STATE OF VERMONT
AGENCY OF TRANSPORTATION

Scoping Report

FOR

Windsor Bridge 55

US 5 over Mill Brook

April 17, 2026



I. Site Information 4
Need 4
Traffic 5
Design Criteria 5
Inspection Report Summary 6
Hydraulics 6
Utilities 6
Right of Way 7
Resources 7
II. Safety 10
III. Maintenance of Traffic 10
Staged Construction Methods 10
Offsite Detour 11
Accelerated Bridge Construction Methods 11
IV. Alternatives Discussion 12
No Action 13
Alternative 1 – Deck Replacement 13
Alternative 2 – Superstructure Replacement 18
Alternative 2w – Superstructure Replacement with Widening 19
Corrosion Mitigation and Protection Measures 19
V. Cost Matrix 21
VI. Conclusion 22
Structure Recommendations 22
Traffic Control 23
VII. Appendixes 24
Appendix A: Site Pictures 25
Appendix B: 2023 Inspection Report 31
Appendix C: Windsor Town Map 50
Appendix D: Geotechnical Scoping Report Data Form 52
Appendix E: Resource ID Completion Memo 57
Appendix F: Archaeological Resource ID 59

Appendix G: Historic Resource ID 63
Appendix H: Natural Resource ID 93
Appendix I: Stormwater Resource ID 98
Appendix J: Landscape Resource ID 101
Appendix K: Environmental Specialist Memo 106
Appendix L: Preliminary Hydraulics Memo 109
Appendix M: Hazardous Waste Urban Soils Map 111
Appendix N: Floodplain Rivers Map 113
Appendix O: Utilities Memo 115
Appendix P: Traffic Memo – VTrans 118
Appendix Q: Community Questionnaire 121
Appendix R: WJE Materials Testing and Condition Assessment Report 127
Appendix S: Basis of Estimate 388
Appendix T: Basis of Schedule 403
Appendix U: Plans 412

I. Site Information

Bridge 55 is a single-span, open spandrel concrete arch bridge carrying US 5 over Mill Brook in Windsor, VT. The bridge carries two lanes of northbound/southbound traffic, and a single sidewalk. The sidewalk is located on the west, upstream, side of the bridge and continues off the bridge to the north and south. The bridge spans 90 feet, and is composed of two parallel arch ribs, spandrel columns rising from the arches, and 8 transverse floor beams supporting the bridge deck. Bridge 55 was constructed in 1930; an asphalt overlay was placed on the deck with other approach roadway work in 1943. The bridge has undergone no major reconstruction in its history.

Roadway Classification	Major Collector
Bridge Type	Open spandrel concrete arch
Bridge Length	90 feet
Year Built	1930, rehabilitation 1943
Ownership	Town of Windsor

The existing conditions are based on routine inspection reports, special inspection and material testing reports, and site visits. See correspondence in the Appendix for more detailed information.

Need

The Windsor bridge is listed in the Vermont State Register of Historic Places, holding significance as the only remaining open spandrel concrete arch bridge in Vermont; therefore, efforts to preserve the existing bridge are desired to keep it in service as long as possible. The Windsor bridge is currently categorized as being in overall fair to satisfactory condition. However, rehabilitation and repair are needed to preserve, to the extent possible, the original fabric and character defining features of the historic bridge. The following is a list of deficiencies of Bridge 55:

- Deterioration of the bridge rail, including cracking, efflorescence, spalling and broken or missing balusters.
- Widespread deterioration of the deck, including efflorescence and staining, delamination, spalling, exposed and rusted reinforcement on the deck underside and fascias.
- Deck joints over floor beams 3 and 6 allow leakage onto the floor beams and arches below
- Extensive deterioration of floor beams 3 and 6, including heavy spalling, exposed and corroded reinforcement and loss of transverse reinforcement.
- Isolated spalling on the arch ribs and the presence of corrosion causing chlorides in the concrete.
- Deterioration and section loss in the abutment and wingwall footings

See the Alternatives for additional discussion of repair needs.

Traffic

Projected bridge volumes for the opening (2029) and future (2049) design year are provided below.

AADT		DHV		%T		%D		ADTT		ESALs	
2029	2049	2029	2049	2029	2049	2029	2049	2029	2049	(2029~2049)	(2029~2069)
3259	3522	390	410	4.4%	6.2%	61%	61%	263	400	1155000	2630000

Design Criteria

The design standards for this project are the Vermont State Standards (VSS), dated October 22, 1997, A Policy on Geometric Design of Highways and Streets (Green Book), 7th Edition, and the VTrans Structures Design Manual, dated 2018. Minimum standards are based on the roadway classification, traffic volumes and a design speed of 25 mph.

Design Criteria	Source	Existing Condition	Minimum Standard	Comment
Approach Lane and Shoulder Widths	VSS Section 5.5	11'/1'+/-	10-11'/2' offset to curb	Village criteria applies, does not meet offset to curb
Bridge Lane and Shoulder Widths	VSS Section 5.5, Table 5.4	11'/1'+/-	10-11'/2' offset to curb (26' clear)	Village criteria applies, does not meet clear width and offset to curb*
Clear Zone Distance	VSS Table 5.5	No Issues Noted	1.5' (3' along radii)	Curbed Village Criteria applies
Banking	VSS Section 5.12, 5.13	Varies	2% (max)	Curbed Village Criteria Applies
Speed	VSS Section 5.3	25 mph (Posted)	25 mph (Design)	
Horizontal Alignment	AASHTO Table 3-10	Tangent Bridge	R_{min}= 134' @ 8%	
Vertical Grade	VSS Table 5.7	0.4% - 1.2%	9% (max) for level terrain	Crest curve over bridge
K Values for Vertical Curves	VSS Table 5.1	Kcrest = 114	20 crest / 30 sag @25 MPH	
Vertical Clearance	VTrans SDM 2.1.2.3	Over water, no issues noted	N/A	No changes to hydraulic opening anticipated
Stopping Sight Distance	VSS Table 5.1	1276'	150' @ Level	
Bicycle/Pedestrian Criteria	VSS Table 5.9	Existing sidewalk on upstream side	2' shoulder for cyclists	Existing shoulders are 1'
Bridge Railing (and Approach Railing)	Structures Design Manual Section 13.2	3-Rail Aluminum	TL-2	
Structural Capacity	Structures Design Manual Section 3.4.1	Structurally Sufficient (49 Tons Inventory Rating)	Design Live Load: HL-93	Existing load rating from the Inventory and Inspection Report, Draft revised load rating pending.

**Section 5.7 states that reconstruction of bridges on Collectors favors preservation within existing footprints, in order to ensure compatibility with the Vermont setting and to reduce costs and environmental impacts.*

Inspection Report Summary

Deck Rating	5 (Fair)
Superstructure	6 (Satisfactory)
Substructure	6 (Satisfactory)
Channel	8 (Very Good)

8/8/2023 – Structure is in okay condition having areas of progressing deterioration scattered throughout. Deck soffit has areas of saturated concrete with moderate to heavy efflorescence leakage, scattered small rust stains and areas of heavy scaling scattered throughout the fascia's. Section 6 between floor beams #5 and #6 has ~6'-0" x 6'-0" spall that has exposed the transverse steel reinforcing. Abutments have minor to moderate efflorescence leakage along the outer portions with typical small areas of light scaling surrounded by some very light honeycombing along the abutment stem. Structure will need rehabilitation project in near future to deter further deterioration with structure continuing to age. – S. P.

For details, see the 2023 Routine Inspection Report and the 2025 Material Testing and Condition Assessment Report, located in Appendix B and R, respectively.

Hydraulics

Bridge 55 is located within a FEMA Special Flood Hazard Area and is subject to inundation by the 100-year flood. Rehabilitation options will not result in permanent impacts to the low chord elevation or stream width between abutments. However, concrete repairs on the arch ribs, abutments, and wingwalls may have temporary impacts.

The bridge abutments are founded on rock and the bridge is not identified as scour critical.

Utilities

The existing utilities are shown in Appendix U and are as follows:

There is a 8" municipal watermain running through the bridge abutments but the section crossing the bridge is currently abandoned according to the Town of Windsor's water operator.

The current waterline line runs up Union Street. There are water valves to the west of the project site near Lowell Street.

The town of Windsor also has sewer infrastructure running up Union Street at the intersection.

There is a 6 conduit ductbank running through the bridge abutments. Consolidated Communications has three of these conduits in this ductbank. Other utilities that may be located in this duct bank require confirmation.

Velco has a substation to the north of this project site with transmission lines running aerially through the project area. These transmission line towers are mounted on piers within the river

bed upstream and downstream of the bridge. There is a transmission tower on the upstream side of the bridge which is within 2 ft of the bridge structure.

Green Mountain Power has several 3 phase lines running in this area crossing the bridge, and several aboveground and underground services in the area including a multiunit housing complex to the northwest of this project area.

Bridge rehabilitation will require demolition and removal of concrete elements such as the deck and floor beams. Active utilities currently supported under the bridge deck will require temporary relocation during construction. It is anticipated that the work may require large equipment, such as cranes, placed on the bridge approaches, necessitating temporary relocation of the overhead utilities crossing US 5 at the south approach and running parallel the bridge along the downstream edge of deck. While temporary utility relocations may be required, the assumption is that the work can be performed without requiring significant permanent impacts to, or relocation of, the transmission lines and towers.

Right of Way

The existing right-of-way is shown on the existing condition layout sheet in Appendix U. Bridge rehabilitation is not anticipated to require Right-of-Way acquisition.

Resources

The environmental resources present at the project location are shown in Appendix E-N.

Biological

Wetlands and Watercourses

No wetlands are located within the vicinity of the project.

Bridge 55 spans Mill Brook, which flows into the Connecticut River, located approximately 0.2 miles to the southeast.

Wildlife Habitat

The project location is primarily urban, with no significant wildlife habitat.

Rare, Threatened, and Endangered Species

The project site is within range of the federally endangered Northern long eared bat and the Dwarf Wedge mussel. There is potential habitat for Northern long eared bats under the bridge and a bat survey should be conducted prior to construction. Occurrence of the Dwarf Wedge mussel has been recorded in the nearby Connecticut River. A mussel survey should be conducted prior to construction.

Agricultural Soils

Prime agricultural soil has been recorded near the project site.

Hazardous Materials

There are multiple hazardous waste sites identified within the Windsor area, including a petroleum release site downstream from the bridge within the northeast quadrant. These sites are not expected to be impacted by the bridge rehabilitation.

The proposed project is located within a mapped Urban Background Soils area. Additional coordination with the Project Contamination Engineer is recommended. Disturbed soils within this project should be kept on site or follow current 'Notice to Bidders' guidance.

Floodplains

There are FEMA Floodplains mapped within the project area and a Flood Hazard Area/ River Corridor Permit may be required.

Historic

Seven historic resources listed on the Vermont State Register of Historic Places or the National Register of Historic Places, including Bridge 55, are located in the vicinity of the project site.

Windsor Village Historic District (Windsor-1)

The Windsor Village Historic District was originally listed in the National Register of Historic Places (NRHP) in 1975 with boundary expansions in 1997, and 2014 to incorporate additional historically relevant properties. The property inventory includes 238 contributing buildings, two structures, and three sites, significant under Criteria A and C for its role in early Vermont and the rise of precision industries and machine tool manufacturing. Bridge 55 is located directly along the southeastern boundary of the historic district; therefore, there may be some interruptions experienced during the design and construction phases of the bridge project. Replacement of the railing along the bridge and adjacent approaches will be selected with the intent to preserve the historical nature of the surrounding area.

Windsor Bridge No. 55, US Route 5 over Mill Brook, Windsor (Windsor-2)

Originally constructed in 1930, this concrete open spandrel arch bridge is listed in the SRHP and eligible for the NRHP. It is one of only four bridges of its type in Vermont and the sole example built by Gerald B. Owens, retaining high integrity of design, materials, and workmanship. Despite some deterioration, the bridge remains a well-preserved representation of early 20th-century engineering aesthetics. The proposed rehabilitation goals aim to extend the service life of the structure while preserving the historic features of the historic bridge.

American Precision Museum, 196 Main Street, Windsor (Windsor-3)

This National Historic Landmark was originally the Robbins & Lawrence Armory and Machine Shop, built in 1846. It played a pivotal role as the site where the improvement of interchangeable parts allowed for the increase of mass production. The building retains its original brick walls, slate roof, and multipaned windows, and its NHL boundary includes the adjacent Windsor Bridge

No. 55. Any work completed on the bridge wingwalls will preserve and protect the existing assets of the American Precision Museum.

NAMCO Block, 1-17 Union Street, Windsor (Windsor-4)

This SR/NRHP-listed property consists of nine four-story brick row houses with distinctive double-bow fronts, constructed between 1920 and 1922 with very minimal updates made to the original structure primarily in the form of minor window replacements. NAMCO Block is located at the northwest corner of Bridge 55. The replaced railing will be adjacent to the building but will be chosen as to minimally impact the historic nature of the area.

Industrial Sites on Mill Brook Historic District, Middle/Lower Dam Area, Mill Brook Falls, Windsor (Windsor-5)

The middle/lower dam area encompasses historic industrial sites around two dam locations on Mill Brook and Union Street, central to Windsor's early industrial development. It is listed in the Vermont State Register of Historic Places and eligible for the National Register of Historic Places. The area is located downstream from Bridge 55 and should not see any impacts from design and construction.

Site No. 14, Grist Mill Ruins, North Bank of Mill Brook, East of U.S. Route 5, Windsor (Windsor-5a)

Dating back to the late 18th century, the remains include the foundation and partial wall of a former gristmill where a wooden dam would've been used to power the site. The area includes multiple contributing resources that illustrate connections to Windsor's economic and industrial development. The Grist Mill Ruins are located downstream of Bridge 55.

Site No. 15, Foundry Wheel House Remains, South Bank of Mill Brook, East of U.S. Route 5, Windsor (Windsor-5b)

Situated on the south bank of Mill Brook, this mid-19th-century site contains remnants of a dry-laid fieldstone foundation, likely part of a substantial industrial facility. Although in poor condition, it remains significant for its connection to Windsor's industrial growth with ties to the American Precision Museum.

Landscape

Bridge 55 is located in an urban village setting with existing sidewalks, utilities, and work within a riparian area that constrain design. Key resources requiring protection include street trees at Main/Bridge and Main/Maple Streets, trees and garden areas at the Union Square apartment building, and the bridge's distinctive historic concrete railing. In accordance with the 2019 Windsor Town Plan, the project should preserve the bridge's historic character, maintain pedestrian and bicycle accessibility (including at least one sidewalk), minimize tree clearing and

riparian disturbance, implement tree protection and riparian planting where needed, and consider pedestrian-oriented design features that support lower vehicle speeds.

Archaeological

Bridge 55 is located in an area of historical significance. Archaeologically sensitive areas are located on the southeast and southwest embankments. Archaeological Phase I testing is recommended if construction activities are planned to take place in either quadrant.

Stormwater

Bridge 55 spans Mill Brook, which is on the 2016 Stressed Waters List in this area.

Existing storm drain inlets are located on the bridge approaches, and are anticipated to be within the rehabilitation construction limits. These inlets currently drain directly into Mill Creek. The rehabilitation project should consider opportunities to reduce stormwater runoff into Mill Creek as part of the design.

II. Safety

Crash history data is not readily available at this location via the VTrans public crash data query tool, and there were no safety concerns noted in the community questionnaire. Crash history can be further evaluated in the design phase, however given the limited scope of work outside of the bridge, additional safety measures are not anticipated as part of the project.

III. Maintenance of Traffic

Staged Construction Methods

Partial width, or staged construction methods are often employed to mitigate traffic impacts, by locating vehicular traffic on a portion of the bridge cross-section, while construction activities progress on the remaining width. Then, the traffic is shifted to the newly constructed portion of the bridge, and construction activities are repeated on the remaining existing structure. Staged construction often results in increased construction duration due to decreased efficiency, but this duration is offset by maintaining traffic access over the bridge during construction. Temporary barriers are typically employed to safely separate traffic from the work zone. The existing bridge deck provides only 24-feet of roadway width, excluding the raised sidewalk. In a staged construction configuration, a single traffic lane and shoulders would occupy 11-feet at a minimum, and the temporary barrier would occupy 3-feet, leaving only 9-feet of width for the construction zone. This narrow width would limit construction access, increase safety risk for construction workers and vehicles, and it would not be adequate width for traffic in the second stage of construction.

Additionally, the floor beams are supported by only two columns or arch ribs. In a staged construction configuration, one half of the floor beam and its support would be removed, requiring a temporary support structure in the middle of the bridge, parallel to the arch ribs. The support structure would need to carry traffic while partial floor beams are removed.

Due to the narrow width of the existing bridge deck, and the non-redundant support for the floor beams, staged or part-width construction is not recommended for rehabilitation of Bridge 55.

Offsite Detour

Given the limited available width for construction and concerns with staged construction on the non-redundant, two-rib arch bridge, an offsite detour was considered. In reviewing the feasibility of a detour, the relatively low traffic volume, and the single season duration of construction make this a viable option. To mitigate the impact to the traveling public, a local detour route(s) in conjunction with a truck detour route was considered.

Local detour: Maple Street, located less than 0.1 miles south of the bridge, is currently a one-way street in the eastbound direction. The community questionnaire noted that the Town would be in favor of making this a two-way street to accommodate a detour. The total detour length would be 0.4 miles and would use Union Street. Another option is to use Maple Street as the pedestrian and bicycle detour route, maintaining the existing travel lane, and creating a temporary sidewalk and/or bike lane. Lowell Street and Clough Ave could then be used as the local detour, accommodating two-way vehicular traffic. Given the limited detour length, and the proximity of the Windsor Fire Department to the detour routes, potential impacts to emergency services responses would be expected to be minimal. While the existing pavement width is adequate and the work could be primarily accomplished with striping and signing, some minor improvements at the intersection of Maple Street and Union Street may be required, and some retiming of the signal at Union Street and Route 5 should be considered. The detour routes will be evaluated further in final design to confirm turning radii, coordinate with stakeholders and impacted services, and to finalize the level of potential improvements needed to support the detour.

Truck Detour: There are approximately 4% trucks according to the traffic memo, and the local detour will not accommodate large vehicles. As such, the formal truck detour under consideration utilizes I-91 exits 8 and 9 to bypass Windsor and Route 5. The total detour length from each end of the bridge is 18.7 miles, but most detour users would likely be traveling less distance, and the distance on I-91 is 9 miles.

Accelerated Bridge Construction Methods

Accelerated Bridge Construction (ABC) refers to construction methods that significantly decrease the duration of traffic disruption, typically utilizing prefabricated elements or installing preconstructed bridges assembled offsite and moved into position. A full bridge closure is still required but the duration is decreased by using ABC. Installation of prefabricated bridge elements often requires larger or specialty equipment at an increased cost. ABC methods for the rehabilitation of Bridge 55 were evaluated for cost and schedule impacts. The analysis assumed that mobile cranes positioned at each approach would be required to lift precast floor beams and deck panels, weighing up to 30,000 lbs, into place. This analysis shows that for all actionable alternatives, the time savings is approximately 1-2 months with 5-10% increase in project

construction cost. Due to the small size of Bridge 55, and the short duration required for rehabilitation using traditional construction methods, the relative benefits of ABC are limited.

IV. Alternatives Discussion

The proposed rehabilitation alternatives are based on findings from load ratings, hands on inspection, material testing, and qualitative service life assessments. With the primary goal of preserving historic Bridge 55, the rehabilitation alternatives address elements with observed deterioration and compromised structural capacity and define project scope to increase the service life and reliability of the bridge.

The following alternatives were considered in support of this project:

1. No Action
2. Alternative 1 – Deck Replacement
3. Alternative 2 – Superstructure Replacement
4. Alternative 2w – Superstructure Replacement with Deck Widening

To quantify repair needs, the severity of deterioration is expressed as a Distress Quantity Ratio (DQR), which sums the mapped surface area of delamination, spalls, freeze-thaw damage, and previous repair areas (which typically require repair during rehabilitation) divided by the total exposed surface area for any element. This ratio, expressed as a percentage, represents the portion of the element likely needing rehabilitation, with 0% indicating no visible areas require intervention and 100% representing that the entire visible surface shows signs of deterioration requiring repair. Generally, DQR greater than 10% indicates the need for repair activities, while DQR ranging from 20% - 40% indicates major rehabilitation is required. Distress like cracking, moisture staining, rust staining, and efflorescence signal potential future problems but usually don’t warrant immediate repair. These indicators are excluded from the DQR but are still considered when evaluating long-term durability and rehabilitation action required.

The anticipated remaining service life refers to the time, without corrective action or intervention, until the quantity of deterioration in an element reaches the threshold for rehabilitation. For this report, the anticipated remaining service life was categorized as follows:

- Short – rehabilitation action is already required or likely to be needed within 5 years
- Moderate – rehabilitation action is likely to be needed in 5 to 20 years
- Long – rehabilitation action is likely not to be needed for more than 20 years

The conditions indicating repair scope are described only briefly here. More detailed descriptions of the deterioration mechanisms, and discussion of service life assessment, are available in the Materials Testing and Condition Assessment Report (March 2025), included in Appendix R.

No Action

This alternative would involve leaving the bridge in its current condition. A good rule of thumb for the “No Action” alternative is whether the bridge can stay in place without any work being performed on the bridge in the next 10 years. Therefore, this alternative is not recommended as multiple elements of the bridge have a short remaining service life and are anticipated to become compromised within 5 years.

Alternative 1 – Deck Replacement

Deck

The roadway portion of the deck's topside is currently obscured by an asphalt overlay, limiting direct visual assessment of the top surface of the original deck to only the topside of the sidewalk. Distress in the asphalt overlay was seen in the presence of cracking along the original deck expansion joints located along floor beams 3 and 6 as well as some areas that exhibited delamination from the deck below. Existing patches in the overlay indicate previous topside distress. The exposed sidewalk topside shows a DQR of 20% due to widespread delamination with areas of previous repairs and areas of exposed reinforcement contributing to the distress. While the overlay likely protected the deck below from deterioration more than the exposed sidewalk, the condition of the overlay cannot be used as an accurate representation of the quality of the structural system below.

The underside and edges of the deck also show signs of deterioration with an overall DQR of 16% for the entire deck, and with more concentrated wear near the expansion joints and the north end of the bridge. The underside of bay 8 exhibits a DRQ of 64%, with large areas of delaminated and spalled concrete, some of which have exposed and corroded reinforcing. The deck fascia on both sides of the bridge shows heavy deterioration in the form of surface disintegration, areas of dense cracking, and exposed, corroded reinforcement. Corrosion in reinforcement indicates current section loss, or signifies potential for future section loss, and reduction in load carrying capacity which cannot be addressed through surface repairs. Material testing indicates high chlorides and corrosion activity, which are signs of decreased remaining service life.

A complete deck replacement is recommended because significant deterioration is present across multiple areas of the deck that are visible, while the asphalt overlay masks underlying damage and cannot be relied upon as an accurate indicator of deck condition, making full replacement the most reliable long-term solution. Alternative 1 assumes replacement in-kind with the existing deck width and cross-section. The existing deck thickness varies and includes a void below the raised sidewalk. The replacement deck is assumed to be uniform thickness with a separately poured raised sidewalk.



Figure 1. Deterioration at Deck Underside and Fascia

Railing

The railings exhibit a large quantity of visible deterioration with prevalent cracking, large areas of exposed and corroded reinforcement, and missing or broken elements. The anchorages to the deck are of questionable reliability due to the deterioration of the deck overhangs previously discussed. The railings are original to the structure, and do not meet current NCHRP crash testing requirements.

Due to widespread deterioration, the railings are beyond their useful service life. Because the deck is recommended for full replacement, the severely deteriorated railings should also be reconstructed in their entirety. Replacing both together ensures structural reliability and durability for the desired extended service life of the structure. The existing balustrade railings contribute to the historic integrity of the bridge, and the replacement railing should retain the character of the structure while meeting current crash protection requirements. Constructing the approach railings on moment slabs should be considered in final design to minimize the vehicle impact loading on the existing wingwalls.



Figure 2. Deterioration in bridge railing

Floor Beams

Overall, the floor beams have a DQR of 11%. However, the deterioration condition varies significantly with the location of the floor beam. Floor beams 3 and 6 are located under the original deck joints, and have DQRs of 27% and 35% respectively. Widespread areas of delamination and spalls have exposed the reinforcement. Advanced corrosion has resulted in section loss in both longitudinal and transverse reinforcement in the portion of these floor beams spanning between the arch ribs, reducing the load carrying capacity of these members. At the remaining floor beams, which were not exposed by deck joints, the center portions are in generally good condition, exhibiting less deterioration with only small, localized areas of delamination and spall. Several overhang portions of the floor beams on both the upstream and downstream sides of the bridge exhibit deterioration including widespread cracking, delamination, efflorescence, and exposed and corroded reinforcement.

Floor beams 3 and 6 have exceeded their useful service life. Due to the advanced deterioration, active corrosion and section loss in the reinforcement, complete replacement of floor beams 3 and 6 is recommended.

The overhangs of the remaining floor beams have an anticipated short remaining service life of less than 5 years in their current condition. For this alternative, reconstruction of the floor beam overhangs is recommended, preserving in-place reinforcement bars to provide structural continuity where corrosion has not compromised the steel. If rebar exposed during construction exhibits section loss or corrosion, removals could extend beyond the width of the in-place columns and complete floor beam replacement may be required.



Figure 3. Underside of floor beam 6



Figure 4. Typical deterioration at floor beam overhangs

Columns & Tie Beams

The concrete columns exhibit relatively low distress, with localized spalling and delamination at the lower corners of the columns and smaller isolated distress on other portions of the columns where concrete cover is low. Due to the small surface area of the columns, DQR was not calculated for these elements.

The columns are in generally good condition except as discussed above, with a long estimated remaining service life. Therefore, for this alternative, the recommended course of action is localized concrete crack and surface repairs at all areas of deterioration.

The tie beams exhibit relatively limited deterioration with only small locations of spalls and delamination. The overall DQR for the tie beams is 4% with no individual tie beam presenting a DRQ larger than 5%. Localized crack and surface repairs are recommended as part of this alternative.



Figure 5. Typical deterioration at columns (left) and tie beams (right)

Arches

The arches ribs have an overall DQR of 10% with larger quantities of deterioration located in the outer thirds of the arch spans closest to the abutments. Various locations throughout the arch ribs exhibit isolated areas of cracking, delamination, spalling, and exposed reinforcement. Previous surface patches have deteriorated and failed. The identified deterioration does not appear to be caused by overloading or structural distress. Material testing indicates the presence of corrosion-causing chlorides in the concrete, and the arch springlines nearest the stream are susceptible to saturation and freeze-thaw deterioration.

The remaining service life of the arch ribs without intervention is estimated to be less than 20 years, before deterioration progresses to a condition requiring significant rehabilitation. The appearance and structural system of the arch ribs contribute to the historic nature of the bridge and preserving these members is critical to the service life of the historic structure. Localized crack repair and concrete surface repair are recommended. Improved service life may be achieved by protecting the arch ribs from water penetration through surface coating. Corrosion mitigation measures should also be considered to delay future deterioration caused by chlorides already present in the concrete. See additional discussion under Corrosion Mitigation and Protection Measures.



Figure 6. Localized deterioration in the arches at the upstream rib north springline (left) and downstream rib (right)

Abutments and Wingwalls

The faces of the abutment walls and wingwalls are in generally good condition, with a DQR ranging from 4% – 12%. Isolated areas of unsound concrete are present on both abutments and in the visible areas of the wingwalls. The upstream south wingwall has concentrated deterioration below a drain outlet near the top of the wall. Much of the upstream north wingwall and both downstream wingwalls are obscured by soil, so while they could not directly be inspected, the top portions that were visible showed signs of surface erosion, cracking and efflorescent below the deck joints with severity decreasing with distance away from the joint. The recommended

course of remediation for these issues includes surface repairs and crack repairs on all locations of deterioration.

The abutment and wingwall footings exhibit a DQR ranging from 10 – 29%. Deep freeze-thaw deterioration and surface erosion is present on the top and side faces of the footings near the waterline, and below the drain outlet at the south upstream drain. On the north abutment footing, the presence of a small waterfall under the bridge has begun to undermine the footing. Cracking, staining, and effloresced are present on visible footing surfaces, indicating signs of wear due to freeze-thaw cycling and water saturation.

The abutment wall and wingwalls are anticipated to have a long remaining service life and require localized surface and crack repairs as part of the rehabilitation project. In contrast, the footings exhibit more severe deterioration and a short anticipated remaining service life. Targeted deep repairs to remove freeze-thaw damaged concrete, and the construction of new concrete facing to address ongoing freeze-thaw and water-related damage are recommended at the abutment and wingwall footings. To address the additional issues caused by the drainage onto the upstream substructure, it is recommended that the drainage be redirected to not outlet directly onto the wingwall.



Figure 7. Staining and section loss at the upstream south wingwall (left) and surface erosion at the north abutment footing (right)

Alternative 2 − Superstructure Replacement

Alternative 2 builds upon all the repair and preservation measures identified in Alternative 1 but recommends the replacement of the of all floor beams and columns in their entirety, with the new columns anchored directly into the existing arches. Since significant intervention is already required to replace two of the floor beams, reconstruct the overhangs, and accommodate new deck construction; the increased element removals will result in only minimal cost increases and schedule impacts. Full floor beam replacement provides consistency across the structure rather than leaving a mix of old and new elements with differing strength and durability, and mitigates

the risk of quantity growth from overhang reconstruction. This uniformity improves predictability of long-term structural behavior and simplifies future maintenance. Replacing both the floor beams and columns also creates an opportunity to evaluate a jointless bridge design, which could eliminate the recurring deterioration caused by failing expansion joints but could also result in increased flexural demands in the columns. Additional evaluation during preliminary design will assess the capacity of the existing arches and columns to accommodate a jointless bridge. Additionally, reconstructing the columns provides flexibility to adjust element depths as needed during final design, while maintaining the bridge's profile grade.

Alternative 2w − Superstructure Replacement with Widening

This alternative incorporates all the repairs and replacements outlined in Alternative 2 while also widening the new bridge deck by up to 4'-0", for a total deck width of up 37'-6". Public engagement meetings with the Town of Windsor and the Community Questionnaire (Appendix Q) indicate that the community desires improved pedestrian and bicycle facilities. This alternative does not consider additional width to provide a dedicated bike lane. The additional width would increase the existing shoulders from 1'-0" to 3'-0", while maintaining the existing lane and sidewalk widths. The widened section would extend off the bridge and transition to the existing cross-section beyond the bridge approaches. The widening offers an opportunity to address current deficiencies while improving long term usability for all bridge users. Additional evaluation during final design is required to determine the structural impacts on the arches, abutments and wingwalls to carry the additional weight of the widened deck and floor beams and to support the railings and sidewalk at the approaches. The widening, although not providing full bikes lanes, would allow for future flexibility along the corridor if widening or other bicycle facilities are considered on the approaches, and would provide shoulders that meet the minimum offset to curb of 2'. The existing approach roadway and shoulders predominantly match the existing lane and shoulder widths on the bridge, which means that any widening on the bridge does not necessarily provide immediate benefit along the corridor. Further evaluation of widening will consider constraints posed by transmission line towers located within approximately 2' of the existing bridge both upstream (east) and downstream (west). Note - the plans in Appendix U show the widening centered about the existing baseline. To avoid impacts to the transmission tower on the southwest quadrant of the bridge, the widening either needs to taper in to match existing right after the bridge, or the baseline needs to be shifted to the east (downstream). If this option is selected, additional geometric analysis will be required in final design.

Corrosion Mitigation and Protection Measures

The following measures may be applied alone or in combination to extend the service life of existing or new elements of Bridge 55.

Concrete Surface Coatings and Sealers

The application of water-resistant coatings or sealers is recommended to protect repaired and existing concrete elements from further deterioration. These coatings serve as a barrier against moisture and chloride intrusion, which are primary contributors to corrosion of embedded reinforcement and freeze-thaw damage. Depending on the type of coating or sealer, re-application is expected after 10-20 years. While this is recommended, it is not a required step and will have to be further evaluated with the state and local historians to determine if the value of the coatings supersedes the potential impacts to the historic nature of the bridge.

The cost and schedule estimates for all the Alternatives considered include concrete surface coating on all exposed surfaces of the existing or replaced floor beams, columns, arches, wingwalls and abutment faces. Localized concrete surface repairs, combined with a coating or sealer, are anticipated to provide a service life of 25 years for the arch ribs.

Cathodic Protection

Cathodic protection techniques reduce the rate of corrosion in reinforcing steel by interrupting the electrochemical process causing corrosion. Commonly used cathodic protection systems include chloride extraction, metalizing and discrete sacrificial anodes. The sacrificial anodes are composed of electrically active metal, typically zinc, which are embedded within the existing concrete at strategic locations and electrically connected to the existing reinforcement. The anodes are consumed over time while the reinforcing steel is protected. The design life of sacrificial anode systems is typically 20-30 years. By actively protecting the reinforcement, the anodes extend the service life of the existing concrete elements beyond what surface treatments or patch repairs alone can achieve. Additional discussion of cathodic protection alternatives available is included in the Materials Testing and Condition Assessment Report (March 2025) included in Appendix R.

The costs and impacts of cathodic protection vary depending on the type of system selected and magnitude of application. Further evaluation, including additional testing and analysis, is recommended during final design to select the appropriate cathodic protection system and identify the targeted locations of application. For cost and schedule evaluation of the Alternatives presented, cathodic protection was assumed to include discrete anodes installed in the existing floor beams and arches to remain for that Alternative. The combination of local concrete surface repairs, cathodic protection system, and application of a coating or sealer is anticipated to extend the service life of the arches to approximately 50 years.

V. Cost Matrix

		Do Nothing	Traditional Construction			ABC Construction		
			Alt 1 - Deck Replacement	Alt 2 - Superstructure Replacement	Alt 2w - Superstructure Replacement with Deck Widening	Alt 1 - Deck Replacement	Alt 2 - Superstructure Replacement	Alt 2w - Superstructure Replacement with Deck Widening
COSTS	FY 25 Rehabilitation Cost	-	$3,644,600	$4,224,600	$5,148,600	$3,710,600	$4,362,600	$5,476,600
	FY 25 Corrosion Mitigation	-	$41,600	$10,400	$10,400	$41,600	$10,400	$10,400
	FY 25 Maintenance of Traffic	-	$40,000	$40,000	$40,000	$40,000	$40,000	$40,000
	FY 25 Construction Engineering + Contingencies 20%	-	$745,240	$855,000	$1,039,800	$758,440	$882,600	$1,105,400
	Escalation 3.5% annually 2025-2030	-	$839,228	$962,831	$1,170,937	$854,093	$993,912	$1,244,811
	FY 30 Total Construction Costs + CEC	-	$5,310,668	$6,092,831	$7,409,737	$5,404,733	$6,289,512	$7,877,211
	Preliminary Engineering 8% [1]	-	$542,353	$604,926	$710,279	$549,879	$620,661	$747,677
	FY 30 Total Project Costs	-	$5,853,021	$6,697,757	$8,120,016	$5,954,611	$6,910,173	$8,624,888
SCHEDULE	Construction Duration	None	6.5 Months	7.5 Months	8 Months	5.5 Months	5.5 Months	6.5 Months
	Traffic Control During Construction	None	Full Closure	Full Closure	Full Closure	Full Closure	Full Closure	Full Closure
ENGINEERING	Typical Section – Bridge [2]	33’-6” Out-to-Out, 1’ Shoulder, 2-11’ Lanes, 1’ Shoulder	33’-6” Out-to-Out, 1’-2½” Shoulder, 2-11’ Lanes, 1’-10” Shoulder	33’-6” Out-to-Out, 1’-2½” Shoulder, 2 11’ Lanes, 1’-10” Shoulder	36’-5½” Out-to-Out, 3’ Shoulder, 2 11’ Lanes, ’3’ Shoulder	33’-6” Out-to-Out, 1’-2½” Shoulder, 2 11’ Lanes, 1’-10” Shoulder	33’-6” Out-to-Out, 1’-2½” Shoulder, 2 11’ Lanes, 1’-10” Shoulder	36’-5½” Out-to-Out, 3’ Shoulder, 2 11’ Lanes, 3’-Shoulder
	Bike/Ped Access	6’ Sidewalk	6’ Sidewalk	6’ Sidewalk	6’ Sidewalk	6’ Sidewalk	6’ Sidewalk	6’ Sidewalk
	Utility	Temporary Impacts	Temporary Impacts	Temporary Impacts	Temporary Impacts	Temporary Impacts	Temporary Impacts	Temporary Impacts
	ROW Acquisition	None	None	None	None	None	None	None
	Design Life	Short (0-5 yrs)	Long (> 20 yrs)	Long (> 20 yrs)	Long (> 20 yrs)	Long (> 20 yrs)	Long (> 20 yrs)	Long (> 20 yrs)

[1] Includes additional testing and cathodic protection system selection

[2] Assumes 1’-2” width of new railing

VI. Conclusion

Structure Recommendations

Based on the need to preserve Bridge 55 in Windsor, Alternative 2 – Superstructure Replacement is recommended to achieve reliable structural performance and preserve, to the extent possible, original fabric and character defining features of the historic structure. Superstructure widening (Alternative 2w) to improve the bridge shoulders will continue to be evaluated for structural feasibility and site constraints in final design. The existing bridge railing, deck and floor beams exhibit extensive deterioration; and localized repairs to these elements are not anticipated to provide significant improvement in their service life. The arch ribs are significant components contributing to the historic importance of the bridge and are prioritized for preservation. Superstructure replacement will address the elements with the most deterioration and provide improved protection to the existing elements to remain. A jointless deck, which will be investigated in final design, will reduce the effects of leakage onto the existing arches and abutments. Further protection of the arches through coatings and cathodic protection can provide a long service life extension, up to 50 years, to these historically significant elements of the Bridge. Alternative 2 includes the following rehabilitation and repairs:

Deck Replacement

The deck shows widespread deterioration, with delamination, spalling, and reinforcement exposure masked in part by the asphalt overlay. Full deck replacement is recommended as the most reliable long-term solution, ensuring structural integrity and eliminating hidden damage.

Railing Replacement

The railings are severely deteriorated with cracking, exposed reinforcement, and unreliable anchorage due to deck overhang deterioration. Since the deck is being replaced, full railing replacement is recommended to restore safety and extend service life.

Floor Beams

All floor beams, including the severely deteriorated beams 3 and 6 and the overhang portions, should be fully replaced to eliminate the mix of old and new elements with differing strength and durability. This approach ensures uniform structural performance and provides the most reliable long-term solution given the major interventions already required.

Columns

All columns are recommended to be replaced as part of Alternative 2. Full replacement provides design flexibility for eliminating deck joints variation in superstructure depth while maintaining profile grade requirements.

Arches

Localized concrete and crack repairs, along with correction of failed past repairs is recommended at deteriorated locations.

Tie Beams

Surface and crack repairs at locations of deterioration is recommended to maintain the anticipated long service life of the tie beams.

Abutments and Wingwalls

Localized repairs are recommended, along with redirecting drainage to prevent further concentrated deterioration. Scour mitigation and deep repairs at the footings are recommended to address freeze-thaw and water-related damage.

Corrosion Mitigation and Protection

Further evaluation of coatings/sealants and cathodic protection systems is required during the preliminary design phase to select cost-effective strategies that maintain the historical integrity of the existing bridge elements. Surface coatings or sealants are recommended to protect new and existing bridge elements from water intrusion, if they are found to have limited adverse historical impacts. Installation of cathodic protection in the arch ribs is recommended to provide further protection and service life extension for the arches.

Traffic Control

Staged construction and a full detour were considered. Staged construction is not recommended due to insufficient available construction zone, and concerns with constructability of a partial width rehab related to the concrete arch. The full detour allows for lower cost and shorter duration construction, and the limited traffic volume and available detour routes make a detour a feasible option. The use of a shorter local detour on Maple Street, Clough Street and Lowell Ave, and a longer truck detour using I-91 Exits 8 and 9 is the recommended option. Detour routes will be evaluated further in final design to confirm feasibility and coordinate with the Town before finalizing the detour routes.

Due to the small scale of the bridge, the schedule sees minimal improvements by using accelerated bridge construction methods as compared to the increase in cost; therefore, ABC construction is not recommended for the rehabilitation of Bridge 55.

VII. Appendixes

Appendix A: Site Pictures
Appendix B: 2023 Inspection Report
Appendix C: Windsor Town Map
Appendix D: Geotechnical Scoping Report Data Form
Appendix E: Resource ID Completion Memo
Appendix F: Archaeological Resource ID
Appendix G: Historic Resource ID
Appendix H: Natural Resource ID
Appendix I: Stormwater Resource ID
Appendix J: Landscape Resource ID
Appendix K: Environmental Specialist Memo
Appendix L: Preliminary Hydraulics Memo
Appendix M: Hazardous Waste Urban Soils Map
Appendix N: Floodplain Rivers Map
Appendix O: Utilities Memo
Appendix P: Traffic Memo – VTrans
Appendix Q: Community Questionnaire
Appendix R: WJE Materials Testing and Condition Assessment Report
Appendix S: Basis of Estimate
Appendix T: Basis of Schedule
Appendix U: Plans

Appendix A: Site Pictures

All photos courtesy of Wiss, Janney, Elstner Associates, Inc.



Western Elevation of Bridge 55, Looking Southeast



Eastern Elevation of Bridge 55, Looking Southwest



Elevation of Downstream North Wingwall, Looking Northwest



View of Upstream South Wingwall, Looking Southeast



Elevation of North Abutment, Looking Northeast



Elevation of South Abutment, Looking Southwest



Plan View of Deck Topside, Top of Photo is Southeast



Deterioration of Deck Underside, Looking Northwest



Deterioration of Upstream Fascia, Looking Northeast



Deck Railing Elevation, Looking Southwest

Appendix B: 2023 Inspection Report

VERMONT
AGENCY OF TRANSPORTATION

Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023



Town: 263 - WINDSOR

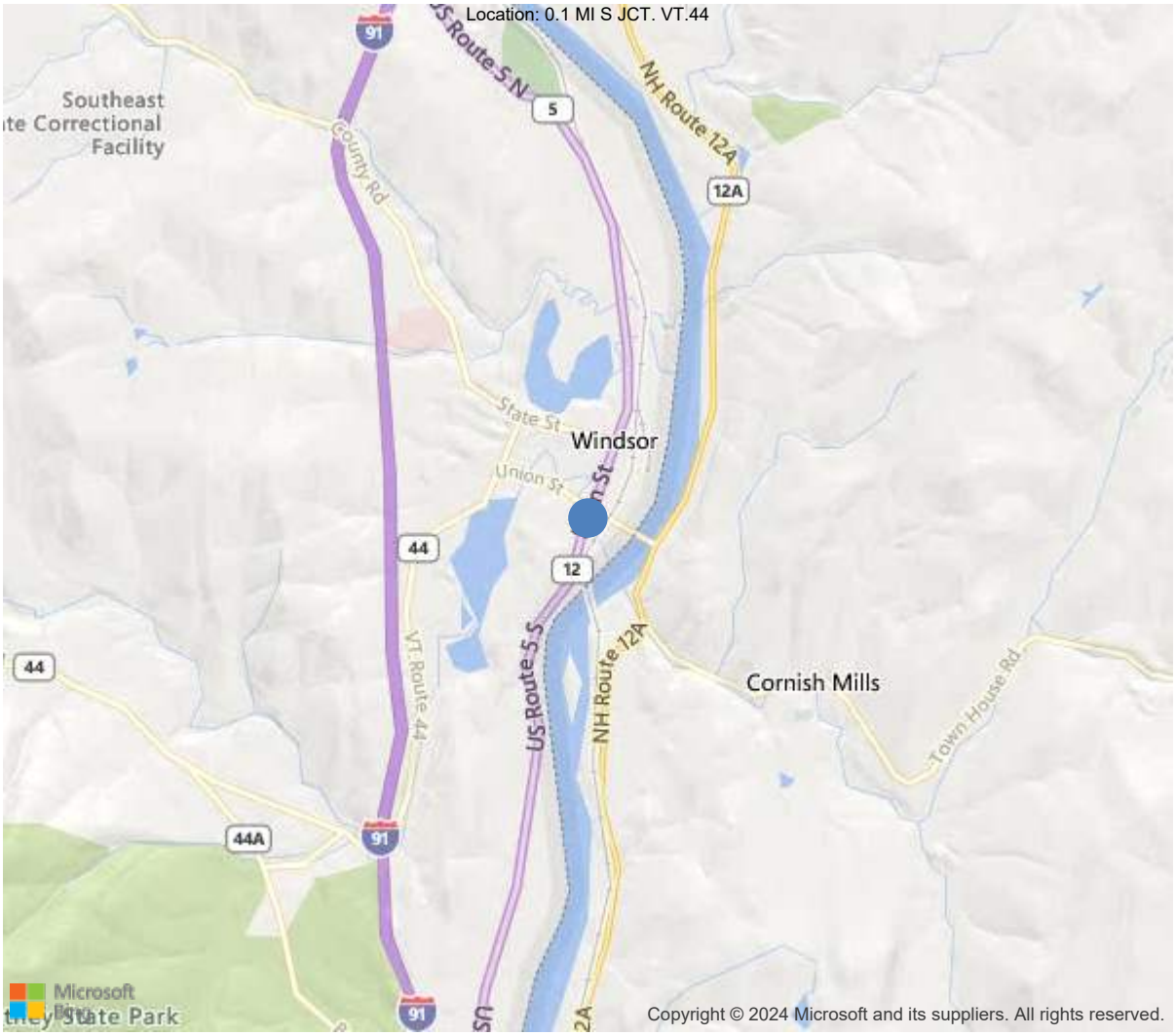
District 4, 27 - WINDSOR County

Owner: 3 - Town or Township Highway Agency

Maintenance Responsibility: 3 - Town or Township Highway Agency

VERMONT
AGENCY OF TRANSPORTATION

Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023



43.47494, -72.38882

VERMONT
AGENCY OF TRANSPORTATION

Route US5 /

Structure #00055 / (Routine)

US 00005 ML over MILL BROOK

Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023

IDENTIFICATION	
(1) State Names	50 - Vermont
(8) Structure Number	200113005514232
(5) Inventory Route	1
(2) Highway Agency District	4 - District 4
(3) County Code	27 - WINDSOR
(4) Place Code	84925
(6) Features Intersected	MILL BROOK
(7) Facility Carried	US 00005 ML
(9) Location	0.1 MI S JCT. VT.44
(11) Mile Point	0 mi
(12) Base Highway Network	No
(13) LRS Inventory Rte & Subrte	
(16) Latitude	43.4749416666667
(17) Longitude	-72.3888166666667
(98) Border Bridge State Code	
(99) Border Bridge Structure No.	
STRUCTURE TYPE AND MATERIAL	
(43) Main Structure Type	111
Material	1 - Concrete
Type	11 - Arch - Deck
(44) Approach Structure Type	00
Material	0 - Other
Type	0 - Other
(45) No. of Spans in Main Unit	1
(46) No. of Approach Spans	0
(107) Deck Structure Type	1 - Concrete Cast-in-Place
(108) Wearing Surface/Protective System	
Type of Wearing Surface	6 - Bituminous
Type of Membrane	0 - None
Type of Deck Protection	0 - None
AGE AND SERVICE	
(27) Year Built	1930
(106) Year Reconstructed	1943
(42) Type of Service	55
On	5 - Highway-pedestrian
Under	5 - Waterway
(28) Lane	
On	2
Under	0
(29) Average Daily Traffic	3300
(30) Year of ADT	2018
(109) Truck ADT	6 %
(19) Bypass, Detour Length	2 mi
GEOMETRIC DATA	
(48) Length of Maximum Span	88 ft
(49) Structure Length	90 ft
(50) Curb or Sidewalk Width	
Left	6 ft
Right	0.5 ft
(51) Bridge Roadway Width Curb to Curb	30.4 ft
(52) Deck Width Out to Out	33.5 ft
(32) Approach Roadway Width (W/Shoulders)	25 ft
(33) Bridge Median	0 - No median
(34) Skew	0 Deg
(35) Structure Flared	0 - No flare
(10) Inventory Route Min Vert Clear	99.99 ft
(47) Inventory Route Total Horiz Clear	30.4 ft
(53) Min Vert Clear Over Bridge Rdwy	99.99 ft
(54) Min Vert Underclear	0 ft
Ref:	
(55) Min Lat Underclear RT	0 ft
Ref:	
(56) Min Lat Underclear LT	0 ft
NAVIGATION DATA	
(38) Navigation Control	0 - No navigation control on w
(111) Pier Protection	
(39) Navigation Vertical Clearance	0 ft
(116) Vert-Lift Bridge Nav Min Vert Clear	0 ft
(40) Navigation Horizontal Clearance	0 ft

CLASSIFICATION	
(112) NBIS Bridge Length	Y
(104) Highway System	0
(26) Functional Class	7 - Rural Major Collector
(100) Defense Highway	1 - The inventory route is on
(101) Parallel Structure	N - No parallel structure exis
(102) Direction of Traffic	2 - way traffic
(103) Temporary Structure	
(105) Federal Lands Highways	0 - N/A
(110) Designated National Network	0 - The inventory route is not
(20) Toll	3 - On free road. The structu
(21) Maintain	3 - Town or Township Highway A
(22) Owner	3 - Town or Township Highway A
(37) Historical Significance	5 - Bridge is not eligible for
CONDITION	
(58) Deck	5
(59) Superstructure	6
(60) Substructure	6
(61) Channel & Channel Protection	8
(62) Culverts	N
LOAD RATING AND POSTING	
(31) Design Load	2 - M 13.5 / H 15
(63) Operating Rating Method	2
(64) Operating Rating	
Type	2 - Allowable Stress(AS)
Rating	49
(65) Inventory Rating Method	2 - Allowable Stress(AS)
(66) Inventory Rating	
Type	
Rating	15
(70) Bridge Posting	5 - Equal to or above legal loads
(41) Structure Open/Posted/Closed	A - Open, no restriction
APPRAISAL	
(67) Structural Evaluation	4
(68) Deck Geometry	4
(69) Clearances, Vertical/Horizontal	N
(71) Waterway Adequacy	8
(72) Approach Roadway Alignment	8
(36A) Bridge Railings	0 - Inspected feature does not meet
(36B) Transitions	0 - Inspected feature does not meet
(36C) Approach Guardrail	0 - Inspected feature does not meet
(36D) Approach Guardrail Ends	0 - Inspected feature does not meet
(113) Scour Critical Bridges	8 - Bridge foundations determined t
PROPOSED IMPROVEMENTS	
(75) Type of Work	31 - Replacement of bridge or
(76) Length of Structure Improvement	117 ft
(94) Bridge Improvement Cost (Multiply value by 1000)	$ 1470
(95) Roadway Improvement Cost (Multiply value by 1000)	$ 150
(96) Total Project Cost (Multiply value by 1000)	$ 1620
(97) Year of Improvement Cost Estimate	2020
(114) Future ADT	3465
(115) Year of Future ADT	2028

INSPECTIONS *

(90) Inspection Date			08/08/2023
(91) Frequency			24
(92) Critical Feature Inspection	Done	Freq. (Mon)	Date
A: Fracture Critical Detail	No		
B: Underwater Inspection	No		
C: Other Special Inspection			

* The inspection date and frequency information in this box contains the current NBI date and frequency information. Please refer to the report header for the date this inspection was conducted.

Deck

ELEMENTS	DESCRIPTION	UNITS	TOTAL	CS1	CS2	CS3	CS4
12	Reinforced Concrete Deck	SF	3008	2176	256	576	0
1080	Delamination/Spall/Patched Area	SF	72	0	36	36	0
1120	Efflorescence/Rust Staining	SF	640	0	100	540	0
1130	Cracking (RC and Other)	SF	120	0	120	0	0
331	Reinforced Concrete Bridge Railing	LF	280	0	280	0	0
1130	Cracking (RC and Other)	LF	280	0	280	0	0
804	Concrete Fascia	LF	280	20	260	0	0
1120	Efflorescence/Rust Staining	LF	260	0	260	0	0

58 - Deck (5 - FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.)
Reinforced concrete deck is in fair condition having areas of saturated concrete with moderate to heavy efflorescence leakage, scattered small rust stains and areas of heavy scaling scattered throughout the deck soffit near the outer portions near fascias. Section 6 between floor beams #5 and #6 has ~6'-0" x 6'-0" spall that has exposed the transverse steel reinforcing. Some various small areas of rust stains and lighter efflorescence leakage with saturation are present along the interior portions of deck soffit. Deck soffit surrounding previous curb slot drains (currently paved over) has areas of spalling with exposed steel reinforcing, saturation, scaling, efflorescence leakage and rust staining with four (4) areas present on both the upstream and downstream sides.

200 - Existing Wearing Surface Depth (3")

A21 - Deck Wearing Surface Condition (Good)
Asphalt is in fairly good condition having minor wearing in the wheel paths of both travel lanes.

A24 - Deck Curb Condition (Satisfactory)
Concrete curbs are in satisfactory condition having a few areas of light scaling and various hairline cracks scattered throughout.

A25 - Deck Sidewalk Condition (Satisfactory)
Reinforced concrete sidewalk is present on the upstream side only having a few small scattered rust stains and some areas of light scaling along the top surface. The sidewalk soffit has a few areas of saturation with efflorescence leakage and small rust stains in the surrounding areas.

A39 - Deck Fascia Condition (Fair)
Both the upstream and downstream fascia's are in fair condition having areas of light to heavy concrete scaling along the lower portions and areas of map cracking with some efflorescence leakage.

B.C.05 Bridge Railing Condition Rating (SATISFACTORY - Widespread minor or isolated moderate defects.)
Reinforced concrete balustrade rail is in okay condition having vertical cracking in nearly every post ranging from hairline up to ~1/8" in width. Reinforced top rail surface has some various cracking with light efflorescence leakage present. Reinforced concrete posts also have some various cracking with some efflorescence leakage and areas of spalling present.

B.C.08 Bridge Joints Condition Rating (GOOD - Some minor defects.)
Asphaltic plug joints are present over both abutments in fairly good condition.

APPROACH

72 - Approach Roadway Alignment (8 - Equal to present desirable criteria)

Roadway alignment is straight and flat.

A13 - Approach Rail Condition (Satisfactory)

Reinforced concrete balustrade rail is in okay condition having vertical cracking in every post ranging from hairline up to ~1/8” in width. Areas of scaling / have exposed steel reinforcing scattered throughout.

B.C.06 Bridge Railing Transitions Condition Rating (NOT APPLICABLE - Component does not exist.)

Superstructure

ELEMENTS	DESCRIPTION	UNITS	TOTAL	CS1	CS2	CS3	CS4
144	Reinforced Concrete Arch	LF	180	152	22	6	0
1080	Delamination/Spall/Patched Area	LF	18	0	12	6	0
1120	Efflorescence/Rust Staining	LF	10	0	10	0	0
155	Reinforced Concrete Floor Beam	LF	268	201	0	67	0
1080	Delamination/Spall/Patched Area	LF	67	0	0	67	0

59 - Superstructure (6 - SATISFACTORY CONDITION - structural elements show some minor deterioration.)

A50 - Super Verticals/Diagonals Condition (Satisfactory)
Floor beam's #1, #2, #7 and #8 have two (2) vertical reinforced concrete columns a piece in satisfactory condition having some various cracking with small areas of concrete popping off exposing the steel reinforcing due to lack of cover.

A55 - Lateral Bracing Condition (Good)
Cast in place concrete cross bracing is present along the arch below floor beams #1, #2, #7 and #8 in fairly good condition having a few small hairline cracks in the concrete.

A56 - Floor Beams Condition (Fair)
Eight (8) reinforced concrete floor beams are in fair condition. Floor beams #3 and #6 have moderate to heavy efflorescence leakage with small rust stains from the upper portions and heavy spalling along the lower sections that has exposed the transverse steel reinforcing and multiple steel stirrups that have heavy rusting. Floor beams #3 and #6 have break in deck over them allowing leakage through deck causing the deterioration. Remaining floor beams are in okay condition with the outer portions having minor to moderate cracking with heavy efflorescence leakage and some areas of concrete scaling.

A64 - Arch Condition (Satisfactory)
Two (2) reinforced concrete arch's are in satisfactory condition having a few small areas of rust stains and small spalls scattered throughout the underside of both arches. The upstream arch has a large spalled area along the underside near abutment #2 that has exposed all of the longutdnial steel reinforcing with concrete above being loose. Concrete patches are present along the underside near abutment #1 on both the upstream and downstream side with a few cracks in the patch.

B.C.07 Bridge Bearings Condition Rating (NOT APPLICABLE - Component does not exist.)

VERMONT
AGENCY OF TRANSPORTATION

Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023

Substructure

ELEMENTS	DESCRIPTION	UNITS	TOTAL	CS1	CS2	CS3	CS4
205	Reinforced Concrete Column	EA	8	8	0	0	0
215	Reinforced Concrete Abutment	LF	67	0	34	33	0
1120	Efflorescence/Rust Staining	LF	50	0	17	33	0
1190	Abrasion/Wear (PSC/RC)	LF	17	0	17	0	0
800	Reinforced Concrete Wing/Retaining Wall	EA	4	0	4	0	0
1120	Efflorescence/Rust Staining	EA	4	0	4	0	0

60 - Substructure (6 - SATISFACTORY CONDITION - structural elements show some minor deterioration.)
Reinforced concrete abutment #1 is in satisfactory condition having minor to moderate efflorescence leakage along the outer portions with typical small areas of light scaling surrounded by some very light honeycombing along the abutment stem.

Reinforced concrete abutment #2 is in satisfactory condition having minor to moderate efflorescence leakage along the outer portions with typical small areas of light scaling surrounded by some very light honeycombing along the abutment stem.

A77 - Retaining/Wingwall Condition (Good)
Reinforced concrete retaining walls are in fairly good condition having some hairline cracks.

A78 - Abutment Footings Condition (Satisfactory)
Both the upstream and downstream footings have minor to moderate scaling along the top surface and along the upper edge with some various cracking scattered throughout.

CHANNEL

61 - Channel Condition (8 - Banks are protected or well vegetated. River control devices such as spur dikes and embankment protection are not required or are in a stable condition.)
Mill brook flows at a sharp angle upstream of structure directed towards abutment #2. Both abutments are poured on ledge. Channel bottom is a mixture of ledge throughout and silt/gravel on the upstream end. Channel banks are on ledge with some various stone throughout with some vegetation growth along the upper portions.

B.C.10 Channel Protection Condition Rating (VERY GOOD - Some inherent defects.)
Abutments are poured on ledge.

B.C.11 Scour Condition Rating (Insignificant scour.)

GENERAL OBSERVATION

Structure is in okay condition having areas of progressing deterioration scattered throughout. Deck soffit has areas of saturated concrete with moderate to heavy efflorescence leakage, scattered small rust stains and areas of heavy scaling scattered throughout the fascia's. Section 6 between floor beams #5 and #6 has ~6'-0" x 6'-0" spall that has exposed the transverse steel reinforcing. Abutments have minor to moderate efflorescence leakage along the outer portions with typical small areas of light scaling surrounded by some very light honeycombing along the abutment stem. Structure will need rehabilitation project in near future to deter further deterioration with structure continuing to age.

VERMONT
AGENCY OF TRANSPORTATION

Asset **#200113005514232**(Routine)
District: 4, **County:** 27
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023

ELEMENTS	DESCRIPTION	UNITS	TOTAL	CS1	CS2	CS3	CS4
12	Reinforced Concrete Deck	SF	3008	2176	256	576	0
1080	Delamination/Spall/Patched Area	SF	72	0	36	36	0
1120	Efflorescence/Rust Staining	SF	640	0	100	540	0
1130	Cracking (RC and Other)	SF	120	0	120	0	0
144	Reinforced Concrete Arch	LF	180	152	22	6	0
1080	Delamination/Spall/Patched Area	LF	18	0	12	6	0
1120	Efflorescence/Rust Staining	LF	10	0	10	0	0
155	Reinforced Concrete Floor Beam	LF	268	201	0	67	0
1080	Delamination/Spall/Patched Area	LF	67	0	0	67	0
205	Reinforced Concrete Column	EA	8	8	0	0	0
215	Reinforced Concrete Abutment	LF	67	0	34	33	0
1120	Efflorescence/Rust Staining	LF	50	0	17	33	0
1190	Abrasion/Wear (PSC/RC)	LF	17	0	17	0	0
331	Reinforced Concrete Bridge Railing	LF	280	0	280	0	0
1130	Cracking (RC and Other)	LF	280	0	280	0	0
800	Reinforced Concrete Wing/Retaining Wall	EA	4	0	4	0	0
1120	Efflorescence/Rust Staining	EA	4	0	4	0	0
804	Concrete Fascia	LF	280	20	260	0	0
1120	Efflorescence/Rust Staining	LF	260	0	260	0	0

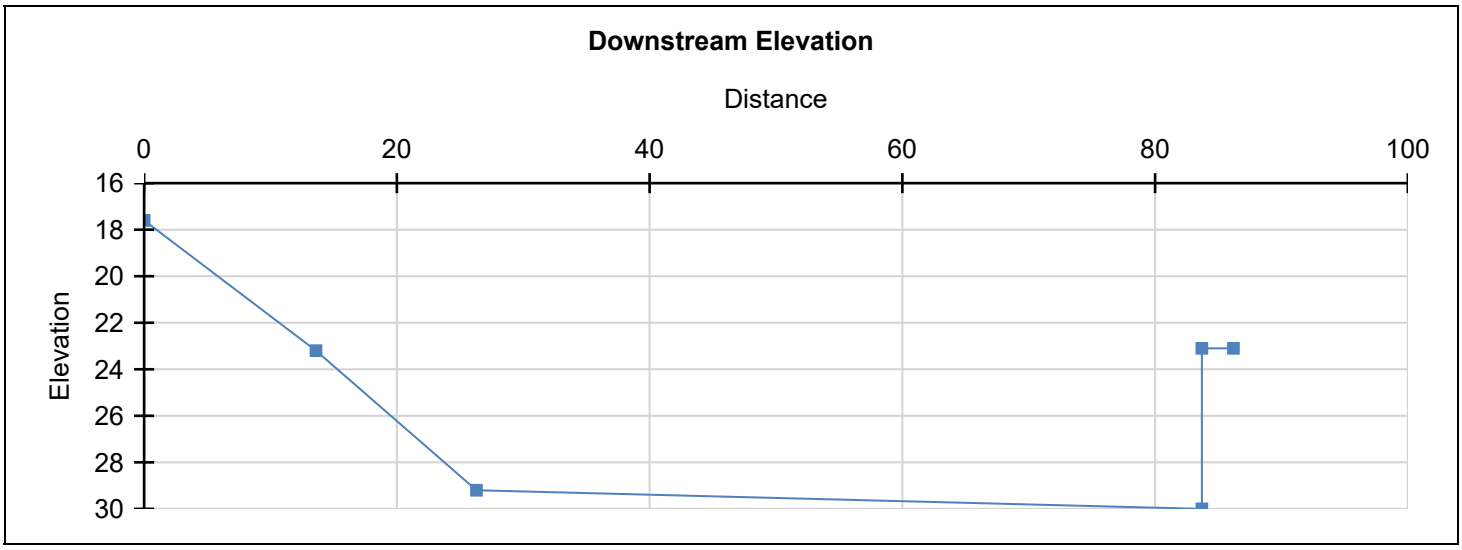
VERMONT
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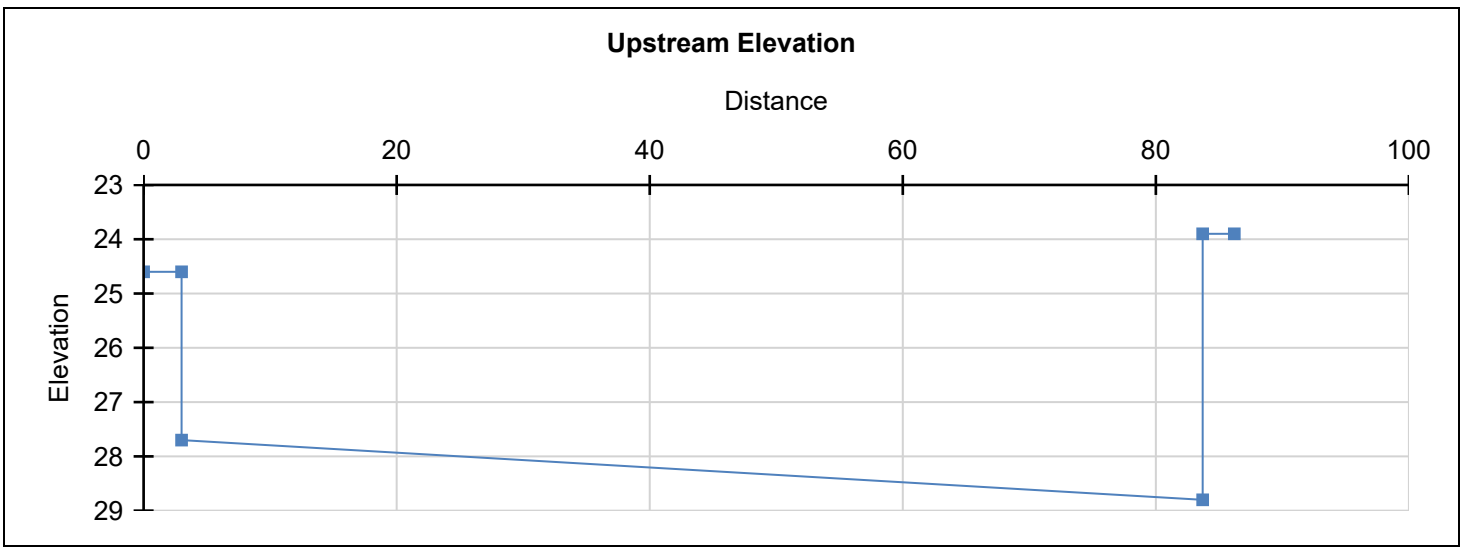
Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023

Channel Profile

Waterway Flow:	Left to Right (With Stationing)	Top of Water:	
Origin:	Deck Underside (Abutment #1)	Bottom of Beam:	

Station	Distance	Downstream	Upstream
Abutment #1	0	17.6	24.6
Top of Footing	3		24.6
Bottom of Footing	3		27.7
Channel Bank	13.6	23.2	
Edge of Channel	26.3	29.2	
Bottom of Footing	83.7	30	28.8
Top of Footing	83.7	23.1	23.9
Abutment #2	86.2	23.1	23.9





VERMONT
AGENCY OF TRANSPORTATION

Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
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Upstream Elevation



Abutment 2 Approach



Wearing Surface from Abutment 2



Asphaltic Plug Joint at Abutment 2

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Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023



Upstream Bridge Rail from Abutment 2



Downstream Channel



Upstream Channel



Asphaltic Plug Joint at Abutment 1

VERMONT
AGENCY OF TRANSPORTATION

Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023



Abutment 1 Approach



Downstream Bridge Rail from Abutment 1



Cracking in Downstream Bridge rail near Midspan



Downstream Fascia from Abutment 2



Cracking in upstream sidewalk near Midspan



Cracking in Downstream Bridge rail near Midspan



Downstream Wingwall at Abutment 1



Downstream End of beam 1

VERMONT
AGENCY OF TRANSPORTATION

Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023



Superstructure facing abutment 2



Spalling and Exposed rebar on beam 3



Spalling and Exposed rebar on beam 6



Upstream bank erosion above Abutment 1

VERMONT
AGENCY OF TRANSPORTATION

Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023



Spalling on Downstream abutment 2 footing



Abutment 1



Efflorescence on Abutment 1



Spalling and Exposed rebar on upstream arch beam at Abutment 2

VERMONT
AGENCY OF TRANSPORTATION

Route US5 /
Structure #00055 / (Routine)
US 00005 ML over MILL BROOK
Team Lead: Stephen Piro, **Inspection Date:** 08/08/2023



Spalling and Exposed rebar in Bay 6



Downstream End of beams facing Abutment 2



Downstream End of beams facing Abutment 1



Patching on upstream arch beam at Abutment 1



Upstream Arch beam from Abutment 1



Upstream Wingwall at Abutment 1



Drainage pipe on bottom of Abutment 1



Upstream Abutment 2 Wingwall



Upstream Edge of Bridge from Abutment 2



Superstructure from Abutment 2

Appendix C: Windsor Town Map

County-Town Code: 1423

WINDSOR, WINDSOR COUNTY - 2025

TOWN HWY. No.	LENGTH IN MILES CLASS 3	CLASS 4
5	3.06	
6	2.86	
7	1.56	
8	0.72	
9	1.15	
10	1.77	
11	0.50	
13	0.42	
14	1.30	
15	0.10	
16	0.54	
17	0.22	
18	0.23	
19	0.06	
20	0.27	
21		(0.35)
22	0.20	
23	0.72	
24	0.47	
25	0.22	
27	0.04	
28	0.16	
29	0.05	
30	0.29	
31	0.07	
32	0.22	
33	0.13	
34	0.23	
35	0.18	
36	0.14	
37	0.04	
38	0.05	
39	0.28	
40	0.06	
41	0.05	
42	0.04	
43	0.08	

TOWN HWY. No.	LENGTH IN MILES CLASS 3	CLASS 4
44	0.07	
45	0.17	
46	0.29	
47	0.13	
48	0.08	
49	0.46	
50	0.16	
51	0.07	
52	0.45	
53	0.10	
54	0.04	
55	0.23	
56	0.11	
57	0.12	
58	0.05	
59	0.27	
60	0.16	
61	0.09	
62	0.11	
63	0.06	
64	0.19	
65	0.03	
66	0.03	
68	0.06	
69	0.16	
70	0.04	
71	0.07	
73	0.02	
74	0.07	
75	0.06	
76	0.04	
77	0.28	
78	0.40	
79	0.24	
80	0.68	
81	0.23	
TOTAL	24.30	0.35

TOWN HWY. No.	LENGTH IN MILES "NOT UP TO STANDARD"
44	(0.07)
65	(0.30)
70	(0.27)
TOTAL	0.64

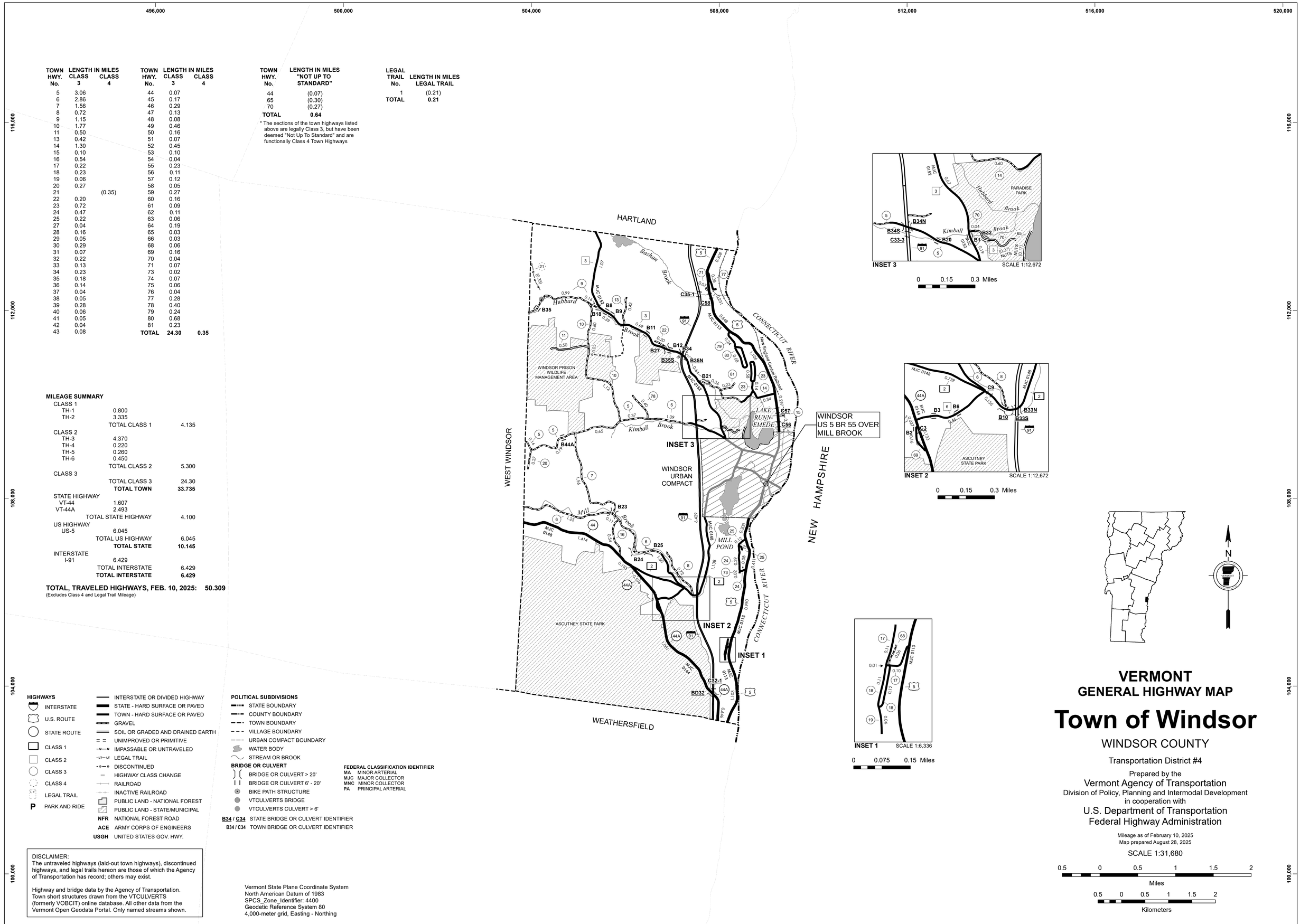
* The sections of the town highways listed above are legally Class 3, but have been deemed "Not Up To Standard" and are functionally Class 4 Town Highways

LEGAL TRAIL No.	LENGTH IN MILES LEGAL TRAIL
1	(0.21)
TOTAL	0.21

MILEAGE SUMMARY

CLASS 1		
TH-1	0.800	
TH-2	3.335	
TOTAL CLASS 1		4.135
CLASS 2		
TH-3	4.370	
TH-4	0.220	
TH-5	0.260	
TH-6	0.450	
TOTAL CLASS 2		5.300
CLASS 3		
TOTAL CLASS 3		24.30
TOTAL TOWN		**33.735**
STATE HIGHWAY		
VT-44	1.607	
VT-44A	2.493	
TOTAL STATE HIGHWAY		4.100
US HIGHWAY		
US-5	6.045	
TOTAL US HIGHWAY		6.045
TOTAL STATE		**10.145**
INTERSTATE		
I-91	6.429	
TOTAL INTERSTATE		6.429
TOTAL INTERSTATE		**6.429**

TOTAL, TRAVELED HIGHWAYS, FEB. 10, 2025: 50.309
(Excludes Class 4 and Legal Trail Mileage)



HIGHWAYS

- INTERSTATE
- U.S. ROUTE
- STATE ROUTE
- CLASS 1
- CLASS 2
- CLASS 3
- CLASS 4
- LEGAL TRAIL
- P PARK AND RIDE

- INTERSTATE OR DIVIDED HIGHWAY
- STATE - HARD SURFACE OR PAVED
- TOWN - HARD SURFACE OR PAVED
- GRAVEL
- SOIL OR GRADED AND DRAINED EARTH
- UNIMPROVED OR PRIMITIVE
- IMPASSABLE OR UNTRAVELED
- LEGAL TRAIL
- DISCONTINUED
- HIGHWAY CLASS CHANGE
- RAILROAD
- INACTIVE RAILROAD
- PUBLIC LAND - NATIONAL FOREST
- PUBLIC LAND - STATE/MUNICIPAL
- NFR NATIONAL FOREST ROAD
- ACE ARMY CORPS OF ENGINEERS
- USGH UNITED STATES GOV. HWY.

POLITICAL SUBDIVISIONS

- STATE BOUNDARY
- COUNTY BOUNDARY
- TOWN BOUNDARY
- VILLAGE BOUNDARY
- URBAN COMPACT BOUNDARY
- WATER BODY
- STREAM OR BROOK

BRIDGE OR CULVERT

- BRIDGE OR CULVERT > 20'
- BRIDGE OR CULVERT 6' - 20'
- BIKE PATH STRUCTURE
- VTCULVERTS BRIDGE
- VTCULVERTS CULVERT > 6'
- B34 / C34 STATE BRIDGE OR CULVERT IDENTIFIER
- B34 / C34 TOWN BRIDGE OR CULVERT IDENTIFIER

FEDERAL CLASSIFICATION IDENTIFIER

- MA MINOR ARTERIAL
- MJC MAJOR COLLECTOR
- MNC MINOR COLLECTOR
- PA PRINCIPAL ARTERIAL

DISCLAIMER:
The untraveled highways (laid-out town highways), discontinued highways, and legal trails hereon are those of which the Agency of Transportation has record; others may exist.

Highway and bridge data by the Agency of Transportation. Town short structures drawn from the VTCULVERTS (formerly VOBCIT) online database. All other data from the Vermont Open Geodata Portal. Only named streams shown.

Vermont State Plane Coordinate System
North American Datum of 1983
SPCS_Zone_Identifier: 4400
Geodetic Reference System 80
4,000-meter grid, Easting - Northing

VERMONT
GENERAL HIGHWAY MAP

Town of Windsor

WINDSOR COUNTY

Transportation District #4

Prepared by the
Vermont Agency of Transportation
Division of Policy, Planning and Intermodal Development
in cooperation with
U.S. Department of Transportation
Federal Highway Administration

Mileage as of February 10, 2025
Map prepared August 28, 2025

SCALE 1:31,680

0.5 0 0.5 1 1.5 2
Miles

0.5 0 0.5 1 1.5 2
Kilometers

For more information contact: Vermont Agency of Transportation, Division of Policy, Planning & Intermodal Development - Mapping Section, 219 North Main Street, Barre, VT 05641 Telephone: 802-828-2600.

This map was funded in part through grants from the Federal Highway Administration, U.S. Department of Transportation. The representation of the authors expressed herein do not necessarily state or reflect those of the U. S. Department of Transportation.

Appendix D: Geotechnical Scoping Report Data Form

Geotechnical Scoping Report Data Form

General Project Information

Project Name:	WINDSOR BF 0113(88)				
Project Pin #	22J404				
Requestor Name:	Laura Stone, P.E., Structures Scoping PM				
Prepared By:	C. Ewald, P.E., Geotechnical Engineering Manager				
Date:	12/6/2023				
Location Information of Structure:	Town: Windsor; Route: Route 5; Mile Marker: 3.34				
Structure Type:	Bridge	Structure ID #:	Br 55	Conceptual Treatment Type:	Deck
General Project Description:	The existing structure is a concrete arch that extends over Mill Brook. Foundations appear to be spread footings on rock.				

Geological Information

Surficial Map Description:	Glaciolacustrine deposit, lake sand – well sorted sand, no pebbles or boulders.
Bedrock Map Description:	Connecticut Valley Trough of the Waits River Formation, Slate, Quartzite – Predominately dark to light-grey, lustrous, carbonaceous chlorite-biotite-muscovite-quartz slate, phyllite, or schist, contains thin beds of quartzite and only sparse layers of punky weathering limestone.

Record Plan Information

Are there Record Plans?	Yes ☒ No ☐
Record Plans ID #	43s1488
Record Plan Notes:	Record plans show footings on ledge, stepping as required. Notes include approximate ledge and founding structure on suitable foundation material, or one foot into ledge elevation.

Bedrock Depth Information

Are there Historical Borings?	Yes ☐ No ☒ Unknown ☐

Historical Boring Information:

Project Name	Distance from Project (ft)	# of Borings	Top of Bedrock Elevation (ft)	Rock Type
N/A	N/A	N/A	N/A	N/A

Link to Historical Boring Information:	N/A
Is there any bedrock depth information from well data near the project location?	Yes ☐ No ☒

Well Data Information:

Well Report #	Bedrock Depth (ft)	Distance from Project (ft)
N/A	N/A	N/A

Are Bedrock Outcrops Present at the Site?	Yes ☒ No ☐
Bedrock Depth General Comments:	Bedrock outcrops can be visually seen throughout project site, in the river and at foundation locations.

General Site Conditions

Site Visit Conducted?	Yes ☐ No ☒

Date of Site Visit:	N/A
Are there Overhead Utilities at the Site?	Yes ☒ No ☐
Are there environmental Hazards Present at the Site?	Yes ☒ No ☐
Information regarding found Environmental Hazards:	There is a known hazardous site just downstream of the bridge on the northeast quadrant of the bridge. Environmental site assessments have been completed and are available online – the site is impacted by a petroleum release. I recommend Andy Shively be consulted early on this project if substructure replacement and excavation are required.
Site Condition Notes:	Utilities, environmental concerns, and site access will provide significant challenges to attaining boring information at this site if a full structure replacement is chosen. Time and cost should be built into the schedule and funds to accommodate.

Recommendations

Are Borings Needed in the Scoping Phase?	Yes ☐ No ☒
Subsurface Investigation Recommendations:	If a replacement structure is the chosen alternative here, then a well thought out subsurface investigation should be conducted after a final alignment is chosen. Borings coupled with geophysical investigation would be used to characterize the site and identify bedrock elevation for the bridge foundations. There is also an independent retaining wall that would likely require an evaluation into the current condition and need for replacement, including borings along the alignment as well.
Foundation & Structure Type Recommendations:	Based on the information reviewed, if a full bridge replacement is chosen as the preferred alternative, foundation options for a replacement structure are likely to include spread footings founded on bedrock.

The information provided is utilized from the databases and references noted in the Reference Section below. This form has been completed to the best of staff and reviewer knowledge.

Please reach out to us if you have any questions or concerns.

Staff Name & Title: CEE

Callie Ewald, P.E., Geotechnical Engineer	Callie.ewald@vermont.gov/802.595.4589

Reviewer Name & Title: END

Eric Denardo, P.E., Geotechnical Engineer	Eric.denardo@vermont.gov/802 595-4754

References:

Doll, C. G., 1970, Surficial Geologic Map of Vermont, Vermont Geological Survey, Montpelier, VT.

Ratcliffe, N. M., Stanley, R. S., Gale, M. H., Thompson, P. J., Walsh, G. J., 2011, Bedrock Geologic Map of Vermont, Vermont Geological Survey, Montpelier, VT.

Vermont Agency of Natural Resources Department of Environmental Conservation, Natural Resources Atlas, www.anr.vermont.gov/maps/nr-atlas%20, accessed 11/20/2023.

Appendix E: Resource ID Completion Memo

OFFICE MEMORANDUM

AOT - PDB - ENVIRONMENTAL SECTION

RESOURCE IDENTIFICATION COMPLETION MEMO

TO: Laura Stone, Project Manager
FROM: Lee Goldstein, Environmental Specialist
DATE: April 16, 2024
Project: **Windsor BF 0113(88) 22J404**

ENVIRONMENTAL RESOURCES:

Archaeological Resources:	X	Yes		No	See Archaeological Resource ID Memo
Historic Resources:	X	Yes		No	See Historic Resource ID Memo
Wetlands:		Yes	X	No	See Natural Resource ID Memo
Aquatic Organism Passage:		Yes	X	No	See Natural Resource ID Memo
Agricultural Soils:	X	Yes		No	See Natural Resource ID Memo
Wildlife Habitat:		Yes	X	No	See Natural Resource ID Memo
Endangered Species:	X	Yes		No	See Natural Resource ID Memo
Stormwater Considerations:	X	Yes		No	See Stormwater Resource ID Memo
Landscape Considerations:	X	Yes		No	See Landscape Resource ID Memo
6(f) Properties:		Yes	X	No	See Environmental Specialist Resource ID Memo
Hazardous Waste:		Yes	X	No	See Environmental Specialist Resource ID Memo
Contaminated Soils:	X	Yes		No	See Environmental Specialist Resource ID Memo
Wild Scenic Rivers:		Yes	X	No	See Environmental Specialist Resource ID Memo
Act 250 Permits:		Yes	X	No	See Environmental Specialist Resource ID Memo
FEMA Floodplains:	X	Yes		No	See Environmental Specialist Resource ID Memo
Flood Hazard Area:	X	Yes		No	See Environmental Specialist Resource ID Memo
River Corridor:	X	Yes		No	See Environmental Specialist Resource ID Memo
Protected Lands:		Yes	X	No	See Environmental Specialist Resource ID Memo
US Coast Guard:		Yes	X	No	See Environmental Specialist Resource ID Memo
Lakes and Ponds:		Yes	X	No	See Environmental Specialist Resource ID Memo
Scenic Highway/ Byway:	X	Yes		No	See Environmental Specialist Resource ID Memo
Environmental Justice:		Yes	X	No	See Environmental Specialist Resource ID Memo
Other:		Yes	X	No	See Environmental Specialist Resource ID Memo

cc: Project File

Appendix F: Archaeological Resource ID

VERMONT

Brennan Gauthier
VTrans Senior Archaeologist
Vermont Agency of Transportation
Project Delivery Bureau
Environmental Section
tel. 802-279-1460
Brennan.Gauthier@Vermont.gov

To: Lee Goldstein, VTrans Environmental Specialist

From: Brennan Gauthier, VTrans Senior Archaeologist

Date: 3/29/2024

Subject: Windsor BF 0113(88) - Archaeological Resource Identification

Dear Lee,

I have completed my field and desk review of Bridge No. 55 (Mill Brook Bridge) on US Route 5 in the town of Windsor, Windsor County, Vermont. Mill Brook Bridge is a concrete spandrel arch bridge that spans a deep gorge that contains Mill Brook upstream from the confluence with the Connecticut River. Replacing an earlier covered bridge in the same location, this structure was originally built in 1930 after being closed to foot and vehicular traffic due to deterioration and oversized truck weight loads that were not accounted for during the original late 1800s construction. The structure was later reconstructed in 1943 to accommodate the ever-changing needs of vehicular traffic in Windsor.

Windsor Bridge No. 55, US Route 5, Windsor, Windsor County, Vermont

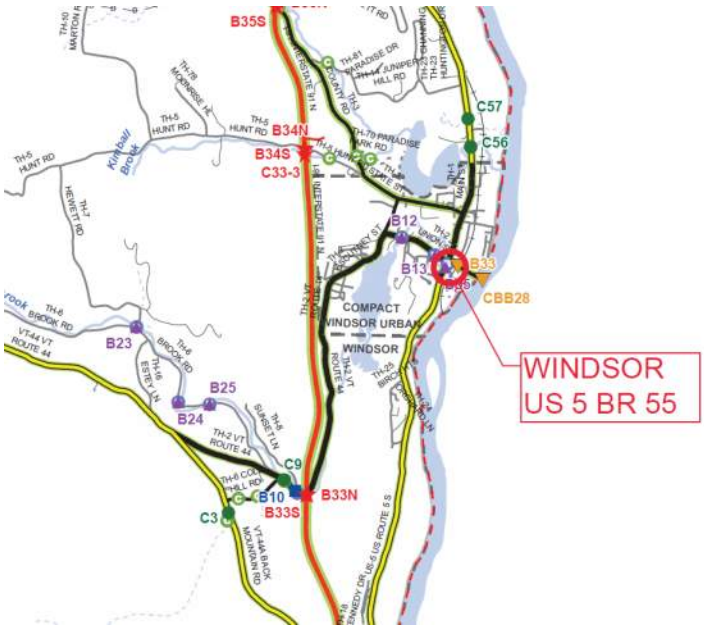


Figure 1: Bridge Location.

VTrans Working to Get You There
Vermont Agency of Transportation

Mill Brook Bridge is directly adjacent to the Robbins and Lawrence Armory and Machine Shop, a National Historic Landmark (NHL). Additionally, several industrial mills were present in the general area around the bridge location in the 19th and 20th centuries, making this a unique area of industrial production in Vermont at the time. Given the significant history present in the project area, a field visit was conducted to ascertain what historical archaeological site presence was likely in the general area around the structure. Evidence of mill remains was noted downstream along the banks of Mill Brook, primarily elements of pen stock features and various rusted mill machinery. Although likely to be impacted by construction, special care should be taken to avoid disturbance before a proper historical context can be researched.

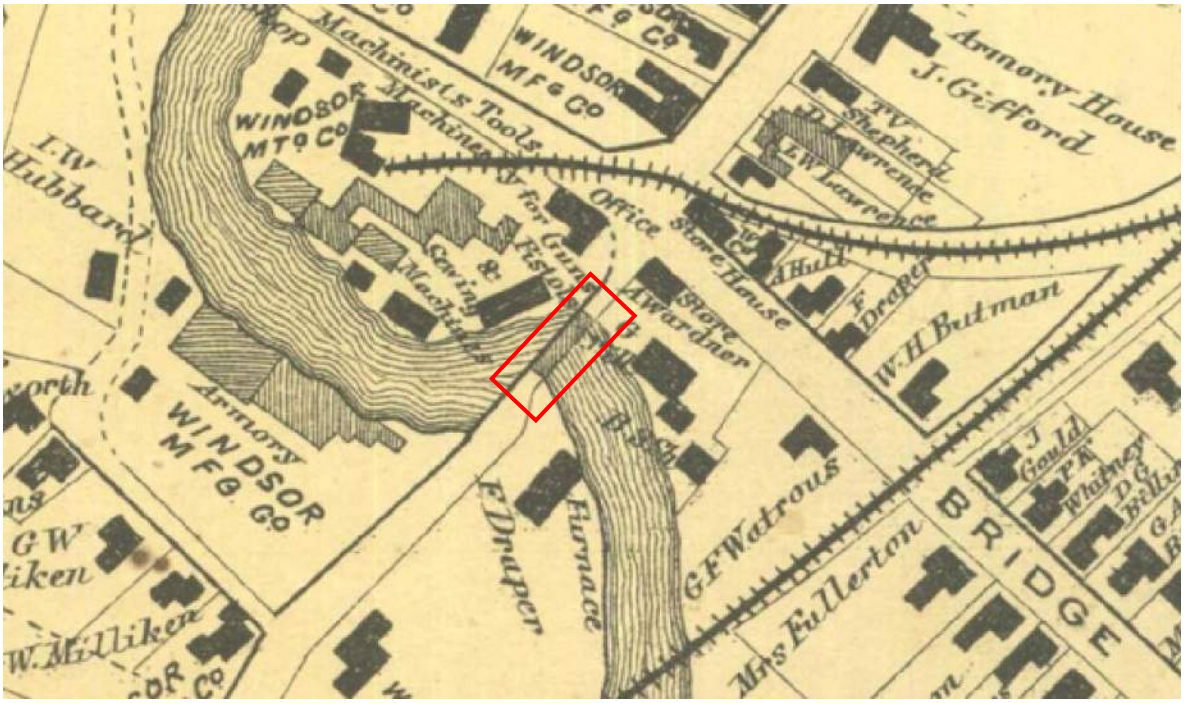


Figure 2: Ca. 1880 Map Showing Mill Locations (Bridge Highlighted in Red).

The yard to the southeast of the bridge that leads to a steep bank leading to the river appears to be undisturbed and has been identified as archaeologically sensitive. Additionally, the land to the northwest that abuts the Windsor Precision Museum has been marked as sensitive due to its relation to the Robbins and Lawrence Armory and Machine Shop. The two northern quadrants exhibit signs of previous disturbances likely related to the demolition of structures in the 20th century.



Figure 3: Archaeologically Sensitive Quadrants.

Archaeological Phase I testing is recommended if construction activities are planned to take place in either quadrant. Feel free to reach out with any questions or comments that may arise as part of the scoping process. The sensitivity lines have been added to the archaeology geodatabase for inclusion in future plans.

Sincerely,

Brennan

Brennan Gauthier | Senior Archaeologist
Vermont Agency of Transportation
Barre City Place, 4th Floor,
219 North Main Street
Barre City, VT 05641
(802) 279-1460

Appendix G: Historic Resource ID

Historic Resources Identification Survey Bridge No. 55 over Mill Brook, U.S. Route 5 Windsor BF 0113(88)

Windsor, Windsor County, Vermont



Prepared for:

Vermont Agency of Transportation
219 North Main Street
Barre, Vermont 05641

Prepared by:

WSP USA Inc.
433 River Street, 7th Floor
Troy, New York 12180

April 5, 2024

Historic Resources Identification Survey
Bridge No. 55 over Mill Brook, U.S. Route 5
Windsor BF 0113(88)

Windsor
Windsor County, Vermont

Prepared for:

Vermont Agency of Transportation
219 North Main Street
Barre, Vermont 05641

Prepared by:

Amber Courselle, Austin White, and Camilla McDonald

WSP USA Inc.
433 River Street, 7th Floor
Troy, New York 12180

April 5, 2024

Abstract

On behalf of the Vermont Agency of Transportation (VTrans), Barre, WSP USA Inc. (WSP) of Troy, New York, completed a historic resources identification survey (ID Report) involving the anticipated future repair and/or replacement of 27 bridges throughout the state. This particular report addresses Windsor Bridge No. 55 over Mill Brook, U.S. Route 5, Windsor County. The scope of work for these resource identification projects includes reconnaissance-level surveys to identify historic buildings, structures, objects, districts, landscapes, and Section 4(f) properties in the preliminary aboveground survey area (survey area) that may be directly, indirectly (including views of the project from the survey area), and/or cumulatively impacted by the potential project. As the forthcoming bridge projects have no specific plans, each bridge and their proximal historic resources were documented in ID Reports to inform the project designers regarding historic and Section 4(f) resources and for National Register of Historic Places eligibility determinations in the Section 106 review. WSP performed no archaeological investigations for these ID Reports.

The survey area extends 39.6 meters (130 feet) around either end of Windsor Bridge No. 55, including all four quadrants, which includes all areas that may be directly impacted by either rehabilitation or replacement of the current bridge and have substantial visibility of the current bridge.

The goal of the survey was to identify (1) buildings, structures, objects, districts, and landscapes in the survey area previously listed in the Vermont State Register of Historic Places/National Register of Historic Places (SR/NRHP) (the criteria for both are identical), (2) previously unsurveyed historic aboveground resources in the survey area that may be eligible for listing in the SR/NRHP, and (3) historic, park, recreational, or refuge Section 4(f) properties. The survey also evaluated the potential effects of the project on viewsheds associated with any SR/NRHP-listed and -eligible properties. As the project is still in the planning stages and may take several years to be implemented, WSP identified properties that meet the 45-year mark for NRHP evaluation.

This reconnaissance-level survey included background research and fieldwork, which was conducted December 5, 2023. All surveys were undertaken in accordance with the guidelines and criteria established by the National Park Service and in 36 Code of Federal Regulations 60.4, and the Advisory Council on Historic Preservation's Reasonable and Good Faith Identification Standard.

The survey area contains a total of eight resources: the NRHP-listed Windsor Village Historic District (WVHD); the American Precision Museum, a National Historic Landmark (NHL); NAMCO Block; the SRHP-listed Bridge No. 55; Mill Brook Historic District (MBHD), Middle/Lower Dam Area, consisting of three contributing resources (including the American Precision Museum); and one previously unsurveyed architectural resource (Table A-1). No historic, park, recreational, or refuge Section 4(f) properties were identified in the survey area.

It is WSP's opinion that the WVHD and the rest of the previously surveyed properties should remain listed in the SR/NRHP. It is also WSP's opinion that Windsor Bridge No. 55 is eligible for the SR/NRHP as it meets the Criterion C registration requirements outlined in the Multiple Property Documentation Form *Metal Truss, Masonry and Concrete Bridges of Vermont, 1820–1978* as an early, well-preserved example of the concrete open spandrel arch type. However, in WSP's opinion the previously unsurveyed resource is not eligible for listing in the SR/NRHP.

NHLs, such as the Robbins and Lawrence Armory and Machine Shop, are judged by the Secretary of the Interior to possess national significance, and agencies are therefore required "to the maximum extent possible undertake such planning and actions as may be necessary to minimize harm to the landmark" (54 USC 306107). Given that the boundary of the Robbins and Lawrence Armory and Machine Shop NHL is

adjacent to Bridge No. 55, there is potential for either the rehabilitation or replacement of the bridge to adversely affect the Armory building and/or other contributing resources within the NHL boundary. Current documentation for the NHL is dated and does not fully account for the significance of landscape features such as the retaining wall, which is close to the abutment for Bridge No. 55. In WSP's opinion VTrans should initiate early consultation with Region 1 of the National Park Service (NPS) about the potential project and, if possible, obtain additional information from the NPS about the significant character-defining elements of the NHL and what resources within the boundary are contributing to the significance of the NHL. If the NPS does not have this information, in WSP's opinion an intensive survey should be conducted in consultation with the VDHP and the NPS to understand the significant resources within the NHL.

Other historic resources, such as the NAMCO Block, are near the bridge. Precautions should be taken to limit both direct effects to the building from construction equipment and activities as well as potential vibration effects from construction activities (whether for bridge rehabilitation or replacement). As Bridge No. 55 is SRHP-listed/NRHP-eligible, rehabilitation of the bridge should be considered first in any future project to avoid and minimize potential adverse effects to the historic bridge and surrounding historic properties. Bridge rehabilitation should follow the Secretary of the Interior's Standards to preserve the character-defining features and avoid an adverse effect. Replacement of the bridge would result in an adverse effect. Design of the replacement bridge could mimic the design of the current bridge or previous bridges at the site to be sympathetic to the adjacent WVHD.

TABLE A-1: NRHP ELIGIBILITY RECOMMENDATIONS FOR PREVIOUSLY AND NEWLY IDENTIFIED HISTORIC RESOURCES IN THE SURVEY AREA

MAP ID (Windsor-)	RESOURCE NAME	LOCATION	PREVIOUS ELIGIBILITY	WSP ELIGIBILITY OPINION	PHOTOGRAPH
1	Windsor Village Historic District	Multiple, Windsor	Listed, SR/NRHP	Listed, SR/NRHP	
2	Windsor Bridge No. 55 over Mill Brook	U.S. 5, Windsor	Listed, SRHP	Listed, SRHP; Eligible, NRHP	
3	American Precision Museum	196 Main Street, Windsor	Listed, SR/NRHP (NHL; MBHD, Middle/Lower Dam Area)	Listed, SR/NRHP (NHL; MBHD, Middle/Lower Dam Area)	

MAP ID (Windsor-)	RESOURCE NAME	LOCATION	PREVIOUS ELIGIBILITY	WSP ELIGIBILITY OPINION	PHOTOGRAPH
4	NAMCO Block	1-17 Union Street, Windsor	Listed, SR/NRHP	Listed, SR/NRHP	
5	Mill Brook Historic District, Middle/Lower Dam Area	Mill Brook Falls, Windsor	Listed, SRHP	Listed, SRHP; Eligible, NRHP	
5a	Site No. 14, Grist Mill Ruins	North Bank of Mill Brook, East of US 5, Windsor	Listed, SRHP	Listed, SRHP; Eligible, NRHP	
5b	Site No. 15, Foundry Wheel House Ruins	South Bank of Mill Brook, East of U.S. 5, Windsor	Listed, SRHP	Listed, SRHP; Eligible, NRHP	
6	Multifamily Residence	189 Main Street, Windsor	Not evaluated	Not Eligible	

Table of Contents

Page

Abstract i
List of Figures v
List of Tables v

I. Introduction 1
A. Project Description 1
B. Objectives 1
II. Methodology 4
III. Historic Context 5
A. Historical Overview of Windsor County 5
B. Historical Overview of the Town of Windsor 5
1. Bridge Street 6
2. Main Street 6
IV. Survey Results 7
A. Vermont SRHP/NRHP-Listed Properties 7
1. Windsor Village Historic District (Windsor-1) 7
2. Windsor Bridge No. 55, US Route 5 over Mill Brook, Windsor (Windsor-2) 10
3. American Precision Museum, 196 Main Street, Windsor (Windsor-3) 11
4. NAMCO Block, 1-17 Union Street, Windsor (Windsor-4) 13
5. Industrial Sites on Mill Brook Historic District, Middle/Lower Dam Area, Mill Brook Falls, Windsor (Windsor-5) 13
a. Site No. 14, Grist Mill Ruins, North Bank of Mill Brook, East of U.S. Route 5, Windsor (Windsor-5a) 14
b. Site No. 15, Foundry Wheel House Remains, South Bank of Mill Brook, East of U.S. Route 5, Windsor (Windsor-5b) 15
B. Newly Surveyed Resource 16
1. Multifamily Residence, 189 Main Street, Windsor (Windsor-6) 16
V. Conclusions and Recommendations 17
VI. References Cited 20

List of Figures

Page

1 Location of Project 2
2 Project Survey Area 3
3 Location of Surveyed Architectural Resources in Survey Area 8
4 Windsor Village Historic District 9
5 NHL Boundary for American Precision Museum (Rettig 1974) 12
6 Industrial Sites in Mill Brook Historic District, Middle/Lower Dam Area 14

List of Tables

Page

A-1 NRHP Eligibility Recommendations for Previously and Newly Identified Historic Resources in the Survey Area ii
1 Previously and Newly Identified Historic Resources in Survey Area 7
2 NRHP Eligibility Recommendations for Previously and Newly Identified Historic Resources in Survey Area 17

I. Introduction

A. Project Description

On behalf of the Vermont Agency of Transportation (VTrans), Barre, WSP USA Inc. (WSP) of Troy, New York, completed a historic resources identification survey (ID Report) involving the anticipated future repair and/or replacement of 27 bridges throughout the state. This particular report addresses Windsor Bridge No. 55 over Mill Brook, U.S. Route 5, Windsor County. The scope of work for these resource identification projects includes reconnaissance-level surveys to identify all historic and Section 4(f) properties in the preliminary aboveground survey area (survey area), including the bridge, that may be directly, indirectly (including views of the project from the survey area), and/or cumulatively impacted by the potential project.

The project is located along U.S. Route 5 in the Town of Windsor, Windsor County (Figure 1). The survey area extends 39.6 meters (130 feet) around either end of the bridge, including all four quadrants based on the visibility and proximity with regard to the structure's dimensions (Figure 2).

B. Objectives

The goal of the survey was to identify (1) buildings, structures, objects, districts, and landscapes in the survey area previously listed in the Vermont State Register of Historic Places/National Register of Historic Places (SR/NRHP) (the criteria for both are identical), (2) previously unsurveyed historic aboveground resources in the survey area that may be eligible for listing in the SR/NRHP, and (3) historic, park, recreational, or refuge Section 4(f) properties. The survey also evaluated the potential effects of the project on viewsheds associated with any SR/NRHP-listed and -eligible properties. The investigation included background research and fieldwork. Fieldwork was conducted on December 5, 2023.

Determinations of eligibility for the NRHP followed the guidelines and criteria established by the National Park Service (NPS) (NPS 2002) and in 36 Code of Federal Regulations (CFR) 60.4, and the Advisory Council on Historic Preservation's Reasonable and Good Faith Identification Standard. In 2001 the Vermont Division for Historic Preservation (VDHP) changed the Vermont SRHP criteria to make them identical to the NRHP criteria, and all resources then listed in the Vermont SRHP were deemed eligible for the NRHP, creating a single class of historic properties and thereby streamlining the historic preservation permitting process in Vermont. As the project is still in the planning stages and may take several years to be implemented, WSP identified properties that meet the 45-year age mark for evaluation for the NRHP.

This report contains six chapters. Following the introduction in Chapter I, Chapter II describes the survey's methodology. Chapter III provides the historic context for the project vicinity. Chapter IV describes the survey results, and the conclusions and recommendations appear in Chapter V. Chapter VI contains the references cited.

This investigation was conducted under the direction and supervision of WSP Contract Manager Jospeh Tomberlin. Senior Architectural Historian Camilla McDonald supervised the QA/QC process. Architectural Historian Austin White conducted fieldwork, and Architectural Historian Amber Courselle conducted research and report writing with assistance from Mr. White. Principal Cartographer/GIS Analyst Jacqueline L. Horsford prepared the graphics. Technical Editor Evelyn Dsouza edited the report.

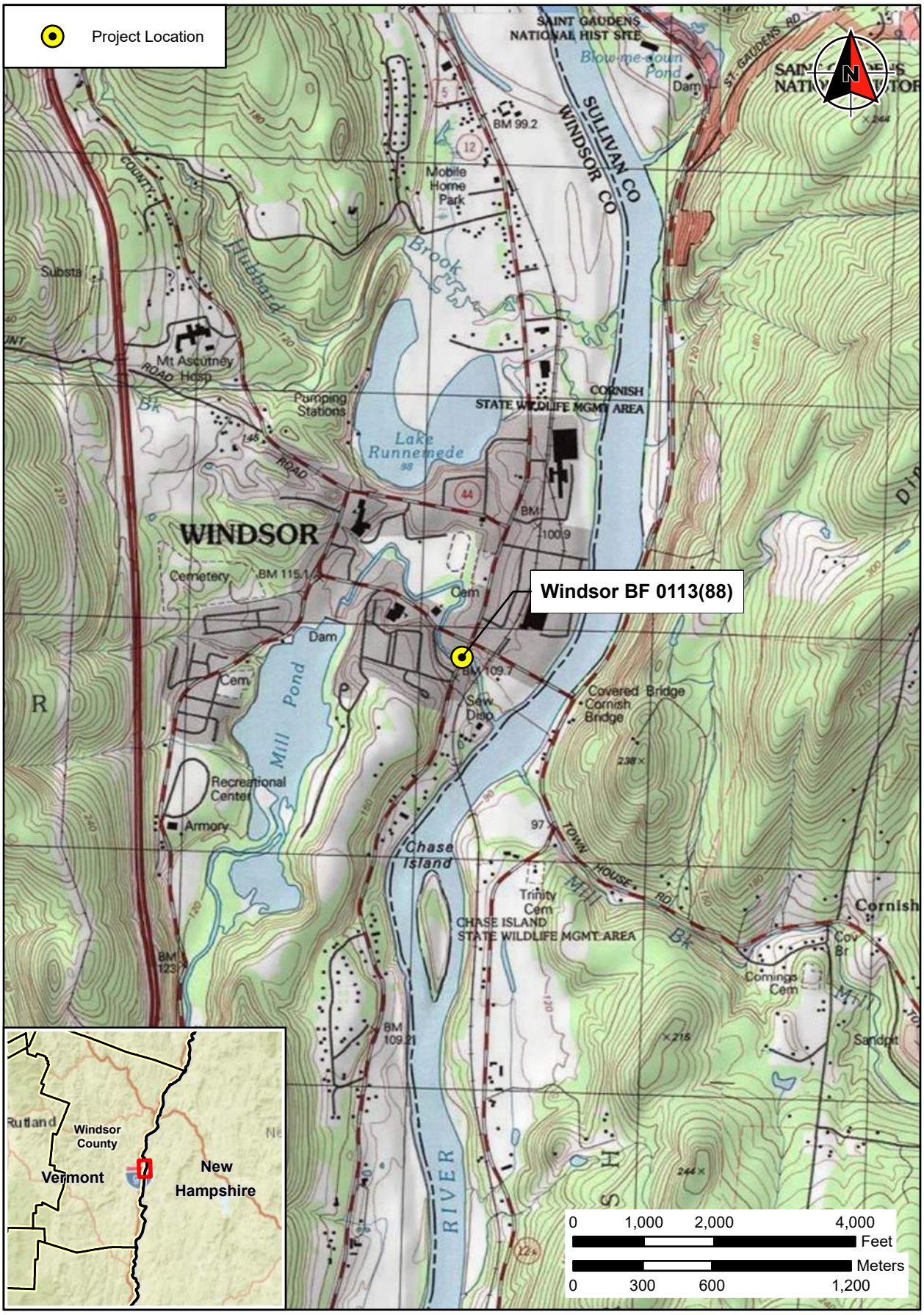


FIGURE 1: Location of Project (ESRI USA Topo Maps 2019 [USGS Windsor])

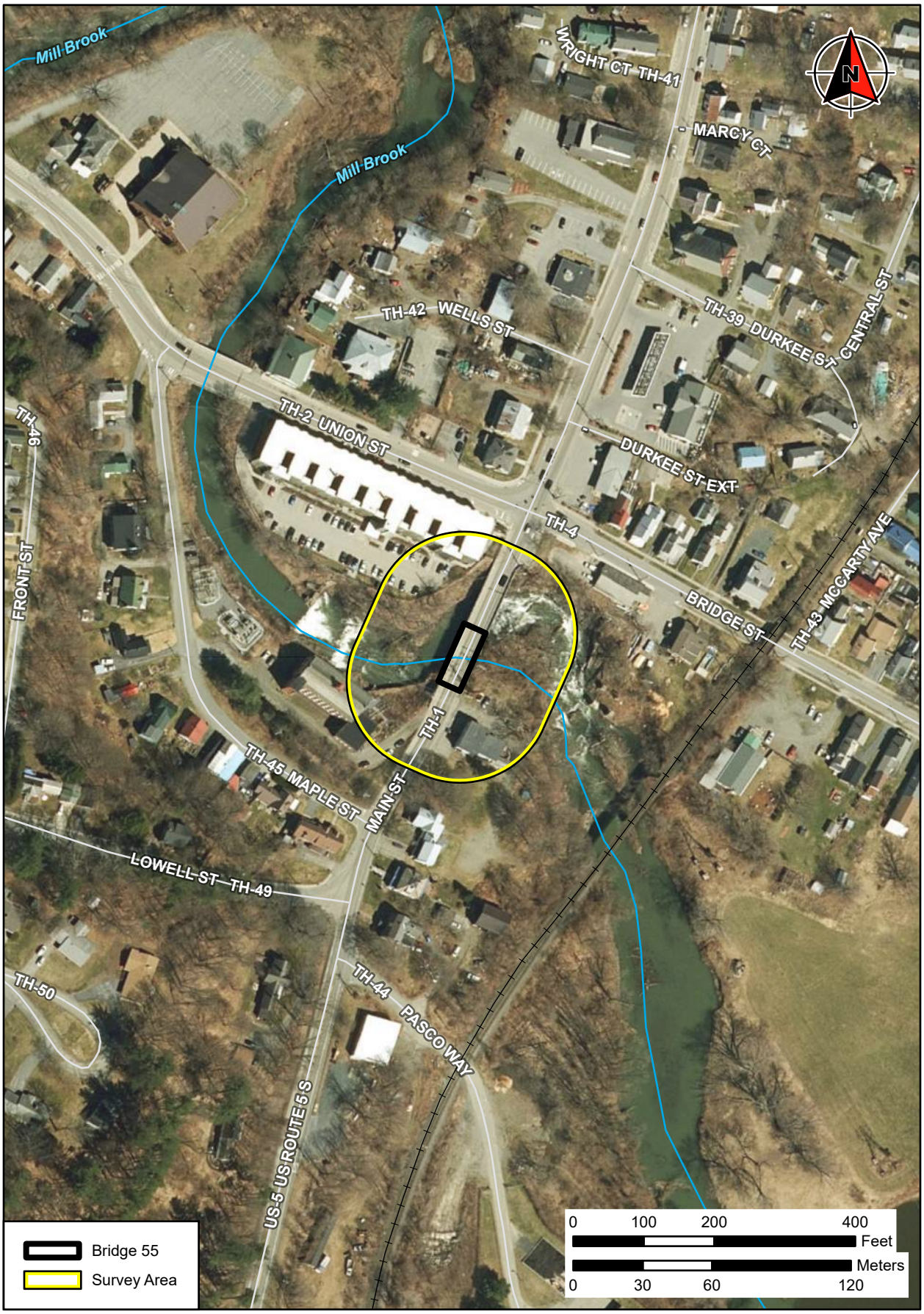


FIGURE 2: Project Survey Area (ESRI World Imagery 2022)

II. Methodology

WSP's primary task for this survey was to identify historic properties in the survey area listed in or eligible for listing in the NRHP. WSP reviewed site files at VDHP, identifying documented resources in the survey area that are already either listed in or eligible for listing in the NRHP. Location information on the identified properties was mapped, and nomination forms and eligibility determination data were copied for comparison against current conditions during the field survey. Available historic context data on the development of the community in the survey area were gathered from VDHP files to assist in the evaluation of additional historical resources identified during the field survey.

The field survey checked the continued existence of the historic properties identified during the site file check and collected information on each property's architectural and historical integrity and eligibility for listing in the NRHP. Each resource in the survey area was documented through digital photographs and narrative field notes. Some properties were not visible from the right-of-way, and those properties were examined through historical and current aerial photographs to determine their age.

WSP followed the NRHP criteria in evaluating each resource. According to the NRHP criteria for evaluation, properties may be eligible for the NRHP if:

A. they are associated with events that have made a significant contribution to the broad patterns of our history; or
B. they are associated with the lives of significant persons in our past; or
C. they embody the distinctive characteristics of a type, period, or method of construction, or that represent the work of a master, or that possess high artistic values, or that represent a significant and distinguishable entity whose components may lack individual distinction; or
D. they have yielded or may be likely to yield, information important in history or prehistory [NPS 2002:7].

WSP's assessments of eligibility were further guided by the Multiple Property Documentation (MPD) form *Multiple Property Documentation for Agricultural Resources in Vermont* (VDHP 1991), which establishes standards of integrity for listing agricultural resources in Vermont in the SR/NRHP.

Results of the background research and field survey were analyzed under the established criteria to determine the NRHP eligibility of each architectural resource, whether previously recorded or newly identified.

III. Historic Context

A. Historical Overview of Windsor County

The first permanent British settlement in Vermont was Fort Dummer, built in 1724 in Brattleboro (Vermont State Parks 2020); however, Euro-American settlement did not begin in earnest until the 1770s. With Vermont's independence from New York in 1777 came the reorganization of county boundaries. By 1781 Cumberland County had been divided into Orange, Windham, and Windsor counties (The Newberry Library 2008). Permanent settlement in Windsor County began in the mid- to late 1700s and concentrated along the Ottaquechee and Connecticut rivers. The population peaked in the 1830s and steady declined from 1840 to 1890, dropping 21 percent before again steadily rising into the twenty-first century.

Woodstock became the shire town of Windsor County, and Hartford grew into the county's most populous town; Chester was the first town, chartered in 1754 (Aldrich and Holmes 1891:102). Chester holds a unique place in the history of the United States of America as the first town to declare its independence from Great Britain, in 1774, two years before the rest of the country (Virtual Vermont 2007). The landscape of the county is marked by many hills and valleys, with lands excellent for pasturage. Farmers in in the region kept large flocks of sheep in the early part of the nineteenth century and cows in the latter part. During the nineteenth century the farmers of Chester also produced rye, corn, barley, oats, peas, beans, potatoes, and other crops.

Windsor County has historically ranked as a leader in agricultural production in the state. The cultivation of hops for brewing beer and other malted beverages became a popular crop in the county and throughout the eastern and northeastern counties of Vermont by the mid-1800s. The hop industry declined at the turn of the century. Windsor County also prospered as a leader in potato growing, sheep raising, and cheese production during the nineteenth century (VDHP 1989). Agricultural census data show that the number of farms in the county peaked in 1880 at 4,455 and dropped to 3,753 by 1890; however, the county still led the state in number of farms (United States [U.S.] Department of the Interior 1880:171; 1890:189). The population was back on the rise by the mid-1950s, a response to the growing tourism industry, and farms had decreased to 1,609. Today the county ranks third, behind Franklin and Addison counties, in the state in number of farms (United States Department of Agriculture [USDA] 2017: Vermont 223–226).

B. Historical Overview of the Town of Windsor

[This section is taken from the Windsor Village Historic District (Additional Documentation and Boundary Increase, 2014) National Register of Historic Places Registration Form (Papazian, 2014).]

The history and growth of the Town of Windsor begins with the industry established at Mill Brook in the 1760s, with its three falls harnessed for waterpower. The falls formed the basis for the building of a community by providing waterpower for subsistence industries like grist and sawmills. After the initial establishment of these two types of mills, the waterpower available on the Mill Brook made possible the development of early commercial industry. The early mills were soon joined by local industries producing ironwork, shoes, leather goods, textiles, and other products. In the 1830s, the new concept of precision manufacturing made Windsor a leader of the nascent machine tool industry by producing rotating pumps, firearms, sewing machines and tools to build the new machines. The new "American System" of interchangeable machined parts was created upon the skill and genius of Windsor engineers and machinists such as Lemuel Hedge, Richard Lawrence, Frederick Howe, George Gridley, and Frank Cone. The inventions and innovations coming out of the Windsor machine shops in the late 19th century were to be the foundation for Windsor's industrial growth and the spread of the new technology throughout the United States. The Robbins & Lawrence Armory and Machine Shop, located on the banks of the Connecticut River immediately south of the Windsor Village Historic District, was built in 1846 to fabricate rifles with

interchangeable parts. It played an essential role in the industrial development of the village (designated a National Historic Landmark in 1966).

In the 20th century, the proximity to water power was no longer a factor for the machine tool industry in Windsor and the Windsor Machine Company relocated and expanded to the river banks near the railroad. The huge new plant was opened in 1910 and the production of automatic lathes increased dramatically during World War I. By 1916, the company was acquired by the National Acme Company of Cleveland, which continued and greatly expanded the construction and rehabilitation of housing for a very large workforce. Also in 1916, The Cone Automatic Lathe Co. began production in its new village plant a bit further north of National Acme on the railroad line. The wartime production of these plants during World War I changed the face of Windsor with an explosion of population. The production of these plants was also essential to the war effort during World War II, although by then the National Acme Plant had become the shoe products division of the Goodyear Tire and Rubber Co. Machine tool and other industries, such those at Windsor, were given top priority in national security.

There were other factors that led to the growth and prosperity evidenced by Windsor Village in the late eighteenth and nineteenth century. The village owes its development and growth to a number of important determining stimuli. In 1777, representatives of the New Hampshire and New York Grants, meeting in the Old Constitution House (#78), adopted the first constitution of the Republic of Vermont. From 1777 to 1808 the village was the part-time capitol of Vermont, and from 1778 to 1786 was the county seat of Windsor County. The result was an early influx of people with professional skills and training. Lawyers, physicians, merchants, businessmen and entrepreneurs recognized the political and economic opportunities in the new republic's and then state's seat of government. In 1809, the first Vermont State Prison was erected in the village on State Street. The prison supported numerous local commercial services and supplied a local labor force for some of the village's manufacturing concerns. Windsor's location on the Connecticut River, and the later infusion of the Vermont Central Railroad, provided the village's commercial and manufacturing prosperity by providing river transportation and eventually, direct rail transportation with the rest of New England and the eastern United States.

1. Bridge Street

Just above the Main and Bridge Street intersection, Mill Brook provided power for the machine shops that were established toward mid-century (ca. 1845). A map of Windsor village showing new construction between 1853 and 1869 indicates that only one new building was erected on Bridge Street during that period, increasing the number of buildings on the already well developed street from 23 to 24. Presumably, then, the majority of the houses already existed at the start of Windsor's industrial boom. One might also speculate that Bridge Street's modestly decorated, gable-front vernacular houses, in such close proximity to the machine shops, were residences for the skilled machinists or their supervisors.

2. Main Street

U.S. Route 5 is the northern continuation of Main Street and was originally known simply as "the River Road" or the "Road to Hartland" because it was the main north-south corridor paralleling the Connecticut River. It was likely Windsor's first main road and was part of the Connecticut River Turnpike, chartered in 1800 and extending from Bellows Falls to Thetford.

IV. Survey Results

The survey area extends 39.6 meters (130 feet) around either side of Windsor Bridge No. 55 over Mill Brook, on U.S. Route 5. The survey area consists of a medium-density village setting.

WSP identified seven previously surveyed architectural properties that are 45 years old or older in or adjacent to the survey area (Figure 3; Table 1). These are the Windsor Village Historic District (WVHD), the American Precision Museum (a National Historic Landmark [NHL]), NAMCO Block, the SRHP-listed Bridge No. 55, and Mill Brook Historic District (MBHD), Middle/Lower Dam Area consisting of three contributing resources (including the American Precision Museum).

WSP also identified one previously unsurveyed architectural resource in or adjacent to the survey area (see Figure 3 and Table 1). This is a multifamily residence at 189 Main Street.

WSP did not identify any historic, park, recreational, or refuge Section 4(f) resources in or adjacent to the survey area.

TABLE 1: PREVIOUSLY AND NEWLY IDENTIFIED
HISTORIC RESOURCES IN SURVEY AREA

MAP ID (Windsor-)	SR/NRHP ELIGIBILITY	NAME	ADDRESS
1	Listed, SR/NRHP	Windsor Village Historic District	Multiple, Windsor
2	Listed, SRHP	Windsor Bridge No. 55 over Mill Brook	U.S. 5, Windsor
3	Listed, SR/NRHP (NHL; MBHD, Middle/Lower Dam Area)	American Precision Museum	196 Main Street, Windsor
4	Listed, SR/NRHP	NAMCO Block	1-17 Union Street, Windsor
5	Listed. SRHP	Mill Brook Historic District, Middle/Lower Dam Area	Mill Brook Falls, Windsor
5a	Listed, SRHP	Site No. 14, Grist Mill Ruins	North Bank of Mill Brook, East of U.S. 5, Windsor
5b	Listed, SRHP	Site No. 15, Foundry Wheel House Ruins	South Bank of Mill Brook, East of U.S. 5, Windsor
6	Not evaluated	Multifamily Residence	189 Main Street, Windsor

A. Vermont SRHP/NRHP-Listed Properties

1. Windsor Village Historic District (Windsor-1)

The WVHD is a residential, commercial, and industrial district originally listed in the NRHP in 1975 that has been expanded twice—first in 1997 and again in 2014. The 1975 district contained 78 contributing buildings and excluded auxiliary and non-contributing buildings. The 1997 amendment increased the property count to 81 total contributing buildings with the inclusion of three residential properties. The most recent enlargement was substantial and encompassed adjoining residential, commercial, and industrial areas along Main Street, Depot Avenue, State Street, and Village Green minor secondary streets, extending east

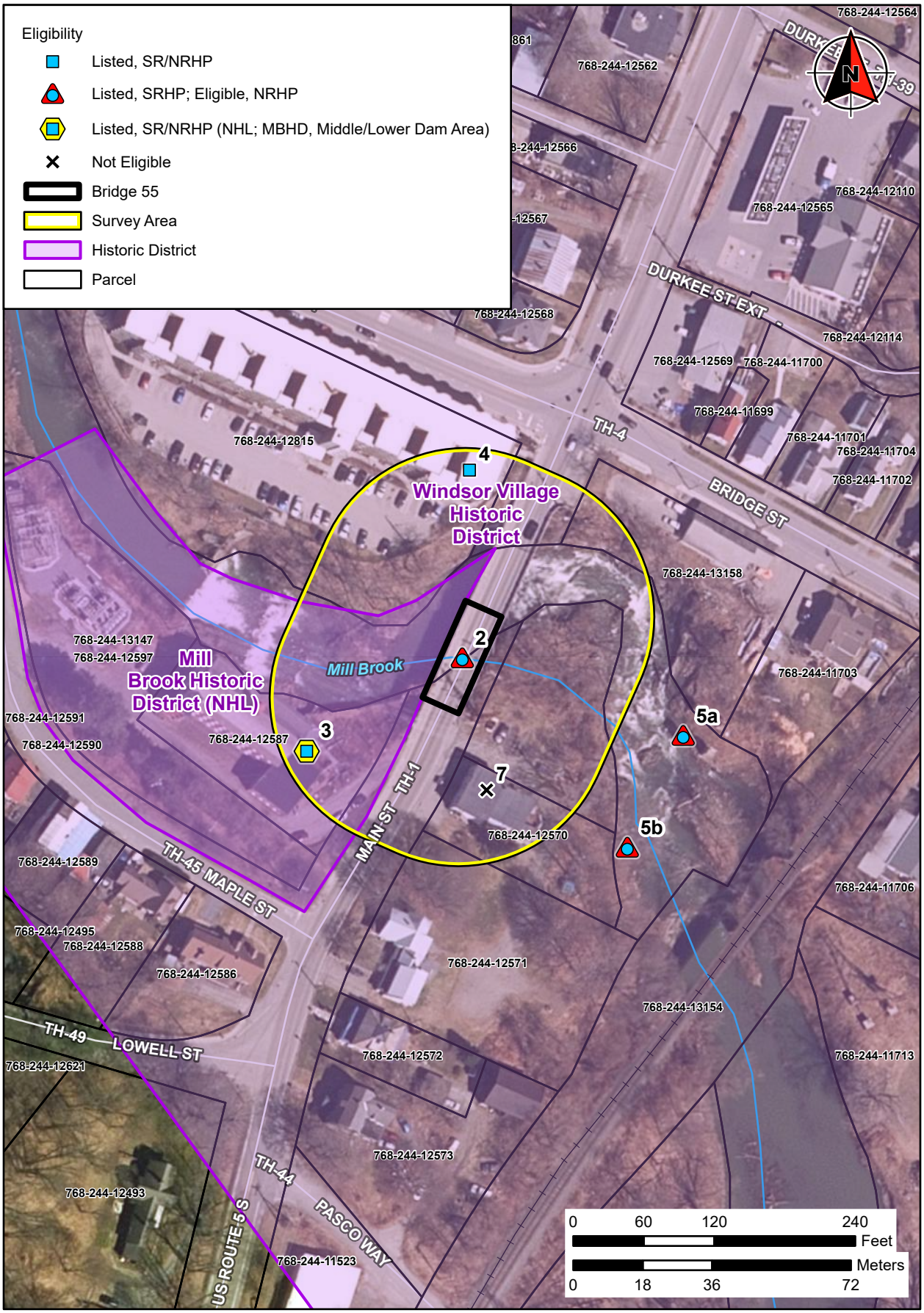


FIGURE 3: Location of Surveyed Architectural Resources in Survey Area (ESRI World Imagery 2022)

to the Connecticut River and west along State Street, which includes Paradise Park (Figure 4). The updated property inventory constitutes 238 contributing buildings, two contributing structures, three contributing sites, 58 non-contributing buildings, and two non-contributing structures. Significant under Criteria A and C, the district has a period of significance from 1770 to 1958: "[The Village of Windsor] represents a type of physical town development that is both significant in the early history of Vermont as well as representative of the later growth of precision industries and machine tool manufacturing in the Connecticut River Valley" (Papazian 2014:129).

