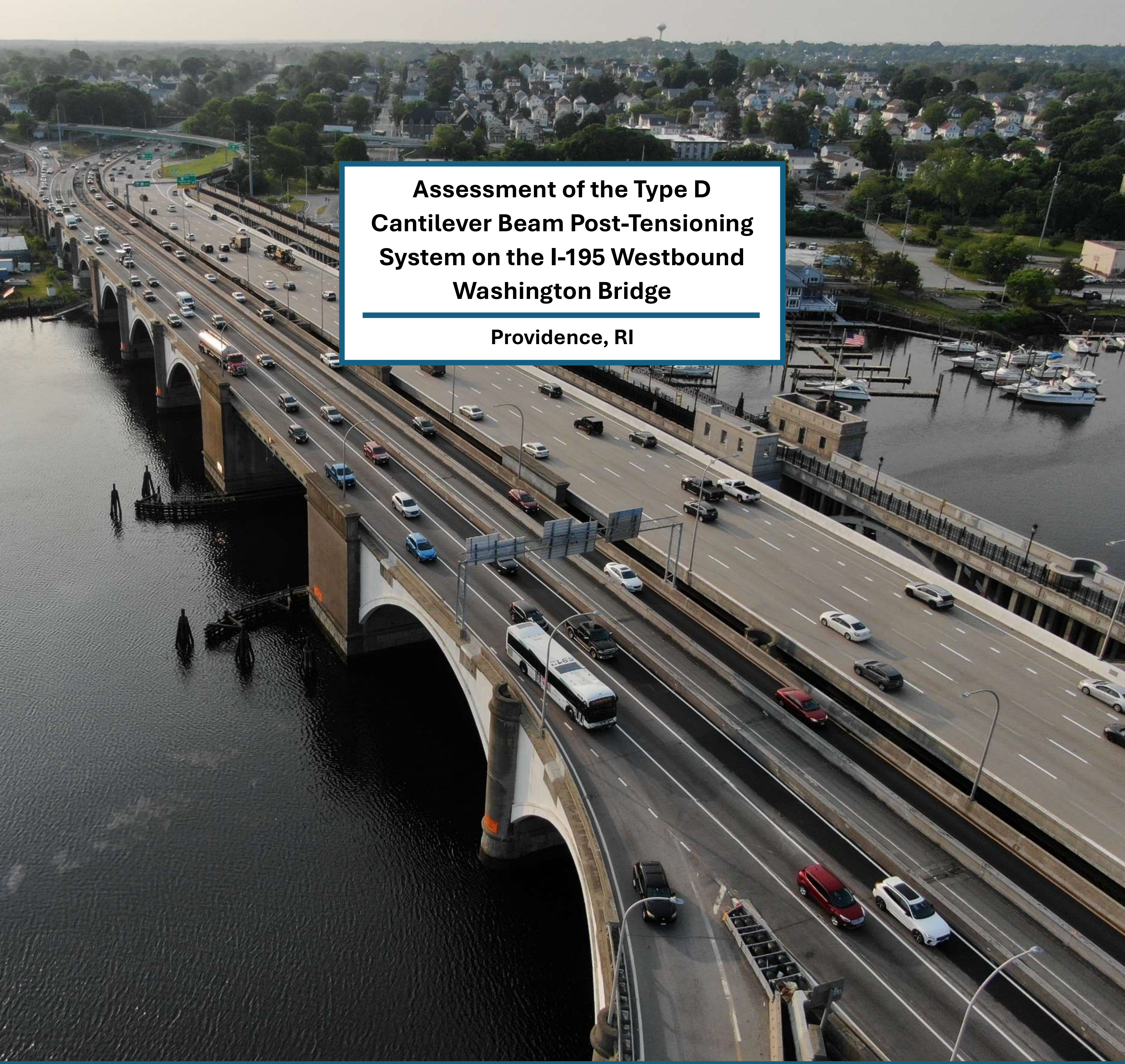
Assessment of the Type D Cantilever Beam Post-Tensioning System on the I-195 Westbound Washington Bridge

Providence, RI



Submitted To:
Rhode Island Department of Transportation

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Section 1. Executive Summary

The I-195 Westbound Washington Bridge is a complex structure composed of 18 spans of varying structural types including post-tensioned (PT) cantilever beams, drop-in prestressed girder beams, simply supported steel beams, and simply supported prestressed girder spans. The bridge is inherently non-redundant in global configuration due to the balanced and unbalanced nature of the bridge. The cantilever beams themselves do not have internal PT redundancy or adequate access to allow for the inspection, maintenance, or replacement of the PT elements.

During inspections on December 8th and 11th, 2023, as part of the on-going Design Build project for the Washington Bridge, VHB inspectors identified that tie-downs along the exterior PT cantilever beams in multiple locations at Piers 6 and 7 had failed. The mobilization efforts to repair the failed tie-down rods provided additional access to the ends of the PT cantilever beams at Piers 6 and 7. Inquiries from the contractor, on January 6th, 2024, regarding the sequencing of repairs to the beam ends and the tie-down rods led to the need for a more thorough assessment of the condition of the PT system within the cantilever beams along the full bridge in the immediate weeks following.

This report constitutes the assessment of PT systems for the Type D cantilever beams located at pier 6 and 7 following field inspections, field testing, and structural analysis of this portion of the structure. Additional inspection and field testing along the remainder of the bridge is ongoing, however currently available documentation is provided within this report. The field inspections and field testing performed on the structure to date have documented the following deficiencies:

- Failure of multiple Tie-Down Rods at Pier 6 and 7
- Exposed PT anchorage assemblies, undergoing active corrosion
- Exposed PT grout ports, undergoing active corrosion, with voids and soft grout present
- Significant voids within the PT ducts
- Soft grout within the PT ducts
- Suspected delamination of the grout within the PT ducts
- Corrosion of PT tendons within the PT ducts
- Unsound concrete in the anchorage development zone of the concrete beams
- Web cracking along the PT ducts at beams throughout the structure
- Concrete of beams at Pier 6 and 7 vulnerable to freeze-thaw damage
- Deck joints leaking above PT anchorage assemblies throughout the bridge

The PT system within the Type D cantilever beams are critical to the beam's load carrying capacity and the stability of the bridge. The type of deterioration documented indicates that the protective concrete around PT anchorages and grout within the PT system is compromised and has failed in locations. The concrete around the anchorage is non-air-entrained, unsound, cracked, and delaminating. The grout within the PT ducts has significant voids, large segments of soft grout, and shows evidence of active corrosion occurring.

The isolated beam line analysis and sectional capacity analysis performed for the Type D Cantilever beams at Piers 6 and 7 indicate insufficient capacity of the beams to support live load traffic without significant repairs and strengthening of the system, the viability of which is limited. The failure mode of the Type D Cantilever beams is anticipated to be non-ductile and therefore would present with limited or no flexural cracking as a warning sign before a potential failure occurs. This poses significant risk from a safety and an asset management perspective as the bridge may not present adequate structural distress prior to failure for routine inspection to detect.

Comparing the capacity findings of this assessment to the 2018 Load Rating of the additional cantilever types indicates that the structural deficiencies documented are anticipated to be found throughout the bridge. It is anticipated the Type A, A1, B, and C cantilevers will have significantly lower capacity than reported in the 2018 report and be found to be more critically deficient than the Type D cantilever beam as presented in this analysis. Repairs and strengthening of the bridge would be a complex operation that carries risk with regards to viability and sufficiency to fully address the deficiencies within the system. The ability to thoroughly identify and access all areas of the PT systems that require repair carries risk. This is due to the inaccessibility of locations on the structure and the geometry of the PT beams themselves. The viability of repairs requires the removal and replacement of the existing deck to lower locked-in load within the PT beams. Removing the existing deck carries risk due to the significant load redistribution that has occurred with the loss of the tie-downs at piers 6 and 7. The ability to reuse the existing beams for anchoring of external strengthening is a risk due to the unsound concrete present at the beam ends. The repair and strengthening options for the Washington Bridge are limited, complex, and do not completely mitigate the identified risks with the structure.

Deck joints along the bridge are located at PT cantilever beam ends where PT anchorage assemblies are present. Water infiltration through the deck joints is an ongoing concern as it allows for the continued deterioration of this critical location. The systems to protect against this type of deterioration at the PT anchorage assemblies are documented to be compromised and in locations have failed, specifically the concrete around the anchorage and grout within the ducts of the PT system. The risk of this deterioration leading to the potential loss of a PT anchorage and cable will only increase with time. The risk associated with the loss of a PT anchor cannot be fully addressed through repairs or retrofitting of the structure.

The risk of a progressive collapse of the bridge is believed to be low in a dead load only configuration of the structure, with no traffic on the bridge and the tie-down rods functioning as-designed. This risk increases with time as deterioration of the system continues and with the failed tie-downs located at pier 6 and 7 unrepaired. The ability to assess this continued deterioration by visual inspections is limited due to it occurring within the anchorage assemblies and with the PT ducts of the beams. This risk of a progressive collapse is reduced, however, once the installation of the new tie-down rod system, to repair the failed tie-downs located at piers 6 and 7, is completed.

The Washington Bridge is unique from a national perspective and a bridge inventory perspective for Rhode Island. The structure is unique given its inherently non-redundant systems combined with the magnitude and types of deficiencies documented, based on the experience of this writer and review of nationally documented studies. The bridge poses significant challenges for its inspection, maintenance, and repair due to its as-designed initial detailing and construction. The repairs and strengthening required to address the documented deterioration and deficiencies along the structure are significant, have limited viability, and have significant risk associated with them. To fully eliminate these risks and improve inspection, maintenance, and repair access would require the demolition and replacement of the superstructure, and potentially a full bridge replacement. From an asset management perspective, consideration should be given to these, and other topics covered in this report in the decision-making process to determine the immediate next steps and long-term viability and sufficiency of the structure.

Section 2. Scope of Analysis

VN Engineers, Inc. (VN) as a subconsultant to VHB, the lead design firm for the Barletta Heavy Division and Aetna Bridge, Design Build Joint Venture (JV), has been tasked with providing an assessment of the post-tensioned (PT) cantilever beams for the I-195 West Bound Washington Bridge located in Providence, Rhode Island, as part of an Emergency Repair Project. This assessment of the PT beams is focused on the Type D cantilevers located at Piers 6 and 7. This was done through a field investigation survey, structural analysis of an isolated partial beam line, integration of Field-Testing data performed by others, analysis of a Type D beam's sectional structural capacity, identification of risks within the PT system, and qualitative review of repair and strengthening options.

Section 3. Background

The design for the I-195 West Bound Washington Bridge located in Providence, Rhode Island, was completed in 1967 and the bridge was opened to traffic in 1969. The complex structure is composed of 18 spans of varying structural types including PT cantilever beams, drop-in prestressed girder beams, simply supported steel beams, and simply supported prestressed girder spans.

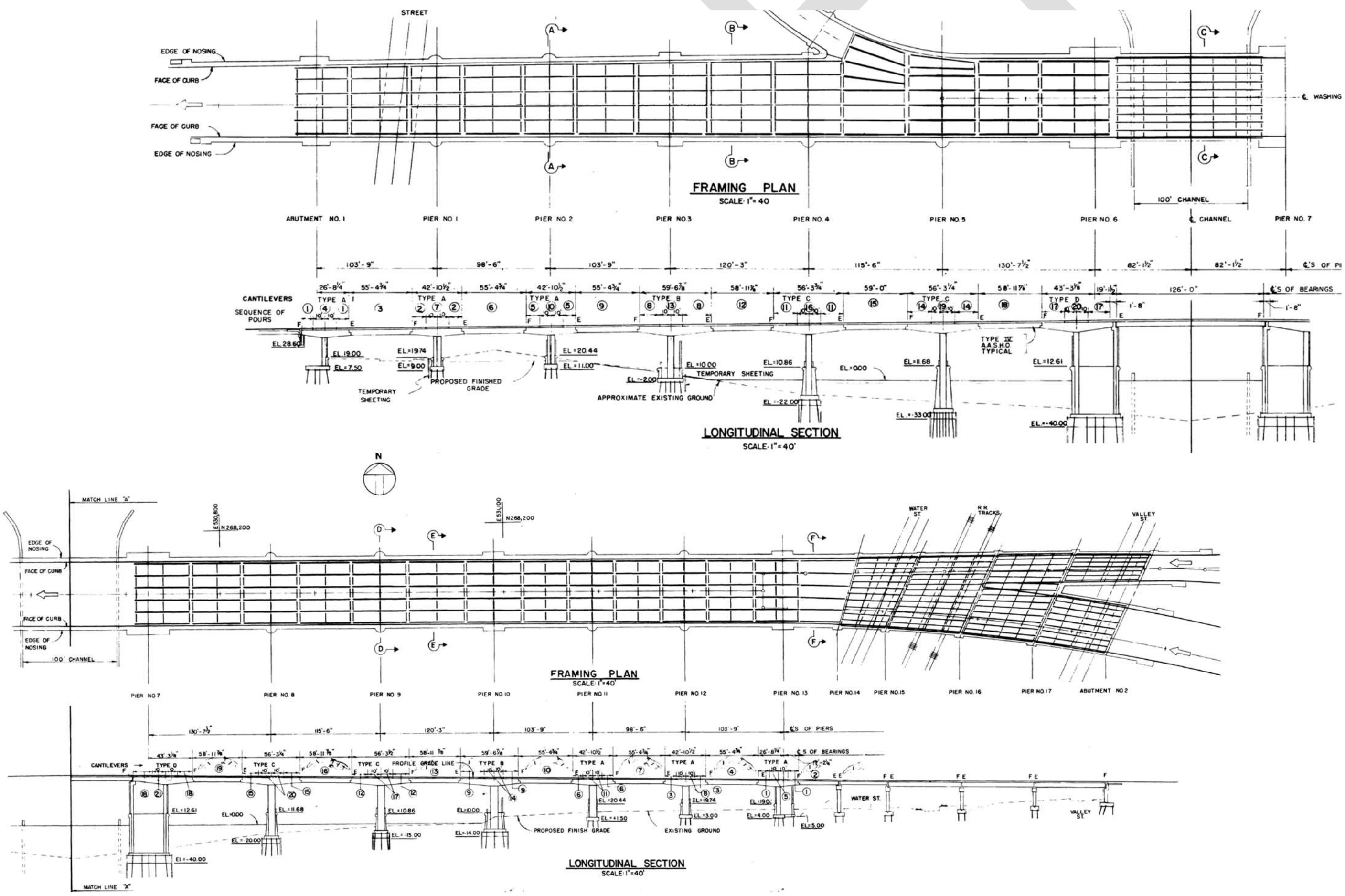

Figure 1: Layout of Washington Bridge

There are 14 PT cantilever beams located above piers along six beam lines within the concrete spans for the structure. There are 5 different types of PT cantilever beams within the bridge (Type A-1, Type A, Type B, Type C, and Type D). Each PT cantilever beam type is unique in its length, global geometry, number of PT cables, and mild-reinforcement detailing. The PT cantilever beams have two general conditions within the bridge, a balanced cantilever configuration and an unbalanced cantilever configuration.

