Alignment and Structure Study Report

Bethel BHF 0241(38)
Vermont Route 12 over Gilead Brook
Bethel, VT

Prepared for

Vermont Agency of Transportation

May 2013

CHA Project No. 23825

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CHA

<u>Alignment and Structure Study Report</u>

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EXECUTIVE SUMMARY

The intent of the Bethel BHF 0241(38) project is to provide a new bridge structure to carry Vermont Route 12 over the Gilead Brook in Bethel. The preferred roadway alignment and structure type selected was determined based upon the key issues, listed below in the order of importance as perceived at the time in which this report was developed:

1. No permanent historical or archaeological impact to the Old Christ Church property located in the northeastern quadrant of the project

2. Limited impacts to other environmental resources

3. Limited closure of Vermont Route 12 which is the primary alternate route for Interstate 89 in this area of the state

4. Limited impacts to adjacent properties and need for property acquisitions and/or displacements

5. Bridge structure cost

6. Aesthetics of the proposed bridge structure

As discussed with the Vermont Agency of Transportation (VTrans), CHA has evaluated three horizontal alignments, two vertical profiles, and three structure types. In accordance with Section 6.1.4.3 of the VTrans Structures Manual, 2004 Edition, CHA recommends the preferred alignment and structure type, summarized below, with supporting information and details of the evaluations presented in subsequent sections of this report.

- The replacement of Bridge No. 38, which carries Vermont Route 12 over Gilead Brook, will be constructed on an alignment which closely follows the existing alignment.

- Traffic on Vermont Route 12 will be maintained on-site by a temporary roadway constructed to the east of the existing structure with a temporary bridge to cross Gilead Brook.

- The proposed replacement structure will be three span, continuous steel multi-girder.

- The proposed superstructure will be supported by integral abutments and conventional hammerhead piers all constructed from cast-in-place concrete.

In accordance with Section 6.1.4.2 of the VTrans' Structures Manual, 2004 Edition, an Advantages/Disadvantages Matrix based on the key issues listed above is presented in Appendix B and summarizes each developed alternative.

BACKGROUND

The purpose of this report is to describe significant existing features at the project location, present the information determined from the preliminary hydrologic and hydraulic analysis, summarize findings contained in the preliminary foundations recommendations report, discuss potential resource impacts within the project limits, evaluate prospective alignment solutions, assess structure alternatives and provide recommendations, including estimated construction costs, for the proposed Vermont Route 12 crossing.

Project Location

A project location map is contained in Appendix A.

For the purposes of this Alignment and Structure Study Report, Vermont Route 12 is considered to be oriented in a north-south direction.

Existing Bridge Structure

The existing bridge, constructed in 1928, carries Vermont Route 12 over Gilead Brook in a 326 ft long, four span arrangement with no skew.

The superstructure is comprised of steel multi-girder (with five girders) approach spans and two steel deck truss interior spans (with two longitudinal trusses). All four spans are simply supported with the approach spans each having a length of 40’-0” and the two interior spans each having a length of 120’-2”. A concrete deck is supported by the multi-girder system in the approach spans and by steel floorbeams in the interior span which are then supported by the steel deck trusses.

The multi-girder approach spans are supported on conventional concrete abutments and framed into the top of the steel deck trusses. The steel deck trusses are supported on concrete piers.

The southern abutment and southernmost pier are founded on spread footings on ledge at an unknown depth. The remaining three substructures are founded on timber piles driven to an unknown depth.

There are overhead utility lines running parallel to the roadway adjacent to each fascia. These utilities are supported by poles adjacent to the existing abutments and are not supported by or attached to the existing bridge. Along the western side of the roadway the overhead lines consist of electric, cable TV, and telephone. A buried telephone line exists along the eastern side of the roadway and becomes an aerial line at the location of the existing bridge.

The original cast-in-place concrete deck was replaced in 1971. According to available plans, the rehabilitation also included replacement of the expansion joints, the bearings, the deck drainage system, the concrete backwalls at the abutments, and portions of the concrete approach slabs. New drainage inlets were also installed behind the abutments.

Existing Approach Roadway

Vermont Route 12 is a two lane, non-NHS (National Highway System), rural major collector roadway with an average daily traffic (ADT) volume of approximately 4300 vehicles. Approximately 6.2% of daily traffic consists of heavy trucks generated by local commercial and industrial land uses. The posted speed limit is 50 mph in the project area. Vermont Route 12 is also the alternate route in this area of the state in the event that Interstate 89 is closed to traffic.

The existing roadway consists of uncurbed, bituminous concrete pavement. The southern roadway approach is approximately 28 feet wide and relatively tangent for a distance of approximately 320 feet from the existing bridge joint. Beyond that distance, there is a point of curvature (PC) for a horizontal curve (radius of 1400 feet) to the east. The northern roadway approach is approximately 26 feet wide and follows a 175 feet tangent from the bridge joint to a point of curvature (PC) and curves to the east with a radius of 780 feet.

Spring Hollow Road (TH-84) intersects Vermont Route 12 from the east creating a skewed intersection within the southern approach to the bridge. The intersection is located approximately 250 feet south of the existing bridge.

Along the northern approach to the bridge, Gilead Brook Road (TH-7) intersects Vermont Route 12 from the west creating a T-intersection. The intersection is located approximately 400 feet from the northern end of the existing bridge.

The existing bridge is on a tangent grade of 5.1% between two sag vertical curves on the southern and northern approach. Beyond the adjacent vertical curves, the grade along the southern approach is approximately -3.4% and the grade on the northern approach is approximately 7.8%.

The project area contains a combination of an open and a closed drainage system for stormwater. The northern approach allows water to run off into ditches along the existing roadway. On the southern approach, there are drop inlets in both shoulders of the northbound and southbound lanes which outlet into ditches at the toes of their respective embankments along Vermont Route 12. There is a third drop inlet in the pavement on the south side of Spring Hollow Road (TH-84). The corresponding outlet pipe runs underneath the town highway and outlets into a ditch along the eastern side of Vermont Route 12.

Other utilities within the project area include overhead electrical, cable TV, and telephone wires which run along the western side of the roadway. Buried telephone wires runs along the eastern side of the roadway becoming overhead at the existing bridge.

<u>Existing Feature Crossed</u>

A preliminary review of the Flood Insurance Study (FIS) for Windsor County (September 2007), indicates that there is no detailed information available for Gilead Brook. As such, CHA developed a hydraulic model to evaluate the existing bridge and to determine the allowable open area for a replacement structure in order to meet VTrans and Federal Emergency Management Agency (FEMA) design guidelines.

Gilead Brook flows from west to east in the vicinity of the bridge. At the upstream approach, Gilead Brook is approximately 100 ft wide and 1.5 ft deep during ordinary high water. It should be noted that the channel and banks sustained significant damage during Tropical Storm Irene in August of 2011. There have been no plans to restore Gilead Brook to its pre-Irene planform and as such, the survey for the Alignment and Structure Study Report reflects the current condition of the riparian corridor.

The results of the modeling indicate that the existing structure exceeds the VTrans' hydraulic design guidelines, providing over 25 ft of freeboard during the 50-year flood event.

A scour evaluation of the existing bridge indicated that the alignment of the pier with approaching flow results in predicted depths that are 50% larger than those associated with a well-aligned substructure.

<u>Subsurface Investigations</u>

A Preliminary Foundations Recommendations for Alignment and Structure Study Report was completed by CHA on March 4, 2013 (See Appendix F).

Based on a review of the *Surficial Geologic Map of Vermont*, the existing bridge foundations are likely located within horizontally bedded deposits of gravel. Glacial till deposits are noted to the north and south. It is anticipated that the natural soils above bedrock may contain numerous cobbles and boulders due to the observed cobbles on the river banks and notes on the 1928 record drawings.

Additionally, according to the *Bedrock Geologic Map of Vermont*, conglomerate and conglomerate quartzite bedrock cross the site from northwest to southeast and is anticipated to be the predominate rock type at the existing bridge. Quartz-muscovite phyllite and silicic phyllite, and garnet-rich biotite-muscovite-quartz schist are located immediately northeast of the site. Bedrock is exposed at the toe of the southern slope.

KEY ISSUES

VTrans' Historic Preservation Office (SHPO) and Archaeological Issues

Coordination with VTrans identified several historical/archaeological resources in the vicinity of the project. These include a historic church and cemetery located in the northeastern quadrant of the project area, and a stone retaining wall and stone fence posts located along the former Vermont Route 12 right of way. VTrans' Historic Preservation Officer and VTrans' Archaeology Officer determined that the church and cemetery are the only resources of concern from a historical and archaeological perspective, and that impacts to the church property should be avoided, if possible. It should be noted that VTrans' Right-of-Way Section is currently researching the property boundaries in the vicinity of the project.

Additionally, the existing bridge is listed on the National Register of Historic Places by the National Park Service. The bridge was constructed in 1928 following the destruction of the previous structure during the flood of 1927. It is considered historic due to the unique riveted steel Warren deck truss superstructure which was a standardized method of construction of long span bridges during the 1928 -1930 reconstruction period in Vermont when approximately 1600 bridges were built across the state. The original bridge railings were made of angles and channels with a latticed upper railing supported on T-section stanchions. This railing was replaced in the deck rehabilitation in 1971 with the new rails incorporating an ornamental lattice detail to match the original.

Other Environmental Issues

In an internal VTrans' office memorandum dated February 14, 2011, it was noted that no wetlands, agricultural soils, nor species of special concern were observed to be in the vicinity of the crossing of Vermont Route 12 and Gilead Brook. Gilead Brook is a cold-water trout stream and therefore in-stream construction activities need to be minimal and erosion control should be strictly enforced. It should be noted that the referenced memo was written at a time when the bridge was being considered for rehabilitation and not replacement. CHA will coordinate with the VTrans' Environmental Section to make final determinations on these issues during final design.

There is a reclaiming project currently ongoing, Bethel-Randolph STP 2921(1), which overlaps with the subject bridge replacement project limits. According to the resource identification file for the reclaiming project, there may be endangered and threatened species and habitat in the vicinity of the bridge. However from the Categorical Exclusion Environmental Analysis Sheet dated January 14, 2013, item 6 "Threatened and Endangered Species and Habitat Present in the Project Area" is checked "NO". CHA has progressed this study under the assumption that there are no endangered and threatened species and habitat in the project vicinity and will coordinate with VTrans' Environmental Section during final design for a final determination.

Maintenance and Protection of Traffic (M&PT) on Vermont Route 12

In this rural area of the state, Vermont Route 12 is the alternate route for traffic in the event that Interstate 89 is closed to traffic. Additionally, it is one of only a few north-south routes connecting residents between the towns of Bethel and Randolph. The closure of Vermont Route 12 would significantly impact the time it takes first responders to reach residents and businesses in the event of an emergency. It would particularly affect White River Valley Ambulance which is located at the southern end of the project limits. Therefore, the maintenance of traffic on Vermont Route 12 during construction is of high importance.

Right-of-Way Issues

According to current tax maps the properties immediately to the south of Gilead Brook are owned by Richard A. Davis on both sides of Vermont Route 12. On the southwestern property, approximately 120 feet from the existing bridge, there is a residence structure located approximately 25 feet from the edge of the existing roadway. There are also two driveways providing access to this structure.

Farther south within the project limits, there is a property on the western side of Vermont Route 12 owned by Richard & Madge Davis. This property has a residence structure located approximately 85 feet from the edge of the existing roadway with a single driveway for access. On the eastern side of Vermont Route 12, there is a property owned by White River Valley Ambulance with a driveway at the end of the proposed project limits that provides access to an emergency vehicle garage facility.

The property in the immediate northwestern quadrant is owned by Edith Reynolds c/o Robert Reynolds with no structures located near the existing roadway. In the immediate northeastern quadrant, there are two properties, one owned by Jeffrey Townsend with no structure (and ownership may be disputed according to VTrans' Right-of-Way personnel), and Old Christ Church which has the church structure and cemetery with driveway access, and has been determined a historical and archaeological resource as noted in the previous section.

Farther north within the project limits, there are two properties, one on each side of Vermont Route 12 which are owned by Clarence & Barbara Wright. There is a residence structure located on the western property approximately 60 feet from the existing edge of roadway with a driveway for access. On the eastern property there is a private road, Tyson Justin Road, which intersects Vermont Route 12.

At the proposed northern limit of the project, there is a property on the eastern side of Vermont Route 12 owned by Brian & Susan Curtis. There are no structures or driveways associated with this property within the proposed project limits.

There are two town highways within the project limits. Spring Hollow Road (TH-84) has a skewed intersection on the eastern side of Vermont Route 12 approximately 250 feet south of the existing bridge. Gilead Brook Road (TH-7) has a T-intersection on the western side of Vermont Route 12 approximately 400 feet north of the existing bridge.

Proposed Bridge Structure Costs

The selection of the preferred alignment and M&PT alternatives will be based on the various impacts to the previous four key issues and will not be influenced by monetary costs. However, the structure alternatives will not impact the four preceding key issues to an extent where a decision can be made between the three alternatives. Therefore, the selection of the preferred structure alternative will be based on construction cost and aesthetic considerations (see section below for aesthetic issues).

Proposed Bridge Structure Aesthetics

Due to the uniqueness of the existing deck truss superstructure and the ornamental lattice detail on the bridge railings, VTrans' Historic Preservation Officer requested that aesthetic considerations be including in the selection of the proposed replacement bridge structure.

ALIGNMENT ALTERNATIVES

CHA investigated three unique alignment alternatives for the replacement of the existing bridge as follows:

- Alignment Alternative 1: Offline West Alignment
- Alignment Alternative 2: Offline East Alignment
- Alignment Alternative 3: Online Alignment

As part of the initial scope of this study, two vertical profile alternatives were proposed. One of these profiles was for the precast concrete spandrel arch structure. This structure alternative was eliminated early in the evaluation process and is not included in this study. Therefore, there is now only a single vertical profile alternative (see the Structures Alternatives section for additional information regarding the elimination of the arch structure alternative).

The issue regarding the M&PT of Vermont Route 12 during construction will be considered separately in the following section and subsequently not affect the selection for the preferred alignment alternative.

The proposed bridge structure construction cost and aesthetics are irrelevant to the alignment alternatives and will be taken into consideration only in the Structure Alternatives section.

Alignment Alternative 1: Offline West Alignment

The Offline West Alignment Alternative would provide a new bridge shifted approximately 37 feet to the west of the existing crossing to accommodate a horizontal curve meeting AASHTO requirements for a design speed of 50 mph and maximum superelevation of 8%. The design criteria were established utilizing the Vermont State Design Standards.

This alignment alternative would not be in conflict with the SHPO and archaeological issues.

At this point in time, there is no evidence that this alignment alternative would impact threatened and endangered species and habitat.

This alignment would require ROW acquisitions from all adjacent property owners within the project limits and would affect corresponding driveway access to the properties. Specifically, significant permanent ROW acquisitions would be required from the Richard and Madge Davis (approx. 7,800 SF), Richard A. Davis (approx. 33,000 SF), Edith and Robert Reynolds (approx. 26,000 SF), and Clarence and Barbara Wright (approx. 21,000 SF) properties along the western side of Vermont Route 12. Additionally, the residence structure on the Richard A. Davis property would need to be demolished and the resident displaced. These types and quantities of ROW acquisitions are considered to be a significant impact on the property owners and therefore this is not a viable alternative for this project. For these reasons, it was eliminated.

Alignment Alternative 2: Offline East Alignment

The Offline East Alignment Alternative would provide a new bridge shifted to the east of the existing crossing. Due to the close proximity of the Old Christ Church property to the bridge in the northeastern quadrant, no alignment to the east could be created which did not significantly impact this historical and archaeological resource. Therefore, Alignment Alternative 2 is not a viable alternative for this project and was eliminated.

Alignment Alternative 3: Online Alignment

The Online Alignment Alternative will provide a new bridge in the same location as the existing crossing.

This alignment alternative will not be in conflict with the SHPO and archaeological issues.

At this point in time, there is no evidence that this alignment alternative will impact threatened and endangered species and habitat.

This alignment will require only minor permanent ROW acquisitions from all adjacent property owners within the project limits and will affect corresponding driveway access to the properties. These types and quantities of ROW acquisitions are not considered to be a significant impact on the property owners. CHA will coordinate with VTrans during final design in the acquisition of the required ROW.

This alternative satisfies the project intent and will not significantly impact the SHPO, archaeological, other environmental, and ROW issues. For these reasons, it is recommended as the PREFERRED ALTERNATIVE. Details of this alternative are provided in the remainder of this section (also see Plan, Profile, and Typical Sections in Appendix C).

This alternative will require approximately 500 feet of full-depth roadway reconstruction on the southern approach, and approximately 250 feet on the northern approach. An additional 300 feet of cold planing will be required on each approach to transition from existing pavement to full-depth reconstruction.

The Online Alignment Alternative was developed in accordance with the approved design criteria presented in Appendix D for a rural major collector roadway with a design speed of 50 mph.

The proposed approach roadway consists of 11 foot travel lanes and 4 foot shoulders. According to the Vermont State Design Standards, the minimum width for Vermont Route 12 should be 11 foot lanes and 3 foot shoulders based on the design speed and traffic volume in the vicinity of the project. The proposed roadway width was established through coordination with VTrans' proposed reclamation project, Bethel-Randolph STP 2921(1), which is providing 11 foot travel lanes and 4 foot shoulders. Bethel-Randolph STP 2921(1) ties into both approaches to this bridge replacement project.

The typical approach roadway cross slope will be normal crown with a shoulder break. Superelevation on the approaches to the bridge has been designed to be consistent with AASHTO guidance for the design of high speed rural roads.

The horizontal curvature for the proposed alignment satisfies the minimum radii requirements associated with a maximum superelevation of 8% and a 50 mph design speed. The vertical alignment utilized a design speed of 40 mph in order to meet the minimum requirements set in the Vermont State Design Standards for degree of curvature and stopping sight distance. Because the design speed of the proposed roadway is within 10 mph of the 50 mph posted speed, the proposed design does not require formal design exception in accordance with Section 5.3 of the Vermont State Design Standards provided that adequate warnings are posted.

The profile incorporates southern and northern approach grades of -3.56% and 7.87%, respectively and a 750 feet vertical sag curve extending across the proposed bridge to provide a stopping sight distance of 313 feet.

The proposed curbing on the bridge combined with the sag curve extending across the bridge and normal crown cross slope requires drop inlets and associated outlet pipes to be installed on both the northern and southern approaches to convey stormwater runoff from the pavement surface to Gilead Brook. The inlets on the northern approach will be utilized to prevent excess water from flowing onto the bridge while the inlets on the southern approach will catch water flowing off the bridge towards the low point of the sag curve. CHA anticipates that the inlets on the northern approach will likely be designed to discharge into the embankment toe on the western side of the roadway and flow overland to the bank of Gilead Brook. The inlets on the southern approach will likely be designed to discharge to a vegetated swale located along the embankment toe on the eastern side of the roadway.

Roadside barrier will be designed based on the requirements set in the 2011 Roadside Design Guide which warrants rail based on traffic volume, slope, and embankment height. Due to restrictions of the existing corridor (driveways and side roads), guardrail will be designed to approximately match the existing length with approach rail connected to the proposed bridge rail.

The construction of this alternative will also affect the intersections of Vermont Route 12 with Gilead Brook Road (TH-7) and Spring Hollow Brook Road (TH-84). The designs of these intersections will be completed during final design and will be coordinated with VTrans and the Town of Bethel.

MAINTENANCE AND PROTECTION OF TRAFFIC ALTERNATIVES

In order to maintain traffic during the construction of the replacement bridge structure, which is the third key issue of this project, there are three possible alternatives. First, traffic can be maintained on the existing bridge while a new bridge is built offline. This would require an offline alignment. However, both offline alignments were dismissed in the previous section and therefore this alternative has been eliminated. The remaining two alternatives for M&PT, the off-site detour and on-site temporary bridge, are discussed below.

M&PT Alternative 1: Off-Site Detour

The off-site detour will have no significant effect on the SHPO and archaeological issues or the other environmental issues. However, a reasonable detour route which can accommodate truck traffic does not exist due to the prohibitive length (22 miles) and the limited classifications of roadways that would be traversed on that detour.

The anticipated construction duration for this project is approximately two years. Even when considering accelerated bridge construction techniques, Vermont Route 12 would need to be closed for a minimum of three to four months in order to demolish the existing superstructure, drive piles for the proposed foundations, install prefabricated substructure units, erect the superstructure, and install precast concrete deck panels. The use of Vermont Route 12 as the alternate route for Interstate 89 in the event that the interstate needs to be closed would be precluded by bridge closure. In addition there would be an adverse impact on emergency services if the bridge was closed. Closure of Vermont Route 12 for the duration stated above is considered unacceptable.

Minimizing the closure time of Vermont Route 12 using staged construction and maintaining traffic on an alternating one way scenario is also not feasible. The existing superstructure consisting of two trusses supporting the structural deck which means it is not feasible to stage the construction with only one truss in service while constructing the new bridge adjacent.

For these reasons M&PT Alternative 1 is not a viable alternative for this project and was eliminated.

M&PT Alternative 2: On-Site Temporary Bridge

Traffic will be directed off of existing Vermont Route 12 onto a temporary alignment which will run parallel and be offset approximately 80 feet to the west of the existing and proposed alignment of Vermont Route 12. The proposed on-site temporary bridge will be placed on this alignment and cross Gilead Brook downstream of the existing structure. Due to the right-of-way restrictions and requirements set in the Vermont State Design Standards, the temporary detour was designed for a speed of 30 mph. Consequently, advanced advisory speed plaques will be required to notify drivers of the speed reduction. The on-site temporary alignment and bridge will provide appropriate M&PT during construction and will have a minor impact on the traveling public.

It is not anticipated that the on-site temporary bridge will be in conflict with the SHPO and archaeological issues.

At this point in time, there is no evidence that the temporary bridge will impact threatened and endangered species and habitat. It is possible, based on the type of temporary bridge chosen, that substructures for the temporary bridge will be constructed along the banks for Gilead Brook. During final design, information will be provided on the construction documents which will inform the contractor that in-stream construction activities shall be minimal.

