



Scarptec, Inc.
P.O. Box 326
Monument Beach, MA.
02553-0326

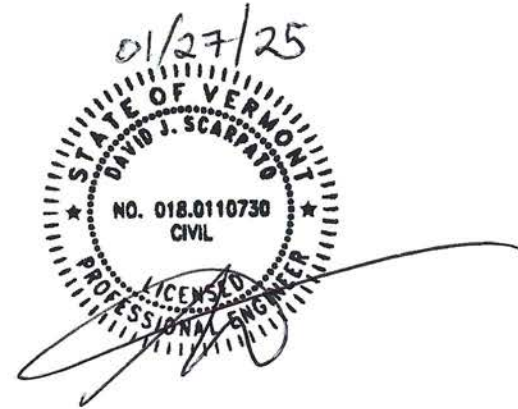
Tel: 603.361.0397
www.scarptec.com

27 January 2025
Internal Project No. 24-09

GEODesign, Inc.
84 Granite Shed Lane, Unit 1
Montpelier, VT 05602

Attention: Mr. Jason Gaudette, P.G., LEED, AP
Associate

Subject: Concept Development Report
Phase 3 Rockfall Mitigation Design
I-91 Southbound, Mile Marker 92.55
Fairlee, Vermont



Dear Jason,

Scarptec Inc. (Scarptec) is pleased to submit this Concept Development Report (CDR or "Report") to GEODesign Inc. (GEODesign) and the Vermont Agency of Transportation (VTrans), summarizing the Phase 3 rock slope mitigation design recommendations for replacement of an existing mitigation system at the Palisades rock cut at Mile Marker (MM) 92.55 on the southbound side (west side) of U.S. Interstate Highway Route 91 (i.e., "I-91") in Fairlee, Vermont. The existing rockfall mitigation system was impacted most recently by a rockslide event on 27 February 2024, when an estimated 100 to 200 cubic yards (CY) of rock debris apparently toppled from the slope, overwhelming an existing double-twist wire mesh rockfall drape and blocking the two southbound lanes. Phase 3 permanent rockfall mitigation design efforts were predicated by the winter 2024 failure event, as described in more detail below

The sections below present a brief project background, results of our site investigations, recommended rock slope mitigation design elements and our assumptions and understandings.

1. BACKGROUND

1.1 February 27, 2024 Rockslide

At approximately 11:00 p.m. on 27 February 2024, a rockslide occurred at the Palisades rock slope at approximately mile marker 92.55 on I-91, including an estimated 100 to 200 cubic yards (CY) of rock debris, some of which impacted the southbound (SB) lanes with boulders exceeding 3 feet (ft.) in maximum dimension. No injuries or significant vehicular damage were reported. The rockslide originated from the upper reaches of the 300-ft.-high slope, and ripped through existing



rockfall drape netting, installed as part of a rock slope mitigation project in 1996.^{1,2} VTrans employed this mitigation in response to an increasing frequency of rockfall events, dating back to 1985 and 1987.³ The most recent rockslide caused full closure of the north and southbound highway lanes for emergency response and assessment. Northbound traffic was allowed to resume the following day, and one lane of southbound traffic was opened after deployment of temporary concrete barriers and other traffic control.

In the days following the rockslide event, VTrans commissioned Haley & Aldrich, Inc. (H&A) to conduct emergency geotechnical services to assess the existing conditions and determine the cause of the rockslide. We understand that the cause of the slide was attributed to the toppling failure of pillars of rock, loosened by freeze/thaw effects in nearly vertical, dilated joints parallel to the roadway. The 2024 failure mechanism appears similar to a smaller rockfall event that occurred in 2018, located approximately 600 ft. north of the most recent rockslide.⁴

1.2 Original Construction

VTrans completed the original I-91 construction in Fairlee by 1971.^{1,3} To widen the corridor for four lane traffic, VTrans excavated lower sections of the rock slope using perimeter control blasting (i.e., presplitting) to construct four (4) presplit benches along the slope, each less than about 2 ft. wide and about 30 ft. high. These benches were cut at a 4 vertical to 1 horizontal angle (4V:1H; ~76 degrees), with the half-barrel casts at about 3 ft. spacings. The drilling was splayed out at the end of at least one bench to match the existing natural rock slope angle. Trim blasting likely was also completed above the presplit face to remove overhanging or unstable rock masses, based on a few vertical half-barrel casts near the top of the slope, and the presence of some old rusty winching cables adjacent to the crest access road.

1.3 1996 Mitigation

In summer/fall 1996, due to recurring rockfall events from above the presplit benches in 1973, 1985, 1987 and 1990^{1,3} that involved vehicle strikes and southbound roadway closures, VTrans designed and constructed a rockfall drape system as part of an interstate safety program. During this work, southbound traffic on I-91 was detoured onto adjacent U.S. Route 5 between Exits 16 and 15 to accommodate construction, including a large crawler crane with about 250 ft. of boom length. Prior to drape placement, the rock slope was cleared of vegetation and hand scaled (over three weeks). Four large rock blocks that appeared potentially unstable were secured with ten (10) rock bolts. These consisted of #8 (1 in. dia.) galvanized, 30 ft. long, Grade 150 threadbars

¹ Eliassen, T.D., 1997. Wire Mesh Netting of the Palisades, Interstate 91, Fairlee, Vermont. NESGE September/October Meeting, 13 p.

² Westerman, D.S., Eliassen, T.D., Finucane, R., and Wright, S.F., 2003. Highway Geology Symposium Field Trip Guide for Central Vermont, 54th Highway Geology Symposium, Burlington, Vermont, September 25, 2003, p. 3-7.

³ Baskerville, C.A., Lee, F.T. and Ratté, C.A., 1993. Landslide Hazards in Vermont. U.S. Geological Survey Bulletin 2043, 23 p.
<https://pubs.usgs.gov/bul/2043/report.pdf>

⁴ Golder Associates Inc., May 5, 2019. Preliminary Rockfall Mitigation Design, Fairlee Escarpment Southbound Interstate 91, Milepost 92.7, Fairlee, Vermont, 33 p. <https://vtrans.vermont.gov/sites/aot/files/FairleeRockfallMitigationDesign%20dated%205-10-19%20LINK.pdf>



installed in 1-5/8 in. dia. drill holes using a drill on a platform suspended from the large crane and grouted using fast and slow setting resin. The threadbars were then tensioned to a proof load of 90 kips and locked off on a bearing plate at 75 kips.

The 1996 rockfall drape consisted of Maccaferri No. 11 gauge, galvanized double-twist mesh made of steel wire with a tensile strength of 70 ksi. The mesh was suspended by 3/4-in. dia. galvanized wire rope attached to vertical Grade 60 galvanized #8 (1-in. dia.) threadbars above the crest of slope and grouted using resin cartridges (in rock) and cementitious grout (in overburden, if present). The drape was extended to about 30 ft. above the slope toe. Two cranes, one located above the crest of the slope, and one staged on the southbound lanes, were used to install the drape. According to publicly available reference documents,^{1,2} the installed draped mesh quantity was approximately 196,146 ft².

Due to occasional rockfalls damaging the double-twist material, including rockfalls in the past four (4) years emanating from the presplit portion of the slope, VTrans conducted repairs in 2022 consisting of placement of double-twist mesh patches attached to the damaged mesh.

1.4 Current Mitigation

VTrans has divided the rockfall mitigation project into three interrelated phases of work:

- Phase 1 – Emergency Response in the rockslide area, consisting of temporary rockfall barrier/maintenance and protection of traffic, removal of damaged draped mesh, scaling of loose rock remaining in the failure area, design and installation of rock dowels, and removal of associated debris. This phase was conducted from early March to early April 2024, by the general contractor J.A. McDonald, Inc. (JAM) and specialty rock slope contractor Apex Rockfall Mitigation, LLC (Apex). H&A conducted professional services during this phase, including construction observation, aerial drone services, and rock dowel design. This phase of work is now complete.
- Phase 2 – Interim Mitigation of the overall slope face, consisting of removal of draped mesh at the slope crest (to the slope crest and to about 200 ft. north of the southern end of the mesh), clearing/removal of vegetation, scaling of remaining loose rock previously covered by the drape, additional rock dowel design and installation, and temporary cable lashing. Due to the potential for scaled rock and other debris affecting I-91 and U.S. Route 5, southbound traffic was detoured onto U.S. Route 5 from Exit 16 to Exit 15 until November 23, 2024. JAM/Apex used two temporary, portable rockfall catchment barriers to protect northbound traffic (a primary 1,000 kJ barrier on the right side of the travel lane, and a secondary 35 kJ barrier on the left shoulder of the passing lane). The passing lane was opened to southbound traffic from November 23, 2024, to January 17, 2025. H&A continued to provide geotechnical design services and periodic construction



observation services during this phase. This phase started in mid-April 2024 and was completed on January 17, 2025.

- Phase 3 – Long-Term Rockfall Mitigation Design, consisting of developing, evaluating, and designing a “permanent” (i.e., long-term) rock slope mitigation design (the subject of this report).

2. SITE INVESTIGATION

2.1 Site Geology

Initial regional geologic mapping identified the bedrock as Late Ordovician Fairlee Quartz Monzonite, an igneous pluton assigned to the Late Ordovician Highlandcroft plutonic suite of western New Hampshire.⁵ This mapping describes the lithology consisting of crushed and foliated greenish gray, with local pink tinges, coarse-grained granitic rock, containing bluish-gray quartz, perthitic microcline and oligoclase or andesine, with secondary chlorite, sericite and green biotite. Later mapping correlated the Fairlee quartz monzonite with similar rocks of the Devonian Bethlehem Gneiss to the east in New Hampshire (Indian Pond Pluton), transported to the west on the upper block of the Ammonusic Fault in a post-Taconian extensional tectonic regime.^{6,7} Updated regional geologic mapping assigns the Fairlee Quartz Monzonite an Early Devonian age (410±5 Ma), with a lithology described as greenish-gray, pink-tinged, weakly foliated, coarse-grained to porphyritic biotite granite.⁸ Numerous xenoliths and felsic dikes occur within the rock mass.

