Stantec

COBBOSSEE TRAIL FEASIBILITY STUDY

WIN 013344.10
Cobbossee Trail Extension
Gardiner, Maine

February 25, 2022

Prepared for:

City of Gardiner

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1.0 INTRODUCTION AND SUMMARY

For many years, the City of Gardiner has been planning a Cobbossee Stream multi-use trail. Construction of the portion connecting the Kennebec River Rail Trail corridor to the Arcade/Water Street intersection will be completed in 2022. The trail will eventually be extended from the Arcade/Water Street intersection to cross the Cobbossee Stream on Winter Street and proceed down Summer Street. The purpose of this project is to investigate different trail alignments to optimize cost and constructability to connect from the southern end of Summer Street back to Water Street via a new trail bridge crossing of the Cobbossee Stream near the abandoned railroad trestle, which is to be removed by the Maine Department of Transportation (MaineDOT).

This report describes the process of selecting the two alignment alternatives and some of the considerations that went into choosing a recommended alternative.

2.0 ALIGNMENTS AND DEVELOPMENT

2.1 PROCESS OF SELECTING ALIGNMENTS

2.1.1 Initial Review of Alignments

During the preparation of the proposal, Stantec conducted a site visit and qualitatively concluded that an alignment upstream of the trestle was not practical due to existing grade challenges, limited construction access, and likelihood of high cost due to extensive retaining walls. This study therefore focused on alignment alternatives downstream as agreed upon during the August 4, 2021 Trail Committee meeting.

The original alignment concepts, presented to the Trail Committee on August 4, 2021, showed an alignment running roughly parallel to the existing trestle to stay within the State ROW to the extent possible and an alignment further downstream that was positioned to minimize the span length. Further review of the clearance required between the parallel structure and the existing trestle prompted a decision by the Department to remove the deteriorating trestle.

No longer having to work around the existing trestle opened up options for reusing the existing trestle alignment and northern abutment area with a new bridge structure. Stantec discussed three bridge options with the Trail Committee during the November 15, 2021 meeting:

- Green Alignment (Option 1A) – This alignment runs along the existing alignment and either re-uses the existing pier or includes a new pier
    - Reuses existing northern abutment area
    - Requires two spans
    - New southern abutment is inside of the stream but outside the FEMA regulatory floodway
    - Considerations that went into choosing a new pier over reusing the pier are discussed further below

- Blue Alignment (Option 1B) – This skewed alignment re-uses the existing northern abutment landing area
  - Reuses existing northern abutment area
  - Single span
  - New southern abutment is outside of the stream and the FEMA regulatory floodway
- Original skewed downstream alignment (Option 2) – This alignment is identical to the downstream alignment presented to the City on August 4, 2021
  - Has more ROW impacts

Discussion and a vote during the meeting narrowed this down to the two options discussed in this report: the one re-using the existing alignment with a new pier (now called Option 1) and the one re-using the existing northern abutment landing area (now called Option 2).

2.1.2 Considered Alignments

Figure 1 depicts the considered trail alignments. Below is a description of the options from the north at the end of Summer Street and progressing southward. The trail crosses the Cobbossee Stream on a multi-use bridge and continues to Water Street. Appendix A contains renderings from a viewpoint south of the existing north abutment looking upstream. Appendix C contains the Conceptual Plans for both options. For reference, the span of the multi-use bridge in downtown Gardiner is 77 feet.

2.1.2.1 Option 1

The alignment for Option 1 is shown in blue on Figure 1 and starts at Summer Street and swings out parallel to the existing railroad alignment to connect with the trestle alignment. In Figure 2, one can see the existing trail with sheet pile wall on the left and the end of Summer Street. Not precisely following the existing alignment eliminates the curve from the bridge, which crosses the stream in one 100-foot span, lands on a pier built in the footprint of one of the existing concrete piers and continues in another 129-foot span. The downward grade of the bridge crossing is 2.92%. From the bridge, people traveling on the path would see up and downstream. The downstream view is shown in Figure 3. The southern landing of the bridge has about 100 feet of retaining wall along the stream as the alignment stays within the footprint of the existing trestle alignment. As the alignment curves away from the stream, the grade begins to climb at 7.00% with an ADA compliant "rest area" on the stream side. Approaching the southern abutment and end of the trestle, the trail eventually ducks to the east of the existing alignment to preserve the section of the trestle that will remain as a viewing platform. Traveling southward and up the grade, this platform will be on the right with the stream beyond. Figure 4 shows the end of the trestle looking north, which is the perspective of the viewing platform. In the fall and winter, when the foliage is less dense, the stream would be more visible from this location. At the top of the slope, the trail continues to follow the existing alignment, first at a gentle 1.29% grade and then increasing to 5.00% grade as the trail cuts across a corner of the Warren Parcel to connect to Water Street at a 1.45% grade.

Figure 1. Trail Alignment Crossing Options

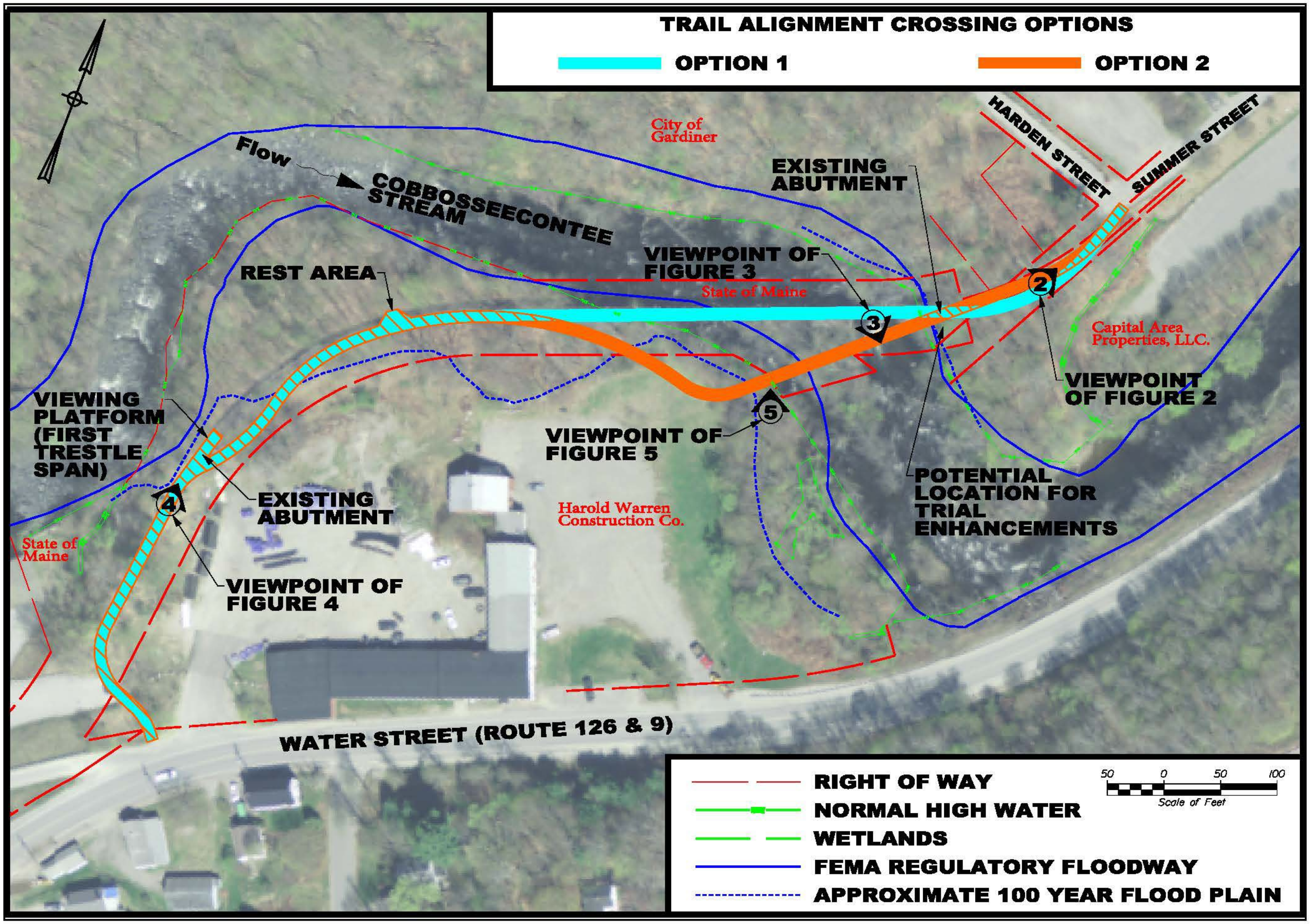


Figure 2. View looking north toward Summer Street with sheet pile wall along track alignment.

Figure 3. View looking downstream from the existing trestle.

Figure 4. View looking north at existing southern trestle bay that will be preserved for viewing platform. Trail will continue down the slope on the right.

2.1.2.2 Option 2

The alignment for Option 2 starts at Summer Street and follows the existing railroad alignment to the existing northern abutment before crossing diagonally south of the existing trestle alignment at a 2.77% grade. This approach is similar to Option 1 and Figure 2 shows the view looking north toward Summer Street. The view from the bridge would show long views of both upstream and downstream, with a bird's eye view of the island in the middle of the stream and Figure 3 shows the downstream view. The bridge span is 185 feet and lands on the Warren parcel before sweeping back north to the existing trestle alignment. Figure 5 provides the view looking at the stream from the southern abutment.

Once Option 2 connects to the existing trestle alignment, the alignment for Option 2 matches that of Option 1. As the alignment curves away from the stream, the grade begins to climb at 7.00% with an ADA compliant "rest area" on the stream side. Approaching the southern abutment and end of the trestle, the trail eventually ducks to the east of the existing alignment to preserve the section of the trestle that will remain as a viewing platform. Traveling southward and up the grade, this platform will be on the right with the stream beyond. Figure 4 shows the end of the trestle looking north, which is the perspective of the viewing platform. In the fall and winter, when the foliage is less dense, the stream would be more visible from this location. At the top of the slope, the trail continues to follow the existing alignment, first at a gentle 1.29% grade and then increasing to 5.00% grade as the trail cuts across a corner of the Warren Parcel to connect to Water Street at a 1.45% grade.

Figure 5. View looking at southern abutment location for Option 2

2.2 PERMITTING

Stantec limited in-stream work and environmental impacts to the extent possible to limit wetland impacts to below the MaineDEP Permit By Rule (PBR) and ACOE Category II levels to help expedite the permitting process. MaineDOT will coordinate the environmental process and eventual preparation of permit applications, including NEPA, Section 106, Section 4(f), Section 7, and hazardous materials as part of the Local Project Administration (LPA) process. Cultural/historical resources have not been identified in the project site to date.

This project is located within FEMA's special flood hazard zone AE, so the City of Gardiner requires a Floodplain Management Permit, which would be prepared in the following phase. Eliminating impacts to the floodway and maintaining at least one foot of vertical clearance over the FEMA 100-year base flood elevation will keep this process simpler. The current design for both options maintains this one foot minimum of vertical clearance.

Impacts that will require permitting include any clearing, permanent structures or embankments, temporary construction impacts such as cofferdams and crane pads, and underwater excavation (dredge) within the 100-year floodplain. Combined permanent and temporary impacts for both options are expected to be less than the MaineDEP PBR limits, but Option 1 will have 40% to 50% more wetland impacts than Option 2.

Based on Stantec's experience on the Gardiner Bridge Street project, this project may require Section 7 consultation for endangered fish species. However, we do not anticipate a restrictive in-water work window or unusual limits on the types of in-stream work that can occur.

2.3 UTILITIES

Survey and ROW investigation have only identified a sewer easement, depicted on the plans. However, as part of the utility coordination process, Stantec sent out Letter #1 on 1/20/2022 to the utilities listed below to ensure that none are missed during further design efforts. This letter and responses received as of Feb 25, 2022, from the utilities are in Appendix F.

- Central Maine Power
- Charter
- Consolidated Communications
- Gardiner Water District
- City of Gardiner
- Summit Natural Gas
- Oxford Networks
- MaineCom Services

2.4 RIGHT-OF-WAY

Both alignment options generally remain within the existing State ROW but do have impacts to the Capital Area Properties parcel on the north side of the stream and Harold Warren Construction Company parcel on the south side of the stream. Both Options 1 and 2 have anticipated temporary grading impacts to the Capital Area Properties and the Harold Warren Construction Company and an anticipated acquisition on the southern corner of the Harold Warren Construction Company parcel. Alignment Option 2, however, has an additional anticipated acquisition on the northern corner of the Harold Warren Construction Company parcel where the alignment was positioned to minimize the bridge crossing length. Both alignment options will need temporary rights for construction access also. Stantec understands that the City has met with the owner to initiate discussions about the trail project. Property Owner Reports (PORs) have been sent to the affected property owners and are included in Appendix E.

2.5 BRIDGES

2.5.1 Superstructure Type and Aesthetics

Stantec considered two types of multi-use trail bridges for this location: prefabricated H-section trusses and modular steel trusses. While there are as many bridge types for pedestrian bridges as there are for vehicular bridges, prefabricated and modular bridges have many advantages. They are faster to construct, are less expensive, and require minimal engineering. The manufacturers design the bridges according to a performance specification and have templates and experience that result in significant gains in efficiency both in design and materials. For this reason, customization significantly increases costs.

Prefabricated H-section trusses like the one built in 2019 in downtown Gardiner are the most common multi-use trail bridge type (see Figure 6). Available from several different manufacturers, these steel truss bridges are shipped in one or two pieces and assembled on site before being installed in a single placement. These bridges can be weathering steel, galvanized, or galvanized and painted, and can have timber or concrete decks.

Figure 6. Prefabricated H-section truss bridge, galvanized and painted, in Gardiner, ME. This bridge was shipped in two pieces that were bolted together in a laydown area and then lifted into place as a single piece with a crane.

Modular steel truss bridges are often used for temporary vehicular or pedestrian bridges (see Figures 7 and 8). They consist of multiple 10-foot-long prefabricated steel truss panels that are shipped in shipping containers and assembled on site. These bridges are more utilitarian in appearance since the thinner truss panels often must be doubled or tripled for larger spans. They come with a galvanized finish. The contractor would be required to provide the pressure treated timber stringers to support the deck, the IPE deck, and the pressure treated timber railing. Stantec's estimate has included costs for these items in the lump sum of the modular steel trusses.

Modular steel truss bridges are typically launched from one end support or foundation by continually constructing individual panels and pushing the truss further out into the span. The launching end is often counter-weighted until the receiving end lands on the far support or a crane assists on the receiving end as needed. This serves to reduce the need for heavier cranes and associated crane pads, which is beneficial in constrained sites or sites with varied topography.

Figure 7. Modular steel temporary pedestrian bridge with timber railing in Kennebunk, ME.

Figure 8. This close-up of the modular bridge shows the two outer truss layers, timber railing and sets of bolts that connect the panels together. Note that the surface in this photo does not represent the anticipated IPE timber deck.

The finish of the selected superstructure option is not entirely an aesthetic decision. Cost and structure life are also important to consider. Weathering steel, while the least costly finish, is the most likely to deteriorate, especially in a moist environment. H-section trusses are often built using closed tube members, and condensation can build up due to temperature and moisture differentials on the inside and outside of the members. Moisture causes corrosion on the inside of the steel, but since the members are closed, this corrosion might advance past tolerable levels before the structure shows any distress. A 50-to-75-year design life would not be a guarantee for a weathering steel truss, and MaineDOT does not recommend use of weathering steel for closed members. A galvanized finish would protect the structure and would be the next least expensive. Galvanizing and painting would provide the most options for customizing the appearance of the structure, but painting is only available for the H-section truss option.

Maintenance of the coating will vary depending on use of salt and sand and any damage to the structure. Galvanized coatings require touch-ups when damaged to prevent continued deterioration, so the bridge should be inspected for coating deterioration regularly. If painted, the structure would likely require re-painting at least once during its lifetime, as paint will dull and eventually peel as a result of long-term weathering or damage.

Both bridge options include an IPE timber deck, which matches the surface of the multi-use bridge in downtown Gardiner shown in Figures 6 and 9. IPE is a durable, dense wood that is rot, mold, fungus, and insect resistant. It has a long life and is relatively maintenance-free unless a board gets damaged by snow removal equipment. The surface is also more slip-resistant than other timber options.

2.5.2 Constructability

The steep slopes, heavy vegetation, and lack of existing access to the site create challenges for construction. Clearing and temporary easements will be required for crane and construction vehicle access at both ends of the bridge.

Figure 9. Installation of the Cobbossee Trail Multi-Use Bridge in downtown Gardiner using a single 110-ton crane for the 10-foot-wide, 76.5-foot span bridge.

The major challenge for both alignment options is that prefabricated H-section trusses are heavy, and the available locations for crane placement result in large pick radii. The larger the pick radius, the less capacity a crane has. For both bridge alignment options, a sheet pile retaining wall running parallel to the train track at the north end limits the size, placement, and movement of the crane, because a crane needs room to rotate as it swings to install the bridge. As a result, single-crane or even two-crane installation may not be possible. Potential solutions for this issue are as follows:

- Place a crane on the island
  - Downsides to this include the need for clearing on the island, which is a permanent wetland impact, and the need for temporary access to the island, which would be subject to flood flows and would also be a temporary wetland impact and additional cost.
- Assemble the bridge in place instead of in a laydown area.
  - Would require falsework at the splice point to hold it in place.
  - Requires multiple large cranes.
- Launch the bridge from one side
  - This option works best with a modular bridge since they are designed to be assembled and launched as they are constructed. The bridge would be launched as a cantilever with a counterweight on the back end until a crane can assist from the far abutment area. This reduces the need for heavier cranes and associated crane pads, which is beneficial in constrained sites or sites with varied topography.

- This could also work with a prefabricated truss with falsework in the island area and a second crane at the opposite end.

Temporary ROW rights outside of State ROW will be needed at both ends of the bridge for installation. While the method of construction would be up to the project contractor, the ROW process would account for necessary laydown and access areas during final design.

2.5.3 Substructure Type

The substructure options described below were developed with guidance from the Geotechnical Engineering Report included in Appendix G. The soils in the area are mostly dense glacial till, which can develop sufficient bearing resistance for a multi-use bridge, meaning that piles will not be required. Additional soils information and analysis may be required for final design.

Abutments

The abutment alternatives that Stantec initially considered were:

- Full height cantilever abutments on piles
- Modular steel bin-type walls
- Stub abutments founded on mechanically stabilized earth (MSE) walls at the north end
- Bent-type substructure unit at the south end to connect to a switchback ramp

These alternatives were originally developed assuming alignments that avoided the existing railroad bridge. MaineDOT's decision to remove the existing bridge allowed the option of reusing the existing bin wall at the north end of the project. Stantec also determined that the grades are gradual enough that a switchback ramp is not necessary to meet ADA requirements for the trail, so the bent-type substructure was no longer a beneficial option.

Stub abutments on MSE walls are an aesthetically pleasing option that is commonly used elsewhere in New England. However, MSE walls require specialized design and significant excavation for installation. They are constructed using a concrete facing with long steel reinforcing strips to stabilize the soil, so installation may require much larger excavation footprints than a conventional cantilever abutment would. MSE walls are more common in embankment fill areas for bridges on completely new alignments than they are for cut areas. Additionally, while MSE wall technology has improved, there have been past failures with MSE walls adjacent to waterways because of the potential for the backfill to wash out during a flood event. For these reasons, these are not the best option for this location.

Conventional cast-in-place concrete, full-height cantilever abutments on spread footings have several advantages in this location. They:

- Do not require specialized engineering,
- Can be founded directly on the glacial till,
- Do not require deep excavation,
- Are adaptable to unforeseen subsurface conditions,

- Are the most resistant to flood flows, and
- Do not require specialized materials.

Pier

Stantec investigated the costs for rehabilitating versus replacing-in-kind the existing pier for the Option 1 alignment and shared the results with the Gardiner Trail Committee during the November 15, 2021, meeting. Items and considerations for re-using the existing pier included:

- Closer inspection of the pier, including sounding with hammers, to determine areas of repair
    - Challenges: Access – need to get to all parts of the pier – difficult to do when water is high, need to be able to put a ladder in the channel or mobilize climbing inspectors to reach all parts of the pier
    - Costs: Access equipment & labor
- Concrete coring to determine concrete strengths for design, laboratory testing of cores
    - Challenges: Access for taking cores
    - Costs: Labor, coring, lab testing
- Determine whether there are piles or not, and if so, what condition they're in
    - Challenges: Unknown pier foundations within the stream is a tough investigation, can be very costly only to determine that there are no piles. Traditional borings, probes, and geophysical are not going to help determine what's under the concrete footing.
        - The most reliable method would be excavation to expose the foundation, but that poses many challenges between needing a cofferdam and getting an excavator out to the pier
        - Bending wave tests work on timber piles, and are similar to sonic/echo impulse tests, which are non-destructive tests that will determine the presence of piles.
        - If there are piles, then need to figure out what load capacity they were driven to, which can be very difficult without any existing drawings, pile records, or at least the historical borings used. This is especially true with timber piles, which may have deteriorated, especially in a wet environment, and are lower capacity to begin with.
        - Sometimes only way to vet re-use of piles is with load testing. Load testing here may be difficult considering it is a train bridge with poor access and the superstructure load rating is unknown.
    - Costs:
        - Cofferdam
        - Geotechnical exploration to determine unknown foundations for design
        - Potential load testing if piles are present

- If there are no pier piles and the pier is not founded on bedrock (rock unlikely, considering depth of rock in borings), then extensive pier scour counter measures (heavy riprap or precast cable mat in the channel, will need regular maintenance inspections) and potential foundation strengthening (drill down through pier footing and add micropiles) may be needed. If there is any scour undermining now, may have to address that with grout bags.
    - Challenges: Might need cofferdam to install, access for installation
    - Costs: Heavy riprap to fix current scour issues noted in the prior bridge study inspection report
- There is no guarantee that the existing pier would be sufficient for current design codes and new loading; pedestrian trusses can exert surprising force magnitudes, even on a pier that was originally designed for train loading. If we were to pursue this and find that we needed to strengthen the pier, that would add a lot of cost. Repairs might consist of chipping away existing deteriorated concrete, adding a concrete jacket and reinforcing, and/or drilling micropiles, including access, cofferdam, and verification testing. We would also need to modify and build up the rest of the pier with more concrete to accept the new superstructure, which effectively adds a third substructure unit to the project even with re-use.
    - Challenges: Access – would need temporary work platform and trestle for equipment to access the pier
    - Costs:
        - Micropile mobilization and verification testing, temporary work trestle, concrete, piles
        - Concrete and rebar to build up pier

The new pier would replace the existing pier in kind to limit impacts to the floodway. It would require additional basic geotechnical exploration, new concrete, and riprap. Starting fresh with the foundations eliminates the uncertainty of the existing foundations and the condition of the existing concrete The estimated cost to rehabilitate the pier is approximately $20,000-$300,000 more than a new pier. For this reason, Stantec recommends a new pier. The committee agreed during the November 15, 2021 meeting.

Note that either reusing the pier or designing a new pier for Option 1 will add cost to the design versus Option 2, which has no pier.

Retaining Wall

Option 1 will require an approach retaining wall to keep the sideslopes out of the FEMA regulated floodway. Although there are numerous retaining wall options, Stantec considered cast-in-place concrete retaining walls to estimate an initial upper limit cost for these retaining walls. Bin walls such as Contech's galvanized Bin-Wall system or MSE walls would be less expensive options for retaining walls in this location. However, MSE walls are not recommended for the reasons outlined in the abutment section above, and MaineDOT usually does not recommend bin walls because they have had issues with design lives of less than 75 years. Refining the type of retaining wall in this location could reduce the cost, which would be considered in the next phase of the project if this option were progressed.

An option to eliminate the retaining walls would be to move the abutment back and increase the bridge span. However, this increases the span to over 200 feet, which would be far more expensive than the retaining walls and even more difficult to construct than the single-span Option 2.

2.5.4 Hydraulics

Stantec performed a hydrologic, hydraulics, and stream scour analysis and reporting as part of the downtown Gardiner bridges and multi-use trail project, which is around 1900 feet downstream of this location. Stantec gathered flood information and modeling from FEMA resources, estimated storm flows from MaineDOT, and gaging station data for that analysis. Based on our current review of the available FEMA material for this upstream project location, the flood clearances above the FEMA regulated 100-year base flood elevation (BFE) of approximately Elevation 39.2 (NAVD 88) will not control the bridge design or layout.

The main hydraulic concern for either alignment option is avoiding impacting the FEMA regulated floodway, since permanent impacts to the floodway result in changes to FEMA flood insurance mapping. This is the main factor driving the length of the spans for Option 1, which follows the existing trestle alignment. To limit the span lengths, the south abutment lands just outside the FEMA floodway, and a retaining wall keeps the trail sideslopes out of the floodway. The proposed Option 2 bridge span length places the south abutment outside of the floodway and trail sideslopes do not impact it.

Future design efforts will require additional hydraulics analysis to assess changes to the stream based on changes to the hydraulic opening. For Option 2, this is likely to be a simple analysis since removing the existing bridge significantly increases the hydraulic opening, and Option 2 is outside of the floodway and almost entirely outside of the 100-year floodplain. A basic analysis would likely show that there is a net increase in hydraulic opening. However, Option 1 returns obstructions to the floodway and floodplain, including a pier and a long retaining wall, and a more detailed hydraulics analysis would be required to determine if there is a change to the 100-year regulatory flood levels. This would add additional design cost compared to Option 2.

2.6 ADDITIONAL TRAIL ELEMENTS

In addition to the trail and the bridge, there are several trail enhancements that would enrich this section of trail and could be developed as funding becomes available. A limited list of possible enhancements is below. A potential location for some of these enhancements can be seen in Figure 1 near the northern abutment. Enhancements include:

- Trail lighting
- Bridge lighting
- Benches
- Interpretive panels
- Public art
- Picnic areas

The City could coordinate with MaineDOT during demolition of the existing truss to salvage railroad ties or other parts of the bridge for re-use in benches, tables, interpretive panel supports, stair treads, art installations, or other creative endeavors.

2.6.1 Lighting

The AASHTO Guide for the Development of Bicycle Facilities recommends lighting for when night usage is expected, such as when paths serve commuters or when they intersect with roadways. Lighting can also be added in areas where there might be safety or security risks. For pedestrian lighting, the guide recommends closely spaced short (about 15 foot tall) poles with dark-sky-compliant luminaries, which direct light downwards. The single-luminary ornamental light standards in downtown Gardiner would meet these requirements. Additionally, pedestrian-level bridge lighting can add both visibility and aesthetic interest.

The Kennebec River Rail Trail is a good example of a trail with some lit and unlit sections. The majority of the trail that goes through the woods is unlit. However, in locations where the trail intersects with the roadway, the crossings are illuminated. The trail also passes by some buildings that have floodlights for increased security.

Lighting for this section of trail will depend on what the City envisions for future usage. If the City anticipates mostly daytime usage, lighting may not be necessary except at the south end where the trail connects to Water Street. However, if nighttime usage is anticipated, the City may wish to plan for future lighting on the trail and the bridge. At a minimum, Stantec recommends a cobra head light on the utility pole closest to the new crosswalk on Water Street. Conduit and junction boxes for future light installation are relatively inexpensive and can be installed during excavation for the trail, which will provide maximum flexibility for future lighting.

Lighting conduit would typically be connected to the existing grid, but solar is also an option here. Individual solar lights may be overly impacted by tree cover and dark days, but a free-standing solar power source could help power the trail lights.

2.6.2 Other

The north end of the trail, which will require clearing for construction access and bridge erection, is an excellent location for a small park. Benches or picnic tables could provide a place for visitors to rest and view the stream, while interpretive panels could educate visitors on the history of the trestle. This would also be an excellent opportunity to display sculptures or other art. Trails that have already been worn into the ground from frequent visitors could be turned into spur trails to the stream (note that these would not be ADA-compliant).

2.7 ALTERNATIVES AND COSTS

2.7.1 Construction Costs

The construction costs, excluding lighting, landscaping, or additional aesthetic treatments, for each of the alignment and truss options are noted in Table 1. Table 2 provides those optional elements excluded from Table 1. The total construction costs will depend on what bridge type (Line items 3a to 3c) from Table 1 and additional trail elements (Table 2) the City chooses to proceed with.

Table 1 – Construction Costs for Trail Approach and Bridge

Line Item	Option 1 Cost (2022)	Option 2 Cost (2022)
1. Trail Approach Items	$268,920	$232,035
2. Bridge Items (NOT including truss bridge)	$642,250	$179,400
3a. H-Section Galvanized Truss Bridge	$840,000	$807,500
3b. H-Section Galvanized & Painted Truss Bridge	$880,000	$850,000
3c. Modular Steel Galvanized Truss Bridge	$595,000	$565,000
4. Mobilization	$150,617	$97,644
TOTAL TRAIL APPROACH & BRIDGE OPTIONS (includes 15% contingency)		
TOTAL Trail & Bridge: H-Section Galvanized Truss (1 + 2 + 3a + 4)	**$2,188,000**	**$1,515,000**
TOTAL Trail & Bridge: H-Section Galvanized & Painted Truss (1 + 2 + 3b + 4)	**$2,234,000**	**$1,563,000**
TOTAL Trail & Bridge: Modular Steel Galvanized Truss (1 + 2 + 3c + 4)	**$1,906,000**	**$1,236,000**

Table 2 – Construction Costs for Optional Trail Elements

Item	Option 1 Cost (2022)	Option 2 Cost (2022)
5. Landscaping	$4,500	$4,500
6. Conduit/junction boxes for lighting	$48,000	$48,000
7. Ornamental light standards and foundations	$110,000	$110,000
8. Cobra head light for south end of trail	$2,500	$2,500
9. Bridge conduit and lighting	$73,000	$59,000
10. Interpretive panel	$2,500	$2,500

2.7.2 Program Costs

The total cost of the trail, including construction, engineering, permitting, construction services, and right-of-way is challenging to estimate due to the number of variables at this stage. Stantec's estimates of probable costs for engineering and permitting, construction services, and right-of-way are below.

Table 3 – Program Costs		
Item	**Option 1 Cost (2022)**	**Option 2 Cost (2022)**
Range of Construction Costs (Table 1 Only)	$1,906,000 - $2,234,000	$1,236,000 - $1,563,000
Engineering & Permitting	$250,000	$150,000
Construction Services (8%)	$153,000-$179,000	$100,000-$125,000
ROW	TBD	TBD
TOTAL PROGRAM COSTS (not including ROW, landscaping, lighting, or other aesthetic treatments)	**$2.3 million to $2.7 million**	**$1.5 million to $1.8 million**

2.8 RECOMMENDED ALTERNATIVE

Table 4 – Recommended Alternatives

Criteria	Option 1	Option 2
Permitting/Environmental Impacts	Portion of south bridge abutment inside the normal high-water line will be wetland impacts. Contains in-water work subject to Section 7 endangered fish species permitting.	Island clearing will be wetland impacts. Contains very little in-water work subject to Section 7 endangered fish species permitting.
ROW	Mainly uses the existing State ROW with some permanent impacts at the north and south ends of the trail. Temporary easements needed for construction.	Permanent impacts at the south end of the bridge and the south end of the trail. Temporary easements needed for construction.
Constructability	Two bridge spans will be lighter and easier to install than Option 2.	Single long-span bridge is heavy and will require some innovation to install.
Substructure type	Two abutments, one pier, and an extensive retaining wall at the south side.	Two abutments.
Bridge Hydraulics	Barely outside of the FEMA-regulated floodway. South abutment and retaining walls are entirely on the 100-year floodplain. Will require extensive analysis to show that proposed solution is not a decrease in hydraulic opening from existing bridge.	Outside of the FEMA-regulated floodway and mostly outside of the 100-year floodplain except for a small corner of the south abutment. By inspection will not decrease hydraulic opening compared to existing bridge.
Design Phase Considerations	Additional hydraulic analysis. Additional boring needed. Pier design.	Design of abutments and wingwalls is straightforward.

Stantec recommends Option 2 with a galvanized coating for the selected bridge as the most feasible option for a stream crossing in this location because:

- The construction cost is significantly less than Option 1 and given the limited funding available through the City's own means and MaineDOT. Seeking funding through grants is very competitive and the difference in the two options is approximately $800,000.
- There are fewer wetland impacts.
- The configuration of the bridge results in a net positive increase to the hydraulic opening as compared to the existing bridge.
- The in-water work is minimized as much as possible due to lack of a pier, which will simplify permitting, especially for Section 7 endangered species.
- The design phase will not require additional borings, extensive hydraulics analysis, or pier design.
- The anticipated ROW impacts and associated costs are likely manageable.
- The galvanized coating will minimize maintenance costs and maximize design life and safety.

APPENDICES

Appendix A RENDERINGS

Existing Train Trestle

Option 1
H-Section Truss - Galvanized

Option 1
H-Section Truss - Galvanized and Painted

Option 1
Modular Truss - Galvanized

Option 2
H-Section Truss - Galvanized

Option 2
H-Section Truss - Galvanized and Painted

Option 2
Modular Truss - Galvanized

Appendix B PRELIMINARY ESTIMATE

Stantec 2211 Congress Street Suite 380 Portland, ME 04102	Quantity Summary City of Gardiner WIN 13344.00 Cobbossee Trail Extension Feasibility Study		Initials	Date
		Calc'd By:	THG,LSF	1/21/2022
		Checked By:	PLP, KLW	1/25/2022
		Revised By:	LSF	2/15/2022
		Checked By:	THG,DDT	2/22/2022

Item No.	Item Description	Unit	Unit Price	Option 1 Quantity	Option 2 Quantity	Option 1 $ (2022)	Option 2 $ (2022)
201.11	CLEARING	AC	$25,000.00	0.7	0.8	$17,500.00	$20,000.00
203.20	COMMON EXCAVATION	CY	$35.00	400	420	$14,000.00	$14,700.00
203.24	COMMON BORROW	CY	$30.00	1150	1350	$34,500.00	$40,500.00
203.25	GRANULAR BORROW	CY	$50.00	800	250	$40,000.00	$12,500.00
206.082	STRUCTURAL EARTH EXCAVATION - MAJOR STRUCTURES	CY	$50.00	700	300	$35,000.00	$15,000.00
304.10	AGGR SUBB COURSE - GRAVEL	CY	$50.00	330	390	$16,500.00	$19,500.00
403.209	HOT MIX ASPHALT 9.5 MM (INCIDENTALS)	T	$220.00	130	140	$28,600.00	$30,800.00
409.15	BITUMINOUS TACK COAT - APPLIED	G	$25.00	32	35	$800.00	$875.00
502.219	STRUCTURAL CONCRETE, ABUT & RET WALLS	LS/CY	$1,200.00	275	105	$330,000.00	$126,000.00
502.239	STRUCTURAL CONCRETE PIERS	LS/CY	$1,200.00	65	0	$78,000.00	$0.00
504.5101	MISC. BRIDGE REPAIRS	LS	$10,000.00	1	1	$10,000.00	$10,000.00
507.0842	ORNAMENTAL PEDESTRIAN RAILING	LS/LF	$250.00	167	42	$41,750.00	$10,500.00
511.07	COFFERDAM:	LS	Varies	1	1	$192,500.00	$42,900.00
530.01	STEEL PEDESTRIAN BRIDGE	LS	Varies	1	1	-	-
			H-SECTION GALVANIZED			$840,000.00	$807,500.00
			H-SECTION GALVANIZED AND PAINTED			$880,000.00	$850,000.00
			MODULAR STEEL GALVANIZED			$595,000.00	$565,000.00
607.22	CEDAR RAIL FENCE	LF	$60.00	530	580	$31,800.00	$34,800.00
610.16	HEAVY RIPRAP	CY	$100.00	320	240	$32,000.00	$24,000.00
615.07	LOAM	CY	$80.00	87	99	$6,960.00	$7,920.00
618.14	SEEDING METHOD NUMBER 2	UN	$50.00	14	16	$700.00	$800.00
619.1201	MULCH	UN	$40.00	14	16	$560.00	$640.00
659.10	MOBILIZATION	LS	Varies	1	1	$150,617.00	$97,643.50

Total Opinion of Probable Construction Cost in 2022 Dollars

	Option 1	Option 2
Subtotal - Trail Items	$268,920.00	$232,035.00
Subtotal - Bridge Items (exclu. Bridge)	$642,250.00	$179,400.00
H-Section Galvanized Bridge	**$840,000.00**	**$807,500.00**
H-Section Galvanized & Painted Bridge	**$880,000.00**	**$850,000.00**
Modular Steel Galvanized Bridge	**$595,000.00**	**$565,000.00**
Mobilization	**$150,617.00**	**$97,643.50**
TOTAL* H-SECTION GALVANIZED BRIDGE & TRAIL	**$2,188,000.00**	**$2,089,000.00**
TOTAL* H-SECTION GALVANIZED & PAINTED BRIDGE & TRAIL	**$2,234,000.00**	**$1,451,000.00**
TOTAL* MODULAR STEEL GALVANIZED BRIDGE & TRAIL	**$1,906,000.00**	**$1,810,000.00**

*Includes 15% contingency. Does not include lighting, landscaping, or additional aesthetic treatments

ADDITIONAL TRAIL ELEMENTS

Item No.	Item Description	Unit	Unit Price	Option 1 Quantity	Option 2 Quantity	Option 1 $ (2022)	Option 2 $ (2022)
621	LANDSCAPING	LS	$ 4,500.00	1	1	$4,500.00	$4,500.00
634.160	HIGHWAY LIGHTING	LS	Varies	1	1	-	-
		CONDUIT/JUNCTION BOXES FOR TRAIL				$48,000.00	$48,000.00
		ORNAMENTAL LIGHTS AND FOUNDATIONS				$110,000.00	$110,000.00
		COBRA HEAD LIGHT				$2,500.00	$2,500.00
		BRIDGE CONDUIT AND LIGHTING				$73,000.00	$59,000.00
645.51	SPECIAL SIGNING (INTERPRETIVE PANEL)	LS	$ 2,500.00	1	1	$2,500.00	$2,500.00

Appendix C PLANS

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

GARDINER
KENNEBEC COUNTY
COBBOSSEE STREAM CROSSING
13344.10
PROJECT LENGTH: 0.28 MILES
TRAIL EXTENSION PROJECT

PLAN LEGEND

Town, County, State
Property Lines
R/W Lines-Existing
R/W Lines-Proposed

Culvert-Existing
Culvert Proposed

Curbing	Existing	Proposed
Type 1		
Type 3		
Type 5		

Outline of Bodies of Water
Ledge
Buildings

Trees — Conifer — Deciduous
Tree Line
Clearing Limit Line — CLL — CLL

Centerline-Existing
Centerline-Proposed — 10+00
Travelway-Existing
Travelway-Proposed
Railroad

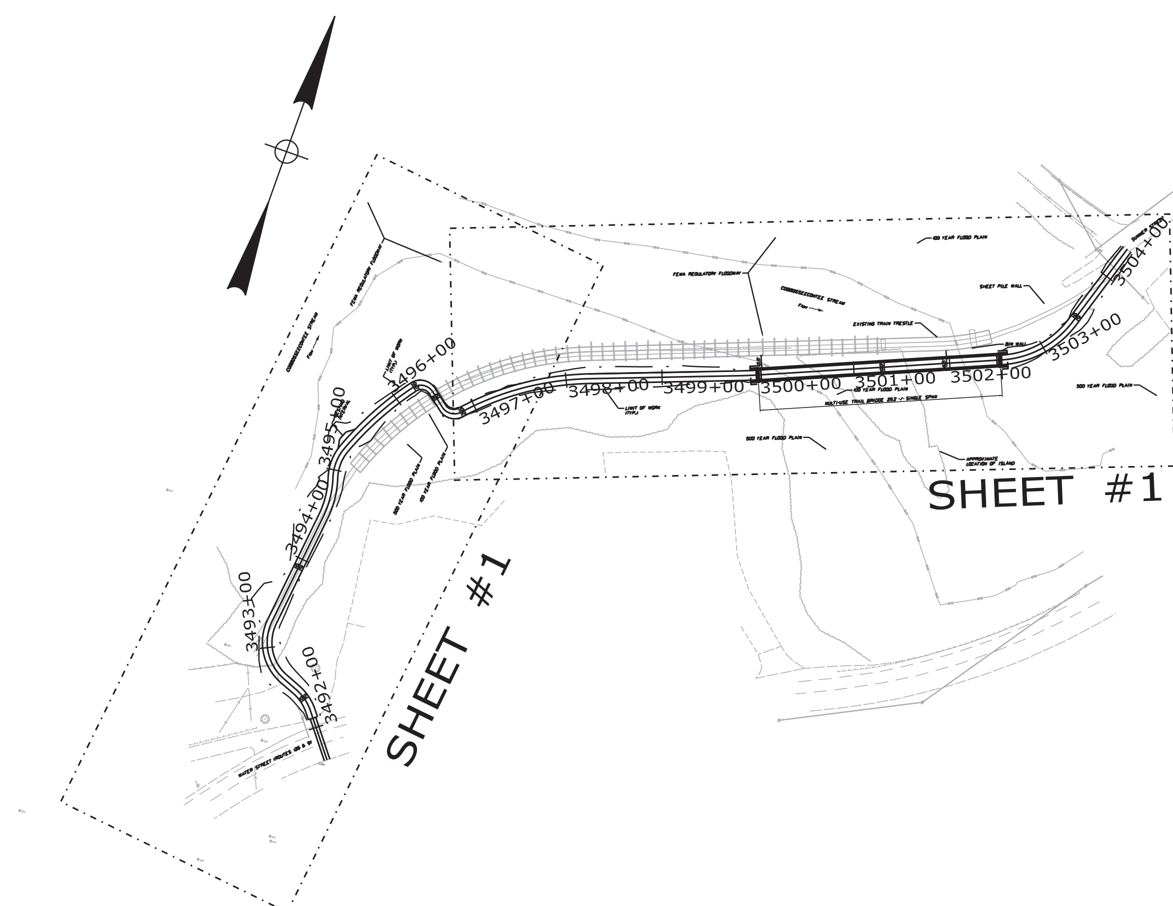
Catch Basins — Existing — Proposed
Manholes — Existing — Proposed
Proposed Underdrain
Proposed Ditch
Existing Ditch

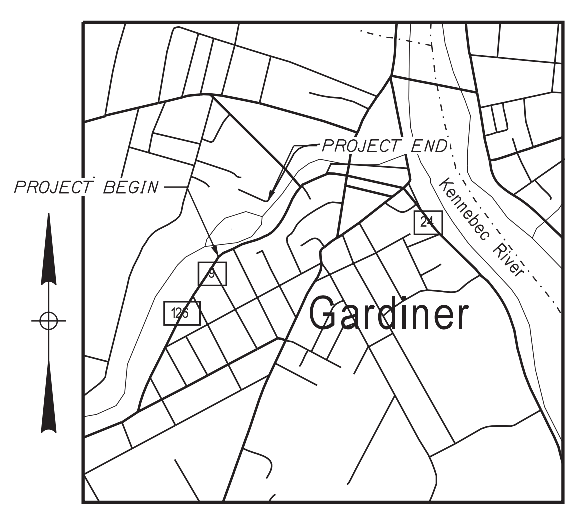
Utlity Poles — Existing — Proposed
Fire Hydrants — Existing — Proposed
Existing Water Line
Existing San. Sewer
Existing San. Sewer Manhole

Guardrail-Existing
Guardrail-Proposed
Guardrail-Cable, Other

INDEX OF SHEETS

TRAIL FEASIBILITY STUDY
CONCEPTUAL PLANS
FEBRUARY 25, 2022





LOCATION MAP

0.25 0 0.25 0.5
Scale in Miles

Stantec

PROJECT LOCATION:	BEGINNING AT END OF SUMMER STREET EXTENSION AND EXTENDING SOUTH CROSSING COBBOSSEECONTEE STREAM AND TERMINATING AT CROSSING ALONG WATER STREET.
PROGRAM AREA:	CITY OF GARDINER
OUTLINE OF WORK:	TRAIL CONSTRUCTION INCLUDING BRIDGE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

APPROVED	DATE
COMMISSIONER:	
CHIEF ENGINEER:	

PROJECT INFORMATION

		SIGNATURE	P.E. NUMBER	DATE
PROGRAM	CITY OF GARDINER			
PROJECT MANAGER	JERRY DOUGLASS			
DESIGNER	LAUREN MEEK			
CONSULTANT	STANTEC			
PROJECT RESIDENT				
CONTRACTOR				
PROJECT COMPLETION DATE				

WIN 13344.10

GARDINER
COBBOSSEE STREAM

TITLE SHEET

SHEET NUMBER
1
OF 23

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Username: lflanders
Date:2/24/2022

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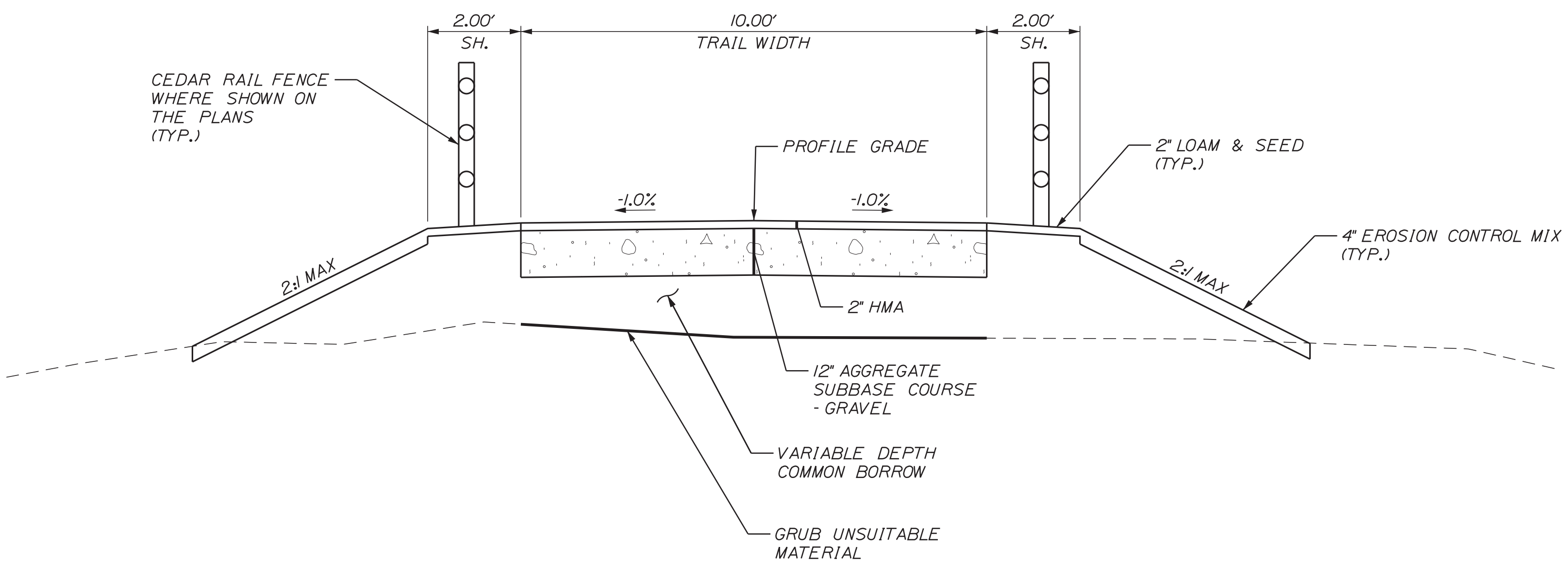

