STATE OF VERMONT
AGENCY OF TRANSPORTATION

Scoping Report

FOR

Springfield IM 091-1(74)

Interstate Route 91, Bridges 26 N/S over the Black River

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1. SITE INFORMATION

The bridges carry Interstate Route 91 Northbound and Southbound over the Black River in the town of Springfield and are located 0.4 miles south of Exit 7. The existing conditions were gathered from a combination of a Site Visit, the Inspection Reports, and the existing Survey. See correspondence in the Appendix for more detailed information.

Roadway Classification	Principal Arterial - Interstate
Bridge Type	3-Span Welded Steel Plate Girder with suspended middle span
Bridge Length	310 feet
Year Built	1965
Ownership	State of Vermont

1.1. Need

The following is a list of deficiencies of Springfield Bridges 26 Northbound and Southbound

1. Pin and Hanger Suspenders are fracture critical
2. Aluminum Bridge Railing is substandard
3. Transitions are substandard
4. Areas of heavy deterioration and exposed reinforcing steel on curbs

1.2. Traffic

A traffic study of this site was performed by the Vermont Agency of Transportation. The traffic volumes are projected for the years 2017 and 2037.

	NORTHBOUND		SOUTHBOUND	
TRAFFIC DATA	2017	2037	2017	2037
AADT	5700	6600	5700	6600
DHV	920	1100	1200	1400
ADTT	1300	2100	1300	2000
%T	16.1	22.5	16.0	21.7
%D	100	100	100	100

1.3. Design Criteria

The design standards for this bridge project are the AASHTO A Policy on Geometric Design of Highways and Streets (Green Book, 2011 Edition), VTrans Structures Design Manual (5th Edition, 2010), and the VTrans Hydraulics Manual (2015 Edition). Minimum standards are based on an AADT of 6600 and a design speed of 70 mph.

Design Criteria	Source	Existing Condition	Minimum Standard	Comment
Approach Lane and Shoulder Widths	AASHTO Green Book Section 8.2.4	2-12' Lanes 4' High Speed Shoulder 12' Low Speed Shoulder	2-12' Lanes 4' High Speed Shoulder 10' Low Speed Shoulder	
Bridge Lane and Shoulder Widths	AASHTO Green Book Section 10.8.3	2-12' Lanes 3' High Speed Shoulder 3' Low Speed Shoulder	Same as approach roadway (preferred)[1]	Meets minimum tolerable criteria
Clear Zone Distance	AASHTO Roadside Design Guide, 4th Edition, 2011 – Table 3-1	No issues noted. Existing Guardrail at all bridge approaches.	Foreslopes: 30' Backslopes: 22' - 30'	
Banking	AASHTO Green Book Section 8.2.4 and 8.2.6	Normal crown (Parabolic on bridges)	2% (minimum) 6-8% (maximum)	
Speed	AASHTO Green Book Section 8.2.1	65 mph (design)	50mph(minimum) 70mph(desirable)	
Horizontal Alignment	AASHTO Green Book Table 3-9	Tangent – No curves on the bridges	R(min)=2,040' @ 6.0%	
Vertical Grade	AASHTO Green Book Table 8-1	Bridges located on a 0.500% grade	5% (maximum) for mountainous terrain	
K Values for Vertical Curves	AASHTO Green Book Table 3-34 and 3-36	Bridge profiles are straight	247 crest / 181 sag	
Vertical Clearance Issues		No issues noted	No less than existing	Prelim. Hydraulic Assessment
Stopping Sight Distance	AASHTO Green Book Table 3-1	Not limited by bridge	730'	
Bicycle/Pedestrian Criteria		Bicycles/Pedestrians Prohibited on Interstate	Bicycles/Pedestrians Prohibited on Interstate	
Bridge Railing	Structures Manual Section 13	Aluminum three rail	TL-4	Substandard
Hydraulics	VTrans Hydraulics Section	Adequate		Slight chance over overtopping roadway, stable for scour (Insp. Reports)
Structural Capacity	SM, Ch. 3.4.1	Design Live Load: H20	Design Live Load: HL-93	

[1]AASHTO Green Book Section 4.4.2 – "Partial Shoulders are sometimes used where the full shoulders are unduly costly, such as on long (over 200-ft) bridges or in mountainous terrain."

1.4. Inspection Report Summary

	Northbound	Southbound
Deck Rating	6 Satisfactory	5 Fair
Superstructure Rating	7 Good	5 Fair
Substructure Rating	8 Very Good	7 Good
Channel Rating	8 Very Good	8 Very Good

From the Structure Inspection, Inventory, and Appraisal Sheet:

1/9/2015 – (Northbound) The full depth hole on the north end of the suspended section in span #4 on the downstream side of the joint and in the curb have been repaired and is in good condition ~FRE/MJK

6/18/2014 – (Northbound) …Beams should be spot cleaned and painted ~FRE/TJB

6/18/2014 – (Southbound) Beams should be spot cleaned and painted. Curbs should be cleaned and patched. Tear drop rail on the upstream side at midspan should be repaired. ~FRE/TJB

6/8/2012 – (Northbound) Structure is in good condition however the curbs should be cleaned and patched. Debris at pier #1 should be removed. ~FRE/SJH

6/8/2012 – (Southbound) Curbs should be cleaned and patched. Structure should be spot cleaned and painted. Joint areas in the soffit should be cleaned and patched. ~FRE/SJH

4/12/2010 – (Northbound) Right bridge rail along span No. 1 is in need of repairs. (Southbound) Left bridge rail of span No. 3 is in need of repairs. (Northbound and Southbound) Large concrete cavities along both curb areas are in need of repairs. The joint areas above both pin and hanger locations of the suspended span are in need of major repairs to arrest ongoing corrosion activities of the girder end areas. Servi-lift inspection performed on 7/15/2010. PLB

1.5. Hydraulics

From Preliminary Hydraulics Report (PHR):

"The existing bridges are more than adequate hydraulically…if the bridges are rehabilitated, there should be no changes that would reduce the waterway area below elevation 308.0', which includes abutments and fill material. Bottom of beams should be above elevation 315.0'."

1.6. Utilities

There are no utilities on the bridges, overhead, or buried in the approaches.

1.7. Right of Way

The existing Right-of-Way is shown on the Existing Conditions Layout Sheet. The bridges are located well within the Right-of-Way and it is anticipated that additional rights will not be needed.

1.8. Resources

The resources present at this project are summarized below, shown on the Existing Conditions Layout Sheet, and discussed in greater detail in the memos included in the Appendix.

1.8.1. Biological:

Wetlands/Watercourses

Class II wetlands are located in all four quadrants of the project area. If impacts are to occur to the mapped wetlands, a formal wetland delineation will be required before any impacts can be calculated for permitting purposes.

Wildlife Habitat

The project area is not considered significant for wildlife habitat on a statewide or northeastern US level, however there is evidence of small mammals crossing under the existing structure. It is recommended by VTrans Environmental Biologist that the riprap under the bridge be grubbed to allow for easy animal movement under the bridge.

The Black River is an Essential Fish Habitat, therefore the VT Fish and Wildlife AOP guidelines will need to be followed to accommodate aquatic organism passage.

Rare, Threatened and Endangered Species

The project area is within the mapped habitat for dwarf wedgemussels. Wedgemussels are both state and federally listed as endangered, and are state listed as S1 (Very Rare (Critically Imperiled)). Any work in the water or that leads to disturbance to the waters may require a survey and relocation of wedgemussels within the project area.

A vascular plant ranked as S3 (Uncommon (Vulnerable)) has been observed north of the northeast quadrant. Work in this area should be avoided. If it is unavoidable, an inventory of these plants should be performed to identify any existing plans to minimize impacts.

The Northern Long-eared Bat is listed by the US Fish & Wildlife Service as threatened and by the Vermont Fish & Wildlife Department as endangered. Guidance from FHWA and FRA indicates that all trees greater than or equal to three inches in diameter that exhibit cracks, crevices, holes, and peeling bark are considered suitable roost trees. Therefore, a habitat assessment will be required before any necessary tree clearing, unless the clearing can be conducted from November 1st through April 15th.

Agricultural

There are two Prime Agricultural soils within the project area. Ninigret fine sandy loam 0-8% slopes (prime ag soil-9B) is located in all four quadrants. Podunk fine sandy loam 0-3% slopes (prime ag soil-24) is located in the northeast quadrant.

1.8.2. Archaeological

There is a high density of archaeological sites within the vicinity of the project area; at least 7 known sites are nearby, most of which are pre-contact. Areas of archaeological sensitivity were found within all four quadrants of the project site and are marked on the Arch Sensitive Lines map with the Archaeological memo in the Appendix.

1.8.3. Historic

This project is considered exempt for above-ground resources per the Section 106 Exemption Regarding Effects to the Interstate Highway System (see correspondence included in Appendix).

1.8.4. Hazardous Materials:

There are no known hazardous materials in the project area.

1.8.5. Stormwater:

As noted in the Resource ID Completion Memo: "OSW unlikely, Springfield CMG PARK(32) permit is nearby; need calculations for CSW". When the project scope is determined, this will be revisited.

2. CONDITION ASSESSMENT

While the bridges are not coded structurally deficient, there are two existing conditions that are deficient and require immediate action: the fracture critical pin and hanger suspenders should be retrofitted, and the substandard aluminum bridge railing should be replaced. The condition of the deck, expansion joints, superstructure steel, high level bearings, and substructure were evaluated to determine additional repairs and develop appropriate combinations for rehabilitation alternatives. Additionally, a cursory seismic evaluation was performed to understand the anticipated performance and vulnerability of the structure for the design earthquake and the outcome of a seismic retrofit strategy.

2.1. <u>Deck Patching</u>

The bridge decks have been classified as Satisfactory and Fair. There is visible deterioration to the underside, particularly near the joints, and the deck cores (see Appendix) indicate that the deck has been overlaid. While deck patching appears to be a viable repair on its own, when combined with other repairs it becomes less appropriate.

For example, the severely deteriorated curb cannot receive a new steel railing, and it is likely that the deck below the curb will be inadequate to structurally support a new curb. Thus, it is anticipated, that to replace the substandard aluminum railing with a TL-4 steel railing, a full depth section of deck along each fascia will need to be removed and replaced to receive a new concrete curb. Additionally, if the expansion joints are to be replaced (as discussed in Section 2.4), a section of deck (full depth by full bridge width) on either side of the joint will need to be removed and replaced. It does not seem practical, from a life-cycle cost perspective, to perform extensive full-depth repairs and patching to a deck from 1965 that has been previously patched and overlaid.

2.2. Deck Replacement

Full deck replacement should be considered to increase the life of the structure. Deck replacement would involve removing the existing deck in its entirety and placing a new deck on the existing steel girders. A concrete F-Shape TL-5 bridge barrier will be included with a full depth deck replacement alternative. The F-Shape barrier is narrower than the existing curb, which will increase the total roadway width by about two feet.

2.3. Pin and Hanger Retrofit

2.3.1. Catcher's Mitt

The first retrofit considered is a "Catcher's Mitt" detail that will not eliminate the pin and hanger, but will provide a safety upgrade and a contingency support. This detail consists of connecting a steel beam to the bottom flange of the cantilevered girder and extending it below the suspended girder to "catch" that girder in the event of suspender failure (see Figure 1). This is an appropriate repair if the deck and existing high level bearings are to remain; the steel repair can be completed from below the deck and the loads and expansion will not be altered.

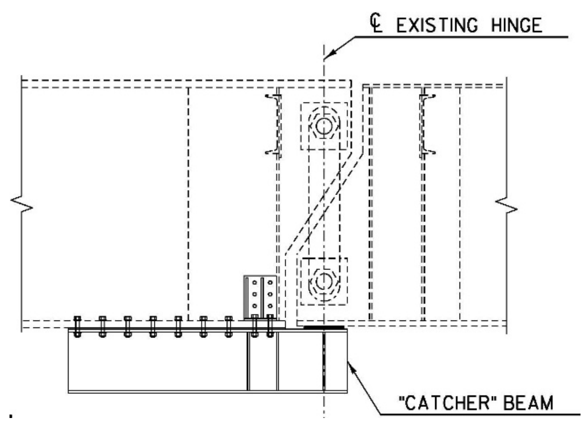


Figure 1: "Catcher's Mitt" Retrofit

It should be noted that this alternative will marginally reduce the flood elevation freeboard, which differs from the assumptions made during the Preliminary Hydraulics Assessment.

The viability of this retrofit is dependent on the results of the next pin test, which is anticipated to be scheduled for 2017.

2.3.2. Seated Girder

As an alternative to the "Catcher's Mitt" retrofit, a "Seated Girder" detail can be considered; it will provide a more robust repair since it eliminates the pin and hanger, but it will maintain the existing suspended span configuration so it is appropriate if the existing bearings are to remain. This detail is shown in Figure 2, and consists of removing the pin and hanger and a section of girder on either side of it, and splicing in two new steel sections, one bearing on the other. To perform this repair, the deck above must be removed and the girders must be supported – this could be achieved with suspended "saddle" beams.

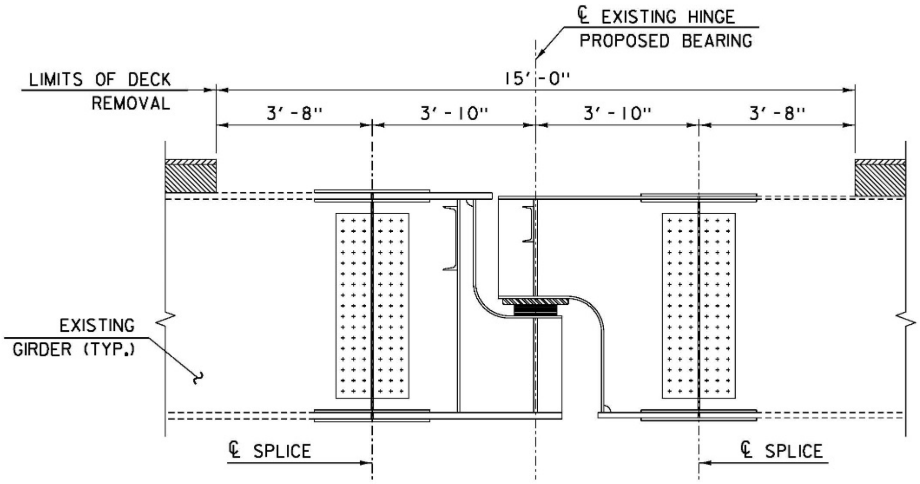


Figure 2: "Seated Girder" Retrofit

2.3.3. Continuous Splice

The third retrofit under consideration is a bolted field splice that will make the girders continuous across all three spans, referred to as the "Continuous Splice" retrofit (see Figure 3). This would require the same level of effort as the Seated Girder retrofit; the deck above the repair must be removed and the girders supported, which could be achieved with suspended "saddle" beams. The pin and hanger and a section of girder on either side of it would be removed, and a new steel section spliced between. This repair changes the load distribution and expansion of the structure and is only considered in combination with a bearing replacement.

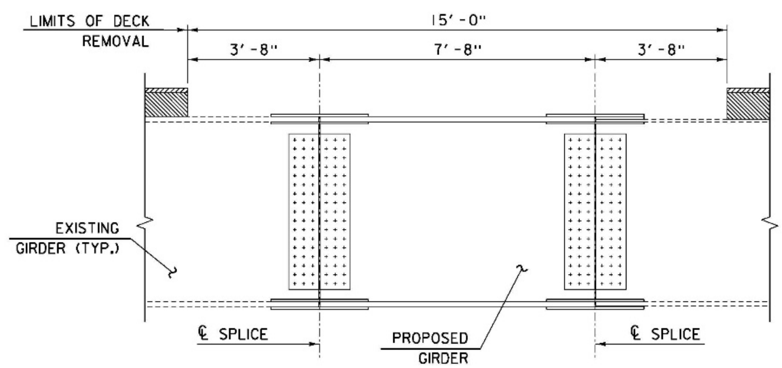


Figure 3: "Continuous Splice" Retrofit

2.4. Expansion Joints

The expansion joints over the pin and hanger are leaking and should be replaced as part of every rehabilitation alternative. The calculated range of thermal movement, under a 150-degree temperature differential, is 2.34" - this can be accommodated by the Vermont Joint. A product such as Emseal Bridge Expansion Joint System (BEJS) can also be considered under this range of movement. The BEJS is a product that MassDOT has had recent success for retrofit joint replacements. If the pin and hanger is retrofitted with a Continuous Splice, the deck will be made continuous and the expansion joints will be placed at the abutments behind the backwall. The expansion under the continuous configuration will increase to approximately 3.8", which can be accommodated by the Fingerplate Vermont Joint.

2.5. Bearing Replacement

The existing girders are supported by high level bearings, and a few of the expansion rocker bearings have extended beyond the adjacent bearings. While this is not an urgent matter and may indicate that they were originally set incorrectly, it could also suggest that the bearing lines are not moving uniformly and may be "walking". In that event it is likely that they will continue to shift and eventually will require replacement. It is not reasonable to jack the structure only to realign the dislocated bearings thus bearing repair has not been included with any of the rehabilitation alternatives. Instead, the alternatives either assume the bearings will remain or that the structures will be jacked and the bearings will be replaced with steel reinforced elastomeric bearing pads. It should also be noted that high level rocker bearings have a demonstrated history of being vulnerable to toppling in seismic events.

It is recommended that replacement bearings be designed according to AASHTO Method B. While bearings designed according to Method B require more extensive material and fabrication testing than Method A, this increased knowledge of the bearing's material properties permits Method B bearings to provide a more refined design. The additional testing, according to the fabricators and testing laboratories, does not significantly increase construction costs because many fabricators perform the testing as part of their normal QA/QC procedures. Method B design would be required if bearing replacement were to be used as part of a seismic retrofit strategy.

It is also recommended that the replacement bearings be designed to let the bridge "float". A "floating bridge" is a structure fully carried on elastomeric bearings without defined fixed or expansion bearings. This detail relies on keeper blocks or shear keys and backwalls to contain the required displacement that must be accommodated during a seismic event in the substructure, and these components are designed elastically to do so. This concept reduces the demand on individual "fixed" substructure units by distributing to all substructure units, and is an appropriate rehabilitation option for these bridges. The "floating bridge" concept also improves the expansion characteristics of the structure, by allowing expansion and contraction from its own center of stiffness, and not a defined fixed point.

2.6. Substructure

The substructures are in very good (northbound) and good (southbound) condition. It is anticipated that after sounding their surfaces, only minor repairs will be required. Each rehabilitation alternative includes a nominal quantity of shallow concrete patch repairs and pressure injection of cracks.

2.7. Preliminary Seismic Evaluation

The structures convey an interstate highway over the Black River, and are thus considered critical essential bridges. A preliminary seismic analysis was performed to develop a basic understanding of how the bridges might perform during the design seismic event. The approach and assumptions of this analysis are discussed in detail in the Appendix, and proposed methods for addressing the anticipated deficiencies is discussed in Section 3.7.

Two conditions were evaluated; the existing condition, and a proposed condition with replacement bearings and an isolated superstructure. For the existing condition, the analysis shows that the pier stems are inadequate for flexure with a capacity-to-demand (C/D) ratio of 0.33, and the piles will be overstressed. For the proposed condition, the analysis shows that the pier stem is still inadequate, though the C/D ratio has increased to 0.67, and the piles are just adequate (at approximately 92% of yield).

It should be noted that this analysis is very simplified and hence very conservative. For these bridges, a detailed seismic analysis should be performed to provide more refined seismic loads, but it is very likely that retrofitting is warranted. Because these are multispan bridges, classified as critical essential and located on a Seismic Zone 2 site, this refinement would include a multimode analysis of the entire structure (AASHTO LRFD Table 4.7.4.3.1-1).

3. ALTERNATIVE DISCUSSION

3.1. No Action

This alternative leaves the bridge in its current condition, which means the deficiencies of the structure (the fracture critical pin and hanger suspenders and the substandard bridge railing and transitions) would not be addressed. For these reasons, the No Action alternative is not recommended. No cost estimate has been provided for this alternative since there are no immediate cost considerations.

3.2. Alternative 1: "Catcher's Mitt" Retrofit

This alternative would retrofit the pin and hanger detail with the "Catcher's Mitt", which is a safety upgrade that provides a contingency support for the existing detail against fatigue failure. The existing bridge railing, expansion joints, and high level bearings would remain and the substructure surfaces would be repaired. Since the steel and substructure repairs can be completed from below the deck, traffic would not be affected.

Advantages: This alternative provides an immediate safety upgrade to the pin and hanger, while keeping the cost to a minimum. There would be minimal impacts to biological resources and no impact to the nearby archaeologically sensitive resources.

Disadvantages: The bridge deck, expansion joints, substandard railing, and high level bearings remain.

3.3. Alternative 2a: Deck Replacement and "Seated Girder" Retrofit

This alternative would include the associated deck replacement repairs discussed in Section 2.2, and would replace the pin and hanger detail with the "Seated Girder" detail. The bridge railing would be a concrete TL-5 F-Shape barrier, the expansion joints would be replaced and the substructure surfaces would be repaired. The high level bearings would remain and the superstructure steel would be blast cleaned and painted. Traffic can be maintained by any of the methods described in Section 4, however it is understood that median crossovers would be preferred by VTrans for a deck replacement.

Advantages: The pin and hanger detail is eliminated, the substandard bridge railing is replaced, the superstructure is cleaned and painted, and the design life is approximately 40 years. There would be minimal impacts to biological resources and no impact to the nearby archaeologically sensitive resources.

Disadvantages: Higher cost compared to Alternative 1 and the high level bearings remain. The shoulder widths would remain substandard.

3.4. Alternative 2b: Deck Replacement, "Continuous Splice" Retrofit and Bearing Replacement

This alternative would include the associated deck replacement repairs discussed in Section 2.2, and would retrofit the pin and hanger detail with the Continuous Splice. This necessitates jacking the bridge and replacing the high level bearings with steel reinforced elastomeric bearings. The bridge railing would be a concrete TL-5 F-Shape barrier, the expansion joints would be moved to the abutments, the substructure surfaces would be repaired, and the superstructure steel would be blast cleaned and painted. Traffic can be maintained by any of the methods described in Section 4, however it is understood that median crossovers would be preferred by VTrans for a deck replacement.

Advantages: The suspended span is eliminated, and the continuous girders on new bearings will provide upgraded seismic performance, as compared to the "Seated Girder" detail. The substandard bridge railing is replaced and the entire superstructure will be blast cleaned and painted. The design life of a deck replacement is approximately 40 years. There would be minimal impacts to biological resources and no impact to the nearby archaeologically sensitive resources.

Disadvantages: Higher up front cost, as compared to Alternative 2a, to jack the bridge and replace the bearings. The shoulder widths would remain substandard.

3.5. Alternative 3: Superstructure and Bearing Replacement

This alternative would involve removing and replacing the existing superstructure and high level bearings with a similar structure consisting of a concrete deck on continuous welded steel girders and steel reinforced elastomeric bearings. It is assumed that the replacement superstructure would be of similar width and cross section as existing, with an F-Shape TL-5 barrier. If the structure were to be widened to increase the shoulder widths, the substructures would require significant modifications and the interstate may require realigning. For these reasons, a wider superstructure is only considered for the Bridge Replacement Alternative below. Additionally, surficial substructure repairs (patching and sealing cracks) would be included in this alternative. Traffic can be maintained by median crossovers or an off-site detour, as described in Section 4.

Advantages: This alternative addresses the deficiencies of the bridge and provides a new superstructure with a longer design life and seismic upgrades. The design life is limited by the remaining life of the substructure, which is estimated to be 50 years.

Disadvantages: This alternative has a higher cost and longer construction duration than the deck patching or replacement alternatives, and the bridge width would remain substandard. Additionally, the condition of the superstructure does not necessitate replacement.

3.6. Alternative 4: Bridge Replacement

This alternative would involve removing and replacing the bridge in its entirety on the same alignment. The new structure would be about 10-ft wider, to increase the shoulder at the low speed lane to 10-ft and the high speed lane to 4-ft, and to maintain both 2-ft offsets. This would require an additional girder and increased girder spacing. The new piers and abutments would be a similar type to existing and in the same locations, and wider to accommodate the wider superstructure. It is assumed that the existing steel H-piles could be reused, and that additional steel H-Piles would be installed to accommodate the widening. Traffic can be maintained by median crossovers or an off-site detour, as described in Section 4.

Advantages: This alternative provides a new structure with the longest design life, full shoulder widths that match the approaches, and inherent seismic robustness.

Disadvantages: This alternative has the highest cost and longest construction duration, thereby causing the greatest disturbance to the community. It would also have the greatest impact to biological resources, would likely impact the archaeologically sensitive areas. Additionally, the condition of the superstructure and substructure does not warrant replacement.

3.7. Seismic Retrofit Strategy

A seismic retrofit should be considered for this structure. Preliminary evaluation shows the piers to be very seismically vulnerable and likely to suffer significant damage in the event of a design earthquake. As seismic retrofits primarily affect the substructure they may be performed in conjunction with any of the preferred superstructure alternatives depending on the desired approach to the retrofit. Two areas of work that will affect the superstructure retrofit strategies are the disposition of the pin and hanger details and the bearings. Seismic retrofit strategies can be tailored to the preferred superstructure alternatives.

The Catcher's Mitt and Seated Girder retrofits will require additional longitudinal and lateral restraint be added to improve seismic resistance. This may be accomplished for the lateral direction with the addition of diaphragms and for the longitudinal direction with the addition of restrainer cables. The Continuous Splice retrofit is the most robust upgrade seismically as it removes the potential for a loss of support failure within the span.

Bearing replacement would eliminate the potential loss of stability failure to the high level fixed and rocker bearings. Replacement bearings can also be designed to isolate the superstructure from the substructure, changing the structure's period and reducing earthquake loads on the bridge substructure.

The bridge piers and pier foundations appear to be very seismically vulnerable and should be strengthened. From the preliminary analysis, this work could include construction of a reinforced concrete jacket around the stem, installation of additional piles, reinforcement of the stem to footing connection, and strengthening of the footings. Keeper blocks could be constructed at the abutments to restrain the structure transversely while reducing demands on the piers. The abutments could be upgraded to fully resist the seismic forces, or the potential damage to the abutments could be accepted as a tradeoff for reduced damage to the piers.

The overall seismic retrofit strategy would begin with tailoring the approach to the desired superstructure rehabilitation program and a complete seismic analysis. The analysis would identify the specific seismic vulnerabilities of the structure and the best remedies for retrofitting. The retrofit work can be complete or partial depending on the outcomes of the evaluation and acceptable levels of risk for the various components that may fail in an earthquake.

Advantages: Improved resistance to earthquakes reduces risk for loss of the structure in the event of a design earthquake and reduced damage in lower level earthquakes. Once a complete evaluation is performed during design, a more comprehensive retrofit program can be developed and decisions made as to extent of the work compared to risk of failure.

Disadvantages: Depending on the results of the analysis and the desired retrofit program the work may be relatively simple and inexpensive to very complex and costly. As a minimum the structure should be analyzed so that risks may be specifically identified and more informed decisions may be made as to what deficiencies to address. A major disadvantage to not analyzing the structure seismically is all outcomes remain unknown.

4. MAINTENANCE OF TRAFFIC

The Vermont Agency of Transportation reviews each new project to determine suitability for the Accelerated Bridge Program, which focuses on faster delivery of construction plans, permitting, and Right of Way, as well as faster construction of projects in the field. One practice that will help in this endeavor is closing bridges for portions of the construction period, rather than providing temporary bridges. In addition to saving money, the intention is to minimize the closure period with faster construction techniques and incentives to contractors to complete projects sooner. The Agency will consider the closure option on most projects where rapid reconstruction or rehabilitation is feasible. The use of prefabricated elements in new bridges will also expedite construction schedules. This can apply to decks, superstructures, and substructures. Accelerated Construction should provide enhanced safety for the workers and the travelling public while maintaining project quality.

In order to construct the project, traffic will need to be maintained. The following options have been considered:

4.1. Option 1: Off-Site Detour

This option would close the bridges to traffic during construction and reroute traffic from I-91 to US Route 5 and back to I-91. Between I-91 Exit 6 and Exit 7, US Route 5 would be the obvious detour route as the roadway parallels the existing I-91 alignment. The detour has an end-to-end distance of approximately 8.5 miles, takes 12 minutes to drive, and has an additional travel distance of 1.4 miles. A map of the detour routes, northbound and southbound, can be found in the Appendix.

	I-91 Northbound Detour Route	I-91 Southbound Detour Route
Through Route	7.4 miles	6.7 miles
Detour Route	8.8 miles	8.2 miles
Added Miles	1.4 miles	1.5 miles
Detour Travel Time	12 minutes	12 minutes

The detour would substantially increase the volume of traffic on Route 5 for the duration of the project. An increase in traffic along the detour route poses several concerns related to safety and roadway capacity.

There are two locations along the US Route 5 detour route that are listed on the High Crash Location List, one of which is the US5/VT131 intersection in Weathersfield that ranks 39 of 50 for high crash locations in Vermont. Traffic signal timing, intersection capacity, and turning radii for the high percentage of truck traffic that would utilize this detour during construction would need to be addressed prior to implementation of this detour. Additionally, the community has expressed concern for safety and congestion if all I-91 traffic were to be detoured onto US-5 for this project.

The US Route 5 detour route includes two bridges with reduced or non-existent shoulders. Detoured traffic including trucks traveling at reduced speeds over these bridges would be subject to delay at these locations and pose a concern for safety.

It is also noted that existing public transit service along I-91 by The Current (Southeast Vermont Transit, formerly Connecticut River Transit) would be impacted by the detour.

Advantages: Utilizing an off-site detour would eliminate the need to use phase construction, use a temporary bridge, or construct median crossover roadways to maintain traffic. This option would decrease the cost and amount of time required to construct a project in this location. The impacts required to construct a project in this location would also be reduced for this option. Many times by decreasing the impacts, the length of time to develop the project can be decreased. The safety of construction workers will be improved by removing traffic from the construction site.

Disadvantages: Traffic flow would not be maintained through the project site during construction resulting in a disruption to Interstate I-91 regional and long haul traffic. Added traffic volume including heavy truck traffic would be directed to use US Route 5 and will impact the operations of the at grade intersections and the I-91 Exit 7 Park & Ride, residential, commercial, and retail business driveways on US Route 5 along the detour route. There are also safety concerns since there are two high crash locations along the detour route. Roadway and intersection improvements on the detour route that are beyond the scope of this project would be a prerequisite to implementing the detour. Public transit that uses I-91 would be impacted and the local community has expressed potential safety issues and traffic concerns with an off-site detour.

4.2. Option 2: Phased Construction

Phased construction is the maintenance of traffic on the bridge while rehabilitating or replacing the structure one lane at a time. This allows the road to remain open during construction and minimizes impacts to adjacent property and environmental resources. While the time required to develop a phased construction project would remain the same, the time required to complete a phased construction project increases because some of the construction tasks have to be performed multiple times. In addition to the increased design and construction costs mentioned above, the costs also increase for phased construction because of the inconvenience of working around traffic and the effort involved in coordinating the joints between the phases. Another negative aspect of phased construction is the decreased safety of the workers and vehicular traffic, which is caused by increasing the proximity and extending the duration that workers and moving vehicles are operating in the same confined space. Phased construction is usually considered when the benefits include reduced impacts to resources and decreased costs and development time by not requiring the purchase of additional ROW.

Based on the current traffic volumes, it is acceptable to close one lane of traffic, and maintain one lane of traffic in both the northbound and southbound directions. Additionally, based on the existing bridge widths, it is possible to phase traffic without widening the bridge beyond the standard or shifting the horizontal alignment. However, if the project consists of deck patching and pin and hanger retrofits that require the deck to be removed above the steel repair (see Section 2.3), the resulting lane width will be temporarily reduced to 11-ft and this will require that all oversized and wide load truck traffic utilize an off-site detour. See the Appendix for the recommended phasing layout plans.

Advantages: Traffic flow would be maintained through the project site during construction. Also, this option would have minimal impacts to the existing median and land beyond the limits of the existing paved surface.

Disadvantages: Costs could be higher and construction duration could be longer since many construction activities in this case would have to be performed twice. Additionally, since cars are traveling near construction activity, there is decreased safety. There would be some delays and disruption to traffic since the roadways would be reduced to a single lane. Also, depending on the chosen repair alternative, there is the potential that all oversized and wide load truck traffic would be required to utilize the US Route 5 off-site detour between I-91 Exit 6 and Exit 7, and the local community has expressed potential safety issues and traffic concerns with an off-site detour.

4.3. Option 3: Temporary Bridge

A temporary bridge would allow the existing bridges to be closed to traffic without the need to divert traffic to an off-site detour. A temporary bridge would be placed between the two bridges within the existing median. Additional costs would be incurred to use a temporary bridge including the cost of the bridge itself, installation and removal, and restoration of the disturbed area.

Additional studies would be triggered by potential impacts to sensitive areas within the median at the crossing over the Black River.

Advantages: Traffic flow can be maintained along the I-91 corridor. Lane reductions would not be required.

Disadvantages: This traffic control option would be costly and time consuming as construction activities would take an additional construction season to set up the temporary bridge. This option would have adverse environmental impacts to and along the banks of the Black River.

This option for traffic management is a major construction project by itself. Since a detour route is available, and the traffic volumes are such that median crossovers (described below in Section 4.4) are feasible, a temporary bridge is not reasonable and will not be developed further in this report.

4.4. Option 4: Median Crossovers

The implementation of temporary median crossover detour roadways would allow the project to be constructed with the bridges closed, one at a time. The travel lanes on both I-91 northbound and I-91 southbound would be reduced from two lanes to one lane each and allow the opposing direction to crossover the median and use the opposite bridge to cross the Black River. Based on the current traffic volumes, it is acceptable to close one lane of traffic and maintain a single travel lane of traffic in both the northbound and southbound directions. This project location is good fit for crossovers since there are no obstructions in the median, no tree clearing will be required, and the elevation change between the northbound and southbound barrels is minimal. The temporary crossover roadway earthwork in the median can be left in place for future use if the pavement is removed and replaced with 5" of topsoil and seeded.

To the north and to the south of the project site, I-91 bridges over US Route 5 and the Toonerville Rail Trail present a constraint to the layout of temporary median crossover roadways, however, it is expected that the single lane crossover roadways would meet a 55mph design speed without impacting them. The crossovers would be located immediately to the north and south of the project bridges. Full access to and from Exit 7 on the north side of the project would be maintained. It should be noted that if median crossovers are in place over the winter season, a minimum width of 14-ft is required. The existing bridge geometry can provide a maximum width of 14-ft curb-to-curb.

Advantages: Utilizing median crossover detour roadways would eliminate the need to use a temporary bridge or phase construction to maintain traffic. The work zone would be safer, as compared to phased construction. Traffic flow would be maintained through the project site during construction. The remaining median crossover earthwork would provide immediate and future cost savings since the roadways would not be removed and would be available for future maintenance or rehabilitation of these bridges or other work in the vicinity.

Disadvantages: There would be some delay and disruption to traffic, since the roadways would be reduced to a single lane. There would be added cost to construct the median crossover roadways and to remove them after use.

5. ALTERNATIVE SUMMARY

- **Alternative 1**: Catcher's Mitt Retrofit
  Traffic maintenance only required to access the work below the deck

- **Alternative 2a**: Deck Replacement, F-Shape Barrier, Seated Girder Retrofit
  Traffic maintained by median crossovers

- **Alternative 2b**: Deck Replacement, F-Shape Barrier, Continuous Splice Retrofit and Bearing Replacement
  Traffic maintained by median crossovers

- **Alternative 3**: Superstructure Replacement
  Traffic maintained by median crossovers

- **Alternative 4**: Bridge Replacement
  Traffic maintained by median crossovers

6. COST MATRIX[1]

Springfield I-91 26N/S(IM 091-1(74))		Do Nothing	Alt 1	Alt 2a	Alt 2b	Alt 3	Alt 4
			Catcher's Mitt	Deck Replacement		Superstructure Replacement	Bridge Replacement
				Seated Girder	*Bolted Splice*		
COST	Bridge Cost	$0	$450,000	$3,400,000	$4,880,000	$5,470,000	$9,770,000
	Clean & Paint Exist. Structure	$0	*included above*	$1,470,000	$1,470,000	$0	$0
	Removal of Structure	$0	$0	$280,000	$280,000	$2,230,000	$2,870,000
	Roadway	$0	$0	$250,000	$250,000	$300,000	$500,000
	Maintenance of Traffic	$0	$50,000	$900,000	$900,000	$900,000	$900,000
	Construction Costs	$0	$500,000	$6,300,000	$7,780,000	$8,900,000	$14,040,000
	Construction Engineering + Contingencies	$0	$150,000	$1,449,000	$1,893,000	$2,670,000	$4,212,000
	Total Construction Costs with CEC	$0	$650,000	$7,749,000	$9,673,000	$11,570,000	$18,252,000
	Preliminary Engineering[2]	$0	$65,000	$627,900	$820,300	$1,041,300	$1,642,680
	Right-of-Way	$0	$0	$0	$0	$0	$0
	Total Project Costs	**$0**	**$715,000**	**$8,376,900**	**$10,493,300**	**$12,611,300**	**$19,894,680**
	Total Project Costs w/o Clean & Paint	**$0**	**$715,000**	**$6,906,900**	**$9,023,300**	**$12,611,300**	**$19,894,680**
SCHEDULING	Project Development Duration[3]	NA	2 years	2 years	2 years	3 years	3 years
	Construction Duration	NA	8 months	24 months	24 months	30 months	36 months
	Closure Duration (If Applicable)	NA	NA	NA	NA	NA	NA
ENGINEERING	Typical Section - Roadway (feet)	4-12-12-10	4-12-12-10	4-12-12-10	4-12-12-10	4-12-12-10	4-12-12-10
	Typical Section - Bridge (feet)	3-12-12-3	3-12-12-3	4-12-12-4	4-12-12-4	4-12-12-4	4-12-12-10
	Geometric Design Criteria	Substandard Width	No Change Substandard Width	Improved Substandard Width	Improved Substandard Width	Improved Substandard Width	Improved
	Traffic Safety	No Change	No Change	Improved	Improved	Improved	Improved
	Alignment Change	No	No	No	No	No	No
	Bicycle Access	No Change	No Change	No Change	No Change	No Change	No Change
	Hydraulic Performance	Adequate	Adequate	Adequate	Adequate	Adequate	Adequate
	Pedestrian Access	No Change	No Change	No Change	No Change	No Change	No Change
	Utility	No Change	No Change	No Change	No Change	No Change	No Change
OTHER	ROW Acquisition	No	No	No	No	No	No
	Road Closure	No	No	No	No	No	No
	Design Life	< 10 years	15 years	40 years	40 years	50 years	80 years

[1]Costs are estimates only, used for comparison purposes.
[2]Preliminary Engineering costs are estimated starting from the end of the Project Definition Phase.
[3]Project Development Durations are starting from the end of the Project Definition Phase.