The Fairlee Pluton has been extensively impacted by shearing and fault movement, including a prominent fault daylighting subhorizontal to north-dipping on the rock slope face. Drag folds on the upper block suggested a normal sense of movement;² however detailed field mapping by H&A and Scarptec indicates that the fault surface dips steeply to the northwest and is likely a high-angle reverse transpression fault. Several splays from this fault occur with similar suspected relative movement. This is consistent with a reverse fault mapped as bounding the eastern portion of the pluton roughly adjacent to the west bank of the Connecticut River.⁷ This complex faulting, along with orogenic events has resulted in the formation of at least six (6) prominent joint discontinuity sets crosscutting the rock mass.

⁵ Hadley, J.B., 1951. Geology of the Bradford-Thetford Area, Orange County, Vermont. Vermont Geologic Survey, Bulletin No. 1, 36 p., 2 plates. https://anrweb.vt.gov/PubDocs/DEC/GEO/Bulletins/Hadley_1950.pdf

⁶ Moench, R.H., 1990. The Piermont Allochthon, Northern Connecticut Valley Area, New England – Preliminary Description and Resource Implications. Chapter J, U.S. Geological Survey Bulletin 1887, Summary Results of the Glens Falls CUSMAP Project, New York, Vermont and New Hampshire, p. J1-J23. <https://pubs.usgs.gov/bul/1887/report.pdf>

⁷ Moench, R.H., 2007. The Piermont allochthon revisited and redefined at its type locality: Discussion. Geological Society of America Bulletin, March/April 2007, v. 119, no. 3, p. 493-499.

⁸ Ratcliffe, N.M., Stanley, R.S., Gale, M.H., Thompson, P.J., and Walsh, G.J., 2011. Bedrock Geologic Map of Vermont. U.S. Geological Survey Scientific Investigations Map 3184, 3 sheets, scale 1:100,000. <https://dec.vermont.gov/geological-survey/publication-gis/VTrock>



Overburden materials above the rock slope crest, inclusive of soils, are thin and likely less than five (5) ft. and consist of a regolith of angular quartz monzonite rock blocks contained within a matrix of modified ablation glacial till and slope wash (colluvium). Low outcrops and subcrops of quartz monzonite occur above the slope crest. The Laurentide ice sheet flowed generally southeasterly, forming the Palisades (as a roche moutonnée), and likely “plucked” rock blocks from the southeast face, contributing to the fractured nature of the slope. Glacial erratic cobbles and boulders up to 12 ft. in longest dimension exist above the crest of the slope. Small talus aprons, estimated to be less than a few feet thick, exist on the upper portions of the slope at the south end, where the slope angle is moderate (e.g., 45 to 50 degrees). The overburden materials are generally covered with grassy and brushy vegetation, with locally large diameter hard wood varieties and white pine trees.

2.2 Field Investigations & Observations

During the Phases 1 and 2 work, Scarptec conducted 11 site visits between 21 March 2024 and 10 December 2024 to conduct the following activities:

- Observe emergency mitigation activity
- Observe rockfall behavior from scaling operations
- Check on the performance of the temporary rockfall barrier systems
- Obtain representative photos of site conditions (including drone surveys)
- Collect geologic, geotechnical and laboratory data
- Assist in temporary rock slope mitigation design at the south end of the project
- Select rope rappels to observe post-scaling conditions
- Upper slope (above current mesh) conditions that may pose long-term rockfall issues
- Collect detailed information for the Phase 3 design

A key aspect of rock slope mitigation design is a three-dimensional understanding of rock mass discontinuities. During our site visits in July, August and October 2024, we collected geologic and geotechnical information for 159 discontinuities (joints, faults and mineral veins). The data included location (either stationing or GPS outcrop locations), discontinuity type, dip angle, dip angle direction, persistence, aperture, infilling, shear strength, surface roughness, shape, water, spacing and pertinent notes/comments. These data are plotted on the lower hemisphere stereographic projection (“stereonet”) presented in Figure 1 (Attachment No. 1). Our field discontinuity data are summarized in Table 1 (Attachment No. 2). The discontinuities were further assigned to six (6) joint sets based on a minimum concentration density of 2.7%. The orientation of these joint sets aided in evaluating and designing mitigation elements and measures, as described below.

During our site visit in August 2024, we also subcontracted S.W. Cole Engineering Inc. (SW Cole) to obtain shallow NQ size (approx. 1-7/8 in. dia.) rock core from the toe of slope using a small electric powered core drill. SW Cole conducted unconfined compression strength (UCS) and unit



weight (rock density) index testing of select, intact rock core samples per ASTM D7012-10, Method C. We used this data in our mitigation designs. Table 2 below summarizes the index testing results, and Attachment No. 3 contains SW Cole's laboratory report.

Table 2
Geotechnical Laboratory Index Testing Results Summary

Sample ID	#1A	#1B	#2	#3	#4	#6	#7	Average
Station	89+49	89+49	89+77	90+04	90+05	89+48	89+06	n/a
Depth [in.]	0-4	6-10	0.5-4.5	3-7	2-6	0.5-4.5	1.5-5.5	n/a
Unit Weight [pcf]	167.2	168.4	166.8	168.7	169.2	167.3	167.9	167.9
UCS [psi]	8,770	10,100	4,750	5,150	5,320	6,100	11,150	7,334

Notes:

pcf = pounds per cubic foot

psi = pounds per square inch

Core sample #5 was excluded as it did not have intact core lengths > 4-in. per ASTM requirements.

2.3 Phase 2 Temporary Rockfall Mitigation Support

The southern approximately 200 linear feet of the existing double-twist mesh was left in place, with its removal, hand scaling and permanent mitigation to be included during Phase 3. Scarptec provided support to the project team to review site conditions of several rock masses and blocks that were possibly unstable and could present rockfall hazards that could overwhelm the mesh during the 2024 to 2025 winter season. Temporary mitigation consisted of removing one small rock block, cable lashing two rock masses and one rock block, and securing the northern edge of the cut mesh with wire rope cable anchors.⁹ JAM/Apex completed this mitigation on January 15, 2025. Additional details of this separate scope of work are not covered further within this Report.

3. ROCKFALL MITIGATION DESIGN

The Fairlee rock slope is extensive, with a maximum vertical height of approximately 320 ft., from approximately elevation (El.) 440 ft. to 760 ft., and an overall maximum slope length of more than 1,200 ft. Our rockfall mitigation design scope of work area extended from VTrans highway baseline stationing (Sta.) 83+00 to 90+00. We understand that other parties completed geotechnical evaluation(s) of the slope north of approximate Sta. 90+00 in the years prior to our 2024 work. The proposed Phase 3 mitigation system extent is approximately 640 ft. long, extending from Sta. 83+50 on the south end to Sta 89+90 on the north end. The north end extent of the new mesh is about 30 ft. longer than the 1996 draped mesh to address potential rockfall hazards that may impact the roadway shoulder. The south end extent of the proposed mesh is

⁹ Scarptec, Inc., 21 October 2024. Temporary Rockfall Mitigation Recommendations. Emergency Rockfall, Fairlee IM 091-2(96), I-91 SB Mule Marker 92.55, Stas. 83+50 to 85+50, Fairlee, Vermont, 51 p., submitted to GEODesign, Inc.



about 10 to 20 ft. longer to wrap the mesh around a natural corner, stopping where rockfalls are anticipated to land in the southern talus field and not impact the roadway. The mitigation extends from the crest of the slope, generally where the slope steepness is less than 25 degrees, to 15 ft. above the slope toe as shown in Figures 2, 3 and 4 (Attachment No. 1). The following paragraphs provide details on the proposed mitigation elements.

3.1 Removal of Remaining Mesh and Temporary Rockfall Mitigation Elements

As discussed above, the southern 200 ft. of existing double-twist mesh was not removed during Phase 2, and three rock masses/blocks were temporarily secured. To access the slope to complete hand scaling of loose rock beneath the mesh, and either reinforce or remove the temporary mitigation elements, most of the existing double twist mesh will need to be removed as shown on Figure 2. Supporting double twist mesh anchors should be cut flush with the slope and surface hardware removed. We note that upper portions of the existing double-twist mesh can remain in place; specifically mesh that was installed above the crest in relatively flat areas, and in the upper southern portion of the slope where the slope angle is approximately 45 degrees or less. New pinned mesh can be installed over the old mesh in this area.

The Phase 2 temporary mitigation elements like wire rope cable lashing can be cut and removed within the scaling limits, and the rock masses/blocks can either be scaled or reinforced further with spot rock dowels, shotcrete and pinned mesh. This will be subject to evaluation of the VTrans engineer/geologist at the time of the Phase 3 work.

3.2 Hand Scaling

Hand scaling is recommended for the entire rock slope face as the rock slope will endure the 2024 to 2025 winter shutdown period, and may be affected by freeze-thaw activity, intense rainfall, etc., that may pose a rockfall hazard during construction. Hand scaling is envisioned to be more intense in the remaining southern 200 ft. of the slope where the existing mesh will be removed, as scaling has not occurred in this area since 1996. Hand scaling methods include rock scalers on rappel using hand tools such as a mine scaling bar and periodic use of air bags.

3.3 Anchored Mesh System (i.e., Pinned Mesh)

The extent of the mitigation of the upper portion of the rock slope is shown in Figure 3. The pinned mesh consists of Geobrigg's Tecco® G65/4mm mesh anchored to #10 (1.25-in. dia.) Grade 75, galvanized threadbar anchors 15 ft. long (typical) placed on a staggered pattern ranging from 8 ft. X 8 ft. to 12 ft. X 12 ft., depending on slope steepness. These anchorage elements are referred to as "Wire Mesh Dowels" in the referenced contract drawing set. This mesh is constructed of high tensile strength (>256 ksi) steel wire with a zinc-aluminum corrosion protection coating, as compared with the Maccaferri double-twist mesh, which is composed of steel wire with a tensile strength of 70 ksi. The anchors will be installed in drill holes with a



diameter of 3.5-in., and inclined downward 15 degrees to 25 degrees, depending on slope steepness. The anchors will be grouted using tremied cementitious grout. Note that in areas where the existing double twist mesh is to remain, it will need to be locally cut to drill the anchors.

