I-195 Washington Bridge (700)

Forensic Evaluation and Procedural Audit Related to PT Tie-Down Failures

I-195 SB Washington Bridge over Seekonk River
Providence, RI

DRAFT REPORT

April 5, 2024
WJE No. 2023.7858.0

PREPARED FOR:

Mr. John Preiss
State Bridge Engineer
Rhode Island Department of Transportation
Two Capitol Hill
Providence, Rhode Island, 02903

PREPARED BY:

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WJE

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Michael C. Brown, PhD, PE
Associate Principal

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INTRODUCTION

Wiss, Janney, Elstner Associates, Inc. (WJE) was engaged by the Rhode Island Department of Transportation (RIDOT) to perform a forensic audit of events preceding failure of post-tensioned (PT) tie-down rods at Pier 7 of the southbound I-195 Washington Bridge north structure (700). This document summarizes our findings.

PURPOSE AND SCOPE OF REVIEW

WJE was tasked with performing a review of records and history leading to the post-tensioned rod failures and to provide its assessment of:

- what events and conditions led to the failure of the rods
- whether the agency's decision to close the bridge to traffic was reasonable
- whether conditions leading to the failure could have or should have been foreseen
- whether conditions discovered after the failure revealing serious deterioration of the post-tensioned cantilever beams should have been foreseen
- what actions or policy recommendations may be recommended to prevent similar types of events from occurring in the future

WJE's efforts consisted of two major thrusts:

1. Forensic metallurgical analysis of samples taken from the failed PT tie-down rods
2. Detailed review of agency records to understand the design, construction, maintenance, inspection, repair, and rehabilitation history of the structure in general, with particular focus on the post-tensioned concrete cantilever beams and associated support corbels/pedestals and tie-down rods at Piers 6 and 7.

Information used by WJE to perform these studies was provided by Mr. John Preiss, State Bridge Engineer, RIDOT. Throughout the period of this study, WJE was aware of but did not play a role in the design of mitigation repairs that would potentially allow the bridge to reopen to traffic. RIDOT made a conscious decision to firewall WJE from day-to-day meetings and discussions so as not to have those discussions bias WJE's review.

BACKGROUND

The subject bridge is an 18-span structure comprised of prestressed and post-tensioned concrete multi-girder approach spans; 6 spans to the west and 11 spans to the east of a steel multi-girder main span (Span 7). There is also a curved three-span prestressed concrete box girder ramp to Gano street exiting to the north at the west end. The Structure Inventory & Appraisal (SI&A) information in the July 2023 inspection report indicates the bridge is 1,903.87 ft long with a maximum span length (Span 7) of 130.60 ft. The structure is nominally 75 ft wide out-to-out and 68 ft curb-to-curb with a total deck area of 152,958.0 square feet. The bridge is a unique design, possibly the only one of its kind in the country. One of the most unique aspects is its incorporation of balanced and unbalanced post-tensioned concrete cantilever beams that alternately support prestressed concrete drop-in beams. At the ends of the cantilevered bridge segments, the weight of drop-in beams is countered at the unbalanced end of the cantilevers by restraint via high-strength post-tensioned tie-down rods that attach to pier walls or an abutment. The bridge was closed to traffic on December 11, 2023 after construction workers reported observation of a full fracture of one of the PT rods used to tie down a cantilever beam at Span 7 to Pier 7.

WJE personnel Michael Brown and John Cocca visited the bridge on December 16, 2023. During the site visit the fractured bars and the configuration and condition of the cantilever beams and structural walls at Piers 6 and 7 that the post-tensioned (PT) rods were designed to connect were observed. WJE observed fractured rods at two of four corners of Span 7, at Cantilever A and Cantilever F at Pier 7. At the time, a third rod, Cantilever A at Pier 6, was suspected of also being fractured due to observed movement of the cantilever/rod, but a fracture could not be visibly confirmed. A report noted bouncing of cantilevers and gaps at bearings of adjacent girder lines at the Pier 6 and Pier 7 connections.[1] Photos of the fractured bars taken by WJE on December 16, 2023 are shown in Figures 1 and 2. The bars were corroded on the exterior perimeter and the cross-section was reduced/tapered in the regions around the fractures.

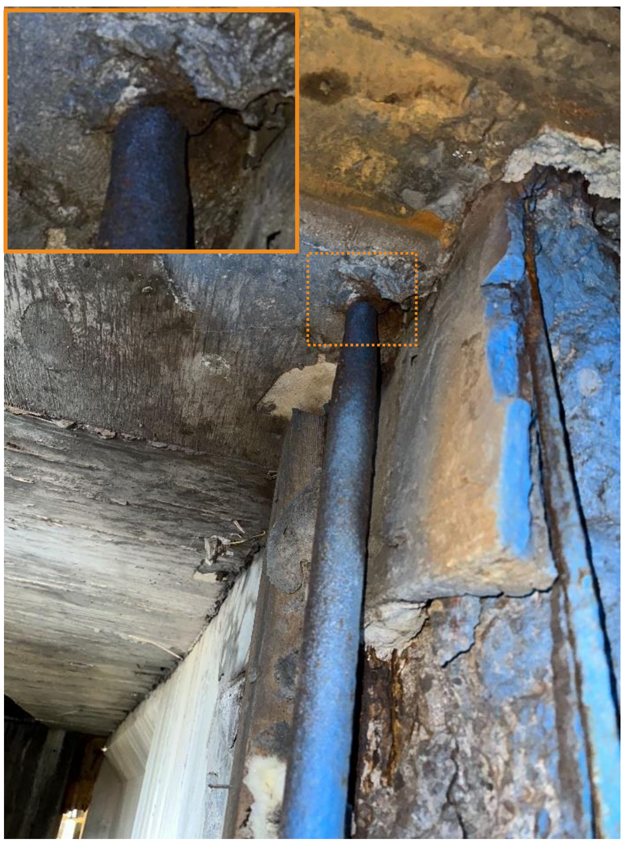
Figure 1. Pier 7 Cantilever A fractured tie-down rod


Figure 2. Pier 7 Cantilever F fractured tie-down rod

DOCUMENTATION REVIEW

To review the structure's composition and history predating the reported failure, WJE requested access to original design documents, inspection reports, special study reports, rehabilitation project plans,

[1] *VHB Visit Findings 12-11-23 (Additional Info) (Reduced).pdf*

specifications, structural calculations, and contract documents, as well as construction inspection daily reports and summaries.

Original Design and Construction (1967)

WJE reviewed original design plans, applicable standards and (non-project-specific) specifications, structural calculations, and shop drawings. Project-specific specifications were not available. The original design for the bridge was completed in January 1967 by Charles A. Maguire & Associates Engineers of Providence, RI. The design referenced the then current American Association of State Highway Officials (AASHO) specifications, including the Standard Specifications for Highway Bridges, 9th edition[2], and the 1965 edition of Rhode Island Standard Specification for Road and Bridge Construction[3].

Framing Plan and Cantilever Design

The following summarizes aspects of the original design relevant to the post-tensioned cantilevers, prestressed drop-in beams, Pier 6 & 7 support corbels and post-tensioned tie-down rods. The approach structures in Spans 1 through 6 and 8 through 13 comprise post-tensioned concrete balanced cantilever beams on concrete piers that support prestressed concrete drop-in girders between them (Figure 3).

Unbalanced post-tensioned concrete cantilever beams at end spans at Abutment 1 (C-1 through C-6) and Piers 6 and 7 (C37 through C42 and C-43 through C-48, respectively) support drop-in beams at one end and are tied down at the abutment or pier wall at the other to provide counter force (**Error! Reference source not found.**). The unbalanced cantilevers were designated “Type D” in the drawings and the length of cantilever at the tie down end was longer than the drop-in end (Figure 5). These end cantilever segments are tied down with high-strength PT rods to reinforced concrete corbels on the face of Abutment 1 to balance Span 1 drop-in beams and to structural walls of Piers 6 and 7 to either side of the steel main span, Span 7, to counterbalance the loads of the drop-in beams in Spans 6 and 8. At the east end of the bridge, the last cantilever segment is balanced, supporting a drop in beam that also rests directly on Pier 14, so no tie-down detail is required.

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[2] *Standard Specifications for Highway Bridges*, Ninth Edition, American Association of State Highway Officials, Washington, D.C. 20004, 1965

[3] *Standard Specifications for Road and Bridge Construction*, State of Rhode Island and Providence Plantations Department of Public Works, Division of Roads and Bridges, State Office Building, Providence Rhode Island, Revision of 1965

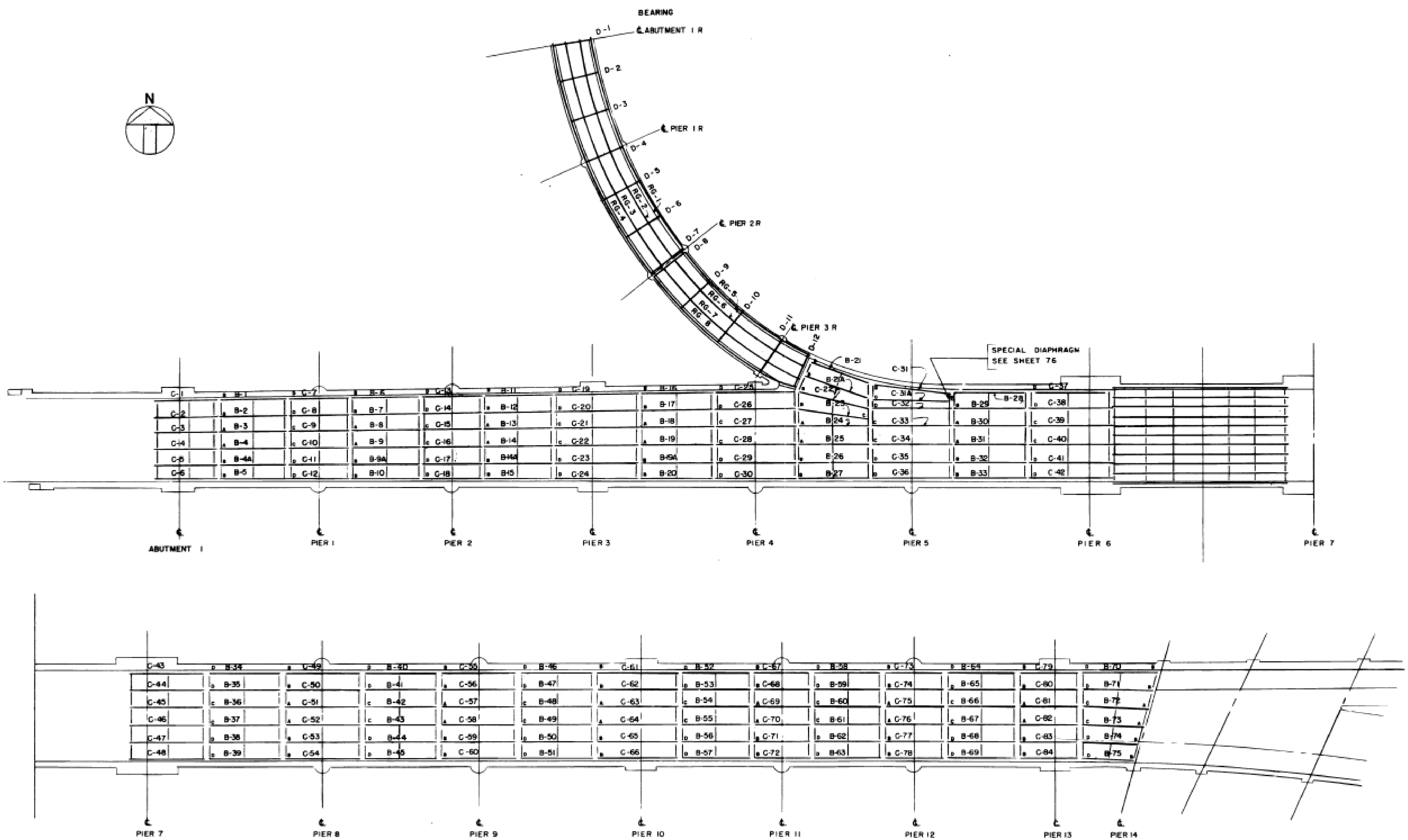


Figure 3. Full Framing Plan with concrete beam designations (Sheet 70, Framing Plan) of original plans (1967).

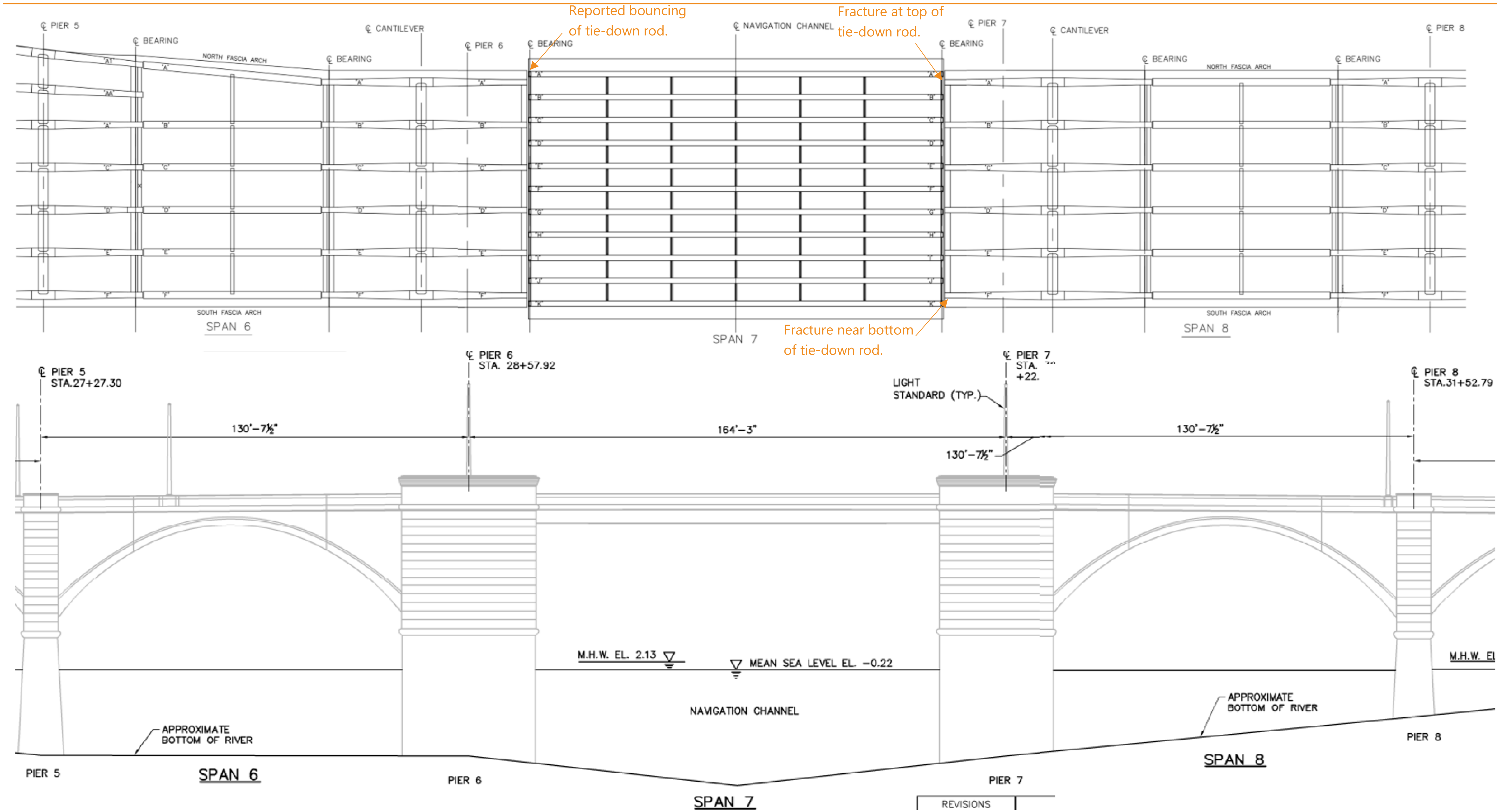


Figure 4. Framing Plan and Elevation of Spans 6 through 8, southbound I-195 Washington Bridge North (700)

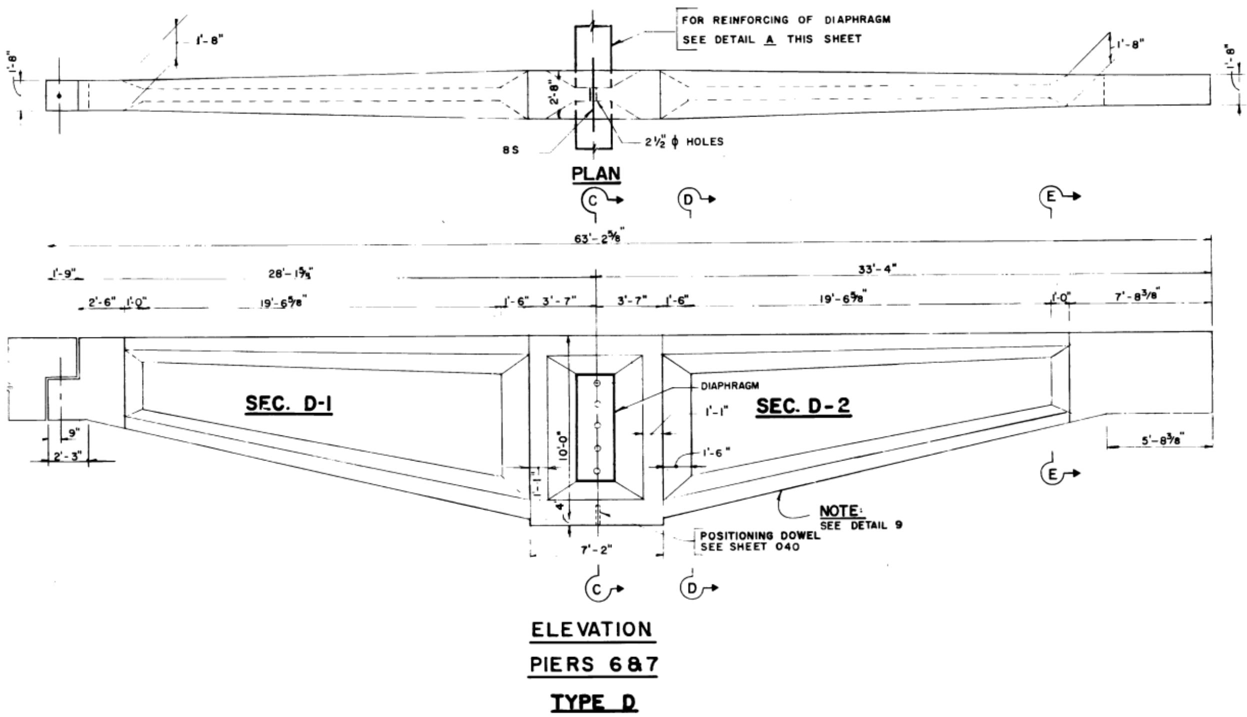


Figure 5. Type D Cantilever Beam at Pier 6 & 7 (Sheet 74, Cantilevers Sheet 1 of original plans, 1967).

PT Tie-down Rods

At the subject span ends, there are six concrete girder lines (A-F), each with twin tie-down rods connected to an anchor plate at the top that straddles the cantilever beam (Figure 6). The tie-down rods are generally embedded in the diaphragms at ends of the cantilever beams and extend down into the corbels, making them thus not visible for inspection. However, at the four outward corners at girder lines A and F of Piers 6 and 7, the outboard tie-down rods lie to the exterior of the cantilever beam ends, where there are no diaphragms, and are exposed from the top of the beam seat to the bottom of the concrete deck. Original design drawings required the prestressing rods to be 1 $^{3}/_{8}$″ diameter high strength pre-stressing rod with threaded ends (Figures 7 and 8).

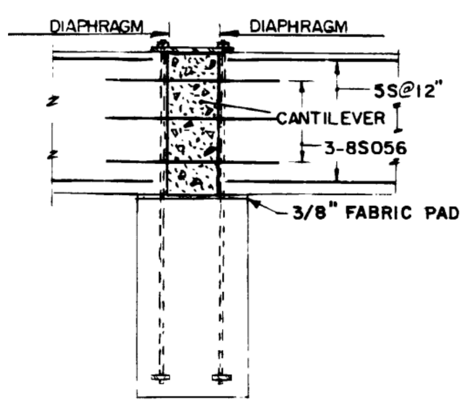


Figure 6. Section depicting PT anchor plates and tie-down rods at end span cantilever supports (Sheet 143, Detail Sheet 8 of original plans, 1967).

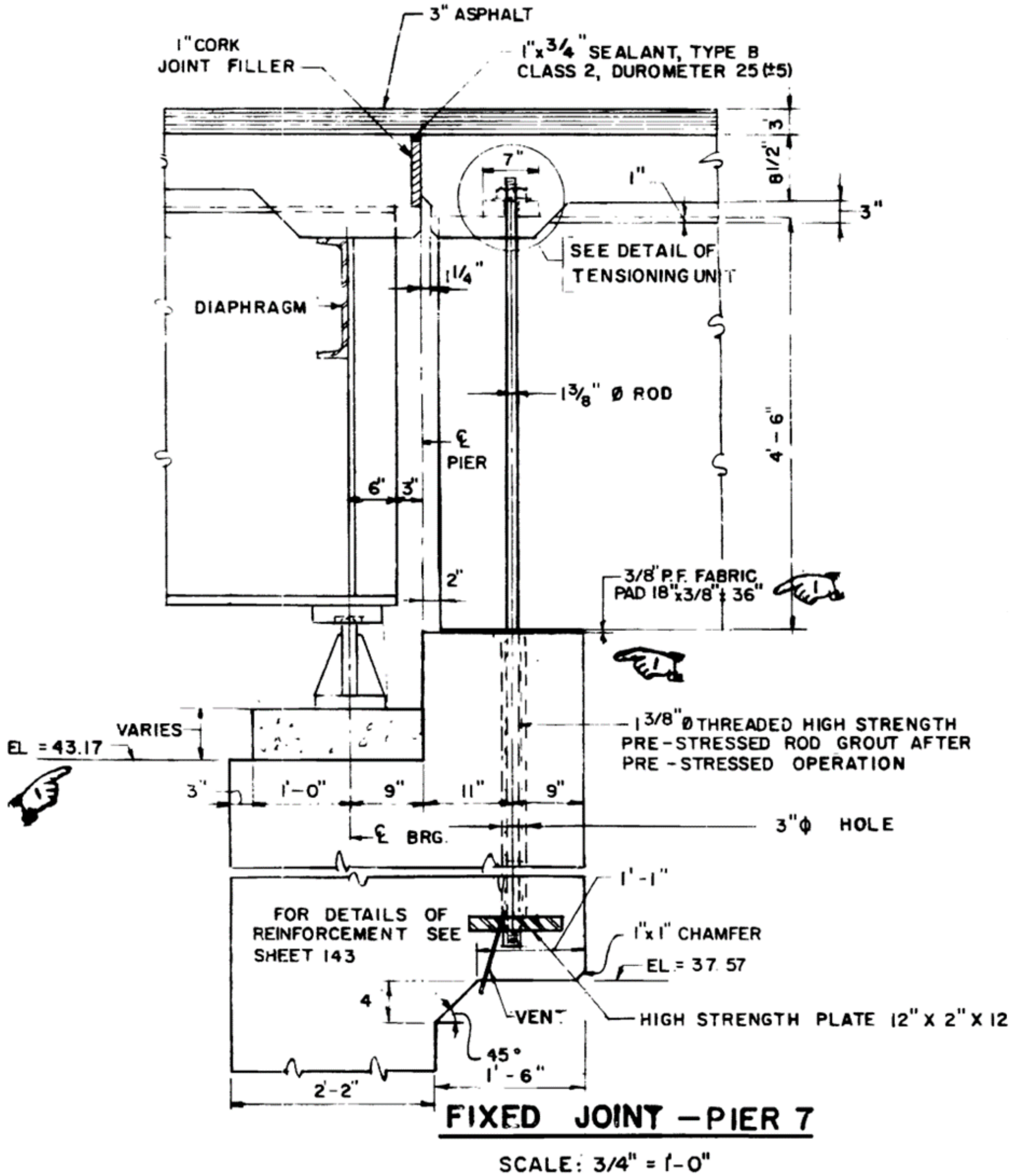


Figure 7. Section depicting steel and concrete beam bearing and PT anchor plates and tie-down rod details for Pier 7 (Sheet 139, Detail Sheet 4 of original plans, 1967).

