Geotechnical Engineering Report

Bennington BF 1000(20)
VT Rt 9, Bridge 6 Over Walloomsac
Bennington, Vermont

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Sign-off Sheet

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1.0 Introduction

This report presents the results of the geotechnical exploration and analysis for the replacement of Bridge No. 6 over the Walloomsac River located in Bennington, Vermont. The geotechnical scope of work consisted of drilling test borings, evaluating the subsurface conditions, and providing geotechnical engineering recommendations for the support of the proposed bridge design. A subsurface test boring program consisting of three test borings was drilled to evaluate the soil and rock conditions in the area of the proposed bridge.

Geotechnical design recommendations were made for the proposed bridge using the following documents:

- *VTrans Structures Design Manual, Fifth Edition* (VTrans Structures Design Manual); and
- *AASHTO LRFD Bridge Design Specifications, 8th Edition/2017* (AASHTO 2017).

Background information was provided in the following documents:

- Conceptual bridge plans, entitled *Proposed Improvement Bridge Replacement Project, Town of Bennington, County of Bennington, Bridge No. 6* prepared by Stantec and dated April 21, 2020; and
- Two sheets from the design plans for the existing bridge, dated August 12, 1923 (1923 Bridge Plans).

Elevations in this report are in feet and referenced to the vertical datum NAVD88. The horizontal datum is NAD (83) 2007.

2.0 Site and Project Information

The existing bridge is a single-span structure located on VT Route 9 approximately 0.5 miles east of the intersection with VT Route 7. The existing bridge carries VT Route 9 over the Walloomsac River. The river flows from north to south and the bridge spans the river in an east to west direction. In the area of the bridge, the bottom of the river channel ranges from approximately El. 721 to El 722. The existing bridge consists of the reinforced concrete deck supported by concrete abutments. The 1923 Bridge Plans indicate the abutments bear at El. 89.5 based on the datum used for the project. Based on the current datum (NAVD88) the existing abutments have been estimated to bear at El. 720. Based on the recently drilled test borings the existing abutments are soil supported. The clear span of the bridge is 42 feet. The width of the bridge is 59.67 feet. The bridge supports a two-lane roadway and a sidewalk on both sides. Wingwalls ranging from 9 to 10 feet long are located at each bridge corner and run parallel to the river. The northern side of the bridge supported trolley car tracks. A water line, sewer line and a telecommunication line are supported under the existing bridge deck. The 1923 Bridge Plans are provided in Appendix A.

We understand the existing bridge will be replaced with a new single-span structure consisting of precast concrete superstructure with a length of approximately 59 feet and width of approximately 62 feet. The site location is shown in Figure 1 entitled “Project Location Plan”.

3.0 Subsurface Information

3.1 LOCAL GEOLOGY

The site is located on the south side of the Walloomsac River valley. Based on the Surficial Geologic Map of the Bennington Area, Vermont (Vermont Geological Survey Open File Report VG2017-1:Plate 1) dated 2017, the area immediately adjacent to the river consists of alluvium. The alluvium is described as fine sand, silt, and gravel. The majority of the valley floor is covered with alluvial fan deposits consisting of gravel, silt, and sand, often poorly sorted. The edges of the valley consist of fluvial terrace deposits consisting of fine sand, silt, and gravel. The valley walls and hill tops are general covered with glacial till.

Based on the Bedrock Geology Map of Vermont dated 2011, the bedrock in the area of the site consists of well-bedded dolostone of the Winooski Dolostone Formation. Bedrock outcrops were not observed in the immediate area of the project site.

3.2 SUBSURFACE EXPLORATION

The exploration program consisted of three test borings drilled from October 12 to 15, 2020. The tests borings are designated as B-1 through B-3. Borings B-1 and B-2 were drilled in the area of the proposed Abutment No. 1 (west abutment). Boring B-3 was drilled in the area of the proposed Abutment No. 2 (east abutment). Boring B-2 encountered refusal a shallow depth and five attempts were made to advance the boring. The attempts where designated B-2A through B-2E. The locations of the test borings are shown on the attached Figure 2, entitled "Boring Location Plan."

The test borings were drilled by New England Boring of Derry, New Hampshire. A truck-mounted drill rig equipped with 4-inch diameter flush-joint steel casing or 4.25-inch inside diameter hollow stem augers was used to advance the borings through the soil overburden. Bedrock was cored at borings B-1 and B-3 using a NQDC double-walled core barrel.

Soil samples were obtained by driving a 24-inch long, 2-inch outside diameter split spoon sampler with a 140-pound safety hammer falling 30 inches, in substantial accordance with ASTM D1586, the Standard Penetration Test (SPT). The blows for each 6-inches of penetration are recorded for a total of 24-inches. The sum of the blows required to drive the sampler from 6-inches to 18-inches penetration is referred to as the Standard Penetration Resistance, or N-value, which is an index of measure of in-situ soil density or consistency. In accordance with VTrans practice, N values for granular soils less than 5 are considered to be very loose, between 5 and 10 loose; between 11 and 24 medium dense; between 25 and 50 dense; and greater than 50 very dense. The SPTs were conducted using a safety hammer driven with a rope and cathead; as such a value of 1.0 is recommended for the hammer energy correction factor (CE). Soil samples from the test borings were visually classified in the field by Stantec personnel. The boring logs include the visual descriptions. It should be noted that environmental samples were collected as part of the investigation and the majority of the soil retrieved from the split spoon sampler was sent for laboratory analytical testing. The locations were determined by measuring from existing site features. Boring logs are provided in Appendix B. A summary of the boring locations is provided in the table below.

Table 1 – Boring Locations and Elevations

Boring	Structure	Northing (ft)	Easting (ft)	Station (ft)	Offset (ft)	Grnd Elev. (ft)	Bedrock Elev. (ft)	Depth to Bedrock (ft)
B-1	Abut No. 1	138801.53	1456422.58	12+30.94	17.73 RT	730.49	716.49	14.0
B-2A	Abut No. 1	138826.55	1456403.72	12+18.86	11.17 LT	730.61	--	--
B-2B	Abut No. 1	138825.81	1456400.81	12+15.86	11.17 LT	730.56	--	--
B-2C	Abut No. 1	138823.87	1456401.31	12+15.86	9.17 RT	730.58	--	--
B-2D	Abut No. 1	138823.37	1456399.37	12+13.86	9.17 LT	730.56	--	--
B-2E	Abut No. 1	138825.10	1456406.15	12+20.86	9.17 LT	730.67	--	--
B-3	Abut No. 2	138810.02	1456515.30	13+23.48	32.24 RT	730.81	717.81	13.0

Notes: (1) Boring B-2 attempted at several locations but encountered an obstruction at shallow depths.

4.0 Summarized Subsurface Conditions

The subsurface conditions encountered are based on widely spaced explorations and variations in conditions should be anticipated. In general, the test borings encountered granular fill overlying a cobble/boulder deposit overlying bedrock. Subsurface conditions encountered are summarized in the following paragraphs:

4.1 ABUTMENT NO. 1 (WEST SIDE)

Two borings were drilled at Abutment No 1. B-1 was drilled through the overburden and 21 feet into the bedrock. B-2 encountered multiple shallow refusals on an unknown concrete structure. The following subsurface conditions were encountered at the location of B-1:

4.1.1 Pavement

The boring encountered 6 inches of asphalt pavement.

4.1.2 Fill Material

The fill was encountered from below the pavement to a depth of 8.0 feet below the ground surface. The fill generally consisted of brown silty gravelly sand. The recorded N-values ranged from 8 to 12 blows per foot (bpf), indicating a loose to medium dense consistency. The fill is likely associated with the backfill for the bridge.

4.1.3 Cobble/Boulder Deposit

A deposit of cobbles and boulders was encountered from 8.0 to 14.0 feet below the ground surface. The presence of the cobbles and boulders was primarily determined by drilling action and drilling resistance. Roller bit and casing refusal was encountered at 8 feet which required the boring be relocated 3 feet east of original location. One split spoon sample conducted at a depth of 9 to 10 feet and returned rock fragments. The recorded N-value was 34 blows per foot (bpf), indicating a dense consistency for the soil portion of the deposit. This deposit is likely part of the stream channel prior to construction of the existing bridge.