Both the temporary alignment and temporary bridge structure will require temporary ROW acquisitions from the Richard A. Davis (approx. 14,000 SF) and Jeffrey Townsend (approx. 31,000 SF) properties along the eastern side of Vermont Route 12. These quantities of ROW acquisitions are not considered to be a significant impact on the property owners. CHA will coordinate with VTrans during final design in the acquisition of the required ROW.

The temporary alignment will begin near the intersection of Vermont Route 12 and Spring Hollow Road (TH-84) causing Spring Hollow Road (TH-84) to be temporarily impacted. CHA will coordinate with VTrans and the Town of Bethel during final design to ensure that access is maintained to Spring Hollow Road (TH-84).

This alternative satisfies the M&PT requirements and does not significantly impact the SHPO, archaeological, other environmental, and ROW issues. For these reasons, it is recommended as the PREFERRED ALTERNATIVE (see Plan, Profile, and Typical Sections in Appendix C).

STRUCTURE ALTERNATIVES

Based on the information provided on the Structure Inspection, Inventory, and Appraisal Sheet completed on November 30, 2011 for the existing bridge the original design live load was an AASHO H-15. Since no strengthening of the bridge members has occurred in the past, the bridge is inadequate to carry today's standard loads. A load rating analysis for the existing deck trusses was completed during the first phase of this project by CHA. As a result of this analysis, it was shown that the deck truss members need to be significantly strengthened. Strengthening of the truss members would likely require closing the bridge to traffic for a significant period of time. Due to extent of the strengthening needed, it was determined during a meeting with VTrans' Project Manager Mark Sargent that a rehabilitation of the existing deck trusses was not economically feasible and a complete bridge replacement was warranted. Subsequent coordination with the VTrans' Historic Preservation Officer and the VTrans' Archaeology Officer led to concurrence of the need for a complete bridge replacement. Also, due to the unique railing on the existing bridge, SHPO requested that the proposed bridge should have an aesthetic bridge railing that satisfies the TL-4 requirements.

CHA investigated four unique structure types for the replacement of the existing bridge. Prior to the scoping of the Alignment and Structure Study Report, a precast concrete spandrel arch structure was briefly discussed in a meeting with VTrans' Project Manager Mark Sargent. Therefore, the original scope of this study included the arch as an alternative. In the evaluation process, it became evident that only one of the arch foundations could be constructed on rock due to site conditions. Compounding this difficulty, an arch at this site would be relatively flat, generating high lateral loads on the arch foundations. An arch foundation needs to be designed to limit horizontal displacements to maintain the structural integrity of the arch. Early in the evaluation process, it was determined that the use of an arch would be very problematic at this location. The arch structure was therefore dropped from this study as a feasible alternative.

The remaining three structure alternatives considered are as follows:

- Structure Alternative 1: Two Span, Continuous Steel Multi-Girder
- Structure Alternative 2: Three, Simple Span, Precast/Prestressed Concrete Beams
- Structure Alternative 3: Three Span, Continuous Steel Multi-Girder

All three alternatives will be constructed using integral abutments. The abutments will be located behind the existing abutments. The preliminary foundation recommendations for the integral abutments and piers requiring deep foundations are steel H-piles driven to rock. At pier locations where the bedrock is in close proximity to the existing ground or channel bottom, the pier will be supported by spread footings bearing on rock.

The roadway profile places a sag vertical curve on the bridge. While this is not ideal from an aesthetic point of view, it is acceptable as long as the low point of the sag is not on the bridge. The low point of the sag occurs south of the structure and there is no location along the bridge profile in which the instantaneous tangential grade is less than 1% as specified by Section 2.1.1.3 of the VTrans' Structures Manual, 2010 Edition. Also, as this bridge crosses a water feature, the aesthetic issue is mitigated.

The bridge section is the same for all three alternatives and consists of a 9" cast-in-place concrete structural deck with one 11'-0" travel lane in each direction with 5'-0" shoulders. A transition from the 4'-0" shoulders on the adjacent highway sections to the 5'-0" shoulders on the bridge will occur before and after the approach slabs. VTrans' standard bridge railing, galvanized 2 rail box beam, will be used with an ornamental steel lattice treatment attached to the fascia side of the rail posts for aesthetic purposes. The added steel lattice treatment will be in the style of the railing on the existing bridge. The total out-to-out width of the bridge is 33’-0”.

Based on preliminary models for both the two span and three span configurations, the existing hydraulic opening at the bridge will be increased, resulting in no change in 50- or 100-year water surface elevations through the bridge reach.

Scour evaluations for the proposed alternatives indicated that scour depths for the substructures orientated perpendicular to the roadway are 50% larger than those aligned with the approaching flow. While skewing the substructures would reduce scour depths, there are substantial benefits to squaring up the bridge such as lower initial construction costs and lower future maintenance costs. During final design, CHA will investigate the potential benefit of skewing the substructures as well as developing other details, such as reducing the pier thickness, to mitigate scour.

All three alternatives will have the same impacts, or lack of impacts, to the SHPO, archaeological, other environmental, M&PT, and ROW issues. Therefore, the selection of the alternative will be based on the bridge structure construction cost and aesthetics.

<u>Structure Alternative 1: Two Span, Continuous Steel Multi-Girder</u>

This structure alternative would consist of two 182 foot long continuous spans of steel (weathering) plate girders with parabolic haunches at the piers. A single concrete hammerhead pier would be located at the mid-length of the bridge and is assumed to be supported by a steel H-pile deep foundation pending a formal subsurface investigation. This alternative would place a pier in the middle of the Gilead Brook channel and is not perceived to be as aesthetically pleasing as a three span bridge. This alternative does not have the lowest construction cost of the alternatives at $2,880,000. For these reasons, it was eliminated.

<u>Structure Alternative 2: Three, Simple Span, Precast/Prestressed Concrete Beams</u>

This structure alternative would consist of three 113’-4” long simple spans of precast/prestressed concrete beams. Of the two concrete hammerhead piers, one is assumed to be supported by a steel H-pile deep foundation and one is assumed to be supported by a spread footing on rock, pending a formal subsurface investigation. Due to the inherit camber in precast/prestressed beams, in combination with the sag vertical curve, large deck haunches would be required at several locations on the concrete beams. This alternative was not perceived to be as aesthetically pleasing as a haunched three span steel girder. This alternative does not have the lowest estimated construction cost of the alternatives at $2,670,000. For these reasons, it was eliminated.

<u>Structure Alternative 3: Three Span, Continuous Steel Multi-Girder</u>

This structure alternative consists of three continuous spans of 100 foot long approach spans and a 140 foot middle span. It is a steel (weathering) multi-girder system with parabolic haunches at the piers. There are two concrete hammerhead piers located at the edges of the existing channel. One is assumed to be supported by a steel H-pile deep foundation and one is assumed to be supported by a spread footing on rock, pending a formal subsurface investigation. This alternative is judged to be the most aesthetically pleasing of those considered. This alternative also has the lowest construction cost of the alternatives at $2,630,000. For these reasons, it is recommended as the PREFERRED ALTERNATIVE (see Plan, Profile, and Typical Sections in Appendix C).

COST ESTIMATE AND CONSTRUCTABILITY CONSIDERATIONS

A preliminary cost estimate was developed for the three span continuous steel multi-girder structure constructed on the online alignment. A summary of items and associated unit costs is included in Appendix E; with supporting calculations available upon request.

The estimated cost for roadway items associated with Alignment Alternative 3 is **$1,170,000** (2013).

The estimated cost for temporary items associated with Alignment Alternative 3 is **$30,000** (2013).

The estimated cost for traffic and safety items associated with Alignment Alternative 3 is **$140,000** (2013).

The estimated cost for the temporary bridge items associated with M&PT Alternative 2 is **$1,300,000** (2013).

The estimated cost for structure items associated with Structure Alternative 3 is **$2,680,000** (2013).

The total project cost, including mobilization/demobilization and a nominal contingency to account for minor items is approximately **$6,550,000** (2013).

The demolition and removal of the existing structure is included in the estimated cost for the proposed structure.

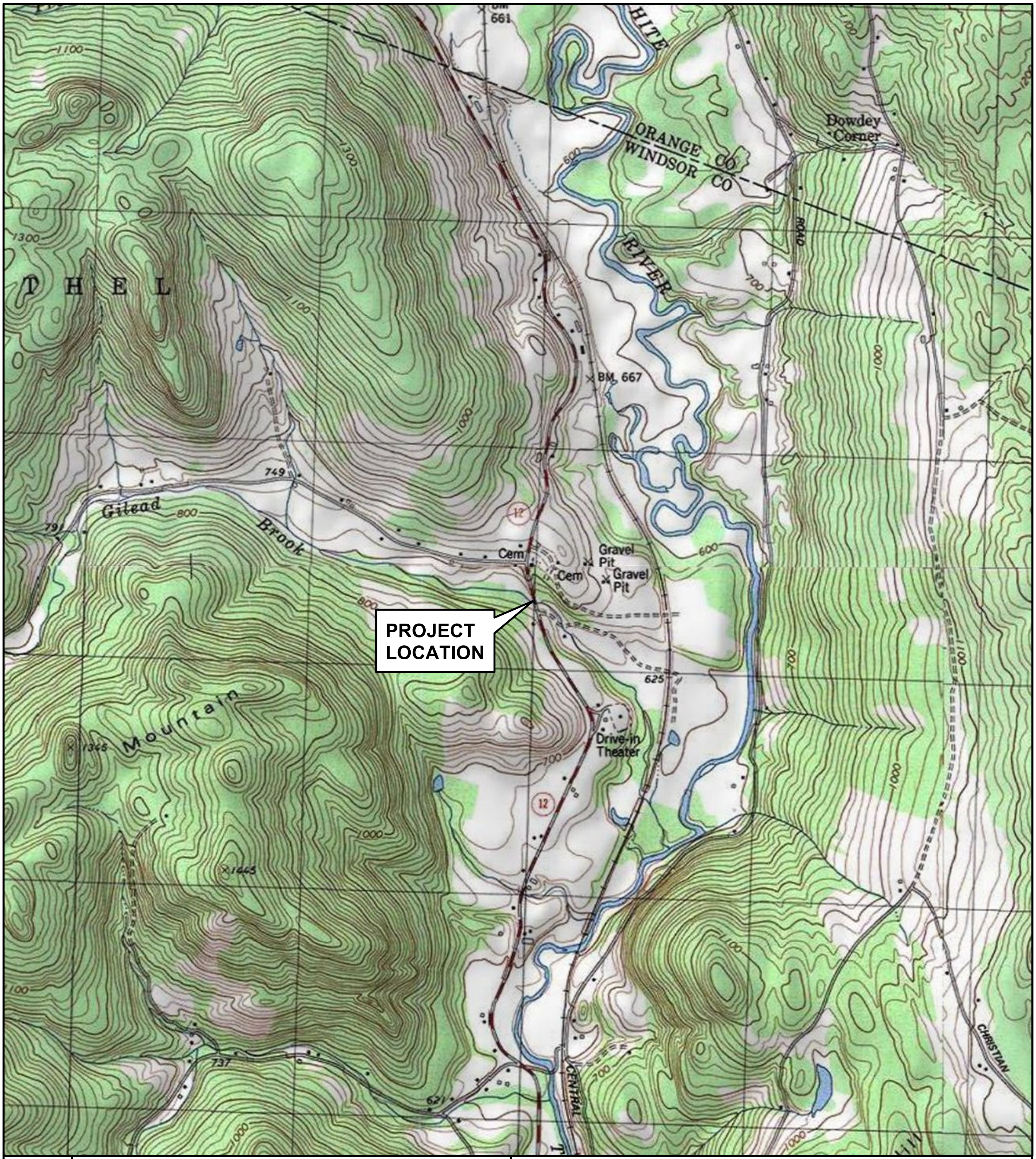
To facilitate construction of the proposed bridge and demolition of the existing structure, overhead utilities currently located along both sides of the roadway will require temporary or permanent relocation. CHA will coordinate with VTrans and the affected utilities for the relocation during final design.

To facilitate installation of the temporary bridge, cofferdams, the proposed cast-in-place concrete piers, and the multi-girder superstructure, it is likely that a temporary causeway will be required. Development of a preliminary sequencing plan indicated that the southeastern quadrant would be the most effective location for construction of the temporary causeway. It is anticipated that U.S. Army Corps of Engineers Permits will be required in order to construct this causeway.

CHA will discuss these constructability considerations with the VTrans' Structures, Hydraulic, Environmental and Utility Sections, as this project progresses during final design.

Appendix A

Project Location Map



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Project Location Map

Scale 1" = 2000'	CHA File No: 23825	Vermont Route 12 Over Gilead Brook – Bridge No. 38 (Bethel) Bethel, Windsor County, VT Randolph and Bethel USGS Quads

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Appendix B

Advantage/Disadvantage Matrix

ADVANTAGE/DISADVANTAGE MATRIX

VERMONT ROUTE 12 OVER GILEAD BROOK - BRIDGE 38

ALTERNATIVE	SHPO & Archaeological Issues[1]	Other Environmental Issues[2]	M&PT on Vermont Route 12	Right-Of-Way Issues	Proposed Bridge Structure Cost	Proposed Bridge Structure Aesthetics	Critical Perceived Benefit(s)	CONCLUSIONS
ALIGNMENT ALTERNATIVES								
Offline West Alignment	• No impact	• No impact to other environmental issues	• See M&PT Alternatives	• Significant property taking in northwestern and southwestern quadrant • House condemnation in southwestern quadrant	• See M&PT and Structure Alternatives	• See Structure Alternatives	**• Maintain traffic on existing structure during construction**	**• Dismissed due to significant ROW impacts and house condemnation in southwest quadrant [3]**
Offline East Alignment	• Significant historical and archaeological impacts to Old Christ Church in northeast quadrant	• No impact to other environmental issues	• See M&PT Alternatives	• Significant property taking in northeastern and southeastern quadrants	• See M&PT and Structure Alternatives	• See Structure Alternatives	**• Maintain traffic on existing structure during construction**	**• Dismissed due to significant historical and archaeological impacts**
Online Alignment	• No impact	• No impact to other environmental issues	• See M&PT Alternatives	• Minimal impact	• See M&PT and Structure Alternatives	• See Structure Alternatives	**• Minimum historical and archaeological, other environmental, and right-of-way impacts**	**• SELECTED ALIGNMENT ALTERNATIVE**
MAINTENANCE AND PROTECTION OF TRAFFIC ALTERNATIVES								
Off-site Detour	• No impact	• No impact	• Closure of Vermont Route 12[4] • 20 mile interstate detour • Emergency vehicle access not maintained	• No impact	• No significant cost	• See Structure Alternatives	**• Minimum construction cost**	**• Dismissed due to significant M&PT impact**
On-site Temporary Bridge	• No impact	• Minor temporary impact to other environmental issues	• Minor impact due to reduced speed on Vermont Route 12 at the temporary bridge	• Significant temporary impacts	**$1,300,000.00**	• See Structure Alternatives	**• Minimum impact to traveling public/emergency vehicle access maintained**	**• SELECTED MAINTENANCE AND PROTECTION OF TRAFFICE ALTERNATIVE**
STRUCTURE ALTERNATIVES								
Two Span, Continuous Steel Multi-Girder	• See Alignment Alternatives	• Minimal permanent environmental impact due to single, intermediate pier in channel	• See M&PT Alternatives	• See Alignment and M&PT Alternatives	**$2,930,000.00**	• The continuous steel girders, haunched at the piers, and hammerhead piers provide a aesthetically pleasing structure.	**• Minimal environmental / cultural impacts • Haunched girders and hammerhead piers contribute to aesthetics**	**• Dismissed due to construction cost**
Three, Simple Span, Precast/Prestressed Concrete Beams	• See Alignment Alternatives	• Minor permanent environmental impact due to the two piers at the edges of the channel	• See M&PT Alternatives	• See Alignment and M&PT Alternatives	**$2,710,000.00**	• Multi span precast concrete girders are not considered as aesthetically pleasing	**• Precast concrete girders are less susceptible to corrosion**	**• Dismissed due to construction cost and least aesthetically pleasing**
Three Span, Continuous Steel Multi-Girder	• See Alignment Alternatives	• Minor permanent environmental impact due to the two piers at the edges of the channel	• See M&PT Alternatives	• See Alignment and M&PT Alternatives	**$2,680,000.00**	• The continuous steel girders, haunched at the piers, with well proportioned span lengths, and hammerhead piers provides the most aesthetically pleasing structure.	**• Minimum construction cost • Haunched girders, well proportioned span lengths, and hammerhead piers provides the most aesthetically pleasing structure.**	**• SELECTED STRUCTURE TYPE ALTERNATIVE**
NO-BUILD ALTERNATIVE	***No Construction Cost or Impacts are associated with No-Build alternative.***							**• Dismissed due to non-fulfillment of project intent**

[1] All alignment alternatives include the demolition of the deficient existing structure in the final condition. The existing structure is a historical resource.

[2] No significant impacts are anticipated to threatened and endangered species and habitat for all alternatives. Temporary environmental impacts are similar for all structure type alternatives - causeway(s), cofferdam(s), and dewatering will be required for all alternatives.

[3] The cost of property acquisition and displacement of a local resident has been assumed to render this a non-preferred alignment alternative. The actual cost of the acquisition/displacement could be assessed by VTrans and compared to the temporary bridge cost.

[4] Vermont Route 12 is the alternate route for traffic in the event that Interstate 89 is closed to traffic.

Appendix C

Plan, Profile and Project Typical Sections

GENERAL PLAN

CHA

GENERAL PLAN (1 OF 3)

PROJECT NAME: BETHEL BRIDGE
PROJECT NUMBER: BHF 0241(38)

FILE NAME: z10c216_plan_01.dgn
PROJECT LEADER: M. SARGENT
DESIGNED BY: R. HENDERSON
DWG. NO.: PLN-01

PLOT DATE: 3/21/2013
DRAWN BY: P. ROTH
CHECKED BY: D. GAZALKOWSKI
SHEET 1 OF 8

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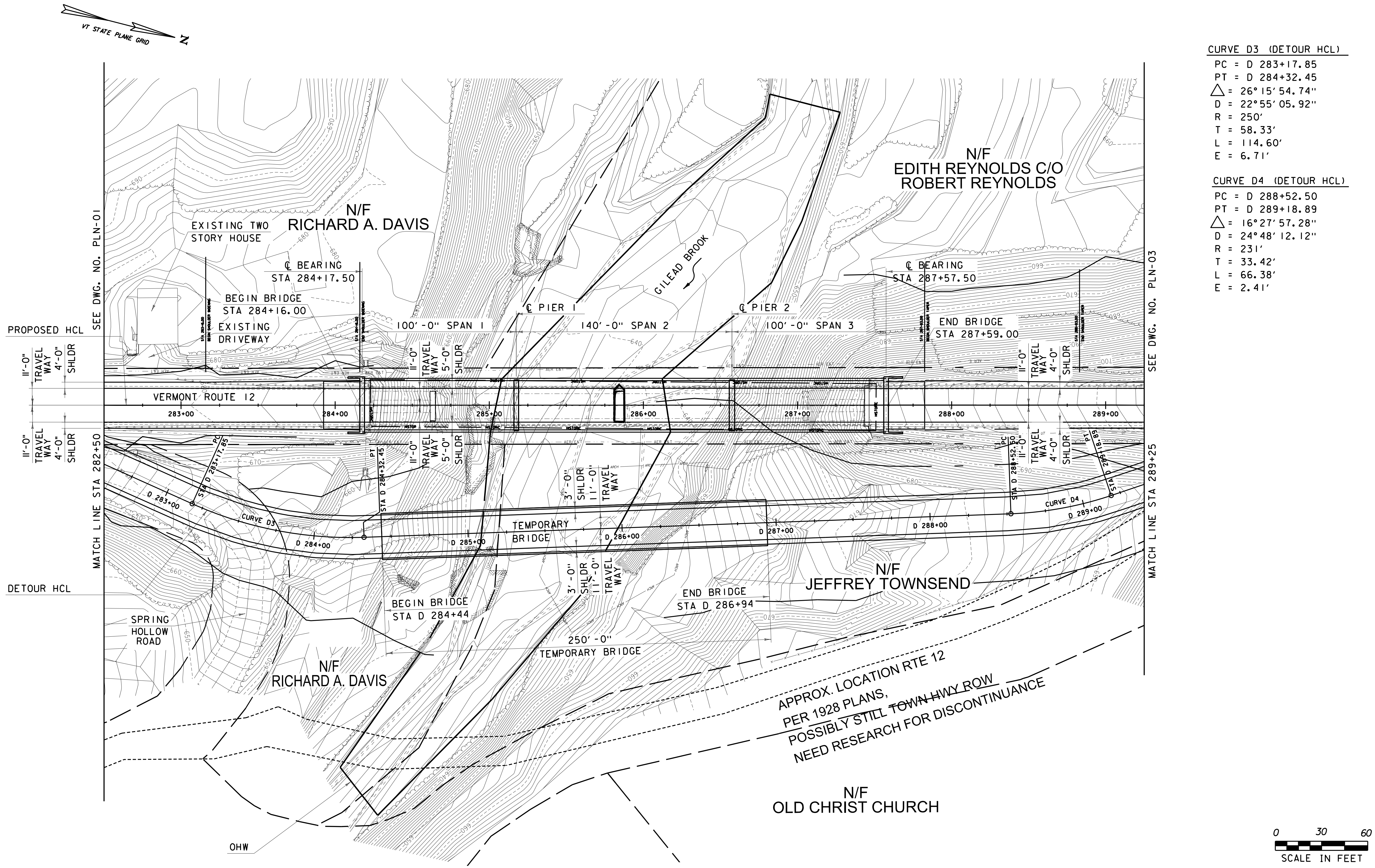
USER = 3293



GENERAL PLAN

PROJECT NAME:	BETHEL BRIDGE
PROJECT NUMBER:	BHF 0241(38)

GENERAL PLAN (2 OF 3)