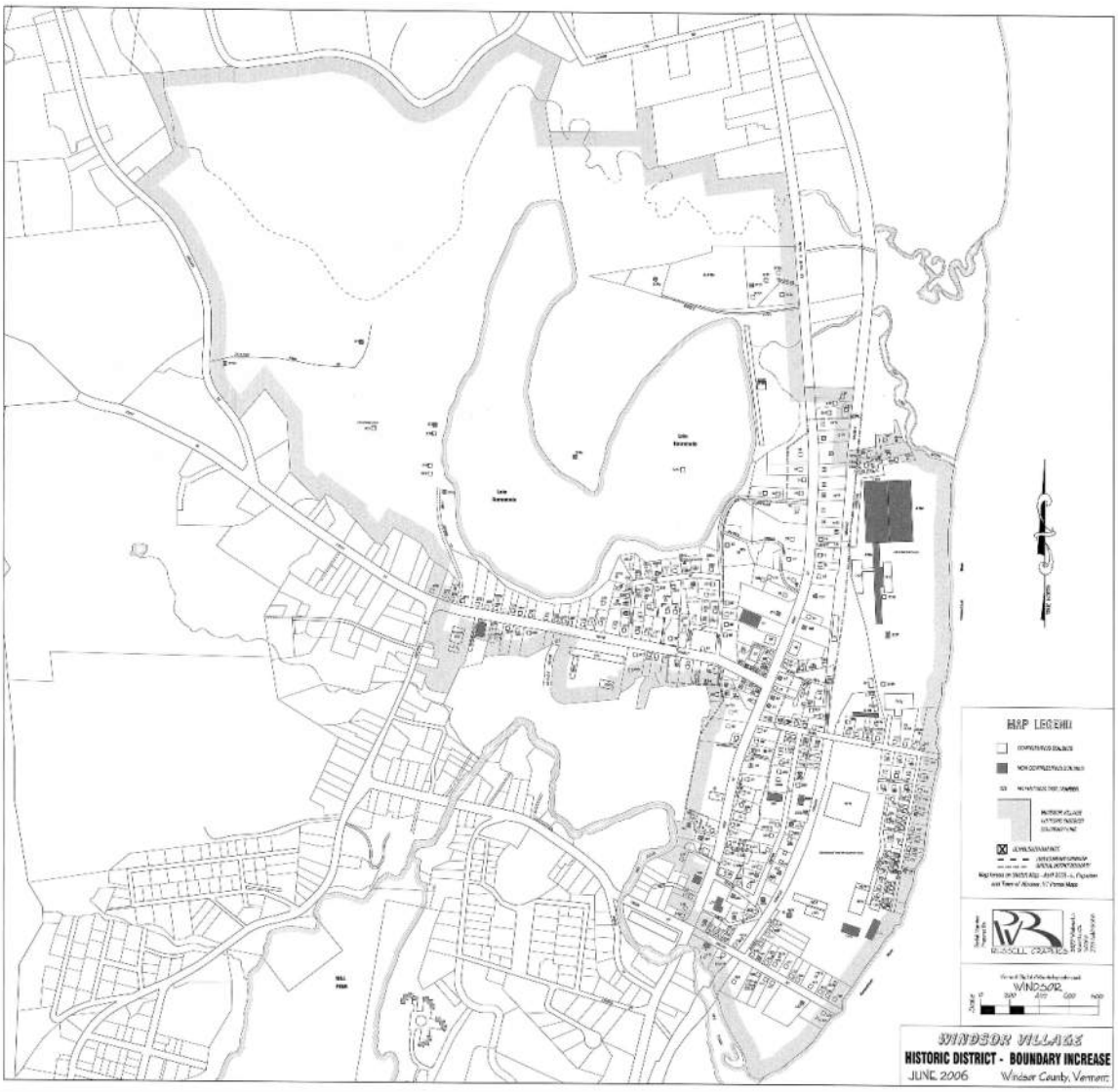


FIGURE 4: Windsor Village Historic District (Papazian 2014)

2. Windsor Bridge No. 55, US Route 5 over Mill Brook, Windsor (Windsor-2)

Resource Name	Windsor Bridge No. 55, U.S. Route 5 over Mill Brook
VTrans ID No.	BF 0113(88)
Location	U.S. Route 5, Windsor
Parcel ID	N/A
Date(s) of Construction	1930; 1943
NRHP Recommendation	Remain Listed, SRHP; Eligible, NRHP



Overview of Bridge No. 55 from NHL



Retaining Wall Adjacent to Bridge



Retaining Wall and Tower Base Adjacent to Bridge

Windsor Bridge No. 55 over Mill Brook is a one span, 88-foot-long, 33.5-foot-wide concrete open spandrel arch bridge. The structure carries two lanes of U.S. Route 5 over Mill Brook. The concrete cast-in-place deck has concrete railings on both sides composed of urn-shaped balusters topped by a large, wide capstone. The railings are divided in 10-foot intervals by square piers, all of which bear brass plates of previous lamp fixtures. The northwest pier is incised with the construction date. A sidewalk runs along the west side (southbound) of the bridge. The concrete superstructure consists of two central elliptical ribbed arches which taper toward the crowns, joined by four struts. The superstructure is supported by eight large open double tee-beams, all with crossbeams shaped, from bottom to top, as large and small coves, then filleted at the plain fascia. The outer four beams rest atop the abovementioned struts. The bridge is anchored by abutments embellished with tapered pilasters and shallow coves. The bridge is in generally fair condition. The deck exhibits areas of efflorescence leakage, scaling, and rusting staining ranging from moderate to heavy. Nearly every rail post has verticals cracking, along with the top rail where efflorescence staining is present. The fascia suffers from varying degrees of scaling with areas of cracking. The superstructure is in overall good condition featuring areas of light cracking, minor exposure of steel reinforcing spalling, scaling, rust staining, and effloresce leaking and staining. The abutments are in generally satisfactory condition and exhibit varying egress of efflorescence leakage, scaling, and honeycombing. The wingwalls and footings have hairline cracks, with scaling on the latter.

Windsor Bridge No. 55 over Mill Brook was listed in the SRHP as one of only four concrete open spandrel arch bridges in Vermont. Constructed in 1930, it is the only bridge in the state constructed by Gerald B. Owens and the only example of the company's work in the state. Windsor Bridge No. 55 meets Criterion C registration requirements outlined in the MPD *Metal Truss, Masonry and Concrete Bridges of Vermont, 1820–1978* (Louis Berger 2018:F59–F61) as an early, well-preserved example of the last surviving concrete open spandrel arch type bridge in the state (the other is the bi-state Vilas Bridge in Bellows Falls).

Bridge No. 55 has a high degree of integrity. The bridge has not been moved or replaced since its construction, and the surrounding area's medium-density residential, commercial, and industrial character has remained largely unchanged since 1930; it therefore retains its integrity of location and setting. Bridge No. 55 retains its integrity of design and materials as a good example of a largely unaltered, early twentieth-century concrete open spandrel bridge with extant character-defining arches and balustrade railings. The bridge's intact arches and balustrade railings demonstrate the artistic and stylistic expressions incorporated into early twentieth-century bridge designs and it therefore retains its integrity of workmanship. The bridge's integrity of feeling and association also remain largely intact and still convey a sense of early twentieth-century bridge design and transportation planning in a medium-sized Vermont community. In consideration of these factors, it is WSP's opinion that the Windsor Bridge No. 55 over Mill Brook remains eligible for listing in the SRHP and is eligible under Criterion C for listing in the NRHP.

3. American Precision Museum, 196 Main Street, Windsor (Windsor-3)

Resource Name	American Precision Museum
VTrans ID No.	N/A
Location	196 Main Street, Windsor
Parcel ID	560196-000
Date(s) of Construction	1846
NRHP Recommendation	Remain Listed as an NHL



The Robbins and Lawrence Armory and Machine Shop, now the American Precision Museum, has experienced very few changes since its listing as an NHL in 1966. The coursed rubble foundation, brick walls, multipaned windows, and slate roof with dormers and central hexagonal cupola remain intact. Thus, the NHL retains excellent integrity of location, design, setting, materials, workmanship, feeling, and association, and still conveys its historic significance as a mid-nineteenth-century industrial facility.

Significant in the areas of industry and commerce, the building was the site of the design and manufacture of machine tools that "accelerated the industrial revolution in America by improving the production of interchangeable parts and thus stimulating mass production" (Rettig 1974). The boundary of the NHL, which runs along the edge of Main Street and Bridge No. 55, includes the entire parcel owned by the museum (Figure 5). It is WSP's opinion that the American Precision Museum should remain listed in the SR/NRHP and designated as an NHL.

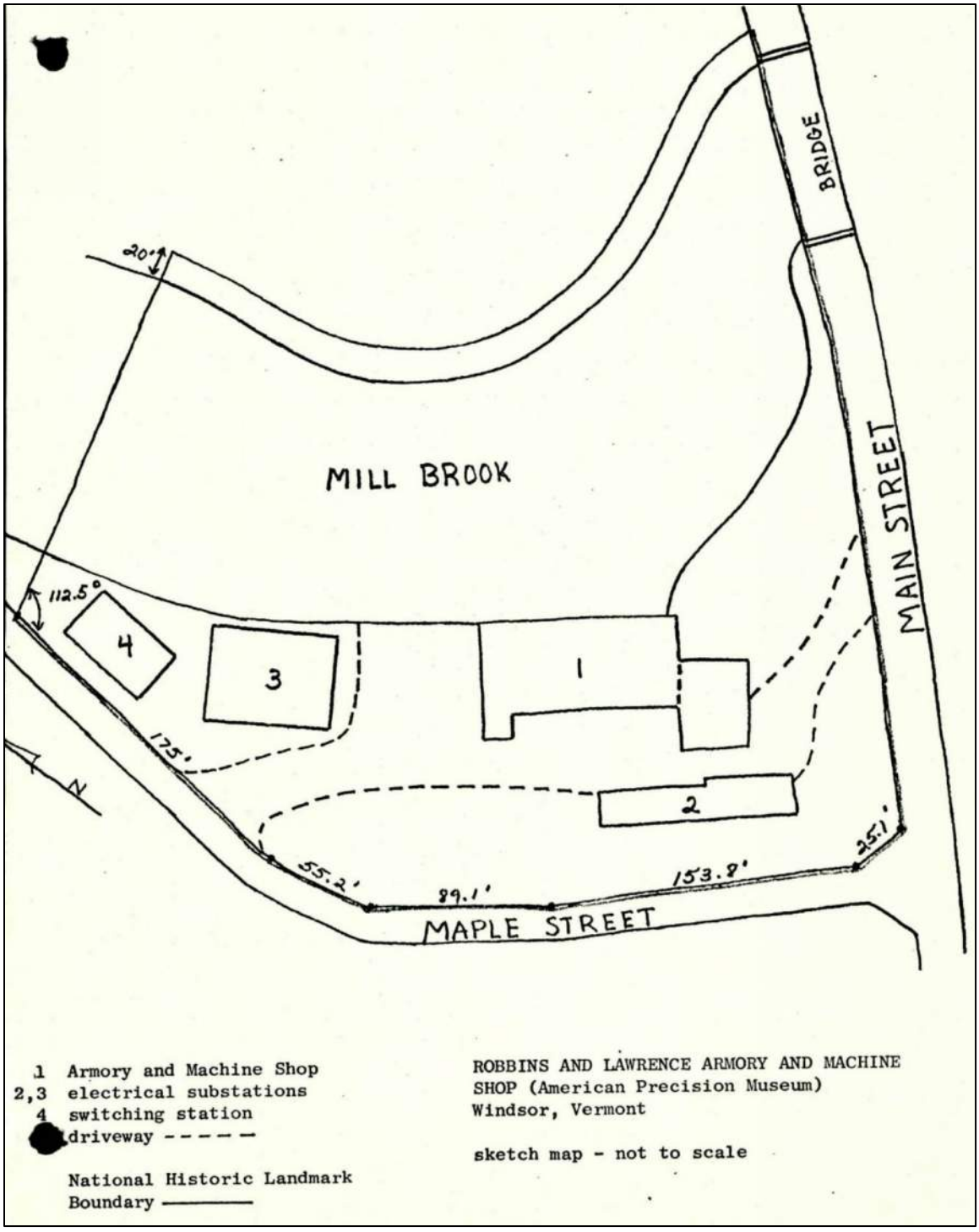


FIGURE 5: NHL Boundary for American Precision Museum (Rettig 1974)

4. NAMCO Block, 1-17 Union Street, Windsor (Windsor-4)

Resource Name	NAMCO Block
VTrans ID No.	N/A
Location	1-17 Union Street, Windsor
Parcel ID	790001-000
Date(s) of Construction	1920–1922
NRHP Recommendation	Remain Listed, SR/NRHP



This unique block of nine double-bow front row houses is four stories high and constructed of brick. Though some windows have been replaced, they retain the same glazing pattern as the original sash. The property retains good overall integrity with modifications limited to some replacement, sympathetic windows. Therefore, it is WSP's opinion that the NAMCO Block should remain listed in the SR/NRHP.

5. Industrial Sites on Mill Brook Historic District, Middle/Lower Dam Area, Mill Brook Falls, Windsor (Windsor-5)

The Mill Brook in Windsor, with its three natural falls creating a 60-foot drop in one-third of a mile, was the reason for the location of the settlement of Windsor in 1764. Although the first settler was apparently Captain Steel Smith, the first industrial development was Israel Curtis's sawmill and log dam at the upper falls and gristmill and log dam at the lower falls both constructed by 1769. Soon afterward, another sawmill was also built by Thomas Cooper above the gristmill at the "middle falls." For many years, these two establishments were the most important in town. The Middle/Lower Dam Area is concentrated around the two dam sites on the Mill Brook and Union Street (Papazian 1999:1, 7) (Figure 6).

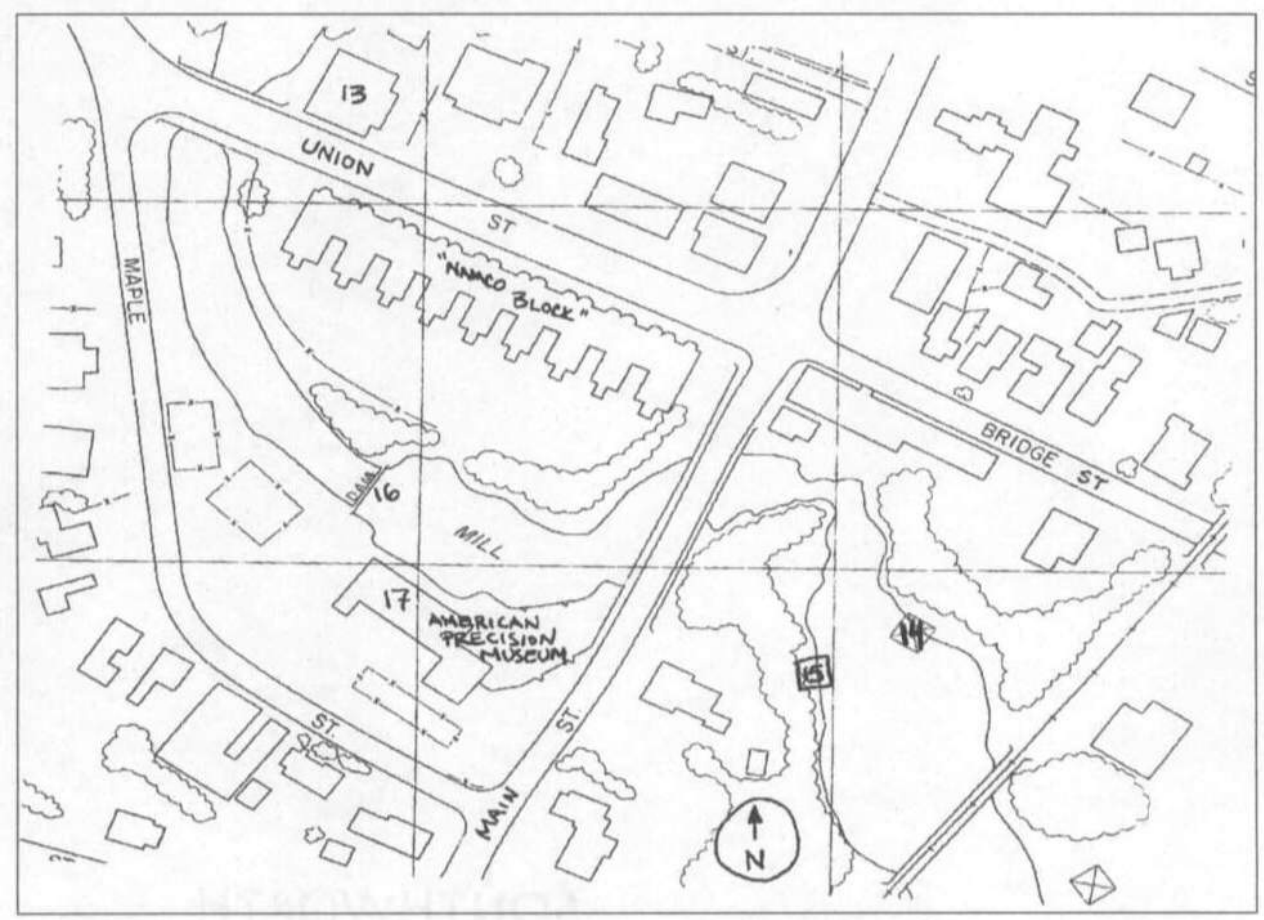


FIGURE 6: Industrial Sites in Mill Brook Historic District, Middle/Lower Dam Area (Papazian 1999)

a. *Site No. 14, Grist Mill Ruins, North Bank of Mill Brook, East of U.S. Route 5, Windsor (Windsor-5a)*

Resource Name	Site No. 14, Grist Mill Ruins
VTrans ID No.	N/A
Location	North Bank of Mill Brook, East of U.S. Route 5, Windsor
Parcel ID	N/A
Date(s) of Construction	ca. 1791
NRHP Recommendation	Remain Listed, SRHP; Eligible, NRHP



The ruins are a contributing resource of the SRHP-listed Industrial Sites on MBHD, Middle/Lower Dam Area. The remains consist of a rubble stone foundation and the main wall of a former gristmill. The rebuilt foundation is roughly square in plan with 20- to 25-foot-tall, 3-foot-thick stone walls. Located on the western wall facing the dam are two stone arches, one open at water level and the other higher and infilled with stones. The mill would have presumably drawn power from Mill Brook using a lower wood dam. Associated features include stone piers, which supported a penstock.

The resource retains integrity of location, setting, feeling, and association, and is representative of Windsor's early industrial history and economic development. Therefore, it is WSP's opinion that Site No. 14, Grist Mill Ruins, should remain listed in the SRHP and therefore eligible for listing in the NRHP.

b. *Site No. 15, Foundry Wheel House Remains, South Bank of Mill Brook, East of U.S. Route 5, Windsor (Windsor-5b)*

Resource Name	Site No. 15, Foundry Wheel House Remains
VTrans ID No.	N/A
Location	South Bank of Mill Brook, East of U.S. Route 5, Windsor
Parcel ID	N/A
Date(s) of Construction	ca. 1850
NRHP Recommendation	Remain Listed, SRHP; Eligible, NRHP



This resource contributes to the SRHP-listed Industrial Sites on MBHD, Middle/Lower Dam Area. The remnants of the foundry are in poor condition and consist of a portion of a dry laid fieldstone foundation. The resource's location and orientation indicate that it supported a substantial industrial facility complete with mills, dams, and a turbine.

Despite its condition, the resource is representative of Windsor's mid-nineteenth-century industrial development. Furthermore, it may be strongly linked to the nearby American Precision Museum. It is WSP's opinion that Site No. 15, Foundry Wheel House Remains, should remain listed in the SRHP and therefore eligible for listing in the NRHP.

B. Newly Surveyed Resource

1. Multifamily Residence, 189 Main Street, Windsor (Windsor-6)

Resource Name	Multifamily Residence
VTrans ID No.	N/A
Location	189 Main Street, Windsor
Parcel ID	560195-000
Date(s) of Construction	ca. 1920
NRHP Recommendation	Not Eligible



This resource is a two-story, three-bay, frame multifamily residence. The rectilinear plan building has a flat roof with a molded cornice carried by cornerboards. The majority of the exterior is sheathed in wood clapboards, but the first-story façade is covered by vertical composite boards. The fenestration is primarily composed of replacement six-over-six double hung sashes and paired eight-over-eight windows light the first two bays of the ground floor, all in wood surrounds with modest crowns; the openings for the latter windows appear to have been boarded. The primary entrance is a modern multilight door occupying the third bay. Secondary entrances with similar door types are located on the second stories of the south elevations of the primary unit and a rear addition, both accessed by external wood staircases. A gabled, single-story tertiary addition is located on the east elevation of the larger, earlier expansion. A photograph dated circa 1933 shows that the resource was built as a Texaco service station (American Precision Museum).

The building has been altered by modern exterior materials and is not architecturally significant. Therefore, it is WSP's opinion that the resource is not eligible for listing in the SR/NRHP.

V. Conclusions and Recommendations

On behalf of VTrans, WSP completed an ID Report involving the anticipated future repair and/or replacement of 27 bridges throughout the state. This particular report addresses Windsor Bridge No. 55 over Mill Brook, U.S. Highway 5, Windsor County. The survey's purpose was to identify and evaluate all structures and buildings in the survey area, including the bridge, that may be directly, indirectly, and/or cumulatively impacted by the potential project.

WSP identified eight resources in the survey area: the NRHP-listed WVHD; the American Precision Museum, an NHL; NAMCO Block; the SRHP-listed Bridge No. 55; MBHD, Middle/Lower Dam Area, consisting of three contributing resources (including the American Precision Museum); and one previously unsurveyed architectural resource. No historic, park, recreational, or refuge Section 4(f) resources were identified in the survey area.

It is WSP's opinion that the WVHD and the rest of the previously surveyed properties should remain listed in the SR/NRHP. Windsor Bridge No. 55 meets Criterion C registration requirements outlined in the MPD *Metal Truss, Masonry and Concrete Bridges of Vermont, 1820–1978* (Louis Berger 2018:F59-F61) as an early, well-preserved example of the concrete open spandrel arch type. It is WSP's opinion that the previously unsurveyed resource is not eligible for listing in the SR/NRHP.

TABLE 2: NRHP ELIGIBILITY RECOMMENDATIONS FOR PREVIOUSLY AND NEWLY IDENTIFIED HISTORIC RESOURCES IN SURVEY AREA

MAP ID (Windsor-)	RESOURCE NAME	LOCATION	PREVIOUS ELIGIBILITY	WSP ELIGIBILITY OPINION	PHOTOGRAPH
1	Windsor Village Historic District	Multiple, Windsor	Listed, SR/NRHP	Listed, SR/NRHP	
2	Windsor Bridge No. 55 over Mill Brook	U.S. 5, Windsor	Listed, SRHP	Listed, SRHP; Eligible, NRHP	
3	American Precision Museum	196 Main Street, Windsor	Listed, SR/NRHP (NHL; MBHD, Middle/Lower Dam Area)	Listed, SR/NRHP (NHL; MBHD, Middle/Lower Dam Area)	

MAP ID (Windsor-)	RESOURCE NAME	LOCATION	PREVIOUS ELIGIBILITY	WSP ELIGIBILITY OPINION	PHOTOGRAPH
4	NAMCO Block	1-17 Union Street, Windsor	Listed, SR/NRHP	Listed, SR/NRHP	
5	Mill Brook Historic District, Middle/Lower Dam Area	Mill Brook Falls, Windsor	Listed, SRHP	Listed, SRHP; Eligible, NRHP	
5a	Site No. 14, Grist Mill Ruins	North Bank of Mill Brook, East of US 5, Windsor	Listed, SRHP	Listed, SRHP; Eligible, NRHP	
5b	Site No. 15, Foundry Wheel House Ruins	South Bank of Mill Brook, East of U.S. 5, Windsor	Listed, SRHP	Listed, SRHP; Eligible, NRHP	
6	Multifamily Residence	189 Main Street, Windsor	Not evaluated	Not Eligible	

Given that the boundary of the Robbins and Lawrence Armory and Machine Shop NHL is adjacent to Bridge No. 55, there is a potential for either the rehabilitation or replacement of the bridge to adversely affect the Armory building within the NHL and/or other contributing resources. Current documentation for the NHL is dated and does not fully account for the significance of landscape features such as the retaining wall, which is close to the abutment for Bridge No. 55. WSP recommends VTrans initiate early consultation with Region 1 of the NPS about the potential project and, if possible, obtain additional information from the NPS about the significant character-defining elements of the NHL and what resources within the boundary are contributing to the significance of the NHL. If the NPS does not have this information, WSP recommends an intensive survey be conducted in consultation with VDHP and the NPS to understand the significant resources within the NHL.

Other historic resources, such as the NAMCO Block, are near the bridge. Precautions should be taken to limit both direct effects to the building from construction equipment and activities as well as potential vibration effects from construction activities (whether for bridge rehabilitation or replacement). Since Bridge No. 55 is SRHP-listed/NRHP-eligible, replacement of the bridge would result in an adverse effect. Design of the replacement bridge could mimic the design of the current bridge or previous bridges at the site to be sympathetic to the adjacent WVHD. If the bridge will be rehabilitated, work should follow Secretary of the Interior Standards to preserve the character-defining features and avoid an adverse effect.

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1933 *View of Main Street, Looking North, ca. 1933.* Photograph on file, American Precision Museum, Windsor, Vermont.

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Appendix H: Natural Resource ID

VERMONT

Jessie Johnson
VTrans Biologist
State of Vermont
Environmental Section
Barre City Place
219 Main St.
Barre City, VT 05641
https://vtrans.vermont.gov/

Agency of Transportation

To: File
From: Jessie Johnson, VTrans Biologist
Date: Thursday, September 28, 2023
Subject: Windsor BF 0113(88) – Natural Resource ID

I have completed natural resource identification for Windsor BF 0113(88). This project is located on Bridge 55, on Main Street in Windsor, Vermont. My evaluation has included wetlands and watercourses, wildlife habitat, agricultural soils, and rare, threatened, and endangered species.

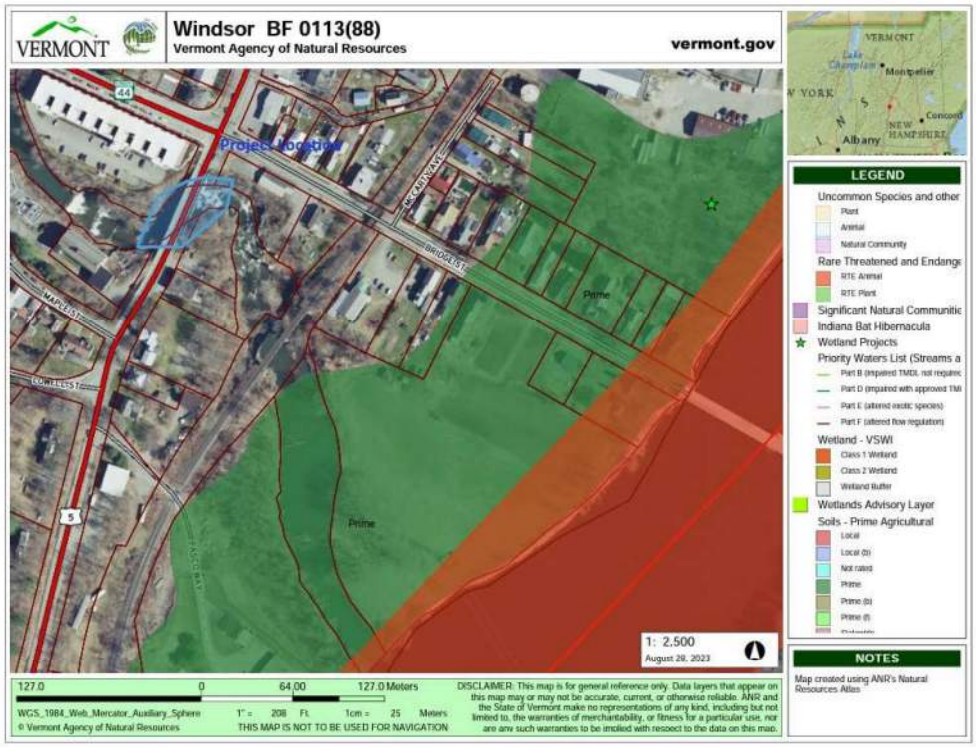


Figure 1: A map of the natural resources near the project location

Wetlands and Watercourses
I have reviewed existing VSWI, Advisory Wetland Mapping, USFWS Wetland Mapper, and conducted a field visit on 9/11/2023. I have discovered that no wetlands are found within the vicinity of the project. Bridge 55 crosses Mill Brook, which flows into the Connecticut River, which is located just southeast of the project location.

Wildlife Habitat
The land surrounding the project area is primarily urban, and does not have any significant wildlife habitat.

Rare, Threatened, and Endangered Species
I queried the USFWS IPaC website and the project is within the range of the federally endangered Northern long eared bat (*Myotis septentrionalis*) and the Dwarf Wedge mussel (*Alasmidonta heterodon*). There is potential suitable Northern long eared bat habitat under the bridge, and a bat survey should be conducted prior to construction. The Vermont Natural resource Atlas depicted that an occurrence of the Dwarf wedgemussel was recorded in the nearby Connecticut River. It is quite possible that mussels could be present within the project location. A mussel survey is recommended prior to the construction of this project.

Agricultural Soils
Podunk fine sandy loam, 0 to 3 percent slopes, occasionally flooded, which is classified as a prime agricultural soil, has been recorded near the project location.

Pictures from the 9/11/2023 Field Visit

















Appendix I: Stormwater Resource ID

State of Vermont
Environmental Section
219 North Main Street
Barre, Vermont 05641
Vtrans.vermont.gov

Agency of Transportation

[phone] 802-498-5787

To: Project file
From: Heather Voisin, VTrans Green Infrastructure Engineer
Date: January 30, 2024
Subject: Windsor BF 0113(88) - Stormwater Resource ID Review

Project Description: I have reviewed the project area for Windsor BF 0113(88) for stormwater related regulatory and water quality concerns. At this time, the project scope has not yet been defined, so this review is based on the surrounding area and potential stormwater impacts. The project involves Bridge 55 on US Route 5, which crosses Mill Brook in the village of Windsor. My evaluation has included the review of existing imagery and mapping (Google Street View from August 2023 and ANR Natural Resource Atlas) to capture existing stormwater features and existing drainage.

Regulatory Considerations
Once the scope of the project has been determined, an assessment of how much impervious area is involved will determine if an Operational Stormwater permit is required for the project. There do not appear to be any existing stormwater permits in the immediate vicinity of the site area.

Mill Brook in this area is listed as being on the 2016 Stressed Waters List due to "impoundment de-silting, developed land runoff" and pollutants associated with sedimentation and stormwater. While this designation should have no bearing on the project, the project should aim to reduce sedimentation and stormwater impacts to the extent possible.

Existing Drainage
Based on a review of available mapping, it appears that the bridge is curbed and drainage from most of the surrounding roads is also collected via a closed drainage system of inlets and pipes.

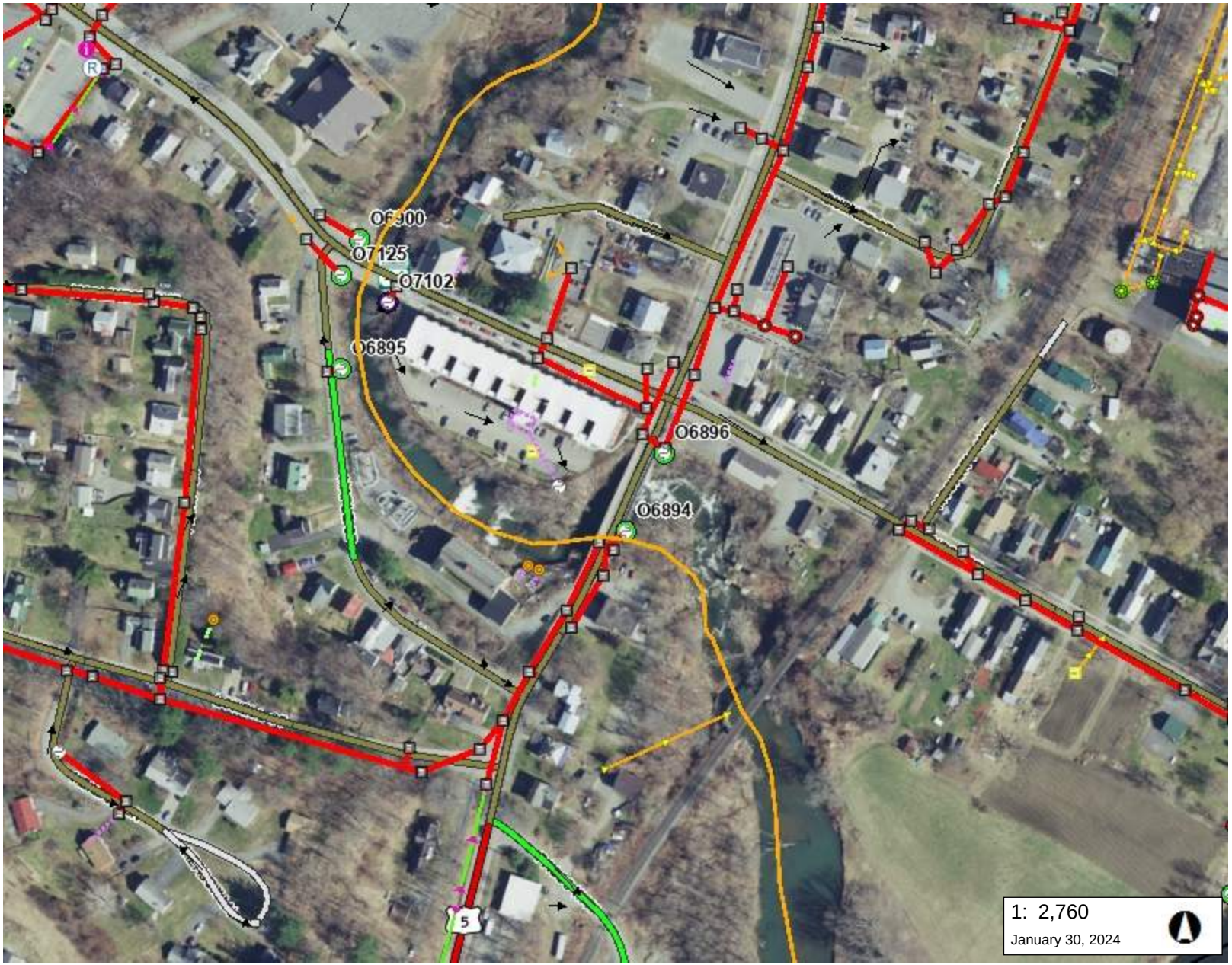
Design Considerations
Depending on the extent of the project limits and roadway approach work, it may be worthwhile to consider opportunities to improve the drainage from the surrounding area. To the extent possible, it is encouraged that it be aligned with the VTrans Phosphorus Control Highway Drainage Management Standards, as this may allow future credit toward achieving phosphorus reduction goals required by the Agency's TS4 permit.

VERMONT

SW Resource ID

Vermont Agency of Natural Resources

vermont.gov





LEGEND

- 303(d) List of Impaired Stream
- Stressed Waters List (Streams

Stormwater Permits (Issued)

- Operational
- Construction
- Industrial - NOI
- Industrial - NOX
- Other

Stormwater Permits (Pending)

- Operational
- Construction
- Industrial - NOI
- Industrial - NOX
- Other

Existing stormwater point

- Pipe Cross (not connected)
- Catchbasin
- Dry Well
- Drop Inlet
- Grate/Curb Inlet
- Yard drain
- Junction Box
- Stormwater Manhole
- Outfall
- Culvert inlet
- Culvert outlet
- Pond outlet structure

NOTES

Map created using ANR's Natural Resources Atlas

140.0 0 70.00 140.0 Meters

WGS_1984_Web_Mercator_Auxiliary_Sphere 1" = 230 Ft. 1cm = 28 Meters

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Appendix J: Landscape Resource ID

VERMONT
AGENCY OF TRANSPORTATION

State of Vermont | Agency of Transportation
Environmental Section
219 North Main
Barre, VT 05641
Vtrans.vermont.gov

To: Project File
From: Bonnie Kirn Donahue, *VTrans Landscape Architect*
Date: January 18, 2024
Project: **Windsor BF 0113(88) 22J404**
Subject: Landscape (LA) Clearance for Resource ID

I have reviewed the proposed area for **Windsor BF 0113(88) 22J404**, and found the following:

SITE DESCRIPTION
Bridge 55 is located on Main Street in Windsor, VT. The surrounding area is urban in character, with an existing sidewalk network.

EXISTING CONDITIONS
The following items/conditions were found on site that could influence design decisions:

1. Context/setting:
    a. The project is located in an urban area
    b. This project is located in a downtown or village area

2. Presence of utilities:
    a. This project includes utilities that should be considered.

3. Riparian buffer:
    a. This project includes work within a riparian area and may benefit from a planting plan.

4. Trees to protect:
    a. This project includes trees that should be protected, including:
        i. Street trees at Bridge and Main Street
        ii. Trees at the Union Square apartment building at Bridge and Main Street.
        iii. Street trees at Maple and Main Street

5. Presence of hazard trees
    a. Desktop review. No hazard trees were identified.

6. Special site features:
    a. This project includes special site features that should be protected.

i. The concrete railing of this bridge is unique. Consider how to protect or recreate this railing to reflect the existing railing.
ii. Gardens at the Union Square apartment building

7. Plants observed: (this is not a complete list of species on site)
    a. Desktop review. No species were identified. See natural resources clearance.

8. Invasive species observed: (this is not a complete list of species on site)
    a. Desktop review. No species were identified. See natural resources clearance.

9. Accessibility & Active Transportation:
    a. This project includes accessible sidewalks and crosswalks that should be protected.
    b. This project includes bicycle facilities that should be protected.
    c. This project would benefit from lower speeds supported by pedestrian design accommodations.

COMMUNITY RESOURCES

Per the 2019 Windsor Town Plan, the following municipal policies apply to this project:

1. GOALS:
    a. "To provide for a safe, convenient, economic and energy efficient transportation system that respects the integrity of the natural environment, including public transit options and facilities for pedestrians and bicycles.
    b. To maintain a functional and attractive roadway network that preserves the rural and historic character of Windsor.
    c. To provide a multi-modal transportation network that supports the development goals and policies of the Town Plan." (page 27)
2. POLICIES:
    a. "Roads within the downtown shall be designed for slow speeds and narrow streets in order to further smart growth principles, improve pedestrian safety and protect the historic character of the area." (page 27)
    b. "Structural improvements to Bridge 55 on Main Street shall not diminish the historic character of this concrete arch structure." (page 27)
    c. "Improve bicyclist safety along US Route 5 between the downtown and industrial park with any major state repaving or reconstruction project by narrowing travel lanes and increasing shoulder widths." (page 27)

RECOMMENDATIONS

1. Minimize tree clearing in this area.
2. Minimize disturbance in the riparian buffer.
3. Develop a riparian planting plan for any disturbed riparian areas on this project.
4. Tree protection shall be used for any trees with canopies within the area of construction, including:
    a. Street trees at Bridge and Main Street
    b. Trees at the Union Square apartment building at Bridge and Main Street.
    c. Street trees at Maple and Main Street
5. Follow the goals and polices in the 2019 Windsor Town Plan related to historic character of the bridge, and pedestrian/bicyclist safety and infrastructure.

6. Protect, maintain, and improve the accessibility, function, and safety of active transportation infrastructure, such as:
    a. Maintain at least one sidewalk on the bridge.
    b. Consider including infrastructure for bicyclists, like wider shoulders.
    c. Explore the feasibility of pedestrian design accommodations that lower speeds (curb extensions, etc.).
7. Develop a plan for managing invasive species in the project area.

NOTES

1. I am available to assist with landscape architectural design, including planting plans, plant lists, hardscape/pedestrian access plans, etc. (bonnie.donahue@vermont.gov).

ATTACHMENTS



Figure 1: Protect street trees (right) and garden space (left)



Figure 2: Protect historic character of bridge (existing railings contribute to this). Maintain sidewalk on at least one side of the bridge.



Figure 3: Protect street trees near the intersection of Main & Maple

Appendix K: Environmental Specialist Memo

VERMONT

State of Vermont
Highways-PDB-Environmental
219 N. Main Street
www.aot.state.vt.us

Agency of Transportation

Date: September 15, 2023
Environmental Specialist: Lee Goldstein
Project: **Windsor BF 0113(88) 22J404**

6(f) Properties:
There aren't any 6(f) Properties within the project area.

Hazardous Waste:
Multiple hazardous waste sites are identified within the proposed project area. However, due to project scope, impacts to these sites are not anticipated. Coordination with the Hazard Waste Coordinator is not recommended at this time.

Contaminated Soils/ Urban Background Soils
The proposed project is located within a mapped *Urban Background Soils* area (See ANR Atlas Map in the folder). Additional coordination with the Hazard Waste coordinator is recommended. Disturbed soils within this project should be expected to be kept on site, or follow current 'Notice to Bidders' guidance.

Wild Scenic Rivers:
There aren't any designated Wild Scenic Rivers within the project area.

Act 250 Permits:
There aren't any Act 250 Permits within the project area.

FEMA Floodplains:
There are FEMA Floodplains mapped within the project area and a Flood Hazard Area/ River Corridor Permit may be required if there are impacts.

River Corridor:
There are River Corridors mapped within the project area and a Flood Hazard Area/ River Corridor Permit may be required if there are impacts.

Protected Lands:
There aren't any Protected Lands within the project area.

US Coast Guard:
There aren't any US Coast Guard navigable waterways within the project area. In addition, there is a dam located upstream of the bridge.

Lakes and Ponds:
There aren't any lakes or ponds within the project area.

Scenic Highway/ Byway:
There are Scenic Highway/ Byways within the project area and further coordination with the Environmental Section may be required.

Environmental Justice:
Determination of no adverse impacts:
"No minority or low-income populations have been identified that would be adversely impacted by the proposed project as determined above. Therefore, in accordance with the provisions of E.O. 12898 and FHWA Order 6640.23A, no further EJ analysis is required."

Other:
There aren't any other resources within the project area.

Appendix L: Preliminary Hydraulics Memo

VERMONT

State of Vermont
Structures and Hydraulics Section
Barre City Place, 219 North Main Street
Barre, Vermont 05641
vtrans.vermont.gov

Agency of Transportation

[phone] 802-595-6493

TO: Laura Stone, P.E., Scoping Engineer/AOT Project Manager

CC: Patrick Ross, Hydraulics Engineer

FROM: Madeline Glow, Hydraulics Project Engineer

DATE: November 30, 2023

SUBJECT: WINDSOR BF 0113(88) Preliminary Hydraulics Memo (#22J404)
Site location: Bridge 55, US Route 5, 200 ft south of VT-44, Mill Brook
Coordinates: 43.47494, -72.38882

The existing structure is a concrete arch bridge with a concrete cast in place deck built in 1930. Bridge 55 spans Mill Brook, which flows south directly into the downstream confluence of the Connecticut River approximately 0.2 miles from the structure.

There is a FEMA Flood Insurance Study for Windsor county. This study indicates that this structure is within Zone AE, an area that corresponds to Special Flood Hazard Areas (SFHA) inundated by the 100-year flood with base flood elevations determined.

Bridge 55 is not listed on the VTrans Scour Critical Bridge list as both abutments look to be founded on ledge, minimizing the scour concern here. Both the superstructure and substructure are rated as "Satisfactory" condition from the latest 2021 Bridge Inspection report. Survey indicates that the elevation of the crown of the arch is 339.26 feet.

We recommend that all rehabilitation work, as part of the proposed deck replacement scope, be done above the crown of the arch. A 2-D hydraulic model was not developed for this project as the proposed project scope is not anticipated to result in elevation changes of the concrete arches or work below the arch crown elevation. If there are any changes to the scope where work is proposed at or below the arch crown elevation (e.g. superstructure replacement, shoring, abutment work, etc.) please contact the VTrans Hydraulics Unit.

Appendix M: Hazardous Waste Urban Soils Map

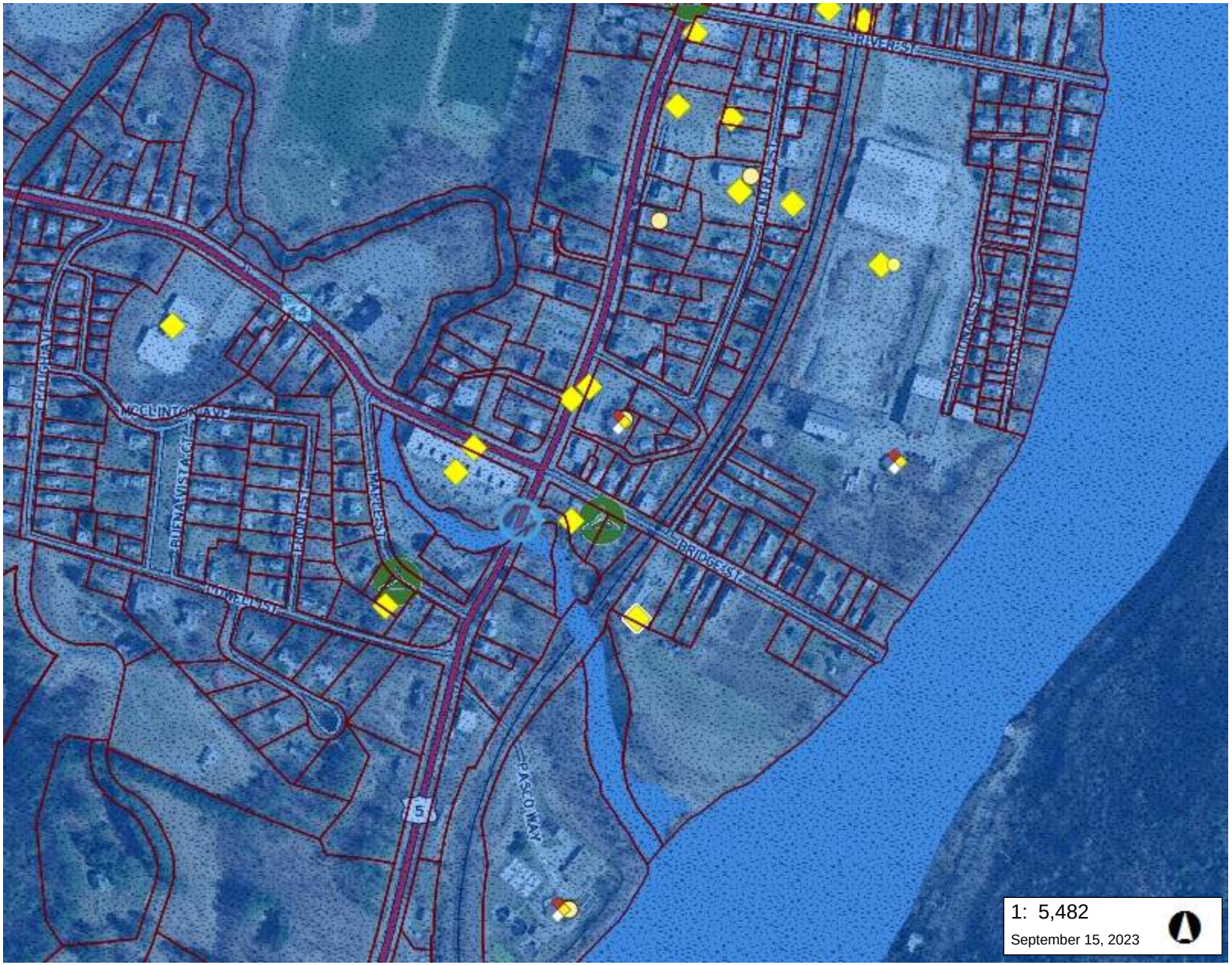
VERMONT

Windsor BF 0113(88)

Vermont Agency of Natural Resources

vermont.gov





LEGEND

- Landfills
  - OPERATING
  - CLOSED
- Land Use Restrictions
  - Class IV GW Reclass
  - Class VI GW Reclass
  - Deed Restriction
  - Easement
  - Land Record Notice
  - Other
- Hazardous Site
- Hazardous Waste Generators
- Brownfields
- Salvage Yard
- Aboveground Storage Tank
- Underground Storage Tank (w
- Dry Cleaner
- Urban Soil Background Areas
- Parcels (standardized)
- River Area
- Railroads
- Roads
  - Interstate
  - US Highway; 1
  - State Highway
  - Town Highway (Class 1)
  - Town Highway (Class 2,3)

278.0 0 139.00 278.0 Meters

WGS_1984_Web_Mercator_Auxiliary_Sphere 1" = 457 Ft. 1cm = 55 Meters

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NOTES

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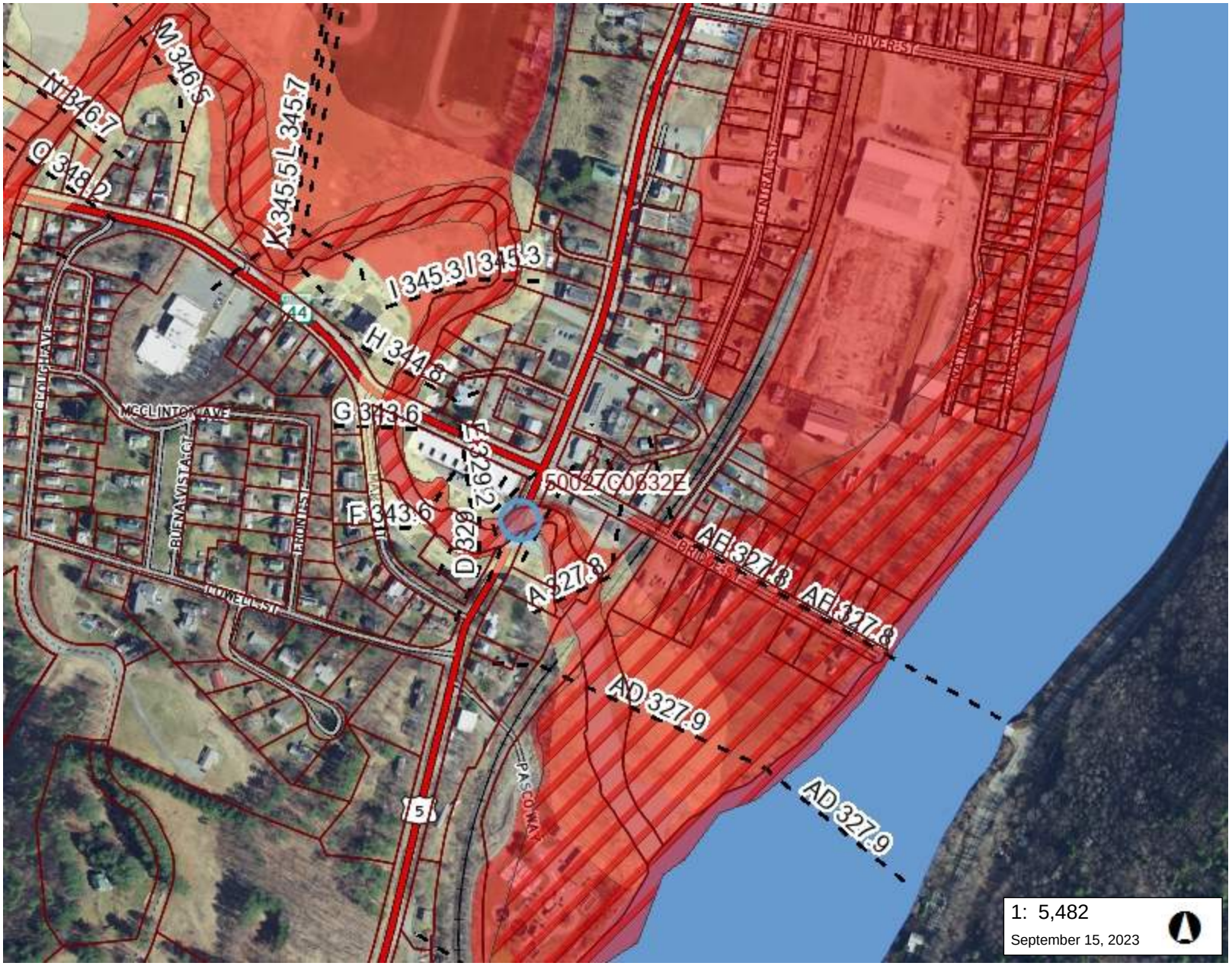
Appendix N: Floodplain Rivers Map

VERMONT

Windsor BF 0113(88)

Vermont Agency of Natural Resources

vermont.gov





LEGEND

- DFIRM X-Sections
- DFIRM - Letter of Map Revisio
- DFIRM Panels
- DFIRM Floodways
- Flood Hazard Areas (Only FEM
  - AE (1-percent annual chance floodp
  - A (1-percent annual chance floodpl
  - AO (1-percent annual chance zone feet)
  - 0.2-percent annual chance flood ha
- River Corridors (Aug 27, 2019)
- .5 - 2 sqmi.
- .25-.5 sqmi.
- Parcels (standardized)
- River Area
- Railroads
- Roads
  - Interstate
  - US Highway; 1
  - State Highway
  - Town Highway (Class 1)
  - Town Highway (Class 2,3)
  - Town Highway (Class 4)
  - State Forest Trail
  - National Forest Trail
  - Legal Trail
  - Private Road/Driveway
  - Proposed Roads

NOTES

Map created09/15/2023 using ANR's Natural Resources Atlas

278.0 0 139.00 278.0 Meters

WGS_1984_Web_Mercator_Auxiliary_Sphere 1" = 457 Ft. 1cm = 55 Meters

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Appendix O: Utilities Memo

Windsor BF 0113(88) Pin: 22j404

Existing Utilities within Project Limits Report

VT Route 5- BR 55

AERIAL

-Green Mountain Power Corporation
-Consolidated Communications
-Comcast Communications
-FirstLight Fiber
-VELCO

UNDERGROUND

-Green Mountain Power
-Consolidated Communications
-Comcast Communications
-Vermont Telephone Company

MUNICIPAL

Town of Windsor Water
Town of Windsor Sewer

There is a 8" municipal watermain running through the bridge abutments but the section crossing the bridge is currently abandoned according to the Town of Windsor's water operator. The current waterline line runs up Union Street. There are water valves to the west of the project site near Lowell Street.

The town of Windsor also has sewer infrastructure running up Union Street at the intersection.

There is also a 6 conduit ductbank running through the bridge abutments. Consolidated Communications has three of these conduits in this ductbank. I am confirming the other utilities in this ductbank.

Velco has a substation to the north of this project site with transmission lines running aerially through the project area. These transmission line towers are mounted on piers within the river bed to the north and south of the bridge. There is a transmission tower on the northerly side of the bridge which is within 2 ft of the bridge structure.

Windsor BF 0113(88) Pin: 22j404

Existing Utilities within Project Limits Report

VT Route 5- BR 55

Green Mountain power has several 3 phase lines running in this area crossing the bridge, and several aboveground and underground services in the area including a multiunit housing complex to the north of this project area.

There could be large impacts to these utilities depending on the construction practices and limits.

Appendix P: Traffic Memo – VTrans

AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

OPERATIONS & SAFETY BUREAU DATA MANAGEMENT UNIT

TO: Daniel Beard

FROM: Becca Mitchell, AOT Data Analyst

DATE: September 25, 2023

RE: WINDSOR BF 0113(88) 22J404
US-5 Windsor to US-5 Windsor
Length: 0.0000 Begin Mile Marker: 3.3380 End Mile Marker: 3.3380

Please find below the requested traffic data for the above referenced 2029 project. The data consists of the AADTs and DHVs for 2029 and 2049, as well as the 20-year (2029 ~ 2049) and 40-year (2029 ~ 2069) flexible ESALs.

If you have any questions, please call me at (802) 279-0502 or email me at becca.mitchell@vermont.gov.

AADT = Annual Average Daily Traffic **DHV** = Design Hour Volume **%T** = Percentage of Trucks during Peak Hour **%D** = Highest Directional Percentage during Peak Hour **ADTT** = Average Daily Truck Traffic **ESALs** = (Flexible) Equivalent Single Axle Loads

CC: Data Analysis Files

Section	AADT		DHV		%T		%D		ADTT		ESALs	
	2029	2049	2029	2049	2029	2049	2029	2049	2029	2049	(2029~2049)	(2029~2069)
1	3259	3522	390	410	4.4%	6.2%	61%	61%	263	400	1155000	2630000

Appendix Q: Community Questionnaire

Project Summary

This project, BF 0113(88), focuses on bridge 55 on US Route 5 in Windsor, Vermont. The bridge is deteriorating and needs either a major maintenance action or replacement. Potential options being considered for this project include targeted repairs, deck replacement, deck & superstructure replacement, and a replacement with a new bridge. It is possible that VTrans will recommend a road closure and detour traffic away from the project site for the duration of the work. Efforts will be made to limit the detour to State roads.

Community Considerations

1. Are there regularly scheduled public events in the community that will generate increased traffic (e.g. vehicular, bicycles and/or pedestrians), or may be difficult to stage if the bridge is closed during construction? Examples include annual bike races, festivals, parades, cultural events, weekly farmers market, concerts, etc. that could be impacted? If yes, please provide approximate date, location and event organizers' contact info.
   A: No, no regularly scheduled events.

2. Is there a "slow season" or period of time from May through October where traffic is less or no events are scheduled**?**

   A: No, traffic patterns are consistent throughout the summer.

3. Please describe the location of the Town garage, emergency responders (fire, police, ambulance) and emergency response routes that might be affected by the closure of the bridge, one-way traffic, or lane closures and provide contact information (names, address, email addresses, and phone numbers.

   A: Maple St. is one way (east) but can be changed to accommodate 2 way traffic during the construction process. The Cornish-Windsor covered bridge and 12A provide a convenient option for passenger vehicles if needed for non-emergency service.

   Fire Chief: Kevin McAllister K.Mcallister@windsorvt.org 802-674-9043
   Police Chief: Jennifer Frank jennifer.frank@vermont.gov 802-674-9042
   Highway Foreman: Pete Johnson windsorhighway@gmail.com 802-674-6000
   Fire and police are located at 29 Union St. Windsor
   Highway is located at 30 Central St. Windsor

4. Are there businesses (including agricultural operations and industrial parks) or delivery services (fuel or goods) that would be adversely impacted either by a detour or due to work zone proximity?
   A: No.

5. Are there important public buildings (town hall, community center, senior center, library) or community facilities (recreational fields, town green, etc.) close to the project?
   A: Town Hall and the Town wastewater treatment plant are close to the project

6. What other municipal operations could be adversely affected by a road/bridge closure or detour?
   A: Municipal water & sewer operations - minor impact

7. Are there any town highways that might be adversely impacted by traffic bypassing the construction on other local roads? Please indicate which roads may be affected and their condition (paved/unpaved, narrow, weight-limited bridges, etc), including those that may be or go into other towns.
   A: Maple St. - paved
   Lowell St. - paved
   Clough Ave. - paved
   Bridge St. - Paved
   Cornish - Windsor covered bridge - height restricted
   NH 12a - paved

8. Is there a local business association, chamber of commerce, regional development corporation, or another downtown group that we should be working with? If known, please provide name, organization, email, and phone number.
   Mount Ascutney Regional Commission - Jason Rasmussen 802-674-9201
   jrasmussen@marcvt.org
   Springfield Regional Development Corp. - Bob Flint 802-8853061
   bobf@springfielddevelopment.org

9. Are there any public transit services or stops that use the bridge or transit routes in the vicinity that may be affected if they become the detour route?
   Southeast Vermont Transit: Randy Schoonmaker 802-464-0164 randys@moover.com

Schools

1. Where are the schools in your community and what are their yearly schedules (example: first week in September to third week in June)?

   Schools are all located at 19 Ascutney St. Windsor. School year is 4th week in August to 3rd week in June

2. Is this project on specific routes that school buses or students use to walk to and from school?

   No

3. Are there recreational facilities associated with the schools nearby (other than at the school)?

   No

Pedestrians and Bicyclists

1. What is the current level of bicycle and pedestrian use on the bridge?

   Minimal

2. Are the current lane and shoulder widths adequate for pedestrian and bicycle use?

   No

3. Does the community feel there is a need for a sidewalk or bike lane on the bridge?
   Yes

4. Is pedestrian and bicycle traffic heavy enough that it should be accommodated during construction?

   No, minimal use due to lack of bike/ped lane

5. Does the Town have plans to construct either pedestrian or bicycle facilities leading up to the bridge? Please provide any planning documents demonstrating this (scoping study, master plan, corridor study, town or regional plan). **Sidewalk currently exists on the west side of the bridge and connects to municipal sidewalks at either end.**

6. In the vicinity of the bridge, is there a land use pattern, existing generators of pedestrian and/or bicycle traffic, or zoning that will support development that is likely to lead to significant levels of walking and bicycling? **Current development density contributes significantly to walking and bicycling in the area. There are no current plans that are expected to add to the existing volume.**

Design Considerations

1. Are there any concerns with the alignment of the existing bridge? For example, if the bridge is located on a curve, has this created any problems that we should be aware of?

   No

2. Are there any concerns with the width of the existing bridge?

   A: Yes, too narrow for bike traffic

3. Are there any special aesthetic considerations we should be aware of?

   A: "Concrete arch" design should be maintained. Original bridge contained light posts, the Town would like new posts included in the design

4. Does the location have a history of flooding? If yes, please explain.

   A: Tropical Storm Irene created high water levels, but did not flood bridge

5. Are there any known Hazardous Material Sites near the project site? **No**

6. Are there any known historic, archeological and/or other environmental resource issues near the project site?**. The American Precision Museum abuts the bridge. A retaining wall owned by the museum directly abuts the bridge from below the water level to street level. The area along the brook by the bridge had been home to a variety of manufacturing facilities some remnants of those facilities remain.**

7. Are there any utilities (water, sewer, communications, power) attached to the existing bridge? Please provide any available documentation. **No. A sewer line had been suspended from the bridge but that was removed some time ago and the connections have been capped.**

8. Are there any existing, pending, or planned municipal utility projects (communications, lighting, drainage, water, wastewater, etc.) near the project that should be considered? **No**

9. Are there any other issues that are important for us to understand and consider? Detours have the potential to significantly impact surrounding neighborhoods. Given the height restrictions of the Cornish – Windsor covered bridge detour plans for truck traffic should be well marked at the I-91 8 and 9 exits.

Land Use & Zoning

1. Please provide a copy of your existing and future land use map or zoning map, if applicable.

   This link will bring you to a copy of our zoning map.

2. Are there any existing, pending or planned development proposals that would impact future transportation patterns near the bridge? If so, please explain. **No.**

3. Is there any planned expansion of public transit or intercity transit service in the project area? Please provide the name and contact information for the relevant public transit provider.

   Southeast Vermont Transit: Randy Schoonmaker 802-464-0164 randys@moover.com

Communications

1. Please identify any local communication outlets that are available for us to use in communicating with the local population. Include weekly or daily newspapers, blogs, radio, public access TV, Facebook, Front Page Forum, etc. Also include any unconventional means such as local low-power FM.
   A: Cable access TV
   Municipal Social Media
   Weekly community newsletter

2. Other than people/organizations already referenced in this questionnaire, are there any others who should be kept in the loop as the project moves forward?
   American Precision Museum located next to bridge:

   A: Steve Dalessio steve@americanprecisionmuseum.org 802 674-5781

Appendix R: WJE Materials Testing and Condition Assessment Report

WJE

Windsor Bridge 55 Carrying US5 Over Mill Brook

Materials Testing and Condition Assessment Report

Windsor, Vermont



FINAL REPORT

March 5, 2025

WJE No. 2023.7260

PREPARED FOR:

HNTB Corporation
3 Executive Park Drive
Bedford, New Hampshire 03110

PREPARED BY:

Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, Illinois 60062
847.272.7400 tel

Windsor Bridge 55 Carrying US5 Over Mill Brook

Materials Testing and Condition Assessment Report

Windsor, Vermont

Arne Johnson, PE (IL, IA, IN, KY, MN, MO, OH, WI)
Principal and Project Manager

Tanner Swenson, PE (MN, ND, WI)
Senior Associate

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CONTENTS

1. Introduction 1
2. Bridge Description and Repair History 4
2.1. General Description and Nomenclature 4
2.1.1. Bridge Deck and Railings 4
2.1.2. Floor Beams and Columns 7
2.1.3. Arch Ribs 10
2.1.4. Abutments and Wing Walls 12
2.1.5. Water Main and Utilities 14
2.2. Bridge Inspection and Repair History 14
2.3. Historical Significance 15
3. Bridge Inspection Work Plan 16
3.1. General Approach and Methodology 16
3.1.1. Phase 1 – Overall Element Level Bridge Inspection 16
3.1.2. Phase 2 – Follow-Up Study Area Testing and Material Sampling 17
3.2. Background Regarding Relevant Deterioration Mechanisms 18
3.2.1. Corrosion of Steel Embedded in Concrete 18
3.2.1.1. Carbonation-Induced Corrosion 19
3.2.1.2. Chloride-Induced Corrosion 20
3.2.2. Cyclic Freezing and Thawing Deterioration 21
3.2.3. Alkali-Silica Reaction Deterioration 22
3.2.4. Distress from Mechanical Action 23
3.3. Follow-Up Testing Methods 23
3.3.1. Field Testing 23
3.3.1.1. Material Sampling 23
3.3.1.2. Delamination Survey 25
3.3.1.3. Corrosion Survey 25
3.3.1.4. Reinforcement Cover Survey 29
3.3.1.5. Field Carbonation Testing 30
3.3.2. Laboratory Analysis 31
3.3.2.1. Concrete Compressive Strength and Elastic Modulus 31
3.3.2.2. Chloride Testing 31

3.3.2.3. Petrographic Analysis ... 32
3.3.2.4. Reinforcing Steel Strength and Composition Testing ... 32
4. Phase 1 - Overall Element Level Inspection Results ... 34
4.1. Documentation of Inspection Conditions ... 34
4.2. General Observations by Element ... 37
4.2.1. Bridge Deck and Railings ... 37
4.2.2. Floor Beams and Columns ... 41
4.2.3. Arch Ribs and Tie Beams ... 45
4.2.4. Abutments and Wing Walls ... 49
4.2.5. Water Main and Utilities ... 53
4.3. Conditions That Could Affect Structural Analysis and Load Rating ... 53
4.4. Distress Quantity Ratios by Element ... 53
4.4.1. Definition of Distress Quantity Ratio ... 53
4.4.2. Summary of Distress Quantity Ratios by Element Category ... 54
5. Phase 2 - Follow-Up Study Area Testing and Material Sampling Results ... 59
5.1. Field Testing and Material Sampling ... 60
5.1.1. Half-Cell Potential ... 62
5.1.2. Concrete Resistivity ... 67
5.1.3. Corrosion Rate ... 67
5.1.4. Carbonation and Reinforcement Cover ... 68
5.2. Laboratory Testing ... 68
5.2.1. Concrete Samples ... 68
5.2.2. Concrete Compressive Strength ... 70
5.2.3. Chloride Concentration ... 70
5.2.4. Petrographic Analyses ... 75
5.2.5. Reinforcing Steel Properties ... 80
6. Findings ... 81
6.1. Findings That Could Impact As-Inspected Structural Model ... 81
6.2. Comparison with Minnesota Open Spandrel Concrete Arch Bridges ... 81
6.3. Interpretation of Distress Quantity Ratios and Thresholds for Preservation Actions ... 83
6.4. Deterioration Mechanisms and Anticipated Remaining Service Life by Element ... 84
6.4.1. Deck ... 88

6.4.2. Railings ... *89*
6.4.3. Floor Beams ... *89*
6.4.4. Columns and Tie Beams ... *90*
6.4.5. Arch Ribs ... *90*
6.4.6. Abutments ... *91*
6.4.7. Wing Walls ... *92*
7. Summary and Considerations for Rehabilitation Alternatives ... 93
7.1. Deck ... 93
7.2. Railings ... 94
7.3. Floor Beams ... 94
7.3.1. At Expansion Joints ... *94*
7.3.2. Away from Expansion Joints ... *94*
7.3.3. Overhangs at Upstream and Downstream Edges ... *94*
7.4. Columns and Tie Beams ... 94
7.5. Arch Ribs ... 95
7.5.1. Potential Corrosion Mitigation and Protection Measures for Arch Ribs ... *96*
7.5.1.1. Coatings and Sealers ... *96*
7.5.1.2. Crack Repairs ... *96*
7.5.1.3. Cathodic Protection ... *97*
7.5.1.4. Metalizing ... *98*
7.5.1.5. Electrochemical Chloride Extraction (ECE) ... *99*
7.5.2. Selection of Corrosion Mitigation Solution ... *101*
7.6. Abutments ... 102
7.7. Wing Walls ... 102
7.8. Water Main and Utilities ... 103
APPENDIX A. BRIDGE INSPECTION WORK PLAN
APPENDIX B. PHASE 1 FIELD SURVEY DOCUMENTATION (PLANNOTATE SHEETS)
APPENDIX C. PHASE 2 STUDY AREA FIELD TESTING DOCUMENTATION
APPENDIX D. PETROGRAPHIC EXAMINATION REPORT
APPENDIX E. METALLURGICAL TESTING REPORT

1. INTRODUCTION

As part of a team led by HNTB Corporation (HNTB) and retained by the State of Vermont Agency of Transportation (VTrans), Wiss, Janney, Elstner Associates, Inc. (WJE) has completed a condition assessment and materials testing program for existing Windsor Bridge 55. The scope of work was defined in Task Order No. WJE009.01 from HNTB and performed in accordance with HNTB's prime agreement with VTrans for this project.

Completed in 1930, Windsor Bridge 55 (Windsor Bridge) carries US Route 5 over Mill Brook in Windsor, Vermont. The bridge is a single-span, open-spandrel concrete arch bridge measuring approximately 90 feet long by 33 feet wide. The deck is supported by eight floor beams, which are supported by two conventionally reinforced concrete arch ribs spaced at 20 feet on-center and with an 18-foot rise. The four floor beams nearest the crown of the arch frame directly into the arch ribs, while the other four are supported by spandrel columns extending down to the arch ribs, where cross beams frame between the ribs. At each spring line, the arch ribs frame into concrete abutments supported on spread footings. Overall views of the bridge are shown in Figure 1.1 and Figure 1.3. A bridge location map is shown in Figure 1.4.



Figure 1.1. Overall view of Windsor Bridge 55 from the downstream/east side.



Figure 1.2. Overall view of Windsor Bridge 55 from the upstream/west side.



Figure 1.3. Aerial view of bridge (arrow), looking east.

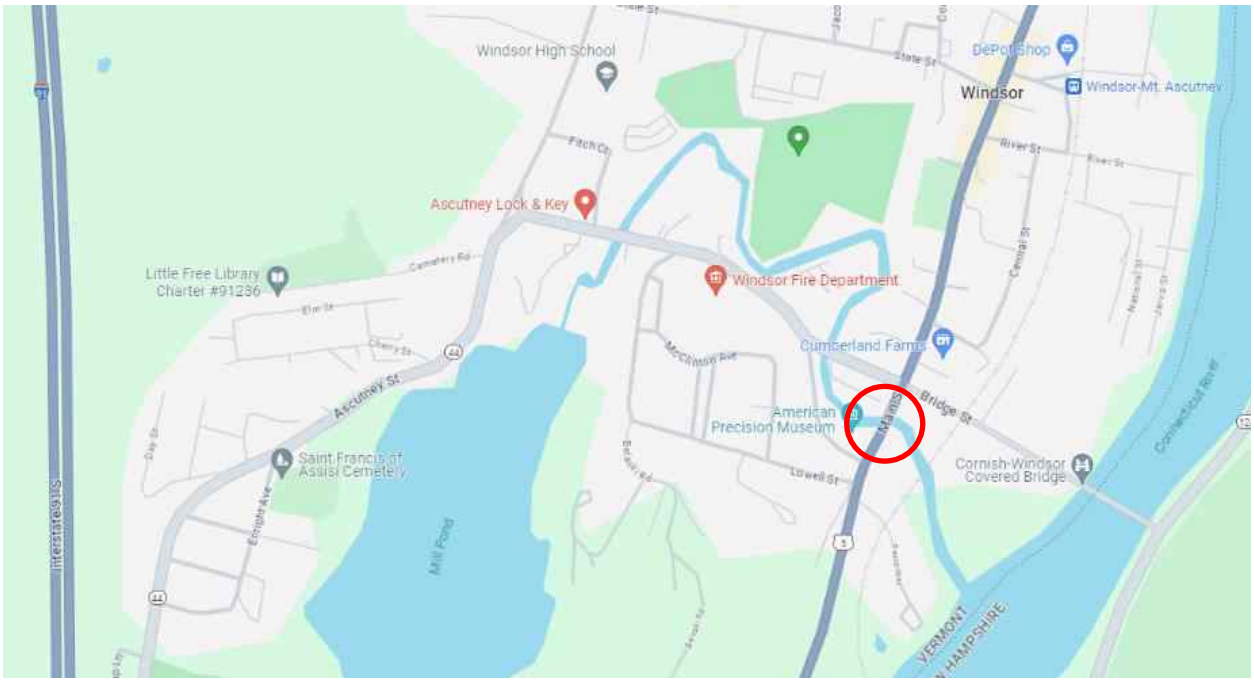


Figure 1.4. Location map with bridge circled in red (from Google Maps).

2. BRIDGE DESCRIPTION AND REPAIR HISTORY

2.1. General Description and Nomenclature

The Windsor Bridge, an open-spandrel concrete arch bridge constructed circa 1930, spans 90 feet in the general north-south direction. Mill Brook flows west to east beneath the bridge toward the Connecticut River. The bridge deck that carries two lanes of traffic is supported by eight floor beams, numbered consecutively from south to north and referred to as FB-1 through FB-8.[1] The floor beams are supported on columns or directly on two arch ribs (Figure 2.1). The arch ribs frame into abutments on each end. Wing walls extending beyond each abutment retain soil that supports the roadway approaching the bridge.

The following sections describe the construction of the bridge elements as documented in the original construction drawings. Some commentary based on field observations is added where relevant. Refer to Chapters 4 and 6 for information related to WJE's field findings and as-inspected conditions.

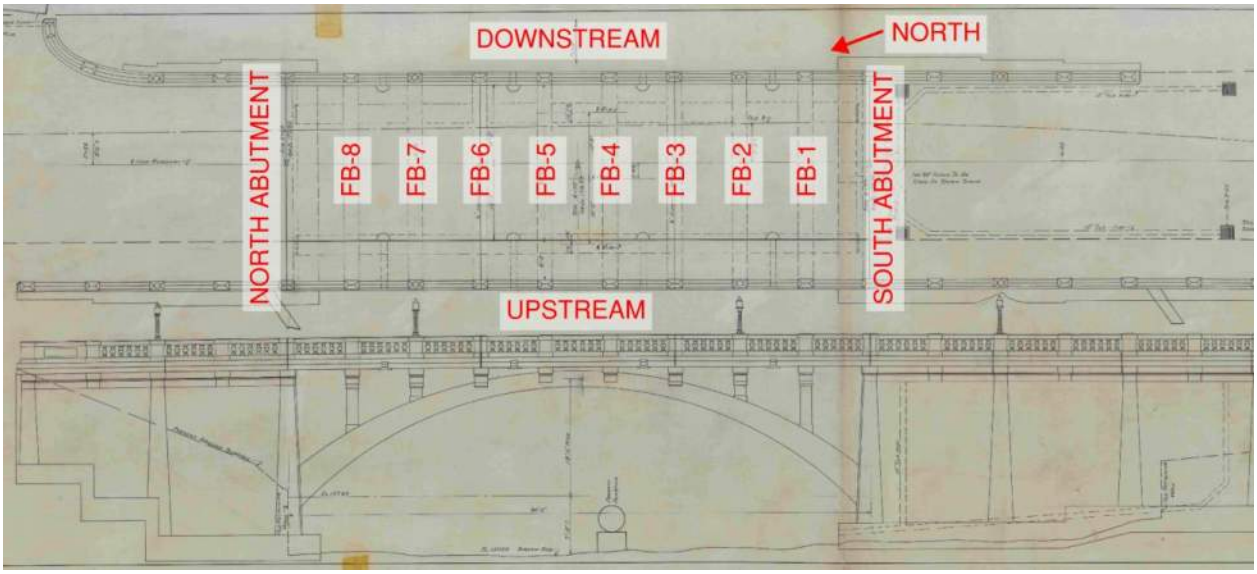


Figure 2.1. Bridge plan (top) and elevation (bottom) showing bridge orientation and floor beam naming convention (excerpt from original drawings).

2.1.1. Bridge Deck and Railings

The existing concrete bridge deck, which appears to be original, carries two lanes of traffic and includes a sidewalk along the upstream edge. The deck is 33 feet 6 inches wide and consists of a 24-foot-wide roadway, 6-foot-wide sidewalk, and 1-foot 4-inch-wide railings. The deck is 10 inches thick within the roadway and steps up 8 inches at the sidewalk. The sidewalk consists of a 6-inch-thick slab and 1-foot-7-inch-deep drop beam below the railing. The deck has a cross slope of 2 inches in 12 feet and a longitudinal slope of 1 percent downward from south to north.

[1] Similar numbering has been used in recent inspection reports; however, the original drawings label FB-1 as beam 1L, FB-2 as 2L, FB-3 as 3L, FB-4 as 4L, FB-5 as 4R, FB-6 as 3R, FB-7 as 2R, and FB-8 as 1R.

The deck was constructed with transverse expansion joints (referred to as deck joints in this report) at the third points of the bridge (i.e., over FB-3 and FB-6). Each 30-foot-long deck section is reinforced with mats of 3/4-inch-diameter reinforcing bars top and bottom of the deck. The deck is connected to the floor beams with hooked reinforcing bar dowels, except at FB-3 and FB-6 where only bottom reinforcing steel and no hooked dowels are present. An approximately 2-1/2-inch-thick asphaltic concrete overlay is present over the roadway, and asphaltic plug joints are present at each end (this report uses "asphalt overlay" or simply "overlay" to indicate the existing overlay in the roadway). No overlay is present on the sidewalk. Based on Google Streetview imagery, an asphalt overlay was present in 2008 (oldest photo available) and was replaced sometime between then and 2012. The deck was cast with four drain openings each on the upstream and downstream sides of the bridge, but the drains were closed off with the installation of the overlay.

The concrete balustrade railings consist of 10-foot-long sections with six ornamental balusters supporting the top and bottom rails. Between sections are 2-foot-wide posts aligned over the floor beams. The railings are approximately 3 feet 5 inches tall at the posts and 3 feet 3 inches tall between the posts. Within the bridge span, joints are present within the posts, aligned with the abutments and deck joints (i.e., at FB-3 and FB-6). Beyond the bridge span to the north and south, slip joints with copper castings are present every 20 to 30 feet between the posts and top rails. The railings are supported on the deck within the bridge span and by the wing walls beyond. The upstream railings extend beyond the bridge span approximately 100 feet to the south and 70 feet to the north, and the downstream railings extend approximately 40 feet to the south and 50 feet to the north. The original drawings indicate that light posts were installed on top of select posts; however, the light posts were removed at an unknown date and are no longer present.

Representative views and details of the bridge deck and railings, taken from the original bridge drawings, are shown in Figure 2.2 through Figure 2.5.

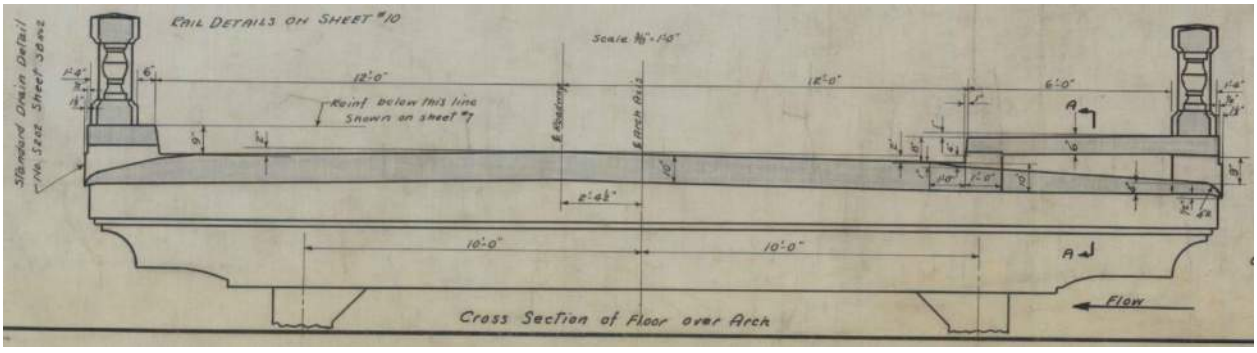


Figure 2.2. Transverse bridge deck section through drain (excerpt from original drawings). Deck concrete is shown shaded on top of the floor beam beyond. Note that no overlay was present in the original construction.

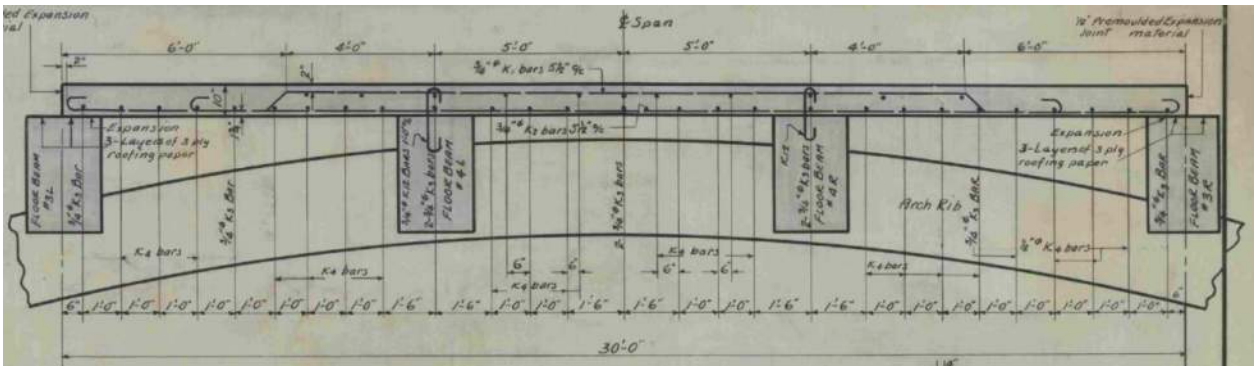


Figure 2.3. Longitudinal bridge deck section through center 30-foot deck section and floor beams FB-3 through FB-6 (excerpt from original drawings).

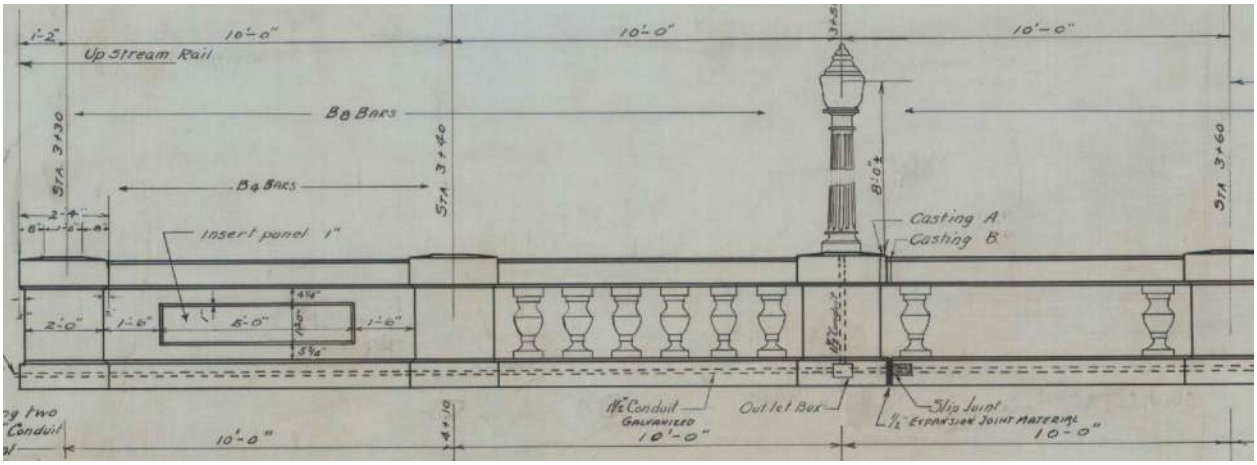


Figure 2.4. Railing elevation (excerpt from original drawings).



Figure 2.5. View of deck and railings looking south, taken during Phase 1 bridge inspection.

2.1.2. Floor Beams and Columns

The bridge deck is supported by eight concrete floor beams spaced at 10 feet on center. The floor beams are approximately 3 feet deep between arch ribs and cantilever beyond the ribs, where the depth varies to create an ornate profile. The upstream cantilevered end is built up approximately 1 foot to support the sidewalk slab. The floor beams are 2 feet wide at the bottom and 2 feet 4 inches wide at the top. FB-3, FB-4, FB-5, and FB-6 frame directly into the arch ribs, while FB-1, FB-2, FB-7, and FB-8 are supported by 1-foot 9-inch square columns that are supported on the ribs.

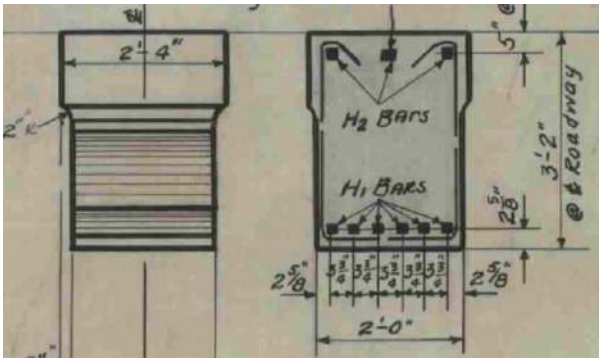


Figure 2.6. Floor beam end elevation (left) and typical section (right) (excerpt from original drawings).

The floor beams are reinforced longitudinally with 1-1/8-inch square bars with hooked ends, six at the bottom of the section and two to three at the top, and transversely with 1/2-inch-diameter U-shaped stirrups spaced at 6 to 12 inches on center.

The columns, which rise from the arch ribs to support the floor beams, are typically 1 foot 9 inches square. Each column is reinforced longitudinally with eight 1-inch square bars that are hooked at each end and extend into the floor beam and arch rib, and transversely with 3/8-inch-diameter ties at 12 inches on center.

Representative views and details of the floor beams and columns, taken from the original bridge drawings are shown in Figure 2.6 through Figure 2.11.

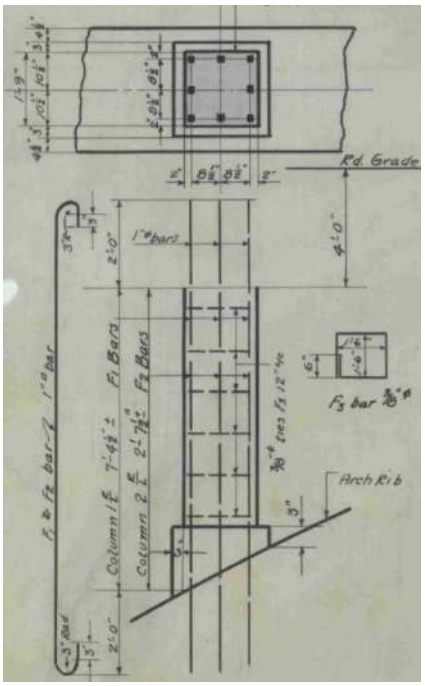


Figure 2.7. Column section and elevation (excerpt from original drawings).

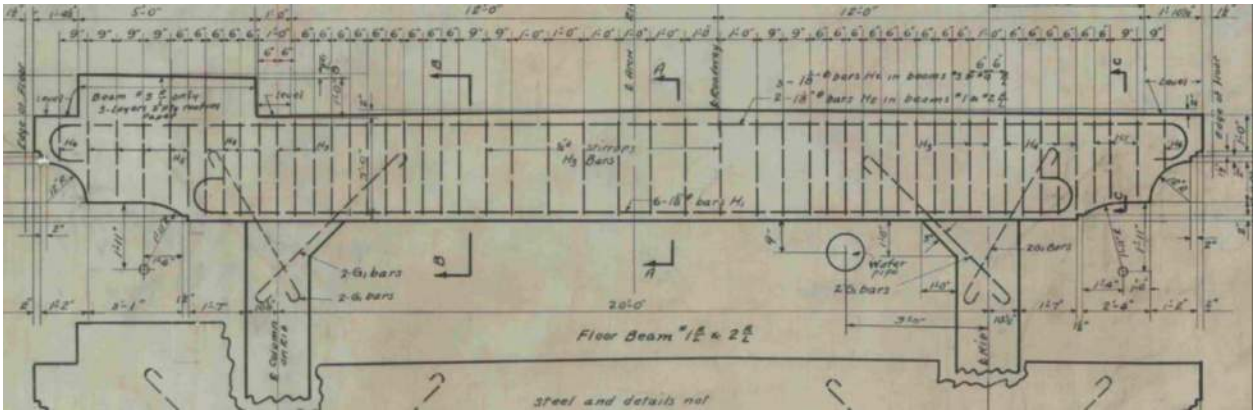


Figure 2.8. Floor beam typical elevation (excerpt from original drawings).



Figure 2.9. View of floor beams, deck underside, and arch rib undersides, looking north.



Figure 2.10. Typical upstream floor beam end.



Figure 2.11. Typical downstream floor beam end.

2.1.3. Arch Ribs

The floor beams and columns are supported by two 3-foot-wide arch ribs (also referred to as "arches"), spaced at 20 feet on center, that span between the south and north abutments. The ribs vary in depth from 2 feet 6 inches at the crown to approximately 5 feet at the spring lines. The arch ribs rise 18 feet over their 90-foot span and are laterally braced by 2-foot-3-inch by 1-foot-6-inch rectangular tie beams (also referred to as "cross beams" on the original drawings) aligned with FB-1, FB-2, FB-7, and FB-8, while lateral bracing is provided directly by FB-3 through FB-6. The ribs are reinforced longitudinally with six 1-1/8-inch square bars top and bottom near the spring line and 7/8-inch-diameter bars top and bottom in the remainder of the span. The ribs are reinforced transversely with built-up stirrups consisting of 1/2-inch-diameter hooked and U-shaped bars at 12 inches on center.

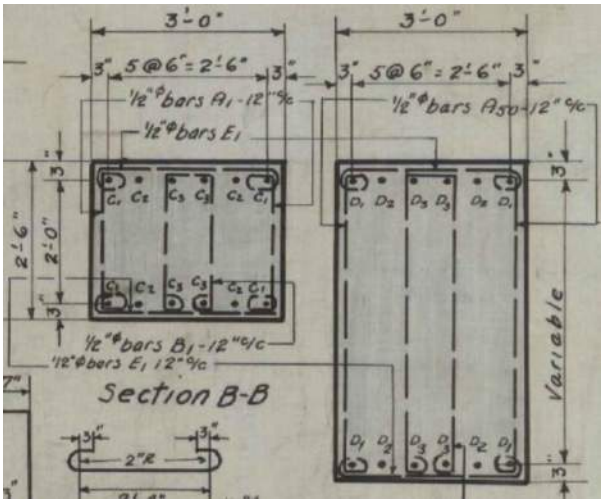


Figure 2.12. Arch rib sections through crown (left) and near spring line (right) (excerpt from original drawings).

Representative views and details of the arch ribs, taken from the original bridge drawings, are shown in Figure 2.12 through Figure 2.14.

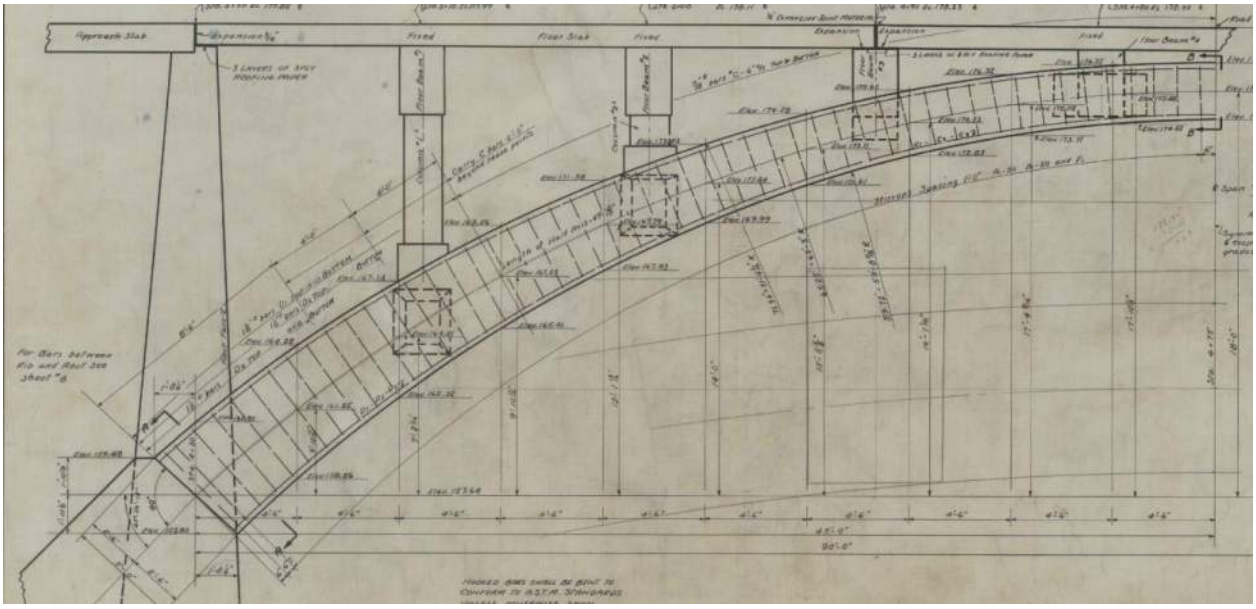

Figure 2.13. Arch rib half elevation (excerpt from original drawings).



Figure 2.14. Overall view of arch ribs and bridge underside, taken during Phase 1 inspection.

2.1.4. Abutments and Wing Walls

The north and south abutments each consist of a 25-foot-tall wall supported on an approximately 4-foot-deep spread footing. The walls are battered on both faces and are approximately 5 feet thick at the base and 2 feet thick at the top. The footings extend 3 feet beyond the exterior face of the wall and approximately 5 feet beyond the interior face. The interior faces of the walls are reinforced vertically with 1-1/8-inch square bars at variable spacing (6 inches on center at the base and 4 feet on center at the top) and horizontally with 1/2-inch-diameter bars at 2 feet on center. The exterior faces are reinforced vertically with 1/2-inch-diameter bars at 3 feet on center and horizontally with 1/2-inch-diameter bars at 1 foot on center.

At their spring lines, the arch ribs frame into what is referred to as "rib abutments" on the original drawings. The rib abutments are trapezoidal concrete masses that are integral with the "main abutment" wall, into which the longitudinal arch rib reinforcing bars are anchored. The rib abutments are 3 feet wide at their top (i.e., where the rib frames in), 12 feet wide at the base, and extend approximately 9 feet behind the main abutment wall. The bottom of the rib abutment is keyed and extends approximately 3 feet below the bottom of the main abutment footing.

Wing walls of similar construction as the main abutment walls extend beyond the abutments, following the same profile as the deck railings. The wall heights and footing elevations vary to accommodate the varying slopes of adjacent grade. The wing and abutment walls are backfilled with soil that supports the roadways and sidewalks approaching the bridge. Concrete and stone retaining walls that are not part of the bridge construction are present abutting the north and south wing walls on the upstream side.

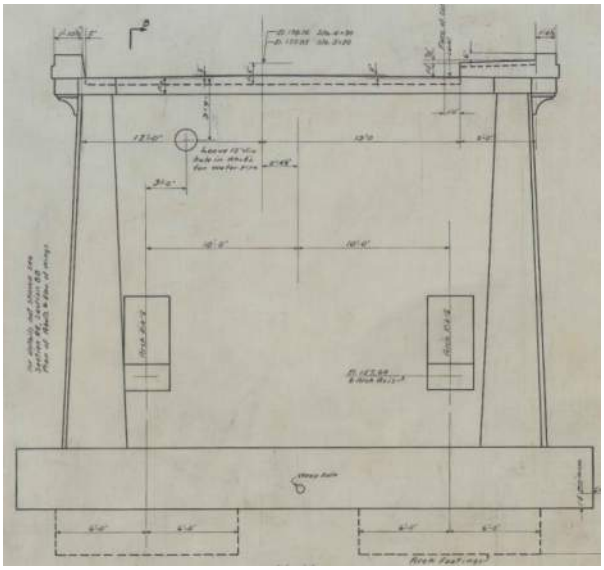


Figure 2.15. Abutment elevation (excerpt from original drawings).

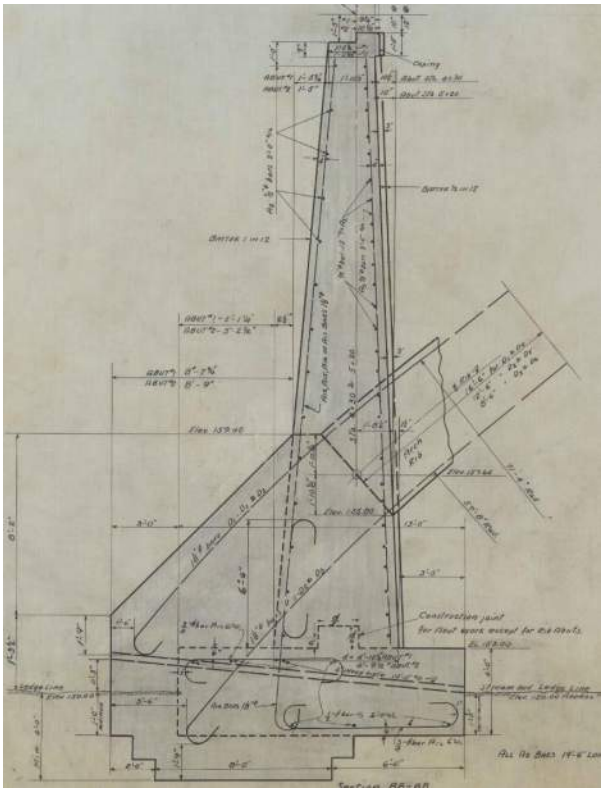


Figure 2.16. Abutment section (excerpt from original drawings).

Drains are present in the roadway shoulders within the north and south approaches. Based on the original plans, the drains are connected by 12- to 18-inch-diameter concrete or clay tile pipes that, based on observations made during our inspection, discharge at outlets located near the base of the south abutment and near the top of the upstream wing wall. Corrugated metal pipes were observed on the slopes along the north upstream and south downstream wing walls, but it was not clear whether they are connected to the roadway drains.

Representative views and details of the abutments and wing walls, taken from the original bridge drawings, are shown in Figure 2.15 through Figure 2.22.



Figure 2.17. South abutment.



Figure 2.18. North abutment.



Figure 2.19. Upstream south wing wall.



Figure 2.20. Downstream south wing wall.


Figure 2.21. Upstream north wing wall.


Figure 2.22. Downstream north wing wall.

2.1.5. Water Main and Utilities

A water main and other smaller utility conduits are suspended below the bridge deck. The water main is an approximately 12-inch-diameter steel pipe supported by steel hangers embedded in the deck underside. The water main was installed during the original construction of the bridge and is no longer in use. The utilities consist of a bundle of nine 4-inch-diameter PVC pipes supported by steel angle and threaded rod hangers mounted to the floor beams and are not original to the bridge. The water main and utilities pass into openings in the abutment walls at each end of the bridge.

2.2. Bridge Inspection and Repair History

The bridge is subject to routine inspections every two years, the most recent of which was completed on August 8, 2023. The 2023 inspection report indicates that the deck is in "fair" condition, noting distress at the deck underside and edges. Both the superstructure and substructure were noted to be in "satisfactory" condition, with comments regarding distress at floor beams FB-3 and FB-6 below the deck joints, past repairs and exposed reinforcing steel in the arches, and "minor to moderate scaling" at the abutment footings.

Records of past repair or rehabilitation projects were not provided to WJE; however, past repair efforts were evident based on observed concrete repair material on the arches and deck underside, as well as non-original concrete observed below the asphalt overlay in core samples taken through the deck by WJE as part of this assessment. The 2023 routine bridge inspection report provides a "Year Reconstructed" date of 1943, but it is not clear what work was done at that time. Street view imagery available on Google Maps[2] shows that deck work was performed between October 2008 and July 2012, including replacement of the asphalt overlay and concrete repairs at the sidewalk.

[2] Google Maps (2025). Available from https://www.google.com/maps/@43.4750639,-72.3888898,19.29z?entry=ttu&g_ep=EgoyMDI1MDExMC4wIKXMDSoASAFQAw%3D%3D [Accessed January 15, 2025].

2.3. Historical Significance

The bridge inspection reports note that the Windsor Bridge is not eligible to be listed on the National Register of Historic Places (NRHP), the reasons for which are unknown to WJE. However, it is our understanding that the Windsor Bridge is the last remaining open spandrel concrete arch bridge in Vermont and for that reason has historical significance to the State. The bridge is located approximately 100 feet downstream from the Robbins and Lawrence Armory and Machine Shop (also known as the American Precision Museum) that was listed on the NRHP as a National Historic Landmark in 1966.

3. BRIDGE INSPECTION WORK PLAN

3.1. General Approach and Methodology

As required by the Task Order, WJE developed a Draft and Final Bridge Inspection Work Plan based on review of project documents and comments from the project team. The work plan documents were submitted to HNTB for distribution to VTrans and other involved parties before the inspection began. Draft work plans were submitted on July 26, 2024, and, upon receiving and incorporating all comments, the Final Inspection Work Plan was submitted on August 8, 2024. The final plan is reproduced in APPENDIX A and is summarized below.

The overall objective of the inspection was to gather sufficient data to determine the deterioration mechanisms by element of the bridge and, on that basis, to develop service life estimates by element of the bridge, as well as considerations for structural repairs to be incorporated into the bridge rehabilitation alternatives.

The work was performed in two distinct phases: 1) an initial element level inspection of all components of the bridge, and 2) testing and material sampling in representative study areas. Information from the materials testing and condition assessment was provided to HNTB for use in their bridge analysis and load rating.

3.1.1. Phase 1 – Overall Element Level Bridge Inspection

The inspection began with an element level inspection of all above water and above grade components of the bridge in general accordance with the American Association of State Highway and Transportation Officials (AASHTO) "Manual for Bridge Element Inspection", Second Edition, 2019. The objectives of the overall inspection were to identify and document:

- The range and types of distress and deterioration conditions present in the various elements
- Signs and evidence of the root causes of the distress and deterioration conditions present
- Prudent locations for follow-up field testing and material sampling (in Phase 2)
- Locations and quantities of deterioration so that approximate repair quantity estimates can be developed

The results of the Phase 1 inspection are presented in Chapter 4 and discussed in Chapter 6. The methods and other details of the inspection are included in the full inspection plan that is appended. In general, a close-up visual inspection was performed of all exposed bridge surfaces. All locations where close-up inspection indicated conditions that may warrant a repair were mechanically sounded, and representative locations where no visual distress was observed were also mechanically sounded for confirmation.

The Phase 1 work was performed on August 12 through 15, 2024. Traffic control, which was provided by the Town of Windsor, consisted of closing the upstream/southbound lane to allow approximately 7-1/2 hours of inspection time per field day. For access to locations not accessible from ground, WJE rented an under-bridge inspection unit (UBIU or snooper), which was operated from within the southbound lane closure. Operation of the UBIU from the other lane was not possible due to overhead power lines along the downstream side of the bridge.

The following elements were included in the inspection:

- Deck: reinforced concrete deck, wearing surface, sidewalk
- Deck railings
- Superstructure: floor beams, columns, arch ribs
- Substructure: abutments, footings, and wing walls down to the waterline

The following items were excluded from the inspection:

- Drains and drain pipes
- Water main and utilities (conduit, piping, supports), except that WJE noted obviously failed supports, especially where related to concrete distress
- Retaining walls beyond abutment wing walls
- On-grade sidewalks beyond bridge span
- Elements below water-line and below grade

3.1.2. Phase 2 – Follow-Up Study Area Testing and Material Sampling

After the Phase 1 overall inspection, WJE commenced Phase 2 including non-destructive field testing and material sampling, followed by laboratory analysis of samples and synthesis of collected data. The objectives of the testing and materials evaluation work were as follows:

- Determine by testing and laboratory analysis the root causes of the distress and deterioration conditions present in the various elements
- Provide a basis for projecting future performance of the structural elements of the bridge
- Quantify the properties of the concrete and steel needed to conduct the structural load rating (by HNTB) and service life evaluation
- Provide a basis for developing conceptual repair methods and details that address the root causes of and extent (e.g., depth) of the deterioration as part of a rehabilitation to extend the life of the bridge

The deterioration mechanisms considered and the test methods employed are described in the remainder of this chapter. The results of the Phase 2 work are presented in Chapter 5 and discussed together with the Phase 1 results in Chapter 6.

The Phase 2 follow-up testing and material sampling work were performed on October 14 through 17, 2024. Access and traffic control were similar to the Phase 1 inspection. Moore/Kidder Concrete Cutting was retained as a subcontractor to WJE for removal of concrete core samples and patching the resulting holes with cementitious repair material. In total, 17 study areas were accessed close-up for follow-up testing, 18 core samples were extracted, and 5 steel samples were removed.

3.2. Background Regarding Relevant Deterioration Mechanisms

The following sections provide background regarding the deterioration mechanisms that are most relevant to historic concrete in general and the Windsor Bridge in particular. These include the following:

- Corrosion of steel embedded in concrete
  - Carbonation-induced corrosion
  - Chloride-induced corrosion
- Cyclic freezing and thawing deterioration
- Alkali-silica reaction (ASR) deterioration
- Distress from mechanical action

3.2.1. Corrosion of Steel Embedded in Concrete

Corrosion of reinforcement in concrete, initiated by carbonation and/or chloride ion contamination, is a common cause of structure degradation. Corrosion of reinforcing steel in new concrete typically does not occur because cement hydration products are highly alkaline (pH of 12.5 to 13.5) by nature, and this quickly produces a stable, thin oxide film (or passive film) on the surface of reinforcing bars embedded in concrete. This passive film impedes corrosion. There are two primary mechanisms that can develop as the structure ages, resulting in the destruction of the passive film (depassivation) and causing corrosion of reinforcing steel: carbonation and chloride ion contamination.

When these two processes, singularly or in combination, are coupled with moisture and oxygen, corrosion of the reinforcing bars in the concrete will proceed. Where the depassivation occurs first, the steel becomes anodic (or corrodes) and supports the reaction that, in the presence of water, produces red rust (hydrated ferric oxide) and other corrosion products. Adjacent areas of the steel become cathodic (non-corroding), where oxygen and water react. Both anodic and cathodic reactions, in combination with an electronic current path (the steel) and an ionic current path (the concrete) are needed to complete the corrosion cell. Once the corrosion cell develops, the corrosion products (rust) that result occupy a much larger volume than the steel from which they were formed. This increased volume leads to expansive pressures inside the concrete that result in cracking, delamination, and ultimately spalling of the cover concrete.

The rate at which corrosion proceeds is controlled by many factors, such as dissolved oxygen availability, moisture content, resistivity of concrete, and temperature. Concrete acts as an impediment to the flow of water, chloride ions, carbonation reactions, and oxygen. Thus, the depth of cover over the bars, the presence of cracks, and the permeability of the concrete all influence the rate that corrosion will occur. It is a rule of thumb that corrosion rates of steel in concrete typically double for a temperature increase of 18°F (Tuutti, 1982)[3], although it has been suggested that the rate may increase by as much as a factor of five for that magnitude of temperature increase (Broomfield, 2007)[4]. The ratio of the anodic area to

[3] Tuutti, K. 1982. *Corrosion of Steel in Concrete.* Stockholm, Sweden: Swedish Cement and Concrete Research Institute.

[4] Broomfield, John P. 2007. *Corrosion of Steel in Concrete.* New York: Taylor and Francis.

cathodic area can also control the corrosion rate; the condition where small anodes are surrounded by large cathodes produces the most rapid corrosion.

3.2.1.1. Carbonation-Induced Corrosion

Carbonation of concrete occurs when carbon dioxide present in the air diffuses through pores in the concrete and reacts with moisture and cement hydration products within the concrete. The main reaction is calcium hydroxide within the cement paste reacting with carbon dioxide to form calcium carbonate. Carbonation of portland cement paste has two distinct effects, one chemical and one physical. The chemical effect is to lower the pH of the pore solution from approximately 13 to about 9 or less. The protective passive film on the bar starts to break down at a pH of 10 to 11, permitting active corrosion to develop (Broomfield, 2007). The physical effects of carbonation are irreversible shrinkage and a moderate increase in density of the carbonated layer. Carbonation also can free chloride ions that were chemically bound in the aluminate phases of the cement paste, further aggravating corrosion of embedded steel.

Once depassivation has occurred and sufficient oxygen is available, the corrosion rate in concrete is strongly influenced by the resistivity of the concrete (Alonso, Andrade, & Gonzalez, 1988)[5]. This is because concrete forms the ionic current path, and a more resistive concrete will slow current. The resistivity of concrete is strongly influenced by moisture in the concrete; this can be quantified in relation to the relative humidity within the concrete (Enevoldsen, Hansson, & Hope, 1994)[6]. The relationship between corrosion rate and relative humidity is different depending on whether the corrosion is prompted by carbonation or chloride contamination (Broomfield, 2007). Corrosion rates increase as relative humidity within the concrete increases beyond 70 percent in chloride-contaminated concrete and beyond 85 percent in carbonated concrete. The corrosion rates reach a peak at 95 to 97 percent relative humidity, above which the additional moisture in the concrete impedes the ingress of the oxygen necessary to support the cathodic reaction.

In practical terms, when the carbonation depth reaches the depth of the reinforcing steel, the reinforcement is no longer protected by an alkaline environment and corrosion of that reinforcement may ensue. Carbonation-induced corrosion generally manifests first at the reinforcement that has the least cover, and as such is often seen at shallow cover conditions.

Although not carbonation-induced corrosion, another related corrosion mechanism is atmospheric corrosion. Atmospheric corrosion occurs when steel is directly exposed to the atmosphere, such as when reinforcing steel is exposed within a “rock pocket” (voided area due to original poor consolidation) at the concrete surface or when the reinforcing steel cover is so shallow that the bar is partly exposed on the concrete surface.

[5] Alonso, C., Andrade, C., & Gonzalez, J. A. 1988. “Relation between resistivity and corrosion rate of reinforcements in carbonated mortar made with several cement types.” *Cement and Concrete Research, 18*(5), 687-698.

[6] Enevoldsen, J. N., C. M. Hansson, and B. B. Hope. 1994. "The Influence of Internal Relative Humidity on the Rate of Corrosion of Steel Embedded in Concrete and Mortar." *Cement and Concrete Research* 24 (7): 1373-1382.

3.2.1.2. Chloride-Induced Corrosion

In the absence of carbonation, chloride ions must accumulate to a critical concentration for corrosion to initiate on reinforcing steel that is embedded in sound, alkaline concrete. The onset of corrosion is governed by the time required for chloride in the environment to penetrate through the concrete cover over the steel and build up at the bar depth to levels above the chloride threshold value. Chloride ions can also be present in the concrete from initial construction in the form of admixtures used to accelerate strength gain or in contaminated aggregate, such as sea sand.

When chloride ions accumulate to a sufficient concentration—known as the chloride corrosion threshold—the naturally occurring passive oxide film protecting the steel from corrosion breaks down. The chloride corrosion threshold is dependent on a number of conditions within the concrete, including cement content and chemistry, moisture conditions, steel chemistry and surface conditions, and proximity and condition of other embedded steel elements. The chloride corrosion threshold for initiation of corrosion of embedded mild, uncoated steel is considered to be approximately 0.20 percent total (acid-soluble) chloride by weight of cement in non-carbonated concrete (Broomfield, 2007); note, as stated below and later in this report, this is equivalent to approximately 0.030 percent weight concrete for typical modern concrete mix designs. This is the lowest chloride concentration at which corrosion may be expected to initiate, provided that all other conditions conducive to corrosion are present. The likelihood, severity, and rate of corrosion increase as chloride concentrations increase above this threshold level.

It is important to recognize that corrosion of the reinforcing is not certain at chloride concentrations at and above the chloride corrosion threshold, since multiple environmental factors affect the influence of chloride concentration on corrosion. Further, since testing of existing structures is performed on samples of concrete, not cement, a conversion is needed based on the content of cement in the concrete mix and the unit weight of the sampled concrete. For typical normal weight concrete, a value of 0.030 to 0.035 percent by weight of concrete is often cited as the chloride threshold within the construction industry, as this provides a conservative limit to prevent corrosion. At the same time, published literature contains widely varying statements about the chloride corrosion threshold, indicating that there is not a consensus within the industry and practice.

Based on available research and WJE's experience, the risk of corrosion due to elevated chloride concentrations can be interpreted in detail as follows:

Table 3.1. Summary of Interpretation Thresholds for Chloride Concentrations

Risk of chloride-induced corrosion	Uncoated (Black Bar) Reinforcement		Epoxy-Coated Reinforcement	
	(% chloride by weight of cement)[1]	(% chloride by weight of concrete)[2]	(% chloride by weight of cement)[3]	(% chloride by weight of concrete)[2]
Very Low	<0.2	<0.03	<0.3	< 0.05
Low	0.2-0.4	0.03-0.06	0.3-0.7	0.05- 0.11
Moderate	0.4-1.0	0.06-0.15	0.7-1.9	0.11-0.29
High	>1.0	>0.15	> 1.9	>0.29

[1] *From (Broomfield, 2007)*

[2] *Concentrations by weight of concrete assume cement content of 564 lbs/cu yd (i.e., a 6 bag mix)*

[3] *Based on previous work performed by WJE (Lawler et al. 2021)* [7]

The above threshold values refer to bare uncoated steel reinforcing bars and epoxy-coated reinforcing bars in concrete, separately. The chloride content required to initiate corrosion on epoxy coated reinforcing bars is higher.

Chloride ions can be chemically or physically bound to the cement paste as they ingress into the material. This chloride is typically referred to as bound chloride; in contrast, chloride remaining dissolved in the pore solution is referred to as free chloride. Because the chloride binding is reversible, depending on both chloride concentration and pore solution pH, testing for total chloride content (bound plus free chloride) is used as the basis for quantifying concentrations and for interpretation of corrosion thresholds.

Chloride contamination and carbonation exhibit a synergistic effect, promoting corrosion when both occur in concrete beyond what would be expected by one mechanism alone. If the concrete is carbonated, with a pH less than approximately 10, the presence of even low levels of chloride will encourage corrosion of mild steel. In addition, chloride is hygroscopic and tends to keep moisture within the concrete. Furthermore, the presence of chloride lowers the resistivity of concrete, supporting more rapid corrosion rates (Enevoldsen, Hansson, & Hope, 1994).

3.2.2. Cyclic Freezing and Thawing Deterioration

Cyclic freezing and thawing deterioration (F/T deterioration) occurs in non-air-entrained concrete when the concrete becomes saturated with water and the moisture trapped within the concrete pore structure is subjected to freezing. Concrete distress occurs because water increases in volume as it freezes, generating high capillary water pressure within the concrete. When internal concrete pressure exceeds the concrete's tensile strength, the concrete may crack. Cracking of the concrete permits additional ingress of moisture, and subsequent freezing and thawing events often lead to progressively deeper deterioration, which

[7] John Lawler, Jonah Kurth, Stephen Garrett and Paul Krauss, "Statistical Distributions for Chloride Threshold of Reinforcing Bars," ACI Materials Journal, V.118, Issue 2, March 2021 (DOI: 10.14359/51730411).

typically manifests itself as parallel planes of narrow cracks. This mechanism eventually results in erosion (disintegration and loss) of the concrete surface as pieces of loose concrete become dislodged. More rapid deterioration is common within areas of greater moisture exposure, including below drain outlets and at the corners and waterline regions of concrete elements situated in bodies of water. Concretes with higher porosity associated with higher water-to-cement ratio are typically more susceptible to freeze-thaw distress.

Modern concrete, that is concrete produced after about the 1940s, relies on entrained air to avoid freezing and thawing deterioration. Entrained air is a network of fine, closely spaced air bubbles that are intentionally created in fresh concrete through the use of chemical admixtures. These bubbles remain in the concrete after it hardens and provide spaces where water can move when nearby moisture within the concrete freezes. This relieves the pressures created by freezing and thawing exposure and prevents damage to the concrete.

Note that shallow, surficial freeze-thaw deterioration is often termed "scaling." Most of the freeze-thaw deterioration that is present in the Windsor Bridge is deeper and not merely surficial. As such, the term "freeze-thaw deterioration" is generally used in this report, referring to subsurface deterioration.

3.2.3. Alkali-Silica Reaction Deterioration

Alkali-silica reaction (ASR) is a deleterious, expansive, secondary reaction that can occur in concrete that contains sufficient amounts of alkali-hydroxides in the cement paste, as well as certain types of susceptible silica-bearing rock particles in the aggregate (commonly referred to as "reactive aggregates"). For ASR to take place, the concrete must also have a relative humidity of at least 80 percent. Both the composition and the internal structure of a rock are important factors that affect a rock's susceptibility to reaction.

The reaction produces a gel (microcrystalline material) that is especially absorptive and can expand if moisture is available. Pressure is exerted on the surrounding concrete as the gel and the particles swell. If this pressure exceeds the strength of the surrounding concrete, cracking will occur. Classical ASR, as described above, where the three conditions required for ASR are present throughout the concrete can take as long as 10 years to develop and often eventually affects the entire depth of the structure. However, variations in the alkali concentration, reactive aggregates, and moisture distributions may influence the location and the severity of damage to the concrete where the reaction occurs.

Evidence of ASR includes ASR gel or associated reaction product on the boundary of aggregates (also known as reaction rims), paste darkening, microcracks, and cracks. When the ASR gel becomes abundant, cracks typically extend from the reactive aggregates into the paste. The severity of ASR distress in a structure is influenced by the ASR development and the characteristics of the material and structure type, including but not limited to mixture proportions, exposure conditions (moisture level and temperature), reinforcement, and type and geometry of the structure.

Both ASR and F/T deterioration require abundant moisture in the concrete. ASR cracks usually originate in or near reactive aggregate particles and then extend outward into the paste, whereas F/T cracks usually occur primarily in the paste. Nonetheless, if F/T cracks develop, they allow more water to penetrate the

concrete, which can trigger or exacerbate ASR distress. In this manner, the two mechanisms can be synergistic.

3.2.4. Distress from Mechanical Action

Distress from mechanical action occurs when external forces cause physical damage to structural elements. Mechanical action distress is also called structural distress, especially when the distress affects the structural performance of an element. Examples include structural cracking or spalling of concrete elements due to external loads or forced displacements, such as the loads/displacements that thermal expansion and contraction may impose on the columns of open spandrel concrete arch bridges. Distress from mechanical action is in contrast to deterioration that develops in an element over time due to internal stresses from material degradation, such as from corrosion of reinforcing steel or freezing and thawing.

3.3. Follow-Up Testing Methods

3.3.1. Field Testing

3.3.1.1. Material Sampling

The material samples collected for laboratory testing included concrete cores, concrete powder samples, and steel reinforcing bars. The conditions of these samples were documented in the field after removal from the structure and upon receipt in WJE's laboratory in Northbrook, Illinois. The samples collected are summarized in Chapter 5 and the appendices.

3.3.1.1.1. Concrete Core Removal and Powder Sampling

Concrete core samples were extracted with a diamond impregnated, water-cooled core drill. After removal, the core holes were photographed and then patched with a rapid-setting, cementitious repair material. The condition of each core was documented with photographs and notes, including location on the structure and condition of the sample. Concrete samples were selected for either compressive strength testing, chloride concentration profile evaluation, evaluation by petrographic analysis, or some combination of these tests.

Powder samples were collected through drilling into the concrete soffit (deck underside) at incremental depths. The powder samples were tested only for chloride concentration, to supplement the core sample testing.



Figure 3.1. Example of concrete core removal from within the UBIU basket.



Figure 3.2. Example of concrete core removal from grade.

3.3.1.1.2. Reinforcing Bar Removal

Reinforcing bar samples were extracted from locations where cover concrete had spalled exposing the embedded reinforcing steel. Locations were selected in coordination with HNTB from locations of relatively low element stress. Steel samples were tested for tensile strength and chemical composition.



Figure 3.3. Samples removed from redundant reinforcing bars at spalled field modification near north end of upstream arch rib (cut end of bars at red arrows).



Figure 3.4. Various reinforcing steel samples removed from spalled area of previous deck drain.

3.3.1.2. Delamination Survey

As part of the Phase 1 overall inspection and the Phase 2 study area testing, selected regions of concrete were mechanically-sounded and visually assessed for delamination (i.e., laminar separation of the top layer of concrete from the substrate concrete often caused by corrosion of embedded reinforcing steel). The deck topside was surveyed for delaminations by chain dragging the asphalt overlay and the exposed concrete sidewalk; other concrete elements were hammer-sounded. Note that chain dragging the asphalt overlay can detect debonding of the overlay but most likely does not detect delamination of the concrete slab below. Assessment methods were in general accordance with ASTM D4580, *Standard Practice for Measuring Delaminations in Concrete*.

3.3.1.3. Corrosion Survey

Corrosion surveys were performed to assess corrosion of the embedded reinforcing steel using half-cell potential testing, concrete resistivity measurements, and corrosion rate testing, where applicable. These test methods are described below. Note that corrosion rate testing (using the iCOR instrument) was not successful in some study areas due to existing conditions including dry and possibly frozen cover concrete, deeper reinforcement cover, and limitations of the test equipment.

3.3.1.3.1. Corrosion Potential (Half-Cell Potential Testing)

Half-cell potential (HCP) testing provides an indication of corrosion risk for reinforcing steel in concrete. Highly negative potential (voltage) readings indicate a high probability that active corrosion is occurring. HCP measurements do not locate spalls, delaminations, or other damage sites; however, these conditions are often associated with corrosion, and thus usually coincide with more negative potential readings. Anodic (corroding) regions that have not yet caused delaminations or spalls can be identified by this technique, and thus HCPs can be used as an indicator of regions likely to become damaged by corrosion in the near future.