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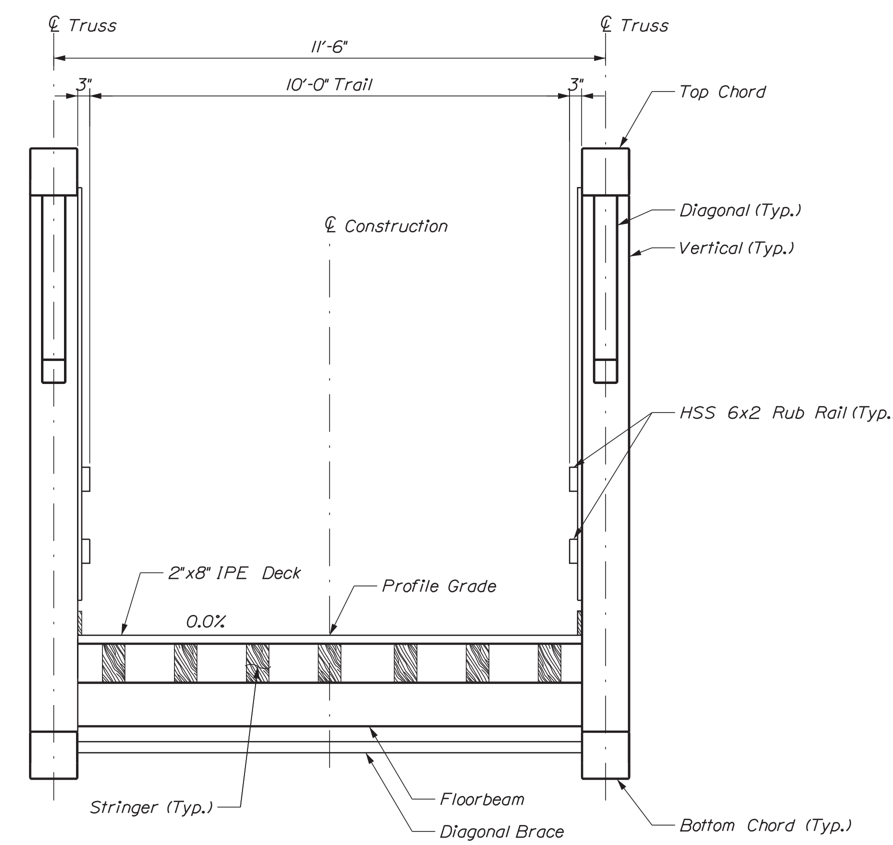

CONCEPTUAL TYPICAL BRIDGE SECTION

Stantec

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

MULTIMODAL PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GARDINER
COBBOSSEE STREAM

TRAIL TYPICAL SECTIONS

SHEET NUMBER
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OF 23

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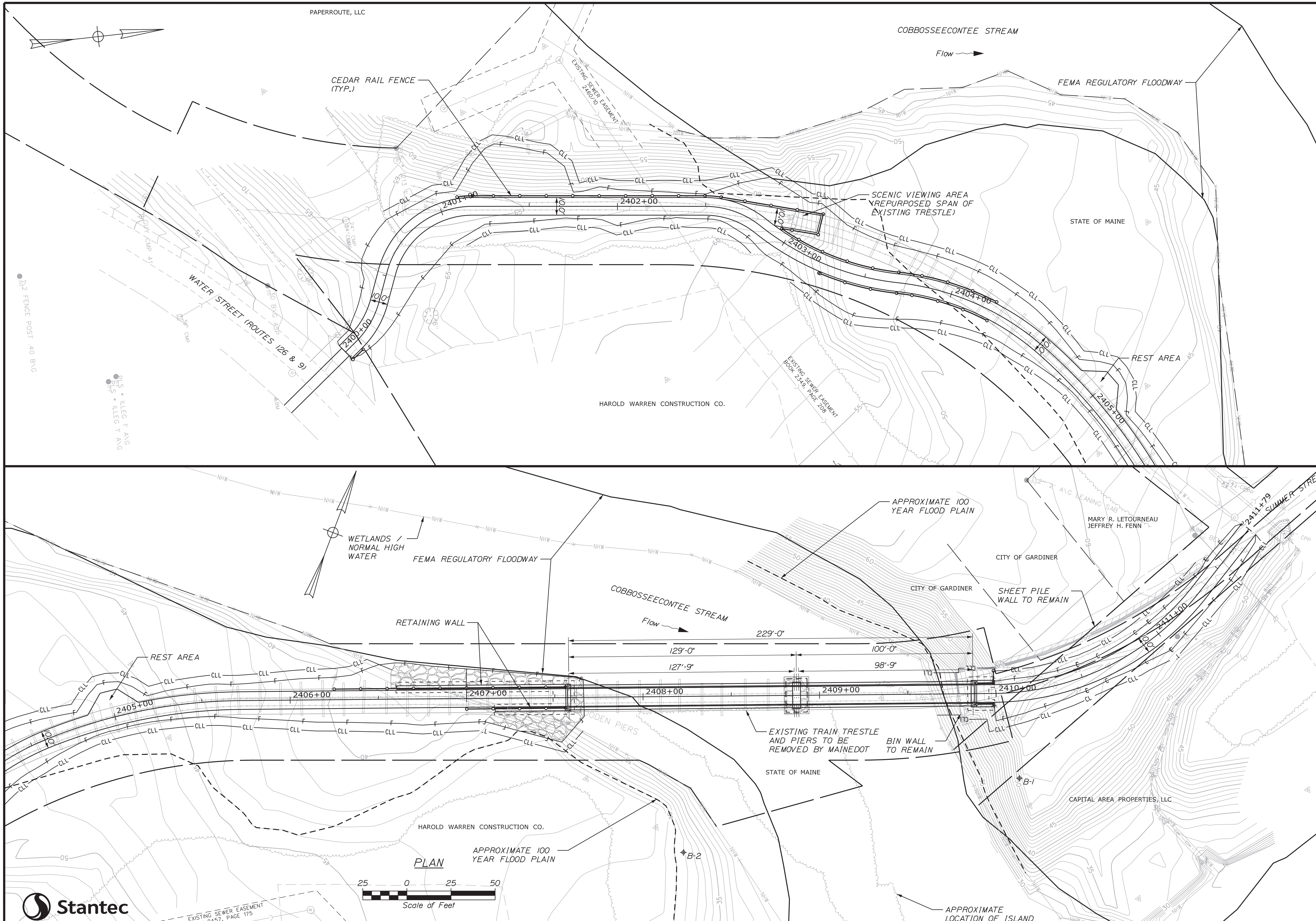
Username: lflanders
Date:2/24/2022



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

MULTIMODAL PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

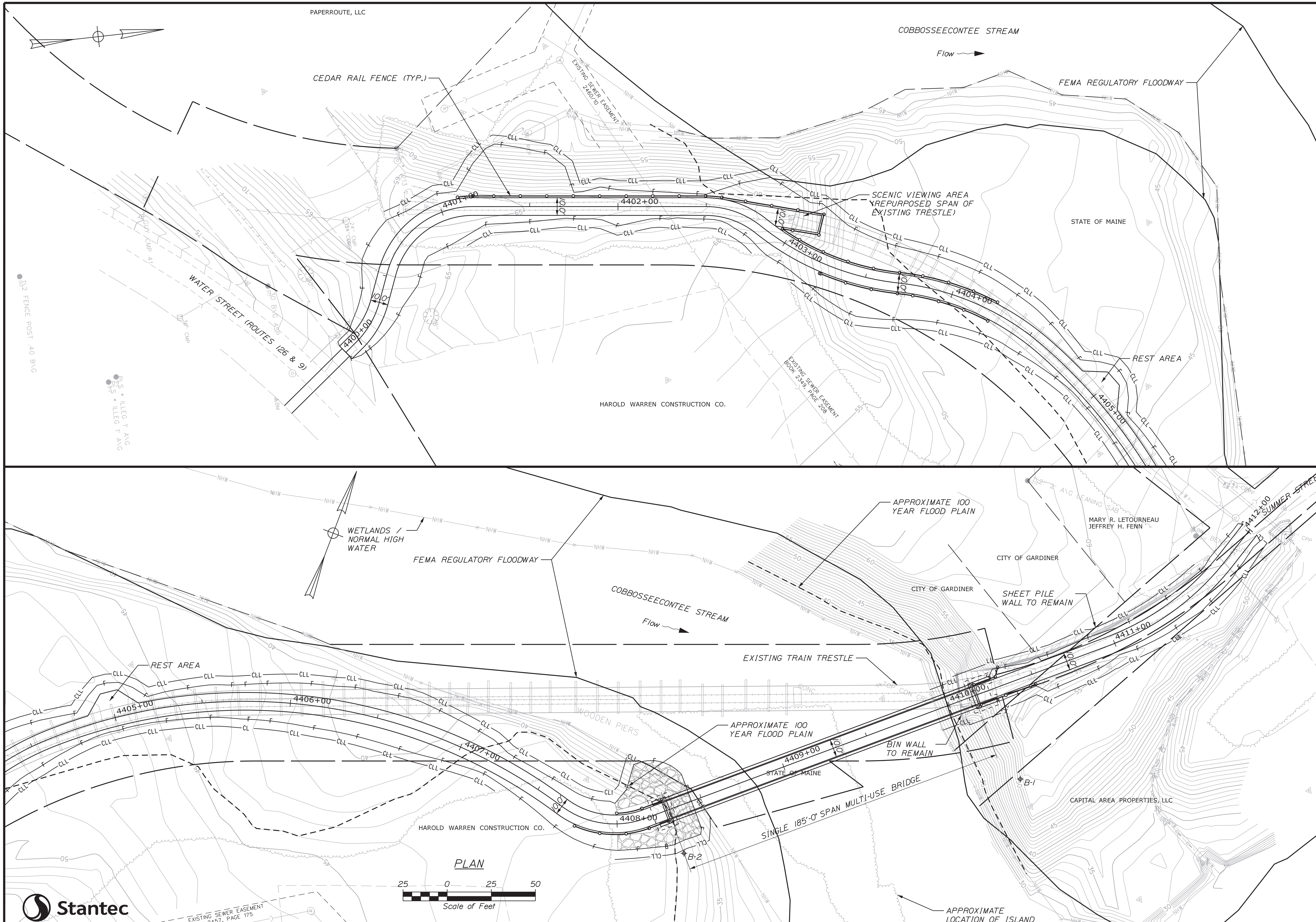
GARDINER
COBBOSSEE STREAM

TRAIL PLAN OPTION 1

SHEET NUMBER

3

OF 23



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Username: lflanders

Date:2/24/2022

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN 13344.10

MULTIMODAL PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GARDINER
COBBOSSEE STREAM

TRAIL PLAN OPTION 2

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OF 23

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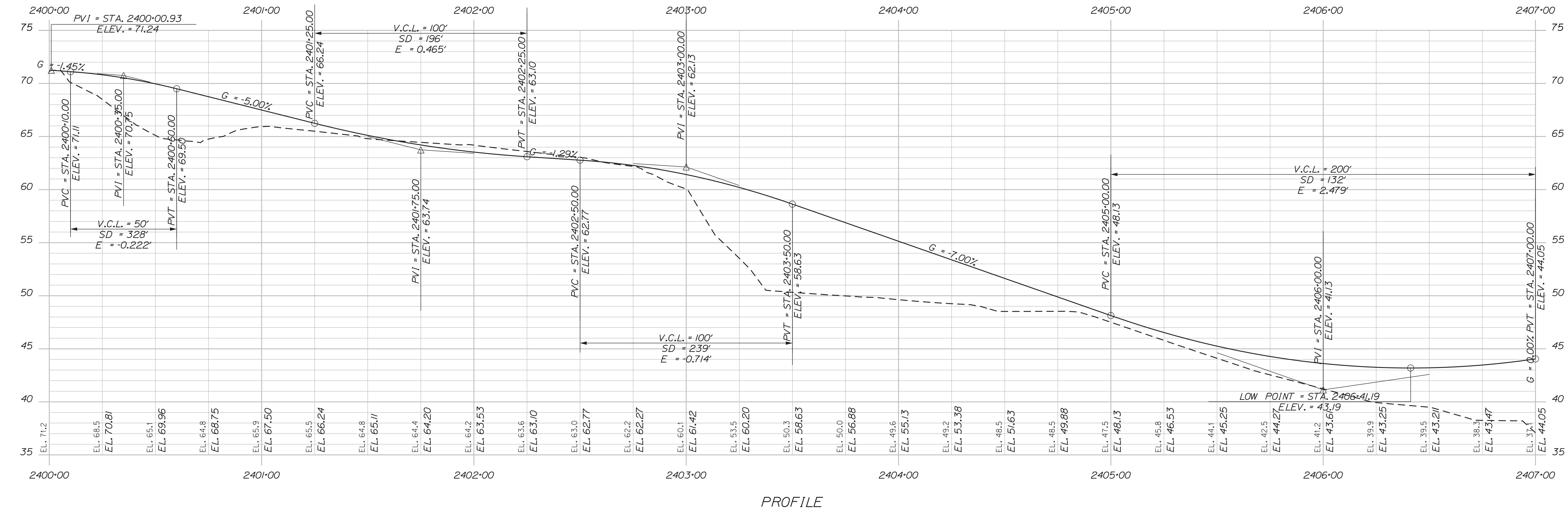

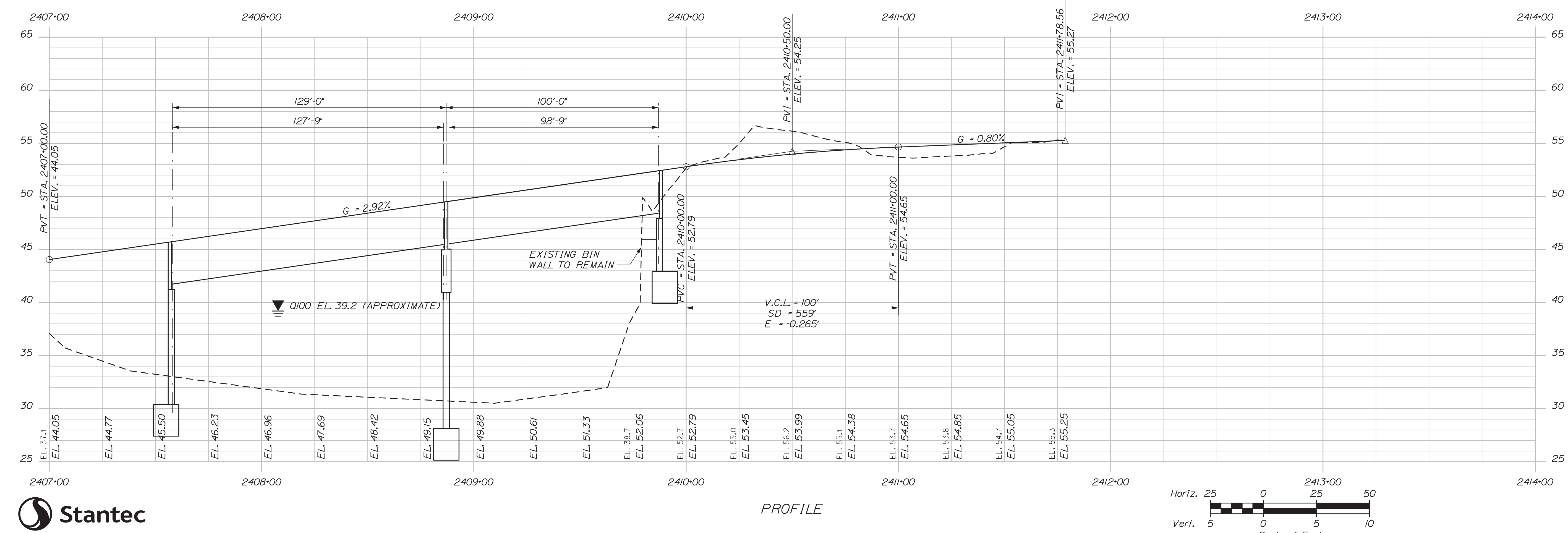

Stantec

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN 13344.10

MULTIMODAL PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GARDINER
COBBOSSEE STREAM

PROFILE OPTION 1

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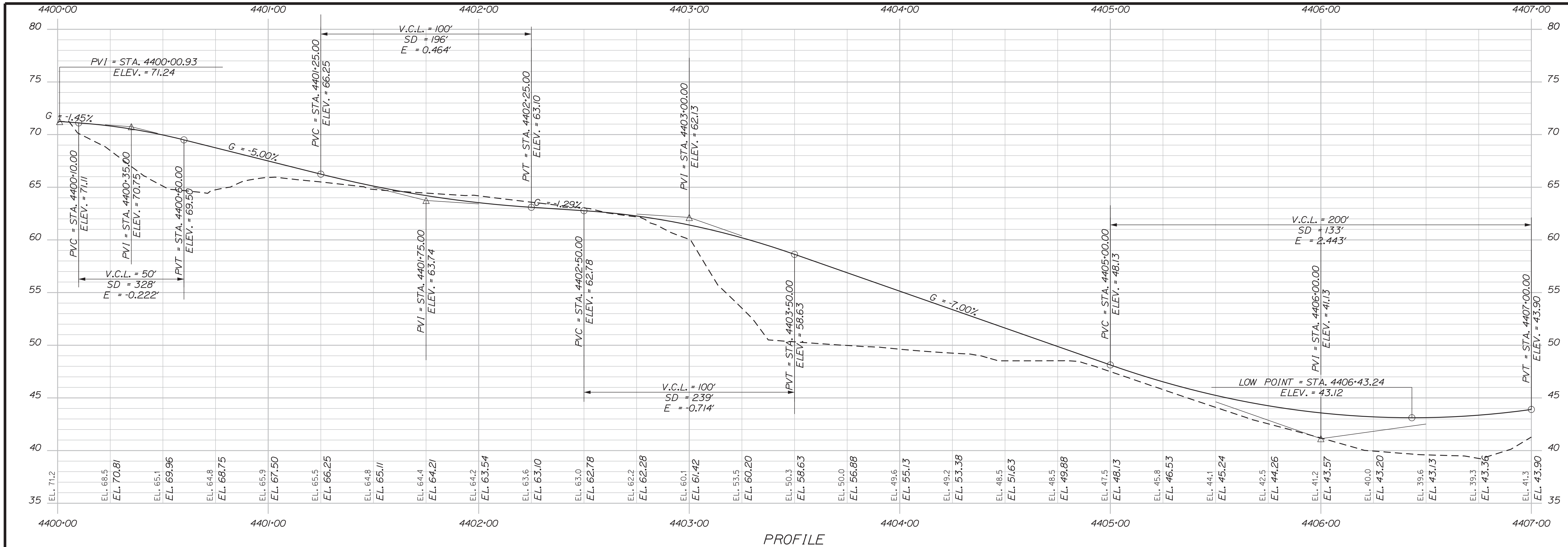

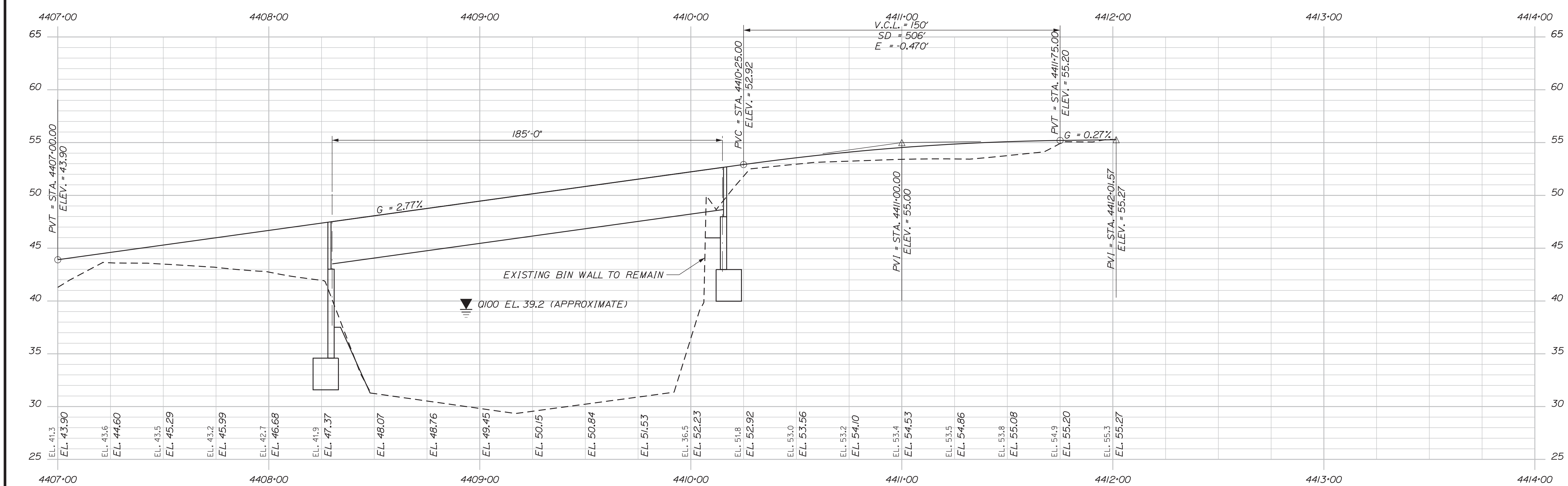

Stantec

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

MULTIMODAL PLANS

SIGNATURE

P.E. NUMBER

DATE

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
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GARDINER
COBBOSSEE STREAM

PROFILE OPTION 2

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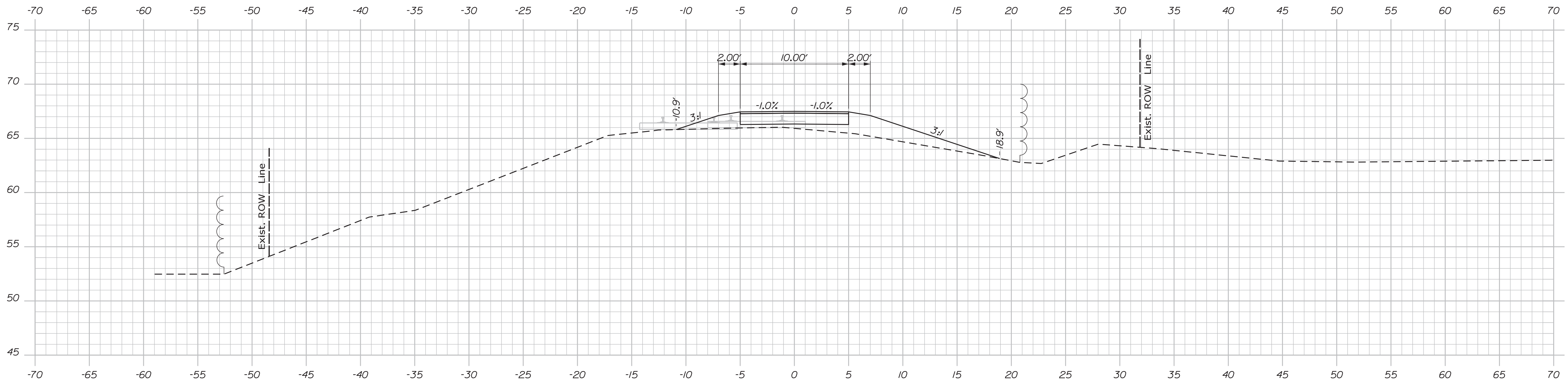

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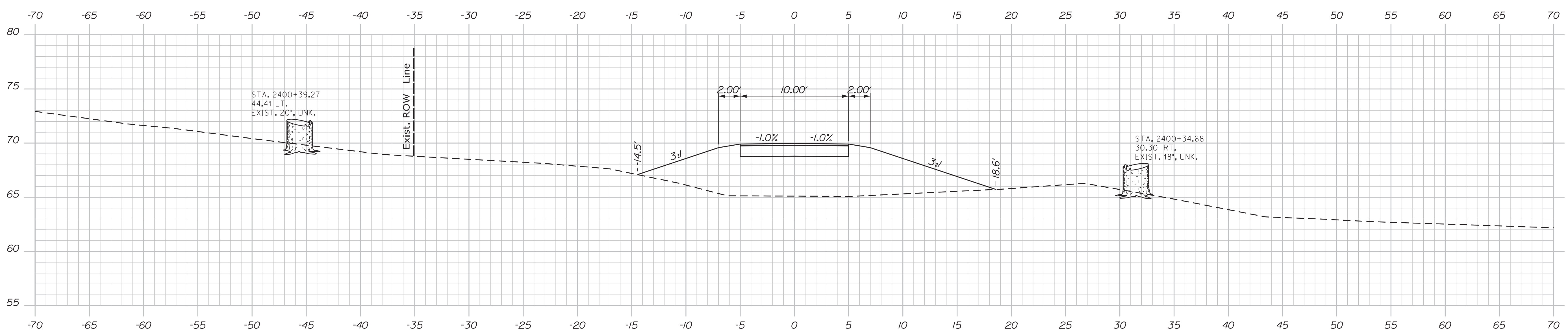

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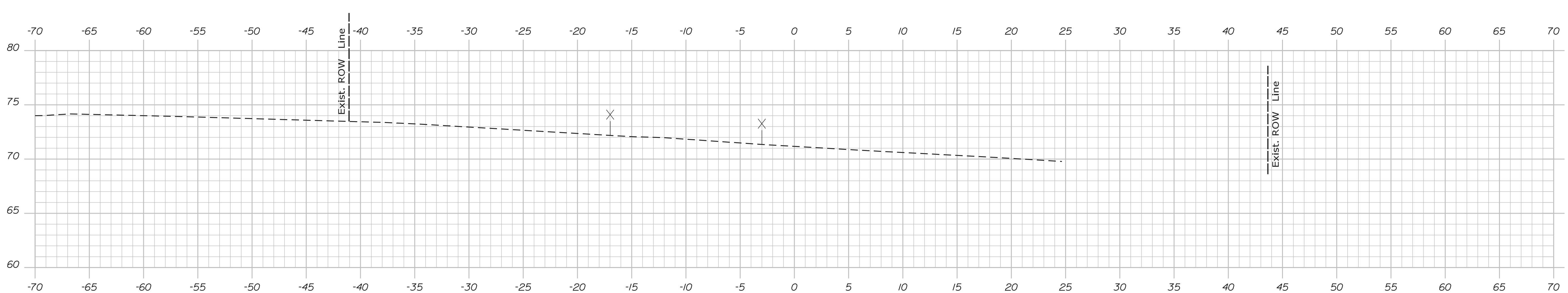

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS / BRIDGE PLANS

SIGNATURE

P.E. NUMBER

DATE

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 1

SHEET NUMBER
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OF 23

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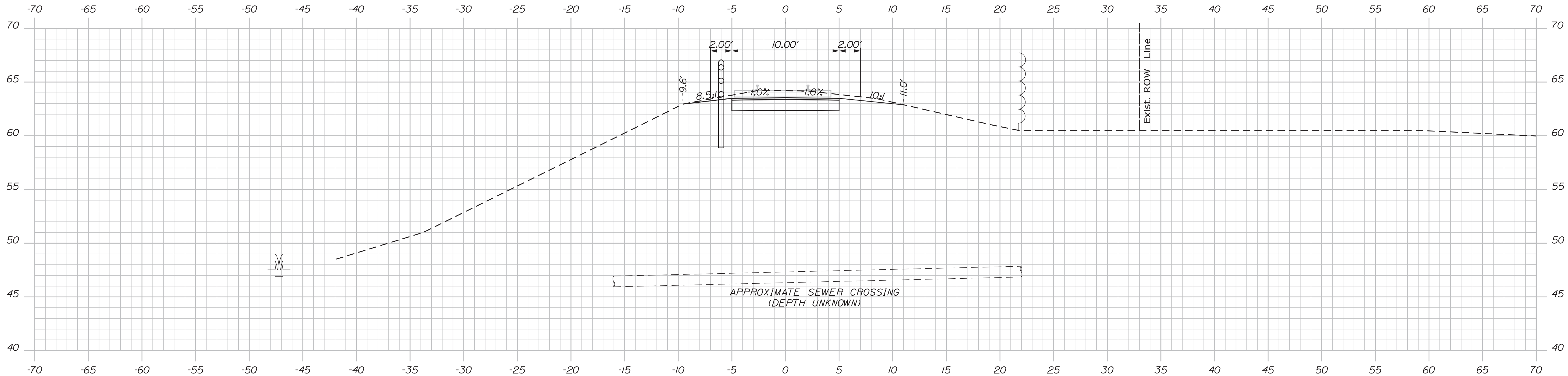

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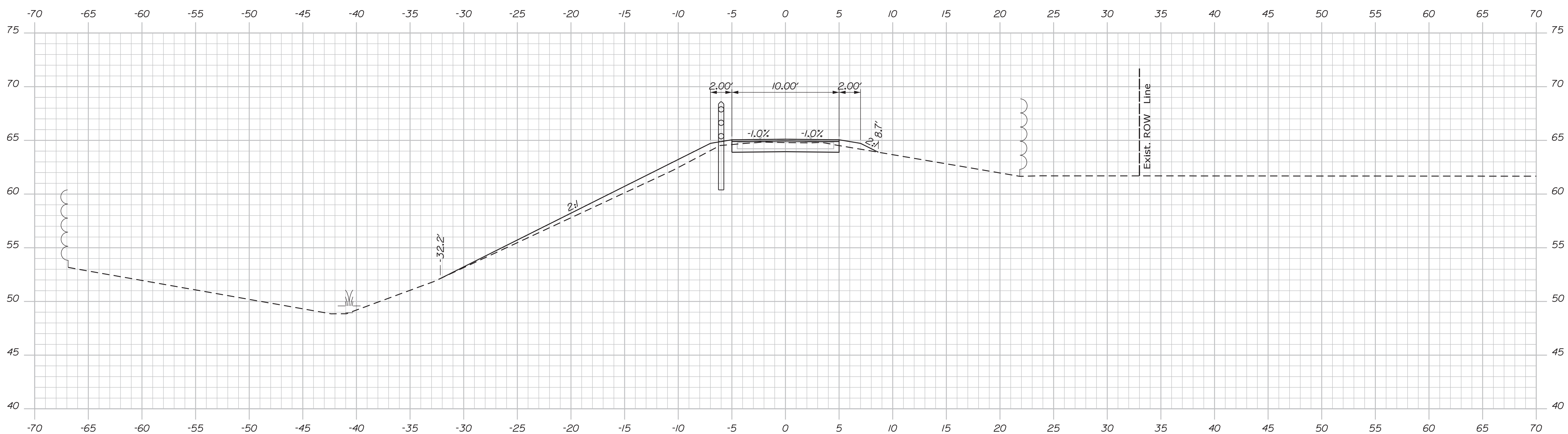

2401+50.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
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REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 1

SHEET NUMBER

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OF 23

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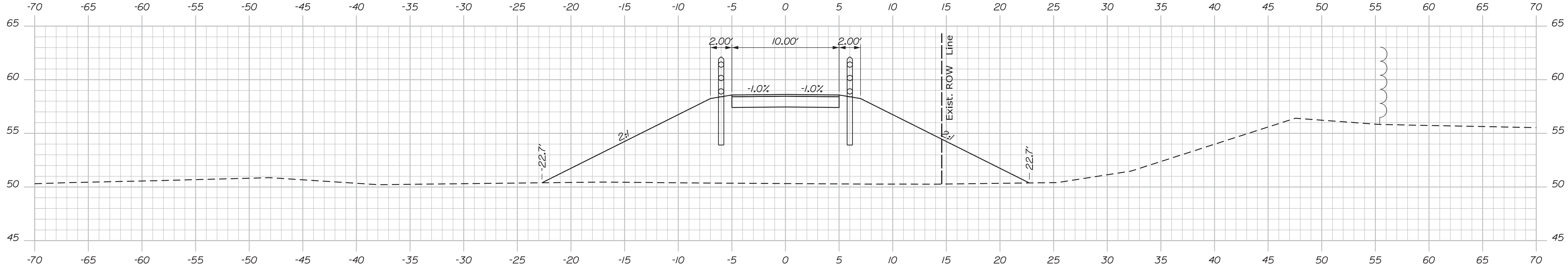

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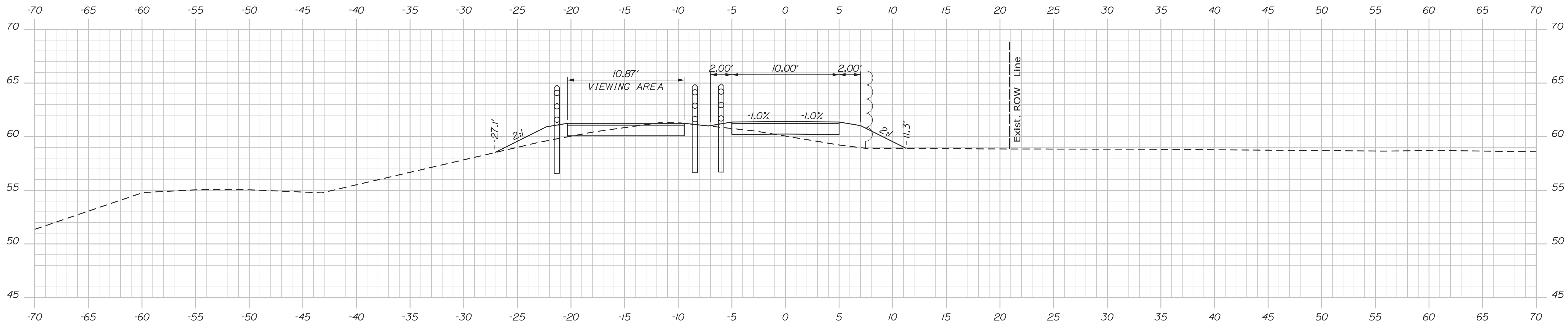

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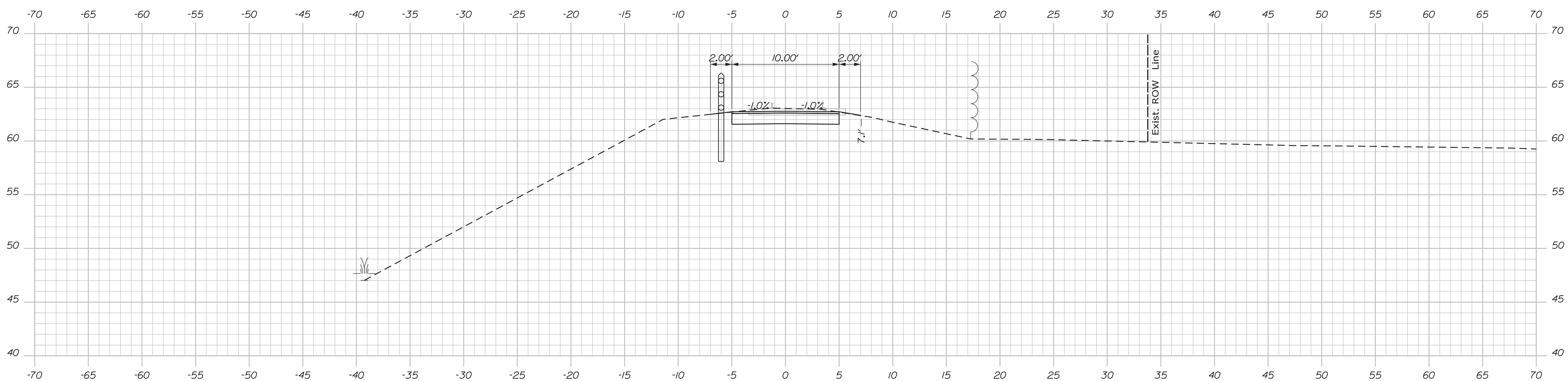

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN 13344.10

BRIDGE PLANS / HIGHWAY PLANS

		BY	DATE	SIGNATURE
PROJ. MANAGER	J. DOUGLASS			
DESIGN-DETAILED	P. POTTLE	PLP	08\2021	
CHECKED-REVIEWED	L. MEEK	LEM		
DESIGN2-DETAILED2	L. FLANDERS	LSF		
DESIGN3-DETAILED3				
REVISIONS 1				P.E. NUMBER
REVISIONS 2				
REVISIONS 3				DATE
REVISIONS 4				
FIELD CHANGES				

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 1

SHEET NUMBER
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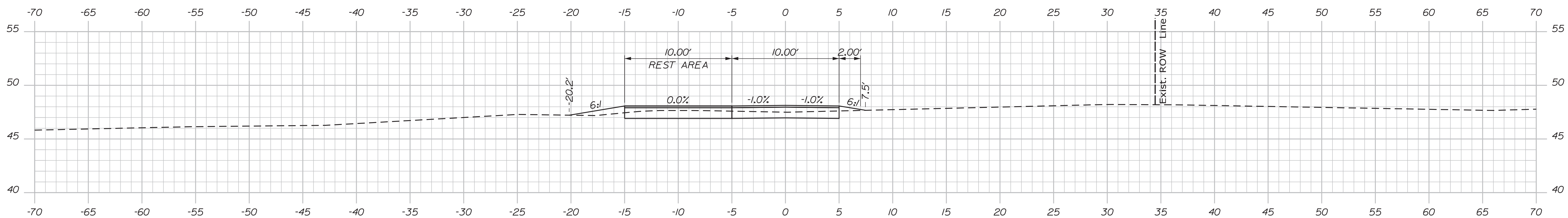
OF 23



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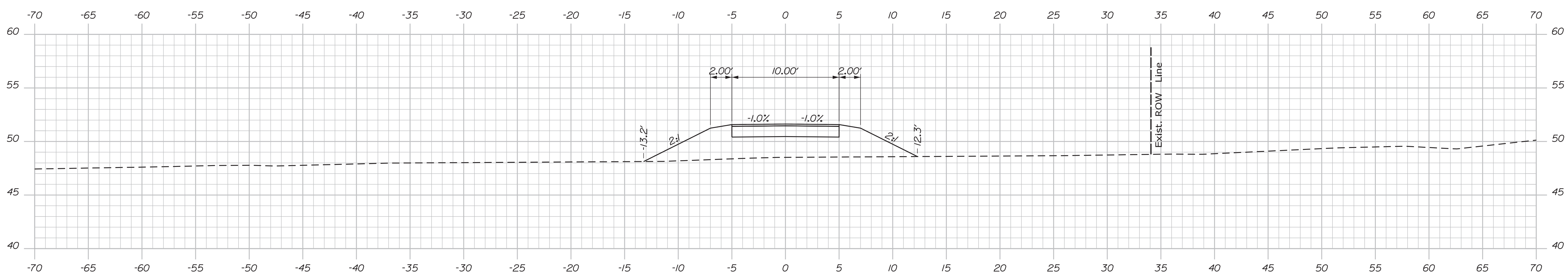

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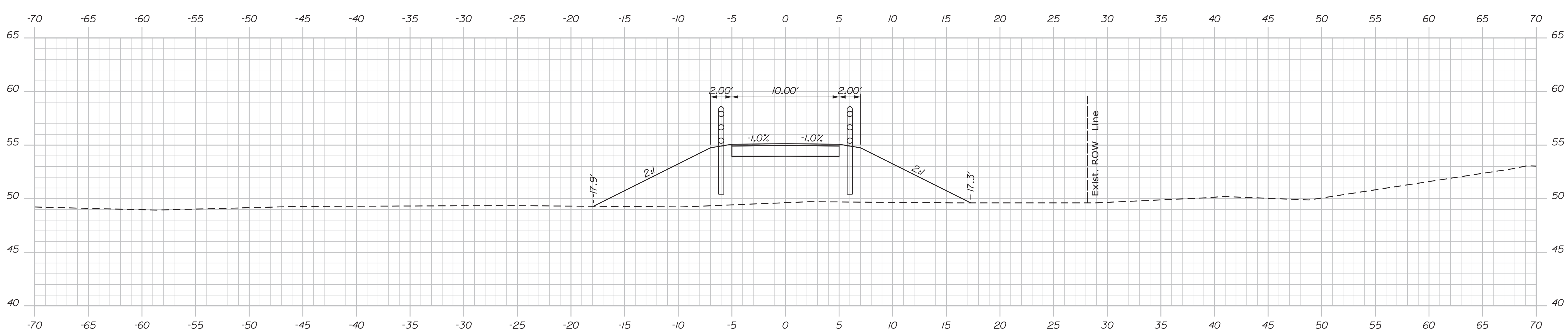

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Division: BRIDGE
Username: lflanders
Date:2/24/2022

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 1

SHEET NUMBER

10

OF 23

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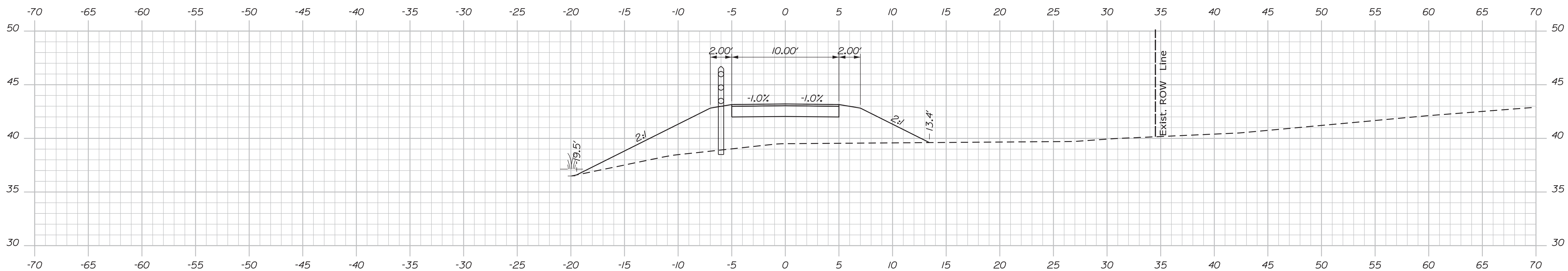

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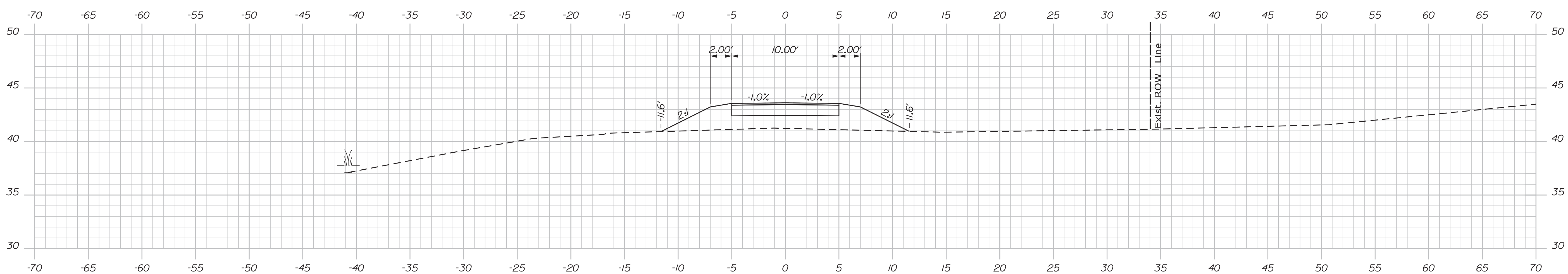

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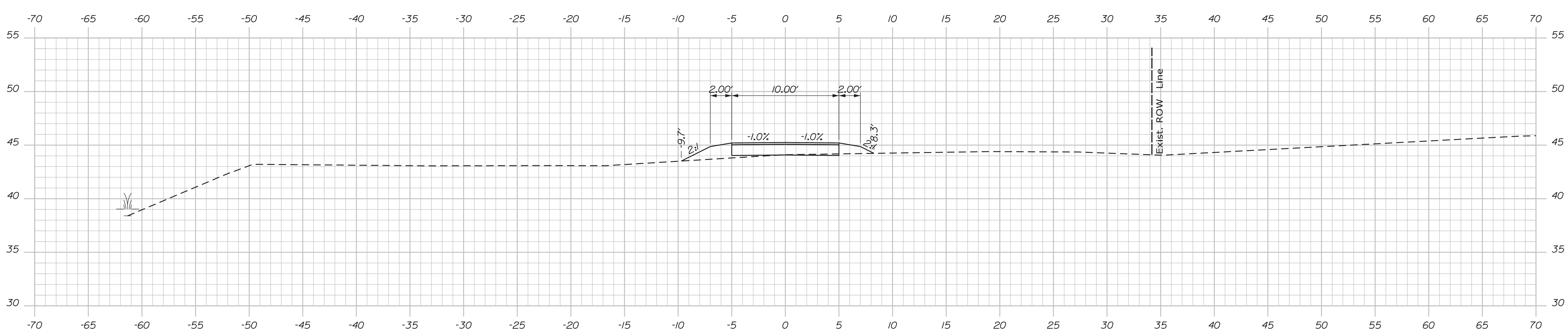

2405+50.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 1

SHEET NUMBER

11

OF 23

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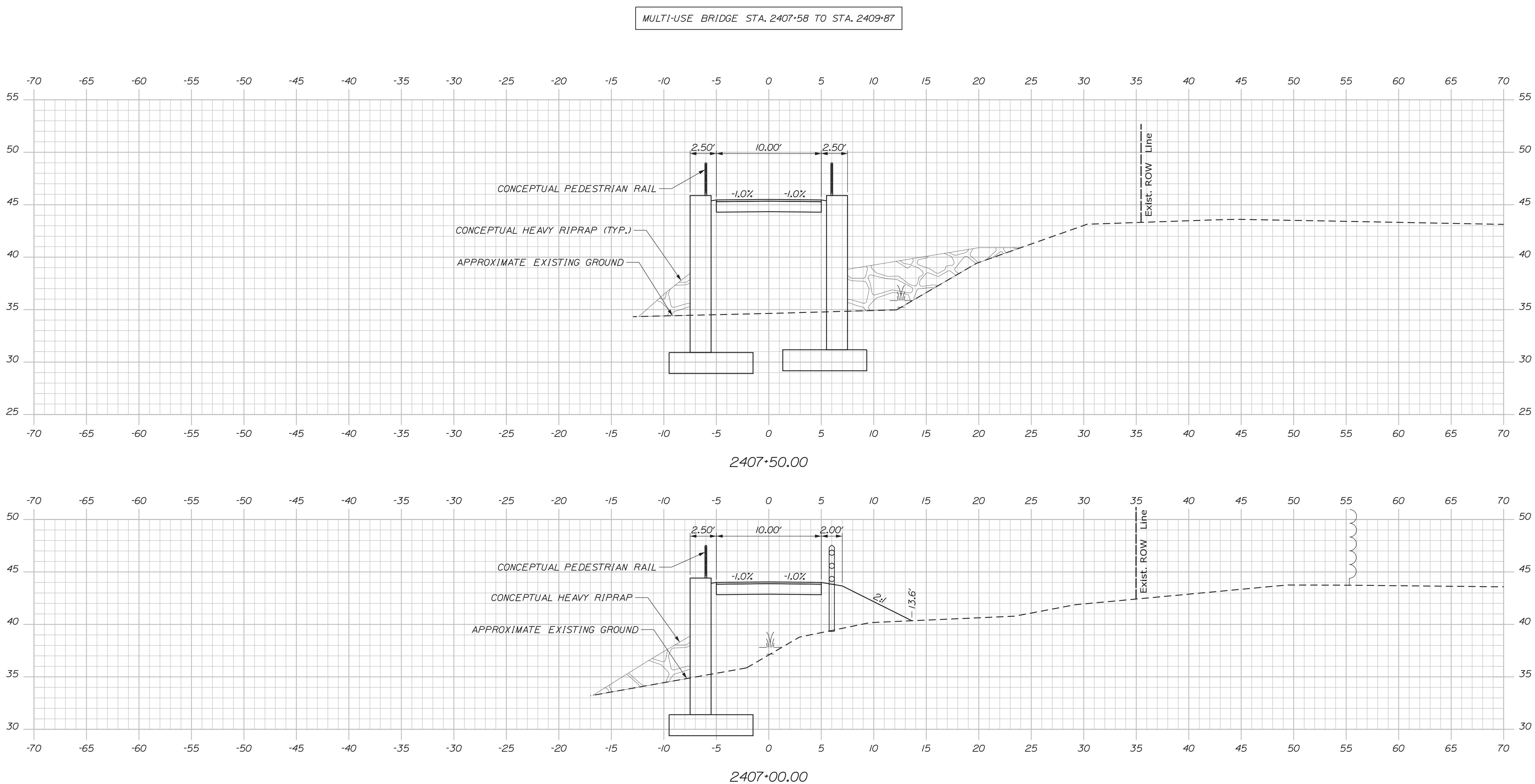