4.1.4 Bedrock

The top of bedrock was encountered at a depth of 14.0 feet below the ground surface. Four 5-foot long bedrock cores were obtained from boring B-1. The bedrock was described as light gray, moderately hard, slightly weathered to fresh, dolostone. The joints ranged from low angle to high angle, rough, slightly discolored and tight to partly open. The RQD values were 38, 60, 53 and 80 percent and the RMR values were estimated to range from 27 (poor rock) to 49 (fair rock). Photographs of the rock cores are included in Appendix C.

4.2 ABUTMENT NO. 2 (EAST SIDE)

Test boring B-3 was drilled at Abutment No. 2. The following subsurface conditions were encountered at the abutment:

4.2.1 Pavement

The borings encountered 6 inches of asphalt pavement.

4.2.2 Fill Material

The fill was encountered from below the pavement to a depth of 8.0 feet below the ground surface. The fill generally consisted of brown silty gravelly sand. The recorded N-values ranged from 16 to 39 blows per foot (bpf), indicating a medium dense to dense consistency. The fill is likely associated with the backfill for the bridge.

4.2.3 Cobble/Boulder Deposit

A deposit of cobbles and boulders was encountered from 8.0 to 13.0 feet below the ground surface. The presence of the cobbles and boulders was primarily determined by drilling action and drilling resistance. Two split spoon samples were attempted. The sample at a depth from 9 to 11 feet had a recorded N-value of 19 blows per foot (bpf), indicating a medium dense consistency, but refused before penetrating the full 24 inches. The sample at a depth of 11 feet refused with no penetration. This deposit is likely part of the stream channel prior to construction of the existing bridge.

4.2.4 Bedrock

The top of bedrock was encountered at a depth of 13.0 feet below the ground surface. Four 5-foot long bedrock cores were obtained from boring B-3. The bedrock was described as light gray, moderately hard, slightly weathered to fresh, dolostone. The joints ranged from low angle to moderately dipping, rough, fresh and tight to partly open. The RQD values were 23, 83, 87 and 92 percent and the RMR values were estimated to range from 32 (poor rock) to 46 (fair rock). Photographs of the rock cores are included in Appendix C.

4.3 CONCRETE OBSTRUCTION AT ABUTMENT NO. 1

Boring B-2 was intended to be drilled at the northern side of Abutment No. 1. Five attempts were made to advance the boring each attempt encountered 3 inches of asphalt pavement, overlying either steel plate or a concrete structure. It should be noted that this boring was location in the area of a now defunct trolley car line. Additionally, the original roadway surface was reportedly concrete. After discussions between the geotechnical engineer and drilling contractor it was decided not to advance the boring through the bottom of the unknown structure. The table below summarizes the conditions encountered at B-2.

Table 2 – Summary of Conditions at B-2

Boring	Asphalt Thickness (inches)	Comments
B-2A	3	Auger refusal on concrete at a depth of 3 inches.
B-2B	3	Auger refusal on steel at a depth of 3 inches.
B-2C	3	Advanced auger through 5 inches of concrete. Encountered a circular void with a 22-inch diameter partially filled with sand/gravel. The sides and bottom appeared to be concrete. The boring was not advanced through the bottom of the void.
B-2D	3	Auger refusal concrete at a depth of 3 inches.
B-2E	3	Advanced auger through 5 inches of concrete. Encountered a void partially fill with sand and gravel, similar to the void encountered at B-2C.

4.4 GROUNDWATER

Given the granular nature of the soils and the proximately to the river channel, the groundwater in the area of the proposed bridge is expected to coincide with the water level in the river. At the time of drilling there was approximately 1 foot of water in the stream channel, which corresponds to approximately El. 722 to 723. Groundwater and river water levels will vary over time due to seasonal changes in precipitation and temperature, snowmelt, and surrounding and on-site drainage characteristics.

4.5 LABORATORY TESTING

The amount of soil retrieved from the borings was relatively small and the majority of the soil was use for the environmental testing program. Only two jar samples remained for the geotechnical program. These samples were not sent for laboratory testing. Rock samples were not sent for testing.

5.0 Discussions and Recommendations

Based on the subsurface conditions encountered in the test borings, the bedrock is relatively shallow and suitable for supporting the abutments and wingwalls on conventional spread footings bearing directly on the bedrock or on a concrete sub-footing placed on the bedrock. The layer of cobbles and boulder is not considered to be a suitable bearing material. As an alternative to a spread footing foundation the abutments and wingwalls could be supported on drilled micropiles bearing in the bedrock. The micropile foundations in combination with precast modular bridge units would likely decrease the construction duration of the project. Recommendations for both options are provided below.

Prior to final design we recommend the following be conducted to delineate the limits of the unknown concrete structure encountered at the location of B-2:

- Review any historical records pertaining to the existing bridge structure and roadway construction. In particular, any historic records of the trolley car tracks that were located below the west bound lane should be reviewed.
- Conducted a geophysical investigation to delineate the horizontal distance and depth of the structure. The investigation should focus on the areas behind each abutment. A geophysical firm should be contacted to determine which geophysical method is appropriate.
- After review of historic records and conducting the geophysical investigation, conduct additional borings and/or probes to define the depth to bedrock in the area below the west bound lanes.

5.1 SPREAD FOOTING RECOMMENDATIONS

Based on the subsurface conditions encountered in the borings, the abutments and wingwalls can be supported by conventional shallow spread footings bearing directly on bedrock or spread footings bearing on a concrete sub-footing cast on bedrock. In the area of the Abutment No. 1 the bedrock was encountered at approximately El. 716.49. In the area of the Abutment No. 2 the bedrock was encountered at approximately El. 717.81.

Abutments and wingwalls should be proportioned for all applicable load combinations specified in the AASHTO 2017 and shall be designed for all relevant strength, extreme and service limits states. The design of the abutments and wingwalls founded on spread footings at the strength limit state shall consider bearing resistance, eccentricity (overturning), sliding and reinforced concrete structural failure. For bearing, a resistance factor, (φ), of 0.45 shall be applied to the nominal bearing resistance.

For scour protection the footings should be founded directly on the bedrock surface cleaned of all loose, weathered and erodible rock. Because the bedrock will not scour, strength and extreme limit states design do not need to consider foundation resistance after the design flood event.

For the extreme limit state, the abutment and wingwalls should be designed for bearing resistance, eccentricity, sliding and structural failure with respect to the extreme event load conditions relating to applicable hydraulic events and ice loads. Resistance factors, (φ), for the extreme limit state shall be taken as 1.0.

For the service limit state, the abutment and wingwalls should be designed for settlement, horizontal movement, bearing resistance, sliding and eccentricity. The global stability of foundations is typically evaluated at the Service I Load Combination and a resistance factor, (φ), of 0.65. However, shear failure along the joints in the rock mass below the foundations is not anticipated and a global stability evaluation is not needed for this site.

For footings bearing on bedrock, the eccentricity (e) of the loading at the strength limit state shall not exceed 0.45 of the footing width in either direction. As presented in AASHTO Section 10.6.3.3 for foundations on bedrock:

$$|e| < 0.45b, \text{ where b is the footing width}$$

5.1.1 Bearing Resistance

The substructure footings shall be proportioned to provide stability against bearing resistance failure. A factored bearing resistance of 20 ksf may be used for sizing the footing and to limit settlement when evaluating the service limit state. This value was obtained from the Table C10.6.2.6.1-1 of the AASHTO 2017 code, using the recommended values for the "weathered or broken bedrock" category, which is expected to be conservative for the bedrock at the site.

The bearing resistance of the footings should be analyzed at the strength limit state using a factored bearing resistance of 29 ksf. This factored value is based on a nominal bearing resistance of 64 ksf and applying a bearing resistance factor, (φ_b) of 0.45 for spread footing bearing bedrock in accordance with Table 10.5.5.2.2-1 of AASHTO 2017.

