March 1, 2024

Mr. Paul Paquin, Chair
Town of Hull Conservation Commission
253 Atlantic Ave.
Hull, MA 02045

Subject: 51 Harborview Road
Hull, Massachusetts 02045
Response to Comments
CEC Project 334-762

Dear Mr. Paquin:

On behalf of the Applicant for the above referenced property, Civil & Environmental Consultants, Inc. ("CEC") has prepared this memorandum and attached Contractor's Qualifications, Soil Nail and Slope Stability Design Narrative (the "Design Narrative") and Aerial Overlay Exhibits in response to comments provided in a memorandum by Mr. Chris Krahforst, dated February 1, 2024.

The comments provided are summarized below in italics, followed by CEC's response in bold.

CONSERVATION COMMISSION COMMENTS

1. *Describe purpose, document functionality, and assess efficacy of interim measure. (See how this fits with 2-j and –p below.)*

CEC Response: The purpose of the interim measure is to divert stormwater away from the backside of the existing wall and down the existing slope via a drain pipe. Impermeable liner was also installed over the ground surface from the backside of the wall to about 15 feet beyond to eliminate stormwater infiltration at the back of the wall. We inspected the initial installation and determined that the contractor had used a perforated pipe connected to the catch basin, rather than the solid pipe that we specified. That has been corrected, and the pipe end has been located above the elevation of wave runup and in a dissipation area as specified.

2. *Meet requests as outlined in bullets within the Peer-Review (see pages 8-12), particularly:*

a. *Ground water effects including addressing the assumptions used in the plan reviewed that may not be valid as discussed at the 1/30 hearing (design standards factoring a fully saturated slope) and included in the peer review.*

CEC Response: CEC performed a revised stability analysis assuming a fully saturated slope (i.e. groundwater at the ground surface along the entirety of the slope. Because this is considered a transient condition similar to seismic loading, CEC understands a factor of safety of at least 1 will be considered acceptable for design purposes. Refer to the Soil Nail and Slope Stability Design Narrative (the "Design Narrative") included as Attachment B for additional information, which demonstrates compliance with the design factor of safety.

b. *Gabion basket comments (bottom of page 8 to top of p.9) and concerns raised by CZM regarding excess weight of gabion baskets on bank stability.*

CEC Response: The contractor will install drilled piles along the bottom row of the gabions to increase the global stability of the gabions and bank which will provide an adequate factor of safety (i.e. greater than 1.5). The design submittal will be reviewed by CEC. Gabion basket layout, size, subgrade information, and backfill requirements are shown on Detail 6 on Sheet C800 and the adjacent Gabion Notes. Revised calculations are provided in the Design Narrative included as Attachment B.

c. *Comment on proposed grading (1st bullet, p. 9)*

CEC Response: The existing seawall is not on the Applicant's property and is not part of this project. In the Commission's review of the 2019 NOI for this property, the Commission received a report that concluded the "failure of the [Town's] stone block wall at the toe of the slope has caused a progressive failure of the above supported soil." "Adequate toe protection will need to be designed and installed to prevent additional sliding failures and scour." It is our understanding that the Town has had a project under consideration for at least 5 years to repair the seawall on the Town's land.

As part of the Fitzgerald project, vegetation will be stablished on the landslide material within the 51 Harborview property. Notes have been added to the plans confirming that erosion control matting shall be placed on construction with slopes greater than 3:1 (in addition to vegetation) and perimeter erosion controls are shown at the toe of slope.

Additionally, the Sewer Department expressed concern about whether the landslide material may have resulted in ground movement around the sewer line. The sewer department reported having inspected the sewer line and concluded that it appeared to have shifted laterally by a half inch. There is no evidence to indicate that the slide of the material on this site actually caused any shift of the sewer as an offset of that amount could have been present when the sewer line was first installed or after it was installed due to minor lateral movements that occur behind many retaining walls as earth pressures push against the walls. Nevertheless, the present question is what effect could the currently vegetated and stable soil above the sewer line now have on the lateral displacement of the sewer. We believe that the additional existing soil above the sewer would have no additional effect on the sewer line. Rather, to the extent that this property poses any concern about lateral movement – the Soil Nail work should be approved as soon as possible so that the upper slope is stabilized and effectively eliminates the risk of another failure of the upper slope and new lateral movement.

Additionally, temporary construction activity in the vicinity of the existing sewer main is not anticipated to be problematic due to the existing depth of the sewer main and lightweight equipment to be used to execute the work.

d. Comment raised over concern that Soil Nail Structure (SNS) drainage strips can tend to clog-address how this will be mitigated/addressed. (2nd bullet, page 9)

<u>CEC Response:</u> As stated in the GZA Memorandum dated January 18, 2024, installation of drainage strips are typical practice during construction of soil nail structures and have been proven effective for the life of the structure. The drainage strips specified in CEC's construction drawings have geotextile separation fabric fused to the inner core to minimize the migration of fine particles into the drainage system. The drainage system will include weepholes installed at the base of the soil nail facing that ties into the rip rap berm proposed at the base of the facing. The weepholes can be cleaned out from the ground surface periodically, if necessary. However, the strip drain portion of the drainage system will be inaccessible after construction due to placement of the shotcrete.

e. Comment to limit seepage migration from previous installation(s) (middle, p. 9)

<u>CEC Response:</u> As part of the interim solution, a french drain was installed just north of the new edge of the patio. The drain pipe is about 11 feet back from the edge of the existing wall and ties into a drainage manhole towards the northeast corner of the property. The French drain is a 4" perforated pipe surrounded by crushed stone and is designed to catch runoff from the patio. Impermeable liner is installed below the stone bedding and drain

to eliminate infiltration near the face of the existing retaining wall. In addition, the existing grading from the back of the existing retaining wall to the French drain is pitched towards the drain and away from the wall.

A second french drain was also installed along the east edge of the patio, which ties into the same drainage manhole as the above noted drain. This drainage manhole contains a pipe which connects to a catch basin, and an outlet pipe is installed in this catch basin which drains down the slope. See the Concept Drawings in the Revised Design Narrative for additional details.

f. Comment to capture runoff from patio, etc. (p.9)

CEC Response: Above response addresses current conditions. For the final conditions once the slope is repaired, the recently installed French drains behind the existing retaining wall will be replaced with trench drains which will tie into the existing catch basin near the northeast corner of the top of bluff. The impermeable liner will also be restored to minimize infiltration behind the soil nails.

g. Include existing wall in design drawings and see comment #4 below (p. 9)

CEC Response: The existing wall has been added to the design drawings.

h. Address concerns about additional vegetation (coastal bank) removal (p. 9, near bottom)

CEC Response: The proposed grading and soil nail layout, especially in the westerly portion of the site, has been modified in an effort to minimize disturbance where current vegetation exists. Areas that are disturbed during construction of the soil nails will be re-vegetated.

i. Incorporate discussion on 310 CMR 10 relevant performance standards (coastal banks, bottom p.9 in "Additional Comments" section and last bullet on p. 11

CEC Response: See below for responses to the Regulations under Section 10.30 subsections 6 through 8. Regulations are identified with italicized font with CEC's comments below.

WHEN A COASTAL BANK IS DETERMINED TO BE SIGNIFICANT TO STORM DAMAGE PREVENTION OR FLOOD CONTROL BECAUSE IT IS A VERTICAL BUFFER TO STORM WATERS, 310 CMR 10.30(6) THROUGH (8) SHALL APPLY:

(6) Any project on such a coastal bank or within 100 feet landward of the top of such coastal bank shall have no adverse effects on the stability of the coastal bank.

The analyses supporting the soil nail design indicate that the soil nail design will have a factor of safety greater than 1.5 for long term stability, which is typically the accepted standard for similar engineering designs. The proposed soil nails will have no adverse effects on the stability of the coastal bank; it will only improve it.

(7) Bulkheads, revetments, seawalls, groins or other coastal engineering structures may be permitted on such a coastal bank except when such bank is significant to storm damage prevention or flood control because it supplies sediment to coastal beaches, coastal dunes, and barrier beaches.

The portion of the coastal bank being modified as part of this project is not supplying sediment to the adjacent coastal beach; therefore this standard is not applicable.

(8) Notwithstanding the provisions of 310 CMR 10.30(3) through (7), no project may be permitted which will have any adverse effect on specified habitat sites of rare vertebrate or invertebrate species, as identified by procedures established under 310 CMR 10.37.

Based on our review of the estimated habitat map of state-listed rare wetlands wildlife, the project is not located within a mapped habitat. It is not anticipated to have an adverse effect to any nearby habitat areas.

It is therefore CEC's conclusion that the project as designed and presented in this NOI fully complies with the regulatory conditions of 310 CMR 10.30.

j. *Address "perched groundwater table" concerns (p. 10)*

<u>CEC Response</u>: CEC performed an additional stability analysis with the assumption that groundwater is at the ground surface (i.e. a fully saturated slope). The results of our analysis indicated the slope will have a factor of safety above 1.0 for the fully saturated condition, which is within an appropriate factor of safety for this very conservative analysis. Refer to the revised Design Narrative for additional information on this analysis.

k. Address how interim measure fits within the long term project (p.10)

<u>CEC Response</u>: The two French drains will be replaced with at-grade trench drains. The outlet pipe may need to be slightly moved as the gabions are being installed.

l. Specify design life (for all alternatives, p.10)

<u>CEC Response</u>: Refer to Section 9.0 of the revised Design Narrative for information on the design life of the soil nails and gabions.

m. See bullet regarding alternative that restores coastal bank and etc. (2nd bullet up from bottom p. 10), including removal of a portion, if not all, of the patio and deck within 20+ feet from the coastal bank.

<u>CEC Response</u>: See response to item 5 below.

n. Address construction sequencing issue and other issue raised last bullet, bottom p. 10.

<u>CEC Response</u>: Refer to Section 8.0 of the revised Design Narrative for additional information on construction sequencing.

o. Address bullet (top of p. 11) about construction logistics and constructability aspects of the SNS. Address equipment and earth loading on the sewer force main alignment and access from the beach to the embankment.

<u>CEC Response</u>: Refer to response to comment "C" for additional detail regarding discussion of the soil above the sewer main. Refer to Section 8.0 of the revised Design Narrative for additional information on construction logistics and constructability aspects.

p. Please review and comment on CZM's review of this project (1st bullet, middle p. 12)

<u>CEC Response</u>: Responses to CZM comments are below:

- **Drilling equipment and access are indicated in Section 8 of the revised Design Narrative.**
- **Soil nails are regularly installed in glacial till soils. Drill bits typically consist of tungsten carbide which can readily advance through stiff soil, cobbles, boulders, and rock.**

- **The shotcrete facing will contain drain strips behind the facing to relieve the facing of hydrostatic pressures. Further, proposed grading will pitch water away from the backside of the facing, and recently installed drainage features will also help drain water away from the backside of the facing.**
- **The re-grading of the coastal bank has been modified to minimize disturbance to the bank.**
- **The design as currently proposed results in pulling back of the top of bank by more than 7-ft in some locations when compared to the historic grades of the site prior to the slope failure. Refer to the Aerial Overlay Exhibits included as Attachment C for a comparison of the conditions from 2010 prior to the construction of the deck, 2020 after the construction of the deck, current conditions and proposed conditions.**

  Pulling back the top of the bank further to create a more stable slope will result in additional disturbance to the coastal bank and will result in grade differentials at the property line which would require the construction of retaining structures at each side lot line property line. That is inconsistent with the Commission's policy to avoid construction within 10 feet of a property line; and would require the structurally engineered walls to provide lateral support to each of the abutting properties.
- **Comments on additional weight from the rip rap and gabions: Much of the slope near the east property line will be re-graded to a shallower slope except for areas immediately adjacent to the property line where grades cannot change. The shallower slope will be more stable than it currently is, particularly due to the toe key of rip rap proposed at the bottom of the rip rap section (see Detail 5 on Sheet C800). Further, to increase the stability of the gabion baskets, the contractor will install vertical piles/soil nails along the base of the baskets to increase the global stability factor of safety.**
- **Erosion control vegetation will be established over most of the area below the soil nails, and rip rap will be established over the other areas where the proposed slopes are too steep to establish vegetation.**
- **Regarding end effect erosion – the grades at the property line are generally staying the same, and there will not be concentrated flow on the rip rap near the property line which is usually the cause of end effect erosion.**

q. Establish bank monitoring protocol to capture short-term and long-term assessment of bank stability (last bullet, p. 12)

<u>CEC Response</u>: In the interim while the permanent stabilization solution is being planned, CEC recommends the establishment of deformation monitoring points at each existing column along the column line closest to the existing wall, in addition to every other column along the column line supporting the middle of the deck. The points can consist of PK nails placed on the columns; the northing, easting, and elevation data should be read every month. Once the project has been completed, CEC can establish an updated protocol for additional points to monitor long-term stability of the coastal bank. Routine visual observation should also be performed and CEC should be notified of any visual indications of movement if observed.

3. *Provide a quality/qualitative alternatives analysis. The Commission (and abutters) are interested in more detail on restoring coastal bank to natural conditions (see 2-l, -o above), including but not limited to:*
    - *Pulling the top of the coastal bank further away from the coast,*
    - *Regrading to a lesser slope, and*
    - *Revegetation with deep-rooted salt tolerant vegetation (see CZM recommendations).*

<u>CEC Response</u>: The performance standards of 310 CMR 10.30 applicable to this project do not require such an alternatives analysis. The key is stability. Nevertheless, our responses are as follows:

- **Bullet 1: The design as currently proposed results in pulling back of the top of bank by more than 7-ft in some locations when compared to the historic grades of the site prior to the slope failure. Pulling the top of the coastal bank further away from the coast is not the applicant's objective and would result in additional disturbance to the coastal bluff, resulting in removal of large volumes of coastal bank material that is inconsistent with the standard under the 10.30 regulations and CZM policy to not remove materials that provide a buffer to coastal flooding. Further, pulling the top of the coastal bank further back would result in significant grade differentials at the property line which would likely require the design and construction of engineered retaining structures to provide lateral support for each of the abutting properties.**
- **Bullet 2: Much of the existing northerly slope at 51 Harborview will be re-graded to a lesser slope once the soil nails are installed. Regrading of the entire coastal bank on this site to a less steep slope would require either moving large volumes of materials onto the site, while completely removing all vegetation and then needing to use engineered materials and reinforcement to hold that large volume of unconsolidated material in place; or removing a volume of material at the top of**

the slope, which we addressed above. Refer to Response to Comment 5 for additional description of this alternative. These options are not consistent with the goals of the applicant, and would create a grade differential at each of the side lot lines that would require construction of engineered retaining walls to provide lateral support for those properties. We concluded that neither of these are viable or preferable alternatives from an engineering perspective, regardless of the applicant's goals.

- **Bullet 3: It is the intention of the applicant to revegetate the exposed and altered slopes below the soil nails with deep-rooted salt tolerant vegetation. The proposed 2:1 slopes which are proposed along much of the bank are shallow enough to accommodate vegetation. For steeper slopes such as those near the east abutter, rip rap will be installed.**

4. *Address the Toe of the Coastal Bank. The Commission suggested a stabilization plan that details how this portion of the coastal bank will be included in the overall design (see 2 -f above).*

CEC Response: Currently, the toe of the coastal bank is mostly vegetated and is at a shallow enough slope where it is currently stable, other than the long term instability caused by the failure of a part of the Town's seawall on the Town's property. CEC recognizes that part of the toe is in a wave zone subject to wave action. CEC also is aware that the Town has a project it committed to previously to repair that stone revetment/seawall. CEC anticipates that repair of the stone revetment will protect the toe of the coastal bank from addition erosion. This NOI does not propose work on any property other than that owned by the Applicant.

5. *Address the viability, practicality and constructability aspects of rebuilding the coastal bank to match adjacent existing soil and vegetation and how this approach differs in performance with the proposed SNS.*

CEC Response: Fill placement using standard earthwork will not achieve soil shear strengths comparable to the strengths of surrounding coastal bank slopes comprised of over-consolidated glacial till and will not be adequate for long-term stability. Re-establishing the coastal bank to match the adjacent gradients will require removal of all of the vegetation on the coastal bank, and then installation of geogrid reinforcement elements to reconstruct the bank as a reinforced soil slope (RSS) due to the steep slope gradient. Though an alternatives analysis is not required under 10.30, we would not support proposing a project that requires removal of all of a coastal bank's vegetation, when an alternative is available (the SNS as proposed) that will fully stabilize the upper

slope and directly alter only a small fraction of the existing surface area of that bank and disturb minimal existing vegetation, Although reconstructing the bank as an RSS may be technically feasible, it is CEC's opinion that RRS as a remediation option is impractical. The construction of RSS would result in significantly more earth disturbance to establish a uniform, flat fill area, which is bearing on competent glacial till material and sufficiently sized to achieve adequate geogrid embedment lengths. The fill area would need to extend horizontally into the slope approximately 15 to 20 feet at the elevation of the existing coastal bank toe of slope and would require all vegetation, significant removal of existing material and importing of soil. The heavy equipment needed for such a project is also not consistent with the concerns the Town officials have expressed about construction impacts to the sewer alignment.

6. *Summarize where soil nail technology has been applied including in coastal settings.*

CEC Response: The following bullets provide a brief summary of the use of soil nails applied in coastal settings. Refer to the attached Statement of Qualifications from GeoStabilization International included as Attachment A for additional information on the projects noted below and for additional relevant projects.

- **Pearl Street Slope Repair Project – Laguna Beach, California – In June of 2021, a crumbling rock slope on a coastal bluff required repair. An all terrain access spider excavator, similar to what would be used at the 51 Harborview property, was utilized to install rock anchors (equivalent to soil nails but for rock) to stabilize the coastal bluff. Biodegradable engine oil and hydraulic fluid was used to minimize environmental concerns. The excavator was able to walk up the steep slope to install the top row of anchors.**
- **Spanish Fort Bluff – Emergency Bluff Repair – Spanish Bluff, California – Poor drainage followed by a heavy rain led to the failure of a bluff along the backyard of several residential homes. To prevent additional erosion, a soil nails were installed at the top of the failed sloped and rip rap was installed below the soil nails to move stormwater down and off the slope in a controlled manner.**

7. *Summarize the projects completed to date on the property including any effects relative to slope failure and how the proposed project will address these effects. Also include how the proposed SNS will not have any adverse effect on the stability of the coastal bank.*

CEC Response: On Page 5 of GZA's peer review memo, a time history diagram is present which documents the approximate dates of relevant events at the 51 Harborview property pertaining to the stability of the slope. Based on discussion with the applicant, this timeline is generally correct, although CEC disagrees that the deck as built differed in any

substantive way from the approved design, CEC also generally agrees with GZA's conclusions regarding the effects of the timber wall construction activities with the tie backs and trenches, and that this contributed to the instability which was originally triggered by the failure of the seawall at the toe of the slope. Prior to the work on the timber wall, we do not believe that the construction work caused instability to the slope. The construction of the deck did not destabilize the slope. The deck contains small footings that would not have destabilized the slope, and no infiltration is associated with that project. The slope failure that occurred prior to and that led to the 2019 proposal of the timber wall with tie backs was reviewed by a geotechnical engineer at the time who concluded that it was triggered by the failure of the seawall at the base of the slope.

The proposed soil nails will not have an adverse effect on the stability of the coastal bank. Soil nails will reinforce the existing slope while minimizing additional disturbance to the slope, and will tie the top of the slope together to act as a unit. Drainage improvements to the top of the slope and drainage integrated into the soil nails will relieve the SNS from buildup of additional pressures from a heavy rain event. Of the options proposed for remediation, soil nails are the least disruptive to the existing slope. Past disturbance has led to past failures; and it is our intention to stabilize the slope in a way that minimizes disturbance.

Soil nails are a well-established technology which have been used to stabilize coastal banks under similar geologic and topographic conditions (refer to attached GeoStabilization International Statement of Qualifications).

8. *Address how surface and groundwater will be controlled at the top of the embankment and along the slope. Also address how surface flow on the SNS itself will be controlled/captured/mitigated to prevent further erosion/washout below the SNS.*

CEC Response: See response for 2e and 2f above.

9. *Address how water is prevented from migrating in the drilled holes for the soil nails and how that water will be controlled/mitigated from entering the slope face or underlying soil.*

CEC Response: The drilled holes for the soil nails will be backfilled with cement grout to the ground surface, which will create a watertight seal around the perimeter of the drilled holes.

10. *What measures are necessary to provide support below the SNS and below the remaining slope of the coastal bank beneath the SNS?*

CEC Response: By re-grading the slope immediately below the soil nails to a 2:1 and establishing salt-tolerant vegetation, the slope will be stable. Refer to the slope stability outputs the Revised Design Narrative for the factor of safety down the center of the slope.

11. *Address protective measures to prevent erosion and washout directly below the SNS and lower coastal bank slope interface.*

CEC Response: The rip rap buttress at the bottom of the soil nails is designed to dissipate energy from runoff on the face of the soil nails and prevent erosion of the vegetated slope below the soil nails. Surface water from the area above the soils nails will be collected in the system installed for the interim measure, as it will be modified for the SNS (see response to comments 2e and 2f). Groundwater from behind the soils nails will be managed by the soil nail design itself, which has drainage strips (see discussion in 2d above.)

We hope that you find these responses helpful in your evaluation of the coastal bank stabilization project. Please feel free to contact us with any questions at tsousa@cecinc.com or kskulte@cecinc.com or via phone at (774) 501-2176.

Sincerely,

CIVIL & ENVIRONMENTAL CONSULTANTS, INC.

Tony Sousa, P.E.	Karlis Skulte, P.E.
Project Manager	Principal

Enclosures: Attachment A - GeoStabilization International Statement of Qualifications
Attachment B – Soil Nail and Slope Stability Design Narrative (with attachments)
Attachment C – Aerial Overlay Exhibits

ATTACHMENT A

GEOSTABILIZATION INTERNATIONAL STATEMENT OF QUALIFICATIONS

STATEMENT OF QUALIFICATIONS

GeoStabilization International®

Prepared by GeoStabilization International® (GSI®)

This document contains trade secrets and/or confidential data (collectively, "data") that is exempt from state and federal freedom of information acts, shall not be disclosed outside the Government, and shall not be duplicated, used, or disclosed—in whole or in part—for any purpose other than to evaluate this Statement of Interest. If, however, a contract is awarded to GeoStabilization International, LLC as a result of—or in connection with—the submission of this data, the Government shall have the right to duplicate, use, or disclose the data inside the Government to the extent required for the parties to perform under the contract. Notwithstanding the foregoing, this data shall not be disclosed outside the Government without first obtaining the written authorization of GeoStabilization International, LLC.

With the submission of this document, GeoStabilization International, LLC hereby requests prompt written notice of any request for disclosure of its trade secret or confidential data, as well as the maximum amount of time available under the applicable statute to object in writing to any disclosure of such information.

About GeoStabilization International®

GeoStabilization International® (GSI®) was founded in Colorado in 2002 and offers design-build slope, landslide, and rockfall mitigation across the globe. We employ a minimum of fifty (50) 4-5-person crews and 50+ registered Professional Engineers. Our expertise, tools & technologies, worldwide reach, and design / build / warranty approach allow us to repair virtually any slope stability problem in any kind of geologic setting.

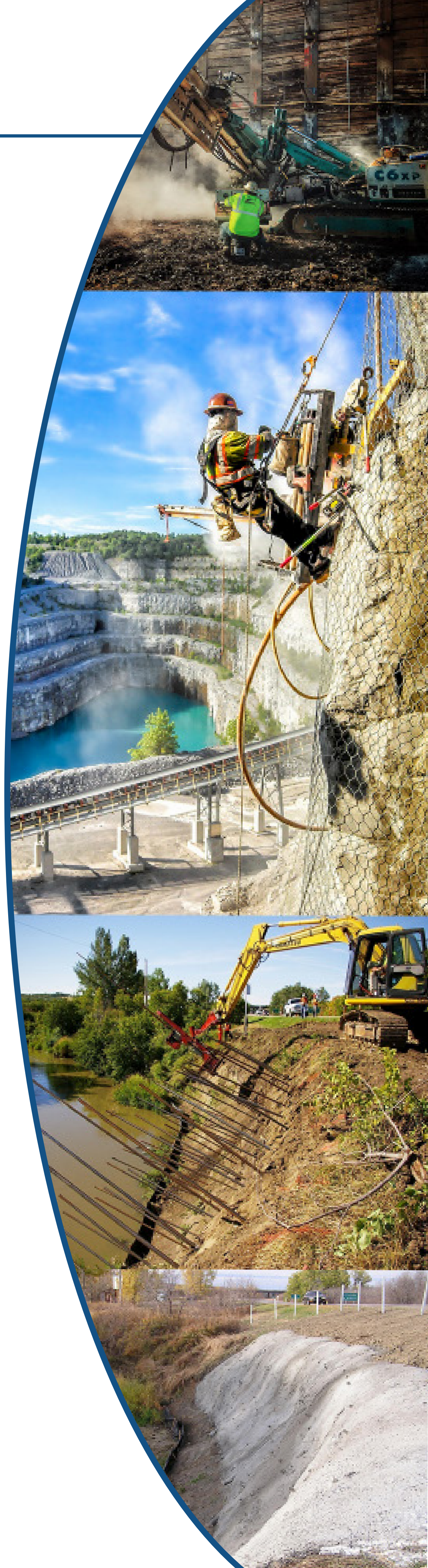
GeoStabilization's comprehensive service capabilities include engineering design and physical stabilization, slope/bluff stabilization; landslide remediation; rockfall mitigation; grouting & foundations; temporary shoring; GSIAbutments™; MSE & historic rock wall repair; rail subgrade and embankment stabilization; micropiles, ground anchors & underpinning; and bridge rehabilitation.

Our tools include patented Soil Nail Launchers™, SPIDER excavators/drills, and purpose-built limited access drill rigs. Our patented technologies include SuperNails™, BioWall® system, ScourMicropiles™, and Geosynthetically Confined Soil® (GCS®) walls. Additionally, our complete equipment inventory includes SuperNailers™ (modified excavator-based rock drills), crane basket drills, compressors, grout plants, shotcrete pumps, welders, and a variety of support trucks, trailers, and other related equipment. Most of the equipment has been custom built or modified specifically for slope, rockfall, or landslide mitigation.

These competencies, tools, and technologies provide our customers the assurance that every geohazard emergency will be solved utilizing a solution designed specifically for that application as GeoStabilization International does not ascribe to the assumption that one solution fits all applications. More importantly, GeoStabilization can and oftentimes does utilize multiple technologies (compaction grouting, launching soil nails, rockfall scaling, etc.) on our projects to leverage their advantages into cost effective solutions for our customers – all without the time-consuming coordination and lost efficiencies encountered when multiple companies are involved.

For more than eighteen years, GeoStabilization has provided more than 6,000 cost effective, technically superior solutions, many completed under emergency declarations and requiring expedited design and rapid mobilization.

GeoStabilization is decentralized and mobile, with a primary focus on supporting geohazard mitigation in emergency response scenarios. As a result of our organization's focus, we can reliably commit to responding to geohazard events within twenty-four hours.



GSI
GeoStabilization International®

Core Values

GeoStabilization's integrated engineering and construction offering provides each of our clients the fastest possible response time, ensures seamless integration of design and construction, and allows rapid assessment and implementation of design changes when unanticipated subsurface conditions are encountered during construction.

GeoStabilization's approach on every project is to provide an initial no-cost site visit and geohazard evaluation (with preliminary engineering). A key point of emphasis for our engineers is to evaluate each site objectively. We will recommend the best approach(es) for remediation, even if it doesn't include our equipment and/or technologies.

We credit our success to the founding philosophy of serving the client with the best technologies, the best solutions, and the fastest possible service. Through many years of training, experience, and this founding philosophy, GSI's project managers and constructors are among the most qualified and most experienced in our industry.

GSI Core Values

Since our inception as a company of 4 people to the 500+ family members to date, we have adhered strictly to our ethical values. These Core Values represent who we are and provide, without question, the standard of behavior by which we conduct our business. These principles are the heart, soul, and character of GeoStabilization International and are:

- Always do the Right Thing
- Ensure Client Best Value
- Advance the Health, Safety, and Future of our Family
- Bring Your Best, Nothing Less
- Advantage by Innovation

GeoStabilization International's reputation is upheld and enhanced or diminished by each person's decision, action and sense of business ethics. Therefore, GSI has established a Third-Party Code of Ethics and Business Conduct that we hold our subcontractors, vendors, suppliers, agents, consultants and business partners to; which mirrors the standards we set for our own employees. This Code may not cover every issue that could arise, but sets the principles and methodology to help guide us in the attainment of a common goal.

GeoStabilization International®

GSI Safety Program/Record

GeoStabilization's training and certification program is one of the best in the industry. Employing a full-time training department, GeoStabilization places great emphasis to ensure its employees are properly trained. We want each employee to be proficient in his or her current tasks and to assume ever-increasing responsibilities on the job-site. Currently 377 GeoStabilization employees are OSHA 10 certified and 126 are OSHA 30 certified. Additionally, 60 employees are ACI Certified Nozzleman. Sixteen (16) rockfall technicians employees are Level I/Level III SPRAT/IRATA trained.

GeoStabilization International has an excellent safety record as demonstrated by our earning the ADSC's Outstanding Safety Program Award for the past six years. Currently our Experience Modifier (EMR) is 0.62 [industry standard is 1.00]. All this translates into the knowledge that GeoStabilization is one of the safest geotechnical contractors in the industry.

ADSC 2014-2021 SAFETY AWARD WINNER

Fast-paced, cutting-edge, and industry leading projects require a fast-paced, cutting-edge, and industry leading safety program. GeoStabilization employs a full time, dedicated safety team, maintains a written safety plan, and trains/re-trains all employees in the most current practices of the industry. In addition to daily toolbox talks and weekly, monthly, and quarterly training.

THE ASSOCIATION OF GEOHAZARD PROFESSIONALS SAFETY RECOGNITION Recipient 2022

Our Credentials: OSHA 10 certified workforce; Purpose-built safety program, specifically designed and developed for geohazard mitigation; all-disciplines company safety committee focusing on geohazard mitigation safety challenges and solutions; SPRAT certification for rope-access slope work; certified equipment operators; CPR and First Aid training across all roles; defensive driving courses.

The table below contains the supplemental safety statistics and data demonstrating GeoStabilization's dedication to maintaining our culture of safety. The specific metrics are described as follows:

CERTIFIED Gold Shovel STANDARD

1. Modification Rate Record (EMR) - GeoStabilization's interstate worker's compensation experience modification rate. [Industry Average: 1.00 (source EMR)]
2. OSHA Inspection Record metrics include the number of recordable incidents and the lost time rate.
3. The number of fatalities GeoStabilization has experienced during the past five years – including the current year.
4. The actual number of man-hours worked for each of the past five years.

Safety Metric	2019	2020	2021	2022	2023
Modification Rate Record (EMR):	0.81	0.76	0.69	0.62	0.62
Incident Rate and Lost Time Rate:					
Recordable Rate	1.24	0.71	1.51	0.71	0.69
Lost Time Rate	0.17	0	0.43	0	0
Number of Fatalities:	0	0	0	0	0
Number of Man-Hours Worked:	999,995	1,410,809	922,199	1,114,241	1,444,948

GSI GeoStabilization International®

REPRESENTATIVE PROJECT EXPERIENCE

GeoStabilization has completed over 6,000 successful projects to-date, involving a variety of innovative design and construction elements. Slope stabilization measures most often utilize passive reinforcement elements such as soil nails and/or micropiles, but sometimes include other components such as toe berms/shear keys, sheet pile/soldier pile walls, GRS walls, compaction grouting, and high-tensile wire mesh.

A key point of emphasis for our engineers/geologists is to evaluate a site objectively and recommend the best approaches for remediation, even if they don't include our proprietary equipment and/or technologies. This philosophy assures our clients that every geohazard emergency will be addressed with a context-sensitive solution designed specifically for a given site. Often, the most efficient solution involves a combination of remediation measures, such as launched soil nails for rapid temporary stabilization (to halt movement) combined with engineered fill buttress for long-term permanent stabilization (at a higher factor of safety).

We have included projects that illustrate the breadth of GeoStabilization's capabilities in the following sections.

GSI

GeoStabilization International®

Geohazard Mitigation Experts

CASE STUDY

PEARL STREET SLOPE REPAIR PROJECT

The City of Laguna Beach, located in Southern California, had a critical need for coastal bluff stabilization for a failing slope section adjacent to a public access stairwell. GeoStabilization overcame limited access conditions, coastal tides, and environmental challenges to provide a turn-key solution.

PROJECT OVERVIEW

In June 2021, the city of Laguna Beach in Southern California desired to fix the crumbling rock slope behind a public beach entrance. GeoStabilization overcame environmental and access challenges to provide the solution.

The entire California coastline, now part of the California Coastal National Monument, is geologically complex and subject to numerous local, state, and federal protection measures to ensure its long-term health.

The city's Pearl Street beach entrance point has a minimally invasive public access stairwell installed to allow pedestrians from the street above to traverse to the beach below. The rock slope behind the beach area was weathering, which threatened the bluff's long-term stability as well as the residences perched above the beach.

The City of Laguna Beach was uncertain how to install the solution's rock anchors and initially thought they would access the slope from the crest with drilling equipment. GeoStabilization's provided an ecologically-friendly solution that included switching anchor size, changing the grouting plan, and utilizing all-terrain access drill equipment that used biodegradable fluids to access the slope from the beach and elevate the drill mast to all rock bolt locations.

LEVERAGING INNOVATIVE SOLUTIONS

This project's biggest challenge was accessing the marine bluff to install the needed rock bolts. Our engineers developed a repair plan utilizing a specialty drill mounted on a walking excavator (commonly known as a SPIDER) to install the rock anchors. This innovative equipment uses biodegradable engine oil and hydraulic fluid, providing an green alternative to a conventional excavator or drilling equipment. The SPIDER reached the topmost anchor locations by "walking" up the slope immediately adjacent to the stairs.

Our engineers designed the solution with #11, GR 75 epoxy-coated rock bolts placed in 4-inch diameter holes. Additionally, the beach access permitted faster drilling that reduced the project's duration from six to five days and reduced the project's cost to the taxpayers.

While drilling and installing the rock bolts, the crew had to navigate challenging coastal working conditions with work hours dictated by the rise and fall of the daily tides. Additionally, an unexpected environmental challenge developed as some of the drilled holes began filling with petroleum-contaminated groundwater and began weeping onto the beach. Our team successfully overcame all challenges and completed the project to the customer's and neighbor's satisfaction.

DRILLING FROM BEACH LEVEL

- *Project completed 20% quicker while adhering to strict environmental regulations*
- *GeoStabilization's design-build capabilities provided an 18% reduction in project cost*
- *GeoStabilization's tools and technologies were ideally suited for coastal bluff application*

DRILL'S EXTENDED REACH FOR UPPER BOLTS

ABOUT GEOSTABILIZATION

Our passion is to innovate and implement optimized solutions that protect people and improve infrastructure.

Assessment - No-obligation site visit by a qualified geotechnical engineer.

Analysis - Fixed-cost proposal developed from available geotechnical and site visit information.

Design - In-house engineering team assists or completes the design.

Installation - Crews can mobilize within 24 hours; designs optimized in real time.

Warranty - Multi-year performance warranty covers materials, installation, and overall system performance.

GEOHAZARD EXPERTS

- ✓ Over 5,000 projects completed since 2002
- ✓ Industry Leader for Safety; annually recognized by the ADSC and the AGHP
- ✓ Leading geohazard solutions provider for federal, state, local governments, major O&G, rail, mining, and utility operators, and general contractors

AWARD-WINNING SERVICES

GeoStabilization International® credits its success to the founding philosophy of serving the client with the newest technologies, the best solutions, and the fastest possible service. Through many years of training, experience, and this founding philosophy, our engineers and constructors now stand as the most qualified and experienced in the industry.

ENR 2018-2021 REGIONAL BEST PROJECTS

ETHISPHERE® WORLD'S MOST ETHICAL COMPANIES® 2019-2020

ADSC 2014-2021 SAFETY AWARD WINNER

Dave Brogan, MS, PE
Cell: 207-801-0469 // dave.brogan@gsi.us

CalTrans District 4, Hwy 1, SON, Mile 15.5

Sonoma County, CA

REPRESENTATIVE PROJECT

PROJECT HIGHLIGHTS

Owner:
CalTrans District 4

Prime Contractor:
Said Najafi
Ghilotti Construction
246 Ghillotti Ave, Santa Rosa, CA 95407

Project Start/Completion Date:
3/7/2017 - 3/30/2017

Key Personnel:
Project Development Engineer: Andy Bowman, PE
Project Manager: Brett Gustafson
Operations Manager: Ronald Priestly
Superintendent: Kenyon Acosta
Engineer: Cameron Lobato

Project Description:
GeoStabilization International provided a permanent stabilization system below the roadway platform that was experiencing severe erosion adjacent to Hwy 1 in Sonoma County. The installation's design utilized 150 linear feet of Self-Drilling SuperNails® and 2,179 square feet steel-reinforced shotcrete (6-in thick), 50 square feet steel-reinforced shotcrete (12-in thick) facing, 1,462 SF non-reinforced shotcrete (Area 3 only), and 1,669 SF stained shotcrete.

GeoStabilization International®

Coastal Bluff Stabilization Using Launched Soil Nails

Crescent City, CA

REPRESENTATIVE PROJECT

PROJECT HIGHLIGHTS

Client:
Del Norte County
Steve Devlin, PE

Project Start/Completion Date:
12/08-01/09

Key Personnel:
Project Development Specialist: James Chinchiolo

Project Summary:

Sections of coastal bluff along Pebble Beach Drive in Cresent City, Del Norte County, were experiencing erosion. In the past, rip rap had been installed to help mitigate the receding slopes. Because of permitting and environmental issues, the project engineer decided launched soil nails were the best option for a permanent stabilization.

Results from six geotechnical borings and CPT testing indicated soil types ranging from fat clay to unconsolidated sand - conditions favorable to launched nails.

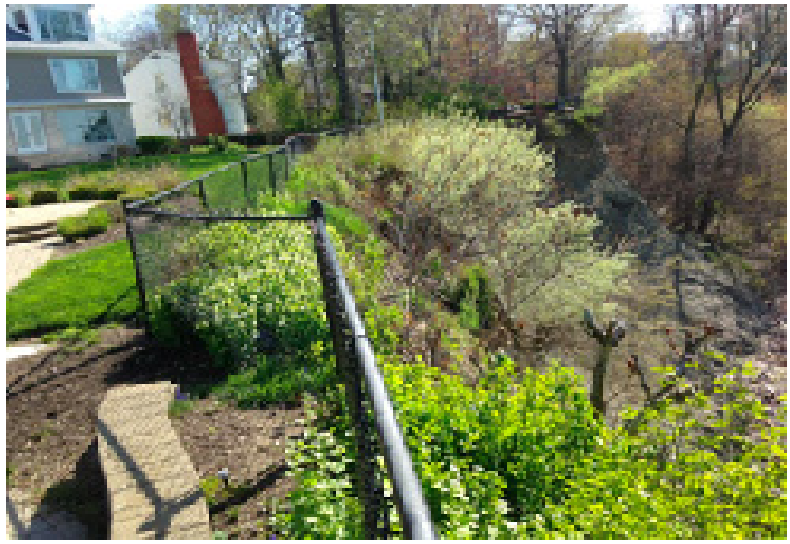
The nails were installed in a triangular pattern. The type of nail used was a fiberglass nail with a steel tip. Over 350 launched soil nails were installed. The Soil Nail Launcher was attached to the boom of an excavator. It was able to sit on the ledge of the bluff and extend down and over the slope to install the nails. Drainage galleries were installed in areas of the wall where excess pore pressure behind the shotcrete wall was anticipated.

There were issues with launched nails going too deep into the soil. To rectify this air pressure in the launcher was lowered. Another issue involved the steel tips getting shot off of the nails. An aesthetical issue was also presented with protruding drainage pipes. This was solved by cutting the pipes flush with the slope and painting them a color that matched the surrounding wall.

Ricker Residence

Lake Erie, Ohio

REPRESENTATIVE PROJECT



PROJECT HIGHLIGHTS

Client:
Mr. Scott Ricker

Project Start/Completion Date:
9/19/2017-9/29/2017

Key Personnel:
Project Engineer: Greg Bachman, PE
Project Manager: Jared McDowell

Project Description:
Perched high atop a soil cliff on Lake Erie, the backyard of the Ricker residence was sloughing off into the lakeshore below. Caused by the ground's differential weathering, this regular geologic occurrence was reducing the backyard's footprint and could eventually imperil the livability of the entire property. An engineered solution was required that would arrest the slope's deterioration and "harden" the crest to prevent further erosion.

At the project's commencement, our crew cleared, excavated, and reshaped the bluff to achieve a slope of approximately 0.5H:1V. Then multiple rows of up to 20-foot long self-drilling SuperNails™ were installed into the slope face to 12-feet below the slope's crest; with their ends protruding through the soil approximately 5-inches. The nails were then grouted for their full length of the embedment to ensure a satisfactory pull-out resistance was achieved. Finally, a 12-foot tall band of steel mesh reinforced shotcrete, nominal 6" thickness, was installed to strengthen the crest.

GSI GeoStabilization International®

Spanish Fort Bluff - Emergency Bluff Repair

Spanish Fort, Alabama

REPRESENTATIVE PROJECT

PROJECT HIGHLIGHTS

Client:
City of Spanish Fort
John Amburgey

Project Start/Completion Date:
7/9/2014-10/27/2014

Key Personnel:
Project Engineer: Reid Bailey, PE
Project Manager: Matt Swoboda

Project Description:
When poor drainage and a damaged culvert couldn't handle a localized heavy rain, a bluff providing the backyard to many residential properties failed. The subsequent erosion caused a large canyon to form, the wall of which stopped within 6-feet from the back of the houses. To arrest the erosion and prevent its further spread, a slope stabilization system was needed.

Given the size of the resulting unstable earth mass below and near the house platforms and the depth of soft subsurface conditions, our solution addressed both the slope and stormwater drainage. The area below the repair area was unloaded with the construction of a shotcreted soil nail retaining system at the top of the bluff where erosion. As part of this retaining system, up to 15 feet of soil down from the current grade was removed. This soil removal reduced the slope angle below the soil nail wall system and improved the lower slope's factor of safety. Then riprap and a designed drainage system were installed to move future stormwater off the slope in a controlled manner.

LIMITED ACCESS DRILLING

GeoStabilization International®

LIMITED ACCESS DRILLING

Just as conventional reinforcement technologies are rarely suited for the challenges associated with geohazard mitigation, conventional drill rigs are no match for the difficult conditions encountered on emergency geohazard sites. That's why GeoStabilization International® engineers design each of their purpose-built rigs to deal specifically with geohazard repair conditions. From helicopter-mobile Wagon Drills, to excavator-mounted Soil Nail Launchers™ and SuperNailers, to crane-basket mounted drills and Spider-Excavator Mounted "Walking" drill rigs, no company in North America has a more extensive, diverse, or capable fleet. All of GeoStabilization's purpose-built equipment provides the following benefits:

Fast Installation

In geohazard mitigation, speed of installation is critical. Active landslides don't slow down or stop when you start working on them. Unstable rock masses have a very short time frame when they can stabilized in place. Using the best tool for the job results in fast installation, which correlates directly with site safety. Using the wrong tool for the job, or only having a few tools to choose from can directly lead to total roadway collapse. For example, the speed of the Soil Nail Launcher™ and GeoStabilization's purpose-built SuperNailers™ prevented total collapse of Interstate 75 in Campbell County, Tennessee, both in 2005 and again in 2012. Conventional drill rigs could not match the installation rates of either tool, and would not likely have succeeded in preventing failure at either site.

Mobility

Active landslides and unstable rock masses, by definition, exist near a factor of safety of 1.0. This means that the driving forces causing failure and the resisting forces preventing movement are nearly equal. Any disturbance to this delicate balance can trigger total failure and for this reason, GeoStabilization specifically tailors its purpose-built fleet to work without major access road and significant site disturbance and excavation. This "light" site footprint can also yield environmental benefits and a reduction in permitting requirements.

Access

Many geohazard sites are relatively inaccessible with conventional drill rigs. Unstable rock masses can be hundreds of feet above the roadway elevation. Landslides can occur on remote sections of gas pipelines. Transmission tower sites can be miles from the nearest road. For this reason nearly all of GeoStabilization's fleet can travel the most questionable of roads, and much can be mobilized using helicopters, cranes, or high-reach lift equipment.

DRILL RIGS

DRILL RIGS

MICROPILE APPLICATIONS

In-Situ Reinforcement

Slope stabilization & earth retention	Ground strengthening	Settlement reduction	Structural stability

Structural Support

Earth retaining structure foundations	Foundations for new structures	Underpinning existing foundations	Seismic retrofitting
Scour protection	Repair/replace existing foundations	Stop/prevent movement	Upgrade foundation capacity

GSI Management Team

DOMINIC IVANKOVICH, M.B.A., CEO, Dominic joins GeoStabilization after more than twenty-eight years of engineering, marketing, and business development experience, including more than ten years leading large global businesses across essential industries. With a degree in Chemical Engineering from Montana State University and an MBA from the Kellogg School of Management at Northwestern University, Dominic has a rich track record of driving growth acceleration and market share gains via improved client satisfaction, innovation, commercial execution, and strategic acquisitions in core and new markets. Additionally, Dominic's significant depth in acquisition integration and construction management is critical to GeoStabilization's future as it continues to grow in current and new mission-critical applications consistent with the company's mission.

NATE BEARD, PE, VP CLIENT MANAGEMENT, Mr. Beard directly oversees the implementation of design, management, and construction of all of GSI's specialty earth retention systems in the United States utilizing the sciences of Soil Nailing, Tiebacks, Micropiles, as well as Geosynthetically Confined Soils. As an engineer with more than 16 years of experience, including 12 years working with GSI, Nathan brings a wealth of relevant past experience. A former Captain in the U.S. Air Force and Systems Engineering Chief at Patrick Air Force Base, he is adept at facilitating the delivery of high quality plans – reviewed with constructability in mind – to the field with exceptional precision. Nathan is a registered professional engineer in over a dozen states. Nathan has authored multiple papers on stabilization technology and regularly presents workshops across the United States and Canada.

ANDREW SWEATMAN, SR. DIRECTOR OF PMO, As the newest addition to the Service Delivery Leadership Team, Andrew Sweatman brings a wealth of experience and expertise to his role as Senior Director of the Project Management Office (PMO) at GSI. With over 15 years of proven leadership in diverse operational environments, Andrew joins us from Amazon.com, where he most recently served as Regional Director overseeing operations across 14 fulfillment centers. Andrew's track record of success extends beyond operational excellence, encompassing a keen understanding of project management principles and methodologies. He brings a robust skill set in leadership development, project execution, and stakeholder engagement. Andrew oversees the US Soils Project Management team, as well as spearheading crew and equipment scheduling initiatives.