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS / BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 1

SHEET NUMBER
12
OF 23

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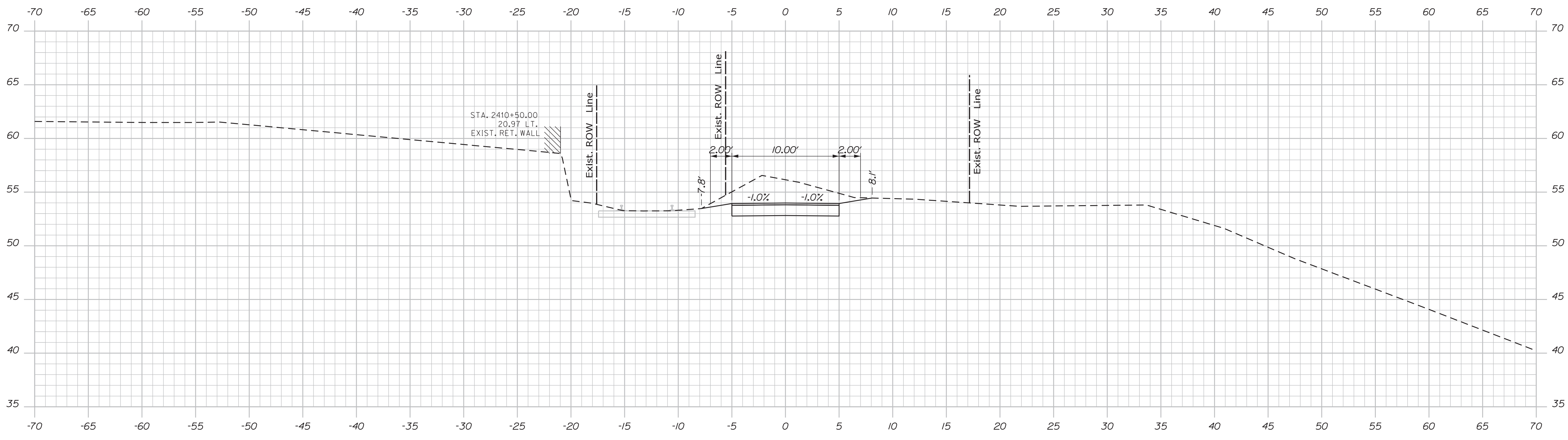

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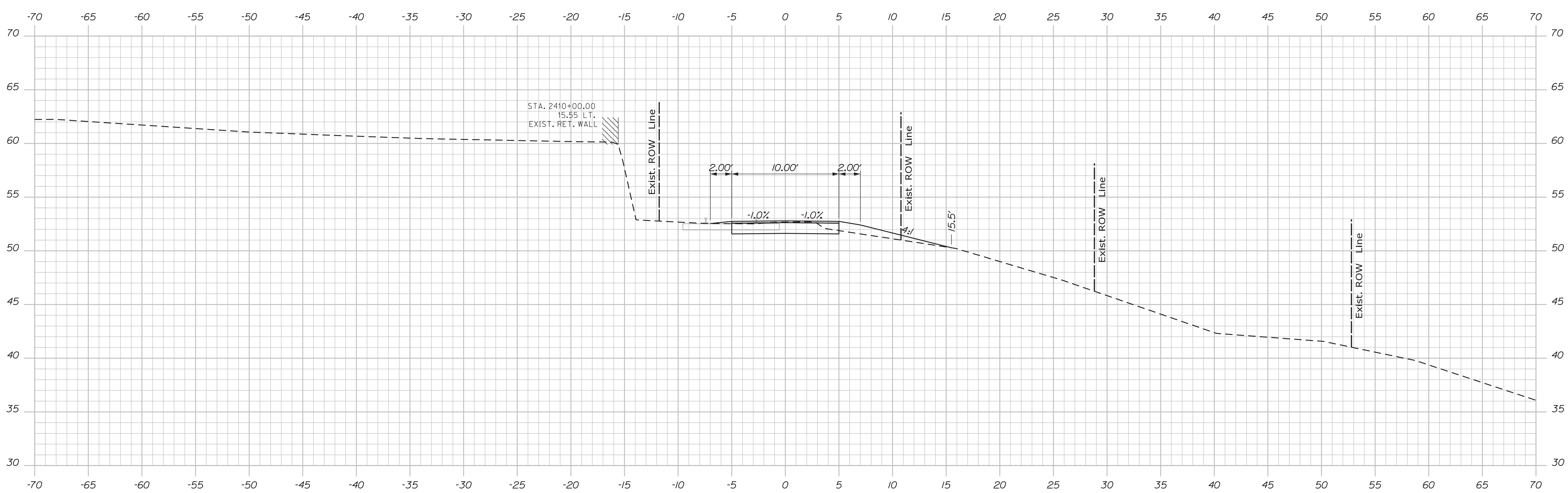

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

BRIDGE PLANS
HIGHWAY PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

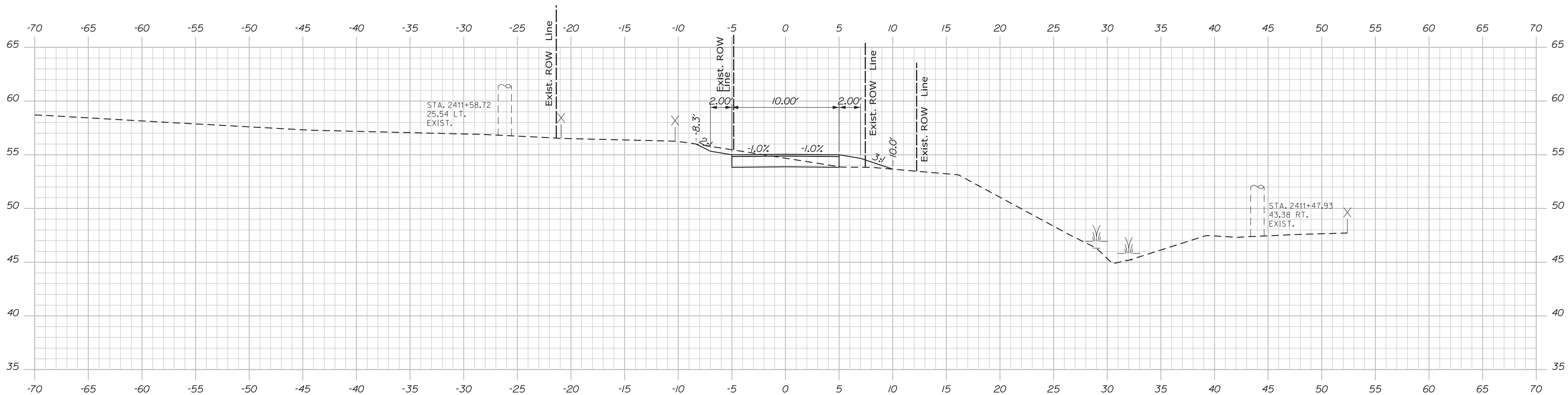
GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 1

SHEET NUMBER
13
OF 23

Sta. 2410+00.00 to Sta. 2410+50.00

Filename: ...\10\multimode\MSTA\xsec1.dgn
Division: BRIDGE
Username: lflanders
Date:2/24/2022



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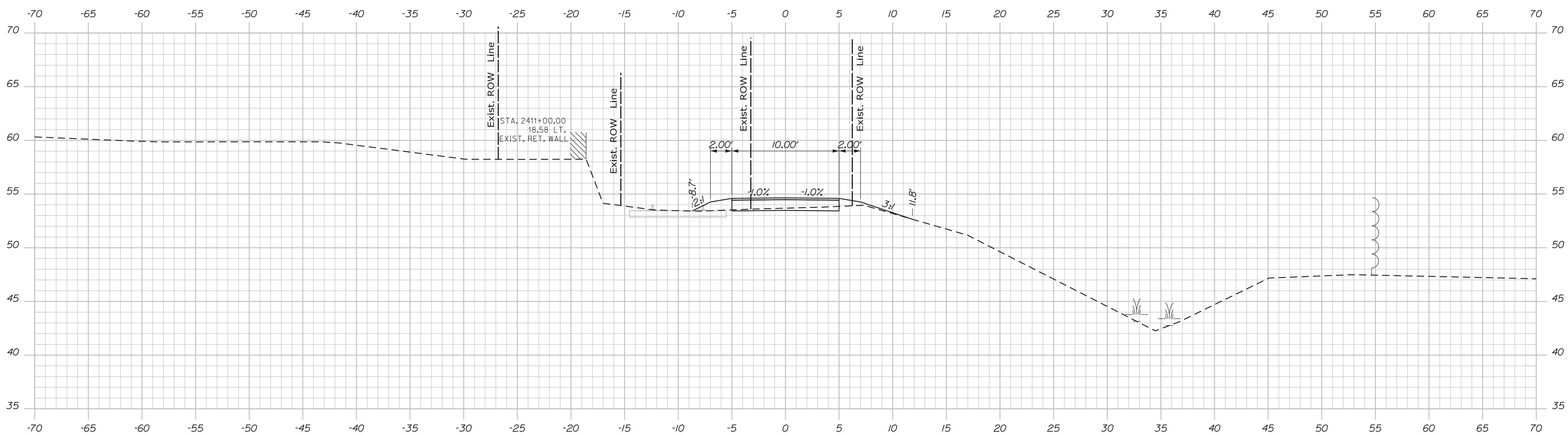

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS

PROJ. MANAGER	J. DOUGLASS		
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DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
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REVISIONS 4			
FIELD CHANGES			

SIGNATURE

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GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 1

SHEET NUMBER
14
OF 23

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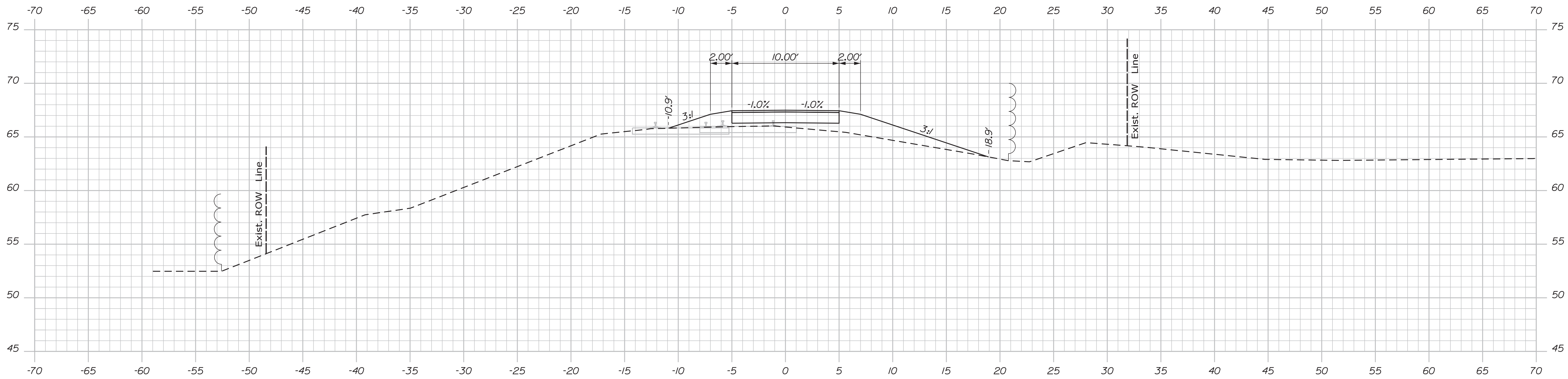

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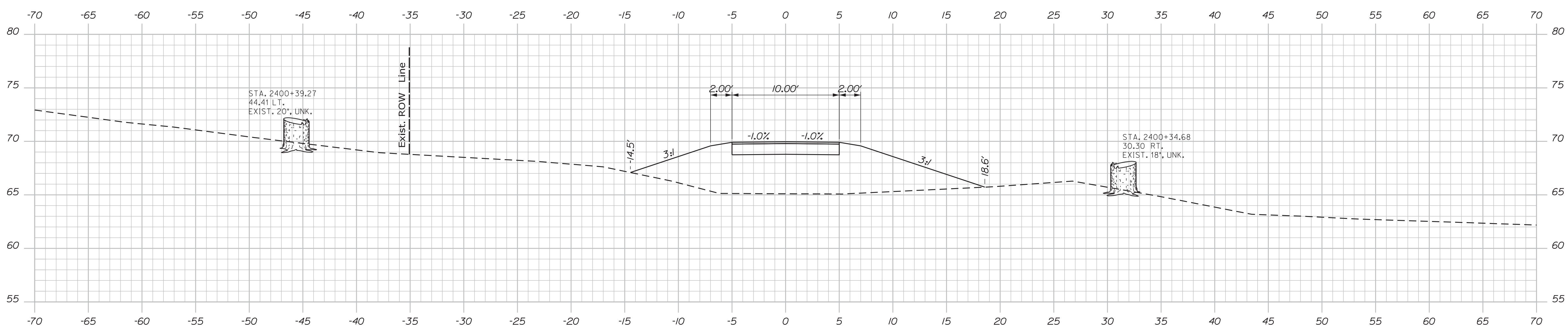

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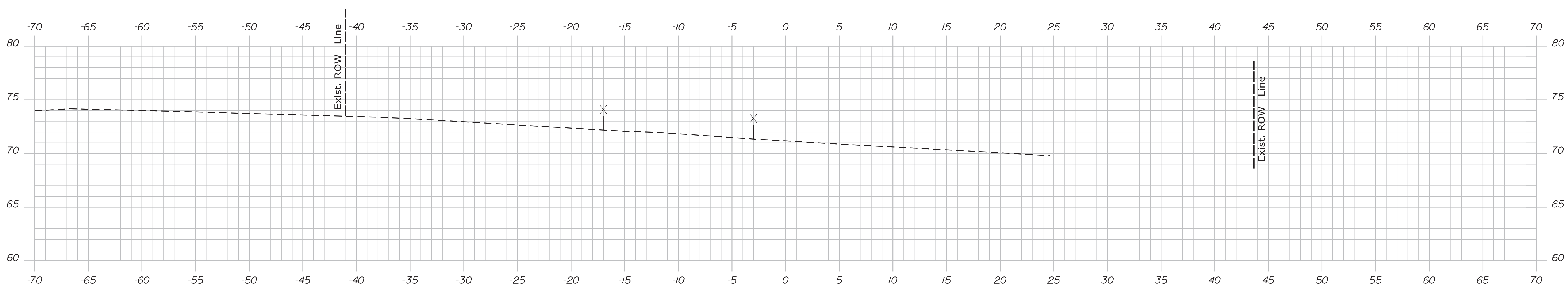

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

BRIDGE PLANS
HIGHWAY PLANS

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REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

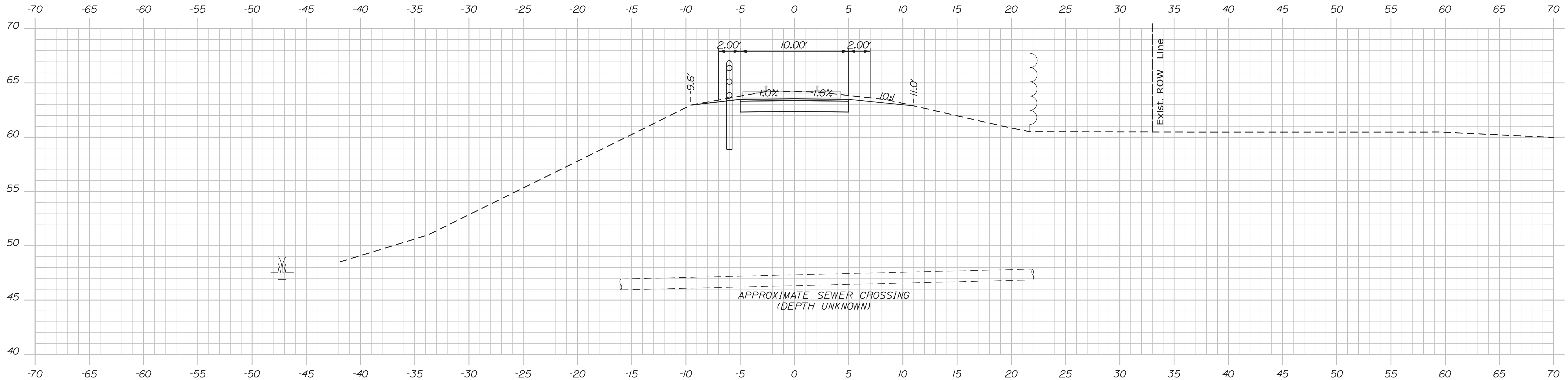
GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 2

SHEET NUMBER

15

OF 23



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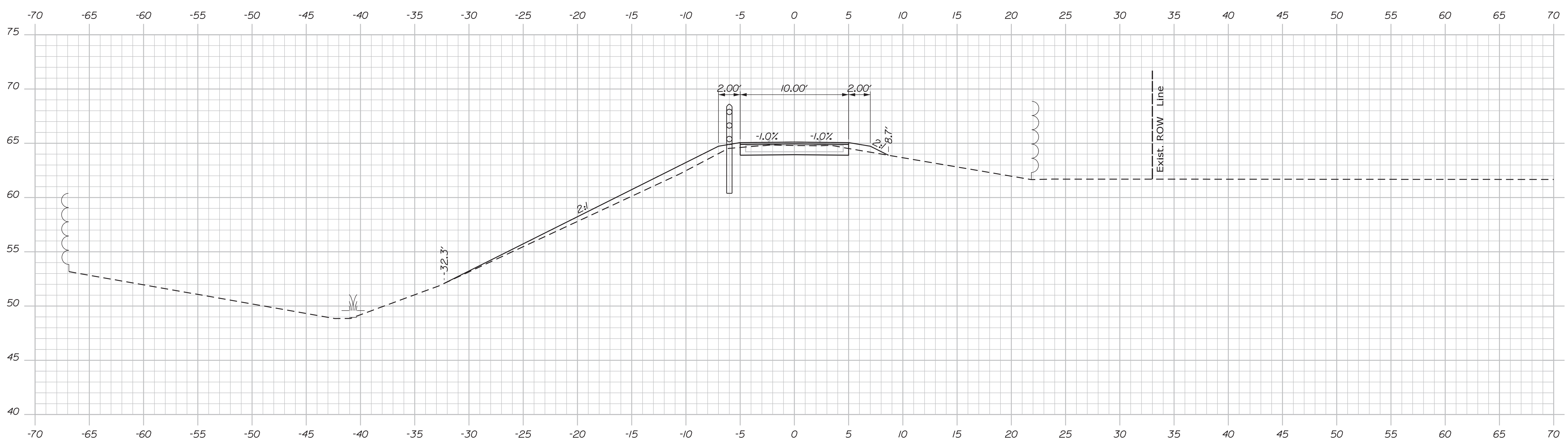

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Date: 2/24/2022
Username: lflanders
Division: BRIDGE
Filename: ...\10\multimode\MSTA\xsec1.dgn

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS
BRIDGE PLANS

SIGNATURE
P.E. NUMBER
DATE

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DESIGN2-DETAILED2	L. FLANDERS	LSF	
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FIELD CHANGES			

GARDINER
COBBOSSEE STREAM

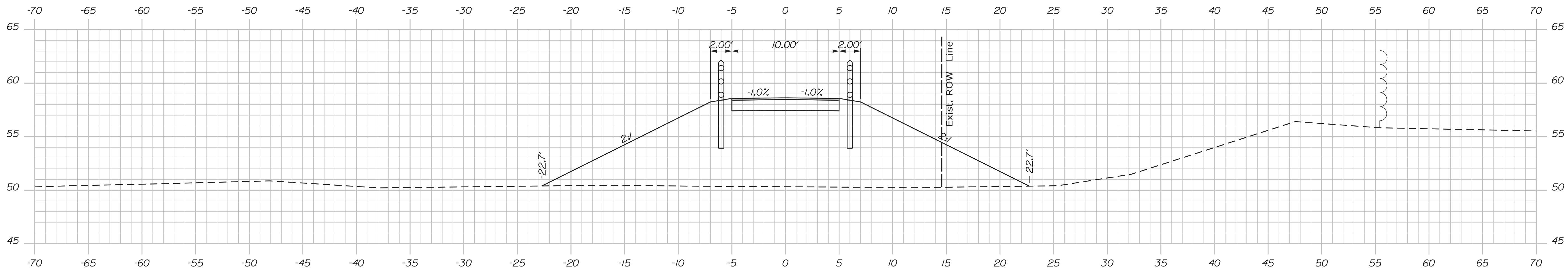
CROSS SECTIONS OPTION 2

SHEET NUMBER

16

OF 23

Filename: ...\10\multimode\MSTA\xsect.dgn Division: BRIDGE Username: lflanders Date:2/24/2022



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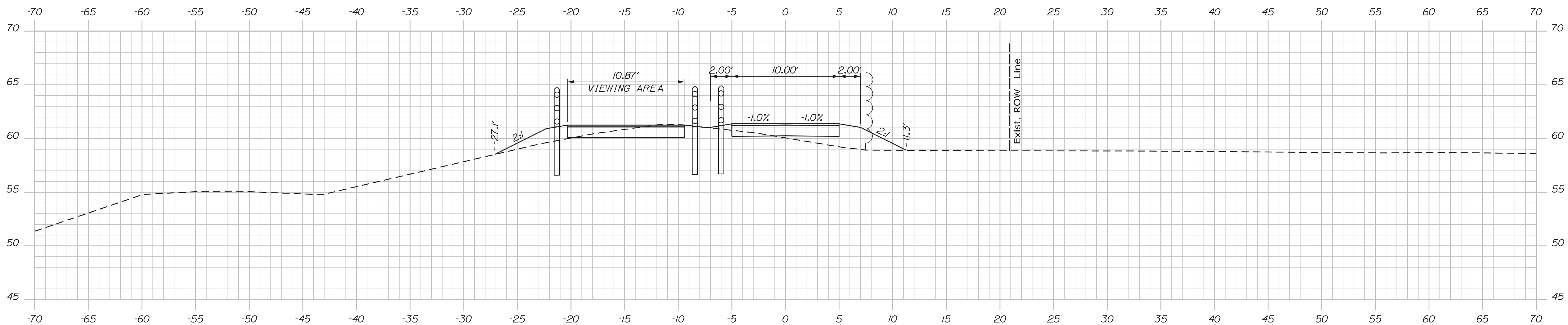

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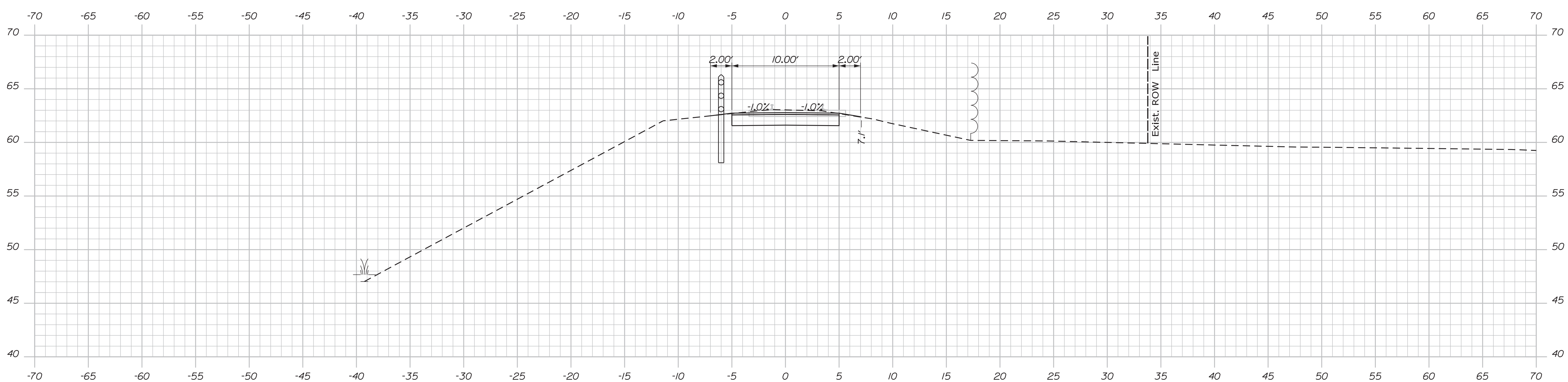

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

BRIDGE PLANS
HIGHWAY PLANS

PROJ. MANAGER	J. DOUGLASS	BY	DATE
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REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 2

SHEET NUMBER
17
OF 23

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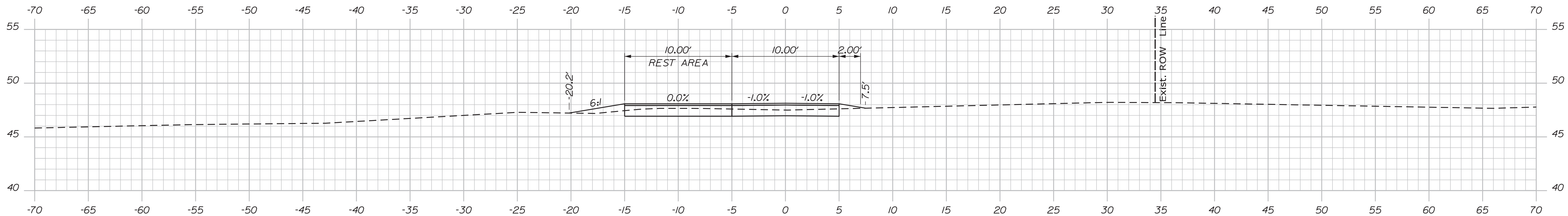

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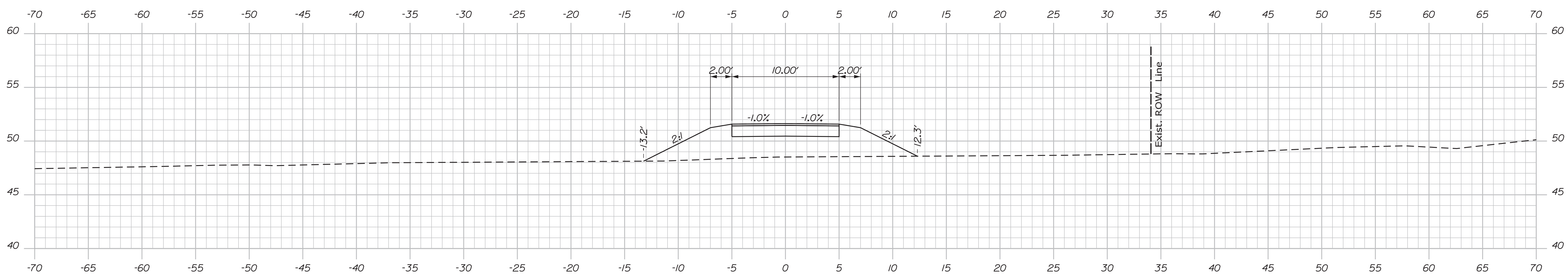

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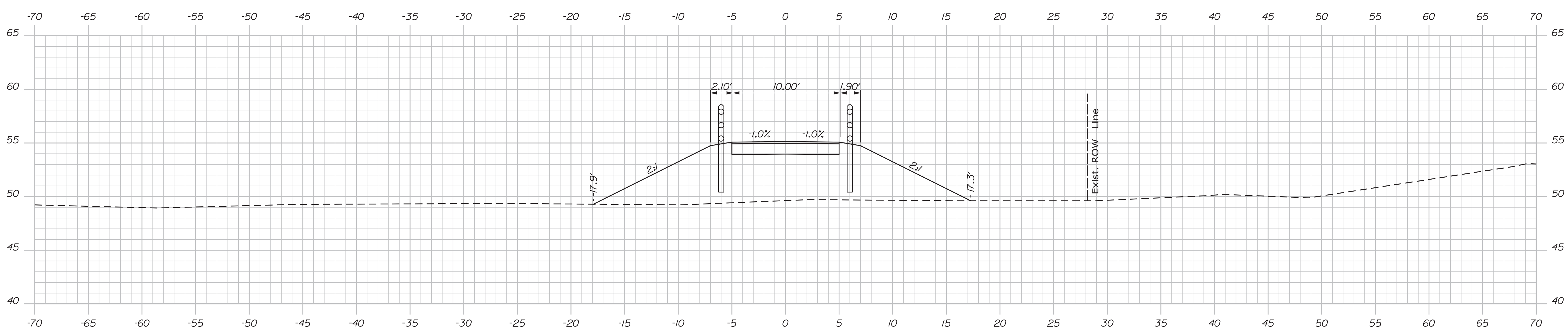

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS

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DATE

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 2

SHEET NUMBER
18
OF 23

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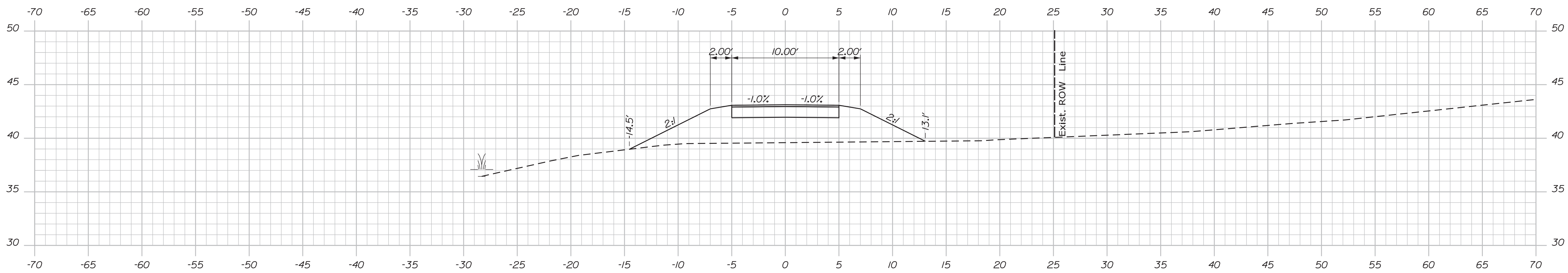

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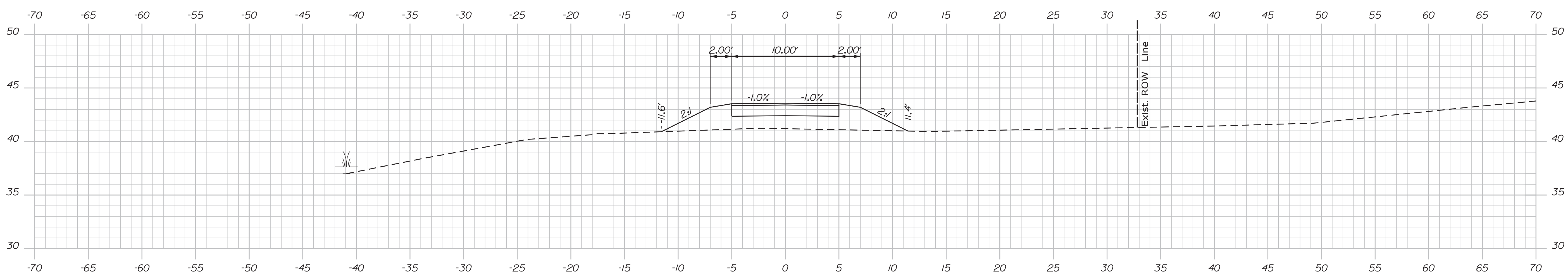

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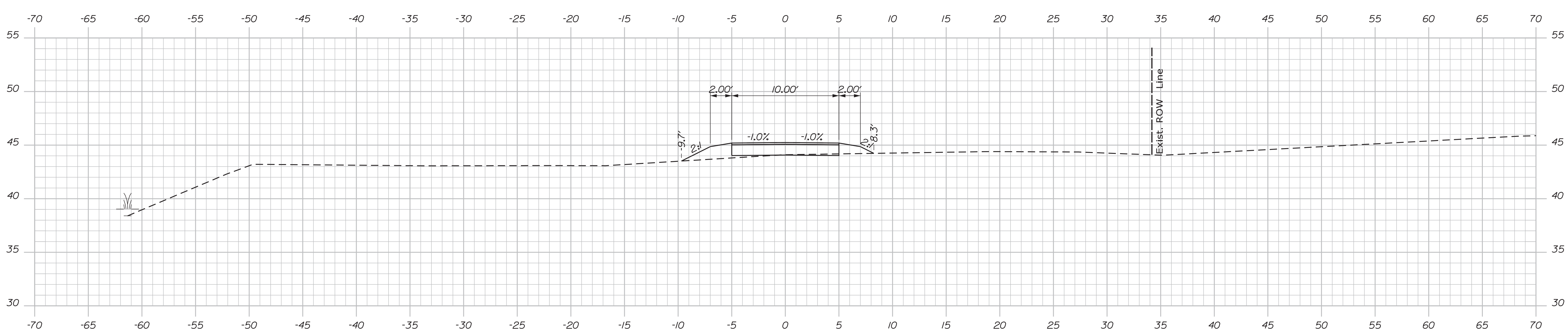

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS

PROJ. MANAGER	J. DOUGLASS	BY	DATE
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REVISIONS 4			
FIELD CHANGES			

SIGNATURE

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GARDINER
COBBOSSEE STREAM

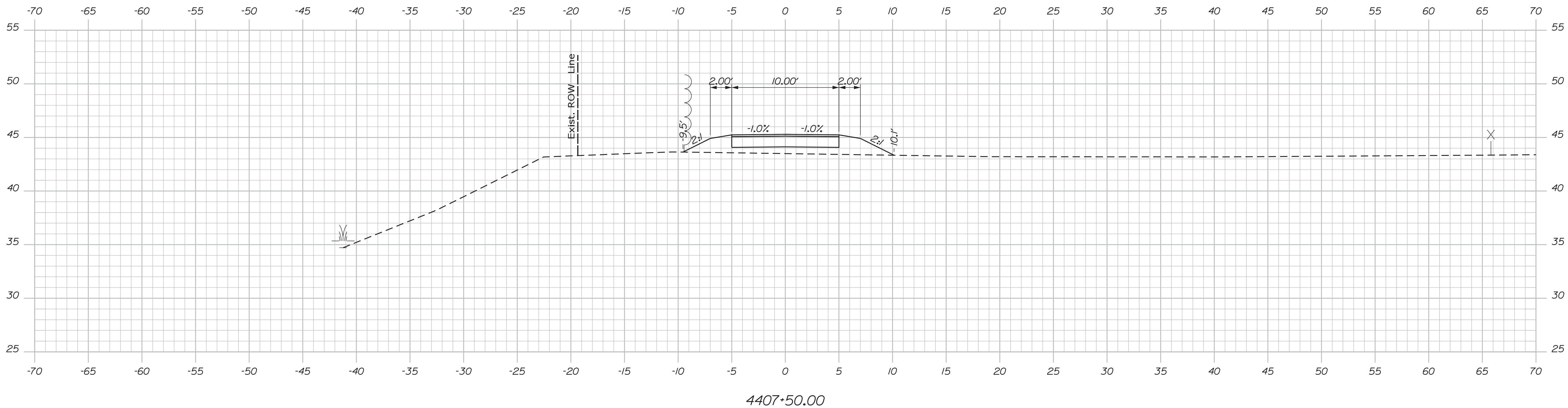
CROSS SECTIONS OPTION 2

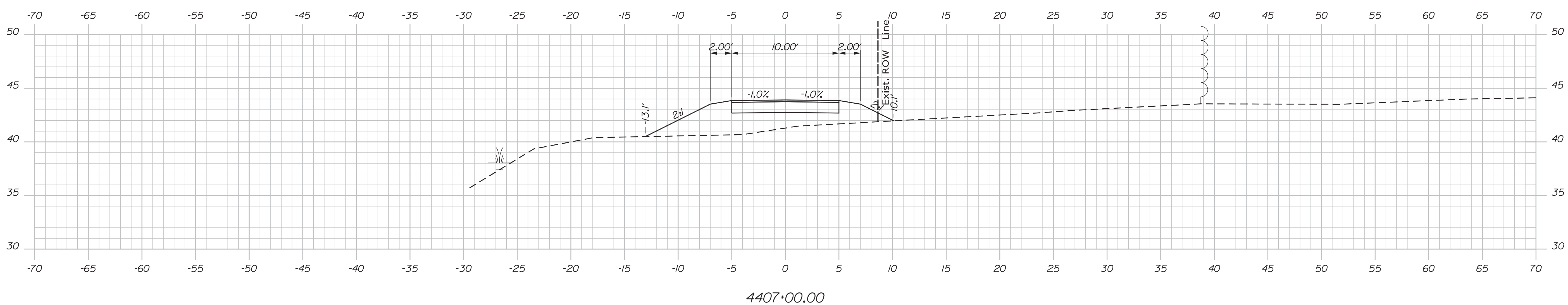
SHEET NUMBER

19

OF 23

Filename: ...\10\multimode\MSTA\xsec1.dgn Division: BRIDGE Username: lflanders Date:2/24/2022





STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

HIGHWAY PLANS / BRIDGE PLANS

PROJ. MANAGER	J. DOUGLASS	BY	DATE
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
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REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 2

SHEET NUMBER
20
OF 23

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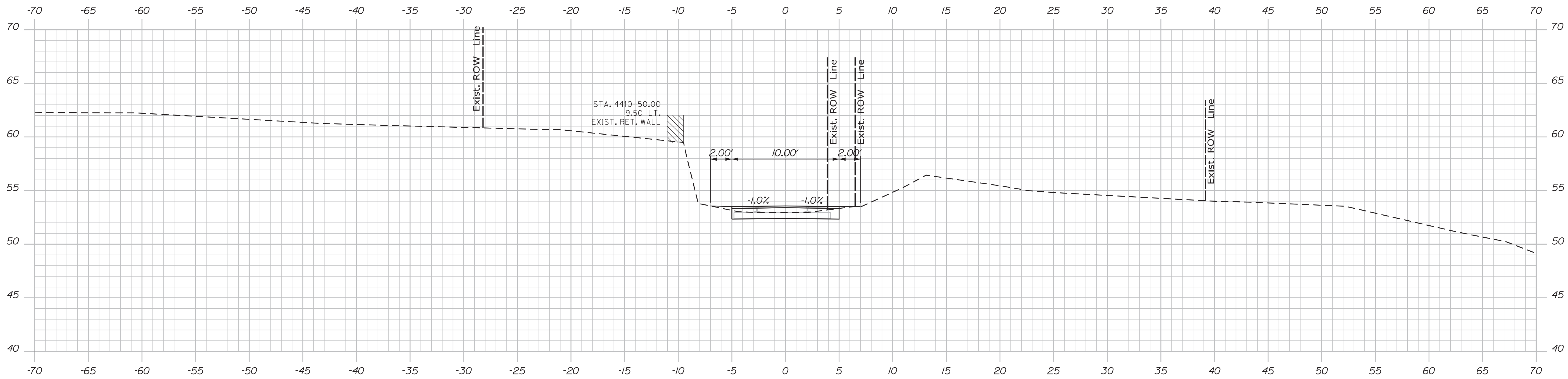
Filename: ...\10\multimode\MSTA\xsec1.dgn   Division: BRIDGE   Username: lflanders   Date:2/24/2022



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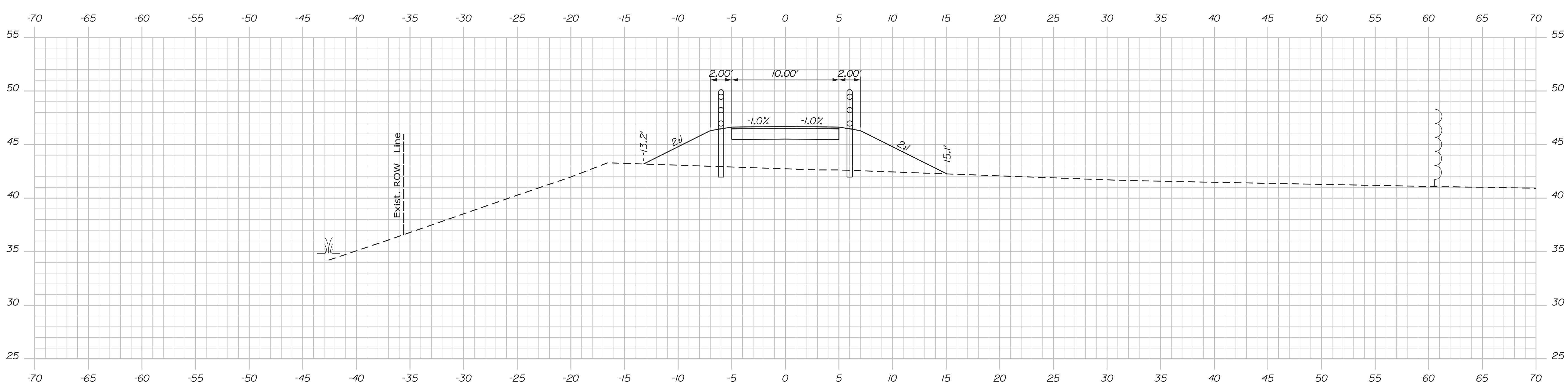
MULTI-USE BRIDGE STA. 4408+30 TO STA. 4410+15



4408+00.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

HIGHWAY PLANS / BRIDGE PLANS

WIN
13344.10

SIGNATURE

P.E. NUMBER

DATE

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
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DESIGN2-DETAILED2	L. FLANDERS	LSF	
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REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 2

SHEET NUMBER
21
OF 23

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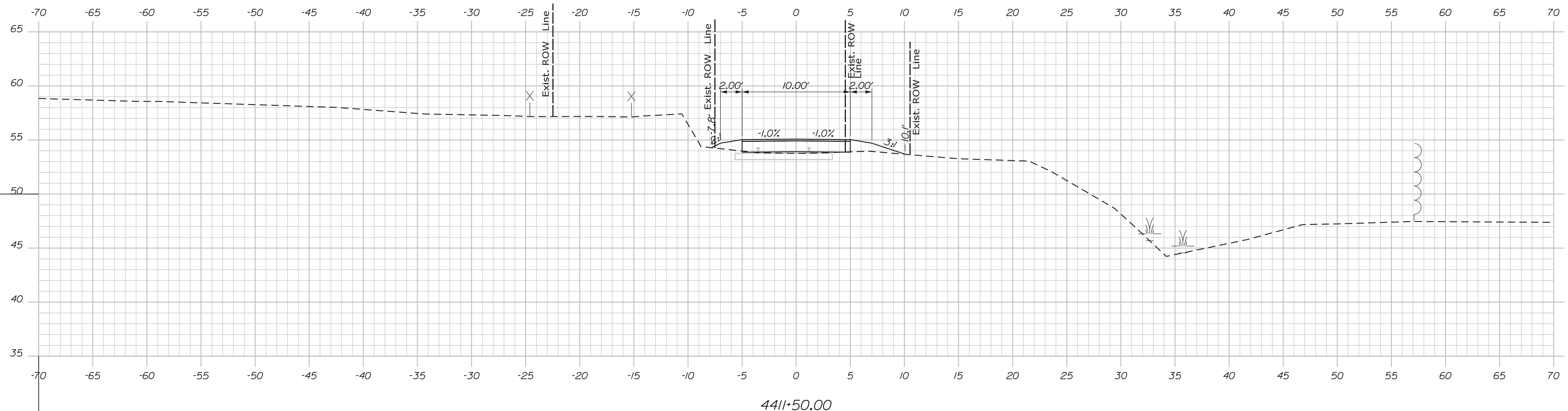
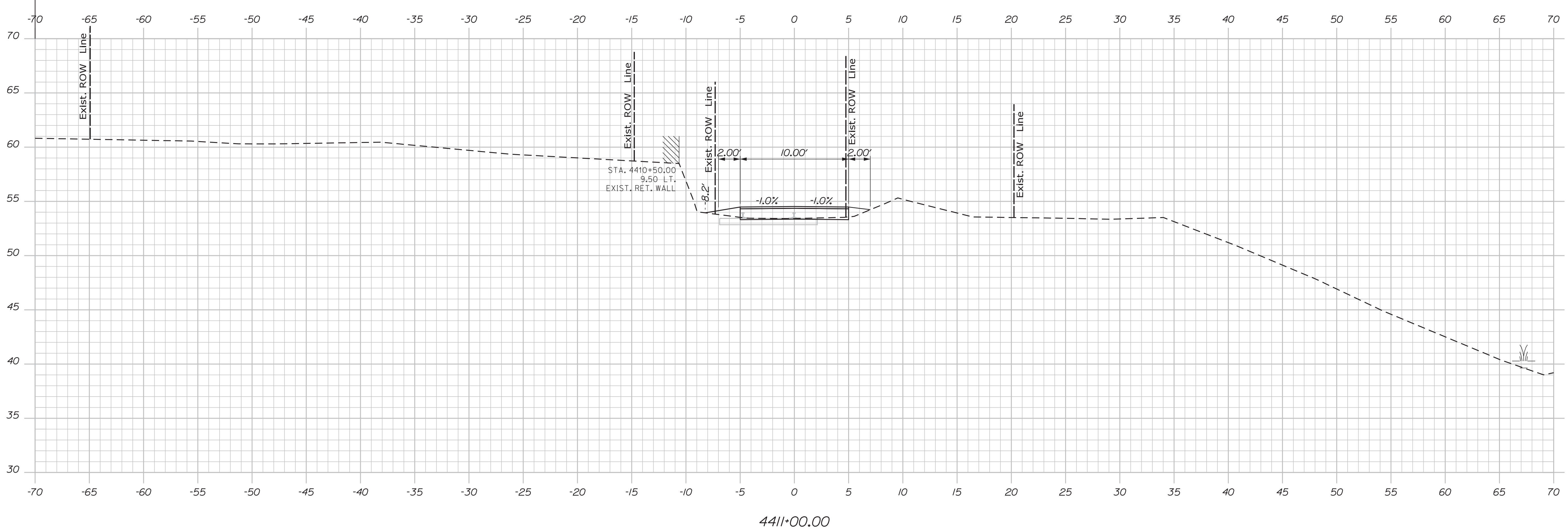
Division: BRIDGE