The mesh will be secured to the slope using Geobrugg P33 galvanized spike plates and a hex nut torqued to 11 kips of tension force on the anchors. The pinned mesh includes double-leg, ¾-in. dia. galvanized wire rope anchors (WRAs), also placed in 10-ft.-long drill holes of 3.5-in. dia. These are placed along the perimeter of the mesh to hold a 5/8-in. dia. galvanized wire rope to help secure the edges of the pinned mesh. To help secure the mesh to areas of the slope with highly variable topography, we include the use of ¾-in. dia. wire rope as lashing between supplemental wire rope anchors. A thin mantle of overburden (estimated to be less than 5 ft. thick) exists at some portions of the uppermost slope, just below the crest, as shown in Figure 3. In these areas, Type I Erosion Control Product (ECP) will be placed beneath the mesh to reduce loss of easily erodible soils and promote vegetation growth.

3.4 Wire Mesh Drape (i.e., Draped Mesh)

The extent of the mitigation of the lower portion of the rock slope is shown in Figure 3. The draped mesh also consists of Geobrugg's Tecco® G65/4mm mesh, but it is anchored to a horizontal ¾ (0.75)-in. dia. wire rope cable, which is anchored to double-leg, ¾-in. dia. galvanized WRAs, also placed in 10 ft. long drill holes of 3.5-in. dia., placed at approximately 15-ft. spacings along the top. Supporting WRA's should be drilled 3- to 6-ft. above the bottom of the pinned mesh system so that no gap in mesh coverage exists. The draped mesh also contains vertical 5/8-in. cables at approximately 45-ft. spacings placed from the top horizontal cable to a bottom 5/8-in. dia. horizontal wire rope cable placed approximately 15 ft. above the slope toe. The mesh at the bottom will be hemmed around the bottom horizontal cable. The vertical 5/8-in. cables will be extended below the horizontal cable to form "pig tails" for periodic cleanout to remove accumulated debris using an excavator bucket attached to the pig tail.

3.5 Rock Dowels

Due to the presence of large rock blocks that either cannot be scaled safely or that could form rockfall hazards that could overload the mesh, rock dowels will be needed in select areas, as shown in Figure 3. These spot rock dowels occur in both the upper and lower portions of the mesh systems. The dowels will comprise #10 (1.25-in. dia.) Grade 75, galvanized threadbar that is nearly identical in construction elements as the mesh anchors used for the pinned mesh but have a square bearing plate and may be longer depending on drilling conditions (est. to be 20 ft. on average). These rock reinforcement elements are referred to as "Rock Dowels" in the construction drawings. Rock dowels in the pinned mesh areas may receive spike plates and be used to help tie the mesh to the slope. Rock dowels within draped mesh areas will receive bearing plates with the mesh over the dowel.



The design of the rock dowels was based on bar capacity and reinforcement of an assumed equivalent rock block size of 525 ft³. Total reinforced joint capacity was estimated using the method suggested by Spang and Egger (1990), resulting in a total joint capacity (106 kips) approximately 39% higher than the ultimate shear strength (76 kips) of just the bar alone. The recommended 10 ft. minimum rock dowel embedment depth past potential failure planes encountered during construction was determined based on an allowable (working) grout to rock bond strength of 125 psi (Safety Factor of 2.0 applied to ultimate bond strength).

3.6 Shotcrete

Several rock blocks and masses on the slope that can be or have been secured with rock dowels contain open, dilated near vertical joints (up to 5 ft. wide) with broken/weathered rock infill that can retain and collect water/ice, and thus can form rockfall sources due to hydrostatic uplift and ice jacking. To prevent water/ice infiltration and further movement, dry mix shotcrete will be used to fill these voids and help secure the rock blocks. Where needed the shotcrete can be reinforced with short rock nails, rebar and welded wire fabric. The shotcrete also includes provisions for drainage using a drainage geotextile board. Shotcrete areas thicker than 2 ft. will incorporate 0.5-in. diameter (#4) galvanized rebar longitudinal sections (with field bends as needed) for enhanced flexural resistance.

3.7 Drains

During our observations of the slope in 2024, there are several areas in the lower portion of the slope that seep groundwater, including some areas sufficient to support vegetative growth. This groundwater can exacerbate degradation of the slope through ice jacking at the surface during the winter months. To relieve groundwater pressure and to direct it away from the face, a series of drilled drains are designed to target the wetter areas, as shown in Figure 3. Drilled drains are also designed to help drain shotcrete and rock dowel areas. The drains will consist of 3.5-in. dia. boreholes, drilled at least 20 ft. deep, and inclined upward at 10 degrees from the horizontal, with a short PVC pipe grouted into the collar.

3.8 Upper Slope Boulder Removal

During our evaluations to support temporary mitigation at the southern end of the slope, we noted a glacial erratic boulder approximately 20 to 30 ft. above the slope crest. The boulder is estimated to have a mass exceeding 15 tons and is visibly depressing the shallow overburden. While not deemed a potential source of major rockfall, slope wash, freeze/thaw and root jacking effects could move the boulder down slope towards the crest over time. As mitigation work will be conducted adjacent to the boulder, we recommend that VTrans demolish/break up the boulder and remove the debris, as shown in Figure 4.



3.9 Upper Backslope Additional Pinned Mesh Areas

During our evaluations to support temporary mitigation at the southern end of the slope, we noted a rock block exceeding 100 lbs. that had been retained by small trees on top of the mesh, just below the crest of the mesh (Rock Block 7).¹⁰ The source area for this rock block was determined to be further up slope where the slope angle was as great as 30 degrees. Subsequent observations indicate the presence of two low outcrops above this area with loose, dilated rock blocks. Due to the elevation of these outcrops (exceeding 450 ft. vertically above the roadway), rolling rocks emanating from these outcrops could attain enough energy to reach the roadway. To mitigate these areas, we include two additional pinned mesh areas as shown in Figure 4. These areas will contain the same elements as the pinned mesh, but as they are relatively small in area and do not require Wire Mesh Dowels within the mesh limits but just WRAs on the perimeter wire rope around the outcrops.

3.10 Pull Testing

As part of construction quality control in rock slope mitigation projects using the recommended solid steel threadbar anchors and WRAs, pull testing is used to check that the contractor's materials and methods meet the minimum strength testing when the anchors are loaded. The design includes conducting pull testing of sacrificial anchors using a calibrated hydraulic center hole jack and two (2) independent incremental dial gauges to measure extension under load. The test protocols described in the project specifications and shown in the detail address sacrificial test anchors constructed in the same manner as production anchors, but with a specific lower bond length and an upper unbonded free stressing length. The testing will use a design test load of 16.5 kips per bonded foot. The location of the sacrificial test anchors will be representative of site conditions, but also likely placed in areas where confirmation testing can be conducted at a later date to determine if bond strength loss occurs (if any) after several years.

4.0 KEY ASSUMPTIONS & LIMITATIONS

Please note that the findings in this Concept Development Report are based on the following key assumptions and limitations:

1. We assume VTrans will compile the bidding documents, which will use the design drawings, along with site civil features, such as right-of-way (ROW), existing geometry and stationing, presence of any easements, drainage, topography and environmental/permit restrictions that will be surveyed by VTrans or its consultants.
2. We understand that VTrans will complete final insertion of the rock slope mitigation drawing and detail elements, and that these items will be assembled into project Contract Documents by VTrans or their Prime Civil/Highway Engineer.
3. Scarptec does not offer any guarantees that the proposed rockfall mitigation solutions will mitigate all rockfall hazards that may be present at this site. We have evaluated the



hazard(s) in accordance with industry standards of practice and have communicated these results to GEODesign and VTrans within this Report.

4. Rockfall mitigation measures recommended herein are not intended or designed to mitigate icefall hazards, if and where they may exist; however, use of rock drains and slope scaling may help reduce this potential hazard.
5. Note that if slope failures occur or if slope condition(s) changes prior to anticipated construction commencing in spring 2025, Scarptec may need to re-evaluate the conclusions in this Report, as slope geometry and conditions may have changed such that the Phase 3 design's effectiveness may have been affected. This also applies if VTrans elects to push proposed construction out to beyond 2025.
6. Our Scope included evaluation of existing portions of the slope between Sta. 83+00 and 90+00. Technical evaluation(s) in areas outside these shown limits were not included within our analyses. As such, rockfall mitigation recommendations also shown herein should not be extended outside the slope limits shown in Figures 2 to 3, or at other sites without Scarptec's prior review and assessment.
7. All mitigation elements recommended herein will require field engineering during construction to adapt the intended design to encountered conditions.
8. Rockfalls and rockslides/landslides are sporadic and unpredictable. Causes range from human activities (construction, traffic, illegal dumping, etc.) to environmental events (weather, earthquakes, etc.). Because of the multiplicity of factors potentially affecting such events, and the random nature of rockfall trajectories, design of rockfall mitigation is not, and cannot be considered an exact science that guarantees the safety of individuals and property; however, by the application of sound engineering principles to a predictable range of parameters and using generally accepted and properly designed protection measures in identified risk areas, the risk of injury and property loss can be substantially reduced. Inspection and maintenance of such systems are essential to ensure the serviceability of the systems and to maintain an improved protection level. These systems can also be degraded by impact damage including rock strikes on system infrastructure, sharp rocks that cut mesh fabric or rock strikes that exceed the service or design limits of a particular system. Degradation of the system can also occur through corrosion caused by pollution or other human activity factors.
9. Scarptec accepts no liability for rock slope mitigation elements designed and installed by other parties. This includes the Phase 1 and Phase 2 mitigation elements, which we assume are designed as interim measures until the Phase 3 construction is complete.
10. Our design services were substantially completed prior to completion of the Phase 2 rock slope mitigation work. The figures and recommendations herein contain photos or references to conditions that may differ from those realized at the start of Phase 3 construction.
11. Scarptec will be engaged for periodic site visits during Phase 3 construction to observe that conditions and assumptions relied upon for our design are still valid and present for proposed spring 2025 construction.