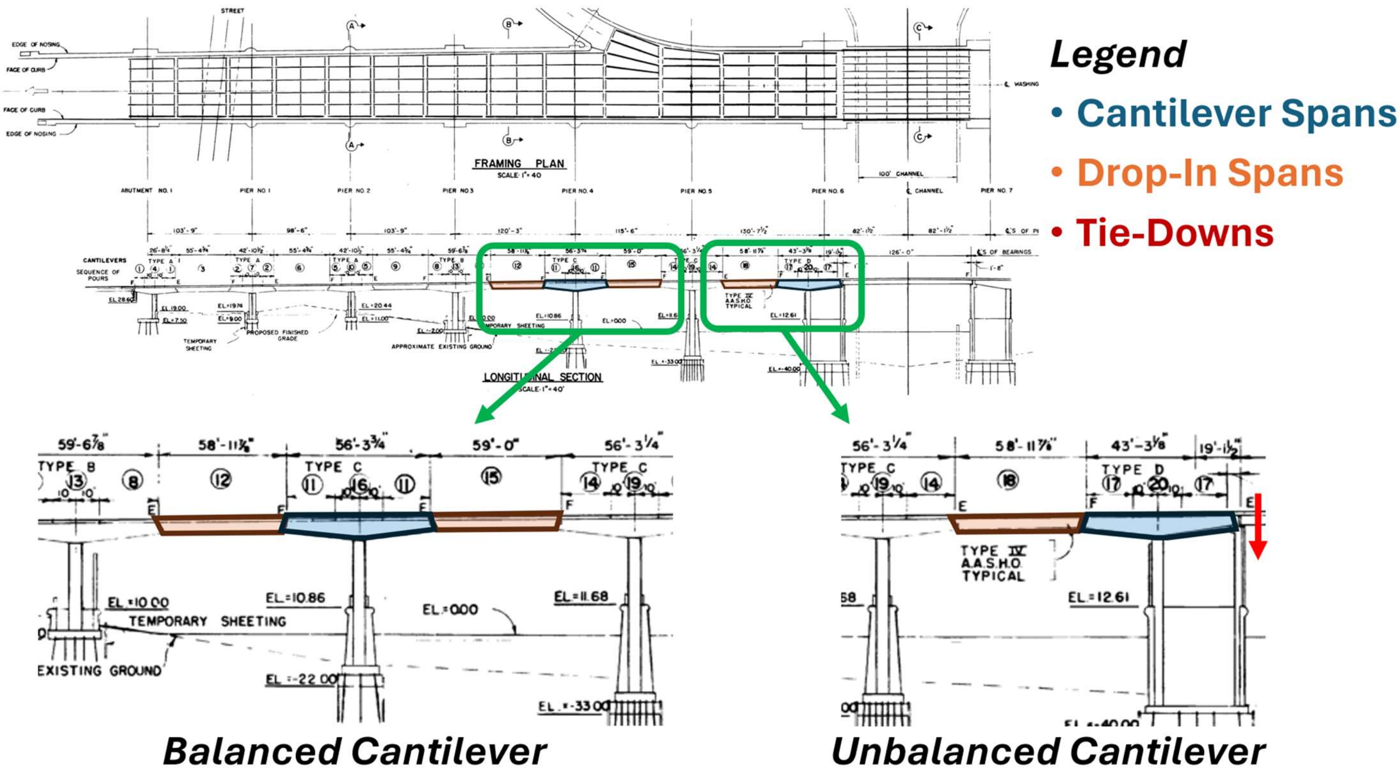

Figure 2: Balanced and Unbalanced Cantilever Configuration

In the balanced cantilever configuration, stability of the cantilever beam is established by the weight of adjacent drop-in prestressed girder spans and vertical post-tensioning rods anchoring the cantilever beam to the supporting pier. In the unbalanced cantilever beam configuration, a drop-in prestressed girder span is only located on one end of the cantilever. The stability of the unbalanced cantilever is maintained by post-tensioned tie-down rods located on the opposite end of the beam from the drop-in span. Each unbalanced cantilever beam has two tie-down rods holding the beamline. Only the exterior facing tie-down rods on the exterior beamlines are visually accessible for inspection. The remainder of the tie-down rods are encased in a concrete diaphragm between adjacent beam lines. The unbalanced cantilever beams are not anchored to their supporting pier and are free to rotate about a centrally located positioning dowel, or pintle. The unbalanced cantilever configuration with associated tie-downs is located at three pier locations along the bridge, the West Abutment, Pier 6, and Pier 7.

The Washington Bridge underwent significant repairs during 1996 that included repairing deteriorated cantilever beam ends, repairing web cracks and spalls, grouting of voids within ducts of the post-tensioned concrete beams, repairing delamination in beam webs, and installing of seismic catcher blocks at the ends of the cantilever beams to catch drop-in beams in the case of a seismic event.

Section 3.1 Post-Tensioned Concrete

As an introduction to the post-tensioned concrete system on the Washington Bridge that is the focus of this assessment report, a brief conceptual overview of post-tensioned concrete is warranted. Concrete is a composite material comprised of aggregate (stones or sand) bonded together via a cementitious paste that cures over time into a solid matrix. Concrete is innately strong when loaded in compression and is weak when loaded into tension. Under application of a load that produces bending (like in a beam on a bridge), one face of the beam will go into compression while the opposite face will go into tension. In a pure concrete beam, the tension face under load will crack potentially leading to failure, see figure below.

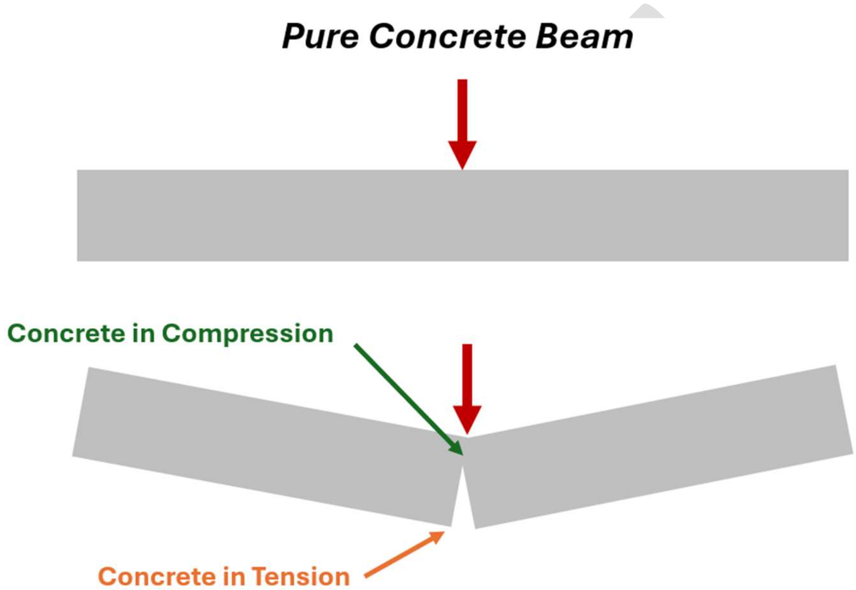

Figure 3: Pure Concrete Beam Example

The concept of a reinforced concrete beam utilizes the compressive strength characteristics of concrete along with the tension strength of steel reinforcement. In a reinforced concrete beam, steel reinforcement bars are placed near the tension face of the beam. As the beam is loaded and a crack forms, the steel reinforcement engages and picks up the tension load. Adding reinforcement to a concrete beam mitigates the size of cracks that would form under load and increase the load carrying capacity of the beam.

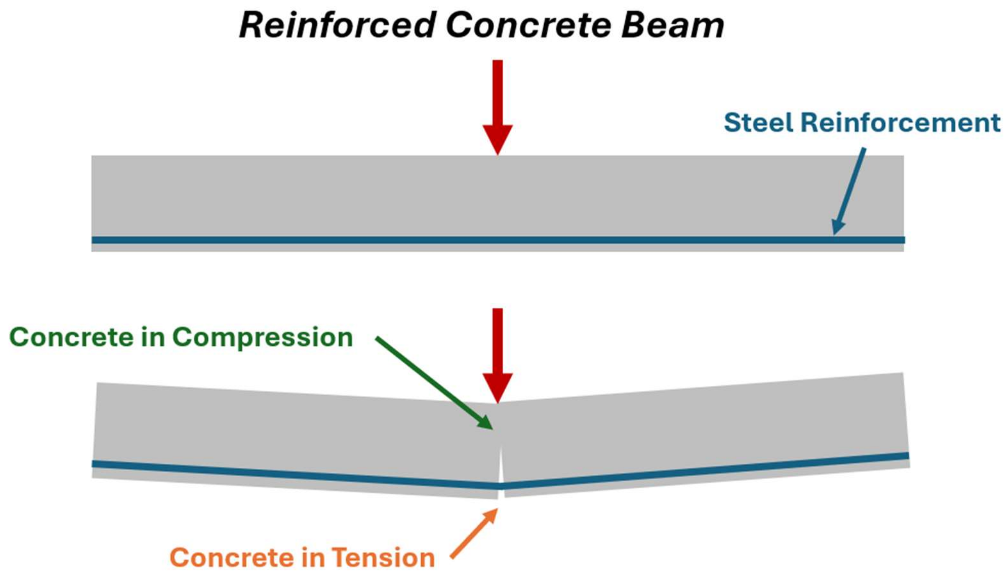

Figure 4: Reinforced Concrete Beam Example

In post-tensioned concrete, high tensile strength steel cables or bars are tensioned and then anchored into the ends of a concrete beam. When the steel cables are anchored into the beam, they pre-compress the concrete putting the beam into a state of compression. When a post-tensioned beam is loaded, the goal would be for no tension to develop. As load is applied to the beam the pre-compression would be reduced at the tension face but not overcome.

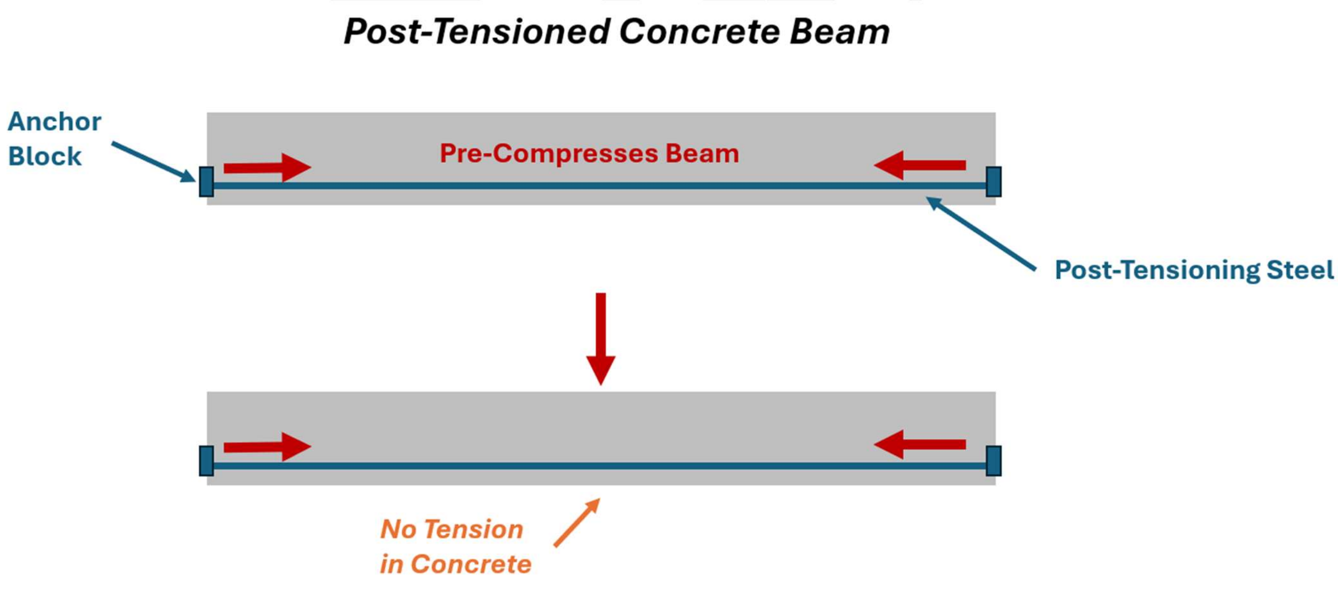

Figure 5: Post-Tensioned Concrete Beam Example

Eliminating tensile stresses in a concrete beam is advantageous. A leading cause of deterioration of concrete structures is water and chloride infiltration into steel reinforcement, which can lead to corrosion of steel, spalling and delamination of concrete, loss of reinforcement, loss of post-tensioning force, and other forms of deterioration. This water and chloride infiltration is accelerated by the presence of cracks within concrete beams.

Pre-compressing concrete elements through post-tensioning can help mitigate crack development and propagation and can add increased load carrying capacity to a beam element.

It is important to note that there are several critical components to a post-tensioned concrete beam.

- The Anchor Block or Anchorage Assembly - This is located at the ends of the beam and anchors the tensioning cables. This element is under high compressive loads.
- Anchorage Zone Concrete - This concrete is located directly behind the anchor blocks and where the load from the anchor block is passed into the concrete beam and is developed.
- Tensioning Cables (PT Tendons or Strands) - These run from one anchor to another and are stressed to develop the pre-compression force in the concrete beam. The cables are under high tension loads and are vital for maintaining the precompression force in the beam.
- The PT Ducts – Tensioning cables (tendons) are run through ducts within a concrete beam. These ducts protect the tension cables and define their profile.

Post-tensioned concrete elements are classified as either bonded or unbonded, depending on the type of filler material used in the ducts. Bonded systems have a concrete grout injected into ducts that surround the tension cables bonding the cables to the duct and the concrete beam. In a bonded system the concrete beam and the tendons behave as a composite system. Unbonded systems either have a flexible filler material injected into the ducts or have no filler. In unbonded systems the concrete beam and the tendons do not behave as a composite system and are not strain-compatible.

The application of prestressed and post-tensioned concrete for infrastructure projects emerged after World War II and became more readily used during the 1950s. This is an important reference point when considering the design of the Washington Bridge, as it was designed during the mid-1960s. Therefore, only about two decades of industrial knowledge on the performance, maintenance, inspection, and repair of post-tensioned concrete structures could be used to inform the design, detailing, and construction of the Washington Bridge.

For context, the post-tensioned concrete beams on the Washington Bridge were designed as a bonded system with concrete grout injected into their ducts. The tension cables are ½” strands that are stressed-relieved, with an ultimate capacity of 270 ksi, per construction shop-drawings of the stressing operations, see Appendix A.

Section 3.2 *Emergency Repair Project*

VHB on December 8th, 2023, inspected the steel span located between piers 6 and 7 following painting operations associated with the ongoing Design Build project for the Washington Bridge. During this inspection, it was identified that three tie-downs located on the facade face of the exterior PT cantilever beams in multiple locations at piers 6 and 7 had failed. A further inspection of the tie-downs located at piers 6 and 7 was performed on December 11th, 2023, by VHB. This inspection concluded that additional tie-downs were no longer performing as designed, due to the observed bouncing of beam lines and the up-lift from their bearings on four out of six beam lines at pier 6 and two of the six beam lines at pier 7. The loss of these tie-downs poses a stability concern for the bridge. Therefore, the I-195 West Bound Washington Bridge was closed on the afternoon of December 11th, 2023. An emergency repair project was then assigned to the Design Build team to address the identified tie-down deficiency.