CHA

FILE NAME: z10c216_plan_02.dgn
PROJECT LEADER: M. SARGENT
DESIGNED BY: R. HENDERSON
DWG. NO.: PLN-02

PLOT DATE: 5/20/2013
DRAWN BY: P. ROTH
CHECKED BY: D. GOZALKOWSKI
SHEET 2 OF 8

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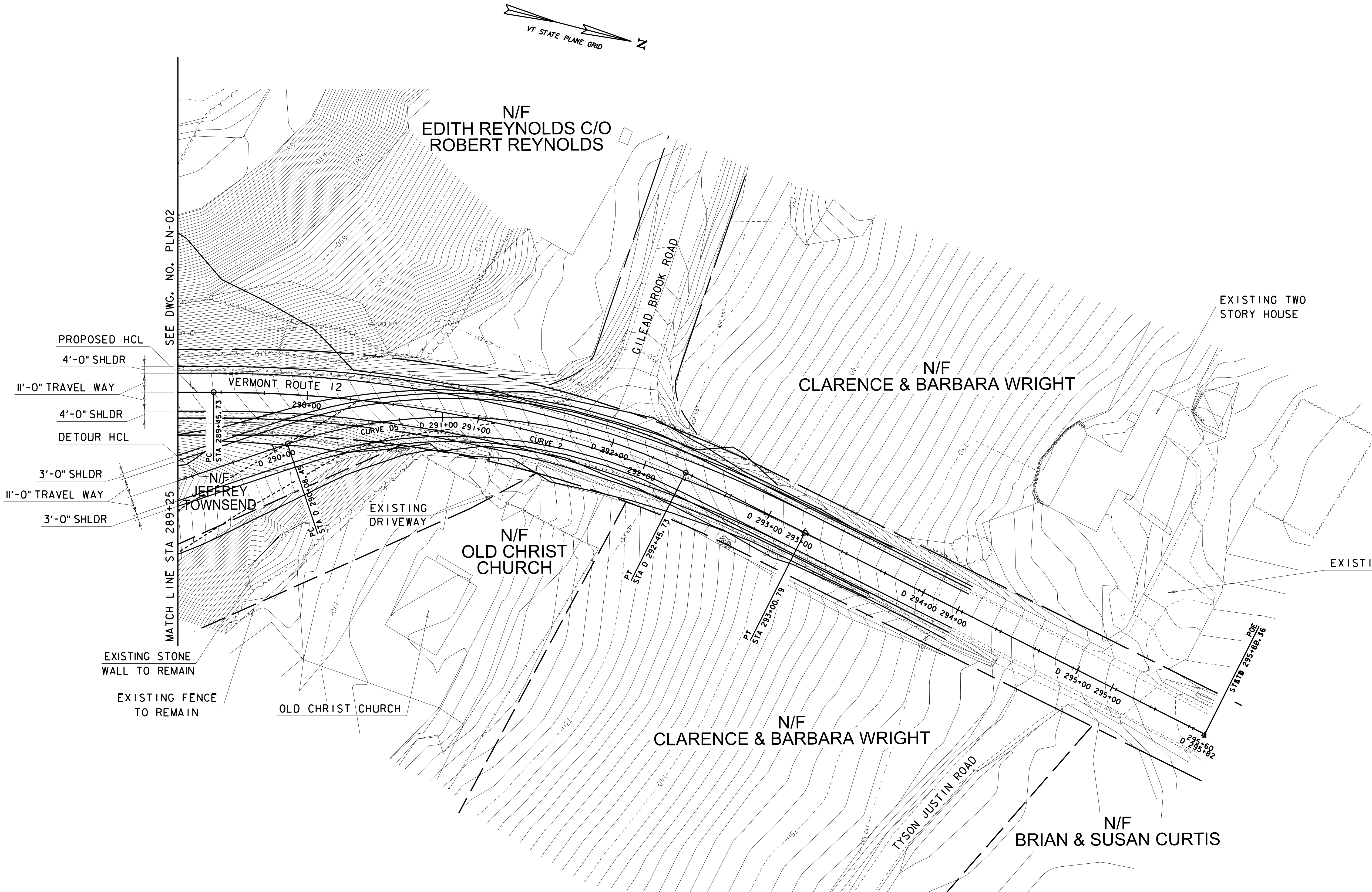


GENERAL PLAN

CURVE D2 (PROPOSED HCL)
PC = 289+45.73
PT = 293+00.79
△ = 26° 46' 05.03"
D = 7° 32' 20.11"
R = 760'
T = 180.83'
L = 355.07'
E = 21.22'

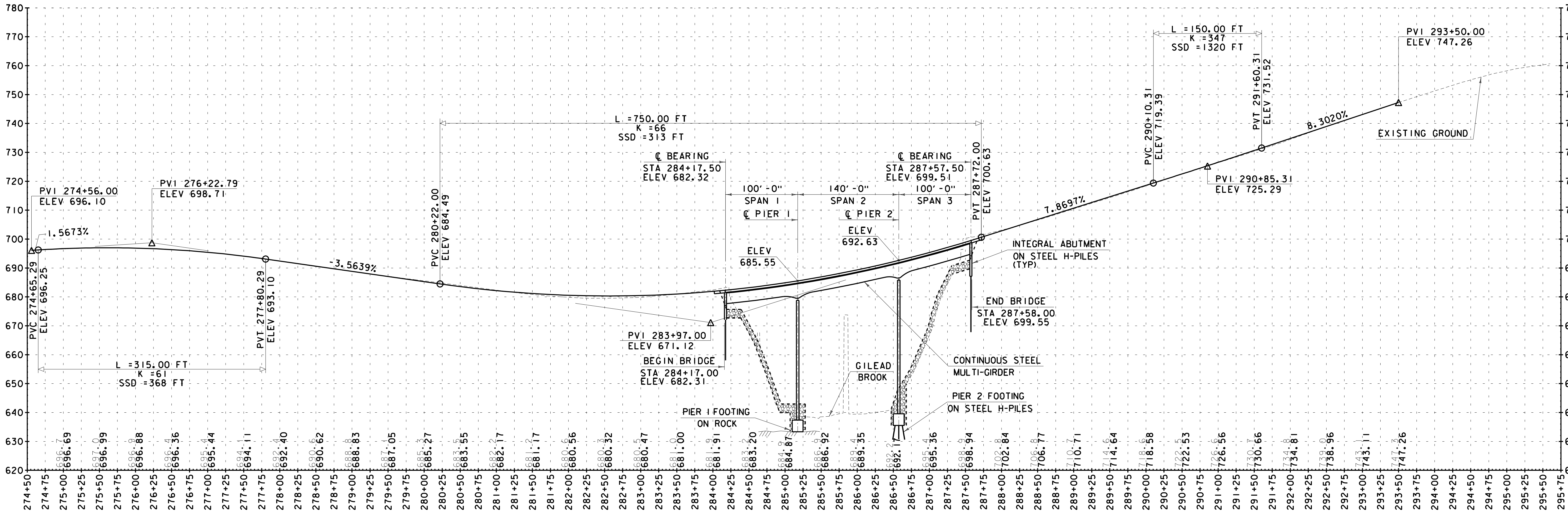
CURVE D5 (DETOUR HCL)
PC = D 290+08.45
PT = D 292+45.73
△ = 45° 19' 02.32"
D = 19° 05' 54.94"
R = 300'
T = 125.24'
L = 237.28'
E = 25.09'

0 30 60
SCALE IN FEET

CHA
III Winners Circle, PO Box 5269 • Albany, NY 12205-0269
Main: (518) 453-4500 • www.chacompanies.com

GENERAL PLAN (3 OF 3)

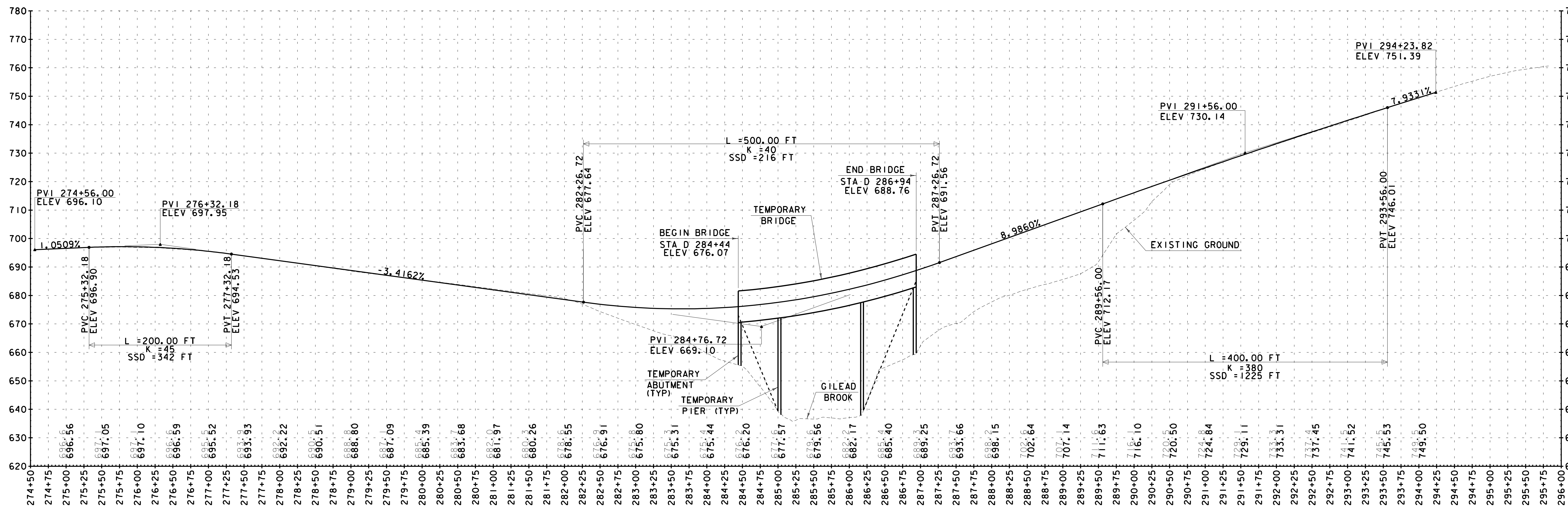
PROJECT NAME: BETHEL BRIDGE	
PROJECT NUMBER: BHF 0241(38)	
FILE NAME: z10c216_plan_03.dgn	PLOT DATE: 3/21/2013
PROJECT LEADER: M. SARGENT	DRAWN BY: P. ROTH
DESIGNED BY: R. HENDERSON	CHECKED BY: D. GOZALKOWSKI
DWG. NO.: PLN-03	SHEET 3 OF 8



PROPOSED PROFILE (3 SPAN)

CHA

PROPOSED PROFILE (3 SPAN)	PROJECT NAME: BETHEL BRIDGE PROJECT NUMBER: BHF 0241(38)
	FILE NAME: z10c216_pro_01.dgn PROJECT LEADER: M. SARGENT DESIGNED BY: R. HENDERSON DWG. NO.: PRO-01
	PLOT DATE: 3/21/2013 DRAWN BY: P. ROTH CHECKED BY: D. GOZALKOWSKI SHEET 4 OF 8



DETOUR PROFILE

DETOUR PROFILE	PROJECT NAME: BETHEL BRIDGE
	PROJECT NUMBER: BHF 0241(38)
FILE NAME: z10c216_pro_02.dgn	PLOT DATE: 3/21/2013
PROJECT LEADER: M. SARGENT	DRAWN BY: P. ROTH
DESIGNED BY: R. HENDERSON	CHECKED BY: D. GOZALKOWSKI
DWG. NO.: PRO-02	SHEET 5 OF 8

CHA

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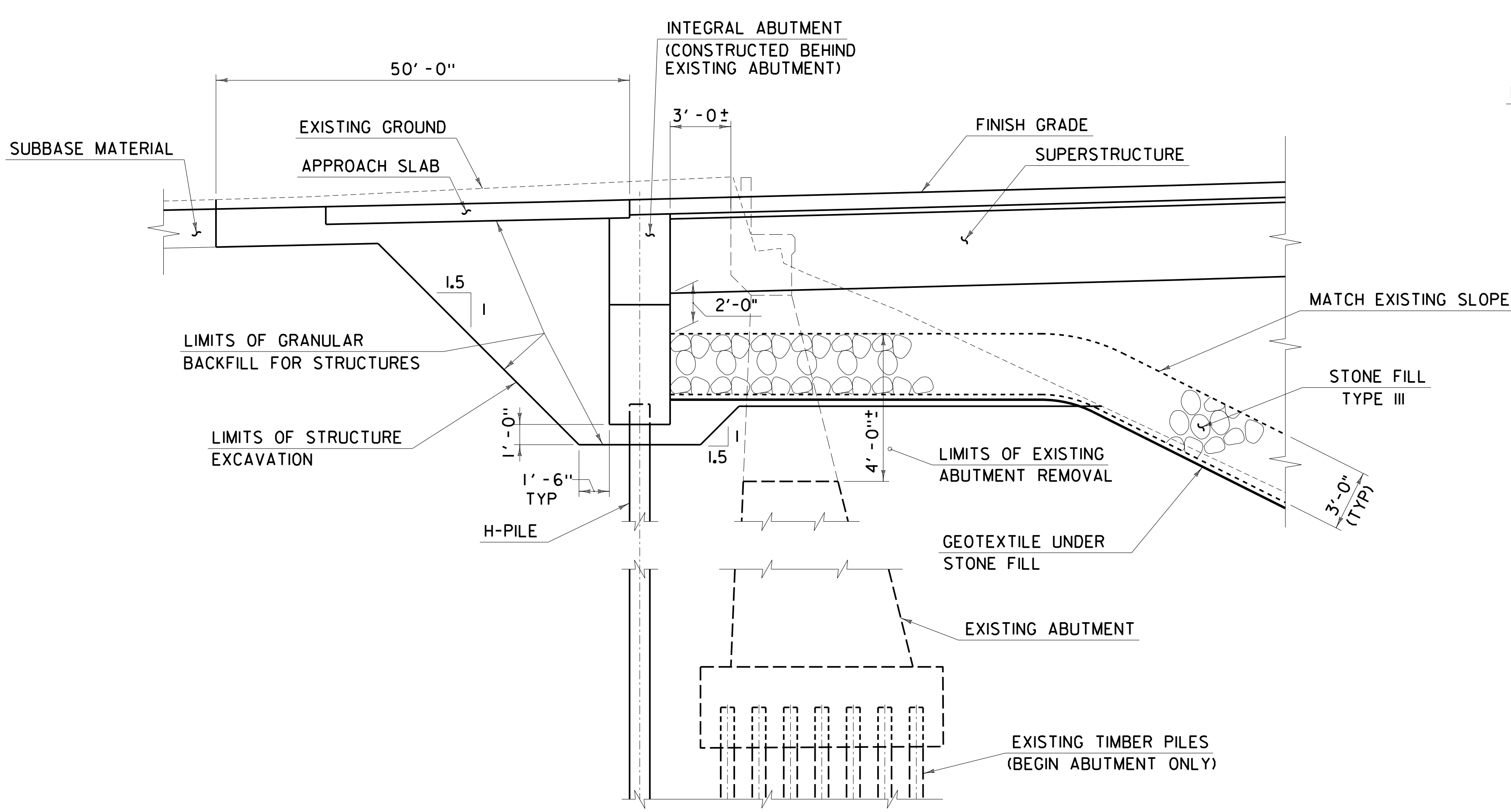
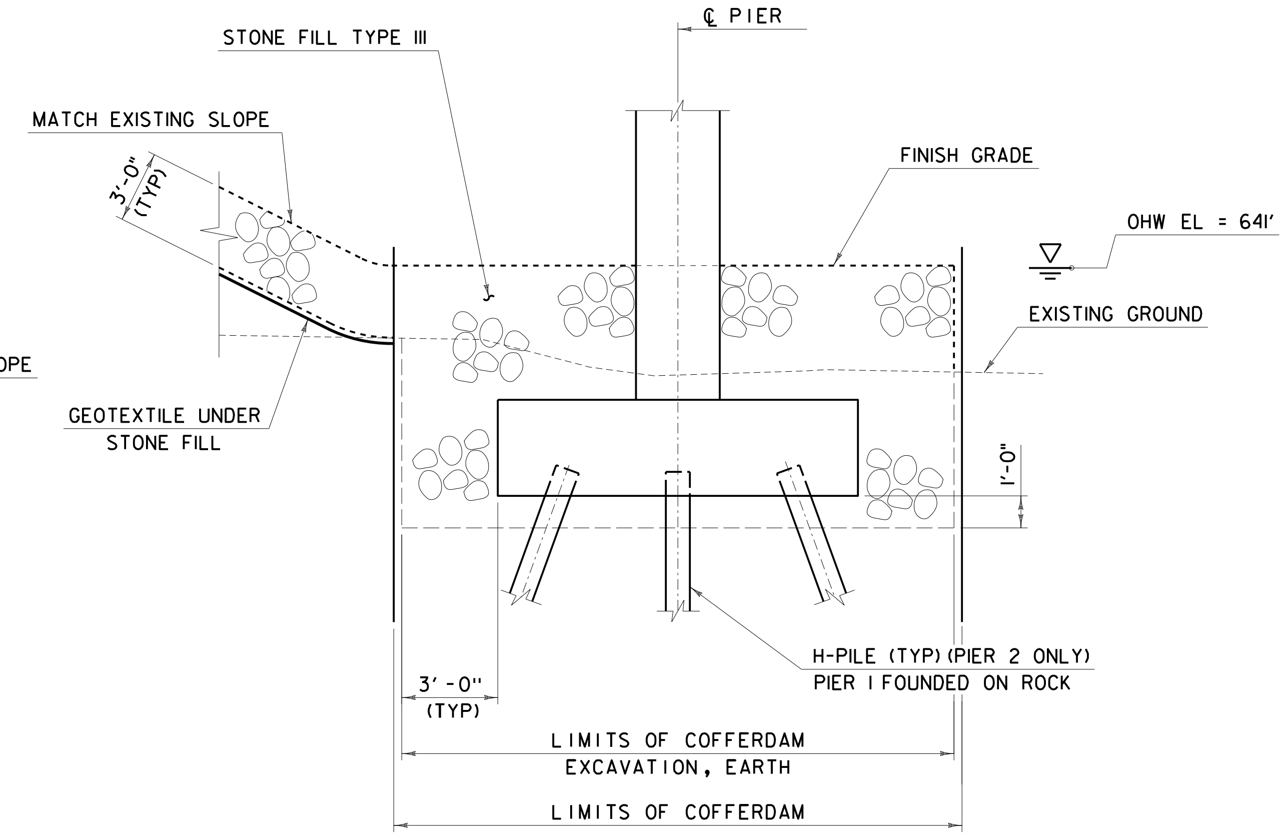
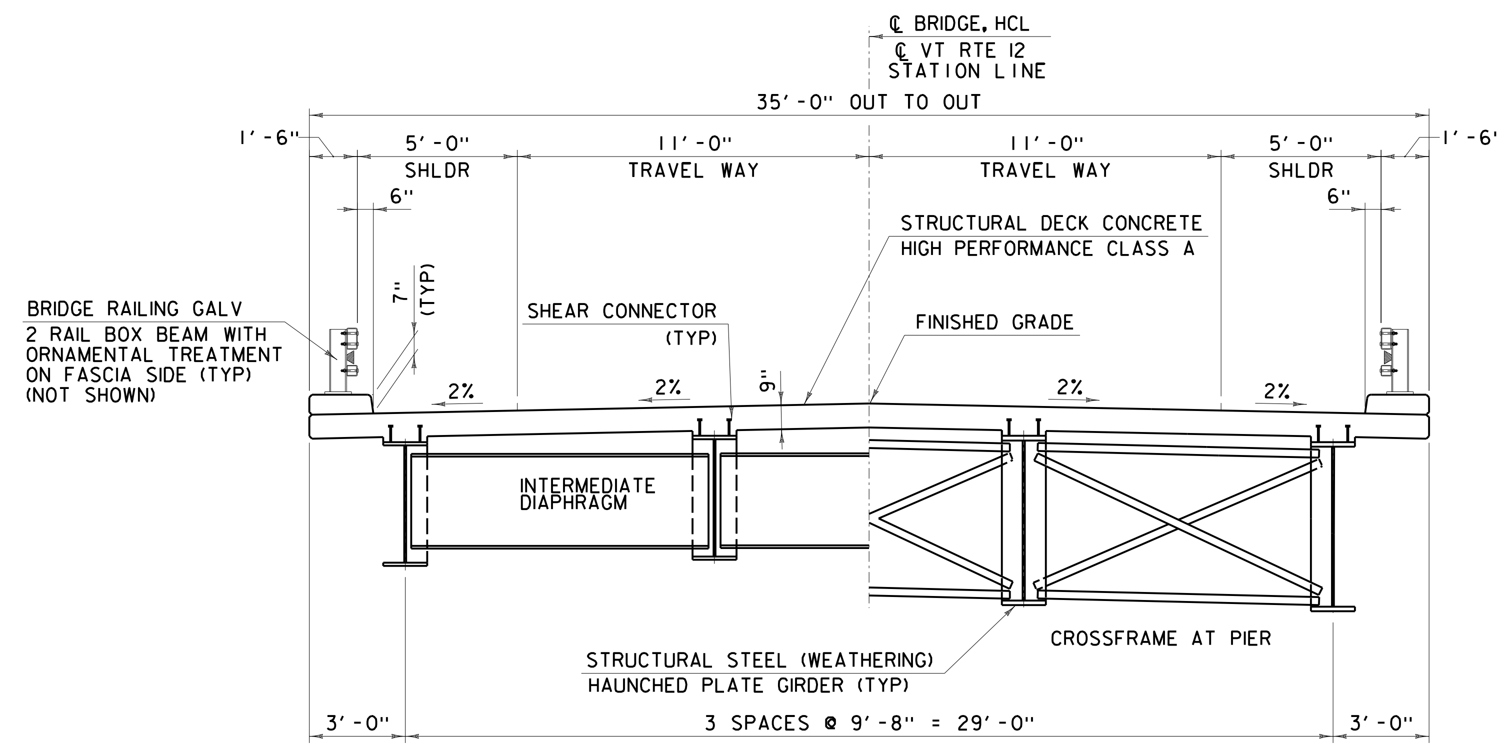
DATE/TIME = 3/21/2013
USER = 3293



TYPICAL ABUTMENT SECTION
(BEGIN ABUTMENT SHOWN, END ABUTMENT SIMILAR)



TYPICAL PIER SECTION
(PIER 1 SHOWN, PIER 2 SIMILAR)