ATTACHMENT B

SOIL NAIL AND SLOPE STABILITY DESIGN NARRATIVE

1.0 SUMMARY AND PURPOSE

Civil & Environmental Consultants, Inc. (CEC) prepared this Design Narrative for stabilizing the slope at the rear of the 51 Harborview Road property in Hull, Massachusetts. The slope is located at approximate coordinates 42.307492°, -70.906191° (latitude, longitude). Based on the existing slope conditions and subsurface conditions at the site, CEC is recommending stabilizing the slope with a combination of soil nails, slope re-grading with rip rap armoring, and the installation of gabion baskets near the east property line.

The purpose of this design narrative is to provide a detailed description of the analyses performed to support our conceptual remediation design (Concept Design), included as Attachment 1, for stabilization of the slope. This narrative includes the results of analyses performed to develop the remedial design, site observations, laboratory testing, soil and imported stone material properties, structural reinforcement elements associated with the soil nail and shotcrete facing, and our design assumptions.

2.0 REFERENCES AND ATTACHMENTS

The following attachments are included with this design narrative:

1. Conceptual Remediation Design Drawings
2. Stability Cross Section Location Plan
3. Back Analysis and Outputs
4. Existing Conditions Calculations and Outputs
5. SNAIL Analysis Outputs
6. Global Stability Analysis Outputs – Proposed Soil Nails
7. Gabion Basket Stability Analyses
8. Rip Rap Transition Slope Stability Analysis
9. Test Boring Log
10. Laboratory Test Results
11. FHWA Soil Nail Reference Manual, Publication No. FHWA-NHI-14-007, February 2015 (Partial Version)
12. Williams Form Engineering – Grade 75 All-Thread Rebar Properties

<u>References (not included as attachments):</u>

1. FHWA Soil Nail Reference Manual, Publication No. FHWA-NHI-14-007, February 2015 (Complete Version)
2. California Department of Transportation (Caltrans) SNAIL User Guide – March 2020
3. Caltrans Trenching and Shoring Manual – August 2011
4. “Standard Handbook for Civil Engineers” McGraw -Hill, 1983

3.0 BACKGROUND INFORMATION

3.1 Site Conditions: CEC performed an initial reconnaissance at the site on July 18, 2022, to document the site and slope conditions and again on June 7, 2023, to note any visual changes to site conditions. The existing Retaining Wall was constructed at the top of the coastal bank and parallels the existing patio and deck structure constructed behind the wall. The patio is supported on wood columns and concrete piers. A copy of the Retaining Wall drawings was provided to CEC by the Client.

Since the initial slope failure, the area below and immediately behind the existing Retaining Wall has been subjected to significant erosion which has resulted in rills, outwash of portions of the retained soils, and partial undermining of the solider piles for approximately two-thirds of the wall length. CEC understands the Client placed quick setting foam behind the wall as a measure to backfill outwashed portions of the retained material and reduce the potential for additional erosion from surface water.

In November of 2023, a contractor was hired to construct drainage improvements at the top of the bank. The As-Built drainage improvement are shown on the attached Conceptual Drawings.

3.2 Proposed Slope Remediation: The proposed remediation will use a combination of site grading, construction of soil nails with shotcrete facing, a rip rap transition slope, and placement of gabion baskets to stabilize the slope near the east property line. As part of stabilizing the slope, most of the existing Retaining Wall will be removed and the slope flattened. This will require a staged deconstruction of the existing Retaining Wall to maintain stability of the slope during the staged excavation and installation of the proposed soil nails.

CEC originally provided two design alternatives for stabilizing the slope between the end of the existing Retaining Wall and the east property line. Alternative 1 included a concept to end the soil nail facing about 6.5- feet from the property line and install gabion baskets adjacent to the east property line to improve the stability of the slope in this area and keep all improvements within the 51 Harborview Road property. Alternative 2 includes a concept to extend the soil nails into the east abutter's property with the abutter's permission. Based on discussions with the Conservation Commission and interested parties, we understand Alternative 2 is not an option at this time and has been removed from this revised calculation package.

3.3 Subsurface Investigation and Laboratory Testing: CEC subcontracted Soil X Corporation (Soil X) to drill one test boring at the site on June 28, 2023, to evaluate the subsurface conditions. Due to access restrictions, the test boring (Test Boring B-1) was performed at the front of the property. The ground surface elevation at the test boring location was approximate Elevation 58', or approximately 5-feet above the top of the Retaining Wall elevation.

Soil X sampled the soil zone using hollow-stem auger and split-spoon sampling methods in general accordance with the standard penetration test (SPT) outlined in ASTM D1586. Soil samples were collected at approximate 3-foot center-to-center intervals from at each test boring. A split-spoon sampler is a 2-inch outside-diameter (OD) tube that captures soil material inside when driven below ground. Upon retrieval, the sampler is split open for removal and identification of the captured soils. The SPT consists of driving a split-spoon sampler through soil using a 140-pound hammer freely falling a distance of 30 inches and recording the number of blows required to drive the split-spoon sampler through three successive 6-inch increments. The sum of the number of blows required to drive the split-spoon sampler through the second and third increments is the N-value of the soil. Soil N-values are commonly used to estimate soil density, compressibility, and shear strength. The test boring extended to approximately 30.1-feet below the existing ground surface (bgs).

CEC encountered existing fill to a depth of approximately 3-feet bgs, then glacial till to the bottom of the test boring. CEC defines existing fill as soil derived from natural soil, bedrock, or processed materials that were placed by artificial methods, such as construction, waste disposal, or dumping. The existing fill sampled consisted of loose silty sand. Glacial till is soil deposited by and underneath a glacier, generally consisting of a heterogeneous, unstratified mixture of clay, sand, gravel, and boulders. The glacial till encountered primarily consisted of very dense silty sand with gravel.

CEC subcontracted Thielsch Engineering, Inc. to perform laboratory material properties testing on select samples of glacial till from Test Boring B-1 (Sample S-4, 15-16.4' bgs) and surficial bulk samples (Samples GS-1, 0-1' bgs and AS-1, 2-3' bgs) obtained in the vicinity of the Retaining Wall. Laboratory testing included ASTM D2216 (moisture content), ASTM D6913 (particle size distribution), ASTM G57 (soil resistivity), ASTM G51 (pH), and ASTM D4327 (sulfate and chloride) tests. The tested samples classified as SP (poorly graded sand with gravel) and SM (silty sand with gravel; clayey silty sand with gravel) according to the Unified Soil Classification System (USCS). Moisture contents of the samples ranged from 1.9% to 10.1%. Samples GS-1 and AS-1 classified as non-plastic. The sample obtained from Test Boring B-1 visually classified as plastic, however, Atterberg limits testing was not performed. Fines contents of the samples ranged from 1.9% to 41.0%. Chloride, sulfate, and pH contents measured in Sample GS-1 were 16 mg/L, 13 mg/L, and 9.1, respectively. Electrical resistivity testing performed on this sample yielded resistivities of 33,600 Ohm-cm and 17,400 Ohm-cm for the as-received and saturated conditions, respectively. Attachment 10 includes the complete laboratory testing results.

4.0 MATERIAL PARAMETERS

The parameters used for the various materials modeled in the analyses represent the effective strength parameters and are based on the data obtained from the test boring information, laboratory testing, visual observation of the landslide geometry and conditions, and our slope stability back analysis.

Existing Fill behind the Retaining Wall (Existing Fill): CEC defined the material properties for the existing fill placed behind the Retaining Wall based on assumed values typical of the material gradation reflected in the laboratory testing. CEC also assumed the material is relatively use. Therefore, values typical of loose to medium dense silty sand were utilized (reference 3).

Colluvium: CEC defined the existing colluvium soils towards the bottom of the slope based on the gradation laboratory test results from the Glacial Till soils, with the assumption that the material is relatively loose from previous disturbance.

Glacial Till: CEC defined the material properties for the glacial till based on the back-analysis, laboratory testing, and empirical correlations from SPT N-values.

Riprap: CEC defined the proposed riprap material based on the gradation/weight requirements of Dumped Riprap in accordance with Table M2.02.2-1 of the Massachusetts DOT Standard Specifications for Highways and Bridges, 2023 Edition.

Gabion Stone: CEC defined the proposed gabion stone based on typical values for loose gravel (reference 3).

Leveling Pad: CEC defined the proposed leveling pad which would be placed below the gabion baskets based on the gradation of MassDOT Processed Gravel.

The material strength properties utilized in our slope stability and soil nail analyses are shown in Table 1 below. Tables 2 – 5 include the material properties of the components of the soil nails and facing.

Table 1 – Material Strength Parameters

Material	Unit Weight (pcf)	Angle of Friction (degrees)	Cohesion (psf)	Nominal Bond Strength (psi)
Existing Fill	130	32	0	1.5
Colluvium	120	32	0	--
Glacial Till	135	36	50	20
Riprap	120	45	0	--
Gabion Stone	120	35	0	--
Leveling Pad	140	38	0	--

Table 2 – Soil Nail Parameters

Soil Nail	Nail Diameter (inch)	Borehole Diameter (inch)	Grout Compressive Strength (psi)	Steel Yield Strength (ksi)
#8 Grade 75 All-Thread Rebar	1.00	4	4,000	75

Table 3 – Bearing Plate Parameters

Item	Dimensions (inch)			Hole Diameter (inch)	Steel Yield Strength (ksi)
	Width	Height	Thickness		
Bearing Plate	8	8	0.750	1.5	50

Table 4 – Bearing Plate Stud Parameters

Item	No. per bearing plate	Head Diameter (inch)	Head Thickness (inch)	Length (inch)	Shaft Diameter (inch)	Spacing (inch)	Tensile Strength (ksi)
Bearing Plate Studs	4	1.25	0.380	5	0.750	5	65

Table 5 – Shotcrete Facing Parameters

Shotcrete Facing	Thickness (inch)	Shotcrete Compressive Strength (psi)	Welded Wire Mesh Reinforcement	Vertical Waler Reinforcement[1]	Horizontal Waler Reinforcement[1]	Reinforcement Yield Strength (ksi)
				Type – Length (L) – Spacing (S)		
Initial	4	4,000	4x4 W4.0x4.0	#4 Rebar L = 3 feet S = 12-inch c.c.	#4 Rebar L = 3 feet S = 12-inch c.c.	60
Final	8	4,000	--	#4 Rebar L = continuous S = 12-inch c.c.	#4 Rebar L = continuous S = 12-inch c.c.	60

1. Initial shotcrete facing layer includes two vertical and two horizontal waler bars at each soil nail.

5.0 DESIGN CONSIDERATIONS AND ASSUMPTIONS

CEC utilized the following design considerations and assumptions to perform the analysis.

5.1 Back-Analysis on Existing Conditions:

- To develop strength parameters for the Glacial Till soils, CEC performed a back analysis based on the slope to the west of the site.
- The existing slope to the west of the site is vegetated and appears stable. It slopes from approximate El. 63 feet to about El. 20 feet on an approximate 1.6H:1V grade. An approximately 5-foot-high stone revetment is present at the toe of the slope with a beach beyond.
- CEC considered that the slope to the west of the site is stable and selected this slope due to its consistent geometry in the out of plane (i.e. east to west) direction.

5.2 Soil Nail Construction:

Installation of the proposed soil nails will involve staged deconstruction/excavations. Up to three rows of soil nails will be installed in the area shown on Attachment 1. Towards the west end of the soil nail alignment, the height will decrease to match the re-graded slope. The proposed soil nail elevations, inclinations, and embedment depths are described herein and listed on Table 6 in Section 6.2. The soil nails will be spaced 4-feet center-to-center (c.c.) vertically and 4-feet c.c. horizontally, with the upper soil nail row positioned at approximate Elevation 51. The staging will consist of performing excavations to approximately 1 to 2-feet below the proposed soil nail elevations; drilling, installing soil nail tendons, and backfilling the boreholes with grout; and applying an initial shotcrete facing within a 2-day period following the excavations for each stage. After completing the final excavation stage, a final shotcrete facing layer will be applied over the entire soil nail array. The proposed soil nails will have a 15-degree batter with the top of the facing positioned approximately 3-feet behind the existing Retaining Wall. The proposed soil nails will also incorporate drainage strips at the rear of the shotcrete facing and weep holes to reduce the potential for hydrostatic pressures behind the facing. The drainage strips will tie into a riprap buttress that will be installed at the bottom of the facing, with erosion control plantings installed beyond the buttress.

<u>Slope Preparation:</u> During soil nail construction, the temporary excavations will not exceed 4-feet before application of the temporary shotcrete facing. Excavation for the next row of soil nails will not proceed until the shotcrete facing has adequately cured. The contractor performing the work will stage the deconstruction of the existing Retaining Wall to maintain temporary stability during installation of the soil nails.

<u>5.3 Gabion Basket Construction:</u>

The gabion baskets construction will utilize 4mm galvanized steel wire mesh baskets with dimensions of 3’x3’x3’ (length x width x height). Each row of baskets will be stepped back 6-inches from the front face of the lower row. The gabion baskets will be installed perpendicular to and in front of the slope stabilized using soil nails to reduce the height of the temporary excavation (see Attachment 1 for additional details). The conceptual design shows the soil nail installation being constructed from approximate Elevation 53’ to Elevation 41’ followed by installing gabions between approximate Elevation 41’ to Elevation 50’. The alignment of the gabion baskets will be perpendicular to the soil-nail-stabilized slope for a distance of about 12 feet, then returning the gabion baskets parallel to the soil nails. This will allow regrading of the top of the slope where existing grades near the property line are steeper than 1:1. Construction of the gabion baskets will generally result in temporary excavation heights of around 3 to 8 feet at the rear of the baskets. The total height of the gabion baskets will range from 3 to 9 feet with the embedment of the lowest row ranging from 1 to 3 feet. Refer to Attachment 1 for additional details.

6.0 ANALYSIS METHODOLOGY

CEC performed the analyses for this project in six general stages. For each phase, the material properties in Section 4.0 were utilized. The location of each stability cross section is shown in Attachment 2.

The first stage involved performing a back analysis to estimate the material properties of the glacial till based on the adjacent stable slope geometry, subsurface conditions, and groundwater conditions for the existing slope (see Attachment 3). The back-analysis consisted of iteratively modifying the material strength properties for the glacial till to obtain a conservative factor of safety (FS) of the stable slope to the west of the site. Slope stability software Slide Version 9.0 was used to calculate the minimum global FS for this analysis and subsequent global stability analyses listed below. The program uses 2D limit equilibrium methods to determine the minimum FS. CEC utilized a conservative non-circular failure analysis to determine the minimum FS within the observed failure limits. Our analyses used the Spencer's Method considering a Cuckoo, non-circular search with optimization to calculate all FS.

The second stage involved performing an analysis along the middle of the existing Retaining Wall. CEC determined the tensile resistance of the existing tiebacks and performed a global stability analysis to calculate the minimum FS (see Attachment 4). CEC performed this analysis in an effort to quantify the proposed improvements relative to the existing conditions. The analysis was performed along Cross Sections A-A as shown on the Conceptual Design

The third stage involved the dimensioning of the proposed soil nail layout utilizing SNAIL geotechnical engineering software developed by the California Department of Transportation (Caltrans). The SNAIL software evaluates the suitability of the reinforcing elements for the soil nail and shotcrete facing components of the system. CEC selected input parameters regarding soil properties, slope geometry, and the system components based on the information provided herein.

The fourth stage of the analysis was a global stability analysis of the soil nails. The objective of this analysis was to evaluate the length of the soil nails regarding the overall stability of the slope. CEC calculated the minimum FS for our anticipated critical scenario represented by Cross Sections A-A as shown on the Conceptual Design. During the analysis process, the soil nail geometry was iteratively modeled to determine the combination of features that resulted in a FS of 1.5 or greater for permanent conditions.

The fifth stage of the analysis included performing stability analyses near the east property line where it is necessary to transition into the existing property and therefore, soil nails cannot be utilized. In this area, CEC is proposing the upper portion of the slope be supported with stacked gabion baskets.

The sixth and final stage of the analysis included performing stability analyses on the proposed rip rap transition slope near the west extent of the proposed soil nails.

6.1 Back Analysis of Existing Conditions Slope West of 51 Harborview Road

CEC performed a global stability analysis on the existing stable slope to the west of the site as a basis to develop strength parameters for the glacial till present at the site. A back analysis was also considered along the steep slope at the east property line, but due to inconsistent geometry in the out of plane dimension, the west slope was considered more reliable. It is CEC's opinion that the existing conditions factor of safety for the existing slope to the west of the site should be above 1.0. CEC evaluated the slope based on strength parameters equal to a cohesion of 50 psf and a friction angle of 36 degrees, which resulted in a factor of safety of 1.17.

While groundwater was not encountered in the boring performed on the property, for conservatism CEC calculated a groundwater level for the back analysis by utilizing the Frimpter method. CEC selected two index wells located in Hanson and Dover, MA that were installed in a similar geological setting to the site to estimate the probable high groundwater level at the site. The probable high groundwater level at the site was determined to be at El. 29.3 based on this analysis. In the stability model, CEC assumed the phreatic surface was a straight line between El. 29.3 and the FEMA flood zone El. 20 at the bottom of the slope. A worksheet utilized to calculate the probable high groundwater level is provided in Attachment 3 along with the results of CEC's back analysis. The location of the stability cross sections are shown in Attachment 2.

6.2 Evaluation of Existing Retaining Wall

CEC performed a global stability analysis along the center of the existing Retaining Wall to quantify the effect of the proposed improvements and also to approximate the tensile resistance provided by the existing tiebacks when the soil nails are installed and before they are cut. CEC calculated the tensile strength provided by the existing tiebacks using guidance provided in Reference 3. Because the bottom level of tiebacks is generally at the centroid of the triangular pressure distribution behind the Retaining Wall, CEC assumed the bottom row of tiebacks were resisting the full earth pressures behind the Retaining Wall. Calculations are provided in Attachment 4 and a summary of the results is shown in Table 8below.

6.3 SNAIL Evaluation

CEC utilized the geometry depicted on the cross sections and material parameters discussed in Section 4.0 as the basis for the site-specific subsurface conditions considered in the SNAIL evaluation. With this combination of inputs, the software checks a variety of potential failure modes. Snail analyzes soil nail system stability based on force limit equilibrium. The software generates bi-linear surfaces through the toe of the facing, or tri-linear surfaces which pass below the toe and daylight in front of the facing, to calculate and search for the minimum factor of safety of the selected analysis scenario. The software evaluates pullout along the length of the soil nail; yield of the soil nail; punching shear, flexure, and stud tensile of the facing; flexure and bearing stress of the bearing plate; and bearing stress of the facing shotcrete with regard to the bearing plate. CEC's analysis also evaluated the slope FS for the various construction stages correlating to temporary excavations.

CEC also input approximate loads from the columns supporting the deck above the top of the slope. Three rows of columns exist at the site; the first row is about 5 feet back from the existing Retaining Wall. The second row of columns is about 20 feet back from the first row of columns. The third row of columns is beyond the limits of the soil nails and was not included. CEC assumed a deck load of 55 psf between dead and live loads, and calculated the approximate tributary area based on the column spacing. Using a 10-inch diameter concrete pier, CEC estimated 12,664 psf for the deck load per interior column and 6,332 psf for the deck load for the perimeter columns closest to the existing wall. For temporary conditions during the soil nail installation, these loads were reduced based on a dead load of 15 psf, as the deck is not currently in use by the owner and will not be fully utilized until the slope repair is completed. These reduced loads correspond to an interior column load of 3,419 psf and a perimeter column load of 1,710 psf.

The proposed slope considers the soil nail facing with a 15-degree batter, which involves an excavation extending approximately 3-feet behind the top of the existing Retaining Wall. The soil nail system utilizes a nail layout spaced at 4-feet c.c. vertically by 4-feet c.c. horizontally, with the upper row of soil nails starting at El. 51. The shotcrete facing for the soil nails extends from the ground surface, at approximate Elevation 53, to approximate Elevation 41 at its lowest point. The soil nail installation considers a 4-inch borehole and the grout-to-ground bond strengths listed in Table 1 in Section 4.0. The proposed soil nail lengths based on the critical section are shown in Table 6 below.

Table 6 – Soil Nail Elevations, Embedment, and Inclination

Soil Nail Row	Approximate Soil Nail Elevation	Embedment Length (ft)	Soil Nail Inclination (degrees)
1	51	35	15
2	47	25	10
3	43	20	10

The FHWA minimum recommended factor of safety values are provided in in Table 7 below, and each criteria was met in the SNAIL analysis (see Table 9 below). Full SNAIL outputs are included in Attachment 5.

Table 7 – FHWA Minimum Recommended Safety Factors

Failure Mode	Factor of Safety
Overall	1.5
Short Term Conditions	1.3
Pullout Resistance	2.0
Tendon Tensile Strength (Grades 60 and 75 Steel)	1.8
Facing Flexure	1.5
Facing Punching Shear	1.5
Headed Stud Tensile Strength	2.0

6.4 Global Stability Analysis – Soil Nails

CEC utilized the design consideration discussed in Section 5.0 as the basis for the site-specific subsurface conditions considered in global stability analysis. In addition, CEC utilized two groundwater levels; the “normal” groundwater level based on the Frimpter analysis described in Section 6.1, and a groundwater level at the ground surface to represent fully saturated slope conditions. The groundwater level at the ground surface was only utilized for the proposed conditions case once the soil nails are completed and not during construction of the soil nails.

In addition to the geometry components of the proposed soil nail arrangement, which are shown on the geotechnical cross section, soil nail inputs required in the global stability analysis include the force application method, tensile force orientation, bond strength per linear foot of soil nail, the tensile strength of the soil nail, the shear strength of the soil nail, and the plate capacity at the soil nail head. CEC selected the active force application method for use in the analysis. In accordance with generally accepted practice when modeling supports in slope stability programs using the active force application method, the input value for the tensile strength of the soil nail is its ultimate strength divided by a factor of safety of 1.8 per FHWA Soil Nail

Reference manual (Attachment 11). The tensile force orientation was selected as parallel to the reinforcement as significant deformation would have to take place for the force to be oriented in another manner. The plate capacity utilized in the stability model was selected based on capacity of the shotcrete facing determined in the SNAIL analysis. CEC conservatively used the facing capacity of the temporary condition for all modeled scenarios.

CEC also input approximate loads from the columns supporting the deck above the top of the slope similar to Section 6.2 above. CEC estimated 12,664 psf for the deck load per interior column and 6,332 psf for the deck load for the perimeter columns closest to the existing wall. For temporary conditions during the soil nail installation, these loads were reduced based on a dead load of 15 psf, as the deck is not currently in use by the owner and will not be fully utilized until the slope repair is completed. These reduced loads correspond to an interior column load of 3,419 psf and a perimeter column load of 1,710 psf.

The results of CEC's global stability analysis for the soil nails are provided as Attachment 6 and a summary of the results is provided in Table 9 below.

6.5 Stability Analyses for Gabion Baskets

CEC performed a calculation to determine the frictional resistance of the gabion basket steel mesh and the underlying subsoil. CEC's calculations conservatively assumed the gabion stone would not extrude through the steel mesh, and therefore, only considered the steel mesh-to-subsoil frictional resistance as part of our calculation. CEC determined a steel mesh-to-subsoil friction coefficient of 0.90 based on the steel mesh diameter, opening size, subsoil friction angle, and steel to soil friction angle between dissimilar materials. For the purposes of our calculation, CEC conservatively reduced the frictional coefficient to 0.70 to better represent typical values for sliding at the base of retaining structures.

CEC utilized SRwall to evaluate sliding, overturning, bearing capacity, and the internal stability for the gabions. Typically, gravity block retaining structures develop a unit-unit interface friction that depends on the applied normal force and friction angle between the blocks. However, the infilled weight of one gabion basket would achieve a normal force that would exceed the weld shear strength. Therefore, our analysis utilized a unit-unit interface shear capacity between the gabion baskets equivalent to the weld shear strength for the welded wire mesh. According to ASTM A974 "Standard Specification for Welded Wire Fabric Gabions and Gabion Mattresses", the average weld shear strength is generally around 70% of the breaking strength of the wire (i.e. ultimate tensile strength). Other manufacturer recommendations suggest the minimum shear weld shear strength should be at least 60% of the ultimate tensile strength. CEC utilized a unit-unit shear strength of 880 lb/ft for the analysis.

CEC then utilized Slide to evaluate the global stability of the gabion baskets. The global stability analysis assumed failure surfaces could only propagate from below the gabion baskets. The results of the SRwall and global stability analysis are presented in Table 9, and the analysis outputs are included in Attachment 7. The location of the stability cross section is shown in Attachment 2. CEC considers the global stability FS for the gabion baskets as conservative, as there is much published research that indicates concave-shaped slopes provide additional confinement that are not quantified in two-dimensional analyses.[1]

6.6 Stability Analyses for Rip Rap Transition Slope

Lastly, CEC performed a global stability analysis on the proposed rip rap slope near the portion of the existing bank that is still intact near the west property line to confirm the slope meets the minimum factor of safety requirements. The results of the global stability analysis are presented in Table 9 and the analysis outputs are included in Attachment 8. The location of the stability cross section is shown in Attachment 2.

7.0 CALCULATED FACTORS OF SAFETY

The following table summarizes the results of the global slope stability analyses and includes a global stability FS for both the back-analysis based on existing conditions and the soil nails.

Table 8 – Back Analysis, Existing Conditions and Calculated Factors of Safety

Cross Section	Analysis	Global Minimum Calculated FS	SNAIL Analysis Calculated FS	Minimum Target FS
West Slope	Existing Conditions – Back Analysis	1.17	--	1.0
Cross Section A-A	Existing Conditions	0.97	--	--
	Proposed Conditions – Normal GW Level	1.77	1.77	1.5
	Proposed Conditions – Fully Saturated Slope	1.05	--	1.0
	Stage 1 – Excavation	1.07	--	1.0
	Stage 1 – Excavation – Temporary Conditions	1.36	3.11	1.3
	Stage 2 – Excavation	1.28	--	1.0
	Stage 2 – Excavation – Temporary Conditions	1.79	3.01	1.3
	Stage 3 – Excavation	1.82	--	1.0
	Stage 3 – Excavation – Temporary Conditions	2.01	2.13	1.3

The following table summarizes the results of the stability analyses performed for the proposed gabion baskets near the east property line and the west rip rap transition slope.

[1] "A Comparison of Slope Stability Analyses in Two and Three Dimensions" – D. Wines, Journal of South African Institute of Mining and Metallurgy, May 2016

Table 9 – Gabion Baskets and Rip Rap Slope Calculated Factors of Safety

Cross Section	Analysis	Global Minimum Calculated FS	Minimum Target FS
East Gabion Baskets	Bearing Capacity	16.6	2.5
	Sliding Resistance	1.8	1.5
	Overturning	2.2	2.0
	Global Stability (East to West)	1.51	1.5
	Global Stability (North to South)	1.15	1.5
	Global Stability (North to South) with Pile Support	1.53	1.5
West Rip Rap Transition Slope	Global Stability – Normal GW	2.16	1.5
	Global Stability – Fully Saturated Slope	1.14	1.0

Note the Global Stability FS for the north to south cross section across the proposed gabion baskets is less than the target value of 1.5. To increase the global stability factor of safety, additional support will be required. CEC performed additional analyses to determine a lateral pile capacity that would be required to increase the global stability factor of safety to at least 1.5 and determined that an 11-kip lateral capacity pile spaced every 3-feet along the base of the gabion baskets would result in a global stability factor of safety greater than 1.5. CEC discussed vertical support options with a specialty contractor; installation of drilled piles using similar materials to the soil nail construction appear to be a feasible option to increase the global stability of the slope near the east abutter. CEC will work with the selected contractor to design the contractor's chosen vertical support to increase the global stability factor of safety of the gabion baskets.

8.0 GENERAL CONSTRUCTION SEQUENCE AND CONSIDERATIONS

CEC anticipates the construction sequence of the soil nails and gabion baskets will be in the following order. This section is for informational purposes only based on CEC's experience with previous soil nail projects. The order in which the work will be done may change based on the contractor's selected means and methods, and the information below is subject to contractor review.

8.1 Erosion Controls

- Erosion Controls will be installed at the toe of the slope to protect sediment from migrating downstream.

- Slope protection will be installed in areas along the limit of work to minimize the potential for slopes to erode during construction.
- The erosion controls will be monitored and inspected by a qualified professional as construction progresses.

8.2 Soil Nail Sequence

- The existing deck post foundations will be monitored during construction. If signs of ground movement that could adversely impact the deck post foundations are observed, the deck post foundations will be stabilized/shored by the contractor.
- Soil nail construction is anticipated to take place with a specialty "spider" excavator directly on the slope. The spider excavator would temporarily access the site from the beach (see two photos below). In addition, the spider excavator is able to step across any sensitive utility crossings (such as the sewer force main at the bottom of the bank) so that sensitive areas are not subjected to additional surcharge.

Photo 1 – Spider excavator installing soil nails

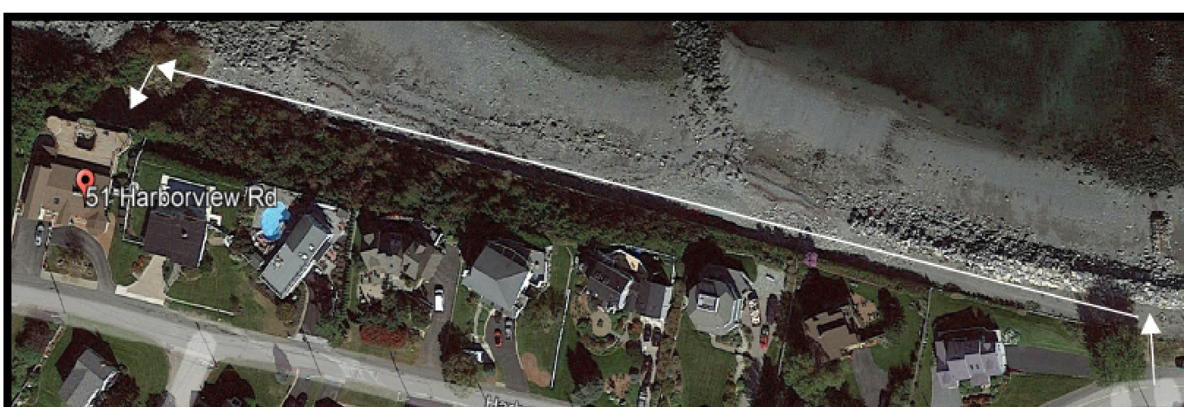


Photo 2 - Anticipated access route for spider excavator

- The timber lagging will be removed incrementally and the soil behind the lagging will be excavated to the grades shown on the plans. The amount of lagging removed in a single sequence will be a function of the behavior of the soil behind the existing wall and how much of the backfill can be exposed without collapse.
- The soil nails will be drilled and grouted. Grout will be mixed from the top of the slope and pumped to the soil nails.
- The strip drains will be installed.
- Welded wire mesh and waler bars will be placed around the soil nails. An initial layer of shotcrete will be sprayed over the soil nail and reinforcement grid. A bearing nut will be placed on each soil nail and the anchor studs will be wet set into the shotcrete. We anticipate the shotcrete will be supplied from the front of the house and pumped to the back.
- Once the initial shotcrete face cures, the next sequence of timber lagging will be removed and the above steps will be repeated.
- At the bottom of the soil nail facing, weephole drainpipes will be installed. The weepholes will connect to the bottom of the strip drains.
- Once the initial shotcrete facing construction is complete, a final shotcrete facing will be constructed. The final facing can be sculpted to achieve an aesthetic look. Upon completion of this step, the soil nails will be complete.
- Riprap will be installed along the base of the soil nails per the plans.

8.3 Gabion Basket Sequence

The gabion baskets will be installed after the soil nails are completed in the following general sequence.

- A 6-inch leveling pad of well-graded sand and gravel will be placed over the footprint of the gabion baskets.
- The proposed gabion baskets are 3-ft by 3-ft by 3-ft and will be filled with gabion stone large enough to prevent loss through the mesh openings and in accordance with the manufacturer’s instructions.
- The filled gabion baskets will be placed according to the layout shown on the plans.
- Rip rap will be placed below the gabion baskets.

8.4 Final Grading and Planting Sequence

After the gabion baskets are installed, grading in accordance with the plans will be required. Excavation of material will take place with the spider excavator and removal of excavated material is anticipated to take place with a conveyor belt system that can be installed from the front of the property down to the bluff (see photo below). The same conveyor belt would also likely be utilized to transport other materials from the front to the back of the property and vice versa, such as rip rap, gabion basket supplies, and construction debris. Fill areas will be benched and compacted into the existing subgrade.

Photo 3 – Example of conveyor belt system that would likely be used to remove excavated material

- On the plans, rip rap is proposed in areas where slopes of steeper than 2:1 are proposed. In areas where rip rap hatch is not present, these areas are graded at a 2:1 and plantings can be established. For additional planting details, refer to the memorandum by Woods Hole Group for the site entitled "51 Harborview Road, Hull MA – Site Restoration and Revegetation", dated November 19, 2019.

- CEC has not proposed any re-grading along the toe of the coastal bank where an existing stone revetment and sewer line owned by the Town of Hull is present. CEC understands that the Town of Hull plans to repair this wall in the future. When additional plans and details are provided for this repair, CEC should be contacted to confirm the repair work does not compromise the slope at 51 Harborview Road.

9.0 DESIGN LIFE AND MAINTENANCE

The design life of permanent soil nails are at least 50 years (Ref. 1). To assess the corrosivity of the on site soils, CEC submitted a soil sample obtained near the face of the existing slope to the laboratory for chloride content, sulfate content, pH, and electrical resistivity testing. The following table provides a summary of the laboratory test results as a comparison to the threshold for determining corrosion potential of soil per Reference 9 above:

Table 10 – Corrosion Test Results vs. Thresholds for Aggressive Soils

Test	Units	Threshold for Non-Aggressive	Result
pH	--	5.0<pH<10	9.1
Resistivity	Ohm-cm	>3,000	33,600/17,400[(1)]
Sulfates	ppm	<200	13
Chlorides	ppm	<100	16

1. Test result indicates as received/saturated.

As indicated in the above table, the soil at the site appears to be generally non-aggressive. However, CEC recognizes the increased corrosion potential at marine sites. As such, CEC recommends the soil nails be coated with Sikagard-62 epoxy coating or similar as an added layer of corrosion protection. In addition, the shotcrete mix will contain Type II cement (moderate sulfate resistance).

In discussion with product manufacturers, CEC recommends the soil nail facing be inspected every five years to maximize the service life. Areas of observed cracking, spalling or rusting should be repaired by a contractor specializing in reinforced concrete repair. In addition, CEC recommends the installation of deformation monitoring points on each column along the column line closest to the existing Retaining Wall and every other column along the center column line at the top of the coastal bank. The points should be read on a monthly basis to determine if the coastal bank is actively moving. When the coastal bank is remediated, CEC should be consulted to modify the deformation monitoring program to include additional points.

CEC also notes that several localized steep areas close to 1:1 in orientation are present within the intact coastal bank to the west of the soil nail repair. If areas of sloughing are observed over time, CEC should be contacted to provide additional guidance. Periodic maintenance of the coastal bank may be required to restore localized areas of the bank and prevent further erosion.

10.0 LIMITATIONS

This calculation package is for conceptual design purposes only. The proposed infrastructure has not been approved and the final design may change with respect to the concepts described in this calculation package and attachments. CEC will update the calculation package as required to incorporate changes that are made to the proposed slope improvements as the project advances into the final design phase.

ATTACHMENT 1
CONCEPTUAL REMEDIATION DESIGN DRAWINGS

LEGEND

- EXISTING SUBJECT PROPERTY LINE
- EXISTING INDEX (MAJOR) CONTOUR
- EXISTING INTERMEDIATE (MINOR) CONTOUR
- EXISTING FEMA FLOOD ZONE
- EXISTING CURB
- EXISTING EDGE OF PAVEMENT
- EXISTING STRUCTURE
- EXISTING STORM PIPE
- PROPOSED INDEX CONTOUR
- PROPOSED INTERMEDIATE CONTOUR
- APPROXIMATE EXTENT OF RIPRAP SURFACE REPAIR
- APPROXIMATE EXTENT OF PROPOSED SOIL NAILS
- TOF= TOP OF FACING
- BOF= BOTTOM OF FACING
- EXISTING STONE BLOCK RETAINING WALL
- PROPOSED GABION BASKETS
- PROPOSED GABION BASKETS

SCALE IN FEET
0 10 20

NOTES

1. REMOVE EXISTING SURFACE GRAVEL AND IMPERVIOUS LINER BEHIND EXISTING TIMBER WALL AS NEEDED DURING CONSTRUCTION OF THE SOIL NAILS. IMPERVIOUS LINER SHALL BE REPLACED AFTER SOIL NAIL CONSTRUCTION IS COMPLETED.
2. ESTABLISH NEW VEGETATION IN ERODED AREAS IN ACCORDANCE WITH MEMO ENTITLED "51 HARBORVIEW ROAD, HULL MA – SITE RESTORATION AND REVEGETATION" BY WOODS HOLE GROUP, DATED NOVEMBER 19, 2019.

REFERENCE

1. AERIAL IMAGERY AND SLOPE TOPOGRAPHY DERIVED FROM UNMANNED AERIAL PHOTOGRAMMETRY DATA COLLECTED BY CEC, INC. ON JUNE 22, 2023. TOPOGRAPHY AT TOP OF SLOPE WITHIN EXISTING PATIO AREA WAS COLLECTED BY CEC DURING A FIELD SURVEY PERFORMED ON FEBRUARY 21, 2024. TOPOGRAPHIC CONTOURS MAPPED AT AN INTERVAL OF 1 FOOT.
2. LOCATION OF STONE BLOCK RETAINING WALL AND EXISTING DECK COLUMNS COURTESY OF SURVEY ENTITLED "TOPOGRAPHIC SITE PLAN OF NORTHERLY PORTION OF #51 HARBORVIEW ROAD, HULL, MA" BY WOODS HOLE GROUP, DATED 09/27/2019.

REVISION RECORD

NO	DATE	DESCRIPTION
1	2/23/2024	RESPONSE TO COMMENTS FROM GZA, CZM, AND CONSERVATION COMMISSION.

Civil & Environmental Consultants, Inc.
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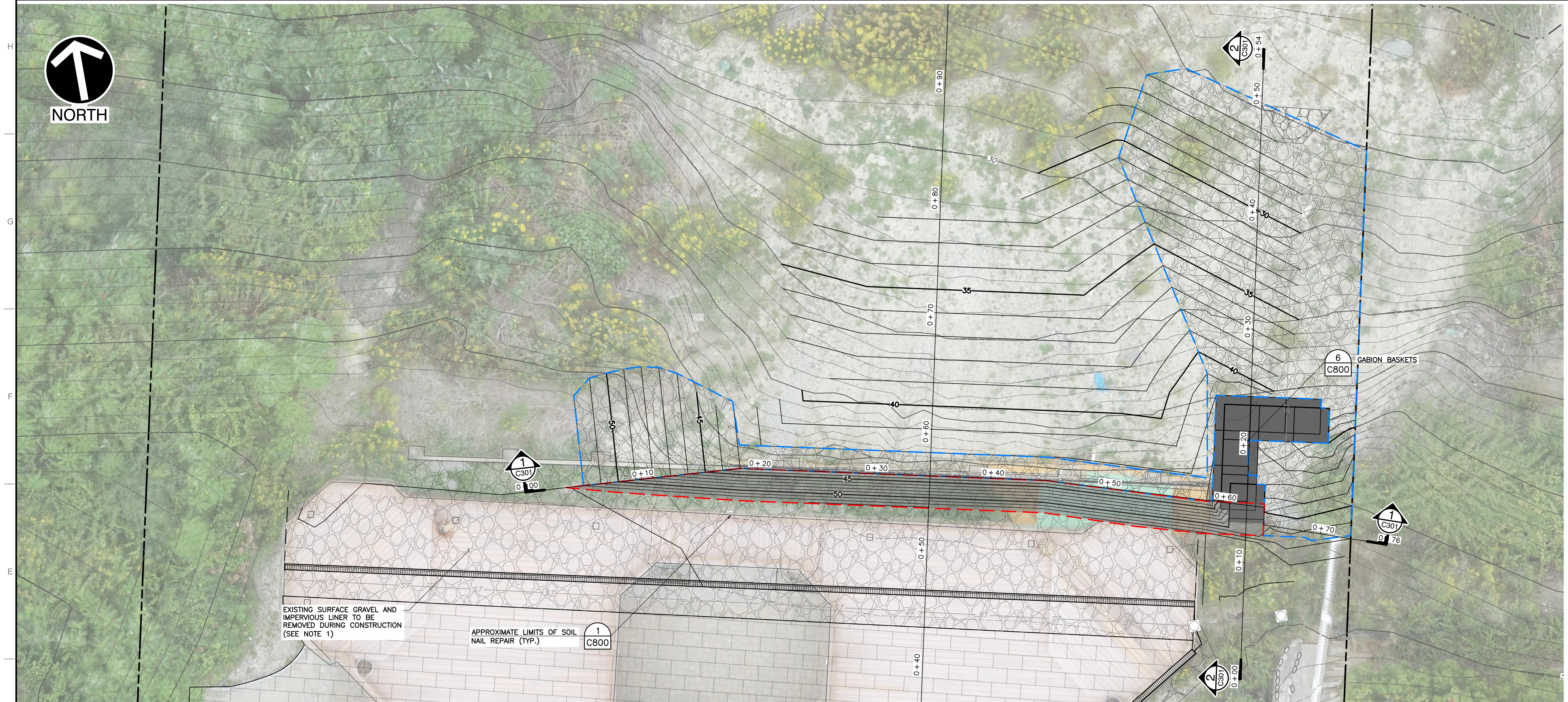
THOMAS P. FITZGERALD
51 HARBORVIEW ROAD
HULL, MASSACHUSETTS 02045

OVERALL PLAN VIEW

DATE:	OCTOBER 2023	DRAWN BY:	JAH
DWG SCALE:	AS SHOWN	CHECKED BY:	TES
PROJECT NO:	324-891		
APPROVED BY:	KPS		

DRAWING NO.:
C300

\\cecinc.com\global\Projects\320-000\324-891\-CADD\Dwg\GT01\324891-GT01-C300-WallPlans.dwg{C300} LS:(2/23/2024 - swilson) - LP: 3/1/2024 4:18 PM

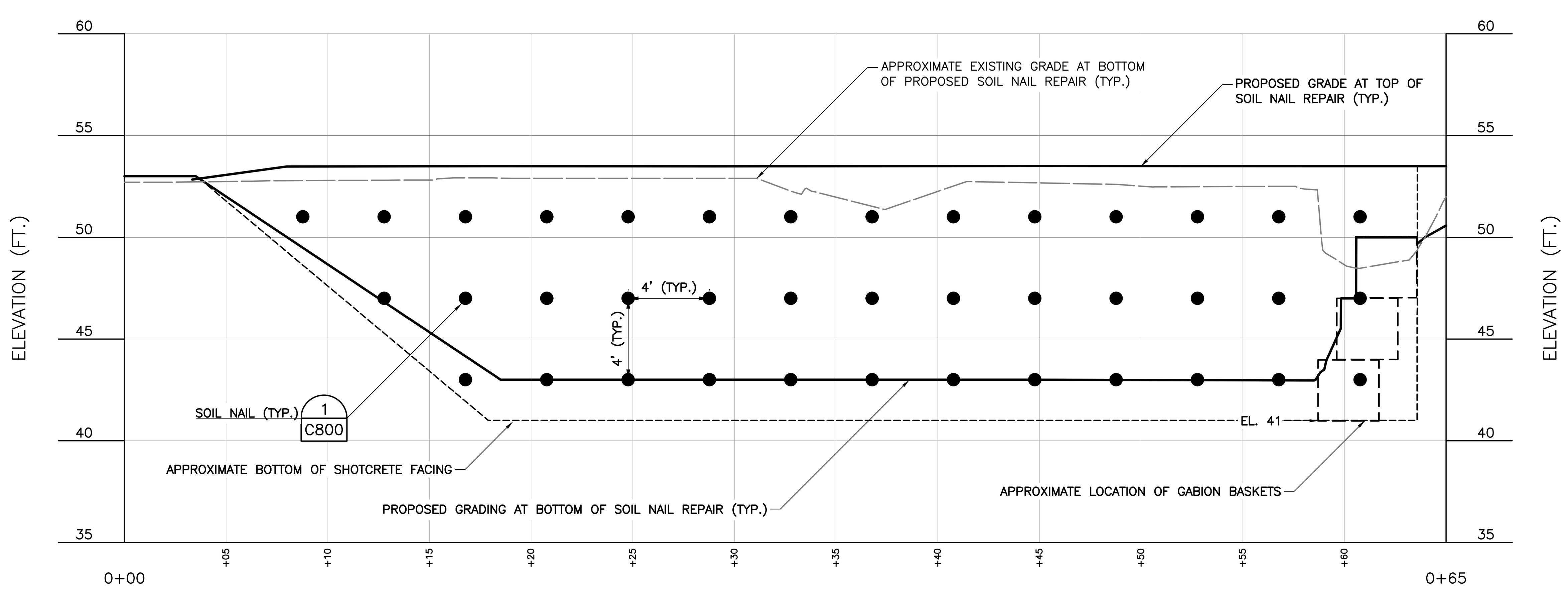


LEGEND

EXISTING SUBJECT PROPERTY LINE	
100	EXISTING INDEX (MAJOR) CONTOUR
	EXISTING INTERMEDIATE (MINOR) CONTOUR
ST	EXISTING STORM PIPE
35	PROPOSED INDEX CONTOUR
	PROPOSED INTERMEDIATE CONTOUR
	APPROXIMATE EXTENT OF RIPRAP SURFACE REPAIR
	APPROXIMATE EXTENT OF PROPOSED SOIL NAILS
	EXISTING STONE BLOCK RETAINING WALL
	PROPOSED GABION BASKETS

SOIL NAIL WITH GABION PLAN VIEW

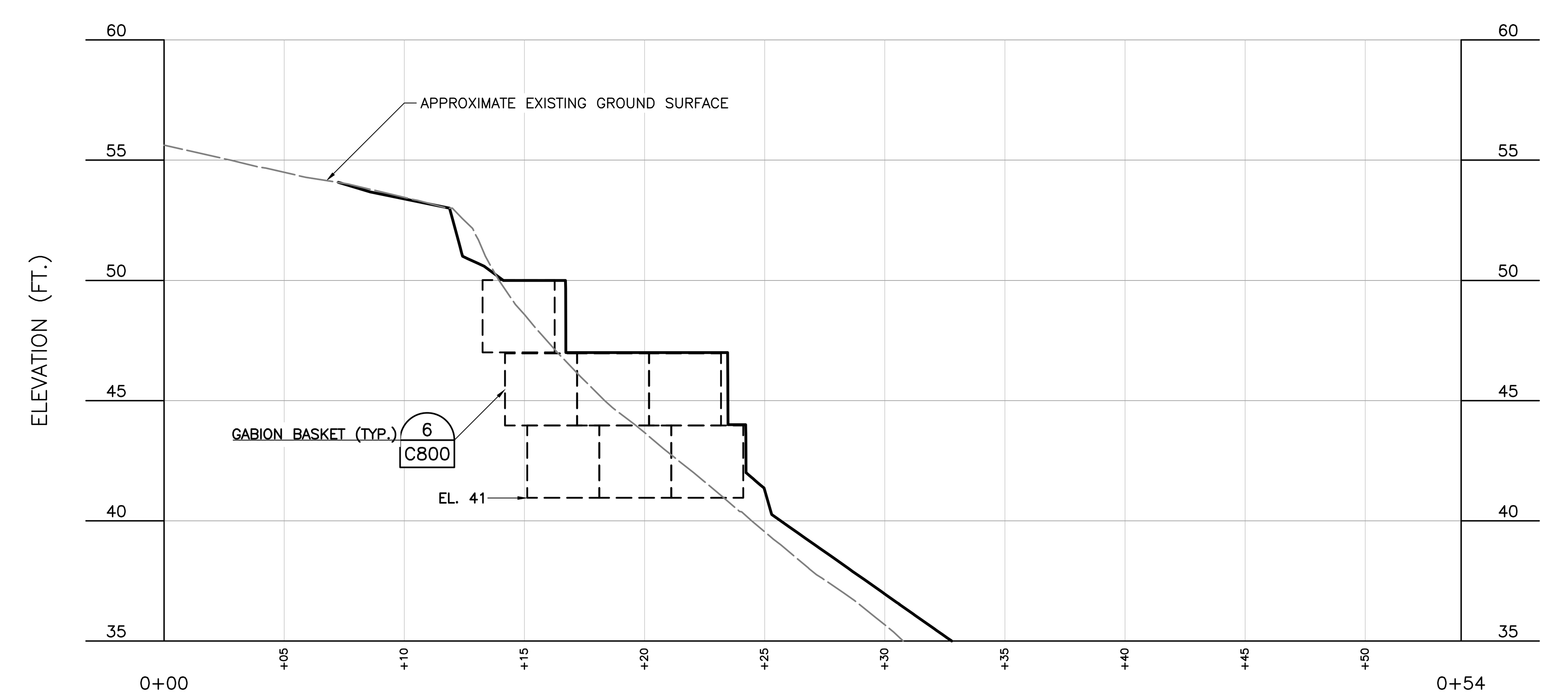
SCALE IN FEET
0 5 10



PROFILE 1

SCALE H:1"=5'; V:1"=5'

SCALE IN FEET
0 5 10



PROFILE 2

SCALE H:1"=5'; V:1"=5'

SCALE IN FEET
0 5 10

LEGEND

	PROPOSED GRADE AT TOP/BOTTOM OF SOIL NAIL REPAIR
	APPROXIMATE EXISTING GRADE AT BOTTOM OF PROPOSED SOIL NAIL REPAIR
	APPROXIMATE BOTTOM OF SHOTCRETE FACING
	GABION BASKET
	SOIL NAIL

REFERENCE

1. AERIAL IMAGERY AND SLOPE TOPOGRAPHY DERIVED FROM UNMANNED AERIAL PHOTOGRAMMETRY DATA COLLECTED BY CEC, INC. ON JUNE 22, 2023. TOPOGRAPHY AT TOP OF SLOPE WITHIN EXISTING PATIO AREA WAS COLLECTED BY CEC DURING A FIELD SURVEY PERFORMED ON FEBRUARY 21, 2024. TOPOGRAPHIC CONTOURS MAPPED AT AN INTERVAL OF 1 FOOT.