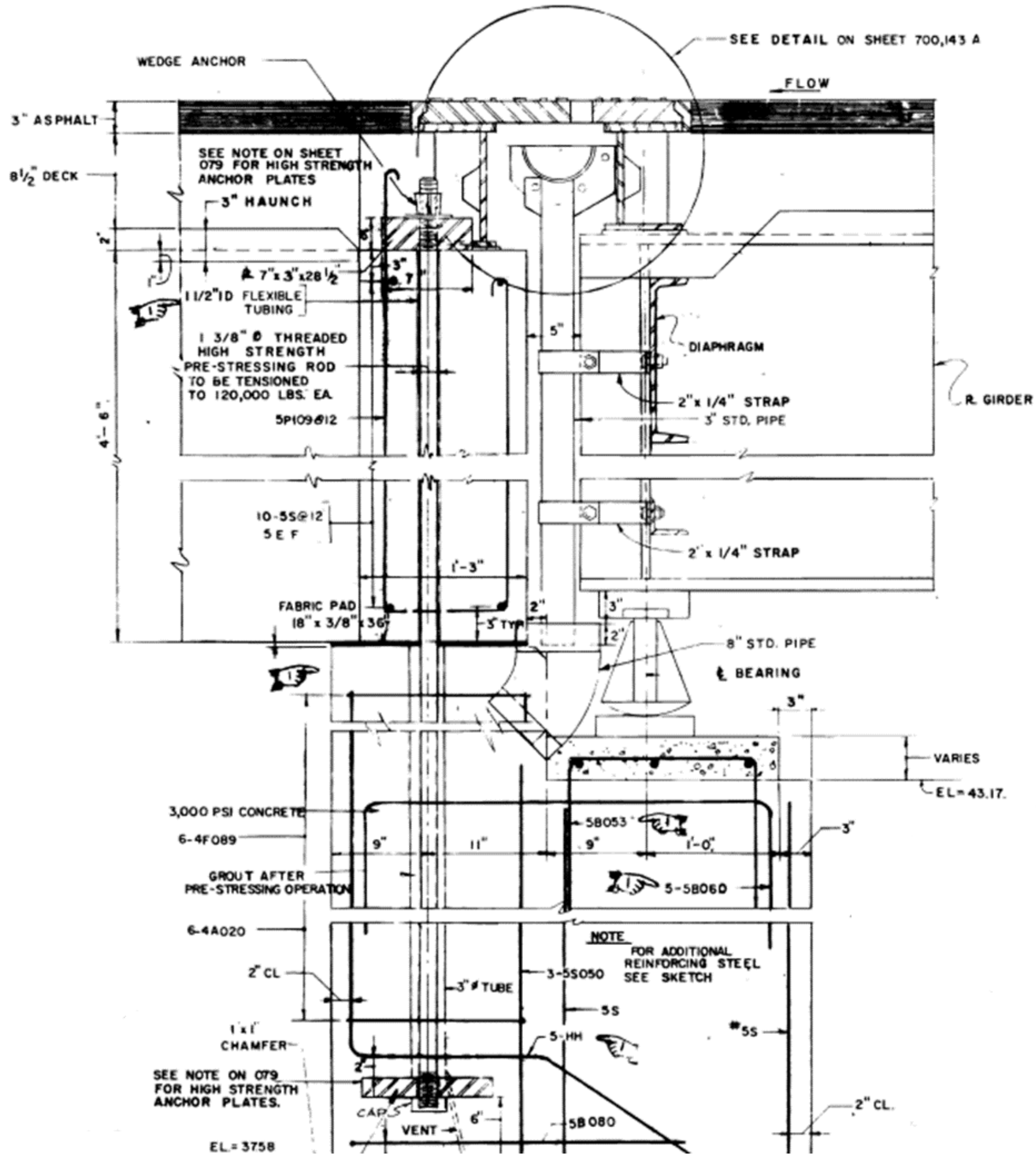


Figure 8. Section depicting steel and concrete beam bearing and PT anchor plates and tie-down rod details for Pier 6 (Sheet 143, Detail Sheet 8 of original plans, 1967).

Per Sheet 79, Cantilever Beam Anchorage of the original plans (1967),

> "The high strength steel to be used for anchor plates, hold down plates and plate washers shown on sheets listed below shall conform to ASTM designation A107-61T Grade 1040 for bar sizes and shall be from rolled billets conforming to the same designation as well as the requirements of ASTM designation A-6 for plate sizes with the following physical requirements:
> Min Ultimate Strength = 91,000 p.s.i.
> Min Yield Point = 50,000 p.s.i."

Structural design calculations for tie-down anchor plates reflect these yield and ultimate strengths.

No standard specification was specifically cited in the plans for the high-strength prestressing/post-tensioning rods and original project specifications were not provided. However, the 1965 Rhode Island Standard Specification for Road and Bridge Construction contains provision for high-tensile-strength alloy bars, which would have been applicable (Figure 9).

M.05.03 PRESTRESSING REINFORCEMENT STEEL. Prestressing reinforcement shall be high-tensile-strength steel wire, high-tensile-strength, seven-wire strand or high-tensile-strength alloy bars as called for in the plans or in the special provisions.

High-tensile-strength steel wire shall be made by the open-hearth or electric furnace process and shall conform to ASTM A 421 (AASHO M 204).

High-tensile strength seven-wire strand shall conform to the requirements of ASTM Designation A 416 (AASHO M 203).

High-tensile-strength alloy bars shall be stress-relieved and then cold stretched to a minimum of 130,000 psi. After cold stretching, the physical properties shall be as follows:

Minimum ultimate tensile strength	145,000 psi
Minimum yield strength, measured by the 0.7 percent extension under load method shall be not less than	130,000 psi
Minimum modulus of elasticity	25,000,000 psi
Minimum elongation in 20-bar diameters after rupture	4 percent
Diameter tolerance	plus 0.03", minus 0.01"

Figure 9. M.05.03 Prestressing Reinforcement Steel, Standard Specifications for Road and Bridge Construction, Revision of 1965, p. 319.

The design was performed according to allowable stress design (ASD) procedures. The applicable AASHO specifications limited stress at design load to the lesser of 0.60 f'_s (min ultimate strength) or 0.80 f_{sy} (nominal yield stress @ 1.0% extension).

Per structural drawings (Sheets 139 and 140), PT tie-down rods were each to be post-tensioned to 120,000 lbf. Structural design calculations indicate the PT rods were assumed to be centered 1 inch from the outside face of each cantilever web face and that expected design loads on the rods was P_{max} = 142.6 kips, or $P_{\underline{working}}$ = 140kips.[4] Based on a calculated cross-sectional area of 1.485 in^2 for a 1 $^3/_8$-inch diameter bar, the calculated effective working stress was 94.3 ksi. Note this would represent 0.65 f'_s and 0.725 f_{sy} if the high strength rod used matched the RIDOT specification. From Stressteel Corp. shop drawings, a

4 *1967 070001 Cantilever Stems and Connections - Design Calcs.pdf*

separate calculation of bar stress accounting for creep, shrinkage and relaxation losses indicated a target applied post-tension stress of 92.63 ksi.

Unfortunately, records WJE reviewed did not specifically identify the steel used for the rods. However, tests of the materials from the two failed rods indicated ultimate strengths of 158.9 and 165.5 ksi, such that design stress was ≤ $0.60f_{su}$ (average stress at ultimate load) based on 140 kip working load.[5]

Cantilever Post-Tensioning

The Type D cantilever beams were internally post-tensioned with 5 cables along 4 profile lines that draped and spanned over the pier stem support from the beam ends (Cables 1, 2 & 3) or between locations along the bottom flange (Cables 4 & 5) as shown in Figures 10 and 11. Type D cantilever beams were constructed using 5,000 psi (f'_c) concrete post-tensioned with cables composed of 12 -$^1/_2$" diameter 7-wire strands.

Tendons were grouted after post-tensioning. Shop drawings indicate the grout used was as shown in Figure 12. The stated range of water (5 to 6 gallons per bag of cement) would be equivalent to a water-to-cement ratio (w/c) range of 0.44 to 0.53 for Mixture A.

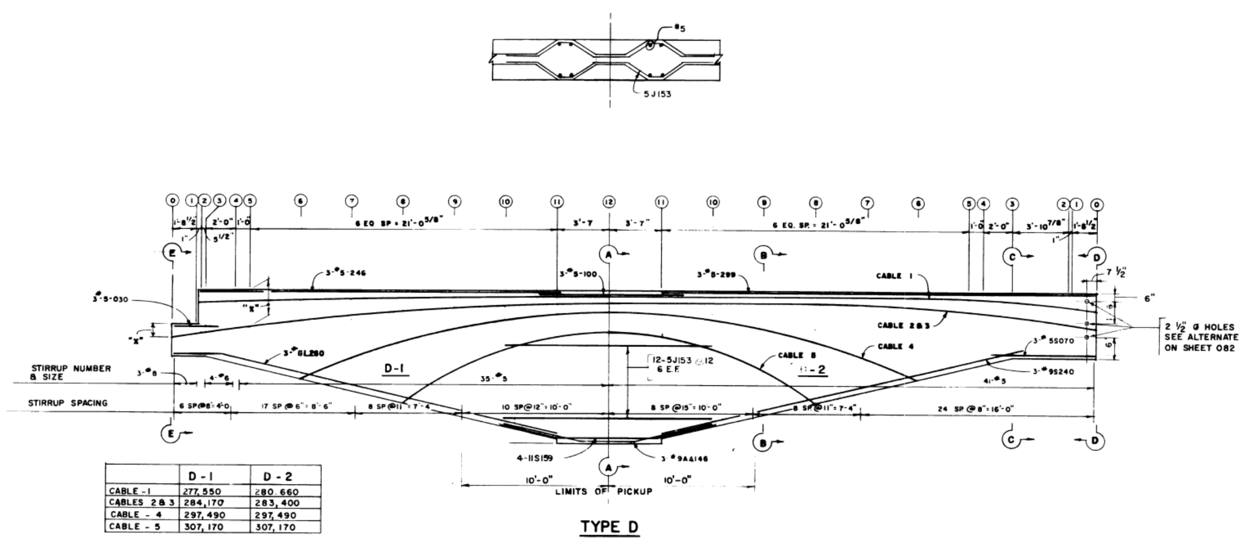


Figure 10. Post-tensioning layout for Type D Cantilever Beams (Sheet 78, Cantilevers Sheet-5 of original plans, 1967).

[5] Forensic Investigation of Failed Post-Tensioned Tie-Down Rods, I-195 SB Washington Bridge North (700), Wiss, Janney, Elstner Associates, Inc., February 19, 2024, *RIDOT Washington Bridge PT Rod Failure Memo 2024-02-19.pdf*

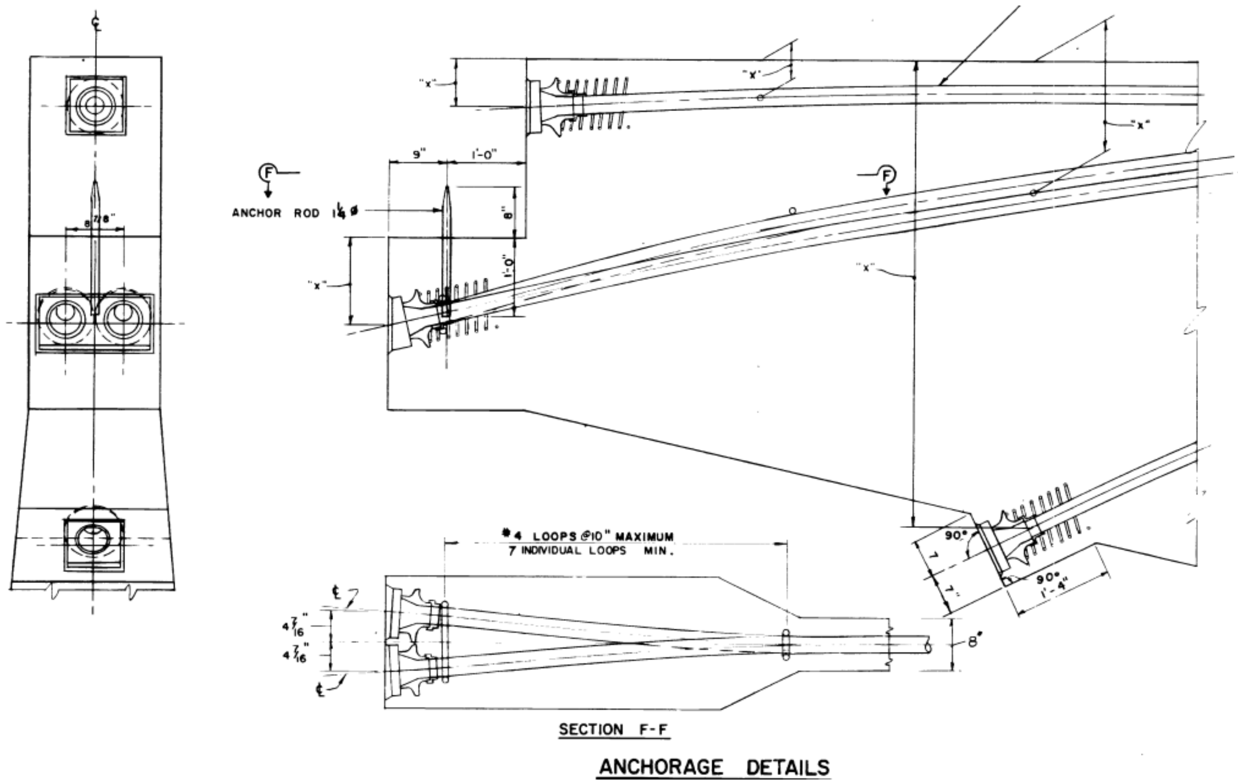


Figure 11. Post-tension anchorage details in cantilever beams (Sheet 78, Cantilevers Sheet 5 of original plans, 1967).

Grout Mixes	Mixture A	Mixture B
Portland Cement	96 lbs.	96 lbs.
Low Carbon Flyash	none	50 lbs.
Admixture (*)	1 lb.	1 lb.
Water	Sufficient water to give desired fluidity. The water-cement ratio is usually kept to 5-6 gallons per sack of cement.	

(*) The admixture recommended above is Intraplast C manufactured by the Sika Chemical Company.

cont.

Figure 12. Grout mix design excerpt from shop drawing.[6]

Lichtenstein Emergency Inspection Report, January 1992[7]

Lichtenstein completed an emergency inspection, testing, and evaluation of the cantilever beams and ship-lap details of the bridge and reported, "the condition of the shiplap details (corbels) in the precast post-tensioned concrete cantilever beams are deteriorating. The damage found is localized to the area below the roadway joints and is more severe on the lower shiplap (corbel), which supports the drop-in spans. There is a precast U-shaped buildout on the end of each corbel which forms a stressing pocket for the two post-tensioning tendons which terminate in the corbel at a heavy steel bearing plate cast into the

[6] *1967 070001 Prestress Shop Drawings and Calcs.pdf*

[7] A.G. Lichtenstein & Associates, Inc., report for RIDOT Contract No. 90127 Washington Bridge – I-95 Westbound Bridge No. 700, January 27, 1992, *WBN_070001_Lichtenstein EM Inspection Testing Report 1992.pdf*

beams…The grout in the stressing pocket and the precast shoulders of the cantilever beams are all showing signs of distress."

The post-tensioning tendons were identified as comprised of Freyssinet 270 ksi 7-wire strand, with dead ends of the wires protruding beyond the standard anchorages. Concern was expressed about stress corrosion resulting from moisture and salt exposure. The report related results of chloride testing and radiography. Ten chloride samples were obtained from the north face at the east end of Beam A in Span 13 (samples 1-6), the north face at the west end of Cantilever F in Span 3 (samples 7-9), and the south face at the west end of Beam E in Span 3 (sample 10). Chloride concentrations were reported to be high 11.47 to 11.75 pcy) but results were unusually uniform in concentration over the 10 samples and therefore deemed suspect. Radiography was used to look for broken strands in two beams; broken strands were not found, however, shadows in the images suggested the presence of voids in the grouted tendons behind the anchorage plates. The report suggested, "Ultrasound or ground penetrating radar testing in these areas might provide more reliable data on concrete condition in the anchorages."

From calculations performed to evaluate stress in the cantilever beams and corbels, the authors concluded that, "Stress in the prestressing strands after calculated losses is estimated at 54% of the ultimate strength of the strands which is below the stress allowed by AASHTO." They later stated, "Due to the structural redundance of the wires, strands and beams and the fact that we calculated strand stresses of approximately 54% of ultimate after losses, there is reason to believe that a failure of some wires would not result in catastrophic collapse, but if one corbel were found with major distress indicative of a shear failure, the failure of adjacent corbels might occur rapidly."

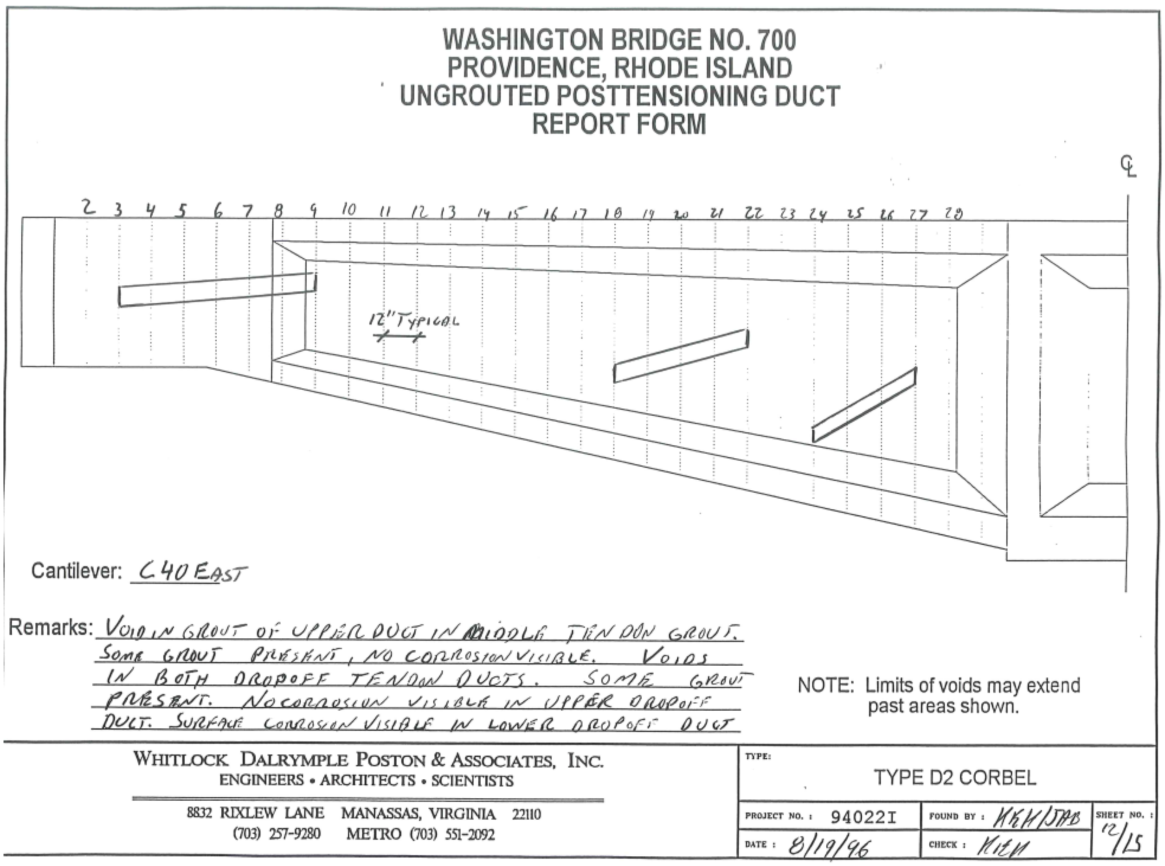
The authors also noted, "The secondary area of concern in the post-tensioned cantilever beams is in the beam webs where cracks through have been found that follow the tendon profile." They surmised, "there are two conceivable causes for this type of cracking: the cracks may have formed during construction during initial tensioning of the strands due to an overstress of the concrete or the cracks could be caused by expansive forces created by corrosion of the tendon ducts. Through a series of calculations, the authors surmised that the calculated principal stresses, at the level of the tendons, would have caused cracking if the concrete strength at the time of post-tensioning were below 4000 psi or even perhaps if slightly above, concluding, "Calculations indicate that the diagonal cracks, which follow the tendon profile in all likelihood were formed during initial tensioning of the tendons" and, "these cracks have not grown and will probably not grow in the future."

1996-1998 Rehabilitation

After identification of the issues with corbels and PT strands, a rehabilitation project was designed in 1996-97. The plans were developed by Vanasse Hangen Brustlin, Inc. (VHB) of Providence, RI. The project included a series of repairs to the corbels of the cantilever beams. In July 1996, while preparing to execute the rehabilitation project, greater deterioration was discovered in the corbel supports of the cantilever-drop-in beam connections than previously anticipated.[8] Further probing brought into question the

[8] Time Analysis: Conception Through Construction to Date, Washington Bridge No. 700, R.I. Contract No. 9603 by David F. Arnold, Arnold Engineering Company, Inc., September 22, 1997, as found in file "*1997-09-10 Correspondence from Sen. Roney to Director Anker.pdf*"

condition of post-tensioning anchorages and the tendons. Whitlock, Dalrymple, and Poston (WDP) were engaged to perform impact echo testing of the cantilever beam webs to investigate suspected voids. Over a series of NDE inspections, WDP identified 81 out of 170 beams (47.6%) containing 139 out of 644 PT ducts (21.6%) with indications of voids or voids and delaminations (Figure 13). The cantilevers at Piers 6 & 7 in Span 7 contained the highest frequency of voids and delamination with indications in 42 out of 120 (35%) ducts (Figure 14). Note Duct "H" in each beam could not be evaluated due to the thicker section and reinforcement configuration.