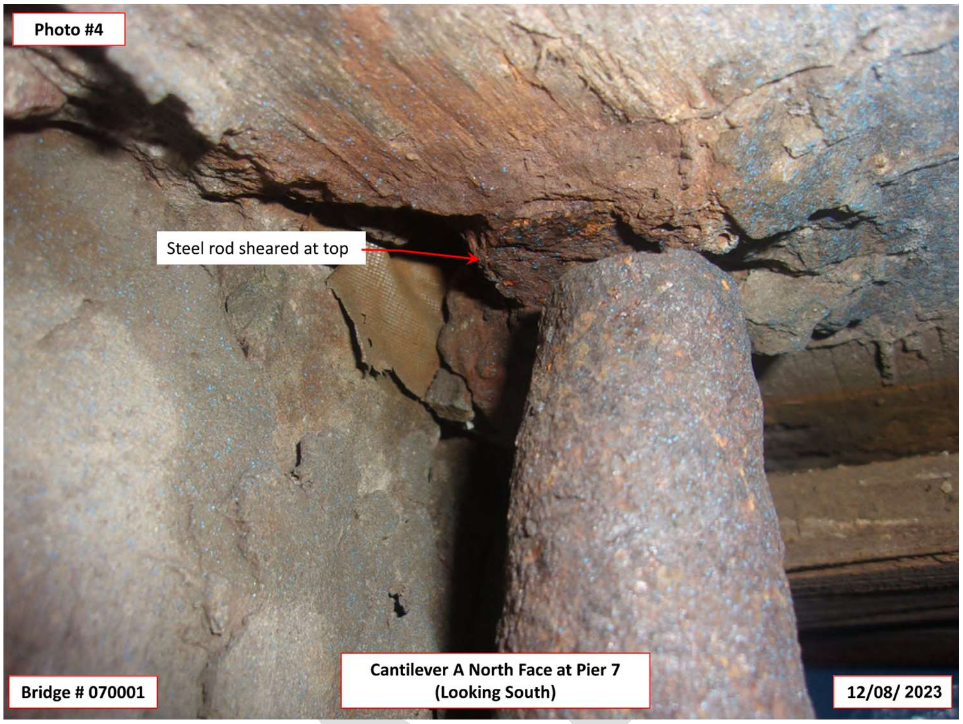

Figure 6: Tie-Down Failure at Beam A Pier 7

To repair the tie-down deficiency, new tie-down rods are to be installed along with additional strengthening of the pier walls at piers 6 and 7 to accommodate the eccentricity of the new tie-down rod's position. The installation of the new tie-down rods and needed pier wall strengthening required additional work-platform access to be installed at piers 6 and 7.

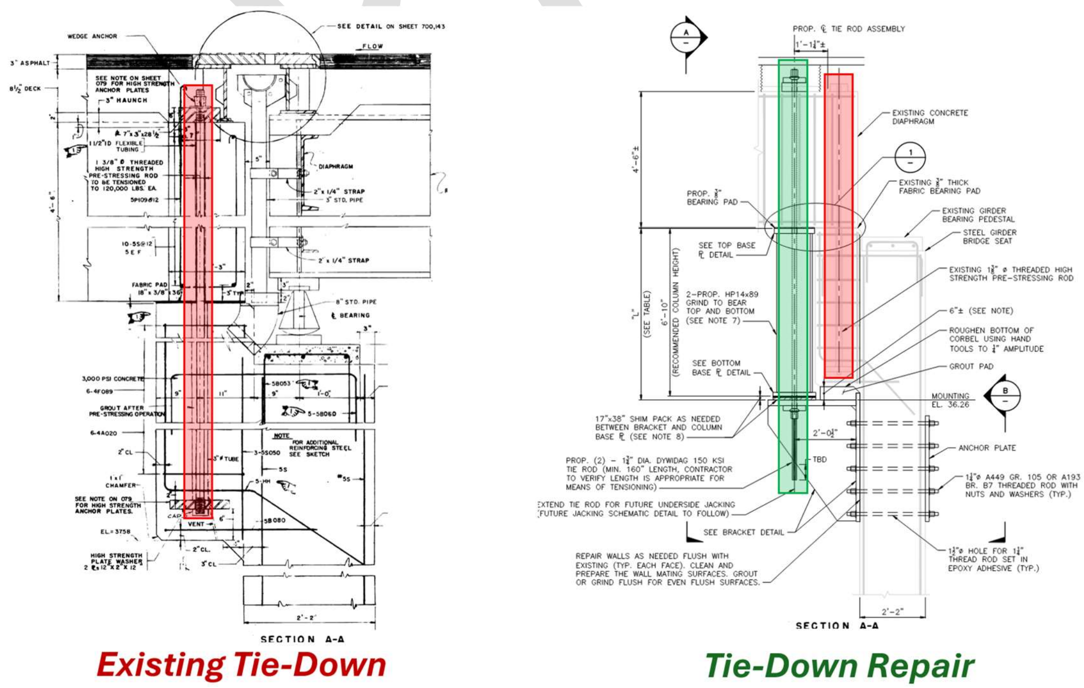

Figure 7: Tie-Down Repair Detail

During the installation of this work platform access, the JV provided pictures of Beam A at pier 7 (the north exterior beam line). These photos were sent to VHB, who forwarded them to VN for review, on the morning of January 6th, 2024. The review conducted by VHB and VN concluded that additional inspection and assessment of the PT cantilever beam ends would be required along with additional repairs at these locations. The additional inspection activities included sounding of the beam ends at piers 6 and 7 to better understand the extent of unsound concrete within these areas and to identify any further deficiencies that would need to be addressed in the beam end repairs. VN at this time identified risks associated with the PT system and the criticality of this location with respect to the cantilever beam's integrity.

Section 4. Field Investigation Survey

With the identification of the exposed anchorage systems at the beam ends at piers 6 and 7, additional resources were mobilized to the project site to aid in the assessment of the structure. This included field inspection personnel from VN, VHB, Michael Baker International (MBI), AECOM, and BDI. VN had personnel onsite from January 13th through January 17th, 2024. The focus of VN's field investigation was to assess the condition of the PT anchorage system for beams at piers 6 and 7 and to investigate if concerns identified at these locations were present or likely at other locations along the structure. A summary of critical findings from this field investigation survey is presented below.

Section 4.1 *PT Anchorage Systems*

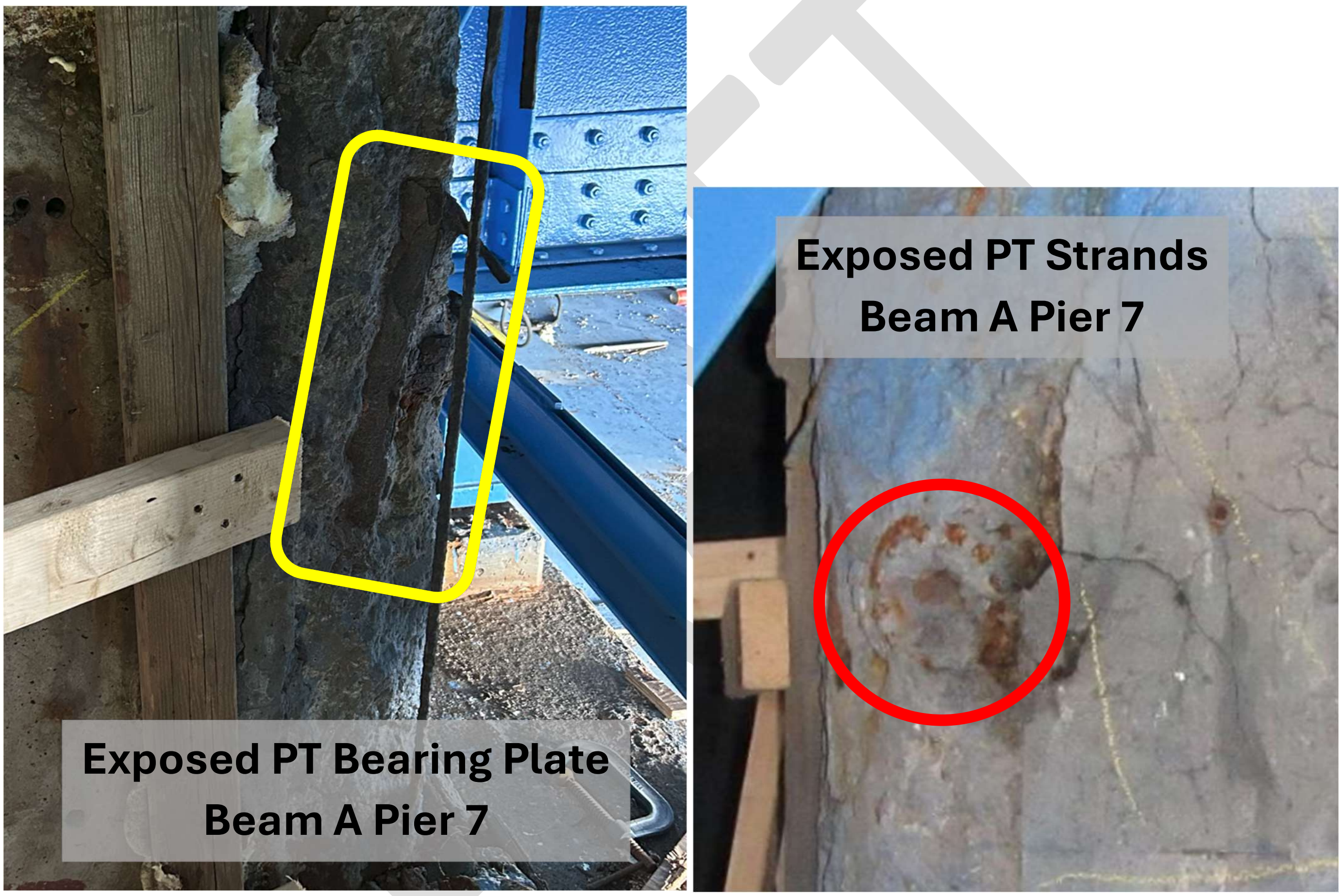

Figure 8: Exposed PT Anchorage at Beam A at Pier 7

The anchorage systems for multiple PT cables for the exterior beam A in span 6 and span 8 at piers 6 and 7, respectively, were observed to be exposed. The concrete spalls exposed the PT bearing plate, highlighted in yellow in the above image. The PT bearing plate, a critical element of the PT system, transfers force from the PT tendons into the concrete beam. Each PT cable at these anchor locations carries over 200,000 lbs of force. Each cable is anchored to the base plate through chucks. The ends of these strands and their anchoring chucks were found to be exposed with active corrosion of the steel occurring. Individual exposed strands can be seen in the above photo in the red highlighted circle. At the center of these anchored strands is the grout port for the PT anchor system. This port is where grout would be injected into the ducts of the PT system.

The concrete around the PT anchorage system was observed to be unsound with significant cracking, hollow areas, and delamination observed. The contractor removed unsound concrete via chipping to allow for further inspection, assessment, and preparation for future repairs of the ends and sides of the beams at piers 6 and 7. For Beam A at Pier 6, the north exterior beam, unsound concrete was removed to a depth of 2.875". Unsound concrete was still observed at this depth within the beam. VN halted continued chipping operations at this location from proceeding deeper to prevent removal of concrete within the PT anchorage development region. The approximate location of this unsound concrete is identified in yellow in the below image.

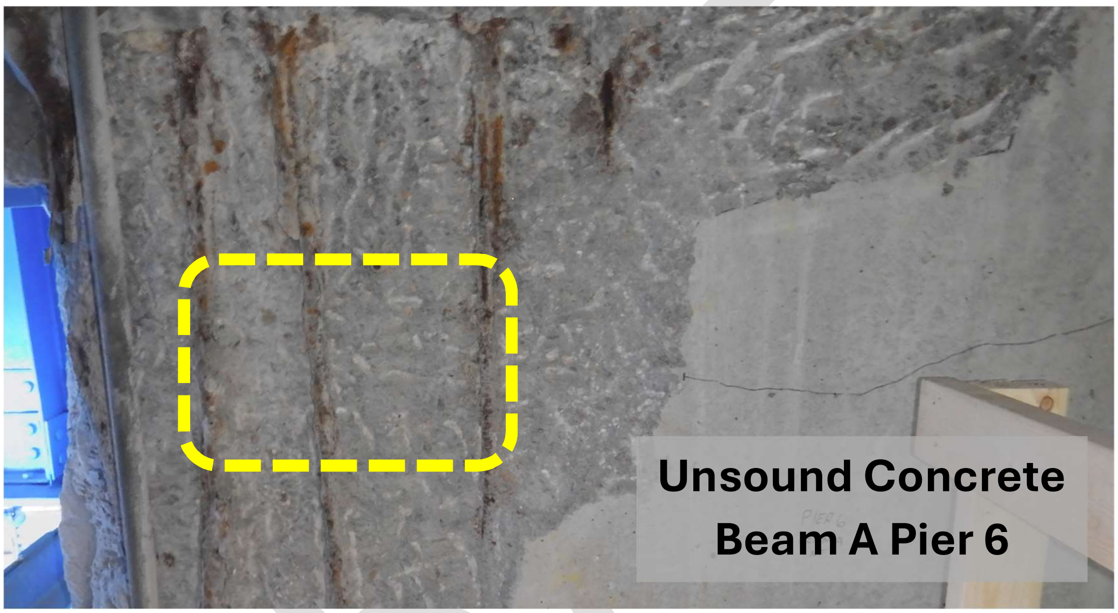

Figure 9: Unsound Concrete at Beam A at Pier 6

As of January 17th, 2024, after removal of unsound concrete, PT anchorage heads were found to be exposed at every beam end along pier 6, except at beams D and F. Removal of unsound concrete at Beam D at Pier 6 is not accessible due to the position of the steel beam end stiffeners for the adjacent span. Therefore, visual assessment of this beam was not able to be performed since the unsound concrete couldn't be removed. The end of Beam F at Pier 6 has been patched in the past, and the patched concrete around the PT anchorage was found to be sound at the time of inspection. With respect to beam ends along Pier 7, PT anchorage heads were exposed at every beam. This includes the Pier 7 end of Beam F which was previously patched. The patch was found to be unsound around one of the PT Anchorage heads with surface corrosion present on the steel elements, see photo below.

Figure 10: Exposed PT Anchorage Beam F at Pier 7

Section 4.2 *PT Grout Ports*

After removal of unsound concrete, PT anchorage systems at multiple beam locations were found to be exposed. At Beam E at Pier 7, concrete was removed via sounding operations and exposed a PT anchorage head and the grout port. The PT grout port was observed to have heavy corrosion, a full diameter void, and soft grout present.

Figure 11: Voids and Soft Grout within PT Grout Port on Beam E at Pier 7

Heavy corrosion, voids, and soft grout were found at multiple PT anchorage assemblies within the grout port at beam ends at Piers 6 and 7. At Pier 7 Beam B for example, all three PT anchorage heads were exposed with the removal of unsound concrete and two of the three exposed grout ports contained voids and soft grout, see photo below. The number of locations observed with this form of defect indicates a widespread issue with the grout system for the structure.