We hope these recommendations are beneficial to the project team. Please contact us if you would like to discuss the conclusions contained in this Report.

Sincerely,
SCARPTEC, INC.

A blue ink signature of David J. Scarpato, consisting of a stylized 'D' and 'S' followed by a horizontal line.

David J. Scarpato, P.E. (VT)
President & Principal
Geohazard Engineer

A blue ink signature of Peter C. Ingraham, featuring a stylized 'P' and 'I' followed by a horizontal line.

Peter C. Ingraham, P.E.
Senior Rock Engineering Consultant

A blue ink signature of Jay R. Smerekanicz, featuring a stylized 'J' and 'S' followed by a horizontal line.

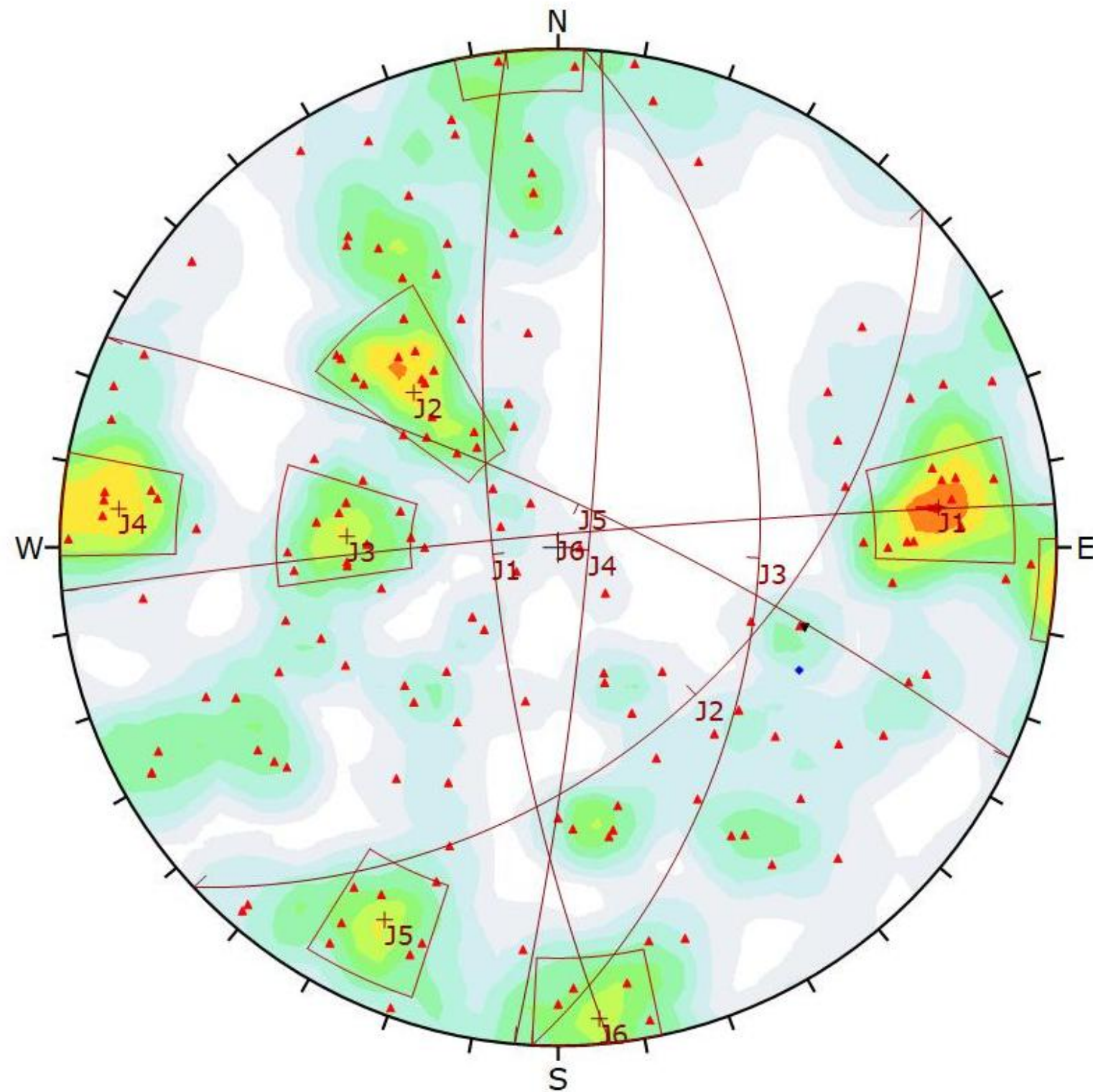
Jay R. Smerekanicz, P.G., C.P.G.
Senior Engineering Geologist

Attachments:

1. Figure Nos. 1 to 4 (4 sheets)
2. Table 1 – Discontinuity Data Summary (3 sheets)
3. Geotechnical Laboratory Data (11 sheets)

ATTACHMENT NO. 1

Figure Nos. 1 through 4



Symbol	TYPE	Quantity
◆	fault	1
▲	joint	157
▼	vein	1

Color	Density Concentrations
	0.00 - 0.45
	0.45 - 0.90
	0.90 - 1.35
	1.35 - 1.80
	1.80 - 2.25
	2.25 - 2.70
	2.70 - 3.15
	3.15 - 3.60
	3.60 - 4.05
	4.05 - 4.50

Contour Data	Pole Vectors
Maximum Density	4.14%
Contour Distribution	Fisher
Counting Circle Size	1.0%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m	■	75	264	J1
2m	■	46	137	J2
3m	■	46	93	J3
4m	■	83	95	J4
5m	■	79	25	J5
6m	■	87	355	J6

Plot Mode	Pole Vectors
Vector Count	159 (159 Entries)
Hemisphere	Lower
Projection	Equal Angle

- NOTES:
1. DISCONTINUITY DATA COLLECTED JULY 02 THROUGH OCTOBER 29, 2024.
 2. SEE TABLE 1 FOR DISCONTINUITY SUMMARY.

DATE	NO.	REASON ISSUED / REVISED	DRAWN	DESIGN	CHECK
12/12/2024	1	DESIGN BASIS REPORT	JRS	JRS	DJS
12/18/2024	2	VTRANS REVIEW	JRS	JRS	DJS
12/24/2024	3	FINALIZED SET	JRS	JRS	DJS



DESIGNED FOR: **GEODesign, Inc.**

INTERNAL PROJECT NO.: 24-09 PROJECT LOCATION: FAIRLEE, VERMONT

CONCEPTUAL PHASE 3 ROCK SLOPE MITIGATION
FAIRLEE ROCK SLOPE I-91 SB MM 92.55
VTRANS IM 091-2(97)

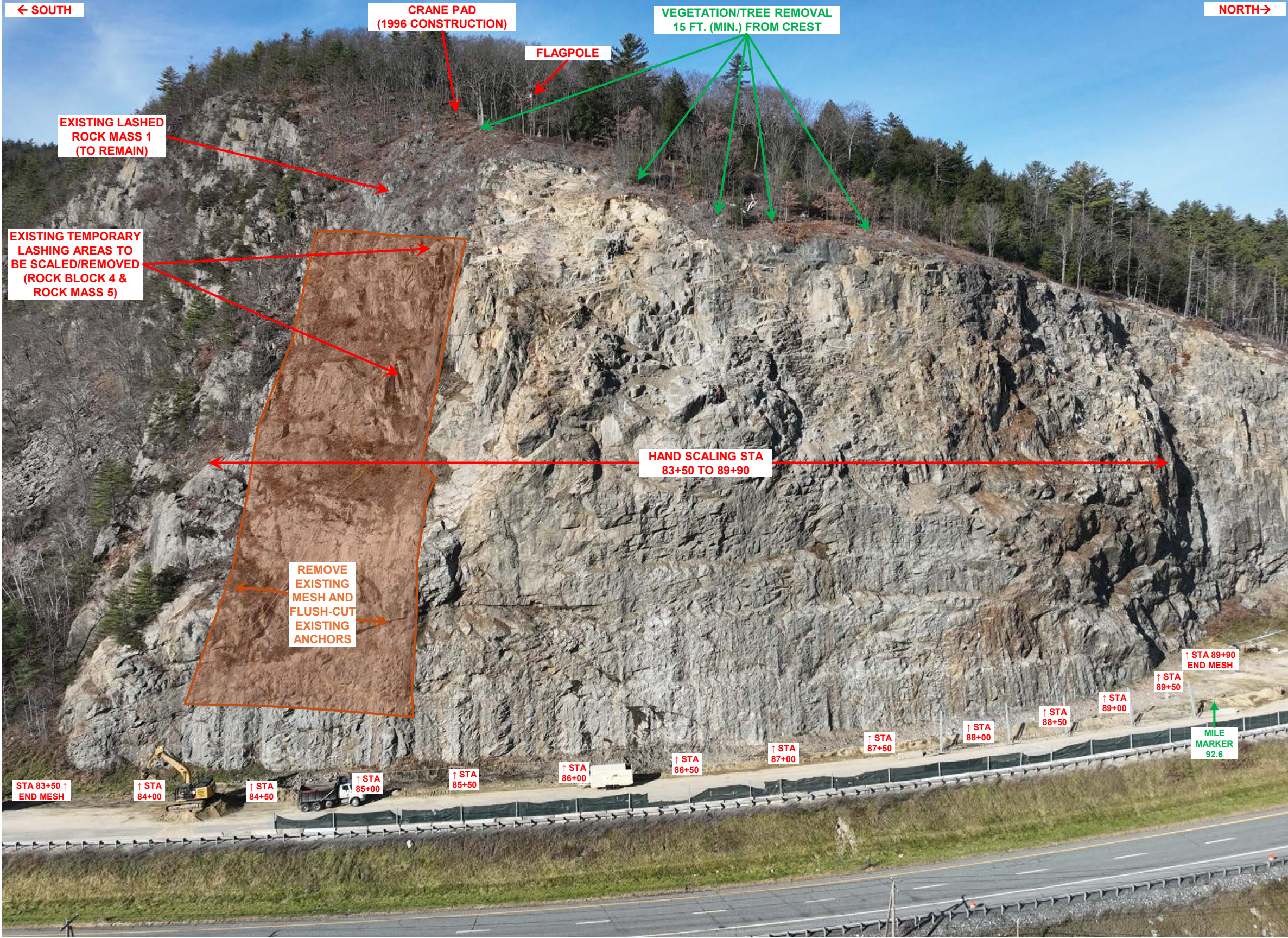
FIGURE TITLE

DISCONTINUITY STEREONET

FIGURE NUMBER

1

1 OF 4



- NOTES:
- 1. DRONE PHOTO TAKEN BY VTRANS 11/19/2024.
 - 2. ALL MITIGATION ELEMENT LAYOUT AND PROJECT STATIONING SHOWN IS APPROXIMATE.
 - 3. PHASE 3 SITE CONDITIONS WILL DIFFER FROM THOSE SHOWN HERE.
 - 4. ALL PRELIMINARY LIMITS, DIMENSIONS AND QUANTITIES INDICATED HEREIN ARE APPROXIMATE AND MAY CHANGE PENDING COMPLETION OF PHASE 2 WORK, SCHEDULED FOR 12/21/2024. FINAL LOCATIONS AND EXTENTS OF MITIGATION WILL REQUIRE FIELD ENGINEERING DURING CONSTRUCTION.