WASHINGTON BRIDGE NO. 700
PROVIDENCE, RHODE ISLAND
UNGROUTED POSTTENSIONING DUCT
REPORT FORM

Cantilever: C40 EAST

Remarks: VOID IN GROUT OF UPPER DUCT IN MIDDLE TENDON GROUT. SOME GROUT PRESENT, NO CORROSION VISIBLE. VOIDS IN BOTH DROPOFF TENDON DUCTS. SOME GROUT PRESENT. NO CORROSION VISIBLE IN UPPER DROPOFF DUCT. SURFACE CORROSION VISIBLE IN LOWER DROPOFF DUCT

NOTE: Limits of voids may extend past areas shown.

WHITLOCK DALRYMPLE POSTON & ASSOCIATES, INC.
ENGINEERS • ARCHITECTS • SCIENTISTS
8832 RIXLEW LANE MANASSAS, VIRGINIA 22110
(703) 257-9280 METRO (703) 551-2092

TYPE: TYPE D2 CORBEL
PROJECT NO.: 94022I | FOUND BY: KEH/JAB | SHEET NO.: 12/15
DATE: 8/19/96 | CHECK: KEH

Figure 13. Example impact echo findings from WDP inspections

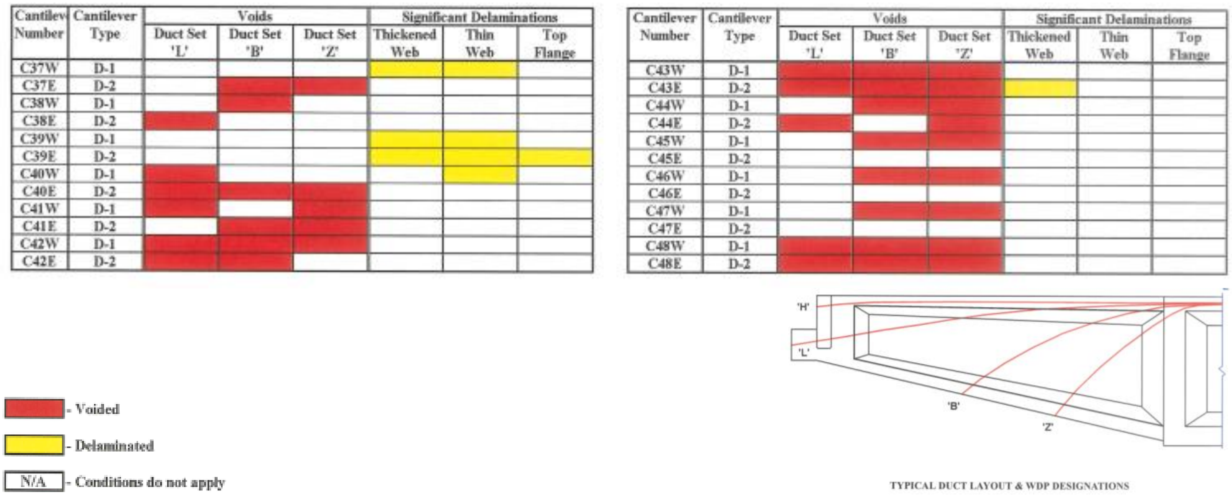


Figure 14. Summary of Type D cantilever impact echo findings compiled from WDP report.[9]

Special provisions have been retained for the impact echo testing and retrofit grouting.[10] A series of change orders were issued during the contract to address the required NDE testing, additional repairs to ends of cantilevers, additional repairs to address cantilever beam delaminations and the retrofit grouting to address voids detected in the cantilever beam post-tensioned ducts.[11,12,13] The WDP summary report indicates voids in ducts of most cantilever spans. However, it is unclear from available documents whether retrofit grouting was applied to cantilevers in all spans or primarily to the Pier 6 and 7 cantilevers. Project records and change order documents indicate 125 one-square-foot tendon openings (with cost unit "each") were made to enable retrofit grouting during the project.[13] It is not clear how many cumulative linear feet of tendon were retrofit grouted.

Inspection Reports (2001 to 2023)

Access to inspection reports was provided via AASHTOWare BrM and SharePoint. WJE received documentation of the biennial routine inspections, underwater inspections conducted on a four-year cycle, and a series of special inspections, as listed in APPENDIX A.

Table 1. Special inspections were performed during years alternate to routine inspections from 2016 forward as the bridge was placed on a 12-month inspection cycle primarily due to progressing deterioration. Some special inspections were requested to investigate specific issues.

[9] *1997-07-02 Impact Echo Testing Results WDP.pdf*

[10] *1996-03-20 Contract Documents - Special Provisions Pages S-84 to S-87.pdf*

[11] *1998-07-14 ROC 057 - Item #259 - Pier 6 and 7 Pedestan and Cantilever Repairs.pdf*

[12] *1998-11-02 ROC 064 - Item #269 - Delamination Repairs.pdf*

[13] *1996-10-31 ROC 008 - Impact Echo - Item #76 - Retrofit Grout of Post Tensioning.pdf*

Table 1 shows the reported General Condition Ratings (GCR) for Deck, Superstructure and Substructure components (items 58, 59, & 60, respectively) as defined in the National Bridge Inspection Standards (NBIS)[14] and the Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (Coding Guide).[15]

For reference, the FHWA Coding Guide defines these GCRs shown as follows:

> "6 SATISFACTORY CONDITION - structural elements show some minor deterioration.
>
> 5 FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
>
> 4 POOR CONDITION - advanced section loss, deterioration, spalling or scour."

Table 1. Washington Bridge 700 inspection reports reviewed

Report Date	Inspection Firm	Inspection Type	General Condition Rating		
yyyy-mm-dd			**Deck**	**Superstructure**	**Substructure**
2001-06-24	RIDOT	Routine	5	5	5
2003-08-26	Fuss & O'Neill	Underwater	-	-	-
2007-07-01	AI Engineers	Routine	6	4	5
2009-06-29	Specialty Diving Services	Underwater	-	-	-
2009-08-07	TranSystems	Routine	6	4	5
2011-08-03	Michael Baker	Routine	6	4	5
2013-08-02	AI Engineers	Routine	6	4	4
2013-08-07	Collins Engineers	Underwater	-	-	-
2015-07-28	AECOM	Routine	6	4	4
2016-07-15	TranSystems	Special	6	4	4
2017-07-24	Collins Engineers	Routine	6	4	4
2017-07-24	Collins Engineers	Underwater	-	-	-
2017-10-27	AECOM	Special	-	-	-
2018-07-24	Michael Baker	Special	-	4	4
2019-07-24	AECOM	Routine and Special	6	4	4
2020-07-22	AECOM	Special	6	4	6
2021-07-23	Jacobs	Routine and Underwater	6	4	6
2022-07-22	TranSystems	Special	6	4	6
2023-07-21	AECOM	Routine	6	4	6
2023-12-17	AECOM	Special	6	4	6

[14] National Bridge Inspection Standards - Bridge Inspection - Safety Inspection - Bridges & Structures - Federal Highway Administration (dot.gov), *https://www.fhwa.dot.gov/bridge/nbis.cfm*

15 Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, Report No. FHWA-PD-96-001, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., December 1995.

The progression of inspection reports demonstrates the transition from solely NBIS-based inspections to incorporate element-level inspection as the department adopted formal bridge management systems software, first within Pontis and then in AASHTOWare BrM. For example, the 2007 inspection report lists the addition of 30 Commonly Recognized (CoRe) bridge elements and associated quantities into Pontis for the bridge.[16] The August 2011 routine inspection by Michael Baker made significant changes to element quantities, such as removing railings and adding to parapets and adding concrete diaphragms. The July 2015 inspection by AECOM reflected the transition from CoRe elements to National Bridge Elements, Bridge Management Elements, and Agency-Developed Elements (NBE/BME/ADE) as reflected in the Manual for Bridge Element Inspection (MBEI).[17] In this transition, all elements were adjusted to have four condition states for each element in similar format (CS1 = good; CS2 = fair, CS3 = poor, and CS4 = severe) with associated quantities (e.g., LF, SF, EA.) as appropriate to the element. MBEI introduced Defects with associated condition states (1 to 4) to describe and quantify specific types of deficiencies such as *Delamination/Spall/Patched Area* (1080) and *Cracking (RC and Other)* (1130), which are aggregated to form the overall element condition state. Where multiple defects occur in the same defined space, the defect in the most severe condition state is reported.

Over the period from 2007 to 2023, six different engineering firms conducted routine and special inspections of the bridge (plus underwater inspections), with some firms completing multiple inspections, but per RIDOT policy, consecutive routine inspections were not conducted by the same firm.

From the review, the following are highlights from the inspection reports as pertaining to the cantilever beams, drop-in beams, corbels, and tie-down rods, with emphasis on the Piers 6 & 7 cantilevers at Span 7. Each inspection report comprises a series of files in a folder, so individual file references are not provided. These passages have been edited slightly for grammar but largely convey the language as stated in the reports.

Routine Inspection by RIDOT, 2001

In this inspection, condition ratings were assigned to individual elements (this preceded Pontis implementation and CoRe element inspection). Overall superstructure condition rating was 5. The superstructure rating for Span 6 was 5 based on PT and RC beam condition and Span 8 superstructure rating was 6 based on PT cantilevers. Span 7 appeared to only reference the steel-framed span. In notes for Span 6 and 8, respectively it was noted “East cantilever has limited access due to pier 6 curtain walls” and “West cantilever has limited access due to pier 7 curtain walls” general observations of PT girders and corbels include web cracks, evidence of repairs, wet conditions due to overhead joint leakage, and honeycombed or cracked concrete on corbels.

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[16] AASHTO *Guide for Commonly Recognized (CoRe) Structural Elements*, American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., May, 1998.

[17] *Manual for Bridge Element Inspection*, 1st Edition (MBEI-1), American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C., 2013.

Underwater Inspection by Fuss & O'Neill, 2003

Report gave an overall rating of 7 for channel and channel protection (NBI Item 61) and overall rating of 6 for substructure.

Routine Inspection by AI Engineers, July 2007

The superstructure general condition rating (GCR), NBI item 59, was downgraded from 5 to 4 based on prestressed girder spalls with exposed strands and reinforcing bars at random locations. The deck GCR, NBI Item 58, was upgraded from 5 to 6 based on observed satisfactory condition of underside of deck. During this cycle, inspectors added types, quantities, and condition states for 30 CoRe elements. Cracks were noted, 0.005-inch width, on the Span 6 cantilever elevations that seem to align with PT ducts.

Routine Inspection by TranSystems, August 2009

The condition of lower areas of Pier 6 & 7 interior wall faces are noted as unconfirmed due to inaccessibility beneath catwalks. Asphaltic plug joints in all mainline spans except span 5 in good condition. Evidence of joint leakage was noted at random locations below the joints. For Element 109 Prestressed Concrete Open Girders (these are the post-tensioned cantilever beams), 11 entries reflect defects in cantilever beams of Spans 6 and 8 at Piers 6 and 7, respectively. The field notes page containing Span 7 and tied-down cantilevers of Spans 6 & 8 have very few notes on the cantilevers and focus mainly on the Span 7 steel span.

Routine Inspection by Michael Baker International, August 2011

Asphaltic plug joints in spans 3, 7, 8, 12 and at pier 14 have adhesion separation up to 25% of the length of the joint by up to ½" wide in the travelway and up to 1½" wide in the right shoulder. Evidence of joint leakage was noted at random locations below the joints and isolated areas of joint filler material were hanging down.

Cantilever ends at corbels typically have cracks with isolated efflorescence and rust, hollow areas and spalls with exposed anchorage plates (plates typically have laminated rust with negligible section loss). Pier 6 & 7 walls have hollow areas and spalls, and above the deck have hollow areas throughout entire face up to 4' x 3' with cracks with isolated efflorescence and rust. Piers 6 & 7 have high reinforced walls on all sides with a catwalk. There are scattered hairline cracks at the tops of these walls and insides were inspected from the catwalk and have random hairline cracks and the diaphragm supports at the span 7 sides typically have hollow areas and spalls with exposed rebar.

Cracking, efflorescence, and spalls clearly related to the embedded PT tendon profiles at Span 6, Girder E and Span 8, Girder B. Corbels inside of Pier 7 at catwalk [show] evidence of spalls and cracks. Corroded rocker bearings A of Span 7 at Pier 6 and Pier 7 evidence significant joint leakage above.

Routine Inspection by AI Engineers, August 2013

Substructure GCR, NBI Item 60, changed from 5 (Fair) to 4 (Poor). The inspection included critical findings regarding Span 13 Girder F and Span 6 Girder F webs and Span 14 Girders A & F loss of section and bearing. Cantilever ends at corbels typically exhibit cracks with isolated efflorescence and rust, hollow areas and spalls with exposed anchorage plates (plates typically have laminated rust with negligible section loss). Also, there is a spall up to 18″ x 12″ x 6″ deep exposing the anchor plate and undermining the bearing pad at east corbel to girder B in span 6. Pier 6 & 7. Interior walls and support columns exhibit areas of heavy spalls with corroded rebars on walls up to 3′ x 2′ x 5″ deep and columns up to full height by full width (2.5′) x 6″ deep. Field notes show cracking on web faces of cantilevers at Pier 6 that appear to align with PT ducts. WJE's review noted that field notes for Span 7 do not address cantilever portions of Spans 6 and 8 within the Pier 6 and Pier 7 walls.

Routine Inspection by AECOM, July 2015

The post-tensioned concrete cantilever girders in Spans #1 through #14 were reported to be in overall satisfactory to fair condition. Element 109 - Post Tensioned Cantilever I-Girders condition states (CS) quantities were adjusted to reflect the current conditions. [There is] isolated severe spalling with reduced bearing area and fully exposed/corroded stirrups to drop-in girder ends and cantilever girder ends. The defects table lists web diagonal cracks noted in many cases to follow the path of post-tension cables emanating at anchorage blocks. Numerous girders (16%) exhibit hairline diagonal web cracks that follow the path of post tension cables. These cracks generally start at the free end of the cantilever near the post-tension anchorage blocks and can extend up to 10′+ to the top of the webs. Isolated girders have hollow areas and shallow spalling along these cracks or cracked and hollow grout pocket patches. Post-tension anchorage blocks on the underside of the bottom flanges are typically cracked with efflorescence and rust staining. Numerous blocks are hollow or spalled with isolated exposed steel anchorage plates. In Span #7, the ends of the cantilever girders exhibit typical spalling up to full height x up to 7″ deep over the bearings with multiple fully exposed, debonded, and broken stirrups. The concrete cantilever girder pedestals on the interior walls of Piers 6 east wall and 7 west wall (behind the steel girder seats) exhibit typical spalling up to full height by full length by up to 7″ deep and undermine the cantilever girders.

Special Inspection by TranSystems, July 2016

This special inspection was for the superstructure and substructure only, to inspect the deteriorated condition of elements. For Element 109, prestressed girders and corbels typically exhibit spalls with exposed rebar, section loss on exposed rebars, hollow areas, concrete patches, efflorescence, rust stains and leakage stains. There are scattered cracks (some structural shear cracks). In Span 7 at interiors of Piers 6 and 7, the cantilever girder thin bearing pads are undermined up to 7" deep due to pedestal spalls and exhibit moderate to heavy crushing or bulging.

The post-tensioned concrete drop-in corbels in Spans #1 through #14 exhibit scattered hairline cracking open up to 0.012" wide, with few locations showing wider cracks, hollow areas, spalls with exposed reinforcing plates and efflorescence/rust stains at deteriorated locations. There are scattered locations with heavy accumulation of pigeon debris which limits inspection access, and isolated locations with evidence of leakage.

The corbels exhibit typical honeycombing of lower faces up to 2" deep, with scattered hairline cracking, hollow areas, efflorescence and rust staining. In multiple locations the cracks and hollow areas extend to the corbel undersides with spalling. Isolated underside [areas of the corbels] have heavier deterioration with multiple cracks, leakage stains, hollow areas and spalls up to 2" deep with exposed reinforcement along corbel edges…The lower end faces of the corbels exhibit typical intermittent hollow areas and spalls at the corners; some with exposed corroded reinforcing plate and up to 3.5" deep; some elastomeric bearings are undermined due to spalls. The upper end faces of the corbels beyond the drop-in beam bearings exhibit typical scattered spalls up to 2" deep with rust staining and exposed and corroded post-tension reinforcing plates.

Routine Inspection by Collins Engineers, July 2017

In Span #7 at the interior of Piers #6 and #7, the ends of the cantilever girders exhibit spalling up to full height by up to 8” deep over the bearings with multiple fully exposed, debonded, and broken rebars. The cantilever support pedestals on the interior walls of Piers #6 east wall and Pier# 7 west wall (behind the steel girder seats) exhibit random hairline cracks, isolated hollow areas and spalls without and with exposed rebar which undermine the masonry plates. The spalling on the cantilever support pedestals has exposed and debonded rebar, section loss on exposed rebar, and isolated broken rebar. The cantilever support pedestals exhibit uneven bearing pedestals and missing/deteriorated grout pads resulting in gaps under the masonry plates and loss of bearing area at random bearings. There are several defects which have been repaired or in the process of being repaired during the inspection.

The east pier wall at Pier #6 and the west pier wall at Pier #7 exhibit hollow areas and spalls. The cantilever support pedestals on the interior walls of Pier #6 east wall and Pier #7 west wall exhibit hollow areas. The cantilever support pedestals exhibit uneven bearing pedestals and missing/deteriorated grout pads resulting in gaps under the masonry plates. The east pier wall at Pier #6 and the west pier wall at Pier #7 exhibit spalls with exposed and debonded rebar with section loss. The cantilever support pedestals on the interior walls of Piers #6 east wall and Pier #7 west wall exhibit spalls with exposed rebar. The spalling on the cantilever support pedestals have exposed and debonded rebar, section loss on exposed rebar, and isolated broken stirrups.

Special Inspection by Michael Baker, July 2018

The purpose of this special inspection is to monitor the condition of the superstructure and substructure due to deteriorated condition per BI-011 on file dated 10/26/15. Note, rehabilitation construction activities are on-going and were occurring at the time of this special inspection. Based on the results of this special inspection, the bridge overall is in poor condition. Substructure (Rating = 4) – The substructure has hollow areas and spalls at the cantilever pedestals. The pier walls that support span 7 have cracking. The cantilever support pedestals on the interior walls of Piers 6 east wall and Pier 7 west wall (behind the steel girder seats) have scattered up to 16” long x 3/16” wide vertical and horizontal cracks, and up to 3’ high x full pedestal width concrete patches. Amended quantities of defects in Element 109 – Prestressed Open Concrete Girder / Beam (incl. new spalls, exposed reinforcing bar and delaminations in Span 7) and Element 210 – Reinforced Concrete Pier Wall (some due to ongoing repairs). Several long diagonal cracks were noted in webs of cantilever beams, some potentially aligned with PT ducts.

Routine & Special Inspection by AECOM, July 2019

"Rehabilitation construction is on-going and there are multiple defects that have been repaired or are in the process of being repaired…There is scaffolding in place throughout the structure allowing access to the drop -in girder ends and corbels.” For Element 109, the corbels exhibit typical cracked, hollow and spalled areas with exposed post tensioned anchor plates on the drop-in span sides throughout. The other faces and undersides exhibit isolated cracks, hollow areas and minor spalls. The cantilever girders exhibit typical hairline diagonal cracks along the post-tensioned cable lines, some sealed and unsealed, isolated vertical cracks and hollow area over the pier columns and typical hollow/spalled post-tensioned anchor blocks on the undersides. Other remaining areas exhibit random minor cracked, hollow and spalled areas. The cantilever ends in Span #7 at Pier #6 and Pier #7 (accessed via the catwalks on the interior walls of the piers) exhibit typical hollow areas/spalls up to full height with fully exposed and debonded stirrups and reduced bearing areas.

Special Inspection by AECOM, July 2020

"The special inspection includes the superstructure and substructure…There is scaffolding in place throughout the structure (from previous bridge rehabilitation construction) allowing access to the drop-in girder ends and corbels. There is typical construction debris scattered through the scaffolding."

The condition rating for Item 60 - Substructure has been increased from (4 - Poor) to (6 - Satisfactory) based on the repairs which have been made throughout the bridge substructure elements.

The cantilever ends in Span #7 at Pier #6 and Pier #7 (accessed via the catwalks on the interior walls of the piers) exhibit typical hollow areas/spalls up to full height with fully exposed and debonded stirrups and reduced bearing areas. The Element 109 Defect Table for Spans 6 - 8, lists several cantilever beams that exhibit diagonal cracks aligned with PT anchorage blocks.

Note this inspection report contains similar or identical phrasing of observations for many elements to the preceding routine inspection, and the element condition tables contain many typographical errors that resemble optical character recognition errors rather than typing errors. However, the photographs do appear to be original to this inspection.

Routine and Underwater Inspection by Jacobs, July 2021

This inspection reports very few changes in conditions or defects overall and none to Elements 109 and 210. The report contains identical typographical errors to those found in the preceding special inspection report by AECOM. Photographs appear to be original to this inspection. There were 4ft CS3 reported for scour undermining (Element 220) from the 2021 UW report.

Special Inspection by TranSystems, July 2022

For Element 109, there were changes to defects 1080 (4 ft improved from CS3 to CS2) and 1090 (5 ft improved from CS3 to CS2). The only other changes referred to graffiti and nesting birds.

Routine Inspection by AECOM, July 2023

For Element 109, some improvements were reflected, with quantities moving from CS3 & CS4 to CS2. Specifically, condition state quantities changed from: 11,650 ft CS1; 1,299 ft CS2; 1,464 ft CS3; 130 ft CS4 to 11,647 ft CS1; 1,397 ft CS2; 1,394 ft CS3; 105 ft CS4.

Special Inspection by AECOM, December 2023

The inspection, necessitated by a critical finding during construction, noted some deterioration of Element 109 from 11,647 ft CS1; 1,397 ft CS2; 1,394 ft CS3; 105 ft CS4 to 11,621 ft CS1; 1,408 ft CS2; 1,409 ft CS3; 105 ft CS4.

Cantilever Beam Element Condition State Progression

The subject post-tensioned concrete cantilever beams for this structure are catalogued as Element 109, *Prestressed Concrete Open Girder* in Pontis/BrM. The reported condition states from each inspection are tabulated in Table 2. Note that a significant change occurred between 2013 and 2015 reflecting a dramatic increase in the percentage of the element rated in CS1. Since no rehabilitation occurred during this period, the change in condition is likely attributable to the transition from Core Elements to NBEs and BMEs with incorporation of quantified Defects. This was an administrative change and does not reflect a commensurate improvement in condition. Rather, it is analogous to measuring with a different scale (e.g., meter vs. yard). Figure 15 shows the proportions in CS1 through CS4 after 2013. The jump in CS4 quantities in 2016 reflects discovery of additional defects during the special inspection. Conversely, the improvement in condition between October 2017 and July 2018 would be associated with the rehabilitation project underway at that time. However, little significant improvement in condition occurred after that time and some previously good portions of the girders continued to deteriorate.