Username: lflanders

Date: 2/24/2022





STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN 13344.10

HIGHWAY PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
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FIELD CHANGES			

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GARDINER
COBBOSSEE STREAM

CROSS SECTIONS OPTION 2

SHEET NUMBER
22
OF 23

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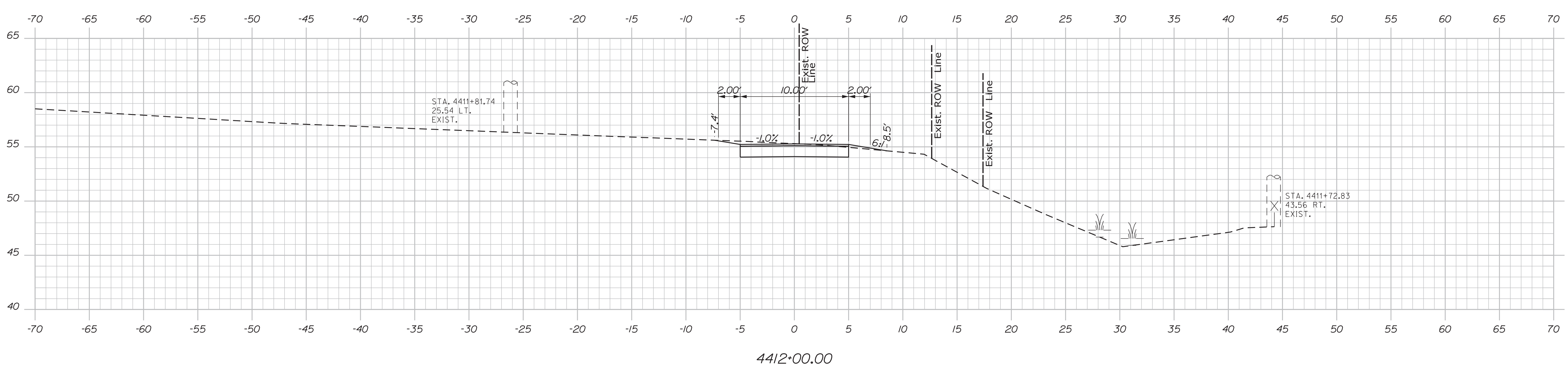

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN
13344.10

BRIDGE PLANS
HIGHWAY PLANS

PROJ. MANAGER	J. DOUGLASS	BY	DATE
DESIGN-DETAILED	P. POTTLE	PLP	08\2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
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REVISIONS 4			
FIELD CHANGES			

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GARDINER
COBBOSSEE STREAM

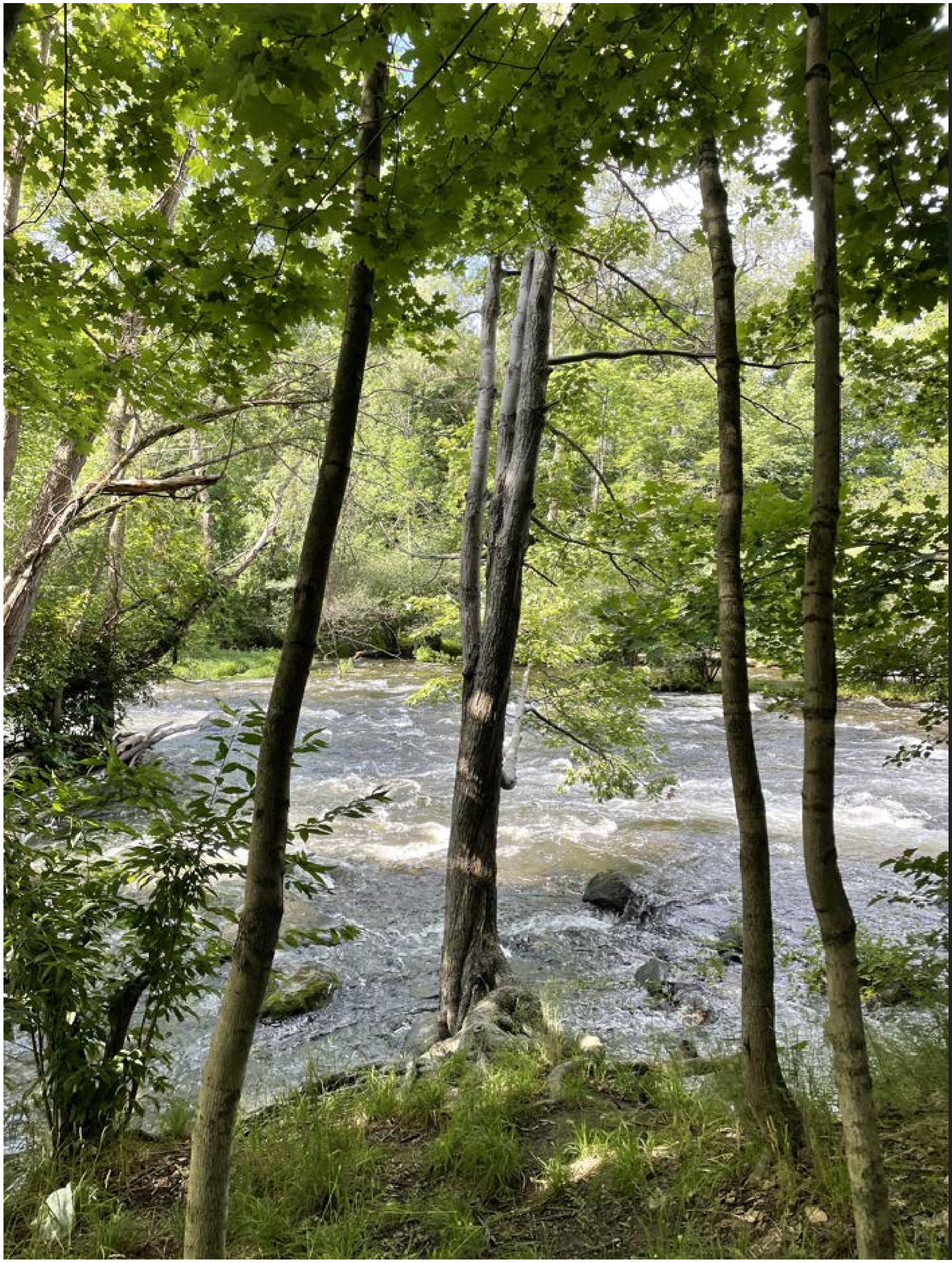
CROSS SECTIONS OPTION 2

SHEET NUMBER
23
OF 23

Sta. 4412+00.00 to Sta. 4412+00.00

Appendix D MEETING HANDOUTS

AUGUST 5, 2021

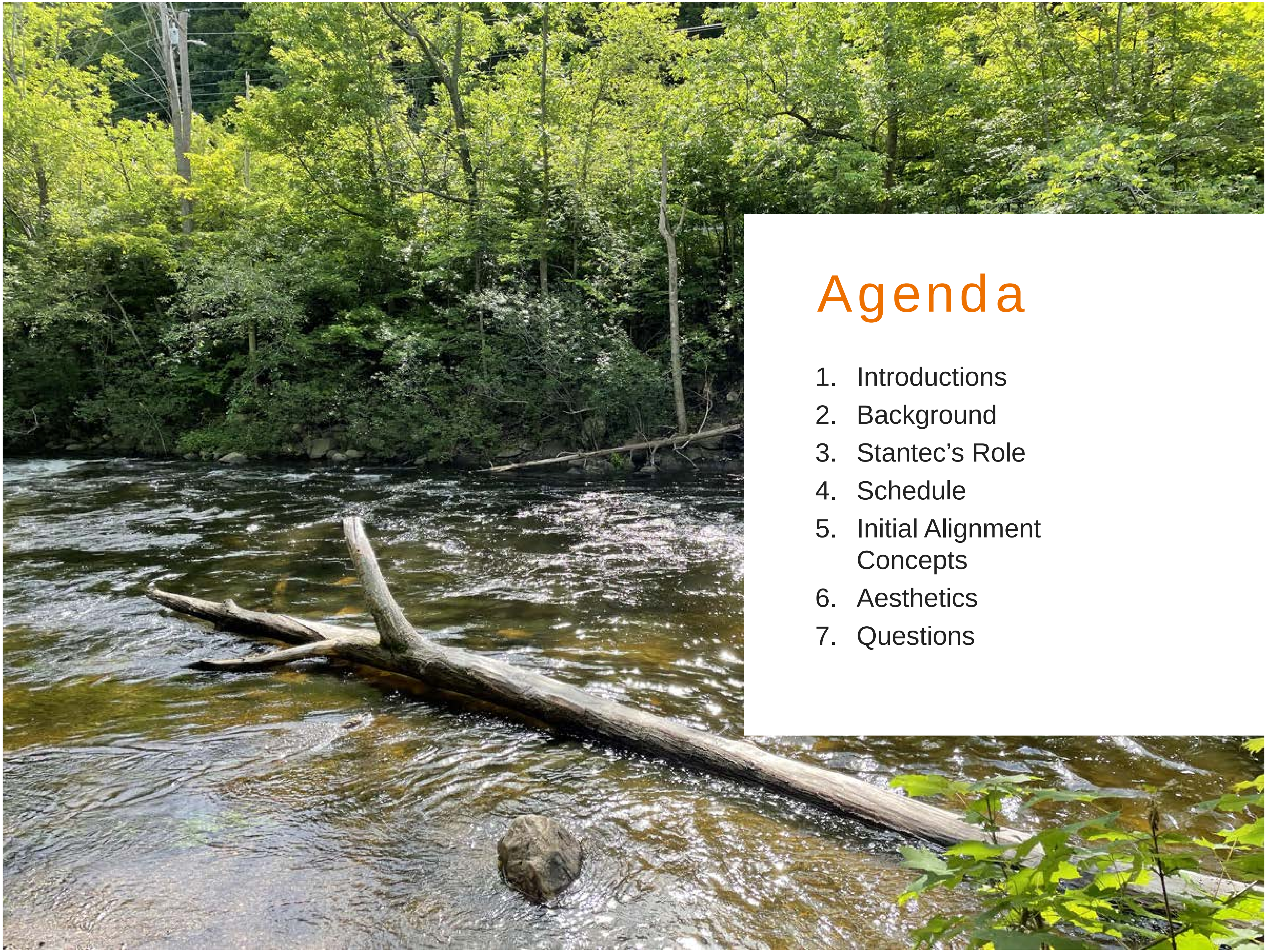


City of Gardiner

Cobbossee Trail Feasibility Study

Stantec

Safety Moment



Agenda

1. Introductions
2. Background
3. Stantec's Role
4. Schedule
5. Initial Alignment Concepts
6. Aesthetics
7. Questions

Introductions

Project Background

- 2005 Corridor Study
- 2018 PDR Update
- Train Trestle Condition

Stantec's Role

Meetings

- 1 in-person kick-off meeting (on-site 7/28)
- 2 trail committee meetings (virtual)
- 2 submittal review meetings with City (virtual)
- 2 environmental coordination meetings with DOT (virtual)

Deliverables

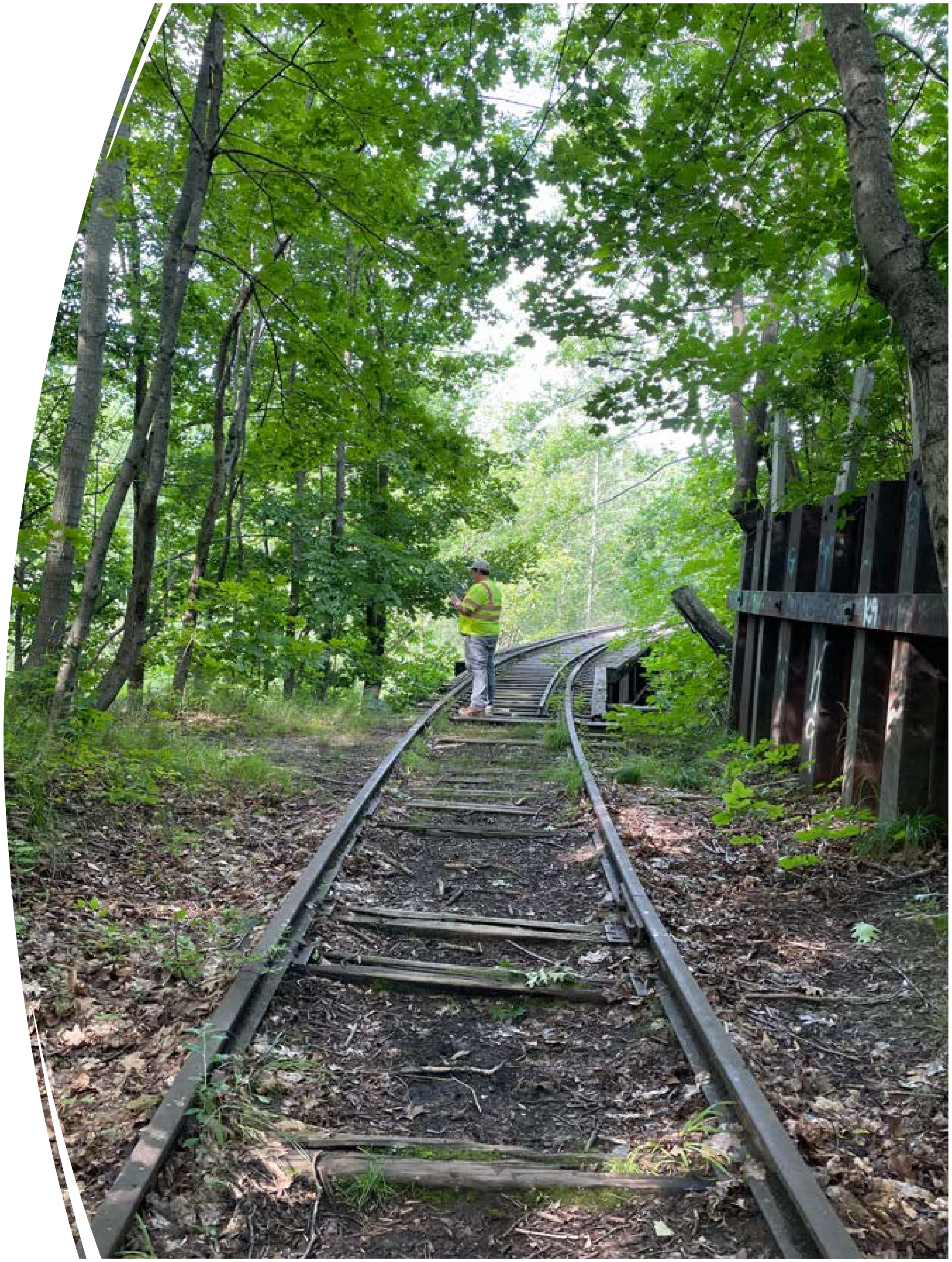
- Survey and Wetland Delineation
- Utility Coordination
- Feasibility Study
  - Report
  - General Plans
  - Renderings
  - Construction Cost Estimate
  - Potential ROW impacts identified

Schedule

Notice to Proceed	**7/19/21**
Survey Complete	**8/20/21**
Submit Progress Plans to City	**8/27/21**
Review Meeting with City	***Week of 9/13/21***
Borings	**9/13/21 to 9/17/21**
Submit Draft Feasibility Study to City	**11/19/21**
Review Meetings with City and Trail Committee	***Week of 11/29/21***
Submit Final Feasibility Study to City	**12/23/21**
Stantec's contract expiration	**12/31/21**

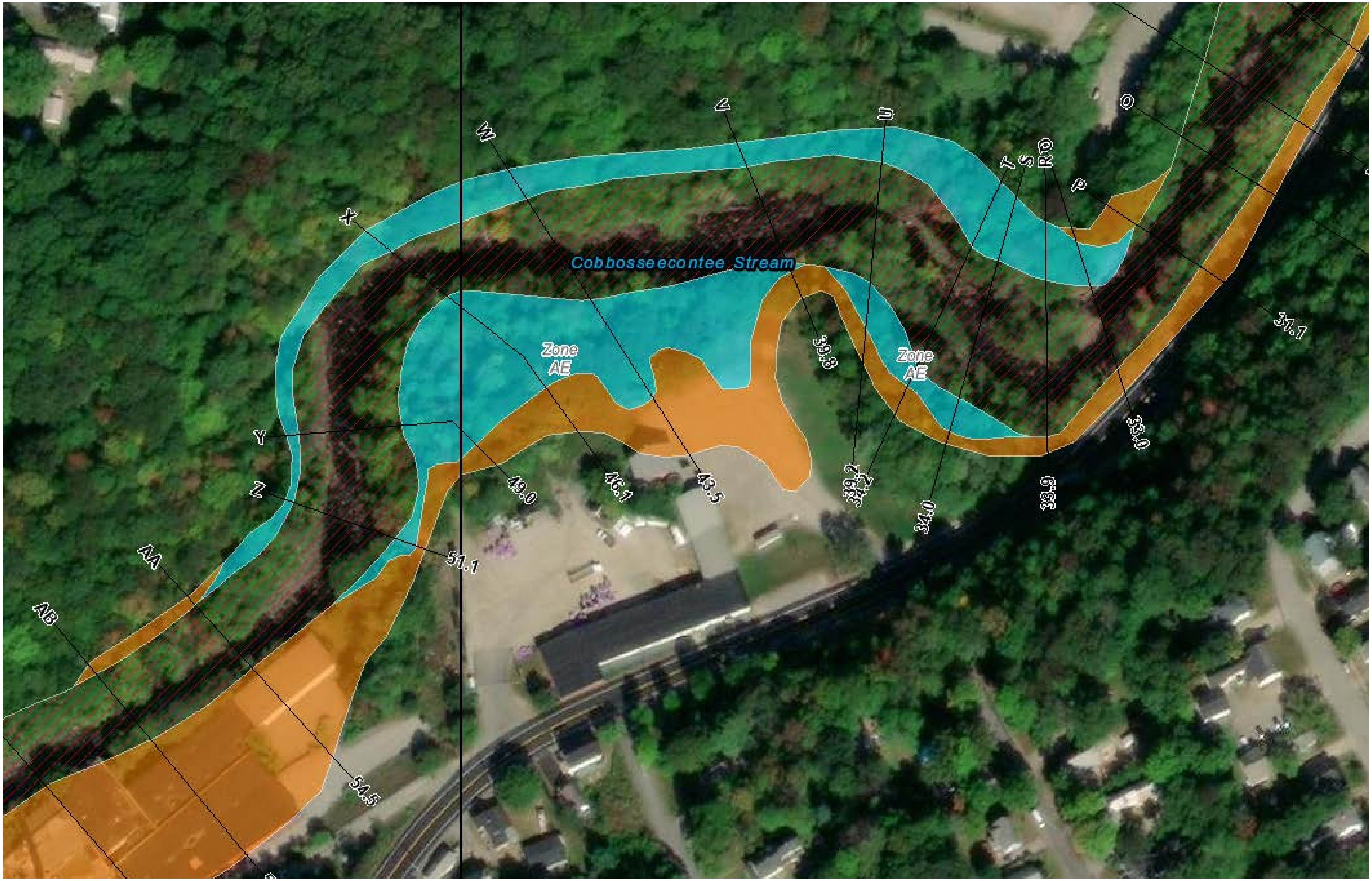
Initial Alignment Concepts

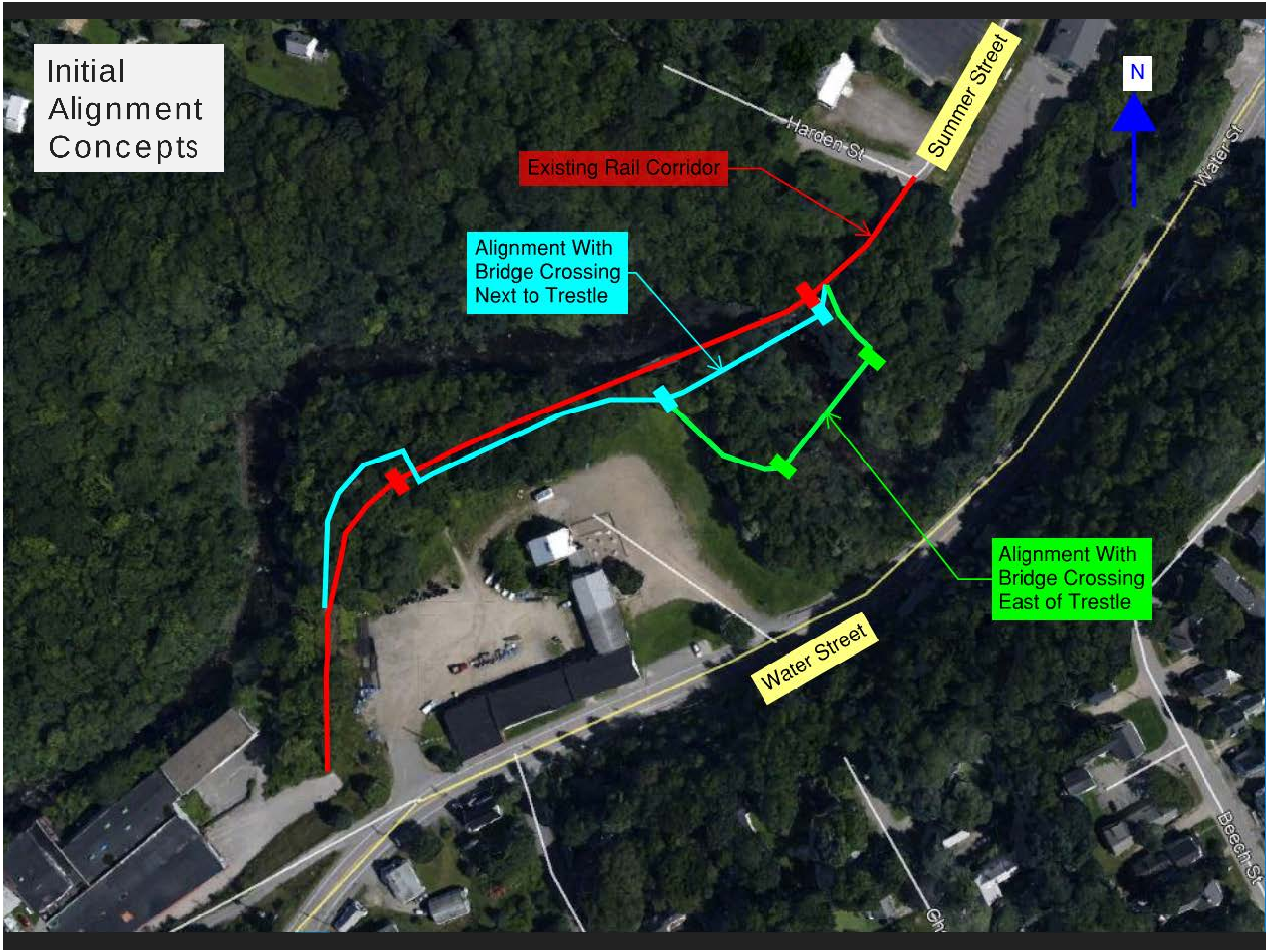
- North bank alignment
- Alignment with bridge crossing next to trestle
- Alignment with bridge crossing east of trestle
- Considerations:
  - FEMA Floodway
  - ROW
  - View of trestle
  - Bridge span length with no piers
  - Abutment locations
  - Trail width & grade
  - Use of fence
  - Connection to Water Street sidewalk



FEMA Floodway

Red Stripes = Regulatory Floodway
Cyan = 100 Year Flood
Orange = 500 Year Flood




Initial Alignment Concepts
N
Summer Street
Harden St
Existing Rail Corridor
Water St
Alignment With Bridge Crossing Next to Trestle
Alignment With Bridge Crossing East of Trestle
Water Street
Beech St

Initial Alignment Concepts – North Bank

North Bank Alignment

- Extremely steep – requires significant retaining walls
- Retaining wall construction access very limited
- Much less feasible than other alignment options

Initial Alignment Concepts – Next to Truss

Initial Alignment Concepts – East of Truss

Initial
Alignment
Concepts

The Trail Beyond the River

Aesthetics

- Approach surface
- Lighting
- Truss (weathering steel/painted)
- Truss travel surface (concrete/IPE)

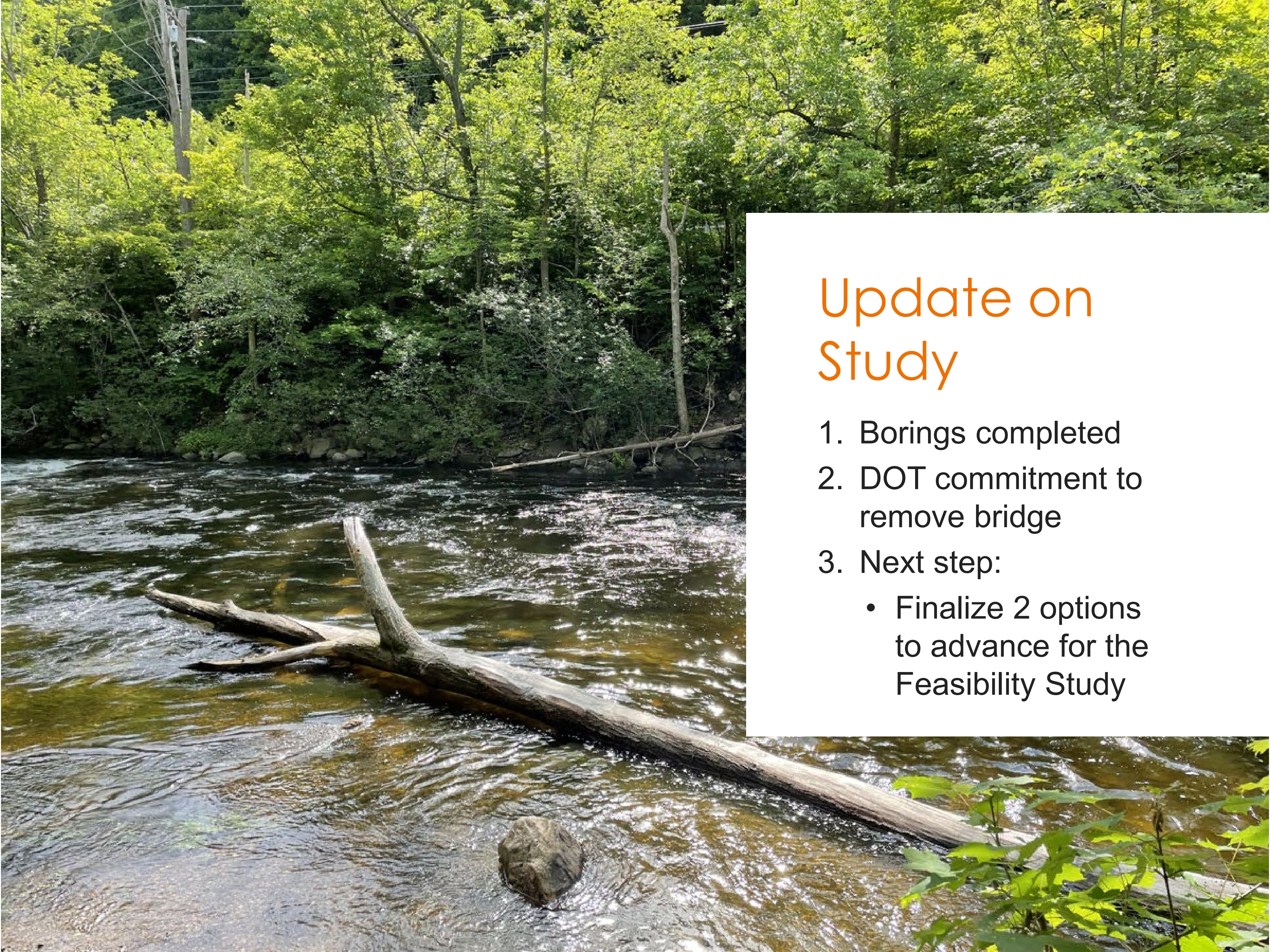
Questions?

NOVEMBER 15, 2021

City of Gardiner

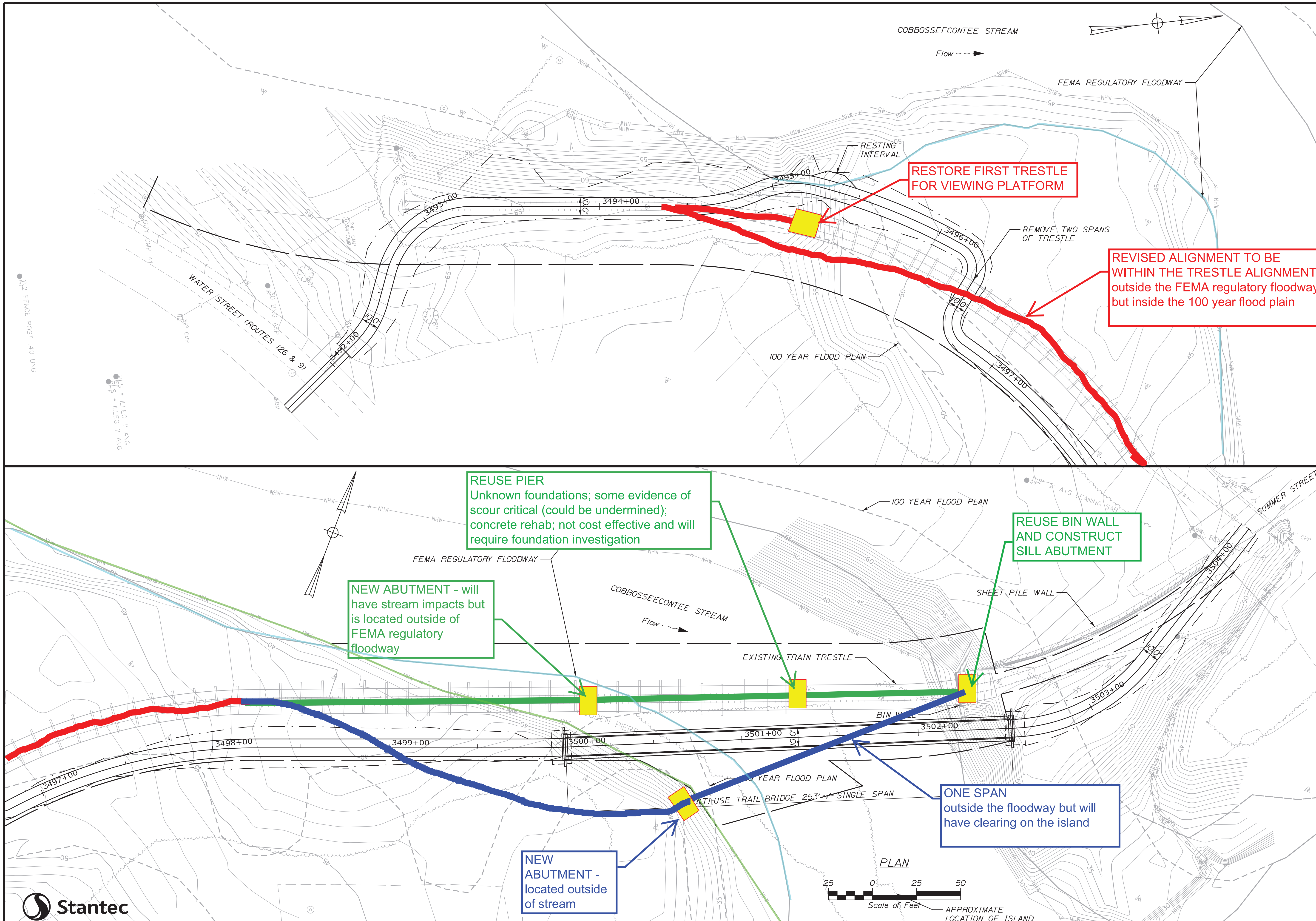
Cobbossee Trail Feasibility Study

Stantec



Update on Study

1. Borings completed
2. DOT commitment to remove bridge
3. Next step:
   - Finalize 2 options to advance for the Feasibility Study



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Division: HIGHWAY
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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN 13344.10

MULTIMODAL PLANS

	BY	DATE
PROJ. MANAGER	J. DOUGLASS	
DESIGN-DETAILED	P. POTTLE	PLP 08\2021
CHECKED-REVIEWED	L. MEEK	LEM
DESIGN2-DETAILED2	L. FLANDERS	LSF
DESIGN3-DETAILED3		
REVISIONS 1		
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REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

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P.E. NUMBER
DATE

GARDINER
COBBOSSEE STREAM
TRAIL PLAN OPTION 1

SHEET NUMBER
3
OF 25

Option 1

Revise with consideration of removing the wooden trestle and steel spans

Option 1A (Green)

- Reuse of Existing Northern Abutment
- Reuse of the Pier vs. New Pier
- 2 Spans
- New Southern Abutment inside of stream & outside of FEMA Regulatory Floodway

Option 1B (Blue)

- Reuse of Existing Northern Abutment
- 1 Span
- New Southern Abutment outside of stream & FEMA Regulatory Floodway

Realignment South of Stream (Red)

- Alignment in footprint of trestle
- Less disturbance/ clearing
- Less ROW impacts
- Remain out of the FEMA Floodway

Conceptual Cost Difference (vs Option 1B)

+$300,000 - $580,000 Reuse of Pier

+$280,000 New Pier

Option 1A (Green)

REUSE OF EXISTING PIER

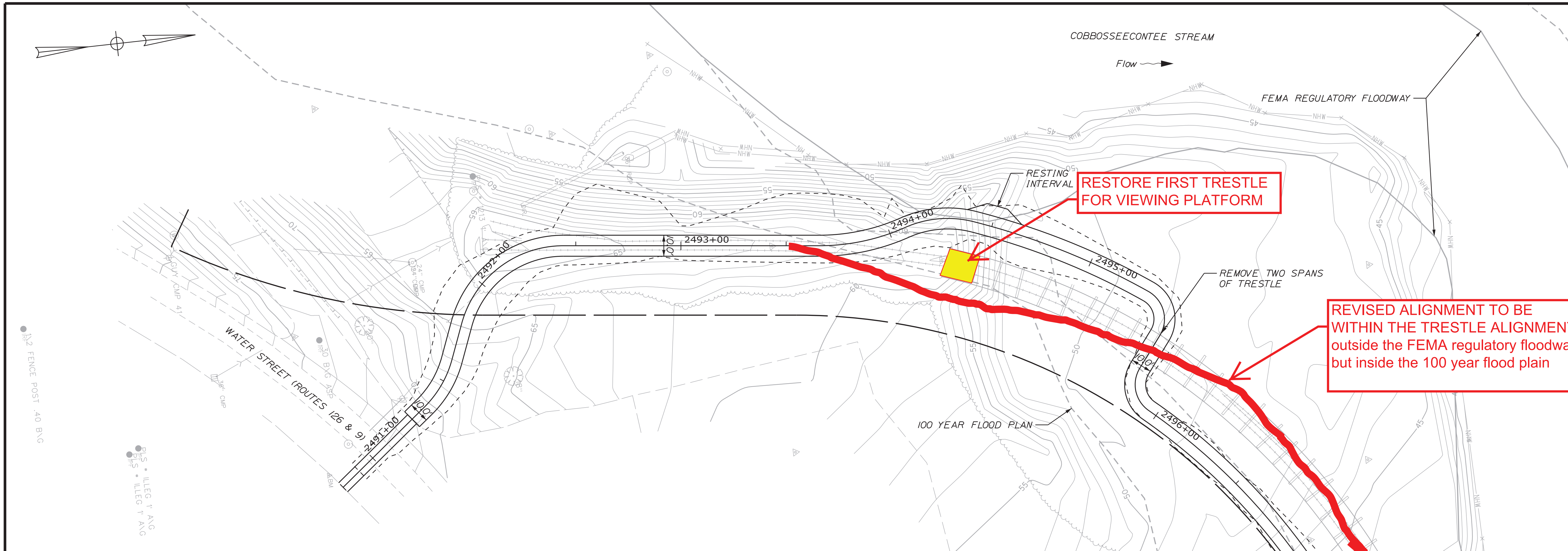
(+$300,000 - $580,000 vs. Blue)

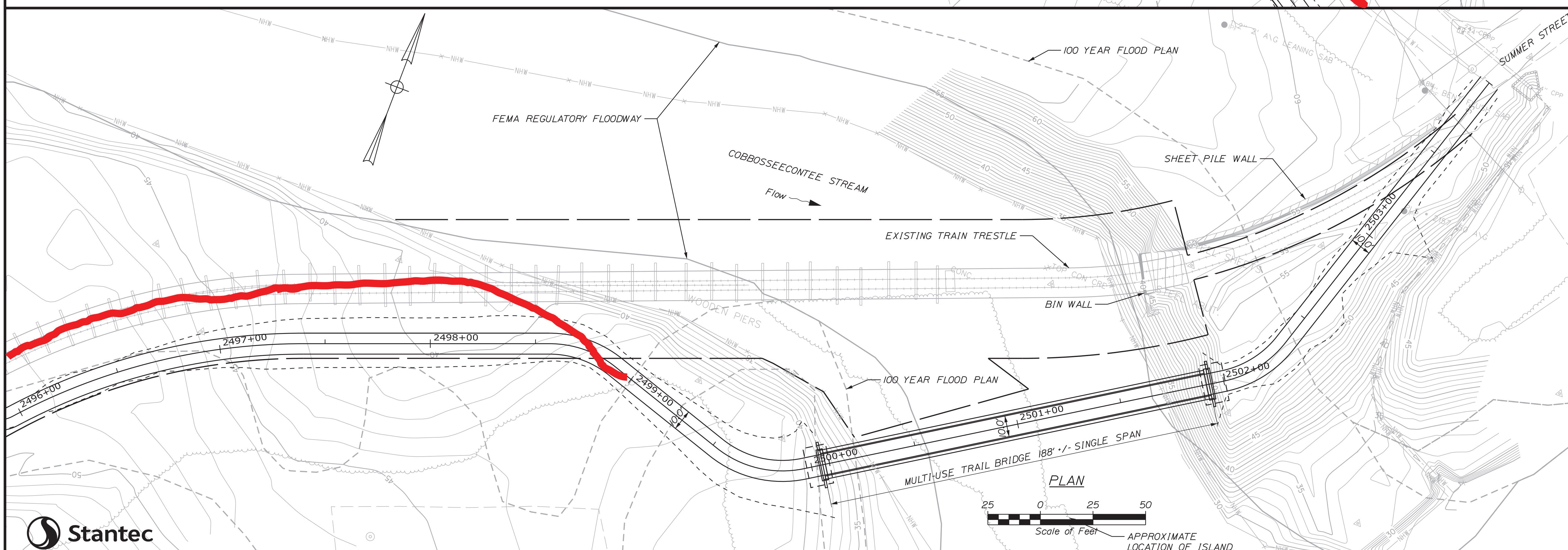
- Risk of Unknown Scope for Construction Phase until after Investigation
- Age (100+year old structure)
- Investigation of Foundation Material and Condition of Concrete
- Could Require Micropiles (depending on depth of existing footing and soil conditions)
- Rehab / Building Up of Pier
- Scour Protection

NEW PIER

(+$280,000 vs. Blue)

- Investigation of Foundation Material (Boring in Stream)
- Construction of Pier
- 75+ Years Service Life





STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1334410

WIN 13344.10

MULTIMODAL PLANS

		BY	DATE
PROJ. MANAGER	J. DOUGLASS		
DESIGN-DETAILED	P. POTTLE	PLP	08/2021
CHECKED-REVIEWED	L. MEEK	LEM	
DESIGN2-DETAILED2	L. FLANDERS	LSF	
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

GARDINER
COBBOSSEE STREAM
TRAIL PLAN OPTION 2

SHEET NUMBER
4
OF 25

Date:8/26/2021
Username: lmeek
Division: HIGHWAY
Filename: ...\MSTA_genplon_option2.dgn

Option 2

Revise with consideration of removing the wooden trestle and steel spans

Realignment South of Stream (Red)
• Alignment in footprint of trestle • Less disturbance/ clearing • Less ROW impacts • Remain out of the FEMA Floodway

FEBRUARY 7, 2022

City of Gardiner

Cobbossee Trail Feasibility Study

Stantec

Update on Study

1. Draft to Committee on 1/25/2022
2. Next steps:
    - Review Comments from Committee - 2/8/2022
    - Incorporate Comments
    - Submit Final Report – 2/25/2022
    - City Council Recommendation

Alignment Options

Option 1 (Cyan)

- Reuses the existing trestle alignment
- Two bridge spans (129 ft. +100 ft.)

Option 2 (Orange)

- Reuses existing north abutment landing and crosses south of the existing trestle and connects back to the existing trestle alignment
- One bridge span (185 ft.)

OPTION 1
OPTION 2
FLOW
COBBOSSEECONTEE STREAM
HARDEN STREET
SUMMER STREET
WATER STREET (ROUTE 126 & 9)
WATER STREET
RIGHT OF WAY
NHW
NATURAL HIGH WATER
PFO
WETLANDS
FEMA REGULATORY FLOODWAY
APPROXIMATE 100 YEAR FLOOD PLAIN
50
0
50
100
Scale of Feet

Alignment Options – Right of Way

Option 1 (Cyan)

- Mainly uses the existing State ROW
- Warren Parcel - Permanent acquisition near Water Street
- Temporary easements needed for construction

Option 2 (Orange)

- Warren Parcel - Permanent acquisition near Water Street and at the south end of the bridge
- Temporary easements needed for construction

Alignment Options – Engineering & Permitting

Option 1 (Cyan)

- Design of two abutments, one pier, and retaining walls
- Additional geotechnical exploration required
- Extensive hydraulics analysis (unclear whether there is an increase in hydraulic opening)
- Extensive in-water work subject to Section 7 endangered fish species permitting; Restrictive window for construction
- Requires minimal clearing in the island

Option 2 (Orange)

- Design of two abutments
- Minimal hydraulics analysis (by inspection, net positive increase in hydraulic opening compared to existing)
- Very little in-water work
- Requires clearing in the island

Bridge Options

Conceptual Cost Difference

Modular steel truss costs 20%-30% less than the H-section truss

H-Section Truss

- Heavy crane picks
- Comes in galvanized or galvanized + painted

Modular Steel Truss

- More constructable
- Comes in galvanized only

Existing Train Trestle

Option 1
H-Section Truss - Galvanized

Option 1
H-Section Truss - Galvanized and Painted

Option 1
Modular Truss - Galvanized

Option 2
H-Section Truss - Galvanized

Option 2
H-Section Truss - Galvanized and Painted

Option 2
Modular Truss - Galvanized

Alignment Options – Construction Costs

Option 1 (Cyan)

- $2,058,000(H-Section Truss)
- $1,915,000 (Modular Steel Bridge)
- Pier and retaining walls contribute the most to additional cost

Option 2 (Orange)

- $1,394,000 (H-Section Truss)
- $1,256,000 (Modular Steel Bridge)

Cost Assumptions

- Does not include lighting, landscaping, or additional aesthetic treatments
- Does not include Engineering, ROW or Construction Services

Appendix E PROPERTY OWNER REPORT

PROPERTY OWNER REPORT

Municipality: Gardiner
WIN: 013344.10
Report By:
Date:

Tax Map 33
Block
Lot 102

Owner of Property: Harold Warren Const. Co.

Spouse:

Contact Person for Companies/Organizations: Michael Warren

Legal Address: 131 Nehumkeag Way Pittston, ME. 04345

Mailing Address: Same

Home Phone: —— Business Phone: 207 458-0751

Can Owner be Contacted at Home? Yes ☒ No ☐ Work? Yes ☒ No ☐

PROPERTY INFORMATION

Deed Information: Book: 5271 Page: 262 Date: 12-1996
12803 86 12-2017

Remarks:

Name of Previous Owner: Area leasing and Development / State of Maine

Boundary Line Markers: Yes ☒ No ☐ Iron Pin ☒ Granite Mon. ☒ Other ☐

If a fence or hedge exists, do you or your neighbor own the fence or hedge? N/A

Date Building Built: ? Any Cemeteries on Property? Yes ☐ No ☒

Is Property or Building registered as a Historic Site? Yes ☐ No ☒

Is Property considered Park lands, commonly referred to as 4F lands? Yes ☐ No ☒

Is Property currently licensed with the Federal Energy Regulatory Commission (FERC)? NO

Has Property been Surveyed? Yes ☐ No ☐ ? Is Survey Recorded? Yes ☐ No ☐

Names of Surveyor: ? May We Obtain a Copy? Yes. ☐ No ☐

Approximate Frontage on Highway: 1091± Total Area of Lot: 4.68± A

Water Supply: Drilled Well (ft.) Dug Well (ft.) Spring ☐ Municipal ☒ Well Point (ft.) Other:

Has Supply been Adequate for the Past Five Years? Yes No. of People:

Is Location of Water Source Known? Yes ☐ No ☐ (Show on sketch of property) N/A

Sewage: Septic Tank & Leach Field ☐ Cesspool ☐ Municipal ☒ Other

Is Location of Sewage System Known? Yes ☐ No ☐ (Show on sketch of property) N/A

Private Pipes or Wires Into or Under Highway? Yes ☐ No ☒ Pipes ☐ Wires. ☐

Property Use Residence: Yes ☐ No ☒ Owner Occupied or Rented

If Rented, No. of Units: Number of Tenants:

If Farm Property: Count of Stock Acres of Pasture N/A

Acres Cultivated Acres Leased Acres Non-Locus to Farm: N/A

Is Property under Maine Tree Growth Law? Yes ☐ No ☒

If Commercial Property: Type Owner Occupied or Leased:: leased

Name of Lessee: Prefered Pump / Sebago lake dist.

Underground Storage Tank(s)? Yes ☐ No ☒ Gas ☐ Diesel ☐ Heating Oil ☐

Chemical or Hazardous Substances ☐ (Show on sketch of property)

Do you own the Tank(s)? Yes ☐ No ☐ If no, name of owner:

Owners Comments:

PLEASE MAKE SKETCH OF PROPERTY: (Use Separate Sheet if Necessary)

Signed [signature]

The information in this report is necessary for the development of transportation projects.
Thank you for your assistance.

Appendix F UTILITY COORDINATION LETTER #1 RESPONSE

*** IMMEDIATE RESPONSE REQUESTED ***

January 20, 2022

Town/City: Gardiner, ME
Project WIN: 013344.10
Location: Old Train Trestle Near Summer St

RE: Identification of Utility Facilities

To whom it may concern:

The City of Gardiner is planning the construction of a multi-use trail and bridge, located near the old train trestle crossing Cobbossee Stream between Summer Street and Water Street/Route 126.

Enclosed you will find a location map to further assist you in locating the proposed project.

Please complete and return the brief questionnaire attached to this letter. The information provided here will allow our project designers to recognize the presence of existing facilities or plans to install additional facilities within the next five years. Your responses will enable us to better coordinate our work with you throughout this project.