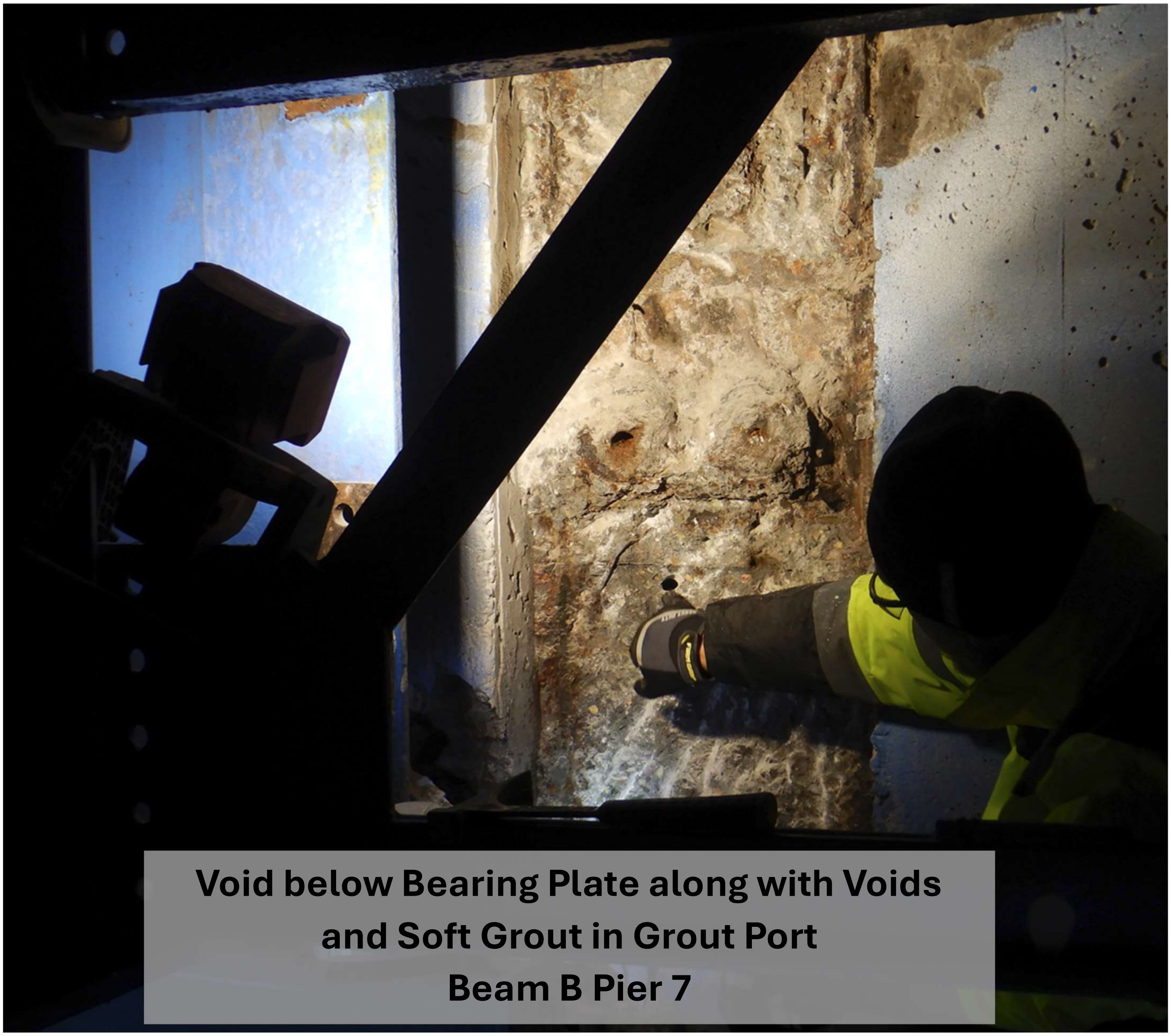

Figure 12: Deficiencies at End of Beam B at Pier 7

With respect to Beam B at Pier 6, additional defects were observed including a void at the base of the PT anchor bearing plate and unsound concrete within the bearing column for the beam after 5” to 6” of concrete depth was removed from the beam end. Cracking of the concrete around aggregate in this unsound area was observed. Additional unsound concrete at this depth could not be removed without risk of compromising the bearing capacity of the beam. Repair of this beam end would be complicated due to the existing tie-down rods at this beam believed to be intact and pre-tensioned with a force of approximately 240,000 lbs. While in the field, VN directed that no construction loading was to be placed above this beam line and in the adjacent bays until this beam end could be repaired.

The type of deterioration observed around and within the PT anchor grout ports indicates a compromised grouting of the PT systems. This allows for corrosion of the PT strands directly behind the PT bearing plate with no method of directly observing the degree of corrosion, section loss of strands, or potential loss of a strand. These findings confirmed the need for a more intensive investigation into the condition of the grout and strands along the PT beams away from the anchorage zones. This additional investigation was started by BDI on January 15th and is still ongoing along the bridge at the time of this report. Initial findings from BDI's investigation are discussed in the Field-Testing Data section of this report.

Section 4.3 *Web Cracking*

Longitudinal web cracking was observed along multiple beam lines following the path of the PT ducts within the beams. These cracks were variable in length and width, with the longest being in excess of 97" in length and up to 0.04" in width. When comparing these cracks to historical records, it was documented that the crack lengths and width have been increasing with time. Example of this crack propagation is the web cracking of Beam A at Pier 6 along the south face of the web. In 1996, per the rehabilitation plans, documentation of the existing conditions of this crack indicated an approximate 60" in length. As of January 13th, 2024, this crack has grown to 96" in length. This propagation in crack length indicates these cracks are not solely from the original tensioning operations of the PT system, which can occur, but are a sign of active distress on the system.

Figure 13: Web Cracking along Beam A at Pier 6

Section 4.4 *Joints at Piers 6 and 7*

During the course of the field investigation, rain and snow occurred for two days, allowing the team to observe significant water infiltration through the deck joints at piers 6 and 7, which flowed along the back face of the PT beams under the deck joints. Water infiltration was most significantly observed around the facade beam ends, but was observed at the back face of interior beam ends as well. This water was observed to flow over beam ends, directly over the PT anchorage locations. VN directed the contractor to “caulk” the deck joints around the beam ends to help mitigate water infiltration into the exposed anchorage systems during this assessment period.

Section 4.5 Other Locations Along the Bridge

The field investigation performed by VN was not limited to the beams located piers 6 and 7, but included inspection of the beams at the West Abutment, isolated beam lines at piers 3, 4, 5, 8, 9 and 10. The beam ends at these locations, where the PT anchorages for these beams are located, are not accessible to visual inspection and sounding due to the original design and detailing of the West Abutment, the presence of drop-in prestressed girders, and supplemental concrete added as part of the seismic retrofit at cantilever beam ends.

The general condition of the beams that were accessible, however, were found to be consistent with the condition of the beams are piers 6 and 7, including observations of web cracking along anticipated duct paths and hollow areas along beam webs.

Additionally, significant spalls were observed along the bottom flange and web of Beam F at the West Abutment as can be seen in the below photos.

Figure 14: Spalls and Condition Photos for Beam F at West Abutment

It should be noted that the beam ends, which contain the PT anchorages, and tie-down rods for all beam lines at the West Abutment are not accessible for visual inspection due to the original design and detailing. These are critical components of the structural system for the bridge, which cannot be accessed for inspections, routine maintenance, or repair without removal of the approach roadway and excavation of the soil against the abutment. The tie-down rods and beam ends are encased in a concrete diaphragm that is cast directly against soil, which was observed to be damp during the field investigation. A photo of the observed condition of this end diagram at the west abutment is provided below.

Figure 15: Observed Condition of End Diaphragm at West Abutment

The condition of joints at the beam ends at other spans observed by the team were consistent with the findings at piers 6 and 7, where significant water infiltration at the ends of the PT cantilever beam ends was observed. The photos below document the water infiltration along the end diaphragm for the Pier 9 cantilever beams E and F within span 9. Water was observed to pool on the drop-in span bearing seat at the cantilever beam end. This is the location of multiple PT anchorage assemblies for these cantilever beams.

Figure 16: Water Infiltration at Deck Joints along Bridge

A complete record of VN's field notes and pictures taken during our field investigation is provided in Appendix B.

Section 5. Field-Testing Data

BDI, under an inspection contract with Michael Baker International for RIDOT, is performing non-destructive testing (NDT) and collecting concrete cores and samples for additional assessment of the structure. VN did not lead this effort, but assisted in the recommendation for testing, the recommendation for inclusion of displacement sensor monitoring, identification of locations where testing would be recommended, and review of the testing data to aid in the assessment of the structure's condition. A summary of critical findings from BDI's ongoing testing operations is summarized here, as it pertains to the assessment of the Pier 6 and Pier 7 PT cantilever type D beams.

Section 5.1 *Displacement Sensor Monitoring*

Displacement sensors have been placed along the structure to provide active monitoring for movement of the cantilever beams during assessment and repair activities. This active monitoring allows for notification of when significant movement occurs on the structure, which could be an indication of additional failures of the remaining tie-down rods at Pier 6, Pier 7, and at the West Abutment. At the time of this report, no significant movement at these sensor locations has been documented. See Appendix C for sensor location placement.

Section 5.2 *Concrete Sampling and Testing*

During the week of January 22nd, 2024, six concrete cores were taken along beam ends at piers 6 and 7. Three of these concrete cores were taken for compressive strength testing and three were taken for petrography analysis. Locations of where the concrete cores were taken is provided below.

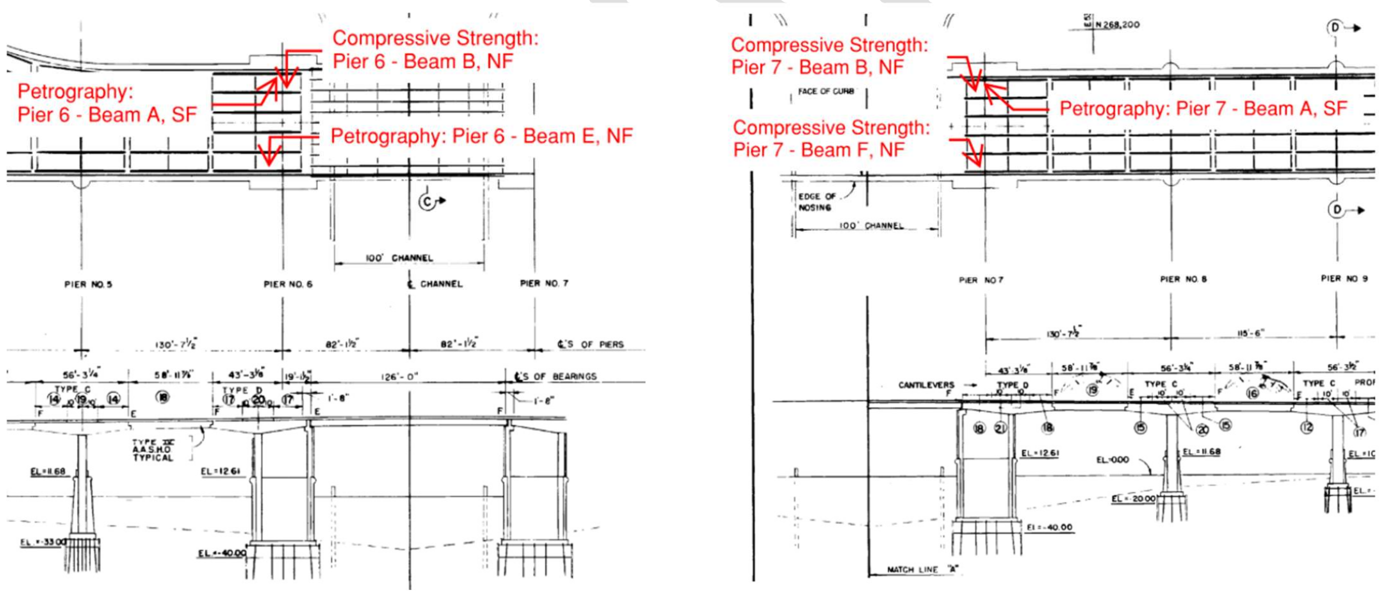

Figure 17: Locations of Concrete Sampling

The concrete compressive strength tests resulted in a breaking strength of the samples between 7.37 and 8.77 ksi. The design strength of the concrete beam elements is specified as 5 ksi for the tested beams. This indicates that the concrete compressive strength for the beams tested at piers 6 and 7 meets and exceeds the specified design strength requirements.

The results of the petrography analysis indicated that the concrete is non-air-entrained. Air-entrained concrete has a system of microscopic air bubbles that form during the mixing and curing of concrete. These air bubbles increase the freeze-thaw durability of the concrete and also increase its resistance to scaling caused by de-icing chemicals. The concrete samples taken from beams at piers 6 and 7 indicate the concrete is non-air-entrained, and therefore is at a greater risk of deterioration caused by freeze-thaw cycling. Ettringite was observed in concrete and grout fragments taken from Pier 6 and Pier 7 Beam A, which is an indicator that the concrete element has been exposed to elevated moisture levels for long periods of time.

The presence of ettringite and identification of the concrete as non-air-entrained is consistent with the deterioration observed at the beam ends at Pier 6 and Pier 7 where significant water infiltration was observed at join locations.

Section 5.3 Ground Penetrating Radar (GPR) and Ultrasound: Condition Assessment

To aid in the assessment of the PT tendons and grout within the ducts, BDI used GPR and Ultrasound Shear Wave Tomography (MIRA). The MIRA technology scans the outside of a concrete beam to develop a 3D scan of the beam's interior, like an ultrasound or medical CAT scan. These 3D scans of the interior of the beam can be used to locate potential discontinuities in the concrete, delamination of the grout, soft grout, or voids within the ducts. An example of the results of this technology is presented below for the south face of Beam A at Pier 7.

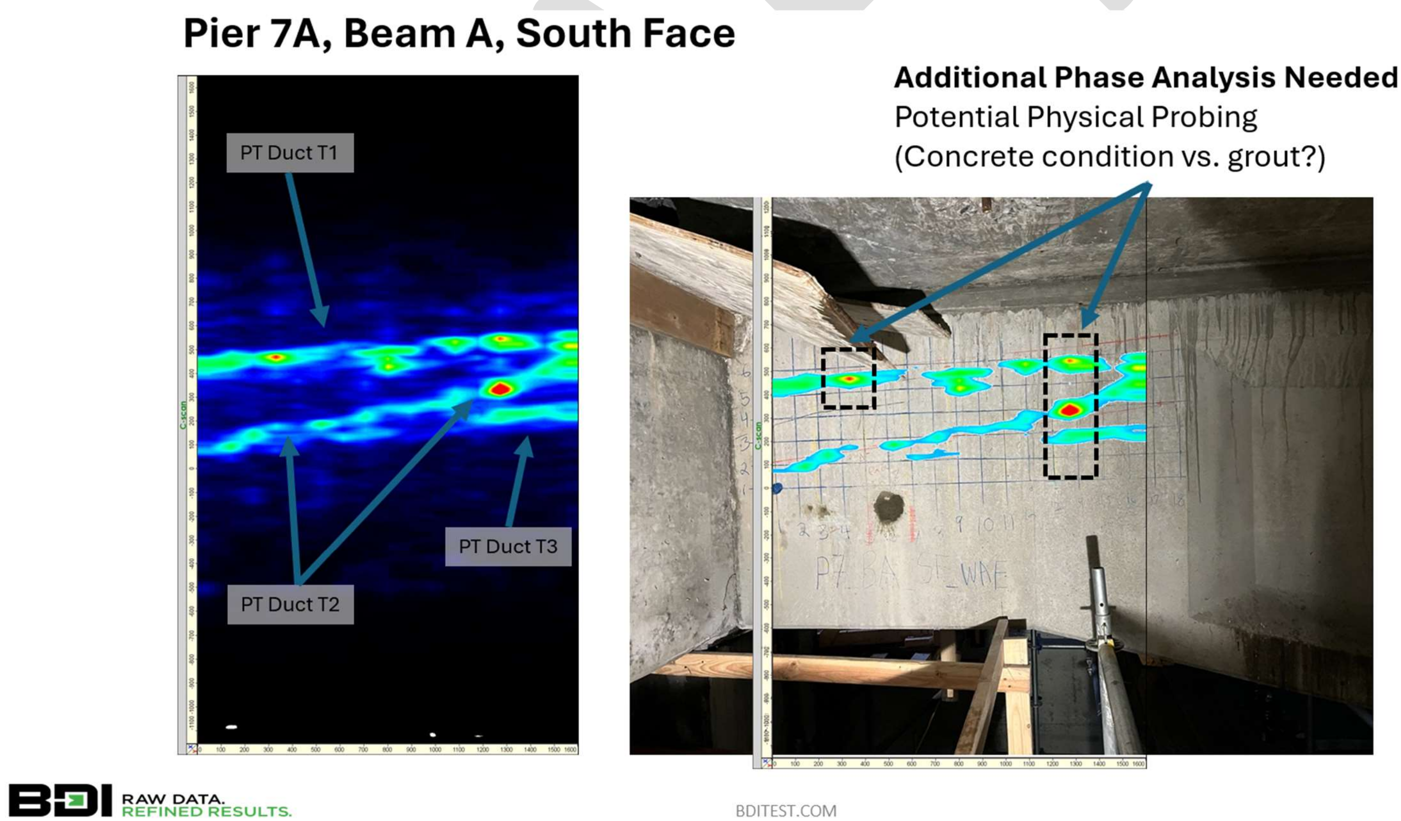

Figure 18: Example of MIRA Scans for Beam A at Pier 7 - South Face

These scans can be further post-processed through a phase analysis to help identify what type of deficiency could be found within a duct or surrounding concrete. Scans of the beams located at piers 6 and 7 were analyzed by BDI and several locations were identified for further testing via physical probing.