7. CONCLUSION

We recommend **Alternative 2b**; to replace the bridge deck and thereby replace the railing with a concrete F-Shape TL-5 barrier, replace the pin and hanger with a continuous splice, move the expansion joints to each end, replace the high-level bearings with steel-reinforced elastomeric pads, and perform surficial substructure repairs. Depending upon VTrans systemwide evaluation of available budget and needs, the rehabilitation of these structures could also be sequenced. The first phase could consist of the “Catcher’s Mitt” (Alternative 1) to eliminate the undesirable pin and hanger detail. Alternative 2b could then be implemented in years to follow.

We also recommend a detailed seismic evaluation be performed during the next design phase and a comprehensive seismic retrofit program be developed for the structure. A more informed decision may be made at that time as to the extent of the upgrades that should be performed.

<u>Structure:</u>

The selected pin and hanger retrofit will likely be decided by a system-wide evaluation of available budget and measured risk. The “Catcher’s Mitt” is the lower-end repair that will provide an immediate safety upgrade and will have the shortest project duration and cost, however the deck and substandard bridge railing will remain. The “Seated Girder” is the mid-end repair that eliminates the pin and hanger and replaced the deck, but maintains the suspended span configuration and the existing high level bearings. Lastly, the “Continuous Splice” is the high-end repair, that replaces the pin and hanger and provides seismic upgrades by replacing the high-level bearings and uniformly distributing the seismic loads to the substructures. This option will likely have a longer design life and require less maintenance than the “Seated Girder”.

<u>Traffic Control</u>:

Median crossovers are the preferred method of traffic maintenance. Median crossovers would avoid narrow 11-ft lanes during construction which would force wide loads and oversized vehicles to follow the US Route 5 detour route.

The AADT for I-91 northbound is 5,700 with over 16% trucks. The AADT for I-91 southbound is 5,700 with over 16% trucks. An off-site detour/closure of I-91 would result in interstate traffic on US Route 5 for the duration of the project causing delay along the detour route and traffic and safety concerns in the community. The detour route is undesirable for this volume of traffic and the high percentage of trucks, thus this option is not appropriate for this project. The temporary bridge option would allow I-91 to maintain 2 lanes in both directions but brings added environmental impacts, costs, and an increased project duration. This option is not appropriate for this project.

APPENDIX

8.1. Site Photographs

Bridge 26N, Facing North

Bridge 26N, Facing South

Underside of Bridge 26N, from Abutment #2

From underside of Bridge 26N Abutment #2, facing Bridge 26S

Face of Abutment #2 of Bridge 26N

South Face of Pier #1, from Abutment #1 of Bridge 26S

Bridge 26N Hinge

Deck Condition at Expansion Finger Joint, Located above Hinges (Bridge 26S shown)

Pin Condition (Bridge 26S shown)

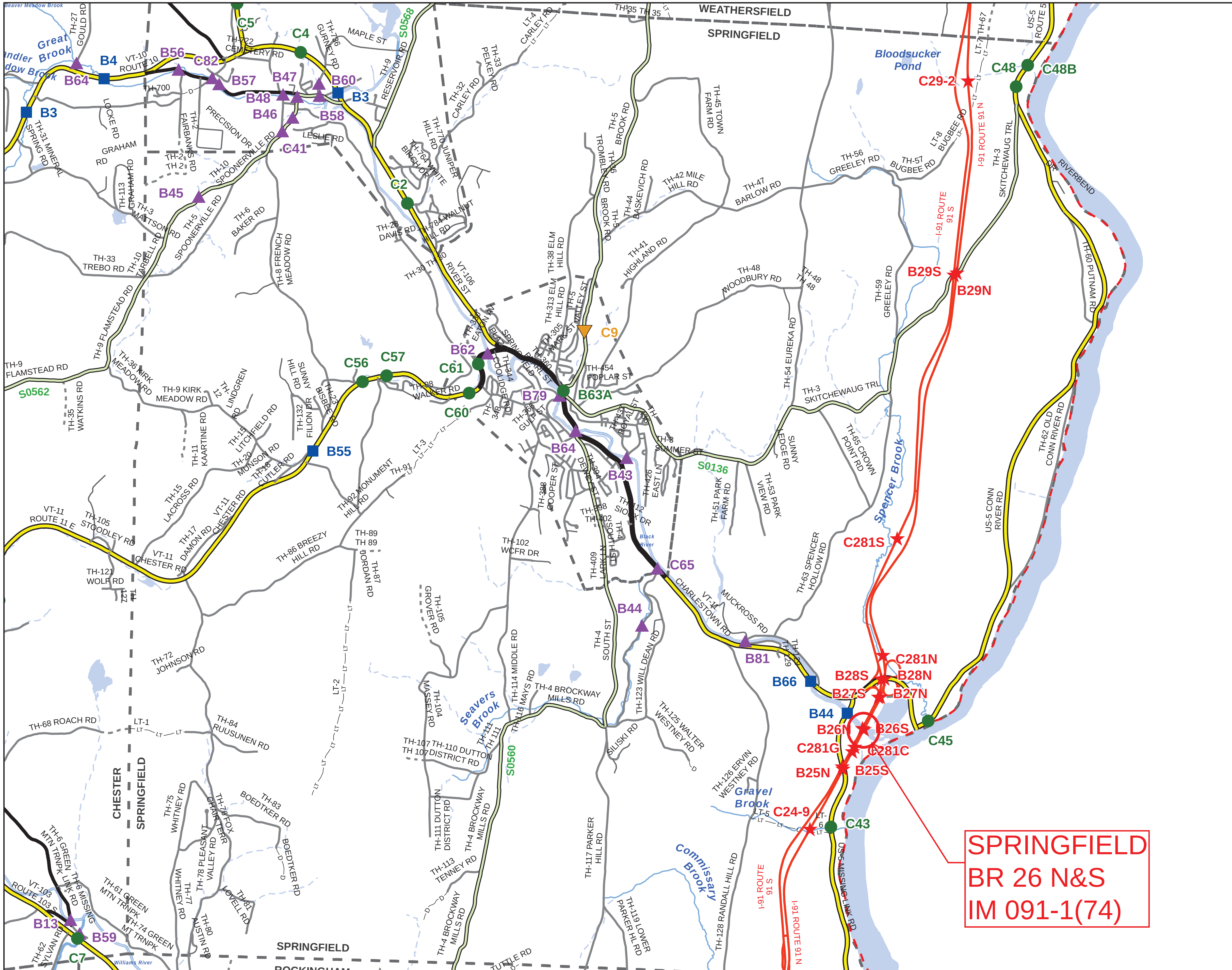
Deck Condition above Pin (Bridge 26S shown)

Bridge Curb Deterioration (Bridge 26S shown)

Bridge Curb Deterioration (Bridge 26S shown)

APPENDIX

8.2. Town Map and Bridge Location

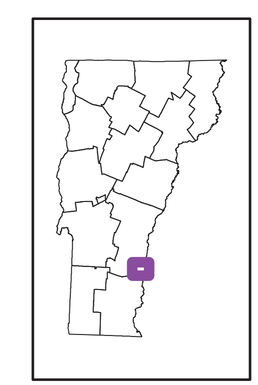


Scale 1:47,965

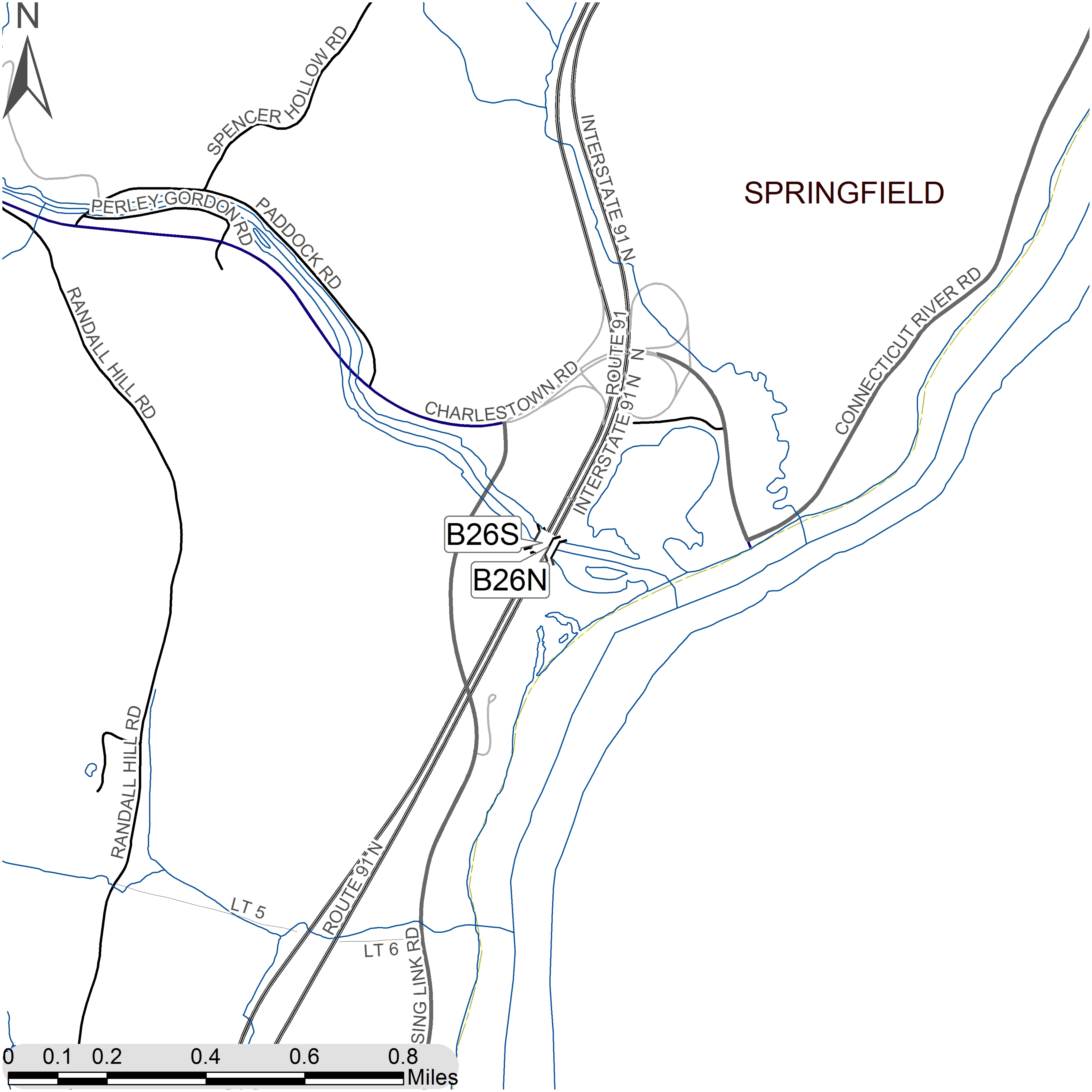
N

- INTERSTATE
- STATE LONG
- STATE SHORT
- TOWN LONG
- FAS/FAU
- FAS/FAU HWY
- INTERSTATE
- STATE HIGHWAY
- CLASS 1
- CLASS 2
- CLASS 3
- CLASS 4
- LEGAL TRAIL
- PRIVATE
- DISCONTINUED
- DISTRICT
- POLITICAL BOUNDARY
- NAMED RIVERS-STREAMS
- UNNAMED RIVERS-STREAMS

Produced by:
Mapping Unit
Vermont Agency of Transportation
August 2011



SPRINGFIELD
WINDSOR COUNTY
DISTRICT # 2


N
SPRINGFIELD
SPENCER HOLLOW RD
PERLEY GORDON RD
PADDOCK RD
RANDALL HILL RD
CHARLESTOWN RD
INTERSTATE 91 N
ROUTE 91 N
INTERSTATE 91 N
CONNECTICUT RIVER RD
B26S
B26N
RANDALL HILL RD
LT 5
LT 6
ROUTE 91 N
SING LINK RD
0
0.1
0.2
0.4
0.6
0.8
Miles

APPENDIX

8.3. Bridge Inspection Reports

STRUCTURE INSPECTION, INVENTORY and APPRAISAL SHEET

Vermont Agency of Transportation ~ Structures Section ~ Bridge Management and Inspection Unit

Inspection Report for SPRINGFIELD | *bridge no.:* 0026N | *District:* 2
Located on: I 00091 ML *ove* BLACK RIVER | *approximately* 0.4 MI S EXIT 7 | *Owner:* 01 STATE-OWNED

CONDITION

Deck Rating: 6 SATISFACTORY
Superstructure Rating: 7 GOOD
Substructure Rating: 8 VERY GOOD
Channel Rating: 7 GOOD
Culvert Rating: N NOT APPLICABLE
Federal Str. Number: 200091026N14182
Federal Sufficiency Rating: 077.1
Deficiency Status of Structure: ND

AGE and SERVICE

Year Built: 1965 *Year Reconstructed:* 0000
Service On: 1 HIGHWAY
Service Under: 5 WATERWAY
Lanes On the Structure: 02
Lanes Under the Structure: 00
Bypass, Detour Length (miles): 01
ADT: 005500 *% Truck ADT:* 13
Year of ADT: 1998

GEOMETRIC DATA

Length of Maximum Span (ft): 0130
Structure Length (ft): 000316
Lt Curb/Sidewalk Width (ft): 1
Rt Curb/Sidewalk Width (ft): 1
Bridge Rdwy Width Curb-to-Curb (ft): 30
Deck Width Out-to-Out (ft): 35.2
Appr. Roadway Width (ft): 038
Skew: 00
Bridge Median: 1 OPEN MEDIAN
Min Vertical Clr Over (ft): 99 FT 99 IN
Feature Under: FEATURE NOT A HIGHWAY OR RAILROAD
Min Vertical Underclr (ft): 00 FT 00 IN

STRUCTURE TYPE and MATERIALS

Bridge Type: 3 SP CONT WLD GIRDER
Number of Approach Spans 0000 *Number of Main Spans:* 003
Kind of Material and/or Design: 4 STEEL CONTINUOUS
Deck Structure Type: 1 CONCRETE CIP
Type of Wearing Surface: 6 BITUMINOUS
Type of Membrane 2 PREFORMED FABRIC
Deck Protection: 0 NONE

APPRAISAL *AS COMPARED TO FEDERAL STANDARDS

Bridge Railings: 0 DOES NOT MEET CURRENT STANDARD
Transitions: 0 DOES NOT MEET CURRENT STANDARD
Approach Guardrail 1 MEETS CURRENT STANDARD
Approach Guardrail Ends: 1 MEETS CURRENT STANDARD
Structural Evaluation: 7 BETTER THAN MINIMUM CRITERIA
Deck Geometry: 4 MEETS MINIMUM TOLERABLE CRITERIA
Underclearances Vertical and Horizontal: N NOT APPLICABLE

Waterway Adequacy: 8 SLIGHT CHANCE OF OVERTOPPING ROADWAY

Approach Roadway Alignment: 8 EQUAL TO DESIRABLE CRITERIA

Scour Critical Bridges: 8 STABLE FOR SCOUR

DESIGN VEHICLE, RATING, and POSTING

Load Rating Method (Inv): 1 LOAD FACTOR (LF)
Posting Status: A OPEN, NO RESTRICTION
Bridge Posting: 5 NO POSTING REQUIRED
Load Posting: 10 NO LOAD POSTING SIGNS ARE NEEDED
Posted Vehicle: POSTING NOT REQUIRED
Posted Weight (tons):
Design Load: 5 HS 20

INSPECTION and CROSS REFERENCE

X-Ref. Route:
Insp. Date: 062014 *Insp. Freq. (months)* 24 *X-Ref. BrNum:*

INSPECTION SUMMARY and NEEDS

1/9/2015 The full depth hole on the north end of the suspended section in span #4 on the downstream sideof the joint and in the curb have been repaired and is in good condition. ~FRE/MJK

6/18/2014 The full depth hole on the north end of the suspended section at the face of the curb should be repaired soon to stop the water and sand form dripping on to the pin and plates. Beams should be spot cleaned and painted. ~FRE/TJB

6/8/2012 Structure is in good condition however the curbs should be cleaned and patched. Debris at pier #1 should be removed. ~FRE/SJH

04/12/2010 Right bridge rail along span No.1 is in need of repairs. Large concrete cavities along both curb areas are in need of repairs. The joint areas above both pin and hanger locations of the suspended span are in need of major repairs to arrest ongoing corrosion activities of the girder end areas. Servi-lift inspection performed on 07/15/2010. PLB

STRUCTURE INSPECTION, INVENTORY and APPRAISAL SHEET

Vermont Agency of Transportation ~ Structures Section ~ Bridge Management and Inspection Unit

Inspection Report for SPRINGFIELD **bridge no.:** 0026S **District:** 2

Located on: I 00091 ML **ove** BLACK RIVER **approximately** 0.4 MI S EXIT 7 **Owner:** 01 STATE-OWNED

CONDITION

Deck Rating: 5 FAIR
Superstructure Rating: 5 FAIR
Substructure Rating: 7 GOOD
Channel Rating: 8 VERY GOOD
Culvert Rating: N NOT APPLICABLE
Federal Str. Number: 200091026S14182
Federal Sufficiency Rating: 065
Deficiency Status of Structure: ND

AGE and SERVICE

Year Built: 1965 Year Reconstructed: 0000
Service On: 1 HIGHWAY
Service Under: 5 WATERWAY
Lanes On the Structure: 02
Lanes Under the Structure: 00
Bypass, Detour Length (miles): 01
ADT: 005500 % Truck ADT: 13
Year of ADT: 1998

GEOMETRIC DATA

Length of Maximum Span (ft): 0130
Structure Length (ft): 000316
Lt Curb/Sidewalk Width (ft): 1
Rt Curb/Sidewalk Width (ft): 1
Bridge Rdwy Width Curb-to-Curb (ft): 30
Deck Width Out-to-Out (ft): 35.2
Appr. Roadway Width (ft): 038
Skew: 00
Bridge Median: 1 OPEN MEDIAN
Min Vertical Clr Over (ft): 99 FT 99 IN
Feature Under: FEATURE NOT A HIGHWAY OR RAILROAD
Min Vertical Underclr (ft): 00 FT 00 IN

STRUCTURE TYPE and MATERIALS

Bridge Type: 3 SP CONT WLD GIRDER
Number of Approach Spans 0000 Number of Main Spans: 003
Kind of Material and/or Design: 4 STEEL CONTINUOUS
Deck Structure Type: 1 CONCRETE CIP
Type of Wearing Surface: 6 BITUMINOUS
Type of Membrane 2 PREFORMED FABRIC
Deck Protection: 0 NONE

APPRAISAL *AS COMPARED TO FEDERAL STANDARDS

Bridge Railings: 0 DOES NOT MEET CURRENT STANDARD
Transitions: 0 DOES NOT MEET CURRENT STANDARD
Approach Guardrail 1 MEETS CURRENT STANDARD
Approach Guardrail Ends: 1 MEETS CURRENT STANDARD
Structural Evaluation: 5 BETTER THAN MINIMUM TOLERABLE CRITERIA
Deck Geometry: 4 MEETS MINIMUM TOLERABLE CRITERIA
Underclearances Vertical and Horizontal: N NOT APPLICABLE
Waterway Adequacy: 8 SLIGHT CHANCE OF OVERTOPPING ROADWAY
Approach Roadway Alignment: 8 EQUAL TO DESIRABLE CRITERIA
Scour Critical Bridges: 8 STABLE FOR SCOUR

DESIGN VEHICLE, RATING, and POSTING

Load Rating Method (Inv): 1 LOAD FACTOR (LF)
Posting Status: A OPEN, NO RESTRICTION
Bridge Posting: 5 NO POSTING REQUIRED
Load Posting: 10 NO LOAD POSTING SIGNS ARE NEEDED
Posted Vehicle: POSTING NOT REQUIRED
Posted Weight (tons):
Design Load: 5 HS 20

INSPECTION and CROSS REFERENCE

Insp. Date: 062014 Insp. Freq. (months) 24
X-Ref. Route:
X-Ref. BrNum:

INSPECTION SUMMARY and NEEDS

6/18/2014 Beams should be spot cleaned and painted . Curbs should be cleaned and patched. Tear drop rail on the upstream side at midspan should be repaired. ~FRE/TJB

6/8/2012 Curbs should be cleaned and patched. Structure should be spot cleaned and painted. Joint areas in the soffit should be cleaned and patched. ~FRE/SJH

04/12/10 Left bridge rail of span No.3 is in need of repairs. Large concrete cavities along both curb areas are in need of repairs. The joint areas above both pin and hanger locations of the suspended span are in need of major repairs to arrest ongoing corrosion activities of the girder end areas. Servi-lift inspection performed on 07/15/2010. PLB

APPENDIX

8.4. Deck Core Results

AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Gary Sweeny, Structures

From: Jim Wild, Structural Concrete Engineer

Date: December 13, 2016

Subject: Springfield IM 091-1(74) Bridge 26N

1. INTRODUCTION

On May 27, 2016 Jonathan Griffin from Structures contacted Jim Wild, Structural Concrete Engineer, to inquire if the Structural Concrete Unit would be able to analyze the deck concrete from bridge 26N on I91 as part of the scoping project for this bridge. The proposed project scoping consists of either removing and replacing the curbs and deck overhangs or completely replacing the deck.

Bridge No. 26N is located on Interstate 91 North bound south of Exit 7 and crosses over the Black River in Springfield, Vermont. The bridge was built in 1965 by Perini Corp. of Framingham Mass. The concrete came from Charleston Redimix Inc. in Charlestown New Hampshire. The bridge is approximately 310 feet long, with the low end of the bridge at the north end. According to the original design plans the deck was to be 7.5” thick with 1.5” of cover over the top mat and 1.125” clear on the bottom.

Analysis of the concrete deck included taking concrete core samples to determine compressive strength, and concrete powder samples for use in determining concentration and depth of chloride penetration. In addition, both chemical and petrographic analysis were performed on core samples to determine the presence and severity of alkali-silica reaction (ASR).
Contained herein are the results of this field sampling and laboratory analyses, followed by a summary of findings and final recommendation.

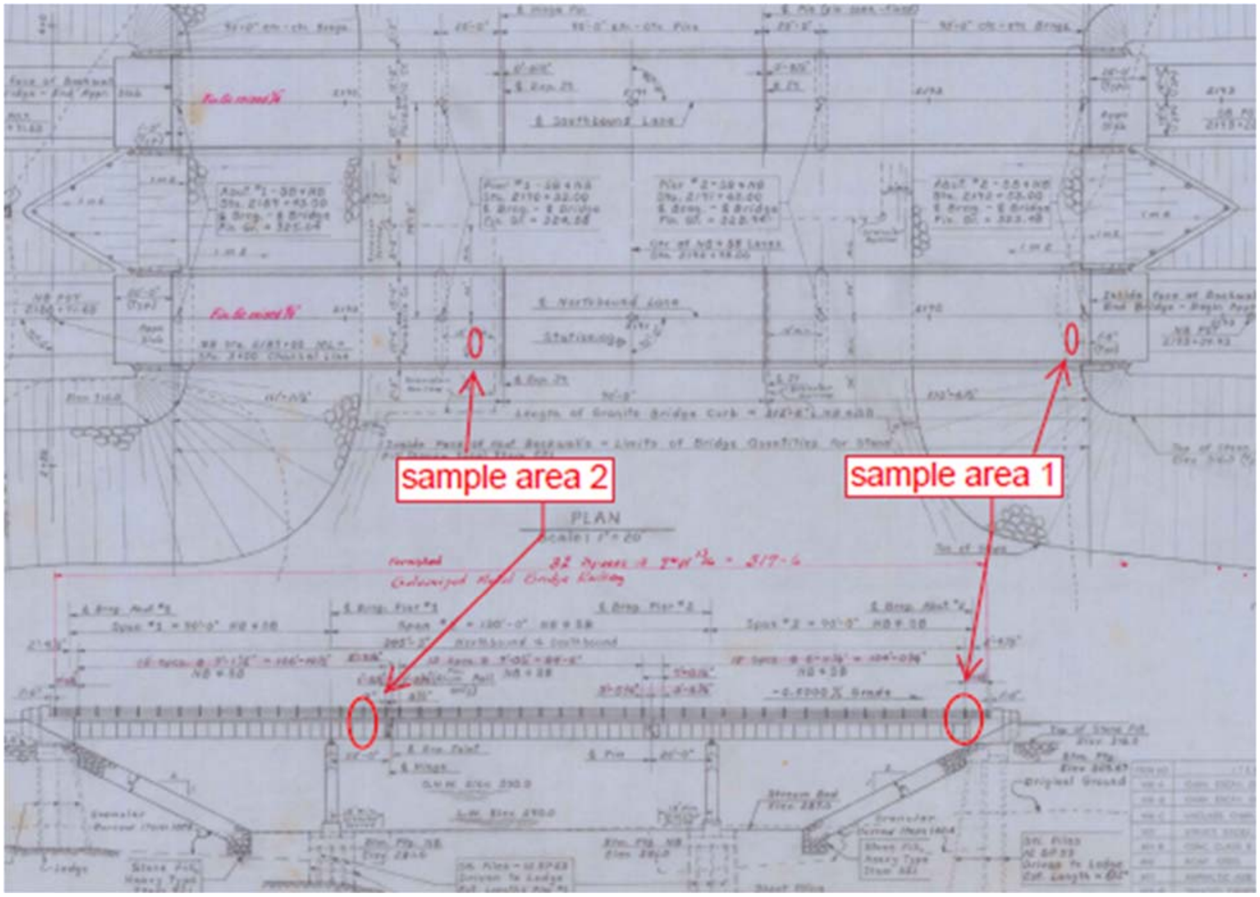
2. FIELD SAMPLING AND OBSERVATIONS

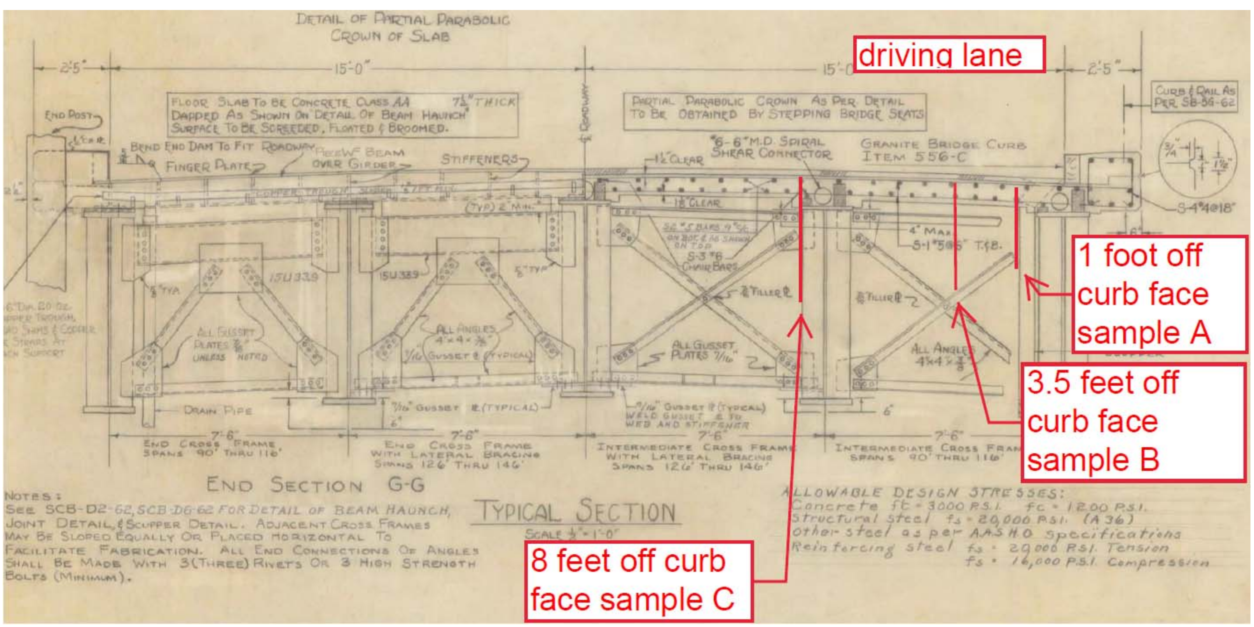
The field sampling was conducted on July 19, 2016. The final sampling location plan is as follows;

Location #1. 6 feet from the end of the deck on the low (North) end of the bridge
Location #2. 208 feet from the end of the deck on the low (North) end of the bridge

Sample area 1 was chosen because it is at the low end of the bridge deck which should see the longest duration of brine runoff. The second sample area was chosen as it would be approximately 8 feet south of the hinge joint which should see the longest duration of brine runoff for the southern span.

The following plans show approximate sample locations.





Core and chloride sample locations for BR 26N were measured perpendicular to the granite curb face. Each sample area had cores and chloride samples taken at 1 foot, 3.5 feet, and 8 feet perpendicular to the face of the East curb, noted as ‘Sample A’, ‘Sample B’ and ‘Sample C’ in Image #2 above. Ground Penetrating Radar (GPR) was used for each sample location to locate the rebar grid to avoid coring into rebar. One core was taken per AASHTO T24 from each location (A, B, and C) along with chloride samples at each location. Cores of 4" diameter were extracted for compressive strength and ASR evaluation. The Vtrans Drilling unit assisted in coring by using their trailer mounted core rig. The deck was cored to a depth of approximately 6.5" in hopes that the cores would break off at 6" to 5.5" in length. Samples of concrete dust were extracted for chloride concentration testing using a Hilti drill with 1" diameter bit. Dust samples were obtained at five ½" depth increments representing a total depth of 2.5 inches from the deck surface.

In 1989 there was a deck overlay project that was performed by The Bridge Construction Corp. of Augusta ME. The record plans were reviewed from this project. General plan notes say to use concrete Class AA for Class II repair at two inches of depth. Bridge specific notes indicate there were very high corrosion readings for this deck. Final quantity of Class II repair was 987.22 square yards which is roughly 95% of the deck surface. They also replaced over 2000 linear feet of rebar.

Coring was done to a depth of approximately 6 to 6.5 inches. The recovered length of the cores were 4 to 5 inches in length which was approximately the interface of the 3/8" overlay concrete to the remaining original concrete. This depth is well below the two inches described in the General plan notes to remove the existing concrete to. There were a few cores in which some of the original concrete remained attached to the bottom of the core, allowing visual identification of this interface between the 3/8" concrete overlay to the original ¾" mix. This would also indicate that the interface of the overlay to original concrete has a good bond. In the appendix there is a picture showing the 3/8" to ¾" concrete interface.

Unfortunately, because the extracted concrete is of the 3/8" overlay mix, the condition of the remaining original concrete is unknown. The plan set also indicated that a sheet membrane was to be put down then paved over. There was a lack of membrane adhesion, or lack of membrane, on the top of some of the cores indicating that the membrane may be failing which could possibly allow the intrusion of water and chlorides under it. All cores were found to have no visual indication of ASR.

Below are the locations of where the cores were extracted from:

Table 1: Sample Core Locations BR26N Area 1

Core number	Station (ft)*	Offset (ft)**	Core length (in)***
1A	6	1	4
1B	6	3.5	5
1C	6	8	5

* Station is measured from the North end of bridge, parallel to the curb face, increasing to the South
** The offset is measured perpendicular from face of the East curb to the sample location.
*** Length prior to preparation for capping

Table 2: Sample Core Locations BR26N Area 2

Core number	Station (ft)*	Offset (ft)**	Core length (in)***
2A	208	1	5.25
2B	208	3.5	5.25
2C	208	8	5.5

Chloride samples were taken at the same offset as the cores and approximately a foot added to the south of the core station length.

3. LABORATORY TESTING

The core samples were tested for compressive strength per AASHTO T24. Core samples obtained from the field are cut in the lab to produce cylinder ends that are flat and parallel. Petrographic analysis, ASTM 295-98, for ASR was performed on thin sections obtained from the core end cutoffs. After the cores were tested for compression strength, pieces of the cores were given to the VAOT Chemist to undergo uranyl acetate screening for ASR per AASHTO T299. The chloride penetration profile was done in accordance with ASTM C1218.

3.1 COMPRESSIVE STRENGTH (AASHTO T24). After the core ends are cut, the cores are capped with Sulphur in order to create a bearing surface that will evenly distribute the compressive stress. AASHTO T24 requires cores to be a minimum of 1:1 to 2.1:1 ratio of length to diameter after capping for compression testing. Also no reinforcing steel should be in the cut core. If a core does have rebar in it perpendicular to the axis, it is up to the specifier whether to use it or not and to determine the effect on the strength results. All core samples met the requirements of AASHTO T 24 and with no rebar present.

Table 3: Sample Core Strength BR26N Area 1

Core number	Station (ft)*	Offset (ft)**	Corrected Strength (psi)
1A	6	1	N/A***
1B	6	3.5	7460
1C	6	8	6830

* Station is measured from the North end of bridge parallel to the curb face, increasing to the South
** The offset is measured perpendicular from face of the East curb to the sample location.
***N/A – core length after capping was too short. The uncorrected PSI was 8543psi

Table 4: Sample Core Strength BR26N Area 2

Core number	Station (ft)*	Offset (ft)**	Corrected Strength (psi)
2A	208	1	2690
2B	208	3.5	7340
2C	208	8	7560

3.2 CHLORIDE PROFILE (ASTM C1218). Chloride samples were taken at each core location approximately a foot down station as measured parallel to the curb. Five samples at ½” depth increments were taken from each location to a total approximate depth of 2.5”.

The concentration of chlorides required to initiate corrosion of steel is influenced by many variables. Some of which are pH of the concrete, the depth of carbonation, water to cement ratio, the amount of cement in the concrete along with several other variables. New concrete may take 7000 – 8000 parts per million (PPM) where old concrete could be as low as 100 PPM (PCA – Corrosion of Embedded Metals). Other research has shown that approximately 0.15% for water-soluble chlorides by mass of cement at the steel surface could initiate corrosion (Whiting – 1997). This would translate to approximately 260 PPM, assuming 660 lbs/cy of cement and a cubic yard of concrete weight of 3800 pounds. We have chosen 350 PPM as the threshold value where we would consider corrosion to initiate. This may be a conservative value but due to the amount of variables and unknown values of those variables with older concrete and typically shallower cover depths over the rebar, we believe this is a realistic value.

Five of the samples had chloride concentrations below the threshold value at the 1.5” to 2” depths. Sample location 1B had very high chloride concentration readings ranging from 2900 PPM in the first half inch to 2500 PPM in the 2 to 2.5” depth. There was very little reduction of concentration per depth. The chloride concentrations for the depths of each sample are not what would be typically expected. Typically, there would be an 80-150 PPM decrease per 0.5” of depth. The chloride results for this deck showed virtually no decrease in values from the 0.5” depth and down. One possible reason is the 3/8” is very dense and has inhibited chloride migration and the values recorded are the background values in the concrete

Based on the results of the samples collected there is no concern of chloride induced corrosion of the rebar, with the exception of sample location 1B which had over 7 times the threshold value.

3.3 ASR SCREENING (AASHTO T299). ASR screening was done on pieces of the cores after they were tested for compression strength. The cores were lightly wrapped with plastic in order to keep the broken pieces in approximately their correct original orientation. The analysis of the screening had 1 core rated high, 1 at moderate, 1 at low and 3 at very low.

3.4 PETROGRAPHIC ANALYSIS FOR ASR ASTM 295-98). The thin section petrographic analysis was performed on core specimen cut-offs recovered from concrete cut from each end of the compression cores prior to capping. Due to the short recovery lengths of the cores there was no opportunity on some cores to get a cut end section large enough to get a thin section sample from. The petrographic analysis was completed on two of the cores to confirm the ASR activity ratings from the screening.

Thin section C160365T and C160365B, sampled from location 2B, had no sign of ASR activity in the thin section. This correlates well with the ASR screening which had this core rated as very low with no visible reaction.

There were observed gel filled cracks in sample C160361B, sampled from location 1B. The chemist's screening rated this core as high for ASR Activity. There is strained quartz in the concrete aggregate that is contributing the silica for the alkalis to react with. Research has indicated that the reaction of strained quartz is slow reacting and could take up to 20 or more years before showing deleterious effects (Jensen V. – 1993). The age of the overlay is 27 years old. There is a high concentration of ASR gel but due to the slow reaction, it has taken a number of years for this concentration to build up.

The presence of strained quartz in the samples tested appears to be inconsistent. Some samples had aggregate containing strained quartz while others had none present. The petrographic analysis indicates there could be a possible blend of gravel and crushed stone coarse aggregates in the mix which would be a reason as to why some areas show no ASR activity and others are showing high ASR activity. The gravel source may contain the strained quartz.

4. SUMMARY

Based on the visual observation of the cores after they were extracted there appears to have been a rehabilitation job in which the top 4-5 inches was removed and a 3/8" inch concrete mix overlay was put down. All cores at both sample areas showed this characteristic. Further research into the history of the bridge confirms that there was a deck overlay project in 1989. The concrete analysis was performed on this overlay 3/8" concrete mix as these were the only sections of cores that were recoverable.

The compressive strengths for all the cores, except core 1A, ranged from 2690 psi to 7560 psi. Core 1A was too short after capping but was still broken to get an uncorrected 8543 psi. Core 1B also had an uncorrected strength of 8543 psi with a corrected strength or 7460 psi. Core 1B was roughly 0.1 inch taller than core 1A. Assuming the 0.1" of length difference is not significant it would be reasonable to estimate that the strength of core 1A would be greater than 7000 psi. Excluding the low compressive strength result of core 2A, the remaining cores had a range of 730 psi which is in reason of cast cylinders. Record plans indicate the deck was originally designed for 3000 psi design strength. All the overlay cores, excluding core 2A, exceeded the 3000 psi design strength.