REVISION RECORD

NO	DATE	DESCRIPTION
1	2/23/2024	RESPONSE TO COMMENTS FROM GZA, CZM, AND CONSERVATION COMMISSION.

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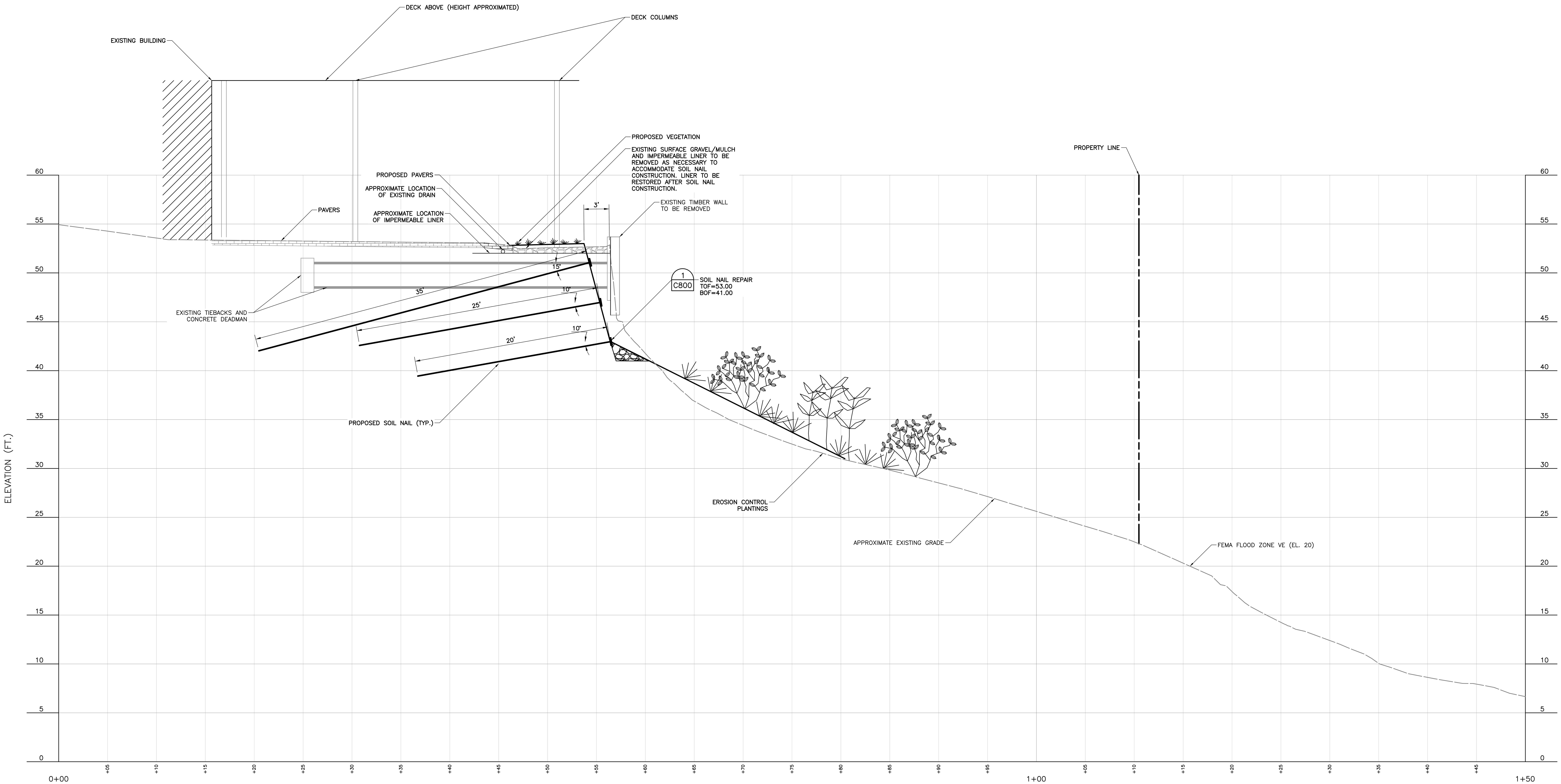
SOIL NAILS WITH GABIONS
PLAN AND PROFILES

DATE:	OCTOBER 2023	DRAWN BY:	JAH
DWG SCALE:	AS SHOWN	CHECKED BY:	TES
PROJECT NO:	324-891	APPROVED BY:	KPS

DRAWING NO.:
C301



PROFILE A

SCALE H:1"=5'; V:1"=5'

SCALE IN FEET

0 5 10

REVISION RECORD		
NO	DATE	DESCRIPTION
1	2/23/2024	RESPONSE TO COMMENTS FROM GZA, CZM, AND CONSERVATION COMMISSION.

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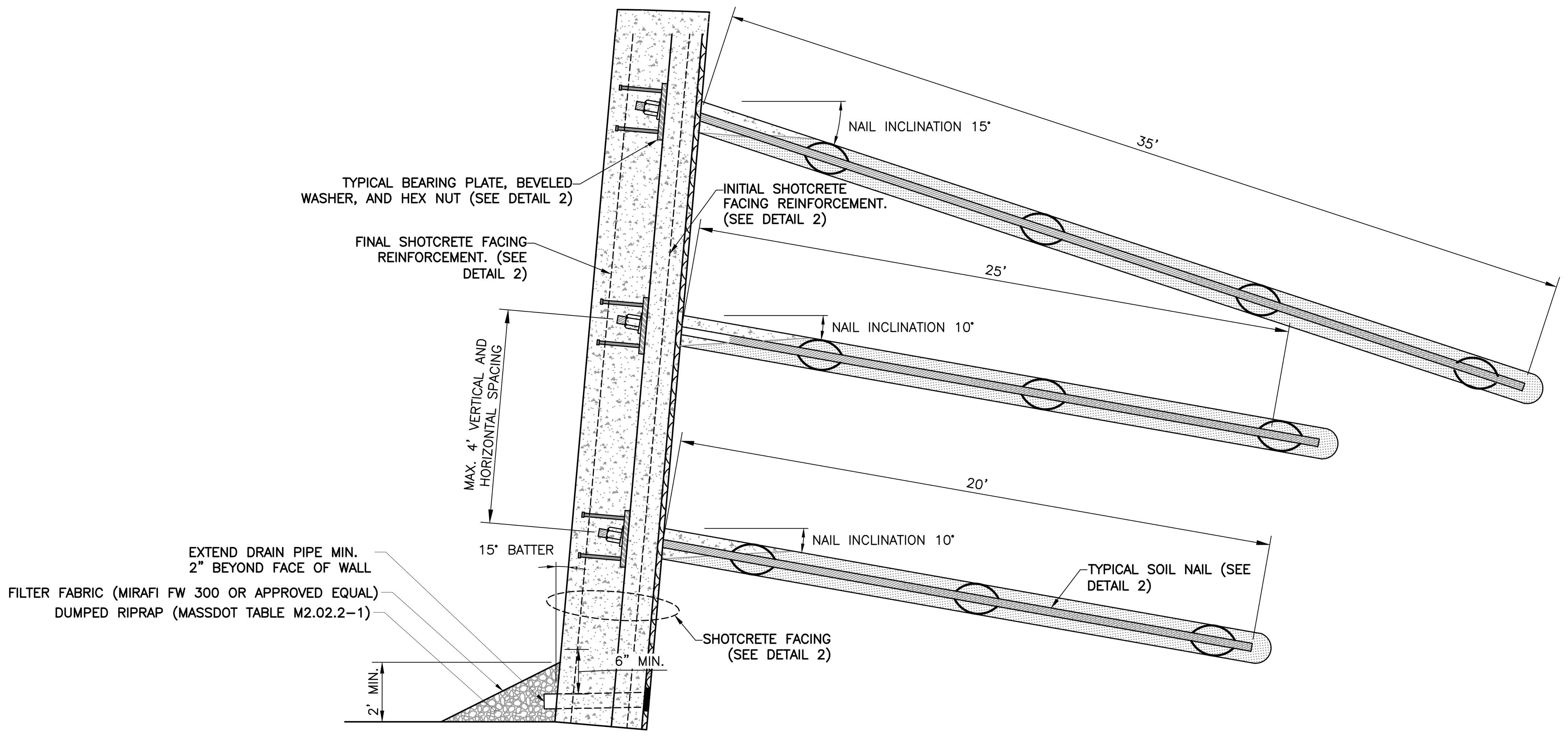
GEOTECHNICAL CROSS SECTION

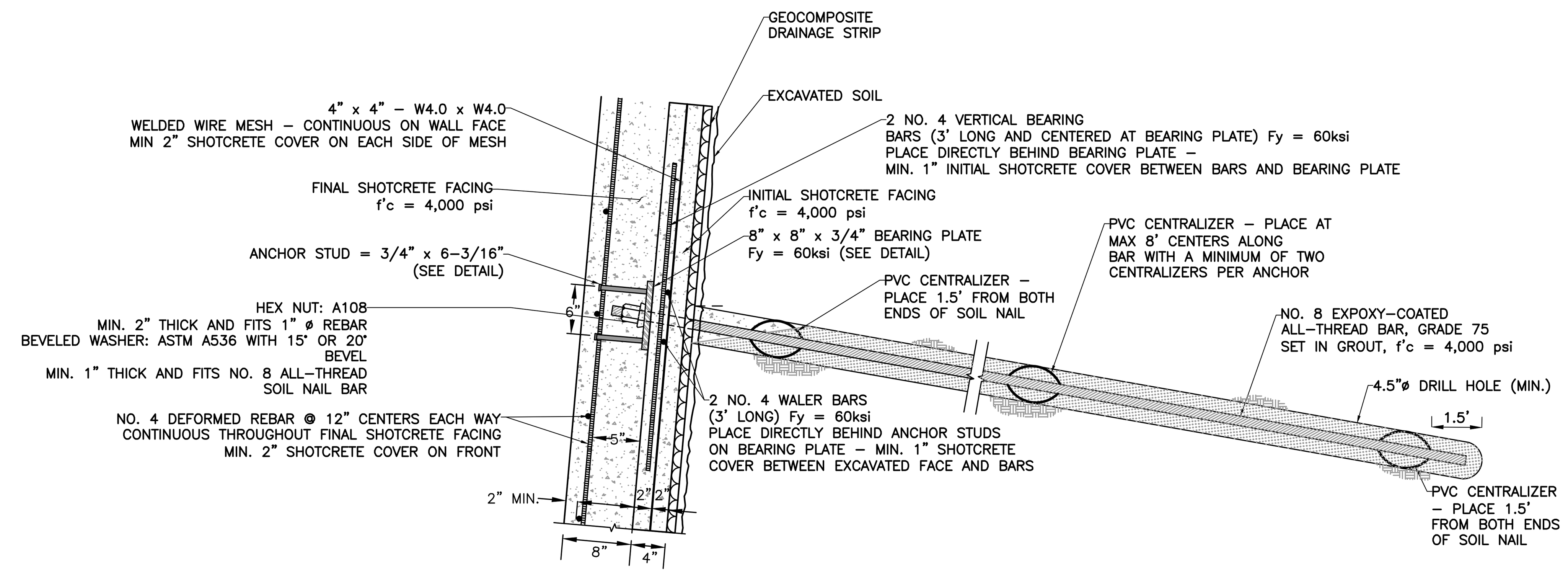
DATE:	OCTOBER 2023	DRAWN BY:	JAH
DWG SCALE:	AS SHOWN	CHECKED BY:	TES
PROJECT NO:	324-891	APPROVED BY:	KPS

DRAWING NO.:

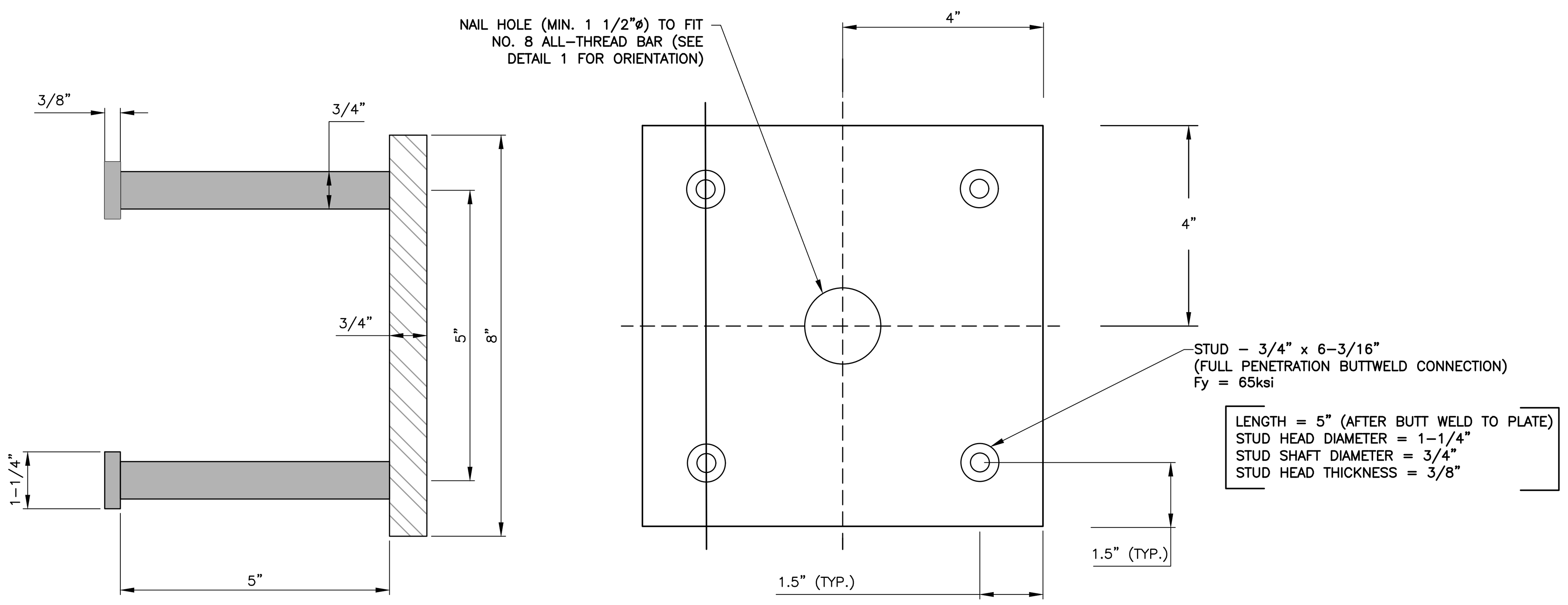
C302



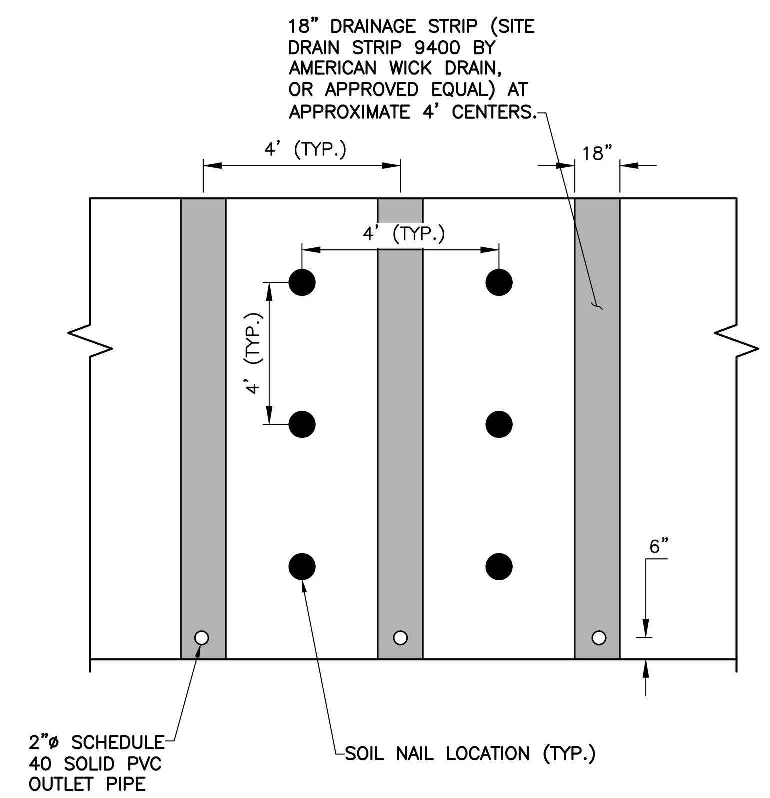
DETAIL 1
SOIL NAIL REPAIR
N.T.S.



DETAIL 2
SHOTCRETE FACING AND SOIL NAIL
N.T.S.

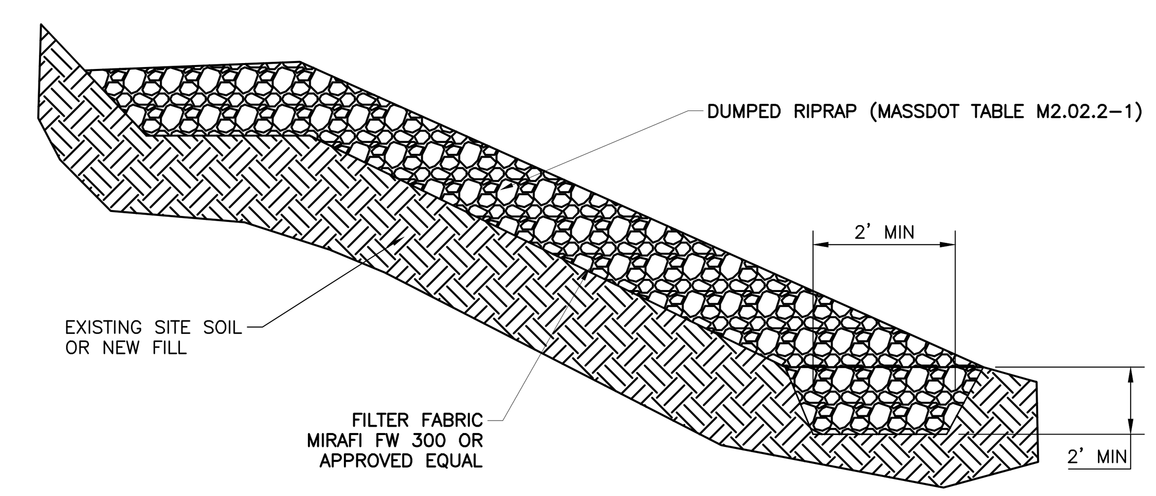


DETAIL 3
BEARING PLATE WITH STUD
N.T.S.



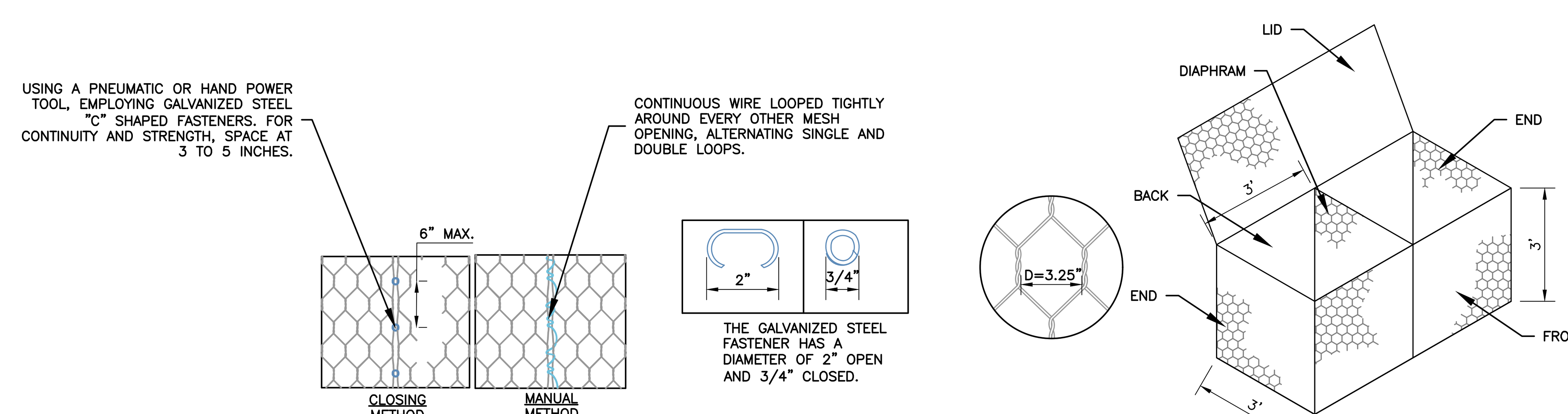
NOTE:
1. SPLICE DRAINAGE STRIPS ACCORDING TO MANUFACTURER'S RECOMMENDATIONS. (12" MIN.)
2. INSTALL ADDITIONAL DRAINAGE STRIPS AT SEEP LOCATIONS AS NECESSARY.

DETAIL 4
SOIL NAIL DRAINAGE STRIPS
N.T.S.



NOTE:
1. ARMOR SLOPE WITH RIPRAP (MASSDOT TABLE M2.02-1) AT A MINIMUM THICKNESS OF 1-FOOT WHERE APPLICABLE FOR EROSION CONTROL.

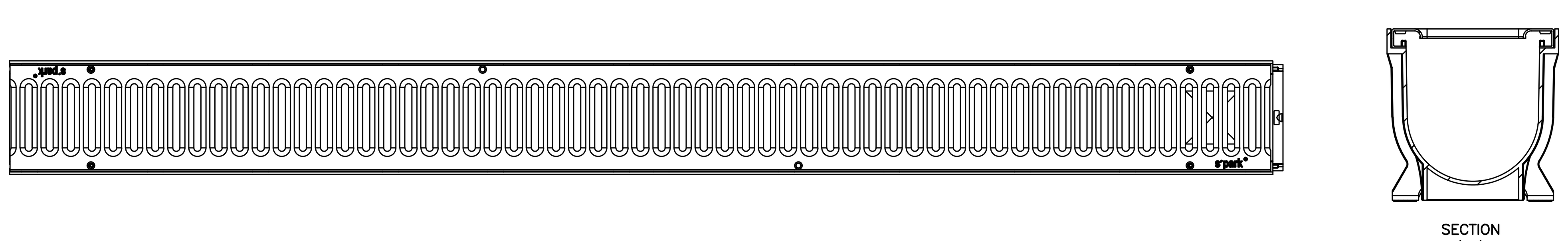
DETAIL 5
SURFICIAL RIPRAP ARMOR
N.T.S.



GABION NOTES:

1. GABIONS SHALL BE CONSTRUCTED OF NON-WELDED (TWISTED) GALVANIZED WIRE MESH AND CONFORM TO ASTM A975. THE USE OF FASTENER RINGS IN THE ASSEMBLY OF THE GABION BASKETS IS ALSO PERMITTED. PROVIDING THE RINGS HAVE AT LEAST THE SAME TENSILE STRENGTH, ZINC COATING, AND SIZE AS THE WIRE USED IN THE BODY OF THE GABION.
2. INSTALL GABIONS IN ACCORDANCE WITH THE DESIGN DRAWINGS AND THE MANUFACTURER'S SPECIFICATIONS.
3. PLACE GABIONS ON A MINIMUM 6-INCH THICK LAYER OF SAND AND GRAVEL (MASSDOT TABLE M1.03.1) COMPACTED TO 95% OF THE MAXIMUM DRY DENSITY) AS DETERMINED BY ASTM D698.
4. FILL MATERIAL FOR GABIONS SHALL CONSIST OF GABION STONE. THE DIMENSIONS OF THE ROCK OR STONE FILLINGS SHALL BE SUCH AS TO PREVENT LOSS OF MATERIAL THROUGH MESH OPENINGS WITH A MINIMUM AMOUNT OF VOIDS.
5. ROCK SHALL BE PLACED IN 12-INCH LIFTS. THE FILL LAYER SHALL NEVER BE MORE THAN 12 INCHES HIGHER THAN ANY ADJOINING CELL.
6. SUFFICIENT HAND MANIPULATION OF THE ROCK SHALL BE PERFORMED TO MINIMIZE VOIDS AND ACHIEVE A MAXIMUM DENSITY OF ROCK IN THE GABION.

DETAIL 6
GABION BASKETS
N.T.S.



DETAIL 7
TYPICAL TRENCH DRAIN
N.T.S.

GENERAL NOTES:

1. ESTABLISH EROSION CONTROL PLANTINGS ON SLOPES WHERE RIP RAP IS NOT PROPOSED. PLANTINGS SHALL BE ESTABLISHED IN ACCORDANCE WITH MEMORANDUM BY WOODS HOLE GROUP ENTITLED '51 HARBORVIEW ROAD, HULL MA – SITE RESTORATION AND REVEGETATION', DATED NOVEMBER 19, 2019.

REVISION RECORD

NO	DATE	DESCRIPTION
1	2/23/2024	RESPONSE TO COMMENTS FROM GZA, CZM, AND CONSERVATION COMMISSION.

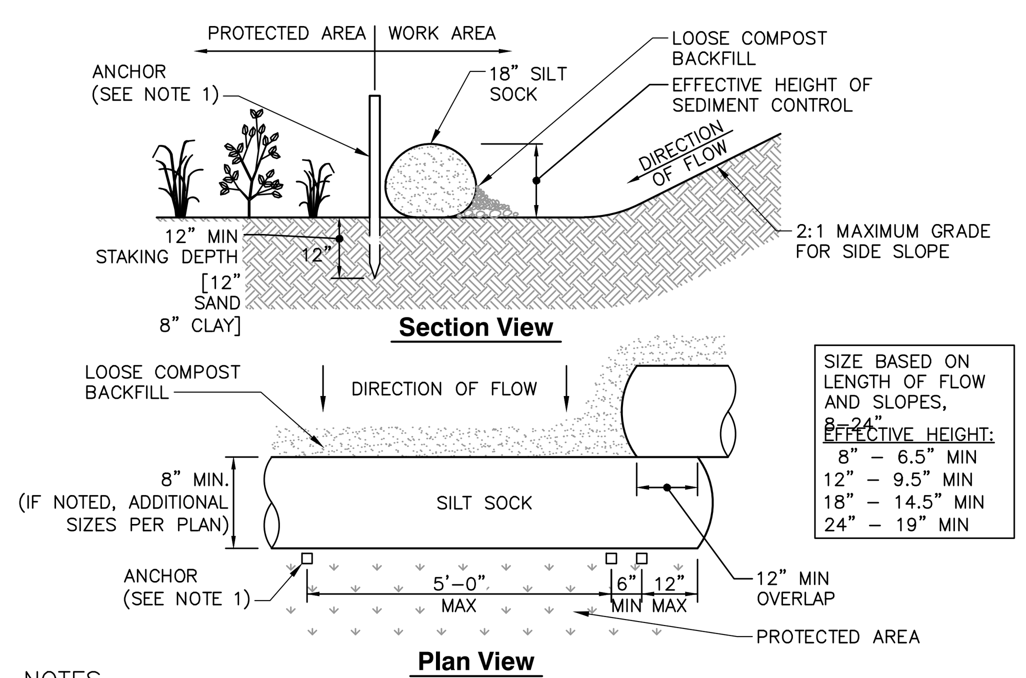
Civil & Environmental Consultants, Inc.
31 Bellows Road · Raynham, MA 02767
Ph: 774.501.2176 · 866.312.2024 · Fax: 774.501.2669
www.cecinc.com

THOMAS P. FITZGERALD
51 HARBORVIEW ROAD
HULL, MASSACHUSETTS 02045

DETAILS

DATE:	OCTOBER 2023	DRAWN BY:	JAH
DWG SCALE:	AS SHOWN	CHECKED BY:	TES
PROJECT NO:			324-891
APPROVED BY:			KPS

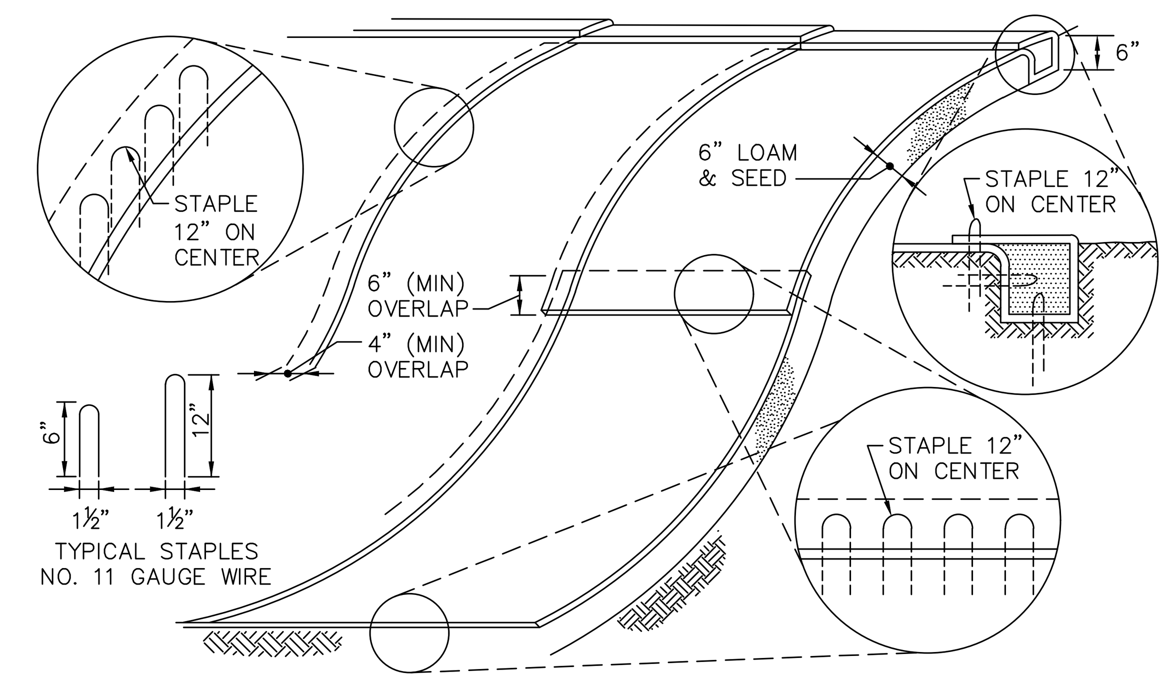
DRAWING NO.:
C800



NOTES

1. ANCHOR TO BE A 2" X 2" X 36" HARD WOOD STAKE, OR APPROVED EQUAL.
2. COMPOST FOR SILT SOCK FILL MATERIAL TO BE PROVIDED BY THE MANUFACTURER IN CONJUNCTION WITH THE ENGINEER TO PROVIDE THE REQUIRED REMOVAL OF SEDIMENT OR OTHER POLLUTANTS FROM RUNOFF.
3. COMPOST MATERIAL SHALL BE DISPERSED ON SITE, AS DETERMINED BY THE LOCAL CONSERVATION AGENT OR THE DESIGN ENGINEER.
4. SILT SOCK SHALL BE INSPECTED PER LOCAL AND STATE REQUIREMENTS. REPAIR OR REPLACEMENT SHALL BE PERFORMED PROMPTLY, AS NEEDED.
5. METHOD OF INSTALLATION SHALL BE AS PER MANUFACTURER'S RECOMMENDATIONS.

DETAIL 8
SILT SOCK
N.T.S.



CONSTRUCTION NOTES

1. BEGIN AT THE TOP OF BLANKET INSTALLATION AREA BY ANCHORING BLANKET IN A SIX (6) INCH DEEP TRENCH. BACKFILL AND COMPACT TRENCH AFTER STAPLING.
2. ROLL THE BLANKET DOWN THE SLOPE IN THE DIRECTION OF THE WATER FLOW.
3. THE EDGES OF BLANKETS MUST BE STAPLED WITH APPROXIMATELY FOUR (4) INCH OVERLAP WHERE TWO (2) OR MORE STRIP WIDTHS ARE REQUIRED.
4. WHEN BLANKETS MUST BE SPLICED DOWN THE SWALE, PLACE UPPER BLANKET END OVER LOWER END WITH SIX (6) INCH MINIMUM OVERLAP AND STAPLE BOTH TOGETHER.
5. MORE FASTENERS MAY BE REQUIRED DUE TO BLANKET COMPOSITION, SOIL TYPE, SURFACE UNIFORMITY AND SLOPE STEEPNESS. METHOD OF INSTALLATION SHALL BE AS PER MANUFACTURER'S RECOMMENDATIONS.
6. TO BE PLACED ON SLOPES 3:1 OR GREATER, AS SHOWN ON PLANS.

DETAIL 9
EROSION CONTROL BLANKET
N.T.S.

GENERAL NOTES:

1. ESTABLISH EROSION CONTROL PLANTINGS ON SLOPES WHERE RIP RAP IS NOT PROPOSED. PLANTINGS SHALL BE ESTABLISHED IN ACCORDANCE WITH MEMORANDUM BY WOODS HOLE GROUP ENTITLED '51 HARBORVIEW ROAD, HULL MA – SITE RESTORATION AND REVEGETATION', DATED NOVEMBER 19, 2019.

REVISION RECORD

NO	DATE	DESCRIPTION
1	2/2/2024	RESPONSE TO COMMENTS FROM GZA, CZM, AND CONSERVATION COMMISSION.

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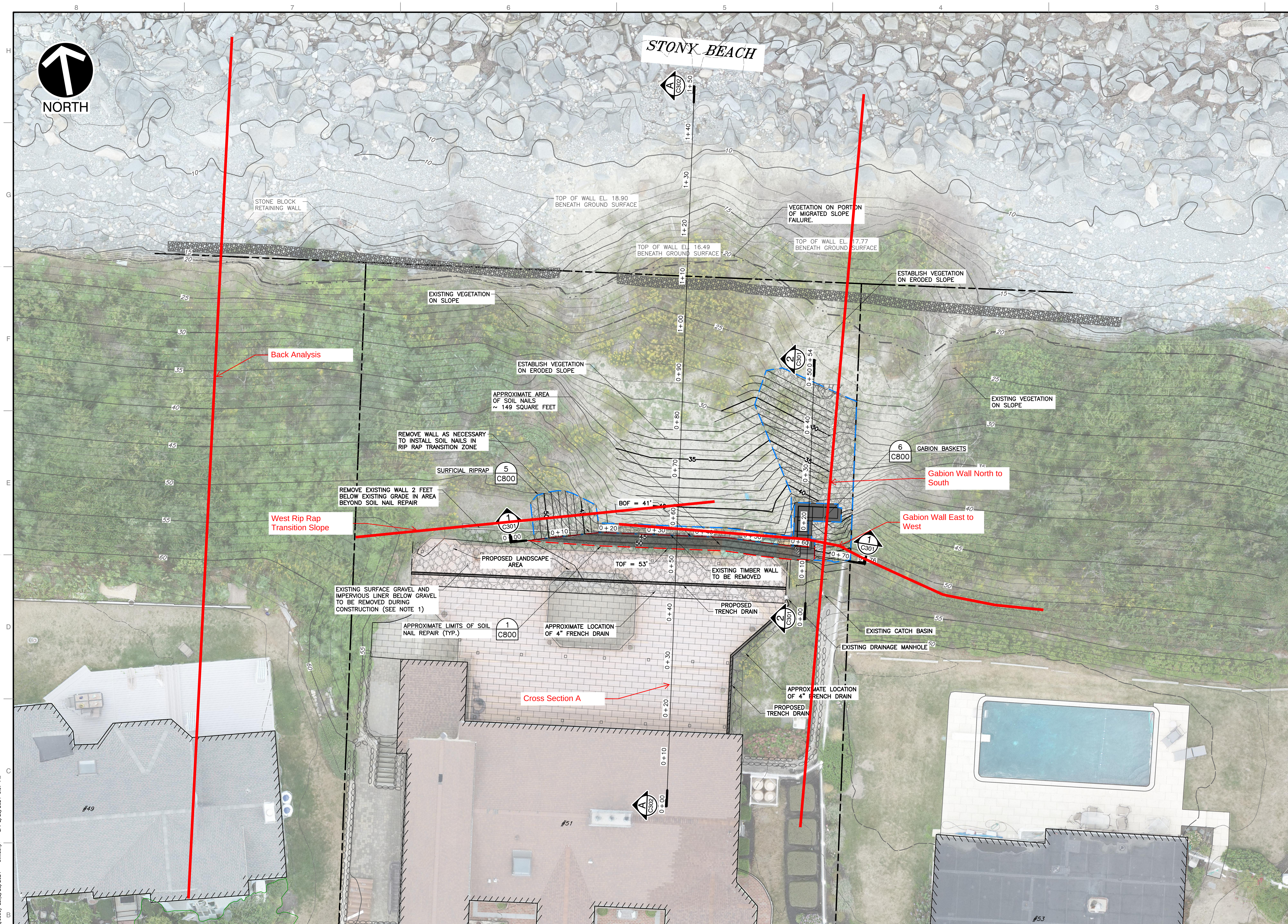
51 HARBORVIEW ROAD
HULL, MASSACHUSETTS 02045

DETAILS

DATE:	OCTOBER 2023	DRAWN BY:	JAH
DWG SCALE:	AS SHOWN	CHECKED BY:	TES
PROJECT NO:	324-891	APPROVED BY:	KPS

DRAWING NO.:
C801

ATTACHMENT 2
STABILITY CROSS SECTION LOCATION PLAN



LEGEND

- EXISTING SUBJECT PROPERTY LINE
- EXISTING INDEX (MAJOR) CONTOUR
- EXISTING INTERMEDIATE (MINOR) CONTOUR
- EXISTING FEMA FLOOD ZONE
- EXISTING CURB
- EXISTING EDGE OF PAVEMENT
- EXISTING STRUCTURE
- EXISTING STORM PIPE
- PROPOSED INDEX CONTOUR
- PROPOSED INTERMEDIATE CONTOUR
- APPROXIMATE EXTENT OF RIPRAP SURFACE REPAIR
- APPROXIMATE EXTENT OF PROPOSED SOIL NAILS
- TOF= TOP OF FACING
- BOF= BOTTOM OF FACING
- EXISTING STONE BLOCK RETAINING WALL
- PROPOSED GABION BASKETS

REVISION RECORD

NO	DATE	DESCRIPTION
1	2/23/2024	RESPONSE TO COMMENTS FROM GZA, CZM, AND CONSERVATION COMMISSION.

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THOMAS P. FITZGERALD
51 HARBORVIEW ROAD
HULL, MASSACHUSETTS 02045

Stability Cross Sections

DATE:	OCTOBER 2023	DRAWN BY:	JAH
DWG SCALE:	AS SHOWN	CHECKED BY:	TES
PROJECT NO:	324-891	APPROVED BY:	KPS

DRAWING NO.:

SCALE IN FEET
0 10 20

NOTES

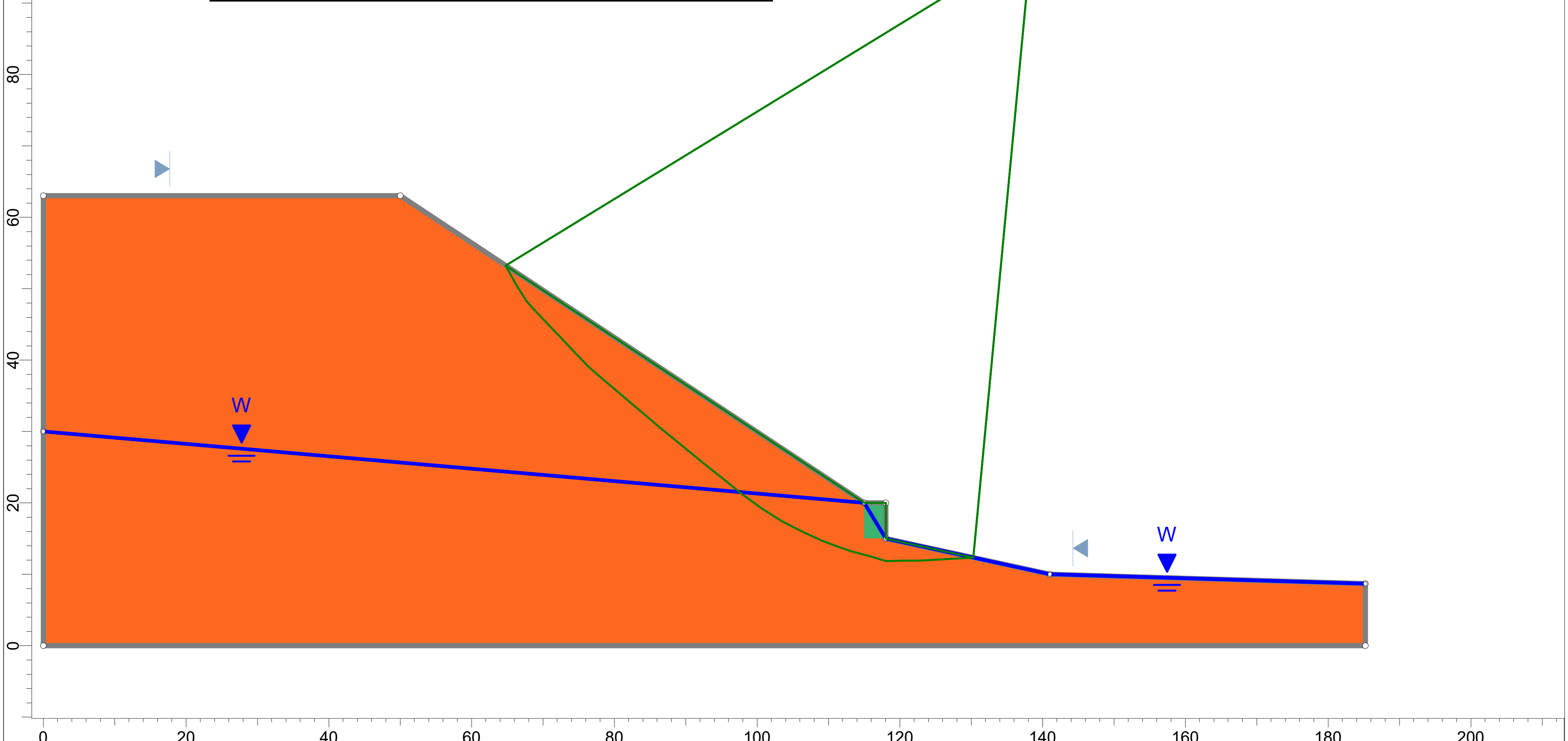
1. REMOVE EXISTING SURFACE GRAVEL AND IMPERVIOUS LINER BEHIND EXISTING TIMBER WALL AS NEEDED DURING CONSTRUCTION OF THE SOIL NAILS. IMPERVIOUS LINER SHALL BE REPLACED AFTER SOIL NAIL CONSTRUCTION IS COMPLETED.
2. ESTABLISH NEW VEGETATION IN ERODED AREAS IN ACCORDANCE WITH MEMO ENTITLED "51 HARBORVIEW ROAD, HULL MA – SITE RESTORATION AND REVEGETATION" BY WOODS HOLE GROUP, DATED NOVEMBER 19, 2019.

REFERENCE

1. AERIAL IMAGERY AND SLOPE TOPOGRAPHY DERIVED FROM UNMANNED AERIAL PHOTOGRAMMETRY DATA COLLECTED BY CEC, INC. ON JUNE 22, 2023. TOPOGRAPHY AT TOP OF SLOPE WITHIN EXISTING PATIO AREA WAS COLLECTED BY CEC DURING A FIELD SURVEY PERFORMED ON FEBRUARY 21, 2024. TOPOGRAPHIC CONTOURS MAPPED AT AN INTERVAL OF 1 FOOT.
2. LOCATION OF STONE BLOCK RETAINING WALL AND EXISTING DECK COLUMNS COURTESY OF SURVEY ENTITLED "TOPOGRAPHIC SITE PLAN OF NORTHERLY PORTION OF #51 HARBORVIEW ROAD, HULL, MA" BY WOODS HOLE GROUP, DATED 09/27/2019.