Table 2. Element 109 Prestressed Concrete Open Girder – Condition State History

Date	Total	Unit	CS1		CS2		CS3		CS4	
July 2007	14,543	FT	40.0%	5,817	30.0%	4,363	20.0%	2,909	10.0%	1,454
June 2009	14,543	FT	40.0%	5,817	30.0%	4,363	20.0%	2,909	10.0%	1,454
August 2009	14,543	FT	40.0%	5,817	30.0%	4,363	20.0%	2,909	10.0%	1,454
August 2011	14,543	FT	53.2%	7,739	30.0%	4,363	15.0%	2,181	1.8%	260
August 2013	14,543	FT	53.2%	7,737	30.0%	4,363	15.0%	2,181	1.8%	262
August 2013	14,543	FT	53.2%	7,737	30.0%	4,363	15.0%	2,181	1.8%	262
July 2015	14,543	FT	84.9%	12,347	10.0%	1,454	4.0%	576	1.1%	166
July 2016	14,543	FT	80.6%	11,724	4.3%	629	11.5%	1,673	3.6%	517
July 2017	14,543	FT	80.6%	11,721	4.3%	632	11.5%	1,673	3.6%	517
October 2017	14,543	FT	80.6%	11,721	4.3%	632	11.5%	1,673	3.6%	517
July 2018	14,543	FT	80.7%	11,733	8.7%	1,268	9.7%	1,407	0.9%	135
July 2019	14,543	FT	80.7%	11,733	8.7%	1,268	9.7%	1,407	0.9%	135
July 2020	14,543	FT	80.1%	11,650	8.9%	1,290	10.1%	1,468	0.9%	135
July 2021	14,543	FT	80.1%	11,650	8.9%	1,290	10.1%	1,468	0.9%	135
July 2022	14,543	FT	80.1%	11,650	8.9%	1,299	10.1%	1,464	0.9%	130
July 2023	14,543	FT	80.1%	11,647	9.6%	1,397	9.6%	1,394	0.7%	105
December 2023	14,543	FT	79.9%	11,621	9.7%	1,408	9.7%	1,409	0.7%	105
January 2024	14,543	FT	79.9%	11,623	9.7%	1,409	9.7%	1,406	0.7%	105

Source: RIDOT AASHTOWare BrM

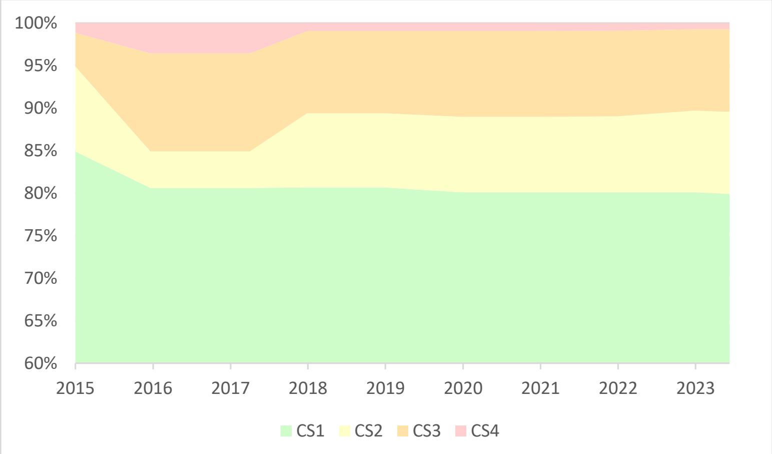

Figure 15. Element 109 Condition State History (2015 to 2023)

2016-2019 Rehabilitation

In January 2015, AECOM issued a Final Technical Evaluation Report[18] for Washington Bridge 700, starting with a summary of a special inspection completed in 2014 and progressing to recommendations for a rehabilitation.

In its summary of the 2014 special inspection, regarding the prestressed girders, AECOM noted:

- "Drop-In Prestressed Beam Ends, Reinforced Concrete Diaphragms and Post-Tensioned Cantilever Corbels exhibit prevalent cracked, hollow sounding and spalled concrete with multiple exposed stirrups, longitudinal reinforcement and prestressing strand ends below drop-in span deck joints. Isolated spalls in these areas reduce drop-in beam bearing area and/or undermine bearing pads throughout the bridge.
- Post-Tensioned Cantilever Beams exhibit typical hairline web cracking running parallel to the post-tensioned cables with prevalent spalling of concrete anchorage blocks exposing moderately to heavily corroded steel anchorage plates throughout. Isolated beams exhibit hairline vertical cracking and hollow concrete over bearing locations."

The proposed major repairs included:

- concrete beam end repairs,
- bearing repairs,
- deck repairs,
- beam end strengthening (FRP),
- pier cap strengthening (FRP),
- waterproofing membrane and asphalt wearing surface replacement, and
- joint repairs and elimination.

[18] Final Technical Evaluation Report - Washington Bridge North No. 700, Providence and East Providence, Rhode Island RI Contract No. 2014-EB-003, AECOM technical Services, Inc., January 2015, *2014-EB-003 Washington Bridge Technical Evaluation Report.pdf*.

Several parts of the structure were identified as requiring strengthening to increase the live load carrying capacity from HS-20 to HL-93 design loading, for which externally applied fiber reinforced polymer (FRP) was proposed to strengthen concrete beam and pier cap members.

AECOM's report stated, "Two joint elimination scenarios were evaluated and a scenario that eliminates 65% or 23 of the existing 35 deck joints is recommended as the preferred alternative. An approximate 6' wide portion of deck will be removed and reconstructed at the 23 locations where joints will be eliminated. The remaining 12 joints will consist of strip seal joints (8 locations), asphaltic plug joints (2 locations) and pourable joint seals (2 locations). The impact that joint elimination has on the pier columns was investigated and all pier columns were found to be adequate." Figure 16 shows the original joint configuration and 2 joint replacement scenarios. Figure 17 presents the resulting link slab location plan for the selected scenario 2, as shown in rehabilitation plans.

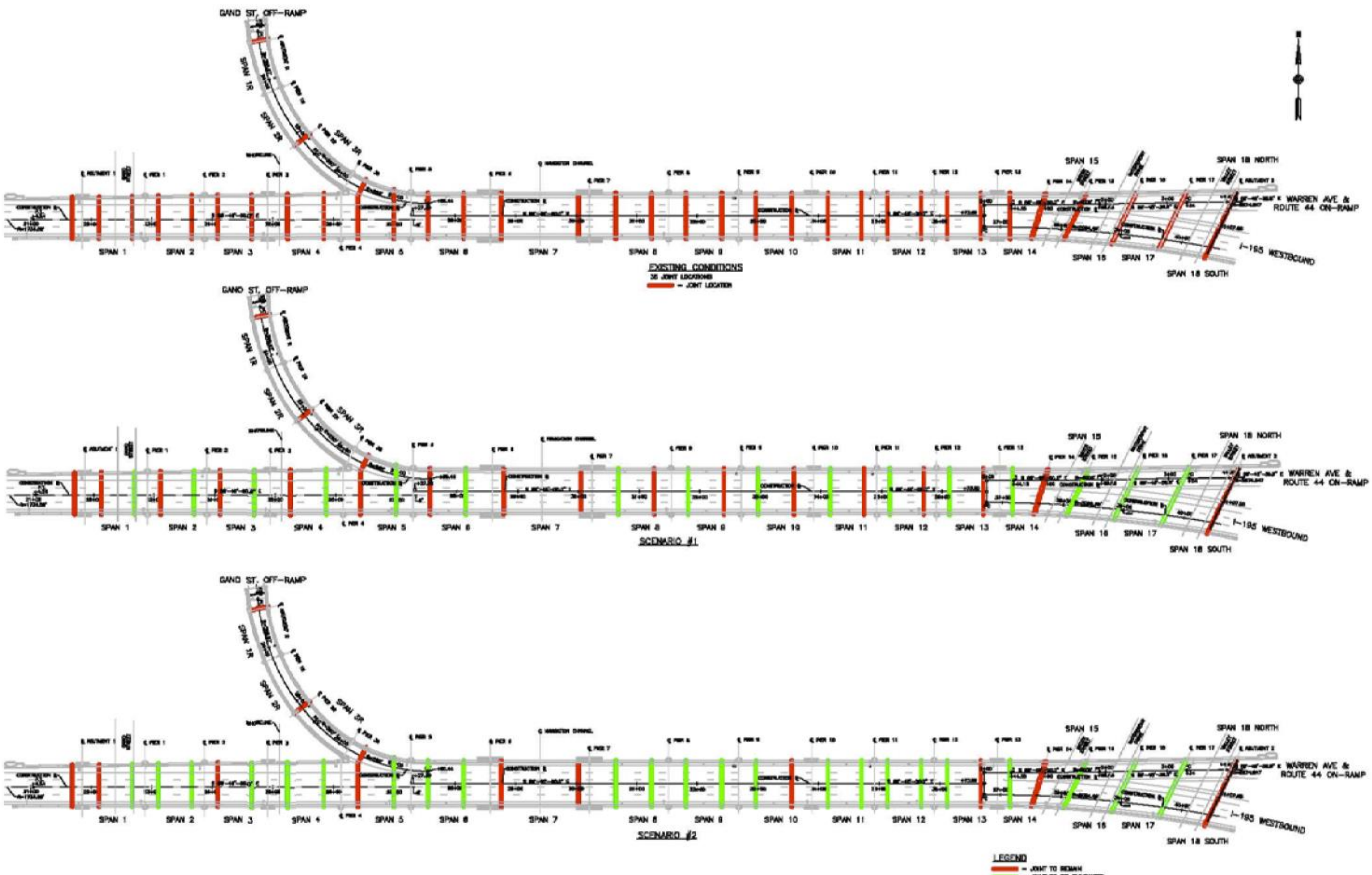

Figure 16. Joint elimination and replacement schemes, AECOM Final Technical Evaluation Report, January 2015. Top is as-constructed, and middle and bottom are two alternatives. Red indicates joint to be replaced (strip seal, asphalt plug joint, or pourable seal) and green indicates joint to be eliminated with deck closure pour.

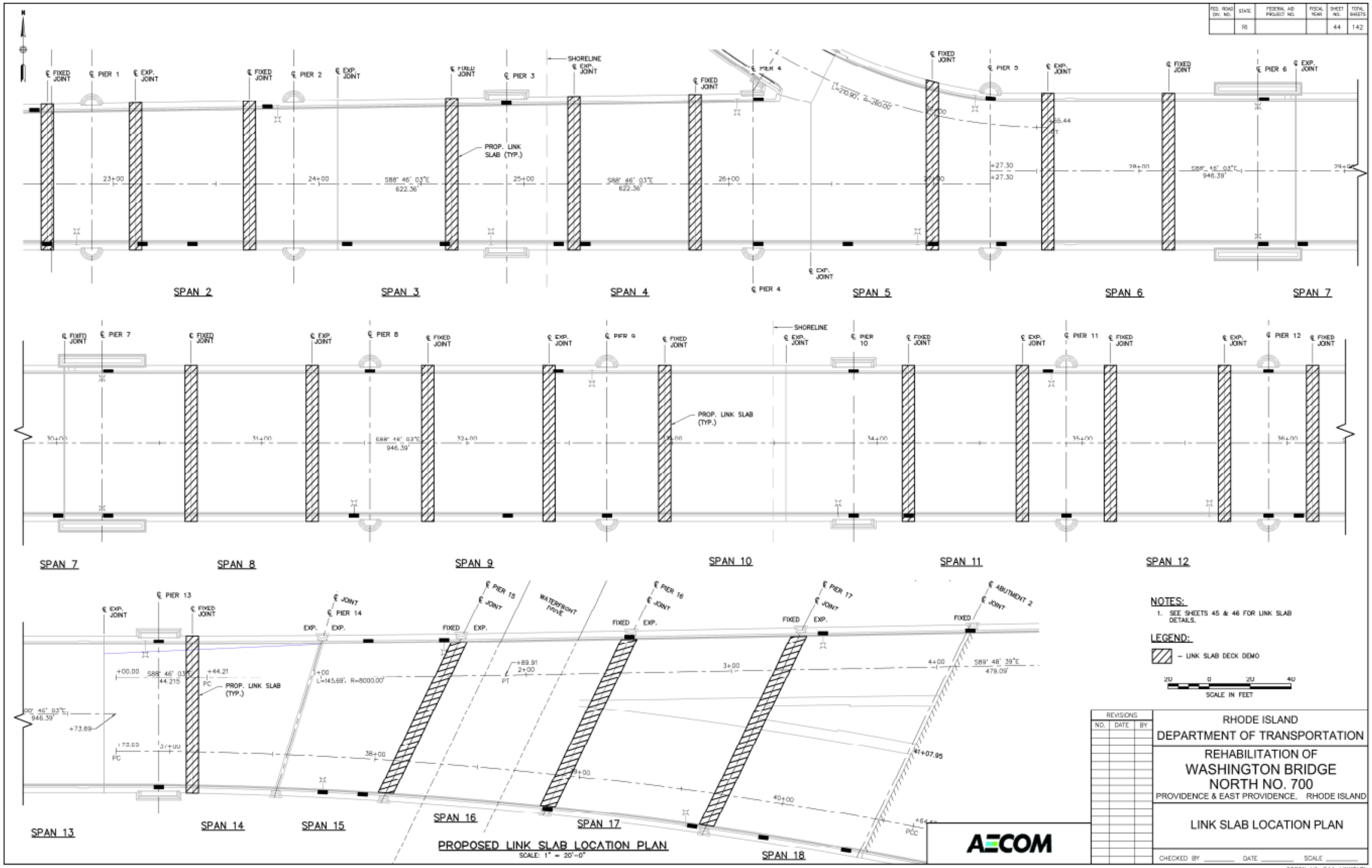

Figure 17. Link slab Location Plan from 2016 Rehabilitation plans, Sheet 44, RIDOT Contract 2016-CB-059, 2016.

Structural Modeling

Using CSI Bridge 2014, "AECOM modeled the portion of the structure from span 1 thru span 14…The models accounted for dead loads, live loads, braking force, temperature loads and site specific seismic loads. Geometry was based on the existing 1967 drawings."

"Except for span 7, spans 1 thru 14 consist of prestressed concrete drop-in beam spans with variable depth post tensioned cantilever beams. This superstructure is supported by multi-column pier bents founded on deep pile foundations. The beams support a reinforced concrete deck with a three inch asphalt wearing surface. The deck is modeled as shell elements and beams, diaphragms and columns are modeled as frame elements. Each drop-in span is supported by a fixed and expansion bearing and this has been accounted for in the beam end releases defined in the model. The foundation pile cap and pile system were not modeled as part of the preliminary engineering effort and the restraint at the base of the reinforced concrete rectangular pier columns was taken as fully fixed for rotation and translation. It is anticipated that this full fixity can be more accurately modeled in final design by replacing full column fixity with a linear spring that reflects the stiffness provided by the pile layout."

In the summary of design loads for its analysis, AECOM noted, "Per RIDOT 3.8, assume a Tmax of 100 degrees and a Tmin of 0 degrees. Assume an initial temperature of 50 degrees when evaluating temperature increase and an initial temperature of 70 degrees when evaluating temperature drop. So Trise = 100-50 = 50 degrees; Tfall = 70 – 0 = 70 degrees (entered as -70 in CSI Bridge)."

AECOM determined, "No columns were found to need strengthening when checked against the existing joint configuration and for both of the proposed joint elimination scenarios. " In the Final Technical Evaluation Report, when considering evaluation of existing beam capacities and need for member strengthening, AECOM stated "The 2012 rating results for drop-in span post-tensioned corbels and post-tensioned cantilever girders produced rating factors above statutory levels for all load cases and limit states; thus a re-evaluation was not necessary."

"A link slab is proposed where joints are proposed for elimination. With Scenario 2, the link slab detail would be constructed at 23 locations. A link slab is comprised of a reinforced concrete deck with a length that extends approximately 5% to 7% of each adjacent span. A bond breaker is applied between the slab and the top flange of the beam to prevent composite action in this region." Figure 18 reflects the detail issued for bid in the rehabilitation plans.

To accommodate the restraint added by the partial depth closure pours, the joints at the ends of the newly adjoined units were to be reworked to accommodate greater anticipated movement. Figure 19 shows the proposed detail for a strip seal at a drop-in-cantilever corbel support, for example.

Concerning the detailing of the joints to remain AECOM stated, "If deck continuity is constructed at 23 of the existing joints, 12 joints would remain on the bridge. The appropriate joint type at these 12 locations is based primarily on the amount of thermal movement that is anticipated. Table 7 – 1 [see Table 3] summarizes the total design movement that will need to be accommodated at each proposed joint location. The existing movement is provided for reference. As expected the proposed movement is greater than the existing due to the elimination of 65% of the deck joints."

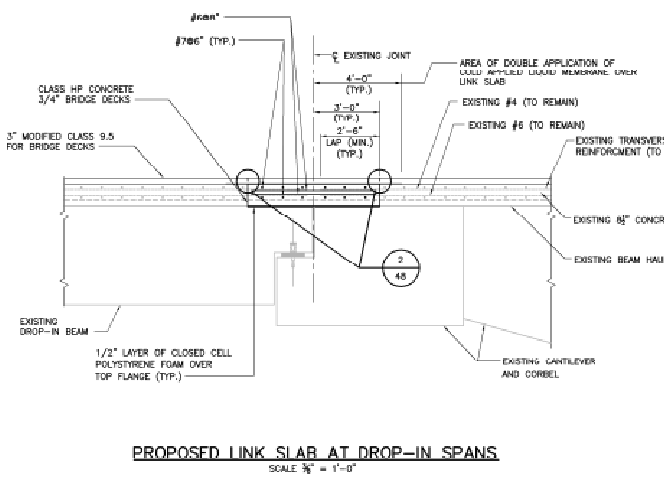

Figure 18. Link Slab Detail, Sheet 45, RIDOT Contract 2016-CB-059, 2016.

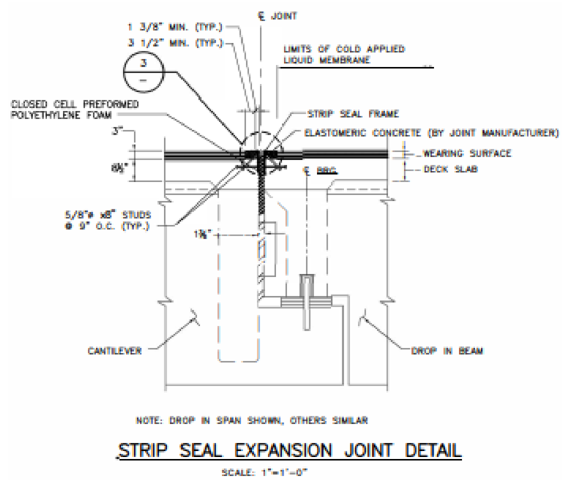

Figure 19. Strip Seal Joint Detail, Sheet 48, RIDOT Contract 2016-CB-059, 2016.

Table 3 represents a selective excerpt from the cited Table 7 – 1, showing only the joints where it is proposed that joints remain and accommodate movement of the groups of adjacent spans connected by proposed joint closures. Note that the recommended scenario two to eliminate 23 of 35 joints results in closure of the three joints immediately west of Pier 6 east joint and closure of the 5 joints immediately east of Pier 7 west joint. These closures would result in concentration of thermal movement over segments west of Pier 6 for approximately 236.2 feet of the bridge, and over segments east of the Pier 7 joint over approximately 352.6 feet of the bridge. As a result, based on the values reported by AECOM in its report, movement at Span 7, Pier 6 cantilever was predicted to increase from 1.588 to 3.226, a factor of 2.0 and movement of Span 7, Pier 7 cantilever was predicted to increase from 0.482 to 2.808, a factor of 5.8.

Table 3. Total design movement to be accommodated at joint locations.

Joint Location	Total Movement Existing	Total Movement Proposed	Recommended Joint Type
West Abutment East Joint	0.983	1.328	Strip Seal
Pier 2 East Joint	0.822	1.536	Strip Seal
Pier 4; East Joint	0.915	2.105	Strip Seal
Pier 6; East Joint	1.588	3.226	Strip Seal
Pier 7; West Joint	0.482	2.808	Strip Seal
Pier 10; West Joint	0.954	2.684	Strip Seal
Pier 13; West Joint	1.119	1.624	Strip Seal
Pier 14	-	TBD	Strip Seal
East Abutment	< 0.25	<0.25	Pourable Joint Seal
Gano St Ramp Span 1R	<0.5	<0.5	Asphaltic Plug
Gano St Ramp Span 2R	<0.5	<0.5	Asphaltic Plug

Source: Excerpt from Table 7 -1, AECOM Final Technical Evaluation Report, 2015

It is not clear from this report that AECOM's evaluation explicitly considered the configuration of the tie-down rods at the Pier 6 & Pier 7 expansion joints to remain after adjacent joints were closed. Although a detailed analysis is discussed of the influence of the closure on loads induced in the cantilever piers, the analysis does not appear to recognize or consider changes in forces (tension, moment, or shear) in the tie-down rods at the unbalanced ends of the cantilevers at Piers 6 & 7 or Abutment 1.

Construction

The project was issued under Rhode Island Contract No. 2016-CB-059 and awarded to Cardi Corporation in December 2016, with a target completion date of October 18, 2019. WJE was given access to contract and bid documentation and the design plans. However, WJE received very limited access to information regarding the execution of the construction. It is evident that construction began in early 2017. Early beam repair work concentrated at the east end of the bridge but scaffolding and work platforms were placed throughout the bridge and demolition for repairs appeared to have been undertaken over many areas of the bridge simultaneously. Deck joint removal activities had begun, with demolition of joints along the southern lanes taking place along the length of the bridge. However, no link slabs were placed during the course of construction. Reportedly significant traffic control issues occurred during construction, causing the work to be discontinued, and leaving many excavated repairs opened and incomplete for an extended period of months. The contractor was required to replace the removed joint seals and return the deck to a traversable condition. Eventually, RIDOT canceled the contract with most of the repairs not completed, including FRP strengthening and link slab placements.

2021-2023 Rehabilitation

Following cancellation of the 2016 construction contract, the project was re-issued as a design-build (DB) contract. Contract 2021-DB-020 was ultimately issued to a team of Barletta Engineering Heavy Division and Aetna Bridge Company, with design engineers VHB supported by VN Engineers, Inc.

Link Slab Design

The design under this DB followed largely the scope of the 2016 project, to include cantilever repairs, link slab installation, and FRP strengthening. However, the link slab locations were revised to closer reflect the Scenario #1 concept under AECOM's Final Technical Evaluation Report, wherein alternating joints were replaced with link slabs and with Emseal joints, thereby reducing the number of link slabs to be installed and the number of spans over which the deck would be continuous. In VHB's design, 14 link slabs were to be installed, rather than 23 (Figure 20).

