Pedestrian Bridge Collapse Over SW 8th Street
Miami, Florida
March 15, 2018

Accident Report

NTSB/HAR-19/02
PB2019-101363

National
Transportation
Safety Board

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Highway Accident Report

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National
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490 L'Enfant Plaza, S.W.
Washington, D.C. 20594

National Transportation Safety Board. 2019. ***Pedestrian Bridge Collapse Over SW 8th Street, Miami, Florida, March 15, 2018*****. Highway Accident Report NTSB/HAR-19/02. Washington, DC.**

Abstract: On Thursday, March 15, 2018, about 1:46 p.m., a partially constructed pedestrian bridge crossing an eight-lane roadway in Miami, Florida, experienced a catastrophic structural failure in the nodal connection between truss members 11 and 12 and the bridge deck. The 174-foot-long bridge span fell about 18.5 feet onto SW 8th Street, which consists of four through travel lanes and one left-turn lane in the eastbound direction, and three through travel lanes in the westbound direction. Two of the westbound lanes below the north end of the bridge were closed to traffic at the time of the collapse; however, one westbound lane and all five eastbound lanes were open. On the day of the collapse, a construction crew was working on retensioning the post-tensioning rods within member 11, connecting the bridge canopy and the deck at the north end. Eight vehicles located below the bridge were fully or partially crushed. One bridge worker and five vehicle occupants died. Five bridge workers and five other people were injured.

The investigation focused on the following safety issues: bridge design and construction plan errors, and unique bridge characteristics and mechanisms of failure; independent peer review of complex bridge design; shortcomings in oversight of evaluation of and response to significant observed bridge structure distress prior to collapse; and lack of redundancy guidelines in specifications for pedestrian and concrete truss bridges.

As a result of this investigation, the National Transportation Safety Board (NTSB) makes new safety recommendations to the Federal Highway Administration, the Florida Department of Transportation, the American Association of State Highway and Transportation Officials, and FIGG Bridge Engineers, Inc.

Contents

Figures and Tables

Acronyms and Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AASHTO LRFD	*AASHTO LRFD Bridge Design Specifications*
ABC	accelerated bridge construction
ACI	American Concrete Institute
BIRM	*Bridge Inspector's Reference Manual* [FHWA]
Bolton, Perez	Bolton, Perez and Associates Consulting Engineers
BUILD	Better Utilizing Investments to Leverage Development [DOT]
CEI	construction engineering and inspection (or construction engineering inspector)
CFR	*Code of Federal Regulations*
DBB	conventional design bid–build
D/C	demand-to-capacity
DOT	US Department of Transportation
EOR	engineer of record
ERC	Electronic Review Comments
FAC	*Florida Administrative Code*
FAST	Fixing America's Surface Transportation (Act)
FDOT	Florida Department of Transportation
FIGG	FIGG Bridge Engineers, Inc.
FHWA	Federal Highway Administration
FIU	Florida International University
FIUPD	Florida International University Police Department
kip	1,000 pounds-force

ksi	kilopound per square inch
LAP	local agency program (agreement)
LRFD	Load and Resistance Factor Design
$\mu\varepsilon$	microstrain
MDFR	Miami-Dade Fire Rescue
MDPD	Miami-Dade Police Department
MOT	maintenance of traffic
η_R	redundancy factor
NTSB	National Transportation Safety Board
P.E.	professional engineer
PPM	*Plans Preparation Manual* (FDOT)
psi	pounds per square inch
PT	post-tensioning (rod or tendon)
SDO	Structures Design Office [FDOT]
SPMT	self-propelled modular transporter
TAP	Transportation Alternatives Program [DOT]
TFHRC	Turner-Fairbank Highway Research Center [FHWA]
TIGER	Transportation Investment Generating Economic Recovery [DOT]
UCPP	University City Prosperity Project [FIU]
USC	*United States Code*

Glossary of Bridge-Related and Other Terms

Terms italicized at first mention within definitions are included in the glossary. Any terms taken from the Federal Highway Administration (FHWA) *Bridge Inspector's Reference Manual* are denoted with "BIRM" (FHWA 2012).[1]

Abutment: Part of the bridge ***substructure*** at either end that transfers ***loads*** from the ***superstructure*** to the foundation and provides lateral support for the approach roadway embankment (BIRM).

Accelerated bridge construction (ABC): Construction that uses innovative planning, design, materials, and methods in a safe and cost-effective manner to reduce onsite construction time when building new bridges or replacing or rehabilitating existing bridges.

Axial force: The compression or tension force acting in a structural member.

Bearing: A support element that transfers loads from the superstructure to the substructure while permitting limited movement (BIRM).

Bending moment: Reaction induced when an external force is applied to a structural element.

Blister: A concrete block cast on the top or side of a concrete ***member*** that typically provides access to a ***post-tensioning*** anchorage.

Bridge load rating: Determination of the live load-carrying capacity of a bridge using bridge plans and supplemented by information from a field inspection (BIRM).

Canopy: Top horizontal member of the Florida International University (FIU) pedestrian bridge.

Cantilever: A structural member that has a free end projecting beyond a support; or a length of span overhanging a support (BIRM).

Capacity: Ability of a structure to resist applied loads.

Chamfer: A transitional edge between two faces of an object, sometimes defined as a form of bevel.

Chord: A generally horizontal member of a ***truss*** (BIRM).

Clamping force: The compressive (vertical) force that contributes to ***interface shear*** resistance.

Cold joint: A joint or discontinuity resulting from a delay in concrete placement of sufficient duration that the freshly placed concrete cannot intermingle with the previously placed, already hardened, concrete.

[1] See Bridge Inspector's Reference Manual, accessed September 23, 2019.

Compression: A type of stress involving pressing together, which tends to shorten a member; the opposite of ***tension*** (BIRM).

Compression member: Any structural member subjected to a compressive force. In a truss bridge, some structural members (***chord*** or ***diagonal***) are always under ***compression***; some are always under tension; and some, depending on the configuration of the structure and the loading, change from compression to tension and vice versa.

Concrete competency: Refers to whether fresh concrete is appropriately consolidated; whether any segregation occurred during placement or consolidation; and whether layers of unvibrated concrete might have formed, particularly adjacent to formwork. Concrete vibration—which removes air bubbles that have the potential to substantially weaken the structure—is a critical step once concrete has been poured.

Concrete truss bridge: The FIU bridge was designed as a two-***span***, single-plane concrete truss containing longitudinal, ***transverse***, and truss member post-tensioning. The truss structure was complemented architecturally with a central pylon and steel pipe stays. Concrete truss bridges are exceedingly rare. Research has revealed no other designs similar to the FIU bridge. Generally, truss bridges are constructed primarily of steel.

Confinement reinforcement: Placement of reinforcement bars normally oriented transverse to the primary axis of a reinforced concrete member, whose purpose is to surround and strengthen the core of the member.

C-pier: A ***cantilevered*** cap that extends from one side of the ***pier*** column with the footing offset from the pier, in the same direction as the cap, to resist overturning moments. The final shape form is similar to the letter "c." A c-pier is typically used when an obstacle below, usually a roadway, would conflict with the normal placement of the pier.

Curing: A process that begins immediately after concrete is placed and finished, and involves maintaining moisture and temperature conditions throughout the concrete for an extended period of time (BIRM).

Dead load: Static load due to the weight of a structure itself; also referred to as self-weight (BIRM).

Deck: Portion of a bridge that provides direct support for vehicular and pedestrian traffic, supported by a superstructure (BIRM).

Deck truss bridge: A truss bridge with the truss structure underneath the roadway or walkway, which supports traffic or pedestrians traveling along the top of the main structure.

Demand: Design loads imposed on structural members that need to be resisted or supported by the structure.

Design–build: A system of contracting whereby one entity performs both architectural/engineering design and construction under a single contract.[2]

Diagonal: A sloping structural member of a truss or bracing system (BIRM). The FIU bridge diagonals connected the bridge ***canopy*** and the bridge ***deck***.

Diaphragm: A transverse member placed within a member or superstructure system to distribute stresses and improve strength and rigidity (BIRM).

Distress: A physical manifestation of deterioration that is apparent on or within a structure, including cracking, delamination, and spalling of concrete.

End bent: A support at the end of a bridge structure that transfers and resists vertical and lateral loads; consists of columns and a cap beam.

Falsework: A temporary wooden or metal framework built to support the weight of a structure during construction and until it becomes self-supporting (BIRM).

Fatigue: Tendency of a member to fail at a stress below the yield point when subjected to repetitive loading (BIRM).

Finite element analysis: Analysis of a structure based on a computer model of its material or design. A finite element analysis model describes a virtual assembly of simplified structural elements to approximate a complex structure. The behavior of the structure is then calculated by combining the actions of the interconnected simpler elements.

Fracture-critical member: A steel member in tension, or with a tension element, whose failure would likely cause a portion of a bridge or an entire bridge to collapse (BIRM).

Girder: A horizontal flexural member that is the main or primary support for a structure; any large beam, especially if built up (BIRM).

Interface shear surface: The contact area between two concrete elements that transfers opposing forces across the joint. In the case of a ***cold joint***, the roughness (friction) and associated cohesion across the interface ***shear*** surface and the magnitude of the forces compressing the two surfaces provide resistance to interface shear.

Horizontal component: Shearing force on the ***interface shear surface*** at the end of an inclined or vertical truss member.

Laitance: A thin, flaky layer of hardened but weak hydrated cement and fine sand; occurs when cement and fine aggregates rise to the surface, commonly a result of excess water in the cement.

[2] See the Design-Build Institute of America website, accessed September 23, 2019.

Lap splice: A connection created by overlapping two lengths of ***rebar***. From a structural point of view, the most critical aspect of a lap splice is the length of overlap—requirements for which vary with both rebar size and the specific structural application.

Live load: A temporary dynamic load, such as vehicular traffic, that is applied to a structure; also accompanied by vibration or movement affecting its intensity (BIRM).

Load: A force carried by a structure component (BIRM).

Load posted: A limited loading indicating that a bridge cannot safely take greater loads (BIRM).

Lower chord: The bottom horizontal, or almost horizontal, member of a truss; often consists of multiple shorter chord members connected at ***nodes***.

Megashores: An ultra-heavy-duty modular ***shoring*** and propping system designed for high axial forces.

Member: An individual angle, beam, plate, or built-up piece intended to become an integral part of an assembled frame or structure (BIRM). Members are the major structural elements of the truss (chords, diagonals, and ***verticals***).

Node (or nodal region): Located at any part of a bridge in which truss members (chords, diagonals, and verticals) are connected. In the FIU bridge, the canopy was the top chord, and the deck was the bottom chord.

Nondestructive evaluation: Also referred to as nondestructive testing or nondestructive inspection, this evaluation does not damage the test object. Technologies for nondestructive evaluation include x-ray and ultrasound sensors that can detect such defects as cracking and corrosion.

Nonredundant structure: A structure with fewer load paths (or main supports) than necessary to maintain stability following the failure of a critical component, likely resulting in its collapse.

Pier: A substructure unit that supports the spans of a multispan superstructure at an intermediate location between its ***abutments*** (BIRM).

Post-tensioning: A method of prestressing concrete using steel rods or strands that are stretched after the concrete has hardened. This stretching puts the concrete in compression, with the compressive stresses intended to counteract tensile (tension) forces experienced by the concrete.

Post-tensioning (PT) rod: Prestressing steel rod inside a plastic duct or sleeve, positioned in the formwork before the placement of concrete. PT rods are large-diameter threaded rods secured with large nuts and anchor plates to lock their ends in place so they can be tensioned and/or detensioned as necessary. A PT rod is tensioned after the concrete has gained strength but before service loads are applied to the structure.

PT tendon: Strand of PT wire that is tensioned, then held taut by clamps at each end, and typically cannot be detensioned without cutting the strands. PT tendons were located in the main span bridge deck and canopy.

Rebar: Reinforcing steel bars often used in concrete structures for added strength and stability. Standard rebar classifications rate the bars by diameter as follows: size 4 = 0.50 inch, size 5 = 0.625 inch, size 6 = 0.75 inch, size 7 = 0.875 inch, size 8 = 1.0 inch, size 9 = 1.128 inch, size 10 = 1.27 inch, and size 11 = 1.41 inch.

Redundancy: The capability of a bridge structural system to carry loads after damage to, or the failure of, one or more of its members.

Reinforced concrete: Concrete to which steel is embedded such that the two materials act together in resisting forces. The reinforcing steel (rods, bars, tendons, etc.) helps to absorb the stresses in a concrete structure.

Rocker bearing: A bridge support that accommodates expansion and contraction of the superstructure through a tilting action (BIRM).

Roller bearing: A bridge support that consists of a single roller or a group of rollers housed so as to permit longitudinal thermal expansion or contraction of a structure.

Self-propelled modular transporter (SPMT): A platform vehicle with a large array of wheels. SPMTs are used for transporting massive objects—such as large bridge sections, oil refining equipment, and motors—that are too big in scale or too heavy for truck transport.

Shear: A force that causes parts of a material to slide past one another in opposite directions.

Shim stack: Multiple layers (or plates) of a material (a shim) stacked to provide support—in this case, to support the main span during permanent placement; a shim plate is a single layer.

Shoring: A process of temporarily supporting a structure with shores (props) to prevent collapse or during repairs or alterations.

Span: Horizontal space between two supports of a structure. A simple span rests on two supports, one at each end, the stresses on which do not affect the adjoining spans. A continuous span consists of a series of consecutive spans (three or more supports) that are rigidly connected (without joints) so that ***bending moment*** and shear are transmitted from one span to another.

Specifications: A detailed description of requirements, materials, and tolerances for construction that are not shown on drawings; also known as "specs" (BIRM).

Stirrup: A steel bar bent into a "U" or box shape and installed perpendicular to, or at an angle to, the longitudinal reinforcement; used to resist shear and diagonal tension stresses in a concrete structural member.

Strain: Ratio of the change in length of a material to the original unaffected length; may be compressive or tensile.

Strut: A region of concrete, internal to the structure, that carries compressive forces along the load path. The strut is a component of the strut and tie modeling calculation methodology by which complex stress patterns within reinforced concrete structures are modeled as triangular truss elements; the methodology can be applied to many concrete structural elements.

Substructure: Bridge structure that supports the superstructure and transfers loads from it to the foundation; main components are abutments, piers, footings, and pilings.

Superstructure: Bridge structure that receives and supports traffic or pedestrian loads and, in turn, transfers those loads to the substructure; includes the bridge deck, structural members, parapets, handrails, sidewalk, lighting, and drainage features.

Tendon: A prestressing steel cable, strand, or bar that provides a clamping load to produce compressive stress to balance tensile stress.

Tension: Stress that tends to pull apart material; the opposite of compression (BIRM).

Tension truss member: Any member of a truss that is subjected to tensile (tension) forces. In a truss bridge, some structural members are always under compression; some are always under tension; and some, depending on the structural configuration and loading, change from compression to tension and vice versa.

Transverse: Perpendicular to the longitudinal axis; a transverse member helps distribute stresses and improves strength and rigidity.

Truss: A bridge superstructure made up of members whose ends are linked at nodes. The structure is composed of connected elements, typically forming triangular units, where the members act as a single object.

Upper chord: Top horizontal, or almost horizontal, member of a truss. The upper chord often consists of multiple shorter chord members connected at nodes.

Vertical component: Compressive or ***clamping force*** on the interface shear surface at the end of an inclined or vertical truss member that contributes to interface shear resistance.

Vertical truss member: A vertical member connecting the upper and lower chords at nodes.

Yield stress: Stress above the elastic limit at which permanent (plastic) deformation occurs.

Executive Summary

Investigation Synopsis

On Thursday, March 15, 2018, about 1:46 p.m., a partially constructed pedestrian bridge crossing an eight-lane roadway in the city of Miami, in Miami-Dade County, Florida, experienced a catastrophic structural failure in the nodal connection between truss members 11 and 12 and the bridge deck. The 174-foot-long bridge span fell about 18.5 feet onto SW 8th Street, which consists of four through travel lanes and one left-turn lane in the eastbound direction, and three through travel lanes in the westbound direction. Two of the westbound lanes below the north end of the bridge were closed to traffic at the time of the collapse; however, one westbound lane and all five eastbound lanes were open.

The pedestrian bridge was under construction as part of the Florida International University University City Prosperity Project. On the day of the collapse, a construction crew was working on retensioning the post-tensioning rods within member 11, connecting the bridge canopy and the deck at the north end. About 1:46 p.m., a video camera on a construction pickup truck traveling east, approaching the bridge, recorded the collapse sequence. The video showed the blowout of the concrete north of truss member 12, and the truss losing geometric stability. Eight vehicles that were located below the bridge were fully or partially crushed, seven of which were occupied. One bridge worker and five vehicle occupants died. Five bridge workers and five other people were injured.

Probable Cause

The National Transportation Safety Board (NTSB) determines that the probable cause of the Florida International University (FIU) pedestrian bridge collapse was the load and capacity calculation errors made by FIGG Bridge Engineers, Inc., (FIGG) in its design of the main span truss member 11/12 nodal region and connection to the bridge deck. Contributing to the collapse was the inadequate peer review performed by Louis Berger, which failed to detect the calculation errors in the bridge design. Further contributing to the collapse was the failure of the FIGG engineer of record to identify the significance of the structural cracking observed in this node before the collapse and to obtain an independent peer review of the remedial plan to address the cracking. Contributing to the severity of the collapse outcome was the failure of MCM; FIGG; Bolton, Perez and Associates Consulting Engineers; FIU; and the Florida Department of Transportation to cease bridge work when the structure cracking reached unacceptable levels and to take appropriate action to close SW 8th Street as necessary to protect public safety.

Safety Issues

The investigation of the collapse of the FIU pedestrian bridge focused on the performance of the northernmost nodal region (11/12 node) of the 174-foot-long main span. The failure of this nodal region was the triggering event for the bridge collapse. Factors in the collapse included bridge design errors, inadequate peer review of the bridge design, poor engineering judgment and response to the cracking that occurred in the region of eventual failure, and lack of redundancy in the bridge design. Specifically, the investigation focused on the following safety issue areas:

- **Bridge design and construction plan errors (section 2.3) and unique bridge characteristics and mechanisms of failure (section 2.4).** The uniqueness of designing a concrete truss bridge led to the circumstances that accounted for the collapse of the pedestrian bridge. The bridge design team made two errors that resulted in the under-design of the nodal area (11/12) that failed, resulting in the collapse. First, the design team underestimated the demand (loads imposed on structural members) that would be acting on the nodal area. The investigation compared postcollapse calculations for the demands on the node with the design calculations. This comparison found that the demand for the node was nearly twice what the design team had calculated. The investigative report discusses how this error was made. Second, the design team also overestimated the capacity of the node to resist shear (horizontal force) where the nodal region (11/12) was connected to the bridge deck. This overestimation was the result of the designer using incorrect loads and load factors in its calculations. These two design errors resulted in a node that lacked the capacity to resist the shear force pushing the node to the end of the bridge. The NTSB recommends improving the discussion of calculating demand loads and capacity resistance in bridge design guides.
- **Independent peer review of complex bridge design (section 2.5).** Errors in design may occur, but systems should be in place to catch those errors when they do occur. In this case, a firm was hired to independently review the bridge design for errors. However, the review conducted by this firm did not evaluate the nodes of the bridge truss where they connected with the bridge deck and canopy, nor did it consider the multiple stages the bridge construction involved. Although the design reviewer recognized that he should have examined the nodes and stages, he indicated that there was not enough budget or time to evaluate those factors. Contributing to this review failure was the reviewing firm’s lack of qualification to do the work. Further, no specific guidelines call for nodes or construction stages to be included in independent bridge design reviews. The NTSB recommends changes to bridge design review procedures to ensure that bridge nodes and construction stages are included in independent design reviews.
- **Shortcomings in oversight of evaluation of and response to significant observed bridge structure distress prior to collapse (section 2.6).** As soon as the bridge had to support its own weight, cracks appeared at the under-designed nodes, particularly node 11/12. Over the next 19 days, the cracks grew until the bridge collapsed. The construction and inspection firms working on the bridge were aware of the cracks and reported the cracks to the design firm, asking for guidance. The engineer of record at the design firm repeatedly indicated that the cracks were of no safety concern. On the day of the collapse, the firms met to discuss a plan by the engineer of record to remediate the cracks. The bridge collapsed as the firms were implementing the remediation plan. In addition, the repair work was conducted without closing the road below the bridge to traffic. The NTSB recommends changes to Florida bridge construction oversight procedures to emphasize the need for bridge and road closures

to protect public safety when structural cracking (beyond what sound engineering judgment considers acceptable) occurs and to increase state oversight of complex bridge construction.

- **Lack of redundancy guidelines in specifications for pedestrian and concrete truss bridges (section 2.7).** The design of the pedestrian bridge did not include redundancy in the bridge load path. As a result, when the 11/12 nodal region failed, the bridge collapsed. The design firm incorrectly believed that the bridge had a redundant design. For typical bridge designs, a bridge designer would use a safety factor greater than one to ensure that the bridge was over-designed to prevent a collapse. The NTSB, recognizing that no design guidance exists discussing redundancy in concrete truss bridges, recommends that bridge design guides include a discussion of redundancy in concrete bridge designs.

Recommendations

As a result of this investigation, the NTSB makes safety recommendations to the Federal Highway Administration, the Florida Department of Transportation, the American Association of State Highway and Transportation Officials, and FIGG Bridge Engineers, Inc.

1 Factual Information

1.1 Pedestrian Bridge Collapse

1.1.1 Location and Design

The collapse of the Florida International University (FIU) University City Prosperity Project (UCPP) pedestrian bridge occurred in the city of Miami, in Miami-Dade County, Florida. The bridge site is located 11 miles west of downtown Miami, on the west side of the intersection of SW 8th Street and SW 109th Avenue (figure 1). Also designated as US Highway 41 and State Route 90, SW 8th Street is an eight-lane highway with a speed limit of 45 mph.

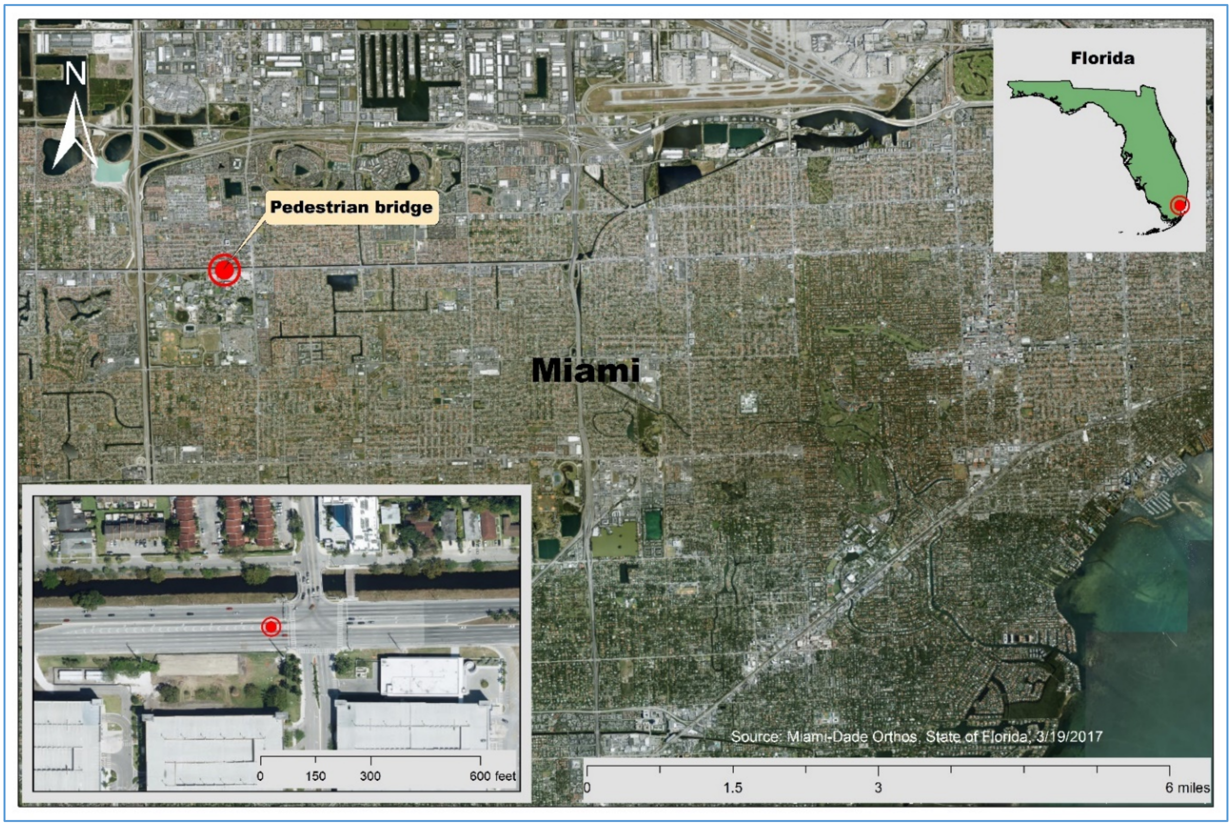

Figure 1. Location of SW 8th Street–SW 109th Avenue intersection, with inset maps showing closeup view of intersection and Miami's location in south Florida.

The pedestrian bridge was to serve as an elevated transit bridge for pedestrians and bicyclists crossing the travel lanes of SW 8th Street and the Tamiami Canal. The main bridge section—spanning the south pier to the pylon pier—was 174 feet long, and the walking deck surface was elevated 18.5 feet. The back span, as designed, was to extend 99 feet from the pylon pier, across the Tamiami Canal, and end at the north pier at the same height elevation (see figure 2).[1]

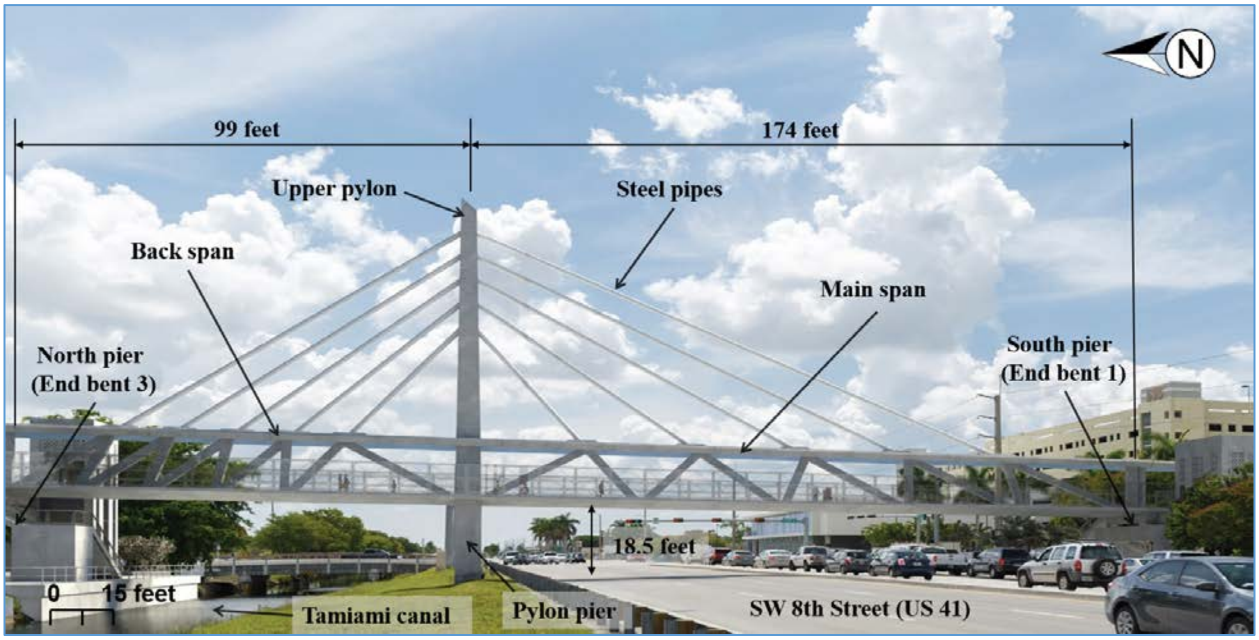

Figure 2. East-view rendering of completed UCPP pedestrian bridge. Note: This depiction does not illustrate the 11-foot northward shift of the bridge to accommodate a future westbound express bus lane on SW 8th Street or the construction of a bulkhead wall on the south Tamiami Canal bank. (Source: FIU, annotated by NTSB)

Architecturally, the bridge featured a 109-foot-tall upper pylon; 10 diagonal steel pipes, with lights, connecting the canopy to the upper pylon; a staircase at the north end and a grand staircase at the south end; and north and south elevators from the deck to street level. The bridge design included an almost 32-foot-wide concrete deck and an overhead concrete canopy connected vertically by a single row of concrete diagonal and vertical supports in the center. The bridge canopy, 15 feet above the deck, was approximately 16 feet wide.

A 4-foot-wide raised concrete median separated the left-turn lane (and four through lanes) of SW 8th Street eastbound from the three westbound lanes. The total distance from the south curb

[1] (a) See the glossary for definitions of bridge-related and other terms used in this report. (b) The south pier, pylon pier, and north pier are also referred to in construction plans as piers 1, 2, and 3, respectively. Throughout this report, they are referred to as the south, pylon, and north piers. These piers have also been referred to as "end bent" locations; however, for reader orientation, this report labels them as "piers." (c) During the design process, the Florida Department of Transportation (FDOT) requested that the bridge have the flexibility to accommodate a 12-foot-wide westbound express bus lane on the north side of SW 8th Street. As a result, FIGG Bridge Engineers revised the general plan and elevation, the general notes (2 of 2), the bridge hydraulics recommendation sheet, and the foundation layout drawings to show the new locations of the pylon pier and the north and south plaza landing areas. A new bulkhead wall on the south side of the Tamiami Canal was also introduced. The FDOT Structures Design Office did not require an independent peer review of the revised documents, because there were no alterations or revisions to the bridge structural design. The new locations increased the horizontal clearance from the edge of the pavement to the face of the pylon pier from 5 feet 6.25 inches to 16 feet 6.25 inches (a difference of 11 feet).

line to the north curb line was about 115 feet.[2] Figure 3 shows the FIU design rendering for the completed bridge project.

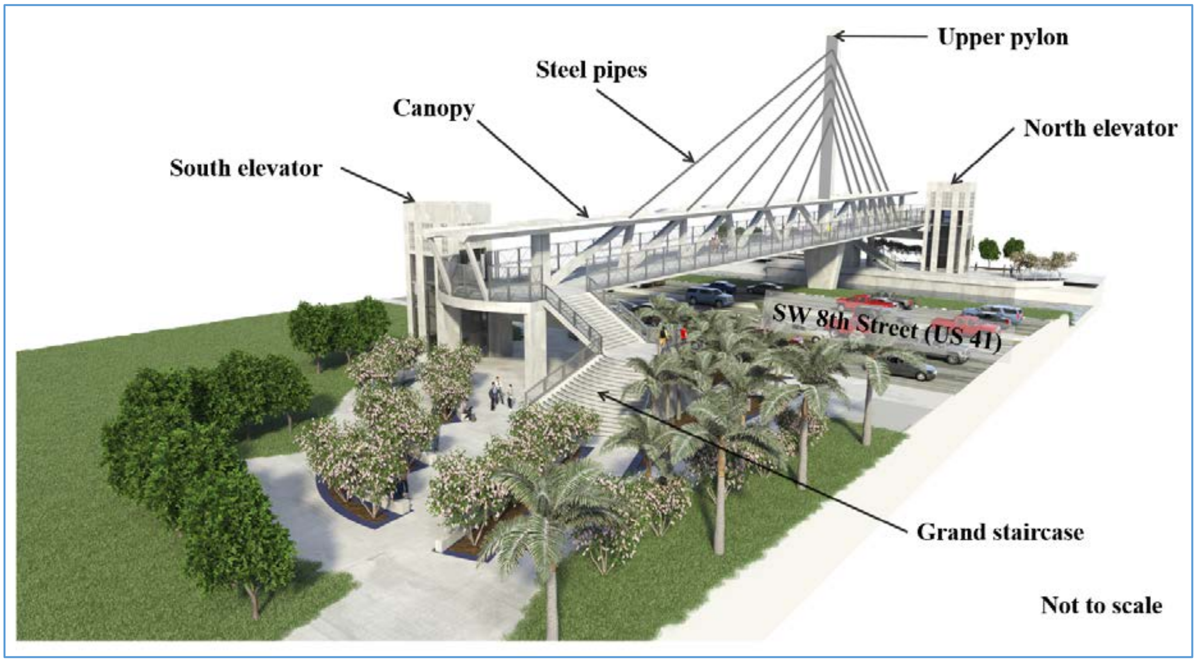

Figure 3. Design rendering of completed pedestrian bridge. (Source: FIU, annotated by NTSB)

1.1.2 Precollapse and Collapse Timeline of Events

On March 15, 2018, about 1:46 p.m., the partially constructed bridge main span was in place on the south pier and pylon pier when it experienced a catastrophic failure in the truss member 11/12 nodal region and bridge deck.[3] At the time of the collapse, vehicular traffic was stopped below the main span for a red light for SW 8th Street at the traffic-controlled intersection with SW 109th Avenue (figure 4). A construction crew was working on retensioning internal post-tensioning (PT) rods within a main span truss member (11) that connected the bridge canopy and the deck at the north end (at the pylon pier).[4] (See appendix A for additional information on the National Transportation Safety Board [NTSB] launch to the scene and parties to the investigation.)

[2] In 2017, the average annual daily traffic in the vicinity of the pedestrian bridge was 60,000 vehicles.

[3] At 1:53 p.m. on the day of the collapse, the Miami International Airport weather station, located about 6 miles east-northeast of the bridge site, reported a temperature of 73°F, clear skies, winds from the north at 5 mph, and visibility unrestricted at 10 statute miles or more. The roadway surfaces were dry, with no precipitation reported in the previous 24 hours.

[4] (a) Throughout this report, we refer to PT rods in the diagonal members and longitudinal and transverse PT tendons in the deck and canopy. (b) PT rods are large-diameter threaded rods secured with large nuts and anchor plates to lock them in place so they can be tensioned and/or then detensioned as necessary. (c) PT tendons are strands of post-tensioning wire that—once tensioned—are held taut by clamps at each end and typically cannot be detensioned without cutting the strands.

Figure 4. Still images from FIU parking garage camera, showing east views of pedestrian bridge, March 15, 2018, precollapse (top) and postcollapse (bottom). (Source: FIU video camera)

1.1.3 Emergency Response

The Miami-Dade Police Department (MDPD) received the 911 call reporting that the pedestrian bridge had collapsed at 1:47 p.m. Multiple units were dispatched and en route by 1:49 p.m.; and they arrived on scene by 1:52 p.m. The FIU Police Department (FIUPD), with jurisdiction for the university campus (including the area surrounding the pedestrian bridge), dispatched officers at 1:48 p.m.; they were on scene by 1:52 p.m.[5] The Sweetwater Police Department, with jurisdiction for the municipality of Sweetwater to the north of the FIU campus, also initiated a response to the incident at 1:48 p.m. and immediately dispatched officers. The Florida Highway Patrol, sharing jurisdiction for the area with the MDPD, received the call at 1:56 p.m., and arriving units were on scene by 2:03 p.m. The city of Doral Police Department sent units to the scene for mutual aid at 2:03 p.m.

Miami-Dade Fire Rescue (MDFR), with jurisdiction for the location, dispatched 12 units starting at 1:49 p.m. The MDFR battalion chief assumed incident command for rescue and

[5] The FIUPD dispatched 14 units and 33 officers. The FIU emergency operations center was opened within minutes of the 911 call and provided support to emergency responders.

emergency medical service response. In total, 10 people were transported to the Kendall Regional Medical Center, and one person self-transported to the hospital.

1.2 Injuries

A six-person construction crew was working on the bridge at the time of the collapse. One worker was fatally injured, four were seriously injured, and one received minor injuries. In addition, seven occupied vehicles stopped below the bridge were fully or partially crushed.[6] Five vehicle occupants were fatally injured, two were seriously injured, and three received minor injuries. An eighth vehicle, parked in the westbound lanes, was unoccupied and partially crushed. (See figure 5 and table 1.)

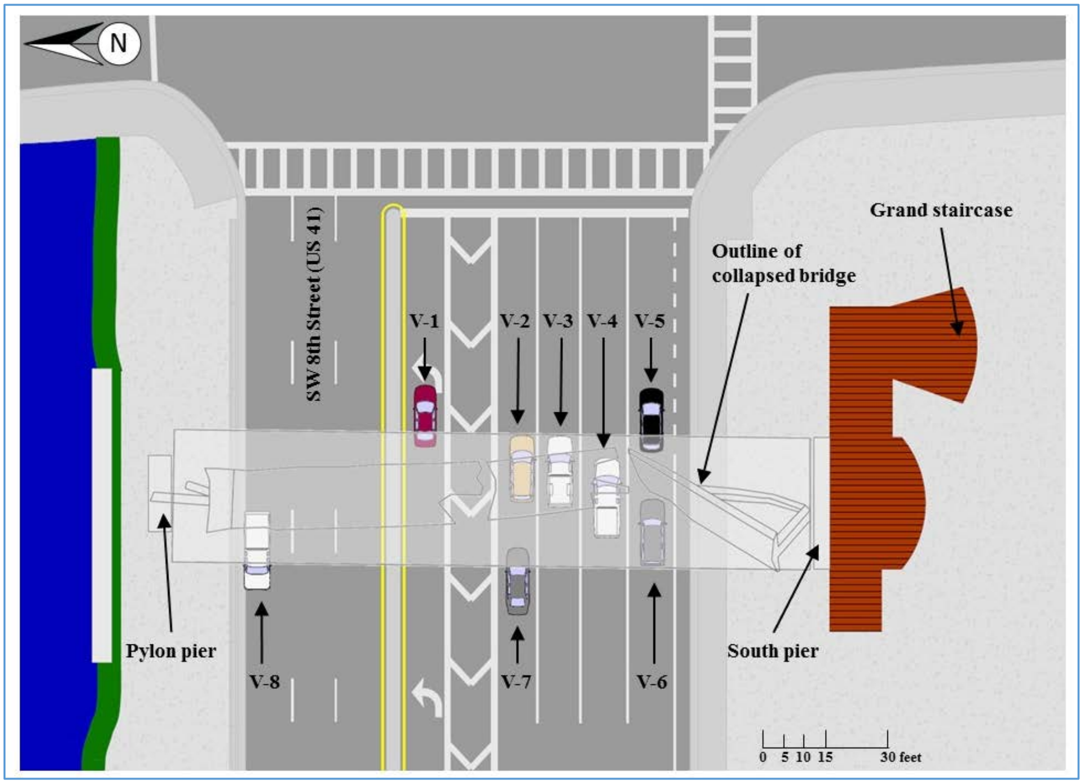

Figure 5. Scene diagram of vehicles under or proximate to bridge at time of collapse, with vehicle reference numbers.

[6] Vehicles 1–7 were traveling east on SW 8th Street at the time of the bridge collapse: (a) Vehicle 1: a 2008 Honda Civic, occupied by a 22-year-old female driver who received minor injuries. (b) Vehicle 2: a 2015 Jeep Cherokee, occupied by a 60-year-old male driver who was fatally injured. (c) Vehicle 3: a 2006 Chevrolet 1500 pickup truck, occupied by a 53-year-old male driver and a 57-year-old male passenger. Both were fatally injured. (d) Vehicle 4: a 2014 Ford F150 pickup truck, occupied by a 39-year-old male driver who was fatally injured. (e) Vehicle 5: a 2011 Nissan Rogue, occupied by a 32-year-old female driver who received minor injuries. (f) Vehicle 6: a 2008 Toyota 4-Runner sport utility vehicle, occupied by an 18-year-old female driver who was fatally injured and a 19-year-old male passenger who was seriously injured. (g) Vehicle 7: a 2015 Kia Optima, occupied by a 42-year-old female driver who received minor injuries and a 34-year-old male passenger who was seriously injured. (h) Vehicle 8: a 2014 Chevrolet 2500 pickup truck—unoccupied—owned by Structural Technologies, a construction contractor, was parked on the northwest shoulder of SW 8th Street, facing west.

Table 1. Injuries among bridge construction workers and vehicle occupants.

Injury Severity[a]	Fatal	Serious	Minor	None	TOTAL
Bridge construction workers	1	4	1	0	6
Vehicle occupants	5	2	3	0	10
TOTAL	6	6	4	0	16

[a] Although 49 *Code of Federal Regulations* (CFR) Part 830 pertains only to the reporting of aircraft accidents and incidents to the NTSB, section 830.2 defines fatal injury as any injury that results in death within 30 days of the accident, and serious injury as any injury that: (1) requires hospitalization for more than 48 hours, commencing within 7 days from the date of injury; (2) results in a fracture of any bone (except simple fractures of fingers, toes, or nose); (3) causes severe hemorrhages, nerve, or tendon damage; (4) involves any internal organ; or (5) involves second- or third-degree burns, or any burn affecting more than 5 percent of the body surface.

1.3 Characteristics of FIU Pedestrian Bridge

1.3.1 Design and Construction of the Bridge

The design and construction of bridges generally involve three basic stages: conceptual, design, and construction. A brief overview of this process is provided below to help the reader understand the detailed processes described later in the report.

For the conceptual stage, FIU decided that a pedestrian bridge was needed across SW 8th Street to link two portions of the campus that were disconnected by a heavily traveled highway. FIU, in cooperation with the city of Sweetwater, received Transportation Investment Generating Economic Recovery (TIGER) and Transportation Alternatives Program (TAP) grant funds from the US Department of Transportation (DOT) to design and construct the bridge; the project also included streetscaping and transit improvements. A local agency program (LAP) agreement was executed between FIU and the Florida Department of Transportation (FDOT) to establish consistent and uniform practices for authorizing local agencies (such as FIU) to use federal-aid funds. FIU issued a request for proposals that included the owner’s intent for the bridge and other related criteria. FIU entered into a contract with T.Y. Lin International to prepare a report that documented the pedestrian bridge design criteria, which included the architectural vision for the bridge and described the bridge requirements, specifications, and references. Although not a requirement of the design in the T.Y. Lin report, reference was made to accelerated bridge construction (ABC) and pedestrian bridge design concepts.

For the design stage, FIU entered into a design-build contract with MCM to construct the bridge and a standard professional services agreement with Bolton, Perez and Associates Consulting Engineers (Bolton, Perez) to administer, monitor, and inspect the bridge as it was constructed. MCM, the design builder, entered into a standard form of agreement with FIGG Bridge Engineers (FIGG), the design consultant, to provide professional design and engineering services that included final design, release for construction drawings, and specifications associated with the bridge. FIU coordinated each of these contracts with FDOT and the Federal Highway

Administration (FHWA), because federal funds were being expended on this project. Further, although FDOT had delegated its project oversight to FIU, when issues arose, FDOT was called in to consult.

All bridge projects should achieve the objectives of safety, constructability, and serviceability as prescribed in the *American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications* (*AASHTO LRFD*). These objectives are met through the theory of reliability based on current statistical knowledge of loads and structural performance. In *AASHTO LRFD* design, the anticipated loads or demands on the bridge are conservatively estimated, and the structural system is designed to have the capacity to resist those demands. FIGG entered into an agreement with the firm Louis Berger to perform an independent peer review of the bridge plans, which required an independent verification of the design to ensure that the bridge would be designed with the capacity to support the demands on it.

The final stage was the construction of the bridge, which was done by MCM, following FIGG's construction plans. Bolton, Perez was responsible for overseeing the construction for FIU and, by extension, FDOT.

1.3.2 Unique, Complex Bridge Design

The pedestrian bridge was designed to be a two-span, single-plane concrete truss bridge. It had a total length of 273 feet, with a main span of 174 feet and a back span of 99 feet. The main span crossed SW 8th Street, and the back span crossed the Tamiami Canal. The bridge design included a concrete deck and a concrete canopy connected by a single row of concrete diagonal and vertical support members, which extended down the center of the bridge. To replicate the aesthetics of a cable-stayed bridge, each diagonal truss member was designed to a different angle and length and aligned with 10 steel pipe stays (cables fanning out from a tall mast, referred to as the upper pylon; see figure 6).[7] This configuration created irregularly shaped diagonal truss members with different angles and lengths.

[7] The upper pylon was designed to extend approximately 109 feet high and symbolized the location of the cross street, SW 109th Avenue.