The factored bearing stress shall not exceed the factored compressive resistance of the footing concrete, which may be taken as $0.3f'_c$. Where, f'_c is the compressive strength of the concrete.

The footing shall be at least 3 feet wide regardless of the bearing material or applied bearing pressure.

5.1.2 Settlement

We anticipate the roadway approaches at both ends of the proposed bridge will not be significantly modified. Placement of fill will result in negligible densification of the underlying soils and negligible settlement of roadway. Given the granular nature of the soils at the site any roadway settlement is expected to occur immediately after construction. Given the relatively light bridge loads the footings bearing on the bedrock are expected to settle less than 0.5 inches.

5.2 MICROPILE FOUNDATION RECOMMENDATIONS

As an alternative to supporting the abutments with footings bearing on bedrock the abutments could be supported on a system of drilled micropiles. The micropiles will reduce the depth of the temporary earth support and will reduce the dewatering effort. Resulting in a shorter construction duration. Recommendations for the design and construction of micropiles are provided below.

Micropiles consist of steel casing advanced through an unsuitable bearing soil with an uncased (or bonded) zone formed within either a dense soil zone or bedrock. Based on the borings the subsurface conditions the amount of soil at the site is relatively thin, therefore the micropiles should be founded in the bedrock.

Common casing sizes have an outside diameter of 5-1/2, 9-5/8 and 10.75 inches. The most common casing size is 9-5/8 inches. The annular space between the bar and casing is filled with 4,000 to 5,000 psi grout placed using tremie methods. The internal reinforcing bars typically range from #8 to #18 Grade 75 bars. The reinforcing bar should be continuous for the entire length of the pile and centered within the pile casing.

5.2.1.1 Micropile Tip Resistance

Micropiles derive their axial resistance primarily from side resistance. The tip resistance is small compared to the side resistance and is conservatively ignored.

5.2.1.2 Side Resistance in Compression

Table 3 provides the per foot nominal and factored geotechnical resistance. The resistance is based 9-5/8 inch diameter cased micropile extending through the soil and plunged 2 feet into the bedrock. The actual rock socket diameter was assumed to be 7.5 inches in diameter. The capacity for the micropiles bearing soil are based on a nominal grout-to-soil bond strength of 20 ksf. The bond strength was obtained from AASHTO 2017 Table C10.9.3.5.2-1.

Table 3 - Micropile Compression Resistance Bearing in Soil

Estimated Top Bond Zone Elevation(1)	Resistance Per Foot of Pile	
	Nominal (kips/ft)	Factored(2) (kips/ft)
Abut No. 1 – El. 714.5 Abut No. 2 – El. 715.8	39	22

(1) Assumes a 2-foot plunge depth below the top of bedrock.
(2) Based on a resistance factor of 0.55.

Additional recommendations:

- The rock sockets should have a minimum length of 10 feet.
- The micropiles should be designed using permanent steel drill casing advanced at least 2 feet into the bedrock. This is referred to as the plunge depth.
- In accordance with AASHTO 2017 Table C10.5.5.2.5-1, a resistance factor equal to 0.55 is recommended for axial compression because presumptive grout-to-ground bond strengths have been used. In accordance with AASHTO 2017 Table C10.5.5.2.5-1, once load tests are completed the resistance factor can be increased to 0.7.

5.2.1.3 Side Resistance in Tension

If tension tests are performed on the piles, then the factored uplift resistance of the piles will equal the factored compression resistance of the pile. If a tension test is not performed, then the factored uplift resistance can be estimated as 50 percent of the factored compression resistance, As per AASHTO 2017 C10.9.3.7. The nominal uplift resistance of the micropile group should be taken as the lesser of:

- The sum of the individual pile uplift resistance.
- The uplift resistance of the pile group considered as a block. A unit weight of 150 pcf can be used for the bedrock.

5.2.1.4 Soil Parameters for Lateral Pile Analysis

Lateral capacity of pile typically requires the use of computer software such as LPile or GROUP. For the analysis, the fill should be modelled as being above the water table. The cobble/boulder deposit and bedrock should be modelled as being below the water table. The following parameters in Table 4 can be used for the analysis.

Table 4 – Parameters for Lateral Capacity Analysis

Soil Type	Unit Weight (pcf)		Spring Constant (pci)		Uniaxial Compressive Strength (psi)
	Above Water	Below Water	Above Water	Below Water	
Existing Fill	110	48	25	20	--
Cobble/Boulders	130	68	60	90	--
Bedrock (strong rock)	150	88	--	--	3000(1)

(1) Assumed value.

5.2.1.5 Micropile Testing

The micropiles should be load tested in accordance with AASHTO 2017 section 10.9.3.5.4. Testing should consist of verification and proof tests. One verification test should be conducted on a non-production pile. At least one proof test should be conducted at the location of each pier, but not less than 5 percent of the number of total piles. Testing should be conducted in accordance with ASTM D1143 for compression and ASTM D3689 for tension.

5.2.2 Lateral Earth Pressures

The following recommendations are for the design of the abutments and wingwalls:

- Walls that are free to rotate at the top should be designed based on active earth pressure (K_a) and compacted Granular Backfill for Structures. Walls with level backfill should be designed using K_a equal to 0.28 and a unit weight of 140 pounds per cubic foot (pcf) for the backfill. Walls with a backfill slope of 2 horizontal to 1 vertical (2H: 1V) should be designed using K_a equal to 0.42 and a unit weight of 140 pcf for the backfill. These earth pressures are based the Coulomb theory and an angle of internal friction equal to 34 degrees. The interface friction angle between the backfill and the concrete wall was conservatively assumed to be zero.
- We recommend walls that retain earth be backfilled horizontally with a minimum of 4 feet of compacted fill meeting the requirements of VTrans Item No. 704.08 (Granular Backfill for Structures) in order to provide free draining and less frost susceptible materials in this zone.
- The walls should be designed for a live load surcharge equivalent to the earth fill height summarized in AASHTO LRFD Tables 3.11.6.4-1 and 3.11.6.4-2.
- In addition to the free draining granular backfill, weep holes should be installed in the abutment and wingwalls. In accordance with the 2010 VTrans Structures Manual, weep holes should be 4 inches in diameter with a maximum center to center spacing of 10 feet. The elevation of the weep holes shall be at the highest of either the approximate ordinary low water elevation or 1 foot above top of footing.

- For calculating nominal sliding resistance (R_n) for footings bearing on bedrock we recommend using Tan(δ) = 0.60 (where δ = 31 degrees), this value was obtained from Table 3.11.5.3-1 of the AASHTO LRFD 2017 code and considered to be conservative. Based on Table 10.5.5.2.2-1 of AASHTO LRFD 2017 code, a resistance factor (φ_τ) equal to 0.80 should be used. In accordance with the 2010 VTrans Structures Manual the nominal passive resistance (R_{ep}) for soil in front of the retaining walls should be ignored.

- If the bedrock surface is observed to slope steeper then 4H: 1V at the subgrade elevation, then the bedrock surface should be benched to create level steps or excavated to be completely level. Alternatively, rock dowels extending from the footings (or sub-footing) into the bedrock may be used to resist sliding forces and improve stability.

5.2.3 Frost Depth

Based on the map entitled *Frost Penetration Map, 90% Reliability*, in the *Vermont Agency of Transportation, Pavement Design Guide*, the frost depth at the site is 55 inches. Therefore, we recommend the footings be founded at least 55 inches below the surrounding ground surface.

5.2.4 Scour Considerations

If the footings are founded on the bedrock or founded on a sub-footing bearing on bedrock, the bedrock at the site is not considered to be scour susceptible.

5.2.5 Seismic Design Parameters

Because the proposed bridge is a single span structure, we anticipate seismic analysis will not be required. However, if needed, we have developed the following seismic design parameters based upon subsurface conditions encountered in the test borings and AASHTO 2017 Section 3.10.