With regard to Piers 6 and 7 in Span 7, the joints at the concrete cantilever to steel superstructure transition at the pier walls were retained. The deck joints above the first cantilever corbel-to-drop-in beam support from the Pier 6 and Pier 7 pier walls in Span 7 were to be replaced with a link slab (Figure 21). The effective length of continuous deck increased from about 63'3" to 120'6", or approximately double. Figure 22 shows the intended details for the joint demolition and link slab installation at the drop-in spans. Figure 23 shows a concrete repair detail in the vicinity of the Pier 6 and Pier 7 cantilever tie-down connections.

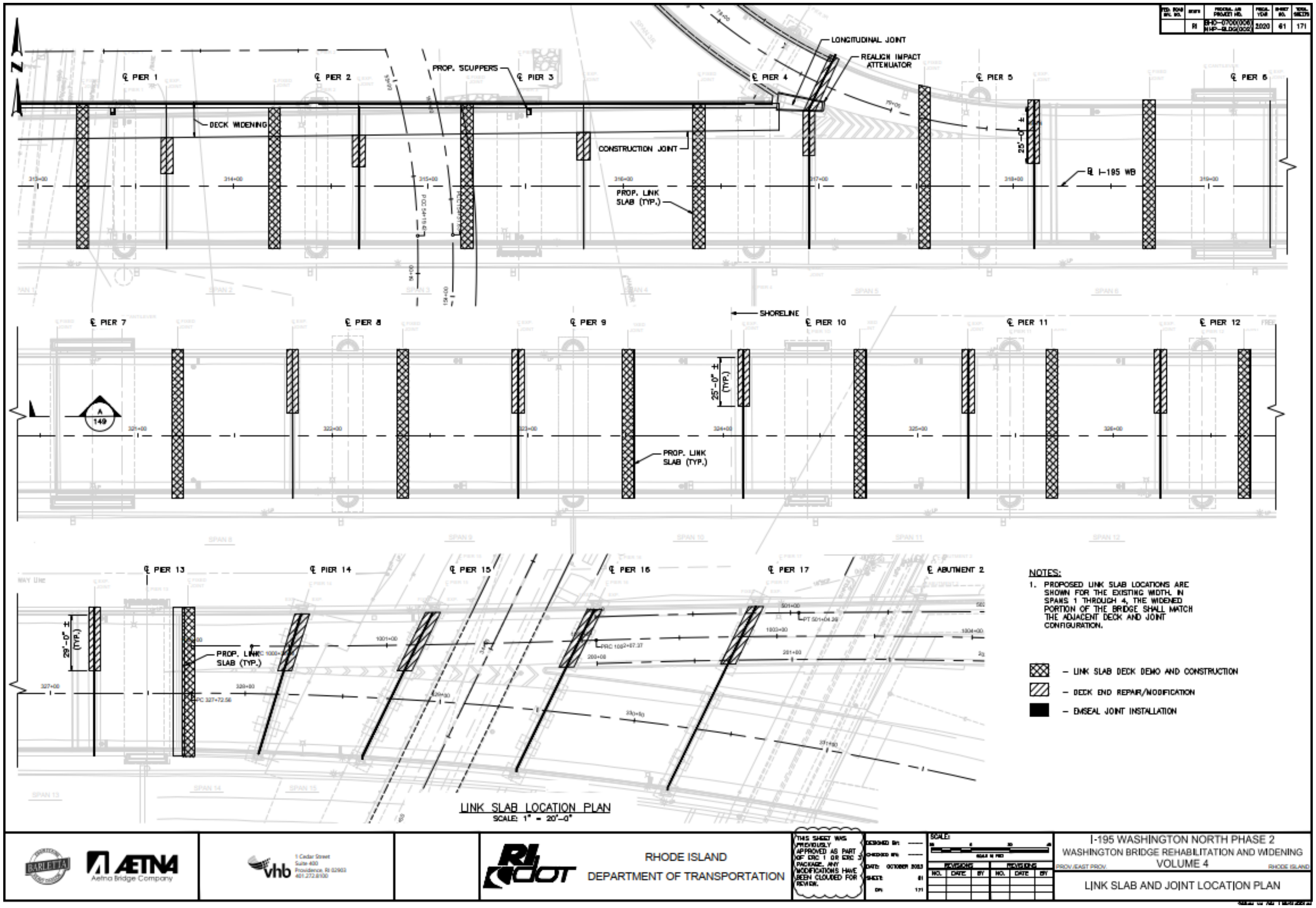

Figure 20. Link Slab and Joint Location Plan for Contract 2021-DB-020 (Sheet 61 of IFC Submission, I-195 Washington North Phase 2 Rehabilitation, Volume 4).

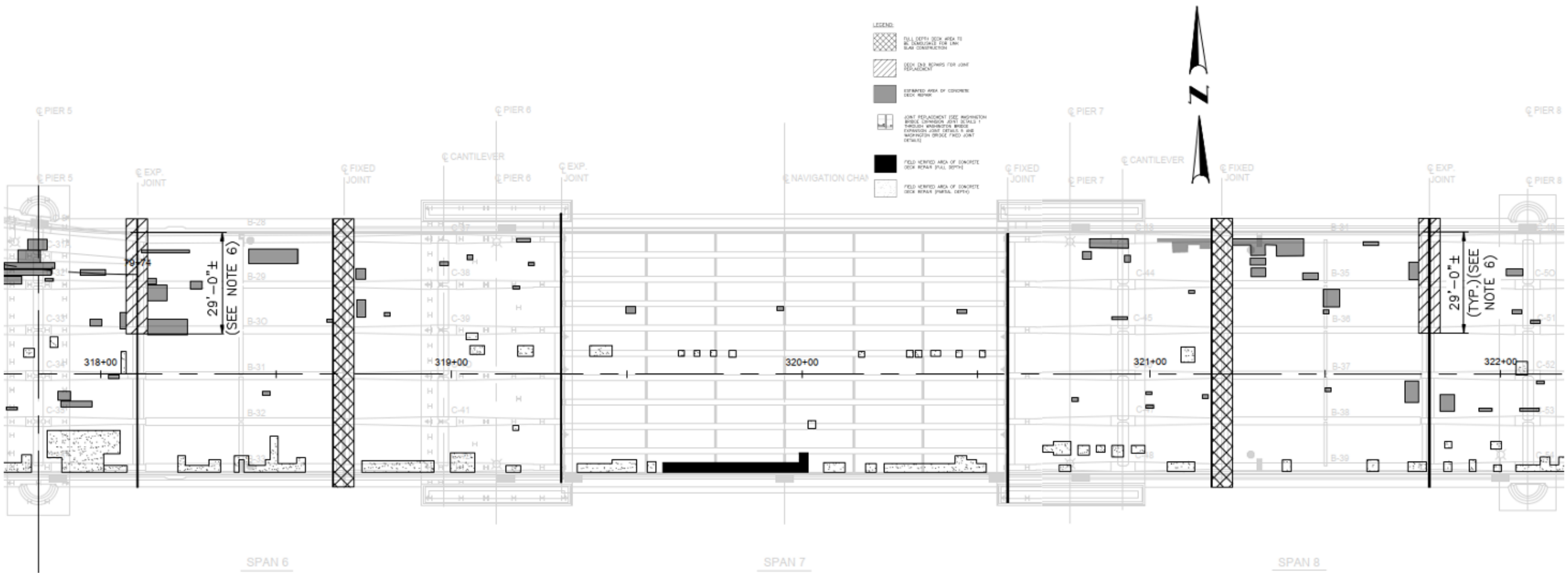

Figure 21. Spans 6 through 8 repair plan for Contract 2021-DB-020 (Compiled from excerpts of Sheets 57 and 58 of IFC Submission, I-195 Washington North Phase 2 Rehabilitation, Volume 4).

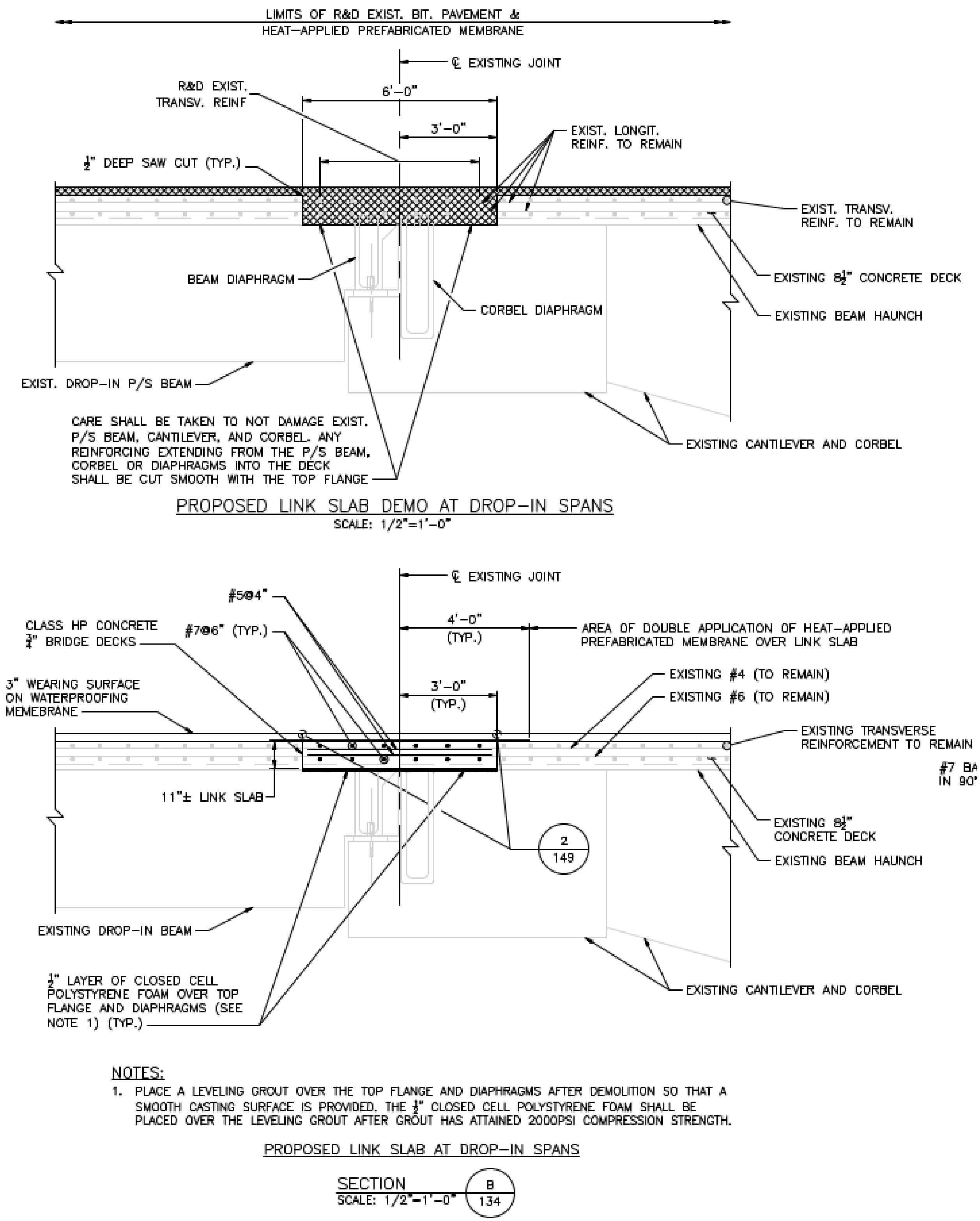

Figure 22. Link Slab Details for Contract 2021-DB-020 (Sheet 134 of IFC Submission, I-195 Washington North Phase 2 Rehabilitation, Volume 4).

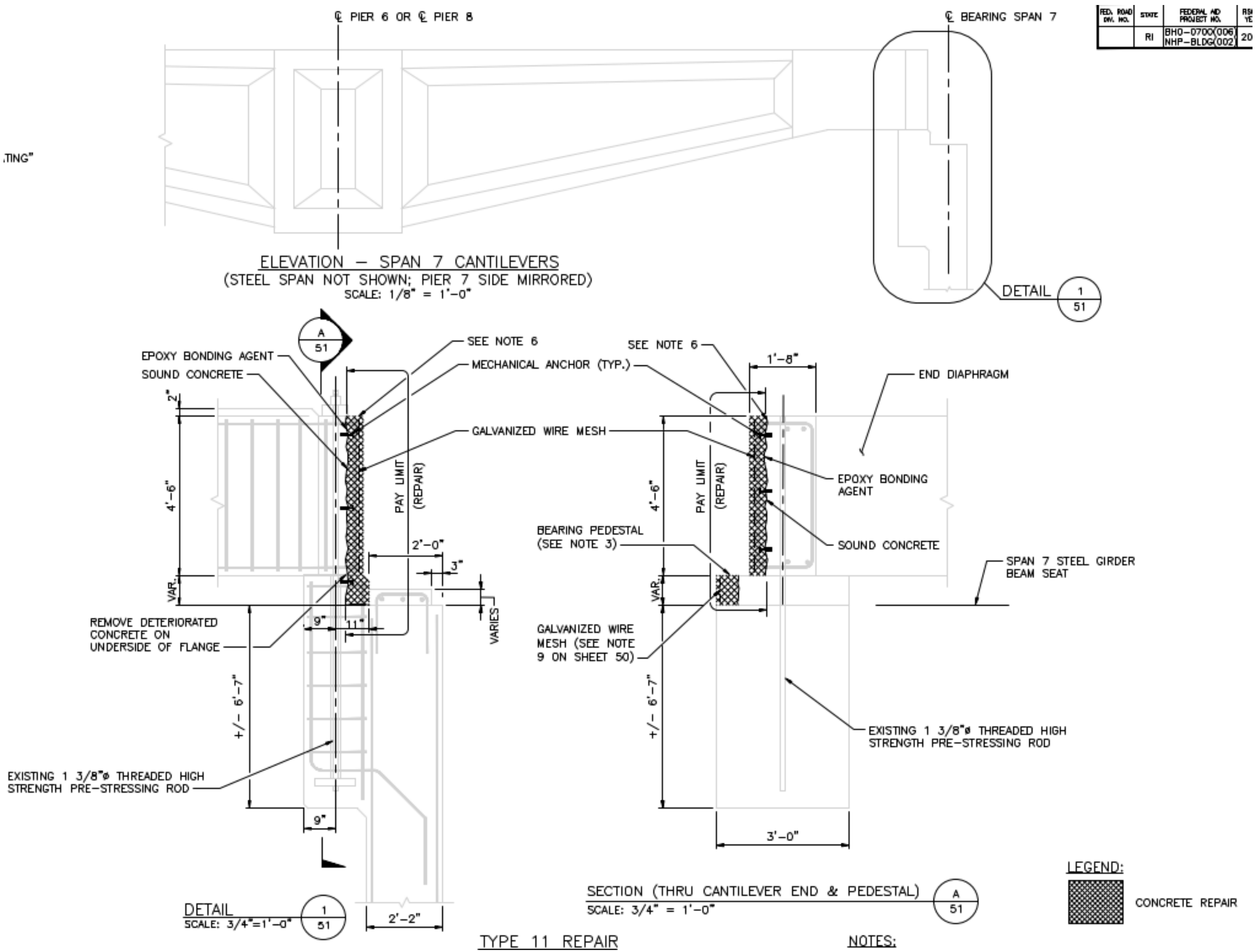

Figure 23. Type 11 Cantilever Repair Detail at Span 7 for Contract 2021-DB-020 (Sheet 51 of IFC Submission, I-195 Washington North Phase 2 Rehabilitation, Volume 4).

Issued for Construction (IFC) structural calculations state the following:

> "The Link Slab is designed for negative moment due to service load rotations only. There is a concern with the existing fixed bearings at Washington Bridge 700 as a fixed-fixed support will result in a large axial force in the link slab, which will create an equally large axial force (couple) on the existing pintles. Furthermore, replacing the expansion joints with link slabs is a concern since the structure is comprised of prestressed and post-tensioned members. By locking in thermal expansion between substructure elements the superstructure members will be put into compression and therefore reduce the prestress and post-tensioning stresses in the beams and could compromise their structural capacity. Therefore, the fixed joints will be evaluated for link slab replacement and the annular space around the pintles will be reviewed to confirm rotation can occur without engaging the pintle and causing a significant force couple."[21]

Analysis then presents determination of length of the link slab and calculation of end rotation of girders considering thermal gradient loading. The calculations state, "Only drop in span will be investigated for rotation. It is assumed the cantilever span will not deflect much." The analysis then considered axial forces on the link slab from live load and temperature. A discussion of bearing connection capacity focused solely on the pier support connection to check capacity and then annular space around the pintles based on shifting the point of rotation from the pintle to the top of the beam at the link slab. Finally, cracking moment, crack control and crack width of the link slabs were evaluated, followed by link slab moment capacity.

IFC structural calculations did not evaluate changes in temperature-related joint movement caused by link slabs over the cantilever beams at Piers 6 and 7, though movement was calculated for the steel-framed portion of Span 7.[19] This calculation focused on setting width of the expansion joint.

WJE did not find calculations to evaluate loading of the tie-down bars at the Pier 6 and Pier 7 cantilever connections. The following passage was found regarding tie-downs at the Abutment 1 support:

> "<u>External Springs and Boundary Conditions</u>
>
> *PT Beam 1 Supports*
>
> The west end of the PT beam located at Abutment 1 is restrained against uplift via a Tie Down detail consisting of prestressing rods cast into reinforced concrete and held in place by high strength steel plates attached to the rods via tightened nuts at either end. The result is a system that restricts lateral and vertical movement of the PT Beam end with respect to the supporting cantilever stem. Additionally, the base of the PT beam is restrained from moving laterally via a tapered steel down rod, but not significantly restrained against rotation.
>
> - Tie-down Point for PT Beam 1:
>   - Beam Lines A, B, and D thru F
>   - Rotational Degrees of Freedom – All Free
>   - Translational Degrees of Freedom – Only X and Z Fixed
>   - Beam Line C
>   - Rotational Degrees of Freedom – All Free
>   - Translational Degrees of Freedom – Only X and Z Fixed"[21]

Figure 24 was included as part of this passage and shows the Abutment 1 tie-down detail and the cantilever beam support and pintle detail (highlights by VHB). No similar discussion is presented in the IFC calculations for cantilever tie-down locations at Piers 6 and 7.

[19] Volume 4 - Washington Bridge Rehabilitation and Widening IFC Submission October 2023, VHB, *Volume 4 - IFC Calculation Book.pdf*

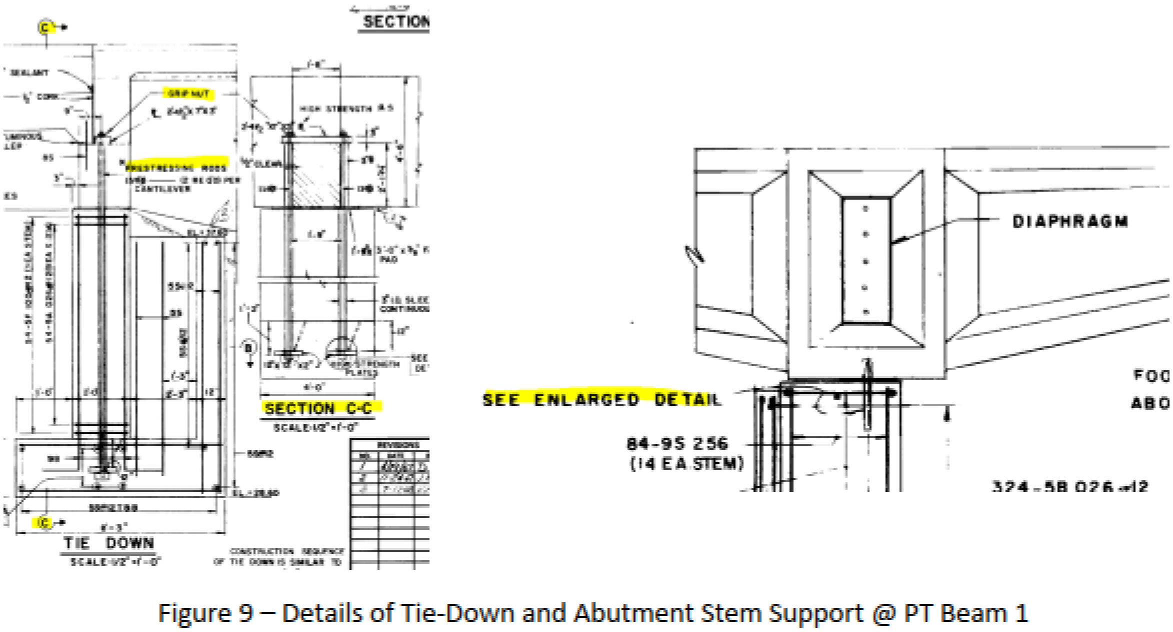

Figure 24. Excerpt regarding Abutment 1 cantilever restraint from IFC Calculations[20]

Construction

For the 2021 design-build contract, the work was divided into five stages to support maintenance of traffic. Each stage represented a lane on the bridge, with Stage 1 being the leftmost high-speed lane and Stage 4 being the rightmost low-speed lane, with Stage 5 representing a final closure immediately left of Stage 4 (Figure 25). At the time the fractured tie-down rods were discovered, the contractor had completed Stages 1, 2, and 3, Stage 4 was partially complete, and Stage 5 had not started. The link slab work started on the bridge's east end and progressed in a westerly direction.