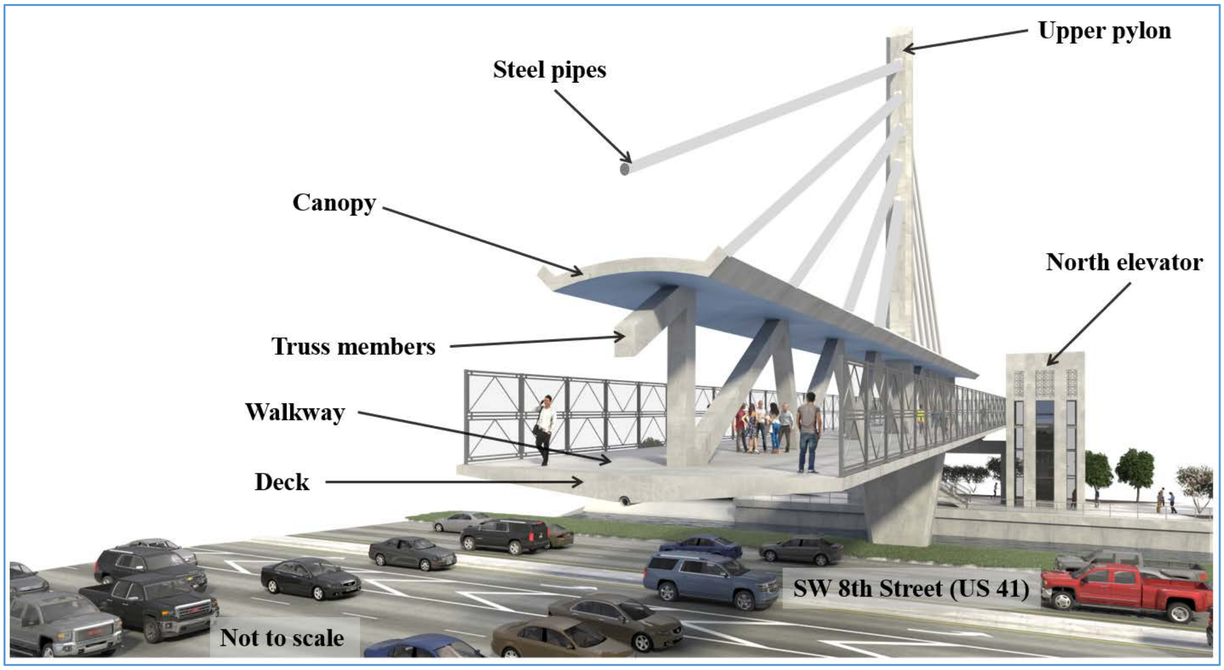

Figure 6. Cross-section rendering of pedestrian bridge, north view. (Source: FIU, modified by NTSB)

The main span included 12 truss members, whose placement and numbering are shown in figure 7. Truss members were aligned along the structure's centerline. Each of the members measured 1 foot 9 inches wide (transverse direction) and ranged from 2 to 3 feet deep (longitudinal direction). In addition to the upper pylon and 10 steel pipes, the design incorporated north- and south-end staircases and elevators. The steel pipes were functional structural members designed to increase the bridge's natural frequency—that is, to dampen the vibrations from pedestrian traffic. They were not load-carrying structural elements.

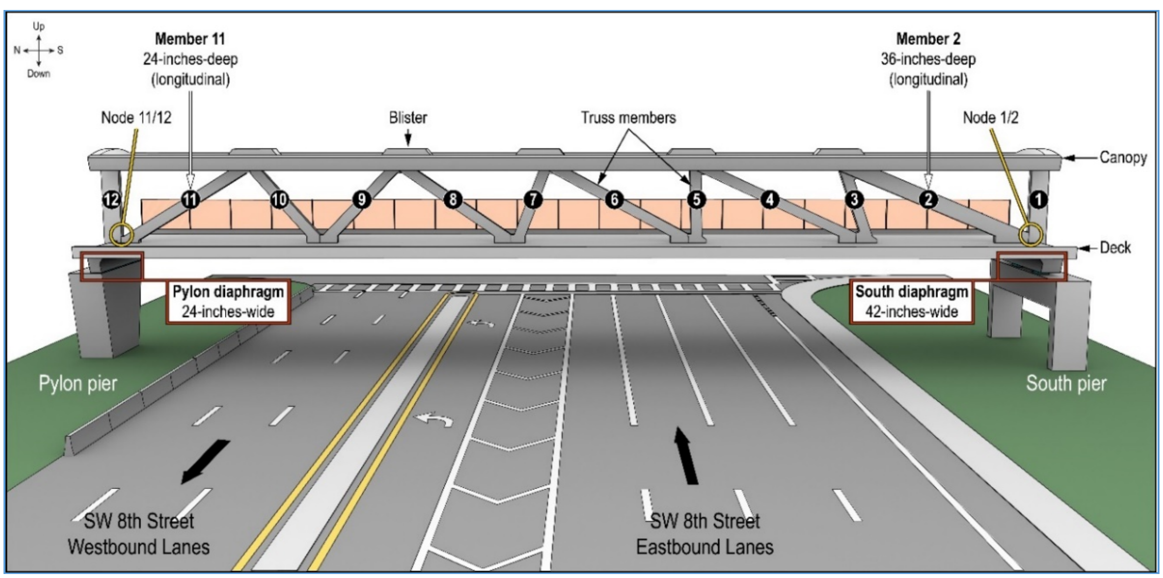

Figure 7. Nomenclature of bridge components and numbering of diagonal and vertical truss members on main span of pedestrian bridge, east view.

The concrete deck acted as the bottom chord of the truss and contained longitudinal and transverse post-tensioning, while the concrete canopy acted as the top chord of the truss and contained longitudinal post-tensioning.[8] Truss members were aligned along the centerline of the cross section. Main span truss members (numbered 3, 5, 6, 7, 8, and 10) were to be permanently post-tensioned via internal PT rods (truss members 1, 4, 9, and 12 were not post-tensioned). To address temporary construction conditions, members 2 and 11 were temporarily post-tensioned, again via internal PT rods (see section 1.6.4).

Five blisters were located on the top of the main span canopy to accommodate the truss member PT rod anchorages and to provide a platform for the steel pipe stays that connected to the upper pylon. The back span over the Tamiami Canal, the upper pylon, and the steel pipes had not yet been constructed at the time of the collapse.

1.3.3 Redundancy Requirements

A typical truss bridge design includes two parallel trusses that are connected with lateral supports and bracing. Although there is a wide range of variability in truss bridge design, trusses are often constructed using evenly spaced and similarly shaped triangular sections, running along each side of the bridge. The pedestrian bridge had a single truss constructed of asymmetrical triangular supports running down the center. Concrete truss bridges are rare; NTSB research found no other designs similar to the pedestrian bridge. Although truss bridges can be made from a variety of construction materials, they are typically constructed of steel (see figure 8) because of the inherent need for trusses to carry both compressive and tensile forces.[9]

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[8] Post-tensioning of concrete is a process by which the special reinforcing steel that is embedded in the structure to hold regions of concrete, which would normally be in tension, is stressed into a state of compression, even under loading. The post-tensioning steel is placed in ductwork within the structure before the concrete is poured. After the concrete has cured enough to develop sufficient strength to withstand the post-tensioning, the post-tensioning steel is stressed, and it pulls the concrete into compression.

[9] As a construction material, concrete performs well in compression but poorly in tension.

Figure 8. Typical truss bridge with one set of regularly spaced vertical truss pieces running along each side, with top lateral bracing constructed of steel. (Source: *Santa Clarita, California*)

For the design of structures, the *AASHTO LRFD* defines redundancy as "the quality of a bridge that enables it to perform its design function in a damaged state" and redundant member as "a member whose failure does not cause failure of the bridge" (AASHTO 2015). For the design of concrete structures, the *AASHTO LRFD* offers no specific discussion of redundancy or redundant members, nor do the FDOT *Structures Design Guidelines* (FDOT 2015b).

The introduction to the *AASHTO LRFD* states that all bridges shall be designed to achieve the objectives of constructability, safety, and serviceability (AASHTO 2015) and that multiple-load-path and continuous structures should be used unless there are compelling reasons not to use them. AASHTO recommends that a strength limit state a redundancy factor (η_R) of at least 1.05 for calculating the capacity of nonredundant members (and 1.00 for conventional levels of redundancy). The FDOT guidelines (2015b) use similar language for redundancy factors.

1.4 Video Study, Collapse Sequence, and Damage

A video camera mounted in the interior of a pickup truck traveling east (in the third lane from the right) on SW 8th Street on approach to the bridge recorded the collapse sequence.[10] The video shows the main span in precollapse condition, the blowout of the concrete north of member 12, the truss losing geometric stability, and the separation of the deck from the pylon pier.

The video includes the entire bridge span structure. Workers are visible on the canopy of the main span in the vicinity of truss member 10, and eastbound traffic is stopped underneath the bridge. Figure 9 shows the video frame with time stamp 13:46:43:881. Debris consistent with concrete dust blowout is observed on the north side of the main span diaphragm—at the location of the deck, diaphragm, and pylon pier.[11] Figure 10, the video still image with time stamp 13:46:44:046, shows the beginning collapse sequence, with the north end at truss member 10 hinging downward as compared with its position in the precollapse recorded images. The video captures the continued hinge movement of the main span structure, which rotates downward as a rigid structure. Full-width fracturing of the canopy is visible north of the member 10/11 nodal region and in the deck structure north of the member 9/10 nodal region. Figure 11, the video image with time stamp 13:46:44:310, shows the main span completely collapsed, with members 4, 3, 2, and 1 intact at the south end.

Figure 9. Still image (time stamp 13:46:43:881) from in-vehicle mounted video camera on pickup truck traveling east on SW 8th Street, showing concrete dust and debris blowout at north end (pylon pier), March 15, about 1:46 p.m.

[10] The NTSB also examined two other video recordings, details of which are available in the NTSB public docket for this investigation (HWY18MH009): (a) An original video from the owner of a cell phone who recorded the bridge collapse from the Miami-Dade County camera located at the southeast corner of the SW 8th Street–SW 109th Avenue intersection. The original video from the county camera was not available because the rewind feature allows playback for only 30 minutes, after which the video is automatically deleted. The cell phone video was taken during the time of available playback. (b) An original video from FIU that captured the bridge collapse from three cameras (two located in high parking garages and one located in a high dormitory). The video is a compilation of 1-minute time lapse photographs, not a continuous feed of live video over a 24-hour period.

[11] The 13:46:43:881 time stamp designates 1:46 p.m., 43 seconds, and 881 milliseconds.

Figure 10. Still image (time stamp 13:46:44:046) from in-vehicle mounted video camera on pickup truck traveling east on SW 8th Street, showing full-width canopy fracture and deck fracture areas at north end (pylon pier), March 15, about 1:46 p.m.

Figure 11. Still image (time stamp 13:46:44:310) from in-vehicle mounted video camera on pickup truck traveling east on SW 8th Street, showing main span completely collapsed, March 15, about 1:46 p.m.

The video recording of the entire event progressed in less than 2 seconds. The downward hinging motion of the structure increases with the fracture north of member 10, while the structure south of member 10 appears to remain rigid and rotates downward. Once the main span deck slides off the pylon pier, the canopy has collapsed onto the deck, and the deck has fallen onto the westbound lanes of SW 8th Street. The south end of the main span is visible rotating on the south pier anchorage. Truss members 12 and 11 collapse, while members 10 through 5 are pulverized between the canopy and the deck, as the deck progressively collapses onto the vehicles and roadway from the pylon pier toward the south pier. The main span canopy and the deck collapse onto SW 8th Street across the left, left center, and right center lanes—while the canopy; truss members 4, 3, 2, and 1; and the deck remain visible and still attached to the south pier (refer to figure 11, time stamp 13:46:44:310).

During the collapse sequence, which initiates at the north end, the entire span falls and breaks into multiple sections in the north-to-south direction, as depicted in figure 12. The south and pylon piers appear to have remained vertical and stationary postcollapse, with minimal damage.

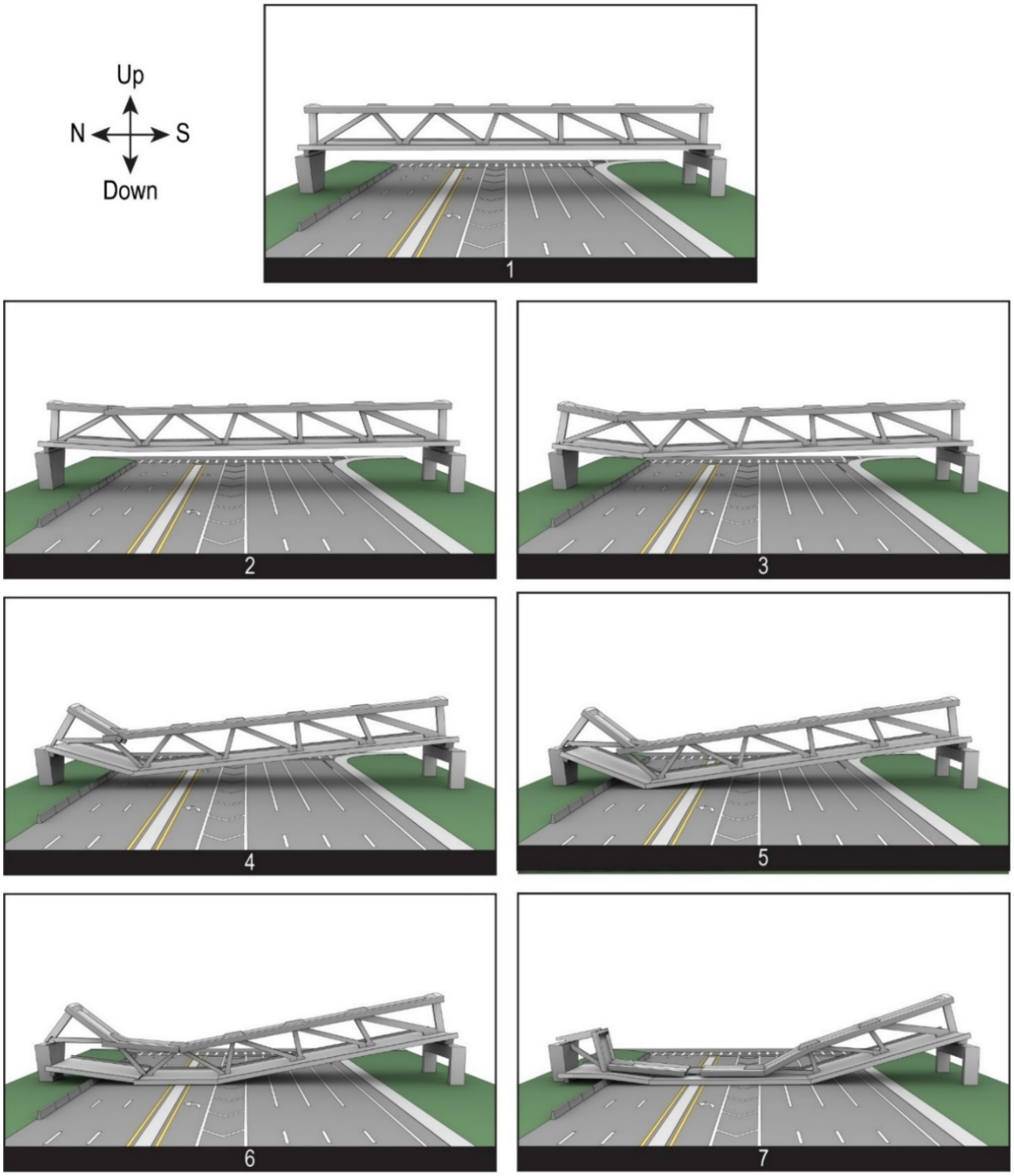

Figure 12. Collapse sequence diagram, facing east, depicting bridge's precollapse condition in phase 1 through postcollapse position (onto SW 8th Street) in phase 7.

NTSB investigators directed initial recovery of the collapsed span, in situ inspections and documentation, and removal of the remaining structural components. Individual pieces were documented and moved to the FDOT yard and subjected to NTSB examination, as discussed in section 1.10.

1.5 Bridge Construction Contracts

1.5.1 FIU University City Prosperity Project

1.5.1.1 FIU–FDOT Responsibilities. On September 5, 2013, the DOT notified FIU that it was selected to receive an $11.4 million TIGER grant, including funds to construct "a new pedestrian bridge over a busy arterial road."[12] The FHWA is charged with implementing the Federal-Aid Highway Program in cooperation with the states and local governments.[13] The grant agreement was executed on June 5, 2014, between the FHWA, the FIU Board of Trustees, and FDOT—which was responsible for local agency compliance with all applicable federal laws and requirements. Per the terms and conditions of the TIGER grant, FDOT was to:

- Act as a limited agent for FIU to assist in the receipt and disbursement of the grant monies.
- Perform other administrative and oversight duties with respect to the grant and the project (as FIU and FDOT shall agree among themselves).
- Comply with all applicable federal laws, regulations, executive orders, policies, guidelines, and requirements as they relate to the duties it assumed under the agreement.[14]

The FDOT LAP agreement establishes consistent and uniform practices for authorizing other local agencies to use federal-aid funds provided through FDOT for project planning, project

[12] The TIGER program funds supported investments in roads, bridges, transit, rail, ports, or intermodal transportation. The DOT Better Utilizing Investments to Leverage Development (BUILD) discretionary grants program replaced TIGER in fiscal year 2018. FIU also received a $1 million TAP grant. TAP invested in smaller scale transportation projects, such as pedestrian and bicycle facilities. The Fixing America's Surface Transportation (FAST) Act replaced TAP with a set-aside of funds under the Surface Transportation Block Grant Program. For financing and budget details, see the bridge factors group chairman report in the NTSB public docket for this investigation (HWY18MH009).

[13] The Federal-Aid Highway Program provides financial assistance for construction, maintenance, and operation of the nation's 3.9 million-mile highway network, including the Interstate Highway System, primary highways, and secondary local roads. See the FHWA webpage on federal aid essentials for local public agencies, accessed September 23, 2019.

[14] Per the Consolidated and Further Continuing Appropriations Act: The Florida International University Board of Trustees, University City Prosperity Project, FHWA FY2013 TIGER grant no. 12.

development, design, right-of-way relocation and acquisition, and construction (FDOT 2013).[15] Per 23 CFR Chapter I, FDOT acts as the supervising agency and ensures that LAP projects receive adequate supervision and inspection and are developed according to approved plans and specifications.[16] FDOT, under this assignation of responsibilities, may permit local agencies to carry out its assumed responsibilities on locally administered projects. (See section 1.12 for more details on LAP agreements.)

1.5.1.2 Contracted Parties for Design, Construction, and Inspection.[17] As part of the FIU UCPP, T.Y. Lin International prepared a report on criteria for the analysis and design of the pedestrian bridge (T.Y. Lin 2014). Also, in June 2014, FIU issued a request for proposals to solicit qualifications, competitive bids, and technical proposals from a design–build firm.[18] The anticipated responsibilities of each party are outlined below. The pedestrian bridge was to be completed by early 2019.

T.Y. Lin. The design criteria were intended to provide general guidance for the architectural and structural elements of the bridge. The report stated that "selection criteria will be weighed heavily toward an innovative design that represents the intentions of this project, creating a distinctive landmark for the region." (T.Y. Lin 2014; see appendix B for the list of required specifications and references for the bridge work.) On the design of the bridge, the report stated that a thorough study had been conducted and that "in the end it was determined that a truss or a hybrid of sorts was the best typology."[19] The report further stated that "Redundancy factors shall be determined in accordance with FDOT's SDG Section 2.10."

Under ABC—for bridges constructed in a staging area and launched, slid, or otherwise transported into final location—the specified design criteria were as follows:[20]

> B. Temporary Support Structures: Provide design of all temporary structures meeting AASHTO Design Guide for Bridge Temporary Works. Show dimensions,

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[15] The term "local agency" includes, but is not limited to, a county, an incorporated municipality, a metropolitan planning organization, an expressway or transportation authority, a special road or bridge district, or a regional governmental unit. Certification cannot be granted to a private corporation or nonprofit organization. According to the FDOT LAP manual, 23 *United States Code* (USC) 106(g) states that the keys to compliance and reducing state and federal risk factors related to compliance are local agency staff experience and cooperation and state-sponsored training of local agency staff.

[16] FDOT is subject to review, monitoring, and oversight by the FHWA.

[17] See appendix C for an organizational chart of the contracted parties discussed in this report.

[18] Design–build relies on a "single point of responsibility contract" and is used to minimize risk for the project owner and to reduce the delivery schedule by overlapping the design and construction phases. Traditional construction projects appoint a designer separate from a builder.

[19] T.Y. Lin also stated that "one of the major parameters governing the selection of a truss typology was the ability to seamlessly integrate the required 8-foot missile fence over the roadway into the structure and skin of the bridge. The missile fence should not stick out as its own discrete component but should contribute as a feature that is woven into the holistic design and as such function for the sake of providing shade, safety, reinforcement of the geometry, and so forth." [Note: A "missile fence" is designed to prevent pedestrians on the bridge from throwing projectiles into the path of vehicles traveling below.]

[20] Note that items A, D, and G from this quoted material are not pertinent to this discussion, so they were not included in this selection.

alignments, and elevations of temporary supports relative to those of the permanent supports;

C. Permanent Superstructure: Design permanent superstructure including the maximum anticipated and maximum allowed deflections of the ends relative to mid-span as a result of any temporary support conditions necessitated by the chosen method of moving the bridge;

E. Bridge Movement Plan: Detail the sequence and procedures for attaching the Bridge Movement System to the superstructure and actively engaging the load. Show inspection access points under or around the superstructure at lift locations and attachment points. Provide anticipated height change limitations or stroke limits of the jacking systems for the bridge movement systems. Include all scheduling and Traffic Control Plans.

F. Monitoring Plan: Provide a plan for monitoring structure deflections during the move. Include details of all instrumentation, locations of benchmarks, and locations of reference points in the BSA and at the final bridge location. Include details for measuring the deflections of the structure immediately after lifting and immediately before settling the structure.

FIU Request for Proposals. The request for proposals specified the following:

G. Structure Plans

1. Bridge Design Analysis: The Engineer of Record for bridges shall analyze the effects of the construction-related loads on the permanent structure. These effects include but are not limited to construction equipment loads, change in segment length, change in construction sequence, etc. The Engineer of Record shall review all specialty engineer submittals (camber curves, falsework systems, etc.) to ensure compliance with the contract plan requirements and intent.

2. Criteria: (a) All plans and designs are to be prepared in accordance with *AASHTO LRFD Bridge Design Specifications*, Department Standard Specifications, Structures Manual, Plans Preparation Manual, Department Standard Drawings, Supplemental Specifications, Special Provisions, and directions from the State Structures Design Engineer, Temporary Design Bulletins, Structures Design Office and/or District Structures Design Engineer.

H. Specifications

FDOT Specifications may not be modified or revised. The Design-Build Firm shall also include all Technical Special provisions, which will apply to the work in the proposal. Technical Special Provisions shall be written only for items not addressed by Department Specifications and shall not be used as a means of changing Department Specifications.

The request for proposals stated that the design–build firm was responsible for—

Detailed plan checking as outlined in the *Plans Preparation Manual* (*PPM*); as described in the RFP [request for proposals]; and the Design and Construction

> criteria package. This includes a checklist of the items listed in the PPM for each completed phase submittal. Bridge submittals may be broken into architecture, foundation, substructure, superstructure, approach spans, and main channel spans.
>
> Prior to submittal to the OWNER (FIU), 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 certification 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 plan 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.

MCM and Bolton, Perez and Associates Consulting Engineers. On January 14, 2016, FIU entered into a design–build contract with MCM to perform all work and furnish all materials, equipment, supplies, and labor necessary to construct the pedestrian bridge. On September 23, 2016, FIU also entered into a contract with Bolton, Perez to administer, monitor, and inspect the pedestrian bridge "such that the project was constructed in reasonable conformity with the plans, specifications, and special provisions of the construction contract." As the construction engineering and inspection (CEI) contractor, Bolton, Perez was required to (1) observe the MCM work to determine its progress and quality; (2) identify and report significant discrepancies to FIU; and (3) direct MCM to correct such observed discrepancies.[21]

FIGG Bridge Engineers and Louis Berger. On April 28, 2016, MCM entered into a design–builder and design–consultant contract with FIGG to provide professional bridge design and engineering services and to serve as the engineer of record (EOR). FIGG then contracted with Louis Berger on September 16, 2016, to conduct the project-required independent peer review. (See section 1.9.) As lead partner for the bridge design team, FIGG provided to MCM the final design, construction drawings, and specifications necessary to construct a complete and fully operational project (in accordance with FIU requirements and contract reference documents).

Structural Technologies, The Corradino Group, and Barnhart Crane and Rigging. MCM contracted with Structural Technologies to conduct post-tensioning of the pedestrian bridge; and Bolton, Perez contracted with The Corradino Group to inspect the post-tensioning work. MCM

[21] On November 10, 2015, FDOT recommended to the FHWA that the design-build contract for the FIU UCPP be awarded to MCM. On August 23, 2016, FDOT recommended that the CEI contract be awarded to Bolton, Perez. The FHWA concurred on the contract awards to MCM on November 16, 2015, and to Bolton, Perez on September 12, 2016.

also contracted with Barnhart Crane and Rigging to move the main span from the adjacent ABC casting yard onto the pylon pier abutment and south pier.[22]

1.5.2 Scope of Work

The scope of work agreement stated that FIU would provide the conceptual design drawings for the pedestrian bridge.[23] FIGG was responsible for managing the design team as the lead partner and for acting as the single point of contact with MCM, the design-builder. FIGG was responsible for completing the final structural design and preparing contract documents, including analysis and design of the bridge superstructure, substructure, and foundations related to the final construction contract documents. According to the contract, the bridge was to be designed to meet the following criteria:

- *AASHTO LRFD Bridge Design Specifications*, 7th edition with 2015 interims (AASHTO 2015).
- FDOT *Structures Manual, Structures Design Guidelines,* January 2015 (FDOT 2015b).
- *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges*, 2nd edition, 2009 (AASHTO 2009).

1.6 Bridge Construction

1.6.1 Construction Stages

ABC broadly refers to a method of bridge construction that focuses on minimizing the disruption of traffic when building new bridges or replacing or rehabilitating existing bridges. The ABC process uses planning, design, materials, and methods to reduce onsite construction time. The ABC process for the pedestrian bridge included constructing the main bridge span in an adjacent staging area (the casting yard) to avoid obstructing the SW 8th Street trafficway.

The pedestrian bridge was to be constructed in eight stages.[24] For the purposes of this report, to represent the different stages (conditions) of the main span structure's support during ABC construction, the eight stages are condensed into the following four stages:

[22] Considerations for this type of bridge building must include construction location; traffic volumes; bridge size, shape, and composition; and environmental conditions. ABC candidates typically have high traffic volumes; a construction site with a sufficient area for prebuild of a bridge span; a bridge span capable of being supported in a condition that is different from the support to be provided in the permanent location; and access to an efficient detour route for vehicle, pedestrian, and bicycle traffic.

[23] Design drawings were to include the landing areas and the rail and elevator structures. FIU would also coordinate general civil design items.

[24] Stage 1, substructure casting; stage 2, superstructure precasting; stage 3, erection of main span; stage 4, casting of back span; stage 5, continuity tendons and casting of upper pylon; stage 6, installation of pipe support system; stage 7, installation of bridge components; and stage 8, installation of landings. See appendix D for the detailed steps in construction stages 1–8.

- Stage 1: Construction of the bridge substructure elements and the fabrication of the main span truss members in the casting yard.
- Stage 2: Transport of the main span from the casting yard using a self-propelled modular transporter (SPMT) and its placement in a "simply supported" condition on the south pier and pylon pier, then detensioning truss members 2 and 11.[25]
- Stage 3: Construction of the back span, supported on the pylon pier and north pier.
- Stage 4: Connection of the main span, back span, and pylon pier; construction of the upper pylon; and installation of steel pipe stays (see figure 13 for representation of stages 2–4).

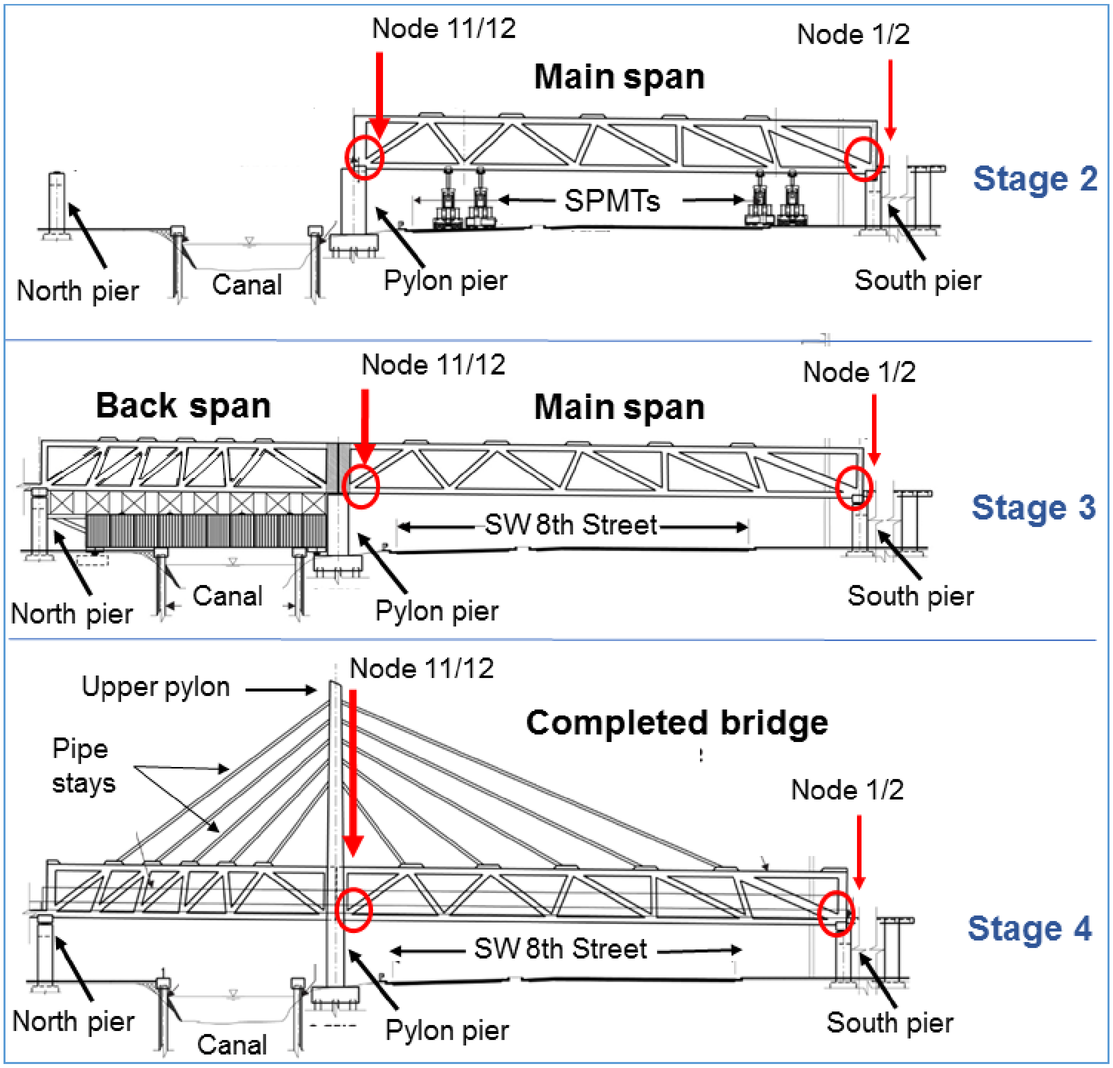

Figure 13. Construction stages 2–4, east view. (Source: FIGG, annotated by NTSB)

[25] A simply supported structure has a pinned support (or a connection type that will allow for rotation, but not horizontal or vertical movement) on one end and a roller, or similar, support (that will allow both rotation and horizontal movement) at the other end.

1.6.2 Concrete Casting of Main Span (Stage 1)

1.6.2.1 Process Descriptions: Falsework, Cold Joint, Interface Shear. The pedestrian bridge superstructure casting sequence included three distinct pours of concrete (concrete casting phases): with the bridge deck cast first, the truss members cast second, and the structure canopy cast last.[26] Concrete was mixed and placed in a semiliquid state into preconstructed formwork, which was designed to hold the concrete in a desired shape while supporting the material's self-weight. Each concrete pour was to be completed sequentially, with significant time allotted between pours to allow for development of the concrete's mechanical properties.[27] Falsework was used to support the formwork and the poured structure.

Sections of formwork are filled to the top with concrete, which must harden prior to the installation of additional formwork and the placement (pour) of subsequent semiliquid batches of concrete. Subsequent concrete castings rest on the earlier, hardened castings. The surface between the two castings is referred to as a "cold joint."[28] (See figure 14.)

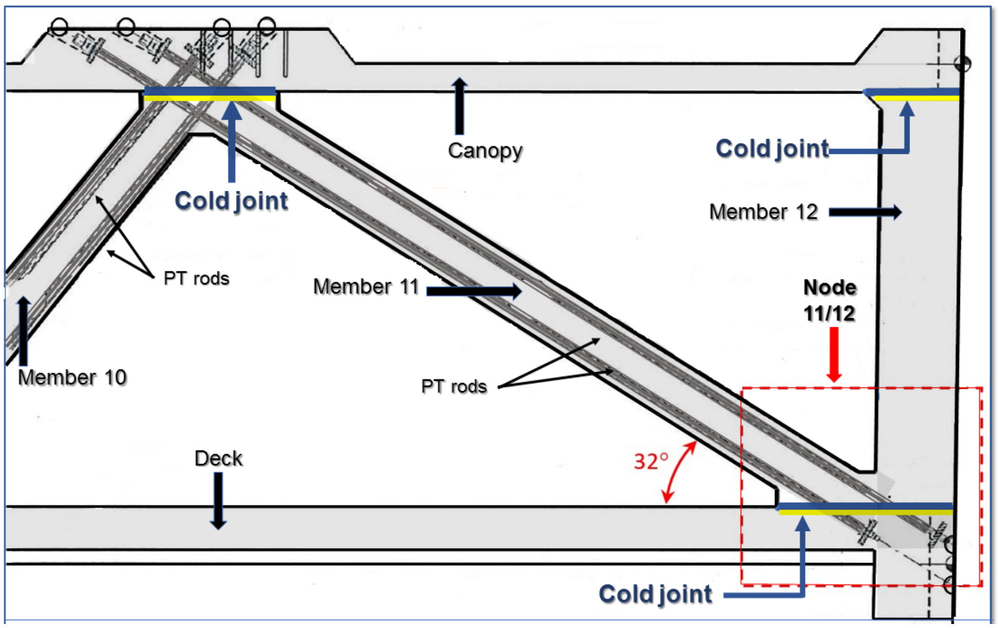

Figure 14. Cold joints on pedestrian bridge main span north end, at intersection of members 10, 11, and 12 at canopy and at node 11/12 connection to deck at pylon pier. [Note: View is looking west]

[26] The entire deck—the first pour—was cast and allowed to harden. Truss members 1 through 12 were then cast atop the deck and allowed to harden. For the third pour, the canopy concrete was cast atop the truss members. The truss member 11/12 nodal region includes portions of the first two concrete pours.

[27] Formwork is removed after the concrete undergoes a series of chemical reactions and hardens, and is capable of carrying the necessary loads. The placement of concrete in a construction project progresses as multiple discrete batches are deposited into the formwork. Where successive concrete batches meet, they are agitated (vibrated) to consolidate the material and merge concretes from successive semiliquid batches.

[28] A cold joint is a discontinuity where one layer of concrete reaches final set before subsequent concrete is placed (American Concrete Institute [ACI] 2018).

The bridge design plans—with phased concrete placement—resulted in cold joints between concrete pours at each end of each truss member: one located at the bottom of the truss member (the bridge deck to truss member interface) and the other at the top of the truss member (the truss member to canopy interface). Both the upper and lower surfaces of each truss member were designed to transfer forces between the truss members and the canopy or bridge deck, respectively, by transferring shear forces across the concrete-to-concrete interface surfaces, commonly referred to as interfaces.[29] The horizontal plane where truss members 11 and 12 (nodal region) connected to the deck was a cold joint.

1.6.2.2 Design and Concrete Placement Requirements for Cold Joints. Concrete construction commonly requires that cold joint regions receive special design considerations and concrete placement techniques to enhance their performance. The FDOT (2015a) *Standard Specifications for Road and Bridge Construction,* under section 400-9.3, Construction Joints–Preparation of Surfaces, calls for the following:

> 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.

The resistance provided along a concrete interface is dependent on its characteristic, such as a monolithic concrete interface, the interface of a cold joint with substrate concrete that may have been intentionally roughened, or a similar cold joint interface that is unroughened. AASHTO (2015, pp. 5–86) describes intentionally roughened concrete as a "clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 inch." Concrete roughening is commonly performed after the concrete has been placed in the formwork and consolidated but before it hardens.[30]

The construction process for the pedestrian bridge included multiple concrete placements, with cold joints at specific locations. In particular, one cold joint occurred on a horizontal plane at the intersection of the deck and members 11 and 12. The concrete for the deck was cast first and

[29] (a) In a cold joint region, the mechanical properties of a structural concrete element differ locally—unlike in a monolithic concrete structure, which is formed from a single concrete pour. Because the concrete castings harden prior to the placement of a secondary concrete pour, chemical and mechanical bonds form at the interface between the two castings (that is, at the cold joint). (b) Compressive force is related to both the amount of reinforcing steel crossing the interface surface and the amount of permanent loading whose line of action is perpendicular to the interface. Occasionally, supplemental compressive forces supplied through other means—such as post-tensioning—might be included.

[30] Consolidating concrete causes it to spread into place, fill formwork, and encapsulate embedded objects, such as steel reinforcement bars or drain pipes.

cured—with members 11 and 12 cast thereafter.[31] Upon postaccident examination, the hardened surface of the first concrete placement was found not to have an intentionally roughened surface, and a portion of the failure surface under member 11 was found to coincide with this cold joint. For pylon diaphragm connection points, the FIGG plans did specify a 0.25-inch amplitude for surface roughening. However, the FIGG plans did not specify this surface roughening at the cold joints between the truss members and the deck or canopy.[32]

1.6.2.3 Construction Stage 1 Activities. As indicated earlier, stage 1 involved the following activities:

- Casting of the superstructure concrete.
- Curing of all placed concrete.
- Application of PT tendons in the deck and canopy, and PT rods in the truss members—including the temporary post-tensioning of PT rods within truss members 2 and 11 in the casting yard.

The main span was cast above ground level and supported by temporary falsework—which supported the weight of the bridge along its entire length. (See figure 15.) Beginning on February 24, as the falsework was sequentially removed from the middle of the span toward the ends of the span, the load was transferred through the main span structural load-carrying elements to the temporary end supports, known as megashores. Once the falsework was completely removed, the bridge was entirely self-supporting.

[31] (a) Curing is a process that begins immediately after concrete is placed and finished; it involves maintaining moisture and temperature conditions throughout the concrete for an extended period of time. Properly cured concrete will have an adequate amount of moisture for continued hydration and strength development, and will be stable against volume changes and resistant to freezing and thawing. (b) As noted earlier, the casting process for the superstructure included three distinct concrete pours: for the deck, for the truss members, and for the canopy. The truss member 11/12 nodal region included portions of the first two concrete pours.

[32] FIGG specifications for a 0.25-inch amplitude for surface roughening were documented in the release for construction plans (sheets B-24B and B-25). FIGG did not include this specification for the truss members and the deck and canopy shown on sheets B-37, B-38, and B-41.

Figure 15. Temporary falsework supporting the bridge span in the casting yard on February 24, 2018, being sequentially removed from the middle of the span toward the ends of the span. (Source: MCM)

1.6.3 Concrete-Embedded Elements Within Truss Member 11/12 Nodal Region and Deck

The concrete fabrication of the bridge superstructure (detailed in construction plans) included the placement of steel reinforcement bars (rebar) of varying diameters, PT rods, and various hollow pipes within the concrete.[33] For member 11, figure 16 shows the steel reinforcement within the truss nodal region. Also shown in the figure are two 4.5-inch-diameter vertical pipes (referred to as pipe sleeves) at the vertical column east face of member 12 (the west face also contained two identical pipe sleeves).[34] A horizontal 8.625-inch drain pipe ran in a north–south direction down the deck centerline, through the nodal region and the diaphragm immediately under truss member 12, and out the north face.

[33] Steel reinforcement bars, termed "rebar," are often used in concrete structures for added strength and stability.

[34] The pipe sleeves provided a conduit through which vertical PT rods and size 11 rebars could pass for use in future construction stages.

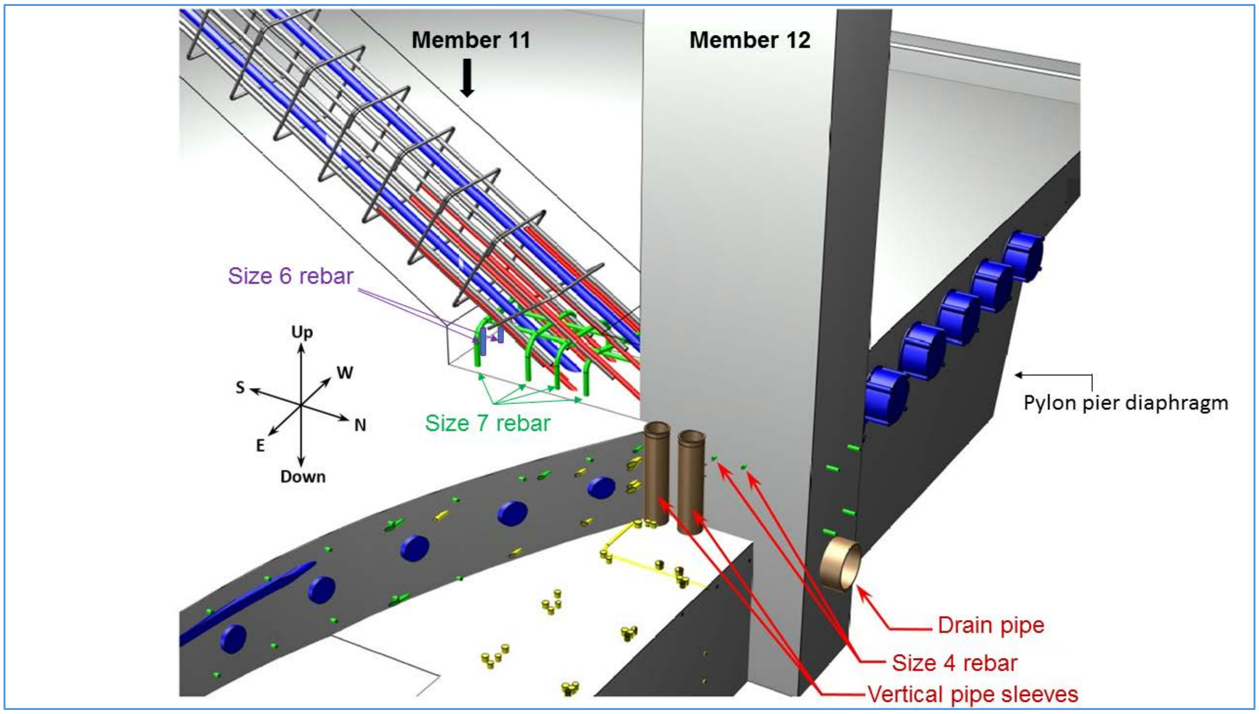

Figure 16. Main span, north end, showing steel rebars in member 11, vertical pipe sleeves, end of drain pipe location, and size 4 rebar at member 12. (Source: FHWA 2019)

The member 11 and 12 reinforcing steel embedded in the concrete deck is shown in red and green in figures 16 and 17. Additional steel reinforcement not embedded in the deck and reinforcing truss members 11 and 12 is shown in gray. The PT rods in member 11 are shown in blue. The confinement reinforcements for members 11 and 12 are shown as gray rectangular-looped bars; they consisted of size 4 rebar hoops spaced at 12 inches along the length of the member. In member 12, the longitudinal reinforcement consisted of three size 11 rebars in the member south face and three size 7 rebars each in the member east, north, and west truss member faces.[35]

[35] The lower ends of these vertical reinforcements were anchored into the bottom of the north diaphragm (nodal region) and extended above the deck level. These vertical steel bars were lap-spliced with matching vertical bars beginning just above the cold joint at the base of truss member 12—except for the size 11 rebar on the center of the south face and the size 7 rebar on the center of the north face, which conflicted with the placement of the horizontal drain pipe. At these two locations, FIGG placed the lower bar in the lap splice to hook within the volume of concrete immediately above the drain pipe.