The Work Identification Number (WIN) assigned to this project is 013344.10 and should be used on any future correspondence regarding this project. This project is currently in a feasibility/planning stage only and does not have a planned advertisement date.

If you have any questions or concerns, please feel free to contact me at 207-303-7435, or by email at paul.pottlejr@stantec.com. Thank you for your cooperation.

Sincerely,

Paul Pottle, PE
Senior Transportation Engineer
Stantec Consulting Services, Inc.

Enclosures: Questionnaire Response Form
Project Location Map

**** IMMEDIATE RESPONSE REQUESTED ****

RE: 013344.10 Date

Town/City: Gardiner, ME
Project WIN: 013344.10
Location: Old Train Trestle Near Summer St

Utility Coordinator: Paul Pottle, PE
Stantec Consulting Services, Inc.
2211 Congress Street, Suite 380
Portland, ME 04102
Phone: 207-303-7435
E-Mail: paul.pottlejr@stantec.com

Please complete the following short questionnaire and fax, email or send via mail. The following may be filled out electronically in Microsoft Word by using the "TAB" key.

Utility: **Date Form Submitted:**

1. Does the utility you represent presently have facilities within the project limits? ☐ Yes ☐ No

2. What type of facilities do you have in the project area? ☐ Underground ☐ Aboveground

3. Pole Owner:

Attachments:

4. Do you plan on installing any facilities within the project limits in the next 5 years? ☐ Yes ☐ No

6. Contact person for project coordination:
Name:
Address:
Tel:
Cell:
Fax No:
E-mail:

6. Contact person for construction:
Name:
Address:
Tel:
Fax No:
E-mail:

7. Comments

** IMMEDIATE RESPONSE REQUESTED **

RE: 013344.10 Date

Town/City: Gardiner, ME
Project WIN: 013344.10
Location: Old Train Trestle Near Summer St

Utility Coordinator: Paul Pottle, PE
Stantec Consulting Services, Inc.
2211 Congress Street, Suite 380
Portland, ME 04102
Phone: 207-303-7435
E-Mail: paul.pottlejr@stantec.com

Please complete the following short questionnaire and fax, email or send via mail. The following may be filled out electronically in Microsoft Word by using the “TAB” key.

Utility: Spectrum Cable **Date Form Submitted:** 02/01/2022

1. Does the utility you represent presently have facilities within the project limits? ☐ Yes ☒ No

2. What type of facilities do you have in the project area? ☐ Underground ☐ Aboveground

3. Pole Owner:

Attachments:

4. Do you plan on installing any facilities within the project limits in the next 5 years? ☐ Yes ☒ No

6. Contact person for project coordination:

Name:	Stefanie Worster
Address:	83 Anthony Ave, Augusta ME 04330
Tel:	207-620-3441
Cell:	207-592-4788
Fax No:	
E-mail:	stefanie.worster@charter.com

6. Contact person for construction:

Name:	Same As Above
Address:	
Tel:	
Fax No:	
E-mail:	

7. Comments

** IMMEDIATE RESPONSE REQUESTED **

RE: 013344.10 Date

Town/City: Gardiner, ME
Project WIN: 013344.10
Location: Old Train Trestle Near Summer St

Utility Coordinator: Paul Pottle, PE
Stantec Consulting Services, Inc.
2211 Congress Street, Suite 380
Portland, ME 04102
Phone: 207-303-7435
E-Mail: paul.pottlejr@stantec.com

Please complete the following short questionnaire and fax, email or send via mail. The following may be filled out electronically in Microsoft Word by using the "TAB" key.

Utility: Consolidated Communications **Date Form Submitted:** 1/24/2022

1. Does the utility you represent presently have facilities within the project limits? X Yes ☐ No

2. What type of facilities do you have in the project area? ☐ Underground X Aboveground

3. Pole Owner: CMP

Attachments:

4. Do you plan on installing any facilities within the project limits in the next 5 years? ☐ Yes X No

6. Contact person for project coordination:
Name: Marty Pease
Address:
Tel:
Cell: 207-272-7993
Fax No:
E-mail: martin.pease@consolidated.com

6. Contact person for construction:
Name:
Address:
Tel:
Fax No:
E-mail:

7. Comments Aerial cable on Water St

** IMMEDIATE RESPONSE REQUESTED **

RE: 013344.10 Date

Town/City: Gardiner, ME
Project WIN: 013344.10
Location: Old Train Trestle Near Summer St

Utility Coordinator: Paul Pottle, PE
Stantec Consulting Services, Inc.
2211 Congress Street, Suite 380
Portland, ME 04102
Phone: 207-303-7435
E-Mail: paul.pottlejr@stantec.com

Please complete the following short questionnaire and fax, email or send via mail. The following may be filled out electronically in Microsoft Word by using the "TAB" key.

Utility: MaineCom **Date Form Submitted:** 1/21/22

1. **Does the utility you represent presently have facilities within the project limits?** ☐ Yes ☒ No
2. **What type of facilities do you have in the project area?** ☐ Underground ☐ Aboveground
3. **Pole Owner: CMP**

   Attachments:

4. **Do you plan on installing any facilities within the project limits in the next 5 years?** ☐ Yes ☒ No
6. **Contact person for project coordination:**
   Name:
   Address:
   Tel:
   Cell:
   Fax No:
   E-mail:
6. **Contact person for construction:**
   Name:
   Address:
   Tel:
   Fax No:
   E-mail:
7. **Comments No MaineCom**

**** IMMEDIATE RESPONSE REQUESTED ****

RE: 013344.10 Date

Town/City: Gardiner, ME
Project WIN: 013344.10
Location: Old Train Trestle Near Summer St

Utility Coordinator: Paul Pottle, PE
Stantec Consulting Services, Inc.
2211 Congress Street, Suite 380
Portland, ME 04102
Phone: 207-303-7435
E-Mail: paul.pottlejr@stantec.com

Please complete the following short questionnaire and fax, email or send via mail. The following may be filled out electronically in Microsoft Word by using the "TAB" key.

Utility: Firstlight Fiber **Date Form Submitted:** 1/21/2022

1. Does the utility you represent presently have facilities within the project limits? ☐ Yes X No

2. What type of facilities do you have in the project area? ☐ Underground ☐ Aboveground

3. Pole Owner:

Attachments:

4. Do you plan on installing any facilities within the project limits in the next 5 years? ☐ Yes X No

6. Contact person for project coordination:

Name: Michael Ellingwood
Address: 14 Resilient Circle Brunswick Me
Tel: 207-333-3471
Cell: 207-462-2759
Fax No: na
E-mail: mellingwood@firstlight.net

6. Contact person for construction:

Name:
Address:
Tel:
Fax No:
E-mail:

7. Comments

** IMMEDIATE RESPONSE REQUESTED **

RE: 013344.10 Date

Town/City: Gardiner, ME
Project WIN: 013344.10
Location: Old Train Trestle Near Summer St

Utility Coordinator: Paul Pottle, PE
Stantec Consulting Services, Inc.
2211 Congress Street, Suite 380
Portland, ME 04102
Phone: 207-303-7435
E-Mail: paul.pottlejr@stantec.com

Please complete the following short questionnaire and fax, email or send via mail. The following may be filled out electronically in Microsoft Word by using the "TAB" key.

Utility: Gardiner Water District **Date Form Submitted:** 01/20/2022

1. Does the utility you represent presently have facilities within the project limits? x☐ No

2. What type of facilities do you have in the project area? ☐ Underground ☐ Aboveground

3. Pole Owner:

Attachments:

4. Do you plan on installing any facilities within the project limits in the next 5 years? x☐ No

6. Contact person for project coordination:

Name: Paul Gray
Address: PO Box 536 Gardiner Me. 04345
Tel: 582-5500
Cell: 317-6562
Fax No: 582-3093
E-mail: paul.gray@roadrunner.com

6. Contact person for construction:

Name: Same
Address:
Tel:
Fax No:
E-mail:

7. Comments **We do have buried water mains on Water St on the edge of the pavement**

CMP (Letter 1 Not Filled Out)

Flanders, Lauren

From:	Laney, Timothy <Timothy.Laney@cmpco.com>
Sent:	Tuesday, January 25, 2022 9:29 AM
To:	Pottle, Paul
Cc:	Grard, Jeffrey R.
Subject:	RE: EXTERNAL:WIN 013344.00 City of Gardiner Multi-Use Path and Bridge Utility Letter 1
Attachments:	WIN013344_utility_letter1.docx

No facilities in project area.

Internal Use

From: Pottle, Paul <Paul.PottleJr@stantec.com>
Sent: Thursday, January 20, 2022 2:25 PM
To: Laney, Timothy <Timothy.Laney@cmpco.com>; dlpormeconstleadership@charter.com; MDOT_Requests <MDOT_Requests@fairpoint.com>; Paul Gray <paul.gray@roadrunner.com>; capplebee@gardinermaine.com; matwater@tilsontech.com; mellingwood@firstlight.net; Garth Vdoviak <GVdoviak@summitnaturalgas.com>
Cc: Meek, Lauren <Lauren.Meek@stantec.com>; Jerry Douglass <jdouglass@gardinermaine.com>
Subject: EXTERNAL:WIN 013344.00 City of Gardiner Multi-Use Path and Bridge Utility Letter 1

Hello,

The City of Gardiner is in the planning and development stage of a new Multi-Use Path and Bridge along a section of the Cobbossee Stream from Summer Street to Water Street. Attached, please find a utility coordination letter, brief questionnaire and project location map.

Please complete and return the questionnaire within 2 weeks of receipt.

If you have any questions or concerns, please feel free to contact me. Thank you for your cooperation.

Sincerely,

Paul Pottle P.E.
Senior Transportation Engineer

Direct: 207 887-3513
Mobile: 207 303-7435
Paul.PottleJr@stantec.com

Stantec
2211 Congress Street Suite 380
Portland ME 04102-1955

Appendix G GEOTECHNICAL ENGINEERING REPORT

Geotechnical Engineering Report

Cobbossee Stream Crossing
Cobbossee Trail Extension
Gardiner, Maine

February 25, 2022

Prepared for:

City of Gardiner, Maine

Prepared by:

Stantec Consulting Services, Inc.
5 Dartmouth Drive, Suite 200
Auburn, New Hampshire 03032

Revision	Description	Author		Quality Check		Independent Review	
0	Report	T. Dykstra	2/21	B. Foley	2/23	L. Gillen-Hughes	2/23

This document entitled Geotechnical Engineering Report was prepared by Stantec Consulting Services Inc. (“Stantec”) for the account of City of Gardiner, Maine (the “Client”). Any reliance on this document by any third party is strictly prohibited. The material in it reflects Stantec’s professional judgment in light of the scope, schedule and other limitations stated in the document and in the contract between Stantec and the Client. The opinions in the document are based on conditions and information existing at the time the document was published and do not take into account any subsequent changes. In preparing the document, Stantec did not verify information supplied to it by others. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that Stantec shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party as a result of decisions made or actions taken based on this document.

(signature)

Trey Dykstra, PE
Associate/Geotechnical Engineer

Reviewed by ____
(signature)

Brian Foley, EIT
Geotechnical Designer

Approved by____
(signature)

Liam Gillen-Hughes
Geotechnical Designer

Table of Contents

LIST OF TABLES

FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

Stantec Consulting Services, Inc. (Stantec) has performed a geotechnical exploration and analysis for the design and construction of a pedestrian bridge to be constructed as a part of an extension to the Cobbossee Trail. The project involves extending the trail from Water Street (Route 9/126) near its intersection with Elm Street to the west end of Summer Street. The extension will generally follow an existing inactive rail line. The proposed alignment will require the construction of a bridge to carry the trail across the Cobbossee Stream. This report provides geotechnical recommendations for the proposed bridge over the Cobbossee Stream.

The project location is shown on Figure 1 – Site Location Plan. Elevations referenced in this report are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

The geotechnical design recommendations contained in this report have been developed in accordance with the following:

- AASHTO, Load Resistance Factor Design (AASHTO LRFD) Bridge Design Specifications, Ninth Edition, 2020; and
- Maine Department of Transportation, Bridge Design Guide (BDG), 2003 Edition with 2014 Updates

1.1 SITE DESCRIPTION

The site is located approximately 2,500 feet from the confluence of the Cobbossee Stream and the Kennebec River in Gardiner, Maine. The Cobbossee Stream generally flows through the project area in a west to east direction. The north side of the project area is located on a river terrace and is generally wooded. The south side of the project is lower in elevation and is wooded. The existing railroad trestle is located to the west of the proposed pedestrian bridge. The surrounding area is generally developed with residential or commercial development. The 100-year flood is El. 39.2 in the area of the proposed bridge.

1.2 PROJECT DESCRIPTION

Two options for the stream crossing are currently being considered. This report addresses option 2, which is located south (downstream) of the existing railroad trestle. The proposed structure will be a single-span bridge, with an anticipated length of 185 feet and approximately 12 feet wide. The bridge is expected to be a steel truss with an IPE decking. The south and north abutments are expected to be located at Sta 4408+30 and Sta 4410+15, respectively. In the area of the south abutment the ground surface slopes downward towards the river at a grade of 2.5H:1V from approximately El. 40 to El. 31. The north abutment is located on a river terrace. The ground surface slopes downward towards the river at a grade of 1.3 H:1V from El. 53 to El. 31. An existing bin wall is located at the north abutment of the existing railroad trestle. We understand the bin wall will remain in place and the proposed north abutment will be founded behind the bin wall. We understand the design plans for the bin wall are not available and therefore the exact construction of the wall is not known.

Because the boring (B-1) for the north abutment was drilled 55 feet from the proposed north abutment location, the soil conditions encountered at B-1 may differ from the conditions at the proposed abutment location. The soil conditions at the proposed abutment location will likely contain fill associated with the construction of the existing bin wall. Fill was not encountered at the B-1 location. Therefore, once the location of the north abutment has been finalized, we recommend that a boring be drilled to fully evaluated the conditions below the proposed abutment footing.

2.0 SUBSURFACE CONDITIONS

Subsurface conditions for the site were evaluated by using published information and conducting a test boring program. Our findings are provided in the paragraphs below.

2.1 LOCAL GEOLOGY

The site is located near the confluence of Cobbossee Stream and the Kennebec River. The surficial soils in the project area are mapped on the Surficial Geology, Gardiner Quadrangle, Maine, Maine Geological Survey Open File No. 09-8, 2009. The surficial soil in the general area of the site is the Presumpscot Formation which consists of silt, clay and sand sized particles deposited in deep ocean water. The map indicates that the surficial soil in the area of the north abutment is a terrace consisting of sand and gravel. Glacial till is also mapped at the ground surface in the area of the project.

Based on a map in the publication entitled “Bedrock Geology of Gardiner 15’ Quadrangle, Maine” Maine Geological Survey Open File No. 84-8, 1984 the bedrock is mapped as Silurian-Ordovician age rocks of the Vassalboro Formation. The composition of the bedrock is noted as biotite granofels.

2.2 TEST BORING PROGRAM

The subsurface investigation program consisted of two test borings drilled by New England Boring Contractors of Derry, New Hampshire between October 25 to 28, 2021. The as-drilled boring locations are presented on Figure 2 – Boring Location Plan. The test borings were observed and logged by a Stantec geotechnical engineer. The soil samples were visually classified in the field in accordance with the Burmister soil classification system. Details of drilling and sampling methods are indicated on the borehole logs presented in Appendix A of this report.

The borings were drilled with an ATV mounted Mobile B-57 drill rig equipped with 4-inch steel casing. Soil samples were obtained by driving a 24-inch long, 2-inch outside diameter split spoon sampler with a 140-pound automatic hammer falling 30 inches, in substantial accordance with ASTM D1586, the Standard Penetration Test (SPT). The blows for each 6-inches of penetration are recorded for a total of 24-inches. Because the SPT was conducted using an automatic hammer, an energy correction factor of 1.3 was used to correct the raw blow counts. Rock core samples were obtained using a NX double-walled core barrel.

Table 1 – Boring Locations and Elevations

Boring	Station	Offset (ft)	Abutment	Ground Elev. (ft)	Top of Bedrock	
					Depth (ft)	Elev. (ft)
B-2	4408 + 30	0	South	39	69.8	-30.8
B-1	4410 + 14	0	North	52	NE	NE

NE = Not Encountered

2.3 SOUTH ABUTMENT

Test boring B-2 was drilled at the proposed location of the south abutment. The subsurface conditions encountered at the boring are summarized in the paragraphs below. In general, the test boring encountered alluvium, upper glacial till, outwash and bedrock.

2.3.1 Alluvial

An alluvium deposit was encountered from the ground surface to a depth of 5 feet below the ground surface. The deposit was described as gray/black, medium to fine sand, some silt. The recorded N-value was 4 indicating a loose consistency.

2.3.2 Upper Glacial Till

This deposit was encountered from a depth of 5 to 30 feet below the ground surface. The deposit was generally described as gray, medium to fine sand, some coarse to fine gravel, some silt, trace clay. Due to the depositional nature of glacial till cobbles and boulders are to be expected within this deposit. The recorded N-values ranged from 80 to greater than 100 bpf, indicating a very dense consistency.

2.3.3 Outwash

An outwash deposit was encountered from a depth of 30 to 69.8 feet below the ground surface. The deposit was generally described as gray, medium to fine sand, little silt. The recorded N-values ranged from 59 to 77 bpf, indicating a very dense consistency.

2.3.4 Bedrock

The top of bedrock was encountered at 69.8 feet below the ground surface. The bedrock was cored in three runs from 70.9 to 82.8 feet. The bedrock was described moderately hard, fresh, gray to white, medium to fine grained Biotite Granofels. The joints are low angle, very close, rough, and tight to open. The rock Quality Designation ranged from 7 to 23 percent, indicating a very poor quality. A photograph of the rock cores is provided in Appendix B.

2.4 NORTH ABUTMENT

Test boring B-1 was drilled approximately 55 feet south of the proposed location of the north abutment. At the time the boring was drilled, the north abutment was to be located south of the current proposed location. The subsurface conditions encountered at the boring are summarized in the paragraphs below. In general, the test boring encountered a surficial layer of silty clay overlying upper glacial till, outwash and lower glacial till.

2.4.1 Marine Deposit

A surficial marine deposit was encountered from the ground surface to a depth of 4 feet. The deposit was described as gray/brown, silty clay, some fine sand. The recorded N-values were 5 and 35 indicating a medium stiff and hard consistency.

2.4.2 Upper Glacial Till

This deposit was encountered from a depth of 4 to 57 feet below the ground surface. The deposit was generally described as brown or gray/brown, medium to fine sand, some silt, some coarse to fine gravel. A boulder was cored from 34 to 37 feet below the ground surface. Due to the depositional nature of glacial till cobbles and boulders are to be expected within this deposit. The recorded N-values ranged from 45 to greater than 100 bpf, indicating a dense to very dense consistency. The majority of the deposit has a very dense consistency.

2.4.3 Outwash

An outwash deposit was encountered from a depth of 57 to 80 feet below the ground surface. The deposit was generally described as gray, medium to fine sand, little fine gravel, little to trace silt. The recorded N-values ranged from 62 to 74 bpf, indicating a very dense consistency.

2.4.4 Lower Glacial Till

This deposit was encountered from a depth of 80 to 92 feet below the ground surface. The deposit was generally described as brown or gray, medium to fine sand, some coarse to fine gravel, some silt. Due to the depositional nature of glacial till cobbles and boulders are to be expected within this deposit. The recorded N-value was 88 bpf, indicating a very dense consistency.

2.4.5 Bedrock

Bedrock was not encountered within the depth of the test boring.

2.5 GROUNDWATER

Due to the proximity to the stream channel, groundwater levels in the area of the abutments are expected to generally coincide with the water level in the stream channel. The stream channel water level and groundwater levels will vary over time due to seasonal changes in precipitation and temperature, snowmelt, and surrounding and on-site drainage characteristics. Based on observations made during the drilling program the water in the channel was at approximately El. 31.

3.0 LABORATORY TESTING

Geotechnical laboratory tests were conducted on representative soil samples obtained from the test borings to assist in classification, evaluate engineering properties and evaluate corrosion potential. Laboratory testing was conducted by GeoTesting Express of Acton, MA or TEI Testing services under contract to GeoTesting Express. Results of the soil tests are included in Appendix C and are summarized in the tables below.

3.1 SOIL TEST RESULTS

Geotechnical soil testing consisted of grain size distribution and moisture content, which were conducted in accordance with ASTM D2216 and ASTM D6913, respectively.

Table 2 – Summary Grain Size Distribution Testing

Boring/ Sample No.	Depth (feet)	Soil Description	Moisture Content (%)	Gravel (%)	Sand (%)	Fines (%)
B-1/S-4	8-10	Medium to fine SAND, some Silt, little fine Gravel	13.0	16.8	51.8	31.4
B-1/S-10	30-31.1	Fine SAND and Silt, some coarse to fine Gravel	9.1	22.4	38.9	38.7
B-2/S-5	9-10.8	Medium to fine SAND and Silt, little fine Gravel	8.9	15.6	45.3	39.1
B-2/S-12	45-47	Medium to fine SAND, trace Silt, trace fine Gravel	17.0	3.5	91.8	4.7

3.2 BEDROCK TEST RESULTS

Geotechnical bedrock testing consisted of bulk density and compressive strength, which were conducted in accordance with ASTM D7012 Method C.

Table 3 - Bedrock Laboratory Testing Summary

Boring	Core Run	Approx. Elevation (ft)	Rock Type	Bulk Density (lb/ft^3)	Failure Type	Compressive Strength (lb/in^2)
B-2	C-1	70.9	71.6	174	Intact	25,355

3.3 CORROSION TEST RESULTS

Corrosion testing consisted of pH, water-soluble chloride and water-soluble sulfate, which were conducted in accordance with ASTM D4972, AASHTO T-291-18 and AASHTO T-290-20, respectively.

Table 4 – Summary of Corrosion Testing

Boring No.	Sample No.	Depth (feet)	pH of Soil in Distilled Water	pH of Soil in Calcium Chloride	Chloride(1) (mg/kg)	Sulfate(1) (mg/kg)
B-1	S-3	5 – 6.3	5.9	5.5	19	< 10
B-2	S-4	7 – 8.3	7.2	6.8	11	< 10

Notes: (1) Detection limit is 10 mg/kg.

4.0 DISCUSSIONS AND RECOMMENDATIONS

Our recommendations provided below are based upon the subsurface information obtained, laboratory test data, and our understanding of the proposed construction. The following sections provide recommendations for the proposed bridge pier and abutment foundations. Results of our slope stability analysis are also presented in the sections below.

4.1 PRELIMINARY STRUCTURAL LOADS

Preliminary structural loads were provided for the abutments to be used in the geotechnical analyses. This information is provided below.

Table 5 – Summary of Structural Loads

Actual Footing Width (feet)	Service Condition		Strength Condition	
	Maximum Pressure (ksf)	Effective Footing Width (feet)	Maximum Pressure (ksf)	Effective Footing Width (feet)
12	3.43	9.63	4.73	8.74

4.2 SOIL ENGINEERING PARAMETERS

Engineering parameters have been developed for the soils at the site based on the test boring program and laboratory test results. The table below provides the soil engineering parameters used in our analyses.

Table 6 – Summary of Soil Strength Parameters

Soil Stratum	Unit Weight, γ_m (pcf)	Drained Conditions		Undrained Conditions	
		Effective Friction Angle, φ' (Degrees)	Cohesion, c' (psf)	Friction Angle, φ (Degrees)	Cohesion, c (psf)
Silty Clay	120	28	0	0	2000
Alluvium	110	29	0	29	0
Upper Glacial Till	135	38	0	38	0
Outwash	130	36	0	36	0
Lower Glacial Till	135	38	0	38	0

4.3 FOUNDATION RECOMMENDATIONS

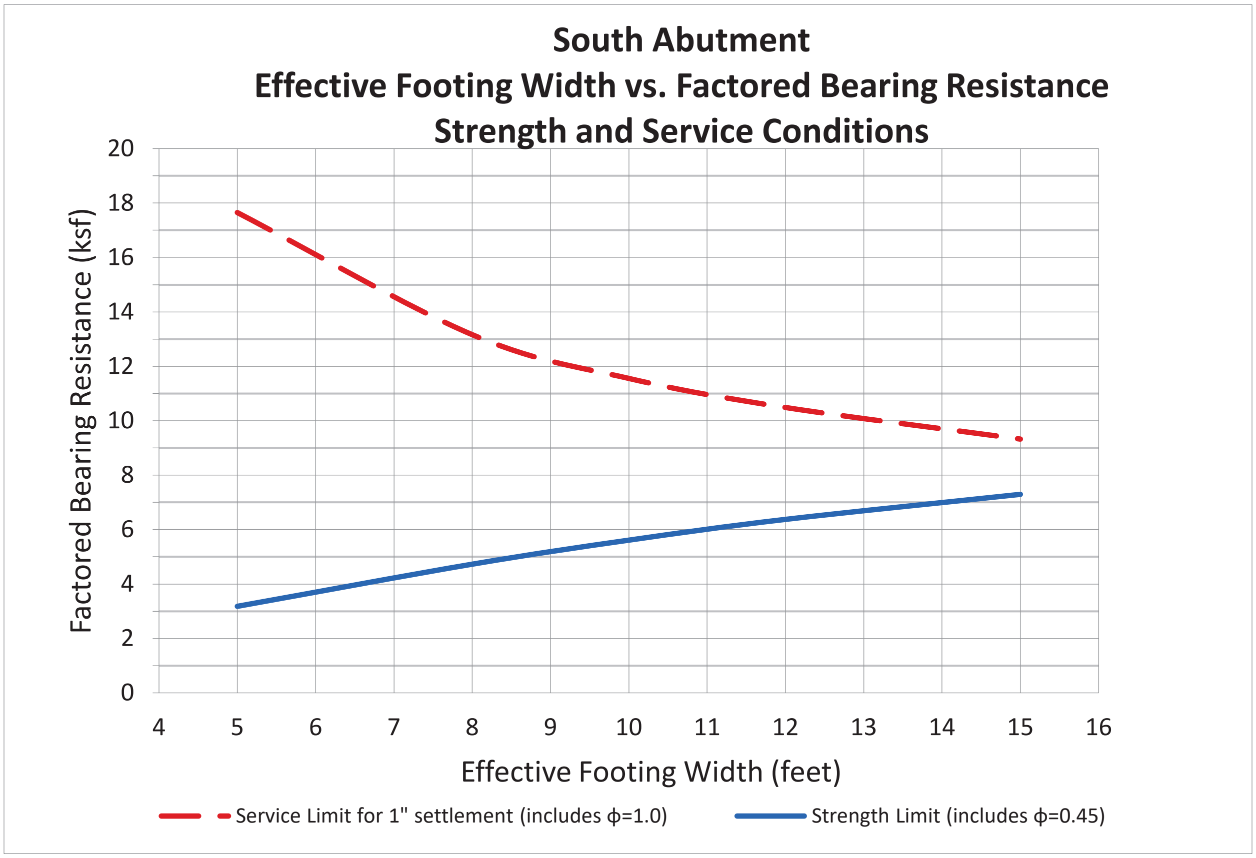
The proposed bridge will be supported on abutments located on the south and north sides of the river. Bearing resistances for the south and north abutments are provided in the paragraphs and sections below.

The bearing resistance for the footings should be evaluated at the service and strength limit states using the figures below. Section C10.6.2.1 of the AASHTO LRFD 2021 indicates the design of footings is frequently controlled by settlement at the service limit state. However, when the abutments are located at the top of a slope or within a slope the nominal bearing resistance for the strength limit state must be reduced to account for the sloping ground surface. The nominal bearing resistance for each abutment have been reduced in accordance with AASHTO LRFD 2021 Section 10.6.3.1.2c. The result is the strength limit state will likely control the size of the footing.

The service limit state resistance presented in figures below is based on a maximum settlement of 1 inch and includes a resistance factor (φb) equal to 1.0. The strength limit state resistance shown in the figures includes a resistance factor (φb) equal to 0.45 and have been reduced for the sloping ground surface. The vertical bearing pressure should be calculated assuming a uniformly distributed pressure over an effective base is as shown in LRFD Figure 11.6.3.2-1. The footing widths shown on the figures is the effective footing widths.

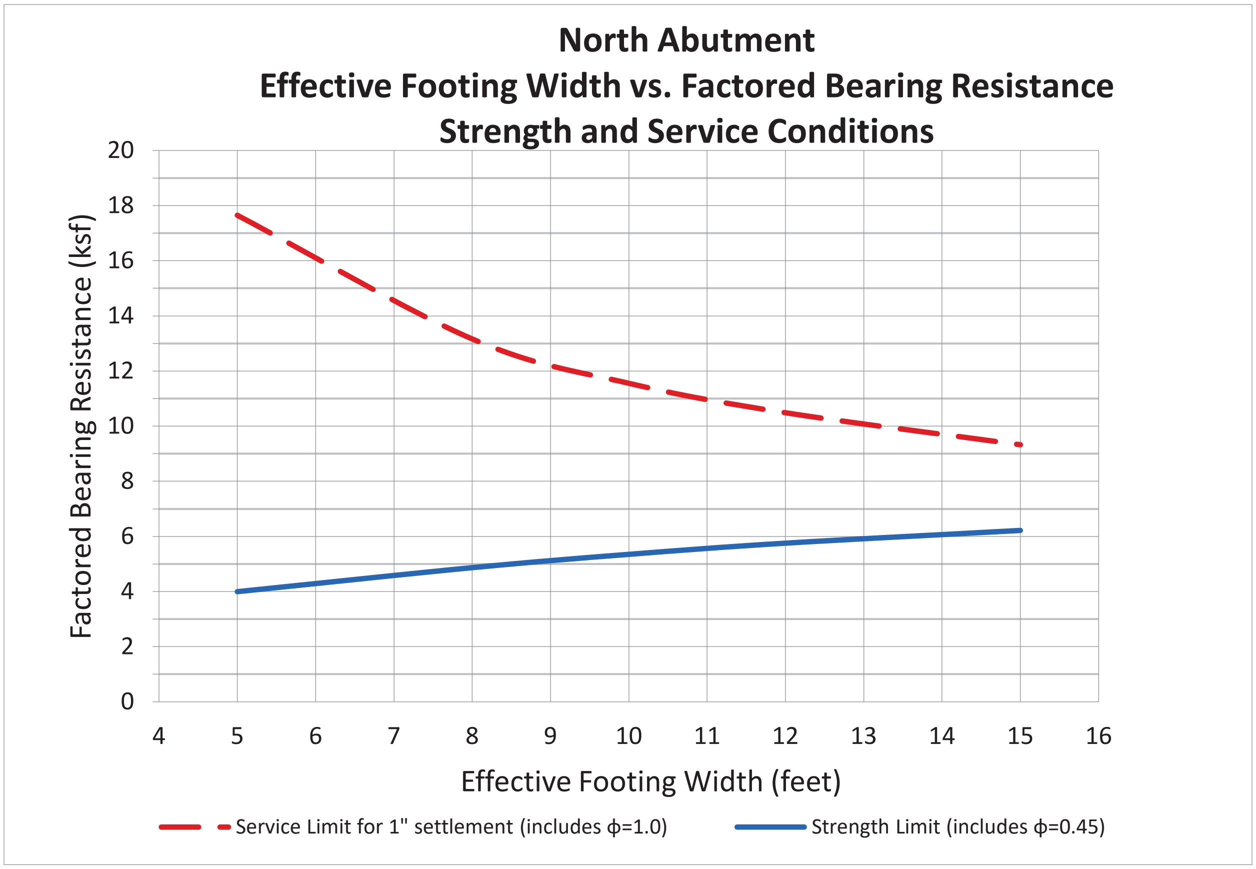
4.3.1 South Abutment

We anticipate the bottom of the proposed abutment footing will bear at approximately El. 32. Based on the subsurface conditions encountered in B-2 the proposed footing will bear on very dense glacial till. Based on the proposed footing location, footing width and slope geometry, the reduction coefficient for bearing resistance (RC_{BC}) is 0.60.



4.3.2 North Abutment

We anticipate the bottom of the proposed abutment footing will bear at approximately El. 40. Based on the subsurface conditions encountered in B-1, it is anticipated the proposed footing will bear on or just above very dense glacial till. Based on the proposed footing location, footing width and slope geometry, the reduction coefficient for bearing resistance (RC_{BC}) is 0.39.



4.3.3 Lateral Earth Pressures

The following recommendations are for the design of the bridge abutments and wing walls:

- Abutment and wingwalls that are free to rotate at the top should be designed based on active earth pressure (K_a) and compacted Gravel Borrow backfill. Assuming the ground surface behind the walls will be level and the Rankine theory, we recommend using K_a equal to 0.28. The unit weight for the Gravel Borrow is 135 pounds per cubic foot (pcf). The use of the Rankine theory in this case is conservative.
- The walls should be designed for a live load surcharge equivalent to the earth fill height summarized in AASHTO LRFD Tables 3.11.6.4-1 and 3.11.6.4-2.

- For calculating nominal sliding resistance for cast-in-place concrete footings bearing on the naturally deposited glacial till we recommend the values in the table below. We have also provided values for Gravel Borrow, for the case where gravel Borrow is placed between the glacial till and footing. The nominal passive resistance (R_{ep}) for soil in front of the retaining walls should be ignored.

Table 7 – Summary Sliding Factors

Footing Subgrade	Coefficient of Friction (Tanδ) BDG Table 3-3	Resistance Factor Strength Limit State (φ_s) BDG Table 5-3	Resistance Factor Extreme Limit State
Glacial Till	0.40	0.80	1.0
Gravel Borrow	0.50	0.80	1.0

4.3.4 Seismic Design Parameters

We have developed the following seismic design parameters based upon subsurface conditions encountered in the test borings, LRFD AASHTO Section 3.10 and the U.S. Seismic Design Maps software on the USGS website:

Table 8 – Summary of Seismic Parameters

Parameter	Value	Reference
Site Class (Stiff Soil) profile, based on the average N-value for the upper 100 feet of soil profile between 15 and 50 bpf.)	Site Class “D”	Table 3.10.3.1
Peak Ground Acceleration (PGA)	0.077 g	Figure 3.10.2.1-1
Acceleration Coefficient (A_s)	0.124 g	Equation 3.10.4.2-2
Spectral response acceleration at 0.2-second (Ss)	0.160 g	Figure 3.10.2.1-2
Design spectral acceleration at 0.2-second (S_{DS})	0.256 g	Equation 3.10.4.2-3
Spectral response acceleration at 1.0-second (S1)	0.045 g	Figure 3.10.2.1-3
Design spectral acceleration at 1.0-second (S_{D1})	0.108 g	Equation 3.10.4.2-6
Seismic Zone, based on a $S_{D1} < 0.15$ g	Seismic Zone 1	Table 3.10.6-1

4.3.5 Liquefaction Analysis

Liquefaction is a condition when a soil undergoes continued deformation during the course of cyclic stress applications induced by an earthquake where pore water pressure becomes equal to the confining pressure (e.g. effective stress approaches zero) and large deformations occur. Significant factors influencing liquefaction include grain size distribution of sand, fines content, in-situ density, and vibration characteristics (e.g. design earthquake and acceleration coefficient). Liquefaction generally occurs in saturated, relatively loose (N values less than 15 bpf) sandy soils with low fines content (less than 30 percent). Based on the dense to very dense consistency of the glacial till and glacial outwash the soils are not considered to be susceptible to liquefaction.

4.3.6 Frost Depths

Foundations placed on soil should be founded below the frost depth at the site. The frost depth at the site was estimated using the method provided in Section 5.2.1 of the BDG. Based on Figure 5-1, the design freezing index is estimated to be 1550 F-degrees days. The soil is considered to be coarse grained but has a significant amount of fine-grained soil. The moisture content for the soil near the foundation ranges between 8.9 to 13 percent. Using Table 5-1, the estimated frost penetration will range from 4.6 to 6.4 feet. A design frost depth of 5.5 feet is recommended for foundations at this site.

4.3.7 Embankment Settlement

On the south side of the project approximately 5 feet of embankment fill will be placed to achieve the final grade. Based on the subsurface conditions encountered at B-2 the soils in the area of the proposed embankment fill are granular. Settlement due to the embankment fill will occur as the embankment fill is placed. Long term consolidation settlement is not expected to occur. On the south side of the project the proposed grade changes are less than 2 feet and the settlement is expected to be negligeable.

4.3.8 Stability Analysis

The global stability of the proposed geometry was analyzed using the computer program Slope/W which is part of the GeoStudio Suite of programs. The stability analyses were conducted along the trail alignment at the location of the south and north abutments. The proposed geometries were modelled using a slope in front of the south and north abutments of 2H:1V and 1.7H:1V, respectively. The embankment and abutment slopes were both analyzed for static and seismic conditions. The seismic condition included a horizontal seismic force of 0.062g, which corresponds to 50 percent of the Acceleration Coefficient (A_s). A surcharge of 100 psf was used to model the load along path. The load from the bridge was modelled using a surcharge of 3.5 ksf applied across the effective width of the footing. The 3.5 ksf surcharge corresponds to the maximum bearing resistance for the service condition.

Based on LRFD Article 11.6.2.3, slopes that support a structure require a factor of safety (FOS) greater than 1.5 for static conditions. This is consistent with the criteria in Section 5.9.2 of the BDG. For seismic condition a FOS of greater than 1.0 is acceptable. The results of the stability analysis are presented in the table below. In each case the calculated Factor of Safety (FOS) is greater than the required FOS for both static and seismic conditions. The stability calculations are presented in Appendix D.

Table 9 – Slope Stability Summary

Stability Location	Static Condition		Seismic Condition	
	Required FOS	Actual FOS	Required FOS	Actual FOS
South Abutment	1.5	2.14	1.0	1.96
North Abutment	1.5	1.66	1.0	1.53

4.3.9 Corrosion Potential

Based on the results of the laboratory corrosion testing the naturally deposited soils are not expected to be corrosive to concrete or steel.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 PROTECTION OF EXISTING UTILITIES

Although not anticipated, above grade and below grade utilities lines located along the proposed bridge and trail alignment should be protected from damage during construction. Utilities lines that conflict with the proposed construction should temporarily and/or permanently rerouted.

5.2 TEMPORARY EARTH EXCAVATION SUPPORT

Unbraced excavations for foundation construction are anticipated to cave. Therefore, excavations will require temporary shoring. Based on the very dense soil conditions and likely presence of cobbles with the glacial till, we anticipate that driving steel sheet piles will not be feasible. Temporary shoring consisting of a soldier pile and lagging wall may be required, with the solider piles placed in a predrilled hole. Ultimately, the method of temporary earth support is the responsibility of the contractor.

All excavations should be conducted in accordance with current OSHA requirements under the observation and responsibility of the project contractors. Temporary earth support should be designed by a professional engineer licensed in the State of Maine and submitted for review. Excavation slopes and the area adjacent to temporary earth support systems should be checked regularly for signs of instability and flattened as required. Surface run-off should be directed away from excavation.

5.3 CONSTRUCTION DEWATERING

Excavations in the area of the abutments may encounter ground water. When encountered, the Contractor should be prepared to install a dewatering system capable of maintaining the groundwater depth at least 2 feet below the bottom of the excavation. Specifications should require that the Contractor divert surface water runoff away from excavations so that the glacial till subgrade does not become saturated. Precipitation that results in standing water in the excavation should be removed immediately

5.4 SUBGRADE PREPARATION

Once rough graded and immediately prior to placing backfill or concrete for the abutments, the subgrade should be proof-rolled under the direction of a geotechnical engineer to detect any weak or unstable areas that should be repaired prior to proceeding with further work. Proof-rolling should be performed with a minimum of six passes using a large vibratory plate compactor or small vibratory roller (e.g., trench roller). If the subgrade contains an elevated silt content or is within 2 feet of the ground water elevation, the proof compaction should be performed in static mode. Methods of repair of low strength, excessively dry or wet, and/or frozen soil are discussed under “Subgrade Stabilization”.

5.5 SUBGRADE STABILIZATION

The subgrade soils may require stabilization after exposure to construction traffic disturbance, and specifications should contain provisions for subgrade repair. The subgrade should be graded to promote positive runoff to a suitable drainage feature during construction. All excavations and exposed subgrade should be maintained in a moist, but unsaturated, condition throughout construction. The degree of subgrade disturbance will be dependent on the Contractor's means and methods, such as coordinating site activities around anticipated precipitation, and protecting exposed subgrade due to disturbance from excess moisture and construction equipment traffic.

Subgrade repair can include one or more of the following:

1. Scarification, moisture conditioning, and recompacting (sand/gravel soils only).
2. Over-excavation to a stable subgrade.
3. Partial over-excavation and stabilization with coarse graded aggregate and/or geotextile.

Every effort should be made to minimize disturbance of the on-site soils by construction traffic and surface runoff. The on-site soils are moisture sensitive and will deteriorate when subjected to repeated construction traffic and likely will require removal and replacement. The services of a geotechnical engineer should be retained to inspect soil conditions during construction and verify the suitability of the prepared foundation subgrade for support of the design loads.

5.6 BACKFILL, PLACEMENT, AND COMPACTION

Backfill materials should be comprised of clean soil and/or aggregate, free of organics, deleterious materials, ice, snow, and waste of any kind. One gradation test (AASHTO T-88) and one Modified Proctor (AASHTO T-180) test should be performed for each source of imported backfill. Excavated naturally deposited soils to be reused as fill will require multiple tests. When placing Structural Fill or reusing existing soils as backfill, the soil moisture content range should be ±2 percent of its optimum moisture content as determined by Modified Proctor. Backfill should be placed in uniform lifts not exceeding 12-inches loose thickness when using large vibratory rollers. When vibratory plate compactors are used for compaction the maximum loose lift thickness should be 6 inches. Within the zone of influence of the abutment footing the backfill should be compacted to at least 95% of the maximum dry density per AASHTO T-180. The percent compaction should be determined in the field using a nuclear density meter. A minimum of two in place density tests should be performed for each lift of fill placed.

5.7 MATERIAL REUSE

Stantec anticipates the excavated soils will have elevated silt content and will not be suitable for reuse as Gravel Borrow or Granular Borrow but, may be suitable for reuse as Common Borrow. Reuse of the existing fill materials will be contingent on careful inspection in the field by visual observation and/or test pit excavations prior to and during construction in accordance with the recommendations provided herein. Any deleterious materials and miscellaneous debris that may be encountered during excavation activities within the fill should be removed from the site.

On-site materials placed as fill should be sealed on a daily basis using a smooth drum roller to promote drainage and prevent ponding of storm water. Alternatively, imported fill materials may be used to attain the desired grades and expedite earthwork operations during wet weather periods.

5.8 WINTER CONSTRUCTION

Frozen subgrade should be removed and replaced with compacted Gravel Borrow. If excavation and backfilling operations are conducted during winter months, at the end of each day, the fill should be covered with a sacrificial 6-inch-thick layer of structural backfill that is to be stripped off at the start of the day to remove any hoar frost. The subgrade would then be compacted, and field density testing performed to ensure that the required compaction has been achieved. Optionally, the subgrade may be protected with insulated blankets and/or heated with circulated glycol lines to prevent the subgrade from freezing. Imported and/or on-site stockpiled backfill material should be covered with insulated blankets to minimize snow intrusion and/or rainfall infiltration. Any surficial frozen soil in the stockpile/borrow pit should be removed prior to placement in the work area.

5.9 BACKFILL TESTING

The project specifications should require the Contractor to provide test results provided by an approved soil testing laboratory along with a sample of the imported fill material or any on-site material proposed for reuse. The analyses of the proposed materials should include gradation (AASHTO T-88) and moisture-density relationships (AASHTO T-180) and be submitted for approval by the project Geotechnical Engineer. The placement of backfill should be monitored by a qualified soils technician to observe and make accurate records regarding proof-compaction operations of the subgrade prior to backfill placement, types of materials used, thickness of lifts, densities, percent compaction, type of compaction equipment and number of passes, etc.

5.10 EMBANKMENT CONSTRUCTION

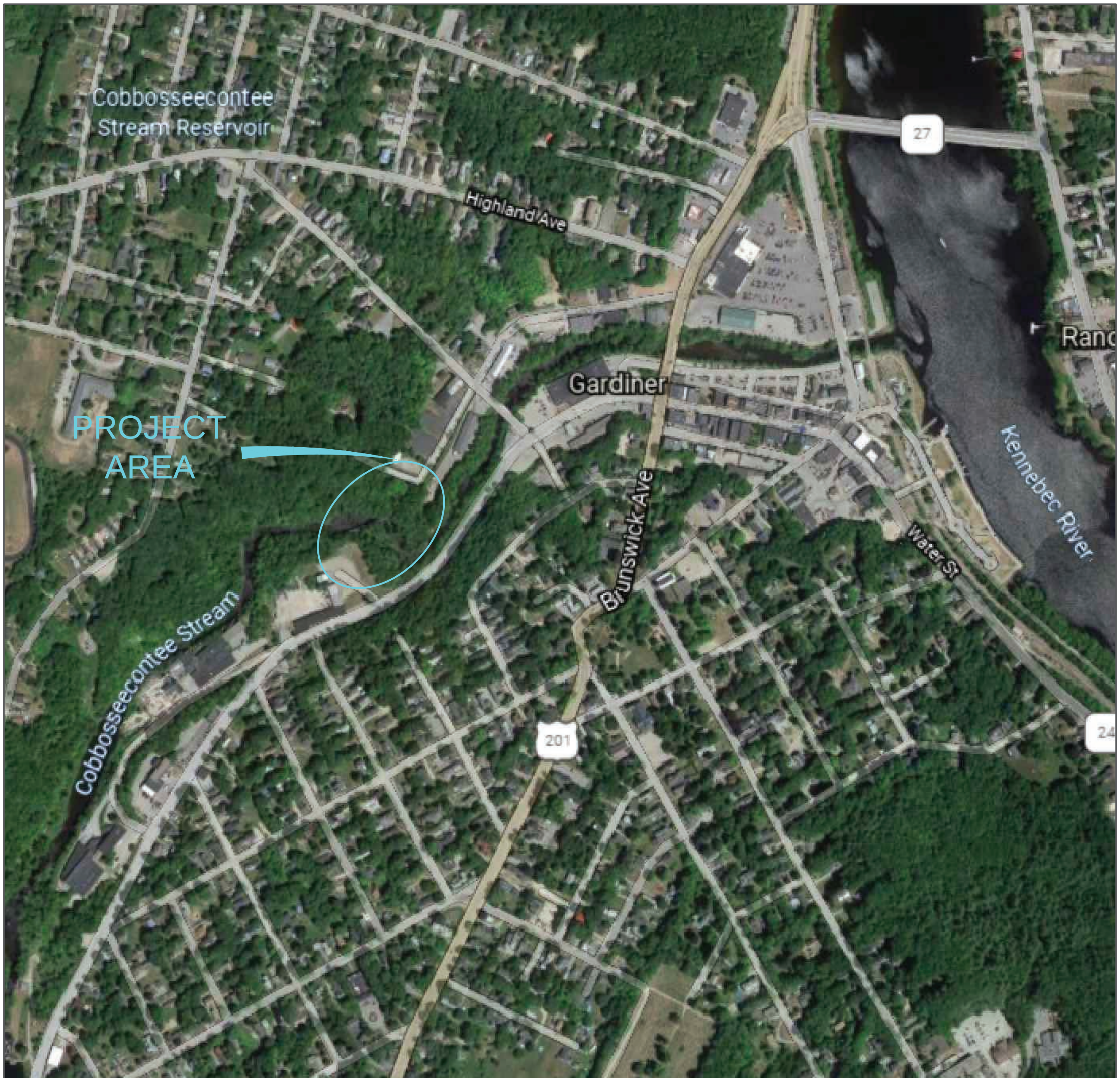
Construction of embankment slopes shall be conducted in accordance with MaineDOT Standard Specifications Section 203, Excavation and Embankment. Maximum lift thickness and minimum compaction requirement are provided in Section 203. The embankments should be constructed of soil meeting the requirements of MaineDOT Item No. 703.18, Common Borrow.