TYPICAL BRIDGE SECTION

TYPICAL SECTIONS	PROJECT NAME: BETHEL BRIDGE PROJECT NUMBER: BHF 0241(38)	
	FILE NAME: z10c216_sec_typ_01.dgn PROJECT LEADER: M. SARGENT DESIGNED BY: R. HENDERSON DWG. NO.: TS-01	PLOT DATE: 5/16/2013 DRAWN BY: P. ROTH CHECKED BY: D. VIENI SHEET 6 OF 8

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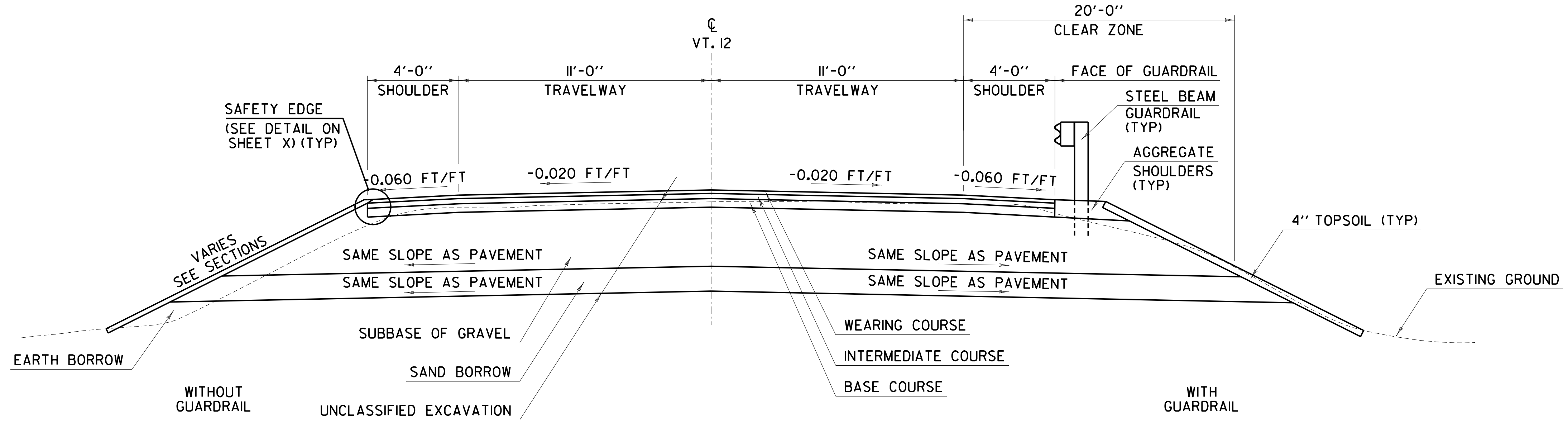
FILE NAME = V:\Projects\ANY\K2\23825\10C216\Consultants\bridge\z10c216_sec_typ_01.dgn
DATE/TIME = 5/16/2013
USER = 3293

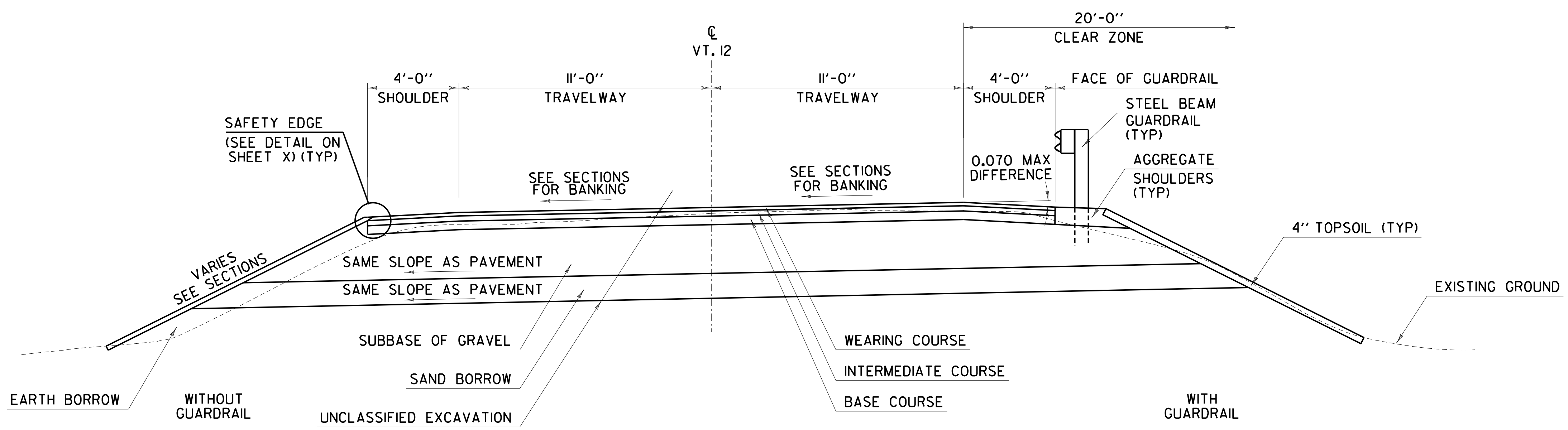
TYPICAL ROADWAY SECTION
VT ROUTE 12

MATERIAL ITEM	THICKNESS TOLERANCE
SURFACE	
-PAVEMENT COURSES (TOTAL DEPTH)	+/- 1/4"
-AGGREGATE SURFACE COURSE	+/- 1/2"
SUBBASE	+/- 1"
SAND BORROW	+/- 1"

1.5" SUPERPAVE BITUMINOUS CONCRETE PAVEMENT, WEARING COURSE, TYPE IVS
2" SUPERPAVE BITUMINOUS CONCRETE PAVEMENT, INTERMEDIATE COURSE, TYPE IVS
3.5" SUPERPAVE BITUMINOUS CONCRETE PAVEMENT, BASE COURSE, TYPE IIS
18" SUBBBASE OF GRAVEL
18" SAND BORROW



NORMAL SECTION
STA 282+50.00 - STSA 284+16.00



BANKED SECTION
STA 279+25.00 - STS 282+50.00
STA 287+83.00 - STA 290+25.00

NOT TO SCALE

TYPICAL SECTIONS SHEET #1	PROJECT NAME: BETHEL BRIDGE PROJECT NUMBER: BHF 0241(38)	
	FILE NAME: z10c216_typ_01.dgn PROJECT LEADER: D.E.G. DESIGNED BY: D.P.C. DWG. NO.: TYP 1	PLOT DATE: 3/21/2013 DRAWN BY: M.E.D CHECKED BY: J.P.S. SHEET 7 OF 8

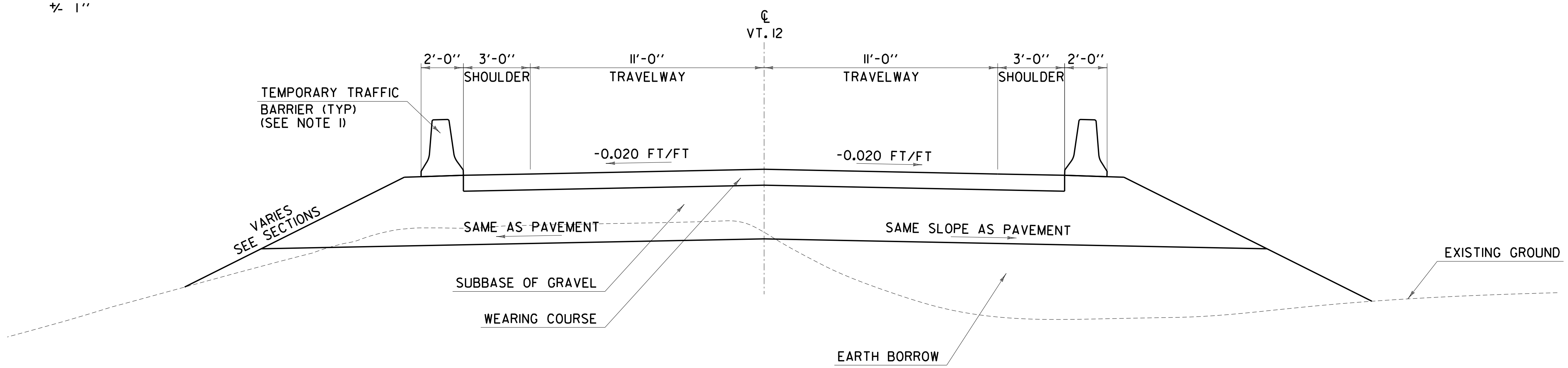
CHA

FILE NAME = V:\Projects\ANY\K2\23825\10C216\Consultants\highway\z10c216_typ_01.dgn
DATE/TIME = 3/21/2013
USER = 3293

MATERIAL ITEM	THICKNESS TOLERANCE
SURFACE	
-PAVEMENT COURSES (TOTAL DEPTH)	+/- 1/4''
-AGGREGATE SURFACE COURSE	+/- 1/2''
SUBBASE	+/- 1''
SAND BORROW	+/- 1''

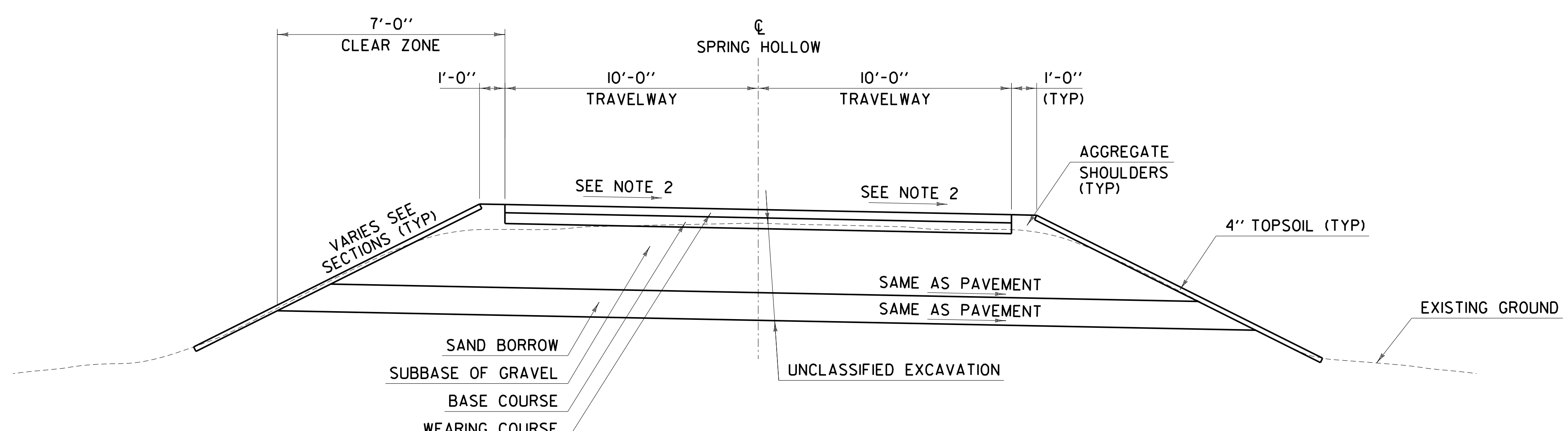
TYPICAL DETOUR SECTION
VT ROUTE 12

3'' SUPERPAVE BITUMINOUS CONCRETE PAVEMENT, WEARING COURSE, TYPE IVS
12'' SUBBASE OF GRAVEL



TYPICAL SECTION
SPRING HOLLOW ROAD

3'' SUPERPAVE BITUMINOUS CONCRETE PAVEMENT, WEARING COURSE, TYPE IVS
3'' SUPERPAVE BITUMINOUS CONCRETE PAVEMENT, BASE COURSE, TYPE IIS
24'' SUBBASE OF GRAVEL
12'' SAND BORROW



NOT TO SCALE

NOTES:

1. ALL TEMPORARY TRAFFIC BARRIER SHALL BE PINNED AND HAVE RETROREFLECTIVE MARKERS ON BARRIER.
2. SEE SPRING HOLLOW CROSS SECTIONS FOR CROSS SLOPE INFORMATION

CHA
18 Winners Circle, PO Box 5269 • Albany, NY 12205-0269
Main (518) 453-4500 • www.chacompanies.com

TYPICAL SECTIONS SHEET #2	PROJECT NAME: BETHEL BRIDGE	
	PROJECT NUMBER: BHF 0241(38)	
	FILE NAME: z10c216_typ_02.dgn	PLOT DATE: 3/21/2013
	PROJECT LEADER: D.E.G.	DRAWN BY: M.E.D
	DESIGNED BY: D.P.C.	CHECKED BY: J.P.S.
	DWG. NO.: TYP 2	SHEET 8 OF 8

FILE NAME = V:\Projects\ANY\K2\23825\10C216\Consultants\highway\z10c216_typ_02.dgn
DATE/TIME = 3/21/2013
USER = 3243

Geometric Design Criteria

Appendix D

BETHEL BHF-0241(38) - VERMONT ROUTE 12 OVER GILEAD BROOK

VERMONT ROUTE 12

GEOMETRIC DESIGN CRITERIA - RURAL MAJOR COLLECTOR ROAD				
Design Elements		**Standard Criteria**	**Proposed Criteria**	**Primary Reference / Secondary Reference (If Applicable)**
Design Speed (mph)		Posted Speed	50 [1]	Section 5.3, VT State Standards
Traffic Volumes (ADT)		-	4300	Traffic Research Unit Memo (Carr to Sargent, 1/23/13)
Lane Width (ft)		9 - 11	11	Table 5.3, VT State Standards / Figure 8-3, VTrans Road Design Manual
Shoulder Width (ft)		3	4 [2] / 5 (bridge) [6]	Table 5.8, VT State Standards / Figure 8-3, VTrans Road Design Manual
Cross Slope	Travel Lane	2%	2%	Section 5.12, VT State Standards
	Shoulder	6%	6%	Figure 8-3, VTrans Road Design Manual (No VT State Standards Reference)
Maximum Superelevation Rate		8%	8% [3]	Section 5.13, VT State Standards
Minimum Radius (ft)		758	1400 (50 mph) / 760 (50 mph) [5]	Table 3-7, AASHTO [4]
Maximum Grade		7%	8% [1]	Table 5.6, VT State Standards
Sight Distance	Stopping (ft)	400	275 [1]	Table 5.1, VT State Standards
	Corner (ft)	550	440 [1]	Table 5.2, VT State Standards
Clear Zone	Fill Slope (ft)	20	20	Table 5.5, VT State Standards
	Cut Slope (ft)	14	14	Table 5.5, VT State Standards
Horizontal Clearance (ft)		10 (min) to 30 (max)	20	Figure 8-3, VTrans Road Design Manual
Bridge Width (ft)		Match Roadway	32 [6]	Section 5.7, VT State Standards
Structural Capacity		AASHTO HL-93 LRFD	AASHTO HL-93 LRFD	Section 3.4.1 VTrans Structures Design Manual

Notes:

1 Posted speed on Vermont Route 12 is 50 mph. Per Section 5.3 of the VT State Standards, a design speed up to 10 mph lower than the legal (posted) speed may be used without formal design exception, provided appropriate warnings are posted.
2 Add one foot on bridges per note "a" under Table 5.8 of the VT State Standards.
3 Per Section 5.13 of the VT State Standards, superelevation should be limited to 6% where a side road intersects the outside of a main road curve
4 Full reference: *A Policy on Geometric Design of Highways and Streets* , AASHTO, 2011.
5 Radius values correspond to superelevation of 8%.
6 Shoulder widths of 5ft on the proposed bridge have been requested by VTrans due to "Complete Streets" considerations over the design life of this project.

BETHEL BHF-0241(38) - VERMONT ROUTE 12 OVER GILEAD BROOK

SPRING HOLLOW ROAD

GEOMETRIC DESIGN CRITERIA - LOCAL ROADS				
Design Elements		**Standard Criteria**	**Proposed Criteria**	**Primary Reference / Secondary Reference (If Applicable)**
Design Speed (mph)		Posted Speed	25 [1]	Section 6.2, VT State Standards
Traffic Volumes (ADT)		-	-	
Lane Width (ft)		7-10	10	Table 6.3, VT State Standards / Figure 8-4, VTrans Road Design Manual
Shoulder Width (ft)		0-1	0	Table 6.3, VT State Standards / Figure 8-4, VTrans Road Design Manual
Cross Slope	Travel Lane	2%	2%	Section 6.11, VT State Standards
	Shoulder	2%	2%	Figure 8-4, VTrans Road Design Manual
Maximum Superelevation Rate		8%	8% [2]	Section 6.12, VT State Standards
Minimum Radius (ft)		134	110 (20 mph) [4]	Table 3-7, AASHTO [3]
Maximum Grade		11%	11%	Table 6.6, VT State Standards
Sight Distance	Stopping (ft)	150	150	Table 6.1, VT State Standards
	Corner (ft)	275	275	Table 6.2, VT State Standards
Clear Zone	Fill Slope (ft)	7	7	Table 6.5, VT State Standards
	Cut Slope (ft)	7	7	Table 6.5, VT State Standards
Horizontal Clearance (ft)		5 (min) to 10 (max)	7	Figure 8-4, VTrans Road Design Manual

Notes:

1 Per Section 6.2 of the VT State Standards, a design speed up to 10 mph lower than the legal (posted) speed may be used without formal design exception, provided appropriate warnings are posted.

2 Superelevation should not exceed 8% on paved roads and limited to 6% on unpaved roads.

3 Full reference: *A Policy on Geometric Design of Highways and Streets* , AASHTO, 2011.

4 Radius values correspond to superelevation of 8%.

Construction Cost Estimate

Appendix E

CLOUGH HARBOUR & ASSOCIATES LLP

			PROJECT	PHASE	ORG
COMPLETED BY:	RDH		23825	2000	29000
CHECKED BY:	DPC / JMF	SHEET #:	1	OF	1
PROJECT NAME:	VT RTE 12 BRIDGE 38 (BETHEL)	DATE:	5/15/2013		
PROJECT LOCATION:	BETHEL, VT	SUBJECT:	SUMMARY (BRIDGE & HWY)		

ITEM NO.	DESCRIPTION	Quantity	Unit	Unit Cost	Total Cost
204.25	STRUCTURE EXCAVATION	220	CY	$13	$2,860
204.30	GRANULAR BACKFILL FOR STRUCTURES	80	CY	$35	$2,800
208.30	COFFERDAM EXCAVATION, EARTH	245	CY	$15	$3,675
208.35	COFFERDAM EXCAVATION, ROCK	50	CY	$91	$4,550
208.40	COFFERDAM	2	LS	$20,000	$40,000
501.33	CONCRETE, HIGH PERFORMANCE CLASS A	375	CY	$775	$290,625
501.34	CONCRETE, HIGH PERFORMANCE CLASS B	390	CY	$630	$245,700
504.10	FURNISHING EQUPMENT FOR DRIVING PILING	1	LS	$50,000	$50,000
505.XX	STEEL PILING, HP X	775	LF	$80	$62,000
505.45	DYNAMIC PILE LOADING TEST	3	EA	$3,800	$11,400
506.55	STRUCTURAL STEEL, PLATE GIRDER	423634	LB	$2	$804,905
507.11	REINFORCING STEEL, LEVEL I	39500	LB	$1	$39,500
507.13	REINFORCING STEEL, LEVEL III	86900	LB	$3	$260,700
508.15	SHEAR CONNECTORS	5460	LS	$3	$16,380
509.10	LONGITUDINAL DECK GROOVING	1185	SY	$5	$5,333
525.10	REMOVAL OF EXISTING BRIDGE RAILING	650	LF	$9	$5,850
525.33	BRIDGE RAILING, GALVANIZED 2 RAIL BOX BEAM	690	LF	$100	$69,000
528.11	TWO-WAY TEMPORARY BRIDGE	1	LS	$1,300,000	$1,300,000
529.10	REMOVAL OF BRIDGE PAVEMENT	865	SY	$12	$10,380
529.15	REMOVAL OF STRUCTURE	1	LS	$500,000	$500,000
529.25	REMOVAL OF CONCRETE OR MASONRY	900	CY	$135	$121,500
531.15	BEARING DEVICE ASSEMBLY, HIGH LOAD MULTI-ROTATIONAL	8	EA	$3,700	$29,600
613.XX	STONE FILL, TYPE III	520	CY	$45	$23,400
621.72	GUARDRAIL APPROACH SECTION, GALVANIZED 2 RAIL BOX BEAM	4	EA	$2,700	$10,800
649.31	GEOTEXTILE UNDER STONE FILL	780	SY	$3	$2,340
653.35	VEHICLE TRACKING PAD	1485	CY	$40	$59,400
				TOTAL STRUCTURES COST =	$3,980,000
201.10	CLEARING AND GRUBBING, INC. INDIVIDUAL TREES AND STUMPS	1	LS	$ 30,000	$ 30,000
203.17	UNCLASSIFIED EXCAVATION	3000	CY	$ 16	$ 48,000
203.30	EARTH BORROW	18500	CY	$ 20	$ 370,000
203.31	SAND BORROW	1700	CY	$ 25	$ 42,500
210.10	COLD PLANING, BITUMINOUS CONCRETE PAVEMENT	7250	SY	$ 7	$ 50,750
301.15	SUBBASE OF GRAVEL	3000	CY	$ 20	$ 60,000
402.12	AGGREGATE SHOULDERS	100	TON	$ 40	$ 4,000
490.30	SUPERPAVE BITUMINOUS CONCRETE PAVEMENT	2800	TON	$ 135	$ 378,000
601.2615	18" CPEP(SL)	200	LF	$ 42	$ 8,400
604.18	PRECAST REINFORCED CONCRETE DROP INLET WITH CAST IRON GR	5	EACH	$ 2,850	$ 14,250
613.10	STONE FILL, TYPE I	2100	CY	$ 27	$ 56,700
621.205	STEEL BEAM GUARDRAIL, GALVANIZED W/8 FEET POSTS	1000	LF	$ 15	$ 15,000
621.60	ANCHOR FOR STEEL BEAM RAIL	4	EACH	$ 645	$ 2,580
621.90	TEMPORARY TRAFFIC BARRIER	1700	LF	$ 15	$ 25,500
630.10	UNIFORMED TRAFFIC OFFICER	650	HR	$ 60	$ 39,000
630.15	FLAGGERS	950	HR	$ 21	$ 19,950
631.10	FIELD OFFICE, ENGINEERS	1	LS	$ 20,000	$ 20,000
641.10	TRAFFIC CONTROL	1	LS	$ 50,000	$ 50,000
649.31	GEOTEXTILE UNDER STONE FILL	6500	SY	$ 2	$ 13,000
651.15	SEED	50	LB	$ 8	$ 400
651.18	FERTILIZER	400	LB	$ 4	$ 1,600
651.2	AGRICULTURAL LIMESTONE	1	TON	$ 560	$ 560
651.25	HAY MULCH	1	TON	$ 815	$ 815
651.35	TOPSOIL	400	CY	$ 40	$ 16,000
651.4	GRUBBING MATERIAL	6000	SY	$ 6	$ 36,000
652.10	EPSC PLAN	1	LS	$ 5,000	$ 5,000
652.20	MONITORING EPSC PLAN	40	HR	$ 43	$ 1,720
652.30	MAINTENANCE OF EPSC PLAN (N.A.B.I.)	1	LU	$ 5,000	$ 5,000
653.20	TEMPORARY EROSION MATTING	6000	SY	$ 2	$ 12,000
	TOTAL ROADWAY, TEMPORARY AND TRAFFIC & SAFETY COST =				$1,330,000
				SUBTOTAL =	$5,310,000
635.11	MOBILIZATION AND DEMOBILIZATION (7% OF SUBTOTAL)				$380,000
				15% CONTINGENCY =	$860,000
		TOTAL PROJECT COST =		$6,550,000	

STRUCTURES =	$ 2,620,000
ROADWAY =	$ 1,170,000
TEMPORARY =	$ 1,390,000
TRAFFIC & SAFETY =	$ 140,000

Appendix F

Preliminary Foundations Recommendations for Alignment and Structure Study Report

Interoffice Memorandum

To: Ryan Henderson

From: Jennifer MacGregor & Katy Adnams, P.E.