Table 5 – Summary of Seismic Parameters

Parameter	Reference
Site Class "C" – Very Dense Soil and Soft Rock profile, based on the average N-value for the upper 100 feet of soil profile greater than 50 bpf.	Table 3.10.3.1-1
Peak Ground Acceleration (PGA) = 0.06 g	Figure 3.10.2.1-1
Acceleration Coefficient (As) = 0.072 g	Equation 3.10.4.2-2
Spectral response acceleration at 0.2-second (S_s) = 0.136 g	Figure 3.10.2.1-2
Design spectral acceleration at 0.2-second (S_{DS}) = 0.164 g	Equation 3.10.4.2-3
Spectral response acceleration at 1.0-second (S_1) = 0.042 g	Figure 3.10.2.1-3
Design spectral acceleration at 1.0-second (S_{D1}) = 0.071 g	Equation 3.10.4.2-6
Seismic Zone 1, based on a SD1 < 0.15 g	Table 3.10.6-1

5.2.6 Liquefaction Analysis

Liquefaction is a condition when a soil undergoes continued deformation during the course of cyclic stress applications induced by an earthquake where pore water pressure becomes equal to the confining pressure (e.g. effective stress approaches zero) and large deformations occur. Significant factors influencing liquefaction include grain size distribution of sand, fines content, in-situ density, and vibration characteristics (e.g. design earthquake and acceleration coefficient). Liquefaction generally occurs in saturated, relatively loose (N values less than 15 bpf) sandy soils with low fines content (less than 30 percent). Given the soil below the water table consists of cobbles and boulders with bedrock at a depth of 13 to 14 feet, the site is not considered to be susceptible to liquefaction.

6.0 Construction Consideration

6.1 COFFERDAMS/TEMPORARY EARTH SUPPORT

Given the anticipated bottom of the footings will be at or below the water in the river channel, cofferdams will be required to allow the footings to be constructed in-the-dry. Due to the presence of the cobble and boulder layer and shallow bedrock, cofferdams consisting of driven sheet piles will not be possible. Therefore cofferdams will likely consist of a series of large sand bags or portable dam type structures. In accordance with Section 208.06 of the VTrans *2018 Standard Specifications for Construction,* the Contractor shall prepare detailed plans and a schedule of its operation for each cofferdam specified in the Contract. The design and structural details of the cofferdam shall be signed, stamped, and dated by a Professional Engineer (Structural or Civil).

As with the cofferdams, driven sheet piles are not feasible due to the cobble and boulder layer. Additionally, the shallow bedrock will prevent the sheets from being driven below the bottom of the proposed excavation and thus the sheet piles will not develop resistance from being cantilevered. Any temporary shoring system will need to be internal braced and/or restrained using tiebacks. However, tiebacks may be difficult to install due to the presence of below ground utilities in the project area. Feasible shoring systems include internally braced slide rail type systems and soldier pile and lagging systems where the piles are placed in predrilled rock sockets.

6.2 CONSTRUCTION DEWATERING

Groundwater is expected to seep from fractures exposed at the bedrock surface and from the cobble/boulder deposit. It is anticipated that temporary dewatering can be accomplished by a system of shallow sumps and pumps. Sumps should be equipped with filter fabric to prevent the loss of fine-grained soils during pumping. Water pumped from the excavations should be discharged to settling ponds to allow fine particles to settle out prior to discharge to the river. Water should be discharged in accordance with all applicable permits and regulations.

6.3 PLACEMENT OF BACKFILL

Fill should be placed systematically in horizontal lifts not more than 12 inches in loose lift thickness prior to compaction. Cobbles larger than 8 inches in any direction should be removed from the fill prior to placement. Compaction equipment should preferably consist of large self-propelled vibratory rollers. Where hand-guided equipment is used (e.g. small vibratory plate compactor), the loose lift thickness shall not exceed 6 inches. Cobbles larger than 4 inches should be removed from the fill prior to placement.

Embankment fills should be compacted to at least 90 percent of the material's maximum dry density as determined in accordance with AASHTO T-99. Granular Backfill for Structures, or other select material placed within the roadway base section shall be compacted to at least 95 percent of the material's maximum dry density as determined in accordance with AASHTO T-99.

Backfill placed behind abutments and wingwalls shall meet the requirements of Granular Backfill for Structures (VTrans Item No. 704.08). Granular Backfill for Structures shall consist of satisfactorily graded, free draining granular material reasonably free from loam, silt, clay, and organic material and meet the following gradation:

Table 6 – Granular Backfill for Structures Gradation

Particle Size	% Passing by Weight
3-inch	100
No. 4	45 – 75
No. 100	0 – 12
No. 200	0 – 6

The soil moisture content range should be ±2 percent of its optimum moisture content and Granular Backfill should be placed in uniform lifts not exceeding 12 inches loose thickness compacted to at least 95 percent of Maximum Dry Density as determined by AASHTO T99 Method D. The percent compaction is determined in the field by AASHTO T191 (sand cone) or AASHTO T310 (nuclear moisture-density meter).

6.4 PROTECTION OF UTILITIES

Utilities within the area of the proposed excavations should be properly braced, temporarily re-routed and/or protected from construction activities/disturbance.

6.5 BEDROCK SUBGRADE PREPARATION

All soil, loose rock, weathered rock, and erodible rock should be removed from beneath the proposed abutment footings prior to placing concrete for the footings. The foundation subgrade should be inspected and approved by a qualified geotechnical engineer or geologist. The surface of the bedrock should be prepared as flat as possible, with all areas of the subgrade flatter than 4 horizontal to 1 vertical (4H:1V). If the bedrock surface is observed to slope steeper then

4H:1V at the subgrade elevation, the bedrock surface should be benched to create level steps or excavated to be completely level. Alternatively, rock dowels extending from the footings into the bedrock may be used to resist sliding forces and improve stability.

7.0 LIMITATIONS

7.1 USE OF REPORT

This report has been prepared for the exclusive use of the Vermont Agency of Transportation (VTrans) and their respective assigns and designees. This report is not intended for the use or reliance of other (third) parties, without the express consent of Stantec and VTrans. Any use, which a third party makes of this report, or any reliance on decisions made based on this report, is the responsibility of such third parties. Further, the findings of this study apply only to the specific Site and project described herein. The findings herein are inapplicable to other Sites, and to developments of different grading, layout, loading, and performance requirements. Stantec accepts no responsibility for damages, real or perceived, suffered by parties as a result of decisions made or actions based on the unintended and/or inappropriate use of this report.

This Geotechnical Report provides recommendations, and is intended for informational use, requiring interpretation by the owner, design team, and contractor for the design and construction of the project, and interpretation of final quantities and construction costs. The Geotechnical Report is not intended, or suitable, by itself, for use as a technical specification or to determine quantities. Anticipated quantities and/or costs may be provided in the Geotechnical Report; such information is an Engineer's interpretation, and may vary dramatically from contractor bids, which are based on potentially differing interpretations, and several other variables not available or considered by the Engineer.

7.2 SUBSEQUENT INVOLVEMENT

The geotechnical process incorporates initial exploration and recommendations as summarized herein, and is followed by continuous involvement during key design and construction benchmarks. The recommendations provided herein are based on preliminary information and assumptions regarding proposed site grading, structural loading and performance requirements. It is recommended that Stantec review final foundation, grading, and other applicable plans to assess whether or not these recommendations require modification.

During construction, additional soil samples should be analyzed in the laboratory for moisture content, gradation, and moisture density relationship tests to evaluate the reuse of onsite soils (existing fill and natural sand strata) as backfill material.