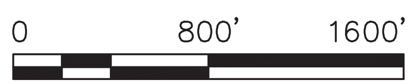
Prior to placing fill for embankment construction, existing vegetation, unsuitable existing fill materials, asphalt, topsoil and other organic or deleterious material should be removed to expose suitable subgrade soils. Where proposed slopes are constructed against existing slopes, the existing slope should be continuously benched by excavating steps into the existing slope in accordance with Standard Specification Section 203.09 of the MaineDOT Standard Specifications. The entire area of the new embankment should be placed in horizontal lifts and compacted. Unsuitable materials should not be wasted in the outer portion of fill slopes. Offsite waste disposal areas shall be established in accordance with Section 203.06 of the MaineDOT Standard Specifications.

We anticipate that slopes that are 2H:1V or flatter will be treated with loam and seed to provide long term erosion control. Short-term protection can be provided by utilizing temporary erosion control matting, MaineDOT Item No. 613.319. Slopes steeper than 2H:1V should be treated with a 4-inch thick geocell confinement system.

Unsuitable soils or soils that become disturbed during construction of the embankments should be completely excavated from the subgrade and replaced with compacted granular borrow. Granular borrow should conform to MaineDOT Standard Specification 703.19, Granular Borrow. The granular borrow should be compacted to 92 percent of the Modified Proctor maximum dry density (AASHTO T-180).

FIGURES





ORIGINAL SHEET - ANSI A

DECEMBER 9, 2021
195602201

Stantec

5 Dartmouth Drive, Suite 200
Auburn, NH
www.stantec.com

Client/Project

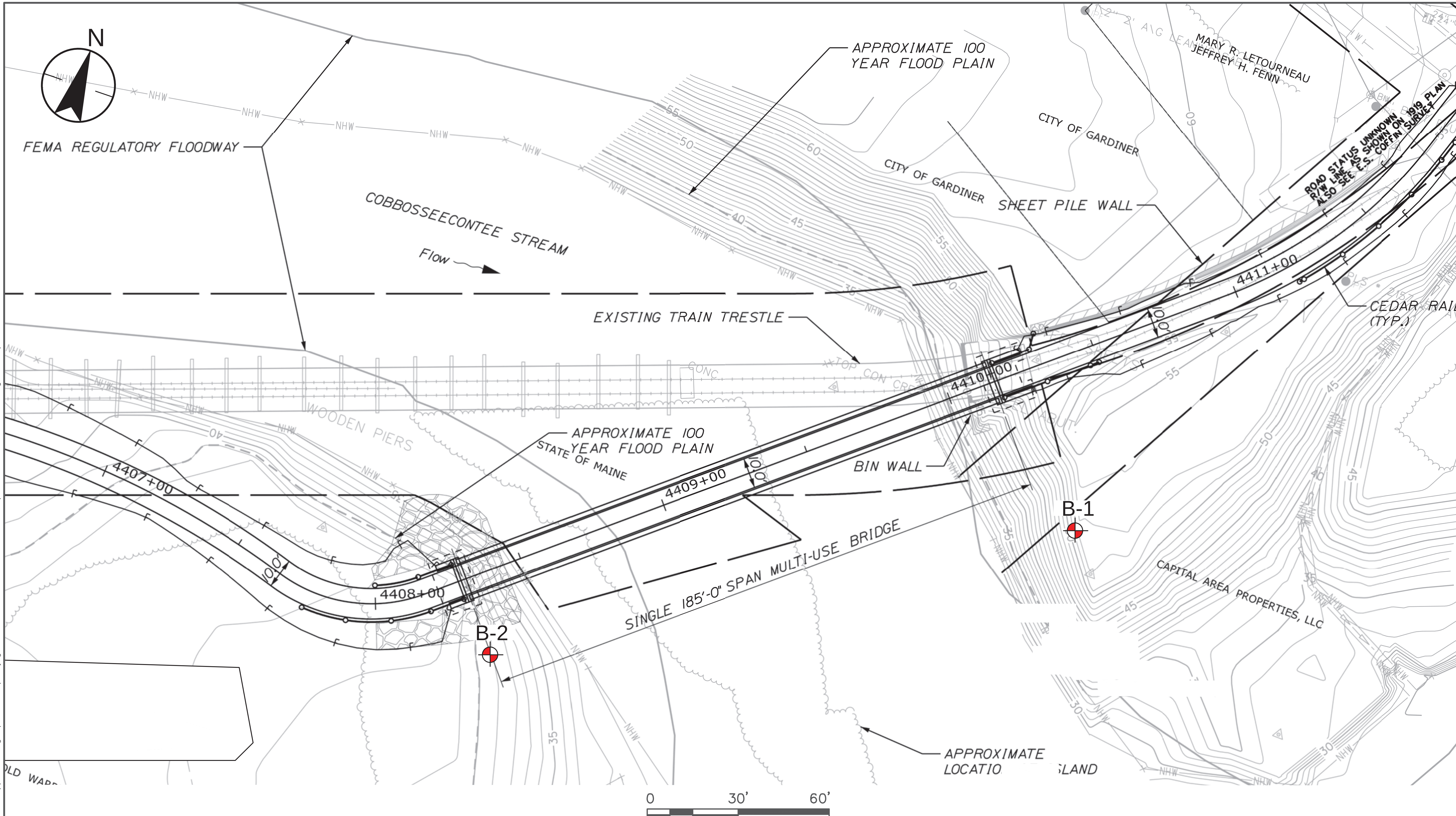
COBBOSSEE STREAM CROSSING
COBBOSSEE TRAIL EXTENSION
GARDINER, MAINE

Figure No.

1

Title

Site Locus Plan



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Stantec

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Tel: 603.669.8672
www.stantec.com

Notes

1. BORINGS B–1 THROUGH B–2 WERE DRILLED FROM OCTOBER 25 TO OCTOBER 28, 2021 BY NEW ENGLAND BORING CONTRACTORS OF HERMON, ME UNDER THE SUPERVISION OF STANTEC PERSONNEL.
2. LOCATIONS OF THE TEST BORINGS WERE DETERMINED BY MEASURING TO EXISTING SITE FEATURES AND ARE TAKEN TO BE APPROXIMATE.

Legend

B-1 LOCATION AND DESIGNATION OF BORINGS

Client/Project
COBBOSSEE STREAM CROSSING
COBBOSSEE TRAIL EXTENSION
GARDINER, MAINE

Project No.
179450405

Title
EXPLORATION LOCATION PLAN

Revision	Date
#0	2021.12.09

Reference Sheet	Figure No.
-	2

APPENDICES

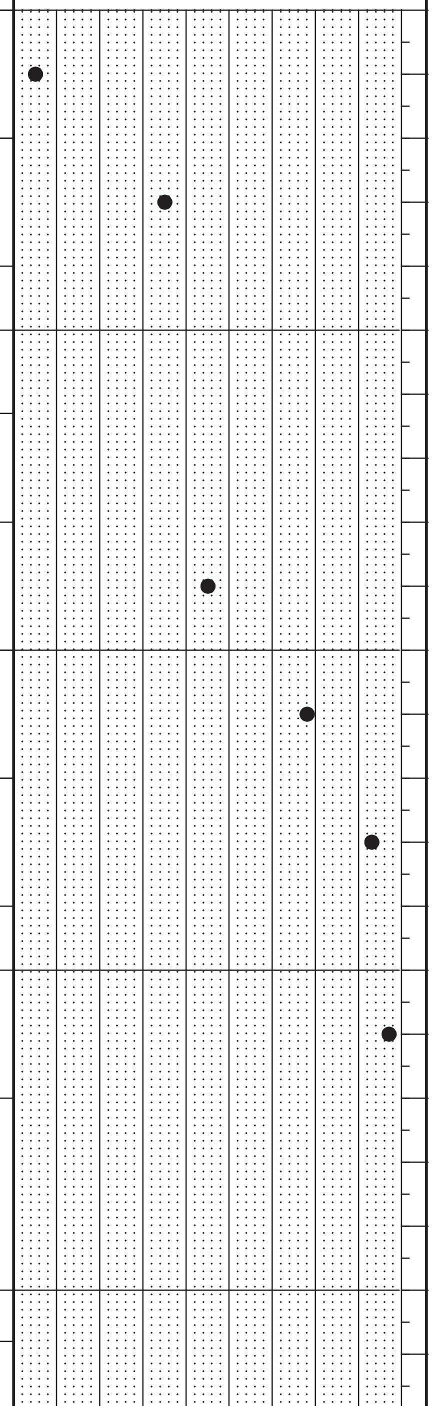
Appendix A – Test Boring Logs

Stantec BOREHOLE LOG B-1

CLIENT: City of Gardiner Maine
LOCATION: Cobbossee Stream Crossing, Gardiner, ME
EXPLORATION DATE: 10/25/2021 to 10/26/2021
GROUND EL.: 53
STATION:
OFFSET:
WATER LEVEL: 22
PROJECT No.: 17945405
EXPLORATION No.: B-1
DATUM: NAVD 88

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	TYPE	NUMBER	RECOVERY (in.)	SPT blows / 6"	SPT N-Value	SPT N(60) VALUE
0	53.0	Medium stiff, gray/brown, silty CLAY, some fine Sand.	SS	1	18	2 2 3 8	5	
	51.0	-SILTY CLAY- Hard, gray/brown, silty CLAY, some fine Sand.	SS	2	13	14 16 19 27	35	
	49.0							
5	48.0	Very Dense, gray/brown, medium to fine SAND, some fine gravel, someSilt.	SS	3	12	45 42 50/3"	R	
	46.7							
	45.0	Dense, gray/brown, medium to fine SAND, some fine gravel, some Silt.	SS	4	11	20 23 22 20	45	
10	43.0	Very Dense, gray/brown, medium to fine SAND, some fine gravel, someSilt.	SS	5	14	30 33 35 45	68	
	41.0	Very Dense, gray/brown, medium to fine SAND, some fine gravel, some Silt.	SS	6	15	38 35 48 45	83	
	39.0	-UPPER GLACIAL TILL-						
15	38.0	Very Dense, gray/brown, medium to fine SAND, some fine gravel, some Silt.	SS	7	16	45 48 39 30	87	
	36.0							
20	33.0	Very Dense, gray/brown, medium to fine SAND, some fine gravel, some Silt.	SS	8	9	60 50/3"	R	
	32.2							

Undrained Shear Strength - tsf: 1 2 3 4
Water Content & Atterberg Limits W_P W W_L
Dynamic Penetration Test, blows/foot ★
Standard Penetration Test, blows/foot ●
10 20 30 40 50 60 70 80 90



Driller: NEBC, Hermon, ME; Supervisor: Stantec: Liam Gillen-Hughes
Rig Type: Mobile B-57 ATV Rig; Hammer Type: Auto; 4-inch diameter casing, 2" Split Spoon Sampler

△ Unconfined Compression Test
☐ Field Vane Test ■ Remolded
✖ Pocket Penetrometer / Torvane

Continued Next Page

STN13-GEO-I-VOC BORING LOGS.GPJ JW NHP.GDT 11/29/21

Stantec BOREHOLE LOG B-1

CLIENT: City of Gardiner Maine
LOCATION: Cobbossee Stream Crossing, Gardiner, ME
EXPLORATION DATE: 10/25/2021 to 10/26/2021
STATION:
OFFSET:
GROUND EL.: 53
WATER LEVEL: 22
PROJECT No.: 17945405
EXPLORATION No.: B-1
DATUM: NAVD 88

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	NUMBER	RECOVERY (in.)	SPT blows / 6"	SPT N-Value	SPT N(60) VALUE
25	28.0 / 27.2	Very Dense, gray/brown, medium to fine SAND, some fine gravel, some Silt.	SS	9	10	37 50/4"	R	
30	23.0 / 21.7	Very Dense, gray/brown, medium to fine SAND, some fine gravel, someSilt.	SS	10	15	40 50 50/4"	R	
		-UPPER GLACIAL TILL-						
	19.0	Cored boulder from 34 to 37 feet.						
35	16.0							
	14.0 / 13.7	Very Dense, gray/brown, medium to fine SAND, some fine gravel, someSilt.	SS	11	2	50/4"	R	
40								
	9.0							

Undrained Shear Strength - tsf: 1 2 3 4
Water Content & Atterberg Limits: W_P W W_L
Dynamic Penetration Test, blows/foot ★
Standard Penetration Test, blows/foot ●
10 20 30 40 50 60 70 80 90

Driller: NEBC, Hermon, ME; Supervisor: Stantec: Liam Gillen-Hughes
Rig Type: Mobile B-57 ATV Rig; Hammer Type: Auto; 4-inch diameter casing, 2" Split Spoon Sampler

△ Unconfined Compression Test
☐ Field Vane Test ■ Remolded
✖ Pocket Penetrometer / Torvane

Continued Next Page

Stantec BOREHOLE LOG B-1

CLIENT: City of Gardiner Maine
LOCATION: Cobbossee Stream Crossing, Gardiner, ME
EXPLORATION DATE: 10/25/2021 to 10/26/2021
GROUND EL.: 53
STATION:
OFFSET:
WATER LEVEL: 22
PROJECT No.: 17945405
EXPLORATION No.: B-1
DATUM: NAVD 88

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (in.)	SPT blows / 6"	SPT N-Value	SPT N(60) VALUE
45	7.2	Very Dense, gray/brown, medium to fine SAND, some fine gravel, someSilt.			SS	12	13	40 50 60 50/3"	116	
	4.0									
50	2.0	Very Dense, gray/brown, medium to fine SAND, some fine gravel, someSilt.			SS	13	20	36 42 50 47	92	
		-UPPER GLACIAL TILL-								
55	-2.0	Very Dense, gray/brown, medium to fine SAND, some fine gravel, someSilt.			SS	14	15	30 40 38 41	78	
	-4.0									
60	-7.0	Very dense, medium to fine SAND, trace Silt.			SS	15	18	22 27 35 42	62	
	-9.0									
		-OUTWASH-								
65										

Undrained Shear Strength - tsf: 1 2 3 4

Water Content & Atterberg Limits W_P W W_L
Dynamic Penetration Test, blows/foot ★
Standard Penetration Test, blows/foot ●

10 20 30 40 50 60 70 80 90

Driller: NEBC, Hermon, ME; Supervisor: Stantec: Liam Gillen-Hughes
Rig Type: Mobile B-57 ATV Rig; Hammer Type: Auto; 4-inch diameter casing, 2" Split Spoon Sampler

△ Unconfined Compression Test
□ Field Vane Test ■ Remolded
✖ Pocket Penetrometer / Torvane

Continued Next Page

Stantec BOREHOLE LOG B-1

CLIENT	City of Gardiner Maine	STATION		PROJECT No.	17945405
LOCATION	Cobbossee Stream Crossing, Gardiner, ME	OFFSET		EXPLORATION No.	B-1
EXPLORATION DATE	10/25/2021 to 10/26/2021	GROUND EL. 53	WATER LEVEL 22	DATUM	NAVD 88

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (in.)	SPT blows / 6"	SPT N-Value	SPT N(60) VALUE	Standard Penetration Test, blows/foot
70	-17.0	Very dense, gray, medium to fine SAND, little fine Gravel, little Silt.			SS	16	18	25 30 44 45	74		● 74
	-19.0										
75											
80	-27.0	-LOWER GLACIAL TILL-									
85											

Undrained Shear Strength - tsf: 1, 2, 3, 4

Water Content & Atterberg Limits W_P W W_L

Dynamic Penetration Test, blows/foot ★

Standard Penetration Test, blows/foot ●

10 20 30 40 50 60 70 80 90

Driller: NEBC, Hermon, ME; Supervisor: Stantec: Liam Gillen-Hughes
Rig Type: Mobile B-57 ATV Rig; Hammer Type: Auto; 4-inch diameter casing, 2" Split Spoon Sampler

△ Unconfined Compression Test
□ Field Vane Test ■ Remolded
✖ Pocket Penetrometer / Torvane

Continued Next Page

Stantec **BOREHOLE LOG** **B-1**

CLIENT: City of Gardiner Maine | STATION: | PROJECT No.: 17945405

LOCATION: Cobbossee Stream Crossing, Gardiner, ME | OFFSET: | EXPLORATION No.: B-1

EXPLORATION DATE: 10/25/2021 to 10/26/2021 | GROUND EL.: 53 | WATER LEVEL: 22 | DATUM: NAVD 88

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (in.)	SPT blows / 6"	SPT N-Value	SPT N(60) VALUE
90	-37.0	Very dense, gray, medium to fine SAND, some fine Gravel, some Silt.			SS	17	14	33 41 47 38	88	
	-39.0									
95										
100										
105										
110										

Undrained Shear Strength - tsf: 1, 2, 3, 4

Water Content & Atterberg Limits (W_P, W, W_L)

Dynamic Penetration Test, blows/foot ★

Standard Penetration Test, blows/foot ●

10 20 30 40 50 60 70 80 90

Driller: NEBC, Hermon, ME; Supervisor: Stantec: Liam Gillen-Hughes

Rig Type: Mobile B-57 ATV Rig; Hammer Type: Auto; 4-inch diameter casing, 2" Split Spoon Sampler

△ Unconfined Compression Test

□ Field Vane Test ■ Remolded

✖ Pocket Penetrometer / Torvane

Stantec

BOREHOLE LOG

B-2

CLIENT City of Gardiner Maine | STATION | PROJECT No. 17945405

LOCATION Cobbossee Stream Crossing, Gardiner, ME | OFFSET | EXPLORATION No. B-2

EXPLORATION DATE 10/27/2021 to 10/28/2021 | GROUND EL. 39 | WATER LEVEL 8 | DATUM NAVD 88

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (in.)	SPT blows / 6"	SPT N-Value	SPT N(60) VALUE
	39.0									
0		Loose, dark gray/black, medium to fine SAND, some Silt.			SS	1		2 2 2 2	4	
	37.0	Loose, dark gray/black, medium to fine SAND, some Silt.			SS	2		2 2 2 3	4	
	35.0	-ALLUVIUM-								
5	34.0	Very Dense, dark brown, medium to fine SAND, some fine Gravel, some Silt.			SS	3		34 38 50/2"	R	
	32.8									
	32.0	Very dense, gray, medium to fine SAND, some fine Gravel, some Silt, little Clay.		▽	SS	4		25 33 50/3"	R	
	30.7									
	30.0	Very dense, gray, medium to fine SAND, some fine Gravel, some Silt, little Clay.			SS	5		42 55 50 50/4"	105	
10										
	28.2	-UPPER GLACIAL TILL-								
	25.0	Very dense, gray, medium to fine SAND, some fine Gravel, some Silt, little Clay.			SS	6		21 35 50/5"	R	
15	23.5									
	19.0									
20		Very dense, gray, medium to fine SAND, some fine Gravel, some Silt, little Clay.			SS	7		45 40 40 40	80	
	17.0									

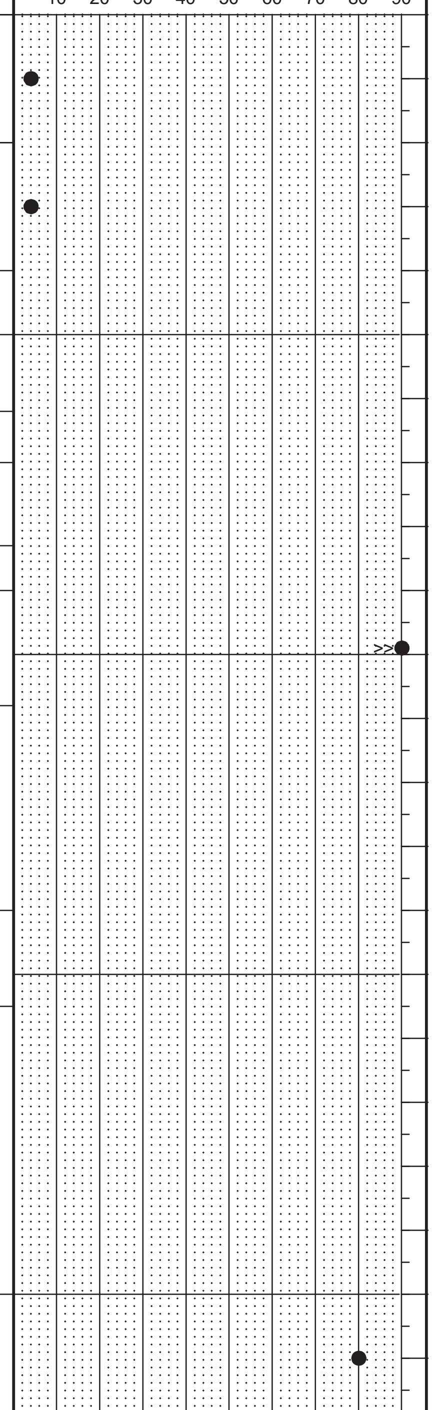
Undrained Shear Strength - tsf: 1 2 3 4

Water Content & Atterberg Limits W_P W W_L

Dynamic Penetration Test, blows/foot ★

Standard Penetration Test, blows/foot ●

10 20 30 40 50 60 70 80 90



Driller: NEBC, Hermon, ME; Supervisor: Stantec: Liam Gillen-Hughes

Rig Type: Mobile B-57 ATV Rig; Hammer Type: Auto; 4-inch diameter casing, 2" Split Spoon Sampler

△ Unconfined Compression Test

☐ Field Vane Test ■ Remolded

✖ Pocket Penetrometer / Torvane

Continued Next Page

Stantec BOREHOLE LOG B-2

CLIENT: City of Gardiner Maine
STATION:
PROJECT No.: 17945405
LOCATION: Cobbossee Stream Crossing, Gardiner, ME
OFFSET:
EXPLORATION No.: B-2
EXPLORATION DATE: 10/27/2021 to 10/28/2021
GROUND EL.: 39
WATER LEVEL: 8
DATUM: NAVD 88

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	TYPE	NUMBER	RECOVERY (in.)	SPT blows / 6"	SPT N-Value	SPT N(60) VALUE
25	14.0 / 13.2	Very dense, gray, medium to fine SAND, some fine Gravel, some Silt, little Clay.	SS	8		35 50/3"	R	
30	9.0 / 7.0	Very dense, gray, medium to fine SAND, little Silt.	SS	9		20 28 35 38	63	
35	4.0 / 2.0	Very dense, gray, medium to fine SAND, little Silt.	SS	10		24 35 42 49	77	
		-OUTWASH-						
40	-1.0 / -3.0	Very dense, gray, medium to fine SAND, little Silt.	SS	11		18 35 36 38	71	

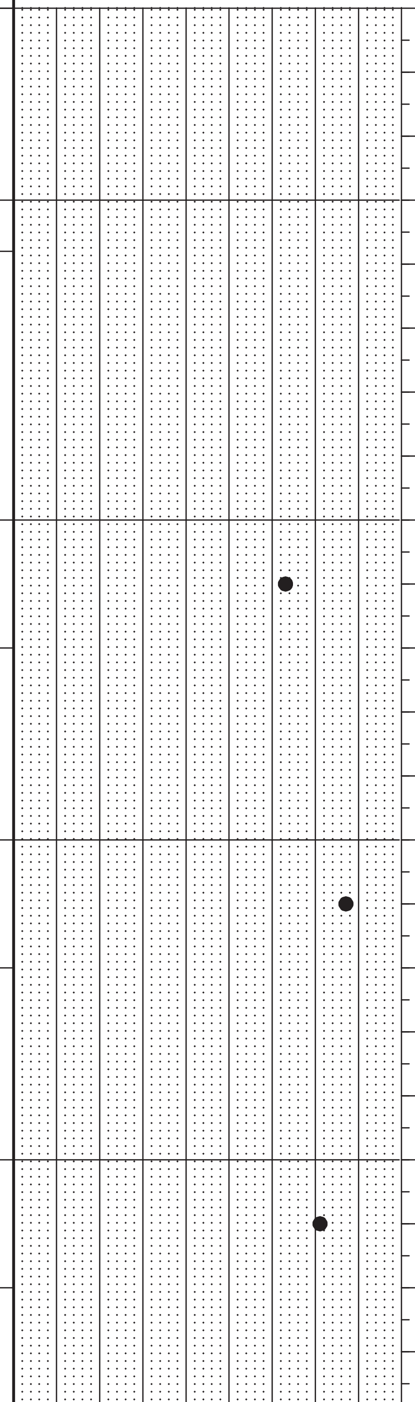
Undrained Shear Strength - tsf: 1 2 3 4

Water Content & Atterberg Limits: W_P W W_L

Dynamic Penetration Test, blows/foot ★

Standard Penetration Test, blows/foot ●

10 20 30 40 50 60 70 80 90



Driller: NEBC, Hermon, ME; Supervisor: Stantec: Liam Gillen-Hughes
Rig Type: Mobile B-57 ATV Rig; Hammer Type: Auto; 4-inch diameter casing, 2" Split Spoon Sampler

△ Unconfined Compression Test
□ Field Vane Test ■ Remolded
✖ Pocket Penetrometer / Torvane

Continued Next Page

Stantec BOREHOLE LOG B-2

CLIENT: City of Gardiner Maine
LOCATION: Cobbossee Stream Crossing, Gardiner, ME
EXPLORATION DATE: 10/27/2021 to 10/28/2021
GROUND EL.: 39
STATION:
OFFSET:
WATER LEVEL: 8
PROJECT No.: 17945405
EXPLORATION No.: B-2
DATUM: NAVD 88

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (in.)	SAMPLES SPT blows / 6"	SAMPLES SPT N-Value	SPT N(60) VALUE
45	-6.0	Very dense, gray, medium to fine SAND, little Silt.			SS	12		22 31 33 42	64	
	-8.0									
50	-11.0	Very dense, gray, medium to fine SAND, little Silt.			SS	13		15 25 34 47	59	
	-13.0	-OUTWASH-								
55	-16.0	Very dense, gray, medium to fine SAND, little Silt.			SS	14		18 22 38 49	60	
	-18.0									
60	-21.0	Very dense, gray, medium to fine SAND, little Silt.			SS	15		29 27 34 36	61	
	-23.0									
65										

Undrained Shear Strength - tsf: 1 2 3 4
Water Content & Atterberg Limits: W_P W W_L
Dynamic Penetration Test, blows/foot ★
Standard Penetration Test, blows/foot ●
10 20 30 40 50 60 70 80 90

Driller: NEBC, Hermon, ME; Supervisor: Stantec: Liam Gillen-Hughes
Rig Type: Mobile B-57 ATV Rig; Hammer Type: Auto; 4-inch diameter casing, 2" Split Spoon Sampler

△ Unconfined Compression Test
□ Field Vane Test ■ Remolded
✖ Pocket Penetrometer / Torvane

Continued Next Page

BOREHOLE LOG

B-2

CLIENT: City of Gardiner Maine
STATION:
PROJECT No.: 17945405

LOCATION: Cobbossee Stream Crossing, Gardiner, ME
OFFSET:
EXPLORATION No.: B-2

EXPLORATION DATE: 10/27/2021 to 10/28/2021
GROUND EL.: 39
WATER LEVEL: 8
DATUM: NAVD 88

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (in.)	SPT blows / 6"	SPT N-Value	SPT N(60) VALUE
	-30.8									
70		Advanced roller bit into bedrock from 69.8 to 70.9 feet.								
	-31.9	Moderately hard, fresh, gray to white, medium to fine grained Biotite Granofels. Joints are low angle, very close, rough and tight. RQD = 17 %.			C	1	27	min/ft 3 2.5 2.5 2		
75	-35.9	Moderately hard, fresh, gray to white, medium to fine grained Biotite Granofels. Joints are low angle, very close, rough and tight. RQD = 23%. -VASSALBORO FORMATION-			C	2	32	min/ft 2 2.5 2.5		
80	-38.8	Moderately hard, fresh, gray to white, medium to fine grained Biotite Granofels. Joints are low angle, very close, rough and tight. RQD = 7%.			C	3	60	min/ft 2 2.5 2.5 2 2		
	-43.8	Bottom of boring at 82.8 feet below ground surface. Terminated in bedrock.								
85										

Undrained Shear Strength - tsf: 1 2 3 4

Water Content & Atterberg Limits W_P W W_L

Dynamic Penetration Test, blows/foot ★

Standard Penetration Test, blows/foot ●

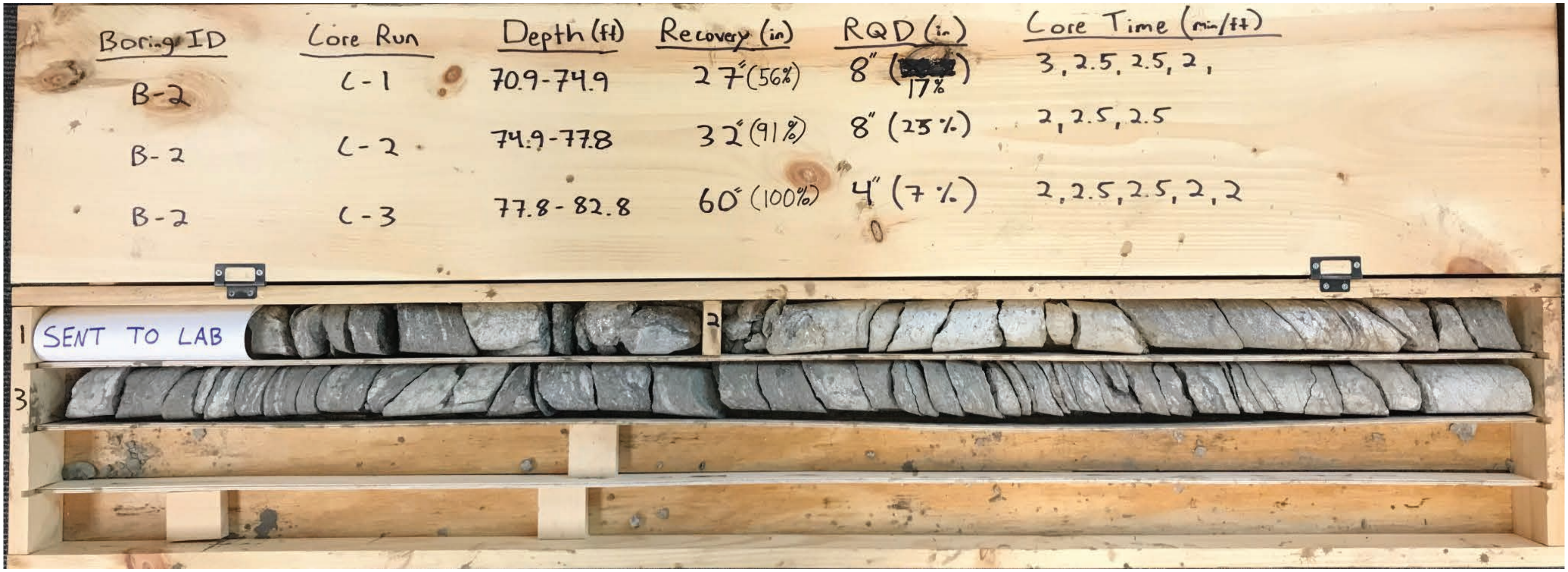
10 20 30 40 50 60 70 80 90

Driller: NEBC, Hermon, ME; Supervisor: Stantec: Liam Gillen-Hughes
Rig Type: Mobile B-57 ATV Rig; Hammer Type: Auto; 4-inch diameter casing, 2" Split Spoon Sampler

△ Unconfined Compression Test
□ Field Vane Test ■ Remolded
✖ Pocket Penetrometer / Torvane

Appendix B – Rock Core Photograph

APPENDIX B
Cobbossee Stream Crossing
Gardiner, Maine



Appendix C – Laboratory Test Results

Client:	Stantec Inc.				
Project:	Cobbossee Trail Bridge				
Location:	Gardiner, ME			Project No:	GTX-314550
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	11/11/21	Checked By:	bfs
Depth :	---	Test Id:	638804		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
B-1	S- 4	8-10 ft	Moist, reddish brown silty sand with gravel	13.0
B-1	S- 10	30.0-31.1 ft	Moist, gray silty sand with gravel	9.1
B-2	S- 5	9.0-10.8 ft	Moist, dark gray silty sand with gravel	8.9
B-2	S- 12	45-47 ft	Moist, gray sand	17.0

Notes: Temperature of Drying : 110° Celsius

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pH of Soil by ASTM D4972

Boring ID	Sample ID	Depth	Visual Description	pH of Soil in Distilled Water	pH of Soil in Calcium Chloride
B-1	S-3	5.0-6.3 ft	Moist, grayish brown silty sand with gravel	5.9	5.5
B-2	S-4	7.0-8.3 ft	Moist, dark greenish gray silty sand	7.2	6.8

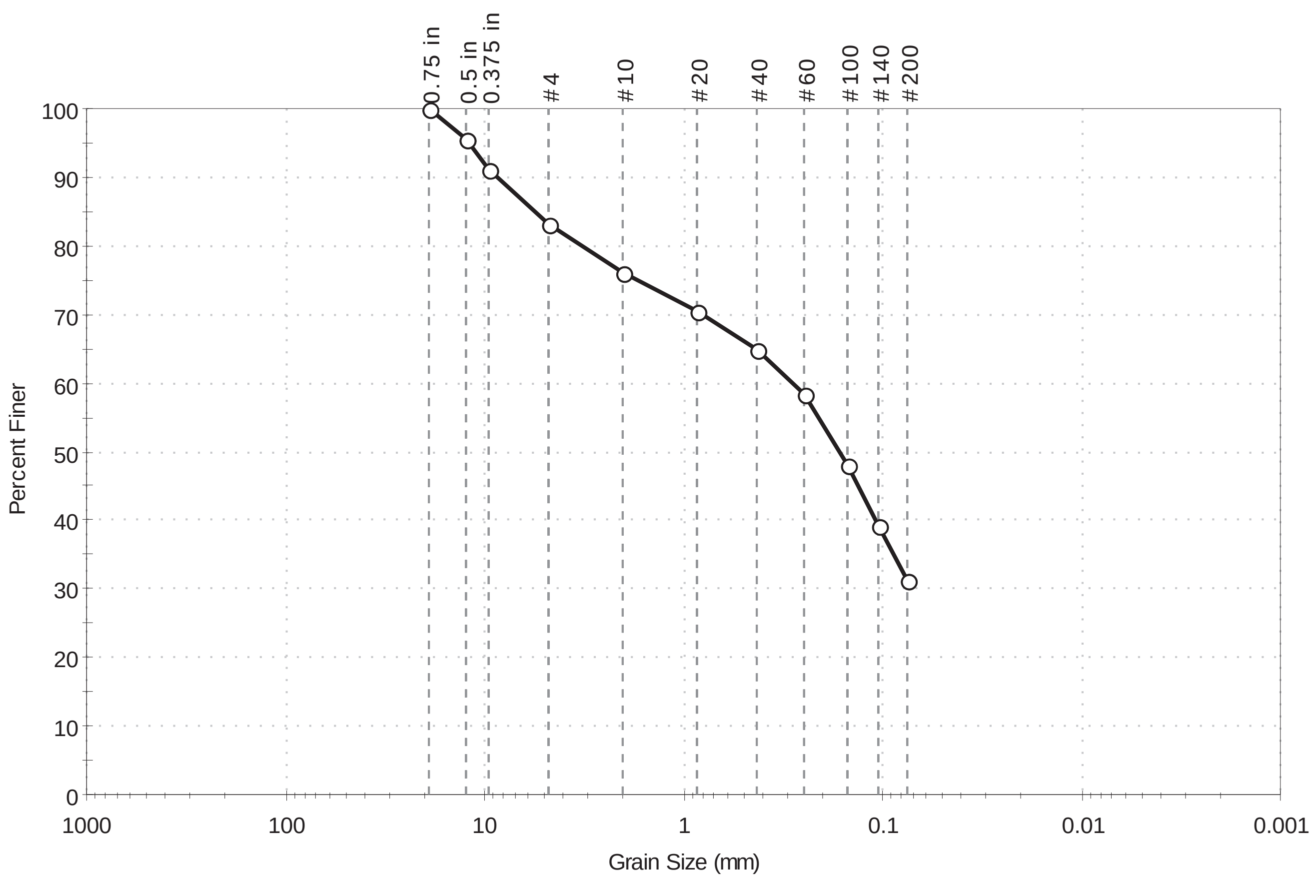
Notes: Sample Preparation: screened through #10 sieve

Method A, pH meter used

Client:	Stantec Inc.				
Project:	Cobbossee Trail Bridge				
Location:	Gardiner, ME			Project No:	GTX-314550
Boring ID:	B-1	Sample Type:	bag	Tested By:	ckg
Sample ID:	S-4	Test Date:	11/08/21	Checked By:	bfs
Depth :	8-10 ft	Test Id:	638807		

Test Comment: ---
Visual Description: Moist, reddish brown silty sand with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	16.8	51.8	31.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	95		
0.375 in	9.50	91		
#4	4.75	83		
#10	2.00	76		
#20	0.85	71		
#40	0.42	65		
#60	0.25	58		
#100	0.15	48		
#140	0.11	39		
#200	0.075	31		

Coefficients

D_{85} = 5.5626 mm　　D_{30} = N/A

D_{60} = 0.2870 mm　　D_{15} = N/A

D_{50} = 0.1660 mm　　D_{10} = N/A

C_u = N/A　　C_c = N/A

Classification

ASTM　N/A

AASHTO　Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

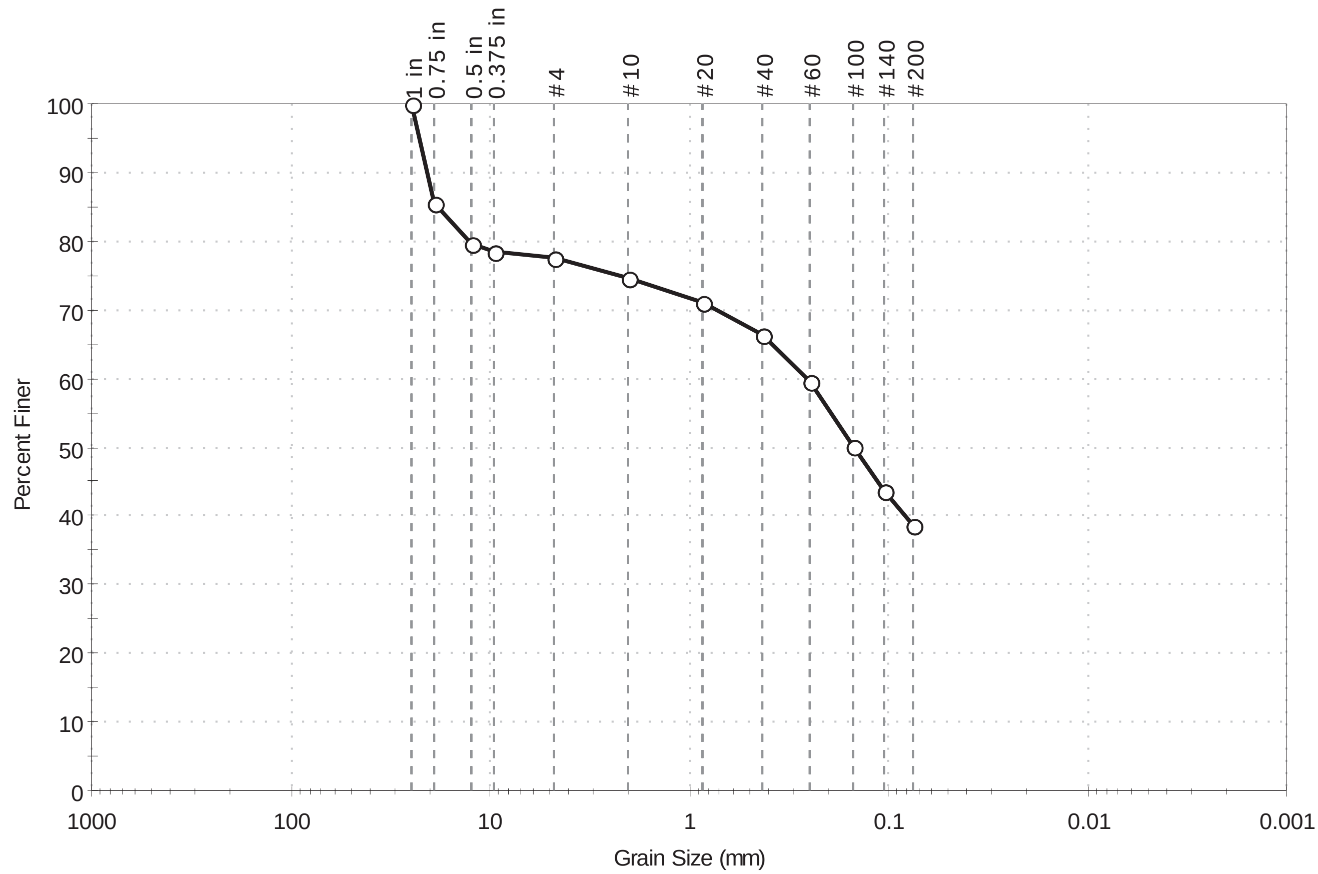
Client:	Stantec Inc.		
Project:	Cobbossee Trail Bridge		
Location:	Gardiner, ME	Project No:	GTX-314550
Boring ID: B-1	Sample Type: bag	Tested By:	ckg
Sample ID: S-10	Test Date: 11/08/21	Checked By:	bfs
Depth : 30.0-31.1 ft	Test Id: 638809		

Test Comment: ---

Visual Description: Moist, gray silty sand with gravel

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	22.4	38.9	38.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	85		
0.5 in	12.50	80		
0.375 in	9.50	78		
#4	4.75	78		
#10	2.00	75		
#20	0.85	71		
#40	0.42	66		
#60	0.25	60		
#100	0.15	50		
#140	0.11	44		
#200	0.075	39		

Coefficients

D_{85} = 18.3244 mm　D_{30} = N/A

D_{60} = 0.2591 mm　D_{15} = N/A

D_{50} = 0.1484 mm　D_{10} = N/A

C_u = N/A　C_c = N/A

Classification

ASTM　N/A

AASHTO　Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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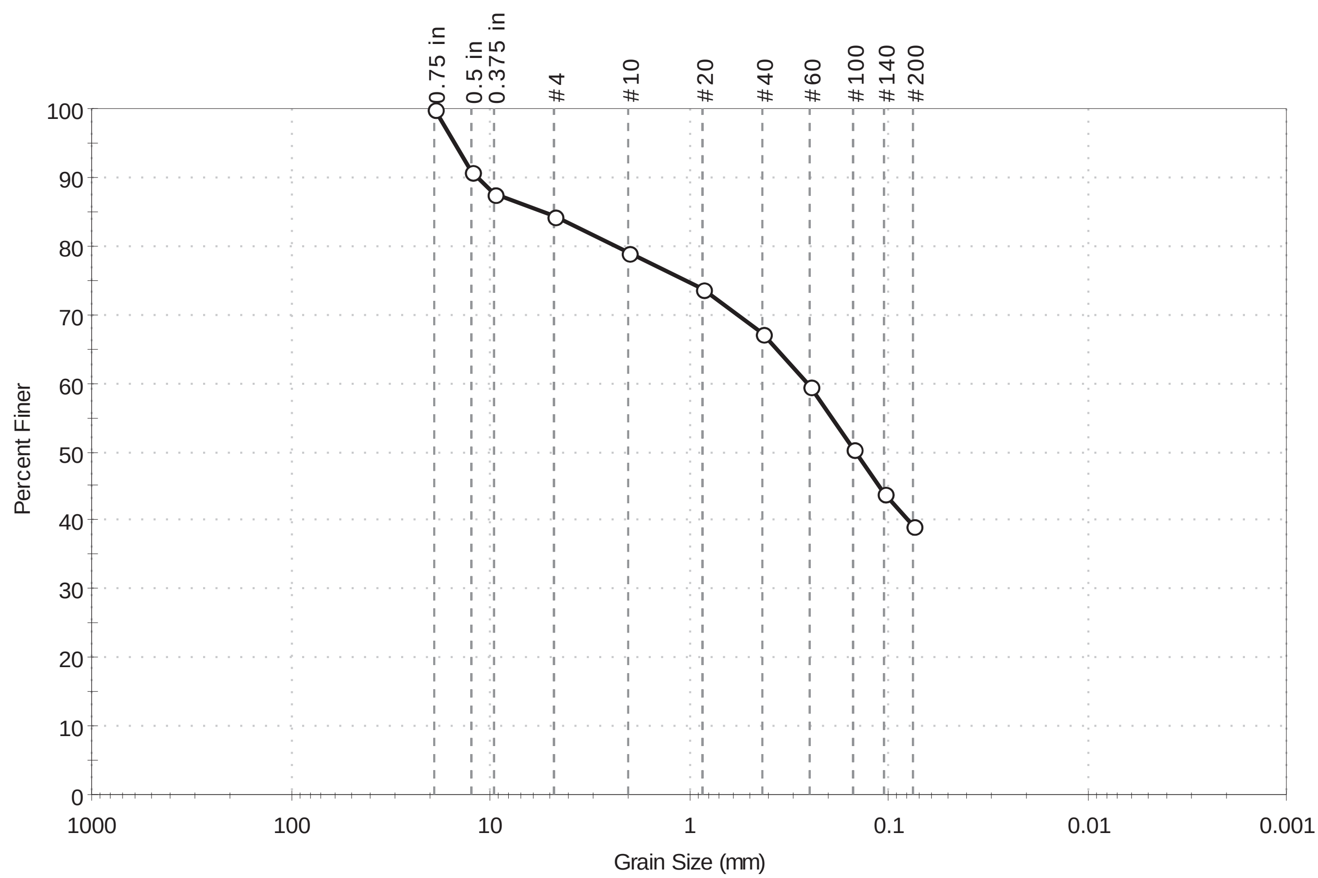
Client:	Stantec Inc.				
Project:	Cobbossee Trail Bridge				
Location:	Gardiner, ME			Project No:	GTX-314550
Boring ID:	B-2	Sample Type:	bag	Tested By:	ckg
Sample ID:	S-5	Test Date:	11/08/21	Checked By:	bfs
Depth :	9.0-10.8 ft	Test Id:	638806		

Test Comment: ---

Visual Description: Moist, dark gray silty sand with gravel

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	15.6	45.3	39.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	91		
0.375 in	9.50	88		
#4	4.75	84		
#10	2.00	79		
#20	0.85	74		
#40	0.42	67		
#60	0.25	60		
#100	0.15	50		
#140	0.11	44		
#200	0.075	39		

Coefficients

D_{85} = 5.3579 mm | D_{30} = N/A

D_{60} = 0.2580 mm | D_{15} = N/A

D_{50} = 0.1467 mm | D_{10} = N/A

C_u = N/A | C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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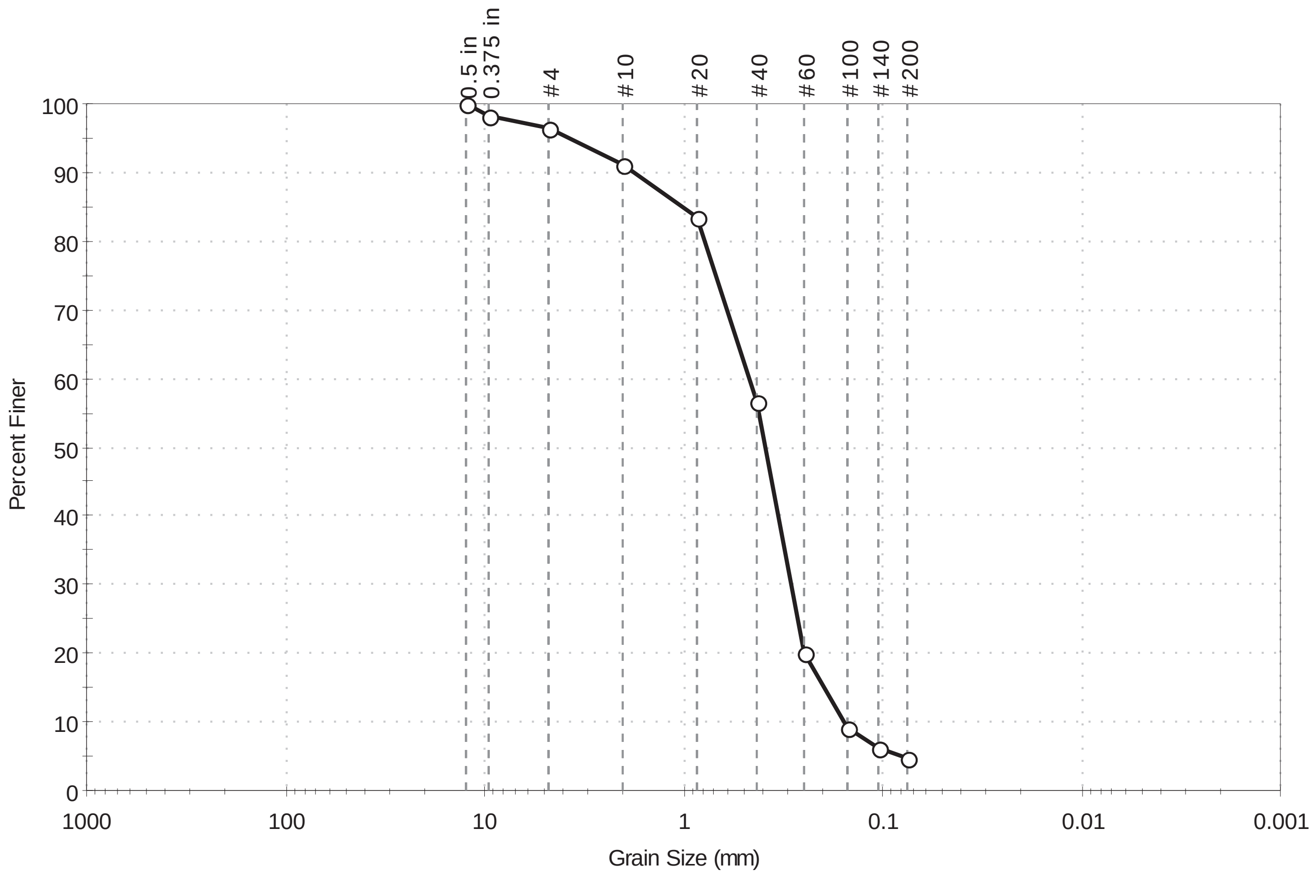
Client:	Stantec Inc.		
Project:	Cobbossee Trail Bridge		
Location:	Gardiner, ME	Project No:	GTX-314550
Boring ID: B-2	Sample Type: bag	Tested By:	ckg
Sample ID: S-12	Test Date: 11/08/21	Checked By:	bfs
Depth : 45-47 ft	Test Id: 638808		
Test Comment:	---		
Visual Description:	Moist, gray sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	3.5	91.8	4.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	96		
#10	2.00	91		
#20	0.85	84		
#40	0.42	57		
#60	0.25	20		
#100	0.15	9		
#140	0.11	6		
#200	0.075	4.7		

Coefficients

D_{85} = 0.9956 mm — D_{30} = 0.2888 mm

D_{60} = 0.4622 mm — D_{15} = 0.1971 mm

D_{50} = 0.3855 mm — D_{10} = 0.1555 mm

C_u = 2.972 — C_c = 1.160

Classification

ASTM Poorly graded SAND (SP)

AASHTO Fine Sand (A-3 (1))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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GEOTESTING EXPRESS INCORPORATED
125 NAGOG PARK
ACTON MA 01720-3451
USA

Analysis No.	TS-A2109935
Report Date	10 November 2021
Date Sampled	04 November 2021
Date Received	08 November 2021
Where Sampled	Acton, MA USA
Sampled By	Client

This is to attest that we have examined: Soil: Project: Cobbossee Trail Bridge; Site Location: Gardiner, ME; Job Number: GTX-314550

When examined to the applicable requirements of:

AASHTO T-291-18	"Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil" Method B
AASHTO T-290-20	"Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil"

Results:

AASHTO T 291 – Chloride Method B

Sample		Results		Detection Limit
		ppm (mg/kg)	%[1]	
B-1		19.	0.0019	10.
S-3	5.0 – 6.3'			
B-2		11.	0.0011	
S-4	7.0 – 8.3'			

NOTE: [1]Percent by weight after drying and prepared as per the Standard.