DATE	NO.	REASON ISSUED / REVISED	DRAWN	DESIGN	CHECK
12/12/2024	1	DESIGN BASIS REPORT	JRS	JRS	DJS/PCI
12/18/2024	2	VTRANS REVIEW	JRS	JRS	DJS
12/24/2024	3	FINALIZED SET	JRS	JRS	DJS



DESIGNED FOR:
GEODesign, Inc.

INTERNAL PROJECT NO.: 24-09 PROJECT LOCATION: FAIRLEE, VERMONT

**CONCEPTUAL PHASE 3 ROCK SLOPE MITIGATION
FAIRLEE ROCK SLOPE I-91 SB MM 92.55
VTRANS IM 091-2(97)**

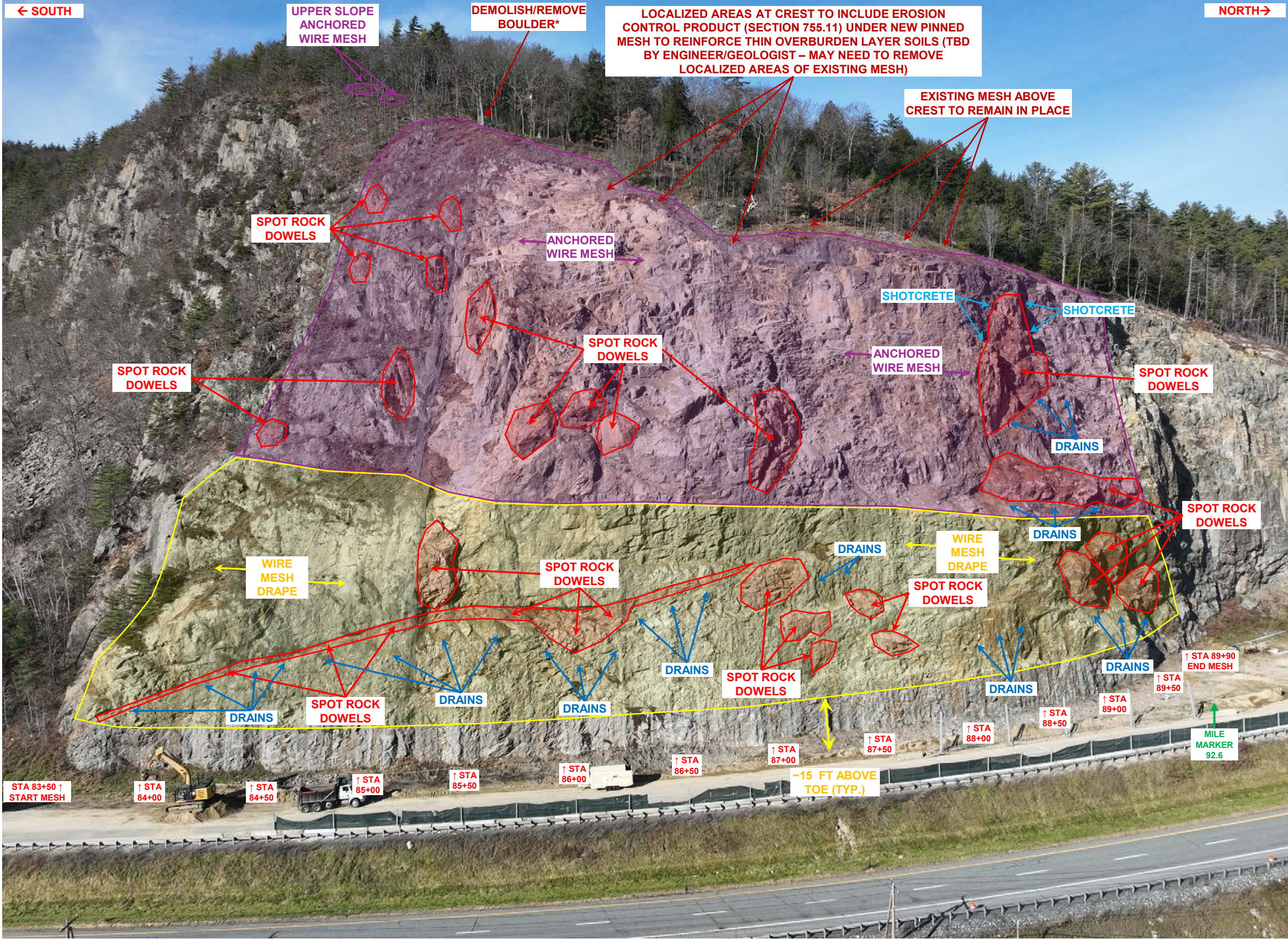
FIGURE TITLE

**HAND SCALING EXTENT, EXISTING
MESH/LASHING REMOVAL, AND
VEGETATION/TREE CLEARING**

FIGURE NUMBER

2

2 OF 4



- NOTES:
1. DRONE PHOTO TAKEN BY VTRANS 11/19/2024.
 2. ALL MITIGATION ELEMENT LAYOUT AND PROJECT STATIONING SHOWN IS APPROXIMATE.
 3. PHASE 3 SITE CONDITIONS WILL DIFFER FROM THOSE SHOWN HERE.
 4. ALL PRELIMINARY LIMITS, DIMENSIONS AND QUANTITIES INDICATED HEREIN ARE APPROXIMATE AND MAY CHANGE PENDING COMPLETION OF PHASE 2 WORK, SCHEDULED FOR 12/21/2024. FINAL LOCATIONS AND EXTENTS OF MITIGATION WILL REQUIRE FIELD ENGINEERING DURING CONSTRUCTION.
 5. * - SEE FIGURE 4 FOR DETAILS.

DATE	NO.	REASON ISSUED / REVISED	DRAWN	DESIGN	CHECK
12/12/2024	1	DESIGN BASIS REPORT	JRS	JRS	DJS/PCI
12/18/2024	2	VTRANS REVIEW	JRS	JRS	DJS
12/24/2024	3	FINALIZED SET	JRS	JRS	DJS



DESIGNED FOR: **GEODesign, Inc.**

INTERNAL PROJECT NO.: 24-09 PROJECT LOCATION: FAIRLEE, VERMONT

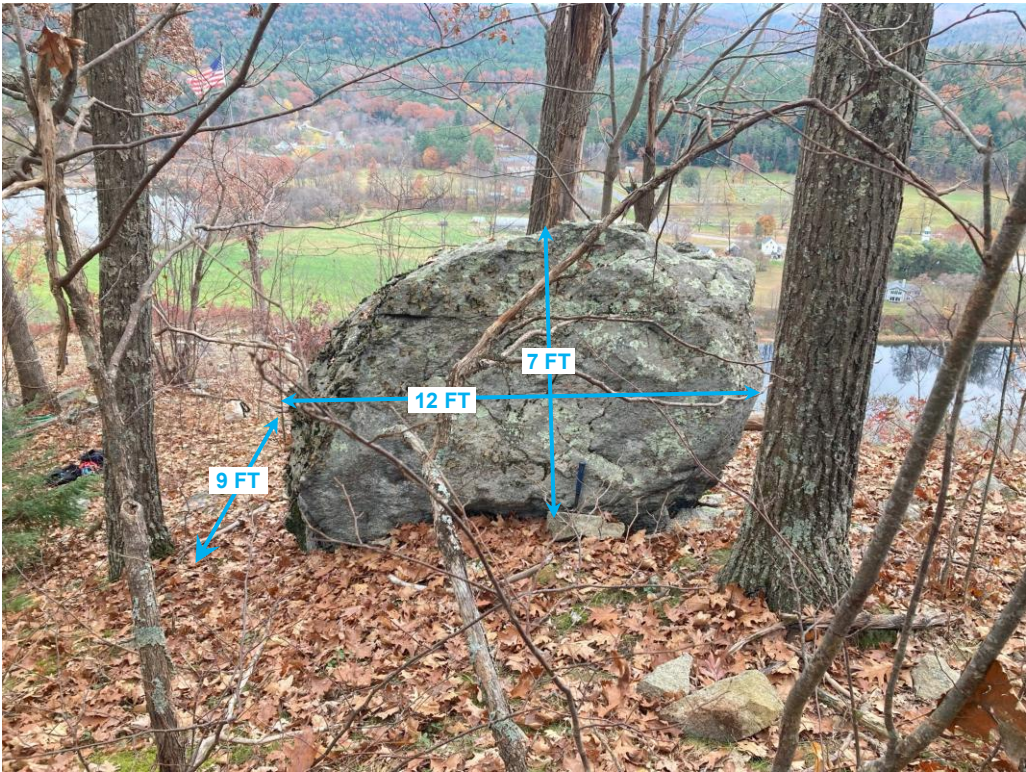
CONCEPTUAL PHASE 3 ROCK SLOPE MITIGATION
FAIRLEE ROCK SLOPE I-91 SB MM 92.55
VTRANS IM 091-2(97)