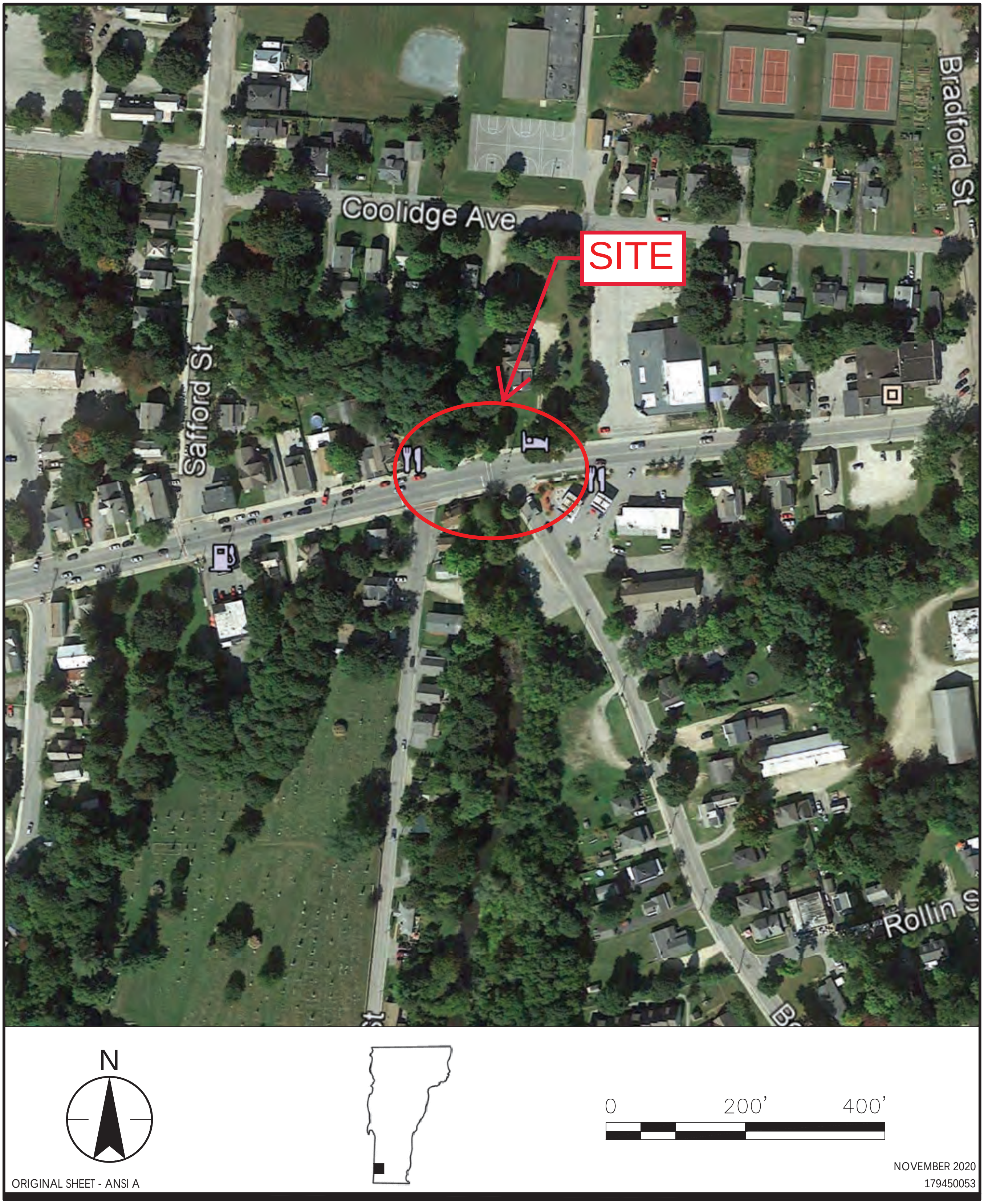
Stantec should be retained to observe excavations and subgrade preparation to assess whether the intent of these recommendations is followed during construction, and whether or not other appropriate and/or cost-effective solutions may be warranted based on the actual conditions encountered. Further, a soil exploration is a random sampling of a Site. Should any conditions at the Site at any point during the project be encountered that differ from those summarized in the report, Stantec should be notified immediately in order to permit reassessment of these conditions and the recommendations contained in the report.

7.3 REPRESENTATION AND INTERPRETATION OF DATA

Surficial and subsurface information presented herein is based on field measurements obtained during the course of the exploration and site reconnaissance. The precision and accuracy of surficial data is a function of the references, benchmarks, methods and instruments employed, as summarized in the report. Subsurface data is based on measurements within the borehole or test pit using the sampling methods described on the exploration logs. The completeness, precision, and accuracy of such data is a function of the frequency and type of exploration and sampling employed, as well as the precision and accuracy of the surface location and elevation of the borehole, and may vary from actual conditions encountered during excavations. Subsurface conditions between, beyond and below explorations, may vary dramatically from the nearest exploration, due to natural geologic action, deposition and weathering, or man-made activities.

Groundwater levels were recorded during the time periods and frequencies noted on the explorations. It is important to note that groundwater levels are disrupted by the exploration, and require equilibration periods to determine actual hydrostatic levels, which exceed the duration of the measurement period. Multiple hydrostatic groundwater levels may exist, including perched or trapped water, which may not necessarily be accurately represented by one water level reading. Groundwater levels fluctuate due to seasonal variations, adjacent surface water bodies, precipitation, and on-Site and nearby land use.

Figures



ORIGINAL SHEET - ANSI A

NOVEMBER 2020
179450053

Stantec

5 Dartmouth Drive
Auburn, NH 03032
www.stantec.com

Client/Project

VERMONT AGENCY OF TRANSPORTATION
BENNINGTON BF 1000(20)
VT ROUTE 9, BRIDGE 6, OVER WALLOOMSAC

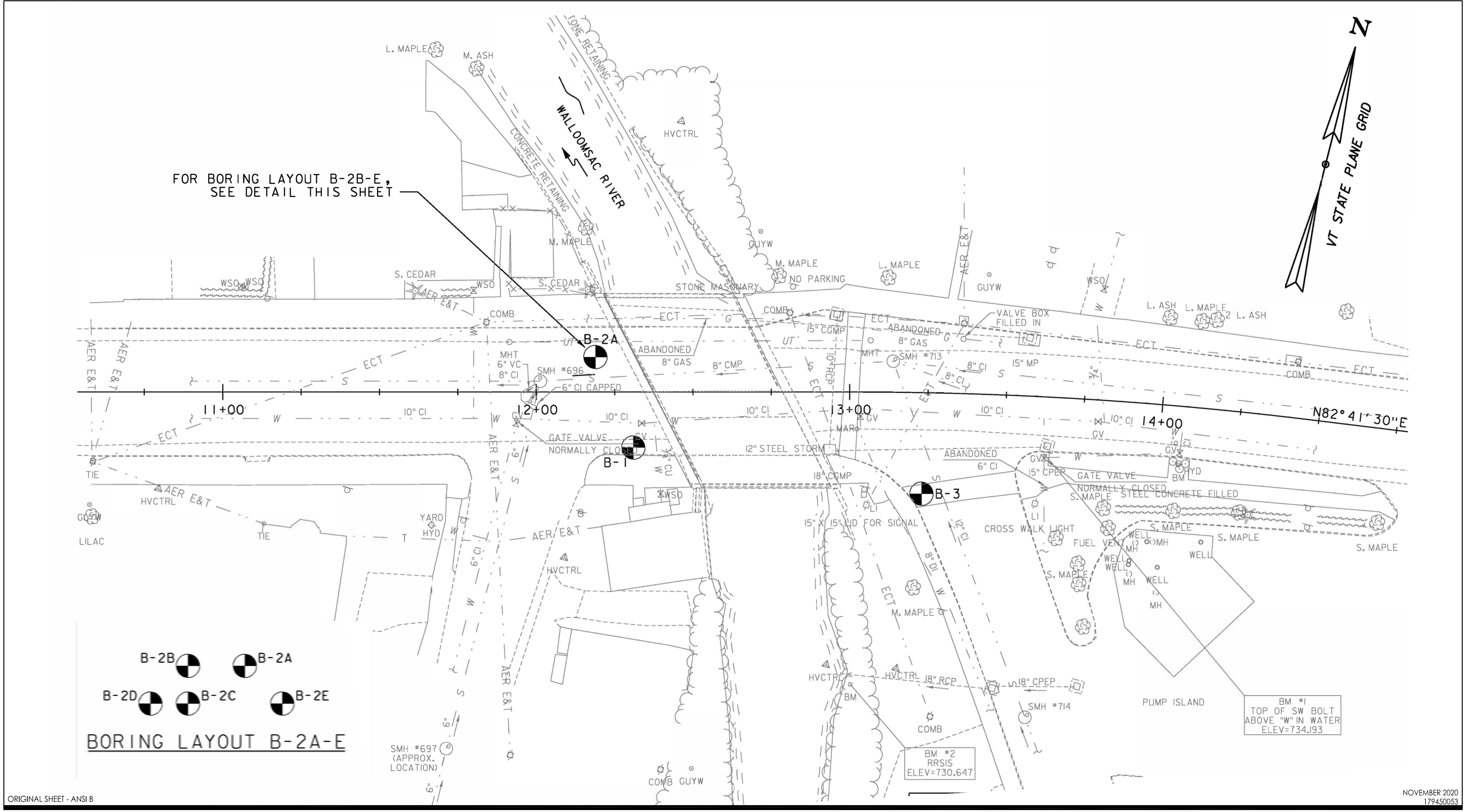
Figure No.