AASHTO T 290 – Sulfates (Soluble)

Sample		Results		Detection Limit
		ppm (mg/kg)	%[1]	
B-1		< 10.	< 0.0010	10.
S-3	5.0 – 6.3'			
B-2		< 10.	< 0.0010	
S-4	7.0 – 8.3'			

NOTE: [1]Percent by weight after drying and prepared as per the Standard.

END OF ANALYSIS

USEPA Laboratory ID UT00930

Merrill Gee P.E. – Engineer in Charge

Client:	Stantec Inc.			
Project:	Cobbossee Trail Bridge			
Location:	Gardiner, ME		Project No:	GTX-314550
Boring ID: B-2		Sample Type: ---	Tested By:	tlm
Sample ID: C-1		Test Date: 11/08/21	Checked By:	smd
Depth : 70.9-71.6 ft		Test Id: 638810		
Test Comment:	---			
Visual Description:	See photograph(s)			
Sample Comment:	---			

Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C

Boring ID	Sample Number	Depth	Bulk Density, pcf	Compressive strength, psi	Failure Type	Meets ASTM D4543	Note(s)
B-2	C-1	70.9-71.6 ft	174	25355	3	Yes	---

Notes: Density determined on core samples by measuring dimensions and weight and then calculating.

All specimens tested at the approximate as-received moisture content and at standard laboratory temperature.

The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.

Failure Type: 1 = Intact Material Failure; 2 = Discontinuity Failure; 3 = Intact Material and Discontinuity Failure
(See attached photographs)

Client:	Stantec, Inc.	Test Date:	11/5/2021
Project Name:	Connossee Trail Bridge	Tested By:	ak
Project Location:	Gardiner, ME	Checked By:	smd
GTX #:	314550		
Boring ID:	B-2		
Sample ID:	C-1		
Depth:	70.9-71.6 ft		
Visual Description:	See Photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.47	4.47	4.47		
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	630.27				
Bulk Density, lb/ft^3	174			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90^o)	0.00040	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00040	-0.00050

Difference between max and min readings, in:

0° = 0.00010 90° = 0.00090

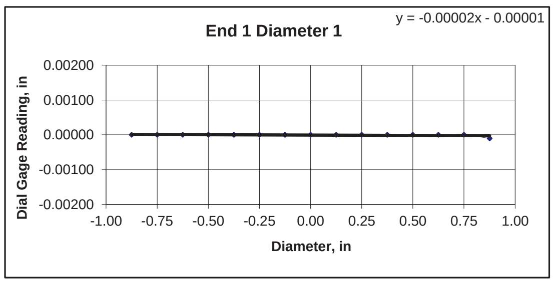
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00020
Diameter 2, in (rotated 90^o)	-0.00060	-0.00040	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00020	0.00020	0.00020	0.00030

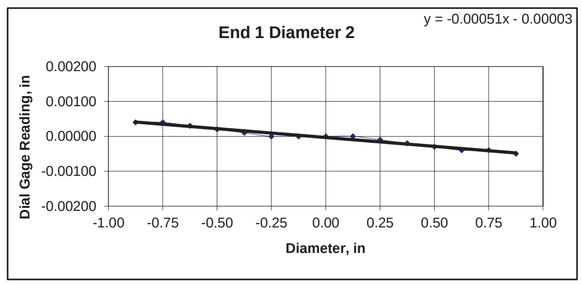
Difference between max and min readings, in:

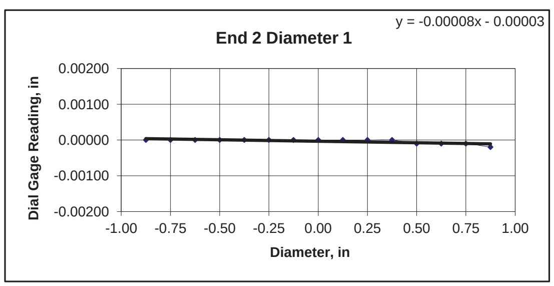
0° = 0.0002 90° = 0.0009

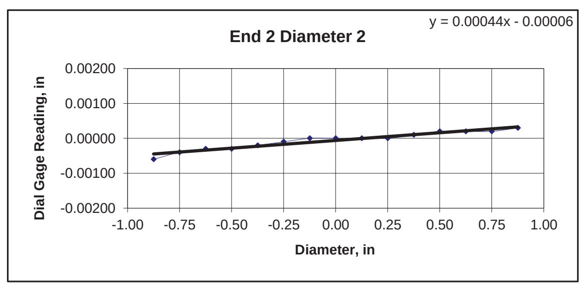
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00045

Flatness Tolerance Met? YES









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00002
Angle of Best Fit Line:	0.00115
End 2:	
Slope of Best Fit Line	0.00008
Angle of Best Fit Line:	0.00475
Maximum Angular Difference:	0.00360

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00051
Angle of Best Fit Line:	0.02898
End 2:	
Slope of Best Fit Line	0.00044
Angle of Best Fit Line:	0.02537
Maximum Angular Difference:	0.00360

Parallelism Tolerance Met? YES
Spherically Seated

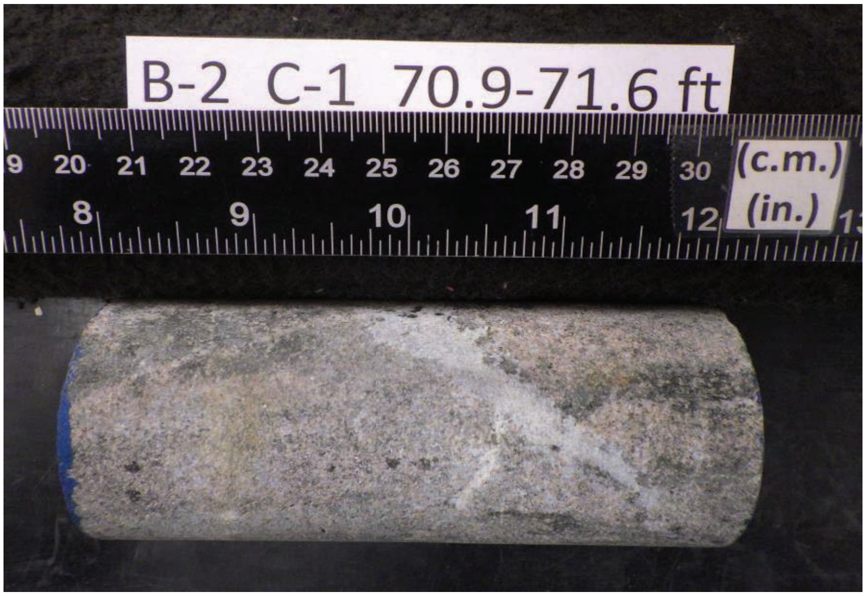
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES
Diameter 2, in (rotated 90^o)	0.00090	1.980	0.00045	0.026	YES
END 2					
Diameter 1, in	0.00020	1.980	0.00010	0.006	YES
Diameter 2, in (rotated 90^o)	0.00090	1.980	0.00045	0.026	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Stantec, Inc.
Project Name:	Connossee Trail Bridge
Project Location:	Gardiner, ME
GTX #:	314550
Test Date:	11/8/2021
Tested By:	kdp
Checked By:	smd
Boring ID:	B-2
Sample ID:	C-1
Depth, ft:	70.9-71.6 ft



After cutting and grinding

After break

Appendix D – Calculations

Soil Strength

Alluvium Deposit

B-2

N60' = 8 and 8 blows per foot
(corrected for depth and hammer efficiency)

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Use phi = 29, which is conservative

Performed By: TAD
Checked By: BJF

Table 2-9

RELATIVE DENSITY OF SAND VERSUS N

N Value (blows/ft or 305 mm)	Relative Density	D_r (%)
0 to 4	very loose	0 to 15
4 to 10	loose	15 to 35
10 to 30	medium	35 to 65
30 to 50	dense	65 to 85
> 50	very dense	85 to 100

Dr = 30 %

Source: Terzaghi and Peck (27), p. 341 and Lambe and Whitman (6), p. 31.

Use SM soil type

Dry Unit Wt = 1.60 x 62.4pcf = 100 pcf
Moist Unit Wt = 1.10 x 100 = 110 pcf, Use 115 pcf

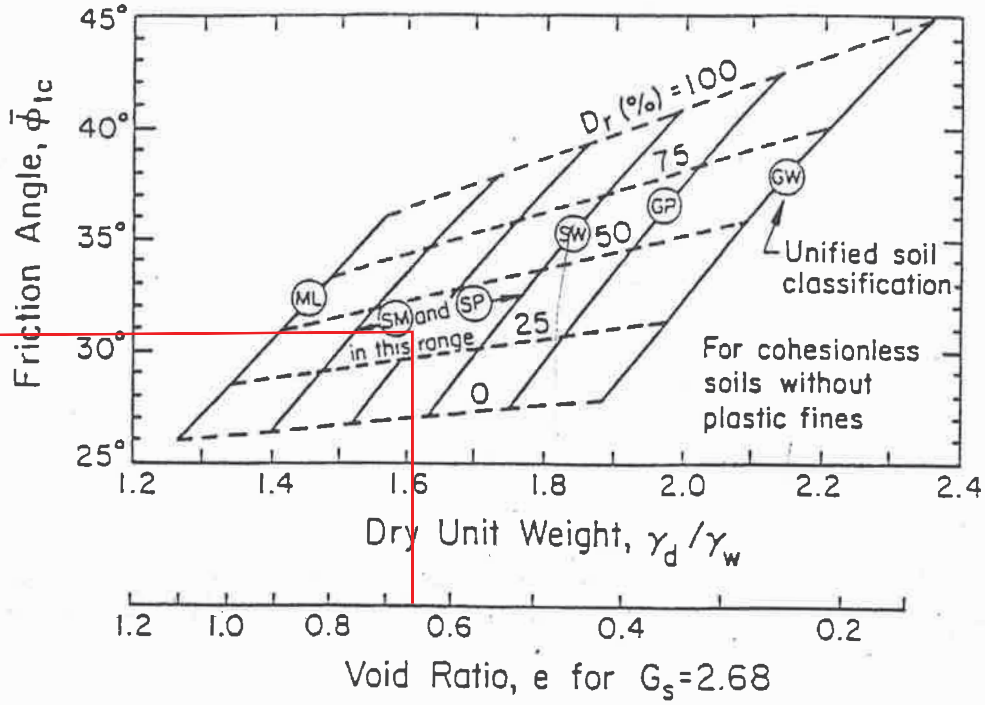


Figure 4-8. $\bar{\phi}_{tc}$ versus Relative Density and Unit Weight

Source: NAVFAC (6), p. 7.1-149.

Performed By: TAD
Checked By: BJF

Outwash Deposit

B-1 and B-2

Average for B-1 N60' = 58
Average for B-2 N60' = 67
(Corrected for Depth and hammer efficiency)

Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Use phi = 36 which is conservative

Performed By: TAD
Checked By: BJF

Table 2-9

RELATIVE DENSITY OF SAND VERSUS N

N Value (blows/ft or 305 mm)	Relative Density	D_r (%)
0 to 4	very loose	0 to 15
4 to 10	loose	15 to 35
10 to 30	medium	35 to 65
30 to 50	dense	65 to 85
> 50	very dense	85 to 100

Dr = 90 %

Source: Terzaghi and Peck (27), p. 341 and Lambe and Whitman (6), p. 31.

Use SP soil type

Dry Unit Wt = 1.90 x 62.4pcf = 118.5 pcf
Moist Unit Wt = 1.10 x 118.5 = 130 pcf

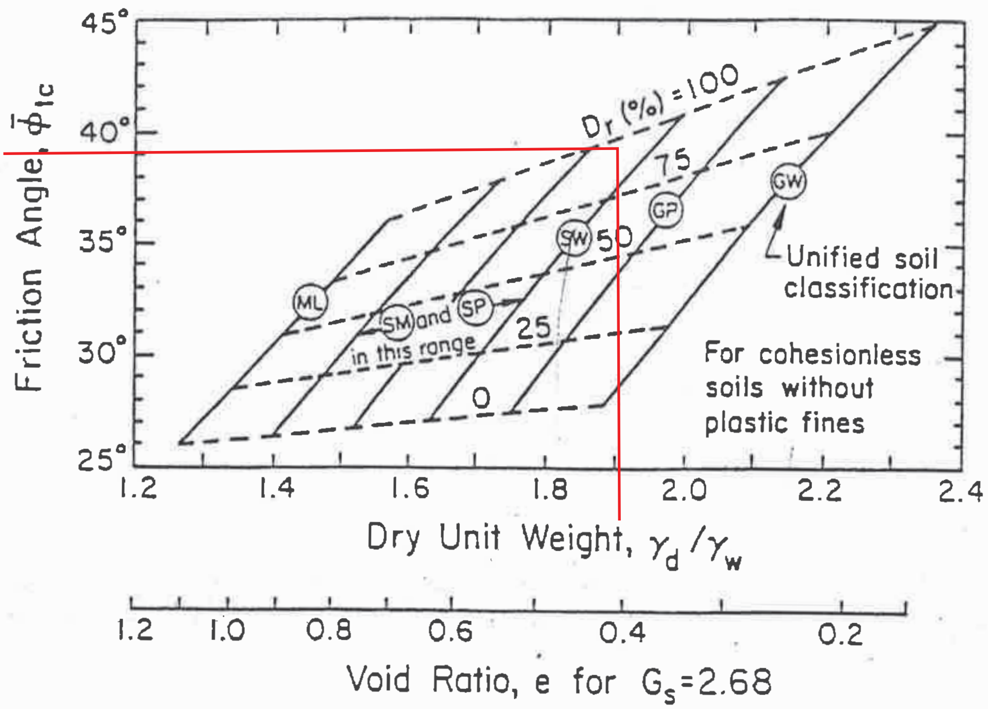


Figure 4-8. $\bar{\phi}_{tc}$ versus Relative Density and Unit Weight

Source: NAVFAC (6), p. 7.1-149.

Performed By: TAD
Checked By: BJF

Glacial Till Deposit

B-1 and B-2

Average for B-1 N60' = 118
Average for B-2 N60' = 145
(Corrected for Depth and hammer efficiency)

Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Use phi = 38 which is conservative and typical for glacial till

Moist Unit Weight = 135 pcf typical for glacial till

Performed By: TAD
Checked By: BJF

Table 2-9

RELATIVE DENSITY OF SAND VERSUS N

N Value (blows/ft or 305 mm)	Relative Density	D_r (%)
0 to 4	very loose	0 to 15
4 to 10	loose	15 to 35
10 to 30	medium	35 to 65
30 to 50	dense	65 to 85
> 50	very dense	85 to 100

Dr = 100 %

Source: Terzaghi and Peck (27), p. 341 and Lambe and Whitman (6), p. 31.

Use SP soil type

Dry Unit Wt = 1.90 x 62.4pcf = 118.5 pcf
Moist Unit Wt = 1.10 x 118.5 = 130 pcf

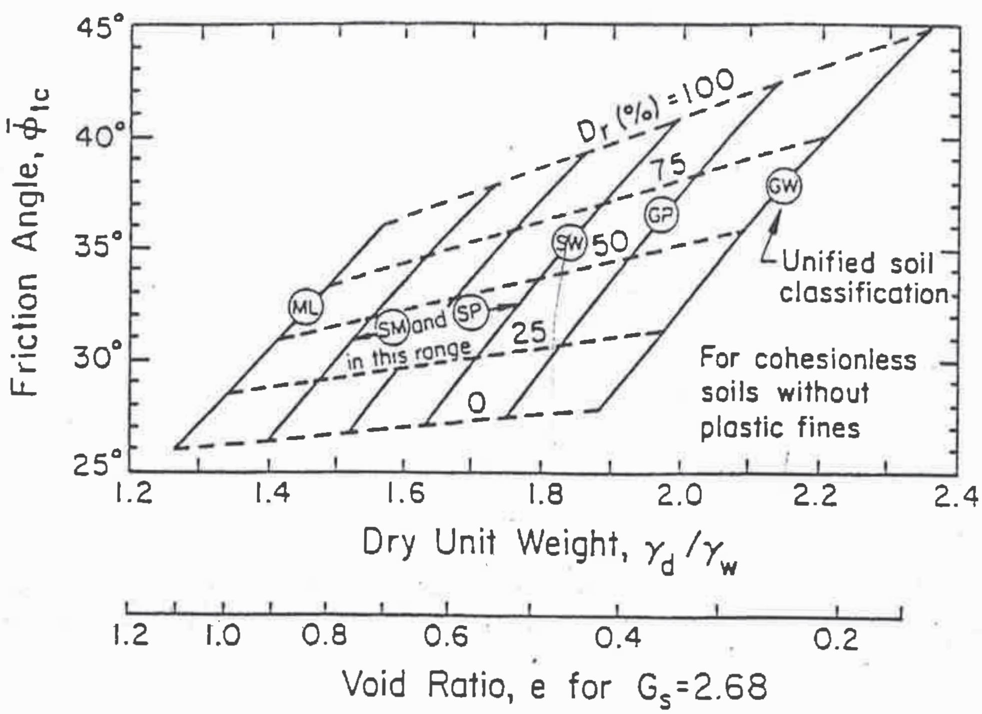


Figure 4-8. $\bar{\phi}_{tc}$ versus Relative Density and Unit Weight

Source: NAVFAC (6), p. 7.1-149.

Performed By: TAD
Checked By: BJF

EFFECTIVE FRICTION ANGLE OF CLAY

Correlation between N60 Values and Undrained Shear Strength

Consistency	Field Identification	Undrained Shear Strength, S_u (psf)	Standard Penetration Test Blowcount* (blows/ft)
Very soft	Easily penetrated several inches by fist	< 250	< 2
Soft	Easily penetrated several inches by thumb	250 - 500	2 - 4
Medium	Can be penetrated several inches by thumb with moderate effort	500 - 1000	4 - 8
Stiff	Readily indented by thumb but penetrated only with great effort	1000 - 2000	8 - 15
Very stiff	Readily indented by thumbnail	2000 - 4000	15 - 30
Hard	Indented with difficulty by thumbnail	> 4000	> 30

**The correlation between undrained strength and SPT blowcount is rather unreliable*

Reference: From Peck, et al. 1974

Range = 10 to 70
Use Su = 2000 psf
This will be conservative

Assume phi = 28 degrees, this is conservative.

Performed By: TAD
Checked By: XX

Stantec

Bearing Resistance

LRFD BEARING RESISTANCE PLOTS

PROJECT NAME: Multi-Use Trail over Cobbessee Stream
PROJECT NUMBER: 179450405
LOCATION: SOUTH ABUTMENT (Boring B-2)

FACTORED BEARING RESISTANCE

Engineering Inputs:

Total unit weight of soil, ϒ:

Above footing (pcf)	125.0 pcf	Compacted granular fill
Below footing (pcf)	135.0 pcf	Glacial Till
Friction angle, ϕ	38	Very Dense Glacial Till
Groundwater depth, D_w	0.0 ft	
Undrained shear strength	0.0 psf	
RC_{BC} (slope reduction)	0.60	Reduction coefficient is based on AASHTO Table 10.6.3.1.2c-2. Calculation is included.
Footing Depth, D_f (feet)	0.0 ft	Set to zero because footing is on slope, AASHTO 10.6.3.1.2c-1
Footing Length, L (feet)	19.0 ft	From Plans
Bearing Strata	Dense sand	

Footing Depth	0 ft	0 ft	0 ft	0 ft	0 ft
Effective Footing Width	5.0 ft	8.0 ft	10.0 ft	12.0 ft	15.0 ft
Footing Length	19.0 ft	19.0 ft	19.0 ft	19.0 ft	19.0 ft
Bearing Strata	Dense sand	Dense sand	Dense sand	Dense sand	Dense sand
Strength Limit State Resistance Factor	0.45	0.45	0.45	0.45	0.45
Nominal Bearing Resistance, q_n	11.8 ksf	17.5 ksf	20.8 ksf	23.6 ksf	27.0 ksf
qn(sloping ground) RC_{BC} x q_n	7.1 ksf	10.5 ksf	12.5 ksf	14.2 ksf	16.2 ksf
Strength Limit (includes ϕ=0.45)	3.2 ksf	4.7 ksf	5.6 ksf	6.4 ksf	7.3 ksf
Service Limit for 1" settlement (includes ϕ=1.0)	17.7 ksf	13.2 ksf	11.6 ksf	10.5 ksf	9.3 ksf

Bearing Restance for 1" Settlement, $q_o=(144*E_s*B_z*S_e)/[(1-v^2)*sqrt(A')]$

Nominal bearing resistance, $q_u= c*N_{cm}+\Upsilon*D_f*N_{qm}*C_{wq}+.5*\Upsilon*B*N_{\Upsilon m}*C_{w\Upsilon}$

BEARING AND SETTLEMENT CALCULATION FACTORS					
Poisson's Ratio, v	0.3	0.3	0.3	0.3	0.3
Youngs Nodulus, E_s	11	11	11	11	11
Shape factor, B_z	1.19	1.12	1.10	1.09	1.09
N_c	61.4	61.4	61.4	61.4	61.4
S_c	1.21	1.34	1.42	1.50	1.63
N_{cm}	74.27	81.99	87.14	92.28	100.01
N_q	48.9	48.9	48.9	48.9	48.9
S_q	1.21	1.33	1.41	1.49	1.62
N_{qm}	58.9	65.0	69.0	73.0	79.0
C_{wq}	0.50	0.50	0.50	0.50	0.50
N_Υ	78	78	78	78	78
S_Υ	0.89	0.83	0.79	0.75	0.68
$N_{\Upsilon m}$	69.8	64.9	61.6	58.3	53.4
$C_{w\Upsilon}$	0.5	0.5	0.5	0.5	0.5

*For the modified bearing capacity factors: dq is conservatively assumed to equal 1.0. The effect of the load inclination is assumed to be minor and therefore the load inclination factor is assumed to be 1.0.

*Shape rigidity factor interpolated from Table 10.6.2.4.2-1 based on Length/Base ratio.

*Applied vertical stress, q_o, is the ultimate pressure transered from the footing in which all load factors equal 1 and includes the footing weight itself.

*Shape rigidity factor interpolated from Table 10.6.2.4.2-1 based on Length/Base ratio.

Table C10.4.6.3-1 Elastic Constants of Various Soils (Modified after U.S. Departement of the Navy, 1982; Bowles, 1988)

Soil Type	Typical Range of Youngs Modulus Values, Es (ksi)		Poisson's Ratio, v	
Clay:	**Lower**	**Upper**	**Lower**	**Upper**
Soft clay	-	-	0.40	0.50
Medium stiff	0.347	2.08	0.40	0.50
Stiff clay	2.08	6.94	0.40	0.50
Very stiff clay	6.94	13.89	0.40	0.50
Silt	0.278	2.78	0.30	0.35
Fine Sand:				
Loose fine sand	1.11	1.67	0.25	0.25
Medium dense fine sand	1.67	2.78	0.25	0.25
Dense fine sand	2.78	4.17	0.25	0.25
Sand:				
Loose sand	1.39	4.17	0.20	0.36
Medium dense sand	4.17	6.94	0.20	0.36
Dense sand	6.94	11.11	0.30	0.40
Gravel:				
Loose gravel	4.17	11.11	0.20	0.35
Medium dense gravel	11.11	13.89	0.20	0.35
Dense gravel	13.89	27.78	0.30	0.40

Use Es = 11 ksi because the very dense glacial till is mostly sand and silt.
Use v = 0.35, which is conservative

Table 10.6.2.4.2-1 Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, B_z (avg.)	Rigid, B_z
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

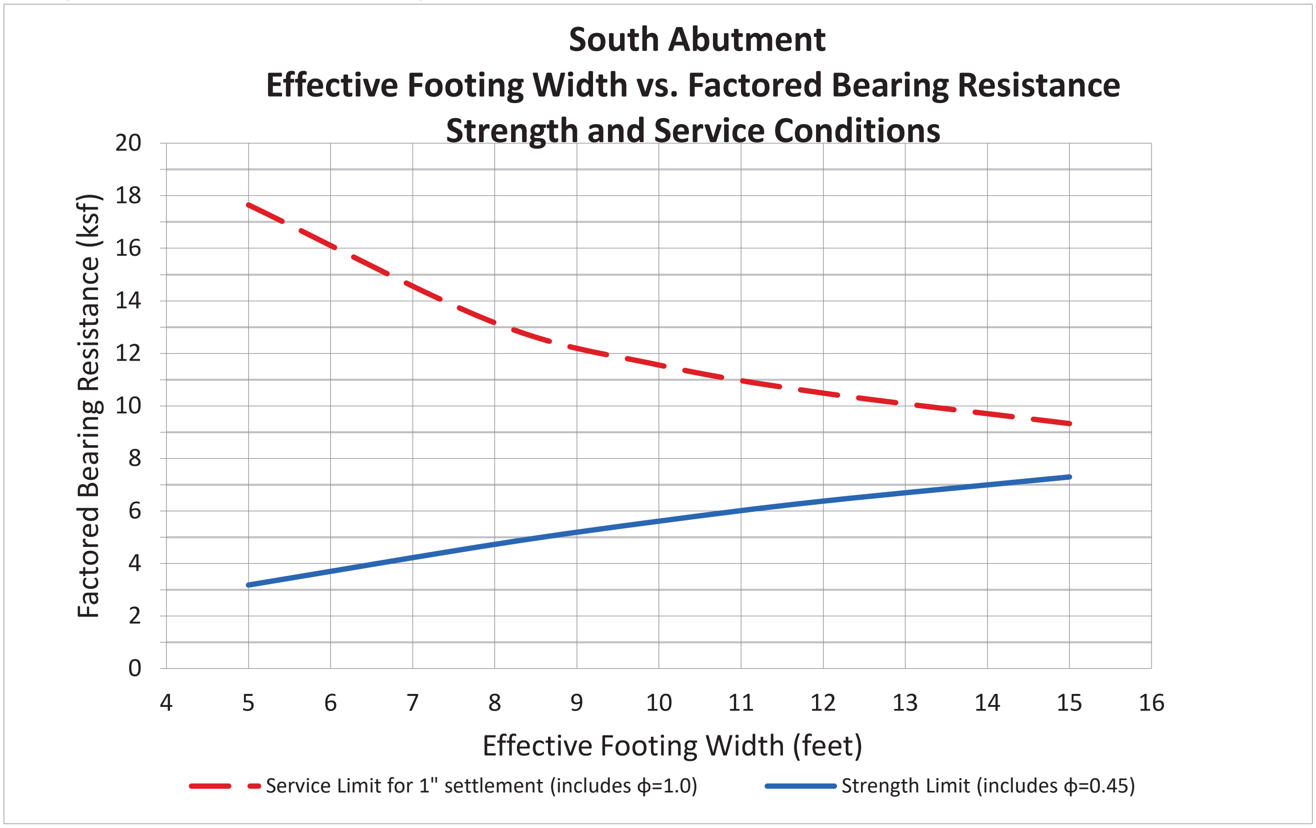
Table 10.6.3.1.2a-1 Bearing Capacity Factors N_c (Prandri, 1921), N_q (Reissner, 1924), and N_γ (Vesic,1975)

ϕ_f	N_c	N_q	N_γ
0	5.14	1	0
20	14.8	6.4	5.4
21	15.8	7.1	6.2
22	16.9	7.8	7.1
23	18.1	8.7	8.2
24	19.3	9.6	9.4
25	20.7	10.7	10.9
26	22.3	11.9	12.5
27	23.9	13.2	14.5
28	25.8	14.7	16.7
29	27.9	16.4	19.3
30	30.1	18.4	22.4
31	32.7	20.6	26.0
32	35.5	23.2	30.2
33	38.6	26.1	35.2
34	42.2	29.4	41.1
35	46.1	33.3	48.0
36	50.6	37.8	56.3
37	55.6	42.9	66.2
38	61.4	48.9	78.0
39	67.9	56.0	92.3
40	75.3	64.2	109.4

Table 10.6.3.1.2a-2 Coefficients C_{wq} and $D_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0	0.5	0.5
D_f	1.0	0.5
>1.5B+D_f	1.0	1.0

Strength and Service Limit State Figure



LRFD BEARING RESISTANCE PLOTS

PROJECT NAME:	**Multi-Use Trail Over Cobbossee Stream**
PROJECT NUMBER:	**179450405**
LOCATION:	**NORTH ABUTMENT (B-1)**

FACTORED BEARING RESISTANCE

Engineering Inputs:

Total unit weight of soil, Υ:		
Above footing (pcf)	125.0 pcf	Compacted granular fill
Below footing (pcf)	135.0 pcf	Glacial Till
Friction angle, ϕ	38	Very Dense Glacial Till
Groundwater depth, D_w	7.0 ft	
Undrained shear strength	0.0 psf	
RC_{BC} (slope reduction)	0.39	Reduction coefficient is based on AASHTO Table 10.6.3.1.2c-2. Calculation is included.
Footing Depth, D_f (feet)	0.0 ft	Set to zero because footing is on slope, AASHTO 10.6.3.1.2c-1
Footing Length, L (feet)	19.0 ft	Based on Plans
Bearing Strata	Dense sand	

Footing Depth	0 ft	0 ft	0 ft	0 ft	0 ft
Effective Footing Width	5.0 ft	8.0 ft	10.0 ft	12.0 ft	15.0 ft
Footing Length	19.0 ft	19.0 ft	19.0 ft	19.0 ft	19.0 ft
Bearing Strata	Dense sand	Dense sand	Dense sand	Dense sand	Dense sand
Strength Limit State Resistance Factor	0.45	0.45	0.45	0.45	0.45
Nominal Bearing Resistance, q_n	22.8 ksf	27.7 ksf	30.5 ksf	32.8 ksf	35.4 ksf
qn(sloping ground) RC_{BC} x q_n	8.9 ksf	10.8 ksf	11.9 ksf	12.8 ksf	13.8 ksf
Strength Limit (includes ϕ=0.45)	4.0 ksf	4.9 ksf	5.3 ksf	5.8 ksf	6.2 ksf
Service Limit for 1" settlement (includes ϕ=1.0)	17.7 ksf	13.2 ksf	11.6 ksf	10.5 ksf	9.3 ksf

Bearing Restance for 1" Settlement, $q_o=(144*E_s*B_z*S_e)/[(1-v^2)*sqrt(A')]$

Nominal bearing resistance, $q_u= c*N_{cm}+\Upsilon*D_f*N_{qm}*C_{wq}+.5*\Upsilon*B*N_{\Upsilon m}*C_{w\Upsilon}$

BEARING AND SETTLEMENT CALCULATION FACTORS					
Poisson's Ratio, v	0.3	0.3	0.3	0.3	0.3
Youngs Nodulus, E_s	11	11	11	11	11
Shape factor, B_z	1.19	1.12	1.10	1.09	1.09
N_c	61.4	61.4	61.4	61.4	61.4
S_c	1.21	1.34	1.42	1.50	1.63
N_{cm}	74.27	81.99	87.14	92.28	100.01
N_q	48.9	48.9	48.9	48.9	48.9
S_q	1.21	1.33	1.41	1.49	1.62
N_{qm}	58.9	65.0	69.0	73.0	79.0
C_{wq}	1.00	1.00	1.00	1.00	1.00
N_{Υ}	78	78	78	78	78
S_{Υ}	0.89	0.83	0.79	0.75	0.68
$N_{\Upsilon m}$	69.8	64.9	61.6	58.3	53.4
$C_{w\Upsilon}$	1.0	0.8	0.7	0.7	0.7

*For the modified bearing capacity factors: dq is conservatively assumed to equal 1.0. The effect of the load inclination is assumed to be minor and therefore the load inclination factor is assumed to be 1.0.

*Shape rigidity factor interpolated from Table 10.6.2.4.2-1 based on Length/Base ratio.

*Applied vertical stress, q_o, is the ultimate pressure transered from the footing in which all load factors equal 1 and includes the footing weight itself.

*Shape rigidity factor interpolated from Table 10.6.2.4.2-1 based on Length/Base ratio.

Table C10.4.6.3-1 Elastic Constants of Various Soils (Modified after U.S. Departement of the Navy, 1982; Bowles, 1988)

Soil Type	Typical Range of Youngs Modulus Values, Es (ksi)		Poisson's Ratio, v	
	Lower	Upper	Lower	Upper
Clay:				
Soft clay	-	-	0.40	0.50
Medium stiff	0.347	2.08	0.40	0.50
Stiff clay	2.08	6.94	0.40	0.50
Very stiff clay	6.94	13.89	0.40	0.50
Silt	0.278	2.78	0.30	0.35
Fine Sand:				
Loose fine sand	1.11	1.67	0.25	0.25
Medium dense fine sand	1.67	2.78	0.25	0.25
Dense fine sand	2.78	4.17	0.25	0.25
Sand:				
Loose sand	1.39	4.17	0.20	0.36
Medium dense sand	4.17	6.94	0.20	0.36
Dense sand	6.94	11.11	0.30	0.40
Gravel:				
Loose gravel	4.17	11.11	0.20	0.35
Medium dense gravel	11.11	13.89	0.20	0.35
Dense gravel	13.89	27.78	0.30	0.40

Use Es = 11 ksi because the very dense glacial till is mostly sand and silt.
Use v = 0.35, which is conservative

Table 10.6.2.4.2-1 Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, B_z (avg.)	Rigid, B_z
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

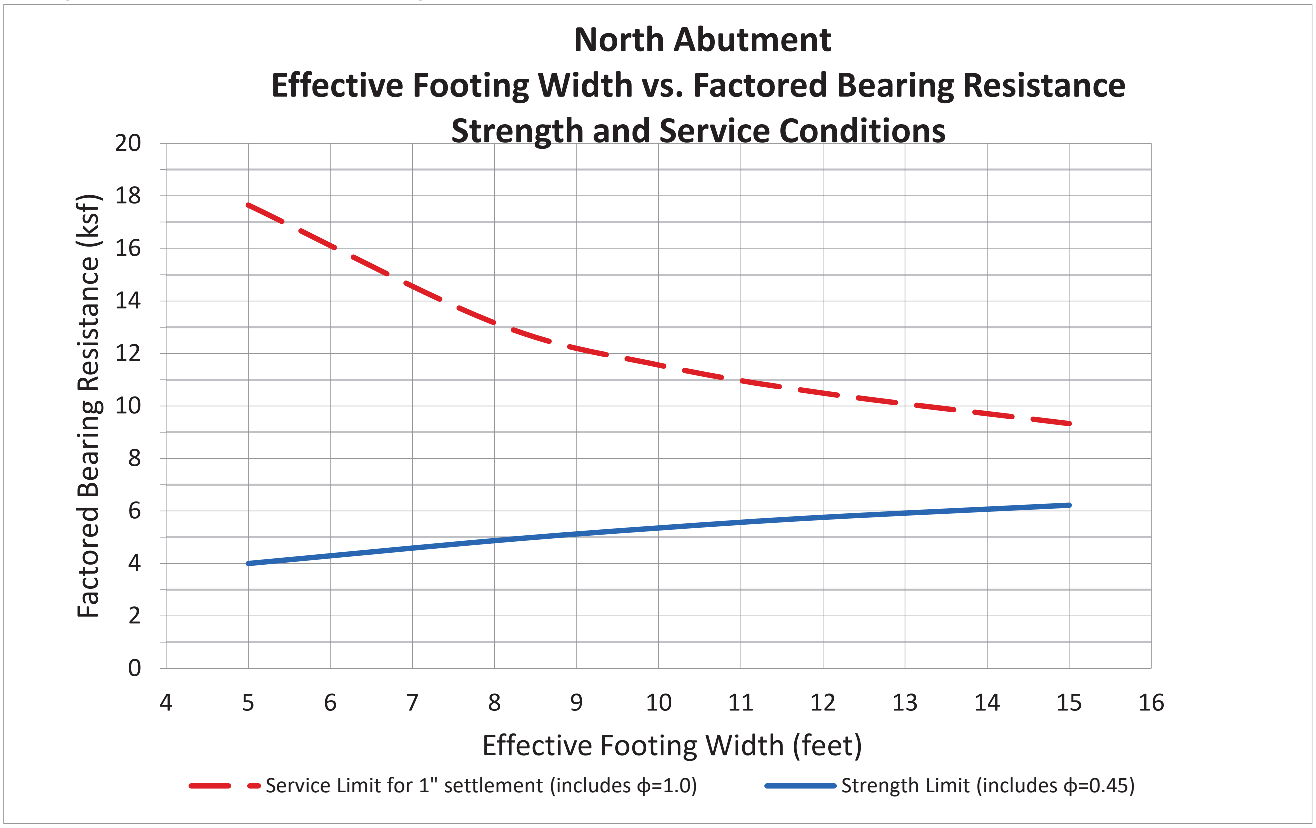
Table 10.6.3.1.2a-1 Bearing Capacity Factors N_c (Prandri, 1921), N_q (Reissner, 1924), and N_Υ (Vesic,1975)

ϕ_f	N_c	N_q	N_Υ
0	5.14	1	0
20	14.8	6.4	5.4
21	15.8	7.1	6.2
22	16.9	7.8	7.1
23	18.1	8.7	8.2
24	19.3	9.6	9.4
25	20.7	10.7	10.9
26	22.3	11.9	12.5
27	23.9	13.2	14.5
28	25.8	14.7	16.7
29	27.9	16.4	19.3
30	30.1	18.4	22.4
31	32.7	20.6	26.0
32	35.5	23.2	30.2
33	38.6	26.1	35.2
34	42.2	29.4	41.1
35	46.1	33.3	48.0
36	50.6	37.8	56.3
37	55.6	42.9	66.2
38	61.4	48.9	78.0
39	67.9	56.0	92.3
40	75.3	64.2	109.4

Table 10.6.3.1.2a-2 Coefficients C_{wq} and $D_{w\Upsilon}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\Upsilon}$
0	0.5	0.5
D_f	1.0	0.5
$>1.5B+D_f$	1.0	1.0

Strength and Service Limit State Figure



Slope Bearing Reduction

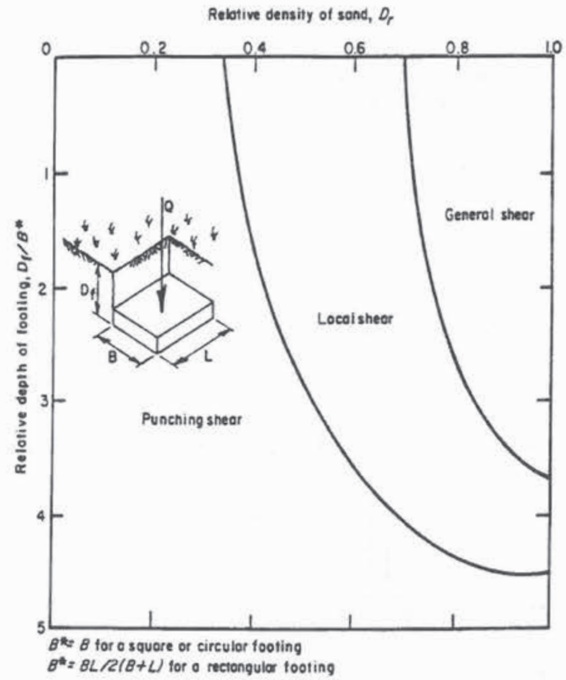


Figure C10.6.3.1.2b-1—Modes of Bearing Capacity Failure for Footings in Sand

10.6.3.1.2c—Considerations for Footings on Slopes

For footings constructed on or adjacent to slopes, the nominal bearing resistance shall be determined using a reduction coefficient (RC_{BC}) as presented in Tables 10.6.3.1.2c-1 and 10.6.3.1.2c-2. The reduction coefficient should be applied directly to the nominal bearing resistance calculated from Eq.10.6.3.1.2a-1 for footings on level ground and supported on the same foundation soil conditions.

The nominal resistance of footings on or adjacent to slopes shall be taken as:

$$q_{n-sloping\ ground} = RC_{BC}q_n = RC_{BC}\left(cN_c + 0.5\gamma BN_\gamma\right) \quad (10.6.3.1.2c\text{-}1)$$

where:

$q_{n\text{-}sloping\ ground}$ = the nominal footing bearing resistance considering the effect of sloping ground (ksf)

RC_{BC} = reduction coefficient for bearing resistance due to slope effects (dim)

and other variables are as defined in Article 10.6.3.1.2a and Figure 10.6.3.1.2c-1. The bearing capacity factors N_c and N_γ are obtained in accordance with Article 10.6.3.1.2a.

Reduction coefficients (RC_{BC}) should be determined using the definitions illustrated in Figure 10.6.3.1.2c-1

C10.6.3.1.2c

A rational approach for determining a modified bearing resistance for footings on or adjacent to a slope is presented in Leshchinsky (2015) and Leshchinsky and Xie (2016). These methods are considered valid and applicable to structure foundations in addition to the MSE retaining wall example presented in the reference papers. The reduction coefficients provided in Tables 10.6.3.1.2c-1 and 10.6.3.1.2c-2 are modified and reconfigured by the author of the cited papers to allow for more convenient use in practice. See the original papers for the complete tabulation of reduction coefficient values.

The reduction coefficients are applicable to purely cohesive, purely cohesionless and c- ϕ soils. The RC_{BC} factors are based on no footing embedment for footings either on or adjacent to slopes and may be conservative for deep footing embedment depths.

Limit analysis, or limit equilibrium analysis, should be considered to estimate the nominal bearing resistance of footings on or adjacent to slopes composed of soils and/or site conditions that are not consistent with the parameters and conditions described in the reference documents (i.e. embedment >0, layered soils, steeper slopes).