WJE performed HCP testing in general accordance with ASTM C876 *Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete* over selected areas of the structure. The HCP surveys were performed by establishing an electrical connection (ground) to the reinforcement and placing a reference electrode on the surface of the concrete. Before commencing HCP measurements, electrical continuity testing was performed in each portion of the structure to verify the electrical continuity between two distant electrical connections to the reinforcing steel. Potentials were measured using a standard copper-copper sulfate half-cell reference electrode and a high-impedance volt meter. Potential measurements were performed in a grid pattern or over areas of interest (e.g., over reinforcing bars). All HCP measurements are presented in terms of mV versus copper-copper sulfate electrode (CSE) and have been corrected for temperature to equivalent value at 75 ºF, in accordance with ASTM C876.

Half-cell potentials can be influenced by a number of parameters, including temperature, concrete resistivity, measurement circuit resistivity, and electrochemical conditions at the steel reinforcement. Concrete resistivity is affected by moisture, chloride content, and surface carbonation, Electrochemical conditions at the steel are affected by the cement pore chemistry, oxygen availability, and chloride concentration. Saturated concrete causes very negative potentials because the oxygen availability is limited, and thus affects the passive film on the bar. Carbonated concrete has a higher resistivity than

uncarbonated concrete and can result in positive potentials, but if the carbonated layer is deep enough it may also destabilize the passive film on the bar and allow corrosion.

Typical ranges for half-cell potentials in a number of conditions are provided in Table 3.2 along with an associated risk of corrosion. Separately, guidelines for interpretation of the half-cell data per ASTM C876 are shown in Table 3.3.

Table 3.2. Typical Half-Cell Potential Ranges (RILEM TC-154)

Concrete Condition	Typical Range of Half-cell potentials, mV vs CSE, with [risk of corrosion activity]		
	Chloride-Contaminated	Carbonated	Chloride-Free
Humid, non-saturated	-600 to -400 [high]	-400 to +100 [moderate]	-200 to +100 [low]
Saturated, oxygen-starved	-1000 to -900 [low]	no data	-1000 to -900 [low]
Dry	no data	0 to +200 [low]	0 to +200 [low]

Table 3.3. Half-Cell Potential Corrosion Risk (ASTM C876)

Uncarbonated or Chloride-Driven Corrosion (based on uncoated rebar in non-saturated conditions)	
HCP vs. CSE	Corrosion Activity
> -200 mV	low - 10% probability of corrosion
-200 to -350 mV	moderate - increasing probability of corrosion
< -350 mV	high - 90% probability of corrosion

Interpretation of HCPs using the guidelines in ASTM C876 (Table 3.3) is generally applicable for chloride-induced corrosion in uncarbonated, atmospherically-exposed elements, and may be applicable to epoxy-coated bars if the coating has a number of damage locations and good electrical connectivity. However, these guidelines are not applicable where moisture conditions are highly variable and carbonation has developed. In such conditions, interpretation of corrosion risk by examination of potential gradients is preferred over assessment of risk by absolute potential values. Areas of more negative potential surrounded by areas of more positive potential suggest the areas of more negative potential are anodic and likely undergoing corrosion, while those of more positive potential are cathodic and not generating corrosion product. Other things being equal, higher corrosion currents will flow between points of high and low potential that are closer to each other. As a guideline, potential differences of 150 mV over a 3-foot distance indicate active corrosion.



Figure 3.5. HCP survey with CSE electrode and volt meter (tie beam).



Figure 3.6. HCP survey with CSE electrode and volt meter (column).

3.3.1.3.2. Concrete Resistivity

Concrete surface resistivity was measured with a four-pin Wenner-type probe (Respiod by Proceq, or iCOR by Giatech). Prior to taking readings, reinforcing steel in the vicinity was located with ground-penetrating radar (GPR). Readings were taken avoiding reinforcing bars, or if unavoidable, perpendicular to a single bar such that the bar did not influence the current path between probe points.

Concrete resistivity is correlated with the rate of corrosion in concrete structures. As resistivity increases, the rate of corrosion decreases at locations where corrosion is occurring. The categories described in Table 3.4 were used to correlate the resistivity measurement to the rate of corrosion.

Table 3.4. Interpretation of Resistivity Measurements

Resistivity (kΩ-cm)	Corrosion Rate
> 100	Negligible. Cannot distinguish between active and passive steel.
50 to 100	Low. Corrosion rates likely to be low.
10 to 50	Moderate. Moderate to high corrosion rates possible in active areas.
< 10	High. Resistivity is not the controlling factor in corrosion rates.

Sources: (Broomfield, 2007); iCOR operating manual; Resipod operating manual.


Figure 3.7. Resistivity survey with Resipod (tie beam).


Figure 3.8. Resistivity survey with Resipod (column).

3.3.1.3.3. Corrosion Rate

Instantaneous corrosion rate testing was performed to identify locations and rate of active corrosion of reinforcement. The corrosion rate was measured at selected locations where the half-cell potentials indicated corrosion might be occurring. Corrosion is a dynamic process, and the results presented are instantaneous measurements of the rate of corrosion; consequently, changes in the site conditions, particularly temperature and moisture content, will influence the rate of corrosion. Note that the calculation of the corrosion rate is sensitive to many factors, including assumptions made about the surface area and the corrosion activity of the steel polarized during the test. Therefore, some variability in corrosion rate measurements should be expected (Broomfield, 2007). The purpose of corrosion rate testing is to supplement the findings from other corrosion assessment methods, rather than to be used independently as an indicator of future performance.

The corrosion rate was measured using the Connection-less Electrical Pulse Response Analysis (CEPRA) technique, using the iCOR by Giatec. This method has known sensitivities to surface condition and concrete cover; however, it is an industry accepted method for instantaneous corrosion readings.

Using the CEPRA technique, the instrument measures the electrical response of a reinforcing bar to constant AC current. The frequency of the current is swept low to high, and the system response is analyzed. Because the response to the current sweep of a corroding rebar is different from that of a non-corroding rebar, the rate of corrosion can be assessed. This is illustrated schematically in Figure 3.9 and Figure 3.10 below. Concrete resistivity is also measured during this process.

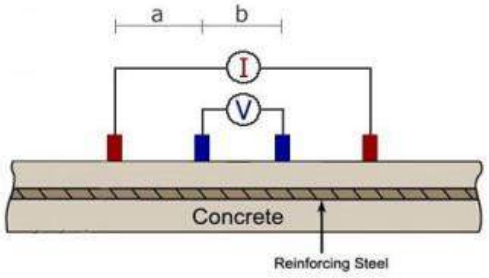


Figure 3.9. Configuration of four probes on the surface of concrete in CEPRA corrosion rate testing (from iCOR manual).

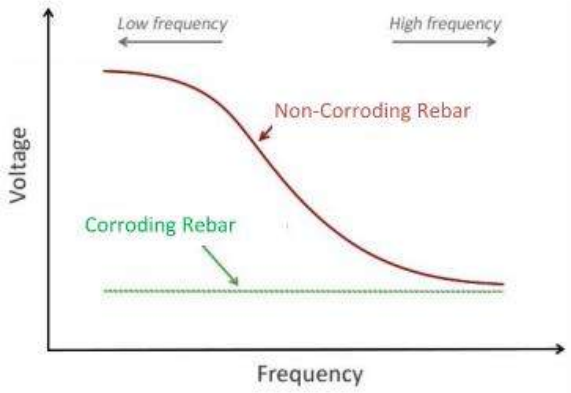


Figure 3.10. Schematic illustration of voltage-frequency response of a corroding and non-corroding rebar, as evaluated by CEPRA corrosion rate testing (from iCOR manual).

Qualitative interpretations of ranges of corrosion rate measurements are summarized in Table 3.5. Note, these values were developed for uncoated reinforcing bars. Epoxy coating on the bars can limit the area that is available to participate in the measurement but, based on WJE's previous experience with corrosion rate testing of epoxy-coated bars, these interpretations of corrosion rate are still reasonable.

Table 3.5. Interpretation of Corrosion Rate Measurements

Corrosion Rate from iCOR (µA/cm^2)*	Classification
< 1.0	Passive / Low
1 to 3	Moderate
3 to 10	High
> 10	Severe

** Values based on manufacturer's literature*

3.3.1.4. Reinforcement Cover Survey

To determine locations and concrete cover of reinforcing steel, ground penetrating radar (GPR) was used in combination with limited drilling to bars for direct measurement of cover (i.e., calibration/verification). GPR involves the use of a high-frequency radar antenna, which transmits electromagnetic radar pulses along a discrete longitudinal scan at the surface of a structural element. Electromagnetic signals reflected from material interfaces having different dielectric properties (i.e., reinforcing steel embedded in concrete) are collected by the antennae and displayed graphically for direct interpretation in the field. Guidelines for GPR considered during this work included ACI 228.2R *Nondestructive Test Methods for Evaluation of Concrete in Structures* and ASTM D6432 *Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface Investigation.*

GPR testing was completed using a handheld GPR 'StructureScan Mini' unit manufactured by GSSI, operating at a central frequency of 2600 MHz. The results of the testing were verified and calibrated by drilling holes to bars and directly measuring the cover depth at representative locations in each study area; these drill holes were also used for half-cell ground locations. These methods were also used in combination with visual assessment and sounding to select locations for core sampling.



Figure 3.11. GPR survey on arch rib underside.



Figure 3.12. GPR survey on deck topside through asphalt overlay (core removal in background).

3.3.1.5. Field Carbonation Testing

Carbonation of concrete was measured in the field by applying a pH indicator (phenolphthalein) to freshly exposed concrete surfaces. The pH indicator was applied to most core samples immediately after extraction. Also, in the study areas, incremental drilling was performed (at approximately 1/16-inch increments) and the pH indicator was applied to the drill holes after each increment. The pH indicator changes from colorless to bright purple above a pH of about 8.5, which is approximately the pH threshold level associated with depassivation of the steel due to carbonation. The depth at which the color change was observed is assumed to be representative of the depth of the carbonation front. These values were recorded for the elements surveyed.

Carbonation testing was also repeated on fresh, fractured surfaces on concrete core samples as part of the petrographic examination of cores.



Figure 3.13. Field carbonation measurement by incremental drilling and pH indicator.



Figure 3.14. Example of incremental drilling to determine depth of carbonation.

3.3.2. Laboratory Analysis

Materials testing was performed at WJE's laboratory facility in Northbrook, Illinois as described below.

3.3.2.1. Concrete Compressive Strength and Elastic Modulus

The compressive strength testing was performed on 13 concrete core samples removed from the abutments (2 cores), arches (4 cores), columns (1 core), deck (2 cores), floor beams (3 cores), and wing walls (1 core). Core samples were prepared and tested in general accordance with ASTM C42 *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*. Per ASTM C42, length/diameter (L/D) correction factors were applied as appropriate. Results of the compressive strength tests were reported to HNTB soon after the Phase 2 field work so that they could be considered in the bridge load rating.

3.3.2.2. Chloride Testing

Chloride content testing was performed on selected core samples and the powder samples to determine whether chloride contamination from deicing salts has accumulated to sufficient levels to promote corrosion. Selected core samples were cut into slices at pre-determined depths, and the slices were pulverized to facilitate acid-soluble chloride content analysis in general conformance with ASTM C1152 *Test Method for Acid-Soluble Chloride in Mortar and Concrete*. The acid-soluble chloride content represents both water-soluble and chemically-bound chloride within the cement paste and any chloride that may be present in the aggregate. Chloride concentrations listed in this report are given in terms of percent by weight of concrete.

Typically, three to six 1/8- to 1/4-inch-thick slices were cut from the tested core samples, at depths ranging from 1/4 to 4 inches from the exterior surface. Slices were cut at increments from the exposed surface at depths equal to or exceeding the depth of reinforcing steel. On select cores, chloride concentrations were measured at a sufficient depth such that diffusion of surface chlorides was unlikely, so that any measured chloride concentration could be assumed to be an initial chloride concentration or "background" chloride concentration. Based on the obtained chloride profiles, which taper to near zero chloride levels with depth from the surface, no background (e.g., admixed chloride as an accelerator) or bound chloride (e.g., in the aggregate) is present in the tested samples.

The water-soluble chloride content was determined for the powder samples removed from the deck underside, as well as representative slices from the cores to confirm no bound chlorides were present. Water-soluble chloride analysis was performed in general accordance with ASTM C1218, *Standard Test Method for Water-Soluble Chloride in Mortar and Concrete*.

3.3.2.3. Petrographic Analysis

Concrete core samples selected for petrographic analysis were examined visually and microscopically in accordance with the applicable procedures of ASTM C 856 *Standard Practice for Petrographic Examination of Hardened Concrete*. A petrographic examination consists of a series of observations that are interpreted to draw conclusions about the composition, quality, and cause(s) of any deterioration present. Testing was performed by WJE petrographers at WJE's laboratories.

The core samples were first visually assessed to categorize the macro-characteristics of the concrete and different concrete mixtures present. Then, based on these results, in-depth petrographic examinations were performed on samples from representative elements and condition states.

For the in-depth petrographic examinations, each core was generally cut in half parallel to the core axis using a continuous-rim, water-cooled diamond saw blade. One of the resulting plane surfaces was lapped (ground smooth and polished) using progressively finer grinding abrasives or diamond-impregnated plates. A thin section (a 15- to 20-micrometer thick slice taken from the concrete specimen) was also prepared for selected cores. The lapped surfaces and the thin sections were examined microscopically using methods outlined in ASTM C856. Powder immersion mounts of the paste from selected cores were also prepared and examined to determine the composition of cementitious materials and secondary deposits. Depth of carbonation from the surface was also tested by breaking the cores and applying phenolphthalein solution and/or universal pH solution onto freshly exposed concrete surfaces.

In some cases termed "limited" or "partial" petrographic examinations, the examinations focused on macroscopic conditions with the concrete like the extent of visible cracking and apparent freeze-thaw distress and did not include thin section analysis.

3.3.2.4. Reinforcing Steel Strength and Composition Testing

Tensile strength testing and chemical composition determination were conducted on the reinforcing steel samples removed from the bridge. Testing was performed by WJE metallurgists at WJE's laboratories.

Tension tests were performed using a 120-kip Satec universal test machine and 500-kip Riehle test machine with mechanical and pneumatic grips, respectively. Tests were performed in general accordance with the tension test methods specified in ASTM A370-24 "*Standard Test Methods and Definitions for*

Mechanical Testing of Steel Products." The grade of the reinforcing bar was determined by comparing the yield strength and tensile strength results to the requirements specified in ASTM A15 (1914) *Standard Specification for Billet-Steel Concrete Reinforcing Bars,* the reinforcing bar standard in effect at the time of bridge construction.

Chemical composition tests were performed using an optical emission spectroscopy (OES) in accordance with ASTM E415-21, *Standard Test Method for Analysis of Carbon and Low-Alloy Steel by Spark Atomic Emission Spectrometry*. Chemical composition results were compared with the chemical composition properties specified for all grades of ASTM A15 (1914) deformed reinforcing bars.

4. PHASE 1 - OVERALL ELEMENT LEVEL INSPECTION RESULTS

4.1. Documentation of Inspection Conditions

Phase 1 included an overall element level inspection of the bridge from end to end. Inspection observations, notes, and locations of photographs were documented electronically using tablet computers (iPads) prepopulated with the relevant elevation and plan view field sheets. The field sheets were developed by WJE based on review of original drawing information provided by HNTB and VTrans, as well as from field observations and photographs where drawings were not available or complete.

Plannotate – Electronic Data Collection

Plannotate® is WJE's in-house electronic field-note platform. Deployed on iPads for use in the field, it allows multiple users to make annotations to PDFs in real-time and tag photographs to specific locations. Data from Plannotate is uploaded to a remote cloud server and stored in a database so that engineers in the field and in the office are updated on status in real-time. This system also allows project work to stop and resume exactly where the last inspector left off, which provides flexibility in performing assessments when access may be limited to irregular time periods or operational demands. It also aids in subsequent inspections because inspectors will have a record of previous inspections findings (such as in Phase 2 of this project). Because WJE developed the software in-house, the Plannotate system is customizable by WJE software experts to fit the needs of any particular structure or defined assessment plan. As such, Plannotate allows for improved efficiency, organization and consistency in data collection.

Condition State Definitions

For the Phase 1 inspection, Plannotate was customized to record conditions and observations in line with the condition states defined in the AASHTO "Manual for Bridge Element Inspection" and as customized for specific applications by some State Departments of Transportation (DOT).[8] This includes definitions of cracking, delaminations, spalls, efflorescence, moisture/rust staining, patches, scale, wear and abrasion. WJE added other types of concrete distress that were applicable to this inspection but not specifically defined in the AASHTO guide, namely freeze-thaw deterioration conditions states.

The condition state definitions utilized for this project are summarized in Table 4.1 below, and the legend showing their representation on the Plannotate field sheets is reproduced in Figure 4.1. Note that the colors darken as the severity of the condition worsens. Representative photographs of the non-standard "Freeze-Thaw" condition states created by WJE for this project are provided in Figure 4.2 through Figure 4.5.

[8] Vermont does not have a state-specific customization, so this assessment relied on the federal standard, documents published by other states, and WJE's experience with this bridge type.

Table 4.1. Customized Plannotate Condition States for Overall Inspection

Condition State	Units	Definition for Purpose of Inspection			
		CS1	CS2	CS3	CS4
		Good	Fair	Poor	Severe
Crack	LF	<0.012" or sealed	0.012 to 0.050"	> 0.050"	Structural impact
Delamination, spall, or exposed rebar	SF	N/A	Delamination, or spall < 1" deep	Spall 1-4" deep or exposed bars	Spall >4" deep or severe rebar section loss
Efflorescence	SF	N/A	Light leaching, little build-up	Heavy leaching with build-up	N/A
Freeze-thaw distress	SF	Map cracking only	Fair, surface scaling, no exposed bars	Poor, up to 2" deep, no exposed bars	Severe, 2" or more deep, exposed bars
Moisture or rust staining	SF	N/A	Moisture or minor rust stains (chairs)	Moderate moisture of severe rust stains	N/A
Patch	SF	N/A	Sound patch	Deteriorated patch	Immediate repair recommended
Scale, wear, or abrasion	SF	<1/4" deep	1/4" to 1/2" deep	1/2" to 1-1/2" deep	> 1-1/2" deep

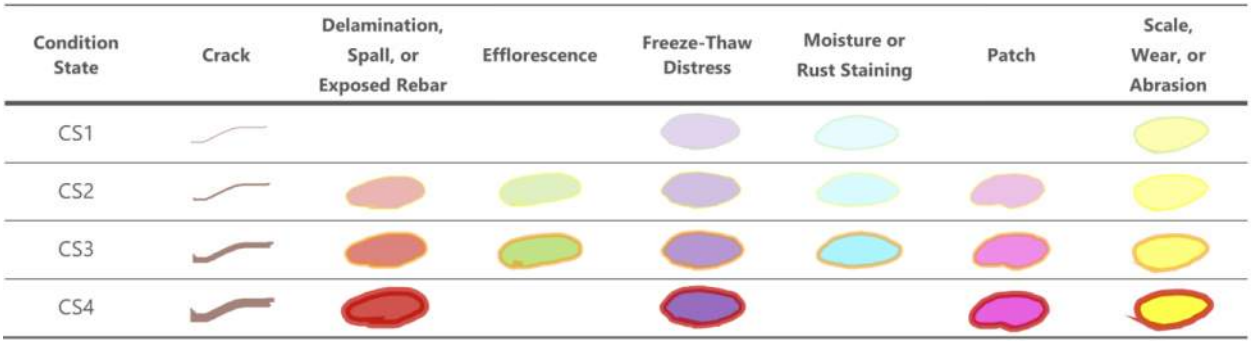


Figure 4.1. Customized legend for condition states shown on field sheets.

Freeze-Thaw Condition States



Figure 4.2. **Freeze-Thaw Condition State 1.**
Map cracking and moisture staining only.



Figure 4.3. **Freeze-Thaw Condition State 2.**
Surface scaling without substantial material loss.



Figure 4.4. **Freeze-Thaw Condition State 3.**
Up to 2 inches (by depth) of apparent concrete deterioration.



Figure 4.5. **Freeze-Thaw Condition State 4.**
Concrete disintegration and erosion extending greater than 2 inches in depth from original section, or exposed bars.

A complete copy of the field inspection notes is provided in APPENDIX B. Data are also saved in the Plannotate software for future reference and review. The Phase 1 inspection observations are summarized in the following sections by element.

4.2. General Observations by Element

4.2.1. Bridge Deck and Railings

- Deck Underside and Fascia
  - Variable-sized areas of delaminated or spalled concrete were present throughout the underside of the deck (Figure 4.6). Isolated areas with narrow longitudinal cracks, moisture/rust staining, and efflorescence were observed. A large concrete repair, likely full depth of the deck, was present between FB-7 and FB-8 (Figure 4.7), adjacent to a large area of delaminated original concrete.
  - The deck fascia, on both the upstream and downstream edges of the bridge, was heavily deteriorated, with large areas of dense cracking, heavy efflorescence, and apparent freeze-thaw distress as evident by surface disintegration/erosion (Figure 4.8 and Figure 4.9).
  - The concrete around original, now abandoned drain openings was generally spalled, and exposed reinforcing steel with significant section loss was typically present (Figure 4.10).
- Deck Topside
  - The asphalt overlay on the deck topside was in fair condition. Visible distress in the overlay included transverse cracks aligned with deck joints over FB-3 and FB-6 (Figure 4.11) and an approximately 0.030-inch-wide crack between the north plug joint and bridge overlay. Isolated areas of delaminated asphalt overlay were identified through chain dragging and hammer sounding (Figure 4.12).
  - The sidewalk topside exhibited widespread delaminated concrete with areas of sound and unsound previous repairs and areas of exposed reinforcing steel (Figure 4.12).
- Railings
  - Wide cracking was present at most railing elements, with many areas of spalled concrete and exposed rebar; some balusters were nearly gone (Figure 4.13 and Figure 4.14).
  - Some of the copper castings at the railing slip joints were cracked or fractured. Longitudinal, transverse, and vertical offsets were observed up to approximately 2 inches, 3 inches, and 1 inch, respectively (Figure 4.15 and Figure 4.16).



Figure 4.6. Example of spalled concrete on deck underside.



Figure 4.7. Large previous repair, likely full-depth of the deck, as viewed from the deck underside between FB-7 and FB-8.



Figure 4.8. Upstream deck fascia general condition.



Figure 4.9. Downstream deck fascia general condition.



Figure 4.10. Example of severe deterioration at now-abandoned drain location.



Figure 4.11. Crack in asphalt overlay located over joint in concrete deck below.

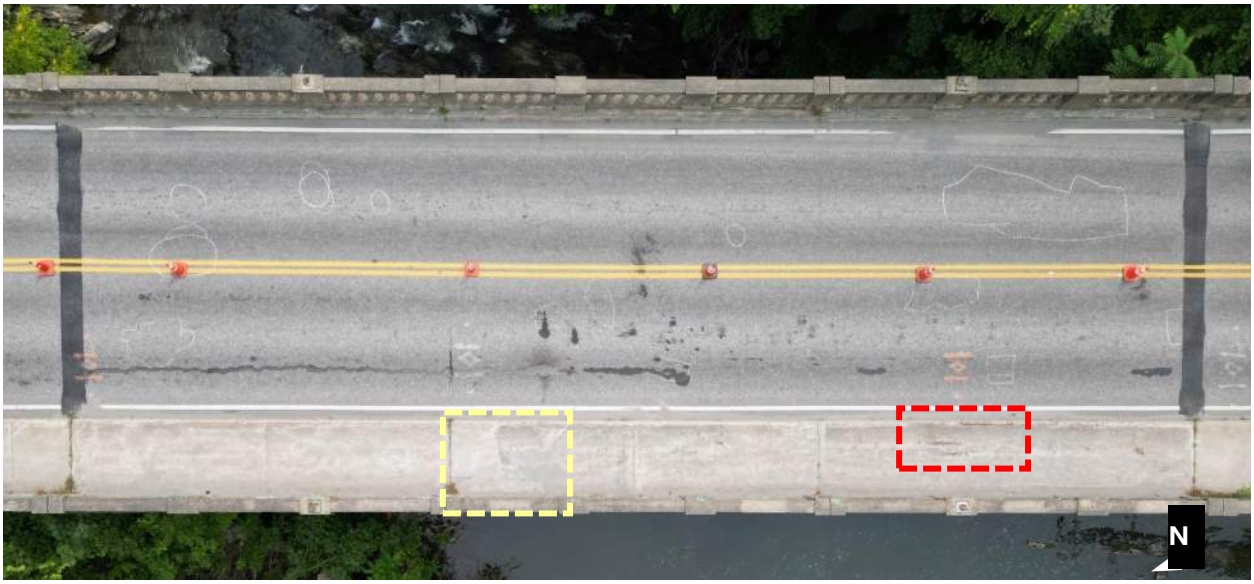


Figure 4.12. Overall view of deck topside showing areas of delaminated overlay, unsound sidewalk repairs (yellow box), and exposed sidewalk reinforcement (red box).



Figure 4.13. Example of railing conditions.



Figure 4.14. Example of severe deterioration in railings, with exposed reinforcement at top rail and baluster and one baluster nearly gone.



Figure 4.15. Longitudinal and vertical offsets at slip joint.



Figure 4.16. Transverse offset and fractured copper casting at slip joint in railing.

4.2.2. Floor Beams and Columns

- The conditions of the center portions of the floor beams (between the arch ribs) varied:
  - FB-3 and FB-6, located beneath original deck joints, were very heavily deteriorated (Figure 4.17 and Figure 4.18). The undersides of these beams were spalled full-length, exposing reinforcing steel with significant section loss. Most of the horizontal legs of the beam stirrups were fractured or missing, and the longitudinal reinforcement was exposed with heavy corrosion. Field measurements showed cross section loss to the longitudinal bars of up to 33 percent. Horizontal cracks with heavy efflorescence were typically present along the top of the floor beams approximately 6 inches below the deck. Areas of delaminated concrete on the vertical faces were present along this crack as well as along the underside spalling.
  - At the eight floor beams located away from the deck joints, minimal deterioration was observed (Figure 4.19). Observed conditions generally consisted of isolated areas of delaminated or spalled concrete, usually at locations of shallow cover or original poor consolidation (Figure 4.20) and narrow cracks (i.e., generally less than 0.020 inches wide).
- In general, the ornate cantilevered floor beam ends (also referred to as overhangs) were significantly distressed at both the upstream and downstream sides of the bridge.
  - The most severe distress was observed at FB-3 and FB-6 and consisted of apparent freeze-thaw deterioration. Surface erosion was up to 5 inches deep, and soft concrete paste extended approximately 6 inches deeper (for a total of 11 inches of affected concrete depth), based on limited hammer drilling. The overhangs often exhibited heavy efflorescence, wide cracks (greater than 1/4 inch), delaminated and spalled concrete, and exposed and corroded steel reinforcing steel (Figure 4.21 and Figure 4.22).
  - The ends of the other six floor beams generally exhibited similar, though slightly less severe distress conditions compared to the floor beam ends at FB-3 and FB-6 (Figure 4.23 and Figure 4.24). Spalling and surface erosion were less frequent, but widespread cracking, efflorescence, and delaminated concrete were typical. Several floor beam ends were cracked down the center, with crack widths up to approximately 1/4 inch (e.g., FB-5, Figure 4.25).
  - Minimal deterioration was observed at the upstream floor beam ends of FB-1, FB-2, and FB-8 (Figure 4.26).
- At the columns, delaminated concrete and spalls were typically present at the bottom corners. Longitudinal reinforcing bars were exposed at some corner spalls, and one bar below the upstream end of FB-2 had significant section loss (Figure 4.27). Other isolated areas of delaminated or spalled concrete were present away from the bottom corners, some of which were related to near-surface transverse reinforcing steel or original poor consolidation (Figure 4.28).
- No cracking that appeared to be related to structural distress (e.g., shear, flexure) or movement was observed within the floor beams or columns.



Figure 4.17. Severe deterioration at center of FB-3.



Figure 4.18. Severe deterioration at center of FB-6.



Figure 4.19. Typical floor beam away from deck joint.



Figure 4.20. Isolated concrete delamination at floor beam away from deck joint.



Figure 4.21. Floor beam overhang condition below deck joint (upstream FB-3).



Figure 4.22. Floor beam overhang condition below deck joint (upstream FB-6).



Figure 4.23. Floor beam overhang condition away from deck joint (downstream FB-7).



Figure 4.24. Floor beam overhang condition away from deck joint (downstream FB-2).



Figure 4.25. Floor beam overhang condition away from deck joint (downstream FB-5).



Figure 4.26. Floor beam overhang condition away from deck joint and with less deterioration (upstream FB-2).



Figure 4.27. Example of concrete spalling often present at bottom corner of column (column below FB-2).



Figure 4.28. Concrete delamination and spalling at bottom corner of column and at reinforcement with shallow cover.

4.2.3. Arch Ribs and Tie Beams

- Isolated areas of spalled or delaminated concrete were present throughout the arch ribs.
  - The areas of delaminations and spalls were typically small (i.e., less than 3 square feet), and were generally related to original poor consolidation or reinforcing steel with shallow cover (Figure 4.29).
  - Larger areas (i.e., up to approximately 15 square feet) of delamination were present on the arch undersides, typically near apparent original construction joints or lift lines, but sometimes corresponding with larger areas of poor concrete consolidation (Figure 4.30).
  - Concrete surface repairs were present on the arch ribs at multiple locations (Figure 4.31 and Figure 4.32). The repairs were typically 5 square feet or smaller, appeared to have been performed using hand-patching techniques, and most had failed (i.e., exhibited cracking, delamination, and/or spalling).
- Cracking was present in the arch ribs near their spring lines.
  - A relatively wide crack was present at the downstream rib underside, extending from the north abutment to approximately FB-8 (Figure 4.33). The crack was approximately 0.075 inch wide near the abutment and narrowed as it approached FB-8. Concrete along the crack was generally sound. This type of crack (singular, relatively wide, longitudinally-oriented crack) was the only one of this kind observed in the arch ribs. See discussion under Chapter 6.
  - Closely spaced approximately 0.025- to 0.030-inch-wide cracks were present on the interior and exterior faces of the downstream arch rib near the south abutment (Figure 4.34). Concrete along the cracks was generally sound.
  - Map cracking was present on the undersides of both arches at the south end of the bridge (Figure 4.35). The cracks were typically less than 0.010 inches wide, but some were up to 0.020 inches wide. Concrete along the cracks was generally sound.
  - No apparent structural cracking or other significant structural distress from applied loads or mechanical actions were observed in the arch ribs.
- An approximately 15-square-foot area of spall with exposed reinforcing steel was present on the upstream arch underside below FB-8 (Figure 4.36 and Figure 4.37). The area had evidence of previous repair material. The current spall was approximately 6 inches deep, with 15 longitudinal bars and multiple rows of stirrups exposed. The longitudinal bars were corroded, with slight section loss, and the bottom legs of the stirrups were fractured or missing within the spall. It appeared that the original concrete was poorly consolidated around the longitudinal reinforcement. The longitudinal bars are congested within the spall, apparently due to supplemental reinforcement and a lap splice that were added to address arch alignment during original construction.
- Moisture staining, efflorescence and surface scaling were present on the vertical arch faces and topsides below FB-3 and FB-6 (i.e., deck joint locations; Figure 4.38 and Figure 4.39). Deterioration at the arch topsides was otherwise minimal.
- At the tie beams, deterioration was limited and generally consisted of small (i.e., less than 2 square feet), isolated areas of delaminated or spalled concrete, typically related to poor consolidation or reinforcement with shallow cover (Figure 4.40).



Figure 4.29. Example of concrete spall related to near-surface reinforcement (i.e., shallow cover).

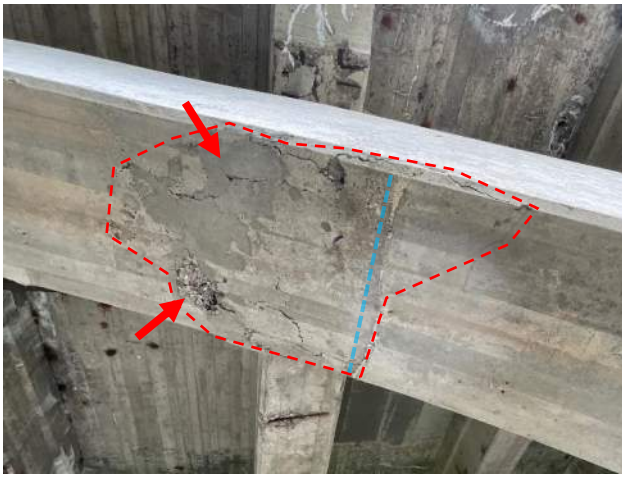


Figure 4.30. Example of delaminated concrete (dashed outline), failed repair (upper arrow), and poor consolidation (lower arrow) on arch rib underside near apparent cold joint (blue line).



Figure 4.31. Example of concrete repairs on arch rib (arrows).



Figure 4.32. Example of failed repair on arch rib.



Figure 4.33. Atypical, longitudinally-oriented, wide crack on downstream arch underside at north abutment.



Figure 4.34. Crack on downstream arch vertical face at south abutment (arrow).



Figure 4.35. Map cracking on arch underside near south abutment.



Figure 4.36. Large, deep spall with exposed reinforcement on upstream arch rib near north abutment.



Figure 4.37. Close-up of exposed reinforcement on upstream arch rib near north abutment.



Figure 4.38. Example of heavy efflorescence and surface scaling on vertical face of arch rib below deck joint.



Figure 4.39. Example of efflorescence on arch topside below deck joint.



Figure 4.40. Example of tie beam spalls at locations of reinforcement with shallow cover (arrows).

4.2.4. Abutments and Wing Walls

- Abutment Walls
  - Small, isolated areas of cracking, efflorescence, and delaminated concrete were present at both abutments, particularly at the upstream side (Figure 4.41 through Figure 4.44). The upstream and downstream cornices were generally deteriorated, with heavy efflorescence, surface erosion, cracking, and delaminated concrete (Figure 4.45).
  - Single, wide cracks (up to 1/4 inch wide) were present at the top of both abutment walls, aligned with vertical construction joints in the wall below (Figure 4.46).
  - Areas of light surface scaling were present on both abutment walls.
  - Moisture staining was present originating from water main and utility openings at both abutments.
- Wing Walls
  - At the upstream south wing wall, deterioration was primarily concentrated below a drain outlet located near the top of the wall (Figure 4.47). Horizontal cracking up to approximately 0.070 inches wide was present between pilasters located approximately 10 feet to either side of the drain outlet. The cracks extended through previous repair material surrounding the outlet (Figure 4.48). Approximately 10 square feet of delaminated original concrete was present around the repair material. Moisture staining and light surface scaling were present directly below the drain.
  - The upstream north, downstream north, and downstream south wing walls were largely inaccessible and/or obscured by soil or vegetation. However, the tops of all four wing walls were visible and, and deterioration, including surface erosion, cracking, and heavy efflorescence, was present below the deck joints (Figure 4.49). Cracking and efflorescence extended horizontally along the cornice, decreasing in severity away from the joint.
- Footings at Abutments and Wing Walls
  - Severe freeze-thaw deterioration and surface erosion was present at the north downstream abutment/wing wall footing corner (Figure 4.50 and Figure 4.51) and in the footing below the drain outlet at the south upstream wing wall (Figure 4.47). Less severe freeze-thaw deterioration was also evident in other areas of the footings.
  - Undermining was observed at an approximately 3-foot length of north abutment footing below the downstream arch rib, at the “plunge pool” at the base of a small waterfall along the face of the footing (Figure 4.50).
  - Cracking with efflorescence and moisture staining was generally present on the vertical faces of the footings (Figure 4.51). Some horizontally-oriented separations (cracks) were present in the footing faces, apparently due to original poor consolidation along cold joints or lift lines; concrete along some of the cold joints or lift lines had deteriorated, widening the separations (Figure 4.52).



Figure 4.41. Overall view of north abutment.



Figure 4.42. Overall view of south abutment.



Figure 4.43. Conditions at upstream side of north abutment.



Figure 4.44. Conditions at upstream side of south abutment.



Figure 4.45. Cornice condition at downstream side of north abutment.



Figure 4.46. Crack at top of north abutment wall originating at vertical construction joint below.

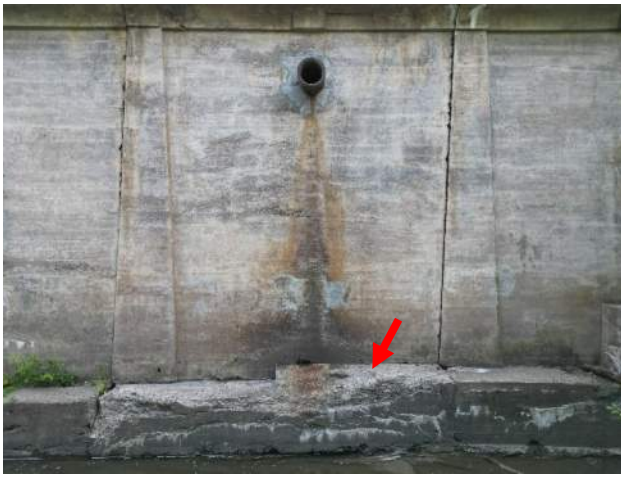


Figure 4.47. Conditions below drain at upstream south wing wall, including footing freeze-thaw deterioration and surface erosion (arrow).



Figure 4.48. Cracking near drain at upstream south wing wall (arrows).



Figure 4.49. Erosion, cracking, and efflorescence at downstream north wing wall.



Figure 4.50. Footing freeze-thaw deterioration and erosion at downstream corner of north abutment footing. Undermining of the footing was observed at the base of the waterfall (arrow).



Figure 4.51. North abutment footing freeze-thaw deterioration and surface erosion (right) and dense cracking/efflorescence (left).



Figure 4.52. Horizontal separations (cracks) with concrete deterioration along original construction joints or lift lines with poor consolidation (arrows).

4.2.5. Water Main and Utilities

- The abandoned water main pipe and its hanger attachments to the deck underside were heavily corroded. The pipe was fractured at multiple locations (Figure 4.53) and many of the hangers had corroded through such that they were no longer supporting the pipe (Figure 4.54).



Figure 4.53. Water main pipe fracture.



Figure 4.54. Corroded water main hanger.

4.3. Conditions That Could Affect Structural Analysis and Load Rating

Parallel with WJE's field inspections and materials testing, structural analysis and load rating of the bridge was performed by HNTB. To facilitate the analysis and load rating, WJE provided HNTB with advance results from the materials testing and condition assessment. This included detailed condition information on the floor beams below the deck joints, including section loss measurements for the reinforcing steel in the floor beams. Concrete compressive strength test results and reinforcing steel mechanical strength test results were also provided. See Chapter 5 for detailed test results.

4.4. Distress Quantity Ratios by Element

4.4.1. Definition of Distress Quantity Ratio

A useful way to analyze the overall bridge inspection data is to calculate total quantities of observed distress that would warrant a repair in a rehabilitation (referred to as "distress quantities" in this report). To that end, the condition state data can be conceptually split into two groups. The first group includes primary measures of concrete deterioration and distress such as delaminations, spalls, freeze-thaw deterioration, and previously repaired (patched) areas. Distress represented by these types of conditions and associated condition states are usually repaired in a major preservation or rehabilitation program. The second group of condition states can be considered leading indicators of future distress. These indicators include moisture staining, rust staining, and efflorescence. These types of distress are typically not repaired but should be considered in determining the extent or need for preventive maintenance. Cracking can fall into either grouping; for this analysis cracking is considered to be a leading indicator of future distress and quantified in the second group.

To allow interpretation of the structure's current condition, the condition state data from the overall inspection was aggregated into a single "distress quantity ratio" (DQR) for each bridge element. The DQR was calculated by adding the mapped areas of all the below-listed primary measures of concrete deterioration for a given element and dividing by the total exposed surface area of that element. The following conditions were included in the calculation of DQR:

- Delaminated and spalled areas: CS2 (delaminated), CS3 (moderate-depth spalls), and CS4 (deep spalls)
- Pre-existing patches and repairs: CS2 (currently sound patches) and CS3 (failed or debonded patches)[9]
- Freeze-thaw distress: CS3 (moderate, with up to 2 inches of deterioration depth) and CS4 (severe, more than 2 inches of deterioration depth).

As such, the DQR for any given element is essentially the percentage of mapped surface area on that element that would likely need to be repaired in a rehabilitation, and the value will fall between 0 and 100 percent. The DQR can also be viewed as a relative rating between elements, where 0 is the best and 100 is the worst condition.

The leading indicators of distress (cracking, moisture staining, etc.) were not included in the DQRs. This is because the presence of these types of distress alone does not presently require a concrete surface repair. However, their incidence was later considered when assessing element durability and the need for future repairs or other preventative measures.

4.4.2. Summary of Distress Quantity Ratios by Element Category

WJE used the inspection notes to calculate the DQR for each element category in the bridge[10], which are presented in detail in Table 4.2 through Table 4.7. The DQR values are color coded as follows:

- Green – DQR less than 10%
- Yellow – DQR 10 to 19%
- Orange – DQR 20 to 29%
- Red – DQR 30% or higher

The highlights of the findings are briefly summarized below and are discussed in Chapter 6 for further interpretation. Note that the "distress quantity ratio" as defined above is sometimes simply referred to as "distress quantity" as a percentage in this report.

[9] Note that for the purpose of this assessment, both currently sound and unsound previous repairs, as documented in the bridge inspection, were included in the distress quantities. Although this is appropriate for an initial assessment, it should be understood that not all currently sound repairs are visually detectable. (For the Windsor Bridge we believe this is a small amount since the bridge is not coated and has not been repaired extensively; at the 3rd Avenue Bridge, a large number of previous repairs were not visually detectable.) Also, whether or not currently sound repairs need to be replaced in a rehabilitation project depends on the target service life extension. For example, currently sound repairs could likely remain in a service life extension of 10-20 years; however, repairs will likely fail before 50 years, and so they would need to be replaced in a rehabilitation meant to extend the service life by 50 years. The decision of how to treat currently sound repairs needs to be further evaluated in the design phase of the project.

[10] DQR were not calculated for the railings due to the obviously severe and widespread distress in this element of the bridge.

- Deck
  - A high DQR in Bay 6 of the deck underside was due to relatively large areas of spalled and delaminated concrete.
  - A very high DQR in Bay 8 of the deck underside was due to delaminated concrete comprising almost half of the bay at the downstream side, as well as an approximately 50 square foot apparent previous full-depth concrete repair.
  - The DQR for the deck topside within the asphalt overlay area was the amount of apparently delaminated/debonded asphalt overlay as determined by mechanical sounding and is not directly representative of the structural concrete deck condition below the asphalt. The DQR identified for the sidewalk topside is accurate since the concrete is exposed.
- Floor Beams and Columns
  - All floor beam DQRs were below 8 percent except for FB-3 and FB-6. The floor beams located beneath deck joints had much higher DQRs due to more advanced deterioration associated with the more severe exposure.
  - The column DQRs appear high due to the relatively small surface area of the columns; spalling/delamination that was present at the lower corners of the columns represented a significant proportion of the small column total area.
- Arch Ribs and Tie Beams
  - The DQRs for the arch ribs averaged 10 percent, with distress slightly more concentrated at the north end of both ribs. The outer thirds of the arches had higher DQR's than the middle third, except the south third of the downstream arch rib had a lower DQR.
  - The DQRs for the tie beams were all 5 percent or lower.
- Abutment and Wing Walls
  - The high DQR at the north abutment footing was due to a large area of erosion and freeze-thaw distress at the downstream side. The exposed portion of the south abutment footing had a lower DQR, but most of the south abutment footing was concealed by rock/soil.
  - DQRs were not calculated for the upstream north, downstream north, or downstream south wing walls because a significant portion of their surface areas was inaccessible and/or concealed by vegetation/soil. The DQR for the upstream south wing wall is based on the portion that was accessible for inspection.

Table 4.2. Distress Quantity Ratios - **Deck**

Element	Location	Distress Quantity (square feet)	Surface Area (square feet)	Distress Quantity Ratio
Deck	**Underside – Total**	**369**	**2,358**	**16%**
Deck	Underside – Bay 1 (S. Abut. to FB-1)	6	262	2%
Deck	Underside – Bay 2 (FB-1 to FB-2)	13	262	5%
Deck	Underside – Bay 3 (FB-2 to FB-3)	13	262	5%
Deck	Underside – Bay 4 (FB-3 to FB-4)	20	262	8%
Deck	Underside – Bay 5 (FB-4 to FB-5)	15	262	6%
Deck	Underside – Bay 6 (FB-5 to FB-6)	83	262	32%
Deck	Underside – Bay 7 (FB-6 to FB-7)	32	262	12%
Deck	Underside – Bay 8 (FB-7 to FB-8)	169	262	64%
Deck	Underside – Bay 9 (FB-8 to N. Abut.)	18	262	7%
Deck	**Topside – Sidewalk**	**110**	**540**	**20%**
Deck	**Topside – Asphalt Overlay**	**150**	**2,160**	**7%**

Table 4.3. Distress Quantity Ratios – **Floor Beams**

Element	Location	Distress Quantity (square feet)	Surface Area (square feet)	Distress Quantity Ratio
Floor Beam	**Total**	**226**	**1,956**	**11%**
Floor Beam	FB-1	13	256	5%
Floor Beam	FB-2	7	256	3%
Floor Beam	FB-3	69	249	27%
Floor Beam	FB-4	12	223	5%
Floor Beam	FB-5	18	220	8%
Floor Beam	FB-6	84	240	35%
Floor Beam	FB-7	12	256	5%
Floor Beam	FB-8	11	256	4%

Table 4.4. Distress Quantity Ratios - **Columns**

Element	Location	Distress Quantity (square feet)	Surface Area (square feet)	Distress Quantity Ratio
Column	**Total**	**49**	**260**	**19%**
Column	Downstream - FB-1	4	51	8%
Column	Upstream - FB-1	6	51	13%
Column	Downstream - FB-2	2	18	11%
Column	Upstream - FB-2	6	18	34%
Column	Downstream - FB-7	5	14	35%
Column	Upstream - FB-7	0	14	0%
Column	Downstream - FB-8	15	47	32%
Column	Upstream - FB-8	11	47	24%

Table 4.5. Distress Quantity Ratios – **Arch Ribs**

Element	Location	Distress Quantity (square feet)	Surface Area (square feet)	Distress Quantity Ratio
Arch Rib	**Total**	**212**	**2,222**	**10%**
Arch Rib	**Downstream - Total**	**92**	**1,111**	**8%**
Arch Rib	**Upstream - Total**	**120**	**1,111**	**11%**
Arch Rib	Downstream – S. Abut to FB-3 (south third)	12	407	3%
Arch Rib	Downstream – FB-3 to FB-6 (middle third)	30	297	10%
Arch Rib	Downstream – FB-6 to N. Abut. (north third)	50	407	12%
Arch Rib	Upstream – S. Abut to FB-3 (south third)	44	407	11%
Arch Rib	Upstream – FB-3 to FB-6 (middle third)	11	297	4%
Arch Rib	Upstream – FB-6 to N. Abut. (north third)	65	407	16%

Table 4.6. Distress Quantity Ratios – **Tie Beams**

Element	Location	Distress Quantity (square feet)	Surface Area (square feet)	Distress Quantity Ratio
Tie Beam	**Total**	**22**	**516**	**4%**
Tie Beam	FB-1	4	129	3%
Tie Beam	FB-2	6	129	5%
Tie Beam	FB-7	6	129	5%
Tie Beam	FB-8	6	129	4%

Table 4.7. Distress Quantity Ratios – **Abutments and Wing Walls**

Element	Location	Distress Quantity (square feet)	Surface Area (square feet)	Distress Quantity Ratio
Abutment	**Total**	**187**	**1,714**	**11%**
Abutment	**South - Total**	**57**	**868**	**7%**
Abutment	**North - Total**	**130**	**846**	**15%**
Abutment	South - Wall	56	718	8%
Abutment	South - Footing	1	150	1%
Abutment	North - Wall	86	696	12%
Abutment	North - Footing	44	150	29%
Wing Wall	**Upstream South – Total**	**106**	**1675**	**6%**
Wing Wall	Upstream South - Wall	51	1400	4%
Wing Wall	Upstream South - Footing	55	275	20%

5. PHASE 2 - FOLLOW-UP STUDY AREA TESTING AND MATERIAL SAMPLING RESULTS

Based on the findings from Phase 1, select areas of the structure were selected by WJE for the Phase 2 follow-up testing and material sampling. The results of the field and laboratory testing performed as part of the follow-up study area testing and material sampling are summarized in this chapter, and in APPENDIX C. The goal of the follow-up testing was to characterize the distress observed during the overall element level inspections. Locations were selected to cover the range of material and exposure characteristics, with the goal being to identify the deterioration mechanisms present in each element category. The results of the follow-up testing form the basis for the qualitative interpretations of durability potential (i.e., anticipated remaining service life) for each element category as presented in Chapters 6 and 7.

In total, 17 areas received non-destructive testing, 18 core samples and 3 powder samples were extracted, and 5 steel samples were removed and tested. The locations of the study areas (SA), cores and powder samples are shown in Figure 5.1 and Figure 5.2. The study areas were selected to provide a spatial distribution across the bridge and to represent the range of different structural elements and exposures. Additional information regarding each study area and material sample are provided in the following sections.

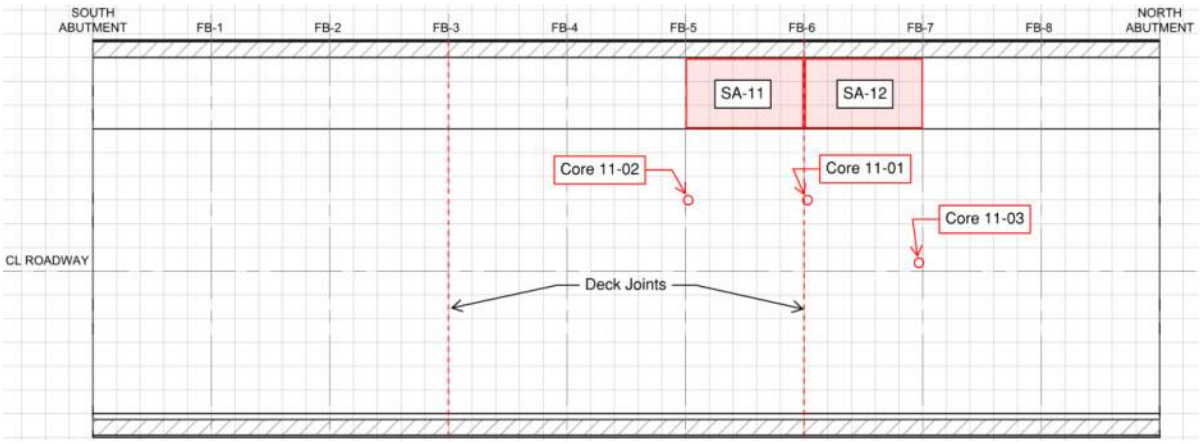


Figure 5.1. Deck plan showing locations of deck topside study areas and deck concrete core samples.

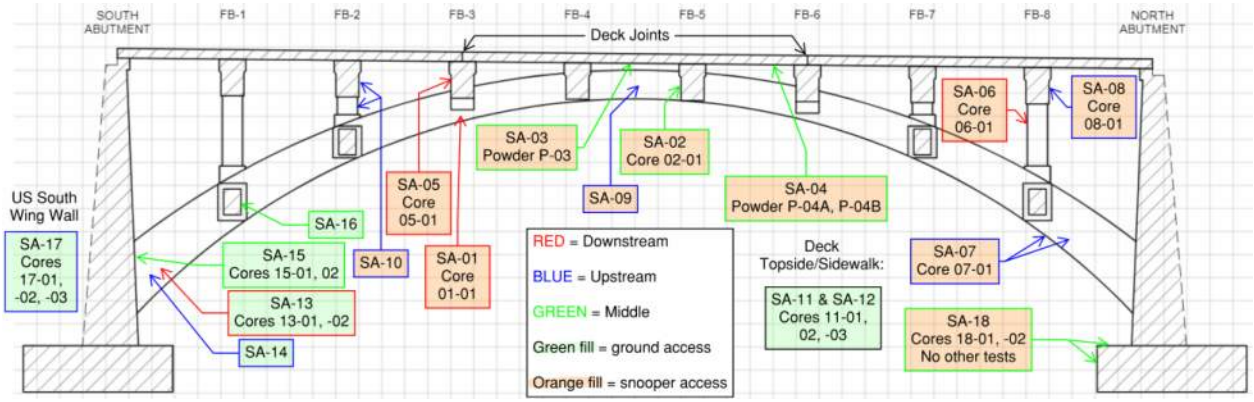


Figure 5.2. Elevation view showing general locations of all study areas and concrete samples, except for deck topside.

5.1. Field Testing and Material Sampling

Field testing results for study areas SA-01 through SA-17[11] are summarized in Table 5.1, organized according to element type. The test results include half-cell potential measurements, concrete resistivity measurements (using both the Resipod and iCOR), corrosion rate measurements, reinforcing steel cover measurements, and field carbonation depth testing. The findings of the field testing are summarized in more detail in the sections that follow.

[11] No field testing was performed at SA-18; only concrete core samples were extracted from this location.

Table 5.1. Summary of Results of Field Testing at Study Areas

Study Area ID	Element	Location	Description	Half-Cell Potential		Resistivity (Resipod)		Resistivity (iCOR) (kΩ-cm)	Corr. Rate (iCOR) (μA/cm²)	Reinf. Cover		Carb. Depth (in)
				Avg. (mV)	Min. (mV)	Avg. (kΩ-cm)	Min. (kΩ-cm)			Min. (in)	Max. (in)	
SA-03	Deck	Underside	Bay 5 (FB-4 to FB-5)	-24	-84	559	432	-	-	1-1/2	1-3/4	~0
SA-04	Deck	Underside	Bay 6 (FB-5 to FB-6)	-359	-547	53	30	-	-	1-1/2	1-7/8	1/8
SA-11	Deck	Topside	Sidewalk, Bay 6 (FB-5 to FB-6)	-431	-580	73	22	-	-	2	2-3/4	-
SA-12	Deck	Topside	Sidewalk, Bay 7 (FB-6 to FB-7)	-359	-524	90	59	-	-	2	2-3/4	-
SA-02	Floor Beam	FB-5	South face, downstream	-93	-356	226	50	319	0.2	1-1/8	2-7/8	1/8
SA-05	Floor Beam	FB-3	South face, downstream	-314	-497	42	12	-	-	2-1/8	2-1/8	1
SA-08	Floor Beam	FB-8	North face, upstream	-103	-160	284	103	-	-	1-5/8	1-3/4	~0
SA-10	Floor Beam	FB-2	North face, upstream (includes column)	-81	-131	224	86	-	-	1	3	1/8
SA-06	Column	FB-8	Downstream, south and exterior faces	-118	-267	378	271	-	-	1-3/8	2	~0
SA-01	Arch	Downstream	Exterior face, FB-3	-355	-445	43	32	32	0.3	0	3-1/4	3/16
SA-07	Arch	Upstream	Interior face, Bay 9 (FB-8 to N. Abut.)	-220	-299	150	126	118	0.1	2-1/2	3-1/8	1/4
SA-09	Arch	Upstream	Exterior face, Bay 5 (FB-4 to FB-5)	-110	-167	112	87	147	0.1	1-3/8	2-3/4	1/8
SA-13	Arch	Downstream	Interior face, Bay 1 (S. Abut. to FB-1)	-216	-378	73	67	-	-	3-7/8	4	~0
SA-14	Arch	Upstream	Interior face, Bay 1 (S. Abut. to FB-1)	-254	-393	83	59	142	0.2	2-3/4	2-7/8	~0
SA-16	Tie Beam	FB-1	North face	-40	-79	175	166	-	-	3/4	3-1/8	1/8
SA-15	Abutment	South	Wall	-300	-399	65	31	-	-	4	5	1/4
SA-17	Wing Wall	US South	Wall (below drain)	-390	-492	74	44	-	-	3-1/4	3-1/4	1/8

5.1.1. Half-Cell Potential

Table 5.1 and the figures in this section summarize the half-cell potential (HCP) data. Note, HCP data were temperature-corrected to 75°F based on recorded ambient temperature at time of readings (ambient temperatures ranged from 39 to 49°F). Readings are color coded in the table to represent probabilities of corrosion activity based on thresholds defined in Chapter 3, consistent with ASTM C876:

- Green – Low probability of active corrosion (>-200 mV)
- Yellow – Moderate/low probability of active corrosion (-200 to -274 mV)
- Orange – Moderate/high probability of active corrosion (-275 to -349 mV)
- Red – High probability of active corrosion (≤-350 mV)

The following summarizes the findings of the HCP testing by element type:

- Deck
  - HCP readings on the deck underside away from deck joints (i.e., SA-03 between FB-4 and FB-5) were all more positive than -100 mV (i.e., low probability of corrosion). No visible distress was present within this study area.
  - HCP readings on the deck underside near a deck joint (i.e., SA-04 between FB-5 and FB-6) were all more negative than -350 mV (i.e., high probability of corrosion). This study area was located adjacent to and partially within an area of delaminated concrete.
  - HCP readings on the topside of the sidewalk (i.e., SA-11 and SA-12) were all more negative than -350 mV (i.e., high probability of corrosion). Approximately half of the readings from SA-11 were taken within previous repairs that were in generally poor condition.
- Floor Beams and Columns
  - A comparison between HCP readings and visual conditions of floor beams away from and directly below a deck joint is shown in Figure 5.3 through Figure 5.6.
    - Figure 5.3 shows a contour plot of the HCP readings and Figure 5.4 shows the visible condition at SA-02, which was located away from the deck joints at FB-5. In general, HCP readings were more positive than -200 mV (i.e., low probability of corrosion). HCP readings trended more negative near the extreme downstream end of the overhang, where visible distress included cracking, efflorescence, and delaminated concrete on the downstream face.
    - Figure 5.5 shows a contour plot of the HCP readings and Figure 5.6 shows the visible condition at SA-05, which was located directly below a deck joint at FB-3. The average HCP reading was -314 mV and the minimum was -497 mV, indicative of a high probability of corrosion. Visual indicators of ongoing corrosion were visible across the entire surface of the floor beam, including spalling and reinforcement section loss along the bottom of the beam between the arches, cracking, and heavy efflorescence.
  - HCP readings at the upstream overhangs away from deck joints (i.e., SA-08 and SA-20 on FB-8 and FB-2, respectively) were all more positive than -200 mV (i.e., low probability of corrosion). Minimal visual distress was present at these locations.
  - At one column (i.e., SA-06 at the downstream column under FB-8), the average HCP reading was -118 mV and the minimum was -267 mV (i.e., moderate probability of corrosion). A short column

was tested as part of SA-10 at FB-2 and all HCP readings were more positive than -200 mV. Spalling and delaminated concrete were present at the bases of both columns.

- Arch Ribs
  - A comparison between HCP readings and visual conditions of the arch ribs away from and directly below a deck joint is shown in Figure 5.7 through Figure 5.10.
    - Figure 5.7 shows a contour plot of the HCP readings and Figure 5.9 shows the visual condition at SA-09, which was located away from the deck joints between FB-4 and FB-5. HCP readings were all more positive than -200 mV (i.e., low probability of corrosion). Other than one small, localized spall associated with low reinforcement cover, there was no visible distress.
    - Figure 5.8 shows a contour plot of the HCP readings and Figure 5.10 shows the visual condition at SA-01, which was located directly below a deck joint at FB-3. HCP readings were all more negative than -350 mV (i.e., high probability of corrosion). Distress included: one spall due to low reinforcement cover; an area of delaminated concrete; and staining and efflorescence associated with exposure from the deck joint above.
  - At the interior face of the upstream arch below FB-8 (SA-07), the average HCP reading was -220 mV and the minimum was -267 mV, which is indicative of a moderate probability of corrosion. An area of delaminated concrete along the bottom arch corner, corresponding with a location of exposed longitudinal reinforcement on the arch underside, was present within this study area.
  - Near the south spring lines of both arch ribs (i.e., SA-13 and SA-14 at the interior faces of the downstream and upstream arch ribs, respectively), the average HCP readings were -216 mV and -254 mV, and the minimum readings were -378 and -393 mV. The minimum readings occurred immediately adjacent to the abutment and decreased to values less negative than -200 mV away from the abutment. Saturated soil conditions are known to affect half-cell potential readings, and it is likely that readings in the arch ribs directly adjacent to the abutment were affected by the abutment reinforcement. Similar readings were measured on the abutments and wing walls as discussed below. An approximately 0.025-inch-wide crack was present within SA-13, but distress was otherwise minimal at both study areas.
  - HCP readings at the tie beam below FB-1 (SA-16) were all more positive than -100 mV (i.e., low probability of corrosion). Other than one small spall due to low reinforcement cover, distress was minimal at this study area.
- Abutment/Wing Wall
  - Note that, as discussed in Chapter 3, earth retaining structures are known to have more negative values and therefore the standard interpretation of half-cell potential readings is generally not applicable.
  - At the south abutment wall (SA-15), the average HCP reading was -300 mV and the minimum was -399 mV. Delaminated concrete and moisture staining beneath the utility conduit opening was present within a portion of the study area.
  - Below the drain outlet on the downstream south wing wall (SA-17), the average HCP reading was -390 mV and the minimum was -493 mV. Delaminated concrete and cracking were present along horizontal reinforcement within the study area.

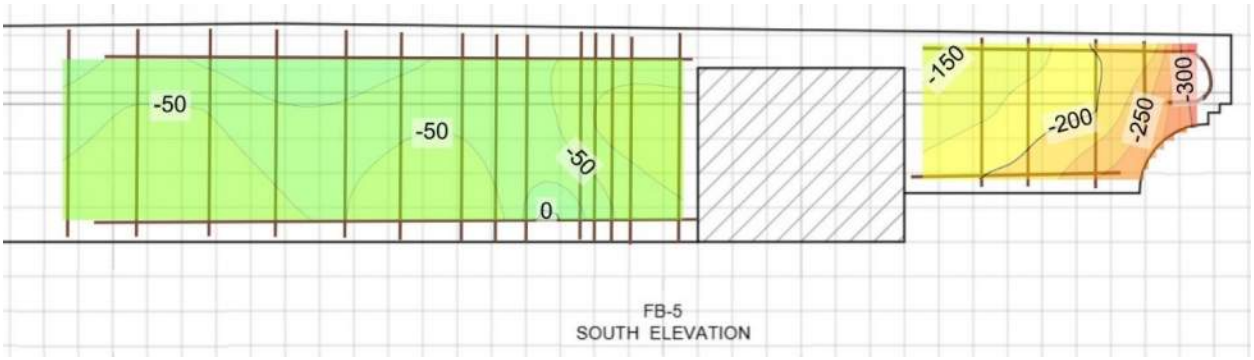


Figure 5.3. HCP contour plot for SA-02 (FB-5; away from joint; average -93 mV, minimum -356 mV).



Figure 5.4. View of SA-02. No visual distress where HCP readings are less negative, cracking and efflorescence visible where HCP readings are more negative at overhang. Blue lines drawn on beam indicate reinforcement identified using GPR and wet spots show where the surface was pre-wetted to take readings.

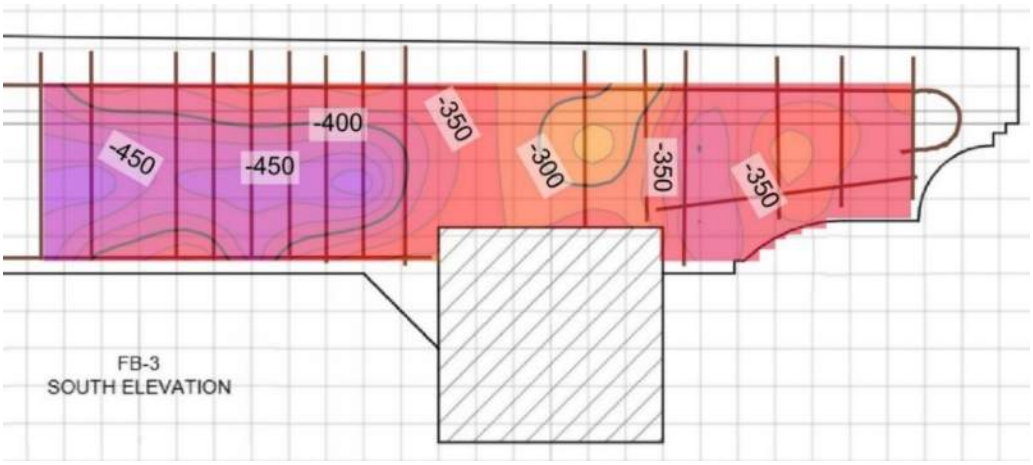


Figure 5.5. HCP contour plot for SA-05 (FB-3; at deck joint; average -314 mV, minimum -497 mV).



Figure 5.6. View of SA-05. Evidence of deterioration is visible throughout entire area.

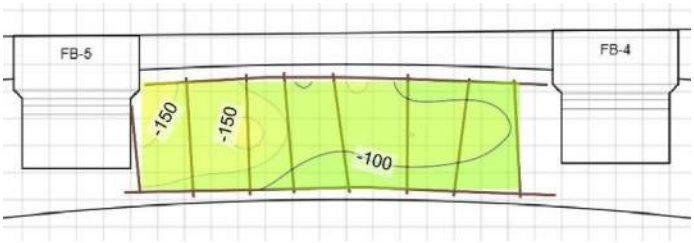


Figure 5.7. HCP contour plot for SA-09 (away from deck joint; average -110 mV, minimum -167 mV).

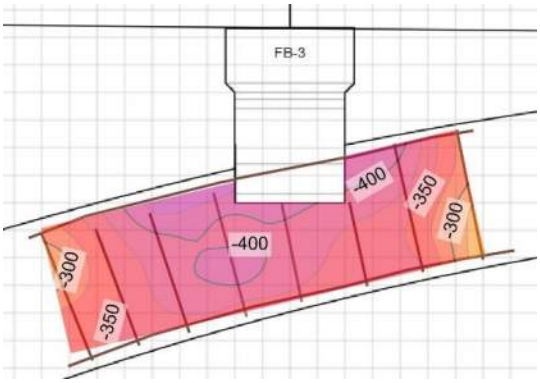


Figure 5.8. HCP contour plot for SA-01 (at deck joint; average -355 mV, minimum -445 mV).



Figure 5.9. View of SA-09. No visual distress except small spall due to near-surface reinforcement. Blue lines drawn on arch indicate reinforcement identified using GPR and wet spots show where the surface was pre-wetted to take readings.



Figure 5.10. View of SA-01. Distress included a spall due to near-surface reinforcement, an area of delaminated concrete (red outline), and efflorescence from the deck joint above. Blue lines drawn on arch indicate reinforcement identified using GPR and wet spots show where the surface was pre-wetted to take readings.

5.1.2. Concrete Resistivity

The concrete resistivity readings reported in Table 5.1 are based on several measurements taken at various points throughout each study area, and therefore the average and minimum values are reported. The values in the table are color coded to represent likely levels of corrosion rate if corrosion has initiated based on the thresholds defined in Chapter 3:

- Green - Negligible (>100 kΩ-cm)
- Yellow –Low (51 to 100 kΩ-cm)
- Orange – Moderate (11 to 50 kΩ-cm)
- Red – High (≤10 kΩ-cm)

The following summarizes the findings related to concrete resistivity measured using the Resipod (iCOR resistivity readings will be discussed in the following section):

- Deck
  - Readings were indicative of negligible corrosion rate on the deck underside away from deck joints (SA-03), and low to moderate corrosion rate on the deck underside near a deck joint (SA-04).
  - Readings were indicative of low to moderate corrosion rate on the deck topside at the sidewalk (SA-11 and SA-12).
  - Resistivity testing cannot be performed through asphalt.
- Floor Beams and Columns
  - Readings were indicative of negligible corrosion rate away from deck joints at SA-02 (FB-5) except for at the downstream overhang where readings were indicative of moderate corrosion rate.
  - Readings were indicative of moderate corrosion rate below the deck joint at SA-05 (FB-3).
  - Readings were indicative of negligible corrosion rates at SA-08 (FB-8), SA-10 (FB-2), and SA-06 (downstream column at FB-8), except at the upstream column within SA-10 where they were indicative of low corrosion rate.
- Arch Ribs
  - Readings were indicative of moderate corrosion rate below the deck joint at SA-01.
  - Readings were indicative of negligible to low corrosion rate at all other arch rib and tie beam study areas (i.e., SA-07, -09, -13, -14, and -16).
- Abutment/Wing Wall
  - Readings were indicative of low to moderate corrosion rate at SA-15 and SA-17.

5.1.3. Corrosion Rate

Concrete resistivity readings reported in Table 5.1 were generally indicative of negligible to low corrosion rates, except in regions where exposure and corrosion activity (as measured through HCP) were identified. Companion resistivity readings taken during corrosion rate testing with the iCOR were typically slightly higher in magnitude, but generally consistent with the trends measured using the Resipod.

Corrosion rate data was obtained using the iCOR at several locations in the arches and floor beams. Overall, these readings indicate a low rate of active corrosion at the time of measurement. This result is expected given that corrosion rate is directly proportional to temperature, which ranged from 39 to 49°F

at the time of measurement. The readings can be used as a baseline for subsequent investigations should long-term monitoring be performed for the structure.

5.1.4. Carbonation and Reinforcement Cover

The following summarizes the findings related to the field measured cover depth over reinforcement and field carbonation depths, as reported in Table 5.1:

- Except where near-surface reinforcement was exposed, carbonation depths did not reach or exceed the measured reinforcement cover at any study area.
- Deck
  - Cover depths at the deck underside ranged from 1-1/2 to 1-7/8 inches, and carbonation depths did not exceed 1/8 inch.
  - Cover depths at the sidewalk ranged from 2 to 2-3/4 inches. No carbonation measurements were taken at the sidewalk topside.
- Floor Beams and Columns
  - Cover depths to the floor beam stirrups varied widely and ranged from 1 to 2-7/8 inches, with an average of approximately 2 inches. Carbonation depths did not exceed 1/8-inch except at SA-05, where up to 1 inch of carbonation was measured at a crack.
  - Cover depth measurements at columns were 1-3/8 inches (stirrup at SA-06), 2 inches (vertical bar at SA-06), and 3 inches (vertical bar at SA-10). Carbonation depths at both columns were negligible.
- Arch Ribs
  - Cover depths to the arch rib stirrups varied widely and ranged from 1-3/8 to 4 inches, with an average of approximately 2-1/2 inches. Near-surface reinforcement (i.e., nearly zero cover depth) was present at SA-01 and SA-09. Carbonation depths did not exceed 1/4 inch.
  - Cover depths to the tie beam stirrups at SA-16 ranged from 3/4 to 1-3/4 inches on the north face and from 2-5/8 to 3-1/8 inches on the south face. Carbonation depth did not exceed 1/8 inch.
- Abutment/Wing Wall
  - Cover depth measurements on the abutment wall at SA-15 were 4 inches to the vertical bar and 5 inches to the horizontal bar. Carbonation depth did not exceed 1/4 inch.
  - Cover depth measurements to two separate vertical bars on the upstream south wing wall at SA-17 were both 3-1/4 inches. Carbonation depth did not exceed 1/8 inch.

5.2. Laboratory Testing

5.2.1. Concrete Samples

A total of 18 concrete core samples were extracted from the structure. Each sample corresponded with a study area and was selected for further laboratory testing, including concrete compressive strength testing, chloride concentration testing, and petrographic analysis (full or partial). A log of all core samples and the tests performed on each are summarized in Table 5.2, and the following sections summarize the results of each test. A comprehensive core log that also provides a summary of macro-characteristics of each core (e.g., cracking, reinforcement) is provided in APPENDIX C.

Table 5.2. Concrete Sample Log and Laboratory Tests Performed

Core ID	Study Area ID	Element	Location	Comp. Strength	Chloride Content	Petro. (Full*)	Petro. (Partial*)
01-01	SA-01	Arch	Downstream, FB-3	X	X	X	-
02-01	SA-02	Floor Beam	FB-5	X	X	-	-
05-01	SA-05	Floor Beam	FB-3	X	X	-	-
06-01	SA-06	Column	FB-8	X	-	-	-
07-01	SA-07	Arch	Upstream, FB-8	X	X	-	-
08-01	SA-08	Floor Beam	FB-8	X	X	X	-
11-01	SA-11	Deck	Topside within roadway, FB-6	-	X	-	-
11-02	SA-11	Deck	Topside within roadway, FB-5	X	X	X	-
11-03	SA-11	Deck	Topside within roadway, FB-7	X	-	-	-
13-01	SA-13	Arch	Downstream, S. Abut.	X	X	X	-
13-02	SA-13	Arch	Downstream, S. Abut.	X	-	-	-
15-01	SA-15	Abutment	S. Abut. Wall	X	-	X	-
15-02	SA-15	Abutment	S. Abut. Wall	X	X	-	-
17-01	SA-17	Wing Wall	Upstream S. Wall	X	X	-	X
17-02	SA-17	Wing Wall	Upstream S. Footing	-	-	-	X
17-03	SA-17	Wing Wall	Upstream S. Footing	-	-	-	X
18-01	SA-18	Abutment	N. Abut Footing	-	-	-	X
18-02	SA-18	Abutment	N. Abut Footing	-	-	-	X

**See Section 5.2.4 for definition of petrographic analysis types.*

5.2.2. Concrete Compressive Strength

A total of 13 concrete cores were tested for compressive strength. Testing was performed in WJE's laboratories in general accordance with ASTM C42 *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*. Test results are summarized in Table 5.3.

Table 5.3. Concrete Compressive Strength Test Results

Core ID	Element	Capped Length (in)	Diameter (in)	L/D	Correction Factor	Compressive Strength (psi)
11-02	Deck	5.4	3.7	1.5	0.96*	10,072
11-03	Deck	6.2	3.7	1.7	0.97*	7,730
02-01	Floor Beam	7.4	3.7	2.0	none	5,360
05-01	Floor Beam	7.4	3.7	2.0	none	4,920
08-01	Floor Beam	5.2	3.7	1.4	0.95*	7,219
06-01	Column	7.4	3.7	2.0	none	5,150
01-01	Arch	7.5	3.7	2.0	none	6,920
07-01	Arch	7.4	3.7	2.0	none	5,450
13-01	Arch	6.7	3.7	1.8	none	7,300
13-02	Arch	7.4	3.7	2.0	none	7,170
15-01	Abutment	6.8	3.7	1.8	none	5,220
15-02	Abutment	7.3	3.7	1.9	none	6,230
17-01	Wing Wall	7.4	3.7	2.0	none	5,620

** Per ASTM C42, L/D correction factors are only applicable to low-density and normal density concretes having strengths between 2,000 and 6,000 psi. While C42 doesn't explicitly provide correction factors for these higher strengths, the correction factors were applied in this case as a conservative measure (likely underestimating strength by a small amount).*

*** Note that preparation time before testing was slightly longer than the testing time in the C42 standard due to the transport of the cores from the site to the laboratory; the effect on the results in this case are likely small, if any.*

5.2.3. Chloride Concentration

Chloride concentration testing was performed on 10 concrete core samples extracted across the structure, as well as on 3 powder samples extracted from the deck underside within study areas SA-03 and SA-04. The powder samples were tested for water-soluble chlorides. The cores were tested for acid-soluble chlorides with a few slices selected for comparative water-soluble testing; background chloride concentrations were negligible and thus acid-soluble chloride testing was deemed appropriate for evaluating chloride-induced corrosion risk. The chloride profiles for the deck, floor beams, arch ribs, and abutment/wing wall are shown in Figure 5.11 through Figure 5.14, respectively. Note that for the purpose of the chloride profile charts, the corrosion threshold is conservatively shown to be 0.03 percent chloride by weight of concrete; see Chapter 3 for further discussion of corrosion thresholds. Design cover depths shown on the profile charts were interpreted from the original drawings. The following summarizes the findings:

- Deck
  - Topside
    - Chloride concentration testing was performed in the core samples, with depth measured from the top of the original concrete deck. The asphalt overlay was not tested.
    - Cover to the top layer of reinforcement was not measured through the asphalt overlay. The original drawings indicate the top layer of reinforcement is 2 inches from the top of the structural deck, but the drawings do not specify whether that dimension indicates clear cover or distance to the center of the bar. Therefore, for a 3/4-inch diameter bar, the specified cover is either 1-5/8 or 2 inches.
    - Core 11-01 was located adjacent to a deck joint over FB-6, and the top section of the core consisted of approximately 5-1/2 inches of non-original concrete (i.e., likely repair material) over approximately 5-1/2 inches of original concrete. Chloride concentrations were above the threshold in the non-original concrete to the estimated depth of reinforcement.
    - Core 11-02 was located away from a deck joint over FB-5. Non-original concrete (i.e., likely repair material) was present at the top of the core with depth varying from approximately 1 to 2 inches. Chloride concentrations were below threshold to the estimated depth of reinforcement, which was within the region of non-original concrete. Chloride concentrations were above threshold in the original concrete just beyond (below) the non-original concrete. In other words, above-threshold chlorides are present in the original deck concrete just below the repair material, likely from deep chloride penetration into the deck concrete before the repair was installed.
  - Underside
    - Underside chloride concentration tests were performed on concrete powder samples. The samples were obtained by hammer drilling on the deck underside in approximately 1/4-inch increments and capturing the powder in sample bags.
    - Samples P-03 and P-04B from SA-03 and SA-04, respectively, had chloride concentrations below threshold. Sample P-03 was located approximately midway between FB-4 and FB-5 (approximately 14 feet from the nearest deck joint) and was not near any visible deterioration. Sample P-04B was located near FB-7 in Bay 7 (FB-6 to FB-7, approximately 8 feet from the nearest deck joint) and was also not near any visible deterioration.
    - Sample P-04A from SA-04 had chloride concentrations above threshold and was located approximately midway between FB-6 and FB-7 (approximately 5 feet from the nearest deck joint), near a large area of delaminated concrete.

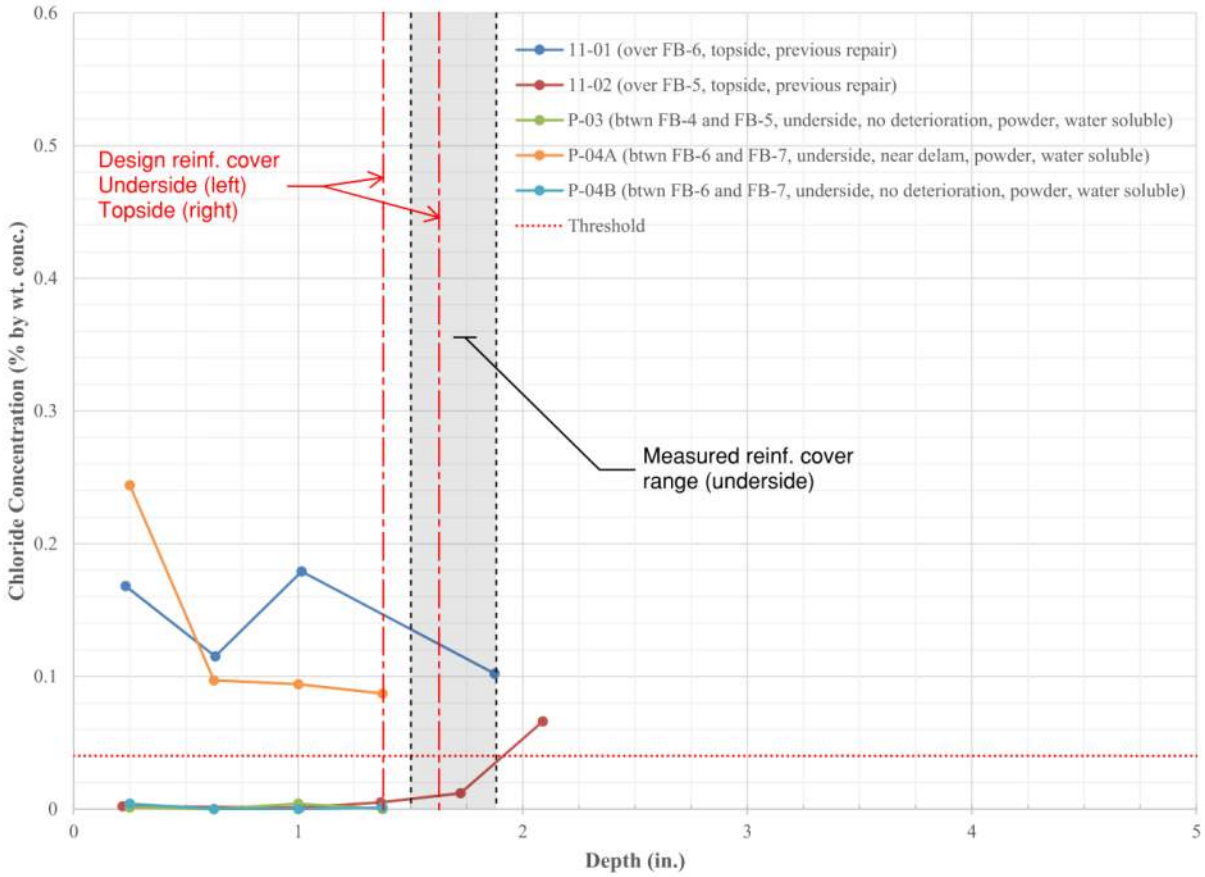


Figure 5.11. Chloride concentrations for **deck samples.**

- Floor Beams
  - Core 05-01, located at FB-3 (i.e., below a deck joint), had chloride concentrations significantly above threshold at the depth of reinforcement.
  - Cores 02-01 and 08-01, located at FB-5 and FB-8 (i.e., away from deck joints), respectively, had chloride concentrations below threshold at the depth of reinforcement.

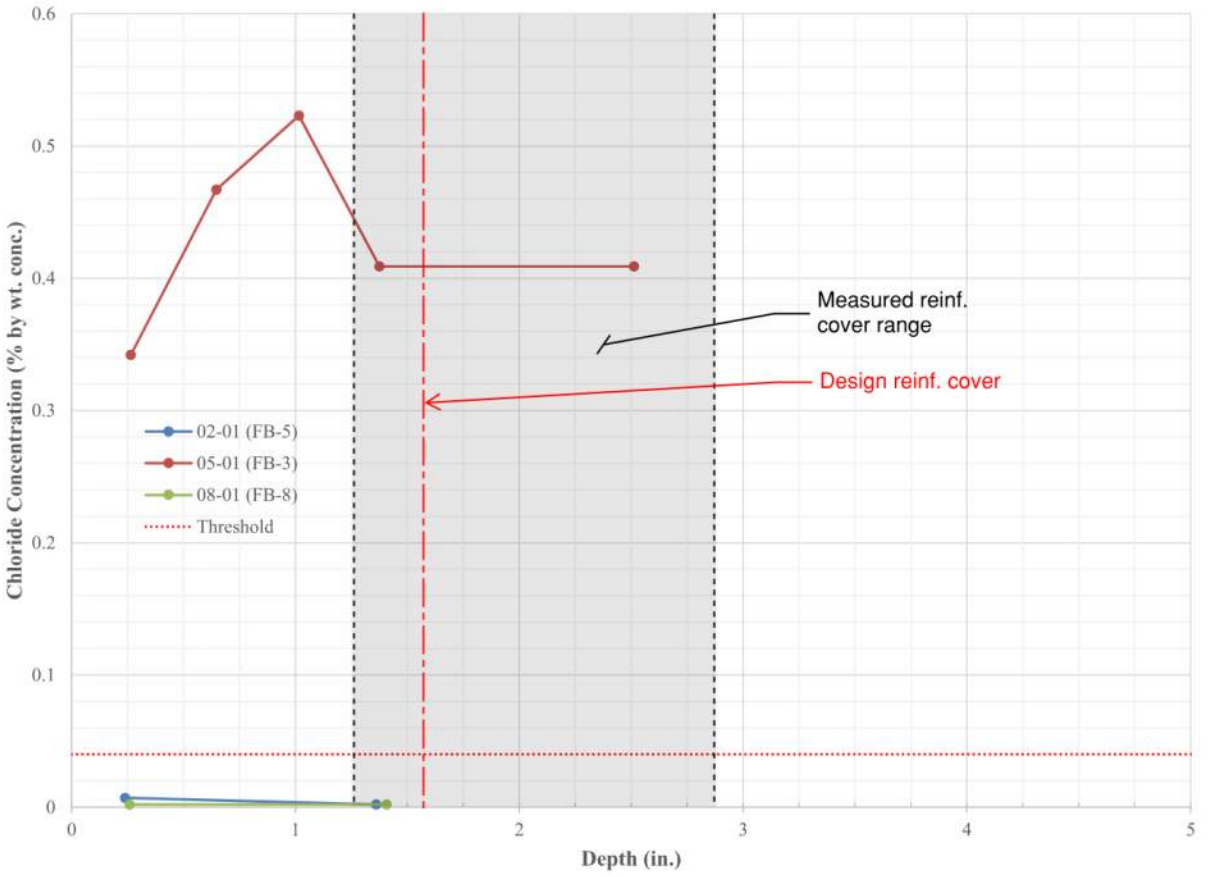


Figure 5.12. Chloride concentrations for **floor beam samples.**

- Arch Ribs
  - Cores 01-01 and 07-01 had chloride concentrations above threshold within the range of measured reinforcement cover depths. The concentrations peaked at approximately 1 inch deep, then decreased until below threshold at depths around 3 to 3-1/2 inches.
    - Core 01-01 with above-threshold chloride to a depth of near 3 inches was located on the exterior face of the downstream arch rib, below a deck joint at FB-3.
    - Core 07-01 also with above-threshold chloride to a depth of near 3.5 inches was located on the interior face of the upstream arch rib, below FB-8 near the arch spring line.
  - Core 13-01 had chloride concentrations well below threshold at the depth of reinforcement. The core was located on the interior face of the downstream arch rib, near the south abutment.

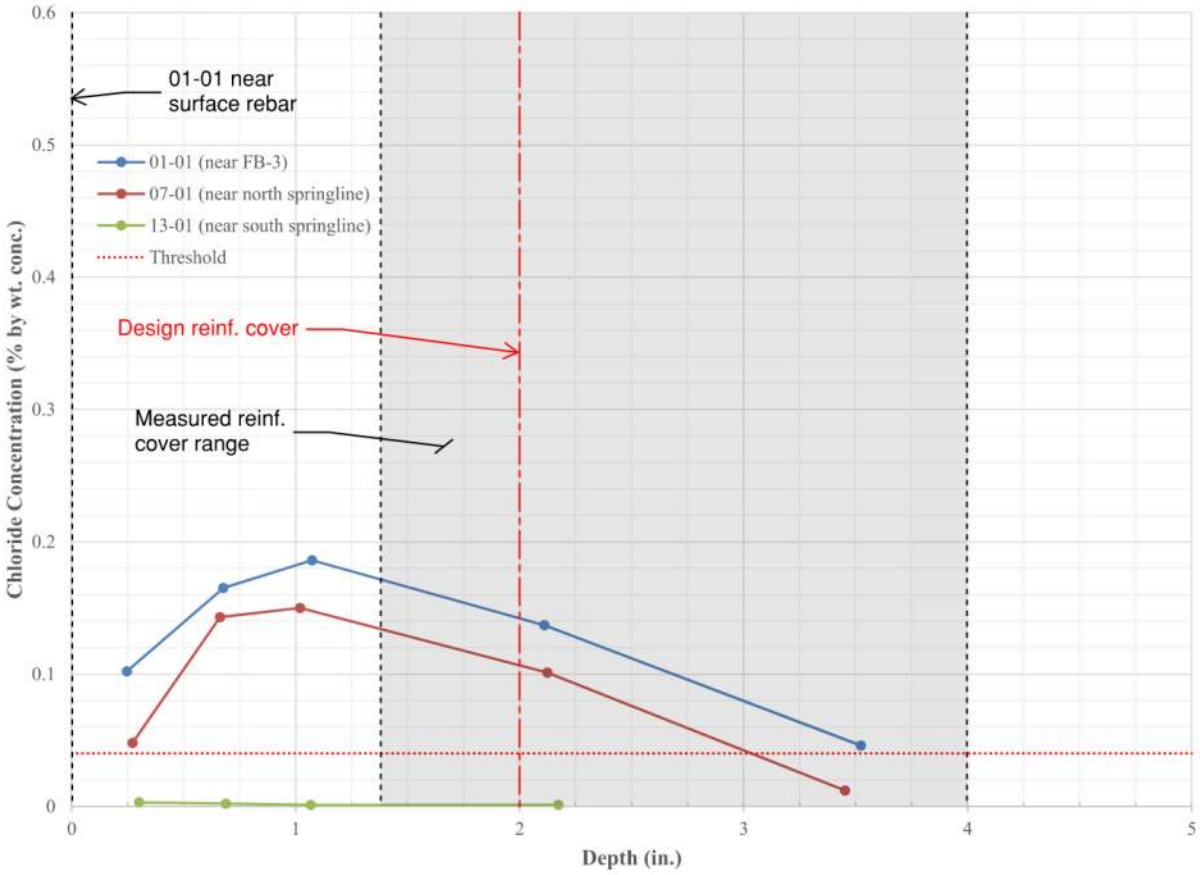


Figure 5.13. Chloride concentrations for **arch rib samples.**

- Abutment/Wing Wall
  - Core 17-01 was located on the upstream south wing wall, below the drain outlet, and had chloride concentrations above threshold to the depth of reinforcement.
  - Core 15-02 was located on the south abutment wall and had chloride concentrations below threshold to the depth measured, which was less than the depth of reinforcement.

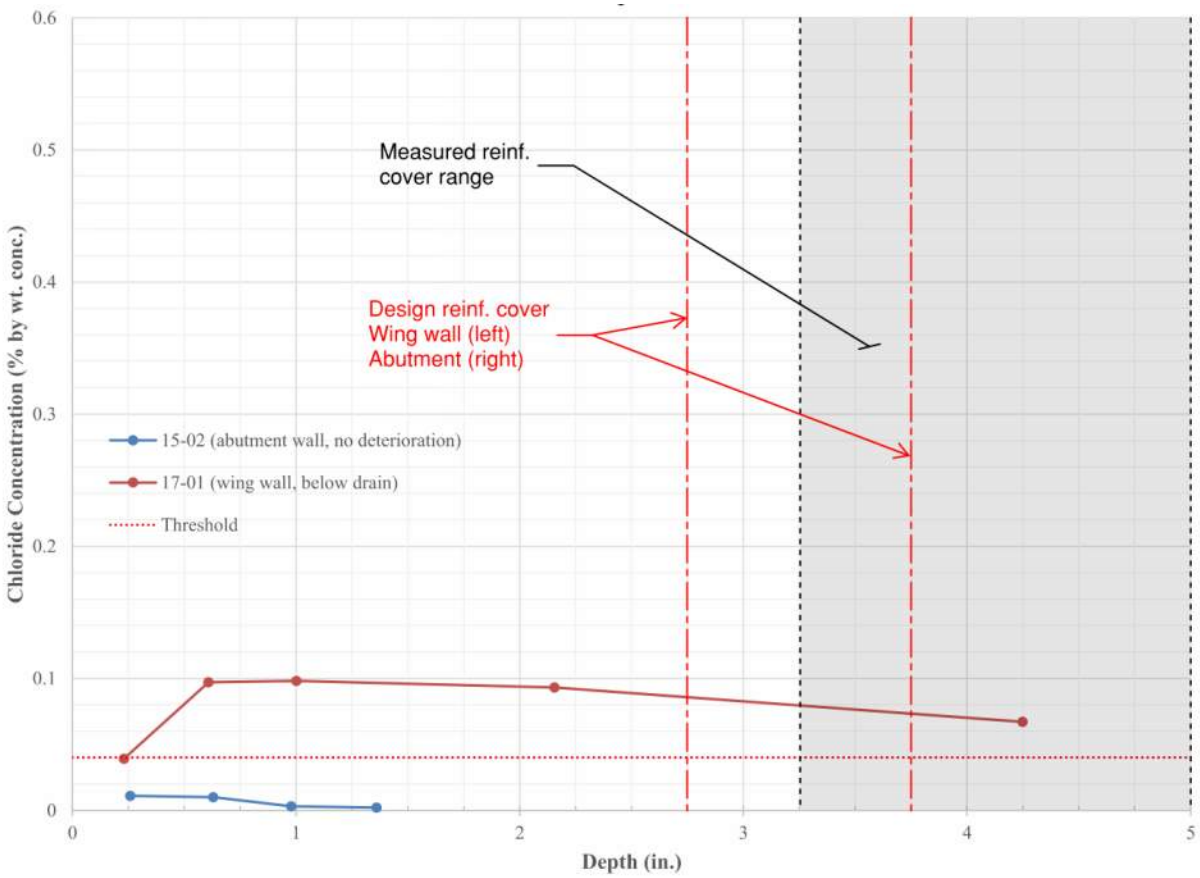


Figure 5.14. Chloride concentrations for **abutment and wing wall samples.**

5.2.4. Petrographic Analyses

As described in APPENDIX D, of the 18 cores removed from structure, five were selected for detailed petrographic examination, including Cores 01-01, 08-01, 11-01, 13-01, and 15-01 (Table 5.4). Additionally, five cores were selected for limited (partial) petrographic examination, including Cores 17-01, 17-02, 17-03, 18-01, and 18-02 (Table 5.5).

Table 5.4. As-Received Condition of Concrete Cores for **Detailed Petrographic Examination**

Sample ID	Element/ Coring Orientation	Length (inches)	Concrete Repair Present?	Macro Cracking Present?	Core Notes	Area Examined
01-01	Arch/ Horizonal	12	No	No	Flat formed exterior surface	Exterior 2.5 inches
08-01	Floor Beam/ Horizontal	10	No	No	Formed exterior surface with exposed sand; Voids adjacent to rebar and some coarse aggregate particles	Exterior 2 inches
11-01	Deck/ Vertical	10.5, not including asphalt topping	Yes, 5.25 inches thick	Yes, surface-parallel cracks in interior nominal 3.5 inches, through and around aggregate	Contains two concretes and asphalt overlay; waterproofing on top surface of repair concrete and bottom surface of substrate concrete; a cementitious layer underlying the bottom waterproofing on interior surface; Asphalt overlay nominally 2.25 inches thick, debonded (not examined)	Full Core Length
13-01	Arch/ Horizontal	11.5	No	Yes, overlapping crack tips on exterior surface, and surface-perpendicular crack extending from exterior surface through a few aggregate particles up to 3.25 inches deep	Flat formed exterior surface with overlapping crack tips	Exterior 3.5 inches
15-01	Abutment/ Horizontal	10.5	No	No	Weathered exterior surface with exposed fine aggregate and slightly crumbling paste	Exterior 2 inches

Table 5.5. As-Received Condition of Concrete Cores for **Limited Comparative (Partial) Petrographic Examination**

Sample ID	Element/ Coring Orientation	Length (inches)	Concrete Repair Present?	Macro Cracking Present?	Core Notes	Area Examined
17-01	Retaining Wall/ Horizontal	11.5	No	No	Weathered exterior surface with exposed coarse and fine aggregate	Exterior 2.5 inches
17-02	Retaining Wall/ Vertical	13	No	Yes, “map cracking” on exterior surface and intermittent surface-parallel cracking through full thickness of core, through and around aggregate	Slightly uneven exterior surface, but no exposed aggregate	Full Core Length
17-03	Retaining Wall/ Horizontal	21	No	Yes, cracked into many pieces, crumbling in exterior nom. 11 inches, through and around aggregate, generally parallel or oblique to surface but some cracks are irregular	Full length not definite because exterior surface was not retained, but exterior-most segment is only partial diameter, second segment is nominally 6.5 inches and cracked, and interior-most segment is nominally 10 inches with relatively less cracking but still has low volume of cracks	Full Core Length
18-01	Abutment/ Horizontal	12 to 13.5	No	Yes, fractured at 6.275 to 8.5 inches from exterior, mostly through aggregate, white deposits on fractured faces of some aggregate particles; low volume of aggregate with internal cracking	Weathered exterior surface; internal cracking in some aggregate	Full Core Length
18-02	Abutment/ Vertical	8.5	No	Yes, fractured into several pieces, through and around aggregate	Weathered/eroded (inferred) exterior surface, with abundant coarse and fine aggregate standing proud; Core is crumbled into several pieces, length cannot be measured but reportedly 8.5 inches to surface	Full Core Length

Petrographic examination of the 10 core samples listed above was conducted in general accordance with the methods and procedures outlined in ASTM C856, Standard Practice for Petrographic Examination of Hardened Concrete. The cores were sectioned and then lapped to achieve a smooth, fine matte finish suitable for examination with a stereomicroscope. For the five cores selected for detailed petrographic examination, a thin section was prepared and examined using a petrographic (polarized-light) microscope at magnifications from 50X to 400X. For the five cores selected for limited petrographic examination, only lapped sections were prepared and examined, and the examinations were more high-level and primarily to evaluate the extent of visible cracking and apparent freeze-thaw distress.

5.2.4.1 Petrographic Observations

Bulk concrete general characteristics - The concrete in each of the cores that is original to the bridge construction contains similar constituents in relatively similar proportions. This concrete, which will be referred to as the "bulk concrete" is composed of natural metamorphosed siliceous gravel coarse aggregate and natural siliceous sand fine aggregate dispersed within a non-air-entrained portland cement paste. The concrete in the core samples is overall well consolidated, and distribution of constituents is uniform.

The coarse aggregate is composed of siliceous metamorphic rocks, mainly including schist, phyllite, and quartzite as well as small amounts of non-metamorphosed siliceous rocks including calcite-bearing sandstone, shale, and weathered granitic to basaltic rocks. Based on cursory geologic review of the project site area, it is likely that these aggregates were quarried near the project site. The coarse aggregate is well graded and well distributed, with a nominal top size from 3/4 to 1 inch in the superstructure elements and up to 1-1/2-inch top size in the abutment and wing wall footings. The sand is mainly composed of quartz, quartzite, granite, feldspar, shale, schist, chert, mica, and small amounts of miscellaneous rocks and mineral particles. Portions of the coarse and fine aggregate are reactive with alkalis in the paste and can support ASR.

Paste-aggregate bond is overall moderately strong to strong. Thin section examination reveals the cement particles are larger in size than modern portland cement, typical for historic concrete. Based on the observed paste features, the water-to-cement (w/c) ratio of the cores is judged to be moderate. It can be difficult to estimate w/c in historic concrete due to significant changes in concreting practice and lack of reference samples, but estimated w/c ranges are from 0.43 to 0.53 with minor variations between cores.

The depth of carbonation was typically shallow, from negligible (less than 1/16 inch) to 1/4 inch from the exterior surface. Deeper carbonation, from 1/2 to 1 inch, was present in Core 15-01 that had poor consolidation and/or surficial scaling at its exterior surface. Air content in the bulk concrete was less than 3 percent, with only entrapped air voids present.

ASR in bulk concrete – For the cores removed from locations where water saturation was likely at times, including the cores from the abutment and wing wall footings near the water line and the deck at the expansion joint, moderate to severe ASR distress was observed. For the other cores, which were taken from more free-draining concrete elements/surfaces, ASR gel was observed in some lapped sections, but there were only trace signs of ASR distress.

Freeze-thaw in bulk concrete – The cores exhibiting moderate to severe ASR also exhibited signs of F/T distress, which occurs when non-air-entrained concrete gets critically saturated with water and is then

subjected to freezing and thawing cycles. The bulk concrete is not air entrained (air-entraining technology was not available at the time of original construction) and has an estimated w/c higher than 0.45; both factors are expected to lead to greater vulnerability to cyclic freezing and thawing than modern concretes when the concrete is critically saturated. The maximum depth of microcracking in the cores due to cyclic F/T distress was observed to be as follows:

- 10.5 inches in Core 11-01 (deck at expansion joint)
- 13 inches in Core 17-02 (wing wall footing vertical near waterline)
- 10 inches in Core 17-03 (wing wall footing horizontal near waterline)
- 8 inches in Core 18-01 (abutment footing horizontal near waterline)
- 2.5 inches in Core 18-02 (abutment footing vertical near waterline)

ASR and freeze-thaw acting in combination in bulk concrete – All of the core samples manifesting moderate to severe ASR distress also manifested F/T distress and were extracted from locations where critical water saturation is likely. As such, the cracking distress is the result of both the F/T and ASR mechanisms acting in combination.

Repair materials – The deck core examined petrographically (Core 11-01) had several inches of bonded repair material present at its surface (the top of the original deck). The repair material is air-entrained portland cement concrete, also containing fly ash, with 1/4-inch top size aggregate and moderately low w/c. Such material is typical of bridge deck repair concrete.

5.2.4.2 Petrographic Examination Summary and Discussion

Petrographic examinations of the 10 cores revealed varying extents of distress. The cores removed from elements not subject to water saturation were found to be in generally good condition, with only trace amounts of microcracking due to either ASR or F/T.

Advanced distress was only observed in those cores subject to critical water saturation, including the abutment and wing wall footings and the deck at expansion joint. The more advanced distress was mainly caused by cyclic freeze-thaw, ASR, or a combination of both. (Note that the cores were not removed over reinforcing steel experiencing chloride-induced corrosion; chloride-induced corrosion distress is addressed elsewhere in this report.) It is judged that cyclic F/T likely initiated near-surface cracks, creating pathways for moisture entry, and accelerating ASR distress, with both mechanisms worsening the condition.

ASR-related distress was most severe in cores that likely exhibited preexisting cracking caused from cyclic F/T or other causes. The primary reactive constituent is strained quartz in various metamorphic rocks, which typically reacts slowly. Given that this concrete is nearly 100 years old, significant changes in ASR are not expected if moisture exposure conditions remain unchanged.

Other than ASR and cyclic freeze-thaw distress as noted above, no other internal distress mechanisms such as internal or external sulfate attack or salt crystallization distress were identified in the petrographic examination.

5.2.5. Reinforcing Steel Properties

As detailed in APPENDIX E, yield strength and ultimate strength were determined for four reinforcing steel samples, and chemical composition properties were determined for five reinforcing steel samples.[12] The samples were extracted by WJE from locations where the reinforcement was exposed on the bridge and where element stress should be low. The properties were compared to requirements specified in ASTM A-15 (1914) *Standard Specification for Billet-Steel Concrete Reinforcing Bars*, which was the reinforcing bar standard in effect at the time of original construction. The results are summarized in Table 5.6 and Table 5.7 below. As shown, the chemical and tensile properties of the samples were generally consistent with either Structural Steel Grade (minimum yield strength of 33 ksi), Intermediate Grade (minimum yield strength of 40 ksi), or Hard Grade (minimum yield strength of 50 ksi), as listed in ASTM A15 (1914). Multiple samples exhibited concentrations of phosphorus that exceed the requirements of ASTM A15 (1914), which could reduce weldability.

Some of the samples had section loss due to corrosion. Bar size was determined by weighing the samples and calculating the bar size using the density of steel. Strain extensometers were placed on the bars in the regions that exhibit the least corrosion pitting in order to minimize the effect of the corrosion. The strength results determined using this method should provide a reasonable estimate of actual steel yield strength and a conservative estimate of ultimate strength.

Table 5.6. Reinforcing Steel Tensile Test Results

Sample/Test ID	Element	Bar Type	Bar Size (in)	Yield Strength (ksi)	Ultimate Strength (ksi)	ASTM A15 (1914) Grade
11215	Arch Rib	Round, deformed	3/4	56.9	89.0	Hard
11216	Railing	Round, deformed	11/16	46.0	76.8	Intermediate
11217	Deck	Round, deformed	7/16	44.5	63.4	Structural Steel
11218	Arch Rib	Square, deformed	1-1/8	42.9	68.9	Intermediate

Table 5.7. Reinforcing Steel Chemical Composition Test Results

Sample/ Test ID	Carbon (%)	Manganese (%)	Phosphorus (%)	Sulfur (%)	Silicon (%)	Copper (%)	Nickel (%)	Chromium (%)	Molybdenum (%)
11215	0.36	0.53	0.01	0.038	0.15	<0.05	0.1	0.07	<0.03
11216	0.30	0.93	0.058	0.057	<0.05	<0.05	<0.05	<0.05	<0.03
11217	0.31	0.83	0.144	0.127	<0.05	<0.05	<0.05	<0.05	<0.03
11218	0.23	0.67	0.075	0.056	<0.05	<0.05	<0.05	<0.05	<0.03
N/A*	0.23	0.86	0.071	0.046	<0.05	<0.05	<0.05	<0.05	<0.03

**No Sample/Test ID was assigned because no tensile tests were performed. Sample was obtained from deck edge.*

[12] Chemical composition was determined for five total samples; the fifth sample could not be tested for mechanical properties due to advanced section loss.

6. FINDINGS

6.1. Findings That Could Impact As-Inspected Structural Model

Findings of the materials testing and condition assessment that could impact the as-inspected structural analysis model and load rating were provided by WJE to HNTB during the course of the Phase 2 assessment. This included concrete compressive strength results, reinforcing steel strength testing results, and reinforcing steel section loss measurements for the floor beams below the deck joints. See Sections 4.2.2 and 5.2 of this report for summaries of this data, which are not repeated here.

6.2. Comparison with Minnesota Open Spandrel Concrete Arch Bridges

To provide a frame of reference, the general condition of the Windsor Bridge was compared to previous assessments performed recently by HNTB and WJE for the 3rd Avenue Bridge (constructed c. 1918, Figure 6.1) and the Franklin Avenue Bridge (constructed c. 1923, Figure 6.2), both of which are open spandrel concrete arch bridges constructed in Minnesota around the same time as the Windsor Bridge (c. 1930). WJE performed a condition investigation, in-depth evaluation, and repair design and construction observations for restoration of the Franklin Avenue Bridge between 2007 and 2017 and the 3rd Avenue Bridge between 2017 and 2024. The general findings for each component of the three bridges are compared and contrasted in the table below.



Figure 6.1. Overall view of 3rd Avenue Bridge after rehabilitation.



Figure 6.2. Overall view of Franklin Avenue Bridge after rehabilitation.

Table 6.1. Comparison of Distress Quantities in Windsor Bridge and Similar Bridges in Minnesota

Bridge Element Category	Sub-Category	Percentage of Surface Area Exhibiting Distress (DQR)		
		Franklin Avenue (2008 and 2013)*	3rd Avenue (2017)*	Windsor Bridge (2024)*
Deck	Topside	1 to 9% (deck replaced c. 1980)	0 to 2% (deck replaced c. 1980)	N/A concrete, 7% of asphalt overlay debonded (deck is original)
	Underside	3% (deck replaced c. 1980)	13 to 16% (deck replaced c. 1980)	11% (up to 64% in single bay)
Floor Beams	Non-Expansion Joints	2% (all floor beams replaced c. 1980)	1% (all floor beams replaced c. 1980)	3 to 8% (floor beams are original)
	Expansion Joints	10% (all floor beams replaced c. 1980)	34% (all floor beams replaced c. 1980)	27 to 35%
Spandrel Columns	Non-Expansion Joints	2% (all columns replaced c. 1980)	2% (some columns replaced c. 1980)	N/A (low) (columns are original)
	Expansion Joints	2% (all columns replaced c. 1980)	10 to 36% (some columns replaced c. 1980)	N/A (low) (columns are original)
Abutments	--	10 to 20%	1%	8 to 12%
Arch Ribs	--	14% (north arch) 20% (south arch)	9 to 24% (outside ribs) 1 to 10% (middle ribs) 11 to 14% (barrel arches)	4 to 10% (middle thirds) 3 to 16% (outer thirds) 10% (overall)
Pier Walls	Piers without Joints	3 to 6%	1 to 4%	No piers
	Piers with Joints	33 to 36%	7 to 17%	No piers
Pier Bases (Exposed Footings)	--	Not quantified separately	20 to 100%	10 to 29%

** Indicates year of WJE inspection*

As shown, the amount of deterioration of the Windsor Bridge is in many respects similar to the Minnesota bridges. For example, the exposed portions of the footings have advanced deterioration due to cyclic freeze-thaw damage of the critically saturated non-air-entrained concrete near the waterline. A similar degree of deterioration is present on the arches, which are original on all three bridges.

Differences are that the deck of the Windsor Bridge is original and is more severely deteriorated, whereas the decks of the Minnesota bridges were replaced in the 1980's and were again starting to show deterioration at the time of WJE's assessments. Similarly, the floor beams of the Windsor Bridge are original and very severely deteriorated below expansion joints, whereas the floor beams of the Minnesota Bridges were replaced in the 1980's and were again showing deterioration.

6.3. Interpretation of Distress Quantity Ratios and Thresholds for Preservation Actions

The distress quantity ratios (DQRs) and condition state quantities presented in Chapter 4 provide a quantitative view of the current condition of the structure. WJE understands VTrans does not have a state-specific guideline for interpretation of distress quantities relative to preservation actions. Therefore, WJE referenced other federal and state guidelines.

The Federal Highway Administration (FHWA) "Bridge Preservation Guide" provides a framework for a systematic approach to a preservation maintenance program. This document is intended to mesh with the National Bridge Inventory (NBI) inspection programs and associated bridge ratings that are performed. The document defines a few key terms:

- **Bridge Preservation** – Actions or strategies that prevent, delay, or reduce deterioration of bridges of bridge elements, restore the function of existing bridges, keep bridges in good condition and extend their life. Preservation actions may be preventive or condition-driven.
- **Preventive Maintenance** – A planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system (without substantially increasing structural capacity).
- **Rehabilitation** – Major work required to restore the structural integrity of a bridge as well as work necessary to correct major safety defects.
- **Replacement** – Total replacement of a structurally deficient or functionally obsolete bridge.

The AASHTO "Guide to Bridge Preservation Actions", Appendix A, provides a compilation of source documents that contain guidance on preservation actions. These include the AASHTO and FHWA documents as well as a number of state DOT documents. Most of these guidelines provide actions based on quantities of individual condition states that have been documented (CS1, CS2, etc.).

The guideline adopted in this assessment for determining appropriate preservation actions, and in turn estimating remaining service life as further explained below, is the Minnesota DOT "Bridge Preservation and Improvement Guidelines," last updated April 15, 2022. The MnDOT document establishes general guidelines for corrective actions based on aggregated condition states defined as area of unsound concrete, which is comparable to the distress quantities calculated herein.

The two main categories of actions in the MnDOT document are "Preservation" and "Improvement." Preservation can take the form of either preventative maintenance or major preservation, and improvement can include both bridge rehabilitation and bridge replacement. Major preservation activities include, but are not limited to, actions such as joint repair or replacement; deck overlays; partial deck replacement; concrete element patching repairs; or installation of cathodic protection. Bridge rehabilitation activities are more extensive types of work, such as full-scale replacement of components (deck, superstructure, substructure), bridge widening, or major structural repairs that increase capacity.

The guideline thresholds for each of these actions are presented in Table 6.2. For interpretation of the thresholds presented for bridge decks, the Windsor Bridge is categorized as a Rural Major Collector, with an ADT of 3,300 (last counted in 2018).

Table 6.2. MnDOT Guideline Thresholds for Preservation Actions, Utilized Herein for Anticipated Remaining Service Life Estimation

Component	Element Category	Preventive Maintenance	Major Preservation[1]	Rehabilitation[2]
Deck	Wearing Surface	≤ 2% Do Nothing or Spot Repairs	2 to 10% Mill and Patch	10 to 25%: Deck Repairs and Re-Overlay >25%: Replace Deck
	Deck and Slab Underside			
	Sidewalk			
	Joints	≤ 10%	> 10%	--
Superstructure	Floor Beams	≤ 10%	10 to 20%	>20%
	Columns			
	Arch Ribs			
	Tie Beams			
Substructure	Abutments	≤ 10%	10 to 40%	>40%
	Wing Walls			
Other Elements	Pedestrian Railing	≤ 15%	> 15%	--

[1] *Major preservation activities include but are not limited to actions such as joint repair or replacement; deck overlays; partial deck replacement; concrete element patching repairs; or installation of cathodic protection.*

[2] *Rehabilitation activities include more extensive types of work, such as full-scale replacement of components (deck, superstructure, substructure), bridge widening, or major structural repairs that increase capacity.*

The criteria in Table 6.2 are based on a percent of the structure that is in condition state CS3 or CS4. For concrete elements, in general, this refers to "unsound" concrete, or concrete that has spalled, exhibits exposed reinforcement, or contains wide cracks, but also can include more minor aspects such as build-up of efflorescence or surface scaling. Notably, however, this criteria does not always include delaminated areas of concrete. WJE defined the "distress quantity ratio" (see Section 4.4.1) to include CS3 and CS4 conditions and to also include delaminations and freeze-thaw damage because these conditions will also require concrete repair. As such, the "distress quantity ratios" calculated by WJE based on the inspection findings are considered directly comparable to the thresholds in Table 6.2.

6.4. Deterioration Mechanisms and Anticipated Remaining Service Life by Element

The following sections summarize the deterioration mechanisms and expected performance for each of the element categories investigated. Based on WJE's interpretation of the overall element level inspections and the follow-up testing results, the anticipated remaining service life of each element category (i.e., the element's "durability potential") was assessed. Predictions of future performance presented herein are relative and qualitative. Numerical service life modeling has not been performed.

The anticipated remaining service life is used here to describe, in relative terms, the time until the quantity of distress in a given element, without corrective action or intervention, is likely to reach or exceed the

guideline threshold for a "Rehabilitation" action as listed in Table 6.2.[13, 14] For the purpose of this report, the anticipated remaining service life was categorized as follows:

- Short – "Rehabilitation" action threshold already reached or likely to be reached within 5 years;
- Moderate – "Rehabilitation" action threshold likely to be reached in 5 to 20 years; and
- Long – "Rehabilitation" action threshold likely not to be reached for more than 20 years

For example, the observed condition of floor beams below deck joints is already beyond rehabilitation thresholds, and the anticipated remaining service life of these elements before rehabilitation is short. In contrast, the columns, tie beams, and floor beams away from deck joints have a long anticipated remaining service life considering the generally good condition and low risk for widespread deterioration due to the distress mechanisms expected to define their life.

Based on the current distress quantity, the identified deterioration mechanisms, and severity of deterioration, the anticipated remaining service life of each element category and sub-category was assessed and is presented in the following tables.

Table 6.3 below provides an overall summary of the distress observed and the distress quantity ratios by bridge element category and sub-category. Refer to Chapters 4 and 5 for more detailed information.

Table 6.4 below provides an overall summary of the primary deterioration mechanisms and anticipated remaining service life determined based on this assessment. In this table, "chloride" and "carbonation" refer to the types of corrosion mechanism that will influence life. The table is again divided according to bridge element category and sub-category.

The text sections following the tables provide a more detailed explanation of the conditions and anticipated remaining service life for each element type. Considerations for rehabilitation in light of these findings are discussed in the next chapter.

[13] What constitutes the "end of life" of a member must be defined in a service life analysis. Using percent of damage to define end of life is one rational method, which is used herein. The most relevant percentage of damage values to use in this context are those used by various state DOT's including the Minnesota DOT's guideline thresholds for a "Rehabilitation" corrective action, as provided in Table 6.2.

[14] As defined for the purpose of this current condition evaluation report, the "anticipated remaining service life" is the future life of an element assuming that corrective action or other interventions are not taken at this time. Service life estimates for bridge elements after they have been treated with various rehabilitation alternatives will be studied in the rehabilitation alternatives and design portions of this project to follow.

Table 6.3. Summary of Distress and Distress Quantity Ratios by Bridge Element

Element Category	Sub-Category	Distress Typically Observed	Distress Quantity Ratio
Deck	Topside	Distress generally not visible due to asphalt overlay, but previous patching of asphalt indicates previous topside distress in the concrete deck below; core samples from deck contain repair material on the topside indicating previous deck distress.	N/A (covered by asphalt overlay, which was 7% debonded)
	Underside – bays adjacent to expansion joints	Widespread water staining, delaminations and spalls with exposed and corroded reinforcing steel; field and material testing show high chlorides and very active corrosion activity	5 to 32%
	Underside – bays away from expansion joints	Typically less widespread water staining, delaminations and spalls, although field and material testing show elevated chlorides and active corrosion in some areas; Bay 8 has a large spalled area and a large full-depth repair	2 to 7% (except for Bay 8 at 64%)
Railings	--	Severe and widespread concrete distress in the form of wide cracking, delaminations and spalls with exposed and corroded reinforcing; some elements almost gone	N/A (>50%)
Floor Beams	At expansion joints	Widespread water staining, delaminations and spalled with exposed and severely corroded reinforcing steel (stirrups often corroded away, bottom steel with section loss)	27 to 35%
	Away from expansion joints	Generally good condition except for unsound areas of concrete at isolated locations of original honeycombing or shallow cover conditions	3 to 8%
	Overhangs at upstream and downstream edges	Widespread map cracking with efflorescence indicative of freeze-thaw and ASR distress, with some areas of resulting concrete disintegration at edges; three overhangs along upstream edge appear in good condition	Included in above
Columns	--	Generally good condition except for corrosion-induced spalls at bottoms of some columns and at isolated locations of original honeycombing or shallow cover; no structural cracking at bottoms of columns	Area too small for objective DQR
Tie Beams	--	Generally good condition except for unsound areas of concrete at isolated locations of original honeycombing or shallow cover conditions	3 to 5%

Element Category	Sub-Category	Distress Typically Observed	Distress Quantity Ratio
Arch Ribs	Middle thirds	Relatively good condition except for unsound areas of concrete at isolated locations of original honeycombing or shallow cover conditions	4 to 10% middle thirds
	Outer thirds	Occasional, isolated locations of delamination and spalling with exposed reinforcement, often at locations of original honeycombing or shallow cover conditions; testing shows some active corrosion and elevated chlorides; surface map cracking indicative of early freeze-thaw or ASR distress at a few isolated locations	3 to 11% south thirds 12 to 16% north thirds
Abutments	Walls	Relatively good condition except for unsound areas of concrete at isolated locations of original honeycombing or shallow cover conditions	8 to 12%
	Footings	Deep F/T and ASR distress on top and face of footing near water line; material testing shows F/T depth up to 13-inch deep	10 to 29%
Wing Walls	Walls	Generally good condition except for unsound areas of concrete at isolated locations of original honeycombing or shallow cover conditions; area below drain outlet at southwest wing wall has heavy water staining and core sample indicates elevated chlorides and evidence of F/T and ASR distress	4% (limited survey)
	Footings	Deep F/T and ASR distress on top and face of footing near water line; material testing shows F/T depth up to 13-inch deep	20% (limited survey)

Table 6.4. Summary of Anticipated Remaining Service Life by Bridge Element

Element Category	Sub-Categories	Primary (and Secondary) Deterioration Mechanisms	Anticipated Remaining Service Life (Without Corrective Action or Intervention)
Deck	Topside	Chloride	Short
	Underside – adjacent to expansion joints	Chloride (F/T and ASR)	Short
	Underside – away from expansion joints	Chloride	Moderate to short
Railings	--	Likely chloride, F/T and ASR	Short
Floor Beams	At expansion joints	Chloride	Short
	Away from expansion joints	Chloride, carbonation*	Long
	Overhangs at upstream and downstream edges	Chloride, F/T and ASR	Short
Columns	--	Chloride, carbonation*	Long
Tie Beams	--	Carbonation*	Long
Arch Ribs	Middle thirds	Carbonation*	Long
	Outer thirds	Chlorides, carbonation*	Moderate
Abutments	Walls	Carbonation*	Long
	Footings	F/T and ASR	Short
Wing Walls	Walls	Carbonation*	Long
	Footings	F/T and ASR	Short

** Carbonation is currently shallow, and the carbonation front advances slowly; thus, remaining life is long for elements where carbonation is the primary deterioration mechanism. Carbonation-induced corrosion and ensuing concrete distress are only currently present at locations of surficial poor consolidation or shallow cover over reinforcing steel; distress will continue to develop slowly as the carbonation front reaches the next shallowest steel.*

6.4.1. Deck

Topside. The topside of the driving surface of the original concrete deck is obscured by an asphalt overlay. Based on the original drawings, an overlay was not present at the time of original construction but was added at some point, possibly as early as the major bridge repairs performed in the 1940's. The present overlay is underlain by a membrane that provides some protection to the original concrete deck below, but whether any previous overlays had a membrane is unknown. Approximately 7 percent of the present asphalt overlay is debonded, and two of the three cores taken from the deck contained concrete repair material at the topside of the original concrete deck. The topside of the sidewalk, which has always been exposed, has widespread concrete delamination (distress quantity of 20 percent for sidewalk topside alone), spalling, and very high HCP's indicative of active reinforcing steel corrosion. The distress quantity in the sidewalk topside is well above the guideline threshold for a "Rehabilitation" action. Chloride concentrations in the two cores tested from the deck are above the corrosion threshold at or beyond the level of the reinforcing steel, even for the core taken away from the expansion joint. The deck core taken at the expansion joint also had deep freeze-thaw and ASR damage.

Underside. On average across the deck underside, the distress quantity is 11 percent. However, adjacent to the two deck expansion joints, the deck underside has very active corrosion potentials and a higher percentage of unsound concrete, near or above the guideline threshold for a "Rehabilitation" action. Near the joints, powder samples from the deck underside showed above-threshold chloride contents at the depth of the reinforcing steel.

Away from the expansion joints, the deck underside has less (but still widespread) areas of delaminated concrete, with some isolated spalls and exposed reinforcement. Some areas of active corrosion were identified although concrete powder samples indicated below-threshold chloride contents. One area in Bay 8 near the north end of the bridge has a very large spall and a full-depth repair. The distress quantity for Bay 8 alone is 64 percent.

Deck Edges. The deck edges at the upstream and downstream sides of the bridge deck have advanced deterioration from both chloride-induced corrosion of reinforcement and freeze-thaw and ASR. Large areas of the deck edges have spalled.

Anticipated Remaining Service Life. The primary deterioration mechanism for the deck of the Windsor Bridge, which is original to the structure, is chloride-induced corrosion. Chloride contents at the level of the reinforcing steel, both at topside and underside areas of the deck, are above the corrosion threshold. The deck edges also exhibit advanced freeze-thaw and ASR deterioration. Current distress quantities for the sidewalk topside and the worst underside deck areas are already above the guideline thresholds for a "Rehabilitation" action. In view of these factors, the anticipated remaining service life of the deck is judged to be short (see definitions above for "end of service life" and the time periods for the different categories of remaining service life).

6.4.2. Railings

The concrete balustrade railings, which are original to the bridge construction, are severely deteriorated with some baluster concrete nearly gone. Although the railings were not tested, the damage is likely due to a combination of chloride- and carbonation-induced corrosion, freeze-thaw deterioration, and ASR deterioration. The railings are supported on the deck edges that have advanced freeze-thaw and ASR deterioration.

Anticipated Remaining Service Life. The railings are severely deteriorated beyond their useful service life and warrant replacement.

6.4.3. Floor Beams

At expansion joints. Two of the eight floor beams are located below deck joints (FB-3 and FB-6). At these beams, advanced concrete deterioration is present, as evident by widespread delamination, spalling, and/or exposed and corroded reinforcing steel over approximately 30 percent of their area. The bottom longitudinal reinforcing bars have cross sectional area loss up to approximately 30 percent due to the corrosion. Most of the stirrups are corroded completely through near the bottoms of the beams. Field and laboratory testing confirm that chloride-induced corrosion is actively occurring, no doubt stemming from long-term leakage of chloride-laden water through the deck joints.

Away from expansion joints. The six floor beams located away from bridge expansion joints (FB-1, FB-2, FB-4, FB-5, FB-7 and FB-8) are in much better condition than those at the joints. At these beams, only 3 to

8 percent of the beam surface area is unsound, and field and laboratory testing show that carbonation and chlorides have not penetrated very far into the concrete. The small, isolated areas of concrete surface deterioration are due to carbonation- or chloride-induced corrosion of reinforcing steel where cover is very shallow or where original surficial honeycombing has allowed deeper carbonation locally.

Overhangs. The portions of the floor beams that cantilever (overhang) beyond the columns exhibit more advanced deterioration than the portions of the floor beams between the columns. The more advanced deterioration at the overhangs is due to freeze-thaw and ASR deterioration acting together with the deterioration from chloride-induced corrosion. Water saturation of the concrete, a requisite for freeze-thaw deterioration, has apparently occurred in the overhangs due to runoff over the deck edges, leakage around previous deck drains, and wind-driven precipitation. Notably less deterioration is present in three of the floor beam overhangs (FB-1, FB-2, and FB-8) as compared to the other thirteen.

Anticipated remaining service life. The full length of the two floor beams located below expansion joints (FB-3 and FB-6) have advanced deterioration due to chloride-induced corrosion of reinforcement and are already beyond their useful service life. Similarly, most of the overhangs of the remaining six floor beams, except for three overhangs, are severely deteriorated and beyond their useful service life due to freeze-thaw and ASR deterioration acting in combination with chloride-induced corrosion.