[20] Volume 4 - Washington Bridge Rehabilitation and Widening, IFC Submission October 2023, VHB, *Volume 4 - IFC Calculation Book.pdf*

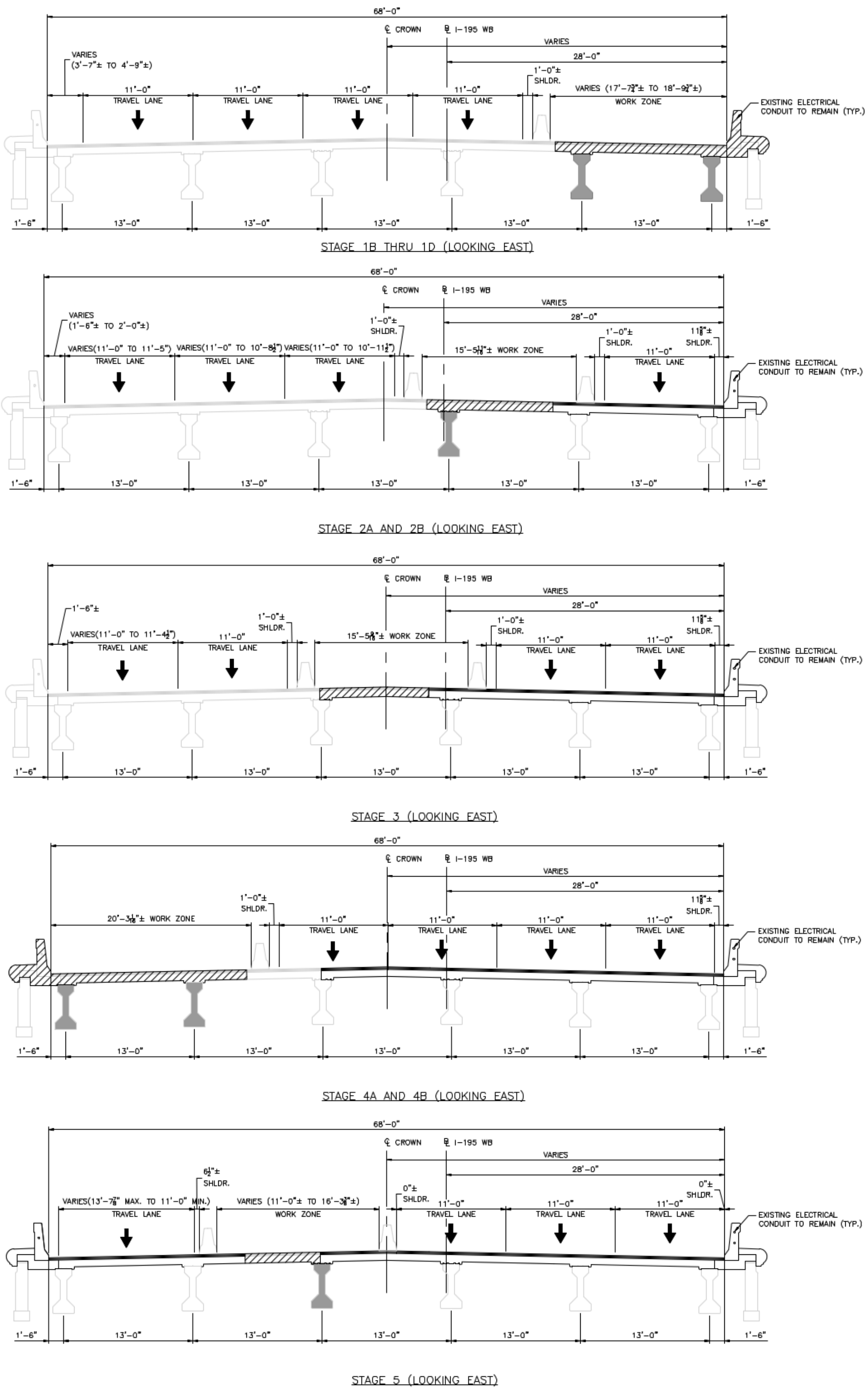

Figure 25. Stages 1 through 5 construction sequence for Contract 2021-DB-020 (Sheets 17, 20, 22, 24, & 26 of IFC Submission, I-195 Washington North Phase 2 Rehabilitation, Volume 4).

According to records provided by the RIDOT Construction Resident Engineer, link slab demolition and placements completed for Stages 1 through 4 took place between March 2022 and October 2023. The DB team completed Stage 1 in October of 2022; Stage 3 was completed next in February of 2023; followed by Stage 2 in July 2023. Stage 4 was approximately 40% complete when work stopped. No work was completed on Stage 5.

Table 4. Demolition start and link slab pour dates for Stages 1 through 4 (from RIDOT Construction Resident Engineer)

Span #	Stages							
	1		3		2		4	
	Start Demo Existing	Pour New	Start Demo Existing	Pour New	Start Demo Existing	Pour New	Start Demo Existing	Pour New
1	4/26/22	10/6/22	1/5/23	2/28/23	6/19/23	7/13/23		
2	4/26/22	10/6/22	1/5/23	2/28/23	6/14/23	7/13/23		
3	4/24/22	10/6/22	1/5/23	2/28/23	6/14/23	7/13/23		
4	4/21/22	9/29/22	1/5/23	2/28/23	6/13/23	7/13/23		
5	4/17/22	9/29/22	1/5/23	2/28/23	6/8/23	7/13/23	9/27/23	10/5/23
6	4/12/22	9/29/22	12/27/22	2/28/23	6/8/23	7/13/23	11/1/23	
7	4/6/22	9/21/22	12/19/22	2/28/23	6/15/23	6/15/23	10/31/23	10/31/23
8	4/1/22	9/22/22	12/15/22	1/17/23	6/6/23	7/13/23	10/23/23	
9	3/21/22	9/22/22	12/12/22	1/17/23	6/6/23	7/13/23	10/11/23	
10	3/13/22	9/21/22	12/8/22	1/17/23	6/7/23	7/13/23	10/11/23	
11	3/9/22	9/22/22	12/8/22	1/17/23	6/9/23	7/13/23	10/11/23	
12	3/7/22	9/23/22	12/5/22	1/17/23	6/6/23	7/13/23	10/11/23	
13	3/3/22	9/24/22	12/5/22	1/17/23	6/7/23	7/13/23	10/11/23	
14	3/1/22	9/25/22	12/5/22	1/17/23	6/2/23	7/13/23	12/5/23	

The planned contract work was discontinued on or about December 11, 2023 in conjunction with the bridge closure and the DB team's efforts were shifted to evaluation and retrofit design to supplement the failed tie-down rods.

DISCUSSION

This evaluation has primarily focused on details and events related to two specific sets of components:

- the post-tensioned tie-down rods, fracture of which was impetus for closure of the bridge to traffic, and
- the post-tensioned concrete cantilever beams, and more explicitly the post-tensioning tendons and corbel drop-in beam supports.

Tie-Down Rods

The I-195 Washington Bridge (700) is a unique structure. The use of post-tensioned elements to provide counter-balance to gravity loads on a cantilever element, although not entirely without precedent, is not a common design configuration for highway bridges. This method of framing could be argued to be complex in its configuration.

Inspections

The bridge was designed and built in an era before the NBIS; the U.S. Congress originally required the Secretary of Transportation to establish these standards in 1968 and the original NBIS was published in 1971.[21] Element-level inspection evolved during the lifespan of the structure, with CoRe elements implemented in the mid-2000s and MBEI adopted circa 2014. No standard or NBE element exists in the MBEI (or previously under CoRe elements) to represent the tie-down rods and thereby promote direct scrutiny of those elements.

WJE's review did not reveal existence of an "Owner's Manual", or operations and maintenance (O&M) manual, to call attention to complex elements (e.g., tie-down rods) or to specify associated inspection and maintenance procedures. In modern practice, the designer of a complex structure, upon construction and commissioning, would be required by many jurisdictions to provide an O&M manual to guide methods and frequency of recommended actions and highlight unusual or critical components.

In accordance with NBIS requirements, the FHWA Bridge Inspector's Reference Manual (BIRM) defines a fracture critical member (FCM) as "a steel member in tension or with a tension element, whose failure probably causes a portion of or the entire bridge to collapse". Bridges that contain fracture critical members are considered fracture critical bridges."[22] "Two criteria exist for a bridge member to be classified as fracture critical. The first criterion deals with the forces in the member, such that members that are in tension or members that have fibers or elements that are in tension meet the first criterion. The second criterion is that its failure causes a total or partial collapse of the structure. Therefore, recognition and identification of a bridge's degree of redundancy is crucial. Redundancy is defined as a structural condition where there are more elements of support than are necessary for stability. There are three basic types of redundancy to consider in bridge design:

- Load path redundancy - designs that have three or more main load-carrying members or load paths between supports
- Structural redundancy - designs which provide continuity of load path from span to span
- Internal redundancy - a bridge member contains three or more elements that are mechanically fastened together so that multiple independent load paths are formed

According to the NBIS, fracture critical members (FCMs) are to be inspected at regular intervals not to exceed 24 months. FCM criteria have prevailed since the mid-1990s and are being gradually superseded. Federal regulations now define a Nonredundant Steel Tension Member (NSTM) as a primary steel member fully or partially in tension, and without load path redundancy, system redundancy, or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse.[23] FCM criteria are being

[21] National Bridge Inspection Standards - Bridge Inspection - Safety Inspection - Bridges & Structures - Federal Highway Administration, *https://www.fhwa.dot.gov/bridge/nbis.cfm*, accessed April 1, 2024.

[22] Bridge Inspector's Reference Manual (BIRM), FHWA-NHI-16-013, Federal Highway Administration U.S. Department of Transportation, Washington, DC, November 2015.

[23] 23 CFR 650.305 Subpart C—National Bridge Inspection Standards, Definitions.

replaced by criteria for NSTMs under the Specifications for the National Bridge Inventory, but applicability remains similar.[24]

Per RIDOT's Bridge Inspection Manual, and in accordance with BIRM, "prior to the bridge inspection, the team leader is responsible for planning and preparing for the inspection, which includes reviewing the bridge structure file and evaluating any bridge site conditions (such as confined spaces, nondestructive evaluation and traffic control). While performing the field inspection, the team leader is responsible for all judgments made concerning a bridge's condition, including recognizing and reporting any critical findings, as well as maintaining safe inspection practices throughout the entire bridge inspection. Upon completion of the bridge inspection, the team leader finalizes the bridge inspection report and submits all required information within the specified timeframe.[25]

The statewide program manager is assigned the duties and responsibilities for bridge inspection, reporting and inventory. These duties and responsibilities may then be delegated by the statewide program manager to project managers (Consultants) and team leaders within the State. Although the statewide program manager may choose to delegate some or all functions to other bridge inspection personnel, the statewide program manager retains all responsibility for bridge inspection operations for which he or she was assigned.

Though the Washington Bridge 700 cantilever beam configuration is arguably complex, the tie-downs do not technically represent fracture critical members (FCM), or non-redundant steel tension members (NSTMs). The tie-down rods do not represent internal redundancy, since there are only two at each connection, and the bridge does not demonstrate structural redundancy since the cantilever to drop-in beam connections represent simple (i.e., not continuous) girder configuration. However, there is load path redundancy (six cantilevers per span with two tie-down rods per cantilever). As such, the tie-down rods were not called out for annual hands-on inspections normally associated with FCMs and NSTMs.

The AASHTO Manual for Bridge Evaluation (MBE) indicates, "The areas of the structure to be closely monitored are those determined by previous inspections and/or load rating calculations that are critical to the bridge's load-carrying capacity. If, during a routine inspection, there is an area of the structure that requires a closer, more detailed inspection to determine its impact on safety or load-carrying capacity (e.g., a crack in a steel member), then perform and document an additional in-depth inspection of that area." This requirement is not restricted to FCMs and would be applicable to deterioration of the tie-down rods.

All of the tie-down rods at Abutment 1 and all but the four external fascia tie-downs at Piers 6 and 7 (two each at beam lines A and F) were encapsulated in concrete diaphragms and therefore not visible to inspectors; only the tie-down rods at the corners of Piers 6 and 7 could be observed without destructive probing. The unbalanced ends of Span 7 cantilevers where these tie-down rods existed were located inside the enclosed areas of Piers 6 and 7 such that they could not be accessed by under-bridge inspection units. Access was limited to that enabled by the existing catwalks along the interior pier walls,

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[24] Specifications for the National Bridge Inventory, FHWA-HIF-22-017, Federal Highway Administration U.S. Department of Transportation, Washington, DC, March 2022.

[25] RIDOT Bridge Inspection Manual, Rhode Island Department of Transportation, 2013.

unless extraordinary measures were undertaken. The elevation of catwalks was such that the base of tie rods at support corbels was above the inspector's eye level. The two tie-down rods along the north face of beam line A at Piers 6 and 7 could be observed from the access hatch and ladder used to reach the catwalk inside the pier enclosures. However, without conscious effort to focus on the rods while descending the ladder, they could be easily overlooked.

As a result of the factors outlined above, there appears to have been a general failure to recognize the significance of the tie-down rods. In addition to RIDOT, six different nationally recognized engineering firms alternately performed one or more routine or special safety inspections of the bridge from 2001 to 2023. None of the inspection reports seemed to consider the PT tie-down rods as being any different in nature than the mild reinforcing steel in the adjacent concrete cantilever beam ends and the reinforced concrete diaphragms and wall corbels.

Though considerable attention was given to deterioration of the corbels and loss of bearing under the cantilevers in the inspection reports and subsequent rehabilitation designs, little focus was given to the high-strength tie-down rods. Since the tie-downs were encapsulated within the diaphragms, an inspector not alerted to the post-tensioned tie-down configuration would likely mistake the cantilever beam end support condition as a normal pad bearing, which functions in compression (supporting downward gravity loads), not in tension (resisting uplift of an unbalanced cantilever). To recognize this condition would require careful consideration of the overall structural configuration and thorough review of the design details.

During routine and special inspections, many photos were taken of the external tie-down rods, though these appear to be without exception incidental to documentation of deteriorating conditions of the adjacent reinforced concrete or structural steel elements. In those photos, the progressive corrosion and loss of section, particularly near the upper and lower concrete interfaces, can be captured in a few. For many others, only a small portion of the center section of the bars can be seen, partially obscured by other elements in the foreground. Figures 26 through 36 present photos from inspection reports from 2015 through 2023 that contain the subject exterior tie-down rods in Span 7. The degree of corrosion and section loss are clearly visible in most of these photos. However, note that annotations provided by inspectors do not clearly identify these as post-tensioned high-strength elements. Specifically, annotations in Figures 29 and 30 point directly to the tie-down rods but describe spalls and exposed rebar with section loss.

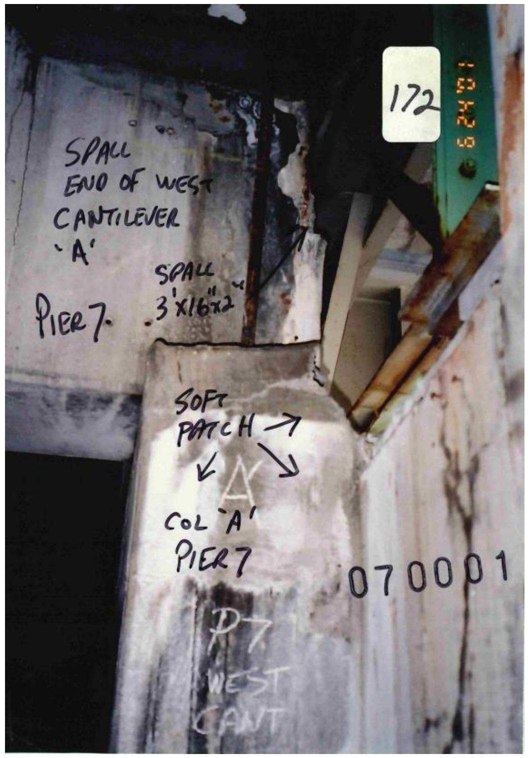

Figure 26. Inspection photo of exterior tie-down rod for cantilever beam A at Pier 7 taken June 2001 (RIDOT).

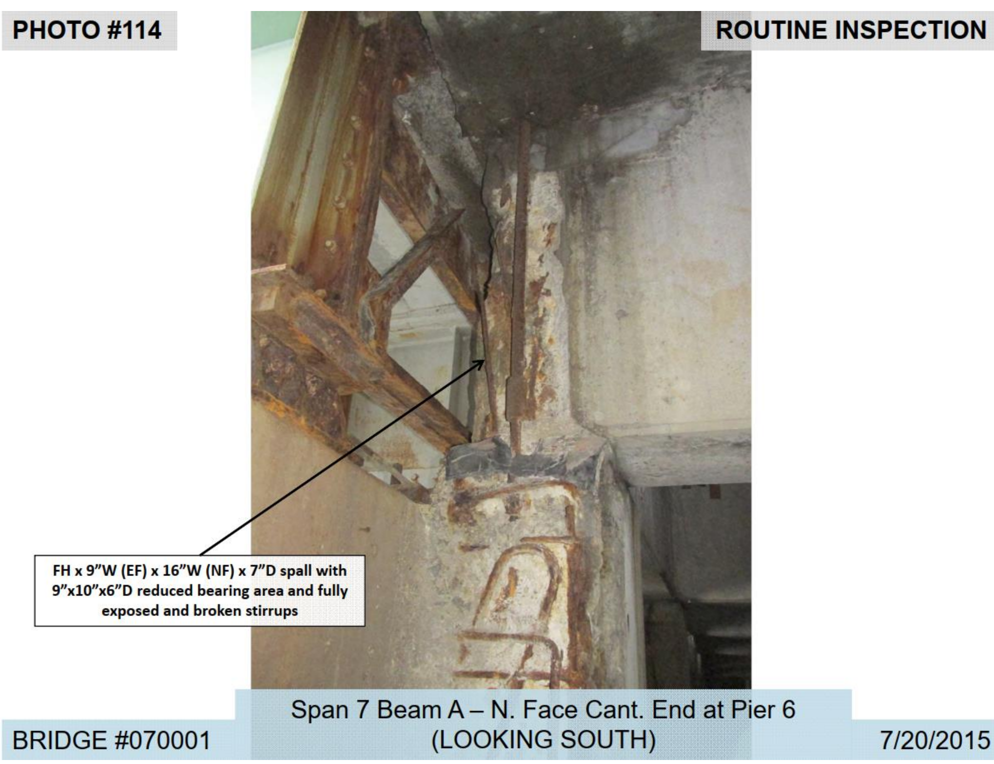

Figure 27. Inspection photo of exterior tie-down rod for cantilever beam A at Pier 6 taken July 2015 (AECOM).

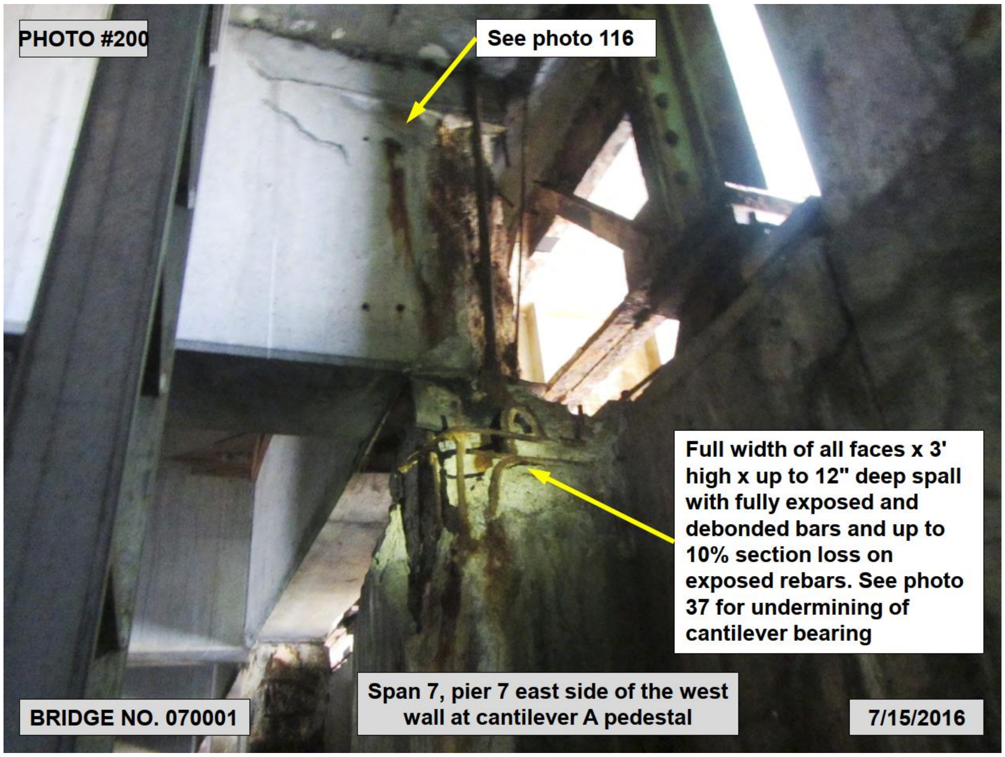

Figure 28. Inspection photo of exterior tie-down rod for cantilever beam A at Pier 7 taken July 2016 (TranSystems).

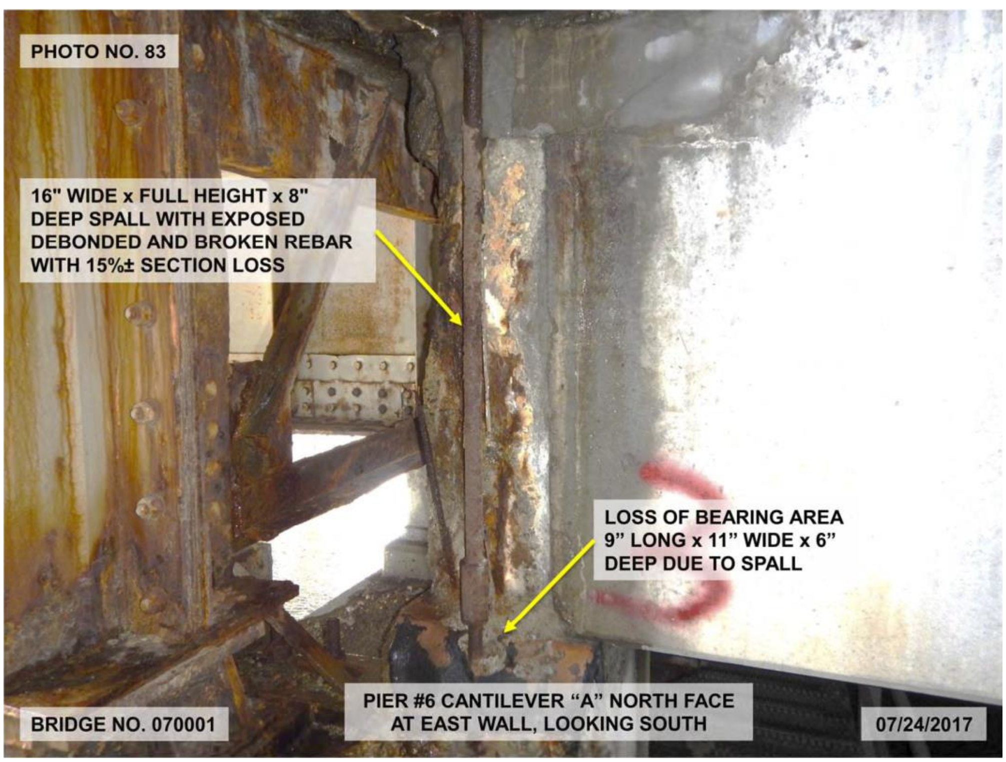

Figure 29. Inspection photo of exterior tie-down rod for cantilever beam A at Pier 6 taken July 2017 (Collins).