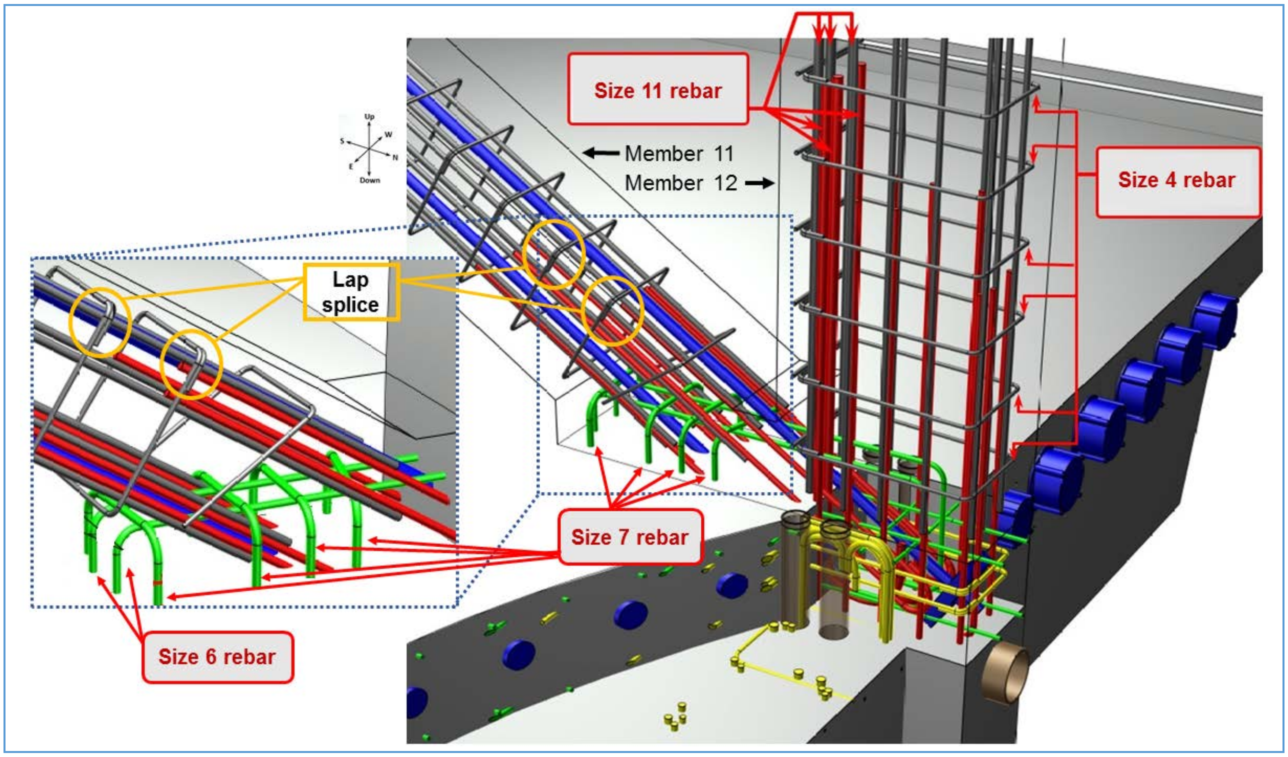

Figure 17. Main span, north end, showing rebar detailing in member 11, member 12, and node 11/12. Inset shows another view of rebar in node 11/12 and detail of lap splice from member 11. (Source: FHWA 2019)

Figure 18 is a cross section of the reinforcement within member 12, showing the size 11 rebars (1.4 inch diameter) for longitudinal reinforcement as larger circles on the left (south face). The remaining circles represent the size 7 rebars (0.875-inch diameter) for longitudinal reinforcement around the perimeter. The confinement rectangular hoop in member 12 was composed of size 4 rebars (0.5 inch diameter).

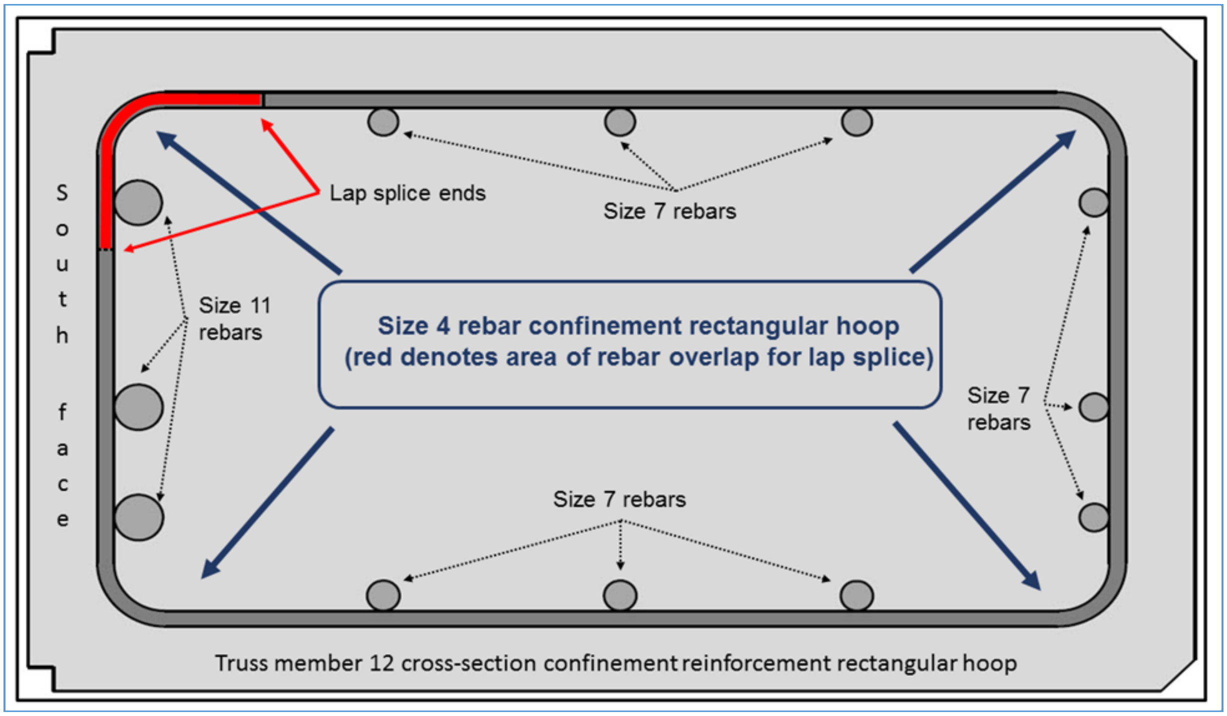

Figure 18. Cross section of truss vertical member 12 concrete-embedded rebar confinement hoop and rebars with lap splice.

As described earlier, permanent internal PT tendons ran the full span length in the longitudinal direction in the main span concrete deck (which was 31 feet 8 inches wide) and within the 16-foot-wide canopy. The deck also included permanent internal transverse PT tendons. The main span truss members 3, 5, 6, 7, 8, and 10 were permanently post-tensioned via PT rods, while members 1, 4, 9, and 12 were not post-tensioned. All longitudinal and transverse PT tendons in the deck and canopy, along with members 2 and 11 (to address temporary construction conditions), were stressed before the falsework was removed from under the deck.

PT rods are large-diameter threaded rods secured with large nuts and anchor plates to lock their ends in place so they can be tensioned and/or detensioned as needed. Specialized hydraulic equipment was used to provide the force necessary to apply stress (tensioning force) to the PT rods, as shown in figure 19. As described earlier, blisters were located on the top of the canopy to accommodate the PT rod anchorages.

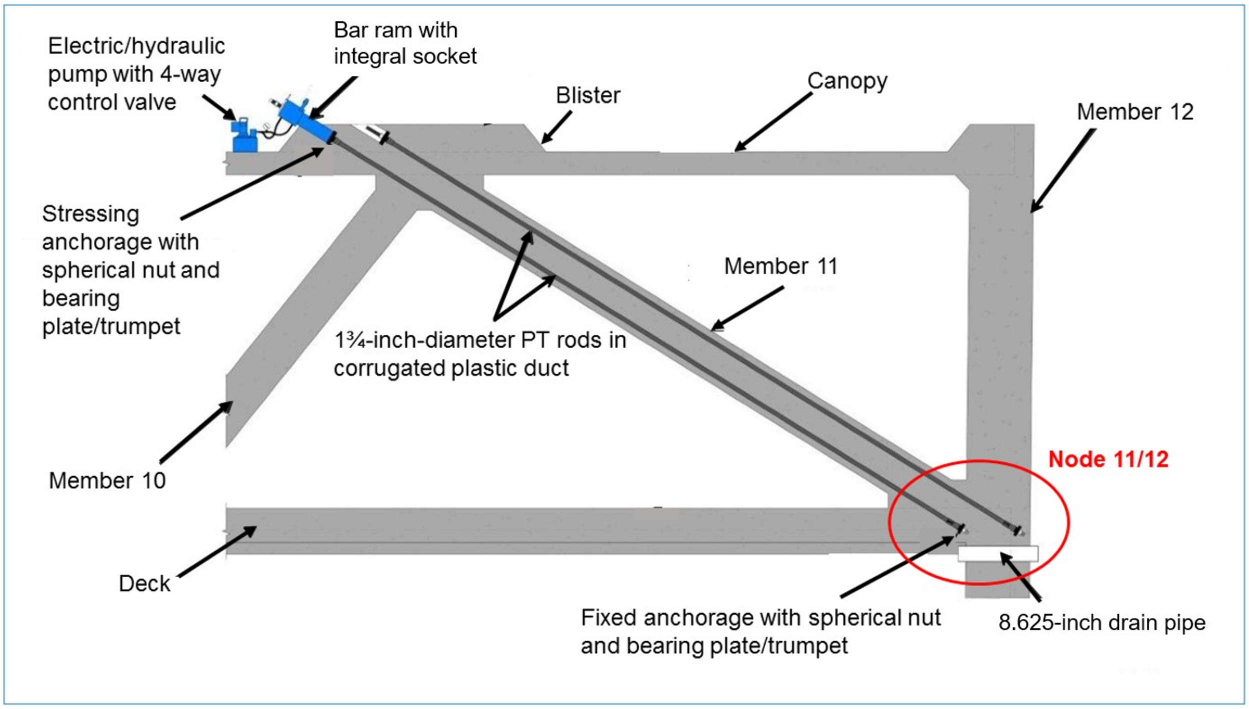

Figure 19. Main span, north end, showing post-tensioning specialized equipment in relation to location of PT rods in member 11. (Source: Structural Technologies, annotated by NTSB)

Once the span was permanently in place on the south and pylon piers, the tension and compression members were designed to support the main span at each end.[36] When asked by NTSB investigators why the self-weight plus post-tensioning creates a situation in which all members are in compression, FIGG replied as follows:

> The truss is designed as a prestressed concrete element. As such, prestressing steel is used in members subject to tension forces to limit the net tension in the member under full design loads. Loads that the truss was designed for that were not applied at the time include pedestrian live load, wind loads, and thermal loads, which would result in tension in certain members. The compression provided by prestressing counteracts the tension generated from the loads.

In its *Post-Tensioning Tendon Installation and Grouting Manual*, the FHWA (2013) mentions in reference to the performance of concrete structures that—

> . . . the tensile strength of concrete is only about 10% of its compressive strength. . . . plain concrete members are likely to crack when loaded. Reinforcing steel can be embedded in the concrete members to accept tensile stresses which plain

[36] Because support placement of the self-propelled modular transporters caused the concrete members at the ends to be in tension, FIGG stressed the PT rods to temporarily compress the members and counteract this effect. In the casting yard, the falsework supported the forms and provided support to the ends in a similar condition as when placed on the permanent piers.

concrete cannot resist. The resulting reinforced concrete members may crack, but they can effectively carry the design loads.[37]

1.6.4 Transport of Main Span to Permanent Location (Early Stage 2)

On March 10, using two SPMTs, Barnhart Crane and Rigging moved the prefabricated main bridge span, weighing 950 tons, from the casting yard adjacent to eastbound SW 8th Street to its permanent location on the piers.[38]

FIGG structurally evaluated the main span design to determine the SPMT locations for transport of the main span. The SPMTs were positioned under the bridge deck at the member 3/4 and 9/10 nodal regions. Diagonal truss members 2 and 11 were determined to require temporary post-tensioning to prepare for placement onto the SPMTs. Figure 20 illustrates SPMT support locations with self-weight forces only (gravity loads), as well as the self-weight plus post-tensioning for the main span in its simple span position. (SPMT support locations are also shown on figure 20.) Figure 21 illustrates the self-weight forces only (gravity loads), as well as the self-weight plus the forces from the PT rods.

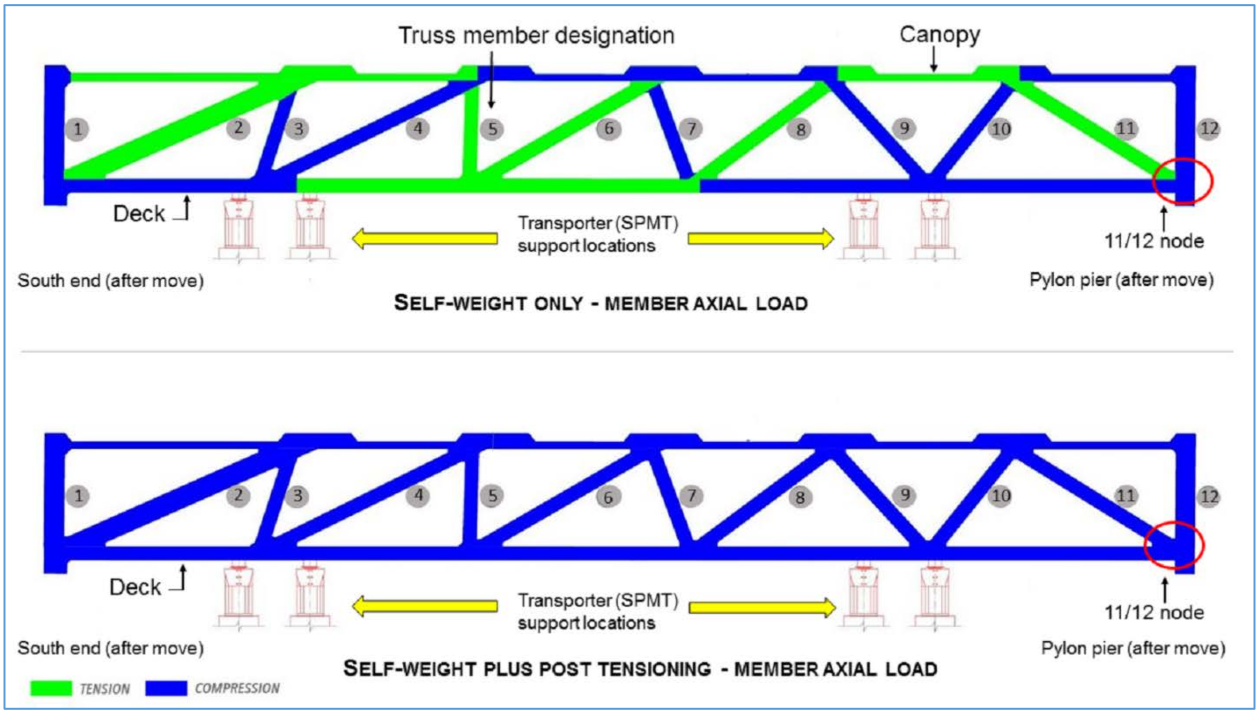

Figure 20. Main span tension and compression members for diagonal truss members 2 and 11, which supported each end during SPMT move from casting yard to south and pylon piers. Top illustration is shown without post-tensioning; bottom illustration is shown with actual loads. (Source: FIGG, annotated by NTSB)

[37] Reinforcing is selected assuming that the tensile zone of the concrete carries no load and that tensile stresses are resisted only by tensile forces in the reinforcing bars. Cracks in reinforced concrete are normally very small and well distributed.

[38] The main span included the deck, diagonal and vertical truss members, and the canopy.

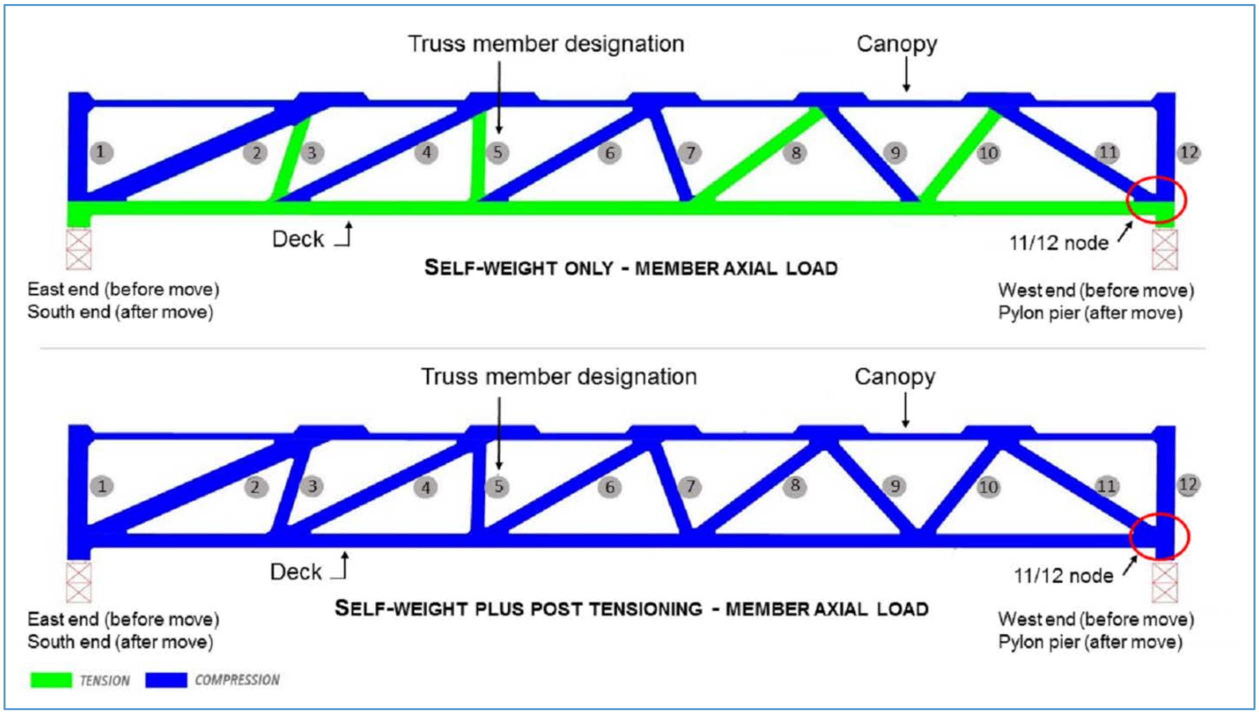

Figure 21. Self-weight axial forces only (top) for diagonal truss members 2 and 11, which are combined with post-tensioning forces concurrently acting on main span to produce net axial forces (bottom) at each end before transport and on permanent pier following transport. Top illustration is shown without post-tensioning; bottom illustration is shown with actual loads. (Source: FIGG, annotated by NTSB)

Traffic on SW 8th Street was detoured during the installation period, and the entire roadway was closed to facilitate movement of the structure.

Each SPMT included four shoring stands located symmetrically about the longitudinal and transverse centerlines of the main span truss. Each shoring stand supported a hydraulic jack assembly, also positioned symmetrically about the centerline of the main span.[39] Two truss assemblies were installed at both ends of the SPMTs to connect them to ensure that they maintained the proper spacing throughout the travel path. The bridge span was raised and lowered by simultaneously extending or retracting eight hydraulic jack assemblies (four pairs located at discrete locations along the length of the span). One operator controlled the steering and forward/reverse functions for the entire system, as well as the leveling of the hydraulic suspension on the north SPMT, while the second operator controlled only hydraulic suspension leveling for the south SPMT (see figure 22).

[39] The four pairs of hydraulic jack assemblies each supported a beam and were positioned transversely to the main span. Two wedge-shaped hardwood mats (matching the angle of the tapered bottom of the main span flange) were installed on top of each beam, symmetrically about the longitudinal main span centerline. Loads to each jacking assembly were equalized through valves in the hydraulic control system. Steel mats were placed in the gravel staging area, as well as adjacent to the curb and median. The mats provided a solid surface for the tires to ensure adequate traction during transport and to smoothly transition over the curb and median. To resist acceleration/deceleration forces, securement chains connected the shoring system to the decks of the SPMTs.

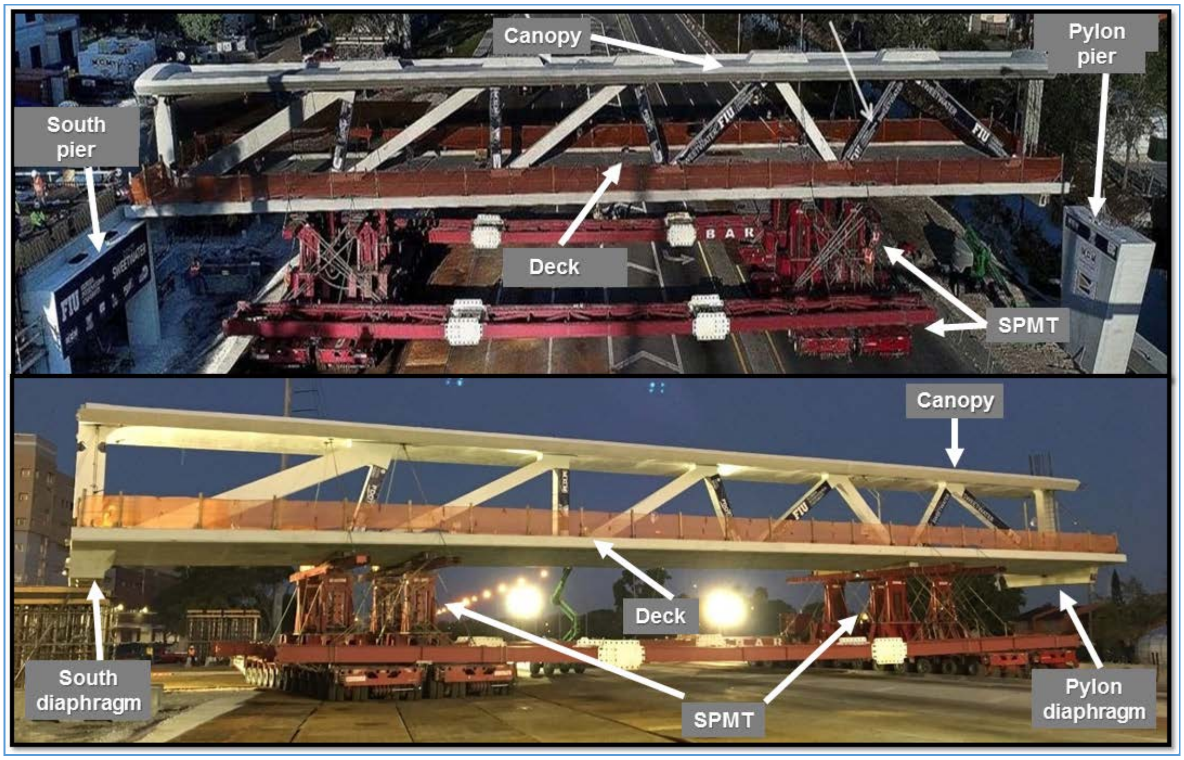

Figure 22. SPMT being moved overnight on March 10 (bottom) and main span being placed on south and pylon piers, facing west (top). Transport occurred over 8 hours, starting at 4:30 a.m., with placement of final permanent pier support concluding at 12:30 p.m. (Source: FIGG, annotated by NTSB)

Before the transport of the main span—because it was sensitive to torsional forces (that is, twisting along the longitudinal axis)—FIGG specified the limiting criterion for allowable twist angle tolerance to +/-0.5 degree.[40] This twist criterion applied to transport and final pier placement over the length of the 95-foot span located between the SPMTs. A system was developed to monitor transverse rotation at three cross sections: at the centers of the two lift points (SPMTs) and at midspan. Twist was computed as the difference in rotation angle between the two lift point cross sections. These data were calculated and displayed in real time by the monitoring system so that corrective action could be taken if the specified twist tolerance was approached. (Strain measurements were recorded for the duration of transport of the bridge span, but these values were not examined in real time because there were no defined limits or stop criteria.)

Beginning at 4:30 a.m. on March 10, the main span was moved via the SPMTs and placed on the permanent pylon and south pier supports. This operation was completed by 12:30 p.m. (refer to figure 22). During transport of the main span, the +/-0.5 degree tolerance—a function of the rate at which twist was occurring, the time to make an "all stop" decision, and the time to execute the command—was exceeded in two instances:

[40] FIGG had initially established an allowable tolerance of +/-0.17 degree for the twist angle. Barnhart Crane and Rigging informed FIGG and MCM that the transport system could not accommodate a twist angle of less than 0.5 degree. Subsequently, FIGG further analyzed the span and revised the allowable twist angle tolerance to +/- 0.5 degree.

- At the peak static twist value of approximately 0.65 degree.
- During the final alignment process. Just prior to the bridge being placed on the south and pylon piers (1 and 2, respectively), a peak twist angle of 0.75 degree occurred for about 4 minutes as the bridge came in contact with one of the bearing pads on the south pier.[41] During this time, the recorded strain changed by 200 microstrain (με) at the top of member 12 and by 30 με at the bottom of member 12. (For comparison, the change in strain in the same locations on member 12, according to Barnhart, was 200–800 με [at top] and 20–25 με [at bottom] during the lift and set evolutions, respectively.) The MCM interpretation of the change in strain in the same locations on member 12 was 500–1,000 με (at top) and 40–120 με (at bottom) during the lift and set evolutions, respectively. The strains in member 11 and node 11/12 were not measured.

Global deformation in the form of span deflection and flexural rotation was measured at the time of the lift and for the final placement. All global deformations—such as rotation, twist, and deflection—indicated that the condition of the span after the move was nearly identical to its initial state.

Once the main span was permanently placed on the south and pylon piers, and the SPMT supports were removed, the PT rods in diagonals 11 and 2 were detensioned as specified in the plans. SW 8th Street was reopened to traffic about 6:53 p.m.[42]

1.6.5 Permanent Placement of Main Span (End of Stage 2)

The original bridge plan shows the temporary PT rod layout in members 2 and 11. No specific procedures were issued for the detensioning operation on the permanent pier location.[43] Per the design plans, diagonal truss members 2 and 11 were prestressed with PT rods to 280 kips (1 kip = 1,000 pounds-force) to provide compressive force to counteract the tensile forces resulting from the SPMT vehicle support locations under the main span. On March 6, 2018, in an email from FIGG to MCM, FIGG specified that “The PT rods in members 2 and 11 are only required for the temporary support condition during the movement of the span. Therefore, the PT rods can be destressed after span 1 is supported on the permanent supports (pylon and end bent 1).”

The diaphragms at the end bent and pylon pier locations were located at the ends of the bridge deck to transfer the main span weight to the supporting pier and end bents. The diaphragm

[41] During the bridge alignment process, the procedure was to align the south end of the main span with the bearings and then set the north end. As the bridge was being aligned, the span came into contact with the southwest bearing, which induced twist because the bridge was not yet exactly oriented with the pier. As the bridge was lowered, the induction of a new support condition at the southwest bearing caused the twist value to quickly change. An “all stop” call was made. Barnhart immediately stopped movement and adjusted the rotation to bring the twist back within specifications. The correction was completed in about 10 minutes. At the end of this adjustment, the bridge was no longer in contact with the bearing pads. During both exceedances of the +/-0.5 degree tolerance, the north end of the bridge was floating and had not yet made contact with the pylon pier support.

[42] Onsite at the time of detensioning were Structural Technologies, which performed the PT rod detensioning with its equipment and operators; George’s Crane Service, which provided the crane and operator; MCM, which performed construction management; Bolton, Perez, which conducted CEI oversight; and The Corradino Group, which conducted post-tensioning inspection.

[43] FIGG bridge plan sheet B-38. Detensioning PT rods is a common post-tensioning operation.

at end bent 1 measured 4 feet high, 3.5 feet wide, and 20 feet long; and the diaphragm at the pylon pier measured 4 feet high, 2 feet wide, and 20 feet long. Four discrete shim stacks were used to temporarily support the main span on the pylon pier (two on the east side of the diaphragm and two on the west side).

Available in various thicknesses, shims—or shim stacks—are often used to assist in the placement and leveling of large precast structures. Unlike when the bridge was supported across the entire width of the diaphragm in the casting yard, this new bearing configuration would have changed the path of the forces within the structure and the location where these forces were transferred from the diaphragm to the pylon pier. These shims were to be encapsulated in grout later in the construction process to re-establish the full-length contact between the bottom of the diaphragm and the top of the pylon pier. However, no shim stacks were located directly beneath the truss line (centerline of bridge) when the SPMTs lowered the bridge to its final position.

On March 10, within hours of placement of the main span truss on the permanent support piers (as part of the end of step 2), construction crew members detensioned the PT rods within truss diagonal members 2 and 11. In late February, a crack had been observed in the truss member 11 and 12 nodal region; another was found in the member 1 and 2 nodal region (see section 1.7). Starting on March 13, two remedial measures were taken to address observed distress (cracking) in the member 11/12 nodal region above the pylon pier. These measures, which were not included in the FIGG design or the planned construction stages, included the following:

- Placing an additional shim between the underside of the deck diaphragm and the top of the pylon pier within the truss member 12 footprint on March 13 (see figure 23).
- Retensioning the PT rods in truss member 11 on March 15.[44]

[44] No note appears on the original FIGG design plans to restress the PT rods in member 11.

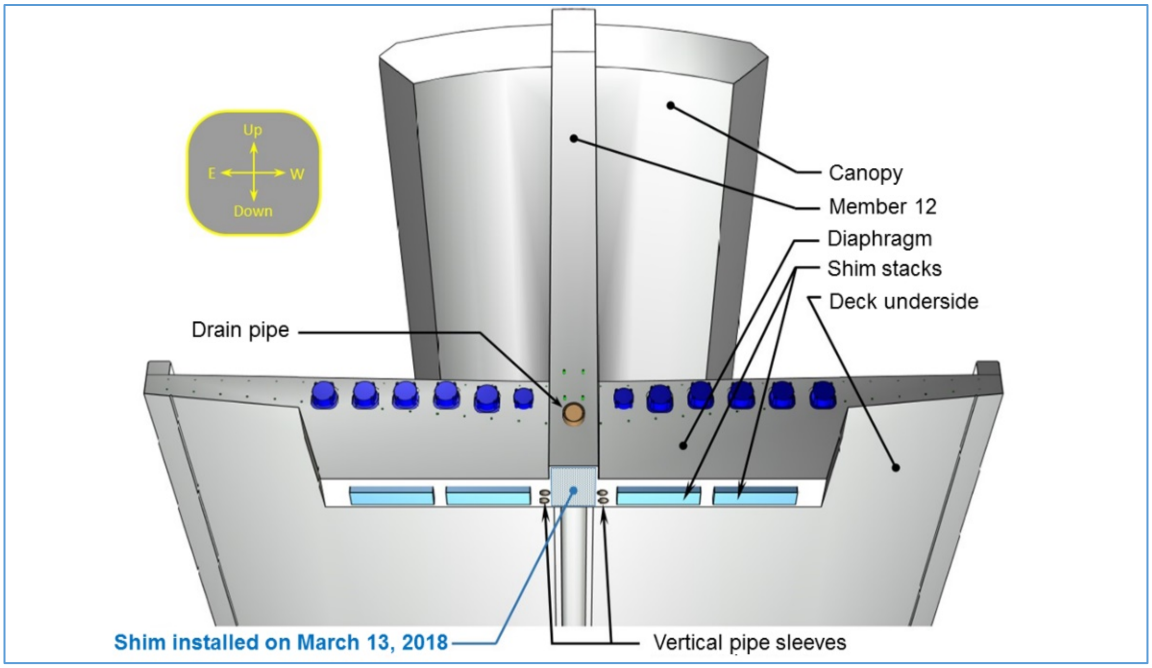

Figure 23. View of underside of post-SPMT support location of pylon diaphragm onto pylon pier, showing shim stacks on east and west side of diaphragm centerline. Post-SPMT support location is at underside of pylon pier deck. (Source: FHWA 2019)

The collapse occurred immediately after the retensioning of the member 11 PT rods. (See next section for additional information on post-tensioning work performed before the collapse.)

1.7 Precollapse Activities and Documentation of Concrete Cracking

Cracks in concrete occur for a variety of reasons (for example, restrained shrinkage, thermal effects, and structural loading). Crack widths range from sizes smaller than can be detected by unaided human vision (approximately 0.01 inch wide) up to large widths that indicate gross separation of the two opposing portions of the structure. For reference, in a reinforced concrete element, cracks with widths up to approximately 0.016 inch are often considered generally acceptable, with formal acceptance depending on the purpose of the structure and the location of the cracks.[45] Days before the collapse of the pedestrian bridge, extensive cracks, documented as more than 40 times larger in width than generally accepted cracks for a reinforced concrete structure, were observed in the member 11/12 nodal region.

In this case, the cracking of the reinforced concrete during construction began at least 3 weeks before the collapse of the main span. The severity and extent of the cracking progressed, with structural distress at some locations worsening and new locations of distress displayed as cracking became apparent. The distress was observed and documented by parties involved in the design and construction of the bridge, as discussed below.

[45] The *AASHTO LRFD* cites 0.017-inch-wide cracks for environmental exposure conditions, where they can be tolerated due to reduced concerns with appearance or corrosion (AASHTO 2015). The ACI discusses 0.016-inch-wide cracks as being reasonable in certain favorable environmental conditions (ACI 2008).

1.7.1 Bolton, Perez Reports on Main Span Concrete Cracking

Before and after the SPMT move, Bolton, Perez (the CEI) sent three reports (on February 13 and 28 and on March 13) to MCM documenting the condition of concrete cracks in the main span.[46] In addition, FDOT, FIU, FIGG, MCM, and Barnhart employees took photographs of the cracking.[47]

When Bolton, Perez sent the February 13 report, members 2 and 11 had already been stressed. The falsework under the canopy and deck had not yet been fully removed. The FIGG EOR was to document the significance of the findings, the condition of the structure, and the requirements for repair. On February 16, FIGG responded to the Bolton, Perez report (see appendix E).

On February 24, during the removal of the span-supporting formwork, construction personnel working on the structure reported hearing a loud, distinct sound of concrete cracking that came from the structure. Construction activities were briefly halted, and the structure was inspected. A crack was found in the truss member 11 and 12 nodal region near and at the truss member 11 intersection with the deck. A similar crack was found in the member 1 and 2 nodal region at the opposite end of the bridge span. Bolton, Perez provided the second crack report to MCM on February 28, after all the members had been stressed and the falsework had been removed. However, Bolton, Perez mislabeled the general location of the documented cracks on the report photographs.[48] On March 7, FIGG notified MCM of the mislabeling and also provided preliminary comments in response to share with Bolton, Perez (see appendix E). As discussed below, Bolton, Perez emailed the third crack report to MCM on the morning of March 13.

1.7.2 Documentation of Cracking Among Parties

Following submission of the first two crack reports and at the time of the third report, numerous emails and other forms of communication were shared among the contractors working on the pedestrian bridge:

- ***March 8–10:*** Before the SPMT move, Barnhart and Bolton, Perez documented a crack at the pylon pier diaphragm (also referred to as diaphragm 2) west face, while the main span was in the casting yard. After the SPMT move, Bolton, Perez photographed the same crack from both the east and west faces, showing that it had not grown or enlarged during the SPMT move on March 10. The transport of the main span onto the piers concluded about 12:30 p.m., and diagonal supports 2 and 11 were detensioned immediately afterward. Between 12:29 and 12:31 p.m. on March 10, FIGG took several photographs of diaphragm 2 (at the pylon pier). MCM also took photographs during the detensioning of truss member 11.

[46] The Bolton, Perez reports to MCM and the FIGG responses were sent via email. (See appendix E.)

[47] Photographs documenting the cracking began to be taken on February 13 and continued to be taken through the morning of the collapse on March 15.

[48] For example, a label should have pointed to the chamfer region at the end of truss member 11 and the connection to the bridge deck, or to the chamfer region at the end of truss member 2 and the connection to the deck.

Bolton, Perez monitored and documented the growth of the cracks beginning on March 11 to determine if they were active or dormant.

- ***March 12:*** MCM documented the development of cracks at the northern end of the precast main span (diaphragm at pylon pier, north and south faces) on March 12. At 4:51 p.m., MCM sent an email to FIGG transmitting 16 photographs of diaphragm 2, with two photographs of cracking at node 11/12. MCM wrote that some of the cracks were rather large and of concern, requesting that FIGG review them and advise of any required course of action. Per FIGG, it received no phone calls or correspondence from MCM between March 10—when the photographs were taken—and March 12, when the email was sent.

- ***March 13:*** FIGG opened the email from MCM at 7:45 a.m. and telephoned about 9:30 a.m. to discuss the timeline of the observed cracking and how the cracks had evolved when detensioning the PT rods of member 11. MCM reported during this call that the cracking depicted in the photographs sent by email on March 12 had been present *before* the detensioning of the PT rods and that it had worsened afterward.

  At 9:45 a.m., FIGG responded to MCM, indicating that the cracking was not a safety issue and recommending that plastic shims be placed underneath diaphragm 2 at the bridge centerline. Per FIGG, all discussion focused on diaphragm 2; there was no discussion of the member 11/12 nodal region.

  A series of internal FIGG emails (at 11:58 a.m. and 1:44 p.m.) confirmed that the cracks were observed prior to the detensioning of diagonal truss member 11 and grew slightly afterward. NTSB investigators reviewed the emailed photographs and, based on the review, documented that the cracks appeared to have significantly progressed after the detensioning of member 11.

- ***Also on March 13:*** At 10:59 a.m., Bolton, Perez emailed the third crack report to MCM, stating that, "As discussed earlier, I recommend we monitor and document the growth of these cracks to determine if these are active and developing further or dormant. Please let us know the outcome of the EOR analysis and course of action."[49] At 5:00 p.m., FIGG telephoned MCM to provide a verbal update on its evaluation. According to FIGG—because MCM stated that (1) the cracks at the north end of the precast main span had grown since first observed on March 10, after the detensioning of the temporary PT rods in members 2 and 11; and (2) the cracking had worsened since the PT rods were detensioned—it recommended restressing the temporary PT rods in member 11 to return it to its previous state when the cracks were known by MCM to have been smaller. At 5:18 p.m., FIGG emailed a response to MCM, confirming their earlier telephone conversation and the FIGG determination that the cracking was not a safety issue and its recommendation that member 11 be restressed.

Figure 24 is a photograph taken by MCM that documents the cracking observed on March 13.

[49] The photographs were identical to ones sent by MCM to FIGG on March 12 at 4:51 p.m.

Figure 24. Cracks of 3–4 inch depth at northern end of precast main span, along west side of diaphragm 2 (north view), March 13, 11:17 a.m. (Source: MCM)

- ***March 14:*** At 1:38 p.m., MCM replied to FIGG by email to discuss the cracks in the area of nodes 1l and 12 and included additional photographs of cracking on the north face of diaphragm 2 and on the shims placed under the type 2 diaphragm at the bridge centerline. Table E-1 (in appendix E) documents the emails between Bolton, Perez; MCM; and FIGG on March 13–14, 2018.

Table 2 summarizes communications regarding the identification and assessment of structural damage in the member 11/12 nodal region at the north face of diaphragm 2 after movement of the main span and detensioning of the PT rods in members 2 and 11 on March 10. In this documented communication, the FIGG EOR and the design manager clearly express that the cracks are not a safety concern.

Table 2. Selected communications related to cracks in member 11/12 nodal region, for March 13-15, 2018.

Date	Time	Communication Method	Response
March 13	9:45 a.m.	Email from FIGG design manager to MCM	“We do not see this as a safety issue”
--	4:13 p.m.	Voice mail message from FIGG EOR to FDOT	“But from a safety perspective, we don’t see that there’s any issue there, so we’re not concerned about it from that perspective”
--	5:18:22 p.m.	Email from FIGG design manager to MCM	“Again, we have evaluated this further and confirmed that this is not a safety issue”
March 14	10:50 a.m.	Email from MCM to Structural Technologies	“FIGG has further evaluated and confirmed that the cracks encountered on the diaphragm do not pose a safety issue and/or concern”

March 15	9:00 a.m.	Presentation by FIGG EOR at meeting with FDOT; FIU; MCM; Bolton, Perez (and others)	“And, therefore, there is no safety concern relative to the observed cracks and minor spalls”
--	--	Meeting minutes prepared by Bolton, Perez	“FIGG assured that there was no concern with safety of the span suspended over the road”
--	--	Meeting minutes of March 15 prepared by FIGG	“Based on the discussions at the meeting, no one expressed concern with safety of the span suspended over the road”

1.7.3 Day of Collapse

On March 15, about 8:00 a.m., the FIGG EOR and another employee went onto the main span and viewed the cracks at the member 11/12 nodal region—in advance of a 9:00 a.m. presentation by FIGG to FDOT, FIU, MCM, and Bolton, Perez to discuss the cracking. The FIGG employee (not the EOR) took seven photographs of the cracks. The FIGG EOR did not view these photographs before, during, or after the 9:00 a.m. meeting. Prior to the presentation, MCM provided site inspection photographs to the FIGG EOR, and he stated during the meeting that “the cracks look more significant in person than on the photographs.”[50] The bridge collapsed about 1:46 p.m. Postcollapse, FIGG provided the photographs to NTSB investigators, accompanied by a description of the location of the cracks, with date, time, and source (see figures 25 and 26).

Figure 25. Cracks at bottom of diagonal member 11 (west view), March 15. (Source: FIGG)

[50] This statement was documented by Bolton, Perez in its March 15 meeting minutes. FIGG does not provide the statement in its version of the meeting minutes. See the NTSB public docket for this investigation (HWY18MH009).

Figure 26. Cracks at bottom of diagonal member 11 (east view), March 15. (Source: FIGG)

Postcollapse, NTSB investigators interviewed a FIGG design manager, who stated the following regarding the rationale for restressing the PT rods on the morning of March 15:

> . . . CEI and MCM had observed some cracking there at that north end region. And they said that after destressing the [PT] rods, it was observed that the cracking had gotten slightly worse . . .
>
> But as part of the recommendations coming out of the end of the day Tuesday [March 13] based on the observations from MCM and CEI that the cracks got a little bit worse when they detensioned the PT rods, the direction from the design team was well, let's go back one step backwards, you know, from the design standpoint and go ahead and reinstall those PT rods on the north side only for truss member 11. Not truss member 2; only truss member 11.

FIGG engineers confirmed to NTSB investigators that there was no specific sequence for (1) stressing the PT rods in the casting yard, or (2) detensioning the member 2 and 11 PT rods after transporting the main span onto the piers. FIGG provided a specific sequence for restressing the member 11 PT rods, and the collapse occurred immediately after this activity.

1.7.4 Restressing of Member 11

As described in section 1.7.2 and appendix E, on March 13 at 5:18 p.m., FIGG responded to the third Bolton, Perez crack report in an email, stating: "As you and I just discussed, please find the additional recommendations and requests below that FIGG thinks will be beneficial for the structure. Again, we have evaluated this further and confirmed that this is not a safety issue."

According to the email, on March 15, member 11 was to be restressed to its original stressing force (its condition in the casting yard) of 280 kips (or 280,000 pounds), in the increments shown below:

- Stress top rod to 50,000 pounds (50 kips).
- Stress bottom rod to 50,000 pounds.
- Stress top rod to 100,000 pounds.
- Stress bottom rod to 100,000 pounds.
- Stress top rod to 150,000 pounds.
- Stress bottom rod to 150,000 pounds.
- Stress top rod to 200,000 pounds.
- Stress bottom rod to 200,000 pounds.
- Stress top rod to 250,000 pounds.
- Stress bottom rod to 250,000 pounds.
- Stress top rod to 280,000 pounds, final force.
- Stress bottom rod to 280,000 pounds, final force.

Postcollapse, on August 27, 2018, FIGG confirmed to NTSB investigators that no specific sequence was provided for the order of stressing individual PT rods in the casting yard or detensioning the member 2 and 11 PT rods after transport of the main span. FIGG stated—

> A specific sequence for the order of stressing the PT bars within a given member was not provided as no specific order was required. There was no specific order for destressing the temporary PT bars in members 2 and 11, just that they could be destressed after the precast Span 1 (main span) was placed on the bearings/supports at end bent 1 and the lower pylon.

Upon receipt of the March 13, 5:18 p.m., email from FIGG, at 10:43 a.m. on March 14, MCM began coordinating the restressing of member 11 with Structural Technologies as a "rushed request." Structural Technologies explained that its work crews were out of town and asked MCM to check with the FIGG EOR to ascertain whether other work was necessary prior to the restressing operation (such as epoxy injection to fill voids and avoid further cracking before implementing 560,000 pounds of force on this area [that is, 280,000 pounds each for the top rod and the bottom rod]).[51]

[51] The email from Structural Technologies to MCM on March 14 reads: "Saturday we brought in the crew from out of town to detension the rods per your request. The cracking was observed on the bridge. However, prior to additional work on the bridge, the EOR needs to analyze the bridge before any additional work is done. For me to just bring guys to the site Thursday will delay other projects. Please check with the EOR if work is to be completed prior to the stressing . . . After the EOR reviews the area, we can schedule the appropriate crew whether being epoxy injection crew first or the stressing crew."

MCM responded, indicating that Structural Technologies misunderstood, and that FIGG had further evaluated and confirmed that the cracks encountered on the diaphragm did not pose a safety issue or concern.[52] Therefore, the restressing was to be done as promptly as possible because no other work was considered necessary. Structural Technologies prepared the rushed request for the change order the same day (March 14) and requested that MCM approve it so that work crews could be mobilized and onsite the following day between 9:30 and 10:00 a.m. MCM formally approved the change order on March 14.

The restressing of member 11—which was performed on March 15—was not shown on the design plans prepared by FIGG. The FIGG EOR stated in a March 20 interview with NTSB investigators that the purpose of restressing member 11 was to address the cracking and bring the main span back to its "pre-existing condition" in the casting yard. He emphasized that he did not think the restressing of member 11 was a change to the design plans because it was bringing the structure back to its "pre-existing condition." For this reason, the change in the build/construction was not independently peer reviewed. The FDOT guide specifications for pedestrian bridges indicates in section 10.3 that any design calculations, details, or changes must be signed and sealed by a professional engineer (P.E.) licensed in the state of Florida (FDOT 2015b).