Chloride results at the 1.5” to 2” depths, which is approximately where the first mat of steel is located, are below the 350 PPM threshold for initiation of corrosion of the reinforcing, excluding sample location 1B in which the chloride concentration was 7 times the threshold value at the 1.5” to 2” depth.

The concrete in all core locations was sound.

Membrane adhesion, if there was a membrane, was poor for some of the core locations. If chloride rich water is able to get between the membrane or asphalt and the concrete surface interface, this could accelerate concrete deterioration quickly by letting the chloride solution pond on the concrete.

The majority of the concrete is experiencing very little ASR distress according to the samples represented here. The cores were visually examined after they were extracted and there were no visible ASR signs. The VAOT Chemist performed ASR screening and rated the ASR activity in one core as high, one core was moderate, one core low and three cores very low. Petrographic analysis was performed by the VAOT Geologist on two selected cores, C160361 and C160365. The petrographic analysis confirmed the presence of gel filled cracks in the one core rated as high, C160361, from the screening process. The second core, C160365, did not have ASR activity which correlates to the ASR screening by the VAOT Chemist. The original concrete in the lower portion of the deck is unknown. It could be assumed that there is ASR activity in the original concrete as that has been the case for other decks of the same vintage that have been tested in the area.

It was observed that the adhesion of the membrane, or lack thereof, to the concrete surface was inconsistent. At some core locations, the membrane/pavement would easily detach from the concrete yet in others it had good adhesion and had to be scraped off. This could be an indication that more areas on the deck are experiencing the same lack of adhesion.

CONCLUSION

The areas that were sampled had roughly 50 to 70% of the original depth removed and replaced with a 3/8” concrete overlay, assuming the overlay was replaced to the original deck thickness. Record plan as built quantities indicate 95% of the deck surface area had Class II repair done to it. The interface of the overlay to the original concrete seemed to provide a good bond. The core strengths are in excess of design 3000 psi strength, except for core 2A which was 90% of design strength. The deck surface appears to still be competent. Excluding sample location 1B, there is no concern of chloride induced corrosion of the reinforcing in the areas where chloride samples were collected. The concrete for the most part has low to no ASR activity.

The condition of the concrete under the overlay is unknown. Concrete analyzed from other decks in the area of the same vintage have been found to have high ASR gel contents. The strength of the remaining underlying original concrete is unknown.

The recommendation, based on the concrete represented in this analysis, is that it is possible to get another 10 to 20 years of service from the superstructure concrete.

Enclosures: Core compressive strength spread sheet
Chloride concentration spread sheet
ASR petrographic report
ASR petrographic result spread sheet
ASR screening spread sheet
Pictures of Chemist's ASR screening
Plan sheet of bridge section typical
Pictures of the onsite sampling visit

cc: Electronic Read File
Project File

Core break information

Project: Springfield IM 091-1(74) BR26N
Material: Concrete Core from Deck, NB
Sampled By: J. Wild
Sampled Date : 7/21/16
Received Date: 7/21/16
Tested Date: 8/2/2016
Tested By: D. Tillberg/TJ Davison
Reviewed By: J. Wild

Lab #	Core Id	Test Date	Test Time	Weight	Avg. Core Ht. (in.)	Density lb/cf	Load (#)	PSI	Corrected PSI	Avg. Test Ht. (in.)	Avg. Diam. (in.)	Corr. Factor	L/d
C160356	1A*	8/2/2016	14:21	4.28	3.72	159.2	106600	8543	#N/A	3.723	3.986	#N/A	0.994
C160361	1B	8/2/2016	14:36	3.96	3.81	144.1	106500	8543	7460	3.810	3.984	0.874	1.015
C160360	1C	8/2/2016	14:31	3.96	4.11	133.5	95500	7667	6830	4.114	3.983	0.891	1.087
C160362	2A	8/2/2016	14:42	6.33	4.23	207.7	37600	3018	2690	4.229	3.983	0.893	1.094
C160365	2B	8/2/2016	14:51	4.31	4.19	142.2	103100	8258	7340	4.194	3.987	0.889	1.078
C160363	2C	8/2/2016	14:46	4.36	4.20	143.7	106000	8484	7560	4.195	3.989	0.891	1.088

Note*: less than 4" length, cannot correct psi

Chloride penetration profile table

Springfield IM 091-1(74) Bridge 26N chloride concentration per depth (PPM)					
sample location	depth				
	0-0.5 inch	.5-1 inch	1-1.5 inch	1.5-2 inch	2-2.5 inch
1A	230	220	200	260	270
1B	2900	2800	2500	3000	2500
1C	210	220	220	220	220
2A	360	360	320	250	230
2B	130	30	30	20	50
2C	180	20	80	20	40

AGENCY OF TRANSPORTATION OFFICE MEMORANDUM

To: Jim Wild, Composite Materials Engineer

From: EJT Ethan J. Thomas, Transportation Geologist via CEE Callie Ewald, P.E., Geotechnical Engineering Manager

Date: October 26, 2016

Subject: Springfield IM 091-1(74), Bridges 26 N & S Petrographic Analysis for ASR

1.0 INTRODUCTION

The VTrans Construction and Materials Bureau Central Laboratory performed petrographic analyses on concrete cores taken from the Northbound (NB) and Southbound (SB) decks of Bridge No. 26 on Interstate 91 in Springfield, Vermont. The petrographic analyses were performed in order to determine the level of alkali-silica-reactivity (ASR) present in the cores. This memo documents the method of analysis conducted and includes a summary of results.

2.0 ANALYSES

The core samples were first screened by the Agency Chemist prior to determining the petrographic analysis testing plan. Uranyl Acetate Staining was performed by the Chemist on all of the concrete cores to determine the initial degree of ASR. Ratings were assigned to each core ranging from low to very high. These results can be found in a separate report, and were then used to determine which cores to perform petrographic analyses.

Cores were chosen for petrographic analysis based on the initial screening as well as the need to evaluate a range of activity to correlate the initial screening to the thin section analysis. When possible, the top and bottom sections of the cores that were rated as having an ASR Activity of High were analyzed petrographically. Two cores, one from each deck, rated as having an ASR Activity of High could not be analyzed due to the cores breaking in the field too close to the 4-inch minimum length required for compressive strength testing. These shorter cores didn't allow any space for a thin section to be created from the core prior to compressive strength testing. Some of the cores rated as having an ASR Activity of Low-Moderate and Low were analyzed petrographically as well to get the full range of Activity analyzed.

Five core samples were evaluated petrographically utilizing a polarized-light microscope according to ASTM 295-98.[1] A polarized-light microscope is a compound transmitted-light microscope to which components have been added to enable the determination of the optical properties of translucent substances. Polarizing filters and special analyzers allow for the

[1] ASTM C295-98**,** 2001: *Standard Guide for Petrographic Examination of Aggregates for Concrete*, American Society for Testing and Materials, Annual Book of ASTM Standards, Vol. 04.02, Concrete and Aggregates, West Conshohocken, PA, pp. 180-187.

identification of mineral species and other physical properties of rock specimens. All minerals exhibit certain optical properties that can aid in identification.

One to two thin sections were prepared from each core for a total of seven thin sections. Four thin sections were created from the SB deck and three thin sections were created from the NB deck. Table 1 shows the correlation between thin section ID number, original core location, and initial Activity rating from the Uranyl Acetate Screening performed by the Agency Chemist.

Table 1: Thin section number and location information.

Thin Section #	Location on Core	Location	Bridge	Initial Rating
C160355B	Bottom	1A	SB	Low – Small percentage of aggregate mildly affected.
C160358B	Bottom	1C	SB	Low/Moderate – Most aggregate mildly affected, no gel filled
C160361B	Bottom	1B	NB	High – All aggregate highly affected.
C160364B	Bottom	1B	SB	High – All aggregate affected, numerous gel filled voids.
C160364T	Top	1B	SB	High – All aggregate affected, numerous gel filled voids.
C160365B	Bottom	2B	NB	Very Low – No visible reaction.
C160365T	Top	2B	NB	Very Low – No visible reaction.

Creating a thin section consists of cutting ¾-inch blocks from the core samples; grinding the ¾-inch blocks with a specimen polisher until the sample surface is perfectly flat and smooth, and air drying the sample blocks overnight. Sample blocks are then mounted to a glass thin section slide using two-part epoxy and are allowed to cure overnight. Cured samples are then cut and ground to approximately 30-microns in thickness using a thin section machine with a diamond saw and grinding wheel. Final polishing of the thin section is then accomplished by hand grinding using water and #600 grit silicon carbide on a thick piece of glass. This process to create a section approximately 30 microns in thickness is needed in order to accurately identify minerals using a polarize-light microscope.

Since only one core had a High initial rating from the Springfield NB deck, and this core broke off in the field at the 4 inch minimum for compression testing, it was decided that the chip used to make the thin section would be analyzed further with the binocular stereoscopic microscope. This chip corresponds to thin section # C160361B. This chip was polished to remove saw marks from the thin section creation and analyzed to determine any macroscopic symptoms of ASR under 1X to 7X magnification.

3.0 RESULTS

The results of the thin section analysis are shown below in Table 2. Four of the seven thin sections show signs of ASR, including gel filled voids, reaction rims around reactive aggregate, and micro cracking through aggregate. Figure 1 below shows a photomicrograph of an ASR gel filled void with micro cracking of surrounding aggregate. Gel filled voids are the byproduct of silica reacting with alkalis from the cement paste. Reaction rims are zones where the silica has begun to react with the cement paste.[2] The primary reactive aggregates are strained quartz within gneisses and quartzites. Strained quartz tends to take a longer period of time to develop ASR and as such have been referred to as slow/late-expanding ASR. Research has shown that slow/late-expanding ASR can take up to 20 years to show deleterious effects.[3] The thin sections that show no signs of ASR activity lack quartzite and gneiss.

Table 2: ASR Results, Aggregate Size, and Mineral Descriptions.

Thin Section #	Observed ASR Activity	Aggregate Size	Sample Mineral Description
C160355B	No signs of ASR activity.	3/8”	Primarily amphibolite with a minor amount of quartz and free mica (muscovite & biotite).
C160358B	Reaction rims present around some quartzite grains. No gel filled voids or micro cracking present.	3/4”	Amphibolite, quartzite, granite, and opaques.
C160361B	Reaction rims around schist and quartzite. Some gel filled voids. Micro cracking observed through quartzite grains in chip.	3/8”	Quartzite, phyllite, garnet gneiss, amphibolite, opaques.
C160364B	Large gel filled voids, some micro cracking through paste and into aggregate.	3/4”	Quartzite, rare garnet gneiss, amphibolite, phyllite, some free mica (biotite) and opaques.
C160364T	Numerous small gel filled voids and reaction rims around some quartzite grains.	3/4"	Quartzite, garnet amphibolite, garnet gneiss, phyllite, and opaques.
C160365B	No signs of ASR activity.	3/8”	Granite, amphibolite, quartz, phyllite, and opaques.
C160365T	No signs of ASR activity	3/8”	Granite, amphibolite, some garnet schist, quartz, and free mica (muscovite & biotite).

[2] FHWA-HIF-13-019**,** 2013: *Alkali-Aggregate Reactivity (AAR) Facts Book*. Thomas, M. D. A., author. Federal Highway Administration, Office of Pavement Technology, Washington, DC, pp. 1-212.

[3] Jensen, V., 1993: *Alkali Aggregate Reaction in Southern Norway*. Doctor Technical Thesis. The Norwegian Institute of Technology, University of Trondheim, Trondheim, 1993, 262 pp

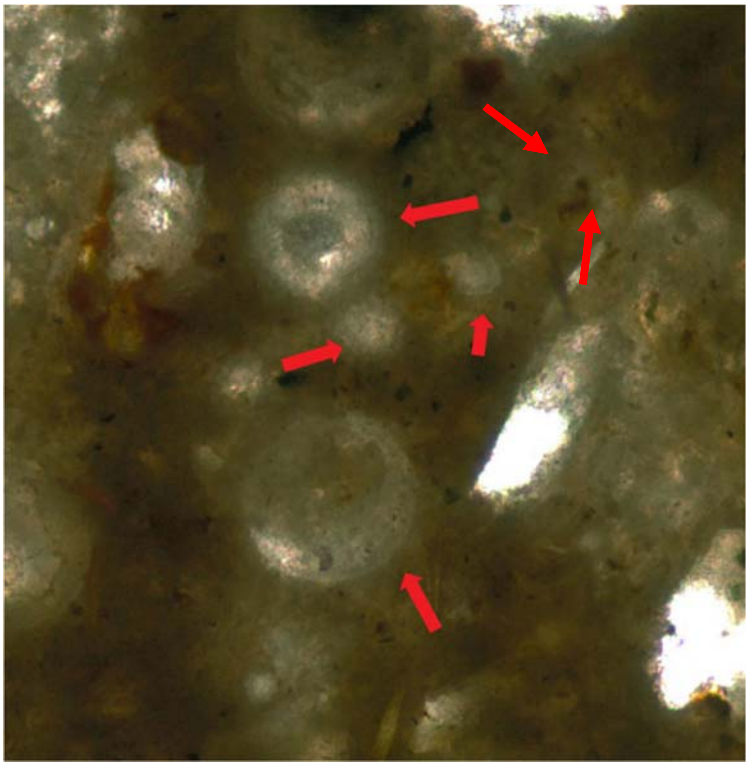


Figure 1: Photomicrograph of Thin Section # C160361B. Gel filled voids within cement paste. View is with 10x magnification and plane polarized light.

The size of the coarse aggregate in the cores for the NB bridge and some of the SB bridges was 3/8". Some of the SB bridge cores contained 3/4" aggregate. The aggregate grains within the thin sections examined range from well-rounded to sub-angular. This disparity in angularity could indicate that the source of the aggregate used within the concrete is a blended source, possibly a gravel mixed with crushed stone.

4.0 CONCLUSION

Four of the seven thin sections examined showed signs of ASR activity including gel filled voids, reaction rims around quartzite and gneiss aggregate grains, and micro cracking through paste and into adjacent aggregate. ASR gel filled micro cracks were observed in the chip cut for thin section # C160361B. The three thin sections that did not show signs of ASR activity lacked quartzite and gneiss aggregate. The findings of the petrographic analysis are in agreement with the Uranyl Acetate Screenings performed by the Agency Chemist.

If you have any questions, would like to discuss these results, or require anything further from us, please do not hesitate to contact us at (802) 828-2561 or Ethan.Thomas@vermont.gov.

cc: Electronic Read File/DJH
Project File/CEE / EJT

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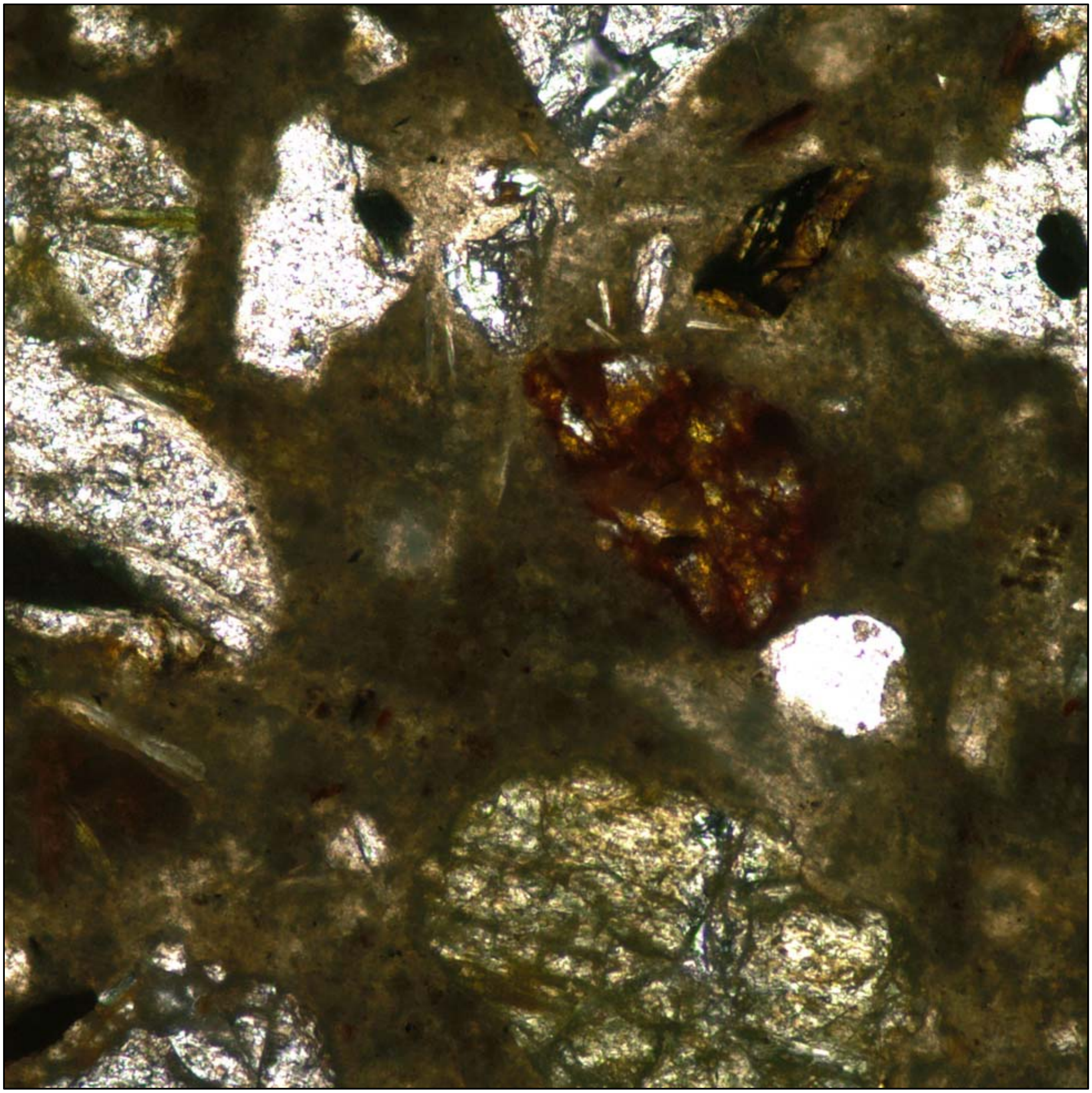
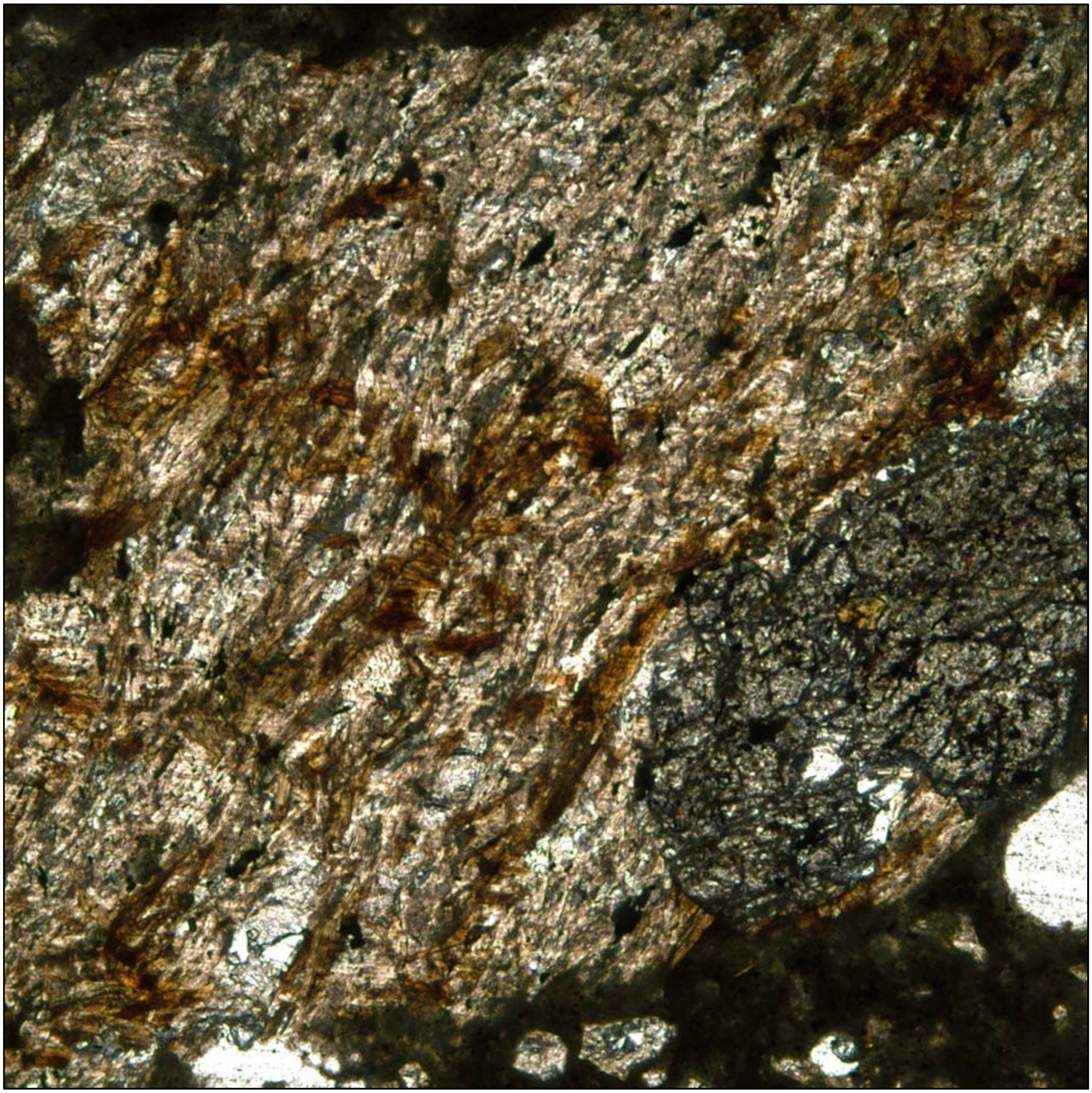
APPENDIX 1

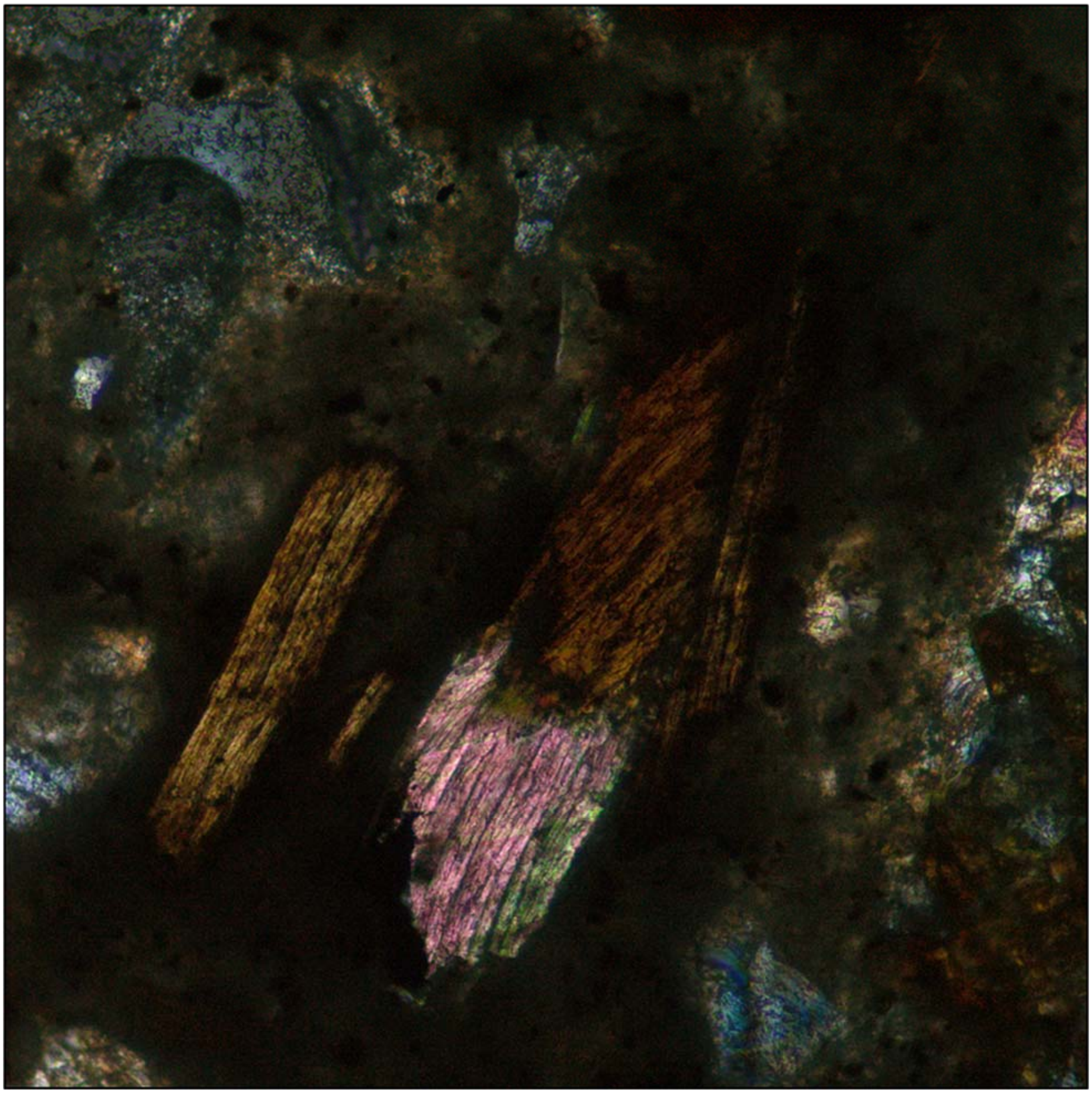
PHOTOMICROGRAPHS



C160365B: Typical view of paste. View is with 10x magnification and with plane polarized light.



C160365T: Photomicrograph of garnet-schist. View is with 4x magnification and with plane polarized light.



C160365T: Photomicrograph of biotite mica and muscovite mica in paste. View is with 20x magnification and with crossed polarized light.

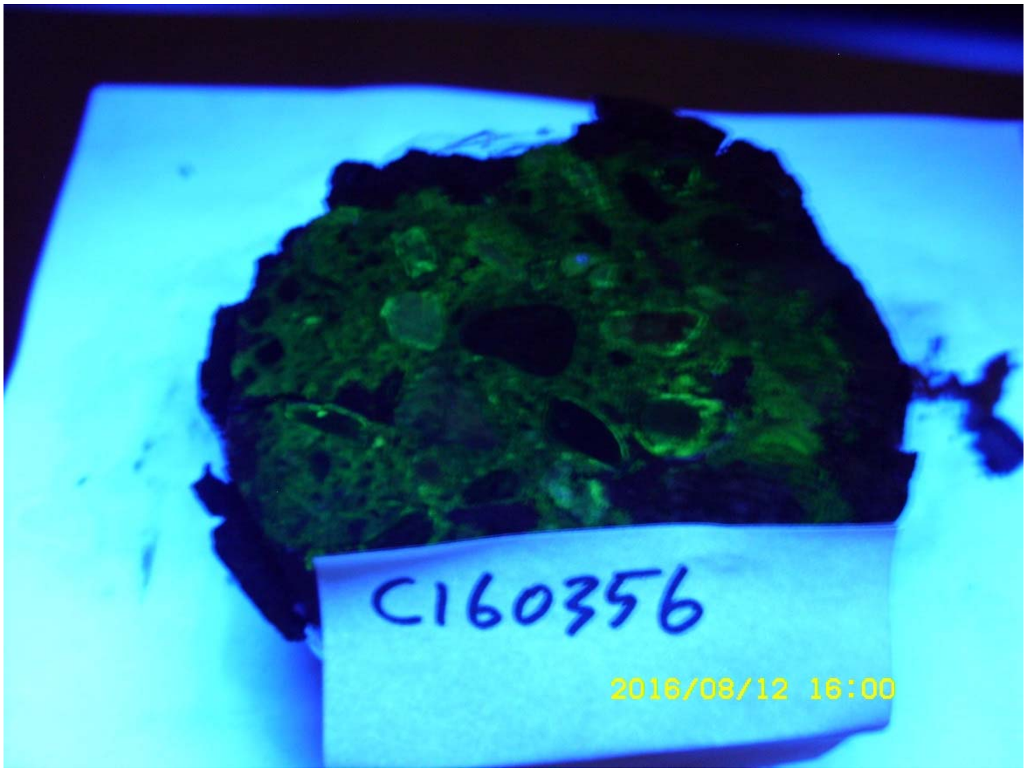
ASR screening information

Springfield IM 091-1(74) BR 26N ASR Screening				
Lab #	location	agg size	ASR Activity	Comments
C160356	1A	3/8	Moderate	Aggregate has reacted but not severely.
C160361	1B	3/8	High	All aggregate highly affected.
C160360	1C	3/8	Low	Some aggregate mildly affected.
C160362	2A	3/8	Very Low	No visible reaction.
C160365	2B	3/8	Very Low	No visible reaction.
C160363	2C	3/8	Very Low	No visible reaction.

ASR petrographic thin section descriptions

ASR Petrographic Analysis - Springfield IM 091-1(74) BR 26 N					
Thin Section #	Location on Core	Core Location	Bridge	Observed ASR Activity	Sample Mineral Description
C160361B	Bottom	1B	NB	Reaction rims around schist and quartzite. Some gel filled voids. Micro cracking observed through quartzite grains in chip.	Quartzite, phyllite, garnet gneiss, amphibolite, opaques.
C160365B	Bottom	2B	NB	No signs of ASR activity	Granite, amphibolite, quartz, phyllite, and opaques
C160365T	Top	2B	NB	No signs of ASR activity	Granite, amphibolite, some garnet schist, quartz, and free mica (muscovite & biotite).

ASR Screening Pictures

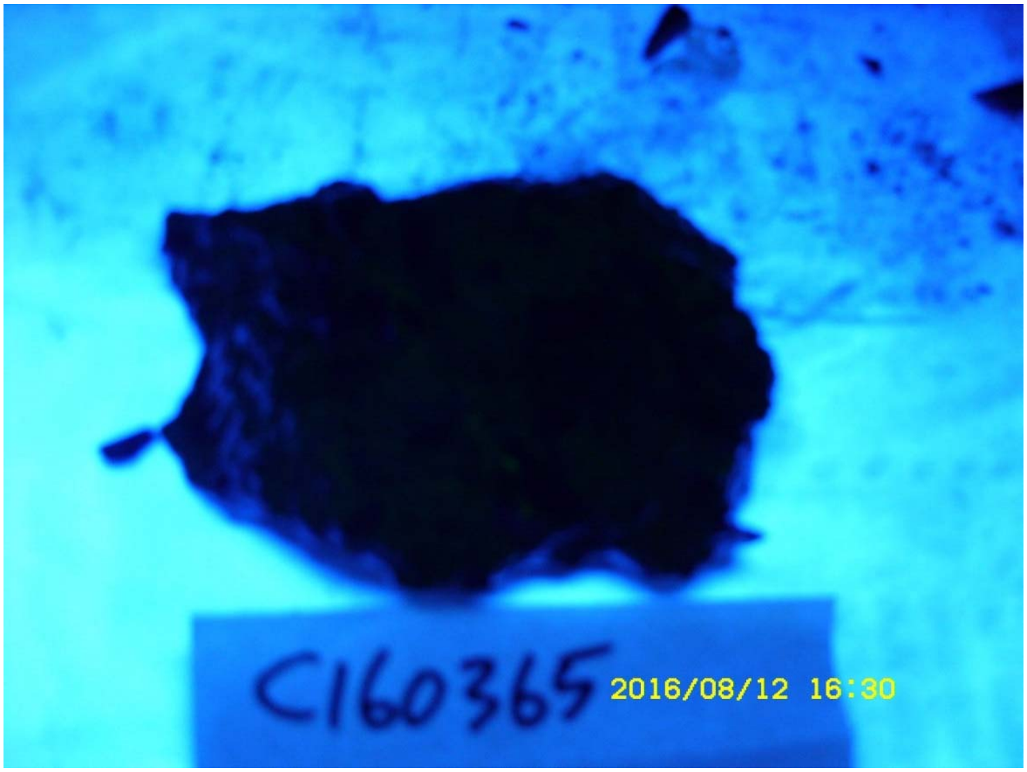


Core 1A from sample area 1, rated moderate ASR activity

Core 1B from sample area 1, rated high ASR reactivity

Core 1C from sample area 1, rated low ASR reactivity

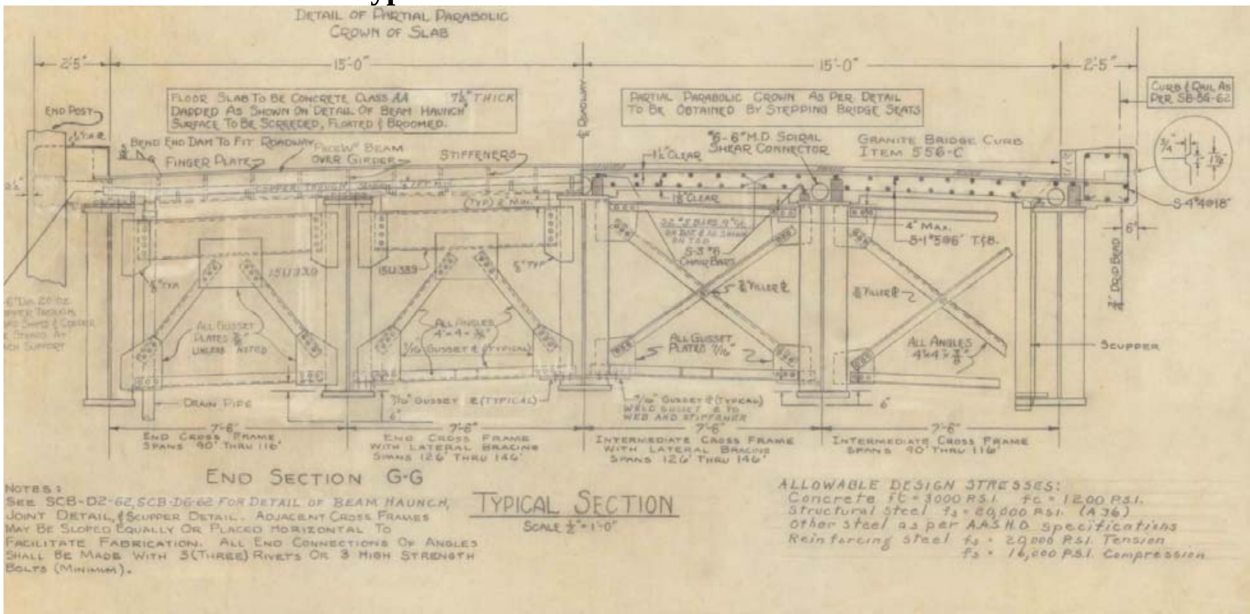
Core 2A from sample area 2, rated very low ASR reactivity



Core 2B from sample area 2, rated very low ASR reactivity

Core 2C from sample area 2, rated very low ASR reactivity

Plan sheet of deck section typical



Plan sheet as built of bridge section typical

On Site Sampling Pictures

Sample area 1, North end of the bridge. GPR unit, on the curb, that was used to layout the rebar grid prior to coring

Sample area 2 core location 3, approximately 8 feet south of hinge joint on second span

Core 1B showing interface of 3/8” aggregate concrete to 3/4” concrete.

Trailer mounted core rig used by Vtrans Drilling unit used to cut the cores.

AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Gary Sweeny, Structures

From: Jim Wild, Structural Concrete Engineer

Date: December 14, 2016

Subject: Springfield IM 091-1(74) Bridge 26S

1. INTRODUCTION

On May 27, 2016 Jonathan Griffin from Structures contacted Jim Wild, Structural Concrete Engineer, to inquire if the Structural Concrete Unit would be able to analyze the deck concrete from bridge 26S on I91 as part of the scoping project for this bridge. The proposed project scoping consists of either removing and replacing the curbs and deck overhangs or completely replacing the deck.

Bridge No. 26S is located on Interstate 91 South bound south of Exit 7 and crosses over the Black River in Springfield, Vermont. The bridge was built in 1965 by Perini Corp. of Framingham Mass. The concrete came from Charleston Redimix Inc. in Charlestown New Hampshire. The bridge is approximately 310 feet long, with the low end of the bridge at the north end. According to the original design plans the deck was to be 7.5” thick with 1.5” of cover over the top mat and 1.125” clear on the bottom.

Analysis of the concrete deck included taking concrete core samples to determine compressive strength, and concrete powder samples for use in determining concentration and depth of chloride penetration. In addition, both chemical and petrographic analysis were performed on core samples to determine the presence and severity of alkali-silica reaction (ASR).
Contained herein are the results of this field sampling and laboratory analyses, followed by a summary of findings and final recommendation.