The central portion of the six floor beams located away from joints (FB-1, FB-2, FB-4, FB-5, FB-7 and FB-8) are in generally good condition, with only isolated areas of unsound concrete at locations of shallow cover or original honeycombing. Field and laboratory testing indicate low chlorides, shallow carbonation, and low corrosion activity. Anticipated service life for these portions of the floor beams and the three floor beam overhangs is judged to be long, considerably longer than 20 years.

6.4.4. Columns and Tie Beams

Except for small, localized areas of unsound concrete, the columns and tie beams across the bridge are in good condition, with relatively small percentages of distress quantity. Localized distress includes delamination and spalls at the bases of some columns, likely due to poor consolidation and/or shallow cover and carbonation- or chloride-induced corrosion. Also, distress is present at isolated locations across the columns and tie beams with locally shallow cover and/or original honeycombing. Test results show low corrosion activity, low chlorides, and shallow carbonation. No distress that appeared to have been induced by structural loads was observed.

Anticipated remaining service life. In light of these factors, the anticipated remaining service life of the columns and tie beams is judged to be long, considerably longer than 20 years.

6.4.5. Arch Ribs

Overall. Current distress quantities for the arch ribs range from 3 to 16 percent, with an average of 10 percent over all the surfaces. No distress that appeared to have been induced by structural loads was observed.

Middle thirds. The middle thirds of both the upstream and downstream ribs exhibit somewhat less deterioration (distress quantity of 4 to 10 percent) and did not show active corrosion or elevated chlorides in the testing. Deterioration in this portion of the ribs is driven by slowly progressing carbonation (still

generally very shallow) that reaches bars with shallow cover and causes small, isolated carbonation-induced corrosion spalls.

Outer thirds. The outer thirds of the arches have a somewhat greater amount of deterioration than the middle thirds, especially the north thirds (12 to 16 percent distress quantity in the north thirds). Field and laboratory testing showed active corrosion and above-threshold chlorides in some areas of the arches that are located below the deck expansion joints or in the outer thirds of the arches where chloride-laden water could run down from leaking expansion joints. The chloride profiles for the three cores taken from the arches, along with the measured reinforcing steel cover results, are telling (see Figure 5.13): cover to the stirrups in the arches varied widely between test locations and cover was sometimes quite shallow; chloride contents in the core taken from the middle third were very low; chloride contents in the two cores taken from the outer thirds were above the corrosion threshold to a depth of approximately 3 inches, which means that bars in these zones with cover less than 3 inches are prone to chloride-induced corrosion.

The arches are also prone to freeze-thaw and ASR deterioration, which based on the testing should only be severe in zones of critical water saturation. For the arches, this could be at the arch springlines and below columns. These mechanisms are not currently manifesting much distress in the arches, except possibly at the far south ends of the arches where map cracking was observed; however, it should be guarded against in the future by limiting water penetration into the arch concrete.

The precise cause of the atypical relatively wide longitudinally-oriented crack near the north springline of the downstream arch rib is unclear. The crack is generally consistent with internal expansion of the concrete; it does not appear to be related to structural loading. Possible sources of the expansion include ASR, bursting forces from reinforcing steel corrosion (albeit no corrosion stains or coincident delaminations were evident), and/or freezing of water in an internal void. The nature of the crack (i.e., one singular, wide crack rather than narrower closely-spaced cracks) suggests that the reinforcing steel cover is deep at this location. If rehabilitation of the arch ribs is pursued, testing could be performed at this location to further assess the cause of the cracking.

Anticipated remaining service life. The anticipated remaining service life of the middle third of the arches is long, considerably more than 20 years. The anticipated remaining service life of the outer thirds of the arches is moderate to long, perhaps 10 to 20 years without intervention before reaching a distress quantity at the guideline threshold for a “Rehabilitation” action.

6.4.6. Abutments

Walls (faces). The faces of the abutment walls have only isolated areas of concrete distress and relatively small percentages of distress quantity (approximately 10 percent). Field and lab testing show moderate corrosion potential and low chloride contents away from areas exposed to moisture, and moderate to high corrosion potential in areas exposed to moisture (e.g., beneath the utility conduit opening).

Footings. The exposed top and side faces of the footings below the abutments (as well as the footings below the wing walls – see below) typically exhibit a high distress quantity ratio (up to approximately 30 percent) and deep concrete deterioration due to freeze-thaw and ASR deterioration, up to 13-inches deep based on petrographic examination. The footing of the north abutment is in worse condition than at the south. Because of their proximity to the water line, the footing surfaces experience repeated saturation

with water, which together with the non-air-entrained nature of the original concrete makes them very susceptible to freeze-thaw damage. Freeze-thaw cracking likely initiated first but then cracks allow more water penetration and ASR deterioration contributes due to reactive aggregates present in the mix (see petrographic examination summary in Section 5.2.4). Considering the limited reinforcement in the footings, carbonation and chloride-induced corrosion are not expected to cause much distress.

Wide horizontal cracks are present in some portions of the footing faces that appear to be original cold joints (lift lines) with poor concrete consolidation that have eroded over time due to water entry and the aforementioned mechanisms. Also, one small area below the north footing exhibits localized scour.

Although the footings are still functioning adequately, they have reached or nearly reached their anticipated remaining service life according to the definitions adopted in this report.

Anticipated remaining service life. The anticipated remaining service life of the abutment faces is long. However, the remaining service life of the abutment footings is categorized as short due to the advanced levels of freeze-thaw deterioration currently present and the propensity for this deterioration to worsen with continued exposure to water saturation and freeze-thaw cycles. Based on past performance, a rough estimate for further propagation of freeze-thaw damage into the footing concrete is 1/8 inch of depth per year of continued exposure.

6.4.7. Wing Walls

Walls (faces). Similar to the abutment wall faces, the wing wall faces have only isolated areas of concrete distress and relatively small percentages of distress quantity. For the walls that were accessible for testing, field and laboratory testing showed little evidence of chloride-related distress. The core that was tested from below the drain outlet in the south wall, downstream side, had elevated chlorides, presumably due to outfall of chloride-laden drain water. However, the reinforcing steel in the wing walls is relatively deep and widely spaced. Corrosion of these bars, if it occurs, would result in only localized spalls that can be readily repaired as part of regular maintenance.

Footings. The footings of the wing walls exhibit similar distress quantities and the same distress mechanisms as the footings of the abutments. Although the footings are still functioning adequately, their anticipated remaining service life is short according to the definitions adopted in this report.

Anticipated remaining service life. The anticipated remaining service life of the wing wall faces is anticipated to be long. However, similar to the abutment footings, the remaining service life of the abutment footings is categorized as short due to the advanced levels of freeze-thaw deterioration currently present and the propensity for this deterioration to worsen with continued exposure to water saturation and freeze-thaw cycles.

7. SUMMARY AND CONSIDERATIONS FOR REHABILITATION ALTERNATIVES

WJE as a subconsultant to HNTB performed an inspection and testing program for the Windsor Bridge (Bridge 55 Over Mill Brook) in Windsor, Vermont from May 2024 through February 2025. WJE's work was performed in two phases: Phase 1 consisted of an overall element level inspection of all components of the bridge, and Phase 2 consisted of field testing, material sampling in representative study areas, and laboratory testing of samples. The scope of the work was defined in the Bridge Inspection Work Plan. The results of the inspection and testing were provided incrementally in various meetings and correspondence in order to facilitate HNTB's parallel load rating studies. A comprehensive summary of the inspection and testing program is provided in this report. This chapter also provides findings that follow from the assessment results and that should be considered in the development of rehabilation alternatives.

7.1. Deck

The deck topside is obscured by the asphalt overlay, but based on the condition of the asphalt and exposed sidewalk, as well as the concrete repair material found in core samples taken from the deck, widespread deterioration of the deck topside below the overlay is probable. Lab testing of core samples from the deck topside indicates chloride contents above the corrosion threshold at the depth of the reinforcing steel; the core at the expansion joint also exhibited deep freeze-thaw and ASR deterioration. Accordingly, deck topside distress is currently advanced and will likely accelerate in the coming years.

The deck underside exhibits elevated chlorides, active corrosion, and a relatively high percentage of current distress (average distress quantity of 11 percent across whole underside, maximum distress quantity in a single bay of 64 percent at Bay 8 near the north end of the bridge). Where underside distress is present, full-depth deck repairs would be appropriate. The deck edges also have advanced freeze-thaw and ASR deterioration, with deep spalling.

Given these factors, anticipated remaining service life of the deck (i.e., time until rehabilitation thresholds are reached, without intervention) is judged to be short (less than 5 years). Away from the expansion joints, the deck is typically in better condition than near the expansion joints; however, test results show that some deck areas away from the joints also have active corrosion and elevated chlorides, likely associated with through-thickness deck cracks. Also, one bay (Bay 8 near the north end of the bridge) is in notably worse condition. As such, even deck areas away from the expansion joints have a short, or at best moderate, remaining life.

Given the condition of the deck, complete replacement of the deck should be included in any rehabilitation alternative intended to provide a long service life extension. For a rehabilitation with a shorter service life extension, it may be feasible to retain portions of the deck away from the expansion joints and to replace the deck and floor beams at the joints. Localized repairs to the deck topside and underside would also be needed. With removal and replacement of the existing overlay, this scheme should achieve a service life extension of approximately 15 to 20 years. If this option is pursued, the viability of saving the existing deck should be reviewed after the asphalt overlay is removed. Corrosion mitigation to achieve a longer service life from the existing deck does not seem cost effective.

7.2. Railings

The concrete balustrade railings have exceeded their useful service life and are beyond practical repair. Anchorage of the railing to the deck edges is questionable due to deterioration of the overhangs. Replacement of the railings should be anticipated in any rehabilitation alternative.

7.3. Floor Beams

7.3.1. At Expansion Joints

The two floor beams located below expansion joints (FB-3 and FB-6) exhibit very advanced and widespread deterioration due primarily to chloride-induced corrosion, as well as freeze-thaw and ASR deterioration at their overhangs. These two beams have exceeded their useful service life and based on their poor condition warrant replacement in any rehabilitation scheme.

7.3.2. Away from Expansion Joints

The central portion of the six floor beams located away from expansion joints (FB-1, FB-2, FB-4, FB-5, FB-7 and FB-8) are in good condition (distress ratio less than 10 percent), show limited chloride contamination, and have a long remaining service life. With only minor, isolated surface repairs, these portions of the floor beams (along with the columns that support them – see below) could be salvaged in rehabilitation alternatives that have a long service life extension.

7.3.3. Overhangs at Upstream and Downstream Edges

Three of the twelve floor beam overhangs located away from expansion joints are in good condition and could be salvaged as part of a rehabilitation project (FB-1, FB-2 and FB-8, all at the upstream edge). The other nine floor beam overhangs have advanced deterioration and should be replaced in any rehabilitation alternative. Given that most of the overhangs need replacement (including these nine plus the four located at expansion joints), it may be most practical to replace them all.

7.4. Columns and Tie Beams

Similar to the central portion of the floor beams away from joints, the columns and tie beams are generally in good condition, show limited chloride contamination, and have a long remaining service life. With only minor, isolated surface repairs, these elements could be salvaged in rehabilitation alternatives that have a long service life extension. (Note that none of the columns or tie beams are located below deck expansion joints.)

7.5. Arch Ribs

Current distress quantities for the arch ribs range from 3 to 16 percent, with an average of 10 percent over all the surfaces. The middle thirds of the arches are in somewhat better condition than the outer thirds. For the middle thirds, the deterioration mechanism is primarily carbonation-induced corrosion at locations of surficial poor consolidation or reinforcing steel with shallow cover. For the outer thirds, there are also zones of elevated chlorides and chloride-induced corrosion apparently due to leakage of chloride-laden water through the deck joints. Although not currently manifesting distress associated with freeze-thaw and ASR, the arches are also prone to deterioration from these mechanisms if the concrete is allowed to become critically saturated with water. For this reason, long-term rehabilitation schemes should include measures to limit water penetration into the arch concrete.

As shown in Table 6.1, these distress quantities are similar, or lower, than those documented in the arches of two Minnesota bridges of similar type, vintage and exposure. The arches for the two Minnesota bridges had similar deterioration mechanisms, except that their concrete did not exhibit the same propensity for ASR as the Windsor Bridge concrete. The arches of the Minnesota bridges were rehabilitated recently to achieve a service life extension of at least 50 years using repair and corrosion mitigation methods designed by WJE and HNTB (see Johnson et al. 2017, 2019, 2022 and 2024).[15, 16, 17, 18] Similar rehabilitation methods are options for the arch ribs of this bridge.

Long-term service life of the arches of the Windsor Bridge is achievable if corrective actions are taken to address the deterioration mechanisms affecting this structure. This would include well-designed localized concrete repairs and protection of the arch concrete from water penetration (such as by crack repairs and application of an appropriate water-resistant coating or sealer). Since above-threshold chlorides are present at the depth of the reinforcing bars in some areas of the arches, corrosion mitigation measures to enhance durability should also be considered. Options for corrosion mitigation are discussed below. The fundamentals behind each of these methods are described in WJE-authored articles (see Johnson et al. 2004 and 2019 and Gaudette and Slaton 2007).[19, 20, 21]

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[15] *The Franklin Avenue Bridge – Part 1: History, Investigation and Rehabilitation*, Arne Johnson, John Lawler, Dan Enser, Travis Konda, and Paul Backer, ACI's Concrete International, 2017.

[16] *Lessons in Cathodic Protection from Soldier Field and the Franklin Avenue Bridge*, Arne Johnson, John Lawler, and Michael Murphy, APT Journal of Preservation Technology, 2019.

[17] *Concrete Rehabilitation Design for the Historic 3rd Avenue Bridge*, Arne Johnson, Tanner Swenson, and Dan Enser, ICRI Concrete Repair Bulletin, 2022.

[18] *3rd Avenue Bridge Rehabilitation (2024 Repair Project of the Year Winner)*, Arne Johnson and Tanner Swenson, ICRI Concrete Repair Bulletin, 2024.

[19] *Soldier Field Stadium: Corrosion Mitigation for Historic Concrete*, Arne Johnson and S. K. Lee, APT Journal of Preservation Technology, 2004.

[20] *APT Preservation Brief 15: Preservation of Historic Concrete*, Paul Gaudette and Deborah Slaton, Association of Preservation Technology, 2007.

[21] *Lessons in Cathodic Protection from Soldier Field and the Franklin Avenue Bridge*, Arne Johnson, John Lawler, and Michael Murphy, APT Journal of Preservation Technology, 2019.

7.5.1. Potential Corrosion Mitigation and Protection Measures for Arch Ribs

The following are corrosion mitigation and protection measures that may be used alone or in combination to extend the life of the arch ribs. Example applications of various corrosion mitigation and protection options from previous WJE projects are shown in Figure 7.1 through Figure 7.8 below. The electrochemical methods are illustrated in Figure 7.9 through Figure 7.12.

7.5.1.1. Coatings and Sealers

Water-resistant coatings and sealers applied to the surface of the concrete can mitigate concrete deterioration mechanisms that are water-driven and hence extend the service life of concrete. This includes all the mechanisms causing distress in the Windsor Bridge concrete, as explained in this report.

Film-forming coatings are often considered inappropriate for use on a historic structure, unless the structure was coated historically. Film forming coatings typically change the color and appearance of a surface, and higher build coatings can also mask the concrete surface texture, such as original form-board lines. Depending on the type of coating and surface preparation, a coating may need to be reapplied after 20 years.

An alternative can be clear, penetrating concrete sealers, such as silanes, which can prevent up to 90 percent of water from penetrating into the concrete compared to untreated concrete by rendering fine (< 10 mils) cracks and pores within the concrete hydrophobic. However, they do not bridge or fill large cracks. Once applied, penetrating sealers cannot be effectively removed and are therefore considered irreversible. Their effectiveness does decrease over time, meaning they require periodic re-application to remain effective, at intervals of approximately 10 to 20 years.

7.5.1.2. Crack Repairs

As noted above, concrete sealers do not bridge or fill cracks wider than about 10 mils. Also, coatings may not have sufficient elasticity to bridge wide or moving cracks. As such, crack sealing repairs should be employed for wider and moving cracks in combination with concrete surface sealing or coating.



Figure 7.1. Example of crack repairs and film-forming coating on historic concrete surface (3rd Avenue Bridge).



Figure 7.2. Example of crack repairs and film-forming coating on historic concrete surface (Franklin Avenue Bridge).

7.5.1.3. Cathodic Protection

Corrosion is an electrochemical process in which electrons flow between cathodic (positively charged) and anodic (negatively charged) areas on a metal surface. Corrosion occurs at the anodes. Cathodic protection (CP) is a technique used to reduce the rate of corrosion of reinforcing steel in concrete by introducing an electrical current from an external source to make the steel surface the cathode (rather than the anode) of the electrochemical cell. There are two types of cathodic protection systems: impressed-current (active) system and sacrificial anode (passive, galvanic) systems.

Impressed current CP involves using a DC rectifier to pass the electrical current from an inert anode to a reinforcing steel mat. The anode is typically a titanium or mixed metal oxide mesh, ribbon, or probe that is implanted within the concrete or the concrete repair. The applied current forces the entire reinforcing steel mat to become the "cathode" in the electrochemical cell, and as such not to corrode. The anode is consumed gradually over time, but impressed current systems can last 50 years or more. Impressed current CP is the most effective means of mitigating steel corrosion and has been used in practical structural applications since the 1970s. However, impressed current CP systems require a continuous electrical power source at the structure, and ongoing monitoring, adjustment, and maintenance is essential to ensure a proper protection current. For this reason, impressed current CP systems are not commonly used for bridge rehabilitation projects except in states like Florida that have devoted internal resources to support their maintenance. Further and specifically relevant to the Windsor Bridge concrete, the high current levels associated with impressed current systems may cause local accumulation of alkali metals and exacerbate ASR concerns unless the system is specifically designed to mitigate this risk.

Sacrificial anode CP systems, sometimes called galvanic anode CP systems, are similar to impressed current CP systems, except that electrical current is not externally applied but rather flows naturally by galvanic action. An electrically active metal, typically zinc, is used as the anode and is connected electrically to the embedded reinforcement. Common anodes include zinc or aluminum alloy coatings thermally sprayed on to the concrete surface (sometimes called metalizing or arc-sprayed), zinc sheets mechanically fastened to the concrete surface, and zinc pieces embedded within the concrete (often simply called embedded anodes). The anode is "sacrificial" in that it is consumed over time, while the reinforcing steel, functioning as the cathode in the electrochemical cell, is protected. The main advantages of this technique are maintenance-free operation (operates by its own galvanic action), ability to target the anodes where they are most needed, and oftentimes lower cost compared to other options. Since the anode is consumed over time, periodic reapplication of the anode is required. The design life of sacrificial CP systems is typically 20 to 30 years.



Figure 7.3. Cathodic protection anodes inserted into cored holes in the top of the arch ribs at the 3rd Avenue Bridge.



Figure 7.4. Cathodic protection anodes placed in slots cut into the arch ribs corners at the Franklin Avenue Bridge.

7.5.1.4. Metalizing

Metalizing, also called thermally sprayed or arc-sprayed zinc, is simply one form of sacrificial anode application that is often used on bridges and other structures. It has the advantage of being relatively easy and inexpensive to apply compared to other systems, and it is generally effective at slowing corrosion of near-surface reinforcing bars. Metalizing is essentially a coating of zinc or aluminum alloy on the concrete surface. It is typically topcoated with some type of sealer to limit self-corrosion, and this has the added advantage of mitigating some water ingress into the concrete. However, the combined system also has the disadvantage of masking form-board lines in historic concrete. Practical applications of metalizing also have the limitation of only protecting bars that are no deeper than about 3 to 4 inches from the concrete surface, based on WJE's experience.



Figure 7.5. Example of trial area of metalizing applied to historic concrete surface (Soldier Field Stadium).



Figure 7.6. Example of metalizing applied to modern concrete surface (3rd Avenue Bridge precast girder).

7.5.1.5. Electrochemical Chloride Extraction (ECE)

This technology, first used in the U.S. in the late 1980's, removes chloride ions from concrete electrochemically. Much like impressed current CP, the installation involves a temporary external anode (typically a metallic mesh), a dike or porous covering containing an alkaline electrolyte solution, and a rectifier. A high electrical current is applied from the anode to the reinforcing steel for a period of several weeks. To maintain charge neutrality, free chloride ions in the concrete migrate away from the reinforcing steel, toward the anode, and hydroxyl ions form on the surface of the reinforcing steel. After the process, the external anode and electrolyte, which will have absorbed some of the chloride ions, are removed. The hydroxyl ions also increase the pH at the steel surface and may repassivate the steel.

Drawbacks of this process are that it is relatively costly and can sometimes stain the concrete surface, which may be a significant consideration for a historic structure. The treatment operation can be difficult due to the issues of applying and containing the electrolyte, particularly overhead, which also presents environmental challenges if electrolyte leakage occurs. From WJE's experience, chloride extraction can be successful at removing chlorides near the surface and adjacent to the reinforcing, but it requires careful pre-application testing to establish appropriate operating voltages and duration, as well as quality control during the treatment and verification testing after the treatment. Removal of chlorides deeper than about 3 inches is typically not practical or achievable. As with impressed current systems, ECE may stimulate ASR.

The duration of the benefit of ECE depends on how effectively the chloride ions were removed; once the treatment is concluded, the chlorides can begin the process of diffusion back towards the reinforcing steel, but provided the application was effective, it is reasonable to expect that the development of additional chloride-induced corrosion-related damage would be postponed for at least 20 years.



Figure 7.7. Realkalization treatment (similar to chloride extraction treatment) underway on historic concrete surface (Soldier Field Stadium).



Figure 7.8. Installation of mesh and electrolyte media for electrochemical chloride extraction applied to modern concrete surface (Dunwoody Bridge Project).

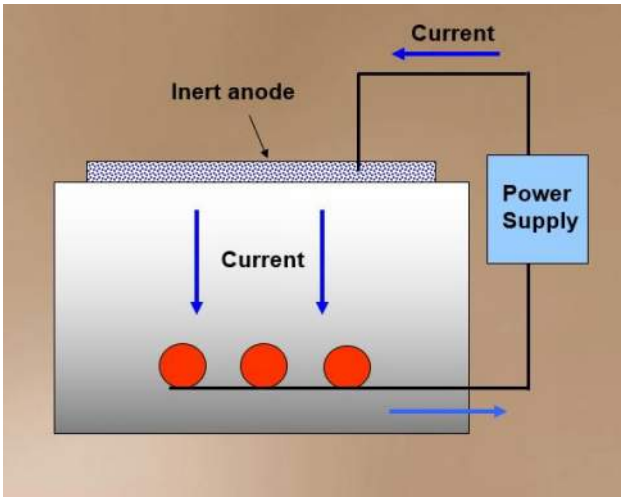


Figure 7.9. Illustration of concrete with reinforcement (red shapes) protected by an **impressed current cathodic protection system** (from Johnson et al. 2004).

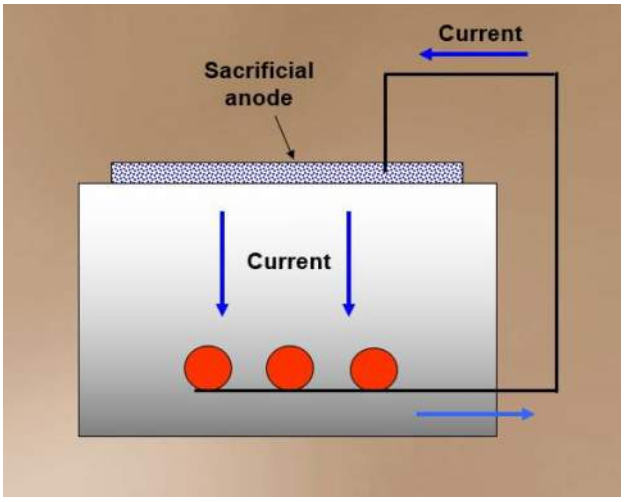


Figure 7.10. Illustration of concrete with reinforcement (red shapes) protected by a **sacrificial anode (galvanic) cathodic protection system** (from Johnson et al. 2004).

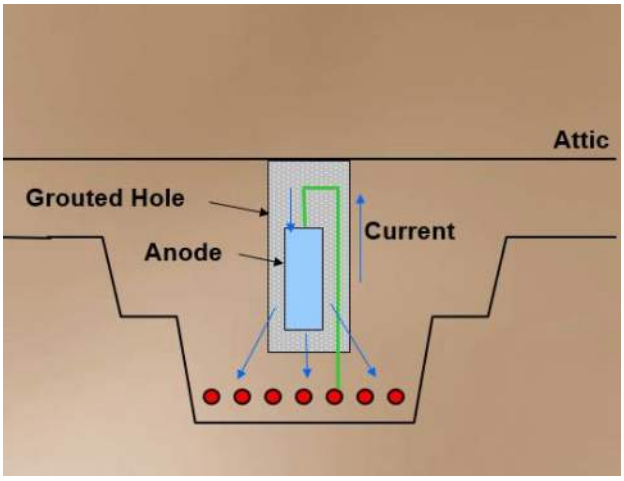


Figure 7.11. Example of a reinforced concrete element protected using **sacrificial anode cathodic protection** with anodes inserted into holes cored from above (from Johnson et al. 2004).

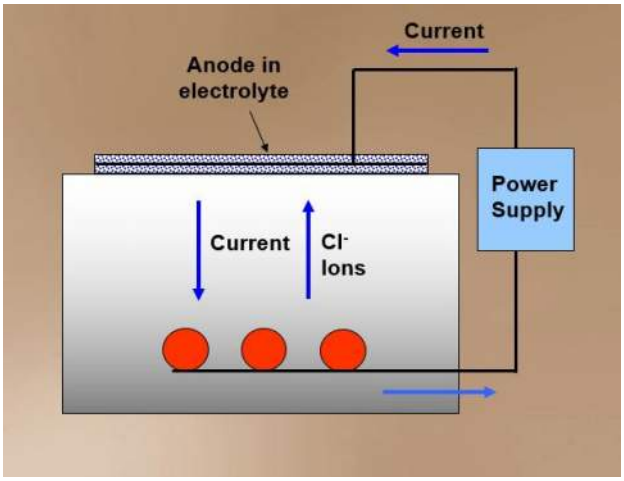


Figure 7.12. Illustration of concrete with reinforcement (red shapes) treated using the **electrochemical chloride extraction** process (from Johnson et al. 2004).

7.5.2. Selection of Corrosion Mitigation Solution

Further testing and analysis are recommended during the rehabilitation design process in order to select the most cost-effective corrosion mitigation solutions for the arches of the Windsor Bridge. Such testing should include additional half-cell potential testing for corrosion activity by zone, GPR surveys for reinforcing steel cover variations, and powder samples for chloride content distribution by zone of the arches. If ECE is considered, further testing and mock-ups could also be helpful to evaluate whether this method is feasible but otherwise not detrimental to the concrete matrix.

Based on our knowledge of corrosion mitigation techniques and the currently available test data for the Windsor Bridge, we recommend that the following points be considered in the development of rehabilitation alternatives, with the final solution being decided based on the further testing and analysis during the design phase:

- To achieve a moderate surface life extension, on the order of **25 years**, we anticipate that localized high quality surface repairs[22] followed by application of a coating/sealer and crack repairs across the arches would be sufficient. (Note that this treatment – localized concrete repairs plus surface coating/sealer – could be repeated after the 25 years, if desired. See section 7.5.1.1.)
- For a longer service life extension, on the order of **50 years**, we anticipate that localized high quality surface repairs plus a combination of protection methods will be needed. In order of increasing cost and effort, such protections methods to be considered include the following:
  - **Distributed CP anodes** installed in holes cored on a regular spacing into the arch ribs, followed by application of a coating/sealer and crack repairs. A line of anodes installed along each arch corner (similar to the installation at the 3rd Avenue Bridge except not staggered, see Figure 7.3) may be sufficient to protect the top and bottom mat of reinforcement. Depending on further test results (see first paragraph above), an additional line of anodes may be needed at the middle of the top and bottom sides of the arches and along the arch sides. If test results show that chlorides are limited to the arch topsides where chlorides pond, the anodes might only be needed on the arch topsides.
  - **Metalizing** applied over the entire arch rib surfaces, followed by overcoating with a water-resistant colored coating. Metalizing should be effective in this application to address corrosion on all surfaces of the arch. The zinc and coating system needs to be reapplied when the zinc is consumed (after 15 to 25 years), and the zinc coating creates a new surface and masks the form-board lines on the historic concrete.
  - **Electrochemical chloride extraction** followed by a coating/sealer and crack repairs. This technique has the potential for a long service life extension, depending on further test results (see first paragraph above). However, as explained above, challenges and potential negative consequences are that the treatment can stain the concrete surface, environmental containment of

[22] For any elements retained in a rehabilitation, the benefits and value of matching the color and texture of the concrete surface repairs to the surrounding original concrete surfaces should be considered in the rehabilitation design process. Options include color matching the repair material to the original concrete color, using concrete stains to blend the repairs with the surrounding original concrete surfaces, matching the texture of the repairs to the surrounding concrete using form-board finishing techniques, and/or covering the repairs with a film-forming water-resistant coating or corrosion mitigation coating.

the electrolyte can be difficult, variable and deep reinforcing steel cover can make treatment complicated, and the treatment can stimulate ASR.

7.6. Abutments

Walls. The abutment walls (faces) are in relatively good condition with low to moderate distress quantities. Field and laboratory test results indicate a low risk for corrosion, no other durability concerns, and a long remaining service life. With only minor, isolated surface repairs, the abutment walls can be salvaged in rehabilitation alternatives that have a long service life extension.

Footings. The exposed top and side faces of the footings below the abutments (and wing walls – see below) typically exhibit a high distress quantity ratio and widespread areas of deep freeze-thaw and ASR deterioration. Some areas have lost several inches of surface concrete due to freeze-thaw disintegration and erosion. Based on petrographic examination of core samples, microcracking due to freeze-thaw deterioration extends as deep as 13 inches from the original surfaces that were cored. This deterioration will continue as long as moisture saturation and freezing and thawing cycles occur.

In any major rehabilitation project, the abutment and wing wall footings should be repaired. The undermining present near the east end of the north abutment also needs to be addressed. Given the widespread, deep deterioration that is present, and the vulnerability of the original concrete to ongoing damage if it remains exposed, a reinforced concrete jacket (new facing) applied to the top and sides of the footings seems to be the practical solution. At least a portion of the depth of the freeze-thaw damaged concrete (6 to 12 inches) should be removed and replaced with a reinforced high-performance concrete layer that is cast to replicate the original footing profile. The new concrete, which will be air-entrained and of high-quality concrete characteristics, will not only replace the damaged concrete but also will provide a water barrier and additional thermal mass so that the original concrete behind the jacket does not experience as much water exposure and as severe freeze-thaw cycling that leads to deterioration. Due consideration should be given in rehabilitation design of the jackets to prevent water penetration through the joints and ends of the jackets.

7.7. Wing Walls

Walls. The wing wall faces that are exposed are in relatively good condition with low distress quantities. Field observations and available test results indicate a low risk for corrosion, no other durability concerns, and a long remaining service life. The area below the drain outlet in the south wall of the upstream side of the bridge has above-threshold chloride levels as deep as the reinforcing steel. However, the reinforcing bars in the wing walls are deep and widely spaced, such that corrosion-related distress should be limited. With only minor, isolated surface repairs, the wing wall faces can be salvaged in rehabilitation alternatives that have a long service life extension. As a first priority, in any maintenance, preservation or rehabilitation scheme, the drain outfalls should be redirected away from the wing wall faces and footings in order to forestall additional erosion and freeze-thaw damage.

Footings. Essentially the same considerations apply to the wing wall footings as for the abutment footings. The most practical rehabilitation scheme for the wing wall footings, as for the abutment footings, is a new concrete jacket. See discussion above under abutment footings.

7.8. Water Main and Utilities

The abandoned water main pipe and its hanger attachments to the deck underside are deteriorated and potentially loose. These elements should be checked and stabilized in the near future and removed as part of any bridge rehabilitation.

APPENDIX A. BRIDGE INSPECTION WORK PLAN

Windsor Bridge 55 Over Mill Brook

Bridge Inspection and Condition Assessment Work Plan

Windsor, Vermont



August 8, 2024
WJE No. 2023.7260

PREPARED FOR:
HNTB Corporation
3 Executive Park Drive
Bedford, New Hampshire 03110

PREPARED BY:
Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, Illinois 60062
847.272.7400 tel

Windsor Bridge 55 Over Mill Brook

Bridge Inspection and Condition Assessment Work Plan

Windsor, Vermont

Arne Johnson, PE, SE
Principal and Project Manager

Tanner Swenson, PE
Senior Associate

August 8, 2024
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CONTENTS

Structure Description **1**

General Scope and Purpose of Inspection **3**

Inspection Schedule **4**

Methods of Access and Equipment **4**

Inspection Procedures **7**

General 7

In-Depth Element Level Bridge Inspection (Phase 1) 7

Follow-up Testing and Material Sampling (Phase 2) 8

Traffic Control Plan **10**

Inspection Team **11**

Safety **12**

APPENDIX A. UBIU Data Sheets

APPENDIX B. Example Bridge Inspection Forms

STRUCTURE DESCRIPTION

Completed in 1930, Windsor Bridge 55 carries US Route 5 over Mill Brook in Windsor, Vermont. The bridge is a single-span, open-spandrel concrete arch bridge measuring approximately 90 feet long by 33 feet wide. The deck is supported by eight floor beams, which are supported by two conventionally-reinforced concrete arch ribs spaced at 20 feet on-center and with an 18-foot rise. The four floor beams nearest the crown of the arch frame directly into the arch ribs, while the other four are supported by spandrel columns extending down to the arch ribs, where cross beams frame between the ribs. At each spring line, the arch ribs frame into concrete abutments supported on spread footings. Overall views of the bridge are shown in Figure 1 and Figure 2, and a location map is shown in Figure 3.



Figure 1. Overall view of Windsor Bridge 55 from the downstream/east side.



Figure 2. Aerial view of bridge, looking east.



Figure 3. Bridge location map (from Google Maps).

GENERAL SCOPE AND PURPOSE OF INSPECTION

The overall objective of the inspection is to gather sufficient data to determine the deterioration mechanisms by element at this particular bridge and, on that basis, to develop structural repair strategies that can be incorporated in bridge rehabilitation alternatives. WJE is performing this work as part of HNTB's team, as described in the project scope of work.

The inspection will begin with an in-depth element level inspection of all above-water components of the bridge. The in-depth inspection will occupy the first week of the inspection, as detailed in the sections to follow. The objectives of the in-depth inspection are to identify and document:

- The range and types of distress and deterioration conditions present in the various elements.
- Signs and evidence of the root causes of the distress and deterioration conditions present.
- Prudent locations for follow-up non-destructive field testing and material sampling.
- Locations and quantities of deterioration so that accurate repair quantity estimates can be developed.

Following the in-depth inspection and analysis of the inspection findings, WJE will carry out a second week of non-destructive field testing and material sampling. The objectives of the testing and materials evaluation work include the following:

- To determine by testing and laboratory analysis the root causes of the distress and deterioration conditions present in the various elements.
- To quantify the properties of the concrete and steel needed to conduct the structural, load rating, and service life analyses.
- To provide a basis for developing repair approaches and details that will address the root causes of the deterioration and be long-lasting, and to further define the extent of the repairs (e.g., how deep the concrete repairs will need to be).

The following elements will be included in the inspection:

- Deck
  - Reinforced concrete deck and sidewalk
  - Asphaltic wearing surface
  - Expansion joints
  - Railings
- Superstructure
  - Floor beams (also called cap beams elsewhere)
  - Spandrel columns
  - Cross beams
  - Arch ribs
- Substructure
  - Abutment walls
  - Footings (where visible above waterline)
- Other Elements
  - Visible/accessible portion of upstream retaining wall adjacent to the north abutment

The following items are excluded from the inspection:

- Inspection of near- or below-water portions of the abutment/footings
- Retaining walls upstream of bridge
- Drains and drainpipes, except that WJE will note general functionality including broken pipes, leakage, and contribution to local deterioration
- Water main
- Utilities (conduit, piping, supports, etc.), except that WJE will note obviously failed supports especially if related to concrete distress

INSPECTION SCHEDULE

Work is scheduled to take place in two phases: an initial in-depth element level bridge inspection of all components of the bridge, followed by testing and material sampling in representative study areas. The details of the two phases of the inspection are described below.

- In-Depth Element Level Bridge Inspection (Phase 1)
  - Dates: August 12-16, 2024
  - Work hours: 9:00 AM to 4:30 PM within lane closure, additional work from ground or sidewalks may be performed outside of these hours.
  - Traffic control: 9:00 AM to 4:30 PM (Tuesday 8/13 through Friday 8/16)
    - Primary lane closure will be in southbound lane to allow operation of UBIU in southbound lane. Work will require switching lane closure to northbound lane for deck and railing inspection for approximately ½ day (no UBIU required for this work in northbound lane).
    - See section below for further details regarding traffic control.
- Follow-up Testing and Material Sampling (Phase 2)
  - Dates: TBD; Town of Windsor indicated September is not desirable due to planned heavy hauling vehicle operation throughout the month. A week in October will likely be selected, if this is acceptable to VTrans as it effects the project delivery schedule.
  - Work hours: TBD
  - Traffic control: TBD

METHODS OF ACCESS AND EQUIPMENT

WJE will provide or rent the following equipment to gain close-up access to the various structural elements and to perform the inspection and testing:

- Under Bridge Inspection Unit (UBIU) – Data sheets for the model A-62 UBIU are appended for reference.
  - Access to below-deck elements will be primarily provided by a 62-foot UBIU. To avoid power lines along the downstream/east side and both ends of the bridge, we anticipate the UBIU will operate within the southbound traffic lane and all below-deck access will originate from the upstream/west side (Figure 4).

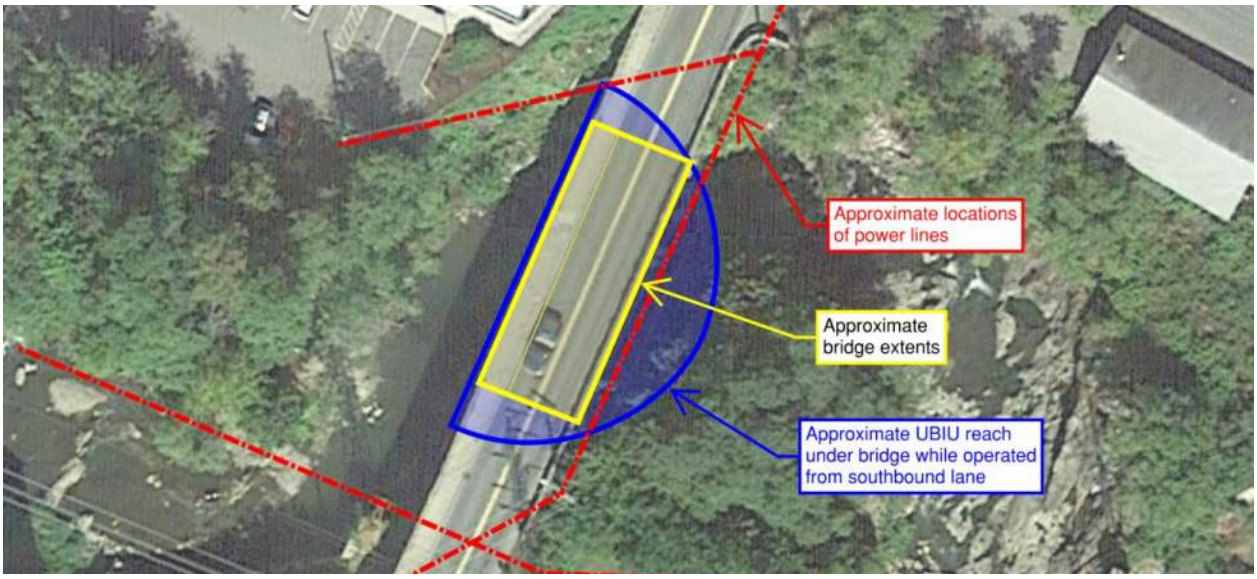


Figure 4. UBIU reach diagram.

- Inspection equipment:
  - Field note-taking equipment (paper base sheets and iPads)
  - Cameras, video camera
  - Flashlights and battery-powered worklights
  - Misc. hand tools - masonry hammers, tape measures, non-permanent marking materials, crack gages, chain drag equipment, calipers, wire brushes, etc.
- Non-destructive testing equipment:
  - Hand-held rebar locators
    - Handheld ground-penetrating radar (GPR)
    - Eddy-current rebar locator
  - Half-cell testing equipment
    - Rolling wheel
    - Single-point probe
  - Corrosion rate testing equipment
  - Concrete resistivity testing equipment
  - Steel ultrasonic thickness gauge
- Material sampling equipment:
  - Concrete coring equipment
    - Typically 4-inch core barrels with extensions for deeper cores where needed
  - Reinforcing steel
    - Electric chipping hammers and hammer drills to expose reinforcing steel as needed
    - Sawzall with metal blades and mini grinder with cutting wheels to remove steel samples
- Standard PPE and equipment for bridge work over water
  - Hard hat
  - Reflective vest

  - Safety glasses
  - Hearing protection
  - Gloves
  - Personal flotation devices (see section on safety below)
  - Work boots
  - Harness and safety lanyard (Y-lanyards or other as needed to maintain 100 percent tie-off)
- Storage: WJE inspection-related equipment (hand tools, PPE, and NDE equipment) will be stored in WJE-owned and/or WJE-rented vehicles. Vehicles will be parked near the work areas when work is underway and will be removed from the job site when work is not underway.

Inspection access for the planned work will vary based on which portions of the structure are to be accessed, as described below.

- Deck Topside and Railings – The deck in the southbound lane will be accessed directly during lane closures to facilitate the deck assessment portion of the inspection work and sample extraction. The railings will be accessed directly during the lane closures and from the sidewalks. For deck and railing inspection in the northbound lane, an approximately 4-hour closure of the northbound lane will be required (see traffic control section below).
- Abutments and Retaining Wall – Access for close-up inspection of the abutments and retaining wall will be from ground and UBIU, potentially supplemented by ladders where access is otherwise not possible. Portions of the abutment footings above the waterline will be inspected to the extent possible.
- Arch Span – Access to the underside of the arch span, including the deck underside, floor beams, spandrel columns, and arch ribs, will be accomplished using the UBIU.
- Access Methods using Drones or Rope Access Techniques
  - Inspectors will seek to access all bridge elements close-up using the above-described access methods. For locations that cannot be accessed close up using these methods, supplemental information may be gained using drone inspection methods or rope access techniques.
  - Drone inspection, if needed, will be performed using WJE-owned and operated equipment. WJE has several small unmanned aerial systems (UAS) and pilots that conduct structural inspections. Each UAS is equipped with a surveillance camera, providing high resolution photographs and video. WJE pilots have remote pilot certificates, knowledge of FAA Part 107 rules and regulations, and extensive flying experience.
  - If rope access techniques are utilized, inspection will be performed by WJE personnel with industrial rope access (IRA) training and experience under the supervision of a SPRAT (Society of Professional Rope Access Technicians) Level III certified rope technician. If utilized, rope access would be during Phase 2 of the inspection effort.

INSPECTION PROCEDURES

General

An in-depth element level inspection will be performed of all above-water components of the bridge. Base sheets for notetaking purposes will be prepared in advance based on available drawings and inspection reports. The base sheets will include all bridge surfaces and element numbering will be consistent with previous inspection reports (i.e., south to north). Notable conditions will be documented using NBIS Condition States (i.e., CS1 through CS4).

In-Depth Element Level Bridge Inspection (Phase 1)

The in-depth element level bridge inspection will be conducted by two WJE personnel. The general daily inspection procedure during the in-depth bridge inspection will be as follows:

1. Pre-job Brief: At the start of each day, all on-site personnel will meet for a pre-job brief. Topics of the pre-job brief will include job-site safety; known traffic control and equipment operations for the day; and planned inspection activities.
2. Inspection preparation:
    a. Collect field sheets and plan view drawings for areas to be inspected.
    b. Identify hazards in the areas to be inspected – loose items, vehicle traffic patterns, electric lines, fall hazards, structure deficiencies, etc.
3. Mark element ID numbers on areas to be inspected with chalk or other non-permanent means. Spray paint will not be used on the structure.
4. Perform close-up visual inspection of all surfaces. Note any surfaces that are not accessible using planned access methods.
5. Mechanically sound all suspect locations (where close-up inspection indicates conditions that may warrant a repair). Mechanically sound representative non-suspect locations to confirm visual indications.
6. Note reinforcing steel corrosion and estimate of section loss where steel is exposed.
7. Document:
    a. Location, type and quantity of observed deterioration, defect, or distress.
    b. Photograph representative and atypical conditions. Document location of photographs.
    c. Confirm structure configuration as shown on inspection field drawings; note areas that differ.
    d. Mark areas where inspection is complete on inspection field drawings.
    e. Field Notes: Plannotate (WJE's in-house electronic field note system using custom tablet software) will be used to document notes, quantities, and photographs. WJE personnel will sync daily all electronic field sheets with the WJE remote server, which is itself backed up daily.
8. At the end of each day, the UBIU will be driven and parked off-site by its driver.

Follow-up Testing and Material Sampling (Phase 2)

Follow-up testing, consisting of non-destructive testing (NDT), inspection openings, and material sampling will be conducted at selected study areas. The primary objective of this phase of the investigation is to identify the relevant causes of distress so that long-lasting repair approaches can be developed.

The selection of study areas for the follow-up testing and material sampling will be performed at the conclusion of the Phase 1 inspection and will be based on element type, existing condition, exposure conditions (e.g. proximity to leaking joints, stream water, etc.), and preliminary assessment of apparent deterioration mechanisms. WJE's understanding of the current bridge conditions will be based on the findings of the in-depth element level bridge inspection and prior inspection reports. The study areas will include representative locations covering the range of conditions observed on each element type of the bridge, from good to more severely deteriorated conditions.

The inspection and testing methods to be employed in this program are outlined in Table 1:

Table 1. Inspection and testing methods.

Item	Method / Description	Purpose
Visual Survey	Visual inspection from UBIU	Characterize and document extent and location of deterioration
Delamination Survey	Hands-on mechanical sounding	Identify delaminations or other deterioration not apparent during visual survey and locations of likely repair (for quantity estimates)
Reinforcement Cover Survey	Ground-penetrating radar (GPR)	Confirm as-built construction (cover and spacing) for structural analysis; support service life modeling
Corrosion Survey: Half-cell Survey	Rolling wheel per ASTM C876	Identify "hot spots" of corrosion activity and areas where future concrete distress is anticipated.
Corrosion Survey: Corrosion Rate	ASTM G59	Assess instantaneous rate of corrosion to support projections of future distress
Corrosion Survey: Resistivity	4-pin Wenner probe	Quantify resistivity of concrete, a controlling factor in corrosion rate and cathodic protection system design
In situ Steel Corrosion Assessment	Visual assessment digital calipers at exposed steel	Determine condition and quantify extent of corrosion-related section loss of reinforcing steel
Reinforcing Bar Sampling	Removal of approx. 3-ft. long	Provide material for laboratory testing
Concrete Sampling	Water-cooled core drilling or hand removal of bulk/chunk samples	Provide material for laboratory testing

The test methods to be employed on the materials samples are outlined in Table 2:

Table 2. Test methods for materials samples.

Item	Method / Description	Purpose
Carbonation Depth	Phenolphthalein pH indicator on all cores	Assess potential for carbonation-related corrosion of reinforcement and support service life modeling
Compressive Strength	ASTM C42	Determine concrete strength to support structural analysis
Chloride Profile	Per ASTM C1152 on slices cut from cores	Assess potential for chloride-related corrosion of reinforcement and support service life modeling
Petrographic Examination	ASTM C856	Assess concrete quality and identify nature and extent of distress mechanisms, including depth of freeze-thaw damage
Steel testing	Tensile and chemistry testing	Asses mechanical properties and chemistry of steel reinforcing bar to support structural modeling

The preliminary testing plan for using these inspection and testing methods is outlined in Table 3 and includes 18 study areas, 15 concrete cores, 17 corrosion surveys, 4 steel samples, 9 chloride profiles, and 5 petrographic examinations. Final quantities may be adjusted based on field conditions.

The follow-up testing and materials sampling will generally be conducted by two WJE personnel. WJE will engage a local coring contractor to perform concrete core sample extraction. Core samples will be taken at most study areas, and this will be supplemented by additional cores at other locations of interest. The location of the cores will be selected based on technical merit of the information to be obtained from the concrete sample and to limit impact on structural performance in coordination with HNTB. The depth of most core samples will be 6 to 12 inches; where warranted, such as at the abutment footings and arch springline, deeper cores may be obtained to investigate the depth of freeze-thaw distress. Inspection openings to obtain samples of reinforcing bar will be made at locations where the impact on structural performance with be limited and will be coordinated with HNTB. Core holes and inspection openings will be repaired with a rapid hardening cementitious repair material.

Following completion of the field inspections, a service life assessment will be performed for the bridge elements. This assessment will consider chloride-induced or carbonation-induced corrosion, as applicable.

Table 3. Preliminary bridge inspection testing plan – anticipated number of study areas and test quantities.

Element	Overall Inspection		Number of Targeted Study Areas[1]					Laboratory Work					Service Life	
	Visual / Element Level Rating	Delamination Survey	Reinforcement Cover Survey[2]	Corrosion Survey[2, 3]	In Situ Steel Corrosion Assessment	Concrete Core Sampling	Steel Sampling	Chloride Profile	Carbonation Depth	Compressive Strength	Petrographic Analysis	Section Loss Quantification[3]	Service Life Estimates	Service Life by Numerical Modeling[3]
Railing	100%	20%	1	-	-	1	-	-	-	-	-	-	Y	-
Deck	100%	100%	4	4	1	3	1	3	3	2	1	-	Y	-
Floor Beams	100%	100%	3	3	1	2	-	2	2	1	1	-	Y	-
Columns	100%	100%	2	2	1	2	-	1	1	1	-	-	Y	-
Tie Beams	100%	100%	1	1	-	-	1	-	-	1	-	-	Y	-
Arches	100%	100%	5	5	2	4	2	2	2	2	2	-	Y	-
Abutments	100%	50%	1	1	-	2	-	1	1	1	1	-	Y	-
Wing Walls	100%	50%	1	1	-	1	-	-	1	1	-	-	Y	-
NW Retaining Wall	100%	0%	-	-	-	-	-	-	-	-	-	-	Y	-
Total	**100%**	**-**	**18**	**17**	**5**	**15**	**4**	**9**	**10**	**9**	**5**	**0**	**9**	**0**

Notes: [1] Each study area will be a minimum of 20 square feet; [2] Corrosion survey includes half-cell potential, corrosion rate, and resistivity testing; [3] Section loss quantification and service life by numerical modeling were removed from the project scope.

TRAFFIC CONTROL PLAN

Maintenance and control of traffic will be provided by the Town of Windsor with assistance from VTrans. The northbound lane of Route 5 will remain open to all traffic and the southbound lane will be closed to all traffic at the bridge during bridge inspection work hours. Portable message boards will be placed on Interstate 91 to alert drivers to the closure with the primary intent to direct truck traffic away from downtown Windsor. Southbound traffic intending to cross the bridge will be directed via signage to detour the bridge by turning right (west) onto Union Street to Route 44/Ascutney Street and back to Route 5; it is anticipated that local non-truck traffic may use southbound Maple Street to avoid the longer detour.

Signage within the general town limits will be provided by the Town and/or by VTrans and deployed by the Town. Signage alerting drivers to the upcoming closure will be placed in advance of the inspections so that the public can plan accordingly. VTrans will provide signage warning drivers of workers on the bridge

to be deployed by the Town during the inspections. The Town will provide and place sufficient cones along the centerline of the roadway and at the ends of the bridge to demarcate and prevent traffic from entering the lane closure. The Town will close the sidewalk across the bridge. A police cruiser provided by the Town will be parked within the lane closure to provide additional protection to the UBIU and workers. WJE's rental vehicle may also be parked within the closure.

The lane closure will begin at 9:00 AM and end at 4:30 PM each workday. The majority of WJE's work will be performed from the southbound lane closure, but approximately 4 hours will be required in the northbound lane. Northbound lane work will not require the UBIU, and WJE anticipates traffic control will consist of the Town temporarily moving the cones and signage to shift northbound traffic to the southbound lane to complete this work.

INSPECTION TEAM

WJE personnel and their roles for the field inspection are listed below.

- Arne Johnson – Project Manager
  - Mobile phone: (847) 508-1516
  - Email: ajohnson@wje.com
- Tanner Swenson – Lead Inspector
  - Mobile phone: (763) 486-3662
  - Email: tswenson@wje.com
- Sam Sherry – In-Depth Element Level Inspection (Phase 1) Inspector
  - Mobile phone: (972) 837-5796
  - Email: ssherry@wje.com
- Stephen Garrett – Follow-Up Testing and Materials Sampling (Phase 2) Inspector
  - Mobile phone: (708) 707-2132
  - Email: sgarrett@wje.com
- Jason Kamman – Safety Coordinator
  - Mobile phone: (317) 691-5303
  - Email: jkamman@wje.com

Primary contact personnel for other involved parties are as follows:

- HNTB
  - Rich Tetreault
    - Mobile phone: (802) 522-0918
    - Email: rtetreault@hntb.com
  - Angela Kingsley
    - Mobile phone: (651) 895-4278
    - Email: akingsley@hntb.com
- VTrans
  - Laura Stone
    - Mobile phone: (802) 917-4996

    - Email: laura.stone@vermont.gov
- Town of Windsor
    - Police Chief – Jennifer Frank
        - Mobile phone: (802) 398-2677
        - Email: Jennifer.Frank@vermont.gov
    - Fire Chief – Kevin McAllister
        - Mobile phone: (802) 585-9893
        - Email: kmcallister@windsorvt.org
    - Town Manager – Tom Marsh (tmarsh@windsorvt.org)
- UBIU Rental Sub
    - McClain & Co., Inc.
        - Phone: (888) 868-2118
        - Email: Gayle McDonald (gmcdonald@mcclain1.com)

SAFETY

An inspection team safety meeting will be held at the beginning of each work shift and will be led by the Lead Inspector. Work site hazards and equipment logistics for the shift will be discussed. All inspection work will be performed in accordance with governing state and federal safety laws and with general WJE's safety policies. Hazards on this site will include, but are not limited to, trips/falls, traffic, aerial lifts, equipment, climbing access methods, above water work, and inclement weather.

OSHA Part 1926.106 requires employees to wear life jackets and that ring buoys and a lifesaving skiff be available when working over or near water. In a letter of interpretation dated September 28, 1999, OSHA clarifies that a life jacket is not required if continuous fall protection is used, as will be the case while tied-off to the UBIU basket. A ring buoy will be available, but a lifesaving skiff is not feasible given the shallow and rocky streambed below the bridge. Life jackets will be worn by WJE employees while working from ground near the waterline.

If needed, the following facility has been identified as the closest medical center for non-emergency medical needs:

Mt. Ascutney Hospital and Health Center

289 County Road

Windsor, VT 05089

Phone: (802) 674-6711

From the north end of the bridge:

1. Head north on Main Street/Route 5 for approximately 1/3 mile.
2. Turn left on State Street and proceed approximately 3/4 mile.
3. Turn right on County Road and proceed approximately 1/3 mile.
4. Destination is on left.

APPENDIX A. UBIU DATA SHEETS

ASPEN A-62

Bridge Inspection Unit

New Technologies Lead the Industry

Design..... Through extensive research utilizing customer input, FEA engineering and repetitive motion testing, the Aspen A-62 was designed from the ground up resulting in greater versatility, simplicity and reliability under all types of conditions. This model has been extended to access areas over, around and under some of your widest bridges to provide you with the pinnacle of "no-compromise" performance and efficiency.



Maneuverability..... Two rotating turntables plus multiple articulating and telescoping booms allow the 40"x 60" rotating platform to deploy off of either side of the truck to access all of your structures. The Aspen A-62 is truly in a league of its own.

Stabilization..... Outriggers are not required and all counterweights stay within the width of the truck body. The redesigned sliding counterweight is installed under the truck bed and operates to either side while the Aspen A-62 is deployed to the opposite side. In the transport mode, the counterweight stays in the center of the truck for improved road handling. A torsion-box subframe and hydraulic axle locks unitize the chassis and truck axles, allowing the vehicle to travel while the unit is fully deployed.

Reliable

Adjustable Turntable



Turntable no. 2 comes equipped with an automatic leveling system, providing smooth platform movement. In addition, the operator can make manual adjustments of +/- 5 degrees to compensate for the slope of the bridge. With the elimination of the leveling arms, the Aspen A-62 has more vertical clearance to get over high fences.

Innovative

Telescoping & Rotating Platform



- 180 degree rotating platform
- Telescoping 4th boom
- Over 13' vertical reach

Durable

Advanced Control System

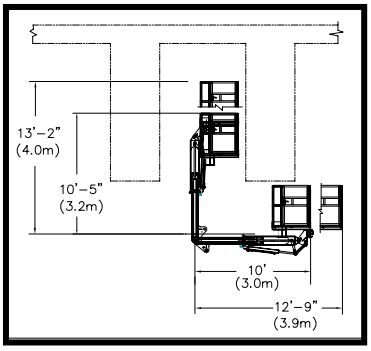


As the most advanced microprocessor control system available, the "Plus 1" Graphical Terminal Interface includes a color display depicting unit operation and individual function performance while monitoring the unit's parameters, hydraulics and enabling simple troubleshooting. Wireless controls are included at both operator stations.

Platform Features

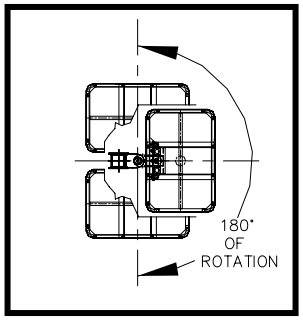
Telescoping 4th boom

The Aspen A-62 has the capability of providing inspectors with over 13' (4m) of vertical reach. This fully hydraulic feature will enable you to get up and behind your deepest girders for a close-up inspection.



Rotating Platform

The added flexibility of a 180 degree rotating platform makes maneuvering into working positions easy, increasing productivity and efficiency.

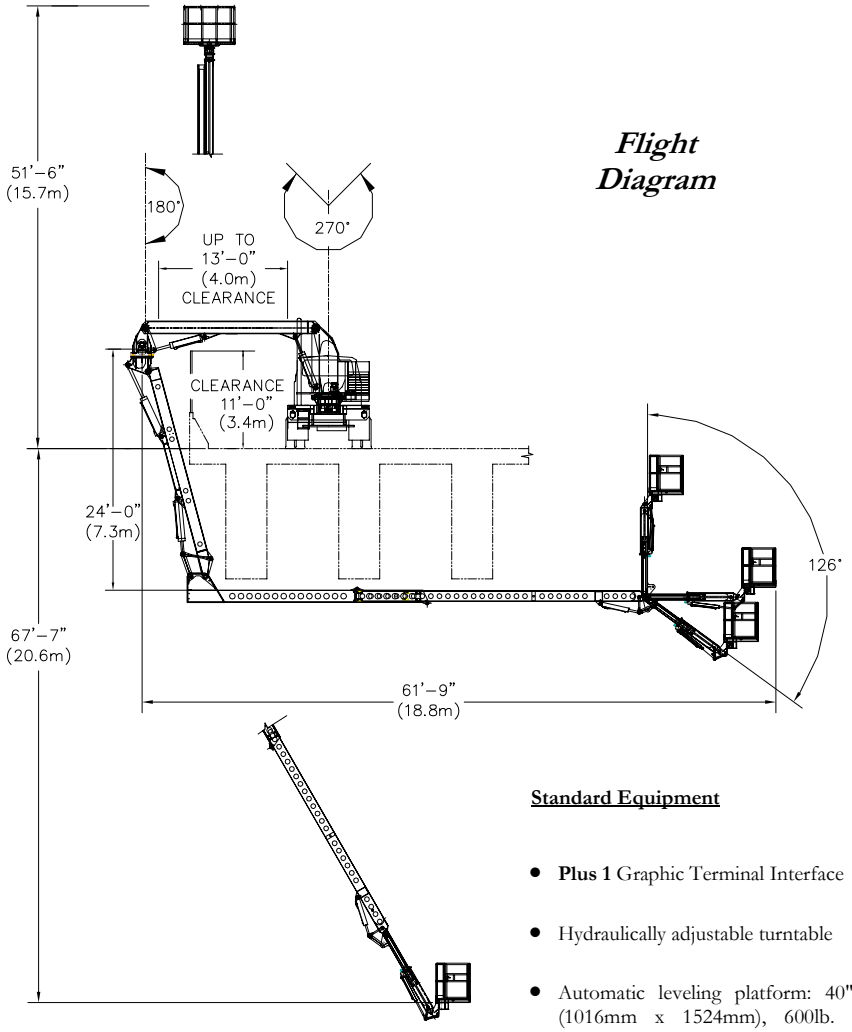


Aspen Aerials, Inc.
4303 West 1st Street
Duluth, MN 55807

Phone: 218-624-1111
Toll Free: 800-888-2773
Fax: 218-624-1714
Web site: www.aspenaerials.com

ASPEN AERIALS

Specifications



Flight Diagram

Horizontal Underbridge Reach	**61'-9" / 18.8m**
Vertical Reach Down	**67'-7" / 20.6m**
Vertical Reach Up	**51'-6" / 15.7m**
Boom no. 1 movement	**+30 to - 35 degrees**
Boom no. 2 movement	**+ 0 to -105 degrees**
Boom no. 3 movement	**+90 to - 60 degrees**
Boom no. 4 movement	**+90 to - 36 degrees**
Space Required on Bridge	**102" / 2.5m**
Basket Capacity	**600 lbs. / 272 kg.**
Basket Size	**40" x 60" x 42" 1010mm x 1520mm**
Overall Length	**40' / 12.2m ***
Overall Height	**13'-3" / 4.0m ***

**** May vary depending on chassis***

Available Options:

- Air Compressor and air line in platform
- Generator with electrical outlet in platform
- Platform Heaters & Floodlights
- Remote Controls

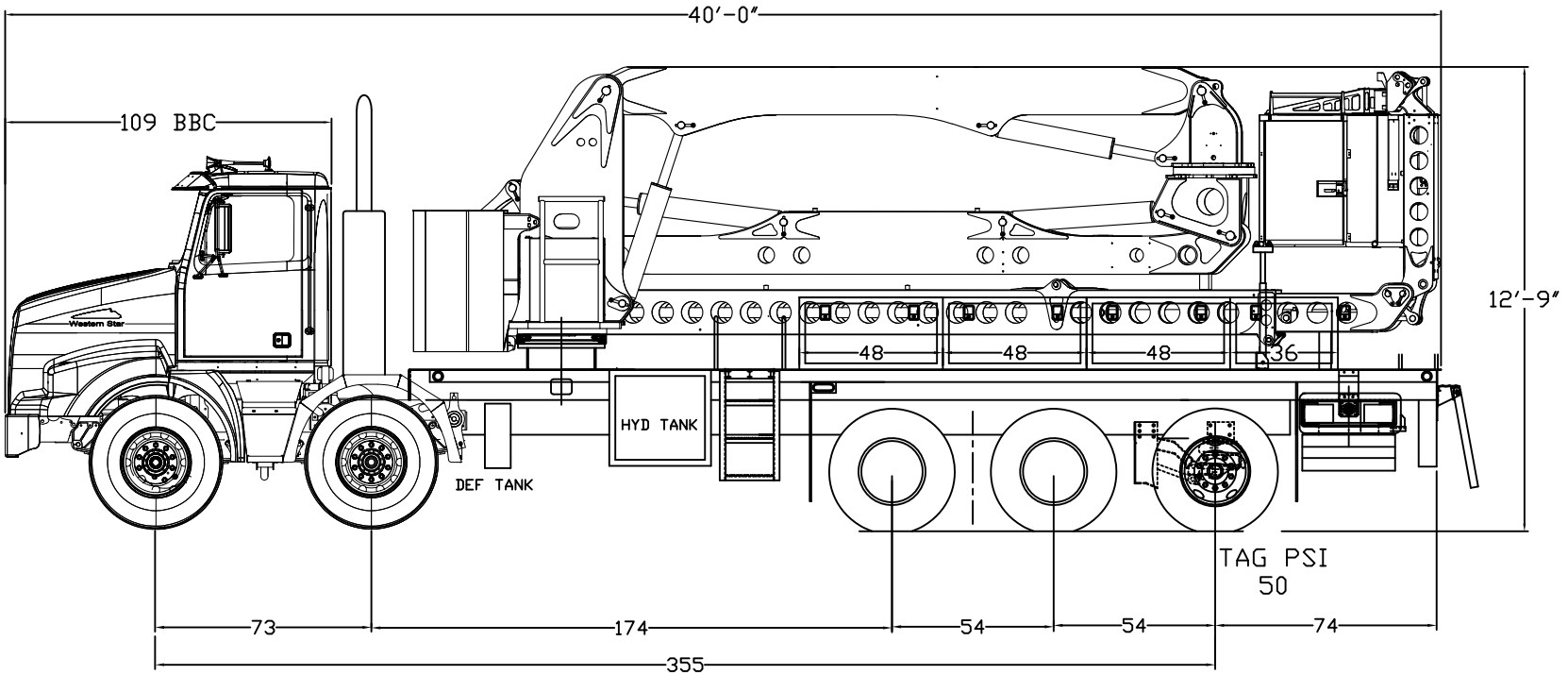
Standard Equipment

- **Plus 1** Graphic Terminal Interface
- Hydraulically adjustable turntable
- Automatic leveling platform: 40" x 60" (1016mm x 1524mm), 600lb. (272kg.) capacity w/access gate
- 2 sets of controls: one set in the platform and one set at the pedestal
- 4 articulating booms (3rd & 4th booms telescope); 2 rotating turrets, rotating platform
- Stability interlocks with monitoring system
- Two power systems: power take-off from the truck's transmission and a back-up diesel power unit
- 12-volt intercom system between platform,



Various chassis configurations available

VEHICLE: 2019 WESTERN STAR 4800
VIN: 5KKSBBDV6KPKV9962
ASPEN S/N : 10327



FRONT STEER	SECOND STEER	TANDEM AXLE	TAG AXLE
WEIGHT	WEIGHT	WEIGHT	WEIGHT
13,260	13,020	30,720	9,000
TIRE	TIRE	TIRE	TIRE
(2) 385/65R22.5	(2) 385/65R22.5	(8) 11R22.5H	(2) 11R22.5H

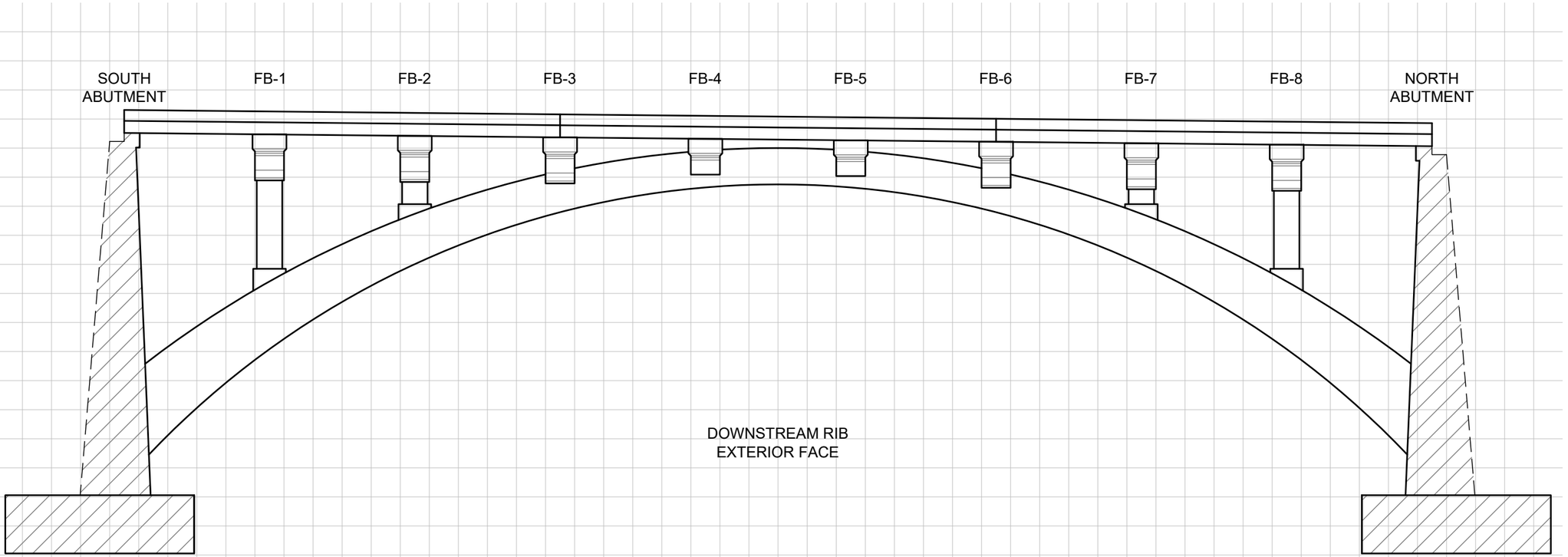
TOTA WEIGHT
66,000 LB

INDIVIDUAL AXLE WEIGHTS MAY VARY +/- 2% OF FIGURE SHOWN

DO NOT SCALE THIS DRAWING
NEW RELEASE AUTHORIZATION
REVISIONS

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APPENDIX B. EXAMPLE BRIDGE INSPECTION FORMS



DOWNSTREAM RIB
EXTERIOR FACE



DOWNSTREAM RIB
TOPSIDE

	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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Windsor, VT 05089

Client
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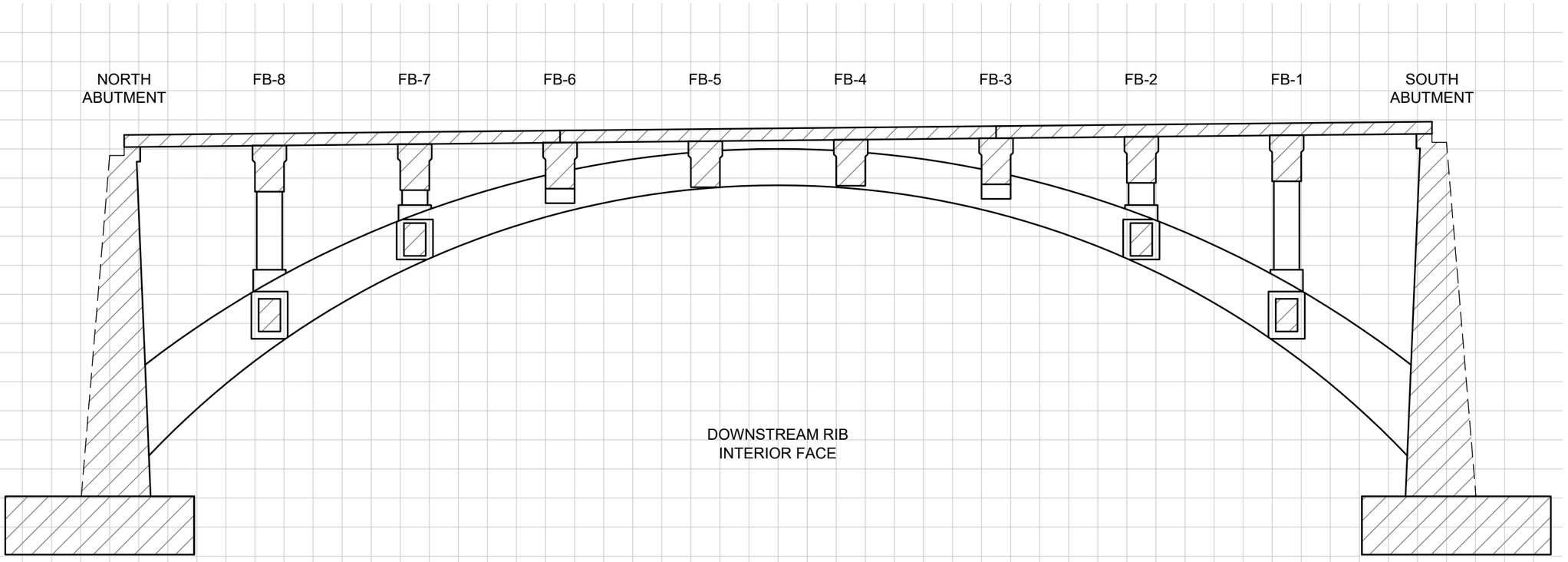
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Date	Value
Drawn	TWS
Checked	APJ
Scale	1/8" = 1'-0"

DS Arch Rib Field Sheet
Sheet Title

Sheet No. **01**



DOWNSTREAM RIB
INTERIOR FACE



DOWNSTREAM RIB
UNDERSIDE (REFLECTED)

	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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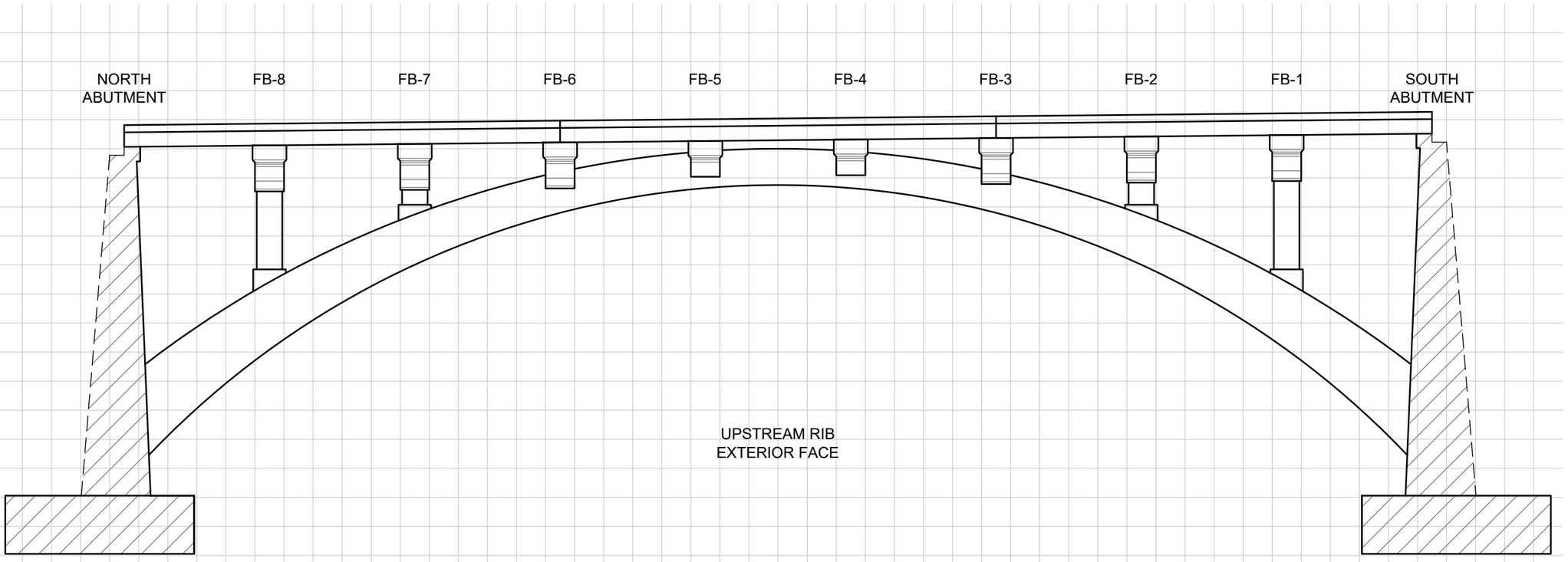
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Checked	APJ
Scale	1/8" = 1'-0"

Sheet Title
DS Arch Rib Field Sheet

Sheet No.
02



UPSTREAM RIB
EXTERIOR FACE



UPSTREAM RIB
TOPSIDE

	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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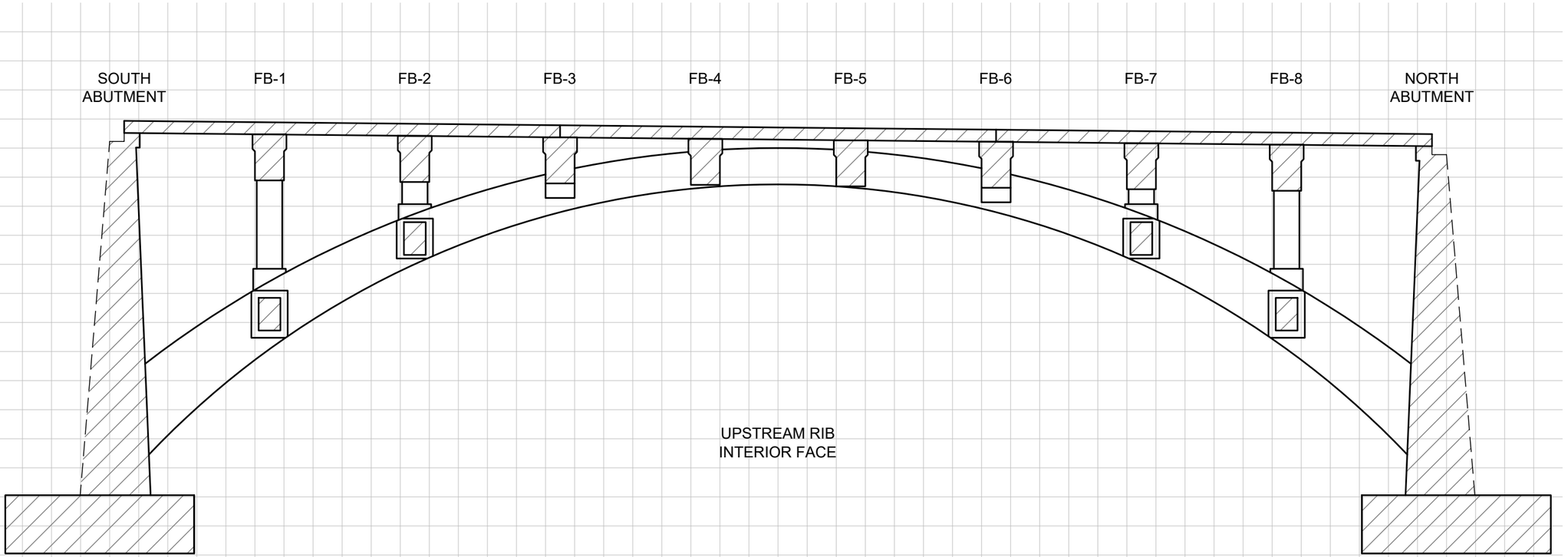
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Scale	1/8" = 1'-0"

Sheet Title
US Arch Rib Field Sheet

Sheet No.
03





UPSTREAM RIB
UNDERSIDE (REFLECTED)

	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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Mark	Date	Description

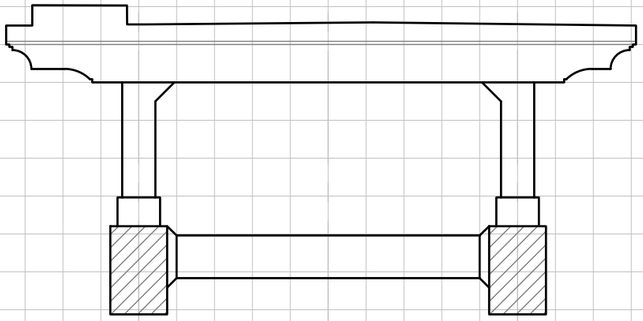
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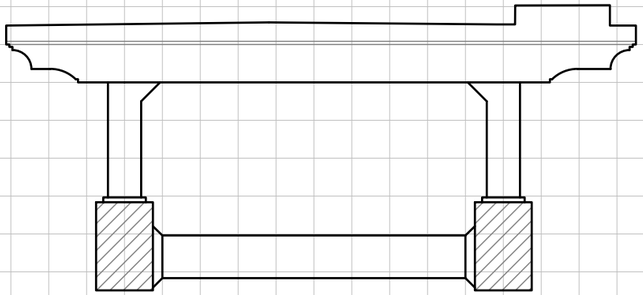
US Arch Rib Field Sheet
Sheet Title

Sheet No. **04**

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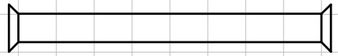
FB-1
SOUTH ELEVATION



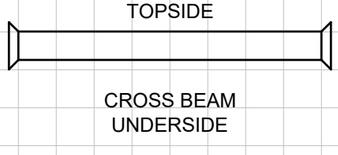
FB-1 NORTH
FACE ELEVATION



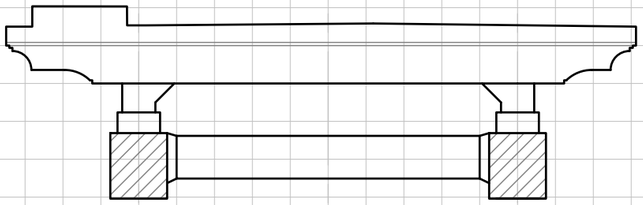
FB-1 UNDERSIDE



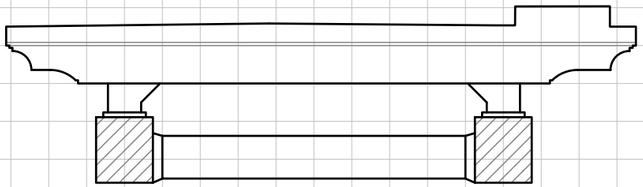
CROSS BEAM
TOPSIDE



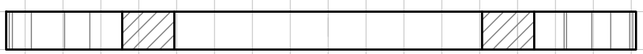
CROSS BEAM
UNDERSIDE



FB-2
SOUTH ELEVATION



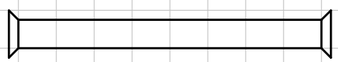
FB-2
NORTH ELEVATION



FB-2 UNDERSIDE



CROSS BEAM
TOPSIDE



CROSS BEAM
UNDERSIDE

	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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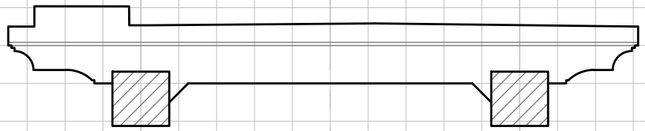
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FB-1-2 Elev Field Sheet
Sheet Title

Sheet No. **05**

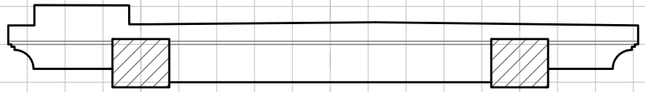
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FB-3
SOUTH ELEVATION



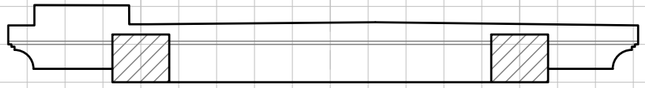
FB-3 UNDERSIDE



FB-4
SOUTH ELEVATION



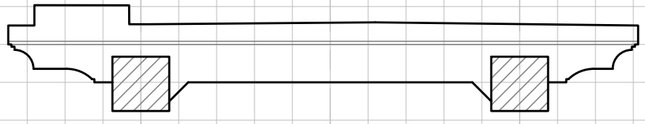
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FB-5
SOUTH ELEVATION



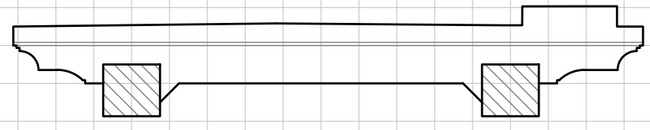
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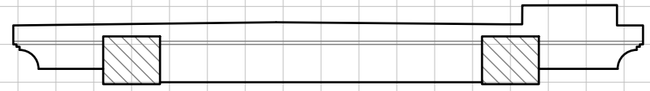
FB-6
SOUTH ELEVATION



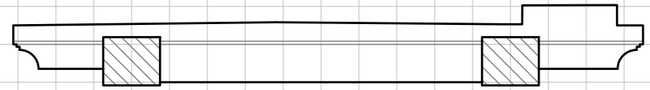
FB-6 UNDERSIDE



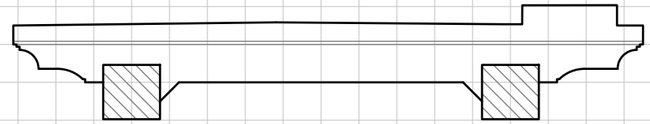
FB-3
NORTH ELEVATION



FB-4
NORTH ELEVATION



FB-5
NORTH ELEVATION



FB-6
NORTH ELEVATION

	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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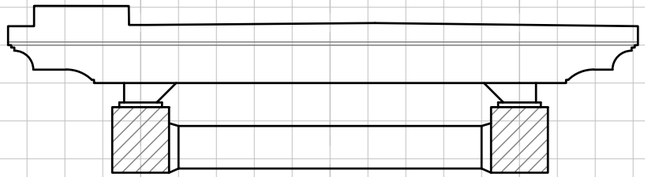
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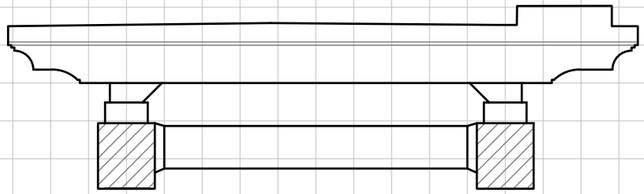
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Date	Value
Drawn	TWS
Checked	APJ
Scale	1/8" = 1'-0"

Sheet Title
FB-3-6 Elev Field Sheet

Sheet No.
06



FB-7
SOUTH ELEVATION



FB-7
NORTH ELEVATION



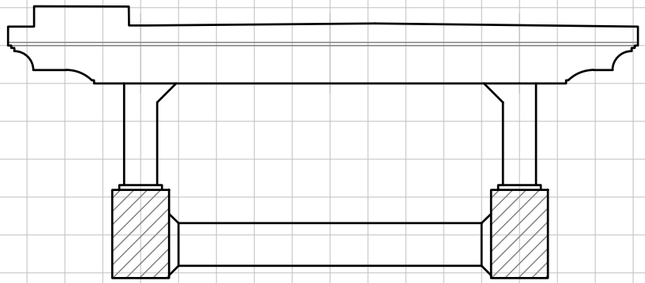
FB-7 UNDERSIDE



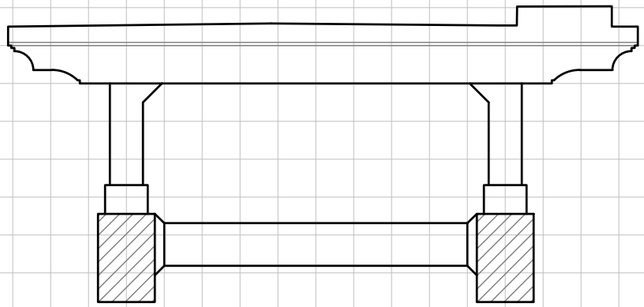
CROSS BEAM
TOPSIDE



CROSS BEAM
UNDERSIDE



FB-8
SOUTH ELEVATION



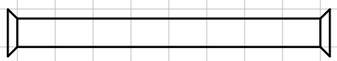
FB-8
NORTH ELEVATION



FB-8 UNDERSIDE



CROSS BEAM
TOPSIDE



CROSS BEAM
UNDERSIDE

	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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Consultants

Project
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Windsor, VT 05089

Client
HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

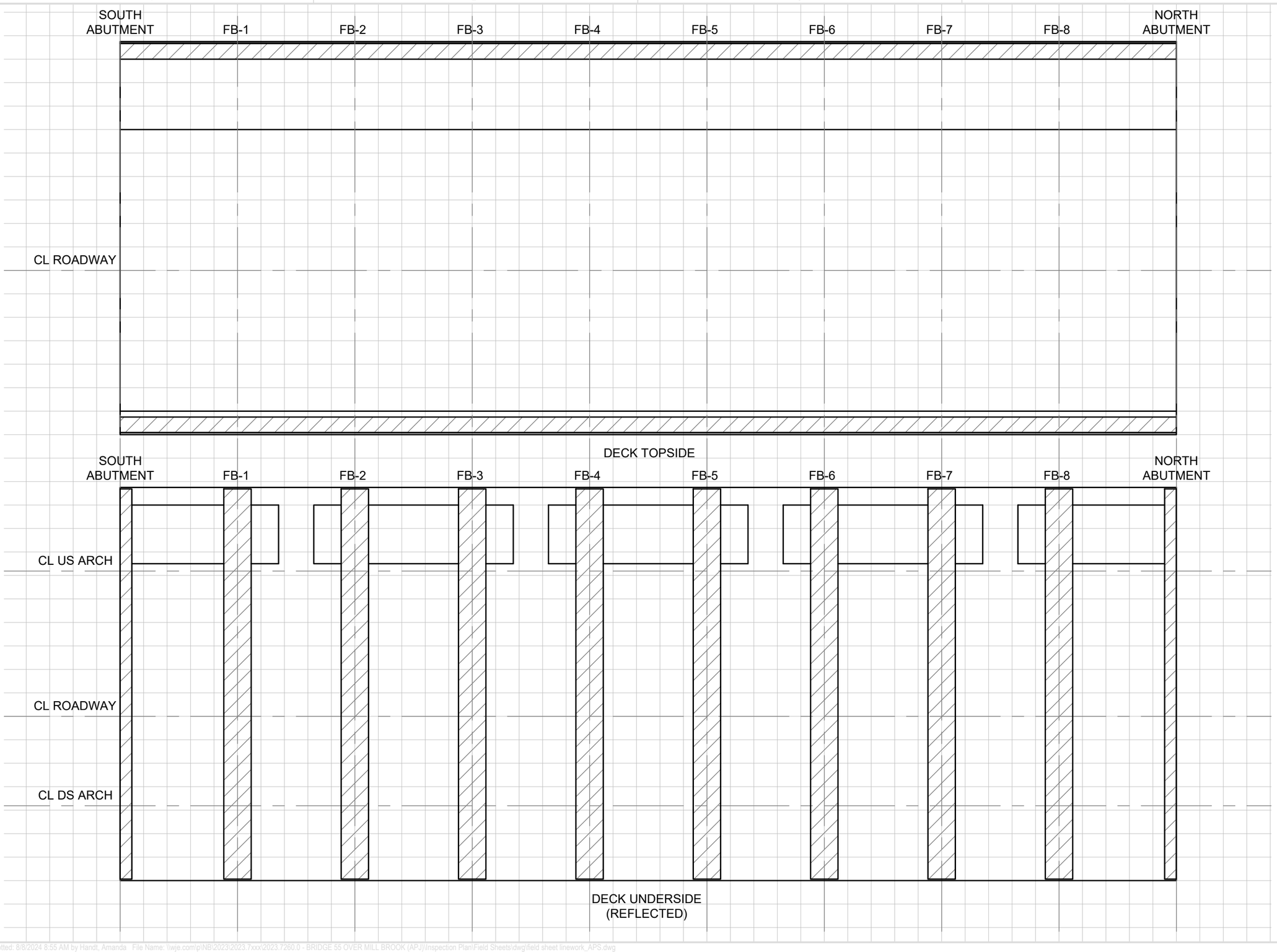
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Date Value
Drawn TWS
Checked APJ
Scale 1/8" = 1'-0"

Sheet Title
FB-7-8 Elev Field Sheet

Sheet No.
07



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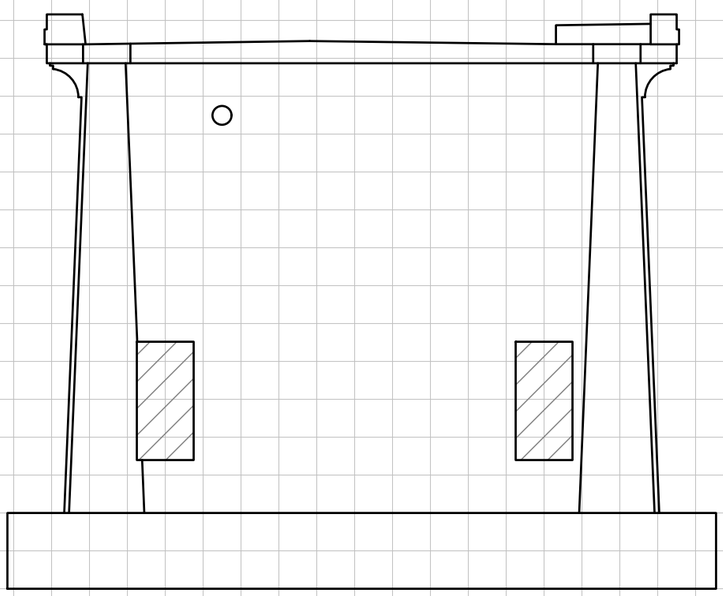
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Checked	APJ
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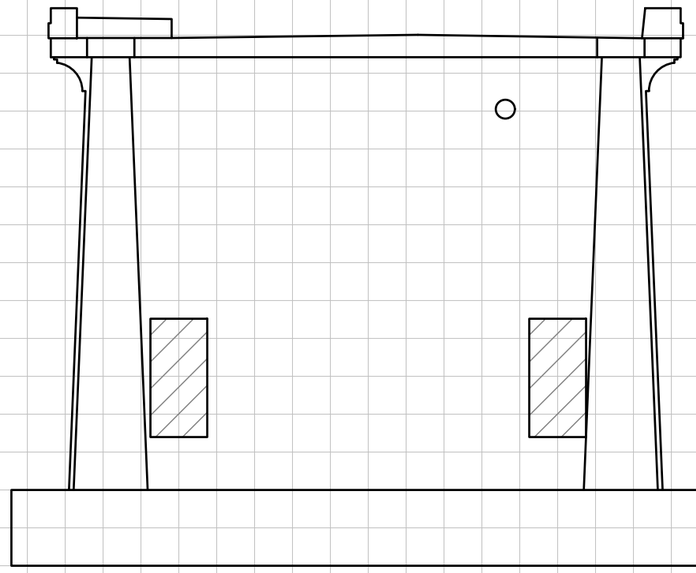
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Sheet No.: **08**

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SOUTH ABUTMENT



NORTH ABUTMENT

	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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Mark	Date	Description

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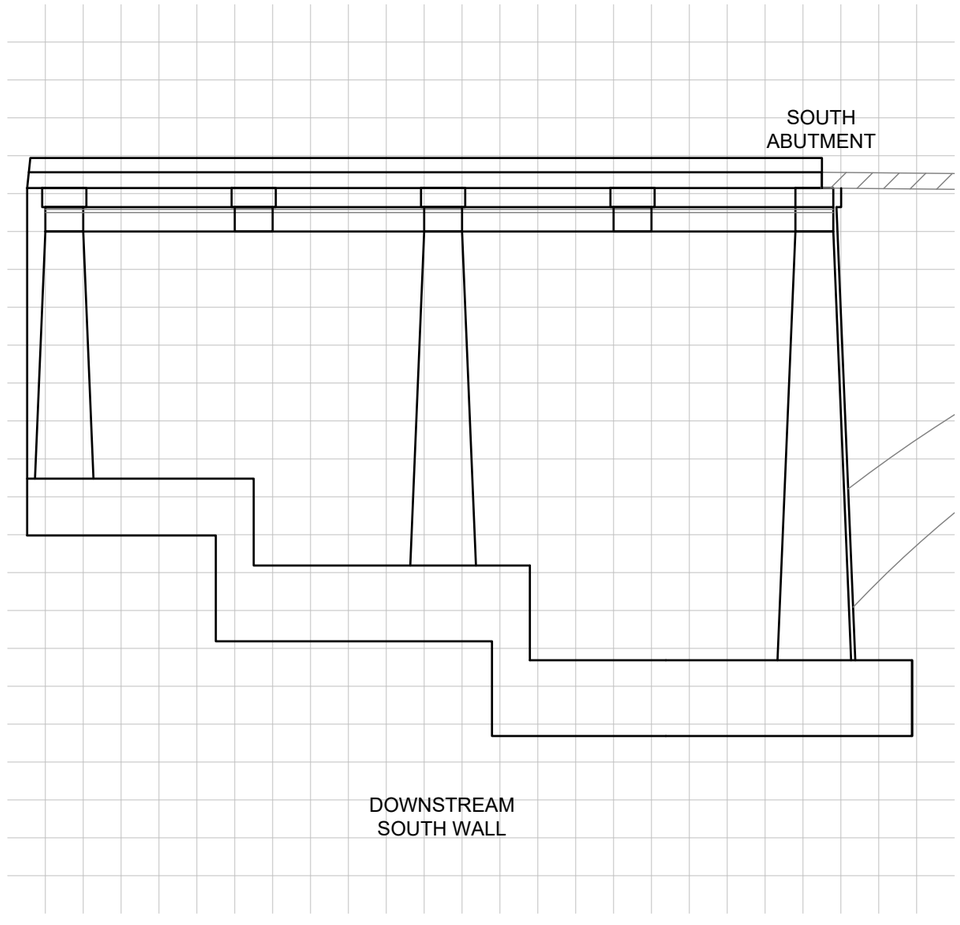
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Sheet Title
Abut Elev Field Sheet

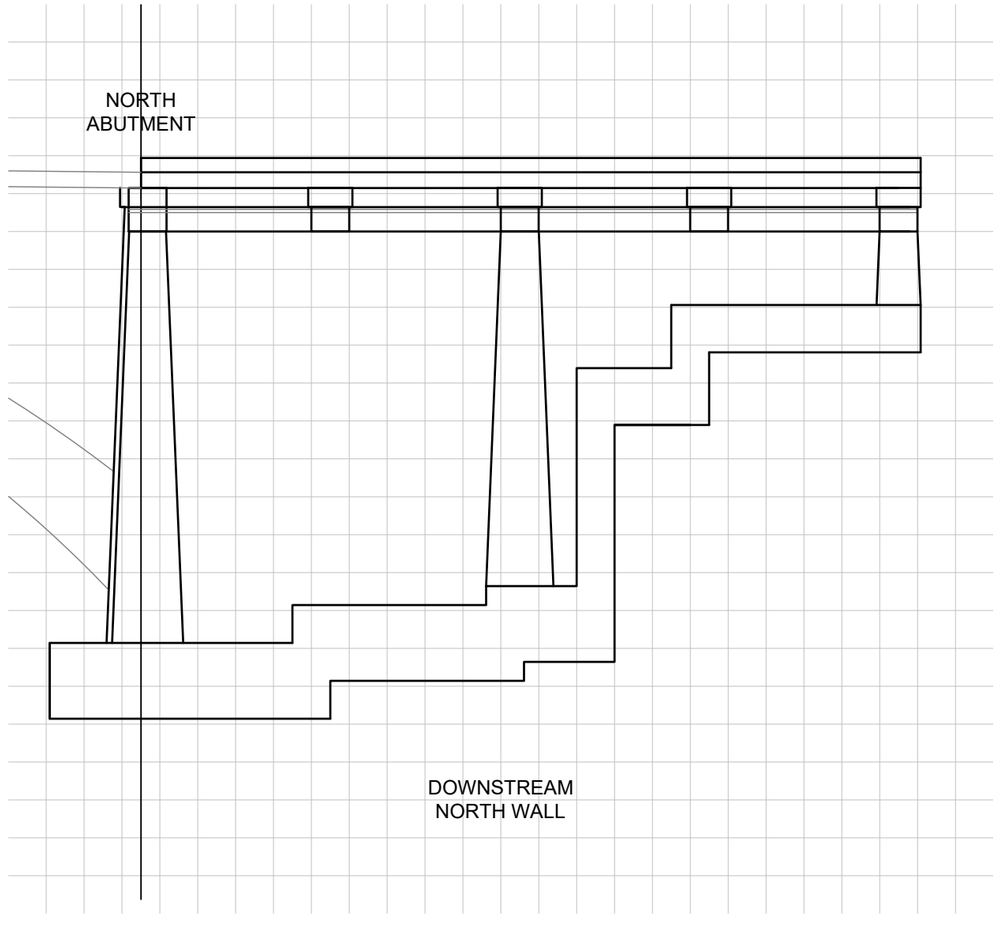
Sheet No.
09

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DOWNSTREAM
SOUTH WALL



DOWNSTREAM
NORTH WALL

	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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Mark	Date	Description

0 1/2" 1"
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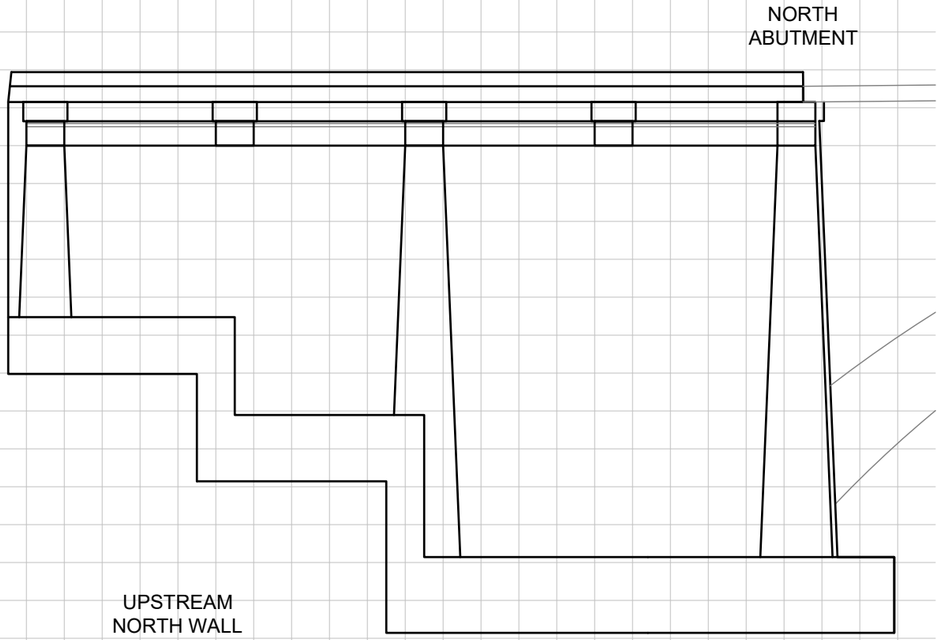
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Sheet Title
DS Walls Field Sheet

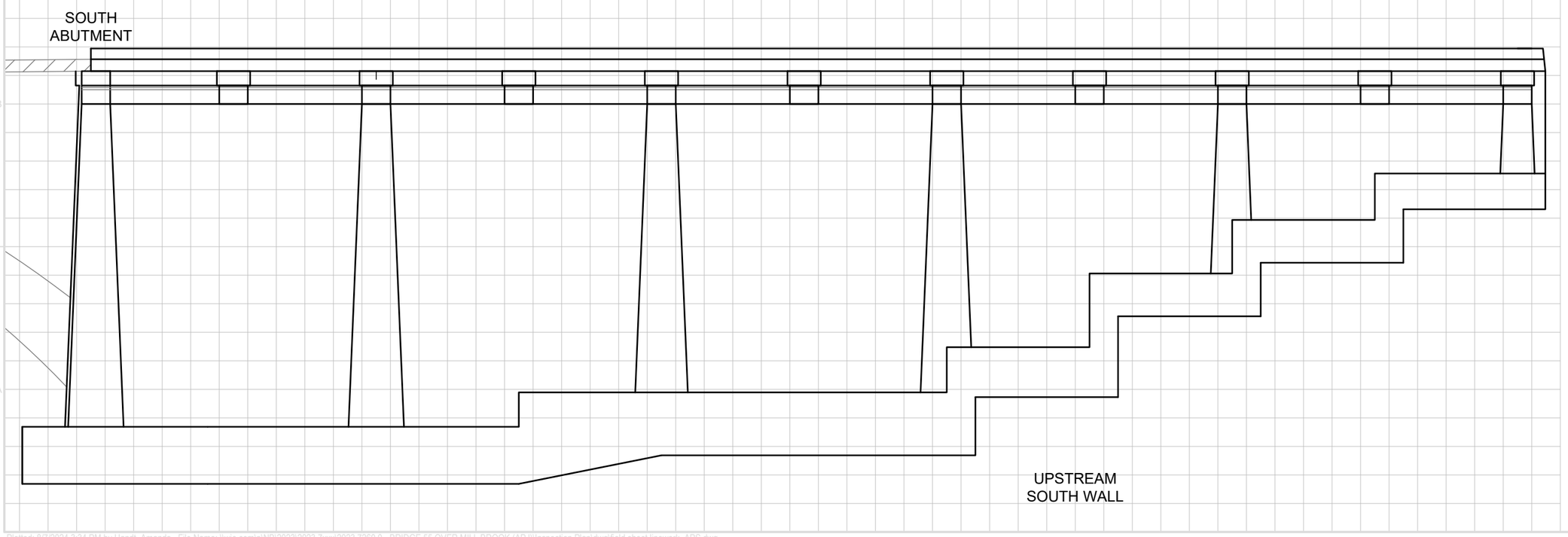
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	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								



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0 1/2" 1"
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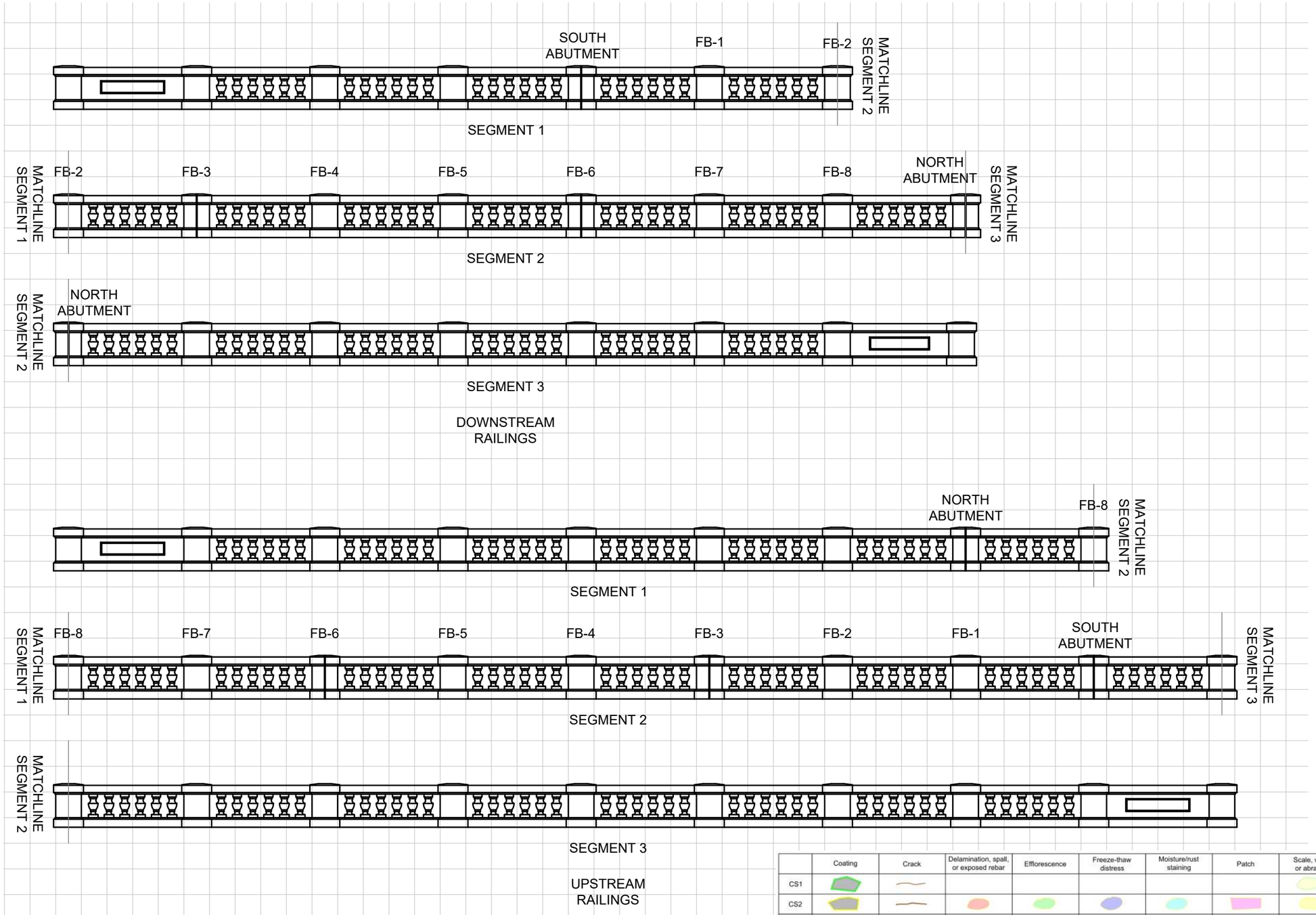
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Date	Value
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Checked	APJ
Scale	1/8" = 1'-0"

Sheet Title: US Walls Field Sheet

Sheet No.: 11

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	Coating	Crack	Delamination, spall, or exposed rebar	Efflorescence	Freeze-thaw distress	Moisture/rust staining	Patch	Scale, wear, or abrasion
CS1								
CS2								
CS3								
CS4								

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Mark	Date	Description

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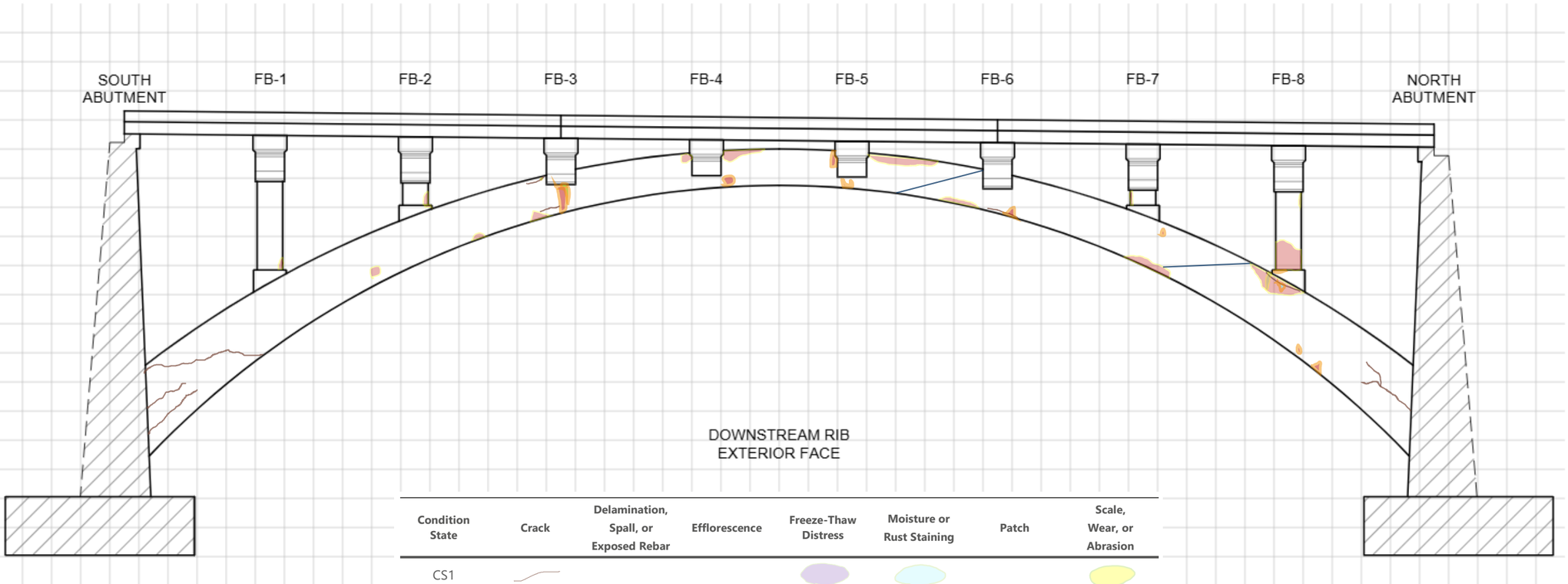
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US Railing Field Sheet
Sheet Title

Sheet No. 12

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APPENDIX B. PHASE 1 FIELD SURVEY DOCUMENTATION (PLANNOTATE SHEETS)



DOWNSTREAM RIB
EXTERIOR FACE

Condition State	Crack	Delamination, Spall, or Exposed Rebar	Efflorescence	Freeze-Thaw Distress	Moisture or Rust Staining	Patch	Scale, Wear, or Abrasion
CS1							
CS2							
CS3							
CS4							



DOWNSTREAM RIB
TOPSIDE

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Mark	Date	Description

0 1/2" 1"
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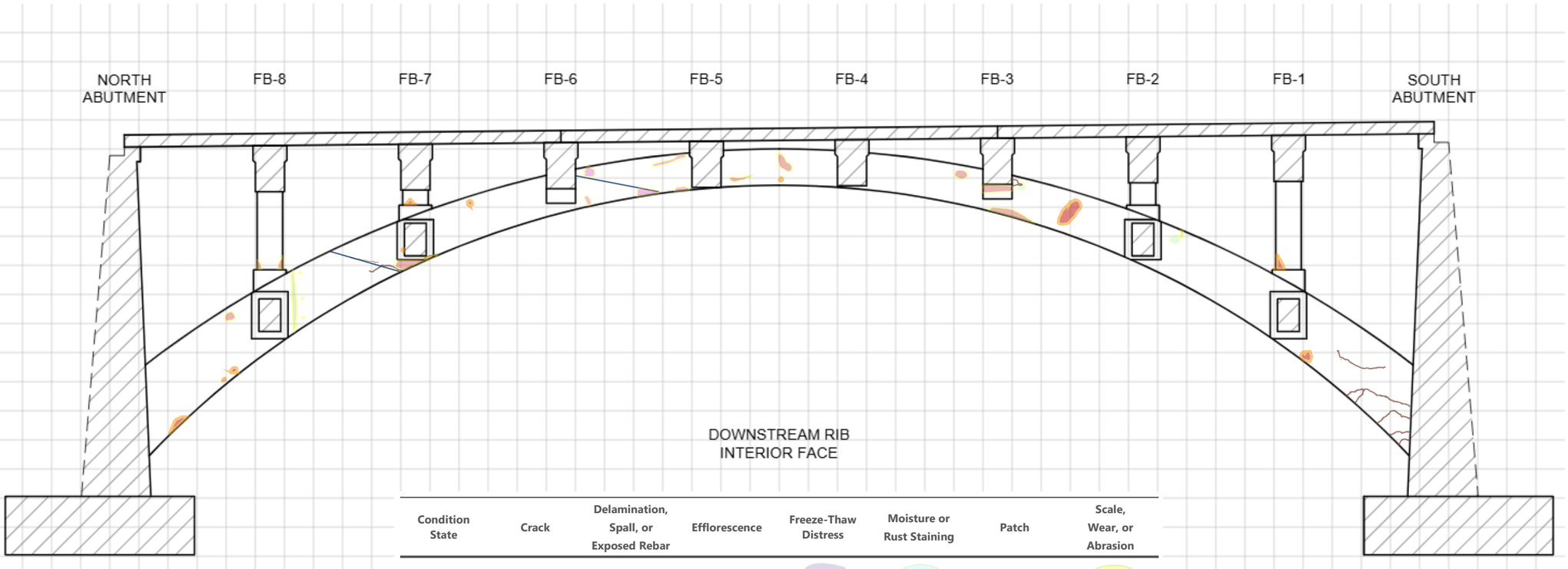
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Date	Value
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Checked	APJ
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DS Arch Rib Field Sheet
Sheet Title

Sheet No. 01

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DOWNSTREAM RIB
INTERIOR FACE

Condition State	Crack	Delamination, Spall, or Exposed Rebar	Efflorescence	Freeze-Thaw Distress	Moisture or Rust Staining	Patch	Scale, Wear, or Abrasion
CS1							
CS2							
CS3							
CS4							



DOWNSTREAM RIB
UNDERSIDE (REFLECTED)

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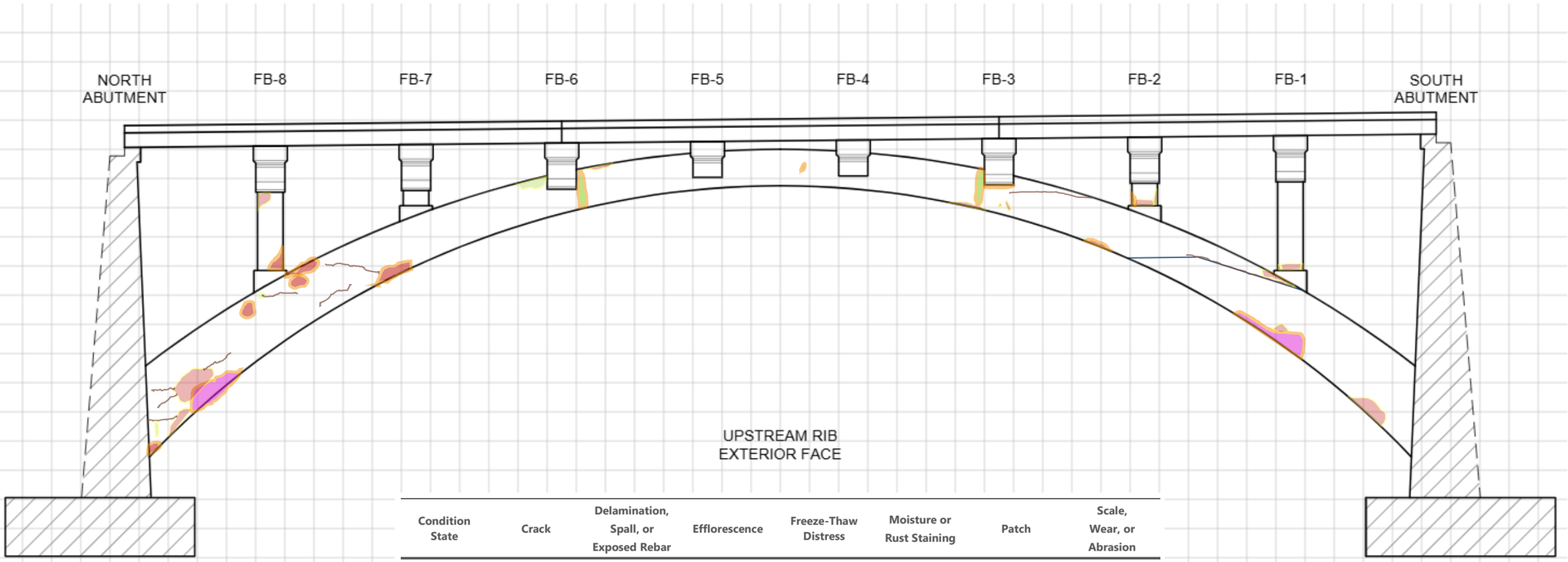
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DS Arch Rib Field Sheet
Sheet Title

Sheet No. 02

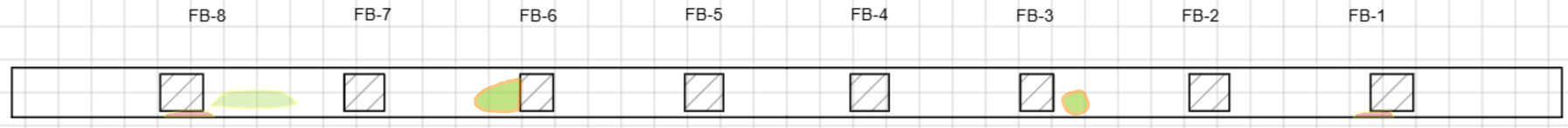
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UPSTREAM RIB
EXTERIOR FACE

Condition State	Crack	Delamination, Spall, or Exposed Rebar	Efflorescence	Freeze-Thaw Distress	Moisture or Rust Staining	Patch	Scale, Wear, or Abrasion
CS1							
CS2							
CS3							
CS4							



UPSTREAM RIB
TOPSIDE

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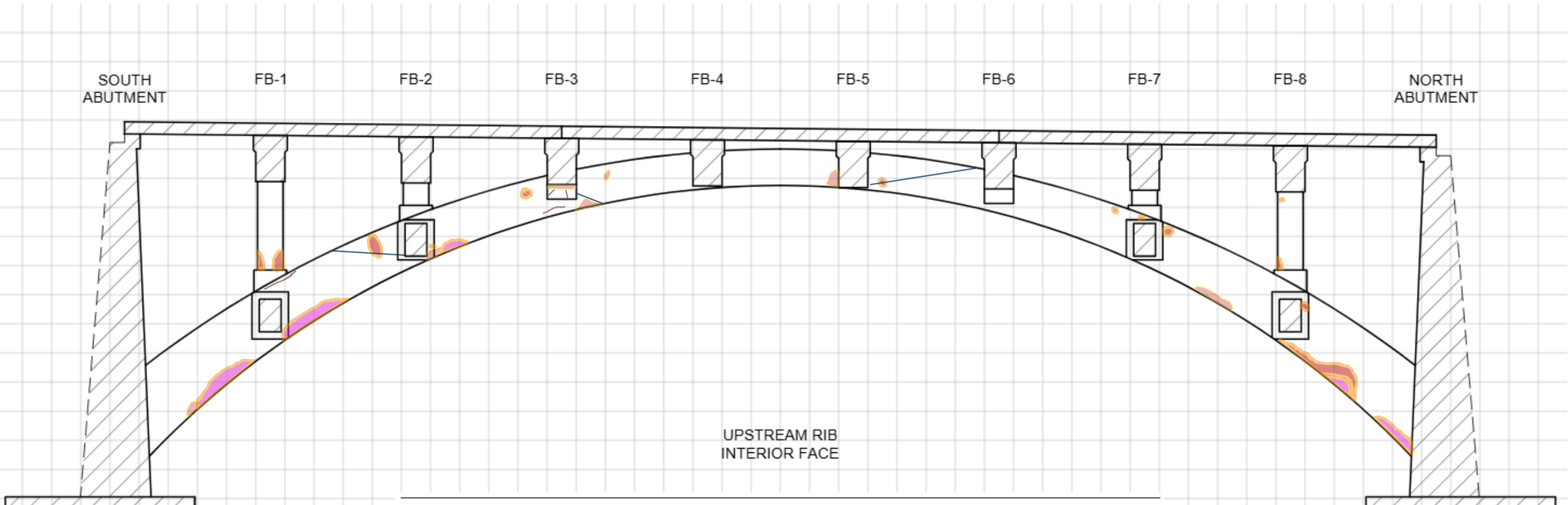
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Checked	APJ
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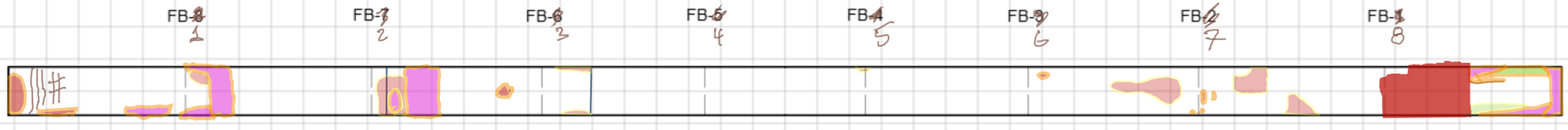
US Arch Rib Field Sheet
Sheet Title

Sheet No. 03



UPSTREAM RIB
INTERIOR FACE

Condition State	Crack	Delamination, Spall, or Exposed Rebar	Efflorescence	Freeze-Thaw Distress	Moisture or Rust Staining	Patch	Scale, Wear, or Abrasion
CS1							
CS2							
CS3							
CS4							



UPSTREAM RIB
UNDERSIDE (REFLECTED)

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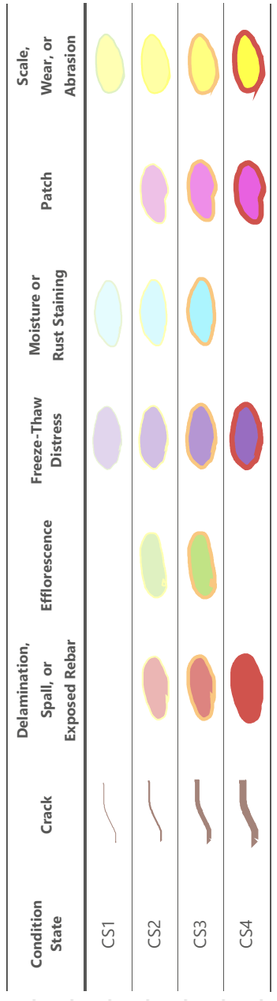
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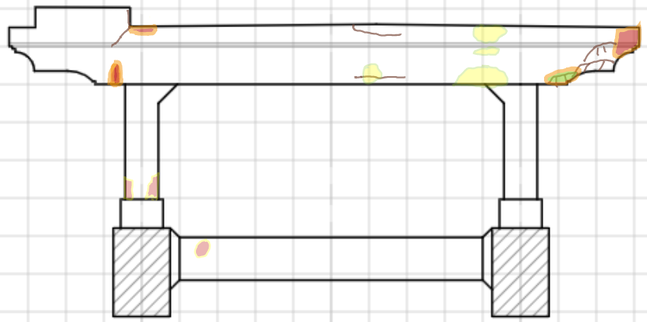
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Checked	APJ
Scale	1/8" = 1'-0"