ATTACHMENT 3
BACK ANALYSIS AND OUTPUTS

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Native Glacial Till		135	Mohr-Coulomb	50	36	Water Surface	Custom	1
Stone Revetment		140	Mohr-Coulomb	5000	0	Water Surface	Custom	1



Civil & Environmental Consultants, Inc.	*Project* 51 Harborview Rd Hull, MA Slope Stabilization		
	Analysis Description West Slope Existing Conditions Back Analysis	*Figure*	
	Drawn By TS	*Company* CEC Inc.	
SLIDEINTERPRET 9.026	*Date* 9/5/23	*File Name* Till Back Analysis West Slope.slmd	

Civil & Environmental Consultants, Inc.

Project Name:	51 Harborview Rd Slope Stabilization	**Date:**	10/3/2023
Project Location:	Hull, MA	**Calculated By:**	MT
Project Number:	324-891	**Checked By:**	TS

FRIMPTER METHOD

Frimpter Equation

$$\frac{S_c - S_h}{Ow_c - Ow_{max}} = \frac{S_r}{Ow_r}$$

S_h	Maximum water level height at site
S_c	Current water level at site
S_r	Probable "range" at the site
Ow_c	Current water level at observation well
Ow_r	Observation well range
Ow_{max}	Maximum water level height at observation well

$$S_h = S_c - \left(\frac{S_r * (Ow_c - Ow_{max})}{Ow_r}\right)$$

Observation Hole/Well #

https://newengland.water.usgs.gov/web_app/frimpter/frimpter.html

Site	MA-HGW 76R HANSON, MA		
Type	Stratified Drift		
Setting	Valley Flat		
Ow_c	4.63	ft	BGS
Ow_r	3.23	ft	BGS
Ow_{max}	2.4	ft	BGS

Site Range

Till	17.0	ft	
Terraces	10.0	ft	
Valley Flat	4.2	ft	
S_r	4.2	ft	BGS

Site Condition

S_c	30.0	ft	BGS
S_h	29.39	ft	BGS

Civil & Environmental Consultants, Inc.

Project Name:	51 Harborview Rd Slope Stabilization	**Date:**	10/3/2023
Project Location:	Hull, MA	**Calculated By:**	MT
Project Number:	324-891	**Checked By:**	TS

FRIMPTER METHOD

Frimpter Equation

$$\frac{S_c - S_h}{Ow_c - Ow_{max}} = \frac{S_r}{Ow_r}$$

S_h	Maximum water level height at site
S_c	Current water level at site
S_r	Probable "range" at the site
Ow_c	Current water level at observation well
Ow_r	Observation well range
Ow_{max}	Maximum water level height at observation well

$$S_h = S_c - \left(\frac{S_r * (Ow_c - Ow_{max})}{Ow_r}\right)$$

Observation Hole/Well #

https://newengland.water.usgs.gov/web_app/frimpter/frimpter.html

Site	MA-DVW 10R DOVER, MA		
Type	Stratified Drift		
Setting	Terrace		
Ow_c	31.42	ft	BGS
Ow_r	5.65	ft	BGS
Ow_{max}	28.92	ft	BGS

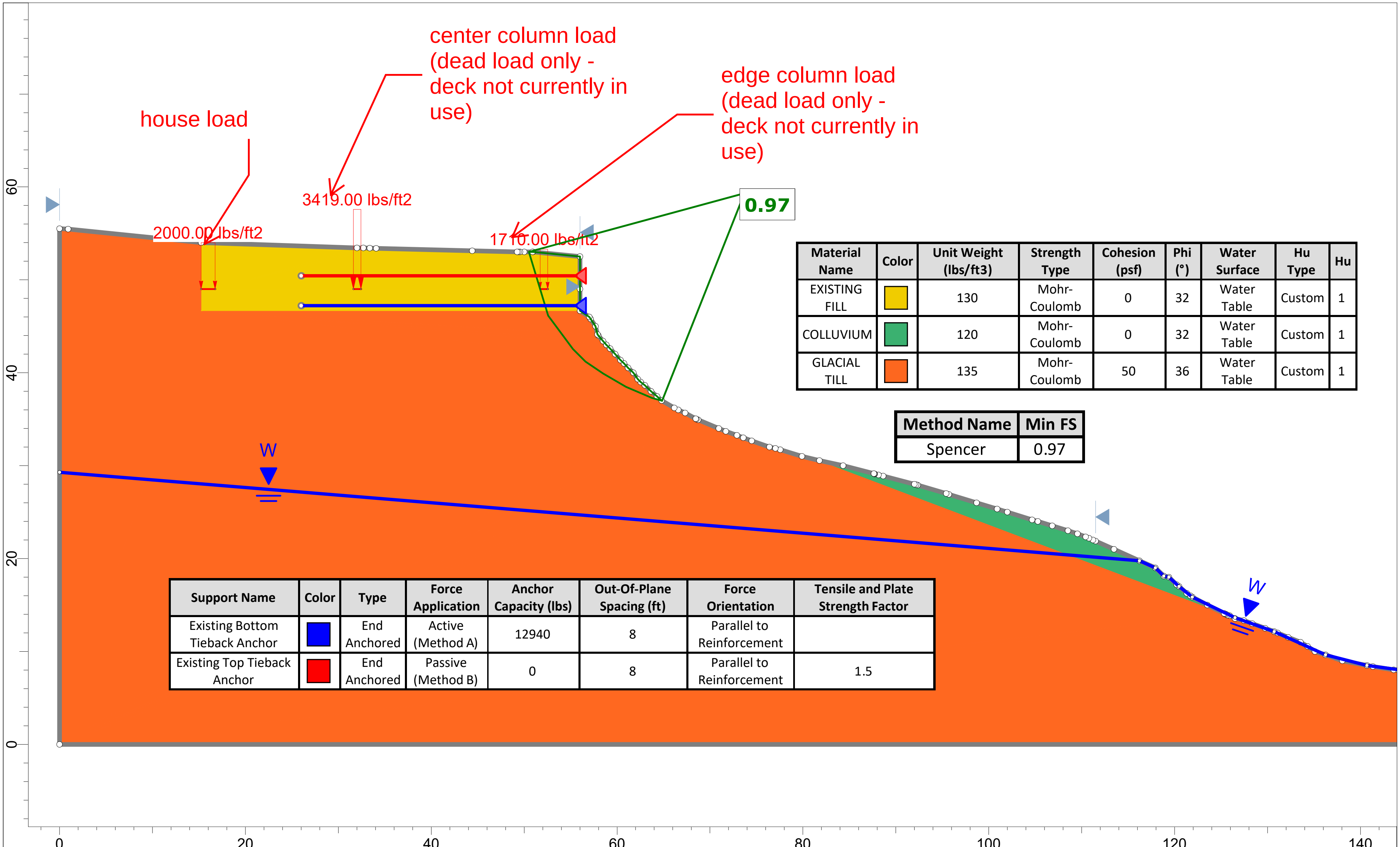
Site Range

Till	17.0	ft	
Terrace	10.0	ft	
Valley Flat	4.2	ft	
S_r	10.0	ft	BGS

Site Condition

S_c	30.0	ft	BGS
S_h	28.67	ft	BGS

ATTACHMENT 4
EXISTING CONDITIONS CALCULATIONS AND OUTPUTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface	Hu Type	Hu
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table	Custom	1
COLLUVIUM		120	Mohr-Coulomb	0	32	Water Table	Custom	1
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table	Custom	1

Method Name	Min FS
Spencer	0.97

Support Name	Color	Type	Force Application	Anchor Capacity (lbs)	Out-Of-Plane Spacing (ft)	Force Orientation	Tensile and Plate Strength Factor
Existing Bottom Tieback Anchor		End Anchored	Active (Method A)	12940	8	Parallel to Reinforcement	
Existing Top Tieback Anchor		End Anchored	Passive (Method B)	0	8	Parallel to Reinforcement	1.5

CEC
Civil & Environmental Consultants, Inc.
SLIDEINTERPRET 9.031

Project	51 Harborveiw Rd Hull		
Analysis Description	Exisiting Condition	*Figure*	
Drawn By	TS/SM	*Company*	CEC Inc.
Date	2/20/24	*File Name*	324-891 - Cross Section A-A.slmd

Client: Tom Fitzgerald **Job Number:** 324-891 **PAGE:** 1
Project: 51 Harborview Rd Slope Stability **Computed by:** G. Swarm **DATE:** 9/8/2023
Detail: Existing Tieback Analysis **Checked by:** T. Sousa

Purpose:

Determine tensile forces in the existing tiebacks behind the soldier pile and lagging wall.

Reference:

1. "Earth Retaining System", by Antonopoulos Company, sheet S-1 dated November 6, 2019
2. State of California Deparment of Transportation Trenching and Shoring Manual, Rev. 1 August 2011
3. Ameircan Institute of Steel Construction, "Base Plate and Anchor Rod Design", Second Edition, 2006
4. CEC Boring Log Boring Number B-1, dated 6/28/23.

General Assumptions:

1. Tiebacks and deadman are a passive system - no pre-tensioning.
2. Assume backfill in anchor trenches is sand and gravel per discussion with owner.
3. No hydrostatic pressures
4. Wall is not restrained at the top. Therefore, active pressures apply.

Approach:

Step 1: Soil parameters were estimated based on discussion with owner and recent boring log data (B-1).
Step 2: Input data from design drawings.
Step 3: Determine wall pressure.
Step 4: Determine anchor capacity and check that it is less than wall pressure.
Step 5: Determine deadman capacity and check that it is less than wall pressure.

General Equations:

Wall Pressure:

Lateral Pressure = 0.5* K_a*ϒ*H^2

where K_a= Active earth coefficient
H= Retained height of wall
ϒ= Unit weight of backfill soil

Anchor Capacity:

Design Tensile Strength = 0.75*F_u*A_t (Page 19, Reference 3)

where F_u = Ultimate tensile stress
A_t= Tensile stress area

Deadman Capacity:

T_{ult} = L(P_p-P_a) (Eq 9-2 Reference 2)

where L= Length of anchor block

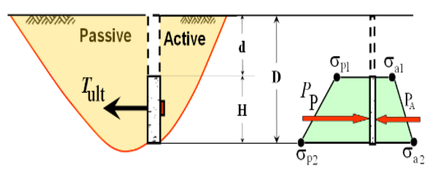
$$P_A = \left[\frac{\sigma_{a1}+\sigma_{a2}}{2}\right]H \quad \text{Eq. 9-13}$$

$$\sigma_{p1} = \gamma d k_p \quad \text{Eq. 9-14}$$

$$\sigma_{p2} = \gamma D k_p \quad \text{Eq. 9-15}$$

$$P_P = \left[\frac{\sigma_{p1}+\sigma_{p2}}{2}\right]H \quad \text{Eq. 9-16}$$

Reference 2 below



Soil Profile:

Refer to Reference 1 and 4 - sand and gravel fill in anchor trenches, surrounded by till.

Soil Property Equations

Total Unit Weight:

<u>Fill Stratum</u>

Fill and till is assumed to have γ_T = 130 and 135 pcf, respectively.

Results:

Tables: Anchor tensile resistance = 12.94 kips, which is equal to lateral earth pressure and less than allowable tensile resistance from steel section.
Since centroid of earth pressure distribution is at the bottom anchor, assume bottom anchor is under full tension of 12.94 kips.

ANCHOR INFORMATION

Anchor Yield Stress	τ_{bar}	36	ksi
Anchor Diameter	d_{bar}	1.25	in
Steel factor of safety	FS_S	1.8	--
Number of Anchor Rows	σ_{bar}	2	--
Horizontal Anchor Spacing	W_{bar}	8	ft

DEADMAN INFORMATION

Unit Weight - soil	γ	130	pcf
Friction Angle - soil	ϕ	32	degrees
Top Depth - deadman	d_t	2	feet
Bottom Depth - deadman	d_b	6.5	feet

WALL INFORMATION

Unit Weight - soil	γ	130	pcf
Friction Angle - soil	ϕ	32	degrees
Height of Wall	H	9	feet
Exposed Height of Wall - front	h_f	9	feet
Exposed Height of Wall - back	h_b	0	feet

Unit Weight -backfill soil	γ	130	pcf	
Friction Angle - backfill soil	ϕ	32	degrees	
Height of Wall	H	9	feet	height of posts
Exposed Height of Wall - front	h_f	9	feet	height of posts
Exposed Height of Wall - back	h_b	0	feet	
(45-ϕ/2) in radians	--	0.5061	radians	
(45+ϕ/2) in radians	--	1.0647	radians	
Active Earth Pressure Coefficient	K_a	0.3073	--	
Passive Earth Pressure Coefficient	K_p	3.2546	--	
Vertical Stress - back	$\sigma_{vy\text{-}b}$	1170	psf	
Vertical Stress - front	$\sigma_{vy\text{-}f}$	0	psf	
Active Lateral Earth Pressure - Wall	P_A	1617.7	lb/ft	
Passive Lateral Earth Pressure (surface to top deadman)	P_P	0.0	lb/ft	
Resulting Lateral Pressure on Wall - per foot	P_W	**1617.7**	**lb/ft**	
Resulting Lateral Pressure on Wall - for anchor spacing	P_{WT}	**12.94**	**kip**	

Anchor Yield Stress	τ_{bar}	36	ksi	from shop drawings
Anchor Diameter	d_{bar}	1.25	in	from shop drawings
Allowable Capacity Per Anchor	$\mathbf{P_{all\text{-}bar}}$	**26.70**	**kip**	

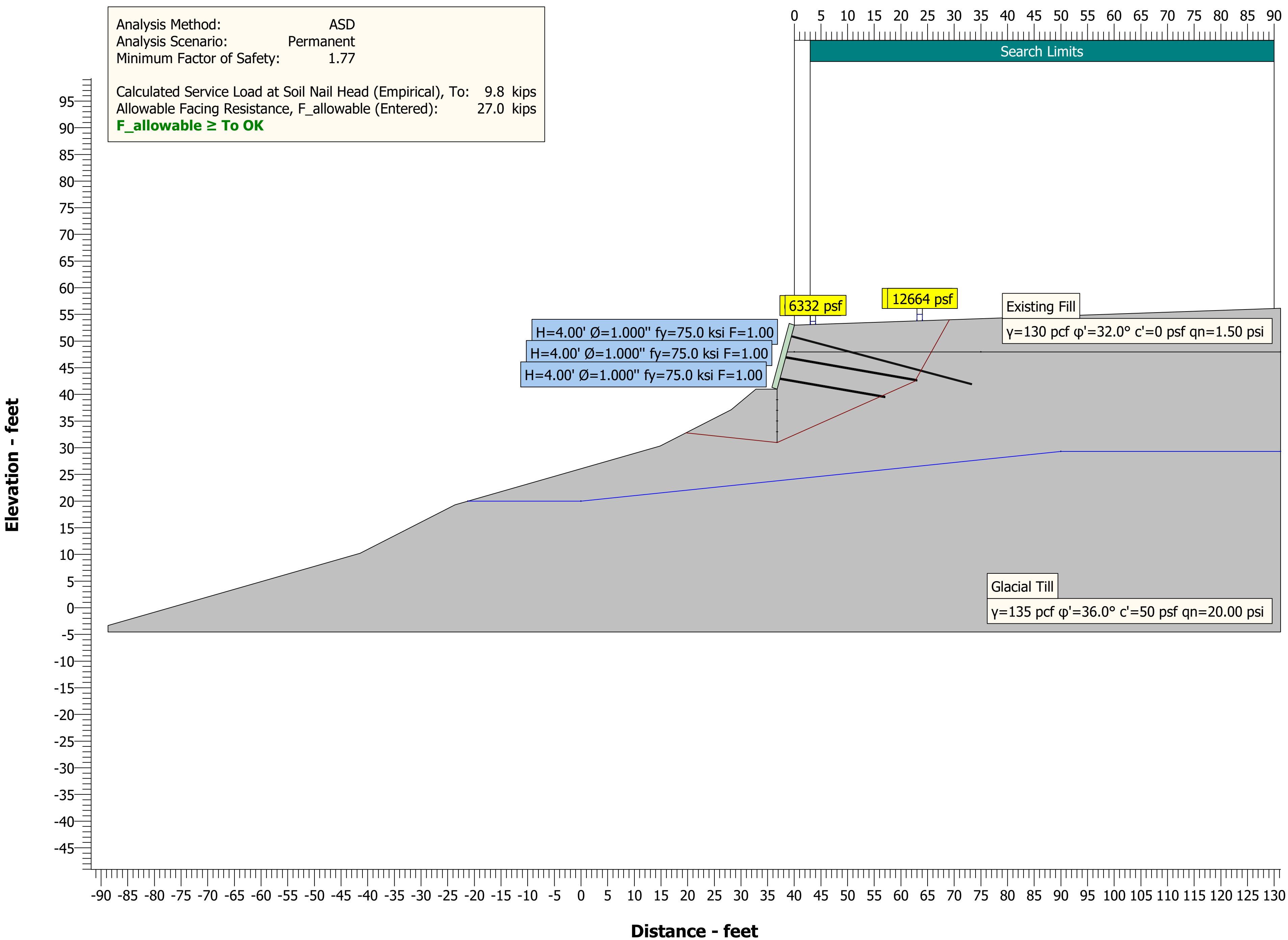
Table 3.1. ASTM F1554 Anchor Rod (rod only) Available Tensile Strength, kips

Rod Diameter, in.	Rod Area, A_b, in.^{2}	LRFD ϕR_n $\phi = 0.75$			ASD R_n/Ω $\Omega = 2.00$		
		Grade 36 kips	Grade 55 kips	Grade 105 kips	Grade 36 kips	Grade 55 kips	Grade 105 kips
⅝	0.307	10.0	12.9	21.6	6.68	8.63	14.4
¾	0.442	14.4	18.6	31.1	9.60	12.4	20.7
⅞	0.601	19.6	25.4	42.3	13.1	16.9	28.2
1	0.785	25.6	33.1	55.2	17.1	22.1	36.8
1⅛	0.994	32.4	41.9	69.9	21.6	28.0	46.6
1¼	1.23	40.0	51.8	86.3	26.7	34.5	57.5
1½	1.77	57.7	74.6	124	38.4	49.7	82.8
1¾	2.41	78.5	102	169	52.3	67.6	113
2	3.14	103	133	221	68.3	88.4	147
2¼	3.98	130	168	280	86.5	112	186
2½	4.91	160	207	345	107	138	230
2¾	5.94	194	251	418	129	167	278
3	7.07	231	298	497	154	199	331
3¼	8.30	271	350	583	180	233	389
3½	9.62	314	406	677	209	271	451
3¾	11.0	360	466	777	240	311	518
4	12.6	410	530	884	273	353	589

Unit Weight - soil	γ	130	pcf
Friction Angle - soil	ϕ	32	degrees
Top Depth - deadman	d_t	2	feet
Bottom Depth - deadman	d_b	6.5	feet
height - deadman	h_d	4.5	feet
(45-ϕ/2) in radians	--	0.5061	radians
(45+ϕ/2) in radians	--	1.0647	radians
Active Earth Pressure Coefficient	K_a	0.3073	--
Passive Earth Pressure Coefficient	K_p	3.2546	--
Vertical Stress (surface to top deadman)	σ_{vy}	260	psf
Vertical Stress (surface to bottom deadman)	σ_{vy}	845	psf
Active Lateral Earth Pressure (surface to top deadman)	P_{at}	79.9	psf
Active Lateral Earth Pressure (surface to bottom deadman)	P_{ab}	259.6	psf
Active Lateral Earth Pressure - deadman	P_A	763.9	lb/ft
Passive Lateral Earth Pressure (surface to top deadman)	P_{pt}	846.2	psf
Passive Lateral Earth Pressure (surface to bottom deadman)	P_{pb}	2750.1	psf
Passive Lateral Earth Pressure - deadman	P_P	8091.7	lb/ft
Ult. Resistance Capacity of Deadman - per foot	P_D	**7327.8**	**lb/ft**
Total Ult. Resistance Capacity of Deadman - for anchor spacing	P_{DT}	**58.62**	**kip**

Allowable Capacity Per Anchor	$P_{all\text{-}bar}$	**26.70**	**kip**
Allowable Load for Number of Anchors	P_{ALL}	**53.4**	**kip**
Resistance Capacity of Deadman - per foot	P_D	**7327.8**	**lb/ft**
Total Resistance Capacity of Deadman - for anchor spacing	P_{DT}	**58.6**	**kip**
Resulting Lateral Load on Wall - per foot	P_W	**1617.7**	**lb/ft**
Resulting Lateral Load on Wall - for anchor spacing	P_{WT}	**12.94**	**kip**
Is $P_{WT} < P_{DT}$?	**YES**		
is $P_{WT} < P_{ALL}$?	**YES**		

ATTACHMENT 5
SNAIL ANALYSIS OUTPUTS


Analysis Method: ASD
Analysis Scenario: Permanent
Minimum Factor of Safety: 1.77
Calculated Service Load at Soil Nail Head (Empirical), To: 9.8 kips
Allowable Facing Resistance, F_allowable (Entered): 27.0 kips
F_allowable ≥ To OK
Search Limits
6332 psf
12664 psf
Existing Fill
γ=130 pcf φ'=32.0° c'=0 psf qn=1.50 psi
H=4.00' Ø=1.000" fy=75.0 ksi F=1.00
H=4.00' Ø=1.000" fy=75.0 ksi F=1.00
H=4.00' Ø=1.000" fy=75.0 ksi F=1.00
Glacial Till
γ=135 pcf φ'=36.0° c'=50 psf qn=20.00 psi
Elevation - feet
Distance - feet

==

Snail

Version: 2.2.2

==

File Information

==

File Name: 324-891 Proposed Final Conditions.snz
Run Date: 02/21/24
Run Time: 15:24:43

==

Project Information

==

Description: Permanent Conditions
Location:
EA:
Project ID: 324-891
Wall No.:
Structure No.:
Station:
Engineer: GSS
Designer

Comments:

51 Harborview

==

Geometry

==

Layout:

Reference Point:

At: Top of Wall
Distance From Origin: 40.00 feet
Elevation Above Origin: 53.00 feet

Wall Dimensions:

Wall Height: 12.00 feet
Facing Angle: 75.00 degrees
Facing Batter: 3.215 :12 H:V

Ground Surface:

Number of lines that define the ground surface above the wall: 1

No.	Angle degrees	Distance feet
1	2	

Number of lines that define the ground surface in front of the toe: 6

No.	Angle degrees	Distance feet
1	0	4.00
2	-40	6.00
3	-27	15.00
4	-16	40.00
5	-27	20.00
6	-16	

Soil Layers:

Number of Layers: 2

Layers Below the Top Layer:

Coordinates of the Top of the Layer: feet

Layer	Point 1 Distance	Point 1 Elevation	Point 2 Distance	Point 2 Elevation

2	40.00	48.00	75.00	48.00

Ground Water:

Include Ground Water: Yes
Phreatic Correction: No
Number of Points: 2

No.	Distance feet	Elevation feet
1	0.00	20.00
2	90.00	29.33

Soil Nails

Dimensions and Properties:

Maximum Vertical Spacing: 4.00 feet
Number of Soil Nail Rows: 3
Soil Nail Design Parameters: Varying

No.	Soil Nail Length feet	Inclination From Horizontal degrees	Vertical Spacing feet	Horizontal Spacing H feet	Nail Bar Diameter Ø inches	Nail Bar Yield Strength fy ksi	Bond Strength Factor F
1	35.00	15	2.00	4.00	1.000	75.0	1.00
2	25.00	10	4.00	4.00	1.000	75.0	1.00
3	20.00	10	4.00	4.00	1.000	75.0	1.00

Facing Resistance:

	Temporary	Permanent	Seismic
ASD Allowable Facing Resistance:	23.0	27.0	36.8 kips

Soil Properties

Layer	Description	Unit Weight γ pcf	Friction Angle φ' degrees	Cohesion c' psf
1	Existing Fill	130	32.0	0
2	Glacial Till	135	36.0	50

Loads

Applied Loads:

Seismic:

Horizontal Seismic Coefficient Kh:

External Load:

Apply external load: No

Surcharges:

Apply surcharges: Yes

No.	Distance from Top of Wall Begin feet	Distance from Top of Wall End feet	Load Begin psf	Load End psf
1	23.00	24.00	12664	12664
2	3.00	4.00	6332	6332

Factors of Safety

	Temporary	Permanent	Seismic
Pullout (Distal):	2.00	2.00	1.50
Pullout (Proximal):	2.00	2.00	1.50
Nail Bar Yield:	1.80	1.80	1.35

Search Options

Search Limits:

Begin: 3.00 feet
End: 90.00 feet

Below Toe Searches (BTS):

Perform below Toe Search: Yes
Number of BTS Points: 5
BTS Depth: 10.00 feet
Interface Friction
Reduction Factor: 0.33

Advanced Search Options:

Use Advanced Search Options: No

Results

Analysis:

Method: ASD
Scenario: Permanent

Factor of Safety:

Minimum: 1.77
Found at Search Point: 4
Found at Grid Point: 42
Found at Search Level: 10.00 feet below the toe of the wall

Load at Soil Nail Head:

Calculated Service Load at Soil Nail Head (Empirical), To: 9.8 kips
Allowable Facing Resistance, F_allowable (Entered): 27.0 kips
F_allowable ≥ To OK

Nominal Pullout Resistance:

Layer	Description	Nominal Pullout Resistance klf
1	Existing Fill	0.226
2	Glacial Till	3.016

Results by Search Level:

** Indicates Minimum Factor of Safety

Search Level: At the toe of the wall Facing Design Force = 11.1 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	4.54	6.22	0.00	5.59	87.06	12.12	1	34.9	Facing
							2	39.7	Pullout
							3	28.4	Pullout
2	6.70	14.92	0.00	5.97	54.20	15.30	1	35.7	Facing
							2	33.0	Pullout
							3	26.5	Pullout
3	11.04	23.62	0.00	21.25	79.48	12.93	1	28.5	Pullout
							2	8.8	Pullout
							3	16.3	Pullout
4	3.07	32.32	2.88	25.89	61.11	13.38	1	18.8	Pullout
							2	0.3	Pullout
							3	21.4	Pullout
5	4.31	41.02	0.00	24.61	39.07	21.13	1	16.0	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
6	5.19	49.72	0.00	24.86	28.73	28.35	1	13.2	Pullout
							2	0.0	Pullout

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes Lower Angle degrees	Failure Planes Lower Length feet	Failure Planes Upper Angle degrees	Failure Planes Upper Length feet	Reinforcement Level	Stress ksi	Controlling Resistance Failure Mode
							3	16.3	Pullout
7	6.07	58.42	0.00	23.37	21.67	37.72	1	12.8	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
8	7.07	67.12	0.00	20.13	16.85	49.09	1	14.0	Pullout
							2	0.8	Pullout
							3	16.3	Pullout
9	7.22	75.82	0.00	22.74	15.32	55.03	1	10.4	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
10	7.43	84.52	0.00	25.35	14.08	60.99	1	7.1	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
11	8.37	93.22	0.00	18.64	11.48	76.09	1	11.3	Pullout
							2	0.0	Pullout
							3	16.3	Pullout

Search Level: 2.00 feet below the toe of the wall Facing Design Force = 9.6 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.60	6.22	26.76	6.26	86.85	11.30	1	34.9	Facing
							2	39.8	Pullout
							3	28.5	Pullout
2	3.99	14.92	0.00	5.97	58.16	16.96	1	35.8	Facing
							2	31.6	Pullout
							3	24.5	Pullout
3	5.59	23.62	11.73	21.71	77.09	10.57	1	29.2	Pullout
							2	9.6	Pullout
							3	18.7	Pullout
4	2.53	32.32	6.63	26.03	61.72	13.64	1	18.5	Pullout
							2	0.0	Pullout
							3	12.2	Pullout
5	3.23	41.02	3.56	24.66	40.04	21.43	1	15.4	Pullout
							2	0.0	Pullout
							3	6.0	Pullout
6	3.65	49.72	0.00	24.86	32.15	29.36	1	9.7	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
7	4.02	58.42	0.00	23.37	24.44	38.50	1	8.3	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
8	4.36	67.12	0.00	20.13	19.06	49.71	1	8.8	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
9	4.60	75.82	0.00	22.74	17.31	55.59	1	4.6	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	4.84	84.52	0.00	25.35	15.89	61.51	1	0.8	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
11	5.13	93.22	0.00	18.64	12.95	76.52	1	4.6	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

Search Level: 4.00 feet below the toe of the wall Facing Design Force = 9.6 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode

1	3.17	6.22	40.82	7.39	86.85	11.29	1	34.9	Facing
							2	39.8	Pullout
							3	28.5	Pullout
2	3.09	14.92	15.38	6.19	58.78	17.27	1	35.8	Facing
							2	31.4	Pullout
							3	24.2	Pullout
3	4.03	23.62	13.27	21.84	78.59	11.93	1	28.8	Pullout
							2	9.1	Pullout
							3	10.6	Pullout
4	2.20	32.32	11.17	26.35	61.52	13.55	1	18.6	Pullout
							2	0.1	Pullout
							3	7.7	Pullout
5	2.71	41.02	8.01	24.85	40.18	21.47	1	15.3	Pullout
							2	0.0	Pullout
							3	2.0	Pullout
6	3.04	49.72	8.07	25.11	29.56	28.58	1	12.3	Pullout
							2	0.0	Pullout
							3	2.1	Pullout
7	3.37	58.42	8.72	23.64	22.25	37.87	1	11.8	Pullout
							2	0.0	Pullout
							3	3.4	Pullout
8	3.64	67.12	5.17	20.22	19.25	49.76	1	8.4	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
9	3.88	75.82	4.66	22.82	17.45	55.63	1	4.2	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	4.07	84.52	6.36	17.01	14.08	69.71	1	7.1	Pullout
							2	0.0	Pullout
							3	0.9	Pullout
11	4.26	93.22	5.86	18.74	13.01	76.54	1	4.4	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

Search Level: 6.00 feet below the toe of the wall Facing Design Force = 9.8 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.05	6.22	44.16	7.80	87.19	12.69	1	34.9	Facing
							2	39.7	Pullout
							3	28.4	Pullout
2	2.64	14.92	17.15	6.24	61.62	18.83	1	35.8	Facing
							2	30.5	Pullout
							3	22.8	Pullout
3	3.04	23.62	19.40	22.53	78.12	11.47	1	28.9	Pullout
							2	9.3	Pullout
							3	9.6	Pullout
4	1.98	32.32	16.40	26.95	60.47	13.11	1	19.0	Pullout
							2	0.6	Pullout
							3	5.9	Pullout
5	2.38	41.02	13.25	25.28	39.50	21.26	1	15.7	Pullout
							2	0.0	Pullout
							3	1.1	Pullout
6	2.64	49.72	13.32	25.55	28.93	28.40	1	13.0	Pullout
							2	0.0	Pullout
							3	1.2	Pullout
7	2.88	58.42	0.00	5.84	20.76	56.22	1	14.4	Pullout
							2	0.2	Pullout
							3	3.3	Pullout
8	3.12	67.12	11.36	20.54	19.01	49.69	1	8.9	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes Lower Angle degrees	Failure Planes Lower Length feet	Failure Planes Upper Angle degrees	Failure Planes Upper Length feet	Reinforcement Level	Reinforcement Stress ksi	Reinforcement Controlling Resistance Failure Mode
9	3.34	75.82	10.24	23.11	17.20	55.56	1	4.9	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	3.50	84.52	13.85	87.05	0.00	0.00	1	7.9	Pullout
							2	0.0	Pullout
							3	2.1	Pullout
11	3.65	93.22	12.78	95.58	0.00	0.00	1	5.4	Pullout
							2	0.0	Pullout
							3	0.3	Pullout

Search Level: 8.00 feet below the toe of the wall Facing Design Force = 10.2 kips (Clouterre)

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes Lower Angle degrees	Failure Planes Lower Length feet	Failure Planes Upper Angle degrees	Failure Planes Upper Length feet	Reinforcement Level	Reinforcement Stress ksi	Reinforcement Controlling Resistance Failure Mode
1	2.88	6.22	55.18	9.80	87.05	12.08	1	34.9	Facing
							2	39.7	Pullout
							3	28.4	Pullout
2	2.39	14.92	34.38	7.23	61.27	18.62	1	35.8	Facing
							2	30.6	Pullout
							3	23.0	Pullout
3	2.50	23.62	25.98	23.64	77.16	10.62	1	29.1	Pullout
							2	9.6	Pullout
							3	9.8	Pullout
4	1.86	32.32	22.12	27.91	58.41	12.34	1	19.8	Pullout
							2	3.4	Pullout
							3	5.7	Pullout
5	2.17	41.02	19.11	26.04	37.94	20.80	1	16.8	Pullout
							2	0.3	Pullout
							3	1.8	Pullout
6	2.39	49.72	19.19	26.32	27.56	28.04	1	14.5	Pullout
							2	0.0	Pullout
							3	1.9	Pullout
7	2.62	58.42	0.00	5.84	22.64	56.96	1	11.1	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
8	2.77	67.12	18.33	70.70	0.00	0.00	1	10.4	Pullout
							2	0.0	Pullout
							3	0.7	Pullout
9	2.95	75.82	16.55	79.09	0.00	0.00	1	6.7	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	3.14	84.52	15.12	87.55	0.00	0.00	1	3.4	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
11	3.32	93.22	13.94	96.05	0.00	0.00	1	0.5	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

Search Level: 10.00 feet below the toe of the wall Facing Design Force = 9.8 kips (Clouterre)

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes Lower Angle degrees	Failure Planes Lower Length feet	Failure Planes Upper Angle degrees	Failure Planes Upper Length feet	Reinforcement Level	Reinforcement Stress ksi	Reinforcement Controlling Resistance Failure Mode
1	2.87	6.22	63.16	12.39	86.78	11.07	1	34.9	Facing
							2	39.8	Pullout
							3	28.5	Pullout
2	2.20	14.92	45.05	19.00	80.55	9.09	1	36.3	Facing
							2	24.8	Pullout
							3	19.4	Pullout
3	2.32	23.62	32.67	25.25	75.43	9.39	1	29.6	Pullout

							2	12.3	Pullout
							3	10.6	Pullout
** 4	1.77	32.32	24.00	28.30	60.68	13.20	1	18.9	Pullout
							2	0.5	Pullout
							3	1.5	Pullout
5	2.07	41.02	20.76	26.32	40.46	21.56	1	15.1	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
6	2.25	49.72	20.81	26.59	29.69	28.61	1	12.2	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
7	2.40	58.42	22.27	63.13	0.00	0.00	1	11.7	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
8	2.58	67.12	19.85	71.36	0.00	0.00	1	7.1	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
9	2.76	75.82	17.93	79.69	0.00	0.00	1	2.9	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	2.94	84.52	16.38	88.09	0.00	0.00	1	0.0	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
11	3.16	93.22	15.10	96.55	0.00	0.00	1	0.0	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

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END OF REPORT

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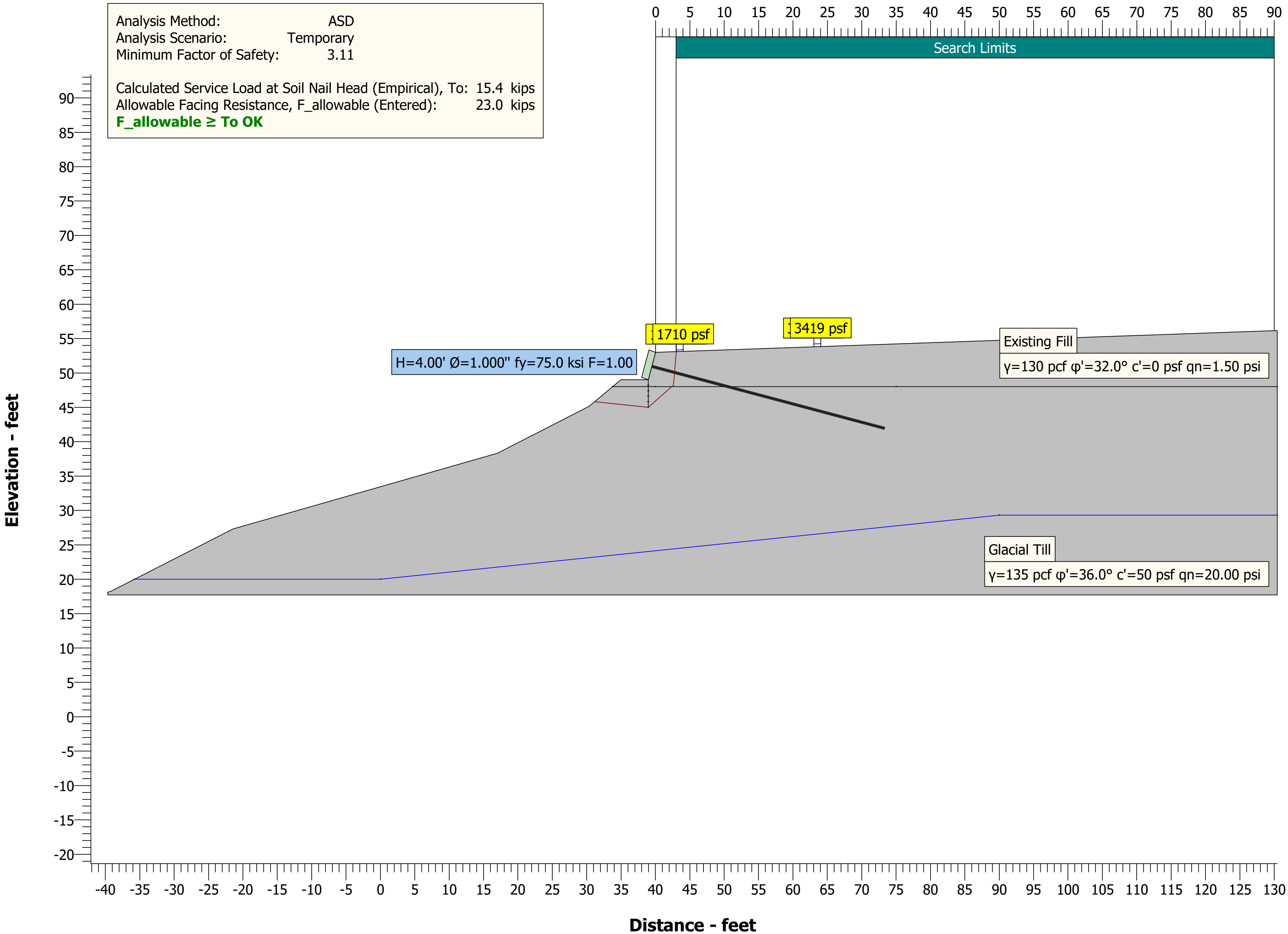


Analysis Method: ASD
Analysis Scenario: Temporary
Minimum Factor of Safety: 3.11
Calculated Service Load at Soil Nail Head (Empirical), To: 15.4 kips
Allowable Facing Resistance, F_allowable (Entered): 23.0 kips
F_allowable ≥ To OK
Search Limits
1710 psf
3419 psf
H=4.00' Ø=1.000" fy=75.0 ksi F=1.00
Existing Fill
γ=130 pcf φ'=32.0° c'=0 psf qn=1.50 psi
Glacial Till
γ=135 pcf φ'=36.0° c'=50 psf qn=20.00 psi
Elevation - feet
Distance - feet

==
Snail

Version: 2.2.2

==
File Information
==

File Name: 324-891 Stage No. 1 - Excavation.snz
Run Date: 02/21/24
Run Time: 15:45:28

==
Project Information
==

Description: Temporary Conditions
Location:
EA:
Project ID: 324-891
Wall No.:
Structure No.:
Station:
Engineer: GSS
Designer

Comments:

51 Harborview

==
Geometry
==

Layout:

Reference Point:

At: Top of Wall
Distance From Origin: 40.00 feet
Elevation Above Origin: 53.00 feet

Wall Dimensions:

Wall Height: 4.00 feet
Facing Angle: 75.00 degrees
Facing Batter: 3.215 :12 H:V

Ground Surface:

Number of lines that define the ground surface above the wall: 1

No.	Angle degrees	Distance feet
1	2	

Number of lines that define the ground surface in front of the toe: 6

No.	Angle degrees	Distance feet
1	0	4.00
2	-40	6.00
3	-27	15.00
4	-16	40.00
5	-27	20.00
6	-16	

Soil Layers:

Number of Layers: 2

Layers Below the Top Layer:

Coordinates of the Top of the Layer: feet

Layer	Point 1 Distance	Point 1 Elevation	Point 2 Distance	Point 2 Elevation

2	40.00	48.00	75.00	48.00

Ground Water:

Include Ground Water: Yes
Phreatic Correction: No
Number of Points: 2

No.	Distance feet	Elevation feet
1	0.00	20.00
2	90.00	29.33

Soil Nails

Dimensions and Properties:

Maximum Vertical Spacing: 4.00 feet
Number of Soil Nail Rows: 1
Soil Nail Design Parameters: Varying

No.	Soil Nail Length feet	Inclination From Horizontal degrees	Vertical Spacing feet	Horizontal Spacing H feet	Nail Bar Diameter Ø inches	Nail Bar Yield Strength fy ksi	Bond Strength Factor F
1	35.00	15	2.00	4.00	1.000	75.0	1.00

Facing Resistance:

	Temporary	Permanent	Seismic
ASD Allowable Facing Resistance:	23.0	27.0	36.8 kips

Soil Properties

Layer	Description	Unit Weight γ pcf	Friction Angle φ' degrees	Cohesion c' psf
1	Existing Fill	130	32.0	0
2	Glacial Till	135	36.0	50

Loads

Applied Loads:

Seismic:

Horizontal Seismic Coefficient Kh:

External Load:

Apply external load: No

Surcharges:

Apply surcharges: Yes

No.	Distance from Top of Wall Begin feet	End feet	Load Begin psf	Load End psf
1	23.00	24.00	3419	3419
2	3.00	4.00	1710	1710

Factors of Safety

	Temporary	Permanent	Seismic
Pullout (Distal):	2.00	2.00	1.50
Pullout (Proximal):	2.00	2.00	1.50
Nail Bar Yield:	1.80	1.80	1.35

Search Options

Search Limits:

Begin: 3.00 feet
End: 90.00 feet

Below Toe Searches (BTS):

Perform below Toe Search: Yes
Number of BTS Points: 5
BTS Depth: 4.00 feet
Interface Friction
Reduction Factor: 0.33

Advanced Search Options:

Use Advanced Search Options: No

Results

Analysis:

Method: ASD
Scenario: Temporary

Factor of Safety:

Minimum: 3.11
Found at Search Point: 1
Found at Grid Point: 38
Found at Search Level: 4.00 feet below the toe of the wall

Load at Soil Nail Head:

Calculated Service Load at Soil Nail Head (Empirical), To: 15.4 kips
Allowable Facing Resistance, F_allowable (Entered): 23.0 kips
F_allowable ≥ To OK

Nominal Pullout Resistance:

Layer	Description	Nominal Pullout Resistance klf
1	Existing Fill	0.226
2	Glacial Till	3.016

Results by Search Level:

** Indicates Minimum Factor of Safety

Search Level: At the toe of the wall Facing Design Force = 15.4 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.82	4.07	6.39	3.69	83.71	3.72	1	29.8	Facing
2	N/A	12.77	0.00	1.28	20.98	12.31	1	29.8	Facing
3	N/A	21.47	0.00	2.15	13.70	19.89	1	30.0	Facing
4	N/A	30.17	0.00	3.02	10.47	27.61	1	30.1	Facing
5	N/A	38.87	0.00	3.89	8.65	35.39	1	30.2	Facing
6	N/A	47.57	0.00	4.76	7.48	43.18	1	30.2	Facing
7	N/A	56.27	0.00	5.63	6.68	50.99	1	30.3	Facing
8	N/A	64.97	0.00	6.50	6.08	58.81	1	30.3	Facing
9	N/A	73.67	0.00	7.37	5.63	66.63	1	30.4	Facing
10	N/A	82.37	0.00	8.24	5.27	74.45	1	30.4	Facing
11	N/A	91.07	0.00	9.11	4.98	82.28	1	30.4	Facing

Search Level: 0.80 feet below the toe of the wall Facing Design Force = 15.4 kips (Clouterre)

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes: Lower Angle degrees	Lower Length feet	Upper Angle degrees	Upper Length feet	Reinforcement: Level	Stress ksi	Controlling Resistance Failure Mode
1	3.40	4.07	14.99	3.79	84.08	3.94	1	29.8	Facing
2	N/A	12.77	0.00	1.28	24.38	12.62	1	29.9	Facing
3	N/A	21.47	0.00	2.15	15.92	20.10	1	30.2	Facing
4	N/A	30.17	0.00	3.02	12.09	27.77	1	30.3	Facing
5	N/A	38.87	0.00	3.89	9.92	35.52	1	30.4	Facing
6	N/A	47.57	0.00	4.76	8.53	43.29	1	30.5	Facing
7	N/A	56.27	0.00	5.63	7.57	51.09	1	30.6	Facing
8	N/A	64.97	0.00	6.50	6.86	58.90	1	30.6	Facing
9	N/A	73.67	0.00	7.37	6.31	66.71	1	30.7	Facing
10	N/A	82.37	0.00	8.24	5.88	74.53	1	30.7	Facing
11	N/A	91.07	0.00	9.11	5.54	82.35	1	30.8	Facing

Search Level: 1.60 feet below the toe of the wall Facing Design Force = 15.6 kips (Clouterre)

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes: Lower Angle degrees	Lower Length feet	Upper Angle degrees	Upper Length feet	Reinforcement: Level	Stress ksi	Controlling Resistance Failure Mode
1	3.33	4.07	25.03	4.04	84.18	4.01	1	29.8	Facing
2	8.82	12.77	2.99	11.51	76.71	5.56	1	30.9	Facing
3	N/A	21.47	0.00	2.15	18.09	20.33	1	30.3	Facing
4	N/A	30.17	0.00	3.02	13.69	27.95	1	30.5	Facing
5	N/A	38.87	0.00	3.89	11.19	35.66	1	30.6	Facing
6	N/A	47.57	0.00	4.76	9.58	43.42	1	30.8	Facing
7	N/A	56.27	0.00	5.63	8.45	51.20	1	30.8	Facing
8	N/A	64.97	0.00	6.50	7.63	59.00	1	30.9	Facing
9	N/A	73.67	0.00	7.37	7.00	66.80	1	31.3	Facing
10	N/A	82.37	0.00	8.24	6.49	74.61	1	32.0	Facing
11	N/A	91.07	0.00	9.11	6.09	82.43	1	32.7	Facing

Search Level: 2.40 feet below the toe of the wall Facing Design Force = 15.7 kips (Clouterre)