1.7.5 Exclusion of CEI Contractor

Bolton, Perez was not involved in precoordination for the restressing operation. It was not informed of the restressing of member 11 until March 15 at 9:00 a.m. As a result, Bolton, Perez was unable to have The Corradino Group (its contracted post-tensioning inspector) onsite for the operation—which was ongoing when the bridge collapsed at 1:46 p.m. After being notified of the restressing of member 11 at the 9:00 a.m. meeting, Bolton, Perez requested a written plan for such and was verbally informed that the procedure was being done incrementally.[53] MCM informed Bolton, Perez that Structural Technologies was currently onsite to perform the restressing operation.

Because Bolton, Perez had just learned of the restressing and did not have its contracted post-tensioning inspector onsite, it dispatched an employee to the canopy to observe the operation. Another employee positioned on the bridge deck during the restressing of the bar tendons did not observe any increase in the length or size of the cracks in member 11.

During the postcollapse investigation, Bolton, Perez reported to NTSB investigators that:

[52] The responding email from MCM to Structural Technologies on March 14 reads: "It seems you misunderstood our conversation of yesterday. As explained, FIGG has further evaluated and confirmed that the cracks encountered on the diaphragm do not pose a safety issue and/or concern the request. Contrary to your email, we will not be stressing the pylon bars yet. What I mentioned to your [sic] yesterday was that truss member #11 needed to be re-stressed as indicated by the EOR. Both PT bars should be stressed to the 280 kips stressing force as listed on plan sheet B-69 and these bars should be stressed in 50-kip increments each, starting with the top pt bar, then bottom pt bar, then back to the top pt bar, etc. Therefore, we asked [you] to schedule the work as promptly as possible and as [sic] no other work is to proceed at this time. As conveyed, we have a crane available for tomorrow, please confirm availability."

[53] FIGG instructed that the cracks be closely monitored at diaphragm 2 and that the restressing be done in 50-kip increments. Based on evaluations, FIGG anticipated that the cracks would either remain the same or, more likely, decrease. If the cracks increased in size, the operation would stop and FIGG would be notified immediately.

> The re-stressing of bar-tendons A and B in Member 11 after erection was verbally communicated by the EOR during the meeting on the morning of March 15, 2018. We assume that the EOR and the contractor had exchanged information regarding the re-stressing operations before the meeting since we now know this work was being set up during the same time the meeting was taking place. The work was part of the Design/Build Team's remedial plan for correcting the cracking occurring at the joint between Members 11 and 12. In addition to the re-stressing work in Member 11, other aspects of the remedial plan were discussed by the EOR during the meeting, including adding additional longitudinal post-tensioning along the bottom of the truss, as well as attaching steel stiffening elements along the top of the truss. This remedial work was not included in the contract plans, and it was requested during the meeting that this remedial work be reviewed and approved for implementation, including peer reviewed, prior to performing the work.
>
> During the meeting, we were informed for the first time that preparations for re-stressing of bar-tendons A and B in Member 11 were on-going and that the work would be taking place immediately. Although we requested a written plan for the work, we were only told verbally that the re-stressing would take place incrementally. Each bar-tendon in member 11 would be stressed in 50K increments each, alternating between bar-tendon A and B, until the full 280K force was applied to each bar-tendon. Given that at this time . . . The Corradino Group, our post-tensioning inspector, was not on site, and that we had just been informed of the post-tensioning operations taking place immediately after the meeting, we dispatched [an employee of] Bolton, Perez to *only observe* the re-stressing operations on the canopy and report the activities. [A] Bolton, Perez [employee] went on the bridge deck to observe the behavior of the cracks in Member 11 during the re-stress of the bar-tendons and did not observe any increase in length or size of the cracks in Member 11.

1.8 Bridge Superstructure Final Design Calculations and Modeling

The final design calculations for the pedestrian bridge superstructure were signed and checked by P.E.s employed by FIGG. These calculations were the basis for the FIGG design plans used by MCM to construct the bridge.[54] The FIGG design provided detailed information on required construction materials, components, and procedures. The FHWA assessments of the FIGG design are referred to herein as the "FHWA postcollapse check."

Structural design typically follows a two-step process in which the designer first analyzes the structure to determine the forces (demands) on the structure's components due to the applied loading, and then designs these components and their connections with sufficient capacity to withstand the forces (demands). The designer uses provisions from the applicable code or specification to ensure that each structural component and connection has enough capacity to withstand the applied demand.

[54] FDOT mandated that the FIGG design meet the requirements of multiple established bridge design and construction specifications. The design was the basis for the FIGG plans.

The provisions in the *AASHTO LRFD* were to be used to design the FIU pedestrian bridge. This document uses a reliability-based approach to ensure adequate structural capacity. The desired reliability is achieved by amplifying the expected loading acting on the structure by a statistically based multiplier (load factor), and by reducing the expected structural capacity by a statistically based multiplier (resistance factor). The separation created by amplifying the expected loading and reducing the expected capacity provides a level of confidence that sufficient capacity will be available to withstand multiple possible and/or extreme loadings. Each design is completed by checking that the reduced capacity is greater than the factored loading effects (demands).

1.8.1 Design Requirements (Demand)

The *AASHTO LRFD* is the primary national guideline for the design of bridge structures, including the FIU pedestrian bridge (AASHTO 2015). Using these specifications, the FHWA postcollapse check focused on design and analysis of the nodal regions.[55]

The pedestrian bridge design requirements mandated use of the *AASHTO LRFD* load factors and load combinations. A load combination is a predetermined set of loadings applied simultaneously to the structure. Each loading has its own unique load factor. The individual loads included in the load combination are then either increased or decreased to produce the most conservative singular total factored force effect (demand) for that load combination. The load combinations and factors are referred to as “limit states.” The FIGG design for interface shear used the “Strength I” limit state to generate the applied force effects (demand) on the truss member nodal regions with the load factors, as provided below:

- Dead load of structural components and nonstructural attachments (DC) = 1.25.
- Pedestrian live load (PL) = 1.75.
- Post-tensioned tendon force (PT) = 1.0.[56]
- Force effect due to uniform temperature (TU) = 0.5.

1.8.1.1 FIGG Bridge Modeling and Analysis. The pedestrian bridge was constructed in multiple stages. Each construction stage generated unique forces on the bridge structure, which had to be designed to withstand those forces.

FIGG used four analytical models for the superstructure final design calculations. The models were used to evaluate three critical construction operations: main span transport during stage 2; main span in simply supported condition with members 2 and 11 both tensioned and detensioned, at the conclusion of stage 2; and completed bridge, at the conclusion of stage 4.

[55] Nodal regions are located at any part of the bridge in which truss members are connected to either the bridge canopy or the deck.

[56] The *AASHTO LRFD* does not have a force effect for primary post-tensioned force, most likely because it is typically on the capacity side of the design. AASHTO does prescribe secondary force effects from post-tensioning on continuous structures (PS). It most likely would not have been appropriate to use secondary force effects (with a 1.25 factor) for designing the pedestrian bridge.

Each model and the corresponding bridge configuration are listed below (refer to figure 13 in section 1.6 for bridge stage configurations). The proprietary names for the two software packages used in the calculations are LARSA and LUSAS.[57]

- LARSA longitudinal model: two-dimensional model of the complete two-span, continuous structure (stage 4).
- LARSA main span erection model: two-dimensional model of the main span supported at SPMT locations during transport (stage 2).
- LUSAS simple support: three-dimensional solids model of the main span simply supported at the south landing abutment and the pylon pier (stage 2).
- LUSAS fixed pylon: three-dimensional solids model of the main span simply supported at the south landing abutment and fixed at the pylon pier, adjusted to replicate the main span performance in its completed condition (stage 4).

Each analytical model generated multiple force effects for every structural component included. The truss member axial forces were extracted from these analyses and used in the design of each truss member connection. The axial force in the truss member was subsequently resolved into vertical and horizontal components. The vertical component is the compressive or clamping force that contributes to interface shear resistance. The horizontal component is the shearing force on the interface shear surface or the interface shear demand. Figure 27 shows the factored interface shear demand results for each main span truss nodal region generated from the four models used in the FIGG design. Regarding the results from the LUSAS simple support and fixed pylon models, the FIGG design included only the forces identified by the designer as the most critical. Therefore, the results shown in figure 27 reflect only the identified maximums between these two analytical models.

[57] See LARSA website and LUSAS website, accessed September 23, 2019.

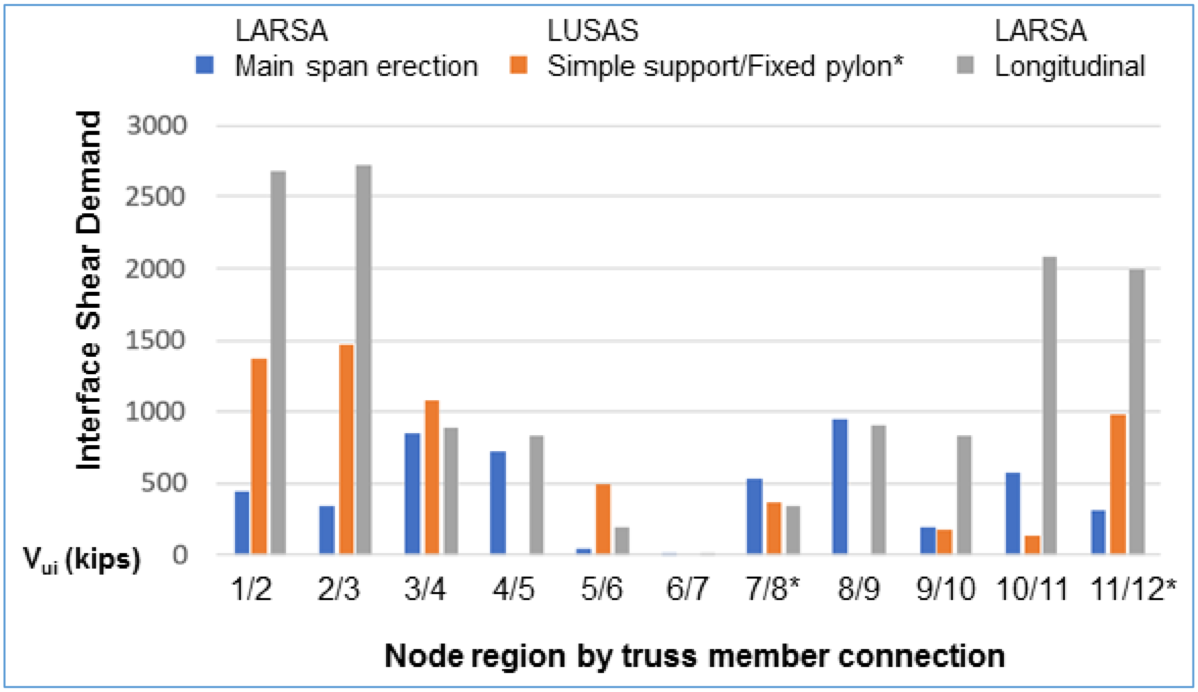

Figure 27. Nodal region interface shear demand results generated for FIGG bridge design, with horizontal axis showing two truss member identification numbers that connect into each nodal region. (Source: FHWA 2019)

The factored interface shear forces shown in figure 27 were generated from the load combination and load factors prescribed in the *AASHTO LRFD* Strength I limit state, which FIGG determined to be the controlling limit state for the interface shear design of all truss cold joints in the nodal regions. Figure 27 shows the significant variations in the interface shear demand generated for each nodal region. The forces generated from the LARSA longitudinal model (for the completed bridge structure) generated the largest forces for the nodal regions at the north and south ends of the main span. However, the FIGG design exclusively used the results from the LUSAS simple support and fixed pylon modeling combination for the design of every main span nodal region.

1.8.1.2 Design Capacity Calculations. The superstructure final design calculations (dated February 2017) for the pedestrian bridge included the calculations for the connections between the truss elements and the deck and canopy. These calculations followed the *AASHTO LRFD Bridge Design Specifications* (seventh edition, with 2015 interims [AASHTO 2015]). FIGG used the *AASHTO LRFD* design provisions in article 5.8.4, Interface Shear Transfer–Shear Friction, for design of the truss member–bridge deck and canopy connections. Table 3 shows the governing equation to determine connection capacity, referred to as “nominal capacity.”

Table 3. *AASHTO LRFD Bridge Design Specifications* equations.

***AASHTO LRFD* Equation**	***AASHTO LRFD Bridge Design Specifications***	
Nominal Capacity (governing equation to determine connection capacity)		
5.8.4.1-3	$V_{ni} = cA_{cv} + \mu\ (A_{vf}f_y + P_c)$	
	V_{ni}	nominal interface shear resistance (kip)
	c	cohesion factor specified in article 5.8.4.3 (ksi)
	A_{cv}	area of concrete considered to be engaged in interface shear transfer (inches^2)
	μ	friction factor specified in article 5.8.4.3 (dimensionless)
	A_{vf}	area of interface shear reinforcement crossing shear plane within area A_{cv} (inches^2)
	f_y	yield stress of reinforcement but design value not to exceed 60 (ksi)
	P_c	permanent net compressive force normal to shear plane; if force is tensile, 0.0 (kip)[a]
[a] The permanent net compression, P_c, is beneficial to developing interface shear capacity. The *AASHTO LRFD*, article 3.4.1, states: "Where the permanent load increases the stability or load-carrying capacity of a component or bridge, the minimum value of the load factor for that permanent load shall also be investigated . . . both extreme combinations may need to be investigated by applying either the high or the low load factor as appropriate."		
Upper Limits on Nominal Capacity		
5.8.4.1-4	$V_{ni} \leq K_1 f'_c A_{cv}$	
5.8.4.1-5	$V_{ni} \leq K_2 A_{cv}$	
	In which: $A_{cv} = b_{vi} L_{vi}$	
	f'_c	specified 28-day compressive strength of weaker concrete on either side of 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)
	b_{vi}	interface width considered to be engaged in shear transfer (inches)
	L_{vi}	interface length considered to be engaged in shear transfer (inches)
Nominal Resistance		
5.8.4.1-2	$V_{ri} = \varphi\ V_{ni} \geq V_{ui}$	
	V_{ni}	nominal interface shear resistance (kip)
	V_{ui}	factored interface shear force due to total load based on applicable strength load combination (kip)
	φ	resistance factor; for normal weight concrete, value is 0.90

- The *AASHTO LRFD* defines the dead load factor as either 1.25 (to generate a maximum) or 0.90 (to generate a minimum). Although this provision is not explicitly required in the *LRFD* when calculating interface shear capacity, the FHWA strongly recommends using the 0.90 factor on dead load when determining $\boldsymbol{P_c}$ to obtain a minimized interface shear capacity value. The FHWA does not consider it appropriate to use the maximum load factor. The highest dead load factor that can reasonably be justified when determining $\boldsymbol{P_c}$ is 1.0. The 1.25 load factor was used in the FIGG design calculations; because the 0.90 load factor is not an explicit requirement for interface shear, the FHWA postcollapse check used a load factor of 1.0.

- The *AASHTO LRFD* sets upper limits on the capacity generated from equation 5.8.4.1-3 (see table F-4 for a comparison of modeling results). The nominal interface shear resistance, $\boldsymbol{V_{ni}}$, cannot exceed values determined from equations 5.8.4.1-4 and 5.8.4.1-5. The least of these three nominal capacity determinations is the controlling nominal interface shear resistance. The *AASHTO LRFD* provisions require that the nominal resistance calculated from the equation above be reduced by multiplying it by a resistance factor, $\boldsymbol{\varphi}$. This reduced value is the "factored interface shear resistance" (capacity) and is designated $\boldsymbol{V_{ri}}$. It must be greater than the factored interface shear force (demand), designated $\boldsymbol{V_{ui}}$, for the appropriate strength limit state, as specified in article 5.8.4.1.

1.8.2 Interface Shear Design Calculations (Capacity)

The FIGG superstructure final design calculations (dated February 2017) for the pedestrian bridge followed the *AASHTO LRFD* interface shear provisions described in section 1.8.1. Each variable selected is explained as noted:

- $\boldsymbol{c}$ **= 0.0 ksi**: cohesion factor, indicates that the computed capacity does not include the effects of cohesion.
- $\boldsymbol{\mu}$ **= 1.0**: friction factor, indicates that the interface surface is clean and free of laitance, with surface roughened to an amplitude of 0.25 inch.
- $\boldsymbol{K_1}$ **= 0.25**: fraction of concrete strength, indicates either normal weight or lightweight concrete placed monolithically against a clean concrete surface and free of laitance, with surface intentionally roughened to an amplitude of 0.25 inch.
- $\boldsymbol{K_2}$ **= 1.5 ksi**: limiting interface shear factor, indicates normal-weight concrete placed monolithically.
- $\boldsymbol{\phi}$ **= 0.90**: resistance factor for shear, per *AASHTO LRFD* article 5.5.4.2-1.
- $\boldsymbol{f_y}$ **= 60 ksi:** yield strength of reinforcing steel that crosses the interface plane.
- $\boldsymbol{f'_c}$ **= 8.5 ksi**: specified 28-day compressive strength, indicates the weakest concrete compressive strength on either side of the interface.

Appendix F provides the interface shear capacity calculations included in FIGG's superstructure final design calculations for each nodal zone connection to the bridge deck (see table F-3).

1.9 Louis Berger Independent Peer Review

1.9.1 FDOT *Plans Preparation Manual* Requirements for Peer Reviews

Per the FDOT *Plans Preparation Manual* in effect at the time of the project advertisement and the governing language of the request for proposals, the pedestrian bridge was classified as a category 2 structure—the design of which required an independent peer review (FDOT 2014a). The bridge was so classified because it was a post-tensioned concrete bridge; and it used design concepts, components, details, and construction techniques with a history of less than 5 years of use in Florida. It was required that the independent peer review firm have no other involvement with the project and be prequalified in accordance with *Florida Administrative Code* (FAC) Rule 14-75.

The 2017 edition of the *Plans Preparation Manual* further clarified that a bridge is a category 2 structure when any of the following are present (FDOT 2017):

- New bridge types.
- New materials used to construct bridge components.
- New bridge construction methods.
- Nonstandard or unusual bridge component-to-component configurations and connection details.
- Items not covered by the *Standard Specifications for Road and Bridge Construction* (FDOT 2015a).

The pedestrian bridge was a concrete truss configuration with a single line of diagonal and vertical supports (new bridge type). It was constructed using high-performance concrete containing titanium dioxide (new materials); ABC techniques (new construction methods); and an irregular pattern of diagonal support members, each with different angles and lengths (nonstandard or unusual bridge component configurations and connection details).

The bridge also was classified as a category 2 structure because of a modification made to the general specifications for the design–build contract, which replaced Division 1, General Requirements and Covenants, of the *Standard Specifications for Road and Bridge Construction* (FDOT 2015a). However, Division II, Construction Details, and Division III, Materials, remained in effect and were incorporated by reference into the contract.

Both the 2014 and the 2017 editions of the *Plans Preparation Manual* (FDOT 2014a and 2017) indicate that—

> The peer review is intended to be a comprehensive, thorough independent verification of the original work. An independent peer review is not simply a check of the EOR's plans and calculations; it is an independent verification of the design

> using different programs and independent processes than what was used by the EOR.
>
> In addition, the manuals specify that—
>
> All independent peer reviews include, but are not limited to, the independent confirmation of the following when applicable:
>
> 1. Compatibility of bridge geometry with roadway geometrics including typical sections, horizontal alignment, and vertical alignment. Minimum lateral offsets and vertical clearance requirements.
> 2. Compatibility of construction phasing with Traffic Control Plans.
> 3. Conflicts with underground and overhead utilities.
> 4. Compliance with AASHTO, Department, and FHWA design requirements.
> 5. Conformity to Department Design Standards.
> 6. Structural Analysis Methodology, design assumptions, and independent confirmation of design results.
> 7. Design results/recommendations (independent verification of the design).
> 8. Completeness and accuracy of bridge plans.
> 9. Technical Special Provisions and Modified Special Provisions where necessary.
> 10. Constructability assessment limited to looking at fatal flaws in design approach.

For items 6 and 7 above, when category 2 superstructure elements are designed with software using refined analyses (for example, Grid, Finite Element Method, etc.), the peer review consultant is required to verify the design results by using a different program/method. The FDOT *Plans Preparation Manual* did not have a specific requirement for all nodes and connections of category 2 bridge structures to be so checked and verified (FDOT 2014a).

1.9.2 FIU/FIGG Contractual Requirements for Peer Review

1.9.2.1 Scope and Certification. FDOT specifications for the project required an independent peer review to be conducted by a separate firm. The FIU request for proposals in June 2014 from design–build firms specified that a peer review analysis by an independent engineering firm was required, as follows:

> . . . 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 plan’s 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.

According to the September 30, 2015, FIGG and MCM technical proposal, the quality management plan indicated that (1) the independent review would be performed by FIGG prior to 90 percent plan submittal; and (2) the final component submittal to FIU would be completed by a different design office within FIGG, which would compare the calculations with the original design to verify adequacy. The quality management plan also stated that FIGG would lead the design quality team to ensure that all aspects of the design followed prescribed procedures. The plan further indicated that the original FIGG design team would not be involved with the FIGG team that completed the independent review. MCM and FIGG's agreement, dated April 28, 2016, stated that—

> FIGG will perform the final structural design and contract document preparation including analysis and design of the bridge superstructure, substructure, and foundations related to preparation of final construction contract documents. This work includes:
>
> > 8. Design quality control and quality assurance in accordance with the project Professional Service Quality Control Plan, including independent design check of bridge.

At a meeting on June 30, 2016, FDOT informed FIGG that an independent peer review performed by an independent engineering firm was required.[58] Shortly thereafter, FIGG requested bids from independent peer review firms.[59] Louis Berger submitted a bid and scope of work, and FIGG followed up on July 5, 2016, in an email that stated: "we want to further coordinate with you and your team on performing the independent peer review for the above referenced [FIU bridge] project" and provided a link with preliminary information about the bridge project. Louis Berger confirmed to FIGG via email on July 6, 2016, that it was FDOT-prequalified for work type 4.3.1. (See section 1.9.2.2 below.)

On August 10, 2016, Louis Berger sent an email to FIGG stating, "Please note the quote we have is for a very thorough scope and creation of independent models. Please inform FIU on evaluating bids, as a lesser fee may be associated with less effort/value."[60] A later internal FIGG email sent on August 10 listed the subconsultants who had bid to perform the independent peer review as Louis Berger (bid for scope of work at $110,000) and two other firms whose bids were $85,000 and $63,000.

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[58] See June 30, 2016, meeting minutes in the NTSB docket for this investigation (HWY18MH009).

[59] On February 6, 2017, FIGG and MCM agreed to a change order to the contract that stated "FIGG had not included an independent peer review (independent firm) in the proposal cost for this project. FDOT specifications require an independent peer review by a separate firm." As part of the change order, MCM agreed to pay the cost of the peer review fee, which the change order stated "represents the increase to FIGG's Contract for this scope" to the cost of the peer review at $61,000.

[60] The Louis Berger email to FIGG also stated, "We would appreciate an opportunity to respond with a BAFO (Best and Final Offer) if necessary to be fair and level the assumptions."

In an email sent to Louis Berger on August 11, FIGG indicated that the original scope of work remained unchanged, but the fee had been revised to $61,000. Louis Berger confirmed both these elements in an email to FIGG sent later that day. In addition to reducing its fee for the project, Louis Berger reduced its timeframe for the project from 10 weeks to 7 weeks, to meet FIGG's requirements. The contract between FIGG and Louis Berger, dated September 16, 2016, indicated that "Louis Berger will perform Independent Peer Review for the concrete pedestrian bridge plans in accordance with the project and request for proposal requirements and FDOT *Plans Preparation Manual* (Chapter 26)."

1.9.2.2 Independent Peer Review Work. Louis Berger reviewed the bridge foundation, substructure (end bents and center tower), and superstructure. The design plans included construction sequencing (including construction sequence drawings), and covered main span precasting, transport of main span, and placement of the main span between end bent 1 (south pier) and the pylon pier. The design plans also included the post-tensioning stressing and destressing sequences and phases.

Louis Berger performed the independent peer review using ADINA, a finite element model software program, in accordance with provision 6 of the 2014 FDOT *Plans Preparation Manual* (FDOT 2014a, discussed above). Postcollapse, Louis Berger confirmed to NTSB investigators that it analyzed the design of the entire bridge structure (in its completed state) and the forces within the members themselves. The Louis Berger engineer told investigators that—

> My model was for the structure as one structure. Doing construction sequence staging analysis was not part of our scope. And again, doing such an analysis requires much more time than what we agreed about [with FIGG].

NTSB investigators confirmed that the independent peer review analyzed the entire structure as one unit but did not analyze the different construction sequence configurations and nodal areas of each member. In a postcollapse interview, the Louis Berger engineer conducting the peer review stated that—

> . . . in the beginning, I suggested to do this kind of analysis, to analyze the connections. I'm talking about the nodes, or the joints to analyze the connections. However, the budget and time to do this actually was not agreed upon with the designer.

FDOT's *Plans Preparation Manual* (FDOT 2014a) required that the following documents be provided by Louis Berger with plan submittals for category 2 bridges during its independent peer review:

- ***90 Percent Plan Submittal**:* (1) A tabulated list of all review comments from the independent review engineer and responses from the designer, and (2) a standard peer review certification letter. All outstanding comments and issues in the letter must be resolved and implemented prior to the 100 percent plan submittal.

- ***100 Percent Plan Submittal***: A certification letter signed and sealed by the independent review engineer stating that all review comments were adequately addressed and that the design complied with all FDOT and FHWA requirements.

Louis Berger did not provide 90 percent certification letters. (See appendix G.) FDOT informed the NTSB on April 26, 2018 (postcollapse), that the company met the intent of the 90 percent certification, as stated: ". . . the 90% certification was not provided, however, since it is an 'in progress' certification and because we received the final certification which included the review of the 90% work, it would have been included in the 100% review. Therefore, the intent of the 90% was met."

In addition, emails from FDOT to FIGG dated August 23 and 29, 2016, confirmed that FDOT was in agreement with MCM/FIGG regarding submittal of the 90 percent superstructure plans without the independent peer review documentation. The email stated that "Based on our discussion today, we understand that the Department [FDOT] would be ok [okay] with MCM/FIGG submitting the 90% Superstructure Plans to FIU/FDOT without the Independent Peer Review documentation."

Louis Berger did submit 100 percent certification letters, signed and sealed, which stated "hereby certifies that an independent peer review of the above-referenced submittal has been conducted in accordance with [FDOT's] Chapter 26 of the Plans Preparation Manual and all other governing regulations." The 100 percent certification letters (see appendix G) were signed by Louis Berger's independent review engineer, with the following certification statement:

> I certify that the component plans listed in the letter have been verified by independent review, that all review comments have been adequately resolved, and that the plans are in compliance with Department [FDOT] and FHWA requirements presented in the Contract Documents.

The FDOT and FHWA requirements called for Louis Berger to check constructability considerations of the bridge by the *AASHTO LRFD*, section 2.5.3, and by the FDOT *Structures Design Guidelines*, sections 2.13, 4.58, 4.59, and 6.10. Both the *AASHTO LRFD* and FDOT's guidelines were required for Louis Berger's scope of work and included the investigation of the structure during various construction phases. The certification letters were submitted to FIU at the following intervals:

- 100 percent bridge foundation plans: submittal no. 1, September 13, 2016.
- 100 percent bridge substructure plans: submittal no. 2, September 29, 2016.
- 100 percent bridge superstructure plans: submittal no. 3, February 10, 2017.

FDOT performed a quality assurance review of Louis Berger's independent peer review documentation as submitted on September 13 and 29, 2016, and February 10, 2017. FDOT requested the technical proposal and associated documents with the independent peer reviewer's comments, comment responses, and final signed and sealed certification letters as part of its quality assurance review on November 7, 2017. FIGG provided the requested documentation the same day, and FDOT acknowledged receipt of the documentation.

1.9.3 FDOT Qualification of Independent Peer Review Firms

FDOT requires an independent peer review firm to be prequalified in accordance with FAC Rule 14-75—which, among other factors, establishes minimum technical qualification standards by type of work for professional services consultants. Specific to the FIU bridge design (a complex truss bridge), FDOT required that the independent peer review firm be qualified under work type 4.3.1 (complex bridge design-concrete) defined below:

- ***4.3.1 Complex Bridge Design-Concrete:*** Design for the construction, rehabilitation, widening, or lengthening of concrete superstructures for the structure types that include estimated span(s) longer than 400 feet, tunnels, cable-stayed bridges, suspension bridges, truss spans, concrete arch bridges, and bridges requiring unique analytical methods or other design features not commonly addressed in AASHTO publications.

To qualify for subcategory work type 4.3.1, any independent peer review firm would need to employ at least three P.E.s registered with the Florida State Board of Professional Engineers, and each P.E. would be required to have a minimum of 5 years of structural bridge design experience in complex concrete bridges (as defined for work group 4.3.1). According to FDOT records from 2013 through 2018, neither Louis Berger U.S., Inc., nor its predecessor—Louis Berger Group, Inc.—was qualified by FDOT for work type 4.3.1. Additionally, FDOT records indicate that Louis Berger Group, Inc., applied for work type 4.3.1 in 2013, but FDOT did not approve the application due to the applicant's lacking engineers with the required experience in this work type.[61]

As stated earlier, when FIGG requested bids from independent peer review firms, Louis Berger submitted a bid and also informed FIGG via email on July 6, 2016, that it was prequalified by FDOT for work type 4.3.1.[62] While performing this contract work as an independent peer review firm without prequalification from FDOT, Louis Berger submitted to FIU the 100 percent certification letters signed and sealed for the complex bridge design.

At the time that FIGG was procuring the independent peer review, the FDOT website listed Louis Berger as prequalified for work type 4.3.1, complex bridge design-concrete. Postcollapse, the NTSB requested that FDOT review its website for the accuracy of this information. FDOT reported that a technical error had occurred during the processing of its physical records into the

[61] See FDOT letter (in the NTSB public docket for HWY18MH009) dated March 18, 2013, to Louis Berger indicating "insufficient" status for 4.3.1 complex bridge design-concrete. FDOT stated that "resumes did not document 5 years of design experience in the particular structure type." FDOT also stated "this work group primarily applies to the design of post-tensioned concrete superstructures. Structures design work performed by individuals was too vague to properly [assess] design experience."

[62] See Louis Berger email in the NTSB docket dated July 6, 2016, indicating "Yes. I confirm we still have the FDOT complex bridge-concrete prequal. [prequalification] as the web-site indicates."

website-generated report.[63] According to FDOT, its website is informational only and is not intended to be used as a substitute for due diligence in consultant teaming.[64] FDOT issues a letter to prequalified consultants detailing the specific work types for which they have been approved. To verify prequalification status, consultants may request the prequalification letter directly from the firm being considered for peer review services or from FDOT.

1.10 Postcollapse Recovery of Structure and Materials Testing

Following the collapse, numerous samples of concrete, steel rebar and PT rods, and post-tensioning equipment were collected onsite and shipped to the FHWA Turner-Fairbank Highway Research Center (TFHRC) for materials testing and assessment. Additionally, portions from the north end of the failed structure were moved to an FDOT maintenance yard for inspection, with some items then being sent to the TFHRC for testing and assessment. These materials included pieces of member 11, blister 10/11, member 12, and a section of the northernmost deck with node 11/12.[65]

1.10.1 Post-Tensioning System Performance Testing

Just prior to the bridge collapse on March 15, construction workers were applying tension to PT rods on the north end of the bridge span. Postcollapse, the hydraulic jack used for retensioning was found affixed to the lower PT rod in member 11, and the oil pump was located within debris on the bridge canopy.[66] The pressure gauge and hoses were found between the hydraulic jack and the oil pump and remained connected.

Steel samples were also collected on scene for materials testing, including the top portion of the lower PT rod from member 11 and a PT rod from the stockpile of unused building materials stored on the bridge (south end). Sections of steel rebar from the concrete samples—the lower portions of members 11 and 12—were collected at the FDOT yard (see section 1.10.2 for test results).

[63] The copy of the printout from FDOT's website showing Louis Berger's qualification for work type 4.3.1 was undated, and it was not clear to investigators when the printout was downloaded from the FDOT website. At the request of NTSB investigators, FDOT reviewed its website and confirmed that it appeared that Louis Berger Group, Inc., was at one time listed on the department's website-generated prequalification report for work type 4.3.1, complex bridge design-concrete.

[64] Postcollapse, FDOT told the NTSB that "the ultimate burden of identifying work type capabilities is with the firm performing the work." (In this instance, the firm performing the peer review work was Louis Berger.)

[65] The TFHRC received a total of 43 concrete samples. Of these samples, 23 were recovered directly from the bridge structure (bridge deck and canopy) by means of core drilling; 15 were received from two engineering firms (Professional Service Industries and Universal Engineering Sciences) hired to conduct quality control testing; and 5 (2 concrete cylinder samples in molds and 3 concrete pieces [chunks]) were recovered from the collapse site.

[66] The jack had a hollow-core, double-acting configuration. "Hollow-core" refers to the hole in the center through which the PT rod could pass. "Double-acting" refers to the pair of hydraulic oil chambers into which oil could be pumped to either advance the jack (and apply tension to the PT rod) or retract it (and either unload or disengage the jack from a loaded and locked-off PT rod).

The TFHRC assessed the jack used to tension member 11, which was recovered with the piston advanced and the check valve closed. Testing showed that the jack was performing tensioning operations at the time of the bridge collapse.[67] Pressure gauge testing results showed that the relationship between the pressure and the force produced by the jack was linear and also verified the calibration data provided to the NTSB by the post-tensioning vendor.[68]

1.10.2 Concrete and Steel Materials Testing

1.10.2.1 Concrete Materials. Per the FIU "Release for Construction" plans, the canopy, member, and deck concrete was specified to meet class VI standards, with a minimum 28-day compressive strength of 8,500 pounds per square inch (psi), and to be in accordance with section 346 "Portland Cement Concrete," of the FDOT *Standard Specifications for Road and Bridge Construction*—which specifies the air content of class VI concrete to be within the range of 1.0–6.0 percent (FDOT 2015a).

The competency of concrete is of critical importance to the overall performance of any concrete structure. In the vicinity of the member 11/12 nodal region, concrete competency was assessed through the cutting and sampling of retained portions of the node.[69]

All concrete core specimens from the canopy and bridge deck were tested. The TFHRC documented that all samples tested met the compression test requirement and were within the specified range for total air content. Although there are no specifications for concrete tensile strength, the post-failure fracture surfaces of the tension specimens were all perpendicular to the direction of loading, which is typical of brittle materials that fail in tension.

1.10.2.2 Steel Materials. Per the FIU "Release for Construction" plans, the PT rods were specified to be steel grade 150, meeting ASTM A722 (uncoated high-strength steel bars; ASTM 2014b). The TFHRC testing of machined round and full-size rods from both the unused construction rods and the section of PT rod from member 11 met the specified minimum yield strength, tensile strength, and percent elongation at fracture.[70] Also per the plans, the rebar was specified to be steel grade 60, meeting ASTM A615 (deformed and plain carbon-steel rebar; ASTM 2014a).

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[67] During post-tensioning operations, the jacking contractor monitors oil pressure and, through a calibration chart, knows how much load is being applied by the jack at any given pressure.

[68] TFHRC testing of the hydraulic actuation system focused on assessing performance relative to that which the tensioning contractors assumed the system was delivering during bridge construction.

[69] (a) A vertical cut was completed through the retained deck portion of the member 11/12 nodal region, parallel to and 20 inches east of the east face of member 12. The concrete—both here and on the parallel cut 20 inches west of the west face of member 12—was observed to be well consolidated and did not exhibit segregation. In addition, no honeycombing was identified on either cut face. (b) Concrete in other nearby portions of the nodal region was also examined. No honeycombing or segregation was observed in the cores extracted from the deck at the east and west extents of the north end. The concrete in a core extracted from the top deck surface, centered under where member 11 met the deck, was also judged to be competent. Finally, no competency issues were identified in the fractured surfaces of the lower portions of retained members 11 and 12.

[70] The TFHRC tested the samples for chemical composition and documented levels of phosphorous and sulfur below the ASTM A722-specified maximums.

The rebar was tested to ASTM specifications, and the TFHRC found that it met the minimum yield strength, tensile strength, and percent elongation at fracture for the respective sizes.[71]

The positioning of steel reinforcement is a key aspect of predicting the overall performance of a concrete structure. The size and approximate position of steel rebar in the vicinity of the member 11/12 nodal region was examined after the bridge collapse to assess whether there were any deficiencies in rebar size or location. Rebar extending from retained pieces of the bridge and cut sections of the retained deck component were examined to compare with construction plans.

No significant deviations from the construction plans were identified through the sampled assessment of rebar sizes and locations.[72] In constructing the nodal region, MCM met the expectation established on the FIGG plans, which was consistent with the assumptions made in the FIGG design.

1.11 Open Traffic Lanes and Roadway Closures

1.11.1 FDOT, FIU, and Consultant Authorities

FDOT has plenary authority over state rights-of-way and state bridges in Florida and may direct or authorize partial or complete road closures as necessary. For the pedestrian bridge (a LAP project), FDOT did not have an onsite inspector monitoring the construction of the bridge, nor was it required to do so.[73]

The contract between FIU and Bolton, Perez (as CEI) summarized the scope of services and performance in the *Construction Project Administration Manual*, as follows (FDOT 2012):

> **4.1.3 Background**
>
> The Department must ensure the Consultant CEI is performing services in accordance with the scope of services and the contract.
>
> **4.1.4 Role of Consultant CEI**
>
> The Department has representation in administering construction projects through Professional Services contracts. Hence, the authority of the CCEI firm's lead person, such as the Senior Project Engineer, and the CCEI Project Administrator shall be identical to the Department's Resident Engineer and Project Administrator, respectively, and shall be interpreted as such. The Consultant is required to exercise their professional judgment in performing their obligations and responsibilities under the contract. However, the Consultant must seek input from the Construction Project Manager. Therefore, the Department vests the Consultant with the

[71] Samples of steel rebar at sizes 5, 8, and 11 were tested. Deformation from the collapse precluded testing of the size 7 rebar that had been extracted from member 12.

[72] See the NTSB public docket for photographs of the deck cross section examination process (HWY18MH009).

[73] The LAP agreement executed on June 23, 2014, by FIU and FDOT indicated that the pedestrian bridge project would be performed in accordance with all applicable FDOT procedures, guidelines, manuals, standards, and directives.

> responsibility of administering the project(s) and to implement actions based on their authority, subject to the requirements of Section 4.1.6.

The contract stated that the Bolton, Perez project administrator and senior project engineer had identical authority to the FDOT project administrator and resident engineer. However, the contract also specified that Bolton, Perez was to seek input from the construction project manager (FIU), as necessary, in exercising its professional judgment. FDOT entrusted Bolton, Perez with the responsibility of administering the project and implementing actions based on that authority.

Although the authority of Bolton, Perez was identical to the FDOT project administrator and resident engineer, the CEI did not have complete authority to act on its own. Bolton, Perez was to act collectively with FDOT/FIU in providing recommendations and advising, as stated in the CEI scope of work. Section 4.1 of the *Construction Project Administration Manual* sets forth directions concerning the administration of the consultant CEI contract and provides FDOT/FIU with procedures for evaluating the performance of Bolton, Perez.

The following language was included in the contract between FIU and Bolton, Perez as exhibit B:

> **8.0 Performance of the Consultant**
>
> During the term of this Agreement and all supplemental amendments thereof, the Department and/or FIU will review various phases of Consultant operations, such as construction inspection, materials sampling and testing, and administrative activities, to determine compliance with this Agreement. The Consultant shall cooperate and assist representatives in conducting the reviews. If deficiencies are indicated, remedial action shall be implemented immediately. Recommendations and Consultant responses/actions are to be properly documented by the Consultant. No additional compensation shall be allowed for remedial action taken by the Consultant to correct deficiencies.

This language is similar to the language in the *Construction Project Administration Manual* (FDOT 2012):

> **4.1.12 Consultant Performance**
> **Resident Level Responsibilities**
>
> During the early stages of the construction project, the Construction Project Manager shall thoroughly evaluate the performance of the CCEI Firm to ensure the CCEI Firm is demonstrating the necessary knowledge, skills and experience to make decisions in accordance with the Consultant’s Contract. Any deficiencies in the performance of the CCEI Firm will necessitate remedial action, including but not limited to, reassignment of personnel, replacement of personnel, and increase in the frequency of monitoring and inspection activities, and increase in the scope and frequency of training of the Consultant personnel.

The standard form of agreement between MCM and FIGG (the EOR) stated that “Nothing in this agreement shall relieve FIGG of responsibility for errors, inconsistencies, or omissions in the services.”

The FAC, at 61G15-30.002, "Definitions Common to All Engineer's Responsibility Rules," provides the following definition of the EOR: "A Florida professional engineer who is in responsible charge for the preparation, signing, dating, sealing and issuing of any engineering document(s) for any engineering service or creative work." At 61G15-18.011, "Definitions," the code further states—

> As used in Chapter 471, F.S., and in these rules where the context will permit the following terms have the following meanings:
>
> (1) "Responsible Charge" shall mean that degree of control an engineer is required to maintain over engineering decisions made personally or by others over which the engineer exercises supervisory direction and control authority. The engineer in responsible charge is the Engineer of Record as defined in subsection 61G15-30.002(1), F.A.C.
>
> (a) The degree of control necessary for the Engineer of Record shall be such that the engineer:
>
> 1. Personally 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. In making said engineering decisions, the engineer shall be physically present or, if not physically present, be available in a reasonable period of time, through the use of electronic communication devices, such as electronic mail, videoconferencing, teleconferencing, computer networking, or via facsimile transmission.

The contract specifications implied—and the Florida statutes and rules cited above required—that the EOR had the authority to direct or authorize partial or complete road closures as necessary to protect the health, safety, and welfare of the public. Ensuring the safety of the public is emphasized as being every P.E.'s responsibility and obligation, especially when the engineer is in "responsible charge" of the design.

The National Society of Professional Engineers—in its engineering code of ethics (see NSPE code-ethics)—also mandates the same high standard: "Engineers, in the fulfillment of their professional duties, shall hold paramount the safety, health, and welfare of the public."

The contract between FIU and MCM stated—

> Until acceptance by FIU, the work shall be under the charge and custody of MCM. MCM shall take every necessary precaution against injury or damage to the work by the action of the elements or from any other cause whatsoever arising either from the execution or non-execution of the work and shall rebuild, repair, restore and make good, without additional compensation, all injury or damage to any portion of the work.

The contract further specified—

> FIU's Associate Vice President of Facilities Management may appoint Engineer's assistants who are authorized to call to the attention of MCM any failure of the work or materials to meet the contract documents, and have FIU to reject materials or suspend the work until any questions at issue can be referred to and decided by FIU's Associate Vice President of Facilities Management or his/her duly authorized representative.

1.11.2 Traffic Control

The FDOT *Construction Project Administration Manual* provides recommended actions to shut down a project due to maintenance of traffic (MOT) deficiencies (FDOT 2014b):

> **9.1.8 Recommended Action to Shut Down a Project Due to MOT Deficiencies**
> (1) Any 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.

The FDOT *Standard Specifications for Road and Bridge Construction* (FDOT 2015a) provides guidance to ensure the worksite traffic supervisor performs the following duties:

> **102-3.2 Worksite Traffic Supervisor**
>
> 4. Immediately corrects all safety deficiencies and does not permit minor deficiencies that are not immediate safety hazards to remain uncorrected for more than 24 hours.

MOT can be described as the maintenance of traffic to accomplish temporary traffic control. It is a process of establishing a work zone and providing related transportation management on streets and highway rights-of-way. MOT deficiencies did not apply to the pedestrian bridge, however; and the specific language to the contractor (MCM) providing recommended actions to shut down a project was not applicable in this case.

MCM and FIGG were familiar with the FDOT automated system to facilitate lane closures and, on two separate occasions, had requested the closure of traffic lanes, as noted below:

- On January 31, 2018, MCM requested an as-needed two-lane blanket road closure until April 27, 2018, for westbound traffic on SW 8th Street. The closure extended from SW 11th Avenue to 500 feet west of SW 10th Avenue. FDOT approved the request on February 6, 2018.
- MCM engaged FIGG to assist with the application for one permit for closing SW 8th Street for movement of the precast concrete bridge span.[74] On December 12, 2017, FIGG so requested on behalf of MCM, and FDOT worked with local municipalities to permit a full closure of SW 8th Street for transport of the main span to its final position.

[74] This was change order 8 to the MCM and FIGG agreement.

> The general use permit included the bridge movement plans and was approved by FDOT on February 5, 2018.