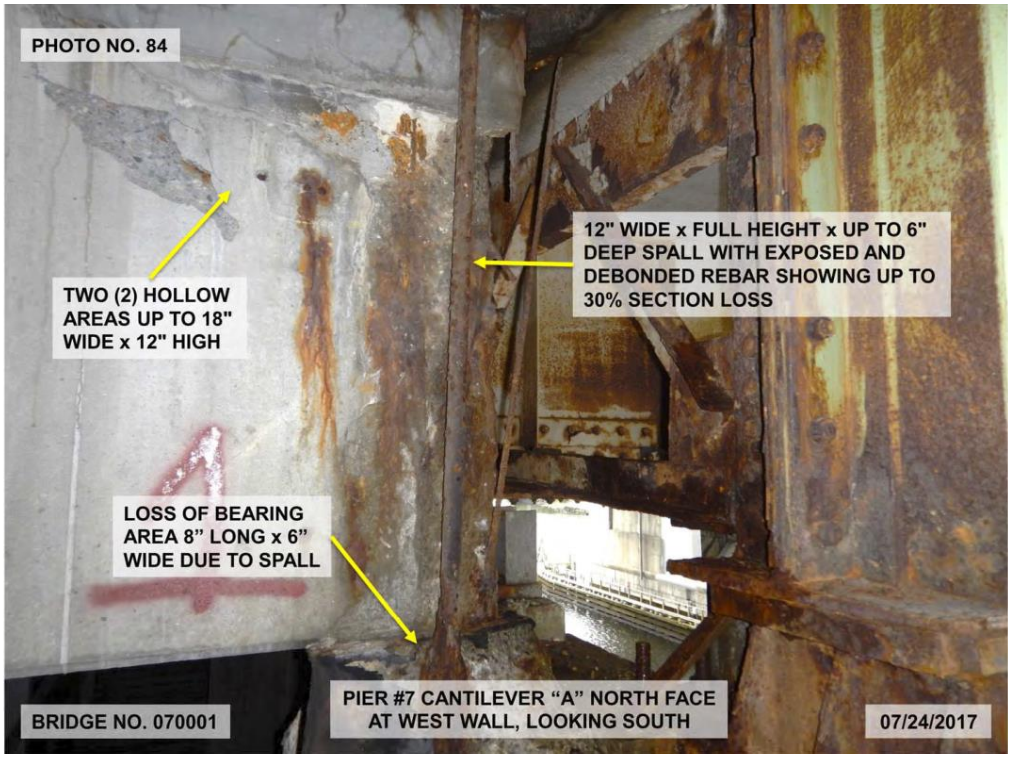

Figure 30. Inspection photo of exterior tie-down rod for cantilever beam A at Pier 7 taken July 2017 (Collins).

Figure 31. Inspection photo of exterior tie-down rod for cantilever beam A at Pier 7 taken July 2017 (Collins).

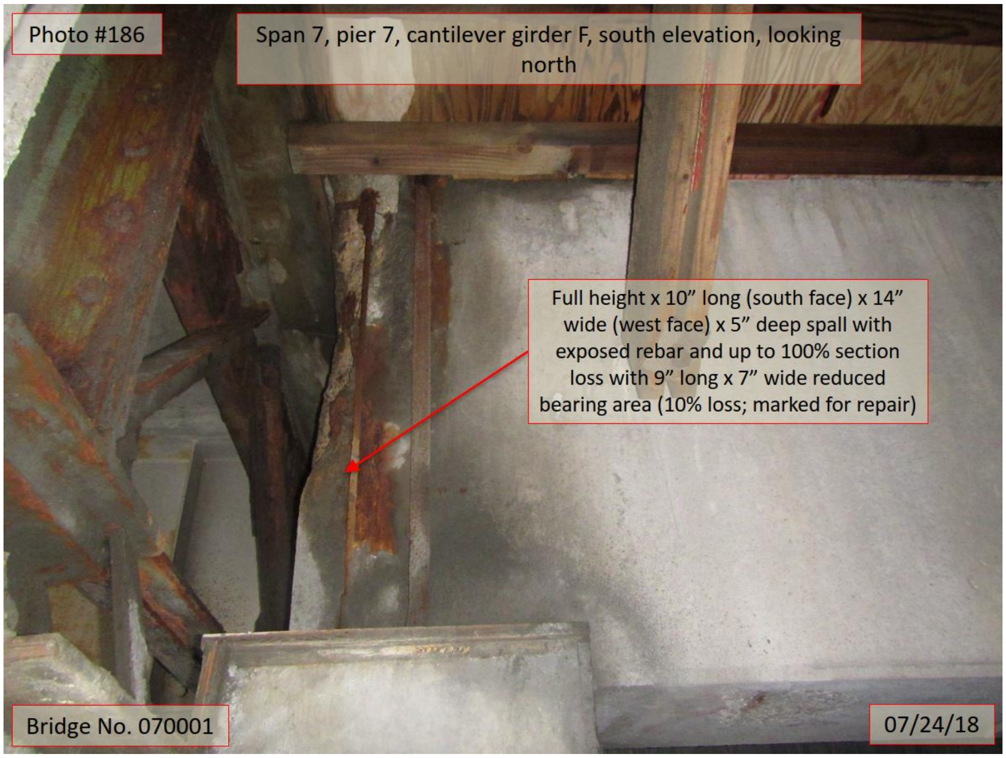

Figure 32. Inspection photo of exterior tie-down rod for cantilever beam F at Pier 7 taken July 2018 (Michael Baker).

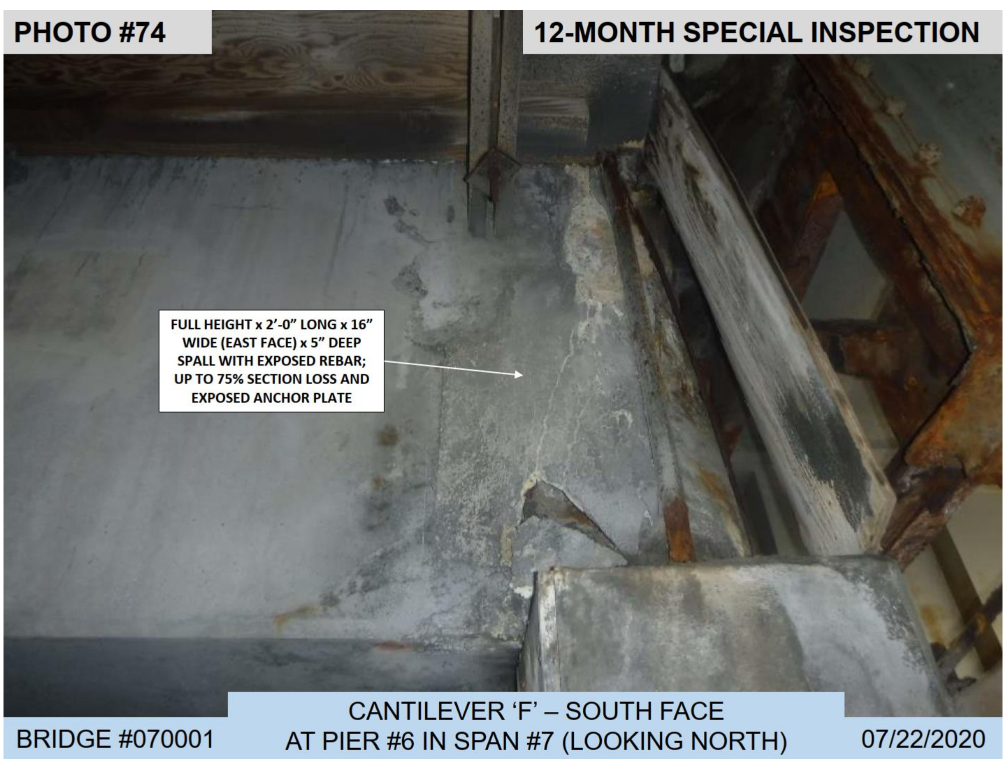

Figure 33. Inspection photo of exterior tie-down rod for cantilever beam F at Pier 6 taken July 2020 (AECOM).

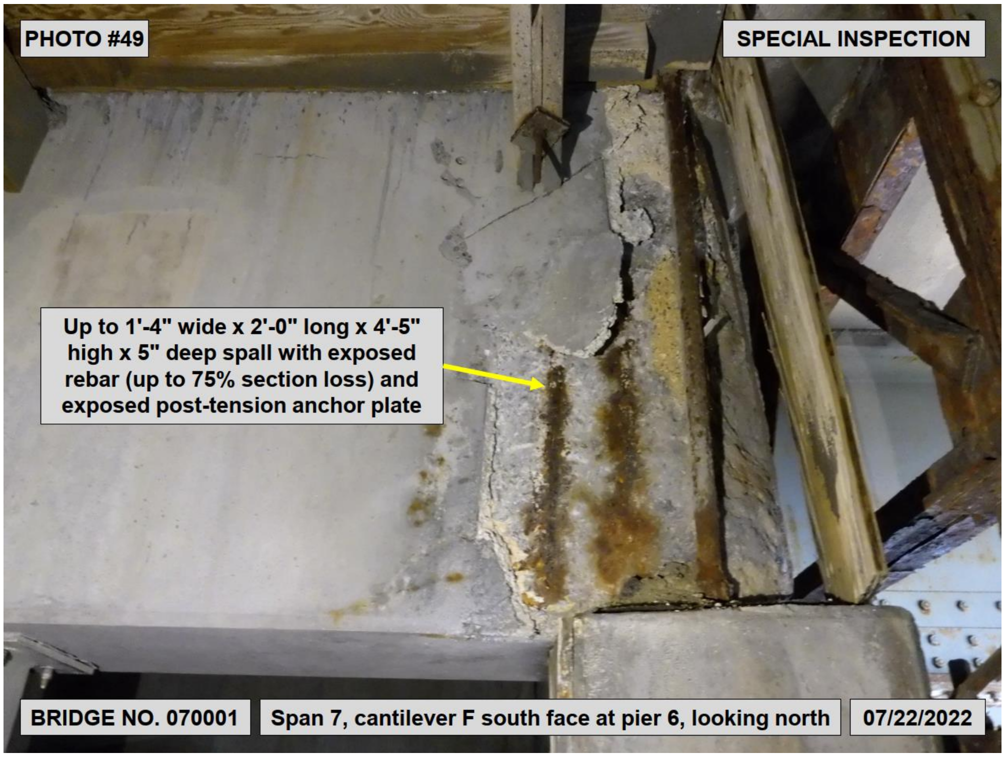

Figure 34. Inspection photo of exterior tie-down rod for cantilever beam F at Pier 6 taken July 2022 (TranSystems).

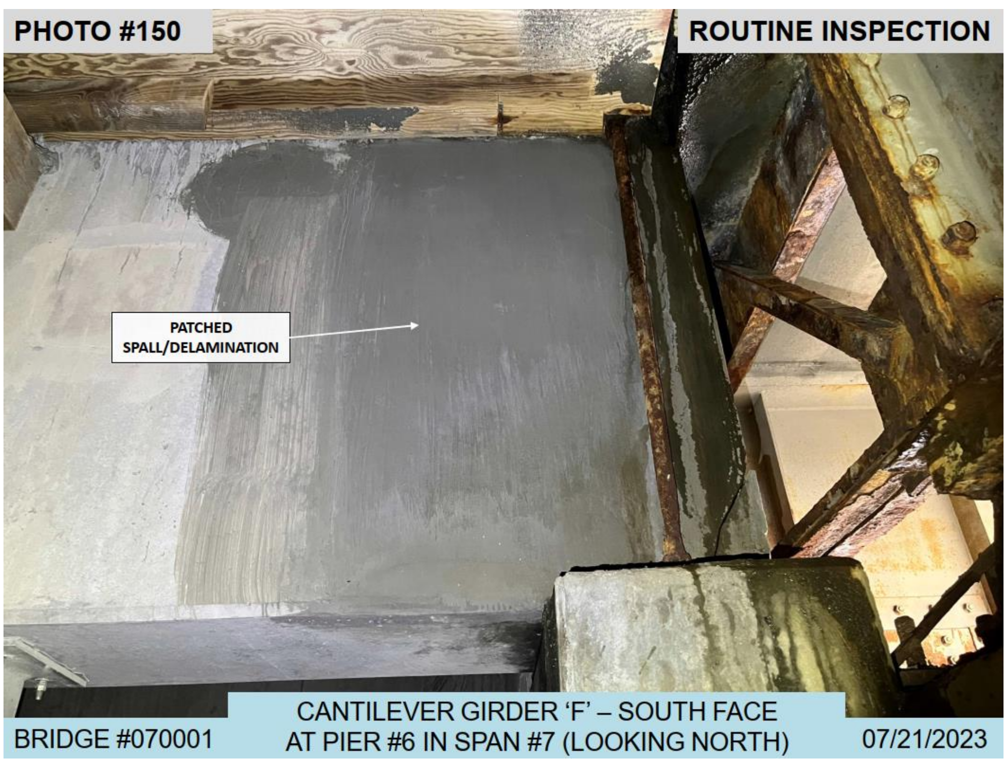

Figure 35. Inspection photo of exterior tie-down rod for cantilever beam F at Pier 6 taken July 2023 (AECOM).

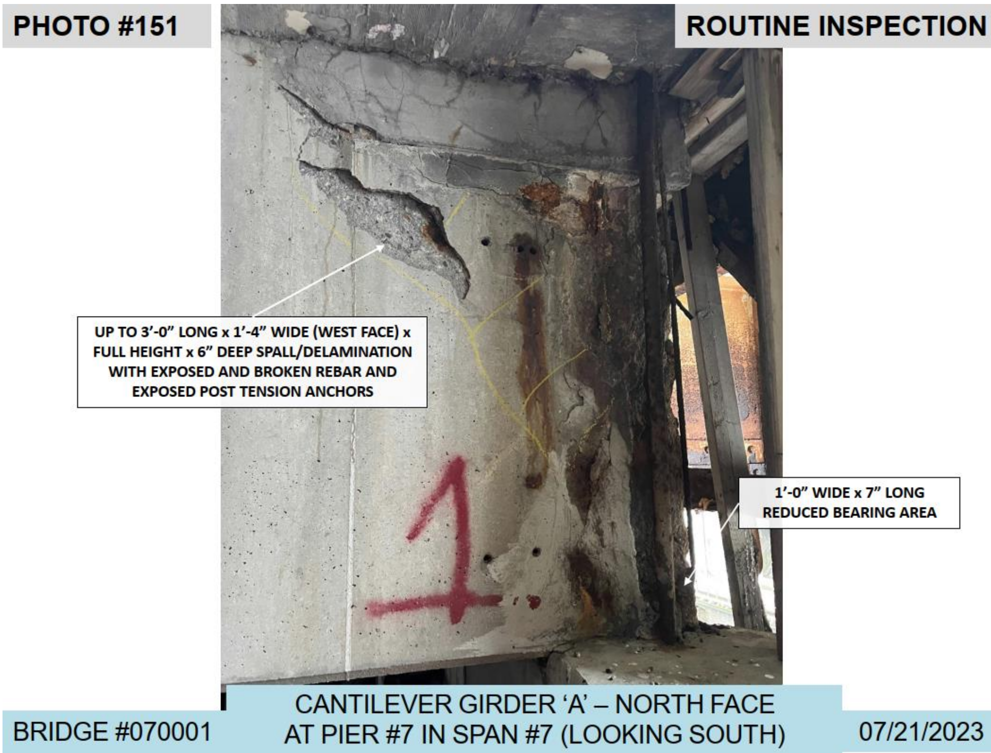

Figure 36. Inspection photo of exterior tie-down rod for cantilever beam A at Pier 7 taken July 2023 (AECOM).

Rehabilitation

For the 2016 Rehabilitation design, AECOM performed detailed girder line analysis and load rating to assess the implications of proposed joint elimination on the cantilevers themselves and moment induced in the pier stems. They recommended a scenario to eliminate 23 of 35 joints, which would have resulted in increasing movement at the Pier 6 joint by 2× and the movement at the Pier 7 joint by more than 5×. The AECOM report stated that thermal loading was considered. Their report tabulated increases in joint displacement in joints to remain at Span 7 after elimination of 3 joints to the west and 5 joints to the east. Although these increased movements were reported, no consideration seems to have been given to the moment and shear being introduced into the tie-down rods at the Pier 6 and 7 unbalanced cantilever supports.

Ultimately, the AECOM joint elimination strategy was never implemented due to complications with the construction project and its cancellation. The concept was picked up by VHB under the "Phase 2" DB project issued in 2021. As noted, VHB's design did not adopt AECOM Scenario 2, which would have eliminated multiple joints in sequence along the bridge, instead opting to eliminate one of the two corbel joints in each cantilever/drop-in span. Thus, cumulative increase in thermal movement at joints at Pier 6 and Pier 7 were less than planned in the prior project. Nonetheless, VHB calculations do not reflect direct consideration of changes in thermal movement at the Pier 6 and Pier 7 joints on the cantilever tie-down rod detail. The calculation summary for the Abutment 1 connection states the girder end is free to rotate in all directions and free to translate in the Y (longitudinal) direction and only fixed in the X and Z (transverse and vertical) directions, which does not appear to be correct given the restraint of the tie-down rods.

The result of continuity over the adjacent drop-in joint would be increased axial strain in the deck due to cumulative thermal movement and potential added girder rotation at the beam connection to the pier wall. As temperatures decrease, a higher net axial tensile force would be applied at the deck level at the joint. The external tie-down rods would then experience an increase in bending moment, which is not advisable for this material. Though WJE has not performed structural modeling to confirm, it is believed that the increased cumulative contraction at the deck level induced by the link slab coupled with restraint of the cantilever at the pier pedestal would also increase uplift at the pier wall support, thereby increasing tensile load on the tie-down rods. Specifically, considering the low toughness of the high-strength steel found by WJE's forensic testing, increased tensile loads on rods with stress concentrations at segments with reduced diameter due to heavy local corrosion and pitting combined with low temperatures and dynamic vehicular loading, could cause a very small initial surface defect to lead to bar fracture.[26]

WJE's forensic report concluded that the PT tie-down rods fractured due to tensile overstress which was at least partially attributable to loss of section caused by environmentally induced corrosion, where rods had lost 48% and 57% of cross-sectional area. The forensic evaluation did not identify fatigue-related striations on the fracture surfaces, though corrosion may have obscured some of the details. Increased stresses on the tie-down rods in tension, bending or shear, would only have served to exacerbate this condition.

The observed fractures occurred in the external tie-down rods. It has not yet been confirmed whether any of the encapsulated tie-down rods have also fractured, but emergency inspections in December 2023 indicated movement ("bouncing") between the seat and diaphragm that suggest fracture is likely. According to the original design, the cantilever beams are seated on a $^3/_8$-inch fabric pad with diaphragms cast to either side and transverse reinforcement continuing through the cantilever end (Figure 6). Thus tie-down rods encapsulated within the diaphragms would experience a concentrated shear force at the point where the rods transition from the diaphragm into the beam seat/corbel.

The two fractured rods were discovered during December 2023. The sequence of link slab closures is illustrated in the timeline of Figure 37. Specifically, closures for Span 6 (adjacent to Pier 6) occurred in September 2022, February 2023, and July 2023. Likewise, Span 7 closures (adjacent to Pier 7), occurred in September 2022, February 2023, and June 2023. The exact time of the fractures cannot be determined, but they were not observed during the July 2023 routine inspection. WJE's forensic evaluation noted that the thickness of corrosion product on the fractured surfaces suggests the fractures had not occurred very close to the time of discovery. However, it is likely fracture occurred within the timeframe that the link slabs were placed.

[26] Forensic Investigation of Failed Post-Tensioned Tie-Down Rods, I-195 SB Washington Bridge North (700), Wiss, Janney, Elstner Associates, Inc., February 19, 2024, *RIDOT Washington Bridge PT Rod Failure Memo 2024-02-19.pdf*

2022																																												
	Feb	Mar				Apr				May					Jun				Jul					Aug				Sep				Oct					Nov				Dec			
	27	06	13	20	27	03	10	17	24	01	08	15	22	29	05	12	19	26	03	10	17	24	31	07	14	21	28	04	11	18	25	02	09	16	23	30	06	13	20	27	04	11	18	25
	Stage 1																																				Stage 3							
SPAN LINK POURED																														7	4	1												
																														10	5	2												
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	01	08	15	22	29	05	12	19	26	05	12	19	26	02	09	16	23	30	07	14	21	28	04	11	18	25	02	09	16	23	30	06	13	20	27	03	10	17	24	01	08	15	22	29
	Stage 3									Stage 2																						Stage 4												
SPAN LINK POURED			8						1															7				1												5				
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Figure 37. Link slab placement timeline

Post-tensioned Concrete Cantilever Beams

The following summarizes the evolution of condition of the post-tensioned concrete cantilever beams.

Inspections

The 1992 evaluation by Lichtenstein investigated cracking in the webs of the cantilever beams that followed the post-tensioning tendon profiles. Their investigation concluded that cracking along tendon profiles likely occurred at original construction and were not growing. Radiography at two anchorages indicated voids might be present behind the anchor plates. Lichtenstein suggested NDE stress methods or ground penetrating radar might be used to further investigate, but no probing of the tendons to determine if voids exist occurred or was recommended at the time. Grouts from the era this bridge was constructed were primarily simple cement grouts, with little or no admixture for volume stabilization. Grouts of this era were known to bleed and shrink. Even a grout of this type with appropriate water content has been shown to experience approximately 4% shrinkage, resulting in voids within grouted tendons. If water-to-cement proportions were not carefully controlled, additional water would lead to greater bleeding and voids, or potential segregation of the grout, wherein sulfate components of the cement float to the top and create a porous, punky, low-pH layer that is conducive to corrosion.

Although Lichtenstein anticipated no crack growth, future inspections reported growing length of cracks along tendons in the cantilever beams, even after regrouting in 1998.

Rehabilitation

During the 1996-1998 rehabilitation contract, the project team encountered significantly greater deterioration in cantilevers, corbels and drop-in beam ends than previously thought, including voids in tendons and delaminations in the webs of cantilevers. Special provisions were issued to perform nondestructive evaluation using impact echo and indications were explored by creating 1 square foot openings. Voids detected by impact echo and confirmed by drilling were retrofit grouted. Over 125 locations were opened following these procedures. Exact locations and quantities of retrofit grout were not shown in available documents, but it is assumed they occurred through Spans 1 through 14. Note that the top "H" tendon of the cantilevers was not surveyed by impact echo and thus presumably did not receive retrofit grout.

Though details available to WJE are sparse, the 2016 rehabilitation seemed to have been plagued by poor coordination and very little emphasis on progressing the work. Although repairs were heavily concentrated at the east end of the bridge, the distribution and progress of patch repairs seemed very haphazard. Inspection reports document accumulation of construction debris and eventually bird feces on the scaffolding and falsework that remained in place for years. There were instances where back-to-back annual inspections showed very little change in element condition states that would reflect the benefit of ongoing repairs. These conditions are corroborated by some inspection photos that show the same incomplete repair conditions over a period of years. Inspection photos showed persistent incomplete patch repairs with poor preparation and opened areas remaining exposed for extended periods. "Prepared" patches contained corroded reinforcement and severed prestressing strands that were not cleaned or primed, nor supplemented with prestressing splices. Many areas were not appropriately squared off or chipped to required depths below reinforcement.

In addition, the 2016 repair plans called for FRP strengthening of cantilever beam corbels and drop-in beam ends; inspection photos do not reflect that any significant FRP wraps were ever installed under either rehabilitation project.