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes: Lower Angle degrees	Lower Length feet	Upper Angle degrees	Upper Length feet	Reinforcement: Level	Stress ksi	Controlling Resistance Failure Mode
1	3.25	4.07	35.38	4.49	84.05	3.92	1	29.8	Facing
2	7.11	12.77	6.76	11.57	76.80	5.59	1	30.9	Facing
3	N/A	21.47	0.00	2.15	20.21	20.59	1	30.5	Facing
4	43.91	30.17	0.00	3.02	15.28	28.15	1	30.7	Facing
5	N/A	38.87	0.00	3.89	12.44	35.83	1	30.9	Facing
6	N/A	47.57	0.00	4.76	10.61	43.56	1	31.4	Facing
7	N/A	56.27	0.00	5.63	9.34	51.32	1	32.8	Facing
8	N/A	64.97	0.00	6.50	8.40	59.11	1	34.0	Facing

9	N/A	73.67	0.00	7.37	7.68	66.90	1	35.0	Facing
10	N/A	82.37	0.00	8.24	7.10	74.71	1	35.8	Facing
11	N/A	91.07	0.00	9.11	6.64	82.52	1	36.6	Facing

Search Level: 3.20 feet below the toe of the wall Facing Design Force = 15.4 kips (Clouterre)

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes: Lower Angle degrees	Lower Length feet	Upper Angle degrees	Upper Length feet	Reinforcement: Level	Stress ksi	Controlling Resistance Failure Mode
1	3.16	4.07	38.57	4.69	84.69	4.40	1	29.8	Facing
2	6.08	12.77	7.54	11.59	78.15	6.22	1	30.9	Facing
3	11.21	21.47	0.00	19.32	74.82	8.20	1	29.8	Pullout
4	13.10	30.17	0.00	24.14	53.71	10.19	1	28.6	Pullout
5	32.57	38.87	0.00	3.89	13.69	36.01	1	32.1	Facing
6	81.90	47.57	0.00	4.76	11.65	43.71	1	34.2	Facing
7	174.68	56.27	0.00	5.63	10.22	51.46	1	35.8	Facing
8	118.52	64.97	0.00	6.50	9.16	59.23	1	37.2	Facing
9	132.79	73.67	0.00	66.30	52.88	12.21	1	28.6	Pullout
10	89.62	82.37	0.00	74.13	50.63	12.99	1	28.6	Pullout
11	70.19	91.07	0.00	81.96	48.64	13.78	1	28.6	Pullout

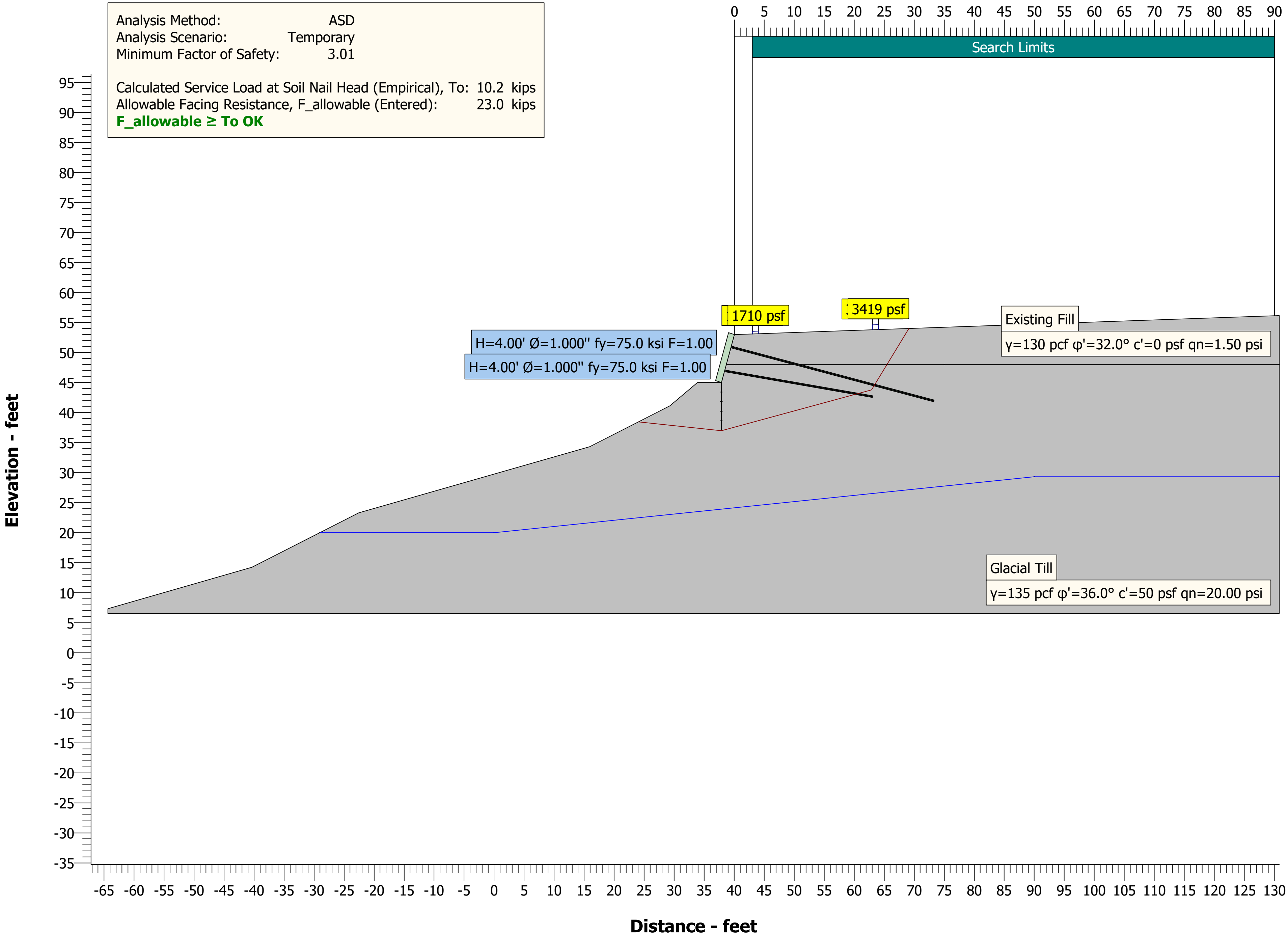
Search Level: 4.00 feet below the toe of the wall Facing Design Force = 15.4 kips (Clouterre)

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes: Lower Angle degrees	Lower Length feet	Upper Angle degrees	Upper Length feet	Reinforcement: Level	Stress ksi	Controlling Resistance Failure Mode
** 1	3.11	4.07	41.50	4.89	85.21	4.88	1	29.8	Facing
2	5.12	12.77	12.38	11.77	77.76	6.02	1	30.9	Facing
3	9.12	21.47	2.58	19.34	74.69	8.13	1	29.8	Pullout
4	7.46	30.17	0.00	21.12	44.89	12.78	1	25.5	Pullout
5	7.49	38.87	2.75	19.46	23.34	21.17	1	29.6	Pullout
6	15.94	47.57	0.00	23.79	22.03	25.66	1	22.7	Pullout
7	16.14	56.27	0.00	22.51	16.39	35.19	1	23.1	Pullout
8	22.64	64.97	0.00	19.49	12.68	46.62	1	25.8	Pullout
9	19.39	73.67	0.00	22.10	11.55	52.64	1	23.4	Pullout
10	24.86	82.37	0.00	24.71	10.65	58.67	1	22.7	Pullout
11	26.10	91.07	0.00	18.21	8.70	73.70	1	26.1	Pullout

END OF REPORT

Analysis Method: ASD
Analysis Scenario: Temporary
Minimum Factor of Safety: 3.01

Calculated Service Load at Soil Nail Head (Empirical), To: 10.2 kips
Allowable Facing Resistance, F_allowable (Entered): 23.0 kips
F_allowable ≥ To OK



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Snail

Version: 2.2.2

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File Information
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File Name: 324-891 Stage No. 2 - Excavation.snz
Run Date: 02/21/24
Run Time: 15:52:03

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Project Information
==

Description: Temporary Conditions
Location:
EA:
Project ID: 324-891
Wall No.:
Structure No.:
Station:
Engineer: GSS
Designer

Comments:

51 Harborview

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Geometry
==

Layout:

Reference Point:

At: Top of Wall
Distance From Origin: 40.00 feet
Elevation Above Origin: 53.00 feet

Wall Dimensions:

Wall Height: 8.00 feet
Facing Angle: 75.00 degrees
Facing Batter: 3.215 :12 H:V

Ground Surface:

Number of lines that define the ground surface above the wall: 1

No.	Angle degrees	Distance feet
1	2	

Number of lines that define the ground surface in front of the toe: 6

No.	Angle degrees	Distance feet
1	0	4.00
2	-40	6.00
3	-27	15.00
4	-16	40.00
5	-27	20.00
6	-16	

Soil Layers:

Number of Layers: 2

Layers Below the Top Layer:

Coordinates of the Top of the Layer: feet

Layer	Point 1 Distance	Point 1 Elevation	Point 2 Distance	Point 2 Elevation

2	40.00	48.00	75.00	48.00

Ground Water:

Include Ground Water: Yes
Phreatic Correction: No
Number of Points: 2

No.	Distance feet	Elevation feet
1	0.00	20.00
2	90.00	29.33

Soil Nails

Dimensions and Properties:

Maximum Vertical Spacing: 4.00 feet
Number of Soil Nail Rows: 2
Soil Nail Design Parameters: Varying

No.	Soil Nail Length feet	Inclination From Horizontal degrees	Vertical Spacing feet	Horizontal Spacing H feet	Nail Bar Diameter Ø inches	Nail Bar Yield Strength fy ksi	Bond Strength Factor F
1	35.00	15	2.00	4.00	1.000	75.0	1.00
2	25.00	10	4.00	4.00	1.000	75.0	1.00

Facing Resistance:

	Temporary	Permanent	Seismic
ASD Allowable Facing Resistance:	23.0	27.0	36.8 kips

Soil Properties

Layer	Description	Unit Weight γ pcf	Friction Angle φ' degrees	Cohesion c' psf
1	Existing Fill	130	32.0	0
2	Glacial Till	135	36.0	50

Loads

Applied Loads:

Seismic:

Horizontal Seismic Coefficient Kh:

External Load:

Apply external load: No

Surcharges:

Apply surcharges: Yes

No.	Distance from Top of Wall Begin feet	Distance from Top of Wall End feet	Load Begin psf	Load End psf
1	23.00	24.00	3419	3419
2	3.00	4.00	1710	1710

Factors of Safety

	Temporary	Permanent	Seismic
Pullout (Distal):	2.00	2.00	1.50
Pullout (Proximal):	2.00	2.00	1.50
Nail Bar Yield:	1.80	1.80	1.35

Search Options

Search Limits:

Begin: 3.00 feet
End: 90.00 feet

Below Toe Searches (BTS):

Perform below Toe Search: Yes
Number of BTS Points: 5
BTS Depth: 8.00 feet
Interface Friction
Reduction Factor: 0.33

Advanced Search Options:

Use Advanced Search Options: No

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Results

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Analysis:

Method: ASD
Scenario: Temporary

Factor of Safety:

Minimum: 3.01
Found at Search Point: 4
Found at Grid Point: 37
Found at Search Level: 8.00 feet below the toe of the wall

Load at Soil Nail Head:

Calculated Service Load at Soil Nail Head (Empirical), To: 10.2 kips
Allowable Facing Resistance, F_allowable (Entered): 23.0 kips
F_allowable ≥ To OK

Nominal Pullout Resistance:

Layer	Description	Nominal Pullout Resistance klf
1	Existing Fill	0.226
2	Glacial Till	3.016

Results by Search Level:

** Indicates Minimum Factor of Safety

Search Level: At the toe of the wall Facing Design Force = 18.1 kips (Clouterre)

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes: Lower Angle degrees	Lower Length feet	Upper Angle degrees	Upper Length feet	Reinforcement: Level	Stress ksi	Controlling Resistance Failure Mode
1	4.28	5.14	9.93	4.70	85.97	7.31	1	29.8	Facing
							2	37.3	Facing
2	26.06	13.84	0.00	12.46	80.65	8.52	1	31.2	Facing
							2	25.9	Pullout
3	598.81	22.54	0.00	2.25	23.24	22.08	1	30.6	Facing
							2	38.1	Facing
4	59.41	31.24	0.00	3.12	17.78	29.53	1	30.9	Facing
							2	36.9	Pullout
5	N/A	39.94	0.00	3.99	14.53	37.14	1	33.7	Facing
							2	35.0	Pullout
6	N/A	48.64	0.00	4.86	12.40	44.82	1	36.1	Facing
							2	33.5	Pullout
7	N/A	57.34	0.00	5.73	10.89	52.56	1	37.9	Pullout
							2	32.1	Pullout
8	N/A	66.04	0.00	6.60	9.77	60.31	1	36.2	Pullout
							2	31.0	Pullout

9	N/A	74.74	0.00	7.47	8.90	68.09	1	34.8	Pullout
							2	29.9	Pullout
10	N/A	83.44	0.00	8.34	8.21	75.88	1	33.6	Pullout
							2	29.0	Pullout
11	N/A	92.14	0.00	9.21	7.65	83.67	1	32.5	Pullout
							2	28.1	Pullout

Search Level: 1.60 feet below the toe of the wall Facing Design Force = 19.3 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.83	5.14	22.75	5.02	86.21	7.78	1	29.8	Facing
							2	37.4	Facing
2	6.47	13.84	4.59	12.50	81.26	9.11	1	31.4	Facing
							2	24.6	Pullout
3	8.58	22.54	0.00	20.29	77.67	10.56	1	29.0	Pullout
							2	9.4	Pullout
4	5.69	31.24	0.00	24.99	59.52	12.32	1	19.4	Pullout
							2	8.2	Pullout
5	8.46	39.94	0.00	23.97	34.35	19.35	1	19.4	Pullout
							2	8.2	Pullout
6	9.83	48.64	0.00	24.32	24.77	26.79	1	17.9	Pullout
							2	8.2	Pullout
7	11.50	57.34	0.00	22.94	18.52	36.29	1	18.6	Pullout
							2	8.2	Pullout
8	12.65	66.04	0.00	33.02	19.71	35.08	1	10.8	Pullout
							2	8.2	Pullout
9	12.18	74.74	0.00	29.90	15.14	46.46	1	10.9	Pullout
							2	8.2	Pullout
10	12.31	83.44	0.00	25.03	12.02	59.72	1	15.2	Pullout
							2	8.2	Pullout
11	13.54	92.14	0.00	36.86	12.98	56.74	1	10.8	Pullout
							2	8.2	Pullout

Search Level: 3.20 feet below the toe of the wall Facing Design Force = 19.3 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.43	5.14	36.23	5.74	86.28	7.93	1	29.8	Facing
							2	37.4	Facing
2	5.05	13.84	10.56	12.67	81.52	9.39	1	31.4	Facing
							2	24.5	Pullout
3	6.62	22.54	3.36	20.32	78.13	10.96	1	28.9	Pullout
							2	9.3	Pullout
4	4.37	31.24	2.80	25.02	60.39	12.65	1	19.0	Pullout
							2	3.2	Pullout
5	5.24	39.94	0.00	23.97	38.08	20.30	1	16.7	Pullout
							2	0.1	Pullout
6	5.66	48.64	0.00	24.32	27.80	27.50	1	14.3	Pullout
							2	0.0	Pullout
7	6.13	57.34	0.00	22.94	20.88	36.83	1	14.1	Pullout
							2	0.0	Pullout
8	6.57	66.04	0.00	33.02	22.13	35.65	1	2.5	Pullout
							2	0.0	Pullout

9	6.77	74.74	0.00	22.42	14.71	54.09	1	12.3	Pullout
							2	0.0	Pullout
10	6.90	83.44	0.00	25.03	13.51	60.07	1	9.2	Pullout
							2	0.0	Pullout
11	7.24	92.14	0.00	36.86	14.54	57.12	1	0.0	Pullout
							2	0.0	Pullout

Search Level: 4.80 feet below the toe of the wall Facing Design Force = 19.3 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.17	5.14	48.11	6.93	86.20	7.76	1	29.8	Facing
							2	37.4	Facing
2	4.00	13.84	17.64	13.07	81.49	9.35	1	31.4	Facing
							2	24.5	Pullout
3	5.19	22.54	7.59	20.47	78.22	11.04	1	28.9	Pullout
							2	9.2	Pullout
4	3.73	31.24	6.31	25.15	60.52	12.70	1	19.0	Pullout
							2	2.2	Pullout
5	4.38	39.94	3.37	24.01	38.50	20.41	1	16.4	Pullout
							2	0.0	Pullout
6	4.69	48.64	3.39	24.36	28.09	27.57	1	13.9	Pullout
							2	0.0	Pullout
7	5.01	57.34	3.67	22.98	21.07	36.87	1	13.8	Pullout
							2	0.0	Pullout
8	5.43	66.04	0.00	19.81	18.01	48.61	1	11.2	Pullout
							2	0.0	Pullout
9	5.55	74.74	3.91	22.48	14.78	54.11	1	12.0	Pullout
							2	0.0	Pullout
10	5.73	83.44	3.57	25.08	13.55	60.08	1	9.1	Pullout
							2	0.0	Pullout
11	6.10	92.14	0.00	18.43	12.20	75.42	1	7.9	Pullout
							2	0.0	Pullout

Search Level: 6.40 feet below the toe of the wall Facing Design Force = 19.3 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.08	5.14	57.45	8.60	85.94	7.27	1	29.8	Facing
							2	37.3	Facing
2	3.28	13.84	25.43	13.80	81.15	8.99	1	31.3	Facing
							2	24.6	Pullout
3	4.15	22.54	12.60	20.79	77.97	10.82	1	28.9	Pullout
							2	9.3	Pullout
4	3.31	31.24	10.48	25.42	59.93	12.47	1	19.2	Pullout
							2	3.2	Pullout
5	3.76	39.94	7.47	24.17	38.21	20.33	1	16.6	Pullout
							2	0.0	Pullout
6	4.00	48.64	7.51	24.53	27.79	27.49	1	14.3	Pullout
							2	0.0	Pullout
7	4.26	57.34	8.10	23.17	20.79	36.80	1	14.3	Pullout
							2	0.2	Pullout
8	4.58	66.04	4.80	19.88	17.94	48.59	1	11.3	Pullout
							2	0.0	Pullout

9	4.72	74.74	8.59	22.68	14.52	54.05	1	12.9	Pullout
							2	0.2	Pullout
10	4.92	83.44	7.84	25.27	13.28	60.02	1	10.1	Pullout
							2	0.0	Pullout
11	5.18	92.14	5.44	18.51	12.09	75.39	1	8.5	Pullout
							2	0.0	Pullout

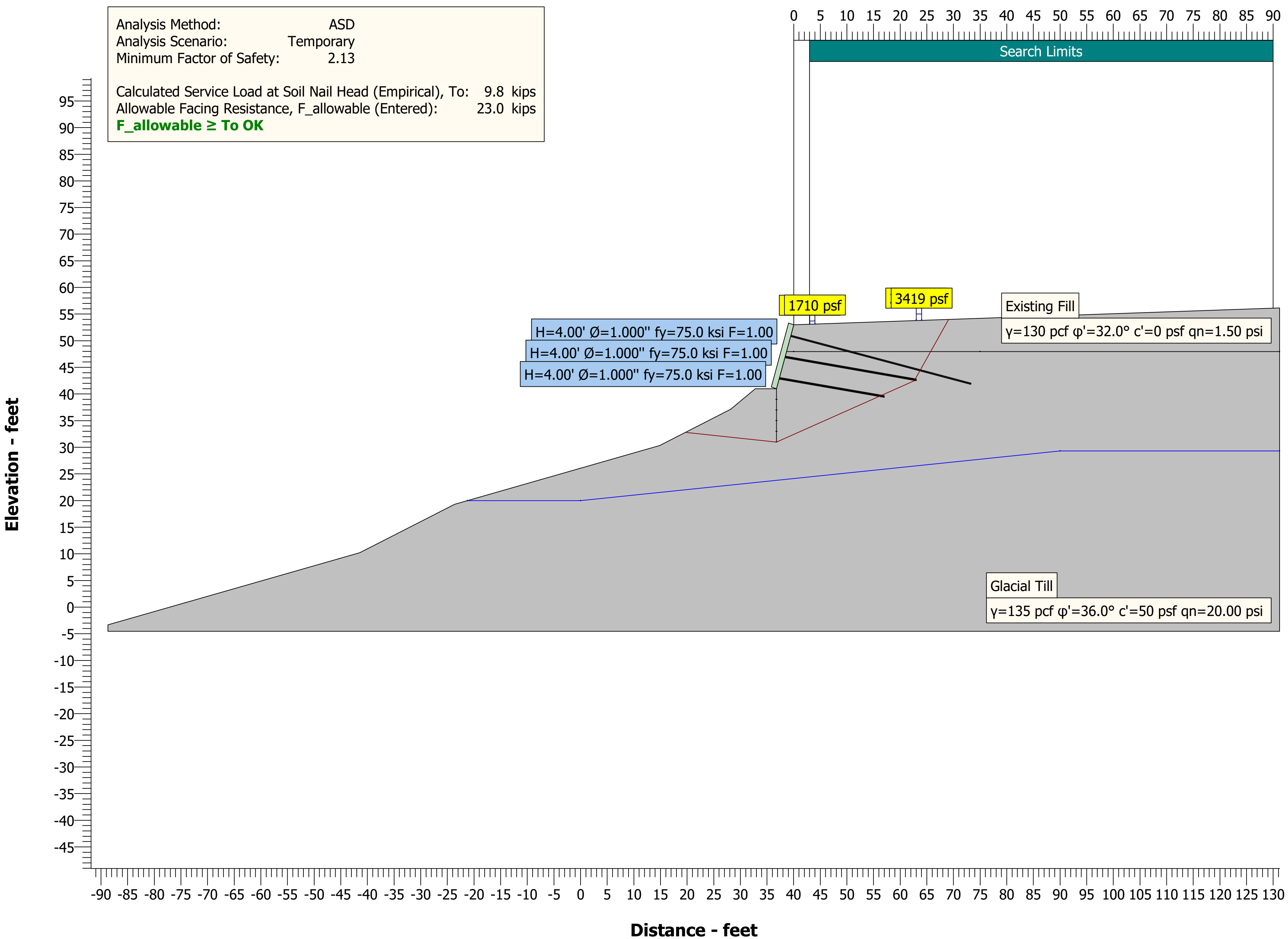
Search Level: 8.00 feet below the toe of the wall Facing Design Force = 10.2 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.06	5.14	60.11	9.29	86.35	8.07	1	29.8	Facing
							2	37.4	Facing
2	3.09	13.84	27.78	14.08	82.00	9.94	1	31.5	Facing
							2	24.4	Pullout
3	3.61	22.54	20.34	19.23	65.79	10.99	1	32.2	Pullout
							2	13.7	Pullout
** 4	3.01	31.24	15.23	25.91	58.53	11.97	1	19.7	Pullout
							2	5.2	Pullout
5	3.33	39.94	12.23	24.52	37.19	20.06	1	17.3	Pullout
							2	0.9	Pullout
6	3.52	48.64	12.26	24.89	26.90	27.27	1	15.3	Pullout
							2	0.1	Pullout
7	3.81	57.34	13.20	23.56	20.04	36.62	1	15.7	Pullout
							2	1.9	Pullout
8	3.97	66.04	10.43	20.15	17.51	48.48	1	12.4	Pullout
							2	0.0	Pullout
9	4.17	74.74	9.39	22.73	15.82	54.38	1	8.8	Pullout
							2	0.0	Pullout
10	4.33	83.44	12.72	85.54	0.00	0.00	1	12.3	Pullout
							2	0.1	Pullout
11	4.51	92.14	11.74	94.11	0.00	0.00	1	10.1	Pullout
							2	0.0	Pullout

END OF REPORT

Analysis Method:	ASD
Analysis Scenario:	Temporary
Minimum Factor of Safety:	2.13

Calculated Service Load at Soil Nail Head (Empirical), To: 9.8 kips
Allowable Facing Resistance, F_allowable (Entered): 23.0 kips
F_allowable ≥ To OK



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Snail

Version: 2.2.2

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File Information

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File Name: 324-891 Proposed Final Temporary Conditions.snz
Run Date: 02/21/24
Run Time: 15:57:16

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Project Information

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Description: Temporary Conditions
Location:
EA:
Project ID: 324-891
Wall No.:
Structure No.:
Station:
Engineer: GSS
Designer

Comments:

51 Harborview

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Geometry

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Layout:

Reference Point:

At: Top of Wall
Distance From Origin: 40.00 feet
Elevation Above Origin: 53.00 feet

Wall Dimensions:

Wall Height: 12.00 feet
Facing Angle: 75.00 degrees
Facing Batter: 3.215 :12 H:V

Ground Surface:

Number of lines that define the ground surface above the wall: 1

No.	Angle degrees	Distance feet
1	2	

Number of lines that define the ground surface in front of the toe: 6

No.	Angle degrees	Distance feet
1	0	4.00
2	-40	6.00
3	-27	15.00
4	-16	40.00
5	-27	20.00
6	-16	

Soil Layers:

Number of Layers: 2

Layers Below the Top Layer:

Coordinates of the Top of the Layer: feet

Layer	Point 1 Distance	Point 1 Elevation	Point 2 Distance	Point 2 Elevation

2	40.00	48.00	75.00	48.00

Ground Water:

Include Ground Water: Yes
Phreatic Correction: No
Number of Points: 2

No.	Distance feet	Elevation feet
1	0.00	20.00
2	90.00	29.33

Soil Nails

Dimensions and Properties:

Maximum Vertical Spacing: 4.00 feet
Number of Soil Nail Rows: 3
Soil Nail Design Parameters: Varying

No.	Soil Nail Length feet	Inclination From Horizontal degrees	Vertical Spacing feet	Horizontal Spacing H feet	Nail Bar Diameter Ø inches	Nail Bar Yield Strength fy ksi	Bond Strength Factor F
1	35.00	15	2.00	4.00	1.000	75.0	1.00
2	25.00	10	4.00	4.00	1.000	75.0	1.00
3	20.00	10	4.00	4.00	1.000	75.0	1.00

Facing Resistance:

	Temporary	Permanent	Seismic	
ASD Allowable Facing Resistance:	23.0	27.0	36.8	kips

Soil Properties

Layer	Description	Unit Weight γ pcf	Friction Angle φ' degrees	Cohesion c' psf
1	Existing Fill	130	32.0	0
2	Glacial Till	135	36.0	50

Loads

Applied Loads:

Seismic:

Horizontal Seismic Coefficient Kh:

External Load:

Apply external load: No

Surcharges:

Apply surcharges: Yes

No.	Distance from Top of Wall Begin feet	End feet	Load Begin psf	Load End psf
1	23.00	24.00	3419	3419
2	3.00	4.00	1710	1710

Factors of Safety

	Temporary	Permanent	Seismic
Pullout (Distal):	2.00	2.00	1.50
Pullout (Proximal):	2.00	2.00	1.50
Nail Bar Yield:	1.80	1.80	1.35

Search Options

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Search Limits:

Begin: 3.00 feet
End: 90.00 feet

Below Toe Searches (BTS):

Perform below Toe Search: Yes
Number of BTS Points: 5
BTS Depth: 10.00 feet
Interface Friction
Reduction Factor: 0.33

Advanced Search Options:

Use Advanced Search Options: No

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Results

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Analysis:

Method: ASD
Scenario: Temporary

Factor of Safety:

Minimum: 2.13
Found at Search Point: 4
Found at Grid Point: 42
Found at Search Level: 10.00 feet below the toe of the wall

Load at Soil Nail Head:

Calculated Service Load at Soil Nail Head (Empirical), To: 9.8 kips
Allowable Facing Resistance, F_allowable (Entered): 23.0 kips
F_allowable ≥ To OK

Nominal Pullout Resistance:

Layer	Description	Nominal Pullout Resistance klf
1	Existing Fill	0.226
2	Glacial Till	3.016

Results by Search Level:

** Indicates Minimum Factor of Safety

Search Level: At the toe of the wall Facing Design Force = 19.0 kips (Clouterre)

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Failure Planes: Lower Angle degrees	Lower Length feet	Upper Angle degrees	Upper Length feet	Reinforcement: Level	Stress ksi	Controlling Resistance Failure Mode
1	4.60	6.22	13.68	5.12	83.49	10.96	1	29.8	Facing
							2	36.8	Facing
							3	29.7	Pullout
2	8.91	14.92	0.00	13.42	83.15	12.50	1	31.7	Facing
							2	24.1	Pullout
							3	16.3	Pullout
3	10.54	23.62	0.00	21.25	79.48	12.93	1	28.5	Pullout
							2	8.8	Pullout
							3	16.3	Pullout
4	5.42	32.32	0.00	25.85	63.59	14.53	1	17.8	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
5	6.51	41.02	0.00	24.61	39.07	21.13	1	16.0	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
6	7.00	49.72	0.00	24.86	28.73	28.35	1	13.2	Pullout
							2	0.0	Pullout

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Lower Angle degrees	Lower Length feet	Upper Angle degrees	Upper Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
							3	16.3	Pullout
7	7.55	58.42	0.00	23.37	21.67	37.72	1	12.8	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
8	7.96	67.12	0.00	33.56	22.98	36.45	1	0.9	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
9	8.18	75.82	0.00	22.74	15.32	55.03	1	10.4	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
10	8.19	84.52	0.00	25.35	14.08	60.99	1	7.1	Pullout
							2	0.0	Pullout
							3	16.3	Pullout
11	8.82	93.22	0.00	37.29	15.15	57.94	1	0.0	Pullout
							2	0.0	Pullout
							3	16.3	Pullout

Search Level: 2.00 feet below the toe of the wall Facing Design Force = 19.4 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.60	6.22	26.76	6.26	86.85	11.30	1	29.8	Facing
							2	37.5	Facing
							3	28.5	Pullout
2	5.04	14.92	6.13	13.50	83.44	13.05	1	31.8	Facing
							2	24.0	Pullout
							3	13.2	Pullout
3	5.85	23.62	11.73	21.71	77.09	10.57	1	29.2	Pullout
							2	9.6	Pullout
							3	18.7	Pullout
4	3.80	32.32	6.63	26.03	61.72	13.64	1	18.5	Pullout
							2	0.0	Pullout
							3	12.2	Pullout
5	4.07	41.02	0.00	24.61	43.04	22.45	1	13.5	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
6	4.18	49.72	0.00	19.89	27.64	33.67	1	14.4	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
7	4.39	58.42	0.00	23.37	24.44	38.50	1	8.3	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
8	4.57	67.12	0.00	20.13	19.06	49.71	1	8.8	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
9	4.77	75.82	0.00	22.74	17.31	55.59	1	4.6	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	4.97	84.52	0.00	25.35	15.89	61.51	1	0.8	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
11	5.17	93.22	0.00	18.64	12.95	76.52	1	4.6	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

Search Level: 4.00 feet below the toe of the wall Facing Design Force = 9.6 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode

1	3.14	6.22	40.82	7.39	86.85	11.29	1	29.8	Facing
							2	37.5	Facing
							3	28.5	Pullout
2	3.89	14.92	13.74	13.82	83.52	13.21	1	31.8	Facing
							2	24.0	Pullout
							3	13.2	Pullout
3	4.12	23.62	13.27	21.84	78.59	11.93	1	28.8	Pullout
							2	9.1	Pullout
							3	10.6	Pullout
4	3.00	32.32	11.17	26.35	61.52	13.55	1	18.6	Pullout
							2	0.1	Pullout
							3	7.7	Pullout
5	3.23	41.02	8.01	24.85	40.18	21.47	1	15.3	Pullout
							2	0.0	Pullout
							3	2.0	Pullout
6	3.40	49.72	8.07	25.11	29.56	28.58	1	12.3	Pullout
							2	0.0	Pullout
							3	2.1	Pullout
7	3.61	58.42	5.84	17.62	21.53	43.96	1	13.0	Pullout
							2	0.0	Pullout
							3	1.1	Pullout
8	3.78	67.12	5.17	20.22	19.25	49.76	1	8.4	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
9	3.99	75.82	4.66	22.82	17.45	55.63	1	4.2	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	4.14	84.52	6.36	17.01	14.08	69.71	1	7.1	Pullout
							2	0.0	Pullout
							3	0.9	Pullout
11	4.31	93.22	5.86	18.74	13.01	76.54	1	4.4	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

Search Level: 6.00 feet below the toe of the wall Facing Design Force = 9.8 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	3.04	6.22	44.16	7.80	87.19	12.69	1	29.8	Facing
							2	37.6	Facing
							3	28.4	Pullout
2	2.99	14.92	34.44	16.28	80.80	9.32	1	31.3	Facing
							2	24.7	Pullout
							3	21.1	Pullout
3	3.14	23.62	19.40	22.53	78.12	11.47	1	28.9	Pullout
							2	9.3	Pullout
							3	9.6	Pullout
4	2.54	32.32	16.40	26.95	60.47	13.11	1	19.0	Pullout
							2	0.6	Pullout
							3	5.9	Pullout
5	2.73	41.02	13.25	25.28	39.50	21.26	1	15.7	Pullout
							2	0.0	Pullout
							3	1.1	Pullout
6	2.89	49.72	13.32	25.55	28.93	28.40	1	13.0	Pullout
							2	0.0	Pullout
							3	1.2	Pullout
7	3.07	58.42	12.81	17.97	21.30	43.89	1	13.4	Pullout
							2	0.0	Pullout
							3	1.8	Pullout
8	3.23	67.12	11.36	20.54	19.01	49.69	1	8.9	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Lower Angle degrees	Lower Length feet	Upper Angle degrees	Upper Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
9	3.43	75.82	10.24	23.11	17.20	55.56	1	4.9	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	3.58	84.52	13.85	87.05	0.00	0.00	1	7.9	Pullout
							2	0.0	Pullout
							3	2.1	Pullout
11	3.71	93.22	12.78	95.58	0.00	0.00	1	5.4	Pullout
							2	0.0	Pullout
							3	0.3	Pullout

Search Level: 8.00 feet below the toe of the wall Facing Design Force = 10.2 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	2.86	6.22	55.18	9.80	87.05	12.08	1	29.8	Facing
							2	37.6	Facing
							3	28.4	Pullout
2	2.65	14.92	37.24	16.86	81.69	10.31	1	31.4	Facing
							2	24.5	Pullout
							3	18.4	Pullout
3	2.60	23.62	25.98	23.64	77.16	10.62	1	29.1	Pullout
							2	9.6	Pullout
							3	9.8	Pullout
4	2.30	32.32	22.12	27.91	58.41	12.34	1	19.8	Pullout
							2	3.4	Pullout
							3	5.7	Pullout
5	2.44	41.02	19.11	26.04	37.94	20.80	1	16.8	Pullout
							2	0.3	Pullout
							3	1.8	Pullout
6	2.57	49.72	18.07	20.92	26.91	33.45	1	15.3	Pullout
							2	0.1	Pullout
							3	0.4	Pullout
7	2.75	58.42	15.72	24.27	23.65	38.26	1	9.5	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
8	2.86	67.12	18.33	70.70	0.00	0.00	1	10.4	Pullout
							2	0.0	Pullout
							3	0.7	Pullout
9	3.02	75.82	16.55	79.09	0.00	0.00	1	6.7	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	3.20	84.52	15.12	87.55	0.00	0.00	1	3.4	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
11	3.37	93.22	13.94	96.05	0.00	0.00	1	0.5	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

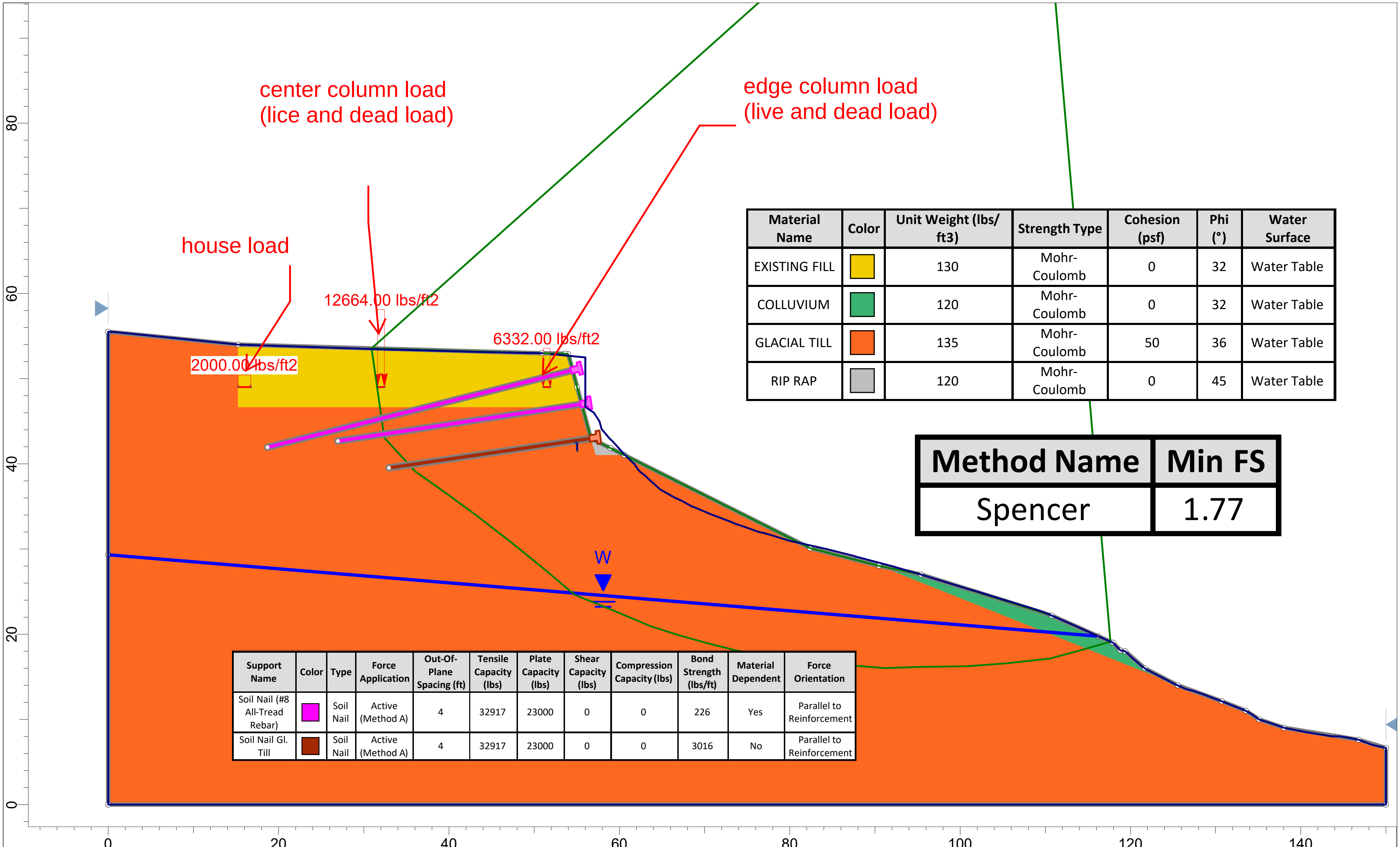
Search Level: 10.00 feet below the toe of the wall Facing Design Force = 9.8 kips (Clouterre)

			Failure Planes				Reinforcement		
			Lower		Upper				
Search Point	Minimum Factor of Safety	Distance From Toe of Wall feet	Angle degrees	Length feet	Angle degrees	Length feet	Level	Stress ksi	Controlling Resistance Failure Mode
1	2.86	6.22	63.16	12.39	86.78	11.07	1	29.8	Facing
							2	37.5	Facing
							3	28.5	Pullout
2	2.43	14.92	45.05	19.00	80.55	9.09	1	31.2	Facing
							2	24.8	Pullout
							3	19.4	Pullout
3	2.42	23.62	28.12	24.10	78.25	11.60	1	28.8	Pullout

							2	9.2	Pullout
							3	6.3	Pullout
** 4	2.13	32.32	24.00	28.30	60.68	13.20	1	18.9	Pullout
							2	0.5	Pullout
							3	1.5	Pullout
5	2.29	41.02	25.35	27.23	35.40	20.13	1	18.6	Pullout
							2	2.5	Pullout
							3	3.2	Pullout
6	2.40	49.72	20.81	26.59	29.69	28.61	1	12.2	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
7	2.50	58.42	22.27	63.13	0.00	0.00	1	11.7	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
8	2.66	67.12	19.85	71.36	0.00	0.00	1	7.1	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
9	2.82	75.82	17.93	79.69	0.00	0.00	1	2.9	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
10	2.99	84.52	16.38	88.09	0.00	0.00	1	0.0	Pullout
							2	0.0	Pullout
							3	0.0	Pullout
11	3.21	93.22	15.10	96.55	0.00	0.00	1	0.0	Pullout
							2	0.0	Pullout
							3	0.0	Pullout

END OF REPORT

ATTACHMENT 6
GLOBAL STABILITY ANALYSIS OUTPUTS – PROPOSED SOIL NAILS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table
COLLUVIUM		120	Mohr-Coulomb	0	32	Water Table
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table
RIP RAP		120	Mohr-Coulomb	0	45	Water Table

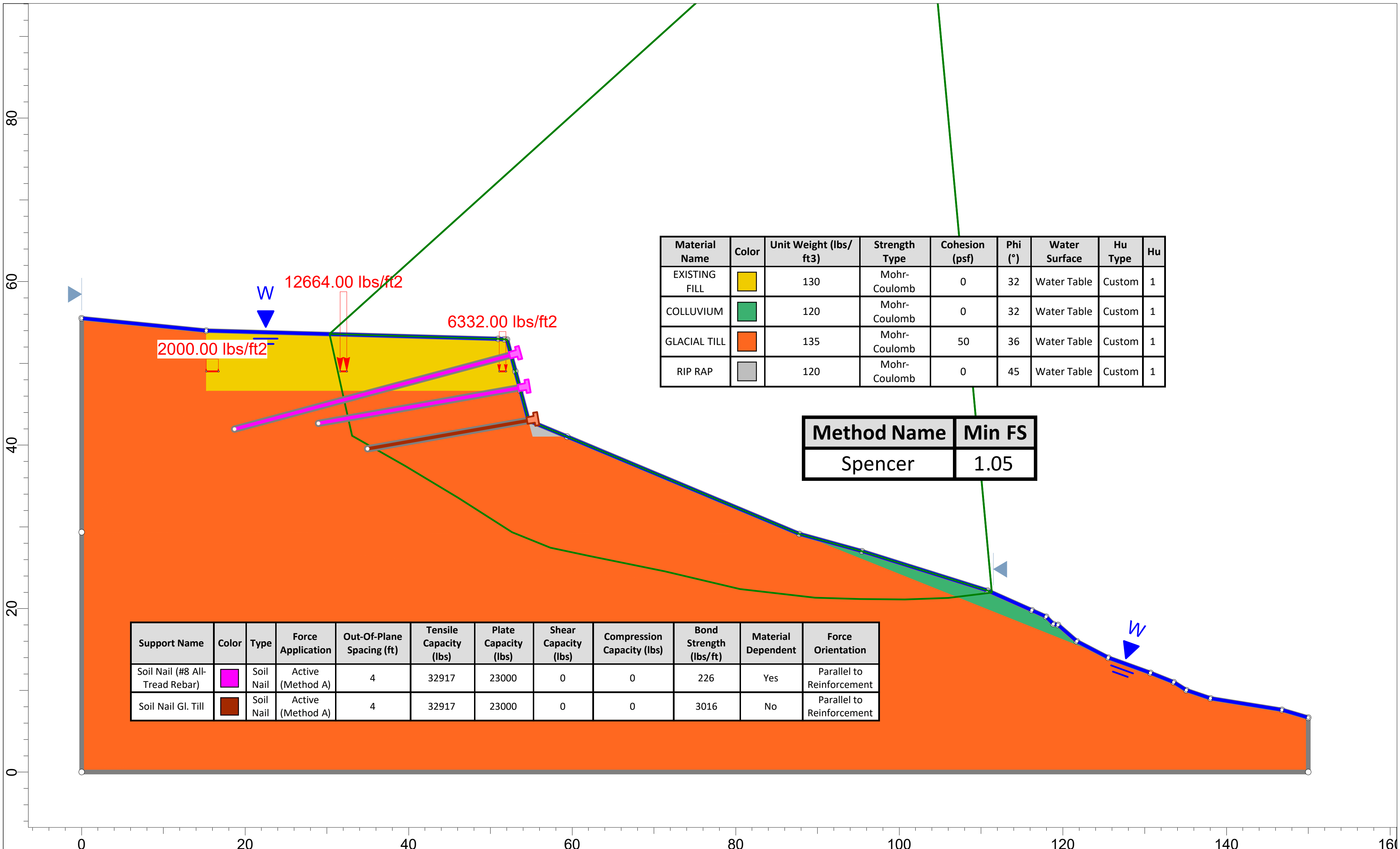
Method Name	Min FS
Spencer	1.77

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Shear Capacity (lbs)	Compression Capacity (lbs)	Bond Strength (lbs/ft)	Material Dependent	Force Orientation
Soil Nail (#8 All-Tread Rebar)		Soil Nail	Active (Method A)	4	32917	23000	0	0	226	Yes	Parallel to Reinforcement
Soil Nail Gl. Till		Soil Nail	Active (Method A)	4	32917	23000	0	0	3016	No	Parallel to Reinforcement

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.031

Project	51 Harborveiw Rd Hull, MA Slope Stabilization
Analysis Description	Proposed Conditions
Figure	
Drawn By	TS/SM
Company	CEC Inc.
Date	2/20/24
File Name	324-891 - Cross Section A-A.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface	Hu Type	Hu
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table	Custom	1
COLLUVIUM		120	Mohr-Coulomb	0	32	Water Table	Custom	1
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table	Custom	1
RIP RAP		120	Mohr-Coulomb	0	45	Water Table	Custom	1