1

Title

PROJECT LOCATION PLAN

\\us0287-ppfs01\workgroup\1956\temporary\GEOTECHNICAL\179450053 - Bennington Rt 9 Bridge\AutoCAD\Boring Location Plan.dwg
2020/11/19 9:15 PM By: Foley, Brian



ORIGINAL SHEET - ANSI B

NOVEMBER 2020
179450053

Stantec

5 DARTMOUTH DRIVE
AUBURN, NEW HAMPSHIRE
www.stantec.com

Legend

B-1 ● Location and designation of test boring

0 30' 60'

Notes

1) Test borings were drilled by New England Boring of Derry, New Hampshire on October 12 through October 15, 2020 under supervision of Stantec personnel.

2) The borings were located in the field by taping from existing site features.

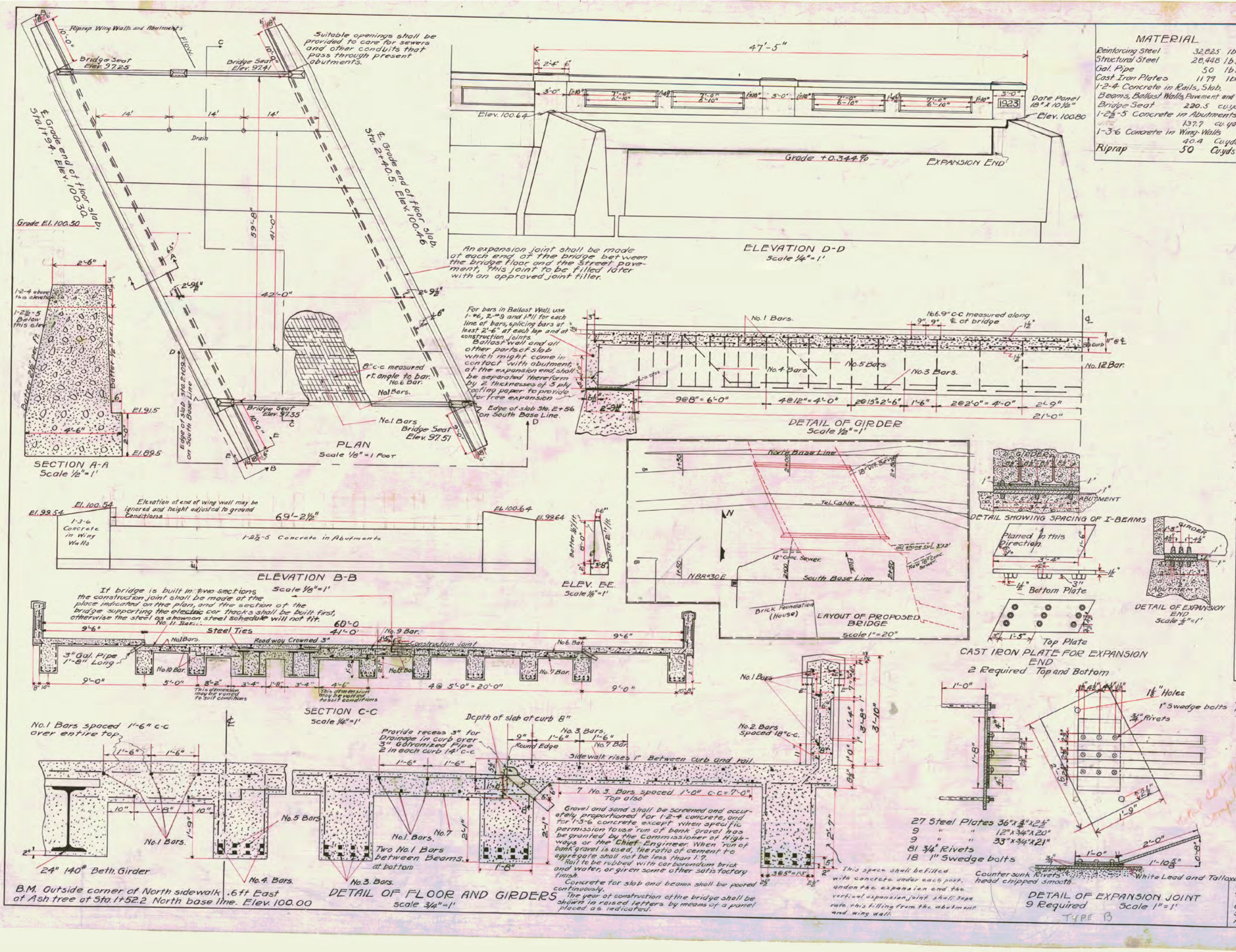
Client/Project
VERMONT AGENCY OF TRANSPORTATION
BENNINGTON BF 1000(20)
VT ROUTE 9, BRIDGE 6, OVER WALLOOMSAC

Figure No.
2

Title
BORING LOCATION PLAN

Appendix A

1923 Bridge Plans



Gravel and sand shall be screened and accurately proportioned for 1-2-4 concrete, and for 1-3-6 concrete except when specific permission to use "run of bank" gravel has be granted by the Commissioner of Highways or the Chief Engineer. When "run of bank" gravel is used, the ratio of cement to aggregate shall not be less than 1:7.

Rail to be rubbed with carborundum brick and water, or given some other satisfactory finish.

Concrete for slab and beams shall be poured continuously.

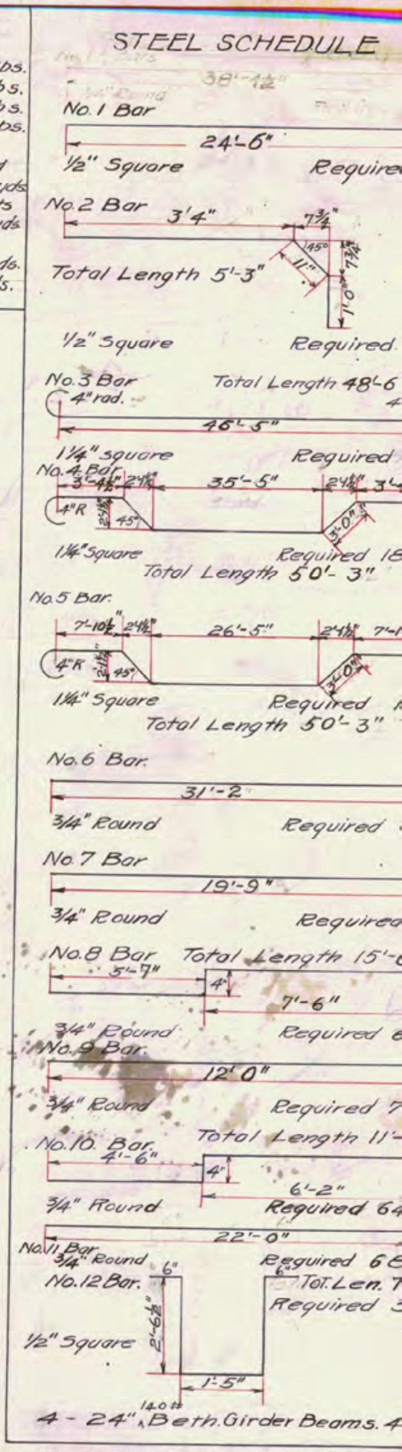
The year of construction of the bridge shall be shown in raised letters by means of a panel placed as indicated.