Copy Scott Doehla & Dale Gozalkowski

Date: March 4, 2013

Re: **Preliminary Foundations Recommendations for Alignment and Structure Study Report**
Proposed Bridge Replacement – Bethel BRF 0241(38)
Bethel, VT
CHA Project No. 23825.2000.32000

PROJECT UNDERSTANDING

The existing two-lane, four (4) span bridge on Vermont Route 12 crossing the Gilead Brook is scheduled for replacement. Based upon recent site survey and observations during a site visit on January 19, 2013, there are bedrock outcrops on the southern bank. Per the 1928 record drawings, the existing southern abutment and pier are supported on spread footings bearing on rock at about El. 650 and El. 647, respectively. The record plans further indicate that the center pier, northern pier, and northern abutment are supported on timber piles with estimated length of 25 feet. The drawings do not indicate if the piles were driven to rock or bear within the natural soil deposits.

The replacement bridge will be approximately on the same horizontal alignment as the existing structure. The Alignment and Structure Study is considering two (2) and three (3) span alternatives (see Appendix C of the Alignment and Structure Study Report for preliminary plans the preferred alternative). We understand that integral abutments are the preferable design for this project provided there is sufficient depth to bedrock for the piles to achieve fixity. The following table summarizes the preliminary estimated service loads on the bridge abutments and piers.

Location	Two-span alternative		Three-span alternative	
	dead load	live load	dead load	live load
Beginning Abutment	686.5 kip	460.8 kips	451.7 kip	376.0 kip
Pier(s)	2751.0 kips	986.2 kips	1672.5 kips 1673.3 kips	738.2 kips 738.8 kips
Ending Abutment	686.5 kips	461.0 kips	451.3 kips	376.0 kips

The following sections present preliminary recommendations for foundation types based for use in the Alignment and Structure Study Report. Geotechnical borings at the substructure locations and corresponding geotechnical analysis will be required for final design. A proposed geotechnical scope of work has been submitted to VTrans.

REGIONAL GEOLOGY

CHA reviewed the following publications to assess the regional geologic conditions:

- Doll, C.G., W. M. Cady, J. B. Thompson, and M. P. Billings (1970) "Surficial Geologic Map of Vermont." Vermont Geological Survey.
- Ratcliffe, N.M., R.S. Stanley, M.H. Gale, P.J. Thompson, and G.J. Walsh (2011) "Bedrock Geologic Map of Vermont," U.S. Geological Survey Scientific Investigations Map 3184.

According to the *Surficial Geologic Map of Vermont*, the bridge is likely located within horizontally bedded deposits of gravel. Glacial till deposits are noted to the north and south. It is anticipated that the natural soils above bedrock may contain numerous cobbles and boulders due to the observed cobbles on the river banks and notes on the 1928 record drawings.

According to the *Bedrock Geologic Map of Vermont*, conglomerate and conglomerate quartzite bedrock cross the site from northwest to southeast and is anticipated to be the predominate rock type at the permanent bridge. Quartz-muscovite phyllite and silicic phyllite, and garnet-rich biotite-muscovite-quartz schist are located immediately northeast of the site and may be the predominate rock type below the temporary bridge. Bedrock is exposed at the toe of the southern slope.

FOUNDATION ALTERNATIVES

Based upon the anticipated subsurface conditions, we anticipate that the bridge foundations will be supported on a combination of spread footings and deep foundations. Suitable deep foundations systems include driven H-piles, drilled minipiles, or drilled shafts. Driven H-piles are the preferred alternative for integral abutments. We have included the other options for the alternatives analysis.

DRIVEN H-PILES

Driven piles will most likely be driven to bedrock to maximize the available structural capacity of the steel pile and thereby reduce the number of piles required. The capacity of driven piles bearing on bedrock will be controlled by the structural pile strength. Since the piles will bear on bedrock, settlement under the applied foundation loads is anticipated to be on the order of magnitude of the steel compression under the applied axial load. The H-Pile section should be selected from Table 4.5.1.5-1 in the VTrans Integral Abutment Bridge Design Guidelines. Examples are provided in the following table.

Pile Section	Nominal Structural Pile Resistance (kips)	Strength Limit State Capacity, $\varphi = 0.65$ (kips)
HP10x57	840	545
HP12x74	1090	708
HP14x102	1500	975

The resistance factor applied in this table is based upon confirming the driving criteria through dynamic pile load testing of at least 2 percent of the production piles at each structure.

Piles shall be spaced no closer than 5.8 feet to allow the piles to be designed as a single pile. Closer spacing may require the group pile design.

Installation of driven piles may be complicated by the presence of cobbles and boulders in the embankment fill and natural soil. The presence of these challenging conditions will be confirmed during the geotechnical exploration program.

SPREAD FOOTINGS ON ROCK

Spread footings are likely to be the appropriate foundation type for the southern pier in the 3-span configuration based on the presence of bedrock outcrops. Indications from drawings for the existing bridge suggest that deep foundations will be required at the other substructure locations. Based upon the rock types described in the regional geology, we recommend a bearing resistance at the service limit state of 10 tons per square foot (tsf) for preliminary design. For strength limit state, we anticipate that the strength of the concrete will control the design of footings bearing on bedrock.

DRILLED SHAFTS

Drilled shafts bearing in bedrock can develop very high capacity per element due to the large area of the base. However, drilled shafts can be difficult to advance past obstructions such as cobbles and boulders. Additionally, casing and slurry will be required due to high groundwater conditions and anticipated sandy soil. The greater lateral stiffness of drilled shafts compared to driven piles may be advantageous for the piers within Gilead Brook if scour depths are large.

The end bearing capacity is highly dependent on the joint pattern in the rock. For preliminary design, we recommend a nominal end bearing capacity of 50 tsf. Side friction resistance through the overburden soils should be ignored. The following table provides estimated end bearing capacities for use in preliminary design.

The center-to-center spacing of drilled shafts shall be no closer than 4 times the shaft diameter. Closer spacing is possible with a reduced capacity to account for overlapping zones of influence, which can be accounted for in final design.

Diameter (feet)	**Service Limit State Capacity, φ = 1.0 (kips)**	**Strength Limit State Capacity, φ = 0.50 (kips)**
3	700	350
4	1250	625
5	1960	980

DRILLED MINIPILES

The new bridge foundation could also be supported on drilled minipiles developing their capacity through a rock socket. Minipiles can be drilled through cobbles and boulders. For preliminary design, we recommend a grout-to-ground capacity of 15 ksf in the bedrock. Side friction resistance through the overburden soils should be ignored. The following table provides examples of pile capacity.

Diameter (inches)	Rock Socket Length (ft)	Service Limit State Capacity, φ = 1.0 (kips)	Strength Limit State Capacity, φ = 0.55 (kips)
7	10	275	150
	15	410	225
8	10	310	170
	15	470	255

The center-to-center of minipiles shall be no less than 5 feet or 3 times the pile diameter, whichever is greater.

FOUNDATION RECOMMENDATIONS

ABUTMENTS

The VTrans Integral Abutment Bridge Design Guidelines indicate that a minimum 16 foot pile embedment is required. There is about a 50 foot difference in elevation between the existing bridge deck and river bottom. An approximately 20-foot-high embankment was constructed for the southern abutment and an approximately 35-foot-high embankment was constructed for the northern abutment. Therefore, we anticipate that the minimum embedment criteria can be met.

We recommend supporting the abutments on H-piles driven to bedrock. The pile section should be selected from Table 4.5.1.5-1 of the VTrans Integral Abutment Bridge Design Guidelines based upon the load capacity required.

SOUTHERN PIER

Bedrock is exposed at the toe of the southern slope in the vicinity of the southern pier location for the three span alternative (about Sta. 285+16.5). Therefore, it is anticipated that this pier can be supported on a spread footing bearing on rock.

CENTER PIER AND NORTHERN PIER

The existing center pier and northern pier are reportedly supported on driven timber piles. Therefore, it is recommended that the foundation for the center pier for the two span alternative (at about Sta. 268+00) or northern pier for the three span alternative (at about Sta. 268+58) be supported on H-piles driven to bedrock. However, if scour depths are problematic for allowable unsupported lengths of slender driven piles, drilled shafts may be a preferable alternative.

CLOSING

The recommendations contained herein are intended for preliminary foundation design prepared as part of the Alignment and Structure Study Report. A geotechnical exploration program and engineering analysis will be required for final design.

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Appendix G

Hydrologic and Hydraulic Analysis

Hydrologic and Hydraulic Analysis

for

Bethel BHF 0241(38)

Vermont Route 12 over the Gilead Brook

Windsor County, Vermont

Prepared for:

Vermont Agency of Transportation

March 2013

CHA Project No. 23825

Prepared by:

CHA

III Winners Circle

Albany, New York 12205

(518) 453-4500

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TABLES

1. General

Bridge 00038 carries Vermont Route 12 over Gilead Brook in the Town of Bethel, Vermont. The bridge consists of a four span steel truss structure with a total of three piers. Two of the three piers are located out of the main channel, while Pier 2 (middle) is in the center of Gilead Brook. A preliminary review of the Flood Insurance Study (FIS) for Windsor County (September 2007), indicates that there is no information available for Gilead Brook. As such, CHA developed a hydraulic model to evaluate the existing bridge and to determine the allowable open area for a replacement structure in order to meet Vermont Agency of Transportation (VTrans) and Federal Emergency Management Agency (FEMA) design guidelines. CHA utilized existing data as available to facilitate the development of the hydraulic model; however updated survey was required due to the significant changes in channel geometry which occurred as a result of Tropical Storm Irene.

2. Hydrologic Assessment

As recommended in the VTrans Hydraulic Manual, a minimum of three hydrologic methods were utilized to develop design flows for Gilead Brook. A brief description of each is provided in the bulleted items below.

- USGS StreamStats Regression

  Discharges were developed from the StreamStats online application. CHA verified the StreamStats watershed delineation was consistent with the USGS quadrangle map. The discharges are based on the USGS Water-Resources Investigations Report, Flow-frequency characteristics of Vermont Streams, 2002.

- Gage Comparison (Area Relationship)

  Discharges were developed from the historic records of USGS Gage 01142500 on Ayers Brook in Randolph, VT. The Randolph gage was selected based on its proximity, similarities in watershed characteristics and extensive period of record (73 years). The respective event flows for the USGS Gage were developed by a HEC-SSP (Version 2.0) Bulletin 17B Analysis and were converted to a unit flow per square-mile of contributing watershed. These values were then transferred to Bridge 00038 using an area relationship referenced from the VTrans Hydraulic Manual.

- 1974 Regression Equations

  Discharges were developed using the methodologies detailed in the publication *Progress Report on Flood Magnitude and Frequency of Vermont Streams Regression Equations*.

Event discharges were developed for each hydrologic method and compared for the 2-, 10-, 50-, 100- and 500-year (yr) events. The 500-yr event discharge was extrapolated from known data where necessary. The resulting discharges for each method are provided in Table 1.

Table 1 - Design Flows

Hydrologic Method	2-yr	10-yr	50-yr	100-yr	500-yr
StreamStats Regression	457	861	1,130	1,340	2,200
Gage Comparison	**414**	**870**	**1,560**	**1,974**	**3,351**
1974 Regression	375	718	1,040	1,212	1,590

CHA utilized the flows provided by the Gage Comparison hydrologic method, which provided the most conservative discharge and resulting water surface elevation for the design flood events.

3. Hydraulic Analysis

3.1. Methodology

Water surface profiles were generated using the U.S. Army Corps of Engineers' River Analysis System Software (HEC-RAS, Version 4.1). This model was developed to compute the hydraulic parameters needed to analyze scour at the bridge, as well as to evaluate potential countermeasure designs for the proposed structure. The study reach begins 1,750 feet (ft) downstream of the Vermont Route 12 crossing and extends upstream along Gilead Brook for 2,200 ft, ending approximately 400 ft upstream of the existing bridge. Mixed flow scenarios were modeled for the 2-, 10-, 50-, 100- and 500-yr flood events and starting water surface elevations were based on normal depth in the downstream reach. In addition, a convergence test was performed to ensure the water surface elevation at Bridge 00038 was independent of the downstream boundary condition.

The geometry (cross sections and bridge geometry) required for the hydraulic model was developed from a combination of USGS Digital Elevation Model (DEM), field survey and record plans. The cross-section geometry was extracted from the USGS DEM using the Army Corps of

Engineers' HEC-GeoRAS extension for ArcView. Individual cross-sections were edited in HEC-RAS based on the data obtained from a site survey (October 2012). Manning's "n" values and the contraction and expansion coefficients for each cross-section were based on field conditions documented during an October 2012 site visit. All elevation data used in the modeling and presented in this document is referenced to the North American Vertical Datum of 1988 (NAVD88).

3.2. Ordinary High Water

Gilead Brook was observed to be a steep gradient stream that sustained significant damage from Tropical Storm Irene in August 2011. Due to the damage sustained the OHW elevation could not easily be identified during the October 2012 CHA field visit. As a result, CHA used the 2-yr discharge (in accordance with guidance provided in the VTrans Hydraulic Design Manual) to develop an estimate for the ordinary high water (OHW) elevation along the study reach of Gilead Brook.

3.3. Existing Hydraulics

Based on survey data, the existing bridge deck varies from approximately 700.5 ft at the left (northern) abutment to approximately 683.9 ft at the right (southern) abutment. The low chord was defined as the bottom of the deck truss in the main span and varies from an elevation of 681.1 ft at Pier 1 (northern) to 668.7 ft at Pier 3 (southern). In addition, flood flows are largely contained within the main channel throughout the bridge reach with Froude numbers routinely approaching one (critical depth). The downstream bridge that carries Spring Hollow Road over Gilead Brook was significantly damaged during Tropical Storm Irene, as a result CHA modeled the structure as though it had been replaced in kind, therefore addressing any possible tail water condition at the subject bridge. A summary of the hydraulic analysis is presented in Table 2.

Table 2 - Existing Condition Hydraulic Data

Design Parameters	Design Events			
	2-yr (OHW)	50-yr	100-yr	500-yr
Peak Discharge (ft^3/sec)	414	1,560	1,974	3,351
Water Surface Elevation[1] @ Approach Sect (ft)	640.3	641.7	642.0	642.9
Freeboard Provided (ft)	28.4	27.0	26.7	25.8

Average Velocity @ Structure (ft/sec)	5.4	7.0	7.4	8.2

[1]All elevations are referenced to NAVD88.

3.4. Design Criteria

In accordance with the VTrans Hydraulics Manual (1998), bridges located on principal arterials should, where practical, be designed to convey the 50-yr flood event with a minimum clearance of 1.0 ft between the water surface elevation and the low chord of the structure. In addition, consideration should be given to the potential effects of the 100-yr flood on adjacent properties, the environment, hazards to human life and floodplain management criteria. Given the results of the hydraulic analysis, the existing bridge exceeds the minimum freeboard requirements and does not create a significant constriction to the regulatory floodplain.

3.5. Proposed Hydraulics

The proposed two-span design includes a single pier which is to be placed near the middle of the channel. Similar to the existing configuration, the pier is not skewed to the bridge deck which results in an angle of attack for the pier of approximately 20 degrees. The bottom of the steel in the two-span design varies from approximately 695.4 ft at the left (northern) abutment to approximately 676.4 ft at the right (southern) abutment. The low chord is defined as 676.4 ft which is the lowest bottom of steel elevation. A summary of the hydraulic analysis for the two-span option is presented in Table 3.

Table 3 - Proposed Two-Span Hydraulic Data

Design Parameters	Design Events			
	2-yr (OHW)	50-yr	100-yr	500-yr
Peak Discharge (ft^3/sec)	414	1,560	1,974	3,351
Water Surface Elevation[1] @ Approach Sect (ft)	640.2	641.3	641.6	642.6
Freeboard Provided (ft)	36.2	35.1	34.8	33.8
Average Velocity @ Structure (ft/sec)	4.5	7.2	7.7	7.7

[1]All elevations are referenced to NAVD88.

The proposed three-span design includes two piers; Pier 1 (northern) will be located out of the channel while Pier 2 (southern) will be placed near the southern edge of the existing channel. Similar to the existing configuration, the piers are not skewed to the bridge deck which results in an angle of attack of 20 degrees. The bottom of the steel in the three-span design varies from approximately 695.4 ft at the left (northern) abutment to approximately 676.4 ft at the right (southern) abutment. The low chord is defined as 676.4 ft which is the lowest bottom of steel elevation. A summary of the hydraulic analysis for the three-span option is presented in Table 4.

Table 4 - Proposed Three-Span Hydraulic Data

Design Parameters	Design Events			
	2-yr (OHW)	50-yr	100-yr	500-yr
Peak Discharge (ft^3/sec)	414	1,560	1,974	3,351
Water Surface Elevation[1] @ Approach Sect (ft)	640.1	641.3	641.6	642.6
Freeboard Provided (ft)	36.3	35.1	34.8	33.8
Average Velocity @ Structure (ft/sec)	5.3	7.1	7.7	7.8

[1]All elevations are referenced to NAVD88.

The results of the hydraulic analyses indicate that either proposed structure exceeds the VTrans hydraulic design guidelines, providing over 35 ft of freeboard during the 50-yr flood event. However, given the topography of the surrounding area and the limitation on the vertical curvature of the approach roadways, the proposed structure is not expected to have a significant reduction in hydraulic opening. In fact, the proposed design maintains the current hydraulic opening, resulting in minimal change (decrease) in 50- and 100-yr water surface elevations through the bridge reach due to the proposed pier stem geometry.

4. Scour Analysis

CHA evaluated two potential designs to replace the existing structure that carries Vermont Route 12 over Gilead Brook. In order to support the cost benefit analysis associated with the required foundation design, scour depths were generated for each of the proposed alternatives.

4.1. Streambed Soils

The streambed material in the study reach of Gilead Brook was field classified as fine to coarse gravel and cobles with some boulders and little sand. Although there was evidence of shallow bedrock noted along the right (southern) edge of the channel near the bridge, the current pier is supported by a pile foundation. There was no boring data available on the record plans, and as such, for the purposes of the structure type study, CHA conservatively used a median diameter of 0.25 inches (fine gravel) to evaluate the potential scour depths. As the preferred alternative advances to final design, CHA will refine the soil characterization based on actual boring logs; however we do not anticipate any significant changes in the predicted scour depths.

4.2. Potential Scour

The total scour depth is generally assessed as the sum of any long-term trends of the streambed, the computed contraction scour depth and the computed local scour depths in accordance with guidelines set forth in the Federal Highway Administration Publication, “Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges, Fifth Edition, April 2012” (HEC-18).

Gilead Brook sustained significant damage as a result of Tropical Storm Irene in August of 2011. Field observations and recent survey indicate that the thalweg has a potential to migrate or shift. As such, although significant aggradation/degradation is not expected, it is imperative that all substructures are designed using the maximum expected velocity, given the potential for channel migration.

Contraction scour occurs when the flow area of a stream is reduced either by a natural constriction or by a bridge. A decrease in the flow area results in an increase in average velocity and bed shear stress throughout the contracted section. Whether or not a contracted section, such as a bridge, will experience scour is dependent on the competence of the flow to transport bed material into or out of the section. A review of the HEC-RAS scour analysis indicates that the flow in the approach section has the potential to transport the soils into the bridge opening during each of the events investigated. As a result, the live-bed scour equation was used to evaluate the potential contraction scour at the subject bridge. Based on field observations and the approach sections developed in the HEC-RAS model, the bridge does not create a significant constriction to flood flows. The contraction scour estimates for the proposed designs confirm this assessment, with no scour predicted for the 100- or 500-yr flood events.

Local scour at a pier is a function of the geometry of the pier and footing, the composition of the bed material and the flow characteristics at the upstream bridge fascia. The flow characteristics of interest for local pier scour are the velocity and flow depth immediately upstream of the structure, the alignment of the pier in relation to the approaching stream (angle of attack) and the potential for pressure flow. In the case of both proposed bridge designs, the piers are not skewed to the bridge deck, which results in an angle of attack of approximately 20 degrees. In addition, all of the substructures in or near the main channel will experience moderate to high velocities during the 100- and 500-yr storm events. Pressure flow conditions are not expected.

The fifth edition of HEC-18 has removed the armoring coefficient (K_4) from the CSU equation and replaced it with a separate analysis developed by the Federal Highway Administration (FHWA) to predict pier scour in coarse bed materials. Given the limited information on the underlying soils, CHA did not utilize FHWA methodology, opting for the more conservative CSU equation in order to evaluate pier scour for each replacement alternative. Finally, since the HEC-RAS model calculated an initial scour depth exposing the proposed pile caps, the HEC-18 Case 2 complex pier scour equations were utilized at each substructure. For the purpose of this analysis, the HEC-18 Case 2 complex pier scour equations were applied using the maximum potential velocity and the corresponding water depth at each pier.