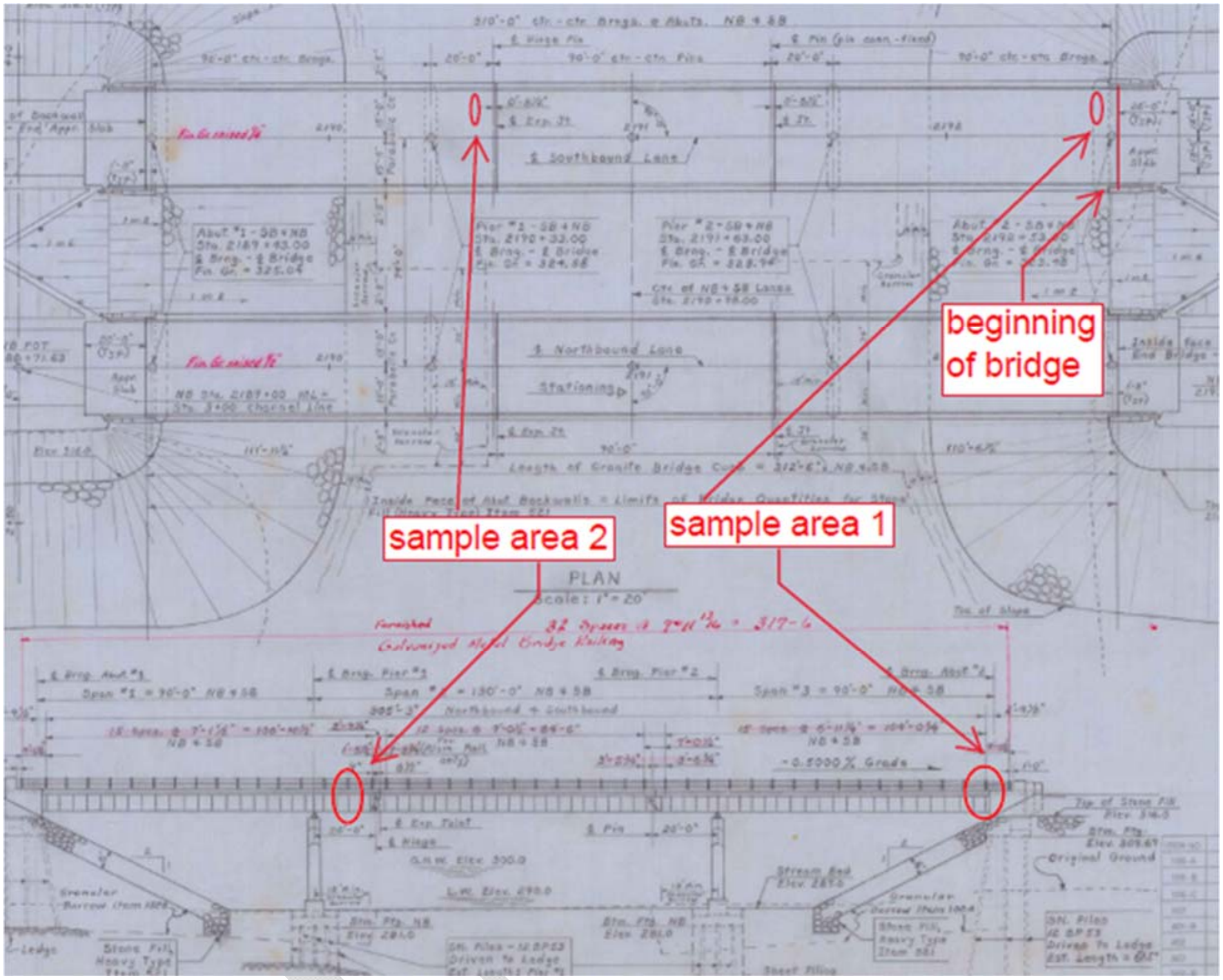
2. FIELD SAMPLING AND OBSERVATIONS

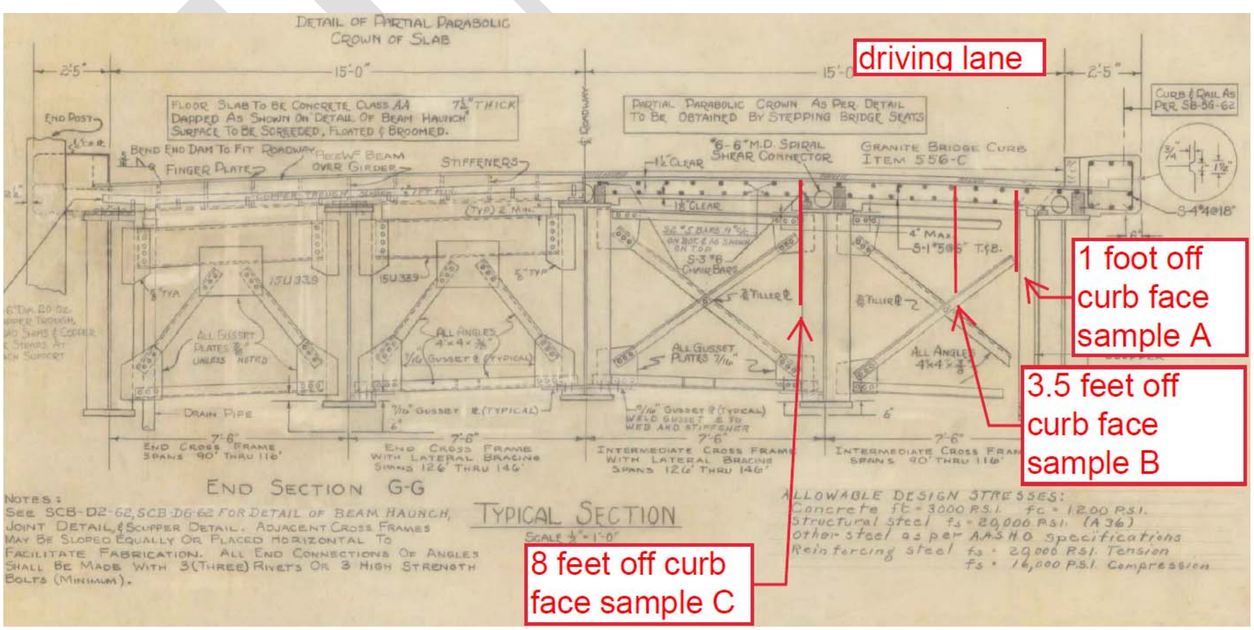
The field sampling was conducted on July 19, 2016. The final sampling location plan is as follows;

Location #1. 6 feet from the end of the deck on the low (North) end of the bridge
Location #2. 208 feet from the end of the deck on the low (North) end of the bridge

Sample area 1 was chosen because it is at the low end of the bridge deck which should see the longest duration of brine runoff. The second sample area was chosen as it would be approximately 8 feet south of the hinge joint which should see the longest duration of brine runoff for the southern span.

The following plans show approximate sample locations.


beginning
of bridge
sample area 2
sample area 1
PLAN
Scale: 1" = 20'


DETAIL OF PARTIAL PARABOLIC CROWN OF SLAB
driving lane
1 foot off
curb face
sample A
3.5 feet off
curb face
sample B
8 feet off curb
face sample C
END SECTION G-G
TYPICAL SECTION
GRANITE BRIDGE CURB ITEM 556-C
ALLOWABLE DESIGN STRESSES:

Core and chloride sample locations for BR 26S were measured perpendicular to the granite curb face. Each sample area had cores and chloride samples taken at 1 foot, 3.5 feet, and 8 feet perpendicular to the face of the West curb, noted as ‘Sample A’, ‘Sample B’ and ‘Sample C’ in Image #2 above. Ground Penetrating Radar (GPR) was used for each sample location to locate the rebar grid to avoid coring into rebar. One core was taken per AASHTO T24 from each location (A, B, and C) along with chloride samples at each location. Cores of 4” diameter were extracted for compressive strength and ASR evaluation. The Vtrans Drilling unit assisted in coring by using their trailer mounted core rig. The deck was cored to a depth of approximately 6.5” in hopes that the cores would break off at 6” to 5.5” in length. Samples of concrete dust were extracted for chloride concentration testing using a Hilti drill with 1” diameter bit. Dust samples were obtained at five ½” depth increments representing a total depth of 2.5 inches from the deck surface.

In 1989 there was a deck overlay project that was performed by The Bridge Construction Corp. of Augusta ME. The record plans were reviewed from this project. General plan notes say to use concrete Class AA for Class II repair at two inches of depth. Project specific notes indicate there were very high corrosion readings for this deck. Final quantity of Class II repair was 477.22 square yards which is roughly 46% of the deck surface. They also replaced roughly 1500 linear feet of rebar.

Coring was done to a depth of approximately 6 to 6.5 inches. Two of the cores were full depth original concrete. The other four cores were 3/8” mix and had recovered lengths of 4 to 5 inches which was approximately the interface of the 3/8” overlay concrete to the remaining original concrete. This depth is well below the two inches described in the General plan notes to remove the existing concrete to. There were a few cores in which some of the original concrete remained attached to the bottom of the core, allowing visual identification of this interface between the 3/8” concrete overlay to the original ¾” mix. This would also indicate that the interface of the overlay to original concrete has a good bond. In the appendix there is a picture showing the 3/8” to ¾” concrete interface

The plan set also indicated that a sheet membrane was to be put down then paved over. There was a lack of membrane adhesion, or lack of membrane, on the top of some of the cores indicating that the membrane may be failing which could possibly allow the intrusion of water and chlorides under it. All cores were found to have no visual indication of ASR.

Below are the locations of where the cores were extracted from:

Table 1: Sample Core Locations BR26S Area 1

Core number	Station (ft)*	Offset (ft)**	Core length (in)***
1A	6	1	4.5
1B	6	3.5	6
1C	6	8	7.75

* Station is measured from the North end of bridge, parallel to the curb face, increasing to the South
** The offset is measured perpendicular from face of the West curb to the sample location.

*** Length prior to preparation for capping

Table 2: Sample Core Locations BR26S Area 2

Core number	Station (ft)*	Offset (ft)**	Core length (in)***
2A	208	1	5
2B	208	3.5	4.25
2C	208	8	5.25

Chloride samples were taken at the same offset as the cores and approximately 1.5 feet down station from the core sample locations.

3. LABORATORY TESTING

The core samples were tested for compressive strength per AASHTO T24. Core samples obtained from the field are cut in the lab to produce cylinder ends that are flat and parallel. Petrographic analysis, ASTM 295-98, for ASR was performed on thin sections obtained from the core end cutoffs. After the cores were tested for compression strength, pieces of the cores were given to the VAOT Chemist to undergo uranyl acetate screening for ASR per AASHTO T299. The chloride penetration profile was done in accordance with ASTM C1218.

3.1 COMPRESSIVE STRENGTH (AASHTO T24). After the core ends are cut, the cores are capped with Sulphur in order to create a bearing surface that will evenly distribute the compressive stress. AASHTO T24 requires cores to be a minimum of 1:1 to 2.1:1 ratio of length to diameter after capping for compression testing. Also no reinforcing steel should be in the cut core. If a core does have rebar in it perpendicular to the axis, it is up to the specifier whether to use it or not and to determine the effect on the strength results. All core samples met the requirements of AASHTO T 24 with no rebar present, with the exception of core 2C, lab ID C160366, which did contain rebar perpendicular to the primary axis of the cylinder. The compressive strength was within range of the others so the result was considered valid for the analysis.

Table 3: Sample Core Strength BR26S Area 1

Core number	Station (ft)*	Offset (ft)**	Corrected Strength (psi)
1A***	6	1	7510
1B	6	3.5	7980
1C	6	8	8460

* Station is measured from the North end of bridge parallel to the curb face, increasing to the South
** The offset is measured perpendicular from face of the West curb to the sample location.

Table 4: Sample Core Strength BR26S Area 2

Core number	Station (ft)*	Offset (ft)**	Corrected Strength (psi)
2A***	208	1	6730
2B***	208	3.5	2690
2C***+	208	8	7680

*** Appears to be 3/8” concrete mix
+ Core had rebar running perpendicular to the axis approximately 2 inches down from the top surface.

3.2 CHLORIDE PROFILE (ASTM C1218). Chloride samples were taken at each core location approximately a foot down station as measured parallel to the curb. Five samples at ½” depth increments were taken from each location to a total approximate depth of 2.5”.

The concentration of chlorides required to initiate corrosion of steel is influenced by many variables. Some of which are pH of the concrete, the depth of carbonation, water to cement ratio, the amount of cement in the concrete along with several other variables. New concrete may take 7000 – 8000 parts per million (PPM) where old concrete could be as low as 100 PPM (PCA – Corrosion of Embedded Metals). Other research has shown that approximately 0.15% for water-soluble chlorides by mass of cement at the steel surface could initiate corrosion (Whiting – 1997). This would translate to approximately 260 PPM, assuming 660 lbs/cy of cement and a cubic yard of concrete weight of 3800 pounds. We have chosen 350 PPM as the threshold value where we would consider corrosion to initiate. This may be a conservative value but due to the amount of variables and unknown values of those variables with older concrete and typically shallower cover depths over the rebar, we believe this is a realistic value.

Sample location 1B had chloride concentration above the threshold value at the 1.5” to 2” depth by 1.4 times the threshold value. Sample locations 1A, has a concentration at the threshold value. The other sample locations were below the threshold value. Sample locations 1B and 1C appeared to be the original ¾” concrete mix. All other core locations were of 3/8” concrete mix.

There was very little reduction of chloride concentration per depth for the 3/8” concrete. The chloride concentrations for the depths of each sample are not what would be expected. Typically, there would be an 80-150 PPM decrease per 0.5” of depth. The chloride results for this deck showed virtually no decrease in values from the 0.5” depth to the maximum depth tested. One possible reason is the 3/8” is very dense and has inhibited chloride migration and the values recorded could be the background values in the concrete.

3.3 ASR SCREENING (AASHTO T299). ASR screening was done on pieces of the cores after they were tested for compression strength. The cores were lightly wrapped with plastic in order to keep the broken pieces in approximately their correct original orientation. The analysis of the screening had 2 cores rated high, 1 core at low-moderate, 2 cores at low, and 1 core at very low.

3.4 PETROGRAPHIC (ANALYSIS FOR ASR ASTM 295-98). The thin section petrographic analysis was performed on core specimen cut-offs recovered from concrete cut from each end of the compression cores prior to capping. Due to the short recovery lengths of some of the cores there was no opportunity to get a cut end section large enough from both ends to get a thin section sample. The petrographic analysis was completed on four of the cores to confirm the ASR activity ratings from the screening.

Thin section C160355B, sampled from location 1A, had no sign of ASR activity in the thin section. This correlates well with the ASR screening which had this core rated as low.

Thin section C160358B, sampled from location 1C had some reaction rims around some of the aggregate. The chemist’s screening rated this core as low-moderate for ASR Activity.

Thin section C160364T and C160364B, sampled from location 1B had reaction rims around some of the aggregate, gel filled voids and cracking through the quartzite grains. The chemist’s screening rated this core as high for ASR Activity

The petrographic analysis correlates well with results of the screening. There is strained quartz in the concrete aggregate that is contributing the silica for the alkalis to react with. Research has indicated that the reaction of strained quartz is slow reacting and could take up to 20 or more years before showing deleterious effects (Jensen V. – 1993). The age of the overlay is 27 years old. There is a high concentration of ASR gel but due to the slow reaction, it has taken a number of years for this concentration to build up.

The presence of strained quartz in the samples tested appears to be inconsistent. Some samples had aggregate containing strained quartz while others had none present. The petrographic analysis indicates there could be a possible blend of gravel and crushed stone coarse aggregates in the mix which would be a reason as to why some areas show no ASR activity and others are showing high ASR activity. The gravel source may contain the strained quartz.

4. **SUMMARY**

Based on the visual observation of the cores after they were extracted there appears to have been a rehabilitation job in which the top 4-5 inches was removed and a 3/8” inch concrete mix overlay was put down. Four of the six cores showed this characteristic. Further research into the history of the bridge confirms that there was a deck overlay project in 1989. The concrete analyzed was on the original 3/4” concrete and the overlay 3/8” concrete mix.

The compressive strengths for all the cores, except core 2B, were above the 3000psi original design strength as stated on the plans. The strength ranged from 2690 psi, this is core 2B, to 8460 psi. Core 2C had a piece of rebar running perpendicular to the axis of the core approximately 2 inches down as measured from the surface. The strength result from this core was included in the analysis as it did not appear to create an erroneous result. Excluding core 2B, the range of results was 1730 psi. There are many factors that can affect the strength of concrete placed when comparing one area to another. Some of these could be differing air content, amount of consolidation, different concrete loads, and several others.

Chloride results at the 1.5” to 2” depths, which is approximately where the first mat of steel is located, are at or below the 350 PPM threshold for initiation of corrosion of the reinforcing, excluding sample location 1B. Sample location 1B had a chloride concentration of 1.4 times the threshold value at the 1.5” to 2” depth.

The concrete in all core locations was sound.

The ASR activity ranges from high to very low from the Chemist’s screening. It did not matter if it was original concrete or the newer overlay concrete. Both concrete types had a high and low ASR activity sample. The cores were visually examined after they were extracted and there were some cores that had some visible ASR signs and others that had no visible ASR signs. Petrographic analysis was performed by the VAOT Geologist on three selected cores, C160355, C160358, and C160364.

The petrographic analysis confirmed the screening rating of high for C160364 due to the presence of gel filled voids and cracks in the paste and aggregate.

Cores C160355 and C160358 petrographic analysis confirmed the ASR screening of low and low to moderate, respectively, as no ASR activity was noted in C160355 and only a few reaction rims around some aggregate with no gel filled voids were observed in C060358.

It was observed that the adhesion of the membrane, or lack thereof, to the concrete surface was inconsistent. At some core locations, the membrane would easily detach from the concrete yet in others it had good adhesion and had to be scraped off. This could be an indication that more areas on the deck are experiencing the same lack of adhesion.

CONCLUSION

According to the record plan as-built quantities, 46% of the total area of the deck had Class II repair along with approximately 1500 linear feet of rebar replaced. Four of the six core locations that were sampled had roughly 50 to 70% of the original depth removed and replaced with a 3/8” concrete overlay.

The core strengths are in excess of design 3000 psi strength, except for core 2A, which had 90% of design strength. The deck surface appears to still be competent. There appeared to be good bond between the overlay and existing original concrete.
Except for sample location 1B, there is no concern of chloride induced corrosion of the reinforcing in the areas where chloride samples were collected.

The concrete for most of the core locations demonstrated little to no ASR activity. One of the original concrete core samples did exhibit some cracking in the paste and through the quartzite aggregate.

Concerning the condition of the original concrete under the overlay, it would be reasonable to assume the majority of this original concrete under the overlay would exhibit properties in line with the two samples that consisted of original full depth concrete. This would lead to the conclusion that the original concrete under the overlay is still sound

The recommendation, based on the concrete represented in this analysis, is that it is possible to get another 10 to 20 years of service from the superstructure concrete.

Enclosures: Core compressive strength spread sheet
Chloride concentration spread sheet
ASR petrographic report
ASR petrographic result spread sheet
ASR screening spread sheet
Pictures of Chemist’s ASR screening
Plan sheet of bridge section typical
Pictures of the onsite sampling visit

cc: Electronic Read File
Project File

Core break information

Project: Springfield IM 091-1(74) BR26S
Material: Concrete Core from Deck, SB
Sampled By: J. Wild
Sampled Date : 7/19/16
Received Date: 7/19/16
Tested Date: 8/2/2016
Tested By: D. Tillberg/TJ Davison
Reviewed By: J. Wild

Lab #	Core Id	Test Date	Test Time	Weight	Avg. Core Ht. (in.)	Density lb/cf	Load (#)	PSI	Corrected PSI	Avg. Test Ht. (in.)	Avg. Diam. (in.)	Corr. Factor	L/d
C160355	1A	8/2/2016	14:21	4.28	3.93	150.4	106600	8519	7510	3.929	3.992	0.881	1.046
C160364	1B	8/2/2016	14:46	4.36	5.31	113.3	106000	8465	7980	5.309	3.993	0.943	1.361
C160358	1C	8/2/2016	14:36	3.96	6.44	85.4	106500	8565	8460	6.442	3.979	0.988	1.673
C160357	2A	8/2/2016	14:31	3.96	3.69	149.5	95500	7694	6730	3.686	3.976	0.875	1.022
C160359	2B	8/2/2016	14:42	6.33	4.14	212.4	37600	3022	2690	4.139	3.980	0.890	1.085
C160366*	2C	8/2/2016	14:51	4.31	4.86	123.0	103100	8266	7680	4.856	3.985	0.930	1.248

Note*: rebar running perpendicular to axis, approximately 2" down from the surface

Chloride penetration profile table

Springfield IM 091-1(74) Bridge 26S chloride concentration per depth (PPM)

sample location	depth				
	0-0.5 inch	.5-1 inch	1-1.5 inch	1.5-2 inch	2-2.5 inch
1A	410	360	350	360	350
1B	840	690	590	500	500
1C	230	250	70	20	30
2A	130	110	120	130	130
2B	200	200	170	190	190
2C	220	190	200	180	190

AGENCY OF TRANSPORTATION OFFICE MEMORANDUM

To: Jim Wild, Composite Materials Engineer

From: EJT Ethan J. Thomas, Transportation Geologist via CEE Callie Ewald, P.E., Geotechnical Engineering Manager

Date: October 26, 2016

Subject: Springfield IM 091-1(74), Bridges 26 N & S Petrographic Analysis for ASR

1.0 INTRODUCTION

The VTrans Construction and Materials Bureau Central Laboratory performed petrographic analyses on concrete cores taken from the Northbound (NB) and Southbound (SB) decks of Bridge No. 26 on Interstate 91 in Springfield, Vermont. The petrographic analyses were performed in order to determine the level of alkali-silica-reactivity (ASR) present in the cores. This memo documents the method of analysis conducted and includes a summary of results.

2.0 ANALYSES

The core samples were first screened by the Agency Chemist prior to determining the petrographic analysis testing plan. Uranyl Acetate Staining was performed by the Chemist on all of the concrete cores to determine the initial degree of ASR. Ratings were assigned to each core ranging from low to very high. These results can be found in a separate report, and were then used to determine which cores to perform petrographic analyses.

Cores were chosen for petrographic analysis based on the initial screening as well as the need to evaluate a range of activity to correlate the initial screening to the thin section analysis. When possible, the top and bottom sections of the cores that were rated as having an ASR Activity of High were analyzed petrographically. Two cores, one from each deck, rated as having an ASR Activity of High could not be analyzed due to the cores breaking in the field too close to the 4-inch minimum length required for compressive strength testing. These shorter cores didn't allow any space for a thin section to be created from the core prior to compressive strength testing. Some of the cores rated as having an ASR Activity of Low-Moderate and Low were analyzed petrographically as well to get the full range of Activity analyzed.

Five core samples were evaluated petrographically utilizing a polarized-light microscope according to ASTM 295-98.[1] A polarized-light microscope is a compound transmitted-light microscope to which components have been added to enable the determination of the optical properties of translucent substances. Polarizing filters and special analyzers allow for the

[1] ASTM C295-98**,** 2001: *Standard Guide for Petrographic Examination of Aggregates for Concrete*, American Society for Testing and Materials, Annual Book of ASTM Standards, Vol. 04.02, Concrete and Aggregates, West Conshohocken, PA, pp. 180-187.

identification of mineral species and other physical properties of rock specimens. All minerals exhibit certain optical properties that can aid in identification.

One to two thin sections were prepared from each core for a total of seven thin sections. Four thin sections were created from the SB deck and three thin sections were created from the NB deck. Table 1 shows the correlation between thin section ID number, original core location, and initial Activity rating from the Uranyl Acetate Screening performed by the Agency Chemist.

Table 1: Thin section number and location information.

Thin Section #	Location on Core	Location	Bridge	Initial Rating
C160355B	Bottom	1A	SB	Low – Small percentage of aggregate mildly affected.
C160358B	Bottom	1C	SB	Low/Moderate – Most aggregate mildly affected, no gel filled
C160361B	Bottom	1B	NB	High – All aggregate highly affected.
C160364B	Bottom	1B	SB	High – All aggregate affected, numerous gel filled voids.
C160364T	Top	1B	SB	High – All aggregate affected, numerous gel filled voids.
C160365B	Bottom	2B	NB	Very Low – No visible reaction.
C160365T	Top	2B	NB	Very Low – No visible reaction.

Creating a thin section consists of cutting ¾-inch blocks from the core samples; grinding the ¾-inch blocks with a specimen polisher until the sample surface is perfectly flat and smooth, and air drying the sample blocks overnight. Sample blocks are then mounted to a glass thin section slide using two-part epoxy and are allowed to cure overnight. Cured samples are then cut and ground to approximately 30-microns in thickness using a thin section machine with a diamond saw and grinding wheel. Final polishing of the thin section is then accomplished by hand grinding using water and #600 grit silicon carbide on a thick piece of glass. This process to create a section approximately 30 microns in thickness is needed in order to accurately identify minerals using a polarize-light microscope.

Since only one core had a High initial rating from the Springfield NB deck, and this core broke off in the field at the 4 inch minimum for compression testing, it was decided that the chip used to make the thin section would be analyzed further with the binocular stereoscopic microscope. This chip corresponds to thin section # C160361B. This chip was polished to remove saw marks from the thin section creation and analyzed to determine any macroscopic symptoms of ASR under 1X to 7X magnification.

3.0 RESULTS

The results of the thin section analysis are shown below in Table 2. Four of the seven thin sections show signs of ASR, including gel filled voids, reaction rims around reactive aggregate, and micro cracking through aggregate. Figure 1 below shows a photomicrograph of an ASR gel filled void with micro cracking of surrounding aggregate. Gel filled voids are the byproduct of silica reacting with alkalis from the cement paste. Reaction rims are zones where the silica has begun to react with the cement paste.[2] The primary reactive aggregates are strained quartz within gneisses and quartzites. Strained quartz tends to take a longer period of time to develop ASR and as such have been referred to as slow/late-expanding ASR. Research has shown that slow/late-expanding ASR can take up to 20 years to show deleterious effects.[3] The thin sections that show no signs of ASR activity lack quartzite and gneiss.

Table 2: ASR Results, Aggregate Size, and Mineral Descriptions.

Thin Section #	Observed ASR Activity	Aggregate Size	Sample Mineral Description
C160355B	No signs of ASR activity.	3/8”	Primarily amphibolite with a minor amount of quartz and free mica (muscovite & biotite).
C160358B	Reaction rims present around some quartzite grains. No gel filled voids or micro cracking present.	3/4”	Amphibolite, quartzite, granite, and opaques.
C160361B	Reaction rims around schist and quartzite. Some gel filled voids. Micro cracking observed through quartzite grains in chip.	3/8”	Quartzite, phyllite, garnet gneiss, amphibolite, opaques.
C160364B	Large gel filled voids, some micro cracking through paste and into aggregate.	3/4”	Quartzite, rare garnet gneiss, amphibolite, phyllite, some free mica (biotite) and opaques.
C160364T	Numerous small gel filled voids and reaction rims around some quartzite grains.	3/4"	Quartzite, garnet amphibolite, garnet gneiss, phyllite, and opaques.
C160365B	No signs of ASR activity.	3/8”	Granite, amphibolite, quartz, phyllite, and opaques.
C160365T	No signs of ASR activity	3/8”	Granite, amphibolite, some garnet schist, quartz, and free mica (muscovite & biotite).

[2] FHWA-HIF-13-019**,** 2013: *Alkali-Aggregate Reactivity (AAR) Facts Book*. Thomas, M. D. A., author. Federal Highway Administration, Office of Pavement Technology, Washington, DC, pp. 1-212.

[3] Jensen, V., 1993: *Alkali Aggregate Reaction in Southern Norway*. Doctor Technical Thesis. The Norwegian Institute of Technology, University of Trondheim, Trondheim, 1993, 262 pp

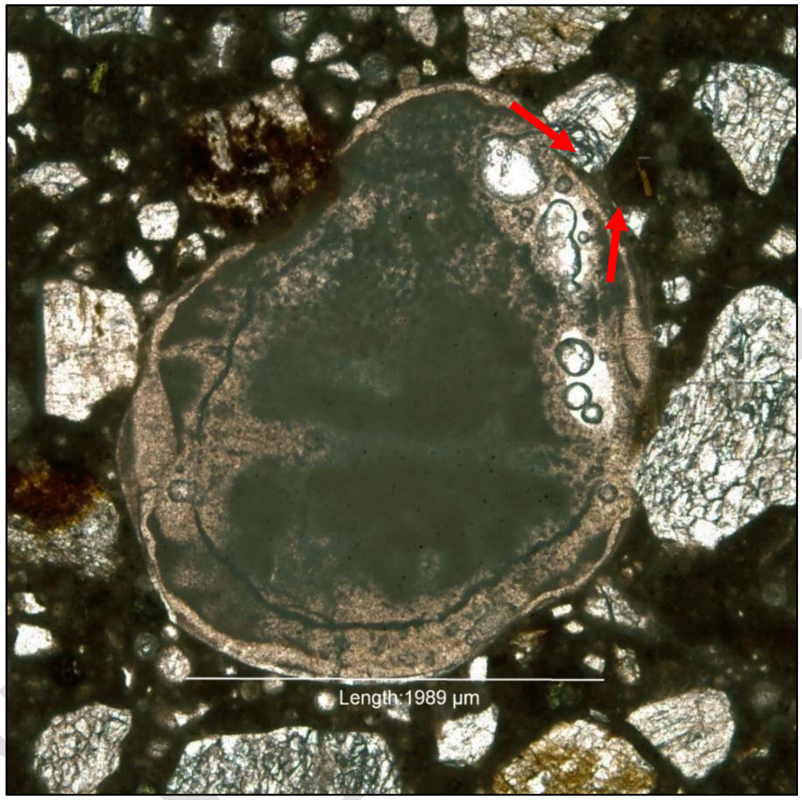


Figure 1: Photomicrograph of Thin Section #C160364B showing large ASR gel filled void. Red arrows show ASR gel filled cracks extruding into adjacent aggregate grain. Photo taken at 4X under plane polarized light.

The size of the coarse aggregate in the cores for the NB bridge and some of the SB bridges was 3/8". Some of the SB bridge cores contained 3/4" aggregate. The aggregate grains within the thin sections examined range from well-rounded to sub-angular. This disparity in angularity could indicate that the source of the aggregate used within the concrete is a blended source, possibly a gravel mixed with crushed stone.

4.0 CONCLUSION

Four of the seven thin sections examined showed signs of ASR activity including gel filled voids, reaction rims around quartzite and gneiss aggregate grains, and micro cracking through paste and into adjacent aggregate. ASR gel filled micro cracks were observed in the chip cut for thin section # C160361B. The three thin sections that did not show signs of ASR activity lacked quartzite and gneiss aggregate. The findings of the petrographic analysis are in agreement with the Uranyl Acetate Screenings performed by the Agency Chemist.

If you have any questions, would like to discuss these results, or require anything further from us, please do not hesitate to contact us at (802) 828-2561 or Ethan.Thomas@vermont.gov.

cc: Electronic Read File/DJH
Project File/CEE / EJT

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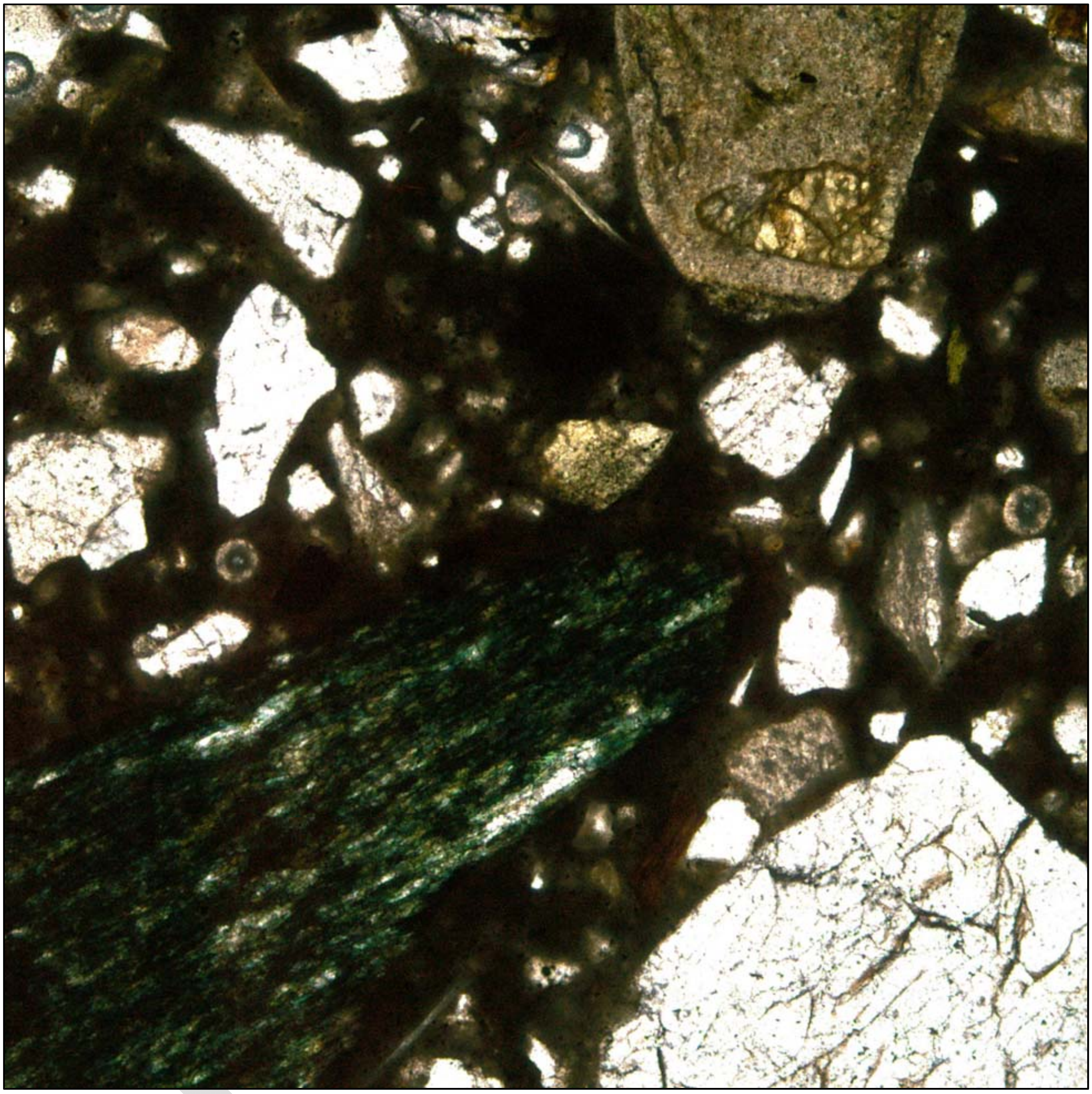
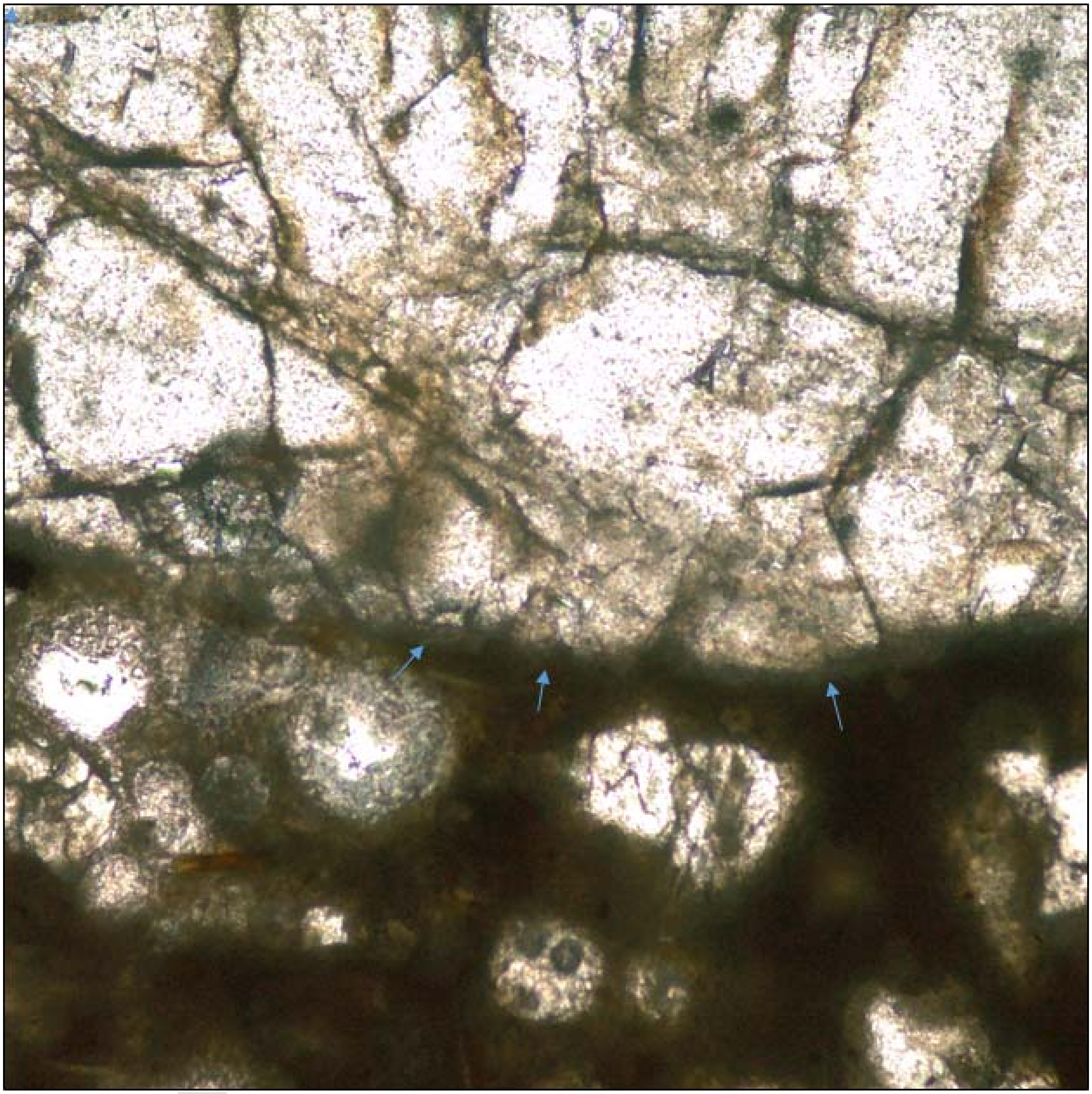
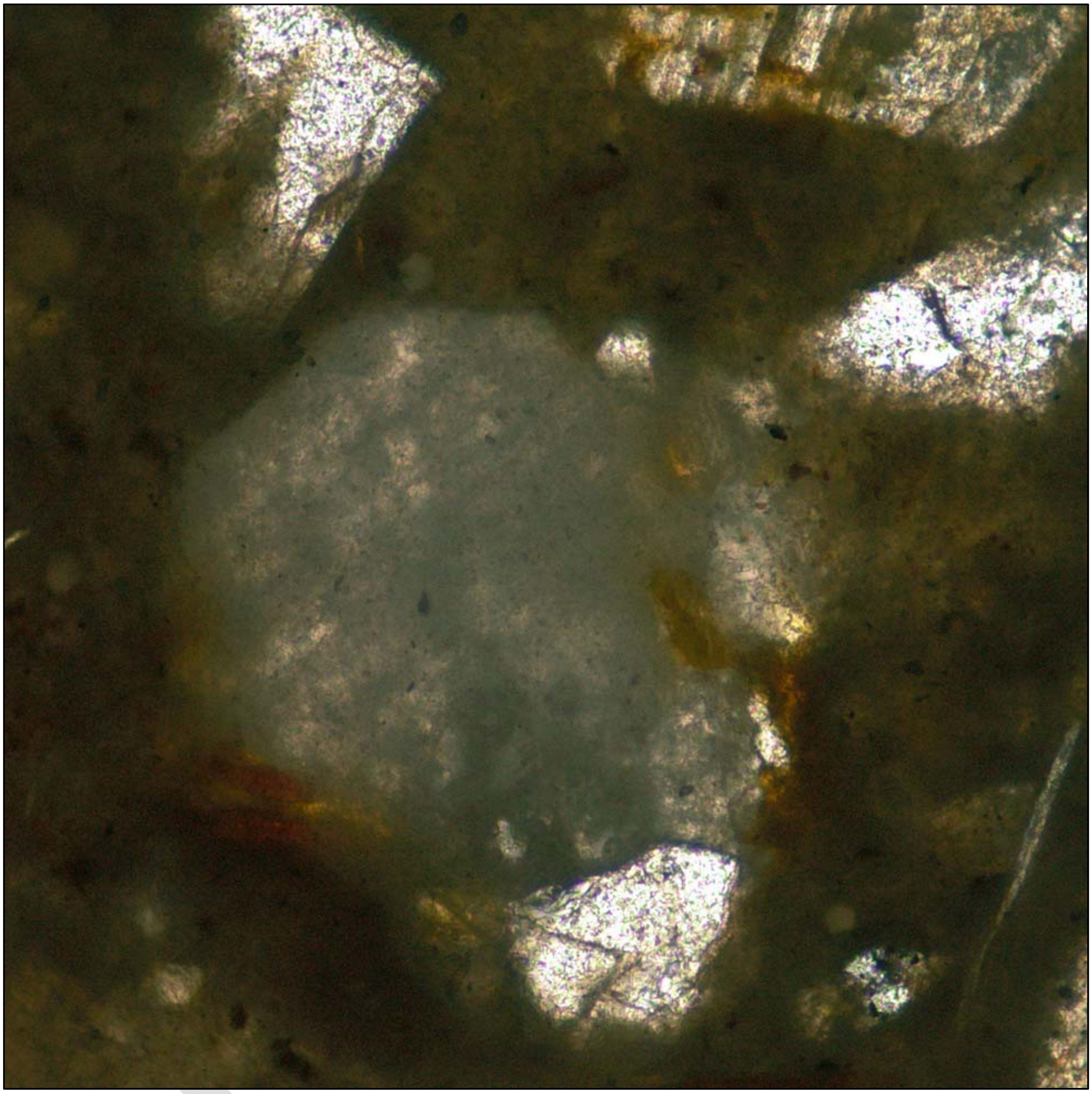
APPENDIX 1
PHOTOMICROGRAPHS



C160355B: Typical view of thin section showing paste and aggregate. View is with 4x magnification and with plane polarized light.