The crack growth along the faces of the PT tendons in cantilever beams did not appear to have been specifically investigated in preparation for the 2016 and 2021 rehabilitation projects. While FRP was to be applied to strengthen the corbels, no further probing of the tendons appears to have been conducted to evaluate potential causes for the continued propagation of cracks.

Overall, the bridge superstructure general condition rating remained 4 – Poor for more than a decade while known defects continued to grow and multiply. Many maintenance recommendations were repeated over multiple inspection cycles without being addressed. The failure of the 2016-2019 rehabilitation project resulted in extended exposure and greater deterioration. However, had the original link slab design been implemented, the failure of the tie-down rod connections may have been accelerated.

CONCLUSIONS

What events and conditions led to the failure of the tie-down rods?

The fracture of the PT tie-down rods appears to have resulted from a combination of the following:

- Severe reduction of local cross-sectional area due to advanced corrosion.

- Lack of recognition by inspectors of the importance of the tie-down rods to the structural stability of the bridge.
- Poor maintenance of the structure (e.g., leaking joints, corroding reinforcement, and cracked and spalled concrete) over an extended period of decades.
- A lack of toughness of the original high-strength rod materials, which were based on the standards applicable at the time of construction. Though technically not out of specification, they do present greater risk than those that comply with modern standards.
- Possibly to a lesser extent, failure during rehabilitation design to properly characterize boundary conditions associated with the presence of the tie-down rods, and their effect on expansion and contraction of the bridge between expansion joints. Calculations were found for influence at Abutment 1, but not for Piers 6 and 7.

Was the agency's decision to close the bridge to traffic reasonable?

It is our opinion that Rhode Island Department of Transportation was entirely justified in closing the bridge as a precaution to protect both the traveling public and workers at the bridge.

Should conditions leading to the failure have been foreseen?

Many of the issues highlighted in this report show that program managers, bridge inspectors, and designers should have and could have been aware of problems that were developing. Greater attention needed to be paid to the importance of the post-tensioning systems in overall structural performance and stability of the bridge.

Should conditions discovered after the failure that revealed more serious condition of cantilever beams have been foreseen?

The deteriorating condition of the cantilever beams and corbels was clear. Given experience within the industry with poorly grouted post-tensioning systems on other bridges, more attention should have been given to signs of continuing cracking in the beams along the tendons, exposure of post-tensioning anchorages, and advanced deterioration of cantilever beam ends. Though the 1996-98 rehabilitation undertook retrofit grouting of the tendons, it is not clear that all voids had been treated or that the treatment was 100 percent effective.

What actions or policy recommendations are recommended to prevent similar types of events from occurring in the future?

The entity responsible for safety inspections within RIDOT should conduct a review of structures that could be considered complex to identify critical elements or details, to include but not limited to NSTMs. For critical portions of a structure that do not have a specific element defined in the bridge management system, consider establishing specific agency defined elements (ADEs) to ensure that the elements are properly inspected/addressed with each applicable inspection.

In addition, RIDOT should review its processes for prioritizing and following up on work recommendations provided in inspection reports.

CLOSING

WJE's evaluation is based on review of the documents and references cited herein as provided by Rhode Island Department of Transportation. Should additional information become available, WJE reserves the right to amend its assessment based on the new information.

APPENDIX A. INSPECTION REPORT FILES REVIEWED

Table A-5. Routine, underwater, and special inspection files received for review

Inspection2001-06-26
2001-06-26 Inspection Report - Inspection Comments.pdf
2001-06-26 Inspection Report Photos 1-180.pdf
2001-06-26 Inspection Report.pdf
Inspection2003-08-26
2003-08-26 Inspection Report - Sub-Aqueous Inspection.pdf
SUBAQUEOUS 2003 070001 Washington Bridge N.pdf
Inspection2007-07-01 (UNMX)
Changes.doc
Clearance Dropline 01-9.pdf
Field Notes 01-51.pdf
field notes.doc
Photos 01-67.pdf
title.doc
Inspection2009-06-29 (MDVX)\Subaqueous
Bridge No 070001 - Sub Aqueous Inspection Report.pdf
Inspection2009-08-07 (UUIQ)
700Y09.PDI
8-7-09 SIA Inspection Report.pdf
Changes to PONTIS SI A.pdf
Element 105 RConcBoxGirder.doc
Element 109 (cantilever).xls
Element 109 (simply supported).xls
Element 144 ReinfConcArch.doc
Field Notes.pdf
Hydraulic_clearance 1.pdf
Hydraulic_clearance 2.pdf
Hydraulic_clearance 3.pdf
Photo_1-137.pdf
Pontis Notes Tab 'Inspection Notes'.doc
Vertical_clearance 1.pdf
Vertical_clearance 2.pdf
Vertical_clearance 3.pdf
Vertical_clearance 4.pdf
Inspection2011-08-03 (YGVV)
070001 BI005 Bridge Load Rating Posting Form 08-04-11.pdf
070001 Changes in Pontis.pdf
070001 RIDOT Hydraulic - Electronic (1 of 3).pdf
070001 RIDOT Hydraulic - Electronic (2 of 3).pdf

070001 RIDOT Hydraulic - Electronic (3 of 3).pdf
2011Pontis.pdf
Element 105 RConc Box Girder Notes.pdf
Element 109 Prestressed Concrete Open Girder Notes.pdf
Element 144 RConc Arch Notes.pdf
Element 210 RConc Pier Wall Notes.pdf
Element 310 Elastomeric Bearing Notes.pdf
Field Notes.pdf
Gano Street Vertical Clearance.pdf
Photo_1-159.pdf
Pontis Notes Tab 'Bridge Notes'.pdf
Time Log.pdf
Valley Street Vertical Clearance (1 of 2).pdf
Valley Street Vertical Clearance (2 of 2).pdf
Water Street Vertical Clearance.pdf

Inspection2013-08-02 (MBPU)

070001_RIDOT Hydraulic 1 of 2.pdf
070001_RIDOT Hydraulic 2 of 2.pdf
700Y13.PDI
700_Element 105 RConc Box Girder Notes.pdf
700_Element 109 Prestressed Concrete Open Girder Notes.pdf
700_Element 144 RConc Arch Notes.pdf
700_Element 210 RConc Pier Wall Notes.pdf
700_Element 310 Elastomeric Bearing Notes.pdf
700_Field Notes.pdf
700_Pontis Changes.doc
700_Pontis Notes Tab 'Bridge Notes'.pdf
700_Title.doc
BI006_Bridge Critical Findings.pdf
Br 070001 Superstructure General Notes.docx
Br 070001 Span 12 Superstructure Deficiencies.docx
Br 070001 Span 13 Superstructure Deficiencies.docx
Br 070001 Span 17 Superstructure Deficiencies.docx
Br 070001 Span 5 Superstructure Deficiencies.docx
Br 070001 Span 8 Superstructure Deficiencies.docx
Br 070001 Span 9 Superstructure Deficiencies.docx
Critical Findings Photos.pdf
Photo_01-172.pdf
Report_08_02_13.pdf
VerticalClearanceTwoLane Gano Street.pdf
VerticalClearanceTwoLane Valley Street 1 of 2.pdf
VerticalClearanceTwoLane Valley Street 2 of 2.pdf
VerticalClearanceTwoLane Water Street.pdf
VerticalClearanceTwoLane Waterfront Drive.pdf

Inspection2013-08-07 (XLNE)\Subaqueous

070001_2013.dwg
700Y13.PDI
BI 008 Bridge 070001 Inspection Submittal Checklist.pdf
RIDOT BR070001_UW-08-07-13.pdf

Inspection2015-07-28 (ETXM)

070001 Element Write-Ups and Tables.pdf
070001 Photos_1-203.pdf
070001 Pontis Changes.doc
070001 Vertical Clearance Form - Span 1.pdf
070001 Vertical Clearance Form - Span 15.pdf
070001 Vertical Clearance Form - Span 16.pdf
070001 Vertical Clearance Form - Span 18.pdf
Element 12 and 321 - Reinforced Concrete Deck and Approach Slab.pdf
Element 105 - Reinforced Concrete Closed Box Girder Defects Table.pdf
Element 105 - Reinforced Concrete Closed Box Girder.pdf
Element 107 and 8370 - Steel Open Girder and Diaphragms.pdf
Element 109 - Drop-In I-Girders (Spans 1-6 and 8-14) Defects Table.pdf
Element 109 - Drop-In Post Tensioned Concrete Corbels (Spans 1-6 and 8-14) Defec
Element 109 - Post Tensioned Cantilever I-Girders (Spans 1-6 and 8-14) Defects T
Element 109 - Prestressed Concrete Open Girder.pdf
Element 109 - PS Concrete I-Girders (Spans 15-18) Defects Table.pdf
Element 144 - Reinforced Concrete Arch - Fascia Arches (Spans 1-6 8-13 and 1R-3R
Element 144 - Reinforced Concrete Arch.pdf
Element 205 210 and 234 - Reinforced Concrete Pier Column Wall and Cap.pdf
Element 215 8213 and 8218 - Reinf Concrete Abutment Return Wall and Backwall.p
Element 300 301 and 8305 - Deck Joints.pdf
Element 310 311 and 313 - Bearings.pdf
Element 331 8335 8336 and 8398 - Railings and Sidewalks.pdf
Element 8060 and Additional - Scuppers Utilities and Additional Bridge Notes.pdf
Element 8366 and 8367 - Rip Rap and Slope Blocks.pdf
Element 8371 - Concrete Diaphragms Defects Table.pdf
Element 8371 - Concrete Diaphragms.pdf
Inspection Report (Routine) 150728.pdf
Substructure Defects Table.pdf

Inspection2016-07-15 (NTGU)\Special

Br 700 Defects Tables Combined.pdf
Br 700 Element Summary Notes Combined.pdf
Changes to BrM SI A.pdf
Inspection Report Special 160715.pdf
Photo_1-218.pdf

Inspection2017-07-24 (KPWQ)

070001_Element 107_Element 1000_BrM_Notes.pdf
700 Vertical Clearance Form - Span 1.pdf
700 Vertical Clearance Form - Span 15.pdf

700 Vertical Clearance Form - Span 16.pdf
700 Vertical Clearance Form - Span 18.pdf
700_Additional_Inspection_Notes_BrM_Notes.pdf
700_BrM_Element and NBI_Data_Changes.pdf
700_BrM_Element_Notes.pdf
700_Element 105_Element 1080_BrM_Notes.pdf
700_Element 105_Element 1090_BrM_Notes.pdf
700_Element 105_Element 1120_BrM_Notes.pdf
700_Element 105_Element 1130_BrM_Notes.pdf
700_Element 107_Element 1000_BrM_Notes.pdf
700_Element 109_Element 1080_BrM_Notes.pdf
700_Element 109_Element 1090_BrM_Notes.pdf
700_Element 109_Element 1100_BrM_Notes.pdf
700_Element 109_Element 1110_BrM_Notes.pdf
700_Element 109_Element 1120_BrM_Notes.pdf
700_Element 110_Element 1080_BrM_Notes.pdf
700_Element 110_Element 1090_BrM_Notes.pdf
700_Element 110_Element 1130_BrM_Notes.pdf
700_Element 16_Element 1080_BrM_Notes.pdf
700_Element 16_Element 1090_BrM_Notes.pdf
700_Element 16_Element 1120_BrM_Notes.pdf
700_Element 16_Element 1130_BrM_Notes.pdf
700_Element 205_Element 1080_BrM_Notes.pdf
700_Element 205_Element 1090_BrM_Notes.pdf
700_Element 205_Element 1130_BrM_Notes.pdf
700_Element 210_Element 1080_BrM_Notes.pdf
700_Element 210_Element 1090_BrM_Notes.pdf
700_Element 210_Element 1120_BrM_Notes.pdf
700_Element 210_Element 1130_BrM_Notes.pdf
700_Element 215_Element 1080_BrM_Notes.pdf
700_Element 215_Element 1120_BrM_Notes.pdf
700_Element 215_Element 1130_BrM_Notes.pdf
700_Element 234_Element 1080_BrM_Notes.pdf
700_Element 234_Element 1090_BrM_Notes.pdf
700_Element 234_Element 1120_BrM_Notes.pdf
700_Element 234_Element 1130_BrM_Notes.pdf
700_Element 310_Element 2240_BrM_Notes.pdf
700_Element 8371_Element 1080_BrM_Notes.pdf
700_Element 8371_Element 1090_BrM_Notes.pdf
700_Element 8371_Element 1120_BrM_Notes.pdf
700_Element 8371_Element 1130_BrM_Notes.pdf
BI 008 Bridge 700 Inspection Submittal Checklist.pdf
Photos_1-247.pdf

Inspection2017-07-24 (KPWQ)\Subaqueous

070001_2017.dwg
RIDOT Br 070001_UW_2017-07-24.pdf

Inspection2017-10-27 (JTUG)\Special

070001 BrM Changes.doc
070001 Dapped End Special Inspection Photos.pdf
10-27-2017 S Beam Elevations - Underside.pdf
10-27-2017 S Beam Ends-1.pdf
10-27-2017 S Beam Ends-2.pdf
10-27-2017 S Framing Plans Spans 1-14.pdf

Inspection2018-07-24 (YLOH)\Special

070001 BI 008 Bridge Inspection Submittal Checklist.doc
070001 Changes in AASHTOWare BrM.doc
070001 Changes in AASHTOWare BrM.pdf
070001 Cover Letter.pdf
070001 Photo_1-207.pdf
070001 Report Text.pdf
070001 Vertical Clearance Form - Span 1.pdf
070001 Vertical Clearance Form - Span 15.pdf
070001 Vertical Clearance Form - Span 16.pdf
070001 Vertical Clearance Form - Span 18.pdf
Bridge 070001 Additional Notes.pdf
Bridge 070001 Elem 105 Defect 1130 Table.pdf
Bridge 070001 Elem 105 Defect Table.pdf
Bridge 070001 Elem 107 Defect Table.pdf
Bridge 070001 Elem 109 Defect Table.pdf
Bridge 070001 Elem 109 Shear Crack Table.pdf
Bridge 070001 Elem 16 Defect Table.pdf
Bridge 070001 Elem 205 Defect Table.pdf
Bridge 070001 Elem 210 Defect Table.pdf
Bridge 070001 Elem 215 Defect Table.pdf
Bridge 070001 Elem 234 Defect Table.pdf
Bridge 070001 Elem 110 Defect Table.pdf
Bridge 070001 Elem 310 Defect Table.pdf
Bridge 070001 Elem 8371 Defect Table.pdf

Inspection2019-07-24 (UMYT)

070001 Additional Inspection Notes.pdf
070001 BI 008 Bridge Inspection Submittal Checklist.pdf
070001 Bridge Inspection AASHTOWare BrM RIDOT Schedule Webpage Checklist.pdf
070001 BRM Change Sheet.pdf
070001 Elem 105 Defect 1130 Table.pdf
070001 Elem 105 Defect Table.pdf
070001 Elem 105 Underside Sketches.pdf
070001 Elem 107 Defect Table.pdf
070001 Elem 109 Defect Table.pdf
070001 Elem 109 Shear Crack Table.pdf
070001 Elem 110 Defect Table.pdf

070001 Elem 12 Defect Table.pdf
070001 Elem 16 Defect Table.pdf
070001 Elem 205 Defect Table.pdf
070001 Elem 210 Defect Table.pdf
070001 Elem 215 Defect Table.pdf
070001 Elem 234 Defect Table.pdf
070001 Elem 8371 Defect Table.pdf
070001 RIDOT Bridge Inspection Cover Letter.pdf
070001 Span 1 North Elevation.pdf
070001 Vertical Clearance Form - Span 1.pdf
070001 Vertical Clearance Form - Span 15.pdf
070001 Vertical Clearance Form - Span 16.pdf
070001 Vertical Clearance Form - Span 18.pdf
070001_Photos_1-292 reduced.pdf
2019-07-24 (UMYT).pdf

Inspection2020-07-22 (AOUC)\Special

070001 - Gano Box Girders - Water and Pigeons - Email_07-10-2020.pdf
070001 Additional Inspection Notes.pdf
070001 BRM Change Sheet.doc
070001 BRM Change Sheet.pdf
070001 Elem 105 Defect 1130 Table.pdf
070001 Elem 105 Defect Table.pdf
070001 Elem 105 Underside Sketches.pdf
070001 Elem 107 Defect Table.pdf
070001 Elem 109 Defect Table.pdf
070001 Elem 109 Shear Crack Table.pdf
070001 Elem 110 Defect Table.pdf
070001 Elem 16 Defect Table.pdf
070001 Elem 205 Defect Table.pdf
070001 Elem 210 Defect Table.pdf
070001 Elem 215 Defect Table.pdf
070001 Elem 234 Defect Table.pdf
070001 Elem 8371 Defect Table.pdf
070001_Photos_1-149.pdf
2020-07-22 (AOUC).pdf
BI008 Bridge Inspection Submittal Checklist.pdf
Bridge Inspection AASHTOWare BrM RIDOT Schedule Webpage Checklist REV06212019.do
Bridge Inspection AASHTOWare BrM RIDOT Schedule Webpage Checklist.pdf
RIDOT Bridge Inspection Cover Letter.pdf

Inspection2021-07-23 (HMIB)

070001 Additional Inspection Notes.doc
070001 Additional Inspection Notes.pdf
070001 BI-008 Bridge Inspection Submittal Checklist.doc
070001 BI-008 Bridge Inspection Submittal Checklist.pdf
070001 BrM Changes.doc

070001 BrM Changes.pdf
070001 Cover Letter.pdf
070001 Elem 105 Defect 1130 Table.pdf
070001 Elem 105 Defect 1130 Table.xlsx
070001 Elem 105 Defect Table.pdf
070001 Elem 105 Defect Table.xlsx
070001 Elem 105 Underside Sketches.pdf
070001 Elem 107 Defect Table.pdf
070001 Elem 107 Defect Table.xlsx
070001 Elem 109 Defect Table.pdf
070001 Elem 109 Defect Table.xlsx
070001 Elem 109 Shear Crack Table.pdf
070001 Elem 109 Shear Crack Table.xlsx
070001 Elem 110 Defect Table.pdf
070001 Elem 110 Defect Table.xlsx
070001 Elem 12 Defect Table.pdf
070001 Elem 12 Defect Table.xlsx
070001 Elem 16 Defect Table.pdf
070001 Elem 16 Defect Table.xlsx
070001 Elem 205 Defect Table.pdf
070001 Elem 205 Defect Table.xlsx
070001 Elem 210 Defect Table.pdf
070001 Elem 210 Defect Table.xlsx
070001 Elem 215 Defect Table.pdf
070001 Elem 215 Defect Table.xlsx
070001 Elem 234 Defect Table.pdf
070001 Elem 234 Defect Table.xlsx
070001 Elem 8371 Defect Table.pdf
070001 Elem 8371 Defect Table.xlsx
070001 Report.pdf
070001 Schedule Webpage Checklist.doc
070001 Schedule Webpage Checklist.pdf
070001 Vertical Clearance Form - Span 1.pdf
070001 Vertical Clearance Form - Span 15.pdf
070001 Vertical Clearance Form - Span 16.pdf
070001 Vertical Clearance Form - Span 18.pdf
Pages 1-100 from 070001 Photos 1-213 Reduced.pdf
Pages 101-213 from 070001 Photos 1-213 Reduced.pdf

Inspection2021-07-23 (HMIB)\Subaqueous

070001 Underwater Report.pdf

Inspection2022-07-22 (SVFV)

070001 - Photos_1-86.pdf
070001 Additional Inspection Notes.pdf
070001 Cover Letter.pdf
070001 Elem 105 Defect 1130 Interior Table.pdf

070001 Elem 105 Defect Interior Table.pdf
070001 Elem 105 Underside Exterior Sketches.pdf
070001 Elem 107 Defect Table.pdf
070001 Elem 109 Defect Table.pdf
070001 Elem 109 Shear Crack Table.pdf
070001 Elem 110 Defect Table.pdf
070001 Elem 16 Defect Table.pdf
070001 Elem 205 Defect Table.pdf
070001 Elem 210 Defect Table.pdf
070001 Elem 215 Defect Table.pdf
070001 Elem 234 Defect Table.pdf
070001 Elem 8371 Defect Table.pdf
070001 ReportInFrame.pdf
BI 008 Bridge Inspection Submittal Checklist.pdf
BI 011 Special Inspection Requirements.pdf
BrM Notes.docx
Changes to BrM.pdf

Inspection2023-07-21 (SAES)

070001 Additional Inspection Notes.docx
070001 Additional Inspection Notes.pdf
070001 BI008 Bridge Inspection Submittal Checklist.pdf
070001 BrM Changes Sheet.docx
070001 BrM Changes Sheet.pdf
070001 Elem 105 Defect 1130 Interior Table.pdf
070001 Elem 105 Defect 1130 Interior Table.xlsx
070001 Elem 105 Defect Interior Table.pdf
070001 Elem 105 Defect Interior Table.xlsx
070001 Elem 105 Underside Exterior Sketches.pdf
070001 Elem 107 Defect Table.pdf
070001 Elem 107 Defect Table.xlsx
070001 Elem 109 Defect Table.pdf
070001 Elem 109 Defect Table.xlsx
070001 Elem 109 Shear Crack Table.pdf
070001 Elem 109 Shear Crack Table.xlsx
070001 Elem 110 Defect Table.pdf
070001 Elem 110 Defect Table.xlsx
070001 Elem 12 Defect Table.pdf
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070001 Elem 205 Defect Table.pdf
070001 Elem 210 Defect Table.pdf
070001 Elem 210 Defect Table.xlsx
070001 Elem 215 Defect Table.pdf
070001 Elem 215 Defect Table.xlsx
070001 Elem 234 Defect Table.pdf

070001 Elem 234 Defect Table.xlsx
070001 Elem 8371 Defect Table.pdf
070001 Elem 8371 Defect Table.xlsx
070001 RIDOT Bridge Inspection Cover Letter.pdf
070001 RIDOT Schedule Webpage Checklist.pdf
070001 Vertical Clearance Form - Span 1.pdf
070001 Vertical Clearance Form - Span 15.pdf
070001 Vertical Clearance Form - Span 16.pdf
070001 Vertical Clearance Form - Span 18.pdf
070001_Photos_1-300.pdf
2023-07-21 (SAES).pdf

Inspection2023-12-17 (FZFQ)

0001 BrM Changes Sheet.docx
070001 BrM Changes Sheet.docx
070001 BrM Changes Sheet.pdf
070001 Elem 109 Defect Table.pdf
070001 Elem 109 Defect Table.xlsx
070001 Elem 12 Defect Table.pdf
070001 Elem 12 Defect Table.xlsx
070001 Elem 215 Defect Table.pdf
070001 Elem 215 Defect Table.xlsx
070001 Elem 8371 Defect Table.pdf
070001 Elem 8371 Defect Table.xlsx
070001 Photos 1-52.pdf
070001 RIDOT Bridge Inspection Cover Letter.pdf
2023-12-17 (FZFQ).pdf
BI008 Bridge Inspection Submittal Checklist_Fillable PDF.pdf
BrM RIDOT Schedule Webpage Checklist_Fillable PDF.pdf