FIGURE TITLE	FIGURE NUMBER
ROCK SLOPE MITIGATION	3
	3 OF 4



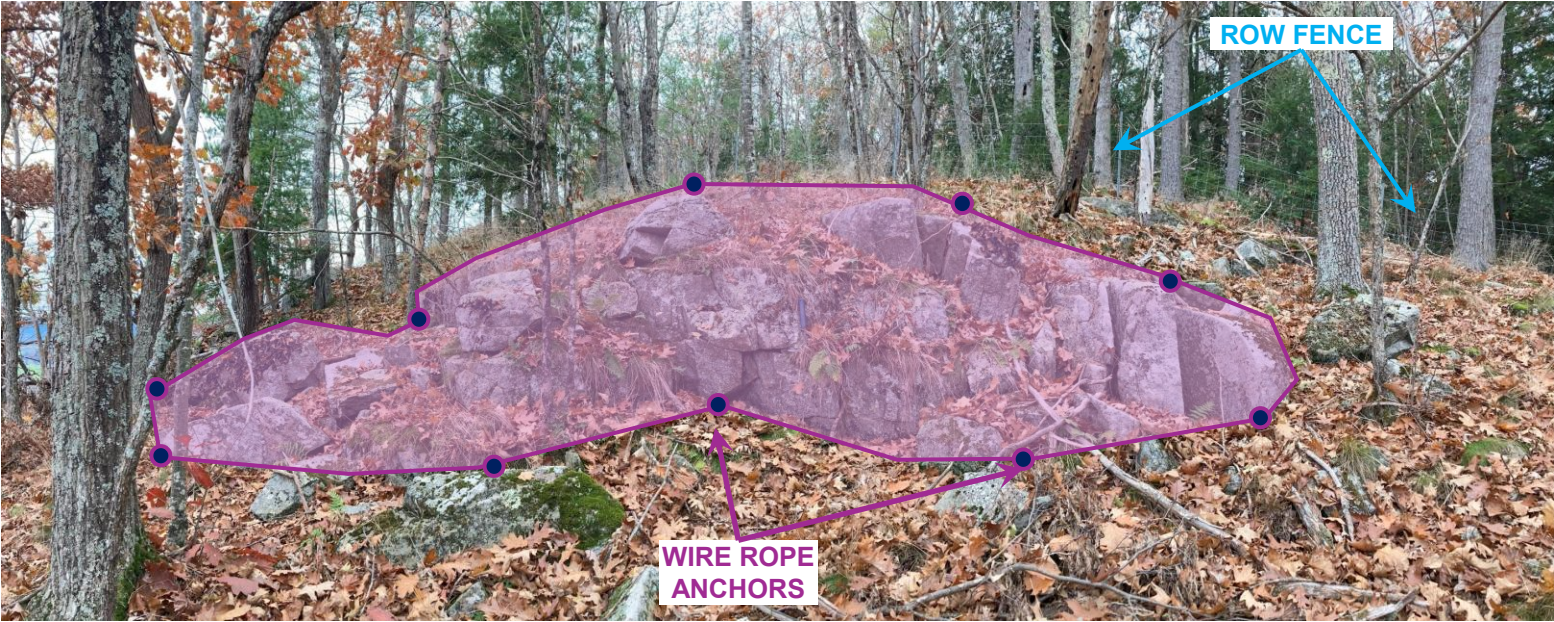
OUTCROP OC-8

- SCALING/TREE REMOVAL AS NECESSARY
- PLACE ANCHORED WIRE MESH, EST. AREA = 300 FT²
- SIX (6) WIRE ROPE ANCHORS, 5 FT (MIN.) EMBEDDED IN BEDROCK



BOULDER

- DEMOLISH BOULDER IN PLACE USING NON-EXPLOSIVE METHODS, DISPOSE OF DEBRIS AWAY FROM CREST OF SLOPE TO AREA APPROVED BY ENGINEER
- EST. VOLUME = 28 YD³



OUTCROP OC-10

- SCALING/TREE REMOVAL AS NECESSARY
- PLACE ANCHORED WIRE MESH, EST. AREA = 400 FT²
- EST. TEN (10) WIRE ROPE ANCHORS, 5 FT (MIN.) EMBEDDED IN BEDROCK

NOTES:

1. PHOTOS TAKEN BY SCARPTec 10/28-29/2024.
2. OUTCROP AND BOULDER LOCATIONS SHOWN ON FIGURE 3.
3. ALL MITIGATION ELEMENT LAYOUT SHOWN IS APPROXIMATE. LOCALIZED TREE/VEGETATION REMOVAL MAY BE NEEDED FOR CONSTRUCTION.
4. PHASE 3 SITE CONDITIONS WILL DIFFER FROM THOSE SHOWN HERE.
5. ALL PRELIMINARY LIMITS, DIMENSIONS AND QUANTITIES INDICATED HEREIN ARE APPROXIMATE AND MAY CHANGE PENDING COMPLETION OF PHASE 2 WORK, SCHEDULED FOR 12/21/2024. FINAL LOCATIONS AND EXTENTS OF MITIGATION WILL REQUIRE FIELD ENGINEERING DURING CONSTRUCTION.

DATE	NO.	REASON ISSUED / REVISED	DRAWN	DESIGN	CHECK
12/12/2024	1	DESIGN BASIS REPORT	JRS	JRS	DJS/PCI
12/18/2024	2	VTRANS REVIEW	JRS	JRS	DJS
12/24/2024	3	FINALIZED SET	JRS	JRS	DJS



DESIGNED FOR:	GEODesign, Inc.
INTERNAL PROJECT NO.: 24-09	PROJECT LOCATION: FAIRLEE, VERMONT
CONCEPTUAL PHASE 3 ROCK SLOPE MITIGATION FAIRLEE ROCK SLOPE I-91 SB MM 92.55 VTRANS IM 091-2(97)	

FIGURE TITLE
UPPER SLOPE MITIGATION

FIGURE NUMBER
4
4 OF 4

ATTACHMENT NO. 2

Table 1 – Discontinuity Data Summary

TABLE 1
DISCONTINUITY SUMMARY
FAIRLEE ROCK SLOPE, I-91 SB MM 92.55
GEODESIGN/VTRANS
VTRANS IM 091-2(97)
FAIRLEE, VERMONT

Discontinuity			Dip [°]	Uncorrected Dip Direction State Plane Grid North [°]	Corrected ¹ Dip Direction State Plane Grid North [°]	Set	Persistence [ft]	Aperture [in]	Infilling	Shear Strength ²	Surface Roughness	Discontinuity Shape	Water	Spacing [ft]	Notes/Comments
Sta	ID	Type													
83+67	1	joint	32	355	341		10	0.2	none	medium	rough	planar	dry	1	
83+69	2	joint	52	075	061		4	0.4	none	medium	rough	planar	dry	0.5	
83+72	3	joint	36	117	103	3	20	0.4	none	low	very rough	planar	dry	2	friable
83+75	4	joint	76	303	289		15	0.1	none	high	rough	undulating	dry	1	
83+82	5	joint	52	334	320		10	0.2	none	high	rough	planar	dry	1	
83+85	6	joint	79	190	176		4	0.6	FeO _x	medium	rough	planar	dry	1.5	past water seepage
83+89	7	joint	52	110	096	3	20	0.5	none	high	very rough	planar	dry	5	slickensides
83+95	8	joint	84	169	155		15	0.3	none	high	very rough	planar	dry	4	
84+08	9	joint	60	003	349		20	tight	none	high	very rough	planar	dry	3	
84+14	10	joint	32	158	144	2	20	tight	none	high	rough	planar	dry	10	
84+17	11	joint	54	083	069		2	tight	none	high	rough	planar	dry	10	
84+18	12	joint	86	263	249		3	tight	none	high	very rough	undulating	dry	5	
84+19	13	joint	75	340	326		25	0.2	none	medium	rough	undulating	dry	6	
84+23	14	joint	86	075	061		8	0.2	none	high	rough	curved	dry	5	
84+27	15	joint	29	155	141	2	30	0.5	none	high	rough	curved	dry	3	both ends curved to NE
84+34	16	joint	86	120	106		6	0.1	none	high	very rough	undulating	dry	3	
84+36	17	joint	50	349	335		10	0.2	none	high	rough	planar	dry	3	
84+50	18	joint	48	113	099	3	20	0.2	organics	medium	rough	stepped	seep	5	water producer
84+55	19	joint	53	171	157		10	0.1	none	medium	rough	undulating	dry	4	
84+51	20	joint	83	180	166		15	0.4	organics	medium	rough	planar	seep	1.5	plants
84+60	21	joint	47	159	145	2	20	0.2	none	high	rough	undulating	dry	1.5	
84+60	22	joint	68	343	329		8	tight	none	high	rough	planar	dry	2	conjugate of #21
84+64	23	joint	87	286	272		4	0.1	none	high	rough	undulating	dry	4	
84+71	24	joint	46	100	086	3	20	0.3	none	medium	rough	planar	dry	2	
84+70	25	joint	42	140	126	2	40	0.2	none	medium	rough	undulating	seep	2	
84+75	26	joint	31	147	133	2	60	1.5	soil	medium	rough	planar-undulating	seep	5	calcium carbonate
84+80	27	joint	40	150	136	2	20	0.2	organics	medium	rough	planar-undulating	seep	5	
84+85	28	joint	65	194	180		4	2	none	high	very rough	planar-undulating	dry	0.2	
84+89	29	joint	84	077	063		10	0.1	none	high	very rough	planar-irregular	dry	6	
84+93	30	joint	47	116	102	3	4	tight	none	medium	rough	planar-undulating	dry	0.5	
85+04	31	joint	56	099	085	3	8	0.1	broken rock	high	very rough	planar-undulating	dry	3	
85+04	32	joint	57	103	089	3	3	tight	none	medium	rough	irregular	dry	1.5	
85+07	33	joint	54	302	288		6	0.05	none	high	very rough	planar	dry	1	
85+15	34	joint	59	089	075		20	0.1	none	medium	rough	planar-undulating	dry	6	
85+20	35	joint	85	110	096	4	20	0.1	none	low	rough	planar-undulating	dry	4	local rockfall
85+20	36	joint	46	099	085	3	3	0.05	silt	high	very rough	undulating-circular	dry	1.5	
85+28	37	joint	72	070	056		12	tight	none	medium	rough	planar-undulating	dry	2	
85+35	38	joint	46	057	043		20	tight	none	low	smooth	planar-circular	dry	0.3	
85+50	39	joint	59	049	035		80	0.4	organics/silt	medium	rough	planar-undulating	seep	3	
85+53	40	joint	75	261	247		4	tight	none	medium	rough	planar	dry	1	
85+54	41	joint	71	034	020	5	30	0.05	none	low	smooth	planar-undulating	dry	4	wedge former
85+58	42	joint	65	186	172		10	tight	FeO _x	medium	rough	undulating	dry	0.5	
85+60	43	joint	78	281	267	1	12	0.05	none	high	rough	planar	dry	1	
85+64	44	joint	62	170	156		18	0.6	organics/silt/CaCO ₃	high	very rough	undulating	seep	4	wedge former
85+70	45	vein	55	302	288		20	1	broken rock	low	smooth	planar-stepped	dry	2	quartz vein
85+78	46	joint	47	155	141	2	20	0.4	none	medium	rough	planar-circular	dry	4	
85+95	47	joint	35	026	012		10	0.05	none	medium	rough	planar-circular	seep	1	
85+94	48	joint	56	144	130	2	10	0.1	CaCO ₃ /slicks/chl	high	very rough	planar-undulating	seep	2.5	
85+94	49	joint	63	283	269		10	tight	none	medium	very rough	undulating	damp	2.5	
85+98	50	joint	78	019	005		20	0.2	none	medium	rough	planar	damp	3	
86+13	51	joint	57	014	360		30	0.05	none	medium	very rough	planar	damp	3	
86+14	52	joint	42	105	091	3	15	0.5	organics/silt	medium	very rough	planar-curved	seep	3.5	rockfall area
86+31	53	joint	30	104	090		15	0.2	broken rock/silt	high	very rough	planar-undulating	seep	0.75	
86+31	54	joint	82	034	020	5	5	0.05	none	medium	rough	undulating	dry	4	
86+36	55	joint	89	034	020		10	0.5	soil	medium	very rough	undulating	seep	n/a	slickensides
86+50	56	joint	52	158	144	2	5	tight	none	low	rough	planar	seep	2	
86+50	57	joint	40	091	077		15	0.1	none	medium	rough	planar	seep	1.5	