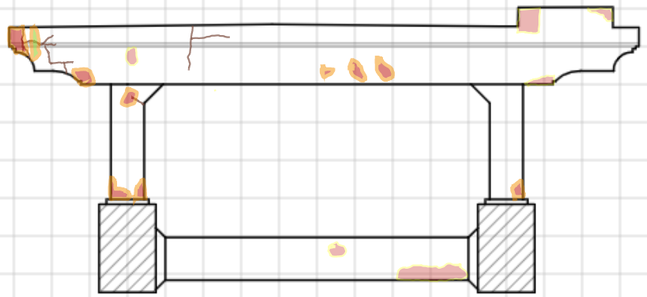
US Arch Rib Field Sheet
Sheet Title

Sheet No. 04





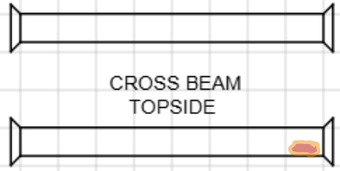
FB-1
SOUTH ELEVATION



FB-1 NORTH
FACE ELEVATION

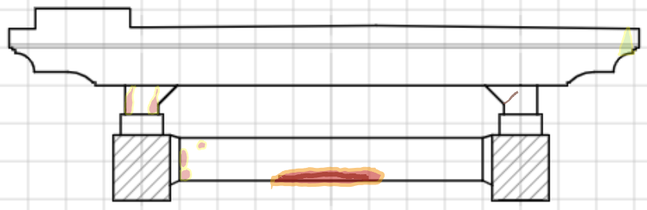


FB-1 UNDERSIDE

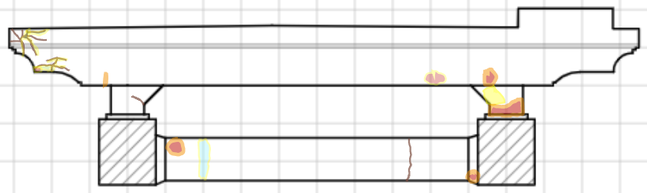


CROSS BEAM
TOPSIDE

CROSS BEAM
UNDERSIDE



FB-2
SOUTH ELEVATION



FB-2
NORTH ELEVATION



FB-2 UNDERSIDE



CROSS BEAM
TOPSIDE



CROSS BEAM
UNDERSIDE

US

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Consultants

Project
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client
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3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

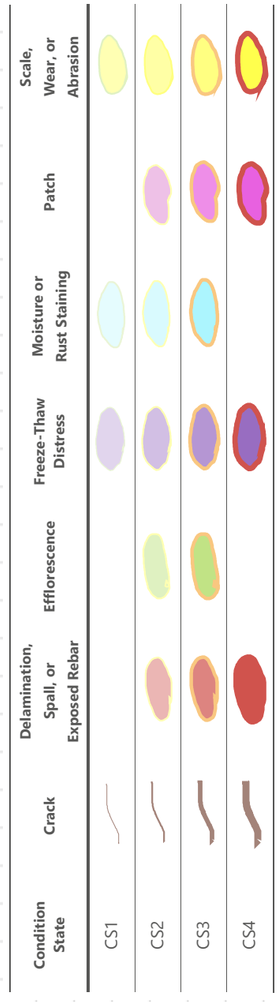
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Date	Value
Drawn	TWS
Checked	APJ
Scale	1/8" = 1'-0"

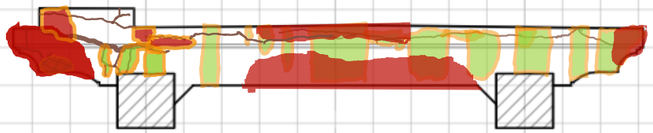
FB-1-2 Elev Field Sheet
Sheet Title

Sheet No. 05

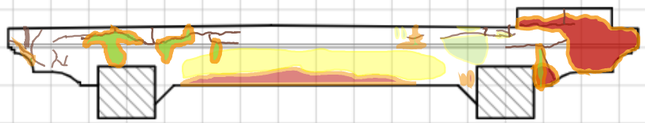
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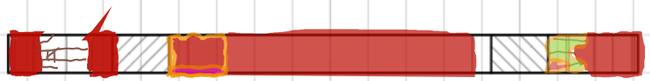




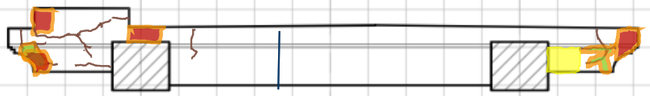
FB-3
SOUTH ELEVATION



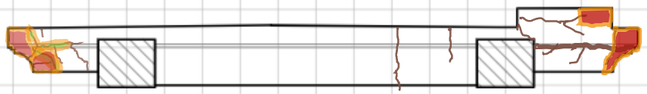
FB-3
NORTH ELEVATION



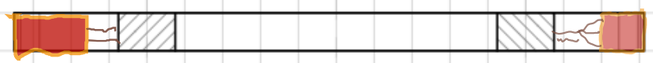
FB-3 UNDERSIDE



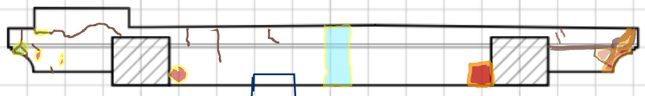
FB-4
SOUTH ELEVATION



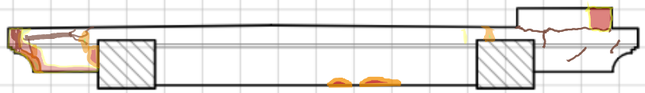
FB-4
NORTH ELEVATION



FB-4 UNDERSIDE



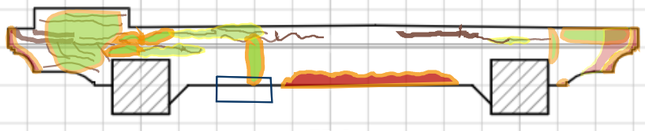
FB-5
SOUTH ELEVATION



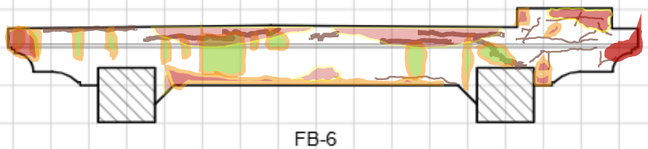
FB-5
NORTH ELEVATION



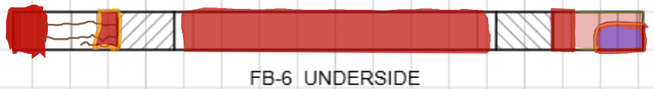
FB-5 UNDERSIDE



FB-6
SOUTH ELEVATION



FB-6
NORTH ELEVATION



FB-6 UNDERSIDE

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Mark	Date	Description

0 1/2" 1"
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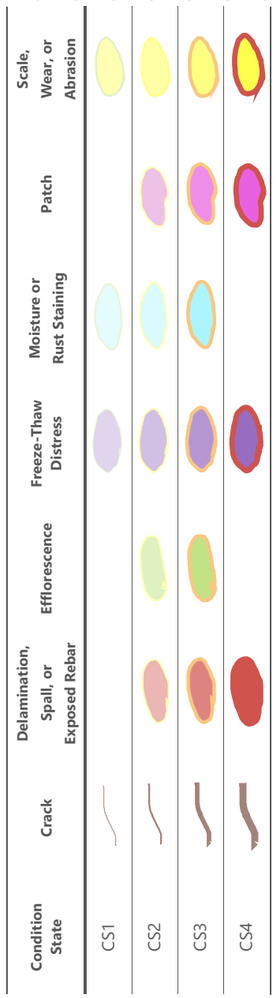
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Date	Value
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Checked	APJ
Scale	1/8" = 1'-0"

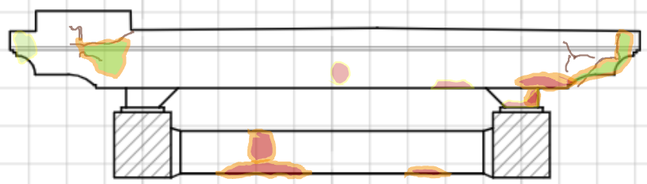
FB-3-6 Elev Field Sheet
Sheet Title

Sheet No. **06**

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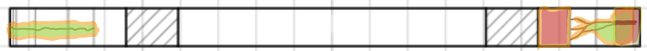




FB-7
SOUTH ELEVATION



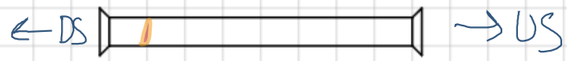
FB-7
NORTH ELEVATION



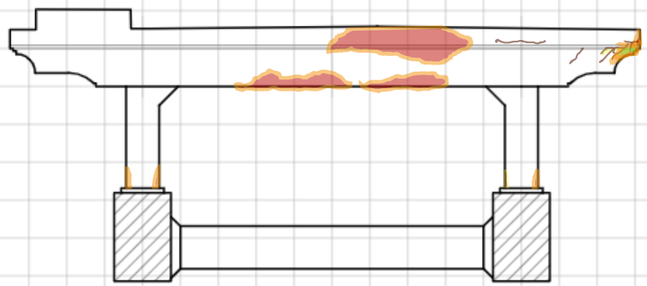
FB-7 UNDERSIDE



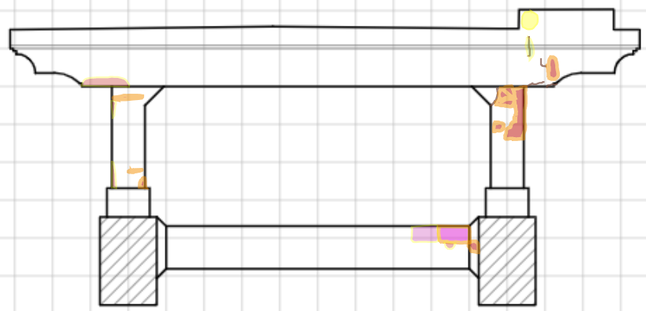
CROSS BEAM
TOPSIDE



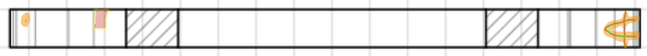
CROSS BEAM
UNDERSIDE



FB-8
SOUTH ELEVATION



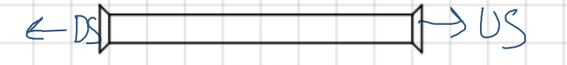
FB-8
NORTH ELEVATION



FB-8 UNDERSIDE



CROSS BEAM
TOPSIDE



CROSS BEAM
UNDERSIDE

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Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

Project No.	2023.7260
Date	Value
Drawn	TWS
Checked	APJ
Scale	1/8" = 1'-0"

FB-7-8 Elev Field Sheet
Sheet Title

Sheet No. **07**

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SOUTH ABUTMENT | FB-1 | FB-2 | FB-3 | FB-4 | FB-5 | FB-6 | FB-7 | FB-8 | NORTH ABUTMENT

CL ROADWAY

DECK TOPSIDE

SOUTH ABUTMENT | FB-1 | FB-2 | FB-3 | FB-4 | FB-5 | FB-6 | FB-7 | FB-8 | NORTH ABUTMENT

CL US ARCH

CL ROADWAY

CL DS ARCH

DECK UNDERSIDE (REFLECTED)

Condition State: CS1 CS2 CS3 CS4

Crack

Delamination, Spall, or Exposed Rebar

Efflorescence

Freeze-Thaw Distress

Moisture or Rust Staining

Patch

Scale, Wear, or Abrasion

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r Mill Brook
T 05089

poration
Park Drive
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Mark	Date	Description

0 1/2" 1"

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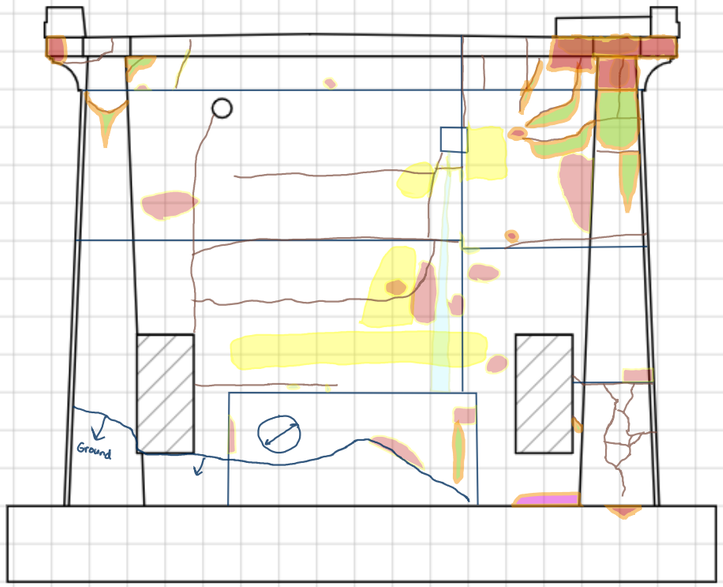
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Checked	APJ
Scale	1/8" = 1'-0"

Sheet Title: Deck Field Sheet

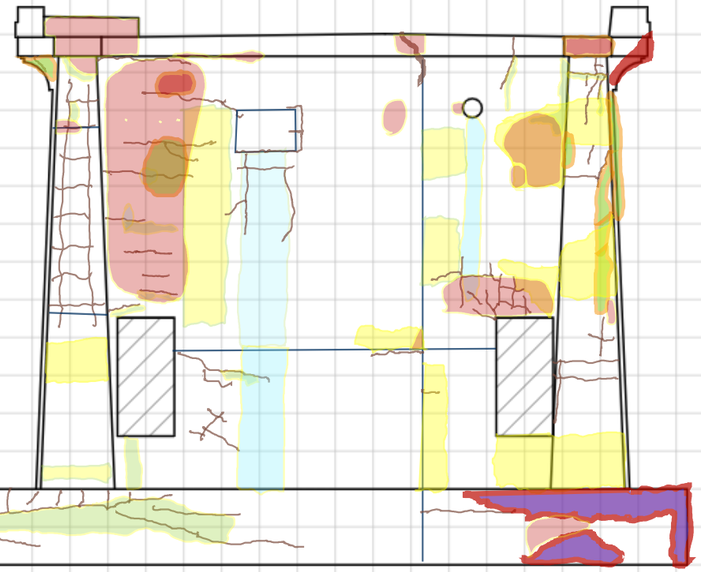
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SOUTH ABUTMENT



NORTH ABUTMENT

Condition State	Crack	Delamination, Spall, or Exposed Rebar	Efflorescence	Freeze-Thaw Distress	Moisture or Rust Staining	Patch	Scale, Wear, or Abrasion
CS1							
CS2							
CS3							
CS4							

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Mark	Date	Description

0 1/2" 1"
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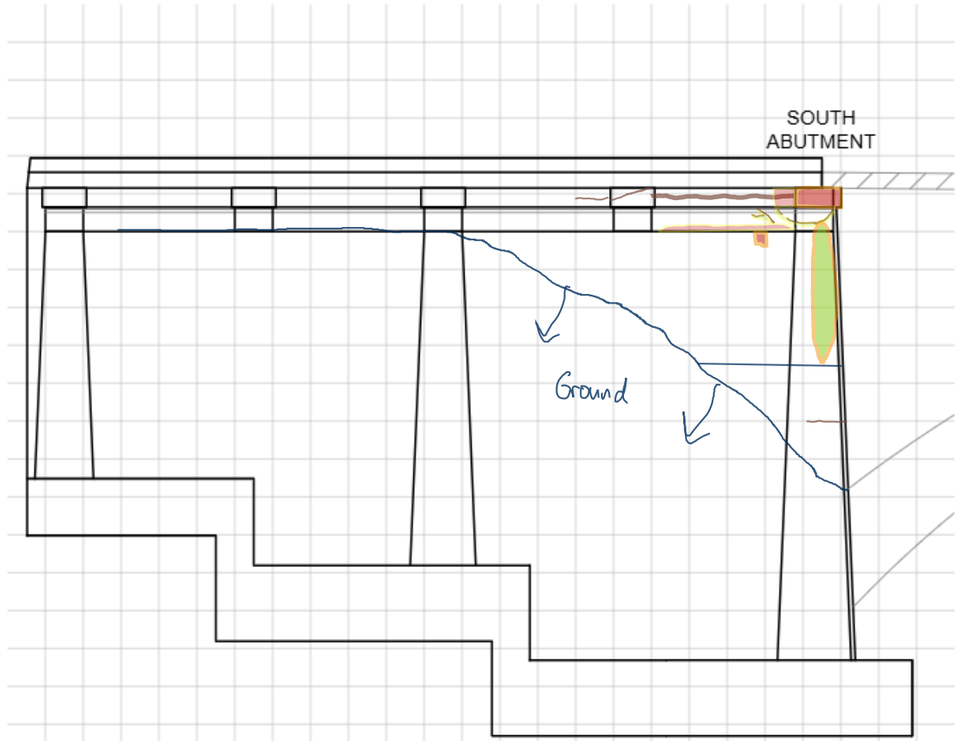
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Sheet Title
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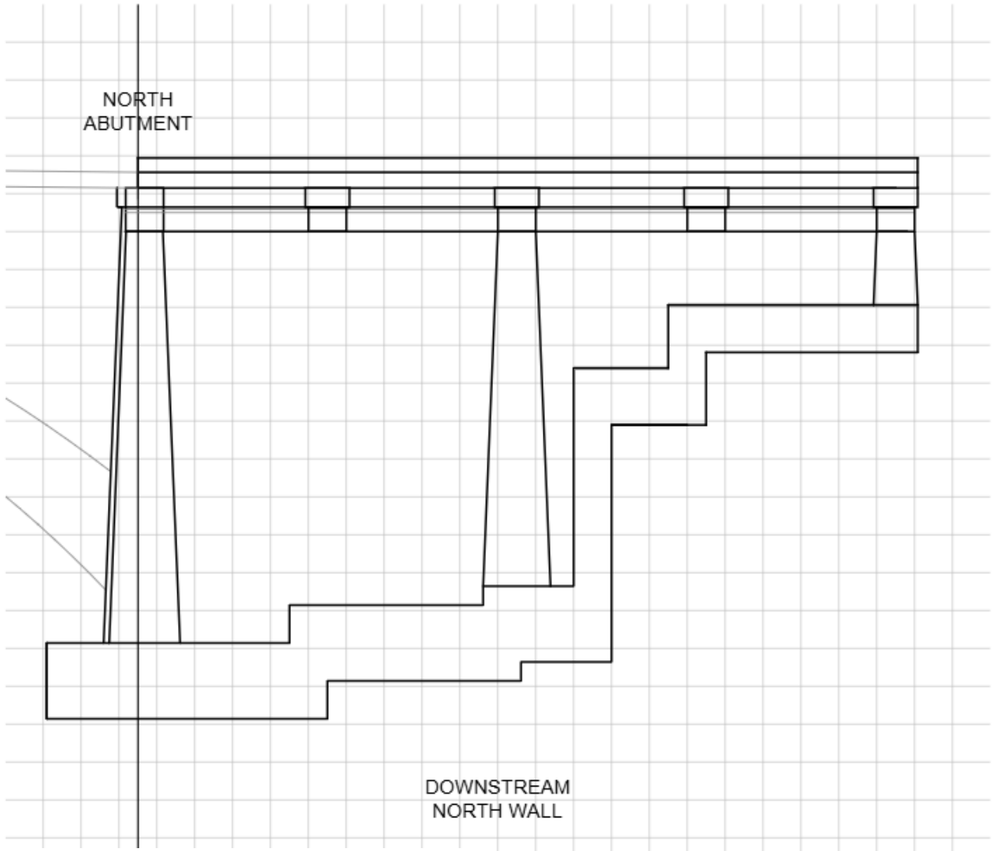
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09

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DOWNSTREAM
SOUTH WALL



DOWNSTREAM
NORTH WALL

Condition State	Crack	Delamination, Spall, or Exposed Rebar	Efflorescence	Freeze-Thaw Distress	Moisture or Rust Staining	Patch	Scale, Wear, or Abrasion
CS1							
CS2							
CS3							
CS4							

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0 1/2" 1"
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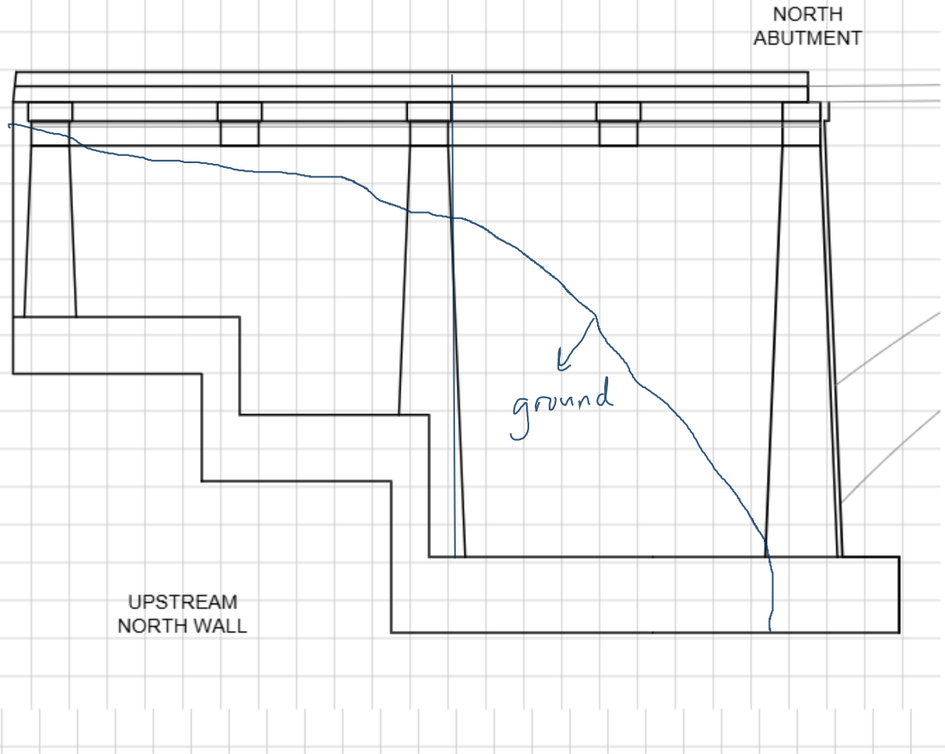
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Sheet Title
DS Walls Field Sheet

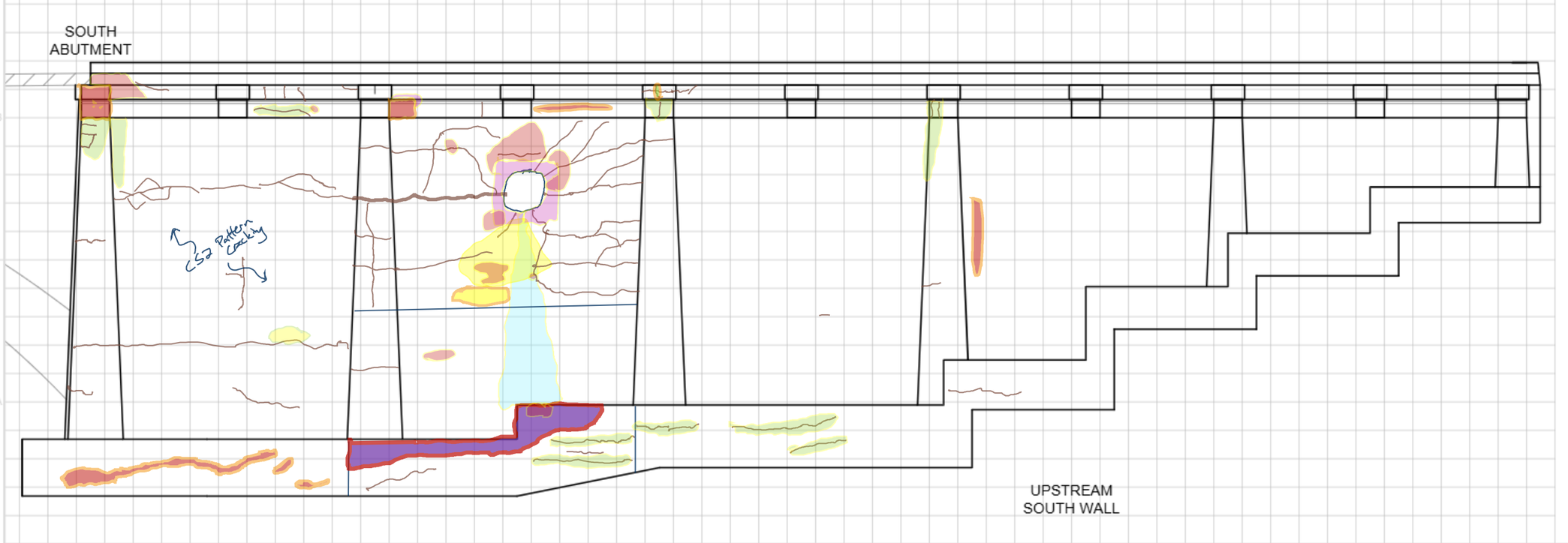
Sheet No. **10**

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Condition State	Crack	Delamination, Spall, or Exposed Rebar	Efflorescence	Freeze-Thaw Distress	Moisture or Rust Staining	Patch	Scale, Wear, or Abrasion
CS1							
CS2							
CS3							
CS4							



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Mark	Date	Description

0 1/2" 1"
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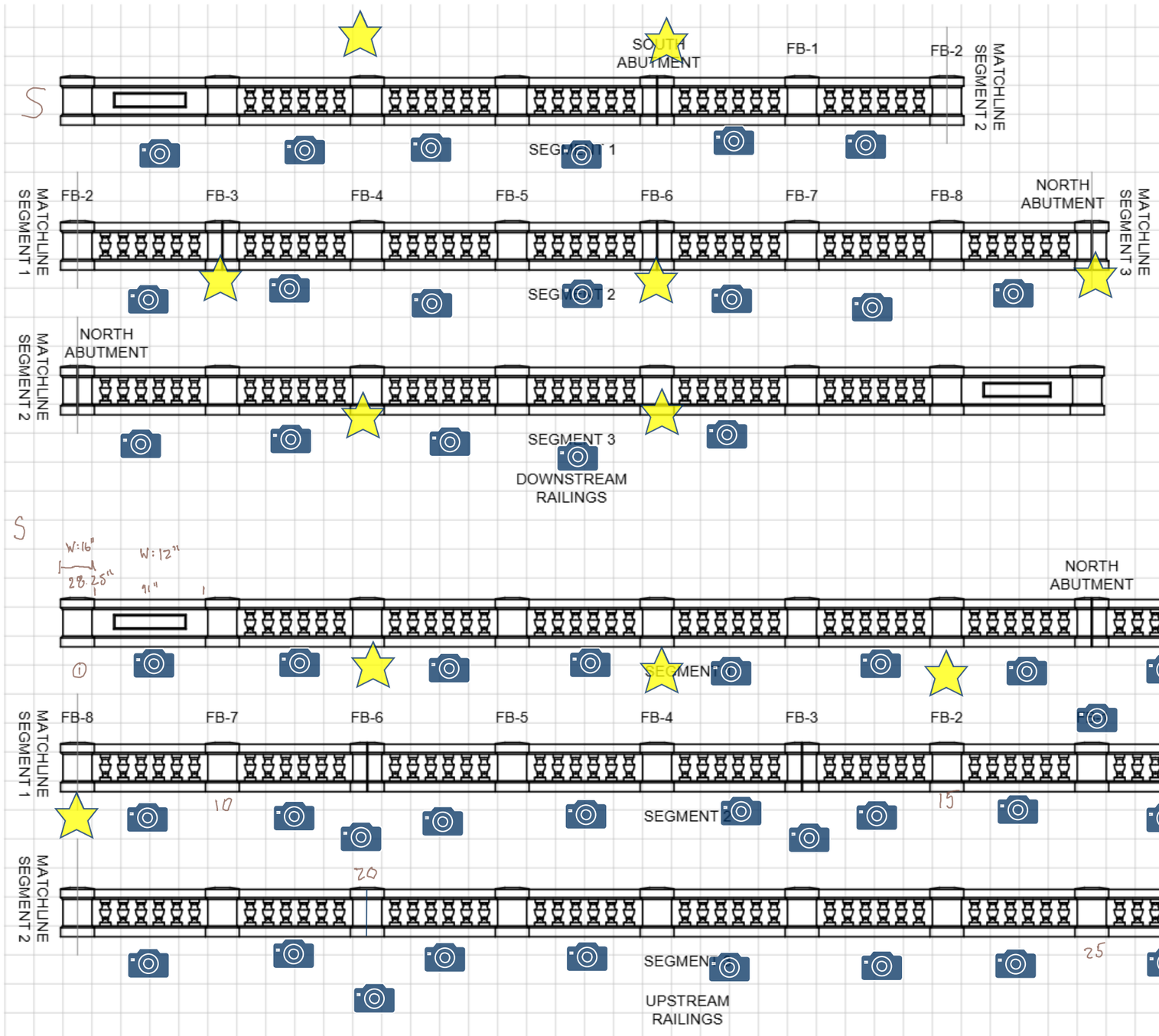
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Date	Value
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Checked	APJ
Scale	1/8" = 1'-0"

Sheet Title: US Walls Field Sheet

Sheet No.: 11

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Drawn	TWS
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US Railing Field Sheet
Sheet Title

Sheet No. **12**

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APPENDIX C. PHASE 2 STUDY AREA FIELD TESTING DOCUMENTATION

Core Label	Full Petro ?	Part. Petro ?	Chlor. ?	Comp. Str.?	Comp. Str. (psi)	Element	Location	Description	Conditions	Diam. (in.)	Length (in.)	Overlay Notes	Reinf. Condition Notes	Crack Features	Freeze-Thaw Notes	Other Notes
01-01	Y		Y	Y	6920	Arch	DS	Ext face, under FB-3	Downhill from exp. jt	3.75	12.0					Flat exterior surface, exposed fine aggregate; fractured interior surface
02-01			Y	Y	5360	Floor Beam	FB-5	South face, middle/DS	Near crown, good	3.75	12.5					Flat exterior surface, exposed fine aggregate; fractured interior surface
05-01			Y	Y	4920	Floor Beam	FB-3	South face, middle/DS	Below joint, bad	3.75	10.5					Flat exterior surface, showing low volume of localized paste loss and distress but generally intact; fractured interior surface
06-01				Y	5150	Column	FB-8	DS, south and ext faces	Minor damage some sides	3.75	13.3					Formed exterior surface; void space along exterior of bar and some coarse aggregate particles
07-01			Y	Y	5450	Arch	US	Underside/int face, Bay 9	Exposed rebar	3.75	12.0					Formed exterior surface; fractured interior surface
08-01	Y		Y	Y	7600	Floor Beam	FB-8	North face, US end	Near end, very good	3.75	10.0		Uncoated No. 8; 8 in. cover; Flash corrosion, no other observed corrosion Bar is diagonal bar extending to col, not main top bar Dwgs call it 3/4" dia			Formed exterior surface; void space along exterior of bar and some coarse aggregate particles; fractured interior surface
11-01			Y			Deck	Topside	Over FB-6, mid SB lane	Near crack over old joint	3.75	10.5	5.3 in. deep; Relatively smaller aggregate No. 4; Contact with bulk concrete uneven but intimate contact	Uncoated No. 6; 2.25 in. cover; Flash corrosion, no other observed corrosion Cover from top = 7.5", cover shown left is from underside because this is a bottom bar	Surface-parallel cracks interior nom. 3.5 inches, through and around agg	Bottom nom. 3.5 inches	Adhered possible waterproofing and other cementitious material on interior surface; Asphalt overlay nom. 2.25 inches thick, debonded
11-02	Y		Y	Y	10540	Deck	Topside	Over FB-5, centerline of roadway	near area of underside deck distress	3.75	10.3	2.3 in. deep; Relatively smaller aggregate No. 4; Contact with bulk concrete uneven but intimate contact	Uncoated No. 6?; Flash corrosion, no other observed corrosion Cover from top = 8.25", 1.25 in. cover from underside because this is a bottom bar			Minor honeycombing in interior 1 inch Asphalt overlay nom. 2.75 inches thick, debonded; Uneven, possibly formed interior surface

Core Log

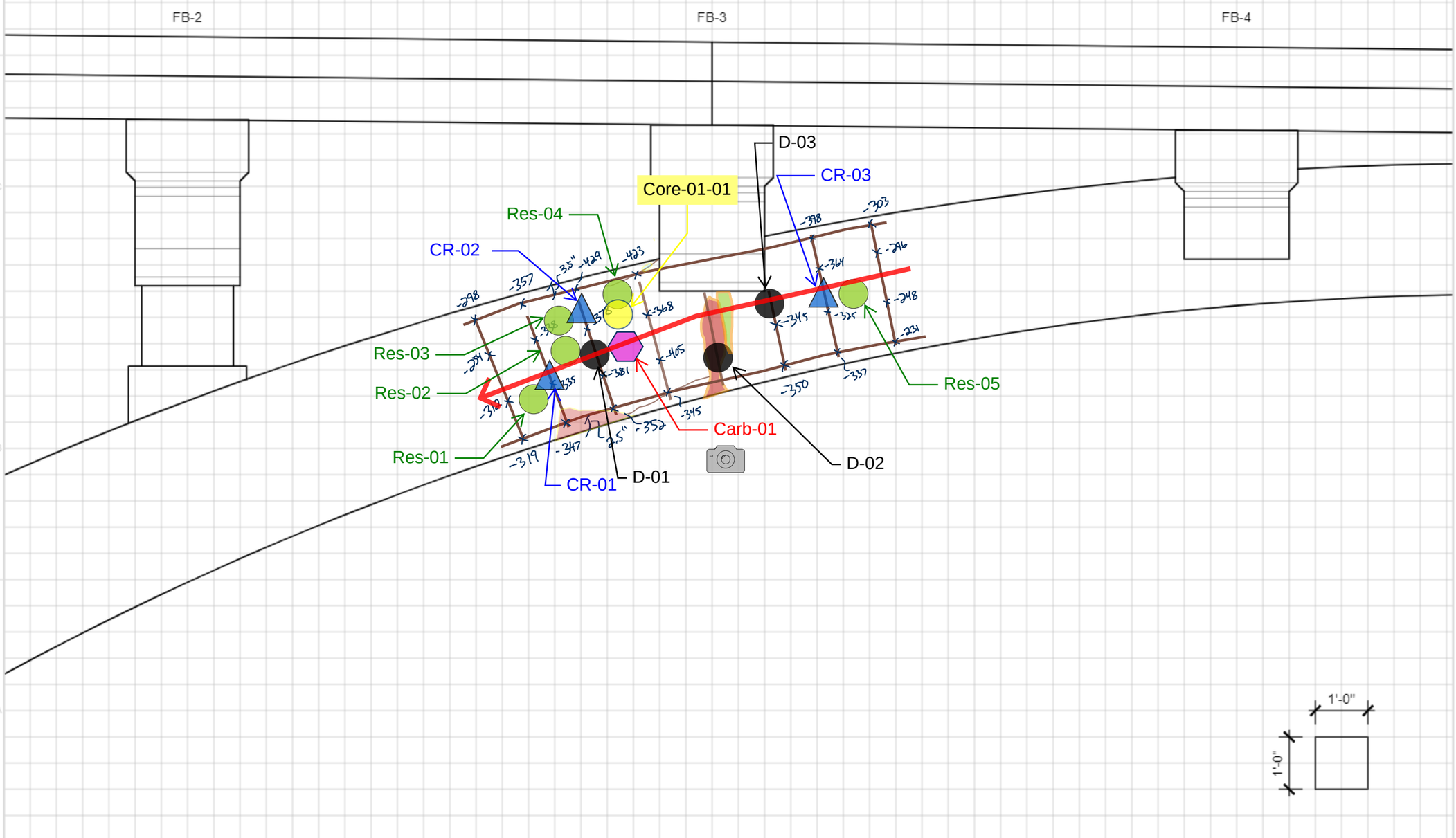
Core Label	Full Petro ?	Part. Petro ?	Chlor. ?	Comp. Str.?	Comp. Str. (psi)	Element	Location	Description	Conditions	Diam. (in.)	Length (in.)	Overlay Notes	Reinf. Condition Notes	Crack Features	Freeze-Thaw Notes	Other Notes
11-03			Y	Y	7930	Deck	Topside	Over FB-7, mid SB lane	underside somewhat distressed	3.75	10.0		Uncoated No. 6; Flash corrosion, no other observed corrosion Cover from top = 7.25", 2 in. from underside because this is a bottom bar			Asphalt layer nom. 2 inches thick, debonded; Uneven, possibly formed bottom surface
13-01	Y		Y	Y	7300	Arch	DS	Int face, Bay 1	Map cracking near springline	3.75	11.5			Surface-perpendicular crack extending from exterior surface through agg. Up to 3.25 inches deep; overlapping crack tips on surface		Flat exterior surface; overlapping crack tips on exterior; fractured interior
13-02	N			Y	7170	Arch	DS	Int face, Bay 1	Map cracking near springline	3.75	12.0					Formed exterior surface, minor craze cracking; Fractured interior surface; 1.5" agg along interior surface
15-01	Y			Y	5220	Abutment	South	Middle	near scaling/delam	3.75	10.5					Weathered exterior surface, exposed fine aggregate and slightly crumbling paste; Fractured interior surface; 1.5" top size
15-02			Y	Y	6230	Abutment	South	Middle	Good	3.75	9.25 to 11.25					Formed exterior surface with some exposed fine aggregate; Fractured interior surface
17-01		Y	Y	Y	5620	Retaining Wall	US South	Wall below drain, footing	Scaling, cracking, delam, F/T	3.75	11.5					Weathered exterior surface, exposed coarse and fine aggregate; Fractured interior surface
17-02		Y				Retaining Wall	US South	Top of footing	severe erosion on adjacent front face	3.75	13.0			Intermittent surface-parallel cracking through full thickness of core, through and around aggregate	Through full thickness of core, but core is still in one piece	Slightly uneven exterior surface, but no exposed aggregate; fractured interior surface

Core Log

Core Label	Full Petro ?	Part. Petro ?	Chlor. ?	Comp. Str.?	Comp. Str. (psi)	Element	Location	Description	Conditions	Diam. (in.)	Length (in.)	Overlay Notes	Reinf. Condition Notes	Crack Features	Freeze-Thaw Notes	Other Notes
17-03		Y				Retaining Wall	US South	Face of footing	severe erosion	3.75	21.0			Cracked into many pieces, crumbling in exterior nom. 11 inches, through and around aggregate, genreally parallel to oblique ot surface but some cracks are irregular	Through full thickness of core, but especially exterior nom. 11 inches	Need to rule out ASR- some aggregate has white material on it on fractured surfaces; Full thickness not definite because extioer surface was not retained,but exterior-most segment is only partial diameter, second segment is nom. 6.5 inches but cracked, and interior-most segment is nom. 10 inches with relatively less cracking but still has low volume of cracks; fractured interior surface
18-01		Y				Abutment	North	Front face of footing below US rib	near cracking and efflor	3.75	12 to 13.5			Fracture 6.275 to 8.5 inches from exteriormostly through aggregate, white deposits on some aggregate particles on fractured face; low volume of aggreaget internal agg. Particles		Weathered exterior surface; internal cracking in some aggregate, but doesn't really look like freeze thaw- need to look for ASR; fractured interior surface, uneven fracture
18-02		Y				Abutment	North	Top of footing below DS rib	severe erosion	3.75	8.5?			Fractured into several pieces, through and aroud aggregate	Likely crumbled from F/T, but the pieces cannot be put back together so difficult to say	(Assumed exterior surface) weathered, showing coarse and fine aggregate; Crumbled into several pieces, length cannot be measured but note on core says it was 8.5 inches to surface; Not much cracking in intact pieces of core fractured (assumed) interior surface

Core Log

DOWNSTREAM RIB
EXTERIOR FACE



1'-0"
1'-0"

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Consultants

Project
Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client
HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

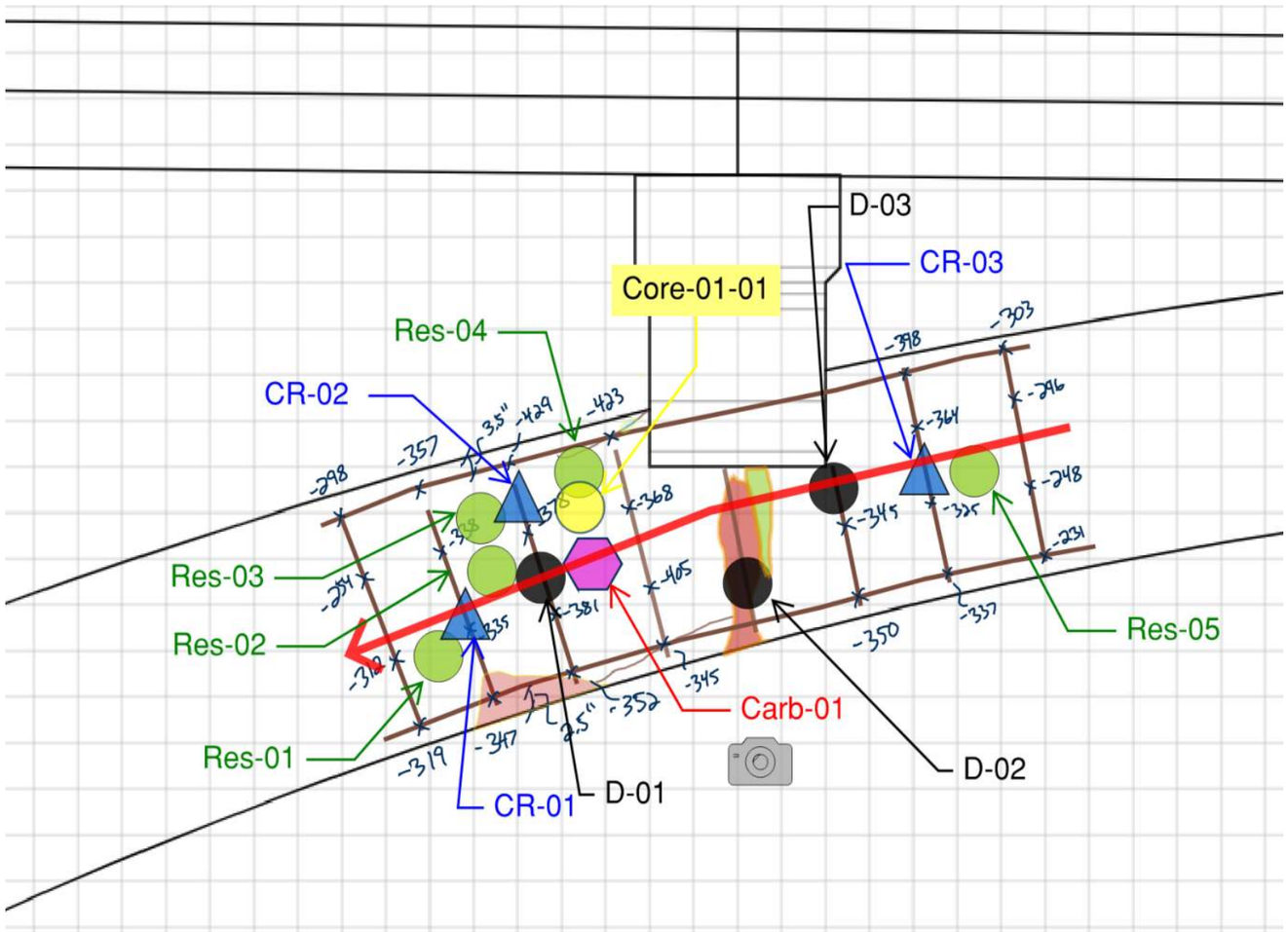
Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 1

Sheet No.: 01

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HCP Readings (as recorded and shown on sketch)

-298	-357	-429	-423
-254	-338	-378	-368
-312	-335	-381	-405
-319	-347	-352	-345

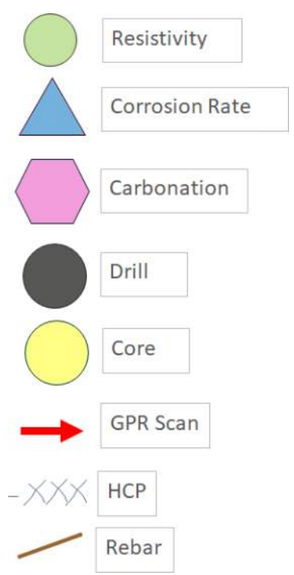
	-398	-303
	-364	-296
-345	-325	-248
-350	-337	-231

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	44
HCP Correction (mV)	-15.5

HCP Readings (corrected to 75 deg F)

-314	-373	-445	-439
-270	-354	-394	-384
-328	-351	-397	-421
-335	-363	-368	-361

	-414	-319
	-380	-312
-361	-341	-264
-366	-353	-247



Study Area SA-01_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_01.pdf	**Res-03**						44	46	47	46
SA_01.pdf	**Res-01**						43	29	30	34
SA_01.pdf	**Res-04**						29	33	34	32
SA_01.pdf	**Res-05**	10/14/2024		51	44	Yes	37	33	39	36
SA_01.pdf	**Res-02**	10/14/2024		51	44	Yes	68	65	68	67

Drill Holes

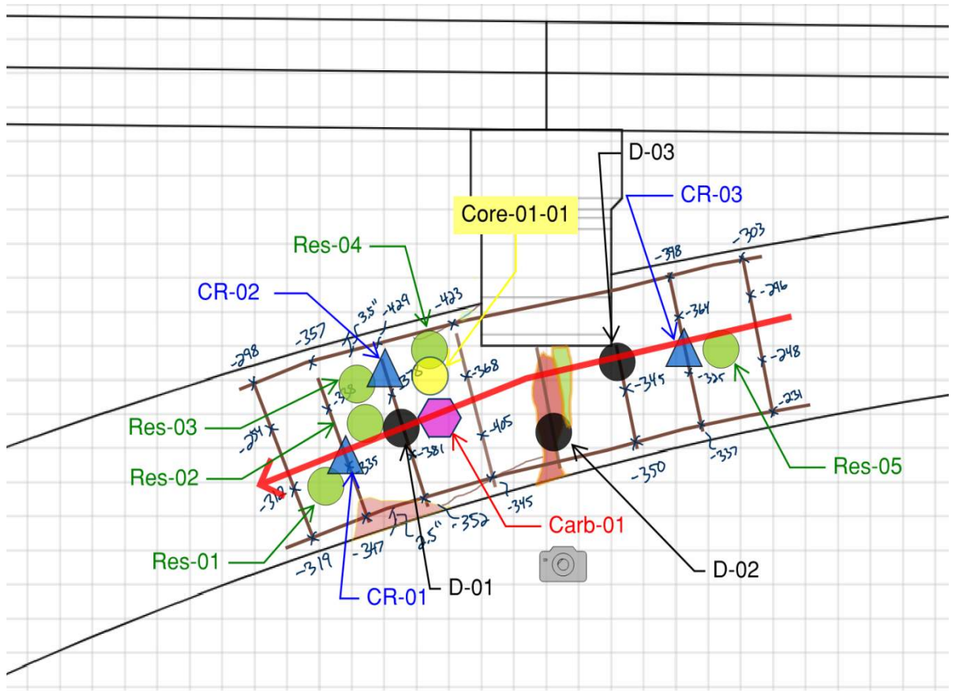
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_01.pdf	**D-02**	10/14/2024	Ground on exposed bar	0	
SA_01.pdf	**D-03**	10/14/2024		2.875	
SA_01.pdf	**D-01**	10/14/2024	1 to 3 = 500 ohm 56 mV 1 to 2 = 100 ohm 5 mV Do double ground at 1 and 3	3.25	

Corrosion Rate - iCOR

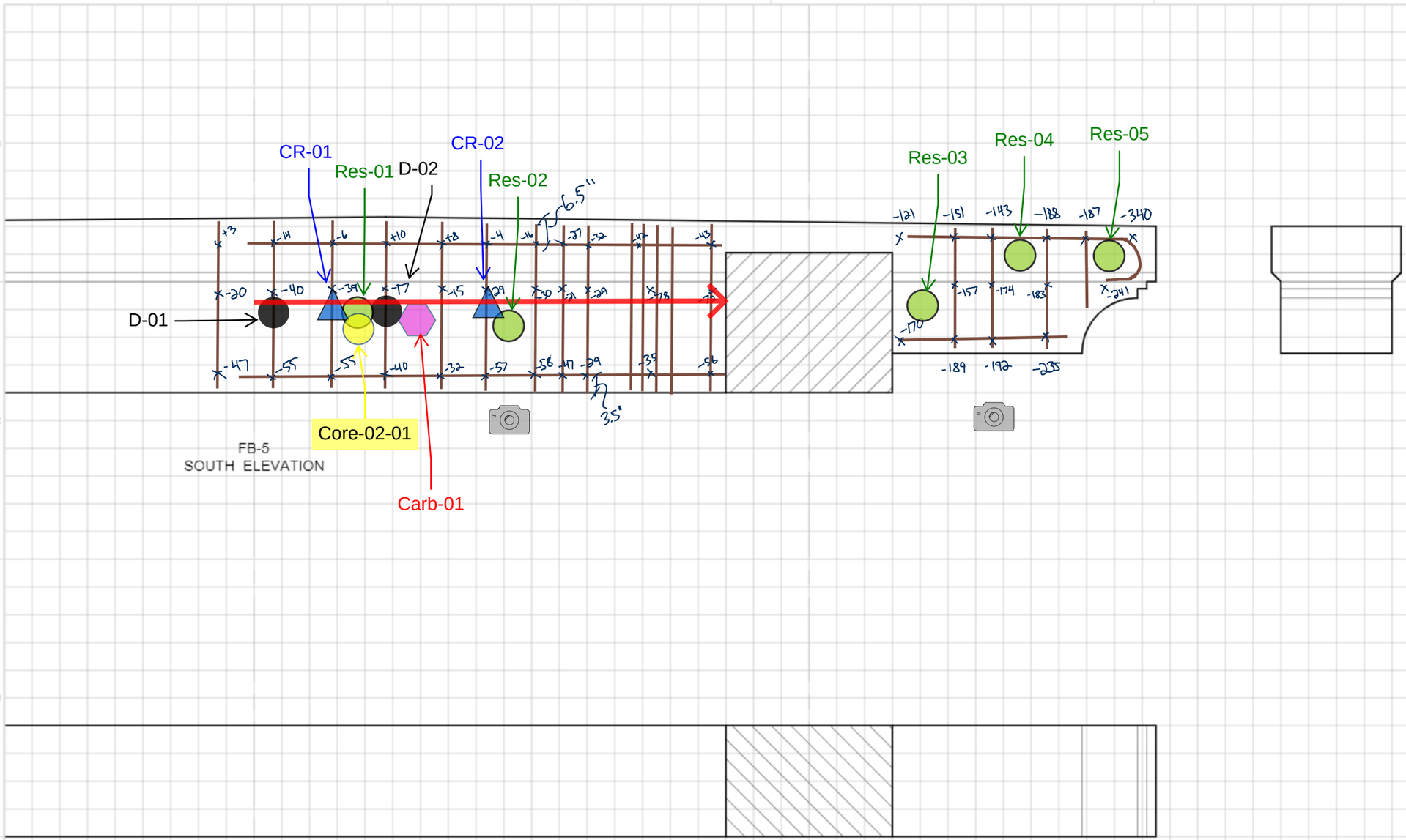
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SA_01.pdf	**CR-01**	10/14/2024	0.3 mA/cm^2 48 kOhm-cm	Yes
SA_01.pdf	**CR-03**	10/14/2024	0.3 mA/cm^2 32 kOhm-cm	Yes
SA_01.pdf	**CR-02**	10/14/2024	0.2 mA/cm^2 61 kOhm-cm	Yes

Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
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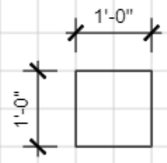


Study Area SA-01_Notes



FB-5
SOUTH ELEVATION

FB-5 UNDERSIDE



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Project

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Bridge 55 Over Mill Brook
Windsor, VT 05089

Client

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3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

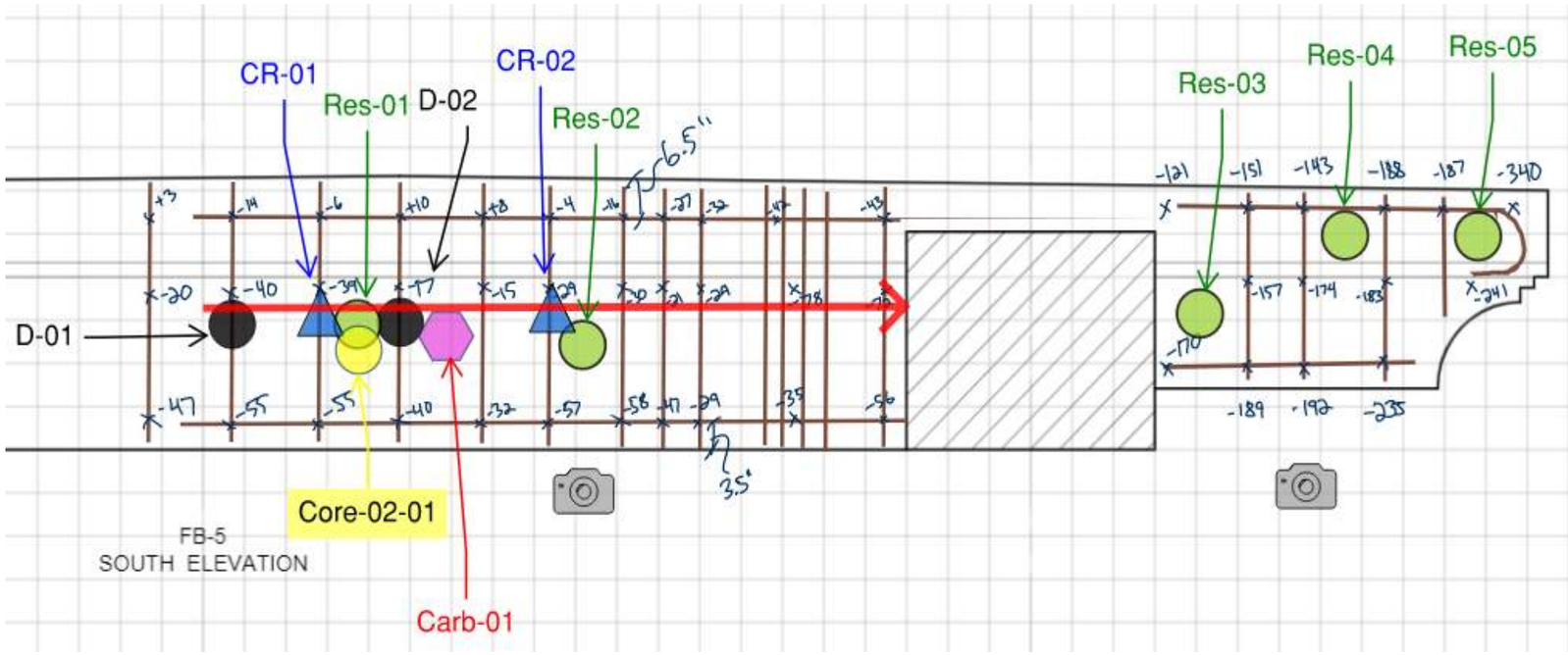
0 1/2" 1"

THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 2

Sheet No.: **02**



Resistivity
Corrosion Rate
Carbonation
Drill
Core
GPR Scan
HCP
Rebar

HCP Readings (as recorded)

3	-14	-6	10	8	-4	-16	-27	-32	-42	-43
-20	-40	-39	-17	-15	-29	-30	-21	-29	-78	-72
-47	-55	-55	-40	-32	-57	-58	-29	29	-35	-56

-121	-151	-143	-188	-187	-340
	-157	-174	-183	-241	
-170	-189	-192	-235		

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	44
HCP Correction (mV)	-15.5

HCP Readings (corrected to 75 deg F)

-13	-30	-22	-6	-8	-20	-32	-43	-48	-58	-59
-36	-56	-55	-33	-31	-45	-46	-37	-45	-94	-88
-63	-71	-71	-56	-48	-73	-74	-45	14	-51	-72

-137	-167	-159	-204	-203	-356
	-173	-190	-199	-257	
-186	-205	-208	-251		

Study Area SA-02_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_02.pdf	**Res-05**			51	44		62	59	68	63
SA_02.pdf	**Res-04**			51	44		49	50	50	50
SA_02.pdf	**Res-03**			51	44		76	65	55	65
SA_02.pdf	**Res-01**			51	44		478	623	562	554
SA_02.pdf	**Res-02**			51	44		380	375	432	396

Drill Holes

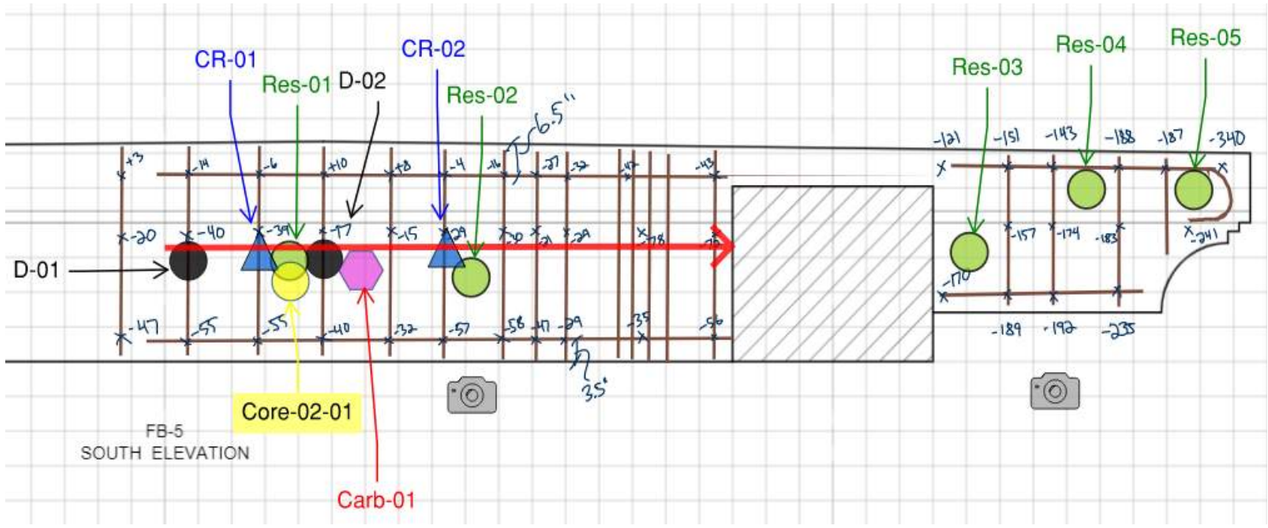
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_02.pdf	**D-01**			2.5	
SA_02.pdf	**D-02**	10/14/2024	12 mV continuity Double ground	2.625	200

Corrosion Rate - iCOR

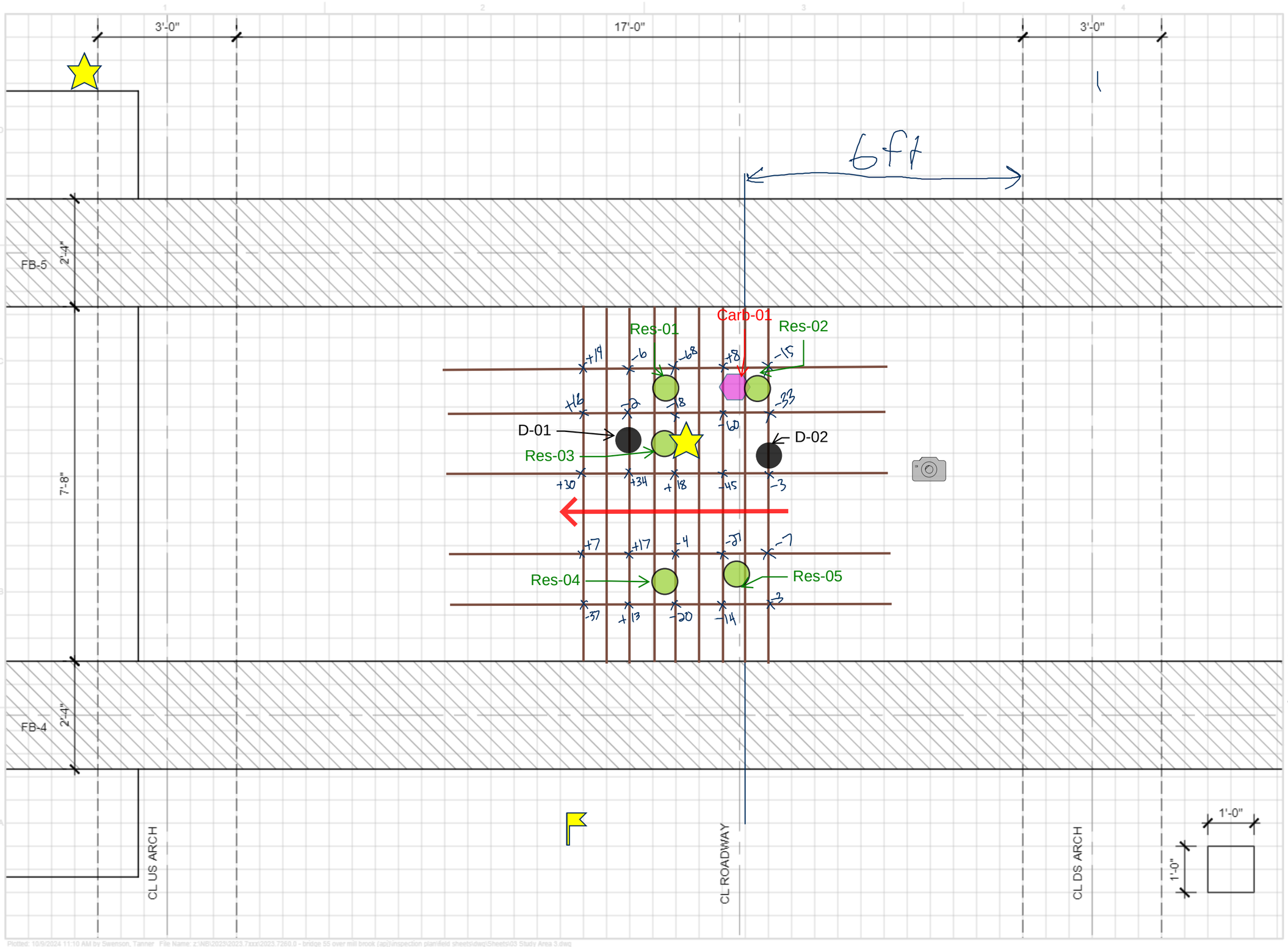
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SA_02.pdf	**CR-02**	10/14/2024	0.22 mA/cm^2 319 kOhm-cm	Yes
SA_02.pdf	**CR-01**	10/14/2024	0.04 mA/cm^2 385 kOhm-cm	Yes

Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_02.pdf	**Carb-01**	0.125	



Study Area SA-02_Notes



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Project

Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client

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Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

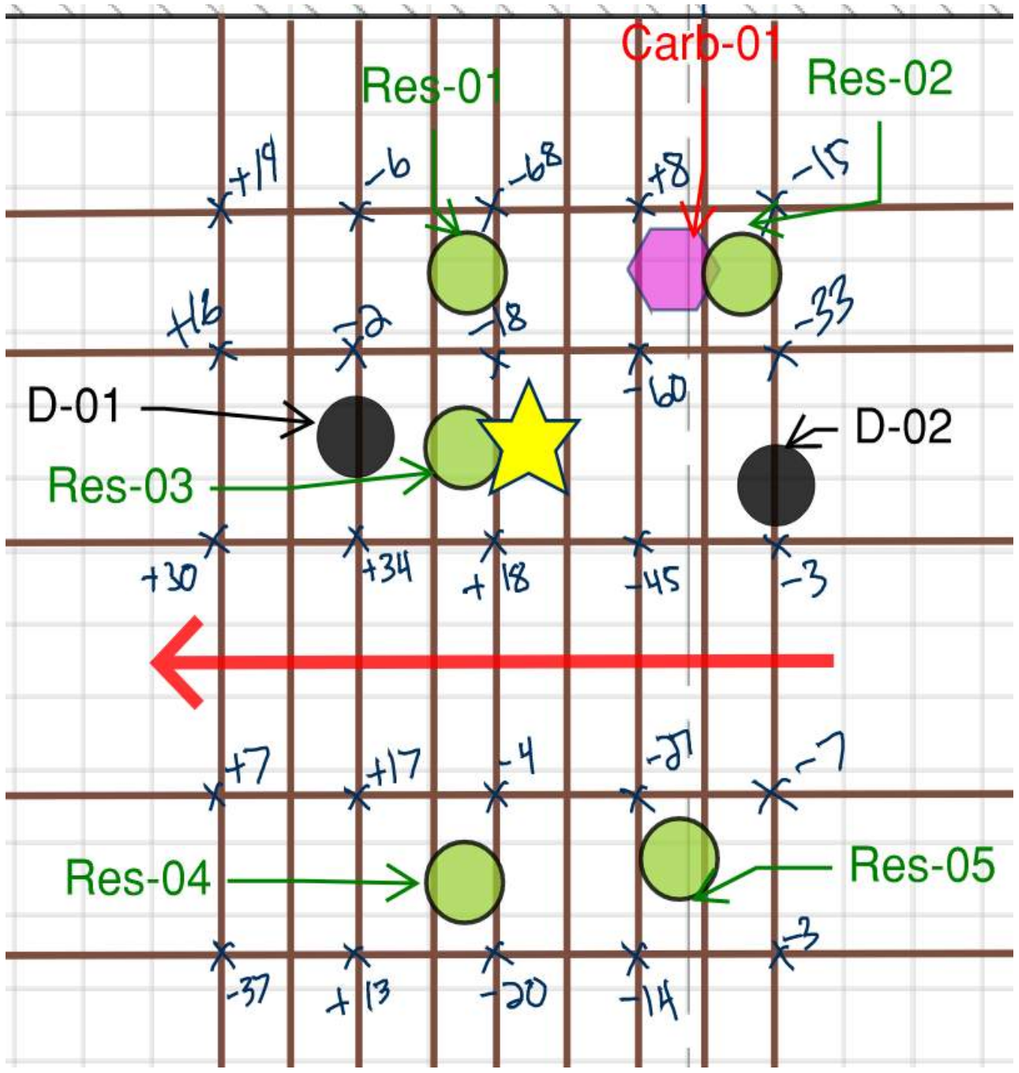
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Date	October 14, 2024
Drawn	TWS
Checked	APJ
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Sheet Title: Study Area 3

Sheet No.: 03

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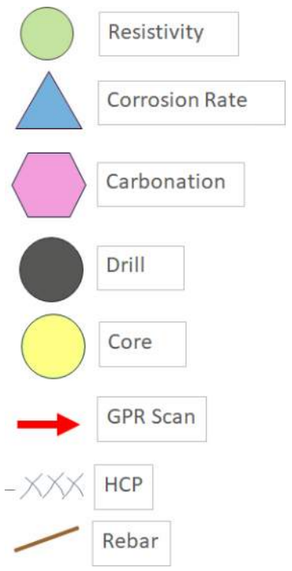
HCP Readings (as recorded and shown on sketch)

19	-6	-68	8	-15
16	-2	-18	-60	-33
30	34	18	-45	-3
7	17	-4	-27	-7
-37	13	-20	-14	-3

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	44
HCP Correction (mV)	-15.5

HCP Readings (corrected to 75 deg F)

4	-22	-84	-8	-31
1	-18	-34	-76	-49
15	19	3	-61	-19
-9	2	-20	-43	-23
-53	-3	-36	-30	-19



Study Area SA-03_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_03.pdf	**Res-05**						401	492	504	466
SA_03.pdf	**Res-04**						373	483	462	439
SA_03.pdf	**Res-02**						490	320	485	432
SA_03.pdf	**Res-01**		Overflow							Overflow
SA_03.pdf	**Res-03**						860	735	1100	898

Drill Holes

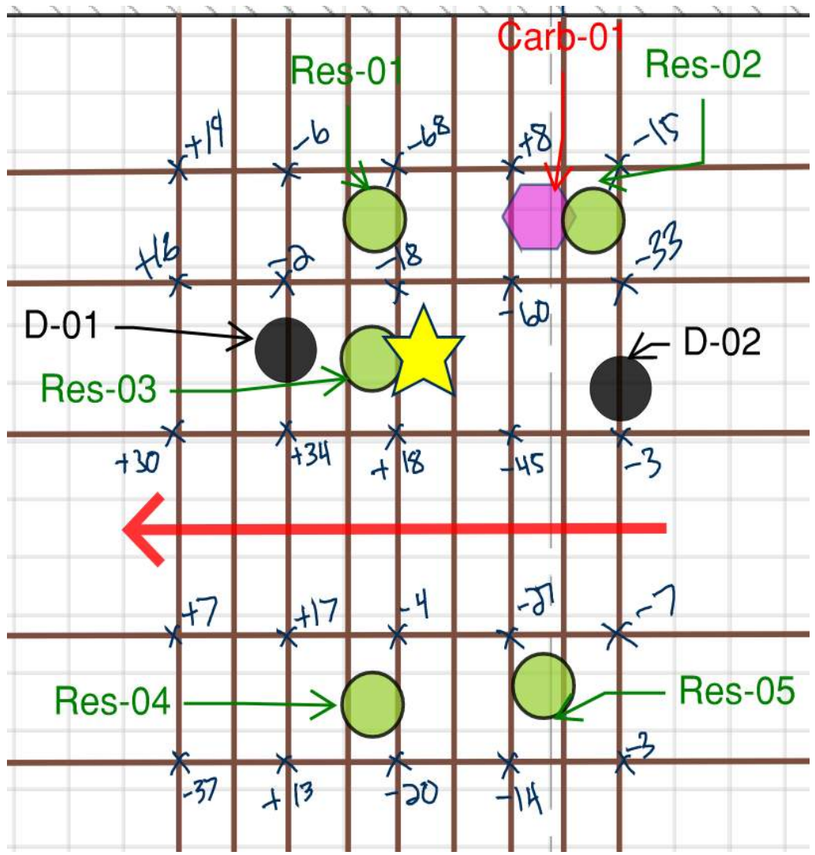
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_03.pdf	**D-01**			1.75	
SA_03.pdf	**D-02**	10/14/2024	Continuity to FB-5 SA-02 = 90 ohm 30mV	1.5	0.8

Corrosion Rate - iCOR

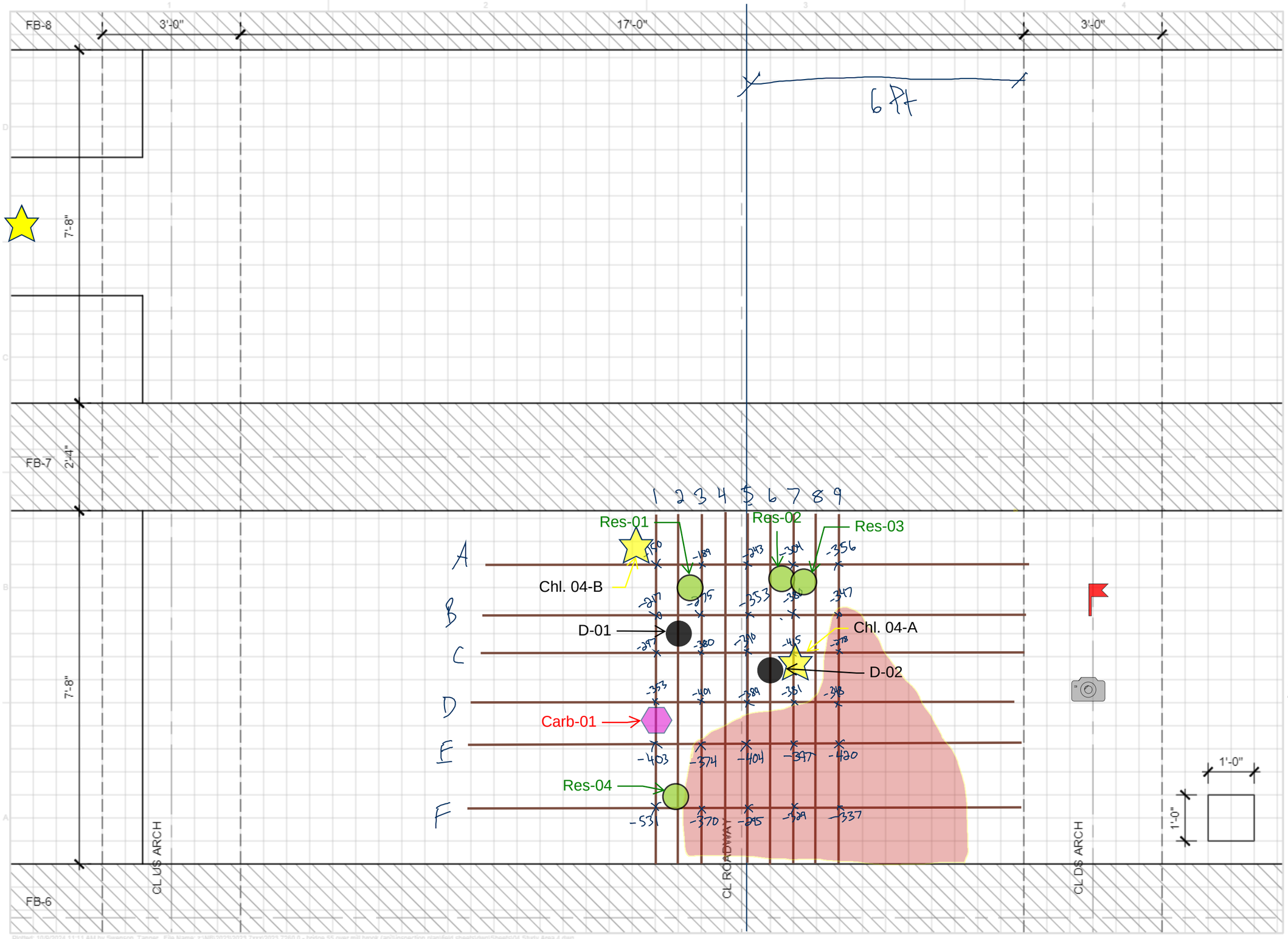
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Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_03.pdf	**Carb-01**	0	Negligible



Study Area SA-03_Notes



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Windsor, VT 05089

Client
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Bedford, NH 03110

Mark	Date	Description

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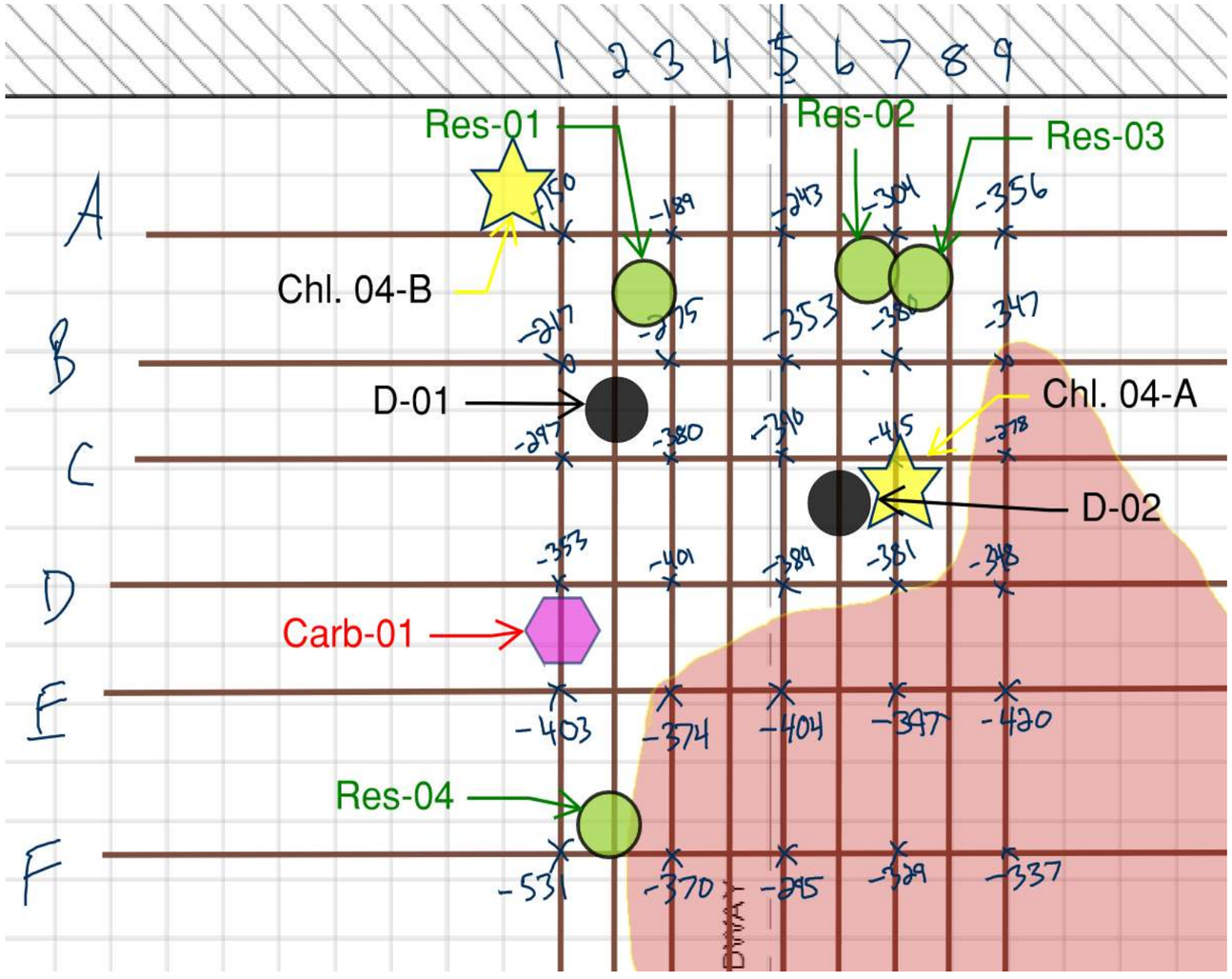
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Date	October 14, 2024
Drawn	TWS
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Scale	1/2" = 1'-0"

Sheet Title: Study Area 4

Sheet No.: 04

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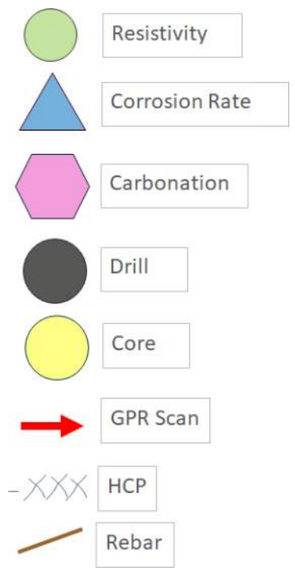
HCP Readings (as recorded and shown on sketch)

-150	-189	-243	-304	-356
-217	-275	-353	-380	-347
-297	-380	-390	-415	-278
-353	-401	-389	-381	-348
-403	-374	-404	-397	-420
-531	-370	-295	-329	-337

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	44
HCP Correction (mV)	-15.5

HCP Readings (corrected to 75 deg F)

-166	-205	-259	-320	-372
-233	-291	-369	-396	-363
-313	-396	-406	-431	-294
-369	-417	-405	-397	-364
-419	-390	-420	-413	-436
-547	-386	-311	-345	-353



Study Area SA-04_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_04.pdf	**Res-03**	10/14/2024				Yes	46	51	59	52
SA_04.pdf	**Res-02**	10/14/2024				Yes	62	61	62	62
SA_04.pdf	**Res-04**	10/14/2024				Yes	31	25	34	30
SA_04.pdf	**Res-01**	10/14/2024				Yes	74	67	69	70

Drill Holes

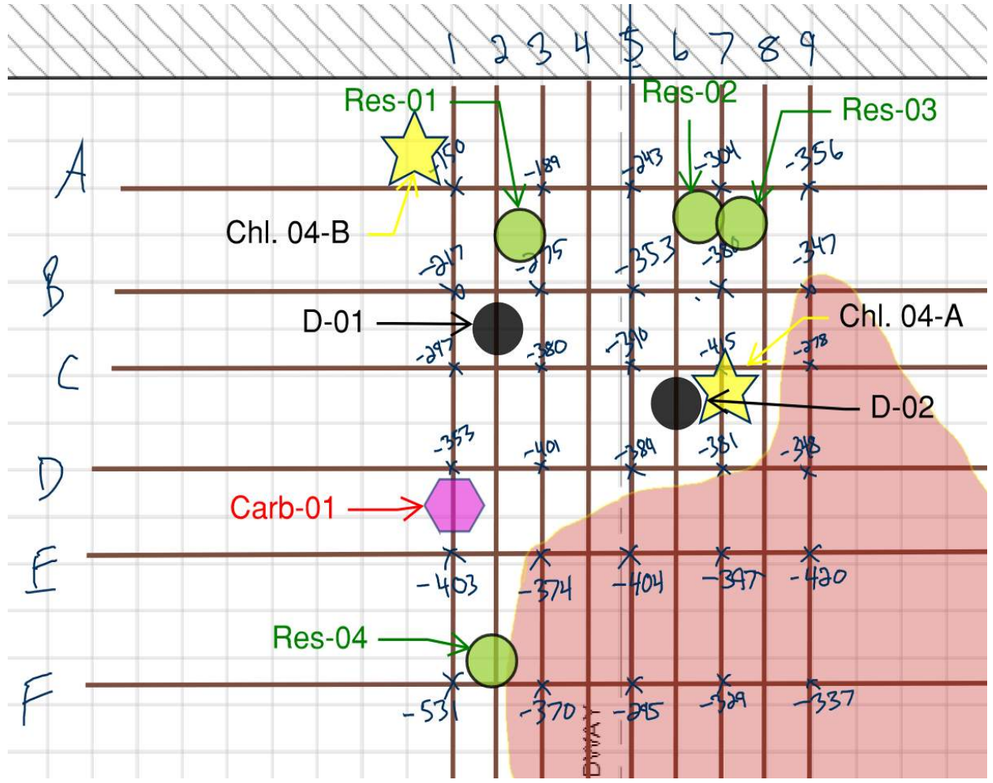
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_04.pdf	**D-01**	10/14/2024		1.875	0.3
SA_04.pdf	**D-02**			1.5	0.3

Corrosion Rate - iCOR

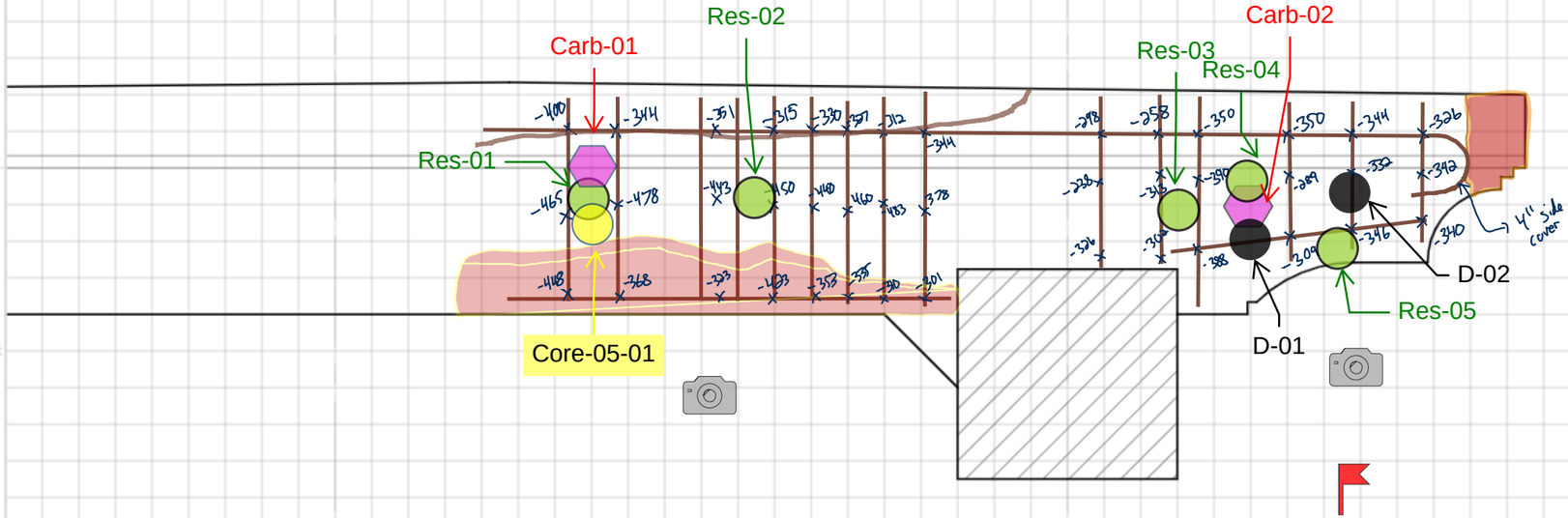
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used

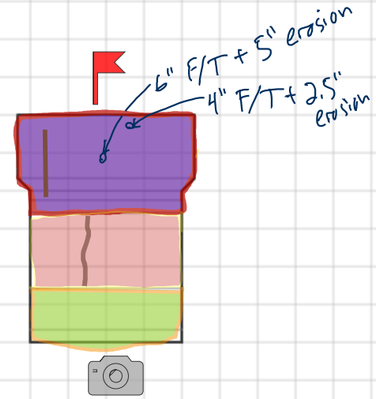
Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_04.pdf	**Carb-01**	0.125	

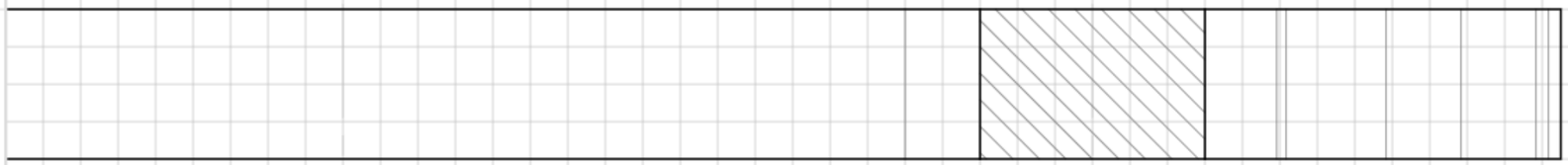


Study Area SA-04_Notes

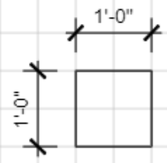




FB-3
SOUTH ELEVATION



FB-3 UNDERSIDE



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Plotted: 10/9/2024 11:11 AM by Swenson, Tanner File Name: z:\NB\2023\2023.7xxx\2023.7260.0 - bridge 55 over mill brook (apj)\inspection plan\field sheets\dwg\Sheets\05 Study Area 5.dwg

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Consultants

Project

Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client

HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

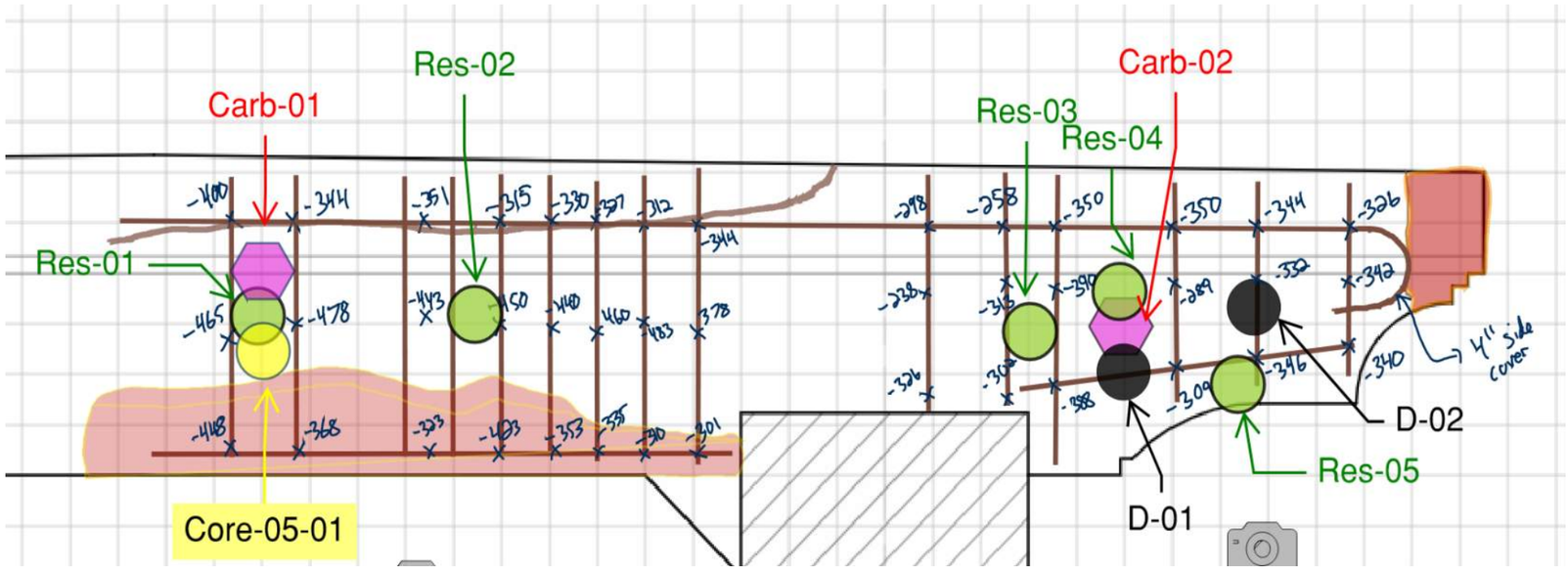
Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 5

Sheet No.: **05**



HCP Readings (as recorded and shown on sketch)

-400	-344	-351	-315	-330	-327	-312	-344
-465	-478	-443	-450	-440	-460	-483	-378
-448	-368	-323	-423	-353	-335	-310	-301

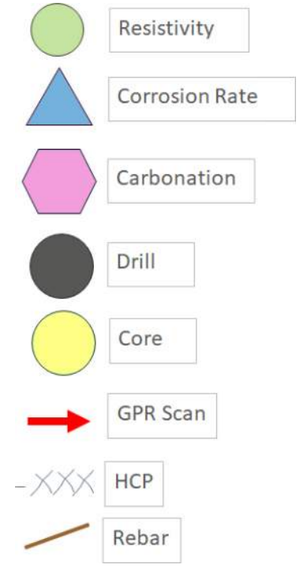
-298	-258	-350	-350	-344	-326
-238	-313	-390	-289	-332	-342
-326	-302	-388	-309	-346	-340

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	48
HCP Correction (mV)	-13.5

HCP Readings (corrected to 75 deg F)

-414	-358	-365	-329	-344	-341	-326	-358
-479	-492	-457	-464	-454	-474	-497	-392
-462	-382	-337	-437	-367	-349	-324	-315

-312	-272	-364	-364	-358	-340
-252	-327	-404	-303	-346	-356
-340	-316	-402	-323	-360	-354



Study Area SA-05_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_05.pdf	**Res-01**						11	11	14	12
SA_05.pdf	**Res-02**						12	14	17	14
SA_05.pdf	**Res-03**					Yes	69	57	55	60
SA_05.pdf	**Res-04**						67	58	43	56
SA_05.pdf	**Res-05**	10/14/2024				Yes	59	66	76	67

Drill Holes

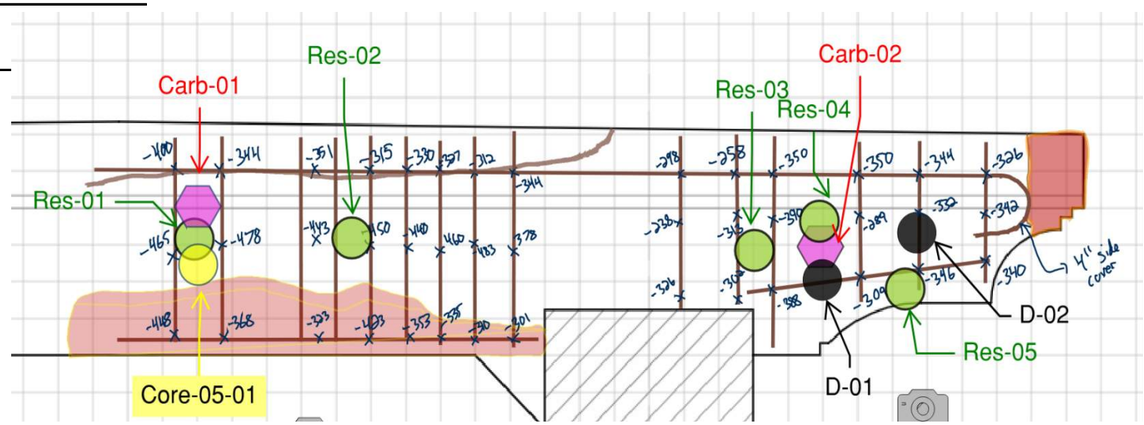
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_05.pdf	**D-01**			3.625	
SA_05.pdf	**D-02**	10/14/2024	Between FB-3 grounds 0.7 mV, 120 ohmTo arch SA-01, 2 to 12 mV	2.125	

Corrosion Rate - iCOR

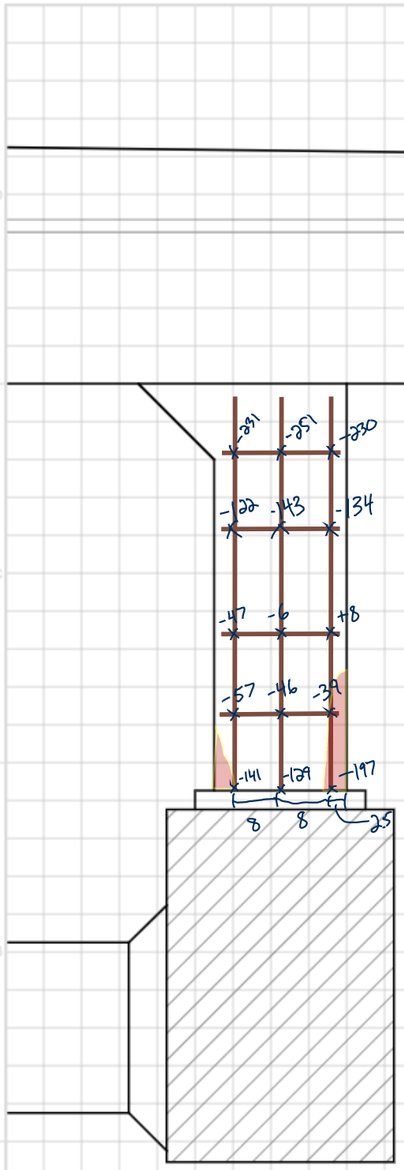
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used

Carbonation

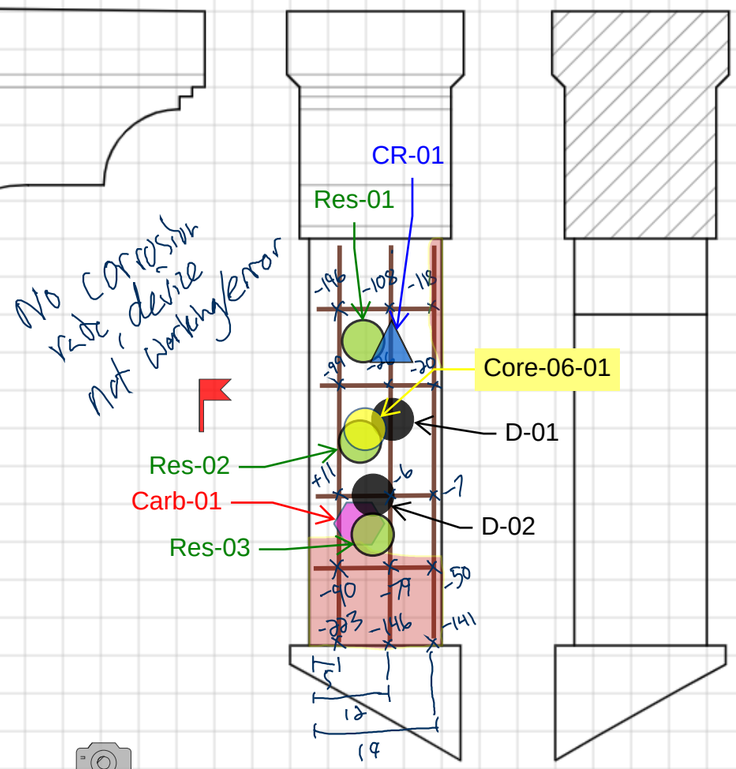
document-name	Location ID	Carb. Depth (in)	Comments
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SA_05.pdf	**Carb-02**		Variable 0.25-1"



Study Area SA-05_Notes

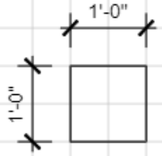


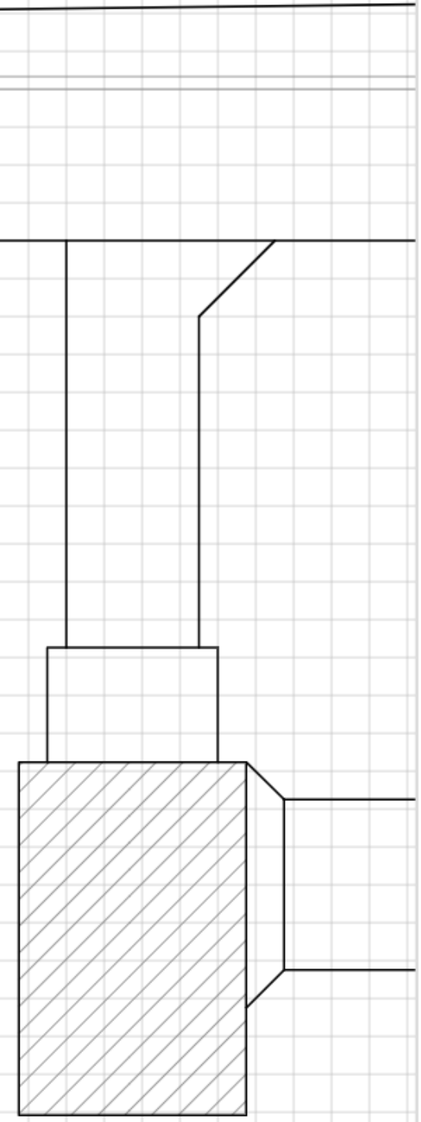
FB-8
SOUTH ELEVATION



FB-8
DS FACE

FB-8
US FACE





FB-8
NORTH ELEVATION

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Consultants

Project
Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client
HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

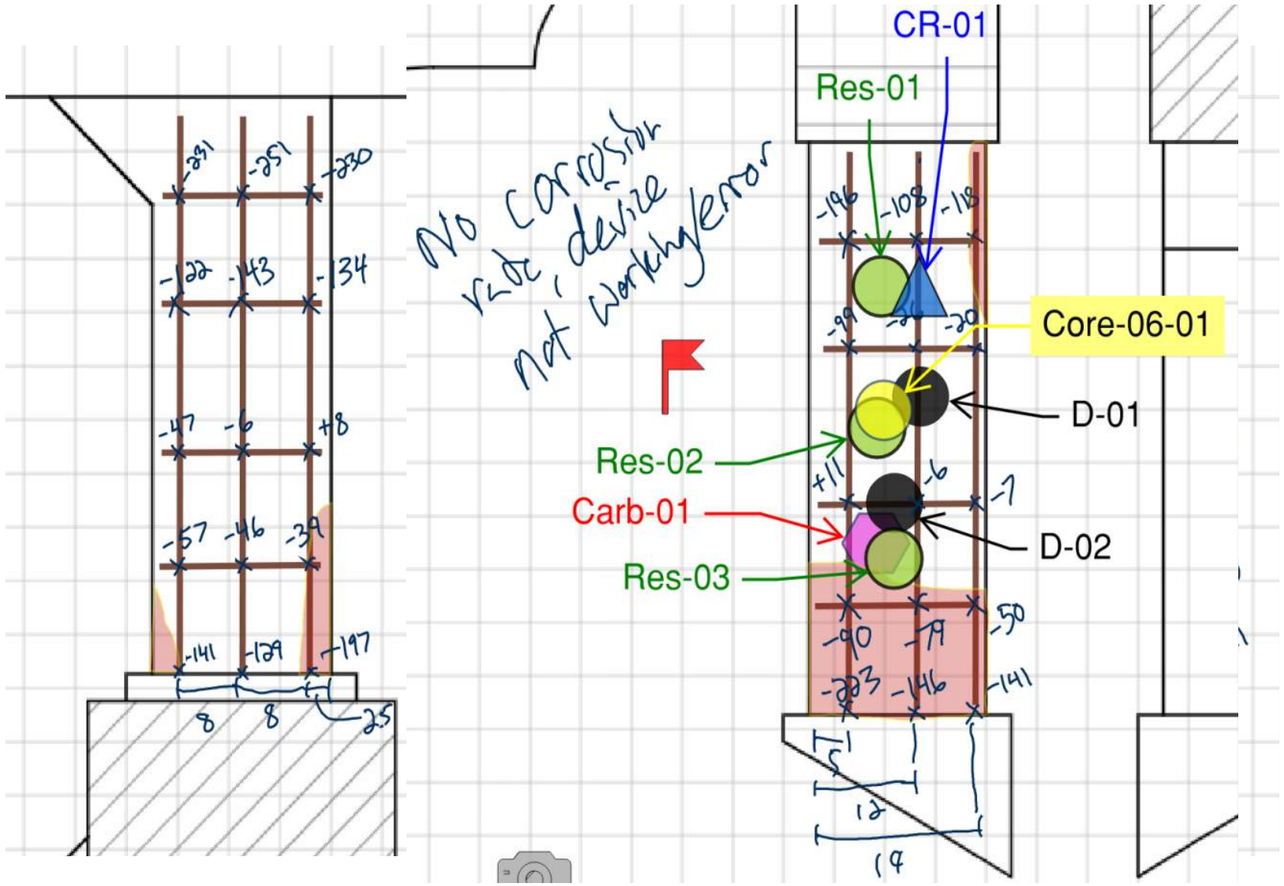
Project No.	2023.7260
Date	October 14, 2024
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Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 6

Sheet No.: **06**

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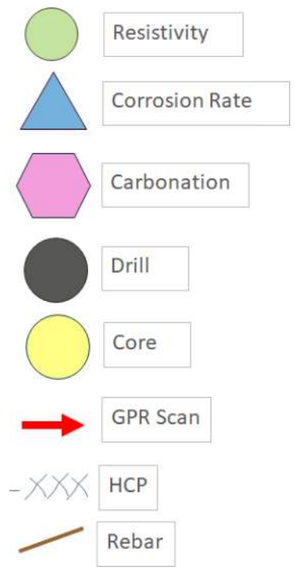
HCP Readings (as recorded and shown on sketch)

-231	-251	-230	-196	-108	-118
-122	-143	-134	-99	-26	-20
-47	-6	8	11	-6	-7
-57	-46	-39	-90	-79	-50
-141	-129	-197	-223	-146	-141

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	44
HCP Correction (mV)	-15.5

HCP Readings (corrected to 75 deg F)

-247	-267	-246	-212	-124	-134
-138	-159	-150	-115	-42	-36
-63	-22	-8	-5	-22	-23
-73	-62	-55	-106	-95	-66
-157	-145	-213	-239	-162	-157



Study Area SA-06_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_06.pdf	**Res-01**	10/15/2024					370	402	403	392
SA_06.pdf	**Res-03**	10/15/2024	In delam				327	276	209	271
SA_06.pdf	**Res-02**	10/15/2024					539	476	396	470

Drill Holes

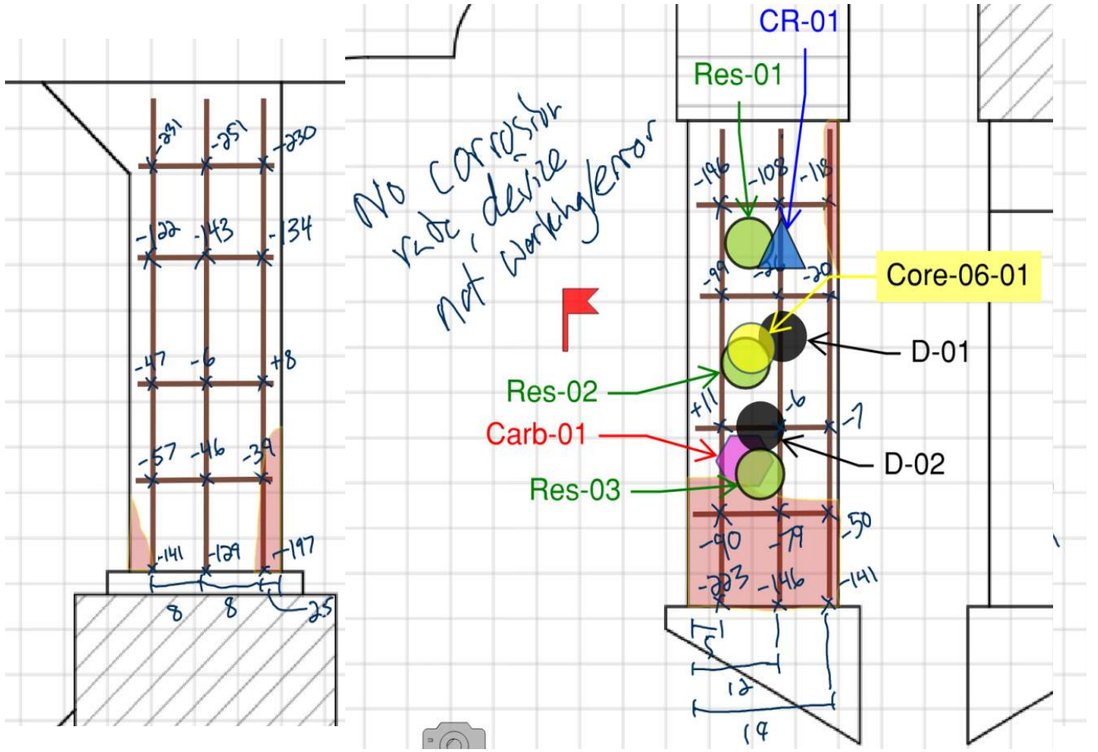
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_06.pdf	**D-02**	10/15/2024		1.375	
SA_06.pdf	**D-01**		6.7 ohm, 0.3 mV	2	

Corrosion Rate - iCOR

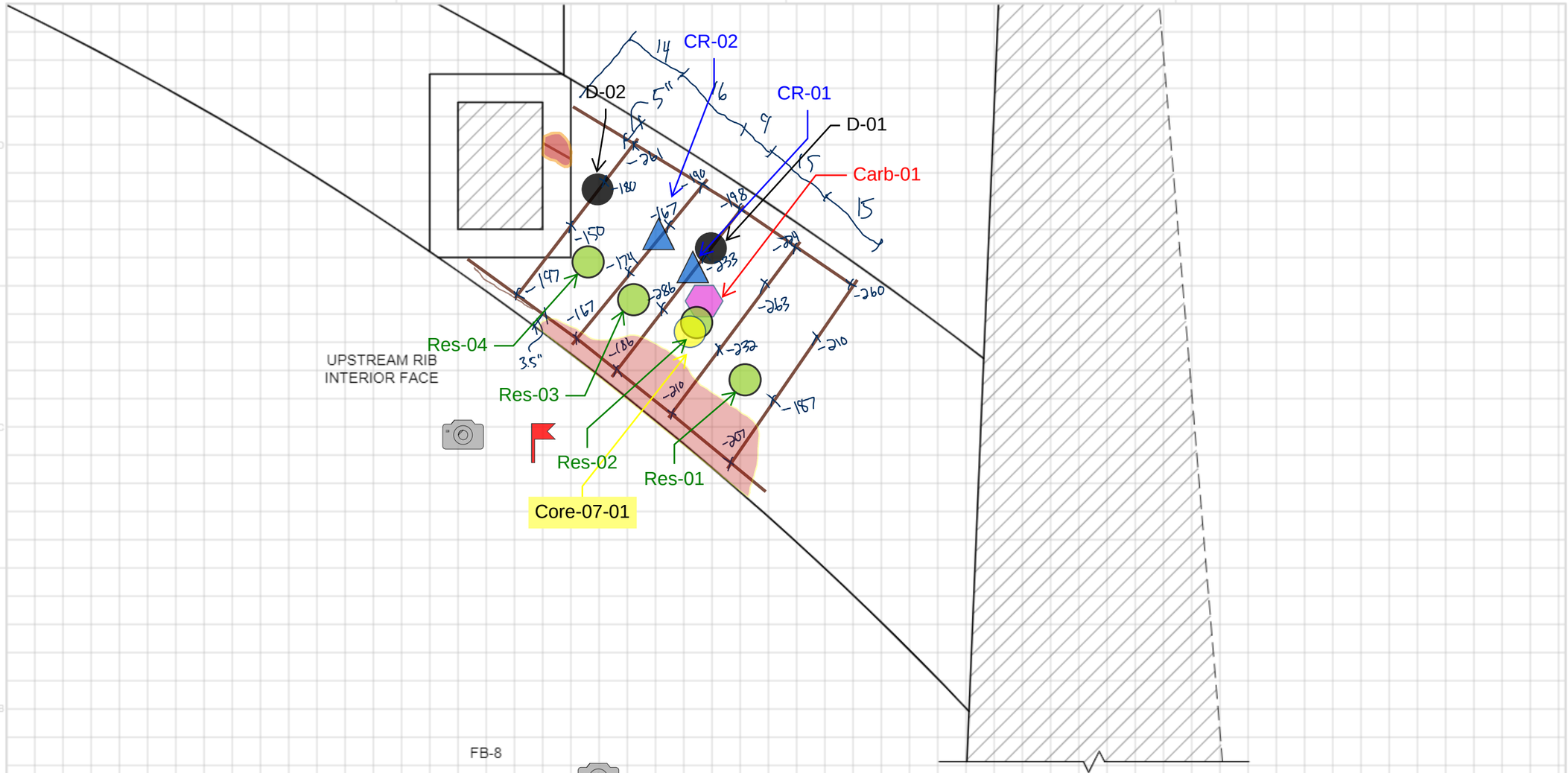
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used
SA_06.pdf	**CR-01**	10/14/2024	mA/cm^2 kOhm-cm	Yes

Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_06.pdf	**Carb-01**	0	Negligible



Study Area SA-06_Notes



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San Diego | San Francisco | Seattle | South Florida | Washington, DC

Consultants

Project
Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client
HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

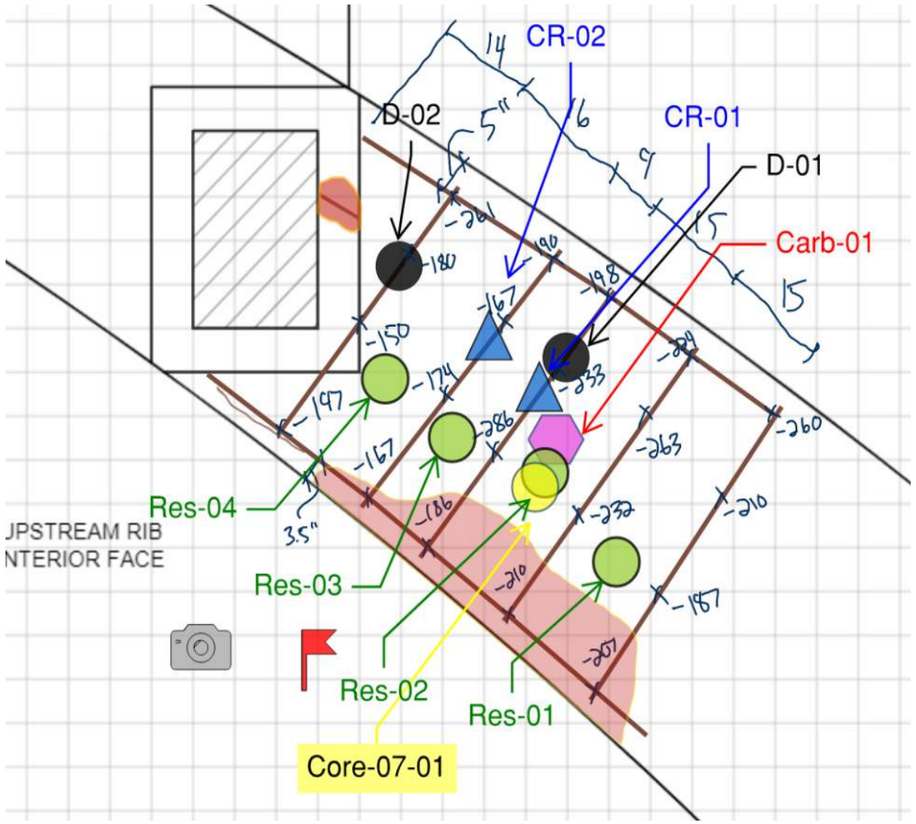
Project No.	2023.7260
Date	October 14, 2024
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Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 7

Sheet No.: **07**

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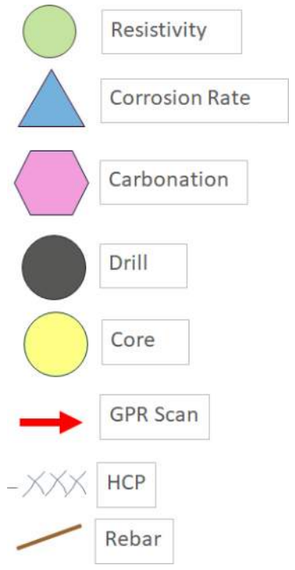
HCP Readings (as recorded and shown on sketch)

-261	-190	-198	-224	-260
-180	-167	-233	-263	-210
-150	-174	-286	-232	-187
-147	-167	-186	-210	-207

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	49
HCP Correction (mV)	-13

HCP Readings (corrected to 75 deg F)

-274	-203	-211	-237	-273
-193	-180	-246	-276	-223
-163	-187	-299	-245	-200
-160	-180	-199	-223	-220



Study Area SA-07_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_07.pdf	**Res-01**						129	140	138	136
SA_07.pdf	**Res-02**						122	122	135	126
SA_07.pdf	**Res-03**						173	240	167	193
SA_07.pdf	**Res-04**						158	133	140	144

Drill Holes

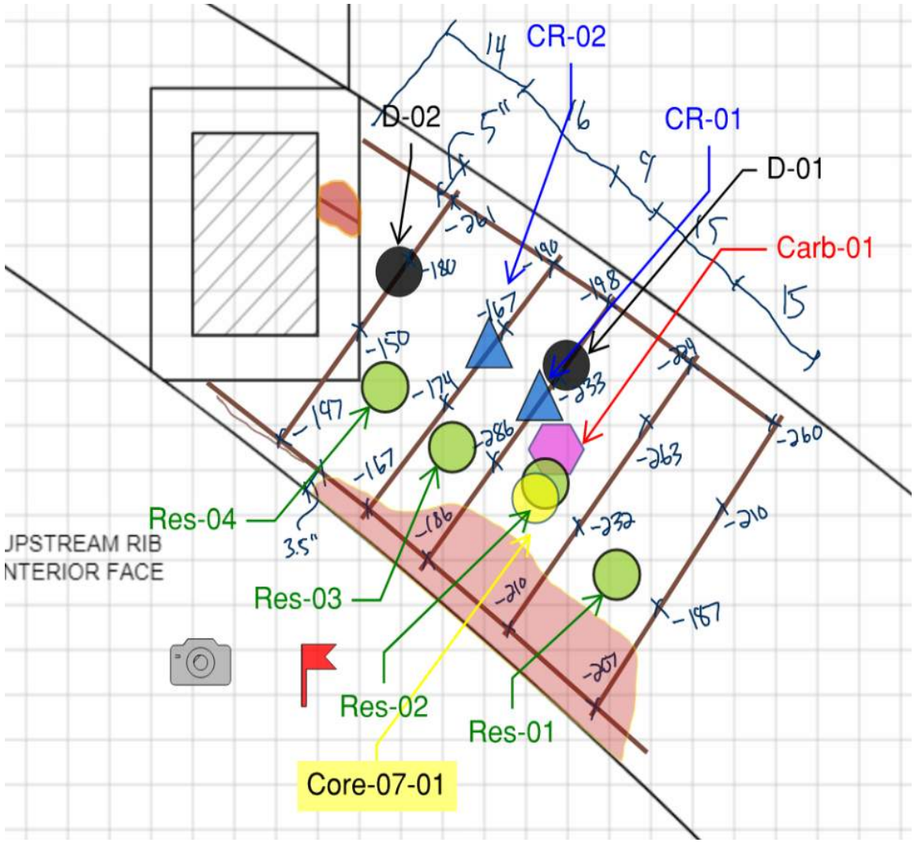
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_07.pdf	**D-01**	10/15/2024	42 ohm, 6 mV, double ground	3.125	
SA_07.pdf	**D-02**			2.5	

Corrosion Rate - iCOR

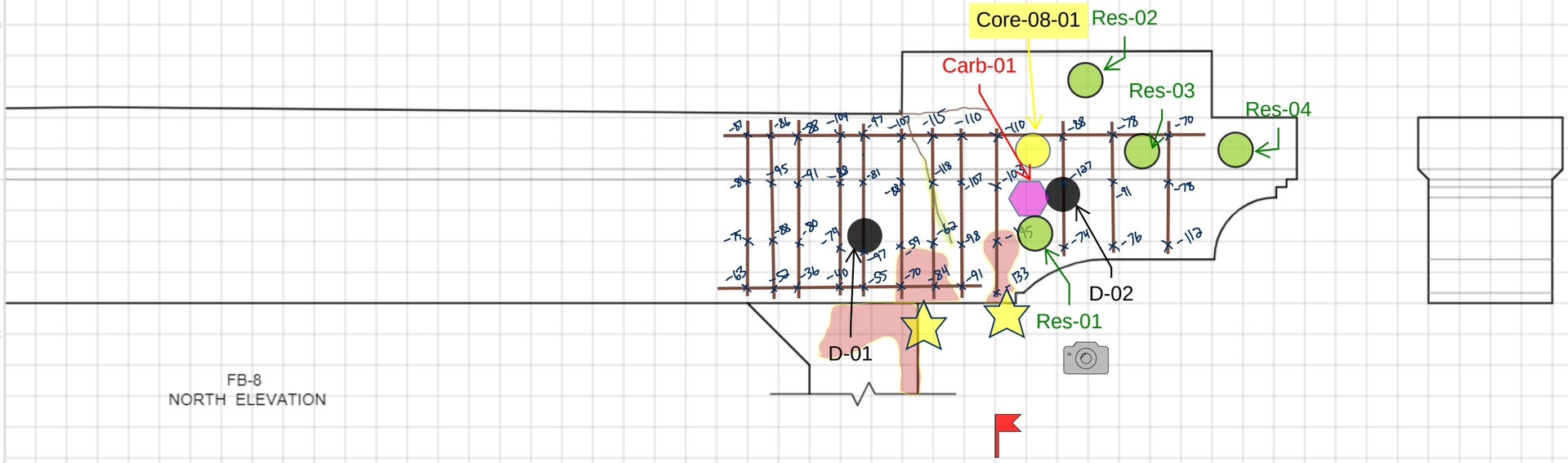
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used
SA_07.pdf	**CR-01**	10/15/2024	0.08 mA/cm^2118 kOhm-cm	Yes
SA_07.pdf	**CR-02**	10/15/2024	0.09 mA/cm^2204 kOhm-cm	Yes

Carbonation

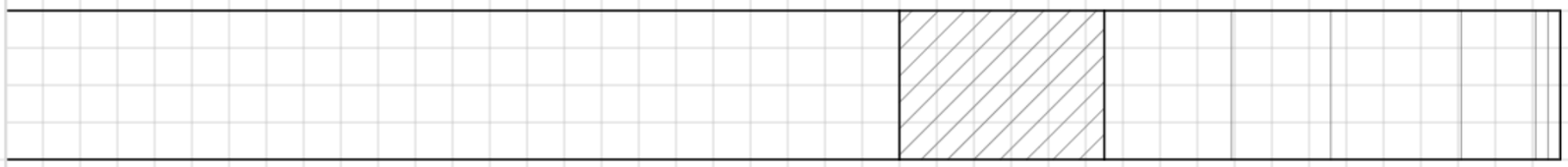
document-name	Location ID	Carb. Depth (in)	Comments
SA_07.pdf	**Carb-01**		1/8 to 1/4



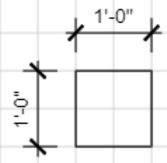
Study Area SA-07_Notes



FB-8
NORTH ELEVATION



FB-8 UNDERSIDE



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Pittsburgh | Philadelphia | Portland | Princeton | Raleigh | San Antonio
San Diego | San Francisco | Seattle | South Florida | Washington, DC

Consultants

Project

Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client

HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

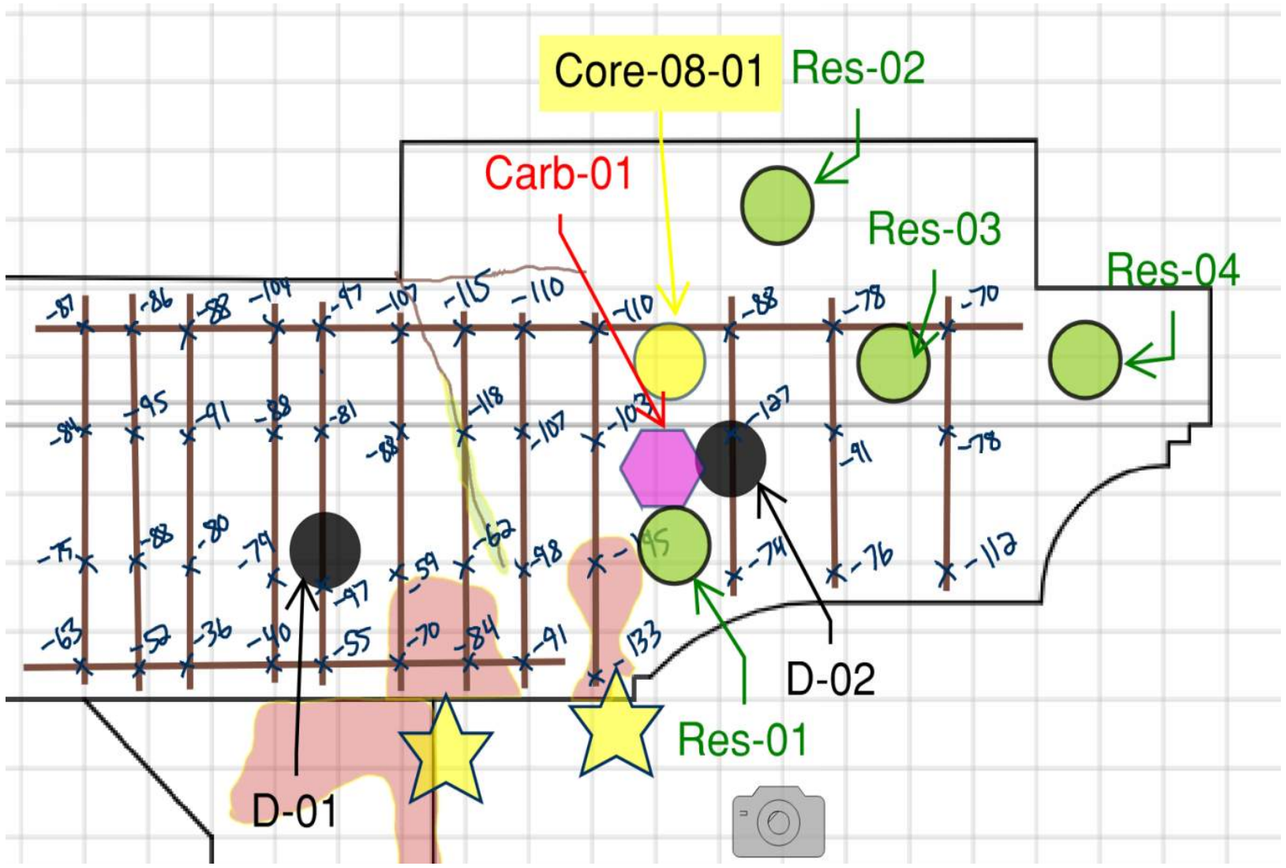
Mark	Date	Description

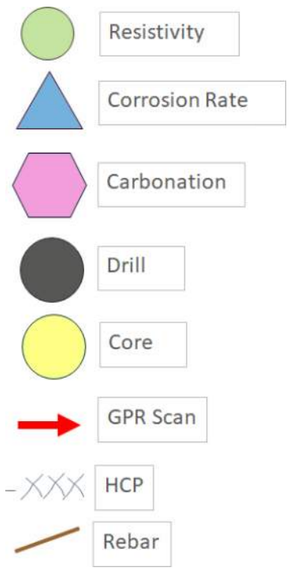
0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 8

Sheet No.: **08**





HCP Readings (as recorded and shown on sketch)

-87	-86	-88	-104	-97	-107	-115	-110	-110	-88	-78	-70
-84	-95	-91	-88	-81	-88	-118	-107	-103	-127	-91	-78
-75	-88	-80	-74	-97	-59	-62	-98	-145	-74	-76	-112
-63	-52	-36	-40	-55	-70	-84	-91	-133			

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	45
HCP Correction (mV)	-15

HCP Readings (corrected to 75 deg F)

-102	-101	-103	-119	-112	-122	-130	-125	-125	-103	-93	-85
-99	-110	-106	-103	-96	-103	-133	-122	-118	-142	-106	-93
-90	-103	-95	-89	-112	-74	-77	-113	-160	-89	-91	-127
-78	-67	-51	-55	-70	-85	-99	-106	-148			

Study Area SA-08_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_08.pdf	**Res-03**						299	234	204	246
SA_08.pdf	**Res-04**						114	98	98	103
SA_08.pdf	**Res-01**						153	189	207	183
SA_08.pdf	**Res-02**						537	613	657	602

Drill Holes

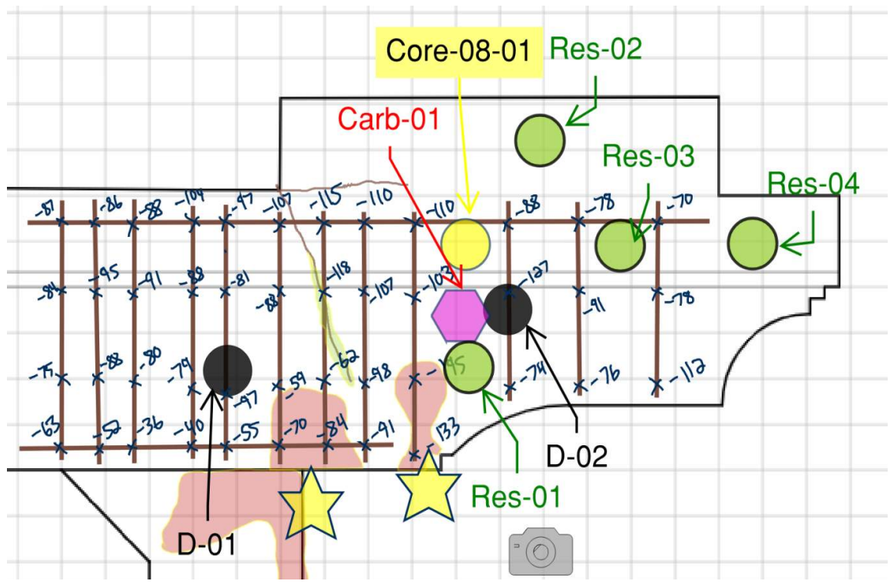
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_08.pdf	**D-01**	10/15/2024	Continuous with column bar, 0 mV, 0.8 ohm. Discontinuous with other drill hole and exposed stirrup 70mV, 700 kOhm. Double ground for half cell	1.75	
SA_08.pdf	**D-02**	10/15/2024	Continuous with exposed stirrup, 4 mV, 70 ohm. Discontinuous with other drill hole and exposed column bar 70mV, 700 kOhm. Double ground for half cell	1.625	

Corrosion Rate - iCOR

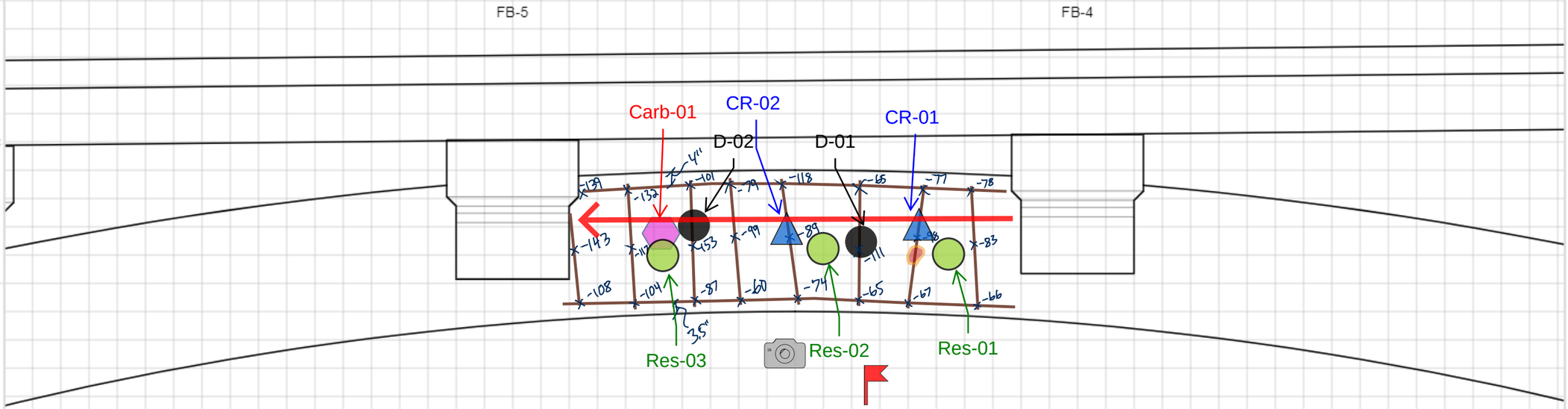
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used

Carbonation

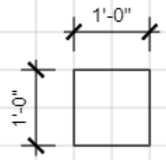
document-name	Location ID	Carb. Depth (in)	Comments
SA_08.pdf	**Carb-01**	0	Negligible



Study Area SA-08_Notes



UPSTREAM RIB
EXTERIOR FACE



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Consultants

Project

Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client

HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

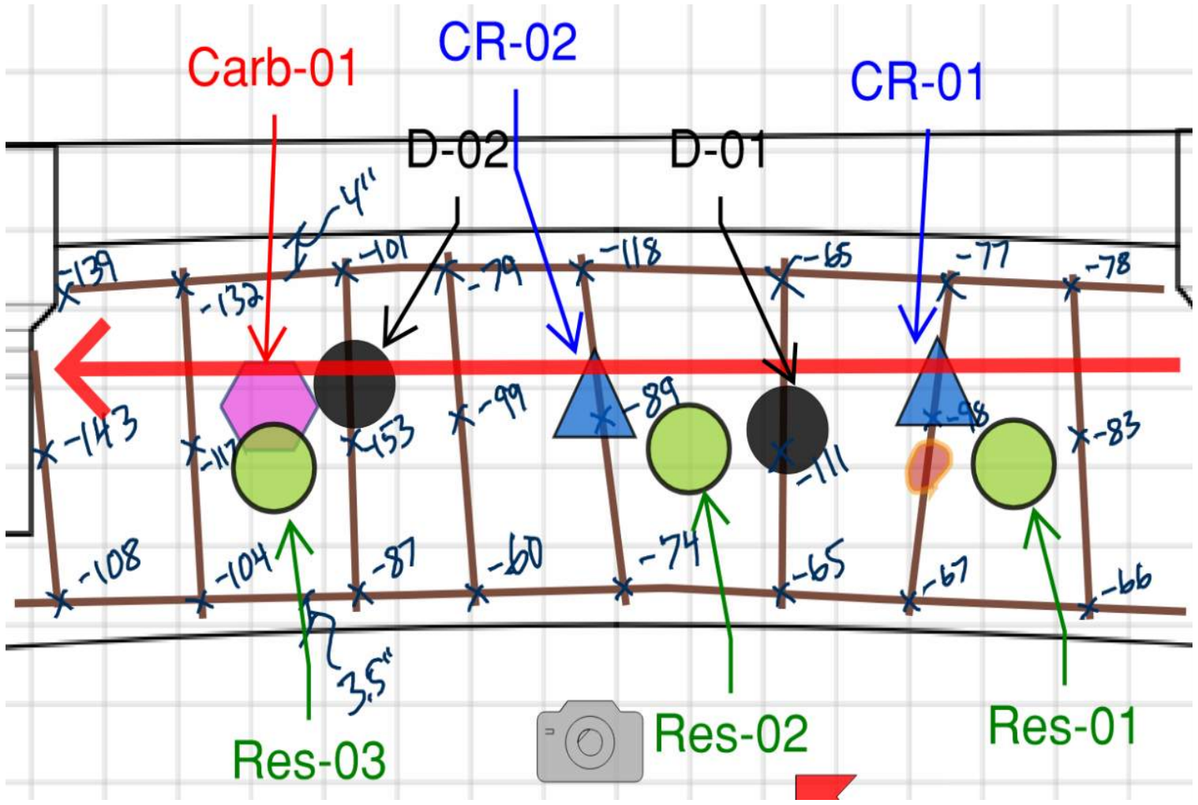
Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

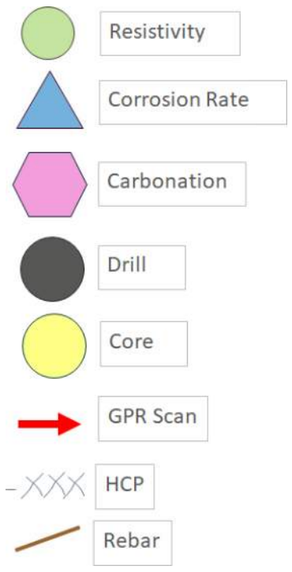
Sheet Title: Study Area 9

Sheet No.: **09**

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Plotted: 10/9/2024 11:11 AM by Swenson, Tanner File Name: z:\NB\2023\2023.7xxx\2023.7260.0 - bridge 55 over mill brook (apj)\inspection plan\field sheets\dwg\Sheets\09 Study Area 9.dwg





HCP Readings (as recorded and shown on sketch)

-139	-132	-101	-79	-118	-65	-77	-78
-143	-117	-153	-99	-89	-111	-98	-83
-108	-104	-87	-60	-74	-65	-67	-66

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	47
HCP Correction (mV)	-14

HCP Readings (corrected to 75 deg F)

-153	-146	-115	-93	-132	-79	-91	-92
-157	-131	-167	-113	-103	-125	-112	-97
-122	-118	-101	-74	-88	-79	-81	-80

Study Area SA-09_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_09.pdf	**Res-01**						99	113	106	106
SA_09.pdf	**Res-02**						144	146	143	144
SA_09.pdf	**Res-03**						84	89	87	87

Drill Holes

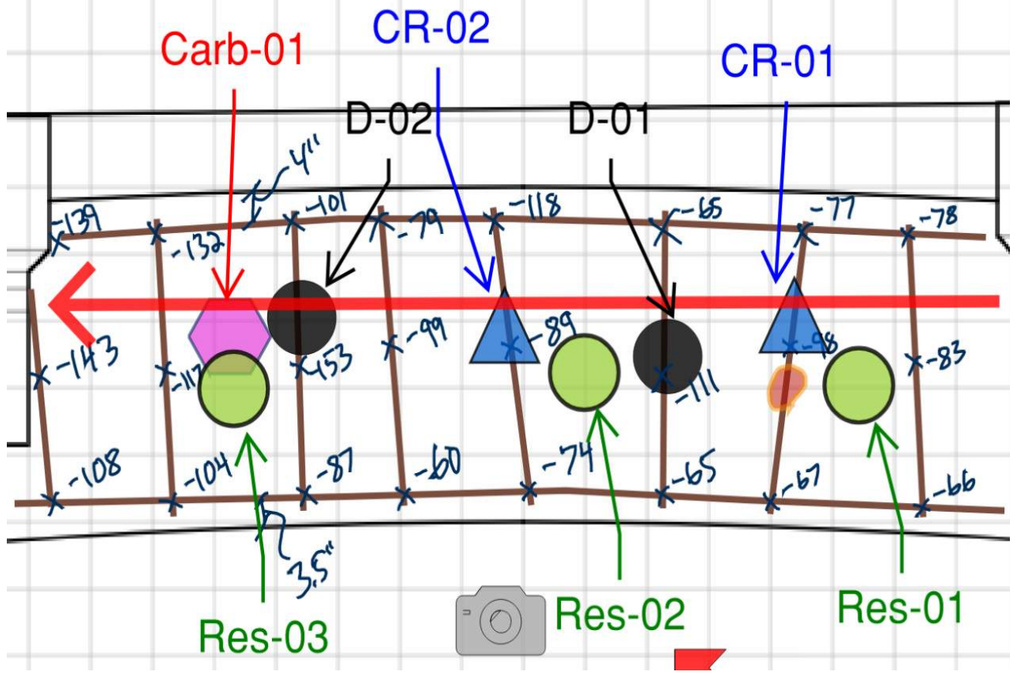
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_09.pdf	**D-02**	10/15/2024		2.375	0.3
SA_09.pdf	**D-01**	10/15/2024		2	0.3

Corrosion Rate - iCOR

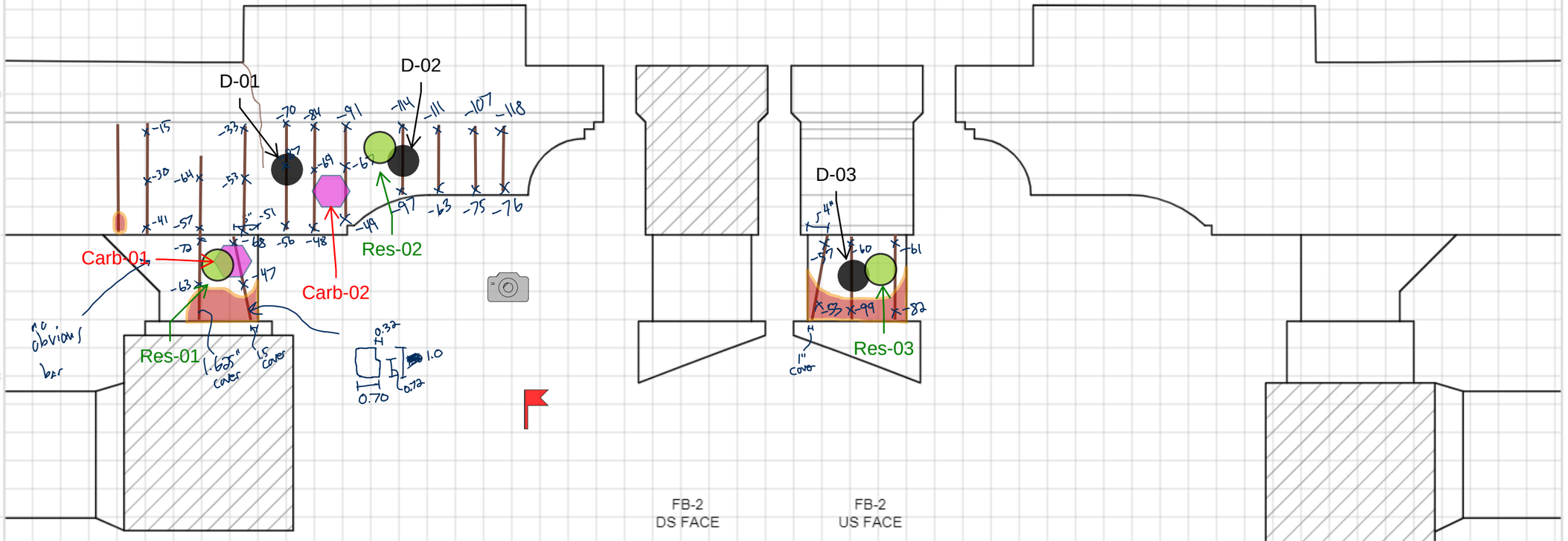
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used
SA_09.pdf	**CR-01**	10/15/2024	ERROR mA/cm^2147 kOhm-cm	Yes
SA_09.pdf	**CR-02**	10/15/2024	0.1 mA/cm^2172 kOhm-cm	Yes

Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_09.pdf	**Carb-01**	0.125	



Study Area SA-09_Notes

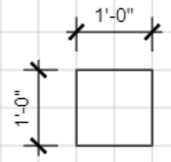


FB-2
NORTH ELEVATION

FB-2
DS FACE

FB-2
US FACE

FB-2
SOUTH ELEVATION



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Consultants

Project
Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client
HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

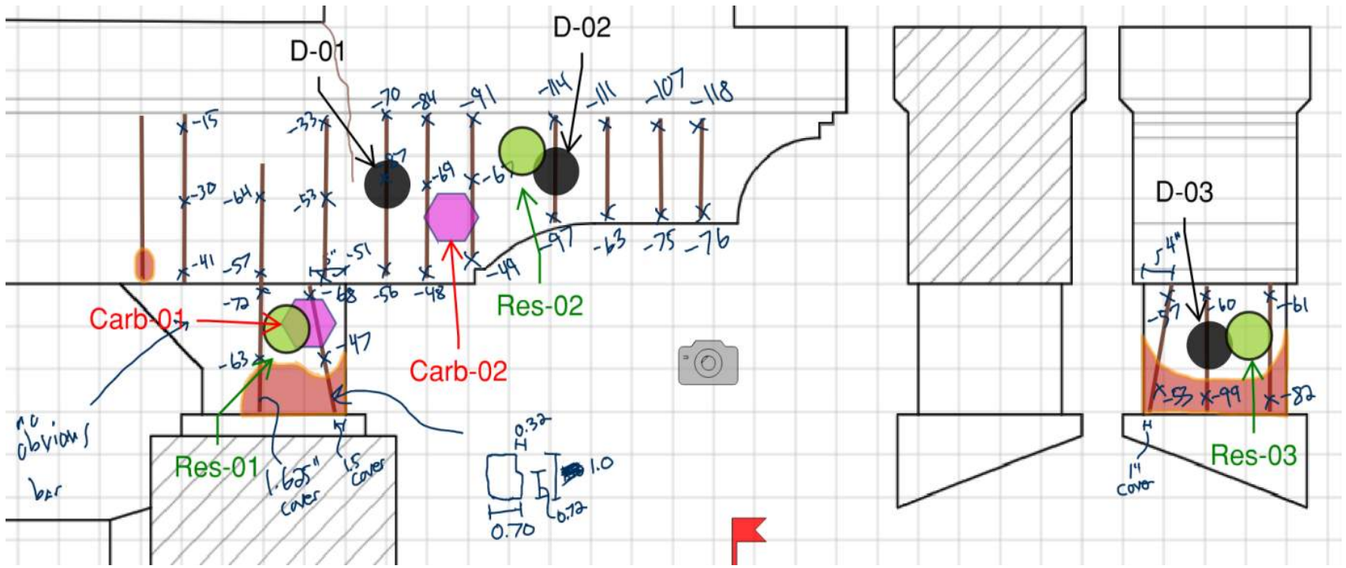
Mark	Date	Description

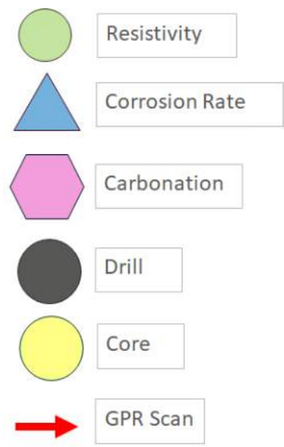
0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 10

Sheet No.: **10**





HCP Readings (as recorded and shown on sketch)

-15		-33	-70	-84	-91	-114	-111	-107	-118
-30	-64	-53	-87	-69	-67	-97	-63	-75	-76
-41	-57	-51	-56	-48	-49				
	-72	-68							
	-63	-47							

-57	-60	-61
-53	-99	-82

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	49
HCP Correction (mV)	-13

HCP Readings (corrected to 75 deg F)

-28		-46	-83	-97	-104	-127	-124	-120	-131
-43	-77	-66	-100	-82	-80	-110	-76	-88	-89
-54	-70	-64	-69	-61	-62				
	-85	-81							
	-76	-60							

-70	-73	-74
-66	-112	-95

Study Area SA-10_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_10.pdf	**Res-03**						461	358	355	391
SA_10.pdf	**Res-01**						96	90	73	86
SA_10.pdf	**Res-02**						220	214	153	196

Drill Holes

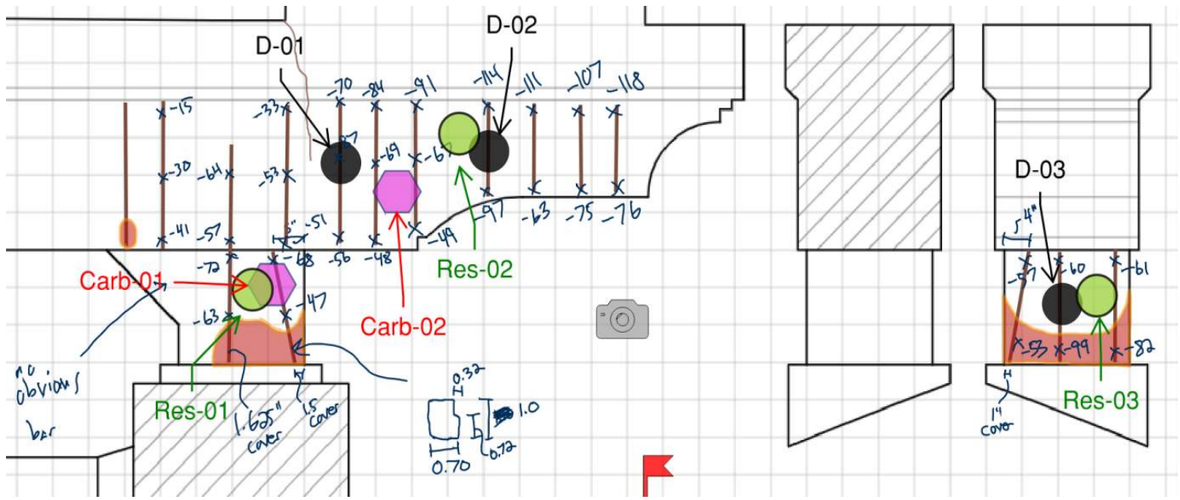
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_10.pdf	**D-01**	10/15/2024		1	
SA_10.pdf	**D-02**	10/15/2024	0.8 mV, 50 Ohm to left hole	1.375	
SA_10.pdf	**D-03**	10/15/2024	190 Ohm 41 mV to right FB hole, 141 Ohm 39 mV to left hole	3	

Corrosion Rate - iCOR

document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used

Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_10.pdf	**Carb-01**	0	Negligible
SA_10.pdf	**Carb-02**	0.125	



Study Area SA-10_Notes

Res-12-01
Res-12-02
Res-12-03
D-12-01
D-12-02
Core-11-02
Core-11-01
Core-11-03
Res-11-01
Res-11-02
Res-11-03
D-11-01
D-11-02
FB-7
FB-6
FB-5
7'-8"
2'-4"
7'-8"
CL ROADWAY
CLUS ARCH
1'-0"
1'-0"

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Bridge 55 Over Mill Brook
Windsor, VT 05089

Client
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3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

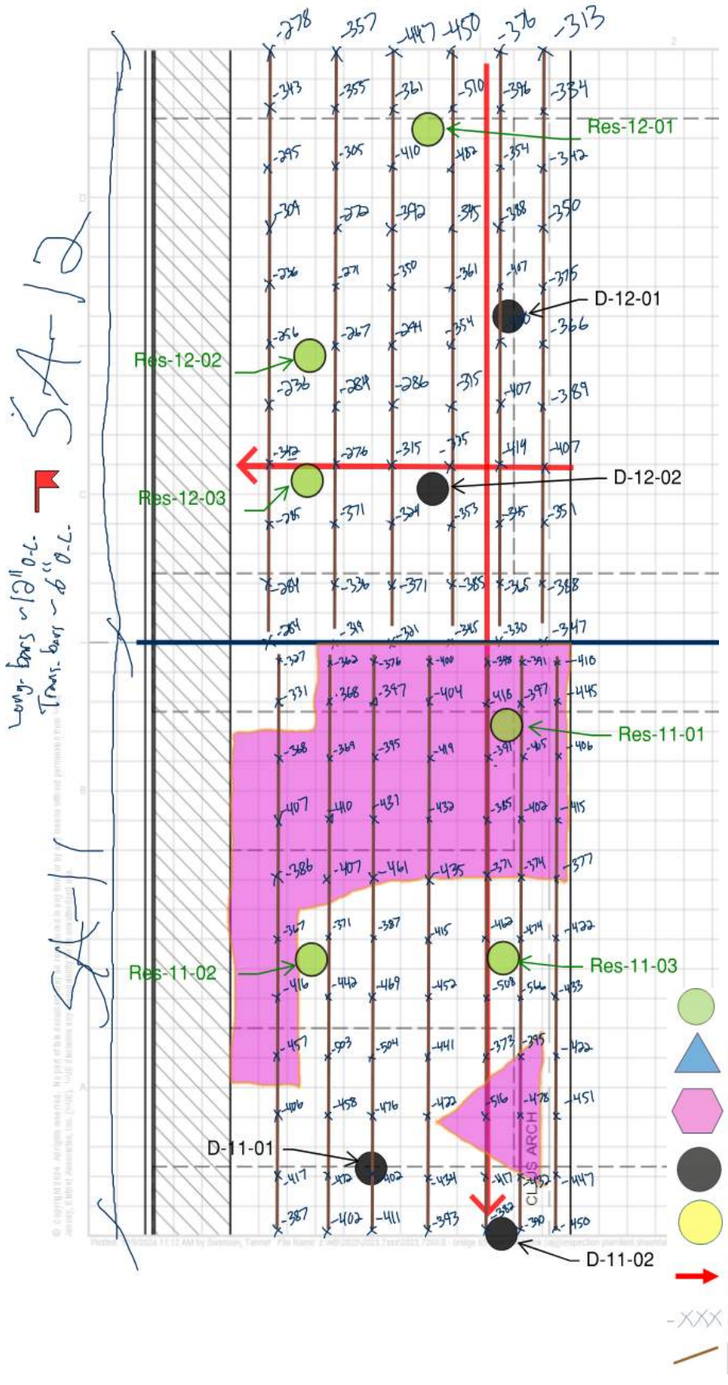
Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 11

Sheet No.: 11

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Plotted: 10/9/2024 11:12 AM by Swenson, Tanner File Name: I:\WJE\2023\2023.7xxx\2023.7260.0 - bridge 55 mill brook (apj)\inspection plan\field sheets\dwg\Sheets\11 Study Area 11.dwg



HCP Readings (as recorded and shown on sketch)

-278	-357	-447	-450	-376	-313
-343	-355	-361	-510	-396	-334
-295	-305	-410	-482	-354	-342
-309	-272	-392	-345	-388	-350
-236	-271	-350	-361	-407	-375
-256	-267	-294	-354	-400	-366
-236	-284	-286	-315	-407	-389
-312	-276	-315	-335	-419	-407
-285	-371	-324	-353	-345	-351
-284	-336	-371	-385	-365	-388
-284	-319	-321	-345	-330	-347

HCP Readings (corrected to 75 deg F)

-292	-371	-461	-464	-390	-327
-357	-369	-375	-524	-410	-348
-309	-319	-424	-496	-368	-356
-323	-286	-406	-359	-402	-364
-250	-285	-364	-375	-421	-389
-270	-281	-308	-368	-414	-380
-250	-298	-300	-329	-421	-403
-326	-290	-329	-349	-433	-421
-299	-385	-338	-367	-359	-365
-298	-350	-385	-399	-379	-402
-298	-333	-335	-359	-344	-361

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	48
HCP Correction (mV)	-13.5

HCP Readings (as recorded and shown on sketch)

-327	-362	-376	-400	-348	-391	-410
-331	-368	-397	-404	-418	-397	-445
-368	-369	-395	-419	-391	-405	-406
-407	-410	-431	-432	-385	-402	-415
-386	-407	-461	-435	-371	-374	-377
-367	-371	-387	-415	-462	-474	-422
-416	-442	-469	-452	-508	-566	-433
-457	-503	-504	-441	-373	-395	-422
-406	-458	-476	-422	-516	-478	-451
-417	-472	-402	-434	-417	-432	-447
-387	-402	-411	-393	-382	-390	-450

HCP Readings (corrected to 75 deg F)

-341	-376	-390	-414	-362	-405	-424
-345	-382	-411	-418	-432	-411	-459
-382	-383	-409	-433	-405	-419	-420
-421	-424	-445	-446	-399	-416	-429
-400	-421	-475	-449	-385	-388	-391
-381	-385	-401	-429	-476	-488	-436
-430	-456	-483	-466	-522	-580	-447
-471	-517	-518	-455	-387	-409	-436
-420	-472	-490	-436	-530	-492	-465
-431	-486	-416	-448	-431	-446	-461
-401	-416	-425	-407	-396	-404	-464

Study Area SA-11 and 12_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_11.pdf	**Res-12-02**					Yes	134	135	117	129
SA_11.pdf	**Res-12-03**						85	81	79	82
SA_11.pdf	**Res-12-01**					Yes	71	57	48	59
SA_11.pdf	**Res-11-01**						171	157	158	162
SA_11.pdf	**Res-11-02**						20	20	25	22
SA_11.pdf	**Res-11-03**					Yes	36	38	33	36

Drill Holes

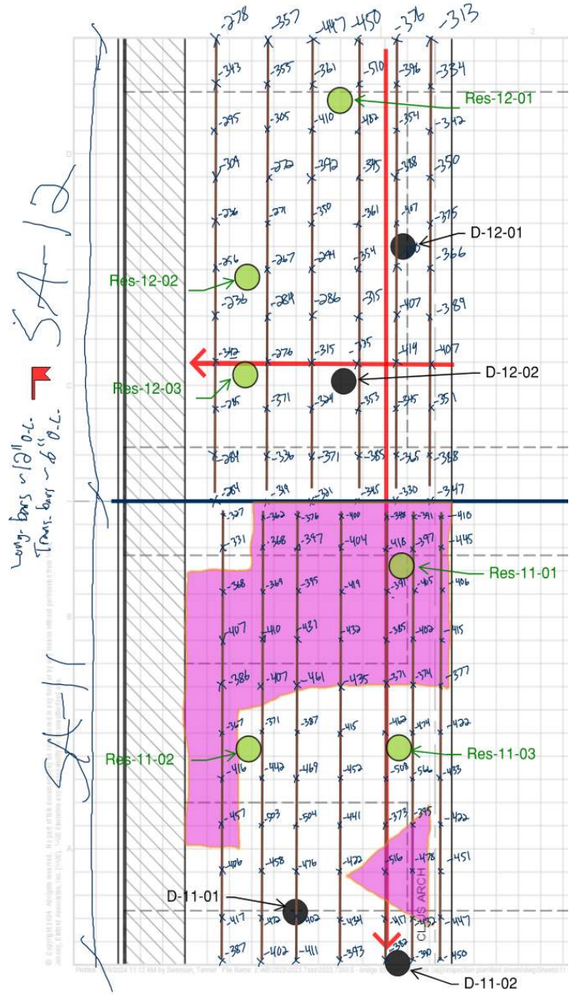
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_11.pdf	**D-12-01**			2	
SA_11.pdf	**D-12-02**			2.75	
SA_11.pdf	**D-11-01**	10/16/2024		2.25	
	D-11-02		On transverse bar, 8 Ohm 9mV to nearest hole		
SA_11.pdf			30 Ohm 1 mV to SA-12 holes	2.25	

Corrosion Rate - iCOR

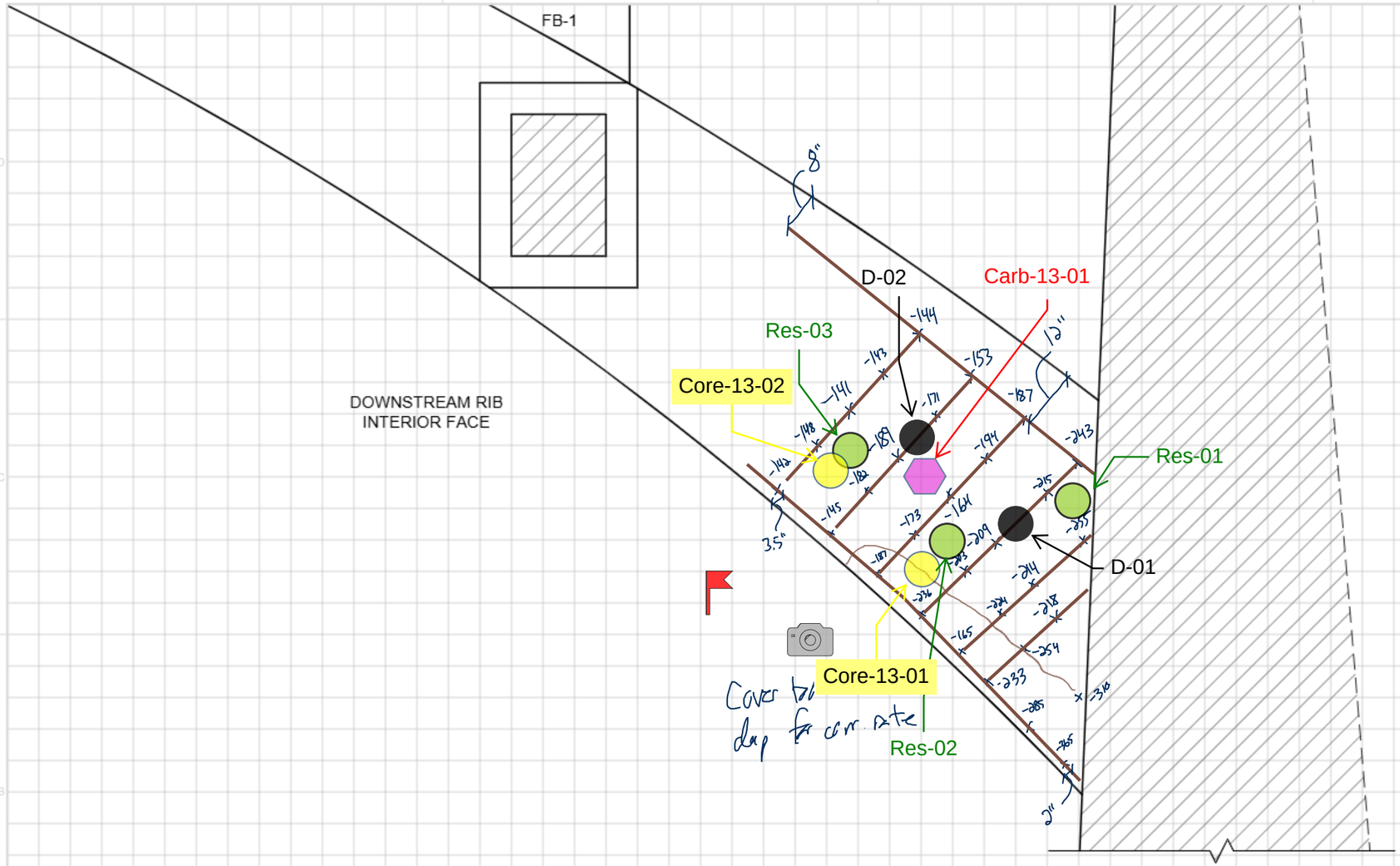
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used

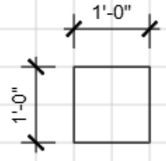
Carbonation

document-name	Location ID	Carb. Depth (in)	Comments



Study Area SA-11 and 12_Notes





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Consultants

Project

Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client

HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

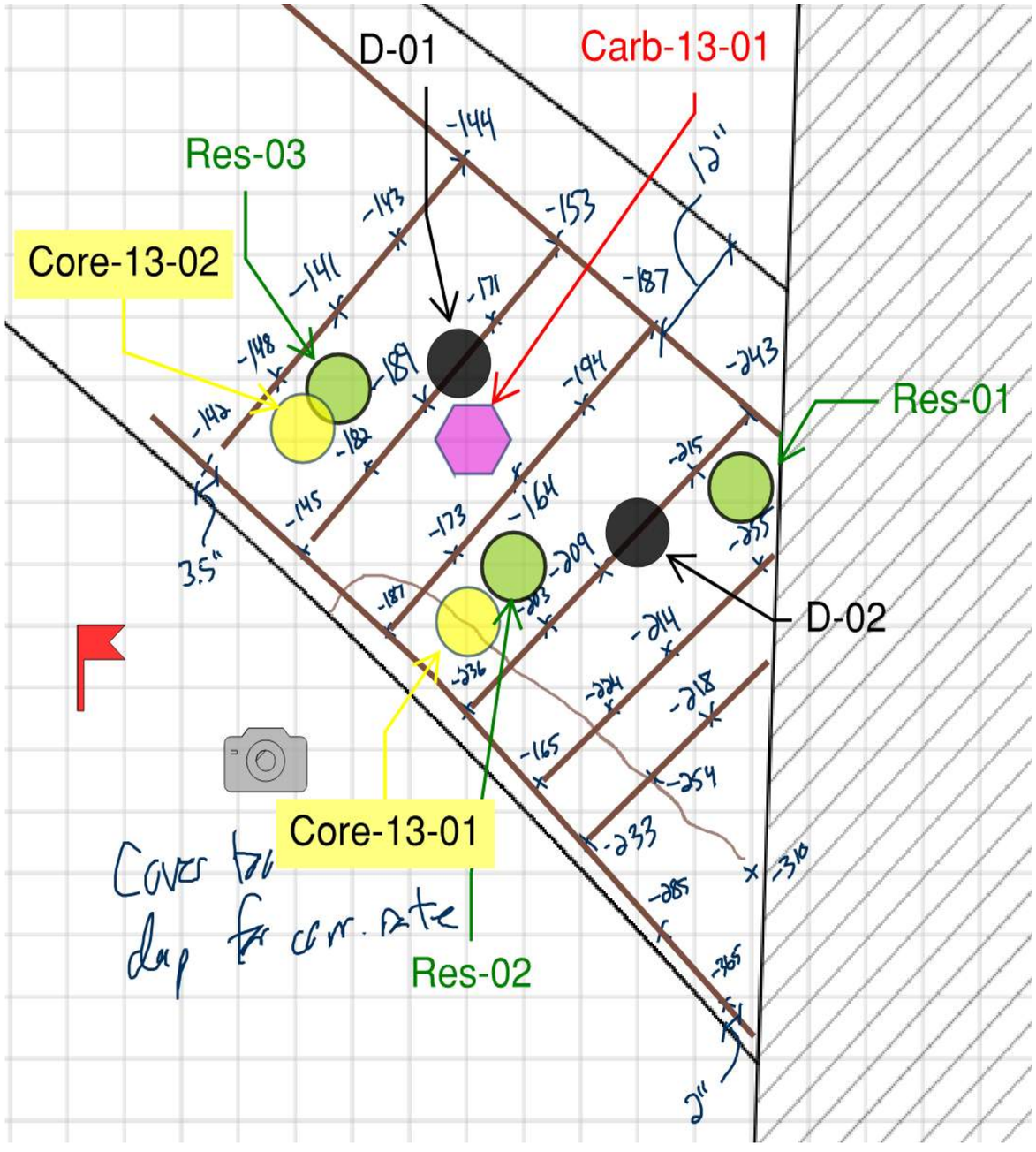
Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 13

Sheet No.: 13

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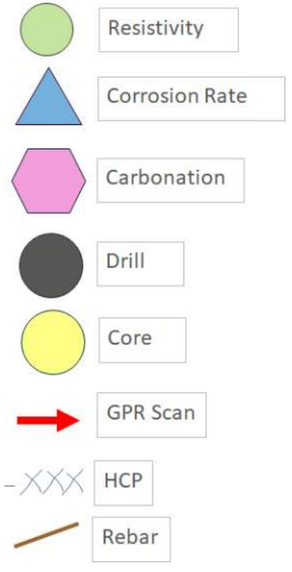
HCP Readings (as recorded and shown on sketch)

-144	-153	-187	-243				
-143	-171	-194	-215	-255			
-141	-189	-164	-209	-214	-218		
-148	-182	-173	-203	-224	-254	-310	
-142	-145	-187	-236	-165	-233	-285	-365

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	49
HCP Correction (mV)	-13

HCP Readings (corrected to 75 deg F)

-157	-166	-200	-256				
-156	-184	-207	-228	-268			
-154	-202	-177	-222	-227	-231		
-161	-195	-186	-216	-237	-267	-323	
-155	-158	-200	-249	-178	-246	-298	-378



Study Area SA-13_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_13.pdf	**Res-01**	10/15/2024				Yes	83	65	64	71
SA_13.pdf	**Res-02**	10/15/2024				Yes	93	71	85	83
SA_13.pdf	**Res-03**					Yes	68	74	58	67

Drill Holes

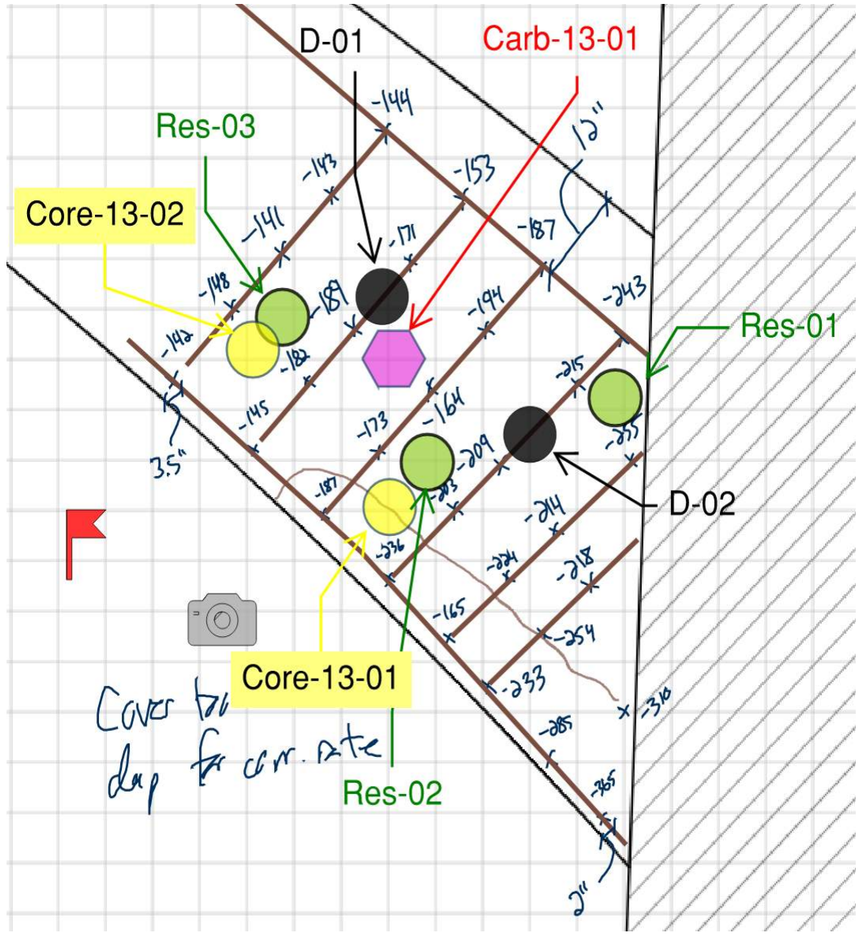
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_13.pdf	**D-01**			3.875	0.2
SA_13.pdf	**D-02**	10/15/2024		4	

Corrosion Rate - iCOR

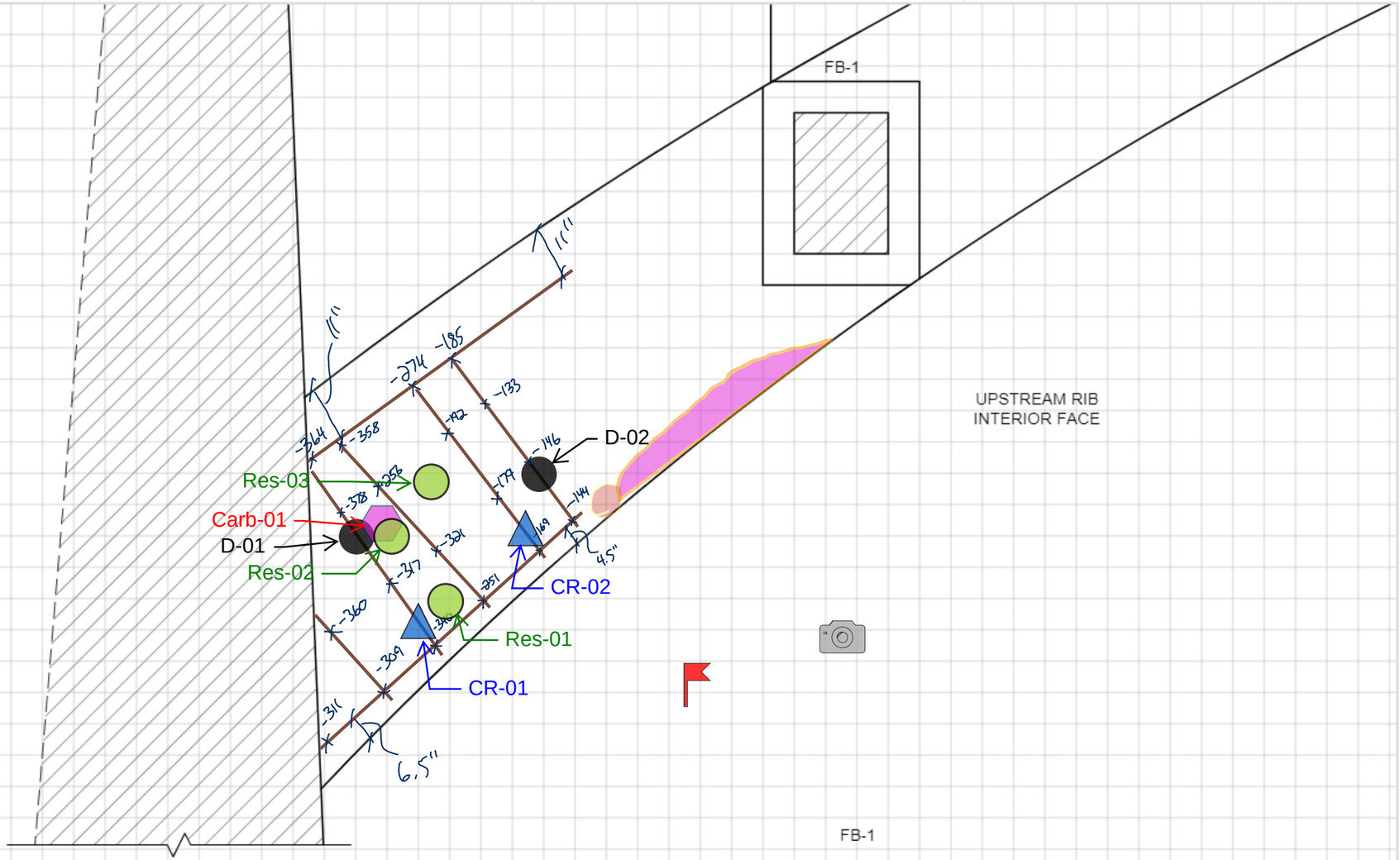
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used

Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_13.pdf	**Carb-01**	0	Negligible



Study Area SA-13_Notes



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Consultants

Project
Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client
HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

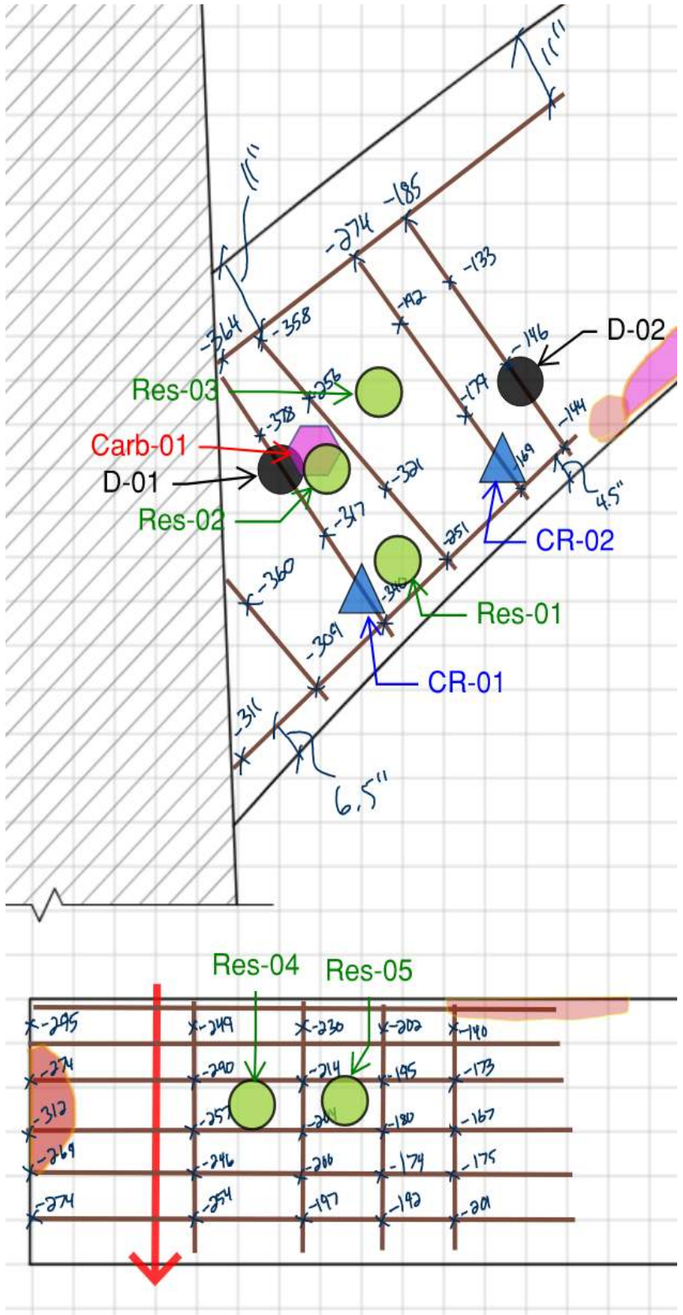
Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

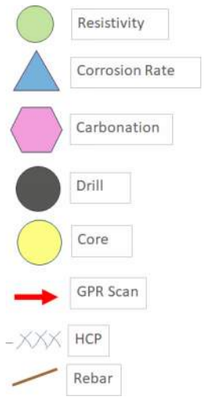
Sheet Title: Study Area 14

Sheet No.: 14

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Plotted: 10/9/2024 11:12 AM by Swenson, Tanner File Name: z:\NB\2023\2023.7xxx\2023.7260.0 - bridge 55 over mill brook (apj)\inspection plan\field sheets\dwg\Sheets\14 Study Area 14.dwg





HCP Readings (as recorded and shown on sketch)

	-364	-358	-274	-185
	-378	-256	-192	-133
-360	-317	-321	-179	-146
-311	-309	-340	-251	-169	-144

HCP Readings (corrected to 75 deg F)

	-379	-373	-289	-200
	-393	-271	-207	-148
-375	-332	-336	-194	-161
-326	-324	-355	-266	-184	-159

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	46
HCP Correction (mV)	-14.5

HCP Readings (as recorded and shown on sketch)

-295	-249	-230	-202	-140
-274	-290	-214	-195	-173
-312	-257	-204	-180	-167
-269	-246	-200	-174	-175
-274	-254	-197	-192	-201

HCP Readings (corrected to 75 deg F)

-310	-264	-245	-217	-155
-289	-305	-229	-210	-188
-327	-272	-219	-195	-182
-284	-261	-215	-189	-190
-289	-269	-212	-207	-216

Study Area SA-14_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_14.pdf	**Res-05**						47	60	73	60
SA_14.pdf	**Res-04**						64	51	61	59
SA_14.pdf	**Res-03**						141	124	143	136
SA_14.pdf	**Res-02**						85	94	90	90
SA_14.pdf	**Res-01**						60	74	77	70

Drill Holes

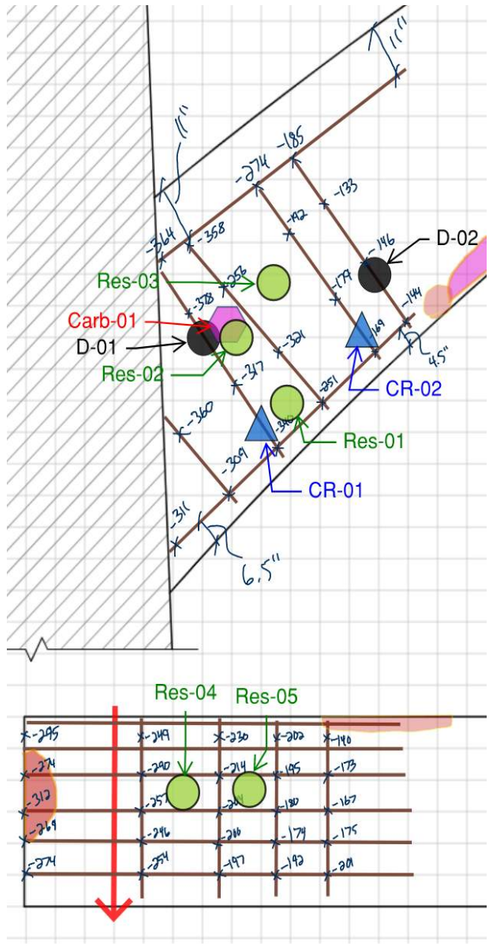
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_14.pdf	**D-02**			2.75	
SA_14.pdf	**D-01**	10/15/2024	3.5 ohm, 0 mV continuity	2.875	3.5

Corrosion Rate - iCOR

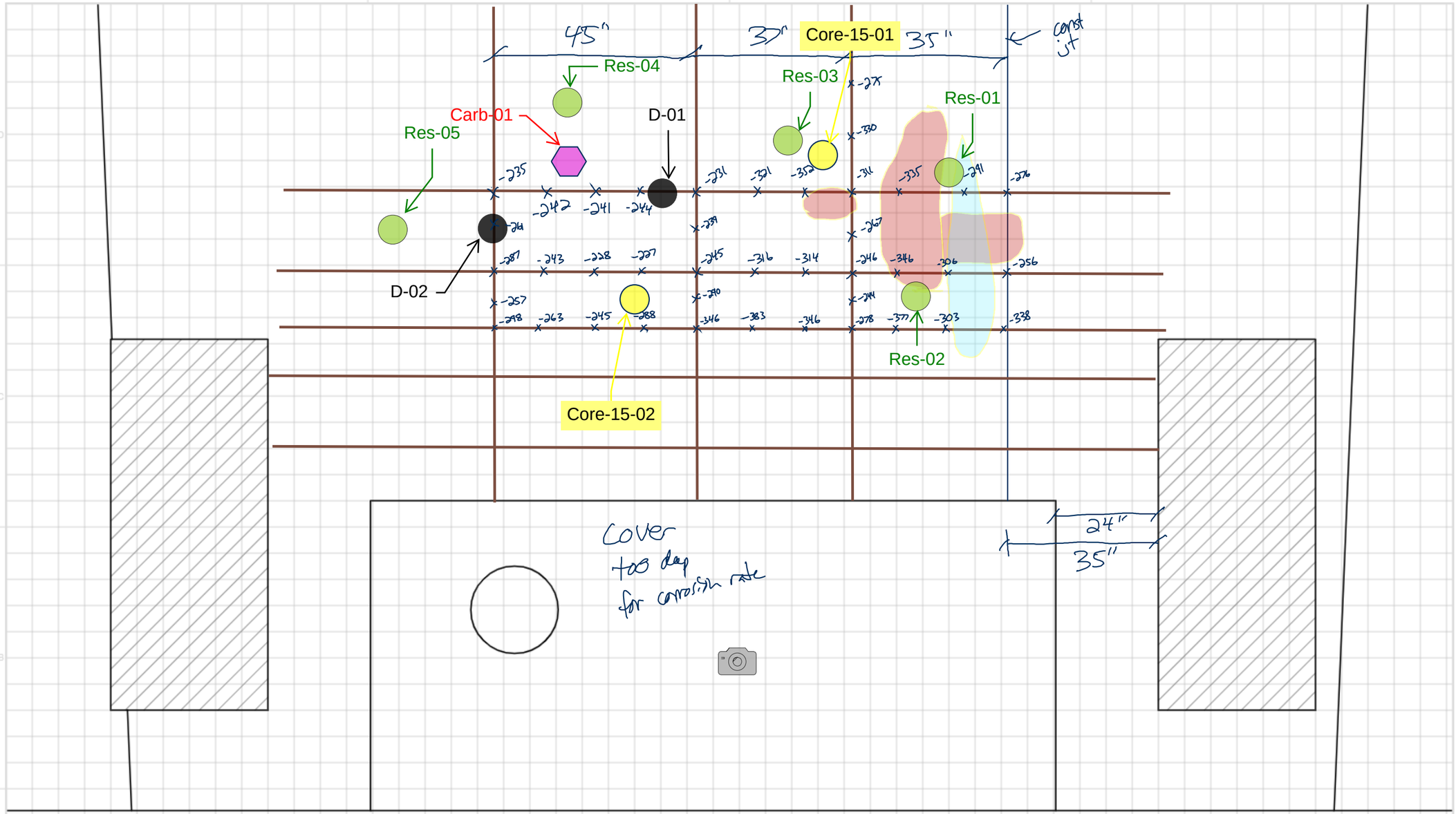
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used
SA_14.pdf	**CR-01**	10/15/2024	ERROR mA/cm^2ERROR kOhm-cm	Yes
SA_14.pdf	**CR-02**	10/15/2024	0.2 mA/cm^2142 kOhm-cm	Yes

Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_14.pdf	**Carb-01**	0	Negligible



Study Area SA-14_Notes



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Consultants

Project
Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client
HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

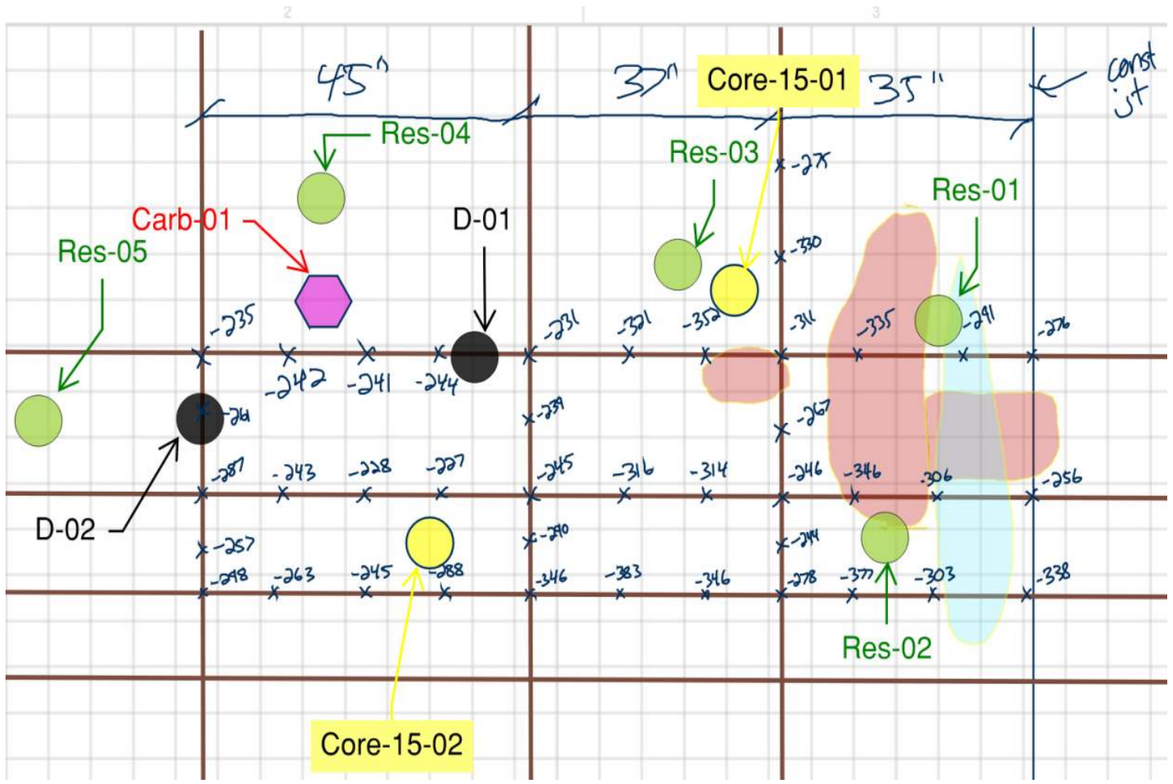
Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 15

Sheet No.: 15

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Plotted: 10/9/2024 11:12 AM by Swenson, Tanner File Name: z:\WJE\2023\2023.7xxx\2023.7260.0 - bridge 55 over mill brook (apj)\inspection plan\field sheets\dwg\Sheets\15 Study Area 15.dwg



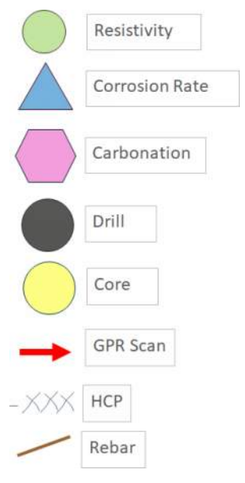
HCP Readings (as recorded and shown on sketch)

							-275			
							-330			
-235	-242	-241	-244	-231	-321	-352	-311	-335	-291	-276
-212				-239			-267			
-287	-243	-228	-227	-245	-316	-314	-246	-346	-306	-256
-257				-290			-244			
-298	-263	-245	-288	-346	-383	-346	-278	-377	-303	-338

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	44
HCP Correction (mV)	-15.5

HCP Readings (corrected to 75 deg F) **HCP Readings (corrected to 75 deg F)**

							-291			
							-346			
-251	-258	-257	-260	-247	-337	-368	-327	-351	-307	-292
-228				-255			-283			
-303	-259	-244	-243	-261	-332	-330	-262	-362	-322	-272
-273				-306			-260			
-314	-279	-261	-304	-362	-399	-362	-294	-393	-319	-354



Study Area SA-15_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_15.pdf	**Res-03**						32	34	28	31
SA_15.pdf	**Res-02**		Already wet			No	55	56	68	60
SA_15.pdf	**Res-01**	10/14/2024	Already wet			Other	94	115	146	118
SA_15.pdf	**Res-05**	10/14/2024		51		Yes	51	41	38	43
SA_15.pdf	**Res-04**	10/14/2024		51	44	Yes	70	74	67	70

Drill Holes

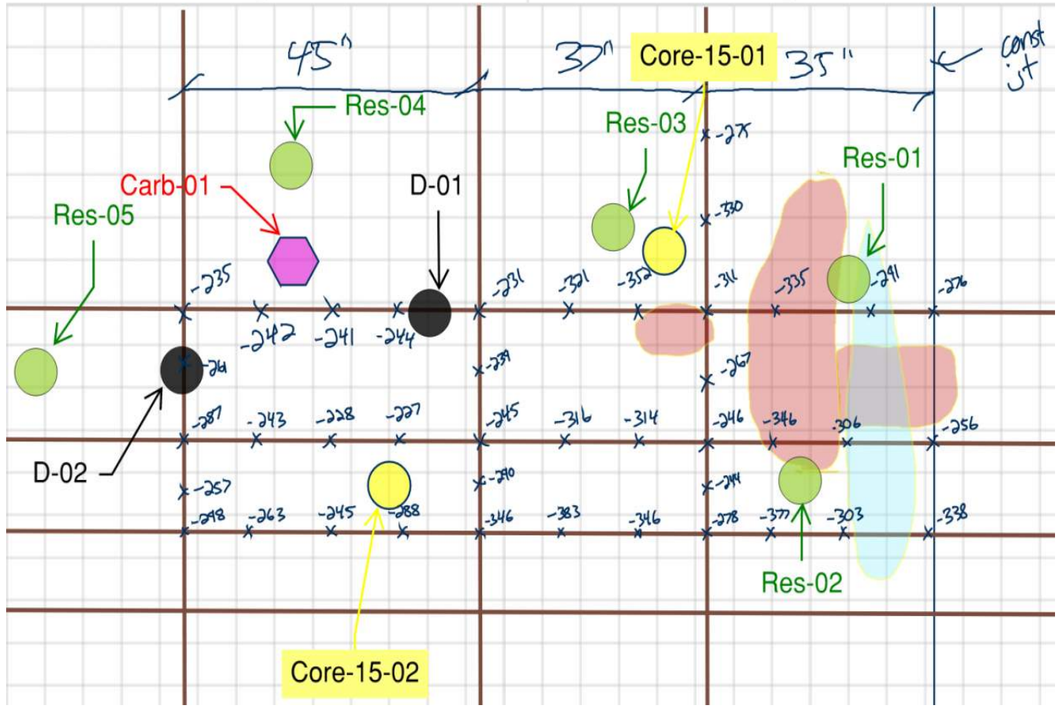
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_15.pdf	**D-01**			5	0.2
SA_15.pdf	**D-02**	10/14/2024		4	0.2

Corrosion Rate - iCOR

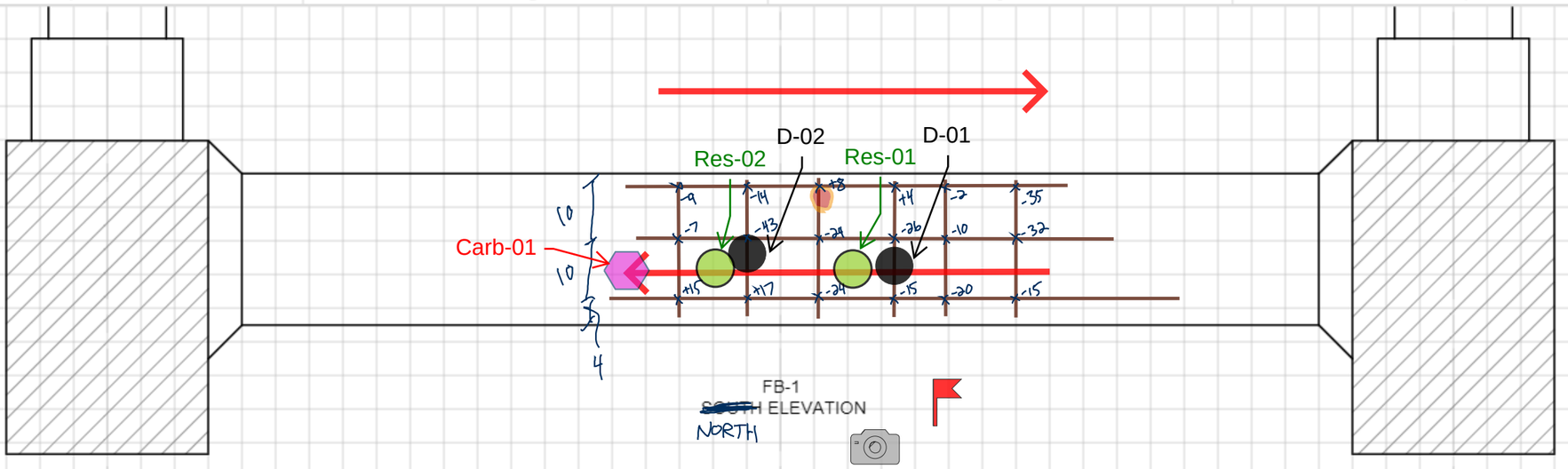
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used

Carbonation

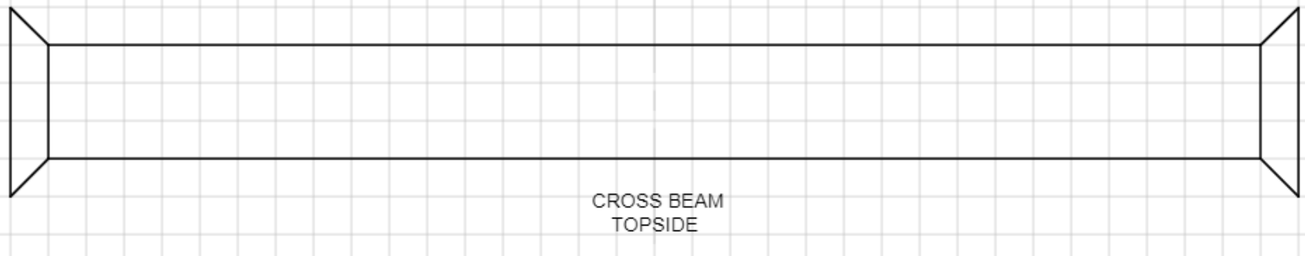
document-name	Location ID	Carb. Depth (in)	Comments
SA_15.pdf	**Carb-01**	0.25	



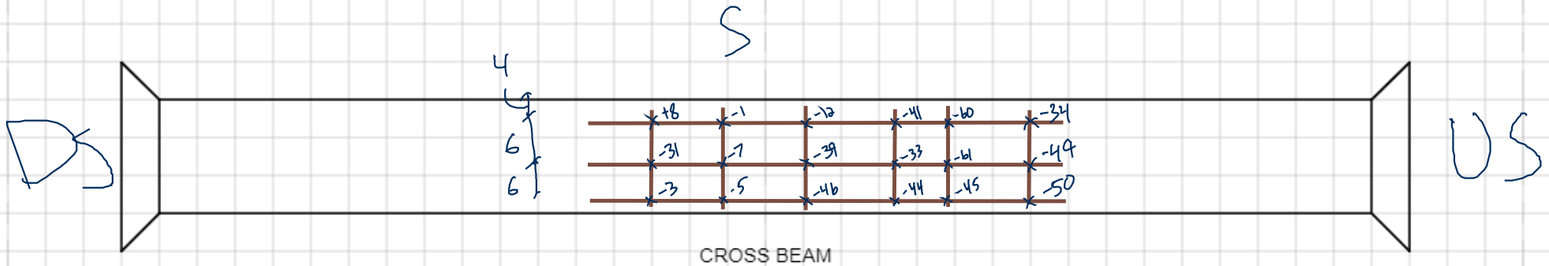
Study Area SA-15_Notes



FB-1
~~SOUTH~~ ELEVATION
NORTH

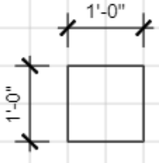


CROSS BEAM
TOPSIDE



CROSS BEAM
UNDERSIDE

N



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Project
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Bridge 55 Over Mill Brook
Windsor, VT 05089

Client
HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

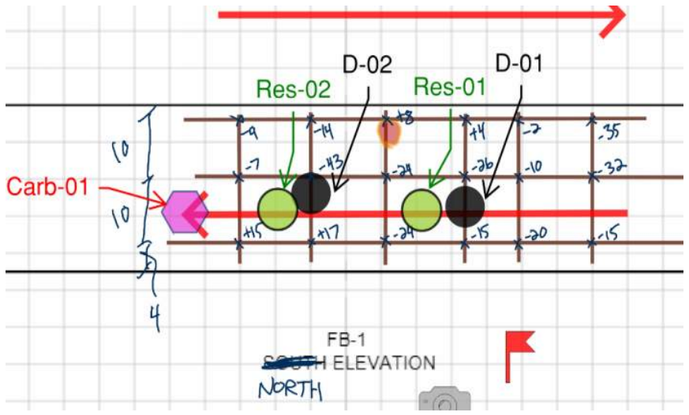
Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

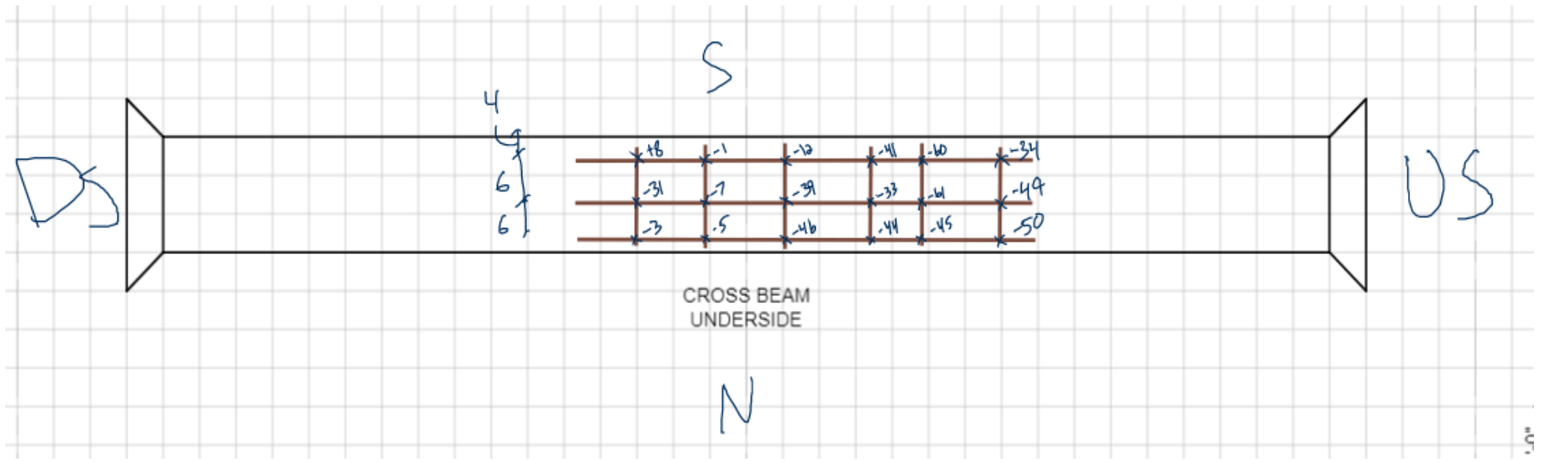
Sheet Title: Study Area 16

Sheet No.: **16**

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Plotted: 10/9/2024 11:13 AM by Swenson, Tanner File Name: z:\NB\2023\2023.7xxx\2023.7260.0 - bridge 55 over mill brook (apj)\inspection plan\field sheets\dwg\Sheets\16 Study Area 16.dwg



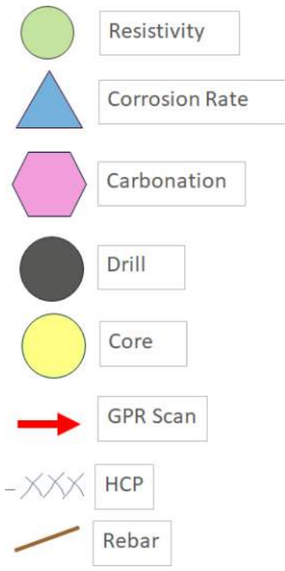


HCP Readings (as recorded and shown on sketch)

-9	-14	8	4	-2	-35
-7	-43	-24	-26	-10	-32
15	17	-24	-15	-20	-15

HCP Readings (as recorded and shown on sketch)

8	-1	-12	-41	-60	-34
-31	-7	-39	-33	-61	-49
-3	-5	-46	-44	-45	-50



Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	39
HCP Correction (mV)	-18

HCP Readings (corrected to 75 deg F)

-27	-32	-10	-14	-20	-53
-25	-61	-42	-44	-28	-50
-3	-1	-42	-33	-38	-33

HCP Readings (corrected to 75 deg F)

-10	-19	-30	-59	-78	-52
-49	-25	-57	-51	-79	-67
-21	-23	-64	-62	-63	-68

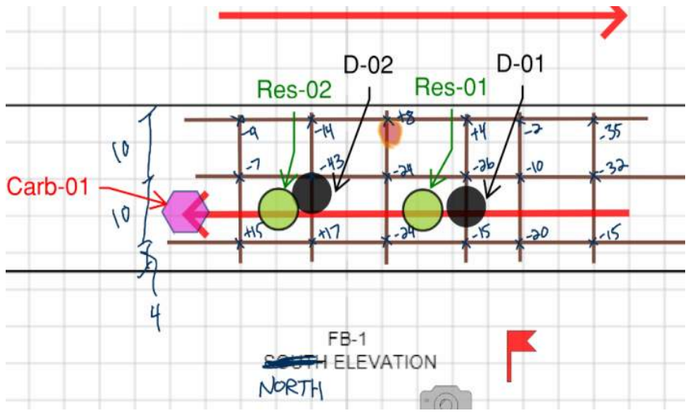
Study Area SA-16_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_16.pdf	**Res-02**						186	174	190	183
SA_16.pdf	**Res-01**	10/16/2024					167	172	160	166

Drill Holes

document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_16.pdf	**D-01**	10/16/2024		1.5	
SA_16.pdf	**D-02**	10/16/2024	40 ohm, 0.3 mV	0.75	

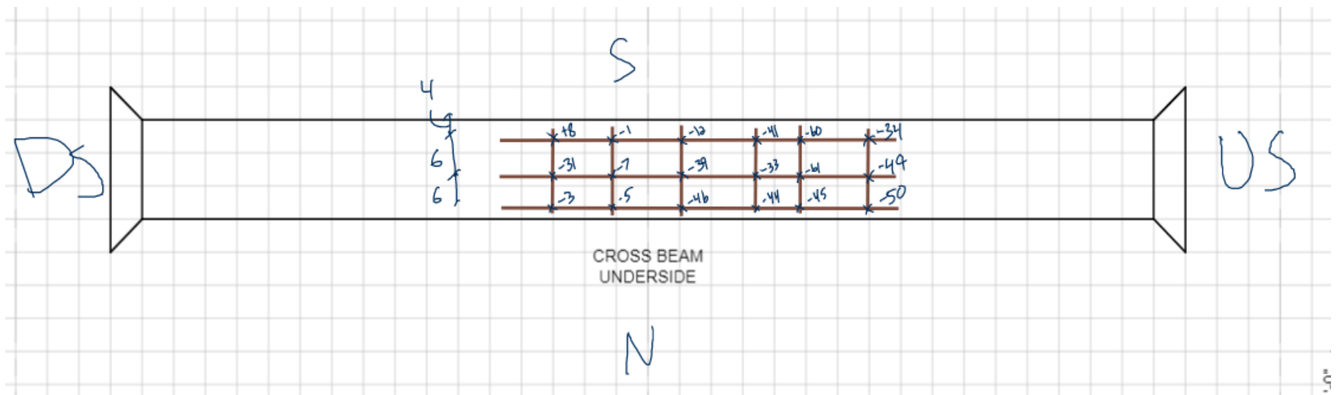


Corrosion Rate - iCOR

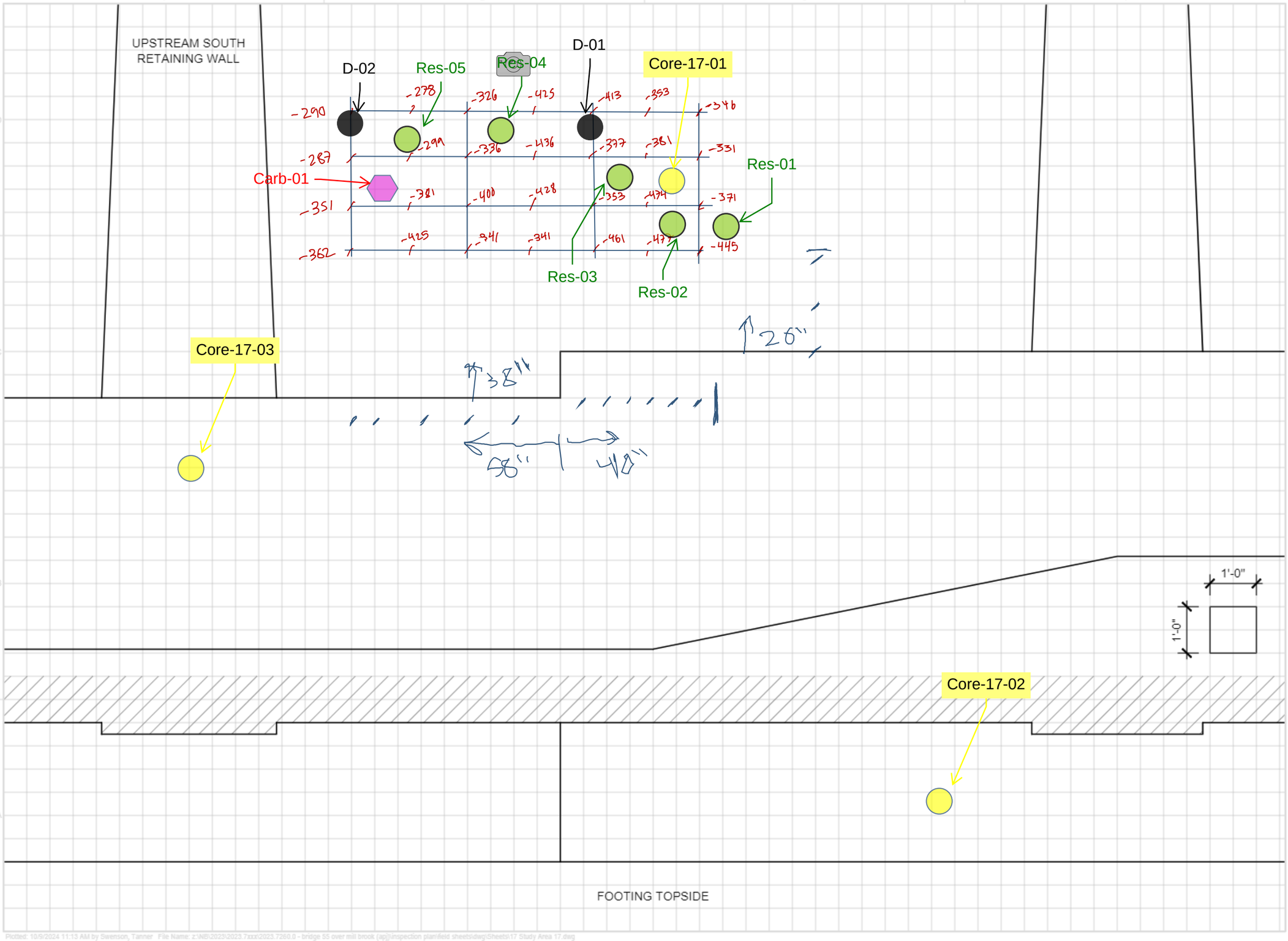
document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used

Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_16.pdf	**Carb-01**		0 to 0.125"



Study Area SA-16_Notes



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Project

Windsor Bridge 55
Bridge 55 Over Mill Brook
Windsor, VT 05089

Client

HNTB Corporation
3 Executive Park Drive
Bedford, NH 03110

Mark	Date	Description

0 1/2" 1"
THIS SHEET PLOTS FULL SIZE AT 11x17 (INCHES)

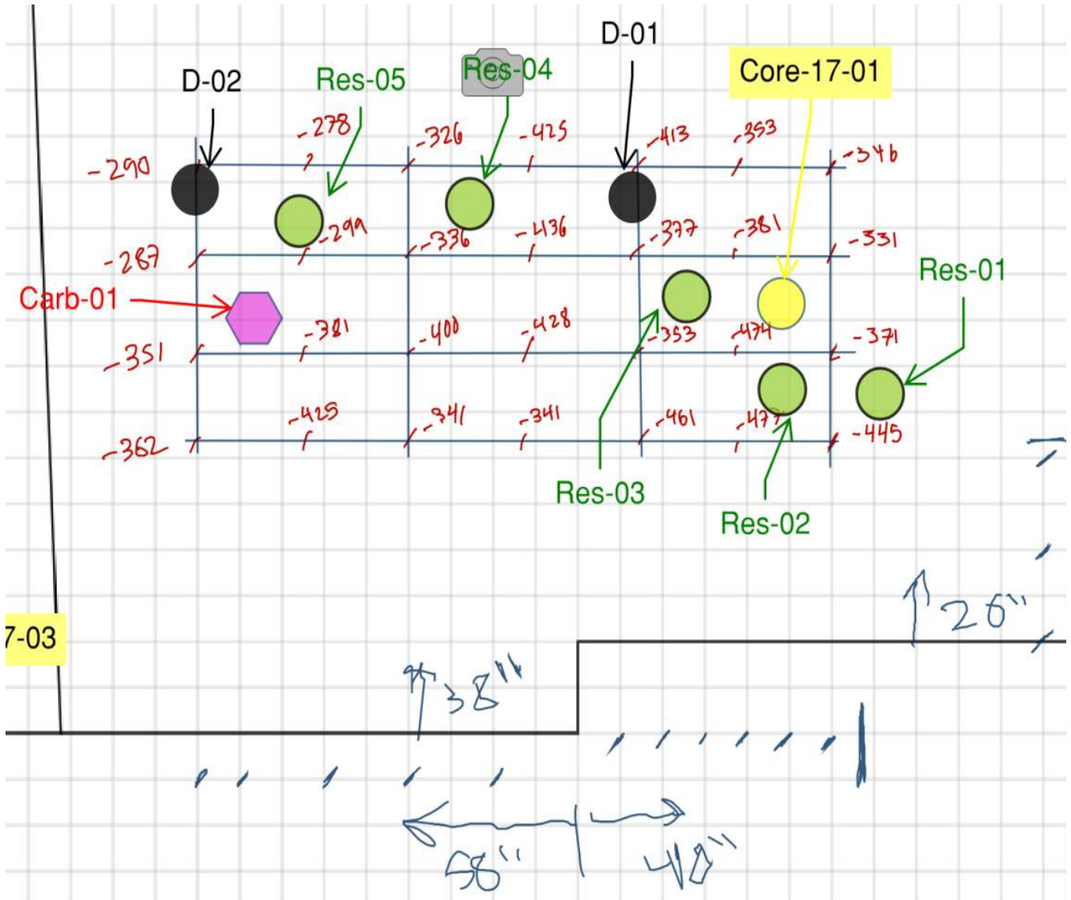
Project No.	2023.7260
Date	October 14, 2024
Drawn	TWS
Checked	APJ
Scale	1/2" = 1'-0"

Sheet Title: Study Area 17

Sheet No.: 17

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Plotted: 10/9/2024 11:13 AM by Swenson, Tanner File Name: z:\NB\2023\2023.7xxx\2023.7260.0 - bridge 55 over mill brook (ap)\inspection plan\field sheets\dwg\Sheets\17 Study Area 17.dwg



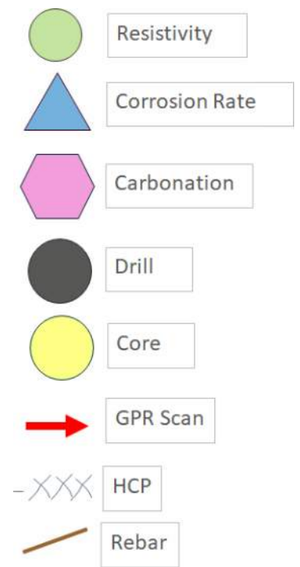
HCP Readings (as recorded and shown on sketch)

-290	-278	-326	-425	-413	-353	-346
-287	-299	-336	-436	-377	-381	-331
-351	-381	-400	-428	-353	-474	-371
-362	-425	-341	-341	-461	-477	-445

Correction Factor (mV/°F)	-0.5
Correction temp (°F)	75
Ambient temp at reading (°F)	45
HCP Correction (mV)	-15

HCP Readings (corrected to 75 deg F)

-305	-293	-341	-440	-428	-368	-361
-302	-314	-351	-451	-392	-396	-346
-366	-396	-415	-443	-368	-489	-386
-377	-440	-356	-356	-476	-492	-460



Study Area SA-17_HCP

Resitivity - Resipod

document-name	Location ID	Date Tested	Notes/ Locations	Surface Temperatue (deg F)	Ambient Temperature (deg F)	Pre-wetting Used	Reading 1 (kΩ-cm)	Reading 2 (kΩ-cm)	Reading 3 (kΩ-cm)	Avg.
SA_17.pdf	**Res-01**	10/16/2024					37	50	51	46
SA_17.pdf	**Res-02**	10/16/2024					60	59	54	58
			930am ET							
SA_17.pdf	**Res-05**	10/16/2024	Below distress	44	45	Yes	110	104	139	118
SA_17.pdf	**Res-04**	10/16/2024	Dry zone			Yes	105	123	88	105
SA_17.pdf	**Res-03**	10/16/2024	Below drain, lower resistivity than adjacent tests			Yes	47	48	36	44

Drill Holes

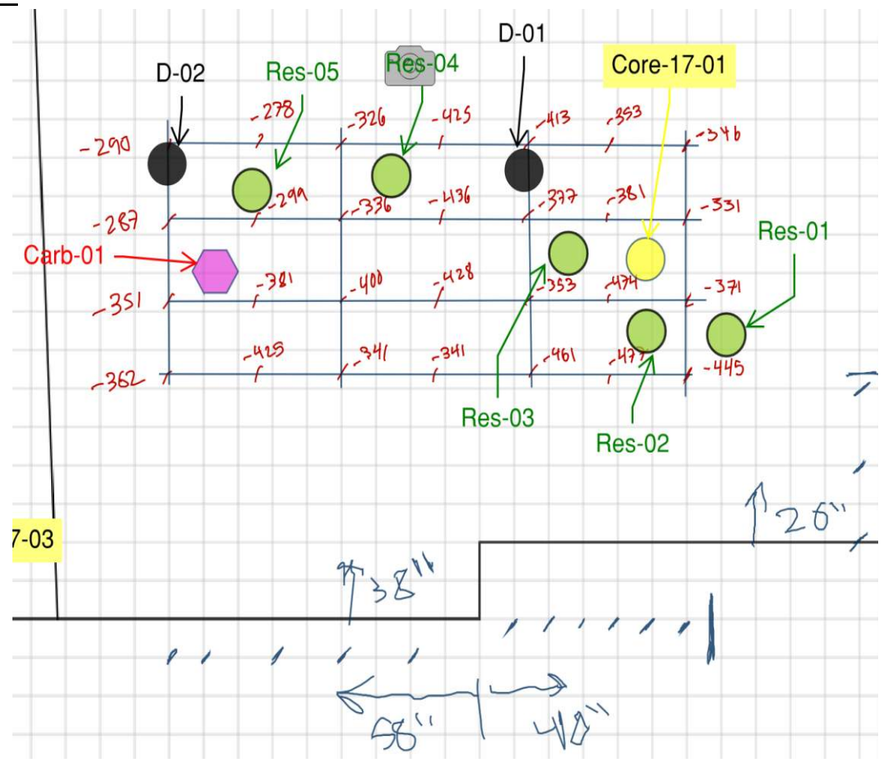
document-name	Location ID	Date Measured	Notes/ Locations	Measured Depth (in.)	Ground Resistance (ohm)
SA_17.pdf	**D-01**	10/16/2024		3.25	
SA_17.pdf	**D-02**	10/16/2024	0.2 ohm , 0 mV	3.25	0.2

Corrosion Rate - iCOR

document-name	Location ID	Date Tested	Notes/ Locations	Pre-wetting Used

Carbonation

document-name	Location ID	Carb. Depth (in)	Comments
SA_17.pdf	**Carb-01**	0.125	



Study Area SA-17_Notes

APPENDIX D. PETROGRAPHIC EXAMINATION REPORT

MEMORANDUM | January 9, 2025

BRIDGE 55 OVER MILL BROOK IN WINDSOR, VERMONT

Petrographic Examination of Concrete Cores

WJE PROJECT NO. 2023.7260

TO Tanner Swenson, Stephen Garrett, Arne Johnson

FROM Hugh Hou, Petrographer, Janney Technical Center
Corrie Piehowski, Project Associate, Janney Technical Center

As requested, petrographic examination was conducted on concrete cores extracted from different elements of the Windsor Bridge 55 on US Route 5 in Windsor, Vermont. The bridge was reportedly constructed in 1930 and repaired in 1943. Elements of the bridge are currently exhibiting various signs of distress including visible corrosion of exposed reinforcing steel, moisture-related staining and efflorescence, concrete cracking, and concrete spalling. WJE was retained to conduct a condition assessment of the bridge which included visual assessment, nondestructive evaluation, and material testing. Petrographic examination supplemented the field investigations and was conducted to assess the concrete quality and identify distress mechanisms, including but not limited to freeze-thaw distress, to aid in the condition assessment.

SAMPLES

Eighteen concrete cores were received at WJE's Janney Technical Center laboratory in Northbrook, Illinois on October 25, 2024. The cores were labeled by WJE staff following extraction in the field, and those assigned numbers are used throughout this memorandum. The cores were nominally 3-3/4 inches in diameter and varied in length. The cores were extracted at different conditions across the structure. The cores generally exhibited a formed or varying eroded exterior surface and a fractured interior surface that extended both around and through coarse aggregate particles.

As part of the broader material assessment, the cores were subjected to compressive strength testing, chloride ion content analysis, and petrographic examination. The general composition and condition for all eighteen cores samples was reviewed as received, and based on this incoming inspection cores were selected for more detailed tests. Of the eighteen cores, five were selected for detailed petrographic examination, including Cores 01-01, 08-01, 11-01, 13-01, and 15-01 (Table 1). Additionally, five cores were selected for limited petrographic examination, including Cores 17-01, 17-02, 17-03, 18-01, and 18-02 (Table 2). Only the exterior region was examined for four of the five cores that were subjected to detailed petrographic examination, as the remaining length was designated for compressive strength testing (Table 1). The full length was examined for four of the five cores selected for limited examination, primarily to evaluate extent of the apparent freeze-thaw distress (Table 2). Brief descriptions of the as-received cores selected for petrographic examination are provided in Table 1 and Table 2, and images of the cores are shown in Figure 1 through Figure 10.

Table 1. As-Received Condition of Concrete Cores for Detailed Petrographic Examination

Sample ID	Element/Coring Orientation	Length (inches)	Concrete Repair Present?	Macro Cracking Present?	Core Notes	Area Examined
01-01	Arch/Horizonal	12	No	No	Flat formed exterior surface	Exterior 2.5 inches
08-01	Floor Beam/Horizontal	10	No	No	Formed exterior surface with exposed sand; Voids adjacent to rebar and some coarse aggregate particles	Exterior 2 inches
11-01	Deck/Vertical	10.5, not including asphalt topping	Yes, 5.25 inches thick, reportedly a previous concrete repair	Yes, surface-parallel cracks in interior nominal 3.5 inches, through and around aggregate	Contains two concretes and asphalt overlay; waterproofing on top surface of repair concrete and bottom surface of substrate concrete; a cementitious layer underlying the bottom waterproofing on interior surface (Figure 3); Asphalt overlay nominally 2.25 inches thick, debonded (not examined)	Full Core Length
13-01	Arch/Horizontal	11.5	No	Yes, overlapping crack tips on exterior surface, and surface-perpendicular crack extending from exterior surface through a few aggregate particles up to 3.25 inches deep	Flat formed exterior surface with overlapping crack tips	Exterior 3.5 inches
15-01	Abutment/Horizontal	10.5	No	No	Weathered exterior surface with exposed fine aggregate and slightly crumbling paste	Exterior 2 inches

Table 2. As-Received Condition of Concrete Cores for Limited Comparative Petrographic Examination

Sample ID	Element/Coring Orientation	Length (inches)	Concrete Repair Present?	Macro Cracking Present?	Core Notes	Area Examined
17-01	Retaining Wall/Horizontal	11.5	No	No	Weathered exterior surface with exposed coarse and fine aggregate	Exterior 2.5 inches
17-02	Retaining Wall/Vertical	13	No	Yes, "map cracking" on exterior surface and intermittent surface-parallel cracking through full thickness of core, through and around aggregate	Slightly uneven exterior surface, but no exposed aggregate	Full Core Length
17-03	Retaining Wall/Horizontal	21	No	Yes, cracked into many pieces, crumbling in exterior nom. 11 inches, through and around aggregate, generally parallel or oblique to surface but some cracks are irregular	Full length not definite because exterior surface was not retained, but exterior-most segment is only partial diameter, second segment is nominally 6.5 inches and cracked, and interior-most segment is nominally 10 inches with relatively less cracking but still has low volume of cracks	Full Core Length
18-01	Abutment/Horizontal	12 to 13.5	No	Yes, fractured at 6.275 to 8.5 inches from exterior, mostly through aggregate, white deposits on fractured faces of some aggregate particles; low volume of aggregate with internal cracking	Weathered exterior surface; internal cracking in some aggregate	Full Core Length
18-02	Abutment/Vertical	8.5	No	Yes, fractured into several pieces, through and around aggregate	Weathered/eroded (inferred) exterior surface, with abundant coarse and fine aggregate standing proud; Core is crumbled into several pieces, length cannot be measured but reportedly 8.5 inches to surface	Full Core Length

PETROGRAPHIC EXAMINATION

Methods

Detailed and limited petrographic examinations of the ten cores listed above were conducted in accordance with the methods and procedures outlined in ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*. A nominal 3/4-inch thick section was cut longitudinally from the center within the exterior region (exact depths listed in Table 1 and Table 2) of Cores 01-01, 08-01, 11-01, 13-01, 15-01, and 17-01 using a water-cooled continuous-rim diamond saw blade. Cores 17-02, 17-03, 18-01, and 18-02 were cut in half longitudinally along the entire length of the core. The surface of one of the resulting saw-cut sections of each core was sprayed with a pH indicating phenolphthalein solution to determine the depth of carbonation of the paste. Another resulting surface from each core was lapped to achieve a fine matte finish suitable for examination with a stereomicroscope. Lapped surfaces and freshly fractured surfaces were examined using a stereomicroscope (5X to 63X magnification) equipped with a digital camera and a calibrated reticle for measurements. The appearance of lapped sections and sprayed faces are shown in Figure 11 through Figure 46. Fresh fractured surfaces were created in the lab and examined to assess the paste-aggregate bond and secondary deposits (Figure 47 through Figure 49). Selected lapped sections were also stored for three days in a room with a controlled relative humidity at 100% and temperature of approximately 73°F (Figure 50).

A thin section was prepared from the exterior region of Cores 01-01, 08-01, 13-01, and 15-01 and across the contact line between the apparent concrete repair and bulk concrete in Core 11-01 to assess the compositional and textural characteristics of the concrete and refine estimation of water to cement ratio (w/c), and to assess for any indications of distress mechanisms within the concrete. Thin sections were examined using a petrographic (polarized-light) microscope at magnifications ranging from 50X to 400X. Thin-section photomicrographs are shown in Figure 51 through Figure 56. The w/c was estimated based on the paste features, including but not limited to paste color, hardness, porosity/water absorptivity, residual cementitious materials, hydration products, and paste-aggregate bond. The Mohs Hardness Index (MHI) of the paste was measured using a Mohs Hardness Testing Kit. A higher hardness of the paste typically indicates a lower w/c. The lapped surface of typical concrete has an MHI between 2 and 4.

Various units of measurement are used in this report. For convenience, the following conversions are provided for reference: 1 mm = 1000 µm = 39 mil = 0.039 inch; 1 mil = 0.001 inch = 25 µm.

Alkali-Silica Reaction and Cyclic Freeze-Thaw

Alkali-Silica Reaction (ASR) distress can occur when alkalis in the concrete paste react with silicas in certain aggregates (reactive aggregates) to form a gel-like material. When water is present, the gel can expand, which can cause the concrete to crack. In this examination, qualitative ranking of damage related to ASR based on our experience is summarized in Table 3. This scale will be used in the following sections to evaluate the severity of ASR-related damage identified during the petrographic examination of the samples. Note that the ASR ranking is for qualitative comparison purposes only for the examined samples; it is not intended to directly represent or correlate with the severity of the distress observed in the field structure. The severity of the distress in a structure resulting from ASR is affected by the ASR development (Table 3) and the characteristics of the material and structure type, including but not limited to mixture proportion, exposure conditions (moisture level and temperature), reinforcement, and type and geometry

of the structure. As described above, the lapped sections of a few of the cores were also stored after initial examinations in a room with a temperature approximately 73°F and controlled relative humidity of 100% for approximately 3 days and further examined after the conditioning to evaluate development of ASR.

Table 3. Petrographic qualitative ranking of distress related to ASR

ASR Severity Ranking	Petrographic Observations
Absent	No ASR gel or associated reaction product on the boundary of aggregates (also known as reaction rims), paste darkening, microcracks, or cracks
Trace	Limited evidence of ASR such as ASR gel and reaction rims, with no associated distress
Minor	Infrequently observed ASR gel and microcracks extending into paste from reactive aggregates
Moderate	Commonly observed ASR gel and microcracks extending into paste from reactive aggregates
Severe	Abundant ASR gel and microcracks extending into paste from reactive aggregates

Cyclic freeze-thaw (F/T) distress occurs when water fills the voids of a porous material and then freezes and expands. In concrete, F/T distress can occur when non-air-entrained concrete, which has limited void space, is critically saturated with water and then is subjected to repeated freezing and thawing cycles. Distress typically manifests first with internal microcracking in the paste (usually extending around rather than through aggregates). Microcracking in turn allows more water to penetrate the concrete, which exacerbates the problem during repeated freezing and thawing. Damage usually propagates from the exterior surface inward, with advanced distress manifesting as a dense network of F/T cracks and eventual paste disintegration (crumbling). Concretes with higher porosity associated with higher w/c are typically more susceptible to freeze-thaw distress.

ASR and F/T distress both require abundant moisture in the concrete. ASR cracks usually originate in or near reactive aggregate particles and then extend outward into the paste, whereas F/T cracks usually occur primarily in the paste. Nonetheless, if F/T cracks develop, they allow more water to penetrate the concrete, which can trigger or exacerbate ASR distress. In this manner, the two mechanisms can coexist and be interdependent, as discussed later in this report.

PETROGRAPHIC OBSERVATIONS AND FINDINGS

The concrete in each of the cores that is original to the bridge construction contains similar constituents in relatively similar proportions, and likely all represent the same basic concrete mixture regardless of structure elements. This concrete, which will be referred to as the "bulk concrete" is composed of natural metamorphosed siliceous gravel coarse aggregate and natural siliceous sand fine aggregate dispersed within non-air-entrained portland cement paste. The concrete in the core samples is overall well consolidated, and distribution of constituents is uniform. Core 11-01 also contained a bonded apparent concrete repair material that represents a different concrete mix; the date of placement of this mix is unknown, but is more modern than the bulk concrete mix, and more recent than the repairs reported in 1943. A more detailed description of individual cores is provided in Table 4. The following descriptions pertain to all cores, unless stated otherwise.

Bulk Concrete Mix

The gravel coarse aggregate is composed of siliceous metamorphic rocks, mainly including schist, phyllite, and quartzite as well as small amounts of non-metamorphosed siliceous rocks including calcite-bearing sandstone, shale, and weathered granitic to basaltic rocks. Based on cursory geologic review of the project site area, it is likely that these aggregates were quarried near the project site. The coarse aggregate is well graded with a nominal top size ranging from 0.75 to 1.5 inch (Table 4) and generally uniformly distributed throughout the examined areas. The sand is mainly composed of quartz, quartzite, granite, feldspar, shale, schist, chert, mica, and small amounts of miscellaneous rocks and mineral particles. Portions of the coarse and fine aggregate are reactive with alkalis in the paste via ASR, as noted above. The extent of ASR-related distress is described below and in Table 4.

On laboratory-induced fracture surfaces, the paste is moderately absorptive to water and moderately hard to hard, with an MHI between 3.5 and 4. Paste-aggregate bond is overall moderately strong to strong with fractures propagating generally through but sometimes around aggregate particles. Thin section examination reveals the cement particles are larger in size than modern portland cement, typical for historic concretes. The hydration of portland cement appears overall advanced, with large cement particles exhibiting a readily visible hydration rim and an un-hydrated inner core. Calcium hydroxide crystals in the paste from hydration of portland cement were moderate in size, moderate to moderately high in volume, and frequently interconnected. Based on the observed paste features, the w/c of the cores is judged to be moderate. It can be difficult to estimate w/c in historic concrete due to significant changes in concreting practice and lack of reference samples, but estimated w/c ranges are from 0.43 to 0.53 with minor variations between cores (Table 4).

The depth of carbonation ranged between the cores, from negligible (less than 1/16-inch) to up to 1 inch from the exterior surface (Table 4). As noted in Table 1, some core surfaces were eroded or fragmented, so the depth of carbonation on those cores was only measured very locally and may not represent the full depth of carbonation in the field.

Air content in the bulk concrete was less than 3 percent, with only entrapped air voids present.

Ettringite secondary deposits were frequently observed in air voids and occasionally along the paste-aggregate interface or in cracks. The ettringite deposits were consistent with exposure of the concrete to moisture, materials dissolution, and reprecipitation. No evidence of ettringite replacing calcium-silicate-hydrate (C-S-H) paste or widespread filling the paste-aggregate interface was observed. No evidence of internal or external sulfate attack or salt-related distress was observed.

ASR in Bulk Concrete

ASR has affected the concrete to varying extents, ranging from trace or minor to severe. Many cores, including Cores 11-01 (bulk concrete only), 17-02, 17-03, 18-01, and 18-02, exhibited evidence of moderate to severe ASR. This evidence includes frequent ASR gel in voids, numerous cracked schist particles with occasional material loss, and cracks/microcracks partially filled with ASR gel extending from reactive particles. The disproportionately high number of microcracks in ASR-affected particles and the high percentage of particles fractured along the broken surfaces of the concrete during lab examinations provide compelling evidence of ASR (Figure 21, Figure 22, Figure 26, Figure 36 through Figure 49). Exposure of the selected lapped sections to moisture in the laboratory induced negligible to a very low

volume of additional ASR gel deposits (Figure 50), indicating a continued potential, albeit very low based on the age of the concrete, for further ASR. The reactive aggregate was mainly schist, with some quartzite and strained quartz in other silicate rocks also affected.

The severity of ASR, based on the criteria in Table 3 and our experience with similar projects, is summarized below:

- *Trace to Minor:* Cores 01-01, 08-01, 11-01 (apparent concrete repair), 15-01, and 17-01
- *Minor:* Core 13-01
- *Moderate:* Cores 18-01 and 18-02
- *Severe:* Cores 11-01 (bulk concrete), 17-02, and 17-03

It should be noted that severe ASR-related damage in Core 11-01 was only observed in the bulk concrete; distress was not observed in the assumed relatively more modern apparent concrete repair (containing fly ash, see characterization in material-specific section below) on top of the bulk concrete. The bulk concrete exhibited frequent cracking through reactive aggregate particles throughout the full depth of the layer, which encompasses the bottom nominal 5 inches of the core; the cracking probably occurred or existed before the placement of the apparent concrete repair and pre-existing deteriorated concrete was not fully replaced. The core also intersected a small portion of a No. 5 rebar (Figure 3). The rebar contained only a few minor stains and appeared to be in good condition overall.

Cyclic Freeze-Thaw

The concrete showing moderate to severe ASR, including Cores 11-01 (bulk concrete only), 17-02, 17-03, 18-01, and 18-02, has also likely been significantly affected by cyclic freeze-thaw. These two distress mechanisms are expected to exacerbate each other, causing more severe damage to the concrete than either mechanism alone. The bulk concrete is not air entrained (air-entraining technology was not available at the time of original construction) and has an estimated w/c of likely higher than 0.45; both factors are expected to lead to greater vulnerability of cyclic freeze-thaw when the concrete is critically saturated. Presence of surface-parallel cracks in many of the cores (Cores 11-01 bulk concrete, 17-02, 17-03, 18-01, and 18-02) appeared to be consistent with frost damage. However, it is difficult to precisely separate freeze-thaw distress and ASR-related damage since moisture accelerates both types of distress.

All of the core samples manifesting moderate or severe ASR distress also manifest F/T distress were extracted from locations where critical water saturation is likely at times (i.e., below deck drains, near the water line, and next to deck expansion joints). As such, the cracking that is present could be the result of both the F/T and ASR mechanisms, likely acting in combination (see further discussion below).

Apparent Concrete Repair, Core 11-01

Core 11-01 contains apparent concrete repair material which is 5.25 inches thick (Figure 18, Figure 19). Overall, the repair layer appears to be well consolidated and in generally good condition. Indications of ASR or F/T are not observed in the repair.

The apparent concrete repair is composed of crushed siliceous coarse aggregate and siliceous sand fine aggregate dispersed within an air entrained portland cement and fly ash paste. The sand appeared to be manufactured, based on angularity of most particles, but it could be natural or a combination of both. The coarse aggregate has a top size of 1/4-inch, is angular to subrounded, and is typically composed of white,

black, and pink crystals. The aggregate is generally well dispersed throughout the thickness of the layer, with a slightly lower volume of particles observed in the top nominal 1/4-inch. The coarse aggregate can geologically be classified as granite or diorite. The fine aggregate is composed of natural siliceous sand including quartz, feldspar, mica, and fine fractions of the coarse aggregate.

The paste in the repair is a medium gray color composed of portland cement and fly ash (Figure 53). On laboratory induced fracture surfaces, the paste has a microgranular texture, vitreous to subvitreous texture, and is somewhat absorptive to water and is judged to be moderately hard to hard. Paste to aggregate bond was judged to be strong, with fractures propagating through aggregate particles. Based on the observed paste features, the w/cm (water to cementitious materials ratio) of the repair concrete is judged to be moderately low, ranging from 0.38 to 0.43.

The apparent concrete repair is air entrained, with spherical voids generally evenly dispersed throughout the thickness of the layer, although a slightly densified layer along the top surface was observed. Estimated air content is 5 to 7 percent. Secondary deposits were not observed in the air voids.

A thin (nominally 1 mm) cementitious skim layer was observed between the apparent concrete repair and the bulk concrete, and was likely applied to promote bond between the two concretes. The skim was composed of natural siliceous quartz sand dispersed within a non-air entrained paste including portland cement, silica fume, and likely other specialized cements (Figure 20). The skim exhibits infrequent microcracking. Each of the materials in the system (the apparent concrete repair, the cementitious skim, and the bulk concrete) appear to be in intimate contact, and accumulations of voids or microcracks along or between the layers were not observed.

Table 4. Description of Bulk Concrete

Parameter		Core 01-01	Core 08-01	Core 11-01	Core 13-01	Core 15-01	Core 17-01	Core 17-02	Core 17-03	Core 18-01	Core 18-02
Type of Petrographic Examination, Region Examined		Detailed, Exterior Region	Detailed, Exterior Region	Detailed, Full Length	Detailed, Exterior Region	Detailed, Exterior Region	Comparative, Exterior Region	Comparative, Full Length			
Coarse Aggregate	Lithology	Natural siliceous metamorphic gravel, including mainly schist and quartzite as well as small amounts of shale, chert, calcite-bearing sandstone, weathered granitic to basaltic rocks									
	Top Size (in)	3/4	3/4	3/4	3/4	1	3/4	1	1	1-1/2	1-1/2
	Shape, Color	Rounded to subangular, Elongate to Equant, White, sparkly light gray, light brown, dark brown, dark gray, olive green, speckled white and black, and more									
Fine Aggregate	Lithology	Natural siliceous sand including quartz, quartzite, granite, feldspar, shale, schist, chert, mica, and small amounts of miscellaneous rocks and mineral particles.									
	Shape, Color	Subrounded to subangular, similar colors to coarse aggregate, plus translucent									
Paste	Color	Light beige gray, discolored orange near exterior surface	Mottled light to medium light gray		Light beige gray	Light beige gray, discolored orange near exterior surface	Very light gray	Mottled very light to medium light gray		Mottled very light to light gray	Mottled very light to medium light gray
	Hardness, MHI value	Hard, 4	Moderately hard, between 3.5 and 4			Hard, 4	Moderately hard, between 3.5 and 4				
	Absorp-tivity	Low	Moderate	Low	Moderate	Low	Moderate				
	Estimated w/c	0.43 to 0.48	0.45 to 0.50	0.44 to 0.49	0.45 to 0.50	0.43 to 0.48	0.45 to 0.50				

Parameter		Core 01-01	Core 08-01	Core 11-01	Core 13-01	Core 15-01	Core 17-01	Core 17-02	Core 17-03	Core 18-01	Core 18-02
Air	Description of Voids	Spherical to rounded, irregularly shaped voids, well distributed throughout paste	Spherical to rounded, irregularly shaped voids, occasional gaps between paste and aggregate well distributed throughout paste	Spherical to rounded, irregularly shaped voids, well distributed throughout paste			Spherical to rounded, irregularly shaped voids, occasional gaps between paste and aggregate well distributed throughout paste	Spherical to rounded, irregularly shaped voids, infrequent large voids throughout, well distributed throughout paste	Spherical to rounded, irregularly shaped voids, well distributed throughout paste		
	Estimated Volume %	<3%									
Cracking		Infrequent microcrack-ing within aggregate	Infrequent microcrack-ing within aggregate, extends into paste, randomly oriented	Frequent cracking within aggregate extending into paste and other aggregate particles, typically parallel to bottom surface	Surface-perpendic-ular cracks through and around aggregate, likely from drying shrinkage/ thermal distress; infrequent microcrack ing within aggregate	Infrequent microcracking within aggregate		Core fractured into several pieces; frequent cracking within aggregate extending into paste and other aggregate; surface perpendic-ular shrinkage	Core fractured into several pieces; frequent cracking within aggregate extending into paste and other aggregate particles, especially in exterior nominal 10	Core fractured into several pieces; frequent cracking within aggregate extending into paste and other aggregate particles	

Parameter	Core 01-01	Core 08-01	Core 11-01	Core 13-01	Core 15-01	Core 17-01	Core 17-02	Core 17-03	Core 18-01	Core 18-02
							cracks typically 1/8-inch deep	to 12 inches		
Nominal Depth of Carbonation, inches	3/8	1/4	Negligible from both layers	1/8	1/2 to 1	1/4	Negligible	Negligible, but limited exterior surface retained	1/8	Negligible, but limited exterior surface retained
Unit Weight (lb/ft^3)	151.3	152.6	150.4	150.8	152.8	154.1	148.3	153.3	148.9	150.9
Extent of ASR	Trace to Minor	Trace to Minor	Severe	Minor	Trace to Minor	Trace to Minor	Severe	Severe	Moderate	Moderate
Cyclic Freeze-Thaw	Not observed	Not observed	Observed through full thickness of bulk concrete (10.5 inches from concrete surface)	Not observed	Not observed	Not observed	Observed through full thickness of core (13 inches from concrete surface)	Observed through exterior nominal 10 inches	Observed through exterior nominal 8 inches	Observed through exterior nominal 2.5 inches

Note: Unit weight was calculated using sample dimensions for Cores 01-01, 08-01, 13-01, 15-01, and 17-01, and roughly according to ASTM C642 for Cores 11-01, 17-02, 17-03, 18-01, and 18-02; samples tested via C642 contained portions of epoxy as well as cracking which may cause variations in measurements.

SUMMARY AND DISCUSSION

Bulk Concrete: Detailed petrographic examinations of Cores 01-01, 08-01, 11-01, 13-01, and 15-01 and comparative, limited examinations of Cores 17-01, 17-02, 17-03, 18-01, and 18-02, extracted from various locations of the Windsor Bridge in Windsor, Vermont, revealed varying extents of distress mainly caused by cyclic freeze-thaw, ASR, or a combination of both. It is judged that cyclic freeze-thaw cycles have likely initiated near-surface cracks, creating pathways for moisture, and accelerating ASR distress, with both mechanisms worsening the condition.

The concrete represented by the ten cores is composed of well-graded natural siliceous metamorphic gravel coarse aggregate with a 3/4 to 1-1/2 inch top size and natural siliceous sand fine aggregate dispersed within an overall well-consolidated, non-air-entrained portland cement paste with a moderate to moderately high w/c. Additional details of the concrete characteristics can be found in Table 4. The distribution of constituents within the concrete was fairly uniform.

The concrete is susceptible to cyclic freeze-thaw distress when saturated due to lack of air entrainment and moderately high porosity associated with a w/c estimated higher than 0.43 for all cores. ASR and cyclic freeze-thaw are judged to have contributed substantially to the cracking observed in the cores extracted from zones of likely water saturation (albeit to varying extents) and adversely affected the concrete performance. Cracking of abundant hard dense coarse aggregate particles is clear evidence of damage caused by ASR, rather than frost damage alone. ASR-related distress was most severe in cores that likely exhibited preexisting cracking caused from cyclic freeze-thaw or from corrosion-related cracking or other causes which could not be determined petrographically or not represented in the examined cores. A detailed classification of ASR severity is described above and summarized in Table 3.

The overall concrete condition of the cores is summarized as follows:

- *Good to Fairly Good:* Cores 01-01, 08-01, 11-01 (apparent concrete repair), 15-01, and 17-01
- *Fair:* Core 13-01
- *Fair to Poor:* Cores 18-01 and 18-02
- *Poor:* Cores 11-01 (bulk concrete), 17-02, and 17-03

The poor condition of the bulk concrete in Core 11-01 may be attributed to exposure, as the core was extracted vertically from the bridge deck and over a horizontal beam. It is possible that water collected between the bridge deck and the beam, allowing for saturation and freeze-thaw distress to occur. The apparent concrete repair contains fly ash, is air entrained, and has estimated w/c lower than the original concrete. It did not exhibit indications of ASR distress or cyclic freeze-thaw.

Other than ASR and cyclic freeze-thaw distress noted above, no other internal distress mechanisms such as internal or external sulfate attack or salt crystallization distress were observed. No substantial staining or rust was observed in the examined cores. However, only two of the examined cores contained reinforcing steel, and the portion containing steel in Core 08-01 was not used for detailed examination; the lack of corrosion product or lack cracks filled with rust in the examined cores should not be interpreted as absence of corrosion elsewhere in the structure. Relatively low depths of carbonation in the examined cores will serve to protect steel from corrosion, but factors such as chloride content or moisture ingress may promote localized corrosion.

ASR Potential: The primary reactive constituent is strained quartz in various metamorphic rocks, which typically reacts slowly via ASR. Conditioning lapped cross sections of the cores in a 100% relative humidity room for a few days produced only negligible to minor additional ASR gel or deposits. Given that this concrete is nearly 100 years old, significant changes in ASR are not expected soon. However, once cracks form and are not repaired, they allow moisture, deicing salts, alkali movement, and air to penetrate deeper into the concrete. These factors can worsen corrosion, ASR, and frost damage.

The most relevant test for assessing ASR potential is the concrete expansion test, ASTM C1293 *Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction*. Among the three primary factors influencing ASR in the subject concrete, alkali levels and moisture may limit ASR potential and kinetics. Reducing moisture exposure is critical in mitigation of ASR as well as cyclic freeze-thaw. There seems to be enough reactive aggregate that is unlikely to be fully consumed. Therefore, the concrete expansion test is likely useful for predicting the ASR potential.

Although further testing is possible, significant changes in ASR distress are not expected soon based on available information. Furthermore, both the F/T and ASR distress mechanisms are manifesting concurrently in this case, and protecting the concrete from either requires limiting moisture ingress into the concrete. As such, further testing to quantify the ASR potential does not seem warranted at this time.

Apparent Concrete Repair: Core 11-01 features a 5.25-inch thick bonded apparent concrete repair in good condition, with no signs of ASR or distress caused by cyclic freeze-thaw. The repair mainly consists of crushed diorite (or its metamorphic equivalent) coarse aggregate and natural siliceous sand fine aggregate uniformly distributed within an air-entrained portland cement and fly ash paste with an estimated w/cm of 0.38 to 0.43. The repair has an estimated air content of 5 to 7% and shows no secondary deposits in the air voids. A thin cementitious skim layer, likely applied to enhance bonding, is present between the apparent concrete repair and the bulk concrete, with all materials in close contact and minimal voids or microcracks. As described earlier, presence of fly ash, entrained air voids, and low w/cm likely have mitigated ASR and frost damage, and also suggest that the repair was made after 1943 (latest date of reported repairs).

FIGURES



Figure 1. Core 01-01 (Arch/Horizontal), as-received. The exterior surface is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image.

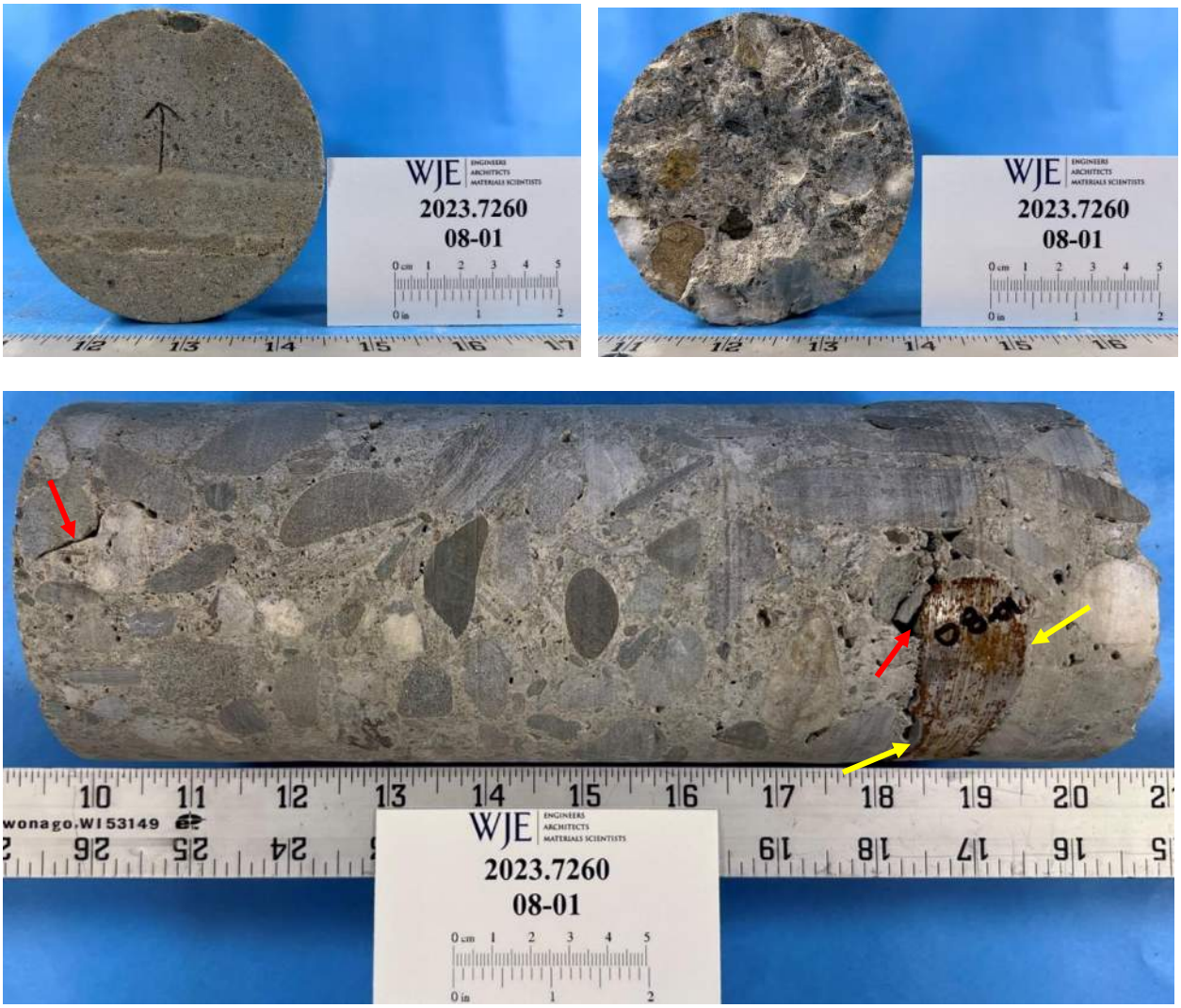


Figure 2. Core 08-01 (Floor Beam/Horizontal), as-received. The exterior surface is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image. Voiding is indicated with red arrows. Embedded steel is indicated with yellow arrows.

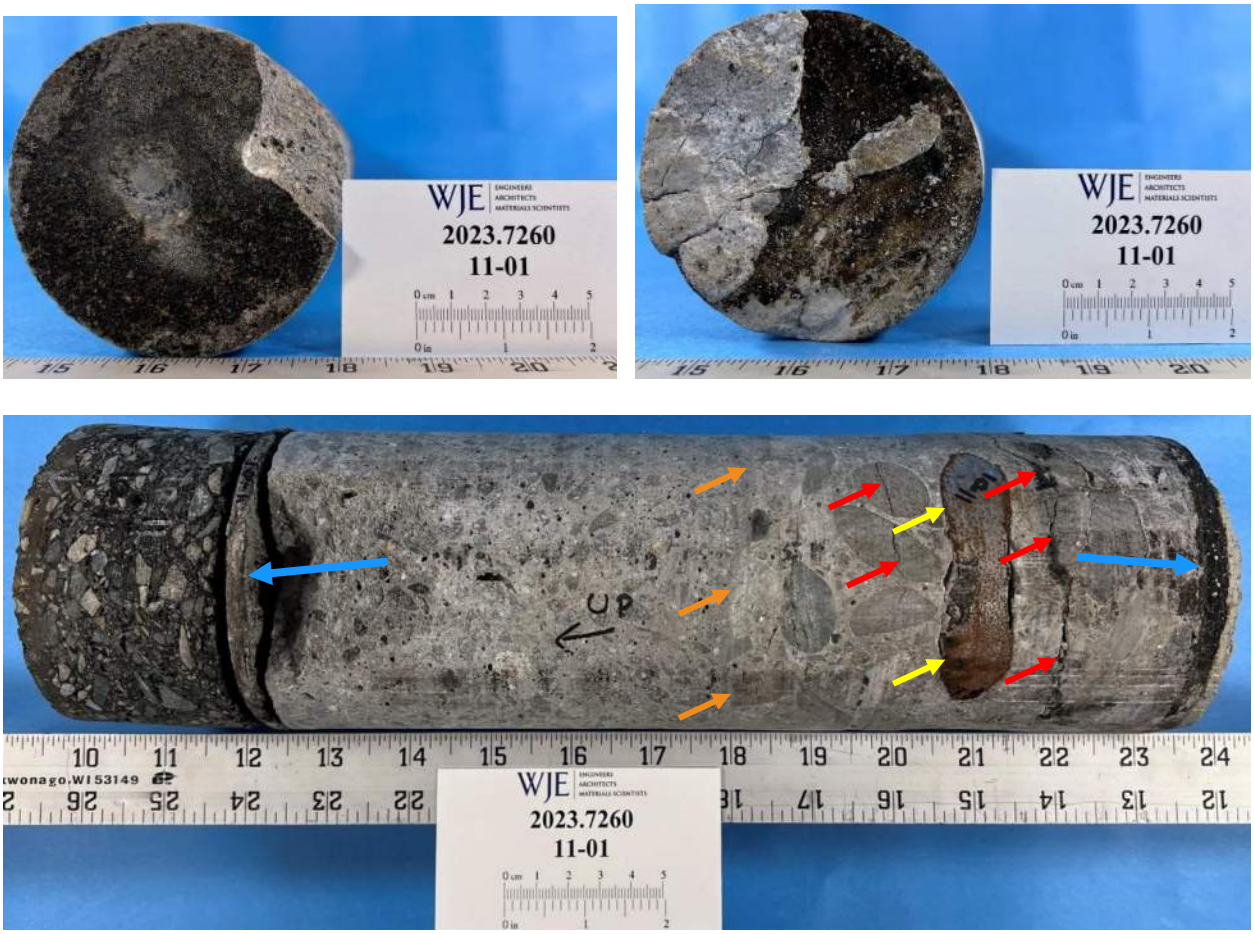


Figure 3. Core 11-01 (Deck/Vertical), as-received. The exterior surface of the concrete is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image. Cracking is indicated with red arrows. Embedded steel is indicated with yellow arrows. Orange arrows indicate the contact between the apparent concrete repair and bulk concrete. Blue arrows indicate suspected waterproofing layers.

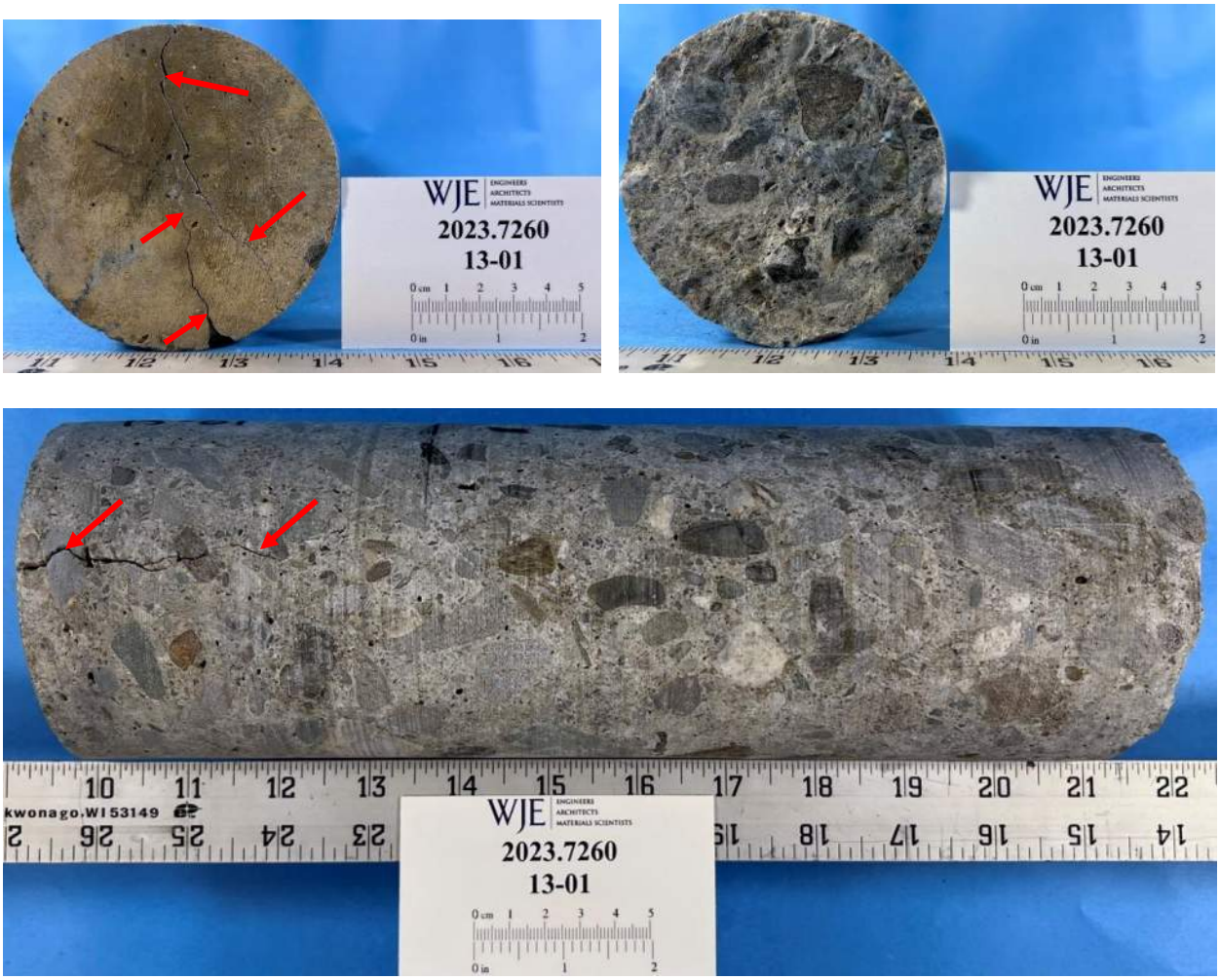


Figure 4. Core 13-01 (Arch/Horizontal), as-received. The exterior surface is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image. Cracking is indicated with red arrows.



Figure 5. Core 15-01 (Abutment/Horizontal), as-received. The exterior surface is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image.



Figure 6. Core 17-01 (Retaining Wall/Horizontal), as-received. The exterior surface is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image.

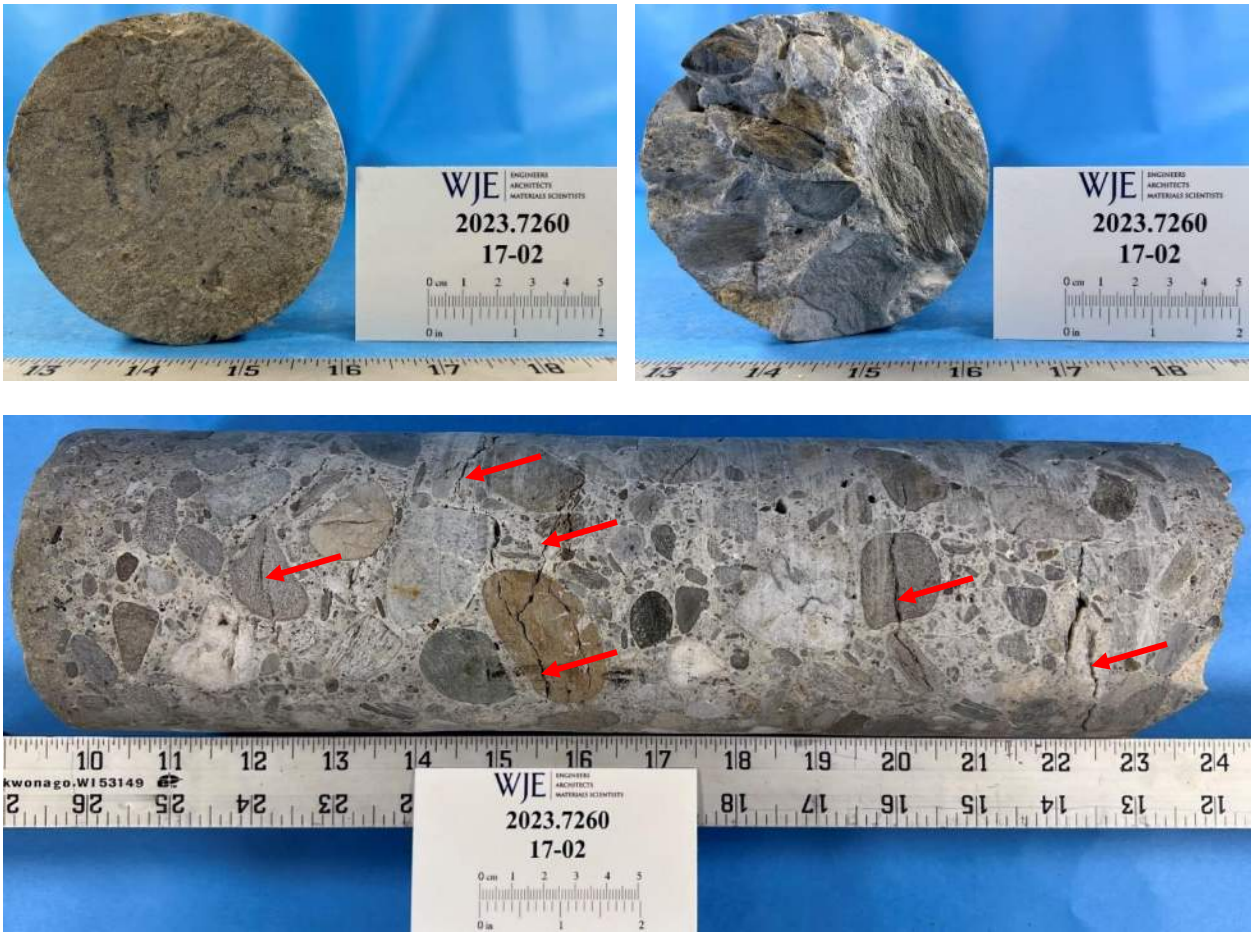


Figure 7. Core 17-02 (Retaining Wall/Vertical), as-received. The exterior surface is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image. Cracking is indicated with red arrows.



Figure 8. Core 17-03 (Retaining Wall/Horizontal), as-received. The exterior surface is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image. Cracking is indicated with red arrows. Note that some of the fragments from the exterior region of the core are also shown.



Figure 9. Core 18-01 (Abutment/Horizontal), as-received. The exterior surface is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image. Cracking is indicated with red arrows.

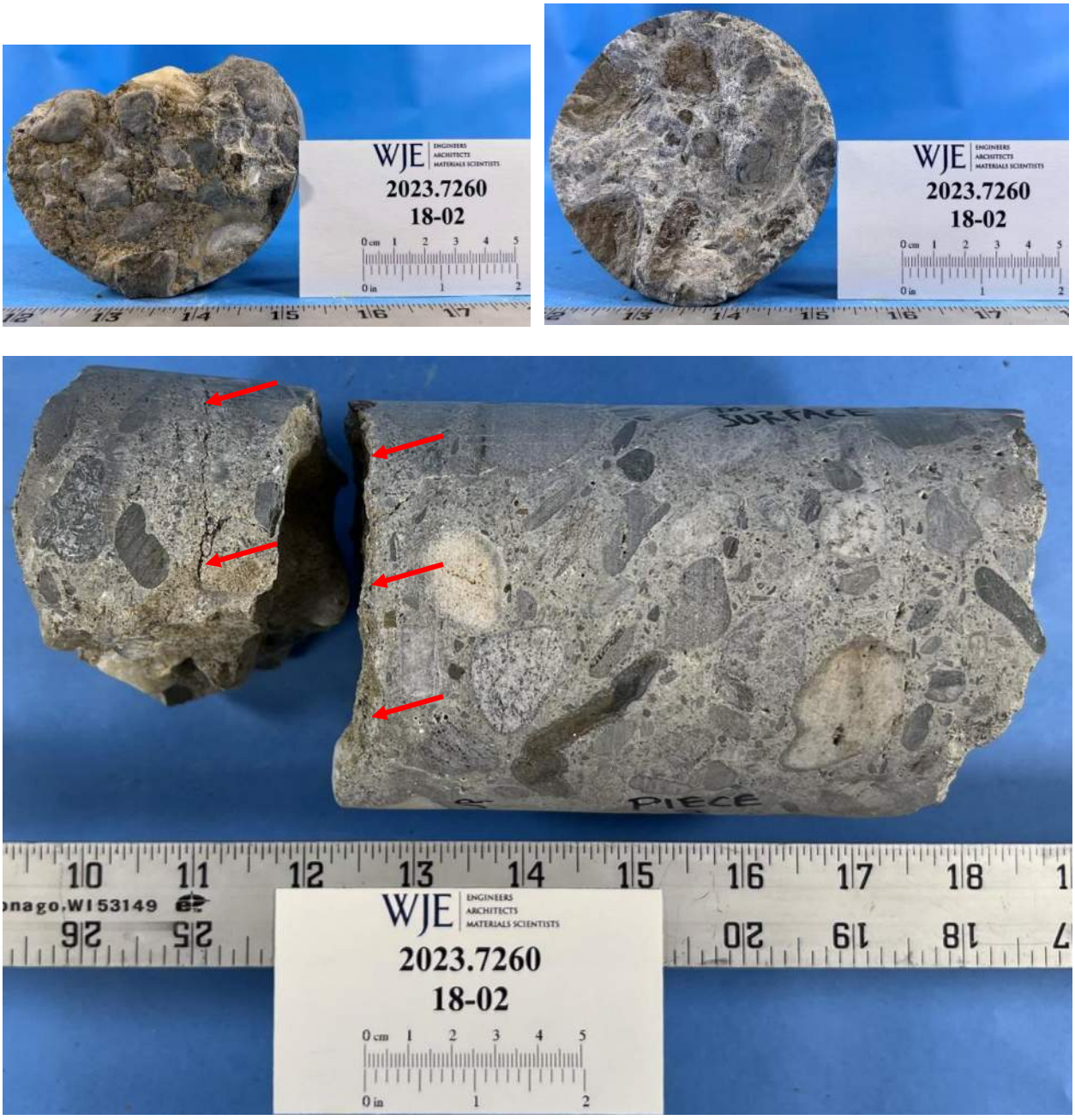


Figure 10. Core 18-02 (Abutment/Vertical), as-received. The exterior surface is in the top left image, the interior surface is in the top right, and a longitudinal view with the exterior surface facing left is in the bottom image. Cracking is indicated with red arrows.

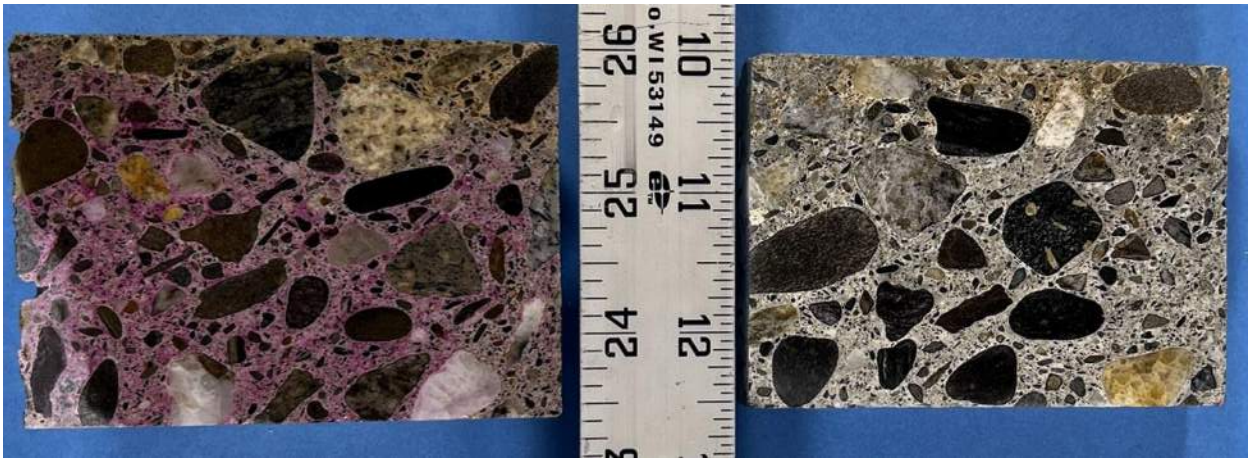


Figure 11. Core 01-01 (Arch/Horizontal). Lapped longitudinal cross section (right) and cut face freshly sprayed with phenolphthalein solution of the exterior surface region (left). The top surface is towards the top of the image. Note paste discoloration near the surface, which corresponds roughly to the depth of carbonation.

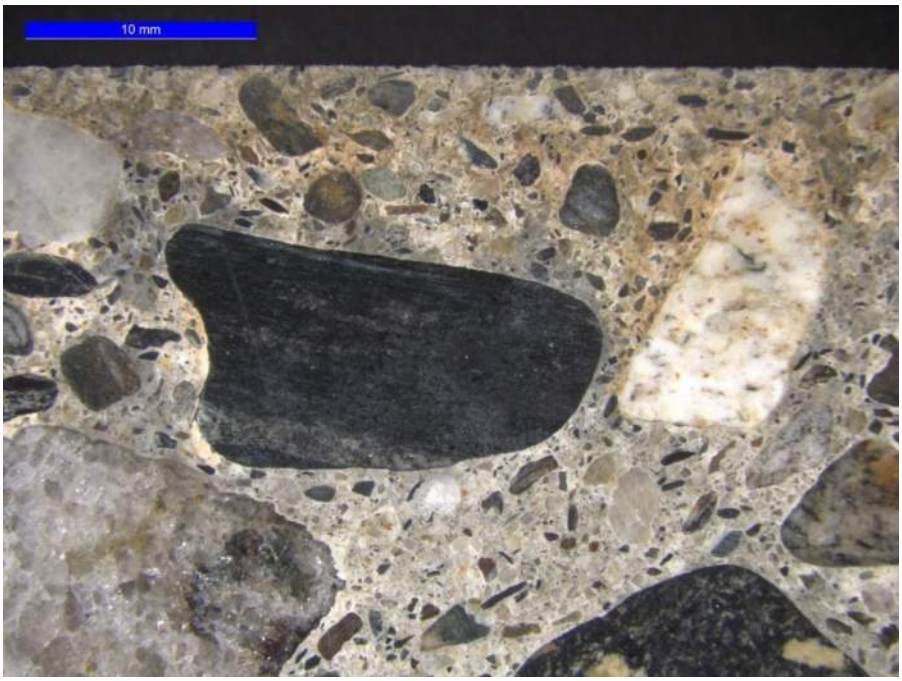


Figure 12. Core 01-01 (Arch/Horizontal). Exterior surface showing orange paste discoloration along the surface, likely from carbonation.

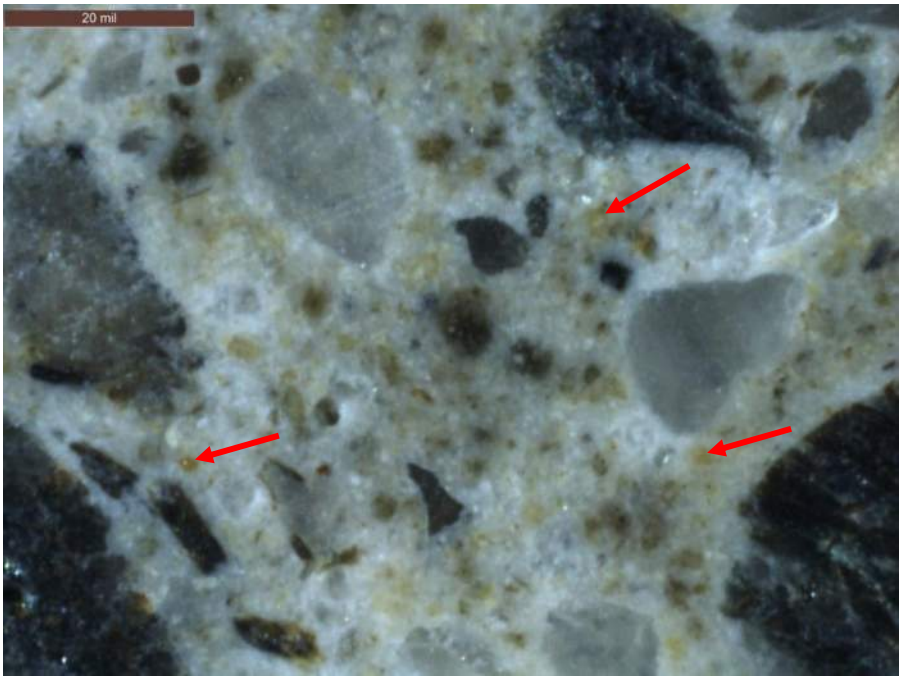


Figure 13. Core 01-01 (Arch/Horizontal). Close-up image of the paste showing suspected old cement particles (red arrows), typically brown in color.

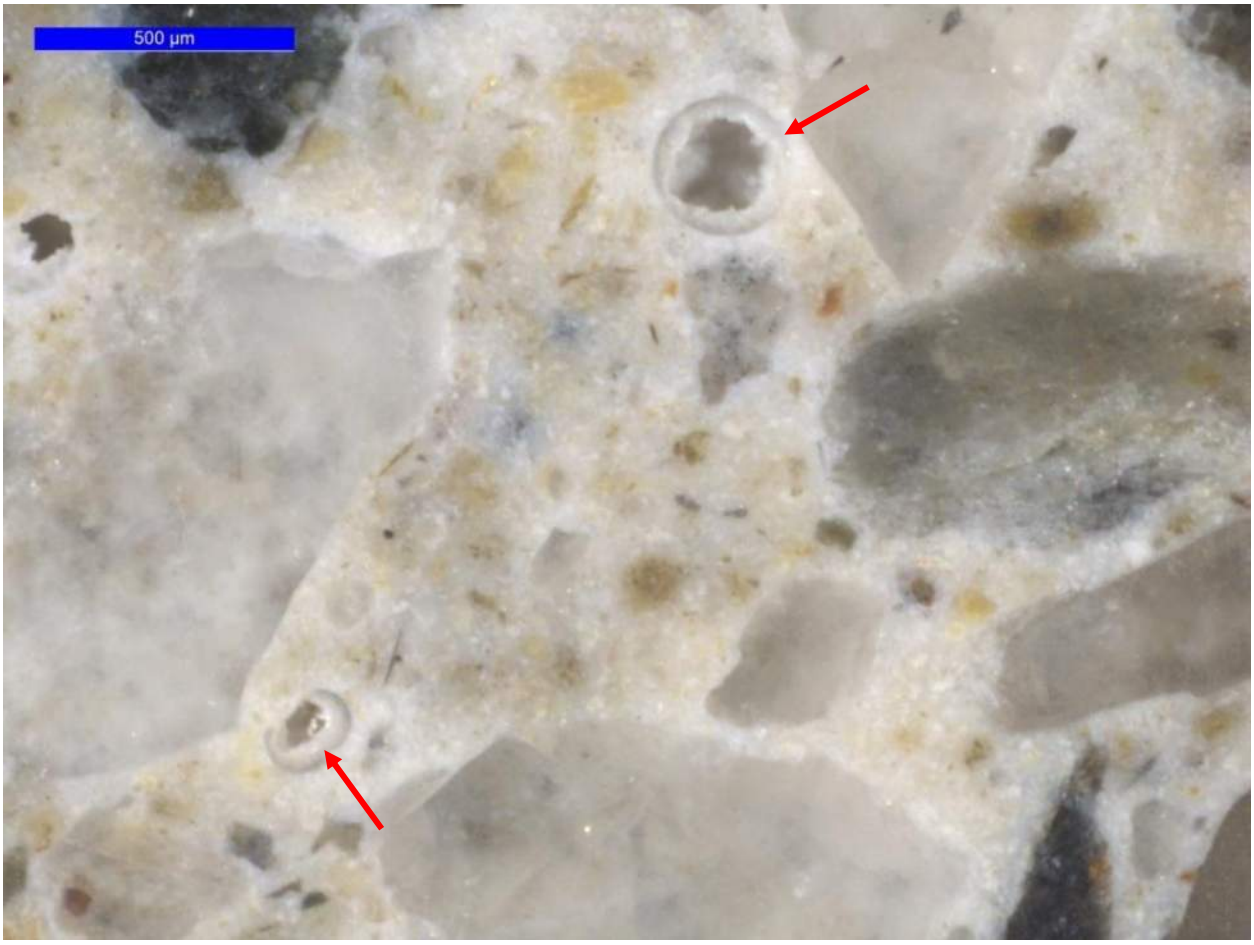


Figure 14. Core 01-01 (Arch/Horizontal). Close-up image showing secondary deposits, likely ettringite, within air voids (red arrows).

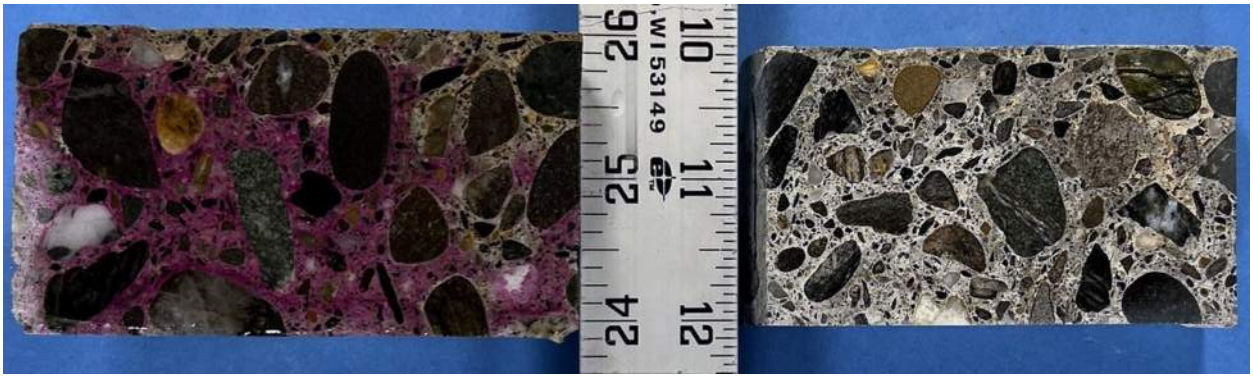


Figure 15. Core 08-01 (Floor Beam/Horizontal). Lapped longitudinal cross section (right) and cut face freshly sprayed with phenolphthalein solution of the exterior surface region (left). The exterior surface is to the top. Note paste discoloration near the surface, which corresponds roughly to the depth of carbonation.

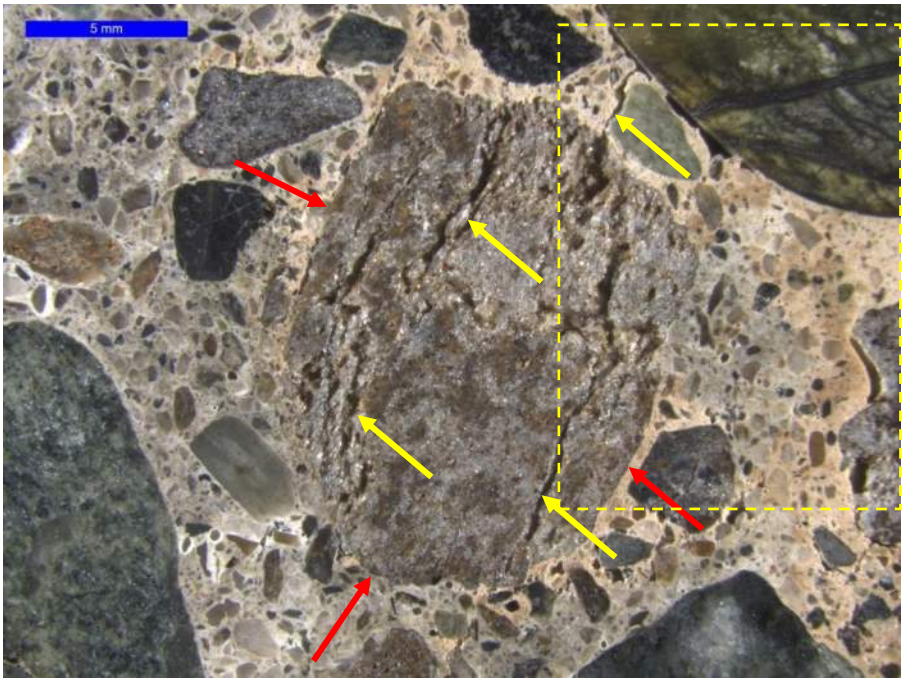


Figure 16. Core 08-01 (Floor Beam/Horizontal). Reactive coarse aggregate particle (red arrows) exhibiting internal fracturing (yellow arrows) that propagate into surrounding paste. The area within the yellow box is shown in the figure below.

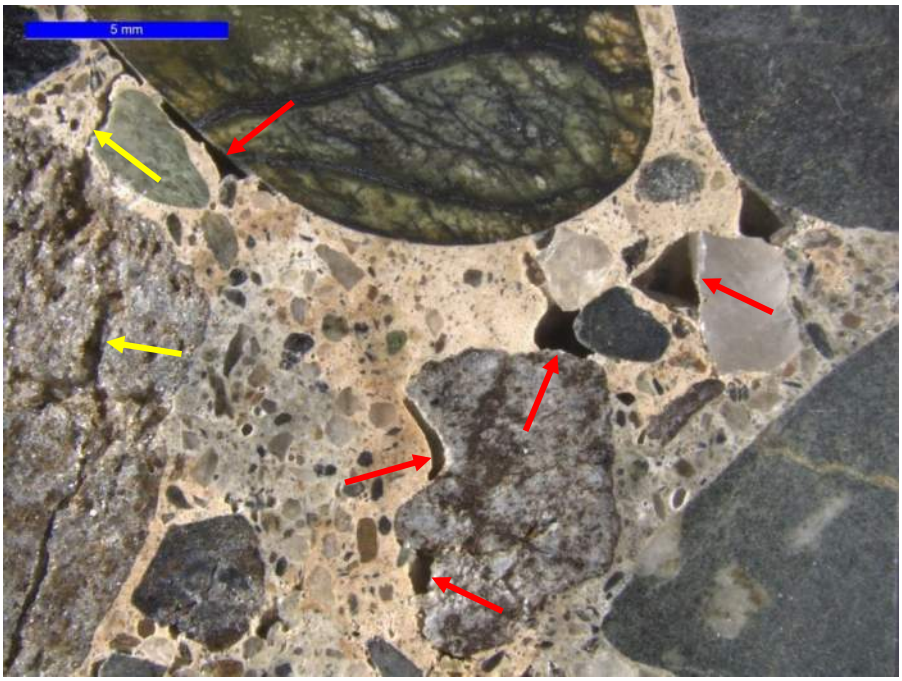


Figure 17. Core 08-01 (Floor Beam/Horizontal). Higher-magnification image of the area highlighted in Figure 16 showing cracking extending from a reactive coarse aggregate particle into surrounding paste (yellow arrows) and gaps and irregularly shaped voids between paste and aggregate (red arrows).

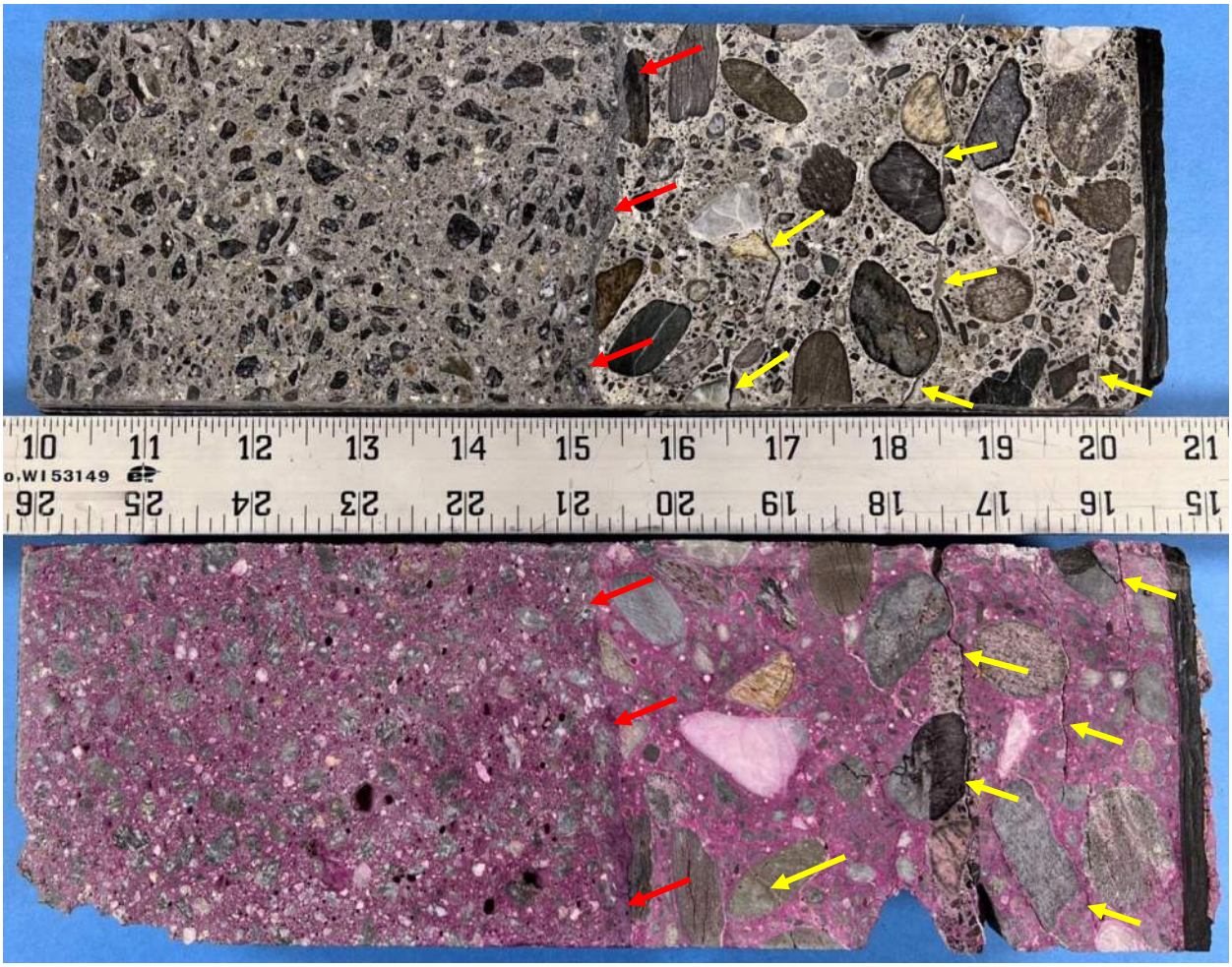


Figure 18. Core 11-01 (Deck/Vertical). Lapped longitudinal cross section (top) and freshly sprayed cut face of the full length of the core. Red arrows indicate the contact between the apparent concrete repair (left of the contact) and bulk concrete. Cracking in bulk concrete below repair concrete indicated by yellow arrows.

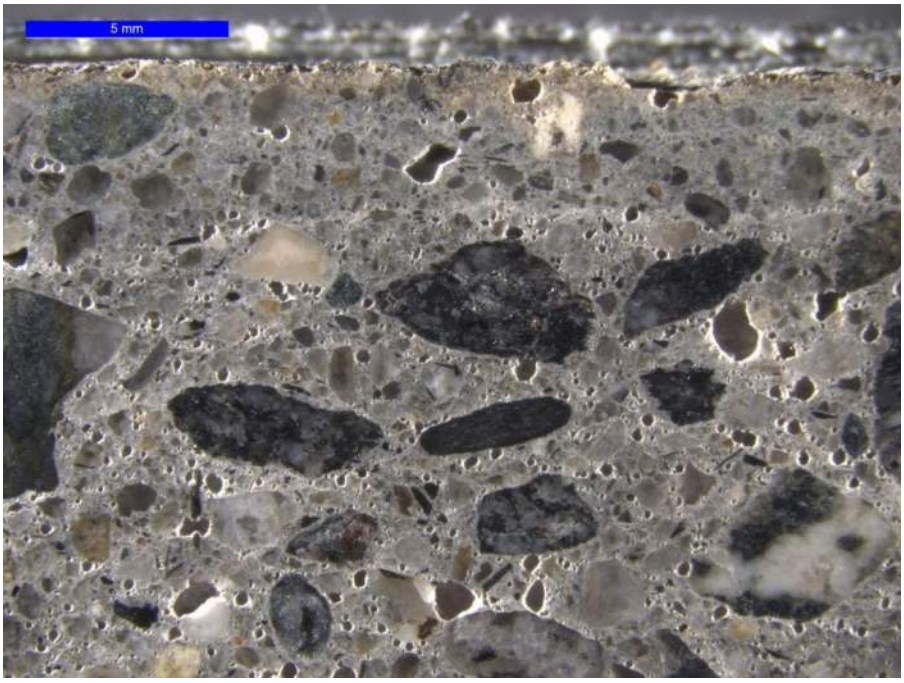


Figure 19. Core 11-01 (Deck/Vertical). Top surface region of the apparent concrete repair mix.

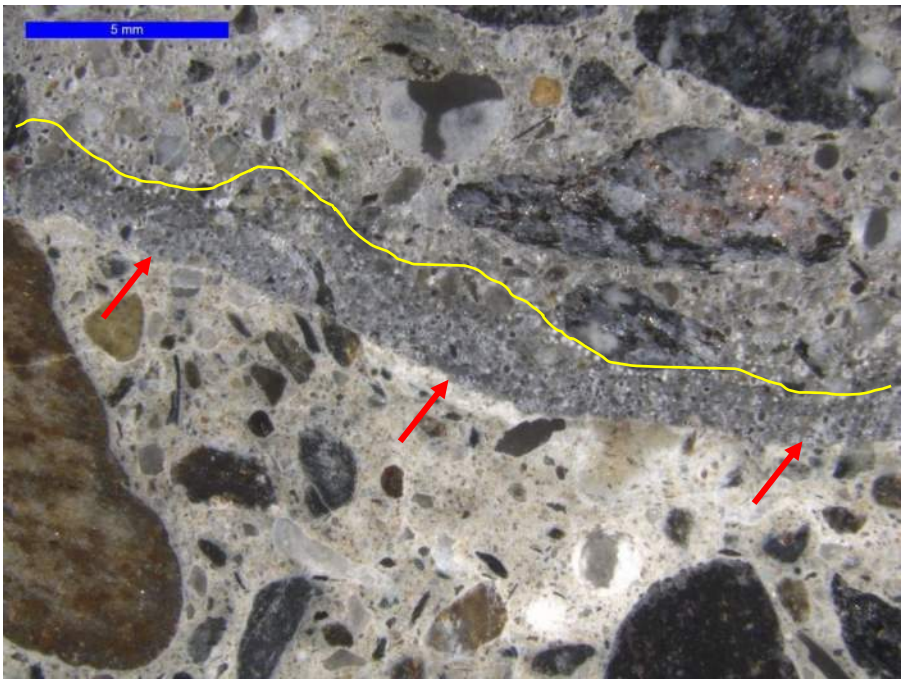


Figure 20. Core 11-01 (Deck/Vertical). Contact region between the apparent concrete repair mix (above yellow line), a potential bonding cement coat (between red arrows and yellow line), and the bulk concrete mix (below red arrows). The materials appear to be in intimate contact.

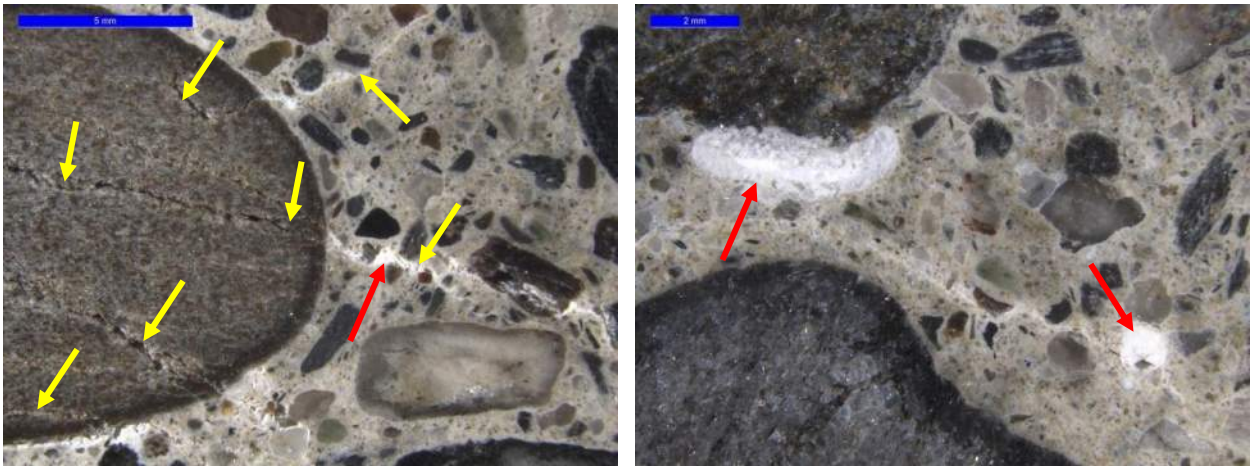


Figure 21. Core 11-01 (Deck/Vertical). Evidence of ASR, including internal fracturing of reactive aggregate particles extending into paste (left image, yellow arrows) and deposits of ASR gel in voids and cracks (red arrows).

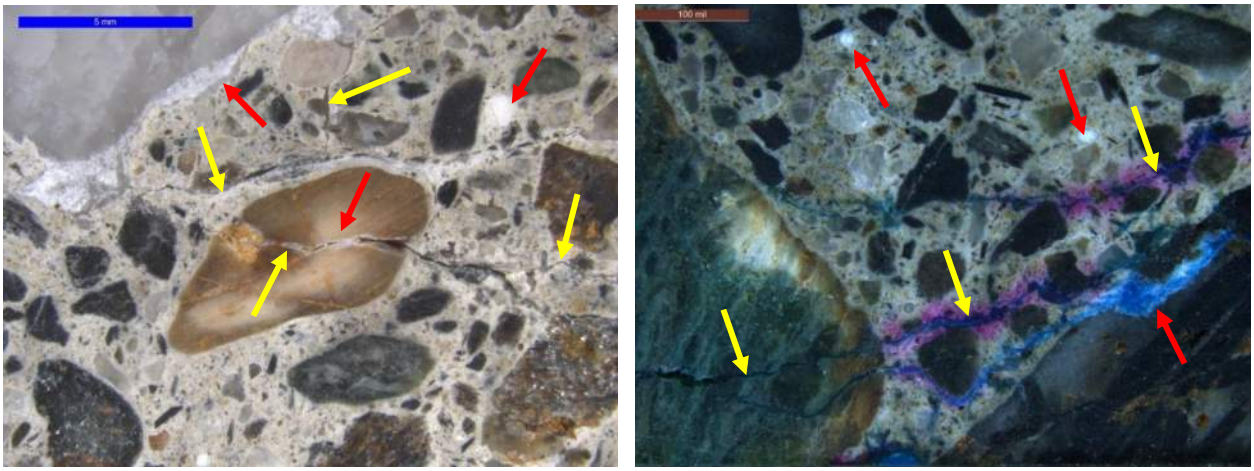


Figure 22. Core 11-01 (Deck/Vertical). Evidence of ASR, including cracking within paste and internal fracturing of reactive aggregate particles extending into paste (yellow arrows) and deposits of ASR gel in voids and cracks (red arrows).

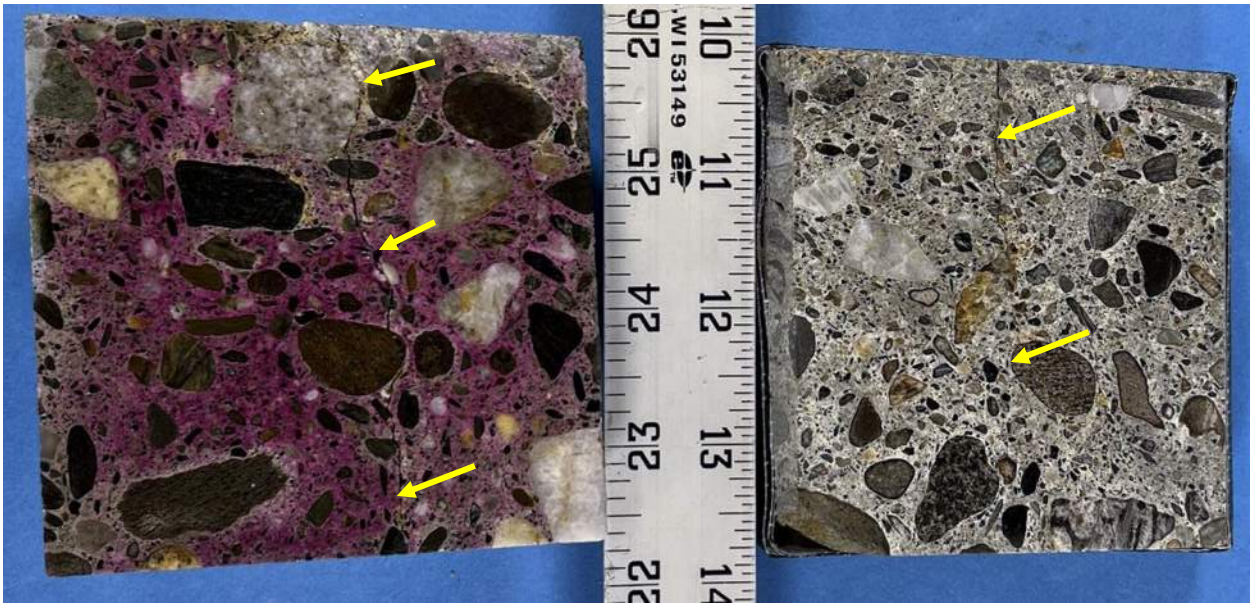


Figure 23. Core 13-01 (Arch/Horizontal). Lapped longitudinal cross section (right) and cut face freshly sprayed with phenolphthalein solution of the exterior surface region (left). The exterior surface is at the top of the image. Yellow arrows indicate a crack propagating from the exterior surface. Greater depth of carbonation was observed along the crack, probably suggesting the crack is not new.

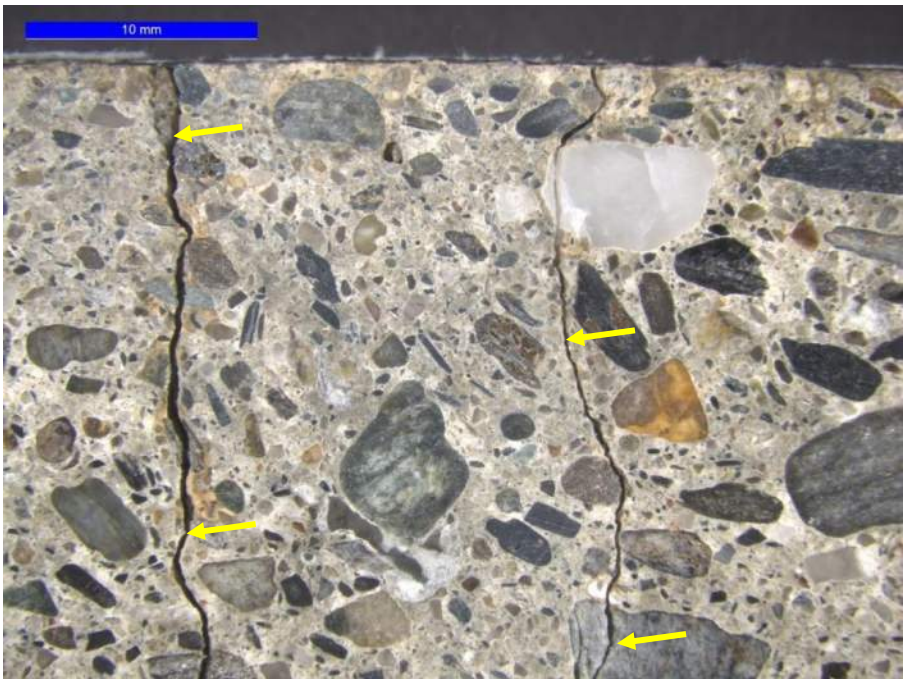


Figure 24. Core 13-01 (Arch/Horizontal). Exterior surface region of showing two surface-perpendicular cracks (yellow arrows) that extend around and through aggregate particles. Causation of cracking is unclear but may have been initiated by drying shrinkage and then exacerbated over time by ASR and/or F/T mechanisms.

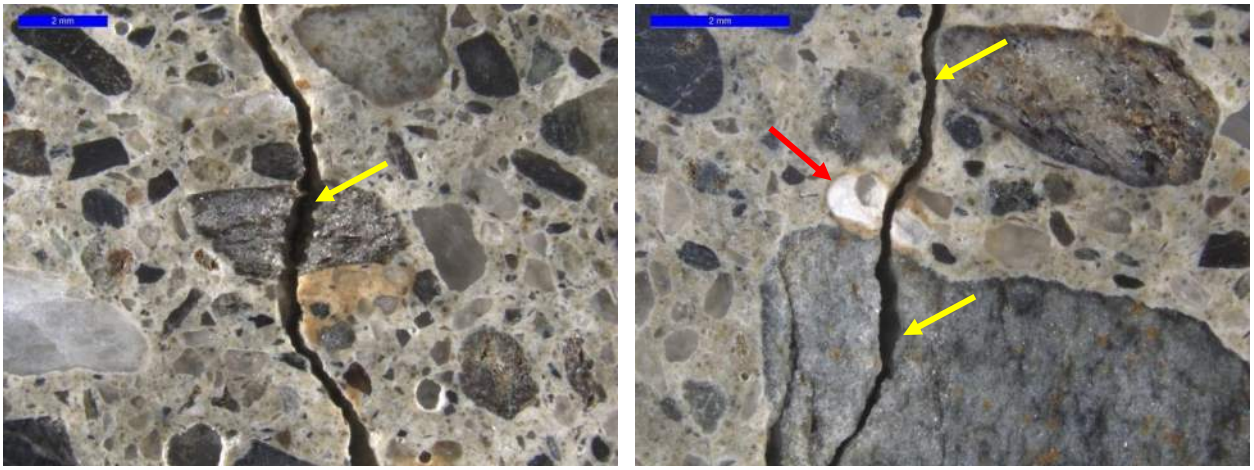


Figure 25. Core 13-01 (Arch/Horizontal). Surface-perpendicular cracks propagating through aggregate (yellow arrows). A small deposit of ASR gel was observed along one area of the crack (red arrow).

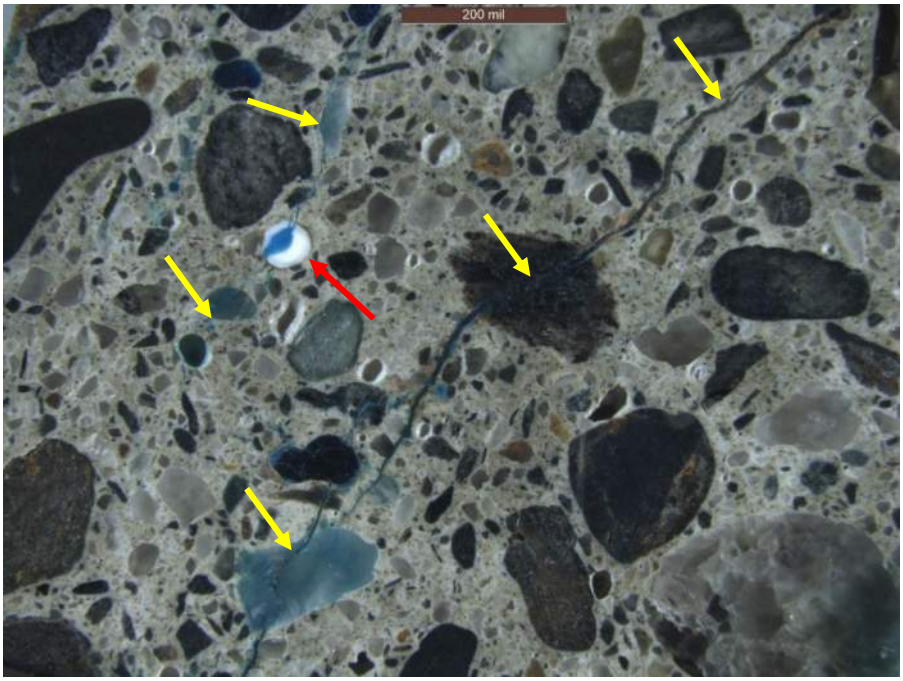


Figure 26. Core 13-01 (Arch/Horizontal). Evidence of ASR, including cracking within paste and internal fracturing of reactive aggregate particles extending into paste (yellow arrows) and deposits of ASR gel in voids and cracks (red arrow).



Figure 27. Core 15-01 (Abutment/Horizontal). Lapped longitudinal cross section (right) and cut face freshly sprayed with phenolphthalein solution of the exterior surface region (left). The exterior surface is at the top of the image. Note paste discoloration near the surface, which corresponds roughly to the depth of carbonation.

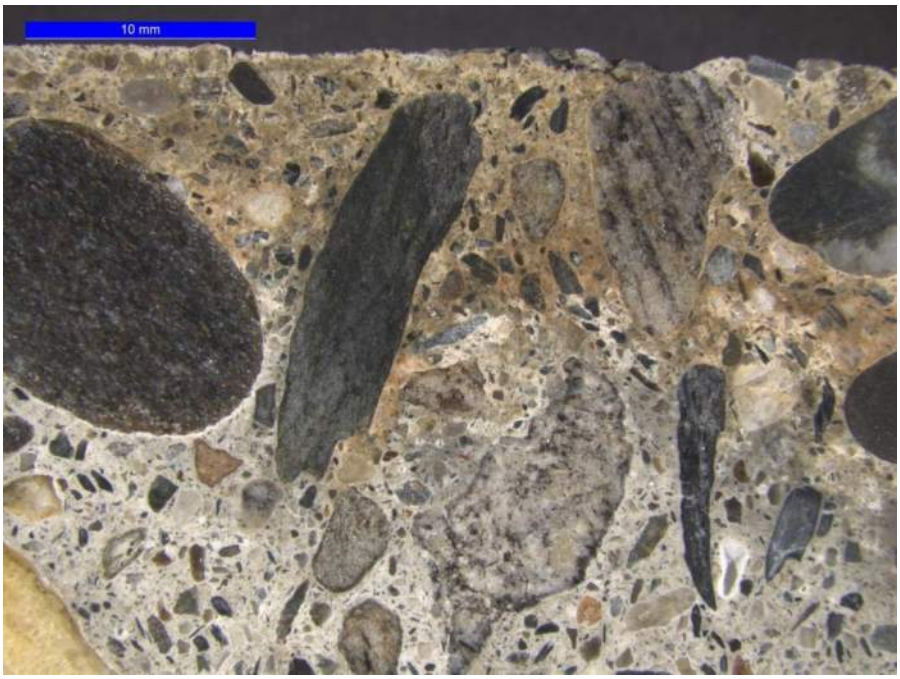


Figure 28. Core 15-01 (Abutment/Horizontal). Exterior surface showing orange paste discoloration along the surface, likely from carbonation.

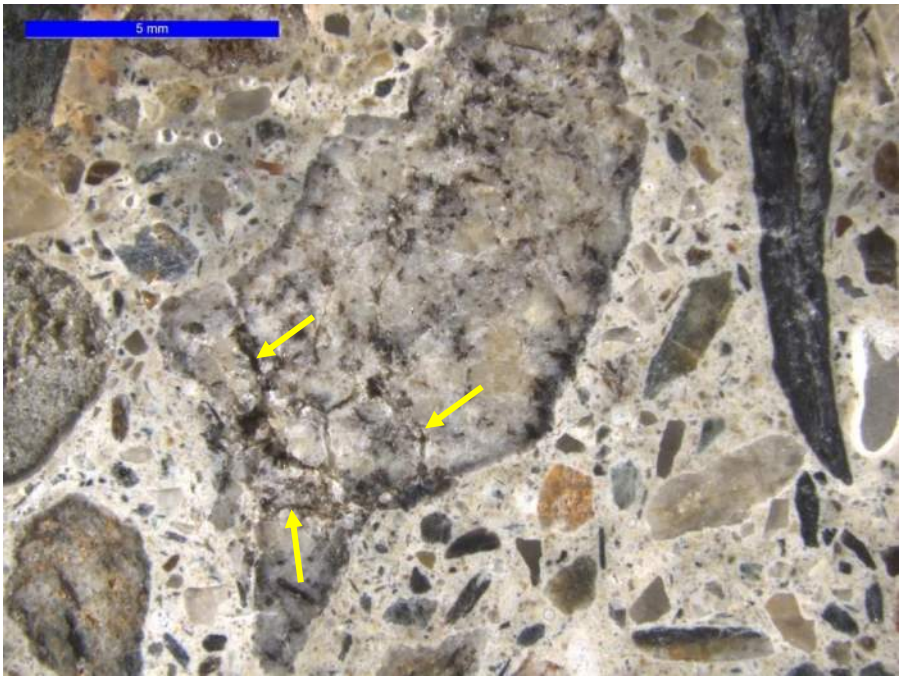


Figure 29. Core 15-01 (Abutment/Horizontal). Coarse aggregate particle exhibiting internal fracturing (yellow arrows).

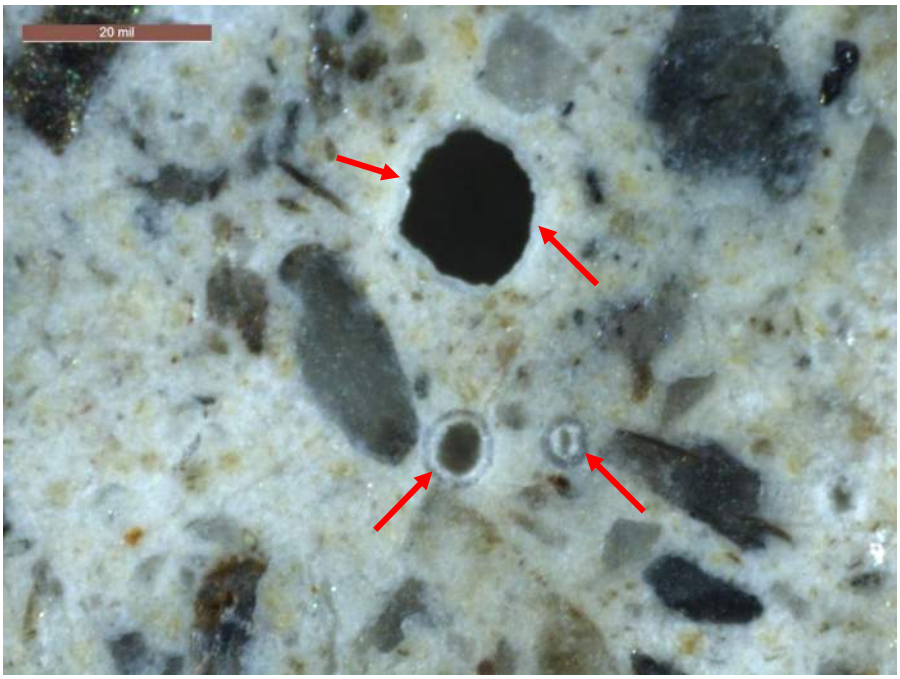


Figure 30. Core 15-01 (Abutment/Horizontal). Close-up image showing secondary deposits, likely ettringite or a mixture of ettringite and calcium hydroxide, within air voids (red arrows).

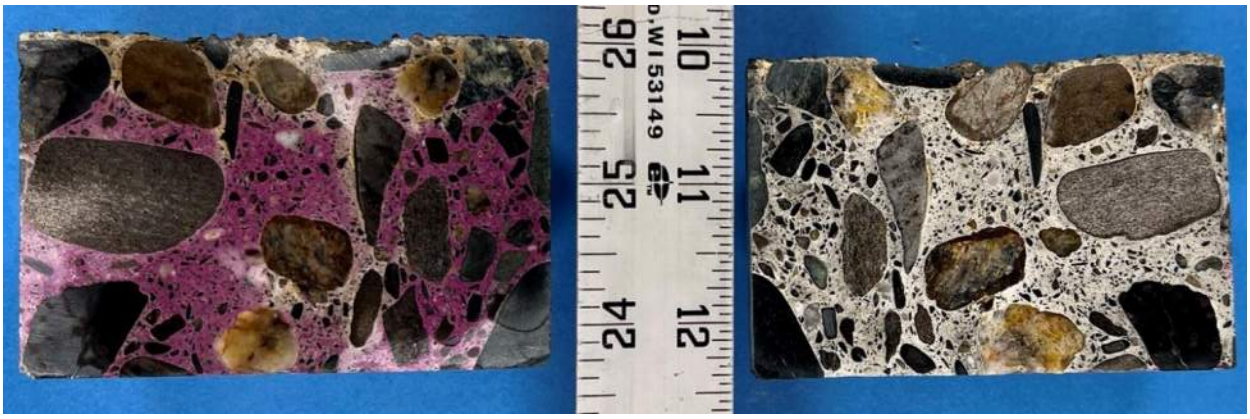


Figure 31. Core 17-01 (Retaining Wall/Horizontal). Lapped longitudinal cross section (right) and cut face freshly sprayed with phenolphthalein solution of the exterior surface region (left). The exterior surface is at the top of the image. paste discoloration near the surface, which corresponds roughly to the depth of carbonation.

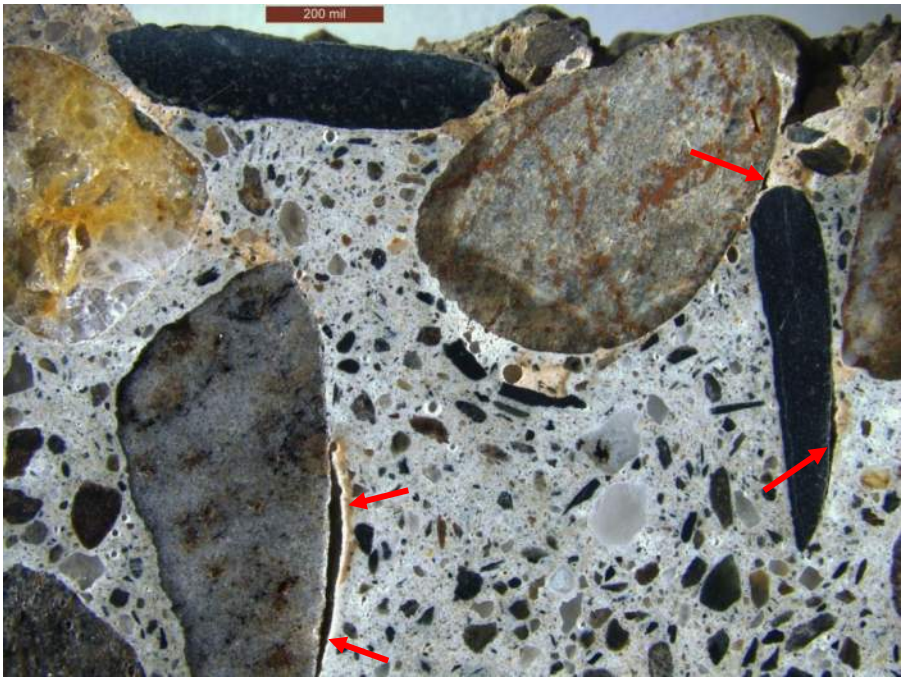


Figure 32. Core 17-01 (Retaining Wall/Horizontal). Exterior surface region, showing gaps between paste and aggregate (red arrows) and paste discoloration, likely from carbonation, along the gaps.

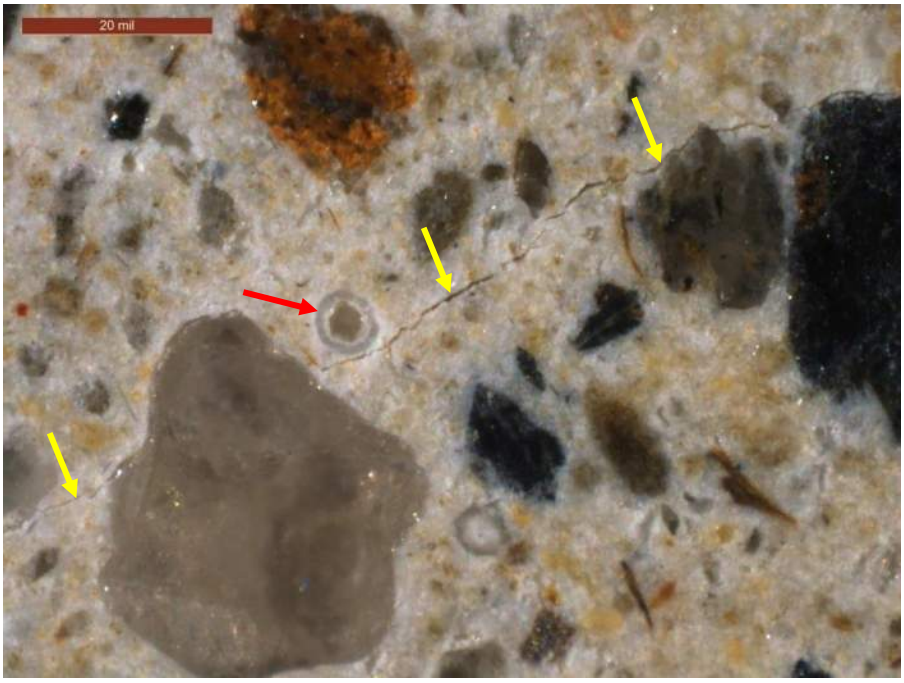


Figure 33. Core 17-01 (Retaining Wall/Horizontal). Close-up image of ettringite secondary deposits in voids (red arrows) and microcracking (yellow arrows).

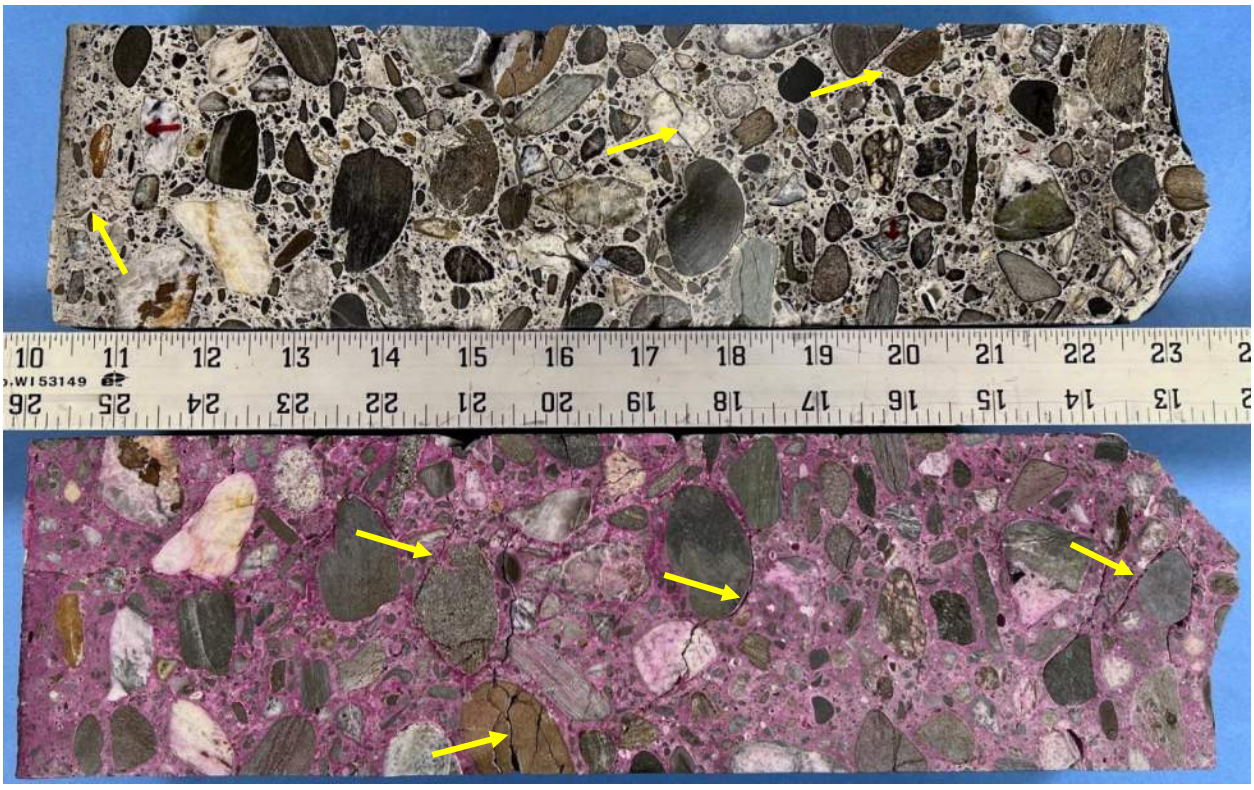


Figure 34. Core 17-02 (Retaining Wall/Vertical). Lapped longitudinal cross section (top) and freshly sprayed cut face of the exterior surface region. Yellow arrows indicate cracks within paste and aggregate.

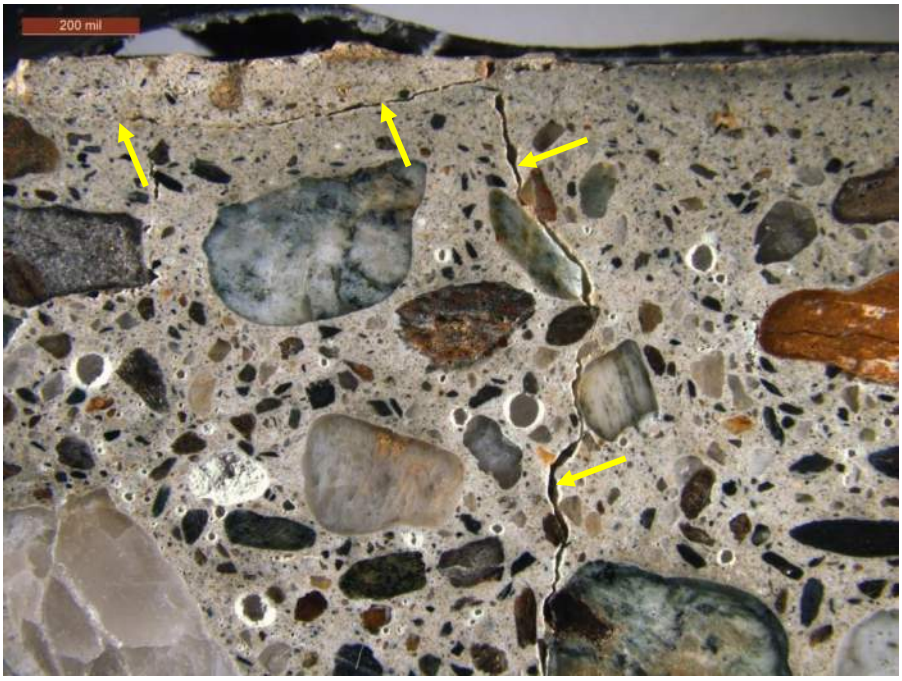


Figure 35. Core 17-02 (Retaining Wall/Vertical). Exterior surface region showing cracking (yellow arrows) oriented perpendicular and parallel to the surface.

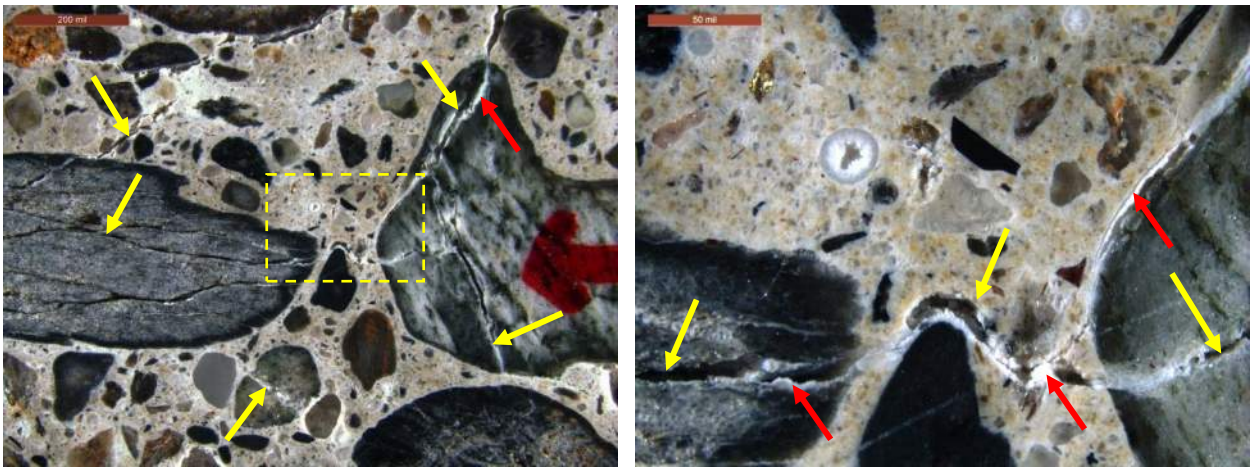


Figure 36. Core 17-02 (Retaining Wall/Vertical). Evidence of ASR, including internal fracturing of reactive aggregate particles extending into paste (yellow arrows) and deposits of ASR gel in cracks (red arrows). The area within the yellow box in the left image is shown at higher magnification on the right.

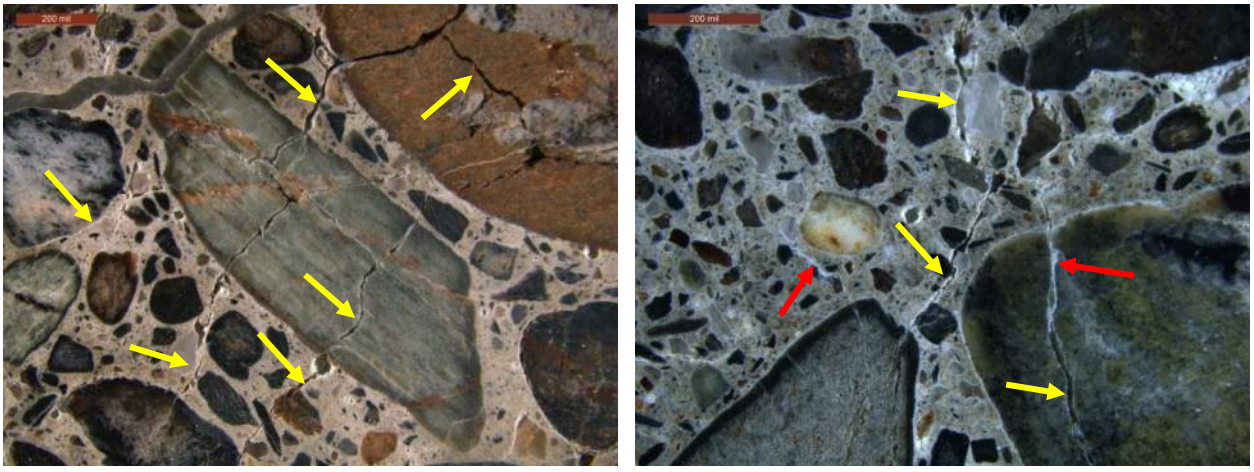


Figure 37. Core 17-02 (Retaining Wall/Vertical). Evidence of ASR, including internal fracturing of reactive aggregate particles extending into paste (yellow arrows) and deposits of ASR gel in voids and cracks (red arrows).

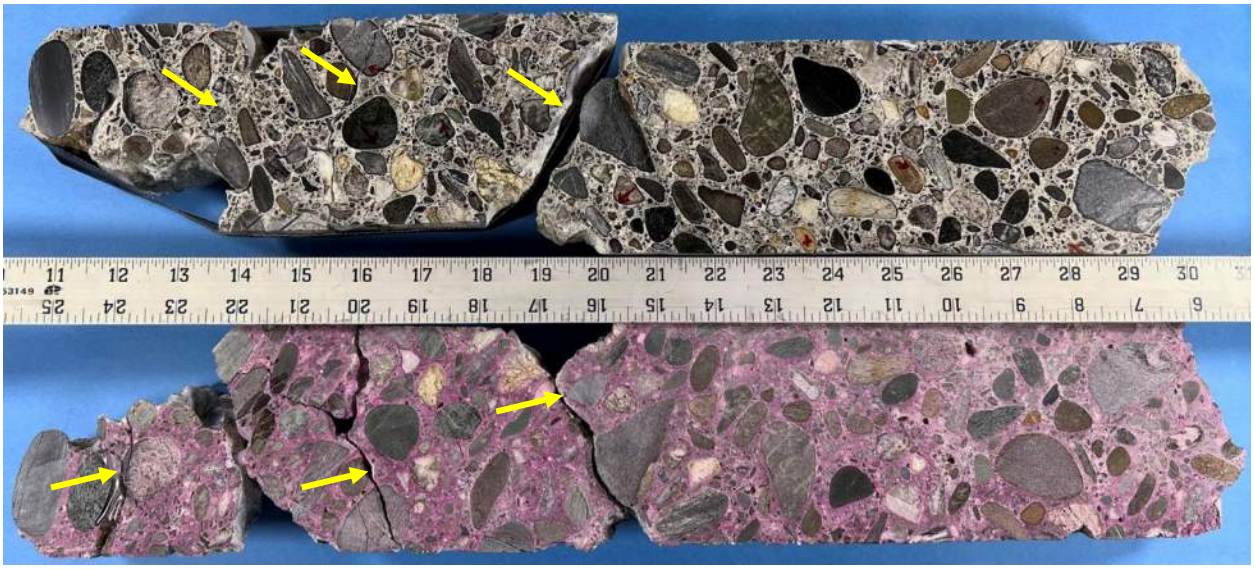


Figure 38. Core 17-03 (Retaining Wall/Horizontal). Lapped longitudinal cross section (top) and freshly sprayed cut face of the exterior surface region. Yellow arrows indicate cracks within paste and aggregate.

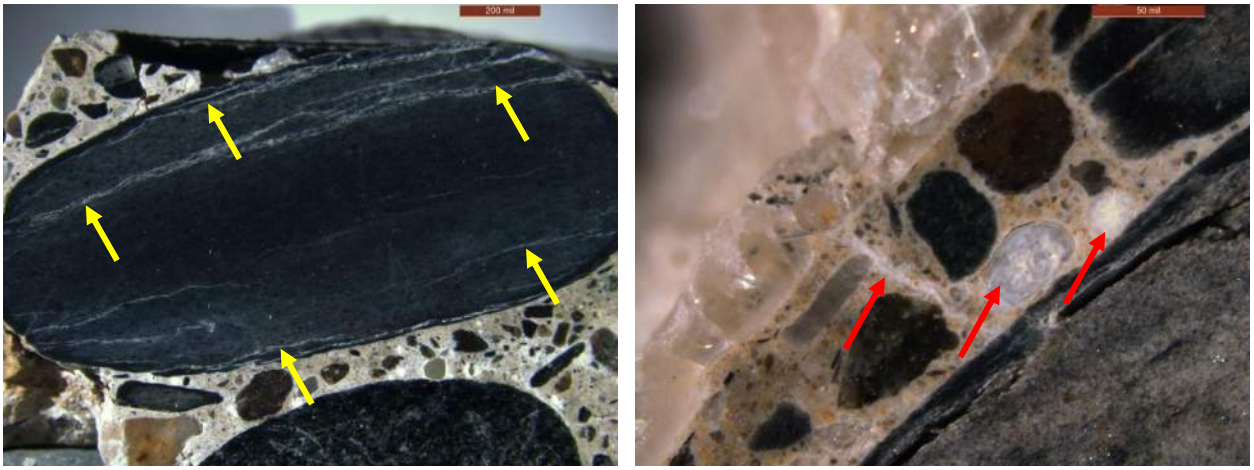


Figure 39. Core 17-03 (Retaining Wall/Horizontal). Evidence of ASR, including internal fracturing of reactive aggregate particles extending into paste (left image, yellow arrows) and deposits of ASR gel in voids and cracks (red arrows).

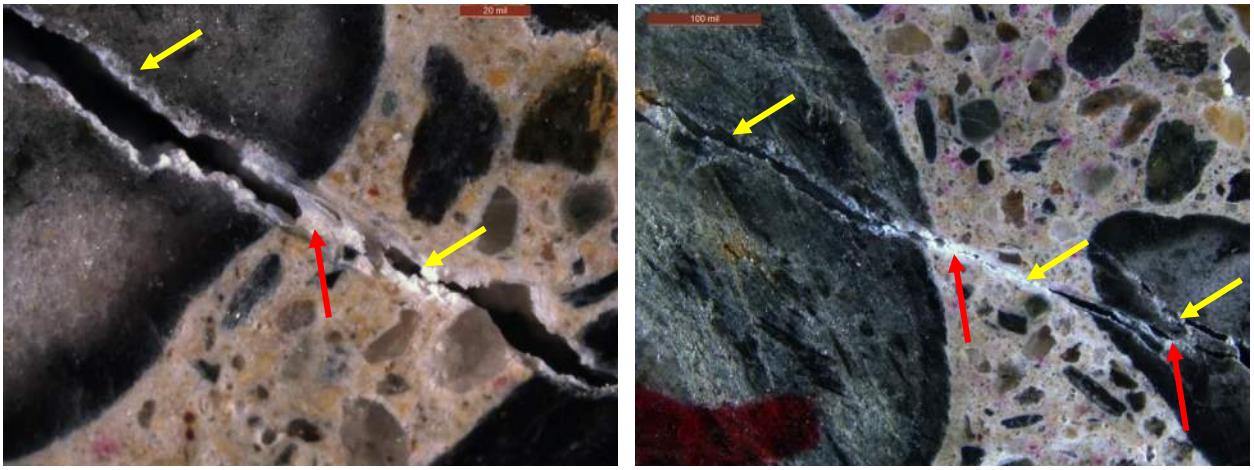


Figure 40. Core 17-03 (Retaining Wall/Horizontal). Evidence of ASR, including internal fracturing of reactive aggregate particles extending into paste (yellow arrows) and deposits of ASR gel in voids and cracks (red arrows).

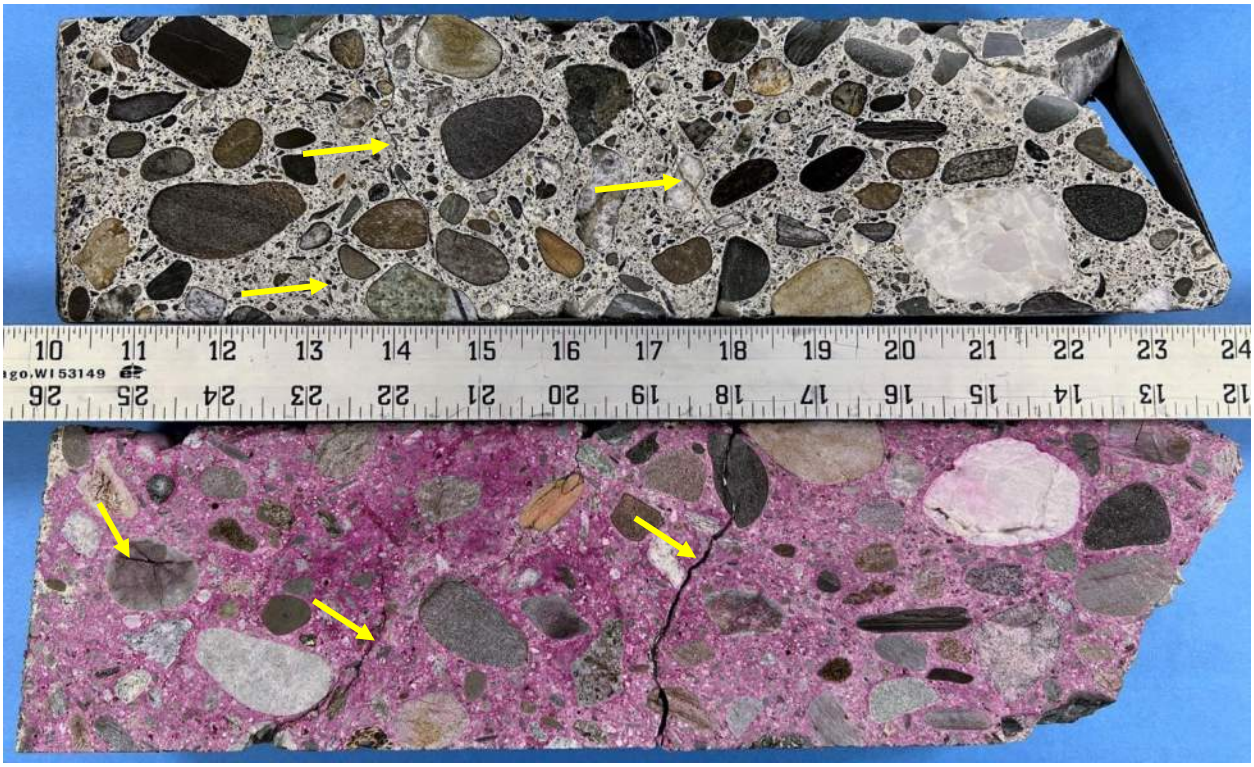


Figure 41. Core 18-01 (Abutment/Horizontal). Lapped longitudinal cross section (top) and freshly sprayed cut face of the exterior surface region. Yellow arrows indicate cracks within paste and aggregate.

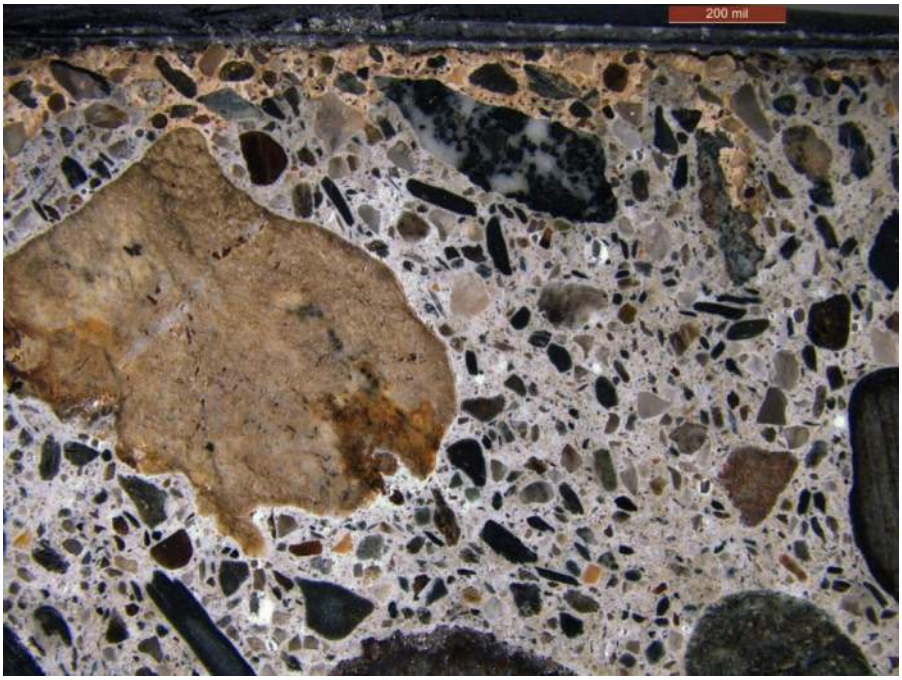


Figure 42. Core 18-01 (Abutment/Horizontal). Exterior surface region, showing orange paste discoloration, likely from carbonation, along the surface.

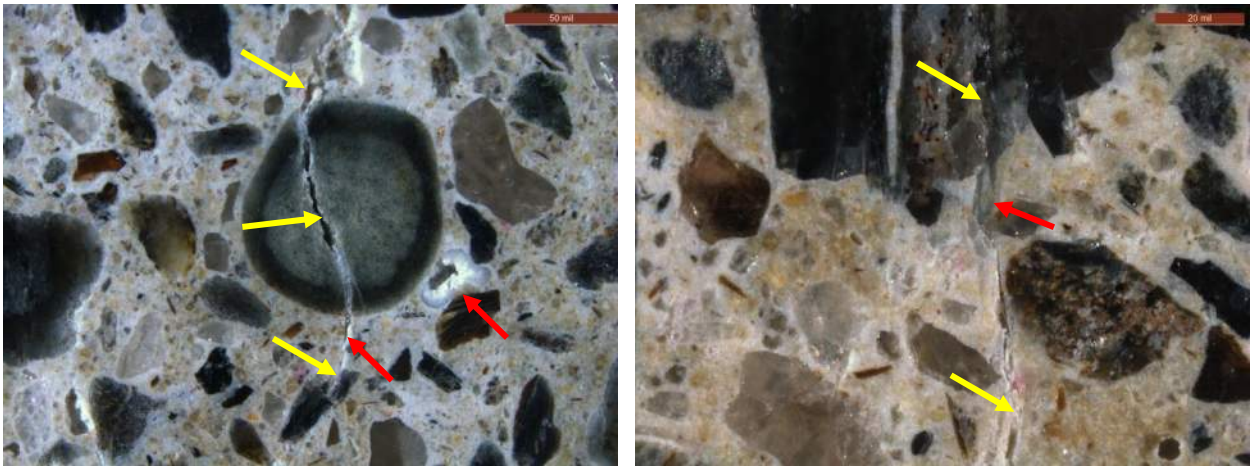


Figure 43. Core 18-01 (Abutment/Horizontal). Evidence of ASR, including internal fracturing of reactive aggregate particles extending into paste (yellow arrows) and deposits of ASR gel in voids and cracks (red arrows).

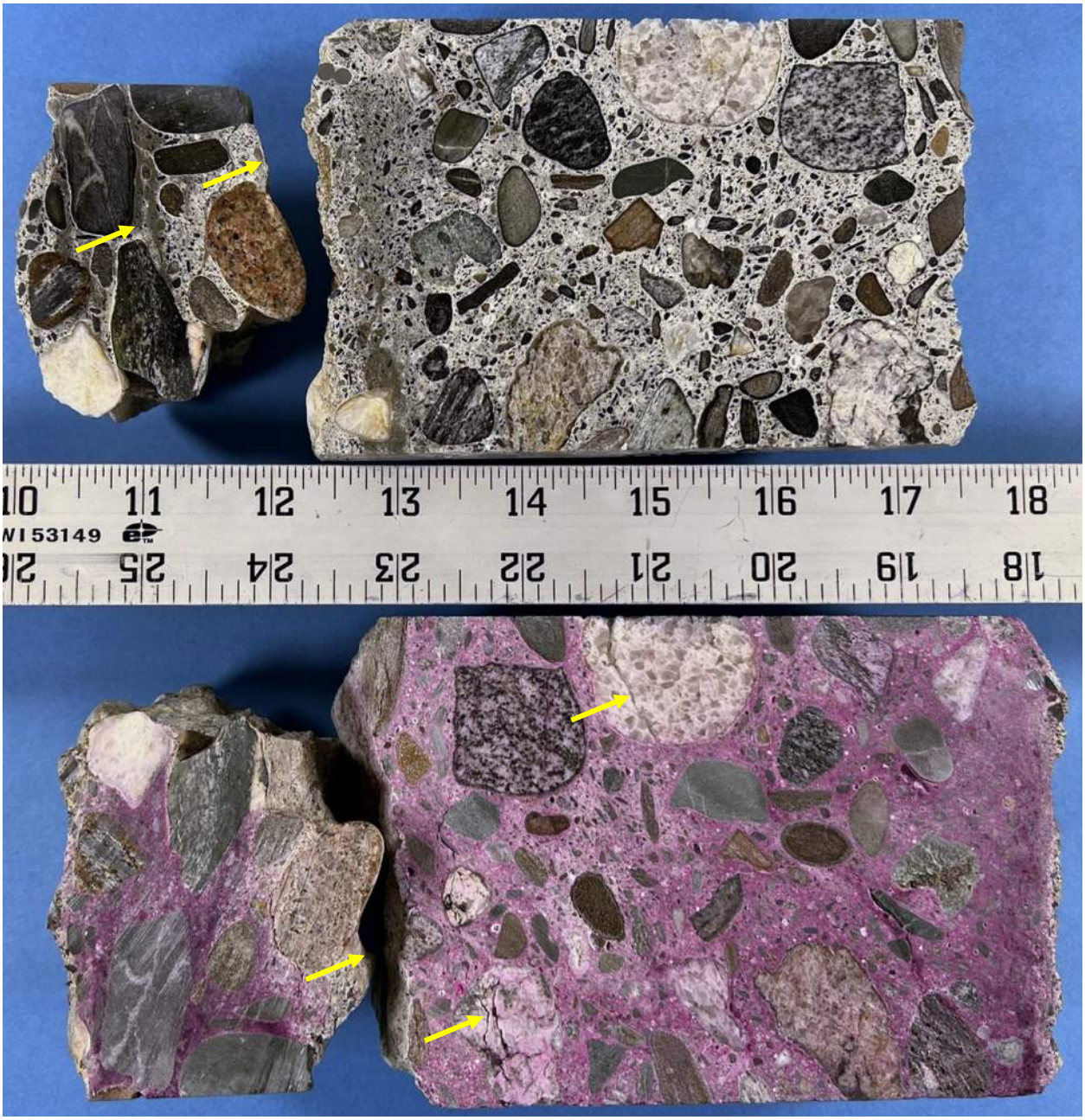


Figure 44. Core 18-02 (Abutment/Vertical). Lapped longitudinal cross section (top) and freshly sprayed cut face of the exterior surface region. Yellow arrows indicate cracks within paste and aggregate.

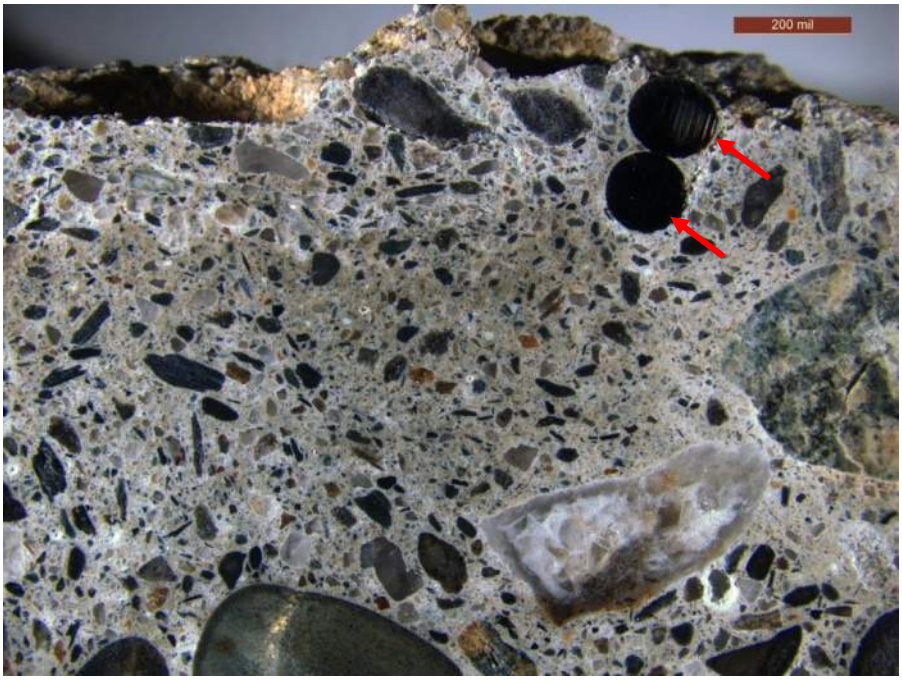


Figure 45. Core 18-02 (Abutment/Vertical). Exterior surface region, showing and irregularly surface due to fracturing. Red arrows indicate two steel wires near the surface.

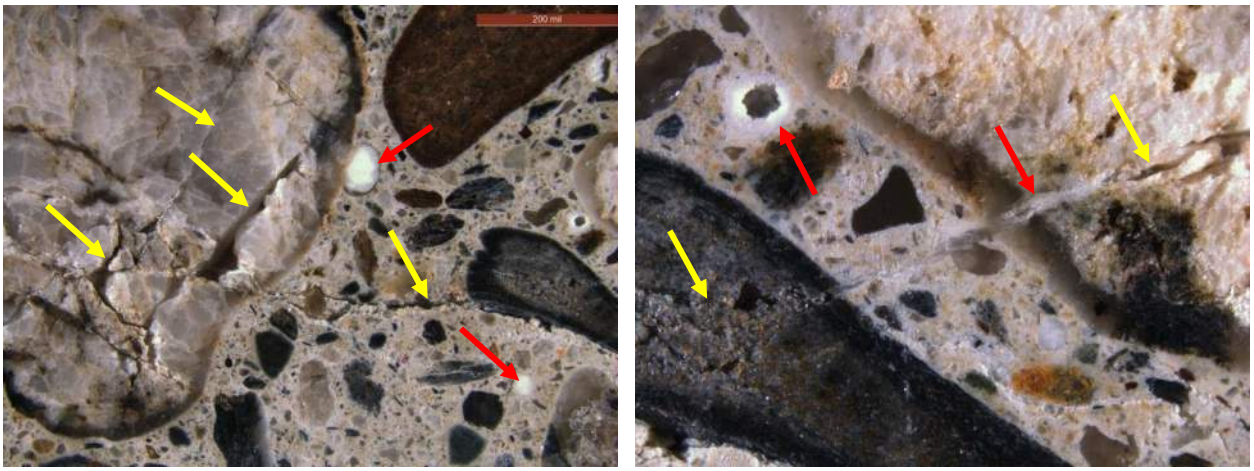


Figure 46. Core 18-02 (Abutment/Vertical). Evidence of ASR, including internal fracturing of reactive aggregate particles extending into paste (yellow arrows) and deposits of ASR gel in voids and cracks (red arrows).

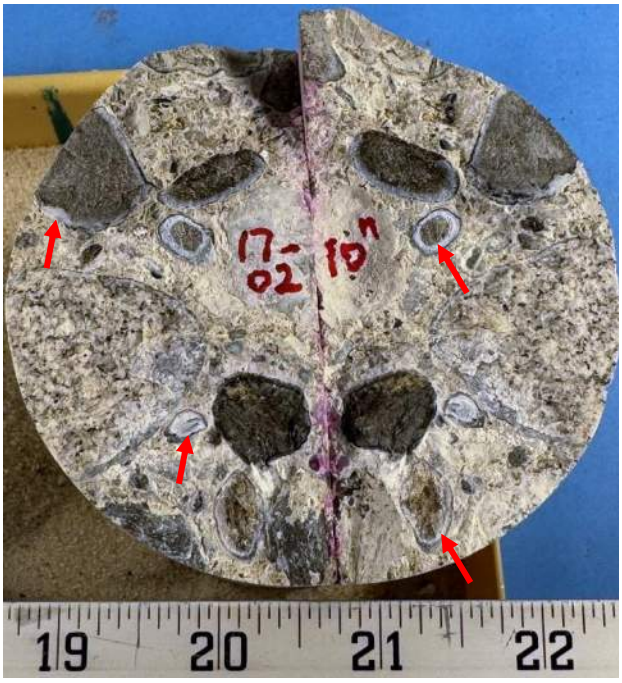


Figure 47. Core 17-02 (Retaining Wall/Vertical). Opposing fracture faces approximately 10 inches below the surface, showing white ASR gel (red arrows) on reactive aggregate particles. The fracture propagated through aggregate particles.



Figure 48. Core 17-02 (Retaining Wall/Vertical). Close-up view of ASR gel along the fracture surface shown in Figure 47. Accumulations of gel and desiccation cracks (yellow arrows) in the gel can be observed.

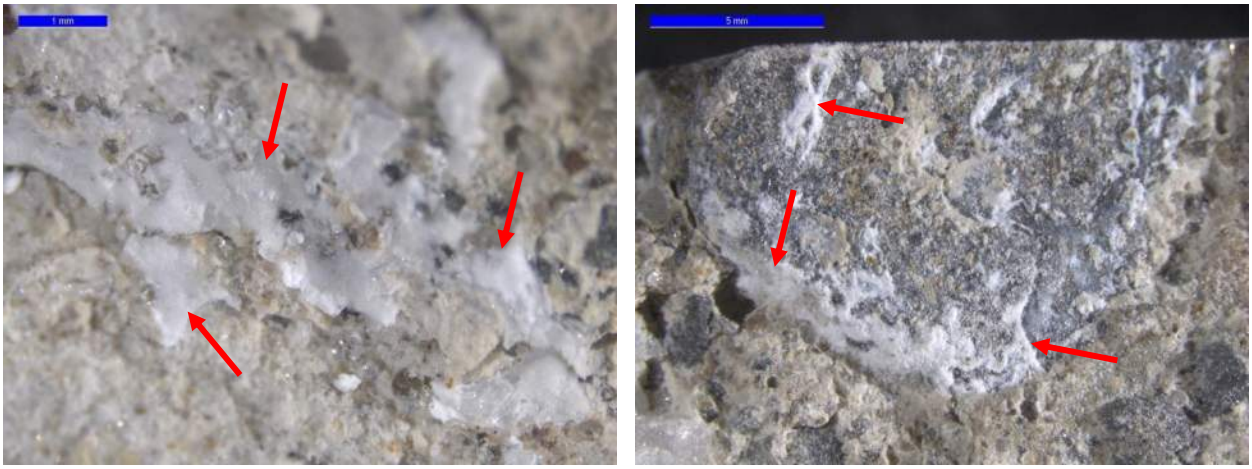


Figure 49. Core 08-01 (Floor Beam/Horizontal). ASR gel (red arrows) on paste and aggregate on a fractured surface of Core 08-01.

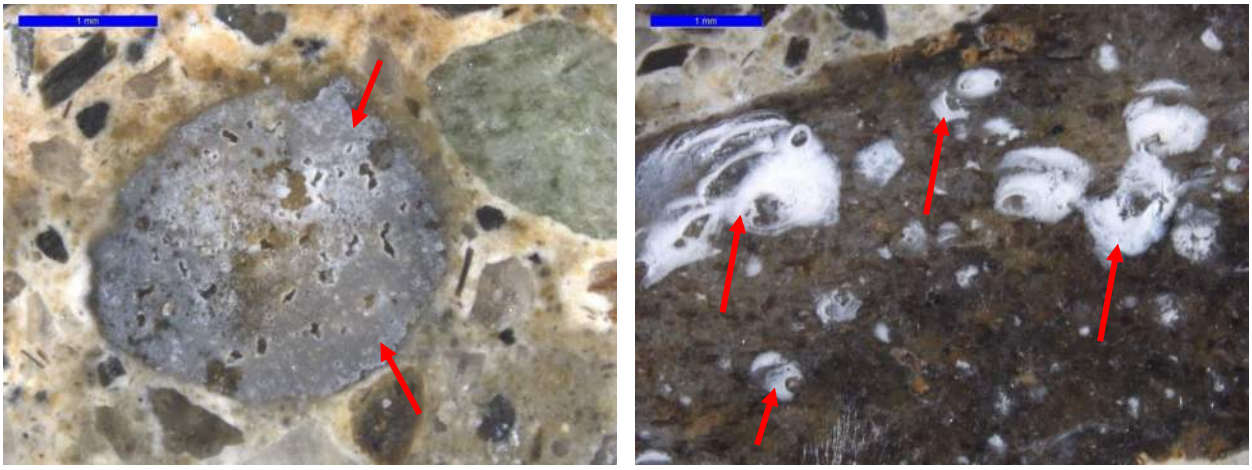


Figure 50. Core 01-01 (Arch/Horizontal). ASR gel (red arrows) that grew on the surface of reactive aggregate particles in Core 01-01 after being placed in 100% relative humidity at 73°F for approximately three days. The entire aggregate particle in the left image is covered in a thin layer of gel, giving it a hazy appearance.

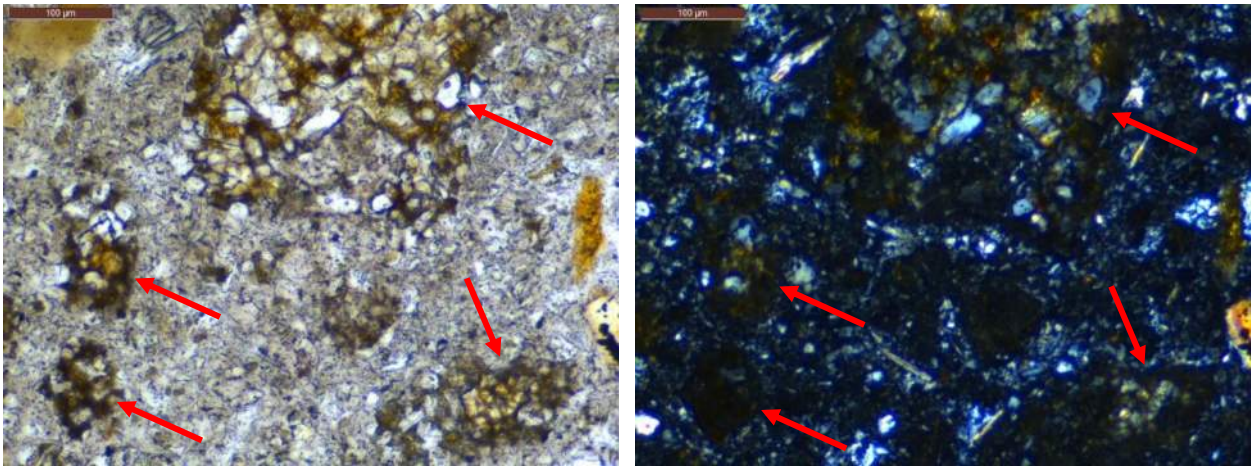


Figure 51. Core 08-01 (Floor Beam/Horizontal). Plane-polarized light (left) and cross-polarized light (right) images of the same area in showing typical portland cement particles in the paste (red arrows). The cement is relatively coarse compared to modern cement, which is typical for cement in concrete of this age.

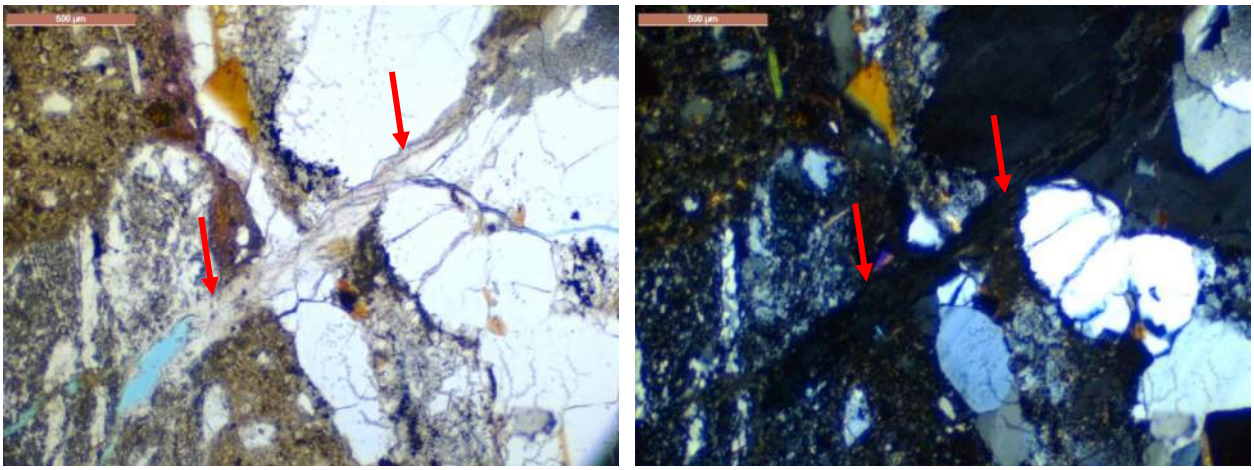


Figure 52. Core 11-01 (Deck/Vertical). Plane-polarized light (left) and cross-polarized light (right) images of the same area showing ASR gel (red arrows) within a crack extending from a reactive aggregate particle into surrounding paste.

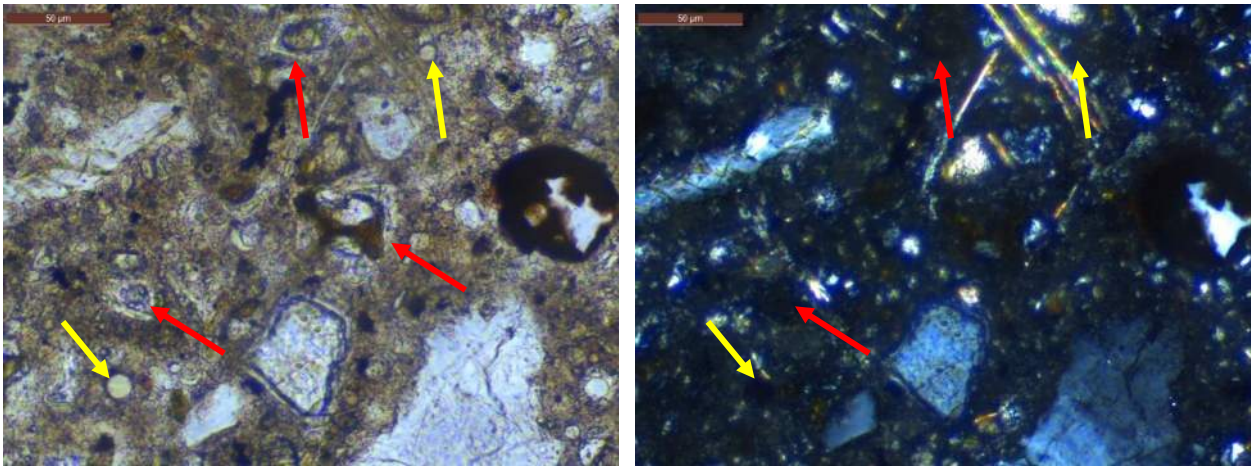


Figure 53. Core 11-01 (Deck/Vertical). Plane-polarized light (left) and cross-polarized light (right) images of the same area within the apparent concrete repair concrete showing typical portland cement particles (red arrows) and fly ash (yellow arrows) in the paste.

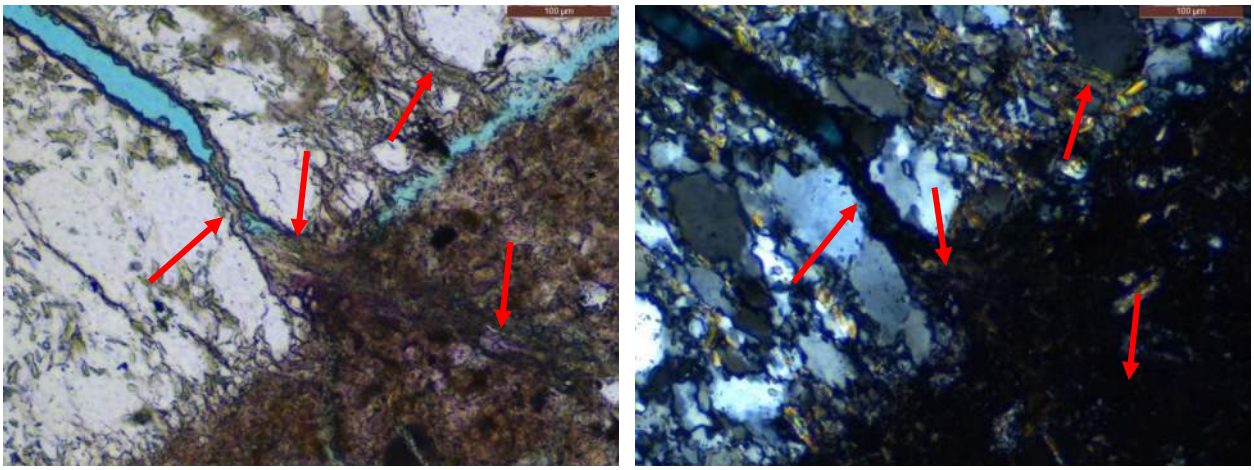


Figure 54. Core 11-01 (Deck/Vertical). Plane-polarized light (left) and cross-polarized light (right) images of the same area in the bulk concrete showing ASR gel (red arrows) within a crack extending from a reactive aggregate particle into surrounding paste.

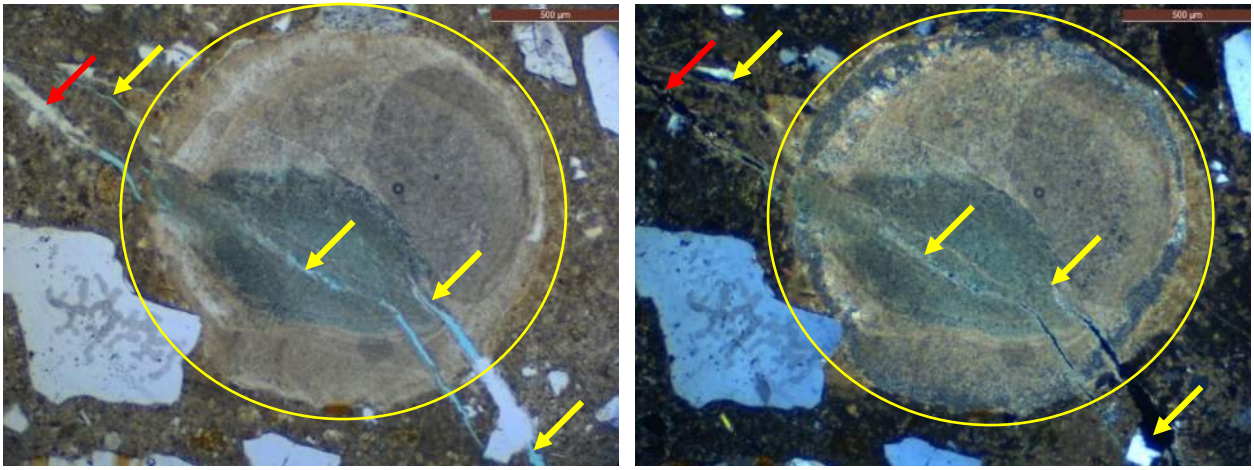


Figure 55. Core 13-01 (Arch/Horizontal). Plane-polarized light (left) and cross-polarized light (right) images of the same area in Core 13-01 showing microcracking (yellow arrows) and ASR gel in cracking (red arrows) and a void (circled). The gel appeared to have partially carbonated or altered.

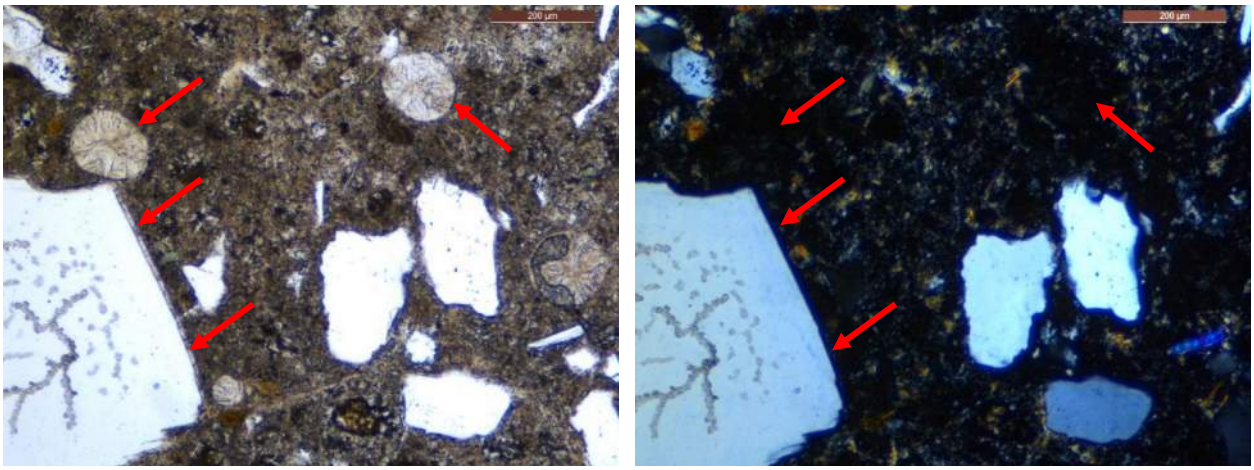


Figure 56. Core 15-01 (Abutment/Horizontal). Plane-polarized light (left) and cross-polarized light (right) images of the same area showing secondary deposits of ettringite in air voids and in a gap between paste and a sand particle (red arrows).

APPENDIX E. METALLURGICAL TESTING REPORT

Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, Illinois 60062
847.272.7400 tel
www.wje.com

MEMORANDUM | December 4, 2024

BRIDGE 55 OVER MILL BROOK IN WINDSOR, VERMONT

Metallurgical Testing of Existing Reinforcing Bars

WJE PROJECT NO. 2023.7260

TO	Tanner Swenson, Stephen Garrett, Arne Johnson
FROM	Scott Graham, Project Associate, Janney Technical Center

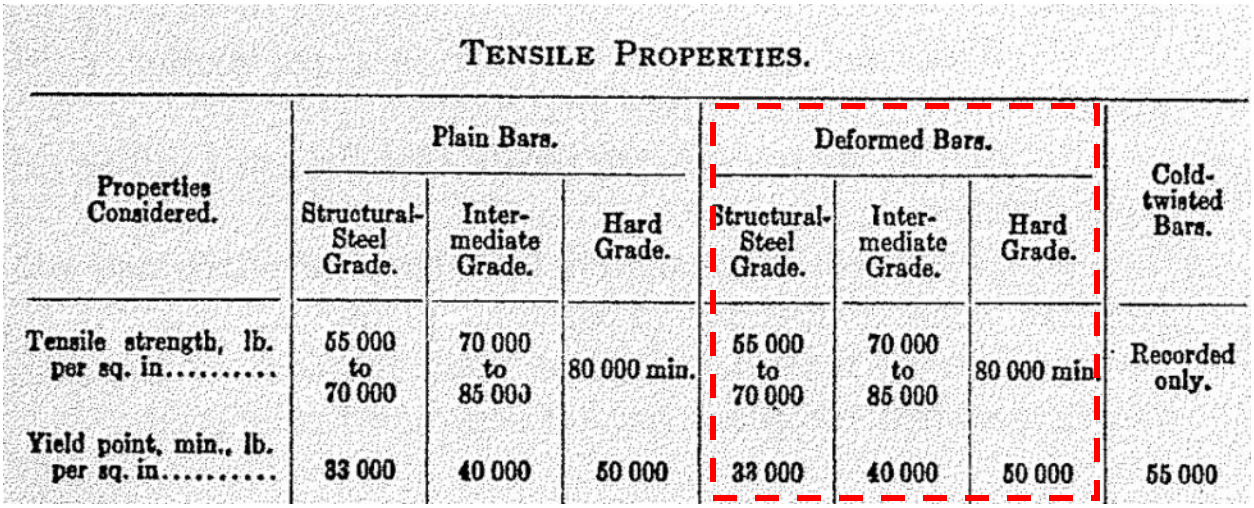
The following is a summary of the monotonic tension testing and chemistry testing conducted on reinforcing bars removed from various locations of Bridge 55 Over Mill Brook, located in Windsor, Vermont. Wiss, Janney, Elstner Associates, Inc. (WJE) is an accredited testing laboratory, recognized by ANSI National Accreditation Board (ANAB) under Certificate AT-2564.

Tension tests were performed on November 21, 2024 and November 25, 2024 at WJE laboratories in Northbrook, Illinois using the 120-kip Satec universal test machine having mechanical grips and the 500-kip Riehle test machine having pneumatic grips. The intent of the tests was to determine the yield strength and tensile strength of the reinforcing bars. Tests were performed in general accordance with the tension test methods specified in ASTM A370-24 "*Standard Test Methods and Definitions for Mechanical Testing of Steel Products*." Results are summarized in Table 1 below. The grade of the reinforcing bar was determined by comparing the yield strength and tensile strength results to the requirements specified in ASTM A-15 (1914) *Standard Specification for Billet-Steel Concrete Reinforcing Bars;* the reinforcing bar standard in effect at the time of bridge construction, reportedly 1930. These requirements are shown in Figure 1.

Table 1. Tension Test Results

Test I.D. No.	Bridge Location	Bar Type	Bar Size	Bar Area (in^2)	Yield Strength (0.2% Offset) f_{ya}		Tensile Strength, f_{ua}			Grade
					(kips)	(ksi)	(kips)	(ksi)	$\%f_{ya}$	
11215	Arch Rib	Round, deformed	3/4"	0.44	25.0	56.9	39.1	89.0	156%	Hard
11216	Railing	Round, deformed	11/16"	0.37	17.0	46.0	28.4	76.8	167%	Intermediate
11217	Deck	Round, deformed	7/16"	0.15	6.7	44.5	9.5	63.4	142%	Structural Steel
11218	Arch Rib	Square, deformed	1-1/8"	1.27	54.4	42.9	87.5	68.9	161%	Intermediate*

*The tensile strength listed in the table does not conform to the specified tensile strength listed for Intermediate deformed bars however the fracture occurred in a portion of the bar that had more significant section loss. It is therefore our opinion that the load at fracture under-represents the load at fracture had the bar been uncorroded. Considering how close the listed tensile strength is to the tensile strength required for Intermediate deformed bars, it is our opinion that an uncorroded bar would have satisfied the tensile strength required for Intermediate deformed bars.



TENSILE PROPERTIES.

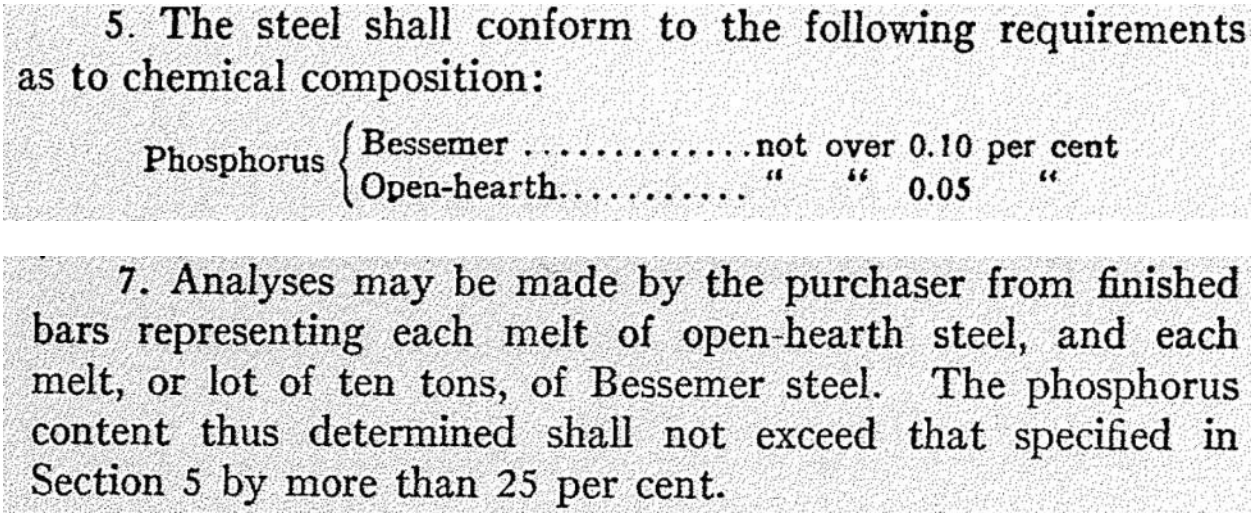
Properties Considered.	Plain Bars.			Deformed Bars.			Cold-twisted Bars.
	Structural-Steel Grade.	Intermediate Grade.	Hard Grade.	Structural-Steel Grade.	Intermediate Grade.	Hard Grade.	
Tensile strength, lb. per sq. in.....	55 000 to 70 000	70 000 to 85 000	80 000 min.	55 000 to 70 000	70 000 to 85 000	80 000 min.	Recorded only.
Yield point, min., lb. per sq. in.....	33 000	40 000	50 000	33 000	40 000	50 000	55 000

Figure 1. Excerpt from ASTM A-15 (1914) showing tensile strength requirements for reinforcing bars.

Chemical composition tests were performed on November 26, 2024 at WJE laboratories. Test samples were evaluated by optical emission spectroscopy (OES) in accordance with ASTM E415-21, *Standard Test Method for Analysis of Carbon and Low-Alloy Steel by Spark Atomic Emission Spectrometry*. Test results are shown in Table 2. Test results for the "Arch Rib 1-1/8" and "Deck 7/8" samples are generally consistent with the chemical composition properties specified for all grades of ASTM A15 (1914) deformed reinforcing bars (see Figure 2). Test results for the "Railing 11/16", "Arch Rib 3/4", and Deck 7/16" samples exhibited concentrations of phosphorus that exceed the requirements of ASTM A15 (1914). If welding to any of these bars is considered, we recommend that carbon equivalent (CE) calculations and metallographic examinations be performed in order to determine guidelines so an appropriate welding procedure can be developed for each bar.

Table 2. Chemical Composition Test Results

Sample	Arch Rib 1-1/8" Square	Deck 7/8" Round	Railing 11/16" Round	Arch Rib 3/4" Round	Deck 7/16" Round
Carbon	0.36%	0.30%	0.31%	0.23%	0.23%
Manganese	0.53	0.93	0.83	0.67	0.86
Phosphorus	0.010	0.058	0.144	0.075	0.071
Sulfur	0.038	0.057	0.127	0.056	0.046
Silicon	0.15	<0.05	<0.05	<0.05	<0.05
Copper	<0.05	<0.05	<0.05	< 0.05	<0.05
Nickel	0.10	<0.05	<0.05	<0.05	<0.05
Chromium	0.07	<0.05	<0.05	<0.05	<0.05
Molybdenum	<0.03	<0.03	<0.03	<0.03	<0.03



5. The steel shall conform to the following requirements as to chemical composition:

Phosphorus	Bessemer	not over 0.10 per cent
	Open-hearth	" " 0.05 "

7. Analyses may be made by the purchaser from finished bars representing each melt of open-hearth steel, and each melt, or lot of ten tons, of Bessemer steel. The phosphorus content thus determined shall not exceed that specified in Section 5 by more than 25 per cent.

Figure 2. Excerpts from ASTM A-15 (1914) showing chemical composition requirements for reinforcing bars.

Appendix S: Basis of Estimate

Basis of Estimate – Concept Selection

This estimate was created "contractor style" with Heavy Bid Software. The assumed project bid items and quantities were estimated based on the as-built drawings. This estimate conforms to an AACE Class 4 estimate.

Total construction cost was calculated in 2025 dollars, the table below adds 20% contingency to 2025 dollars and provides escalated value for upcoming fiscal years. Each year is escalated by 3.5% per year to account for inflation.

FY	Alt 1	Alt 1 ABC	Alt 2	Alt 2 ABC	Alt 2w	Alt 2w ABC
2025	$ 3,726,200	$ 3,792,200	$ 4,275,000	$ 4,413,000	$ 5,199,000	$ 5,527,000
20%	$ 745,240	$ 758,440	$ 855,000	$ 882,600	$ 1,039,800	$ 1,105,400
2025	$ 4,471,440	$ 4,550,640	$ 5,130,000	$ 5,295,600	$ 6,238,800	$ 6,632,400
2026	$ 4,627,940	$ 4,709,912	$ 5,309,550	$ 5,480,946	$ 6,457,158	$ 6,864,534
2027	$ 4,789,918	$ 4,874,759	$ 5,495,384	$ 5,672,779	$ 6,683,159	$ 7,104,793
2028	$ 4,957,565	$ 5,045,376	$ 5,687,723	$ 5,871,326	$ 6,917,069	$ 7,353,460
2029	$ 5,131,080	$ 5,221,964	$ 5,886,793	$ 6,076,823	$ 7,159,166	$ 7,610,832
2030	**$ 5,310,668**	**$ 5,404,733**	**$ 6,092,831**	**$ 6,289,512**	**$ 7,409,737**	**$ 7,877,211**

Schedule – The assumed construction start date is April 1, 2030. The estimate is based on 5-day work week at 10 hours per day with standard holidays. The schedule provides duration for time dependent features in the estimate.

Labor /Fringe wages are based on Vermont Prevailing Wages – Southern. Davis-Bacon wages were lower. Productivity rates/crew composition are based on HNTB historical data.

Permanent materials are assumed to be tax exempt. Consumable materials are taxed at 6% state sales tax rate with 0% local sales tax rate.

Equipment is assumed to be owned by the Contractor with rates per Blue Book.

Civil bid items contain small quantities which lead to relatively high prices compared to standard VTrans bid prices. Architectural bridge repairs with no access below require extensive scaffolding. Structural items are also small quantities, and higher than usual unit prices due to quantities and the difficultly in performing the construction. All concrete repair is assumed to 4" thick for quantities supplied by WJE. This cost can be further refined as design progresses. All existing concrete members received point and patch surface treatment to create a uniform color and appearance.

Repairs to the abutment footings will require access at the river level. It is assumed that the contractor will not be able to drive sheet pile or piling and will have to use super sacks filled

with sand to create a dry working environment. It is also assumed that the contractor will perform this work during low flow periods.

Overhead power lines run along the east side of the bridge and near the north abutment. Depending on voltage, cranes are required to be a minimum of ten feet and possibly more from these lines. It is assumed they will be relocated prior to construction and this cost is not included in the estimate.

Mobilization includes hauling of equipment, overhead staff, contractor field office, project dumpsters, maintenance and small tools. Markups include 3% insurance & bonds, and 15% for contractor overhead and profit. Program engineering costs are calculated separately from the conceptual construction costs.

Alternate 1 is conventional construction methods. The road is closed to traffic. Utilities are assumed to be moved before construction. Tiered scaffolding is installed from the arches. See the attached falsework plan. Each approach will have a crane for swinging scaffolding, falsework, forms and materials to workers. Railings and deck are removed, processed into rubble and hauled to a legal dumpsite.

Substructure repairs can begin once the falling debris has been cleared. Floor beams 3 and 6 are replaced. Soffit falsework is planned for all floor beam removal, replacement and repairs. Falsework is assumed to be designed by the contractor based on their preferred means and methods. After the deck is poured, decorative concrete railings will replace the existing concrete rail.

Alternate 1 ABC modifies the replaced floor beams and deck into precast sections.

Alternate 2 is similar to Alternate 1 but expands the floor beam replacement to all 8 floor beams. Columns are also replaced for each floor beam. Cast in place (CIP) concrete construction was priced for the columns, floor beams and bridge deck. Rebar quantities were modified to reflect estimated quantities of CIP concrete.

Alternate 2 ABC includes CIP columns, precast floor beams, and precast deck panels.

Alternate 2w widens the structure by 6 feet on the east side. The approaches get floor beams that extend over the wingwalls to create the widening. The deck concrete is also extended by 6 feet. Rebar quantities were modified to reflect estimate CIP concrete.

Alternate 2w ABC uses extended length precast floor beams on the approaches. Precast deck panel quantities are increased for the approaches and deck widening.

Alternate 1 and Alternate 1 ABC assume a higher quantity of anodes to preserve the existing substructure. Alternate 2 options replace the substructure and therefore less anodes were assumed for preservation.

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
205	CLEARING & GRUBBING	1.000	LS	8,500.00	8,500.00
210	REMOVE BITUMINOUS PAVEMENT	10,000.000	SF	4.00	40,000.00
215	REMOVE SIGNS	2.000	EA	110.00	220.00
220	STRUCTURE EXCAVATION	200.000	CY	142.00	28,400.00
225	GRANULAR BORROW FOR STRUCTURES	100.000	CY	108.00	10,800.00
230	REMOVE SIDEWALK	2,600.000	SF	2.00	5,200.00
235	REMOVE CATCH BASIN	5.000	EA	520.00	2,600.00
301	SUBBASE DENSE GRADED CRUSHED STONE	50.000	CY	150.00	7,500.00
401	AGGREGATE SURFACE COURSE	50.000	CY	190.00	9,500.00
404	TACK COAT	10.000	CWT	75.00	750.00
406	BITUMINOUS PAVEMENT	250.000	TON	300.00	75,000.00
500	TIERED SCAFFOLDING & STAIR TOWERS	1.000	LS	385,000.00	385,000.00
505	REMOVE BRIDGE DECK	2,700.000	SF	40.00	108,000.00
510	REMOVE RAILING	500.000	LF	72.00	36,000.00
515	REMOVE FLOORBEAMS	14.000	CY	10,225.00	143,150.00
520	CAST IN PLACE FLOORBEAMS	14.000	CY	6,361.00	89,054.00
535	REPAIR FLOORBEAMS 1,2,4,5,7,8	183.000	SF	845.00	154,635.00
540	REPAIR COLUMNS	122.000	SF	295.00	35,990.00
545	REPAIR ARCH RIBS	832.000	SF	220.00	183,040.00
550	REPAIR TIE BEAMS	55.000	SF	265.00	14,575.00
555	REPAIR ABUTMENTS	468.000	SF	275.00	128,700.00
560	REPAIR WINGWALLS	590.000	SF	425.00	250,750.00
562	EPOXY CRACK REPAIR	2,066.000	LF	180.00	371,880.00
565	COATING ALL SURFACES - SILANE	9,838.000	SF	2.00	19,676.00
570	REINFORCING STEEL	28,800.000	LB	3.80	109,440.00
575	CONCRETE BRIDGE DECK	70.000	CY	4,900.00	343,000.00
580	CONCRETE BRIDGE RAILING	500.000	LF	348.00	174,000.00
585	CORROSION MITIGATION - ANODES	800.000	EA	52.00	41,600.00
590	BRIDGE EXPANSION JOINTS - ASPHALTIC PLUG	80.000	LF	258.00	20,640.00
595	SCOUR REPAIR	2.000	EA	31,000.00	62,000.00
598	FUTURE BRIDGE UTILITY CONDUIT	800.000	LF	37.50	30,000.00
605	CATCH BASIN INLETS	6.000	EA	3,100.00	18,600.00
615	18" REINFORCED CONCRETE PIPE	265.000	LF	75.00	19,875.00
620	PAVEMENT MARKINGS 4"	2,000.000	LF	1.25	2,500.00

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
625	PAVEMENT MARKING 24" STOP BAR	12.000	LF	50.00	600.00
645	SIDEWALK	2,600.000	SF	12.00	31,200.00
650	VALLEY GUTTER	75.000	LF	43.00	3,225.00
655	CURB & GUTTER	350.000	LF	22.00	7,700.00
660	DRIVEWAY APRON	230.000	SF	23.00	5,290.00
665	TOPSOIL	50.000	CY	200.00	10,000.00
670	HYDROSEEDING	2,000.000	SF	1.25	2,500.00
675	SIGNS	3.000	EA	600.00	1,800.00
680	FIELD OFFICE, ENGINEER'S	1.000	LS	20,000.00	20,000.00
685	QC/QA TESTING	1.000	LS	19,000.00	19,000.00
690	EROSION CONTROL	1.000	LS	53,000.00	53,000.00
695	MOBILIZATION	1.000	LS	594,110.00	594,110.00
696	CPM SCHEDULE	6.000	EA	1,200.00	7,200.00
699	TRAFFIC CONTROL - ALL INCLUSIVE	1.000	LS	40,000.00	40,000.00
	Bid Total				**$3,726,200.00**

20 % Contingency = $745,240
Total w/Contingency = $4,471,440

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
205	CLEARING & GRUBBING	1.000	LS	8,500.00	8,500.00
210	REMOVE BITUMINOUS PAVEMENT	10,000.000	SF	4.00	40,000.00
215	REMOVE SIGNS	2.000	EA	110.00	220.00
220	STRUCTURE EXCAVATION	200.000	CY	142.00	28,400.00
225	GRANULAR BORROW FOR STRUCTURES	100.000	CY	108.00	10,800.00
230	REMOVE SIDEWALK	2,600.000	SF	2.00	5,200.00
235	REMOVE CATCH BASIN	5.000	EA	520.00	2,600.00
301	SUBBASE DENSE GRADED CRUSHED STONE	50.000	CY	150.00	7,500.00
401	AGGREGATE SURFACE COURSE	50.000	CY	190.00	9,500.00
404	TACK COAT	10.000	CWT	75.00	750.00
406	BITUMINOUS PAVEMENT	250.000	TON	300.00	75,000.00
500	TIERED SCAFFOLDING & STAIR TOWERS	1.000	LS	385,000.00	385,000.00
505	REMOVE BRIDGE DECK	2,700.000	SF	40.00	108,000.00
510	REMOVE RAILING	500.000	LF	72.00	36,000.00
515	REMOVE FLOORBEAMS	14.000	CY	10,225.00	143,150.00
520	PRECAST FLOORBEAMS	14.000	CY	8,800.00	123,200.00
535	REPAIR FLOORBEAMS 1,2,4,5,7,8	183.000	SF	845.00	154,635.00
540	REPAIR COLUMNS	122.000	SF	295.00	35,990.00
545	REPAIR ARCH RIBS	832.000	SF	220.00	183,040.00
550	REPAIR TIE BEAMS	55.000	SF	265.00	14,575.00
555	REPAIR ABUTMENTS	468.000	SF	275.00	128,700.00
560	REPAIR WINGWALLS	590.000	SF	425.00	250,750.00
565	EPOXY CRACK REPAIR	2,066.000	LF	180.00	371,880.00
570	COATING ALL SURFACES - SILANE	9,838.000	LS	2.00	19,676.00
576	PRECAST CONCRETE BRIDGE DECK PANELS (10' L X 6' W)	45.000	EA	10,350.00	465,750.00
580	CONCRETE BRIDGE RAILING	500.000	LF	348.00	174,000.00
585	CORROSION MITIGATION - ANODES	800.000	EA	52.00	41,600.00
590	BRIDGE EXPANSION JOINTS - ASPHALTIC PLUG	80.000	LF	258.00	20,640.00
595	SCOUR REPAIR	2.000	EA	31,000.00	62,000.00
598	FUTURE BRIDGE UTILITY CONDUIT	800.000	LF	37.50	30,000.00
605	CATCH BASIN INLETS	6.000	EA	3,100.00	18,600.00
615	18" REINFORCED CONCRETE PIPE	265.000	LF	75.00	19,875.00
620	PAVEMENT MARKINGS 4"	2,000.000	LF	1.25	2,500.00
625	PAVEMENT MARKING 24" STOP BAR	12.000	LF	50.00	600.00

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
645	SIDEWALK	2,600.000	SF	12.00	31,200.00
650	VALLEY GUTTER	75.000	LF	43.00	3,225.00
655	CURB & GUTTER	350.000	LF	22.00	7,700.00
660	DRIVEWAY APRON	230.000	SF	23.00	5,290.00
665	TOPSOIL	50.000	CY	200.00	10,000.00
670	HYDROSEEDING	2,000.000	SF	1.25	2,500.00
675	SIGNS	3.000	EA	600.00	1,800.00
680	FIELD OFFICE, ENGINEER'S	1.000	LS	20,000.00	20,000.00
685	QC/QA TESTING	1.000	LS	19,000.00	19,000.00
690	EROSION CONTROL	1.000	LS	53,000.00	53,000.00
695	MOBILIZATION	1.000	LS	612,654.00	612,654.00
696	CPM SCHEDULE	6.000	EA	1,200.00	7,200.00
699	TRAFFIC CONTROL - ALL INCLUSIVE	1.000	LS	40,000.00	40,000.00
	Bid Total				**$3,792,200.00**

20 % Contingency = $758,440
Total w/Contingency = $4,550,640

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
205	CLEARING & GRUBBING	1.000	LS	8,500.00	8,500.00
210	REMOVE BITUMINOUS PAVEMENT	10,000.000	SF	4.00	40,000.00
215	REMOVE SIGNS	2.000	EA	110.00	220.00
220	STRUCTURE EXCAVATION	200.000	CY	142.00	28,400.00
225	GRANULAR BORROW FOR STRUCTURES	100.000	CY	108.00	10,800.00
230	REMOVE SIDEWALK	2,600.000	SF	2.00	5,200.00
235	REMOVE CATCH BASIN	5.000	EA	520.00	2,600.00
301	SUBBASE DENSE GRADED CRUSHED STONE	50.000	CY	150.00	7,500.00
401	AGGREGATE SURFACE COURSE	50.000	CY	190.00	9,500.00
404	TACK COAT	10.000	CWT	75.00	750.00
406	BITUMINOUS PAVEMENT	250.000	TON	300.00	75,000.00
500	TIERED SCAFFOLDING & STAIR TOWERS	1.000	LS	385,000.00	385,000.00
505	REMOVE BRIDGE DECK	2,700.000	SF	40.00	108,000.00
510	REMOVE RAILING	500.000	LF	72.00	36,000.00
515	REMOVE COLUMNS	6.000	CY	6,300.00	37,800.00
520	REMOVE FLOORBEAMS	56.000	CY	10,225.00	572,600.00
530	CAST IN PLACE COLUMNS	6.000	CY	7,300.00	43,800.00
535	CAST IN PLACE FLOORBEAMS	56.000	CY	6,361.00	356,216.00
545	REPAIR ARCH RIBS	832.000	SF	220.00	183,040.00
550	REPAIR TIE BEAMS	55.000	SF	265.00	14,575.00
555	REPAIR ABUTMENTS	468.000	SF	275.00	128,700.00
560	REPAIR WINGWALLS	590.000	SF	425.00	250,750.00
562	EPOXY CRACK REPAIR	1,664.000	LF	180.00	299,520.00
565	COATING ALL SURFACES - SILANE	9,838.000	SF	2.00	19,676.00
570	REINFORCING STEEL	45,600.000	LB	3.80	173,280.00
575	CONCRETE BRIDGE DECK	70.000	CY	4,900.00	343,000.00
580	CONCRETE BRIDGE RAILING	500.000	LF	348.00	174,000.00
585	CORROSION MITIGATION - ANODES	200.000	EA	52.00	10,400.00
590	BRIDGE EXPANSION JOINTS - ASPHALTIC PLUG	80.000	LF	258.00	20,640.00
595	SCOUR REPAIR	2.000	EA	31,000.00	62,000.00
598	FUTURE BRIDGE UTILITY CONDUIT	800.000	LF	37.50	30,000.00
605	CATCH BASIN INLETS	6.000	EA	3,100.00	18,600.00
615	18" REINFORCED CONCRETE PIPE	265.000	LF	75.00	19,875.00
620	PAVEMENT MARKINGS 4"	2,000.000	LF	1.25	2,500.00

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
625	PAVEMENT MARKING 24" STOP BAR	12.000	LF	50.00	600.00
645	SIDEWALK	2,600.000	SF	12.00	31,200.00
650	VALLEY GUTTER	75.000	LF	43.00	3,225.00
655	CURB & GUTTER	350.000	LF	22.00	7,700.00
660	DRIVEWAY APRON	230.000	SF	23.00	5,290.00
665	TOPSOIL	50.000	CY	200.00	10,000.00
670	HYDROSEEDING	2,000.000	SF	1.25	2,500.00
675	SIGNS	3.000	EA	600.00	1,800.00
680	FIELD OFFICE, ENGINEER'S	1.000	LS	20,000.00	20,000.00
685	QC/QA TESTING	1.000	LS	19,000.00	19,000.00
690	EROSION CONTROL	1.000	LS	53,000.00	53,000.00
695	MOBILIZATION	1.000	LS	595,043.00	595,043.00
696	CPM SCHEDULE	6.000	EA	1,200.00	7,200.00
699	TRAFFIC CONTROL - ALL INCLUSIVE	1.000	LS	40,000.00	40,000.00
	Bid Total				**$4,275,000.00**

20 % Contingency = $855,000
Total w/Contingency = $5,130,000

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
205	CLEARING & GRUBBING	1.000	LS	8,500.00	8,500.00
210	REMOVE BITUMINOUS PAVEMENT	10,000.000	SF	4.00	40,000.00
215	REMOVE SIGNS	2.000	EA	110.00	220.00
220	STRUCTURE EXCAVATION	200.000	CY	142.00	28,400.00
225	GRANULAR BORROW FOR STRUCTURES	100.000	CY	108.00	10,800.00
230	REMOVE SIDEWALK	2,600.000	SF	2.00	5,200.00
235	REMOVE CATCH BASIN	5.000	EA	520.00	2,600.00
301	SUBBASE DENSE GRADED CRUSHED STONE	50.000	CY	150.00	7,500.00
401	AGGREGATE SURFACE COURSE	50.000	CY	190.00	9,500.00
404	TACK COAT	10.000	CWT	75.00	750.00
406	BITUMINOUS PAVEMENT	250.000	TON	300.00	75,000.00
500	TIERED SCAFFOLDING & STAIR TOWERS	1.000	LS	385,000.00	385,000.00
505	REMOVE BRIDGE DECK	2,700.000	SF	40.00	108,000.00
510	REMOVE RAILING	500.000	LF	72.00	36,000.00
515	REMOVE COLUMNS	6.000	CY	6,300.00	37,800.00
520	REMOVE FLOORBEAMS	56.000	CY	10,225.00	572,600.00
530	CAST IN PLACE COLUMNS	6.000	CY	7,300.00	43,800.00
535	PRECAST FLOORBEAM	56.000	CY	8,800.00	492,800.00
545	REPAIR ARCH RIBS	832.000	SF	220.00	183,040.00
550	REPAIR TIE BEAMS	55.000	SF	265.00	14,575.00
555	REPAIR ABUTMENTS	468.000	SF	275.00	128,700.00
560	REPAIR WINGWALLS	590.000	SF	425.00	250,750.00
562	EPOXY CRACK REPAIR	1,664.000	LF	180.00	299,520.00
565	COATING ALL SURFACES - SILANE	9,838.000	SF	2.00	19,676.00
570	REINFORCING STEEL	12,400.000	LB	3.80	47,120.00
576	PRECAST CONCRETE BRIDGE DECK PANELS (10' L X 6' W)	45.000	EA	10,350.00	465,750.00
580	CONCRETE BRIDGE RAILING	500.000	LF	348.00	174,000.00
585	CORROSION MITIGATION - ANODES	200.000	EA	52.00	10,400.00
590	BRIDGE EXPANSION JOINTS - ASPHALTIC PLUG	80.000	LF	258.00	20,640.00
595	SCOUR REPAIR	2.000	EA	31,000.00	62,000.00
598	FUTURE BRIDGE UTILITY CONDUIT	800.000	LF	37.50	30,000.00
605	CATCH BASIN INLETS	6.000	EA	3,100.00	18,600.00
615	18" REINFORCED CONCRETE PIPE	265.000	LF	75.00	19,875.00
620	PAVEMENT MARKINGS 4"	2,000.000	LF	1.25	2,500.00

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
625	PAVEMENT MARKING 24" STOP BAR	12.000	LF	50.00	600.00
645	SIDEWALK	2,600.000	SF	12.00	31,200.00
650	VALLEY GUTTER	75.000	LF	43.00	3,225.00
655	CURB & GUTTER	350.000	LF	22.00	7,700.00
660	DRIVEWAY APRON	230.000	SF	23.00	5,290.00
665	TOPSOIL	50.000	CY	200.00	10,000.00
670	HYDROSEEDING	2,000.000	SF	1.25	2,500.00
675	SIGNS	3.000	EA	600.00	1,800.00
680	FIELD OFFICE, ENGINEER'S	1.000	LS	20,000.00	20,000.00
685	QC/QA TESTING	1.000	LS	19,000.00	19,000.00
690	EROSION CONTROL	1.000	LS	53,000.00	53,000.00
695	MOBILIZATION	1.000	LS	599,869.00	599,869.00
696	CPM SCHEDULE	6.000	EA	1,200.00	7,200.00
699	TRAFFIC CONTROL - ALL INCLUSIVE	1.000	LS	40,000.00	40,000.00
	Bid Total				**$4,413,000.00**

20 % Contingency = $882,600
Total w/Contingency = $5,295,600

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
205	CLEARING & GRUBBING	1.000	LS	8,500.00	8,500.00
210	REMOVE BITUMINOUS PAVEMENT	10,000.000	SF	4.00	40,000.00
215	REMOVE SIGNS	2.000	EA	110.00	220.00
220	STRUCTURE EXCAVATION	200.000	CY	142.00	28,400.00
225	GRANULAR BORROW FOR STRUCTURES	100.000	CY	108.00	10,800.00
230	REMOVE SIDEWALK	2,600.000	SF	2.00	5,200.00
235	REMOVE CATCH BASIN	5.000	EA	520.00	2,600.00
301	SUBBASE DENSE GRADED CRUSHED STONE	50.000	CY	150.00	7,500.00
401	AGGREGATE SURFACE COURSE	50.000	CY	190.00	9,500.00
404	TACK COAT	10.000	CWT	75.00	750.00
406	BITUMINOUS PAVEMENT	250.000	TON	300.00	75,000.00
500	TIERED SCAFFOLDING & STAIR TOWERS	1.000	LS	385,000.00	385,000.00
505	REMOVE BRIDGE DECK	2,700.000	SF	40.00	108,000.00
510	REMOVE RAILING	500.000	LF	72.00	36,000.00
515	REMOVE COLUMNS	6.000	CY	6,300.00	37,800.00
520	REMOVE FLOORBEAMS	56.000	CY	10,225.00	572,600.00
530	CAST IN PLACE COLUMNS	6.000	CY	7,300.00	43,800.00
535	CAST IN PLACE FLOORBEAMS	160.000	CY	6,742.00	1,078,720.00
545	REPAIR ARCH RIBS	832.000	SF	220.00	183,040.00
550	REPAIR TIE BEAMS	55.000	SF	265.00	14,575.00
555	REPAIR ABUTMENTS	468.000	SF	275.00	128,700.00
560	REPAIR WINGWALLS	590.000	SF	425.00	250,750.00
562	EPOXY CRACK REPAIR	1,664.000	LF	180.00	299,520.00
565	COATING ALL SURFACES - SILANE	9,838.000	SF	2.00	19,676.00
570	REINFORCING STEEL	74,880.000	LB	3.80	284,544.00
575	CONCRETE BRIDGE DECK	82.000	CY	5,260.00	431,320.00
580	CONCRETE BRIDGE RAILING	500.000	LF	348.00	174,000.00
585	CORROSION MITIGATION - ANODES	200.000	EA	52.00	10,400.00
590	BRIDGE EXPANSION JOINTS - ASPHALTIC PLUG	80.000	LF	258.00	20,640.00
595	SCOUR REPAIR	2.000	EA	31,000.00	62,000.00
598	FUTURE BRIDGE UTILITY CONDUIT	800.000	LF	37.50	30,000.00
605	CATCH BASIN INLETS	6.000	EA	3,100.00	18,600.00
615	18" REINFORCED CONCRETE PIPE	265.000	LF	75.00	19,875.00
620	PAVEMENT MARKINGS 4"	2,000.000	LF	1.25	2,500.00

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
625	PAVEMENT MARKING 24" STOP BAR	12.000	LF	50.00	600.00
645	SIDEWALK	2,600.000	SF	12.00	31,200.00
650	VALLEY GUTTER	75.000	LF	43.00	3,225.00
655	CURB & GUTTER	350.000	LF	22.00	7,700.00
660	DRIVEWAY APRON	230.000	SF	23.00	5,290.00
665	TOPSOIL	50.000	CY	200.00	10,000.00
670	HYDROSEEDING	2,000.000	SF	1.25	2,500.00
675	SIGNS	3.000	EA	600.00	1,800.00
680	FIELD OFFICE, ENGINEER'S	1.000	LS	20,000.00	20,000.00
685	QC/QA TESTING	1.000	LS	19,000.00	19,000.00
690	EROSION CONTROL	1.000	LS	53,000.00	53,000.00
695	MOBILIZATION	1.000	LS	596,955.00	596,955.00
696	CPM SCHEDULE	6.000	EA	1,200.00	7,200.00
699	TRAFFIC CONTROL - ALL INCLUSIVE	1.000	LS	40,000.00	40,000.00
	Bid Total				**$5,199,000.00**

20 % Contingency = $1,039,800
Total w/Contingency = $6,238,800

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
205	CLEARING & GRUBBING	1.000	LS	8,500.00	8,500.00
210	REMOVE BITUMINOUS PAVEMENT	10,000.000	SF	4.00	40,000.00
215	REMOVE SIGNS	2.000	EA	110.00	220.00
220	STRUCTURE EXCAVATION	200.000	CY	142.00	28,400.00
225	GRANULAR BORROW FOR STRUCTURES	100.000	CY	108.00	10,800.00
230	REMOVE SIDEWALK	2,600.000	SF	2.00	5,200.00
235	REMOVE CATCH BASIN	5.000	EA	520.00	2,600.00
301	SUBBASE DENSE GRADED CRUSHED STONE	50.000	CY	150.00	7,500.00
401	AGGREGATE SURFACE COURSE	50.000	CY	190.00	9,500.00
404	TACK COAT	10.000	CWT	75.00	750.00
406	BITUMINOUS PAVEMENT	250.000	TON	300.00	75,000.00
500	TIERED SCAFFOLDING & STAIR TOWERS	1.000	LS	385,000.00	385,000.00
505	REMOVE BRIDGE DECK	2,700.000	SF	40.00	108,000.00
510	REMOVE RAILING	500.000	LF	72.00	36,000.00
515	REMOVE COLUMNS	6.000	CY	6,300.00	37,800.00
520	REMOVE FLOORBEAMS	56.000	CY	10,225.00	572,600.00
530	CAST IN PLACE COLUMNS	6.000	CY	7,300.00	43,800.00
535	PRECAST FLOORBEAM	160.000	CY	8,800.00	1,408,000.00
545	REPAIR ARCH RIBS	832.000	SF	220.00	183,040.00
550	REPAIR TIE BEAMS	55.000	SF	265.00	14,575.00
555	REPAIR ABUTMENTS	468.000	SF	275.00	128,700.00
560	REPAIR WINGWALLS	590.000	SF	425.00	250,750.00
562	EPOXY CRACK REPAIR	1,664.000	LF	180.00	299,520.00
565	COATING ALL SURFACES - SILANE	9,838.000	SF	2.00	19,676.00
570	REINFORCING STEEL	12,400.000	LB	3.80	47,120.00
576	PRECAST CONCRETE BRIDGE DECK PANELS (10' L X 6' W)	62.000	EA	10,350.00	641,700.00
580	CONCRETE BRIDGE RAILING	500.000	LF	348.00	174,000.00
585	CORROSION MITIGATION - ANODES	200.000	EA	52.00	10,400.00
590	BRIDGE EXPANSION JOINTS - ASPHALTIC PLUG	80.000	LF	258.00	20,640.00
595	SCOUR REPAIR	2.000	EA	31,000.00	62,000.00
598	FUTURE BRIDGE UTILITY CONDUIT	800.000	LF	37.50	30,000.00
605	CATCH BASIN INLETS	6.000	EA	3,100.00	18,600.00
615	18" REINFORCED CONCRETE PIPE	265.000	LF	75.00	19,875.00
620	PAVEMENT MARKINGS 4"	2,000.000	LF	1.25	2,500.00

BID PROPOSAL

Biditem	Description	Quantity	Units	Unit Price	Bid Total
625	PAVEMENT MARKING 24" STOP BAR	12.000	LF	50.00	600.00
645	SIDEWALK	2,600.000	SF	12.00	31,200.00
650	VALLEY GUTTER	75.000	LF	43.00	3,225.00
655	CURB & GUTTER	350.000	LF	22.00	7,700.00
660	DRIVEWAY APRON	230.000	SF	23.00	5,290.00
665	TOPSOIL	50.000	CY	200.00	10,000.00
670	HYDROSEEDING	2,000.000	SF	1.25	2,500.00
675	SIGNS	3.000	EA	600.00	1,800.00
680	FIELD OFFICE, ENGINEER'S	1.000	LS	20,000.00	20,000.00
685	QC/QA TESTING	1.000	LS	19,000.00	19,000.00
690	EROSION CONTROL	1.000	LS	53,000.00	53,000.00
695	MOBILIZATION	1.000	LS	622,719.00	622,719.00
696	CPM SCHEDULE	6.000	EA	1,200.00	7,200.00
699	TRAFFIC CONTROL - ALL INCLUSIVE	1.000	LS	40,000.00	40,000.00
	Bid Total				**$5,527,000.00**

20 % Contingency = $1,105,400
Total w/Contingency = $6,632,400

Appendix T: Basis of Schedule

Basis of Schedule

Movement of the electrical lines is key to starting the project on time. Cranes cannot operate within a minimum of 10' of overhead power lines and possibly a greater distance depending on the size of the line. In all alternatives, the assumed construction start date is April 1, 2027. Below is an order of magnitude durations for each alternative. Please note that each alternative has been rounded to the nearest half month:

Alternate	Months
1	6.5
1 ABC	5.5
2	7.5
2 ABC	5.5
2w	8
2w ABC	6.5

There are two major operations in the Work Breakdown Structure for all alternatives. Below are assumptions for each operation:

1. Deck & Floor Beam Replacement/Repairs
    - Two carpenter crews are set on each abutment. One crew works on floor beams 1-2 while the other works on floor beams 5-8.
    - Precast floor beams, precast deck panels and cast in place concrete railings and columns are constructed in the ABC option.
    - Deck and floor beam operations start after the tiered scaffold is installed. The tiered scaffold will provide 100% coverage over Mill Brook and be accessible via ladders and stair towers at each abutment. Both carpenter crews will assemble the scaffold & stair towers.
2. Substructure Repairs
    - Work starts after the scaffolding system is installed and the bridge deck and railings are removed for worker safety.
    - Critical path is through repairs, silane treatment and remove scaffold.
    - 6 person Laborer Crew 1 will work off swing stage scaffold and focus on the wingwalls.

All concrete repairs areas are scheduled based on a maximum depth of 4".

Alternate 1 is built using conventional construction methods. The road is closed to traffic. Utilities are assumed to be moved before construction. Tiered scaffolding is installed from the arches. At each approach, a crane will be staged for bridge demolition as well as swinging scaffolding, falsework, forms and materials to workers. After railings and deck are removed, they will be processed into rubble and hauled to a legal dumpsite. Crews will then work on floor beams 1-4 and 5-8 from each abutment respectively.

In alternative 1, only floor beams 3 and 6 are replaced. Soffit falsework is planned for all floor beam removal, replacement and repairs. Falsework is expected to be required for both floor beam construction and deck construction. The design of this falsework will vary on each contractors means and methods. After the deck is poured, decorative concrete railings will be cast.

Substructure repairs can begin once the removals are complete and falling debris has been cleared. Typical crews will demo with a small chipping hammer to sound concrete, install anodes and place patches. After patching, these crews will place a skim coat on the entire surface area to provide a uniform appearance. Silane treatment will follow 14 days after placement of the skim coat.

Alternate 1 Accelerated Bridge Construction (ABC) includes the same scope of work as Alternate 1 but utilizes precast floor beams 3 and 6 and precast deck panels.

Alternate 2 replaces all floor beams and columns using conventional cast in place construction.

Alternate 2 ABC contains the same scope of work as Alternate 2 but utilizes cast in place columns, precast floor beams, and precast deck panels.

Alternate 2w extends the floor beams and widens the bridge deck to one side. Floor beams would need to be added on the approaches, or the retaining wall would need to be reconstructed to create the widening. Final details of widening construction could impact the schedule.

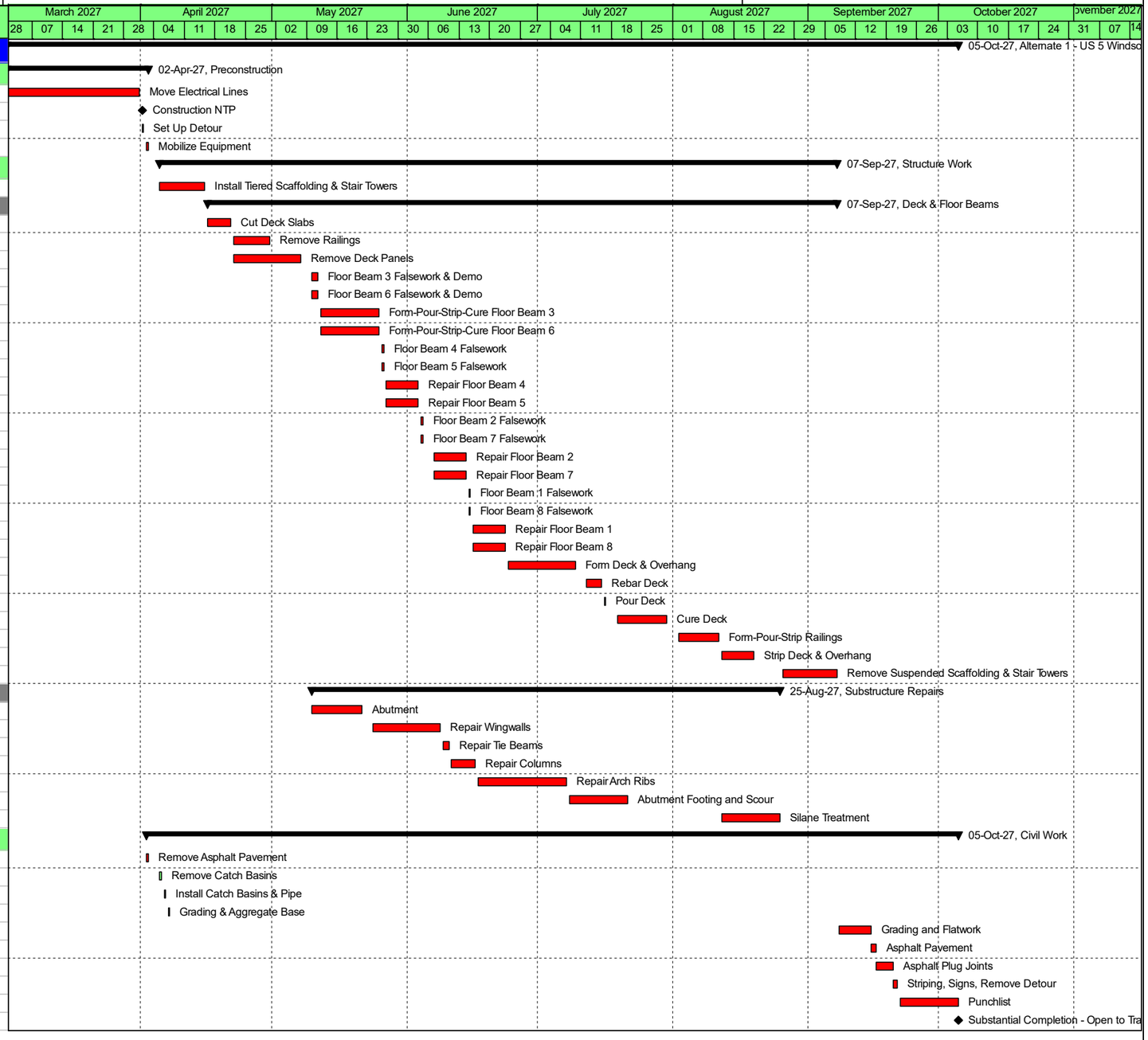
Alternate 2w ABC utilizes precast floor beams lengthened to accommodate a wider travel surface as well as precast deck panels.

Alternate 1 - US 5 Windsor Arch Bridge Repairs

Exhibit #1 - CTD Schedule

Data Date: 01-Mar-27
Printed on: 09-Sep-25 @ 14:05

Activity ID	Activity Name	Original Duration	Start	Finish	Total Float	Notes
Alternate 1 - US 5 Windsor Arch Bridge Repairs		157	01-Mar-27	05-Oct-27	-17	
Preconstruction		25	01-Mar-27	02-Apr-27	-17	
PC100	Move Electrical Lines	23	01-Mar-27	31-Mar-27	-17	
PC110	Construction NTP	0	01-Apr-27		-17	
PC120	Set Up Detour	1	01-Apr-27	01-Apr-27	-17	
PC130	Mobilize Equipment	1	02-Apr-27	02-Apr-27	-17	
Structure Work		112	05-Apr-27	07-Sep-27	-17	
S1000	Install Tiered Scaffolding & Stair Towers	9	05-Apr-27	15-Apr-27	-17	
Deck & Floor Beams		103	16-Apr-27	07-Sep-27	-17	
DF100	Cut Deck Slabs	4	16-Apr-27	21-Apr-27	-17	
DF110	Remove Railings	7	22-Apr-27	30-Apr-27	-12	
DF120	Remove Deck Panels	12	22-Apr-27	07-May-27	-17	
DF130	Floor Beam 3 Falsework & Demo	2	10-May-27	11-May-27	-17	
DF210	Floor Beam 6 Falsework & Demo	2	10-May-27	11-May-27	-17	
DF140	Form-Pour-Strip-Cure Floor Beam 3	10	12-May-27	25-May-27	-17	
DF220	Form-Pour-Strip-Cure Floor Beam 6	10	12-May-27	25-May-27	-17	
DF150	Floor Beam 4 Falsework	1	26-May-27	26-May-27	-17	
DF230	Floor Beam 5 Falsework	1	26-May-27	26-May-27	-17	
DF160	Repair Floor Beam 4	6	27-May-27	03-Jun-27	-17	
DF240	Repair Floor Beam 5	6	27-May-27	03-Jun-27	-17	
DF170	Floor Beam 2 Falsework	1	04-Jun-27	04-Jun-27	-17	
DF250	Floor Beam 7 Falsework	1	04-Jun-27	04-Jun-27	-17	
DF180	Repair Floor Beam 2	6	07-Jun-27	14-Jun-27	-17	
DF260	Repair Floor Beam 7	6	07-Jun-27	14-Jun-27	-17	
DF190	Floor Beam 1 Falsework	1	15-Jun-27	15-Jun-27	-17	
DF270	Floor Beam 8 Falsework	1	15-Jun-27	15-Jun-27	-17	
DF200	Repair Floor Beam 1	6	16-Jun-27	23-Jun-27	-17	
DF280	Repair Floor Beam 8	6	16-Jun-27	23-Jun-27	-17	
DF290	Form Deck & Overhang	12	24-Jun-27	09-Jul-27	-17	
DF300	Rebar Deck	4	12-Jul-27	15-Jul-27	-17	
DF310	Pour Deck	1	16-Jul-27	16-Jul-27	-17	
DF320	Cure Deck	10	19-Jul-27	30-Jul-27	-17	
DF330	Form-Pour-Strip Railings	8	02-Aug-27	11-Aug-27	-17	
DF340	Strip Deck & Overhang	6	12-Aug-27	19-Aug-27	-13	
DF350	Remove Suspended Scaffolding & Stair Towers	9	26-Aug-27	07-Sep-27	-17	
Substructure Repairs		78	10-May-27	25-Aug-27	-17	
SR130	Abutment	10	10-May-27	21-May-27	-6	
SR140	Repair Wingwalls	12	24-May-27	08-Jun-27	-6	
SR120	Repair Tie Beams	2	09-Jun-27	10-Jun-27	-6	
SR100	Repair Columns	4	11-Jun-27	16-Jun-27	-6	
SR110	Repair Arch Ribs	15	17-Jun-27	07-Jul-27	-6	
SR160	Abutment Footing and Scour	10	08-Jul-27	21-Jul-27	-2	
SR150	Silane Treatment	10	12-Aug-27	25-Aug-27	-17	
Civil Work		133	02-Apr-27	05-Oct-27	-17	
CIV10	Remove Asphalt Pavement	1	02-Apr-27	02-Apr-27	-8	
CIV20	Remove Catch Basins	1	05-Apr-27	05-Apr-27	98	
CIV30	Install Catch Basins & Pipe	1	06-Apr-27	06-Apr-27	98	
CIV40	Grading & Aggregate Base	1	07-Apr-27	07-Apr-27	98	
CIV90	Grading and Flatwork	6	08-Sep-27	15-Sep-27	-17	
CIV50	Asphalt Pavement	1	15-Sep-27	16-Sep-27	-17	
CIV60	Asphalt Plug Joints	2	16-Sep-27	20-Sep-27	-17	
CIV70	Striping, Signs, Remove Detour	1	20-Sep-27	21-Sep-27	-17	
CIV100	Punchlist	10	22-Sep-27	05-Oct-27	-17	
CIV80	Substantial Completion - Open to Traffic	0		05-Oct-27	-17	



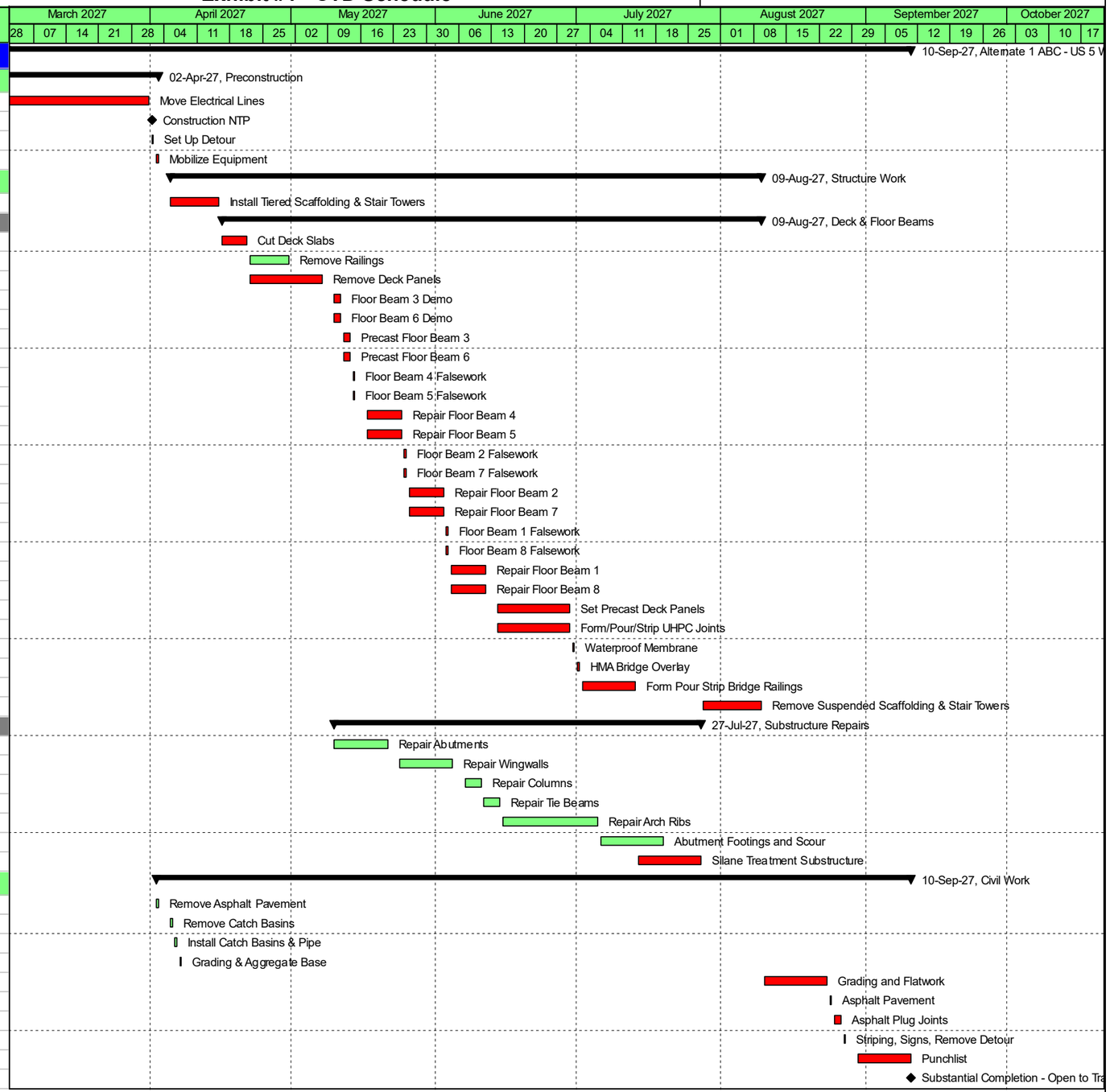
Remaining Level of Effort | Actual Work | Critical Remaining Work | Summary
Actual Level of Effort | Remaining Work | Milestone | WBS Summary Activity

Alternate 1 ABC - US 5 Windsor Arch Bridge Repairs

Exhibit #1 - CTD Schedule

Data Date: 01-Mar-27
Printed on: 09-Sep-25 @ 14:01

Activity ID	Activity Name	Original Duration	Start	Finish	Total Float	Notes
Alternate 1 ABC - US 5 Windsor Arch Bridge Re		140	01-Mar-27	10-Sep-27	0	
Preconstruction		25	01-Mar-27	02-Apr-27	0	
PC100	Move Electrical Lines	23	01-Mar-27	31-Mar-27	0	
PC110	Construction NTP	0	01-Apr-27		0	
PC120	Set Up Detour	1	01-Apr-27	01-Apr-27	0	
PC130	Mobilize Equipment	1	02-Apr-27	02-Apr-27	0	
Structure Work		91	05-Apr-27	09-Aug-27	24	
S1000	Install Tiered Scaffolding & Stair Towers	9	05-Apr-27	15-Apr-27	0	
Deck & Floor Beams		82	16-Apr-27	09-Aug-27	0	
DF100	Cut Deck Slabs	4	16-Apr-27	21-Apr-27	0	
DF110	Remove Railings	7	22-Apr-27	30-Apr-27	5	
DF120	Remove Deck Panels	12	22-Apr-27	07-May-27	0	
DF130	Floor Beam 3 Demo	2	10-May-27	11-May-27	0	
DF210	Floor Beam 6 Demo	2	10-May-27	11-May-27	0	
DF140	Precast Floor Beam 3	2	12-May-27	13-May-27	0	
DF220	Precast Floor Beam 6	2	12-May-27	13-May-27	0	
DF150	Floor Beam 4 Falsework	1	14-May-27	14-May-27	0	
DF230	Floor Beam 5 Falsework	1	14-May-27	14-May-27	0	
DF160	Repair Floor Beam 4	6	17-May-27	24-May-27	0	
DF240	Repair Floor Beam 5	6	17-May-27	24-May-27	0	
DF170	Floor Beam 2 Falsework	1	25-May-27	25-May-27	0	
DF250	Floor Beam 7 Falsework	1	25-May-27	25-May-27	0	
DF180	Repair Floor Beam 2	6	26-May-27	02-Jun-27	0	
DF260	Repair Floor Beam 7	6	26-May-27	02-Jun-27	0	
DF190	Floor Beam 1 Falsework	1	03-Jun-27	03-Jun-27	0	
DF270	Floor Beam 8 Falsework	1	03-Jun-27	03-Jun-27	0	
DF200	Repair Floor Beam 1	6	04-Jun-27	11-Jun-27	0	
DF280	Repair Floor Beam 8	6	04-Jun-27	11-Jun-27	0	
DF290	Set Precast Deck Panels	12	14-Jun-27	29-Jun-27	0	
DF300	Form/Pour/Strip UHPC Joints	12	14-Jun-27	29-Jun-27	0	
DF310	Waterproof Membrane	1	30-Jun-27	30-Jun-27	0	
DF320	HMA Bridge Overlay	1	01-Jul-27	01-Jul-27	0	
DF330	Form Pour Strip Bridge Railings	8	02-Jul-27	13-Jul-27	0	
DF350	Remove Suspended Scaffolding & Stair Towers	9	28-Jul-27	09-Aug-27	0	
Substructure Repairs		57	10-May-27	27-Jul-27	33	
SR130	Repair Abutments	10	10-May-27	21-May-27	6	
SR140	Repair Wingwalls	10	24-May-27	04-Jun-27	6	
SR100	Repair Columns	4	07-Jun-27	10-Jun-27	6	
SR120	Repair Tie Beams	2	11-Jun-27	14-Jun-27	6	
SR110	Repair Arch Ribs	15	15-Jun-27	05-Jul-27	6	
SR170	Abutment Footings and Scour	10	06-Jul-27	19-Jul-27	39	
SR150	Silane Treatment Substructure	10	14-Jul-27	27-Jul-27	0	
Civil Work		116	02-Apr-27	10-Sep-27	0	
CIV10	Remove Asphalt Pavement	1	02-Apr-27	02-Apr-27	9	
CIV20	Remove Catch Basins	1	05-Apr-27	05-Apr-27	98	
CIV30	Install Catch Basins & Pipe	1	06-Apr-27	06-Apr-27	98	
CIV40	Grading & Aggregate Base	1	07-Apr-27	07-Apr-27	98	
CIV90	Grading and Flatwork	10	10-Aug-27	23-Aug-27	0	
CIV50	Asphalt Pavement	1	24-Aug-27	24-Aug-27	0	
CIV60	Asphalt Plug Joints	2	25-Aug-27	26-Aug-27	0	
CIV70	Striping, Signs, Remove Detour	1	27-Aug-27	27-Aug-27	0	
CIV100	Punchlist	10	30-Aug-27	10-Sep-27	0	
CIV80	Substantial Completion - Open to Traffic	0		10-Sep-27	0	



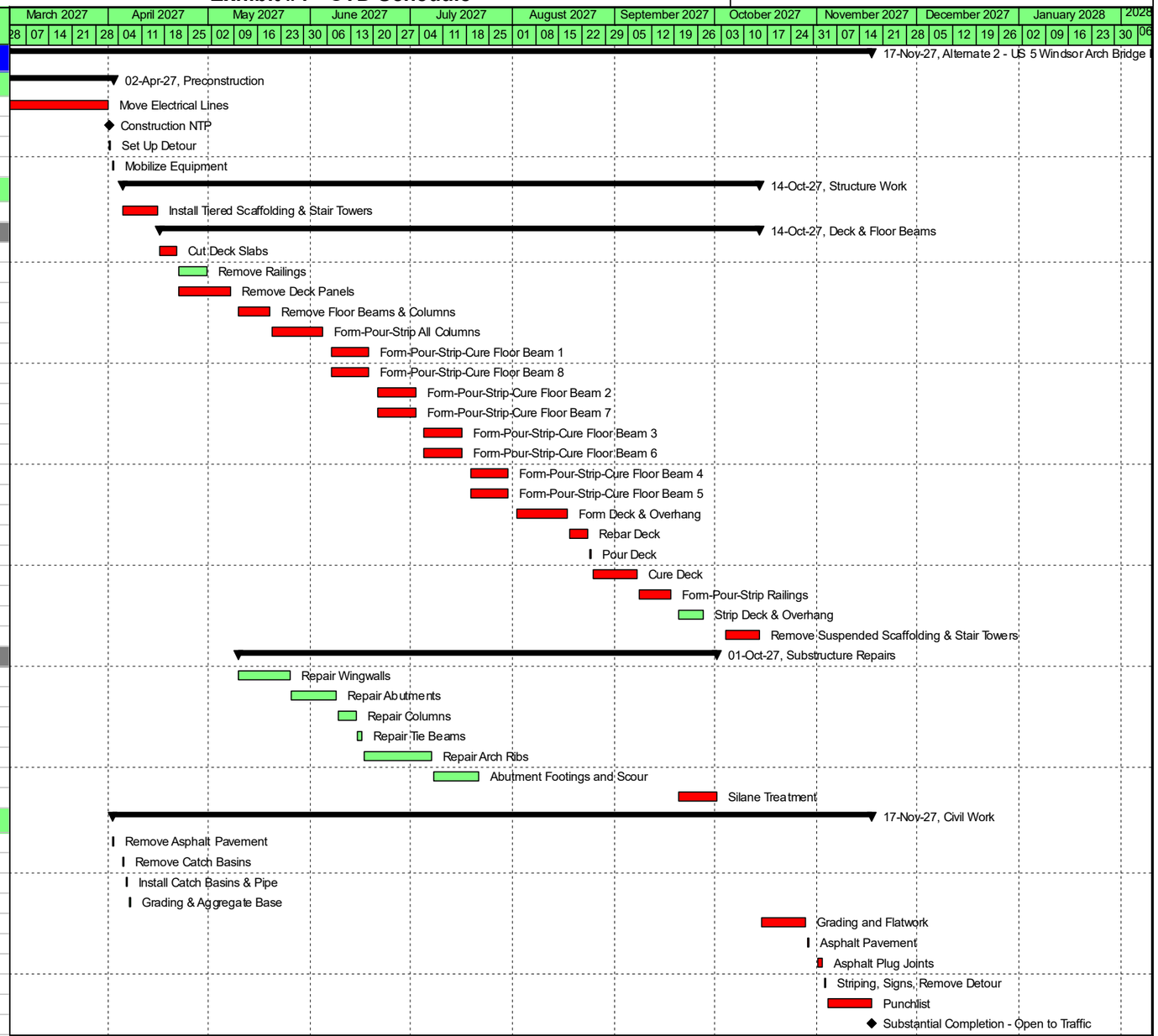
Remaining Level of Effort | Actual Work | Critical Remaining Work | Summary
Actual Level of Effort | Remaining Work | Milestone | WBS Summary Activity

Alternate 2 - US 5 Windsor Arch Bridge Repairs

Exhibit #1 - CTD Schedule

Data Date: 01-Mar-27
Printed on: 09-Sep-25 @ 12:24

Activity ID	Activity Name	Original Duration	Start	Finish	Total Float	Notes
Alternate 2 - US 5 Windsor Arch Bridge Repairs		188	01-Mar-27	17-Nov-27	0	
Preconstruction		25	01-Mar-27	02-Apr-27	0	
PC100	Move Electrical Lines	23	01-Mar-27	31-Mar-27	0	
PC110	Construction NTP	0	01-Apr-27		0	
PC120	Set Up Detour	1	01-Apr-27	01-Apr-27	0	
PC130	Mobilize Equipment	1	02-Apr-27	02-Apr-27	0	
Structure Work		139	05-Apr-27	14-Oct-27	0	
S1000	Install Tiered Scaffolding & Stair Towers	9	05-Apr-27	15-Apr-27	0	
Deck & Floor Beams		130	16-Apr-27	14-Oct-27	0	
DF100	Cut Deck Slabs	4	16-Apr-27	21-Apr-27	0	
DF110	Remove Railings	7	22-Apr-27	30-Apr-27	5	
DF120	Remove Deck Panels	12	22-Apr-27	07-May-27	0	
DF125	Remove Floor Beams & Columns	8	10-May-27	19-May-27	0	
DF360	Form-Pour-Strip All Columns	12	20-May-27	04-Jun-27	0	
DF140	Form-Pour-Strip-Cure Floor Beam 1	10	07-Jun-27	18-Jun-27	0	
DF220	Form-Pour-Strip-Cure Floor Beam 8	10	07-Jun-27	18-Jun-27	0	
DF160	Form-Pour-Strip-Cure Floor Beam 2	10	21-Jun-27	02-Jul-27	0	
DF240	Form-Pour-Strip-Cure Floor Beam 7	10	21-Jun-27	02-Jul-27	0	
DF180	Form-Pour-Strip-Cure Floor Beam 3	10	05-Jul-27	16-Jul-27	0	
DF260	Form-Pour-Strip-Cure Floor Beam 6	10	05-Jul-27	16-Jul-27	0	
DF200	Form-Pour-Strip-Cure Floor Beam 4	10	19-Jul-27	30-Jul-27	0	
DF280	Form-Pour-Strip-Cure Floor Beam 5	10	19-Jul-27	30-Jul-27	0	
DF290	Form Deck & Overhang	12	02-Aug-27	17-Aug-27	0	
DF300	Rebar Deck	4	18-Aug-27	23-Aug-27	0	
DF310	Pour Deck	1	24-Aug-27	24-Aug-27	0	
DF320	Cure Deck	10	25-Aug-27	07-Sep-27	0	
DF330	Form-Pour-Strip Railings	8	08-Sep-27	17-Sep-27	0	
DF340	Strip Deck & Overhang	6	20-Sep-27	27-Sep-27	4	
DF350	Remove Suspended Scaffolding & Stair Towers	9	04-Oct-27	14-Oct-27	0	
Substructure Repairs		105	10-May-27	01-Oct-27	0	
SR140	Repair Wingwalls	12	10-May-27	25-May-27	42	
SR130	Repair Abutments	10	26-May-27	08-Jun-27	42	
SR100	Repair Columns	4	09-Jun-27	14-Jun-27	42	
SR120	Repair Tie Beams	2	15-Jun-27	16-Jun-27	42	
SR110	Repair Arch Ribs	15	17-Jun-27	07-Jul-27	42	
SR160	Abutment Footings and Scour	10	08-Jul-27	21-Jul-27	42	
SR150	Silane Treatment	10	20-Sep-27	01-Oct-27	0	
Civil Work		164	02-Apr-27	17-Nov-27	0	
CIV10	Remove Asphalt Pavement	1	02-Apr-27	02-Apr-27	9	
CIV20	Remove Catch Basins	1	05-Apr-27	05-Apr-27	146	
CIV30	Install Catch Basins & Pipe	1	06-Apr-27	06-Apr-27	146	
CIV40	Grading & Aggregate Base	1	07-Apr-27	07-Apr-27	146	
CIV90	Grading and Flatwork	10	15-Oct-27	28-Oct-27	0	
CIV50	Asphalt Pavement	1	29-Oct-27	29-Oct-27	0	
CIV60	Asphalt Plug Joints	2	01-Nov-27	02-Nov-27	0	
CIV70	Striping, Signs, Remove Detour	1	03-Nov-27	03-Nov-27	0	
CIV100	Punchlist	10	04-Nov-27	17-Nov-27	0	
CIV80	Substantial Completion - Open to Traffic	0		17-Nov-27	0	



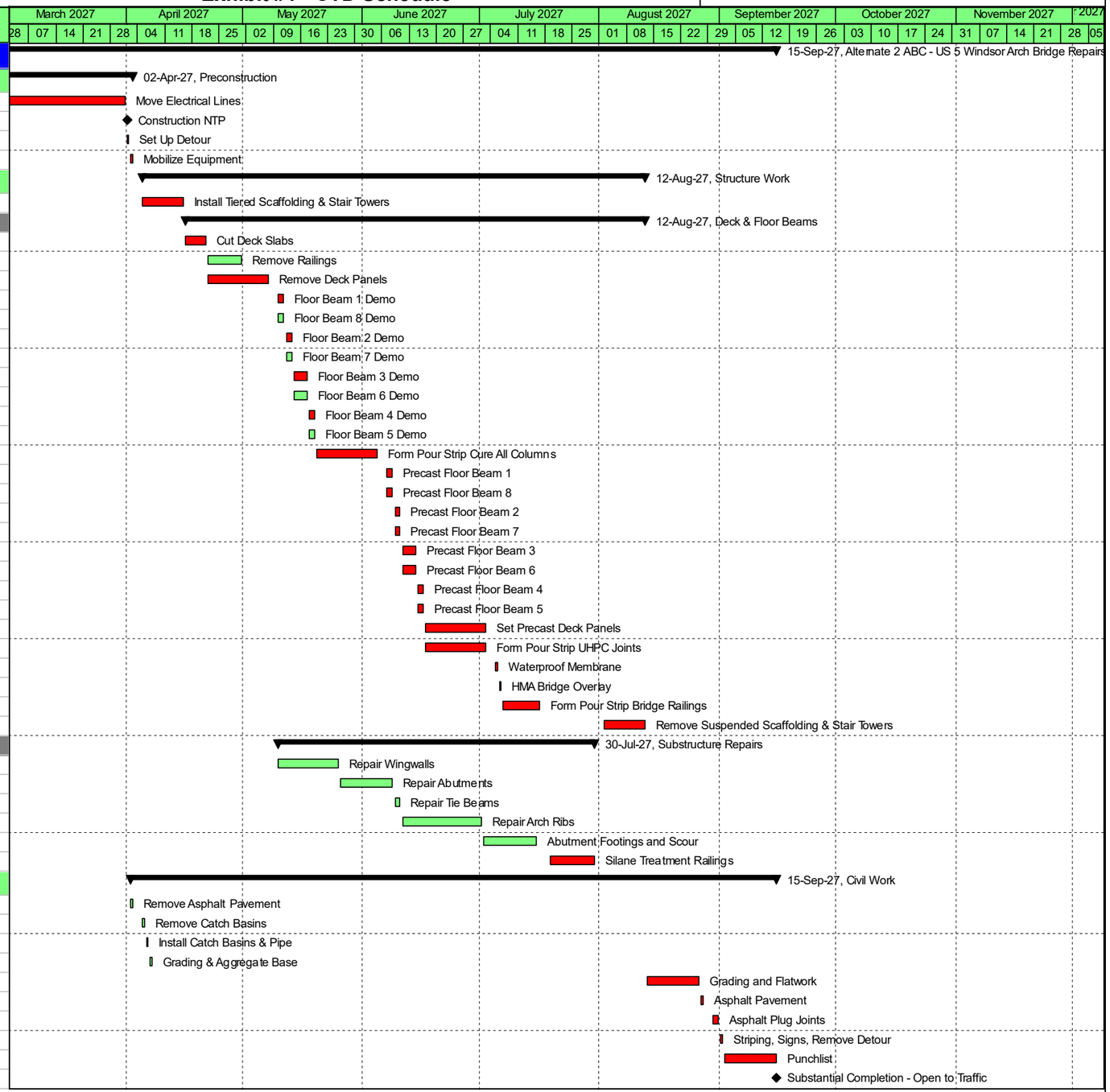
Remaining Level of Effort | Actual Work | Critical Remaining Work | Summary
Actual Level of Effort | Remaining Work | Milestone | WBS Summary Activity

Alternate 2 ABC - US 5 Windsor Arch Bridge Repairs

Exhibit #1 - CTD Schedule

Data Date: 01-Mar-27
Printed on: 09-Sep-25 @ 12:21

Activity ID	Activity Name	Original Duration	Start	Finish	Total Float	Notes
Alternate 2 ABC - US 5 Windsor Arch Bridge Re		143	01-Mar-27	15-Sep-27	0	
Preconstruction		25	01-Mar-27	02-Apr-27	0	
PC100	Move Electrical Lines	23	01-Mar-27	31-Mar-27	0	
PC110	Construction NTP	0	01-Apr-27		0	
PC120	Set Up Detour	1	01-Apr-27	01-Apr-27	0	
PC130	Mobilize Equipment	1	02-Apr-27	02-Apr-27	0	
Structure Work		94	05-Apr-27	12-Aug-27	0	
S1000	Install Tiered Scaffolding & Stair Towers	9	05-Apr-27	15-Apr-27	0	
Deck & Floor Beams		85	16-Apr-27	12-Aug-27	0	
DF100	Cut Deck Slabs	4	16-Apr-27	21-Apr-27	0	
DF110	Remove Railings	7	22-Apr-27	30-Apr-27	5	
DF120	Remove Deck Panels	12	22-Apr-27	07-May-27	0	
DF190	Floor Beam 1 Demo	2	10-May-27	11-May-27	0	
DF270	Floor Beam 8 Demo	2	10-May-27	11-May-27	18	
DF170	Floor Beam 2 Demo	2	12-May-27	13-May-27	0	
DF250	Floor Beam 7 Demo	2	12-May-27	13-May-27	18	
DF130	Floor Beam 3 Demo	2	14-May-27	17-May-27	0	
DF210	Floor Beam 6 Demo	2	14-May-27	17-May-27	18	
DF150	Floor Beam 4 Demo	2	18-May-27	19-May-27	0	
DF230	Floor Beam 5 Demo	2	18-May-27	19-May-27	18	
FB285	Form Pour Strip Cure All Columns	12	20-May-27	04-Jun-27	0	
DF200	Precast Floor Beam 1	2	07-Jun-27	08-Jun-27	0	
DF280	Precast Floor Beam 8	2	07-Jun-27	08-Jun-27	0	
DF180	Precast Floor Beam 2	2	09-Jun-27	10-Jun-27	0	
DF260	Precast Floor Beam 7	2	09-Jun-27	10-Jun-27	0	
DF140	Precast Floor Beam 3	2	11-Jun-27	14-Jun-27	0	
DF220	Precast Floor Beam 6	2	11-Jun-27	14-Jun-27	0	
DF160	Precast Floor Beam 4	2	15-Jun-27	16-Jun-27	0	
DF240	Precast Floor Beam 5	2	15-Jun-27	16-Jun-27	0	
DF290	Set Precast Deck Panels	12	17-Jun-27	02-Jul-27	0	
DF300	Form Pour Strip UHPC Joints	12	17-Jun-27	02-Jul-27	0	
DF310	Waterproof Membrane	1	05-Jul-27	05-Jul-27	0	
DF320	HMA Bridge Overlay	1	06-Jul-27	06-Jul-27	0	
DF330	Form Pour Strip Bridge Railings	8	07-Jul-27	16-Jul-27	0	
DF350	Remove Suspended Scaffolding & Stair Towers	9	02-Aug-27	12-Aug-27	0	
Substructure Repairs		60	10-May-27	30-Jul-27	0	
SR140	Repair Wingwalls	12	10-May-27	25-May-27	11	
SR130	Repair Abutments	10	26-May-27	08-Jun-27	11	
SR120	Repair Tie Beams	2	09-Jun-27	10-Jun-27	11	
SR110	Repair Arch Ribs	15	11-Jun-27	01-Jul-27	11	
SR160	Abutment Footings and Scour	10	02-Jul-27	15-Jul-27	11	
SR150	Silane Treatment Railings	10	19-Jul-27	30-Jul-27	0	
Civil Work		119	02-Apr-27	15-Sep-27	0	
CIV10	Remove Asphalt Pavement	1	02-Apr-27	02-Apr-27	9	
CIV20	Remove Catch Basins	1	05-Apr-27	05-Apr-27	101	
CIV30	Install Catch Basins & Pipe	1	06-Apr-27	06-Apr-27	101	
CIV40	Grading & Aggregate Base	1	07-Apr-27	07-Apr-27	101	
CIV90	Grading and Flatwork	10	13-Aug-27	26-Aug-27	0	
CIV50	Asphalt Pavement	1	27-Aug-27	27-Aug-27	0	
CIV60	Asphalt Plug Joints	2	30-Aug-27	31-Aug-27	0	
CIV70	Striping, Signs, Remove Detour	1	01-Sep-27	01-Sep-27	0	
CIV100	Punchlist	10	02-Sep-27	15-Sep-27	0	
CIV80	Substantial Completion - Open to Traffic	0		15-Sep-27	0	



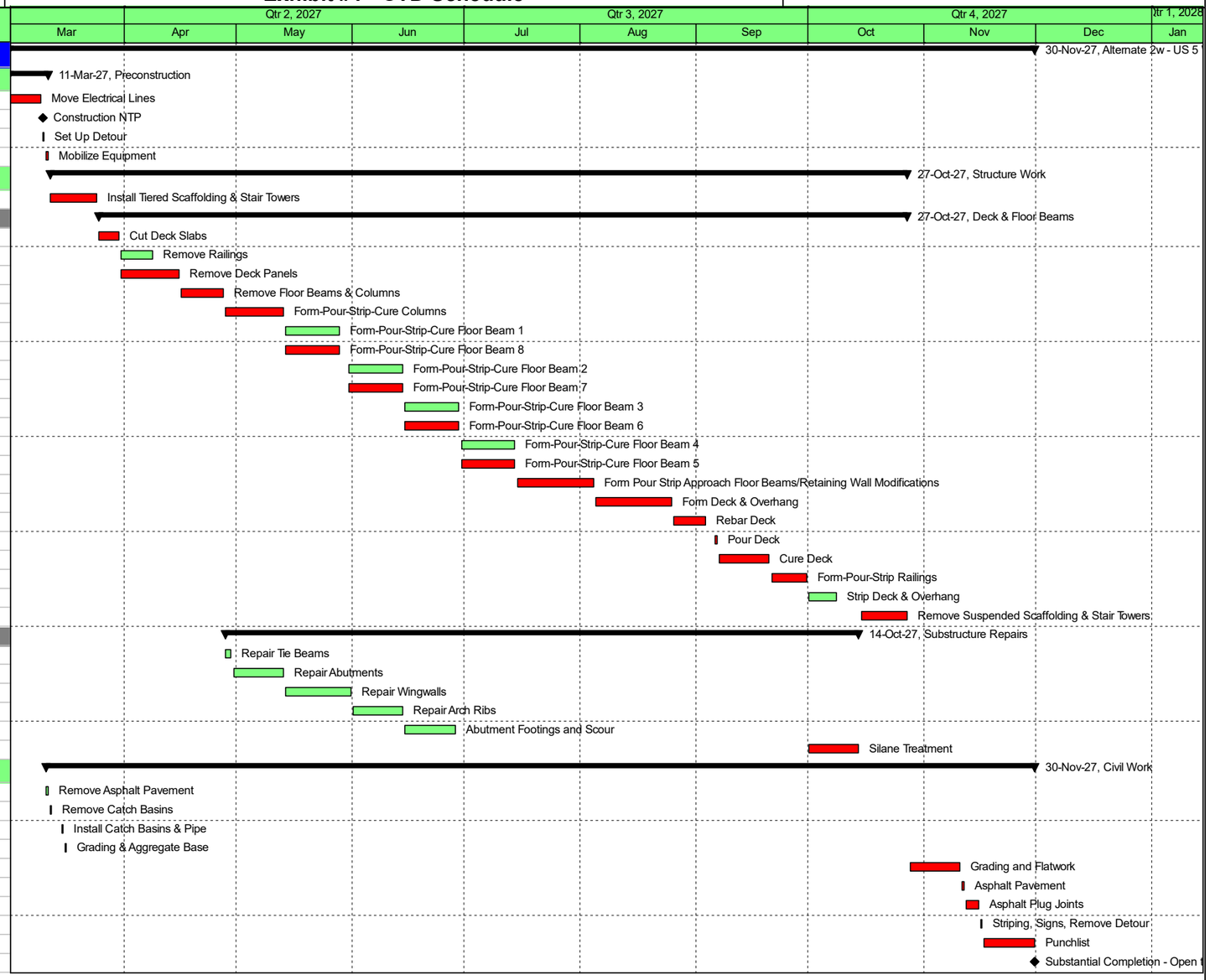
Remaining Level of Effort | Actual Work | Critical Remaining Work | Summary
Actual Level of Effort | Remaining Work | Milestone | WBS Summary Activity

Alternate 2w - US 5 Windsor Arch Bridge Repairs

Exhibit #1 - CTD Schedule

Data Date: 01-Mar-27
Printed on: 09-Sep-25 @ 14:37

Activity ID	Activity Name	Original Duration	Start	Finish	Total Float	Notes
Alternate 2w - US 5 Windsor Arch Bridge Repair		197	01-Mar-27	30-Nov-27	0	
Preconstruction		9	01-Mar-27	11-Mar-27	0	
PC100	Move Electrical Lines	7	01-Mar-27	09-Mar-27	0	
PC110	Construction NTP	0	10-Mar-27		0	
PC120	Set Up Detour	1	10-Mar-27	10-Mar-27	0	
PC130	Mobilize Equipment	1	11-Mar-27	11-Mar-27	0	
Structure Work		164	12-Mar-27	27-Oct-27	0	
S1000	Install Tiered Scaffolding & Stair Towers	9	12-Mar-27	24-Mar-27	0	
Deck & Floor Beams		155	25-Mar-27	27-Oct-27	0	
DF100	Cut Deck Slabs	4	25-Mar-27	30-Mar-27	0	
DF110	Remove Railings	7	31-Mar-27	08-Apr-27	5	
DF120	Remove Deck Panels	12	31-Mar-27	15-Apr-27	0	
DF125	Remove Floor Beams & Columns	8	16-Apr-27	27-Apr-27	0	
DF370	Form-Pour-Strip-Cure Columns	12	28-Apr-27	13-May-27	0	
DF140	Form-Pour-Strip-Cure Floor Beam 1	11	14-May-27	28-May-27	15	
DF220	Form-Pour-Strip-Cure Floor Beam 8	11	14-May-27	28-May-27	0	
DF160	Form-Pour-Strip-Cure Floor Beam 2	11	31-May-27	14-Jun-27	15	
DF240	Form-Pour-Strip-Cure Floor Beam 7	11	31-May-27	14-Jun-27	0	
DF180	Form-Pour-Strip-Cure Floor Beam 3	11	15-Jun-27	29-Jun-27	15	
DF260	Form-Pour-Strip-Cure Floor Beam 6	11	15-Jun-27	29-Jun-27	0	
DF200	Form-Pour-Strip-Cure Floor Beam 4	11	30-Jun-27	14-Jul-27	15	
DF280	Form-Pour-Strip-Cure Floor Beam 5	11	30-Jun-27	14-Jul-27	0	
DF360	Form Pour Strip Approach Floor Beams/Retaining Wal	15	15-Jul-27	04-Aug-27	0	
DF290	Form Deck & Overhang	15	05-Aug-27	25-Aug-27	0	
DF300	Rebar Deck	7	26-Aug-27	03-Sep-27	0	
DF310	Pour Deck	1	06-Sep-27	06-Sep-27	0	
DF320	Cure Deck	10	07-Sep-27	20-Sep-27	0	
DF330	Form-Pour-Strip Railings	8	21-Sep-27	30-Sep-27	0	
DF340	Strip Deck & Overhang	6	01-Oct-27	08-Oct-27	4	
DF350	Remove Suspended Scaffolding & Stair Towers	9	15-Oct-27	27-Oct-27	0	
Substructure Repairs		122	28-Apr-27	14-Oct-27	0	
SR120	Repair Tie Beams	2	28-Apr-27	29-Apr-27	78	
SR130	Repair Abutments	10	30-Apr-27	13-May-27	78	
SR140	Repair Wingwalls	12	14-May-27	31-May-27	78	
SR110	Repair Arch Ribs	10	01-Jun-27	14-Jun-27	78	
SR160	Abutment Footings and Scour	10	15-Jun-27	28-Jun-27	78	
SR150	Silane Treatment	10	01-Oct-27	14-Oct-27	0	
Civil Work		189	11-Mar-27	30-Nov-27	0	
CIV10	Remove Asphalt Pavement	1	11-Mar-27	11-Mar-27	9	
CIV20	Remove Catch Basins	1	12-Mar-27	12-Mar-27	171	
CIV30	Install Catch Basins & Pipe	1	15-Mar-27	15-Mar-27	171	
CIV40	Grading & Aggregate Base	1	16-Mar-27	16-Mar-27	171	
CIV90	Grading and Flatwork	10	28-Oct-27	10-Nov-27	0	
CIV50	Asphalt Pavement	1	11-Nov-27	11-Nov-27	0	
CIV60	Asphalt Plug Joints	2	12-Nov-27	15-Nov-27	0	
CIV70	Striping, Signs, Remove Detour	1	16-Nov-27	16-Nov-27	0	
CIV100	Punchlist	10	17-Nov-27	30-Nov-27	0	
CIV80	Substantial Completion - Open to Traffic	0		30-Nov-27	0	



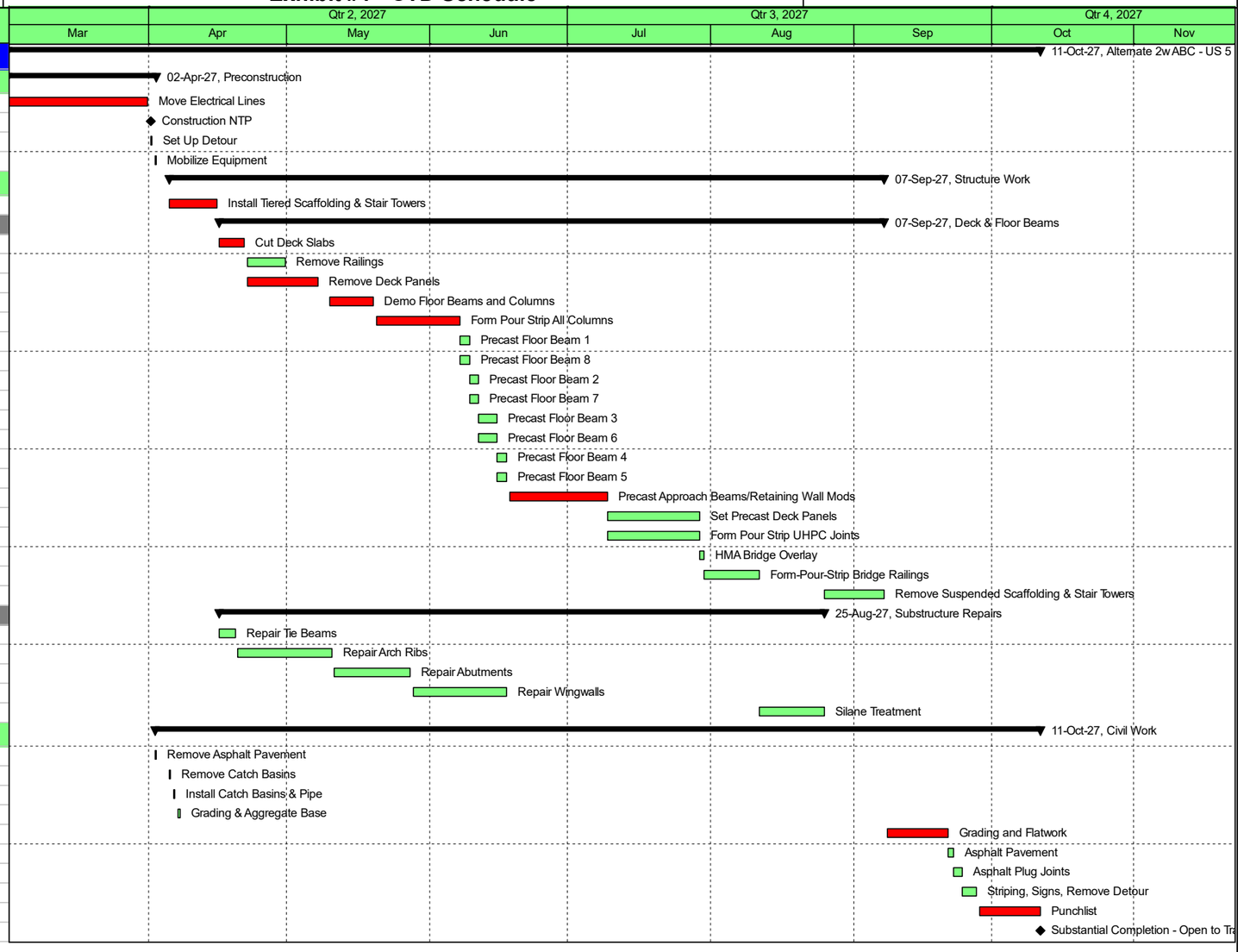
Remaining Level of Effort | Actual Work | Critical Remaining Work | Summary
Actual Level of Effort | Remaining Work | Milestone | WBS Summary Activity

Alternate 2w ABC - US 5 Windsor Arch Bridge Repairs

Exhibit #1 - CTD Schedule

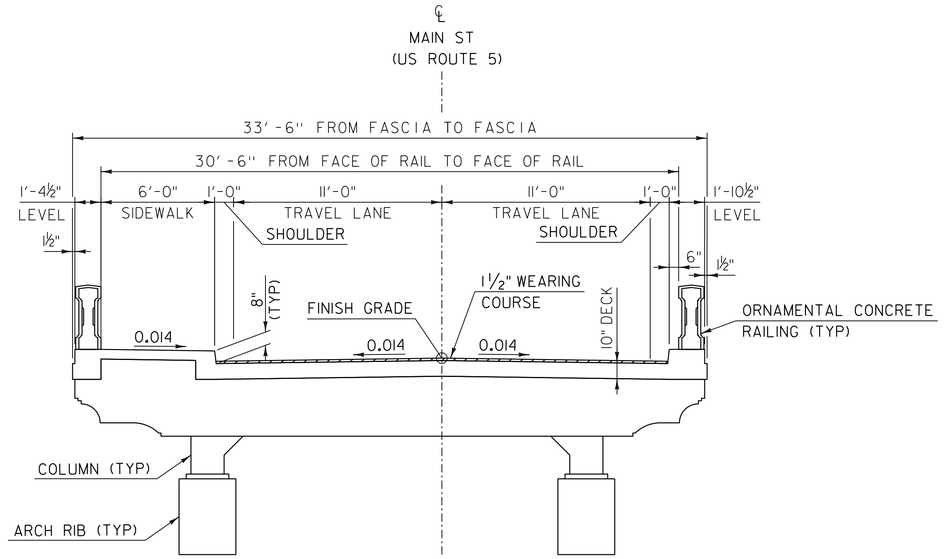
Data Date: 01-Mar-27
Printed on: 09-Sep-25 @ 14:38

Activity ID	Activity Name	Original Duration	Start	Finish	Total Float	Notes
Alternate 2w ABC - US 5 Windsor Arch Bridge R		158	01-Mar-27	11-Oct-27	0	
Preconstruction		25	01-Mar-27	02-Apr-27	0	
PC100	Move Electrical Lines	23	01-Mar-27	31-Mar-27	0	
PC110	Construction NTP	0	01-Apr-27		0	
PC120	Set Up Detour	1	01-Apr-27	01-Apr-27	0	
PC130	Mobilize Equipment	1	02-Apr-27	02-Apr-27	0	
Structure Work		109	05-Apr-27	07-Sep-27	0	
S1000	Install Tiered Scaffolding & Stair Towers	9	05-Apr-27	15-Apr-27	0	
Deck & Floor Beams		100	16-Apr-27	07-Sep-27	0	
DF100	Cut Deck Slabs	4	16-Apr-27	21-Apr-27	0	
DF110	Remove Railings	7	22-Apr-27	30-Apr-27	5	
DF120	Remove Deck Panels	12	22-Apr-27	07-May-27	0	
DF170	Demo Floor Beams and Columns	8	10-May-27	19-May-27	0	
DF190	Form Pour Strip All Columns	12	20-May-27	07-Jun-27	0	
DF180	Precast Floor Beam 1	2	07-Jun-27	09-Jun-27	0	
DF260	Precast Floor Beam 8	2	07-Jun-27	09-Jun-27	0	
DF200	Precast Floor Beam 2	2	09-Jun-27	11-Jun-27	0	
DF280	Precast Floor Beam 7	2	09-Jun-27	11-Jun-27	0	
DF140	Precast Floor Beam 3	2	11-Jun-27	15-Jun-27	0	
DF220	Precast Floor Beam 6	2	11-Jun-27	15-Jun-27	0	
DF160	Precast Floor Beam 4	2	15-Jun-27	17-Jun-27	0	
DF240	Precast Floor Beam 5	2	15-Jun-27	17-Jun-27	0	
DF340	Precast Approach Beams/Retaining Wall Mods	15	18-Jun-27	09-Jul-27	0	
DF290	Set Precast Deck Panels	14	09-Jul-27	29-Jul-27	0	
DF300	Form Pour Strip UHPC Joints	14	09-Jul-27	29-Jul-27	0	
DF320	HMA Bridge Overlay	1	29-Jul-27	30-Jul-27	0	
DF330	Form-Pour-Strip Bridge Railings	8	30-Jul-27	11-Aug-27	0	
DF350	Remove Suspended Scaffolding & Stair Towers	9	25-Aug-27	07-Sep-27	0	
Substructure Repairs		94	16-Apr-27	25-Aug-27	0	
SR120	Repair Tie Beams	2	16-Apr-27	19-Apr-27	25	
SR110	Repair Arch Ribs	15	20-Apr-27	10-May-27	25	
SR130	Repair Abutments	13	11-May-27	27-May-27	25	
SR140	Repair Wingwalls	15	28-May-27	17-Jun-27	25	
SR150	Silane Treatment	10	11-Aug-27	25-Aug-27	0	
Civil Work		134	02-Apr-27	11-Oct-27	0	
CIV10	Remove Asphalt Pavement	1	02-Apr-27	02-Apr-27	9	
CIV20	Remove Catch Basins	1	05-Apr-27	05-Apr-27	119	
CIV30	Install Catch Basins & Pipe	1	06-Apr-27	06-Apr-27	119	
CIV40	Grading & Aggregate Base	1	07-Apr-27	07-Apr-27	119	
CIV90	Grading and Flatwork	10	08-Sep-27	21-Sep-27	0	
CIV50	Asphalt Pavement	1	21-Sep-27	22-Sep-27	0	
CIV60	Asphalt Plug Joints	2	22-Sep-27	24-Sep-27	0	
CIV70	Striping, Signs, Remove Detour	1	24-Sep-27	27-Sep-27	0	
CIV100	Punchlist	10	28-Sep-27	11-Oct-27	0	
CIV80	Substantial Completion - Open to Traffic	0		11-Oct-27	0	



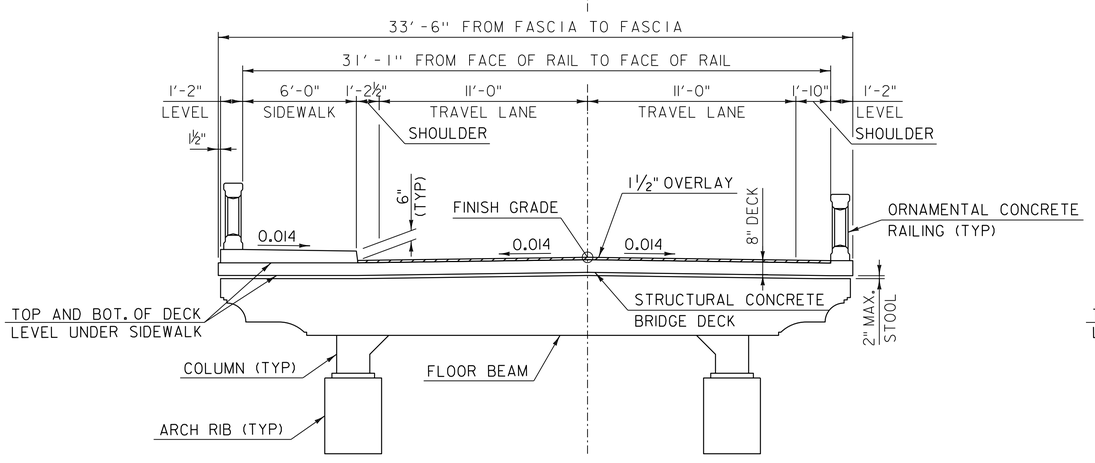
Remaining Level of Effort | Actual Work | Critical Remaining Work | Summary
Actual Level of Effort | Remaining Work | Milestone | WBS Summary Activity

Appendix U: Plans



EXISTING BRIDGE TYPICAL SECTION

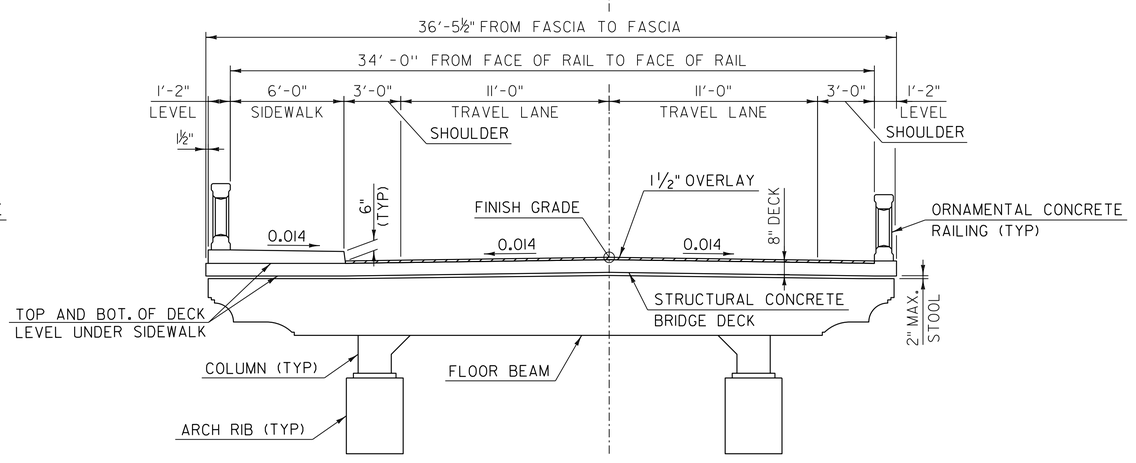
SCALE: ¼" = 1'-0"



PROPOSED BRIDGE TYPICAL SECTION

(WITHOUT WIDENING)

SCALE: ¼" = 1'-0"



PROPOSED BRIDGE TYPICAL SECTION

(WITH WIDENING)

SCALE: ¼" = 1'-0"

PROJECT NAME:	WINDSOR
PROJECT NUMBER:	BF 0113(88)
FILE NAME: z22J404typ.dgn	PLOT DATE: 21-NOV-2025
PROJECT LEADER: R. TETREAULT	DRAWN BY: S. ROMANDINE
DESIGNED BY: S. ROMANDINE	CHECKED BY: A. KINGSLEY
BRIDGE TYPICAL SECTION	SHEET 1 OF 9

HNTB

PRIMARY CONTROL

SPRINGFIELD VT CORS ARP

PID DJ8961
NORTH = 284807.680
EAST = 1646390.520
ELLIP HT = 513.349

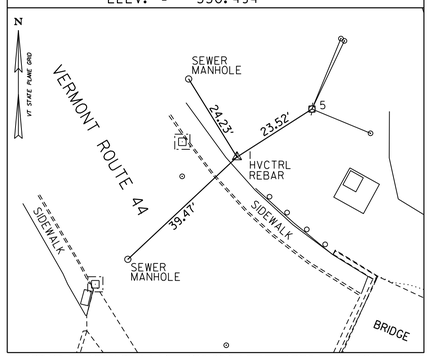
STATION IS A GPS CONTINOUSLY OPERATING REFERENCE STATION. STATION IS THE ANTENNA REFERENCE POINT OF THE GPS ANTENNA. THE MONUMENT IS ATTACHED TO A TWO STORY CONCRETE/BRICK BUILDING WITH AN 8 FT CONCRETE FOUNDATION BUILT IN 1967. THE AST IS A 1.75 INCH DIA GALV PIPE THAT IS 108 INCHES LONG. THE MAST ATTACHED TO A STEEL MOUNTING FRAME WITH THREE ATTACHMENTS CONSISTING OF 3/8 INCH SS THROUGH BOLTS. THE MOUNTING FRAME IS ATTACHED TO THE BUILDING USING 8 ATTACHMENT POINTS. THE TOP 2 and third 2 ARE 1/2 INCH SS BOLTS SECURED TO THE BRICK OR CONC WITH LEAD ANCHORS. THE second 2 and bottom 2 ATTACHMENTS ARE TROUGH BOLTED AND CONSIST OF 1/2 INCH SS THREADED ROD AND NUTS.

PT	NORTH	EAST	ELEV	DESC
2	355313.886	1669528.530	352.415	MAGHUB
11	355704.944	1669612.779	347.767	MAGHUB
12	355774.480	1669604.168	332.867	MAGHUB
13	355724.465	1669540.868	333.928	MAGHUB
30	355149.644	1669706.688	357.986	MAGHUB
31	355276.042	1669877.452	344.630	MAGHUB
40	355619.201	1669943.730	347.606	MAGHUB
41	355478.815	1670018.589	342.223	MAGNAIL
42	355951.459	1670150.505	352.937	MAGHUB
43	355506.945	1669971.417	343.538	MAGHUB
44	355453.730	1669880.035	339.621	MAGNAIL
45	355567.906	1669627.905	343.124	MAGNAIL
311	355111.075	1669957.071	342.202	MAGHUB
312	355097.812	1670023.866	336.990	MAGHUB
400	355489.073	1669964.316	342.139	MAGHUB
401	355464.016	1669925.736	339.727	MAGHUB
402	355437.719	1669877.908	339.068	MAGHUB
440	355506.527	1669683.482	342.503	MAGHUB
441	355466.174	1669685.940	335.858	MAGHUB
442	355453.321	1669732.432	338.842	MAGNAIL
443	355373.397	1669874.453	318.675	MAGHUB
444	355317.730	1669777.581	319.121	MAGHUB
445	355401.418	1669960.559	321.087	MAGNAIL
446	355399.258	1670059.857	307.777	MAGNAIL
447	355132.751	1670111.158	300.411	MAGNAIL

SECONDARY CONTROL

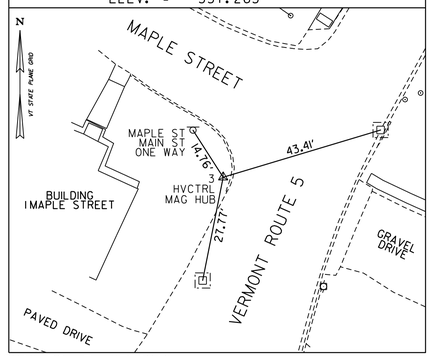
HVCTRL #1

NORTH = 355829.071
EAST = 1669513.012
ELEV. = 350.494



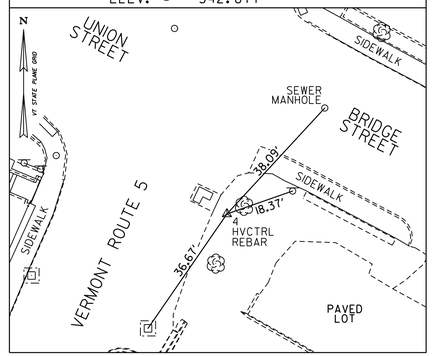
HVCTRL #3

NORTH = 355100.675
EAST = 1669776.530
ELEV. = 357.265



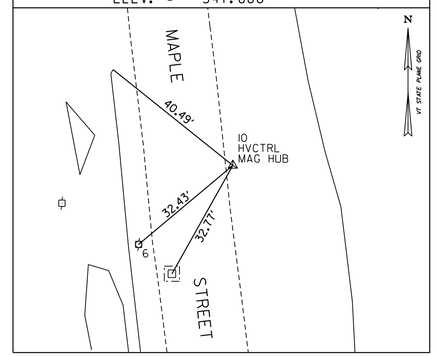
HVCTRL #4

NORTH = 355516.703
EAST = 1670038.589
ELEV. = 342.877



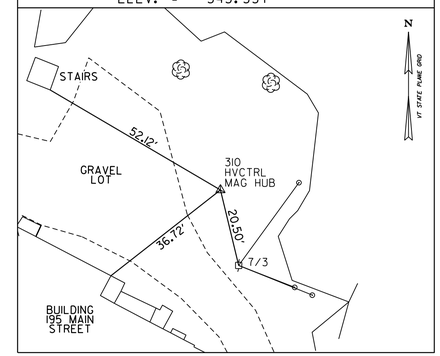
HVCTRL #10

NORTH = 355630.048
EAST = 1669518.998
ELEV. = 347.886



HVCTRL #310

NORTH = 355268.844
EAST = 1669987.716
ELEV. = 343.331



* SURVEY COMPLETED: OCTOBER 6, 2023 BY VSE, M. YEFCHAK-PC, T. YEFCHAK

ALIGNMENT TIES

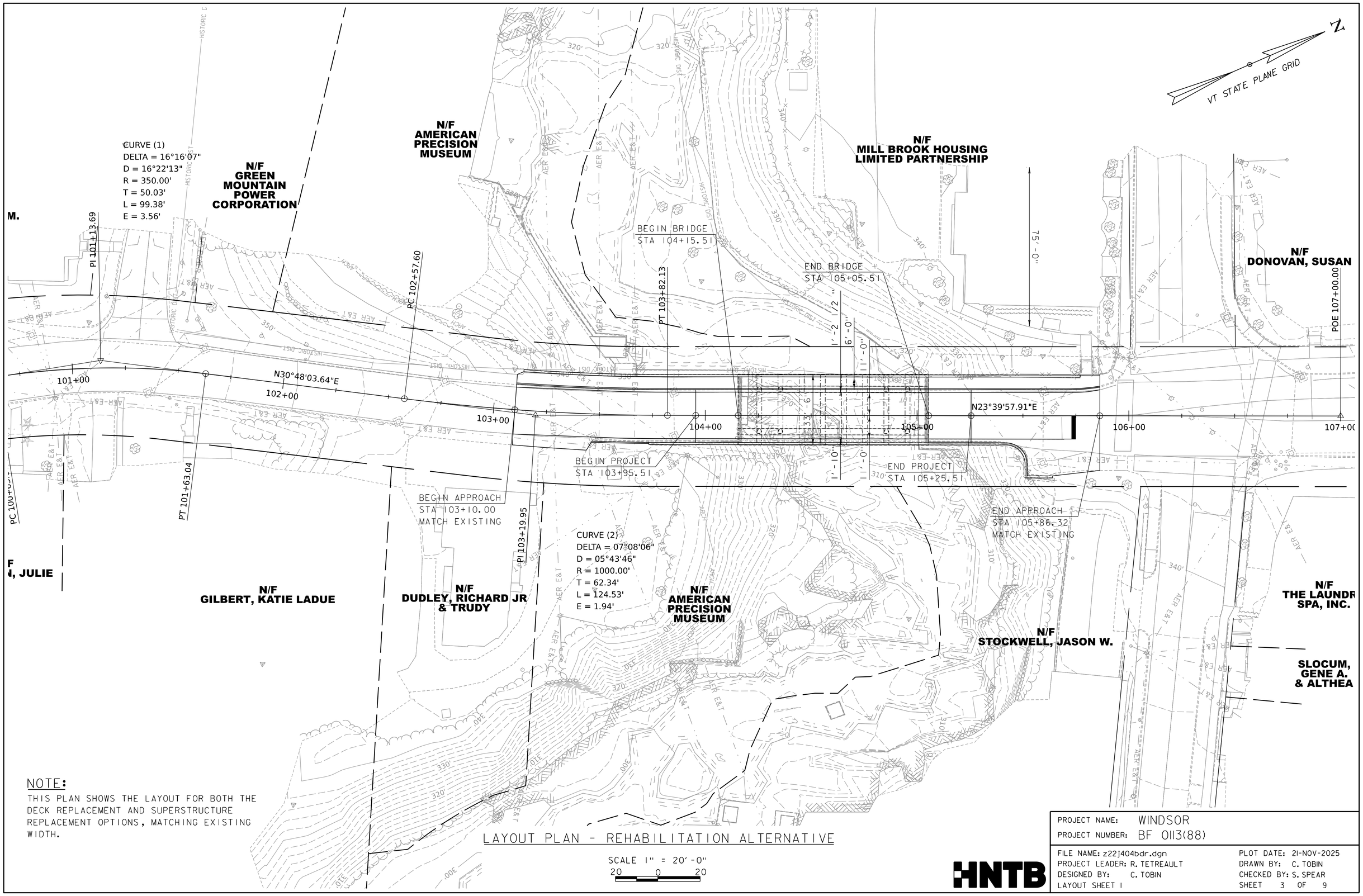
NORTH =	NORTH =	NORTH =	NORTH =	NORTH =
EAST =	EAST =	EAST =	EAST =	EAST =
ELEV. =	ELEV. =	ELEV. =	ELEV. =	ELEV. =

DATUM
VERTICAL NAVD88
HORIZONTAL NAD83(2011)
ADJUSTMENT LSQ

PROJECT NAME: WINDSOR
PROJECT NUMBER: BF 0113(88)

FILE NAME: z22j404ti.dgn
PROJECT LEADER: VTRANS
DESIGNED BY: J. RILEY
TIE SHEET

PLOT DATE: 21-NOV-2025
DRAWN BY: J. RILEY
CHECKED BY: A. McQUEENEY



NOTE:

THIS PLAN SHOWS THE LAYOUT FOR BOTH THE DECK REPLACEMENT AND SUPERSTRUCTURE REPLACEMENT OPTIONS, MATCHING EXISTING WIDTH.

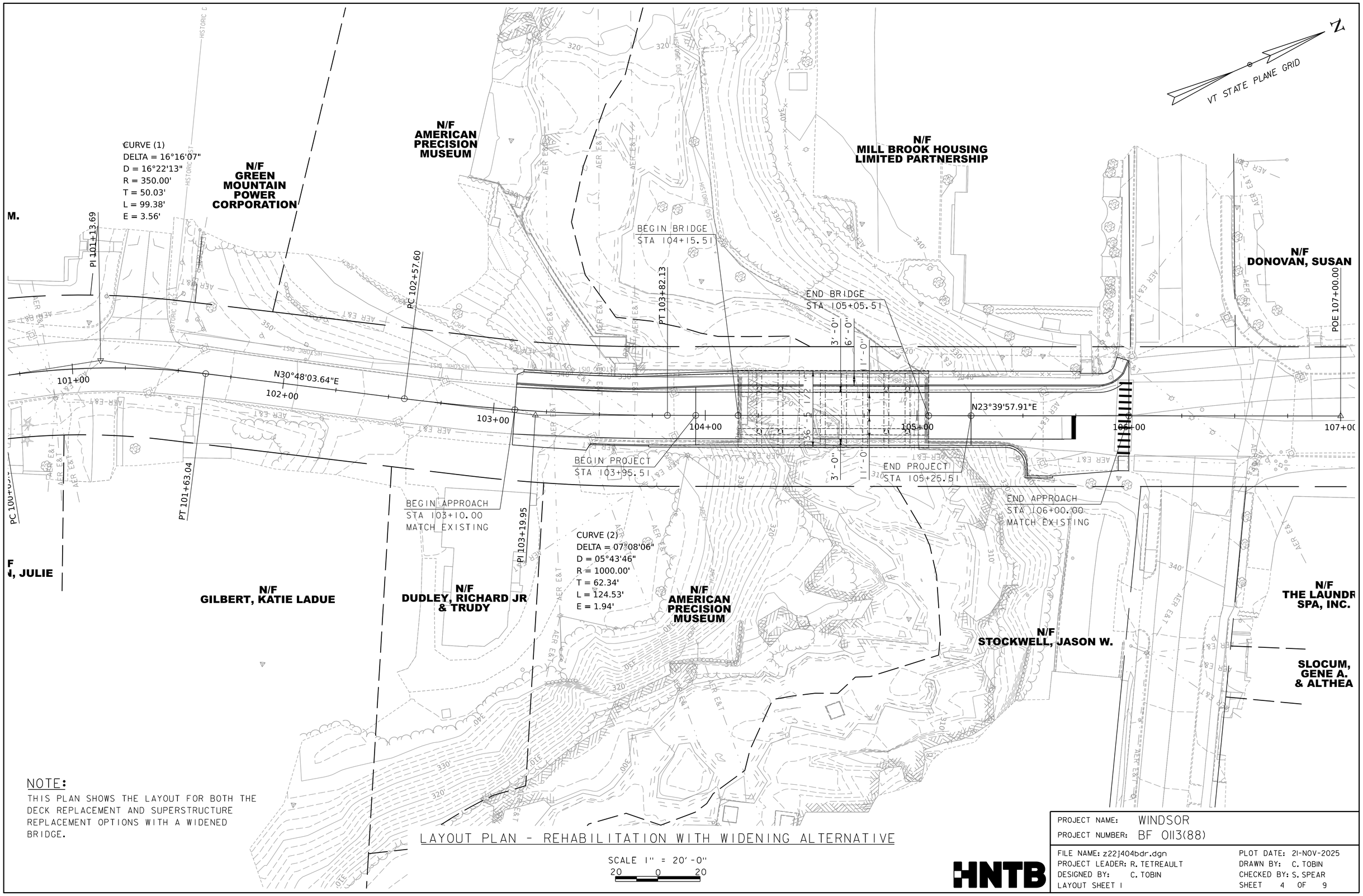
LAYOUT PLAN - REHABILITATION ALTERNATIVE

SCALE 1" = 20'-0"

20 0 20

HNTB

PROJECT NAME:	WINDSOR
PROJECT NUMBER:	BF 0113(88)
FILE NAME: z22j404bdr.dgn	PLOT DATE: 21-NOV-2025
PROJECT LEADER: R. TETREAULT	DRAWN BY: C. TOBIN
DESIGNED BY: C. TOBIN	CHECKED BY: S. SPEAR
LAYOUT SHEET 1	SHEET 3 OF 9



NOTE:

THIS PLAN SHOWS THE LAYOUT FOR BOTH THE DECK REPLACEMENT AND SUPERSTRUCTURE REPLACEMENT OPTIONS WITH A WIDENED BRIDGE.

LAYOUT PLAN - REHABILITATION WITH WIDENING ALTERNATIVE

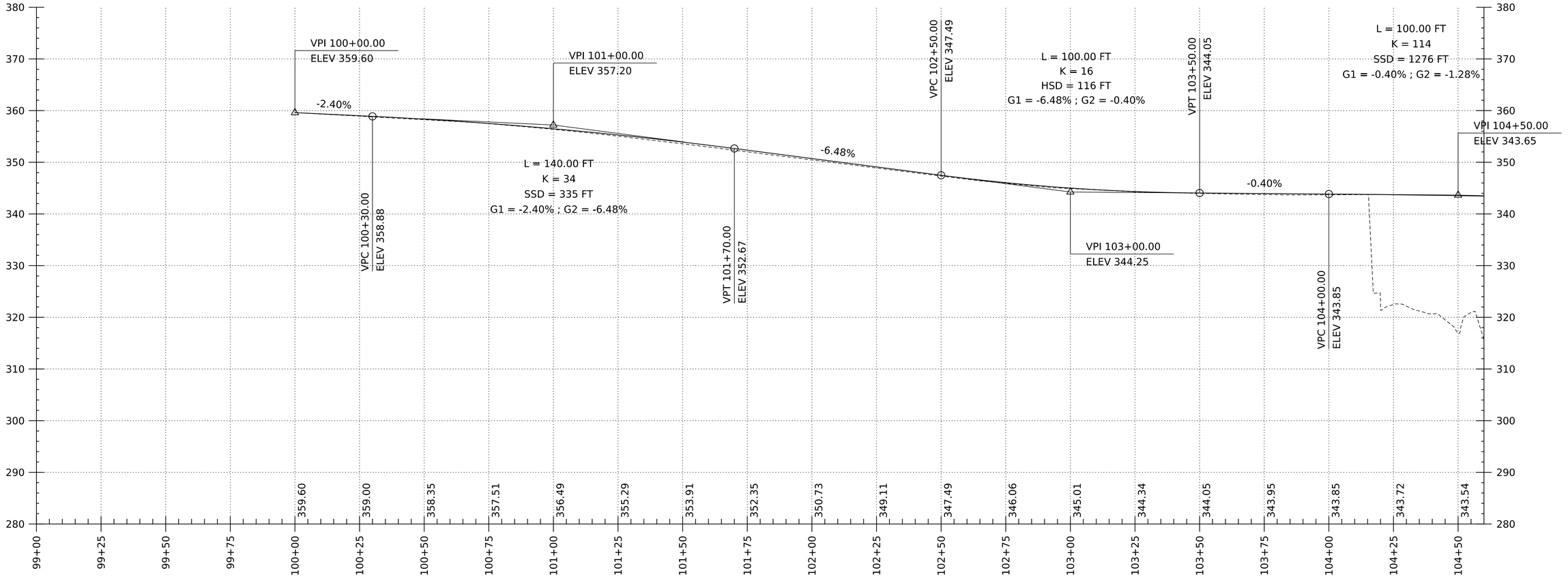
SCALE 1" = 20'-0"
20 0 20

HNTB

PROJECT NAME: WINDSOR
PROJECT NUMBER: BF 0113(88)

FILE NAME: z22j404bdr.dgn
PROJECT LEADER: R. TETREAULT
DESIGNED BY: C. TOBIN
LAYOUT SHEET 1

PLOT DATE: 21-NOV-2025
DRAWN BY: C. TOBIN
CHECKED BY: S. SPEAR
SHEET 4 OF 9

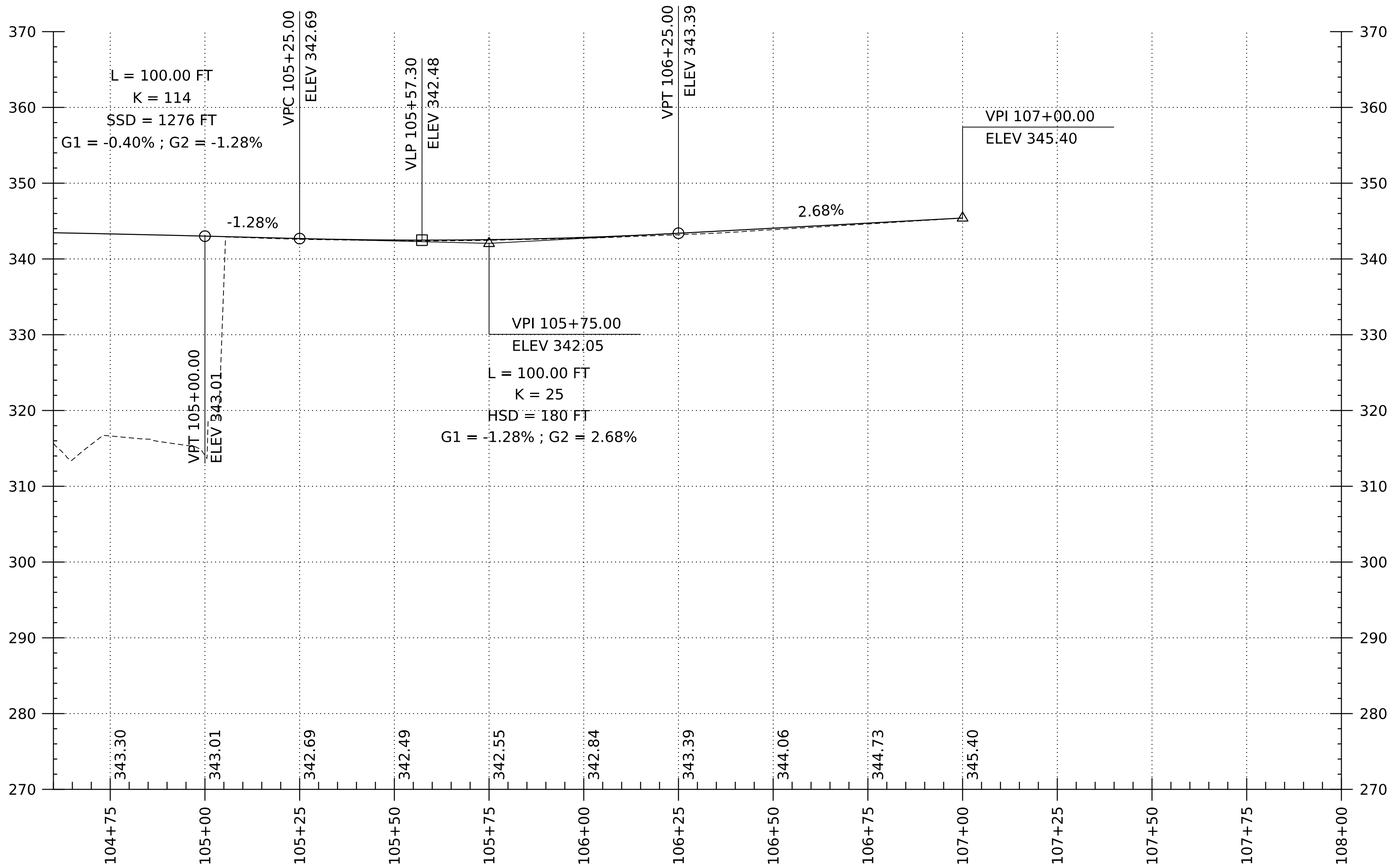


MAIN ST (US ROUTE 5) PROFILE

HORIZONTAL SCALE: 1" = 20'-0"
VERTICAL SCALE: 1" = 10'-0"

PROJECT NAME:	WINDSOR
PROJECT NUMBER:	BF 0113(88)
FILE NAME: z22j404pro.dgn	PLOT DATE: 21-NOV-2025
PROJECT LEADER: R. TETREAULT	DRAWN BY: C. TOBIN
DESIGNED BY: C. TOBIN	CHECKED BY: S. SPEAR
MAIN ST PROFILE	SHEET 5 OF 9

HNTB

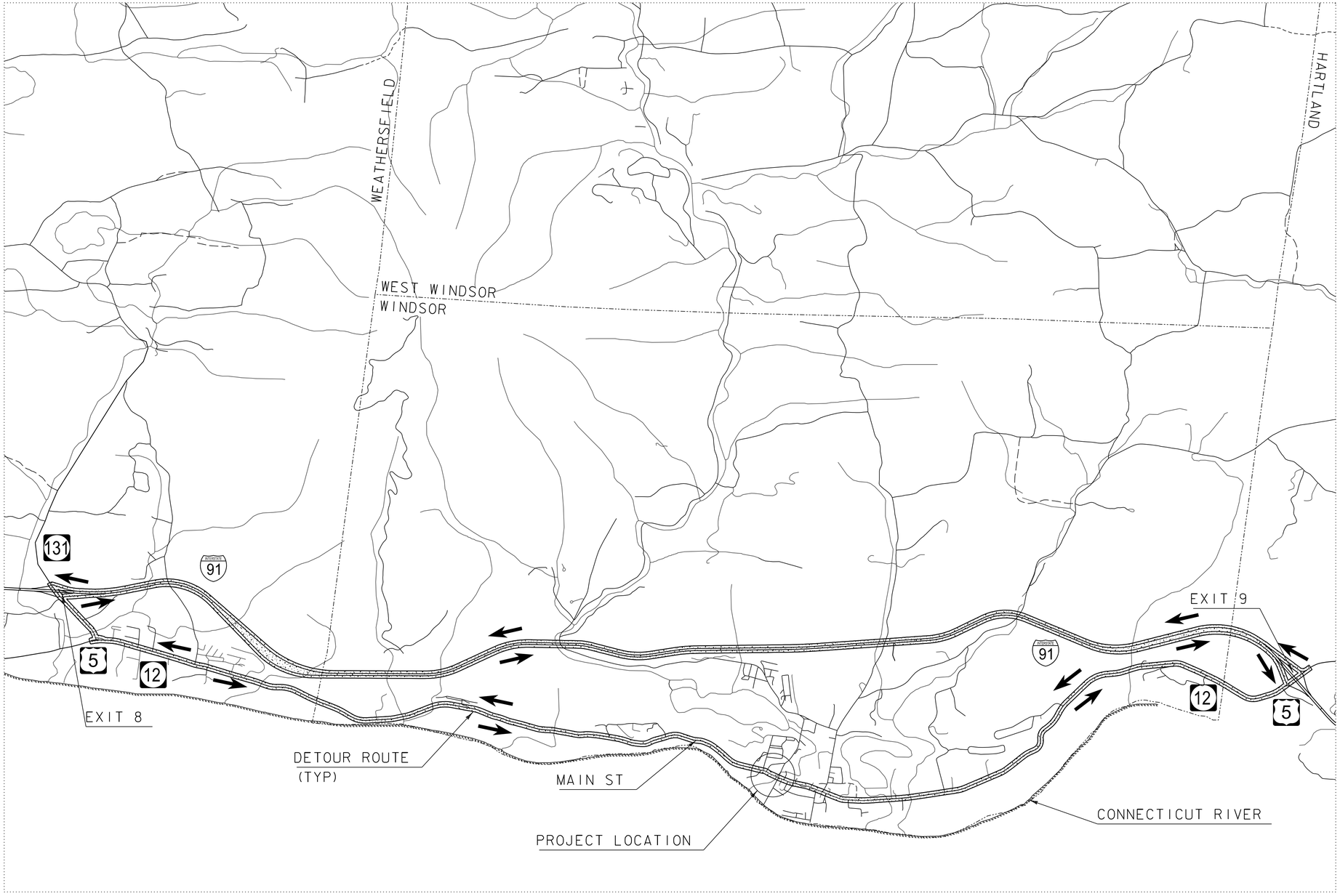


MAIN ST (US ROUTE 5) PROFILE

HORIZONTAL SCALE: 1" = 20'-0"
VERTICAL SCALE: 1" = 10'-0"

HNTB

PROJECT NAME:	WINDSOR
PROJECT NUMBER:	BF 0113(88)
FILE NAME: z22j404pro.dgn	PLOT DATE: 21-NOV-2025
PROJECT LEADER: R. TETREAULT	DRAWN BY: C. TOBIN
DESIGNED BY: C. TOBIN	CHECKED BY: S. SPEAR
MAIN ST PROFILE	SHEET 6 OF 9



DETOUR MAP

(NOT TO SCALE)

DETOUR NOTE:

DETOUR PLAN REFLECTS A COMPLETE CLOSURE AND DISPLAYS THE PROPOSED DETOUR ROUTE. THIS IS INTENDED FOR TRUCK USE AND IMPLEMENTED IN CONJUNCTION WITH A SMALLER LOCAL DETOUR.

HNTB

PROJECT NAME: WINDSOR	
PROJECT NUMBER: BF 0113(88)	
FILE NAME: z22j404detmap.dgn	PLOT DATE: 21-NOV-2025
PROJECT LEADER: R. TETREAULT	DRAWN BY: C. TOBIN
DESIGNED BY: C. TOBIN	CHECKED BY: S. SPEAR
REGIONAL DETOUR MAP	SHEET 7 OF 9

TRAFFIC CONTROL NOTES:
1. ACCESS TO ALL EXISTING DRIVES AND SIDE ROADS WILL BE MAINTAINED.

2. THIS DETOUR PLAN IS A CONCEPTUAL OUTLINE ONLY.

3. ADDITIONAL DESIGN DEVELOPMENT IS ANTICIPATED TO OCCUR IN FINAL DESIGN AND MINOR IMPROVEMENTS TO INTERSECTIONS ALONG THE DETOUR OR SIGNAL TIMING MODIFICATIONS MAY BE REQUIRED.



TRAFFIC CONTROL PLAN - LOCAL DETOUR
SCALE 1" = 80'-0"
80 0 80

HNTB

PROJECT NAME:	WINDSOR
PROJECT NUMBER:	BF 0113(88)

FILE NAME: z22j404TCP.dgn	PLOT DATE: 16-APR-2026
PROJECT LEADER: R. TETREAULT	DRAWN BY: C. TOBIN
DESIGNED BY: C. TOBIN	CHECKED BY: S. SPEAR
TRAFFIC CONTROL PLAN	SHEET 9 OF 9

WINSDOR
LOAM SAND,
25%-60% SLOPES
LOW EROSION POTENTIAL
K=UNKNOWN

URBAN LAND
WINSDOR-AGAWAM,
0%-8% SLOPES
LOW EROSION POTENTIAL
K=UNKNOWN

VT STATE PLANE GRID

N/F
AMERICAN
PRECISION
MUSEUM

N/F
GREEN
MOUNTAIN
POWER
CORPORATION

N/F
MILL BROOK HOUSING
LIMITED PARTNERSHIP

N/F
DONOVAN, SUSAN

N/F
GILBERT, KATIE LADUE

N/F
DUDLEY, RICHARD JR
& TRUDY

N/F
AMERICAN
PRECISION
MUSEUM

N/F
STOCKWELL, JASON W.

N/F
THE LAUNDR
SPA, INC.

SLOCUM,
GENE A.
& ALTHEA

GRASS
GRAVEL
TREES
WATER
PARKING LOT
BUILDING
PAVEMENT
DIRT

N30°48'03.64"E
N23°39'57.91"E

101+00
102+00
103+00
104+00
105+00
106+00
107+00

EASEMENT, BK. 79, PG. 118
EASEMENT, BK. 79, PG. 339
EASEMENT BK. 79, PG. 339
EASEMENT AER BK. 79, PG. 118
AER E&T EASEMENT BK. 79, PG. 118
EASEMENT BK. 61, PG. 515

HISTORIC DIST
AER E&T
ARCH

EXISTING CONDITIONS

SCALE 1" = 20'-0"
20 0 20

HNTB

PROJECT NAME:	WINDSOR
PROJECT NUMBER:	BF 0113(88)
FILE NAME: z22j404bdr_ero.dgn	PLOT DATE: 21-NOV-2025
PROJECT LEADER: R. TETREAULT	DRAWN BY: C. TOBIN
DESIGNED BY: C. TOBIN	CHECKED BY: S. SPEAR
LAYOUT SHEET 1	SHEET 9 OF 9