Substructure Design Calculations
2/9/2017	Final Superstructure Plans
2/10/2017	Updated Final Superstructure Plans, Final Superstructure Design Calculations, Final Superstructure Miscellaneous Details Design Calculations
2/28/2017	RFC Substructure Plans
4/7/2017	RFC Superstructure Plans
4/11/2017	RFC Superstructure and Miscellaneous Detail Design Calculations
6/6/2017	Updated RFC Superstructure Plans

46 NTSB Accellion – FIGG Files – Bridge Calculations - 7. UCPP_RFC_Calculations_Superstructure.pdf, FIGG, April 11, 2017, (p. 5 of 1456), FBE004197.

2.5 Design Review

Multiple teams reviewed FIGG's design calculations and plans. These included an internal team from FIGG's Northeast Regional Office ("NERO"), external consultant Louis Berger, and third-party FDOT. However, the Bridge was classified as a Category 2 structure according to Chapter 26, Section 3.2 of the FDOT Plans Preparation Manual, which required that all pertinent designs and calculations be validated by a qualified Independent Peer Review ("IPR").[47] Importantly, none of these reviews of FIGG's design calculations and plans were performed in a way that met the requirements for such an IPR.

2.5.1 Internal Review within FIGG

FIGG's Southeast Regional Office ("SERO") developed the original Bridge design, so FIGG's NERO was later tasked to perform an internal review of SERO's Superstructure design beginning on October 4, 2016 and ending October 21, 2016.[48] Although there is correspondence related to this task extending into late October 2016,[49] MCM has not seen any draft or final deliverable from the internal review team at NERO to SERO. Regardless, although FIGG had originally intended to use this internal review as the *independent* review of the Superstructure design,[50] FDOT standards required an *external* IPR.

2.5.2 Independent Peer Review by Louis Berger

FIGG thus executed a sub-consultant agreement with Louis Berger for IPR services on October 7, 2016.[51] According to the FDOT Plans Preparation Manual, Section 12 (with emphasis added):

> *The peer review is <u>intended to be a comprehensive</u>, <u>thorough independent verification</u> of the original work. An independent peer review <u>is not simply a check of the EOR's plans and calculations</u>; it is an independent verification of the design <u>using different programs and independent processes</u> than what was used by the EOR.*

Although Louis Berger performed its IPR using a different computer analysis program (ADINA), the <u>IPR did not include critical stages of construction</u> and <u>failed to check FIGG's design on issues directly relevant to the Main Span's collapse</u>, as explained in detail in Section 7. Nonetheless, on September 13, 2016, September 29, 2016, and February 10, 2017, Louis Berger submitted letters to FIU certifying that it had completed its IPR of the 100% Bridge Foundation, Substructure, and Superstructure Plans according to Chapter 26 of the FDOT Plans Preparation Manual and that all outstanding comments had been resolved.[52]

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[47] NTSB Bridge Factors Report, Attachment 44, Plans Preparation Manual, FDOT, 2017, Chapter 26, Section 12; *see also* Plans Preparation Manual, Volume I – English, revised January 1, 2015, FDOT, Section 26.5.

[48] FIGG Memorandum RE: UniversityCity Prosperity Project, Independent Design Review Task 3 – Superstructure Design, October 4, 2016.

[49] Email RE: FIU – Independent Review – 90% Superstructure Design, October 25, 2016.

[50] BT-904| FIU: UniversityCity Prosperity Project Qualifications Submittal, MCM and FIGG, July 30, 2014, p. 75.

[51] NTSB Bridge Factors Report, Attachment 19.

[52] NTSB Bridge Factors Report, Attachment 48.

2.5.3 Review by FDOT

Section 26.5 of FDOT's Plans Preparation Manual says that FDOT's state-level Structures Design Office "has total project development and review responsibility for projects involving Category 2 Structures."[53] Section 26.18 of this manual identifies FDOT responsibilities on bridge projects, like this, that receive funding from non-FDOT sources but cross FDOT assets. Among them are a requirement that FDOT review the project to confirm that it has been designed in accordance with a nationally-recognized code such as AASHTO, American Concrete Institute ("ACI"), or American Institute of Steel Construction ("AISC"), and that the plans meet all district-level permit requirements and procedures. As such, this Project received extensive attention from FDOT throughout its life.

As required, FIGG submitted its design plans and calculations to FIU and FDOT prior to being finalized. FDOT reviewed the 90% and Final plans and calculations and provided numerous written comments and questions to which FIGG responded formally.[54] The types of comments from FDOT demonstrated a detailed technical review of the plans and calculations. Most notably, FDOT's review of FIGG's calculations included Item 28 on the 90% submittal, which pointed out to FIGG that the LARSA 4D model did not include PT in the Truss Web Members and said that PT should be added to the model.[55] FIGG responded to FDOT that its LUSAS model included PT in the Diagonals[56]; however, LUSAS models prepared by FIGG until the time of 90% submittal did not include PT in Member 11. After its 90% submittal, FIGG updated its LUSAS models to include PT in Member 11, but—<u>critically—did not update its design calculations to account for increased axial force in Member 11 and increased shear force in the construction joint between Truss Members 11 and 12 and the Bridge Deck ("Node 11/12")</u>.[57] FDOT did not detect this error in its review.

It is notable that FDOT's review of the Final Substructure Plans included a recommendation from Saul Perez that the surfaces of the precast Bridge that were to be encased in the Pylon concrete be roughened to promote mechanical bonding (Item 20, related to Sheet B-25).[58] FIGG responded that it agreed with the comment, and that they had added a note to that effect. The comment from FDOT did not mention any amplitude for the roughening, but FIGG specified ¼-inch amplitude roughening on Sheet B-25.[59] Saul Perez of FDOT also reviewed the Superstructure Plans, but did not make a similar comment with respect to the construction joints between the Deck and the Truss.[60] FDOT was aware of the importance of indicating surface roughening on the plans but did not recognize its omission on the Superstructure drawings.

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[53] NTSB Bridge Factors Report, Attachment 44; *see* also Plans Preparation Manual, Volume I – English, revised January 1, 2015, FDOT, Section 26.5.

[54] NTSB Bridge Factors Report Attachment 63, FCA-A8(1-108).

[55] NTSB Bridge Factors Report Attachment 63, FCA-A8(100).

[56] *Id.*

[57] NTSB Accellion – FIGG Files – Bridge Calculations - 7. UCPP_RFC_Calculations_Superstructure.pdf.

[58] NTSB Bridge Factors Attachment 63, FCA-A8(94).

[59] RFC Substructure Plans, FBE000202.

[60] NTSB Bridge Factors Attachment 63, FCA-A8(100).

FDOT was the only external party that performed a detailed technical review of both FIGG's plans and calculations. Therefore, FDOT was in a unique position to detect any discrepancy between FIGG's design calculations and the drawings. However, as will be discussed more fully later, FDOT failed to recognize errors and omissions in FIGG's design calculations and drawings.

2.6 Issues with FIGG's Design Plans

Some of the key issues regarding FIGG's design plans are reviewed in this Section, including the various changes to the design, specification of construction joints, and development of reinforcement in the Truss nodes. A more complete review of FIGG's design calculations is discussed in Sections 5 and 6.

2.6.1 Shift to North

FIU and FDOT raised a concern on October 12, 2016 regarding availability of space under the Bridge to accommodate future widening of SW 8th Street.[61] At that time, the Foundations and Substructure plans had already been finalized and the Superstructure plans were 90% complete. Nevertheless, this concern resulted in a plan to shift the Bridge position to the north by approximately 11 feet. This shift was mentioned in several media reports and was erroneously interpreted as an increase in the length of the Main Span that could have potentially contributed to the collapse. This allegation is not correct; the shift did not affect the geometry of the Main Span or require any other design modifications, as the entire Substructure was also moved to the North by the same distance. This change did not contribute to the collapse.

2.6.2 Addition of Temporary Post-Tensioning to End Diagonal Members 2 & 11

For the intended move of the Main Span, FIGG's initial design proposal had SPMTs providing support near the ends of the Main Span and at the first panel points (*i.e.*, the intersection of Members 3-4 and 9-10). But, as shown in Figure 12, FIGG's later RFC drawings in B-109 instead showed SPMTs providing support on either side of these first interior panel points. This change resulted in approximately 35 feet of the Main Span on both the north and the south sides (37' and 33'-9", respectively) to be unsupported (cantilevered past the SPMT support) during transport. This changed the distribution of force in the Web Truss Members, with the most significant change being the creation of tensile forces in Members 2 and 11.

Members 2 and 11 notably did not include any PT bars in FIGG's proposal.[62] Therefore, to address the temporary tension force in these elements during the transportation stage, FIGG added PT bars to Members 2 and 11 to counterbalance this tension by applying an initial compression to concrete and specified that temporary stressing should occur during transport.[63] The stressing was to occur before transport began, with subsequent *de-*stressing to occur after final placement on the South Pier and the Pylon.[64]

61 NTSB Bridge Factors Report, Attachment 34.

62 Design-Build Proposal, p. 115 of 173 (Sheet B-17).

63 NTSB Bridge Factors Report, Attachment 38; RFC Sheet B-109.

64 NTSB Bridge Factors Report, Attachment 38; RFC Sheet B-38.

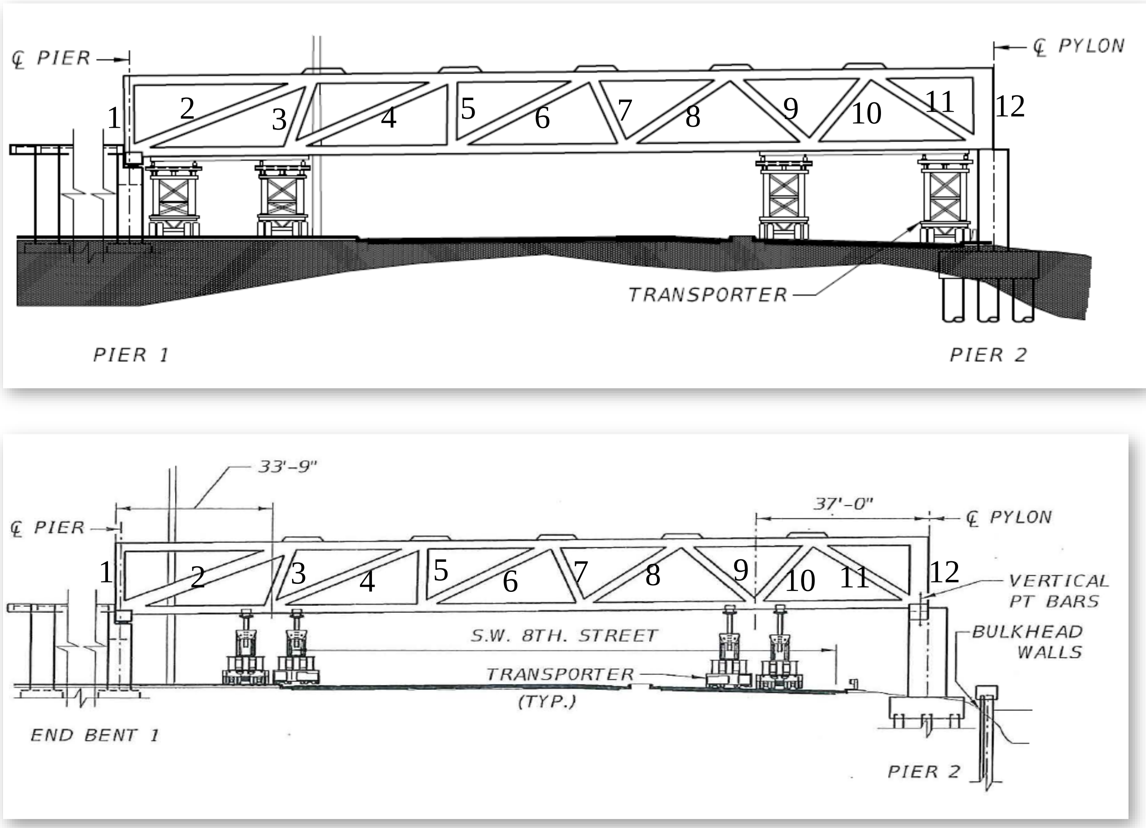

Figure 12. Placement of SPMT support: Proposal (upper) and RFC (lower).[65]

This addition meant that the large compression and shear forces created from the temporary PT in Members 2 and 11 compounded the already large compression and shear forces that existed in these elements both before and after the move. Critically, FIGG did not update its calculations (*i.e.*, interface shear friction capacity) to check this worst-case loading combination of high compression and shear from self-weight *and* PT at Members 2 and 11.[66] It is this combination of compressive forces in Member 11 and resulting shear at Node 11/12, combined with FIGG's failure to accurately verify the structural integrity of the Bridge at these locations, that led to the cracking and final collapse, as discussed in more detail in Sections 5 and 6.[67] FDOT failed to

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65 Design-Build Proposal; NTSB Bridge Factors Report, Attachment 38; RFC Sheet B-109.

66 NTSB Accellion – FIGG Files – Bridge Calculations - 7. UCPP_RFC_Calculations_Superstructure.pdf. FBE005476.

67 The destressing process also caused problems. Although the effect of destressing the PT bars was beneficial in that it reduced the magnitude of compression in the end Diagonals and the shear across the Node 11/12 interface, the destressing process itself was harmful overall. In order to initiate destressing, additional force must first be applied to the PT bars in order to relieve the contact pressure between the nuts and the end plates so that the nuts can be loosened. There is no established standard governing how much force is allowable for the initiation of destressing. It is thus unknown how much additional tensile force was actually applied to the PT bars to initiate destressing, and consequently how much additional compression was induced in the Diagonal concrete Truss Members. It is also unknown what sequence Structural Technologies followed for the destressing, such as unloading increments and patterns of alternation between bars. It appears, however, that there was a temporary condition in which the compression in the end diagonals exceeded the preexisting compression from dead load

note FIGG's omission of PT forces in Member 11 during its review of FIGG's Superstructure design calculations.

2.6.3 Construction Joint Specifications in Drawings

The locations at which Truss Web Members meet the Deck and Canopy are referred to as "construction joints" ("CJ" or "cold joints"). As shown in Figure 13 below, which includes excerpts from Sheet B-37 of FIGG's plans with the construction joint callout indicated, depicts these interface surfaces for the Main Span with horizontal dashed lines. Only one instance is specifically identified, however, and the callout reads, "C.J. (TYP)." These abbreviations inform the user of the drawing that all instances of these dashed lines represent construction joints.

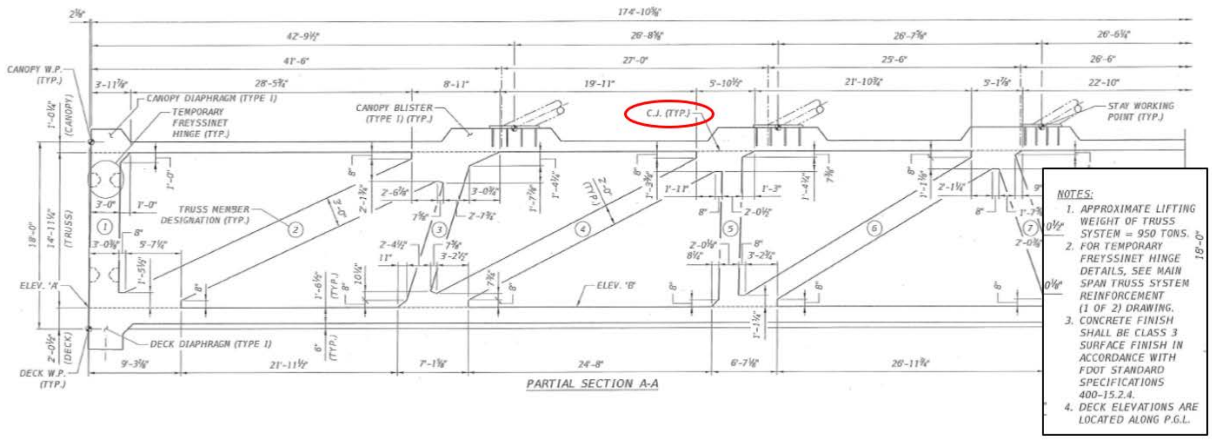

Figure 13. Main Span Truss elevation with construction joints and notes from RFC Sheet B-37.[68]

Based on this B-37 note, and the details of construction stage 2 shown on Sheet B-109, FIGG anticipated and allowed the placement of the Deck concrete to be completed prior to the placement of the Truss Web Member concrete and expected that the Canopy concrete would be placed after the Truss Web Member concrete (thereby creating construction joints). However, FIGG did not provide any instructions regarding surface roughening amplitude at these construction joints, despite the fact that Eddy Leon of FIGG stated in his NTSB interview that intentional roughening is usually called out on the plans when it is intended in the design.[69]

Notably, two Substructure drawings (RFC Sheets B-24B and B-25) identify concrete construction joints in the Pylon Diaphragm. By contrast, in these instances, the notes on the drawing <u>do</u> supply further information and <u>explicitly state that the proposed construction joint "shall be roughened to an amplitude of ¼"</u> prior to casting back span." Excerpts of construction joint detail and the accompanying notes from Sheet B-24B are shown in Figure 14 below.

("DL") and PT forces. This scenario, where the PT force was initially increased to free up the nuts and the end plates at the beginning of the destressing process, likely produced the maximum force carried by the Diagonals (and therefore the shear at Node 11/12 interface) during the life of the structure.

[68] NTSB Bridge Factors Report, Attachment 38; RFC Sheet B-37.

[69] NTSB FIGG Interview Transcripts, Eddy Leon, 223:1-5.

Similarly, for the construction joint between the concrete bearing seat and the top of the end bent in Sheet B-104, FIGG specified a surface roughness amplitude of 1/8-inch.[70] FDOT reviewed all of these drawings and did not take any exception to three different types of specifications[71] for surface roughening amplitude.

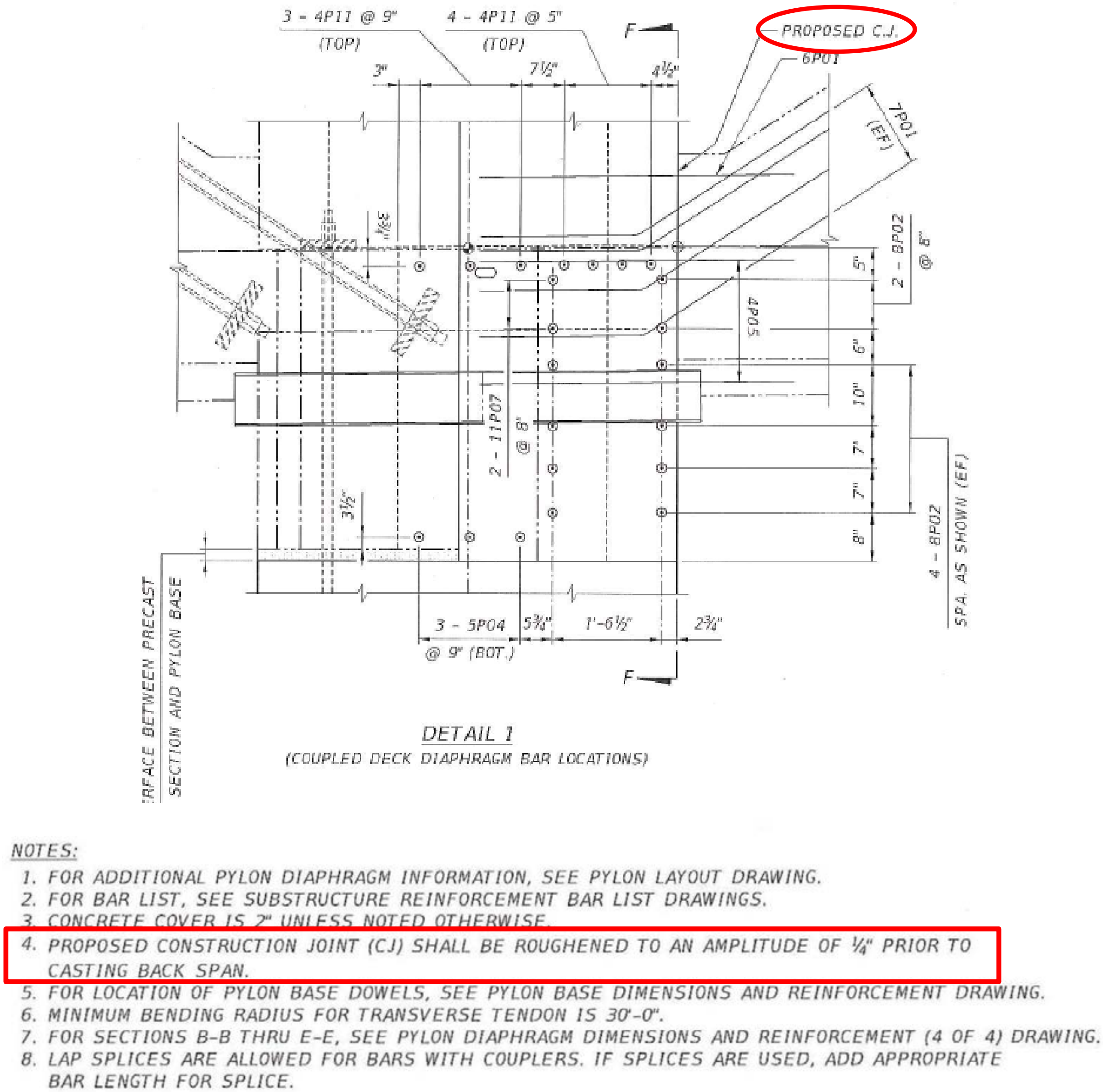

Figure 14. Excerpts of detail with construction joint and notes from RFC Sheet B-24B.[72]

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70 NTSB Bridge Factors Report, Attachment 38; RFC Sheet B-104.

71 No specification, 1/8-inch, and ¼-inch.

72 NTSB Accellion – FIGG Files – Bridge Calculations - 7. UCPP_RFC_Calculations_Superstructure.pdf.

The three different ways of specifying surface roughening amplitude for construction joints in B-37, B-24B, B-25, B-82, and B-104, as well as the statement by Eddy Leon to NTSB, all demonstrate that FIGG was aware of the necessity to indicate a specific surface roughening amplitude explicitly on the drawings when required for implementation in the field.

As listed in the General Notes section of the RFC drawings, different sets of specifications govern the work performed by the Designer and the Contractor. FIGG used the equations and parameters corresponding to intentional roughening of ¼-inch amplitude from AASHTO LRFD Bridge Design Specifications Section 5.8.4.3 in its design calculations. However, any information from the design specifications that is relevant to the construction but is not covered by AASHTO and FDOT construction specifications, needs to be conveyed to the contractor through the drawings. This was done in Sheets B-24B, B-25 and B-104, but not for drawing Sheet B-37 for construction joints in the Main Span. As such, it would not be reasonable for FIGG to assume the parties implementing the plan (MCM, TSG, BPA, and Corradino) would interpret drawing Sheet B-37 as requiring a ¼-inch surface roughening amplitude at construction joints in the Main Span, despite the evidence that the surface was intentionally roughened to some degree (as discussed below).

As discussed earlier, FDOT was the only party external to FIGG that performed a detailed technical review of both FIGG's plans and calculations. Therefore, FDOT was in a unique position to detect the discrepancy between FIGG's inclusion of ¼-inch amplitude surface roughening in its design calculations for the connections between the Truss Webs and the Deck and its failure to note that as a requirement on the Main Span drawings for communication to the Contractor and CEI. However, FDOT failed to recognize that FIGG's design calculations conflicted with its plans for this critical component.

2.6.4 Development of Node Reinforcement

In addition to PT bars and tendons, the Bridge's concrete is reinforced with "mild" steel bars ("rebar"). In order to be effective in resisting tension, mild steel reinforcement must be embedded adequately in concrete to ensure that the capacity of the reinforcing bar can be "developed" to provide the support for which they are intended to provide. The development length depends on the bar diameter, the concrete strength, the configuration of the bar, and other related factors. With specific regard to the Main Span, the bars in the Superstructure nodes needed to have adequate development length both above and below the construction joint. Otherwise, a reduced capacity for the reinforcement should have been considered in the design.

The mild steel reinforcement in the Truss nodes for Members 11 and 12 located at the Deck elevation are shown in Figure 15 and Figure 16. Bars 7S01 are hat-shaped bars placed transverse to the Bridge length, and Bars 6S07 are approximately u-shaped bars placed parallel to the axis of the Bridge. The leading numeral in the bar designations indicate the size of the bar. The drawing excerpt in Figure 16 shows that the 6S07 bars were not to be placed within the bends at the tops of 7S01 bars but were to be located more centrally in the nodes.

AASHTO LRFD Section 5.11.2.6.1 states, “Between anchored ends, each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar.”[73] Because the bar bends for the 7S01 bars did not enclose a longitudinal bar, these #7 bars cannot be considered stirrups. Likewise, because the 6S07 bars are located below the 7S01 bars, the #6 bars also cannot be considered anchored as stirrups. (The transverse section on Sheet B-60 shows the 6S07 bars to be located away from the bends.) Therefore, the minimum development lengths for hooks or straight bars applied.

The top segments of the 6S07 bars were long enough to be considered as the ends of 90-degree hooks. The development length of these bars, considering applicable modifications, was 6.8 inches, which is less than the available eight inches. The 6S07 bars were thus fully developed across the construction joint at the nodes. The top segments of the 7S01 bars did not have enough length to qualify as 90-degree hooks, and the development length of a straight #7 bar in the given configuration is 15.5 inches for the southernmost 7S01 and 12.375 inches for the northern three bars (taking advantage of a beneficial reduction allowed by AASHTO LRFD Section 5.11.2.1.3). The available embedment length above the construction joint was only eight inches. Therefore, the 7S01 bars were not fully developed, and any design calculation that involved these bars across the construction joint at Node 11/12 should have recognized and incorporated an associated strength reduction.

In summary, the 7S01 bars crossing the construction joint at Node 11/12 were not designed to be embedded in concrete deep enough to allow development of their full strength. Therefore, shear strength calculations at Node 11/12 that involve any contribution from 7S01 bars should have considered a reduced capacity of these bars. Accurate estimation of shear capacity at Node 11/12 is essential for explaining the cause of the observed cracking and failure in and around construction joint at Node 11/12. These calculations are discussed in Section 5.

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73 AASHTO LRFD Bridge Design Specifications, 7th Ed. (with the 2015 interim revisions).

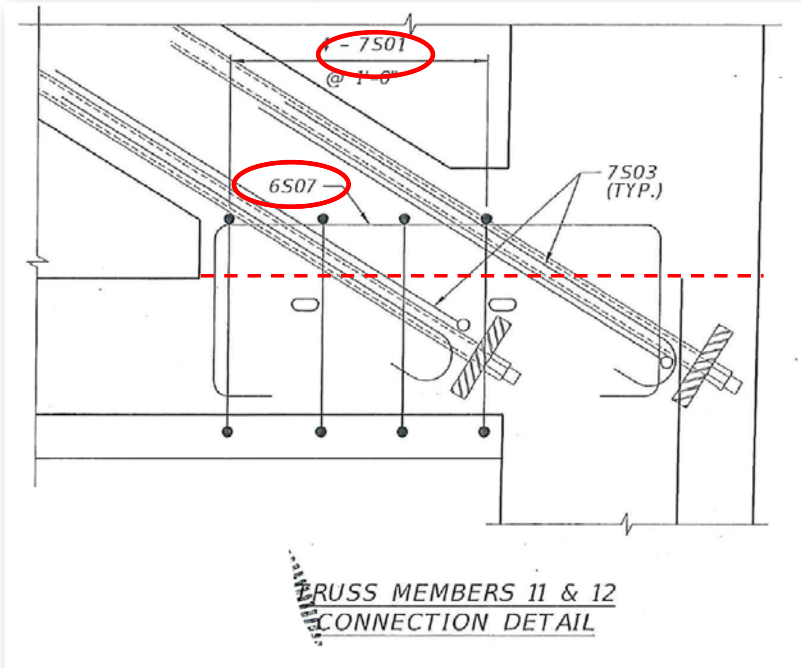

Figure 15. Reinforcement in Node 11/12, Deck construction joint indicated with red dashed line.[74]

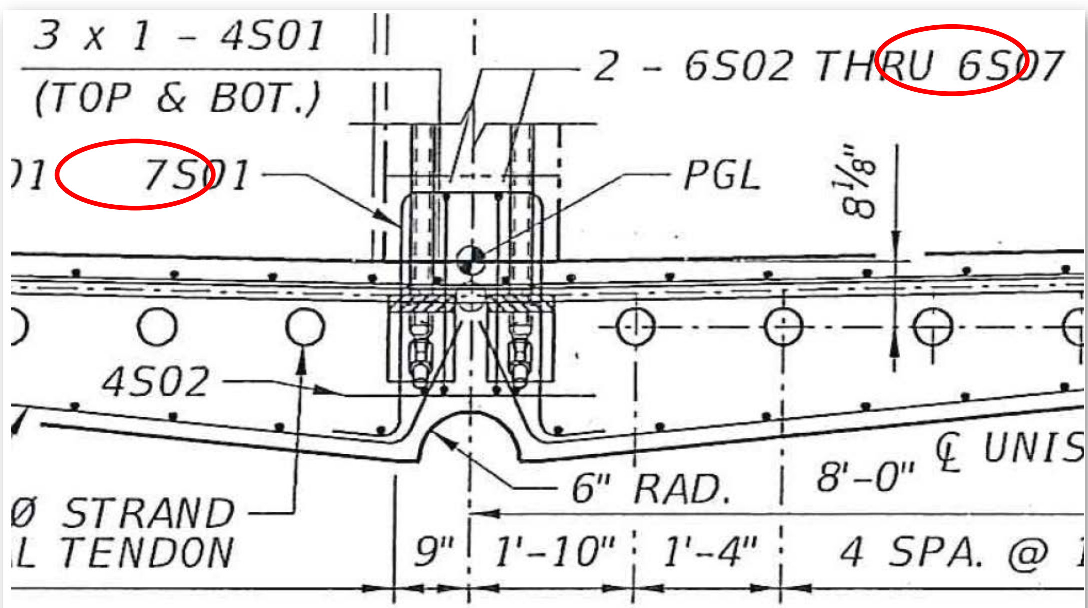

Figure 16. Superstructure Deck cross section, node bars 7S01 and 6S07 indicated.[75]

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[74] NTSB Bridge Factors Report, Attachment 38, Sheet B-61.
[75] NTSB Bridge Factors Report, Attachment 38, Sheet B-60.

3 Construction

By the time the Main Span collapsed, the South Pier, the Pylon Pier, and the Main Span of the Superstructure had all been constructed. Among the various reviewed construction progress photographs taken by the parties depicting the construction and transport of the Main Span, as well as concrete cracking, Figure 17 through Figure 29 are among the most illustrative photographs showing accurate construction implementation of FIGG's plans.

Figure 17. May 30, 2017: Erection of shoring and forms for Main Span.[76]

[76] NTSB Bridge Factors Report, Photo 3.

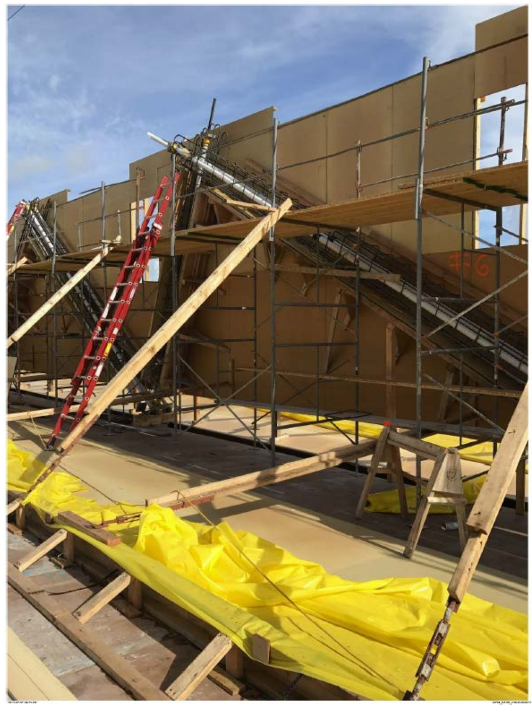

Figure 18. July 7, 2017: Deck and Truss forms.[77]

Figure 19. July 25, 2017: Forms, reinforcement, and PT at Type I Diaphragm and Truss, location of construction joint indicated with red arrow.[78]

[77] NTSB Bridge Factors Report, Photo 6.

[78] MCM Photo Submission to NTSB, MCM_NTSB_OSHA-004070.

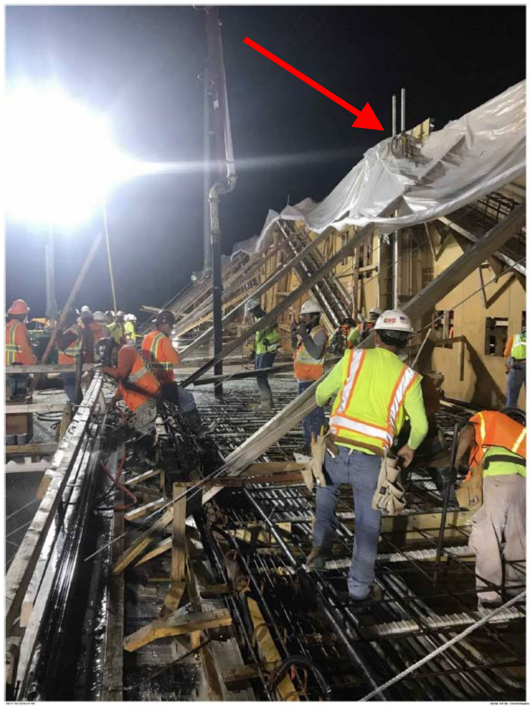

Figure 20. October 19, 2017: Deck concrete placement, location of construction joint indicated with red arrow.[79]

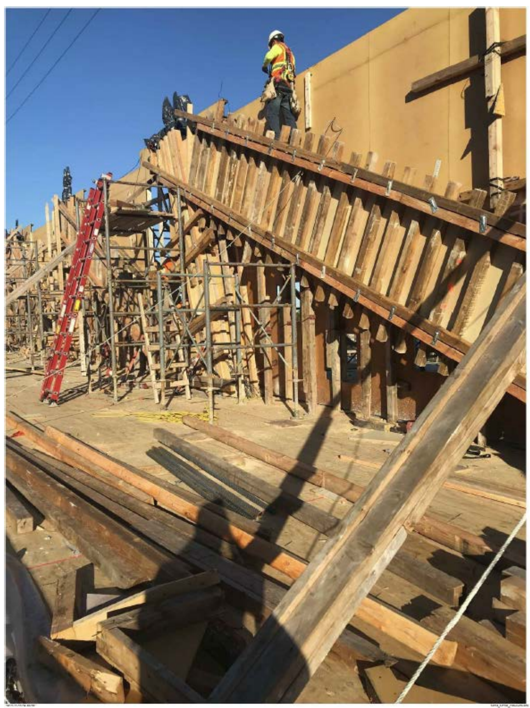

Figure 21. November 3, 2017: Truss Web form closure.[80]

[79] NTSB Bridge Factors Report, Photo 11.

[80] NTSB Bridge Factors Report, Photo 13.

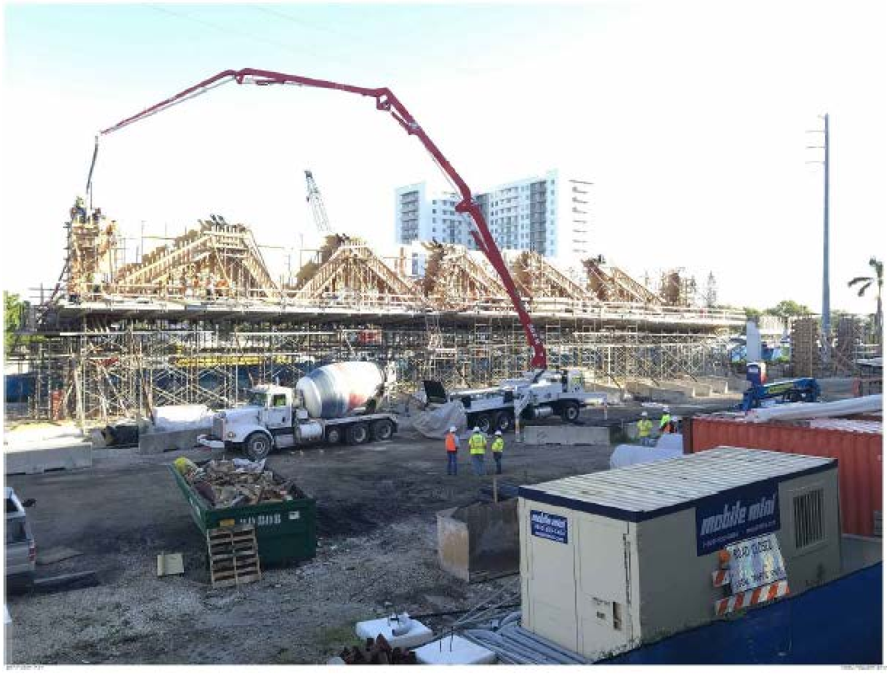

Figure 22. November 6: 2017: Truss concrete placement.[81]

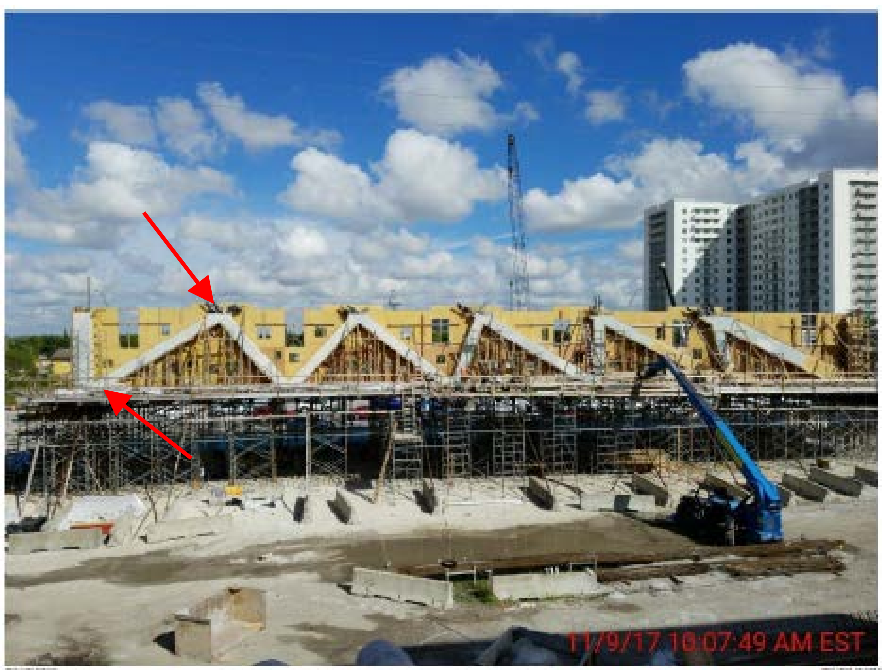

Figure 23. November 9, 2017: Truss form removal, location of construction joints indicated with red arrows.[82]

[81] NTSB Bridge Factors Report, Photo 14.

[82] NTSB Bridge Factors Report, Photo 16.

Figure 24. November 15, 2017: Canopy forms.[83]

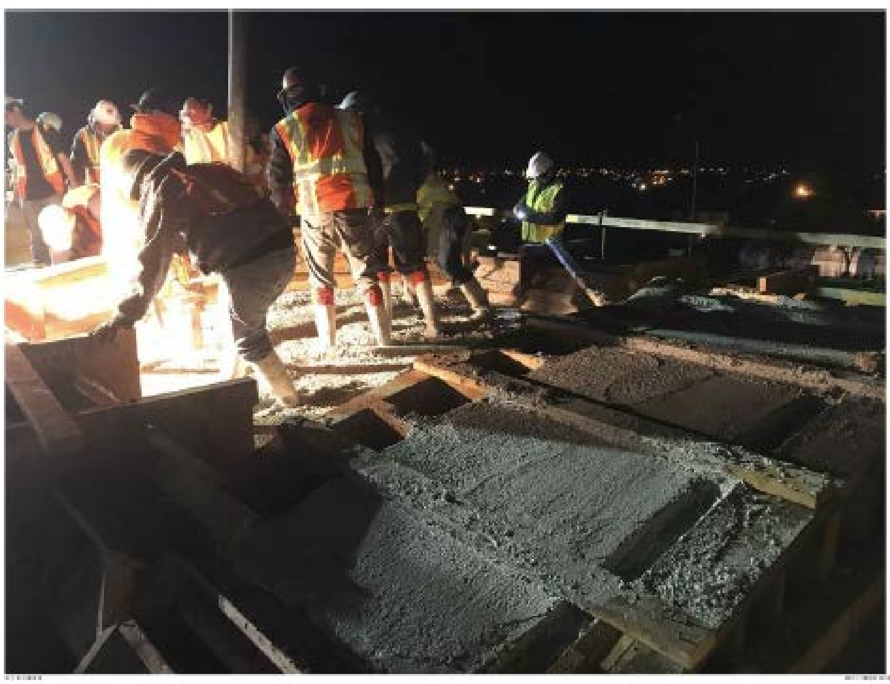

Figure 25. December 14, 2017: Canopy concrete placement.[84]

83 NTSB Bridge Factors Report, Photo 18.

84 NTSB Bridge Factors Report, Photo 25.

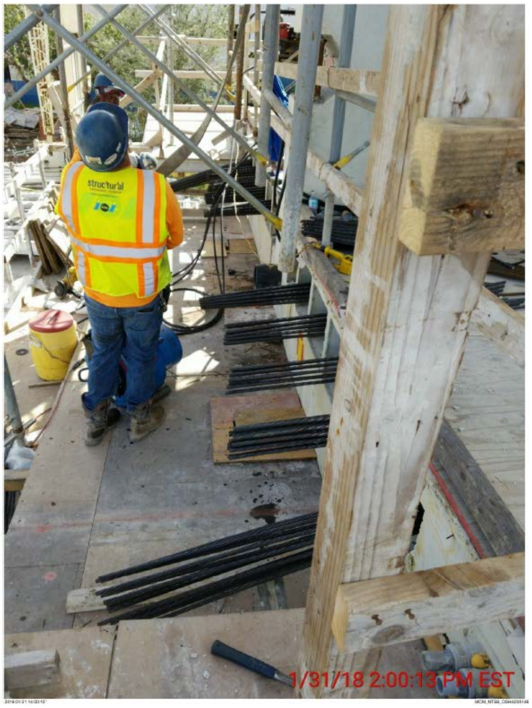

Figure 26. January 31, 2018: Deck PT stressing.[85]

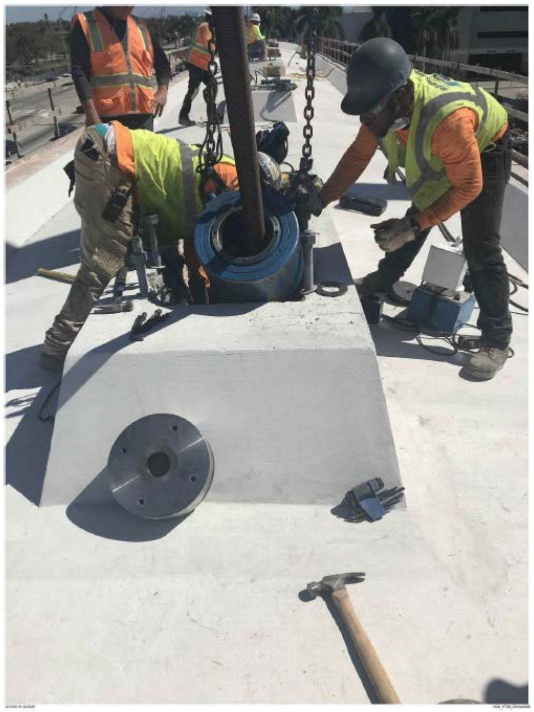

Figure 27. February 17, 2018: Truss post-tensioning.[86]

85 NTSB Bridge Factors Report, Photo 30.

86 NTSB Bridge Factors Report, Photo 32.

Figure 28. February 24, 2018: Formwork and shoring removal.[87]

Figure 29. March 10, 2018: SPMT transport.[88]

[87] NTSB Bridge Factors Report, Photo 40.

[88] NTSB Bridge Factors Report, Photo 53.

Aspects of the construction most relevant to the failure of the Main Span are discussed in more detail in the following sections.

3.1 Main Span Concrete Strength

The General Notes on FIGG's RFC drawing Sheet B-2 require three categories of concrete: Class IV for Substructures and Elevator Towers, Class V (Special) for Precast Piles, and Class VI for the Superstructure and elements of the Pylon. Cemex submitted structural concrete mix designs for the Substructures and Superstructure to MCM dated March 21, 2017 and April 5, 2017,[89] and the anticipated strengths for the three Cemex mixes were 7,340 psi, 7,910 psi, and 10,140 psi, respectively—all of which exceeded FDOT's 28-day compressive strength values of 5,500 psi, 6,000 psi, and 8,500 psi in FIGG's design drawings.[90] Universal Engineering Sciences also performed standard concrete compressive strength tests on samples collected from the Main Span Superstructure, all of which also exceeded the minimum strength of 8,500 psi specified by FIGG.[91] Testing during The NTSB Investigation has not uncovered any problems with Bridge concrete.[92]

3.2 Concrete Pour and Construction Joint Surface Roughening

As further explained in Sections 5 and 6, the issue of Truss construction joints is central to this investigation. During Main Span construction, the Deck concrete was poured first, followed by the Truss Webs, and finishing with the Canopy. Therefore, concrete construction joints existed where new concrete was placed against hardened concrete at the interfaces between each of these portions of the Main Span. Two construction specifications set forth in the General Notes section of the RFC drawings address the requirements for surface roughening applicable to Bridge *Construction* (in contrast to Bridge *Design*).[93]

First, Section 400 of the FDOT Standard Specification is applicable to the construction of concrete structures or components, except for pavements or incidental concrete. Section 400-9 relates to concrete construction joints, and 400-9.3 states,

> *Before depositing new concrete on or against concrete which has hardened, re-tighten the forms. Roughen the surface of the hardened concrete in a manner that will not leave loosened particles, aggregate, or damaged concrete at the surface. Thoroughly clean the surface of foreign matter and laitance and saturate it with water.*

Second, the AASHTO LRFD Bridge Construction Specifications (second edition, 2004) addresses construction joints in section 8.8, and 8.8.2 states,

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[89] Concrete Mix Design, Cemex, MCM (NTSB/OSHA) 000657-000659, March 21 and April 5, 2017.

[90] RFC Drawing Sheet B-2, FBE000172.

[91] Universal Engineering Sciences Compressive Strength of 4" x 8" cylinder Test Specimens. MCM (NTSB/OSHA) 001455 – 001466.

[92] *See* NTSB Investigative Update, August 9, 2018; NTSB Second Investigative Update, November 15, 2018.

[93] RFC Drawing Sheet B-2, FBE000172.

> *Unless otherwise specified in the contract documents, horizontal joints may be made without keys, and vertical joints shall be constructed with shear keys. Surfaces of fresh concrete at horizontal construction joints shall be rough floated sufficiently to thoroughly consolidate the surface and intentionally left in a roughened condition. Shear keys shall consist of formed depressions in the surface covering approximately one-third of the contact surface. The forms for keys shall be beveled so that removal will not damage the concrete.*
>
> *All construction joints shall be cleaned of surface laitance, curing compound, and other foreign materials before fresh concrete is placed against the surface of the joint. Abrasive blast or other approved methods shall be used to clean horizontal construction joints to the extent that clean aggregate is exposed. All construction joints shall be flushed with water and allowed to dry to a surface dry condition immediately prior to placing concrete.*

Importantly, neither of these two standards provides a specific amplitude for surface roughening applicable in all situations. Therefore, unless otherwise specified in the construction contract documents (*i.e.*, the design drawings), a designer cannot premise the strength of the bridge design on the hope that the constructor and inspectors will happen to apply an unspecified surface roughness amplitude. And, as discussed further in Section 2.6 above, FIGG did not specify a surface roughening amplitude for construction joints at the Deck/Canopy to Diagonals/verticals interface, including at Node 11/12.[94] Therefore, none of the construction or inspection contractors, including MCM, BPA, Corradino, and TSG, would have known that the surface had to be roughened to the ¼-inch amplitude that was presumed in FIGG's design calculations of the joint strength.

Notably, MCM, BPA, and FIGG exchanged emails between June 10, 2017 and June 13, 2017 regarding the preparation of construction joints in other parts of the Bridge, namely Foundation Type 3.[95] BPA commented that a bonding agent may be desirable since carrying out the FDOT specification may be difficult in the presence of protruding column reinforcement. FIGG responded by quoting Section 400-9.3 of the FDOT Standard Specification. This conversation demonstrates that the parties did discuss roughening issues and highlights the fact that none of FIGG's instructions related to the Web Truss to Deck construction joints.

The NTSB should conclude that the applicable surfaces were in fact intentionally roughened, consistent with these specifications. Indeed, NTSB conducted several witness interviews after the collapse, and the subject of construction joint roughening was explored. Raphael Urdaneta of BPA, for example, said that he understood that the construction joints between the Deck and the Trusses were to be roughened.[96] Likewise, John Jackson of TSG, the primary contractor performing the concrete work, also clarified that the interfaces were cleaned and roughened during the finishing of the concrete after placement.[97] It is unclear from NTSB's initial interview of Pedro Cortes, Quality Control Technician for MCM, whether roughening was completed in

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94 RFC Drawing Sheet B-37, FBE000230.

95 NTSB Bridge Factors Report, Attachment 64, BPA-2.

96 NTSB BPA Interview Transcripts, Raphael Urdaneta,108:24 -109:23.

97 NTSB TSG Interview Transcript, John Jackson, 24:19 - 25:12.

this area. Specifically, at one point during his interview, Mr. Cortes recalled that no roughening was performed at Diagonal to Deck construction joints.[98] However, later when asked specifically about the construction joint at Diagonal Member 11 and the Deck, Mr. Cortes stated that "there was no construction joint" at that location.[99] In subsequent conversations with MCM to clarify this issue, Mr. Cortes stated that the area around Node 11/12 was cleaned of all debris prior to the pour and that roughening was completed using a chipping hammer.[100]

It is not clear from the interviews, investigation documents, or photographs as to what specific amplitude the surface was roughened, and there is no accurate method to evaluate the existence, degree, or amplitude of any roughness intentionally applied during construction by examining the surfaces after the construction joint has failed. In this case, at Node 11/12, there is clear evidence of sliding of two surfaces that were pressed together with great force. Thus, even if it is assumed there was no bond between these surfaces, *i.e.*, the surfaces were not adhered together, the sliding is bound to have destroyed the rough features of both of those surfaces irrespective of the initial amplitude. Any measurements taken after the incident and on a surface that would have been affected by such sliding would not be representative of its surface roughness prior to the failure. In addition, the surface roughness measured after the failure is likely to be lower because sliding of two rough surfaces under large pressure is akin to the process of polishing.

Figure 30 shows the interface between vertical Member 12 and the Deck prior to Deck concrete placement. Figure 31 shows the forms for the Truss Web Members after the Deck concrete had been placed, with the location of a construction joint shown in the bottom left of the image. And Figure 32 and Figure 33 show the remains of the interface between Node 11/12 and the Bridge Deck after the collapse.

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[98] NTSB MCM Interview Transcript, Pedro Cortes, 24:23 - 26:5.
[99] *Id*., 30:18-31:24.
[100] Declaration of Pedro Cortes, September 20, 2019, MCM (NTSB-OSHA) 021419-021420.

Figure 30. Deck and Web forms and reinforcement prior to pour of Deck concrete (Member 12).[101]

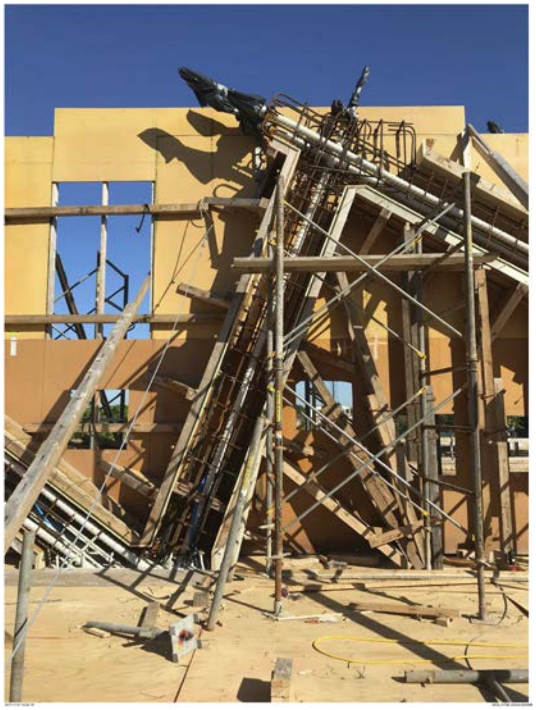

Figure 31. Superstructure forms showing one Web-to-Deck construction joint.[102]

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[101] MCM Photo Submission to NTSB, MCM_NTSB_OSHA-004195.
[102] NTSB Bridge Factors Report, Photo 12.

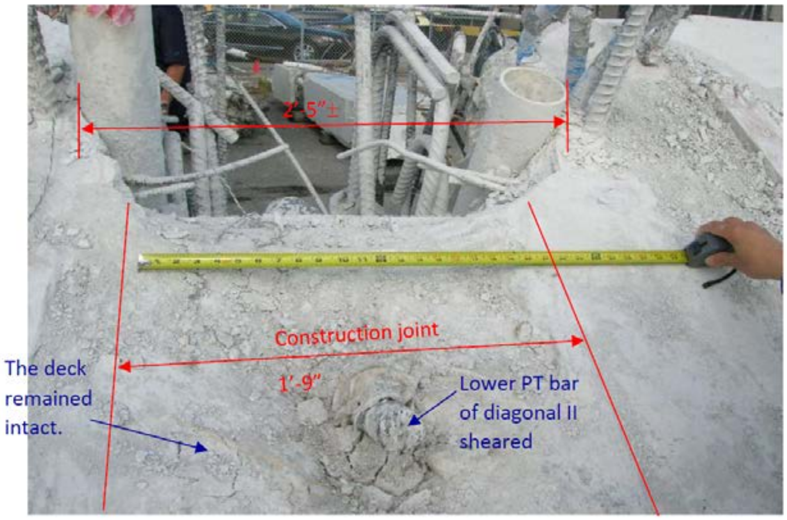
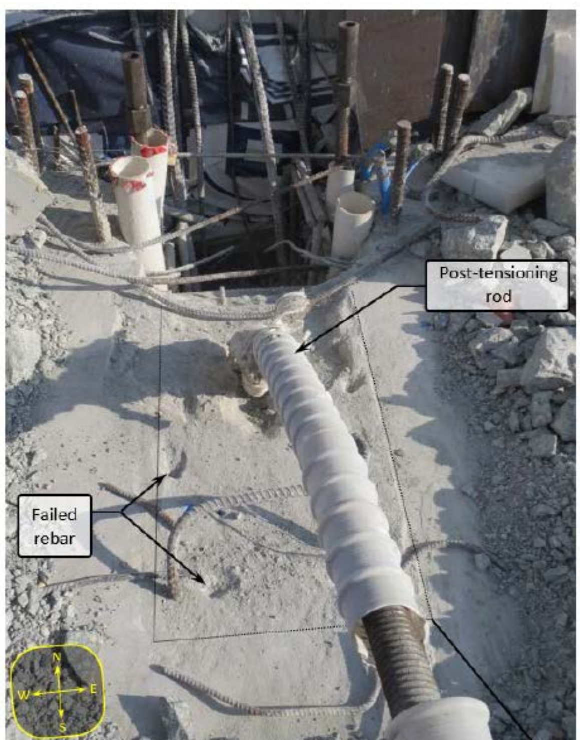


Figure 32. Post-collapse image of the joint between Node 11/12 and the Deck.[103]



Footprint of Member 11 on deck
indicated with superimposed dotted line

Figure 33. Post-collapse image of the joint between Node 11/12 and the Deck.[104]

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[103] Investigation of March 15, 2018 Pedestrian Bridge Collapse at Florida International University, Miami, FL, Occupational Safety and Health Administration, June 2019, ("OSHA Investigation Report"), p. 96.

[104] Turner Fairbank Highway Research Center Factual Report, Concrete Interface Under Members 11 and 12, Federal Highway Administration, October 19, 2018, p. 11.

3.3 BPA Approval of Construction Joint Surface Preparation

In its role as CEI, BPA was responsible for verifying that construction conformed to FIGG's plans and specifications. Accordingly, BPA recently requested that revisions to the draft NTSB Bridge Factors reflect that the email correspondence regarding construction joint preparation that occurred between June 10 and June 13, 2017, as referenced above, was related to construction joints on the south abutment columns.[105] In this comment, BPA clarified that because there was no additional information on the RFC plans for the Main Span Truss construction joints, the provisions of FDOT Standard Specification Item 400-9.3 were followed.

BPA's actions in the field corroborate this fact. Specifically, BPA submitted daily reports of construction on FDOT form 700-10-13. These reports identified which contractors and subcontractors were working, what weather conditions were present, what equipment was on site, and, most importantly, whether the contractors were performing work in accordance with the contract design documents. During the time period from October 18 to December 14, 2017, which covers the time of the placement of the Deck concrete, Truss concrete, and Canopy concrete, BPA did not take exception to any of the preparation of Superstructure construction joints (nor any of the other construction work). For example, BPA noted in its November 4, 2017 daily report that TSG was making final preparations of the Deck prior to the placement of Truss Web concrete on November 6, 2017 but made no direct mention of construction joint preparation and indicated positively that the day's work conformed to the contract documents.[106] Corradino similarly did not raise any issues with surface roughening at these locations.

3.4 First Instance of Cracking Discovered During PT Application on February 6, 2018

On February 6, 2018, after the Main Span Deck PT tendons had been stressed and grouted and Web Truss PT bars in Member 2 and 11 had been stressed but not yet grouted, BPA, MCM, and Corradino noticed minor cracking around several Diagonal Members of the Main Span, specifically Truss Members 3 (south end) and 10 (north end).[107] Truss Members 2 and 11 were the furthest outboard Diagonal Truss Members at each end of the Main Span (Figure 34).

At that time, the Deck and Canopy were still supported on shoring and no cracking was identified in any other Truss Member within the Main Span. BPA created a report discussing the cracking and sent it to MCM on February 13, 2018.[108] MCM forwarded the report to FIGG the same day, and FIGG responded that they would review the report and provide their response.[109] Figure 35 shows photographs of these initial cracks observed by BPA/Corradino.

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105 NTSB Bridge Factors Report, Attachment 64, BPA-2.

106 Email from Carlos Chapman of BPA to Rafael Urdaneta of BPA, November 6, 2017 2:32 AM.

107 *See, e.g.,* NTSB Bridge Factors Report, Photos 1 – 111, and numerous other photos produced during the investigation regarding cracking that developed on the Bridge between February and March 2018, including those produced by MCM.

108 NTSB Bridge Factors Report, Attachment 21.

109 *Id*.

After gathering further information, MCM again transmitted the original report to FIGG on February 15, 2018, clarifying that "all cracks identified are on the West side of the span. Also, and for clarification, these loop around the truss."[110] FIGG responded on February 16, 2018, stating, among other things, that the "CEI's observations of the conditions of Members 3 and 10 after stressing Members 2 and 11 are temporary in nature. The current condition will change as soon as the stressing of the PT bars in Members 3 and 10 is performed."[111] MCM forwarded this response to BPA on February 17, 2018, after also discussing it during the Project weekly meeting on February 16, 2018.[112] FIGG did not express any structural or safety concerns to MCM or BPA at this time.

Figure 34. Photograph of Main Span taken on February 24, 2018.[113]

[110] *Id*.

[111] *Id*.

[112] *Id*.

[113] NTSB Bridge Factors Report, Photo 40.

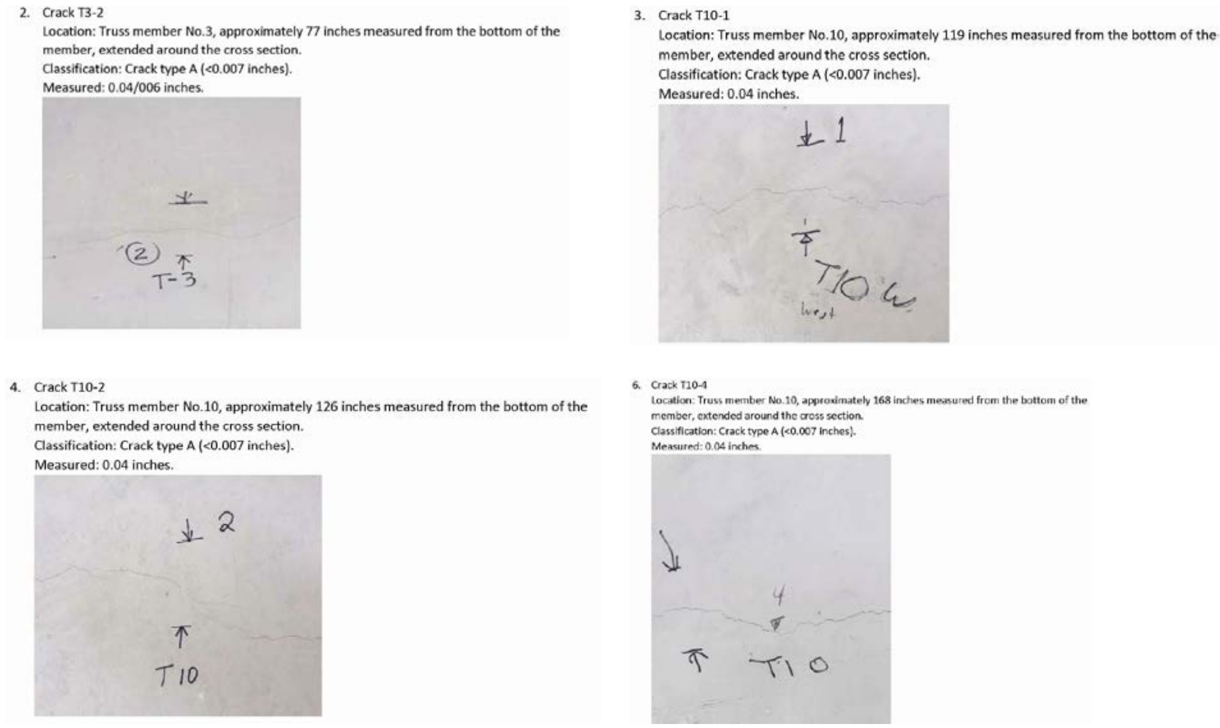

Figure 35. Cracks in Truss Members 3 and 10 on February 6, 2018, after application of PT in Members 2 and 11.[114]

3.5 Second Instance of Cracking Discovered After Shoring Removal on February 24, 2018

Removal of falsework, formwork, and shoring that supported the Main Span during construction began on February 23, 2018 and was completed on February 24, 2018. TSG performed the shoring removal, which RC Group oversaw and inspected. Throughout the process, multiple inspection reports authored by RC Group authorized the shoring removal to continue, and neither TSG nor RC Group ever raised any concerns with cracking on the Main Span during this time.

Upon completion of this effort, however, the Main Span was left self-supporting, *i.e.*, the self-weight that was supported by shoring up until that time was now supported only by the elements of the Main Span itself ("Simply-Supported"). As a result, large forces developed in the Truss Members, and shortly thereafter, MCM and BPA personnel noticed additional cracking on Truss Members 2 and 11. BPA drafted other reports that included photos and drawings of the precise locations and measurements of the cracks, and sent its observations to MCM and FIU on

[114] NTSB Bridge Factors Report, Attachment 21.

February 28, 2018, stating that BPA would monitor cracks on the Bridge.[115] MCM forwarded BPA's emails to FIGG that same day.[116]
Having not received a response from FIGG for more than a week, MCM had to follow up again on March 7, 2018.[117] FIGG then advised, among other things, that the cracking was "***not . . . a structural concern***," and that FIGG was "***not concerned about these very small cracks. . . .***"[118] FIGG further commented that the deflections of the Main Span were within expectations, but clarified that FIGG would need a better description of the locations and widths of the cracks in order to provide their full evaluation.[119] Meanwhile, preparation for transport continued.

Figure 36 shows cracks at the node chamfers near the Deck on February 24, 2018. Figure 37 shows cracks observed on February 28, 2018, after the removal of shoring and forms. FIGG personnel were present at the site on March 9 and 10, 2018, prior to the transport of the Main Span, and FIGG personnel were seen monitoring the existing cracks.[120].

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115 NTSB Bridge Factors Report, Attachment 22.
116 *Id*.
117 *Id*.
118 *Id*.
119 *Id*.
120 *See* NTSB Bridge Factor's Report, FIGG's Photo Submission, FCA-S6 and FCA-S7.

Figure 36. Cracks documented on February 24, 2018, before complete removal of shoring.[121]

[121] NTSB Bridge Factors Report, Photos 36, 37, 38, and 38.

Figure 37. Cracks after removal of forms and shoring on February 26, 2018.[122]

[122] NTSB Bridge Factors Report, Photos 42, 43, 44, and 45.

3.6 Main Span Move using SPMT

On March 10, 2018, the Main Span was moved to its final location through the use of Self-Propelled Modular Transporters ("SPMTs"). After shoring removal was complete, preparation of the ground surface for SPMT setup started on March 1, 2018, which was followed by SPMT part assembly, and installation of tilt meters, strain gages, and displacement sensors. By March 9, 2018, the Main Span was ready to be moved. A test run of the SPMT, including a partial lift and a roll test, was conducted in the evening of March 9, 2018. Barnhart then started moving the Main Span at about 2:20 a.m. ET and completed the move at about 12:30 p.m. ET on March 10, 2018. Figure 38 shows the Bridge supported by SPMTs on SW 8th Street during transportation.

Barnhart oversaw the Main Span movement efforts and developed the details of the move in discussion with FIGG and MCM. Barnhart's drawings were first issued on June 8, 2017, but were later revised to reflect a change in position of the north trailer and the pull-up gantry stroke dimensions, and were finalized on February 27, 2018.[123] BDI, who provided monitoring services during transport, prepared drawings indicating the proposed quantities and locations of strain gauges, tilt meters, and displacement sensors on the Main Span of the Bridge during transport.[124] These drawings were dated December 22, 2017, and FIGG accepted them on February 13, 2018.[125] The drawings do not show any proposed instruments on Diagonal Member 11.

Figure 38. SPMT transport of Superstructure Main Span on March 10, 2018.[126]

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[123] Drawing Number PR1575-G1.0, Revision 01, Barnhart Crane and Rigging, February 27, 2018, MCM (NTSB/OSHA) 000930.

[124] BARNHART CRANE & RIGGING - FIU PEDESTRIAN BRIDGE MOVE - STRUCTURAL MONITORING SYSTEM, BDI Drawing LLT-01, December 22, 2017, MCM (NTSB/OSHA) 000951-956.

[125] *Id*.

[126] NTSB Bridge Factors Report, Photo 53.

During transport, BDI monitored the twist of the Main Span Deck as a “virtual variable” calculated in real-time from the data received from the tilt meters. The twist variable was used to indicate when to stop the transporter and adjust the position of the Bridge, if necessary, such that the twist angle remained within the tolerance for the twist angle throughout the move. During the transport of the Main Span from the staging area to its final position, BDI data shows that the twist tolerance of 0.50 degrees was exceeded two times.[127]

3.7 Destressing PT Bars in Members 2 and 11

After the Bridge was installed, BPA observed cracks in Node 11/12 *before* the destressing of the PT bars in the end Diagonals (Figure 39). A frame from a video of the Main Span just after erection shows diagonal cracks under Member 11 at the south end of Node 11/12, which had been repaired prior to the move. Cracks near Node 11/12 did not open during and immediately after the move (Figure 40).

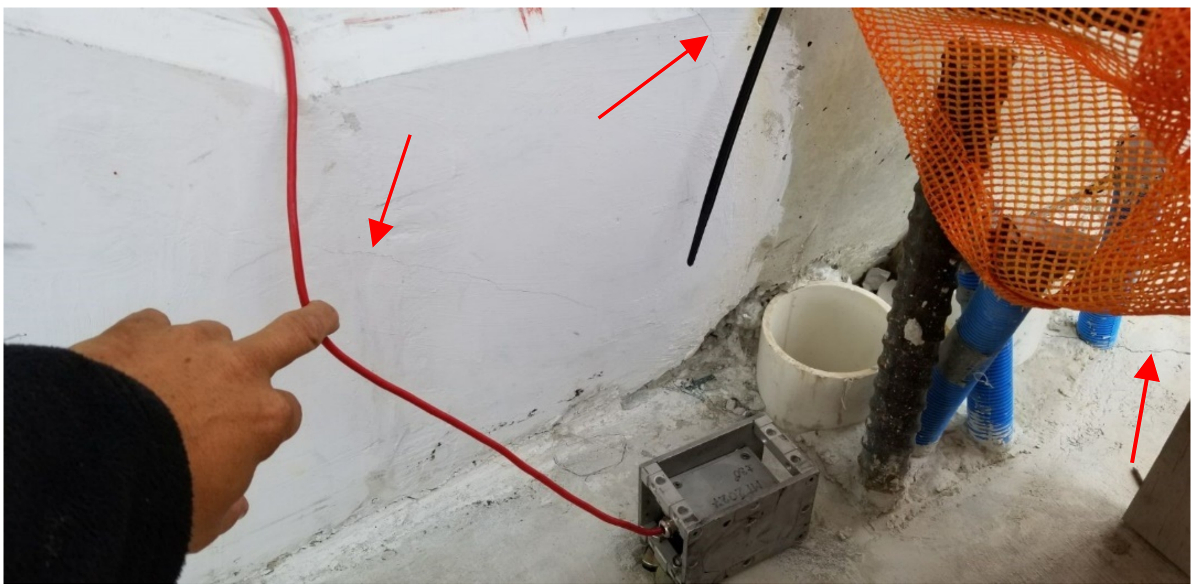

Figure 39. Cracks in and near Node 11/12 after erection and before destressing of Member 11 PT (indicated with red arrows).[128]

[127] This occurred once at 4:45 a.m. ET for approximately 3 minutes and then again at 11:20 a.m. ET for 9 minutes. The maximum twist angles during these times were approximately 0.65 degrees and 0.75 degrees, respectively. The second exceedance of the twist angle tolerance at approximately 11:20 a.m. ET, reportedly occurred when the Main Span came into contact with the southwest bearing pad on the South Pier. The peak twist angle of approximately 0.75 degrees, exceeding the specified twist tolerance by 50%, was measured for approximately 4 minutes.

[128] MCM_NTSB_OSHA-003779 / Bolton-021603.jpg / Carlos Echeverria - Just placed 2.jpg. Also see Bolton-019728.jpg / Carlos Echeverria - Just placed.jpg

Figure 40. Node 11/12 after erection and before destressing of Diagonals 2 and 11 (location of repaired crack indicated with red circle).[129]

Destressing of external Diagonal PT bars began at around 1:30 p.m. ET on Saturday March 10, 2018, and finished around 5:00 p.m. ET the same day.[130] PT bars in Member 11 were destressed first, followed by PT bars in Member 2.[131] Samuel Nunez of Structural Technologies described the destressing process in his interview with NTSB, stating that the threaded PT bar was extended with a coupler nut and another length of threaded rod called a "pull bar," and then force was applied to this extension of the PT bar with the jack until the original nut could be loosened with a ratchet. Structural Technologies was not given any direction from FIGG or MCM regarding how to accomplish the de-tensioning of the PT, *i.e.*, whether the destressing of the two PT bars in Diagonal Members 2 and 11 was to be performed all at once or in small steps, and it was also not specified whether the crew needed to switch between bars after each destressing step.[132]

Alexis Molina of Corradino stated that he observed cracks at Node 11/12 "before they finished the destressing," and that these cracks remained the same after the destressing was complete.[133] Large cracks and spalls were also observed at the top outer edge of the Type II Diaphragm. Photos of cracks in Node 11/12 and Type II Diaphragm taken by Mr. Molina shortly after 3:00 p.m. ET, when destressing of Member 11 PT bars was underway, are shown in Figure 41. Comparing cracks in Figure 37 and Figure 40 from before destressing with the photograph in the upper right corner of Figure 41 shows that the cracks at Node 11/12 were much more severe after the destressing process. The extent of cracks and spalls at the outer edge of the Diaphragm indicates the presence of extensive cracking on the north face of the Diaphragm. Those cracks with a view of the north face were not documented until Monday, March 12, 2018.

[129] Frame from YouTube Video "Up On The FIU-Sweetwater Bridge After The Move" posted by Stuart Grant, https://www.youtube.com/watch?v=SBoE34-WZoE; *See also* FCA-S6.

[130] Field notes of the construction operation covered as senior inspector on Saturday 03-10-2018, CG 000709-CG 000709.pdf

[131] 11.10 Pics 2018.03.10 Pics by Molina IMG_3184.JPG, taken at 1:56PM.

[132] NTSB Structural Technologies Interview Transcript, Samuel Nunez, 49:15 - 50:5-9.

[133] NTSB Corradino Group Interview Transcript, 40:8 - 41:8.

Figure 41. Cracks observed after initiation of destressing on March 10, 2018, at north end of Main Span (indicated with red arrows).[134]

3.8 Crack Progression and Communication Between March 10 and March 15, 2018

After the move was complete and after Structural Technologies de-tensioned Members 2 and 11 pursuant to FIGG's plans, there was a noticeable change in the size and severity of the previously-noted cracking, as documented by photos provided by BPA.[135] MCM learned of the extent of the increased cracking two days later, and immediately followed up with FIGG on Monday, March 12, 2015.[136] Specifically, MCM forwarded BPA's newest crack report showing the expanded cracks, whereby MCM explicitly expressed concern, stating:

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[134] NTSB Bridge Factors Report, Photos 62, 63, 64, and 65.

[135] NTSB Bridge Factors Report, Attachment 24.

[136] *See id*.

> Following our previous emails regarding the noted cracks, and as witnessed on site by FIGG as part of the movement/erection support, attached please find photos depicting the cracks developed prior and post the span 1 erection and/or distressing of truss members 2 & 11 (your team may have most of these pictures). It is our opinion that ***some of these cracks are rather large and/or of concern***; therefore, please review and comment as promptly as possible and advise if there is a required course of action to remedy or address these right away. ***Your immediate attention and response is required***.[137]

MCM also forwarded this correspondence to BPA, which recommended monitoring the cracks.[138] Dwight Dempsey of FIGG responded to MCM on March 13, 2018, stating that FIGG was evaluating this situation as a top priority and that it would make recommendations based on its evaluation.[139] Mr. Dempsey again expressly stated that FIGG ***did not see this as a safety issue***.[140] Nevertheless, FIGG did make several recommendations to address the cracks, namely requiring the placement of supplementary shims ("right away") underneath the Type II Diaphragm at the centerline of the Bridge.[141] The new steel shim under the Type II Diaphragm is shown in Figure 42. Later the same day, Mr. Dempsey of FIGG also responded to MCM and reiterated that FIGG had "evaluated this further and confirmed that ***this is not a safety issue***."[142] Mr. Dempsey also said that FIGG recommended restressing the PT bars as soon as possible, but again, clarified this ***was not a safety concern***. MCM responded by sending additional photos for FIGG's reference and stated they would be monitoring the cracks.[143] MCM sent additional photos to FIGG on March 14, 2018.[144]

Around this same time, on Tuesday, March 13, 2018, Denney Pate, the lead project engineer at FIGG, also left a voicemail message for FDOT employee Thomas Andres describing cracking on the north end of the Main Span but, as FIGG had done with MCM, Mr. Pate reiterated that ***"from a safety perspective" there was no issue, and that FIGG was "not concerned."***[145] FDOT maintains that Mr. Andres was out of the office on assignment and did not hear Mr. Pate's voicemail until he returned on Friday, March 16, 2018, the day following the collapse.[146]

Despite these assurances, FIGG also made plans to visit the Bridge site to further evaluate the issue and to discuss it more fully with all interested parties. FIGG conducted internal analyses of the Bridge and its structural integrity in the meantime, with the intention of sharing its findings at the time of its visit.[147] On March 15, 2018, around 9:00 a.m. ET, FIGG led a meeting with

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137 *Id*. (emphasis added).

138 NTSB Bridge Factors Report, Attachment 26.

139 NTSB Bridge Factors Report, Attachment 27.

140 *Id*.

141 *Id*.

142 *Id*. (emphasis added).

143 *Id*.

144 NTSB Bridge Factors Report, Attachment 29.

145 NTSB Bridge Factors Report, Audio Attachment "Voice mail message from Denney Pate of FIGG to Tom Andres of FDOT on 3-13-2018.m4a."

146 *See* http://www.fdot.gov/info/CO/news/newsreleases/031618_FDOT-Releases-Additional-Information-Regarding-FIU-Bridge-Collapse.pdf.

147 NTSB Bridge Factors Report, Attachments 30 and 32.

MCM, BPA, FIU, and FDOT to discuss the cracking and FIGG's attendant recommendations.[148] Once again, FIGG reported to all parties present that the cracking did not present a safety concern, but also recommended that PT bars of Member 11 be restressed and placed back into a state of tension that day—the purpose of which was seemingly to replicate the off-site staged condition when only minor cracks existed at Members 2 and 11.[149] FIGG represented at this time that its conservative engineering analyses confirmed there was not a structural risk that could lead to a safety concern.[150] The minutes from this meeting are discussed in greater detail below.

Figure 42. Steel shims under vertical Member 12 on March 13, 2018 (indicated with red arrow).[151]

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148 *Id*.

149 NTSB Bridge Factors Report, Attachments 30 and 32.

150 *Id*.

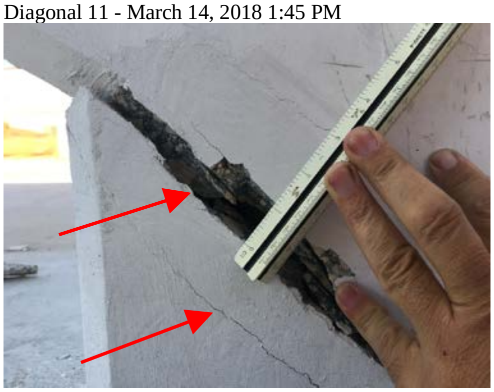
151 NTSB Bridge Factors Report, Photo 87.

3.8.1 Cracking at Nodes 1/2 and 11/12

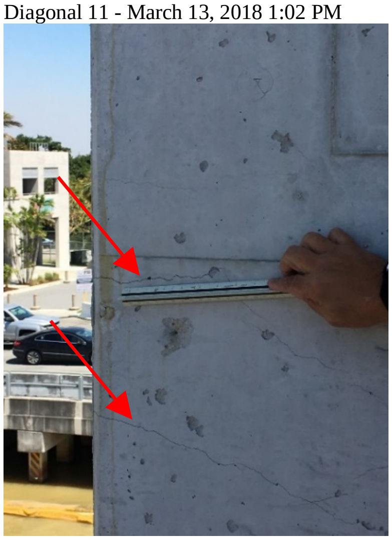
Figure 43 shows cracks observed near the bases of the Truss end Members and north side of Member 12 after the erection of the Main Span and destressing of PT bars in Members 2 and 11.


Diagonal 11 - March 14, 2018 1:45 PM


Diagonal 2 - March 13, 2018 1:44 PM


Diagonal 11 - March 13, 2018 1:02 PM


Diagonal 11 - March 13, 2018 1:17 PM


Vertical 12 - March 14, 2018 1:51 PM


Type II diaphragm and vertical 12 - March 14, 2018 1:51 PM

Figure 43. Cracks observed by BPA near bases of Truss end Members after destressing of PT in diagonal Members 2 and 11 (indicated with red arrows).[152]

3.8.2 Cracking in North Diaphragm

Figure 44 shows the condition of the Type II Diaphragm at the north end of the Main Span after erection of the Main Span and destressing of PT bars in Members 2 and 11.

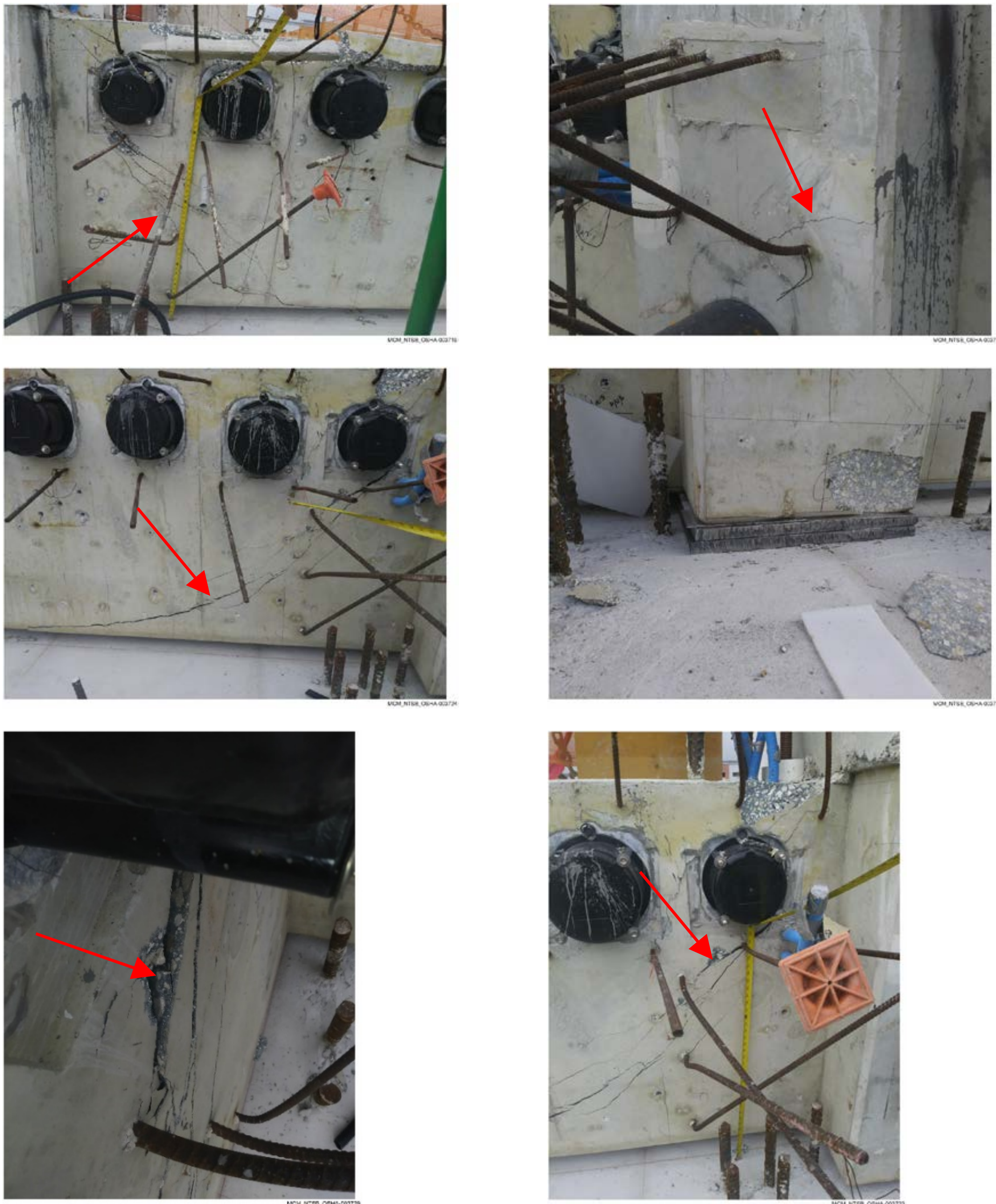


Figure 44. Type II Diaphragm condition after erection and destressing (indicated with red arrows).[153]

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[152] *See* BPA Photo Submissions to NTSB, labeling unknown.
[153] MCM Photo Submission to NTSB, MCM (NTSB/OSHA) 003719, 003720, 003723, 003724, 003729, 003734.

3.9 March 15, 2018 Meeting

On the morning of March 15, 2018, FIGG made a presentation[154] in the MCM field office to representatives of MCM, FDOT, FIU, and BPA.[155]

3.9.1 Review of FIGG Presentation

FIGG presented the chronology of the erection of the Main Span and the discovery of cracks on the north end of the Type II Diaphragm, as well as the results of its analysis to determine the cause of the observed cracking. Some of the key points from FIGG's presentation are as follows (emphasis added throughout):

- Slide 3 – FIGG noted that "immediately after the span move was completed, Franklin Hines (FIGG) and the Project CEI staff inspected the Bridge paying particular attention to regions where previous minor cracking had been noted before the move. The pylon diaphragm end of the span was also visually inspected and nothing of particular interest was noted at the time."

- Slides 4-5 – FIGG received notification of cracks and spalls from MCM's email of 4:52 p.m. on 3/12/2018.

- Slide 6 - FIGG noted that the exposure of the Diaphragm and load on the shim were temporary, and that the final condition will encapsulate the cracks and provide restraint.

- Slide 11 – On this slide titled "Safety," FIGG stated they had conducted "supplemental/independent computations to conclude that there is not any concern with safety of the span suspended over the road." FIGG also confirmed that MCM was notified of this finding by Dwight Dempsey of FIGG. The remainder of the presentation summarized FIGG's analysis supporting this conclusion.

- Slide 13 – Identifies the differences between the support conditions of the Main Span in the casting yard compared to the Pylon/North Pier. On the Pylon/North Pier, the Diaphragm was not supported directly under Member 12. After the discovery of cracks in the north face of the Diaphragm, FIGG instructed MCM to shim the center in apparent recognition that the support conditions were meaningfully different.

- Slides 15-16 – FIGG presented a flexural analysis of the Type II Diaphragm. The analysis assumed identical reactions at each of the four bearing pads, which would only be true if the Diaphragm were infinitely stiff. The computed flexural stress of 350 kips per square foot (2,430 psi) suggested that concrete cracking would be expected.

- Slides 17-20 – FIGG showed the results of a strut-and-tie analysis of the Type II Diaphragm. FIGG used a temporary load combination from AASHTO LRFD (Section 5.14.2.3.4a) that is applicable for segmental concrete bridges that was not appropriate for

[154] NTSB Bridge Factors Report, Attachments 30 and 32.
[155] *Id.*

this Bridge, which is not a segmental bridge. This load combination includes a dead load factor of 1.1 rather than the 1.25 factor in the appropriate Strength I combination from AASHTO LRFD Chapter 3. FIGG's analysis did not include the 20 pounds-per-square-foot ("psf") construction live load required in FIU's Design Criteria. FIGG also discussed whether it was appropriate to use a resistance factor of 0.9 or 1.0. Had FIGG included construction live loads and used the load factor of 1.25 per AASHTO LRFD Specifications, instead of 1.1 that only applied to segmental bridges, this design check would have indicated that the Diaphragm had insufficient strength and was non-compliant with the code requirements.

- Slides 21-24 – FIGG presented a conventional flexural analysis of the Type II Diaphragm. FIGG again used a dead load factor of 1.1 that is applicable for segmental bridges and concluded that the cross section had sufficient flexural capacity. Had FIGG used the applicable load factor of 1.25 per AASHTO LRFD Specifications, its analysis would have shown that the section did not have adequate flexural capacity to meet code requirements.

- Slides 25-34 – FIGG presented two methods of checking vertical shear through the Type II Diaphragm, concluding that the available capacity was adequate. Both methods used the shear friction equations from AASHTO LRFD Section 5.8.4. As in the flexural and strut-and-tie analyses, FIGG used a load factor lower than specified in AASHTO LRFD Specifications, leading to non-conservative assessments of compliance with LRFD requirements. FIGG did not evaluate the nominal shear capacity of the Diaphragm using the Sectional Design Model of AASHTO LRFD Section 5.8.3.

- Slide 35 – FIGG incorrectly concluded there was no safety concern because FIGG mistakenly believed that the preceding AASHTO LRFD design checks for the temporary condition were all satisfied.

- Slides 36-39 – FIGG updated its finite element analysis to reflect the temporary support condition on the Pylon Pier and stated that this analysis suggests that cracks like those observed were possible.

- Slide 40 – FIGG showed pictures where cracks were more severe on the north face of the Diaphragm; however, its finite element analysis did not reveal to them how the spalls on the north edge of the Bridge Deck could have occurred.

- Slides 43-45 – FIGG questioned, but provided no explanation, as to how a change in contact support under the end Diaphragm could have produced the spall observed after transport and erection.

- Slide 46-47 – FIGG presented its analysis of the destressing of PT bars in Member 11. This analysis does not seem to consider the increase in PT force of these bars at the beginning of the destressing operation.

- Slide 48 – FIGG concluded that spalls were minor but could not be explained by engineering analyses, and some cracks required epoxy injection.

Overall, FIGG's presentation did not address the large diagonal cracks in Member 11 or cracking/sliding at the Node-to-Deck interface. FIGG's analysis post-cracking was instead focused mostly on Diaphragm cracking, which failed to recognize and analyze the failure of the shear interface as the root cause of the observed damage in both Node 11/12 and the Type II Diaphragm. This indicates that FIGG misinterpreted the destressing (reduction in PT force in Member 11) to be responsible for cracking, as opposed to the increase in PT force at the very beginning of the destressing process.

Importantly, FIGG did not develop a full understanding of the root cause of the observed cracking before prescribing its repair scheme of restressing PT bars in Member 11. And it appears that FIGG's restressing was not rooted in science and was not independently peer reviewed, but was based on a misinterpretation of the sequence of events and timing of cracking.

Further, Engineers representing FDOT and FIU were present at the meeting and did not take issue with any of FIGG's technical analysis methods or findings. FDOT and FIU engineer representatives also did not object to proceeding with the repair (restressing) immediately after the March 15, 2018 meeting, despite evidence that FIGG did not truly understand the root cause of the observed distress.

3.9.2 Meeting Minutes by BPA

BPA's minutes[156] summarized FIGG's presentation and recorded the questions and answers that arose during the meeting. Some of the key notes from the document are as follows:

- FIGG pointed out that the cracks looked more significant in person than on photographs after their site inspection that was performed prior to the presentation.
- FIGG assured that there was no concern with safety of the span suspended over the road.
- A temporary mechanism to capture the nodal zone and the time-frame to deliver the plan was discussed.
- When asked specifically if temporary shoring was required, FIGG responded that it was not necessary. Rather than providing additional strength to carry the weight vertically, FIGG preferred to provide a mechanism to transfer the load longitudinally from Node 11/12. Per FIGG, steel channels to connect Nodes 11/12 and 9/10 and PT bars to capture some of that force was better than vertical support.
- FDOT requested a copy of FIGG's presentation to give to its structural group.
- FIGG mentioned that the PT bars in their permanent condition have less stress than under construction condition.
- MCM asked BPA if the cracks had been growing, and BPA answered that they had been growing daily.
- When asked by FDOT whether FIGG would continue to investigate why the cracking had occurred, FIGG responded that all they "know is that it just happened." This indicates that FIGG's repair plan of restressing the PT bars was more intuitive rather than based on

[156] NTSB Bridge Factors Report, Attachment 30.

rigorous analysis. Nevertheless, FDOT did not require that the Main Span be shored, even though FIGG conceded that it did not understand the Bridge's behavior.

- BPA asked if the analysis had been peer reviewed and "requested that it wanted more eyes on this." FIGG concurred. FDOT did not require the restressing operation to be delayed until such a peer review could be performed, even though the re-application of PT in Member 11 was a deviation from the original design.
- MCM noted that they would expedite the construction of the Pylon Diaphragm and Back Span.

3.9.3 Meeting Minutes by FIGG

After the collapse, FIGG prepared an *alternative* set of meeting minutes[157] that it labeled "corrected." Some of the notable differences between FIGG's version of the minutes and BPA's version are as follows:

- The title of the meeting was changed to "Temporary Construction Loading Condition" rather than "FIGG Structural Analysis Presentation."
- FIGG removed the phrase "and temporary mechanism to capture nodal zone" from the first bulleted item in the overview, although the section dealing with the capture of nodal zone in the Question and Answers section was not removed.
- FIGG omitted BPA's bullet point stating, "FIGG pointed out that the cracks look more significant in person than on photographs after site inspection performed prior to the presentation," and added some words to this effect on the bottom of the second page.
- FIGG removed itself from the parties that inspected the Bridge after the move and emphasized that FIGG was present as an observer. *This change by FIGG contradicts the statement made by Dwight Dempsey of FIGG in an email to MCM on March 13, 2018, where he states, "FIGG's inspection of the Main Span in this area after the bridge move did not observe this behavior."*[158]
- FIGG changed the date and added the time at which it received email correspondence from MCM regarding cracks.
- FIGG substituted "Based on the discussions at the meeting no one expressed concern with safety of the span suspended over the road," in the place where BPA stated that "FIGG assured that there was no concern with safety of the span suspended over the road." *This change by FIGG seems to misrepresent Slide 11 of FIGG's presentation where it stated, "FIGG had conducted sufficient supplemental/independent computations to conclude that there is not any concern with safety of the span over the road."*
- FIGG added an item stating that its analysis did not predict the spalled areas.

[157] FIGG 3-15-18 Meeting Minutes.pdf.

[158] NTSB Bridge Factors Report, Attachment 31.

- FIGG amplified the item regarding the importance of casting the Pylon by stating that this would increase the reserve strength of the structure and that MCM was tasked with looking into expediting this activity.
- FIGG expanded the description of its response to the question regarding temporary shoring to include a reservation against supporting the structure at a location away from a node and to include discussions of tying Node 11/12 to Node 9/10 with steel channels.
- FIGG classified work related to tying Node 11/12 to Node 9/10 as an “enhancement.”
- FIGG expanded the discussion of the grout to be placed under the Type II Diaphragm.
- FIGG changed its response regarding any predictions of diagonal cracking in the Diaphragm to say that it expected uniform cracking on both sides of the Diaphragm.
- FIGG removed itself from the item in which no response was given to BPA regarding the implementation of a crack monitoring plan.
- FIGG changed an item that related to a question from MCM to BPA regarding crack length in which BPA responded that cracks were growing daily to a statement that BPA and MCM confirmed to FIGG that only small changes had occurred.
- In response to the question about finding out why the cracking occurred, FIGG added that it is prioritizing developing options to improve the situation.
- FIGG removed a statement of its reassurance that means and methods would be considered for the placement of concrete under the Pylon Diaphragm.
- FIGG removed the item related to potential peer review of its analysis.

3.9.4 Meeting Minutes Comments by MCM to NTSB

MCM provided comments to NTSB regarding the meeting minutes drafted by BPA through its counsel, Squire Patton Boggs, in a letter dated April 2, 2018.[159] The letter clarified that revisions to meeting minutes would normally be suggested to the preparer by the various parties in attendance under ordinary circumstances. The situation at the time justified deviating from this usual process, and MCM submitted its proposed revisions to NTSB to assist in its investigation. MCM proposed the following clarifications to BPA’s minutes:

- MCM shared with the attendees that FIGG had contacted FDOT regarding the cracks, leaving a voicemail that had yet to be returned.
- BPA (as CEI) had coordinated with FDOT regarding the lane closures required to position the crane for the restressing operation.
- MCM clarified that it emailed documentation regarding the cracks to FIGG on March 12, 2018, rather than March 13 as originally indicated in BPA’s minutes.
- In its response to the question about whether the temporary mechanism to capture Node 11/12 would have to remain in the structure, FIGG’s response also indicated that its analysis was not yet complete.

[159] MCM Correspondence to NTSB (D. Walsh) through counsel, April 2, 2018.

- The question to FIGG about load restrictions on the north end of the Main Span was asked by both the CEI and MCM.
- For the question from the CEI about whether there will be a crack monitoring plan, MCM stated that the question was asked to FIGG rather than to both FIGG and MCM, and that the CEI asked FIGG to perform such monitoring.
- MCM proposed that the response to the question above should have been recorded as, "FIGG had no response. MCM commented that it already had measures in place to monitor crack movement" rather than stating that neither FIGG nor MCM had any response.
- MCM said that they joined the CEI in asking the question about whether FIGG would stay for the restressing operation.
- MCM's response to the CEI's request for the restressing plan included a description of the procedure, including the alternation between bars and the force increments. Furthermore, MCM said that "FIGG confirmed this and added that it is recommended and prudent to do so to bring the span to its previous state where it sat on temporary supports."
- When asked by the CEI about its schedule for constructing the Pylon Diaphragm and Back Span, MCM stated that it was following both the schedule and sequence of construction and that it would specifically expedite the construction of the Pylon Diaphragm and the Back Span.

With these minor clarifications, MCM agrees with BPA's version of the meeting minutes.

3.10 Tendon Restressing

After reviewing crack photographs and performing additional analysis, FIGG instructed MCM to have the PT bars restressed in Member 11 as soon as possible. These instructions were first delivered in an email from FIGG to MCM on March 13, 2018 at 5:18 p.m. ET.[160] The restressing was to be performed by Structural Technologies and progress in 50-kip increments, alternating between the top and bottom bar, until the full design force of 280 kips was achieved in each bar.[161] FIGG expected that the cracks on the Type II Diaphragm would remain unchanged or close to some degree.[162] FIGG instructed MCM to monitor these cracks and halt the restressing operation if the cracks grew. FIGG did not make any changes to these instructions after inspecting the Bridge on March 15, 2018[163,164] or after the presentation and discussion in the 9:00 a.m. ET meeting that morning.

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[160] NTSB Bridge Factors Report, Attachment 27.

[161] *Id*.

[162] *Id*.

[163] NTSB BPA Interview Transcript, Carlos Chapman, 8:13-22; 12:9-14.

[164] NTSB BPA Interview Transcript, Jose Morales, 37:11-25.

Based on the testimony provided by Structural Technologies and MCM personnel, who were at the top of the Canopy at the time of restressing, the PT force was correctly applied in small increments alternating between the top and the bottom PT bars at each increment.[165,166] And Jose Morales, who was at the Deck level during restressing, did not observe any change in cracks during restressing.[167] Nevertheless, the Bridge was reported to have failed just as the Structural Technologies crew took off the ratchet after applying the very last load increment.

PT bars in Member 11 were stressed in January 2018, destressed on March 10, 2018, and restressed again on March 15, 2018. Each time these bars were stressed to a high fraction of their capacity. This repeated loading and unloading of high-strength threaded bars is highly unusual[168] and could have led to fracture of these bars.[169]

[165] NTSB MCM Interview Transcript, Pedro Cortes, 8:1-23.

[166] NTSB Structural Technologies Interview Transcript, Ramoy Goulbourne, 17:3-10.

[167] NTSB BPA Interview Transcript, Jose Morales, 61:15-19.

[168] Samuel Nunez of Structural Technologies stated that to his knowledge it was not common practice to re-stress PT steel. NTSB Structural Technologies Interview Transcript, Samuel Nunez, 59:3-10.

[169] The custom-made threaded PT bars used in Members 2 and 11, have the same material strength and other characteristics as the standard, ASTM 3125 A490 high-strength structural bolts. The reuse of A490 bolts is prohibited by the Research Council on Structural Connections' "Specification for Structural Joints Using High-Strength Bolts" because they cannot safely undergo multiple high stress cycles. Although there is no evidence that PT bars fractured during restressing operation, restressing of Member 11 PT bars should not have been specified. Had there been a recognition of the similarities between the custom-made PT bars and A490 threaded bolts or a prohibition against re-using PT bars as there is against re-using A490 bolts, the sequence of events leading to the failure would have been interrupted.

4 Traffic Control

Numerous questions have been raised in the NTSB investigation and by the public asking why traffic was allowed to proceed under the Bridge during implementation of FIGG's remedial instructions on March 15, 2018. This section thus discusses how FDOT, FIU, FIGG, BPA and MCM carried out their respective duties with respect to traffic control on the day of the collapse.

4.1 Introduction

FDOT has authority over Florida state rights-of-way and bridges and may therefore direct or authorize partial or complete road closures as necessary.[170] FDOT owns and has such authority over Tamiami Trail (SW 8th Street), which is designated as State Route 90. Under the contract between FIU and BPA (as CEI), BPA's scope of services for the Project included on-site traffic control authority in a collective effort between FIU (through BPA) and FDOT.[171]

For its part as General Contractor for the Bridge, MCM was simply obligated to follow all maintenance of traffic ("MOT") standards and specifications for the Bridge, and to comply with BPA and FDOT directives.[172]

4.2 FIU and FDOT Authority Over Traffic Control

As mentioned above, BPA's general authority for the Project, including over traffic control, was a collective effort between BPA and FDOT.[173] For example, under CPAM section 4.1.4, FDOT "must ensure the Consultant CEI (consultant) [BPA] is performing services in accordance with the scope of services and the contract."[174] In addition, under CPAM section 4.1.5, "the authority of the CEI firm's [BPA's] lead person . . . shall be identical to [FDOT's] Resident Engineer and Project Administrator respectively and shall be interpreted as such."[175] Thus, BPA's authority was essentially identical to FDOT's.[176] BPA and FDOT were both required to implement immediate

170 NTSB Bridge Factors Report, p. 160.

171 *Id*. at p. 161. The relationship between FIU, FDOT, and BPA in this regard was governed by the contract between FIU and BPA, *see* NTSB Bridge Factors Report, Attachment 17 at Ex. B, as well as FDOT's Construction Project Administration Manual ("CPAM").

172 *See* Traffic Control Plans ("TCP"), Cover Sheet R-1. Such standards and specifications included the 2015 FDOT Standard Specifications for Road and Bridge Construction, the 2015 FDOT Design Standard Index Drawings, and the signed and sealed TCP plans. Alfredo Reyna, P.E., who was listed in the TCP as the FDOT Project Manager, was present at the March 15, 2018, meeting.

173 NTSB Bridge Factors Report, p. 161; NTSB Bridge Factors Report, Attachment 17 at Ex. B; CPAM.

174 CPAM, Florida Department of Transportation State Construction Office, Effective: July 1, 2002, Revised: August 27, 2018, Section 4.1 Administration of Consultant CEI Contracts, p. 4.1.4.

175 *Id*. at p. 4.1.5.

176 NTSB Bridge Factors Report, p. 162.

remedial action if any deficiencies in the Project were indicated.[177] Notably, however, BPA "did not have complete authority to act on its own," but rather collectively with FDOT and FIU.[178]

Under CPAM section 9.1.8, "[a]ny MOT deficiency that is considered a severe hazard and life threatening will require immediate corrective action by the Contractor. Failure to correct the hazard immediately is basis to shut down the project and obtain other means to correct the hazard. Prior to any such shut down, document the deficiency with photographs sufficient to support the action."[179] In the contract between FIU and BPA, BPA was required to identify and advise FIU of any significant omissions, substitutions, defects, and deficiencies and the corrective action needed.[180] Likewise, by virtue of the Design-Build contract, FIGG was required to promptly notify MCM of any defects, deficiencies, deviations, omissions, or violations observed by FIGG in the construction of the Project, and make recommendations to MCM on how to proceed.[181]

4.3 MCM Obligations for Traffic Control

Generally, although it was not primarily responsible for traffic control related to the Project, MCM did have responsibilities related to maintaining traffic within the limits of the Project for the duration of the work, including constructing and maintaining detours and furnishing traffic control devices during construction, as needed for safe and expeditious movement of traffic *as specified in the approved plans*.[182] Additionally, under 2015 FDOT Design Standard Index 600, all projects and works on highways, roads, and streets shall have a TCP and all work shall be executed under such established plan and FDOT-approved procedures.[183] In accordance with these specifications and standards, a specific TCP was provided for three separate phases of Bridge construction,[184] and MCM had the necessary personnel and equipment to properly implement the TCP during all three construction phases. However, MCM did not have authority to unilaterally implement a total road closure and was limited to implementing only the approved TCP or any other FDOT-approved procedure.

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[177] *See* NTSB Bridge Factors Report, Attachment 17 at Ex. B; *see also* CPAM, Florida Department of Transportation State Construction Office, Effective: July 1, 2002, Revised: August 27, 2018, Section 4.1 Administration of Consultant CEI Contracts, p. 4.1.7.6.

[178] NTSB Bridge Factors Report, p. 162.

[179] CPAM, Florida Department of Transportation State Construction Office, Effective: July 1, 2002, Revised: August 27, 2018, Section 9.1 Maintenance of Traffic (MOT), p. 9.1.8. Not only did BPA and FDOT not identify any MOT deficiencies in the first instance, neither did they document any such deficiency as required by the CPAM.

[180] *See* NTSB Bridge Factors Report, Attachment 17.

[181] *Id*. at Attachment 18.

[182] *See* 2015 FDOT Standard Specifications for Road and Bridge Construction, Sections 102-1, 102-5.1, and 102-9.

[183] *See* 2015 FDOT Design Standard Index 600.

[184] Phase I maintained traffic on SW 8th Street with all through travel lanes open (4 eastbound lanes and 3 westbound lanes) while construction work activity took place on the north shoulder. *See* TCP, Sheets R-40 through R-45. A Detour Plan was provided for the full closure of SW 8th Street during the time required for moving the Bridge over the travel lanes to its permanent location. *See* TCP, Sheet R-46. Phase II maintained traffic on SW 8th Street with all through travel lanes open after the Bridge had been positioned over the travel lanes. *See* TCP, Sheets R-47 through R-49.

4.4 MCM's Appropriate Action

At the March 15, 2018 meeting, all parties responsible for identifying MOT deficiencies and notifying MCM of any need for changes to the existing TCP were present. Most notably, this included engineering representatives for both FIU and FDOT, as well as the EOR for the Project, FIGG, which was required to report to MCM any problems under the Design-Build contract, and who, per Florida's Administrative Code, "[p]ersonally makes engineering decisions or reviews and approves proposed decisions prior to their implementation, including the consideration of alternatives, whenever engineering decisions which could affect the health, safety and welfare of the public are made."[185]

Critically, no party at the meeting identified *any* MOT deficiencies, let alone any issue that would have led to a full lane closure. Rather, FIGG assured all parties, as it had on multiple occasions since early February 2018, that there were no structural or other safety concerns associated with the Bridge. None of the other qualified engineers present disagreed with FIGG's conclusions and recommended closing traffic, despite FDOT's clear authority to demand a full closure if they deemed it appropriate to correct a MOT deficiency. Importantly, MCM was the only party in the March 15, 2018 meeting that was not a qualified engineer or, in other words, the only party that did not have the expertise sufficient to determine whether the road needed to be closed due to the cracking observed on the Bridge.

Therefore, MCM never had reason to seek closure of all lanes on SW 8th Street, and it certainly could not have done so unilaterally—under these circumstances or otherwise. MCM had not received any request, demand, or notification from FDOT or FIGG to implement a full closure, nor had it received authority to unilaterally implement a full closure.[186] As provided for in the standards and contracts outlined above, MCM had reason to rely on the representatives of the EOR, together with the failure to object by those present who were involved with the Project for the explicit purpose of providing such expertise to proceed with FDOT-approved MOT. As a result, MCM acted reasonably under the circumstances.[187]

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[185] *See* Florida's Administrative Code, 61G15-18.011 Definitions (1)(a)1.

[186] In fact, MCM actually received pushback from FDOT for any lane closures on the day of the collapse. Specifically, on March 14, 2018 at 1:08 p.m. ET, Mr. Saud Khan, FDOT District Six Maintenance of Traffic Specialist, sent an e-mail to MCM (Edwin Vega) criticizing the request to close two lanes of westbound traffic, as it is heavy traffic season and the roadway cannot handle the volumes. Mr. Khan would only allow a single lane closure in each direction during the daytime. However, BPA authorized the westbound double-lane closure under the blanket permit, and notified Mr. Khan. On March 15, 2018 at 9:25 a.m. ET, Mr. Khan sent a critical e-mail to BPA (Rafael Urdaneta, Jose Morales, Carlos Chapman) and MCM (Rodrigo Isaza, Ernie Hernandez, Edwin Vega), indicating that he regretted allowing a daytime two-lane closure because heavy back-ups resulted, and that in the future all lane closure requests must be submitted to BPA before coming to FDOT.

[187] In this regard, it is noteworthy that all instances of bridge closures cited in the NTSB Bridge Factors Report, pp. 166-167, involved shut down by the <u>EOR</u> and/or <u>CEI</u> of the project.

5 Engineering Analysis and Discussion

As part of MCM's investigation of the failure, our team evaluated the Bridge's structural behavior and design details. MCM's analysis included estimating the magnitudes of forces in critical components and connections on the Bridge and evaluating the capacity of those components and connections to resist those forces. These efforts contributed to our overall assessment of potential failure modes and mechanisms, which was then validated by comparing the results to pre- and post-collapse photographs and other factual information discussed herein.

For purposes of the load and resistance calculations discussed below, our analysis can be divided into the following two contexts:

> **Factored loads and factored capacities:** Calculations using factored loads and factored capacities are consistent with those used in structural *design*. Anticipated loads are *increased* by load factors to account for loading uncertainty and to provide a margin of safety. Anticipated member capacities are *reduced* by resistance factors to account for strength uncertainty and to provide a further margin of safety. Results presented below for this context are appropriate for evaluating the design by FIGG.
>
> **Nominal loads and nominal capacities:** Calculations using nominal loads and nominal capacities are used to evaluate structural *failure*. Nominal loads are intended to reflect actual design loading conditions, and nominal capacities are intended to approximate actual capacities. Results presented below for this context are appropriate for evaluating structural failure of critical components and connections of the Main Span.

5.1 Load Effects

MCM calculated the forces in critical members and connections using three independent methods of varying sophistication: (1) hand calculations; (2) finite element analysis ("FEA") with beam elements (using SAP2000 software); and (3) FEA with brick elements (using ABAQUS software). Hand calculations can be performed rapidly, are reliable, and are not as vulnerable to data entry errors as complicated computer analyses. Computer analyses, on the other hand, require more time and computational effort, but typically require fewer simplifying assumptions than do hand calculations. It is thus common practice to use hand calculations to check certain critical results obtained using computer analyses, which MCM did.

The analyses in SAP2000 and ABAQUS were analogous to FIGG's LARSA 4D and LUSAS analyses, respectively (as discussed in Section 6). Using these similar technological approaches allowed MCM to replicate some of FIGG's processes in order to see whether the different approaches had a meaningful influence on the results. The loads, load cases, load factors, and load combinations considered in the MCM analyses were in accordance with Chapter 3 of AASHTO LRFD.

5.1.1 Hand Calculations

In its Simply-Supported condition, the Main Span's member loads could be reasonably approximated as an ideal truss,[188] which allows the magnitudes of the forces in the end diagonals to be computed with reasonable accuracy using hand calculations. Under ideal truss assumptions, Truss Web Members 1 and 12, as well as the end Canopy segments, would be "zero force members," meaning they would not carry any significant axial force. Figure 45 shows the configuration of the Truss under ideal assumptions.

In this case, most of the Bridge's weight reaches the end supports (nodes A and F in Figure 45) by passing through end Diagonal Members 2 and 11. Only the weights of end vertical Members 1 and 12 and half of the end Deck and Canopy segments would reach the supports without first passing through the end Diagonals Members. Therefore, of the 1900-kip (950 ton) weight of the Main Span, all but 360 kips (180 tons) must pass through Diagonal Members 2 and 11 as compressive force. If the remaining 1,540 kips is evenly distributed between the north and south supports, the nominal dead load axial forces in Diagonal Members 2 and 11 would be approximately 1,890 kips and 1,450 kips, respectively, excluding the effects of PT. Adding the prescribed 560 kips of PT in each of these Diagonals, as set forth in FIGG's design plans,[189] increases the nominal axial compressions to 2,450 kips and 2,010 kips, respectively.

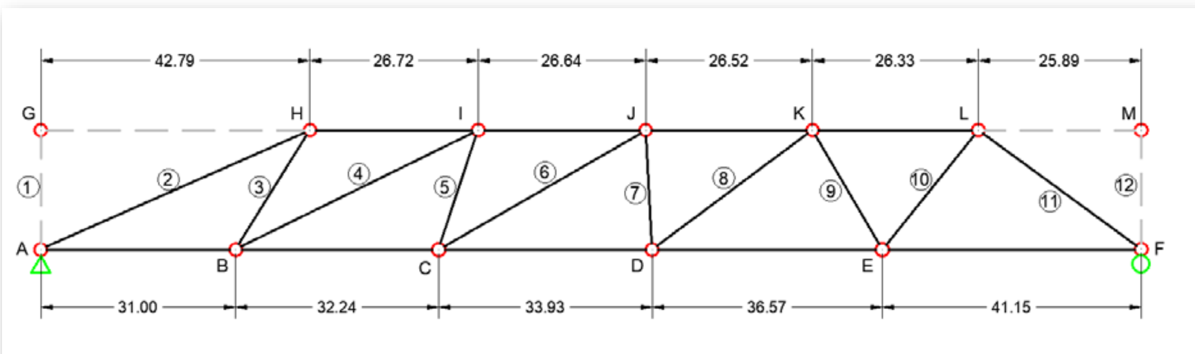


Figure 45. Idealized Main Span Truss.

The axial force in Diagonal Member 11 can be resolved into vertical and horizontal components. The horizontal force is the critical component here because it contributes to interface shear in the Truss-to-Deck connection. The nominal horizontal component of the diagonal axial force is 1,710 kips for Diagonal Member 11. This horizontal force must be resisted by shear friction at Node 11/12, . As discussed later, the value of 1,710 kips for interface shear at Node 11/12 arrived at by simple hand calculation greatly exceeds the value of 571 kips that FIGG extracted from its LUSAS finite element model (*see* Section6.2.1). A hand analysis should have thus revealed to FIGG that it had greatly underestimated the shear force at this critical connection.

[188] This approximation does not account for member end rigidity and mid-joint loading, but these conditions are accounted for in the computer analyses described below.

[189] RFC Superstructure Plan Sheet B-69, FBE000262.

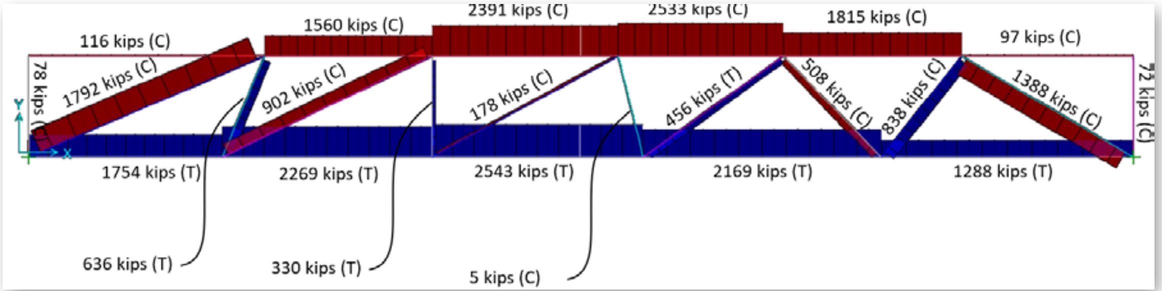
5.1.2 SAP2000 Analysis

As illustrated in Figure 46, our team then used SAP2000 to compute forces in members of the Main Span in its Simply-Supported conditions.[190] Most pertinently, member axial and shear forces were used to evaluate the interface shear at Node 11/12. This analysis is more accurate than simple hand calculations because it does not require the idealizations and assumptions needed to quickly evaluate an ideal truss, and because the rotational restraint at joints, precise distribution of weight, and effects of PT can be explicitly included in the analysis.

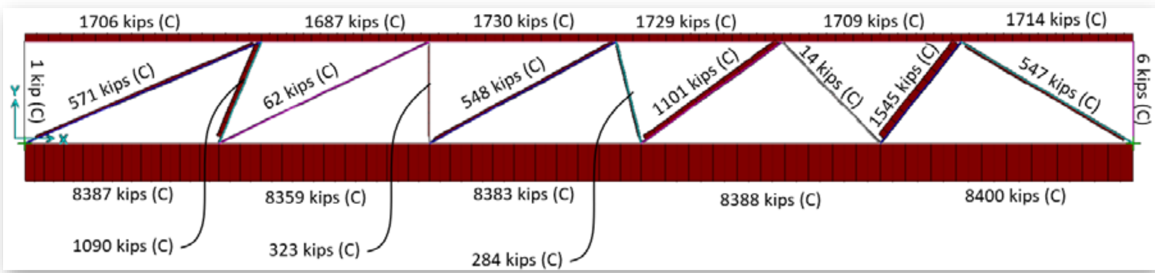
Comparing the SAP2000 results with our hand calculations shows good agreement. For example, the total nominal axial compression in Member 11 in the SAP2000 analysis is 1,935 kips compared to the 2,010 kips arrived at by hand calculations, which is within 4% of the hand-calculated result.[191] Resolving member forces from the SAP2000 analysis also results in a nominal interface shear at Node 11/12 of 1,738 kips, as compared to the 1,710 kips arrived at by hand calculations (also in agreement within 2%). Overall, the results obtained from our SAP2000 computer analysis generally corresponded with the results of hand calculations, thus demonstrating that the Node 11/12 interface carried significantly more force than considered by FIGG in their design calculations.

[190] Dead load and PT loads have been shown separately to illustrate the magnitude of their respective contributions, but dead load was never applied to the Main Span in the absence of PT. The PT in the Deck, Canopy, and Diagonal Members was applied before the removal of the shoring and forms.

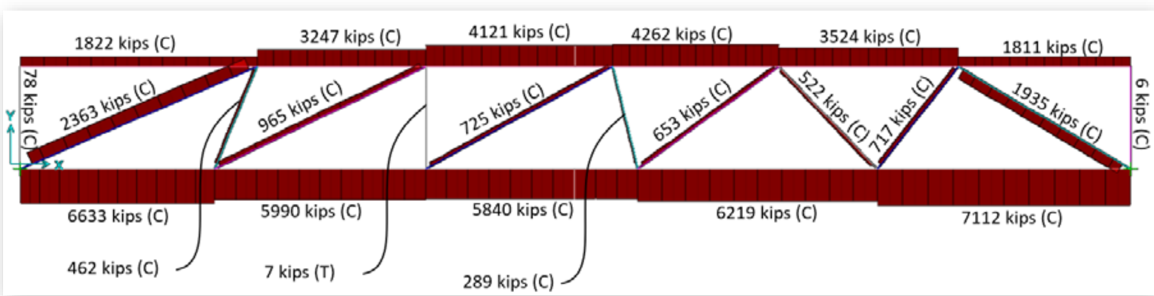
[191] It is expected that the SAP2000 analysis would result in a lower compressive force in Diagonal Member 11 since the rigidity of the connection at the top of vertical 12 in the SAP2000 model will attract some force that is diverted into Diagonal Member 11 under ideal truss assumptions. Furthermore, the SAP2000 model reflects the fact that there would be distribution of the PT force in Diagonal Member 11 into other elements, which the hand calculations did not show under ideal truss assumptions.



Dead Load Only
(This condition never existed in the field by itself)



Post-Tensioning Only
(Applicable before removal of shoring and forms)



Dead Load + Post-Tensioning
(Applicable after removal of shoring and forms)

Figure 46. SAP2000 Analysis Nominal Axial Force Results for Simply-Supported Main Span.

5.1.3 ABAQUS Analysis

To further substantiate the hand calculation and SAP 2000 findings, the team next analyzed the Main Span using FEA software called ABAQUS. In addition to providing a third independent method of verification, this model allowed for a direct extraction of interface shear forces in Truss connections and an evaluation of stresses in the Type II Diaphragm. The ABAQUS results correspond with our hand calculations and with those obtained using SAP2000.

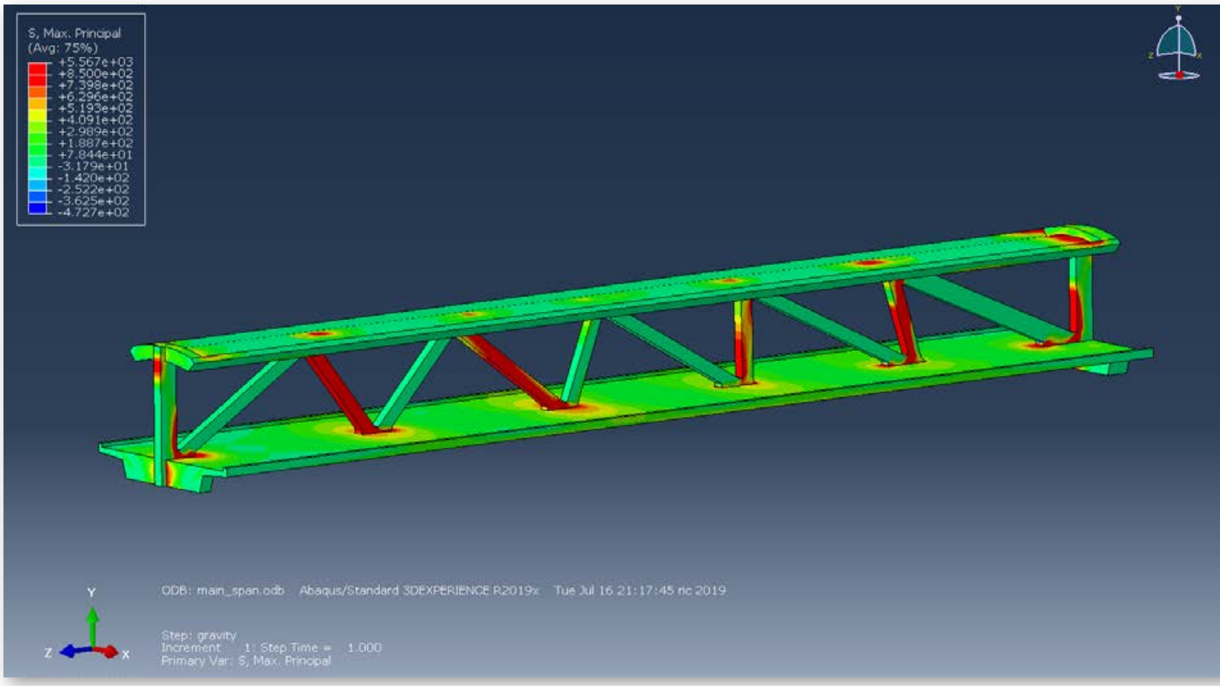


Figure 47. ABAQUS model of Simply-Supported Main Span.

Figure 47 shows a rendering of the ABAQUS model with color contours of principal stress, and Figure 48 shows the directly-extracted interface shear force (reported in pounds) just above the Deck at the bases of Members 11 and 12 for nominal dead load ("DL") with no PT. This nominal DL interface shear value from ABAQUS combined with the effect of PT from hand calculations resulted in a total nominal interface shear value of 1,686 kips, which corresponds with 1,710 kips and 1,738 kips from hand calculations and SAP2000 analysis, respectively.

Interface shear at Node 11/12 from all three models are within 3% of each other, demonstrating the utility and accuracy of each model in this analysis.

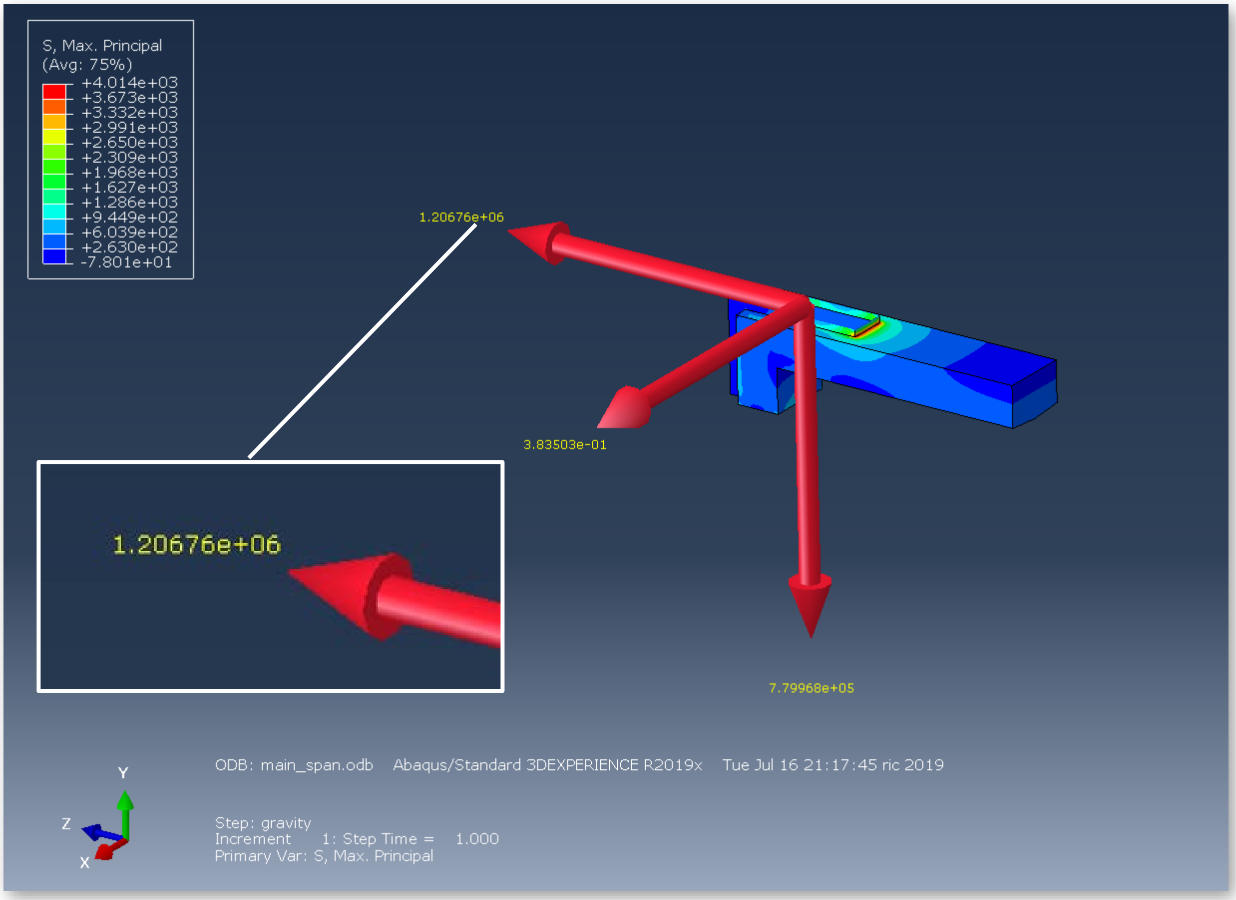


Figure 48. Node 11/12 interface shear force extraction from ABAQUS (DL only).

As shown in Figure 49, our team also developed a separate ABAQUS model to further investigate the behavior of the Type II Diaphragm at the north end of the Main Span, which had the following characteristics:

- Beam elements were used south of Node 10/11 to reduce the computational demands.
- The portion of the Main Span north of Node 10/11 was modeled with quadratic solid elements.
- The beam elements were coupled to the solid elements with kinematic restraints.
- The HDPE shims under the Type II Diaphragm were explicitly modeled.
- Translational restraint in all three directions was provided at the south end of the Main Span. At the Type II Diaphragm, vertical restraint was provided under the HDPE shims, and lateral (transverse) restraint was only provided at the symmetry line of the structure.
- Vertical voids for the PT bars adjacent to Member 12 and a longitudinal void for the drain passing through the center of the Diaphragm were modeled.
- Transverse and longitudinal PT in the Deck and Canopy were modeled with externally-applied forces and pressures. Web Member PT was not modeled.

Figure 50 shows the resulting contours of principal stress. Any color warmer than blue indicates the presence of some tensile stress. Red colors indicate principal tensile stresses over 850 psi, which corresponds to the approximate threshold for concrete cracking in the Superstructure.

According to this analysis, cracking was anticipated on the north side of Member 12, at the south corner of Node 11/12, and on the north face of the Type II Diaphragm. These results are validated by the observations of cracks prior to collapse, as shown in Figure 51 and Figure 52. <u>The extent of the flexural cracks on the north face of Member 11, however, raises questions about the adequacy of the design of this Member that will be addressed in Section 6.</u>

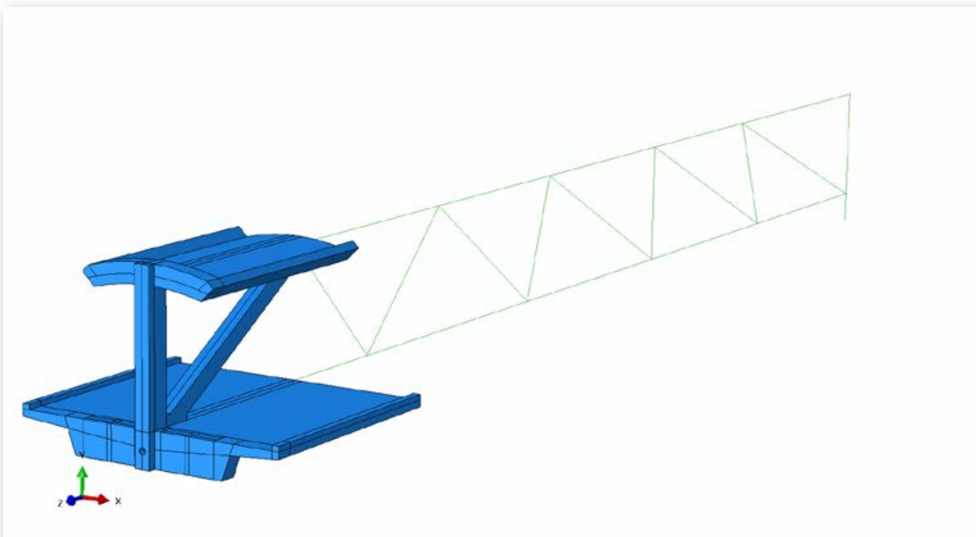


Figure 49. ABAQUS model for investigating Type II Diaphragm.

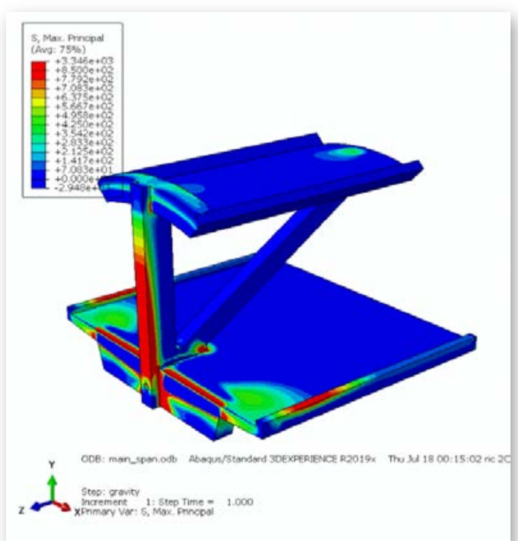


Figure 50. Principal stresses from refined ABAQUS model.

Figure 51. Flexural cracks on north face of vertical Member 12 (indicated with red arrows).[192]

Figure 52. Spalling along top corner of Type II Diaphragm.[193]

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[192] NTSB Bridge Factors Report, Photo 101.

[193] NTSB Bridge Factors Report, Photo 63.

As expected, our three analyses produced comparable interface shear forces at Node 11/12 (agreement within 3%), each of which (singularly and in combination) demonstrates that FIGG's calculations underestimated the overall load on the relevant Bridge elements.

Given that the hand calculations and simpler SAP2000 model were able to provide accurate results, more complex FEA software such as ABAQUS (or in FIGG's case, LUSAS) was not required to compute reliable connection forces for the Main Span in its Simply-Supported condition. FIGG thus unnecessarily increased the complexity of its analysis effort (increasing opportunities for error) by using LUSAS to determine connection forces.

5.2 Resistance Capacity

Section 5.8.4 of AASHTO LRFD provides methods for evaluating the capacity of concrete structures to carry shear forces across interface planes. These interfaces may consist of existing or potential cracks, joints between dissimilar materials, planes separating concrete placed at different times, or the joints between different elements of a structure. The construction joints between the Truss Web Diagonals and the Deck or Canopy are examples of such concrete interfaces. Figure 53 illustrates a concrete interface (outlined by blue dashes) subject to shear force.

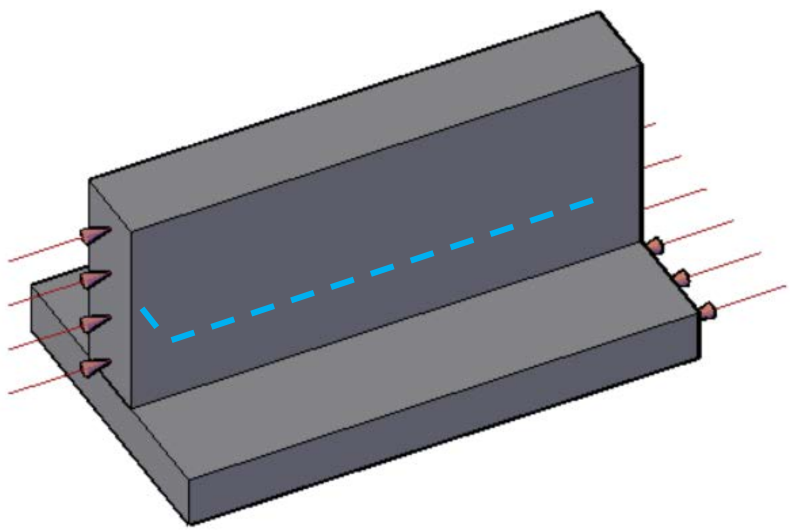


Figure 53. Schematic of Concrete Shear Interface.

Interface shear capacity depends on a number of factors, including the compressive strength of the concrete, the quantity and strength of steel reinforcement crossing the interface, the size of the interface, whether the concrete at the interface was placed at the same time (monolithically, *i.e.*, not a construction joint), or, if not, whether the hardened concrete at a construction joint was intentionally roughened. Interfaces that are not intentionally roughened to an amplitude of ¼-inch are deemed to have lower load-carrying capacity according to AASHTO LRFD, and the specification does not provide any method for estimating interface shear strengths for intentional roughening of other amplitudes.

Designers are of course free to specify other roughness amplitudes, but in such cases the specifications simply would not permit interpolation for roughness amplitudes less than ¼-inch or extrapolation for roughness amplitudes greater than ¼-inch. From the perspective of the AASHTO LRFD design equations, any roughness amplitude smaller than ¼-inch must be treated as not intentionally roughened for the purposes of design calculations.[194] Excerpts from AASHTO LRFD Section 5.8.4 are shown in Figure 54 and Figure 55.

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c) \qquad (5.8.4.1\text{-}3)$$

The nominal shear resistance, V_{ni}, used in the design shall not be greater than the lesser of:

$$V_{ni} \le K_1 f'_c A_{cv}, \text{ or} \qquad (5.8.4.1\text{-}4)$$

$$V_{ni} \le K_2 A_{cv} \qquad (5.8.4.1\text{-}5)$$

in which:

$$A_{cv} = b_{vi} L_{vi} \qquad (5.8.4.1\text{-}6)$$

where:

A_{cv} = area of concrete considered to be engaged in interface shear transfer (in.^{2})

A_{vf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv} (in.^{2})

b_{vi} = interface width considered to be engaged in shear transfer (in.)

L_{vi} = interface length considered to be engaged in shear transfer (in.)

c = cohesion factor specified in Article 5.8.4.3 (ksi)

μ = friction factor specified in Article 5.8.4.3 (dim.)

f_y = yield stress of reinforcement but design value not to exceed 60 (ksi)

P_c = permanent net compressive force normal to the shear plane; if force is tensile, P_c = 0.0 (kip)

194 FIGG drawings B-24B and B-25 specified a surface roughness amplitude of ¼-inch at Substructure construction joints, drawing B-104 specified an amplitude of 1/8-inch for construction joint at top of the end bent, and drawings B-37, B-38, B-41, B-42, B-47, B-48, B-49, and B-82 did not define any amplitude for surface roughening.

f'_c = specified 28-day compressive strength of the weaker concrete on either side of the interface (ksi)
K_1 = fraction of concrete strength available to resist interface shear, as specified in Article 5.8.4.3.
K_2 = limiting interface shear resistance specified in Article 5.8.4.3 (ksi)

Figure 54. Excerpts from AASHTO LRFD Section 5.8.4.1.[195]

The following values shall be taken for cohesion, c, and friction factor, μ:

- For normal-weight concrete placed monolithically:

  c = 0.40 ksi
  μ = 1.4
  K_1 = 0.25
  K_2 = 1.5 ksi

- For normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in.

  c = 0.24 ksi
  μ = 1.0
  K_1 = 0.25
  K_2 = 1.5 ksi

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

  c = 0.075 ksi
  μ = 0.6
  K_1 = 0.2
  K_2 = 0.8 ksi

Figure 55. Excerpts from AASHTO LRFD Section 5.8.4.3.

[195] *See id.*

In conformity with these specifications, our team evaluated the interface shear friction capacity of the connection between Members 11 and 12 with the Bridge Deck at Node 11/12. Three general failure surfaces were investigated:

- Case 1a: where the concrete interface was a single horizontal plane extending from the south end of the node at the chamfer under Member 11 to the north face of vertical Member 12;
- Case 2a: where the horizontal failure surface transitioned to an inclined surface at the south face of vertical Member 12; and
- Case 3: where the failure surface was a monolithic plane at the base of Member 11.

The chamfer at the inside corner of Member 11 did not have sufficient reinforcement to offer significant resistance to any northward shear loading through Member 11. As discussed in Section 2.6, some of the nodal reinforcement at Node 11/12 interface does not have sufficient development and, therefore, would not develop their full capacity. In order to account for these conditions, we also investigated variations of the first two failure surfaces in which the capacity provided by the concrete chamfer at the inside corner of the node under the diagonal was neglected and the incomplete development length of the nodal reinforcement was considered (Cases 1b and 2b). These failure surfaces are shown in Figure 56.

The first of the two failure surfaces in Figure 56 (Cases 1a and 1b) form mostly a single horizontal plane for which interface shear friction capacity according to AASHTO LRFD Section 5.8.4 is applicable. This plane is the construction joint between the Deck and Members 11 and 12. The second (Cases 2a and 2b) consists of four planes: the portion of the horizontal construction joint south of Member 12, two vertically-oriented, trapezoidal surfaces between Member 12 and the Type II Diaphragm, and an inclined surface through the cross section of Member 12. Interface shear friction is applicable for the construction joint and the vertical planes, and conventional section shear according to AASHTO LRFD Section 5.8.3 is applicable for the inclined plane. The trapezoidal vertical planes are only considered to be effective north of the void spaces formed by vertical PVC ducts. The vertical planes are monolithic because they are below the construction joint.

We evaluated the construction joints in Cases 1a, 1b, 2a, and 2b with and without the presence of ¼-inch amplitude intentional roughening. For cases 2a and 2b, the sum of the shear capacities for each of the four planes, as discussed above and shown in Figure 56, comprises the total nominal connection capacity. Table 3 summarizes the nominal interface capacity for each of the selected failure modes. Except for Case 3 (the sloped surface at the base of Member 11),[196] these shear capacities can be compared directly with the nominal interface shear forces of 1,710 kips and 1,738 kips that include the effect of DL and PT, as determined from the hand calculations and the SAP2000 analysis results, respectively.

[196] A further transformation of force is required to determine the shear demand at the base of Member 11.

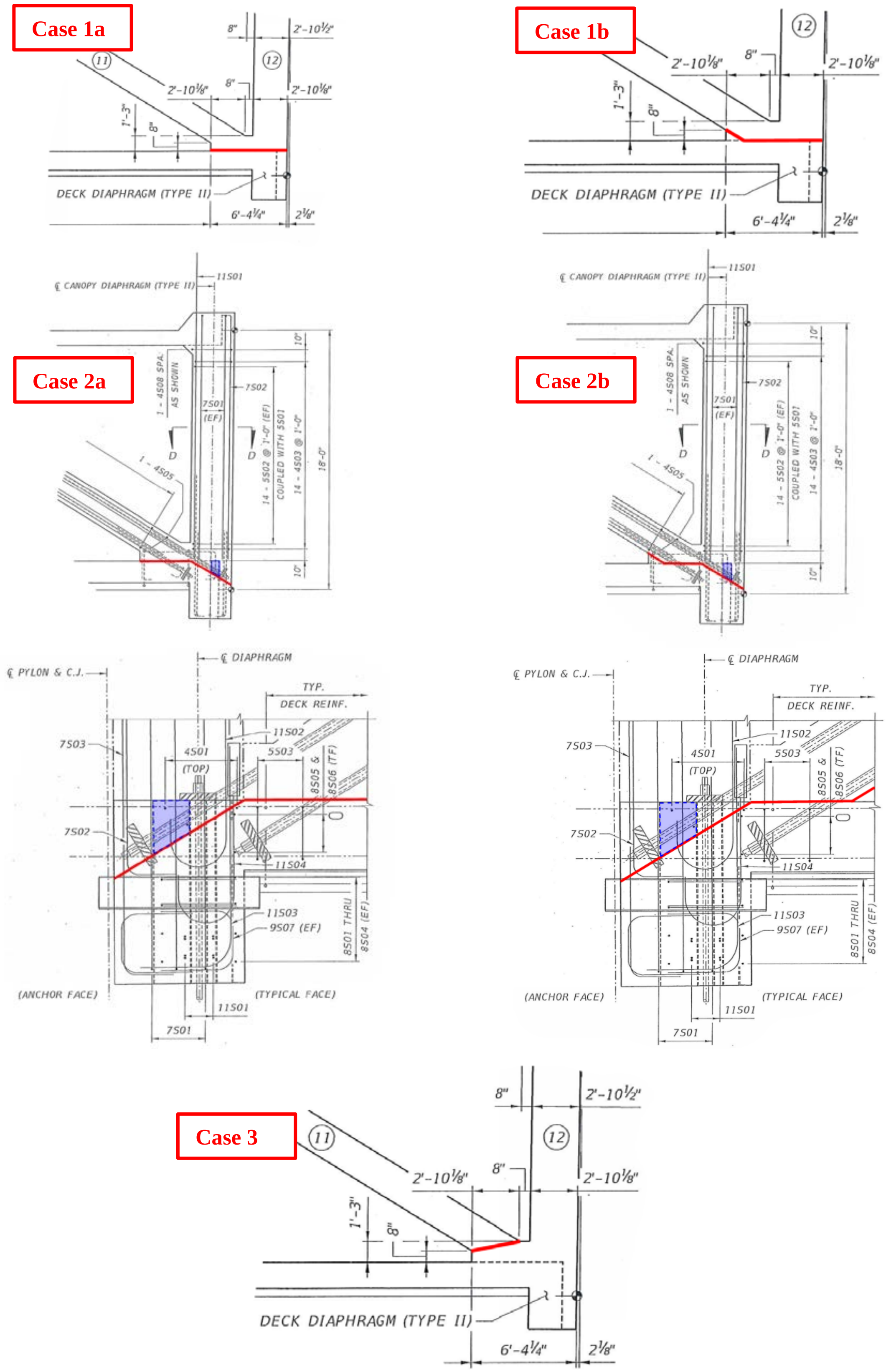


Figure 56. Interface shear failure surfaces.

Table 3. Unfactored Interface Capacities at Node 11/12

Failure Surfaces (from Figure 56)	Shear Strength, with ¼" Intentional Roughening (kip)	Shear Strength, without ¼" Intentional Roughening (kip)
Case 1a (unfactored)	2,402	1,281
Case 1b (unfactored)	2,150	1,147
Case 2a (unfactored)	1,702	1,083
Case 2b (unfactored)	1,450	948
	Monolithic Shear Strength, Roughening N/A (kip)	
Case 3 (unfactored)	1,097	

5.3 Load and Capacity Comparisons

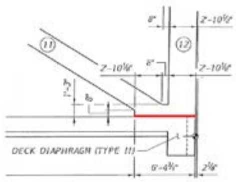
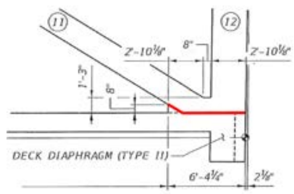
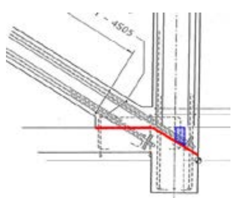
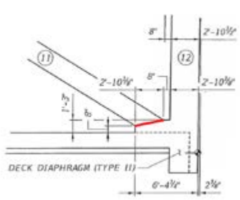
The ratios of structural demand-to-capacity ("DCR") provide a normalized index of design adequacy or failure expectation. When structural demands (loads) exceed the available capacity for an element subjected to some action, then the ratio will be greater than 1.0. DCRs can be evaluated from a design perspective by using factored loads and factored capacities. In this case, when DCR = 1.0, the design is on the threshold of compliance with applicable codes and standards, but failure is not necessarily expected. When the DCR is computed based on nominal loads and nominal capacities, then DCR = 1.0 or greater suggests that failure is plausible or even expected. The higher the value of the DCR, the greater the likelihood of failure.

By way of example, the DCR for nominal interface shear at Node 11/12 for Case 1a without ¼-inch intentional roughening can be calculated using the nominal shear force from the SAP2000 analysis (1,738 kips) and dividing it by the nominal capacity from Table 3 (1,281 kips), resulting in a nominal DCR of 1.36. This demonstrates a high likelihood of failure.

The design and nominal DCR values for the Cases 1 to 3 (Figure 56) investigated in our analysis are shown in Table 4. The results here represent the condition in which the Main Span is Simply-Supported, and PT is present in Diagonal Members 2 and 11. Load and resistance factors used to compute factored loads and factored capacities associated with the design DCRs in the table are per AASHTO LRFD. The demands used in these comparisons originated from the SAP2000 analysis presented in Section 5.1.2, and the capacities were presented in Section 5.2.

As shown in Table 4, "design" DCR values exceed 1.0 for many limit states, even for some that consider the benefit of ¼-inch amplitude intentional roughening of hardened concrete at construction joints. Notably, our analysis of the axial capacity of Diagonal Member 11 (which is not reflected in Table 4) indicates that the factored loads are very nearly equal to the available factored capacity, with a design DCR = 0.99. However, this value also may have been exceeded during destressing because reapplication of full PT force was required to loosen the nuts on the ends of the PT bars.

Table 4. Node 11/12 Interface Shear DCRs[197]

Case	Design DCR		Nominal DCR	
1a 	With ¼” roughening	Without ¼” roughening	With ¼” roughening	Without ¼” roughening
	0.95	**1.78**	0.72	**1.36**
1b 	With ¼” roughening	Without ¼” roughening	With ¼” roughening	Without ¼” roughening
	1.06	**1.99**	0.81	**1.52**
2a 	With ¼” roughening	Without ¼” roughening	With ¼” roughening	Without ¼” roughening
	1.34	**2.11**	**1.02**	**1.60**
2b 	With ¼” roughening	Without ¼” roughening	With ¼” roughening	Without ¼” roughening
	1.58	**2.41**	**1.20**	**1.83**[198]
3 	**1.59**		**1.22**	

[197] Demand-to-capacity ratio (DCR) is the load divided by the structural capacity. Values greater than 1.0 indicate design deficiency for design DCRs and plausible failure modes for nominal DCRs.

[198] This is the highest calculated nominal DCR, which indicates the most likely failure mode.

The “nominal” DCR values in Table 4 also exceed 1.0 in most cases, including all cases without ¼-inch roughening. These limit states warrant closer attention since they represent the most plausible failure modes, based on engineering analysis. This is shown below.

Based on this analysis alone, the failure surface represented by Case 2b is the most likely because it has the highest values of nominal DCR. Most notably, the nominal DCRs for the nodal shear capacity when considering part of the construction joint and section shear of Member 12 exceeds 1.0 even when ¼-inch amplitude intentional roughening is considered.

It should also be emphasized that the demands represented in this analysis do not include the additional PT force required to initiate the destressing of Member 11—if that force were known and included in these calculations, the DCR values for shear in Node 11/12 would increase even further. Additionally, Section 4.10 of FIU’s Design Criteria requires that a minimum of 20 psf construction live load applied to the walking surface be included in the design to account for construction loads.[199] This loading also is not included in the DCRs reported in Table 4, which would also increase the values of the design DCRs and move FIGG’s design even further out of compliance with the required AASHTO LRFD Bridge Design Specifications.

5.4 Comparison of Analysis with Observations

Observations of the Main Span structure’s performance before the collapse and the remains after the collapse can serve to either confirm or invalidate hypothetical failure mechanisms. As shown above, the team’s structural analysis indicates that the most likely failure mechanism was shear in Node 11/12 along the surface that includes part of the Node-to-Deck construction joint and a diagonal plane through Member 12. review of photographs prior to and after the Bridge’s collapse validate the analysis.

For example, Figure 57 shows relative displacement of the nodal concrete with respect to the Deck surface, indicating failure along the construction joint. Figure 58 shows a diagonal crack in the portion of vertical Member 11 that projects beyond the face of the Type II Diaphragm. Figure 59 shows the post-collapse remains of Member 12. A diagonal fracture is visible on the lower surface. Figure 60 shows the remains of the Type II Diaphragm, and the failure surface appears where Member 12 was integral with the Diaphragm above the drain pipe. These images all correspond with Case 2b in Table 4 and validate the results of MCM’s analysis.

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[199] NTSB Bridge Factors Report, Attachment 12; FIU-UniversityCity Prosperity Project Pedestrian Bridge Design Criteria, TY Lin, April 2015.

Figure 57. Northward displacement of Node 11/12 relative to Deck.[200]

[200] NTSB Bridge Factors Report, Photos 78, 79, 84, and 90.

Figure 58. Diagonal crack in vertical Member 12 (indicated with red circle).[201]

Figure 59. Fractured surface at base of Member 12 (indicated with red arrow).

[201] NTSB Bridge Factors Report, Photo 97.

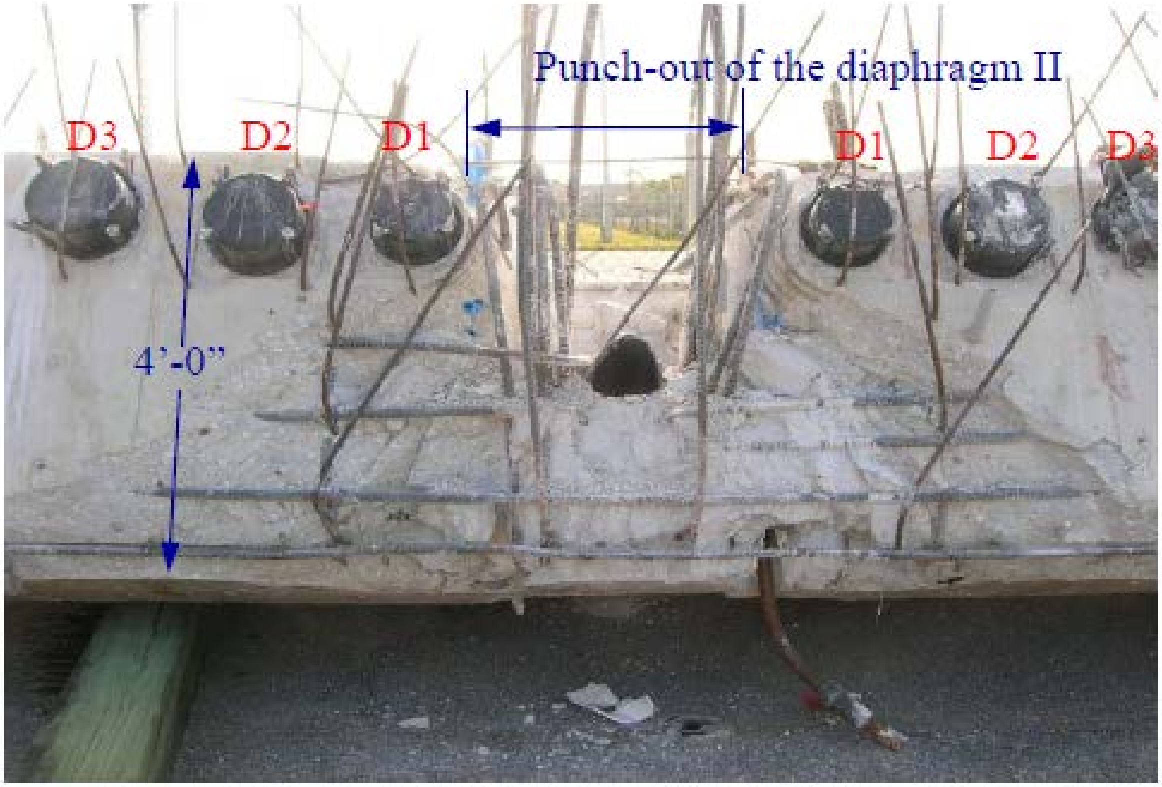

Figure 60. Fractured surfaces in Type II Diaphragm.[202]

The triangularly-shaped fracture surface on the north face of the Diaphragm in Figure 60 appears shallow. The large cracks that developed at this location prior to collapse attracted the most attention from FIGG during the March 15, 2018 meeting. Figure 60 suggests that these cracks were surface spalls created by the northward movement of Node 11/12. This corresponds with the observation in Figure 44 that the lower edges of these cracks were displaced outward from the top edges. The formation of the large diagonal cracks on the north face of the Type II Diaphragm was simply a consequence of the failure of Node 11/12, rather than a failure by itself. This is confirmed by the observation that the cracks seen on the north face of the Diaphragm did not extend to the south face. FIGG's focus on the Diaphragm cracking during the March 12 to 15, 2018 period was thus misplaced; FIGG should have instead focused its investigation on the failure of Node 11/12, *i.e.*, cracking and sliding *under* Member 11. Diaphragm cracking was a secondary failure.

5.5 Analysis Summary

Our team used three independent methods to evaluate the member forces of the Main Span, as well as the interface shear at Node 11/12. Most notably, our ABAQUS FEA model predicted cracking at locations consistent with cracking depicted in pre-collapse photographs. Our evaluation of loads and capacities also resulted in nominal DCRs at Node 11/12 significantly

[202] OSHA Investigations Report, page 96.

greater than 1.0, indicating a high likelihood of failure at that location. And several potential failure modes at Node 11/12 were checked, and the mode associated with the highest DCR (1.83) was consistent with the observed mode of failure.

Ultimately, the results were consistent across the three methods, and the forces at Node 11/12 from our analysis (1,738 kips) were much higher than what FIGG reported (571 kips) in its 90%, Final and RFC design calculations for the Simply-Supported case. Proper hand calculations could have revealed to FIGG that it had greatly underestimated the shear force at this critical connection.[203] FIGG's design at Node 11/12 was found to be noncompliant with the requirements of AASHTO LRFD Bridge Design Specifications for both scenarios, with and without ¼-inch surface roughening at the construction joint, and failure was thus predictable for both scenarios.

[203] As discussed above, FIGG's calculations for shear strength at Node 11/12, also consisting of numerous other errors as discussed herein, relied on the design parameters (cohesion and friction factors) corresponding to intentional roughening of ¼-inch amplitude. However, this design assumption was not communicated to the contractor through a note in the RFC drawings, in contrast to specific notes by FIGG on other RFC drawings such as B-24B, B-25 and B-104 specifying amplitudes of ¼-inch and 1/8-inch.

6 Review of FIGG's Structural Analysis and Design

As an integral part of MCM's analysis, our team also reviewed FIGG's 90%, Final, and RFC Design Calculations and the computational models FIGG has produced related to the same. FIGG relied on LARSA 4D and LUSAS software, as well as electronic worksheets and hand calculations to perform the structural modeling, analysis, and design. Each is discussed below.

6.1 LARSA 4D Structural Analysis

LARSA 4D evaluates structural behavior at many intermediate construction stages, with the ability to include time-dependent effects of concrete creep and shrinkage.

6.1.1 Review of LARSA Output in FIGG's Calculation Submittal

Section III of FIGG's RFC Superstructure design calculations document the LARSA 4D models that FIGG developed and includes input and output for two LARSA 4D models: "Superstructure Model_9.27.2016.lar" (LARSA 1) and "Main Span Erection_9.19.2016_South End Pinned_3' Diagonal.lar" (LARSA 2). FIGG last ran LARSA 1 on September 29, 2016, where it simulated the loading and structural behavior of the Bridge at different stages of construction, not including transport of the Main Span, as shown in Table 5.[204] LARSA 2 considered transport of the Main Span on SPMTs, as shown in Table 6.[205] There are a number of problems with both models.

First, PT forces were not properly taken into consideration. PT bar forces have significant influence on both the axial force in Diagonals and the shear forces in nodes where Diagonals meet the Deck and the Canopy. However, while both LARSA models included Deck and Canopy PT tendons, neither included PT bars in the Diagonals[206] and, thus, the effect of PT force on the shear at nodes was not included in FIGG's analyses. This caused FIGG to significantly underestimate the shear at all node interfaces, especially the end Nodes 1/2 and 11/12.

Second, FIGG miscalculated the properties for Member 12. The cross-sections for all of the Bridge elements are the same in both models, except for vertical Member 12. In the first model, Member 12 has the cross-section properties corresponding to the large (full) Pylon, even in the

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204 Superstructure RFC Design Calculations, UniversityCity Prosperity Project Pedestrian Bridge, FIGG, April 11, 2017, p. 222-223 (FBE004415-004416) and 524-530 (FBE004717-004723). Notably, the list of load cases for this model did not include the construction live load of 20 psf required by FIU's Design Criteria, increasing excessive design loads.

205 *Ibid*, p. 535-538 (FBE004728-004731). Details provided by FIGG show that the SPMT supports were located at 31.28 feet north of the south Bridge end, and at 39.19 feet south of the north Bridge end, corresponding roughly, but not exactly, with the support locations shown on RFC Sheet B-109 and Barnhart's Drawing PR1575-G1.0, which show these support locations as 35.1 feet and 44 feet-9.5 inches, respectively.

206 FIGG's design summary narrative stated that Diagonal PT bar sizes and quantities were selected in order to limit the service level stresses to AASHTO LRFD prescribed limits, *Ibid*, FBE004197, indicating the effect of PT force on the axial force in the Diagonals was likely considered in FIGG's analysis, despite the fact that the effect of PT force on the shear at nodes was not included.

Simply-Supported stage prior to the erection of the Back Span and full Pylon.[207] This indicates the results from the first model for Member 12 may be unreliable for the construction stages prior to integration of the Main Span and the Back Span.

Third, Section IV of FIGG's submittal, Construction Check, provides the Deck and Canopy forces/stresses; and Section V, Longitudinal Design, shows the Web Diagonals forces/stresses for the SPMT support condition.[208] However, neither of these included member load effects for the condition in which the Main Span was Simply-Supported on piers immediately after erection, when the Main Span collapsed. FIGG, therefore, did not use LARSA to check its design under the most critical loading condition of the Main Span.

Table 5. FIGG LARSA 4D Superstructure Model Analysis Stages and Steps

Stage	**Step**	**Activity**
Stage 1 (Day 14)	Step 1: Erect Piers	Construct Structure Group: Piers
Stage 2 (Day 28)	Step 1: Erect Main Span	Construct Structure Group: Main Span
	Step 2: Release Moment at Pylon	Node constrain/release
	Step 3: Fix Joints	Node constrain/release
	Step 4: Stress Main Bottom PT	Tendon Activity: Main Bott. D1L – D6L and D1R – D6R
	Step 5: Stress Main Top PT	Tendon Activity: Main Top C2L – C3L and C2R – C3R
Stage 3 (Day 97)	Step 1: Erect Back Span	Construct Structure Group: Back Span
	Step 2: Stress Back Bottom PT	Tendon Activity: Back Bott. D7L – D9L and D7R – D9R
	Step 3: Stress Back Top PT	Tendon Activity: Main Top C52L and C5R
	Step 4: Stress Top Continuity PT	Tendon Activity: Main Top C1L, C4L, C1R & C4R
	Step 5: Step 3 [sic]	Node constrain/release
Stage 4 (Day 105)	Step 1: Cast Pylon – Top	Construct Structure Group: Pylon
Stage 5 (Day 120)	Steps 1 – 5: Install Stays 1-5	Construct Structure Groups: Stays 1 - 5
Stage 6 (Day 125)	Step 1: Install DC3 (Bridge components)	Load Activity: DC3
Stage 7 (Day 360)	End of construction, no steps	No activity
Stage 8 (Day 500)	No steps	No activity
Stage 9 (Day 1,000)	No steps	No activity
Stage 10 (Day 5,000)	No steps	No activity
Stage 11 (Day 10,000)	No steps	No activity

207 In the second model, Member 12 has the correct section properties (2.875' by 1.75'), which matched the expected cross-section at the time of Main Span movement and before construction of the Back Span.

208 Superstructure RFC Design Calculations, UniversityCity Prosperity Project Pedestrian Bridge, FIGG, April 11, 2017, FBE004991-004998.

Table 6. FIGG LARSA 4D Main Erection Model Analysis Stages and Steps

Stage	Step	Activity
Stage 1 'Stage 2' Day – 28	Step 1: Erect Main Span	Construct Structure Group: Main Span; Node constrain/release; Application of self-weight and dead load
	Step 2: Fix joints	Node constrain/release
	Step 3: Stress Deck PT	Tendon Activity: Main Deck T1L – T6L and T1R – T6R
	Step 4: Stress Canopy PT	Tendon Activity: Main Canopy C2L – C3L & C2R – C3R

In summary, the two LARSA models presented in FIGG's Final Superstructure packages included all the various construction stages but included serious calculation errors and omissions that resulted in a design that was unsafe and non-compliant with the applicable design codes. Moreover, FDOT reviewed FIGG's Superstructure Design Calculations submittal and provided many comments, but failed to detect that FIGG did not check its design of the Main Span connections in the most critical stage, *i.e.*, immediately after the transportation but before destressing of temporary PT in Members 2 and 11.

6.1.2 Review of LARSA Models Provided by FIGG

In addition to the LARSA 4D data output, we also reviewed LARSA 4D models themselves along with their accompanying ASCII format data files. Table 7 identifies the Simply-Supported Main Span models we reviewed and some of their relevant characteristics.

Table 7. FIGG LARSA 4D Models Reviewed

Production Log File Name	Create/Mod Date	Comments
Superstructure_Model_ 9.28.2016_MF.lar	9/27/2016	It is unclear why filename date is later than model creation date. The file was probably used or modified by Manuel Feliciano (MF). They appear contemporaneous with 90% and 100% design calculation model.
20161011_FIU_Superstr_ Indep_Check.lar	10/28/2016	Included PT bars in Diagonals during construction. This model was probably created during FIGG's in-house independent check.

6.1.2.1 Superstructure_Model_9.28.2016_MF.lar

This model was selected for review because it appeared to match the LARSA model presented in FIGG's Final Design Calculations submittal. It was created on September 27, 2016, the same date as in the file name of the first model output that appears in FIGG's Design Calculations submittals. The two models also include the use of the same construction stages and lack of PT in the Truss Web Members. The model was configured so that the user could segregate dead loads, prestress loads, and a combination of all loads, but several problems are notable.

First, the cross-section assigned to vertical Member 12 is 28 square feet, which is much larger than its actual cross-section of 5.03 square feet at the time of Main Span transport. Given this large difference in cross-section, the load effects reported for Member 12 by this model are not expected to be accurate. Second, the PT load effects reported from this model only include the effects of PT applied in the Deck and the Canopy, not the Truss. PT in diagonal Member 11 (or any other diagonal) was not included, the application of which would have increased the axial and horizontal shear forces in Node 11/12 by hundreds of kips. Third, the model did not include the construction live load of 20 psf specified by FIU's Design Criteria, which would have again provided even higher design loads.

Ultimately, the unfactored axial force in Member 11 under dead load was -1,285 kips.[209] Since Member 11 made a 31.2-degree angle with the horizontal plane in this model, the horizontal components of the axial forces can be calculated as a cosine of the axial force in Member 11 and added to the shear force for Member 12 to obtain a combined shear force at Node 11/12. The shear forces from this model for dead load and PT load effects at Node 11/12 were 1,037 and 1,032 kips, respectively (Table 8). The effect of PT on shear at Node 11/12 is thus negligible in this model because it does not include PT in Web Truss Members.

6.1.2.2 20161011_FIU_Superstr_Indep_Check.lar

This model appears to have been generated independently as part of FIGG's internal review process and, therefore, was selected for this review. The model was last updated on October 28, 2018, after FIGG had submitted its 90% design calculations on October 5, 2016. One of the key differences between this model and LARSA model Superstructure_Model_9.28.2016_MF.lar, as well as the LARSA results included in FIGG's calculation submittals in Section 6.1.1, was the inclusion of Truss Web PT bars during intermediate stages of construction. This would have significantly increased the axial force and the shear force at Member 11 and Node 11/12, respectively.

This model considered 13 stages, which characterized the behavior of the Bridge starting from the construction of the Substructures, casting and erection of the Main Span, casting of the Back Span, the erection of the Pylon and stays, and finally through 50,000 days of concrete aging. At the end of Stage 3, the Main Span was Simply-Supported on the South Pier and the Pylon Pier. Vertical Member 12 was modeled with a cross sectional area of 5.25 square feet, which corresponds to a depth of 3 feet and a width of 1.75 feet. This is 1.5 inches deeper than the as-designed Member 12, which has a cross section of 5.03 square feet (2.875 feet x 1.75 feet).

[209] Negative sign denotes compression.

Therefore, the load effects in Member 12 are expected to be more reliable in this model than in FIGG's "Superstructure_Model_9.28.2016_MF.lar" model, which again used a cross-section area of 28 square feet for Member 12.

In this case, the unfactored axial force in Member 11 under DL and PT load were -1,253 kips and -548 kips, respectively,[210] and Member 11 made a 31.2-degree angle with the horizontal plane. Using this angle, the combined shear force from the component of axial force in Member 11 and shear force in Member 12 for DL and PT load effects at Node 11/12 are summarized in Table 8. These forces are unfactored, as they do not include a load factor of 1.25 as specified in AASHTO LRFD Bridge Design Specifications for DL. These also do not include the effect of 20 psf construction live load that was required for design by FIU's Design Criteria. Once again, these omissions would increase the loads reported in Table 8 even further.

In summary, the inclusion of PT in the Diagonals of this model, as it should have been done in all models, increased the combined unfactored shear force for combined dead and PT load by 54%. As discussed earlier, FIGG developed this LARSA model after it submitted its 90% design calculations on October 5, 2016, but this newer analysis was not reflected in the Final or RFC Design Calculation submittal in February and April 2017. If FIGG had correctly updated its design calculations with this increased shear force and compared it with its design shear capacity at Node 11/12, it would have likely discovered its design error and avoided the collapse. The same can be said for FIGG's review of its design in March 2018 after discovery of large cracking on the Bridge.

Table 8. Unfactored Shear Forces from FIGG LARSA Model at Node 11/12 (in kips)

FIGG's LARSA Model	Dead Load	Dead Load + PT[211]
Superstructure_Model_9.28.2016_MF.lar	1,037	1,032
20161011_FIU_Superstr_Indep_Check.lar	1,131	1,592

6.2 LUSAS Finite Element Analysis

FIGG also relied on FEA software LUSAS to (1) evaluate the shear forces at the interfaces between the Truss Web Members and the Deck and Canopy, and (2) verify service-level concrete stresses at various stages of construction.

6.2.1 Review of LUSAS Output in FIGG's Calculation Submittal

Section X of FIGG's RFC Design Calculation submittal includes hand-annotated screenshots from three different LUSAS models:[212]

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210 *Id.*

211 The PT force in LARSA model "Superstructure_Model_9.28.2016_MF.lar" does not include PT in Diagonals.

212 Superstructure RFC Design Calculations, April 11, 2017, FBE005567-005630.

- FIU_Truss_New.mdl – Main Span in its temporary Simply-Supported configuration;
- FIU_Truss_New_Constr_Sup.mdl – Main Span with SPMT support condition; and
- FIU_Truss_New_PylonFixed.mdl – Main Span with the north end restrained from rotation by the completed Pylon and Back Span.[213]

FIGG's screenshots for each of the three models, at both the 90% and Final stages, illustrate the locations and paths of PT tendons and bars, longitudinal normal stress contours (z-direction), deflection contours, and interface shears at Truss Node-to-Deck and Node-to-Canopy interfaces, showing identical model characteristics and results.[214] The first model with Simply-Supported Main Span is the most relevant to this investigation because the Bridge collapsed prior to being integrated with the Back Span and Pylon.

Figure 61 shows the longitudinal normal stress contours for the Simply-Supported Main Span, indicating that dead, live, and PT loads were applied to the model. However, there is no indication that the construction loads specified in FIU's Design Criteria Section 4.10 were included in FIGG's design calculations, which would have increased the design loads and resulting stresses.

Further, Figure 62 shows the locations of PT tendons modeled in the Simply-Supported Main Span. However, PT bars are notably shown in Diagonal Member 2 on the south end of the span, but not in Diagonal Member 11 on the north end of the span. Figure 62 also shows PT bars located in vertical Member 1 and Diagonal Member 4, contrary to RFC drawing Sheet B-38 and the installation in the field. In other words, this arrangement of PT bars does not represent the condition of the Main Span prior to its collapse on March 15, 2018 and is thus unreliable.

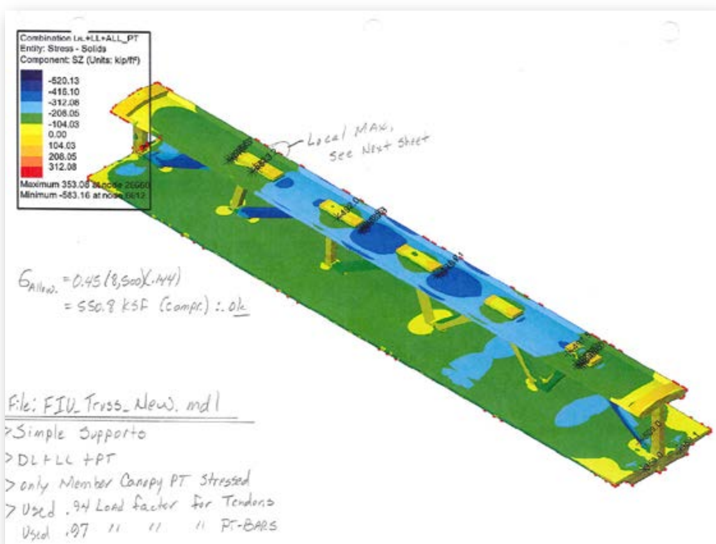

Figure 61. Hand-annotated LUSAS model screenshot.[215]

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[213] The Pylon and Back Span were not modeled directly. Boundary conditions were applied to the north end in an attempt to simulate the restraint they would provide.

[214] Superstructure 90% Design Calculations, UniversityCity Prosperity Project Pedestrian Bridge, FIGG, October 5, 2016.

[215] Superstructure RFC Design Calculations, April 11, 2017, FBE005571.

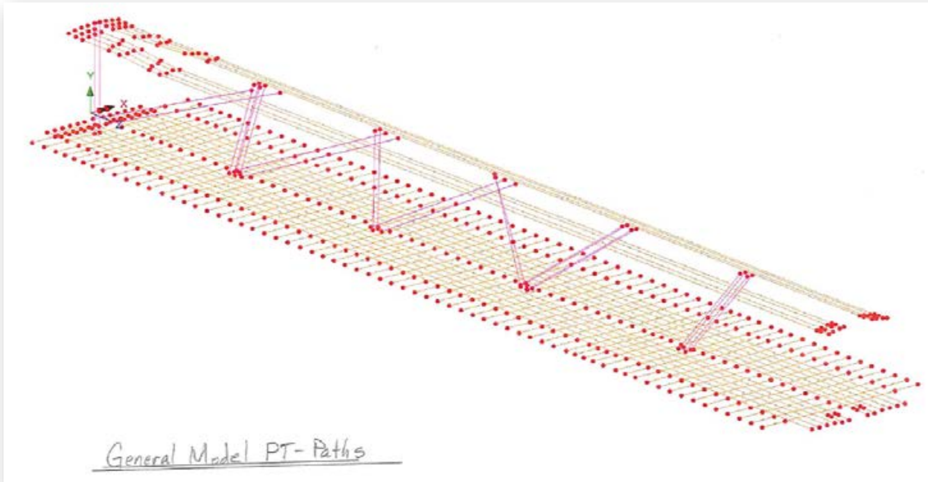

Figure 62. LUSAS screenshot showing PT paths for Simply-Supported Main Span.[216]

The Design Calculation submittals also include screenshots of planar slices through the models at nodal interfaces at the Deck level, which report the internal normal and shear forces on these planes. FIGG's screenshot for the interface at Node 11/12 is shown in Figure 63. The load case producing the forces is not completely visible in the screenshots, but based on the screenshot showing longitudinal normal stress contours in Figure 61, it appears to correspond to the load case called, "DL+LL+ALL_PT."

The value, Fy, in Figure 63 is the total horizontal shear forces occurring at the slices for the given load case and combination, acting in a direction parallel to the length of the Bridge. These horizontal shear forces tend to cause sliding between the Truss Web Members and the Deck, unless the structure has adequate capacity to resist this movement. The value, Fz, is the internal force acting perpendicular (normal) to the slices, and the value, Fx, is the horizontal shear force oriented in a direction perpendicular to the length of the Bridge. All forces in these figures are reported in kips.

In the Simply-Supported condition, nearly all of the weight of the Main Span Truss must pass through the 1/2 and 11/12 interfaces. Summing the vertical forces at these two locations (1,235 and 646.7 kips) yields a reasonable total force of 1,881.7 kips, or 941 tons. This is close to the 950-ton lifting weight given by FIGG on RFC drawing Sheet B-37. However, the distribution of the force between the two interfaces is disproportionate: 66% of the vertical force is passing through Node 1/2, and the remaining 34% is passing through Node 11/12. While the Truss Web Members are not symmetrically configured, there is not enough asymmetry to produce such a large disparity in vertical force distribution. In contrast, the vertical components of DL in end diagonals in FIGG's LARSA model "Superstructure_Model_9.28.2016_MF.lar" are 761 kips (53%) and 674 kips (47%), for Members 2 and 11, respectively. The disparity in the LUSAS output conflicts with FIGG's other analysis model and is likely an indication of an error in the LUSAS model.

216 *Id.*, FBE005568.

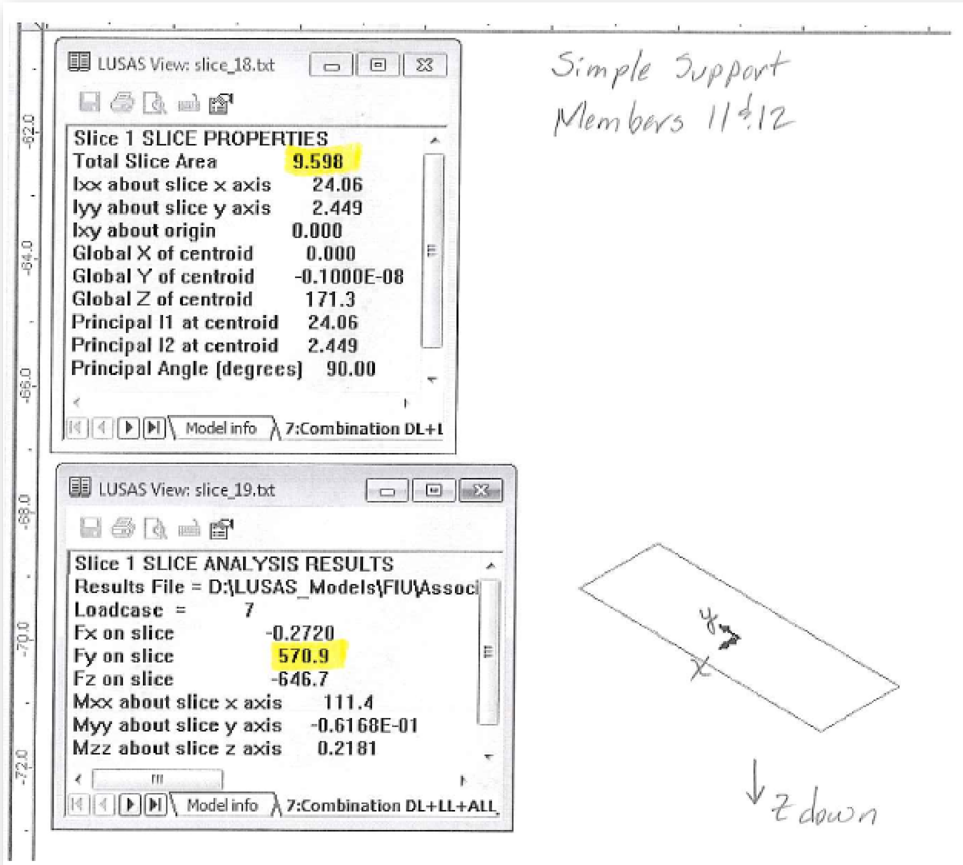

Figure 63. LUSAS screenshot of Node 11/12 interface for the Simply-Supported Model.[217]

Further, axial force in Diagonal Members 2 and 11 would transmit most, but not all, of the weight of the Superstructure to the Substructure Piers. The orientation of the force resultants at Node 1/2 and Node 11/12 would be approximately equal to the orientations of Diagonal Members 2 and 11. And because Diagonal Members 2 and 11 are both oriented at shallow angles to the Deck, the horizontal components of force should be larger than the vertical components. However, the opposite is reflected in FIGG's LUSAS screenshots for the interface shears, again indicating an error in these models.

Finally, aside from a small amount of transverse shear in Members 1 and 12, nearly all the horizontal shear acting on these interfaces is coming from axial force in Diagonal Members 2 and 11. The magnitude of the reported horizontal shear should be related approximately to the axial force in the diagonal by trigonometry. For example, the horizontal shear force reported by FIGG in Figure 63 is 570.9 kips. Considering that Diagonal Member 11 forms a 33-degree angle with the surface of the Deck, the axial force in Diagonal Member 11 would be interpreted to be around (570.9)/cosine(33 degrees), or 681 kips. This is very different from the 1,285 and 1,253 kips axial force represented in the LARSA 4D models for DL (see Section 6.1.2). The LARSA 4D results reported in the calculation submittal also did not include any additional axial force applied through PT bars in Diagonal Member 11.

[217] *Id.*, FBE005580.

Based on the foregoing, our inspection of the LUSAS interface shear forces that FIGG presented in its 90%, Final, and RFC Design Calculation submittals raise serious questions regarding the accuracy of the analysis. The graphical representation of the PT in the model does not match the as-designed condition, the distribution of vertical forces between interfaces is not logical, the orientation of the resultant forces at the interfaces are inconsistent with the Bridge's geometry, and the magnitudes of the forces are inconsistent with other FIGG analyses.

Notably, FDOT also reviewed FIGG's Superstructure Design Calculations, which included results of FIGG's LARSA and LUSAS models. However, FDOT failed to discover the omission of PT bars in Member 11 in FIGG's LUSAS models, the large disparity in vertical force distribution between end nodes, omission of the 20 psf construction live load, and the discrepancy between the axial force in Member 11 and the shear force in Node 11/12 obtained from FIGG's LARSA and LUSAS analyses.

6.2.2 Review of LUSAS Models Provided by FIGG

During the NTSB Investigation, MCM also reviewed the LUSAS models provided by FIGG subsequent to the collapse. Table 9 identifies the Simply-Supported Main Span models reviewed and their most relevant characteristics.

6.2.2.1 Pre-90% Submittal LUSAS Models by FIGG

Our team was unable to reproduce the Node 11/12 interface shear force numbers that FIGG reported in its 90%, Final, and RFC Design Calculations from the LUSAS model files provided. For example, FIGG280209.mdl (last modified on the same date as FIGG's 90% submittal) did not have a load combination called DL+LL+ALL_PT, which was the load combination indicated in FIGG's 90%, Final, and RFC Design Calculations. Summing the DL, LL, and PT load cases for the same slice location and geometry that FIGG used results in a horizontal shear of 793.8 kips rather than the 570.9 kips that FIGG reported.[218] Likewise, FIGG's model FIGG280240.mdl (last updated on September 14, 2016) did have a DL+LL+ALL_PT load case, but when we extracted the interface shear for the same geometry and location, it showed a force of 792.6 kips, again greater than the 570.9 kips FIGG reported.

Ultimately, our review revealed a nuance of FEA that led FIGG to underestimate the nodal interface shears. Specifically, FIGG's Design Calculation submittals showed that FIGG's query of the interface shear forces was exactly at the Deck elevation. This is problematic because taking a slice through the model a small distance above the Deck surface, but still below the bottom of the incoming diagonal, yields an interface shear force significantly larger than if the slice is at the Deck, where interpolation and averaging effects of the numerical model artificially distribute the nodal forces into the Deck elements. As a result, this placement results in a calculated shear force much smaller than the actual shear force. The difference between these two model slice locations for Node 11/12 is illustrated in Figure 64.

[218] *Id.*, FBE005580.

Table 9. FIGG Simply-Supported Main Span LUSAS Models

Bates File	Production Log File Name	Last Update	Comments
FIGG280282.mdl	FIU_Truss_New_1.mdl	9/8/2016	1. No Canopy or Truss Web PT 2. No ALL_PT load combination 3. No hinge in vertical Member 1 4. Continuous support at Pylon
FIGG280283.mdl	FIU_Truss_New_2.mdl	9/9/2016	1. No Truss Web PT 2. No ALL_PT load combination 3. No hinge in vertical Member 1 4. Continuous support at Pylon
FIGG280240.mdl	FIU_Truss_New.mdl	9/14/2016	1. No PT in Diagonal 11 2. PT in vertical 1 2. No ALL_PT load combination 3. No hinge in vertical Member 1 4. Continuous support at Pylon
FIGG280209.mdl	FIU_Truss_New.mdl	10/5/2016 (90% calc date)	1. No PT in Diagonal 11 2. PT in vertical 1 2. No DL+LL+ALL_PT load combination 3. No hinge in vertical Member 1 4. Continuous support at Pylon
FIGG278021.mdl	FIU_Truss_New.mdl	10/7/2016	1. PT in Diagonal 11 2. Separate ALL_PT combination 2. LL factor is zero in DL+LL+ALL_PT combination 3. Member 1 hinges modeled 4. Continuous support at Pylon
FIGG280781.mdl	FIU_Truss_New_Bott Stress_Pylon_Check _Crcks.mdl	3/13/18 (two days before collapse)	1. PT in Diagonal 11 2. No PT in vertical 1 3. No separate ALL_PT combination 4. No hinge in vertical Member 1 5. No longitudinal or transverse restraint at Pylon Pier. 6. No support under member 12 at Pylon Pier.
FIGG280782.mdl	FIU_Truss_New_Bott Stress_Pylon_Check _Crcks_SprngA.mdl	3/14/18 (one day before collapse)	1. PT in Diagonal 11 2. No PT in vertical 1 3. No separate ALL_PT combination 4. No hinge in vertical Member 1 5. No longitudinal or transverse restraint at Pylon Pier. 6. Vertical spring supports under bearing pads at Pylon Pier 7. Portion of vertical 12 north of Type II Diaphragm omitted, PT bar in Diagonal 12 extends beyond volume element, error in executing analysis.
FIGG273201.mdl	N/A	12/5/2018 (post-collapse)	1. PT in Diagonal 11 2. No PT in vertical 1 3. Separate ALL_PT combination 4. LL factor is zero in DL+LL+ALL_PT combination 5. Member 1 hinges modeled 6. Continuous support at Pylon

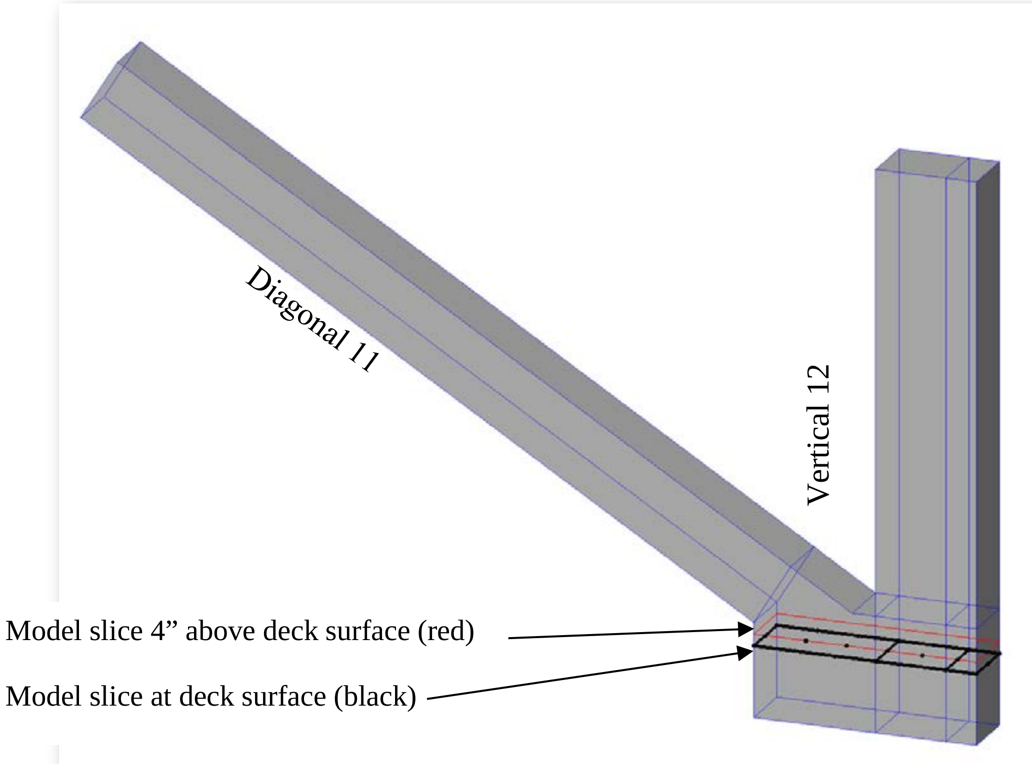

Figure 64. Example LUSAS Model Extract Showing Node 11/12 Slice Locations.

Figure 64 includes the 10-1/2 inches of Member 12 thickness that projects north beyond the Bridge Deck, but FIGG did not include this area in its query of interface shear forces.[219] For model FIGG280209.mdl (October 5, 2016), the DL interface shears for slices at the Deck and four inches above the Deck are 635 and 852 kips, respectively (considering the same slice area as FIGG’s Design Calculation submittals). Similar differences were obtained for the other models. Slicing the volume at the exact interface between the Nodes and the Deck caused FIGG to systematically underestimate the interface shears at the Truss Nodes. This underestimation of interface shears would have been discoverable by FIGG if it had compared their LUSAS results to simple hand calculations.

6.2.2.2 March 2018 LUSAS Models by FIGG

Two days before the Main Span collapsed, on March 13, 2018, FIGG updated another LUSAS model file called, “FIU_Truss_New_Bott_Stress_Pylon_Check_Crcks.mdl.”[220] This model reflected support conditions at the Pylon Pier that were more representative of the *in-situ* conditions. No vertical support was provided directly under vertical Member 12, and longitudinal restraint was removed under the Type II Diaphragm. This model also included a case with PT in Member 11. Interrogating this model for Node 11/12 interface shear forces at the same slice location as FIGG’s Design Calculations (at the Deck surface elevation) showed 1,230 kips of unfactored shear from DL and PT forces, which is more than twice the magnitude of force that

[219] *Ibid*, FBE005580

[220] FIGG280781.mdl.

FIGG reported in its RFC Design Calculations. Taking a slice 4 inches above the Deck, however, which is more reliable as discussed before, gave 1,566 kips of unfactored shear, almost three times the 571 kips that FIGG reported in its 90% and Final submittals for the Simply-Supported case.

Accordingly, our team analyzed this model for the axial compression in Member 11. At a cross-section located at the base of this diagonal, the axial compression was reported to be 1,330 kips, when self-weight and all PT was considered, except for PT in the two end Diagonal Members 2 and 11. This result corresponds with our SAP2000 and ABAQUS analyses (*see* Section 5), and is also comparable to the DL axial force in FIGG's LARSA 4D model. In the condition in which PT force is added to Members 2 and 11, the axial compression in Member 11 was reported by FIGG's updated model to be 1,855 kips. This is reasonable since the applied PT in Member 11 is 560 kips. A simple static analysis shows that the horizontal component of this force produces a shear at the Node 11/12 construction joint of approximately 1,570 kips, which is very close to the value extracted from the slice 4 inches above the Deck. These examples show that FIGG's models had changed substantially since it 90% Design Calculation submittals and the axial force in Member 11 and shear force in Node 11/12 had increased by almost a factor of three. However, FIGG never compared these higher forces with its capacity calculations in its 90%, Final, or RFC submittals and, therefore, did not discover its design error.

Importantly, Slide 29 of FIGG's March 15, 2018 presentation at the Project site shows an axial force of 1,803 kips in Member 11, which is only slightly lower than the value we extracted from FIGG's contemporaneous LUSAS model. In its March 15, 2018 presentation, FIGG demonstrated that it considered a variety of possible explanations for the large cracks that had appeared on the north end of the Bridge and at the Node 11/12 interface. However, FIGG's presentation did not include a re-evaluation of the shear friction capacity of the horizontal construction joint between the Deck and the bottom of Members 11 and 12, even though this limit state was included in its original design calculations. The revisions to the LUSAS model should have alerted FIGG to the fact that the loading of the construction joint was much higher than they originally understood and that their design of Node 11/12 was thus deficient.

None of the LUSAS models reviewed by our team included the construction live load of 20 psf required by FIU's Design Criteria. Inclusion of this additional load would have made the design error even more conspicuous.

6.3 Member and Connection Capacity Checks

The shear forces at Node 11/12 as calculated in the previous section are now compared with FIGG's calculated strengths/capacities. Sections IV through VII of FIGG's RFC submittal included calculations concerning stresses in the Canopy and Deck, buckling of the Canopy under compression, Truss Diagonal PT, Truss Member section strength, transverse flexure of the Canopy and Deck overhangs, and nodal zone design.[221] Documentation of FIGG's member capacity assessments also appears in various locations throughout its Superstructure Design Calculation submittal, and the calculated capacity of the Diaphragms appears in a separate

[221] Superstructure RFC Design Calculations, April 11, 2017, FBE004750-005475.

calculation submittal package, called "Superstructure – Miscellaneous Details RFC Design Calculations," which is dated April 11, 2017.[222] The critical Bridge elements contained within these calculations are evaluated below.

6.3.1 Diagonal Member 11

FIGG's Superstructure Design Calculation submittal did not include a comparison of the factored axial load in Diagonal Member 11 with its available capacity. Rather, FIGG reported a factored axial compressive load of 2,341 kips in Member 11 (LARSA member 732) for the AASHTO LRFD Strength I load combination for the completed Bridge including the Back Span,[223] but never reported a corresponding capacity. FIGG's reported load only slightly exceeds our estimate of factored available capacity of 2,301 kips.

6.3.2 Vertical Member 12

FIGG did not report load effects for Member 12 for the Simply-Supported construction stage, so there is no evidence that FIGG performed a direct comparison between the applied loads and the available capacity for this condition. MCM determined that the *factored* bending moment in Member 12 was 1,131 ft-kips, and that the *factored* capacity was 875 ft-kips for Member 12, indicating that Member 12 was inadequately designed.

6.3.3 Interface Shear at Node 11/12

FIGG's Deck nodal zone capacity calculations appear on FIGG's Superstructure RFC Design Calculations submittal.[224] The tabular format of the calculations indicates they were prepared with a spreadsheet program. The general inputs required for application of AASHTO LRFD Section 5.8.4 appear at the top of the worksheet, and FIGG indicates that the values of the constants, μ, K_1, and K_2 were selected consistent with the assumptions that the surface of the hardened concrete would be intentionally roughened to an amplitude of ¼-inch.[225] This assumption was not carried through in the design drawings and specifications, however, as discussed in detail above. As such, the calculations prepared by FIGG systematically overestimated the strength of these connections, as the design shear capacity of a construction joint with ¼-inch surface roughening amplitude can be up to 90% higher than an identical joint without intentional roughening.

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[222] Superstructure Miscellaneous Details RFC Design Calculations, April 11, 2017, FBE005649-006046.

[223] Superstructure RFC Design Calculations, April 11, 2017, FBE005124.

[224] *Id.*, FBE005476.

[225] *Id.* The shear capacity at the nodal interface is governed by AASHTO LRFD Bridge Design Specifications Equations 5.8.4.1-3, -4 and -5, and is defined as the minimum of the values obtained by the three equations. The parameters c, μ, K_1, and K_2 that are used in these equations depend on the assumed surface roughness condition at the shear interface. On Page 1284 of its RFC Superstructure Calculations, FIGG stated that they selected the values of these parameters for "surface intentionally roughened to an amplitude of 0.25 in.," except parameter c, where they used a value of zero as a conservative estimate. The value of zero for c is closer to the case of no intentional surface roughening. Despite FIGG's choice of zero for c, the cohesion factor, FIGG's overall calculations for nodal shear for all three equations are strongly affected by the assumption of 1/4-inch amplitude for surface roughening and the shear strength calculated by FIGG would have been substantially lower if the same calculations were performed for the case of no intentional roughening.

FDOT reviewed both FIGG's Superstructure Design Calculations and plans but failed to recognize that the surface roughness amplitude relied upon in FIGG's analysis was not communicated to the contractor on the plans – that is, MCM.

The length and width of each Deck nodal zone is also tabulated in FIGG's spreadsheet. The area of the interface is a parameter contributing to the calculated shear friction capacity of the construction joints. FIGG entered a length of 42 inches for Node 11/12, which approximately represents the distance from the south end of the node under Diagonal Member 11 to the south face of vertical Member 12. Neglecting the contribution of Member 12 to the interface is conservative, *i.e.*, FIGG underestimated the shear capacity of Node 11/12 in its calculations. As discussed in Section 5, that did not, however, lead to an overall safe design because FIGG also made gross errors in its calculations of the shear force at Node 11/12.

Another contributor to the interface shear friction capacity of a concrete construction joint is the compressive stress acting perpendicular to the interface. This stress tends to clamp the construction joint together, increasing the friction at the surface and therefore the resistance to sliding. FIGG's spreadsheet tabulates the vertical component of the net compression forces in the Diagonals at each construction joint, which are apparently extracted from the Strength I load combination results from LARSA.

The value of 1,233 kips of factored vertical force at Node 11/12 corresponds well to the Strength I axial force of 2,334 kips in Diagonal Member 11 reported on FIGG's Superstructure RFC Design Calculations submittal for the completed Bridge,[226] with live load applied only on the Main Span. For a factored axial force in Member 11 of 2,334 kips and a vertical component of force of 1,233 kips, the corresponding horizontal component of force would be 1,982 kips. This is more than double the factored interface shear reported by FIGG in the same table, [227] which supposedly also includes the effects of PT. The compressive force of 1,233 kips normal to the Node 11/12 shear interface is reasonable, but the total factored interface shear is not.

FIGG's interface shear friction calculations state that the input values of V_{DC}, V_{LL}, and V_{PT} come from the LUSAS FEA model. (FIGG did not indicate the source of the temperature forces, V_{TU+TD}.) Summing the V_{DC}, V_{LL}, and V_{PT} values in the table for Node 11/12 gives 713 kips, which is equal to the horizontal shear reported on the LUSAS screenshot on FIGG's Superstructure RFC Design Calculations submittal.[228] This case represented the scenario in which the north end of the Main Span was fixed against rotation, simulating the behavior of the completed structure.

By underestimating the shear force at Node 11/12 for the Simply-Supported condition, FIGG incorrectly determined that the shear force was higher after the Bridge was completed and, therefore, did not check what was in fact the most critical design case. In March 6, 2018 email correspondence, MCM requested and received FIGG's confirmation that Diagonal Members 2 and 11 would be de-stressed *after* the Main Span was placed on the permanent supports.[229] This

[226] Superstructure RFC Design Calculations, April 11, 2017, FBE005124.

[227] *Id.*, FIGG005476.

[228] Superstructure RFC Design Calculations, April 11, 2017, FBE 005591.

[229] NTSB Bridge Factors Report, Attachment 27.

indicates that FIGG understood there was a period during which the Main Span would be Simply-Supported (*i.e.*, without the Back Span in place) and still have PT active in the end Diagonals, but apparently did not consider this condition in its structural analysis. This was a critical flaw, as only considering the finished structure for Node 11/12 shear friction capacity checks is not meaningful because the node would have been restrained by the Pylon and Truss Member 14.

There is no evidence that FIGG compared or validated its LUSAS analysis of the nodal shears with either its LARSA results or hand calculations. Had FIGG determined the correct interface shear force that was readily available from its LARSA model, or performed basic checks of its LUSAS models, the anomalous load from its LUSAS model and the unfavorable balance between the load and the capacity could have been detected and the collapse of Bridge prevented.

6.3.4 End Diaphragms

FIGG submitted RFC Design Calculations for miscellaneous Superstructure details on April 11, 2017.[230] This calculation package included consideration of the Deck and Canopy end Diaphragms, PT local zone reinforcement, pipe supports, the south landing canopy, elastomeric bearings, expansion joints, missile guard fence, and drainage supports. We noted multiple discrepancies, as set forth below.

First, FIGG evaluated the capacity of the end Diaphragms to transmit Superstructure loads into the piers using the strut and tie method.[231] FIGG's analysis considered one shim on either side of the Bridge centerline to provide a vertical reaction at the Type II Diaphragm.[232] The actual shim dimensions and layout were not provided until FIGG issued a revised response to RFI 015 on October 19, 2017.[233] The sketch accompanying FIGG's response showed that four shims were required, and each would be 3 feet long rather than the 3.3 feet assumed in FIGG's Design Calculations.[234] It is unclear if FIGG ever updated its Design Calculations to check the adequacy of its Type II Diaphragm design for the revised shim configuration.

Second, FIGG's Design Calculations also included consideration of minimum horizontal and vertical crack control reinforcement as part of the strut and tie analysis.[235] The maximum permissible spacing for either vertical or horizontal crack control reinforcement was computed to be 10 inches, but RFC drawing Sheet B-47 showed that all of the vertical reinforcement was concentrated between the Deck PT ducts, which were 16 inches to 18 inches apart. The distribution of the vertical reinforcement shown on the plans did not meet FIGG's own determination of the maximum spacing.

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[230] Superstructure – Miscellaneous Details, RFC Design Calculations, April 11, 2017, FBE005649.

[231] *Id.*, FBE005659-005683.

[232] *Id.*, FBE005680.

[233] RFI #015 Shim Plate at Pylon Base, Initiated June 6, 2017, Revised Response October 19, 2017.

[234] Superstructure Miscellaneous Details RFC Calculations, April 11, 2017, FBE005680.

[235] *Id.*, FBE005683.

Third, there were differences between FIGG's strut and tie analysis in its original Design Calculations and its analysis presented during the March 15, 2018 meeting at the Project site.[236] For example, the assumption in the original calculations that the vertical reactions under the Diaphragm were located at two shims, closely located to the Bridge centerline, yielded a lower magnitude of tension force in the main longitudinal Diaphragm reinforcement than the assumption FIGG made in March 2018 that the reaction was distributed among four shims.[237] Another difference was in the selection of load and resistance factors. In the original Design Calculations, FIGG used a DL factor of 1.25 for the Strength I combination, but used a load factor of 1.1 in the calculations presented in the meeting.[238] Further, FIGG originally used a resistance factor of 0.9 for the tension steel, but used 1.0 in the March 2018 analysis.[239] We note FIGG calculated a tension tie force of 439 kips in its original Design Calculations,[240] whereas FIGG later calculated a much higher tie force of 593 kips in the analysis it presented on March 15, 2018. The load and resistance factors of 1.25 and 0.9 used in the original design were as required by AASHTO LRFD Bridge Design Specifications. If those were used by FIGG with the updated tension force that FIGG presented on March 15, 2018, it would have demonstrated that FIGG's design of the Type II Diaphragm did not meet the requirements of AASHTO LRFD Bridge Design Specifications.

6.4 Summary

FIGG made several critical errors in its Design Calculations, which led it to erroneously underestimate the load, while overestimating the strength. Because of these errors, FIGG did not recognize the danger of collapse.

FIGG made several mistakes which led to significant underestimation the applied load across the construction joint at Node 11/12. FIGG's LARSA and LUSAS models produced vastly different numbers for shear force across the construction joint at Node 11/12. This should have indicated that at least one of the results was wrong, if not both. FIGG updated its LARSA and LUSAS analyses to include the effect of PT force in Diagonal Member 11 after its 90% design submittal. The shear force at Node 11/12 from the updated models was approximately twice the force FIGG used to design Node 11/12 in its 90% and Final submittal. However, the increased shear force was never used in FIGG's Final design submittal. Compounding these errors was FIGG's neglect of the required 20 psf construction live load as required by FIU's Design Criteria. Had FIGG's design errors been corrected, its calculations would have shown that the Main Span was unable to safely support the applied loads **with or without** the ¼-inch surface roughening.

FIGG overestimated the strength of the construction joint by presuming a surface roughened to a ¼-inch amplitude at the interface between the Deck/Canopy and the Diagonals. The design shear capacity of a construction joint with ¼-inch surface roughening can be up to 90% higher than an identical joint without this magnitude of intentional roughening. However, this key design intent

[236] NTSB Bridge Factors Report, Attachment 32.

[237] *Id.*

[238] *Id.*

[239] *Id.*

[240] NTSB Accellion – FIGG Files – Bridge Calculations - 7. UCPP_RFC_Calculations_Superstructure.pdf, FBE005680.

was never communicated to the contractor – MCM - in any form (*i.e.*, on plan drawings, submittals, or other written or verbal communications) and, therefore, the intentional roughening performed by TSG was performed according to FDOT requirements and AASHTO specifications, but conceivably not ¼-inch amplitude.

FDOT failed to detect FIGG's errors related to omission of the effect of PT in its calculations, large unexplained differences between FIGG's LARSA and LUSAS models, and omission of the 20 psf construction live load that was required per FIU's Design Criteria. FDOT also did not note any deficiencies with FIGG's design of vertical Member 12 and the Type II Diaphragm. Had FIGG's design errors been corrected (by FIGG, Louis Berger, or FDOT) their calculations would have shown that the Main Span was unable to safely support the applied loads **with or without** the ¼-inch surface roughening.

Ultimately, FIGG's design of Members 11, 12 and Type II Diagrams were deficient. FIGG did not properly study its own models in March 2018 or reconcile the updated forces with its original RFC design Calculation submittals. As a result, FIGG failed to understand the cause of distress observed before collapse and the danger posed by restressing of PT bars in Member 11.

7 Louis Berger Peer Review

7.1 Scope of the Independent Peer Review ("IPR")

FIGG retained Louis Berger to provide an Independent Peer Review ("IPR") of the structural design.[241] The Section of Exhibit B of the agreement between FIGG and Louis Berger, dated September 13, 2016, describing the IPR scope is shown in Figure 65.

The Louis Berger Group, Inc. (Louis Berger) will provide independent peer review services for the FIU UniversityCity Prosperity Project in accordance with the RFP.

Independent Peer Review Scope

1. Louis Berger will perform Independent Peer Review for the concrete pedestrian bridge plans in accordance with the project and RFP requirements and FDOT Plans Preparation Manual (Chapter 26).
2. The Independent Peer Review will include the following activities:

Item #	Item Description
1	Develop finite element model for the bridge and estimation of demands on all elements due to different load combinations
2	Peer review of foundation plans
3	Peer review of substructure plans
4	Peer review of superstructure plans

3. The Independent Peer Review will be performed for the following submittals:
   a) Final Foundation and Substructure Plan Submittals
   b) Final Superstructure Plan Submittals
4. This Independent Peer Review scope of work is for the pedestrian bridge structure components only. The elevator structures and stairways/landings are not included in this scope of work.

Figure 65. IPR Scope from Louis Berger Contract.[242]

Based on this agreement, Louis Berger was to perform the IPR in accordance with three separate sets of requirements, each of which is described in detail below:

1. The Project requirements – defined in "*DPQ105 Independent Peer Reviews;*"[243]
2. The RFP requirements – defined in "*Design-Build Maximum Price Request for Proposals*;"[244] and

[241] NTSB Bridge Factors Report, Attachment 19.

[242] *Id*.

[243] UniversityCity Prosperity Project, Design Quality Management Plan, Independent Peer Reviews, DQP105, July 27, 2016 ("IPR DQP105"); see also NTSB Bridge Factors Report, Attachment 11.

[244] IPR DQP105; *see also* NTSB Bridge Factors Report, Attachment 11.

3. FDOT Plans Preparation Manual, Chapter 26 – defined in applicable 2015 version based on the date of the scope of work (September 13, 2016).[245]

Overall, the IPR was supposed to include a review of the Foundations, Substructures, and Superstructure of the Bridge.[246] Most relevant here is the IPR of the Superstructure.

7.2 Project Requirements

The Project requirements with respect to the IPR of the structural design are described in document "*DPQ105 Independent Peer Reviews.*"[247] The key aspects of the IPR as outlined in this document are listed below.

- An IPR is completed to validate the design of Category 2 structures or portions of such.
- The IPR is performed in accordance with FDOT Plans Preparation Manual, Volume 1, Section 26.12, by an independent firm who is familiar with the project requirements but is independent of the preparation of the deliverable and is not otherwise involved in the project itself.
- The IPR reviewer assigned to perform an IPR is responsible for completing the review in accordance with this procedure and the Review Comment Procedure, and for ensuring that the solution designed meets project and standard of practice requirements.
- The IPR reviewer often identifies means of adding value to the design and resolving conflicts in a manner that reflects extensive experience.

7.3 RFP Requirements

Pages 27 and 28 of "*Design-Build Maximum Price Request for Proposals*"[248] describe the IPR requirement for the Bridge plans (Section VI. Design and Construction Criteria, subsection A. General) as shown in Figure 66.

> Prior to submittal to the OWNER, bridge plans shall have a peer review analysis by an independent engineering firm not involved with the production of the design or plans, prequalified in accordance with Chapter 14-75. The peer review shall consist of an independent design check, a check of the plans, and a verification that the design is in accordance with AASHTO, FDOT, and other criteria as herein referenced. The cost of the peer review shall be incurred by the Design-Build Firm. The independent peer review engineer's comments and comment responses shall be included in the 90% plans submittal. At the final plans submittal, the independent peer review engineer shall sign and seal a cover letter certifying the final design and stating that all comments have been addressed and resolved.

Figure 66. IPR requirements from RFP.[249]

[245] *See also* NTSB Bridge Factors Report, Attachment 44.
[246] *E.g.*, NTSB Bridge Factors Report, Attachment 19.
[247] IPR DQP105; *see also* NTSB Bridge Factors Report, Attachment 11.
[248] NTSB Bridge Factors Report, Attachment 11.
[249] *Id*.

7.4 FDOT Plans Preparation Manual - Chapter 26

The FDOT Plans Preparation Manual defines two types of structures based on design difficulty, structural complexity, type of construction materials used, and history of use in Florida: Category 1 and Category 2.[250] Category 1 are simple/traditional bridges and Category 2 are more complicated/sophisticated bridges. The Bridge falls within Category 2.[251] Section 26.12, Independent Peer Review of Category 2 Bridges is thus the applicable section here.

According to Section 26.12 (emphasis added below):

- The designated independent peer review firm shall have no involvement with the project other than conducting the peer review and shall be pre-qualified in accordance with Rule 14-75 of the Florida Administrative Code.

- <u>The peer review is intended to be a comprehensive, thorough independent verification of the original work.</u>

- <u>An independent peer review is not simply a check of the EOR's plans and calculations.</u> It is an <u>independent verification of the design using different programs and independent processes than what was used by the EOR</u>.

- IPR shall include but not be limited to the independent confirmation of the following when applicable:
    - Compatibility of bridge geometry with roadway geometrics including typical sections, horizontal alignment, and vertical alignment. Minimum lateral offsets and vertical clearance requirements.
    - Compatibility of construction phasing with Traffic Control Plans.
    - Conflicts with underground and overhead utilities.
    - Compliance with AASHTO, FDOT and FHWA design requirements.
    - Conformity to FDOT Design Standards.
    - <u>Structural Analysis Methodology, design assumptions, and independent confirmation of design results.</u>
    - <u>Design results/recommendations (independent verification of the design).</u>
    - <u>Completeness and accuracy of bridge plans.</u>
    - Technical Special Provisions and Modified Special Provisions where necessary.
    - <u>Constructability assessment limited to looking at fatal flaws in design approach.</u>

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[250] NTSB Bridge Factors Report, Attachment 44.

[251] *See, e.g.*, NTSB Bridge Factors Report, at Section 19.1.

- When Category 2 superstructure elements are designed with software using refined analyses (*e.g.*, Grid, Finite Element Method, etc.), the peer review consultant shall verify the design results by a different program/method.

7.5 Analysis of the Superstructure IPR

Based on our team's review of the available documents, we noted several deficiencies in Louis Berger's IPR, as explained below:

- Documents provided by Louis Berger include IPR of the Bridge in its final configuration only. Louis Berger did not provide documentation of its IPR for other stages of construction that are shown on the design drawings.[252] It thus appears that Louis Berger did not perform IPR of any of the construction stages for the Main Span.
- Use of ABC for the Main Span was one of the reasons that the Bridge qualified as a Category 2 structure and triggered the FDOT PPM Chapter 26.12 IPR requirements. But it appears Louis Berger also did not include any of the steps of ABC in its IPR. As a result, Louis Berger was unable to identify the numerous errors in FIGG's design of the Main Span prior to integration with the Back Span.
- Elements of a bridge superstructure can be divided into two groups: members and connections. Main superstructure members include items such as the diagonals and verticals, deck and canopy, diaphragms, pylon, and steel pipes. Connections include all interfaces between these members. The material provided by Louis Berger did not include any IPR of the design of the connections. An IPR of the connections of the Diagonals to the Deck and the Canopy for transfer of shear forces across construction joints between the Web Truss and the Deck/Canopy (Figure 67) should have been considered in its review. For a non-redundant structure like the Main Span, failure of a single connection could potentially lead to collapse of the entire structure. Because the connections were not checked by Louis Berger, they failed to discover the design deficiencies in Node 11/12 that caused extensive cracking and the eventual collapse of the Main Span.
- Louis Berger did not provide documents depicting IPR of the design of the Deck and Diaphragm intersections at the ends for their ability to transfer horizontal and vertical shear forces. Therefore, Louis Berger was not able to identify the issues related to development length of rebar or the risk of cracking that was observed prior to the collapse.
- Louis Berger's IPR of the main members was limited to the Diagonals and the verticals under axial load and bending moment and did not include the design for shear capacity across the discontinuities near the member ends. (Figure 68) Therefore, Louis Berger was unable to identify Member 11 design as non-compliant with AASHTO LRFD Bridge Design Specifications.

[252] NTSB Bridge Factors Report, Attachment 38; Drawings RFC B-109 and B-110.

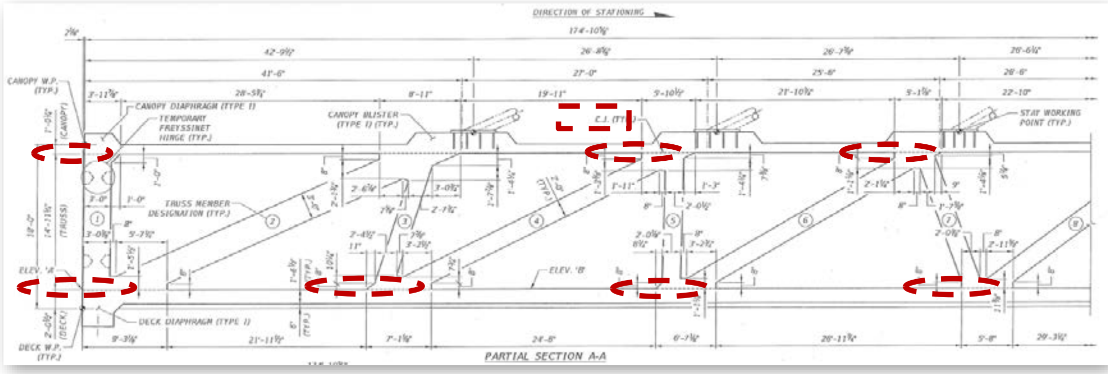


Figure 67. Example of Diagonals to Deck/Canopy construction joints. The construction joint call outs and locations are outlined in red.[253]

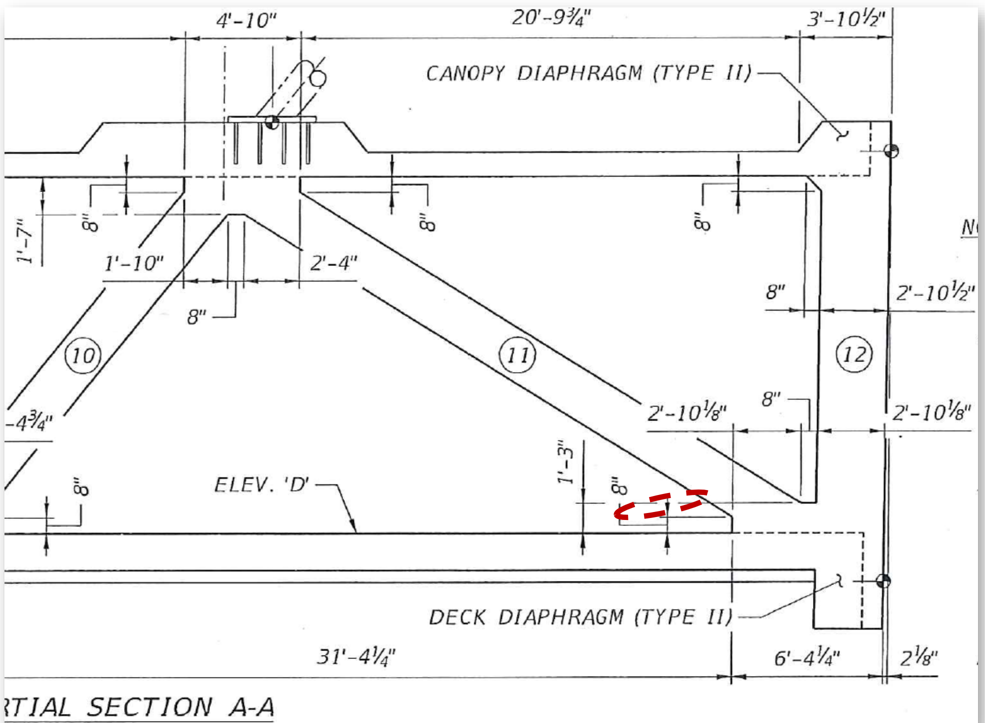


Figure 68. Shear capacity check at discontinuity at base of Member 11, as outlined in red.[254]

- Louis Berger's IPR documentation also did not include the design of other main members such as the Deck, Canopy, Diaphragms, Pylon and steel pipes, or deflection of the Superstructure. This means it did not comply with the requirements of FDOT PPM Chapter 26.12, as it failed to provide "independent confirmation of design results" for these elements.

[253] NTSB Bridge Factors Report, Attachment 38 Drawings RFC B-37.

[254] *Id.*

- Louis Berger's IPR documentation did not include consideration of load reversal in Diagonal Members 2 and 11 during transport, as depicted on Drawing B-109. Louis Berger thus did not perform a comprehensive, thorough independent verification of the design, as required.
- In its IPR documentation, Louis Berger did not appear to consider the condition where Diagonal Members 2 and 11 and their connections are loaded with temporary PT before and after transport of the Main Span. And, in fact, Ayman Shama of Louis Berger confirmed in his NTSB interview that Louis Berger did not consider this condition.[255] FIGG's failure to consider the loading from PT forces together with large compressive force from dead load, as well as the many errors made in calculation of shear force and capacity at Node 11/12, were critical errors by FIGG that directly led to the collapse. Louis Berger did not perform a constructability assessment and, therefore, failed to discover these fatal flaws in FIGG's design.
- Louis Berger failed to note that FIGG did not specify ¼-inch surface roughening amplitude at construction joints between the Diagonals and the Deck on design drawings B-37 and B-41 and the significant adverse effect of the missing specifications on the shear strength at these connections.
- Louis Berger's calculations appear to show that Truss Members 1 and 24 did not meet the required Design Criteria.[256] Figure 69 shows loading points outside of Louis Berger's axial force and bending moment interaction diagrams. However, the documents that Louis Berger provided did not include communications related to any discussion and/or resolution with FIGG on this issue, even though documents reviewed by us show that reinforcement in these members was increased after the 90% submittal.

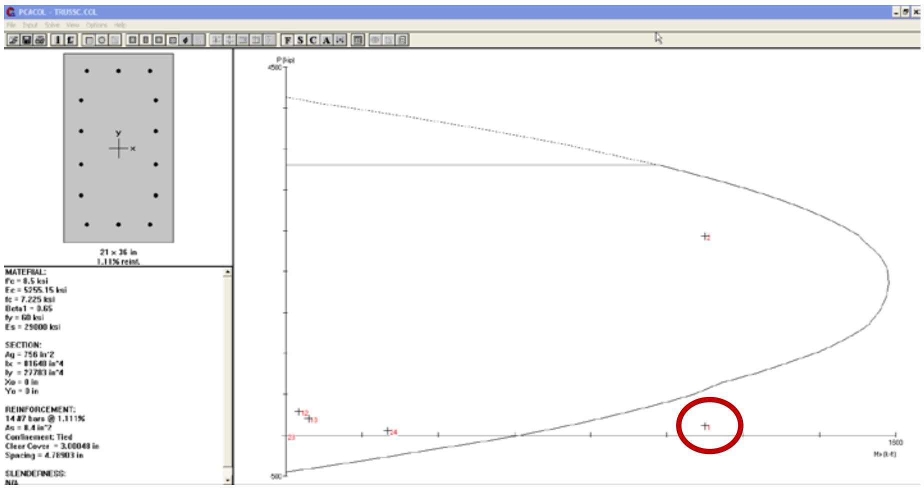
In summary, Louis Berger's IPR did not comply with many of the essential requirements in FDOT PPM Chapter 26.12 and other required specifications. Specifically, Louis Berger did not perform a "comprehensive, thorough independent verification of the original work," did not independently confirm "compliance with AASHTO, FDOT and FHWA design requirements" for connections, did not independently confirm "completeness and accuracy of bridge plans," and did not perform "constructability assessment" nor "independent verification of the design." Louis Berger thus approved a design that was unsafe and did not meet the requirements of applicable design codes.

Louis Berger's IPR also only considered the Bridge in its final configuration, which was not the critical case for axial force in Member 11 and shear at the Node 11/12 construction joint. Its review did not appear to consider the condition where Members 2 and 11 and their connections were loaded with temporary PT before and after transportation of the Main Span, nor did it include review of the shear capacity across the discontinuities near other Truss Member ends. As a result, Louis Berger also failed to identify and correct the errors in FIGG's original design

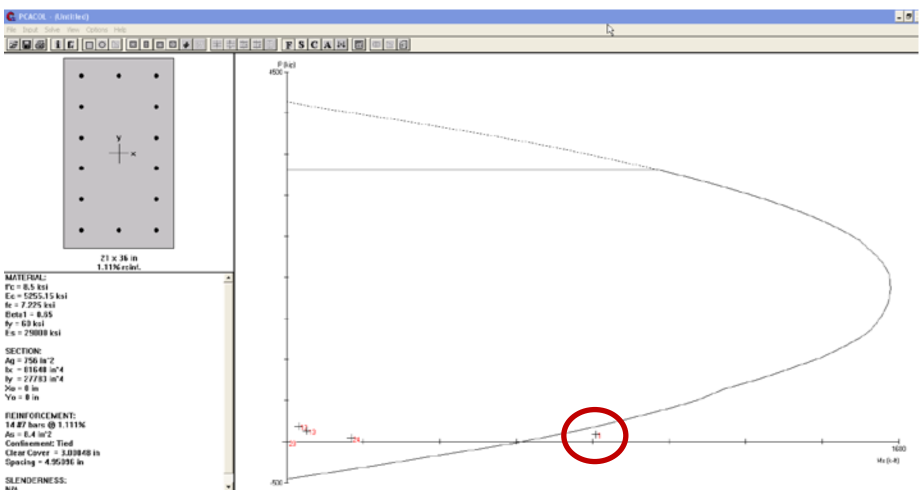
[255] NTSB Louis Berger Interview Transcripts, 24:18 - 25:16.

[256] Louis Berger Production - TRUSS-C-ST5-MX.bmp, TRUSS-C-ST5-MY.bmp, TRUSS-C-SER-MX.bmp, TRUSS-C-SER-MY.bmp.

related to calculations of shear capacity and loads at Node 11/12 of the Main Span during various stages of construction. Louis Berger failed to note that FIGG did not specify the ¼-inch amplitude for surface roughness in its design drawings for Web Truss to Deck construction joints and did not recognize its significant adverse effect on shear strength at Node 11/12.



TRUSS-C-ST5-MX.bmp



TRUSS-C-SER-MX.bmp

Figure 69. Louis Berger column superstructure interaction diagrams.[257] Cross marks circled in red indicate non-conforming loading conditions.

257 Louis Berger Production - TRUSS-C-ST5-MX.bmp, TRUSS-C-SER-MX.bmp.

8 Conclusions

8.1 Factual Findings

1. FIGG made numerous design errors and omissions in its calculations and plans associated with elements critical to the structural integrity of the Bridge during construction, specifically at Node 11/12 of the Main Span. These errors created the prerequisite conditions for failure and led to extensive structural cracking prior to the collapse. Had the design calculations been corrected, they would have shown that the Main Span, as designed by FIGG, was grossly under-designed such that it was unable to support the anticipated loads. The design of the Bridge did not conform with applicable codes and standards regardless of whether the critical construction joint had been roughened to ¼-inch amplitude, as presumed by the Designer but not communicated to the contractors or inspectors. FIGG's design errors include the following:

    a. FIGG did not include the effect of PT force in Member 11 in its design submittals, which led to significant underestimation of the applied shear force. FIGG updated its analysis to include the effect of PT force after its 90% design submittal but did not update the relevant sections of the design calculations in its Final or RFC design submittals.

    b. FIGG did not include a 20-pounds-per-square-foot construction live load in its design calculations, as required by FIU's Design Criteria.

    c. FIGG did not correctly identify the critical shear interface, a mistake that led it to calculate an incorrect shear strength for the failed joint at Node 11/12.

    d. FIGG calculated grossly different numbers for shear force across the construction joint at Node 11/12 using two different analysis methods, i.e., LARSA and LUSAS computer analysis models, but neglected the inconsistency in its own calculations. This inconsistency should have been a clear indication to FIGG that at least one of the results was wrong, if not both. Of the two, FIGG used the lower (incorrect) shear force for its design of Node 11/12, apparently without explaining and resolving the difference.

2. FIGG did not correctly account for, calculate, or communicate the surface roughening criteria at several construction joints, including the failure surface at Node 11/12, upon which FIGG's Bridge design was premised.

    a. The design shear capacity (or strength) of a construction joint can be increased by intentionally roughening the concrete surface between two elements. The shear capacity of a construction joint with ¼-inch surface roughening can be up to 90% higher than an identical joint without intentional roughening. If a designer is to rely on this additional strength, the requirement to intentionally roughen the surface to ¼-inch amplitude prior to placing concrete must be clearly communicated to the contractor.

b. In its design calculations, FIGG considered surface roughening of ¼-inch amplitude at the construction joints between the Deck/Canopy and the Diagonal Truss Members and relied on the additional shear capacity that such roughening provides. However, this key design intent (intentional roughening of ¼-inch amplitude) was never communicated to TSG, MCM, BPA, Corradino, FIU, or any other relevant party, in any form (i.e., on plan drawings, submittals, or other written or verbal communications). Therefore, although the contractors roughened the surfaces in accordance with applicable FDOT and AASTHO Construction Specifications, the specific ¼-inch roughening upon which FIGG relied was possibly not implemented in the field. This resulted in FIGG's calculated shear capacity to be overestimated.

c. Although the Florida Department of Transportation ("FDOT") specifications that require surface roughening at construction joints were applicable to the Bridge, they do not prescribe a specific roughening amplitude. Therefore, without a specific roughening amplitude explicitly prescribed on the design drawings, FIGG should not have presumed ¼-inch amplitude roughening in its calculations.

3. Louis Berger did not perform a proper Independent Peer Review ("IPR") of FIGG's original design plans and calculations, as required by the FDOT Plans Preparation Manual, Chapter 26. As a result, Louis Berger approved a design that was unsafe and did not meet the requirements of applicable design codes. Louis Berger's most critical failures are as follows:

   a. Louis Berger did not analyze the stages of the ABC method, which, ironically, is one of the design features that triggered the IPR requirements of FDOT specifications.

   b. Louis Berger's IPR only considered the Bridge in its final configuration, which was not the critical time period for axial force in Member 11 and shear at the Node 11/12 construction joint.

   c. Louis Berger's review did not consider the condition where Members 2 and 11 and their connections were loaded with temporary PT before and after transport of the Main Span.

   d. Louis Berger failed to identify and correct errors in FIGG's original design related to calculations of shear capacity and loads at Node 11/12 during various construction stages.

   e. Louis Berger's IPR of the design did not include an analysis of shear capacity across the discontinuities near other Truss Member ends.

   f. Louis Berger did not note the missing specification of ¼-inch amplitude for surface roughness on FIGG's design drawings and did not account for the large reduction in the shear strength at Node 11/12 in its IPR.

4. FIGG prescribed an incorrect repair for the observed cracking without fully understanding the reasons for the cracking and underlying conditions that ultimately caused the collapse.

   a. MCM and BPA fully informed FIGG of existing cracking on the Bridge from early February 2018 until the time of the collapse on March 15, 2018. Information provided included numerous photographs, crack measurements, and explicit concerns about the existence of such cracks, and should have been sufficient for an engineering firm like FIGG to appropriately analyze the situation and recognize the dangerous condition.

b. Representatives of FIGG also visited the Project site, including on March 15, 2018, where they directly observed the condition of the Bridge. That same day, FIGG gave a presentation of its supplemental structural analysis to an audience consisting of representatives from MCM, BPA, FIU, and FDOT. During the presentation, FIGG gave assurance that the structure did not present a safety concern.

c. Increased cracking after the Bridge move and destressing should have alerted FIGG to more serious problems. Node 11/12 experienced extensive cracking and loss of capacity during destressing operations on March 10, 2018, which initially required loading the PT bars to their full design force. Preventing further damage would have required either a reduction in applied shear force or an increase in capacity/restraint against sliding. Restressing of the PT bars of Member 11, as prescribed by FIGG, in fact did just the opposite. It increased the load on this construction joint that was already in a precarious condition, causing the final collapse.

d. FIGG also should not have specified restressing of the PT bars in Member 11 because repeated loading and unloading in these highly stressed threaded bars posed a high risk of fracture.

5. FDOT was involved with the Project at all stages of its development and missed multiple opportunities to intervene and prevent the Bridge's collapse.

   a. FDOT failed to identify several serious errors in its review of FIGG's Design Calculations, including FIGG's omission of PT force in Member 11, inconsistencies in FIGG's LUSAS results, omission of 20 psf construction live load in FIGG's analysis, and a large discrepancy in shear force results from FIGG's LARSA and LUSAS models.

   b. FDOT performed a detailed technical review of both FIGG's Design Calculations and plans but failed to recognize the discrepancy between FIGG's reliance upon ¼-inch amplitude surface roughening at the nodal construction joints and the omission of ¼-inch amplitude surface roughening in the plans at those same locations.

   c. FDOT attended the March 15, 2018 meeting and did not take any issue with FIGG's technical analyses or prevent the restressing of Member 11. Even though the restressing procedure was a deviation from the original plans, FDOT representatives did not require an IPR of FIGG's calculations and repair plans.

   d. FDOT did not require that the Main Span be shored, even though FIGG conceded in the March 15, 2018 meeting that it did not fully understand the cause of the cracking.

6. FIGG provided repeated assurances to all relevant parties that there were no safety concerns related to the observed cracking or FIGG's proposed repair plan and, therefore, it also did not identify any need for shoring or traffic closure that could have prevented loss of life.

   a. When asked specifically about the need for temporary shoring in the March 15, 2018 meeting, FIGG indicated that this would not be necessary and may produce a false sense of security. FIGG reiterated that the situation as observed and reported by the parties, and as directly observed by FIGG, was not a safety concern. Thus, only one westbound lane was closed to provide crane access during the restressing operation on March 15, 2018.

b. Based on FIGG's assurances as the EOR on the Project, the remaining parties' decision to keep traffic open was reasonable at the time.

c. The observed failure mechanism at Node 11/12 is consistent with the post-incident analysis performed by MCM, which demonstrates that Node 11/12 was the weakest structural component on the Bridge. The applied loads at the time of failure exceeded the available capacity at Node 11/12, as calculated by FIGG in its design calculations, by more than 50%. Based on the results of MCM's analysis, the failure was predictable and should have been anticipated and corrected.

8.2 Probable Cause

MCM concludes that the probable cause for the Bridge collapse was as follows:

1. There were serious errors and omissions in FIGG's original design calculations and drawings for the Bridge, including a significant underestimation of critical connection forces;
2. Louis Berger did not conduct an adequate Independent Peer Review ("IPR") of FIGG's design and calculations, and did not independently detect FIGG's design calculation and drawing errors;
3. FDOT, who employed licensed professional engineers on the Project, did not recognize multiple design errors and omissions contained within FIGG's calculations and drawings;
4. FIGG lacked proper understanding of the causes for the observed cracking during construction in February and March 2018, selected an incorrect repair of the observed cracking, and failed to appreciate and warn of the dangers posed by both the cracking and the selected repair; and
5. FIGG's repeated assurances (both written and verbal) that there were no safety concerns with the Bridge suspended over the road, dictated that FDOT and FIU not seek closure of the roadway to live traffic during re-stressing operations pursuant to governing traffic control standards and FDOT protocols.

8.3 Safety Recommendations

Based on the NTSB Investigation and the findings in this Report, MCM recommends several changes to design, construction, and inspection practices related to bridge design and construction projects to reduce the risk that future designers will commit similar errors and omissions, and thereby mitigate the risk of similar tragedies.

1. Designers should be required to highlight any significant discrepancies in the results obtained from using different analyses or design methodologies for review by internal and external review teams and any such differences should be resolved before proceeding with the final design.

2. The scope of acceptable IPRs should be expanded to also include any additional analysis and design changes, as well as any repair procedures, made after the original design.
3. Checklists should be required as part of the IPR certification in order to ensure that all key elements of FDOT PPM Chapter 26.12 IPR requirements are addressed.
4. Detailed procedures for destressing post-tensioning should be required in applicable standards and specifications, including (a) limitations on the level of stress applied to initiate destressing, and (2) the precise destressing sequencing when multiple bars are involved.
5. Re-use of high-strength post-tensioning bars should be prohibited in similar fashion to the prohibition on the re-use of similar strength A490 bolts.

September 20, 2019

MCM Response to NTSB "Materials Laboratory Study Report" issued on August 27, 2019 and Turner-Fairbank Highway Research Center Factual Report titled, "Concrete Interface Under Members 11 and 12" dated October 19, 2018.

In a Materials Laboratory Study Report issued by the NTSB on August 27, 2019, the NTSB stated (emphasis added):

> *AASHTO LRFD Bridge Design Specifications **defines** surface roughness as "normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 inch" (Section 5.8.4.3).*[1]

Similarly, the October 19, 2018 Turner-Fairbank Highway Research Center Factual Report titled, "Concrete Interface Under Members 11 and 12,"[2] stated (emphasis added):

> *Roughened concrete is **defined** as a "clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in." Surfaces that are not intentionally roughened are required to be clean and free of laitance.*

MCM disagrees with these purported definitions of "surface roughness" and "roughened concrete," and respectfully asks that the NTSB not adopt the language used in these reports in its investigation of the FIU Pedestrian Bridge Collapse.

The basis for this position and MCM's understanding of the relevant engineering design and construction specifications and Project Documents is discussed below.

Applicable Codes and Standards

As listed in the General Notes section of the RFC drawings, different sets of specifications govern the work performed by the designer (FIGG) and the construction contractors.[3] The design of the Bridge was primarily governed by AASHTO LRFD Bridge *Design* Specifications, and the construction was primarily governed by the FDOT Standard *Construction* Specifications and the AASHTO *Construction* Specifications.

AASHTO LRFD Shear Interface Design Equations

In its design calculations for the nodal construction joints, FIGG used the provisions for shear friction capacity of concrete construction joints from AASHTO LRFD Bridge Design

1 Materials Laboratory Study Report Number 19-043, NTSB Office of Research and Engineering, Materials Laboratory Division, Washington, D.C., August 27, 2019, p 4.

2 FACTUAL REPORT, Concrete Interface Under Members 11 and 12, Federal Highway Administration Turner-Fairbank Highway Research Center, McLean, VA, October 19, 2018, p. 3.

3 RFC Drawing Sheet B-2, FBE000172.

Specifications Section 5.8.4.3. Excerpts from AASHTO LRFD Section 5.8.4.3 are shown in Figure 1, below.

As seen from AASHTO Section 5.8.4.3, the subject provisions are not definitions, but rather language taken from example calculations. Designers are clearly free to specify other surface roughness amplitudes in their designs, as FIGG did for the FIU Pedestrian Bridge Project;[4] but in such cases, the specifications would simply not permit interpolation for roughness amplitudes less than ¼-inch or extrapolation for roughness amplitudes greater than ¼-inch. This does **not** mean, however, that roughening is "defined" as ¼-inch amplitude. It just means that *from the perspective of the AASHTO LRFD design equations* only, any roughness amplitude smaller than ¼-inch must be treated as not intentionally roughened for the purposes of design calculations.

AASHTO and FDOT Construction Specifications

The importance of this distinction is further demonstrated by the fact that AASHTO and FDOT *construction* specifications also do not include any such definition for surface roughening. Specifically, two construction specifications set forth in the General Notes section of the RFC drawings address the requirements for surface roughening applicable to Bridge *Construction* (in contrast to Bridge *Design*). Construction contractors means and methods are governed by these specifications, not by the Design Specifications outlined above.

First, Section 400 of the FDOT Standard Specification is applicable to the construction of concrete structures or components, except for pavements or incidental concrete. Section 400-9 relates to concrete construction joints, and 400-9.3 states:

> *Before depositing new concrete on or against concrete which has hardened, re-tighten the forms. Roughen the surface of the hardened concrete in a manner that will not leave loosened particles, aggregate, or damaged concrete at the surface. Thoroughly clean the surface of foreign matter and laitance and saturate it with water.*

Second, the AASHTO LRFD Bridge Construction Specifications (second edition, 2004) addresses construction joints in section 8.8, and 8.8.2 states:

> *Unless otherwise specified in the contract documents, horizontal joints may be made without keys, and vertical joints shall be constructed with shear keys. Surfaces of fresh concrete at horizontal construction joints shall be rough floated sufficiently to thoroughly consolidate the surface and intentionally left in a roughened condition. Shear keys shall consist of formed depressions in the surface covering approximately one-third of the contact surface. The forms for keys shall be beveled so that removal will not damage the concrete.*
>
> *All construction joints shall be cleaned of surface laitance, curing compound, and other foreign materials before fresh concrete is placed against the surface of the joint.*

[4] Drawings B-37, B-38, B-41, B-42, B-47, B-48, B-49, and B-82 (which did not define any specific roughening amplitude); Drawing B-104 (which specified an amplitude of 1/8-inch for construction joint at top of the end bent); and Drawings B-24B and B-25 (which specified a surface roughness amplitude of ¼-inch at Substructure construction joints).

> *Abrasive blast or other approved methods shall be used to clean horizontal construction joints to the extent that clean aggregate is exposed. All construction joints shall be flushed with water and allowed to dry to a surface dry condition immediately prior to placing concrete.*

Notably, neither of these two standards provides a "standard" or specific definition of roughening or a specific amplitude for surface roughening applicable in all situations. Indeed, this is consistent with general industry practice in which different surface roughening procedures, and different amplitudes, are implemented depending on the specific project and on the designers' exact specifications for the same.

Therefore, MCM once again requests that the NTSB not adopt the incorrect definitional language used in the NTSB "Materials Laboratory Study Report" issued on August 27, 2019 and Turner-Fairbank Highway Research Center Factual Report titled, "Concrete Interface Under Members 11 and 12" dated October 19, 2018.

The following values shall be taken for cohesion, c, and friction factor, μ:

- For normal-weight concrete placed monolithically:

$$c = 0.40 \text{ ksi}$$
$$\mu = 1.4$$
$$K_1 = 0.25$$
$$K_2 = 1.5 \text{ ksi}$$

- For normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in.

$$c = 0.24 \text{ ksi}$$
$$\mu = 1.0$$
$$K_1 = 0.25$$
$$K_2 = 1.5 \text{ ksi}$$

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

$$c = 0.075 \text{ ksi}$$
$$\mu = 0.6$$
$$K_1 = 0.2$$
$$K_2 = 0.8 \text{ ksi}$$

Figure 1. Excerpts from AASHTO LRFD Section 5.8.4.3.

Declaration of Pedro Cortes

1. I, Pedro Cortes, make this declaration for submission to the National Transportation Safety Board ("NTSB") as part of its investigation into the March 15, 2018 collapse of the Main Span of the FIU/Sweetwater Pedestrian Bridge ("Bridge"), NTSB Accident HWY18MH009.

2. I have personal knowledge of the facts set forth herein, and if called upon I could and would competently testify hereto.

3. I am a U.S. Citizen, born in Honduras and employed by Magnum Construction Management, Inc. f/k/a Munilla Construction Management, Inc. ("MCM") in Miami, Florida as a Quality Control Technician, a position I have held continuously since 2013. My job responsibilities include ensuring the quality control of certain MCM projects by ensuring that the construction aspects of those projects are performed according to design and construction specifications. In that role, I conduct physical inspections of job sites and jobs, although I do not conduct structural testing.

4. Beginning in August of 2017, I was MCM's Quality Control Technician for the Bridge project, but was and am also assigned to other MCM jobs. I was standing on the Bridge canopy during the re-stressing of the Span on March 15, 2018 when the Bridge collapsed at approximately 1:47pm Eastern Time.

5. As a result of the collapse, I sustained severe injuries, including multiple lacerations, contusions, and fractures, including a skull fracture. During the several weeks following the collapse, I was in and out of the hospital for surgeries and other treatment. During that time period while I was unable to work and was recuperating, on April 11, 2018, NTSB investigators interviewed me at my home.

6. While the Span was being constructed, I observed preparation of the construction joints between the deck and the trusses. I observed that the interfacing surfaces were cleaned and roughened during the finishing of the concrete after placement, as is customary in the industry when placing construction joints. Specifically, the area around Node 11/12 where the Bridge apparently failed was cleaned of all debris prior to the concrete pour of the truss and readied for the pour using a chipping hammer to ensure a proper bond.

I declare under penalty of perjury under the laws of the State of Florida that the foregoing is true and correct.

Executed this 20 day of September 2019, at Miami, Florida

[redacted]

Pedro Cortes, Declarant