Section 5.4 Physical Probe Testing of Beams at Piers 6 and 7

Six locations were selected along beams at piers 6 and 7 for physical probe testing. To perform the physical probing, concrete is removed to the surface of the PT duct. The PT duct is then opened and the interior is inspected along with a sample of the grout taken for further analysis. The results of physical probing revealed the presence of significant voids, soft grout, and delamination of the grout from the interior of the ducts. The ducts themselves were also observed to be smooth steel tubes. Smooth duct tubes rely on the bond between the concrete and the tube for stress and strain compatibility, where corrugated ducts have a mechanical bonding mechanism.

Figure 19: Photos of Probing at Beam C at Pier 6 - Cable 4

The above images are from the probing performed on Beam C at Pier 6 along cable 4. The rust staining on grout inside of the exposed duct, highlighted in yellow in the above right image, indicates delamination of the grout from the interior of the duct. This delamination of the grout is an indication of the potential debonding of the grout from the duct.

Figure 20: Photos of Probing at Beam B at Pier 6 - Cable 5

The above images are from the probing performed on Beam B at Pier 6 along cable 5. The void found in this cable extended 4ft in one direction along the duct and further than 10 ft in the other direction along the duct. The full extent of the void was not able to be probed due to the maximum length of the borescope camera being 10 ft. Within this void, surface rust is visible along the top of steel duct and along PT tendons.

A draft report from BDI on testing completed to date is provided in Appendix D.

Section 6. Structural Analysis of Isolated Partial Beam Line

To assess the condition of the Type D PT Cantilever Beams a representative structural analysis was performed on an isolated beam line. Beam Line E, the first southern interior beam line, was selected for this representative analysis. This analysis assumes the tie-down located at pier 6 has not failed and is still acting as a hold down at the unbalanced end of the cantilever beam over pier 6. This allows the representative analysis to serve as a baseline of understanding for the condition of the bridge, prior to adding the complexity of the tie-down rod failure. The failure of the tie-downs is an extreme event loading condition for the structure. This extreme event condition was investigated using a version of this beam line approach and is discussed at the end of this section.

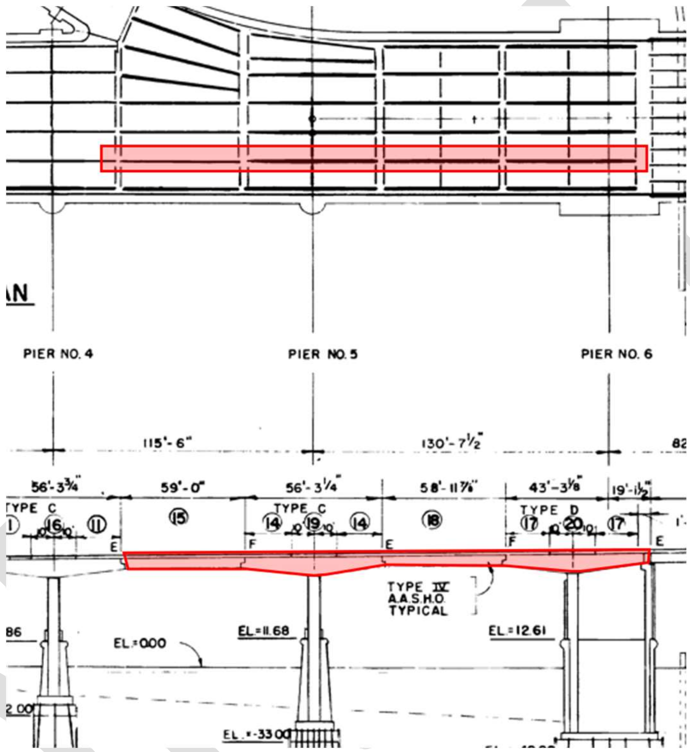

Figure 21: Location of Isolated Beam Modeling Effort

Section 6.1 Modeling Approach to Representative Analysis

The isolated beam line model for the representative analysis was created in LARSA 4D. The model included Beam Line E from the tie-down end of the Cantilever Beam over pier 6 through to the drop-in span at span 5. The limits of the model were selected to minimize modeling development time while still providing representative longitudinal stiffness and load distribution for the cantilever beam over pier 6, which is the focus of the analysis and assessment. LARSA 4D is a finite element analysis program with the capabilities to perform a construction stage analysis. This form of analysis allows for the inclusion of the loading and stress history of a complex structure to be accounted for, including the time-dependent effects of a post-tensioned system, over the course of the structure's service life. Boundary conditions at the ends of the model were defined using a combination of springs and nodal supports. These boundary conditions were calibrated based on past analytical models which were created for the analysis of the widening of the structure as part of the original Design Build project. The concrete beams were defined using beam elements. The deck was defined using shell elements, and the composite behavior between the beams and deck were modeled using rigid links. The construction stage analysis performed started at the casting of the PT cantilever beams, a simulated construction phasing for the bridge, major rehabilitation activities including the 1996 repairs, and the changes in deck continuity associated with the 2016 and current Design Build project. The post-tensioning cables for the Type D cantilever over Pier 6 were modeled within LARSA. The short-term loss constraints were input into LARSA based on available shop-drawing and as-built plans. Long term losses for the system were calculated internally by LARSA per the AASHTO LRFD specifications over the course of the construction stage time history analysis. A more detailed discussion on the development of the extreme event version of this model, which includes the failure of the tie-down at Pier 6, is included in **Appendix X.**

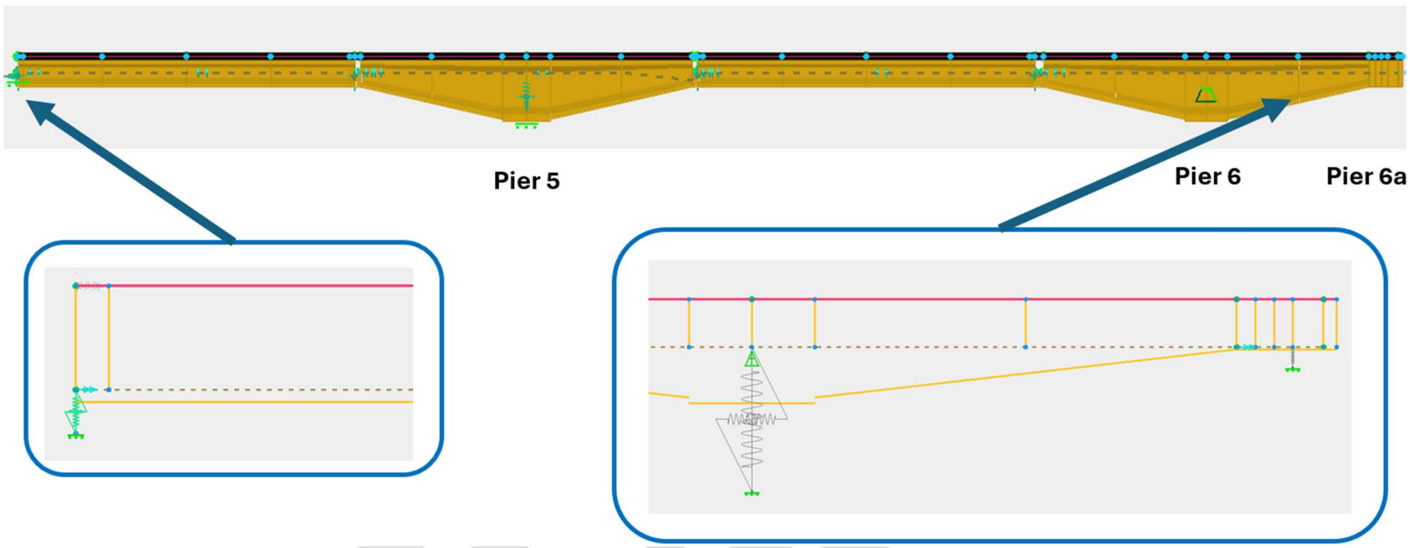

Figure 22: LARSA 4D Isolated Beam Line Model

The model was calibrated and verified against available information in the As-Built Plans, Shop-Drawings, and the current 2018 Load Rating of the Bridge, performed by others. A comparison of the peak dead load, wearing surface, and HL-93 live load moments over Pier 6 are presented in the table below. The 2018 load rating analysis included a more holistic and complex 3D model of the structure. A 3D model allows for a more refined stiffness and load distribution on the structure, particularly in the transverse direction (or between adjacent beam lines). It would be anticipated that the peak moments from a 3D analysis when compared to the isolated beam line analysis would be slightly lower. The comparison between the isolated beam analysis performed and the 2018 Load Rating follows this anticipated trend and good agreeability between the peak moments was found.

Table 1: Peak Negative Moment Comparison

Peak Negative Moment over Pier 6	2024 Isolated Beam Analysis (kip-ft)	2018 Load Rating Analysis (kip-ft)	Percent Difference
Dead Load	4541	4241	6.8%
Wearing Surface Load	570	569	0.2%
HL-93 Live Load	3488	3265	6.6%
Combined Services Loading	8599	8075	6.3%

Section 6.2 Incorporation of Inspected Field Conditions and Field-Testing Data

Based on the review of the as-built drawings, shop drawings, inspected field conditions, and field-testing data, the following assumptions are used in the analysis and assessment of the Type D Cantilever beams at piers 6 and 7.

- ***Tendons are assumed to be unbonded*** – While the original design of the structure intended for the tendons to be bonded, the presence of significant voids, soft grout, and suspected deficient grout along the length of the tendons investigated to date indicates that this is not a valid assumption. Therefore, the system is assumed to be unbonded based on current field conditions in this assessment.
- ***Effectiveness of Tendon Stressing at Services*** – Given the condition of the end-anchorage systems being exposed and with active corrosion occurring, voids present in the anchorage assembly, and the presence of surface rust on the tendons observed during isolated probing, an effectiveness factor of the PT tendons and stressing is assumed at 0.85 (a reduction of 15%) for the service limit state assessment.
- ***Condition Factor of Beam at Strength*** – Given the inspected condition and field-testing data available at the time of this analysis, a "Poor" condition factor is assumed for the strength limit states. A poor condition factor is assumed at 0.85, due to the inability to take accurate field measurements of all deteriorated elements and general condition documented. *(Note: the effectiveness of tendon stressing factor was **not** concurrently applied with the condition factor for the strength assessment to avoid overly reducing the beam's capacity for the inspected condition.)*

Section 6.3 Service Analysis of Representative Type D Cantilever Beams

The AASHTO LRFD Bridge Design Specification, 9th Edition – 2020 (AASHTO), stipulates that concrete elements with unbonded prestressing tendons shall have a zero-tension stress limit during service conditions after all losses are considered. Therefore, under service loading conditions no tension is allowed in the concrete beam per the AASHTO. The Rhode Island LRFD Bridge Design Manual is more restrictive than the AASHTO, stipulating that even for bonded tendons subjected to severe corrosive conditions a zero-tension limit is required.

Service level stress for the Type D cantilever beam over pier 6 were extracted from LARSA 4D analysis for the top and bottom fibers of the PT beam. Stresses were combined using load factors for the Service III limit state, which considers dead and live load concurrently, along with the inclusion of a 0.85 effectiveness factor for the PT force within the system. A plot of Service III stresses for the representative beam line is presented below.

Stresses along the top fiber of the beam were found to exceed the no-tension limit over the pier 6 support, the location of peak bending moment, and along the top of the beam between the cable 4 and cable 5 anchorages. The peak tension stress in the beam under the Service III limit state was found to be 863 psi, which is in violation of the no tension limit for the AASHTO and RIDOT criteria for unbonded post-tensioned concrete elements.

It is worth noting that if no reduction in PT effectiveness is considered, the beam is still found to have tension along its top fiber over the pier and between the cable 4 and 5 anchor heads in violation of AASHTO and RIDOT criteria for unbonded post-tensioned concrete elements.

The representative analysis for the Type D cantilever in a dead load only configuration, with no traffic allowed on the bridge, found the beam to be in a state of global compression with no tension found in the top or bottom fiber.

Concerns with the violation of the tension stress limits are only documented to be present for the type D cantilever beams under live load, when the tie-down rods are working as intended.

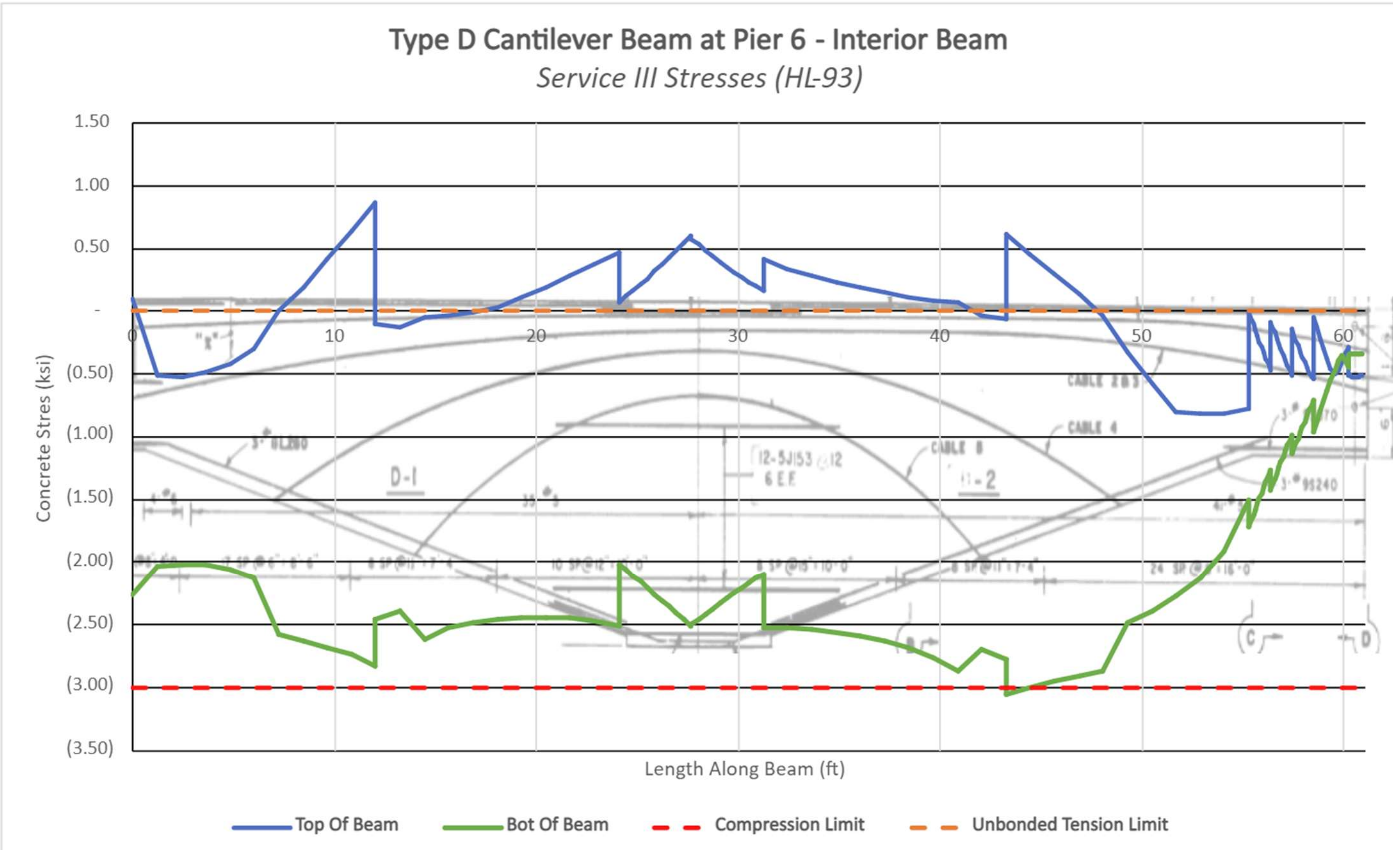

Figure 23: Services III Stress Plot for Type D Cantilever Interior Beam

Section 6.4 Strength Analysis of Representative Type D Cantilever Beams

The strength capacity of a post-tensioned concrete element is dependent on the bonding condition of the tendons of the concrete beam. The Washington Bridge was originally designed and detailed to be in a bonded condition. Based on the documented condition of the tendons from field inspections and field testing the tendons cannot be assumed to be in a bonded condition for the consideration of their strength capacity.