The magnitude of local abutment scour is a function of the geometry of the bridge opening, the alignment of the abutment, the composition of the streambed material and the flow characteristics of the approach section. The proposed designs include abutments that are well elevated and set back from the main channel, in order to maintain the existing road profile. The results of the hydraulic modeling indicate there is limited overbank flow returning at the toe of either of the embankment slopes. Although the embankments for the existing structure sustained minor damage during Tropical Storm Irene, they are currently protected with large rip-rap, which is anticipated to remain in place through the construction of the proposed bridge. In addition, in both of the proposed alternatives, the abutments are not impacted during the 100- or 500-yr flood events. As such, local abutment scour is not expected for either of the proposed designs.

4.3. Proposed Two-Span

The two-span design was assessed for total scour during the 100- and 500-yr flood events. As noted previously, the complex pier scour equations were utilized to obtain an expected scour depth at the pier. According to the flood history of the studied reach, the thalweg was noted to be unstable and has migrated during previous events. Given the proximity of the proposed pier to the centerline of the current channel, CHA utilized the maximum velocity and associated water depth to evaluate the potential pier scour for each flood event. A summary of the total potential scour at each of the substructures is presented in Table 5 on the following page.

Table 5 - Proposed Two-Span Predicted Scour Depths

Flood Event	Predicted Depths (ft)					
	Contraction Scour	Abutment Scour			Pier Scour	
		Left	Right	Max[1]	Pier 1	Max[1]
100-yr	0.0	0.0	0.0	0.0	17.8	17.8
500-yr	0.0	0.0	0.0	0.0	19.8	19.8

[1] Maximum scour depths represent a summation of the contraction and local scour components at an abutment or pier.

The scour depths calculated above assume that the proposed pier will not be aligned with the approaching flow, resulting in an angle of attack of approximately 20 degrees. If the pier is aligned with the approaching flow, it is anticipated that the predicted scour depths would be reduced by approximately 60 percent.

4.4. Proposed Three-Span

The three-span design was assessed for total scour during the 100- and 500-yr flood events. As noted previously, the complex pier scour equations were utilized to obtain an expected scour depth at the piers. According to the flood history of the studied reach, the thalweg was noted to be unstable and has migrated during previous events. Pier 1 (northern) will be located on the left embankment and set back from the channel. Although the 500-yr water surface elevation does not currently impact Pier 1 (northern), given the instability of the channel it was assessed for scour using the velocity and water depth associated with a potential full bank width channel migration. Given the proximity of the proposed Pier 2 (southern) to the thalweg of the current channel, CHA utilized the maximum velocity and corresponding water depth to evaluate the potential pier scour for each flood event. A summary of the total expected scour is presented in Table 6 on the following page.

Table 6 - Proposed Three-Span Predicted Scour Depths

Flood Event	Predicted Depths (ft)						
	Contraction Scour	Abutment Scour			Pier Scour		
		Left	Right	Max[1]	Pier 1	Pier 2	Max[1]
100-yr	0.0	0.0	0.0	0.0	7.1	26.6	26.6
500-yr	0.0	0.0	0.0	0.0	7.9	30.3	30.3

[1] Maximum scour depths represent a summation of the contraction and local scour components at an abutment or pier.

The scour depths calculated above assume that the proposed piers will not be aligned with the approaching flow, resulting in an angle of attack of approximately 20 degrees. If aligned with the approaching flow, it is anticipated that predicted scour depths at Pier 2 would be reduced by approximately 65 percent.

5. Conclusion

In accordance with the VTrans Hydraulics Manual (1998), bridges located on principal arterials should, where practical, be designed to convey the 50-yr flood event with a minimum clearance of 1.0 ft between the water surface elevation and the low chord of the structure. Both of the proposed bridge alternatives satisfy this criterion, providing over 35 ft of freeboard during the design flood event. Although there are substantial differences in the potential scour depths associated with each of the alternatives, all of the piers will ultimately require a pile foundation. As such, CHA will weigh the potential savings in foundation costs versus the increased cost of skewing the structure as the bridge advances to final design.

Appendix H

Selected Existing Bridge Plans

FED. ROAD DIST. NO. 9
STATE VT.
FR. 12 H.
FISCAL YEAR 1928
SHEET NO. 9
TOTAL SHEETS 39
GILEAD BROOK
RANDOLPH
TO BETHEL
TO FARM
O.R. QUIMBY
O.R. QUIMBY
GEORGE POWERS
GEORGE POWERS
TEMPORARY BRIDGE
TEMPORARY DRIVE
POSSIBLE LEDGE
LEDGE
Sta. 5+77.67 ℄ Bearing
Sta. 6+17.67 ℄ Pier
Sta. 7+38.83 ℄ Pier
Sta. 8+60 ℄ Pier
Sta. 9+60 ℄ Bearing
6+00
7+00
8+00
9+00
10+00
11+00
B.M. #2 BLACK CHERRY
SUMMARY OF QUANTITIES
* STRUCTURE EXCAVATION 858 ~~711~~ CU. YDS.
CLASS "A" CONCRETE (FOR BEAMS & SLABS) 229 " "
* CLASS "A" CONCRETE (FOR ABUTMENTS & PIERS) 235 ~~791~~ " "
* CLASS "B" CONCRETE (FOR ABUTMENTS & PIERS) 433 ~~399~~ " "
* REINFORCING STEEL 53124 ~~52750~~ lbs.
STEEL SUPER STRUCTURE (355,800 lbs.) 1-LUMP SUM.
* TIMBER PILING 2100 lin. ft.
* Quantities changed Oct. 22, 1928.
5+0 100' V.C. 404.27
10+50 300' V.C. 435.96
-1.42%
+3.36%
Exp
Fixed
Elev. 430
420
410
400
Elev. of Bottom of Footing 411.18
Abutm. #1
Elev. of Bottom of Footing 400.71'
Pier #1
Foundation: Gravel & Boulders
Elev. of Bottom of Footing 391.98'
Pier #2
El. 408.7' Flood Water Nov. 3 1927
El. 404.0' High Water previous to Nov. 3 1927
L. W. El. 400.0'
Ledge
Pier #3
Elev. of Bottom of Footing 407.0'
Abutm. #2
Elev. of Bottom of Footing 409.59'
CORRECT: A. D. Bishop BRIDGE ENGINEER
DISTRICT No 5
BRIDGE No 24
ROAD No 29
BETHEL VT
WOOD 11-12-27
DUNN 1-21-28
432.8
424.4
423.3
416.4
414.5
408.6
409.3
412.6
412.9
413.4
413.3
402.3
400.0
398.7
399.8
399.2
400.4
401.0
405.0
417.0
419.7
421.2
426.45
430.75
432.65
437.55
440.15
441.75
5+00
+50
+78
6+00
+50
7+00
+10
+40
+50
+60
+70
+80
+90
8+00
+10
+20
+27
+30
+40
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+50
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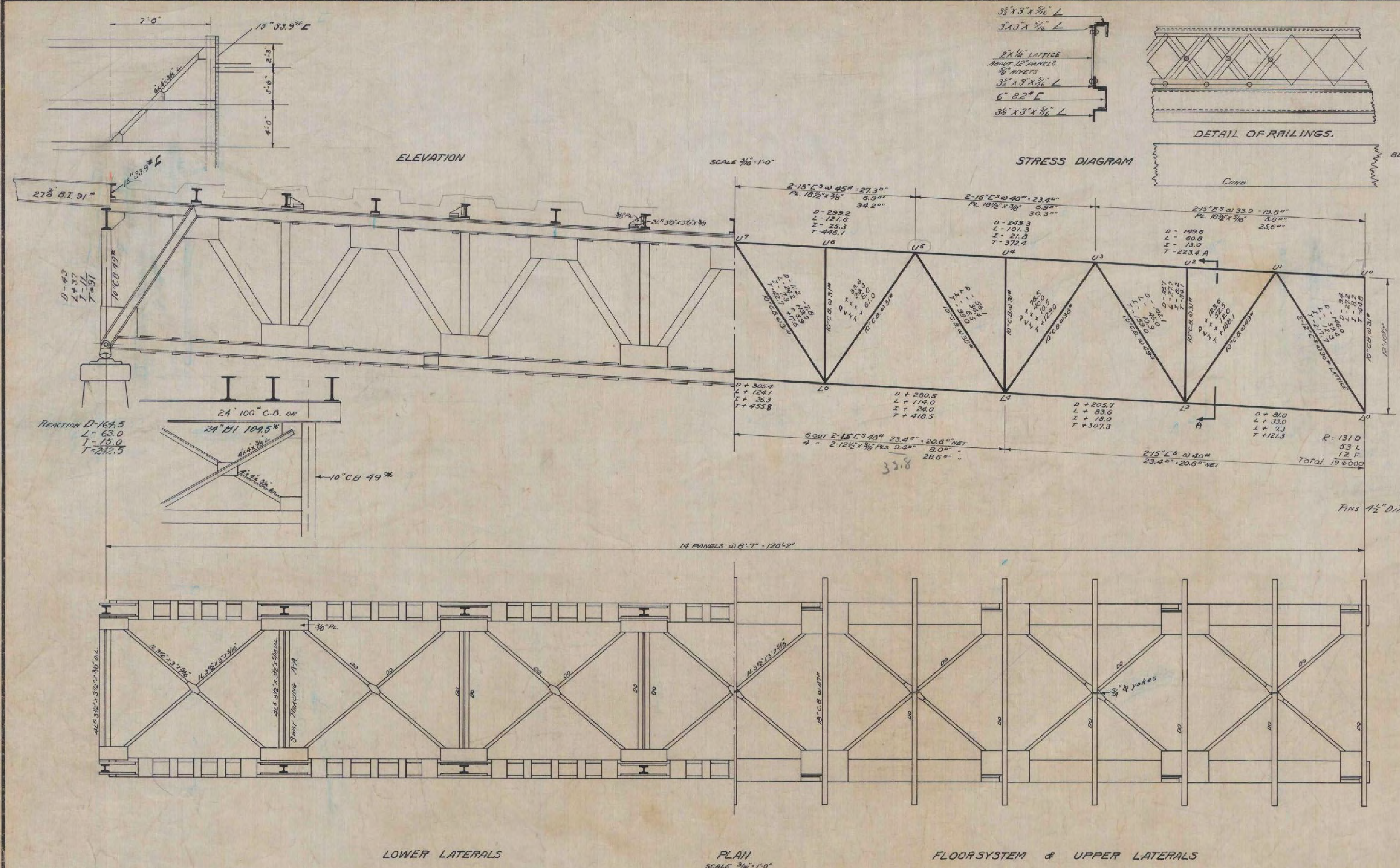
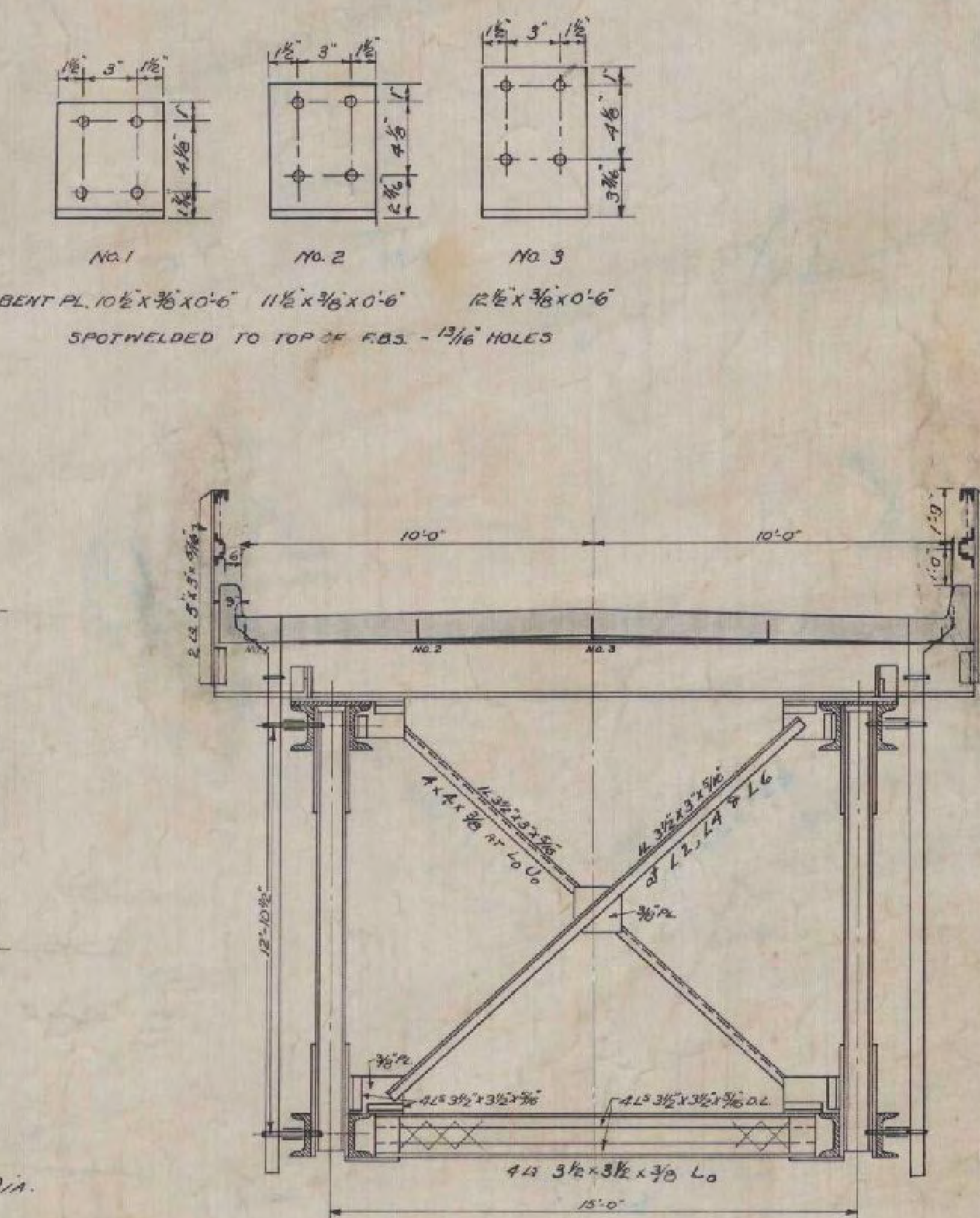
+50
11+00
Series F.R. No. 12 H.
Sheet 9 of 39





CROSS SECTION "A-A"

SCALE 1/4" = 1'-0"

LOADING DATA — FLOORBEAMS DESIGNED FOR,
Dead load 8½" slab plus 25 lbs per sq. ft. paving.
Live load 2-15 Ton trucks - 19' wheelbase.
Impact 30%.

TRUSSES DESIGNED FOR,
Dead load concrete 3200 lbs per ft. of bridge.

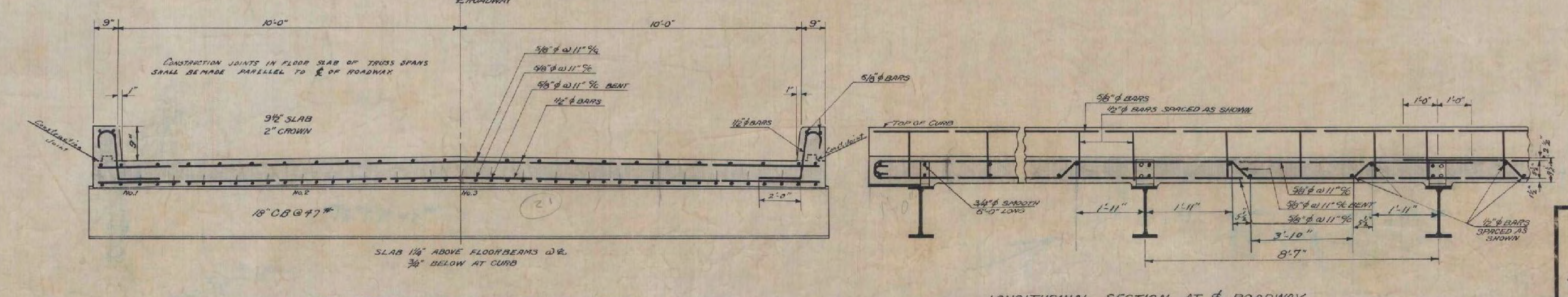
Steele	1150	" " " "
Total	4350	" " " "

Liveload 2H15 AASHO 1926

Impact $\frac{36}{N+18} \times \frac{L \times 250}{10L \times 500}$

Gussets 7/16" Rivets 7/8"

For Standard Details see sheet SB1-55 & 56 without saddles, S10, S11, S12, S13, S15, S21 for arrangement of rods except as otherwise shown.
All checkered apron plates to be 19'-9" long.



CROSS SECTION OF SLAB SHOWING STEEL

SCALE 1/2" = 1'-0"

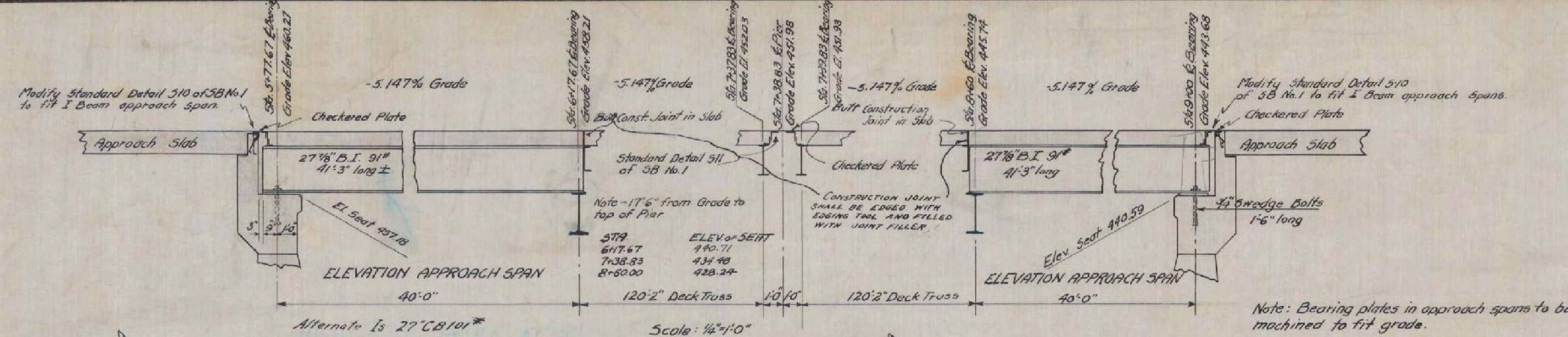
LONGITUDINAL SECTION AT ℄ ROADWAY

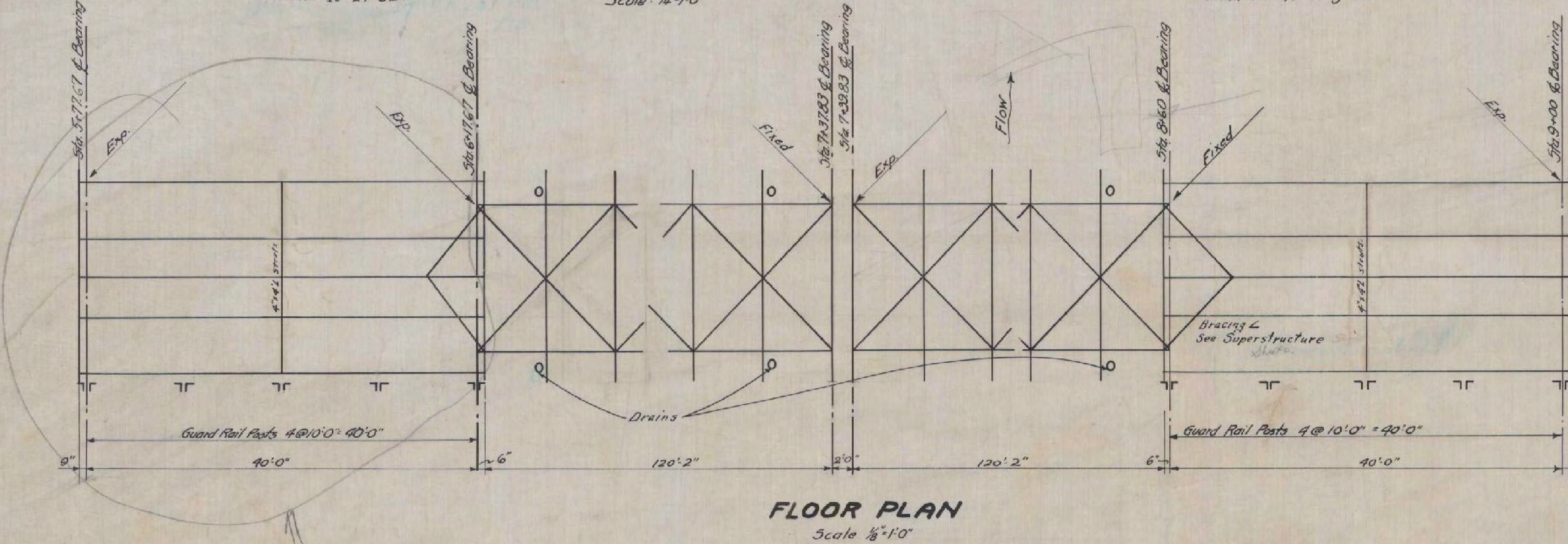
SCALE 1/2" = 1'-0"

SUPERSTRUCTURE DETAILS
BRIDGE OVER GILEAD BROOK AT CHURCH HILL-BETHEL, VT.

ESTIMATED QUANTITIES	
2 TRUSS SPANS	
STRUCTURAL STEEL	307000 LBS.
CLASS A (1-2-4) CONCRETE for Beams & Slabs	158 CU.YDS.
REINFORCING STEEL	27000 LBS.