B.M. Outside corner of North sidewalk .6 ft East of Ash tree at Sta. 1+52.2 North base line. Elev. 100.00

MATERIAL

Reinforcing Steel	32,825 lbs.
Structural Steel	28,448 lbs.
Gal. Pipe	50 lbs.
Cast Iron Plates	1179 lbs.
1-2-4 Concrete in Rails, Slab, Beams, Ballast Walls, Pavement and Bridge Seat	220.5 cu.yds
1-2½-5 Concrete in Abutments	137.7 cu.yds
1-3-6 Concrete in Wing-Walls	40.4 Cu.yds.
Riprap	50 Cu.yds.

STEEL SCHEDULE



All steel to be deformed bars.
Reinforcing steel to conform to Stand Specifications for Billet Steel Concrete Reinforcement Bars of the America Society for Testing Materials.
Serial Designation A 15-14.

Total Cost $12,078.59
Completed Jan. 1924

PLAN
OF
COOPER BRIDGE
BENNINGTON, VT.

APPROVED AUG. 28, 1923
Thurman W. Dix
CHIEF ENGINEER

Vt 9 B6 over Jewett Brook

Designed by: J.L.D. and J.O.D. 8-10-2
Drawn by: F.W. Garran 8-22-2
Traced by. F.W.G, O.E.G.
Checked by: J.O.D. 8-29-2
Series B-351.
App. No. 29 1923. Filed B-40

Appendix B

Test Boring Logs

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

BENNINGTON
BF 1000 (20)
VT Rt 9, Bridge No. 6 Over Walloomsac

Boring No.: **B-1**
Page No.: 1 of 2
Pin No.: z12j606
Checked By: TAD

Boring Crew: New England Boring, Derry, NH, LGH (Stantec)
Date Started: 10/13/20 Date Finished: 10/15/20
VTSPG NAD83: N 138801.53 ft E 1456422.58 ft
Station: 12+30.94 Offset: 17.73' RT
Ground Elevation: 730.49 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	24 in	30 in.
Hammer/Rod Type:	Safety/N	
Rig:	Truck/Mobile B-53	C_E = 1

Groundwater Observations

Date	Depth (ft)	Notes
10/12/20	7.5	
10/13/20	8.0	

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.5 ft								
		Visual Classification, GrSa, brn, Dry, Rec. = 1.0 ft, **-FILL-**				8-7-6 (13)				
2.5		Visual Classification, GrSa, brn, Dry, Rec. = 1.1 ft, **-FILL-**				7-7-6-8 (13)				
5.0		Visual Classification, SiGrSa, brn, Dry, Rec. = 1.5 ft, **-FILL-**				8-7-5-5 (12)				
7.5		Visual Classification, SiGrSa, brn, Dry, Rec. = 1.3 ft, **-FILL-**				6-6-2-3 (8)				
		Field Note:, Boulder at 8 feet could not advance roller or casing, offset boring 3 feet east								
10.0		Visual Classification, Rock Fragments, Rec. = 0.1 ft, **-COBBLE/BOULDER LAYER-**				20-18-16-22 (34)				
12.5		Field Note:, Based on drill action cobbles and boulders are present from approximately 8 to 14 feet., **-COBBLE/BOULDER LAYER-**								
		14.0 ft - 15.0 ft, Advanced roller bit through bedrock from 14 to 15 feet.				Top of Bedrock @ 14.0 ft				
15.0 17.5		15.0 ft - 20.0 ft, Light gray, Dolomite, Moderately hard, Slightly weathered, Poor rock, NQDC, Joints are moderately dipping, rough, slightly discolored, partly open. Highly fractured zone from 17 to 18 feet. RMR = 27	1 (45)	84 (38)	2.5 1.5 1.5 2 2					

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

BENNINGTON
BF 1000 (20)
VT Rt 9, Bridge No. 6 Over Walloomsac

Boring No.: **B-1**
Page No.: 2 of 2
Pin No.: z12j606
Checked By: TAD

Boring Crew: New England Boring, Derry, NH, LGH (Stantec)
Date Started: 10/13/20 Date Finished: 10/15/20
VTSPG NAD83: N 138801.53 ft E 1456422.58 ft
Station: 12+30.94 Offset: 17.73' RT
Ground Elevation: 730.49 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	24 in	30 in.
Hammer/Rod Type:	Safety/N	
Rig:	Truck/Mobile B-53	C_E = 1

Groundwater Observations

Date	Depth (ft)	Notes
10/12/20	7.5	
10/13/20	8.0	

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
22.5	▨	20.0 ft - 25.0 ft, Light gray, Dolomite, Moderately hard, Slightly weathered, Fair rock, NQDC, Joints are moderately dipping, rough, slightly discolored, tight to partly open. RMR = 46	2 (45)	100 (60)	2 2 2 2 2					
25.0 27.5	▨	25.0 ft - 30.0 ft, Light gray, Dolomite, Moderately hard, Fresh, Fair rock, NQDC, Joints are low angle to moderately dipping, rough, slightly discolored, tight to partly open. RMR = 46	3 (25)	100 (53)	2.5 2 2 2.5 2					
30.0 32.5	▨	30.0 ft - 35.0 ft, Light gray, Dolomite, Moderately hard, Fresh, Fair rock, NQDC, Joints are moderately dipping to high angle, rough, slightly discolored, tight to partly open. Highly fractured zone from 33 to 34 feet. RMR = 49	4 (65)	98 (80)	2.5 2 2 1.5 1.5					
35.0 37.5		Hole stopped @ 35.0 ft								

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

BENNINGTON
BF 1000 (20)
VT Rt 9, Bridge No. 6 Over Walloomsac

Boring No.: **B-2A**
Page No.: 1 of 1
Pin No.: z12j606
Checked By: TAD

Boring Crew: New England Boring, Derry, NH, LGH (Stantec)
Date Started: 10/14/20 Date Finished: 10/14/20
VTSPG NAD83: N 138826.55 ft E 1456403.72 ft
Station: 12+18.86 Offset: 11.17' LT
Ground Elevation: 730.61 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	24 in	30 in.

Hammer/Rod Type: Safety/N
Rig: Truck/Mobile B-53 $C_E = 1$

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
	■	Asphalt Pavement, 0.0 ft - 0.3 ft					
		Refusal on concrete, 0.3 ft					
		Hole stopped @ 0.3 ft					
2.5							
5.0							
7.5							
10.0							
12.5							
15.0							
17.5							

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

BENNINGTON
BF 1000 (20)
VT Rt 9, Bridge No. 6 Over Walloomsac

Boring No.: **B-2B**
Page No.: 1 of 1
Pin No.: z12j606
Checked By: TAD

Boring Crew: New England Boring, Derry, NH, LGH (Stantec)
Date Started: 10/14/20 Date Finished: 10/14/20
VTSPG NAD83: N 138826.55 ft E 1456403.72 ft
Station: 12+15.86 Offset: 11.17' LT
Ground Elevation: 730.56 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	24 in	30 in.
Hammer/Rod Type:	Safety/N	
Rig:	Truck/Mobile B-53	C_E = 1

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
	■	Asphalt Pavement, 0.0 ft - 0.3 ft					
		Refusal on steel plate, 0.3 ft Hole stopped @ 0.3 ft					
2.5							
5.0							
7.5							
10.0							
12.5							
15.0							
17.5							