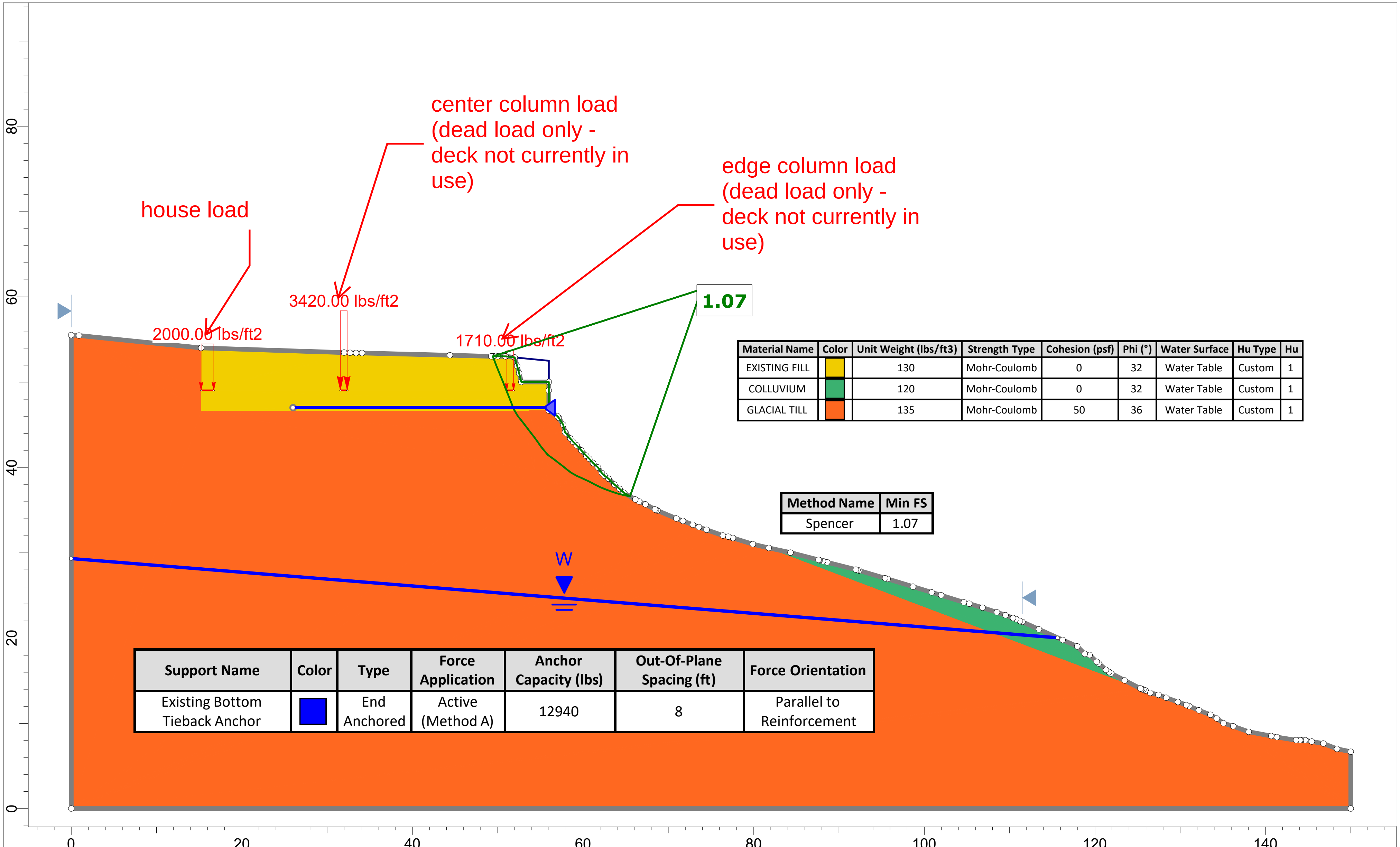
Method Name	Min FS
Spencer	1.05

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Shear Capacity (lbs)	Compression Capacity (lbs)	Bond Strength (lbs/ft)	Material Dependent	Force Orientation
Soil Nail (#8 All-Tread Rebar)		Soil Nail	Active (Method A)	4	32917	23000	0	0	226	Yes	Parallel to Reinforcement
Soil Nail Gl. Till		Soil Nail	Active (Method A)	4	32917	23000	0	0	3016	No	Parallel to Reinforcement

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.031

Project	51 Harborveiw Rd Hull, MA Slope Stabilization		
Analysis Description	Proposed Conditions Fully Saturated	*Figure*	
Drawn By	TS/SM	*Company*	CEC Inc.
Date	2/20/24	*File Name*	324-891 - Cross Section A-A Transient Seepage.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface	Hu Type	Hu
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table	Custom	1
COLLUVIUM		120	Mohr-Coulomb	0	32	Water Table	Custom	1
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table	Custom	1

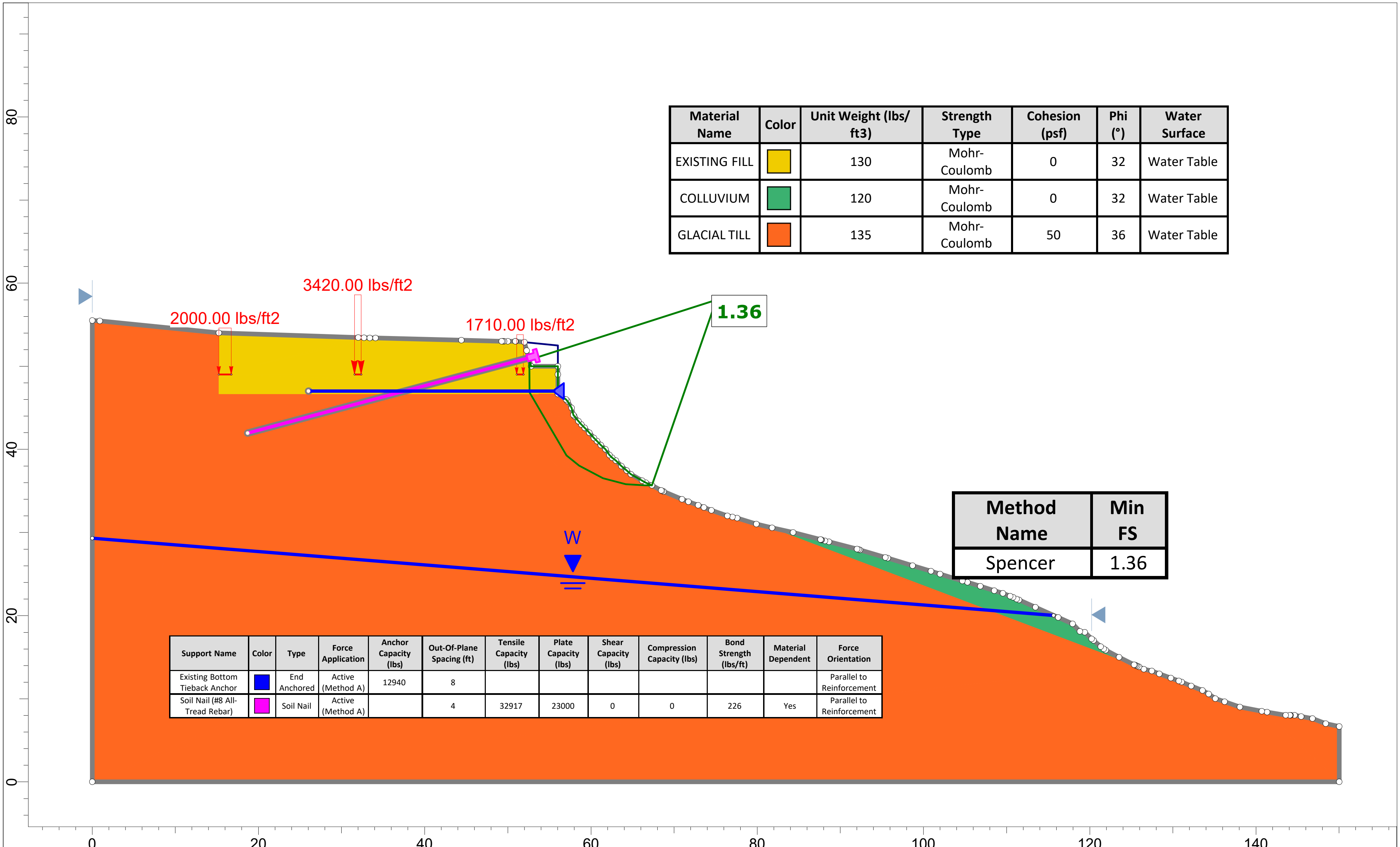
Method Name	Min FS
Spencer	1.07

Support Name	Color	Type	Force Application	Anchor Capacity (lbs)	Out-Of-Plane Spacing (ft)	Force Orientation
Existing Bottom Tieback Anchor		End Anchored	Active (Method A)	12940	8	Parallel to Reinforcement

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.031

Project	51 Harborview Rd Hul, MA Slope Stabilization		
Analysis Description	Stage 1 Excavation	*Figure*	
Drawn By	TS/SM	*Company*	CEC Inc.
Date	2/20/24	*File Name*	324-891 - Cross Section A-A.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table
COLLUVIUM		120	Mohr-Coulomb	0	32	Water Table
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table

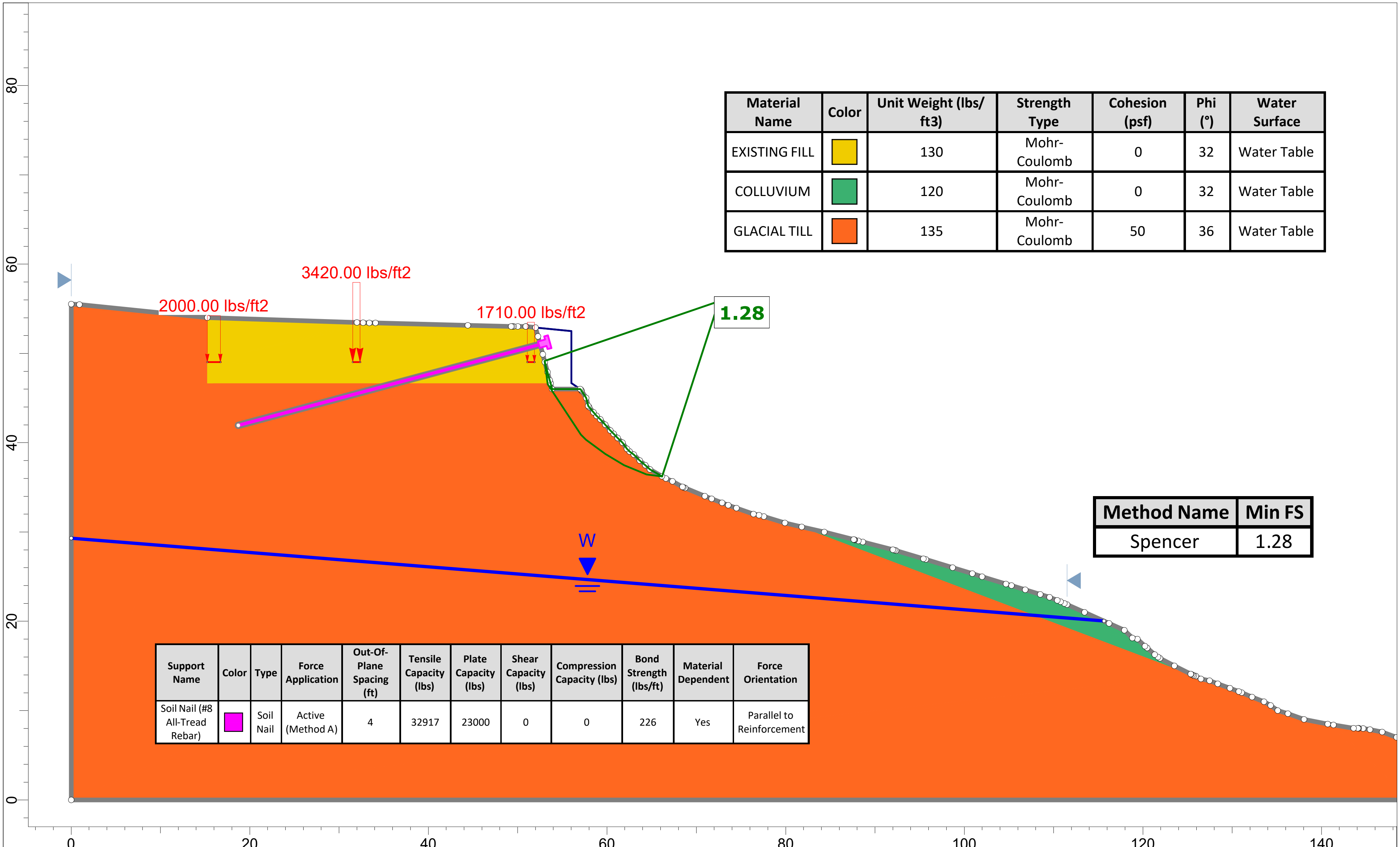
Method Name	Min FS
Spencer	1.36

Support Name	Color	Type	Force Application	Anchor Capacity (lbs)	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Shear Capacity (lbs)	Compression Capacity (lbs)	Bond Strength (lbs/ft)	Material Dependent	Force Orientation
Existing Bottom Tieback Anchor		End Anchored	Active (Method A)	12940	8							Parallel to Reinforcement
Soil Nail (#8 All-Tread Rebar)		Soil Nail	Active (Method A)		4	32917	23000	0	0	226	Yes	Parallel to Reinforcement

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.031

Project	51 Harborview Rd Hul, MA Slope Stabilization
Analysis Description	Stage 1 Temporary Conditions
Figure	
Drawn By	TS/SM
Company	CEC Inc.
Date	2/20/24
File Name	324-891 - Cross Section A-A.slmd



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table
COLLUVIUM		120	Mohr-Coulomb	0	32	Water Table
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table

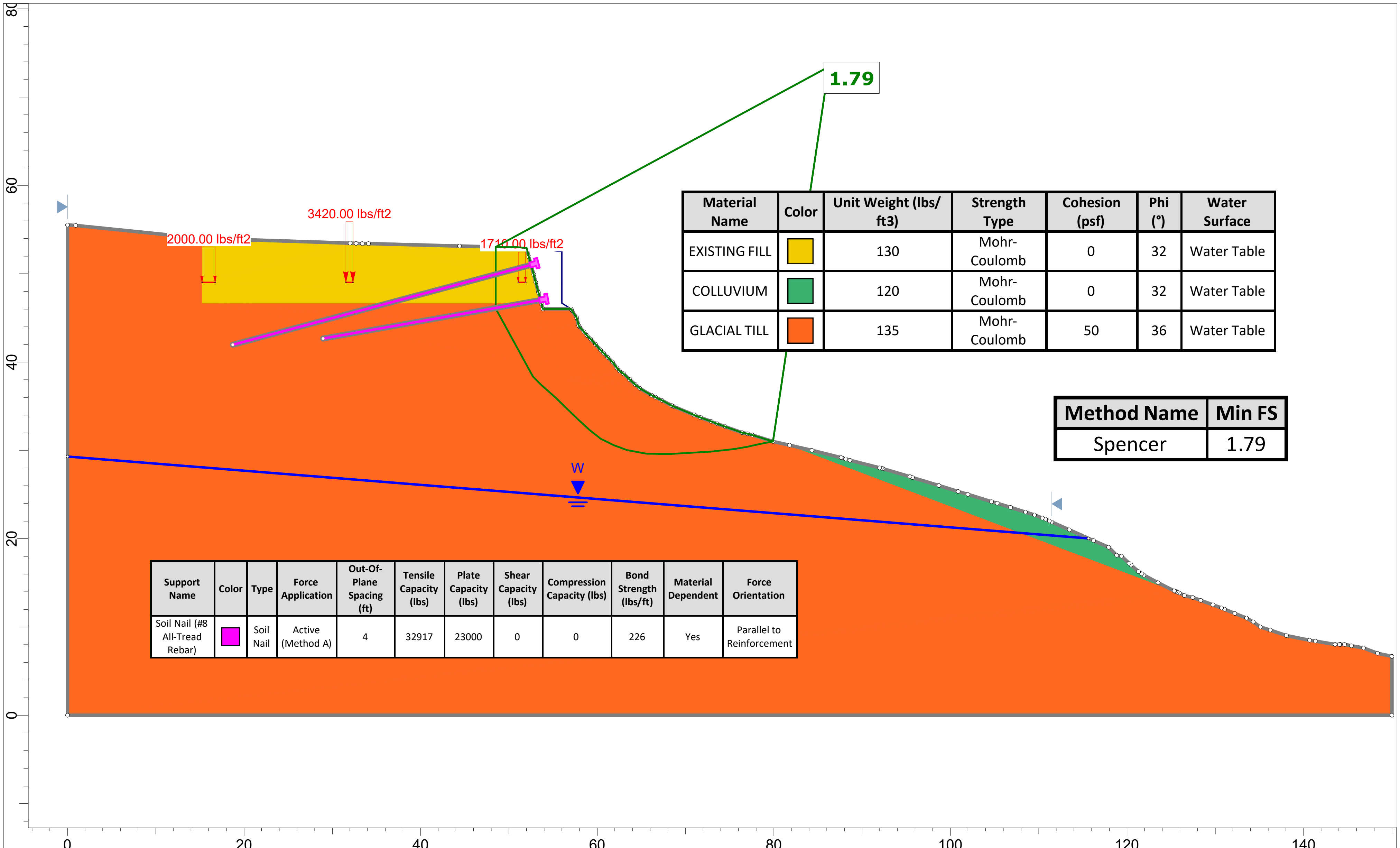
Method Name	Min FS
Spencer	1.28

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Shear Capacity (lbs)	Compression Capacity (lbs)	Bond Strength (lbs/ft)	Material Dependent	Force Orientation
Soil Nail (#8 All-Tread Rebar)		Soil Nail	Active (Method A)	4	32917	23000	0	0	226	Yes	Parallel to Reinforcement

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.031

Project	51 Harborview Rd Hul, MA Slope Stabilization		
Analysis Description	Stage 2 Excavation	*Figure*	
Drawn By	TS/SM	*Company*	CEC Inc.
Date	2/20/24	*File Name*	324-891 - Cross Section A-A.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table
COLLUVIUM		120	Mohr-Coulomb	0	32	Water Table
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table

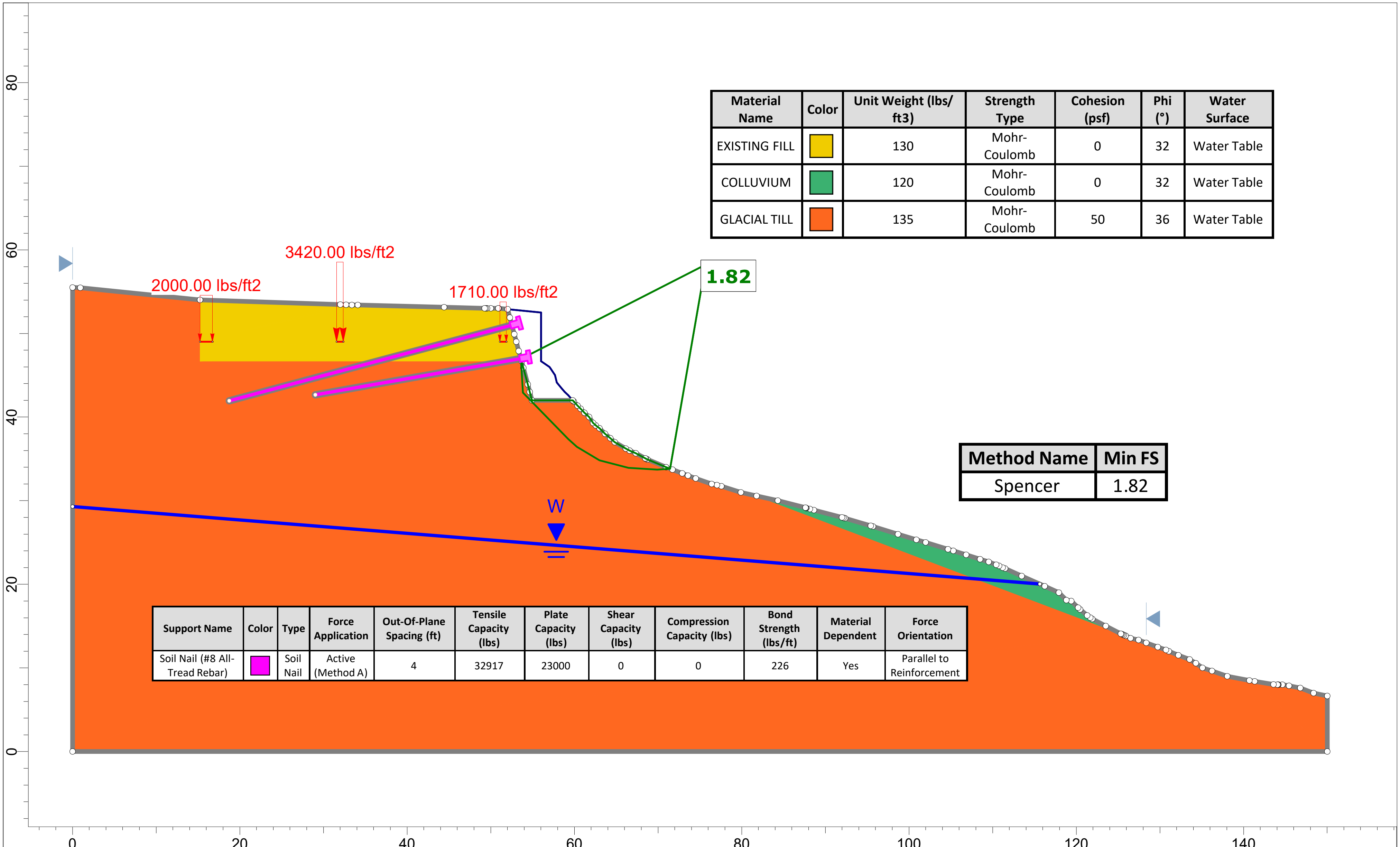
Method Name	Min FS
Spencer	1.79

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Shear Capacity (lbs)	Compression Capacity (lbs)	Bond Strength (lbs/ft)	Material Dependent	Force Orientation
Soil Nail (#8 All-Tread Rebar)		Soil Nail	Active (Method A)	4	32917	23000	0	0	226	Yes	Parallel to Reinforcement

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.031

Project	51 Harborview Rd Hul, MA Slope Stabilization
Analysis Description	Stage 2 Temporary Conditions
Figure	
Drawn By	TS/SM
Company	CEC Inc.
Date	2/20/24
File Name	324-891 - Cross Section A-A.slmd



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table
COLLUVIUM		120	Mohr-Coulomb	0	32	Water Table
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table

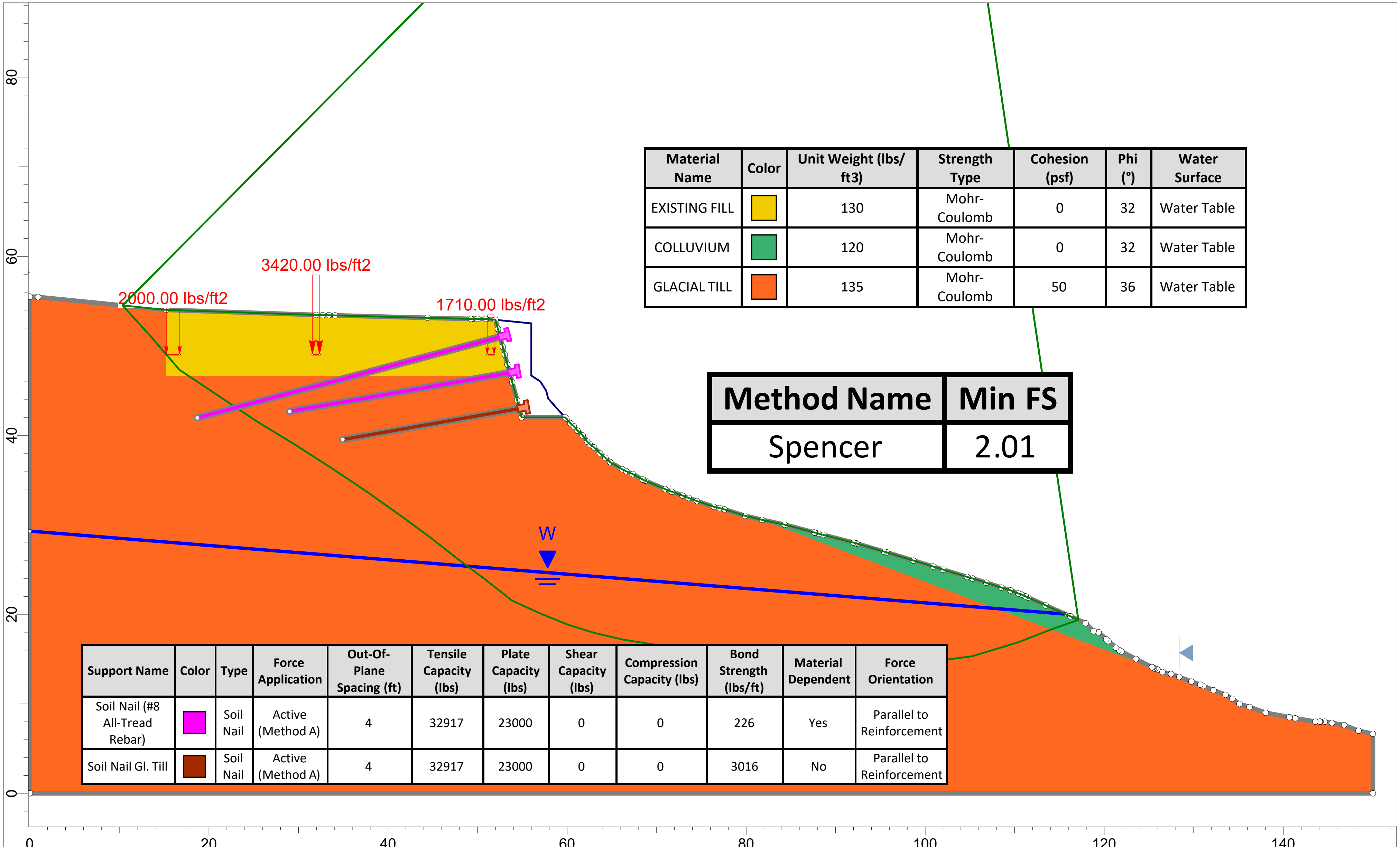
Method Name	Min FS
Spencer	1.82

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Shear Capacity (lbs)	Compression Capacity (lbs)	Bond Strength (lbs/ft)	Material Dependent	Force Orientation
Soil Nail (#8 All-Tread Rebar)		Soil Nail	Active (Method A)	4	32917	23000	0	0	226	Yes	Parallel to Reinforcement

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.031

Project	51 Harborview Rd Hull, MA Slope Stabilization		
Analysis Description	Stage 3 Excavation	*Figure*	
Drawn By	TS/SM	*Company*	CEC Inc.
Date	2/20/24	*File Name*	324-891 - Cross Section A-A.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table
COLLUVIUM		120	Mohr-Coulomb	0	32	Water Table
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table

Method Name	Min FS
Spencer	2.01

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Tensile Capacity (lbs)	Plate Capacity (lbs)	Shear Capacity (lbs)	Compression Capacity (lbs)	Bond Strength (lbs/ft)	Material Dependent	Force Orientation
Soil Nail (#8 All-Tread Rebar)		Soil Nail	Active (Method A)	4	32917	23000	0	0	226	Yes	Parallel to Reinforcement
Soil Nail Gl. Till		Soil Nail	Active (Method A)	4	32917	23000	0	0	3016	No	Parallel to Reinforcement

CEC
Civil & Environmental Consultants, Inc.

Project	51 Harborview Rd Hull, MA Slope Stabilization		
Analysis Description	Stage 3 Temporary Conditions	*Figure*	
Drawn By	TS/SM	*Company*	CEC Inc.
Date	2/20/24	*File Name*	324-891 - Cross Section A-A.slmd

SLIDEINTERPRET 9.031

ATTACHMENT 7
GABION BASKET STABILITY ANALYSES

Civil & Environmental Consultants, Inc.

PROJECT 51 Harborview Rd Slope Stabilization PROJECT NO. 324-891

PAGE 1 OF 2

PREPARED BY VPC DATE 10/11/23 CHECKED BY GSS DATE 10/16/23

Friction coefficient between Gabion and subsoil

$$\alpha_{ds} = \frac{\alpha_s \tan\delta + (1-\alpha_s)\tan\phi}{\tan\phi}$$

α_{ds} = Friction coeff. between gabion and subsoil

α_s = ratio of steel area to gabion-soil contact area

δ = steel-soil friction angle = 0.3ϕ As per NavFac

ϕ = friction angle of subsoil = 38°

$\alpha_s = A_s/(3' \times 3') = A_s/9$ sqft ; A_s = Area of steel

As calc:

wire diameter = 4mm

opening diameter = 75mm

gabion length = 3 ft = 914.4mm

−4mm for leading wire

wire+opening length → $\frac{910.4\text{ mm}}{(75\text{mm} + 4\text{mm})} \cong 11.5 \xrightarrow{\text{round}} 12 + \underset{\text{lead}}{1} = 13$ wires per direction

Wire area = 4mm × (3ft = 914.4mm) = 3657.6 sqmm

× 13 wires

= 47,548.8 sqmm

× 2 directions

= 95097.6 sqmm

− 2704 sqmm overlap

= 92,393.6 sqmm

= 0.99... sqft ≅ 1 sqft

4mm × 4mm = 16 sqmm/overlap

13×13 = 169 overlaps

16×169 = 2704 sqmm of overlapp

$\alpha_s = 1\text{ sqft} / 9\text{ sqft} = 0.\overline{11}$

Friction coeff between gabion and subsoil cont.

$$\alpha_{ds} = \frac{(0.11)\tan(0.3(38^\circ)) + (1-0.22)\tan(38^\circ)}{\tan(38^\circ)}$$

$$\boxed{\alpha_{ds} \cong 0.90}$$

Void Ratio in Gabions

- Not to exceed 0.3
  - Set as 0.25

$$e = 0.25 = \frac{V_v}{V_S} = \frac{0.25}{1} \rightarrow V_t = V_v + V_S \rightarrow \frac{0.25}{1+0.25} = 0.2 = 20\%$$

SRWall (Version 4) Report

Project Identification

Project ID	: **324-891**
Project Name	: **Harborview 51 - East Gabion Baskets**
Owner	:
Client	: **Tom Fitzgerald**
Prepared By	: **GSS**
Company	: **Civil & Environmental Consultants**
Address	:
Telephone	:
Section	:
Project File	: **324-891_10-16-23.prj**
Vendor Data File	:
Date and Time	: **10/13/2023 10:59:47**

Type of Structure	: **Gravity Wall**

Wall Geometry

Design Wall Height(ft)	: **9.00**
Embedment Wall Height(ft)	: **2.00**
Exposed Wall Design Height(ft)	: **7.00**
Number of Segmental Wall Units	: **3**
Wall Inclination(degrees)	: **9.46**

Grades

Top Slope(degrees)	: **27.00**
Broken Back Distance(ft)	: **5.00**

Uniform Distributed Surcharge

Live Load Surcharge(Psf)	: **0.00**
Dead Load Surcharge(Psf)	: **0.00**

Soil Data

Soil Zone	Description	Cohesion (c) (psf)	Friction Angle(Φ) (degrees)	Unit Weight (γ)(pcf)
Retained Soil	**Glacial Till**	**N/A**	**36.00**	**135.00**
Leveling Pad Soil	**Crushed Stone**	**N/A**	**38.00**	**140.00**
Foundation Soil	**Glacial Till**		**36.00**	**135.00**

Segmental Unit Data

Segmental Unit Name	**: 3'x3'x3' Gabion Basket**
Cap Height (Inches)	**: 0.00**
Unit Height (Hu)(Inches)	**: 36.00**
Unit Width (Wu)(Inches)	**: 36.00**
Unit Length (Inches)	**: 36.00**
Setback (Inches)	**: 6.00**
Weight (Infilled)(lb)	**: 3240.00**
Unit Weight (Infilled)(pcf)	**: 120.00**
Center of Gravity(Inches)	**: 18.00**

Unit-Unit Interface Properties

Minimum Shear Capacity(lb/ft)	Shear Friction Angle	Maximum Shear Capacity (lb/ft)
880.00	**1.00**	**880.00**

Vertical Components

Vertical Components of Earth Pressures Used	**: No**

Cofficients of Earth Pressure and Failure Plane Orientation

Retained Soil(Static)(Ka)	**: 0.189**
Retained Soil(Static)(Kah Horizontal Component)	**: 0.169**
External Modified Back Slope(Bext)	**: 8.056**
Orientation of failure plane from horizontal(degrees) for External Stability	**: 54.775**

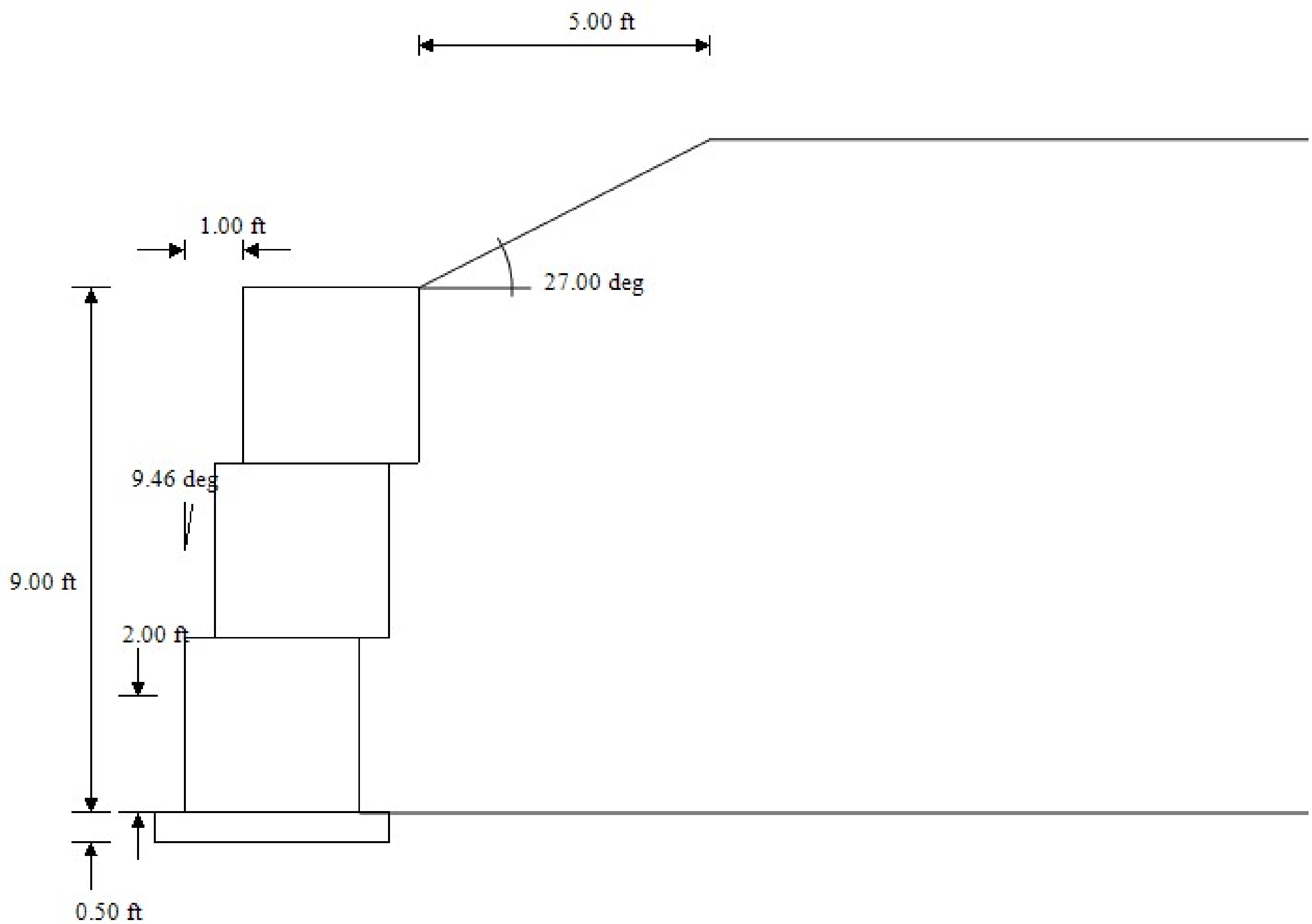
Result of External Stability Static Analysis

	Calculated	Design Criteria
FOS Sliding	1.77	> 1.50
FOS Overturning	2.16	> 2.00
FOS Bearing Capacity	16.58	> 2.50
Base Footing (B)(ft)	3.50	

Results of Internal Stability Static Analysis

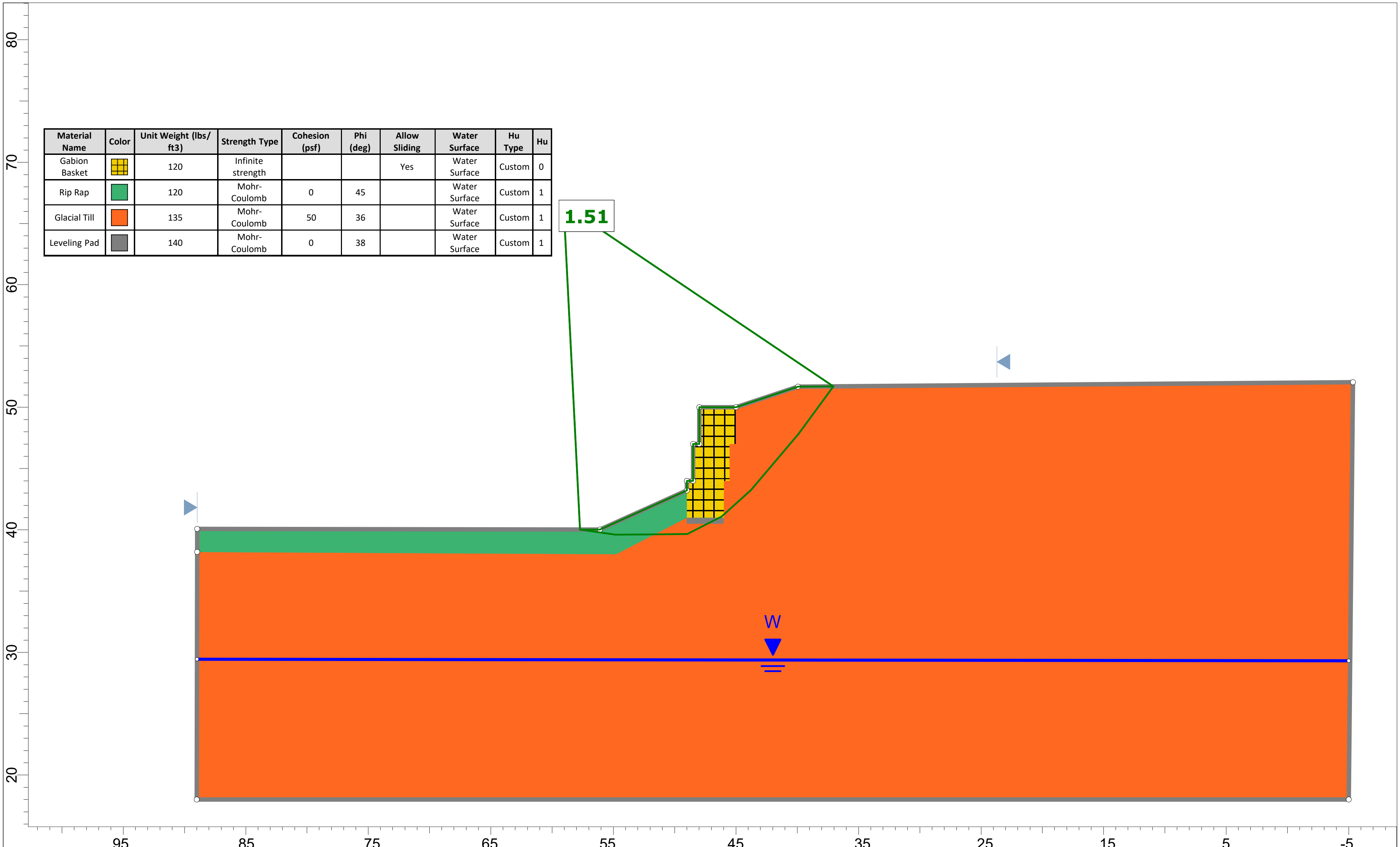
SRW Unit #	Heel Elev (ft)	FOS Shear >=1.50
3	6.00	7.93
2	3.00	1.98

Wall Geometry Layout



Project Identification

Project ID	: **324-891**
Project Name	: **Harborview 51 - East Gabion Baskets**
Owner	:
Client	: **Tom Fitzgerald**
Prepared By	: **GSS**
Company	: **Civil & Environmental Consultants**
Address	:
Telephone	:
Section	:
Vendor Data File	:
Project File	: **324-891_10-16-23.prj**
Date and Time	: **10/13/2023 10:59:47**

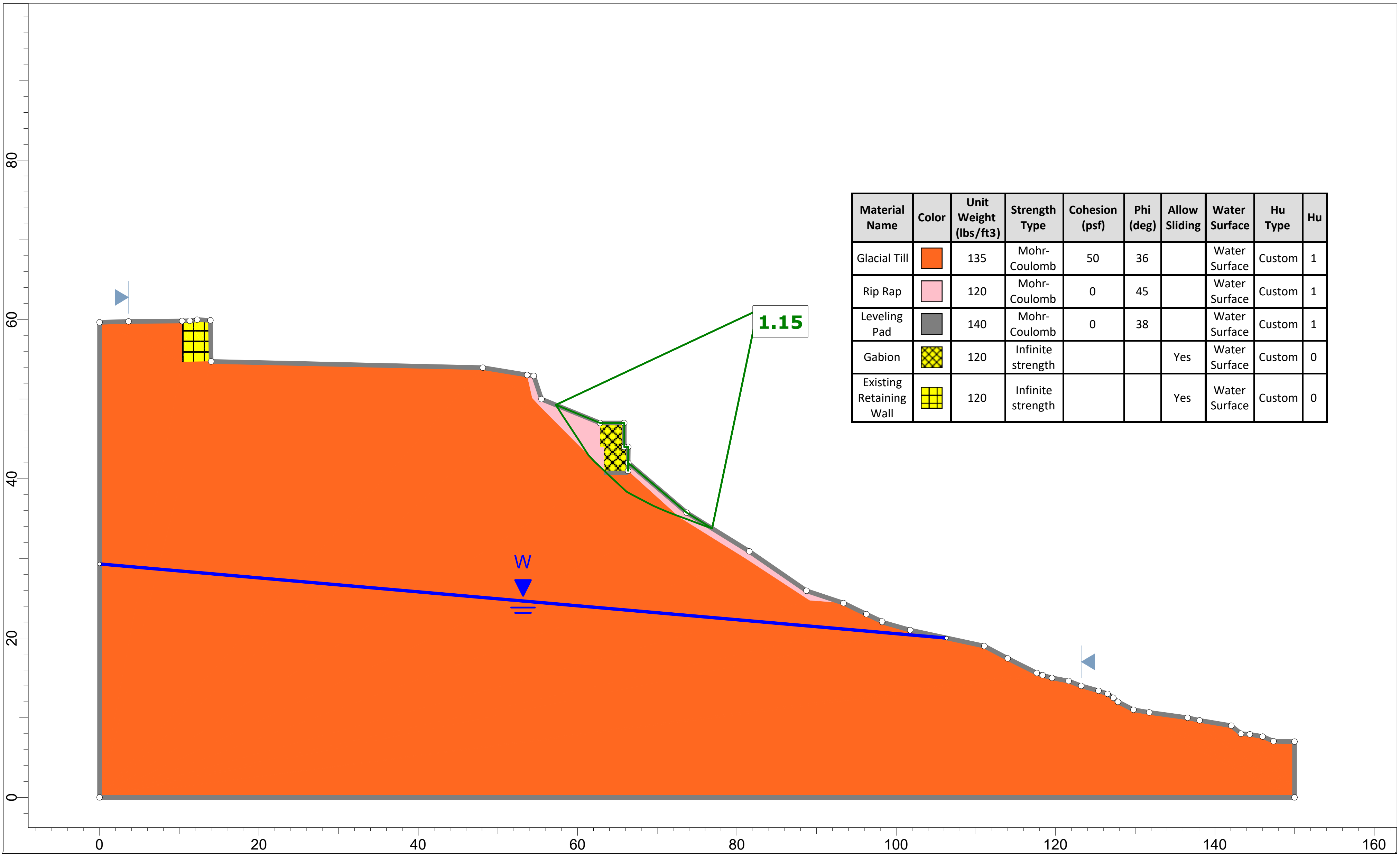


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu
Gabion Basket		120	Infinite strength			Yes	Water Surface	Custom	0
Rip Rap		120	Mohr-Coulomb	0	45		Water Surface	Custom	1
Glacial Till		135	Mohr-Coulomb	50	36		Water Surface	Custom	1
Leveling Pad		140	Mohr-Coulomb	0	38		Water Surface	Custom	1

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.026

Project	51 Harborview Road Slope Stability
Analysis Description	East Gabion Baskets - East-West Section
Figure	
Drawn By	TS
Company	CEC Inc.
Date	10/11/2023
File Name	324-891 Harborview Gabion Gravity SRW - Global.slmd

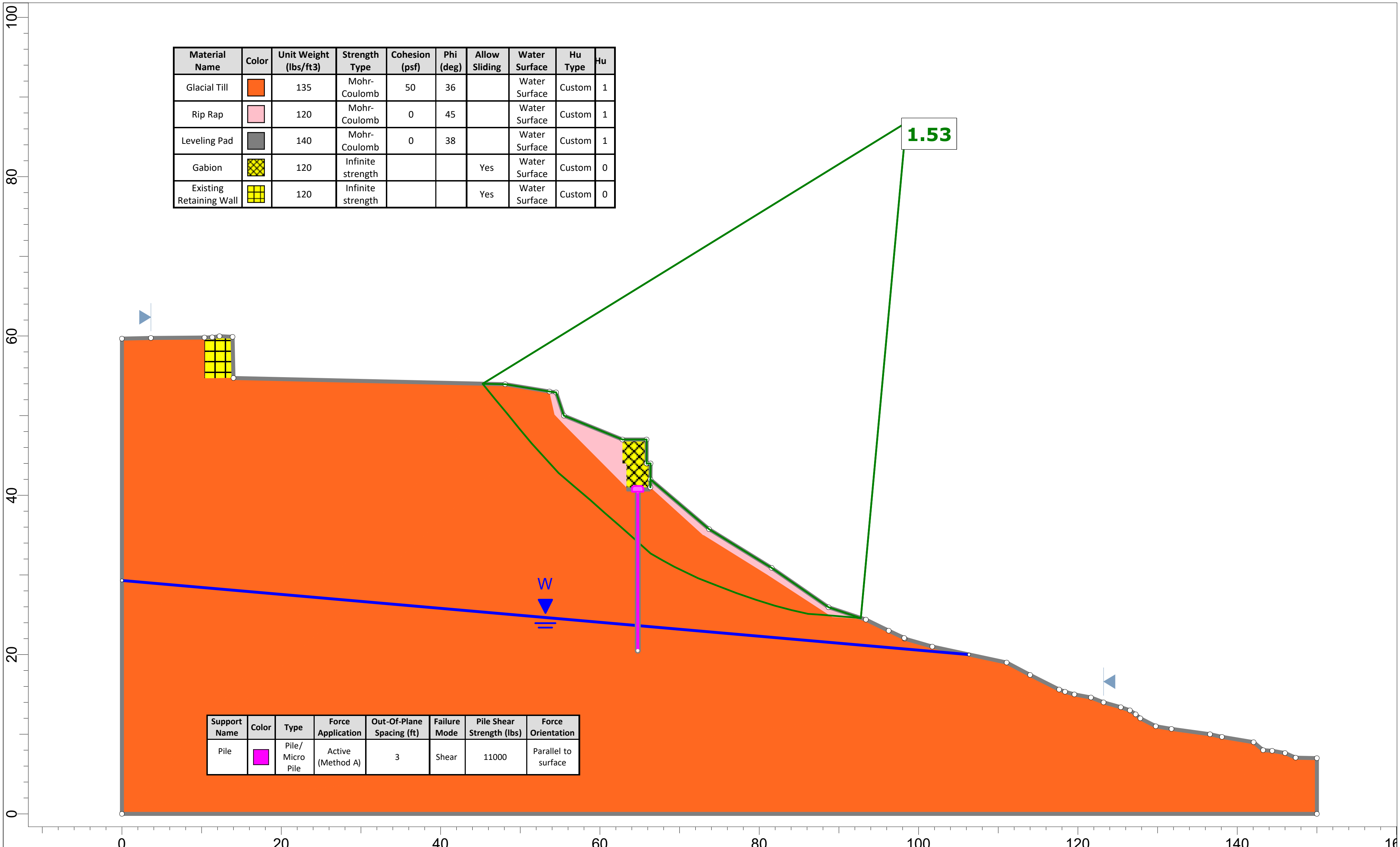


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu
Glacial Till		135	Mohr-Coulomb	50	36		Water Surface	Custom	1
Rip Rap		120	Mohr-Coulomb	0	45		Water Surface	Custom	1
Leveling Pad		140	Mohr-Coulomb	0	38		Water Surface	Custom	1
Gabion		120	Infinite strength			Yes	Water Surface	Custom	0
Existing Retaining Wall		120	Infinite strength			Yes	Water Surface	Custom	0