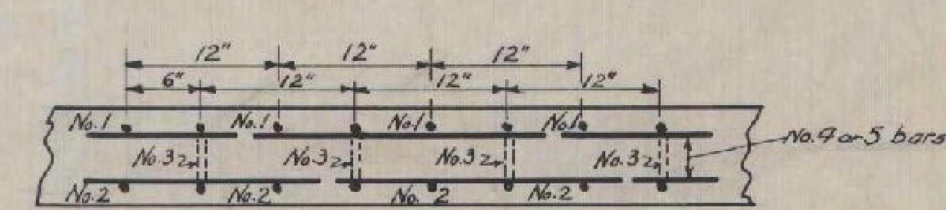
Surveyed by
Designed by
Drawn by F.C. Larsen Jr.
Traced by F.C. Larsen Jr.
Checked by W.R.M. 10/6 28
Series FR No. 1211 Filed
Sheet 10 of 34 Sheets





FLOOR PLAN

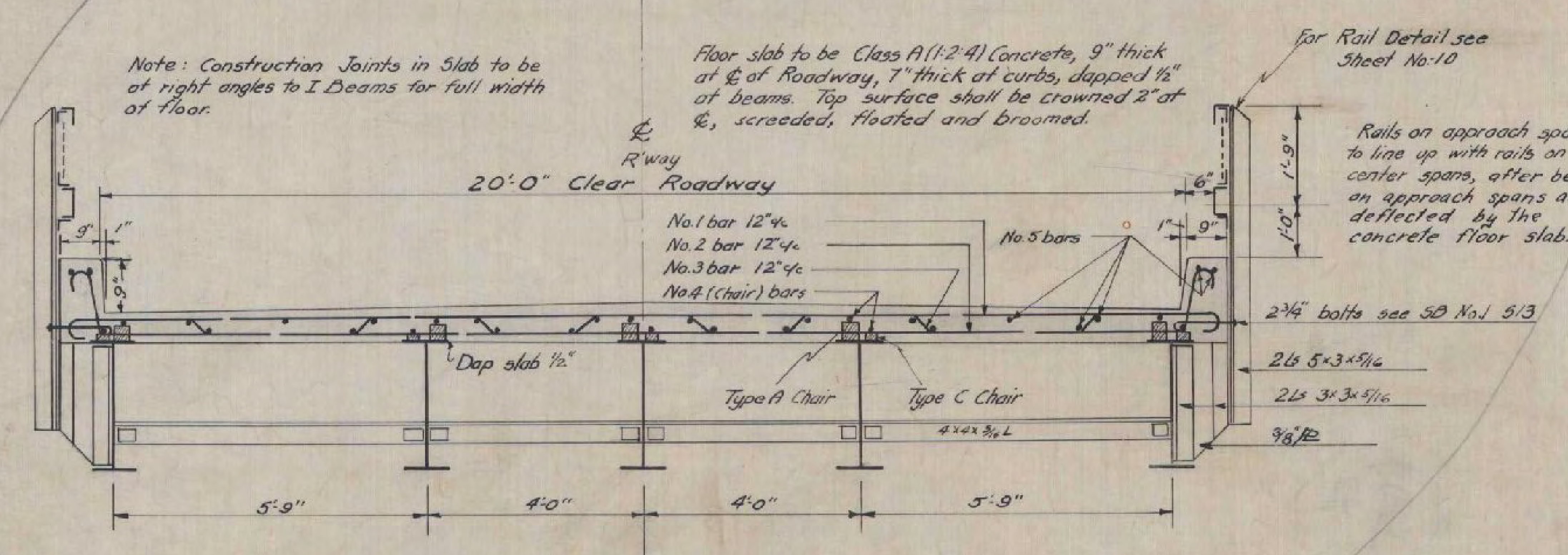
Scale 1/16" = 1'-0"



PART SECTION SHOWING REINFORCING STEEL IN APPROACH SPAN SLAB.

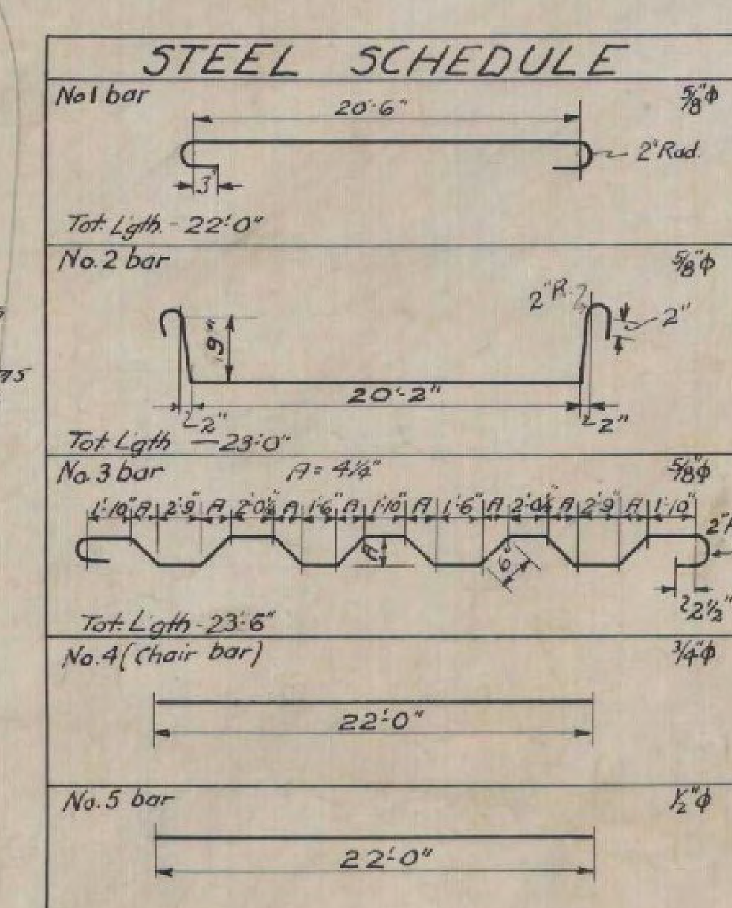
Note: Construction Joints in Slab to be at right angles to I Beams for full width of floor.

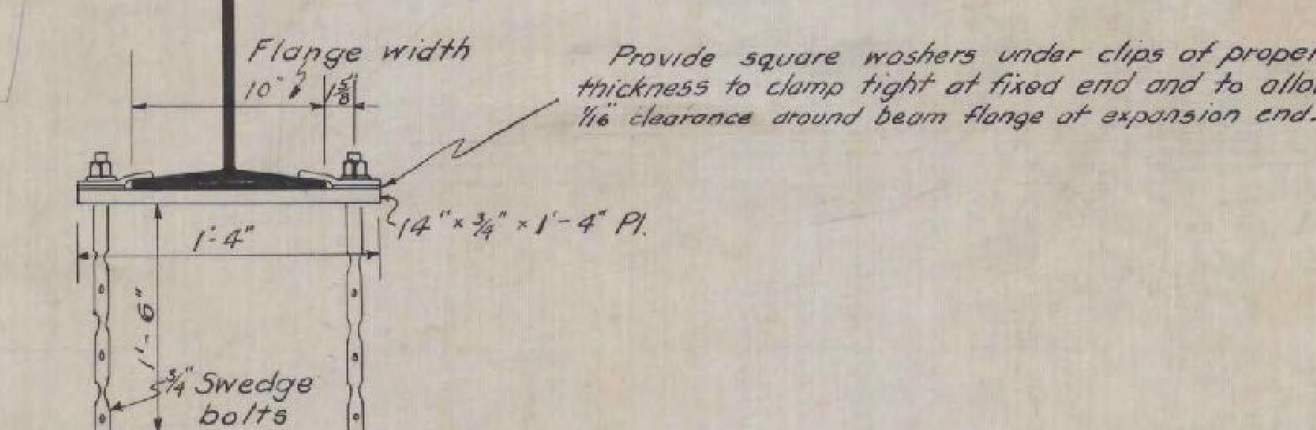
Floor slab to be Class A (1:2:4) Concrete, 9" thick at ⊈ of Roadway, 7" thick at curbs, dapped ½" at beams. Top surface shall be crowned 2" at ⊈, screeded, floated and broomed.



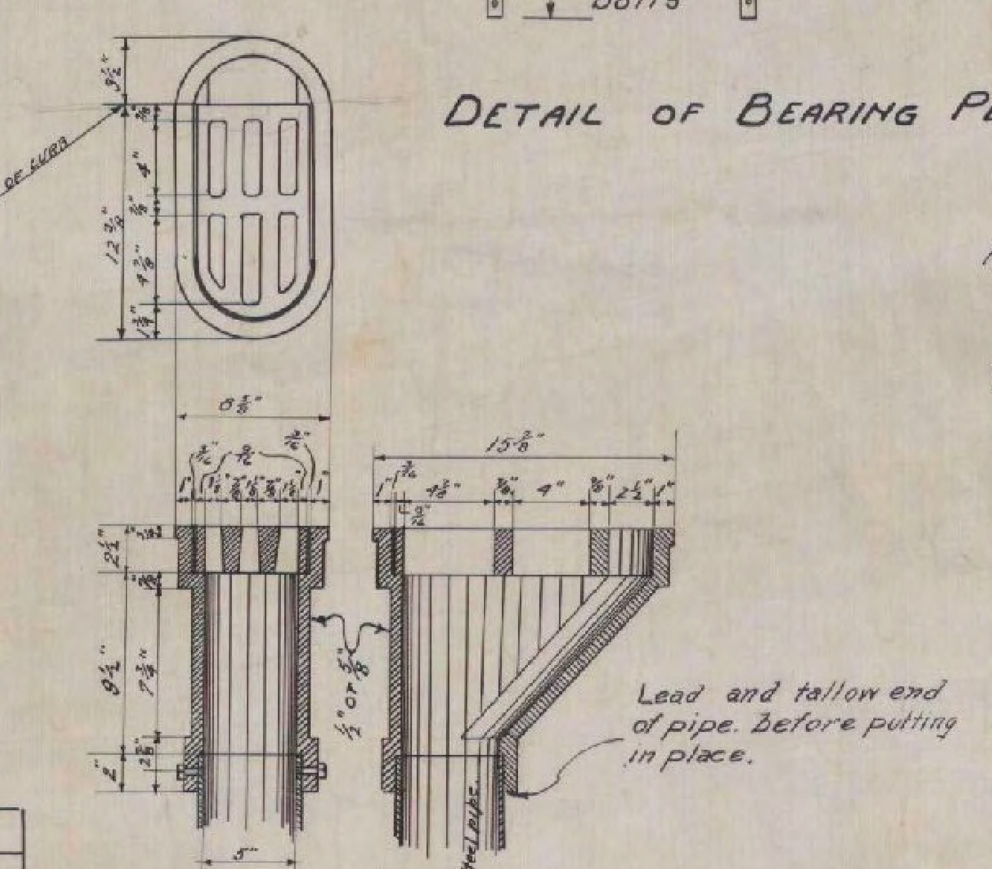
CROSS SECTION OF APPROACH SPAN

Scale: ½" = 1'-0"





DETAIL OF BEARING PLATE



DETAIL OF DRAIN

Scale 1½" = 1'-0"

References:-

S.B. No.1 5-10-11-13-21

S.B. No.2 5-221

See sheet 10 for general notes and references.

SUPERSTRUCTURE DETAILS CHURCH HILL BRIDGE

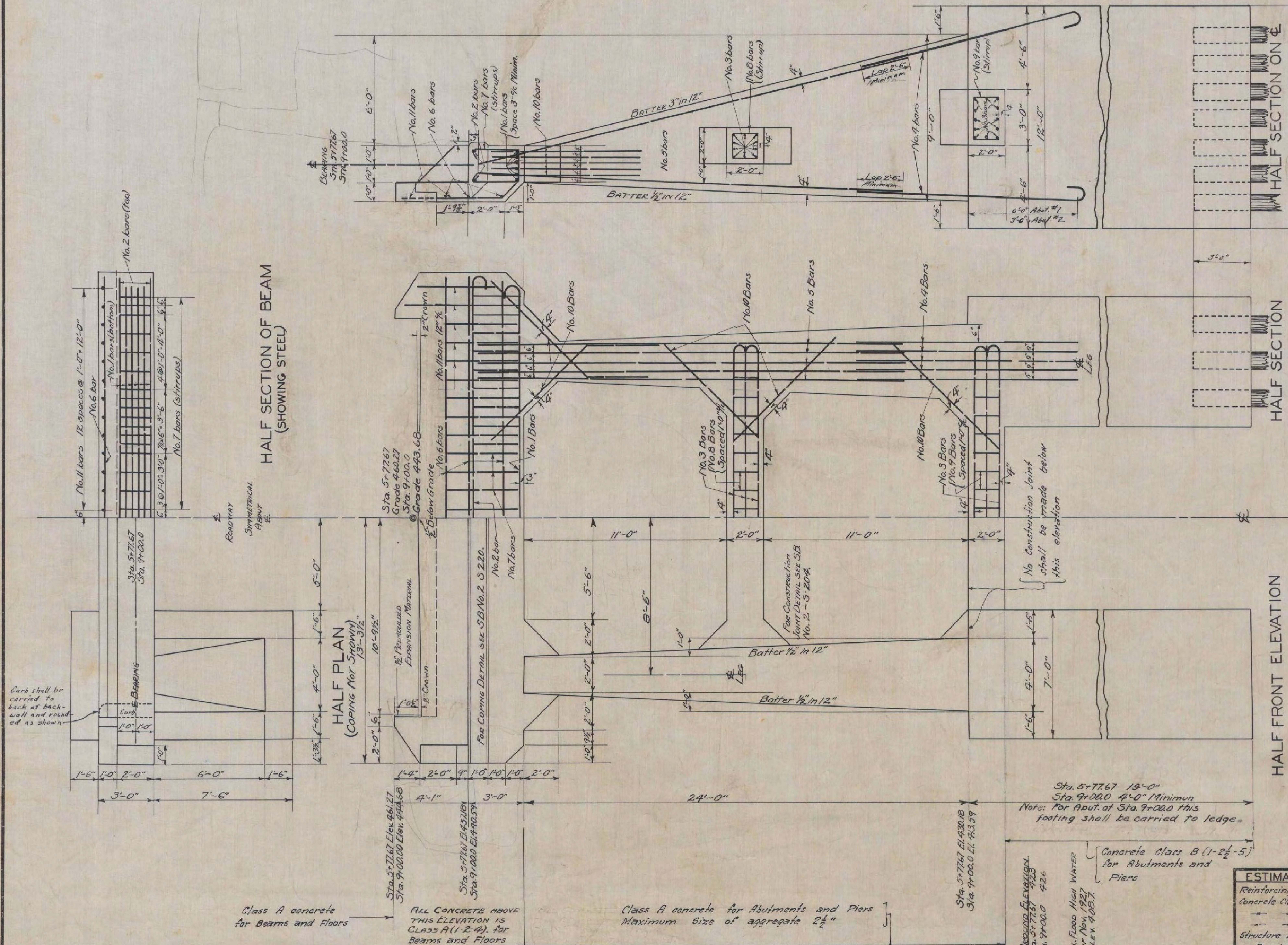
BETHEL, VT.

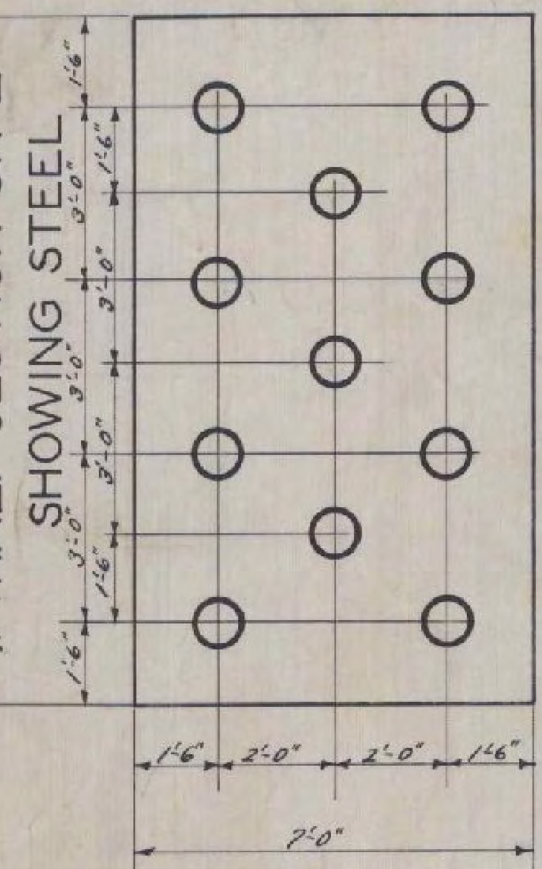
F.R.-12H

ESTIMATED QUANTITIES	
— 2 APPROACH SPANS —	
Structural Steel	48800#
Reinforcing Steel	8400
Concrete, Class A (1:2:4) for Beams & Slabs	49. Cu. Yds.

Surveyed by
Designed by FWG 9/15/28
Drawn by FWG 9/15/28
Traced by A.G. 9/17/28
Checked by W.R.M. 10/6 28
Series FR No. 12H Filed
Sheet 11 of 34 Sheets

FED. ROAD DIST. NO.	STATE	F.R.P.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	VT.	12 H	1928	14	34





PILE PLAN FOR ABUTMENT NO. 1.

PILES ASSUMED AT 25'-0" FOR ESTIMATING PURPOSES ONLY. PILES TO BE DRIVEN TO A 15 TON BEARING CAPACITY.

Note — Bottom of footing elevation lowered 5'-0" and piling added Abutment No. 1. Oct 22, 1928.

REINFORCING STEEL DETAILS

No	Size	L	Tot. Lgh	Remarks
1	1" ⌀	24'-3"	26'-9"	Main steel in beam
2	1¼" ⌀	25'-9"	28'-3"	Steel in top of beam
3	¾" ⌀	18'-0"	20'-6"	Steel in tie beams

No. 4 — ¾" ⌀ — 14'-0" — 4" R — 3" — Tot. Lg'th 15'-3" Steel at bottom of leg.

No.	Size	L	Remarks
5	¾" ⌀	22'-0"	Steel in legs
6	½" ⌀	26'-0"	Tie steel in backwall

No. 7 — ½" rd. — 1'-8" — 2'-7" — 6" — 60° — Tot. Lg'th 7'-10"

No. 8 — ⅜" ⌀ — 3" — 3" — 1'-4" — 1'-4" — Tot. Lg'th 5'-10" Tie steel in center beam

No. 9 — ⅜" ⌀ — 3" — 3" — 2'-4" — 1'-4" — Tot. Lg'th 7'-10" Tie steel in bottom beam

No. 10 — ¾" ⌀ — 7'-0" — 3'-0" — 45° — Tot. Lg'th 10'-0" Steel top of legs

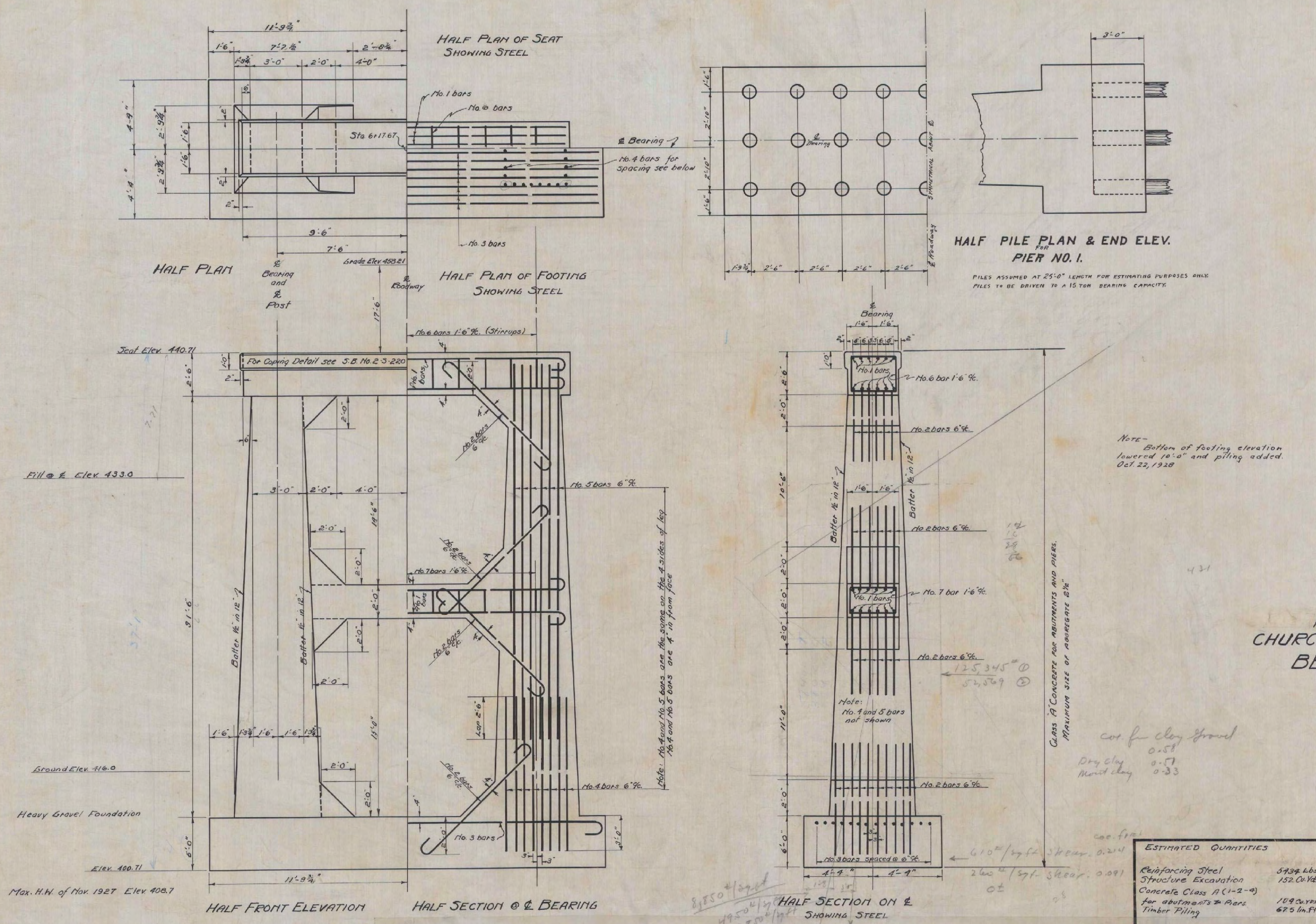
No. 11 — ¾" ⌀ — 3'-6" — 1'-3" — 1'-6" — 45° — Tot. Lg'th 6'-3" Steel in backwall.