TABLE 1
DISCONTINUITY SUMMARY
FAIRLEE ROCK SLOPE, I-91 SB MM 92.55
GEODESIGN/VTRANS
VTRANS IM 091-2(97)
FAIRLEE, VERMONT

Discontinuity			Dip [°]	Uncorrected Dip Direction	Corrected ¹ Dip Direction	Set	Persistence [ft]	Aperture [in]	Infilling	Shear Strength ²	Surface Roughness	Discontinuity Shape	Water	Spacing [ft]	Notes/Comments
Sta	ID	Type		State Plane Grid North [°]	State Plane Grid North [°]										
86+48	58	joint	87	161	147		4	tight	none	low	smooth	planar-curved	dry	n/a	
86+55	59	joint	59	011	357		15	0.1	none	medium	rough	planar	dry	8	aplite vein
86+63	60	joint	45	305	291		15	tight	none	medium	rough	planar	damp	n/a	
86+73	61	joint	30	354	340		50+	0.5	soil	medium	rough	planar	damp	10	
86+83	62	joint	85	108	094	4	12	0.2	broken rock	very high	very rough	undulating	dry	2	
86+90	63	joint	37	056	042		8	tight	none	medium	rough	planar	dry	2	
86+93	64	joint	47	186	172		2	tight	none	high	very rough	undulating	dry	2	wedge former
87+02	65	joint	75	272	258	1	50	0.2	broken rock	medium	rough	undulating/curved	dry	8	wedge former
87+14	66	joint	25	065	051		50+	0.2	broken rock/soil	medium	rough	planar	seep	4	
87+18	67	joint	68	290	276		8	0.05	organics	medium	rough	planar	damp	15	
87+20	68	joint	38	144	130	2	10	0.15	none	medium	rough	undulating	seep	1.5	
87+31	69	joint	61	004	350		6	tight	none	medium	rough	planar/stepped	dry	n/a	
87+29	70	joint	60	145	131	2	20	1	broken rock/organic	high	very rough	planar/curved	seep	5	
87+44	71	joint	75	171	157		10	tight	none	high	very rough	undulating	dry	10	
87+54	72	joint	74	190	176		4	0.15	none	medium	very rough	planar	seep	9	
87+78	73	joint	71	190	176		6	tight	none	high	very rough	planar	dry	1.5	
87+80	74	joint	25	056	042		50+	0.5	silt	medium	very rough	planar/undulating	dry	5	
87+83	75	joint	5	065	271		5	0.05	none	medium	rough	planar	dry	1	
88+00	76	joint	61	145	131	2	10	0.05	FeO _x	high	rough	undulating	dry	5	slickensides
88+00	77	joint	64	254	240		6	tight	CaCO ₃	high	very rough	planar/undulating	dry	5	
88+08	78	joint	15	328	314		3	tight	none	high	very rough	planar	dry	3	
88+23	79	joint	74	305	291		7	tight	CaCO ₃	high	very rough	undulating	dry	5	slickensides
88+20	80	joint	45	123	109		15	0.5	FeO _x /CaCO ₃	medium	very rough	curved	dry	3	slickensides
88+30	81	joint	79	356	342		30	tight	FeO _x	very high	very rough	planar	dry	5	
88+31	82	joint	67	284	270	1	15	tight	none	medium	rough	planar	dry	n/a	
88+50	83	joint	53	154	140	2	30	0.05	CaCO ₃	medium	rough	planar	dry	15	
88+53	84	joint	36	334	320		10	0.15	none	medium	rough	undulating	dry	1.5	
88+72	85	joint	40	350	336		50+	0.75	none	medium	rough	undulating	dry	1.5	
88+80	86	joint	66	174	160		4	tight	none	high	rough	irregular	dry	8	
88+80	87	joint	61	272	258		20	0.2	none	high	very rough	undulating	dry	8	
88+79	88	joint	52	326	312		3	tight	FeO _x	medium	very rough	undulating	dry	6	
OC-1	89	joint	87	124	110		10	0.04-0.08	none	medium	rough	planar	damp	1	OC-1 coordinates: 43.913372, -72.135406
	90	joint	85	044	030	5	3	0.4	none	medium	rough	planar	damp	2	
	91	joint	80	097	083		3	0.12	none	high	rough	stepped	damp	1	
OC-2	92	joint	60	345	331		6	tight	none	medium	rough	planar	damp	1	
	93	joint	80	033	019	5	7	0.12	none	high	rough	planar	damp	1	OC-2 coordinates: 43.913247, -72.135147
	94	joint	73	159	145		3	tight	none	high	rough	stepped	dry	2	
	95	joint	85	129	115		4	0.08	none	medium	rough	planar	damp	0.5	
	96	joint	78	111	097	4	10	0.2	organics	medium	very rough	undulating	damp	3	
OC-3	97	joint	77	045	031	5	5	tight	none	high	rough	curved	dry	3	OC-3 coordinates: 43.913108, -72.135528
	98	joint	84	288	274		10	0.4	none	high	rough	planar	dry	4	
	99	joint	55	124	110		6	0.12	none	medium	very rough	undulating	dry	0.5	
OC-4	100	joint	60	325	311		2	tight	organics	high	rough	planar	damp	2	OC-4 coordinates: 43.910042, -72.134217
	101	joint	71	079	065		4	tight	organics	medium	very rough	undulating	damp	2	
	102	joint	11	074	060		15	0.2	none	very high	very rough	stepped	dry	2	
	103	joint	86	142	128		15	n/a	none	high	very rough	undulating	dry	n/a	
	104	joint	85	206	192		2	n/a	n/a	medium	rough	planar	damp	3	
	105	joint	63	080	066		2	tight	none	high	rough	planar	damp	4	
	106	joint	70	330	316		10	tight	none	high	rough	planar	dry	n/a	
	107	joint	45	062	048		4	0.12	organics	medium	rough	planar	damp	1	
89+01	108	joint	69	341	327		2	n/a	none	medium	rough	curved	dry	4	
89+09	109	joint	58	160	146	2	4	0.04	none	high	rough	undulating	dry	3	
89+11	110	joint	85	014	360	6	4	tight	none	high	rough	planar	dry	5	
89+15	111	joint	65	034	020		9	0.12	organics	high	very rough	undulating	dry	4	
89+27	112	joint	88	196	182	6	20	0.08	none	high	rough	planar	dry	2	
89+35	113	joint	82	044	030	5	4	tight	none	medium	rough	planar	dry	4	