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.026

Project	51 Harborview Rd Hull, MA Slope Stabilization
Analysis Description	East Gabion Baskets - North-South Section
Figure	
Drawn By	TS
Company	CEC Inc.
Date	10/17/23
File Name	



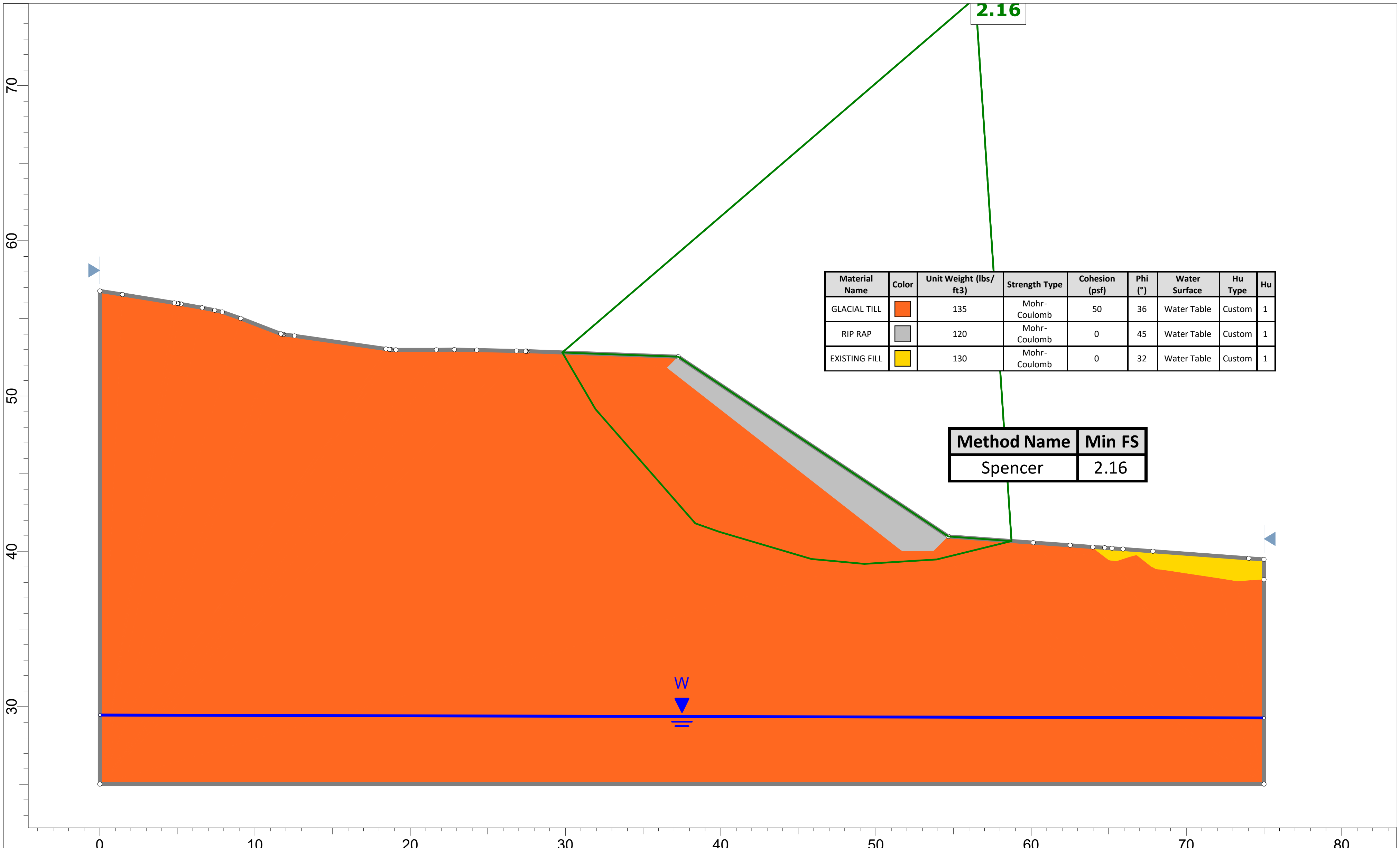
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Allow Sliding	Water Surface	Hu Type	Hu
Glacial Till		135	Mohr-Coulomb	50	36		Water Surface	Custom	1
Rip Rap		120	Mohr-Coulomb	0	45		Water Surface	Custom	1
Leveling Pad		140	Mohr-Coulomb	0	38		Water Surface	Custom	1
Gabion		120	Infinite strength			Yes	Water Surface	Custom	0
Existing Retaining Wall		120	Infinite strength			Yes	Water Surface	Custom	0

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Pile		Pile/ Micro Pile	Active (Method A)	3	Shear	11000	Parallel to surface

CEC
Civil & Environmental Consultants, Inc.
SLIDEINTERPRET 9.026

Project	51 Harborview Rd Hull, MA Slope Stabilization		
Analysis Description	East Gabion Baskets - North-South Section w/Piles	*Figure*	
Drawn By	TS	*Company*	CEC Inc.
Date	10/18/23	*File Name*	

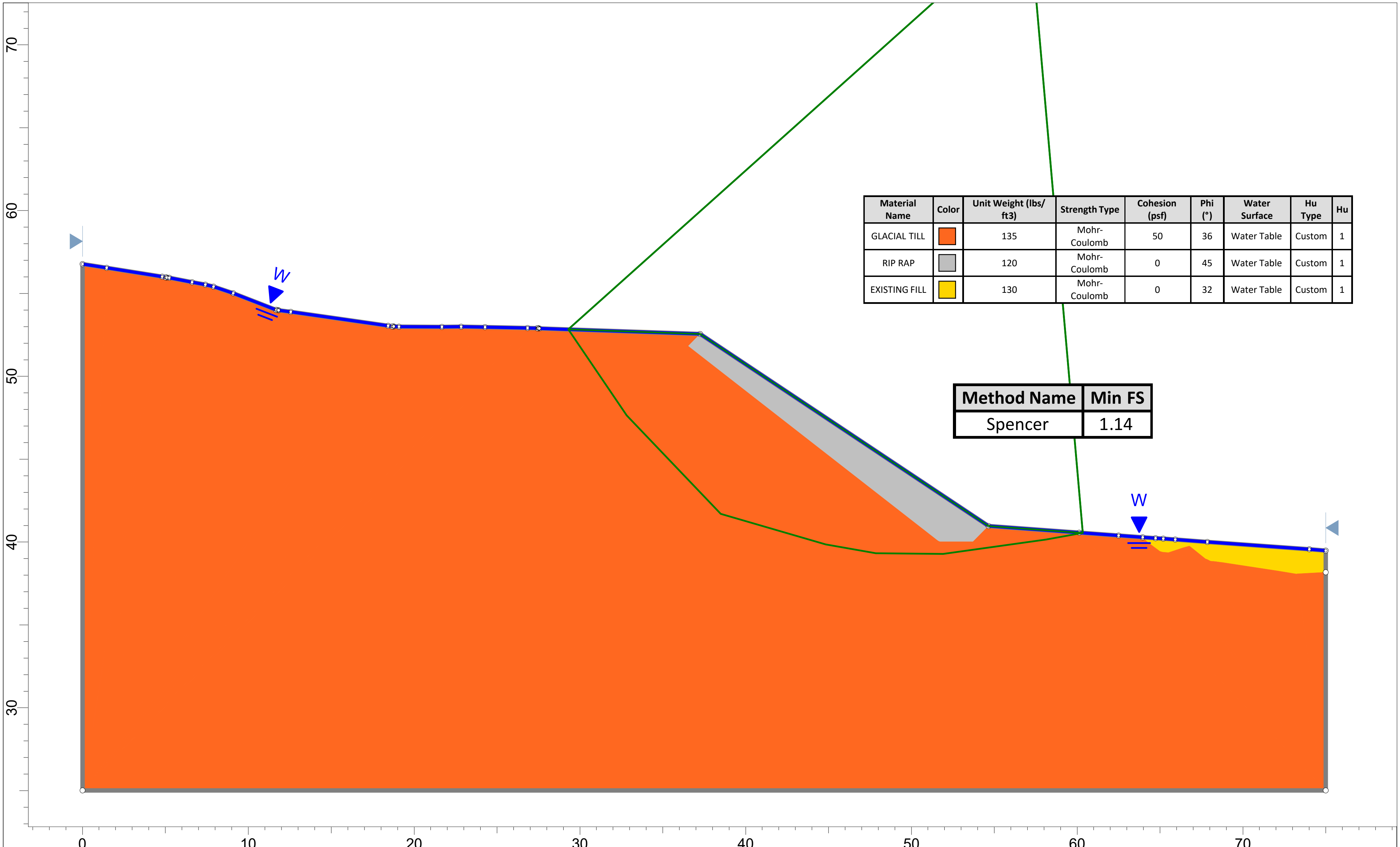
ATTACHMENT 8
RIP RAP TRANSITION SLOPE STABILITY ANALYSIS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface	Hu Type	Hu
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table	Custom	1
RIP RAP		120	Mohr-Coulomb	0	45	Water Table	Custom	1
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table	Custom	1

Method Name	Min FS
Spencer	2.16

CEC Civil & Environmental Consultants, Inc.	*Project* 51 Harborveiw Rd Hull		
	Analysis Description Proposed Conditions	*Figure*	
	Drawn By TS/SM	*Company* CEC Inc.	
SLIDEINTERPRET 9.031	*Date* 2/20/24	*File Name* Rip Rap Side Slope.slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface	Hu Type	Hu
GLACIAL TILL		135	Mohr-Coulomb	50	36	Water Table	Custom	1
RIP RAP		120	Mohr-Coulomb	0	45	Water Table	Custom	1
EXISTING FILL		130	Mohr-Coulomb	0	32	Water Table	Custom	1

Method Name	Min FS
Spencer	1.14

Civil & Environmental Consultants, Inc.

SLIDEINTERPRET 9.031

Project	51 Harborveiw Rd Hull		
Analysis Description	Transient Seepage Condition	*Figure*	
Drawn By	TS/SM	*Company*	CEC Inc.
Date	2/20/24	*File Name*	Rip Rap Side Slope.slmd

ATTACHMENT 9
TEST BORING LOG

Rock Types

Rock Name	Characteristics	Symbol
Claystone	Clay sized particles that are consolidated, lacking fissility.	
Coal	Black and shiny, can break into cubes or conchoidally.	
Conglomerate	Gravel sized grains and larger held together by finer material, called a breccia if clasts are angular.	
Limestone	Effervesses w/ diluted HCl, can be composed of clay up to gravel particles (fossils).	
Sandstone	Primarily sand sized particles modified w/ the descriptor fine, medium, or coarse.	
Shale	Clay sized particles, shale has fissility which is a horizontal sheet-like or laminated feature.	
Siltstone	Composed of silt, normally breaks as irregular chunks.	

Rock Quality Descriptions

Weathering

Completely Weathered: All rock material is decomposed and/or disintegrated. The original rock structure may still be intact.

Highly Weathered: More than half of the rock material is decomposed. Fresh rock is present only as a discontinuous framework or as corestones.

Moderately Weathered: Less than half of the rock material is decomposed. Fresh rock is present at a discontinuous framework or as corestones.

Slightly Weathered: Discoloration or staining indicates weathering of rock material on discontinuity surfaces. Rock may be discolored and softened.

Fresh: No visible signs of rock material weathering.

RQD

Descriptor	%
Very Poor	<25
Poor	25-50
Fair	50-75
Good	75-90
Excellent	>90

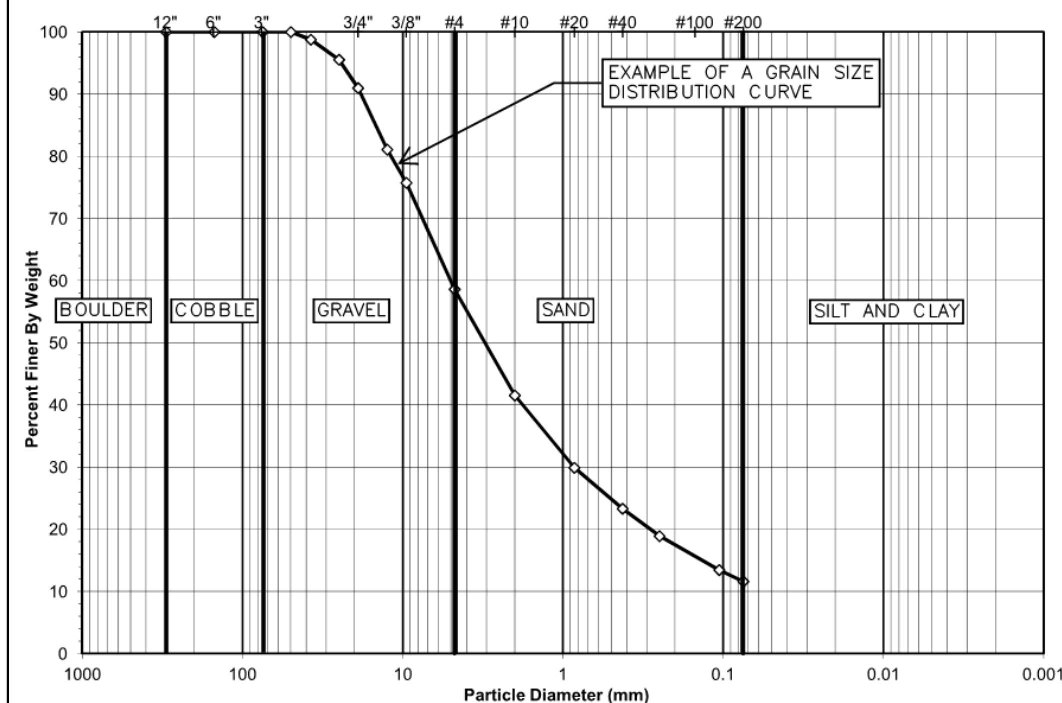
Brokenness

Descriptor	Fracture Spacing (in & ft)
Very Broken	<1 (<0.08)
Broken	1-3 (0.08-0.25)
Moderately Broken	3-6 (0.25-0.5)
Slightly Broken	>6 (>0.5)

Rock Hardness

Descriptor	Field Criterion	Relative Unconfined Compressive Strength
Very Hard	Difficult to break w/ Hammer	> 30,000 psi
Hard	Hand-held sample breaks w/ Hammer	8,000 to 30,000 psi
Medium Hard	Cannot scrape surface w/ knife	2,000 to 8,000 psi
Soft	Cutting or scraping w/ knife difficult	500 to 2,000 psi
Very Soft	Can be cut w/ knife	< 500 psi

Grain Size Distribution Curve



UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)		
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)	
	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)	
	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)		
SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	CH	Inorganic clays of high plasticity, fat clays
	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

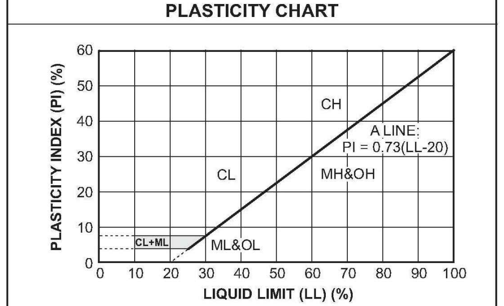
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent .... GW, GP, SW, SP
More than 12 percent .... GM, GC, SM, SC
5 to 12 percent .... Borderline cases requiring dual symbols

PLASTICITY CHART



Glossary

Alluvial Soil or Alluvium: Soil deposited by water in a river, stream, floodplain, or delta.

Bedrock: Materials underlying soil or other unconsolidated surficial materials in which refusal is consistently encountered on lithified, undisturbed, natural bedrock.

Colluvial Soil or Colluvium: Incoherent soil on or at the base of a slope deposited by gravity or slope movement.

Fill: Soil derived from natural soil, rock, or processed materials that was placed by artificial methods, such as construction, waste disposal, or dumping.

Glacial Outwash: Soil, typically sand and gravel, deposited by glacial streams or meltwater in a preexisting valley or over a plain.

Glacial Till: Soil deposited by and underneath a glacier, generally consisting of a heterogeneous, unstratified mixture of clay, sand, gravel, and boulders.

N-Value: The blow count representation of the penetration resistance of the soil determined by the Standard Penetration Test (SPT). It is the sum of the number of blows required to drive the sampler the second and third 6-inch increments (sample depth interval of 6 to 18 inches) and is recorded in blows per foot (bpf). The N-value is considered to be an indication of the relative density of coarse-grained soils (sand and gravel) or consistency of fine-grained soils (silt and clay).

Pocket Pen (PP): Field penetration test performed using a hand-held penetrometer that estimates unconfined compressive strength of cohesive soil in tons per square foot (tsf).

Recovery %: Total length of rock core or soil sample retrieved divided by the total length of the core run or sample interval, expressed as a percentage.

Refusal: The depth at which greater than 50 SPT hammer blows are required to drive the sampling spoon 6 inches or less.

Residual Soil or Residuum: Soil derived from the physical or chemical weathering of the underlying parent bedrock, generally with N-values less than 30 and 50 bpf in cohesive and cohesionless materials, respectively.

Rock Quality Designation (RQD): The sum of the length of intact rock core pieces longer than 4 inches (excluding mechanical breaks) divided by the total length of the core run, expressed as a percentage.

Shelby Tube: A 2" to 3" diameter, thin walled sampling tube that is pushed into the soil to obtain a relatively undisturbed soil sample for geotechnical laboratory tests.

Split Spoon Sampler: A soil sampling tube which is driven, retrieved, and split-open lengthwise for removal and visual inspection, and testing of the soil obtained.

Standard Penetration Test (SPT) ASTM D1586 : Field penetration test consisting of driving a 2-inch outside diameter split-spoon sampler 18 inches using a 140-pound hammer free falling a distance of 30 inches. The number of blows required to advance the spoon through successive 6-inch increments is recorded to determine the N-value.

Weathered Rock: Materials derived from lithified, undisturbed, natural bedrock which are able to be sampled with a split-spoon. Cohesive and cohesionless materials generally have N-values greater than 30 and 50 bpf, respectively.

N-Value Rating

Fine-Grained Soils (Silt and Clay)

Consistency	Blows/ft	PP (tsf)
Very Soft	0-2	<0.25
Soft	3-4	0.25-0.5
Medium Stiff	5-8	0.5-1
Stiff	9-15	1-2
Very Stiff	16-32	2-4
Hard	>32	>4

Coarse-Grained Soils (Sand and Gravel)

Relative Density	Blows/ft
Very Loose	0-4
Loose	5-10
Medium Dense	11-30
Dense	31-50
Very Dense	>50

Unconsolidated Material

Term	Grain Size in mm (in)	Approximate Example Size
Clay and Silt	<.075	can't see grains to barely visible
Fine Sand	0.075-0.4	table salt to sugar
Med. Sand	0.4-2.0 (~<1/16)	openings in a window screen
Coarse Sand	2.0-4.75 (~1/16-1/8)	sidewalk salt
Gravel	4.75-75 (~1/8-3)	pea to tennis ball
Cobble	75-300 (3-12)	tennis ball to basketball
Boulder	>300 (>12)	larger than a basketball

Other Features – Used to describe other identifiable, pertinent features (e.g., angularity of coarse-grained soils, organics, construction debris, etc.)

Term	%
Trace	< 5
Few	5-15
Some	15-45

Moisture Content

Dry: Sample is dusty or obviously dry.
Moist: Anything that does not fit the definition of dry or wet.
Wet: Sample contains free water.

Civil & Environmental Consultants, Inc.

Definitions of Standard Terms and Symbols

Civil & Environmental Consultants, Inc.
31 Bellows Road
Raynham, MA 02767

BORING NUMBER B-1

CLIENT	Mr. Tom Fitzgerald	**PROJECT NAME**	51 Harborview Rd. Slope Stabilization
PROJECT NUMBER	324-891	**PROJECT LOCATION**	Hull, MA
DATE STARTED	06/28/2023 **DATE COMPLETED** 06/28/2023	**GROUND ELEVATION**	58.00 ft **BACKFILL** with Grout
SAMPLING CONTRACTOR	Soil X Corp.	**LATITUDE**	42.307225 **LONGITUDE** -70.906136
SAMPLING METHOD	4-1/4-in Hollow Stem Auger	**WATER LEVELS** AT END OF SOIL SAMPLING	06/28/2023 Not observed
CEC REP	Tony Sousa, P.E. **CHECKED BY** Tyler Reynolds, P.E.	AT END OF CORING	N/A
NOTES		24 HRS AFTER DRILLING	N/A

DEPTH (ft)	GRAPHIC LOG	REMARKS	MATERIAL DESCRIPTION	ELEVATION (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOWS COUNTS (N VALUES)
0–3.0 ft			Brown, Silty Fine to Medium Sand, Trace Fine Gravel, Trace Roots, Dry, Loose, SM, **(FILL)**	58–55.0	SS S-1	25	4-3-2-3 (5)
3.0–5.0 ft		Augers grinding intermittently from 3-30' bgs.		55.0–53.0			
5.0–7.0 ft			Brown, Silty Fine to Medium Sand, Some Fine to Coarse Gravel, Damp, Very Dense, Slight Plasticity, SM **(GLACIAL TILL)**	53.0–51.0	SS S-2	75	15-19-39-23 (58)
10.0–11.5 ft		Split spoon refusal at 11.5' bgs.	Brown, Silty Fine to Medium Sand, Some Fine to Coarse Gravel, Dry, Very Dense, SM **(GLACIAL TILL)**	48.0–46.5	SS S-3	100	27-40-60 (100)
15.0–16.4 ft		Split spoon refusal at 16.4' bgs.	Brown, SILTY FINE TO COARSE SAND, SOME FINE GRAVEL, Dry, Very Dense, SM **(GLACIAL TILL)**	43.0–41.6	SS S-4	93	32-62-63/0.4' (125)
20.0–21.5 ft		Split spoon refusal at 21.5' bgs.	Brown, Silty Fine to Medium Sand, Some Fine to Coarse Gravel, Dry, Very Dense, SM **(GLACIAL TILL)**	38.0–36.5	SS S-5	100	35-60-88 (148)
25.0–25.3 ft		Split spoon refusal at 25.3' bgs.	No Recovery	33.0–32.7	SS S-6	0	75/0.3' (75)
30.0–30.1 ft		Split spoon and auger refusal at 30.1' bgs. Possible bedrock at 30' bgs.	Gray, Coarse Gravel, Dry, Very Dense, GP **(GLACIAL TILL)**	28.0–27.9	SS S-7	100	70/0.1' (70)

End of boring at 30.1 feet

ATTACHMENT 10
LABORATORY TEST RESULTS

Thielsch
DIVISION OF THE RISE GROUP

195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
thielsch.com
Let's Build a Solid Foundation

Client Information:		Project Information:	
Civil & Environmental Consultants		**Slope Stabilization**	
Raynham, MA 02767		**51 Harborview Rd, Hull MA**	
Project Manager:	Tony Sousa	Project Number:	324-891
Assigned By:	Tony Sousa	Summary Page:	1 of 1
Collected By:	Client	Report Date:	07.10.23

LABORATORY TESTING DATA SHEET, Report No.: 7423-G-101

Material Source	Sample ID	Depth (ft)	Laboratory No.	Identification Tests						Corrosivity Tests								Laboratory Log and Soil Description
				As Rcvd Moisture Content %	LL %	PL %	Gravel %	Sand %	Fines %	Resistivity (Mohms-cm)	Chloride (mg/kg)	Sulfate (mg/kg)	Sulfide (mg/kg)	Redox Potential (mv)	pH	Electrical Resist. As Rcvd Ohm-cm @ 60°F	Electrical Resist. Saturated Ohm-cm @ 60°F	
				D2216	D4318		D6913			EPA	D4327		EPA	G200	G51	G57		
Face of Steep Slope	GS-1	0-1	23-S-2805	5.9			25.0	73.1	1.9		16	13			9.1	33600	17400	Brown poorly graded sand with gravel
Top of Wall	AS-1	2-3	23-S-2806	1.9			18.8	40.2	41.0									Brown silty sand with gravel
B-1	S-4	15-16.4	23-S-2807	10.1			24.2	35.4	40.4									Brown clayey silty sand with gravel

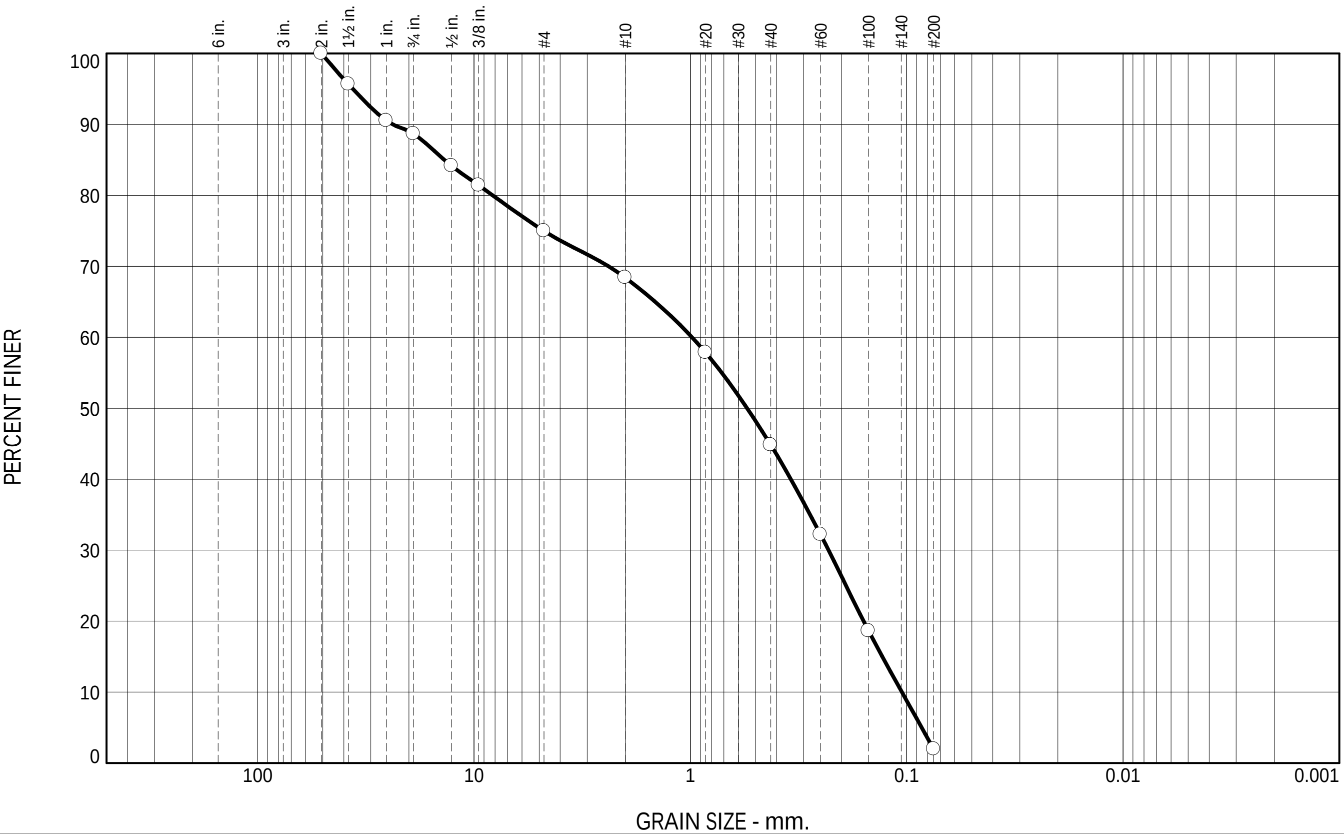
4-Pole Resistivity tested by RB on 7/6/23.

Date Received: 07.03.23

Reviewed By:

Date Reviewed: 07.10.23

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	11.3	13.7	6.6	23.6	42.9	1.9	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1 1/2"	95.7		
1"	90.5		
3/4"	88.7		
1/2"	84.2		
3/8"	81.4		
#4	75.0		
#10	68.4		
#20	57.8		
#40	44.8		
#60	32.2		
#100	18.6		
#200	1.9		

* (no specification provided)

Soil Description

Brown poorly graded sand with gravel

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 23.7988 D_{85}= 13.7048 D_{60}= 0.9832
D_{50}= 0.5468 D_{30}= 0.2298 D_{15}= 0.1297
D_{10}= 0.1053 C_u= 9.34 C_c= 0.51

Classification

USCS= SP AASHTO= A-1-b

Remarks

Source of Sample: Face of Steep Slope Depth: 0-1
Sample Number: GS-1

Date: 07.07.23

Thielsch Engineering Inc.

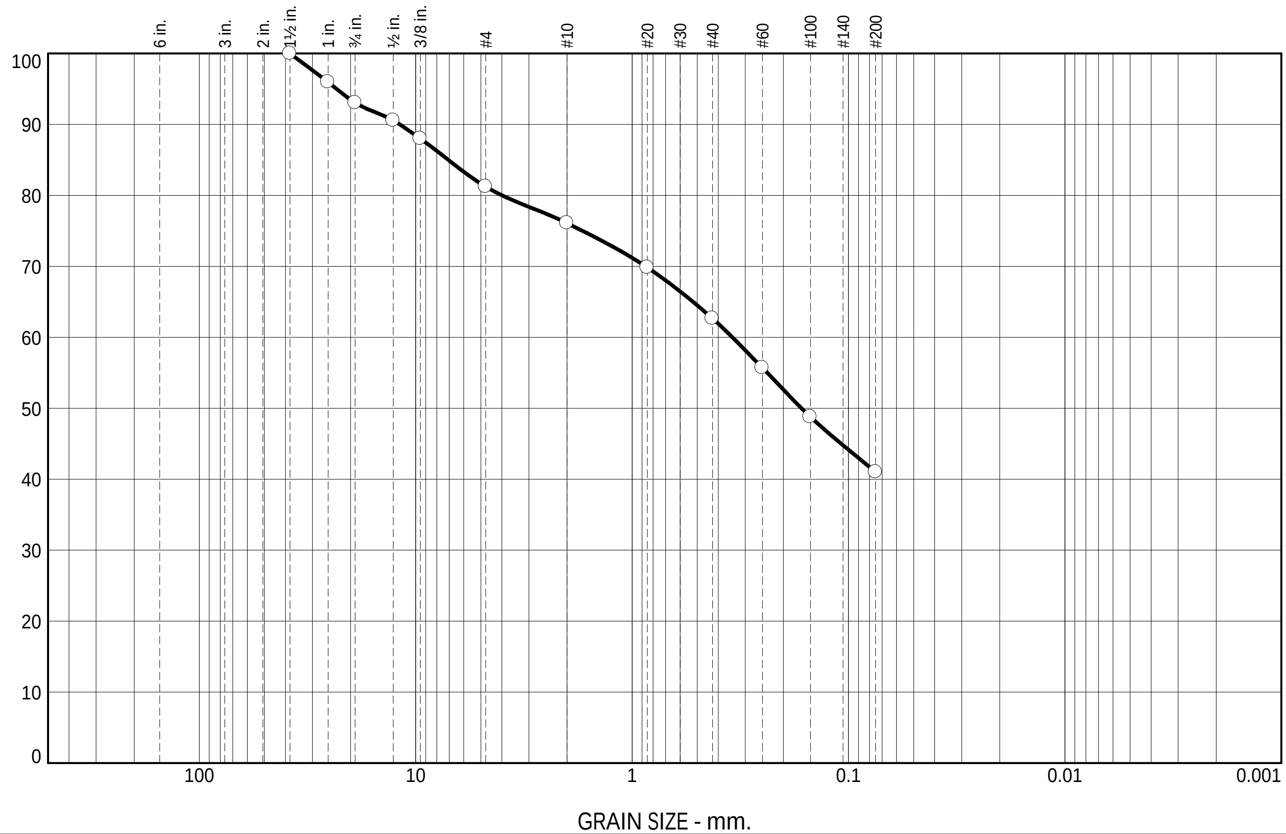
Cranston, RI

Client: Civil & Environmental Consultants, Inc
Project: Slope Stabilization
51 Harborview Rd, Hull MA
Project No: 324-891 Fig. 23-S-2805

Tested By: GO / SP / JB Checked By:

These results are for the exclusive use of the client for whom they were obtained. This report only relates to items inspect and/or tested. No warranty, expressed or implied, is made.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.0	11.8	5.1	13.5	21.6	41.0	

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	96.0		
3/4"	93.0		
1/2"	90.6		
3/8"	88.0		
#4	81.2		
#10	76.1		
#20	69.8		
#40	62.6		
#60	55.7		
#100	48.8		
#200	41.0		

* (no specification provided)

Soil Description

Brown silty sand with gravel

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 11.8336 D_{85}= 7.0685 D_{60}= 0.3442
D_{50}= 0.1650 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Source of Sample: Top of Wall Depth: 2-3'
Sample Number: AS-1

Date: 07.10.23

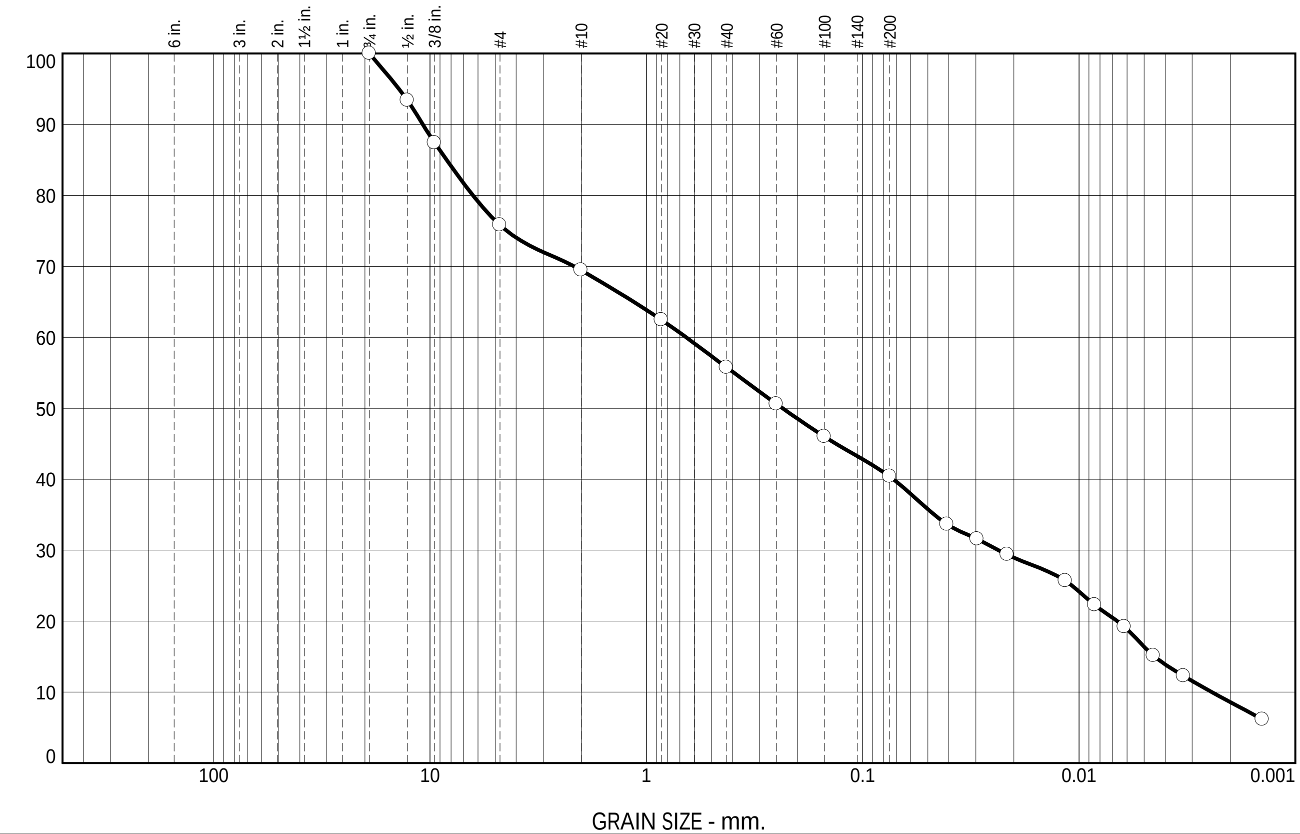
Thielsch Engineering Inc.

Cranston, RI

Client: Civil & Environmental Consultants, Inc
Project: Slope Stabilization
51 Harborview Rd, Hull MA
Project No: 324-891 Fig. 23-S-2806

Tested By: GO / SP / JB Checked By:

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	24.2	6.3	13.8	15.3	31.8	8.6

SIEVE SIZE OR DIAMETER	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	93.4		
3/8"	87.4		
#4	75.8		
#10	69.5		
#20	62.4		
#40	55.7		
#60	50.6		
#100	46.0		
#200	40.4		
0.0406 mm.	33.6		
0.0295 mm.	31.5		
0.0214 mm.	29.3		
0.0115 mm.	25.7		
0.0084 mm.	22.2		
0.0062 mm.	19.2		
0.0045 mm.	15.1		
0.0033 mm.	12.3		
0.0014 mm.	6.1		

* (no specification provided)

Soil Description

Brown clayey silty sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 10.7967 D_{85}= 8.3763 D_{60}= 0.6550
D_{50}= 0.2348 D_{30}= 0.0236 D_{15}= 0.0045
D_{10}= 0.0024 C_u= 268.80 C_c= 0.35

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Sample visually classified as plastic. Sample rolled to 1/4"

Source of Sample: Boring Depth: 15-16.4'
Sample Number: B-1 / S-4

Date: 07.07.23

Thielsch Engineering Inc.

Cranston, RI

Client: Civil & Environmental Consultants, Inc
Project: Slope Stabilization
51 Harborview Rd, Hull MA
Project No: 324-891 Fig. 23-S-2807

Tested By: RB / SL Checked By:

CERTIFICATE OF ANALYSIS

Kris Roland
Thielsch Engineering, Inc.
CTS Cranston
Cranston, RI 02910

RE: Slope Stabilization - CEC (74-23-0002.202)
ESS Laboratory Work Order Number: 23G0006

This signed Certificate of Analysis is our approved release of your analytical results. These results are only representative of sample aliquots received at the laboratory. ESS Laboratory expects its clients to follow all regulatory sampling guidelines. Beginning with this page, the entire report has been paginated. This report should not be copied except in full without the approval of the laboratory. Samples will be disposed of thirty days after the final report has been delivered. If you have any questions or concerns, please feel free to call our Customer Service Department.

Laurel Stoddard
Laboratory Director

REVIEWED
By ESS Laboratory at 5:34 pm, Jul 11, 2023

Analytical Summary
The project as described above has been analyzed in accordance with the ESS Quality Assurance Plan. This plan utilizes the following methodologies: US EPA SW-846, US EPA Methods for Chemical Analysis of Water and Wastes per 40 CFR Part 136, APHA Standard Methods for the Examination of Water and Wastewater, American Society for Testing and Materials (ASTM), and other recognized methodologies. The analyses with these noted observations are in conformance to the Quality Assurance Plan. In chromatographic analysis, manual integration is frequently used instead of automated integration because it produces more accurate results.

The test results present in this report are in compliance with TNI and relative state standards, and/or client Quality Assurance Project Plans (QAPP). The laboratory has reviewed the following: Sample Preservations, Hold Times, Initial Calibrations, Continuing Calibrations, Method Blanks, Blank Spikes, Blank Spike Duplicates, Duplicates, Matrix Spikes, Matrix Spike Duplicates, Surrogates and Internal Standards. Any results which were found to be outside of the recommended ranges stated in our SOPs will be noted in the Project Narrative.

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Slope Stabilization - CEC

ESS Laboratory Work Order: 23G0006

SAMPLE RECEIPT

The following samples were received on July 03, 2023 for the analyses specified on the enclosed Chain of Custody Record.

The client did not deliver the samples in a cooler.

Lab Number	Sample Name	Matrix	Analysis
23G0006-01	GS-1	Soil	D4327

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Slope Stabilization - CEC

ESS Laboratory Work Order: 23G0006

PROJECT NARRATIVE

No unusual observations noted.

End of Project Narrative.

DATA USABILITY LINKS

To ensure you are viewing the most current version of the documents below, please clear your internet cookies for www.ESSLaboratory.com. Consult your IT Support personnel for information on how to clear your internet cookies.

Definitions of Quality Control Parameters

Semivolatile Organics Internal Standard Information

Semivolatile Organics Surrogate Information

Volatile Organics Internal Standard Information

Volatile Organics Surrogate Information

EPH and VPH Alkane Lists

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Slope Stabilization - CEC

ESS Laboratory Work Order: 23G0006

CURRENT SW-846 METHODOLOGY VERSIONS

Analytical Methods

1010A - Flashpoint
6010C - ICP
6020A - ICP MS
7010 - Graphite Furnace
7196A - Hexavalent Chromium
7470A - Aqueous Mercury
7471B - Solid Mercury
8011 - EDB/DBCP/TCP
8015C - GRO/DRO
8081B - Pesticides
8082A - PCB
8100M - TPH
8151A - Herbicides
8260B - VOA
8270D - SVOA
8270D SIM - SVOA Low Level
9014 - Cyanide
9038 - Sulfate
9040C - Aqueous pH
9045D - Solid pH (Corrosivity)
9050A - Specific Conductance
9056A - Anions (IC)
9060A - TOC
9095B - Paint Filter
MADEP 04-1.1 - EPH
MADEP 18-2.1 - VPH

Prep Methods

3005A - Aqueous ICP Digestion
3020A - Aqueous Graphite Furnace / ICP MS Digestion
3050B - Solid ICP / Graphite Furnace / ICP MS Digestion
3060A - Solid Hexavalent Chromium Digestion
3510C - Separatory Funnel Extraction
3520C - Liquid / Liquid Extraction
3540C - Manual Soxhlet Extraction
3541 - Automated Soxhlet Extraction
3546 - Microwave Extraction
3580A - Waste Dilution
5030B - Aqueous Purge and Trap
5030C - Aqueous Purge and Trap
5035A - Solid Purge and Trap

SW846 Reactivity Methods 7.3.3.2 (Reactive Cyanide) and 7.3.4.1 (Reactive Sulfide) have been withdrawn by EPA. These methods are reported per client request and are not NELAP accredited.