ABUTMENTS #1 & #2 AT STA. 5+77.67 AND 9+00 CHURCH HILL BRIDGE BETHEL VT.

ESTIMATE OF QUANTITIES		
Reinforcing Steel	7510	lbs.
Concrete Class A (Beams & Floors)	22	cu. yds
— — A (Abutm. & Piers)	98	" "
— — B (Abutm. & Piers)	118	" "
Structure Excavation For both Abutments.	503	" "
Timber Piling (Abut. #1)	550	lin ft.

Surveyed by
Designed by F.W.G 9/7/28
Drawn by F.W.G 9/7/28
Traced by R.G.M.
Checked by M.F.B 9/18/28
Series F.R. No. 12H Filed
Sheet 14 of 34 Sheets

FED. ROAD DIST. NO.	STATE	[illegible]	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	VT.	F.R. 12H	1928	15	34



Reinforcing Steel Details

Bar	Size	L	Tot. Lgth.	Remarks
1	¾" ϕ	17'-9"	20'-3"	Steel in seat and tie
2	¾" ϕ	8'-0"	10'-6"	Bars at corners
3	1" ϕ	22'-7"	25'-1"	Steel in Footing

Bar 4 — ¾" ϕ — 9'-0" — 4" R — Tot. Lgth 10'-3" Vertical Steel at bottom of legs

Bar 5 — ¾" ϕ — 32'-0" — Vertical Steel in legs

No. 6 — ⅜" ϕ — 2'-4" × 1'-8" — Tot. Lgth 8'-6" Stirrup top beam

No. 7 — ⅜" ϕ — 2'-4" × 1'-4" — Tot. Lgth 7'-10" Stirrup tie beam

For General Notes see S.B. No. 2 S-221

Note – Bottom of footing elevation lowered 10'-0" and piling added. Oct. 22, 1928

PIER NO. 1
CHURCH HILL BRIDGE
BETHEL VT.
F.R. No. 12-H

coe. for clay gravel 0.58
Dry clay 0.51
Moist clay 0.33

125,345 (1)
52,569 (2)

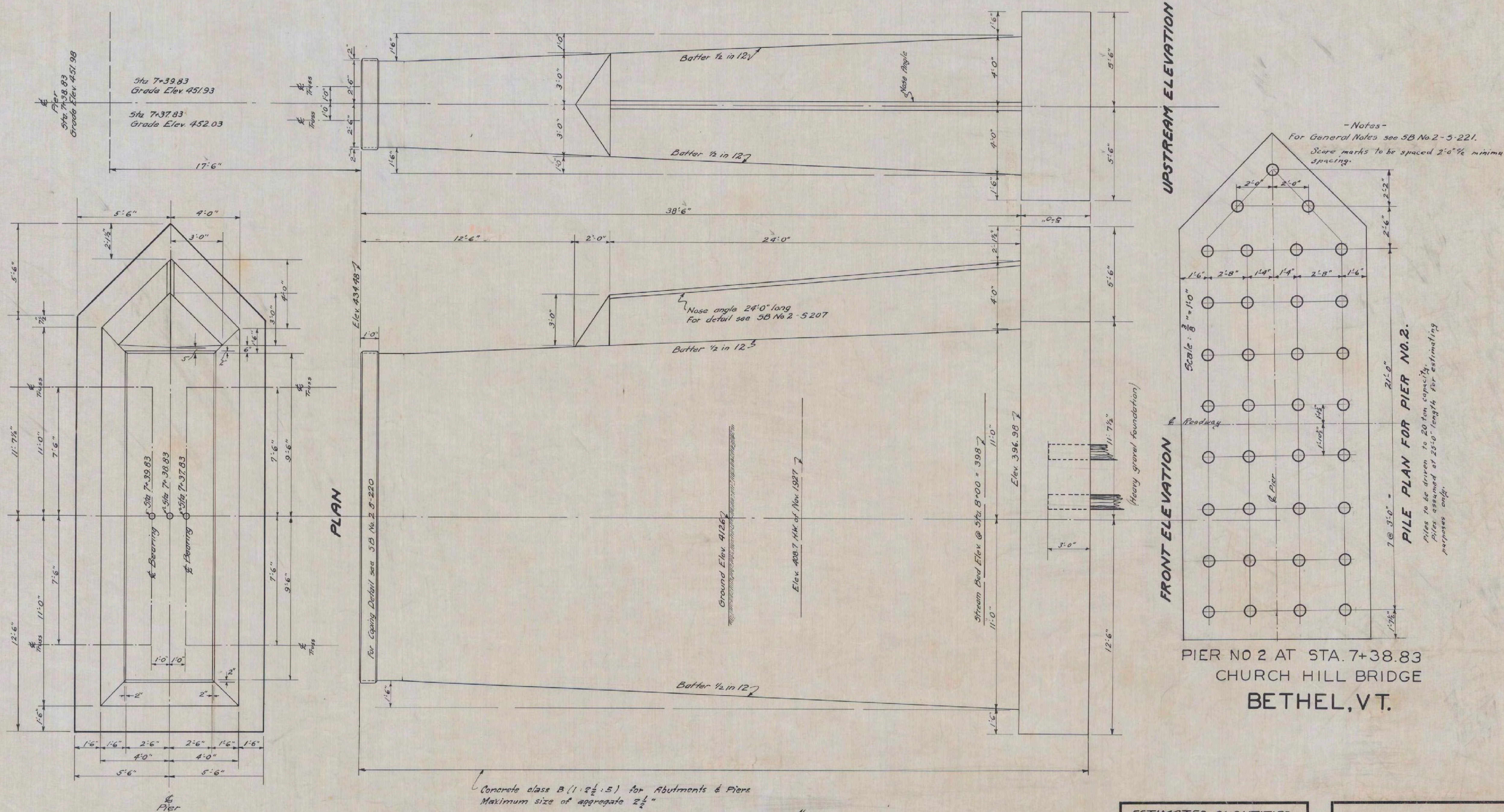
610#/sq.ft shear. 0.214
260#/sq.ft shear. 0.091

Estimated Quantities

Reinforcing Steel	5434 Lbs.
Structure Excavation	152 Cu. Yds.
Concrete Class A (1-2-4) for abutments & Piers.	109 Cu. Yds.
Timber Piling	675 lin. Ft.

Surveyed by
Designed by F.W.G. 9/11/28
Drawn by F.W.G. 9/11/28
Traced by A.R.D. 9/12/28
Checked by M.F.B. 9/12/28
Series FR No. 12H Filed
Sheet 15 of 34 Sheets

FED. ROAD DIST. NO.	STATE	FR	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	VT.	12 H	1928	16	34



-Notes-

For General Notes see SB No. 2-S-221.

Score marks to be spaced 2'0" % minimum spacing.

PIER NO 2 AT STA. 7+38.83
CHURCH HILL BRIDGE

BETHEL, VT.

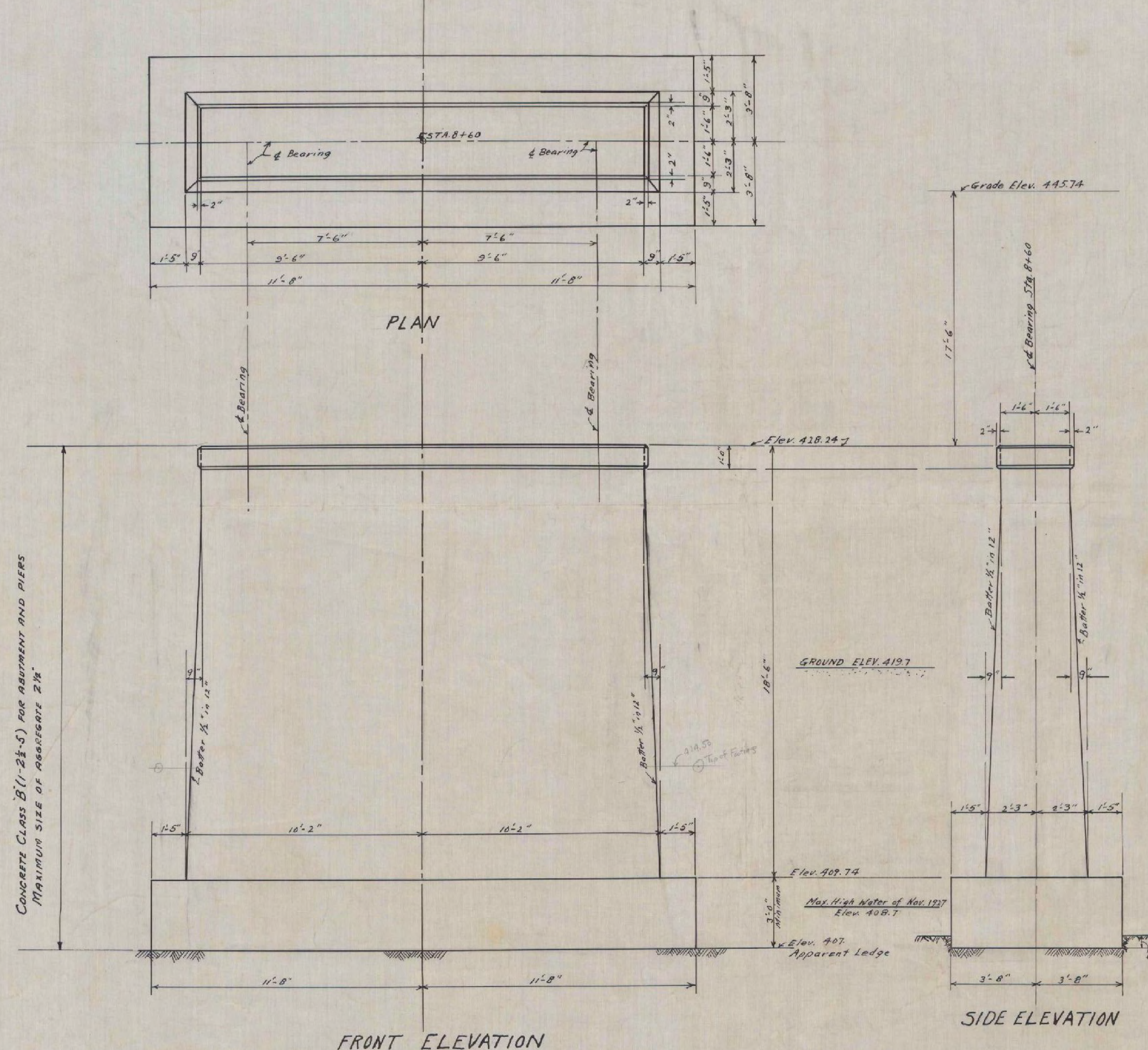
Concrete class B (1:2½:5) for Abutments & Piers
Maximum size of aggregate 2½"

Note - Top of footing elevation raised 1'0" and piling added Oct. 22, 1928.

ESTIMATED QUANTITIES	
Structure Excavation	286 Cu. Yds.
Concrete Class B(1:2½:5) for Abutments & Piers	247 Cu. Yds.
Steel Nosing - Reinforcing Steel	270 #
Piles	875 lin. ft.

Surveyed by		
Designed by	F.W.G.	9/12/28
Drawn by	F.W.G.	9/12/28
Traced by	A.G.	9/12/28
Checked by	M.F.B.	9/13/28
Series FR	No. 12 H	Filed
	Sheet 16 of 34 Sheets	

FED. ROAD DIST. NO.	STATE	[illegible]	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	VT.	FR 12H	1928	17	34



Scale: 3/8" = 1'-0"

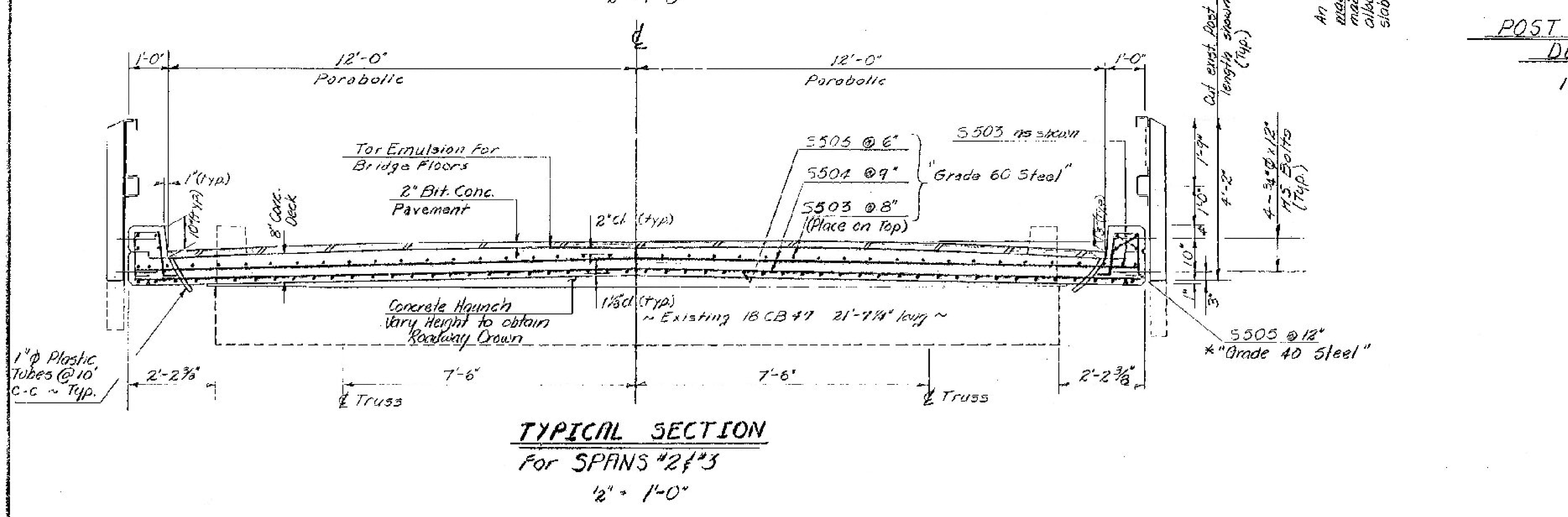
For General Notes, See SB-No.2 - S221
For Coping Details, See SB-No.2 - S220
Score Marker placed 2'-0" c/c Minimum spacing.

PIER NO. 3
CHURCH HILL BRIDGE AT BETHEL, VT.

ESTIMATE OF QUANTITIES	
Structure Excavation	115 Cu. Yds.
Class B (1-2½-5) Concrete for abutments and piers	68 Cu. Yds.

Surveyed by
Designed by J.W.G. 9/11/28
Drawn by J.W.G. 9/11/28
Traced by M.F. Decker 9/12/28
Checked by M.F. Decker 9/12/28
Series FR No. 12 H Filed
Sheet 17 of 34 Sheets


Vt. F.R.P. 12H
To Randolph
To Bethel
Plan of Top Bracing
Erector's Note: Place 4-½" washers at railing connections to each F.P. to make rail straight. See Sh. 12.
Grade of Roadway ⅛" per foot.
K17 at all F.P.2
Crown of Roadway
Assume ℄ P1 @ 100
Fixed
Exp.
Pier 1
Pier 2
Pier 3
ERECTOR. Assume grade at P1 ℄ B.C. = 100 Randolph End. Then grade at ✕ ℄ B.C. = 98'-4½" .. .. ψ ℄ B.C. = 95'-8½"
Erector. 3 Layers of 12 to 14 oz. duck to be laid in red lead under shoes on top of Piers "1", "2", & "3".
El. 408.7' High Water
El. 400' Low Water
~Detail at End~ ~Scale: ½"=1'~
~Detail at End~ ~Scale: ½"=1'-0"~
40-0
14 Panels @ 8-7 = 120-2
120-2
40-0
Sectional Elevation on ℄ of Bridge
Plan of Bottom Bracing
Section at End Piers
C50 Randolph End. C54 Bethel End.
Typical Section
Section at Center Pier
C52 Randolph End C53 Bethel
Sect. A-A Scale: ½"=1'
List of Sheets
1 Erection Plan
2 Castings
3 Trusses
4 do.
5 do.
6 do.
7 Sways
8 do.
9 Lateral Brcg. & Scuppers
10 Fl. Bms. & Apron Pls.
11 Approach Spans
12 Fence
The Boston Bridge Works Inc. Boston, Mass.
Name Bethel, Vt. Church Hill Bridge
Details of Erection Plan
Scale 3/32" & ¼" = 1'-0
Drawn by R.F.W.
Job 5685
Sh. 1



ITEM NO.	ITEM	UNIT	NET			TOTAL	FINAL
	CHAN. EXCAV. OF EARTH	C.Y.					
	CHAN. EXCAV. OF ROCK	C.Y.					
	UNCLASS. CHAN. EXCAV.	C.Y.					
	STRUCT. EXCAV.	C.Y.					
	CONC. CLASS AA (MOD.)	C.Y.					
	CONC. CLASS B (MOD.)	C.Y.					
	REINF. STEEL	LBS.					
	ASPHALTIC-ASB. COATING	S.Y.					
	TREATED TIMBER PILING	L.F.					
	SPLICES FOR STEEL PILING	EA.					
	STEEL PILING	L.F.					
	UNTREATED TIMBER PILING	L.F.					

STATE OF VERMONT
DEPARTMENT OF HIGHWAYS

TOWN OF BETHEL
ROAD NO. VT 12 BRIDGE NO. 38
VT 12 OVER GILEAD BROOK
TYPICAL SECTIONS, DETAILS
SCALE AS SHOWN
SECTION SUPERVISOR J.R.JENNINGS
DRAWN BY P.Plumb & J.A.Richardson CHECKED BY E.L. Leonard
PROJECT NO. BMH 611-8
Br. 102 SHEET 4 OF 16

Summary of the Load Rating Analysis of the Existing Deck Truss

Appendix I

APPENDIX I - Summary of the Load Rating Analysis of the Existing Deck Truss

CHA was scoped in Phase 1 of this project (Bethel BHF 0241(38)) to perform a load rating analysis of the deck trusses of the existing bridge.

This analysis included all the individual truss elements and gusset plates based on the information provided in the 1928 Record Plans and the 1971 rehabilitation plans. No consideration was given to as-built conditions, deterioration, or other deficiencies of the structure in the analysis. Other members of the structure such as the substructures, steel girders in spans 1 and 4, floorbeams in spans 2 and 3, and the structural deck were not analyzed.

The load rating analysis was done in accordance with the following using HL-93 loading:

- VTrans Structures Design Manual, 5th Edition, 2010, by the VTrans Structures Section
- The Manual for Bridge Evaluation, 2nd Edition, 2011, by the American Association of State Highway and Transportation Officials
- AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010 with current interims, by the American Association of State Highway and Transportation Officials
- Publication No. FHWA-IF-09-014, Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges, February 2009, by the Federal Highway Administration

The model used in the analysis consisted of a two-dimensional structural system. Based on structure symmetry about Pier No. 2 and the centerline of Vermont Route 12, a single truss was analyzed – results for the other a three (3) trusses were assumed identical for Phase 1 purposes.

Based on the information provided on the Structure Inspection, Inventory, and Appraisal Sheet completed on November 30, 2011 for the existing bridge the original design live load was an AASHO H-15. Since no strengthening of the bridge members has occurred in the past, the bridge should be inadequate to carry today's standard loads. This was confirmed by the findings of the load rating analysis. See the summary tables on the following sheet for a breakdown of the limiting truss elements (inventory rating factors, RF_{INV}, less than 1 indicate that the applied loads are greater than the design capacity of the member).

<u>Truss Members</u>

	NUMBER	LOCATION	MEMBER	RF_{INV}	RF_{OPER}
BOTTOM CHORD	2	L2-L4	BC1	***0.74***	***0.95***
	4	L4-L6	BC2	***0.79***	***1.02***
	5	L6-L8	BC2	***0.69***	***0.89***
	6	L8-L10	BC2	***0.79***	***1.02***
	7	L10-L12	BC3	***0.74***	***0.95***
TOP CHORD	13	U3-U4	$TC1_{sec2}$	***0.75***	***0.97***
	14	U4-U5	$TC1_{sec2}$	***0.75***	***0.97***
	15	U5-U6	TC2	***0.64***	***0.83***
	16	U6-U7	TC2	***0.64***	***0.83***
	17	U7-U8	TC2	***0.64***	***0.83***
	18	U8-U9	TC2	***0.64***	***0.83***
	19	U9-U10	$TC3_{sec1}$	***0.75***	***0.97***
	20	U10-U11	$TC3_{sec1}$	***0.75***	***0.97***
DIAGONALS	32	L0-U1	D1	***0.71***	***0.92***
	33(T)	L2-U1	D2	***0.68***	***0.89***
	34(C)	L2-U3	D3	***0.68***	***0.88***
	35(T)	L4-U3	D4	***0.71***	***0.93***
	36(C)	L4-U5	D5	***0.68***	***0.88***
	41(C)	L10-U9	D8	***0.61***	***0.79***
	42(T)	L10-U11	D9	***0.77***	***1.00***
	43(C)	L12-U11	D10	***0.61***	***0.79***
	44(T)	L12-U13	D11	***0.75***	***0.97***
	45	L14-U13	D12	***0.64***	***0.83***

<u>Gusset Plate Connections</u>

	Plate			**Rivet**		
LOCATION	RF_{INV}	RF_{OPER}	CRITERIA	RF_{INV}	RF_{OPER}	CRITERIA
L2	**0.86**	**1.11**	Shear (Section A-A)	**0.72**	**0.94**	Shear Capacity (L2-U1)
L4	**1.18**	**1.52**	Shear (Section A-A)	**0.84**	**1.08**	Shear Capacity (L4-U3)
L6	**1.43**	**1.85**	Tension (L6-U5)	**0.65**	**0.84**	Shear Capacity (L6-U5)
U1	**0.73**	**0.95**	Shear (Section A-A)	**0.70**	**0.90**	Shear Capacity (L2-U1)
U3	**1.04**	**1.34**	Compression (U3-U4)	**0.89**	**1.16**	Shear Capacity (L4-U3)
U5	**0.82**	**1.07**	Compression (U5-U6)	**0.65**	**0.84**	Shear Capacity (L6-U5)