C160358B: Faint reaction rims around aggregate (blue arrows). View is with 10x magnification and with plane polarized light.



C160364T: ASR gel filled void in paste. View is with 20x magnification and with plane polarized light.

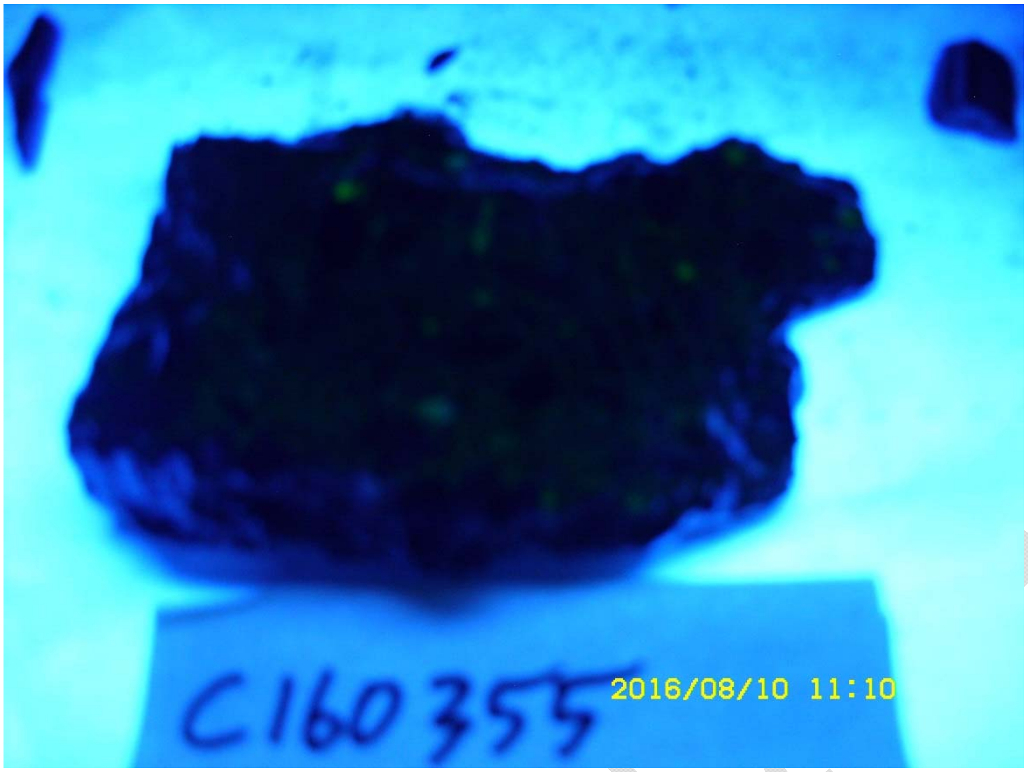
ASR screening information

Springfield IM 091-1(74) BR 26S ASR Screening				
Lab #	location	agg size	ASR Activity	Comments
C160355	1A	3/8	Low	Small percentage of aggregate mildly affected.
C160364	1B	3/4	High	All aggregate affected, numerous gel-filled voids.
C160358	1C	3/4	Low - Moderate	Most aggregate mildly affected, no gel filled voids.
C160357	2A	3/8	Low	Small percentage of aggregate mildly affected.
C160359	2B	3/8	Very Low	No visible activity.
C160366	2C	3/8	High	All aggregate affected, numerous gel-filled voids.

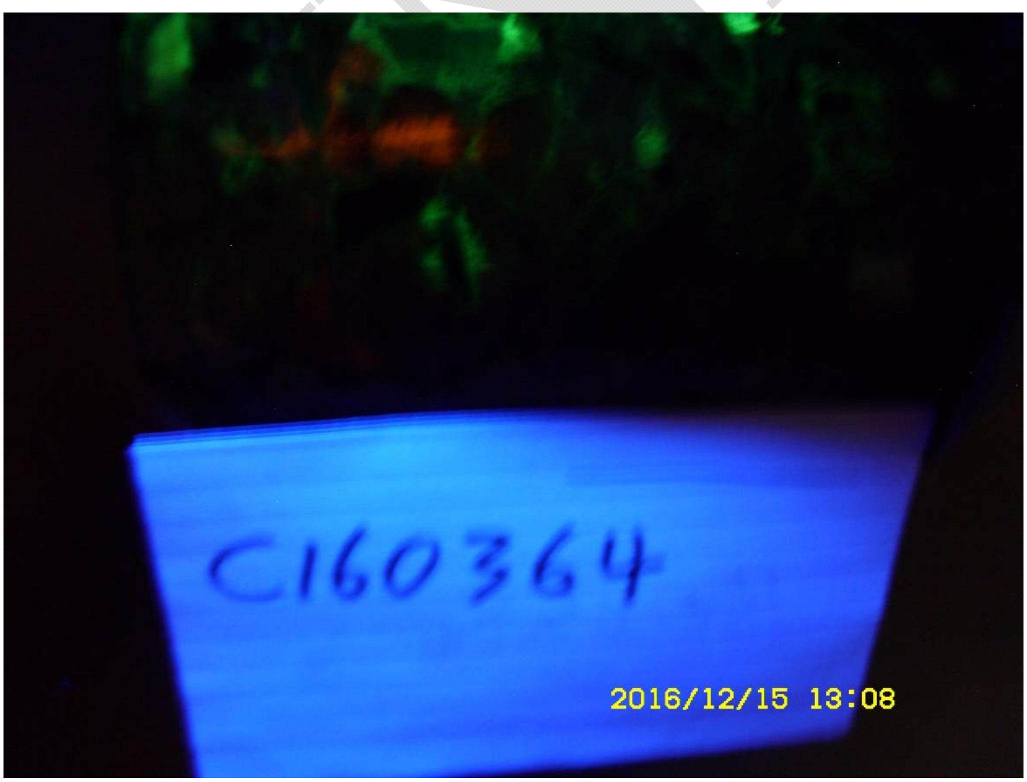
ASR petrographic thin section descriptions

ASR Petrographic Analysis - Springfield IM 091-1(74) BR 26 S					
Thin Section #	Location on Core	Core Location	Bridge	Observed ASR Activity	Sample Mineral Description
C160355B	Bottom	1A	SB	No signs of ASR activity.	Primarily amphibolite with a minor amount of quartz and free mica (muscovite & biotite).
C160358B	Bottom	1C	SB	Reaction rims present around some quartzite grains. No gel filled voids or micro cracking present.	Amphibolite, quartzite, granite, and opaques.
C160364T	Top	1B	SB	Numerous small gel filled voids and reaction rims around some quartzite grains.	Quartzite, garnet amphibolite, garnet gneiss, phyllite, and opaques.
C160364B	Bottom	1B	SB	Numerous small gel filled voids and reaction rims around some quartzite grains.	Quartzite, garnet amphibolite, garnet gneiss, phyllite, and opaques.

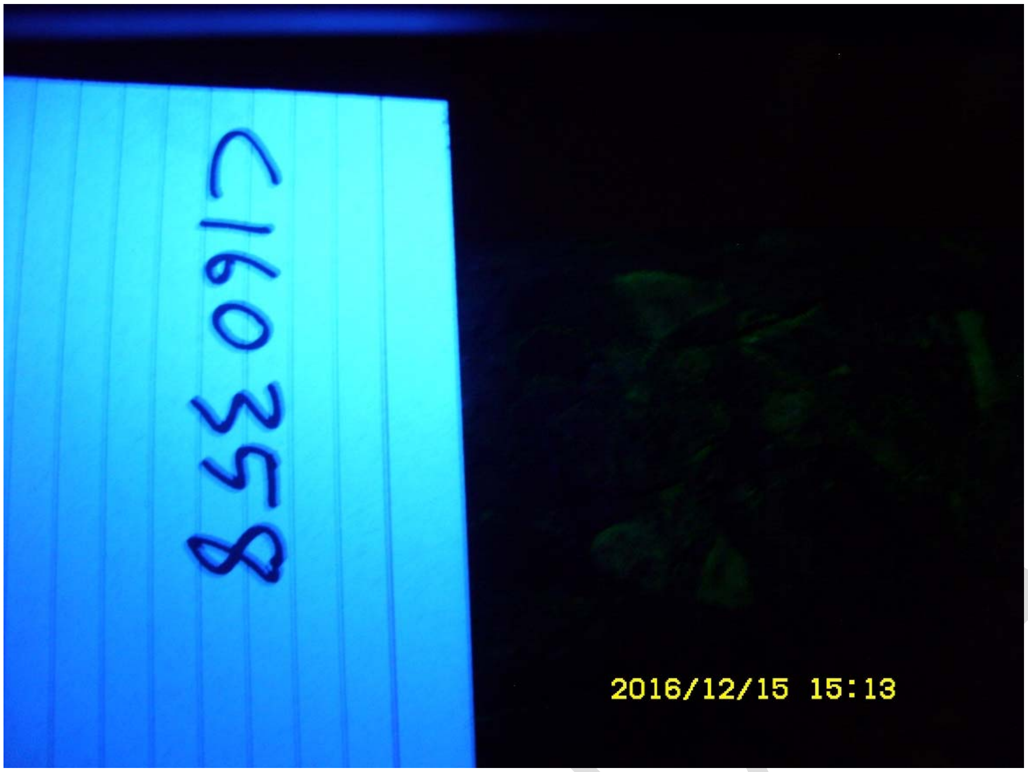
ASR Screening Pictures



Core 1A from sample area 1, rated low ASR activity



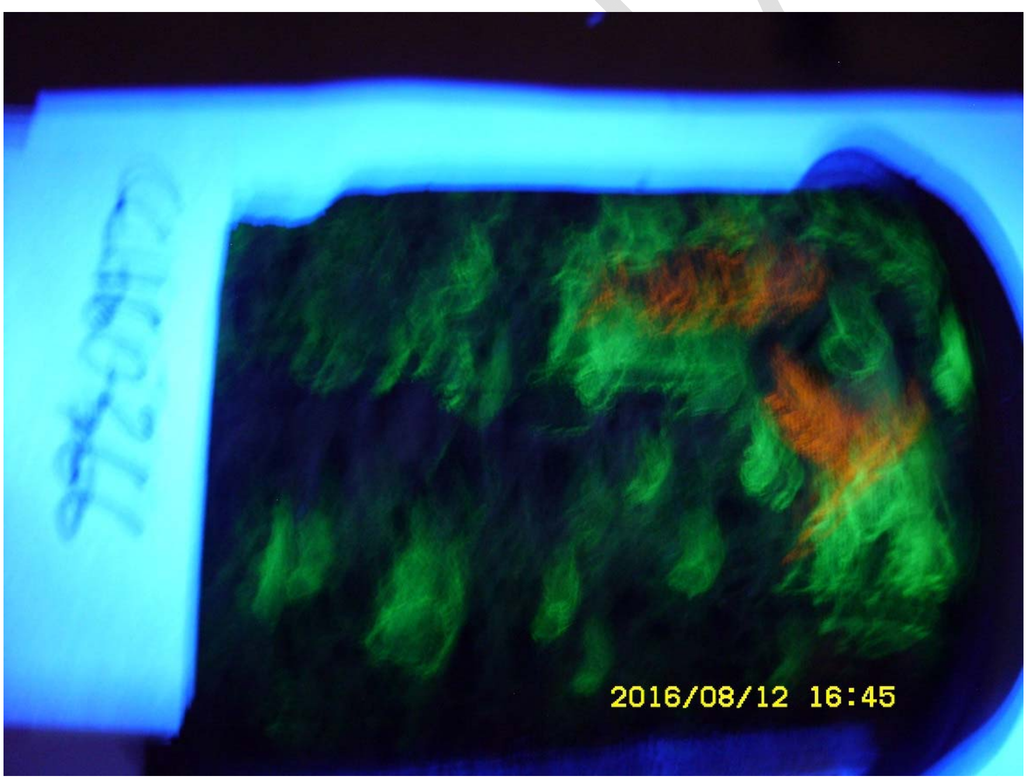
Core 1B from sample area 1, rated high ASR reactivity



Core 1C from sample area 1, rated low-moderate ASR reactivity

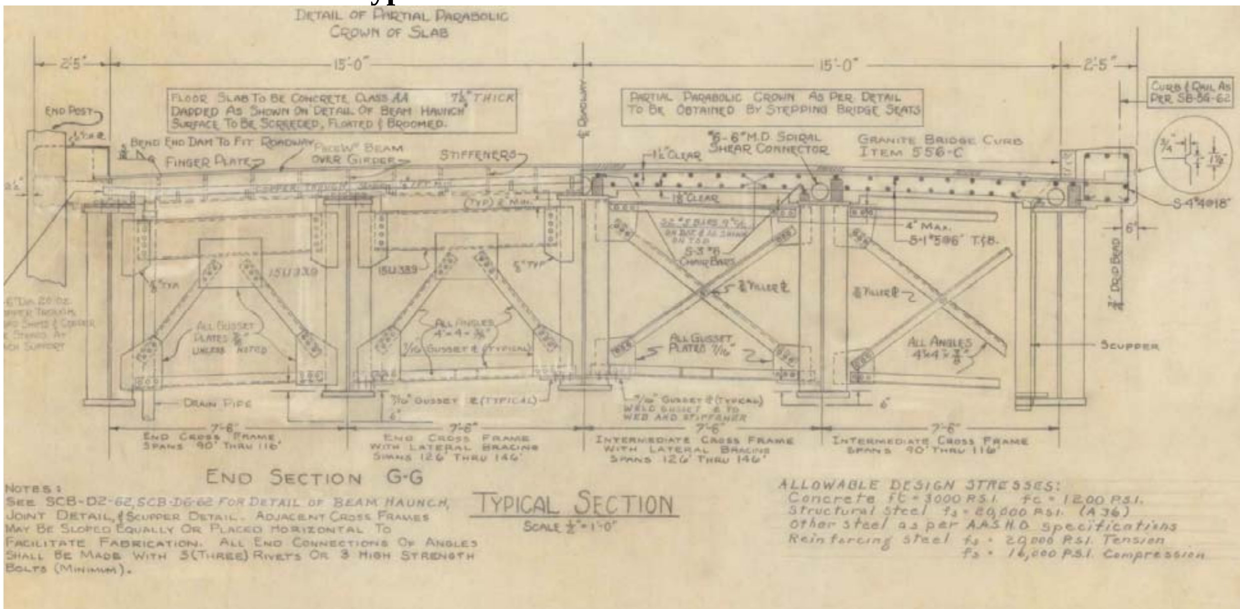
Core 2A from sample area 2, rated low ASR reactivity

Core 2B from sample area 2, rated very low ASR reactivity



Core 2C from sample area 2, rated high ASR reactivity

Plan sheet of deck section typical

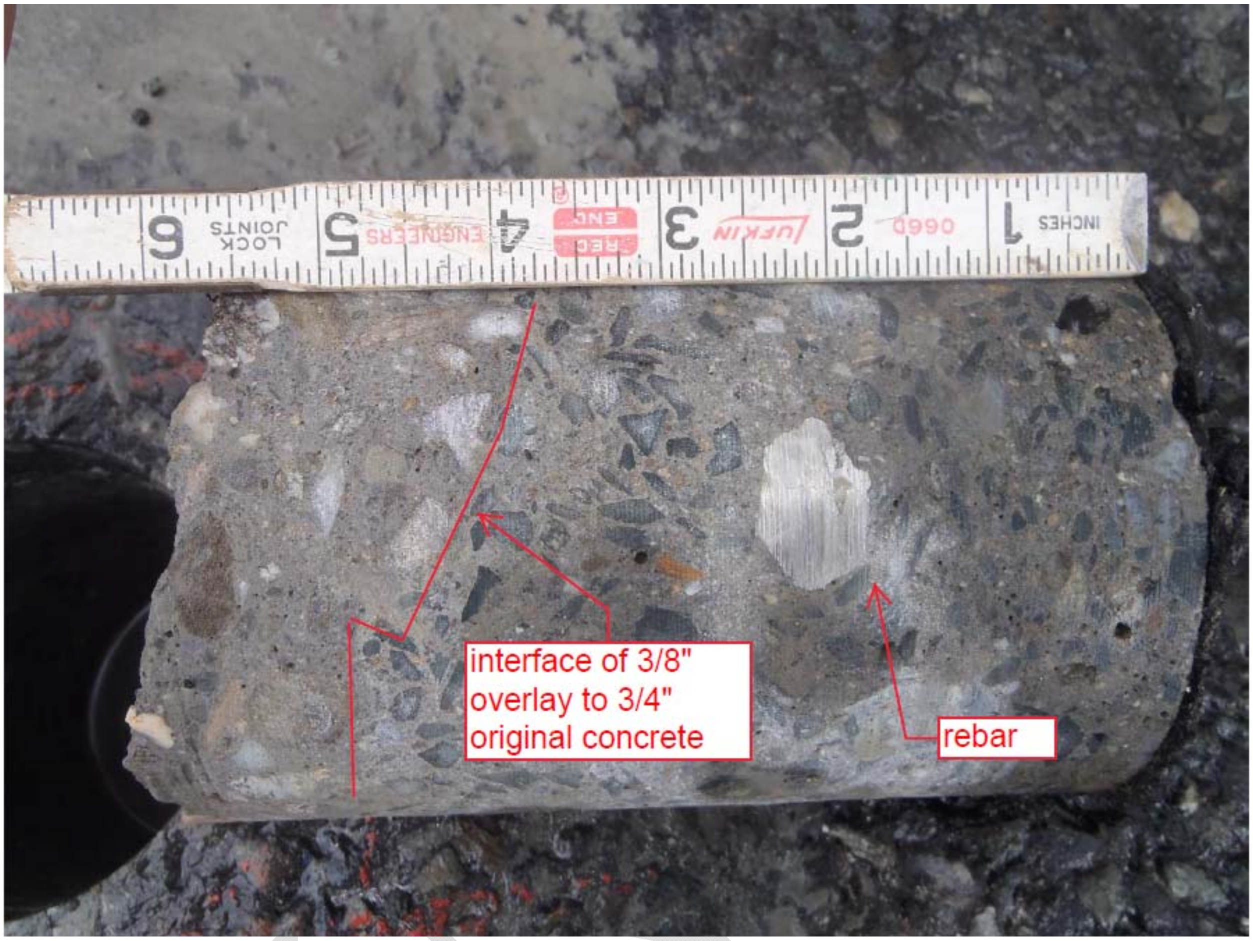


Plan sheet as built of bridge section typical

On Site Sampling Pictures

Sample area 1, North end of the bridge. measuring from end of bridge to core hole locations

Sample area 2 core location 3, approximately 8 feet south of hinge joint on second span



Core 2C showing interface of 3/8" aggregate concrete to 3/4" concrete.

Trailer mounted core rig used by Vtrans Drilling unit used to cut the cores.

File picture showing the GPR unit used and typical grid pattern determined from GPR to avoid coring rebar.

APPENDIX

8.5. Preliminary Hydraulics Report

VT AGENCY OF TRANSPORTATION PROGRAM DEVELOPMENT DIVISION

HYDRAULICS UNIT

TO: Jennifer Fitch, Structures Project Manager

FROM: Leslie Russell, P.E., Hydraulics Project Manager

DATE: 17 November 2015

SUBJECT: Springfield IM 091-1(74)
I 91 BR 26 N & S over the Black River

We have completed our preliminary hydraulic study for the above referenced site, and offer the following information for your use:

The existing structures were built in 1965. They are 3 span continuous welded girder bridges. The decks are cast-in-place concrete with bituminous pavement. The bridges have a clear span of about 305' with 2 piers each. The center spans are 126' long with the two end spans being about 87' long. The bridges are about 25' to 30' above the river. These structures are in the backwater floodplain of the Connecticut River.

The existing bridges are more than adequate hydraulically, as they are above the channel and span the channel, except for the piers.

While the scope of the project has not yet been determined, with the good and very good ratings of the substructures, it is most likely that only the superstructures will be replaced. However, in 2012, it was recommended by Stantec to underpin and place concrete under BR 26S Pier 1 to prevent undermining. If this work has not been done yet, it should be added to the scope of the project. With this in mind and the hydraulic capacity of the existing bridges, it was determined that a detailed hydraulic study would not be necessary at this time.

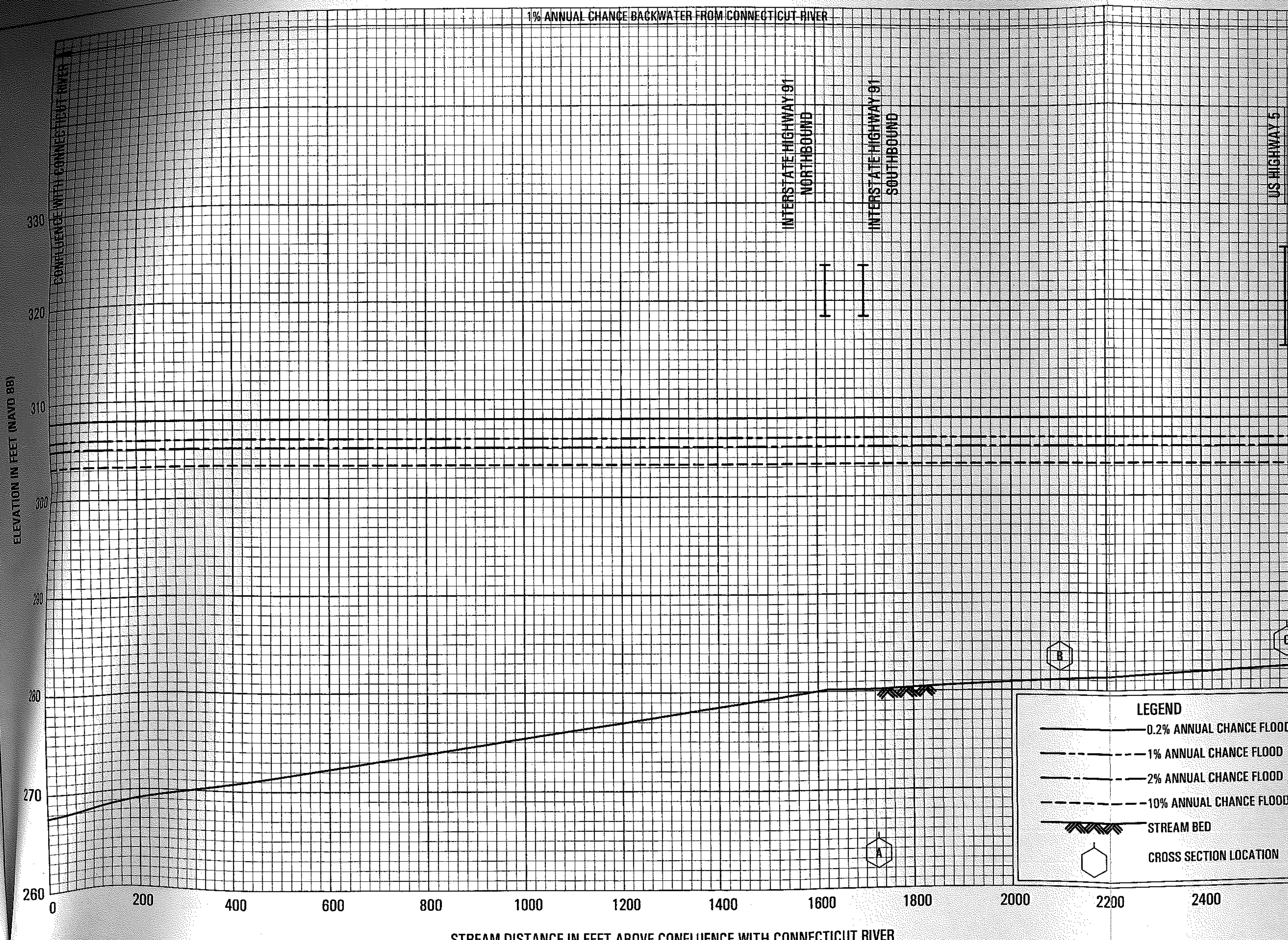
If the bridges are rehabilitated, there should be no changes that would reduce the waterway area below elevation 308.0', which includes abutments and fill material. Bottom of beams should be above elevation 315.0'. If lower bottom of beams is desired, please let us know.

If the bridges are replaced in full, it would be preferable to keep all new piers out of the channel. And new piers should be aligned with the channel.

Please contact us if you have any questions or if it is decided that full replacement of the bridges is needed.

LGR

cc: Hydraulics Project File via NJW



FEDERAL EMERGENCY MANAGEMENT AGENCY

WINDSOR COUNTY, VT

(ALL JURISDICTIONS)

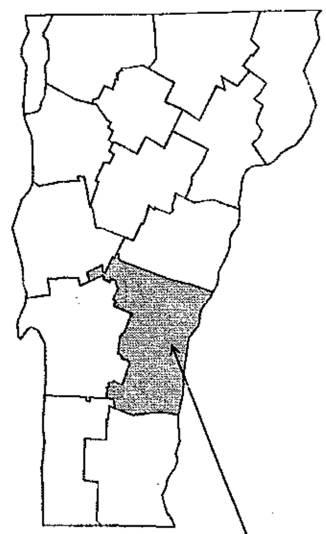
FLOOD PROFILES

BLACK RIVER

FLOOD INSURANCE STUDY

VOLUME 2 OF 4

WINDSOR COUNTY, VERMONT (ALL JURISDICTIONS)



COMMUNITY NAME	COMMUNITY NUMBER
ANDOVER, TOWN OF	500291
BARNARD, TOWN OF	500292
BETHEL, TOWN OF	500143
BRIDGEWATER, TOWN OF	500144
CAVENDISH, TOWN OF	500145
CHESTER, TOWN OF	500146
HARTFORD, TOWN OF	500148
HARTLAND, TOWN OF	500149
LUDLOW, TOWN OF	500150
LUDLOW, VILLAGE OF	500294
NORWICH, TOWN OF	500295
PLYMOUTH, TOWN OF	500151
POMFRET, TOWN OF	500297
READING, TOWN OF	500152
ROCHESTER, TOWN OF	500299
ROYALTON, TOWN OF	500153
SHARON, TOWN OF	500300
SPRINGFIELD, TOWN OF	500154
STOCKBRIDGE, TOWN OF	500155
WEATHERSFIELD, TOWN OF	500156
WEST WINDSOR, TOWN OF	500301
WESTON, TOWN OF	500157
WINDSOR, TOWN OF	500159
WOODSTOCK, TOWN OF	500160
WOODSTOCK, VILLAGE OF	500161

SEPTEMBER 28, 2007

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
50027CV002A

TABLE 3 - SUMMARY OF DISCHARGES – FREE FLOWING FLOODS - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
BARNARD BROOK					
Above confluence with Ottauquechee River	37.2	3,280	5,820	7,210	10,680
Above confluence of Gulf Stream Brook	18.5	1,840	3,450	4,280	6,350
BLACK RIVER					
At confluence with Connecticut River	206.0	3,500	6,020	7,500	13,000
Upstream of Muckross Brook	201.0	3,500	5,600	7,220	12,000
Upstream of Seaver Brook	194.0	3,500	5,000	7,000	10,000
At abandoned dam in Perkinsville	117.0	8,745	15,440	19,130	30,600
At Howard Hill Road	107.9	8,240	14,450	18,000	28,300
Above confluence of Twentymile Stream	85.3	6,900	12,200	15,100	23,700
At Ludlow/Cavendish corporate limits	77.1	6,400	11,300	14,000	22,000
At Pleasant Street Extension	71.2	5,714	10,224	12,797	20,028
Ludlow – At Main Street	68.1	5,460	9,826	12,352	19,900
Above Jewell Brook	57.8	4,850	8,800	11,300	19,500
Above Okemo Mountain Tributary	55.4	4,600	8,400	10,750	19,350
Above Branch Brook	38.8	3,850	7,080	9,300	19,000
At downstream Plymouth corporate limits	32.8	*	*	8,200	*
Upstream of confluence of Patch Brook	27.3	*	*	7,340	*
Upstream of confluence of Buffalo Brook	20.2	*	*	6,130	*
Downstream of confluence of Great Roaring Brook	10.0	*	*	4,010	*
Upstream of confluence of Great Roaring Creek	4.0	*	*	2,300	*
Upstream of confluence of Tinker Brook	0.7	*	*	840	*
BLOODY BROOK					
At confluence with Connecticut River	18.0	*	*	5,538	*
BRANCH BROOK					
At mouth	16.6	3,900	7,600	10,400	18,700
Above Coleman Brook	14.6	3,500	6,950	9,300	16,900

*Data not available

FLOODING SOURCE		FLOODWAY			WATER-SURFACE ELEVATION (FEET NAVD)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Black River								
A	1,725	251	3,569	2.1	306.0	301.8[2]	301.8	0.0
B	2,100	509	7,015	1.1	306.0	301.9[2]	302.9	1.0
C	2,575	248	3,520	2.1	306.0	301.9[2]	302.9	1.0
D	3,300	334	4,621	1.6	306.0	301.9[2]	302.9	1.0
E	4,175	201	2,749	2.7	306.0	301.9[2]	302.9	1.0
F	4,665	337	4,397	1.7	306.0	302.1[2]	302.9	0.8
G	5,425	164	2,617	2.9	306.0	302.1[2]	303.1	1.0
H	6,450	213	2,624	2.9	306.0	302.2[2]	303.2	1.0
I	7,655	205	2,141	3.5	306.0	302.4[2]	303.4	1.0
J	8,175	162	2,219	3.4	306.0	302.5[2]	303.5	1.0
K	8,300	151	2,177	3.4	306.0	302.5[2]	303.5	1.0
L	8,385	127	1,714	4.4	306.0	302.6[2]	303.6	1.0
M	9,365	143	528	14.2	307.7	307.7	307.7	0.0
N	9,675	100	580	12.9	311.6	311.6	311.6	0.0
O	10,585	105	798	9.1	317.8	317.8	317.8	0.0
P	11,665	181	1,403	5.2	320.2	320.2	320.2	0.0
Q	14,255	114	937	7.5	323.0	323.0	323.2	0.2
R	15,050	115	1,091	6.4	324.3	324.3	324.5	0.2
S	16,053	128	1,243	5.6	325.3	325.3	325.3	0.0
T	17,606	123	1,096	6.4	326.6	326.6	326.6	0.0
U	18,030	121	1,171	6.0	327.1	327.1	327.1	0.0
V	18,165	140	1,168	6.0	327.1	327.1	327.1	0.0
W	18,195	293	455	15.4	327.1	327.1	327.1	0.0
X	18,290	123	869	8.1	330.1	330.1	330.1	0.0
Y	19,588	299	1,729	4.1	332.9	332.9	332.9	0.0
Z	20,358	282	1,037	6.8	335.2	335.2	335.2	0.0

[1]Feet above confluence with Connecticut River
[2]Elevation computed without consideration of backwater effects from the Connecticut River

TABLE 7

FEDERAL EMERGENCY MANAGEMENT AGENCY

WINDSOR COUNTY, VT (ALL JURISDICTIONS)

FLOODWAY DATA

BLACK RIVER

APPENDIX

8.6. Preliminary Geotechnical Information

AGENCY OF TRANSPORTATION **OFFICE MEMORANDUM**

To: Chris Williams, P.E., Structures Project Manager

MLM CCB

From: Marcy Meyers, Geotechnical Engineer, via Christopher C Benda, P.E., Soils and Foundations Engineer

Date: July 7th, 2014

Subject: Springfield IM 091-1(74) – BR # 26 N/S Preliminary Geotechnical Information

1.0 INTRODUCTION

We have completed our preliminary geotechnical investigation for the replacement of Bridge #26 N/S on Interstate-91 over the Black River in the Town of Springfield, VT. The subject project consists of replacing the existing three-span steel girder bridges along both the northbound and southbound lanes of I-91. This review included observations made from available information including the examination of historical in-house bridge boring files, as-built record plans, USDA Natural Resources Conservation soil survey records, published surficial and bedrock geologic maps, and water well logs on-file at the Agency of Natural Resources.

2.0 SUBSURFACE INFORMATION

2.1 Previous Projects

Record plans were available for the subject project and contained original boring logs as well as foundation information for both bridges. Abutments No. 1 and 2 for both the NB and SB bridges are founded on steel 12 BP 53 piles driven to bedrock. The two piers for the NB bridge are also founded on steel 12BP 53 piles driven to bedrock. However, the two piers for the SB bridge are founded on spread footings bearing on bedrock. The boring logs revealed top of ledge elevations ranging from 250.5’ to 281.2’ at the project location. Information about the number of piles and estimated pile lengths can be found in Table 1.

Table 1: Current Foundation Information

	Number of Piles	Estimated Avg. Pile Length
Southbound Bridge		
Abutment 1	16	43’-0”
Abutment 2	16	50’-0”
Northbound Bridge		
Abutment 1	16	34’-6”
Pier 1	20	22’-6”
Pier 2	20	32’-6”
Abutment 2	16	61’-6”

Additional surrounding projects were searched for in the Soils & Foundations’ GIS based historical record of subsurface investigations which contains electronic records for the majority of borings completed in the past 10 years. An exploration of this map revealed one project Springfield BRO 1442(26), approximately 8.7 miles away, indicating sandy gravel with shallow bedrock (9-40 feet below the ground surface). Additional surrounding projects were also searched for in our G:drive projects folder and three projects with borings were found: Springfield IM CULV(11) located approximately 5.2 miles from the subject project, Springfield ST CULV(5) located approximately 6.9 miles from the subject project, and Springfield BRO 1442(26) located approximately 8.7 miles from the subject project. Due to the distance these three projects are from the subject project, the soil information should be considered ancillary.

2.2 Water Well Logs & USDA Soil Survey

The Agency of Natural Resources (ANR) documents and publishes all water wells that are drilled for residential or commercial purposes. Based on subsurface information reported by well drilling reports on file at ANR and the USDA web soil survey, the surficial geology in the vicinity of the subject area is expected to consist of a mix of sand, silt, and gravel.

Figure 1 contains the subject project as well as surrounding well locations found using the ANR Natural Resources Atlas. Published online, the logs can be used to determine general characteristics of soil strata in the area. The soil description given on the logs is done in the field, by unknown personnel, and as such, should only be used as an approximation. The specific wells used to gain information on the subsurface conditions are highlighted by a red box. Three water wells within an approximate 2,375 foot radius were used to get an estimate of the depth to bedrock likely to be encountered for BR #26 N/S.

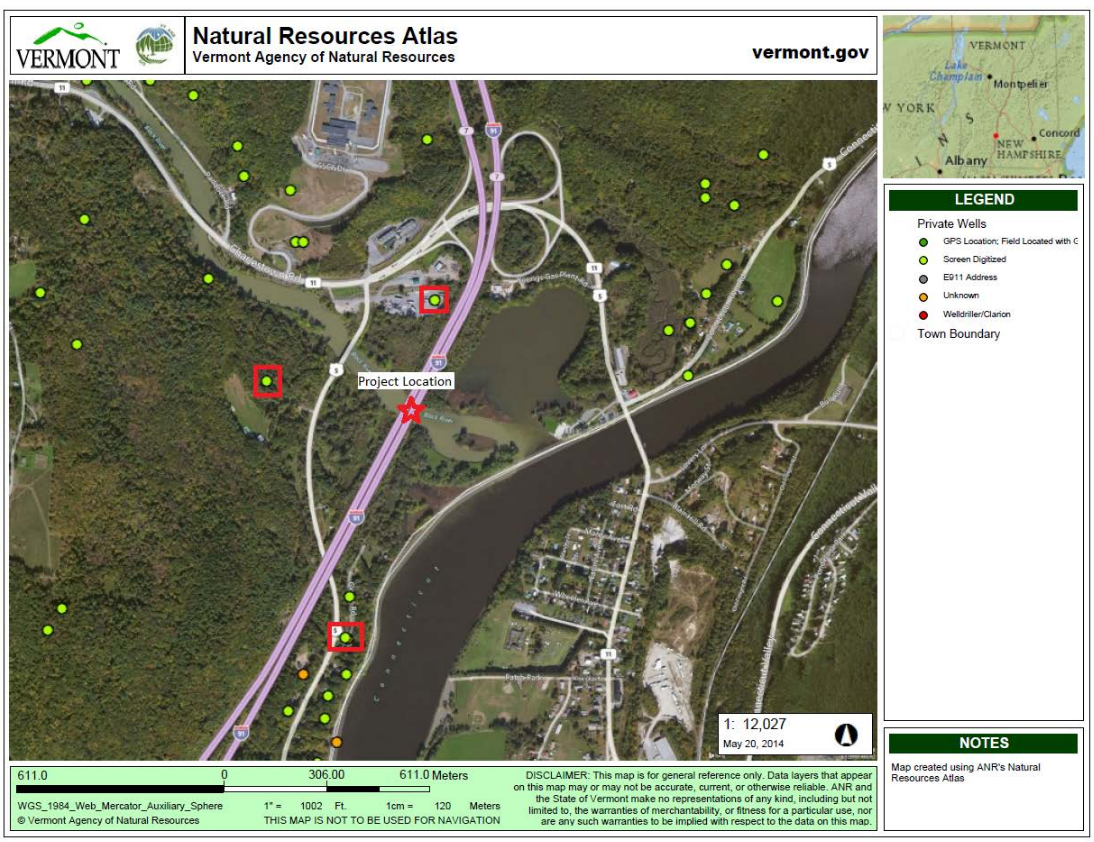


Figure 1. Highlighted Bridge and Well Locations

Table 1 lists the well sites used in gathering the surrounding information, and includes the approximate distance from the bridge project and depth to bedrock for Bridge #26 N/S.

Table 1. Well Information Including Depths to Bedrock

Well Number	Approximate Distance From Project (feet)	Approximate Depth To Bedrock (feet)
102	1145	14
591	1460	7
31331	2375	29

Information from these wells as well as record plans, suggest that shallow bedrock may be encountered during drilling operations. Information about the bedrock, taken from the ANR Natural Resource Atlas, indicates “predominately dark-to light-gray, lustrous, carbonaceous chlorite-biotite-muscovite-quartz slate, phyllite, or schist, contains thin beds of quartzite and only sparse layers of punky weathering limestone”. Based on the USDA Soil Map, the soils to be encountered at BR #26 N/S consist of Ninigret fine sandy loam with 0-8% slopes. These soils are classified as moderately well drained with a depth to bedrock of greater than 80 inches and a depth to groundwater of 18-30 inches.

2.3 Bridge Inspection Photos

Based on the latest bridge inspection report from June 2012, the curbs, structure, and joint areas in the soffit should be cleaned and patched. Additionally, the bridge railing and transitions do not meet federal standards. The northbound bridge substructure was rated as very good and the southbound substructure was rated as good. Photos from the 2012 inspection show curbing requiring patchwork as seen in Figure 2.

Figure 2: BR 26 S Curbing Requiring Patchwork

3.0 RECOMMENDATIONS

Depending on the proposed design, it may be feasible to reuse the existing substructures. Based on the most recent bridge inspection reports from June 8th, 2012, the substructure ratings for BR #26 N and #26 S were rated as very good and good, respectively. However, because preliminary designs have not yet been developed, it is too early to determine whether or not the current substructures will meet the design criteria.

If new substructures do need to be built, we recommend integral abutments or reinforced concrete abutments on spread footings as possible foundation options. If this is the case, we recommend a minimum of two borings be taken at opposite corners of each bridge, as well as at the pier locations, in order to more fully assess the subsurface conditions at the site including, but not limited to, the soil properties, groundwater conditions, and depth to bedrock. If shallow bedrock is present, borings should be performed at all four corners of the bridge, and both corners of the piers, to get a better indication of the bedrock profile across both the abutments and piers.

4.0 CONCLUSION

If you have any questions or would like to discuss this report, please contact us by phone at (802) 828-2561.

cc: DJH/Read File
CCB/Project File
MLM

G:\Soils and Foundations\Projects\Springfield IM 091-1(74)\REPORTS\Springfield IM 091-1(74) Preliminary Geotechnical Information.doc

APPENDIX

8.7. Resource ID Checklist

OFFICE MEMORANDUM

AOT - PDB - ENVIRONMENTAL SECTION

RESOURCE IDENTIFICATION COMPLETION MEMO

Clear Form

TO: Jennifer Fitch, Project Manager

FROM: Lee Goldstein, Environmental Specialist

DATE: 02/29/16

Project: Springfield IM 091-1(74)

ENVIRONMENTAL RESOURCES:

Wetlands:	◉ Yes	○ No	all 4 quadrants (Class 2)
Historic/Historic District:	○ Yes	◉ No	Considered Exempt (Interstate)
Archaeological Site:	◉ Yes	○ No	Site visit 12/10/2015; 7 known sites in vicinity-highly sensitive context-all quadrants
4(f) Property:	○ Yes	◉ No	not for historic
6(f) Property:	○ Yes	◉ No	
Agricultural Land:	◉ Yes	○ No	see NR ID
Fish & Wildlife Habitat:	◉ Yes	○ No	Black River is EFH-AOP required; also COE Cat 2 required
Endangered Species:	◉ Yes	○ No	state/fed-listed dwarf wedgemussel; a survey and relocation within project area possibly required
Hazardous Waste:	○ Yes	◉ No	
Contaminated Soils:	○ Yes	◉ No	
Stormwater:	○ Yes	◉ No	OSW unlikely, Springfield CMG PARK(32) permit is nearby; need calculations for CSW
USDA-Forest Service Lands:	○ Yes	◉ No	
Wildlife Habitat Connectivity:	◉ Yes	○ No	small mammals--per NR ID, grub rip-rap under bridge to facilitate movement for all species; adjacent habitat blocks
Scenic Highway/Byway:	○ Yes	◉ No	
Act 250 Permits:	○ Yes	◉ No	
FEMA Floodplains:	◉ Yes	○ No	AE; also floodway and
Flood Hazard Area/ River Corridor:	◉ Yes	○ No	
Invasive Species:	◉ Yes	○ No	Berberis thunbergii ID at roadside, located SW of immediate project area; knotweed could be present
Coast Guard:	○ Yes	◉ No	
Landscaping:	○ Yes	◉ No	
Environmental Justice:	○ Yes	◉ No	
Source Protection Area:	○ Yes	◉ No	
Other:	◉ Yes	○ No	NLEB language required for CA; also vascular plant n NE quad needs avoidance measures; if unavoidable, a plant inventory is needed

Thanks,

Date: 2016.02.29 15:50:56 -05'00'

cc: Jennifer Fitch
Project File

APPENDIX

8.8. Natural Resources Memo

State of Vermont
Program Development Division
One National Life Drive
Montpelier, VT 05633-5001
www.aot.state.vt.us

[phone] 802-828-3979
[fax] 802-828-2334
[ttd] 800-253-0191

Agency of Transportation

To: Lee Goldstein, VTrans Environmental Specialist

From: James Brady, VTrans Environmental Biologist

Date: January 4, 2016
Subject: Springfield IM 091-1 (74); Natural Resource ID

I have completed my natural resource report for the above referenced project. My evaluation has included wetlands, wildlife habitat, agricultural soils, and rare, threatened and endangered species.

Project Springfield IM 091-1 (74) is located at bridges 26S and 26N on Interstate 91 in the town of Springfield.

Wetlands/Watercourses
Bridges 26S and 26N carry Interstate 91 southbound and northbound barrels over the Black River.

Wetlands are located in all four quadrants of the project, see attached wetland ID map. If impacts are to occur to the mapped wetlands, a formal wetland delineation will be required before impacts can be calculated for permitting purposes.

Wildlife Habitat
The areas in and around the project area are not considered significant for wildlife habitat on a statewide or northeastern US level. That said, there is evidence of small mammals crossing under the existing structures. It is recommended that the riprap under the bridge be grubbed to allow for easy movement under the bridges for all species present in this area.

Rare, Threatened and Endangered Species
This project falls within mapped habitat for the dwarf wedgemussel (Alasmidonta heterodon) which is listed at the state and federal level as endangered and state listed as S1 (*Very rare (Critically imperiled): At very high risk of extinction or extirpation due to extreme rarity (often 5 or fewer populations or occurrences), very steep declines, or other factors*). Any work in the water or that leads to disturbance to the waters in and around this project may lead to a taking. A survey and relocation of any wedgemussels within the project area may be required.

A vascular plant ranked S3 (*Uncommon (Vulnerable): Moderate risk of extinction/extirpation due to restricted range, relatively few populations or occurrences (often 80 or fewer), recent and widespread declines, or other factors*) has been observed north of the northeast quadrant of the project area. Work in this area should be avoided. If this area must be disturbed, it is recommended that an inventory be performed to call out any existing plants to minimize impacts.

Recently, the northern long eared bat was listed by the US Fish & Wildlife Service as threatened and the Vermont Fish and Wildlife Department as endangered throughout the entire state of Vermont. The Federal Highway Administration (FHWA) and Federal Railroad Administration (FRA) have implemented a Range wide

Programmatic Informal Consultation for Indiana Bat and Northern Long-eared Bat. The guidance indicates that all trees ≥ 3” in diameter, that exhibit: cracks, crevices, holes, and peeling bark are considered suitable habitat roost trees. If tree clearing will be required, a habitat assessment will be needed prior to cutting unless trees can be cleared from November 1st through April 15th.

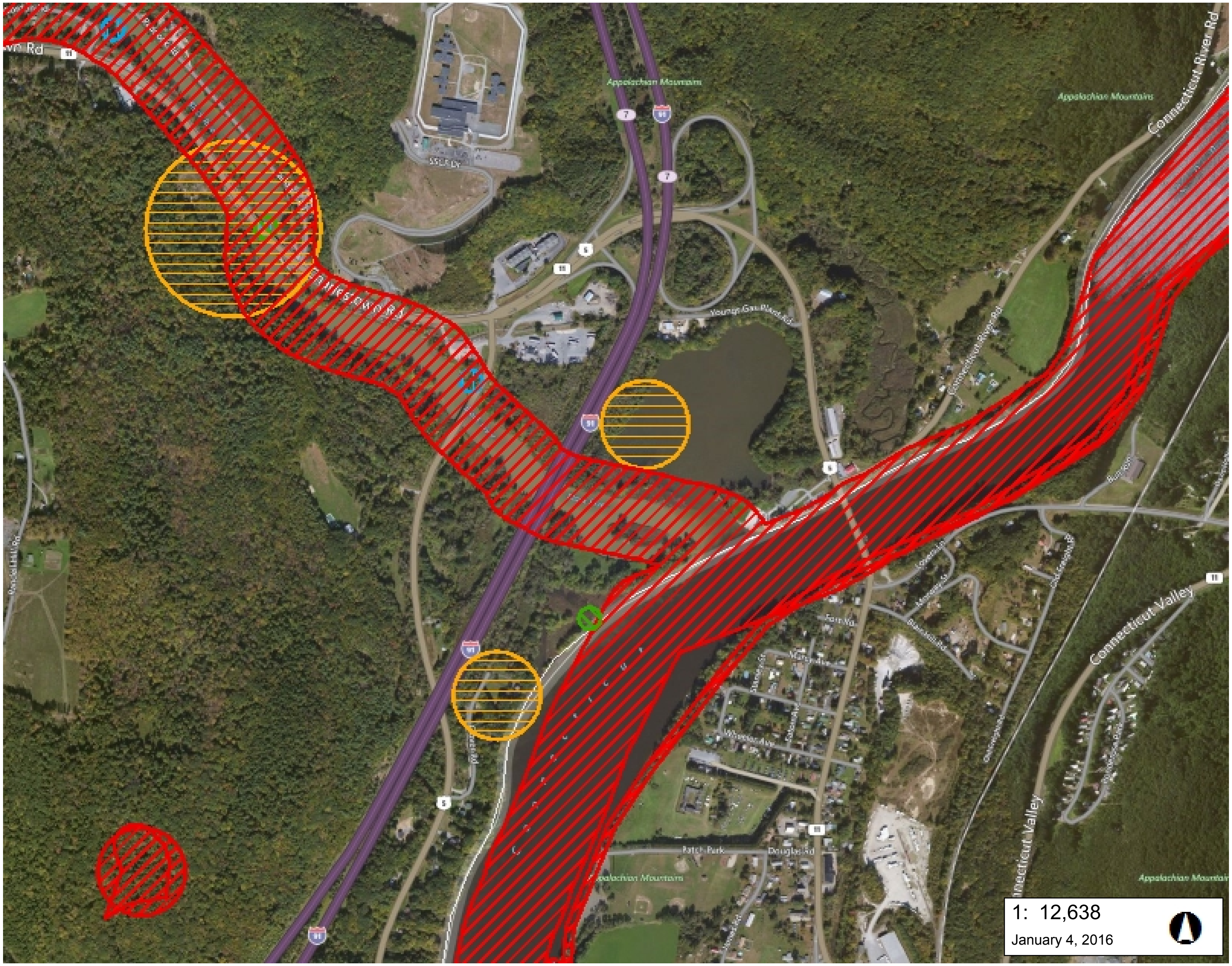
Agricultural Soils:
This project has mapped *Ninigret fine sandy loam, 0 to 8% slopes* (prime ag soil – 9B) in all four quadrants. *Podunk fine sandy loam, 0 to 3% slopes* (prime ag soil – 24) is also found in the northeast quadrant.

Springfield, VT; IM 091-1 (74)
Feet
0
300
600
1,200
N
Legend
WetlandResourceID
SPRINGFIELD
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

Springfield IM 091-1 (74); Resource ID

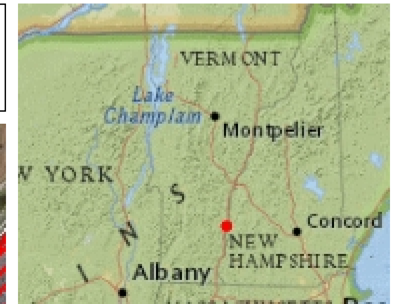
Vermont Agency of Natural Resources

vermont.gov



1: 12,638

January 4, 2016



LEGEND

Rare Threatened Endangered

- Threatened or Endangered
- Rare

Uncommon Species and Other

- Animal
- Plant
- Natural Community

Town Boundary

642.0 | 0 | 321.00 | 642.0 Meters

WGS_1984_Web_Mercator_Auxiliary_Sphere

1" = 1053 Ft. 1cm = 126 Meters

THIS MAP IS NOT TO BE USED FOR NAVIGATION

DISCLAIMER: This map is for general reference only. Data layers that appear on this map may or may not be accurate, current, or otherwise reliable. ANR and the State of Vermont make no representations of any kind, including but not limited to, the warranties of merchantability, or fitness for a particular use, nor are any such warranties to be implied with respect to the data on this map.

NOTES

Map created using ANR's Natural Resources Atlas

APPENDIX

8.9. Archaeological Memo

Jeannine Russell
VTrans Archaeology Officer
State of Vermont
Environmental Section
One National Life Drive
Montpelier, VT 05633-5001
www.aot.state.vt.us

[phone] 802-828-3981
[fax] 802-828-2334
[ttd] 800-253-0191

Agency of Transportation

To: Lee Goldstein, VTrans Environmental Specialist

From: Jeannine Russell, VTrans Archaeology Officer
via Jacquelyn Lehmann, VTrans Archaeology Tech

Date: December 15, 2015

Subject: Springfield IM 091-1 (74) – Archaeological Resource ID

The scope for this project has not yet been fully defined. We have identified archaeological resources within the vicinity of Bridges 26 N/S on US Interstate 91 over the Black River in Springfield, VT.

The VTrans Archaeology Officer and Archaeology Tech visited the site on 12-10-15. This region contains a high density of archaeological sites within the vicinity of the project area. At least seven known sites are nearby the project area, mostly pre-contact. Given the high site density and environmental factors including the river confluence, nearby wetlands, floodplains, proximity to a river, and level knolls, the general area is highly sensitive. Areas of archaeological sensitivity were found within the project area along the northeast, northwest, southeast, and southwest quadrants. These areas are marked on the Arch Sensitive Lines map attached along with historic maps and a project location map associated with the area.

The VTrans Archaeology Officer will issue a formal Section 106 when plans are available.

Please feel free to contact me with any questions.

Thank you,
Jen Russell
VTrans Archaeology Officer

/Prepared by Jacquelyn Lehmann, Archaeology Technician

VTrans Working to Get You There
Vermont Agency of Transportation

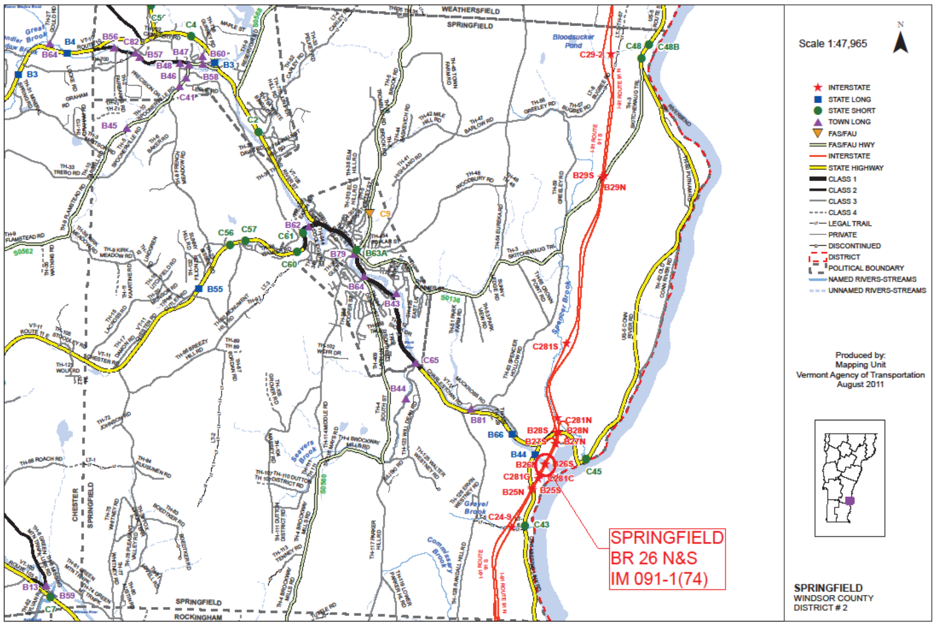


Figure 1: 2015 Project Location Map

Figure 2: Beers Map of Springfield

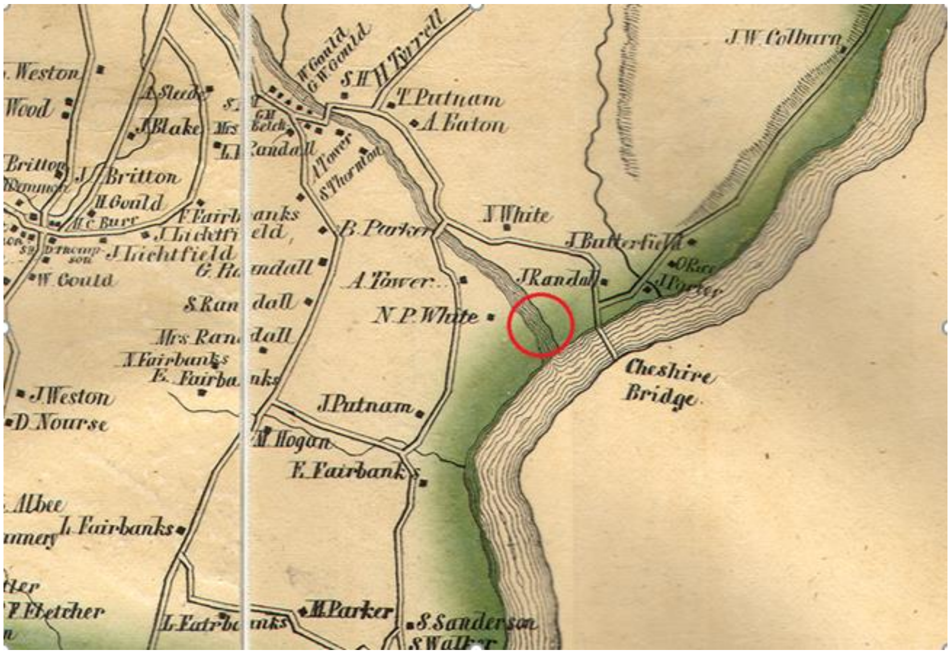
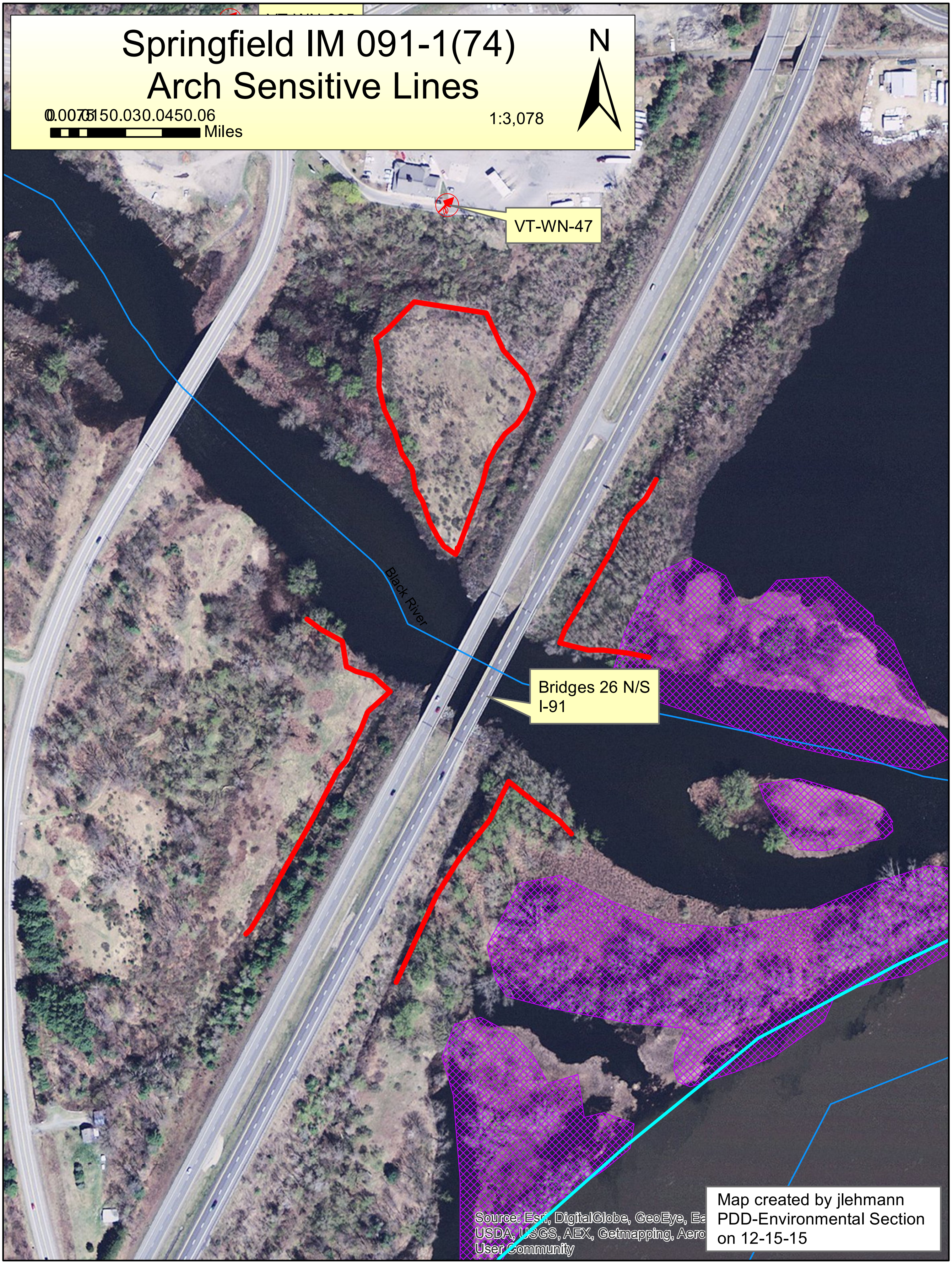


Figure 3: Walling's Map of Springfield



Springfield IM 091-1(74)
Arch Sensitive Lines
0.00750.0150.030.0450.06
Miles
1:3,078
N
VT-WN-47
Black River
Bridges 26 N/S
I-91
Source: Esri, DigitalGlobe, GeoEye, Ea
USDA, USGS, AEX, Getmapping, Aero
User Community
Map created by jlehmann
PDD-Environmental Section
on 12-15-15

APPENDIX

8.10. Historic Memo

Goldstein, Lee

From: Ramsey, Jeff
Sent: Tuesday, February 02, 2016 9:16 AM
To: Goldstein, Lee
Subject: FW: Springfield IM 091-1(74)

Jeff Ramsey
Environmental Specialist Supervisor
Vermont Agency of Transportation
Environmental Section
1 National Life Drive
Montpelier, VT 05633
(802) 828-1278
jeff.ramsey@vermont.gov
VTrans Environmental Section Website

From: Ehrlich, Judith
Sent: Tuesday, February 02, 2016 8:45 AM
To: Ramsey, Jeff <Jeff.Ramsey@vermont.gov>
Cc: Obenauer, Kyle <Kyle.Obenauer@vermont.gov>; Russell, Jeannine <Jeannine.Russell@vermont.gov>; Gauthier, Brennan <Brennan.Gauthier@vermont.gov>
Subject: Springfield IM 091-1(74)

This project is considered EXEMPT for above-ground resources per the Section 106 Exemption Regarding Effects to the Interstate Highway System adopted by the Advisory Council on Historic Preservation on March 7, 2005. (See Federal Register Vol.70/No.46)
Thanks—
Judith

VERMONT

Judith Williams Ehrlich, VTrans Historic Preservation Officer
Vermont Agency of Transportation
Project Delivery Bureau - Environmental Section
One National Life Drive
Montpelier, Vermont 05633 – 5001
judith.ehrlich@vermont.gov
(802) 828-1708

APPENDIX

8.11. Local Response and Input

Community Considerations

1. Are there any 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 bike races, festivals, parades, cultural events, farmers market, concerts, etc. that could be impacted? If yes, please provide date, location and event organizers' contact info.
   No
2. Is there a "slow season" or period of time from May through October where traffic is less?

   Unknown.

3. Please describe the location of emergency responders (fire, police, ambulance) and emergency response routes.

   Springfield Police

   Springfield Fire and Ambulance

   Charlestown (NH) Fire

   Charlestown (NH) Police

   Rockingham/ Bellows Falls Police

   Rockingham/ Bellows Falls Fire

4. Are there businesses (including agricultural operations) that would be adversely impacted either by a detour or due to work zone proximity?

   No. Although if all I-91 traffic detoured onto US-5 there would be significant traffic and potential safety issues.

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?

   No

6. What other municipal operations could be adversely affected by a road/bridge closure or detour?

   If all I-91 traffic detoured onto US-5 there would be significant traffic and potential safety issues.

7. Are there any town highways that might be adversely impacted by traffic bypassing the construction on another local road?
   No. US-5 is the logical detour, running parallel to I-91.

8. Is there a local business association, chamber of commerce or other downtown group that we should be working with?
   Unlikely to be needed, but in case they are:
   Springfield Regional Development Corporation – Bob Flint - bobf@springfielddevelopment.org
   Springfield Regional Chamber of Commerce – Caitlin Christiana - springfieldrcoc@vermontel.net
   Springfield on the Move—Carol Lighthall

Schools

1. Where are the schools in your community and what are their schedules?

   Schools do not use I-91

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

   No

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

   No

Pedestrians and Bicyclists

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

   None. Not Applicable since Interstate

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

   None. Not Applicable since Interstate

3. Does the community feel there is a need for a sidewalk on the bridge?

   None. Not Applicable since Interstate

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

   None. Not Applicable since Interstate

5. Does the Town have plans to construct either pedestrian or bicycle facilities leading up to the bridge? Please provide a planning document demonstrating this (scoping study, master plan, corridor study, town plan).

   None. Not Applicable since Interstate

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?

   None. Not Applicable since Interstate

Communications

1. Please identify any local communication channels that are available for us to use in communicating with the local population. Include weekly or daily newspapers, blogs, radio, public access TV, Front Porch Forum, etc. Also include any unconventional means such as local low-power FM.
   - Springfield Reporter – Newspaper
   - Facebook – Springfield Police Department - https://www.facebook.com/Springfield-Police-Department-Springfield-VT-133631763326692/?fref=ts
   - SAPA TV – Springfield Area Public Access Television

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?

   None known

2. Are there any special aesthetic considerations we should be aware of?
   None known

3. Are there any known Hazardous Material Sites near the project site?

   None known

4. Are there any known historic, archeological and/or other environmental resource issues near the project site?
   None known

5. Are there any other comments that are important for us to consider?
   None known

Land Use & Zoning (to be filled out by the municipality or RPC).

1. Please provide a copy of your existing and future land use map or zoning map, if applicable.
   All available online at http://swcrpc.org/town-of-springfield/ and
   http://www.springfieldvt.govoffice2.com/index.asp?SEC=16A3EF8E-6C1C-45D6-809C-42992634780B&Type=B_BASIC

2. Is there any existing, pending or planned development proposal that would impact future transportation patterns near the bridge? If so please explain.
   None known

3. Is there any planned expansion of public transit service in the project area? If not known please contact your Regional Public Transit Provider.
   Existing public transit service along I-91 by The Current (Southeast Vermont Transit, formerly Connecticut River Transit). No planned expansion of public transit service in project area.
   Contact Rebecca Gagnon for more information – rgagnon@crtransit.org

APPENDIX

8.12. Traffic and Crash Data

AGENCY OF TRANSPORTATION — OFFICE MEMORANDUM

POLICY, PLANNING AND INTERMODAL DEVELOPMENT DIVISION

TO: Christopher Williams, Structures Project Manager

FROM: Maureen Carr, Traffic Analysis Engineer MC
Colin Philbrook, Traffic Analysis Technician CCP

DATE: January 30, 2014

RE: Springfield IM 091-1(74)
I-91, BR #26 NB/SB, MM 41.265

As requested on December 11, 2013, please find in the table below the estimated 2017, 2037 and 2057 traffic data for the subject project.

~ ***Section #1***- *BR #26 Northbound*
~ ***Section #2***- *BR #26 Southbound*

Section	AADT		DHV		%T		%D		ADTT		ESALs	
	2017	2037	2017	2037	2017	2037	2017	2037	2017	2037	(2017 ~2037)	(2017 ~2057)
1	5700	6600	920	1100	16.1	22.5	100	100	1300	2100	8,757,000	20,600,000
2	5700	6600	1200	1400	16.0	21.7	100	100	1300	2000	10,565,000	24,959,000

If you have any questions please call me at x3667.

CC: Chris Cole, Director of Policy, Planning and Intermodal Development
Data Analysis Files

Vertmont Agency of Transportation

General Yearly Summaries - Crash Listing: State Highways and All Federal Aid Highway Systems

From 01/01/08 To 12/31/12 General Yearly Summaries Information

Route: I-91 Continued ...

*	Reporting Agency/ Number	Town	Mile Marker	Date MM/DD/YY	Time	Weather	Contributing Circumstances	Direction Of Collision	Number Of Injuries	Number Of Fatalities	Number Of Untimely Deaths	Direction	Road Group
	VTVSP0400/09D10 1678	Springfield	39.4	07/02/2009	11:31	Cloudy	No improper driving, Driving too fast for conditions, Failure to keep in proper lane	Same Direction Sideswipe	0	0	0	N	SH
	VTVSP0400/08D10 1402	Springfield	39.51	04/19/2008	12:49	Clear	Made an improper turn, Distracted, No improper driving	Same Direction Sideswipe	2	0	0	S	SH
	VTVSP0400/09D10 0583	Springfield	39.6	03/02/2009	11:54	Cloudy	Driving too fast for conditions, Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/10D10 0256	Springfield	39.6	01/24/2010	20:43	Sleet, Hail (Freezing Rain or Drizzle)	Driving too fast for conditions	Single Vehicle Crash	1	0	0	S	SH
	VTVSP0400/12D10 3639	Springfield	39.6	11/09/2012	17:19	Clear	No improper driving	Single Vehicle Crash	0	0	0	N	SH
	VTVSP0400/08D10 1286	Springfield	39.75	04/07/2008	05:32	Fog, Smog, Smoke	Failure to keep in proper lane	Single Vehicle Crash	1	0	0	S	SH
	VTVSP0400/08D10 0428	Springfield	39.95	02/01/2008	12:31	Snow	Failure to keep in proper lane	Single Vehicle Crash	0	0	0		SH
	VTVSP0400/11D10 2128	Springfield	39.95	07/20/2011	17:04	Clear	Failure to keep in proper lane	Single Vehicle Crash	1	0	0	S	SH
	VTVSP0400/11D10 3003	Springfield	40.03	09/29/2011	14:13	Rain	Fatigued, asleep	Single Vehicle Crash	1	0	0	S	SH
	VTVSP0400/09D10 0488	Springfield	40.05	02/20/2009	23:18	Snow	Driving too fast for conditions	Single Vehicle Crash	0	0	0		SH
	VTVSP0400/08D10 0365	Springfield	40.1	01/27/2008	08:12	Snow	Driving too fast for conditions, Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/08D10 4297	Springfield	40.19	12/31/2008	11:10	Snow	Driving too fast for conditions, Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/08D10 0147	Springfield	40.2	01/11/2008	18:49	Clear	No improper driving	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/10D10 3136	Springfield	40.24	11/12/2010	16:36	Clear	Followed too closely, Swerving or avoiding due to wind, slippery surface, vehicle, object, non-motorist in roadway etc	Rear End	0	0	0	N	SH
	VTVSP0400/08D10 0148	Springfield	40.25	01/11/2008	18:30	Clear	Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/08D10 0146	Springfield	40.25	01/11/2008	18:12	Clear	Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/10D10 2871	Springfield	40.25	10/15/2010	16:27	Rain	Failure to keep in proper lane, Driving too fast for conditions	Single Vehicle Crash	0	0	0	N	SH
	VTVSP0400/12D10 0375	Springfield	40.3	01/30/2012	13:17	Clear	Swerving or avoiding due to wind, slippery surface, vehicle, object, non-motorist in roadway etc, Failure to keep in proper lane	Single Vehicle Crash	0	0	0	N	SH
	VTVSP0400/10D10 1706	Springfield	40.4	06/21/2010	07:25	Clear	Exceeded authorized speed limit, Failure to keep in proper lane	Single Vehicle Crash	0	0	0		SH
	VTVSP0400/10D10 0041	Springfield	40.5	01/03/2010	19:25	Snow	Driving too fast for conditions	Single Vehicle Crash	0	0	0	N	SH
	VTVSP0400/09D10 3059	Springfield	40.54	11/11/2009	06:59	Cloudy	No improper driving	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/08D10 3189	Springfield	40.6	09/20/2008	14:49	Clear	Failure to keep in proper lane, Inattention	Single Vehicle Crash	1	0	0		SH
	VTVSP0400/11D10 0869	Springfield	40.72	03/25/2011	13:00	Clear	Visibility obstructed	Same Direction Sideswipe	0	0	0	N	SH
	VTVSP0400/10D10 2435	Springfield	40.88	08/29/2010	19:56	Clear	Failure to keep in proper lane, Fatigued, asleep	Single Vehicle Crash	1	0	0		SH
	VTVSP0400/10D10 2804	Springfield	40.89	10/06/2010	16:45	Fog, Smog, Smoke	Exceeded authorized speed limit, Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/08D10 1200	Springfield	40.9	03/28/2008	00:45	Cloudy	Driving too fast for conditions	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/08D10 4288	Springfield	40.9	12/31/2008	08:07	Snow	Driving too fast for conditions, Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/09D10 1314	Springfield	40.92	05/25/2009	13:50	Cloudy	Fatigued, asleep, Failure to keep in proper lane	Single Vehicle Crash	2	0	0	S	SH

**Crash occurred prior to the last Highway Improvement Project. This data should not be used in a crash analysis. UNK indicates the Mile Marker is Unknown.*

Vermont Agency of Transportation

General Yearly Summaries - Crash Listing: State Highways and All Federal Aid Highway Systems

From 01/01/08 To 12/31/12 General Yearly Summaries Information

Route: I-91 Continued ...