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Slope Stabilization - CEC
Client Sample ID: GS-1
Date Sampled: 07/03/23 09:30
Percent Solids: 95

ESS Laboratory Work Order: 23G0006
ESS Laboratory Sample ID: 23G0006-01
Sample Matrix: Soil

Classical Chemistry

Analyte	Results (MRL)	MDL	Method	Limit	DF	Analyst	Analyzed	Units	Batch
Chloride	**WL 16** (5)		D4327		1	EEM	07/05/23 21:29	mg/kg dry	DG30529
Sulfate	**WL 13** (5)		D4327		1	EEM	07/05/23 21:29	mg/kg dry	DG30529

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Slope Stabilization - CEC

ESS Laboratory Work Order: 23G0006

Quality Control Data

Analyte	Result	MRL	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Qualifier
Classical Chemistry										
Batch DG30529 - General Preparation										
Blank										
Chloride	ND	5	mg/kg wet							
Sulfate	ND	5	mg/kg wet							
LCS										
Chloride	10		mg/L	10.00		101	85-115			
Sulfate	10		mg/L	10.00		101	80-120			

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Slope Stabilization - CEC

ESS Laboratory Work Order: 23G0006

Notes and Definitions

WL	Results obtained from a deionized water leach of the sample.
U	Analyte included in the analysis, but not detected
ND	Analyte NOT DETECTED at or above the MRL (LOQ), LOD for DoD Reports, MDL for J-Flagged Analytes
dry	Sample results reported on a dry weight basis
RPD	Relative Percent Difference
MDL	Method Detection Limit
MRL	Method Reporting Limit
LOD	Limit of Detection
LOQ	Limit of Quantitation
DL	Detection Limit
I/V	Initial Volume
F/V	Final Volume
§	Subcontracted analysis; see attached report
1	Range result excludes concentrations of surrogates and/or internal standards eluting in that range.
2	Range result excludes concentrations of target analytes eluting in that range.
3	Range result excludes the concentration of the C9-C10 aromatic range.
Avg	Results reported as a mathematical average.
NR	No Recovery
[CALC]	Calculated Analyte
SUB	Subcontracted analysis; see attached report
RL	Reporting Limit
EDL	Estimated Detection Limit
MF	Membrane Filtration
MPN	Most Probable Number
TNTC	Too numerous to Count
CFU	Colony Forming Units

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Slope Stabilization - CEC

ESS Laboratory Work Order: 23G0006

ESS LABORATORY CERTIFICATIONS AND ACCREDITATIONS

ENVIRONMENTAL

Rhode Island Potable and Non Potable Water: LAI00179
http://www.health.ri.gov/find/labs/analytical/ESS.pdf

Connecticut Potable and Non Potable Water, Solid and Hazardous Waste: PH-0750
http://www.ct.gov/dph/lib/dph/environmental_health/environmental_laboratories/pdf/OutofStateCommercialLaboratories.pdf

Maine Potable and Non Potable Water, and Solid and Hazardous Waste: RI00002
http://www.maine.gov/dhhs/mecdc/environmental-health/dwp/partners/labCert.shtml

Massachusetts Potable and Non Potable Water: M-RI002
http://public.dep.state.ma.us/Labcert/Labcert.aspx

New Hampshire (NELAP accredited) Potable and Non Potable Water, Solid and Hazardous Waste: 2424
http://des.nh.gov/organization/divisions/water/dwgb/nhelap/index.htm

New York (NELAP accredited) Non Potable Water, Solid and Hazardous Waste: 11313
http://www.wadsworth.org/labcert/elap/comm.html

New Jersey (NELAP accredited) Non Potable Water, Solid and Hazardous Waste: RI006
http://datamine2.state.nj.us/DEP_OPRA/OpraMain/pi_main?mode=pi_by_site&sort_order=PI_NAMEA&Select+a+Site:=58715

Pennsylvania: 68-01752
http://www.dep.pa.gov/Business/OtherPrograms/Labs/Pages/Laboratory-Accreditation-Program.aspx

ESS Laboratory Sample and Cooler Receipt Checklist

Client: Thielsch Engineering, Inc - ESS

Shipped/Delivered Via: Client

ESS Project ID: 23G0006
Date Received: 7/3/2023
Project Due Date: 7/11/2023
Days for Project: 5 Day

1. Air bill manifest present? No
   Air No.: NA
2. Were custody seals present? No
3. Is radiation count <100 CPM? Yes
4. Is a Cooler Present? No
   Temp: 23 Iced with: None
5. Was COC signed and dated by client? Yes

6. Does COC match bottles? Yes
7. Is COC complete and correct? Yes
8. Were samples received intact? Yes
9. **Were labs informed about short holds & rushes?** Yes / No / NA
10. Were any analyses received outside of hold time? Yes / No

11. Any Subcontracting needed? Yes / No
    ESS Sample IDs:
    Analysis:
    TAT:

12. Were VOAs received? Yes / No
    a. Air bubbles in aqueous VOAs? Yes / No
    b. Does methanol cover soil completely? Yes / No / NA

13. Are the samples properly preserved? Yes / No
    a. If metals preserved upon receipt: Date: Time: By/Acid Lot#:
    b. Low Level VOA vials frozen: Date: Time: By:

Sample Receiving Notes:

No cooling metals above 6

14. Was there a need to contact Project Manager? Yes / No
    a. Was there a need to contact the client? Yes / No
    Who was contacted? Date: Time: By:

Resolution:

Sample Number	Container ID	Proper Container	Air Bubbles Present	Sufficient Volume	Container Type	Preservative	Record pH (Cyanide and 608 Pesticides)
1	449644	Yes	N/A	Yes	8 oz jar	NP	

2nd Review
Were all containers scanned into storage/lab? Initials e
Are barcode labels on correct containers? Yes / No
Are all Flashpoint stickers attached/container ID # circled? Yes / No / NA
Are all Hex Chrome stickers attached? Yes / No / NA
Are all QC stickers attached? Yes / No / NA
Are VOA stickers attached if bubbles noted? Yes / No / NA

Completed By: [signature] Date & Time: 7/3/23 10:46
Reviewed By: Clayton Davis Date & Time: 7/3/23 1107

ESS Laboratory
Division of Thielsch Engineering, Inc.
185 Frances Avenue, Cranston, RI 02910-2211
Tel. (401) 461-7181 Fax (401) 461-4486
www.esslaboratory.com

CHAIN OF CUSTODY

ESS LAB PROJECT ID 23G0006

Turn Time: Standard X Rush Approved By: ______

Reporting Limits -

State where samples were collected: MA

Is this project for any of the following: (please circle)
MA-MCP CT-RCP RGP DOD Other________

Electronic Deliverable Yes X No____
Format: Excel___ Access___ PDF X Other_____

Project Manager: Kris Roland
Company: Thielsch Engineering
Address: 195 Frances Ave
Cranston, RI 02910

Project # 74-23-0002.202
Project Name / Client Name:
Slope Stabilization
Civil & Environmental Consultants
Contract Pricing __x___
Special Pricing WO#: ____________

ESS Lab Sample ID	Date	Collection Time	Grab -G Composite-C	Matrix	Sample Identification	# of Container	Sulfate (ASTM D4327)	Chloride (ASTM D4327)								Comment #
1	07.03.23	0930	G	S	GS-1	1	X	X								
Preservation Code: 1-NP, 2-HCl, 3-H2SO4, 4-HNO3, 5-NaOH, 6-MeOH, 7-Asorbic Acid, 8-ZnAct, 9-_CH_3OH____							1	1								
Container Type: P-Poly G-Glass AG-Amber Glass S-Sterile V-VOA							AG	AG								

Matrix: S-Soil SD-Solid D-Sludge WW-Wastewater GW-Groundwater SW-Surface Water DW-Drinking Water O-Oil W-Wipes F-Filter

Cooler Present ______Yes ✓ No
Seals Intact _____ Yes ✓ No NA: _____
Cooler Temperature: 23 No Ice

Sampled by : R. Blake

Comments: Please send reports to kroland@thielsch.com, mcolman@thielsch.com, and rroth@thielsch.com

Relinquished by: (Signature)	Date/Time	Received by: (Signature)	Relinquished by: (Signature)	Date/Time	Received by: (Signature)
Sarah Lefley (CTS)	7.3.23 0949				
Relinquished by: (Signature)	**Date/Time**	**Received by: (Signature)**	**Relinquished by: (Signature)**	**Date/Time**	**Received by: (Signature)**

Please E-mail all changes to Chain of Custody in writing.

Page ____ of ____

ATTACHMENT 11
FHWA SOIL NAIL REFERENCE MANUAL, PUBLICATION NO. FHWA-NHI-14-007, FEBRUARY 2015 (PARTIAL VERSION)

economy to advance the bar and carry spoils to the surface. After the tendon is installed to the desired depth, a heavier final grout mixture is pumped through the hollow bar and the nail is considered complete when the heavier mixture returns to the excavation face, signaling that the lighter drilling grout was flushed from the drill hole and that all drilling spoils have also been removed.

10.3 Bond Strength

Thirty seven HBSNs were installed in four different sites across the U.S. and tested as part of an investigation co-sponsored by FHWA and the International Association of Foundation Drilling (ADSC) (Cadden et al. 2010). The goals of this investigation were to establish appropriate de-bonding procedures for proof-testing HBSNs and to create a database of bond strengths in different ground conditions. Gravity-grouted, solid bar soil nails were also installed and tested for comparison at each of the sites. The results of the tests are summarized in Table 10.1. The bond strength values provided in the table were calculated using the nominal drill bit diameter, not the actual grout body diameter of the HBSNs. Some of the tests did not reach failure; therefore, the range of resistance values for the HBSNs may be somewhat conservative. Note that presumptive values of bond strength, for pressure-grouted micropiles (Sabatini et al. 2005), are provided for similar soil types as a comparison.

Table 10.1: Nominal Bond Strength of Soil Nails in Granular Soils (after Cadden et al. 2010)

Soil Type	Gravity-Grouted Solid Bar Soil Nail (psi)	Hollow Bar Soil Nail (psi)	Pressure-grouted Micropiles (Sabatini et al. 2005) (psi)
Silty Sand (SM)	12.8-17.1	18.7-24.4	Sand (some silt) 10-27.5
Poorly Graded Sand (SP)	13.2	71.2-79.6	Sand (some Gravel) 17.5-52
SP with Gravel	50.5-82.2	-	Sand (some Gravel) 17.5-52
Poorly Graded Gravel (GP) with Sand	53.2-64.5	62.4-156.3	Gravel (some Sand) 17.5-52

Note: Bond strength values from tests are calculated using the nominal drill bit diameter.

The tests showed that HBSNs tend to develop larger bond strength values in granular soils than traditional, gravity-grouted solid bar nails. This is due to the larger drill hole diameter,

Table A.1a: Properties of Solid-Threaded Bars - Grade 60

Bar Designation	Maximum Diameter (w/ threads)	Minimum Cross-Sectional Area	Unit Weight	ASTM Grade	Yield Stress	Yield Load
Conventional	inch	inch^2	lb/ft	Conventional	ksi	kip
#6	0.86	0.44	1.50	60	60	26
#7	0.99	0.60	2.04	60	60	36
#8	1.12	0.79	2.67	60	60	47
#9	1.26	1.00	3.40	60	60	60
#10	1.43	1.27	4.30	60	60	76
#11	1.61	1.56	5.31	60	60	93
#14	1.86	2.25	7.65	60	60	135

Table A.1b: Properties of Solid-Threaded Bars - Grade 75

Bar Designation	Maximum Diameter (w/ threads)	Minimum Cross-Sectional Area	Unit Weight	ASTM Grade	Yield Stress	Yield Load
Conventional	inch	inch^2	lb/ft	Conventional	ksi	kip
#6	0.86	0.44	1.50	75	75	33
#7	0.99	0.60	2.04	75	75	45
#8	1.12	0.79	2.67	75	75	59
#9	1.26	1.00	3.40	75	75	75
#10	1.43	1.27	4.30	75	75	95
#11	1.61	1.56	5.31	75	75	117
#14	1.86	2.25	7.65	75	75	168

Sources: Dywidag, Williams and Contech

Table A.1c: Properties of Solid-Threaded Bars - Grade 150

Bar Designation (inch)	Maximum Diameter (w/ threads)	Minimum Cross-Sectional Area	Unit Weight	ASTM Grade	Ultimate Load [(1)]
Conventional	**inch**	**$inch^2$**	**lb/ft**	**Conventional**	**kip**
¾	0.75	0.37	1.32	150	57
1	1.00	0.85	3.01	150	128
1¼	1.25	1.25	4.39	150	188
1⅜	1.375	1.58	5.56	150	237
1¾	1.75	2.58	9.22	150	400

Sources: Dywidag, Williams and Stressteel

Note: (1) Based on ultimate strength.

Table A.2: Properties of Hollow Bars

CTS/Titan Bar Type

Bar Designation	Effective Outer Diameter	Cross-Sectional Area	Unit Weight	Yield Load (1)	Ultimate Load
Conventional	**inch**	**$inch^2$**	**lb/ft**	**kip**	**kip**
30/16	1.02	0.53	1.8	42.7	55.1
30/14	1.03	0.58	1.9	49.5	61.8
30/11	1.03	0.64	2.2	58.5	72.0
40/20	1.42	1.13	3.8	95.6	121.4
40/16	1.42	1.40	4.8	118.1	148.4
52/26	1.92	1.94	6.7	164.2	208.0
73/56	2.76	2.11	7.3	186.6	232.7
73/53	2.76	2.50	8.9	218.1	260.9
73/45	2.76	3.50	12.0	285.6	356.4
73/35	2.76	4.20	14.2	321.6	419.4

Source: Contech

Note: (1) These bars are alloy steel and have nominal yield strength of 87 ksi.

Table A.3: Properties of Hollow Bars

MAI Bar Type

Bar Designation	Outer Diameter	Cross-Sectional Area	Weight	Average Yield Stress (1)	Average Ultimate Tensile Stress (1)	Yield Load	Ultimate Load
Conventional	**inch**	**inch^2**	**lb/ft**	**ksi**	**ksi**	**kip**	**kip**
R25N	1.00	0.41	1.41	83	111	34	45
R32N	1.26	0.56	1.88	92	112	52	63
R32S	1.26	0.72	2.49	87	112	63	81
R38N	1.50	1.00	3.43	89	112	90	112
R51L	1.50	1.17	3.97	86	106	101	124
T40N	1.57	1.32	4.50	89	112	118	148
R51N	2.00	1.61	5.44	88	112	142	180
T76N	3.00	3.32	11.29	81	108	270	360
T76S	3.00	3.88	13.24	87	110	337	427

Source: Dywidag

Note: (1) The yield strength and ultimate tensile stresses of these bars vary as indicated above, per manufacturer's information.

Table A.4: Properties of Hollow Bars

GEO-Drill Bar Type

Bar Designation	Nominal Outer Diameter	Cross-Sectional Area	Weight	Yield Load (1)	Ultimate Load (1)
Conventional	**inch**	**inch^2**	**lb/ft**	**kip**	**kip**
B7X1-32	1.25	0.556	2.10	47.2	58.4
B7X1-32X	1.25	0.776	2.70	66.0	81.5
B7X1-38	1.50	1.067	3.76	90.7	112.0
B7X1-51	2.00	1.795	6.26	152.0	188.0
B7X1-76	3.00	3.880	13.79	329.0	407.0

Source: Williams

Note: (1) These bars have a yield strength of 85 ksi and an ultimate strength of 105 ksi.

Table A.5: Welded-Wire Mesh (1)

Mesh Designation(2) (3) (4)	Cross-Sectional Area per Unit Width (5)	Weight per Unit Area
in. × in. – in.^{2}/100 × in.^{2}/100	in.^{2}/ft	lb/cft^2 (6)
4x4 - W 1.4 x W 1.4	0.042	31
4x4 – W 2.0 x W 2.0	0.060	44
4x4 – W 2.9 x W 2.9	0.087	62
4x4 – W 4.0 x W 4.0	0.120	88
6x6 – W 1.4 x W 1.4	0.028	21
6x6 – W 2.0 x W 2.0	0.040	30
6x6 – W 2.9 x W 2.9	0.058	42
6x6 – W 4.0 x W 4.0	0.080	58

Source: WRI (2006)

Notes:
(1) These properties are applicable for WWM in Grades 60 or 65. Consult WRI (2006) for WWM properties for Grades 70, 75, and 80.
(2) The first pair of numbers (e.g., 4x4) indicate the mesh opening size (or wire spacing) in the longitudinal and transverse direction, respectively.
(3) Prefix "W" indicates plain wire. A prefix "D" would indicate pre-deformed wire.
(4) The second pair of numbers (e.g., 1.4x1.4) indicates the cross-sectional area of wires (in square inches x 100) in the longitudinal and transverse direction, respectively.
(5) The area per unit width is obtained by dividing the wire cross-sectional area by 100 and by the mesh opening size (or wire spacing, in feet). If the longitudinal and transverse spacings and wire sizes are different, the cross-sectional area per unit width will be different in each direction. Consult WRI (2006) for additional information.
(6) CSF = 100 square feet.

Table A.6: Reinforcing Bars

Bar Designation	Nominal Diameter	Nominal Area
#	**inch**	**inch^2**
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56
14	1.693	2.25
18	2.257	4.00

Source: ACI 318-11

Table A.7: Headed-Studs

Headed Stud Size	Nominal Length (1) (2)	Head Diameter	Shaft Diameter	Head Thickness	Head Area / Shaft Area	Head Thickness / (Head Diameter- Shaft Diameter)
-	L_S	D_{SH}	D_{SC}	t_{SH}	A_{SH} / A_{SC}	t_{SH} / (D_{SH} - D_{SC})
-	inch	inch	inch	inch	-	-
¼ x 4⅛	4⅛	0.50	0.25	0.19	4.0	0.75
⅜ x 4⅛	4⅛	0.75	0.38	0.28	4.0	0.75
⅜ x 6⅛	6⅛	0.75	0.38	0.28	4.0	0.75
½ x 4⅛	4⅛	1.00	0.50	0.28	4.0	0.56
½ x $5\frac{5}{16}$	$5\frac{5}{16}$	1.00	0.50	0.28	4.0	0.56
½ x 6⅛	6⅛	1.00	0.50	0.28	4.0	0.56
⅝ x $6\frac{9}{16}$	$6\frac{9}{16}$	1.25	0.63	0.31	4.0	0.50
¾ x $3\frac{11}{16}$	$3\frac{11}{16}$	1.25	0.75	0.38	2.8	0.75
¾ x $4\frac{3}{16}$	$4\frac{3}{16}$	1.25	0.75	0.38	2.8	0.75
¾ x $5\frac{3}{16}$	$5\frac{3}{16}$	1.25	0.75	0.38	2.8	0.75
¾ x $6\frac{3}{16}$	$6\frac{3}{16}$	1.25	0.75	0.38	2.8	0.75
⅞ x $4\frac{3}{16}$	$4\frac{3}{16}$	1.40	0.86	0.38	2.5	0.73
⅞ x $5\frac{3}{16}$	$5\frac{3}{16}$	1.40	0.86	0.38	2.5	0.73
⅞ x $6\frac{3}{16}$	$6\frac{3}{16}$	1.40	0.86	0.38	2.5	0.73

Source: Stud Welding Associates, Inc.

Notes:
(1) Nominal length indicated is before welding.
(2) After-welding lengths to be considered as follows: For $D_S \leq$ ½", L_S is approximately ⅛" shorter after welding. For $D_S >$ ⅝"L_S is approximately 3/16" shorter after welding.

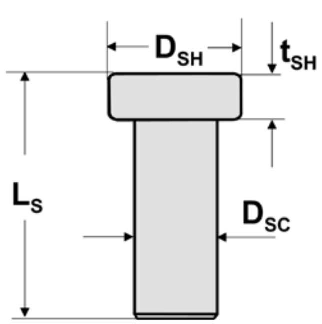


Figure A.1: Illustration. Geometry of headed studs.

Table 5.1: Minimum Recommended Factors of Safety for the Design of Soil Nail Walls Using the ASD Method [1]

Limit State	Condition	Symbol	Minimum Recomm. Factors of Safety, Static Loads	Minimum Recomm. Factors of Safety, Seismic Loads
Overall	Overall Stability	FS_{OS}	1.5[2]	1.1[6]
Overall	Short Term Condition, Excavation	FS_{OS}	1.25-1.33[3]	NA
Overall	Basal Heave	FS_{BH}	2.0[4], 2.5[5]	2.3[5]
Strength – Geotechnical	Pullout Resistance	FS_{PO}	2.0	1.5
Strength – Geotechnical	Lateral Sliding	FS_{LS}	1.5	1.1
Strength – Structural	Tendon Tensile Strength (Grades 60 and 75)	FS_{T}	1.8	1.35
Strength – Structural	Tendon Tensile Strength (Grades 95 and 150)	FS_{T}	2.0	1.50
Strength – Structural	Facing Flexural	FS_{FF}	1.5	1.1
Strength – Structural	Facing Punching Shear	FS_{FP}	1.5	1.1
Strength – Structural	Headed Stud Tensile (A307 Bolt)	FS_{FH}	2.0	1.5
Strength – Structural	Headed Stud Tensile (A325 Bolt)	FS_{FH}	1.7	1.3

Notes: (1) The limit state and symbol nomenclature differ from that presented in the previous version of this manual. Many of these changes reflect the move toward using LRFD terminology as presented in AASHTO (2014).

(2) For non-critical, permanent structures, some Owners may accept a design for static loads and long-term conditions with FS_{OS} = 1.35 when uncertainty is considered to be limited due to the availability of sufficient geotechnical information and successful local experience on soil nailing.

(3) This range of safety factors for global stability corresponds to the case of temporary excavation lifts that are unsupported for up to 2 days before nails are installed. The larger value may be applied to critical structures or when more uncertainty exists regarding soil conditions.

(4) This factor of safety for basal heave is applicable to permanent walls for short-term conditions.

(5) This factor of safety for basal heave is applicable to permanent walls for long-term conditions.

(6) The minimum FS_{OS} for seismic overall stability should be 1.0, when horizontal seismic coefficients are used and these were derived from estimated, allowable seismic deformations.

6.6, the soil pressure is smaller between nails, where the displacements are larger and tend to produce a stress relief. The soil pressure immediately behind the nails is larger than that in the mid-span because the soil confinement is greater.

Eqs. 6.9 through 6.14 are also valid for the final facing provided the parameters of the equations (thickness and material strengths) are selected appropriately for the final facing.

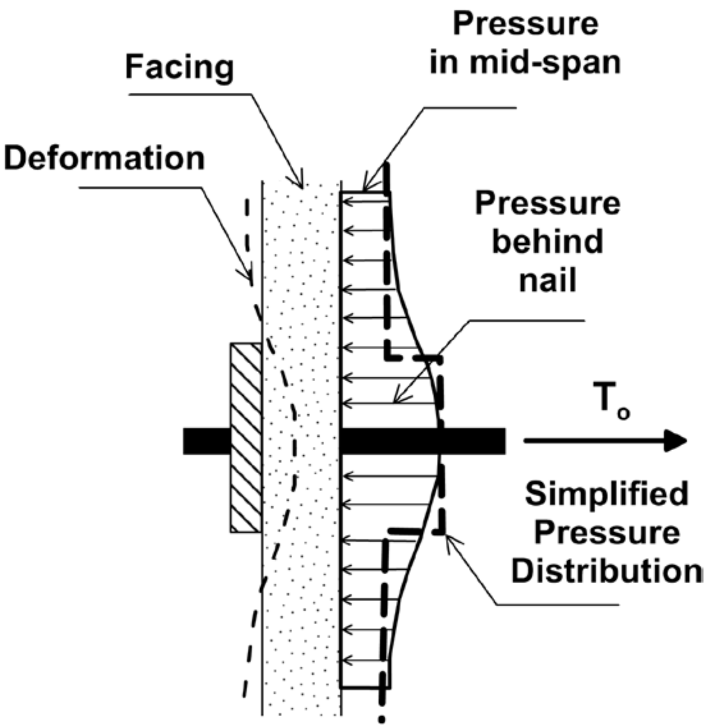


Figure 6.6: Illustration. Soil pressure distribution behind facing. Modified after Byrne et al. (1998).

If Grade 60 steel is used for WWM and rebar, and f'_c = 4,000 psi for shotcrete, the following simplified equations can be used:

$$R_{FF}\left[\text{kip}\right] = 3.8 \times C_F \times f_y\left[\text{ksi}\right] \times F = 228 \times C_F \times F$$

Equation 6.15: Nominal bending resistance for Grade 60 steel WWM/rebar and 4,000 psi shotcrete.

Where:

$$F = \text{smaller of} \begin{cases} \left(a_{vn} + a_{vm}\right)\left[in^2/ft\right] \times \left(\dfrac{S_H\, h\,[ft]}{S_V}\right) \\ \\ \left(a_{hn} + a_{hm}\right)\left[in^2/ft\right] \times \left(\dfrac{S_V\, h\,[ft]}{S_H}\right) \end{cases}$$

Equation 6.16: Definition of F.

Assuming that a_{vn} = a_{vm} = and a_{hn} = a_{hm} = a_s, and S_H = S_V, the following equation can be used for the final facing, where C_F = 1.0:

$$R_{FF}\left[kip\right] = 456 \times a_s \left[in^2/ft\right] \times h_f\,[ft]$$

Equation 6.17: Nominal bending resistance.

C_F is selected from Table 6.5.

Table 6.5: Factor C_F

Facing	**Facing Thickness, h_i or h_f (in.)**	**C_F**
Initial	4	2.0
Initial	6	1.5
Initial	8	1.0
Final	All	1.0

The facing flexural nominal resistance (R_{FF}) for the initial facing is calculated using Equations 6.15 or 6.17. Alternatively, R_{FF} can be estimated from Table 6.6 using results for initial calculations (Section 6.6.5a).

Table 6.6: Nominal Flexure Resistance, R_{FF} (Initial Facing)

Facing thickness, h_i	Nail Spacing Ratio[1]	ρ_{tot} = 0.5 % [2]	ρ_{tot} = 1.0 % [2]	ρ_{tot} = 2.0 % [2]
(in.)	(ft/ft)	R_{FF} (kip) [3] [5]	R_{FF} (kip) [4] [5]	R_{FF} (kip) [4] [5]
4	0.67	12	24	48
4	1	18	36	71
6	0.67	20	40	81
6	1	30	60	120
8	0.67	24	48	95
8	1	36	71	143

(1) Nail spacing ratio = smaller of S_V/S_H or S_H/S_V.
(2) $\rho_{tot} = \rho_n + \rho_m$, ρ_n and ρ_m are the nail head and mid-span reinforcement ratios, respectively, in either direction.
(3) The above values are valid for Grade 60 steel. Multiply the values above by 1.24 for Grade 75 steel.
(4) Divide R_{FF} by 2 for final facing thickness h_f = 4 in. Divide R_{FF} by 1.5. for h_f = 6 in. For h_f = 8 in., use same R_{FF}.

<u>Final Facing</u>

The same steps shown above for the initial facing must be completed for the final facing, considering the appropriate reinforcement, section thickness and material properties.

The facing resistance is verified in LRFD for each of the facings as follows:

$$CDR = \frac{\phi_{FF}\ R_{FF}}{\gamma\ T_o} \geq 1.0$$

Equation 6.18: Capacity-to-demand ratio for bending in facing.

Where:

ϕ_{FF} = resistance factor for bending/flexure in the facing

R_{FF} = nominal resistance for bending/flexure of facing

γ = load factor selected for verifications

T_o = maximum tensile force at soil nail head, as estimated with Eq. 5.1 (Section 5.2.1)

If the resistance is insufficient, increase the thickness of facing, amount of steel, and/or strength of steel and/or of concrete.

6.6.6 Step 7e Verify Facing Punching Shear Resistance

For programs that do not have the capability of verifying the punching shear resistance, the designer must verify the punching shear resistance at the facing using the procedure presented below.

Connectors installed at the nail head may be subjected to a punching shear limit state, which may occur if the nominal shear resistance of the reinforced shotcrete/concrete section around the nails is exceeded. The nominal punching shear resistance must be evaluated for both initial and final facings (Figure 6.7) for:

- Initial facing: bearing plate connection
- Final facing: headed stud connection

At the limit state, a conical shear surface forms in the facing around the nail head. The size of the conical shear surface is affected by the thickness of the facing, and the dimension of the bearing plate (initial facing) and headed studs (final facing).

The nominal facing punching shear resistance, R_{FP}, for either situation must meet the following condition:

$$CDR = \frac{\phi_{FP} \ R_{FP}}{\gamma \ T_o} \geq 1.0$$

Equation 6.19: Capacity-to-demand ratio for punching shear resistance.

Where:

ϕ_{FP} = resistance factor for punching shear in the facing

γ = load factor selected for verification

T_o = maximum tensile force at soil nail head, as defined previously

R_{FP} is estimated as:

$$R_{FP} = C_P \ V_F$$

Equation 6.20: Nominal punching shear resistance at facing.

Where:

C_P = dimensionless factor that accounts for the contribution of the soil support under the nail head to the shear resistance

V_F = concrete punching shear basic resistance acting through the facing section

C_P can be as high as 1.15 if the soil reaction is considered. The contribution from the soil support behind the wall is conservatively assumed to be negligible; therefore, $C_P = 1.0$. The punching shear resistance, V_F, can be calculated as:

$$V_F\left[\text{kip}\right] = 0.58\sqrt{f'_c\ [\text{psi}]}\ \pi\ D'_c[\text{ft}]h_c[\text{ft}]$$

Equation 6.21: Nominal punching shear resistance through facing.

Where:

f'_c = as defined before

D'_c = effective equivalent diameter of the conical slip surface

h_c = effective depth of the conical surface

D'_c and h_c must be selected separately for the initial and final facing, as follows:

<u>Initial facing (Figure 6.7a)</u>

$$D'_c = L_{BP} + h_i$$

Equation 6.22: Effective, equivalent diameter of conical slip surface through initial facing.

$$h_c = h_i$$

Equation 6.23: Effective depth of the conical slip surface, initial facing.

Where:

L_{BP} = bearing plate size

h_i = thickness of initial facing

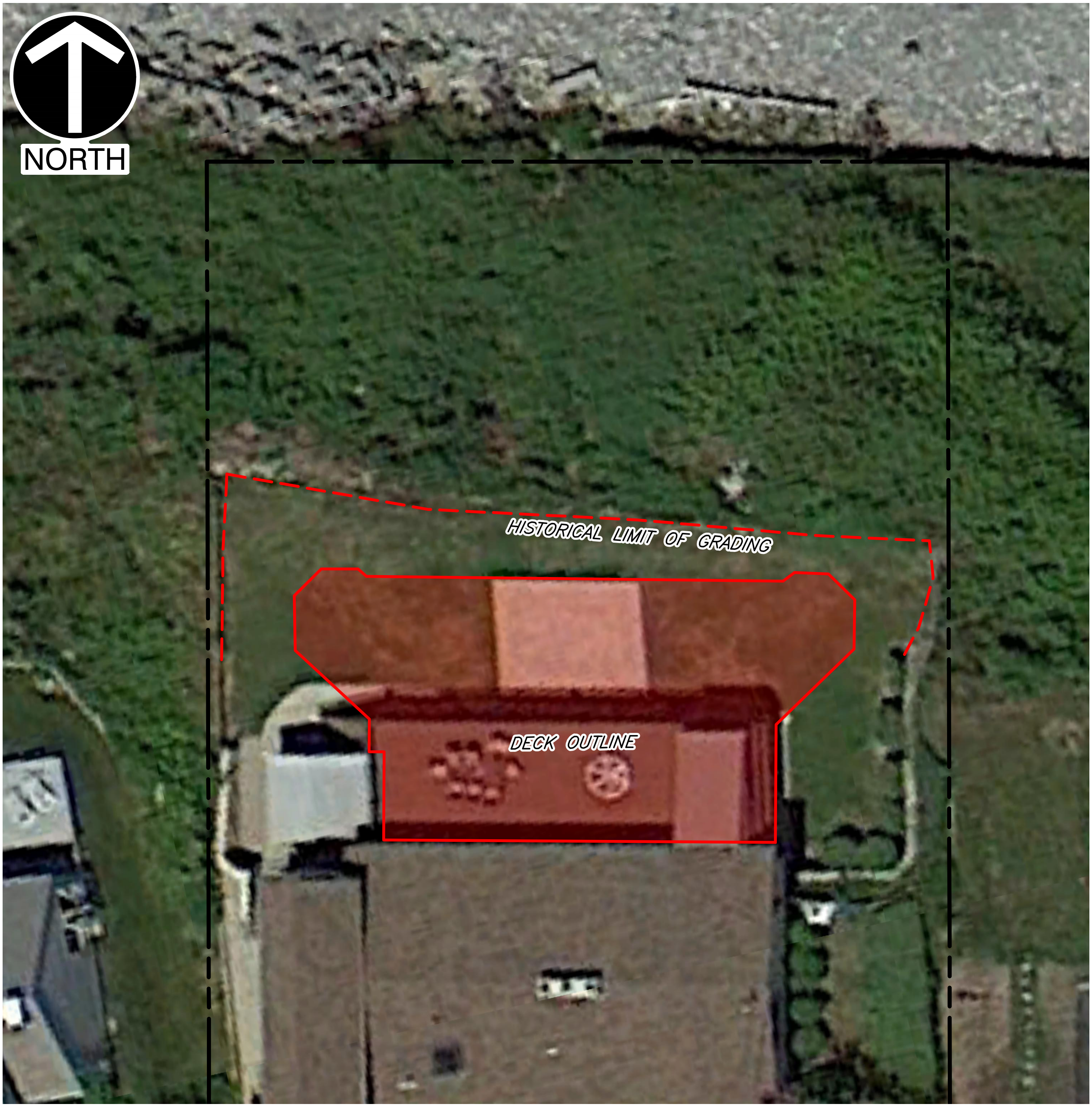
ATTACHMENT 12
WILLIAMS FORM ENGINEERING – GRADE 75 ALL-THREAD REBAR PROPERTIES

GRADE 75 & GRADE 80 ALL-THREAD REBAR

BAR DESIGNATION NOMINAL DIAMETER & PITCH	MINIMUM NET AREA THRU THREADS	MINIMUM ULTIMATE STRENGTH	GRADE 75 MINIMUM YIELD STRENGTH	GRADE 80 MINIMUM YIELD STRENGTH	NOMINAL WEIGHT	APPROXIMATE THREAD MAJOR DIAMETER	PART NUMBER
#6 – 3/4″ – 5 (19 mm)	0.44 in^2 (284 mm^2)	44 kips (196 kN)	33 kips (147 kN)	35 kips (156 kN)	1.5 lbs/ft (2.4 kg/m)	7/8″ (22 mm)	R61-06
#7 – 7/8″ – 5 (22 mm)	0.60 in^2 (387 mm^2)	60 kips (267 kN)	45 kips (200 kN)	48 kips (214 kN)	2.0 lbs/ft (3.0 kg/m)	1″ (25 mm)	R61-07
#8 – 1″ – 3-1/2 (25 mm)	0.79 in^2 (510 mm^2)	79 kips (351 kN)	59 kips (264 kN)	63 kips (280 kN)	2.7 lbs/ft (3.9 kg/m)	1-1/8″ (29 mm)	R61-08
#9 – 1-1/8″ – 3-1/2 (29 mm)	1.00 in^2 (645 mm^2)	100 kips (445 kN)	75 kips (334 kN)	80 kips (356 kN)	3.4 lbs/ft (5.1 kg/m)	1-1/4″ (32 mm)	R61-09
#10 – 1-1/4″ – 3 (32 mm)	1.27 in^2 (819 mm^2)	127 kips (565 kN)	95 kips (424 kN)	102 kips (454 kN)	4.3 lbs/ft (5.5 kg/m)	1-3/8″ (35 mm)	R61-10
#11 – 1-3/8″ – 3 (36 mm)	1.56 in^2 (1006 mm^2)	156 kips (694 kN)	117 kips (521 kN)	125 kips (556 kN)	5.3 lbs/ft (7.9 kg/m)	1-1/2″ (38 mm)	R61-11
#14 – 1-3/4″ – 3 (43 mm)	2.25 in^2 (1452 mm^2)	225 kips (1001 kN)	169 kips (750 kN)	180 kips (801 kN)	7.65 lbs/ft (11.8 kg/m)	1-7/8″ (48 mm)	R61-14
#18 – 2-1/4″ – 3 (57 mm)	4.00 in^2 (2581 mm^2)	400 kips (1780 kN)	300 kips (1335 kN)	320 kips (1423 kN)	13.6 lbs/ft (19.6 kg/m)	2-7/16″ (62 mm)	R61-18
#20 – 2-1/2″ – 2-3/4 (64 mm)	4.91 in^2 (3168 mm^2)	491 kips (2184 kN)	368 kips (1637 kN)	393 kips (1748 kN)	16.7 lbs/ft (24.8 kg/m)	2-3/4″ (70 mm)	R61-20
#24 – 3″ – 2-3/4 (76 mm) *	6.82 in^2 (4400 mm^2)	682 kips (3034 kN)	512 kips (2277 kN)	546 kips (2429 kN)	24.0 lbs/ft (35.8 kg/m)	3-3/16″ (81 mm)	R61-24
#28 – 3-1/2″ – 2-3/4 (89 mm) *	9.61 in^2 (6200 mm^2)	961 kips (4274 kN)	720 kips (3206 kN)	769 kips (3421 kN)	32.7 lbs/ft (48.6 kg/m)	3-3/4″ (95 mm)	R61-28
#32 – 4″ – 2-3/4 (102 mm) *	12.56 in^2 (8103 mm^2)	1256 kips (5587 kN)	942 kips (4190 kN)	1004 kips (4466 kN)	43.0 lbs/ft (64.0 kg/m)	4-1/4″ (108 mm)	R61-32

ATTACHMENT C

AERIAL OVERLAY EXHIBITS



REFERENCE

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SCALE IN FEET

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Civil & Environmental Consultants, Inc.

31 Bellows Road
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THOMAS P. FITZGERALD
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HULL, MA 02045

2010 AERIAL EXHIBIT

DRAWN BY: WD	CHECKED BY: KPS	APPROVED BY: KPS	FIGURE NO.: A1
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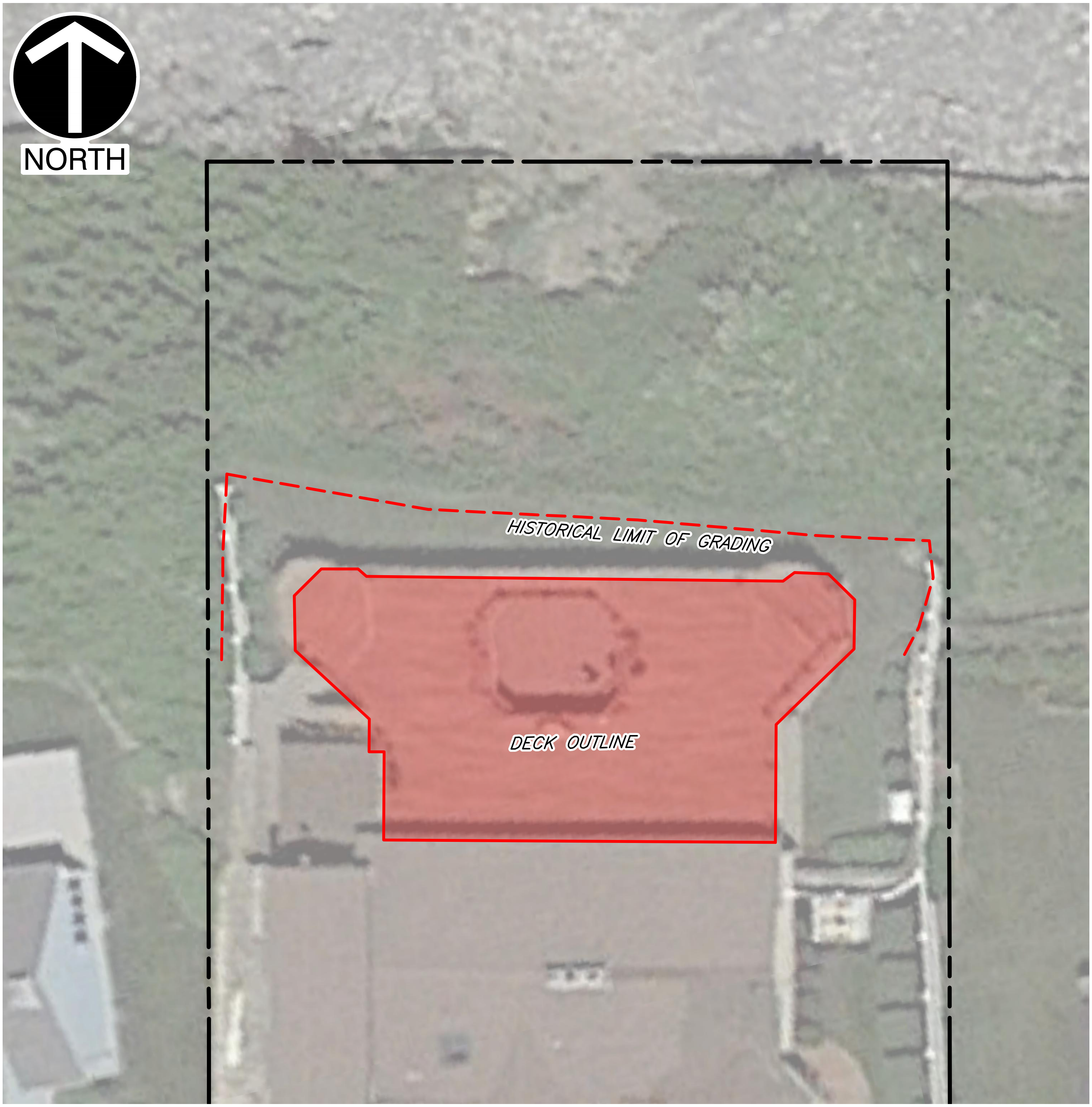

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2020 AERIAL
EXHIBIT

DRAWN BY: WD	CHECKED BY: KPS	APPROVED BY: KPS	FIGURE NO.: A2
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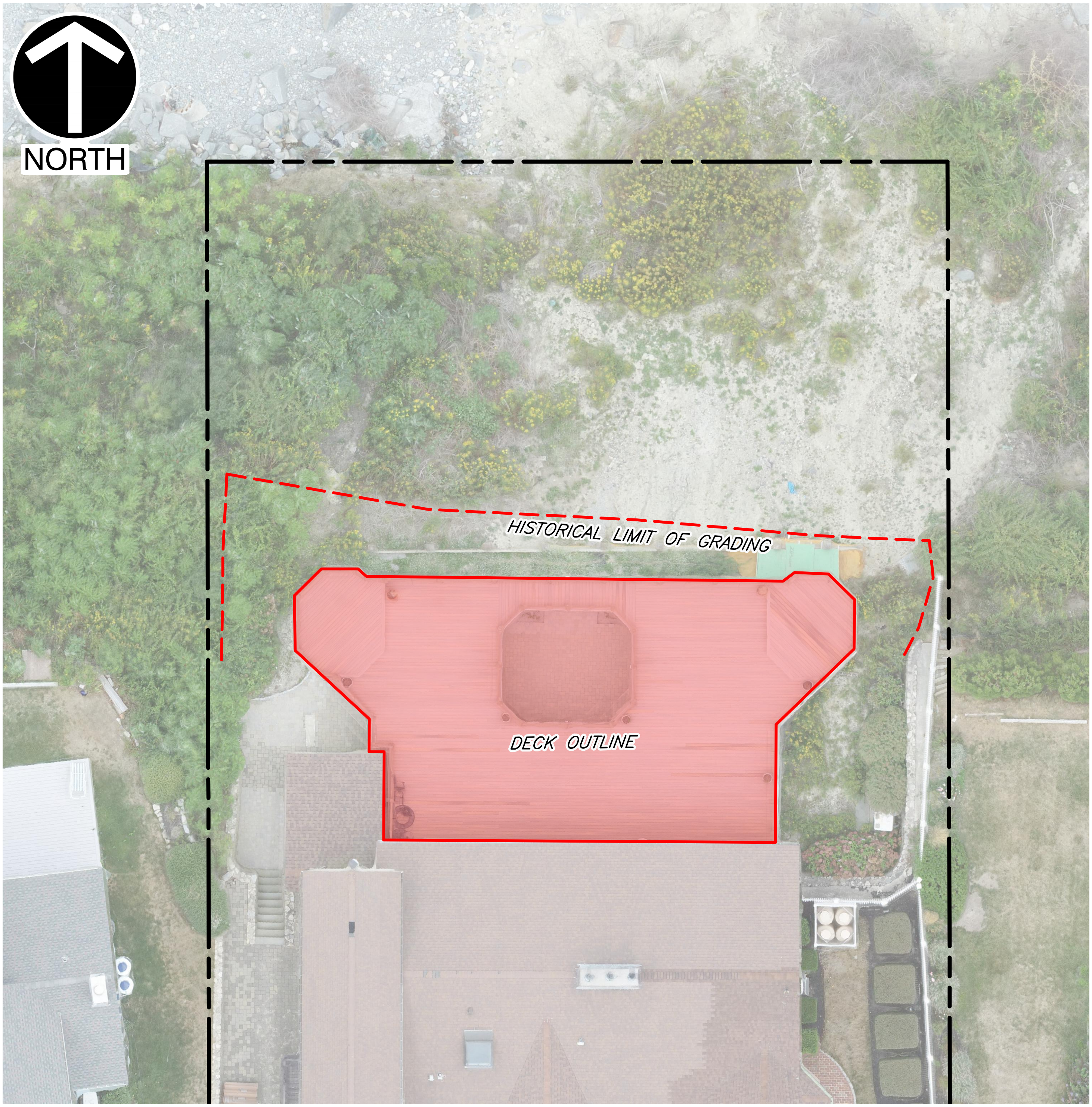

REFERENCE

1. AERIAL IMAGERY AND SLOPE TOPOGRAPHY DERIVED FROM UNMANNED AERIAL PHOTOGRAMMETRY DATA COLLECTED BY CEC, INC. ON JUNE 22, 2023.

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2024 AERIAL EXHIBIT

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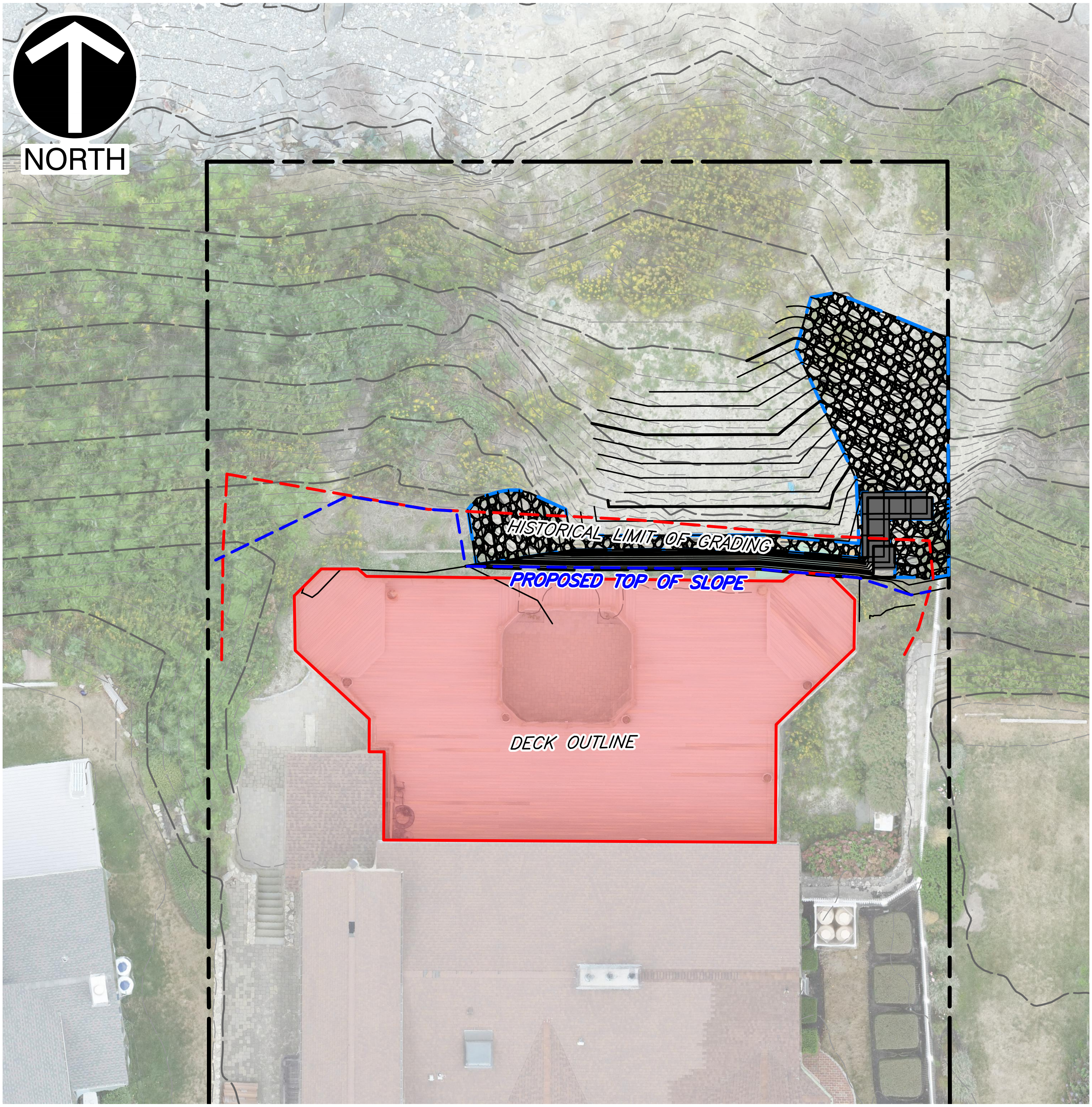

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PROPOSED DESIGN OVERLAY EXHIBIT

DRAWN BY:	WD	CHECKED BY:	KPS	APPROVED BY:	KPS	FIGURE NO.:
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