On FDOT projects, closing a bridge or taking other related safety measures during construction, while not typical, does occur. FDOT provided the following examples in which a bridge was closed to protect the traveling public:

- ***Skyway Bridge transition pier bearing replacement, St. Petersburg, Florida, December 2015:*** During bridge bearing pad replacement operations, where traffic was permitted on the bridge after jacking, the jacking loads caused the diaphragms to spall. The replacement operation was abandoned at the direction of the EOR and the CEI, and the structure was placed back onto the existing bearings.
- ***Memorial Causeway Bridge, Clearwater, Florida, January 2004:*** Severe pier cracking on a balanced cantilever segmented bridge over the Intercoastal Waterway required the EOR and CEI to direct emergency strong-backs and counterweights on the unfinished cantilever to reduce the out-of-balance pier stresses.
- ***Interstate 4 Ultimate Project, Orlando, Florida, April 2018:*** The CEI directed the contractor to shore a c-pier that exhibited cracking.

1.12 FDOT Oversight

1.12.1 LAP Program and FIU Bridge

As discussed in section 1.5, under the terms and conditions of the TIGER grant agreement, FDOT would agree to act as a limited agent for FIU to assist in the receipt and disbursement of TIGER grant monies and to perform other oversight duties as agreed upon between FIU and FDOT. FDOT uses the LAP certification process to determine whether local agencies are qualified to administer federal-aid projects.[75] The purpose of the LAP agreement is to establish consistent and uniform practices for authorizing local agencies to use federal-aid funds provided through FDOT for project planning, project development, design, right-of-way relocation and acquisition, and construction. Per the LAP manual, a local agency is defined as (FDOT 2013):

> A unit of government with less than statewide jurisdiction or any officially designated public agency or authority of such a unit of government that has the responsibility for planning, construction, operation or maintenance of, or jurisdiction over, a transportation facility.

FDOT provided NTSB investigators with the statewide total funding for LAP projects from 2014 through 2018 and the breakdown of how many were design–build projects or conventional design bid–build (DBB) projects (see table 4). A DBB project involves FDOT designing the project

[75] Per the Stewardship and Oversight Agreement (enactment of 23 USC 106(c)), Congress recognized the need to give states more authority to carry out Federal-Aid Highway Program project responsibilities traditionally handled by the FHWA. Under this assignation of responsibilities, FDOT may permit local agencies to carry out its assumed responsibilities on locally administered projects. FDOT is responsible for local agency compliance with all applicable federal laws and requirements.

and assuming the associated risk. Then, following a bid and procurement process, an award is made to a construction company to build the project. Table 5 compares LAP projects by type.

Table 4. Breakdown of total LAP projects, FDOT design–build, and DBB projects statewide, 2014–2018.

LAP Projects	Design–Build Projects	Conventional DBB Projects	Total
493 (21.4%)	134 (5.8%)	1,677 (72.8%)	2,304 (100%)
$.71 billion (4.6%)	$6.1 billion (39.9%)	$8.5 billion (55.5%)	$15.3 billion (100%)

Table 5. LAP projects statewide by type, 2014–2018.

Nonbridge Projects	Bridges	Pedestrian Bridges	Total
483 (98.0%)	7 (1.4%)	3 (0.6%)	493 (100%)
$629 million (89.2%)	$75 million (10.6%)	$1 million (0.2%)	$705 million (100%)

The three LAP pedestrian bridges listed in table 5 include the following:

- Canal Point pedestrian bridge over the L-10 canal, Palm Beach County.
- State Route 5/Overseas Highway pedestrian bridge over Marvin D. Adams Waterway, Monroe County.
- FIU UCPP along SW 109th Avenue and State Route 90/SW 8th Street, Miami-Dade County.

As shown in table 5, pedestrian bridges accounted for less than 1 percent of all FDOT LAP projects over the last 5 years.

Per 23 CFR 635.105, FDOT serves as the prime recipient of federal transportation funds and, as the supervising agency, is responsible for authorizing work by the local agency. In addition, 23 CFR 635.105 requires that the local agency (FIU) have a "responsible charge" as a full-time employee to "maintain familiarity of the day-to-day project operations, including project safety issues."[76]

[76] This is as clarified by the FHWA in an August 2011 memorandum, available in the NTSB public docket for this investigation (HWY18MH009).

FIU received full certification in the LAP program on May 14, 2014. Per FDOT, full certification is reserved for those agencies that demonstrate the qualifications and capability and achieve performance expectations between certification cycles. If the expiration date of the certification occurs during the course of a project, the certification will be considered to remain in effect until the project has been finally accepted by the department and the FHWA.

Although a local agency may be fully certified, FDOT is not relieved of supervision responsibility by certifying a local agency. In addition (FDOT 2013):

> The Department [FDOT] ensures LAP projects receive adequate supervision and inspection and are developed according to approved plans and specifications. The Department [FDOT] final [sic] inspects and accepts all LAP projects.
>
> The Local Agency may administer the project with its own forces or hire a consultant or contractor as appropriate. The local agency controls the day-to-day management and operations of the project.

The FDOT–FIU LAP agreement, executed on June 23, 2014, had the following requirement, as specified in section 2.01:

> The Project will be performed in accordance with all applicable Department [FDOT] procedures, guidelines, manuals, standards, and directives as described in the Department's [FDOT's] Local Agency Program Manual which by this reference is made a part hereof as if fully set forth herein . . .

LAP full certification expires 3 years from the initial certification date unless the expiration occurs during the course of a project, in which case, certification remains in effect until the project has been finalized and accepted by FDOT and the FHWA.[77] The FDOT LAP project agreement contains no language pertaining to closing a bridge when structural cracks are first detected or in situations that require further investigation to ensure the protection of the health, safety, and welfare of the public. FDOT did not conduct a performance evaluation of FIU after the bridge collapse, nor was the FIU LAP certification removed pending the outcome of the NTSB investigation.

1.12.2 FDOT Reviews of New Construction Design Plans

The FDOT *Plans Preparation Manual* required that all structural designs for new construction for FDOT be developed under the direction of the Structures Design Office (SDO).[78] FDOT reviewed the FIU pedestrian bridge project plans because it was a local agency project, pursuant to the LAP agreement between FIU and FDOT. FDOT acted as a pass-through of the federal monies coming in via the TIGER grant to FIU, with the receipt and disbursement of these grant funds. The SDO had total project development and review responsibility, along with

[77] The UCPP contract end date was February 15, 2019, which is when the FIU certification expiration date was anticipated. (See appendix H for more information on LAP certification).

[78] The work could also be done by the District Structures Design Offices or the SDO with the help of the district offices.

structure plan review responsibilities for projects involving category 2 structures and, as appropriate, was to determine when structure component plans should be "Released for Construction."[79]

According to FDOT, there are four phases of bridge project development. Phase one (bridge analysis), occurs during the Project Development and Environment process. Phase 2 includes the development of the bridge-related project constraints based on project-specific requirements and development of the bridge concept plans for inclusion into the requests for proposals.[80] The third phase involves the project procurement process. The fourth phase includes component structure plan reviews in accordance with the requirements of the request for proposal.

For final plans and specifications preparation, the *Plans Preparation Manual* states: there are three phases of work; 60 percent substructure submittal or 60 percent structure plans, 90 percent structure plans, and 100 percent structure plans and specifications. At any time during the project development, the reviewer (FDOT) may require submittal of design calculations. After each of the phases (except the 100 percent structures plans phase), FDOT review comments are sent to the EOR, and the EOR must address each of the comments in writing and resolve each comment before the next submittal.[81] According to FDOT, the review performed on this project by the FDOT SDO—

> was consistent with reviews performed on all projects; it consisted of a high-level review only. We did not perform calculations or review EOR calculations. In addition, this project, like all FDOT Design-Build Category 2 Bridge Projects, required an Independent Peer Review of the bridge design which consists of an independent design verification utilizing different computer software than was used for the design.

Both the design submittal review and approval process were managed through the FDOT Electronic Review Comments (ERC) system website. FDOT's ERC is an application used to track the entire review process (comments and responses) for plan reviews and project submittals in a database. All comments and subsequent comment responses reside in one location, allowing any user easy access to all or partial review data on demand. There were 37 individual reviewers for the FIU bridge plan submittals. The reviewers can be categorized into three groups: FIU and its subconsultant, FDOT and its subconsultants, and FHWA and Miami-Dade County as the third-party reviewers.

FDOT and its subconsultants reviewed the superstructure plans at the 30 percent preliminary, 90 percent, final, and release for construction stages. According to FDOT, all comments were resolved to the FDOT reviewers' satisfaction before the FIU bridge construction

[79] Requests for proposals on those projects where it is anticipated that category 2 bridges will be designed and constructed shall be submitted to the State Structures Design Engineer for review and approval.

[80] FDOT compiled a series of pre-scoping questions that was available on the FDOT website to aid in the development of project-specific constraints. Depending on the complexity of the project and at the discretion of FDOT, this second phase may include a bridge feasibility assessment for the purpose of developing the structure's concept plans.

[81] For any phase, items and drawings from a preceding phase must be included and reflect the comments resolved from the previous phase, as well as the accumulated design and drafting effort required of the current phase.

could begin. The ERC system approval sequence was as follows (see appendix H for the detailed process sequence):

1. FIGG uploaded the plans submittal to FIU, FDOT, and the third-party reviewer through the ERC.
2. FIU, FDOT, and the third-party reviewer then reviewed FIGG's plans submittal and uploaded their comments into the ERC.
3. FIGG reviewed the FIU, FDOT, and third-party comments; then FIGG uploaded its responses into the ERC.
4. FIU, FDOT, and third-party reviewer reviewed FIGG's responses and either approved and closed out the comments or requested a comment resolution meeting to be held.
5. All FIU, FDOT, and the third-party reviewer comments in ERC were required to be approved and closed out by the FDOT reviewer making the comment prior to FIGG making a subsequent submittal or a release for construction.

For the 30 percent preliminary plans submittal, FDOT comments were uploaded to the ERC on March 25, 2016. FIGG responded on April 22, 2016, and FDOT accepted FIGG's response and closed the comments on April 25, 2016. Then, on June 28, 2016, FDOT provided additional comments to the 30 percent preliminary plans submittal, making the following statement at the beginning of the document:

> Comments 1 thru 22 below are for information only. No response is required. The comments are intended to assist in pressing the DBF's [design-build firm's) concept to 90%.

On June 30, 2016, FIGG met with FDOT to review the additional review comments, and FDOT requested that FIGG resubmit the 90 percent foundation and substructure plan submittals with the inclusion of the demand-to-capacity ratios for various components for FDOT review.[82] According to FDOT's draft meeting minutes, FDOT told FIGG:

> The plans need to clearly show the sequence of all stressing. Maintaining stress limits throughout all intermittent phases to avoid cracking of the members will be extremely tricky and will likely necessitate stressing all web members along with some transverse/longitudinal stressing in increments such that members stay in compression. Also predicting where the PT stressing actually goes will be tricky. For instance, any forces imposed on web joints affect all members framing into the joint. Longitudinal stressing of the canopy/walkway will tend to go into the stiff web element and not in the canopy/walkway. Also, the design needs to pay particular [attention to] shear lag affects and member interface shear [horizontal shear] through all phases of stressing.

[82] Documented ERC review comments from the FDOT SDO or its subconsultants did not question redundancy. FDOT told the NTSB postcollapse that the issue of redundancy was discussed as part of a general discussion of FIGG's proposed bridge concept in a meeting between FDOT, FIU, and FIGG on June 30, 2016. However, FIGG stated that, based on its recollection of the meeting, the issue of redundancy was not discussed.

FIGG uploaded the following response in the ERC:

> The 90% superstructure submittal will show in detail, the stressing sequences of the post-tensioning. We agree that the incremental stressing sequence will be important and that the final superstructure design will verify stresses at each of the incremental steps. Shear lag and interface shear were discussed in the above comment clarifications. Relative to these comments, no changes will be made that would alter the 90% foundation design plans.

For all the remaining June 28 FDOT review comments (listed below), FIGG responded in the ERC that the plan submittal review had been accepted and closed by FDOT on April 25 and "It is our understanding that these comments were provided for information only and no response is required at this time. These comments are intended to assist in progressing the DBF's concept to 90% plans." FDOT electronic review comments were as follows:

> There is a concern with tension behind the compression zone due to longitudinal PT of the walkway at the member ends as the top of the web and canopy element gets dragged along (shear lag in region 3).
>
> There appears to be significant shear lag issues in both the canopy and walkway as the stiff web element is being dragged behind the compression zone. The designer needs to pay particular attention in these areas. Moving the canopy continuity tendon to the middle tendon spot may improve the issue. Consider adding additional longitudinal tendons in the added 2 ft. corner chamfers (Comment 4.c.i).

FIGG responded as follows:

> The tendons anchor at the edge of the member, thus a tension field cannot develop behind the compression zone in region 3 during stressing of the tendons. In region 3, the top slab and bottom slab are free to shorten independently. Any differential shortening in this region will result in minor bending moments in the vertical reinforced concrete member. These will be resisted with mild reinforcement in the conventional manner. Relative to this comment, no changes will be made that would alter the 90% foundation design plans.
>
> Relative to the canopy section above (left), the PT [bars] shown were provisional for purposes of various erection methods and sequences. As shown on sheet B-27, the Contractor has elected to CIP [cast in place] the span over the canal, after the precast span is in place. As a result, these PT bars will be eliminated.
>
> Relative to the floor section above (right), the local region bounded by the two "blue triangles" would receive minimal compression from the tendons of the CIP back span. The same location of the precast span includes tendons within this region (see Sheet B-11, Section B-B). During final design of the superstructure, the three tendons of the CIP (on each side of centerline) will be re-spaced to improve the distribution of stresses in this area. The chamfer item was previously addressed above.

Relative to these comments, no changes will be made that would alter the 90% foundation design plans.

In the final release for construction superstructure plans, additional longitudinal post-tensioning tendons in the bridge deck were incorporated that addressed FDOT's review comments about additional longitudinal tendons.

On September 16, 2016, FDOT had specifically commented to FIGG on the *main span truss system* layout (sheet B-36), as shown in figure 28: "Recommend chamfered end blocks to address shear lag at anchors (where the truss members connect to the canopy and bridge deck at the end of the bridge span where the longitudinal PT terminates)."

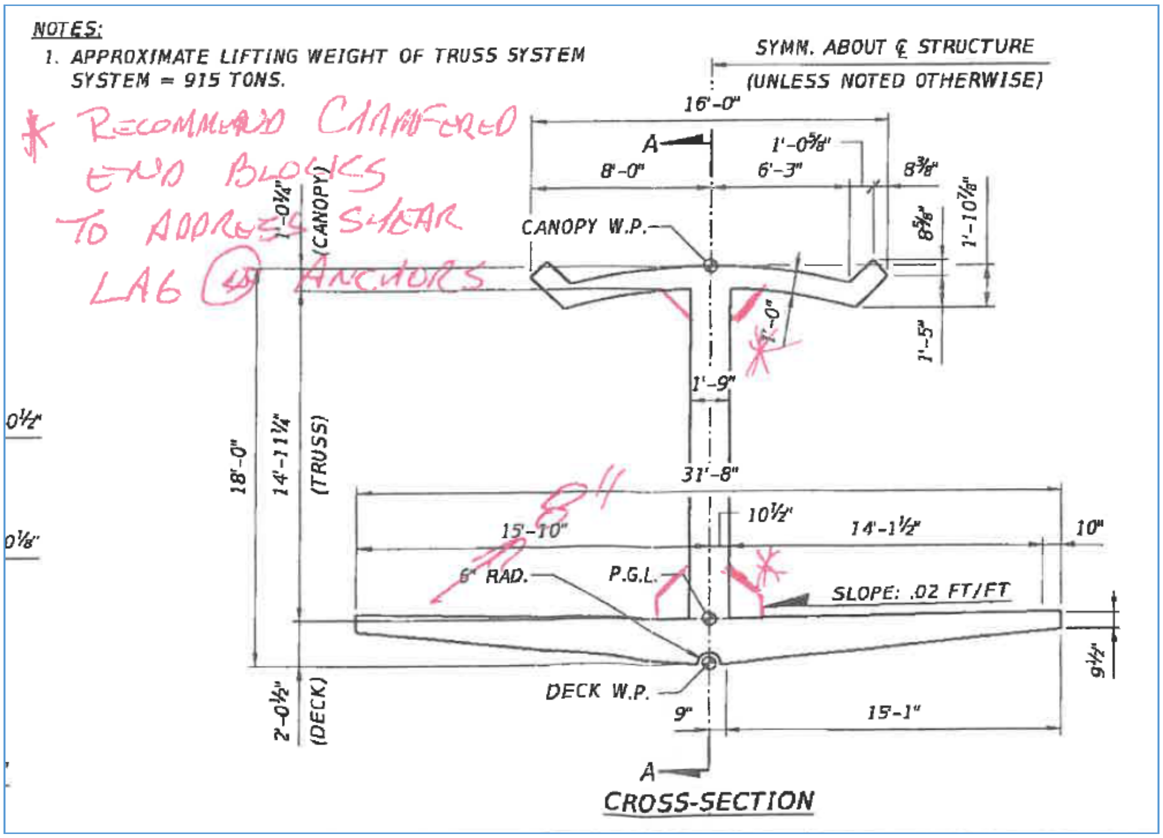

Figure 28. FDOT comments to FIGG on main span truss system layout, recommending chamfered end blocks to address shear lag at anchors. (Source: FDOT)

FIGG reviewed this FDOT recommendation and determined that the chamfered end blocks were not required by the design analysis and would pose a safety or tripping hazard to pedestrians. This FDOT recommendation was not incorporated into the 90 percent, final, or release for construction superstructure plan submittals and FDOT did not submit this as a formal review comment to be addressed during 90 percent plan submittal, final submittals or superstructure plan submittal reviews.

1.13 Postcollapse Actions by FDOT

On July 3, 2019, the governor of Florida signed into law an amendment to state statutes regarding transportation projects. Florida statute section 334.175 reads as follows:

> 334.175 Certification of project design plans and surveys.
> (1) All design plans and surveys prepared by or for the department shall be signed, sealed, and certified by the professional engineer or surveyor or architect or landscape architect in responsible charge of the project work. Such professional engineer, surveyor, architect, or landscape architect must be duly registered in this state.
>
> The text of the 2019 amendment to 334.175 reads as follows:
> (2) For portions of transportation projects on, under, or over a department-owned right-of-way, and regardless of funding source, the department shall review the project's design plans for compliance with departmental design standards.

As of this report date, FDOT is reviewing new draft language for the LAP manual in chapters 9, 19, 20, and 25, to include the following:

> **Chapter 9: Special Contracting Methods for Local Agency Program Projects**
>
> **9.2.7 Design-Build**
>
> For transportation projects on, under, or over an FDOT-owned right-of-way, Florida law requires the Department to review the project's design plans for compliance with FDOT design standards. Chapter 334.175 (2), F.S. In its sole discretion, the Department may reject designs which do not meet Department standards. The Department may also allocate Department-managed resources, including structures engineers and/or project managers to facilitate compliance with applicable design standards.
>
> **Chapter 19: Preliminary Engineering and Design**
>
> **19.1 Overview**
>
> For transportation projects on, under, or over an FDOT-owned right-of-way, Florida law requires the Department to review the project's design plans for compliance with FDOT design standards. Chapter 334.175 (2), F.S. In its sole discretion, the Department may reject designs which do not meet Department standards. The Department may also allocate Department-managed resources, including structures engineers and/or project managers to facilitate compliance with applicable design standards.
>
> **Chapter 20: Plans, Specifications, and Estimates**
>
> **20.1 Overview**
>
> For transportation projects on, under, or over an FDOT-owned right-of-way, Florida law requires the Department to review the project's design plans for

compliance with FDOT design standards. Chapter 334.175 (2), F.S. In its sole discretion, the Department may reject designs which do not meet Department standards. The Department may also allocate Department-managed resources, including structures engineers and/or project managers to facilitate compliance with applicable design standards.

Chapter 25: Maintenance

25.1 Overview

Questions involving road closures needed to prevent imminent risk to the health, safety and welfare of the travelling public must be immediately brought to the attention of appropriate project knowledgeable Department employees. The Department expects sound engineering judgment will be used on all aspects of LAP projects. Crack management and supervision of LAP project structures should be proactively managed, monitored and consistently inspected by an appropriately prequalified structures engineer. Any crack monitoring that involves the health, safety and welfare of the travelling public should be immediately brought to the attention of appropriate project knowledgeable Department employees.

1.14 FDOT Standard Specifications for Road and Bridge Construction

NTSB investigators examined the FDOT specifications related to the pedestrian bridge in effect at the time the FDOT and FIU agreement was signed. Relevant information from the FDOT *Standard Specifications for Road and Bridge Construction*, Construction Details (Division II), Structures, includes the following (FDOT 2015a; pages 377, 391, and 392):

400-9.3 Preparation of Surfaces

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.

400-21.2 Investigation, Documentation, and Monitoring

The Engineer will inspect concrete surfaces as soon as surfaces are fully visible after casting, with the exception of surfaces of precast concrete products produced in offsite plants, between 7 and 31 days after the component has been burdened with full dead load, and a minimum of 7 days after the bridge has been opened to full unrestricted traffic. The Engineer will measure the width, length and depth of each crack and establish the precise location of the crack termination points relative to permanent reference points on the member. The Engineer will determine if coring of the concrete is necessary when an accurate measurement of crack depth cannot be determined by use of a mechanical probe. The Engineer will monitor and document the growth of individual cracks at an inspection interval determined by the Engineer to determine if cracks are active or dormant after initial inspection. The Engineer will perform all final bridge deck crack measurements once the deck is free of all debris and before transverse grooves are cut and after planing is complete for decks that require planing.

400-21.3 Classification of Cracks

The Engineer will classify cracks as either nonstructural or structural and determine the cause. In general, nonstructural cracks are cracks 1/2 inch or less deep from the surface of the concrete; however, the Engineer may determine that a crack greater than 1/2 inch deep is nonstructural. In general, structural cracks are cracks that extend deeper than 1/2 inch. As an exception, all cracks in concrete bridge decks that are supported by beams or girders will be classified as nonstructural and repair will be in accordance with 400-21.5.1. However, if the Engineer determines that repair under 400-21.5.1 is unacceptable, repair in accordance with 400-21.5.2.

400-21.5.2 Structural Cracks

Provide a structural evaluation signed and sealed by the Contractor's Engineer of Record that includes recommended repair methods and a determination of structural capacity and durability to the Engineer. Upon approval by the Engineer, repair the cracked concrete. Complete all repairs to cracks in a member inside a cofferdam prior to flooding the cofferdam.

2 Analysis

2.1 Introduction

This analysis of the March 15, 2018, collapse of the FIU pedestrian bridge focuses on the performance of the northernmost nodal region (11/12 node) of the 174-foot-long main span (section 2.2). The failure of this nodal region was the triggering event for the bridge collapse. The analysis also addresses the following safety issues:

- Bridge design and construction plan errors (section 2.3).
- Unique bridge characteristics and mechanisms of failure (section 2.4).
- Independent peer review of complex bridge design (section 2.5).
- Shortcomings in oversight of evaluation of and response to significant observed bridge structure distress prior to collapse (section 2.6).
- Lack of redundancy guidelines in specifications for pedestrian and concrete truss bridges (section 2.7).

As a result of its investigation, the NTSB established that the allocation of emergency response resources was adequate, and emergency responders were immediately dispatched. The initial 911 call was received at 1:47 p.m., and first responders were on scene within 5 minutes (by 1:52 p.m.). The NTSB concludes that the emergency response by local fire departments and law enforcement personnel was timely and adequate.

TFHRC's testing of postcollapse concrete samples confirmed that the compressive strength and air content of the concrete met project requirements. The samples were observed to be well consolidated and did not exhibit any segregation, and no honeycombing or competency issues were identified. The reinforcing steel bars met the minimum yield strength, tensile strength, and percent elongation at fracture for their respective sizes. Specific mechanical properties were defined for the PT rods, and their tensile strength and elongation at fracture were observed to exceed the specified minimum values.

In summary, the concrete material properties, the reinforcing steel material properties, and the steel PT rod mechanical properties were found to meet or exceed the requirements/values specified in the FIGG bridge design. Therefore, the NTSB concludes that the concrete and steel materials used during construction of the pedestrian bridge were not a factor in its collapse.

The construction process for the bridge required the use of hydraulic jacks to apply tensioning forces to steel rods embedded in the structure. At the time of the bridge collapse, a construction action to post-tension the rods in member 11 had just been completed. TFHRC's performance assessment determined that the jack and associated equipment had operated correctly, including having the capability to apply the full range of loads and doing so in accordance with the specified calibration. Therefore, the NTSB concludes that the hydraulic jack used to post-tension the steel rods in member 11 was operating as expected at the time of the bridge collapse.

2.2 Collapse of Main Span

Section 2.2 provides an overview of the documentation of the concrete cracking and the sequence of failures that led to the destruction of the connection between member 12 and the bridge deck. Using available video recordings and photographic evidence of precollapse concrete fractures and structural distress, the NTSB documented the sequence of events in the collapse of the pedestrian bridge.[83] The FHWA provided technical expertise and resources to evaluate the bridge design and construction processes.

The bridge collapse was preceded by a series of events and documented observations that demonstrated concrete distress in the form of cracking. Construction documentation chronicled the structural performance of the bridge from early stages through the March 15 collapse. The NTSB analysis focused on the truss member 11/12 nodal region, where significant distress cracking was observed prior to the collapse and from which the collapse initiated.[84]

The bridge main span collapsed when the demand placed on the member 11/12 nodal region exceeded the resistance provided by the structure. As the rebars crossing the horizontal and vertical shear planes (of concrete) in the nodal region were pushed beyond their limits, the loads in member 11 were increasingly resisted by member 12, acting as a buttress. This action engaged the vertical reinforcement lap splice in member 12 and, thus, the confinement reinforcement at the lap splice, which failed, leading to the unraveling of the connection between member 12 and the bridge deck (refer to figure 17). An in-vehicle video of the collapse captured the concrete blowout at this connection. Truss members 11 and 12 then translated northward, unimpeded. The video captured the sequence of structural failures caused by the northward movement of members 11 and 12, as their failure caused a total loss of stability, and the bridge span collapsed off the support piers and onto SW 8th Street.

2.3 Bridge Design and Construction Plan Errors

Section 2.3 discusses the design of the nodal regions, interface shear design calculations, analytical models, and safety recommendations related to design and construction plan errors.

2.3.1 Design of Bridge Nodal Regions

As discussed in section 2.1, the NTSB excluded the material and mechanical properties of the concrete—and the steel reinforcing bars and PT rods and equipment—as factors in the bridge collapse. The progression of cracking, with active dislodgment of node 11/12 from the deck, indicated the distress of the structure in the days preceding the collapse and the inability of the bridge to resist interface shear demand at this critical location. Therefore, investigators considered

[83] Concrete distress is the physical manifestation of deterioration that is apparent on or within a structure, including cracking, delamination, or spalling. “Distress” is not to be confused with destress, which is synonymous with detension.

[84] Similar but lesser distress was observed in the member 1/2 nodal region during the weeks leading to the collapse. However, this nodal region did not progress to failure, and it is not the focus of this report.

whether the collapse was due to errors in design consistent with an underestimation of demand or with an overestimation of capacity in node region 11/12 or in the construction of the bridge.

Designers of a complex and unique bridge should always document design approach, analysis, methodology, and key assumptions for all critical construction stages. However, the level of detail included in the FIGG bridge design varied for each construction stage—with the fully completed two-span superstructure having the most thorough documentation and the simply supported condition (its state of collapse) having the least detail.

The insufficient level of detail in the FIGG design calculations resulted in limited information about the design analysis and how it generated the forces (demands) on the under-designed bridge members. Although the NTSB investigation determined that the FIGG analysis contained design errors in the demand forces required for the bridge, without sufficient detail from FIGG, the FHWA postcollapse check could not definitively identify the specific source of the calculated demand errors. Therefore, NTSB investigators developed independent analytical models to determine the design loads that would have been imposed on this node and compared them with the AASHTO-specified limits for concrete and steel reinforcement materials (capacity) (AASHTO 2015).

A nonredundant structure is one with fewer load paths than necessary to maintain stability following the failure of one or more critical components, likely resulting in collapse of the structure. With the single centerline of truss members in the pedestrian bridge design, it would be considered a nonredundant structure.

When designing a structure, it is important to consider uncertainties in the loads the structure will need to support, as well as in the materials that will be used to build the structure. Load and resistance factor design (LRFD) is used to account for these uncertainties. Many types of loads (dead load, live load, wind load, earthquake load, etc.) can occur simultaneously. “Load factors” are applied to the loads or combination of loads to account for these uncertainties. Uncertainties in building materials can occur due to variation in material properties, residual stress in the materials from the fabrication process, fabricated sizes being different from intended measurements, and corrosion or decay of the materials. “Resistance factors” are applied when calculating the ultimate strength of critical sections to account for the uncertainties in the building materials. Using load factors and resistance factors together ensures that a factor of safety is applied to the design of the structure.

The *AASHTO LRFD Bridge Design Specifications*, article 1.3.2.1, applies a redundancy factor to the computed force effects for all limit states. The redundancy factor, $\boldsymbol{\eta_R}$, must be 1.05 or greater for nonredundant members and 0.95 or greater for bridges with an exceptional amount of redundancy. Structures with conventional levels of redundancy have an $\boldsymbol{\eta_R}$ equal to 1.00. Therefore, the redundancy factor can either increase or decrease the demand on a bridge structure. As FIGG inappropriately determined that its design was redundant, it used a redundancy factor of 1.00 (for conventional levels of redundancy). The FHWA concluded that an $\boldsymbol{\eta_R}$ equal to or greater than 1.05 was required for the pedestrian bridge structure, due to the singular load path provided by the single-line truss.[85] The NTSB concludes that (1) the FIGG bridge design was nonredundant because it provided only a singular load path, (2) FIGG used poor judgment when it determined that the bridge was a redundant structure, and then, (3) FIGG erroneously used a redundancy factor of 1.00, which is commonly used for structures with redundant load paths.

2.3.2 FIGG Interface Shear Calculations

Casting a concrete element onto an existing concrete element creates a cold joint. The main span superstructure of the bridge was built offsite in three concrete casting phases, which resulted in cold joints at each end of every truss member: one at the bottom of the member (deck-to-member interface) and the other at the top of the member (member-to-canopy interface).[86] Both interface surfaces of each member were designed to transfer forces through interface shear.

The mechanics of interface shear can transfer forces across a cold joint. The surface roughness (friction), cohesion (chemical bond) across the contact area, and magnitude of the force compressing the two surfaces together provide resistance. This compressive force is related to both the amount of reinforcing steel (rebar) crossing the interface surface and the amount of permanent loading whose line of action is perpendicular to the interface. Occasionally, supplemental compressive forces supplied through other means (such as post-tensioning) might be included.

2.3.2.1 Surface Roughness Contribution to Interface Shear Capacity. NTSB investigators determined that the construction plans inconsistently referenced the surface preparation for cold joints. Multiple drawings in the FIGG plans deliberately noted when cold joints needed to be roughened to a 0.25-inch amplitude to be consistent with the assumptions in the design calculations.[87] However, the construction drawings for the truss nodal regions did not include a note to roughen the surface to a 0.25-inch amplitude or any other specific note pertaining to the surface preparation of interfaces in these areas—in particular, the member 11/12 nodal region—which was inconsistent with the assumptions made in the FIGG design.[88] FIGG calculated the capacity for nodal regions 1/2 and 11/12 at values approximately 20 percent larger than the FHWA postcollapse check calculations for interface shear—which means that the FIGG calculation values overestimated the capacity that was available to resist interface shear and safely support the bridge. However, the NTSB concludes that, even if the cold joint surface of nodal region 11/12 had been roughened to a 0.25-inch amplitude, node 11/12 would not have had sufficient capacity to counteract the demand load for interface shear—and the bridge would still have been under-designed and could have failed. The NTSB further concludes that the FIGG construction

---

[85] However, to provide a clear comparison to the FIGG design, the FHWA check did not use the recommended increased redundancy factor.

[86] The deck was cast and allowed to harden; truss members 1 through 12 were cast atop the deck and allowed to harden; and the canopy concrete was cast atop the truss members.

[87] For pylon diaphragm construction joints, FIGG specified a 0.25-inch amplitude for surface roughening, but it did not include this requirement for the construction joints between the truss members and the deck and canopy.

[88] As stated previously, the interface shear transfer area, A_{cv}, and the reinforcing steel area crossing the interface surface, A_{vf}, were taken from information included in the FIGG plans. The A_{vf} values vary slightly from what was assumed in the FIGG design for nodal regions 1/2, 2/3, and 10/11.

plans inconsistently identified when intentionally roughened surfaces were needed to fulfill the assumptions of the bridge design.

2.3.2.2 Clamping Force Across Interface Shear Zone. Clamping forces increase the effective friction or resistance to sliding. The *AASHTO LRFD* groups multiple types of individual loads into load combinations (also referred to as "limit states"; AASHTO 2015). These combinations are then either increased or decreased to produce the most demanding singular total factored force effect for the element being designed. Load combinations and factors are prescribed in the *AASHTO LRFD*. The FIGG design appropriately included the "Strength I" limit state to generate interface shear demand and obtain the applied force effects (demand) on the truss member nodal regions, with the load factors as follows:

- Dead load of structural components and nonstructural attachments (DC) = 1.25.
- Pedestrian live load (PL) = 1.75.
- Primary post-tensioning force (PT) = 1.0.
- Force effect due to uniform temperature (TU) = 0.5.

FIGG also used the *AASHTO LRFD* specifications for interface shear transfer–shear friction for connection of the truss members to the bridge deck and canopy. The AASHTO governing equation to determine connection capacity, referred to as nominal capacity, states that the permanent net compression, $\boldsymbol{P_c}$, is beneficial to developing interface shear capacity.[89]

The *AASHTO LRFD* states that the load factor on DC (dead load of structural components and nonstructural attachments) can be taken as either 1.25 (to generate a maximum) or 0.90 (to generate a minimum). FIGG used a load factor of 1.25 in its $\boldsymbol{P_c}$ calculation. However, the FHWA strongly recommends the 0.90 factor for dead load when determining a minimized interface shear capacity value for purposes of design.

Typically, a bridge designer would artificially increase the weight of the bridge (through factoring or choosing the correct load factors) so that the designed bridge would be capable of supporting more weight than it really carries. A designer would then also artificially reduce (again through factoring) the capacity of the bridge, which would result in calculations showing that the designed bridge would be able to hold up less weight than it really could. By making these adjustments together (heavier weight and lower ability to hold the weight) in the design, the designer would have provided a safety margin for the actual bridge design, which is considered a factor of safety. By improperly using a load-multiplying factor of 1.25 in the $\boldsymbol{P_c}$ calculation, FIGG effectively increased, and thereby overestimated, the bridge's interface shear capacity by approximately 25 percent. Had FIGG instead correctly used a load factor of 0.90, this would have properly reduced the interface shear capacity by approximately 10 percent.

[89] Specifically, the *AASHTO LRFD* (2015) states, "Where the permanent load increases the stability or load-carrying capacity of a component or bridge, the minimum value of the load factor for that permanent load shall also be investigated . . . both extreme combinations may need to be investigated by applying either the high or the low load factor as appropriate."

The horizontal force component of member 11 acted as the shearing force on the interface shear surface (deck) and was the interface shear demand. The vertical force component of member 11 acted as the compressive, or clamping, force that contributed to interface shear resistance (resisting member 11 from shearing or sliding horizontally off the deck). (See figure 29.) When FIGG made the error of using the incorrect $\boldsymbol{P_c}$ factor in its calculation, this artificially increased the perceived clamping forces available. The artificial increase of weight that resulted from FIGG's incorrect use of a 1.25 load factor (upper bound) led to a determination that there was more clamping force available for resistance to interface shear than actually was available, which resulted in overestimation of interface shear resistance. This means that the actual interface shear resistance was considerably less than FIGG calculated, and the interface shear resistance of member 11 was therefore unable to resist the shear forces, leading to failure at this critical location, causing the bridge collapse.

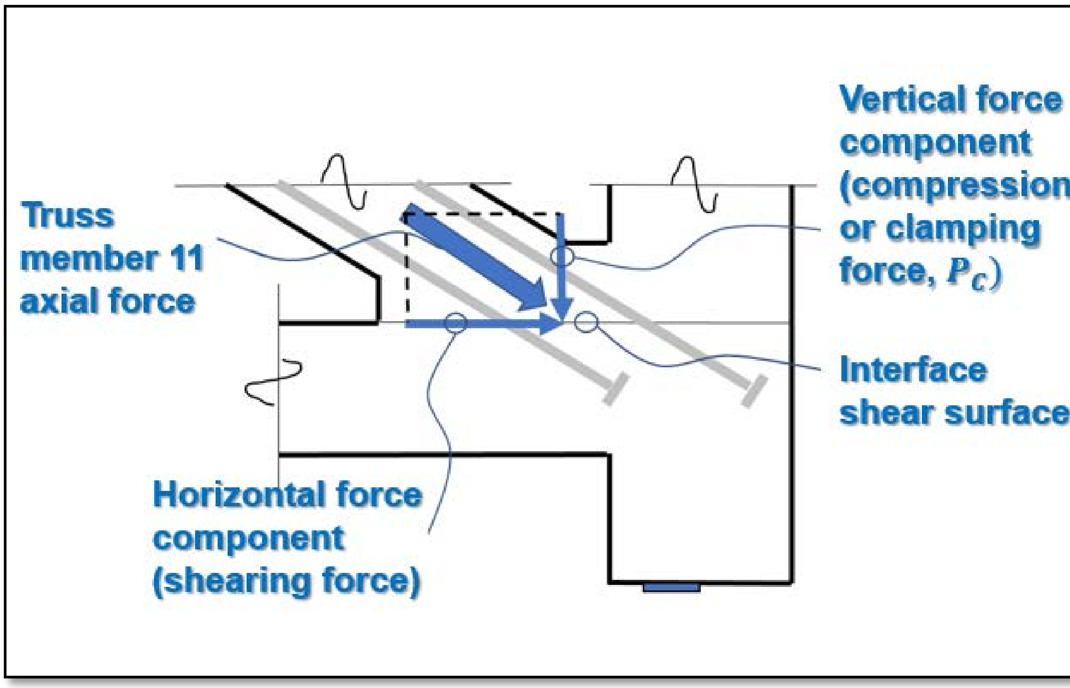

Figure 29. Depiction of axial force from truss member 11 on nodal region interface shear surface. (Source: FHWA 2019)

The *AASHTO LRFD* sets upper limits on the capacity generated from its design calculations, and the least of three nominal capacity determinations is the controlling nominal interface shear resistance. In addition, the *AASHTO LRFD* provisions require that the calculated nominal resistance be further reduced by multiplying it by a resistance factor, which is always equal to or less than 1.0. However, it is significant that for the types of construction materials used in the pedestrian bridge, the *AASHTO LRFD* requires that the nominal resistance be equal to 0.90 for interface shear. This reduced capacity value is referred to as the "factored interface shear resistance." The *AASHTO LRFD* requires that this value be greater than the total factored force effect generated from the governing limit state (Strength I for this design).

Postcollapse, the FHWA identified two consistent errors by FIGG in its interface shear computations for the bridge design. The *AASHTO LRFD,* article 5.8.4.1, states that the value for

$\boldsymbol{P_c}$ used in the capacity calculation must consider only "permanent net compressive force normal to the shear plane." The FIGG design value for $\boldsymbol{P_c}$ considered the compressive forces from both permanent (such as the dead load of the structure) and nonpermanent, or transient, loadings (such as pedestrian live load). In addition, the compressive forces from loading were inappropriately amplified by the Strength I maximum load factors. Therefore, the NTSB concludes that because FIGG (1) did not use the lower bound load factor for determining the governing net compression, $\boldsymbol{P_c}$, in the interface shear; and (2) incorrectly increased and amplified the effects of the clamping force across the interface shear surface, its bridge design calculations resulted in a significant overestimation of capacity. Therefore, the NTSB recommends that FIGG train its staff on the proper use of $\boldsymbol{P_c}$ (the permanent net compressive force normal to the shear plane) when calculating nominal interface shear resistance.

The *AASHTO LRFD* states that bridges must be designed to achieve the objectives of safety, constructability, and serviceability (AASHTO 2015). These objectives are met through the theory of reliability based on current statistical knowledge of loads and structural performance. In *LRFD* design, the anticipated loads on a bridge are conservatively estimated, and the structural system is proportioned to reliably resist those loads. The NTSB concludes that FIGG (1) made significant design errors in the determination of loads, leading to a severe underestimation of the demands placed on critical portions of the pedestrian bridge; and (2) significantly overestimated the capacity of the member 1/2 and 11/12 nodal regions.

2.3.3 FIGG Analytical Models

2.3.3.1 Model Characteristics and Construction Stage Demands. The pedestrian bridge was constructed in multiple stages (as described in sections 1.6.1 and 1.6.2), with each stage generating unique forces on the superstructure and component structures. It was critical that the bridge design consider the forces generated during each construction stage.

The FIGG bridge design used four analytical models to evaluate three critical construction stages: the transport of the main span, the main span in a simply supported condition (supported at the ends, without falsework support in between), and the completed bridge.[90] The two-dimensional LARSA (main span erection and fully completed bridge structure [longitudinal]) and three-dimensional LUSAS (three-dimensional simple support and fixed pylon) were the four models used, as described in section 1.8.[91]

FIGG used the longitudinal and fixed pylon models to generate forces on the *completed* two-span bridge. The structural components included in these two models were different:

[90] The completed bridge structure includes the main span, back span, and all three end bents with piers (south, pylon, and north).

[91] Each analytical model generated multiple force effects for each structural component. The truss member axial forces were extracted from these analyses for use in design of the truss member connections. The axial force in the truss member was subsequently resolved into vertical and horizontal components, as shown in figure 29. As noted earlier, the vertical component is the compressive, or clamping, force that contributes to interface shear resistance. The horizontal component is the shearing force on the interface shear surface, or the interface shear demand.

- The longitudinal LARSA model generated forces for every component in the bridge superstructure.
- The fixed pylon LUSAS model generated forces only for components located in the main span.[92]

These axial forces were extracted from the analytical models of the force transfer across the shear interface surface in the nodal regions, as shown in figure 29 above.

As discussed in section 1.8.1, FIGG identified the most critical forces between the simple support and fixed pylon (LUSAS) models; however, figure 27 results reflect only the identified maximums between the two models. The interface shear forces shown were generated from the load combination and load factors prescribed in the *AASHTO LRFD* Strength I limit state, which was used for the interface shear design of all truss cold joints in the nodal regions. The simple support model generated the governing interface shear force effects for every nodal region except 7/8 and 11/12, for which the fixed pylon model was used. FIGG did not include results from the main span erection model or the longitudinal model for comparison of interface shear demands to determine maximum force effects.[93]

Figure 27 illustrates that the FIGG simple support and fixed pylon (LUSAS) models of the main span (when it was in a simply supported condition between the south pier and the pylon pier) produced results of less than 1,000-kip shear demand at nodal region 11/12. In direct contrast, the longitudinal, fully completed LARSA model produced results of 2,000-kip demand at nodal region 11/12. The forces generated from the longitudinal model (for the completed bridge structure) were the largest for nodal regions at the north and south ends of the main span.

As previously described, the bridge was constructed in multiple stages, and each construction stage generated unique forces on the bridge structure. This means that the bridge's structural components had to be designed to withstand the largest of the forces generated from each of these stages. Specifically, each stage resulted in significantly different interface shear demands generated on the bridge structure. Both the magnitude of the interface shear demand and the concurrent permanent net compression force ($\boldsymbol{P_c}$) needed to be considered in the design for each stage to determine which governing load case should be used for each nodal region. The largest interface shear demand calculated did not necessarily coincide with the governing load case for each nodal region. The FIGG design included only the results from the simple support and fixed pylon modeling combination. After analyzing the models, investigators found that, to properly account for the unique forces on the bridge structure (and to prevent structural failure) at each construction stage, FIGG should have determined the governing interface shear demand for each nodal region as follows:

- Main-span erection: governed for nodal regions 4/5, 6/7, 7/8, and 8/9.
- Simple support: governed for nodal regions 3/4 and 5/6.

[92] This was adjusted to replicate the main span performance in its completed condition.

[93] The FIGG design did not include analysis results from the simple support and fixed pylon models for nodal regions 4/5, 6/7, and 8/9.

- Fixed pylon: did not govern for any nodal regions.
- Longitudinal model: governed for nodal regions 1/2, 2/3, 9/10, 10/11, and 11/12.

Therefore, the NTSB concludes that, based on analytical modeling results, FIGG should have considered the loadings from all critical construction stages when designing the pedestrian bridge and determining the governing interface shear demands.

2.3.3.2 Analysis of Model Results. Postcollapse, to analyze the FIGG models, the FHWA completed four separate structural analyses of the bridge during the construction stage when the collapse occurred (which is considered the simple support position, when the bridge was supported on the piers with truss members 2 and 11 post-tensioned).[94] As shown in table 6, the models generated results showing the interface shear demands (in magnitude only), which—when compared—were found to have very good agreement.

Table 6. FHWA interface shear demand analytical model results for *AASHTO LRFD* Strength I limit state. (Source: FHWA)

Nodal Region	Model 1 (kips)[a]	Model 2 (kips)[a]	Model 3 (kips)[b]	Model 4 (kips)[c]
1/2	2,481	2,476	2,549	2,544
2/3	2,575	2,570	2,589	2,660
3/4	1,090	1,081	1,124	1,061
4/5	977	960	963	940
5/6	628	625	586	624
6/7	519	515	506	515
7/8	338	331	337	381
8/9	19	12	37	61
9/10	80	82	89	34
10/11	1,491	1,504	1,421	1,254
11/12	1,835	1,846	1,806	1,816

[a] Models 1 and 2: two-dimensional grid analyses of beam elements with post-tensioning and nonlinear time-dependent material effects considered.

[b] Model 3: two-dimensional grid analysis of truss elements with post-tensioning effects superimposed.

[c] Model 4: three-dimensional finite element model of solid elements with post-tensioning sequence considered.

Figure 30 compares the interface shear demands generated by the FIGG analytical model (combined simple support and fixed pylon used in FIGG's design calculation) with those generated from the FHWA models. As a reminder, the failure of nodal region 11/12 initiated the collapse. The differences in the model outcomes are shown below. The FHWA models generated significantly higher interface shear demand for the nodal regions at the ends of truss members 2

[94] Each model was developed by one of four FHWA staff members using a different software package.

(regions 1/2 and 2/3) and 11 (regions 10/11 and 11/12), when compared with the FIGG model results. (See appendix F for more details on the differences in the models.)

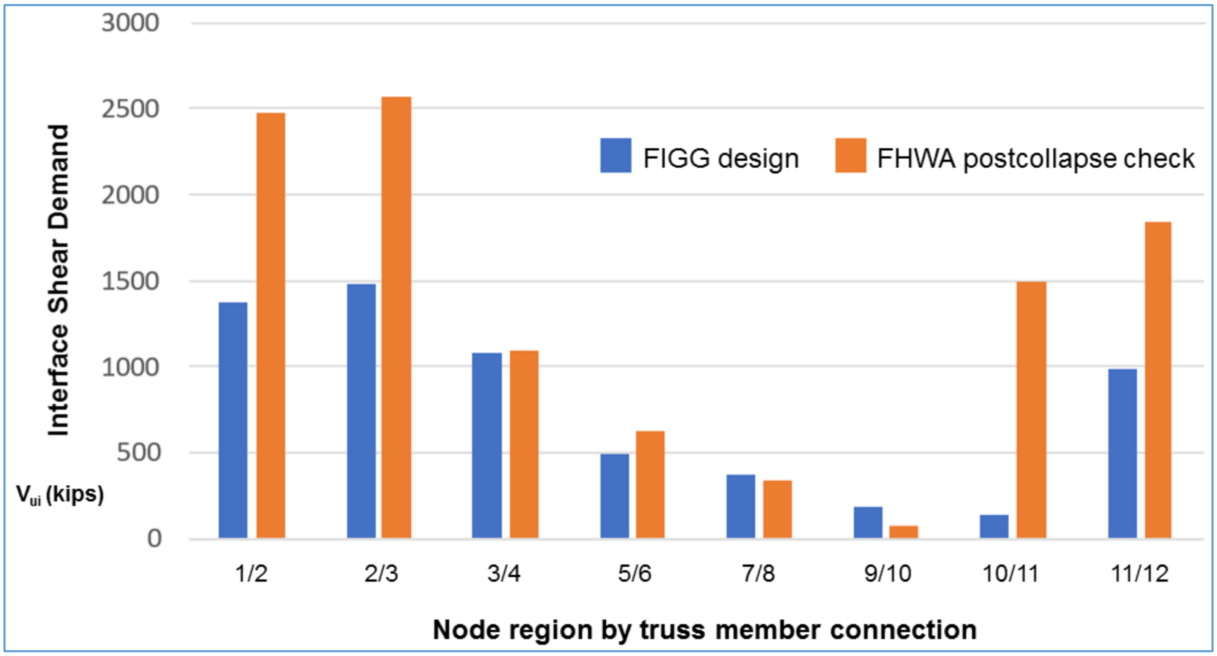

Figure 30. Comparison of nodal region horizontal shear modeling results from FIGG design and FHWA analysis. (Source: FHWA 2019)

FIGG's use of only the simple support and fixed pylon model outcomes resulted in the underestimation of interface shear demand. According to the FHWA models, in maximum force effect, FIGG underestimated interface shear demand by the following approximate numbers:

- Nodal region 1/2: Underestimated by 1,100 kips (or 49 percent).
- Nodal region 2/3: Underestimated by 1,100 kips (or 43 percent).
- Nodal region 10/11: Underestimated by 1,360 kips (or 91 percent).
- Nodal region 11/12: Underestimated by 850 kips (or 46 percent).

For the highly loaded nodal regions connecting truss members 2 and 11 to the deck and canopy, the FHWA analytical models generated interface shear demands lower than those from the FIGG longitudinal model. The NTSB investigation did not assess the accuracy of the FIGG longitudinal model results, because it was clear that FIGG—during its design process—had available model results with nodal region demands (longitudinal) that exceeded the simple support fixed pylon model demands (equaling those acting on the bridge at the time of collapse): however, FIGG used poor engineering judgment and instead chose not to use the higher demand model results (as discussed above).

FIGG did not provide a rationale for the engineering judgment it used when selecting modeling results, which led to consistent underestimation of the interface shear demand across many of the main span truss member nodal region cold joints, even though some of its modeling results produced reasonable estimations for interface shear demand. The NTSB concludes that, in several instances throughout the bridge design process, FIGG models produced reasonable estimations for interface shear demand, but these values were not always used in the design of truss members to resist force demands. For reasons the NTSB could not determine, FIGG ignored the appropriate and reasonable interface shear demand values when sizing the main span truss members to resist demand forces. The NTSB concludes that FIGG's analytical modeling for the bridge design resulted in a significant underestimation of demand at critical and highly loaded nodal regions.

The interface shear transfer area, $\boldsymbol{A_{cv}}$, is the area of surface concrete considered to be engaged and providing interface shear capacity. This area typically encompasses the entire footprint between the connecting elements. However, $\boldsymbol{A_{cv}}$ needs to consider how the interface shear transfer area is affected when it is located along an edge or includes discontinuities. The interface areas located within the nodal region footprint and close to the deck edge had limited capacity to transfer loads to the body of the deck.

Longitudinal post-tensioning, located within the deck, provided the primary mechanism to resist the longitudinal forces delivered by the truss members to the deck. The position of the applied forces from the truss members relative to the post-tensioning location is important. PT tendons can only resist applied forces within their region of influence.

As described previously, interface shear capacity is generated through multiple resistance mechanisms. Resistance is provided by cohesion across the contact area of the two concrete elements and by the roughness between the surfaces. The normal force coinciding with the roughness is a function of the amount of reinforcing steel crossing the interface surface and the amount of permanent compressive loading perpendicular to the interface. Occasionally, supplemental compressive forces supplied through other means (such as post-tensioning) are included in determining the permanent compressive force.

As discussed above, the FIGG design incorrectly calculated the $\boldsymbol{P_c}$ value, which is also used in the design calculation for the interface shear reinforcement area ($\boldsymbol{A_{vf}}$). FIGG's error in calculating the available interface shear capacity resulted in its designing a significantly under-reinforced connection, meaning that insufficient steel rebar was embedded in the concrete between the base of member 11 and the deck. Had FIGG used a more appropriate clamping force calculation (for interface shear capacity), the steel reinforcing bars to be installed between member 11 and the deck would have required a larger interface shear reinforcement area ($\boldsymbol{A_{vf}}$). The FIGG design provided for only eight size 7 rebars (or 4.8 square inches cross-sectional area of reinforcing steel) because FIGG calculations for the required rebar (amount and size) were based on the simple support model (LUSAS), which led to an undersized interface shear reinforcement area ($\boldsymbol{A_{vf}}$). Based on the correct calculation of $\boldsymbol{P_c}$ and demand model, as shown by the FHWA postcollapse check, an additional 13 square inches cross-sectional area of reinforcing steel in the interface shear reinforcement area ($\boldsymbol{A_{vf}}$) should have been provided.[95] Had FIGG used the

[95] There is no single unique solution to the calculation in this simplified example, as the interface area would most likely have to be increased to accommodate the additional reinforcing. However, this example is provided to show a general order of magnitude and to illustrate the significance of the error in the FIGG design.

longitudinal model (LARSA), the calculations would have required a larger cross-sectional area of reinforcing steel at the 11/12 node, leading to a larger interface shear reinforcement area (***A_{vf}***).

Based on FHWA postcollapse check calculations, figure 31 displays ratios in which the maximum interface shear demand (force) acting on a nodal region is divided by the capacity of the nodal region to produce the demand-to-capacity (D/C) ratio. A D/C ratio of less than or equal to 1.0 represents an assessment that the capacity can adequately carry the applied loading (as prescribed by design requirements). The lower the value, the more conservative the design. A D/C value above 1.0 represents an assessment that the capacity provided by the nodal region is not sufficient to carry the applied loading (as prescribed by design requirements). The higher the value, the more under-designed the connection. Figure 31 includes D/C ratios for each nodal region in the main span at the time of collapse.[96]

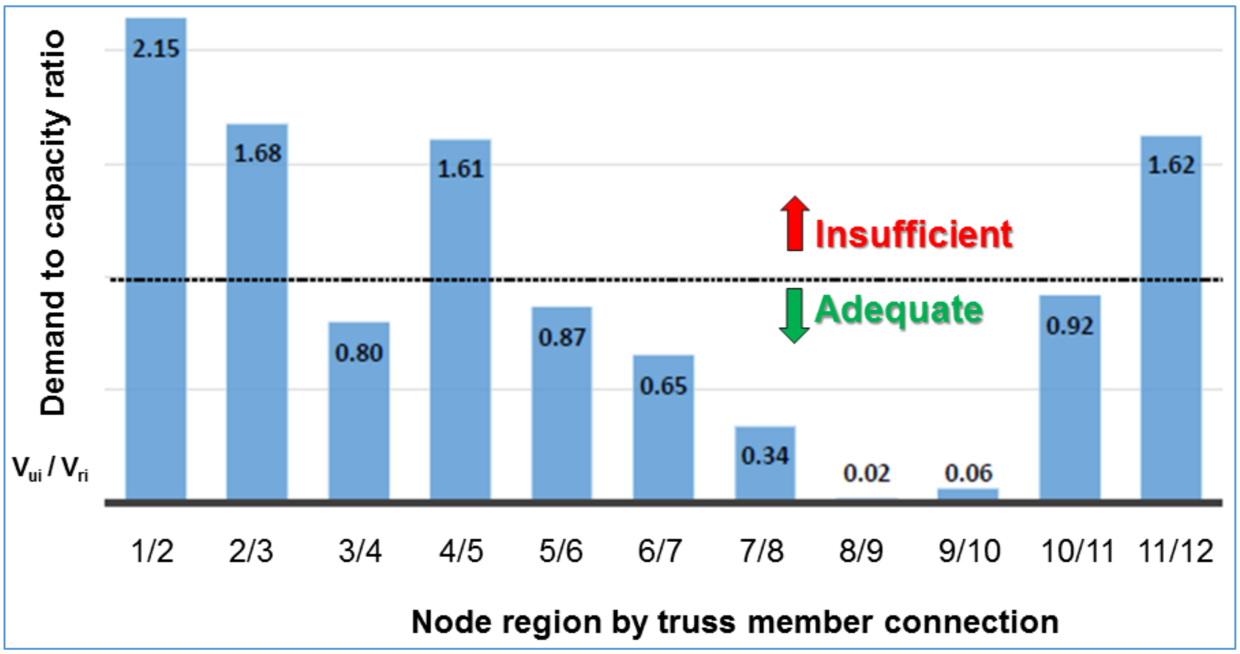

Figure 31. D/C ratios for main span nodal regions. (Source: FHWA 2019)

Four of the main span nodal regions have D/C ratios above 1.0. The member 1/2 nodal region had the highest D/C ratio, indicating that it was the least sufficient to carry the applied load. However, the bridge failed at the member 11/12 nodal region. For example, nodal region 11/12 was framed into a 24-inch-wide diaphragm (also penetrated by four vertical pipe sleeves), while node region 1/2 was framed into a 42-inch-wide diaphragm. In addition, truss member 11 measured one-third less deep than truss member 2 (24 inches deep versus 36 inches deep). Therefore, a more substantial concrete diaphragm provided nodal region 1/2 with an unintended capacity well above that for truss members 11 and 12. The NTSB concludes that the concrete

[96] The maximum demands used to compute the D/C ratios in figure 31 are based on table 6. The capacity is calculated using the interface shear transfer area, ***A_{cv}***, and the cross-sectional area of reinforcing steel crossing the interface plane, ***A_{vf}*,** based on the interface shear capacity results from the FHWA assessment (see appendix F). Cohesion is not included in these capacity calculations. In addition, the compressive force, ***P_c***, acting on each interface surface is the concurrent load generated by the FHWA assessment. The compressive force used for nodal regions 1/2 and 11/12 includes the temporary post-tensioning in truss members 2 and 11.

distress initially observed in nodal region 11/12 is consistent with the underestimation of interface shear demand and the overestimation of identified capacity in the bridge design.

FIGG made significant errors in its determination of loads, severely underestimated the demands placed on critical portions of the bridge, and disregarded the results of its own analytical models' results for appropriate and reasonable interface shear demand values when sizing the main span truss members to resist demand forces. To counteract such failings, the NTSB considers that AASHTO, as a leader in setting technical standards for highway bridge design and construction, could provide additional guidance to bridge designers on concrete bridge structures. Therefore, the NTSB recommends that AASHTO work with the FHWA to develop a requirement that concrete bridge structures be designed with reasonable estimates for interface shear demand, the cohesion and friction contributions to interface shear capacity, and the clamping force across the interface shear surface. Correspondingly, the NTSB recommends that the FHWA assist AASHTO with developing a requirement that concrete bridge structures be designed with reasonable estimates for interface shear demand, the cohesion and friction contributions to interface shear capacity, and the clamping force across the interface shear surface.

2.4 Unique Bridge Characteristics and Mechanisms of Failure

Because the pedestrian bridge was a simple span structure and a single plane truss, no alternate load paths bypassed the cold joint where the member 11/12 nodal region connected to the deck, which was a location subjected to significant shear and axial stresses. Section 2.4 describes the initial indicators of structural distress coinciding with the member 11 and 12 cold joints; analyzes the construction of the member 11/12 nodal region, including the significance of the hollow pipes within the concrete; describes additional structural distress during transport of the main span, including unanticipated variations in shear demand; and analyzes the reapplication of post-tensioning forces in member 11, which increased the demand on the member 11/12 nodal region.

2.4.1 First Structural Distress at Falsework Removal Stage

The main span concrete, including the member 11/12 nodal region, was cast alongside SW 8th Street during the ABC construction process and supported by temporary falsework. The concrete for the deck was cast first and allowed to cure. Members 11 and 12 were cast thereafter, with the fresh concrete from the second pour being cast against the hardened concrete from the first pour. Postaccident, NTSB investigators extensively studied the properties of the cold joint at the intersection of the deck and member 11/12. The hardened surface of the first concrete placement was found not to have an intentionally roughened surface to a 0.25-inch amplitude. A portion of the failure surface under member 11 was found to coincide with this cold joint.

Before moving the main span onto the piers, the falsework that provided continuous support along the length of the span was removed sequentially. As this task was completed, the structure's dead load progressively transferred to the bridge's structural load-carrying elements—and to the temporary support megashores under each end of the main span—until the bridge was fully self-supporting.

During this process, on February 24, a loud, distinct concrete cracking noise was heard, and a crack was found in the member 11/12 nodal region near and at the intersection of truss member 11 with the deck. Section 1.7 discusses the location and path of the crack. Per the FIGG construction plans, eight size 7 rebars and two size 6 rebars were to cross the cold joint under member 11. Although all 10 rebars were present in the structure, the crack passed above the southernmost two size 7 rebars. (See figure 17 inset for the locations of these two size 7 rebars.) This circumstance is attributable to the FIGG design containing a detailing scheme wherein the southernmost two rebars were not anchored on both sides of the critical shear plane at the base of member 11. A portion of the crack bypassed 25 percent of the reinforcing steel area that was, per the FIGG design, intended to offer interface shear resistance at the base of member 11. As a result, the crack did not engage those two reinforcing bars, which would have offered additional resistance to the member 11 northward-thrusting shear demand. The NTSB concludes that the FIGG design of the rebar placement in node 11/12 resulted in less reinforcing steel being available and diminished resistance to the critical interface shear demand, which contributed to the collapse of the bridge.

The location of the crack and the time the crack first appeared—when the falsework was removed and the bridge span was supporting its own weight under dead load application for the first time—are evidence that the demand from the main span dead load exceeded the strength of the concrete at the member 11/12 nodal region.

The opening of a tensile crack at the bottom of member 11 on February 24, combined with the limited cohesion across the cold joint because of the unroughened (relative to the *AASHTO LRFD*) interface surface, resulted in a change in the anticipated load path across the cold joint. In this case, the large northward shear demand parallel to the interface under member 11 would have ceased to be carried in part by the cohesive and tensile resistance of the concrete along the now-cracked plane. Remaining resistance mechanisms would have been a combination of (1) the reinforcing steel bars crossing the interface, (2) the frictional resistance generated by the clamping force on the interface, and (3) the unintended flexural and shear resistance afforded by the member 12 connection to the deck. As a result of the crack formation during removal of the falsework, the centroid (geometric center) of the horizontal shear load-resisting mechanism was effectively shifted northward.

2.4.2 Construction of Member 11/12 Nodal Region

The positioning of steel reinforcement within a concrete structure is a key contributor to its overall performance. Investigators examined the FIGG plans, the photographic documentation captured during construction, and the retained portions from the north end of the failed structure. The assessment focused on reconciliation of the steel reinforcement and the PT rod sizes and locations between the design plans and the as-built structure within the member 11/12 nodal region.

The reinforcement in the member 11/12 nodal region was found to closely match the FIGG plans. Although two specific deviations from the FIGG plans were identified through this reinforcement reconciliation process, the member 11-to-deck interface shear reinforcement position deviations and the PT rod 11S (lower rod) anchor plate position deviation were not considered to be significant. It would not be expected that these deviations would have had any noticeable effect on performance. Instead, they are recognized as minor deviations associated with tying steel reinforcement cages for a reinforced concrete structure (FHWA 2019). Thus—in assessing reinforcing bar sizes and locations—investigators identified no significant deviations from the construction plans. In constructing the nodal region, MCM met the expectation established in the FIGG plans, which was consistent with the assumptions made in the FIGG design. No significant deviations from the construction plans were identified through assessment of the sizes and locations of in-place steel reinforcement.

Based on postcollapse evidence, NTSB investigators determined that the construction process used to complete the concrete pour for the deck-side surface of the member 11/12 nodal region cold joint did not include intentional surface roughening of the concrete under members 11 and 12.

Of more significance to the bridge collapse was that—in addition to the steel bar reinforcing—the main span structure included nonstructural elements (hollow pipes) within the concrete. As described in section 1.6.3, these hollow pipes passed through the member 11/12 nodal region and acted as voids within the concrete mass. The voided areas exhibited a lower stiffness (than concrete) and were less able to resist applied loads than a monolithic concrete region. Therefore, when the pipes passed through a critical region of the structure, the surrounding concrete was subjected to higher stresses, and the voids may have caused an overstress and the unanticipated redirection of an assumed load path. Evidence in the main span bridge debris documented that all five nonstructural pipes bordered the failure planes, demonstrating their imposition on the behavior of a concrete nodal region. The NTSB concludes that the member 11/12 nodal region contained nonstructural voids (four hollow vertical pipe sleeves and the horizontal drain pipe) within the concrete that made it less able to resist applied loads, which contributed to the destabilization of this node through overstress and the subsequent collapse of the main span.

2.4.3 Subsequent Structural Distress During Transport of Main Span

2.4.3.1 Lift and Movement of Main Span. As described in section 2.4.1, the ABC process for the main span build included transport by SPMTs from the casting yard to the permanent installation site. Diagonal members 2 and 11 were pretensioned prior to the move. The cantilevered areas at the north end of the bridge (to be placed on the pylon pier) included the member 11/12 nodal region, and those at the south end (to be placed on the south pier) included the member 1/2 nodal region. In this configuration, the dead load effects were effectively reversed such that the demand on the horizontal shear plane under truss members 11 and 2 changed from an outward to an inward thrust. Once the main span was set on its permanent support piers (pylon and south piers), members 2 and 11 were detensioned, and the dead load shear demand reverted to its original orientation. Thus, the SPMT process imposed one cycle of significant change in the shear demand on the critical horizontal shear plane at the cold joints.

As noted earlier, a crack formed at the member 11/12 nodal region on this horizontal shear plane during removal of the falsework. Although intentionally roughened upper and lower concrete surfaces at the plane would have offered some interlocking resistance due to their matched profiles, the combination of an under-reinforced section and the transport-induced cycle on the previously cracked plane would have likely decreased the interlock. Once the bridge span had been placed on the permanent support piers, the centroid of the shear-resisting mechanisms under members 11 and 12 would likely have shifted farther to the north.

2.4.3.2 Placement of Main Span on Pylon Pier Shim Stacks. In direct contrast to the falsework support configuration that allowed truss member forces to flow directly into the continuous support beneath the truss line, once the main span was placed on its permanent support piers (on March 10), four permanent discrete shim stacks were placed under the pylon pier diaphragm—two to the east and two to the west side. However, no supporting shim stacks were located directly beneath the truss line. Then, on March 13, to address the cracking after detensioning evident at the member 11/12 nodal region, FIGG directed the placement of plastic shims underneath the (type 2) diaphragm at the centerline of the main span.[97]

This new bearing configuration increased the shear demand on the north–south-oriented vertical shear planes in the deck, immediately east and west of the member 11/12 intersection with the deck, as outlined below:

- The shear could not flow directly downward into a support under the diaphragm, now being forced to flow east–west to reach a shim stack support.
- The shear planes were perforated by vertically oriented pipe sleeves that passed upward through the diaphragm and ended adjacent to the east and west sides of member 12, where it adjoined the deck. The pipe sleeves reduced the amount of concrete available to resist the demand placed on these shear planes.
- The transverse post-tensioning in the deck, which would serve as a clamping force on the vertical shear planes, was detailed such that the northernmost post-tensioning point was located 48.5 inches south of the north end of the bridge—at the center of the plane where member 11 met the deck—placing it well south of the vertical shear planes under discussion here.
- The clamping force from the transverse post-tensioning was less effective at the northernmost extent of the span, where large shear forces were generated by the dead load of the structure.

[97] The distress observed in the structure between March 10 and 13, 2018, prompted FIGG to direct MCM to install supplemental support (plastic shims) under the north diaphragm. MCM used a combination of plastic and steel shims. This diaphragm was originally supported on the pylon pier by four shim stacks. Until this time, there had been no support along the centerline of the bridge in the area between the vertical pipe sleeves that passed through the diaphragm. On March 13, at 9:45 a.m., FIGG directed MCM to immediately place a shim under the diaphragm in the area along the bridge centerline, between the existing two innermost shim stacks. The contractor was told that lifting or "jacking" of the bridge was not required; thus, the shim was to be manually inserted in the space between the diaphragm and the pylon pier. Given that the four existing shim stacks were already supporting the weight of the bridge on the pier, the manual insertion of a new shim stack under the diaphragm (that is, without lifting the bridge to rebalance the loads between the five stacks) was not anticipated to have permitted any sizeable load to be reproportioned onto the stack. Thus, the new shim would have attracted load only in a situation where further distress caused a downward movement of the center of the diaphragm relative to the remainder of the diaphragm.

2.4.3.3 Post-Tensioning Force in Diagonal Member 11. The PT rods in truss member 11 served multiple purposes during movement of the main span, as follows:

- The forces imparted by the rods onto member 11 counteracted the tensile forces in the member while the north end of the bridge was cantilevered beyond the north SPMT support point.
- The PT rods also temporarily generated both a beneficial vertical clamping force across the horizontal interface shear plane at the base of member 11 and a detrimental horizontal shearing force across the base of members 11 and 12, pushing northward relative to the deck.

Given that the angle of member 11 relative to the deck was about 32 degrees, the magnitude of the detrimental post-tensioning-induced horizontal shearing force was about 1.6 times the beneficial vertical clamping force.

Once the main span was resting on the pier supports, member 11 was expected to carry only axial compressive forces within the overall structure. Thus, the construction plan called for removal of the post-tensioning forces in member 11 to reduce both the clamping force underneath and the driving force pushing members 11 and 12 northward relative to the deck.

Immediately after the removal of post-tensioning in member 11, the concrete distress previously observed in the member 11/12 nodal region significantly increased. The decrease of the clamping force would have reduced the frictional resistance on the shear plane. Thus, because the large dead load-induced northward shearing force was still present, a large shear-resisting mechanism was required. The resistance mechanism thus engaged could have come from farther toward the north end of the member 11/12 nodal region.

The northward movement, or dislocation, of the upper part of the member 11/12 nodal region relative to the deck was apparent in two observed structural behaviors:

- ***Cracking on the north face of member 12:*** The northward movement of the base of this member, while the top was restrained from translation and rotation by the canopy, caused it to bend in double curvature and exhibit flexural cracking. These north-face cracks were perpendicular to the flexural tensile extreme fiber of the member and were observed within its lower half.
- ***Cracking consistent with interface shear along much of the node and punching shear at the northern extent of the node:*** These effects were observed in the deck around the bases of members 11 and 12.

NTSB investigators determined that two primary mechanisms temporarily resisted the northward dislocation of the nodal region relative to the deck: (1) the lower portion of member 12, whose bottom end was connected to the diaphragm and whose top end was connected to the canopy; and (2) the steel reinforcement that crossed the shear planes under member 11 and beside member 12. Given the orientations of the shear planes, the amount of translation that had already occurred, and the lack of clamping forces, it is likely that the concrete-to-concrete interface shear resistance along these vertical planes did not offer significant resistance.

The column steel reinforcement in member 12 was in place from the bottom of the diaphragm and extended above the deck. The vertical reinforcement, with lap splices beginning just above the deck level, and the confinement reinforcement combined to create a concrete column that was capable of temporarily buttressing much of the load being driven northward by member 11. Within member 12, three size 11 rebars ran vertically along the south face; and three size 7 rebars ran vertically along each of the east, north, and west faces. The lower ends of these reinforcements were anchored into the bottom of the north diaphragm and extended above the deck level. Except for the size 11 rebar on the center of the south face and the size 7 rebar on the center of the north face, the rebars were lap-spliced with matching rebars beginning just above the cold joint at the base of member 12. In the case of the two excepted rebars—which conflicted with the horizontal drain pipe—the lower rebar in the lap splice was detailed by FIGG to hook within the volume of concrete immediately above the drain pipe.

It is important to note that the longitudinal post-tensioning anchorages in the deck were not positioned to resist a significant portion of the northward force applied to the nodal region by member 11. Only horizontal loads applied within the footprint of member 11 had an opportunity to flow outward to the longitudinal post-tensioning anchorages. Because the horizontal interface under member 11 was cracked, and the node was translating northward, it appears that the anchorages would have counteracted only the horizontal force transferred into the deck via the rebar traversing the member 11 footprint shear plane.[98]

NTSB investigators reviewed chronological photographic evidence of the distress observed in the nodal region between the time when member 11 was detensioned and the afternoon of March 14. In particular, photographs taken immediately after the detensioning document the significant increase in distress. Subsequent photographs taken after the completion of detensioning of the PT rods in member 11 show how the cracking in the deck and diaphragm had significantly worsened. Tension cracks parallel to the compression struts in the diaphragm, which would have been carrying the dead load of the bridge through the nodal region and into the shim stacks, were also documented. The upper end of these cracks disconnects the member 12 nodal concrete from the surrounding deck along the vertical shear planes to the east and west. Additional photographs show cracks on the south face of the north diaphragm consistent with the high shear forces caused by the outboard positioning of the shim stacks relative to the centerline of the bridge.

Specifically, photographs captured midday on March 13 through midday on March 14 demonstrate how the distress in the nodal region progressed during the 2 days preceding the collapse. A structural element that has been inelastically damaged (damaged through irreversible action, such as cracking of concrete or yielding of steel) cannot be returned to its original, predamaged state. The structural material would have been permanently changed from its original condition, and the resistance that the bridge material and overall structural system would have offered to subsequent loading would also have been changed. Therefore, the NTSB concludes that, although it may be generally accepted that concrete itself is susceptible to cracking, the rate of premature concrete distress was clear evidence that the structure was progressing toward failure and should have alerted FIGG and MCM to the origin of the distress mechanism that was causing the cracking and the rapidity of cracking progression.

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[98] Only six size 7 rebars were engaged on this shear plane.

2.4.4 Reapplication of Post-Tensioning in Diagonal Member 11

In an attempt to stop the distress observed in the structure, FIGG decided to retension the PT rods in truss member 11 on March 15, the day of the collapse. Although NTSB investigators found no record explicitly describing the intent of this action, it is likely that the objective was to generate clamping force across the horizontal shear interface under member 11. However, the angle of action of member 11 was such that the magnitude of the detrimental horizontal driving force created by applying axial post-tensioning was approximately 1.6 times the beneficial vertical clamping force generated across the horizontal plane.

Postcollapse evidence indicates that, of the two PT rods in member 11, the bottom end of PT rod 11S (lower rod) was well anchored into the deck on the south side of the failure region, while the bottom end of PT rod 11N (upper rod) was anchored into the portion of the nodal region that was experiencing significant cracking distress and had translated northward during the collapse. The contractor executing the post-tensioning operation reported that the bars had just been retensioned at the time of collapse. Thus, it is assumed that the PT rods were sufficiently anchored immediately before collapse to generate their intended loads in member 11. However, given the location of the upper rod anchorage relative to the failure planes, rod 11N is not expected to have generated any significant clamping or driving forces on the interface shear failure planes in the nodal region.

It is apparent that the vertically oriented rebars at the base of member 12 resisted a significant portion of the shear load applied to the region by member 11. Due to the geometry of the node, these forces would have been resisted through a combination of (1) horizontal shear on the rods at a location just above the drain pipe and (2) flexure on member 12 rotating about the east–west axis and causing tension on the south face and compression on the north face at the base of the column. This resistance mechanism was contingent on the vertical rebar in the lower portion of member 12 remaining spliced with the bars in the upper portion of member 12 and also on the entirety of the member 12 cross section remaining geometrically stable.

The retensioning of rods located within member 11 increased demand on and corresponding damage to the member 11/12 nodal region until the distress became critical (see figure 32). Note that the solid red line A–B–C–D–E–F identifies an approximation of the east–west-oriented failure plane. The red cross-hatched region bounded by C–D–E–H identifies an approximation of the north–south-oriented failure planes in the deck. A red cross-hatched plane is located at each of the east and west faces of member 12. Figure 32 also shows the drain pipe (in darker blue fill) that bounded the bottom of the D–E–F failure plane and the vertical pipe sleeves (in light blue fill) that perforated the C–D–E–H failure planes.

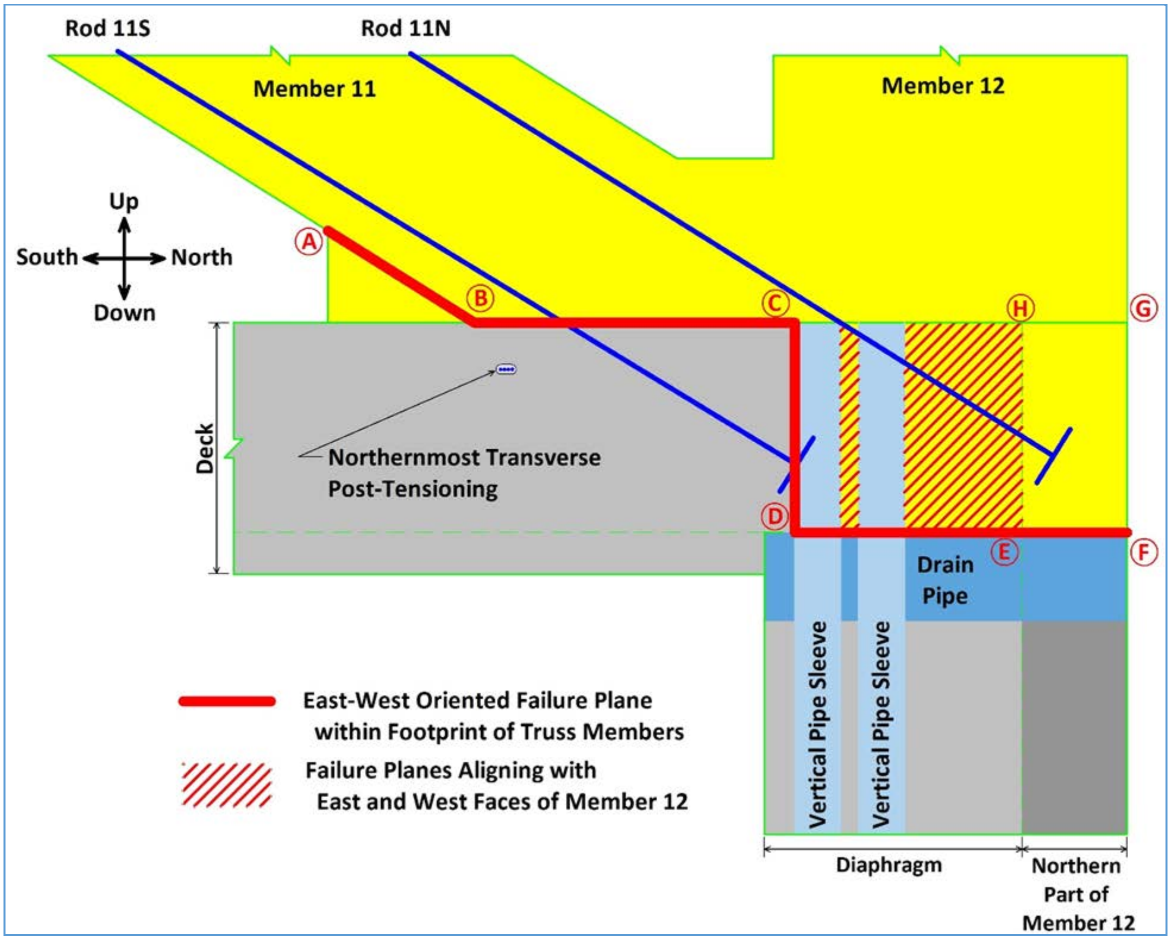

Figure 32. Midline cross section of member 11/12 nodal region, with annotations indicating resistance mechanisms and distress. (Source: FHWA 2019)

2.5 Independent Peer Review of Complex Bridge Design

Section 2.5 discusses how an incomplete and inadequate peer review, conducted by an unqualified peer review firm, led to the failure to recognize the significant under-design of the steel reinforcement within the 11/12 node, which was unable to resist the horizontal shear between diagonal 11 and the bridge deck.

2.5.1 Incomplete Independent Peer Review by Unqualified Firm

Per the FDOT *Plans Preparation Manual*, any independent peer review firm hired was to have no other involvement with the project (FDOT 2014a). Further, FDOT requires that a peer review conducted by an outside firm be "an independent verification of the design using different programs and independent processes than what was used by the EOR." The peer review does not limit itself to checking the plan and calculations of the EOR, but rather is a method to detect errors by reviewing specific compliance with FDOT, AASHTO, and FHWA design requirements.

Although Louis Berger used a different program, called ADINA, a finite element program, which analyzed the entire structure, the firm did not analyze the different sequence stages of construction, focusing on the forces in the members themselves rather than on the nodal areas of each diagonal.[99] According to Louis Berger, such analysis would have added too much time and exceeded the agreed-upon budget. Because the peer review did not include the nodal forces of each diagonal, Louis Berger did not identify the significant under-design of reinforcing that led to the horizontal shear failure between diagonal 11 and the bridge deck.

Both the *AASHTO LRFD Bridge Design Specifications* and FDOT *Structures Design Guidelines* required Louis Berger's scope of work to include analysis of the structure during various construction phases. Because it was a single analysis of the superstructure and not of the individual stages of construction, the Louis Berger independent peer review would not be considered comprehensive or thorough.

Although the FDOT *Plans Preparation Manual* indicates that the peer review is intended to be a comprehensive, thorough, and independent verification of the original work—and lists items typically included in all peer reviews—it does not specifically require that all nodal forces and connections of category 2 bridge structures be checked and verified. FIGG's failure to adhere to the FDOT *Plans Preparation Manual* when initially contracting for an independent peer review firm most likely contributed to the inadequacy of the peer review and its failure to detect the under-design of the bridge. This is because, despite the requirements of the *Plans Preparation Manual*, MCM and the FIGG quality management plan initially indicated that the independent review would be performed by a different design office within FIGG—one not involved in the original design—and would compare calculations with the original design to verify their adequacy. The plan specified that the FIGG design quality team would ensure that all aspects of the design followed the plan procedures. FDOT informed FIGG at a meeting on June 30, 2016, that an independent peer review performed by an independent engineering firm was required. Therefore, in July 2016, MCM required that FIGG hire an independent firm to do the peer review and agreed to cover the additional cost.[100]

In July 2016, Louis Berger submitted the scope of work to FIGG to perform an independent peer review of the bridge plans in accordance with the project, request for proposal requirements, and FDOT's *Plans Preparation Manual*. The original bid from Louis Berger for the work was $110,000, and it included analysis of the connections (nodes and cold joints).[101] However, in August 2016, FIGG received lower bids from other independent peer review firms. On August 10, 2016, Louis Berger specified to FIGG that its independent peer review would be for a "very

[99] NTSB staff obtained the output files from the ADINA finite element program and confirmed that the independent peer review analyzed the entire structure as one structure and did not analyze the different stages of construction, including individual spans or the connection nodal region forces. See ADINA website, accessed September 23, 2019.

[100] On February 6, 2017, MCM and FIGG entered into a change order to their original agreement for the additional peer review fee.

[101] According to the scope of work noted in section 1.9, the following were to be completed: (1) develop finite element model for the bridge and estimation of demands on all elements due to different load combinations; (2) peer review foundation and substructure plans; (3) peer review final foundation, substructure, and superstructure plan submittals.

thorough scope and creation of independent models," and agreed to perform the same scope of work but reduced both its fee (to $61,000) and the project timeframe (from 10 weeks to 7 weeks).

By August 17, 2016, Louis Berger had the notice to proceed and, by August 31, 2016, the modeling and evaluation of demand were to be delivered.[102] Louis Berger submitted the 100 percent foundation and substructure plans to FDOT on September 13 and September 29, respectively. The review of the superstructure plans was to be delivered by October 5, 2016.[103]

Postcollapse, Louis Berger told investigators that, even though the scope of work originally included analysis of the connections (nodes and [cold] joints), because of the lower budget and reduced time to perform the analysis, it did not fulfill the scope of work. The NTSB found no evidence that the scope of work in the contract with FIGG was ever revised. In addition, Louis Berger stated that its model was for the superstructure structure and not the construction sequence staging analysis because that level of analysis required "much more time" than was agreed to with FIGG.

The contract for the pedestrian bridge design required that an independent peer review be conducted of the design and plans, as stated in the *Plans Preparation Manual*, because the bridge was classified as a category 2 structure (FDOT 2014a).[104] In addition, the peer review firm was to be prequalified in accordance with FAC Rule 14-75, which establishes minimum qualification standards by type of work for consultants who seek to provide professional services to FDOT.

According to FDOT records, neither Louis Berger U.S., Inc.—nor its predecessor, Louis Berger Group, Inc.—was ever qualified and had never received a prequalification letter for this specific work type, even though when Louis Berger procured the independent peer review contract with FIGG, the FDOT website listed the firm as prequalified. Louis Berger had provided FIGG with an email attachment of an undated printout from the website.

At the request of NTSB investigators, FDOT confirmed that Louis Berger Group was at one time listed on the website-generated prequalification report for work type 4.3.1, complex bridge design-concrete, due to a technical error in processing physical records into the report. However, according to FDOT, its website is informational only and is not intended to be used as a substitute for due diligence. Moreover, for a firm to be appropriately prequalified, FDOT issues a prequalification letter detailing the specific work types that have been approved.

Although FIGG may not have been aware of the actual prequalification status held by Louis Berger, at all times Louis Berger would have known which FDOT work type prequalification(s) it held and, in fact, it had received an FDOT notification (dated March 18, 2013) of "insufficient" status for 4.3.1 complex bridge design-concrete. Louis Berger was not qualified to conduct the

[102] On September 16, 2016, FIGG finalized the contract with Louis Berger to conduct the independent peer review.

[103] FDOT received the 100 percent superstructure plans on February 10, 2017.

[104] When FIU advertised for its bridge design and build request for proposals in June 2014, the 2014 FDOT *Plans Preparation Manual* was in effect and classified the bridge as a category 2 structure (because it was a post-tensioned concrete bridge, and it used design concepts, components, details, and construction techniques with a history of less than 5 years of use in Florida; FDOT 2014a, pages 26-1 to 26-61).

independent peer review when it signed and sealed the 100 percent certification letters in September 2016 and February 2017.

Because FIGG did not include an allowance for the required independent peer review in its original work proposal to MCM, the project faced a shortened timeframe in which to review a complex bridge superstructure and the substructures within. Even so, the NTSB concludes that Louis Berger was not qualified by FDOT to conduct an independent peer review and failed to perform an adequate review of the FIGG design plans and to recognize the significant under-design of the steel reinforcement within the 11/12 node, which was unable to resist the horizontal shear between diagonal 11 and the bridge deck. The NTSB also concludes that FIGG's failure to adhere to the FDOT *Plans Preparation Manual* requirements for a complex category 2 bridge structure within its work proposal to MCM, calling for an independent firm to conduct a comprehensive peer review, led to the inadequate peer review performed by Louis Berger, which failed to detect the under-design of the bridge. The NTSB further concludes that had the FDOT *Plans Preparation Manual* called for all nodal forces of category 2 bridge structures to be checked and verified by a qualified independent peer review, this collapse might have been prevented.

The NTSB recommends that FDOT revise its *Plans Preparation Manual* to require that the qualified independent peer review for category 2 bridge structures include checking and verifying the design calculations used for all nodal forces.

2.5.2 FDOT Oversight of Independent Peer Reviews

The FDOT *Plans Preparation Manual* required, for the independent peer review of the bridge design plans, a 90 percent plan submittal and a 100 percent plan submittal to FDOT, FIGG, and FIU (FDOT 2014a). The 90 percent submittal was to include a tabulated list of all review comments and responses, and a standard peer review certification letter specifying all outstanding comments and issues to be resolved and implemented prior to the 100 percent plan submittal. The 100 percent submittal was required to have a signed and sealed certification letter stating that all review comments were adequately addressed and that the design complied with all FDOT and FHWA requirements.

Louis Berger did not provide the 90 percent certification letters. FDOT told NTSB investigators that the intent of the 90 percent plan submittal was met, however, because (1) it was considered an in-progress certification, and (2) FDOT received the final certification, which included review of the 90 percent work. In addition, FDOT informed FIGG by email that it agreed with MCM and FIGG in submitting the 90 percent superstructure plans to FIU and FDOT without the independent peer review documentation. Louis Berger did submit the signed and sealed 100 percent certification letters for the bridge foundation, substructure, and superstructure plans.

Although FDOT did not require verification of the independent peer review firm's prequalification when Louis Berger submitted signed and sealed 100 percent certification letters for the bridge foundation, substructure, and superstructure plans, FDOT also failed to verify through its own records—as the agency responsible for qualifying independent peer review firms—that it had, in fact, rejected Louis Berger's 2013 application to perform the work type for this specific independent peer review (work type 4.3.1 complex bridge design-concrete). FDOT missed a critical oversight step in a complex and unique bridge design project when it permitted

MCM and FIGG to submit the 90 percent superstructure plans without the independent peer review documentation. By carrying out this step, FDOT could have verified Louis Berger's qualification to perform the required highly complex level of review needed for this project. The NTSB concludes that, as part of its oversight of LAP projects and new construction, FDOT should have verified Louis Berger's qualifications as an independent peer review firm for complex bridge design-concrete upon receiving the 100 percent certification letters for the bridge foundation, substructure, and superstructure plans.

Therefore, the NTSB recommends that FDOT revise its *Plans Preparation Manual* to require the engineering firm or company independently peer-reviewing bridge design plans to submit a prequalification letter showing that it is qualified in accordance with FAC Rule 14-75 before permitting the firm to sign and seal the 100 percent certification letters indicating that the bridge designs have been peer reviewed.

Although Louis Berger informed FIGG via email on July 6, 2016, that it was prequalified by FDOT for work type 4.3.1, and FDOT's website did contain a technical error in listing Louis Berger as having the FDOT prequalification, FIGG should have verified Louis Berger's technical qualification for the work type needed on the FIU bridge. Therefore, the NTSB concludes that FIGG did not perform its due diligence when it contracted with Louis Berger for the independent peer review of the highly complex and uncommon concrete bridge design. The NTSB recommends that FIGG institute a company policy to obtain a prequalification letter before finalizing any peer review contract with any engineering firm or company being considered to conduct peer review services.