It is worth nothing that the tendons were initially stressed to a low stress level compared to their ultimate capacity. The AASHTO requires for bonded tendons to have their effective stress (the stress level in a tendon after all losses are considered) be over 50% of the ultimate stress capacity of the tendon, per AASHTO Section 5.6.3.1.1. If this limit is not met, the bonded strength capacity equations in the AASHTO cannot be used. This limit is in place to ensure that tendons would yield (i.e. the beam would deform, and the tendons would elongate) to a sufficient extent that the compression fiber of the concrete would reach its ultimate capacity. If the limit is not met, there is risk that the beam would not adequately deform, and the tendons would not elongate enough resulting in a potential failure prior to the estimated capacity per the AASHTO equations. The effective stress in the tendons for the representative Type D cantilever beam over pier 6 varies from 45% to 38% of the ultimate stress capacity of the tendon, which is below the 50% limit. This is an indication that the beam was not designed with adequate ductility per current post-tension concrete design theory and practice.

The strength capacity assessment of the Type D Cantilever Beams assumes an unbonded condition for the tendons. When considering an unbonded post-tensioned concrete system, it is important to note that the system is not composite. In an unbonded system, no strain compatibility between the tendons and the concrete beam is present. The tendons are free to move apart from the concrete beam, and only elongate and pick up additional load based on the global displacement of their anchor points. Therefore, when considering the strength capacity of an unbonded post-tensioned concrete beam the global displacement and limits of the system need to be considered.

For the Type D Cantilever beams, the strength capacity is based on the formulation of a plastic hinge, i.e. a ductile crack, over the Pier 6 support. Therefore, the strength capacity of the beam is directly related to how much this plastic hinge over the pier 6 support can open up, allowing the unbonded tendons to elongate. Based on the global displacement limits on this bridge, a maximum elongation of the plastic hinge of only 1.375” can be accommodated. Any further elongation leads to stability concerns for the system, based on the rotational capacity of the pinned connection at the base of the cantilever beam and the beam’s horizontal displacement interaction with adjacent beams.

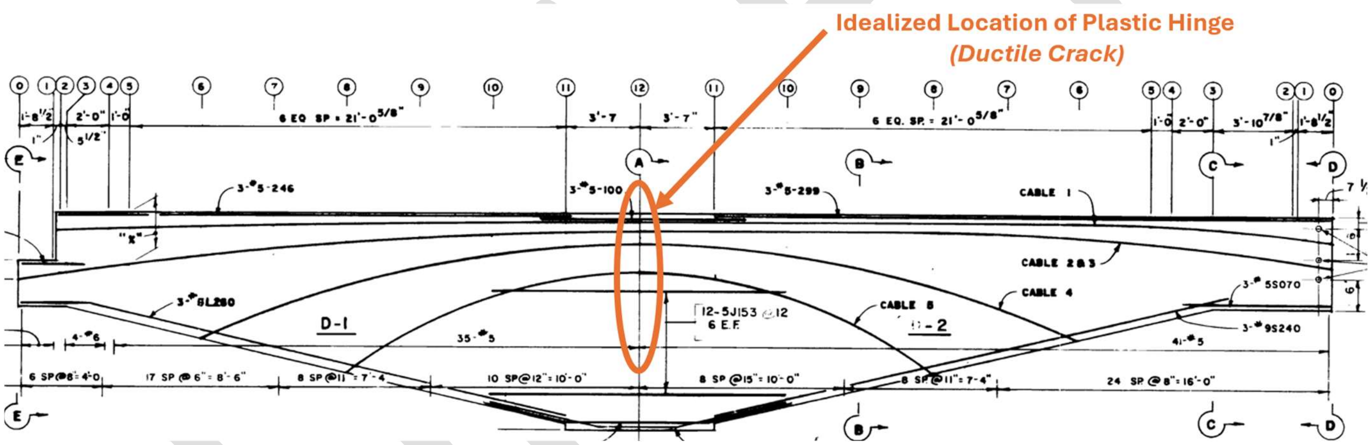

Figure 24: Idealized Location of Plastic Hinge for Type D Cantilevers

To develop the full unbonded strength capacity of the Type D cantilever beam, an elongation at a minimum of 3.1” would be required per equations from AASHTO Section 5.6.3.1.2. This length of elongation cannot be accommodated based on the global displacement and rotational limits of the bridge. Therefore, a maximum elongation limit of 1.375” was used in the determination of the upper bound moment capacity for the beam. With this assumption, the nominal moment capacity of the Type D cantilever beam was estimated to be 13,150 kip-ft.

The formulation of the required plastic hinge and the ability for the tendons to elongate to the geometric 1.375” limit requires the beam to be ductile. AASHTO has limits placed on post-tensioned concrete beams to ensure their ductility including a minimum reinforcement check. This check compares the cracking moment capacity of the beam to the factored strength resistance of the beam, AASHTO Section 5.6.3.3. The cracking moment capacity for the representative Type D cantilever beam was determined to be 12,500 kip-ft. The Type-D cantilever beams violate the minimum reinforcement check within the AASHTO, as the factored strength capacity is determined to be lower than the cracking moment of the beam, as shown below.

- M_n: 13,152 kip-ft *(Nominal Flexural Strength Capacity of Beam)*
- ϕ: 0.9 *(Flexural Strength Resistance Factor for Concrete with Unbonded Tendons)*
- ϕ_d: 0.95 *(Non-Ductile Resistance Factor – AASHTO Section 1.3.3)*
- $M_r = \phi\ \phi_d\ M_n$ = 11,245 kip-ft *(Factored Flexural Strength Resistance of Beam)*
- M_{cr}: 12,506 kip-ft *(Cracking Moment Capacity)*

M_r = 11,245 kip-ft < M_{cr} = 12,506 kip-ft - Criteria Not Meet

Additionally, detailing requirements for bonded reinforcement to be positioned along the web of deep beams is violated, per AASHTO Section 5.6.7. This bonded reinforcement detailing is intended to mitigate crack growth and helps to maintain the integrity of a beam once a crack is formed.

The factored flexural resistance of the beam has been found to be lower than the cracking moment capacity of the beam. The detailing of the beam does not provide adequate crack control reinforcement. Therefore, the ability for the plastic hinge to develop and for the beam to maintain its integrity while deforming to allow the elongation of the tendons to 1.375" is brought into question.

Bridges are designed to be ductile per current codes and design methodologies. This means that as a beam approaches its theoretical failure capacity it should have undergone significant deformation. For concrete beams, this deformation would include the formation of flexural cracks, under loads that should be significantly lower than the beams capacity. The goal of this is to allow the cracking to be seen prior to any potential failure. However, when the factored flexural resistance of a beam is close to or lower than the cracking moment, like it has been determined here for the Washington Bridge, this indicates that little to no cracking would be anticipated prior to a beam's failure. For the Washington Bridge, this means little to no warning should be anticipated prior to a potential failure.

Therefore, an upper and lower bound approach to the estimation of the beam's strength capacity is recommended. The upper bound limit is estimated as the Factored Flexural Strength Resistance assuming its ability to form the required plastic hinge. The lower bound limit is taken as the Factored Cracking Moment Capacity.

It is important to note that for both the upper bound and lower bound capacity estimations, the ability for the beam to undergo significant flexural cracking prior to failure is not anticipated by the analysis. Both failure modes should be considered as non-ductile. Therefore, limited advanced warning via the formation of flexural cracks prior to the potential failure of the beam could occur.

Section 6.5 Strength Load-Carrying Capacity of Type D Cantilever Beams

Taking the upper and lower bound capacity estimates of the Type D Cantilever beams, a load rating analysis was performed per the RIDOT Load Rating Manual and the AASHTO Manual for Bridge Evaluation 3rd Edition (MBE). This analysis was performed for the suite of RIDOT design, legal, and permit vehicles to provide a more complete assessment of the Type D Cantilever Beam's strength load rating capacity. As can be seen from the summary table below, deficient load ratings were found for the Type D Cantilever beams in their inspected condition when incorporating field-testing data available.

Table 2: LRFR Strength I Load Rating Summary for Type D Cantilever Beam

Truck	Upper Bound Rating	Lower Bound Rating
HL-93 (Inv)	0.50	0.42
HL-93 (Opr)	0.65	0.55
H20	1.64	1.38
Type-3	1.35	1.14
Type 3S2	1.04	0.88
Type 3-3	1.02	0.86
SU4	1.23	1.04
SU5	1.09	0.92
SU6	0.98	0.83
SU7	0.88	0.74
RI-3	0.87	0.74
RI-4	0.88	0.75
RI-5	0.68	0.58
RI-5B	0.56	0.48
RI-6	0.65	0.55
RI-OP1	0.67	0.57
RI-OP2	0.68	0.58
RI-OP3	0.62	0.53
RI-OP4	0.49	0.41
RI-OP5	0.76	0.65
EV2	1.16	0.99
EV3	0.78	0.66
RIPTA BUS	1.69	1.43
RI Legal Lane	1.12	0.95

It is important to note that per the 2018 Load Rating Report, the Type D Cantilever Beams were found to have the highest load ratings when compared to the Type A, Type A1, Type B, and Type C cantilever beams. If similar detailed load rating analysis of the Type A, Type A1, Type B, and Type C cantilever beams was performed, it is anticipated they would have significantly lower capacity than reported in the 2018 report and be found to be more critically deficient than the Type D cantilever beam as presented in this analysis.

Section 6.6 Tie-Down Failure Analysis of Type D Cantilever Beams

Following the identification of the failure of several tie-downs, an isolated beam line analysis was performed to assist in the development of the tie-down repair. The purpose of this analysis was to provide order of magnitude forces to design the tie-down repair. The tie-down failure at pier 6 and 7 is considered an extreme event condition for the structure, that results in large displacements, rotations, and transverse twisting of the structure. Beam ends where tie-down failures have occurred have documented uplift in the order of 0.375” to 0.5”. Additionally, Beam A at Pier 6 has been documented to have rotated off its pintle and is bearing against the pier seat, see picture below.

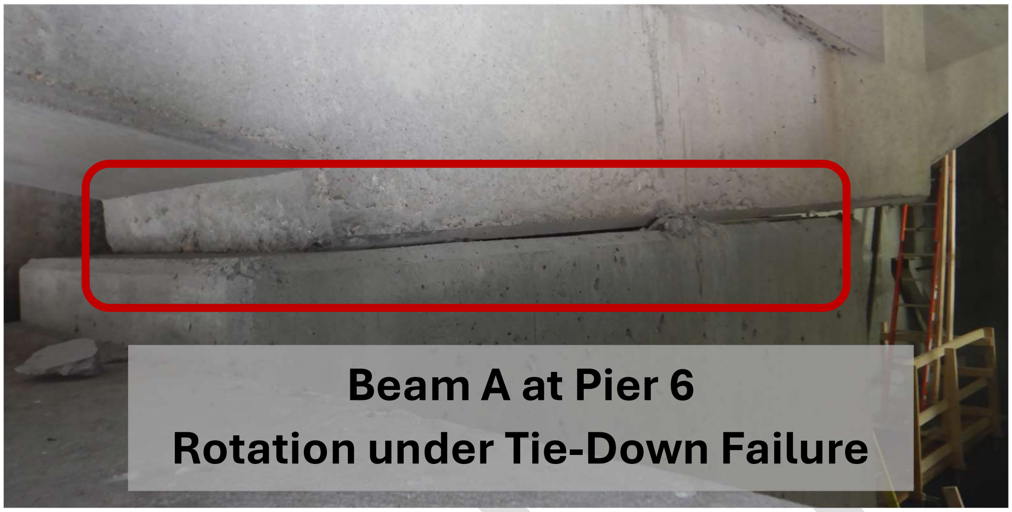

Figure 25: Rotation of Beam A at Pier 6 due to Tie-Down Failure

The isolated beam line analysis performed takes a conservative upper bound modeling approach to this extreme event. The simulated tie-down failure in the analysis results in an uplift at the ends of the type D cantilever of 0.73", which is greater than the field observations, but comparable. See the below image for an exaggerated vertical scale image of the deflected shape of the structure after tie-down failure has occurred. The isolated beam line analytical model does not consider the transverse stiffness of the deck, adjacent beam lines, and diaphragms, which would result in a stiffer resistance to the tie-down failure and a lower uplift of the beam end.

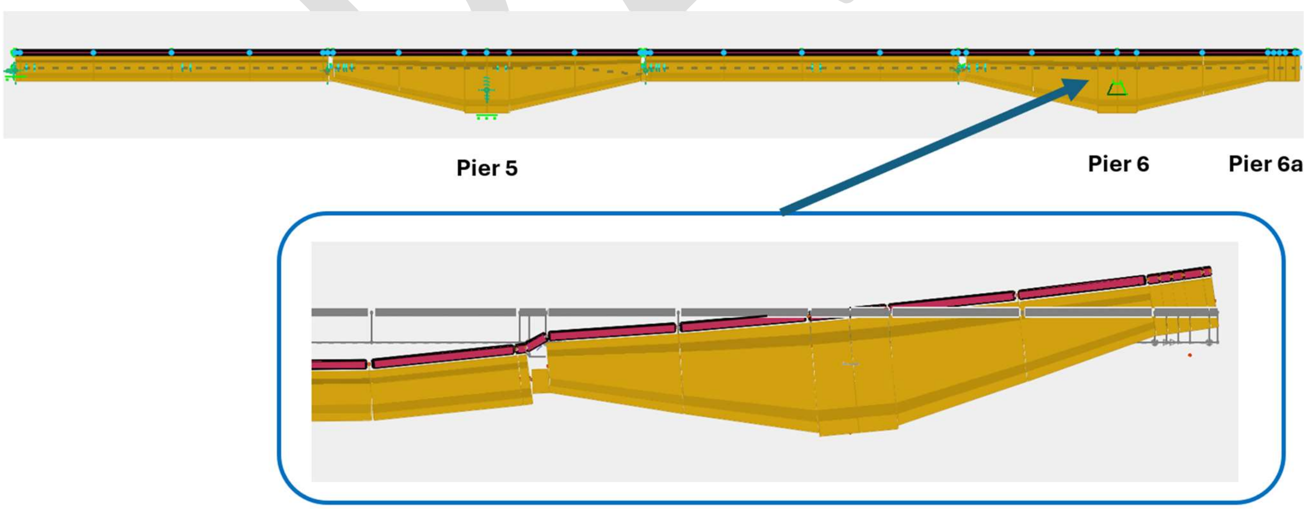

Figure 26: LARSA 4D Model Deflected Shape Under Tie-Down Failure Extreme Event

Once the tie-down rods have failed, stability of the model is maintained via the rotation of the cantilever beam resulting in compression engagement of the link slab between the cantilever beam and the adjacent drop in span and the bearing connection between these beams. Field observations indicate similar rotation and compression engagement of the link slab, along with a twisting of the typical section that is contributing to the resistance to the tie-down failures via the transverse stiffness of the bridge. Significant deadload redistribution has occurred due to the failure of the tie-down rods as is evident from the field documented rotations, displacements and analytical modeling performed to date. Quantifying specifically where this deadload has shifted is a challenging analytical problem. Even with a complex 3D analysis, uncertainty will remain on the magnitude and location of this deadload redistribution. This will be a permanent risk that will need to be accounted for in the maintenance, load rating, bridge posting, and asset management for this structure through the remainder of its service life.

Moments, forces, and stresses taken directly out of this isolated beam line tie-down failure model should be considered an upper bound assessment of the condition of the beam and are conservative. The global behavior of the model is useful, however, in identifying potential areas of concern for more detailed analysis and field observation. For instance, the analysis indicated the potential of tension stress to develop in the cantilever beam near the drop-in span connection. This was flagged as a location of potential distress in the beam and bridge due to the tie-down failure, see figure below for relative position in question.

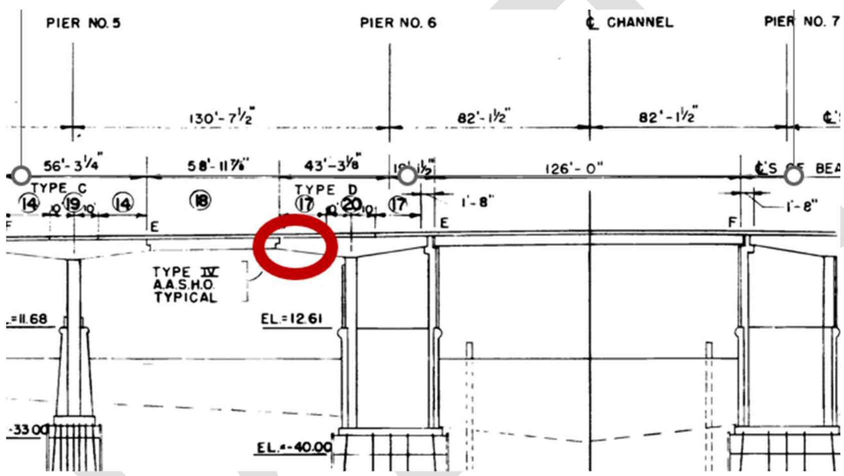

Figure 27: Location of Potential Distress in Cantilever Beam from Analysis

On January 17th, 2024, after reviewing these modeling results VN inspected the condition of Beam A at this suspected location of distress and documented the following. The 1996 seismic retrofit appeared to be separating from the beam end, with a 3/32" crack forming at the interface between the two elements. Unidirectional cracking was observed along the face of the 1996 retrofit, indicative of structural distress. Web cracking along the beam was observed along with potential locations of delamination. See the image below from this field investigation. This area was found to be in distress as indicated by the model.

Figure 28: Inspection Photo of Distress found at End of Beam A near Pier 6 in Span 6

Other locations of this 1996 retrofit were inspected to see if similar patterns of distress were present. While evidence of the 1996 retrofit separating from the cantilever beam is observed throughout the inspected areas of the bridge, the crack pattern on the face of the retrofit seen at Beam A near Pier 6 is unique to locations where tie-down failure has occurred.

Section 7. Qualitative Review of Repair and Strengthening Options

Several key deficiencies have been identified that should be addressed in any potential repair and strengthening alternatives considered for the Washington Bridge. These include the following:

1. The Type D Cantilever Beams are documented to be overstressed for Services III (i.e. tension stresses are present in the beam). *Type A, Type A1, Type B, and Type C Cantilever Beams should be considered to be similarly overstressed, given their relative ratings to the Type D Cantilever Beam per the 2018 Load Rating Report*.
2. The Type D Cantilever Beams are documented to be deficient under Strength I. *Type A, Type A1, Type B, and Type C Cantilever Beams should be considered structurally deficient, given their relative ratings to the Type D Cantilever Beam per the 2018 Load Rating Report*.
3. Water infiltration to PT Anchorage Systems at joint locations along the full length of the bridge.
4. Ongoing corrosion at the anchorages and along the tendons of the ducts of the PT systems.
5. Deficiencies of the grout within the PT ducts.

Section 7.1 Closing Joints along the Length of the Bridge

Closing the joints along the full length of the bridge would significantly mitigate the ability for water infiltration to continue at the PT anchorage locations. Water infiltration is believed to be a key catalyst to the deterioration documented at these anchorage locations due to the concrete being identified as non-air-entrained, per testing. However, the ability to do this would require making either the deck fully continuous along the bridge length or making the deck and beams fully continuous along the bridge length. It is not possible, given the configuration of the cantilever beams and drop-in beams, to make the beams fully continuous. If the deck is made fully continuous, via link slabs or similar detailing, significant changes in the thermal behavior and longitudinal forces that develop along the bridge occur. This would increase the longitudinal forces developed within the superstructure and applied to the substructure. Based on the limited substructure analysis performed to date, increasing the loading on the existing substructure is not advisable.

Closing the joints along the full length of the bridge alone would not be sufficient to repair the structure. Beam repair and strengthening would also be required.

Section 7.2 Repair of Post-Tensioning System and Beams

The repair of the post-tensioning system for the Cantilever Beam would require a system of repairs to include filling of voids in the ducts, grout remediation, tendon impregnation to inhibit future corrosion, the sealing of the anchorage systems, and concrete repairs. Critical points of consideration for such a repair include:

- The repairs along each individual PT duct would be custom and would be dependent on the condition of the grout present in the duct. Areas of soft grout would need to be removed or would require a potential attempt to be made to rehydrate and cure the existing cementitious systems. Voids within the ducts would need to be identified and access to grout them would need to be accommodated.
- Grouting of voids within the ducts is also not without risk. The operation itself is destructive and grout discontinuities between new and old grout can contribute to the formation of macrocell corrosion of the tendons.
- Access to end anchorage locations to clean, re-grout, and seal is not available without the partial demolition of the structure. The drop-in spans block access to this critical end anchorage location at the

supporting ends of the cantilever beams (this detailing is present throughout the length of the bridge). To thoroughly repair these locations would require the removal of the drop-in spans. This is a complex operation that would require partial demolition of structure and then reconstruction of the system.

- The adequacy of this repair approach would be dependent on the ability to identify and repair all locations where these deficiencies are present. This is a significant risk. Not all locations along the PT beams are accessible for testing or for repairs. Refer to the section in this report titled Risks Associated with the PT System, for additional details.

Addressing the grout and anchorage deficiencies within the post-tensioning system of the cantilever beams would mitigate the risk for future deterioration of this critical system within the bridge. However, the capacity deficiencies of the beams would not be addressed by these repairs. The system would still need to be considered unbonded or partially bonded, from a risk perspective. Stress deficiencies due to the effective stressing of the PT system would remain. Therefore, additional strengthening of the beams or system would therefore be required along with these repairs to the post-tensioning system.

Section 7.3 Strengthen Existing Beams – External Post Tensioning Systems

The strengthening of the existing beams with a new external post-tensioning system was a considered option. In such a repair, new post-tensioned PT rods would be installed on either side of the existing cantilever beams to strengthen the system. Such a repair concept does not remove the existing deteriorated PT system within the cantilever beams, but supplements its functionality. Critical points of consideration for such a repair include:

- When the external post-tensioning system is stressed, the existing internal post-tensioning system would be de-stressed. This would require the new system to be designed to take the full design loading of the superstructure. This significant external PT demand would need to be balanced to not overstress the beams in compression. The ability for the existing concrete beams to take such additional compressive force is in question due to unsound concrete documented in the beam ends.
- The existing deck would need to be removed to minimize the locked in deadload stress in the existing cantilever beams prior to stressing of the new system. Then a new deck could be installed.
- External post-tensioning strengthening would not be feasible in all locations along the bridge due to conflicts with substructure elements at the west abutment and along the facade girders.
- The external post-tensioning system would be challenging to anchor to the existing cantilever beams, with the ideal placement of the new anchors conflicting with the concrete installed as part of the 1996 seismic retrofits.
- The anchorage for an external post-tensioning system would need to be anchored at the existing cantilever beam ends, which are documented to contain unsound concrete. The ability to develop adequate compression force to develop the new post-tensioning system is a risk for such a repair concept.
- The anchorages for the new external PT system would be located under deck joints leading to the potential for long term maintenance, repair, and inspection concerns.
- The 60-year-old existing cantilever concrete beams are a critical element for the feasibility of such a strengthening scheme. Additional material testing would be required to confirm the viability of such a system and to estimate the service life extension such a repair would provide.

The feasibility and viability of strengthening the existing cantilever beams with an exterior post-tensioning system given these points is considered low.

Section 7.4 Strengthen Existing Beams – Fiber Reinforced Polymer (FRP)

The strengthening of the existing beams with FRP was a considered option. In such a repair FRP sheets are epoxied to the sides of the existing concrete beams near their tension face. The FRP then shares loading, applied after its installation, with the existing cantilever beams. Critical points of consideration for such a repair include:

- The existing deck would need to be removed to minimize the locked in dead load stress in the existing cantilever beams prior to the application of the FRP. Then a new deck could be installed.
- The existing cantilever beams and their PT systems would remain a primary load carrying element in the bridge.
- The FPR would further limit the ability to inspect the existing cantilever beams in the critical tension zone where the plastic hinge would be anticipated to form under a strength failure condition.
- The ability to apply sufficient FRP to the sides of the beam to provide the required service and strength capacity is in question given the geometry of the cantilever beams.
- The ability to anchor the FRP sufficiently is in question given the geometry of the cantilever beams.

The feasibility and viability of strengthening the existing cantilever beams with FRP is considered low given these points.

Section 7.5 Supplementing the Existing Beams – Sister Beam Alternative

An alternative way of providing added capacity to a bridge is to supplement the existing beams with new beams installed between the existing beam lines. Critical points of consideration for such a system include:

- The new sister beams would need to be installed within the existing framing plan without de-stabilizing the existing beams. This would require removal of the existing diaphragms and poses a stability risk, especially near piers 6 and 7 where significant dead load has been redistributed due to the loss of multiple tie-down rods.
- Load would need to be jacked into the sister beams to allow for sufficient load sharing between the new and existing beam lines.
- The existing cantilever beams and their PT systems would remain a primary load carrying element in the bridge.
- The addition of the sister beam lines would require substructure modifications and increased deadload applied to the existing foundations. Based on the limited substructure analysis performed to date, increasing the loading on the existing substructure is not advisable.

Supplementing the existing beams with new sister beams is not considered a viable option given these points.

Section 7.6 Additional Repair and Strengthening Considerations

Additional repair and strengthening schemes were considered as the assessment of the Type D cantilever beams progressed. These schemes included evaluating isolated beam or beam line replacement. Given the balanced and unbalanced cantilever configuration of the superstructure, replacing isolated elements in the system is very challenging and would require the partial deconstruction of adjacent spans and potentially the use of significant counterweights and temporary shoring to maintain the stability of the bridge. This is further complicated by the redistribution of the deadload that has occurred due to the lost tie-downs at Piers 6 and 7.

To thoroughly repair and strengthen the Washington Bridge would require a combination of the above or similar repair and strengthening concepts. None of the repair and strengthening schemes noted, however, addresses water infiltration at deck joint locations. Water infiltration at these locations directly over the PT anchorage assembles would remain a long-term risk for the structure. The repair and strengthening concepts do not eliminate the tie-down detailing at the West Abutment, Pier 6 and Pier 7 locations. The schemes do not improve the inspection, maintenance, and repair access along the structure, but further complicate it. Additionally, these schemes do not allow for inspection, maintenance, and repair access to the west abutment tie-downs or beam ends with their post-tensioned anchorage systems, which are currently inaccessible.

To fully eliminate these risks and improve inspection, maintenance, and repair access would require a full superstructure replacement and potentially a full bridge replacement. The reuse of the substructure for a superstructure replacement would need to be further investigated to confirm the feasibility and viability of the substructure for reuse. Initial investigations into the feasibility of the substructure for reuse, from a condition and strength perspective, are on-going and being performed by others.

Section 8. Risks Identified on the Washington Bridge

Section 8.1 Potential for Non-Ductile Failure Mode

The isolated beam line analysis and sectional capacity analysis performed for the Type D Cantilever beams at Piers 6 and 7 indicate insufficient capacity of the beams to support live load traffic. The sectional capacity of the beam was found to be comparable to the beam's cracking moment. Therefore, the failure mode of the Type D Cantilever beams is anticipated to be non-ductile. In a non-ductile failure, limited to no flexural cracking could present before a potential failure occurs. This poses significant risk from an asset management perspective as the bridge may not present adequate structural distress via cracking or displacement prior to failure for routine inspections to detect. To further clarify, this means that under sufficient load the bridge could fail with limited to no advanced warning. Given the non-redundant, balanced, and unbalanced nature of the bridge's cantilever beams, the failure of a single beam may not be limited to just that element but could lead to the loss of an entire beam line or could lead to a progressive collapse of multiple beam lines.

Section 8.2 National Perspective

The Washington Bridge is a complex structure given its balanced and unbalanced cantilever configuration coupled with the drop-in spans. The PT system within the cantilever beams is a critical element for the structure and has been documented to have significant deficiencies. These deficiencies include:

- Exposed PT anchorage assemblies undergoing active corrosion
- Exposed PT grout ports undergoing active corrosion with voids and soft grout present
- Significant voids within the PT ducts
- Soft grout within the PT ducts
- Suspected delamination of the grout within the PT ducts
- Corrosion of PT tendons within the PT ducts
- Unsound concrete in the anchorage development zone of the concrete beams
- Web cracking along the PT ducts at beams throughout the structure

These individual deficiencies are not uncommon for post-tensioned infrastructure constructed in the late 1960s. These issues are further complicated for the Washington Bridge due its non-redundancy in global configuration due to the balanced and unbalanced nature of the bridge, and the cantilever beams themselves not having internal PT redundancy or access to allow for the replacement or maintenance of the PT elements.

The National Cooperative Highway Research Program (NCHRP) has published a report on the "Repair and Maintenance of Post-Tensioned Concrete Bridges". This report presents survey findings from state agencies on their experience with repairing and maintaining post-tensioned concrete bridges and presents several case studies for structures that have presented with significant issues relating to their post-tensioning system and how they were addressed or are being managed.

The Washington Bridge is unique in this national context given its non-redundant nature and the number of PT deficiencies documented. For example, the Plymouth Avenue Bridge, in Minneapolis Minnesota, had similar exposed PT anchorage assemblies with surface corrosion, however the grout at the anchorage assembles was found to fill the anchor assembly, without the voids and soft grout encountered on the Washington Bridge. The Varina-Enon Bridge, in Richmond Virginia, was found to have similar grout issues to the Washington Bridge with voids and soft grout within the ducts and even broken tendons within isolated ducts. However, the Varina-Enon

Bridge has sufficient internal redundancy to allow for the de-tensioning and replacement of multiple broken and deteriorated tendons. The Washington Bridge does not have such internal redundancy or sufficient access to anchorage assemblies to allow for such repairs.