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

BENNINGTON
BF 1000 (20)
VT Rt 9, Bridge No. 6 Over Walloomsac

Boring No.: **B-2C**
Page No.: 1 of 1
Pin No.: z12j606
Checked By: TAD

Boring Crew: New England Boring, Derry, NH, LGH (Stantec)
Date Started: 10/14/20 Date Finished: 10/14/20
VTSPG NAD83: N 138826.55 ft E 1456403.72 ft
Station: 12+15.86 Offset: 9.17' LT
Ground Elevation: 730.58 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	24 in	30 in.
Hammer/Rod Type:	Safety/N	
Rig:	Truck/Mobile B-53	$C_E = 1$

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
	■	Asphalt Pavement, 0.0 ft - 0.3 ft					
	▨	5 inches of Concrete, 0.3 ft - 0.7 ft					
		22 inch diameter void, partially filled with soil, 0.7 ft - 2.5 ft					
2.5		Refusal on concrete, 2.5 ft Hole stopped @ 2.5 ft					
5.0							
7.5							
10.0							
12.5							
15.0							
17.5							

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

BENNINGTON
BF 1000 (20)
VT Rt 9, Bridge No. 6 Over Walloomsac

Boring No.: **B-2D**
Page No.: 1 of 1
Pin No.: z12j606
Checked By: TAD

Boring Crew: New England Boring, Derry, NH, LGH (Stantec)
Date Started: 10/14/20 Date Finished: 10/14/20
VTSPG NAD83: N 138826.55 ft E 1456403.72 ft
Station: 12+13.86 Offset: 9.17' LT
Ground Elevation: 730.56 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	24 in	30 in.
Hammer/Rod Type:	Safety/N	
Rig:	Truck/Mobile B-53	C_E = 1

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
	■	Asphalt Pavement, 0.0 ft - 0.3 ft					
		Refusal on concrete, 0.3 ft Hole stopped @ 0.3 ft					
2.5							
5.0							
7.5							
10.0							
12.5							
15.0							
17.5							

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

BENNINGTON
BF 1000 (20)
VT Rt 9, Bridge No. 6 Over Walloomsac

Boring No.: **B-2E**
Page No.: 1 of 1
Pin No.: z12j606
Checked By: TAD

Boring Crew: New England Boring, Derry, NH, LGH (Stantec)
Date Started: 10/14/20 Date Finished: 10/14/20
VTSPG NAD83: N 138826.55 ft E 1456403.72 ft
Station: 12+20.86 Offset: 9.17' LT
Ground Elevation: 730.67 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	24 in	30 in.
Hammer/Rod Type:	Safety/N	
Rig:	Truck/Mobile B-53	$C_E = 1$

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.3 ft					
		5 inches of concrete, 0.3 ft - 0.7 ft					
		Auger encountered a void of unknown depth, partially filled with soil, 0.7 ft - 1.0 ft					
		Hole stopped @ 1.0 ft					
2.5							
5.0							
7.5							
10.0							
12.5							
15.0							
17.5							

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

BENNINGTON
BF 1000 (20)
VT Rt 9, Bridge No. 6 Over Walloomsac

Boring No.: **B-3**
Page No.: 1 of 2
Pin No.: z12j606
Checked By: TAD

Boring Crew: New England Boring, Derry, NH, LGH (Stantec)
Date Started: 10/12/20 Date Finished: 10/12/20
VTSPG NAD83: N 138810.02 ft E 1456515.30 ft
Station: 13+23.48 Offset: 32.24' RT
Ground Elevation: 730.81 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	24 in	30 in.
Hammer/Rod Type:	Safety/N	
Rig:	Truck/Mobile B-53	C_E = 1

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.5 ft								
		Visual Classification, GrSa, brn, Dry, Rec. = 0.75 ft, **-FILL-**				18-16-12 (28)				
2.5		Visual Classification, GrSa, brn, Dry, Rec. = 0.3 ft, **-FILL-**				15-20-17-20 (37)				
5.0		Visual Classification, SiGrSa, brn, Moist, Rec. = 0.25 ft, **-FILL-**				15-18-21-45 (39)				
7.5		Visual Classification, SiGrSa, brn, Wet, Rec. = 0.25 ft, **-FILL-**				10-7-9-10 (16)				
		Field Note:, Cobbles/boulders								
10.0		Visual Classification, GrSa, brn, Wet, Rec. = 0.3 ft, **-COBBLE/BOULDER LAYER-**				10-9-10-50/3" (19)				
12.5		Field Note:, Based on drill action cobbles and boulders are present from approximately 11 to 13 feet., Rec. = 0.0 ft, **-COBBLE/BOULDER LAYER-**				50/0" (R)				
		13.0 ft - 15.0 ft, Advanced roller bit through bedrock from 13 to 15 feet.				Top of Bedrock @ 13.0 ft				
15.0 17.5		15.0 ft - 20.0 ft, Light gray, Dolomite, Moderately hard, Slightly weathered, Poor rock, NQDC, Joints are low angle to moderately dipping, rough, slightly discolored, partly open. RMR = 32	1 (30)	100 (23)	2.5 2.5 2 2 2.5					

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

BENNINGTON
BF 1000 (20)
VT Rt 9, Bridge No. 6 Over Walloomsac

Boring No.: **B-3**
Page No.: 2 of 2
Pin No.: z12j606
Checked By: TAD

Boring Crew: New England Boring, Derry, NH, LGH (Stantec)
Date Started: 10/12/20 Date Finished: 10/12/20
VTSPG NAD83: N 138810.02 ft E 1456515.30 ft
Station: 13+23.48 Offset: 32.24' RT
Ground Elevation: 730.81 ft

	Casing	Sampler
Type:	WASH BORE	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	24 in	30 in.

Hammer/Rod Type: Safety/N
Rig: Truck/Mobile B-53 C_E = 1

Groundwater Observations

Date	Depth (ft)	Notes

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
22.5	▨	20.0 ft - 25.0 ft, Light gray, Dolomite, Moderately hard, Fresh, Fair rock, NQDC, Joints are low angle to moderately dipping, rough, fresh, tight to partly open. RMR = 42	2 (15)	100 (83)	2.5 2 2.5 2 2					
25.0 27.5	▨	25.0 ft - 29.0 ft, Advanced roller bit through bedrock from 25 to 29 feet.								
30.0 32.5	▨	29.0 ft - 34.0 ft, Light gray, Dolomite, Moderately hard, Fresh, Fair rock, NQDC, Joints are low angle to moderately dipping, rough, fresh, tight. RMR = 42	3 (15)	96 (87)	3.5 3.5 2.5 2 2					
35.0 37.5	▨	34.0 ft - 39.0 ft, Light gray, Dolomite, Moderately hard, Fresh, Fair rock, NQDC, Joints are low angle to moderately dipping, rough, fresh, tight. RMR = 46	4 (15)	98 (92)	2.5 2.5 2.5 2 2					
		Hole stopped @ 39.0 ft								

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

Appendix C

Rock Core Photographs

Rock Core Photo B-1

Boring ID	Core Run	Depth (ft)	Recovery (in)	RQD		Drill Time (min/ft)
B-1	C-1	15.0-20.0	52	23 in	38%	2.5, 1.5, 1.5, 2.0, 2.0
B-1	C-2	20.0-25.0	60	36 in	60%	2.0, 2.0, 2.0, 2.0, 2.0
B-1	C-3	25.0-30.0	60	32 in	53%	2.5, 2.0, 2.0, 2.5, 2.0
B-1	C-4	30.0-35.0	59	48 in	80%	2.5, 2.0, 2.0, 1.5, 1.5

Rock Core Photo B-3

Boring ID	Core Run	Depth (ft)	Recovery (in)	RQD	Drill Time (min/ft)
B-3	C-1	15.0-20.0	60	14 in 23%	2.5,2.5,2.0,2.0,2.5
B-3	C-2	20.0-25.0	60	50in 83%	2.5,2.0,2.5,2.0,2.0
B-3	C-3	29.0 - 34.0	58	52 in 87%	3.5,3.5,2.5,2.0,2.0
B-3	C-4	34.0-39.0	59	55 in 92%	2.5,2.5,2.5,2.0,2.0