2.6 Shortcomings in Oversight of Evaluation of and Response to Significant Observed Bridge Structure Distress Prior to Collapse

In addition to presenting safety recommendations for FDOT, section 2.6 discusses how the EOR displayed poor engineering judgment by failing to recognize the extensive, large cracks observed in the member 11/12 nodal region as being abnormal for a reinforced concrete structure; and that the remedial plan developed without peer review by the bridge design-build team failed to adequately address the scale of the concrete cracking, which was a clear indication of the failure of the load-resisting mechanisms of the bridge. It also describes how administrative oversights (1) led to failure to have the post-tensioning inspection contractor onsite on the morning of March 15, and (2) limited the authority of the firm responsible for project administration.

2.6.1 Cracking Characteristics in Member 11/12 Nodal Region

During the 3 weeks preceding the bridge collapse, the structure displayed notable cracking of the reinforced concrete. The distress was observed in four specific locations and was repeatedly documented by parties associated with the design, construction, and operation of the bridge.[105] In particular, the cracking became markedly worse immediately after the detensioning of member 11 on March 10. Cracking and spalling continued to worsen over the following days, with the upper part of the node further dislocating to the north, until the bridge collapsed (see figures 33 and 34).

[105] The locations were the east and west sides of member 11 at the deck and the east and west sides of member 12 at the north end of the deck.

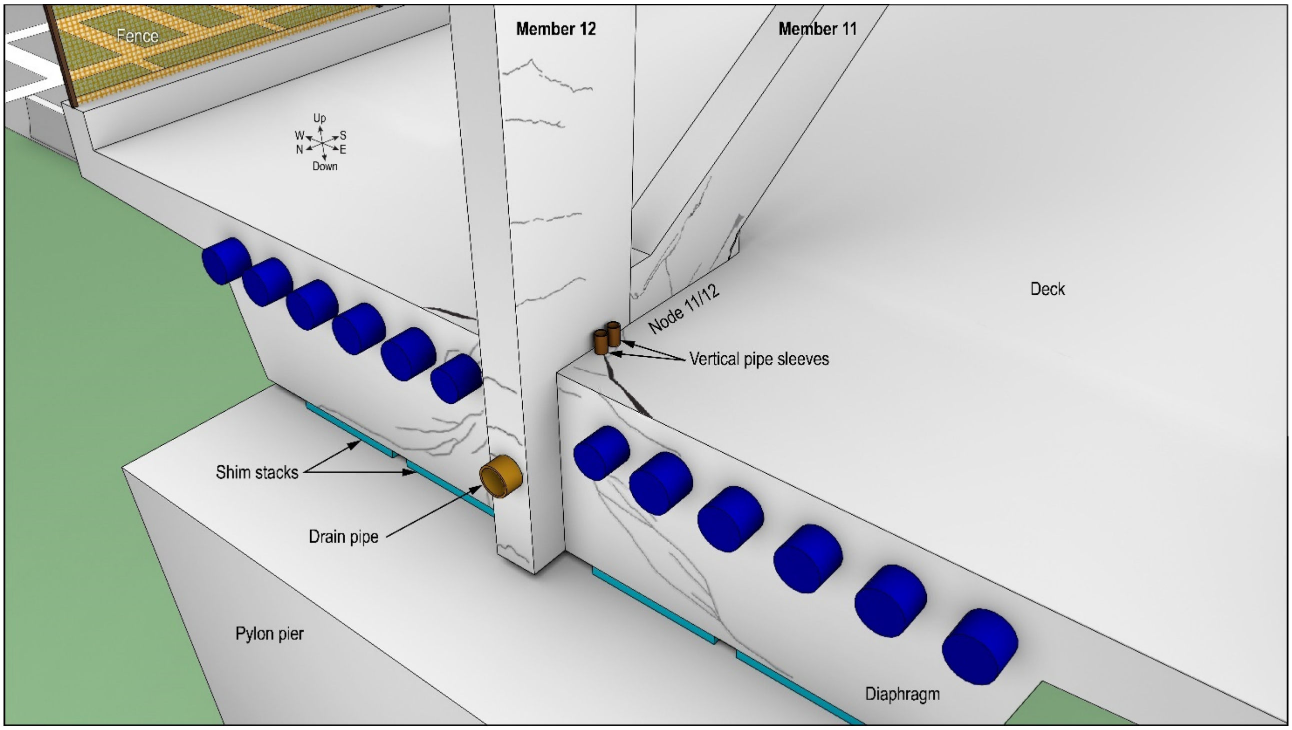

Figure 33. South and west view of extent of cracking at member 11/12 nodal region, deck, and diaphragm, indicating structural distress.

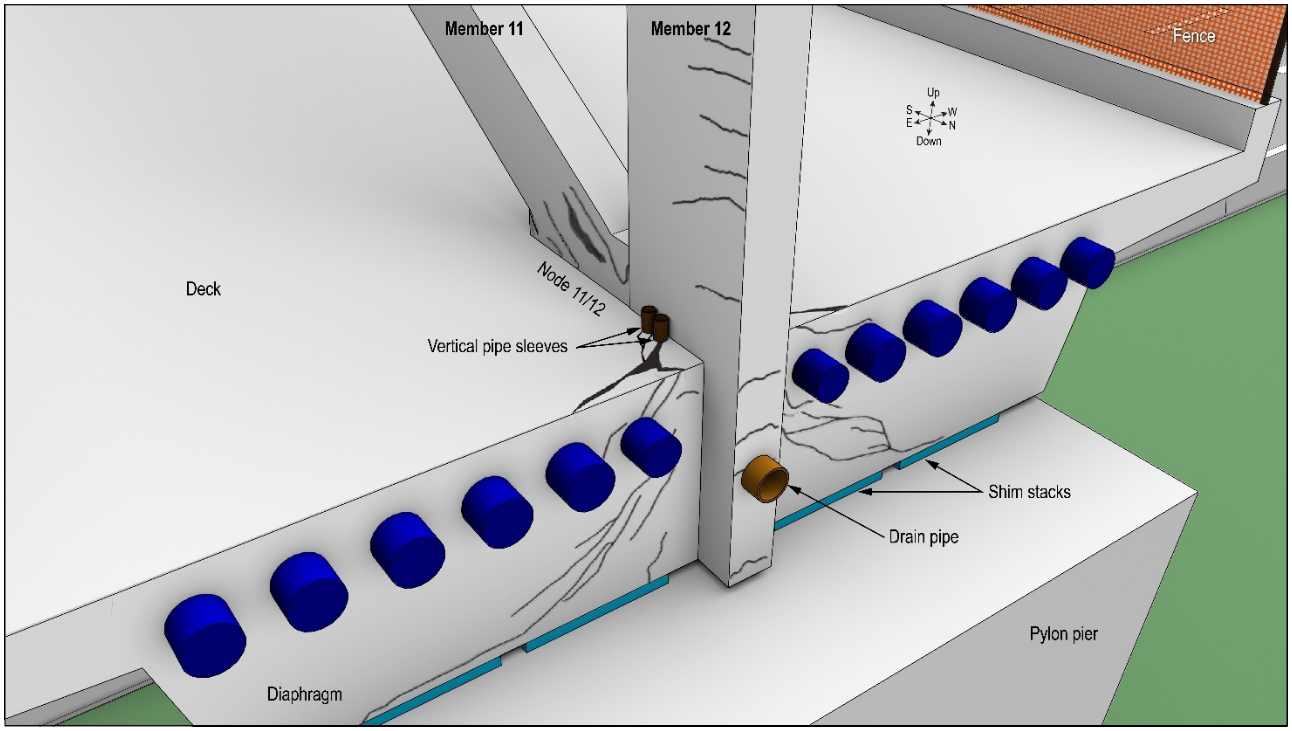

Figure 34. South and east view of extent of cracking at member 11/12 nodal region, deck, and diaphragm, indicating structural distress.

The extensive, large, and wide cracks observed in the member 11/12 nodal region should have been recognized as being abnormal for a reinforced concrete structure. Cracks in concrete can be caused by a variety of factors, such as restrained shrinkage, thermal effects, or structural loading. The size of the cracks ranged from smaller than could be detected by unaided human vision (approximately 0.01 inch [0.25 millimeter]) to gross separation of the two opposing portions of the structure. For reference, in a reinforced concrete element, cracks up to 0.016 inch (0.4 millimeter) wide are often considered generally acceptable, depending on their location and the purpose of the structure.[106]

The structural cracks—located in critical regions of a nonredundant single plane truss—were more than 40 times larger than the widths stated above, as further discussed below:

- The crack sizes were well beyond any level of acceptability.
- The crack widths indicated severe yielding of the reinforcing steel and potentially complete fracture of the reinforcement.
- The cracks resulted from structural loading and were unlike the typical cracks caused by shrinkage.
- The cracks required, per FDOT standards, an engineering evaluation by the bridge design and EOR firm, FIGG.

This scale of cracking in this type of structure was a clear indication that the intended load-resisting mechanisms were failing. As the unanticipated load paths were pushed beyond their limits, the structure exhibited increasing distress until failure.

2.6.2 Precollapse Decision to Retension Member 11

2.6.2.1 FIGG Remedial Plan and Associated Assumptions. On March 15, FIGG presented the remedial plan to retension member 11 to FDOT, FIU, MCM, and Bolton, Perez. This action was not shown on the FIGG design plans, because it was devised to address the structural distress and growing cracks in the main span.[107] The March 15 work was a rushed change order for construction. The post-tensioning inspection contractor (The Corradino Group)—which was not notified until that morning—was not onsite when the work was performed and the collapse occurred.[108]

Postcollapse, the FIGG EOR stated that the retensioning of this truss member would bring the main span back to its "pre-existing condition"—of a previous stage. According to FIGG, this decision was based on judgment that returning the main span to its preexisting condition was not a change to the FIGG design and was the right thing to do. A change to the design plans would

[106] Per AASHTO 2015 and ACI 2008.

[107] The restressing of member 11 to its original stressing force of 280 kips (or 280,000 pounds) was to be accomplished in 50-kip increments. According to FIGG, no specific course of action was formalized except that the retensioning progress in increments. No specific sequence for the order of restressing the top and bottom bars was discussed, and FIGG emphasized that the diaphragm (nodal region) should be closely monitored during the restressing process to ensure that cracking did not increase.

[108] According to Bolton, Perez postcollapse, "This remedial work was not included in the contract plans." It was requested during the meeting that this work be reviewed and approved in advance, including peer review.

have required an independent peer review. The FDOT *Structures Manual*, *Structures Design Guidelines*, indicate that any design changes must be signed and sealed by a P.E. licensed in the state of Florida (FDOT 2015b).

The NTSB does not agree that the retensioning of member 11 would have returned the main span to a "pre-existing condition" for the following reasons:

- The main span was supported in a different manner on both ends (but in locations different from the piers) while in the casting yard versus while it was spanning SW 8th Street. In the casting yard, it was supported on both ends by megashores; over SW 8th Street, it was supported by bearing pads on the south pier and shim plates on the pylon pier.
- The severity of the cracks in the member 11/12 nodal region after March 10 was notable. The cracks had grown significantly compared with the small cracks that appeared on February 24 in the casting yard and clearly indicated the severe yielding and/or fracture of the steel reinforcement.
- The dislocation of the upper portion of the nodal region to the north had changed the geometry of the system and reduced any interlock across shearing planes. Thus, the application of post-tensioning forces along inclined member 11 could not possibly reverse the distress (cracking) and could have unanticipated consequences.

The retensioning of member 11 on March 15 was the final stressing force that resulted in the failure of the member 11/12 nodal region. The FIGG design did not show the restressing of member 11, nor would the restressing have returned the node to its precracking condition. Therefore, the NTSB concludes that the restressing of member 11 was a manipulation of loads that constituted a change to the FIGG design, and, before being implemented, should have been independently peer reviewed and signed and sealed by a P.E.

Moreover, the NTSB concludes that the structural cracking and northward dislocation of the upper part of the member 11/12 nodal region, as documented in the days leading up to the collapse, was strong evidence that the structure was progressing toward failure; and the detensioning of the PT rods located in member 11 significantly increased the damage to the member 11/12 nodal region. In addition, the NTSB concludes that, although the FIGG EOR and design manager were engaged by MCM to assess the increased cracking of the structure, they neither recognized that the singular load path in this nonredundant bridge had been compromised nor took appropriate action to mitigate the risk of failure.

2.6.2.2 Responsibilities and Authorities Among Parties. All parties involved in the LAP project to build the pedestrian bridge were aware of the cracks and their progression, including FDOT, FIU, FIGG, MCM, and Bolton, Perez. Likewise, all parties were present at the meeting on the morning of the collapse, March 15, to hear the FIGG presentation on the worsening structural distress.

The remedial work called for placing workers on the structure without identifying the origin of the distress or determining whether the cracks were structural. Without the post-tensioning inspector available, Bolton, Perez could have authorized the work to be suspended, acting collectively with FDOT and FIU. Moreover, it is important to note that the parties involved failed to discuss the following:

- Restricting all pedestrian and vehicular traffic on SW 8th Street under the main span until shoring was in place and inspected.
- Immediately closing the bridge to construction personnel.
- Installing shoring to fully support the entire bridge weight.
- Using MCM-directed shoring construction techniques that did not require placing workers directly under the bridge.

FIGG and MCM were familiar with the FDOT automated system to facilitate lane closures and had requested the closure of traffic lanes to perform bridge work on two separate occasions.[109] Per the FAC, the FIGG EOR "personally 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." In addition, the contract between FIU and MCM indicated that the FIU associate vice president for facilities management could appoint engineer's assistants to "suspend the work until any questions at issue can be referred to and decided" by FIU administration.

The contract between MCM and Bolton, Perez stated that Bolton, Perez had identical authority to the FDOT project administrator and the FDOT resident engineer. However, the contract also specified that Bolton, Perez was to seek input from the construction project manager (FIU), as necessary, in exercising its professional judgment in performing its obligations and responsibilities. Although FDOT entrusted Bolton, Perez with the responsibility of administering the project and implementing actions based on that authority, as the CEI, it did not have complete authority to act on its own. As stated in the CEI scope of work, Bolton, Perez was to act collectively with FDOT/FIU in providing recommendations and advice (discussed in section 1.5).

FDOT has plenary authority over state rights of way and state bridges, and can direct or authorize partial or complete road closures as necessary. Because the pedestrian bridge was a LAP project, FDOT had no onsite inspector monitoring construction, nor was it required to do so.

2.6.3 Maintenance of Traffic Deficiencies

MOT is a process of establishing a work zone and providing related transportation management and temporary traffic control on street and highway rights of way. The FDOT *Construction Project Administration Manual* recommends actions to shut down a project due to MOT deficiencies, as discussed in section 1.11.2 (FDOT 2014b). On past projects, due to safety concerns, FDOT had occasionally closed a bridge or taken other safety measures during construction; the more typical case is when the CEI orders the contractor to abandon an operation because of safety issues. Examples provided by FDOT in which a bridge was closed to protect the

[109] On January 31, 2018, MCM requested and FDOT issued a two-lane blanket road closure from February to April for westbound traffic on SW 8th Street. On December 12, 2017, on behalf of MCM, FIGG requested and FDOT worked with local municipalities in permitting a full closure of SW 8th Street for the move of the precast concrete main span to its final position.

safety of the public included the Memorial Causeway Bridge in Clearwater in 2004, the Skyway Bridge in St. Petersburg in 2015, and the I-4 Ultimate Project in Orlando in 2018.

Any MOT deficiency that is "considered a severe hazard and life threatening will require immediate corrective action by the Contractor." According to FDOT, the failure to correct the hazard immediately "is basis to shut down the project and obtain other means to correct the hazard."

As described in section 1.14, FDOT's *Standard Specifications for Road and Bridge Construction* state that, "in general, structural cracks are cracks that extend deeper than 1/2 inch." Reinforced concrete element cracks of up to approximately 0.016 inch (0.4 millimeter) wide are often considered generally acceptable; however, the FIU bridge structural cracks were 40 times larger (and located in critical regions of a nonredundant single plane truss) and were documented by FIU, MCM, FIGG, and Bolton, Perez, all of which were capable of recognizing that the cracks were well beyond acceptable dimensions. Therefore, the NTSB concludes that, beginning with the cracking identified on February 24, 2018, the distress in the main span structure was active, continued to grow, and was well documented by all parties involved in the design, construction, and oversight of the bridge. The NTSB further concludes that neither FIU, MCM, FIGG, nor Bolton, Perez took the responsibility for declaring that the cracks were beyond any level of acceptability and did not meet FDOT standards. Additionally, the NTSB concludes that, under the terms and conditions of the contract, Bolton, Perez had the authority to direct or authorize partial or complete road closures as necessary, acting in concert with FDOT and FIU; however, none acted to close the road under the bridge, contributing to the severity of the impact of the bridge collapse.

The NTSB further concludes that LAP agreements require stronger language to clarify that the certified local agency has the authority to immediately close a bridge when structural cracks are first detected or in situations that require further investigation to protect the health, safety, and welfare of the public. Consequently, the NTSB recommends that FDOT revise LAP agreements to specify that when structural cracks are initially detected during bridge construction, the EOR, CEI, design–build firm, or local agency that owns or is responsible for the bridge construction must immediately close the bridge to construction personnel and close the road underneath; fully support the entire bridge weight using construction techniques that do not require placing workers on or directly under the bridge during installation; and restrict all pedestrian, vehicular, and construction traffic on the bridge until the complete support is in place and inspected.

2.6.4 Local Agency Project Oversight

LAP projects are those in which a local agency is certified by FDOT to administer federal-aid funds. FDOT acts as the supervising agency, ensuring that the project is developed according to approved plans and specifications. Over the last 5 years, pedestrian bridges have accounted for 0.6 percent of all FDOT bridge projects. In 73 percent of all LAP projects during this period, FDOT itself designed the project and assumed the associated risk.

In 2014, FIU received full certification as a local agency for 3 years, or until the pedestrian bridge was completed. Accordingly, FIU was responsible for the planning, design, construction, inspection, operation, and maintenance of the project. The FDOT LAP project agreement includes no requirements to close a bridge when structural cracks are first detected or in situations that require further investigation to assess the health, safety, and welfare of the public.

The pedestrian bridge had a unique (concrete truss) and nonredundant design (single row of truss members). Although FDOT was unfamiliar with this type of bridge, it was responsible—as the supervising agency—for ensuring that it was developed according to approved plans and specifications. Moreover, under the LAP program, FIU was responsible for the planning, design, construction, inspection, operation, and maintenance of this unique and complex bridge project.

LAP projects are ideal for low-risk ventures. However, FIU received full certification from FDOT as a local agency, even though it had no P.E.s on staff and relied solely on FIGG, MCM, and Bolton, Perez for a complex bridge project. In addition, FDOT required an independent peer review because the bridge project was unique and highly complex. Therefore, the NTSB concludes that that, given the pedestrian bridge's unique, nonredundant design, FDOT should have ensured that the local agencies involved in the project had adequate staff who were trained and experienced in administering these types of uncommon bridge designs. In addition, the NTSB concludes that FDOT should have provided greater oversight of this complex LAP project to ensure that all safety issues were identified and addressed.

As discussed in section 1.13, as a result of the bridge collapse, in July 2019, Florida's governor signed a law regarding transportation projects so that, for any portions of a transportation project that is on, under, or over a department-owned right of way (regardless of who funds the project), FDOT shall review the project's design plans for compliance with departmental design standards. In addition, FDOT is considering new LAP language stating that, as part of the new law to review project design plans, FDOT may reject designs that do not meet FDOT standards or allocate FDOT-managed resources to facilitate compliance with applicable design standards. With the authority to reject noncompliant design plans that do not meet FDOT standards and the ability to provide structural engineers to facilitate compliance, FDOT is making positive changes to the LAP. The NTSB supports these steps and, because of the nonredundant bridge design in this case and FDOT's unfamiliarity with it, the NTSB recommends that FDOT, to help facilitate compliance with FDOT standards, require its personnel to monitor and inspect all LAP bridge projects determined by the department to have uncommon designs.

2.7 Lack of Redundancy Guidelines in Specifications for Pedestrian and Concrete Truss Bridges

Section 2.7 discusses the need for national and state guidelines on redundancy for the design of reinforced concrete structures and, particularly, for uncommon bridge structures.

As a concrete truss bridge composed of a single row of diagonal supports that extended down the centerline of the structure, the pedestrian bridge design created a situation in which each member was nonredundant—any single component failure would cause failure of the bridge. Traditionally, a truss design has two sets of regularly spaced vertical truss pieces running along each side of the bridge, with top lateral bracing to provide stability between the truss lines.

Postcollapse, FIGG insisted that the pedestrian bridge was redundant by pointing out that longitudinal and transverse tendons were located in the deck and that PT rods were located in the diagonal truss members. Although this statement about the structure's internal redundancy was true, it did not address the structure's lack of load path redundancy and so did not alleviate the problem that, if one member failed, the entire bridge would fail.

Although the *AASHTO LRFD* addresses the design of truss structures and defines requirements for "redundancy" and "redundant members," it does not specifically discuss redundancy in the design of concrete structures (AASHTO 2015). The *AASHTO LRFD* contains an introductory discussion of redundancy, but states only that all bridges must be designed to achieve the objectives of constructability, safety, and serviceability. The bridge design specifications recommend a redundancy factor of at least 1.05 for nonredundant members. The FDOT *Structures Manual* uses similar language on redundancy (FDOT 2015b).

Because of the significant under-design of the nodal region by FIGG, the AASHTO-recommended redundancy factor of 1.05 would not have prevented the bridge collapse. Although FIGG should have recognized that the diagonal truss members were nonredundant and used a redundancy factor of at least 1.05 in its design, there is no AASHTO or FDOT guidance on redundancy specific to concrete structure design. Further, these resources on redundancy in bridge design focus on steel bridges—not the unique design of the concrete truss pedestrian bridge.

Neither the *AASHTO LRFD Bridge Design Specifications* nor the FDOT *Structures Manual* design guidelines discuss specific redundancy requirements for concrete structures. In addition, the *LRFD Guide Specifications for the Design of Pedestrian Bridges* (AASHTO 2009) does not discuss redundancy. The NTSB concludes that, given the serious consequences of the error made by FIGG in assuming that the bridge had a redundant design, when it did not, and the current lack of guidance concerning redundancy design in concrete and pedestrian bridges, design specification publications for concrete and pedestrian bridges should be revised to include redundancy guidance. The NTSB recommends that AASHTO add a discussion about redundancy in the design of concrete structures to section 5 of the *LRFD Bridge Design Specifications*. The NTSB also recommends that AASHTO add a discussion about redundancy to the *LRFD Guide Specifications for the Design of Pedestrian Bridges*, emphasizing uncommon bridge structures. Further, the NTSB recommends that FDOT add a discussion about redundancy to the *Structures Manual, Structures Design Guidelines,* emphasizing uncommon bridge designs, as determined by FDOT.

3 Conclusions

3.1 Findings

1. The emergency response by local fire departments and law enforcement personnel was timely and adequate.
2. The concrete and steel materials used during construction of the pedestrian bridge were not a factor in its collapse.
3. The hydraulic jack used to post-tension the steel rods in member 11 was operating as expected at the time of the bridge collapse.
4. (1) The FIGG Bridge Engineers (FIGG) bridge design was nonredundant because it provided only a singular load path, (2) FIGG used poor judgment when it determined the bridge was a redundant structure, and then, (3) FIGG erroneously used a redundancy factor of 1.00, which is commonly used for structures with redundant load paths.
5. Even if the cold joint surface of nodal region 11/12 had been roughened to a 0.25-inch amplitude, node 11/12 would not have had sufficient capacity to counteract the demand load for interface shear—and the bridge would still have been under-designed and could have failed.
6. The FIGG Bridge Engineers construction plans inconsistently identified when intentionally roughened surfaces were needed to fulfill the assumptions of the bridge design.
7. Because FIGG Bridge Engineers (1) did not use the lower bound load factor for determining the governing net compression, $\boldsymbol{P_c}$, in the interface shear; and (2) incorrectly increased and amplified the effects of the clamping force across the interface shear surface, its bridge design calculations resulted in a significant overestimation of capacity.
8. FIGG Bridge Engineers (1) made significant design errors in the determination of loads, leading to a severe underestimation of the demands placed on critical portions of the pedestrian bridge; and (2) significantly overestimated the capacity of the member 1/2 and 11/12 nodal regions.
9. Based on analytical modeling results, FIGG Bridge Engineers should have considered the loadings from all critical construction stages when designing the pedestrian bridge and determining the governing interface shear demands.
10. In several instances throughout the bridge design process, FIGG Bridge Engineers models produced reasonable estimations for interface shear demand, but these values were not always used in the design of truss members to resist force demands.
11. FIGG Bridge Engineers’ analytical modeling for the bridge design resulted in a significant underestimation of demand at critical and highly loaded nodal regions.
12. The concrete distress initially observed in nodal region 11/12 is consistent with the underestimation of interface shear demand and the overestimation of identified capacity in the bridge design.

13. The FIGG Bridge Engineers design of the rebar placement in node 11/12 resulted in less reinforcing steel being available and diminished resistance to the critical interface shear demand, which contributed to the collapse of the bridge.

14. The member 11/12 nodal region contained nonstructural voids (four hollow vertical pipe sleeves and the horizontal drain pipe) within the concrete that made it less able to resist applied loads, which contributed to the destabilization of this node through overstress and the subsequent collapse of the main span.

15. Although it may be generally accepted that concrete itself is susceptible to cracking, the rate of premature concrete distress was clear evidence that the structure was progressing toward failure and should have alerted FIGG Bridge Engineers and MCM to the origin of the distress mechanism that was causing the cracking and the rapidity of cracking progression.

16. Louis Berger was not qualified by the Florida Department of Transportation to conduct an independent peer review and failed to perform an adequate review of the FIGG Bridge Engineers design plans and to recognize the significant under-design of the steel reinforcement within the 11/12 node, which was unable to resist the horizontal shear between diagonal 11 and the bridge deck.

17. FIGG Bridge Engineers' failure to adhere to the Florida Department of Transportation *Plans Preparation Manual* requirements for a complex category 2 bridge structure within its work proposal to MCM, calling for an independent firm to conduct a comprehensive peer review, led to the inadequate peer review performed by Louis Berger, which failed to detect the under-design of the bridge.

18. Had the Florida Department of Transportation *Plans Preparation Manual* called for all nodal forces of category 2 bridge structures to be checked and verified by a qualified independent peer review, this collapse might have been prevented.

19. As part of its oversight of local agency program projects and new construction, the Florida Department of Transportation should have verified Louis Berger's qualifications as an independent peer review firm for complex bridge design-concrete, upon receiving the 100 percent certification letters for the bridge foundation, substructure, and superstructure plans.

20. FIGG Bridge Engineers did not perform its due diligence when it contracted with Louis Berger for the independent peer review of the highly complex and uncommon concrete bridge design.

21. The restressing of member 11 was a manipulation of loads that constituted a change to the FIGG Bridge Engineers design, and, before being implemented, should have been independently peer reviewed and signed and sealed by a professional engineer.

22. The structural cracking and northward dislocation of the upper part of the member 11/12 nodal region, as documented in the days leading up to the collapse, was strong evidence that the structure was progressing toward failure; and the detensioning of the post-tensioning rods located in member 11 significantly increased the damage to the member 11/12 nodal region.

23. Although the FIGG Bridge Engineers engineer of record and design manager were engaged by MCM to assess the increased cracking of the structure, they neither recognized that the singular load path in this nonredundant bridge had been compromised nor took appropriate action to mitigate the risk of failure.

24. Beginning with the cracking identified on February 24, 2018, the distress in the main span structure was active, continued to grow, and was well documented by all parties involved in the design, construction, and oversight of the bridge.

25. Neither Florida International University, MCM, FIGG Bridge Engineers, nor Bolton, Perez and Associates Consulting Engineers took the responsibility for declaring that the cracks were beyond any level of acceptability and did not meet Florida Department of Transportation standards.

26. Under the terms and conditions of the contract, Bolton, Perez and Associates Consulting Engineers had the authority to direct or authorize partial or complete road closures as necessary, acting in concert with the Florida Department of Transportation and Florida International University; however, none acted to close the road under the bridge, contributing to the severity of the impact of the bridge collapse.

27. Local agency program agreements require stronger language to clarify that the certified local agency has the authority to immediately close a bridge when structural cracks are first detected or in situations that require further investigation to protect the health, safety, and welfare of the public.

28. Given the pedestrian bridge's unique, nonredundant design, the Florida Department of Transportation should have ensured that the local agencies involved in the project had adequate staff who were trained and experienced in administering these types of uncommon bridge designs.

29. The Florida Department of Transportation should have provided greater oversight of this complex local agency program project to ensure that all safety issues were identified and addressed.

30. Given the serious consequences of the error made by FIGG Bridge Engineers in assuming that the bridge had a redundant design, when it did not, and the current lack of guidance concerning redundancy design in concrete and pedestrian bridges, design specification publications for concrete and pedestrian bridges should be revised to include redundancy guidance.

3.2 Probable Cause

The National Transportation Safety Board (NTSB) determines that the probable cause of the Florida International University (FIU) pedestrian bridge collapse was the load and capacity calculation errors made by FIGG Bridge Engineers, Inc., (FIGG) in its design of the main span truss member 11/12 nodal region and connection to the bridge deck. Contributing to the collapse was the inadequate peer review performed by Louis Berger, which failed to detect the calculation errors in the bridge design. Further contributing to the collapse was the failure of the FIGG engineer of record to identify the significance of the structural cracking observed in this node before the collapse and to obtain an independent peer review of the remedial plan to address the cracking. Contributing to the severity of the collapse outcome was the failure of MCM; FIGG; Bolton, Perez and Associates Consulting Engineers; FIU; and the Florida Department of Transportation to cease bridge work when the structure cracking reached unacceptable levels and to take appropriate action to close SW 8th Street as necessary to protect public safety.

4 Recommendations

As a result of its investigation, the National Transportation Safety Board makes the following new safety recommendations.

To the Federal Highway Administration:

Assist the American Association of State Highway and Transportation Officials with developing a requirement that concrete bridge structures be designed with reasonable estimates for interface shear demand, the cohesion and friction contributions to interface shear capacity, and the clamping force across the interface shear surface. (H-19-24)

To the Florida Department of Transportation:

Revise your *Plans Preparation Manual* to require that the qualified independent peer review for category 2 bridge structures include checking and verifying the design calculations used for all nodal forces. (H-19-25)

Revise your *Plans Preparation Manual* to require the engineering firm or company independently peer-reviewing bridge design plans to submit a prequalification letter showing that it is qualified in accordance with *Florida Administrative Code* Rule 14-75 before permitting the firm to sign and seal the 100 percent certification letters indicating that the bridge designs have been peer reviewed. (H-19-26)

Revise local agency program agreements to specify that when structural cracks are initially detected during bridge construction, the engineer of record, construction engineering inspector, design–build firm, or local agency that owns or is responsible for the bridge construction must immediately close the bridge to construction personnel and close the road underneath; fully support the entire bridge weight using construction techniques that do not require placing workers on or directly under the bridge during installation; and restrict all pedestrian, vehicular, and construction traffic on the bridge until the complete support is in place and inspected. (H-19-27)

To help facilitate compliance with Florida Department of Transportation standards, require your personnel to monitor and inspect all local agency program bridge projects determined by the department to have uncommon designs. (H-19-28)

Add a discussion about redundancy to the *Structures Manual*, *Structures Design Guidelines*, emphasizing uncommon bridge designs, as determined by the Florida Department of Transportation. (H-19-29)

To the American Association of State Highway and Transportation Officials:

> Work with the Federal Highway Administration to develop a requirement that concrete bridge structures be designed with reasonable estimates for interface shear demand, the cohesion and friction contributions to interface shear capacity, and the clamping force across the interface shear surface. (H-19-30)
>
> Add a discussion about redundancy in the design of concrete structures to section 5 of the *LRFD [Load and Resistance Factor Design] Bridge Design Specifications*. (H-19-31)
>
> Add a discussion about redundancy to the *LRFD [Load and Resistance Factor Design] Guide Specifications for the Design of Pedestrian Bridges*, emphasizing uncommon bridge structures. (H-19-32)

To FIGG Bridge Engineers, Inc.:

> Train your staff on the proper use of P_c (the permanent net compressive force normal to the shear plane) when calculating nominal interface shear resistance. (H-19-33)
>
> Institute a company policy to obtain a prequalification letter before finalizing any peer review contract with any engineering firm or company being considered to conduct peer review services. (H-19-34)

BY THE NATIONAL TRANSPORTATION SAFETY BOARD

ROBERT L. SUMWALT, III
Chairman

JENNIFER HOMENDY
Member

BRUCE LANDSBERG
Vice Chairman

Report Date: October 22, 2019

Board Member Statement

Vice Chairman Landsberg filed the following concurring statement on October 28, 2019.

Concurring Statement of Vice Chairman Bruce Landsberg re Miami Pedestrian Bridge Collapse

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A bridge-building disaster should be incomprehensible in today's technical world. We have been building bridges in this country for over two hundred years, and long before that in other parts of the world. The science should be well sorted out by now – and for the most part, it is. The investigation clearly highlighted basic design flaws and a *complete lack of oversight by every single party* that had responsibility to either identify the design errors or stop work and call for a safety stand-down, once it was clear that there was a massive internal failure.

The "what" is very clear but the "why" is more elusive. Despite the public's anger, distress, and disappointment, none of the responsible organizations had any intent for this tragic event to occur or to cause any injury or loss of life. Sadly, good intentions do not suffice for competence and diligence.

Engineering schools will use this as a landmark case study for years – and they should. The Engineer of Record (EOR) employed by FIGG Bridge Engineers, Inc., was experienced, but his calculations were erroneous. Reflection on this event should go far beyond merely a technical review. The checks and balances that were required by the Florida Department of Transportation (FDOT) and American Association of State Highway and Transportation Officials guidance and incumbent upon Louis Berger (LB), the peer-reviewing organization, were completely lacking. LB lowered their bid to review the project by 43 percent in order to get the business, but also reduced the scope of the review. The reason given was there wasn't enough money in the project to cover their efforts. That's both disingenuous and unconscionable. It also was in violation of FDOT's requirement that there be an independent second set of eyes to review everything – not just what was economically convenient.

It is likewise incomprehensible and unethical that LB would even bid on a job for which they lacked the requisite qualifications. That FDOT, which was supposed to review the plans, did not know, or think to ask, about their qualifications is more than just an oversight. It's just plain sloppy. Ditto for FIGG. FDOT claimed a technical error on the FDOT website and then, after the collapse, fabricated a disclaimer that they are not responsible for the data that they post. That's unacceptable in my view - either ensure the information is accurate or don't post it.

The bridge was not properly designed, and there was no qualified oversight on that design. When the inevitable began to happen – a creeping, catastrophic material failure, nobody did anything, despite what NTSB Chairman Sumwalt accurately described as the "bridge screaming at everyone that it was failing." Why?

Once the cracking became evident, *not one of the organizations involved was willing to take the essential and unpopular step to call a halt* and close the road. This is similar to the circumstances of the space shuttle Challenger disaster where the decision was made to launch in extremely cold weather. The engineers recommended against it because the O-Rings that were critical to fuel system integrity would be operating outside their design parameters. Rationalization, optimism, and schedule pressure contributed to what has been described in management training circles as "Group Think." Strong and confident personalities persuade everyone that everything will be OK. Despite misgivings and technical expertise that advise against such action, the team moves forward as a group.

It appears that the same mindset was in play here, in every organization: FIGG, LB, MCM (the construction company), Bolton Perez (the engineering firm overseeing the bridge construction), FDOT, and finally, Florida International University. It also appears that every organization absolved themselves of responsibility by rationalizing that if the EOR says it's OK, it must be OK, and if anything bad happens – it's on him. That is not the intent of peer review or safety oversight, and certainly fails the system of checks and balances in place to prevent catastrophes like these.

We've learned this the hard way too many times in transportation modes. The NTSB's stated role is not to lay blame, but some would say that's exactly what we do when we apportion causation. The human failing that affects all of us is complacency. It is not a term the NTSB uses often, but in my opinion, it is present in nearly every accident and crash. We are creatures of habit, and when we become comfortable through long repetitive experience, the guard often comes down - periodically with disastrous results. This is precisely what safety management systems are designed to prevent – to trap errors in process before they become catastrophes. While disasters may be perfectly clear in hindsight, the best organizations take proactive measures - constantly. Schedule pressure, economics, overconfidence, and complacency all work to counter good intentions and too often create tragedy.

It is my fervent hope that the organizations involved will take the NTSB recommendations seriously and quickly implement them. The lives lost and the families disrupted deserve at least that much.

Appendix A: Investigation

The National Transportation Safety Board (NTSB) was notified of the Miami, Florida, pedestrian bridge collapse on March 15, 2018, and dispatched an investigative team to the site. The NTSB established groups to investigate bridge, vehicle, and survival factors. The Federal Highway Administration (FHWA) supported the NTSB in its investigation by providing resources and expertise to evaluate the bridge design and construction processes. Additionally, the FHWA examined and tested physical evidence, including concrete and steel samples, as well as associated construction equipment. The FHWA team consisted of personnel from the Office of Bridges and Structures, the Turner-Fairbank Highway Research Center, the Resource Center, and the Florida Division Office.

Parties to the investigation were the FHWA; FHWA Turner-Fairbank Highway Research Center; US Department of Labor, Occupational Safety and Health Administration (OSHA); US Department of Transportation, Office of the Inspector General; Florida Department of Transportation; Miami-Dade Police Department; Florida Highway Patrol; Florida International University; City of Sweetwater, Florida; Barnhart Crane and Rigging; Bolton, Perez and Associates Consulting Engineers; FIGG Bridge Engineers; MCM; and Structural Technologies.

On June 14, 2019, the NTSB revoked OSHA party status because of a breach of party participation rules. On June 11, contrary to party agreement obligations, OSHA released a report to the public that contained large portions of nonpublic draft NTSB material and failed to provide investigative photographs to the NTSB as required by its status as a party to the investigation.

Chairman Robert L. Sumwalt, III, was the NTSB spokesperson on scene.

Appendix B: T.Y. Lin Report Specifications

2 Specifications and References

All work shall conform to current versions of the following documents. The lists below are in order of precedence.

2.1 FDOT References

- *Standard Specifications for Road and Bridge Construction (Specifications)*
- *Structures Manual (SDG)*
- *Plans Preparation Manual (PPM)*

2.2 AASHTO Specifications

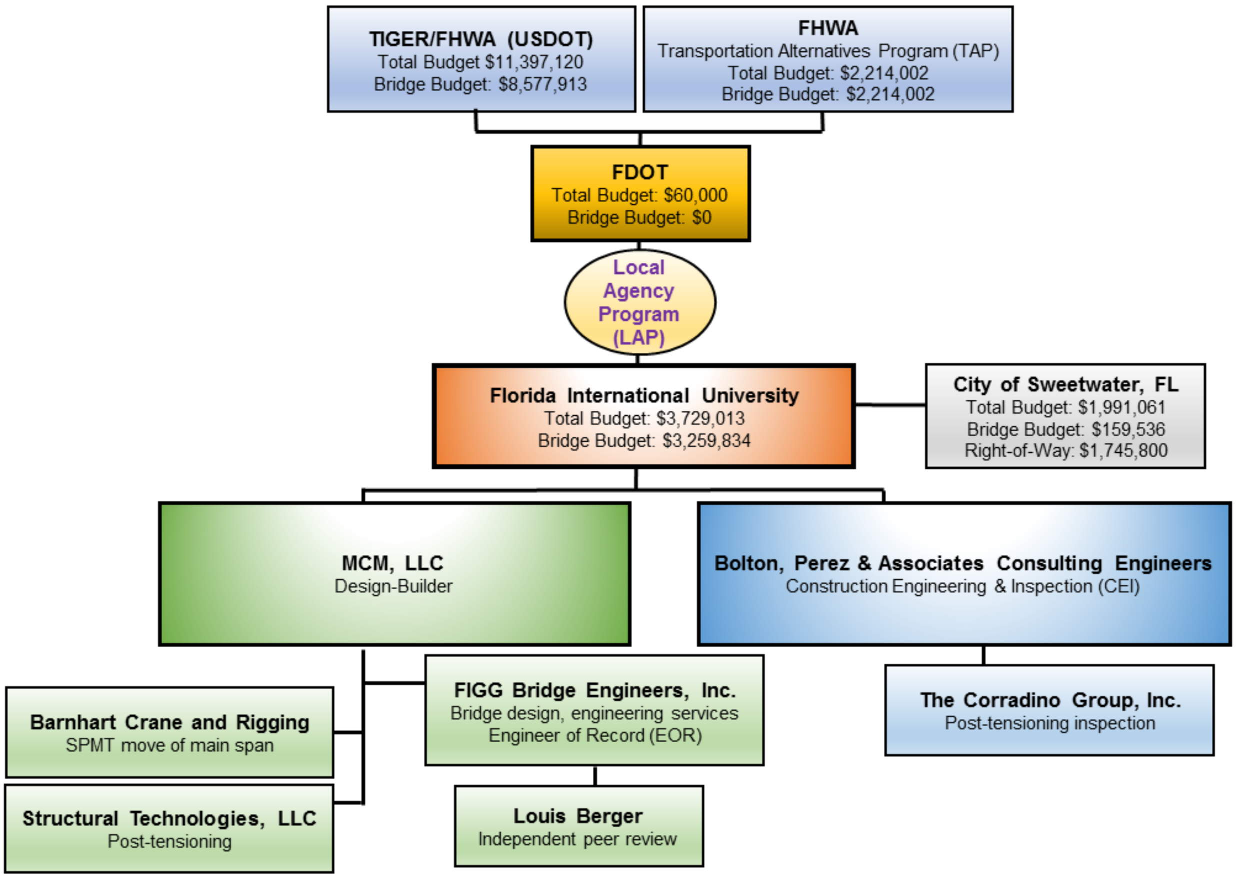
- *LRFD Guide Specification for the Design of Pedestrian Bridges*
- *AASHTO LRFD Bridge Design Specifications (LRFD)*
- *AASHTO/AWS D1.5M/D1.5:2002 Bridge Welding Code*
- *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*
- *AASHTO Guide Design Specifications for Bridge Temporary Works*

2.3 FHWA References

- *FHWA-NHI-05-046, Earth Retaining Structures*
- *FHWA-HI-99-007, Rock Slopes*
- *FHWA-NHI-01-023, Shallow Foundations*
- *FHWA-IF-99-025 Drilled Shafts: Construction Procedures and Design Methods Manual, 1999*

Appendix C: Government–Industry Participation

This flowchart shows the organizational structure of government and industry participants in design and construction of the Florida International University pedestrian bridge.



Appendix D: Construction Phases

Construction stage 1, *substructure casting*, work involved the following steps:

1. Build pier 1 and pier 3 footings at the south and north landings.
2. Build pylon footing and base of pylon.
3. Cast the end bents for both landings.

Construction stage 2, *superstructure precasting*, work involved the following steps:

1. Cast main span superstructure as follows:
   a) Cast deck and diaphragms.
   b) Cast diagonal and vertical members, installing PT bars as shown in sheet B-38.
   c) Cast canopy and top anchor blocks.

2. After concrete compressive strength has reached 6,000 psi, stress post-tensioning of the main span in the following sequence:
   a) Stress deck longitudinal tendons D1.
   b) Stress canopy longitudinal tendons C2.
   c) Stress PT bars in diagonal members 2 and 11.
   d) Stress deck longitudinal tendons in the following sequence: D2, D3, D4, D5, and D6.
   e) Stress bottom slab transverse post-tensioning. Alternated end stressing is required for the transverse tendons.
   f) Stress PT bars in diagonal members 3 and 10.
   g) Stress PT bars in diagonal members 5 and 8.
   h) Stress PT bars in diagonal members 6 and 7.
   i) Stress canopy longitudinal tendons C3.

3. Temporary supports of main span section shall stay in the middle of the cross section during self-propelled modular transporter (SPMT) transport.

Construction stage 4, *casting of back span*, work involved the following steps:

1. Erect temporary beam and falsework.
2. Install bearing pads at end bent 3.
3. Cast intermediate section of the pylon.
4. Cast deck, diagonal member, vertical members, canopy, and top anchor blocks.
5. After concrete compressive strength has reached 6,000 psi, stress post-tensioning of the back span in the following sequence:
   a) Deck longitudinal tendons D7.
   b) Canopy longitudinal tendons C5.
   c) PT bars in diagonal members 15 and 23.
   d) PT bars in diagonal members 16 and 22.
   e) PT bars in diagonal members 17 and 21.
   f) PT bars in diagonal members 18 and 20.
   g) PT bars in diagonal member 19.

h) Deck longitudinal tendons D8 and D9.
i) Bottom slab transverse post-tensioning (transverse tendons require alternated end stressing).

Construction stage 5, *continuity tendons and casting of upper pylon*, work involved the following steps:

1. Install continuity tendons C1 and C4.
2. Cast closure pours in the deck and canopy.
3. After closure pour concrete compressive strength has reached 6,000 psi, stress continuity tendons C1 and C4.
4. Remove falsework over the canal.
5. Stress transverse tendon in the closure of the deck.
6. Cast upper pylon section and north landing deck.
7. Stress transverse tendons of the north landing.

Construction stage 6, *install pipe support system*, work involved the following steps:

1. Connect steel pipes to the superstructure and upper pylon, connecting pipes adjacent to the pylon first.
2. Cast fence concrete curbs on both spans.