Section 8.3 Limited Inspection Access

The geometry and detailing of the Washington Bridge does not allow for the complete assessment of PT systems. The anchorage assemblies for the Type A, Type A1, Type B, and Type C cantilever beam ends are not accessible due to the drop-in span's bearing locations. The concrete installed as part of the 1996 seismic retrofit obscures the ability to inspect the sides of the cantilever beams as well as the anchorage development region. The detailing of the West Abutment prevents the inspection of the PT tie-down rods and the end anchorages for the Type A cantilever beams.

Section 8.4 Deck Joints Located Over PT Anchorages

Concrete testing performed thus far indicates the concrete of the cantilever beams is non-air-entrained. This type of concrete is vulnerable to freeze-thaw damage. Therefore, in locations where water can infiltrate the concrete beams there is risk for freeze-thaw damage. Leaking deck joints are an active source for such water infiltration along the Washington Bridge. These deck joints are located at PT cantilever beam ends and at the critical location of the cantilever beams PT anchorage assemblies, thus allowing for continued deterioration of these beam ends through water infiltration and freeze-thaw cycling.

The continued deterioration of the beam ends around the anchorage assemblies carries risk for the loss of a PT cable through corrosion of the steel elements of the anchorage assemble, the crushing of unsound concrete behind the PT anchorage, and the loss of PT tendons through corrosion. The loss of a single PT cable for a Type D cantilever beam will reduce a beam's structural capacity by approximately 20% and brings into question the ability for the beam to support dead load alone. The risk of losing a PT cable increases as deterioration is allowed to continue at these locations. The likely hood of this type of failure occurring is difficult to quantify as visual inspection of the tendons within the anchorage assembly is not possible. However, the systems to protect against this type of deterioration have been compromised or failed, the concrete around the anchorage and the grout within the PT ducts. The concrete around the anchorage is non-air-entrained, unsound, cracked, and delaminating. The grout within the PT ducts has voids, soft grout, and shows evidence of active corrosion occurring. The risk of losing a PT cable will only increase with time. The ability to mitigate this risk is limited due to the joint configuration of the bridge and placement of the PT anchorages. The deck joints cannot not be fully closed along the bridge and PT anchorages cannot be moved. The risk associated with the loss of a PT anchor cannot be fully addressed through repairs or retrofitting of the structure.

Section 8.5 Ability to Fully Document PT Deficiency

The advanced scanning technologies employed for the assessment of the Washington Bridge have been able to identify locations of deficiency in PT ducts. This includes deficiencies such as the 14ft plus void at Beam B at Pier 6. During the 1996 bridge repairs, impacted echo was used to determine locations of potential voids in the ducts for repair and regrouting. However, the 1996 impacted echo testing did not find the 14ft void found using today's technology, see the below picture for reference.

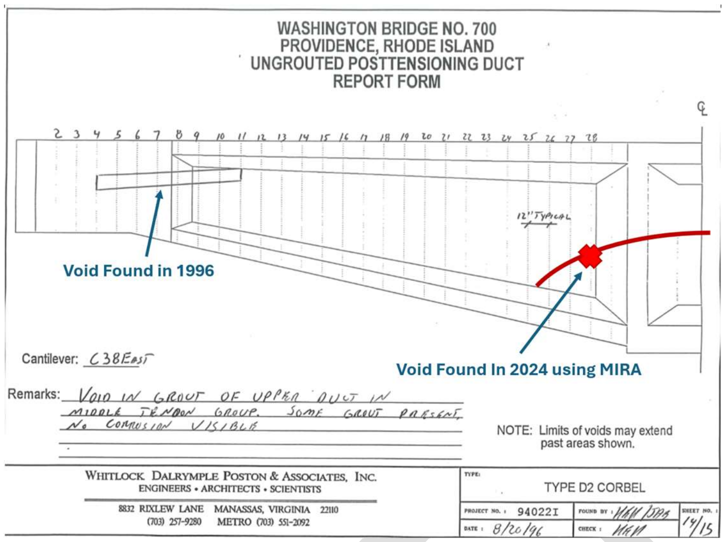

Figure 29: Locations of Voids found from Impact Echo (in 1996) and using MIRA (in 2024) for Beam B at Pier 6

Even using the advanced scanning technologies we have today, risk will remain that not all deficiencies can be identified for repairs or even risk assessment. The geometry of the Washington Bridge cantilever beams does not allow for the scanning of the duct's full length. Additionally, at locations where cracks have already formed on the web of the beam, MIRA technology cannot be used to thoroughly assess the condition of the ducts. Therefore, the risk of unknown deficiencies will always remain for this structure. See the below image identifying locations where MIRA cannot be used (shaded in red), compared to accessible locations due to geometry (yellow).

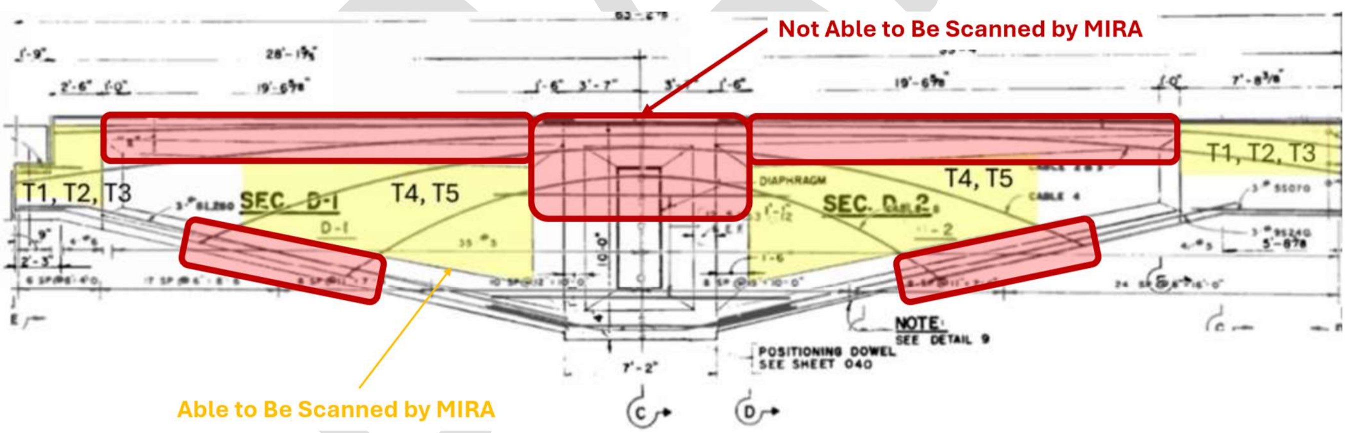

Figure 30: Locations where Scanning Using MIRA can and cannot be preformed

Section 8.6 Potential for Progressive Failure

The Washington Bridge is inherently non-redundant in global configuration due to the balanced and unbalanced nature of the bridge's cantilever beams, and the associated adjacent drop-in spans providing counterbalance weight for the system. If a cantilever beam were to fail, the adjacent drop-in span would lose its support and fall. The falling of this drop-in span would unbalance the next adjacent cantilever beam, which would then be at risk of failing or becoming unstable. This could lead to a progressive failure scenario for the entire beam line, or even risk the loss of adjacent beam lines as the bridge attempts to re-stabilize transversely. The risk of this potential progressive failure is compounded by the redistribution of the load associated with the loss of the tie-down rods at Piers 6 and 7.

PT rods that anchor the cantilever Type A1, Type B, and Type C beams to their respective piers do mitigate this risk to an extent. However, PT rods can be vulnerable to significant dynamic instantaneous loading which can lead to fracture at loads under their ultimate capacity when highly loaded. A potential progressive failure as noted above would be such a dynamic instantaneous loading event.

The potential for such a progressive failure to occur is difficult to quantify. Based on the analysis performed on the Pier 6 and 7 cantilever beams, limited risk is documented under dead load conditions alone with the tie-rods functioning as-designed. However, as is documented in this report, these beams do not have sufficient theoretical capacity to support significant live load. This assessment is valid for the currently documented condition of the beams analyzed. Continued deterioration of the system is on-going, and the conclusions of this assessment could change as this deterioration progresses. Additionally, the Type D cantilever beam failure mode is anticipated to be non-ductile in nature, with limited to no advance warning or early flexural cracking.

The risk of a progressive collapse of the bridge is believed to be low in a deadload only configuration of the structure, with no traffic on the bridge and the tie-down rods functioning as-designed. This risk is reduced once the installation of the new tie-down rod system is implemented. If it did occur, the following effects of such a failure should be considered in a risk assessment of the structure's long-term viability:

- The potential for elements of the superstructure to fall onto Gano Street.
    - *After receiving preliminary findings covered in this report, and in an abundance of caution, RIDOT has directed the contractor to install temporary support beneath the spans of the Washington Bridge that pose a risk to Gano Street, as a mitigation strategy.*
    - *It is important to note that the cantilever beams at the Gano Street crossing are undergoing active monitoring with no documentation of movement in the bridges' current condition.*
- The potential for elements of the superstructure to fall into the river and damage the shared foundations of the I-195 East Bound Bridge.
    - *This has the potential to require lane or full bridge closure of the I-195 East Bridge if such a failure of the Washington Bridge did occur, as the foundations of the I-195 East Bound Bridge would need to be assessed and potentially repaired if struck by falling elements of the Washington Bridge.*
    - *It is important to note that cantilever beams along the full bridge are undergoing active monitoring. Movement is observed along the cantilever beams located at Pier 6 and 7, but within limits associated with the stable condition of the structure.*
- The potential for elements of the superstructure to fall into river impacting vessels below.

Section 9. Summary and Conclusions

The Washington Bridge is a complex structure composed of 18 spans of varying structural types including PT cantilever spans, drop-in prestressed girder spans, simply supported steel beams, and simply supported prestressed girder spans. The bridge is inherently non-redundant in global configuration due to the balanced and unbalanced nature of the bridge. The cantilever beams themselves do not have internal PT redundancy or adequate access to allow for the inspection, maintenance, or replacement of individual PT elements. The field inspections and field testing performed on the structure to date have documented the following deficiencies:

- Failure of multiple Tie-Down Rods at Pier 6 and 7
- Exposed PT anchorage assemblies, undergoing active corrosion
- Exposed PT grout ports, undergoing active corrosion, with voids and soft grout present
- Significant voids within the PT ducts
- Soft grout within the PT ducts
- Suspected delamination of the grout within the PT ducts
- Corrosion of PT tendons within the PT ducts
- Unsound concrete in the anchorage development zone of the concrete beams
- Web cracking along the PT ducts at beams throughout the structure
- Concrete of beams at Pier 6 and 7 vulnerable to freeze-thaw damage
- Deck joints leaking above PT anchorage assemblies throughout the bridge

The Concrete and Grout supporting the PT System is Compromised and has Failed in Locations

The PT system within the Type D cantilever beams are critical to the beam's load carrying capacity and therefore the stability of the bridge. The type of deterioration documented indicates that the protective concrete around PT anchorages and grout within the PT system is compromised and has failed in multiple locations. The concrete around the anchorage is non-air-entrained, unsound, cracked, and delaminating. The grout within the PT ducts has significant voids, large segments of soft grout, and shows evidence of active corrosion occurring.

Type D Cantilever Beams Present with Risk of Failure Without Warning Under Live Load

The isolated beam line analysis and sectional capacity analysis performed for the Type D Cantilever beams at Piers 6 and 7 indicate insufficient capacity of the beams to support live load traffic without significant repairs and strengthening of the system, the viability of which is limited. The failure mode of the Type D Cantilever beams is anticipated to be non-ductile and therefore would present with limited or no flexural cracking as a warning sign before a potential failure occurs. This poses significant risk from a safety and an asset management perspective as the bridge may not present adequate structural distress prior to failure for routine inspection to detect.

Available Repair Options do not Fully Mitigate Risks Associated with Structural Deficiencies

Comparing the capacity findings of this assessment to the 2018 Load Rating of the additional cantilever types indicates that the structural deficiencies documented are anticipated to be found throughout the bridge. It is anticipated the Type A, A1, B, and C cantilevers will have significantly lower capacity than reported in the 2018 report and be found to be more critically deficient than the Type D cantilever beam as presented in this analysis. Repairs and strengthening of the bridge would be a complex operation that carries risk with regards to viability and sufficiency to fully address the deficiencies within the system. The ability to thoroughly identify and access all areas of the PT systems that require repair carries risk. This is due to the inaccessibility of locations on the structure and the geometry of the PT beams themselves. The viability of repairs requires the removal and replacement of the existing deck to lower locked-in load within the PT beams. Removing the existing deck carries risk due to the significant load redistribution that has occurred with the loss of the tie-downs at piers 6 and 7. The ability to reuse the existing beams for anchoring of external strengthening is a risk due to the unsound concrete present at the beam ends. The repair and strengthening options for the Washington Bridge are limited, complex, and do not completely mitigate the identified risks with the structure.

Deck Joints over PT Anchorages Pose a Continual Risk to the Integrity of the Bridge

Deck joints along the bridge are located at PT cantilever beam ends where PT anchorage assemblies are present. Water infiltration through the deck joints is an ongoing concern as it allows for the continued deterioration of this critical location. The systems to protect against this type of deterioration at the PT anchorage assemblies are documented to be compromised and in locations have failed, specifically the concrete around the anchorage and grout within the ducts of the PT system. The risk of this deterioration leading to the potential loss of a PT anchorage and cable will only increase with time. The risk associated with the loss of a PT anchor cannot be fully addressed through repairs or retrofitting of the structure.

Risk of Progressive Collapse and Immediate Stability Measures to be Taken

The risk of a progressive collapse of the bridge is believed to be low in a dead load only configuration of the structure, with no traffic on the bridge and the tie-down rods functioning as designed. Multiple tie-down rods have failed at piers 6 and 7 which increases the risk of instability for the structure. This risk of a progressive collapse can be mitigated by the installation of the new tie-down rod system, to repair the failed tie-downs located at piers 6 and 7. However, this risk increases with time as deterioration of the system continues. The ability to assess this continued deterioration by visual inspections is limited due to it occurring within the anchorage assemblies and within the PT ducts of the beams.

The Repair and Strengthening Required has Limited Viability and Carries Risk

The Washington Bridge is unique from a national perspective and a bridge inventory perspective for Rhode Island. The structure is unique given its inherently non-redundant systems combined with the magnitude and types of deficiencies documented, based on the experience of the writers and review of nationally documented studies. The bridge poses significant challenges for its inspection, maintenance, and repair due to its as-designed initial detailing and construction. The repairs and strengthening required to address the documented deterioration and deficiencies along the structure are significant, have limited viability, and have risk associated with them. To fully eliminate these risks and improve inspection, maintenance, and repair access would require the demolition and replacement of the superstructure, and potentially a full bridge replacement. From an asset management perspective, consideration should be given to these, and other topics covered in this report in the decision-making process to determine the immediate next steps and long-term viability and sufficiency of the structure.

Appendixes

- Appendix A – Washington Bridge Shop Drawing, Stressing Calculations for Cantilever Beams
- Appendix B – VN Field Notes and Photos
- Appendix C – BDI Washington Bridge Sensor Locations, dated February 15, 2024
- Appendix D – BDI Washington Bridge Emergency Report, Version 2, dated February 21st, 2024
- Appendix E – VN Supporting Calculations