*	Reporting Agency/ Number	Town	Mile Marker	Date MM/DD/YY	Time	Weather	Contributing Circumstances	Direction Of Collision	Number Of Injuries	Number Of Fatalities	Number Of Untimely Deaths	Direction	Road Group
	VTVSP0400/10D10 0629	Springfield	40.92	02/25/2010	22:39	Snow	Driving too fast for conditions, Failure to keep in proper lane	Single Vehicle Crash	1	0	0	N	SH
	VTVSP0400/08D10 4177	Springfield	40.95	12/21/2008	12:38	Snow	Driving too fast for conditions, Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/09D10 2224	Springfield	41	08/21/2009	15:02	Rain	Driving too fast for conditions, No improper driving	No Turns, Thru moves only, Broadside ^<	0	0	0	N	SH
	VTVSP0400/12D10 3858	Springfield	41	11/30/2012	01:32	Cloudy	Driving too fast for conditions	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/12D10 0235	Springfield	41.17	01/18/2012	09:40	Clear	No improper driving, Failure to keep in proper lane, Operating vehicle in erratic, reckless, careless, negligent, or aggressive manner	Same Direction Sideswipe	0	0	0	N	SH
	VTVSP0400/09D10 2618	Springfield	41.26	09/28/2009	17:10	Cloudy	Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/08D10 2698	Springfield	41.3	08/07/2008	11:05	Clear	No improper driving, Failure to keep in proper lane	Same Direction Sideswipe	0	0	0	N	SH
	VTVSP0400/12D10 2132	Springfield	41.35	07/10/2012	13:21	Cloudy	Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/09D10 3503	Springfield	41.36	12/23/2009	17:10	Clear	No improper driving, Failure to keep in proper lane	Same Direction Sideswipe	0	0	0	S	SH
	VTVSP0400/08D10 3651	Springfield	41.5	11/07/2008	13:03	Cloudy	Failed to yield right of way, Swerving or avoiding due to wind, slippery surface, vehicle, object, non-motorist in roadway etc, No improper driving	Same Direction Sideswipe	0	0	0	S	SH
	VTVSP0400/09D10 0644	Springfield	41.5	03/09/2009	08:18	Snow	Driving too fast for conditions	Rear End	0	0	0	N	SH
	VTVSP0400/12D10 3963	Springfield	41.55	12/09/2012	08:02	Cloudy	Driving too fast for conditions	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/10D10 1165	Springfield	41.56	04/29/2010	05:49	Fog, Smog, Smoke	No improper driving	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/10D10 1167	Springfield	41.56	04/29/2010	05:49	Fog, Smog, Smoke	No improper driving	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/11D10 3138	Springfield	41.56	10/12/2011	07:00	Clear	Fatigued, asleep	Single Vehicle Crash	1	0	0	S	SH
	VTVSP0400/10D10 1166	Springfield	41.63	04/29/2010	05:40	Cloudy	Driving too fast for conditions, Failure to keep in proper lane	Single Vehicle Crash	0	0	0	S	SH
	VTVSP0400/12D10 1006	Springfield	41.67	03/29/2012	17:40	Cloudy	Failure to keep in proper lane, Driving too fast for conditions	Single Vehicle Crash	1	0	0	N	SH
	VTVSP0400/12D10 2701	Springfield	41.7	08/23/2012	15:54	Clear	No improper driving, Made an improper turn	Same Direction Sideswipe	0	0	0	N	SH
	VTVSP0400/08D10 3860	Springfield	41.8	11/28/2008	18:29	Cloudy	Unknown, Failed to yield right of way, No improper driving	Single Vehicle Crash	0	0	0	N	SH
	VTVSP0400/08D10 4298	Springfield	41.8	12/31/2008	11:48	Snow	Driving too fast for conditions	Single Vehicle Crash	2	0	0	S	SH
	VTVSP0400/12D10 2315	Springfield	41.95	07/28/2012	14:47	Rain	Driving too fast for conditions	Single Vehicle Crash	0	0	0	N	SH
	VTVSP0400/10D10 2629	Springfield	42.15	09/19/2010	08:32	Cloudy	Failure to keep in proper lane, Exceeded authorized speed limit	Single Vehicle Crash	1	0	0	N	SH
	VTVSP0400/11D10 2197	Springfield	42.15	07/25/2011	05:34	Clear	No improper driving	Single Vehicle Crash	1	0	0	S	SH
	VTVSP0400/08D10 2550	Springfield	42.2	07/28/2008	09:40	Clear	Failure to keep in proper lane	Single Vehicle Crash	1	0	0		SH
	VTVSP0400/10D10 0188	Springfield	42.23	01/19/2010	08:10	Snow	Driving too fast for conditions, Failure to keep in proper lane	Single Vehicle Crash	0	0	0	N	SH
	VTVSP0400/12D10 0622	Springfield	42.33	02/24/2012	19:04	Sleet, Hail (Freezing Rain or Drizzle)	Driving too fast for conditions	Single Vehicle Crash	0	0	0		SH
	VTVSP0400/12D10 0294	Springfield	42.51	01/23/2012	16:31	Sleet, Hail (Freezing Rain or Drizzle)	Followed too closely, Driving too fast for conditions	Rear End	0	0	0	N	SH

**Crash occurred prior to the last Highway Improvement Project. This data should not be used in a crash analysis. UNK indicates the Mile Marker is Unknown.*

APPENDIX

8.13. Preliminary Seismic Evaluation

APPENDIX

We performed a preliminary seismic analysis on NB Pier #2. We selected pier #2 as it has the longest piles and the soils are described as Muck and Silt over a mix of Silt, Sand and Clay to Ledge at approximately 30 feet below grade. We evaluated the site classification as Category E with peak ground acceleration as 0.07 from LRFD Figure 3.10.2.1-1. With the corresponding 0.2 second and 1.0 second rock accelerations from Figures 3.10.2.1-2 and 3.10.2.1-3 we constructed the Response Spectrum curve for this site and found that the peak spectral acceleration is 0.4 G for short period, 0.2 second structures, then decaying down to 0.16 G at 1.0 second.

For simplicity, to assess potential seismic vulnerabilities in the pier, we analyzed it as an inverted pendulum in its weak direction with no restraint provided by any fixity from the superstructure. We applied the superstructure dead load reactions as part of the mass of the pendulum, estimated the stiffness based on the un-cracked section of the pier stem and found the period of the pier to be 0.2 seconds, thus conservatively the peak spectral acceleration for this pier is 0.4 G. As part of a seismic retrofit strategy we assumed the superstructure to be isolated and estimated the period of that component of load to be 1.0 second thus reducing the spectral acceleration for the superstructure to 0.16 G. We then combined the reduced load from the superstructure with the full load from the pier to find a revised seismic load. We applied both of these loads at their respective elevations on the pier and calculated shears and moments at the base of the pier stem and at the bottom of the pier footing. We compared these seismic moments to the capacity of the stem in flexure and distributed the shear and moment to the pile group, then evaluated the critical pile for combined axial load and flexure.

For the case of the existing structure we find that the stem is inadequate, with a capacity-to-demand (C/D) ratio of approximately 0.33. For the piles we find that they are overstressed beyond yield with a C/D ratio of about 0.67. For the case of the existing pier with the superstructure isolated, seismic demands drop on the pier by approximately half. We still find the stem to be inadequate for seismic flexure but the C/D ratio increases to 0.67. For the piles in combined axial load and flexure we find that they are just adequate at approximately 92% of yield.

Springfield I-91 N/S over the Black River conveys an interstate highway and so would be considered a critical essential bridge. Based upon our preliminary seismic analysis it is likely the structure would suffer significant damage in a design earthquake. It should be noted that this analysis is very simplified and hence very conservative but it is a good tool to provide a quick evaluation of critical structural components to determine if retrofitting the structure should be considered or if the structure may be deemed adequate from a detailed structural analysis alone. For this structure a detailed seismic analysis should be performed to provide more refined seismic loads but is very likely that seismic retrofitting is warranted.

Because of the classification of critical essential and its location on a Seismic Zone 2 site, a strategy to address seismic deficiencies would include a multi-mode analysis of the entire structure as stated in LRFD Table 4.7.4.3.1-1. The existing high level rocker bearings have a demonstrated history of being vulnerable to toppling in seismic events. Replacement of these bearings with reinforced elastomeric bearings designed as isolation bearings would have a two-fold benefit both

APPENDIX

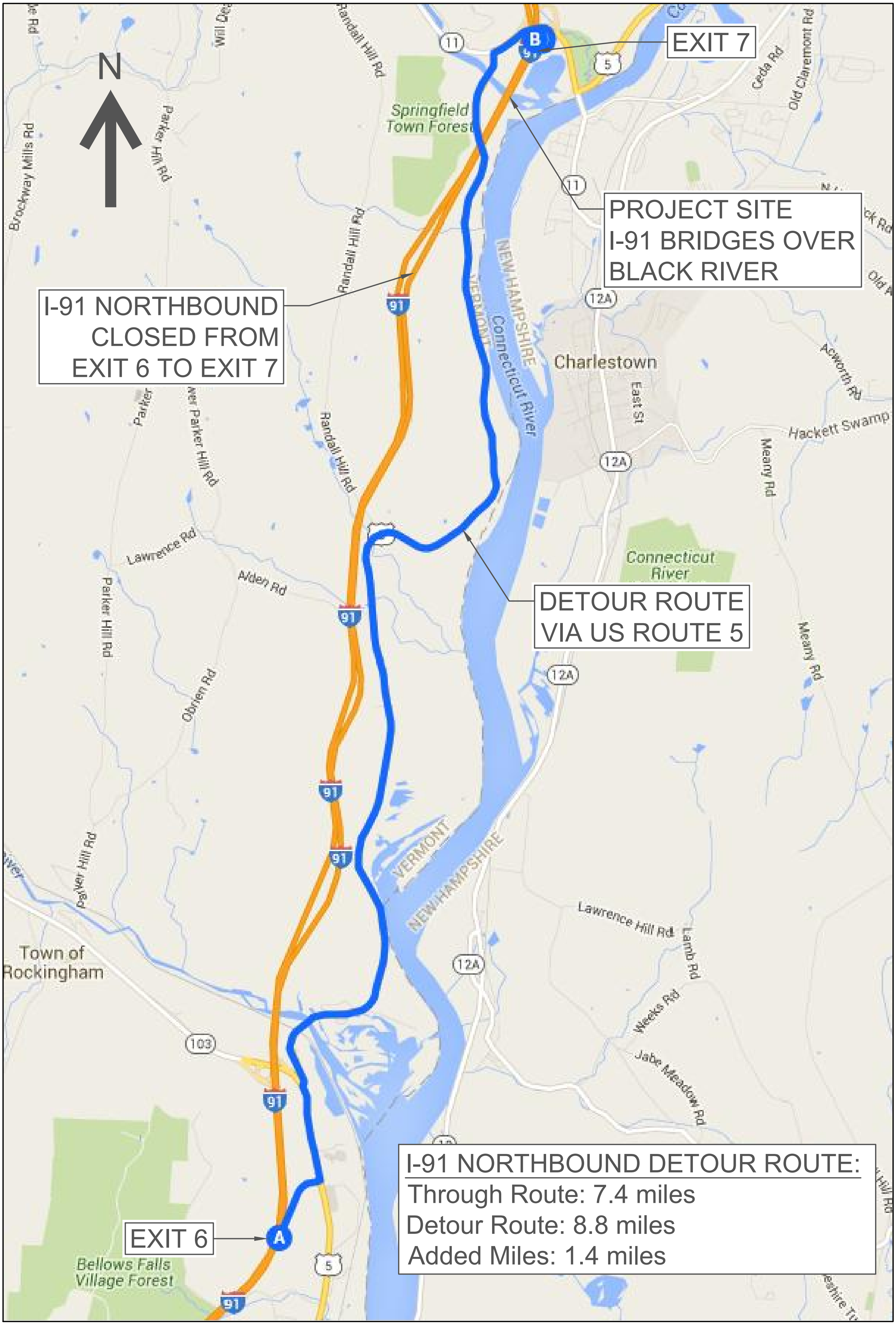
addressing the stability and force reduction concerns. Thus the initial analysis would consider the structure in its existing condition and then in the condition of the superstructure isolated from the substructure with reinforced elastomeric bearings designed as isolation bearings. We have performed a preliminary design on a set of reinforced elastomeric bearings as isolation bearings for the structure and find that they are feasible.

Additional measures would be to strengthen the pier stem if necessary by adding a reinforced concrete jacket around the stem to the height necessary for the required additional capacity in the longitudinal direction. Also, a common problem for older structures is a lack of development length of stem or column reinforcement into caps and footings. A detailed analysis can be performed to determine if the development is adequate for the demands. The pin and hanger details can be addressed by making the structure continuous to improve its overall performance and significantly reduce the likelihood of a loss of support failure, or seismic restrainers such as cables can be added at the hanger locations. The footings would be checked and additional micro-piles could be added to the foundation if necessary.

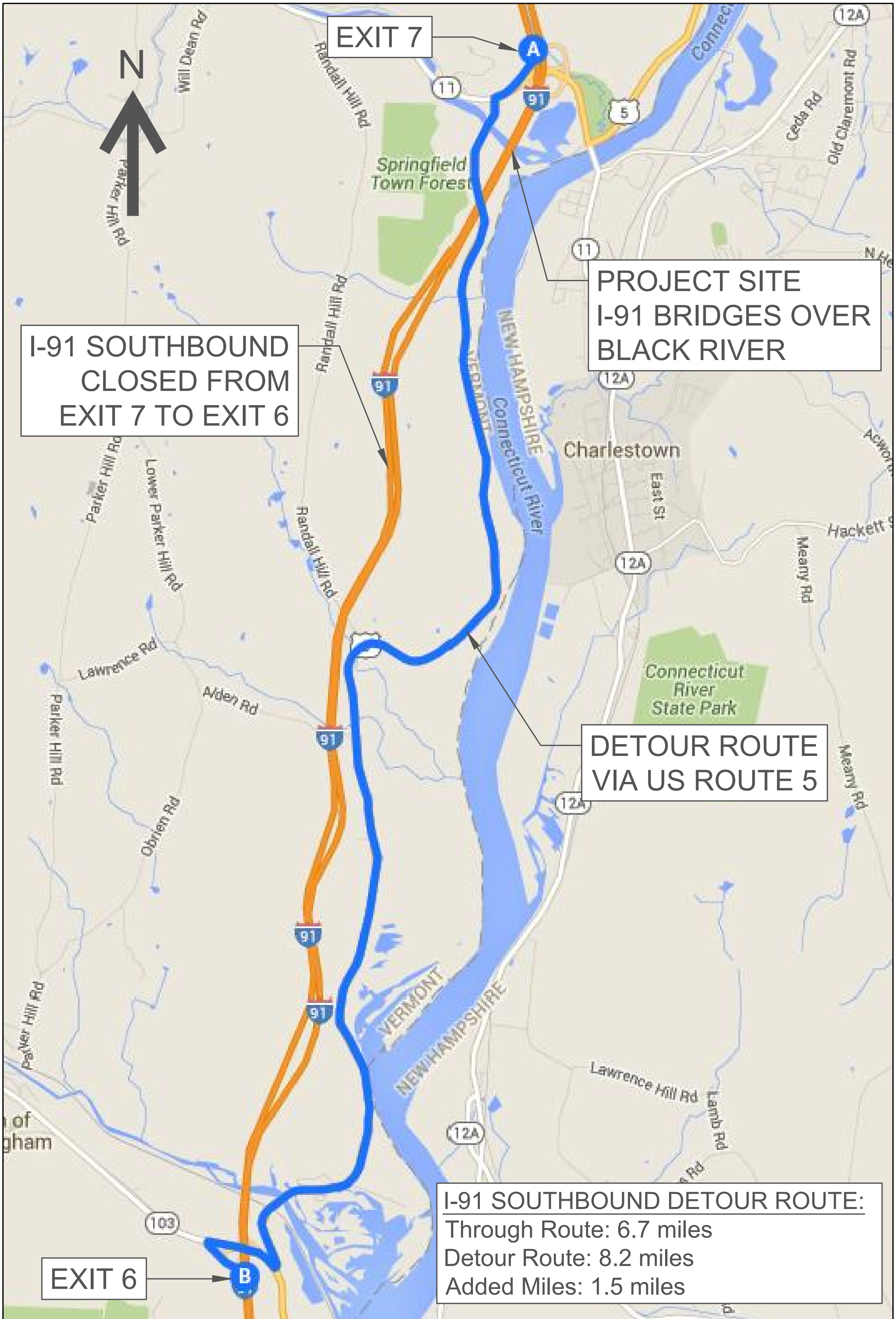
The abutments would be checked for impact from the superstructure on the backwalls in the longitudinal direction and shear blocks could be added to restrain the structure in the transverse direction. More damage can be accepted at the abutments as they are easier to repair after an earthquake and a loss of support failure is less likely. Conversely the abutments could also be upgraded if necessary to fully resist seismic loads.

APPENDIX

8.14. Detours



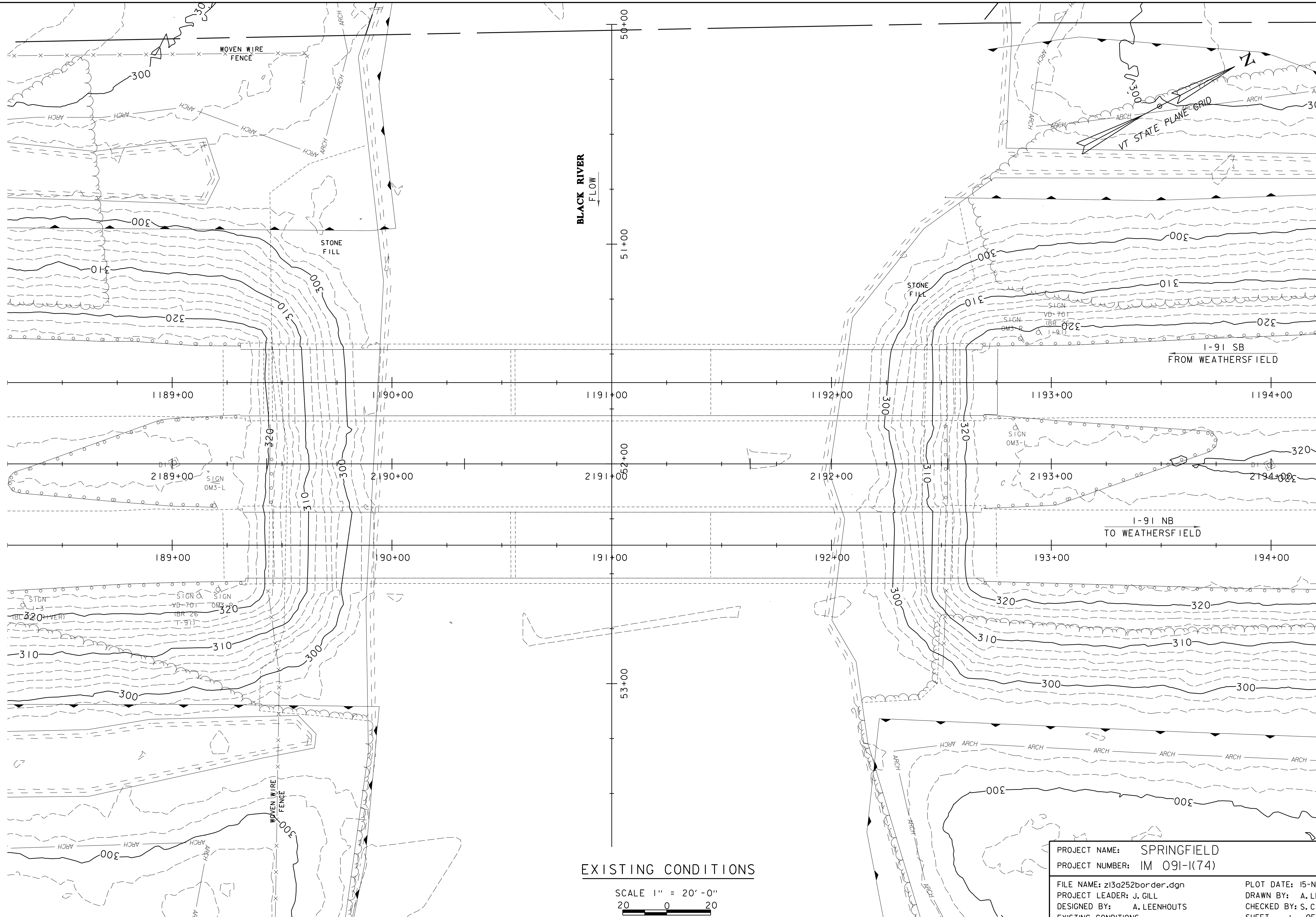
N
EXIT 7
PROJECT SITE
I-91 BRIDGES OVER
BLACK RIVER
I-91 NORTHBOUND
CLOSED FROM
EXIT 6 TO EXIT 7
DETOUR ROUTE
VIA US ROUTE 5
EXIT 6
I-91 NORTHBOUND DETOUR ROUTE:
Through Route: 7.4 miles
Detour Route: 8.8 miles
Added Miles: 1.4 miles
Springfield
Town Forest
Charlestown
Connecticut River
NEW HAMPSHIRE
VERMONT
Connecticut
River
Town of
Rockingham
Bellows Falls
Village Forest
Randall Hill Rd
Parker Hill Rd
Brockway Mills Rd
Lawrence Rd
Alden Rd
Obrien Rd
East St
Hackett Swamp
Meany Rd
Acworth Rd
Old Claremont Rd
Lawrence Hill Rd
Lamb Rd
Weeks Rd
Jabe Meadow Rd



EXIT 7
N
PROJECT SITE
I-91 BRIDGES OVER
BLACK RIVER
I-91 SOUTHBOUND
CLOSED FROM
EXIT 7 TO EXIT 6
DETOUR ROUTE
VIA US ROUTE 5
EXIT 6
I-91 SOUTHBOUND DETOUR ROUTE:
Through Route: 6.7 miles
Detour Route: 8.2 miles
Added Miles: 1.5 miles
Springfield Town Forest
Charlestown
Connecticut River State Park
Connecticut River
NEW HAMPSHIRE
VERMONT
Randall Hill Rd
Parker Hill Rd
Lower Parker Hill Rd
Lawrence Rd
Alden Rd
Obrien Rd
Will Dean Rd
East St
Meany Rd
Hackett
Lawrence Hill Rd
Lamb Rd
Ceda Rd
Old Claremont Rd

APPENDIX

8.15. Plans



EXISTING CONDITIONS

SCALE 1" = 20'-0"
20 0 20

PROJECT NAME: SPRINGFIELD
PROJECT NUMBER: IM 091-1(74)

FILE NAME: z13a252border.dgn	PLOT DATE: 15-NOV-16
PROJECT LEADER: J. GILL	DRAWN BY: A. LEENHOUTS
DESIGNED BY: A. LEENHOUTS	CHECKED BY: S. CARPENTER
EXISTING CONDITIONS	SHEET 1 OF 17

I-91 NORTHBOUND EXISTING/PROPOSED PROFILE

SCALE: HORIZONTAL 1"=20'-0"
VERTICAL 1"=10'-0"

I-91 SOUTHBOUND EXISTING/PROPOSED PROFILE

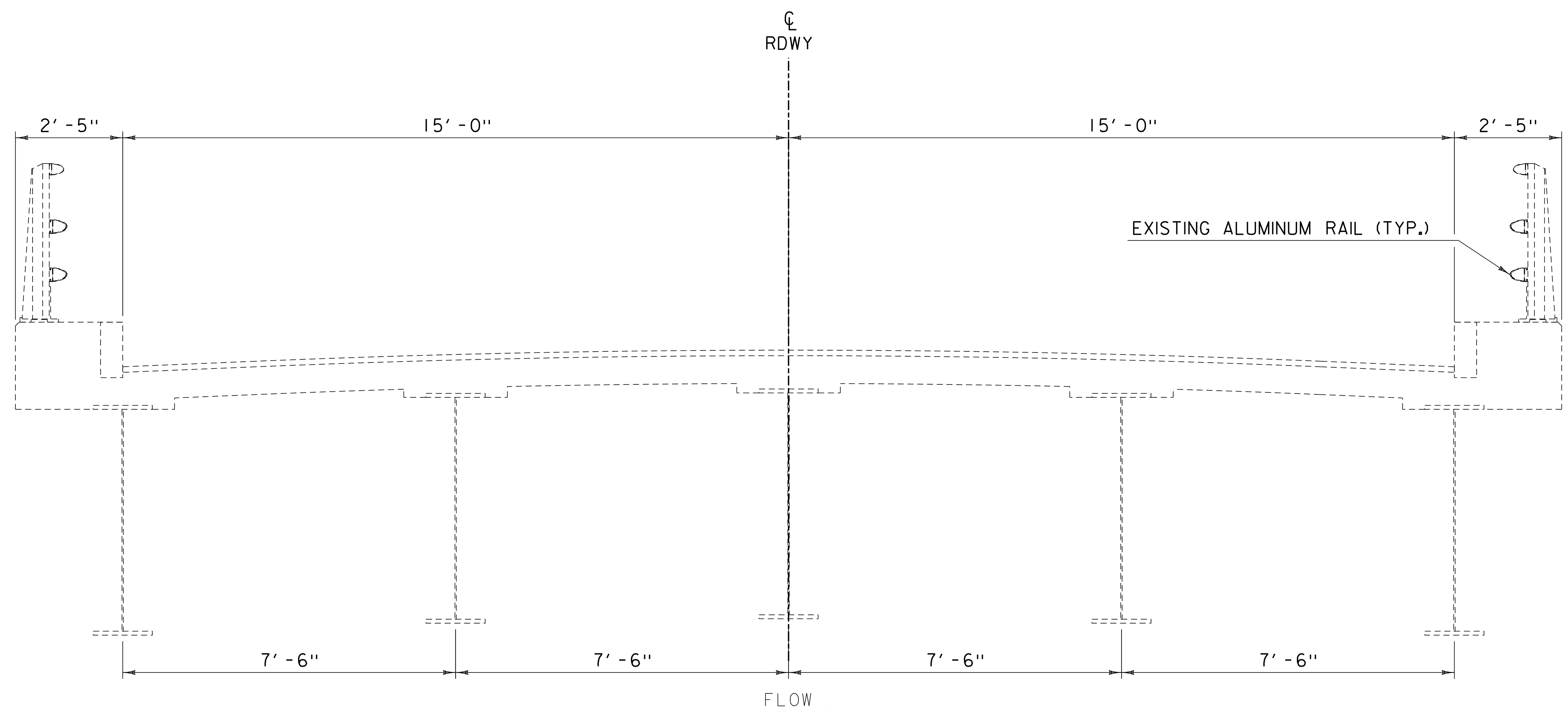
SCALE: HORIZONTAL 1"=20'-0"
VERTICAL 1"=10'-0"

NOTE: EXISTING GROUND IS FROM LIDAR DATA.
TOP OF WATER IS SHOWN

NOTE:
GRADES SHOWN TO THE NEAREST TENTH ARE EXISTING GROUND ALONG ℄
GRADES SHOWN TO THE NEAREST HUNDREDTH ARE FINISH GRADE ALONG ℄

PROJECT NAME: SPRINGFIELD
PROJECT NUMBER: IM 091-1(74)

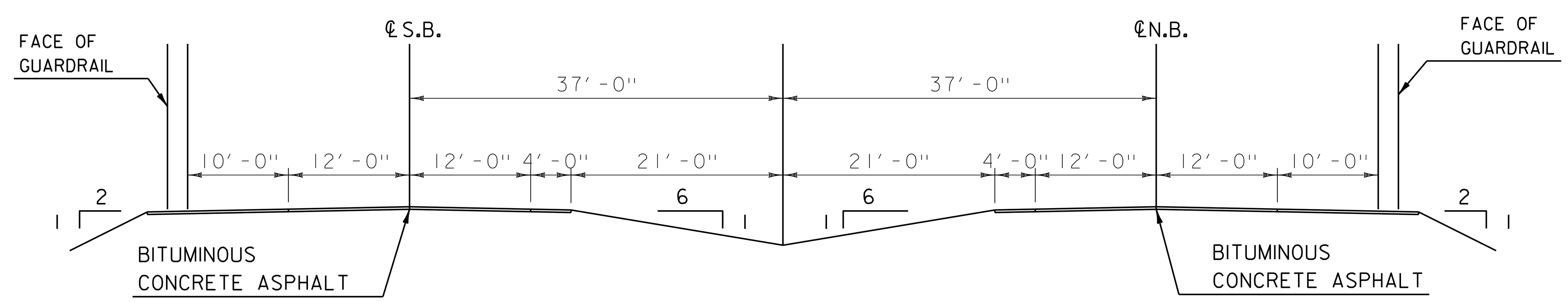
FILE NAME: zl3a252profile.dgn	PLOT DATE: 15-NOV-16
PROJECT LEADER: C.P. WILLIAMS	DRAWN BY: D.D. BEARD
DESIGNED BY: ------	CHECKED BY: ------
EXISTING & PROPOSED PROFILES	SHEET 2 OF 17



TYPICAL EXISTING BRIDGE SECTION

SCALE: $\frac{1}{2}$" = 1'-0"

PROJECT NAME: SPRINGFIELD	
PROJECT NUMBER: IM 091-1(74)	
FILE NAME: z13a252typbdr.dgn	PLOT DATE: 15-Nov-16
PROJECT LEADER: J. GILL	DRAWN BY: J. VELEZ
DESIGNED BY: A. SPERA	CHECKED BY: P. HUCKABEE
EXISTING SECTION	SHEET 3 OF 17



PROPOSED I-91 TYPICAL SECTION
SCALE: 1" = 10'-0"

PROJECT NAME:	SPRINGFIELD
PROJECT NUMBER:	IM 091-1(74)

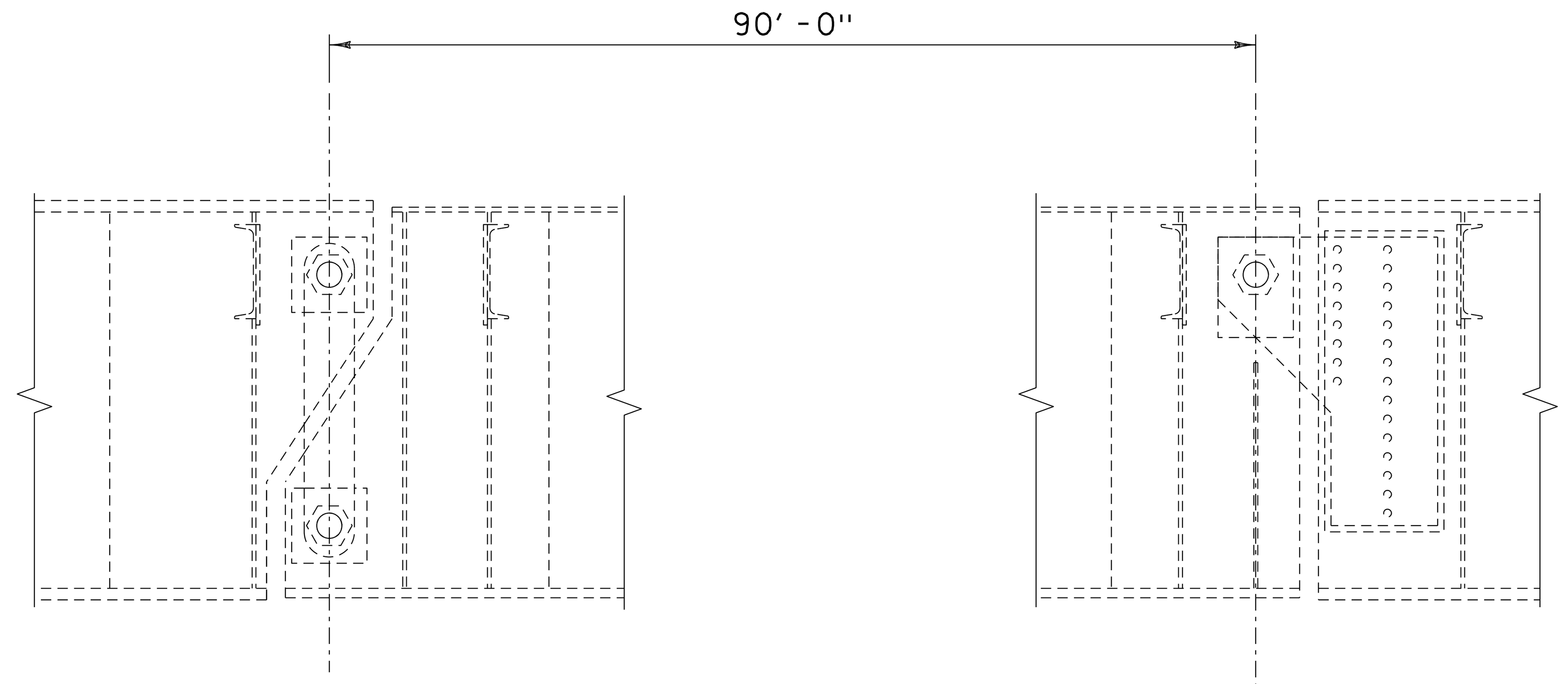
FILE NAME: z13a252typbdr.dgn	PLOT DATE: 15-Nov-16
PROJECT LEADER: J. GILL	DRAWN BY: A. LEENHOUTS
DESIGNED BY: A. LEENHOUTS	CHECKED BY: S. CARPENTER
TYPICAL SECTION	SHEET 4 OF 17

PROPOSED LAYOUT

SCALE 1" = 20'-0"

20 0 20

PROJECT NAME: SPRINGFIELD	
PROJECT NUMBER: IM 091-1(74)	
FILE NAME: z13a252border.dgn	PLOT DATE: 15-NOV-16
PROJECT LEADER: J. GILL	DRAWN BY: A. LEENHOUTS
DESIGNED BY: A. LEENHOUTS	CHECKED BY: S. CARPENTER
PROPOSED LAYOUT	SHEET 5 OF 17

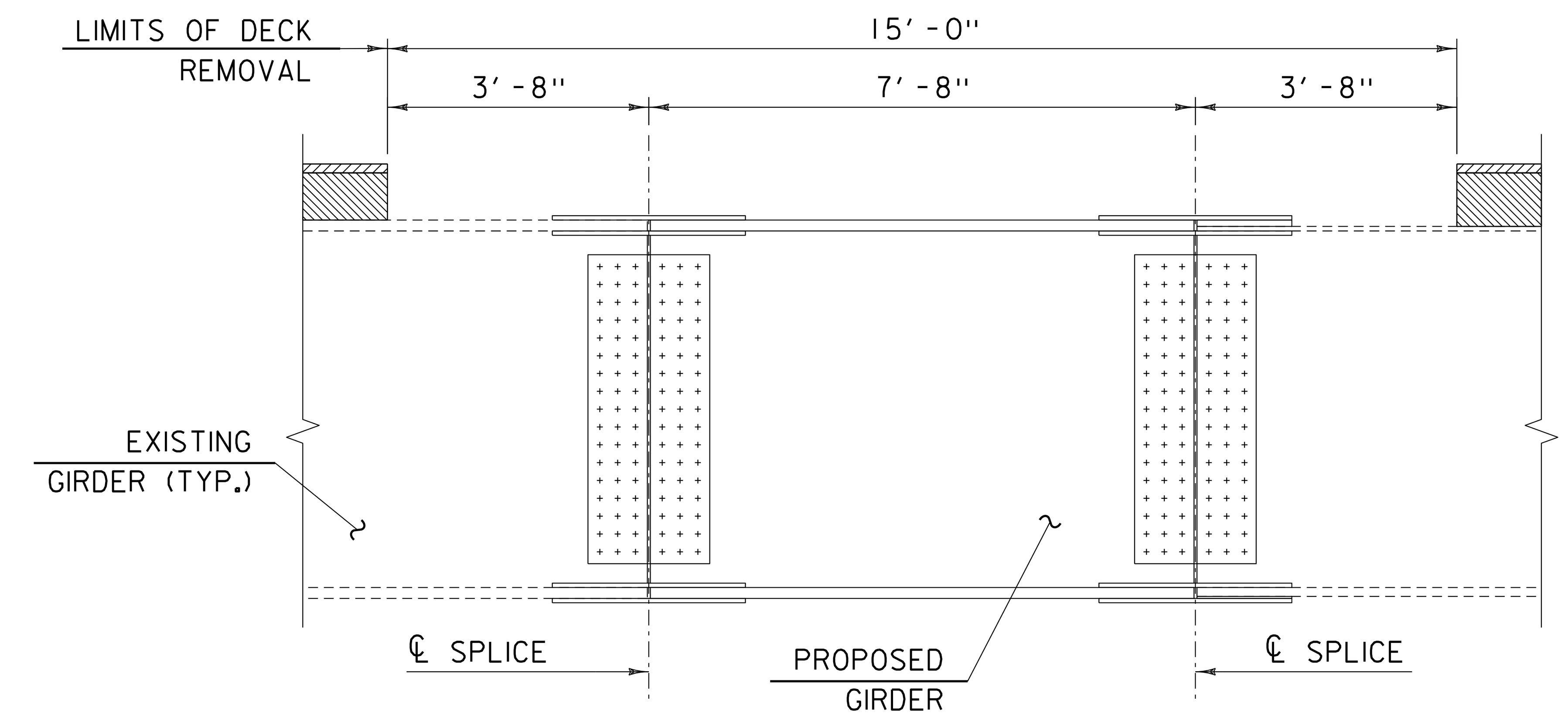


EXISTING HINGE

SCALE: 1/2" = 1'-0"

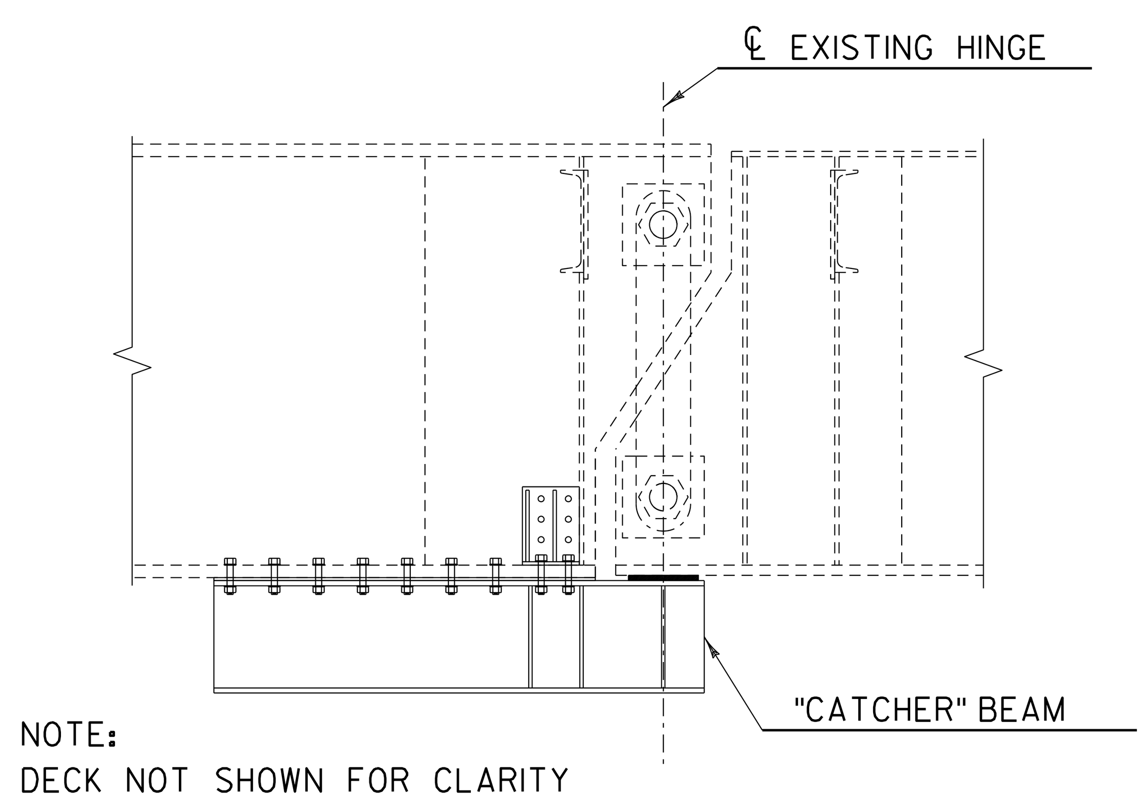
EXISTING PIN SUPPORT

SCALE: 1/2" = 1'-0"



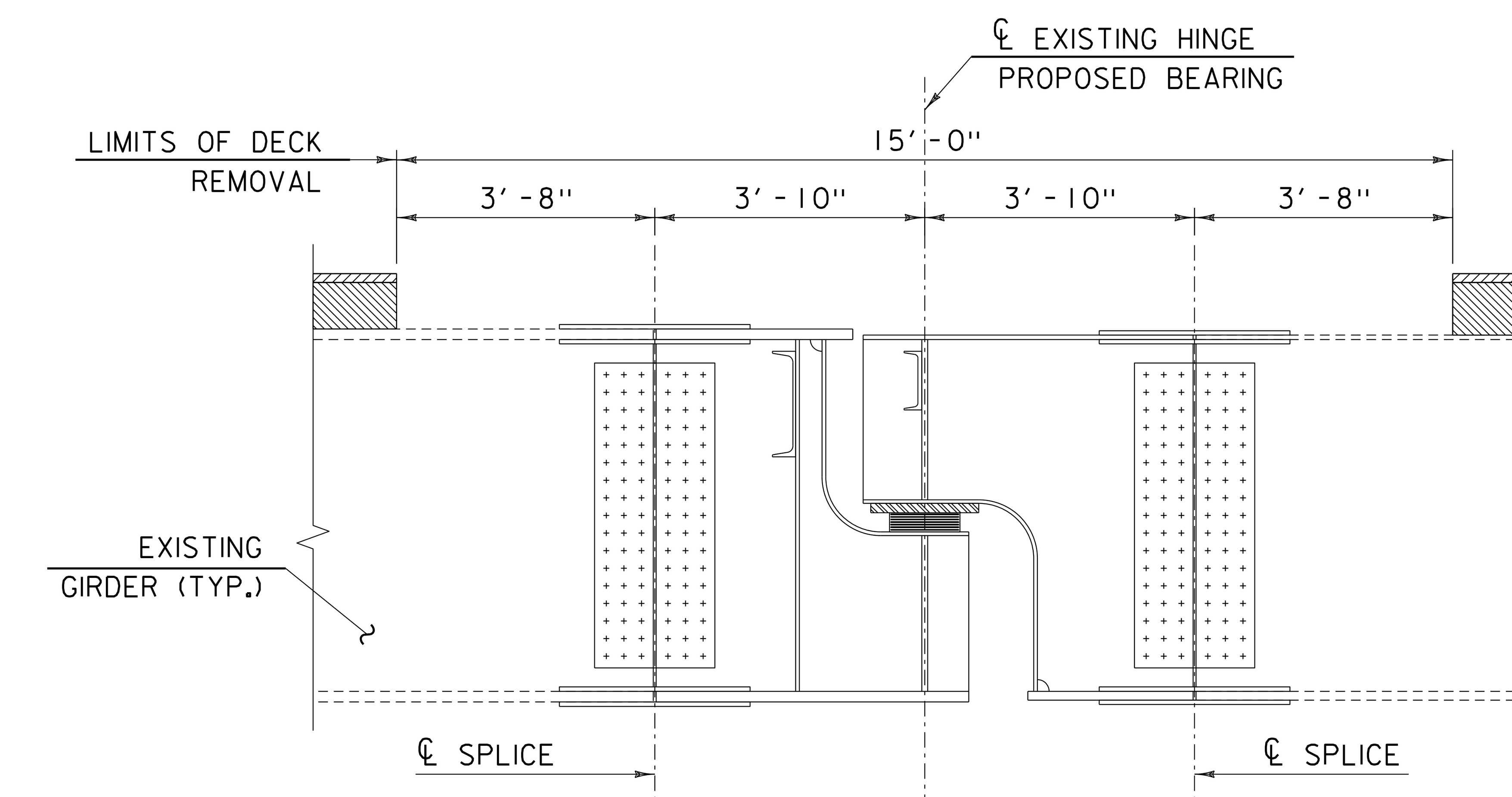
PIN AND HANGER "CONTINUOUS SPLICE" RETROFIT

SCALE: 1/2" = 1'-0"



PIN AND HANGER "CATCHER'S MITT" RETROFIT

SCALE: 1/2" = 1'-0"

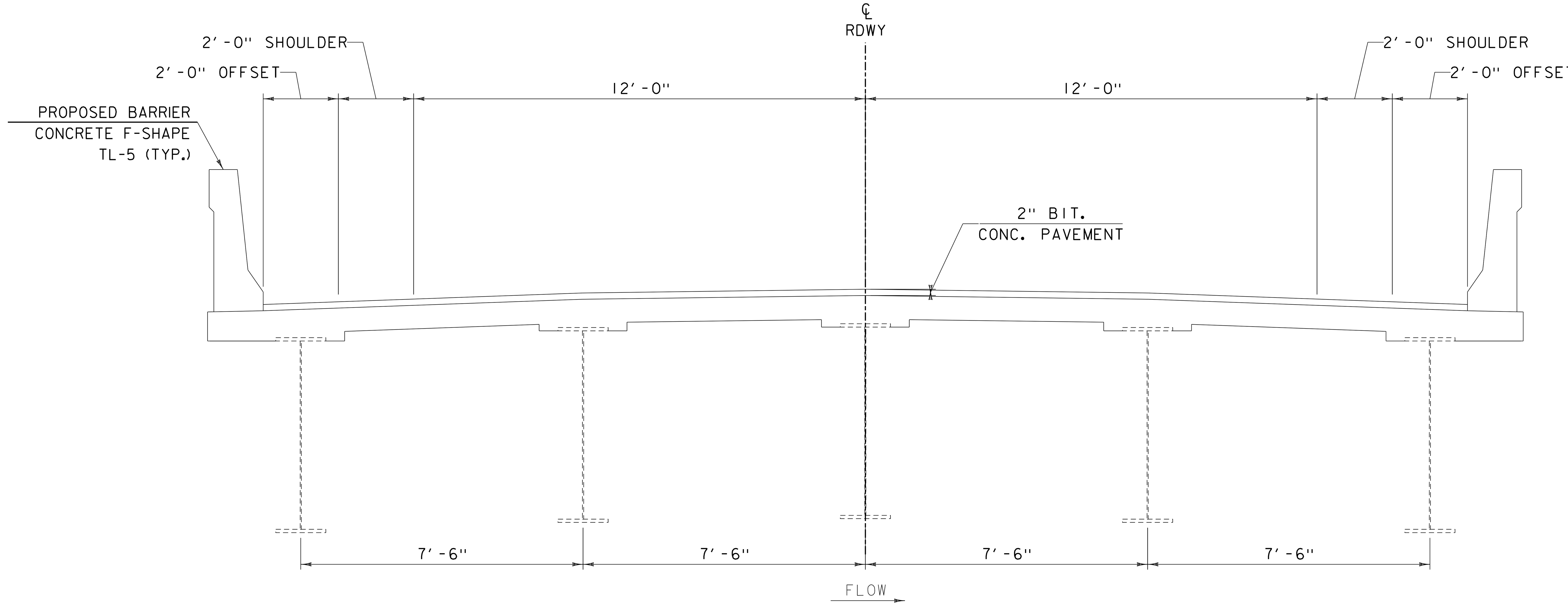


PIN AND HANGER "SEATED GIRDER" RETROFIT

SCALE: 1/2" = 1'-0"

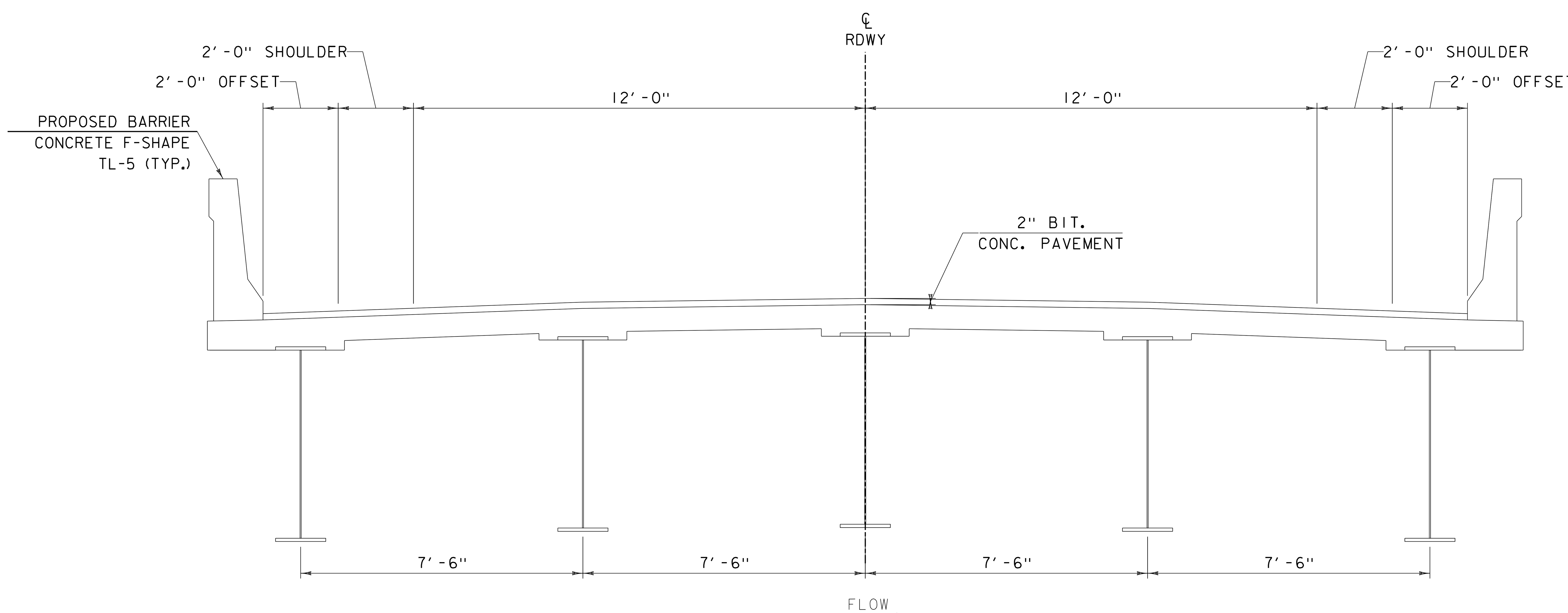
PIN AND HANGER RETROFIT

PROJECT NAME: SPRINGFIELD	
PROJECT NUMBER: IM 091-1(74)	
FILE NAME: z13a252supbdr.dgn	PLOT DATE: 15-Nov-16
PROJECT LEADER: J. GILL	DRAWN BY: J. VELEZ
DESIGNED BY: A. SPERA	CHECKED BY: P. HUCKABEE
PIN & HANGER RETROFIT - SECTION	SHEET 6 OF 17



TYPICAL PROPOSED BRIDGE SECTION - DECK REPLACEMENT (ALT. 1)

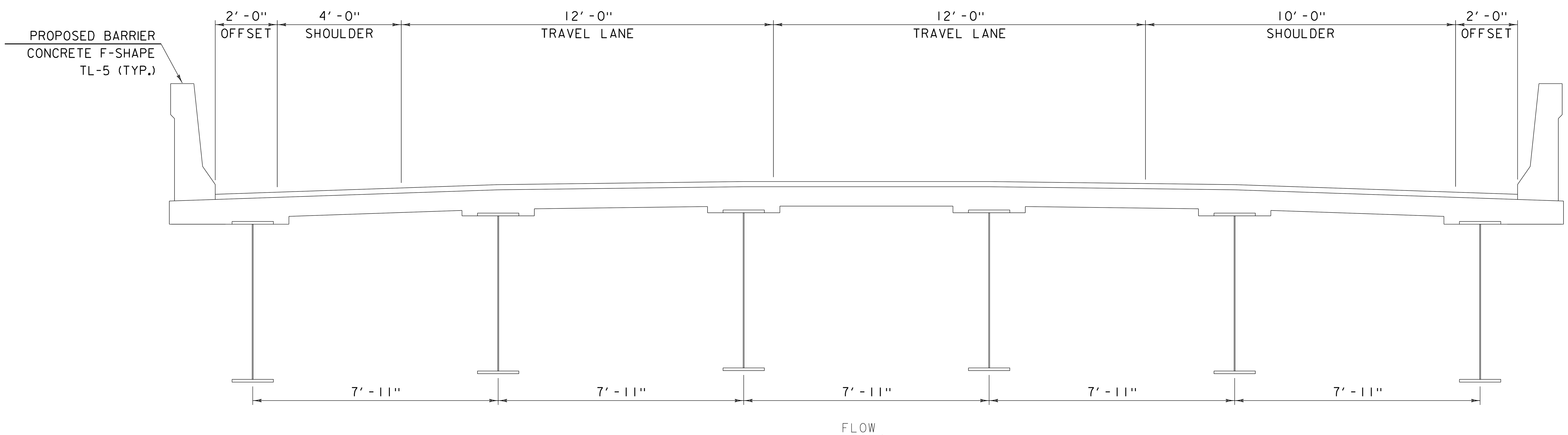
SCALE: 1/2" = 1'-0"



TYPICAL PROPOSED BRIDGE SECTION - SUPERSTRUCTURE REPLACEMENT (ALT. 2)

SCALE: 1/2" = 1'-0"

PROJECT NAME: SPRINGFIELD	
PROJECT NUMBER: IM 091-1(74)	
FILE NAME: z13a252typbdr.dgn	PLOT DATE: 15-Nov-16
PROJECT LEADER: J. GILL	DRAWN BY: J. VELEZ
DESIGNED BY: A. SPERA	CHECKED BY: P. HUCKABEE
ALTERNATIVE SECTION 1 & 2	SHEET 7 OF 17



TYPICAL PROPOSED BRIDGE SECTION - BRIDGE REPLACEMENT (ALT. 3)

SCALE: 1/2" = 1'-0"

PROJECT NAME: SPRINGFIELD	
PROJECT NUMBER: IM 091-1(74)	
FILE NAME: z13a252typbdr.dgn	PLOT DATE: 15-Nov-16
PROJECT LEADER: J. GILL	DRAWN BY: J. VELEZ
DESIGNED BY: A. SPERA	CHECKED BY: P. HUCKABEE
ALTERNATIVE 3 SECTION	SHEET 8 OF 17

PHASED CONSTRUCTION

PHASE I

SCALE 1" = 20'-0"

20 0 20

PROJECT NAME: SPRINGFIELD	
PROJECT NUMBER: IM 091-1(74)	
FILE NAME: z13a252TCborder1.dgn	PLOT DATE: 15-NOV-16
PROJECT LEADER: J. GILL	DRAWN BY: A. LEENHOUTS
DESIGNED BY: A. LEENHOUTS	CHECKED BY: S. CARPENTER
PHASED CONSTRUCTION PHASE I	SHEET 9 OF 17

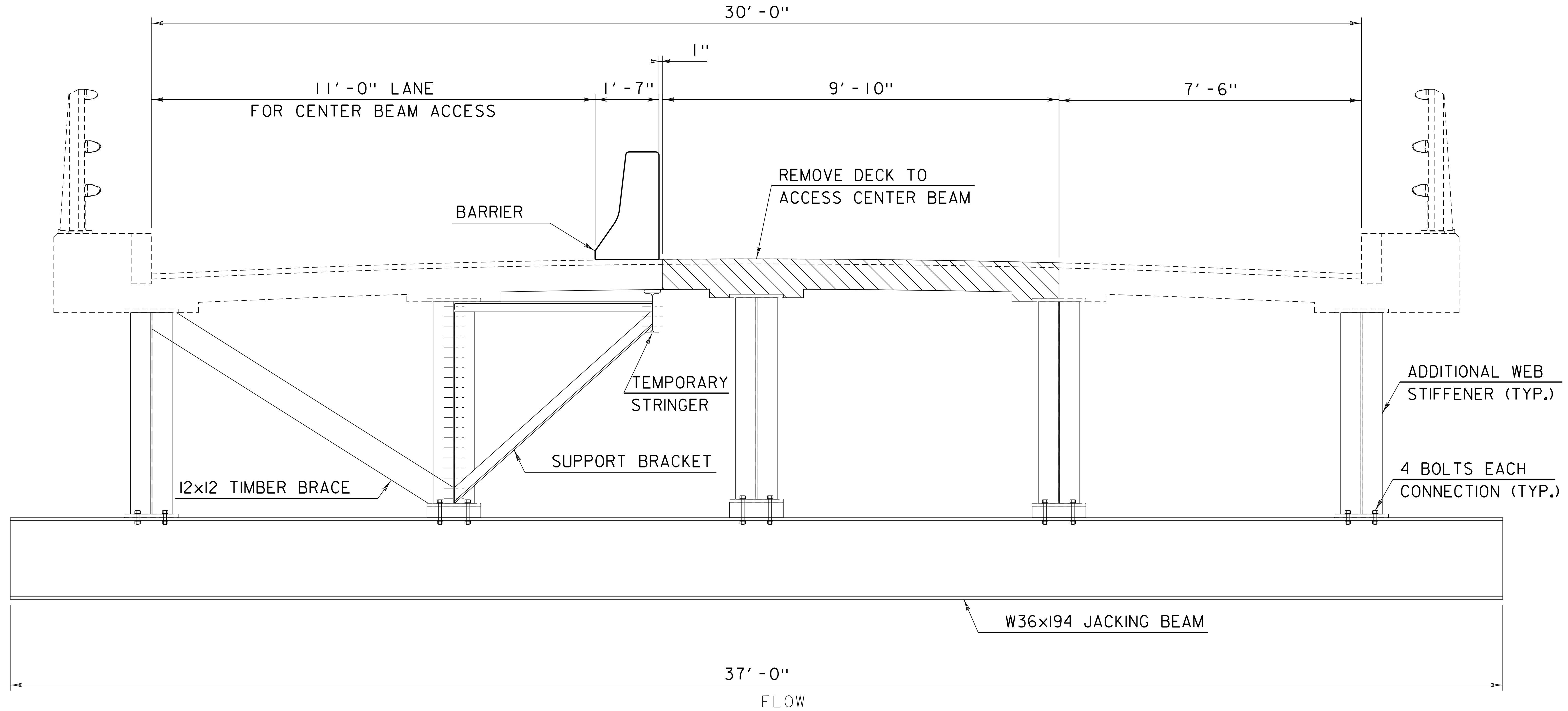
PHASED CONSTRUCTION

PHASE 2

SCALE 1" = 20'-0"

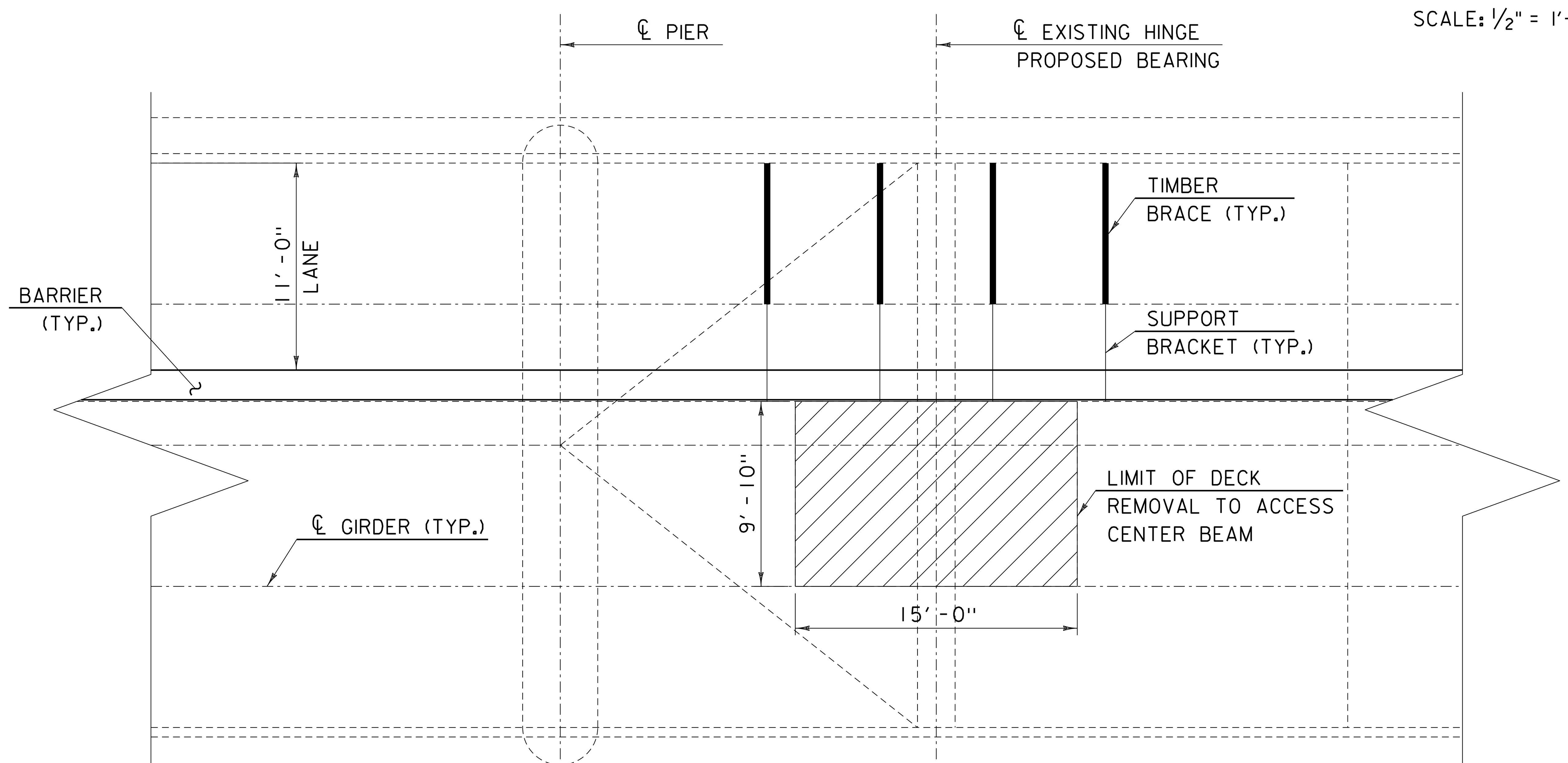
20 0 20

PROJECT NAME: SPRINGFIELD	
PROJECT NUMBER: IM 091-1(74)	
FILE NAME: z13a252TCborder1.dgn	PLOT DATE: 15-NOV-16
PROJECT LEADER: J. GILL	DRAWN BY: A. LEENHOUTS
DESIGNED BY: A. LEENHOUTS	CHECKED BY: S. CARPENTER
PHASED CONSTRUCTION PHASE 2	SHEET 10 OF 17



BRIDGE CROSS-SECTION

SCALE: 1/2" = 1'-0"



NOTE:
FOR OTHER BEAMS, ALIGN BARRIER AT CENTERLINE AND REMOVE DECK AND BARRIER FOR ACCESS TO THE SAME LONGITUDINAL LIMITS (15'-0")

REPAIR LOCATION PLAN VIEW

SCALE: 3/16" = 1'-0"

PHASED CPNSTRUCTION (PIN AND HANGER SEATED GIRDER OR CONTINUOUS SPLICE)

PROJECT NAME: SPRINGFIELD	
PROJECT NUMBER: IM 091-1(74)	
FILE NAME: z13a252supbdr.dgn	PLOT DATE: 15-Nov-16
PROJECT LEADER: J. GILL	DRAWN BY: J. VELEZ
DESIGNED BY: A. SPERA	CHECKED BY: P. HUCKABEE
PIN & HANGER RETROFIT - PHASED CONSTR.	SHEET 11 OF 17

MEDIAN CROSSOVERS

LAYOUT 1

SCALE 1" = 20'-0"

20 0 20

PROJECT NAME: SPRINGFIELD

PROJECT NUMBER: IM 091-1(74)

FILE NAME: z13a252TCborder2.dgn

PROJECT LEADER: J. GILL

DESIGNED BY: A. LEENHOUTS

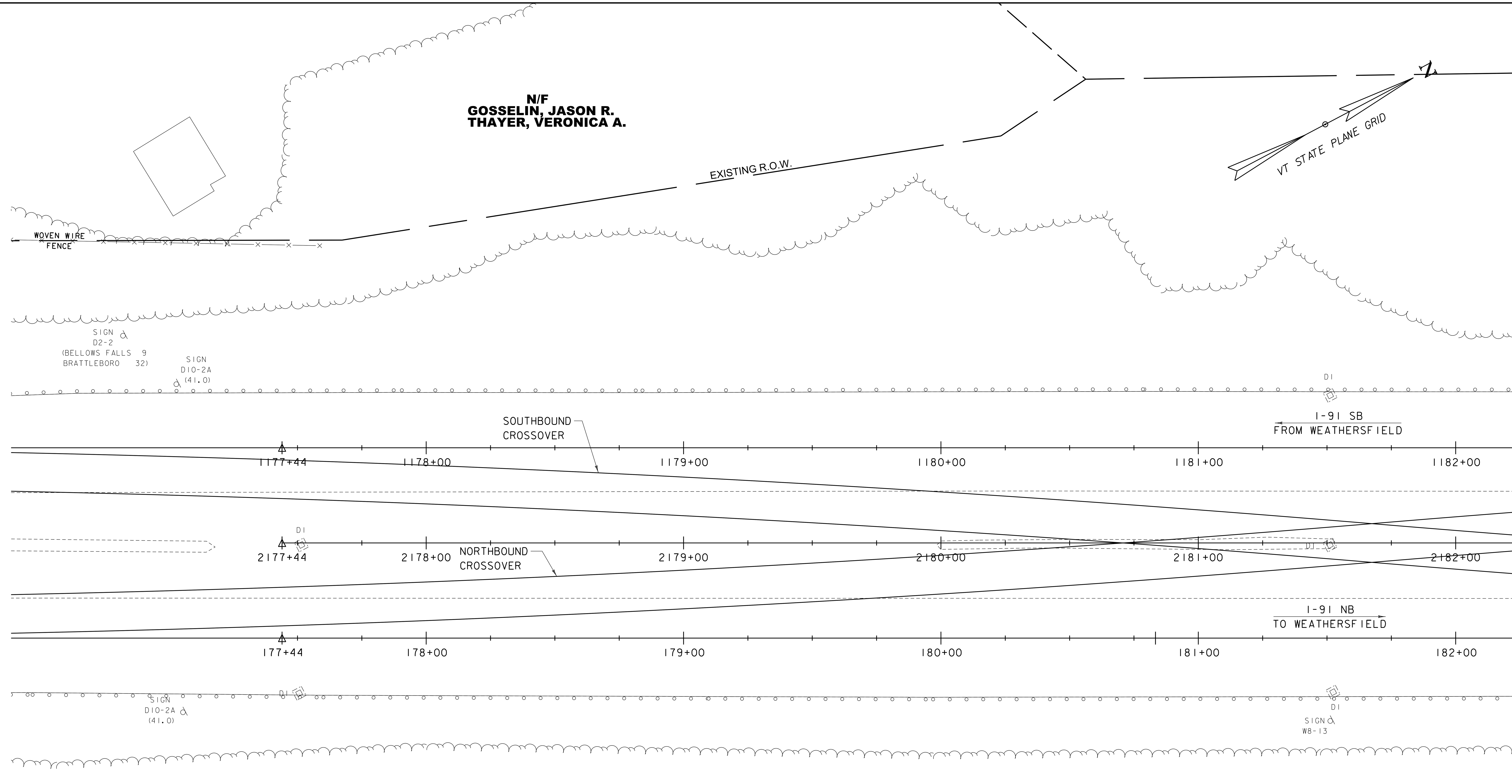
MEDIAN CROSSOVER LAYOUT 1

PLOT DATE: 15-NOV-16

DRAWN BY: A. LEENHOUTS

CHECKED BY: S. CARPENTER

SHEET 12 OF 17



MEDIAN CROSSOVERS

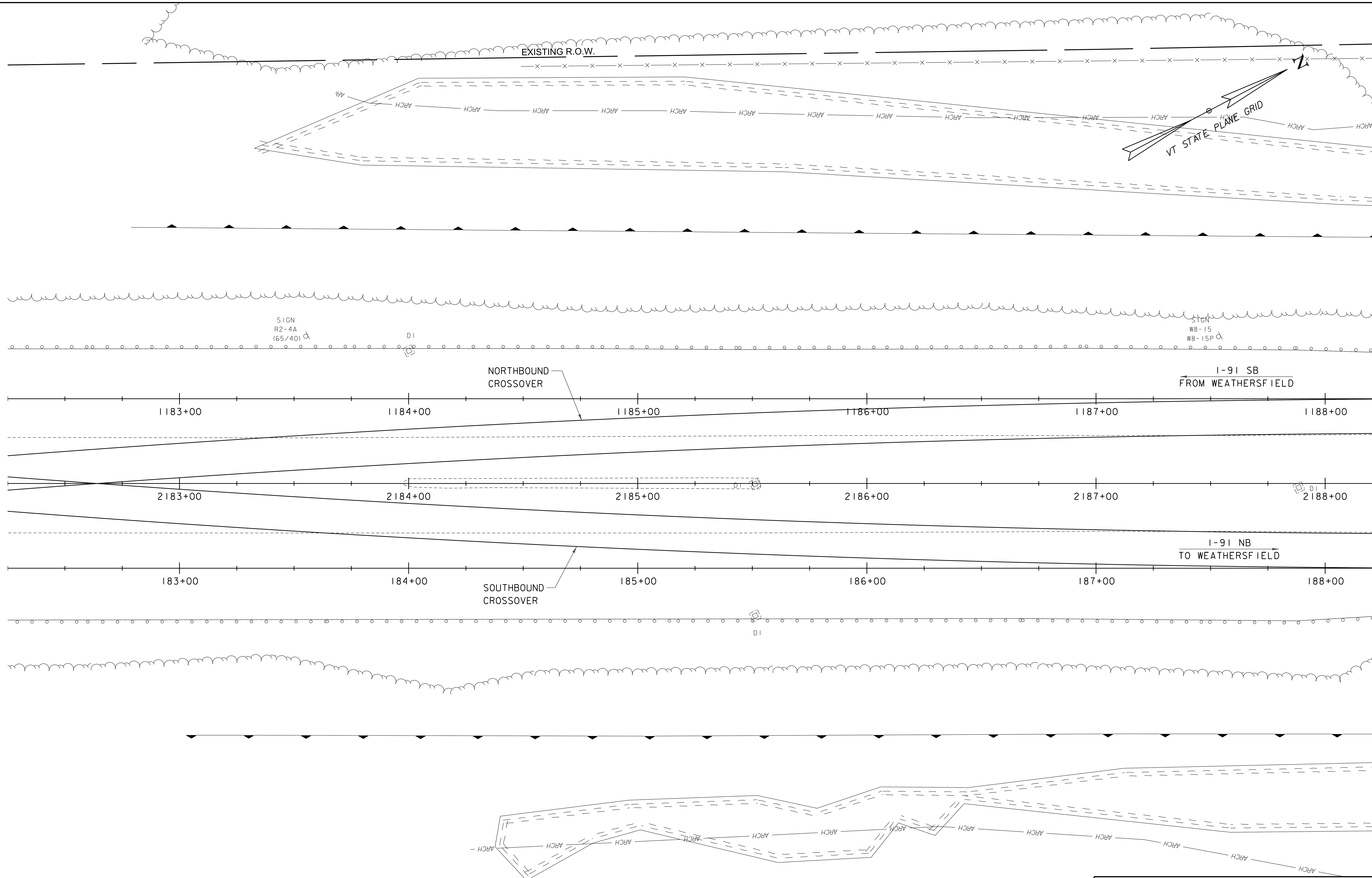
LAYOUT 2

SCALE 1" = 20'-0"

20 0 20

PROJECT NAME:	SPRINGFIELD
PROJECT NUMBER:	IM 091-1(74)

FILE NAME: z13a252TCborder2.dgn	PLOT DATE: 15-NOV-16
PROJECT LEADER: J. GILL	DRAWN BY: A. LEENHOUTS
DESIGNED BY: A. LEENHOUTS	CHECKED BY: S. CARPENTER
MEDIAN CROSSOVER LAYOUT 2	SHEET 13 OF 17



MEDIAN CROSSOVERS

LAYOUT 3

SCALE 1" = 20'-0"

20 0 20

PROJECT NAME: SPRINGFIELD

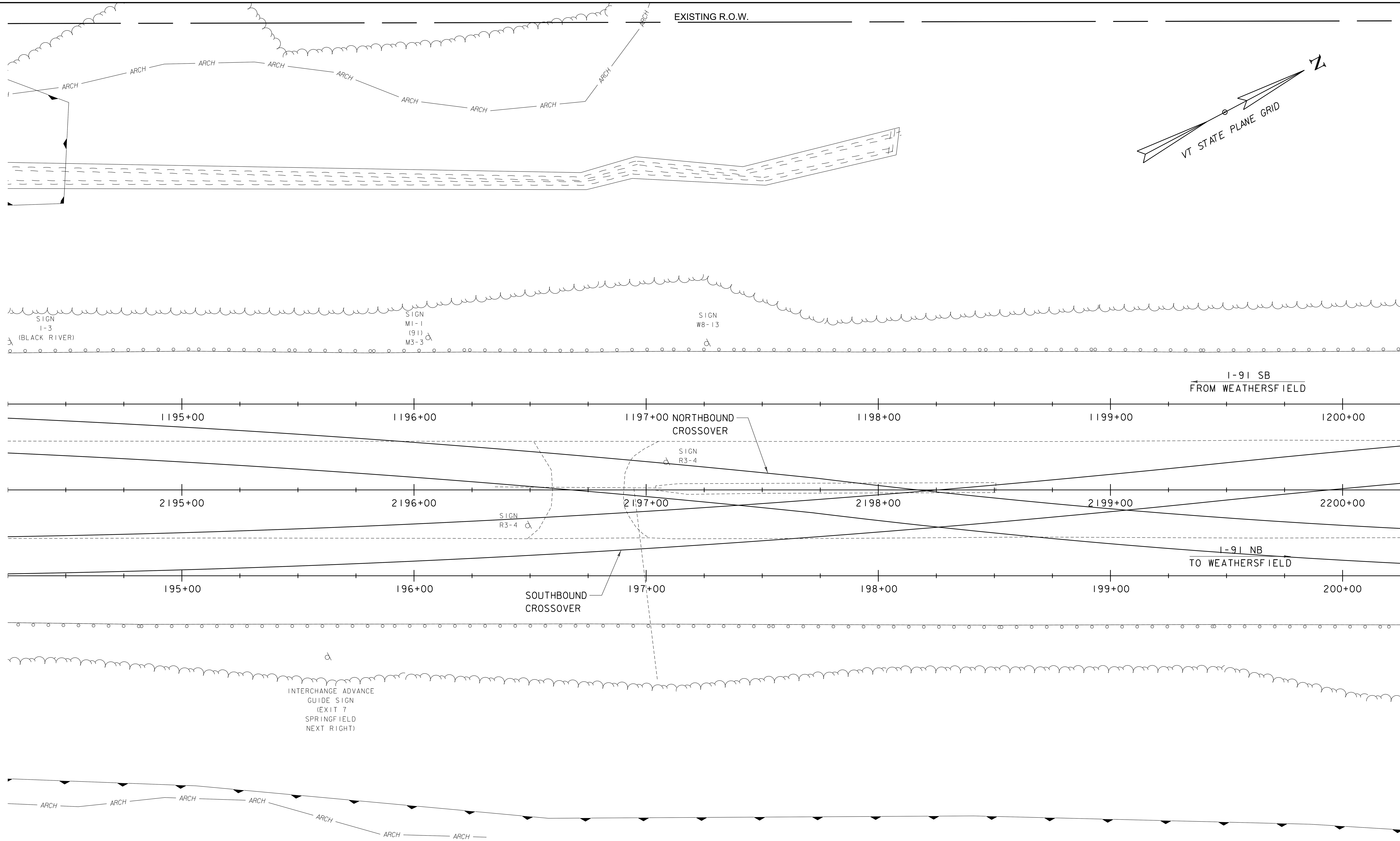
PROJECT NUMBER: IM 091-1(74)

FILE NAME: z13a252TCborder2.dgn	PLOT DATE: 15-NOV-16
PROJECT LEADER: J. GILL	DRAWN BY: A. LEENHOUTS
DESIGNED BY: A. LEENHOUTS	CHECKED BY: S. CARPENTER
MEDIAN CROSSOVER LAYOUT 3	SHEET 14 OF 17

PROJECT NAME: SPRINGFIELD
PROJECT NUMBER: IM 091-1(74)

FILE NAME: z13a252TCborder2.dgn
PROJECT LEADER: J. GILL
DESIGNED BY: A. LEENHOUTS
MEDIAN CROSSOVER LAYOUT 4

PLOT DATE: 15-NOV-16
DRAWN BY: A. LEENHOUTS
CHECKED BY: S. CARPENTER
SHEET 15 OF 17



MEDIAN CROSSOVERS

LAYOUT 5

SCALE 1" = 20'-0"

20 0 20

PROJECT NAME: SPRINGFIELD

PROJECT NUMBER: IM 091-1(74)

FILE NAME: z13a252TCborder2.dgn
PROJECT LEADER: J. GILL
DESIGNED BY: A. LEENHOUTS
MEDIAN CROSSOVER LAYOUT 5

PLOT DATE: 15-Nov-16
DRAWN BY: A. LEENHOUTS
CHECKED BY: S. CARPENTER
SHEET 16 OF 17

MEDIAN CROSSOVERS

LAYOUT 6

SCALE 1" = 20'-0"

20 0 20

PROJECT NAME: SPRINGFIELD
PROJECT NUMBER: IM 091-1(74)

FILE NAME: z13a252TCborder2.dgn
PROJECT LEADER: J. GILL
DESIGNED BY: A. LEENHOUTS
MEDIAN CROSSOVER LAYOUT 6

PLOT DATE: 15-Nov-16
DRAWN BY: A. LEENHOUTS
CHECKED BY: S. CARPENTER
SHEET 17 OF 17