The schematic shown in Figure 10.6.3.1.2c-1 is provided only for illustrating and defining the terms used in the design equations and tables. This figure should not be used as the basis for locating footings on slopes regarding embedment depth and setback.

for footings on or adjacent to slopes. Use linear interpolation to obtain reduction coefficients for values not provided. The slope stability factor, N_s, in Tables 10.6.3.1.2c-1 and 10.6.3.1.2c-2 shall be taken as:

$$N_s = \frac{\gamma H_s}{c} \quad (10.6.3.1.2c\text{-}2)$$

where:

N_s = slope stability factor (dim)
H_s = height of sloping ground surface below bottom of footing (ft)

and other variables are as defined in Article 10.6.3.1.2a.

Use this case - on the slope

North ABUTMENT

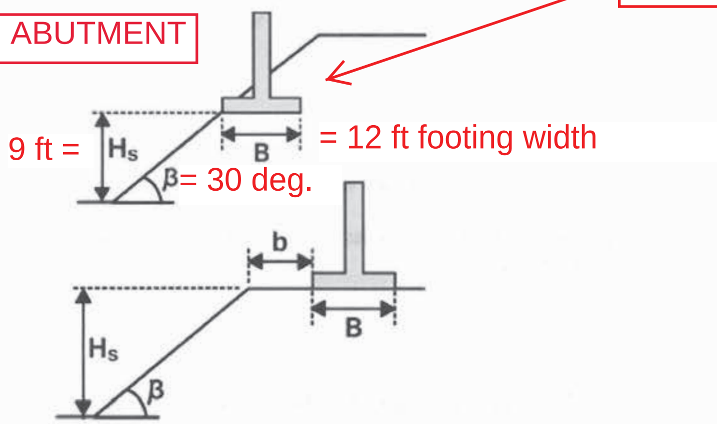


Figure 10.6.3.1.2c-1—Definition of Footing and Slope Geometric Parameters for Determination of RC_{BC}

Soil Friction Angle = 38 deg.

B/Hs = 12/9 = 1.3
b/B = 0/12 = 0

South ABUTMENT
Hs = 1 ft
B = 12 ft
Beta = 26.5 degrees

B/Hs = 12/1 = 12
b/B = 0/12 = 0

Table 10.6.3.1.2c-1—Reduction Coefficients (RC_{BC}) for Footings Placed on Slopes Composed of either Purely Cohesive Soils, (ϕ = 0); Purely Cohesionless Soils (c'=0); or Soils with both Cohesive and Cohesionless Strength Components

			β=10°				β=20°				β=30°				β=40°			
			Ns				*Ns*				*Ns*				*Ns*			
φ (°)	*B/H*	*b/B*	0	2	4	c'=0	0	2	4	c'=0	0	2	4	c'=0	0	2	4	c'=0
0	0.1	0 (On Slope)	0.89	0.89	0.88	0.00	0.89	0.88	0.87	0.00	0.85	0.84	0.83	0.00	0.77	0.76	0.74	0.00
	0.2		0.89	0.88	0.88	0.00	0.89	0.87	0.86	0.00	0.82	0.81	0.78	0.00	0.76	0.73	0.69	0.00
	0.4		0.88	0.87	0.86	0.00	0.89	0.86	0.82	0.00	0.81	0.77	0.66	0.00	0.74	0.68	0.53	0.00
	0.6		0.89	0.87	0.84	0.00	0.88	0.84	0.71	0.00	0.81	0.74	0.53	0.00	0.74	0.64	0.41	0.00
	1		0.87	0.84	0.75	0.00	0.87	0.79	0.56	0.00	0.80	0.66	0.42	0.00	0.73	0.56	0.33	0.00
	1.5		0.87	0.82	0.62	0.00	0.87	0.72	0.47	0.00	0.80	0.61	0.37	0.00	0.73	0.54	0.30	0.00
	3		0.87	0.73	0.47	0.00	0.87	0.67	0.37	0.00	0.83	0.62	0.31	0.00	0.80	0.59	0.28	0.00
20	0.1	0 (On Slope)	0.91	0.91	0.91	0.69	0.80	0.79	0.79	0.22	0.64	0.63	0.61	0.00	0.53	0.52	0.50	0.00
	0.2		0.90	0.89	0.90	0.68	0.75	0.73	0.72	0.21	0.62	0.59	0.56	0.00	0.52	0.49	0.45	0.00
	0.4		0.86	0.86	0.84	0.63	0.73	0.70	0.67	0.22	0.62	0.56	0.51	0.00	0.52	0.45	0.39	0.00
	0.6		0.85	0.84	0.82	0.58	0.73	0.68	0.63	0.22	0.61	0.54	0.47	0.00	0.51	0.41	0.33	0.00
	1		0.85	0.82	0.78	0.58	0.72	0.64	0.58	0.26	0.61	0.50	0.42	0.00	0.52	0.39	0.30	0.00
	1.5		0.86	0.80	0.75	0.58	0.73	0.62	0.54	0.31	0.65	0.50	0.42	0.00	0.60	0.44	0.34	0.00
	3		0.90	0.77	0.72	0.58	0.88	0.66	0.56	0.35	0.86	0.61	0.51	0.00	0.85	0.57	0.46	0.00
30	0.1	0 (On Slope)	0.93	0.92	0.91	0.77	0.65	0.64	0.63	0.40	0.51	0.50	0.48	0.11	0.40	0.37	0.36	0.00
	0.2		0.81	0.82	0.84	0.76	0.64	0.61	0.59	0.39	0.50	0.47	0.44	0.11	0.39	0.35	0.32	0.00
	0.4		0.79	0.79	0.78	0.72	0.63	0.59	0.55	0.37	0.50	0.43	0.39	0.13	0.39	0.32	0.27	0.00
	0.6		0.78	0.77	0.75	0.68	0.62	0.56	0.52	0.36	0.49	0.41	0.36	0.14	0.39	0.30	0.24	0.00
	1		0.79	0.75	0.73	0.67	0.63	0.53	0.49	0.41	0.55	0.41	0.35	0.24	0.48	0.33	0.26	0.00
	1.5		0.79	0.73	0.69	0.66	0.72	0.56	0.50	0.46	0.68	0.47	0.39	0.33	0.64	0.41	0.33	0.00
	3		0.95	0.74	0.70	0.65	0.92	0.66	0.60	0.51	0.90	0.62	0.57	0.43	0.88	0.59	0.51	0.00
40	0.1	0 (On Slope)	0.74	0.77	0.79	0.80	0.52	0.51	0.50	0.38	0.37	0.36	0.34	0.17	0.28	0.26	0.24	0.05
	0.2		0.69	0.69	0.69	0.78	0.51	0.48	0.47	0.37	0.37	0.33	0.30	0.16	0.27	0.23	0.20	0.05
	0.4		0.67	0.69	0.67	0.72	0.50	0.45	0.43	0.36	0.36	0.30	0.26	0.17	0.27	0.20	0.17	0.06
	0.6		0.67	0.67	0.64	0.66	0.50	0.43	0.43	0.34	0.40	0.34	0.26	0.17	0.32	0.22	0.18	0.08
	1		0.69	0.64	0.62	0.70	0.63	0.48	0.43	0.45	0.58	0.39	0.33	0.32	0.54	0.33	0.27	0.24
	1.5		0.76	0.65	0.61	0.74	0.74	0.53	0.48	0.56	0.71	0.47	0.40	0.47	0.68	0.43	0.36	0.41
	3		0.95	0.74	0.71	0.77	0.94	0.68	0.65	0.66	0.91	0.67	0.62	0.62	0.92	0.67	0.59	0.57

phi = 38 deg.

0.46

0.63

SOUTH ABUTMENT
For phi = 30 deg and Beta = 26.5 deg
RCbc = 0.46
For phi = 40 deg and Beta = 26.5 deg
RCbc = 0.63

For phi = 38 deg and Beta = 26.5 deg
RCbc = 0.60
This is conservative because B/H is 12.

Table 10.6.3.1.2c-1—Reduction Coefficients (*RC_{BC}*) for Footings Placed on Slopes Composed of either Purely Cohesive Soils, (ϕ = 0); Purely Cohesionless Soils (*c'*=0); or Soils with both Cohesive and Cohesionless Strength Components

			β=10°				β=20°				β=30°				β=40°			
			Ns				*Ns*				*Ns*				*Ns*			
φ (°)	***B/H***	***b/B***	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**
0	**0.1**	**0 (On Slope)**	0.89	0.89	0.88	0.00	0.89	0.88	0.87	0.00	0.85	0.84	0.83	0.00	0.77	0.76	0.74	0.00
	0.2		0.89	0.88	0.88	0.00	0.89	0.87	0.86	0.00	0.82	0.81	0.78	0.00	0.76	0.73	0.69	0.00
	0.4		0.88	0.87	0.86	0.00	0.89	0.86	0.82	0.00	0.81	0.77	0.66	0.00	0.74	0.68	0.53	0.00
	0.6		0.89	0.87	0.84	0.00	0.88	0.84	0.71	0.00	0.81	0.74	0.53	0.00	0.74	0.64	0.41	0.00
	1		0.87	0.84	0.75	0.00	0.87	0.79	0.56	0.00	0.80	0.66	0.42	0.00	0.73	0.56	0.33	0.00
	1.5		0.87	0.82	0.62	0.00	0.87	0.72	0.47	0.00	0.80	0.61	0.37	0.00	0.73	0.54	0.30	0.00
	3		0.87	0.73	0.47	0.00	0.87	0.67	0.37	0.00	0.83	0.62	0.31	0.00	0.80	0.59	0.28	0.00
20	**0.1**	**0 (On Slope)**	0.91	0.91	0.91	0.69	0.80	0.79	0.79	0.22	0.64	0.63	0.61	0.00	0.53	0.52	0.50	0.00
	0.2		0.90	0.89	0.90	0.68	0.75	0.73	0.72	0.21	0.62	0.59	0.56	0.00	0.52	0.49	0.45	0.00
	0.4		0.86	0.86	0.84	0.63	0.73	0.70	0.67	0.22	0.62	0.56	0.51	0.00	0.52	0.45	0.39	0.00
	0.6		0.85	0.84	0.82	0.58	0.73	0.68	0.63	0.22	0.61	0.54	0.47	0.00	0.51	0.41	0.33	0.00
	1		0.85	0.82	0.78	0.58	0.72	0.64	0.58	0.26	0.61	0.50	0.42	0.00	0.52	0.39	0.30	0.00
	1.5		0.86	0.80	0.75	0.58	0.73	0.62	0.54	0.31	0.65	0.50	0.42	0.00	0.60	0.44	0.34	0.00
	3		0.90	0.77	0.72	0.58	0.88	0.66	0.56	0.35	0.86	0.61	0.51	0.00	0.85	0.57	0.46	0.00
30	**0.1**	**0 (On Slope)**	0.93	0.92	0.91	0.77	0.65	0.64	0.63	0.40	0.51	0.50	0.48	0.11	0.40	0.37	0.36	0.00
	0.2		0.81	0.82	0.84	0.76	0.64	0.61	0.59	0.39	0.50	0.47	0.44	0.11	0.39	0.35	0.32	0.00
	0.4		0.79	0.79	0.78	0.72	0.63	0.59	0.55	0.37	0.50	0.43	0.39	0.13	0.39	0.32	0.27	0.00
	0.6		0.78	0.77	0.75	0.68	0.62	0.56	0.52	0.36	0.49	0.41	0.36	0.14	0.39	0.30	0.24	0.00
	1		0.79	0.75	0.73	0.67	0.63	0.53	0.49	0.41	0.55	0.41	0.35	0.24	0.29	33	0.26	0.00
	1.5		0.79	0.73	0.69	0.66	0.72	0.56	0.50	0.46	0.68	0.47	0.39	0.33	[illegible]	41	0.33	0.00
	3		0.95	0.74	0.70	0.65	0.92	0.66	0.60	0.51	0.90	0.62	0.57	0.43	0.88	0.59	0.51	0.00
40	**0.1**	**0 (On Slope)**	0.74	0.77	0.79	0.80	0.52	0.51	0.50	0.38	0.37	0.36	0.34	0.17	0.28	0.26	0.24	0.05
	0.2		0.69	0.69	0.69	0.78	0.51	0.48	0.47	0.37	0.37	0.33	0.30	0.16	0.27	0.23	0.20	0.05
	0.4		0.67	0.69	0.67	0.72	0.50	0.45	0.43	0.36	0.36	0.30	0.26	0.17	0.27	0.20	0.17	0.06
	0.6		0.67	0.67	0.64	0.66	0.50	0.43	0.43	0.34	0.40	0.34	0.26	0.17	0.32	0.22	0.18	0.08
	1		0.69	0.64	0.62	0.70	0.63	0.48	0.43	0.45	0.58	0.39	0.33	0.32	0.41	33	0.27	0.24
	1.5		0.76	0.65	0.61	0.74	0.74	0.53	0.48	0.56	0.71	0.47	0.40	0.47	[illegible]	43	0.36	0.41
	3		0.95	0.74	0.71	0.77	0.94	0.68	0.65	0.66	0.91	0.67	0.62	0.62	0.92	0.67	0.59	0.57

phi = 38 deg.

NORTH ABUTMENT
For phi = 30 deg and Beta = 30 deg
RCbc = 0.30
For phi = 40 deg and Beta = 30 deg
RCbc = 0.41

For phi = 38 deg and Beta = 38 deg
RCbc = 0.39

Table 10.6.3.1.2c-2—Reduction Coefficients (RC_{BC}) for Footings Placed Adjacent to Slopes Composed of either Purely Cohesive Soils, ($\phi = 0$); Purely Cohesionless Soils (c'=0); or Soils with both Cohesive and Cohesionless Strength Components

			β=10°				β=20°				β=30°				β=40°			
			Ns				*Ns*				*Ns*				*Ns*			
ϕ (°)	***B/H***	***b/B***	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**
0	**0.2**	0	0.89	0.88	0.88	0.00	0.89	0.87	0.86	0.00	0.82	0.81	0.78	0.00	0.76	0.73	0.69	0.00
		0.5	0.97	0.96	0.96	0.00	0.95	0.93	0.91	0.00	0.92	0.89	0.87	0.00	0.86	0.83	0.76	0.00
		1.25	1.00	0.99	0.98	0.00	1.00	0.98	0.96	0.00	1.00	0.97	0.95	0.00	0.95	0.91	0.81	0.00
		2.5	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	0.97	0.84	0.00
		5	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	0.89	0.00
		10	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	0.5	0	0.92	0.91	0.88	0.00	0.85	0.82	0.76	0.00	0.77	0.73	0.63	0.00	0.71	0.65	0.52	0.00
		0.5	0.96	0.95	0.89	0.00	0.92	0.89	0.78	0.00	0.87	0.84	0.68	0.00	0.83	0.76	0.56	0.00
		1.25	0.98	0.97	0.90	0.00	0.96	0.94	0.80	0.00	0.94	0.92	0.71	0.00	0.90	0.83	0.58	0.00
		2.5	1.00	1.00	1.00	0.00	1.00	1.00	0.86	0.00	1.00	1.00	0.79	0.00	1.00	0.93	0.68	0.00
		5	1.00	1.00	1.00	0.00	1.00	1.00	0.95	0.00	1.00	1.00	0.93	0.00	1.00	1.00	0.88	0.00
		10	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	1	0	0.87	0.84	0.75	0.00	0.87	0.79	0.56	0.00	0.80	0.66	0.42	0.00	0.73	0.56	0.33	0.00
		0.5	0.95	0.91	0.82	0.00	0.92	0.83	0.65	0.00	0.86	0.73	0.46	0.00	0.81	0.67	0.40	0.00
		1.25	0.97	0.94	0.83	0.00	0.95	0.87	0.67	0.00	0.92	0.81	0.50	0.00	0.89	0.76	0.46	0.00
		2.5	1.00	0.98	0.88	0.00	1.00	0.97	0.77	0.00	1.00	1.00	0.84	0.00	0.99	0.92	0.63	0.00
		5	1.00	1.00	0.95	0.00	1.00	1.00	0.90	0.00	1.00	1.00	0.84	0.00	1.00	1.00	0.83	0.00
		10	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	2	0	0.87	0.79	0.57	0.00	0.87	0.71	0.44	0.00	0.81	0.62	0.35	0.00	0.75	0.56	0.29	0.00
		0.5	0.97	0.93	0.65	0.00	0.94	0.79	0.49	0.00	0.89	0.72	0.42	0.00	0.85	0.69	0.37	0.00
		1.25	0.99	0.98	0.73	0.00	0.99	0.91	0.57	0.00	0.98	0.86	0.51	0.00	0.96	0.83	0.47	0.00
		2.5	1.00	0.99	0.82	0.00	1.00	0.96	0.69	0.00	1.00	0.95	0.64	0.00	1.00	0.95	0.61	0.00
		5	1.00	1.00	0.96	0.00	1.00	1.00	0.87	0.00	1.00	1.00	0.84	0.00	1.00	1.00	0.81	0.00
		10	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
20	**0.2**	0	0.90	0.89	0.90	0.68	0.75	0.73	0.72	0.21	0.62	0.59	0.56	0.00	0.52	0.49	0.45	0.00
		0.5	0.78	0.87	0.86	0.70	0.74	0.76	0.74	0.40	0.63	0.65	0.63	0.00	0.52	0.56	0.52	0.00
		1.25	0.86	0.92	0.92	0.82	0.83	0.84	0.83	0.70	0.74	0.75	0.74	0.00	0.63	0.66	0.63	0.00
		2.5	0.96	0.98	0.99	0.83	0.95	0.94	0.95	0.84	0.90	0.89	0.90	0.00	0.78	0.81	0.78	0.00
		5	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	0.96	0.98	0.96	0.00
		10	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	0.99	0.99	1.00	0.00
	0.5	0	0.86	0.86	0.84	0.60	0.73	0.70	0.67	0.22	0.62	0.56	0.51	0.00	0.52	0.45	0.39	0.00
		0.5	0.84	0.91	0.92	0.71	0.80	0.80	0.79	0.40	0.70	0.68	0.67	0.00	0.62	0.59	0.56	0.00
		1.25	0.88	1.00	0.97	0.82	0.85	0.88	0.86	0.70	0.76	0.75	0.75	0.00	0.68	0.66	0.64	0.00
		2.5	0.97	1.00	1.00	0.81	0.95	0.97	0.98	0.84	0.90	0.94	0.96	0.00	0.84	0.86	0.87	0.00
		5	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	1	0	0.85	0.82	0.78	0.58	0.72	0.64	0.58	0.26	0.61	0.50	0.42	0.00	0.52	0.39	0.30	0.00
		0.5	0.84	0.91	0.91	0.71	0.81	0.80	0.79	0.46	0.70	0.69	0.67	0.00	0.64	0.62	0.60	0.00
		1.25	0.87	0.95	0.96	0.82	0.85	0.85	0.85	0.73	0.76	0.76	0.75	0.00	0.71	0.70	0.69	0.00
		2.5	0.97	1.00	1.00	0.82	0.95	0.97	0.98	0.83	0.90	0.94	0.97	0.00	0.86	0.89	0.91	0.00
		5	1.00	1.00	1.00	0.83	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	0.83	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	2	0	0.90	0.90	0.90	0.58	0.87	0.86	0.84	0.33	0.84	0.81	0.78	0.00	0.81	0.77	0.74	0.00
		0.5	0.90	0.93	0.93	0.70	0.88	0.88	0.87	0.54	0.84	0.83	0.81	0.00	0.84	0.82	0.81	0.00
		1.25	0.92	0.97	0.99	0.81	0.90	0.92	0.92	0.77	0.86	0.86	0.86	0.00	0.85	0.85	0.84	0.00
		2.5	0.98	1.00	1.00	0.81	0.97	0.98	1.00	0.81	0.93	0.97	1.00	0.00	0.92	0.96	0.99	0.00
		5	1.00	1.00	1.00	0.82	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	0.82	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00

Table 10.6.3.1.2c-2 (cont.)

			β=10°				β=20°				β=30°				β=40°			
			Ns				*Ns*				*Ns*				*Ns*			
ϕ (°)	*B/H*	*b/B*	0	2	4	*c'*=0	0	2	4	*c'*=0	0	2	4	*c'*=0	0	2	4	*c'*=0
30	0.2	0	0.93	0.92	0.91	0.76	0.65	0.64	0.63	0.39	0.51	0.50	0.48	0.11	0.40	0.37	0.36	0.00
		0.5	0.74	0.81	0.80	0.75	0.70	0.66	0.65	0.50	0.57	0.52	0.49	0.21	0.47	0.42	0.39	0.00
		1.25	0.78	0.85	0.86	0.86	0.74	0.73	0.72	0.72	0.63	0.60	0.59	0.38	0.54	0.50	0.47	0.00
		2.5	0.84	0.92	0.93	0.99	0.81	0.82	0.83	0.94	0.72	0.73	0.74	0.74	0.64	0.62	0.61	0.00
		5	0.95	1.00	1.00	1.00	0.93	0.98	1.00	1.00	0.88	0.95	1.00	0.97	0.80	0.85	0.87	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	0.5	0	0.79	0.79	0.78	0.70	0.63	0.59	0.55	0.36	0.50	0.43	0.39	0.13	0.39	0.32	0.27	0.00
		0.5	0.76	0.87	0.87	0.74	0.72	0.71	0.70	0.51	0.58	0.56	0.54	0.24	0.49	0.46	0.43	0.00
		1.25	0.79	0.85	0.92	0.87	0.75	0.73	0.76	0.72	0.63	0.62	0.61	0.45	0.54	0.52	0.50	0.00
		2.5	0.87	0.91	1.00	0.99	0.84	0.85	0.90	0.98	0.74	0.78	0.80	0.80	0.67	0.70	0.71	0.00
		5	0.97	1.00	1.00	1.00	0.95	1.00	1.00	1.00	0.90	1.00	1.00	1.00	0.85	0.94	0.98	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	1	0	0.79	0.75	0.73	0.67	0.63	0.53	0.49	0.41	0.55	0.41	0.35	0.24	0.48	0.33	0.26	0.00
		0.5	0.78	0.87	0.89	0.74	0.75	0.74	0.74	0.51	0.64	0.62	0.60	0.35	0.59	0.56	0.54	0.00
		1.25	0.81	0.90	0.91	0.88	0.78	0.78	0.78	0.72	0.68	0.67	0.66	0.58	0.64	0.62	0.61	0.00
		2.5	0.88	0.99	1.00	0.96	0.85	0.90	0.92	0.95	0.78	0.81	0.84	0.88	0.75	0.78	0.80	0.00
		5	0.97	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.92	1.00	1.00	1.00	0.89	0.98	1.00	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	2	0	0.88	0.88	0.87	0.65	0.87	0.85	0.83	0.48	0.85	0.82	0.80	0.38	0.83	0.80	0.76	0.00
		0.5	0.89	0.91	0.91	0.75	0.89	0.89	0.87	0.58	0.88	0.86	0.84	0.51	0.87	0.85	0.82	0.00
		1.25	0.90	0.92	0.93	0.88	0.90	0.90	0.90	0.75	0.89	0.87	0.87	0.70	0.89	0.87	0.86	0.00
		2.5	0.97	1.00	1.00	1.00	0.96	0.97	0.98	0.98	0.92	0.94	0.96	0.95	0.91	0.92	0.94	0.00
		5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
40	0.2	0	0.69	0.69	0.69	0.78	0.51	0.48	0.47	0.37	0.37	0.33	0.30	0.16	0.27	0.23	0.20	0.05
		0.5	0.65	0.73	0.71	0.74	0.60	0.55	0.53	0.38	0.64	0.38	0.35	0.25	0.34	0.29	0.25	0.13
		1.25	0.68	0.77	0.75	0.86	0.63	0.60	0.58	0.55	0.74	0.44	0.42	0.39	0.39	0.34	0.31	0.25
		2.5	0.72	0.83	0.84	1.00	0.68	0.68	0.68	0.76	0.87	0.53	0.53	0.62	0.45	0.43	0.41	0.48
		5	0.80	0.93	0.95	1.00	0.76	0.82	0.85	1.00	1.00	0.72	0.76	1.00	0.57	0.61	0.63	0.94
		10	0.94	1.00	1.00	1.00	0.91	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.76	0.93	1.00	1.00
	0.5	0	0.67	0.69	0.67	0.69	0.50	0.45	0.43	0.35	0.36	0.30	0.26	0.17	0.27	0.20	0.17	0.07
		0.5	0.68	0.81	0.81	0.73	0.63	0.62	0.61	0.46	0.47	0.44	0.41	0.25	0.39	0.35	0.32	0.09
		1.25	0.70	0.82	0.84	0.85	0.65	0.65	0.66	0.60	0.51	0.49	0.47	0.40	0.43	0.41	0.39	0.18
		2.5	0.76	0.92	0.96	1.00	0.72	0.77	0.80	0.81	0.59	0.62	0.63	0.60	0.54	0.56	0.56	0.37
		5	0.84	1.00	1.00	1.00	0.81	0.91	0.94	1.00	0.71	0.82	0.88	1.00	0.67	0.77	0.83	0.84
		10	0.96	1.00	1.00	1.00	0.94	1.00	1.00	1.00	0.89	1.00	1.00	1.00	0.86	1.00	1.00	1.00
	1	0	0.69	0.64	0.62	0.70	0.63	0.48	0.43	0.45	0.58	0.39	0.33	0.32	0.54	0.33	0.27	0.24
		0.5	0.77	0.81	0.82	0.74	0.75	0.73	0.72	0.49	0.71	0.66	0.62	0.38	0.68	0.62	0.57	0.30
		1.25	0.78	0.84	0.85	0.84	0.77	0.76	0.75	0.64	0.73	0.69	0.66	0.55	0.71	0.66	0.63	0.48
		2.5	0.83	0.92	0.95	1.00	0.81	0.85	0.87	0.85	0.76	0.78	0.79	0.76	0.75	0.76	0.77	0.72
		5	0.89	1.00	1.00	1.00	0.87	0.95	0.98	1.00	0.80	0.90	0.95	1.00	0.80	0.89	0.94	1.00
		10	0.98	1.00	1.00	1.00	0.97	1.00	1.00	1.00	0.94	1.00	1.00	1.00	0.93	1.00	1.00	1.00
	2	0	0.93	0.92	0.89	0.45	0.92	0.90	0.87	0.60	0.91	0.88	0.84	0.53	0.89	0.85	0.81	0.47
		0.5	0.93	0.95	0.93	0.76	0.93	0.92	0.90	0.65	0.92	0.89	0.87	0.64	0.92	0.89	0.86	0.60
		1.25	0.93	0.95	0.94	0.86	0.93	0.93	0.92	0.78	0.93	0.91	0.89	0.74	0.93	0.90	0.88	0.74
		2.5	0.94	0.99	1.00	1.00	0.94	0.98	0.98	0.92	0.94	0.97	0.97	0.87	0.94	0.96	0.96	0.88
		5	0.95	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.98	1.00	1.00	1.00	0.96	1.00	1.00	1.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00	1.00

Seismic Site Class

SEISMIC SITE CALCULATIONS
PROJECT NAME: Cobbossee Trail Over Cobbossee Stream
PROJECT NUMBER: 179450405
LOCATION: Gardiner, Maine

SPT Correction Factor: 1.3
TEST BORINGS USED FOR ANALYSIS:
BORING NO: B-1

N-Value	Corrected N-Value	Thickness (d) [feet]	d/N
5	6.5	2	0.31
35	45.5	3	0.07
100	100	3	0.03
45	58.5	2	0.03
68	88.4	2	0.02
83	107.9	3	0.03
87	113.1	3	0.03
100	100	5	0.05
100	100	5	0.05
100	100	5	0.05
100	100	5	0.05
100	100	5	0.05
92	119.6	5	0.04
78	101.4	5	0.05
62	80.6	9	0.11
74	96.2	18	0.19
88	114.4	12	0.10
Glacial Till	100	8	0.08
	Sum	100	1.34
		N' =	74.7

SITE CLASS: C

> 50 = C
15 to 50 = D
<15 = E

SPT Correction Factor: 1.3
TEST BORINGS USED FOR ANALYSIS:
BORING NO: B-2

Field N-Value	Corrected N-Value	Thickness (d) [feet]	d/N
15	20	8	0.41
4	5	5	0.96
100	100	2	0.02
100	100	2	0.02
100	100	4	0.04
100	100	5	0.05
80	104	5	0.05
50	65	5	0.08
63	82	5	0.06
77	100	5	0.05
71	92	5	0.05
64	83	5	0.06
59	77	5	0.07
60	78	5	0.06
61	79	12	0.15
Bedrock	100	22	0.22
	Sum	100	2.35
		N' =	42.5

SITE CLASS: D

> 50 = C
15 to 50 = D
<15 = E

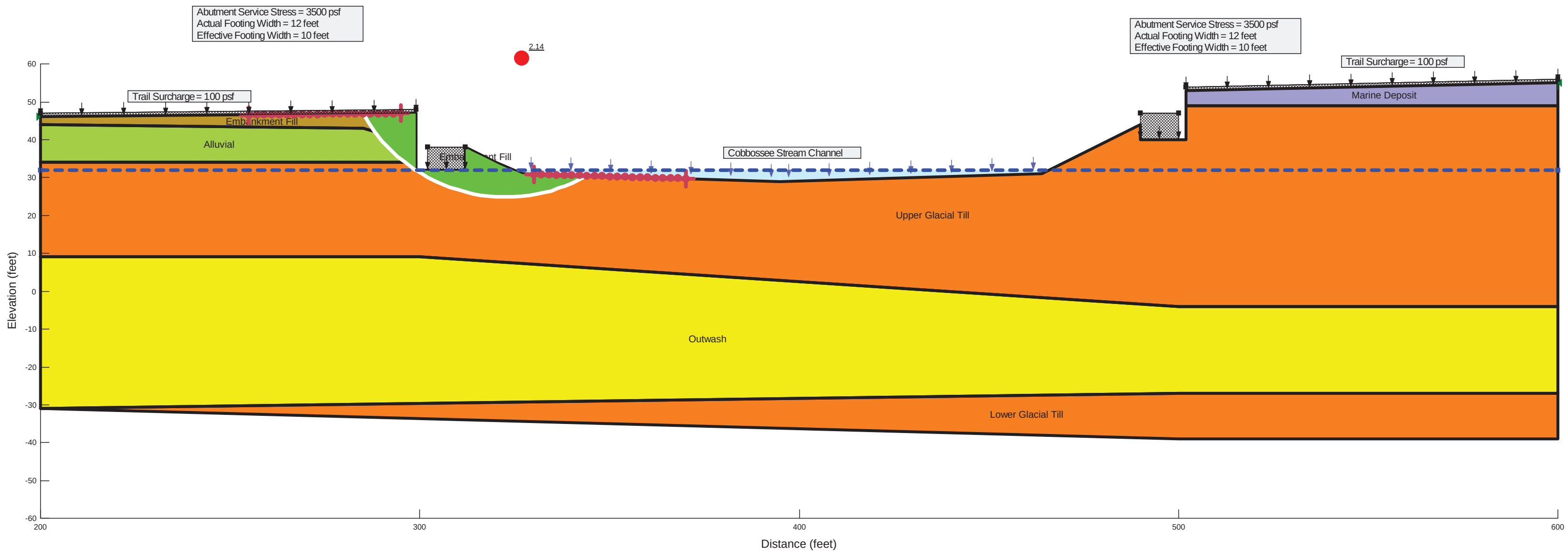
Notes:
-For SPT refusal use N=100 for field and Corrected value
-Bedrock assumed to have SPT N-value of 100 blows per foot
-At B-2 about 8 feet fill will be added. Use corrected N-value of 20 bpf.

Recommend Site Class D for the entire site.

Global Stability

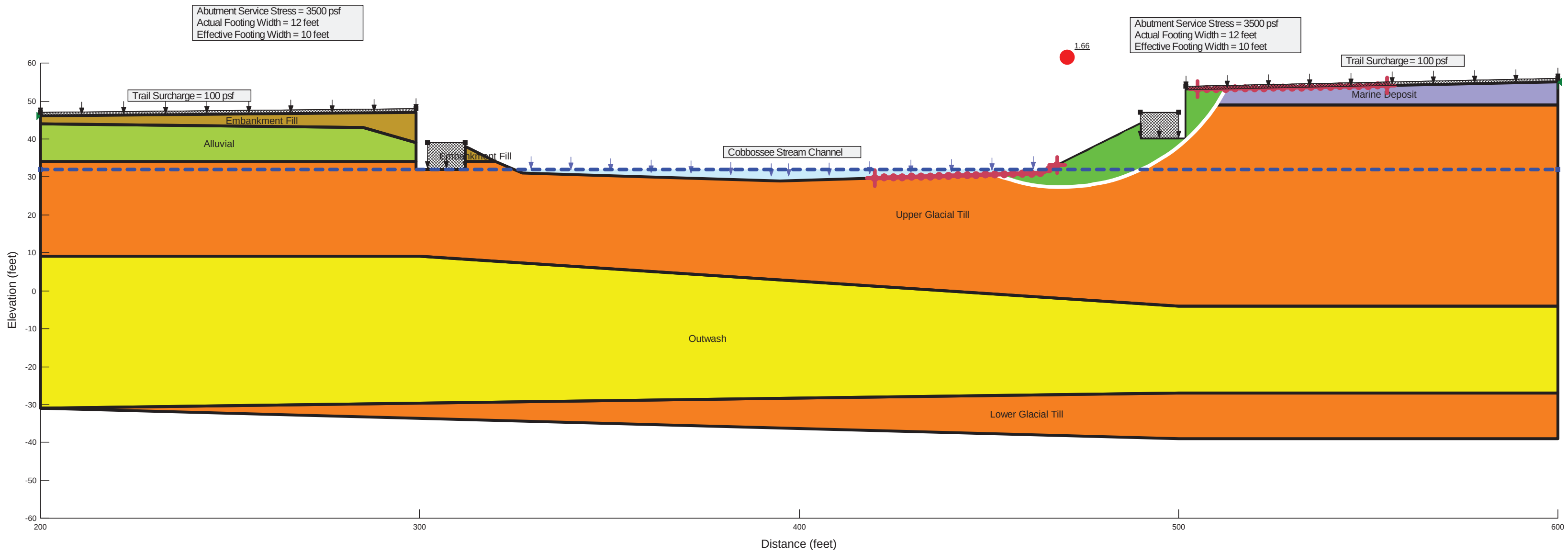
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Alluvial	110	0	29
	Embankment Fill	125	0	34
	Lower Glacial Till	135	0	38
	Marine Deposit	120	0	28
	Outwash	130	0	36
	Upper Glacial Till	135	0	38

Cobbossee Stream Crossing
SouthAbutment
Static Analysis



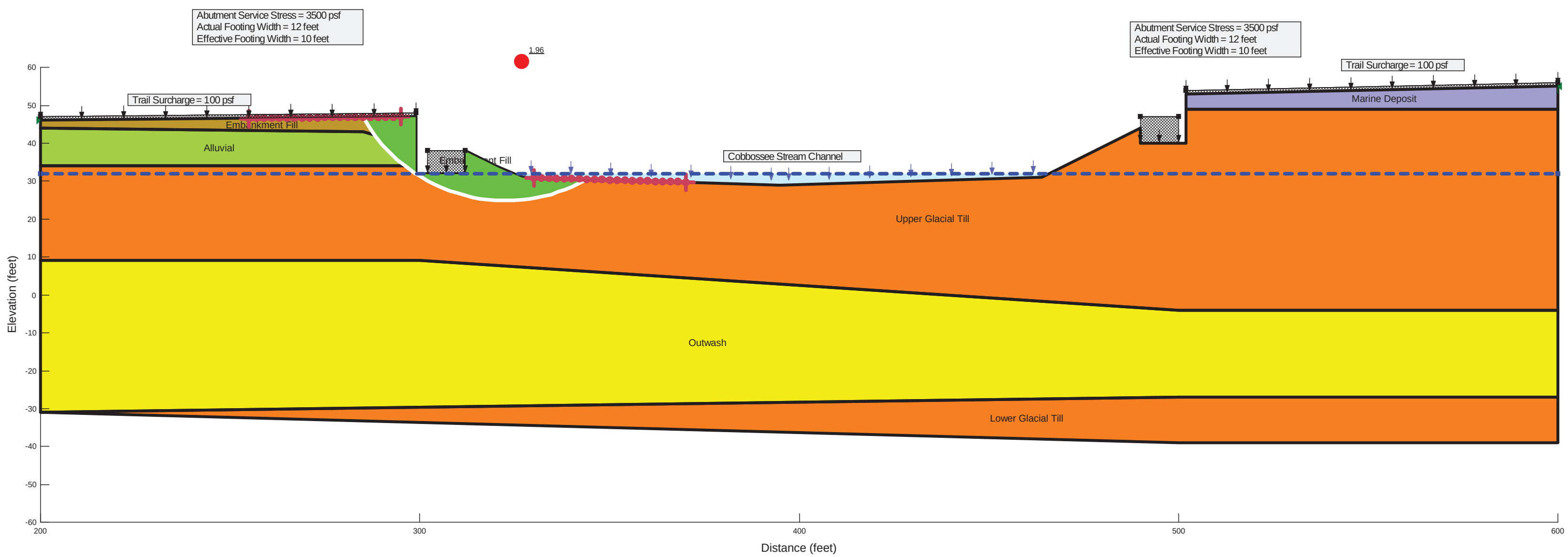
Cobbossee Stream Crossing
North Abutment
Static Analysis

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Alluvial	110	0	29
	Embankment Fill	125	0	34
	Lower Glacial Till	135	0	38
	Marine Deposit	120	0	28
	Outwash	130	0	36
	Upper Glacial Till	135	0	38



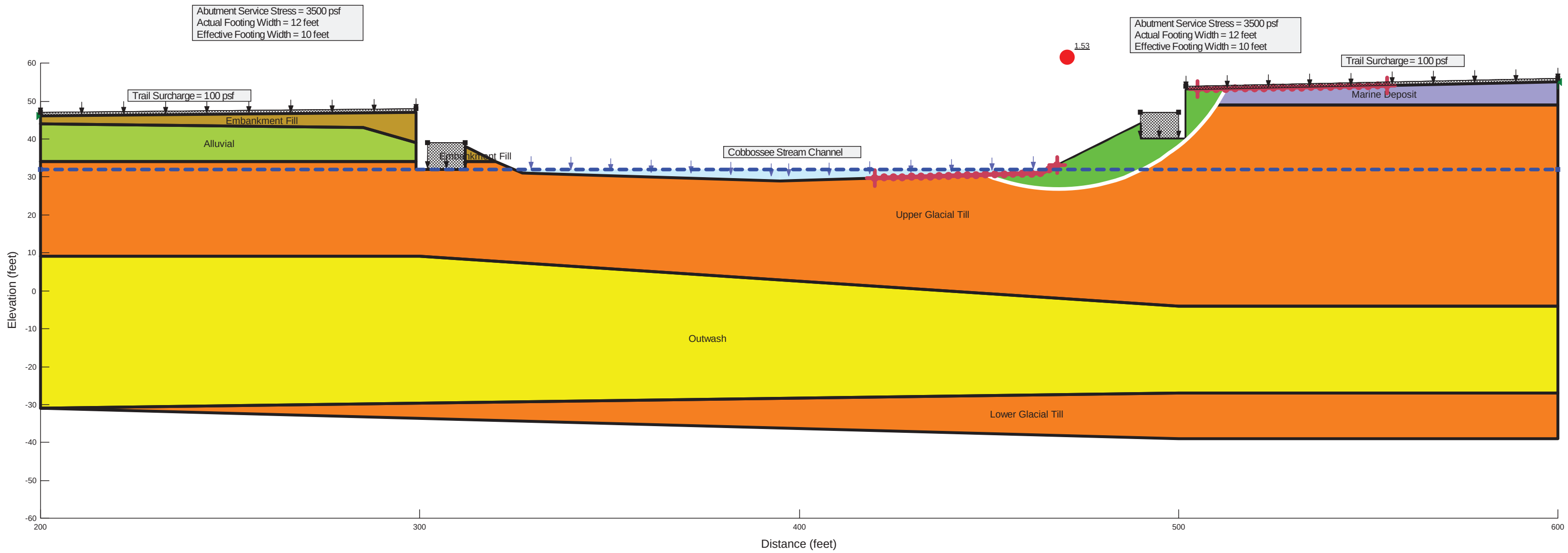
Cobbossee Stream Crossing
South Abutment
Seismic Analysis

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Alluvial	110	0	29
	Embankment Fill	125	0	34
	Lower Glacial Till	135	0	38
	Marine Deposit	120	0	28
	Outwash	130	0	36
	Upper Glacial Till	135	0	38



Cobbossee Stream Crossing
NorthAbutment
Seismic Analysis

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Alluvial	110	0	29
	Embankment Fill	125	0	34
	Lower Glacial Till	135	0	38
	Marine Deposit	120	0	28
	Outwash	130	0	36
	Upper Glacial Till	135	0	38



Frost Depth

5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that sp[...] conditions vary, the depth of frost penetration should be ca[...] Geotechnical Designer.

Use the following Parameters:
-Coarse Grained
-15% moisture
-Freezing Index 1550
-Fines content = 31.4 to 39.1%
-moisture content = 8.9 to 13%

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

By interpolation
Frost Depth = 76.3 inches

By interpolation
Frost Depth = 55.2 inches

Recommend 72 inches or 5.5 feet

Notes: 1. w = water content
2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

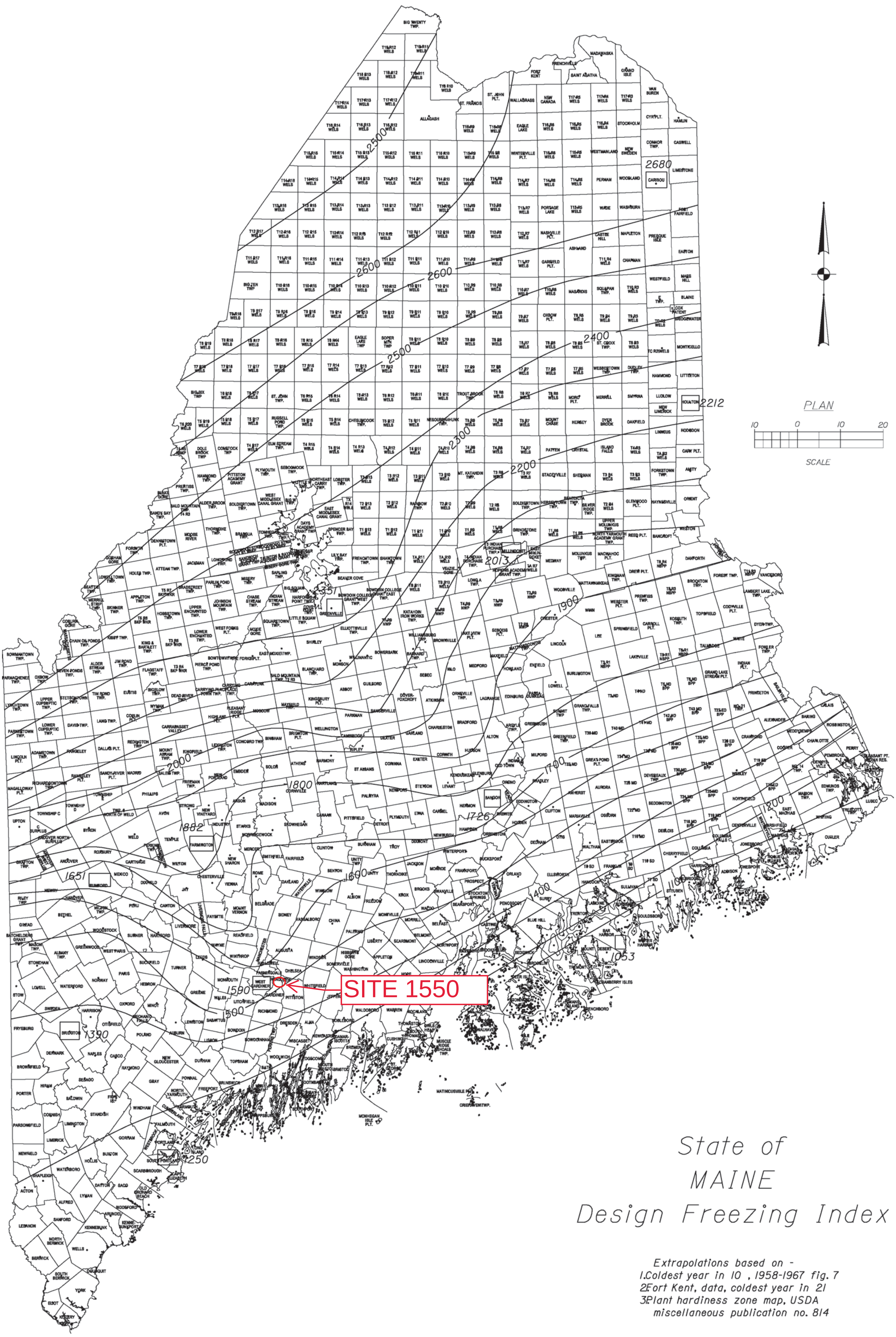


Figure 5-1 Maine Design Freezing Index Map

Example 5-1 illustrates how to use Table 5-1 and Figure 5-1 to determine the depth of frost penetration:

Example 5-1 Depth of Frost Penetration

Given:	Site location is Freeport, Maine
Soil conditions:	Silty fine to coarse Sand

Step 1. From Figure 5-1 Design Freezing Index = 1300 degree-days
Step 2. From laboratory results: soil water content = 28% and major constituent Sand
Step 3. From Table 5-1: Depth of frost penetration = 56 inches = 4.7 feet

Spread footings founded on bedrock require no minimum embedment depth. Pile supported footings will be embedded for frost protection. The minimum depth of embedment will be calculated using the techniques discussed in Example 5-1. Pile supported integral abutments will be embedded no less than 4.0 feet for frost protection.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

The final depth of footing embedment may be controlled by the calculated scour depth and be deeper than the depth required for frost protection. Refer to Section 2.3.11 Scour for information regarding scour depth.

5.2.2 Seal Cofferdams

Seal cofferdams are used when a substructure unit must be constructed with its foundation more than 4 feet below the water table, to counteract the buoyant forces produced during pumping of the cofferdam. Once the cofferdam is constructed, the seal is placed under water and water is then pumped out of the cofferdam. This provides a dry platform for construction of the spread footing, or in the case of a pile foundation, the distribution slab. When a seal is needed, the top of footing or distribution slab is located approximately at streambed, and the depth of seal is calculated based upon the buoyancy of the concrete under the expected water surface during construction. The following formula can be used:

$$145 \cdot y = 62.4 \cdot z$$

where:

145 lb/ft^3 =	unit weight of concrete
62.4 lb/ft^3 =	unit weight of water
y =	the depth of seal from top of seal to bottom of seal
z =	the depth of water from water surface to bottom of seal

6.0 LIMITATIONS

6.1 USE OF REPORT

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

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

6.2 SUBSEQUENT INVOLVEMENT

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

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

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

6.3 REPRESENTATION AND INTERPRETATION OF DATA

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

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