Construction stage 7, *installation of bridge components*, work involved the following steps:

1. Install missile fence.
2. Install expansion joints at end bent 1 and north landing.
3. Install bridge lighting and drainage systems.

Construction stage 8, *installation of landings*, work involved the following steps:

1. Build elevator structures and install elevator systems at both landings.
2. Construct stairways.
3. Install expansion joint at south landing canopy.

Appendix E: Crack Reports and Email Exchanges

Table E-1. Language of selected email exchanges between Bolton, Perez; MCM; and FIGG; February 13–March 14, 2018.

Email Originator and Recipient	Email Language (direct quotes)
February 13, 2018	
Bolton, Perez crack report 1 to MCM	The members showing these small cracks are truss members that share the same blister at the canopy of the already stressed members No 2 (stressed 1/30/18) and No 11 (stressed 1/29/18). We believe, this first stressing operation has temporarily created tension on members No 3 & No 10; thus, creating cross sectional cracks transferring the tension loads to the steel on these members. No other truss members within span 1 show any cracks similar to these shown on members No 3 & No 10. The intent of the report is to inform Design Build Team of these cracks. It is the Design Build responsibility to assess them and determine if these cracks were expected while tensioning and monitor them accordingly if deemed necessary.
February 16, 2018	
FIGG response to Bolton, Perez crack report 1	FIGG received the *Crack Inspection Report* prepared by CEI on February 13, 2018. Subsequent to receiving the report, MCM sent us an e-mail clarifying the location of the reported observations on February 15, 2018. FIGG has reviewed the report and offers the following comments for your consideration: • 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. • The release of the canopy falsework will improve the state of stress in members 3 and 10. • As mentioned in CEI’s report, the observations regarding the current condition of members 3 and 10 are the results of an intermediate step in the stressing operation. • It is recommended that the truss members not be marked with a marker/sharpie as this will lead to discoloration of the concrete.
February 28, 2018	
Bolton, Perez crack report 2 to MCM	Please refer to the pictures attached regarding some cracks seen on truss members of Span 1 after the removal of the formwork. Forward to the EOR for their information. We will monitor these or any other developing cracks on the bridge, but we would like to [sic] the EOR to provide a response and determine if these were expected during the bridge stressing. The one due the [sic] size that we believe needs special attention is the crack shown in photos 5, 6 & 7.

March 7, 2018	
FIGG notified MCM of the unclear locations of crack photographs	FIGG received a Crack Inspection Report prepared by CEI on February 28, 2018. We have evaluated these reports and have the following general comment for your consideration: • CEI Report only contains three (3) pages of photos and one (1) page that shows the elevation view of Span 1 titled *Cracks Location*. The general locations of the cracks are depicted on this elevation view. Using the information provided by CEI, it is not clear from which side of the span the photos were taken since there is not a description of the location for each photo. Therefore, FIGG requests a more descriptive report of the crack locations and width.
FIGG provided MCM with preliminary comments to share with Bolton, Perez	• Photo number 1 appears to be a delamination of the concrete surface of member number 1 near the canopy of Span 1. FIGG does not have a structural concern about this type of imperfection in the finish of the member. MCM will have to repair this imperfection after Span 1 is moved to its final position. • Photo number 2 shows two very small cracks adjacent to the temporary top hinge in vertical member 1. FIGG is not concerned about these very small cracks in this region. Cracks near the temporary hinge were expected. • The elevation of Span 1 shows that photo 3 was taken from the underside of diaphragm number 1. However, the photo appears to be taken at the deck level and its orientation seems to be parallel to the longitudinal axis of member number 2. FIGG is not concerned about this type of crack which seem to be very small and confined to the reinforced concrete chamfer region. • The cracks that are shown on photos 4, 5 and 6 appear to be in the reinforced concrete chamfer region of member 11. The elevation view locates these cracks on the downstation side of diaphragm number 2. It appears that the arrow that shows the general location of the crack was misplaced in the elevation view. These cracks developed at the boundary between the diagonal compression member and its reinforced concrete bottom chamfer. It is anticipated that MCM will seal these cracks in accordance with FDOT Standard Specifications.
March 12, 2018	
MCM email (4:51 p.m.) to FIGG requesting course of action	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.
March 13, 2018	
Internal FIGG emails	• At 11:58 a.m., EOR to FIGG employee: "*Can you ask if any of the cracks were noted before the temporary diagonal bars were destressed, or not.*" • At 1:44 p.m., FIGG employee to EOR: *"…said that cracks were observed prior to detensioning then grew slightly once PT bars were destressed."*

FIGG email response to MCM (9:45 a.m.) regarding March 12 email from MCM	As you and I just discussed, Figg is evaluating this situation as a top priority and will be making recommendations as a result of this evaluation. As of right now, we do not see this as a safety issue but we do recommend that MCM place plastic shims (same as currently being used) underneath the Type 2 diaphragm at the centerline of bridge (this is a 2'-10.5" x 21" area). The shim stack height should be sized to bear against both the top of lower pylon and the bottom of the type 2 diaphragm. Below is a list of facts and other coordination items from our discussion; 1. MCM observed cracks in the Type 2 diaphragm on Saturday afternoon after the SPMT were driven back to the staging area and before the temporrry [sic] pt bars were destressed. It was noted that Figg Inspection of the main span in this area after the bridge move did not observe this behavior. It is not clear as to when this behavior occurred. 2. MCM has destressed the temporary PT bars in the main span. 3. Since Saturday afternoon, MCM has been monitoring the cracks and they have not grown in size. 4. This behavior is only being observed on the north face of the type 2 diaphragm. It is not seen on the south face. MCM to send Figg pictures of the south face of the Type 2 diaphragm and label pictures. 5. MCM will take pictures of the bottom face of the Type 2 diaphragm from both north face (east and west side), south face (east and west side) and east and west face. These pictures are to show the condition of the bottom face and also show the location of the shim stacks to the diaphragm. 6. MCM is to place plastic shims under the Type 2 diaphragm/vertical strut. This is a 2'-10.5" x 21" area to be shimmed. Shims to be placed tight against the top of lower pylon and bottom of type 2 diaphragm. No jacking of bridge is required. These shims need to be placed right away. Figg will be back in contact with MCM to give updates and recommendations from evaluations.
Bolton, Perez crack report 3 to MCM (10:59 a.m.)	As discussed earlier, I recommend we monitor and document the growth of these cracks to determine if these are active and developing further or dormant. Please let us know the outcome of the EOR analysis and course of action.
FIGG response to MCM (5:18 p.m.)	As you and I just discussed, please find the additional recommendations and requests below that FIGG thinks will be beneficial for the structure. Again, we have evaluated this further and confirmed that this is not a safety issue. 1. It is recommended to reinstall the (2) 1-3/8" temporary pt bars in truss member 11 as shown on plan sheet B-38. These are oriented with one bar at top and one bar at bottom of the member section. The temporary pt bars in truss member 2 do not need to be reinstalled or restressed. 2. Both pt bars should be stressed to the 280 kips stressing force as listed on plan sheet B-69 and these bars should be stressed in 50 kip increments each, starting with the top pt bar, then bottom pt bar, then back to the top pt bar, etc. The type 2 diaphragm should be closely monitored during this pt bar stressing process to ensure that the crack size does not increase. Based on our evaluation, we anticipate that the crack size will either remain the same or more probably decrease in size. If the crack size increases, the pt bar stressing shall stop and FIGG be notified immediately.

	3. We understand that MCM was to contact [Structural Technologies] to see when they could be on site to perform pt bar stressing. FIGG recommends to stress these pt bars as soon as possible but again, this is not a safety concern. 4. We request to receive [sic] the concrete break reports from the lab for the bridge deck placement. 5. We understand that MCM is currently placing the shims under the Type 2 diaphragm at the centerline of bridge and will send pictures once complete. MCM will also send pictures of the existing shim stacks to show orientation of shim stack to Type 2 diaphragm.
March 14, 2018	
MCM reply to FIGG (1:38 p.m.)	As we have been discussing, attached please find additional photos for your reference. In addition, FIU/CEI are confirmed for tomorrow at 9:00 a.m. to meet FIGG's team. Lastly, see comments in ***red*** below. [Responses in red are to FIGG's 5:18 p.m., March 13, email]: 1. It is recommended to reinstall the (2) 1-3/8" temporary pt bars in truss member 11 as shown on plan sheet B-38. These are oriented with one bar at top and one bar at bottom of the member section. The temporary pt bars in truss member 2 do not need to be reinstalled or restressed. 2. Both pt bars should be stressed to the 280 kips stressing force as listed on plan sheet B-69 and these bars should be stressed in 50 kip increments each, starting with the top pt bar, then bottom pt bar, then back to the top pt bar, etc. The type 2 diaphragm should be closely monitored during this pt bar stressing process to ensure that the crack size does not increase. Based on our evaluation, we anticipate that the crack size will either remain the same or more probably decrease in size. If the crack size increases, the pt bar stressing shall stop and FIGG be notified immediately. 3. We understand that MCM was to contact [Structural Technologies] to see when they could be on site to perform pt bar stressing. FIGG recommends to stress these pt bars as soon as possible but again, this is not a safety concern. ***Is there a time frame for this? FYI, [Structural Technologies] has been contacted and their crews are currently out of town and are waiting availability confirmation***. 4. We request to receive the concrete break reports from the lab for the bridge deck placement. ***See attached.*** 5. We understand that MCM is currently placing the shims under the Type 2 diaphragm at the centerline of bridge and will send pictures once complete. MCM will also send pictures of the existing shim stacks to show orientation of shim stack to Type 2 diaphragm. ***Shims were installed yesterday (see photos attached; however, if these are consider [sic] temporary, we may need to jack to get them out. We can discuss it further with your team tomorrow inclusive of the grout on this area.***

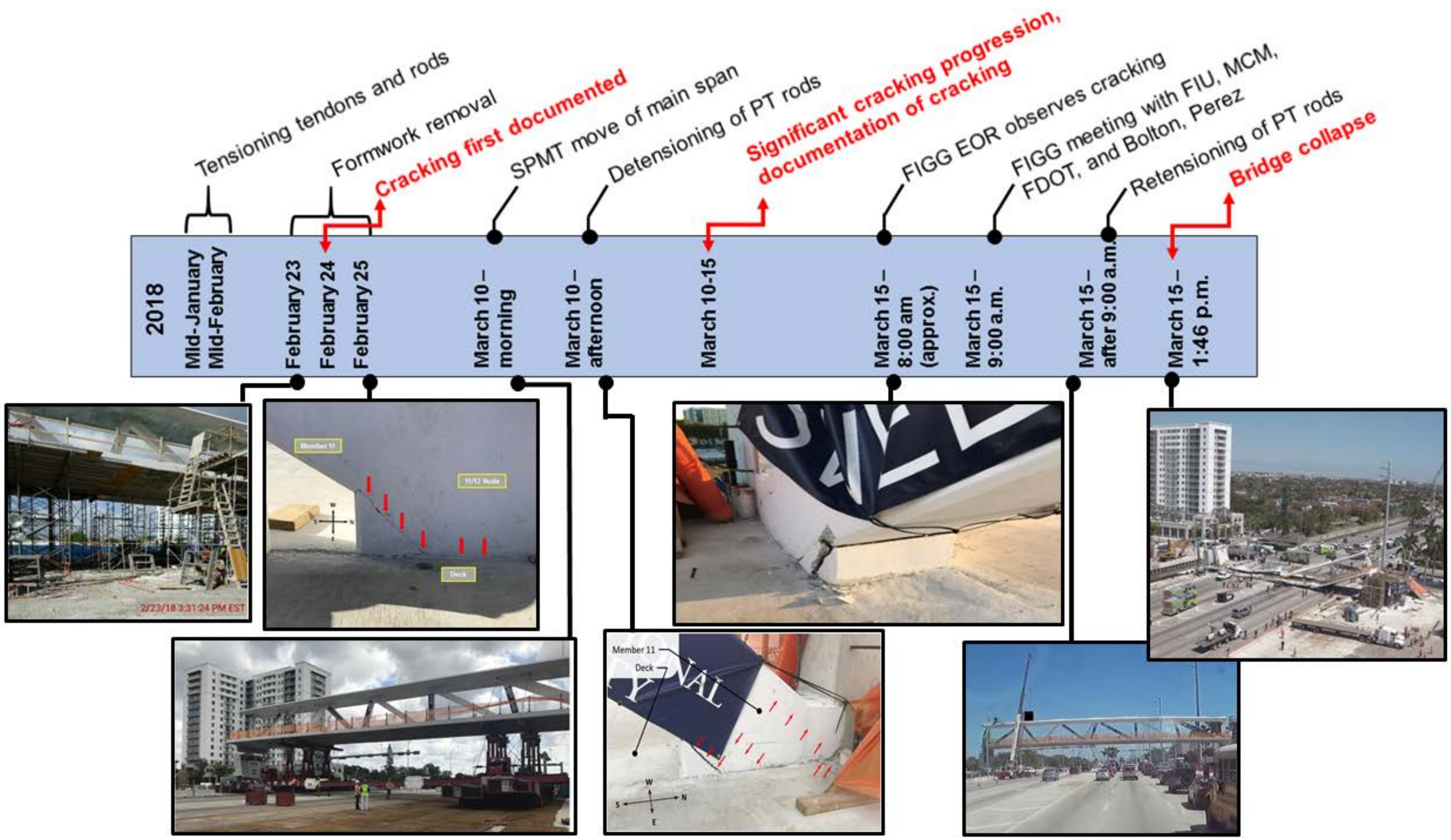


Figure E-1. Timeline of precollapse events.

Appendix F: Interface Shear Calculation Data

The four FHWA models focus on the bridge in the construction stage at the time of collapse (that is, the end of stage 2, the same stage as was assessed in the FIGG Simple Support Model). Three of these models were two-dimensional, the fourth was three-dimensional. The two-dimensional models (#1 and #2) are grid analysis models comprising beam elements with post-tensioning and nonlinear time-dependent material effects considered. The third two-dimensional model is a grid analysis model comprising truss elements in which the post-tensioning effects are superimposed. The fourth model, the three-dimensional model, is a finite element model comprising solid elements with the post-tensioning sequence considered.

The design of the nodal zones (such as the nodal region of truss members 11–12) for the bridge main span used results from the two LUSAS models for the V_{DC}, V_{LL}, and V_{PT} force effects. The larger of the interface shear forces generated by the models at each nodal zone were used. The FIGG LUSAS fixed pylon model generated the larger interface shear force effects for nodal zones at members 7–8 and 11–12. The FIGG LUSAS simple support model generated the larger interface shear force effects for the other nodal zones.

V_{TU+TD} force effects were taken from the FIGG LARSA longitudinal model for all the nodal zones. Values for these force effects are located on pages 934–936 in the superstructure final design calculations. Table F-1 lists the LUSAS model used and corresponding pages in the superstructure final design calculations for the output for V_{DC}, V_{LL}, and V_{PT}. The force effects identified in table F-1 were multiplied by their load factors to calculate the factored interface shear demand V_{ui}. Table F-1 also summarizes the factored interface shear demand calculated for each nodal zone.

Table F-1. Analysis model/superstructure design calculation output location and interface shear summary by nodal zone.

Nodal Region	Model and LUSAS Output Pages for $V_{DC} + V_{LL} + V_{PT}$ Force Effects[a]	Factored Interface Shear Demand V_{ui} (kips) [a,b]
1/2	LUSAS simple support (p. 1382)	1,368 (p. 1283)
2/3	LUSAS simple support (pp. 1285–1287)	1,474 (p. 1284)
3/4	LUSAS simple support (p. 1383)	1,084 (p. 1283)
4/5	Design not included in final calculations	Design not included in final calculations
5/6	LUSAS simple support (p.1384)	491 (p. 1283)
6/7	Design not included in final calculations	Design not included in final calculations
7/8	LUSAS fixed pylon (p. 1396)	370 (p. 1283)

8/9	Design not included in final calculations	Design not included in final calculations
9/10	LUSAS simple support (p. 1386)	181 (p. 1283)
10/11	LUSAS simple support (pp. 1288–1290)	133 (p. 1284)
11/12	LUSAS fixed pylon (p. 1398)	978 (p. 1283)

[a] Page numbers from superstructure final design calculations; see the NTSB public docket for this investigation (HWY18MH009).

[b] 1 kip = 1,000 pounds-force

As stated previously, the interface shear demands calculated for the main span used results from the LUSAS models for all force effects except uniform temperature ($\boldsymbol{V_{TU+TD}}$), which used the LARSA longitudinal model. Although not used in the superstructure final design calculations, the LARSA longitudinal model can generate interface shear demands.

The bridge configuration for the LARSA longitudinal model closely matches the configuration for the LUSAS fixed pylon model. For nodal zone 11–12, the force effects occur in the members but would not be primarily resisted by interface shear because of the resistance provided by the pylon and back span.

The final design calculations used the larger of the interface shear demand values produced by the LUSAS fixed pylon and simple support models. Thus, the LARSA longitudinal model results should be similar to or less than results from the LUSAS models presented in table F-2. For each nodal zone in the main span, table F-2 compares the factored interface shear demands, $\boldsymbol{V_{ui}}$, used in the final design to the factored interface shear demands generated solely from the LARSA longitudinal model, for the Strength I load combination (STR 1/20). Table F-2 shows absolute values for $\boldsymbol{V_{ui}}$.

Table F-2. Comparison of factored interface shear demand modeling.

Nodal Zone	V_{ui} Used in Final Design (calculated using LUSAS models [kips])[a]	V_{ui} Calculated Using LARSA Longitudinal Model (kips)[a]
1–2	1,368	2,683
2–3	1,474	2,719
3–4	1,084	893
4–5	Design not included in final calculations	837
5–6	491	198
6–7	Design not included in final calculations	4
7–8	370	347
8–9	Design not included in final calculations	909

9–10	181	838
10–11	133	2,077
11–12	978	1,990
[a] The LARSA longitudinal model was used to generate V_{TU+TD} force effects. LUSAS models were used to generate all other force effects.		

Table F-3. Nodal region interface shear capacity calculation summary (FIGG design).

Nodal Zone	$A_{cv} = b_{vi} \times L_{vi}$ (inches^2)	A_{vf} (inches^2)	P_c (kips)	*Interface Shear Capacity* $V_{ri} = \varphi V_{ni}$ *(kips)*
C = 0.0 ksi				
1–2	21 x 57.6 = 1,210	8 - #7 = 4.80	1,275	1,407
2–3	21 x 58.0 = 1,218	14 - #6 = 6.16	1,298	1,501
3–4	21 x 85.6 = 1,798	18 - #7 = 10.80	579	1,104
4–5	FIGG design did not calculate for this nodal region			
5–6	21 x 79.3 = 1,664	12 - #7 = 7.20	230	596
6–7	FIGG design did not calculate for this nodal region			
7–8	21 x 68 = 1,428	12 - #7 = 7.20	449	793
8–9	FIGG design did not calculate for this nodal region			
9–10	21 x 67.0 = 1,407	12 - #7 = 7.2	703	1,021
10–11	21 x 58.0 = 1,218	6 - #6 = 2.64	1,210	1,232
11–12	21 x 42.0 = 882	8 - #7 = 4.80	1,233	1,369

Table F-4. Interface shear capacity results from FHWA check.

Nodal Zone	$A_{cv} = b_{vi} \times L_{vi}$ (inches^2)	A_{vf} (inches^2)	P_c (kips)	Interface Shear Capacity $V_{ri} = \varphi V_{ni}$ (kips)
C=0.0 ksi				
1–2	21 x 75.25 = 1,580	10 - #7 = 6.0	925	1,156
2–3	21 x 69.0 = 1,449	12 - #6 = 5.28	1,388	1,534
3–4	21 x 85.6 = 1,798	18 - #7 = 10.80	865	1,361
4–5	21 x 55 = 1,155	10 - #6 = 4.4	411	607
5–6	21 x 79.3 = 1,664	12 - #7 = 7.20	373	725
6–7	21 x 52 = 1,092	10 - #6 = 4.4	617	792
7–8	21 x 68 = 1,428	12 - #7 = 7.20	678	999
8–9	21 x 54 = 1,134	10 - #6 = 4.4	759	921
9–10	21 x 67.0 = 1,407	12 - #7 = 7.20	952	1,246
10–11	21 x 58.0 = 1,218	10 - #6 = 4.4	1,540	1,623
11–12	21 x 42.0 = 882	8 - #7 = 4.80	967	1,130

Pages 934–936 in the superstructure final design calculations provide values for DW1, DC1, FR1, TU3 Diff, and LL2 from the tables below.

Table F-5. Lower node force effects and factored interface shear calculations.

Node	Member	Loads (kips)										
		DW1 (1.50)	DC1 (1.25)	FR1 (1.0)	TU3 Diff (0.5)	LL2 (1.75)	Factored axial Strength Limit State I/20	Compressive Force From PT bars (pg. 842)	Factored axial load with PT bars included	Angle between member and shear plane (deg)	V_{ui}	Sum V_{ui}
1 & 2	2 (Member 705 at Node 703)	82.2	1848.7	-285.8	338.8	352.3	2934.3	0	2934.3	23.9	2683	2683
3 & 4	3 (Member 708 at Node 705)	-34	-608.9	123.1	-58.7	-139.4	-962.325	1120	157.675	71.96	49	893
	4 (Member 711 at Node 707)	16.7	732.1	-229.3	133.9	93.4	941.275	0	941.275	26.24	844	
5 & 6	5 (Member 714 at Node 709)	-13.6	-243.2	120.1	-3.6	-58.8	-309	332	23	88.52	1	198
	6 (Member 717 at Node 711)	-19.8	27.8	-248.2	10.3	-53.5	-331.625	560	228.375	30	198	
7 & 8	7 (Member 720 at Node 713)	2	86	125.4	44.5	4.6	266.2	280	546.2	69.7	189	-347
	8 (Member 723 at Node 715)	-46.3	-587	-186	-75.7	-165.5	-1316.675	1120	-196.675	36.7	-158	
9 & 10	9 (Member 726 at Node 717)	22.2	599.6	108.2	121	89.8	1108.65	0	1108.65	47.44	750	-838
	10 (Member 729 at Node 719)	-55	-870.9	-108.6	-102.2	-209.3	-1697.1	1556	-141.1	51.234	-88	
11 & 12	11 (Member 732 at Node 721)	48.8	1382.1	34.6	311.6	200	2341.225	0	2341.225	31.79	1990	1990

Table F-6. Upper node force effects and factored interface shear calculations.

Node	Member	Loads (kips)										
		DW1 (1.50)	DC1 (1.25)	FR1 (1.0)	TU3 Diff (0.5)	LL2 (1.75)	Factored axial Strength Limit State I/20	Compressive Force From PT bars (pg. 842)	Factored axial load with PT bars included	Angle between member and shear plane (deg)	V_{ui}	Sum V_{ui}
2 & 3	2 (Member 705 at Node 704)	-82.2	-1839.2	285.8	-338.8	-352.3	-2922.4	0.0	-2922.4	23.9	-2672	-2719
	3 (Member 708 at Node 706)	34	614.4	-123.1	58.7	139.4	969.2	-1120	-150.8	71.96	-47	
4 & 5	4 (Member 711 at Node 708)	-16.7	-725.8	229.3	-133.9	-93.4	-933.4	0	-933.4	26.24	-837	-837
	5 (Member 714 at Node 710)	13.6	249.4	-120.1	3.6	58.8	316.8	-332	-15.3	88.52	0	
6 & 7	6 (Member 717 at Node 712)	19.8	-21.5	248.2	-10.3	53.5	339.5	-560	-220.5	30	-191	-4
	7 (Member 720 at Node 714)	-2	-79.5	-125.4	-44.5	-4.6	-258.1	-280	-538.1	69.7	-187	
8 & 9	8 (Member 723 at Node 716)	46.3	593.4	186	75.7	165.5	1324.7	-1120	204.7	36.7	164	909
	9 (Member 726 at Node 718)	-22.2	-593.2	-108.2	-121	-89.8	-1100.7	0	-1100.7	47.44	-744	
10 & 11	10 (Member 729 at Node 720)	55	877.5	108.6	102.2	209.3	1705.4	-1556	149.4	51.234	94	2077
	11 (Member 732 at Node 722)	-48.8	-1375.9	-34.6	-311.6	-200	-2333.5	0.0	-2333.5	31.79	-1983	

- ***Column 1:*** identifies the nodal zone by listing each truss diagonal connecting into the nodal zone.
- ***Column 2:*** identifies the width of the nodal zone (measured transverse to the bridge), labeled $\boldsymbol{b_{vi}}$.
- ***Column 3:*** identifies the length of the nodal zone (measured longitudinally to the bridge), labeled $\boldsymbol{L_{vi}}$.
- ***Column 4:*** identifies the nodal zone interface surface area between the truss diagonals and the deck, labeled $\mathbf{A_{cv}}$—which is calculated by multiplying $\boldsymbol{b_{vi}} \times \boldsymbol{L_{vi}}$.
- ***Column 5:*** identifies the permanent net compression force across the interface surface, labeled $\boldsymbol{P_C}$.
- ***Column 6:*** identifies the shear force (unfactored) acting across the interface surface due to component dead load, labeled $\boldsymbol{V_{DC}}$.
- ***Column 7:*** identifies the shear force (unfactored) acting across the interface surface due to live load, labeled $\boldsymbol{V_{LL}}$.
- ***Column 8:*** identifies the shear force (unfactored) acting across the interface surface due to post-tensioning, labeled $\boldsymbol{V_{PT}}$.
- ***Column 9:*** identifies the shear force (unfactored) acting across the interface surface due to uniform temperature, labeled $\boldsymbol{V_{TU+TD}}$.
- ***Column 10:*** shows the computed factored shear force, $\boldsymbol{V_{ui}}$, which was calculated by multiplying each shear force by its load factor. The displayed value results from: $1.25 \times \boldsymbol{V_{DC}} + 1.75 \times \boldsymbol{V_{LL}} + 1.0 \times \boldsymbol{V_{PT}} + 0.50 \times \boldsymbol{V_{TU+TD}}$.
- ***Column 11:*** shows the computed area of reinforcing steel needed to provide enough capacity so that the factored nominal resistance ($\phi\ \boldsymbol{V_{ni}}$) is greater than the factored interface shear demand.
- ***Column 12:*** lists the reinforcing steel provided in the bridge plans. The syntax of the text is "[number of bars] – [number of legs of each bar] of [size of bar]." Multiplication of [number of bars] times [number of legs of each bar] results in the number of bars of each size that were to be provided across the interface.

Descriptions and page numbers from the superstructure final design calculations for the inputted data for this nodal zone are listed below:

- $\mathbf{b_{vi}}$ = 21.0 inches (dimension shown multiple times on final bridge plans).
- $\boldsymbol{L_{vi}}$ = 42.0 inches (dimension shown multiple times on final bridge plans). This dimension does not include the length of vertical support 12.
- $\boldsymbol{P_C}$ = 1,233 kips (force calculated from the LARSA Longitudinal Model for the completed bridge (see section 28.3 for LARSA model definition)). The $\boldsymbol{P_C}$ force was based on the Strength I load case identified as "STR 1/20" in the LARSA model (see below for STR 1/20 definition). The analysis results for diagonal support 11, identified as member 732 in the LARSA model, were used to generate the $\boldsymbol{P_C}$ value by calculating the vertical component of this member force at the diagonal support 11 to the deck connection (node 721). $\boldsymbol{P_C}$ was calculated using the force effects described below:
    - Dead load of wearing surface and utilities: DW1 = 48.8 kips (page 934).
    - Dead load of structural components: DC1 = 1,382 kips (page 934).
    - Friction load: FR1 = 34.6 kips (page 934).
    - Uniform temperature load: TU3 = 311.6 kips (page 938).
    - Live load: LL2 = 200.0 kips (page 936).
    - The factored axial force in diagonal 11 is computed using the *AASHTO LRFD* Strength I limit state (article 3.4.1-1), which is identified as the STR 1/20 load case in the LARSA model (page 932) and results in: 1.50 × DW1 + 1.25 × DC1 + 1.0 × FR1 + 0.50 × TU3 + 1.75 × LL2 = 2,341.1 kips. $\boldsymbol{P_C}$ is the vertical component of this member force, which can be calculated using the sine of the angle between diagonal 11 and the bridge deck (bridge plans show a 31.79 degree angle). Therefore, $\boldsymbol{P_C}$ = diagonal 11 axial force × sine (31.79) = 2,341.1 kips × sine (31.78) = 1,233 kips.
- $\boldsymbol{V_{DC}}$ + $\boldsymbol{V_{LL}}$ + $\boldsymbol{V_{PT}}$ = 713 kips. These forces were generated by the LUSAS fixed pylon model, which is shown below (see section 28.3 for LUSAS model definition).
- $\boldsymbol{V_{TU+TD}}$ = 55 kips. This force effect was generated by the LARSA longitudinal model for the completed bridge (page 938).
- $\boldsymbol{V_{ui}}$ is calculated by multiplying each shear force by the load factor (1.25 × $\boldsymbol{V_{DC}}$ + 1.75 × $\boldsymbol{V_{LL}}$ + 1.0 × $\boldsymbol{V_{PT}}$ + 0.50 × $\boldsymbol{V_{TU+TD}}$).

Strength I LARSA load combination STR 1/20:

$$STR\ 1/20 = 1.00\ \times (DW1\ \times 1.50 + DC1 \times 1.25 + FR1 \times 1.00 + TU3 \times 0.50 + LL2 \times 1.75)$$

Data shown in blue are calculation input data, and data shown in black are computed data.

Table F-7. Page 1283, superstructure final design calculations.

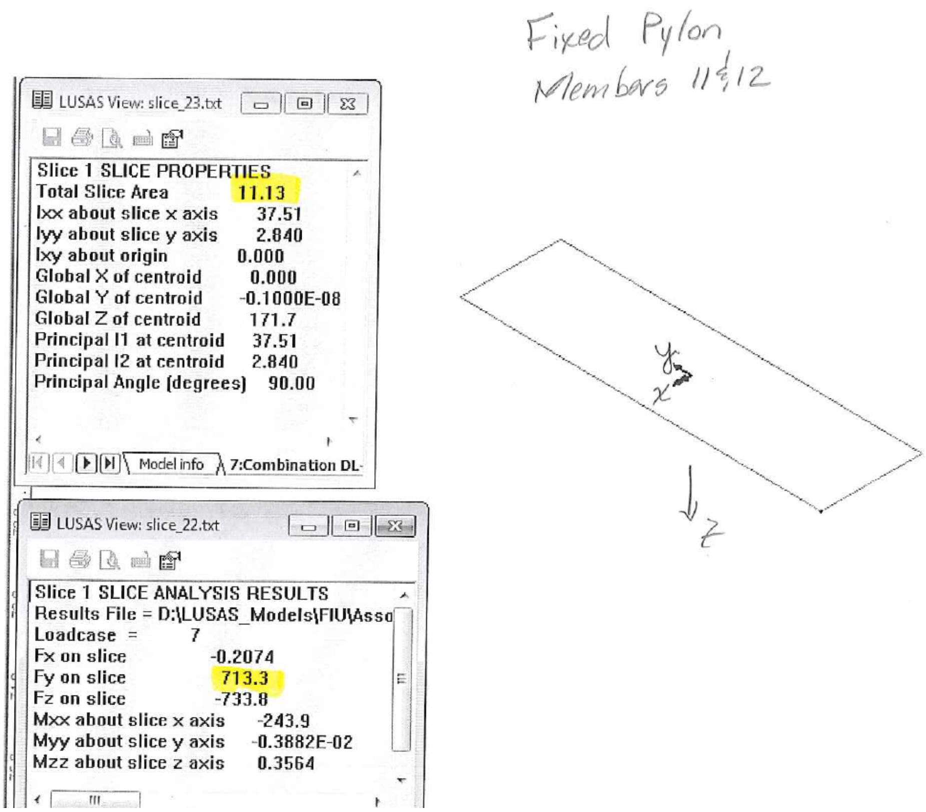
Truss Members	b_{vi} (in)	L_{vi} (in)	A_{cv} (in^2)	P_c (kips)	V_{DC} (kips)	V_{LL} (kips)	V_{PT} (kips)	V_{TU+TD} (kips)	V_{ui} (kips)	A_{vf} (in^2)	A_{vf_prov}
1 & 2	21.0	57.6	1210	1275	670	153	107	310	1368	4.09	4-2 legs of #7
3 & 4	21.0	85.6	1798	579	283	66	564	102	1084	10.43	9-2 legs of #7
5 & 6	21.0	79.3	1664	230	85	20	346	9	491	5.27	6-2 legs of #7
7 & 8	21.0	68.0	1428	449	291	65	-146	76	370	-0.62	6-2 legs of #7
9 & 10	21.0	67.0	1407	703	400	93	-772	-146	181	-8.36	6-2 legs of #7
11& 12	21.0	42.0	882	1233	589	131	-7	55	987	-2.28	4-2 legs of #7

Notes:

1. V_{DC}, V_{LL} and V_{PT} loads from LUSAS F.E. Model (See Finite Element Analysis Calculations)
2. P_c = vertical component of net compression force in diagonals (STRI)

Source: FIGG superstructure final design calculations (February 2017), page 1283.

From page 1398, Superstructure Final Design Calculations:



Appendix G: Independent Peer Review Certification Letters and Schedule for Design Stages and Independent Peer Review

Exhibit 26-B Independent Peer Review Certification Letter (90% Submittal)

Insert Date

Florida Department of Transportation
District ___
[Insert Street Address]

Attn: *[Insert Project Manager/Construction Project Engineer]*

Reference: **Independent Peer Review Category 2 Structures**
Financial Project ID: *[Insert FPID]*
Federal Aid Number: *[Insert Federal Aid Number]*
Contract Number: *[Insert CN]*

Submittal: **90% Bridge *[Insert Component/CSIP]* Plans**
Submittal *[Insert Submittal No.]*
Bridge Number(s): *[Insert Bridge No.(s)]*

Dear *[Insert Project Manager/Construction Project Engineer]*,

Pursuant to the requirements of the Contract Documents, *[Insert the name of the Independent Peer Review Firm]* hereby certifies that an independent peer review of the above-referenced submittal has been conducted in accordance with Chapter 26 of the Plans Preparation Manual and all other governing regulations. Component plans that were included in the peer review are as follows:
[Insert a list of all component plans that underwent an Independent Peer Review]

Outstanding / Unresolved Comments and Issues:
[Provide a statement of outstanding/unresolved comments for the above-referenced review, and actions being taken to resolve issues.]

Certification Statement:
I certify that the component plans listed in this letter have been verified by independent review and are in compliance with all requirements presented in the Contract Documents. Independent Peer Review comments and comment resolutions have been included in this submittal under separate cover.

Please do not hesitate to contact me if you have any questions.

Name of Independent Peer Review Firm	*[Insert Firm Name]*
Name of Independent Peer Reviewer	*[Insert Reviewer Name]*
Title	*[Insert Reviewer Title]*
Signature	______________________
Florida Professional Engineer Lic. No.	*[Insert License Number]*

Exhibit 26-C Independent Peer Review Certification Letter (100% Submittal)

Insert Date

Florida Department of Transportation
District ___
[Insert Street Address]

Attn: *[Insert Project Manager/Construction Project Engineer]*

Reference: **Independent Peer Review Category 2 Structures**
Financial Project ID: *[Insert FPID]*
Federal Aid Number: *[Insert Federal Aid Number]*
Contract Number: *[Insert CN]*

Submittal: **100% Bridge *[Insert Component/CSIP]* Plans**
Submittal *[Insert Submittal No.]*
Bridge Number(s): *[Insert Bridge No.(s)]*

Dear *[Insert Project Manager/Construction Project Engineer]*,

Pursuant to the requirements of the Contract Documents, *[Insert the name of the Independent Peer Review Firm]* hereby certifies that an independent peer review of the above-referenced submittal has been conducted in accordance with Chapter 26 of the Plans Preparation Manual and all other governing regulations. Component plans that were included in the peer review are as follows:
[Insert a list of all component plans that underwent an Independent Peer Review]

<u>Certification Statement:</u>

I certify that the component plans listed in this letter have been verified by independent review, that all review comments have been adequately resolved, and that the plans are in compliance with all Department and FHWA requirements presented in the Contract Documents.

Please do not hesitate to contact me if you have any questions.

Name of Independent Peer Review Firm	*[Insert Firm Name]*
Name of Independent Peer Reviewer	*[Insert Reviewer Name]*
Title	*[Insert Reviewer Title]*
Florida Professional Engineer Lic. No.	*[Insert License Number]*
	[Insert Signature, Date and Seal here.]

Table G-1. Schedule for design stages and independent peer review.

Stage (Plan Phase)	Date to be Delivered		Date Finalized
FIGG (responsible party) – Exhibit C			
DP-1 (Foundations)			
90% Design Submittal	May 2, 2016		
Final Design Submittal	June 21, 2016		
RFC [Release for Construction] Design Plans	July 20, 2016		
DP-1 (Substructure)			
90% Design Submittal	June 10, 2016		
Final Design Submittal	August 1, 2016		
RFC Design Plans	August 30, 2016		
DP-5 (Superstructure)			
90% Design Submittal	August 22, 2016		
Final Design Submittal	October 12, 2016		
RFC Design Plans	November 10, 2016		
*Estimated bridge design deliverables. These dates are based on concurrent review by FDOT and FIU of 20 days for bridge submittals and 15 days for all other submittals (excluding weekends and Owner [FIU] observed holidays)			
Activity	**Date to be Delivered**		**Date Finalized**
Louis Berger (responsible party) – Exhibit B			
Design Services/Scope of Work Provided to FIGG	September 13, 2016		
Contract/agreement signed			September 16, 2016
Notice to Proceed	August 17, 2016		
Modeling and Evaluation of Demand	August 31, 2016		
Review of Final Foundation Plans	September 7, 2016		
Review of Final Substructure Plans	September 21, 2016		
Review of Superstructure Plans	October 5, 2016		
100% Certification Letters Signed and Sealed to FDOT			
1. 100% Foundation Plans			September 13, 2016
2. 100% Substructure Plans			September 29, 2016
3. 100% Superstructure Plans			February 10, 2017

Appendix H: FDOT Local Agency Program Certification

Full certification is reserved for those agencies that demonstrate to the Florida Department of Transportation (FDOT) the qualifications and capability and achieve performance expectations between certification cycles. It is expected that over time the districts will be able to reduce the level of project oversight required to ensure compliance, while not increasing risk within the program. If the expiration date of the certification occurs during the course of a project, the certification will be considered to remain in effect until the project has been final accepted by the department and the Federal Highway Administration (FHWA).

Project-specific certification is reserved for those agencies with limited experience administering federal aid projects, or those that have not produced a consistent number of local agency program (LAP) projects to build experience and maintain consistent knowledge of the program. The districts will need to continue step-by-step project-based oversight of these agencies to mitigate risk, but FDOT staff should seize the opportunity to build these agencies into more consistent program participants (as appropriate). Project-specific certification is limited to off-system roadways, unless approval is provided by the district program management engineer. Project-specific certifications expire once the project closeout is complete.

Full certification life cycle of new local agencies occurs as needed, and subsequent recertification occurs on a 3-year cycle after the date of initial certification. Recertification is based on the local agency's updated subrecipient compliance assessment tool (SCAT), review of financial statements, LAP program training attendance for project personnel, maintenance of experienced project management personnel, and performance evaluations conducted by the district LAP staff at the close of each project administered by the local agency.[1] Full certification expires 3 years from the initial certification date. If a local agency does not produce a project in that 3-year period for any reason, recertification is not applicable. Recertification is dependent on the performance management evaluation process.

Local agency staff turnover is a critical risk factor in achieving successful compliance with all federal-aid highway program requirements. At any such time that a local agency loses key personnel, especially the responsible charge, the local agency's project oversight capability should be reviewed to determine if a change in certification status is warranted or a change to the level of district oversight is required. In the event that the local agency's certification is rescinded or removed, the agency may pursue full certification status at a later date.

Local agencies seeking certification in these areas must demonstrate their level of knowledge, skills, ability, and project experience identified on the LAP certification qualification tool. The required experience shown in table H-1 is necessary regardless of whether the services will be performed by the local agency's own forces or by a consultant or contractor. Contract

[1] See the FDOT LAP webpage, accessed September 23, 2019.

management, administration, and procurement knowledge and experience are critical to secure and ensure adequate oversight of consultants and contractors.

Table H-1. LAP certification areas and requirements.

Certification Area	Minimum Qualifications
Planning	Employees with knowledge of the Metropolitan Planning Organization transportation planning processes; experience with transportation planning studies; and transportation projects of a nature similar to those the agency intends to develop. Refer to Chapter 14-75 of the Florida Administrative Code for minimum planning qualifications required for SHS/NHS projects.
Design	Experience in design with various types of infrastructure projects, particularly projects of a nature similar to those the Local Agency intends to design with federal funds. Florida Professional Engineer registration is required if the Local Agency intends to design a project with its own forces. Training and knowledge of the Americans with Disabilities Act requirements 49 CFR 27, 49 CFR 37, and per the Departments of Justice and Transportation Joint Technical Assistance Memo on Title II of the Americans with Disabilities Act Requirements. Refer to Chapter 14-75 of the Florida Administrative Code for minimum design qualifications required for SHS/NHS projects.
Construction/ Construction Administration	Local Agency staff with experience in providing construction oversight of transportation projects (preferably federally funded), including but not limited to managing contract time, change orders, and construction invoicing. The Local Agency has a materials quality assurance process in place and a process for contract compliance; including but not limited to: Equal Opportunity, Disadvantaged Business Enterprise tracking, and compliance with minimum wage rate decisions and payroll verification. Any inspection and oversight work on the SHS/NHS must comply with the qualifications of work group 10 of Chapter 14-75 of the Florida Administrative Code. An approved design-build procedure is required if the Local Agency will administer a design-build project.

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Table H-2. FDOT's Electronic Review Comment system approval sequence.

Submittal Date	Description	Activity and uploads to FDOT's Electronic Review Comment Portal
2016		
February 26	**30% Preliminary Plans submittal** (uploaded March 8)	
	March 25	114 comments uploaded (104 FDOT, 10 FIU) • 4/22 comments responded to, 8/31 comments approved, closed out
	June 28	FDOT provides to FIGG clarification on comments
	June 29	FIGG provides responses to FDOT comments
	June 30	Comment resolution meeting between FDOT and FIGG
	July 5	Finalized summary of June 30 meeting sent from FIGG to FDOT
	September 14	FIGG provided updated responses to FDOT comments
	September 15	FIGG meeting with FDOT to review updated responses to FDOT comments
	September 16	Finalized summary of September 15 meeting sent from FIGG to FDOT
May 2	**90% Foundation Plans submittal** (Uploaded May 10)	
	June 8	17 comments uploaded (all FDOT) • 6/29 comments responded to, 9/19 comments approved, closed out
	June 30	Comment resolution meeting between FDOT and FIGG
	July 13	**90% Foundation Plans re-submittal** (Uploaded July 14)
	August 3	32 comments uploaded (all FDOT) • 8/17 comments responded to, 9/14 comments approved, closed out
	September 13	**Final Foundation Plans submittal** (Uploaded September 15)
	October 18	22 comments uploaded (all FDOT) • 11/29 comments responded to, 12/15 comments approved, closed out
June 10	**90% Substructure Plans submittal** (Uploaded June 15)	
	July 1	41 comments uploaded (all FDOT) • 8/01 comments responded to, 8/03 comments approved, closed out
	August 1	**90% Substructure Plans re-submittal** (Uploaded August 3)
	August 19	21 comments uploaded (20 FDOT, 1 third-party reviewer) • 9/22 comments responded to, 9/28 comments approved, closed out
	September 29	**Final Substructure Plans submittal** (Uploaded October 17)
	October 28	24 comments (22 FDOT, 2 third-party reviewer) • 11/21 comments responded to • 1/24/2017 comments approved, closed out
September 26	**90% Superstructure Plans submittal** (Uploaded September 28)	
	September 15	Figg meeting with FDOT to preview the 90% Superstructure Plans submittal
	September 16	Draft meeting summary provided by FIGG to FDOT for review, FDOT concurs
		FDOT comments on 90% Superstructure Plans status set are provided to FIGG

	October 17	35 comments uploaded (all FDOT) • 11/11 comments responded to, 12/14 comments approved, closed out
December 9	**RFC Foundation Plans submittal**	
	2017	
January 13	**RFC Substructure Plans submittal, re-submittal on February 28**	
February 10	**Final Superstructure Plans submittal** (Uploaded February 14)	
	May 2	36 comments uploaded (32 FDOT, 4 third-party reviewer) • 5/23 comments responded to, 6/13 comments approved, closed out
February 28	**RFC Foundation Plans re-submittal**	
April 7	**RFC Superstructure Plans submittal**	
Note: (1) Some FDOT comments referred to documents external to the ERC portal that contained multiple additional comments. Total FDOT comments was greater than the number of comments in the FDOT ERC portal. (2) Some comments in ERC refer to previous documents that had contained multiple older comments. Examples include the 90% Foundation Design, comment #15; and the 90% Structural Pylon & Landing Structures Design, comment 1. (3) 30% Preliminary comments were marked "for information only"– no response required due to the preliminary nature of the submittal. All comments on all subsequent submittals required a written response.		

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