TABLE 1
DISCONTINUITY SUMMARY
FAIRLEE ROCK SLOPE, I-91 SB MM 92.55
GEODESIGN/VTRANS
VTRANS IM 091-2(97)
FAIRLEE, VERMONT

Discontinuity			Dip [°]	Uncorrected Dip Direction State Plane Grid North [°]	Corrected ¹ Dip Direction State Plane Grid North [°]	Set	Persistence [ft]	Aperture [in]	Infilling	Shear Strength ²	Surface Roughness	Discontinuity Shape	Water	Spacing [ft]	Notes/Comments
Sta	ID	Type													
89+39	114	joint	56	001	347		20	1	organics	high	very rough	undulating	dry	6	
89+50	115	joint	44	044	030		25	0.2	none	high	rough	undulating	dry	5	
89+57	116	joint	75	081	067		8	tight	none	high	very rough	curved	dry	5	
89+54	117	joint	34	175	161		5	tight	none	medium	rough	undulating	dry	3	
89+65	118	joint	70	065	051		8	0.5	none	high	very rough	irregular	dry	5	
89+84	119	joint	87	055	041		6	0.25	silt	medium	rough	undulating	moist	4	
89+86	120	joint	62	263	249		20	0.08	silt	medium	rough	planar	dry	4	
89+97	121	joint	46	155	141	2	20	0.08	silt	medium	rough	curved	dry	10	
90+09	122	joint	29	174	160		20	tight	none	high	rough	undulating	dry	3	
90+13	123	joint	71	067	053		4	tight	none	medium	rough	planar	dry	3	
90+15	124	joint	85	111	097	4	3	0.04	none	medium	rough	planar	dry	4	
90+25	125	joint	89	105	091	4	3	tight	none	medium	rough	planar	dry	3	
90+29	126	joint	86	075	061		6	tight	none	high	rough	planar	dry	5	
90+35	127	joint	33	108	094	3	5	0.12	none	medium	rough	undulating	dry	3	
90+36	128	joint	74	160	146		3	0.08	silt	high	very rough	undulating	dry	4	
90+39	129	joint	79	112	098	4	10	n/a	n/a	high	rough	planar	dry	6	
90+47	130	joint	55	039	025		6	0.12	n/a	high	very rough	undulating	dry	n/a	
90+56	131	joint	88	055	041		15	0.2	none	high	rough	planar	dry	6	
90+63	132	joint	70	283	269	1	20	0.25	none	high	rough	planar	dry	5	
90+65	133	joint	76	041	027	5	5	0.25	none	high	rough	undulating	dry	8	
90+67	134	joint	77	277	263	1	8	0.12	none	medium	rough	curved	dry	5	
90+69	135	joint	54	144	130	2	15	0.25	wet silty sand (SM)	medium	rough	planar	seeping	4	
90+97	136	fault	57	311	297		150	0.5	broken rock	low	very rough	planar	moist	15	
OC-5	137	joint	64	164	150		6	1	soil	medium	very rough	planar	damp	0.5	OC-5 coordinates: 43.91240, -72.13684
	138	joint	83	275	261	1	6	1	soil	medium	very rough	planar	damp	2	
	139	joint	12	162	148		3	0.25	soil	medium	very rough	planar	damp	0.25	
OC-6	140	joint	83	005	351	6	6	0.25	organics	low	rough	planar	dry	0.5	
	141	joint	80	332	318		3	tight	none	medium	very rough	planar-irregular	dry	2	OC-6 coordinates: 43.91254, -72.13665
	142	joint	89	203	189		3	1	soil	medium	rough	planar	damp	?	
OC-7	143	joint	81	180	166		6	?	?	medium	rough	planar-irregular	dry	2.5	
	144	joint	74	314	300		8	0.25	soil & organics	high	very rough	planar-undulating	dry	1.5	OC-7 coordinates: 43.91253, -72.13685
	145	joint	88	003	349	6	12	0.5	soil & organics	medium	rough	planar-undulating	dry	2.5	
	146	joint	74	248	234		1	tight	none	medium	rough	planar	dry	3	
	147	joint	79	214	200		2.5	0.25	soil & organics	medium	rough	planar	dry	1.5	
OC-8	148	joint	78	274	260	1	3	0.1	soil & organics	high	very rough	planar	dry	0.5	
	149	joint	83	012	358	6	4	?	soil & organics	medium	rough	planar	dry	0.75	OC-8 coordinates: 43.91254, -72.13691
	150	joint	72	107	093		1	tight	soil & organics	medium	rough	planar	dry	4	
	151	joint	71	283	269	1	4	tight	soil & organics	high	very rough	planar	dry	1.5	
	152	joint	69	319	305		8	?	soil	medium	very rough	planar-irregular	dry	2	
	153	joint	76	274	260	1	1	tight	none	medium	very rough	planar	dry	2	
OC-10	154	joint	78	001	347		1	tight	none	medium	rough	planar	dry	0.75	
	155	joint	14	124	110		5	?	organics	medium	rough	planar	damp	2	OC-10 coordinates: 43.91251, -72.13699
	156	joint	89	187	173	6	8	?	organics	medium	rough	planar	dry	4	
	157	joint	80	261	247		3	?	FeO _x	high	very rough	planar-irregular	dry	8	
	158	joint	20	146	132		6	?	?	medium	very rough	planar	damp	?	
	159	joint	70	163	149		1	?	?	high	very rough	planar	dry	?	

Notes:

- Dip direction corrected for magnetic declination (14° west).
- Shear strength of infilling materials/discontinuity as follows:
low = friction angle (ϕ) < 20°
moderate = 20° < ϕ < 30°
high = ϕ > 30°
- Discontinuity measurements obtained July 02 to October 29, 2024.

ATTACHMENT NO. 3

Geotechnical Laboratory Data



Construction Observation Report

Project Name: I-91 Rock Cut Rehabilitation
Location: Fairlee VT
Client: Scarptec, Inc.
Weather: Clear 65 - 75°F
General Contractor: Scarptec, Inc.

Project NO.: 24-1618
Date: 8/28/2024
Client Rep: David Scarpato
S.W.COLE Rep: Dakotah Senesac
Time Onsite: 08:00 - 16:30

Observations / Discussions:

Dakotah Senesac of S.W. Cole Engineering, Inc. was on-site as requested by David Scarpato to perform rock coring operations at the I-91 rock cut for compressive strength and density determinations of the obtained rock core samples. 6 rock cores were obtained via a 2-inch inside diameter diamond tipped core drill from the outer 5 to 13-inches of the existing rock cut face from approximately ground height. A 13-inch length core sample was saw cut into 2 separate core samples for testing. See attached compressive strength and density data, as well as photos of individual core samples pre/post compressive strength testing.

Attachments: 14 Photos, 2 Other

Reviewed by:
9/5/2024

Alan Brown
Alan Brown



13-inch core samples 1A & 1B.



7-inch core sample 2.



9-inch core sample 3.



7.5-inch core sample 4.



5.5-inch core sample 6.



7-inch core sample 7.



Rock core samples after saw cutting for compressive strength determination.



Core 1A break.



Core 1B break.



Core 2 break.



Core 3 break.



Core 4 break.



Core 6 break.



Core 7 break.

Attachments

Report of Uniaxial Compressive Strength of Intact Rock Core

ASTM D7012-10 Method C

Project Name: I-91 Rock Cut Rehabilitation
Project Location: Fairlee, VT
Client: Scarptec
Material Description: Rock Cores
Material Source: I-91 Rock Cut at ground level

Project Number: 24-1618
Lab ID: Multiple
Date Received: 08/28/24
Date Completed: 09/05/24
Tested By: D. Senesac

Sample Info:	#1A, Station	#1B, Station	#2, Station 89+77	#3, Station 90+04
Boring and Sample No.	89+49	89+49		
Sample Depth	0"-4"	6"-10"	.5"-4.5"	3"-7"
Bedrock Classification				
Lab ID	4709W	4710W	4711W	4712W

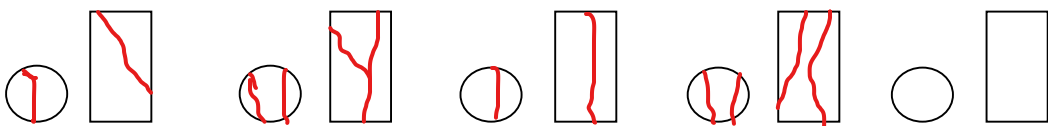
Dimensions:

Length (in)	4.070	3.910	3.980	3.970
Diameter (in)	1.994	1.994	1.994	1.994
Length Capped (in)	4.406	4.187	4.29	4.231
Length/Diameter Ratio	2.0	2.0	2.0	2.0
Weight (lbs)	1.230	1.190	1.200	1.210

Test Results:

Unit Weight (pcf)	167.2	168.4	166.8	168.7
Strength (psi)	8,770	10,100	4,750	5,150

Fracture Location



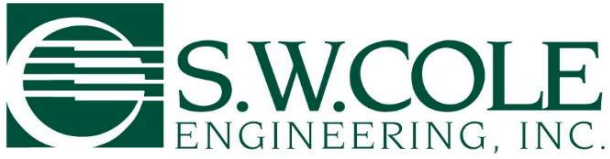
Other Information:

Loading Rate ~300 lbs per second
 Moisture Condition Dry

Comments: See attached photos of core samples.

Reviewed By:





Report of Uniaxial Compressive Strength of Intact Rock Core

ASTM D7012-10 Method C

Project Name: I-91 Rock Cut Rehabilitation
Project Location: Fairlee, VT
Client: Scarptec
Material Description: Rock Cores
Material Source: I-91 Rock Cut at ground level

Project Number: 24-1618
Lab ID: Multiple
Date Received: 08/28/24
Date Completed: 09/05/24
Tested By: D. Senesac

Sample Info:

Boring and Sample No.	#4, Station 90+05	#6, Station 89+48	#7, Station 89+06
Sample Depth	2"-6"	.5"-4.5"	1.5"-5.5"
Bedrock Classification			
Lab ID	4713W	4714W	4715W

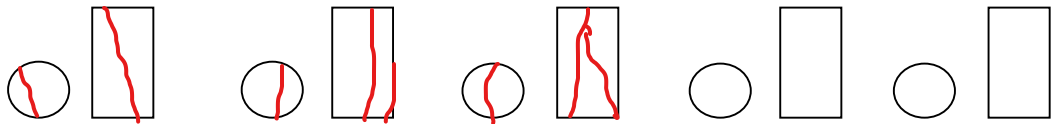
Dimensions:

Length (in)	3.990	3.970	3.890
Diameter (in)	1.994	1.994	1.994
Length Capped (in)	4.189	4.26	4.119
Length/Diameter Ratio	2.0	2.0	2.0
Weight (lbs)	1.220	1.200	1.180

Test Results:

Unit Weight (pcf)	169.2	167.3	167.9
Strength (psi)	5,320	6,100	11,150

Fracture Location



Other Information:

Loading Rate ~300 lbs per second
Moisture Condition Dry

Comments: See attached photos of core samples.

Reviewed By: